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About the Concept of Uncertainty and Methods to Reduce Uncertainty

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Abstract: In condition assessment of a structure a large number of parameters are used, a majority of them having a random characteristic both in instantaneous magnitude and in its evolution. Incompleteness of information at a moment represents an obstacle to complete evaluating of system condition. Condition index estimator is, at its turn, a random variable. It is recommendable to implement permanent programs of investigation and monitoring the structures in operation. When examples are required, variables involved in bridge assessment process are considered.

Rezumat: Asupra conceptului de incertitudine și a metodelor de reducere a incertitudinii

În determinarea stării tehnice a unei structuri se utilizează un număr mare de mărimi, majoritatea având caracter aleator atât în mărimea instantanee, cât și în evoluție. Incompletitudinea informațiilor, la un moment dat reprezintă un impediment în evaluarea completă a stării sistemului, estimatorul stării fiind la rândul său o variabilă aleatoare. Este de aceea recomandabil a se realiza programe permanente de urmărire, chiar de monitorizare a comportării în exploatare a construcțiilor. Pentru exemplificare se consideră variabile implicate în procesul de determinare a stării tehnice a podurilor.

Keywords: construction, bridge, condition assessment, uncertainty.

1. Introduction

Design, construction and operation of structures, as well as their analysis, are affected by many sources of uncertainty. Most often data describing structural behavior are in fact random variables with a certain distribution of probability.

The necessity of developing an infrastructure management system leaded us to the obvious need of knowing and taking into account the uncertainty, its dimension and methods to limit its consequences.

Further, in this paper, we present several considerations on this subject, with examples from highway bridge management, without intending to limit the generality of the approach.

2. Sources of uncertainty

A construction, structure is a complex system. This system is an assembly of many components and is influenced by a multitude of intrinsic and extrinsic

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factors. Its behavior and development may be described only using many variables, each one, as well as their governing relations, being affected by uncertainty.

Uncertainty may be classified in many different types according to its source. It is important to define and to understand these sources while in the process of structural assessment improving the level of knowledge may influence the uncertainty itself. This is possible, within certain limits, according to the sources.

Several attempts of classification have been made. Authors attach ambiguity and vagueness to the uncertainty in defining system structure, parameters, behavior and prediction models [2]. Uncertainty of ambiguity has generally four sources [6]: physical variability, estimation error, model imperfection and human error. Vagueness is related to cognitive sources like quantitative definition of variables, variable interrelations especially in complex systems [7].

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Physical variability (natural variability) of the variables describing basic characteristics: is very difficult to eliminate this type of uncertainty through improving the level of knowledge. Hence, variation of the rupture effort along a steel reinforcement bar may not be practically determined through tests without destroying the bar. Natural variability may also arise in time: effect of road traffic flow action over a particular section of the structure. The fact that variables have a variation in time is important because this influences the way the resulted uncertainty is modeled in probabilistic analysis.

Estimation error (statistical error) results from incompleteness of statistical data from which probabilistic models parameters are estimated. As an example, if average and variance of rupture effort are considered and they are derived from nsamples then estimation error will decrease when nincrease. Hence, an enlargement of quantity of collected data produces a reduction in the uncertainty caused by estimation error. Estimation error may emerge from considering product from sources that have not been considered when first computed the parameters. In this approach, errors may affect perception when considering average and variance for products supplied from steel mills other than those used initially to infer these parameters. It is possible to

reduce uncertainty caused by estimation error trough tests and appropriate sampling.

Human error may emerge in any stage of the life cycle: feasibility study, design solution selection, design, construction, operation, inspection or maintenance of the structure. Uncertainty caused by the human conduct may be reduced by introduction of quality assurance methods that can decrease the rate and the proportion of errors. This type of uncertainty may be reduced by project supervisors for design, inspections and laboratory tests for construction, installation of protection and safety devices for operation errors.

For bridge condition assessment may appear or may be induced errors with multiple cause. They may start in different stages of the inspection and evaluation. Process of assessment and possibility of apparition of errors are presented in Fig.1.

Model imperfections emerge from use of mathematical models to represent real phenomena. This type of uncertainty has two components: one induced by the lack of understanding of the subject and the other due to use of simplified models. In the first case, only in seldom occasions is practical to improve the understanding only to reduce the uncertainty in structural evaluation. Tests may be done to calibrate an older model rather than to



Fig.1. Appearance of errors in bridge condition assessment process

develop a new one. Uncertainty due to simplification can be reduced adopting a more precise model. However, it is not very clear yet whether this type of model gives always better solution, as complex calculations do not necessarily increase precision.

Cognitive source error is related to the availability of knowledge at a certain moment in a certain field, to the capability of individuals to understand, and to the perception filtered by ones judgement. Most of the time defaults and degradations are not identified by measurement, which would be extremely expensive or time consuming, but calling for the opinion of the experts. The final results depend on the competence, correctness, and sight capacity of individuals. Obtained values must be looked mostly as psychological linguistic expressions rather than accurate realization of the reality.

This type of uncertainty may be addressed by:

• developing research programs in psychology with application in engineering comprehension to determine the way humans identify, assess and decide;

• implication of as much as possible individuals in assessment process; and

• programs to develop and use elements of artificial intelligence that would recognize forms and assess level of degradation.

3. Condition assessment

The process of technical condition assessment is described in the next steps [1]:

• Measure of defaults and deterioration magnitude

• Identify the effects of defaults and deterioration on the condition of the structure;

• Establish the set of parameters describing the condition of the structure;

• Compare magnitude of defaults and deterioration with preview records.

Different types of structures have similar responses. Bridges constructed from different material (concrete, steel, composite etc.) have analogue reaction to same loading. However, there are no two identical structures, each construction is unique considering its characteristics (dimensions etc., but mainly the deterioration and degradation mode).

A section of the structure is subject to actions. On the other side the section is capable to support a certain effort determined by its geometrical, physical and mechanical characteristics. Supposing that X_a is the action and X_r is the resistance then we may write the well-known relationship:

$$X_a \leq X_r$$

However, this would be the ideal case when structure is perfectly measurable, elements strictly homogeneous and exterior actions are completely controllable and measurable. Obvious such a case is excluded and both actions and resistance act like random variables. Reasons are multiple (most of them explained before) and some of them may not be completely studied with present techniques (wind, earthquakes).

Whenever we cannot describe these variables using exact algebraic functions it is convenient to choose a probabilistic representation.

Considering sectional actions and resistance as random variables X_a , X_r , they are described by their parameters \bar{x}_a , σ_a , \bar{x}_r , σ_r (average and standard deviation) and functions $f_a(x)$, $F_a(x)$, $f_r(x)$, $F_r(x)$ (probability distribution and repartition function).



Fig. 2 Actions and resistance distribution

As the parameters are not punctual values they may not be simply used in the safety relation. However, it is possible to define probability of safe function:

$$P_S = P_S(x_a \le x_r) = 1 - P_S(x_a > x_r) = 1 - P_c$$
.
Where:

 P_s = probability of safe functioning;

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 P_c = probability of collapse.

$$P_{c} = \int_{0}^{\infty} \left[1 - F_{a}(x) \right] f_{r}(x) dx = 1 - \int_{0}^{\infty} F_{a}(x) f_{r}(x) dx$$

or
$$P_c = \int_0^\infty F_r(x) f_a(x) dx$$
,

and

$$P_{s} = \int_{0}^{\infty} F_{a}(x) f_{r}(x) dx = 1 - \int_{0}^{\infty} F_{r}(x) f_{a}(x) dx.$$

Starting from a statistical study on the structure and its component one may calculate the probability of collapse or the probability of safe function at a certain moment. Obviously, the procedure is laborious and a level of uncertainty is involved.

Evolution of the reliability

Any structure was designed to support particular actions at a given moment. Future actions are considered but methods of prediction have a high level of subjectivism. Also some bridges are built sixty, seventy, even hundred years ago. Technical development makes those assumptions inoperable.



Fig.3 Evolution of actions and resistance.

It is obvious that materials used in a technical work suffer a continuos inherent degradation.

Capacity to support actions decrease permanently in average. Also, actions have o continuos increase following the development of technique, science, and level of life.

Schematically, these evolutions are presented in Fig.3. From the same figure one may see that distributions of actions and resistance are shifting in time following the average. Using these distributions the reliability of the system (*probability of safe function*) is ready to be calculated.



Fig.4 Evolution in time of the reliability. Distribution of reliability estimator.

The evolution of action and resistance implies the evolution of the level of structural reliability (Fig.4). Obtained values of reliability may have significance when compared with some threshold levels that are considered standard.

These levels are established, often arbitrarily, according to moment's objectives, by the administrator, users or authorities. It is possible to exist several considered levels simultaneously. They may serve different purposes: for safety of normal traffic, to ensure special or military transports etc. Level of accepted reliability can be used to determine or to limit the structure's life.

The value of reliability $R_p(t)$ as intrinsic parameter in the environment context is only seldom perfectly comprehended. Lack of total control over traffic, measure methods, material non-homogeneity, error in calculation methods induce a random variable character to the calculated value of reliability $\hat{R}_p(t)$ (reliability estimator). In this case the reliability estimator itself has an average value $\overline{\hat{R}}_p(t)$ and a probability distribution function $\hat{R}_p(x,t)$. However, reliability is a probability and it takes values only in the interval [0,1]. Hence, usual distributions (normal, log-normal etc) may not be used to describe it. A possibility is to represent the variability of estimated reliability using beta distribution (Fig.4).



Fig. 5 Reliability estimation variability for a system

Beta distribution has the advantage of taking values only in a limited interval. Its use corresponds to the implicit assumption that in average there is no possibility that reliability takes a value of 1. In other words, there is no perfection and no absolute safety, which is rational.

According to specific conditions of operation and to traffic actions the degradation curve differ from case to case. From curve's shape and level of acceptable reliability as condition index the length of the life cycle of the structure may be revealed.





In Fig.6 [11] we may see that structures that operate in difficult conditions (or have been incorrectly designed or constructed) have a shorter life than structures that operate in light conditions (no corrosive chemicals, light traffic, low percentage of trucks).

4. Methods to reduce uncertainty

When assessing the technical condition of a bridge a large number of values are evaluated and used. Most of these values have random character and may be described by stochastic variables. Their magnitude, parameters and evolution can be apprehended only with a certain probability.

Incompleteness of information, at a certain moment in time, represents an impediment in complete evaluation of system condition. It is, therefore, recommendable to implement programs of permanent observation, even monitoring, of both material characteristics of bridges and distribution of load actions. The best remedy is knowledge.

Further we represent the consequences of field investigation and measurement on state of knowledge over the bridge and the effort due to traffic (Fig.7), (σ_s , σ_c and $f(\bullet)$ represent effort, resistance and probability distribution function respectively). For complete treatment see Das 1994 [5].



Fig.7 Influence of the investigations and measurement over the perception of the distribution of actions and resistance

A must is also the quality of inspection given by $\rho(s)$ the rate of detection of defaults having magnitude s and the accuracy of determining that magnitude [10], [12]. Rate of detection $\rho(s)$ is also named probability of detection [3], [9]. Rate of detection and dimension error is uncertain. This uncertainty may be incorporate in reliability analysis for the inspected structures or structural components [8], [10], [12].

An important aspect in data analysis is represented by the model. A model is a reduced and not-expensive representation of a system (physical, chemical or biological) in a mathematical, abstract, numerical or experimental form [4]. The model highlights only essential aspects of an existent system or of a system to be constructed. A model of any system respects the general form of a system. A model is a description of the reality that captures a part of the aspects, relevant for the purpose observed. Finally, a model is an imperfect representation using idealizations, implicit assumptions, simplifications generated by and subordinated to the final goal.

It is not necessary a model to describe exhaustively the real mechanisms of the system. It must only to mime its conduct.

Model can be used in simulation. Simulation is a technique of performing experiments, generally using the computer, which implies elaboration of mathematical and logical models to describe the response and evolution of a real system.

Necessity for simulation is generate by the fact that often the real systems can not be studied because of their complexity (difficulties in quantitative evaluation of phenomena, high number of input/output variables, high number of condition states, long time of the phenomena, high costs etc.)

5. Conclusions

The behavior of structures is described by a multitude of variables. It is impossible to completely examine and describe their evolution. Hence, it is difficult to evaluate system condition, which is permanently affected by uncertainty.

However, necessity of developing an infrastructure management system imposes comprehension of system condition at present moment and its evolution in time. Uncertainty must be, therefore, studied, understood, evaluated and its effects limited as much as possible.

Uncertainty does not occur only from natural causes. Human activity is also very important either trough reduce level of knowledge or failure to apply the existent knowledge.

Subject should be further treated with maximum consideration. It is necessary to identify sources of date and facilitate access for researchers.

A close co-operation should start among administration, contractors and universities so that transfer of information, processing, and inference of knowledge to be faster.

To eliminate human error procedure manual should be written for structure inspection and assessment and data acquisition and transmission. Their correct utilization should be also observed.

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Consideration on Use of Fuzzy Sets in Construction Condition Assessment

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Abstract: A structure (i.e. bridge) is probabilistic system acting in a random environment. Due to uncertainty related to human subjectivism, data incompleteness, lack of knowledge, errors inherent to the model, the values describing the defects, degradation and technical condition of structure are expressing the reality only with a certain probability. This probability depends on a multitude of factors only partially known. Consequently, it is possible and recommendable to represent these values as fuzzy sets connected to linguistic expression. The idea is that human mind may perceive, comprehends, and handles linguistic expression easier than exact numerical values. The paper is presenting the achievements in bridge condition assessment using fuzzy sets.

Rezumat: Considerații privind utilizarea seturilor fuzzy în determinarea stării tehnice a unei construcții

O structură (ex. un pod) este un sistem probabilistic ce acționează într-un mediu aleator. Din cauza incertitudinii legate de subiectivismul uman, incompletitudinea datelor, cunoștințe limitate, erori inerente modelului valorile care descriu defecte, degradări sau starea tehnică a structurii exprimă realitatea doar cu o anumită probabilitate. Această probabilitate depinde de o multitudine de factori cunoscuți doar în parte. În consecință, este posibil și recomandabil să se reprezinte aceste valori ca seturi fuzzy legate de expresii lingvistice. Ideea este aceea că mintea umană percepe, înțelege și aplică expresii lingvistice mai ușor decât valorile numerice exacte. Lucrarea prezintă realizări în utilizarea seturilor fuzzy în evaluarea stării tehnice a podurilor.

Keywords: construction, bridge, condition assessment, fuzzy sets, uncertainty.

1. Introduction

Technical condition is a concept accepted as a convention by specialists in a certain field of activity and geographical area.

Generally and simplified we may say:

Technical condition of a structure in civil engineering represents an evaluation of all characteristics of the technical system perceive at a moment in time and which corroborated with influence of the environment determines the present and future conduct of the system from the point of view of the technical goal it was built for.

Obviously, such a definition omits or neglects some aspects:

- It is impossible to take into account all characteristics of the system;
- It is difficult to establish the exact value of parameters and approximations are considered;
- It is impossible to model absolutely exact the behavior of the structural materials;

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• It is difficult to assess the exact value of external factors action onto the system.

2. Technical condition

As presented before, the technical condition of a structure is a quantitative description, which approximates the qualitative description of the behavior of the structure, as a system, in the moment of the assessment.

Technical condition must not be confused with reliability, which is a probability calculable within some limits of uncertainty. The scale of technical condition for elements and the rating scale for the ensemble are arbitrarily chosen and defined by experts. In time, experience and evolution of human knowledge corrected the assessment method.

Current method of description of technical condition is construction of a condition index by aggregation of data concerning individual elements. Such an index may refer to components (for bridges © 2000 Ovidius University Press these would be superstructure, substructure, deck), to the system as a whole or even to the entire network of similar structures. Level of aggregation of data depends on goal and the engineering field addressed.

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In evaluating the technical condition of bridges, first, the identification of defects [2] and their severity assessment must be performed. Based of severity a value is attached to each defect. The value describes the degradation and it is in the range of [1,10], 0 representing the lack of defects. These values are only global evaluations that emerge consequently to inspector's appreciation. Only seldom, precise measurements are performed. Therefore, degree of subjectivism is high and is amplified by the random influence of the environment.

In Romania, technical condition of road bridges are assessed according to Instruction AND522-94, elaborated by INCERTRANS in cooperation with NAR and Bucharest Construction Institute and approved by Ministry of Transportation. According to this regulation 5 indices of material (C_i) and 5 indices of functionality are defined (F_i) .

Each index is computed by penalizing the degradation, as presented before, out of a maximum of 10. The final Technical Condition Index (I_{ST}) is computed as a sum of all the others:

$$I_{ST} = \sum_{i=1}^{5} C_i + \sum_{i=1}^{5} F_i \; .$$

The advantage of the method is extreme simplicity. The disadvantage is the subjectivism.

3. Probabilistic approach

Due to uncertainty related to human behavior, data incompleteness, lack of knowledge, errors inherent to the model, the value describing the defect and degradation is expressing the reality only with a certain probability. This probability depends on a multitude of factors only partially known. Hence, it is possible and recommendable to represent these values as fuzzy numbers.

As shown before, a bridge is a probabilistic system and its process of degradation is stochastic

process. Simultaneously, evaluations of inspectors are rather subjective and are affected by a great number of factors as experience, expertise, physical capacity, visual acuity, psychic condition, weather condition etc.

It appears therefore necessary that technical condition description to be defined by linguistic expressions rather than numerical values. Only after, real numbers or fuzzy values may be attached to these linguistic expressions and further computation would be performed using the attached values. Some countries have already defined such linguistic expressions. The idea is that for bridge acting in an uncertain environment, the bridge itself being a system with high degree of uncertainty, human mind may perceive, comprehend, and handle linguistic expression easier than exact numerical values.

According to AND522/94 regulation, even some hierarchy and gravity level is implied yet there is no linguistic explanation of the grades of penalty applied for degradation.

3.1 Frequency of defects occurrence

Periodic inspection is performed to evaluate technical condition. A sample of 149 bridges has been studied. This represents 4% from the entire population of bridges on national roads network.

Following we present examples of occurrence of defects for subsystems (Fig.1,2,3) (described by an condition index according to current regulations).



Fig.1 Frequency of occurrence of defects for C1 (149 bridges)



Fig.2 Frequency of occurrence of defects for C2 (149 bridges)



Fig.3 Frequency of occurrence of defects for C3 (149 bridges)

4. Use of fuzzy sets for condition assessment

Current mode of assessment is to attach a penalty for each defect. However, degree of degradation is not consequent to an exact measurement but to engineer's judgment so affected by subjectivism and uncertainty.

Uncertainty may occur in to directions: either different level defects rated with the same rank or same level degradation rated different. Resulted subjectivism and uncertainty is possible to model trough fuzzy sets.

4.1 Elements of fuzzy sets theory

• *Confidence interval*: Classic logic attaches to a predicate a logical value discrete well defined:

TRUE=1 or FALS=0. Practice has proved that to a sentence one may attach a degree of true, a probability or a level of confidence [5].

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As an example, whenever a defect is noticed and a value for the level of degradation is attached, that that value is describing the reality only with a certain probability, with the same value being described states, in fact, states from a confidence interval. Hence, if A = a more often the reality is $A_a = [a_1(a), a_2(a)]$. Only when confidence is total we may write for A = athat $A_a = [a, a]$ (to ensure the compatibility for further calculation).

For this kind of numbers normal abgebraic operation may be defined. All these operations takes values over real space R but result are valid also for particular case of discrete values in integer number space N.

• *Fuzzy sets*: Confidence interval contains an implicit assumption. Probability of membership was considered the same for the whole interval so the graphical presentation would be like the one in Fig.4.



Fig.4 Representation for a confidence interval

In the preview example the confidence interval was considered A = [4,8].

However, this represents an idealization. Confidence interval describes only partially the degree of uncertainty. Each value included in the interval has a certain probability to belong to the representation. In other words membership of a value implies a degree of true or a level of confidence. The degree of true or the level of membership to the interval is represented numerically trough a value in [0,1] interval (Fig.5).



Fig.5 Level of membership for a value

The concept of "fuzzy" was firstly introduced by Zadeh [7]. He used this word to generate a generalization of the concept of set of elements.

Consider a universal set X. A fuzzy subset A is defined by its membership function $\mu_A: X \to [0,1]$, which attaches to element $x \in X$ a real number $\mu_A(x)$ in the interval [0,1], where the value $\mu_A(x)$ represents the degree in which x belong to A.

Most often the "fuzzy set" term is used but meaning of subset is implied.

A fuzzy set may be described by the ordered set of pairs of elements x and rank $\mu_A(x)$. It is often written as: $A = \{(x, \mu_A(x)) | x \in X\}$.

Example: $X = \{0,1,2,3,4,5,6,7,8,9,10\}$ the set of grades to penalize degradation. Let consider the fuzzy set of the grades *approximately* 6. Such a set might be $A = \{(4,0.1), (5,0.5), (6,1.0), (7,0.5), (8,0.1)\}.$

This set describes the multitude of real levels that would be possibly ranked with 6 and the level of certainty that this happens.

Another mode to denote a fuzzy set may be:

$$A = \sum_{i>1}^{n} \mu_A(x_i) |x_i|, \text{ meaning:}$$

$$A = 0.1 | 4 + 0.5 | 5 + 1.0 | 6 + 0.5 | 7 + 0.1 | 8,$$

or even:

 $A = \begin{bmatrix} 0.1 | 4 & 0.5 | 5 & 1.0 | 6 & 0.5 | 7 & 0.1 | 8 \end{bmatrix}$ Because $\mu_{4}(x)$ represents a level of true, a

probability of membership and not distribution

probability it is not mandatory that $\int_{X} \mu_A(x) dx$ to take the value 1. As a consequence the maximal possible value for $\mu_A(x)$ is 1.

A fuzzy set A is normal if and only if $\forall x \in X$, $\bigvee_{Y} \mu_A(x) = 1$.

A fuzzy set where $\bigvee_{v} \mu_{A}(x) \neq 1$ may be normalized

dividing each value $\mu_A(x)$ by $\bigvee_{v} \mu_A(x)$ for $\forall x \in X$.

Appropriate operations and concepts have been defined for fuzzy sets [6]: support, height, normality, equality, empty set, containment, complement, intersection, union, algebraic product, bounded product, bounded sum, bounded difference.

A fuzzy set is convex if and only if

 $\mu_A(\lambda x_1 + (1 - \lambda)x_2) \ge \min(\mu_A(x_1), \mu_A(x_2))$ for $\forall x_1, x_2 \in X$ and $\forall \lambda \in [0, 1]$. (Fig.6)



Fig.6 Convex (left) and non-convex (right) fuzzy set

It is often necessary to represent and to use numbers like "approximately m", "close to n". Such numbers have been formally defined as follows ([3],[4],[8]):

A fuzzy number is a convex normalized fuzzy set over R whose membership function is piecewise continuous.

Examples of membership functions for fuzzy numbers are the triangular function or bell-shape function ($\mu_A(x) = \max(0, 1-|x-a|/m)$ where m > 0 or $\mu_A(x) = e^{-b(x-a)^2}$ where $b \ge 1$) (Fig.7).



Fig.7 Examples of membership functions for fuzzy numbers.

As numbers, fuzzy numbers are subject to algebraic operations. Of coarse, the respective operations, addition, subtraction, multiplication and division, are extensions of those defined on **R**. However, attention should be paid to the fact that for fuzzy set it not necessary that $\mu_{A-A}(z)=0$.

4.2 Use of fuzzy sets for defects

When assessing the technical condition of structures one must first identify the defects and evaluate their severity.

As presented before, due to uncertainty, subjectivism, variability, error and lack of knowledge the values describing the defect are expressing the reality only with a certain probability and it is wiser to represent the levels of degradation by fuzzy numbers rather than normal real numbers (Fig.8).



g.8 Representation of values of defects as distributions

When more different defects are found together on the same element current practice is to choose the maximum of penalty values. Applying the *maximum* operation for the two fuzzy numbers a new fuzzy with a new membership function.

For two defects with clear different values (like those in the preview picture) the maximum operation leads to no notable change in the shape of the set in the right. However, if more defects close or equal are found the resulted shape change to right (the calculated severity for more defects increases) Fig.9.



Fig.9 Maximum for more identical defects (example)

An advantage of using fuzzy set results from possibility of describing only mentioned defects and degradations when field inspectors failed to rank the degradation or data is missing from records.

For purpose of better management of data resulted from technical inspection of bridges a software was developed. This software facilitates collection of degradation rating values as they were found in the field. It stores and retrieves these values and it performs the calculation of condition indices for main subsystems as well as for the entire structure. An option was added so that al the calculation may be carried out using fuzzy sets instead of simple numbers (Fig.10, 11).

An automatic conversion is executed each time a degradation submitted and for each level of degradation a fuzzy set is attached describing the subjectivity and uncertainty.

The system is still in tests and work must be done for correct calibration of the shape of sets. However, it clearly demonstrates the possibility to use new mathematical approach in structure condition assessment. It covers the subjectivism and probability that hidden defects exist and their presence may be detected only through their consequences (secondary defects).

Also when, for different reasons, degradation was identified but not quantified still ambiguity may be processed attaching appropriate fuzzy sets on the confidence interval. Lack of numerical data does not affect proper function of software.



Fig.10 Technical condition assessment using fuzzy sets



Fig.11 Graphical representation for condition

5. Conclusion

Technical condition assessment of structures is a difficult task. While the evaluation is basically an expert job without involving a great deal of exact measurement but rather a lot of human skills, sight in good condition, experience and judgment.

Also a structure is a probabilistic system, which is influenced by a random environment. Due to all these and to uncertainty related to data incompleteness, lack of knowledge, errors inherent to the model, the values for degradation are describing the real condition only with a certain probability. Hence, it is possible and recommendable to represent these values as fuzzy numbers.

This representation is possible and the system developed is a proof.

Advantage is offered by the possibility to account for uncertainty and subjectivism. Also it gives the opportunity to consider the defects and degradation that inspectors fail to evaluate or data are missing in old records.

More research is a must for further calibration of sets shape according to the type of defect and severity.

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Dynamic Modal Analysis of Harbor Protection Rock-Fill Breakwaters

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Abstract: The aim of the paper is to present a numerical dynamic analysis of composite rock-fill breakwaters. To conducte the dynamic and seismic analyses for a designed breakwater it is necessary to determine the dynamic characteristics of the structure taking into account the interaction effects with soil foundation water. Numerical results are furnished for a rubble-sloping breakwater of 35.00m height founded at –24.00m in Constantza Port.

Keywords: breakwater, eigen value, interaction response

1. Introduction

A breakwater is a structure protecting a harbor from waves, thereby preventing these from exerting their destructive influence upon area enclosed for shipping reception. Basically, there are two main types of breakwaters: the vertical (or almost vertical) wall type which may be built of natural rock, masonry, wood, steel or concrete and the sloping mound type which may be built of rock, concrete or of rock/concrete/asphalt mixtures [1,4].

Composite rock-fill breakwaters, also named rubble-mounds or rubble sloping breakwaters (Fig. 1) do not have land backing to help them resist the horizontal forces. They must therefore be designed to resist them by shear stress of the base. Since it is difficult underwater to provide protrusions on the base of the breakwater, as is the case for foundation dams, the shear resistance must be developed by friction on the sand or rubble-mound bed.

Since the maximum force due to waves is often due to shock pressure from waves breaking right at the breakwater face, the resultant force may be often of extremely short duration. Also, since these structures can sway small amounts on the soil or stone foundation, we can suppose that the inertia resistance of the structure against the sway may absorb a portion of a shock pressure consequently the shearing forces which the structure are called upon to resist may be reduced by the amount.

The design of rubble-mounds breakwaters has so far been based on the developed of empirical relations with little attempt to understand the basic

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hydrodynamics of the matter. The result has been the development of empirical $K_{\Delta}(2.4)$ (coefficient that varies primarily with the slope of the armor units, roughness of the surface, sharpness of edges and degree of interlocking) values which could just as well be called "ignorance-values" [4] as little effort has been made to explain heir huge variation. This, from a technical-scientific point of view, unsatisfactory situation has caused overestimation as well as underestimations and errors [1,2,3]. It may firmly be stated that the available design formulas are not very safe or suitable for actual design.

The design rules of rubble slopping breakwaters are under continued development [3,5] and the most remarkable feature is probably the procedure of "probabilistic design" as the most practical tool for assessing an optimum design that considers all technical and economic aspects.

This paper is a part from a larger study dedicated to dynamic/seismic analysis of breakwaters (rubblesloping type). The aim of the paper is to present the results of numerical analysis concerning the dynamic characteristics of composite rock-fill breakwaters.

2. Failure mechanisms of composite breakwaters

Stone size of the material available must be great enough to resist the wave forces so that the mound is not flattened during storms. There may also be considerable settlement of the sea bed under the load of the structure. In certain case, this may be increased by the addition of the stone intended to make the sinkage. Therefore, stability may have to be secured by replacing poor bottom material with better material before rock material for the breakwater is dumped

The basic principle is to build up a core and provide it with a protective covering. The core, usually consist of quarry (waste) material ranging in size from a few kilograms to a few hundred kilograms and securing good compactness. In certain cases, a sand fill may provide the inner part of the core.

The protective cover also called armor, must consist of blocks which hinder core material from escaping at the same time as it itself must remain stable against all securing wave action or at least stable enough to prevent collapse under even the worst conditions (although this may result in some damage).

An unit placed in a breakwater armor, in contact with other units is subjected to forces by breaking waves, wave uprush and downrush. During the action the structure is, to a certain extent, filled with water, which exerts hydrostatic pressure on the armor unit. Understandably, it is very difficult to analyses such a problem in all three dimensions. When wave action is assumed to be perpendicular to the face of the structure, the hydrodynamic forces are mainly three-dimensional, but forces are at first considered being twodimensional. This simplifying assumption may cause some errors but will hardly affect the appraisal of the relative importance of the force involved.

The design of rubble-sloping breakwaters implies much verification of general stability and individual stability of units in the armor and genered stability of armor also, under dynamic actions of waves and earthquakes. For that it is necessary to analyze all possible mechanisms of failure. The following scenario has to be considered [1,4]:

a) Slide of units in upper part of armor, when the weight or sizes of rock blocks or artificial concrete blocks are not great enough to resist the wave forces.

b) Slide of armor, generated by insufficient weight or size of the armor units, side slopes, density of armor material and degree of interlocking or wedging between units. c) Degradation of inner side and regressive erosion of breakwaters due to overtopping of waves over the crown of breakwater.

d) Deterioration of the material below the superstructure by waters rushing in (uprush) and out (downrush); the superstructure may then fum counterclockwise and diagonal ruptures appear in the slab.

e) Slide of berme at the foot of the slope as a result of lack of sea bed protection.

f) Settlement and breakage of the armor as a result of loss of fine material from the core by piping on of withdrawal of individual small stones of the underlayers through the interstices of the cover layer.

g) Loss of overall stability of breakwater generated by failure of mass of soil foundation

3. Breakwater – soil foundation – water interaction

The effect of the flexibility of the soil on the dynamic response of structures, particularly when subjected to seismic excitation has been a subject of considerable interest and research in the last twenty years. This effect is important when dealing with very massive structures, such as breakwaters. A number a sophisticated mathematical techniques, elaborate computer codes. and simpler engineering approximation have been developed to account for soil flexibility in a single step, considering the combined soil-structures systems, or in three separate steps, using substructuring techniques [6,11].

While the advantages and disavantages of these two approaches and the conditions under which they will give equivalent and sensible results have been discussed by various authors, some controversy persists regarding the validity or adequacy of each method. A longer emphasis recently is placed on the development of computer codes, rather then the derivation of simplified procedures, comprehensive studies which would shed more light on the importance of various approximations, or adequate justification on the numerical procedures used. Moreover, when studies of this type have been conducted, they have often been more concerned with the evaluation of specific computer programs (not always properly used) than with the investigations of methodologies.

Authors of the paper have been adopted the first procedure, namely, the analyze in a single step of the combined soil structure water system.



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Fig. 1 Typical cross section of a rubble-sloping breakwater (Constantza Port)

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Fig. 2 Breakwater-water-soil foundation - model (A) - modal shape 1



Fig. 3 Breakwater-water-soil foundation - model (A) - modal shape 2



Fig. 4 Breakwaterwater-soil foundation – additional masses of water - model (B) - modal shape 1

Fig. 5 Breakwaterwater-soil foundation – additional masses of water - model (B) - modal shape 2

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Mode	Model A		Mod	lel B	Model C	
Widde	f[Hz]	T[s]	f[Hz]	T[s]	f[Hz]	T[s]
1	0.384	2.604	0.378	2.646	0.423	2.364
2	0.577	1.733	0.431	2.320	0.513	1.949
3	0.623	1.605	0.481	2.079	0.559	1.789
4	0.662	1.511	0.599	1.669	0.626	1.597
5	0.701	1.427	0.628	1.592	0.723	1.383
6	0.802	1.247	0.629	1.590	0.769	1.300
7	0.957	1.045	0.663	1.508	0.792	1.263
8	0.967	1.034	0.675	1.481	0.795	1.258
9	1.105	0.905	0.714	1.401	0.854	1.171
10	1.132	0.883	0.719	1.391	0.863	1.159

Table 1. The eingenvalues for three analyzed models

5. Conclusions

The problem of dynamic analysis of rubblesloping breakwaters as presented is a small part of a large study dedicated to dynamic/seismic analysis of harbor structures. The work is in a beginning phase. The results obtained for determining the eingenvalues of breakwater taking into account the influence of soil foundation deformability and the influence of water, are important to clarify the influence of water mass, soil foundation on the dynamic response of breakwaters. For a massive structure such a rubble-mound embankment submersed in a great part under water and situated on deformable foundation is evidently that each numerical analysis has to be performed on the complete interaction system.

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Dynamic Shear Modulus Determination by Free Torsion Vibration Tests

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Abstract: The paper presents a set of laboratory tests performed in order to determine the most important dynamic characteristic of (sands), which is the dynamic shear modulus at low shear strain. The main purpose of the tests was to determine the shear modulus behavior with the sand moisture content in case of dynamic action. In the same time there were also considered the dry density and oscillation frequency effects. The tests were performed with a free torsion vibration device, considering five levels of moisture content (from the completely dry to the optimum moisture content conditions) at two levels for the dry density. The sand samples were freely vibrated by torsion in three different sequences of the oscillating frequency. From the measured relative displacement – time cyclogram the oscillation period, the damping ratio and the natural frequency were established. The shear modulus is given by $G_o = \rho v_s^2$, but since the density ρ and the shear velocity v_s are determined by the considered test parameters, the max shear modulus was obtained as a function $G_o = f(w, \rho_d, f_n)$.

Keywords: sand, free torsion vibration, low shear strains, dynamic shear modulus.

1. Introduction

The most important soil properties in dynamic analysis are the shear modulus G and the damping ratio D, they determining the shear stress – strain relationship.

There are several laboratory and field methods used for determining the shear modulus and the damping ratio. The main procedures may be summarized as follows (Hardin, Drnevich, 1971):

- Direct determination in laboratory of the hysteretic stress – strain relationship by means of triaxial compression tests or by simple or torsional shear tests, conducted under cyclic loading. This method is considered for moderate to relatively high strains, 0.01 % to 5 %;

- By forced vibration laboratory tests, involving the determination of resonant frequency and the measurement of the soil response at other frequencies. The tests include the application of longitudinal or torsional vibrations (resonant column test) to cylindrical samples, or shear vibrations to samples placed on a shaking table. This method is considered for determination of shear modulus and damping ratio at relatively low to moderate strains, 10^{-4} to 10^{-2} %;

- By free longitudinal or torsional vibration laboratory tests, involving the measurement of the decay in response of the soil sample. This method is considered for relatively low to moderately high strains, 10^{-3} to 1%;

- By field measurements of compression, shear and Rayleigh waves propagation velocities. This method is considered for low strains, 5×10^{-4} %, and does not provide values for the damping ratio.

By these tests the shear wave velocity v_s is established and the maximum shear modulus is directly related to it by the following expression:

$$G_o = \rho v_s^2 \tag{1.1}$$

where ρ is the mass density of the soil.

The present paper shows the procedure and the results of a series of Free Torsion Vibration tests performed at low strain (< 0.02 %) on a common type of sand in order to study the dynamic shear modulus variation with several important parameters: dry density, moisture content, natural frequency.

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2. Laboratory device and tests procedure presentation

The free torsion vibration (FTV) apparatus was developed in order to evaluate the dynamic behavior of soil. The dynamic shear modulus G and the damping ratio D can be defined directly from the test data for various shear strains, from 10^{-3} % to 1%.

The apparatus, presented schematic in the figure 2.1, consists of a modified triaxial cell in which 50 mm and 66 mm diameter samples can be tested. In the figure 2.2 a sand sample is fixed in the apparatus prepared for test performing. Unlike in the schematic presentation the chamber shell was not fixed to the bottom plate, because the confining pressure for the considered tests was relatively low (46 kPa) and applied by suction.



Figure 2.1 Schematic presentation of FTV apparatus

The upper end of the sample is attached to an inertia beam, with a length $l_{arm} = 95$ cm, via a friction free hydraulic bearing. The bottom support of the

sand sample is fixed to the plate and the sample's top is connected to the vertical shaft through which the torsion is applied. A small initial rotation is given to the inertia beam by the pusher pin and thus the sample is loaded by (static) torsional shear.



Figure 2.2 The FTV apparatus

By releasing the beam to move freely, the sample will behave in a damped vibrating spring mode. The torsional displacement is measured by two sensors, one for the an absolute and the other one for a relative measurement, placed symmetrically at a distance of 60 mm with respect to the sample axle. For all the performed tests the initial displacement was fixed at about 0.08 mm. It was set up to register the rotational displacement at every 0.002 seconds, for a total time period of 15 seconds. A sensor placed at the plunger's top also monitors the vertical displacement of the horizontal arm, which should be still during the test performing.

The sinusoidal relationship between the rotation angle and time is logged by a fast data recorder and then analyzed by integrated computer software.

The dynamic shear modulus G is determined from the frequency - rotation angle relationship, and the damping D is indicated by the decaying of the vibration amplitude. Changing driving mass quantity modifies the rotation frequency for a given sample. For the performed tests there were considered three levels for the oscillation frequency, each sample being tested in three steps: with no additional driving mass on the arm, with two symmetrical 5 kg driving masses, and with two symmetrical 10 kg driving masses. In addition to the driving masses there have to be considered the arm mass $m_{arm} = 4.733$ kg, the supports mass $m_{supp} = 2x481.7$ gr (acting each time), and the nuts mass of $m_{nut} = 2x82.6$ gr (acting together with the driving masses). The driving masses were placed at a distance $l_{att} = 25$ cm with respect to the sample axle.

The samples were built from a fine poorly graded (uniform) silty sand, with the following characteristics:

the specific density of solid $\rho_s = 2.65 \text{ gr/cm}^3$;

the minimum density $\rho_{min} = 1.411 \text{ gr/cm}^3$;

the maximum density $\rho_{max} = 1.829 \text{ gr/cm}^3$;

high

the maximum porosity (loosest state) $n_{max} = 46.36$ %; the minimum porosity (densest state) $n_{min} = 32.04$ %; about 66 % of the soil grains are less than 0.2 mm, about 12 % are less than 0.063 mm and about 5 % are less than 0.02 mm;

 $D_{10} = 0.053 \text{ mm}$, $D_{60} = 0.185 \text{ mm}$ and $D_{30} = 0.125 \text{ mm}$.

The maximum dry density - moisture content relationship of the considered sand was obtained by previously performed series of standard Proctor tests (Nicoara, 2001). The highest value for the maximum dry density was obtained about 1.682 gr/cm³ corresponding to the moisture contents of 0 % and about 12 % (the optimum moisture content), respectively. The maximum corresponding degree of compaction D_c, calculated as the percentage of the maximum dry density obtained by the Proctor test with respect to the maximum density, is about 92 %. The strongest effect of the capillary forces upon the compaction result is taking place for the moisture content of about 3.7 %, the maximum dry density obtained for this level being not more than $1.632 gr/cm^{3}$.

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In order to study the relationship of the FTV test results with water content at different levels of the dry density, the tests were performed for two values of the compaction degree. The first range of test samples was aimed to present a high degree of compaction D_c , of about 90 %, and the second range, for a loose situation, was aimed to present a degree of compaction D_c as low as about 84 %. In other words, it can be said that the high compacted sand samples were expected to present a dry density of about 1.65 gr/cm³, a porosity n of about 37.7 % and a relative density D_r of about 57.4 %, and the less compacted samples to present the same characteristics at about 1.54 gr/cm³, 41.9 % and 30.8 %.

 Table 2.1 Characteristics of sand samples for the FTV tests

density									
		diam.	m	m dry	W	dry density	Dc	n	Dr
test no.	test code	[cm]	[gr]	[gr]	[%]	[gr/cm3]	[%]	[%]	[%]
1	D90w0	6.62	818	818	0.0	1.639	89.6	38.2	57.3
2	D90w4	6.67	871	839	3.8	1.656	90.5	37.5	61.8
3	D90w6	6.71	890	840	6.0	1.638	89.6	38.2	57.1
4	D90w8	6.65	897	832	7.8	1.652	90.3	37.7	60.8
5	D90w12	6.68	930	831.8	11.8	1.637	89.5	38.2	56.8
			average :	832.2		1.644	89.9	37.9	58.8

low density								
	diam.	m	m dry	W	dry density	Dc	n	Dr
test no. test code	[cm]	[gr]	[gr]	[%]	[gr/cm3]	[%]	[%]	[%]
1 D84w0	6.6	754	754	0.0	1.520	83.1	42.6	26.0
2 D84w4	6.66	803	774.3	3.7	1.533	83.8	42.2	29.4
3 D84w6	6.65	818	772	6.0	1.533	83.8	42.2	29.4
4 D84w8	6.63	819	760	7.8	1.518	83.0	42.7	25.5
5 D84w12	6.62	846	757	11.8	1.517	82.9	42.8	25.1
	ave	erage :	763.5		1.524	83.3	42.5	27.1

Table 2.2 Characteristics of sand samples for the FTV tests

There were considered five levels for the moisture content values, 0, 4, 6, 8 and 12, respectively, and the desired degree of compaction was achieved by varying the number of layers and blows/layer for each specific sample. In order to obtain a proper material continuity, scrapping the tamped surface ensured the contact between layers. The sand sample contact to the top and bottom elements is ensured by the very rough surfaces of these elements.

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The high of the sample mold was fixed, 14.5 cm, but the sample diameter was measured each time.

The value of sample confining pressure, 46 kPa, was considered at a level which corresponds to the pressure determined by the static weight of a medium size vibratory roller, and it was achieved by suction through the bottom element of the sample. After applying the suction pressure, the two halves of the mold shell are removed and the torsion can be developed freely. The sample moisture content was properly established after the tests performance.

The obtained characteristics of the sand samples are presented in the table 2.1 (Nicoara, 2001). Even if there were not achieved the same desired values for all the samples, their spread from the average levels is less than 1 % for each parameter.

Considering that there was planed the following variation of the studied parameters: two levels of the sand samples dry density (about 1.65 and 1.54 gr/cm³), five levels for the moisture content for each dry density (0 and about 4, 6, 8 and 12 %), three levels for the oscillation frequency for each sample (determined by $m_{att} = 0$, 2x5 and 2x10 kg driving

mass), a total number of 30 free torsion vibration tests were performed.

In order to organize the work for both phases, tests performance and results interpretation, the tests were classified after the aimed values for the considered parameters: D(aimed compaction degree)w(aimed moisture content)m(level of driving mass).

The maximum were selected for each cycle and the oscillation period T was determined by averaging the time periods for the first well-defined cycles (about 15). The maximum shear strains of the sand sample for each cycle were calculated by the following formula:

$$\gamma_{\max} = \frac{(diam) \times (displ)}{2h \times (pos)}$$
(2.1)

where (diam) represents the sample diameter, (displ) represents the relative displacement, (pos) represents the relative sensor position with respect to the sample axle (60 mm) and h represents the sample height.

As for an unforced and damped one-degree-offreedom system, the logarithmic decrement, the damping ratio and the natural frequency were calculated by the following formulae (Das, 1983):

$$\delta = \ln \frac{\gamma_i}{\gamma_{i+1}}, \quad D = \frac{\delta}{\sqrt{\delta^2 + 4\pi^2}} \quad \text{and}$$
$$f_n = \frac{1}{T\sqrt{1 - D^2}} \tag{2.2}$$

where γ_i and γ_{i+1} are the average peak shear strains for two successive cycles, and D is calculated for an average δ from the first distinct cycles considered.

The maximum shear modulus is determined for each test result depending on the sample density and the shear wave velocity (formula 1.1). Since the sample density ρ and the shear wave velocity v_s are determined from the considered test parameters (i.e. moisture content w, dry density ρ_d and natural frequency f_n), the maximum shear modulus will be directed by the following function (Das, 1983):

$$G_o = (1+w)\rho_d \frac{4\pi^2 f_n^2 h^2}{\alpha^2}$$
(2.3)

where h is the sample's height and α is a parameter given by the ratio between the sample's polar moment of inertia J_s and the polar moment of inertia of the driving mass J_m:

$$\alpha tg\alpha = \frac{J_s}{J_m} \tag{2.4}$$

Driving mass = 0 kg

Low density: ρ_d about **1524** kg/m³

$$J_{s} = (1+w)\rho_{d}h\frac{\pi r^{4}}{2}$$
(2.5)
$$J_{m} = J_{arm} + J_{sup} + J_{att} =$$
$$= \frac{m_{arm}l_{arm}^{2}}{12} + m_{sup}l_{att}^{2} + (m_{att} + m_{nut})l_{att}^{2}$$
(2.6)

It is noticed that the parameter α is also depending both on the sample characteristics and the driving system. *r* represents the radius of the cylindrical sample.

3. Tests results and discussion

The values of the shear modulus resulted from all the tests are presented in the tables 3.1 to 3.6, organized after the driving mass, the density level and than depending on the moisture content.

Unfortunately, because of a malfunctioning of the testing device, the tests corresponding to 6 % moisture content for the low-density sand sample didn't supply a workable output.

Table 3.1 Tests D84w0m0 to D84w12m0

Low density.	p _d about 152 4	i kg/m	Table 3.1 Tests D84w0110 to D84w12110			
w	ρ dry	Т	D	f nat	v	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
0	1520	1.51E-01	5.06E-02	6.64	1.92E+02	5.62E+04
3.7	1533	1.29E-01	3.69E-02	7.76	2.16E+02	7.42E+04
6	1533	1.28E-01	3.21E-02	7.79	2.15E+02	7.54E+04
7.8	1518	1.38E-01	3.03E-02	7.27	2.01E+02	6.63E+04
11.8	1517	1.52E-01	3.22E-02	6.57	1.79E+02	5.46E+04

High density: ρ_d about **1644** kg/m³

Table 3.2 Tests D90w0m0 to D90w12m0

W	ρ dry	Т	D	f nat	v	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
0	1639	1.35E-01	3.39E-02	7.39	2.05E+02	6.88E+04
3.8	1656	1.26E-01	3.00E-02	7.96	2.13E+02	7.77E+04
6	1638	1.19E-01	4.53E-02	8.43	2.21E+02	8.51E+04
7.8	1652	1.20E-01	3.07E-02	8.36	2.21E+02	8.66E+04
11.8	1637	1.34E-01	3.58E-02	7.47	1.93E+02	6.78E+04

Driving mass	= 2x5 kg					
Low density:	ρ_d about 1524	4 kg/m^3	Table 3.3 Tests D84w0m5 to D84w12m5			
w	ρ dry	Т	D	f nat	v	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
0	1520	2.90E-01	4.39E-02	3.45	1.59E+02	3.84E+04
3.7	1533	2.55E-01	4.88E-02	3.93	1.74E+02	4.81E+04
7.8	1518	2.65E-01	3.35E-02	3.77	1.66E+02	4.52E+04
11.8	1517	2.86E-01	3.13E-02	3.50	1.52E+02	3.91E+04
High density:	ρ_d about 164	4 kg/m^3		Table 3.	4 Tests D90w0m5 to	D90w12m5
w	ρ dry	Т	D	f nat	v	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
	1 (20	0 41E 01		4 1 5	1.025.02	5 51 5 0 4

[/0]	[Kg/III5]	[3]	[-]	[1/5]	[111/3]	
 0	1639	2.41E-01	3.76E-02	4.15	1.83E+02	5.51E+04
3.8	1626	2.07E-01	2.89E-02	4.83	2.09E+02	7.41E+04
6	1638	2.08E-01	4.57E-02	4.80	2.00E+02	6.98E+04
7.8	1652	2.19E-01	3.22E-02	4.56	1.91E+02	6.52E+04
11.8	1637	2.44E-01	3.28E-02	4.10	1.68E+02	5.18E+04

Driving mass = 2x10 kg

Low density: ρ_d about 1524 kg/m ³					le 3.5 Tests D84w0n	n10 to D84w12m10
w	ρ dry	Т	D	f nat	v	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
0	1520	3.86E-01	4.79E-02	2.59	1.51E+02	3.47E+04
3.7	1533	3.33E-01	4.38E-02	3.01	1.68E+02	4.50E+04
7.8	1518	3.49E-01	3.03E-02	2.87	1.59E+02	4.16E+04
11.8	1517	3.81E-01	3.25E-02	2.63	1.44E+02	3.51E+04

High density:	ρ_d about 164	4 kg/m^3	Table 3	Table 3.6 Tests D90w0m10 to D90w12m10		
W	ρ dry	Т	D	f nat	V	Go
[%]	[kg/m3]	[s]	[-]	[1/s]	[m/s]	[kN/m2]
0	1639	3.22E-01	3.38E-02	3.11	1.73E+02	4.92E+04
3.8	1626	2.96E-01	3.22E-02	3.38	1.85E+02	5.77E+04
6	1638	2.89E-01	4.37E-02	3.47	1.83E+02	5.81E+04
7.8	1652	2.86E-01	3.39E-02	3.50	1.85E+02	6.13E+04
11.8	1637	3.09E-01	2.46E-02	3.23	1.68E+02	5.14E+04

The natural frequency variation with moisture content, presented with respect to the level of dry density, is shown in the figure 3.1. As it was expected, the natural frequency presents little higher values for the denser samples, and also it decreases significantly with the quantity of the driving mass. The largest difference between the high and low density situations appears for the 2x5 kg driving mass, about 23 % in the average values of the natural frequency, and it is about 21 % for the 2x10 kg and 10 % for the 0 kg. The frequency decrease from the 0 kg driving mass situation to the 2x5 kg situation is about 46 %, and to the 2x10 kg situation is about 59 %, while in between the 2x5 kg and the 2x10 situations the decrease is about 24 %.



Figure 3.1 Natural frequency variation with moisture content

Studying the influence of moisture content, it is noticed that its greatest influence upon the natural frequency appears for the 0 kg driving mass, meaning at the largest value of the frequency. It is also noticed that in general, for most of the curves in the figure 3.1, the largest value of natural frequency appears for low moisture content (about 4 %) and its lowest value for dry sand or for the highest moisture content. This is the consequence of the sand stiffening due to the capillary forces that are better developed at low levels of moisture content. Regarding the values obtained for the situation of high density at 0 kg and 2x10 kg driving mass in comparison with the other four situations, it is possible that the test results were not accurately enough. With this reserving note, but still considering that at a level of about 4 % moisture content the natural frequency presents a maximum, the increase with respect to the dry situation is about 7.7 % for high density and 16.9 % for low density in case of 0 kg driving mass, about 16.4 % and 13.9 % in case of 2x5 kg, and about 8.7 % and 16.2 % in case of 2x10kg.

Following the pattern of the natural frequency variation but with a more pronounced influence, the obtained variation of shear modulus with moisture content and with respect to the quantity of driving mass is presented in the figure 3.2. It is also noticed that the shear modulus increases with density for a specific driving mass and the tendency for flatter curves, with lower values, at larger driving masses. This means that for higher frequencies, the moisture content influence upon the shear modulus may be more significant. It also appears that in general the shear modulus presents a maximum for the low level of moisture content of about 4 %. Taking into consideration that there are no measurements for the first part of the Go - w relationship (between dry situation and 4 % moisture content), and also that the results at high density, 0 and 2x10 kg, do not follow the general tendency, it can not be said if there isn't a much more significant increase of the shear modulus for a moisture content at 2 - 3%. In the interval between about 4 % and about 12 %, the shear modulus decreases in general with the moisture content. The shear modulus values obtained for the largest considered moisture content (about 12 %) are almost equal with the values determined for the dry conditions.



Figure 3.2 Maximum shear modulus variation with moisture content

With respect to the dry situation, the shear modulus increases with about 12.9 % at high density and about 32 % at low density for the 0 kg driving mass case, about 34.5 % and 25.3 % for the 2x5 kg case, and about 17.3 % and 29.7 % for the 2x10 kg case. The largest $G_{o(4\%)}/G_{o(dry)}$ ratio for the low density situations is about 1.32 and appears for the 0 kg driving mass. The next lower value is about 1.3

for the 2x10 kg driving mass and than it is about 1.25 for the 2x5 kg situation. In case of high density the largest value appears to be for the 2x5 kg situation (about 1.34) but it is possible that the results obtained for the other two situations are not quite accurate. The shear modulus development with frequency is presented in the figure 3.5. The added polynomial trend lines fit for the considered range of relatively low frequencies, but it cannot be said if these trends maintain for higher frequencies.



Figure 3.3 Shear modulus – natural frequency obtained relationship

4. Conclusions

The obtained test results definitely show the important influence of the moisture content, dry density and oscillation frequency upon the maximum shear modulus behavior. In the same time it is concluded that, by using results of FTV tests, it is possible to estimate the general behavior trend of the shear modulus with the considered parameters for a specific sand.

In order to develop an acceptable empirical model a larger number of tests, for more levels of moisture content (especially in the area of 0 to 6 %) and dry density, are required. The tests should be also extended for several common used types of sand.

For reaching proper test results, the homogeneity of the samples with respect to the

moisture content and dry density should be checked for each sample.

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Estimating the Bearing Capacity to Compression Axial Forces in the Case of in Situ Friction Piles using the Results of Test Loading

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Abstract: The necessity of in situ test loading on piles for buildings having indirect foundations is well known. In most of the cases the tested piles will remain within the future foundation. This is why the axial loading cannot reach the critical value, for which the settlements are no longer stabilized. The computation models presented in this article help the design engineers to estimate the value of the critical loading force, needed for the dimensioning of the foundation

Keywords: In situ friction piles, bearing capacity.

1. Introduction

The evaluation by means of "in-situ" tests of the axial and transversal bearing capacity for indirect pile foundations is extremely useful for design engineers. Usually, due to economical reasons, these tests are made on piles that are to become part of the foundation of the future building. That is why the tests can not be made using loading forces having values up to the expected critical load because it can affect the pile and, of course, the foundation.

In this case the loading steps are stopped within the range of values for the forces corresponding to the serviceability loading of the building and they can be considered as nondestructive test. How to determine then the values for the critical loading that the designer needs in order to choose (verify) the number of piles? The answer to this question is that the value of the critical loading or the bearing capacity can be determined using results of non-destructive tests, from interpretation criterions which anticipate with mathematical laws the behavior of the piles until failure.

The present article shows such criterions and also the results obtained for two case studies.

2. Methods used in the interpretation of the experimental data

By the static compression loading we try to determine, by measurements, the relationship between the intensity of the loading and the value of the piercing of the loaded pile in the soil, as well as establishing the development in time of the settlement under the action of each loading step.

2.1. The method given in STAS 2561/2-81

The loading is made in steps of 1/15 or 1/10 from the estimated final loading. Each step will be maintained until the deformation is stabilized, which is considered to be reached when the settlement for 4 consecutive readings is smaller than 0.1mm. The loading continues until the limit state is reached, which is defined to be the loading for which one of the following conditions is fulfilled:

a) the value of the axial loading for which the mean settlement is greater than 1/10 from the pile's diameter or the its side(in cm).

$$d \ge \frac{d}{10} \tag{1}$$

b) the value of the axial loading for which the value of the settlements for four consecutive readings

at 30 minutes interval, during 24 hours, is greater than 0.1mm.

The results of the measurements are plotted on a graph which includes the variation of

the stabilized settlement, s in time, t, and with the loading, P, as well as the variation of the load P in time (Fig. 1).



Fig.1 Graphical representation of the results of an axial test loading.

2.2. Van der Veen Criterion.

Based on the readings of many tests, the following relationship between loading and settlement is admitted:

$$P = P_{cr} \cdot \left(l - e^{-\alpha \cdot \Delta} \right) \tag{2}$$

from which by logarithmation we obtain:

$$\Delta = m \cdot ln \left(l - \frac{P}{P_{cr}} \right)$$
(3)

where:
$$m = -\frac{1}{\alpha}$$
 (4)

The measurement results $\Delta = f(P)$ are represented in a semi-logarithmic scale having on abscissa the values $ln\left(I - \frac{P}{P_{cr}}\right)$. For the value of

R a linear diagram is obtained. The computation of the bearing capacity will be made starting with the results from the static loading test (Fig. 1). For different values chosen for P we represent in a semi-logarithmic coordinate scale the relationship (3), the value of R being the one for which the graphical representation is a straight line. (Fig. 2)



Fig. 2 Determination of the bearing capacity R using the Van der Veen criterion.

2.3.Parez Criterion

We represent the variation with time of the settlements for each loading step based on the

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measurements taken. The relationship $\Delta = f(t)$ has for a vast range of soils the semi-logarithmic shape.

$$\Delta = \Delta_0 + \delta \cdot lg t \tag{5}$$

The slope δ (Fig. 3a) of the line in the semilogarithmic representation expresses the



Fig. 3 Determining P_{cap} by means of Parez criterion

2.4. Szechy Criterion

Prof. C. Szechy proposes the following procedure for determining the bearing capacity of the piles:

-the piles will be loaded in equal steps that are maintained until the settlements are stabilized; for each loading step after taking the values of the stabilized settlements, the pile is unloaded and the remanent settlement will be measured.

-for each loading cycle, based on the measurements, the elastic and remanent settlement will be computed (Fig. 4); for each loading step the ϵ coefficient is expressed as percentage with the relationship:

$$\varepsilon = \frac{\delta e_i - \delta e_{i-l}}{\delta p_i - \delta p_{i-l}}$$
(6)
in which:

 $-\delta ei$ is the elastic settlement for the i loading cycle,

 $-\delta ei$ -1 is the elastic settlement for the i-1 loading cycle,

deformation speed of the pile under constant loading.

of the pile P_{cap} , the slope variation with the axial load P will be represented (Fig. 3b). The value of the load for which the $\delta = f(P)$ graphic changes the slope

In order to determine the capable capacity

represents P_{cap} of the pile.

 $-\delta pi$ is the remanent settlement for the i loading cycle,

- δpi -1 is the remanent settlement for the i-1 loading cycle.

The value of the loading that corresponds to the maximum value of the ε coefficient represents the bearing capacity R of the pile. (Fig. 4).

The procedure, although justified from the theoretical point of view, has the downside that it requires a long time for obtaining the stabilized settlements under each loading step and is very hard to be used for common purposes. But, associated with another method of extrapolation of the values of stabilized settlements for each loading step, the Szechy procedure can be brought closer to used loading techniques.



Fig. 4: Determination of the bearing capacity R with Szechy criterion

2.5. The hyperbolic model

In this case we the results are represented in a graph as the one shown in Figure 5. The slope change shows us the amount of the loading taken by the lateral surface by friction and by the tip of the pile. We can notice that the first part of the linear representation refers to the lateral surface and for second part we add the value taken by the tip of the pile.



Starting from the variation of the settlement with the loading force, we change the coordinate system, and we will have on the abscissa the ratio between the settlement and the force, and on the ordinate the values of the settlements. The slope change represents the moment in which the resistance on the tip of the pile is fully mobilized. From the equation of the line one can find P_{cr} . In the case of an end bearing pile, the graph will not emphasize a slope change, as the resistance on the tip is mobilized from the very beginning. In the case of a friction pile, from the equation of the line for the first part of the graph one can find out the part taken by friction by the lateral surface.

3. Case study

Pile 1

Pile information: it is a Benoto pile having a diameter of 1080mmm and a length of 10 meters. The borehole reveals the following stratification:

- at the surface, for a depth of about 4m it was found a silty clay, a little sandy at the base, grayish yellow, plastic hard.

- Under this level, for about another 4 meters, the borehole shows the presence of sand, silty sand with clay intrusions.

- It follows a brown clay, with ferruginous concretions at its base.

In this horizon the base of the pile was stopped.

According with the Romanian standards the bearing capacity of this pile it results a critical bearing capacity of P_{cr} =452t.

According with the Van der Veen Criterion one can observe that the diagram becomes a line for a loading of P_{cr} =385t.



Fig. 6: Applying the Van der Veen criterion in order to find out the bearing capacity of the pile

In the case of the Parez criterion, we obtain the value for a capable capacity, and it has a smaller value than the one given by other criterions. This method can show technological mistakes. The results can be modeled function of the available data about the pile execution, the experience (in this type of problems) - and lead to different results (conclusions). For the same pile, different values of the P_{fluaj} are obtained, as it is shown in the following graphs.



Although this method takes into consideration many factors, and it might be more difficult to apply, it is important the fact that eventual mistakes in execution can be observed.

By the procedure Prof. C. Szechy proposes the bearing capacity of the pile is estimated at a value of 225t..



The hyperbolic model shows, beside the value of P_{cr} , the values for the maximum loading taken by the tip of the pile P_v , and for the maximum loading taken by the lateral surface P₁. The graph has an initial part where the loads are exclusively taken by the lateral surface of the pile. The slope changes when the load is also taken by the pile's end. So the following values are obtained:

Table 1: Values for the maximum loading on the tip and on the lateral surface and also P_{cr}



Fig. 9: Determination of P_{cr} with the hyperbolic model . .

. .

Table 2. The results of analytical interpretations								
CRITERION	Type of force	Value (tf)						
STAS 2561/2	P _{cr}	452						
Van der Veen	R	385						
Parez	P _{cap}	270						
Szechy	R	225						
Hyperbolic model	P _{cr}	492						

For the next pile, the same methods were used. Pile information: this pile has a length of 16.50m. The pile is a Benoto pile having the diameter of 1080mm. The borehole shows the following stratification:

- in surface, 1.00m thick is a silty clay, grayish yellow. - between 5.00 - 11.00m the borehole shows the presence of a clayey silt plastic hard, followed by a

medium sand with 1m in thickness. - between 12.00 – 16.00it was found a gravish yellow clay - plastic hard.

Under this level the borehole showed a medium sand, the pile's end being in this layer.

According with the Romanian standards the bearing capacity of this pile it results a critical bearing capacity of P_{cr} =480t.

According with the Van der Veen Criterion one can observe that the diagram straightens for a loading of P_{cr} =430t



Fig. 10: Applying the Van der Veen criterion in order to find out the bearing capacity of the pile.

In the case of Parez criterion we have $P_{cap}=368$ tf.



By the procedure Prof. C. Szechy proposes the bearing capacity of the pile is estimated at a value of 325t.



The hyperbolic model shows in this case an end bearing pile, as the graph does not emphasize a slope change. The value for the critical force is in this case P_{cr} =560tf.

Table 3 Values for the maximum loading on the tip and on the lateral surface and also P_{cr}



Fig. 13: Determination of P_{cr} with the hyperbolic model
Table 4 The results of analytical interpretations							
CRITERION	Type of force	Value (tf)					
STAS 2561/2	P _{cr}	480					
Van der Veen	R	430					
Parez	P _{cap}	368					
Szechy	R	325					
Hyperbolic model	P _{cr}	560					

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In the case of the next pile, it was not possible to apply the Szechy criterion due to the lack of data needed in order to model the results. However, the other criterions offer us the possibility of interpretation of the information and we can obtain relevant results.

Pile information: pile's length is of 25 meters and it has a diameter of 1200mm. It was made under slurry and due to improper cleaning of the hole it lead to fake settlements. This was easy to see in the graphical representation of the data modeled with Parez criterion (fig 15).

According with the Van der Veen Criterion one can observe that the diagram straightens for a loading of P_{cr} =890t





For the Parez criterion it results a value P_{cap} =880tf.



For the hyperbolic model, the results are presented in table 5. As the shape of the graph shows, this pile is a friction pile.





Table 6 The results of analytical interpretations

CRITERION	Type of force	Value (tf)					
STAS 2561/2	P _{cr}	900					
Van der Veen	R	890					
Parez	P _{cap}	880					
Hyperbolic model	P _{cr}	1117					

4. Conclusions

The means of interpretation of the results from the present article shows three important aspects such:

the possibility of determining the axial bearing capacity of the piles using non-destructive methods.
the possibility of evaluation of the components of the critical axial loads such as the

end bearing component and the one taken by the

lateral surface of the pile by friction.

- the identification of possible execution defects due to bentonitic mud deposits formed at the tip of the pile (bottom of the pile's drilling), which can cause "fake" settlements for the first loading steps.

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Evaluation Methods and Technologies of the Quality Characteristics of Buildings

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Abstract: The paper work presents a study concerning some of the most used methods of evaluation of the buildings quality characteristics. The buildings quality estimation can be done through: measuring the characteristics, the comparison of the synthetic indicators of the quality, the damping of the defects and so on. The control of the quality has to include the stages of designing, execution and utilization of the construction. The present study details a few methods of estimating the quality characteristics of buildings.

Keywords: quality indicators, quality characteristics

1. Introduction

The quality of a construction object is a complex concept which includes conception factors of designing, execution and utilization. If we had tried to synthesize these factors in a mathematical formula we should use a quality estimation relation "C" with the form:

$$C = f(p) \cdot f(e) \cdot u \tag{1}$$

Where:

- f(p) the quality of the designing indicator of the quality characteristics possible through the conception designing documentation
- f(e) the quality of the execution indicator of the conformity execution level to the technical documentation stipulations (the technical *execution project*)
- u the utilization quality defined by the customer requests.

Besides these factors, "the quality" in its complexity also includes other conditions: technique, economic, aesthetic, ergonomic, of fiability and mentenability and so on.

The quality characteristics of a building object can be grouped in the following categories:

- functional – regarding the that the object has (dwelling place, fabrication hall, smoke chimney, barrage and so on)

- stuff – regarding the material used and the technological processes of execution.

- psychosensory and social – regarding the aesthetical side, noise, vibrations, thermal isolation, hydrofuge, energy economy, hygiene, environment protection and so on;

- resistance, stability, safe in exploitation and at

fire.

It results thus that to determine the quality of a construction object; we need to determine as many characteristics as we can by noticing it, trying and measuring it.

Qualimetry is the discipline that measures the quality characteristics and estimating the quality.

2. Methods of quality estimation

We present a few methods of quality estimation, like: the measuring of the characteristics, the comparison of the synthetic indicators of the quality, the damping of the defects.

a) The method of measuring the characteristic

This method can be: direct – when a characteristic can be compared to a standard (dimensions weight) or indirect – when the value of the characteristic can be obtained with other measures.

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The measured values, reported to the performances in the field indicate the quality level.

b) The method of comparison the synthetic indicators

The quality indicators generally represent the quantitative expression of the construction objects characteristics, established concerning the conditions of their designing, execution and exploitation. At every quality indicator there is a characteristic that can be measured with the technical means existent

The quality indicator can be simple when it referees to one characteristic and complex when there are more. When the indicator serves to estimate the quality by comparison, is named base indicator.

The correct estimation of the construction objects quality can be made only by systematic approaching of the indicators, thus:

- by the realization stages of the object (designing, execution, exploitation)

- by defining elements of the quality

- depending on the efficiency of the quality (the relation effort—effect).

Taking in account the realization stages of the construction object we can determine the following subgroups of indicators: the designing activity indicators, the execution indicators, the marketing indicators, the exploitation indicators. We can determine the following categories of the quality indicators:

- utilization indicators (functional

characteristics)

- aesthetical indicators (psychosensory characteristics)

- social indicators (social characteristics)

social indicators (social characteristics)
 economic indicators (stuff characteristics)

- technique indicators (stern characteristics) - technique indicators (strength, stability, safe in exploitation and at fire characteristics)

One can also do other classification types but the determination of any quality indicators of a product depends firstly by the complexity and the way of utilization the product. Depending on this the construction object is a complex one which gradually wears out in time use differenced by the industrial products which wear out in totality in their use stage. Regarding the informative source, the quality indicators can be:

- planned indicators - determined after:

- \checkmark intern normative and technique
 - documentation
- ✓ international organizations documentation (I.S.O)

- effective indicators – obtained by processing the data resulted after the measure of the quality characteristics and from the statistical system of evidence of the institution, which reflect different aspects of the quality.

The relation between the effective indicator value and that of the planned indicator m, gives the "quality sign" which shows us the scientifically realization degree of the quality and the dynamic improvement of the quality products.

For complex products (building case) one can determine different quality synthetic indicators "Ci" with the reference value "Cio", based on performances and also the weight "qi", noticed after the importance of the characteristics. We can express in analytical way the quality level:

$$Q = \sum_{i=1}^{n} \frac{c_i}{c_{i0}} q_i$$
 (2)

We can consider the price of the analysed product compared to a reference product "p" having:

$$Q = \frac{p_0}{p} \sum_{i=0}^{n} \frac{C_i}{C_{i0}} q_i$$
(3)

c) The method of damping defects – known as the name of "undeserved" from the French demerite = lack of worth, especially recommended for complex building objects.

The principle of the method consists in a classification of the defects (table 1) and an adequate damping system adopted.(table 2)

A product can have one or more defects from different categories. Arbitrary we adapt a damping system. If we note the number of the defects on categories, with : n_c , n_p , n_s , n_m and the damping with: q_c , q_p , q_s , q_m we determine the undeserved of a volume sample N in relation :

$$D = \frac{n_c q_c + n_p q_p + n_s q_s + n_m q_m}{N}$$
(2)

Table 1.The defects classification on the consequences

Defects classification	Symbol	Defects definition
Critical	С	Defect that stops the function run, making damages or bad accidents. It generates reclamations.
Principal	Р	It reduces the utilization of the product, making dissatisfaction to the owner. Generally, it produces reclamations.
Secondary	S	It doesn't affect too much the utilization possibilities. It is notice but doesn't generate reclamations.
Minor	М	It doesn't reduce the utilization possibility and it hasn't inconvenient. The beneficiaries do not grasp.

We have *specification undeserved* – where the damping is on the percent of defects in every category and *accepting undeserved* – where the damping is based on the possibility of accepting a defect by the beneficiary. The undeserved can be objective when its value Do is fixed on hypothesis referring to the admissible frequency of the defects or on the average value obtained in a certain period of time.

ac	Damping Scale						
Dampin system	m	S	р	с			
Ι	1	3	5	10			
II	1	5	25	125			
III	1	10	50	100			
IV	1	10	100	1000			

Table 2 Damping system for defects

Relation
$$I_p = \frac{D}{D_0}$$

defines the undeserved sign which has the following significations:

Ip = 1 – the quality equal to that of reference

Ip < 1 – the quality is superior to that of reference

Ip > 1 – the quality is inferior to that of reference. The undeserved is good for any complete control method or by sounding on the execution way or at reception and can be followed with a record named "the quality journal".

Although is not the most precise method of control is very rapidly, easy to use and permits the formulation of conclusions and the taking of immediate measures for removing defects.

d) The ELECTRE method (Elimination et Chois Traduissant la Realite), elaborated by a group of researchers from the French Math Society SEMA -- permits the comparison of types of activities, described by many quantitative and qualitative indicators, to establish a hierarchy of these types.

The different expression of the indicators and parameters that characterize the types of activities has the effect of transforming their sizes in marks (unsatisfactory, satisfactory, well, very well), which permit a more simple comparison, intuitive, of the indicators and types that are to be analyzed. Following the various importance of the indicators we difference them by according a coefficient of importance (*Ki*), sizes higher or equal to one. Using different "noticing scales" (like unsatisfactory = 1, satisfactory = 3, well = 5, very well = 7) the marks of every indicator are influenced by the sizes of the importance coefficients. So we can obtain homogeneous sizes, abstract, for any type of indicator.

The method can be used in choosing the best level of quality by comparison more quality characteristics of various levels possible. It has a large area of application and can obtain good results in quality field.

e) The method " zero defects" -- has been utilized for the first time in 1960 by the founders of "Apollo" program, took then by the Japanese companies, South Korea and others, which improved it especially regarding the role of the execution personnel in eliminating defects. The main concept of the method is contrary to that based on the acceptance of a certain "normal" percent of defects, which once respected admits the conclusion of approaching by the wanted objective.

To combat the idea of passive acceptance of a percent of defects at the limit do not means after this method that all the defects are immediately eliminated but the idea that as long as they exist there will be efforts to get rid of them.

This method represents an essential ethical change in the philosophy of quality, in the countenance of quality control doing. Organized until now as an independent activity from the execution compartment and invested with prerogatives to repress any deviation from the documents (projects, standards, rules, tasks) appointing the quality level, it is integrated in the process of obtaining quality, and its ethic becomes the cause of a commune conscience and will responsible of all the execution factors of involving in following the objective.

The method "zero defects", extended on the whole process of existence of the building object (design, execution, exploitation) has at its base the full integration of the control in these stages until the auto control made by every participant.

3. Conclusions

Regarding the evaluation methods of quality characteristics in designing, execution and exploitation of buildings can be:

The control of the buildings quality is a complex process because the object of construction is a complex product.

To determine some quality indicators for different stages of existence of the object (design, execution, exploitation) has in its view the complexity of the object and the possibility of being measured and estimated as much objective as it is possible.

One can see that most of the indicators are well known sizes and studied by other disciplines, how is the compression resistance of the concrete, arrows, fissure, static load and so on.

To study these quality indicators one can use classic methods as well as modern ones which involve the mathematical statistics and the processing of data on the computer.

The managers from the whole world are searching for new methods to study the quality characteristics to improve the buildings quality. The Japanese searched from the very beginning a new road to build constructions of quality and not to obtain the quality by removing the defects. They discovered this way, the statistic control of the production processes developed by Deming. The American teacher had been invited in Japan and from their collaboration resulted the system of quality assurance that was introduced in many countries since then.

Speaking about buildings industry we have to notice that the quality of a building is also given by the stuff quality that has to be improved.

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Evaluation of the Seismic Torsion Effects on Structures

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Abstract: This study brings into attention some series of tests carried out on the shaking table at Laboratory for Earthquake Engineering – National Technical University, Athens [1]. There are presented the effects of the torsional coupling on the behavior of one-story steel structures. The models are designed to be earthquake resistant according to the provisions of the Eurocodes EC3 and EC8. The design of the models under seismic loads was carried out by means of the dynamic analysis in the linear elastic domain. The four series of models are tested on the shaking table. First it is considered the symmetrical model with respect to the major axis. The torsional coupling is realized by some series of models in which: the stiffness center has a fixed position and the center of the mass is mobile and the mass is disposed symmetrically and the stiffness center is placed unsymmetrically. Sinusoidal sweep action and synthetic accelerograms, compatible with the design spectra given by the EC8, have been employed as input motions. The paper presents the results of these tests and performs an analysis of the methods used for the approximately torsional analysis.

Keywords: seismic torsion response

1. Introduction

The design procedure for the torsional effects should consider a number of factors which are, with present knowledge, both difficult to predict and evaluate. These include the influence of the lateral and torsional frequencies, the importance of the adequate design of vertical resisting elements on both sides of centre of the stiffness, accidental eccentricity effects due to a variety of causes, and inelastic structural response. The last of these effects has been found to have a significant influence on the torsional structural response to severe earthquakes, fundamentally altering dynamic response behaviour compared with systems responding in the elastic range [2].

Buildings subjected to earthquakes undergo a lateral (translational) motion in addition to the torsional (rotational) one simultaneously. Such motions are due to [3]:

- i. *natural torsion* in buildings with asymmetric plan (an eccentricity between the centre of mass and centre or rigidity) or asymmetry in stiffness and asymmetry in strength;
- **ii.** *accidental torsion* in all buildings, even those with symmetric plan.

It is evident, when reviewing code provisions [4], that current earthquake-resistant design for torsional effects, whether based on the equivalent static force procedure or the modal analysis one, relies largely on linear elastic theory. In general, a nonlinear inelastic analysis is not required by present code regulations, even for buildings with torsional irregularities, although it may be recommended in some special cases.

2. Methods Proposed in the EUROCODE 8

The guiding principles governing this conceptual design against seismic hazard are:

- i. structural simplicity;
- ii. uniformity and symmetry;
- iii. bi-directional resistance and stiffness;
- iv. torsional resistance and stiffness;
- v. diaphragmatic effect at story level;
- vi. adequate foundation.

With the purpose of seismic design, building structures are distinguished as regular and non-regular. The criteria for regularity in plan and in elevation are given in 2.2.2, 2.2.3 and Annex A [4].

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The above classification implies the following aspects concerning the seismic design:

- 1. The structural model: a) planar model; b) spatial model.
- 2. The method of analysis: a) simplified modal

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response spectrum analysis; b) multi-modal response spectrum analysis.

The allowed simplification of the torsional effects of the EC8 is given in the Table 1.

Type of analysis	Type of structure					
	Two way symmetrical structures	Non-symmetrical structures				
Simplified analysis (clause	$e = \pm e_1 (3.1), \qquad (\text{clause } 3.2)$	$\boldsymbol{e}_{max} = \boldsymbol{e}_0 + \boldsymbol{e}_1 + \boldsymbol{e}_2 \tag{A3}$				
3.3.2).	or $\delta = 1 + 0.6 \ x/L_a$ (3.6)	$\boldsymbol{e}_{min} = \boldsymbol{e}_0 - \boldsymbol{e}_1 \tag{A4}$				
Criteria for regularity: a. 2.2.2; 2.2.3 and Eq.(3.2) or b. 2.2.3; A1 and Eq. (3.2)	(clause 3.3.2.4) Planar model	e_2 (A1) and (A2), (clause A4, fig.A1) Planar model				
Multi-modal approximate analysis	$e_{max} = e_0 + e_1 + e_2$	(A3)				
(Annex A)	$e_{min} = e_0 - e_l$	(A4)				
Criteria for regularity: A1	e_2 (A1) and (A2), (clause A4, fig	.A1)				
	Planar or spatial model	·				

In this Table e represents the eccentricity of the horizontal seismic force, F_i (s. Fig. 1A); the design storey torque is the product of the force F_i by the design eccentricity, e;

 e_0 - actual eccentricity between the CS and CM; e_1 - accidental eccentricity:

$$e_1 = \pm 0.05 L \tag{1}$$

 e_2 - additional eccentricity taking into account the dynamic effect of the simultaneous translational and torsional vibrations.

The equations for calculating e_2 are as follows, where the lower of the two values is taken:

$$e_2 = 0.1(L+B) \sqrt{10 e_0} \le 0.1 (L+B)$$
 (2)

$$e_{2} = \frac{1}{2e_{0}} \left[l_{s}^{2} - e_{o}^{2} - r^{2} + \sqrt{(l_{s}^{2} + e_{0}^{2} - r^{2})^{2} + 4e_{0}^{2}r^{2}} \right]$$
(3)

where:

$$l_s^2 = \frac{1}{12} \left(L^2 + B^2 \right) \tag{4}$$

is the square of radius of gyration (mass radius of gyration of the floor of the roof slab about CM):

$$r^2 = \frac{K_0}{K_y} \tag{5}$$

where K_0 is total torsional stiffness calculated about CS and K_y - the total story stiffness in the lateral direction (OY).

Therefore, EC8 limits the application of the static torsional design procedure specifying certain regularity calculations. If any one of these conditions is not satisfied, the static design procedure is not permitted and generally a multimodal response spectrum analysis is recommended. The code requires that the plan configuration of buildings should be compact and regular and that regularity should be maintained in the vertical configuration such that the stiffness and mass properties he approximately uniform distributed over the building height. EC8 requires that the centres of stiffness of every story should lie approximately on a vertical line and the centre of mass of the individual floor slabs also line on a vertical line.

Further more, EC8 requires that appropriate horizontal regularity must be maintained such that at any given storey the eccentricity, e_0 should not exceed 15% of the torsional radius, r, defined as above. Application of this condition restricts the static design procedure to asymmetric buildings having small eccentricity, even for buildings having high torsional stiffness.

The accidental torsional effects may be determined - in accordance with EC8 (ch. 3.2 and

(3.3.3.3) - using and additional accidental eccentricity, as previously mentioned in Eq.(1).

As a general rule, the expression of the accidental eccentricity is:

$$e_l = \pm \beta L \tag{6}$$

where β is the coefficient defining the accidental eccentricity as a function of *L*.

The EC8 primary (e_{max}) and secondary (e_{min}) design eccentricities are (Fig.1):

$$e_{prim.} = e_{max} = e_f + e_a = \alpha_f e_0 + \beta L \tag{7}$$

$$e_{sec.} = e_{min} = e_r + e_a = \alpha_r e_0 - \beta L \tag{8}$$

Figs. 2 and 3 illustrate the variation of the primary eccentricity amplification factor, α_i , namely:

 $\langle 1 \rangle$ Chandler and Hutchinson [5]:

$$\frac{e_f}{L} = \alpha_f \frac{e_0}{L} = \left(2.6 - 3.6 \frac{e_0}{L}\right) \frac{e_0}{L}$$
(9)

 $\langle 2 \rangle$ New Zeeland Code, 1976 [5]:

$$\alpha_f \frac{e_0}{L} = \left(1.7 - \frac{e_0}{L}\right) \frac{e_0}{L}$$
(10)

 $\langle 3 \rangle$ Dampsey and Tso [5]:

$$\alpha_{f} \frac{e_{0}}{L} = 3 \frac{e_{0}}{L} \quad \text{for } \frac{e_{0}}{L} \le 0.04$$

$$\alpha_{f} \frac{e_{0}}{L} = 0.086 + 0.85 \frac{e_{0}}{L} \quad \text{for } \frac{e_{0}}{L} \ge 0.04$$
(11)

 $\langle 4 \rangle$ Pekau and Rutenberg [5]:

$$\alpha_{f} \frac{e_{0}}{L} = \frac{2.105A^{0.5} \frac{e_{0}}{L}}{A + \frac{e_{0}}{L}} \Omega^{0.1}$$

$$A = \left[0.5 - 4 \left(\frac{e_{0}}{L}\right)^{2} \right]^{0.5}$$

$$\Omega = 1$$
(12)

$$\langle 5 \rangle, \langle 6 \rangle, \langle 7 \rangle$$
:

$$\alpha_f \frac{e_0}{L} = 0.5 \frac{e_0}{L} ; \ \alpha_f \frac{e_0}{L} = 1.0 \frac{e_0}{L} ; \ \alpha_f \frac{e_0}{L} = 1.5 \frac{e_0}{L}$$
(13)

 $\langle 8 \rangle, \langle 9 \rangle, \langle 10 \rangle, \text{ EC8 [4]:}$

$$\alpha_f = 1 + \frac{e_2}{e_0}$$
 for $\frac{L}{B} = 1, 2, \text{ or } 4^{-1}$ (14)

The comparison of the above results shows that EC8 code is generally more restrictive than the National ones [4].

Dynamic analysis performed by many authors (Fig.2) as well as the experimental results presented in this paper (Fig.3) show that for relatively large eccentricities, $e_0 /L \ge 0.1$, the primary amplification coefficient, α_f , has a constant value, equal to 1.5; for the range of eccentricities $e_0 /L < 0.1$, the value of the amplification coefficient should be increased and limited. The graphical representation may be diversified as a function of the ratio L/B.

3. Experimental Verifications

The gained experience from the past earthquakes strongly underline that besides lateral resistance and stiffness, structures have to provide adequate torsion resistance and stiffness.

The last revision of EC8 put a prescription in order to avoid the development of torsional motions which tend to stress, in a non-uniform way, the different structural elements. The arrangements in which the main resisting elements are distributed closely to the building periphery present clear advantages.

The design of the models intended to realize series of systems torsionaly un-coupled (e = 0) and a system torsionaly coupled $(e \neq 0)$ which allow to focus on torsional coupling due to mass and rigidity eccentricities. The models are not scaled to a particular real structure.

The tested models are designed by the specific rules according to EC8 and EC2.

The models are three-dimensional steel frame type, 2.4x2.4 m (span) and 2.0 m height, with different cross-sections of the columns (Fig.4).

The dimensions for the diaphragm and for the columns are shown in Fig.5. The strength

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68 Evaluation of the Seismic.../ Ovidius University Annals of Constructions **3**, **4**, 65-72 (2002) characteristics of the steel are $f_{1y} = 270 \text{ N/mm}^2$, $\varepsilon_{It} = 3^0/_{00}$ and $f_{2y} = 250 \text{ N/mm}^2$, $\varepsilon_{2t} = 45^0/_{00}$.



Fig. 1 - Definition of the e ccentricity

		e > 0		e > 0		e > 0
Case	Α	$e = e_{max}$	В	$e = e_{min}$	C	$e = e_{min}$
				$e_a < e_r$		$e_a > e_r$
General	$e_{max} = e_{pr} = e_f +$	$e_a = \alpha_f e_0 + e_a$	$e_{min} = e_{sec} = e_r$	$e_a = \alpha_r e_0 + e > 0$	$e_{min} = e_{sec} = e_r$	$e_a = \alpha_r e_0 + e_a < 0$
EC8	$e_{max} = e_0 + e_2 -$	$+ e_1$	$e_{min} = e_0 - e_1$	> 0	$e_{min} = e_0 - e_1$	< 0
	$e_f = e_0 + e_2$		$e_r = e_0$		$e_r = e_0$	
	$\alpha_f = (1 - e_2 / e_0)$)	$\alpha_r = 1$		$\alpha_r = 1$	
	$e_a = e_l$		$e_a = e_l$		$e_a = e_1$	
P. 100-92	$e_{max} = e_0 + e_1$		$e_{min} = e_0 - e_1$	> 0	$e_{min} = e_0 - e_1$	< 0
	$e_f = e_0$		$e_r = e_0$		$e_r = e_0$	
	$\alpha_f = 1$		$\alpha_r = 1$		$\alpha_r = 1$	
	$e_a = e_l$		$e_a = e_l$		$e_a = e_l$	













Fig. 4 - General view of the model

3.1.Testing Programme

The primary purpose of the specimen tests was the checking of the mass influence and structural eccentricities. The problem of selecting appropriate parameters in order to obtain accurate response prediction is a complex one.



Fig. 6.- Model A.

Model D1 - two columns of this model have 60x60x6 mm cross-section and two. 60x60x4 mm cross-section. The mass centre (CM) matches the rigidity centre (CS) (s. Fig. 7).



Fig. 7.- Models B1, C1, C3, D1.

3.2. Test Facilities and Instrumentation

The main goal of this paper is to find out the influence of the torsion on structural behaviour using models especially designed for this purpose.

The experimental activities were oriented towards two main directions:

realization of the mass eccentricity; a.

b realization of the stiffness eccentricity.

The mass eccentricity was realized by four weight positions. The stiffness eccentricity was realized with four series of models combined with the mass eccentricity verifications:

Model A - columns with tubular 60x60x6 mm cross-sections. The model is symmetrical with respect to the stiffness. Models A1, ... A4 are with mass eccentricity. The weight positions are shown on Fig. 6.

Model B - three columns of this model have 60x60x6 mm dimensions and one has 60x60x4 mm dimensions. The model is symmetrical with respect to the mass and unsymmetrical with respect to the stiffness (Fig.7).

Model C1 - two columns of this model have 60x60x6 mm dimensions and two have 60x60x4 mm dimensions. The model is symmetrical with respect to the mass. Model C3 is the same as Model C1, but nonsymmetrical with respect to the mass (s. Fig. 7).

The six degrees of freedom (DOFs) shaking table used for the tests is installed at the National Technical University, Laboratory for Earthquake Engineering, in Athens (Greece). The mechanical part of the platform is made by steel and has 4 m x 4 m x 0.6 m dimensions.

Maximum possible weight of the specimen is 100 kN. Maximum force in horizontal plane is 320 kN and 640 kN in Z-direction respectively. Each of the six DOFs can be excited independently or simultaneously.

The shaking table performances are: maximum displacement along each axis: ± 10 cm; maximum rotation about each axis: 7 x 10⁻² rad; operating frequencies for each DOF: 0.1...50 Hz; peak accelerations along each horizontal direction (X, Y) 2.0g and up to 4g for the vertical component; velocity up to 100 cm/s.

The control of the experiment is carried out by analog and digital units. This units ran produce and

combine waveforms of the following types: sinusoidal, sweep sine, random, random based on a response spectrum definition and tabulated entry of field-recorded earthquakes.

The data aquisition system consists of a 64 Channel Digital Control Unit with a PC. The received data (from strain-gauges, accelerometers, displacement transducers, etc.) are stored on disks. These can be compared to similar calculations performed on the desired table motion waveforms. The shaking table output is presented by several means, *i.e.* functions including: response spectra, acceleration time histories, Fourier spectrum and total earthquake energy calculations.

Accelerometers and displacement transducers were placed at the basement (the shaking table self-control transducers) and the first floor level.

The instrumentation for the first level consists of:

a. four horizontal accelerometers (Type ENDEVCO, model 2262A - two in the Y-direction and two in the X-direction);

b. six displacement transducers (Type CELESCO, model PT8101- three in the Y-direction and three in the X-direction).

3.3. Shaking Table Motions Used for Tests

Several ground (*i.e.* shaking table) motions were used in the experimental investigation:

sweep action 0...35 Hz, peak value of 0.lg - action (1);

torsion resonance action - action (2);

sinusoidal action, peak value of 0.3g - action (3);

earthquake action, peak value of 0.3g - action (4).

The dynamic testing programme was complex, leading to a total number of 44 experiments.

First test was carried out to obtain an estimate of the natural frequencies of the system. In this case was used the action (1) in both X and Y-directions.

In order to obtain the damping, the model was subjected to the resonance by the means of action (2).

3.4. Experimental Results

The full set of the experimental results is given in (1). Mean values of the registered natural frequencies are presented in the Table 2.

After post-processing of the experimental results, there were computed the amplification factors

as ratios between the peak values of response and action. These values are summarized in Fig. 3. An important result is found to be a very low value of the torsion damping ratio, $\xi = 0.297\%$.

Гa	ble	2

Model	T _x	Ty	T_{θ}
A1	0.401725	0.445220	0.208980
A2	0.409600	0.426660	0.213330
A3	0.409600		0.204800
A4	0.397780	0.422395	0.232720
B1	0.445210	0.445210	0.222600
C1	0.476530	0.465450	0.238130
C3	0.487610	0.465450	0.238130
D3	0.465450	0.387655	0.242518

4. Conclusions

In the present paper there are shown the obtained testing results in the elastic range that were carried out on the specimens presented in § 3.

The following data and observations are to be mentioned:

The torsion damping of the steel model is very low - about 0.3%.

The coefficients of amplification are generally equal to 1.2...1.5 for medium and large values of the eccentricities ($e_0 / L > 0.1$) and equal to 4...5 for small eccentricities ($e_0 / L < 0.05$), (s. Fig.3).

The model inexactitudes produce great variations of stiffness and inertial characteristics, which affect the eccentricities; these variations affect the torsional response much than the translational response.

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Impact Analysis of Steel Tanks – Sonic Booms

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Abstract: The design and construction of civil and industrial buildings to provide life safety in the face of impact loads is receiving renewed attention from structural engineers. Existing impact design approaches call for modest enhancements in structural design coupled with a buffer zone surrounding the building. This highly effective approach is only feasible where a keep-out zone is available and affordable. To resist impact loads, the first requirement in the assessment of a structure is to determine the threat. For many urban and industrial settings the proximity to (un)regulated car or aircraft traffic brings the terrorist or accidental threat to or within the perimeter of the building. The paper presents a numerical investigation of a real case occurred in July 2001 at S.C. "Sofert" S.A. Bacău, when the thermo–hydro insulation of an ammonium tank was totally destroyed after a low altitude potential flight of a "MiG-21 Lancer" jet aircraft. Beyond the juridical aspects and moral responsibility of all those involved in the event, such threats remains serious and reveal an unacceptable vulnerability of many strategic buildings. A high scale human tragedy has been avoided due to an almost providential chance.

Keywords: impact, shock wave, sonic boom, blast, explosion.

1. Introduction

Impacts and explosions could be encountered in various fields, ranging from domestic environment to military warfare - hurricane and tornado disasters, aircraft and missile impacts, cars, trains and ships collisions, explosions in cars, ships, houses and military establishments, snow/ice impacts, wave impact on the ground and offshore platforms, and many other different impacts in water and in air. The dynamics of impact and explosion are an important consideration in the design of conventional structures, in general, and sensitive and unconventional structures in particular. Accidents causing damage and explosion are a matter of growing concern in many areas such as nuclear, offshore, civil, mechanical, gas, electrical, chemical, aeronautical and naval engineering.

The impact analysis was not possible until 5 to 10 years ago, since the structural response was extremely difficult to be evaluated due to the lack of powerful testing and numerical equipments. The modeling of impact and explosion remains one of the most difficult tasks. It involves structural

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dynamics, load-time relationships, impactor-target interaction, material properties including strain-rate effects and solution convergence procedures. The importance of the structure's ability to withstand the impact loads (such gas explosions, sea wave and snow/ice impacts, ship and missile collisions, dropped objects, seismic shocks etc.) cannot be ignored and the strain rate effect of reinforced concrete elements must be included in the global analysis of concrete structures.

In Romania there are no specific prescriptions regarding the structural design at impact loads to a real risk, and in a very few countries in the world the codes has been properly adapted. Many times the expert's reports often demonstrate that applying minimal safety measures – not exceeding by more than 5 to maximum 10% the general expenses – many lives would certainly have been saved and important collateral damages avoided.

Paulay and Priestley (1992) conclude in [4] that the strain rate effect for the reinforced concrete structures is of low importance and could be neglected. Recent tests strongly contradict their opinions, as it will be shown as follows, based on two relevant references.

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SANDIA Researchers from National Laboratories (U.S. Dept. of Energy) assert that "strength and mode of deformation of rock and other brittle materials such as concrete and ceramic depend on the applied rates of stress or strain. The strength of rock generally increases gradually with about 102 s⁻¹ the effect of increased rate on brittle fracture stress is very strong. Little work has been done in the intermediate range, 10^{-2} to 102 s⁻¹. Some studies have missed the systematic variation of strength with rate in the high rate regime and have simply extrapolated quasi-static data over several orders of magnitude to a single value of "dynamic" strength." ("Material behavior under impulsive loading").

In [2] Ellis and Tsui failed in their attempt to predict the numerical response of the panels, but they admit that the strain rate effect has not been considered. Extrapolating the quasi-static data to analysis leads impact will strong to underestimations of the ultimate load and general structural response. More, it is revealed that the panel stiffness is significantly bigger than expected. This somehow "strange" phenomena is very important in the evaluation of the dynamic general response of the structural element and of the whole structure, respectively.

A proof that the field of impact study of the constructions is of a major interest and that is has captured the interest of the specialists of many countries is also the fact that in 1996 and 1999 in Singapore, there were organized two conferences on impact themes (*Shock and Impact Loads on Structures*), the second conference on this theme taking place at Melbourne, 1997.

2. Real case study

In July 2001 at S.C. "Sofert" S.A. Bacău, the thermo-hydro insulation of the 22,000 cu.m. ammonium steel tank was totally destroyed as a consequence of a low altitude potential flight of a "MiG-21 Lancer" jet aircraft. Beyond the juridical aspects and moral responsibility of all those involved in the event, such threats remains serious and reveal an unacceptable vulnerability of many strategic buildings. A high scale human tragedy has been avoided due to an almost providential chance: the steel tank was empty for periodical technical investigations.



Fig. 1 – The ammonium steel tank before the accident

The preliminary analysis of such structure (according to the present Romanian regulations) revealed that for various current combinations of static, dynamic and exceptional loads – the probability of spoiling the thermo–hydro insulation of the steel reservoir (Fig. 1) is extremely low. Consequently, other exceptional loads have been tried and investigated which could lead to the dramatic damages shown in Figures 2 and 3. Two other possible exceptional loads have been identified, both of them accepted by beneficiary and by the witnesses which assert that the damage occurred immediately after the at low altitude flight of a "MiG-21 Lancer" jet aircraft belonging to a closer military division.



Fig. 2 – The ammonium steel tank after the incident (detail)

The first impact load considered was a typical horizontal shock wave (a sonic boom - ?) generated by



Fig. 3 – The ammonium steel tank after the incident (detail)

a jet aircraft during a flight at a low altitude (Fig. 4) or by accidental ejected burned gases with high velocities (Fig. 5).



Fig. 4 – Sonic boom – impulsive noise (similar to thunder) caused by any object moving in the atmosphere, faster than sound



Fig. 5 - Ejected gases by a MiG 29 aircraft

In another hypothesis a looping acrobatic maneuver has been considered, when the pilot led the plane in an almost vertical position above the tank, projecting the burned gases to the tank roof. Therefore, for this hypothesis there we're identified three possible load situations: a perfect vertical impact and two inclined impacts at 35 and 40 degrees, respectively, respecting to the ground plane.

S.C. "Sofert" S.A. Bacău official requested structural analysis on various numerical models to evaluate the minimum pressure magnitude of the shock wave for which the thermo-hydro insulation is damaged and to identify what kind of impact load could lead to such a severe shatter. In what follows conclusions of the static and dynamic analysis are presented for all four numerical models.

The geometrical and mechanical properties of the ammonium steel tank are as follows:

(i) steel tank:

- diameter 38.00 m;
- height of the tank cylinder 20.50 m;
- relative height of tank roof 5.91 m;
- total height of the tank 26.41 m;
- wall thickness variable from 24 mm at the base to 12 mm at the top;
- equivalent roof thickness 42 mm.
- $E = 2100000 \text{ daN/cm}^2$
- -v = 0.25
- $-\rho = 7850 \text{ kg/m}^3$
- $\sigma_{yield} = 2000 \text{ daN/cm}^2$
- $-\sigma_{\rm u} = 2900 \, {\rm daN/cm^2}$
- (ii) thermal insulation:
 - foam glass thickness 12 cm;
 - mineral wool thickness 5 cm;
 - equivalent thickness of the thermal
 - insulation for numerical analysis 16 cm
 - $E = 10560 \text{ daN/cm}^2$
 - -v = 0.38
 - $-\rho = 270 \text{ kg/m}^3$
 - $-\sigma_u = 4.5 \text{ daN/cm}^2$

(iii) external corrugated aluminum sheet profiles:

- thickness of the aluminum sheet 1.5mm;
- equivalent thickness of the aluminum sheet protected profile 14.9 mm
- $E = 700000 \text{ daN/cm}^2$
- -v = 0.30
- $-\rho = 2700 \text{ kg/m}^3$
- $\sigma_{\text{yield}} = 1380 \text{ daN/cm}^2$
- $\sigma_{\rm u} = 1900 \, {\rm daN/cm^2}$
- $-0_{\rm u} 1900$ ualv/cm

- the corrugated aluminum sheets was equidistant connected at 15 cm with 6 mm screws.

It must be mentioned that the hypothesis of an impact load by a horizontal shock wave generated by the ejected burned gases was extremely low. Flying with a military jet aircraft at a so low altitude, in the existing local conditions is quite impossible. Consequently, a supplementary load of a **thermal pulse** has not been considered as possible, such thermo-structural coupling being insignificant. Structural response at a thermal pulse of about 800-900°C could be relevant only if the plasma jet would be projected from a maximum distance of 10 meters. The velocity of the ejected gases by a MiG-21 Fishbed/Lancer aircraft varies from 190 m/s to 630 m/s, depending by the afterburner diameter of the turbojet engine.

"Unlike the problems created by all the other sources of noises regarding their influence on the structures, the unpleasant effects of the sonic boom usually appear in the pressure field far away from the source and, in very few cases, in the closer one. [...] The damages usually occur at relatively low pressure loads, but applied on large surfaces." [7].

The main technical characteristics of the MiG-21 Lancer jet fighter, without postcombustion (Fig. 6 - v. A & C - 1 pilot, v. B - 2 pilots) are:

- Tumansky engines all versions;
- minimum engine power 40.2 kN;
- maximum engine power 69.7 kN;
- maximum speed 2.1 Mach;
- maximum flying altitude 18.000 m.

Most of the civil and industrial buildings are generally protected for shock wave pressures lower than 100 Pa, generated by sonic booms. USAF and RAF require that all these objectives must resist without any damage at minimum acoustic pressures of 16 psf (750 Pa) applied for maximum 0.15 s.

The high and very high-pressure spectrum frequencies (between 80 Hz and 10 MHz) do not influence the dynamic structural response. Hence, the resonance probability at high frequencies is very reduced, but, due to the Doppler effect, such a phenomenon should not be excluded. As it could be easily seen, this real exceptional load case for the ammonium steel tank seems to be one of the most complex ever imagined structural analysis, for only two loading hypotheses - very sensitive to the entrance data.



Fig. 6 - MiG 21 Lancer jet fighter

Starting from the above mentioned load and model conditions, four numerical models has been developed for the impact analysis with a fluid-gas shock wave. The numerical models have a different number of total finite elements as follows;

- model A 12383 finite elements (Fig. 7);
- model B 12276 finite elements (Fig. 8);
- model C 12240 finite elements (Fig. 9);
- model D 12120 finite elements (Fig. 10);

from which **3960** shell finite elements (with 4 nodes) for each layer of the ammonium reservoir (steel tank, thermal insulation, corrugated aluminum sheets).



Fig. 7 – Numerical model A - horizontal shock wave impact



Fig. 8 – Numerical model B - perfectly vertical shock wave impact



Fig. 9 – Numerical model C - shock wave impact at 40° respecting to ground plane



Fig. 10 – Numerical model D - shock wave impact at 35° respecting to ground plane

For all the numerical models the plastickinematic material was used (when exceeding the material stress crack limit, the finite element is automatically wiped out by the program):

- elastic-perfectly plastic curve (Prandtl curve);

- for the fluid plane (the impactor) a different number of 4 nodes finite elements has been considered as follows:

- for model A 503 finite elements
- for model B 396 finite elements
- for model C 360 finite elements
- for model D 240 finite elements

- the mechanical features of the shock wave: air density air - 1,3 Kg/m³ and the air bulk modulus - E = 150 Mpa.

For both values (air density and bulk modulus), the compression effect generated by the aircraft has not been considered enough relevant by the point of view of the structural analysis. According to military experts' investigations, there are enough insignificant changes of the mechanical properties respecting to supersonic speed up to 2 Mach (Fig. 11). Consequently, this hypothesis may be considered as a supplementary safety measure in evaluation of the structural response.



Between the exterior corrugated aluminum sheets that it have been considered a tie-break contact that was destroyed not at the ultimate stress limit of the screws, but a value of 70% from the yield limit of the corrugated aluminum sheet (950 daN/cm²).

3. Results and conclusions

The minimum pressures for which the corrugated aluminum shield shatters were determined by progressive trials and the following values were obtained: - for *model* A – a 300m/s minimum velocity of the shock wave having a dynamic equivalent pressure of 1430 Pa (Fig. 12);

- for *model* B – a 400m/s minimum velocity of the shock wave having a dynamic equivalent pressure of 2200 Pa;

- for *model* C – a 280m/s minimum velocity of the shock wave having a dynamic equivalent pressure of 1780 Pa (Fig. 13);

- for *model* D – a 220m/s minimum velocity of the shock wave having a dynamic equivalent pressure of 820 Pa;

As a concluding remark it must be mentioned that the model is very sensitive to the velocity of the shock wave and not mainly to the magnitude of the applied pressure. It means that for a lower pressure, but applied with high velocities, the structural behavior is quite similar to a quasi-static analysis, important plastic deformations, without any shatter effect of any layer.



Fig. 12 - *Model A* structural response at a horizontal shock wave

Consequently, the thermo-insulation of the ammonium steel tank may shatter only if there was a strong impact load applied with high velocity. The experts have to identify the most probable source of producing such a shock wave (wind shock waves of minimum 0.1s to maximum 0.25 s, having a minimum velocity of 150 m/s), blast, gas explosions close to the tank, sonic booms, etc.).



Fig. 13 - *Model C* structural response at a 40° inclined shock wave

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Information Processing for Model Adjustment of Damaged Constructions

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Abstract: The paper deals with the problem of structural model correction using some experimental available data for safety evaluation of seismic/dynamic behavior of building's structures. The information processing for seismic analysis is based on estimation methods, typical for identification problem. A global view of the information needed and available for adjustment of structural model used for seismic and dynamic analyses of constructions is presented. Based on a deterministic procedure, the paper present a methodology used for correction of structural models of constructions, exposed to repetitive and cummulative effect of earthquake during lifetime.

Keywords: Indirect identification, linear model, estimation procedure, information processing.

1. Introduction

Damage evaluation in Civil Engineering structures, subjected to general dynamic loading of seismic types, is an issue focused by many researchers, being a complicated matter. The issue of damage evaluation is developed in a multiple stages concept which depends on the clear understanding of problems hierarchy within the proposed or needed performance of structure for a requested safety level, balanced by information processing, specific for each stage. The process of structural modeling to simulate the seismic and dynamic behavior of constructions supposes the following steps:

• A priori knowledge of modeling,

which consist of: modeling of the seismic loading used for the excitation of undamaged/damaged structure; system modeling of the undamaged structure, based on System Theory concept, including the choice of global dynamic model; seismic design of the undamaged structure with respect to codes of practice; definition and modeling of damage based on symptoms description, patterns and features, classification typical for structural micro and macro behavior; structural performance class. • Monitoring on line of damaged structure, accumulating information database,

which supposes: the identification/monitoring during the lifetime of the structure, done by model identification, or inspection on real structure as a time-dependent processes; environmental observations and damage identification based on detection and localization, the relationship between knowledge posteri modeling about damage development; damage evaluation and assessment of the structure; and finally redesign of structure, based on new concept of seismic performance upon deformation-based methods.

Usually, the nondestructive evaluation method is used to identify the damage area location of the structure through post-earthquake inspections. The practical codes are using this method in the expertise of the structural safety and integrity, after each significant seismic event.

The class respectively type of damages determines the class of the damaged structure. For the damage identification, there is a strong tendency to consider the a priori model for structural seismic analyses as a linear model.

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The researches carried out recently highlights the real need for detection damage schemes, especially for large-scale civil engineering structures, subjected to seismic loading. Most approaches see [....] uses the analytical model of the structure in a deterministic procedure for parameter estimation assisted by a corresponding information processing data. The key issue for a practical implementation of such a procedure is: the developing of an accurate model of the structure; developing an accurate chain for excitation force respectively for the structural response; selecting a corresponding environment for information processing of data, as a function of information data type; simulations studies to determine the sensitivity of the estimates to noise, as well as the problem of results uniqueness.

2. Correction Method

The damage class determines the damage structure's class. If, for a given structure, the identified damage is local and moderate, this fact permits the consider a linear behavior for the structure.

Let us assume that the structure is modeled by means of a linear, finite element model with finite degrees of freedom. The response of the structure depends on how its constitutive properties contribute to the stiffness, damping and inertial matrices, K, Cand M respectively. It is assumed that usually, only the linear stiffness K changes due to a certain damage that occurred in structure. Accordingly, the matrix is parameterized by n constitutive parameters, considered as. K(a). The selection of the appropriate parameter estimation algorithm for data processing is linked to the parameterization of the model.

Assuming a linear n degree of freedom model, with viscous damping, the equation of motion in the case of seismic excitation is done in Eq.(1):

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = P(t)$$
(1)

where: M, K, C are: the symmetrical model parameter matrices of order n, x(t) is the compatible vector of displacements and P(t) is the equivalent vector of seismic motion, $P(t) - M\ddot{u}(t)$. In many cases, the real system is divided into subsystems. The corresponding parameter matrices is described by sub-matrices of the same order as the whole system matrices:

$$M = \sum_{e=1}^{S} M_{e}, \quad C = \sum_{p=1}^{E} C_{p}, \quad K = \sum_{i=1}^{L} K_{i}$$
 (2)

Designating M^c , C^c , K^c as the inertia, the damping and respectively the stiffness matrices of the computational model to be corrected, the vector of correction is defined in Eq. (3):

$$M^{c}(a_{n}) = \sum_{c=1}^{S} a_{M_{v}}M_{c} + M'$$

$$C^{c}(a_{c}) = \sum_{p=1}^{L} a_{C_{v}}C_{p} + C'$$

$$K^{\chi}(\alpha_{\kappa}) = \sum_{i=1}^{A} \alpha_{K_{i}}K_{i} + K'$$
(3)

with:

$$a = \{a_j\} = \{a_M, a_C, a_K\}^r, j = 1, 2, ..., J; \quad J = S + R + L$$
(4)

and:

$$a_M = \left\{ a_{M_C} \right\}, \quad a_C = \left\{ a_{C_p} \right\}, \quad a_K = \left\{ a_{K_i} \right\}$$
(5)

Taking the correction parameters vector $\tilde{a} - \{e\}$, where the vector $e^r = (1, \dots, I)$ yields the parameter matrices (2) of the (uncorrected) computational model.

3. Estimation procedure

To improve the existing computational model of the structure, a parameter estimation method is used.

Generally, the estimation concept is characteristic for the Identification of Systems and various authors [1, 3, 4] present the corresponding methods which are used for identification problem (Bayes' Estimation, Maximum Likelihood Estimation, Weighted Least Square Estimation, Least Square Method, Instrumental Variables).

In this paper, the Least Square Method is chosen for estimation. To apply the Least Square Method, the loss function (or aim functional) is defined, [1, 2, 3]:

$$J(a) = V^{T}(a) \cdot G_{V} \cdot V(a)$$
⁽⁶⁾

$$V(a) = \{V_x\}, \quad r = 1, 2, ..., N$$
 (7)

where V(a) is the residuum vector defined, in case of linear model, using a_i parameters in Eq. (8):

$$V(a) = b - DV \cdot a \tag{8}$$

where: *b* is a vector and *DV* is a matrix depending on the a_j parameters; G_v , the weighting matrix is the identity matrix.

There are different possibilities to define the residuum vector [3, 4], as: input residual, output residual, partial residual, etc. After the choice of residuum, the functional must be minimized based on the estimation concept, presented in Eq. (9):

$$\frac{\partial J(a)}{\partial a_j}\bigg|_{a=\ddot{a}} = 0,$$

$$j = 1, 2, \dots, J; \quad J = S + R + L$$
(9)

If the system is linear with respect to the parameters, thus corresponding to the equation (8), the correction's parameters vector is obtained from the next matricial equation, Eq. (10):

$$\widetilde{a} = (DV^T G_v DV)^{-l} (DV^T G_v b)$$
(10)

4. Information processing

For a linear systems represented here by a rigid reinforced concrete frame, it is assumed that the mass matrix \mathbf{M} is defined with sufficient accuracy, than the problem is to correct the stiffness matrix \mathbf{K} , with respect to measured eigen-frequencies of the undamped system.

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In this case, let us regard the eigenvalue problem associated to the undamped model of the system described by Eq. (1):

$$(-\omega_r^2 M + K)u_r = 0 \tag{11}$$

where: ω_r and u_i are the dynamic characteristics of the computational model. The measured eigenvalues will be denoted as $\tilde{\lambda}_r = \tilde{\omega}_r^2$.

The correction parameters vector is given in eq. (12):

$$a_k = \{a_{k_l}\}, \quad l = 1, 2, \dots, L$$
 (12)

and the corrected stiffness matrix, $K^{c}(a_{k})$ is obtained from Eq. (13):

$$K^{c}(a_{k}) = \sum_{l=1}^{L} a_{k_{l}} K_{l} + K'$$
(13)

Using the results from Eq. (4), the residuum vector V(a) is defined by:

$$V^{T} = \left\{ \widetilde{\lambda}_{r} - \lambda_{r} \right\}, \quad r = 1, 2, \dots, N$$
(14)

Then, DV is the matrix of the partial derivatives of the N * L dimensional vector V:

$$DV = -\left[\frac{\delta V}{\delta a_{k_1}}, \frac{\delta V}{\delta a_{k_2}}, \cdots, \frac{\delta V}{\delta a_{k_n}}\right]$$
(15)

Using Eq. (14), DV is becoming:

$$DV = \begin{bmatrix} k_{g_{22}} & \cdots & k_{g_{ln}} \\ \vdots & & \vdots \\ k_{g_m} & \cdots & k_{g_m} \end{bmatrix}$$
(16)

with:

$$k_{g_{12}} = \frac{\delta \lambda_x}{\delta a_{k_2}} = \overline{U}_r^2 K_2 \overline{U}_r$$
(17)

the vector **b**, and the generalized stiffness k'_{gr} are:

$$b^{T} = (\widetilde{\lambda}_{l} - k'_{g_{2}}, \widetilde{\lambda}_{2} - k'_{g_{3}}, \cdots, \widetilde{\lambda}_{n} - k'_{g_{n}})$$
(18)

and:

$$k'_{g_r} = \overline{u}_r^T K' \overline{u}_r \tag{19}$$

where: u_r is the normalized eigenvector of the r modal vibration.

5. Conclusions

Some issues concerning information processing for procedures-based realistic identification of seismic safety of constructions have been discussed. A procedure which processes the information for correcting the models used to simulate the behavior of some RC frame constructions during earthquakes is presented together with an example of application.

The following conclusions appears to be valide:
 a) detection of damage is a complicated matter and depends of a clear understanding of what constitutes an undamaged structures;

b) damage detection depends on the quantities and quality of information data, as upon the available/chosen environment for information processing aswell; c) for large structures, there is a really need for detection schemes of damage.

Also one has to underline the necessity of knowledge transferring from the university laboratory into design office and to link it to practical codes. Especially for countries as Romania, with a damaged infrastructure due to several consecutive earthquakes, there is a real need to complete the database about seismic safety of existing buildings with study of cases for structures, subjected to a given scenario of previous or possible moderate/strong consecutive seismic events through monitoring based simulation of both structural linear or nonlinear behavior.

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Nonlinear Analysis of Seismic Behavior of R.C. Shear Walls with Staggered Openings

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Abstract: Tall buildings with staggered openings shear walls, have been presented minor damage of the structural and nonstructural elements under seismic actions. This paper presents the conclusions, resulting from a nonlinear seismic analysis biographic (push-over) method. Several types of such shear walls have been analyzed with the specialized computer program BIOGRAF 03 [3]. Ductility and failure mechanism are shown in a comparative manner taking into account the position of the openings.

Keywords: shear walls, nonlinear analysis, displacement ductility

1. Introduction

Du to the fact that reinforced concrete structural walls are very effective in providing resistance and stiffness against horizontal loads induced by earthquake, they are frequently used in aseismic structural design all over the world.

The aseismic design codes of buildings, have been permanently completed on the basis of the observations, which were made after studying the behavior of the structural elements of this building during the earthquakes or by experimental and numerical studies.

The development of computer programs on the base of finite elements methodology has allowed to perform nonlinear static or dynamic analysis, a fact that has lead to results as for as ductility on failure mechanism of all structural element of a building are concerned.

The good seismic behavior of building having shear walls with staggered openings are due to rigidity and ductility usually higher to those of shear walls with regular openings.

Experimental researches works on these shear walls indicate the fact that ductility of walls and their failure mechanism are influenced by the position of the openings and the types of reinforcement.

2. The description of analysis method

The reinforcement of these walls have been performed using the computer program AXIS VM 5.0 program.

The nonlinear behaviors of shear walls with staggered openings have been studied, for four wall types with the physical-mechanical characteristics presented in table 1.



Fig.1. Structure discretisation linear anlysis. Oppenig position

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Tabel 1. Geometrical and mechanical characteristics

Geometrical characteristics					
Wall thickness	$b_w = 25 \text{ cm}$				
Story height	$h_s = 2.60 m$				
Wall length	$l_{\rm w} = 5.0 {\rm m}$				
Wall height	$h_{\rm w} = 13.0 \ {\rm m}$				
Opening width	$l_{d} = 1.0 m$				
Opening height	$l_{d} = 2.0 \text{ m}$				
Mechanical characteristics					
Concrete modulus of	E _b =				
elasticity	3E05daN/cm ²				
Compression	f_{cd} =150daN/cm ²				
strength of concrete					
Tensile strength of	$f_{ct} = 11 daN/cm^2$				
concrete					
Steel modulus of	$E_{s}=2,1E06$				
elasticity	daN/cm ²				
Yield strength of steel	f _y =3000daN/cm ²				



Fig.2. Structure discretisation and loads in nonlinear analysis

These walls differ in function of the openings positions characterized by angle α and the quantity of reinforcement.

The analysis has been performed in two steps:

- step 1 – determination of elastic stresses and reinforcement using the AXIS VM 5.0 computer program. To evaluate the reinforcement of cracked zones, due to tension stresses, and the required reinforcement of compressed zones where the stresses are higher than ultimate concrete stress, has been used the AXIS VM5.0 computer program. The geometrical element of the walls and the discretisation are shown in Fig.1.

- step 2 – The behavior of shear walls is performed with BIOGRAF computer program [3], using a push-over methodology.

3. Discretization of structure and the nonlinear analysis

The type of R.C. shear walls used in the analysis has been obtained through its discretizations in 882 nodes and 1644 triangular finite elements.

In the aim to determinate structural failure the potential plastic zones has been divided in a larger number of elements. The horizontal forces have been punctually at the level of every floor, and the vertical forces have been applied in every node (Fig. 2.).

The walls reinforcements do not respect the technological rules because the aim of study is to compare only the behavior of different types of shear walls.

In the aim to of analyzing the behavior of walls at horizontal forces, the direction of these actions have also been changed acting from left or right directions.

The method of analysis is based on progressive reduction of seismic forces and it wants to determine the ductility and the types of failure of these walls. The seismic forces applied to studied shear walls correspond to a building with five levels.

The reduction of reinforced area has been made by multiplication of seismic forces calculated in the elastic domain with R factor. The R factor have had the following values: R=1; 0.5; 0.33; 0.25; 0.20.

In order to obtained some proportional graphics the q factor has been used in the representations: Eq.(1).

$$q = \frac{1}{R} \tag{1}$$

As a result of the reduction of steel ratio on the basis of the presented method of analysis 35 different types of shear walls have been analyzed.

The displacement ductility $:_{0}$ is calculate with Eq. (2):

$$:_{j} = \frac{\Delta_{y}}{\Delta_{c}}$$
(2)

where:

)_y - is horizontal top displacement of the R.C. shear walls at the beginning of reinforcement yielding;

 $)_{c}$ - is horizontal top displacement of the R.C. shear walls the for crushing of concrete.

On the basis of the values of the displacement ductility calculated, the values of the R factor have been determined in concordance with Romanian Seismic Cod R100/92 Eq. (3)

$$\Theta = \frac{1}{\sqrt{2\mu_{\Delta} - 1}} \tag{3}$$

4. Results

4.1. Rigidity reduction

The behavior of shear walls has been examinate by following the horizontal displacement of the top of walls.

The most favorable behavior is obtained for q=2 to 3. Generally one can see that the rigidity of staggered openings is higher than the rigidity of coupled shear walls.



ductility for $\alpha = 90^{\circ}$







Fig.5 Comparative diagrams of displacement ductility for $\alpha = 45^{\circ}$



Fig.6 Comparative diagrams of displacement ductility for $\alpha = 32^{\circ}$



Fig.7 Comparative diagrams of displacement



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Fig.8 Comparative diagrams of displacement



Fig.9 Comparative diagrams of displacement ductility for $\alpha = 18^{\circ}$

4.2. Displacement ductility

Figure 10 present the displacement ductility for coupled shear walls with regular openings ($\alpha = 90^{\circ}$) and Fig. 11-16, the same ductility for different ductility for different angles ($\alpha = 45^{\circ}$; 32°; 18°) and for different directions of seismic loads. One can see that the maximum ductility is obtained for q = 3 to 4 in function of direction of seismic loads.



Fig.10 Comparative diagram of displacement ductility for $\alpha = 90^{\circ}$



Fig.11 Comparative diagram of displacement ductility for $\alpha = 45^{\circ}$



Fig.12 Comparative diagram of displacement ductility for $\alpha = 45^{\circ}$



Fig.13 Comparative diagram of displacement ductility for $\alpha = 32^{\circ}$



Fig. 14 Comparative diagram of displacement ductility for $\alpha = 32^{\circ}$



Fig.15 Comparative diagram of displacement ductility for $\alpha = 18^{\circ}$



Fig.16 Comparative diagram of displacement ductility for $\alpha = 18^{\circ}$

4.3. Yielding modes

From the point of view of the yielding mode, indifferent of the type of reinforcement or the position of openings, all the structural walls with staggered openings register a ductile behavior by yielding of the reinforcements before the concrete crushing.

Generally speaking, first of the reinforcements who reached yielding for α =90° and α =45° was the vertical reinforcements in the zone of openings of the first floor. By the measure of the reduction of the vertical areas of reinforcement (q=3, 4, 5), the reinforcements at the base of the walls reached the flow limit.

The yield stresses of the horizontal reinforcements on the floors level is registered at structural walls with values of α angle α =32° and α =18°. The yielding of horizontal bars at the level of slabs occurs for the walls with opening angles of α =45° and 18°. For high quantity of horizontal bars the yielding is produced in the bars corresponding to the slab over the level 1. By reducing this quantity, the yielding occurs on the bars corresponding to the slab over the level 2.

The yielding of the reinforcements in the structural walls with staggered openings is produced in all the cases at higher horizontal forces than those of the structural walls with ordinary openings, the biggest forces being registered for $\alpha=32^{\circ}$. The values of the horizontal displacements at the top of the walls for yielding of reinforcements are direct proportionally with the values of α angle. There are not high differences between these values for q=1, 2, 3.

From the point of view of the failure of concrete it must noticed that the crush is produced at approximately the same values for the horizontal forces between 36000 daN and 42000 daN for q=1, 2, 3. The force that produces the concrete crush of all types of walls is increasing proportionally with the α angle value. The seismic nonlinear behavior

of structural walls is presented in P- Δ diagrams (Fig. 17-20).

q	α°	Seismic	force	P _{el}	[daN]	$\Delta_{\rm el}$	[cm]	$\mathbf{P}_{\mathbf{y}}$	Δ_y	$P_{min \ crushing}$	$\Delta_{\rm min crushing}$	Ductility	Ψ
		left (+)	right (-)	(+)	(-)	(+)	(-)	[daN]	[cm]	[daN]	[cm]	min μΔ	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
	90	SW 1	SW 1	12678	12678	0,080	0,080	91237	3,90	111270	3,90	2,55	0,49
	45	SW2	SW3	14517	13788	0,080	0,070	93340	3,48	109644	3,48	2,20	0,54
q=1	32	SW4	SW5	14270	13576	0,089	0,076	93128	2,95	114424	2,95	1,53	0,70
	18	SW6	SW7	13740	12920	0,095	0,076	93281	2,87	120974	2,87	1,65	0,66
	90	SW21	SW21	12897	12897	0,086	0,086	49827	1,06	76149	1,06	6,95	0,28
	45	SW22	SW23	20549	23283	0,120	0,130	53144	1,82	72408	1,15	3,59	0,40
q=2	32	SW24	SW25	19078	23522	0,120	0,130	56525	1,70	75455	1,87	1,52	0,70
	18	SW26	SW27	16434	20801	0,110	0,130	59847	1,71	79293	1,71	1,93	0,59
	90	SW31	SW31	12725	12725	0,086	0,086	40616	0,88	57901	0,88	6,49	0,29
	45	SW32	SW33	18837	18672	0,110	0,110	41849	0,90	60084	0,90	6,91	0,28
q=3	32	SW34	SW35	17410	18095	0,110	0,110	39801	0,70	63272	0,70	7,49	0,27
	18	SW36	SW37	15190	16348	0,112	0,107	36797	0,56	69402	0,56	8,64	0,25
	90	SW41	SW41	12643	12643	0,080	0,080	38177	0,92	55069	0,92	9,82	0,23
	45	SW42	SW43	15205	15082	0,096	0,088	38652	0,70	50560	0,94	5,97	0,30
q=4	32	SW44	SW45	14318	14754	0,094	0,087	39852	0,77	55172	0,77	6,62	0,29
	18	SW46	SW47	12678	13639	0,095	0,087	36246	0,58	57636	0,58	6,09	0,30
	90	SW51	SW51	13067	13067	0,086	0,086	38308	0,96	55005	0,96	16,48	0,18
	45	SW52	SW53	12573	12560	0,080	0,073	38489	0,84	47910	1,05	4,90	0,34
q=5	32	SW54	SW55	12000	12287	0,080	0,072	41751	1,67	49666	1,67	2,65	0,48
	18	SW56	SW57	10744	11425	0,082	0,072	38926	0,78	56013	0,78	3,31	0,42

Tab	le 2.	Non	linear	ana	lysis	resul	lts
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Fig.18 Comparative P- Δ diagram $\alpha = 45^{\circ}$



Fig.19 Comparative P- Δ diagram $\alpha = 32^{\circ}$



Fig.20 Comparative P- Δ diagram $\alpha = 18^{\circ}$

4. Conclusion

On the base of results previously presented concerning the seismic non-linear behaviors of the structural walls with staggered openings one can take the following conclusions:

-rigidity of shear walls with staggered openings is higher than the coupled shear walls rigidity;

-they realize a ductile behavior, without special measures of reinforcement in the potential plastic zones.

-the shears walls with staggered openings have higher capacity of seismic energy dissipation.

-the beginning of steel yielding occurs at the seismic forces higher than the one corresponding to coupled shear walls for the same horizontal top displacements.

5. Acknowledgments

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On the Safety Concept in Buildings Affected by Repeatable High Earthquakes

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Abstract : This work presents some comments on the deficiencies of P 100-02 Norm concerning the establishment of the resistance capacity level of buildings. On the basis of own expert experience, the authors present some improvements of the seismic ensurance estimation, taking into account the buildings' vulnerability and their location. The degradation degree of buildings was defined and a scoregrill for the seismic risk classes was elaborated.

Abstract: Evaluarea nivelului de asigurare (siguranță) a construcțiilor ce au suportat mai multe seisme mari se face, în prezent, conform Normativului P 100-92 determinând un grad mare de asigurare la acțiuni seismice $R=S_{cap}/S_{nec}$ și prin încadrarea în clase de risc seismic. In funcție de clasa de importanță a construcției, în P 100-92 sunt date valorile minime pentru R (0,70 – 0,50). Încadrarea construcției în clase de risc seismic (Rsi ... Rsiv) servește la: stabilirea nivelului măsurilor de intervenție și a gradului de urgență a executării acestor măsuri de intervenție. Pe baza expertizării a peste 200 de construcții (65 școli-licee, 45 blocuri de locuințe, 35 biserici și monumente istorice etc.) se propune determinarea nivelului de asigurare (siguranță) diferențiind construcțiile cu privire la: vulnerabilitate, poziția lor în cadrul localităților, valoarea "morală" a lor. Se propune introducerea noțiunii "grad de degradare" care să afecteze S_{cap} , funcție de numărul seismelor suportate de o construcție la care nu s-au făcut intervenții după seisme.

Keywords: seismic safety, seismic risk class, structure degradation degree.

Statistic analysis of Vrancea seisms in our country [1] in the last one thousand years, reveal a great number of high earthquakes occurring in Vrancea with obvious tendency to become more frequent in the XXth century, when evidently, possibilities to measure them more accurately improved.

According to many specialists, *seisms* are considered *high* when over 6,0 seismic degree on Richter magnitude scale.

In 1091 – 1900, according to [1], 59 seisms of 5,5 degree or more were recorded (statistically – one every 14^{th} year), 30 seisms of 6 degree (one every 27^{th} year), 23 of 6,5 degree or more (one every 35^{th} year) and 8 seisms of 7 degree or more (one every one hundred year).

In the XXth century (1900 – 2000), out of 81 seisms, 42 were of 5,5 degree or more (one every $2,3^{rd}$ year), 13 of 6 degree or more (one every $7,7^{th}$

year), 7 of 6,5 degree or more (one every 14th year) and 3 of 7 degree or more (one every 33rd year).

In the XXth century a 7,4 degree on Richter scale seism was recorded on 10^{th} November 1940 (the highest ever being recorded on 26^{th} October 1802 - 7,6 degree).

In an only 13-year period, 4 high seisms occurred: 4th March 1977 of 7,2 degree; 30th August 1977 of 7 degree, 30th and 31st May 1990 of 6,7 degree on Richter scale.

Specialists at the United States Geological Search (USGS) state that around 18 high seisms (7 degree - 7,9 degree) are recorded in the world in the last years, which confirms the tendency of the seismic activity to become more frequent. This tendency in Vrancea seisms is on account of complex factors deriving from endogene and exogene causes that, in their turn, are produced by the internal energy of Earth and by the activity of the continental plates and sub-plates.

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A 100 - 125 - year span on life for a normal building generally accepted (criteria concerning the material and moral degradation being taken into account), than such a building may undergo 10 - 15 high earthquakes ($M \ge 6$), if a 8 - 10 - year recurrence is accepted.

Resisting capacity and behaviour in buildings under seismic activity depend on many parameters connected to: structure type (bearing masonry, reinforced concrete frames, diaphragms, combined structures), building functionality, operating conditions, foundation soil, as well as on a series of

ally repaired, practically of unreserved resistance. It is the case for many blocks of flats in Bucharest, built before the Second World War.

To what concerns the consequences of seisms upon buildings (on the basis of the inventory of ruptures in over 100 buildings - especially churches after the seisms in 1977, 1986 and 1990) it is to be noticed that the synergic character of degradation reprocically potentiateing with every new seism [2] became obvious.

The poor behaviour of buildings after the seism in 1977, proved in fact the deficiencies of codes concerning the establishment of the conventional elastic resistance capacity level (fundamental seismic coeficient) and in the insurance of dimensioning and conformation of the whole and of the one element; that take into account the dynamic and the postelastic behaviour of the structures.

Many cases of serious degradation and collapse were due to inadvertences in the structure design favouring fragile ruptures, localisation of the processus of the post-elastic deformation and energy dissipation.

P100-92 Norm introduces new principles of antiseismic protection and of evaluation of the seismic safety level of buildings.

Buildings may be designed and built with such antiseismic conformation and destruction should be minimum even after high seisms.

Several resistance – structure categories are defined that participed to the antiseismic action, accepting, from project, plastic zones with limited destructions, but avoiding undoubtedly the collapse of the building.

It is generally accepted that the lapses of time in which seisms occur, their manifestation as well as temporally variable parameteres (strength structure material, environment, underground water aggressivity.

It is hard to quantize percentally, how a high seism wears out the power of resistance and the operating safety in different types of structurebuildings, of various ages.

Simplified calculations lead to real, global safety coefficients of $1.2 \dots 1.3$, in buildings that underwent all the high seisms of the XXth century and were not consolidated or capit

their consequences upon buildings are imprevisible, and highly aleatory.

This P100-92 Norm introduces new elements of code in the evaluation of the amplitude of the seismic force, yet, without clearly expressing the repetability and frequence tendency of seisms and the degradation synergic character [3].

The seismic force, according to P100-92 Norm is calculated as follows:

$$S_r = C_r G \tag{1}$$

where:

$$C_r = \alpha K_s \beta_r \psi \varepsilon_r \tag{2}$$

As compared to P 100-81 Norm, elaborated immediately after the seism in 1977, in the P 100-92, C_r coefficient introduces more clearly in the calculation [4]:

- the dynamic effect of the seismic activity,
- the importance of the building,
- the type of structure.
 - Thus:

- α coeficient is introduced, that takes into account the importance class of the building. Four importance classes I – IV are defined, for which α takes values as follows : 1.4 ; 1.2; 1.0; 0.8;

- by changing the seismic division our country into degrees of antiseismic protection (DAP 6, 7, 8, 8.5) with a new seismic division of calculation $A, \dots F$, the value of the coefficient K, increases (e.g.: in Iasi from 0.16 to 0.2);

- β_r coefficient is expressed both by the period of oscilations T_r of the buildings and by seismic

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condions of the area, characterized by the peak period T_p .

These new elements, introduced by P 100-92, determine seismic forces of calculation (of code) 40% - 60% and even with 100% higher, as compared to those calculated with P 100-81 Norm.

The evaluation of the safety level in buildings that underwent several high seisms is calculated presently according to P 100-92 Norm, determining a nominal degree of ensurance to seismic activity:

$$R = S_{cap} / S_{nec}$$
(3)

The capable conventional seismic loading S_{cap} is considered to be that value of the seismic loading that, together with the gravitational ones reach the resistance capacity in the critical sections that determine the resistance and the stability of the structural system.

 S_{nec} value is provided by the relation (1) in which the influence of repeated high seisms and of degradations reciprocically potentiateing themselves does not interfere at all.

For usual buildings, the nominal ensurance degree for seismic activity takes minimal values of $R_{\rm min}$ - 0.7 for the Ist class of importance, 0.6 for the IInd class of importance, 0.5 for the IIIrd and IVth class respectively.

According to [6] the synthesis of the whole activity is made by introducing the building in classes of seismic risk (for the classes $R_{si}...R_{siv}$), useful in establishing the proportions of the intervention measures and of the emergency of their execution.

On the basis of the expert examination of 250 buildings (65 schools – high schools, 45 blocks of flats, 25 historical monuments – churchs etc.), in [6], a scoregrill was elaborated for the seismic risk classes:

$$2.5 \le R_s \text{ I} \le 4.5 < R_s \text{ II} \le 6.5 < < < R_s \text{ III} \le 8.5 < R_s \text{ IV} \le 10$$
(4)

Ten criteria that can determine mainly the *seismic risk class* were selected, and, for each criterium, the score goes from 0 to 1.

- The criteria are the following:
- the seismic area of calculation,
- the category (class) of importance,
- the nominal ensurance degree R,
- undergone seisms,
- height of the building,
- antiseismic conformation,structural degradations,
- hidden degradations.

The values for each criteria are provided by Table 1. Totalized values under 2.5 are considered characteristic for the buildings that may undergo collapse next high seism,, whenever possible in Vrancea region.

To what concerns the S_{cap} value from Eq. (3),

on the basis of the expert examination of many buildings on which no intervention was made after any of the seisms of 1940, 1977, 1986, 1990 (especially blocks of flats and churches), the introduction of the notion of *structure degradation degree* is suggested.

This degradation degree should be equal to the number of high earthquakes (M > 6) the building underwent by the expert examination.

Considering a 100 - 125 span of life for a usual building, and that a high seism, according to the tendency to become more frequent, statistically, occurs every 8 - 10 year, then the maximum degradation degree should be 10 - 15

On the basis of the degradation degree equal to the number of high seisms undergone by a building, the establishing of the *degradation coeficient* (C_d) is suggested, accepting that every seism "consumes" between 3% - 5% of the resistance capacity of the structure, according to the structure type (reinforced concrete frames and diaphragms, bearing masonry). This coeficient, obviously ≤ 1.0 should influence the S_{cap} value in Eq. (3). For instance: a block of flats built in 1920, according to [6], underwent 9 high seisms (M > 6) so 27% of the resistance capacity was consumed, and the degradation coeficient will be $C_d = 0.73$. Once consolidated, the building will have the value 1.0, that is degradation degree 0. On the basis of intense expert examination as well, it is estimated that when determining the nominal degree of ensurance to seismic activities as well as when introducing them into risk classes, two more aspects that the P 100-92 Norm does not comprise should be taken into account:

- the vulnerability of the building,
- the position of the building in the locality.

There are buildings which, by their structural system are extremely vulnerable (e.g. water tower).

To what concerns the second aspect, the same building may be placed in a highly populated and full of buildings area, or in a peripherical area without any other building around, when defining R or the seismic risk class, the position of the building in the inhabited area should be represented by specific coefficients [6].

More and more specialists forecast a major earthquake $(M \ge 7.5)$ in the laps of time that includes 2011, and that is why it is considered that the intervention consolidation measures in buildings affected by the seisms in the last 25 years should be intensified.

If in Moldova hundred of schools – high schools and social-cultural buildings were reabilitated, no block of flats was consolidated, especially those designed and built before the first antiseismic protection norms.

In Bucharest, there are hundred of blocks of flats erected before 1940, that will definitely undergo collapse the next major seism. Although, for many of them, expert examinations and projects were executed, the consolidation action has not stated yet.

Table 1 Criteria - Score

Ist criterium - Seismic area of calculation

Area	А	В	С	D	Е	F	
Score	0.1	0.2	0.4	0.6	0.8	0.95	

IInd criterium - Class / category of importance of the building

Class / category of importance	I/A	II/B	III/C	IV/D	
Score	0.2	0.5	0.75	1.0	

III ^{ra} criterium - <i>R</i> nominal ensurance deg	ree
--	-----

R value		0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.05
Score	Class / category I/A	1.0	0.80	0.70	0.60	0.45	0.30	0.15	0.1
	II/B	-	1.0	0.85	0.70	0.55	0.40	0.25	0.15
	III/C	-	-	1.0	0.80	0.65	0.50	0.35	0.20
	IV/D	_	-	1.0	0.90	0.75	0.60	0.45	0.30

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			a.	usual buil	dings				
Age (years)	100	75	60	45	30	15	10	5	<
Cases	0.05	0.15	0.20	0.45	0.60	0.75	0.95	0.00	

IV th criterium -	Age of the building/ monument	

Age (years)	100	75	60	45	30	15	10	5	≤ 3
Score	0,05	0,15	0,30	0,45	0,60	0,75	0,85	0,90	1

				<i>b</i> . histo	rical mo	numents					
Age (years)	300	250	200	170	150	130	110	90	70	50	30
Score	0.05	0.10	0.15	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90

 V^{th} criterium - Seisms undergone by the building (> 6 Richer degree)

<i>a.</i> usual buildings										
Seisms	0	1	2	3	4	5	6	7		
Score	1	0.90	0.70	0.55	0.40	0.25	0.10.	0.05		

b. historical monuments												
Seisms	3	6	9	12	15	20	25	30	35	40	45	50
Score	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.25	0.20	0.15	0.10	0.05

VIth criterium - Height of building category

Number of	1	2	3	4	5	6	7	8	9	10	11	12 – 14
H (m)	3	6	9	12	15	18	21	24	27	30	33	36 - 42
Score	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.15	0.1	0.05

VIIth criterium - Antiseismic conformation (stipulations of the current norms)

Antiseismic conformation	Very good	Good	Satisfactory	Unsatisfactory
Score	1.0	0.65	0.30	0.1

VIIIth criterium - Structural degradations

Degradation state	Undegradated	Slight	Moderate	Important	Damages	Precollapse
Score	1.00	0.75	0.5	0.25	0.1	0.05

Ductility	Very good	Good	Satisfactory	Unsatisfactory	Unexisting
Score	1.0	0.65	0.30	0.1	0.05

IXth criterium - Structural ductility

Xth criterium - Hidden degradations

Hidden degradations	Any	Moderate	Important	Serious
Score	1.0	0.7	0.3	0.1

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Passive Energy Dissipation System for Framed Structures

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Abstract The paper presents a new structural system consisting of concrete frames in which is installed a passive energy dissipation system. The energy dissipation system has to accomplish the following requirements: large ductility, initial large stiffness, low yield point, adaptability to architectural functions, economical efficiency (low price, short time of construction). The system consists on special concrete panels placed in the plane of frame. Three types of panels were studied and tested. The results of experimental tests, made at static alternate forces, permitted to choose the panel, which accomplishes the requirements mentioned above.

Keywords: energy, dissipation, system, panels, experimental, tests, stiffness, ductility.

1. Introduction

The basic concept of passive control is: - to dissipate the input seismic energy ;

- to reduce energy dissipation demand in the structural elements and consequently minimize potential damage on structure.

Cladding facades and partition panels are generally considered as nonstructural elements and are not allowed to contribute any structural function to the building.

The present research explores ways to use the interaction between the panels and the structure to dissipate energy and thereby reduce building response.

2. Design criterion

The design criterion might be best formulated in terms of energy dissipation. When the partition panels are properly designed they can be used to passively dissipate significant amounts of energy through inelastic hysteretic deformation driven by interstory drift. The interaction panel structure add stiffness to the system and therefore change the dynamic characteristics of the structure. The behavior of the panel is that of an elasto-plastic system, and can contribute at the total stiffness of the frame, increasing it (Fig.1).

The stiffness of the panel must be calibrated in respect of required interstory drift of the frame;

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the seismic response of the structures accordingly the design codes gives large deformations due mainly the post-elastic behavior.



Fig.1. Hysterezis characteristics

The values of the inelastic deformations often exceed the standard's values. Consequentially we can control this deformation increasing the stiffness of the frame by the panel's stiffness.

The effective design criterion could be the ratio of the energy dissipated in the panel to the total input seismic energy for a given design earthquake: E_{panel}

/E_{input.} The objective of the design is to maximize this ratio.The aim of the study is the hysteretic behavior of the panels for seismic energy dissipation. These are panels of precart concrete composed of narrow vertical elements.

The influence of joints hysteretic behavior is also investigated in respect of passiv energy dissipation. Vertical joints are mainly subjected to shear forces which are transmited through the insitu concrete and cross reinforcement of the joint.

The design of the panel resiatance at the lateral forces due to the interaction with the frame's elements involve the design resistance of shear joints, the amount of reinforcement of the panel and the slip along the vertical joints.

The design resistance of shear joints may be determined with the formula [3]:

$$L_{jD} = \frac{1}{\gamma_d} (\beta_1 A_{key} f_{cd} + \beta_2 A_s f_{yd}) \le 0.3A_j f_{cd}$$

 β_1 , β_2 – coefficients for expressing the contribution to the joint resistance of the in-situ concrete; γ_d – complementary partial safety factor; f_{cd} – design strengh of concrete in the joint A_s , f_{yd} – the cross-sectional area of the transverse reinforcement and the design strengh of steel; A_j , f_{cd} – the longitudinal cross-section area of the joint and the design strengh of the in situ concrete.

3.Experimental program

The experimental program was caring out for demonstrate the following characteristic of the panels:

- the bearing capacity
- the hysteretic behavior
- the energy dissipation.

The experimental program was performed on three types of panels:

A. Panel composed of narrow vertical elements of reinforced concrete and cast-in-situ reinforced concrete for the vertical joints; the class of joint's concrete is equal with that of the vertical elements (Fig.2);The joints between the vertical elements are keyed joint type and has horizontal reinforcement bars which are continuously through the all vertical elements.

B. Panel composed of narrow vertical elements of reinforced concrete and cast-in-situ reinforced concrete for the vertical joints; the class of joint's concrete is less than the vertical elements (Fig.2).

The joint between the vertical elements is keyed joint type and has horizontal reinforcement bars which are continuously through the all vertical elements.



Fig.2. The A and B type panel's assembly

C. Panel of precast concrete having vertical discontinue slits (Fig.3).



Fig.3. The B type panel's assembly

The panels have the dimensions less than the frame opening. The gap between the panel and the structural elements will be filled with the insulation material.

The connectors with the structure are placed on the four corners of the panel. These are metal connectors consisting of a roller, two metal pieces and a bolt, that passes through the panel's end (Fig.4).

The experimental program consisted in testing the behavior of the panels to statically alternate loads.

The panel type A behavior was like a full monolithic one.



Fig.4. The panel position on the frame's span and detail of connector

The panel failure was after the diagonal cracks.

The panel type B behavior (fig. 5) had developed two stages of work: first stage the concrete from the joints was cracked after the diagonals of the small concrete prisms formed in

the keyed joint and the second faze when the yielding was occurred in the steel bars from joint.

The relative position between the vertical elements was modified and a drift is occurred. (Fig.6a)

The panel type C presented a failure mechanism through the vertical slits





Fig. 6. The panel behavior: a) the vertical drift of elements; b) the shear joint.

The experimental tests proved that the largest bearing capacity had demonstrated in the case of panel A and also the largest energy dissipation. Nevertheless the panel B behavior (Fig.6) can be better controlled. The stiffness of the panel can be calculated admitting that the panel work is mainly

after the diagonal directions; it can be considered

$$\varDelta x = \Theta \cdot h = \varDelta v \cdot \frac{h}{a}$$

with the following equation :

 Δx - is the horizontal desplacement of the panel; Δv - is the drift between the vertical elements; Θ - is the rotation angle of vertical element's axis; *h* - the height of the panel; *a* - the width of the panel.

The stiffnes of the panel is involved when the admissible level drift of the frame is exceeded.

The joint capacity for energy dissipating is presented in the fig. 7.



Fig. 7. The joint capacity for energy dissipating

It is related the shear force to the displacement Δ of the joint. The behavior was studied for all joints. The ductility of joints from the panel type **B** was between 3 and 8,2 which can be appreciate like good results. The energy dissipated by the joints is the surface of the hysteretic curves.

4. Conclusions

The passive energy absorbing system consists of special panels which can be placed in the frame's span. The panel is composed of narrow vertical elements which have keyed vertical joints.

The experimental tests were performed to statically alternant forces; the results demonstrated that the system has hight ductility and can dissipate the seismic energy.

The panel having the joint concrete of lower class than the class of the vertical elements is recomanded to be use due to the possibility of a better control of stiffness degradation.

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Restoration and Protection of the Structural Parts of the Timber Buildings

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Abstract: Many old buildings, part of them patrimonial, of certain architectural and historic value, have timber structural parts, especially the roofing and flooring parts. There are also some buildings, especially churches, made entirely of timber, and lasting for centuries already, with extreme symbolical value for the Romanian spirituality (as it is known the fact that in Transylvania the building of stone or masonry orthodox churches was forbidden before 1918). Their preservation and restoration implies carefully designed programs, taking into consideration the specific parameters and situation of each building. We present in the article an investigation method for the technical condition of the structural timber parts of the patrimonial civilian buildings, as well as some modern techniques of restoration and preservation of these building parts.

Keywords: timber buildings, timber roofing, timber flooring, restoration, building preservation

1. Introduction

The timber seen as building material supports a larger number of aggressive factors then other usual materials, due to its organic nature.

Its preservation during the normal exploiting conditions in a building implies a systematic control over the state of these structural parts, in order to apply the optimal solutions in due time. The humidity, the insect and fungus action, some human activities or external factors (chemical, physical, meteorological, fire, flood, terrain sliding, earthquakes) substantially increase some of the natural defects. If these deficiencies are not traced in due time, the restoration will be much more expensive and technologically complex (even impossible, implying great loss in case of patrimonial buildings).

The modern methods of investigation and evaluation of the technical condition for timber buildings, in order to apply a preservation or restoration program, implies going through certain stages:

The first stage means making the investigations for establishing:

- the architectural and historical value of the building

- the causes and dimensions of the major damages that were found and the time when

they occurred, as well as productions of their eventual evolution;

- the evolution of these damages and of the complementary ones which occurred, as well as their causes;

- the profitability of certain restoration methods as well as that of a general restoration of the building, as it might imply some financial savings

- the implications of the works upon the environment, the hygiene and the health of the users;

– the estimated price of the rehabilitation operations.

In the next stage the documents needed are drew up:

- the project of reparations, with execution details, including the appropriate technical procedures where necessary, as when used special technical methods;

- the description of the technological process of repairing or, eventually, of partial reconstruction;

- the list containing the building elements that must be repaired and replaced;

- the work progress schedule;

the tracing and quality control system.

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2. Classical and modern techniques for the restoration of the linear timber parts

Besides visual evaluations and diverse destructive methods (as acquiring samples by carving), there are modern methods, not destructive, tomographic or endoscopic (as, for example, the Pilodyn method, that allows, by the help of ultrasounds, the direct determining of the elasticity module), which provide accurate data of the damage, its evolution and causes, allowing the choice of the most appropriate remedy [1].

2.1. Repairing the fissures and leaks

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Inside the structural timber elements appear fissures or leaks under the action of the weather, of physical actions, of mechanical actions, because of structure flaws of the timber, a wrong technical execution of the elements, or other causes.

The typical solution of remedy consists in tightening the leaks with screws or special parts with shape of C or S for fissures at the end of the beams, but all these solutions require penetrations and lead to the weakening of the section.

A solution that doesn't lead to the weakening of the section is that using metallic yokes, but it is hard to put into practice in some cases because it demands penetrating the floors and it is often unacceptable from the aesthetic point of view.

For the fissures that don't exceed 5cm broad, a modern solution is to fill them with synthetic resins. This way the fissuring process is stopped, the leaks are pervaded, a potential insect or fungus aggression is avoided, and also a local and general consolidation of the element being achieved.

2.2. Repairing the structural parts

The consolidation of the stretched parts of timber structures, which are locally damaged, will use metallic joists provided with tightening parts.

For compressed parts, the most frequent damages are those of characteristic curving. Repairing such a structural part means to straighten the part, and to maintain it in this position by increasing its rigidity within the curving plan. For the curved parts, like beams, which are solicited by bending forces, due to the fact that the screw penetrations lead to the decrease of the section

resistance parameters, are applied modern methods of replacing the damaged areas with new, properly treated ones. They are joined with synthetic resins reinforced with glass fibbers, injected in orifices that are precisely calculated using particular schemes for each case, and made into both parts.

In Fig. 1 we present a sample solution of a beam repairing [2].



Fig. 1. Repairing the beam endings

A much alike solution, for elements with important solicitations at axial stretch as the roofing chords, uses steel screws mounted inside special holes made both in the replacing part and the healthy side of the beam that is going to be repaired. After the screw mounting, the holes will be filled with synthetic resins.

After mounting, the joints will be blocked with transversal blockages made of synthetic resin injected inside channels of diameter of 8 to 10 mm and reinforced with glass fibbers, in order to consolidate the area. In Fig. 2. We present the repairing of a roofing chord, which is a stretched part, by removing the damaged area and replacing with a new one [2].



Fig. 2. Repairing a roofing chord

The areas of the structural elements that are most exposed to deterioration are the ones in less ventilated places (beam endings not enough protected inside the brick layers) but also older structures that are not well protected against the action of insects, fungus or humidity. The typical solution of repairing is that of replacing the damaged part with a new part (linked with screws), or with a metallic joist. For compressed elements as pillars the solution mentioned has good results if any eccentricity is avoided, but for beams solicited by bending forces this solution leads to a decrease of resistance, because of the screws.

The beams solicited to bending forces and have damages in the maximal point of the bending moment can be also consolidated with timber or metallic joists. In order to avoid the section weakening that may occur due to the screw penetrations, long joists are used, in order to place the penetrations as far as possible from the maximal bending moment spot.. such beams can be also repaired by suspending them by other bearing parts, which can support them, with yokes [3].

There can also be adopted solutions of consolidating the entire length of the beam by doubling with joists that could sustain entirely or partially the loads of the beam.

If we want to increase the bending rigidity of a specific beam, a solution will be that of consolidating them by suspending them with steel wires, placed in various spots of the beam, as needed, related to the beam opening and the bearing values – Fig. 3.



Fig. 3. Increasing the rigidity of a beam

3. Modern solutions of efficiency in restoration for timber flooring

The old dwellings usually have flooring with one direction beams, or two direction beams, primary and secondary, both ways not supported by rigorous calculations, and generally over-sized. Either for level flooring or attic flooring, the phonoinsulation and thermo insulation are usually made of heavy materials, such as slag, earth, sand. If we remove these materials, replacing them with modern, light materials (mineral wool, extruded polystyrene) we'll reduce the bearings as to place a concrete slab over the timber floor.

That is a solution for the timber old buildings situated in seismic areas, as these areas require an increased horizontal rigidity, which makes the mixed timber-concrete flooring solutions very appropriate.

For applying this method in the restoration of an old timber floor, first of all it is required that we check and repair, if necessary the damaged resistance parts, using either classic or modern methods and materials: joists, yokes, synthetic adhesives and epoxy plasters, or, in certain situations, the complete removal and replacing of the decayed beams. After these preparations are completed, we can go through to the next stage that is mounting the concrete slab with connectors and diverse supporting parts if necessary. The horizontal rigidity of the mixed timber-concrete floor will be obtained with concrete belting on the slab perimeter, mixed within the brickwork and fixed there with steel anchors.

These mixed timber-concrete floors correct some deficiencies of either the timber - only and concrete –

only floors, as following: in the compressed area of the section we find the concrete, which responds best to compression, and in the stretched area we find timber, which, if no deficiencies, has an excellent behavior to this type of solicitation [4].

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This way, a structurally efficient section is obtained, as it has the properties of great rigidity and lightness. Its rigidity is three to four times higher then that of a timber floor, and a bearing capacity about two times higher, for an insignificant heaviness increase. In Fig. 4. we show comparatively the heaviness of the mixed timber – concrete floors, with the timber and concrete – only ones, for the same openings and bearings.



Fig. 4. The floor heaviness (g) in relation with the opening width (l) for a loading of 2.5 KN/m^2 ;

- a) timber floor
- b) timber and concrete floor
- c) concrete floor

The efficiency of this type of flooring is emphasized not only by the decreased expenses, but also by the shown heaviness advantages, a higher execution speed, as the decofration process is avoided. Thus, the mixed structures obtained by this method make an appropriate way of restoration of the timber old buildings, providing an ensemble behavior of much higher performance compared to the initial situation, without changing the building's outlook.

3.1. The mixed timber – concrete flooring technology

The most difficult technical issue of this method is the joining of the two materials, due to the fact that they have different mechanical and hygrothermal behavior.

Thus, the joining is safes with connectors, such as nails, screws, steel parts, thrushes, and so on. Some of the connectors have their own joining systems (screws, nails) the others must be fixed with synthetic adhesives, such as the epoxy resins. In this case we should take into consideration that the adhesive be strong enough and respond to both

surfaces, its strengthening be not very fast, not to release toxic gases, not to change its properties in time.

3.2. The mechanical behavior of the mixed floors

In case that the joining are made of rigid connectors, the calculations will use for the concrete section an equivalent timber section, with the same gravity center, the sliding between the two materials being nil.

If there are used semi-rigid connectors, the sliding between the two will be taken into account during the calculation process. The parameters of the joints mechanical behavior and the rigidity of different types of connectors determine the effort distribution inside the structure. For the connectors, the specific sliding module is defined by the EUROCODE 5.

The calculations that have to be proceeded take place in the field of the elasticity, considering the concrete as well as the connectors from the elasticity behavior point of view. We consider the concrete with no fissures, but if starching efforts appeared we have reinforcement to calculate.

An issue that can occur and which should be taken care of is that of the dimensional variations of the concrete when contracted within the mixed structure. Thus we have supplementary efforts within the connectors, which imply deformations of the entire structure, which lead to new compression and bending solicitations.

4. The protection of the timber structural parts

The climate factors (wind, rain, sun) have a complex action on the timber and timber products. These factors cause erosions, fissures, color and aspect alterations, dimensional changes, resistance modifications depending on the wood species and on the climate zone of the building.

As it is an organic product and contains cellulose and sugars, the timber is attacked by wood - eating insects, and it is, in some conditions, a favorable environment for fungus.

Because of its physical and chemical properties, the wood decomposes progressively at temperatures over 105° C, emitting flammable gases (hydrogen, methane, ethane). During the process of heating, under open flame, at 220° - 250° C, the emitted gases burst into flame, and at 260° - 290° the wood burns with continuous flame. If no contact with the fire, the wood bursts into flame at 330° - 470° C [5].

Preventing the negative effects of the biological agents against wood is obtained in the first place by reducing the humidity of the wood under 20 - 30% and keeping it at this constant as long as the building will last. More, treating the material with disinfecting substances imposes. These substances are to be found on the market under different commercial labels, being either liquid or paste. The treatment methods are, basically, two: thoroughgoing (as the osmosis, impregnation under pressure) and superficial, covering the surface by painting or such.

Both methods require a certain preparation of the timber parts that are to be treated (they must be dry, have no damages produced by insects or fungus, and they must have the final shape).

The wood can not be prepared in order to be not combustible at all, but it can be made hard combustible by constructive solutions (for existent buildings) or chemical solutions as impregnating the timber parts, before using them in construction, with fire-protecting substances.

Among the constructive solutions currently used we mention: painting the surfaces with sodium silicate (soluble glass) suspension or other substances that form isolating coating even under the flame like certain synthetic resins with nitrogen, plastering upon wire netting; building protective brick walls round the sustaining elements (poles); plating the doors with sheet steel in order to restrain the flame from passing to other room.

Chemical solutions refer to the impregnation of the timber with fire protecting substances, that will emit the crystallization water under the action of heat and will create a protecting coating, or will decompose and will emit a large quantity of not burning gases which will keep the oxygen away from the timber surface and dilute the burning gases emitted by the wood mass.

Impregnating the timber with protecting substances is a safer solution than that of covering with fire protecting paints, because the substances penetrate deeper into the wood [6].

There are technical solutions which acquire a deep impregnation of the parts that are already set in place; they usually are local injections made for treating the timber of old roofing's against either fungus or insects (the Cobra method, the Dolger – Wolman method, and so on).

Because the timber must be protected as well against fire and fungus, for decreasing the time of operating and for decreasing the expenses, usually in the fire protecting solutions are introduced antiseptic substances, too.

Impregnating with fire protecting substances is usually mixed with the impregnation with antiseptic substances, as the solutions are combined, and the medium consumption of composition is relatively large (5-8 % mass of wood to impregnate).

An ingenious an relatively simple solution is that of treating the roofs affected by insects and/or fungus with overheated air. The observation of the fungus or insect presence and estate is made directly, visually, and then with specific sound devices. Before applying the overheated air blow in the attic of a building timber roof, the dust is removed from the space with industrial vacuum cleaners then the place is cleaned and tightened. In Fig. 5 we show the distribution of the overheated air inside a roof with or without dwelling conditions.

The lasting of the overheated air blow is determined by the temperature of the air, and will last until the complete destruction of the fungus or insects.



Fig. 5. The distribution of the overheated air within a roof with or without dwelling conditions

5. Conclusions

The structural parts of the timber buildings often have specific defects, due to the characteristic physical and mechanical properties of the wood material. They are also attacked by fungus and insects, as determined by the organic nature of the wood.

For maintaining these structures, some of great importance, due to their historical and architectural relevance, proper solutions are required for preserving the structural safety and also the original aspect of the patrimonial buildings. Such solutions imply the use of modern materials and techniques, approaching the domain of the composite materials (mixing timber with epoxy resins, glass fibbers, concrete, metals), as well as methods of treating the timber against fungus, insects, fire, and physical aggression piece by piece but also *in situ*.

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Seismic Behavior of Steel Structures for Serviceability Limit State

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Abstract New methodology to design structures against seismic actions is developed in very recent decades based on multi-levels: serviceability, damageability and ultimate limit states. The results of practical applications have shown that, in many cases, the serviceability limit state is dominant in design process. Unfortunately, there are no sufficient information concerning the structure-nonstructural elements interaction and proper interstorey drift limits. The paper presents some aspects of the structure verification for serviceability limit state.

Keywords: Earthquake, serviceability limit state, interstorey drift.

1. Introduction

Traditionally seismic codes consider that the life safety is the main goal of the design. But the last important earthquakes, due to the economic and social aspects, have shown that the loss of function has become as much important for seismic design. So, the control of damage represents the key factors especially in steel moment resisting frames, which are usually conditioned by interstorey drift limitation.

Due to this situation, new methodology to design structures against seismic actions is developed based on multi-levels: serviceability, damageability and ultimate limit states. But this methodology is far to be introduced in design practice due to difficulties to change the mentality of designer accustomed to verify the structure response only for one limit state, the ultimate one. Therefore, the developing of a simplest possible methodology, which can be introduced in seismic code, is an urgent requirement.

The paper presents the aspects for the structure verification for serviceability limit state, which must be considered for improving the Romanian code provisions P100/92 for seismic design of steel structures.

2. Serviceability limit state (SLS)

2.1. Assessments of SLS

Unfortunately there is not clear criterion to determine the acceleration corresponding to the SLS.

(i) P100/92 (Romanian code) [1] considers only one limit state, the ultimate one. So, a very important principal mistake is introduced in design by this code. By using the coefficient of reduction ψ some damage in structure are allowed. Contrary, the nonstructural elements are protected, using the interstorey drift limits, against damage. This mistake must urgent eliminate from code, because, especially for steel structures, lead to an important overstrength of structure.

(ii) EUROCODE 8 [2] recognize two different limit states for the structural design. Ultimate limit state (ULS) is mainly addressed to life safety for strong earthquakes, while serviceability limit state (SLS) aims to avoid premature loss of functionality for frequent earthquakes. But the design is based on a primitive methodology, by using the same spectrum for both limit state, the SLS being obtained from ULS by reducing only the acceleration values. This methodology ignores the fact that the characteristics of ground motions are influenced by the earthquake magnitude, the spectra for strong earthquake being very different of the weak earthquakes. The simplification in design by considering the same spectra for both earthquakes is used in order to simplify the work of designers, but leads to a overstrenght of structures which are governed by SLS.

(iii) Gioncu-Mazzolani (RSD) methodology [3] consider three limit states for structural design: serviceability limit state (SLS) for frequent

earthquakes, damageability limit state (DLS) for occasionally moderate earthquakes and ultimate limit state (ULS) for very rare strong earthquakes. For SLS the interstory drift limits are verified through the rigidity of structures (R), for DLS the strength of sections is checking (S) in order to verify if the damage structure is reparable without great difficulties and for USL the ductility capacity is verified through the rotation capacity (D), in order to prevent the structure collapse. For SLS an elastic analysis is used with all the structural and non structural elements undamaged. For DLS an elastoplastic analysis is performed, considering damage of all elements. For ULS a kinematic methodology must be used, the structure being transformed in a global plastic mechanism. The methodology considers the differences in spectra for three limit states and for different earthquakes types, interplate earthquakes (Vrancea) and intraplate earthquakes (Banat), (Fig.1).



Fig.1. Design spectra One can see that for SLS the amplification is heigher than for DLS, but the corner periods are shorter. The

same differences may be observed between interplate

and intraplate earthquakes. For ULS, when the structure is transformed in a global mechanism, no amplification of accelerations occurs.

The following relationship are proposed for SLS:

-interplate earthquakes

$$\beta_{s}(T) = \frac{0.8\eta}{T^{1.1}}; \qquad \beta \le 3.25$$
 (1a)

- intraplate earthquakes

$$\beta_s(T) = \frac{0.18\,\eta}{T^{1.4}}; \qquad \beta \le 4.5$$
 (1b)

The accelerations corresponding to the each limit states is determined from relationship:

$$\frac{a}{a_d} = \left(\frac{P_r}{P_{rd}}\right)^{0.28} \tag{2}$$

where a_d is the acceleration corresponding to DLS determined for a return period of P_{rd} =475 years. For serviceability limit, determined for a return period of P_{rs} =20years, results the acceleration

$$\mathbf{a}_{\rm s} = 0.412 \cdot \mathbf{a}_{\rm d} \tag{3}$$

So, the proposed methodology considers in a proper way the characteristics of seismic actions corresponding to SLS.

2.2. SLS as key-factor in structural design

Frequently, serviceability limit states are determinant in designing steel building, especially in case of moment resisting steel frames in seismic area, for which recent codes provide prohibitive interstorey drift limits.

The shear forces determined for spectra are:

- SLS:
$$F_{bs} = \frac{a_s}{g} \beta_s (T) W$$
 (4a)

-DLS:
$$F_{bd} = \frac{a_s}{g} \beta_s (T) W \frac{1}{q}$$
 (4b)

where g is gravity acceleration, W, structure weight, T, natural period of structure, T_d , natural period of damaged structure, q, the behavior factor (q=1/ ψ).

The ratio between the shear forces corresponding to the two limit states results from the equations (4a, b):

$$\frac{F_{bs}}{F_{bd}} = \frac{a_s}{a_d} \frac{\beta_s(T)}{\beta_d(T)/q}$$
(5)

The ratio (5) is plotted in Fig.2. One can see



Fig.2. Shear forces of damageability and serviceability limit states ratios

that it depends on earthquake type, structure elastic period, behavioral factor and soil conditions. The main factor influencing this ratio is the q factor. For q greater than 4.5 the control of design is given by SLS, showing that the benefit of large behavioral factor may be sometime unprofitable. Fig.2c shows that the use of EC8 provisions leads to an enlarging of the serviceability field. Therefore the interstorey drift checking begins to be the key factor of design, providing considerable overstrenght for members.

2.3. Interstorey drift limits

The interstorey drift control of a structural system is important in order to maintain architectural integrity of nonstructural elements and to avoid human discomfort during the weak but frequent earthquakes. Information concerning the values of available interstorey drifts is given by Mayes [4], Nair [5] and Galambos [6] for additional partition walls (Fig.3). Freeman [7], reported tests for gypsum wallboard and gypsum plaster partitions. De Matteis [8] research works are involved with the trapezoidal sheet infill panels.



Fig.3. Interstorey drift limits for different limit states

In the P100/92 considers the limits d_{lim} =0.0035h for nonstructural elements with brittle behavior and d_{lim} =0.0070h for case than no interaction structurenonstructural elements exists. EC8 considers d_{lim} =0.004h and d_{lim} =0.006h for the same cases. A less severe conditions d_{lim} =0.008h is also proposed in the analysis of EC8 in order to be introduced in practice.

Analyzing the research works and code provisions one can observe that the stage of knowledge on interstorey drift limits does not appear to be advanced in the recent year. One of the most important criteria, which, in many cases, can determine the structure sizing, is based on the obsolute information. So, this is a field of future research works.

3. Panel action in drift control

Because the SLS can lead to a very important overstrenght of structure due to very severe interstorey drift limits considered in codes, a very important design problem is to reduce this source of increasing of structure weight. One way is to consider interstorey drift limits determined in more adequate way that the actual provisions of codes. But this is a problem of future research works. Other way is to consider the effect of panels in the determination of deformability of steel structures.

(i) Panel types. The panel types are presented in Fig.4: cladding panels, masonry panels, gypsum panels and wood panels. The most used panels are the cladding panels with the typologies presented in Fig.5.



Fig.5. Sandwich panels typologies

(ii) Equivalent braces. The most usual modeling is the proposal to replace the infilled panels by an equivalent pin-joined X bracing (Fig.6). The stiffness

of composite system structure-nonstructural elements can be expressed as [9]:



Fig.6. Modelling of cladding contribution

$$k_{eg} = k_f + k_p \tag{6}$$

where $k_{\rm f}$ is the lateral stiffness of the bare frame

$$k_f = 2 \cdot \chi(\beta) = \frac{E \cdot I_c}{L_c^3}$$
(7a)

$$\chi(\beta) = 12 \cdot \frac{1+6 \cdot \chi}{4+6 \cdot \chi} ; \quad \chi = \frac{I_b / L_b}{I_c / L_c}$$
(7b, c)

and k_p is the panel shear stiffness:

$$k_{p} = \frac{E_{d} \cdot A_{d}}{L_{d}} \cdot \cos^{2} \varphi$$
(8)

For masonry infilled frames the effective width w_{ef} is given by Mainstone formula [10]:

$$w_{ef} = 0.175 \cdot (\lambda_h \cdot H)^{-0.4} \cdot (H^2 + L^2)^{1/2}$$
 (9a)

$$\lambda_{h} = \left(\frac{E_{i} \cdot t \cdot \sin 2\varphi}{4 \cdot E_{c} \cdot I_{c} \cdot H_{c}}\right)^{1/4}$$
(9b)

where H, L are the height and span of the frame, E_c , E_i the elastic module of the column and the infill panel, t, the thickness of panel, I_c , the modulus of inertia of the column.

4. Numerical tests

The building (Fig.7) is placed in a seismic area



Fig.7. Worked example-structure subjected to earthquake

(interplate and intraplate) and two interaction types (without and with collaboration of infilled panelmasonry of 15cm for interior walls and 30cm for exterior ones. The interstorey drifts are presented in Fig.8 together with the limit values imposed by EC8. One can see the great differences between the interplatee and intraplate drifts due to the differences in spectra for the two earthquakes. In the first case without considering the interaction with infilled panels, the limit values are exceeded. Contrary, in the case of intraplate earthquake, these values are respected. A very efficient design method to reduce the interstorey drifts is to consider the interaction structure-nonstructural elements. So, by using this interaction, even for interplate earthquakes the interstorey drift limits are satisfied.





with strong earthquakes with $a_d=0.36g$ (resulting from (3) $a_s=0.15g$), with the subsoil corresponding to class C (S=1.35) and the damping factor $\eta=1.2$. The structure analysis is performed with SAP2000 computer program for two earthquake types

5. Conclusions

The design methodology based on three limit states allows to verify the structure behavior in very clear conditions. The serviceability limit state can plays a leader role in many cases, especially if a high value of behavioral factor is used. In this cases, an overstrenght of structure results. In order to equilibrate this situation, two earthquake types are considered, the interplate earthquake (Vrancea) and intraplate earthquake (Banat) with different spectra. In additional, the interaction structure-nonstructural elements are included in analysis, with a very important reduction of interstorey drifts.

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Seismic Performance of a Strengthened Reinforced Concrete Water Tower

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Abstract: Surrounded by several extremely important industrial buildings, utilities and equipments, still sits a 42 m height reinforced concrete water storage tower that provides the local domestic and industrial water supply. The water tower has been severely damaged by recent Romanian earthquakes (1977, 1986, and 1990). The potential for water tower failure during a soon foreseeable earthquake - what Romanians call "The Big One" - is a considerable risk to the development of the unique Romanian synthetic fibres factory ("Moldosin" S.A. – Vaslui). To estimate the degree of risk and the potential consequence of water tower failure is not an easy task. Although the present regulations require that the minimum seismic safety degree of the water towers should be of only 0.5, still, these structures must be often included in a superior risk class, because after a collapse, numberless collateral damages may emerge. The focus of the expertise was to demonstrate to the beneficiary that the water tower will not fail if the proposed strengthening solution will be applied. The seismic response of the strengthened water tower is very complex, and two different accelerograms has been applied for a time history analysis - El Centro, Mexico (1940) and Vrancea, Romania (1977). In the nonlinear analysis the soil-structure interaction has been included to investigate the energy dissipation and/or local dynamic amplifications.

Keywords: water tower, seism, consolidation, safety level.

1. Introduction

The severely seismic events of the last 25 years have seriously affected many civil and industrial buildings of Romania and not only. These damages, together with the lack of professionalism and responsibility of some builders, constitute a serious problem for all those specialists who try to find out solutions for a rapid, safe and rational strenghtening. Increasingly stringent safety requirements for water tower make it necessary to demonstrate the integrity of the structure during a rare, but credibile seismic event. Since this structure is in continual use, complex structural modifications and strengthening would be very difficult, costly and intrusive. Although the present regulation require that the minimum safety level should be of only 0.5, still. these structures should be often included in a superior risk class because if these ones are damaged or will collapse, priceless damages may emerge. In what follows, a fast and efficient strengthening solution is presented in a very concise way for the ISSN-12223-7221

water tower of 300 m³ which belongs to a strategically industrial group in Romania - "Moldosin" SA Vaslui. This strengthening method could be extended as a standard project.

For the transient analysis of the strengthened structure all kinds of interactions (soil-structurefluid) have been considered during two very different seismic events. Previous linear analysis, based on conventional design-code limits, showed that the tower would be significantly overloaded during an earthquake. The assumed linear elastic response gave bending moments much greater than allowed. It was recognized that failure to obey the design code did not imply functional failure. However, significant departure from linear behaviour made it difficult to predict the forces in the concrete structure, and impractical to apply conventional static design code criteria. A non-linear analysis of the structure, working beyond first yield of steel reinforcement and using fundamental failure criteria for concrete, was required to gain confidence that the structure would support a major seismic event.

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2. Structure description. Strengthening solution

The height of the tower is of 42 m, the main tower having 34,70 m, 3,2 m in diameter, and a medium thickness of the wall of 23 cm. The thickness of all six quasi-equidistant reinforced concrete platforms is of 12 cm. The water tank has a total height of 7,30 m, 8,50 m in diameter, and the thickness of the wall is 12 cm. The 44 reinforced concrete piles - circularly and equidistantly disposed, of 6,00 m in length, and having a side of the square cross section of 35 cm - are top connected with a circular raft foundation of 8,50 m in diameter.

The adopted strengthening solution consists in: grout injections of all cracks with an epoxy-type concrete, encapsulating the supplementary slight post-tensioned reinforcements with an uniform covering of 15 cm of high-strength concrete, and 5 radial reinforced concrete counterforts. The transient analysis was carried out for two different accelerograms: El Centro-1940 (Mexico), and Vrancea-1977 (Romania). It is obvious that before the concrete covering to be made, the whole tower surface must be slightly hammered, in order to get a continuous bondline with the old structure. The existing tower should not be drilled for any reason (either for technological or for an anchorage of the new steel reinforcements). No steel connections with the old structure are necessary.

At the base of the tower the proposed 5 counterforts must have a medium thickness of at least 40 cm, a minimum height of 4,00 m, extended on a radius of 7,24 m, and which partially loaded an additional reinforced concrete raft ring of 26.40 m in diameter, which incorporates the existing foundation. The counterforts reinforcements consists in a mixed structure made of rolled sections and ordinary steel reinforcements (Fig.1). The new raft foundation could load a supplementary set of 48 reinforced concrete drilled piles, circularly disposed, in order to get an extra-anchorage of the tower against a possible pulling out, although from the numerical analysis seems to be an overmeasure.

The mechanical properties of the soil were taken out from the original geotechnical reports, and are presented and discussed in the following chapter.



Fig. 1 - Proposed strengthening solution

3. Numerical Model

The numerical model used for strengthened structure in the nonlinear time history analysis had **23188** nodes and **20443** finite elements (Fig. 2 and Fig.3) from which:

- 8 nodes solid finite elements for the structure, (elasto-plastic material) – 5967 elements:

- 8 nodes solid elements for the water in the water reservoir (elastic-fluid material) - 1296 elements;

- 8 nodes solid elements for the soil, (hysteretic soil material) – 11770 elements;

- 4 nodes shell elements (elasto-plastic material) - 1050 elements;

- seismic beams for the reinforced concrete piles (elasto-plastic material) – 120 elements;

- elasto-plastic springs for modelling the soilpile interaction – 20 elements.

Since the seismic response spectra and local possible amplifications are unknowns. Consequently, the model was studied for two completely different accelerograms - El Centro - 1940 (Mexico), and Vrancea - 1977 (Romania). For the second nonlinear analysis a supplementary variant has been performed, considering for the soil, slightly weaker mechanical properties.



Fig. 2 – The numerical model of the water tower

The time history analysis was carried out on a Pentium III System (Intel P III/600 MHz, 128 MB RAM) considering 5 seconds of maximum amplitudes of ground accelerations related to each accelerogram. The elapsed CPU times (to evaluate the final structural response of one numerical model) are presented below, for each phase, and it must be mentioned that there were no substantial differences between the numerical models:

- dynamic relaxation phase 3 h 46';
- time-history analysis itself 26 h 38';
- post-processing phase ~ 3 h.

The geometrical and mechanical properties of the model were considered as follows:

- interior tower: $d_{int} = 3.24m$, g=23 cm -C.12/15: E = 240 000 daN/cm², v = 0.2;
- external covering: d_{ext} =4.00m, g = 15 cm C.20/25: E = 300 000 daN/cm², v = 0.2;
- interior floor diaphragms, roof, water reservoir: g=12cm, C8/10: E=210 000 daN/cm², v = 0.2;
- water: $300 \text{ mc} \text{B} = 21\ 000\ \text{daN/cm}^2$
- counterforts: $g_{med} = 40$ cm, 1 = 5.03 m C.20/25: E=300 000 daN/cm², v = 0.2;
- existing raft foundation: D = 8.50 m, g = 2.00 m, C.12/15: $E = 240\ 000\ \text{daN/cm}^2, \nu = 0.2$;



- $$\begin{split} D_{ext} &= 13.20 \text{m}, \ d_{int} = 8.50 \text{ m}, \ g = 3.00 \text{ m} \\ C.20/25: \ E &= 300\ 000\ da\text{N/cm}^2, \ \nu = 0.2; \\ \text{- reinforced concrete piles: } 35 \text{ x} 35 \text{ cm}^2, \ l=6.00 \text{m}, \\ C.20/25: \ E &= 300\ 000\ da\text{N/cm}^2, \ \nu = 0.2; \end{split}$$
- soil:
- layer 1 (sandy loam): g=7.50 m, $D_{ext} = 12.00$ m, layer 2 (clay marl): g=4.50m, $D_{ext} = 12.00$ m.



Fig.3 - Vertical cross section through the numerical model



The geotechnical information included in the expert's report was totally insufficient to "build" a proper numerical model for the soil, as real as possible. Consequently, the evaluation of the shear

response (Fig. 7) versus the specific deformations, were established correlating the information about similar soil conditions presented by specific literature, from both national and international papers. For the numerical models I and 2b, the shear modulus for sandy loam and clay marl were considered of 200 kN/m² (at 1.75 t/m³ density), and, of 350 kN/m² (at 1.80 t/m³ density), respectively, for which the angles of internal friction are: 10° for saturated soil, and 7° for a semi-drained soil (two thirds of water saturation).



Fig. 5- The Vrancea accelerogram (Romania 1977)



Fig. 6 – The El Centro peak accelerogram used in the time history analysis

For the second case of the time history analysis with the Vrancea accelerogram, the shear modulus values were reduced to 150 kN/m^2 , all other parameters remaining unchanged.

Because between -1,50 m and -2,80 m a layer of fine sand with sandy-gravel mixture was

identified, a special numerical model was tried for which a volume expansion has been considered. This is a specific phenomenon of the granulated soils when the particles subdued one-other, revealing, in fact, the earthquake absorption energy.



Due to the lack of some fundamental data of the expert's reports and of the geotechnical one, even for average data indicated in specific literature, it couldn't be obtained a minimal numerical stability of the numerical model. Hence, such an evaluation of the structural response has been abandoned. It must also be mentioned that only due to the stratification shown in the geotechnical report, the dynamic amplitudes could be increased up to 10 times!

This is why, although the structural response was very laborious and carefully evaluated, it should be considered as an informative one. Thus, a hypothetic accidental amplification should not be excluded, since the site seismic response spectra of is unknown, and the dynamic characteristics of the accelerograms could not be properly estimated.

4. Results, conclusions and recommendations

Case 1. – Because the statistical predictions for the years 2000-2010 reveal that a major seismic event is expected, much bigger than the last one occurred in Vrancea 1977, from the El Centro (Mexico 1940) accelerogram a maximum seismic peak area was selected, between moments 1.168 s and 4.836 s, and the values of those accelerations were increased by 50% due to the possible severe event.





The favourable behaviour of all three numerical models - with small relative displacements at the top of structure, and a balanced distribution of the kinetic energy (Fig. 8) between the water tower itself (part 1 - the existing internal tower; and part 2^{-} the proposed external covering), and the water (part 8) -, is obviously due to the fact that the natural period of the considered accelerogram was completely different by the natural period of the water tower (T=2,2 s). As it was also expected, the increased amplitudes of the accelerations don't lead to a significant increase of the relative displacements values (the maximum allowed relative displacement -H/350 = 12 cm).

Since the natural period of the Vrancea accelerogram is of about 1,5 s, indicating that an oscillation in the resonance domain could be expected, the values of the peak accelerations were increased only by 25%, and the structure has been studied in two different hypothesis, for two different values of the shear modulus of the interacted soil, as follows:

Case 2.a:

- for layers between -2.00m and 7.50m
- $G=150 \text{ kN/m}^2$
- for layers between -7.50m and 12.00m $G=300 \text{ kN/m}^2$

The relative displacements and the internal efforts for this case of structural analysis are much smaller than of the next studied case, and only the variation of the relative displacements has been presented (Fig. 9).



The small values of the absolute displacements recorded for this numerical model may be due to the great deformation ratio of the soil, which seems improve efficiently the seismic energy absorption. More, as it was also shown, because of lack of the geotechnical data regarding the mechanical properties of the soil foundation, and since there is an obvious risk of local dynamic amplification on site, our opinion is such a structural favourable behaviour is not likely to occur (the maximum relative displacements recorded was 10.40 cm).

Case 2.b:

- for layers between -2.00m and 7.50m $G=200 \text{ kN/m}^2$
- for layers between -7.50m and 12.00m $G=350 \text{ kN/m}^2$

The values of the peak accelerations of Vrancea accelerogram (1977 – N-S direction) have also been increased by 25%, all the other data and mechanical characteristics of the model being maintained.

For this numerical model, the relative displacements, given those of the case 2.a, may be due to the "viscosity" effect of the soil foundation, which seriously amortizes the foundation's change of place, phenomenon almost similar with an action of a typical seismic shock (the relative maximum displacement – 16.78 cm, see Fig. 10).

Although in this case the maximum relative displacements of 16.8 cm exceeds the maximum values allowed by Romanian codes (H/350 = 12 cm), it may be considered that numerical model has a favourable structural behaviour during this severe seismic event.

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Fig. 11 – Variation of the resultant shear force [N] (tower cross section at +4.00m)



Fig. 12 - Variation of the resultant bending moment $(M_x+M_y+M_z)$ [Nm] (tower cross section at +4.00m)

The maximum relative displacements was evaluated between the tower top (water reservoir edge) and to the tower cross section at +4.00 m (corresponding counterfort top).

The internal efforts and stresses developed in the structural elements, are, at a first sight, within the expected limits (also prescribed by Romanian codes). A final check is required after the proposed strenghtening method will be applied, and the real data regarding the new implemented steel reinforcements will be considered.

In Fig. 11 and Fig. 12 two graphs are presented: the variation of the resultant shear force and the variation of the resultant bending moment in the critical cross section at +4.00 m (immediately above the top of the counterforts).

The structural response of the strenghtened tower was evaluated with five counterforts having an average thickness of 40 cm, but due to the special situation of the placement, it may be also studied a variant with four counterforts, orthogonally disposed, with an average thickness of 45 cm, but two of them must to be placed, if possible, on the propagation direction of the seismic wave from Vrancea epicenter.

For the external concrete covering a ductile reinforcements (like OB.37) is recommended to be used, and the new reinforcement percentage to be slightly superior (15-20%) to the existing one in the old tower. The new reinforcement will be slightly post-tensioned, but avoiding any possible damage of the old tower by local crushes. The steel reinforcement of the concrete covering will be fixed in a reinforced concrete ring beam, placed at the base of the existing tower, and the counterforts will have a mixed reinforcements with rolled steel profiles and ordinary steel bars as it was shown in Fig. 1.

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Semirigid Steel Structures. Numerical Studies

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Abstract: The semirigidity of frame type structures is presented and incorporated in structural analysis as a geometrical non-linearity. Several analytical bending moment – relative rotation M- θ_r models are introduced and discussed. The non-linear analysis is performed via general displacement method and the load – displacement $(P - \Delta)$ curve is used as an instrumental technique for several study cases.

Keywords: semirigidity, steel frames, geometric non-linearity, relative rotations, $P - \Delta$ curve, ductility.

1. Introduction

Erected steel frame type structures present, alter alia, a specificity leading to clear cut consequences on analysis techniques and their results: the rigidity of the connection, if it is not a hinge. A perfect rigid connection leads to uniform distribution of the rotations of the connecting elements. The discontinuity of a beam – column joint, for instance, exhibiting rather a hinge, corrected through a connecting detail (as web angles) to ensure continuity is, finally, neither of them. It is a flexible connection - a *semirigid joint*. Several features of such a joint are usually ignored (the flexibility of the joint, its form and final dimensions, the geometrical imperfections) and an ideal intersecting point and rigidly fixed elements are considered instead. Basical analytical hypotheses derived from such an ideal connection are stil in use.

The semirigidity of real connections is proved and generally accepted. It came a long way since it was signaled and regarded as an imperfection of the connection to nowadays status as a connection on its own defined in several design codes.



Fig. 1 AISC connections diagram

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Fig. 2 EUROCODE connections diagram

A fundamental concept of the semirigid behaviour is the relative rotation θ_r allowed in such a connection and its nolinear relationship M- θ_r with the associated bending moment M. The American Institute of Steel Construction introduced in 1989 [1] three types of connections (Fig. 1). EUROCODE 3 also recommends [2] a similar practice (Fig. 2) in the design of steel frames.

The important role of M- θ_r relationship is clearly seen and, therefore, a great effort has been invested in the problem of mechanical and analytical models of semirigid connections.

2. Analytical models *M*- θ_r

The large number of experimental and analytical investigations carried out lead to the general acceptance of a compact group of analytical models M- θ_r . The M- θ_r relationship is governed by several mechanical and geometrical parameters and, also, depends on the connection type. Extensive presentations of M- θ_r may be found in [3] and [4]. A basic classification of M- θ_r analytical models may be based on their mathematical form and type and degree of nonlinearity [5]:

- Linear models
- Polynomial models
- Cubic models

- Exponential models
- Differential models

Associated relationships introduced several mechanical parameters of the connecting (angles, bolts) and connected (beam, columns) elements. The most important parameters are: initial stiffness R_i of the connection, relative stiffness of the beam, ultimate bending moment of the connecting section M_u . Also, the geometry of the connected elements (usually of beam – column type) is involved through beam length, its moment of inertia. The ultimate bending of the beam is referred to as M_{cap} .



Fig. 3 Top and seat double web angle connection

The experimental investigations conducted on several beam – column usual types of connections proved the necessity of associating a specifyc *M*- θ_r model to a given beam – column connection type.

The numerical studies presented in what follows have been conducted on steel structures with *top and seat double web beam* – *column* connections (Fig. 3). Several analytical *M*- θ_r (1), (2), (3) may be associated to this type of connection:

• Polynomial model (1)

$$\theta_r = C_1 \left(KM\right)^1 + C_2 \left(KM\right)^3 + C_3 \left(KM\right)^5 \qquad (1)$$

• Exponential models (2)

$$\theta_r = \frac{\left|M\right|}{K_i} \frac{1}{1 - \left|M / M_u\right|^n} \tag{2a}$$

$$\theta_r = \frac{M}{K_i [1 - (M / M_u)^n]^{1/n}}$$
(2b)

$$\frac{M}{M_u} = n \left[\ln \left(1 + \frac{\theta_r}{n\theta_0} \right) \right]$$
(2c)

• Differential model (3)

$$\frac{dM}{d\theta_r} = R_i \left[1 - \left(\frac{M - M_0}{M_u - M_0} \right)^c \right] \quad for \quad M > M_0$$
$$\frac{dM}{d\theta_r} = R_i \quad for \quad M \le M_0 \tag{3}$$

3. Numerical Studies

The behaviour of several spatial and planar steel frames has been analyzed in several cases of beam – column connection and, also, in the case of rigid beam – column connectivity for the purpose of comparison [4].

The criterion of comparison is the load – displacement $(P - \Delta)$ curve. The loading consists of two classes of forces: a gravitational constant uniformly distributed force and a system of lateral

(horizontal) forces applied at floors levels controled by a increasing load parameter λ .

The structure is a six storey spatial frame (Fig. 4) with *top and seat double web angle* beam – column connections.

The intensity (realistical values) of the horizontal forces is $P_i = \alpha_i P$ (where P = 1.00 daN) given in Table 1, while the gravitational force is q = 1800 daN/m for the beams along Oy axis and q = 1000 daN/m for the beams along Ox axis.

Table 1 Loading values									
i	1	2	3	4	5	6			
α_i	400.00	705.00	1050.00	1310.00	1610.00	1914.00			

The elements are made up of rolled profiles (Romanian Standard): 2U30 for columns and I30 for beams. The connecting elements (angles and bolts) have been considered in several cases.

The analytical model *M*- θ_r is of exponential form (2c).

Case study no. 1

This is the (classical) case of rigid beam - column connections.

Case study no. 2

The connecting elements are: bolts M24 and L120x120x10 for the connecting angles ($b_c = 12.5$ cm for top and seat angles leading to $R_k = 113200.0$ kNm/rad; $M_u = 41.88$ kNm and $b_c = 20.0$ cm for web angles leading to $R_k = 95000.00$ kNm/rad; $M_u = 63.00$ kNm).

The corresponding values of connection flexibility is $M_{u}/M_{cap} = 0.76$.

Case study no. 3

In this case the beam - column connections are semirigid with the following parameters: angles of the same type as in Case study no. 2 in the 1st bay of the frame, while in the 2nd bay they are: bolts M16 for the web angles and M12 for the top and seat angles and L70x70x7 for the connecting angles ($b_c = 12.5$ cm for top and seat angles leading to $R_k = 57700.0$ kNm/rad; $M_u = 27.24$ kNm and $b_c =$ 20.0 cm for web angles leading to $R_k = 113000.00$ kNm/rad; $M_u = 33.08$ kNm). The corresponding values of connection flexibility in the 2nd bay of the frame is $M_u/M_{cap} = 0.44$. The corresponding $P - \Delta$ curves are presented in

The corresponding $P - \Delta$ curves are presented in Fig. 5 where the displacement Δ is the lateral displacement of top level node 31 (Fig. 4).

The numerical values of \varDelta are given in Table 2.

Table 2 λ - Δ values

Case study no. 1		Case study no. 2		Case study no. 3	
λ	Δ_{31}	λ	Δ_{31}	λ	Δ_{31}
0.00	0.11	0.00	0.12	0.00	0.10
0.10	0.91	0.10	1.06	0.10	1.03
0.20	1.71	0.20	2.08	0.20	2.10
0.30	2.51	0.30	3.13	0.30	3.17
0.40	3.31	0.40	4.19	0.40	4.26
0.50	4.11	0.50	5.29	0.50	5.39
0.60	4.91	0.60	6.58	0.60	6.76
0.70	5.72	0.70	8.26	0.71	8.82
0.80	6.52	0.77	9.70	0.83	11.62
0.90	7.33	0.87	11.99	0.90	13.80
1.00	8.14	0.90	12.77	1.00	17.21
		1.00	15.55		







4. Conclusions

The $P - \Delta$ curves exhibit both, the expected linear behaviour of the rigid frame and a clear nonlinear behaviour in the case fo semirigid connectivity. The higher is the loading level the deeper is the non-linearity. The lateral displacement of node 31 is 8.14 cm in the case of rigid nodes, 12.77 cm in the case of semirigid nodes of the same parameters all over the frame and 17.21 cm in the case of more flexible joints in the 2nd bay of the frame. It is worth to note, on one hand, the large difference of the values of Δ_{31} in the last two cases while, on the other hand, the overall behaviour of the frame (still exhibits the elastic range, stability and no plateau of the $P - \Delta$ curve. The above remarks lead to yet another important conclusion: that through imposed semirigid nodes the overall behaviour of the structure may be controlled and directed toward a desired ductility without diminishing the bearing capacity of the structure.

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Some Aspects concerning the Design of Concrete Structures for Durability

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Abstract: The modern concept of concrete structures durability has been developed in Europe over the past decade mainly within **CEB** (Comite Euro-International du Beton) and then within **FIB** (Federation Internationale du Beton) and has the aim to ensure a rational and coherent service life design of concrete structures.

In the paper are presented some aspects concerning Design of Concrete Structures for Durability, including deterioration mechanisms, governing parameters of these mechanisms, time development, transport phenomena, environmental aggressivity, design, execution, maintenance, prediction of service life, additional protective measures.

Keywords: Concrete Structures, Durability, Service Life.

1. Introduction

The modern concept of concrete structures durability has been developed in Europe over the past decade mainly within CEB (Comite Euro-International du Beton) and then within *fib* (Federation Internationale du Beton) [1, 2, 3, 7] and is based on consistent engineering models describing the deterioration mechanisms incorporating knowledge form a wide range of technical disciplines, such as statics, materials technology, design, construction, statistics. economy.

Experience from inspection, maintenance and repair of existing structures have been used to identify and calibrate the critical parameters governing these engineering models and to ensure a rational and coherent service life design [2,3].

2. Deterioration mechanisms and governing parameters

Significant deterioration mechanisms are as follows [2]:

Reinforcement corrosion

Corrosion destrois primarily the reinforcement and subsequently the concrete (cracks and spalls)

- Alkali-aggregate reactions
- Chemical attacks
- Freeze-thaw bursting

All the major deterioration mechanisms require sufficient amounts of water.

Chloride based salts are some of the most harmful materials to which concrete can be exposed.

The deterioration mechanisms progress through two phases (Fig.1):





Initiation phase, during which no noticeable weakening of the material or the function of the structure occurs, but some protective barrier is broken down or overcome by the agressive media.

Propagation phase, during which active deterioration develops and loss of function is observed. A number of deterioration mechanisms develop at an increasing rate with time. Reinforcement corrosion is one such important example of propagating deterioration.

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In both the initiation phase and the propagation phase, all important deterioration mechanisms depend on some substance penetrating from the outside into the bulk of the concrete through its surface (Fig.2).



Fig.2 Transport mechanisms for aggressive substances governing the processes during both initiation and propagation phases[2].

The main transport mechanisms are nonlinear with time (more accelerated at the beginning). Therefore a smaller concrete cover than anticipated in the design may lead to severe shortening of service life.

Cyclic wetting and drying will greatly accelerate the rate at which a dissolved aggressive substance enters the concrete and concentrates near the surface of evaporation.

Once the nature of the deterioration of concrete structures is understood, defining the aggressive of the environment in which the structure is placed becomes an essential part of the service life design process. More research is necessary in the next future for classifying environmental aggressivity.

Due to the complex of environmental effects on structures and the corresponding response, it is believed that true improved performance cannot be achieved by improving the materials characteristics alone, but must also involve the elements of architectural and structural design, processes of execution and inspection and maintenance procedures, including preventive maintenance.

3. Design, execution and maintenance

The concrete durability problems experienced in the past shown that they can be avoided in the future if adequate and co-ordinated efforts are imposed to all phases of the building process of defining, planning, construction, maintenance and using the building until the end of its expected lifetime.

A traditional building process is characterised by a specilised input from all parties involved :

- the owner by defining his demands and wishes

- the designers (engineer and arhitect) by preparing design, specifications (including control schemes) and conditions

- the contractor, who will try to follow these intentions in his construction works

One of the important partie have to be add: the user of the structure (the building), who will normaly be responsible for the maintenance of the structure during the period of use.

The influence of the above-mentioned parties on the quality of the final product is essential and all parties are normally responsible for a good quality and lasting structure. The designer will realize that for durability requirements he will have to extend his knowledge or look for specialist assistance and he will recognize the need for adequate education of specialised concrete materials engineers and for an improvement in the education structure, where there seems to be some disharmony between the highly developed computation methods and an adequate knowledge of structural detailing.

All important concrete deterioration process depend on some substance penetrating from the outside into the bulk of the concrete from the surface.

Consequently, much effort must be used to ensure an appropriate quality of the concrete in the exposed outer layer of the structures. A wellcompacted strong concrete skin is needed with low permeability, low diffusivity and without map cracking. Also an adequate thickness of the concrete cover to the reinforcement must be provided.

Having in mind that "Well-constructed structures will be durable", a structure which is easy to construct will be more likely to be constructed properly and hence be durable.

On the other hand complexity in structural form, as well as in construction and use, will increase the sensitivity of the structure to deterioration, shorten service life or require increased efforts in maintenance. Difficult details should be avoided. Reinforcement should be fixed firmly in the form to avoid displacement, which may hamper proper placing and compaction of the concerte or may reduced the thickness of the cover.

Formwork must be stiff and well sealed. Leakage or displacements of the formwork may lead to porous or cracked concrete and to an unsightly surface.

Construction joints should be selected after careful consideration of the effects of reinforcement laps, bending and rebending of bars, anchoring of prestressed tendons and so on.

Durable concrete depends also on good curing. Adequate compaction and good curing are two factors having by far the greatest influence on the durability of concrete structures and this is of particular importance for the concrete in the surface layer. Curing of the concrete is part of the hardening process which ensures an optimal development of the fresh, newly cast concrete into a strong, impermeable, crack-free and durable hardened concrete.

During this initial stage of the life of the concrete it is necessary:

- to use an appropriate hardening process;

- to ensure against damage from drying;

- to ensure against damage through early freezing;

-to ensure against damage from thermal stresses.

4. Prediction of service life

The parameters necessary to quantify the prediction of service life can be listed knowing the mechanisms of deterioration, their governing parameters and the kinetics of the deterioration mechanisms. In both the initiation phase and the propagation phase, all significant deterioration mechanisms depend on some substance penetrating from the outside into the bulk of the concrete through the surface by one or more transport mechanisms.

A reliable service life estimate requires determination of deterioration and transport based on laboratory studies and a subsequent in situ verification of the values used in the initial service life design. The primary task for long service life design is to ensure a sufficiently long initiation period. In practice this is achieved by providing barriers against the penetration and accumulation of the aggressive substance considered. Therefore, the objective is to provide a good protective outer layer of the structures which can be carried out by the following means:

- selecting concrete quality; for example a concrete mix providing low penetrability and high chemical resistance, either by ensuring a low water-cement ratio or the addition of pozzolanic additives or both;

- selecting large concrete cover to the reinforcement, finding an optimal balance between the advatage of a larger cover and the increased risks of cracking;

- ensuring execution procedures which enhance quality, specifically in the outer layers, such as good compaction of the concrete.

This requires structural dimensions and detailing of reinforcement, leaving adequate space for placing concrete and introducing vibrators and good curing of the hardening concrete, requiring moisture control and limitation of temperature differences due to heat of hidration.

Satisfactory service life requires inspection, maintenance and repair. Regular and systematic inspections should be performed in order to identify and quantify possible ongoing deteriorations and to consider the economic consequences of taking either short-term or long-term remedial measures.

Inspection constitutes an integral part of structural safety and serviceability by providing a link between the environmental conditions to which the structure is subjected and the manner in which it performs with time. The nature and frequency of the inspection procedures should be determined with this in mind.

To minimize future maintenance and repair costs the inspections should, to the extent possible, reveal approaching deterioration or lack of adequate performance in due time for preventive maintenance to be applied.

Decisions regarding safety precautions, repairs, strengthening, upgrading, demolition and prevention of recurrence must be made based on the investigation and assessment of concrete structure (specialised technical expertise).

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5. Additional protective measures

Concrete is a durable building material, also under aggressive hot and humid environments, provided it is used **correctly.**

The few, though important, exceptions where additional protection may be needed relate to extreme exposure conditions should be:

- marine splash zones and other similar areas where cyclic wetting and drying with similarly contaminated water;

- foundations and other buried structures in moist soils heavily contaminated by sulphates and chlorides;

- special industrial plants with liquid or gaseous aggressive chemicals, such as refineries, petrochemical plants and desalinisation plants.

The choice of protective measures must be carefully considered in relation to the particular aggressive environment encountered.

Additional protection is considered to be measures such as :

-special additives enhancing the impermeability of concrete

-special admixtures neutralising or inhibiting deterioration mechanisms

-coating to either steel or concrete

-bituminous or polymeric membranes

-phisical lining

-electrochemical protection

-non-corrodible reinforcement such as stainless steel, polymer fibre bars

6. Concluding Remarks

Due to nature of structural concrete, by resembling natural stone, this material was longconsidered an eternally lasting maintenance free building material. During the past say twenty years this opinion has been seriously revised, particularly for reinforced and prestressed structures exposed to aggressive chloride containing environments, due to adverse experience with some such structures, or due to numerous cases of serious premature deterioration reveals for which the cause of damage is not the normally anticipated in-situ variations in material properties, concrete cover, etc, but is due to gross deviations from anticipated values, such as: lower concrete cover, large honeycombing, bad compaction, lower cement content, greater w/c.

Service life of a concrete structure is dependent on design and construction, as presented in the paper, as well as of the owner or the management who takes the decisions for operation and use. Quality requires knowledges and cooperation between all parties involved.

The most important element in improving the quality of structures and their performance is efficient continuing education, where new theories, new technologies and experience gained can be spread to a sufficiently large number of people involved in the design, construction and maintenance of building structures.

fibre reinforcement-polypropilene,glass,steel

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Stability in Semirigidity

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Abstract The problem of stability of semirigid structures is presented in the alternative of large (lateral) displacements. The stability of semirigid steel frames is assessed by the $P - \Delta$ curve. The progressive degradation of stiffness matrix – exhibiting the loose of stability through large displacements – by increasing the loading level is obtained by involving the relative rotation θ_r of the semirigid joint into the geometrically nonlinear stiffness matrix.

Keywords: semirigidity, stability, large displacements, relative rotations geometric nelinearity, stiffness matrix, $P - \Delta$ curve.

1. Introduction

Soon after the semirigidity has been signaled and accepted, its influence on structural behaviour had to be incorporated into the structural analysis and dealt with. A specific modality to take into account the semirigidity of steel frames is to asimilate the neliniarity introduced by the semirigidity to geometrical neliniarity [1]. The abundance of techniques in geometrically nonlinear analysis - mainly in FEM matriceal formulation generated several procedures - via general displacement method - in the analysis of semirigid structures. In the same time, the step-bystep technique required by the geometrical nonlinearity, allowed for taking into account the nonlinear constitutive relation $M - \theta_r$ (bending moment - relative rotation of the semirigid connecting section - usually of beam - column type).

A relative general procedure of semirigid structures analysis – using FEM in general displacement method – allowing, also, for stability analysis given in [2] presents several possibilities to deal with the nonlinearity. A synthesis of stability analysis of semirigid structures may be found in [3] and [4] together numerical studies carried aut on planar and spatial steel frames.

Nevertheless, the semirigidity involves specific problems as well [5]. The main specific problem is the analytical model of the constitutive relation $M - \theta_r$ that has to fit the experimentally obtained curve. The most appropriate relationship $M - \theta_r$ has to be associated to each type of connection usually found in steel structure design [7]. In this matter one has to take into account that the behaviour of the semirigid joint depends, also, on hte loading level and type (monotonical, cyclical).

The nonlinearity of the constitutive relation $M - \theta_r$ leads, in its turn, to a deeply nonlinear load – displacement relation at the structural level.

2. Instability of Semirigid Structures by large Displacements

The problem of instability may be tackled in several ways from buckling of a structural member to analysing the quality of the structural stiffness matrix. Since the semirigidity may be regarded as an imperfection af the connectivity, and viewing the nonlinear characteristic of $M - \theta_r$ relationship, it is very appropriate to consider that the instability of semirigid frame type structures is achieved through large (lateral) displacements, i. e., through a progressive degradation of the (lateral) stiffness matrix of the structure. Once the stability problem is equated to the large displacement analysis, the instrumental aspect is, also, at hand

and consists in assessing the load - displacement $(P - \Delta)$ curve [6]. The form of the $P - \Delta$ curve generated in a geometrically nonlinear analysis exhibits information regarding both, the measure of kinematics

(displacement in the first place) and the quality of equilibrium (stable – uninstable). The pattern of $P - \Delta$ curve and the loading level corresponding to its maximum zone are associated to the (lateral) stiffness matrix of the structure and to its dependence on the structure geometry. The progressive increase in lateral displacements (refering to a frame type structure) is related to the quality of stiffness matrix (of being or not positively defined), quality that depends, in its turn, on the value of the determinant of the matrix. The maximum zone of $P - \Delta$ curve is associated to zero value of this determinant and corresponds to instable equilibrium of the structure.

The nonlinear behaviour of semirigid structures, their, relatively, large displacements mainly in the form of lateral displacements induced by joint rotations require and impose a corresponding geometrically nonlinear analysis. The geometrically nonlinear feature of the behaviour of semirigid structures is, also, strengthen by the tendency of equating the semirigidity to geometrical imperfections and to the classical technique of dealing with such imperfections.

Whatever it is, the relative rotation θ_r may be regarded as a joint (node) displacement and enrolled into the geometrically nonlinear analysis as any other kinematic component of the displacement vector, at both, elemental and structural levels.

Therefore, a stiffness matrix deeply dependent on the relative rotations θ_r can be obtained and deal with. Incorporation of θ_r in the stiffness matrix is simply and can be done in several procedures [2], [3], [4].

The stability analysis based on adapting the stiffness matrix to a reached kinematic (and static) level enjoys a large spectrum of techniques and

algorithms and so does the stability analysis of semirigid structures.

3. Numerical Studies

The authors have performed a large number of numerical studies in stability of semirigid structures using an algorithm based on FEM technique in general displacement formulation. The associated product is ARMIUS developed on a contractual basis research.

In what follows, the case of a six storey two bay steel (planar) frame (Fig. 1) is presented. The loading of the frame is realistically selected and consists of a constant distributed load and a lateral load controlled by the variable parameter λ (Fig. 1).

The frame is equiped with semirigid connections of the type *top and seat double web angles* (Fig. 2).

The associated bending moment – relative rotation $M - \theta_r$ relation is of exponential form (1).

$$\theta_r = n \left(e^{\frac{M}{nM_u}} - 1 \right) \frac{M_u}{R_{ki}}, \quad n = 0.36$$
 (1)

The frame elements are made of rolled steel profiles (Romanian code): 2U30 for columns, I30 for beams and several cases of *top and seat double web angles* connecting substructure allowing for a large spectrum of semirigidity from $M_u/M_{cap} = 0.46$ to 0.96. Here, M_u is the ultimate bending moment of the connecting section, while M_{cap} is the ultimate bending moment of the beam. The case of the rigid connectivity is also presented.

The corresponding $P - \Delta$ curve is presented for top level node 15 (Fig. 3) in the form of $\lambda - \Delta_{15}$ form.

The explicit values of λ and Δ_{15} are presented in Table 1

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Fig. 1 – Six Storey Frame



Fig. 2 - Top and seat double web angle connection



Fig. $3 - P - \Delta$ curves

$1 \text{ able } 1 \lambda - \Delta 1 \text{ values}$															
M _u =60.	.32kNm	M _u =76.	.31kNm	M _u =81.	25kNm	M _u =104	.88kNm	M _u =115	.51kNm	M _u =132	2.28kNm	M _u =	M _{cap}	Ri Conne	gid ections
λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}	λ	Δ_{15}
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0,00	0,13	0,00	0,13	0,00	0,13	0,00	0,13	0,00	0,12	0,00	0,13	0,00	0,12	0,00	0,12
0,10	0,45	0,10	0,42	0,10	0,36	0,10	0,33	0,10	0,32	0,10	0,33	0,10	0,32	0,10	0,29
0,20	0,75	0,20	0,71	0,20	0,59	0,20	0,54	0,20	0,52	0,20	0,54	0,20	0,51	0,20	0,45
0,30	1,12	0,30	1,02	0,30	0,84	0,30	0,75	0,30	0,72	0,30	0,74	0,30	0,71	0,30	0,62
0,40	1,45	0,40	1,33	0,40	1,08	0,40	0,96	0,40	0,93	0,40	0,95	0,40	0,90	0,40	0,78
0,50	1,80	0,50	1,64	0,50	1,33	0,50	1,17	0,50	1,13	0,50	1,16	0,50	1,10	0,50	0,95
0,60	2,15	0,60	1,95	0,60	1,57	0,60	1,39	0,60	1,33	0,60	1,37	0,60	1,29	0,60	1,11
0,70	2,49	0,70	2,26	0,70	1,82	0,70	1,60	0,70	1,53	0,70	1,58	0,70	1,49	0,70	1,28
0,80	2,84	0,80	2,57	0,80	2,06	0,80	1,81	0,80	1,73	0,80	1,79	0,80	1,68	0,80	1,44
0,90	3,19	0,90	2,88	0,90	2,31	0,90	2,02	0,90	1,94	0,90	1,99	0,90	1,88	0,90	1,61
0,95	3,48	1,00	3,19	1,00	2,56	1,00	2,24	1,00	2,14	1,00	2,20	1,00	2,07	1,00	1,77
1,05	3,85	1,10	3,51	1,10	2,81	1,10	2,45	1,10	2,34	1,10	2,41	1,10	2,27	1,10	1,94
1,13	4,26	1,20	3,83	1,20	3,06	1,20	2,67	1,20	2,55	1,20	2,62	1,20	2,47	1,20	2,10
1.18	4.52	1.30	4.15	1.30	3.32	1.30	2.88	1.30	2.75	1.30	2.84	1.30	2.66	1.30	2.27

Table 1 $\lambda - \Delta$ values

1,28	5,00	1,40	4,47	1,40	3,57	1,40	3,10	1,40	2,96	1,40	3,05	1,40	2,86	1,40	2,43
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1.38	5.55	1.50	4.79	1.50	3.83	1.50	3.32	1.50	3.16	1.50	3.26	1.50	3.06	1.50	2.60
1,41	5,78	1,60	5,11	1,60	4,09	1,60	3,54	1,60	3,37	1,60	3,47	1,60	3,26	1,60	2,76
1,51	6,33	1,70	5,44	1,70	4,35	1,70	3,76	1,70	3,58	1,70	3,69	1,70	3,46	1,70	2,93
1,61	6,93	1,80	5,77	1,78	4,59	1,80	3,98	1,80	3,79	1,80	3,90	1,80	3,66	1,80	3,09
1,71	7,51	1,89	6,13	1,86	4,86	1,90	4,20	1,90	4,00	1,90	4,12	1,90	3,86	1,90	3,26
1,81	8,11	1,94	6,31	1,96	5,19	2,00	4,43	2,00	4,21	2,00	4,33	2,00	4,06	2,00	3,42
1,91	8,73	2,04	6,68	2,06	5,57	2,10	4,65	2,10	4,42	2,10	4,55	2,10	4,26	2,10	3,59
1,98	9,16	2,14	7,14	2,16	5,94	2,20	4,88	2,20	4,63	2,20	4,77	2,20	4,46	2,20	3,75
2,08	9,87	2,26	7,74	2,20	6,12	2,30	5,11	2,30	4,85	2,30	4,99	2,30	4,66	2,30	3,92
2,18	10,59	2,36	8,21	2,28	6,46	2,40	5,34	2,40	5,06	2,40	5,21	2,40	4,87	2,40	4,08
2,28	11,32	2,46	8,74	2,38	6,90	2,50	5,57	2,50	5,28	2,50	5,43	2,50	5,07	2,50	4,25
2,38	12,08	2,52	9,08	2,48	7,39	2,60	5,81	2,60	5,50	2,60	5,65	2,60	5,28	2,60	4,42
2,48	12,87	2,62	9,59	2,58	/,8/	2,70	6,05	2,70	5,72	2,70	5,88	2,70	5,48	2,70	4,58
2,38	13,69	2,12	10,17	2,68	8,5/ 8,06	2,80	6.50	2,80	5,94	2,80	0,10 6.22	2,80	5,09	2,80	4,/3
2,00	14,30	2,03	10,83	2,19	0,90	2,91	6 70	2,90	630	2,90	6 56	2,90	5,90	2,90	5.08
2,78	16.48	3.03	12.16	2,89	10.08	3.08	7.11	3,00	6.62	3,00	6.80	3,00	6.33	3,00	5.24
2,00	16.96	3 13	12,10	3.09	10,00	3.18	7 47	3 20	6.85	3 20	7.03	3 20	6 54	3 20	5 41
2.98	18,14	3.23	13.59	3.19	11.26	3.28	7.83	3.30	7.08	3.30	7,27	3.30	6.76	3.30	5.58
2,99	18,67	3,33	14,35	3,25	11,64	3,38	8,19	3,36	7,25	3,40	7,51	3,40	6,97	3,40	5,74
3,00	19,39	3,37	14,72	3,35	12,29	3,48	8,56	3,46	7,50	3,50	7,75	3,50	7,19	3,50	5,91
3,00	19,49	3,47	15,54	3,45	12,96	3,55	8,88	3,52	7,70	3,60	8,00	3,60	7,41	3,60	6,07
3,00	19,58	3,57	16,47	3,55	13,66	3,65	9,29	3,62	8,02	3,70	8,24	3,70	7,64	3,70	6,24
		3,59	16,75	3,65	14,38	3,74	9,71	3,72	8,37	3,80	8,49	3,80	7,86	3,80	6,41
		3,69	17,90	3,73	15,03	3,84	10,18	3,82	8,73	3,90	8,75	3,90	8,09		
		3,75	18,71	3,83	15,90	3,94	10,68	3,92	9,09	4,00	9,00	4,00	8,31		
		3.80	19.89	3.93	16.82	4.04	11.18	4.02	9.45	4.10	9.26	4.10	8.55		
		3 80	20.11	4 03	17.81	4 14	11.69	4.12	9.84	4 20	9.52	4 20	8 78		
		5,00	20,11	4 04	17.88	4 24	12 21	4 22	10.27	4 30	9 79	4 30	9.01		
				4.08	18 73	4 34	12,21	4 32	10.73	4 38	10.03	4 40	9.25		
				4,00	10.75	4.44	13.20	4,52	11 21	4,50	10,05	4,40	9.45		
				4,09	19,20	4,44	13,29	4,42	11,21	4,40	10,50	4,47	9,43		
				4,10	19,99	4,47	13,49	4,32	11,/1	4,34	10,37	4,57	9,70		
				4,10	20,09	4,57	14,11	4,62	12,22	4,64	10,92	4,6/	9,99		
				4,10	20,19	4,6/	14,76	4,72	12,73	4,/4	11,32	4,/5	10,28		
				4,11	20,32	4,77	15,44	4,82	13,26	4,84	11,72	4,85	10,61		
				4,11	20,46	4,87	16,16	4,92	13,79	4,94	12,12	4,95	10,97		
				4,11	20,64	4,97	16,93	5,02	14,34	5,04	12,53	5,05	11,34		
				4,11	20,90	5,07	17,75	5,12	14,91	5,14	12,95	5,15	11,72		
						5,17	18,62	5,15	15,15	5,26	13,48	5,25	12,09		
						5,24	19,24	5,25	15,80	5,36	13,96	5,35	12,48		
								5,35	16,49	5,46	14,49	5,44	12,85		
								5,45	17,21	5,56	15,03	5,54	13,30		
										5,67	15,70	5,64	13,79		
										5,77	16,28	5,74	14,28		
												5,78	14,53		
												5,88	15.05		
												5,98	15,61		

4. Concluding Remarks

A general remark is that semirigidity can be directly incorporated into geometrically nonlinear analysis through involving the relative rotation θ_r into the kinematic (displacement – deformation) fundamental relations of the general displacement method.

Several specific aspects may, also, be mentioned in connection to the degree of flexibility of the joint. It can be seen from the $\lambda - \Delta_{15}$ curve that even in the case of high M_{u}/M_{cap} ratio (corresponding to a semirigid connection close to a rigid one) ther is a significant difference between the two cases. Also, small variations in M_{u}/M_{cap} ratio lead to important variations in the values of lateral displacements.

A relative similar behaviour can be noticed in the case $M_{u'}M_{cap} = 0.84 \div 1.0$ and, also, for the interval $M_{u'}M_{cap} = 0.44 \div 0.76$.

Finally, the numerical results prove the efficienty of both, the formulation of the stability analysis of semirigid structures and of the algorithm.

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Strengthening of the Steel Plate Girders by Prestressing with Rigid Tie Rods

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Abstract: Two methods of strengthening of the steel plate girders are presented in this paper with the aim of carrying capacity increase: one based on tension flange cross section increase and the other using rigid prestressed or unprestressed tie rods added under the bottom flange.

Keywords: steel plate girders, strengthening, prestressing, carrying capacity increase, rigid tie rods

1. Introduction

Static and dynamic physical wear, accidental wear and traffic conditions changes can determine the necessity of some strengthening works which have to be able to ensure the functionality and a safe future use of the bridge structure.

The carrying capacity increase of the steel plate girders, concomitantly with their rigidity increase can be efficiently materialized through the cross flanges section increase or through the rigid prestressed or unprestressed tie rods layed under the tension flange.

The stresses patterns of the strengthened girder are presented in this paper and a numerical example is given here.

2. Stress patterns and deflection size of the strengthened steel plate girders

2.1. Strengthening by flange cross section increasing

Adding strengthening elements on one or on both girder flanges, the increase of the moment of inertia is obtained and implicitly the stresses and deflections under the live loads will diminish.

The stress patterns of the strengthened steel plate girder through a T shape welded element added on the bottom flange are presented in Fig.1.



Fig.1. Stress patterns of the strengthened girder

The state of stresses is the extreme cross section fibers of the steel girder and in the added element to the bottom flange will be:

$$\sigma_s = \frac{M_g}{I} y_s + \frac{\psi M_p}{I_c} y_s'$$
(1a)

$$\sigma_i = \frac{M_g}{I} y_i + \frac{\psi M_p}{I_c} y_i^{\dagger}$$
(1b)

$$\sigma_c = \frac{\psi M_P}{I_c} y_c \tag{1c}$$

where:

 I_c - moment of inertia of the strengthened section; ψ - dynamic coefficient of the live loads.

The favorable effects are also obtained with regard to the elastic girder deflection.

For a girder with a variable cross section the deflection can be evaluated by the relation:

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$$f = \frac{5.5}{48} \frac{M_{\text{max}} L^2}{EI_m}$$
(2)

where I_m is the average moment of inertia:

$$I_m = \frac{\sum I_i l_i}{L}$$

The following values of the girder deflection result:

- unstrengthened girder:

$$f = \frac{5.5}{48} \frac{(M_g + M_p)L^2}{EI_m}$$
(3a)

strengthened girder:

$$f = \frac{5.5}{48E} \left(\frac{M_g}{I_m} + \frac{M_P}{I_m^c} \right) L^2$$
 (3b)

where I_m^c is the average moment of inertia of the strengthened cross section.

2.2.Girder strengthening using rigid tie rods

Strengthening with tie rods consist in adding of a rigid the rod under tension flange made

up by laminated elements: L, U, O or welded sections.

The tie rods can be horizontal or polygonal layed under the bottom flange, Fig.2.





Fig.2.Girders strengthened with rigid tie rods

The tie rods can be prestressed or unprestressed, the prestress of the ties increase their efficiency but complicates the strengthening achievement.

Strengthening design with straight tie rods

The stress pattern can be followed in Fig.3 with regard to the strengthening steps.



Fig.3. State of stresses in the strengthened girder using a rigid tie rod

The state of stresses in girder and in the tie rod taking into account the dynamic effect of the traffic loads will be: - top flange:

- bottom flange:

$$\sigma_{i} = -\frac{X + \psi X_{1}}{A} + \frac{M_{g} + \left[\psi M_{p} - (X + \psi X_{1})e_{r}\right]}{I}y_{i}$$
(11)

$$\sigma_{s} = -\frac{X + \psi X_{1}}{A} - \frac{M_{s} + \left[\psi M_{p} - (X + \psi X_{1})e_{r}\right]}{I}y_{s} \qquad - \text{ tie rod:}$$

$$(4b)$$

$$\sigma_{r} = \frac{X + \psi X_{1}}{A} \qquad (4c)$$

Selftension effort evaluation.

The effort X_1 can be determined by using the static force method to solve the condition equation of the statically indeterminate system, Fig.4.



Fig.4.Selftension effort evaluation in the tie rod

$$\delta_{11}X_{1} + \Delta_{1P} = \Delta_{X1}$$
where:

$$\delta_{11} = \int_{0}^{l_{t}} \frac{m^{2}}{EI} dx + \int_{0}^{l_{t}} \frac{n^{2}}{E_{t}A_{t}} dx = \left(\frac{e_{t}^{2}}{EI} + \frac{1}{E_{t}A_{t}}\right) l_{t}$$

$$\Delta_{P} = \int_{0}^{l_{t}} \frac{M_{P}m}{EI} dx = -\frac{e_{t}}{EI} \Omega$$

$$\Delta_{X1} = -\frac{l_{t}}{EA} X_{1}$$
It is obtained:

$$X_{1} = \frac{\frac{e_{t}}{EI} \Omega}{\frac{e_{t}}{EI} \Omega}$$
(6a)

$$X_{1} = \frac{\overline{EI}^{2}}{\left(\frac{e_{i}^{2}}{EI} + \frac{1}{EA} + \frac{1}{E_{i}A_{i}}\right)l_{i}}$$

 M_p - bending moment diagram given by traffic loads on statically determinate system;

m, n - bending moment and axial force diagrams given by $X_1=1$ on statically determinate system;

 $\Omega\,$ - bending moment diagram area given by traffic loads on the tie rod length.

If the tie rod is a rigid element than $E_t=E$ and relation (6a) becomes:

$$X_{1} = \frac{\frac{e_{t}}{I}\Omega}{\left(\frac{e_{t}^{2}}{I} + \frac{1}{A} + \frac{1}{A_{t}}\right)l_{t}}$$
(6b)

Tie rod effect on the deflection.

The girder deflection is determined with respect to the maximum bending moment taking into account the tie rod reduction effect:

$$f = \frac{5.5(M_g + M_p)}{48EI_m}L^2 - f_t$$
(7a)
where:
$$L^{t}M_m = X_{e}$$
(7a)

$$f_{t} = \int_{0}^{\infty} \frac{M_{t} m}{EI_{m}} dx = \frac{X_{t} e_{t}}{8EI_{m}} \left(L^{2} - 4c^{2} \right)$$
(7b)
$$X_{t} = X + X_{1}.$$

The deflection f_t given by the negative bending moment $M_t = X_t e_t$ (relation 7b) is obtained by using the Mohr-Maxwell method, Fig.5.



Fig.5. Deflection calculation from tie rod effect

where:

3.Numerical example

The state of stresses on the main girders of a steel railway bridge with the span L=20m, Fig.6., is analyzed.



Fig.6.

3.1. Preliminary design elements

 <u>Loads evaluation</u>
 track weight (STAS 1489-78) g₁=800 daN/m
 structure dead weight (STAS 1489-78)

For a trough plate-girder bridge with a span L < 30 m, for load train T 8,5:

g ₂ =44L	+650 = 153) daN/m
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The structure weight is affected by a correction factor $k_8=0,75$ for welded structures and so it results:

$$g = g_1 + k_8 g_2 = 1948 \text{ daN/m}$$

The total dead load is considered: g=1950 daN/m=19,50 kN/m

The maximum bending moment given by dead loads is:

$$M_{g} = \frac{gL^{2}}{8} = \frac{19,5 \times 20^{2}}{8} = 975$$

kNm

is:

<u>The maximum bending moment given by</u> <u>load train T 8,5</u> can be evaluated by the relation: $M_{maxmax} = (10,65L^2 + 106,8L - 320) = 6076$ kNm

The train loads have to be multiplied by the dynamic coefficient ψ , which for welded track

$$\psi = 1,10 + \frac{17}{35 + L} = 1$$

<u>The maximum bending moment</u> will be: $M_{tot} = M_g + \psi M_{maxmax} = 9542 \text{ kNm}$

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Table 1

and for one girder is:

$$M = \frac{M_{tot}}{2} = 4771 \text{ kNm}$$

The girder resistance characteristics are given in Table 1.

Section	Moment of inertia I [cm ⁴]	Modul of resistance W [cm ³]	Cross section area [cm ²]	Average moment of inertia I _m [cm ⁴]	Average cross section area A _m [cm ²]	
mid-span (1-1)	2 831 490	30 611	486	2 531 718	450	
end-span (2-2)	2 165 330	23 409	406	2 331 718		

Resistance capacity checking:

 $\sigma_{\max} = \frac{M}{W} = \frac{4771 \times 10^4}{30611} = 1556 \text{ daN/cm}^2 < \sigma_a$ $\sigma_a = 1600 \text{ daN/cm}^2$ Rigidity (deflection) checking:

$$M_{f} = \frac{1}{2} \left(M_{g} + M_{\max} \right) = 3225,5 \text{ kNm}$$
$$f = \frac{5,5}{48} \frac{M_{f}L^{2}}{EI_{m}} = 2,72 \text{ cm} < f_{a} = \frac{L}{500} = 4 \text{ cm}$$

3.2.Main girder strengthening by increasing the cross section of the tension flange

For a trough plate-girder bridge the strengthening solution adopted consist in a \perp shape welded member under the bottom flange, Fig.7. and the strengthened section characteristics are presented in Fig.8.



The bending stresses will be:

top flange:

$$\sigma_s = \frac{M_s}{W} + \frac{\psi M_p}{I_c} y_s^{\prime} = 1451 \text{ daN/cm}^2$$

- bottom flange:

$$\sigma_i = \frac{M_s}{W} + \frac{\psi M_p}{I_c} y_i = 1232 \text{ daN/cm}^2$$

- strengthening element:

$$\sigma_c = \frac{\psi M_p}{I_c} y_c = 1422 \text{ daN/cm}^2$$

The strengthened girder deflection is:

$$f \cong \frac{5.5}{48E} \left(\frac{M_g}{I_m} + \frac{M_P}{I_c} \right) L^2 = 2,40 \text{ cm}$$

3.3.Main girders strengthening using a simple rigid tie rod

The state of stresses in the main bridge girders is analyzed, by using as a strengthening method a tie rod made up by 2L 100x100x10 at 250

mm distance from the bottom flange, with a length of 17 m, Fig.9.



The tie rod excentricity is:
$$a = 0.25 + 25 = 2.82 = 114.68$$

$$e_t = 92,5 + 25 - 2,82 = 114,68$$
 cm

The diagrams for the tie rod selftensioning effort are presented in Fig.10, where:

$$M_{x} = \frac{M_{\text{max max}}}{0,1936} \left(0,88 \frac{x}{L} - \frac{x^{2}}{L^{2}} \right) \qquad \text{and}$$
$$M_{1} = M_{x} (x = 1,5) = 977 \text{ kNm}$$

It is obtained (Fig.10):





$$\Omega = S_1 + S_2 + S_3 = 41973 \text{ kNcm}^2$$

where:

$$S_{1} = M_{1}l_{t} = 16609$$

$$S_{2} = (M_{\max \max} - M_{1}) \times 0.12L = 5017$$

$$S_{3} = \frac{2}{3}(M_{\max \max} - M_{1})(0.88L - 2c) = 20347$$

The selftension effort in the tie rod is: $X_1 = 330,2 \text{ kN}$ (rel. 6b).

The stresses will be:

- in girder (rel. 4a, 4b):

 $\sigma_s = -1245 \text{ daN/cm}^2$ $\sigma_i = 1054 \text{ daN/cm}^2$

- in tie rod:

 $\sigma_{t} = 1212 \text{ daN/cm}^2$

The tie rod effect on the girder deflection is: $f_t = 0,348 \text{ cm}$ (rel. 7b) $f_{tat} = f - f_t = 2,43 \text{ cm}$

4. Conclusions and observations

The state of stresses and deflection of the initial girder and of the strengthened girder by using the two analyzed methods are presented in Table 2.

Tabl	e	2
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Case	σ_s (reduction [%])	σ_i (reduction [%])	f (reduction [%])	
Initial girder	1556	1556	2,78	
Strengthened girder by bottom flange	1451	1232	2,40	
increase	(93 %)	(79 %)	(86 %)	
Strengthened girder using a simple rigid tie	1245	1054	2,43	
rod	(80 %)	(68 %)	(87 %)	

From the numerical analysis performed here above the following conclusions can be mentionated:

- the strengthening of the steel girders using a rigid tie rod is more efficient in comparison with flange cross section increase at the same material consumption, because the material can be distributed more conveniently;
- the consolidation with rigid tie rod involves a reduced handwork because it is fixed only at the end-span and in some intermediate points, in comparison with a continuous welded element.

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Stress Analysis in Gravity Dams The Case of the Dam with Triangular Section

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Abstract: This work discusses the improvement of the calculation and the verification methods regarding the triangular gravity dams and to compare the results.

Keywords: Stress analysis, gravity dams.

1. Introduction

In the last decades the calculation of the gravity dams appeal to the elasticity theory and to the finite element. This methods allowed a better determination of the stress in the different points of the profiles, but especially help to introduce in the calculation procedures the co-operation effect, the interaction, respecting the whole assembly dam – foundation – accumulation water. For the determination of the stress and of the deformations in a point from the inside of the dam, now we use a few methods.

2. The elementary method (The method rely on Strength of Materials)

In this case we have an eccentric compressive stress.



Fig.1 Dam sections.

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$$T_{x} = H = \frac{\gamma}{2} y^{2}$$
(1) $M_{Z} = \frac{\gamma}{6} y^{3} - \frac{p}{12} y^{3} tg^{2} \beta$ (4)
 $M_{Z} = H \frac{y}{2} - Gy tg \beta (\frac{1}{2} - \frac{1}{2}) = \frac{H}{2} y - \frac{Gy}{6} tg \beta$ (2) $I_{Z} = \frac{y^{3}}{2} tg^{3} \beta$ (5)

$$=H\frac{y}{3}-Gy\,tg\,\beta(\frac{1}{2}-\frac{1}{3})=\frac{H}{3}y-\frac{Gy}{6}\,tg\beta \qquad (2) \qquad I_{Z}=\frac{y^{3}}{12}\,tg^{3}\beta$$

$$G = \frac{p}{2} y^2 tg\beta \tag{3}$$





Fig.2 Dam sections.

(6)

If $tg\beta = m$ then,

$$\sigma_{y=-}\frac{N}{A} \pm \frac{M_z}{I_z} x = -\frac{p}{2} y \pm \left(\frac{2\gamma}{m^3} - \frac{p}{m}\right) x$$

the variation is linear.

$$\sigma_{\text{ymin}} = -py + \frac{\gamma}{m^2} y = \left(\frac{\gamma}{m^2} - p\right) y \tag{7}$$

$$\sigma_{\rm ymax} = -\frac{\gamma}{m^2} y \tag{8}$$

$$\tau_{yx} = \frac{T_x S_z}{bI_z} = \frac{\gamma}{2} y^2 \frac{12}{I y^3 t g^3 \beta} I \left(\frac{y^2 t g^2 \beta}{4} - x^2 \right)$$
(9)

$$S_{Z} = I\left(\frac{y^{2}}{4}tg^{2}\beta - x^{2}\right)$$
(10)

$$\tau_{YX}^{max} /_{X=0} = \frac{3\gamma}{m} y \tag{11}$$

$$\tau_{yx}^{min} = \frac{3\gamma}{my} \left(y^2 - \frac{4}{m^2} \frac{y^2 m^2}{4} \right) = 0$$
 (12) We

obtained a parabolic variation with maximum value on the neutral axis of the section.

3 The calculation of unit stress by the method of elasticity theory.

This method, by which we obtain one image of the valuation and distribution of the stress in the whole body of the dam, rely of the fit of the balancing equation from the elasticity.

We see that, both, the external pressures and the stress from the body of the dam are linear functions by

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the distances to the top, respectively by the usually coordinates.

If we note with σ_x and σ_y the horizontal and vertical normal stress and with τ the tangential stress, they could be like:

$$\begin{cases} \sigma_x = a_1 x + b_1 y \\ \sigma_y = a_2 x + b_2 y \\ \tau_{xy} = a_3 x + b_3 y \end{cases}$$
(13)

For the determination of these six unknowns a_l , b_1, a_2, b_2, a_3, b_3 , we use:

balancing equations from the elasticity are: -

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} = m_x \qquad m_x = 0 \tag{14}$$

$$\frac{\partial \sigma_{y}}{\partial y} + \frac{\partial \tau_{yx}}{\partial x} = m_{y} \qquad m_{y} = \gamma_{b}$$
(15)

- border conditions from dam embankment are: on upstream wall side

$$\begin{cases} p = \sigma_x \\ 0 = \tau_{xy} \end{cases}$$
(16)

on downstream wall side

$$\begin{cases} \sigma_x \cos \beta = \tau_{xy} \sin \beta \\ \tau_{xy} \cos \beta = \sigma_y \sin \beta \end{cases}$$
(17)

$$\begin{cases} \sigma_x = \tau_{xy}m \\ \tau_{xy} = \sigma_ym \end{cases}$$
(18)

Making the derived function on the balance equations and replace the frontier conditions result one system by six equations with six unknowns.

Solving the system we have:

$$\begin{cases} a_{1} = 0 \\ b_{1} = \gamma_{a} \\ a_{2} = -\frac{\gamma_{b}}{\lambda} + \frac{2\gamma_{a}}{\lambda^{3}} \\ b_{2} = \gamma_{b} - \frac{\gamma_{a}}{\lambda^{2}} \\ a_{3} = \frac{\gamma_{a}}{\lambda^{2}} \\ b_{3} = 0 \end{cases}$$

$$\begin{cases} \sigma_{x} = \gamma_{a}y \\ \sigma_{y} = \left(-\frac{\gamma_{b}}{\lambda} + \frac{2\gamma_{a}}{\lambda^{2}}\right)x + \left(\gamma_{b} - \frac{\gamma_{a}}{\lambda^{2}}\right)y \\ \tau_{xy} = \frac{\gamma_{a}}{\lambda^{2}}x \end{cases}$$

$$(19)$$

We could calculate this with the assumption of the elasticity theory choosing one stress function under polynomial form, like:

$$F(x,y) = \frac{A}{6}x^{3} + \frac{B}{2}x^{2}y + \frac{C}{2}xy^{2} + \frac{D}{6}y^{3}$$
(21)

The function must be a double harmonic function and to be accomplished with the balancing conditions on the frontier.

In order that the function must be double harmonic:

$$\Delta^2 F = 0 \qquad \frac{\partial^4 F}{\partial x^4} + 2 \frac{\partial^4 F}{\partial x^2 \partial y^2} + \frac{\partial^4 F}{\partial y^4} = 0 \qquad (22)$$

We see that this condition is accomplished.

The four constants A, B, C, D will be calculate making the conditions on the border line on the both sides of the dam, like following relations:

$$\{p\} = [T_{\sigma}]\{\nu\}$$

$$p_{x} = \sigma_{x} \cos(\nu, x) + \tau_{xy} \cos(\nu, y)$$

$$p_{y} = \tau_{xy} \cos(\nu, x) + \sigma_{y} \cos(\nu, y)$$

$$(23)$$



Fig.3 Dam sections.

$$OA\begin{cases} cos(v, x) = -I\\ cos(v, x) = 0 \end{cases}$$
(24)

$$OB \begin{cases} \cos(\nu, x) = \cos(2\pi - \beta) = \cos\beta\\ \cos(\nu, y) = \cos\left(\frac{3\pi}{2} - \beta\right) = -\sin\beta \end{cases}$$
(25)

On the side OA (upstream of the dam embankment):

$$\begin{cases} yy = (-Cx - Dy)_{x=0} \\ 0 = (Bx + Cy + px)_{x=0} \end{cases}$$
(26)

The constants that results are: $\begin{cases} C = 0\\ D = -\gamma \end{cases}$ (27)

On the side OB (downstream of the dam embankment)

$$\begin{cases} \sigma_X \cos \beta - \tau_{yx} \sin \beta = 0\\ \tau_{xy} \cos \beta - \sigma_y \sin \beta = 0 \end{cases}$$
(28)

$$\begin{cases} (Cx + Dy)\cos\beta + (Bx + Cy + px)\sin\beta = 0\\ -(Bx + Cy + px)\cos\beta - (Ax + By)\sin\beta = 0 \end{cases}$$
(29)

Introducing the valuation of constant *C*, dividing both equations with $\cos \beta$ and remember the note tg $\beta = m$, we obtain:

$$\begin{cases} -\gamma + (B+p)m^2 = 0\\ (B+p)m + (Am+B)m = 0 \end{cases}$$
(30)

The constants who results are:
$$\begin{cases} B = \frac{\gamma}{m^2} - p \\ A = \frac{p}{m} - \frac{2\gamma}{m^3} \end{cases}$$
(31)

Now, we know the all four constants. We will introduce the valuations of the constants in the expressions of the stress and in the final result:

$$\begin{cases} \sigma_{y} = \left(\frac{p}{m} - \frac{2\gamma}{m^{3}}\right)x + \left(\frac{\gamma}{m^{2}} - p\right)y \\ \sigma_{x} = -\gamma y \\ \tau_{yx} = -\left(\frac{\gamma}{m^{2}} - p\right)x - px = -\frac{\beta\gamma}{m^{2}}x \end{cases}$$
(32)

Because the deformations around the *z*-axis are brake (state of the deformation), in the dam appear also normal stress σ_{z} like:

$$\sigma_z = \mu \left(\sigma_x + \sigma_y \right) = \mu \gamma \left[\left(\frac{1}{m} - \frac{p}{\gamma} \right) y - \left(\frac{2}{m^2} - \frac{p}{\gamma} \right) \frac{1}{m} - \gamma y \right]$$

The normal stress σ_z are constant along the dam, and in the median plane they have a linear variation after the x and y directions.

4. The finite element method

The basic idea of this method starts from the possibility to describe the real field of the movements with some values of this in a finite number of points.

If we divide the plane surface of the section in a number of plane figures (triangles, rectangles, quadrilaterals), these elementary surfaces, with dimensions and shapes that was arbitrarily choose, are the finite elements. The crossing points of the network maked in this way are the knots. In the simple form of this method the movements u and v from this knots are calculus parameters, or degrees of freedom. Then, the wording is in movements.

After this, the finite element will be studied separately, manifested in an approximate mode, with the help of some function named interpolation function chosen by the number of freedom degrees, deformation and stress state inside the element.

It is thought that this is a discrete structure in finite element and we note with $\{\delta\}$ the vector of knots movements.

The distribution of the movements in structures is made by:

$$u(x, y, z) = N_1 \delta_1 + N_2 \delta_2 + \dots + N_n \delta_n = [N_i(x, y, z)] \{\delta\}$$

Where $N_I(x, y, z)$ are functions of approximation (of form) defined local for each type of element.

The balancing condition of the structure under the action of external forces will be manifested with the minimisation of the total potential energy (of deformation plus of external forces)

$$E_{p} = \frac{1}{2} \int_{v} \varepsilon^{T} \sigma dV - \left\{\delta\right\}^{T} \left\{R\right\} = min$$
(33)

Where:

 σ is the effort in the structure ε is the specific deformation

R is the external forces thought that it is applied in the knots

 δ is the movements of the knots under the action of the R forces

The product $\{\delta\}^T \{R\} = \delta_1 R_1 + \delta_2 R_2 + \dots$ actually is the mechanical work made by the external forces, storaged like potential energy.

But, the specific deformations are directly tied of the movements.

Like example, in the plane theory of elasticity, ε

$$\varepsilon_{x} = \frac{\partial u_{x}}{\partial x}$$

$$\varepsilon_{y} = \frac{\partial u_{x}}{\partial y}$$
(34)

Where u_x and u_y are the movement projections on the *X* and *Y* direction.

on the directions x and y are:

To generalise, the specific deformations will be obtain with a derived operator applied to the movements: $\varepsilon = L(u(x, y))$

Knowing the relation, the specific deformations could be manifest directly in accordance with the nodal movements $\{\delta\}$:

$$\varepsilon = L(u(x, y)) = L(\Sigma N_i \delta_i) = \Sigma B_i \delta_i = [B] \{\delta\}$$
(35)

Where $[B_i] = L[N_i]$ is a matrix obtained by deriving the functions of approximations N.

If the behaviour of the materials from the structure is elastic, the unit stress σ is tied by the specific deformations ϵ with the matrix of elastic constants:

$$\{\sigma\} = [D]\{\varepsilon\} = [D]\{B\}\{\delta\}$$
(36)

Making the substitutions for ε and σ on the expression of the potential energy.

$$E_{P} = \frac{1}{2} \{\delta\}^{T} \left[\int_{V} [B]^{T} [D] [B] dV \right] \langle \delta \rangle - \{\delta\}^{T} \{R\} = min \quad (37)$$

$$E_{P} = \frac{1}{2} \{\delta\}^{T} [K] [\delta] - \{\delta\}^{T} \{R\} = min$$
(38)

where [K] represents the integral from the bracket named stiffness matrix.

The minimum condition of the potential energy will be manifested making the cancellation successively the partial derivative of this regarding the nodal movements.

$$\min(E_p) \to \frac{\partial E_p}{\partial \delta_i} \bigg|_{i=1,2\dots,n} = 0$$
(39)

Making the derive, where we known [K], who results by making the integral (analytical or numerical) of some known functions, we will have the general system of equations

$$[K]\{\delta\} = \{R\} \tag{40}$$

From the solution of this result the movements of the structure $\{\delta\}$ and after successively from relation (36) the stress from the structure.

We must remark that in all equations (35), (36), (37) we make the calculations independently element by element and the rigidity matrix is a sum

$$\begin{bmatrix} K \end{bmatrix} = \Sigma_e \begin{bmatrix} K \end{bmatrix}^e \tag{41}$$

Where the matrix $[K]^e$ of the elements are selectively added after the contribution of the element that bring to the knot movement of the equation we write.

The major problem in the finite element method is to choose the adequate type of element to the problem who we analyse (once we define the functions of approximation, we will do the calculations after this method).

For the structure analysis there is a big number of type of elements specified and used by a diversity of authors. If the properties of the respectively element are in concordance with the physical problem, by making to become thicker the discreteness, we obtain a solution however close to the exact solution.

5. Static analysis assumptions.

Plane or space analysis.

The system dam – foundation makes, indifferent of the type of the dam, a space system. Generally at

least, result that the static analysis of this must be done like a space analysis in finite element.

We must keep in mind that the necessary time for a space analysis is bigger with 10 to 30 times than the time for plane analysis.

6.Conclusion.

The results obtained with the finite element method are more precisely than results that will obtain with the elementary method or the elasticity theory.

On the case when we keep in mind on the discreteness of the structure and the interaction between the dam and the foundation the results are more precisely. The results obtained in the case of the dam with the triangular section will be compare and analyse the important differences that will appear on these three cases.

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Study of a Soil – Vibratory Roller Response

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Abstract: The paper presents the analytical approach of the sol – vibratory roller response along the dynamic compaction process. The mathematical model was considered for the specific conditions of a compacting plant working on a laid granular material layer. In order to estimate the obtained compaction degree as it is given by a Terrameter Continuous Compaction Control system, the calculations were developed to reach the energy absorbed by the soil during the compaction process. There were considered both modeling cases of the soil – compacting plant system, with one and with two degrees of freedom, respectively. For both, the absorbed energy is determined as a function of the drum amplitude, oscillating frequency and soil stiffness and dumping coefficients. The study searched mainly for the absorbed energy relationship with the sand moisture content, but it also presents the behavior with the dry density and the operating frequency of the vibrating device.

Keywords: sand, vibratory roller, continuous compaction control, absorbed energy.

1. Introduction

The surface compaction of non-cohesive soils is very effectively accomplished by vibrating rollers, which are smooth-wheeled machines fitted with a vibrating mechanism. The method of specification and compaction control depends on the nature of the site and it is very important for obtaining the desired soil properties, especially with a reasonably uniform spread. Moreover, a high-level of quality requires a proper control all over the compacted area, which can be achieved economically by roller-integrated continuous compaction control – CCC.

One of the factors that greatly affect the soil properties and so the compaction process is the moisture (water) content which is largely outside the contractor's control. Because the water content varies inevitably, either as a result of area spreading or due to the weather changes, it is important to determine its implications on the method of compaction control.

The present paper offers an analytical determined response of the soil – vibratory roller system during the compaction process. In order to estimate the compaction value as it is given by the Terrameter Continuous Compaction Control system, the calculations are developed to reach the energy absorbed by the soil during compaction, and to determine its relationship mainly with sand moisture content. It is also discussed the energy behavior with the dry density and with the operating frequency of the vibrating device.

The analytical development is presented for both adopted systems: with one- and with twodegree-of-freedom. For both systems, the absorbed energy is obtained as a function of the drum amplitude, the soil stiffness and damping coefficients, and of the vibrating frequency. The calculations were performed for two levels of dry density. The shear modulus variation with moisture content was determined according to the Wu et.al. model, where the values of the shear modulus at dry conditions were determined from the laboratory tests.

2. The mathematical model of the vibrating roller system

A vibratory roller is generally described by a two-degrees-of-freedom system (fig.2.1) for which only the vertical component of the dynamic force is considered (Yoo, Selig, 1979).

The roller motion is described by the drum displacement z_d and the frame displacement z_f (the part of the frame that influences the drum). The main characteristics considered for the soil – compactor model are: the effective mass of the frame m_f , the mass of the drum m_d and the mass of the eccentrics m_e ; the operating frequency f and the eccentricity e for the vibrating system; the stiffness k_t and the damping c_t parameters for the drum suspension

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system; the stiffness k_{s} and the damping c_{s} parameters for the soil.



The dynamic force produced by the vibrating system is:

 $F_D = F_0 \cdot \sin \omega t = m_e \cdot e \cdot \omega^2 \cdot \sin \omega t \quad (2.1)$

where $\omega = 2 \cdot \pi \cdot f$ is the angular frequency and F₀ is the centrifugal force of the eccentrics.

The drum – soil contact force F_S is a function of the soil parameters (k and c) and soil elastic and plastic deformations. By neglecting the plastic part from the compacted material deformation, during the loading and unloading phases the contact force is:

$$F_s = kz_d + c\dot{z}_d \tag{2.2}$$

During the air phase (the contact between the drum and soil is lost) the contact force is zero.

The efficiency of the compactor is indicated by the ratio of contact (transmitted) force to generated dynamic force: $R_T = F_S/F_D$.

The two differential equations of motion are obtained (Adam, Kopf, 2000):

for the drum (2.3)

$$(m_d + m_e)\ddot{z}_d + c_t(\dot{z}_d - \dot{z}_f) + k_t(z_d - z_f) =$$

$$= m_e e \omega^2 \sin \omega t - F_s$$
and for the frame (2.4)

$$m_f \ddot{z}_f - c_t(\dot{z}_d - \dot{z}_f) - k_t(z_d - z_f) = 0$$

By a special design of the suspension system the stiffness and damping parameters k_t and c_t can be very much reduced, and thus in a simplified mathematical model their contribution can be neglected. The governing differential equation for the obtained one-degree-of-freedom system is:

$$m_r \ddot{z}_d = m_e e \omega^2 \sin \omega t - F_s \tag{2.5}$$

where m_r represents the total mass of the roller $m_d + m_e + m_f$.

Since the homogeneous part from the general solutions of the governing differential equations presented above vanishes in a very short time, for the steady state situation only the particular solutions will be considered. Taking into account the harmonic shape of the driving dynamic force, the particular solutions are assumed also of a harmonic shape.

In the case of the one-degree-of-freedom system, the solution of the equation 2.5 is considered: $z = z_p(t) = U \sin \omega t + V \cos \omega t$ (2.6)

where U and V are constants which are obtained by substituting the first and second derivatives of z in the differential equation.

By considering $z(t) = U \sin \omega t + V \cos \omega t =$

$$= \sqrt{U^{2} + V^{2}} \sin(\omega t - \varphi) \quad \text{where}$$

$$\varphi = \operatorname{arctg} \frac{-V}{U} = \operatorname{arctg} \frac{2D \frac{\omega}{\omega_{n}}}{1 - \frac{\omega^{2}}{\omega_{n}^{2}}} \quad \text{is the phase}$$

angle, the solution for the differential equation 2.5 is obtained:

$$z(t) = \frac{F_o \sin(\omega t - \varphi)}{k \sqrt{\left(1 - \frac{\omega^2}{\omega_n^2}\right)^2 + \left(2D\frac{\omega}{\omega_n}\right)^2}}$$
(2.6')

where there were considered $\omega_n = \sqrt{\frac{k}{m_r}}$ and

 $D = \frac{c}{2m_r \omega_n}$, the natural circular frequency and the

damping ratio for a one-degree-of-freedom system, and F_0 the amplitude of the driving force.

The amplitude of the drum displacement will be marked as:

$$Z = \frac{F_o}{k \sqrt{\left(1 - \frac{\omega^2}{\omega_n^2}\right)^2 + \left(2D\frac{\omega}{\omega_n}\right)^2}}$$
(2.7)

and thus, the drum displacement, velocity and acceleration will be:

$$z(t) = Z\sin(\omega t - \varphi)$$
(2.8)

$$\dot{z}(t) = Z\omega\cos(\omega t - \varphi) \tag{2.9}$$

$$\ddot{z}(t) = -Z\omega^2 \sin(\omega t - \varphi) = -z(t)\omega^2 \quad (2.10)$$

The energy absorbed by the soil, as it is also determined by the Terrameter Continuous Compaction Control system, is calculated by summating the product between the soil – drum interaction force and the drum velocity over a period

$$T = \frac{2\pi}{\omega} \text{ of one cycle:}$$
$$W = \int_0^T (c\dot{z} + kz)\dot{z}dt \qquad (2.11)$$

By substituting $z = U \sin \omega t + V \cos \omega t$ and $\dot{z} = U\omega \cos \omega t - V\omega \sin \omega t$, and performing the integration, it was obtained (Nicoara, 2001):

$$W = c \omega \pi (U^2 + V^2) = \frac{c}{k} F_o Z \omega \pi \qquad (2.11')$$

In the case of the two-degree-of-freedom system (equations 2.3 and 2.4), the solutions for the drum and the frame displacements are assumed:

$$z_d(t) = U_d \sin \omega t + V_d \cos \omega t \qquad (2.12)$$

$$z_f(t) = U_f \sin \omega t + V_f \cos \omega t \qquad (2.13)$$

where U_d , V_d , U_f and V_f are constants which are obtained by substituting the first and second derivatives of z_d and z_f in the differential equations. The expression 2.12 can be written

$$z_d(t) = \sqrt{U_d^2 + V_d^2} \sin(\omega t - \varphi_d)$$

where $\varphi_d = arctg \frac{-V_d}{U_d}$ is the phase angle for the

drum motion, and the drum displacement amplitude is marked as:

$$Z_{d} = \sqrt{U_{d}^{2} + V_{d}^{2}}$$
(2.14)

By substituting the drum displacement and velocity expressions, $z_d = U_d \sin \omega t + V_d \cos \omega t$ and $\dot{z}_d = U_d \omega \cos \omega t - V_d \omega \sin \omega t$ in the expression 2.11, and by performing the integration, it is obtained (Nicoara, 2001):

$$W = c \omega \pi (U_d^{2} + V_d^{2}) = c \omega \pi Z_d^{2} \qquad (2.11'')$$

The two natural frequencies obtained from the frequency equation of the two-degree-of-freedom system are:

$$\omega_{n1,2} = \left[\frac{m_r k_i + m_f (k_i + k_s) \pm \sqrt{(m_r k_i + m_f k_i + m_f k_s)^2 - 4m_r m_f k_i k_s}}{2m_r m_f}\right]^{2/2}$$
(2.15)

The influence of the compacted sand characteristics is exerted through the stiffness and damping coefficients as they were obtained in the semi-infinite elastic cone model (Wolf, 1994):

$$k = \frac{G}{1 - \nu} (3.1a^{0.75}b^{0.25} + 1.6b)$$
(2.16)

$$c = 4ab\sqrt{2G(1+w)\rho_d \frac{1-v}{1-2v}}$$
(2.17)

where G is the maximum shear modulus of the dynamic loaded sand, v is the Poisson's ratio of the sand, a is half of the drum length and $b = \sqrt{2rz - z^2}$ is half of the drum footprint width, with r the drum radius. It is noticed that b is a function of the drum displacement but because its changing with z is very small, for the following

calculations it will be considered as constant. The drum footprint width will be determined by successive calculations from an elastic settlement of the drum under the static weight $z_o k = m_r g$.

The maximum dynamic shear modulus G behavior with water content was determined as follows (Wu et.al., 1984):

determine the degree of saturation

$$S_r = w \frac{\rho_d \rho_s}{\rho_w (\rho_s - \rho_d)}$$
(2.18)

determine the parameter of the model

$$H = (a-1)H_1H_2$$
(2.19)

where
$$H_1 = 1$$
 and $H_2 = \sin(\frac{\pi S_r}{2S_{r(opt)}})$ (2.20)
for $S_r \leq S_{r(opt)}$

$$H_{1} = \frac{1}{2} \left(\frac{100 - S_{r}}{100 - S_{r(opt)}} \right)^{2} \text{ and}$$

$$H_{2} = \sin \left[\frac{\pi (S_{r} + 50 - \frac{3}{2} S_{r(opt)})}{100 - S_{r(opt)}} \right] + 1(2.20^{\circ})$$

for $S_r > S_{r(opt)}$ determine the shear modulus

$$G_o = (1+H)G_{o(dry)}$$
 (2.21)

In this empirical established model, the involved parameters are: w the moisture content, ρ_d , ρ_s and ρ_w the dry, solid and water densities, $S_{r(opt)}$ the saturation degree corresponding to the optimum moisture content and $G_{o(dry)}$ the shear modulus corresponding to the dry conditions.

3. Results of the practical study for a soil – vibratory roller system response

The calculations were conducted for a smooth drum vibratory roller presenting the following parameters (Yoo and Selig, 1980): the mass of the front module $m_r = m_d + m_e + m_f = 4760$ kg; the mass of the frame $m_f = 2040$ kg; the mass of the drum with the vibratory device $m_d + m_e = 2720$ kg; drum width 2a = 1.6 m; drum diameter 2r = 1.2 m; operating frequency f = 25 Hz \Rightarrow the circular frequency $\omega =$

 $2\pi f = 157.08 \text{ rad/s}$; amplitude of the driving force $F_o = 85 \text{ kN}$; stiffness coefficient of the suspension system $k_t = 4400 \text{ kN/m}$; damping coefficient of the suspension system $c_t = 4.4 \text{ kN.s/m}$. The compaction plant was considered to work on an artificial deposit of a fine poorly graded silty sand.

For the situation when the response of the onedegree-of-freedom system was studied, it was considered that the drum is absolutely attached to the frame. The stiffness and damping coefficients of the coupling system were considered zero and the drum mass was considered equal with the total mass of the front module.

The compacted sand was considered for two levels of dry density, the loose sand situation with $\rho_d = 1524 \text{ kg/m}^3$ and the denser sand situation with $\rho_d = 1644 \text{ kg/m}^3$. These two values of the dry density correspond to the levels considered in previous laboratory studies performed in order to estimate the maximum shear modulus at dry condition. Thus, for the low density situation $G_{o(dry)} = 56200 \text{ kN/m}^2$, and for the high density situation $G_{o(dry)} = 68800 \text{ kN/m}^2$. The maximum ratio $a = (G_o/G_{o(dry)})_{max}$ was estimated according to the Wu et.al. model, for a confining pressure of about 46 kN/m², at 1.38.

The soil – vibratory roller system response was determined for a moisture content range between completely dry conditions and 14 %, a value already above the optimum moisture content (12 %) determined by previous standard Proctor tests for the considered sand. Knowing that for the considered sand $D_{10} = 0.053$ mm, the optimum degree of saturation was calculated $S_{r(opt)} = -6.5lg(D_{10}) + 1.5 = 9.79$ % (Wu et.al., 1984).

The Poisson's ratio of the sand was considered for this application as v = 0.33.

The variation with moisture content of the shear modulus, and of the stiffness and damping coefficients are presented in the figures 3.1 to 3.3, for the two considered levels of dry density. It can be noticed that all the three parameters, G_o , k and c, present higher values for the situation of denser sand.

The stiffness coefficient follows the shear modulus variation, and presents the maximum values for the same low levels of moisture content, w at about 2 to 2.5 %. This level of moisture content corresponds to the optimum degree of saturation considered in the Wu et.al. model. Due to the direct proportionality between k and G_o , the ratios between



their values at high and low density levels are almost the same, about 1.2 for the performed application. corresponding to high density are larger than those corresponding to low density.



Figure 3.4 Natural frequency variation with moisture content

Regarding the natural frequency behavior, its value with respect to the operating frequency (157 rad/s for this calculation) is very important. It is noticed that for the present application, the natural frequency curves corresponding to the 1-d-o-f system passes through the value of the operating frequency at two levels of moisture content, at about 0.5 and 10 % for the high density and at about 2 and 4 % for the low density. As a direct effect, the variation of the drum displacement amplitude and of the absorbed energy are presented in the figures 3.5 and 3.6.

In the case of one-degree-of-freedom system, both Z and W present two peaks corresponding to the intersection points of ω_n with ω . The magnitudes of these peaks, as the density level determines it, depend on their position with respect to the optimum saturation degree. The maximum absorbed energy for the loose sand is about 192 and 195 N.m, and for the denser sand, about 171 and 195 N.m. In the area in between the two peaks, about at the level of the optimum saturation degree, the absorbed energy presents a minimum (about 167 N.m and 45 N.m). It is noticed that for the absorbed energy, the ratio between the maximum value and the value corresponding to the dry conditions is about 2.9 for the low density situation, and about 1.4 for the high density situation.

As for the two-degree-of-freedom system, the natural frequency of the drum is all the time above the level of the operating frequency. Thus, both Z and W present the largest values at the completely

Figure 3.3 Damping coefficient variation with moisture content

The damping coefficient c does not follow the G_o variation and, after the moisture content of about 2-2.5 %, it still increases slightly with w.

From the figure 3.4, which presents the natural frequency variation with moisture content for the four cases (the one- and two-degree of freedom systems for the low and high density situations), it is noticed that the drum natural frequency is higher for the 2-d-o-f system and, in the same time, the values

dry or very moist (almost saturated) conditions. At the moisture content corresponding to the optimum saturation degree Z and W present a minimum. The maximum decrease of the absorbed energy with respect to the value corresponding to the dry conditions is about 3.8 for the low density, and about 2.9 for the high density situation.



Figure 3.5 Drum displacement amplitude response in relation with moisture content



Figure 3.6 Energy absorbed by the soil, variation with moisture content

4. Conclusions

The study results show that the sand moisture content significantly influences the drum displacement and consequently the energy absorbed by the sand during the vibratory roller compaction. The influence of the compacted sand characteristics (among which the moisture content) upon the system's response is exerted through the stiffness and damping coefficients k and c.

As it was expected, from the obtained graphs it results that the absorbed energy presents its peaks corresponding to the situations when the operating frequency has about the same value with the natural frequency. Considering this, it can be concluded that for a given type of sand at specific conditions in dry density and moisture content it is important to adjust the operating frequency at the level of the natural frequency corresponding to the actual moisture content of the compacted sand fill.

In case of a compaction plant with one fixed operation frequency, the optimum compaction efficiency can be reached by increasing the soil moisture content till near the level at which the natural frequency curve intersects the operation frequency value. The compaction work performed at these sand characteristics will determine a maximum absorbed energy.

As regarding a further study on the presented subject, it might be useful to improve the analytical model of the soil – vibratory roller system by considering the different operating phases during the dynamic process (loading, unloading and jump or lose of contact) and by taking into consideration the non-linearity of the phenomenon (the plastic settlements).

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Tensometric Measurements and Analysis of Static and Dynamic Stresses in the Gantry Crane Tower

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Abstract: This paper contains the results of experimental analysis of strain and stresses for the crane of S.C. 2x1 Holding Cape Midia Shipyard.

Keywords: Strains measurement, calculus of stresses, verification.

1. Introduction

For the gantry crane of 5/10t-17.2/8.5m T 3472/R-82-M.E.T. of the S.C. "2x1 HOLDING CAPE MIDIA SHIPYARD" S.A. after repair homologation, we was solicited to made tensometric measurements in some points on crane arm and cabin tower. In this paper we will consider exclusively the crane tower case, which is a vertical caisson structure indicated in figure 1.

Numbered point 10, 11, 12, 13, shown in figure 1, represent measuring points. In these points were placed tensometric rosettes. For static measurements the Vishay P3500 Strain Indicator, SB10 balancing units and P3650 maximum peak-minimum peak were used. The balancing of the measuring points was done only once, before static measurements, then dynamic measurements were made.



Fig.1 Crane tower caisson structure

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2. Experimental analysis

The P3650 was balanced under load before each maneuver which consist in hoisting, rotating and crane traveling, tumbling and load lowering, and the dynamic strains, time variable, maximum peak (V.M.) and minimum peak (v.m.) values were read. his mode of operation is shown in figure 2.



Fig. 2 Operation mode

In measuring point no. 13, after 10t load lifted to arm length $L_b = 8,5$ m, the followed static strains values were read:

 $\varepsilon_{0st} = -215 \mu m/m$; $\varepsilon_{45st} = -46 \mu m/m$; and $\varepsilon_{90st} = 73 \mu m/m$.

Dynamic strains according to crane traveling were:

 $\begin{array}{ll} \epsilon_{0din} &= 475 \ \mu m/m; \\ \epsilon_{45din} &= -317 \mu m/m; \\ \epsilon_{90din} &= -90 \ \mu m/m \ (simple \ number \ coincidence \\ between \ symbol \ index \ and \\ strain \ value). \end{array}$

The main strains of the static and dynamic load were calculated with known relation:

$$\varepsilon_{1,2} = \frac{\varepsilon_0 + \varepsilon_{90}}{2} \pm \frac{\sqrt{2}}{2} \sqrt{\left(\varepsilon_0 - \varepsilon_{45}\right)^2 + \left(\varepsilon_{45} - \varepsilon_{90}\right)^2} \tag{1}$$

Then the main direction was calculated according to relation:

$$\varphi = \frac{1}{2} \operatorname{arctg} \frac{2\varepsilon_{45} - \varepsilon_0 - \varepsilon_{90}}{\varepsilon_0 - \varepsilon_{90}} \tag{2}$$

Finally, the principal stresses were calculated with relations:

$$\sigma_{1} = \frac{E}{1 - v^{2}} (\varepsilon_{1} + v \varepsilon_{2})$$

$$\sigma_{2} = \frac{E}{1 - v^{2}} (\varepsilon_{2} + v \varepsilon_{1})$$
(3)

In the static mode of operation the main strains were calculated according to relation (1):

$$\varepsilon_{1,2st} = \frac{-215+73}{2} \pm \frac{1}{\sqrt{2}} \sqrt{\left(-215+46\right)^2 + \left(-46-73\right)^2} ,$$

whence $\epsilon_{1 \text{ st}} = 75,15 \mu\text{m/m}$ and $\epsilon_{2 \text{ st}} = -217,15 \mu\text{m/m}$. Because $\epsilon_{0 \text{ st}} < \epsilon_{90 \text{ st}}, \phi$ indicate the main direction (2).

$$\varphi_{st} = \varphi_{2st} = \frac{1}{2} \operatorname{arctg} \frac{2(-46) + 215 - 73}{-215 - 73} = -4,92^{\circ}$$

as shown in figure 3.



Fig.3 Main direction (2)

The principal stresses were calculated with relations (3), hence:

$$\sigma_{1 \text{ st}} = 23 \text{ MPa}$$
 and $\sigma_{2 \text{ st}} = -44.9 \text{ MPa}$.

In the dynamic mode of operation the main strains were calculated according to relation (1):

$$\varepsilon_{1,2din} = \frac{475 - 90}{2} \pm \frac{1}{\sqrt{2}} \sqrt{\left(475 + 317\right)^2 + \left(-317 + 90\right)^2}$$

whence $\varepsilon_{1 \text{ din}} = 775 \mu \text{m/m}$ and $\varepsilon_{2 \text{ din}} = -390 \mu \text{m/m}$.

The main direction was calculated with relation (2). Because $\epsilon_{0 \text{ din}} > \epsilon_{90 \text{ din}}, \phi$ indicate the main direction (1).

$$\varphi_{din} = \varphi_{1din} = \frac{1}{2} \operatorname{arctg} \frac{2(-317) - 475 + 90}{475 + 90} \cong -30,5^{\circ}$$

as shown in figure 4.





With relations (3), $\sigma_{1din} = 151.8$ MPa and $\sigma_{2din} = -36.3$ MPa was calculated and established the smallest angle between main directions $(1)_{st}$ and $(1)_{din}$.

 $\alpha_1 = 90 - 30,5 + 4,92 = 64.42$ degree

The static stress state was related to main dynamic direction (1) by means of relation:

$$\sigma_{st/din(1)} = \frac{\sigma_{1st} + \sigma_{2st}}{2} + \frac{\sigma_{1st} - \sigma_{2st}}{2} \cos 2\alpha_1 \tag{4}$$

Whence:

$$\begin{split} \sigma_{st/din(1)} &= \frac{23 - 44,9}{2} + \frac{23 + 44,9}{2} \cos 2 \cdot 64,42 \ , \quad \text{or} \\ \sigma_{st/din(1)} &= -10.95 + 33,95 \ \cos \ 128,84 \ , \quad \text{or} \\ \sigma_{st/din(1)} &= -32,24 \ MPa \end{split}$$

Total stress on dynamic main direction (1) was obtained summing σ_{1din} and $\sigma_{st/din(1)}$. Hence:

$$\sigma_{1total} = \sigma_{1din} + \sigma_{st/din(1)} = 151,8 - 32,24 = 119,55$$
 MPa

This situation is shown in figure 5.



Fig.5 Summing stresses

3. Conclusions

When in a measuring point the stress state is the resultant of two stress state, one static stress state and the other one a dynamic stress state, time variable, with different main directions, the resulting stress state can be expressed in two ways.

The strains, the main directions and stresses must be calculated apart both in static and in dynamic mode of operation. Dangerous stress state can be considered that which main directions concur with the main directions of the dynamic mode of operation. In this case static stress can be related to dynamic main directions, computing in this purpose a "static stress related to dynamic main direction (1)" by means of relation (4), through summation of static stresses related to dynamic stress main directions with main dynamic directions, $\sigma_{1 \text{ din}}$.

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The strains corresponding to static stress can be easily and simultaneous read in any directions, hence inclusively for rosette's gauges placed in considered point. The strains corresponding to dynamic stress can read by means of an individual device (such as Vishay's 3650 device) or by a data acquisitions system. In first case is desirable that the reading corresponding to rosette's gauges can be accomplished simultaneously, that is to work with three complete and individual channels (amplifier – peak indicator) for each rosette's gauges.

This is an expensive solution because of the high number of measuring devices (three amplifiers and three peak indicators). By this reason is to be preferred the second solution, by means of a data acquisition system.

If these described solutions aren't available, the problem can be fixed by means of a single measuring system (one amplifier-one peak indicator), reiterating three times in the same conditions each dynamic mode of operation, to catch (for each one) the strain peak value.

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The Analyses of the Stresses from the Reticulate Spatial Structure of the Roof of a Hydroelectric Station

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Abstract: The work is presenting the possibility of studying and also the comparison of the stresses from the reticulate spatial structure's bars in different ways of achievement, interior division and leaning, using the term of : "associated volume stresses".

The method presented permits the graphical visualisation and presentation of the stresses' variation from the bars families, that make up the structure, making easier the choosing of the best solution taking into account the exploiting needs.

Keywords: the associated volume of the stresses, the spatial structures, structure with double layer

1. Introduction

The spatial structures are those construction elements, usually metallic ones, which assure the construction of the roofs with big openings [1]. They are used at covering the openings of 30...100 metres. Their building solutions are of great diversity, they have either plane surfaces, or curve ones.

The structures with plane surfaces, on their turn, can be done like reticulate structures with one, two or three layers. The most used ones are the reticulate structures with one and two layers.

The reticulate spatial structure with double layer are made from two bar families at the superior sole and two at the inferior one, disposed in two directions and outdistanced between themselves, respectively connected with another bar family, which form the diagonals of the structure and witch are bounding with the other families in their nodes.

Depending on the way in which the bar families are laid, which form the soles of the structure and those which form their diagonals, there are a lot of ways to build the plane reticulate structures.

2. The calculation of plan spatial reticulate structures

The plane spatial reticulate structures are usually covering square or rectangular areas, in

which the ratio between the length and the height will be maximum 2, because elseways one can lose the leant on the outline plate effect.

After the loading were settled, which for a better behaviour of the structure must be concentrated in nodes, one passes to the first stage of approximate calculation.

The approximate calculation from the first stage has as a goal the exact establishment of the section of the bars from the structure, which is used in the second stage.

The calculation begins from the equation of the normal deformation on the plane of the plate, produced by the exterior loading:

$$\Delta \Delta W = \frac{p}{D} \tag{1}$$

or written under a developed form:

$$\frac{\partial^4 w}{\partial x^4} + 2 \frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{p}{D}$$
(2)

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where: D is the cylindrical rigidity of the plate:

$$D = \frac{E.I}{1 - \mu^2} = \frac{E.t^3}{12(1 - \mu^2)}$$
(3)

where:

- p is the loading of the plate in daN/m^2 ;
- μ is the Poisson Coefficient;

I - is the plate's inertness (inertia) moment, that can be replaced in equation no.(3) with the calculated value for a strip of 1 centimetre width with:

$$I = \frac{1 \cdot t^3}{12} \tag{4}$$

t - represents the height of the reticulate structure, the distance between the planes of its two soles.

The reticulate spatial structures, that have superior and inferior soles parallel and tied between the selves with diagonals, have a reduced rigidity at twisting, which permits the neglecting of the second term of the equation no. (2) in calculations, the equation becoming:

$$\frac{\partial^4 w}{\partial x^4} + \frac{\partial^4 w}{\partial y^4} = \frac{p}{D} \tag{5}$$

Equation where the first term takes into accounts the bending of the structure towards the "x" axis and the second one towards the "y" axis.

The integrating of the differential equation (5) is done with the finite element method with the help of automatic calculating programs SAP 05, SAP 90, COSMOS etc.

After series of calculations, intermediate stages, which I won't expose in this work, one can reach to the final stage of getting to the final stage of the stresses in the bars of reticulate structures. With these stresses, after splitting almost equal the areas (central, medial and outlined) with close stresses, one can make the dimensioning of the bars, can establish the areas of the section of the bars, which will be used for the exact calculation from the second stage of stresses from bars' structure.

Because the structure is very developed with a great number of finite elements and admits

cemetery axes, one can work with a half, a quarter or an eight from this.

For making easier a fast and rather simple comparison of the stresses from the bars of reticulate structures, which cover the same area, but have different:

- dimensions of the pace (eye) of the net;

- height of the structure;

- ways of leaning on the outline, at the superior and inferior sole level;

- intermediate leanings etc;

one has studied and introduced the "volume associated to stresses" notion, which was marked with $V_N[2]$.

After the calculation of the stresses in the bars of the spatial reticulate structures with one of the automatic calculating programs (SAP 05, SAP 90, SAP 95, COSMOS etc [3] the results which are given under the form of some long rows of tables it does not express an obvious thing, but in first stage an urge for giving up. For this reason it has been measured the idea of association of one 3D representing system, of the stresses and even the unitary tensions and the association of their values to some volumes of stresses or tensions, for the superior, inferior and diagonal soles net.

With the help of utilitarian "PLOT 88"[4]. was done the post-processing operation of the results, obtaining:

• the representation of the stresses in a plain description, got through a interpolation after two directions, using a third degree spleen function;

• the representation of the stresses in a spatial description using a net of maximum 50 x 50 points at base;

• the calculation of the associated volumes to stresses.

Utilitarian "PLOT 88" permits the graphic visualise of the associated volumes to stresses, this being foreseen with a module called "volume.exe", too, with the help of which one can calculate through three different ways of evaluation (the trapezes method, the Simpson method, the Simpson 3/8 method) the volume between a reference plane parallel with the one of the soles plane, the spatial interpolation area resulted through the use of third degree spleen functions and the four planes that make the borders of the bars net (points).

The graphic visualise of the associated volumes to stresses can resemble with the wrapper of the

bending moments diagrams from the girders loaded with mobile forces.

3. Practical example

For a reticulate spatial structure, that has the opening L = 24 m, the length B = 48 m, the net

pace l = 2.4 m, the height h = 1.7 m and loaded with p=265 daN/m² have resulted on bars category and zones, after the calculation [2], the stresses from the table 1:

Table 1	The maximum	stresses in the	bars of t	he superior and	inferior sole ar	nd diagonals
1 4010. 1		sucsses in the	ours or t	ne superior and	micrior sole a	ia alagonais

Zones	The maximum stresses in the bars	The maximum stresses in the	The maximum stresses in the		
	of the superior sole	bars of the inferior sole	bars of the diagonals		
	[daN]	[daN]	[daN]		
А	-33487	34090	8020		
В	-30866	28448	8100		
С	-17050	21607	9050		

Zone A being the central part, zone B being the medial one and the C zone being the outline strip of the studied structure. The bars of the structures have been built from construction pipes, choosing for the same type of bars pipes with equal diameters, modifying only the thickness of these walls as one can see in Table 2:

Table. 2. Dimensions and areas of the bars' families

Zones	Super	ior sole	Inferi	ior sole	Diagonals		
	Bar (D_n)	Area (cm ²)	Bar (D_n)	Area (cm ²)	Bar (D_n)	Area (cm ²)	
А	121 x 5.5	20.00	108 x 6.0	19.20	70 x 4.0	8.29	
В	121 x 5.0	18.20	108 x 5.0	16.20	70 x 4.5	9.26	
С	121 x 4.5	16.50	108 x 4.0	13.10	70 x 5.0	10.02	

The graphic visualise of the associated volumes of the stresses with the help of the utilitarian "PLOT 88" it is presented in Fig. 1, 2, 3, so: the Fig.1.is presenting the associated volumes of the stresses from the superior sole; the Fig. 2 is presenting the associated volumes of the stress from

the inferior sole and Fig. 3 the associated volumes of stresses from the diagonals of the structure.

The calculation of the associated volumes of stresses, visualised in Fig. 1, 2, 3 their values, through the three evaluating methods are presented in Table 3:

Table 3. The evaluating methods of the associated volumes of the stresses in the superior and inferior sole and diagonals

The evaluating method	The superior sole	The inferior sole	The diagonals					
of the volumes	(mc)	(mc)	(mc)					
Trapezoidal	19629580	18033540	396140					
Simpson	19635800	18037640	396398					
Simpson 3/8	19635480	18037560	396462					

The associated volumes of the stresses, got from this table are used further on for their comparison with other values that are calculated for

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the same reticulate structure, but for other ways of forming (dimension of the eyes of the net, heights of the structures, ways of resuming etc.).



Fig. 1. The associated volumes of the stresses from the superior sole



Fig. 2. The associated volumes of the stresses from the inferior sole



Fig. 3. The associated volumes of the stresses from the diagonals

4. Conclusions

The studied method, through which there are associated the stresses from the bars of the reticulate spatial structure, volumes of stresses, is useful in that case in which one wants a quick comparison between much more ways of making up the same structure as in the case of the maximalizing study of those from much more points of view (the weight, the number of bars, the net pace, the thickness of the structure, the leaning ways etc.).

The only big disadvantage of this method would be that the necessary labour for the selection and the manual introduction of data for rolling up the graphic utilitarian, disadvantage that may be eliminated through the conception of a program that can do on its own the selection and the introduction of the data taken from the tables with the precedent rolling of the results.

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The Effect of Material Anisotropy on the Membrane Displacements of Rotation Shells

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Abstract: In this work, a series of systematizations and developments, about the estimation of the displacements for anisotropic rotation shell in the membrane state, is made. One sets out from the general case of the material anisotropy in the tangent plan and there are made particularizations in the orthotropic material case. There are also made particularizations in the case of spherical and conical cupolas, which are the most used in practice. By means over calculus example, it is shown the anisotropy influence on the membrane displacements. The obtained results and conclusions are useful in design.

Keywords: Material anisotropy, rotation shells, membrane state, displacements.

1. Introduction

The reference literature devoted to the isotropic shells specify the geometric, elastic, supporting and loading situations and conditions which are necessary to realize the membrane state [1], [2], [3], [4], [5].

In what regards the anisotropic shells, the membrane state can be achieved with relative accuracy only for smooth shells with material anisotropy. At shells with geometric or structural anisotropy, the stiffening elements, disposed on the surface (ribs, thickenings, waves), introduce the bending effects. At the multilayered shells, although the surface can be smooth, the membrane state is possibly only if the layers are made from the same or different material with similar characteristics. If the lavers are achieved from materials with different characteristics (sandwich type shells with discontinuous stiff cores of honevcomb, waved or folded type, sandwich type shells with light cores foaming, bimetals), appear bending effects coupled with membrane effects.

In general, the stress state in the anisotropic shells represents a strong or weaker coupling between the membrane stress and the bending ones. In many particular practical cases, these effects are uncoupling and the membrane internal forces can be independent determinate from the bending one, by means of the specific equations of the membrane theory. For statically determinate shells, if the anisotropy directions (in particular the orthotropy directions) of the material are the same with the meridian and parallel directions, the membrane internal forces N_{ϕ} , N_{θ} , $N_{\phi\theta} = N_{\theta\phi}$, are not influenced by the material and they can be easily determinate from the well-known equilibrium equations. Between internal forces and the stresses are achieved in average the following equivalence relations:

$$N_{\varphi} = \sigma_{\varphi}h; N_{\theta} = \sigma_{\theta}h; N_{\varphi\theta} = \tau_{\varphi\theta}h; N_{\theta\varphi} = \tau_{\theta\varphi}h; \quad (1)$$

where h is the thickness of the shell.

A specific case, frequently met, is the axisymmetrical case. For the achieving of the symmetrical stress state with respect to the rotation axis (Δ), is necessary to be simultaneously accomplished many conditions. For the isotropic shells these are:

a) constant loading after the parallel circle (not dependent of the angle θ);

b) continuous supporting along the base parallel circle with pendulums whose direction is tangent at the middle surface;

c) complete middle surface ($0 \le \theta \le 2\pi$).

At the shells with material anisotropy, for keeping axisymmetric character of the solicitation,

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near the mentioned conditions, it must be fulfilled the followings:

d) the constitutive material should be relatively homogeneous in the thickness direction or, in the case of an accented nonhomogeneousity, this must keep the axisymmetric character;

e) the material anisotropy should be in the tangent plane at the middle surface of the shell, having the main elastic directions after the meridian and parallel.

The last two conditions involves an elastic axisymmetry in phase with geometric and loading symmetries at points a, b and c. All five conditions lead to the achievement over total axisymmetry, which involves $p_y = 0$ and $N_{\phi\theta} = N_{\theta\phi} = 0$, but the meridian N_{ϕ} and the circumferential internal forces N_{θ} are determined from the two equilibrium equations (projections on the tangent direction at the meridian curved, respectively the parallel circle).

The analytical expressions of the membrane internal forces proceeds from the usual loading (own weight, snow, wind, pressure of some fluids or powders), for particular cases of the rotation shells (spherical and conical), are presented in detail on the reference literature [1], [2], [3], [4], [5].

If one or more of previous conditions for achieving axisymmetrical state is not realized, than the stress and strain state of the shell will be unsymmetrical against the rotation axis. For the isotropic shells, this state can appear if one or more of these conditions are accomplished:

a) the loading varies along the parallel circles in some law (in particular antisymmetric);

b) the outline supporting is unsymmetrical;

c) the middle surface of the shell is incomplete.

For the anisotropic shells, the unsymmetrical stress state appears when the axisymmetrical character of the anisotropy is not maintained (for example in the case of the fibers reinforced composite, whose directions are not in coincidence with the meridian and/or parallel).

2. Physical equations for the tangent anisotropy case

In the general case of some anisotropy of the material in tangent plane, with homogeneity in the shell thickness direction the physical equations are:

$$\begin{cases} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases} = \begin{bmatrix} S_{11} & S_{12} & S_{16} \\ S_{12} & S_{22} & S_{26} \\ S_{16} & S_{26} & S_{66} \end{bmatrix} \begin{bmatrix} \sigma_{\varphi} \\ \sigma_{\theta} \\ \tau_{\varphi\theta} \end{cases}$$
(2)

or, with the membrane internal forces:

$$\begin{cases} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases} = \frac{I}{h} \begin{bmatrix} S_{11} & S_{12} & S_{16} \\ S_{12} & S_{22} & S_{26} \\ S_{16} & S_{26} & S_{66} \end{bmatrix} \begin{bmatrix} N_{\varphi} \\ N_{\theta} \\ N_{\varphi\theta} \end{cases}$$
(3)

where the coefficients S_{ij} (i, j = 1, 2, 6) of the compliance matrix [S], characterize the anisotropic material flexibility and they can be put in correspondence with an equivalence matrix of technical values, which can express the internal forces depending on the strains by the equation:

$$\begin{cases} N_{\varphi} \\ N_{\theta} \\ N_{\varphi\theta} \end{cases} = h \begin{bmatrix} Q_{11} & Q_{12} & Q_{16} \\ Q_{12} & Q_{22} & Q_{26} \\ Q_{16} & Q_{26} & Q_{66} \end{bmatrix} \begin{bmatrix} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases}$$
(4)

where the values Qij (i, j = 1, 2, 6) of the material stiffness matrix [Q], are obtained by inversing the compliance matrix [S].

If the anisotropic main directions of the material shell agree with the meridian, respectively the parallel circle directions, that is an orthotropic material in plane stress state, then the linear strains are uncoupling from the angular strains and, alike, we have the uncoupling of the internal forces:

$$\begin{cases} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases} = \frac{1}{h} \begin{bmatrix} S_{11} & S_{12} & 0 \\ S_{12} & S_{22} & 0 \\ 0 & 0 & S_{66} \end{bmatrix} \begin{bmatrix} N_{\varphi} \\ N_{\theta} \\ N_{\varphi\theta} \end{cases}$$
(5a)

$$\begin{cases} N_{\varphi} \\ N_{\theta} \\ N_{\varphi\theta} \end{cases} = h \begin{bmatrix} Q_{11} & Q_{12} & 0 \\ Q_{12} & Q_{22} & 0 \\ 0 & 0 & Q_{66} \end{bmatrix} \begin{cases} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases}$$
(5b)

The elastic coefficients in the flexibility matrix of the material, are in correspondence with technical values, and they have the expressions:

$$S_{11} = \frac{1}{E_{\varphi}}; \quad S_{22} = \frac{1}{E_{\theta}};$$

$$S_{12} = -\frac{v'_{\varphi}}{E_{\varphi}} = -\frac{v'_{\theta}}{E_{\theta}}; \quad S_{66} = \frac{1}{G}$$
(6)

where E_{φ} , E_{θ} are the elasticity modulus in the tangent direction at the meridian curve and, respectively the parallel circle; $G_{\varphi\theta} = G_{\theta\varphi} = G$ – the shear modulus; $v'_{\varphi\theta} = v'_{\varphi}$, $v'_{\theta\varphi} = v'_{\theta}$ - the Poisson's ratios for tension-compression along the meridian curve respectively the parallel circle direction. In the same way, we introduce the notations:

$$hQ_{11} = \frac{E_{\varphi}h}{I - v'_{\varphi}v'_{\theta}} = B_{\varphi}; \quad hQ_{22} = \frac{E_{\theta}h}{I - v'_{\varphi}v'_{\theta}} = B_{\theta} \quad (7)$$
$$hQ_{12} = B_{\varphi}v'_{\theta} = B_{\theta}v'_{\varphi} = B^{*}; \quad hQ_{66} = B_{\varphi\theta}$$

 B_{ϕ} , B_{θ} , $B_{\phi\theta}$, B^* are the membrane stiffnesses of the shell. Now, the physical equations can be written:

$$\begin{cases} N_{\varphi} \\ N_{\theta} \\ N_{\varphi\theta} \end{cases} = \begin{bmatrix} B_{\varphi} & B^* & 0 \\ B^* & B_{\theta} & 0 \\ 0 & 0 & B_{\varphi\theta} \end{bmatrix} \begin{cases} \varepsilon_{\varphi} \\ \varepsilon_{\theta} \\ \gamma_{\varphi\theta} \end{cases} \quad a) \implies (8)$$
$$\{N\} = [B]\{\varepsilon\} \quad b)$$

where $\{N\}$ – the internal forces vector; $\{\epsilon\}$ – the strain vector; [B] – the membrane stiffness matrix (the extensional stiffness matrix) of the shell.

In a particular case, if the extensional stiffnesses of the orthotropic composite shell in the meridian and the parallel directions are the same, that is

$$B_{\varphi} = B_{\theta} = B; \quad v'_{\varphi} = v'_{\theta} = v' \tag{9}$$

it results the equivalence:

$$Eh = B\left(I - {v'}^2\right) \tag{10}$$

and in what follows it can be used the computing relations from the isotropic shells.

3. Geometrical equations. Developments about the displacements estimations

The membrane displacements u in the parallel circle and w in the normal on the middle surface directions depending of the strains by the following differential equations system [1], [4]:

$$\varepsilon_{\varphi} = \frac{1}{r_{l}} \left(\frac{\partial u}{\partial \varphi} - w \right)$$

$$\varepsilon_{\theta} = \frac{1}{r} \left(\frac{\partial v}{\partial \theta} + u \cos \varphi - w \sin \varphi \right) =$$

$$= \frac{1}{r_{2}} \left(\frac{\partial v}{\partial \theta} \frac{1}{\sin \varphi} + u \cot \varphi - w \right)$$

$$\gamma_{\varphi\theta} = \frac{1}{r_{l}} \frac{\partial v}{\partial \varphi} - \frac{v \cos \varphi}{r} + \frac{1}{r} \frac{\partial u}{\partial \theta} =$$

$$= \frac{1}{r_{l}} \frac{\partial v}{\partial \varphi} - \frac{1}{r_{2}} \left(v \cot \varphi - \frac{1}{\sin \varphi} \frac{\partial u}{\partial \theta} \right)$$
(11)

where r_1 , r_2 are the main curvatures radii of the shell middle surface, r – the parallel circle radius, delimitated from the angle φ (Fig. 1).



Fig. 1. Geometrical parameters for rotation shells (a – spherical; b – conical)

For to determinate the displacements, it is integrates the system (11), using the corresponding boundary conditions written in displacements. In the axisymmetrical case, the v displacement and all the derivatives in terms of θ are zero, thus $\gamma_{\omega\theta} = 0$ and:

$$\varepsilon_{\varphi} = \frac{l}{r_1} \left(\frac{du}{d\varphi} - w \right); \quad \varepsilon_{\theta} = \frac{l}{r_2} \left(u \cot \varphi - w \right) \quad (12)$$

Eliminating w in the Eq. (12), it is obtained the differential equation:

$$\frac{du}{d\varphi} - u\cot\varphi = r_1\varepsilon_{\varphi} - r_2\varepsilon_{\theta} = f(\varphi)$$
(13)

where $f(\phi)$ results replacing ϵ_ϕ and ϵ_θ from Eq. (5a), as it follows:

$$f(\varphi) = \frac{1}{h} \left[(r_1 S_{11} - r_2 S_{12}) N_{\varphi} + (r_1 S_{12} - r_2 S_{22}) N_{\theta} \right]$$
(14)

The displacement u results by the integration of Eq. (13):

$$u = \sin \varphi \int \frac{1}{h \sin \varphi} [(r_1 S_{11} - r_2 S_{12}) N_{\varphi} + (r_1 S_{12} - r_2 S_{22}) N_{\theta}] d\varphi + C$$
(15)

The constant C is determinate from the supporting conditions on the base parallel circle. For example, if the supporting is achieved with pendulums having tangent direction at the meridian, it impose the conditions that for the maximum value of the angle ϕ , the displacement u is null, that is, for $\phi = \phi_m \rightarrow u = 0$ and:

$$C = -\sin\varphi_{m} \int \frac{1}{h\sin\varphi_{m}} \left[(r_{1}S_{11} - r_{2}S_{12})N_{\varphi} + (r_{1}S_{12} - r_{2}S_{22})N_{\theta} \right] d\varphi$$
(16)

The displacement w results from the second geometric equation (12):

$$w = u \cot \varphi - r_2 \varepsilon_{\theta} = u \cot \varphi - \frac{r_2}{h} \left(S_{12} N_{\varphi} + S_{22} N_{\theta} \right)$$

but the first equation from (12) can be used for checking.

In practice, it is determined also the horizontal displacement $\delta_H = \Delta r$, the vertical displacement δ_V and the support rotation β :

$$\delta_{H} = \Delta r = u \cos \varphi - w \sin \varphi = r\varepsilon_{\theta}$$

$$\delta_{V} = u \sin \varphi + w \cos \varphi; \quad \beta = \frac{1}{r_{I}} \frac{dw}{d\varphi}$$
(18)

For the spherical surface (Fig. 1a), where $r_1=r_2=R$, the Eq. (14) becomes:

$$f(\varphi) = \frac{R}{h} \left[(S_{11} - S_{12}) N_{\varphi} + (S_{12} - S_{22}) N_{\theta} \right]$$
(19)

For the conical surface, where $r_1 \rightarrow \infty$, $r_1 d\varphi = dx$, cot $\varphi = tan \alpha$ (Fig. 1b), results:

$$\varepsilon_{\varphi} = \frac{du}{dx} = \frac{1}{h} \left(S_{11} N_{\varphi} + S_{12} N_{\theta} \right)$$

$$\varepsilon_{\theta} = \frac{1}{r_2} \left(u \tan \alpha - w \right)$$
(20)

whence:

$$u = \int \frac{l}{h} \left(S_{11} N_{\varphi} + S_{12} N_{\theta} \right) dx + C$$

$$w = u \tan \alpha - \frac{r_2}{h} \left(S_{12} N_{\varphi} + S_{22} N_{\theta} \right)$$
(21)

4. Results for spherical cupola loaded by snow

Using the elastic coefficients from (Eq. 7) and the expressions of the internal forces from the literature [1], [3], [4], [6], by processing the (Eq. 21) in the case of conical cupola loaded by snow with q intensity (Fig. 2), we obtain the displacements:

$$u = \frac{q \tan \alpha}{4E_{\varphi}h} \left(l - 2v'_{\varphi} \sin^2 \alpha \right) \left(l^2 - x^2 \right)$$
a)

$$w = \frac{q \tan^2 \alpha}{4h} \left[\frac{l}{E_{\varphi}} \left(l - 2v'_{\varphi} \sin^2 \alpha \right) \left(l^2 - x^2 \right) - b \right) (22)$$

$$- \frac{2x^2}{E_{\varphi}} \left(v'_{\theta} - 2 \sin^2 \alpha \right) \right]$$



Fig. 2. Conical cupola loaded by snow
For the following comparative numerical analyses, we use the conditioning relations between the elastic constants of the material as elasticity ratio, \overline{E} , respectively contraction ratio by tension-compression, $\overline{v'}$. These ratios can be highlighted from the physical equation (7), [7], [8]:

$$\frac{B_{\varphi}}{B_{\theta}} = \frac{E_{\varphi}}{E_{\theta}} = \overline{E}, \quad B_{\varphi}v_{\theta}' = B_{\theta}v_{\varphi}', \quad \frac{B_{\varphi}}{B_{\theta}} = \frac{v_{\varphi}'}{v_{\theta}'} = \overline{v'} \quad (23)$$

Using the elasticity ratio \overline{E} and the contraction ratio $\overline{\nu'}$, the expresses (22) of the membrane displacements become:

$$u = -\frac{ql^2}{E_{\theta}h} \frac{\tan\alpha}{4} \left(\frac{1}{\overline{v'}} - 2v'_{\theta}\sin^2\alpha\right) \left(\frac{x^2}{l^2} - l\right)$$
$$w = -\frac{ql^2}{E_{\theta}h} \frac{\tan^2\alpha}{4} \left[\frac{x^2}{l^2} \left(\frac{1}{\overline{v'}} - 2v'_{\theta}\sin^2\alpha + 2v'_{\theta} - -4\sin^2\alpha\right) - \left(\frac{1}{\overline{v'}} - 2v'_{\theta}\sin^2\alpha\right)\right]$$
(24)

The comparative numerical analyses fulfilled for a conical cupola, having at the top the angle $2\alpha =$ 73.74°, is made from a material with $\nu'_{\theta} = 0.25$. For the contraction ratio was taken the values: 0.5, 0.75, 1.0 (isotropic material), 1.5 and 2.0.

The displacements u, to the generatrix direction of the cone, are plotting in Fig. 3 and the normal displacements w are shown in Fig. 4. The values from the diagrams must to be multiply by the constant value $ql^2/E_{\theta}h$.



Fig. 3. The displacements "u" to the generatrix direction



Fig. 4. The displacements "w" to the normal direction on the surface

5. Conclusions

1. It comes out that the important variations of the displacements u, for different contraction ratios, the greatest values are at the top of the cupola (x = 0). Thus, we take as reference point the displacement u in case of the isotropic material (curve u_3), it comes out that:

a) for the sub-unitary ratios $\overline{v'}$, the displacements at the top of the cupola are growing (about 41% for $\overline{v'} = 0.75$ – curve u_2 , respectively 122% for $\overline{v'} = 0.5$ - curve u_1);

b) for the supra-unitary contraction ratios, the displacements u at the top of the cupola are decreasing (about 40% for $\overline{v'} = 1.5$ – curve u₄, respectively 61% for $\overline{v'} = 2.0$ – curve u₅).

2. At the top of the cupola, the displacements w have the like variations as the displacements u.

3. At the base of the cupola, the displacements w results with same values and there are not depending on from the constitutive material, which can be isotropic or orthotropic.

4. In the numerical analyses fulfilled, it kept the ratio $ql^2/E_{\theta}h$ like a common factor. If the geometric parameters l and h can be maintained constantly, like the loading parameter q, but we can not say the same deal about the elastic modulus E_{θ} , which influences the elasticity ratio (also contraction ratio), in accordance with the definition relations (Eq. 23). For to obtain an exactly image about the influence of the material anisotropy on the membrane displacements, in practical cases, the E_{θ} modulus will be numerical replaced.

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The Efforts Study witch Appear on the Joining into two Different Geometrical Covers

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Abstract:

It appears challengers on the contour at the joining into two covers (cylindrical body and semi-spherical bottom) which are generated by the Q_0 effort and the M_0 moment.

Keywords: covers, efforts.

1.Introduction

It appears bend effects at the joining into two different geometrical covers which affect the diaphragm state in case of an external effort.

These effects are generated by the Q_0 effort and the M_0 moment, which it's necessary to be determinate.

2. Apparatus presentation

The making of the 'Communication Apparatus' and the 'Wireless Phone for the divers', imposed the designing of the 'Container for testing the electric cables. The components are shown in Fig. 1.



Fig 1.Continer for testing the electric cables 1Recipient bottom; 2.Cylindrical body; 3.Screwing flange; 4.Arm A; 5.Flange coupling; 6.Lid; 7.Arm B 8.Load equalization; 9.Propping;

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This recipient under inner pressure may be used for testing the strength and tightness of the special electric cables and other apparatus used in diving activities.

H 1660 mm

The spherical solders bottom cylindrical body. The Q_0 effort and the M_0 moment are emphasised by the section between the two covers. (see Fig. 2). The main characteristics of the container are:

work pressure : 38 bars

- pressure for hydraulic check-up : 47 bars
- ringing fluids : water
- dimensions : ø 312*6 mm,

The covers are too solicited by the inner pressure p_i =3,8 MPa.

The unknown quantities Q_0 and M_0 , can be calculated by the continuity condition of the radial movement w and of a rotation angle θ . The w and the θ are shown in Fig. 2.



Fig.2 The joining between the two covers

Writing the radial movement and rotation angle formula caused by the Q_0 effort, M_0 moment and internal pressure p_i , for the two covers.

Because these are soldering, it equals the radial movements and the rotation angles and results an equations system.

The solutions of system are the values $Q_{0}\xspace$ and $M_{0}.$

Then it determines the σ_{1x} and σ_{2x} efforts in any point at x away. 3. Elements of calculation

For the cylindrical body:

r = cylinder ray

 δ_c = cylinder thickness

E = the elasticity modulus

Y= Poisson's constantD = rigidityK = coefficient of pay off

w_c = radial movement of cylindrical body

 θ_c = rotation angle of cylindrical body

R = spherical ray

 δ_s = spherical thickness

 w_s = radial movement of spherical body

 θ_s = rotation angle of spherical body

 λ = coefficient pay off

$$k = \frac{\left[3\left(1 - \gamma^2\right)\right]^{\frac{1}{4}}}{\sqrt{r \times \delta_c}} = 42 \quad (1)$$

$$D = \frac{E \times \delta_{c}^{-3}}{12 \times (1 - \gamma^{2})} = 115,4(2)$$

$$w_{c} = \frac{Q_{0}}{2 \times K^{3} \times D} + \frac{M_{0}}{2 \times K^{2} \times D} + \frac{p \times r^{2}}{2 \times E \times \delta} \times (2 - \gamma)(3)$$

$$\theta_{c} = -\left(\frac{Q_{0}}{2 \times K^{2} \times D} + \frac{M_{0}}{K \times D}\right)(4)$$

$$\lambda = [3 \times (1 - \gamma)]^{1/4} \times \sqrt{\frac{R}{\delta_{s}}} = 0,167(5)$$

$$w_{s} = \frac{2 \times \lambda^{2} \times M_{0}}{E \times \delta_{s}} - \frac{2 \times \lambda \times R \times Q_{0}}{E \times \delta_{s}} + \frac{p \times R^{2}}{2 \times E \times \delta_{s}} \times (1 - \gamma)(6)$$

$$\theta_{s} = \frac{4 \times \lambda^{3} \times M_{0}}{E \times R \times \delta_{s}} - \frac{2 \times \lambda^{2} \times Q_{0}}{E \times \delta_{s}} (7)$$

$$\langle w_{c} = w_{s} \Rightarrow \langle M_{0} = 0,0595 \text{ NM} \approx 0 \text{ NM}(8) \\ w_{c} = w_{s} \Rightarrow \langle M_{0} = -4,9 \text{ N} \approx -5 \text{ N}(9) \rangle$$

 $\begin{array}{c} Considering \ the \ covers \ solicited \ only \ by \ Q_0 \\ and \ M_0 \ , \ the \ calculation \ final \ relations \ are \\ determined \ from \ the \ differential \ equation \ by \ 4 \ order : \end{array}$

$$\frac{d^4w}{dx^4} + 4k^4w = 0(10)$$

Real solutions of the equation are:

$$w = e^{-kx} (A\cos kx + B\sin kx)(11)$$

The calculation final relations are :

$$Q_x(Q_0) = Q_0 e^{-kx} (\cos kx - \sin kx)(12) \qquad T_x$$
$$Q_x(M_0) = -2kM_0 e^{-kx} \sin kx (13) \qquad T_x$$

$$K_{x}(Q_{0}) = \mu Q_{x}(16)$$

$$K_{x}(M_{0}) = \mu M_{x}(17)$$

$$S_{x}(Q_{0}) = 0 (18)$$

$$S_{x}(M_{0}) = 0 (19)$$

$$T_{x}(Q_{0}) = 2krQ_{0}e^{-kx}\cos kx (20)$$

$$T_{x}(M_{0}) = 2k^{2}rM_{0}e^{-kx}(\cos kx - \sin kx)(21)$$

 $M_x(Q_0) = Q_0 e^{-kx} \sin kx$ (14)

 $M_{x}(M_{0}) = M_{0}e^{-kx}(\cos kx - \sin kx)$ (15)

_ 101

$$\theta_x(Q_0) = \frac{2k^2r^2}{\delta E} Q_0 e^{-kx} \cos kx \quad (22)$$

$$\theta_x(M_0) = \frac{4k^3r^2}{\delta E} M_0 e^{-kx} \cos kx$$
(23)

$$w_x(Q_0) = -\frac{2kr^2}{\delta E} M_0 e^{-kx} \cos kx \,(24)$$

$$w_x(M_0) = \frac{2k^2 r^3}{\delta E} M_0 e^{-kx} (\cos kx - \sin kx)(25)$$

The efforts on the outline x away are:

$$\sigma_{Ix} = \frac{S_x(Q_0) + S_x(M_0)}{\delta} \pm 6 \frac{M_x(Q_0) + M_x(M_0)}{\delta^2} (26)$$

$$\sigma_{2x} = \frac{T_x(Q_0) + T_x(M_0)}{\delta} \pm 6 \frac{K_x(Q_0) + K_x(M_0)}{\delta^2} (27)$$

Where:

Where:

+ for the inner surface

- for the external surface

4. Conclusions

All the calculation relations depend on the 4 functions f_i (kx), which are shown in Fig. 3. Observations: For $kx \rightarrow \Pi$ (28), $f_i(kx) \rightarrow 0$ (29).

The f_i functions are paying off wavy and they are null



for $kx = \Pi$.

Fig.3 The f_i pay off wavy functions

For $kx \in (0-\Pi)$, the graph is a semi-wave which has the length = ls

For kls =
$$\pi \Longrightarrow$$
 ls = $\frac{\pi}{k}$.(30),

The tensions are pay off and the effect is null.

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The Modelling of the Behaviour of the Reinforced Concrete through a Macroelement

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Abstract: The present paper has a target to simulate as exactly as possible the modelling of the reinforced concrete.

Keywords: finite element, concrete, truss

1. Introduction

The macroelement I proposed, wich is made up of Truss elements, takes into consideration both the fissuring of stretched section of reinforced concrete and the beginning of its flowing into the



reinforcement. The comparasion of the results is realized by the help of the classic modalities of calculation, trying to emphasize the element's exact behaviour on the entire standing of loading and also the reserve of resistance of the element.



Figure 1 The console study

- - 2

Firstly, we do a study on a console with the length of 0,8m, the section of 40x40cm and with the following characteristics: $E=240000 \text{ daN/cm}^2$ and $I=2.13E-3m^4$, presented in Figure 1.

To this console we apply at its empty end a concentrated force of 1 tf. Using the relation Maxwell-Mohr we can determine the maximum change of place the console:

$$\Delta = \int \frac{Mm}{EI} dx = \frac{1}{EI} \frac{1}{2} \cdot l \cdot P \cdot l \cdot \frac{2}{3} \cdot l = \frac{Pl^3}{3EI}$$
$$\Delta = \frac{1 \cdot 0.8^3 \cdot 10^{-3}}{3 \cdot 0.24 \cdot 10^7 \cdot 2.13 \cdot 10^{-3}} = \frac{0.512}{15336} = 3.33E - 5m$$

The modulus of resistance of the section will be:

$$W = \frac{bh^2}{6} = \frac{40 \cdot 40^2}{6} = 10666 cm^3$$

The maximum tension will be:

$$\sigma = \frac{M}{W} = \frac{1000 \cdot 80}{10666} = 7,5 \, daN \, / \, cm^2$$

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Figure 2 The panel study

The panel by the help of which I realize the simulation of the console of reinforcement concrete, presented in figure 2, will have to behave identically as the console in the strain field of elastic behaviour on the area of the tensions strain smaller than the maximum resistance of strain of the concrete for the considered mark.

The panel in figure 2 is made up only of elements of Truss type. The elements 1 and 2 replace the streched section, respectively the compressed section of concrete. On these elements, linked by the same knots, I put the elements 3 and 4 by the help of which I simulate perfectly the steel reinforcement. These elements have a non-linear behaviour. Thus for the elements which replace the section of reinforced concrete.

I defined a curve of unitary effort a specific deformation conform with the real behaviour of the concrete curve presented in figure 3.

Table 1 Constants of material (stell)

Point	Strain	Stress
1	-0.4	0.
2	-0.3	-0.36e4
3	-0.121e-2	-0.255e4
4	0.	0.
5	0.121e-2	0.255e4
6	0.3	0.36e4
7	0.4	0.

Table 2 Constants of material (concrete)

Point	Strain	Stress
1	-0.4	0.
2	-0.52e-3	0.
3	-0.52e-3	-0.125e3
4	0.	0.
5	0.37e-4	0.9e1
6	0.37e-4	0.



Figure 3 The curve of unitary strain-stress for concrete and stee

As we can observe in figure 3a by the help of this curve we emphasize the maximum resistance of

straining of the concrete, thus simulating its fissuring on the strained area.

The elements 3 and 4 are elements which simulate the reinforced concrete. I realized the curve of strain–stress conform to figure 3b in order to model the beginning of following into the reinforcement.

The elements 5,6,7 and 8 have a linear behaviour with the role of making a rigid panel. The efforts which we obtain in these elements have no practical value.

The initial idea is that this panel must behave identically as the console speaking of change of place. So we realized more rolling regulating the area of the diagonals until we obtained the change of place of the console calculated analiticaly. The rolling tests were realized on the panel of study using an action upon the empty end of the panel with a force that increases incrementally with 10 steps to 100t. The maximum efforts of the concrete are under the maximum limit of strain. At the same time during this rolling the elements that simulate the reinforcement were disconnected. Practically these rollings under the form of tests were made in order to obtain the rigidity when we bend the panel, rigidity that must be identical with the value of the console's rigidity which we study.

Speaking of the axial rigidity it is not the same thing because the areas of elements 1 and 2

are identical to the areas of the concrete sections which they simulate.

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This is a drawback when speaking of a bending as in this way we practically simulate a tension distribution on the streched or compressed area as being constant.

At the same time we should take into account that we are concentrating an equivalent area to a bigger lever's arm (1/2 instead of 0,661/2). From the numerical study realized for the bending I obtained a unitary effort within the elements that simulae the concrete situated somewhere between maximum and minimum obtained with an analytical method by the help of Navier's relation.

In table 3 I presented the modality of evolution of the efforts within the concrete and the reinforcement both for the streched and for the compressed area. We can observe that beginning with step 9 the reinforcement for the streched area penetrates within the plastic and a maximum tension is attained during the compression for the compressed area of concrete. The maximum tension of stretching of the concrete is attained from the first calculation.

The evolution of the changes of place is presented in figure 5. Here we can observe the point where the plastic articulation is formed and also the asimptotic evolution of the changes of place.

Step	<u>1</u>	<u>2</u>	<u>3</u>	4	<u>5</u>	<u>6</u>	<u>7</u>	<u>8</u>	<u>9</u>	<u>10</u>
Stretched	0	0	0	0	0	0	0	0		
Concrete										
Compressed	-13.3	-27	-40	-54	-67	-81.3	-95	-115		
concrete										
Stretched	34.6	737	1097	1464	1831	2199	2550	2550		
steel										
Compressed	-116	-238.1	-357	-476	-594	-713	-833	-1016		
steel										

Table 3



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Figure 4 Time history displacement at the end of the console (when soliciting a bending)

In figure 4 we can observe that at step 7 occurs the entry into folowing of the reinforcement. At the same time we also did a calculation in accordance with the standard, in this way determining the moment capable of double reinforced sections: $M_{cap} = A_a R_a h_a = 20 tm$.

Taking into account that the maximum force applied is of 100t, the length of the console is of 0,8 and the step of calculation at which the folowing of the reinforcement begins is 7 the result is a maximum moment of 56tm, therefore a little more than double as opposed to the one obtained by the standard one. Thus we obtain a reserve of resistance. In table 3 we can observe that the entry into folowing of the reinforcement is produced at the same time with the (crushed concrete) crushing of the concrete. We deduce of this that the idealization realized by the standard by the fact the whole moment is taken over by stretched reinforcement is satisfactory.

Another study can be realized for the console subjected to a simple compression.

In this case the model used resembles a lot to the real one.

I simulated by integrating in an exact period of time the step of time at which the crushing of concrete is produced.

In figure 5 I presented the evolution of displacement at the end of the console. We can clearly observe the point in which the changes of place tend to reach asimptotically to the infinite.

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Figure 5 Time history displacement of the end of the console when solicited by compression

Table 4	(time history	stresses for	compression)
---------	---------------	--------------	--------------

Step	1	2	3	4	5	<u>6</u>	7	8	9	<u>10</u>
Stretched	-13.	-26.	-39.4	-52.	-65.	-78.	-91.	-105.	-118.	0.
Concrete										
Compressed	-13.	-26.	-39.4	-52.	-65.	-78.	-91.	-105.	-118.	0.
concrete										
Stretched	-115.	-230.	-345.	-460.	-575.	-690.	-806.	-921.	-1036	-2550
steel										
Compressed	-115.	-230.	-345.	-460.	-575.	-690.	-806.	-921.	-1036	-2550
steel										

Step	<u>1</u>	<u>2</u>	3	4	<u>5</u>	<u>6</u>	<u>7</u>	8	<u>9</u>	10
Stretched	0.	0.	0.	0.	0.	0.	0.	0.	0.	0
Concrete										
Compressed	0.	0.	0.	0.	0.	0.	0.	0.	0.	0
concrete										
Stretched	330.	670.	1006.	1341.	1677.	2012	2348	2550	2551	2551
steel										
Compressed	330.	670.	1006.	1341.	1677.	2012	2348	2550	2551	2551
steel										

Table 5 (time history stresses for stretched)

2. Conclusion

Through study presented here we can simulate much better the behaviour of reinforced concrete for all type of solicitations. We obtain the point of fissuring of concrete at stretched, crashing point at compression and the entering in plastic of reinforcement.

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The Response of Nonlinear Structures Subjected to Shock

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Abstract: The recent dynamic tests shows that the "pipe" equipment are degraded under the loads from 15 to 20 times greater than those actions that leads to the collapse of the resistance structures. Because these structures behave better than the requirements of the standards and codes, is a reassuring fact but not also economic. In order to guarantee a non-existent degradation (the plastic instability) the designers are forced to limit the pipe's displacements and to reinforce them. A solution in order to solve this problem is to increase the number of classic supports and also the number of self-blocking damper devices. The presence of the functional legerities complicates the dimensioning introducing a shock non-linearity, in case of sudden movements, between the structure and these supports.

Keywords: Nelinear, dinamic, rigidity, shock.

1. Introduction

The nuclear-electrically power/ stations are usually placed in regions where the seismic risk is not an neglected fact that must be take into account, in the design concept of the structures and also for design of electronically, electric and mechanically equipment.

Looking at the mechanically structures the codes and standards defines for each component, strength and deformations criteria under the action of the static, dynamic and seismic loads. The actually settlements aims, among other things, the protection of these structures against the degradations produced by the plastic instability (plastic joints appearance due to great displacements) that could be produced by the earthquakes. These powerfully types of degradations could be observed only in specialized laboratories.

The recent dynamic tests shows that the "pipe" equipment are degraded under the loads from 15 to 20 times greater than those actions that leads to the collapse of the resistance structures.

The over-sizing of the structures comes from the simple modeling of the earthquakes, so that the benefic effects (from the dynamic and transitory

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character of the action) are neglected. The structure's ductility (just a few taken into account) also contribute to in order to explain those deviations. Because these structures behave better than the requirements of the standards and codes, is a reassuring fact but not also economic.

In order to guarantee a non-existent degradation (the plastic instability) the designers are forced to limit the pipe's displacements and to reinforce them. A solution in order to solve this problem is to increase the number of classic supports and also the number of self-blocking damper devices. These devices permit slowly movements produced by the thermal dilatation and also block the pipes in case of sudden movements (produced by the earthquakes). These devices are expensive and needs periodically checking, and in case of damage could block the pipes producing thermal contractions from the temperature variation). This risk could produce degradations from fatigue. So, it is necessary to modify the current practice releasing the self-blocking damper devices (if is possible) in order to keep only the slender pipes sustained by supports with console. This simple technical solution is cheaper and seems more applicable because it allows the dissipation of a part of seismic energy by friction and local plastification, but also needs through knowledge about he dynamic behavior of the slender pipes fixed only by supports. The presence of the functional

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legerities complicates the dimensioning introducing a shock non-linearity, in case of sudden movements, between the structure and these supports. Otherwise, considering the ductility over the accepted local plastification can be extremely delicate.

The actual settles tendencies, concerning a much complete dimensioning than the actual checking criteria are the "reliability" and "degradation level" notions.

The probabilistic methods, exacts or approximate, have been developed to solve some problems of non-linearity. Through these methods there was elaborate techniques of linearity in order to replace non/linear systems with equivalent linear systems, carefully chosen. This "stochastic linearity" concept is attractive while the linearity hypotheses are the basics for the actual computation codes.

The techniques of linearity are still in research. The techniques of linearity applied to the systems subjected to socks presume numerical simulation and/or analytically computation. It is well known that the response's power densities are returned in a small part when classic linearity methods are used. The replacing possibilities of non-linear subsystems into equivalent linear subsystems are restricted in case of big linear structures that include these subsystems.

2. The Shock Non-linearity

The shock non-linearity is essential in the analysis of the pipes behavior subjected to earthquakes.

During the impact between two structures, for instance a pipe and one of the supports, appears new linkage forces. The application zones is localized by the slight deformation of the structures. Practically, it is considered that each of the contact is punctual.

The local aspects of deformation near the supporting zones determine the amplitudes and directions of these forces. The analysis of this problem leads to the contact law's formulation that take into account the elastic, dissipate and, eventually, inertial effects because a fluid is bounded to the structure (the case of the pipes, vapors generators, etc). The main component of the impact force is a normal effort (the tangential component will be neglected).

In what concern the pipes, the tubes of heat changers, etc the structure's type is a bending subjected girder. Every new impact induce vibrating phenomena, that will influence:

- the general behavior of the structure the frequencies could be represented through first longitudinal bending of the girder.
- the local disturbing of the impact zone high frequency vibrations that interfere to transversal modes (shell modes) and that spread waves in the pipe's thickness.

The associate models of these structures are unidimensional and the local deformation aspect of the straight section can be considered through a statically term (Figure 1).



Figure 1: a) Before Shock b) After Shock

 k_o is the pipe's ellipsoidal stiffness and can be obtained by a static computation or tests, after the static displacement δ_r produced by the static load *F*.

The ellipsoidal effect can be considered linear for small static displacement (Figure 2).



Knowing the frequencies of the first transversal modes, in order to represent the general behavior of the structure, it can be chose to cut (at the basis) the longitudinal main modes, starting from this frequency that is superior to the first transversal mod. Not to neglect the girder's stiffness tied to these upper modes (that is important in order to obtain a good approximation of the sock waves) it will be introduce a residual stiffness k_r that is computed taking into account the negligible longitudinal modes.

The usual modeling of the uni-dimensionally girder structures lead to the defining of stiffness to the sock k'.

$$k'^{-1} = k_0^{-1} + k_r^{-1} + k_b^{-1}$$

3. The Equivalent Linearity Method

Because the linear systems are easily to solve, the nonlinear systems study can be done with enough accuracy, by replacing the nonlinear system with an equivalent linear system, who's characteristics are almost the same as those of the initial system.

Booton described this linearity technique of the system subjected to the aleatory actions for the first time in 1953.

SDOF Systems

The simplest mechanical system is the SDOF oscillator, and the motion equation for this system is: $m\ddot{y}(t) + h(y, \dot{y}) = f(t)$ (1)

where: y, \dot{y}, \ddot{y} represents the displacement, the velocity and the acceleration

 $h(y, \dot{y})$ is a function that describe the oscillator non-linearity

f(t) is the aleatory action.

The linearity method concern in replacing the non-linear oscillator with an equivalent linear oscillator. The (1) equation becomes:

$$n\ddot{\widetilde{y}}(t) + c_{eq}\dot{\widetilde{y}}(t) + k_{eq}\widetilde{y}(t) = f(t)$$
(2)

where: $\tilde{y}, \dot{\tilde{y}}, \ddot{\tilde{y}}$ represents the displacement, the velocity and the acceleration of the equivalent linear system

 c_{eq} is the damp viscosity of the equivalent system k_{eq} is the stiffness equivalent linear system (c_{eq} şi k_{eq} must be determined).

It is normally to say that the equivalent linear system's movement is nit identical to the non-linear system and, from this, the solutions $\tilde{y}, \dot{\tilde{y}}, \ddot{\tilde{y}}$ doesn't exactly check the (2) equation, that becomes:

$$m\widetilde{y}(t) + h(\widetilde{y}, \widetilde{y}) = f(t) + \varepsilon(t)$$
(3)

where: $\varepsilon(t)$ is the error that allows to measure the difference between the (1) and (2) equations.

If we subtract the (2) and (3) equations we will get:

$$\varepsilon(t) = h(\tilde{y}, \dot{\tilde{y}}) - c_{eq}\dot{\tilde{y}}(t) - k_{eq}\tilde{y}(t)$$
(4)

In order to determine the c_{eq} and k_{eq} the error $\varepsilon(t)$ must be minimal.

The methods suggest a minimization process of the $\varepsilon(t)$ starting from the identity:

$$\frac{\partial E[\varepsilon(t)^2]}{\partial C_{eq}} = \frac{\partial E[\varepsilon(t)^2]}{\partial K_{eq}} = 0$$
(5)
$$\frac{\langle E[\varepsilon(t)^2]}{\langle E[\varepsilon(t)^2] \rangle} = 0$$

 $(E[\varepsilon(t)^2]$ matematic operator)

Developing the (5) equation will be obtained the system:

$$K_{eq} = \frac{E[h \cdot \tilde{y}]E[\tilde{y}^{2}] - E[h \cdot \dot{\tilde{y}}][\tilde{y} \cdot \dot{\tilde{y}}]}{E[\tilde{y}^{2}]E[\tilde{y}^{2}] - E[\tilde{y} \cdot \dot{\tilde{y}}]^{2}}$$

$$C_{eq} = \frac{E[h \cdot \dot{\tilde{y}}]E[\tilde{y}^{2}] - E[h \cdot \tilde{y}][\tilde{y} \cdot \dot{\tilde{y}}]}{E[\tilde{y}^{2}]E[\dot{\tilde{y}}^{2}] - E[\tilde{y} \cdot \dot{\tilde{y}}]^{2}}$$
(6)

(where h is $h(\tilde{y}, \dot{\tilde{y}})$)

$$E[\tilde{y}, h(\tilde{y}, \dot{\tilde{y}})] \quad si \quad E[\tilde{y}, h(\tilde{y}, \dot{\tilde{y}})] \quad \text{can be}$$

expressed depending on $\tilde{y}si\tilde{y}$

 c_{eq} and k_{eq} are functions of the co-variance matrix's elements, noted cov[Y] like:

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$$\operatorname{cov}[Y] = \begin{bmatrix} E[\tilde{y}, \dot{\tilde{y}}] & E[\tilde{y}, \dot{\tilde{y}}] \\ E[\dot{\tilde{y}}, \tilde{y}] & E[\dot{\tilde{y}}, \tilde{\tilde{y}}] \end{bmatrix}$$
(7)
Using equation (3) will be obtain:
$$\dot{Y} = PY + YP^{T} + F$$
(8)
where

$$P = \begin{bmatrix} 0 & 1\\ -\frac{k_{eq}}{m} & -\frac{c_{eq}}{m} \end{bmatrix} ;$$

$$F = \frac{1}{m} \begin{bmatrix} 0 & E[f\tilde{y}]\\ E[f\tilde{y}] & 2E[f\tilde{y}] \end{bmatrix}$$
(9)

The *F* matrix can be simplified, as follows:

$$E[f(t) \cdot f(t+\tau)] = S_0 \delta(\tau) \tag{10}$$

$$F = \frac{I}{m^2} \begin{bmatrix} 0 & 0\\ 0 & S_0 \end{bmatrix}$$
(11)

If the excitation is stationery y=0 the (8) equation becomes:

$$PY + YP^T + F = 0 \tag{12}$$

which is also called the Liapunov's matrix equation.

The parameters c_{eq} and k_{eq} determination is obtained by solving (8) and (6) or (12) and (6) equation systems in stationary conditions.

It is possible to use an iterative method, as follows:

- it is chosen a first values couple c_{eq} and k_{eq} ;
- it is solved the (8) equation obtaining a first approximation for Y;

it is determined a new couple c_{eq} , k_{eq} from (6) equation. The operation will be repeated until a good convergency will be obtained.

4. Conclusions

The actual settles tendencies, concerning a much complete dimensioning than the actual checking criteria are the "reliability" and "degradation level" notions.

exacts The probabilistic methods, or approximate, have been developed to solve some problems of non-linearity.

The techniques of linearity are still in research. The techniques of linearity applied to the systems subjected to socks presume numerical simulation and/or analytically computation.

The nonlinear systems study can be done with enough accuracy, by replacing the nonlinear system with an equivalent linear system, who's characteristics are almost the same as those of the initial system.

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The Stress And Unit Strain Analysis Study In The Part Of Aspiration Pipe Of Hydro Power Plants With Low Head

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Abstract: The paper presents the analysis study of hydro power plants using the programs based on finite elements method. This study about the part of the aspiration pipe and volute chamber, wants to show the stress and unit strain which appears. Our hydro power plants were calculated with structural static methods. The loads which are applied on the structure gives the moments, after which, the dimensions of the power plants result. The aspiration pipes are solicited by a bending moment which comes by the water pressure from upstream. The part of the pressure manifold is subdued to some big stresses which can't be determined by a structural static path. The finite elements method gives this opportunity to establish the effects of water pressure in the infrastructure of power plants.

Keywords: power plant, stress, unit strain, volute chamber, finite elements.

1. Introduction

There are many problems which appears at the hydrotechinc structures, especially to the dams and power plants, seeing the analysis of the structure. The low power plants which have a pressure wall in the upstream of the power plant, are subdued to the water pressure which has effects at the aspiration pipe and foundation raft. The structure is drawing in 3D and is sectioned through the middle of aspiration pipe, for a better visualization. In this paper we wanted to determine the stresses and the unit strains at a power station from our country named Pîngărați. This power plant is a dam type, which take over the water pressure from the upstream. It is equipped with two Kaplan turbines, with the generating station capacity equal with 11.5 MW, for each group. The head of water is 15 m, and the utilizable discharge is 80 m^3/s . The structure is very compact; having the press water conduit built with reinforce concrete. Also is built with reinforce concrete, the volute chamber and the aspiration pipe. The computer program based on the FEM, gives the results of stresses and unit strains and other results depending of our necessity. We'll see only some of these results which are necessary, like σ_x , σ_y and τ_{xy} . The conclusions of the results (which are gives in the graphics) will be shown in the paper.

2. The loads which action on the power plants

To establish the loads it's very difficult, because the geometry of power plants is various. The loads are sorted about their case of loading. There are two importance case of loading: normal exploitation case of loading and extraordinary case of loading. In the normal case of loading, the loads which appear are: the structural weight, the under pressure, wind and others loads which aren't so important. The water pressure which press on the upstream wall is the most important load which we considered at this structure. When the power station is started the loads are like in the picture:



Figure 2.1 The power station's loads

Other loads which action on the structure of power plants are from the hydro mechanic equipment.

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Here we considered the turbine weight and the weight of aggregates.

The power station is equipped with a Kaplan turbine. The weight of the Kaplan aggregates is determinate by a formula:

$$G_{K} = K(A - 0.108)^{0.7} \cdot \left(1.41 \frac{p^{1.1}}{H^{0.95}} - 0.1A^{0.5} \frac{p^{1.35}}{H^{1.325}}\right) + 229(A - 0.108)^{0.75} \frac{p^{1.125}}{H^{0.938}}$$

K=270 for the reinforcement concrete volute chamber.

$$A=0.108+78.6/n_s$$

 n_s =900 which is specific speed.

The generating station capacity is 11.5 MW, but it must be change in power horse (CP). 1KW=1.36 CP, that means 15 MW=15640 CP. Then the aggregates weight:

$$G_{K} = 270(0.195 - 0.108)^{0.7} \cdot \left(1.41 \frac{15640^{1.1}}{15^{0.95}} - 0.10.195^{0.5} \frac{15640^{1.35}}{15^{1.325}}\right) + 229(0.195 - 0.108)^{0.75} \frac{15640^{1.125}}{15^{0.938}} = 339927 kg$$

G_K=339.927 t

We consider G_K on the area which is loaded with the turbines weight. In n this part, the structure is loaded with 5000 kg/m² and 12000 kg/m², depending of the aggregates type.

Another load which presses on the upstream wall is the water pressure, which is considered variable on the height. The water column height is 15 meters and the pressure at this height of water is $p=15mx1000kgf/m^3x1.00m=15000kgf/m$. The structure weight is calculate by the program, where is considered the gravity which is 9.81 m/s. The other loads on the structure are not especially considered, but on the other parts of structure, we applied a constant load. This will more load the structure on each m^2 with the 500 kgf/m².

3. Geometry presentation of the power plants

The first visualization of the structure is a section through the aspiration pipe. This is seeing in the following picture:



Figure 3.1.

It is shows the volumes of the structure in the different colors. The light blue represents the volume of the foundation plate and upstream wall. The blue color is the volume of the volute chamber. The volume which is represented in the magenta color is the support volume for the electric generator. The volume in red color is the bag wall which separates the machine room.

Another view of the structure is a isometric view, what we can see in the next picture:



and uses consistent tangent stiffness for large strain applications.



Figure 4.1

Next picture will shows the upstream wall and his geometry with the catchments of water at the entrance in the power station.



After the construction of the structure geometry, we mesh all the structure in the finite elements.

4. Element Description and structure meshing

The finite elements which we used at this kind of meshing are the elements named SOLID187. This element is a 3-D 10-Node Tetrahedral Structural Solid, which is a higher order 3-D, 10-node element. SOLID187 has a quadratic displacement behavior and is well suited to modeling irregular meshes (such as those produced from various CAD/CAM systems). SOLID187 provides more nonlinear material models Another characteristic of the element SOLID187:

Table 1:	
Element Name	SOLID187
Nodes	I, J, K, L, M, N, O, P, Q, R
Degrees of Freedom	UX, UY, UZ
Real Constants	None
Material Properties	EX, EY, EZ, ALPX, ALPY, ALPZ, (PRXY, PRYZ, PRXZ or NUXY, NUYZ, NUXZ), DENS, GXY, GYZ, GXZ, DAMP
Surface Loads	Pressures - face 1 (J-I-K), face 2 (I-J-L), face 3 (J-K-L), face 4 (K-I-L)

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In the next picture it can be observed the structure's meshing.



Figure 4.2

5. Evaluations and conclusions about the stresses and unit strains

It is well known that in the technical books is described the classical analysis method on the different parts of the structure. We can only remember the analysis of the volute chamber or aspiration pipe. We'll show only few methods in the following lines.

For the volute chamber which is analyzed like structural frames, some schemes are shown in the next picture:







Figure 5.1



For the aspiration pipe there are schemes like in the picture:



The section through the aspiration pipe and it static scheme:



Figure. 5.4



Figure 5.5

It can be established only approximately results from this analysis. With the finite elements method this is simpler. The program which generates the finite elements and analysis will resolve in few minutes.

In the classically method the stresses are calculates by the formula:

$$\sigma = \frac{\sum V}{A} \left(1 \pm \frac{6e}{B} \right), \text{ where }$$

the ΣV is the vertical component of the forces; A- is the area of foundation, B is the width of the foundation on the eccentricity direction (e).

After running, the results of program are shown in a graphical display. If you want to see the results for some nodes or elements you can open a window where are the results in the table, for each node or element. The differences between the values of the stresses are shown in different colors.

In the first picture we'll give the unit strain. This will be seeing together with the unreformed shape. The most obvious unit strain will be observed at the part where the structures support the weight of the turbine. The translation on the X direction and Y direction are shown in the next picture. The translation of the upstream wall on the X direction is about 8 mm.

The σ_x stress in the program is Sx. Program gives this in the next picture:



Figure 5.6

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The σ_y stress in the program is Sy. Program gives this in the next picture:

Figure 5.7

It could be observe the stress of compression which is with the blue color. The Sy stress is Sy=-37.47 daN/cm². This is normally for the stress compression, but in the comparison with the rest of stresses from the structure, those are significant. This means that the foundation plat of the aspiration pipe is not well dimensioned. Is possible to dimension the aspiration's pipe foundation plate better than that, using a bigger quantity of materials, and less material for the rest of elements, or introduing some empty volumes in the structure.

We will give the Sxy stress which is the τ_{xy} stress. In the next picture it will be seeing:



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Theoretical Problems Regarding the Numerical Modeling of Two Buildings Touching

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Abstract: The paper presents the numerical form of two buildings touching with strength structure consisting of reinforced-concrete frames. It has been used an impact model based on a new solving method for the moving equations defined by MEF, using the analytical and discrete transformation Laplace – L.D.D., these transformations being applied for the whole system with "n" degrees of freedom. We are having in view the constitutive rule of reinforced concrete.

Keywords: impact, reinforced concrete, degradation, touching forces.

1. Introduction

The investigation of the strength structure deterioration after being exposed to different loads generated by the extreme physical conditions and phenomenon (for example: earthquake, sliding, the impact of other objects, and so on...) are based on the numerical analysis with MEF. These investigations together with the suitable experimental methods have a great importance in elaborating the prognosis methods of a structure bearing strength.

Traditionally, such searches are making based on linear or/and non-linear breaking mechanical theory, having as a result an answer over the global structure. Usually, this answer is expressed by a calculation between loading and displacement and elaborating the forming and propagation mechanisms for cracks.

Presently, the study elaborated that is based on relations of continuous degradation mechanics- MDC [1] is very used. MDC [1] describes the material behavior on a very detailed volume level. MDC relations are elaborated conforming to the fundamental theories of thermodynamics for irreversible processes, and specially for applying the computational model.

MDC also includes lots of mechanical properties and characteristics up to the latest state of total degradation.

A special advantage about MDC application is that relations can be extended and also, applied without any difficulties to the stiffness degradation

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estimation, both for the frail and ductile materials. Other advantage, of course, is using the computation automate programs, which are elaborated with MEF in elasto-viscoplasticity theory. All these aspects are important elements, which assure the continuity of post-elastic study.

Two first components describe this kind of model (with impact):

- Constitutive equations about the material degradation and the structure in general;
- The touching forces computation (calculation) together with unstable forces, which load the frontier points of the studied zone.
- Solving method of differential moving equations used in numerical solving with FEM.

2. Constitutive equations

Here in, is taking again the main steps to get the calculations for the elastic model in the analysis with MDC for anisotropic frail materials, type concrete. This model is based on the damage velocity of the deformation potential energy, the calculations being proposed by M. Yazdch, S. Walliappan, W. Zhang [2], [3].

The vectors of internal efforts are $y = \langle y_1 y_2 y_3 \rangle^T$, is

defined by:
$$y = \frac{1}{2} \widetilde{\sigma}^T \frac{\partial \widetilde{E}^{*-1}}{\partial D} \widetilde{\sigma} = \frac{1}{2} \widetilde{\varepsilon}_e^T \frac{\partial \widetilde{E}^*}{\partial D} \widetilde{\varepsilon}_e$$
 (1)

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In (1) has been used the next notes:

 $\widetilde{\sigma}, \ \widetilde{\varepsilon}_e$ - the vector of the tensions and the vector of the elastic deformations in the axis system in which the main directions are tension and deformation;

$$\widetilde{E}^{*} = \begin{bmatrix} \frac{E_{1}(l-D_{1})^{2}}{l-\mu_{12}\mu_{21}} & \frac{E_{2}(l-D_{1})(l-D_{2})\mu_{12}}{l-\mu_{12}\mu_{21}} \\ \frac{E_{1}(l-D_{1})(l-D_{2})\mu_{21}}{l-\mu_{12}\mu_{21}} & \frac{E_{2}(l-D_{2})^{2}}{l-\mu_{12}\mu_{21}} \\ 0 & 0 \end{bmatrix}$$

where: E_1 , E_2 , μ_{12} , μ_{21} si G_{12} are mechanical constants for an anisotropic solid.

The components of y vector determines the degradation of the material on each main direction of the tension:

$$y_{i} = \frac{1}{2} \widetilde{\sigma}_{j} E_{jk,i}^{*-l} \sigma_{k} = \frac{1}{2} \widetilde{\varepsilon}_{j}^{e} \widetilde{E}_{jk,i}^{*} \widetilde{\varepsilon}_{k}^{e}; (i = 1, 2, 3)$$
(3)

The value of the sum $\tilde{y} = y_1 + y_2 + y_3$ will be evaluated by the next formula (4):

$$\overline{y} = \frac{1}{2} \sigma_j \left(\sum_{i=1}^3 \widetilde{E}_{jk,i}^{*-l} \right) \sigma_k = \frac{1}{2} \varepsilon_j^e \left(\sum_{i=1}^3 \widetilde{E}_{jk,i}^{*-l} \right) e_k^e$$
(4)

So, the degradation of the deformation potential energy on all main directions is obtaining as:

$$Y = \frac{1}{2} \widetilde{\sigma} \widetilde{C}^* \widetilde{\sigma} \tag{5}$$

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In which has been made the next notes:

$$\widetilde{C}^{*} = \sum_{i=1}^{3} \frac{\partial E^{*-1}}{\partial D_{i}} = \begin{bmatrix} \widetilde{c}_{11}^{*} & \widetilde{c}_{12}^{*} & \widetilde{c}_{13}^{*} & 0 & 0 & 0\\ \widetilde{c}_{21}^{*} & \widetilde{c}_{22}^{*} & \widetilde{c}_{23}^{*} & 0 & 0 & 0\\ \widetilde{c}_{31}^{*} & \widetilde{c}_{32}^{*} & \widetilde{c}_{33}^{*} & 0 & 0 & 0\\ 0 & 0 & 0 & \widetilde{g}_{23}^{*} & 0 & 0\\ 0 & 0 & 0 & 0 & \widetilde{g}_{31}^{*} & 0\\ 0 & 0 & 0 & 0 & 0 & \widetilde{g}_{12}^{*} \end{bmatrix}$$

$$(6)$$

 D_i , i = 1,2,3 - the main values of the tensions tensor corresponding to the damage of the material D; \widetilde{E}^* - the constitutive anisotropic matrix defined in the axis system; in plane the matrix is:

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$$\frac{2G_{I2}(I-D_{I})^{2}(I-D_{2})^{2}}{(I-D_{I})^{2} + (I-D_{2})^{2}} \end{bmatrix}$$

$$\widetilde{c}_{ii}^{*} = \frac{2}{(I-D_{i})^{3}E_{i}}; \quad i \leq 3$$

$$\widetilde{c}_{ij}^{*} = -\frac{(I-D_{i}) + (I-D_{j})\mu_{ij}}{(I-D_{i})^{2}(I-D_{j})^{2}E_{j}}; i \neq j, i \leq 3, \qquad j \leq 3$$
(7)

$$\widetilde{g}_{ij}^{*} = -\frac{(I - D_{i})^{3} + (I - D_{j})^{3}}{(I - D_{i})^{3} (I - D_{j})^{3} G_{ij}}; \quad i \neq j, i \leq 3, \quad j \leq 3$$

The degradation evolution correlated with the experimental data for an anisotropic material, is described by (8):

$$\dot{D}_{i} = \begin{cases} B_{i}Y_{i}^{ki}, & if \quad Y_{i} > Y_{di} \\ 0, & if \quad Y_{i} \le Y_{di} \end{cases} \quad (i = 1, 2, 3)$$
(8)

In (8), Y_{di} the limit value on the degradation direction "i", and k, B_i represent material parameters. In the method for a finite element, the damage accumulated in the finite element is noted with \overline{D} ; the damage at the time $t = t^{i+1}$ is calculating with (9):

$$\overline{D}_i^{j+1} = \overline{D}_i^j + B_i \theta_k (t^j) \Delta t^j \quad , \quad i = 1, 2, 3$$
(9) where:

$$\theta_{k} = \left(t^{j}\right) = \frac{1}{Ae} \sum_{k=1}^{P} \sum_{e=1}^{P} w_{k} w_{e} \left[y_{i}\left(\xi_{k}, \eta_{e}, t^{j}\right)\right]^{k} H\left[y_{i}\left(\xi_{k}, \eta_{e}, t^{j}\right) - y_{di}\right]$$
(10)

On the above formulas have been made the next notes: A_e - the area of the finite element; H - Heaviside function; (ξ_k, η_e) - the integration points at the time t^i from Gauss integration scheme; w_k, w_e - weight functions in points ξ_k and η_e ; p - the number of the points in each main direction of the tension.

3. The mathematical formula

It is taking in consideration the unstable elastodynamic state for a solid which is deformed by $\Omega \in \mathbb{R}^n$ volume into, OXYZ as axis system end limited by the frontier Γ , for non-homogeneous frontier: on part Γ_u – are specified the shifting, on zone Γ_{σ} -tensions and on zone Γ_c – the contact forces (without penetration) with another solid, as: $\Gamma = \Gamma_{\sigma} U \Gamma_u U \Gamma_c$. The result of the contact between these two solids generates a new contact forces system $q_i(t)$, i = 1,2,3 within the time $[t_1, t_2] \in T_c \times [0, T]$, the system being $f_i \in \Gamma_{\sigma} \times [0, T]$, i = 1,2,3. In this case, the material is homogeneous and anisotrop, with the density $\rho \in \mathbb{R}$.

It will be found the vector functions: $u_i, \dot{u}_i, \ddot{u}_i \in \mathbb{R}^n$, i = 1,2,3; $\varepsilon_{ij}, \sigma_{ij}$, i,j = 1,2,3; $\varepsilon_{ij} = \varepsilon_{ji}; \sigma_{ij} = \sigma_{ji}$, which satisfies the next system of equations:

- the equations of dynamic equilibrium under the next form (11)

$$\sigma_{ij,j} + f_i = \rho \ddot{u}_i \in \Omega \times [0,T] ; \qquad (11)$$

- geometrical equations (12)

$$\varepsilon_{ke}(u) = \frac{1}{2} \left(u_{k,e} + u_{e,k} \right) \in \Omega \times [0,T] ; \qquad (12)$$

- constitutive equations (3)

$$\sigma_{ij} = k_{ijke}^* \varepsilon_{ke} \in \Omega \times [0, T];$$
(13)
- with frontier conditions expressed in tensions:

$$\sigma_{ij}n_{j} = \begin{cases} f_{i} \in \Gamma_{\sigma} \times [0, t_{i}] \\ f_{i} + q_{i} \in (\Gamma_{\sigma} U \Gamma_{c}) \times [t_{1}, t_{2}] \\ f_{i} \in \Gamma_{\sigma} \times [t_{2}, T] \end{cases}$$
(14)

- with frontier conditions expressed in displacements:

$$u_i = 0 \in \Gamma_u \times [0, T]$$
(15)
- initial conditions:

$$u_i(x,0) = u_i^0 \quad x \in \Omega$$

$$\dot{u}_i(x,0) = \dot{u}_i^0 \quad x \in \Omega$$
(16)

Contact condition (constraint) without penetration of the solids is under the next form (17):

$$\begin{bmatrix} t_{ij} \end{bmatrix}^T - \delta_i \ge 0 \in \Gamma_c \times \begin{bmatrix} 0, T \end{bmatrix}$$
(17)

Constitutive equations (13) are completed by the equations forms from (6) to (10) above deduced. There are used the next notations:

- $|t_{ii}|$ - the constraint matrix after the contact;

- *n* - perpendicular vector into a specific point of the frontier $\Gamma_{\dot{\sigma}}$.

4. LD*D* – The solving method for the moving equations defined by MEF. Elastic case.

In specialty literature, a particular attention is given to the direct integration methods of the moving equations. So, there are known lots of such methods, as: Newman, Wilson methods. In these methods, the moving equation for a dynamic system with one degree of freedom is integrated, based on some approximations of velocities and accelerations. This idea is developed in many schemes, in which are additionally introduced control parameters regarding the dissipation and numerical stability of the algorithm.

The elaboration and analyze of such methods is lately presented in many publications.

In LDD method, the amortized character is not a global one.

There are known some methods of high precision that develops the idea about direct integration of the whole system with "n" degrees of freedom.

Thus, in public works [4], [5] is presented a new method of obtaining direct integration schemes for the whole system with "n" equations, under an explicit form.

On the base of LDD Laplace transformations and network-function theory [6] is obtaining a new equivalent of differences, written as integral calculus. The advantage of this new approaching way is obvious.



Fig. 1. The representation of a continuous function through a network function

In the above graphic is presented a continuous arbitrary function with a network function. The values of network function in discret points are equal with the corresponding discret values for the continuous function. With LDD machine, it's making transformations to the differential moving equations system, under a matrix form (18), these transformations being used in numerical solving with MEF:

$$M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = F(t)$$
(18)
with initial conditions:

 $u(0) = u^0$; $\dot{u}(0) = \dot{u}_0$ The result is a 2 integral equations system with difference (19) and (20). The system is equivalent with the next equations:

$$M \{ u[n+2,\varepsilon] - 2u[n+1,\varepsilon] + u[n,\varepsilon] \} + \Delta t C \int_{0}^{l} \{ u[n+2,\lambda] - u[n+1,\lambda] \} d\lambda + \Delta t^{2} K \int_{0}^{l} \{ (\varepsilon - \lambda)[u[n+2,\lambda] - u[n+1,\lambda]] + u[n+1,\lambda] \} d\lambda = \Delta t^{2} \int_{0}^{l} \{ (\varepsilon - \lambda)[f[n+2,\lambda] - f[n+1,\lambda]] + f[n+1,\lambda] \} d\lambda$$
(19)

$$M \{ u[n+2,\varepsilon] - 2u[n+1,\varepsilon] + u[n,\varepsilon] \} + \Delta t C \int_{0}^{l} \{ u[n+2,\varepsilon-\lambda] - u[n+1,\varepsilon-\lambda] \} d\lambda + \Delta t^{2} K \int_{0}^{l} \{ u[n+1,\varepsilon-\lambda] + \lambda[u[n+2,\varepsilon-\lambda] - u[n+1,\varepsilon-\lambda]] \} d\lambda = \Delta t^{2} \int_{0}^{l} \{ f[n+1,\varepsilon-\lambda] + \lambda[f[n+2,\varepsilon-\lambda] - u[n+1,\varepsilon-\lambda]] \} d\lambda$$
(20)

If \mathcal{E} parameter vary between 0 and 1, the forms (19) and (20) appear as (21) and (22) forms.

$$M\{u[n+2] - 2u[n+1] + u[n]\} + \Delta t C \int_{0}^{1} \{u[n+1,\lambda] - u[n,\lambda]\} d\lambda + \Delta t^{2} K \int_{0}^{1} \{\lambda u[n+1,l-\lambda] - (l-\lambda)u[n,l-\lambda]\} d\lambda = \Delta t^{2} \int_{0}^{1} \{\lambda f[n+1,l-\lambda] + (l-\lambda)f[n,l-\lambda]\} d\lambda$$

$$M\{u[n+2] - 2u[n+1] + u[n]\} + \Delta t C \int_{0}^{1} \{u[n+1,\lambda] - u[n,\lambda]\} d\lambda$$
(21)

$$M \{u[n+2] - 2u[n+1] + u[n]\} + \Delta t C \int_{0}^{1} \{u[n+1,\lambda] - u[n,\lambda]\} d\lambda + \Delta t^{2} K \int_{0}^{1} \{\lambda u[n,\lambda] + (1-\lambda)u[n+1,\lambda]\} d\lambda = \Delta t^{2} \int_{0}^{1} \{\lambda f[n,\lambda] + (1-\lambda)f[n+1,\lambda]\} d\lambda$$

$$(22)$$

It easy to check that if the dynamic action f(t) is a polynomial form then u(t) solution can be written as an exponential form $f(t) = de^{i\omega t}$ which has the equivalent into a network function as (23) form $f[n,\varepsilon] = de^{i\tilde{\omega}(n+\varepsilon)}$ (23)

$$f[n,\varepsilon] = de^{i\omega(n+\varepsilon)}$$

where, $\widetilde{\omega} = \omega \Delta t$ and $u(t) = v e^{i\omega t}$.

Using equivalent $u[n,0] = ve^{i\tilde{\omega}t}$ the general formula

$$2M + \Delta tC = 2(2M - \Delta t^2 K)u_{n+1} + (\Delta tC - 2M)u_n + 2\Delta t^2 f_{n+1}$$
(25)
With Simpson formula in 3 points,

$$\int_{0}^{1} f(t)dt \approx \frac{1}{6} \left[f(0) + 4f\left(\frac{1}{2}\right) + f(1) \right]$$
(26)

results the explicit formula (27):

$$\frac{l}{2}(6M + \Delta tC)u_{n+2} = -\Delta t(2C + \Delta tK)u_{n+\frac{3}{2}} + (6M - \Delta t^{2}K)u_{n+1} + \Delta t(2C - \Delta tK)u_{n+\frac{1}{2}} + \frac{l}{2}(\Delta tC - 6M)u_{n} + \Delta t^{2}\left(f_{n+\frac{3}{2}} + f_{n+1} + f_{n+\frac{1}{2}}\right)$$
(27)

Similar, using other numerical integration formulas, it can be obtained high precision schemes. One of them is Gauss-Legendre scheme in which is optimized the time step and its complanation coefficients.

On the base of this scheme there is representing the integral of excitator function (28).

$$\int_{0}^{b} f(t) dt \approx \sum_{i=1}^{m} \alpha_{i} f(t_{i})$$
(28)

A presentation like this is used in approximating the acceleration on earthquake machines.

The multipliers that represent weight force function α_i (*i*=1,2,...,m) and nodes in integration points, at time t_i for the time period [a,b], are calculated with Legendre polynomials.

Into the impact model represented above, is being used the Simpson's explicit integration scheme (26). For (26) is making the analyze of numeric stability of moving equation (29).

$$m\ddot{u} + 2\xi\omega\dot{u} + \omega^2 u = f \tag{29}$$

The characteristic of stability for this method is deduced $\rho \sim \frac{\Delta_t}{T}$, then is compared with the similar

characteristic used in central difference method (Bathe and Wilson method). For $\xi = 0$ and $m = \frac{1}{3}$,

of eigen values is obtained, which reveals the

With well-known formula (24) of trapezium

equivalent necessarily and sufficiently.

results (25), the accurate scheme for central

integration in 2 points:

difference method

 $\int_0^l f(t)dt \approx \frac{l}{2} [f(0) + f(1)]$

the Simpson scheme for LDD method is (30).

$$u_{t+4\Delta t} = -(\Delta t)^2 \omega^2 u_{t+3\Delta t} + \frac{1}{2} (l - 2(\Delta t)^2 \omega^2) u_{u+2\Delta t} + (\Delta t)^2 \omega^2 u_{t+\Delta t} - \frac{1}{4} u_t + \Delta t^2 (f_{t+3\Delta t} + f_{t+2\Delta t} + f_{t+\Delta t})$$
(30)

As matrix:

$$\begin{pmatrix} u_{t+4\Delta t} \\ u_{t+3\Delta t} \\ u_{t+2\Delta t} \\ u_{t+\Delta t} \end{pmatrix} = A \begin{pmatrix} u_{t+3\Delta t} \\ u_{t+2\Delta t} \\ u_{t+\Delta t} \\ u_{t} \end{pmatrix} + L \begin{pmatrix} f_{t+3\Delta t} \\ f_{t+2\Delta t} \\ f_{t+\Delta t} \\ f_{t} \end{pmatrix}$$
(31)

$$A = \begin{pmatrix} -(\Delta t)^2 \omega^2 & \frac{1}{2} (1 - 2(\Delta t)^2 \omega^2) & -(\Delta t)^2 \omega^2 & -\frac{1}{4} \\ 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 \\ 0 & 0 & 1 & 0 \end{pmatrix} (32)$$

(24)

$$\Lambda_4 = \lambda^4 + (\Delta t)^2 \,\omega^2 \,\lambda^3 - \frac{1}{2} \left(I - 2(\Delta t)^2 \,\omega^2 \right) \lambda^2 + (\Delta t)^2 \,\omega^2 \,\lambda^2$$

After calculation, the value $\rho(\Lambda_4) = \frac{I}{\sqrt{2}}$ is lower than that calculated using the central difference

scheme. Comparing the spectral ray variation $\rho(A) \sim \frac{\Delta_t}{T}$ using LDD method with the central difference method, it's observing that optimal step within $10^{-1.6} T < \Delta t < 10^{-0.85} T$ period. varies When $\Delta t < 10^{-1.6}$, the value for spectral ray is insignificantly lower.

5. Conclusions

with high earthquakes, In areas the dimensioning of seismic gaps between buildings is very important so that to be avoided the degradations of the structural and non-structural elements of the buildings. Also, the dimensioning is important for installations and equipment protection inside the buildings that are touching. The touching between 2 buildings appears when, due to different dynamic characteristics, the structures do not synchronous oscillates and the space between them is not sufficiently enough to produce the free displacements. So, it is necessarily to study the structure behavior to the seismic action into the non-linear field of material behavior, so that to get the answer in displacements as accurate as possible. The proper presents the behavior rules for reinforced concrete in the non-linear field and a

It is calculated $\rho(A)$ - spectral ray of operator A, where $\rho(A) = \max \lambda_i$ (i = 1, ..., 4). Similar, the values for (34) are calculated.

$$(34)$$

new method for solving the dynamic system of moving equations. The biggest advantage of the proposed method consists in reducing the calculation time, which in the study of buildings touching takes more time

Thus, the "zones" of the buildings that are touching can be break in very small parts through a network consisting in finite elements. The finite elements mesh is more dense and is useful in studying the local effects that appear in such cases.

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Three Degrees-of-Freedom Model for Instability Coupled Galloping of Iced Conductors

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Abstract: The galloping response of a three-degrees-of freedom system is investigated using perturbation techniques to generate approximate steady-state solutions. Interactions are accommodated between a system's plunge and swing in the along-wind direction and a rotation around the elastic axis. It is considered that a realistic cross-section has eccentricity (its center of mass and elastic axis do not coincide). So, we found a set of non - dimensional equations of motion more complex that those obtained by Blevins and Iwan in concentric case. The Multiple-Time-Scales Method and unification techniques is applied next to obtain the rate equations, from which steady-state (periodic and quasi-periodic) solutions and than stability conditions can be derived.

Keywords: Galloping, three-degrees-of-freedom model, perturbation method.

1. Introduction

Galloping is a low -frequency, high-amplitude oscillation that can occur on an iced electrical transmission line in a steady-side wind. The main feature of this phenomenon is a motion in a single uncoupled mode of vibration in the direction normal to that of the wind, which makes it different from flutter containing coupled modes of vibration. Oscillations of the galloping type are caused by the aerodynamic instability of the cross-section of the body, so that the motion generates forces which increases its amplitudes.

For the theoretical study of this phenomenon has been proposed, in time, a great number of models. This paper is concerned with the galloping of iced conductors modeled as a three-degrees-of-freedom system.

2. Description of model and equations of motion

The basic three-degrees-of-freedom model proposed for galloping's study is illustrated in Figure 1. It consists in a one-dimensional body having a cross-section of arbitrary shape, moving with horizontal and vertical velocities z and y, respectively, in a horizontal wind field V_{ro} and

rotating around the elastic axis with angular velocity

 θ . The dot denotes derivative with respect to time.

The k_y, k_z and k_θ represent the stiffness in y, z and θ directions, respectively, c_y, c_z, c_θ are the corresponding viscous dampers (which are not shown in Fig. 1). It is assumed that a realistic cross-section has the eccentricity e; that is, its center of mass and elastic axis do not coincide.



`Fig. 1 : The three-degrees-of-freedom model

During the motion, between the coordinates of center of mass C and center of rotation O there are the relations

$$z_C = z + e \cos \theta \cong z + e$$
, $y_C = y - e \sin \theta \cong y - e \theta$ (1)

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so the equations of motion are

$$\begin{array}{l}
\overset{\cdots}{mz} + c_{z} z + k_{z} z = F_{z} \\
\overset{\cdots}{my} - S \overleftrightarrow{\theta} + c_{y} y + k_{y} y = F_{y} \\
\overset{\cdots}{J_{0} \overleftrightarrow{\theta}} - S \overset{\cdots}{y} + c_{\theta} \overleftrightarrow{\theta} + k_{\theta} \overleftrightarrow{\theta} = M
\end{array}$$
(2)

where m, S and J_O represent the mass, the static moment and the moment of inertia of the iced conductor's cross-section on unit length.

$$F_{y} = \frac{1}{2} \rho_{a} V_{rel}^{2} dC_{y}(\alpha), F_{z} = \frac{1}{2} \rho_{a} V_{rel}^{2} dC_{z}(\alpha)$$
$$M = \frac{1}{2} \rho_{a} V_{rel}^{2} d^{2} C_{M}(\alpha)$$
(3)

are the aerodynamic forces and moment acting on unit length of cross-section. In deriving Eqs. (1) is considered that angle θ is small. Aerodynamic coefficients C_y, C_z, C_M are continuous functions of angle of attack α and may be expressed as experimentally determined polynomials in this variable. In terms of the true wind and the body velocities, the relative wind V_{rel} and angle of attack α is given by

$$V_{rel} = \sqrt{\left(V_{\infty} - \dot{z} + R\dot{\theta}\cos\gamma\right)^{2} + \left(\dot{y} + R\dot{\theta}\sin\gamma\right)^{2}}$$
$$tg(\theta - \alpha) = \frac{\dot{y} + R\dot{\theta}\sin\gamma}{V_{\infty} - \dot{z} + R\dot{\theta}\cos\gamma}$$
(4)

where $R_1 = R \sin \gamma \cong d/2$ is a characteristic radius of the section. If $y, z \ll V_{\infty}$, we find that

$$V_{rel} \cong V_{\infty} , \quad \alpha \cong \theta - \frac{\dot{y}}{V_{\infty}} - R_1 \frac{\dot{\theta}}{V_{\infty}}$$
 (5)

The representation of C_y , C_z and C_M gives

rise to the nature of the non -linearity in the system. A simple cubic approximation was shown to capture the main features of the galloping [4]. Consequently, a cubic polynomial will be used so the equations of motion became

$$\ddot{z} + \omega_{z}^{2} z = -\frac{c_{z}}{m} \dot{z} + \frac{\rho_{a} V_{\infty}^{2} d}{2m} \left(a_{1} \alpha + a_{2} \alpha^{2} + a_{3} \alpha^{3} \right)$$
$$\ddot{y} - e \ddot{\theta} + \omega_{y}^{2} y = -\frac{c_{y}}{m} \dot{y} + \frac{\rho_{a} V_{\infty}^{2} d}{2m} \left(b_{1} \alpha + b_{2} \alpha^{2} + b_{3} \alpha^{2} \right)$$
$$\ddot{\theta} - \frac{m e}{J_{0}} \ddot{y} + \omega_{\theta}^{2} \theta = -\frac{c_{\theta}}{m} \dot{\theta} + \frac{\rho_{a} V_{\infty}^{2} d^{2}}{2m} \left(c_{1} \alpha + c_{2} \alpha^{2} + c_{3} \alpha^{3} \right)$$
(6)

where

$$\omega_z = \sqrt{\frac{k_z}{m}}, \quad \omega_y = \sqrt{\frac{k_y}{m}}, \quad \omega_\theta = \sqrt{\frac{k_\theta}{J_0}}$$
 (7)

The coupling between equations of motion are caused by eccentricity e as well as by the angle of attack . The non-dimensional form for system (2) are

$$\frac{d^{2}\overline{z}}{d\tau^{2}} + p_{1}^{2} - 2\xi_{z}\frac{d\overline{z}}{d\tau} + \eta_{yz}U^{2}\left(a_{1}\alpha^{2} + a_{2}\alpha^{2} + a_{3}\alpha^{3}\right)$$

$$\frac{d^{2}\overline{y}}{d\tau^{2}} - e\frac{d^{2}\theta}{d\tau^{2}} + \overline{y} = -2\xi_{y}\frac{d\overline{y}}{d\tau} +$$

$$+\eta_{yz}U^{2}\left(b_{1}\alpha + b_{2}\alpha^{2} + b_{3}\alpha^{3}\right)$$

$$\frac{d^{2}\theta}{d\tau^{2}} - \overline{e}\eta\frac{d^{2}\overline{y}}{d\tau^{2}} + p_{2}^{2}\theta = -2\xi_{\theta}\frac{d\theta}{d\tau} +$$

$$+\eta_{\theta}U^{2}\left(c_{1}\alpha + c_{2}\alpha^{2} + c_{3}\alpha^{3}\right)$$
(8)

where

$$\alpha = \theta - \frac{1}{U} \frac{d\bar{y}}{d\tau} - \frac{r_1}{U} \frac{d\theta}{d\tau}$$
(9)

and

$$\begin{aligned} \overline{z} &= \frac{z}{d}, \ \overline{y} = \frac{y}{d}, \ \xi_z = \frac{c_z}{2m\omega_y}, \ \xi_y = \frac{c_y}{2m\omega_y}, \\ \xi_\theta &= \frac{c_\theta}{2J_\theta\omega_y}, U = \frac{V_\infty}{\omega_y d}, \\ \eta_{yz} &= \frac{\rho_a d^2}{2m}, \\ \eta_\theta &= \frac{\rho_a d^4}{2J_0}, \\ \eta &= \frac{\eta_\theta}{\eta_{yz}}, \\ p_1 &= \frac{\omega_z}{\omega_y}, \\ p_2 &= \frac{\omega_\theta}{\omega_y}, \\ \tau &= \omega_y t, \\ r_1 &= \frac{R_1}{d} \end{aligned}$$
(10)

3. Deriving an equivalent system of equations of motion by means of principal coordinates

Suppose the system is initially in stable equilibrium but a parameter changes so that the initial equilibrium became unstable. The aim here is to investigate which subsequent periodic and quasiperiodic motions are possible when the damping and aerodynamic forces (in the right-side of Eqs. (8)) are small compared to the forces arising from the conductor's stiffness and inertia (in the left-side of Eqs. (8)). The terms in the right - side of Eqs. (8) are of order ε ($\varepsilon <<1$ is a positive small parameter). $\varepsilon = 0$ denotes a critical point of system (8) .In this case, the differential linear system is

$$\frac{d^2 \overline{z}}{d\tau^2} + p_1^2 \overline{z} = 0$$

$$\frac{d^2 \overline{y}}{d\tau^2} - \overline{e} \frac{d^2 \theta}{d\tau^2} + \overline{y} = 0$$

$$\frac{d^2 \theta}{d\tau^2} - \overline{e} \eta \frac{d^2 \overline{y}}{d\tau^2} + p_2^2 \theta = 0$$
(11)

Choosing the following solutions for system (11)

$$\overline{z} = A_z \sin(\omega \tau + \varphi), \ \overline{y} = A_y \sin(\omega \tau + \varphi),$$

$$\theta = A_\theta \sin(\omega \tau + \varphi)$$
(12)

we obtain the linear system

$$A_{z}\left(p_{1}^{2}-\omega^{2}\right)=0, A_{y}\left(1-\omega^{2}\right)+\bar{e}\omega^{2}A_{\theta}=0,$$

$$\bar{e}\eta\omega^{2}A_{y}+\left(p_{2}^{2}-\omega^{2}\right)A_{\theta}=0$$
 (13)

The characteristic polynomials of the homogeneous system (13) are

$$\left(p_{1}^{2}-\omega^{2}\right)\left[\left(1-\bar{e}^{2}\eta\right)\omega^{4}-\left(1+p_{2}^{2}\right)\omega^{2}+p_{2}^{2}\right]=0$$
 (14)

and has the solutions

$$\omega_{1C}^{2} = p_{1}^{2}$$

$$\omega_{2C,3C}^{2} = \frac{1 + p_{2}^{2} \pm \sqrt{\left(1 + p_{2}^{2}\right)^{2} - 4p_{2}^{2}\left(1 - e^{-2}\eta\right)}}{2\left(1 - e^{-2}\eta\right)} \quad (15)$$

Now, the linear transformation

$$\begin{pmatrix} \overline{z} \\ \frac{dz}{d\tau} \\ \overline{y} \\ \frac{dy}{d\tau} \\ \frac{d\theta}{d\tau} \\ \frac{d\theta}{d\tau} \end{pmatrix} = \begin{pmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 1 & 0 \\ 0 & 0 & 0 & \omega_{2C} & 0 & \omega_{3C} \\ 0 & 0 & \mu_2 & 0 & \mu_3 & 0 \\ 0 & 0 & 0 & \mu_2 \omega_{2C} & 0 & \mu_3 \omega_{3C} \end{pmatrix} \begin{pmatrix} z_1 \\ z_2 \\ z_3 \\ z_4 \\ z_5 \\ z_6 \end{pmatrix}$$
(16)

where z_1, z_3 and z_5 are the principal coordinates and $\mu_{2,3}$ the distribution coefficients

$$\mu_{2} = \frac{(A_{\theta})_{2}}{(A_{y})_{2}} = \frac{\omega_{2C}^{2} - 1}{\bar{e} \, \omega_{2C}^{2}} = \frac{\omega_{2C}^{2} \bar{e} \, \eta}{\omega_{2C}^{2} - p_{2}^{2}}$$
$$\mu_{3} = \frac{(A_{\theta})_{3}}{(A_{y})_{3}} = \frac{\omega_{3C}^{2} - 1}{\bar{e} \, \omega_{3C}^{2}} = \frac{\omega_{3C}^{2} \bar{e} \, \eta}{\omega_{3C}^{2} - p_{3}^{2}}$$
(17)

transform the system (8) in the equivalent system :

$$\frac{d}{d\tau} \{z\} = [L] \cdot \{z\} + \{N_1\} + \{N_2\}$$
(18)

in which $\{z\}^T = (z_1 \ z_2 \ z_3 \ z_4 \ z_5 \ z_6)$. The [L] is a 6x6 matrix associated with linear terms and $\{N_1\}, \{N_2\}$ are 6x1 vectors whose non-vanishing components are given in Appendix.

4. Determining the periodic and quasi-periodic solutions by means of the Multiple-Time-Scales Method

The autonomous system (18) is weakly nonlinear so we are used the Multiple-Time-Scales-Method [3] for derive the bifurcation equations governing the motion. The two scales are used, namely the normal time $t_1 = \tau$ and the slow time

From Eqs. (18), (19) and (20) and equating coefficients of ε^0 , we obtain the differential equation that permit us to found the first-order approximation

$$\frac{\partial}{\partial t_1} \{ Z_0 \} = \begin{pmatrix} 0 & 1 & 0 & 0 & 0 & 0 \\ -p_1^2 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & \omega_{2C} & 0 & 0 \\ 0 & 0 & -\omega_{2c} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & \omega_{3c} \\ 0 & 0 & 0 & 0 & -\omega_{3c} & 0 \end{pmatrix}$$
$$\cdot \{ Z_0 \} \stackrel{not}{=} [J] \cdot \{ Z_0 \} \qquad (21)$$

The solution (on components) of this equation is

$$Z_{10}(t_{1},t_{2}) = T_{10}(t_{2})\exp(ip_{1}t_{1}) + \overline{T}_{10}(t_{2})\exp(-ip_{1}t_{1})$$

$$Z_{20}(t_{1},t_{2}) = ip_{1}(T_{10}(t_{2})\exp(ip_{1}t_{1}) - \overline{T}_{10}(t_{2})\exp(-ip_{1}t_{1}))$$

$$Z_{30}(t_{1},t_{2}) = T_{30}(t_{2})\exp(i\omega_{2C}t_{1}) + \overline{T}_{30}(t_{2})\exp(-i\omega_{2C}t_{1})$$

$$Z_{40}(t_{1},t_{2}) = i\omega_{2C}(T_{30}(t_{2})\exp(i\omega_{2C}t_{1}) - \overline{T}_{30}(t_{2})\exp(-i\omega_{2C}t_{1}))$$

$$Z_{50}(t_{1},t_{2}) = T_{50}(t_{2})\exp(i\omega_{3C}t_{1}) + \overline{T}_{50}(t_{2})\exp(-i\omega_{3C}t_{1})$$

$$Z_{60}(t_{1},t_{2}) = i\omega_{3C}(T_{50}(t_{2})\exp(i\omega_{3C}t_{1}) - \overline{T}_{50}(t_{2})\exp(-i\omega_{3C}t_{1}))$$
(22)

Additionally, by equating coefficients of ε^1 , we obtain

 $t_2 = \varepsilon \tau$, so that the obtained solutions are valid at time scale $\tau = O(\varepsilon^2)$. Letting

$$\xi_{y,z,\theta} = \varepsilon \cdot \hat{\xi}_{y,z,\theta}, (a_i, b_i, c_i) = \varepsilon \cdot (\hat{a}_i, \hat{b}_i, \hat{c}_i) \quad (19)$$

we consider the following solutions for system (18): $\{z\} = \{Z_0\} + \varepsilon \{Z_1\} + \varepsilon^2 \{Z_2\} + \dots$ (20) where $\{Z_0\} = (Z_{10} \ Z_{20} \ Z_{30} \ Z_{40} \ Z_{50} \ Z_{60}), \text{etc.}$

$$\frac{\partial}{\partial t_1} \{Z_1\} + \frac{\partial}{\partial t_2} \{Z_0\} = [J] \cdot \{Z_1\} + (\hat{L}] - [J]) \cdot \{Z_0\} + \left\{ \hat{N}_1^0 \right\} + \left\{ \hat{N}_2^0 \right\}$$

where non vanishing elements of matrix $[\hat{L}]$ and vectors $\{\hat{N}_{1}^{0}\}, \{\hat{N}_{2}^{0}\}$ are found by replacing values $\xi_{z,y,\theta}$ and $a_{i}, b_{i}, c_{i}, i = \overline{1,3}$, by $\hat{\xi}_{z,y,\theta}$ and $\hat{a}_{i}, \hat{b}_{i}, \hat{c}_{i}, i = \overline{1,3}$, respectively. Furthermore, in vectors $\{N_{1}\}$ and $\{N_{2}\}$ angle α will replace by

$$\alpha_{0} = \mu_{2} Z_{30} - \frac{\omega_{2C}}{U} (1 + r_{1} \mu_{2}) Z_{40} + \mu_{3} Z_{50} - \frac{\omega_{3C}}{U} (1 + r_{1} \mu_{3}) Z_{60}$$
(24)

Neresonant case

$$\left(\frac{p_1}{\omega_{2C}} \neq \frac{m}{n}, \frac{p_1}{\omega_{2C}} \neq \frac{m}{p}, \frac{\omega_{2C}}{\omega_{3C}} \neq \frac{n}{p}, m.n.p \in N\right)$$

By using the first two relations (22) and (23) and eliminating the secular terms on obtains

$$T_{10}(t_{2}) = \frac{1}{2}A_{1} \exp\left(-\hat{\xi}_{z}t_{2} + i\psi_{1}\right),$$

$$A_{1},\psi_{1} = const$$
(25)

From next four relations (22) and (23) and order differential coupled system : vanishing the secular terms, on founds the next first

$$\rho_{3}^{'} = \frac{m_{1}\omega_{2C}}{2}\rho_{3} - \frac{3m_{2}\omega_{2C}^{2}(1+r_{1}\mu_{2})}{4U} \left\{ \frac{1}{2} \left[\left(\frac{\omega_{2C}^{2}}{U} \right)^{2} (1+r_{1}\mu_{2})^{2} + \mu_{2}^{2} \right] \rho_{3}^{2} + \left[\left(\frac{\omega_{3C}^{2}}{U} \right)^{2} (1+r_{1}\mu_{3})^{2} + \mu_{3}^{2} \right] \rho_{5}^{2} \right\} \rho_{3}$$

$$\rho_{5}^{'} = \frac{m_{3}\omega_{3C}}{2}\rho_{5} - \frac{3m_{4}\omega_{3C}^{2}(1+r_{1}\mu_{3})}{4U} \left\{ \frac{1}{2} \left[\left(\frac{\omega_{3C}^{2}}{U} \right)^{2} (1+r_{1}\mu_{3})^{2} + \mu_{3}^{2} \right] \rho_{5}^{2} + \left[\left(\frac{\omega_{2C}}{U} \right)^{2} (1+r_{1}\mu_{2})^{2} + \mu_{2}^{2} \right] \rho_{3}^{2} \right\} \rho_{5}$$

$$\phi_{3}^{'} = -\frac{m_{5}}{2} - \frac{3\mu_{2}m_{2}}{4} \left\{ \frac{1}{2} \left[\left(\frac{\omega_{2C}^{2}}{U} \right)^{2} (1+r_{1}\mu_{2})^{2} + \mu_{2}^{2} \right] \rho_{3}^{2} + \left[\left(\frac{\omega_{2C}\omega_{3C}}{U} \right)^{2} (1+r_{1}\mu_{3})^{2} + \mu_{3}^{2} \right] \rho_{5}^{2} \right\}$$

$$\phi_{5}^{'} = -\frac{m_{6}}{2} - \frac{3\mu_{3}m_{4}}{4} \left\{ \frac{1}{2} \left[\left(\frac{\omega_{3C}^{2}}{U} \right)^{2} (1+r_{1}\mu_{3})^{2} + \mu_{3}^{2} \right] \rho_{5}^{2} + \left[\left(\frac{\omega_{2C}\omega_{3C}}{U} \right)^{2} (1+r_{1}\mu_{2})^{2} + \mu_{2}^{2} \right] \rho_{3}^{2} \right\}$$

$$(26)$$

where

$$m_{1} = \hat{L}_{44}, m_{2} = \frac{\hat{N}_{24}^{0}}{\alpha_{0}^{3}}, m_{3} = \hat{L}_{66}, m_{4} = \frac{\hat{N}_{26}^{0}}{\alpha_{0}^{3}}, m_{5} = \hat{L}_{43} - J_{43}, m_{6} = \hat{L}_{65} - J_{65}$$
(27)

In deriving Eqs. (26) we are used the polar forms :

$$T_{30}(t_2) = \frac{1}{2} \rho_3(t_2) \exp(i\phi_3(t_2)) , T_{50}(t_2) = \frac{1}{2} \rho_5(t_2) \exp(i\phi_5(t_2))$$
(28)

The steady state amplitudes and the stability of the steady state solutions may be determined from Eqs. (26). Using (20), (22), (25) and (28) it is possible to go back to the normal non-dimensional time τ by combining them and to determine the first-order approximation for the solution of equations of motion:

$$\overline{z}(\tau) = z_{1}(\tau) = A_{1} \exp(-\xi_{z} \tau) \cos(p_{1}\tau + \psi_{1}), A_{1}, \psi_{1} \text{ constants}$$

$$\overline{y}(\tau) = z_{3}(\tau) + z_{5}(\tau) = \rho_{3}(\tau) \cos(\omega_{2C} \tau + \phi_{3}(\tau)) + \rho_{5}(\tau) \cos(\omega_{3C} \tau + \phi_{5}(\tau))$$

$$\theta(\tau) = \mu_{2} z_{3}(\tau) + \mu_{3} z_{5}(\tau) = \mu_{2} \rho_{3}(\tau) \cos(\omega_{2C} \tau + \phi_{3}(\tau)) + \mu_{3} \rho_{5}(\tau) \cos(\omega_{3C} \tau + \phi_{5}(\tau))$$
(29)

5. Conclusion

A considerably general study is given for the galloping of an eccentrically iced conductor. It has been found that such a conductor may exhibit more

complicated dynamic behaviour than that which is possible in the cocentric case.

Bifurcation theory leads us to sets of governing equations from the initial equilibrium solution and, consenquently, to explicit asymptotic

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solutions not only for th1e periodic solutions but also for the nonresonant, quasi-periodic motions.

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Appendix

The non-vanishing coefficients used in Eqs. (18) are

$$\begin{split} &L_{12} = 1, L_{21} = -p_{1}^{2}, L_{22} = -2\xi_{z}, L_{23} = \eta_{yz}U^{2}\mu_{2}a_{1}, \\ &L_{24} = -\eta_{yz}Ua_{1}\omega_{2C}(1+\mu_{2}r_{1}), L_{25} = \eta_{yz}U^{2}a_{1}\mu_{3}, \\ &L_{26} = -\eta_{yz}Ua_{1}\omega_{3C}(1+\mu_{3}r_{1}), L_{34} = \omega_{2C}, \\ &L_{43} = -\omega_{2C} + \frac{\omega_{2C}U^{2}\mu_{2}\eta_{yz}}{\mu_{3}-\mu_{2}} \bigg(\mu_{3}b_{1} - \frac{\eta}{p_{1}^{2}}c_{1}\bigg), \\ &L_{44} = \frac{\omega_{2C}^{2}}{\mu_{3}-\mu_{2}} [-2\mu_{3}\xi_{y} + \frac{2\mu_{2}\xi_{\theta}}{p_{2}^{2}} + U\eta_{yz} \cdot \\ &\cdot (1+\mu_{2}r_{1})\bigg(\frac{\eta}{p_{2}^{2}}c_{1} - \mu_{3}b_{1}\bigg)] \\ &\alpha = \mu_{2}z_{3} - \frac{\omega_{2C}}{U}(1+r_{1}\mu_{2})z_{4} + \mu_{3}z_{5} - \\ &- \frac{\omega_{2C}}{U}(1+r_{1}\mu_{3})z_{6} \end{split}$$

 $L_{46} = \frac{\omega_{2C}\omega_{3C}}{\mu_{2} - \mu_{2}} \left[-2\mu_{3}\xi_{y} + \frac{2\mu_{3}\xi_{\theta}}{n_{2}^{2}} + U\eta_{yz} + U\eta_{yz}$ $\cdot (1 + \mu_3 r_1) \left(\frac{\eta}{n^2} c_1 - \mu_3 b_1 \right)], L_{56} = \omega_{3C},$ $L_{63} = \frac{\omega_{3C}}{\mu_3 - \mu_2} \mu_2 U^2 \eta_{yz} \left(\frac{\eta}{n_2^2} c_1 - \mu_2 b_1 \right),$ $L_{64} = \frac{\omega_{2C}\omega_{3C}}{\mu_{3} - \mu_{2}} [2\mu_{3}\xi_{y} - \frac{2\mu_{3}\xi_{\theta}}{n^{2}_{2}} + U\eta_{yz} \cdot$ $\cdot (1 + \mu_2 r_1) \left(-\frac{\eta}{p_2^2} c_1 + \mu_2 b_1 \right)],$ $L_{65} = -\omega_{3C} + \frac{\omega_{3C}}{\mu_3 - \mu_2} \mu_3 U^2 \eta_{yz} \left(\frac{\eta}{n_z^2} c_1 - \mu_2 b_1 \right)$ $L_{66} = \frac{\omega_{3C}^2}{\mu_2 - \mu_2} [2\mu_2 \xi_y - \frac{2\mu_3 \xi_\theta}{n_2^2} + U\eta_{yz} \cdot$ $\cdot (1 + \mu_3 r_1) \left(-\frac{\eta}{n_2^2} c_1 + \mu_2 b_1 \right)]$ $N_{1}^{1}=0$, $N_{1}^{2}=\eta_{yz}U^{2}\alpha^{2}a_{2}$, $N_{1}^{3}=0$, $N_{1}^{5}=0$, $N_{1}^{4} = \eta_{yz} U^{2} \alpha^{2} \frac{\omega_{2C}}{\mu_{2} - \mu_{2}} \left(\mu_{3} b_{2} - \frac{\eta_{2}}{n_{2}^{2}} c_{2} \right)$ $N_{1}^{6} = \eta_{yz} U^{2} \alpha^{2} \frac{\omega_{3C}}{\mu_{3} - \mu_{2}} \left(-\mu_{2} b_{2} - \frac{\eta_{2}}{n_{2}^{2}} c_{2} \right)$ $N_{2}^{1}=0, N_{2}^{2}=\eta_{yz}U^{2}\alpha^{3}a_{3}, N_{2}^{3}=0, N_{2}^{5}=0,$ $N_{2}^{4} = \eta_{yz} U^{2} \alpha^{3} \frac{\omega_{2C}}{\mu_{2} - \mu_{2}} \left(\mu_{3} b_{3} - \frac{\eta_{2}}{n_{2}^{2}} c_{3} \right)$ $N_{2}^{6} = \eta_{yz} U^{2} \alpha^{3} \frac{\omega_{32C}}{\mu_{3} - \mu_{2}} \left(-\mu_{3} b_{3} + \frac{\eta_{2}}{\mu_{2}^{2}} c_{3} \right)$

Unfavorable Influences Caused by the Low Quality of The Rehabilitation Works of the Water and Sewerage Networks

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Abstract: This work presents some observations above the unfavorable influences caused by the low quality of the rehabilitation works of the water and sewerage networks. It contains observations about deficiency of design studies - the missing of actual plans regarding the existing network of utilities, non-correlated surveying measurements, establishing the routes for the designed water and sewerage networks, with no accordance to the existing routes; deficiency regarding the quality of the execution works - the unbind of the road cloth, excavation works, reinforcing the edges, the compaction of the support layer, the post of tubes, filling the risk degree of producing work accidents, raising the risk degree of producing with the analytical ones concerning the effects of free air on the propagation velocity of pressure wave and on the unsteady motion.

Keywords: Influences of low quality in rehabilitation work of networks.

1. Introduction

The rehabilitation of utilities - water networks, sewerage networks, underground telecommunications networks, gases networks, underground networks of the electric systems there are works of a special importance, especially in our country, showing a long period of time exploitation and missing the proper works of maintenance. The remake of these networks need special investments, and in case of private distribution companies, whose target is to obtain huge and immediate profits and the actual conditions of credits, with huge debts and restrictive conditions, there is no expectation. For certain cities were obtained external credits, financed by the PHARE and BERD programs. The use of these credits is necessary to be performed in an efficient way, with responsibility, to be able to assure a maximum efficiency of the investments. Unfortunately, the local representatives, the design units, as well as the Romanian contractors, still treat these investments superficially, not giving them the right importance.

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In the present article, we will deal especially with water and sewerage networks having as object study the rehabilitation of the networks in Cluj Napoca city.

2. Deficiency of design studies

The first activity in rehabilitation is the design phase. As is stated in the design contract the main designer has to make the technical project, as well as the designing of execution details. Important chapters in designing the projects are the soil mechanic studies and surveying measurements, for realizing good rises of the existing networks, establishing areas with different degrees of degradation, on which base to result the necessity and the way of the intervention.

2.1. The missing of actual plans regarding the existing network of utilities.

In many cities in the country, even if there are plans of the networks of utilities in the archive, coming from the execution projects. They have not been updated after different interventions during the time,

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and that implies the need of detailed surveying works, with a big volume of work, and a long time of accomplishment of work. The missing of adequate devices for determining the routes of the underground networks, lead to wrong

recordings, that are not according to the real situation on the site.

2.2. Non-correlated surveying measurements.

The fact that is wanted to establish the rises for the water networks and sewerage networks makes that often to follow only these routes without taking into account the existence of other underground networks in the area. This has unfavorable effects, which appear during execution time, especially when is dealing with sewerage networks, which usually they are posed at a reduced scale in comparison with other networks. The intersection of other networks roues - gases networks, energy system networks, water networks, underground telecommunication networks - impose supplementary constructive measures or can lead to accidental deterioration, that has for execution companies economic and social pursuit leading to non-comfort aspects in the area of works.

2.3. Establishing the routes for the designed water and sewerage networks, with no accordance to the existing routes.

Establishing the routes for the water and sewerage networks through the technical projects and execution details has to take into account the fact that during the execution works, the existing networks has to remain functional, and the assurance of continuity of these utilities being an essential condition. It must not be forget the fact that the whole route, for not to become "accommodation" for the underground fauna.

There must not be admitted also to the project authorization, non final solution of route that can affect the ending of the works in optimum conditions, and this can lead in certain situations to abandon of some routes of new networks, proved as insufficient after the execution.

3. Deficiency regarding the quality of the execution works.

3.1. The unbind of the road cloth.

The road cloth, function of its nature, has to be unbinding on a minimum surface, and after the execution works to be repaired. The use of some nonadequate technologies led to obtaining of some edges of the unbinding of the road clothes to be non-uniform. This fact need supplementary works for repairs, leading to increased costs, having as a secondary effect an unpleasant aspect. The difference of quality of materials used for repairs as well as the missing of the qualified and experienced labor force, especially for the road clothes realized from granite paving blocks – create non-comfort also.

3.2. Excavation works.

Mechanical excavation realized with caterpillar excavator lead to degradation of surface of the road cloth, especially if is realized from asphalt mixture. Even that through the technological measures adopted, was looking for avoiding this insufficient fact, through the use of rubber carpets, missing the supervisory of the works during the execution had produced this unfavorable effects. The resulted excess material from excavation, which was not taken away increased the discomfort from the area and produced supplementary costs through another new operation of loading the material. If there is no protection and sign of the execution works as well as the long duration of maintaining the excavation works unbind represented a permanent potential danger.

3. 3. Reinforcing the edges.

This activity, having as principal goal to assure the work protection measures, which was not made correctly and in time, implies the increasing of the volume of the embankment works, because of the crumbling produced. If the works were not realized in time, another effect is a chain incident, crumbing the edges producing damage of the water pipe, that implies damage of the gas pipe. During the repairs of the water pipe, due to not knowing exactly the route of the
underground electric cable, was produced damage of this one too, that leads to an explosion of the gases accumulated in the existing sewerage networks and in some basements of the buildings in the area. There must be mentioned that this was the cause of some work accidents.

3.4. The compaction of the support layer.

Using modern materials of tubes – HOBAS type tubes from resins reinforced with glass fiber – impose realization of an uniform support layer, well compacted, made of sand, that has to assure an uniform repartition of the loads on the ground, and not to permit the damage of the tubes. Not always was used the corresponding materials and was not realized the optimum compaction of the support layer. Because of this after the realization of the filling works, it was reach the situation of producing some disconnecting of the pipe, due to settlements of the posing bed. Being necessary to remake the respective zone.

3. 5. The pose of tubes

The works of pose were not raise special problem, the technology being easy to apply, with the condition of respecting of some minimum requirements referring especially to the handling of pipes – avoiding producing shocks and hitting with hard materials - the cleaning of the area of connecting and avoiding damage of the lip of the pipe. In the contact zone it was necessary to use some parts of short pipes through the agency of those to obtain a high elasticity degree, in order to prevent breaking the pipe in case of producing of small settlements of these zones.

3. 6. Filling the trenches

The filling works must be performed in two phases as follows:

- 1st phase manual fillings well compacted, realized by thin granular material, till 20 cm over the superior generator of the pipe.
- 2nd phase mechanical fillings realized from earth coming from excavation.

It was found the fact that the conditions of realizing the filling works were not respected, regarding the

quality of used material, the thickness of the filing layer and of the optimum humidity of compaction,

as well as the obtained degree of compactness. In these situations was necessary to remake the works, till they are in the admissible limits.

3.7. The remakes of the road clothe

It was necessary the remake of the road cloth to be done on the surfaces greater than those in the documentation, because of not right unbind and the necessity of a new intervention for remake the edges of the trench, or because of producing of some crumbling following non corresponding reinforcing works. Another cause that leads to the supplementary works of remake the roads was due to non-corresponding to the compaction works producing important settlements in time.

4. Unfavorable effects made by non-corresponding quality

4. 1. Raising the risk degree of producing work accidents.

As we show it above, if the works of reinforcing were not made in time or they were not made right, lead to some work accidents. Performing the works in busy areas, under traffic, with the existing utilities working, create the same problems regarding the necessity of taking protection and working measures, which lead to high production costs.

4. 2. Damage of the network

The low quality of works, especially the compaction works, produce in time unfavorable effects, that need difficult and expensive interventions. Missing the tightness of the water networks lead on the one hand to lose of that water and on the other hand training the adjacent material of pipes. Missing the tightness of the sewerage networks lead on the one hand to pollution of underground water with undesirable effects and hard to be evaluated for environment and on the other hand to the raise of used water flow carried to the combing out station, that follows to be manufactured.

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The most frequent damage is produced in joining points of the pipes and at the contact zone between pipes.

4. 3. Economic evaluation of quality mistakes.

Quality mistakes, function of the moments they were observed, they have different economical effects. If they are repaired during the execution, they can be with minimum costs, through simple interventions. Observing the mistakes during the tightness-checking phase implies a higher cost for labor and materials. The mistakes occurring within the period of guarantee of the work are more higher being necessary appearance of intervention team of the constructor, taking off the function of the networks for a certain period of time, as well as the use of special materials that have a not neglecting cost. Only the simple appearance of some settlements that occur because of the noncorresponding compaction leads to increasing the cost with 150%. This increasing can be avoided through a good supervisory of execution works, through assurance of a corresponding rhythm of execution and through assurance of superior qualified labor.

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Woven Soils

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Abstract: Some new but pressing aspects of the civil engineering, like the protection of the built heritage, are leading to a mutually beneficial convergence of these activities. They can not only benefit from each other but also they impose a new, integral treatment of the whole problem. The results may already be seen and assessed, while their use is rapidly increasing. On such a result are woven soils. The woven elements are linear concrete elements called micropiles and horizontal cement mortar lamellas. The authors introduce a non-linear formulation of the composite soil-concrete, using FEM modeling technique.

Keywords: Woven soils, micropiles, cement mortar lamellas.

1. Introduction

One structure infra and superstructure ensemble must be designed to have an optimum response to diverse solicitations and to comply properly with the site local conditions (foundation soil, hydrometeorological conditions).

Cooperation problems are complex phenomenon influenced by numerous parameters determining the soil behavior and hard to quantify at the present time in engineering analysis.

For the practitioner engineers the only reflection way is to achieve some engineering concepts to offer a certain safety for a structure in front of in situ experimental research results imperfections.

For the soils with uniform characteristics in depth, the problems are relatively clear but for multilayered soils with totally inadequate layers for a bearing role, the things are different.

In lot of situations the bearing capacity of these soils is wrong evaluated, with all the possible consequences.

The present modalities used for infrastructures built in such soil type are:

- foundations made on piles or micropiles,

- injection of foundation soil with lime, clay, cement, synthetic resin suspension.

To resolve such problems, the authors tuned up a method with very good results.

The structural principle is the simultaneous use of micropiles and multilayered injections, obtaining a "woven soil" [1].

2. The description of the procedure

The technical literature has been pointing oout since a few years the use of small dimension linear structural elements called micropiles and the use of injections (ascendant or descendant) [2], [3], [4], [5].

The woven soil, a composite after all, is made using simultaneously the micropiles and the injections (Fig.1).

In this manner, the composite woven soil has the next materials: the soil, the micropiles and the cement mortar lamellas.

The soil is the matrix to be reinforced.

On vertical and oblique directions, the reinforcement is made with micropiles and on horizontal direction with cement mortar lamellas.

It is known that the micropiles can be made with lengths between 6 and 32 meters, depending on the soil conditions.



Fig.1

In order to obtain a proper weaving, the number of micropiles must be 8-15 piece on square meter and the cement mortar lamellas layers must be at 50-80 cm distance from interaxis.

3. Structural analysis

The woven soil is substantially different from the classical foundation soils, having a particular behavior in displacements and strains [6], [7], [8].

The structural analysis is made using elastic

elements and substitute dampers or using a fiber included finite element modeling (Fig.2) [9].

The fibers along direction II (horizontal) are the cement mortar lamellas and along direction I (oblique or vertical), the micropiles.

The angle between the direction I fibers and the Cartesian axis $X^{x\alpha}$ in the Gaussian integrating point with the coordinates (0, +0.573) is

$$\beta_f = \beta - (\beta_0 - \beta_{0f})$$

The relationship between the deformations of the included elements in the soil matrix and the oblique system θ^{α} , in the Cartesian system $X^{x\alpha}$ can be expressed in the next form:

$$\mathbf{\varepsilon}_{f} = \begin{cases} \varepsilon_{11} \\ \varepsilon_{22} \\ \varepsilon_{12} + \varepsilon_{21} \end{cases}_{f} = \begin{vmatrix} F_{11} & F_{12} & F_{13} \\ F_{21} & F_{22} & F_{23} \\ F_{31} & F_{32} & F_{33} \end{vmatrix} \cdot \begin{cases} e_{11} \\ e_{22} \\ e_{12} + e_{21} \end{cases} = \mathbf{F}\mathbf{e}$$



Fig.2

POINT

The expression of the transformation matrix \mathbf{F} is given in work [9].

Considering a linear elastic behavior for the fibers, but with large displacement possibilities, the stresses in the Cartesian system are

 $\sigma^{11}=E_{f}\epsilon_{11}$

and in the oblique system

$$\mathbf{n} = \begin{cases} n_{11} \\ n_{22} \\ n_{12} \end{cases} = \mathbf{F}^{\mathrm{T}} \boldsymbol{\sigma}$$

Figure 3 presents the numerical results regarding the procedure application, relative to the use of micropiles alone.



Fig.3

4. Conclusions

- The weaving of the soil in the case of nonhomogeneous soil having also in their composition deforming layers seems to be a very efficient method relative to the classical soil consolidation methods.

- The weaving of the soil is made applying simultaneously the micropiles and the multilayer injections.

- The proposed procedure is efficient to consolidate the structures damaged by the malfunction of the infrastructure.

- The procedure can be use also to consolidate the earth dams, dikes, road foundations and landing paths.

- This procedure eliminates the use of underpinnings, very delicate works, especially in the case of historical monuments.

- The technology used to make the woven soils allows the total mechanizing of consolidation works.

- The micropiles and the injected cement layers cooperation with the matrix (the soil) makes the total mobilization of all participant factors.

- The woven soils are used to consolidate the surface foundations and for underground works, but also to slope stabilization and deep diggings reinforcing.

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Analytical Method for the Evaluation of the Functioning of the Battery of Wells Equipped with Submerged Pumps functioning in Different exploiting Configurations

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Abstract: The paper presents an analysis method based on the utilization of the analytical characteristics of the system's components that facilitate the evaluation of its working conditions and allows at the same time the establishment of adequate measures for an efficacious control of the working regime. Suitable criterions have also been inserted in this analysis (such as maximum efficiency of the hydro mechanical equipment respectively the optimum diameter of the collecting pipe), in order to improve the energetic and economical performance of the tapping of the ground water front through drilling and making it easier to establish optimal capacity of the tapping fronts – pressure pipe. Case study: a tapping of ground water front constituted of a battery of 20 drilling wells equipped with submerged pumps HEBE 65x2, equidistantly disposed into two branches, on the left bank of the Moldova river.

Keywords: tapping of the ground water front, optimum diameter, energetic and economical performance.

1. Introduction

The ground water resources are generally exploited by using a battery of drilling wells equipped with submerged pumps whose working function leads to energetic costs that are dependent on their working conditions during the evaluation period- for example the calendar year. A frequently encountered case is the tapping of the ground water front – that is relatively homogenous – from the major stream bed, shaped as a battery of wells with quasi-identical features (same hydrodynamic level, same value for the average discharge, same type and dimensions of the submerged pump, same pressure pipes linked to a collector, whose nominal diameter of the tubing length increases towards its downstream end).

In practice, the drill pump is chosen according to the worst parameters: (Q_p, H_p) - the ones associated to the upstream well of the collector - the analysis being effectuated further based on them.

Such an approach will lead to normal results for the water intake working at nominal capacity, but it will have poor economical and energetic performance. Due to pressure loss between the collector's nodes and also because of the variation of the local hydraulic resistance in bounds, in fact, the hydrodynamic head of each pump differs from one well to another, meaning that both the discharge and the efficiency, thus, the power consumption will differ.

Because one pump can have acceptable performance in the dimensional regime, but poor results in other working regimes, the pump mustn't be chosen based only on the dimensional parameters of the installation, as it's not viable energetically and economically. Besides this data, the working parameters of the installation must be known, and its choice must be based on complying with most working regimes.

In the studied case, all working regimes for each well must be analyzed, in real functioning conditions, which can become very laborious when the number of wells (n) increases.

In the following, by taking on a systematical approach of the functioning of the battery of quasiidentical pumping wells, it is advised the use of an analysis method based on the mathematical modeling of the system, facilitated by the use of the analytical characteristics of its own components. The practical way of the method is exemplified on the concrete case of the Moţca 1 water catchment's of the Paşcani municipally water supply, the obtained results being confirmed by the data from the on-site measurements concluded on some of the wells from the specific installation.

2. Theoretical Considerations

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2.1.Working Hypothesis: the links between the discharge and head are studied in a system of *n* wells (fig. 1) that admits usual estimates from practice:

• Battery of drilled wells, equipped with identical submerged pumps, linked to a collecting pipe through identical pressure pipes;

• The submerged pump is specifically chosen to ensure the user's needs so that the well's potential discharge isn't exceeded;

• Water is pumped from an aquiferous with a quasi-horizontal free surface.

2.2The Main Data refers to:

• The pressure characteristic of the pump:

$$H = H_{pf} - K_p \cdot Q^2 \tag{1}$$

• The efficiency characteristic of the pump:

$$\eta = R_1 \cdot Q - R_2 \cdot Q^2 \tag{2}$$

• The hydraulic resistance module of one well's pressure pipe: M_{roj} ;

• The hydraulic resistance module of the tubing lengths between the well's branching $M_{r_i-l_i}$;

- The hydraulic resistance module of the joints:
- - on the lateral branch $M_{r\zeta lj}$
- - on the main branch $M_{r\zeta tj}$

• The hydraulic resistance module of the connecting tubing length (nO): M_{rleg} ;

• The hydrodynamic level of the aquiferous –same in all wells: Z_i

The energetic level for the outflow section of the collecting pipe, ex: the reservoir's water level Z_p .

2.3.*The Equivalent Pressure Characteristic of the Group* (1...j) is represented as follows:

$$H = H_{gf_j} - K_{g_j} \cdot Q^2 \tag{3}$$

with H_{gf} and k_{gj} , j=1,2,...,n, that have to be determined.

Main equations for the current node (j):

• The equivalent pressure characteristic of the active wells group upstream (1-j):

$$H = H_{gf_{l-1}} - (K_{g_{j-1}} + M_{r_{j-1,j}} + M_{r\zeta_{lj}}) \cdot Q^2$$
(4)

• The well's pressure characteristic, reduced to the (*j*) joint:

$$H = H_{pf} - (K_p + M_{ro_j} + M_{r\zeta_{lj}}) \cdot Q^2$$
(5)

• The continuity equation for the (*j*) node:

$$Q_{gj} = Q_{gj-1} + Q_{pj}$$
(6)

• The upstream group's discharge, under the current (H) head in the (j) node:

$$Q_{gj-1} = \sqrt{\frac{H_{gf_{j-1}} - H}{K_{g_{j-1}} + M_{r_{j-1,j}} + M_{r_{\zeta_{ij}}}}}$$
(7)

• The Pj well's discharge, under the same (H) head in j:

$$Q_{pj} = \sqrt{\frac{H_{pf} - H}{K_p + M_{ro_j} + M_{r\zeta_{lj}}}}$$
(8)

• The discharge of the (1...j) group under the given head:

$$Q_{gj} = \sqrt{\frac{H_{gf_j} - H}{K_{g_j}}} \tag{9}$$



• The continuity equation accordingly to the head H:

$$\sqrt{\frac{H_{gf_{j}} - H}{K_{g_{j}}}} = \sqrt{\frac{H_{gf_{j-1}} - H}{K_{g_{j-1}} + M_{r_{j-1,j}} + M_{r_{\zeta_{ij}}}}} + \sqrt{\frac{H_{pf} - H}{K_{p} + M_{ro_{j}} + M_{r_{\zeta_{ij}}}}}$$
(10)

respectively:

$$+\sqrt{\frac{H_{pf}-H}{K_p}}\left(\frac{1}{\sqrt{1+\frac{M_{ro_j}+M_{r\zeta_{lj}}}{K_p}}}\right)$$
(11)

Observation:

If the pumps are identical and placed in wells exploiting the same aquiferous having the free surface quasi-horizontal: $H_{gf_j} = H_{gf_{j-1}} = H_{pf}$ so that by using identical pressure pipes $(M_{ro_i} = M_{ro} = \text{const})$, the resulting equation is:

$$\sqrt{\frac{H_{gf_{j}} - H}{K_{g_{j}}}} = \sqrt{\frac{H_{gf_{j-1}} - H}{K_{g_{j-1}}}} \left(\frac{1}{\sqrt{1 + \frac{M_{r_{j-1,j}} + M_{r\zeta_{ij}}}{K_{g_{j-1}}}}} \right) + \sqrt{\frac{1}{K_{g_{j}}}} = \sqrt{\frac{1}{K_{g_{j-1}}}} \left(\frac{1}{\sqrt{1 + \frac{M_{r_{j-1,j}} + M_{r\zeta_{ij}}}{K_{g_{j-1}}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}{K_{p}}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{r\zeta_{ij}}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{roj}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{roj}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{roj}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{roj}}}{K_{p}}} \right) + \sqrt{\frac{1}{K_{p}}} \left(\frac{1}{\sqrt{1 + \frac{M_{roj} + M_{roj}}}$$

(12)

respectively:

$$\sqrt{\frac{K_p}{K_{g_j}}} = \sqrt{\frac{K_p}{K_{g_{j-1}}}} \left(\frac{1}{\sqrt{1 + \frac{M_{r_{j-1,j}} + M_{r\zeta_{ij}}}{K_{g_{j-1}}}}} \right) + \left(\frac{1}{\sqrt{1 + \frac{M_{ro_j} + M_{r\zeta_{ij}}}{K_p}}} \right)$$
(13)

By using the notation: $v_k = \sqrt{\frac{K_p}{K_{g_k}}}$ the following recursion formula is obtained, that links the hydraulic

resistance coefficients:

$$\nu_{j} = \left(\frac{\nu_{j-1}}{\sqrt{1 + \frac{M_{r_{j-1,j}} + M_{r\zeta_{ij}}}{K_{g_{j-1}}}}}\right) + \left(\frac{1}{\sqrt{1 + \frac{M_{ro_{j}} + M_{r\zeta_{ij}}}{K_{p}}}}\right),$$

where:
$$K_{g_j} = \frac{K_{g_{j-1}}}{v_{j-1}^2}$$
 (15)

allows the head characteristic of the wells group (1...j) to be defined as in equation (3).

Calculation begins at the farthest well from the downstream end (*O*) of the collecting pipe – branched to the *j*=1 node – having: $n_o=0$; $M_{r0,I}$ $M_{r\zeta}$ $I_I=0$; and continues for each node- accordingly to relations (14) and (15) – until is reached the section for which is determined the equivalent hydraulic resistance module of the active wells group. CARPGAR, a program for automated calculations, has been written for practical applications using the above equations, using the BASIC language programming.

For the multiple well tapping fronts, the equivalent hydraulic resistance module can be determined by using the CARPGAR program for 6...8 values of $j \le n$: K_{gk} , K_{gl} ..., K_{gn} so that afterwards, considering $k_g \approx n$ a relation such as can be determined in given conditions:

$$K_{g_n} = \frac{K_{go}}{n^{\alpha}}$$
(16)

2.4. Discharge, head and efficiency of active pumps:

• Head in the downstream end of the collecting pipe (H0):

$$H_0 = Z_p - Z_i \tag{17}$$

• Discharge of the battery of wells depending on the head H0:

$$Q(n, Z_i) = \sqrt{\frac{H_{gf_n} - H_O}{K_{g_n} + M_{rosp}}} = \sqrt{\frac{H_{pf} + Z_i - Z_p}{K_{g_n} + M_{rosp}}}$$
(18)

• The head in the section (O):

$$H_{fcO}(n, Z_i) = H_{pf} - K_{g_n} \cdot \frac{H_{pf} + Z_i - Z_p}{K_{g_n} + M_{rosp}}$$
(19)

• Average discharge of one well from the active battery of n wells:

$$q(n, Z_i) = \frac{Q(n, Z_i)}{n}$$
(20)

• Local head loss on the specified tubing length:

$$h_{rj-1,j} = (M_{rj-1,j} + M_{r\zeta tr_j}) \cdot [q(n, Z_i) \cdot j]^2$$
(21)

passing through the confluence, the hydraulic resistance module is:

$$M_{r\zeta tr_j} = \frac{8}{\pi^2 g} \frac{\zeta_{ctr_j}}{D_j^4}$$
(22)

• Pressure loss on the pressure pipe Pk-k:

$$h_{rlP_{k}k} = M_{rF_{k}k} \cdot [q(n, Z_{i}) \cdot k]^{2}$$
(23)

• In this formula, the hydraulic resistance module of the pressure pipe F_{k} , is:

$$M_{rF_{k}k} = M_{ro} + \frac{8}{\pi^2 g} \frac{\zeta_{cl}}{D_o^4}$$
(24)

• Head, of the node *n*:

$$H_n = H_O + h_{rleg} \tag{25}$$

• Head of current node, commencing from the *n* node:

$$H_{j-1} = H_j + h_{r_{j-1,j}}$$
(26)

• Head at the pressure pipe:

$$H_{pk} = H_k + h_{rlP_kk} \tag{27}$$

• The pump discharge from the drilling well P_k, of the group of n active drillings:

$$Q_{pk}(n, Z_i) = \sqrt{\frac{H_{pf} - H_{pk}}{K_p}}$$
 (28)

• Pump efficiency:

$$\eta (n, Z_i) = R_1 \cdot \sqrt{\frac{H_{pf} - H_{pk}}{K_p}} - R_2 \cdot \frac{H_{pf} - H_{pk}}{K_p}$$
(29)

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• The power-handling capacity of the shaft pump:

$$N(n, Z_i) = \frac{981 \cdot H_{pk}}{R_1 - R_2 \sqrt{\frac{H_{pf} - H_{pk}}{K_p}}}$$
(30)

3. Analysis Method

3.1.The establishment of the final head characteristic of the battery of wells with k active drilling

• All the relative data of the constructive and functional characteristics of the battery of wells – the diagram of the functional and energetic characteristics of the used pump (same type and dimensions), the structure (pipes, elbows, valves, no return valve, water – meter) and the dimensions (*Li*, *Di*) of the pressure pipes between the pump and the collector and also of the tubing lengths between the nodes, are added and systemized.

• Using specific computer programs (such as the SHUFP), the hydraulic resistance modules are calculated and the parameters for the head characteristics and efficiency of the pumps are established (H_{pf} , K_p , R_1 , R_2).

• By using the CARPGAR program and the results from the analysis, the equivalent resistance modules are determined for different exploiting configurations of the battery of wells (K_{gk} , k=1,2...n).

• If the tapping front has two branches linked to the same *reservoir front* junction (*PS*) the equivalent hydraulic resistance modules according to the different exploitation configurations of this ensemble are established by using computation methodology properly to the pump in parallel connection and an adequate computer program.

• Using a statistical approach for processing the connecting sleeves array (k, K_{gk}) , adequate coefficients such as K_{go} and α are established by adjusting to this a power function (16), the MICROCAL ORIGIN programming system can be used for this calculation.

• Considering the fact that $H_{gfk}=H_{pfk}$, the equivalent head characteristics of the well battery are built for

different exploiting configurations with k active drillings:

$$H = H_{pf} - \frac{K_{go}}{k^{\alpha}} \cdot Q^2 \tag{31}$$

3.2 The analysis of working regimes under a given head

• Knowing the head characteristics of the active well battery (31), for the particular case (k=no) and considering the energetic levels of the two characteristic water races of the water project, by using MATHCAD, the first step is to determine:

• - The discharge of the active wells:

$$Q(n, Z_i) = \sqrt{\frac{H_{pf} + Z_i - Z_p}{\frac{K_{go}}{n_o^{\alpha}} + M_{rosp}}}$$
(32)

• - The head in the confluence section of the tapping front's branches:

$$H_{fcO}(n, Z_i) = H_{pf} - \frac{K_{g_n}}{n_o^{\alpha}} \cdot \frac{H_{pf} + Z_i - Z_p}{\frac{K_{g_n}}{n_o^{\alpha}} + M_{rosp}}$$
(33)

• - The average discharge of each active well:

$$q(n,Z_i) = \frac{Q(n,Z_i)}{n_o}$$
(34)

• Once the effective head in the confluence section of the tapping front's branches and the average discharge of each active well are known, the head losses are evaluated for the connecting tubing length Pn-O and head of each one's last nodes.

• By using the equations (21) (26) and (27) in the MATHCAD programming system, passing from one node to another the head for each node is calculated and finally the head for the junction of the active pumps (H_{ok}).

• Accordingly to the effective head (H_{pk}) by using relations (28)...(30) for each active pump is calculated the discharge (Q_{pk}) , the efficiency (η_{pk}) , and the power-handling capacity of the shaft pump (N_{pk}) .

4. Case study

Head characteristics and working regime of the tapping of ground water front 1 Moţca - Paşcani, considering the alimentation of a reservoir with bed level Z_p

Tapping of ground water front 1 Moţca -Paşcani, placed on the left bank of the Moldova River, is made of a battery of 20 drilled wells placed equidistantly in two branches: front 1 -right, with 13 wells and tapping front 1 –left, with 7 wells (fig. 2).

• Submerged HEBE 65x2 electro pumps are being used to pump water from the wells, and these pump's functional and energetic characteristics are analytically represented by the parameters of the head curve ($H_{pf} = 31,22$ m; $K_p = 327571,5$ s².m⁻⁵) and those of the efficiency curve ($R_1 = 18464$; $R_2 = 1903360$).

The Dn 100 pressure pipes are 10 meters in length, have three-90° elbow, one paddle flow meter, a valve and a non-return valve. Due to equivalent roughness k=0,8 mm and local resistances $\Sigma \zeta_i = 9,64$, these are characterized by a hydraulic resistance module $M_{ro} = 11040,6 \text{ s}^2.\text{m}^{-5}$.



Fig. 2. The Moțca - Pașcani taping front of ground water 1 design

• The collecting pipes, having the absolute equivalent roughness of its composing pipes

estimated k=0.8 mm, are characterized – according to nominal diameters, their length and the design

discharge – by the hydraulics resistance modules of the pipes $(M_{r_{j-1,j}})$, and those of the confluences $(M_{r\zeta cl_j}, M_{r\zeta ct_j})$. Their values are calculated using SHUFP program and the results are in the first three columns of tables 1 and 2.

• By using the CARPGAR calculation program with the previously determined data the resistance modules (K_{gj}) for the active well group $(P_1...P_j)$ upstream from the current node (j) are established

and shown – with the corresponding v_j coefficients – in the same tables.

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• By using the CARPGAR calculation program with the previously determined data the resistance modules (K_{gj}) for the active well group $(P_1...P_j)$ upstream from the current node (j) are established and shown – with the corresponding v_j coefficients – in the same tables.

Table 1-The hydraulic resistance modules of the front 1- left components and the par	rameters
of the resulting head characteristic of the active wells group	

J Node	M_{rlat_j}	$M_{r_{j-1,j}}$	$M_{r\zeta ct_j}$	V_j	K_{gj}	ΣM_{rjO}	K_{gO}
1	0	0	1	0.98	333046.0	12850,44	345896,44
2	910	9360	2	1.95	85946.5	3490,44	89436,94
3	725	1089	14	2.92	38356.2	2401,44	40757,64
4	505	1089	25	3.86	21943.6	1312,44	23256,04
5	300	239.6	37	4.82	14084.2	1072,84	15157,04
6	95	239.6	25	5.76	9868.4	833,24	10701,64
7	-105	239.6	60	6.66	7386.0	593,64	7979,64
8	-308	239.6	72	7.51	5812.0	354,04	6166,04
9	-510	74.2	85	8.39	4652.4	279,84	4932,2
10	-715	74.2	96	9.23	3848.2	205,64	4053,84
11	-917	74.2	110	10.25	3115.2	131,44	3246,64
12	-1150	74.2	128	10.92	2745.9	57,24	2803,14
13	-1400	28.6	147	11.57	2445.2	28,62	2473,82

 Table 2-The hydraulic resistance modules of the front 1-right components and the parameters

 of the resulting head characteristic of the active wells group

<i>j</i> Node	M_{rlat_j}	$M_{r_{j-1,j}}$	$M_{r\zeta ct_j}$	$ u_j$	K_{gj}	ΣM_{rjO}	K_{gO}
1	0	0	268	0.98	333046	12496.4	345542.4
2	910	9360	11	1.95	85947.6	3136.4	89084.0
3	725	1089	25	2.92	38359.8	2047.74	40407.5
4	505	1089	37	3.86	21950.0	958.4	22908.4
5	300	239.6	49	4.82	14093.5	718.8	14812.3
6	95	239.6	60	5.75	9893.7	479.2	10372.9
7	-105	239.6	75	6.65	7410.9	239.6	7650.5

• In order to analyze the working regimes of the tapping front in the system, starting from the hydraulic resistance modules K_{gj} , the resistance modules according to the head characteristics for confluence section of the two branches (K_{gOj}) are also determined, and represented in the last column of

tables 1,2, next to the resistance modules of the (*j*-O) tubing length of the collecting pipes.

Based on the equivalent head characteristics, whose parameters $(H_{gfO_k} = H_{pf} \text{ and } K_{gO_k})$ have been determined, the head curves of the battery of wells



that function on the left (fig. 3) and right branch (fig. 4) of the tapping front are calculated.



Fig. 3. Head characteristic of the wells battery – the left tapping front 1, in different configuration having k=1,2,...,13 active wells, exploited between $Z_{imin}=260$ mNM and $Z_{imax}=267$ mNM, on a reservoir having $Z_p=272$ mNM



Fig. 4. Head characteristic of the wells battery – the right tapping front 1, in different configuration having k=1,2,...,7 active wells, exploited between $Z_{imin}=260$ mNM and $Z_{imax}=267$ mNM, on a reservoir having $Z_p=272$ mNM The resulting characteristics of the system made determined by using the equivale

of the two branches, in given conditions, are

determined by using the equivalent hydraulic resistance modules, for the exploiting configurations

of each branch ($K_{gO_m}^s, K_{gO_l}^d$), according to the classic relation for the group of pumps connected in parallel, when $H_{gf_m}^s = H_{gf_k}^d = H_{fp}$

$$\frac{1}{\sqrt{K_{gO_{m+k}}}} = \frac{1}{\sqrt{K_{gO_m}^d}} + \frac{1}{\sqrt{K_{gO_k}^s}}$$
(35)

• Considering the reasonable combinations of m active wells on the right branch of the tapping front and k active wells on the left branch, so that $m+k=8,10,\ldots,20$ the results from table 3 are obtained. The equivalent hydraulic resistance modules of the tapping front 1 Moţca- Paşcani in a couple of exploiting configurations with 8...20 active wells

• The analysis of the results from tables 1,2 and 3 indicate close values of the K_{gO} modules obtained in different combinations with the same total number of active wells, therefore the results have been systemized in table 4.

• Numerical analyzed, the series values from table 4 leads to power function relation type, whose parameters are presented in fig. 5.

The head characteristics of the tapping front 1 can be therefore expressed, for any combination of n active wells (n=8...20), analytically with the relation and graphically with head curves based on this relation:

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$$H = H_{pf} - \frac{K_{go}}{n^{\alpha}} \cdot Q^2 \tag{36}$$

where H_{pf} =31,22 ; K_{go} =326370; α =1,93562, (fig. 6).

• In the case of maximum capacity (n=20) exploitation, reported to the hydrodynamic level of water in the wells, the tapping front provides a catchments discharge $q(20,Z_i)$, under the head in *O*: *Ho*, with the medium discharge of each well $q(20,Z_i)$, whose values are presented in table 5.

The energetic and functional characteristics of the submerged pumps, determined in the previously conditions, according to sections 2 and 3 are represented in fig. 6 that shows: the piezometric lines of the collector and of the pumping wells, the discharge variation and the efficiency of the pumps.

III (a couple of expl	ioning coming	surations with		C 115
Active	wells	K^{d}	K^d K^s		K_{gO}
1 right	1 left	m_{gO_m}	$-gO_k$	wells	
7	13	7650,5	2473,82	20	1005,36
	11		3246,64	18	1190,45
	9		4932,2	16	1517,35
	7		7979,64	14	1953,12
	5		15157,04	12	2614,96
5	13	14812.3	2473,82	18	1246,66
	11		3246,64	16	1506,19
	9		4932,2	14	1983,14
	7		7979,64	12	2653,98
	5		15157,04	10	3745,80
3	9		4932,2	12	2708,8
	7		7979,64	10	3824,87
	5		15157,04	8	5829,59

Table 3-The equivalent hydraulic resistance modules of the tapping front 1 Moţca- Paşcani in a couple of exploiting configurations with 8...20 active wells

Table 4-The calculation values of the equivalent hydraulic resistance module in exploiting configurations with 8...20 active wells

	n	8	10	12	14	16	18	20
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Active wells battery discharge $Q(n, Z_i)$ (m³/s)

0.09

0.1

0.11

0.12

0.13

0.08

 $H_o(Z_{i max})$

0.15

0.16

0.14

6

5 E---0.04

0.05

0.06

0.07

Fig. 6. Head characteristic of the wells battery - the tapping front 1, in different

configurations having $k=1,2,\ldots,20$ active wells, exp	ploited between Z _{imin} =260 mNM
and Z_{imax} =267mNM, on a reservoir ha	aving $Z_p=272 \text{ mNM}$

Table 5-The functional characteristics of the tapping of the ground water front 1, pumping on a reservoir having operating water level Z_p =272 mNM

No	Z_i (mNM)	$Q(m^3/s)$	$H_{O}(\mathbf{m})$	q (l/s)
1	267	0,155	7,33	7,8
2	260	0,133	13,71	6,7



Fig. 7. The energetic and functional characteristics of the submerged pumps of the tapping ground water front 1 Moţca - Paşcani

5. Conclusion

Due to head loss between the collector's nodes but also because of the variation of local hydraulic resistance at confluences, the pump's head differ from one well to another, therefore the discharge and the efficiency of the electro-pumps and their energy consumption differs.

The determination in working conditions of the effective working regimes of each well can be

realized by using an analytical method, elaborated by using a systematical approach of the functioning of the battery of quasi – identical pumping wells, that are exploiting a homogenous aquiferous.

The proposed analysis method is based on the system's mathematical modeling, made easier by the use of the analytic characteristics of its components and the automated data processing system.

The results obtained by applying the proposed method in the case of the tapping front 1 Moţca – Paşcani, concurs very well with the data obtained through on – site measurements of the mentioned installation.

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Consideration on the Hydraulic Resistance of the Confluence T-Joints on The Collector of The Tapping Ground Water Fronts with Submerged Pumps Equipped Wells

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Abstract When determining the resulting head characteristics of the hydraulic systems with numerous elements coupled in parallel on a collecting pipe made of piping tracts with different diameters, calculating the local head loss at confluences, usually produces great discomfort to the specialist conducting the calculations, therefore he usually neglects or rudely approximates these values, thus diminishing the precision of the results.

In order to facilitate the calculation of the working regimes of such installations, this paper presents the way for determining relations that in practical conditions can express the local head loss at confluences, and the local hydraulic resistance modules, as simple order's numbers functions of the singularities, comparative to the upstream end of the collector.

Keywords: the local head loss at confluences, computing relations, local hydraulic resistance modules.

1.Introduction

When determining the resulting head characteristics of the hydraulic systems with numerous elements coupled in parallel on a collecting pipe made of piping tracts with different diameters

The mentioned situation is determined by the particularities of those singularities, making their quantitative characterization extremely laborious:

- for each confluence, the values for two local head coefficients must be established – one for the *lateral* (ζ_{cl}) side, the other for the *main passing* (ζ_{cl}), both expressing the local head loss at confluences in terms of the specific kinetic energy of the flow on the associated components;

- the values of these coefficients depend on a relatively large number of factors: the angle of the confluence (α), the rapport between the transversal section surfaces of the flows that come in contact in the given section surfaces ($a_l = A_l/A_c$; $a_t = A_t/A_c$) and the rapport of the significant discharges ($q_l=Q_l/Q_c$), all their values being different from one collector's confluence to another;

- the tables and diagrams presented in the specialty literature give the values of the two coefficients for the specific kinetic energy of the flow transported by the collectors $(v_c^2 / 2g)$, so that for determining the lateral head loss, its value has to be recalculated, so that it can be associated to the kinetic energy of that specific flow $(\zeta_{cl}^{\ell} - \zeta_{cl} (a_l / q_l)^2)$;



Fig.1-The schematics for the confluence of the flows

- the local hydraulic resistance modules involved in the analysis have to be calculated according to the modifications from one confluence to another of the channel diameters.

In order to facilitate the calculation of the working regimes of such installations, the results obtained trying to establish relations between the local hydraulic resistance modules, as simple functions of ordering numbers of singularity (j) of the upstream end of the collector are presented in the following.

2. Theoretical Considerations

In order to evaluate the local head loss at flow confluences, besides tables and diagrams, the specialty literature presents also some formulas such as those proposed by Levin and Taliev, that, by having a solid theoretical base, contain also the corrections imposed by the calculation comparison of experimental results obtained by Levin, Gardel, Kinne, Petermann and Vogel [1].

For the lateral pipe-line, the coefficient of the locale head loss (ζ_{cl}) is given by the following relation:

$$\zeta_{cl} = h_{rcl} / (v_c^2 / 2g) = B \left[1 + \left(\frac{q_l}{a_l}\right)^2 - 2 \frac{(1 - q_l)^2}{a_l} - 2 \frac{q_l^2}{a_l} \cdot \cos \alpha \right] + K_{lt}$$
(1)

and the coefficient of the local head loss of the passing (ζct) results from the following relation:

$$\zeta_{ct} = h_{rct} / \left(v_c^2 / 2g \right) = 1 + \left(\frac{1 - q_l}{a_l} \right)^2 - 2 \frac{(1 - q_l)^2}{a_t} - 2 \frac{q_l^2}{a_l} \cdot \cos \alpha + K_t$$
(2)

The values of the coefficients *B*, K_b , K_t are systematized in tables 1 and 2. The coefficients K_l and K_t for the confluences with $A_l+A_t>A_c$, become nulls $(K_l=K_t=0)$.

Table 1. The values of the coefficient B

a_l	≤ 0,35	> 0,35	
q_l	0,,,1	≤ 0,4	> 0,4
В	1,0	$0,9(1-q_l)$	0,55

For the lateral pie-line, the coefficient of the head loss at the confluence, is expressed in terms of the specific kinetic energy, by the following relation:

$$\zeta_{cl}^{lat} = h_{rcl} / \left(v_l^2 / 2g \right) = \zeta_{cl} \cdot \left(\frac{a_l}{q_l} \right)^2$$
(3)

Table 2. The values of K_l and K_t coefficients when $A_l + A_t = A_c$

α	al							
	0,10		0,20		0,33		0,5	
	K_l	K_t	K_l	K_t	K_l	K_t	K_l	K_t
30	0	0	0	0	0,17	0,14	0	0,35
45	0	0,05	0	0,14	0	0,14	0	0,30
60	0	0	0	0	0	0,10	0,10	0,25
90	0	0	0,1	0	0,20	0	0,25	0

The confluence of the pipes with circular section is characterized by the diameters D_b , D_b , D_c , respectively associated to the passing of the lateral canalization and the tubing length collector; this is the reason how is determined the particularization of the values used for the determination of the coefficients of the locale head loss:

$$a_l = \frac{A_l}{A_c} = \left(\frac{D_l}{D_c}\right)^2 a_t = \frac{A_t}{A_c} = \left(\frac{D_t}{D_c}\right)^2 \tag{4}$$

For a collector with *n* confluences whose *laterals* have the same diameter $D_c=d$ and transporting the same discharge $Q_l=q$ (Fig. 2) the relative discharge (q_l) becomes a function of *k* (his ordering number); the nods are numbered starting with the farthest upstream confluence:

$$q_{l}(k) = \frac{Q_{l}(k)}{Q_{c}(k)} = \frac{q}{k.q} = \frac{1}{k} , \quad k = 1, 2, ..., n \quad (5)$$

Fig. 2. The schematic of a colector with n lateral communications

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For certain values of the relative area of the *lateral* $a_l \in \{0.2, 0.4, 0.6, 0.8, 1\}$, in the case of the confluence under an angle $\alpha \in \{30^0, 45^0, 60^0, 90^0\}$, for collectors with a=1 – structures with $D_l=D_c$ –

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usually found in practice, by using the MATCHAD program and based on the relations (1) and (2), the diagrams of the coefficients of the locale head loss at the confluences (ζ_{cb} , ζ_{cl}), are obtained (Fig. 3, 4, 5, 6).

In the same condition, by using the relation (3) the corresponding diagrams of the locale head loss at the confluence expressed in terms of the specific



Fig.3 Local head loss coefficient for the 90° confluence reported to collector



Fig.4 Local head loss coefficient for the 60° confluence reported to collector



Fig.5 Local head loss coefficient for the 45° confluence reported to collector

kinetic energy of the directional water-flow (ζ_{cl}^{lat} , ζ^{eol}_{ct}) are obtained. The diagrams are presented in the Fig. 7, 8, 9, 10.



Fig. 6 Local head loss coefficient for the 30° confluence reported to collector



Fig. 7 Local head loss coefficient for the 90° confluence reported to interested communications



Fig. 8 Local head loss coefficient for the 60° confluence reported to interested communications



Fig. 9 Local head loss coefficient for the 45° confluence reported to interested communications

Local head loss depends on the effective discharge according to the locale hydraulic resistance



Fig. 10 Local head loss coefficient for the 30° confluence reported to interested communications

modules – on the lateral pipeline and on the passing. The relations are:

$$M_{r\zeta cl}(D_l, D_c, \alpha, k) = \frac{8.B.(k.a_l)^2}{\pi^2 g} \left[1 + \left(\frac{1}{k.a_l}\right)^2 - 2\frac{(1-k^{-1})^2}{a_l} - 2\frac{k^{-2}}{a_l} \cdot \cos\alpha \right]$$
(6)

$$M_{r\zeta ct}(D_l, D_c, \alpha, k) = \frac{8}{\pi^2 g} \left[1 + \left(\frac{1 - k^{-1}}{a_l}\right)^2 - 2\frac{\left(1 - k^{-1}\right)^2}{a_l} - 2\frac{k^{-2}}{a_l} \cdot \cos\alpha \right]$$
(7)

The specific values for a practical case can be easily determined by using the same program MATHCAD, therefore obtaining pair orders (k, $M_{r,\zeta cl}$), respectively (k, $M_{r,\zeta cl}$), with k = 1,2,...,n.

In order to facilitate the analysis of the working regimes for such hydraulic systems with different operating configurations, the use of the suitable analytical functions for expressing $(k \sim M_r _{\zeta cl})$, respectively $(k \sim M_r _{\zeta cl})$, is recommended. Therefore, by analyzing the value orders using numerical methods (such as the method of the smallest squares), relations can be obtained in the following forms:

-the local hydraulic resistance module at the lateral pipe-line of the confluence:

$$M_{r,cl}(k) = M_{r,lo} - M_{1.}k - M_{2.}k^{2}$$
(8)

-the local hydraulic resistance module of the *passing*:

$$M_{r\zeta ct} = \frac{M_{r\zeta cto}}{k^b} \tag{9}$$

3. Case Study

The local hydraulic resistance modules at the confluences of the tapping of ground water front 1 Moţca-Paşcani

The tapping of ground water front 1 Moţca-Paşcani placed on the left bank of the Moldova river is constituted of a battery of 20 drilling wells equidistantly disposed in two branches: the front 1 on the right bank with 13 wells and the front 1 - on the left side, with 7 wells (Fig. 11).



The water pumping from the wells is made with submerged pumps HEBE 65x2 with quasi - identical energetic and functional characteristics.

• The Dn 100 pressure pipes are 10 meters in length, have three-90° elbow, one paddle flow meter, a valve and a non-return valve. Due to equivalent roughness k=0.8 mm and local resistances $\Sigma \zeta_i = 9.64$, these are characterized by a hydraulic resistance module $M_{ro} = 11040.6 \text{ s}^2 \text{.m}^{-5}$.

By using the calculation program developed for the MATHCAD programming system, considering the mathematical model presented, in §2 and according to the condition of the Moţca-Paşcani tapping front 1, the values for the coefficient of the local head loss at confluences and their hydraulic resistance modules have been determined and the resulting values are systemized in table 3.

Table 3. The local head loss coefficients and the local hydraulic resistance modules at the tapping from	nt
confluences 1 Moțca – Pașcani, characterized by $D_l=d=100$ mm, $D_t=D_c$, $\alpha=90^\circ$ și $q_l(k)=Q_l/Q_c=1/k$	

Nod	D_c	$a = (D/D)^2$	a	۔ lat	col	М	М
k	(mm)	$a_l - (D_l D_c)$	q_l	ς _{cl}	ζ_{ct}	Μ _{rζcl}	$M_{r\zeta ct}$
1	100	1,000	1	0,589	0	486,51	0
2	150	0,444	0,5	0,767	0,75	633,78	122,37
3	150	0,444	0,333	1,198	0,556	986,16	90,645
4	200	0,250	0,25	0,875	0,437	722,75	22,586
5	200	0,250	0,2	0,562	0,360	464,63	18,585
6	200	0,250	0,167	0,125	0,306	103,25	15,774
7	200	0,250	0,143	-0,438	0,265	-362,38	13,696
8	250	0,160	0,125	0,130	0,234	107,05	4,956
9	250	0,160	0,112	-0,203	0,210	-167,84	4,438
10	250	0,160	0,1	-0,587	0,190	-485,03	4,018
11	250	0,160	0,091	-1,022	0,174	-844,50	3,670
12	300	0,111	0,083	-1,509	0,160	-1246	3,377
13	300	0,111	0,077	-2,046	0,148	-1690	3,128

• The variation of the se specific values along the collector can be easily detected by representing them in graphs as functions of the order numbering of the confluence node that is associated the number 1, corresponding to the first upstream node. The variation of the local head loss coefficients is presented in the Fig. 12 and the variation of the hydraulic resistance modules, in the Fig. 13.

• The analysis through numerical methods of the value couples represented in Fig. 12, 13, in order to adjust the function for the best approximation, have led (by using the MICROCAL ORIGIN programming environment) to the determination with good precision for practical cases to relations such as (8) and (9) whose coefficients are:

 $M_{r\zeta 10} = 878,77$; $M_1 = 15,40$; $M_2 = 13,37$ (10 a)respectively:

 $M_{r\zeta to} = 458,82$; b = 1,82207(10 b)

• In the case of the tapping front working at nominal capacity $(q \approx 0,008 \text{ m}^3/\text{s}; Q(k)=k.q)$, its lateral head loss are given by the expression:

$$h_{rcl} = 0,763 - 0,00099.k - 0,00086.k^2 \tag{11}$$

where h_{rel} varies between $h_{r\zeta} = 0,76$ m – on the first well pipe connection (P₁) and $h_{r\zeta} = 0,60m - on$ the P₁₃ well pipe connection, whilst the local head loss at confluences that has reduced values (smaller than 5cm), reduces to negligible values for the last confluences.



Fig. 12. The local head loss coefficients at the Motca - Pascani tapping front's confluences



Fig. 13 The local hydraulic resistance modules at the Motca – Pascani tapping front's confluences

4. Conclusions

By determining the local head loss coefficients at the collector's confluences with multiple lateral communications, based on the semi empirical relations *Levin* – *Taliev* and on the associate hydraulic resistance modules, analytical expressions can be established quite precise particularized for each studied system, by using MATHCAD and MICROCAL ORIGIN programming system, those analytical expressions facilitate the analysis of the effective working regime of the system, specifying the influence of the hydraulic resistances. The case study of the tapping front Moţca – Paşcani clearly indicates that – in these specific conditions – the local resistance of the passing through confluence is significantly low, so that it can be neglected, whilst the local hydraulic resistance of the *laterals* at the confluence present significant values that decrease towards the outlet of the collector, and starting from the 6^{th} node they take negative values so a suction effect of the lateral flow by the collector flow is manifesting.

The evolution of the effective values of the head loss coefficients along the collector allows for a more objective reason for its telescoped piping tract.

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Properties of Maneuverability Characteristics Analogies in Irregular Sea

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Abstract: Prognosis regarding the behaviour in irregular sea of the ships is determined by hard calculations. Hence, it is need of analyzing and generalizing the results of a great amount of information corresponding to many characteristics of maneuverability performance and of floating conditions. Therefore, looking for simplification methods of probability calculus and of compact forms of representation is an actual problem. Solving the engineering problems regarding the estimation and analysis of ship's maneuverability qualities in actual sea conditions is substantially simplified in the case of using the analogies peculiar to the ships in actual and irregular sea. In 1972, Netvetaev published a work in which he had approached the problems regarding the rolling motion. In this paper, the method mentioned above is developed concerning the action of irregular waves on the ship

Keywords:

Solving the engineering problems as regards the probability estimation and the analysis of maneuverability qualities in actual sea conditions is simplified by using the similitude properties regarding the behavior in actual and irregular sea of the ships.

We consider the process by a linear differential equations:

$$\ddot{u} + 2\nu_u \dot{u} + n_u^2 u = f_0 sin(\omega_k t + \varepsilon)$$
⁽¹⁾

where:

 $2v_u$ - damping coefficient;

 $n_u-\mbox{natural/characteristic}$ frequency of the system for oscillation u;

 f_0 – complex amplitudes of forces or disturbing moments;

 ω_k - oscillation frequency or meeting frequency of waves.

Using the solution of equation (1), the oscillation amplitude is represented in a general form:

$$u_0 = k(\sigma_k) u_{st}(\sigma) \tag{2}$$

where:

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$$k(\sigma_k) = \frac{f_I(\sigma)}{\sqrt{\left(l - \sigma_k^2\right)^2 + 4v_{0u}^2 \sigma_k^2}} \qquad - \qquad \text{dynamic}$$

coefficient of the system for oscillation u(t);

 $f_l(\sigma) = l$ - for the oscillations considered in the absolute coordinate system;

 $f_I(\sigma) = \sigma^2$ - for the relative oscillations considered in the coordinate system interdependent with the ship's hull as a rigid/hard solid;

 $\sigma = \frac{\omega}{n_u}$ - dimensionless (relative) frequency of the

waves;

 $\sigma_k = \frac{\omega_k}{n_u}$ - dimensionless (relative) frequency of the ship:

 v_{0u} - dimensionless damping coefficient;

 $u_{st}(\sigma) = f_2(\omega)\chi_u(\omega)r$ - movement of the system on the coordinate for the static action of disturbing force; $f_2(\omega) = 1$ - for the linear oscillations of the hull as a rigid solid;

 $f_2(\omega) = \frac{\omega^2}{g}$ - for the angular oscillations of the hull

as a rigid solid;

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 $\chi_u(\omega)$ - reduced coefficient determining the numerical value of the disturbing force on the coordinate u for a given frequency of the wave; r, ω - amplitude and the frequency of the wave.

The expression (2) determines the amplitude – frequency characteristic of the linear dynamic system for the process u(t).

As for the linearization possibility of the equations, the method mentioned can be also developed on the non-linear system. In order to characterize the intensity of the stochastic process for practical purposes, two sizes are necessary: dispersion D_u and the average value of the oscillation frequency $\overline{\omega}_u$. The latter size is determined by the relation between the dispersion of the process u(t) and the speed $\dot{u}(t)$ by the following equality:

$$\overline{\omega}_u = \sqrt{\frac{\dot{D}_u}{D_u}}$$

By applying the normed spectrum of the waves by means of the formula:

$$S_{\zeta}(\omega) = C \frac{m_0}{\overline{\omega}} \cdot \left(\frac{\omega_m}{\omega}\right)^k exp\left[-\frac{k}{n} \cdot \left(\frac{\omega_m}{\omega}\right)^n\right]$$
(3)

where:

$$C = \frac{n}{\Gamma(a)} \cdot \sqrt{\frac{\Gamma(b)}{\Gamma(a)}} \cdot \left(\frac{k}{n}\right)^{k_n}$$
(4)

and by substituting $\omega = \overline{\omega}x$, $\overline{\omega} = \overline{\sigma}n_u$, $\omega_k = \overline{\omega}x_k$, by means of the formula:

$$D_{u} = \int_{0}^{\infty} S_{u}(\chi, \omega) d\omega =$$

$$= \int_{0}^{\infty} \int_{-\frac{\pi}{2}}^{\frac{\pi}{2}} |\Phi_{u}(\chi, \alpha, \omega)|^{2} \cdot S_{\zeta}(\alpha, \omega) d\alpha d\omega$$
(5)

and by the relation (2) the following expression for the dispersion of the process u(t) is obtained:

$$\frac{D_{u}}{D_{\zeta}} = C \left(\frac{\omega_{m}}{\overline{\omega}}\right)^{k} \int_{0}^{\infty} \frac{f_{l}^{2}(x\overline{\sigma}) \cdot f_{2}^{2}(x) \cdot \chi_{u}^{2}(x\overline{\sigma}) \cdot x^{-k}}{(l - x_{k}^{2}\overline{\sigma}^{2}) + 4v_{\partial u}^{2}x_{k}^{2}\overline{\sigma}^{2}} \cdot exp \left[-\frac{k}{n} \left(\frac{\omega_{m}}{\overline{\omega}}\right)^{n} \cdot x^{-n}\right] dx$$
(6)

where ω_m , $\overline{\omega}$ - maximum frequency and the average value of the wave spectrum frequency.

The left side of the equality (6), including the integral expression of the right side with an approximation of a constant factor, which must be determined from f(x) after the process u(t) is put in a concrete form, represents a dimensionless parameter function characterizing the general dynamic properties of the system (χ_u, v_{0u}) , the energetic spectrum shape $(k, n, \omega_m / \overline{\omega})$ and the oscillation conditions $(\overline{\sigma} = \overline{\omega} / n_u, Fr, \chi)$. Hence, under given floating conditions, the dispersion D_u of the stochastic process u(t), induced by the action of the waves having the dispersion D_{ζ} and the average value of frequency $\overline{\omega}$, is determined by the following formulae:

for the linear oscillations of the hull and for the bending moments:

$$D_{u} = D_{\zeta} S_{u}^{2} \left(\overline{\sigma}, Fr, \chi\right) \tag{7}$$

- for the angular oscillations:

$$D_{u} = D_{\zeta} \overline{\omega}^{4} S_{u}^{2} \left(\overline{\sigma}, Fr, \chi \right)$$
(8)

where S_u – relative standard of the process u(t) (is used as a dimensionless characteristic of the oscillation intensity). Instead of S_u , the size $d_u = S_u^2$ can be taken into account, which is the unit dispersion as it determines D_u for $D_{\zeta} = 1$.

For similar dynamic objects, the standards are equal if the dimensionless frequencies $\overline{\sigma}$ are the same, have the values of the ship's relative speeds and have the same course, that is, for the similar objects A₁ and A₂.

$$S_{u_1}(\overline{\sigma}_1, Fr_1, \chi_1) = S_{u_2}(\overline{\sigma}_2, Fr_2, \chi_1)$$
(9)

if $\overline{\sigma}_1 = \overline{\sigma}_2$, $Fr_1 = Fr_2$, $\chi_1 = \chi_2$ Using Rayleigh's law, instead of the relation (7), the relative standard becomes:

$$S_u = 2u_{0p}(\overline{\sigma})/h_p \tag{10}$$

where:

 $u_{0,p}(\overline{\sigma})$ - amplitude of the process u(t);

 h_p – the wave height;

p – probability to exceed the oscillation amplitude.

The relation is true for any value of probability p, therefore it is indicated for determining the relative standard under the conditions in which the sizes $\overline{\omega}/n_u$, Fr and χ are known and for solving the reverse problem – determination of the oscillation amplitude, if p is the probability to exceed the amplitude, under given floating conditions:

$$u_{0p}(\overline{\sigma}) = \frac{1}{2}h_p S_u(\overline{\sigma}) \tag{11}$$

The formula (4) for a proper choosing of the response function:

$$\Phi_u(v) = \int_0^\infty k_u(t) e^{ivt} dt$$

can be applied for the derivatives of the stochastic time process (speed and acceleration).

By applying, in this case, the formula:

$$D_{u}^{(n)} = \int_{0}^{\infty} \int_{-\pi/2}^{\pi/2} \omega^{2n} |\Phi_{u}(\chi,\alpha,\omega)|^{2} \cdot S_{\zeta}(\alpha,\omega) d\alpha d\omega$$

for nth derivative of dispersion of the stochastic process and by repeating the above transformations for speed and accelerations, we obtain:

$$D_{\dot{u}} = D_{\zeta} \overline{\omega}^{2} d\dot{u} (\overline{\sigma}, Fr, \chi)$$

$$D_{\ddot{u}} = D_{\zeta} \overline{\omega}^{4} d\ddot{u} (\overline{\sigma}, Fr, \chi)$$
(12)

Taking into account the well-known relation for the values $\omega^2 = k \cdot r$, the analogous relations for the dispersion of speeds and angular accelerations are obtained:

$$D_{\dot{\psi}} = D_{\zeta} \overline{\omega}^{6} d\dot{\psi}(\overline{\sigma}, Fr, \chi)$$

$$D_{\ddot{\psi}} = D_{\zeta} \overline{\omega}^{8} d\ddot{\psi}(\overline{\sigma}, Fr, \chi)$$
(13)

In the formulae (12), (13) the size $\sqrt{D_{\zeta}}\overline{\omega}^2$ is proportional to the wave slope, the functions $d\dot{u}, d\ddot{u}$, in the analog sense in which the unit dispersion $du(\overline{\sigma})$ was introduced, have the same invariance property related to the dynamic scales of similar objects, because they are defined only by the following characteristics: amplitude, frequency and dimensionless parameters with the same values for different objects by achieving the similitude conditions.

For practical purposes, the graphic representation of dependencies in the form of (10) is interesting, being applicable to different statically characteristics of the random processes.

The diagrams of parametric curve functions $S(\overline{\omega}/n_u)$ for different values of Fr and χ can be considered as diagrams for oscillations deck wetness and the other processes characterizing the behavior of ship in different sea conditions.

Under the formula (10) the diagrams of oscillations can be compared with the amplitude – frequency characteristics of oscillations $\Phi_u(\omega, Fr, \chi)$ which determine the ship's reaction of the irregular wave action. The similitude confirms the likeness of the external form of curves $S_u(\overline{\sigma})$ and $\Phi_u(\omega)$ and the coincidence of the limit values that, corresponding to the character of the process u(t) considered, for very big and small values of the frequency tends to 0 or 1.



Fig.1 – Dependence of pitching average amplitude for a ship L=120,6m as a function of the navigation conditions: Δ - stationary waves; O - developed



Fig.2 – Diagram of pitching oscillations in meeting waves $(s_{\psi} = 0.23\psi_{0p}/h_p\overline{\omega}^2)$. Resulted from the tests on the prototype: O - for a ship with KL=120,6m; • for a ship with L=160m.

As an example, Figure 1 and 2 show the data resulted from the calculations and the diagrams of

transverse oscillations (pitching) of a ship with a designed length L=120,6m at the motion in meeting waves (when the ship proceeds against the direction of waves propagation).

Calculations: Fr=0; fr=0,12; Fr=0,23. The Roman figures show the waves as a function of Beaufort scale.



Fig.3 – Diagrams of pitching oscillations for carriers sailing on meeting waves $(S_{\psi} = 0.23\psi_{0p}/h_p\overline{\omega}^2)$.

Here, as a characteristic frequency, it is admitted the frequency $\omega_L = \sqrt{\frac{2\pi g}{L}}$, in which *L* is the designed length of the hull and, hence, $\frac{\omega_L}{\omega}$ represents the analogous size of the relative length of the wave $\overline{\lambda} = \frac{\lambda}{L}$. The diagram shape changes as a function of the ship's speed and shows that the influence of the

speed on the amplitude, in the case of pitching, depends on the spectral structure of the waves.

For the long-range waves $\left(0,5 < \frac{\overline{\omega}}{\omega_L} < 1,5\right)$ the vertical oscillations increase with the speed increasing and in the case of small periods $\left(\frac{\overline{\omega}}{\omega_L} > 1.6\right)$ they decrease. In a small provimity of

 $\left(\frac{\overline{\omega}}{\omega_L} > 1,6\right)$ they decrease. In a small proximity of

the average periods $\frac{\overline{\omega}}{\omega_L} \approx 1.5$ (degree 4-5 on

Beauford scale) speed has no influence on the vertical oscillations. The influence of speed on pitching is analogous. The diagrams show the fact that the influence of the wave intensity is equally determined by wave intensity is equally determined by the height and the period of waves.

In practice, the frequency interval $\left(0.5 < \frac{\overline{\omega}}{\omega_L} < 2.5\right)$ the pitching amplitudes increase

at the same time with the increasing of waves in real sea conditions that is, at the same time with the increasing of the height and the period of waves; the oscillation amplitudes can increase faster than the height of waves.

For comparison with the data resulted from calculations, Figure 3 presents some results from the tests on the prototypes finding their concordance.

The data presented in Figure 2 as well as other similar data show that among the fundamental construction parameters of the hull, the designed length of the ship, L, has a significant influence on the pitching under real conditions, which is

expressed by the size
$$\frac{\overline{\omega}}{\omega_L}$$

For the same type of ship, with close ratios between dimensions and close inertia radii, the diagrams of oscillations are a little different.

This thing is presented in Figure 3 in which the data resulted from calculations for 10 carriers are also shown. The variation limits of the fundamental elements of these ships are:

length L[m]	
ratio L/B	
ratio L/T	
fineness wa	erline area coefficient S/LB0,65-0,82

block fineness coefficient V/LBT 0,5-0,8 transverse central inertia radius i_v [m] 21-23

By means of the method mentioned above, a wide range of problems can be solved related to the calculation of the pitching oscillations of the ship.

In a similar way, the oscillation can be determined by means of the formula (11). Especially, the intensity of wave oscillations is given in degrees on Beauford scale. In this case, the calculated values of amplitude are determined in the diagram points situated on the middle of the frequency interval corresponding to the degree on Beauford scale which is equivalent to the supposition regarding the equal probability of a great number of independent experiments (fig.4 and 5).



Fig. 4



Fig. 5

Using the oscillation diagrams, the proper comparison of the oscillations of different ships in comparable navigation conditions and the estimation of the influences of the variations of different building elements of the ship on the oscillations, simplify the systematization of the experimental results on the prototype.

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Statistic Estimation of Relative Oscillations of Wave Profile in the Case of Deck Wetness and Slamming

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Abstract: The successful development of calculation probability method of oscillations of ships on irregular waves and of other processes conditioned by them ensures undeniably the prognosis regarding the maneuverability qualities of ships in actual sea conditions. In this paper it is presented the concept combining the traditional method and those of spectral theory of oscillations which is applied to the deck wetness and slamming as being indicators of maneuverability owing to which frequently reach to reduce the ship's speed in storm conditions.

Keywords: probability method; Slamming; Wave.

We'll consider that for estimating the deck wetness possibility it is enough to know the positions of deck stringer plate related to the profile of the waves meeting with the ship's side. In any transverse section of the ship, the change of hull submerged part related to the wave surface as against the smooth water level without taking into account the pitching oscillations is given by:

$$q(t) = \zeta_v - \zeta_g + x\psi \tag{1}$$

where: ζ_{v} - height of meeting wave;

 ζ_g - height of ship vertical oscillations;

x – distance to midship section ψ - trim-angle.



Fig.1 Coordinate systems used for calculating the deck wetness and ship kinematics during the oscillations

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Essentially, the size q(t) represents the wave profile related to the coordinate system interdependent with the ship and the deck wetness condition to the range x from the center of gravity can be written as follows:

$$q(t) \ge H_f^*(x) \tag{2}$$

where:

 $q_0(x)$ - relative oscillation amplitude of wave level; $H_f^*(x)$ - calculated height of ship's emerged side.

In the capacity of $H_f^*(x)$ the effective emerged side is considered that is different from geometrical emerged side $H_f^*(x)$ witch takes into account the wave level in the considered moment and the variation of emerged side height.

$$H_{f}^{*} = H_{f} - \Delta H_{f} - 0.75 Fr^{2} \frac{L}{l_{p}}$$
(3)

where l_p - length of non-cylindrical part of area bow of the hull.

The dynamic effects determined by the ship's oscillations change the bow relative oscillation amplitude because by the bow flooding the water level is raised more than the statical level of the wave.

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From the experimental data on models [8] for the oscillations forced on the smooth water, an empirical formula was proposed for the bow relative oscillation amplitudes.

$$\frac{\Delta q}{q_0} = \frac{\delta - 0.45}{3} \sqrt{\frac{L}{g}} \omega_k, \quad \delta \in (0.60 \div 0.80) \quad (4)$$

where:

$$\sqrt{\frac{L}{g}}\omega_{k} = \sqrt{2\pi} \frac{\omega}{\omega_{L}} \left(l + \sqrt{2\pi}Fr \frac{\omega}{\omega_{L}} \right)$$

and $\omega_{L} = \sqrt{2\pi\frac{g}{L}}$, L – ship designed length.

Taking into account (4) for calculating the deck wetness at forepeak, the relative oscillation amplitude is determined by:

$$q_0^* = q_0 \left(I + \frac{\Delta q_0}{q_0} \right) \tag{5}$$

At fast ships, the greatest dynamic height of the wave level is observed on the theoretical frame lines 2-3 where the water level can be two times higher at the forepeak.

According to condition (2) the probability of wetness is determined by the integral distribution law of random amplitude q_0 :

$$p(q_0 > H_f^*) = l - F(q_0 = H_f^*)$$
 (6)

The dispersion of relative oscillations q(t) is determined as a dispersion of random size sum considered in formula (1):

$$D_q = D_{\zeta} + D_{\zeta_g} + x^2 D_{\psi} - 2R_{\psi\psi_g} + 2xR_{\zeta\psi} - 2xR_{\zeta_g\psi},$$

where: D_{ζ} , D_{ζ_g} , D_{ψ} - dispersion of process produced by waves, vertical oscillations and pitching; $R_{\zeta\zeta_g}$, $R_{\zeta\psi}$, $R_{\zeta_{g\psi}}$ -correlations of processes $\zeta(t)$, $\zeta_{\sigma}(t)$, $\psi(t)$.

The average period of relative oscillations of wave profile calculated by formula:

$$\overline{\omega}_{u} = \sqrt{\frac{D_{u}}{D_{u}}}$$

$$\overline{T}_{q} = 2\Pi \sqrt{\frac{D_{q}}{D_{q}}}$$
(7)

where
$$D_{\dot{q}} = \int_{0}^{\infty} \omega_k^2 |\phi_q(\omega)|^2 S_{\zeta}(\omega) d\omega$$
 (8)

The problem of deck wetness probability consists in determining how often the random size relating to the deck immersion exceeds the freeboard height (3). The wave profile q(t) will be considered related to the dynamic waterline and not related to smooth water waterline and ,accordingly, the process q(t) and the ship's oscillations have zero as an average value and check the law of Gauss. Therefore, the amplitudes of the process q(t), that is, the extreme values between the passing points through the line q(t)=0 comply with the law of Rayleigh.

$$F(q_0) = l - exp\left(-\frac{q_0^2}{2D_q}\right)$$
(9)

Accordingly, the probability of wetness (5)

$$p(q_0 > H_f^*) = exp\left(-\frac{H_f^{*2}}{2D_q}\right)$$
(10)

If it intersects the average number of deck immersion under the wave level in time unit determining the average passing value of the random function through a given level. In this case, the level is $q = H_f^*$ and the formula is:

$$N_m = \frac{\overline{\omega_q}}{2\Pi} \exp\left(-\frac{H_f^{*2}}{2D_q}\right) \tag{11}$$

where $\overline{\omega_q}$ - the average frequency of the relative oscillations of the wave level.

The average period of immersion of a point situated on the deck is determined as being the ratio between the average value of time during which it is checked the condition $q < -H_f^*$ and the average number of immersions N_m during T_0 :

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$$\overline{T_m} = \frac{\pi}{\omega_q} exp\left(\frac{H_j^{*2}}{2D_q}\right) \left[I - \phi\left(\frac{H_f^{*}}{\sqrt{2D_q}}\right) \right]$$
(12)

where $\phi \left(\frac{\Pi_f}{\sqrt{2D_q}} \right)$ Laplace function.

The statistic characteristics of ship flooding mentioned above can be also applied to estimate the uncover of the ship bottom in a given transverse section of the hull. For this purpose, instead of the instead of calculated height of the freeboard H_f^* it must be considered the draught, T(x), measured as a function of ship's trim ψ to the range x from the centre of gravity using the formulae (9)-(12). So, for bottom uncover calculation as well as for subsequent calculated draught on the meeting waves is not considered. The calculation of relative oscillation dispersion and their speed is made considering the designed couple given by the

dynamic correction with the formula (5). Exposing the ship to the deck wetness depends mainly on the relative length of the wave λ_{I} , therefore, for the ships of different lengths sailing, the wetness degree will be different under different wave conditions. A more complete estimation can be obtained considering the deck wetness under the conditions in which the waves change taking into account the probability of appearing this phenomenon in case of navigation in an established geographic area, that is, using the long range forecast method. It is supposed that the wave intensity is characterized by two parameters (the characteristic height and the average value of time) and by distribution the relative oscillation movements under stationary conditions, like the wave height, according to Rayleigh law.

Therefore, under the formula of total probability for long range distributions of relative oscillations, the probability as the amplitude to exceed a given level is expressed by:

$$Q(q_0 > H_f^*) = \int_0^\infty \int_0^\infty P(h_k, \overline{\omega}) exp\left(-\frac{H_f^{*2}}{2D_q}\right) dh_k d\overline{\omega} \quad (13)$$

where $P(h_k, \overline{\omega})$ - probability of appearance the waves with a characteristic height h_k and average period $\overline{T} = 2\pi$

$$T = \frac{2\pi}{\omega}$$
.

The calculations show that the probability of deck wetness depends on the ship length and increases with decreasing of relative freeboard $\frac{H_f}{L}$. The character of this dependence has a small variation related to the variation $\frac{H_f}{L}$.

An interesting feature $Q(q_0) = f(l)$ is represented by the clear presence of the expressed maximum that irrespective of the freeboard height is situated in a small length interval of L length variation.

Experimentally, it was found that in the North Atlantic at the ship depth $(0.04 \div 0.08)$ L the ships with a length between 20-40m are the most liable to flooding.

In figure 2 it is presented the dependence of the bow freeboard height on the ship length as a function of the ship's relative speed obtained by calculations taking into account the long-range distribution of the relative oscillation in theNorth Atlantic.

This dependence corresponds to the probability of deck wetness that is equal with 1%.



Figure2. Dependence of bow freeboard height on the ship length on the ship with a probability of deck wetness of 1% and different speeds.

Calculations:

- for long-range distributions
- according to Newton data[5]
- under the Load-Lines, 1966[4],
- • bulkcarriers
- • fishing vessels

From the interpretation of the diagram in figure2 results that if the length L<100m, then the board height varies the most intensely and in case of big ships the relative oscillation amplitude depends in a small extent on the hull length and the dependence $H_f/L = f(L)$ has a hyperbolic character. Froude number essentially influences on the distribution time of the relative oscillations. In the interval $[50m \div 250m]$ by increasing of Froude number from 0,2 to 0,3 the relative oscillations amplitude increases from 0,8 to 1,2m which imposes the increasing of the freeboard to keep a probability of deck wetness of 1%.

Further increasing of the relative speed to Fr=0,4 imposes a small increasing of the freeboard. As a comparison, Fig.2 presents the similar data proposed by Newton [5] and the data which reflects the designing practice of the sea ships:

- a) dependence of bow minimum freeboard on the ship length under the Load-Lines Rules;
- b) data regarding the freeboard for a number of bulkcarriers and fishing vessels.

The data proposed by Newton doesn't correspond to the calculation results obtained from the relative oscillation distribution. It is evident that it results from the fact that at the estimation of deck

wetness probability, in the work [5], the irregularity of the sea waves concerning the period doesn't take into account and consequently, the action of spectrum short waves isn't considered.

The practical data for designing has a good approximation with the calculated results and confirms the dependence $H_f(L)$ resulting from the consideration

of the ship relative oscillations in an established geographic navigation area.

For a given probability of the deck wetness, the relative height of the freeboard decreases according to the increasing of the ship length; the increasing of the ship is relative speed entails the increasing of the freeboard.

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Extension of Chèzy's Formula for a Variable Section Pipeline (Conical Pipeline)

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Abstract: In this paper, we present the extension of Chèzy's formula from the case of a simple pipeline to the variable section pipeline, by using the impulse variation theorem and Bernoulli's equation. In the final of this paper, we present some graphics concerning the theoretical relations.

Keywords: variable section pipeline (conical pipeline), pressure, hydraulic charge, velocity, conical angle.

1. Introduction

In hydraulic domain, one of the more used relation is Chèzy's formula. For the simple pipeline, this formula is:

$$v = C\sqrt{RJ} \tag{1}$$

where: v is the velocity, C - Chèzy's coefficient, R - hydraulic radius, J - hydraulic slope.

The question is if this relation may be used for the conical pipeline. In this paper, we present Chèzy's formula for this type of pipeline.

2. Mathematical model

To determine Chèzy's formula, we apply the impulse variation theorem to the liquid into the control surface, which is limited by the sections 1 and 2 (fig. 1).



Fig. 1 Calculus schema

We can write the vectorial relation:

$$\vec{P}_1 + \vec{P}_2 + \vec{F}_f + \vec{G} + \vec{R}_c = \vec{I}_2 - \vec{I}_1$$
(2)

where: P₁, P₂ - the hydrostatic forces exert by the liquid bordered to the left, respective to the right on the liquid limited by the control surface; F_f - the friction force of the liquid by the pipeline wall; R_c - the reaction of the liquid into the control volume; I₁, I₂ - the impulse of the discharge, $I = \beta \rho Q v = \beta \rho \frac{Q^2}{A}$

We project the relation (2) on the axes Ox and Oy (where the horizontal axis Ox coincide with the pipeline axia, Oy is normal to the pipeline axis), and we obtain:

$$Ox: P_1 - P_2 - F_{fx} + R_{cx} = I_2 - I_1$$

$$Oy: R_{cv} - G = 0 \Rightarrow R_{cv} = G$$
(3)

We note τ_x - the tangential average effort at the wall in the section x, the horizontal component of the friction force is done by the relation:

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$$F_{fx} = \frac{\tau_x + \tau_{x+\Delta x}}{2} \pi \frac{D_x + D_{x+\Delta x}}{2} \frac{\Delta x}{\cos \theta} \cos \theta =$$

$$= \frac{\tau_x + \tau_{x+\Delta x}}{2} \pi \Delta x \frac{D_x + D_{x+\Delta x}}{2}$$
(4)

where: θ - conical angle; D_x , $D_{x+\Delta x}$ - the pipeline diameter in the section with x abscise, respective $x+\Delta x$.

By replacing $D_{x+\Delta x} = D_x + 2\Delta x t g \theta$, the relation (4) can be write:

$$F_{fx} = \frac{\tau_x + \tau_{x+\Delta x}}{2} \pi \Delta x \Big(D_x + \Delta x tg \theta \Big)$$
(5)

The next relation does the horizontal component of the reaction of the pipeline wall:

$$R_{cx} = \frac{p_x + p_{x+\Delta x}}{2} \pi \frac{\Delta x}{\cos \theta} \sin \theta \left(\frac{D_x + D_{x+\Delta x}}{2}\right)$$
(6)

We exprime the pressure in the section $x + \Delta x$ function by the pressure in the section x $p_{x+\Delta x} = p_x + \frac{\partial_x}{\partial x} \Delta x$ and we obtain: $R_{cx} = \left(p_x + \frac{1}{2}\frac{\partial p}{\partial x}\Delta x\right) \pi \Delta x tg \theta \left(D_x + \Delta x tg \theta\right)$ (7)

We replace the relations (5) and (7) in the relation (3):

$$p_{X}A_{X} - p_{X+\Delta X}A_{X+\Delta X} - \frac{\tau_{X} + \tau_{X} + \Delta x}{2} \Delta x \pi \left(D_{X} + \Delta x t g \theta \right)$$
$$+ \left(p_{X} + \frac{1}{2} \frac{\partial p}{\partial x} \Delta x \right) \Delta x \pi t g \theta \left(D_{X} + \Delta x t g \theta \right) =$$
$$= \beta \rho Q^{2} \left(\frac{1}{A_{X+\Delta X}} - \frac{1}{A_{X}} \right)$$
(8)

By introducing the expression of the circular

section area $A_x = \frac{\pi D_x^2}{4}$, $A_{x+\Delta x} = \frac{\pi D_{x+\Delta x}^2}{4}$, the relation (8) becomes:

$$p_{x} \frac{\pi D_{x}^{2}}{4} - \left(p_{x} + \frac{\partial p}{\partial x}\Delta x\right)\pi \frac{\left(\pi D_{x} + 2\Delta x tg\theta\right)^{2}}{4} - \frac{\tau_{x} + \tau_{x} + \Delta x}{2}\Delta x \pi \left(D_{x} + \Delta x tg\theta\right) + \left(p_{x} + \frac{1}{2}\frac{\partial p}{\partial x}\Delta x\right)\Delta x \pi tg\theta \left(D_{x} + \Delta x tg\theta\right) =$$

$$= \frac{4}{\pi}\beta\rho Q^{2} \left(\frac{1}{D_{x+\Delta x}^{2}} - \frac{1}{D_{x}^{2}}\right)$$
(9)

We divide the relation (9) by Δx and we make the limit $\Delta x \to 0$. In this case $\Delta p_x \to 0$.

$$\tau_{0x} = -\frac{dp_x}{dx}\frac{D_x}{4} + \frac{16}{\pi^2}\frac{\rho\beta Q^2}{D_x^4}tg\theta$$
(10)

We use the Bernoulli's relation:

$$H_{0} = \frac{\alpha_{x} v_{x}^{2}}{2g} + \frac{p_{x}}{\gamma} + h_{r_{0-x}}$$
(11)

The derivation of the relation (11) by x is:

$$\frac{dp_x}{\gamma dx} + \frac{dh_r}{dx} = \frac{\alpha Q^2}{g A_x^3} \frac{dA_x}{dx}$$
(12)

By replacing the relation (12) in (10), we obtain the relation for τ_x :

$$\tau_{0x} = \gamma \frac{dh_r}{dx} \frac{D_x}{4} - \gamma \frac{\alpha Q^2}{gA_x^3} \frac{D_x}{4} \frac{dA_x}{dx} + \frac{16}{\pi^2} \frac{\rho \beta Q^2}{D_x^4} tg\theta$$
(13)
The derivative of the section area in comparation with x is:

$$\frac{dA_x}{dx} = \frac{d}{dx} \left(\frac{\pi D_x^2}{4} \right) = \frac{\pi}{4} 2D_x \frac{dD_x}{dx} =$$

$$= \frac{\pi D_x}{2} \frac{d(D_0 + 2xtg\theta)}{dx} = \pi D_x tg\theta$$
(14)

We consider $\alpha = \beta \cong 1$ and the expression of τ_x becomes:

$$\tau_x = \gamma \frac{dh_r}{dx} \frac{D_x}{4} \Rightarrow \frac{\tau_x}{\gamma} = R_x J_x \tag{15}$$

where: R_x is the hydraulic radius in the section x, J_x - the hydraulic slope in the section x.

By maintaining the hypothesis which is the base of Chèzy's formula for the simple pipeline, for the conical pipeline this hypothesis is:

$$\frac{\tau_x}{\gamma} = R_x J_x = \frac{v_x^2}{C_x^2} \tag{16}$$

We obtain a formula, which has the same structure like Chèzy's formula for the simple pipeline, by difference, that all terms are functions the section position, respective they depend of the coordonate x:

$$v_x = C_x \sqrt{R_x J_x} \tag{17}$$

where C_x is the local Chèzy's coefficient of the hydraulic resistance.

Using Pavlovsky's formula for the Chèzy's coefficient we may also write:

$$v_x = \frac{1}{n} R_x^{y+0.5} J_x^{0.5} \tag{18}$$

3. Numerical example

We consider a conical pipeline with different conical angle ($\theta = -7^{\circ}, -5^{\circ}, -3^{\circ}, -1^{\circ}, 1^{\circ}, 3^{\circ}, 5^{\circ}, 7^{\circ}$) and

we represent the relation between v_x/v_0 and $\Delta x/D_0$ for different values of $\Delta x/D_0$. We use the relation:





Fig. 2a The variation of v_x/v_0 function $\Delta x/D_0$ for positive conical angle (θ >0)



Fig. 2b The variation of v_x/v_0 function $\Delta x/D_0$ for negative conical angle ($\theta < 0$)

From this graphics (fig. 2a and fig. 2b) results that such conical pipelines can not be long, because the mean velocity in the final section becomes very different compare to the mean velocity in the initial section (very small for $\theta > 0$ and very big for $\theta < 0$). This conclusion confirms our expectations.

4. Conclusions

Chèzy's formula for the simple pipeline can be used in the case of the conical pipeline, considering all terms of the formula function the section position.

To verify, in the future, the theoretical relations we realise in the laboratory an special installation with a conical pipeline which has the angle θ =2,8°.

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The Hydraulic Calculus for Parabolic Section Culvert in Steady Uniform Regime

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Abstract: : The paper presents the equivalence problem of the natural culvert cross section with a parabolic cross section. This equivalence allows a more compelling treatment in the way of expressing the section characteristics in hydraulic calculus as global analytic relation rather than by decomposing the cross section in component regions and getting the global characteristic by summing these partial values. The analysis is for steady regime.

Keywords: Parabolic culvert, aria of the cross section, wet perimeter, hydraulic radius.

1. Introducere

Concerning. the hydraulic calculus for natural or artificial prismatic culverts, with parabolic cross section in technical specialty literature, the parabolic sections appear only as sections depending on one parameter, which intercede in parabola equation, $x^2 = 2py$, sections which have a calculus relationship for critical depth.

The paper considers the case of the section expressed by a relationship that contains two n = 2

parameters, respectively,
$$z = \frac{P}{h}x^2 + 2qx$$

resulting in a substantial improvement of the achievement of the required conditions for section equivalence.

The equivalent cross section of the parabolic culvert is defined as the section, in witch the water, in steady flow allows the same discharge (Q), equal to the discharge in natural culvert, realizing in the same time, the equality conditions of the culvert cross section area, of the surface width, of the maximal depth of water, of the slope and the of mean velocity.

Then is not strict equality between the wet perimeter, the hydraulic radius and the roughness coefficient.

2. The determination of the parabola parameter

Through relatively laborious calculus, we obtain relationships that allow to express the two coefficients of parabolic equation (p and q) as functions of these four parameters, that globally characterize the section of the natural culvert (aria of the cross section, wet perimeter, bottom width, maximal depth).

The equation of parabol is:

$$z = \frac{p}{h}x^2 + 2q|x| \tag{1}$$

If we introduce in this equation $x = \frac{B}{2}$, it results

$$z = h$$

$$pB + 4hq = \frac{4h^2}{B} \tag{2}$$

By integration of the culvert equation, we have successively obtained:

$$A = Bh - 2\int_{0}^{B/2} \left(\frac{p}{h}x^{2} + 2qx\right) dx$$

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$$A = \frac{12Bh^2 - pB^3 - 6qhB^2}{12}$$

We obtain the second equation

$$pB + 6qh = \frac{12h(hB - A)}{B^2}$$
(3)

With the last two equations we can explain the parameters p and q as function of three parameters (aria of the cross section, surface width, maximal depth):

$$q = \frac{2(2hB - 3A)}{B^2} \tag{4}$$

$$p = \frac{12h(2A - hB)}{B^3} \tag{5}$$

For

And
$$\frac{dz}{dx}\Big|_{x=0} > 0 \Rightarrow q > 0 \Rightarrow A < \frac{2hB}{3}$$

 $\frac{d^2 z}{dx^2} > 0 \Longrightarrow p > 0 \Longrightarrow A > \frac{hB}{2}$

This condition for the aria of the cross section results in:

$$\frac{hB}{2} < A < \frac{2hB}{3}$$
(6)

We also determin the wet perimeter by integration:

$$P_{u} = 2 \int_{0}^{B/2} \sqrt{1 + \left(\frac{2p}{h}x + 2q\right)^{2}} dx$$
(7)

$$P_{u} = \frac{h}{p} \left[\left(\frac{pB}{2h} + q \right) \sqrt{1 + 4 \left(\frac{pB}{2h} + q \right)^{2}} - q \sqrt{1 + 4q^{2}} + \frac{1}{2} \ln \frac{\frac{pB}{h} + 2q}{2q + \sqrt{1 + 4 \left(\frac{pB}{2h} + q \right)^{2}}}{2q + \sqrt{1 + 4q^{2}}} \right]$$

We note with h_* - hydraulical depth of the section

$$h_* = \frac{A}{B} \tag{8}$$

$$P_{u} = \frac{1}{24(2h_{*}-h)} \bigg[4(3h_{*}-h)\sqrt{B^{2}+16(3h_{*}-h)^{2}} - 4(2h-3h_{*})\sqrt{B^{2}+16(2h-3h_{*})^{2}} + \frac{1}{2} \bigg]$$

$$+B^{2}\ln\frac{4(3h_{*}-h)+\sqrt{B^{2}+16(3h_{*}-h)^{2}}}{4(2h-3h_{*})+\sqrt{B^{2}+16(2h-3h_{*})^{2}}}\right] (9)$$

In order to equalise the average velocities and respectively the discharges, considering the Chezy and Pavlovski formulas for steady uniform movement we may write:

• for the natural cross section,

$$v = C\sqrt{Ri} = \frac{1}{n}R^{y+0.5}i^{0.5}$$
,

for the equivalent cross section

$$v = C_e \sqrt{R_e i} = \frac{1}{n} R_e^{y+0.5} i^{0.5},$$

Results the roughness coeficient of equivalent parabol

$$n_e = n \left(\frac{R_e}{R}\right)^{y+0.5} = n \left(\frac{P_u}{P_{ue}}\right)^{y+0.5}$$
(10)

The aim is to obtain an adimensional relationships to characterise the simetrical parabolic cross section, so we write:

• the relative wet perimeter:

$$\frac{P_u}{B} = \pi \tag{11}$$

• the relative depth:

$$\frac{h}{B} = \chi \tag{12}$$

• the relative hydraulic depth:

$$\frac{h_*}{B} = \chi_* = \frac{A}{B^2} \tag{13}$$

• the relative hydraulic radius:

$$\rho = \frac{R}{B} = \frac{A}{P_u B} = \frac{h_*}{P_u} = \frac{\chi_*}{\pi}$$
(14)

$$\pi = \frac{1}{6(2\chi_* - \chi)} [(3\chi_* - \chi)\sqrt{1 + 16(3\chi_* - \chi)^2} - (2\chi - 3\chi_*)\sqrt{1 + 16(2\chi - 3\chi_*)^2} + (15)]$$

$$+\frac{1}{4}ln\frac{4(3\chi_{*-\chi})+\sqrt{1+16(3\chi_{*}-\chi)^{2}}}{4(2\chi-3\chi_{*})+\sqrt{1+16(2\chi-3\chi_{*})^{2}}}\right]$$

3. Examples









4. Conclusions

In this paper there are presented the relationships for the equivalence of the basic amounts of the mean velocity and discharge calculus of a natural culvert (roughness coefficient, water depth, width surface, slope, wet parameter and hydraulic radius) with the corresponding amounts of a simetrical parabolic culvert. The relationships we obtained are applied in same certain situations.

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Contributions to the Generalization of the Economical Calculus for The Pressurised Fluid Pipes

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Abstract: There is showed the hydraulic of pressurised fluid pipes from an energetical / economical viewpoint, proposing original and relevant performance indicators, having a high degree of generalisation. The proposed analytical formulae highlight the decisive weight of the diverse influence factors, which permit a justified adoption of the measures for optimisation of the energetical fluids transport pipes, contributing to their economical design and to make technical interventions with advantageous results as concerns the working performances increasing.

Keywords: Hydraulic systems, pressurized fluid pipes, economical calculus.

1. Introduction

In accordance with the reference material [1], the economical calculus consists in the determination of the circular pipe diameter, without the explanation of the working conditions that provide a given flow in the best economical conditions. The economical diameter corresponds to the solution for that the sum of the yearly investments and exploitation expenses is minimum.

In the papers [2] and [5] the authors realised an energetic approach of the pressurised fluid pipes, by fetching original contributions proposing relevant indicators that suggest the best economical solutions. This research is based on some equations from the reference material [1].

In the technical calculus is easy to write the flow as:

$$Q = \left(AC\sqrt{R}\right) \cdot \sqrt{I} = K \cdot \sqrt{I} , \qquad (1)$$

where:

$$K = AC\sqrt{R}$$
(2)

is the flow modulus and depends by the geometrical elements of the pipe section (A) only, because the coefficient *C* is dependent of the hydraulic radius, *R*

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and the inner wall pipe rugosity, being constants for the same conduit.

The relationship between K, C and λ (Darcy's coefficient of the linear specific hydraulic looses is:

$$C = \sqrt{\frac{8g}{\lambda}} \,. \tag{3}$$

We can write:

$$h_r = \frac{LQ^2}{K^2} \tag{4}$$

where $\frac{L}{K^2}$ is the specific hydraulic resistance of the

conduit having the length L and the diameter D, that is the necessary pressure to overcome the flow viscous resistance for an unitary flow rate.

In the case of the cast iron conduits, for the turbulent flow regime is known the relation:

$$K = 6.955 \cdot D^{2.5} \cdot \left(a + \log D\right) \quad \left[\frac{m^3}{s}\right], \tag{5}$$

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where we can substitute D = 4R, the rugosity coefficient *a* having the values 4.53, 3.91 and 3.35 for the clean, normal and respectively dirty conduits.

From the semi-empiric solutions of the turbulent flow in rugged conduits and open channels we reproduce:

$$C = \frac{l}{n} \cdot R^{y}, \tag{6}$$

recommended for R < 0.1 m, that is, for circular conduits, D < 400 mm and the conventional rugosity (experimentally determined) $0.011 \le n \le 0.014$.

The exponent:

$$y = 2.5 \cdot \sqrt{n} - 0.13 - 0.75\sqrt{R} \cdot \left(\sqrt{n} - 0.1\right), \tag{7}$$

or, simplified, $y = 1.5^{\circ} n$.

In the case of the viscous fluids:

$$Q = m \cdot \sqrt{D^5 \cdot I} , \qquad (8)$$

where $I = \frac{h_r}{L}$ is the hydraulic gradient and *m* is a flow coefficient experimentally determined having

flow coefficient, experimentally determined having, for example, the values 4.6, 18.4 and 23, for fuel oil, crude oil and respectively kerosene.

From the above equation we can establish the following relationship:

$$K = m \cdot \sqrt{D^5} , \qquad (9)$$

$$m = \frac{\pi C}{8},\tag{10}$$

and

$$m = \frac{\pi}{8} \cdot \sqrt{\frac{8g}{\lambda}} \,. \tag{11}$$

2. A generalised economical calculus using the hydraulic radius

We define the yearly investment expenses:

$$A = C_A \cdot R^2, \tag{12}$$

where:

$$C_A = 221.3 \cdot a \cdot c_{aux} \cdot c_A \cdot \rho_c \cdot L \cdot \frac{p}{\sigma}$$
(13)

and yearly exploitation expenses:

$$B = C_B \cdot R^{-5}, \tag{14}$$

where

$$C_B = 7.8 \cdot 10^{-6} \cdot n \cdot c_B \cdot \rho \cdot \lambda \cdot L_{ech.tot.} \cdot Q^3 \qquad (15)$$

From the condition for minimise:

$$\frac{\partial}{\partial R} (A+B) = 0,$$

we obtain:

$$R_{ec} = 9.66 \cdot 10^{-2} \cdot \left(\frac{n \cdot c_B \cdot \rho \cdot \lambda \cdot Q^3}{a \cdot c_{aux} \cdot c_A \cdot \rho_c \cdot L \cdot \frac{p}{\sigma}} \right)^{\frac{1}{7}}$$
(16)

The symbols of the magnitudes are:

 $\gamma = \rho \cdot g$ and $\gamma_c = \rho_c \cdot g$ are the specific weight of the fluid and pipe material;

 $\boldsymbol{\sigma}$ - the stress resistance coefficient of the pipe material;

p - inner calculus pressure;

a - yearly repayment quota;

 C_{aux} - auxiliary expenses coefficient;

n - number of the yearly working hours;

 c_A - the specific expenses of the conduit (related to a kilogram);

 c_B - the specific expenses of the energy (related to a kWh).

From the hydraulic radius definition:

$$R = \frac{A}{P} = \frac{S}{C},\tag{17}$$

(A = S - the cross section area, P = C - the wetting perimeter) and obtaining R_{ec} from the equation (16) we can obtain an optimal design of the conduit.

3. A generalised economical calculus using the flow module

On define the yearly investment expenses:

$$A = C_A \cdot K^{4/5}, \tag{18}$$

where:

$$C_A = 15.404 \cdot a \cdot c_{aux} \cdot c_A \cdot \rho_c \cdot L \cdot \frac{p}{\sigma} \cdot m^{-\frac{4}{5}}$$
(19)

and the yearly exploitations expenses:

$$B = C_B \cdot K^{-\frac{23}{5}},$$
 (20)

where:

$$C_B = 7.95 \cdot 10^{-3} \cdot n \cdot c_B \cdot \rho \cdot \lambda \cdot m^2 \cdot L_{ech.tot.} \cdot Q^3 \qquad (21)$$

From the condition

$$\frac{\partial}{\partial K} (A + B) = 0$$

we obtain:

$$K_{ec} = 1.3383 \cdot \left(\frac{C_B}{C_A}\right)^{\frac{5}{27}},\tag{22}$$

where $L_{ech.tot.}$ is the overall equivalent length (the local hydraulic resistances are tantamount to equivalent length) and *m* is the quality dimensional coefficient of the inner conduit surface, that appear in equations (8)...(11).

The equation (22) allows a strongly generalisation of the economical calculus, in

connection with, evidently, the hydraulically calculus formulae (1)...(11).

The calculus is iterative (using successively approximations can be easy runned by a computer): in connection with m (depending directly or indirectly by R) - excepting the case of the viscous fluids, (8) - and with l, $L_{ech.tot}$, too.

4. The grapho-analytical method

This method can offer some facilities for design, orientation or comparative estimate. By a graphical summation we can obtain the curve (A+B), that presents a minimum in the intersection zone of the curves A and B.



Figure 1. The graphical method pinciple

Figure 1 presents the graphical method principle, the independent variable being K, R or D (particular case). The best economical values corresponds to the minimum of the sum curve.

5. Observations and conclusions

The magnitudes K_{ec} and R_{ec} from the equation (16)and (22) aided by (12)...(15), respectively (18)...22) and in connection with the hydraulic formulae (1)...(11) are the characteristic and basic

energetical-economical performance indicators, having a higher degree of generation /complexity/ integrated synthesis than other current theories from the references.

The economical performances increase of the pipes for power fluid transport first require the decrease of the correlated hydraulic resistances.

The proposed relations spot lights the decisive weight of the diverse influence elements: the cross section geometry/ particularly the diameter having explicitly and implicitly, too, an essential importance.

We suggest the correlation of the above study with references [2] and [3] where is defined the fluid carry efficiency.

The proposed original formulae allow a justified adoption of the optimal measures for the energetical carrying fluids and contribute to theirs optimal/economical design ad technical interventions having avatageous results as regards the working performances increasing.

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On The Hydraulic Calculation of Semiforced Culverts

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Abstract: The paper presents rapid and economical solutions for the hydraulic design that ensure the optimum flow at a maximum culvert capacity. The detailed analysis of various types of water flow through pipe culverts and of decisive factors is realised. Then, the study is directed to the rectangular pipe culverts with semiforced flow. In order to design the culvert outlet, computer programs have been created and corresponding graphics have been plotted.

Abstract: Se analizează aspectele hidraulice ale curgerii apei prin podețe tubulare și factorii determinanți. Studiul se concentrează asupra proiectării hidraulice optime și economice pentru podețele tubulare cu curgere semiforțată având secțiunea transversală dreptunghiulară. Pentru stabilirea gabaritului podețului ținând seama de cât mai mulți parametri hidraulici, geometrici și de amplasament, s-a conceput o metodă de calcul analitic și grafic.

Keywords: Culvert outlet, semiforced pipe culvert.

1. Fundamentals

Culverts - the most frequent works of art that cross valleys and short watercourses – are meant to ensure the waters' flowing downunder a railway or a road. On an average, they represent 65% of the total length of the crossing realized through bridges and culverts, which determines the high cost of the communication ways. For this reason, it is necessary to design the culverts according to economic feasibility criteria both in the constructive solution analysis and in the process of establishing of the culvert outlet.

According to the height of the road embankment (h_E), Fig.1, culverts can be divided into: *twin slab culverts* (*or surface culverts*) if $h_E = 0$ [1] and *pipe culverts*, if $h_E \ge 0.50$ m. Pipe culverts represent a simple and more economical way of crossing watercourses, suitable for low water discharges.

According to the transverse section form, Fig. 2, pipe culverts can be classed into: rectangular, circular, ovoid, other forms. In order to use the whole potential of the pipe's cross section, to diminish the hydraulic resistances at the culvert entrance and to protect the embankments from

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water erosion, parallel works meant to guide the flow towards the culverts are necessary. These works implicitly modify the velocity coefficient (φ) and the contraction coefficient (ϵ).

The hydraulic calculation of the pipe culverts is made according to the hypothesis assuming the existence of a prismatic river bed $(\partial \omega / \partial L = 0)$.

According to the upstream water depth, pipe culverts fall into several categories: *free pipe culverts*, if H < (1.2 - 1.4) A and *pressure flow pipe culverts*, if H > 1.2 A. Both categories have a water races transition system which depends on the various hydraulic, geometrical and constructive parameters characterizing the construction.

The types of flow and different kinds of level water races transition at free flow pipe culverts (Fig. 3) are:

1.1. nonsubmerged free flow, when $h_d < (1.2 - 1.4)$ $h_{cr} + i_{cr} (L - l_c)$ characterized by two forms of water

races transition, realized through:

a. backwater curve, if $h_d \leq h_{cr}$; in this case, the design depth will be considered equal with the critic depth $h_{calc} = h_{cr}$;

b. free hydraulic jump, if $h_d > h_{cr}$; the design depth will be $h_{calc} = h_d$;

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Fig. 1. Twin slab culverts and pipe culverts

Fig. 2. Rectangular culverts





a– nonsubmerged; backwater curve; b– nonsubmerged; free hydraulic jump; c– submerged; submerged hydraulic jump; $l_c = (1.5 \div 2.5)(H - h_c) =$ vena contracta position; $l_{ef} =$ effective zone ; $l_e = (2 \div 2.5)(h_{cr} - h_d) =$ outlet zone; $h_c =$ vena contracta depth; $h_d =$ downstream water depth.

1.2. submerged free flow, when $h_d > (1.2-1.4) h_{cr} + i_{cr} (L - l_c)$. In this case, the transition is realized through a submerged hydraulic jump and the design depth is $h_{calc} = h_d$.

Hydraulically, free pipe culverts function like broad sill weirs with lateral contraction, whose sill height is considered to be null. The hydraulic design of the free pipe culverts relies on the hypothesis that the culvert slope is equal with the critical slope $(i_0 = i_{cr})$.

According to the coefficient of admission, the *pressure flow culverts* may be *forced*, when $h_{calc} = A$ and *semi-forced*, when $h_{calc} < A$. The downstream water

depth influences the water flow through *forced culverts* and engenders two forms of outlet: *nonsubmerged outlet*, if $h_d < A$ and *submerged outlet*, if $h_d > A$.

Forced culverts are characterized by hydraulic conditions analogous to those of the nozzles, but differing from these by the fact that the head loss is taken into account. The types of flow and water races transition forms of the *semi-forced* culverts depend on the flow state (Fig.4):

1.3. as far as *the quiet state* is concerned (when $i_0 < i_{cr}$), there are three forms of open levels:

a) backwater curve (when $h_d < h_{cr}$), for which $h_{calc} = h_{cr}$;

b) free hydraulic jump (when $h_d < h_c$ "), for which $h_{calc} = h_{cr}$;

c) submerged hydraulic jump (when $h_d > h_c$ "), for which $h_{calc} = h_d$.

1.4. *the rapid state* (when $i_0 > i_{cr}$); the open level is a downstream decreasing curve. In this case, $h_{calc} = h_0$, where h_0 represents the depth of the normal water level corresponding to i_0 .

Semi-forced pipe culverts work similarly to big orifices.



Fig. 4. Semi-forced pipe culverts a - backwater curve; b - free hydraulic jump; c - submerged hydraulic jump; d - rapid state

2. Semiforced rectangular culverts

The stream occupies the whole pipe section only at the entrance; as for the rest of the construction, the flow is characterized by an open level and the stream occupies just 50–60% of the cross section. The contracted section is located in the vicinity of the entrance and has the depth h_c :

$$h_c = \varepsilon_V A \tag{1}$$

in which ε_V represents the vertical contraction coefficient, Table 1.

The hydraulic design is based on Bernoulli's relation and on the calculation relations of the large orifices. The Bernoulli's relation is applied between the upstream section (of the depth H) and the contracted section (of the depth h_c), considering the Coriolis coefficient $\alpha = 1$.



Fig. 5 Diagram for preliminary design of semiforced culverts a.- logarithmic scale, b.- decimal scale



Fig. 6 Design of the semiforced rectangular culverts' cross section

	φ				
Entrance type		Free	Semiforced	$\epsilon_{\rm V}$	
		culvert	culvert		
Simple	0.80	0.85	0.60	0.60	
tympanum	0.80	0.85	0.00	0.00	
Cone quarters	0.85	0.90	0.64	0.64	
or wings	0.85	0.90	0.04	0.04	
Hydraulic	0.95	1.00	0.65	0.65	
tympanum	0.95	1.00	0.05	0.05	

Table 1.	Velocity coefficient (φ),contraction
	coefficients (ϵ, ϵ_V) of pipe culverts.

So, we finally get to the mathematical formulae necessary for determining the velocity and the water discharge. Thus, velocity can be written as:

$$v = \varphi \sqrt{2 g \left(H - \varepsilon_V A\right)} \tag{2}$$

and the culvert letting out capacity is given by the relation:

$$Q = \varepsilon_c \ \omega \ v = \mu \ A \ B \ \sqrt{2 g \left(H - \varepsilon_V \ A\right)}$$
(3)

The water depth at the culvert entrance can be derived from the eq. (3):

$$H = \frac{Q^2}{2g\,\mu^2 A^2 B^2} + \varepsilon_V A \tag{4}$$

In order to design semiforced rectangular culverts a program based on the equations (2), (3), (4) has been devised. The diagram illustrated by the figures 5 and 6 can be used for the same purpose if $\varepsilon_V = 0.6$.

3. Conclusions

- The reviewed theoretical analyses have suggested that it is possible to improve the existing design methods of pipe culverts.
- The computational programs and the diagrams introduce explicitly many parameters of flow.
- The efficiency of these methods was confirmed by verifications on some existing culverts.

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Monitored System for Hyperbaric chambers

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Abstract: During diving activities in hyperbaric chambers, all the gas parameters must be controlled and monitored. For this objective it was designed and realized a PC-based data acquisition system, which displays and records values of pressure and temperature of the gas used for the diving. There are also displayed the concentrations of oxygen and carbon dioxide. Data are converted in MS Access database and MS Excel files.

Keywords: hyperbaric chambers, data acquisition, breathing gases.

1. Introduction

In the Hyperbaric Laboratory from the Romanian Navy Diving Center there are performed simulated dives for scientifically research or for divers training purposes.

Dives are performed in a diving system which consists of two hyperbaric chambers and a wet simulator. The environmental conditions created here try to simulate as much as possible the real environment from the sea. There are considered: gas pressure in chambers, composition of the breathed gas, its temperature and water temperature in the simulator.

The diving activity is controlled and monitored from a central control panel. Here are displayed pressure within the chambers, oxygen and carbon dioxide concentrations, temperature on floor and ceiling levels, and the water temperature.

At the present moment pressure information is displayed on the panel by mechanical pressure gauges and by digital displays connected to the existing pressure transducers. The oxygen and carbon dioxide concentrations are measured by specialized analog gas analyzers. Temperature values are displayed digital.

Because the central electronic measuring and recording system for gas parameters is non-functional, it was decided its replacement with a modern PC-based data acquisition system. This is the system that will be presented here.

2. Presentation of the monitored system

2.1 Hardware

The system consists of:

- 3 pressure transducers
- 6 temperature sensors
- signal conditioners
- data acquisition board
- PC
- customer software

Data concerning gas concentrations comes from the specialized analyzers. System configuration schematic is shown in Fig.1.



Fig.1 Schematic of the PC based data acquisition system

Data is acquired from three specific places: hyperbaric chamber, personnel transfer chamber and wet simulator.

The parameters variation can be considered slow so that the scanning rate is one per second.

The measurement accuracy is given by sensors accuracy, which is $\pm 0.1\%$ FS for pressure transducers, and $\pm 0.1^{\circ}$ C for temperature sensors, RTD type. Another important factor for accuracy is the data acquisition board resolution and accuracy. For first tests it was used a 12 bit AD/DA ISA bus board. For final solution it is considered to be used a better board, 16 bit - PCI bus with improved capabilities. **2.2.** Software-Fig.2 The panel of the monitoring program

The software program for this application was written in Visual Basic. Two main programs were developed. First of them controls the data acquisition board, the reading, displaying and recording of the data. The scanning event is triggered by the Visual Basic control Timer which is set to 1 second interval. The board used for tests does not have trigger mode.

At start, the program creates a database and three binary files for data recording. Then checks conditions required by the board – address, existence, etc. As an event based program, it waits then for commands.

The second program transfers data acquired to a database.

The main form of the monitoring program is shown in Fig.2.



Fig.2 The panel of the monitoring program

Data sampled form hyperbaric chambers is displayed on the form for the selected chamber and can be recorded for all of them or individually.

Gas concentration data is read and recorded from the desired chamber. It is available an alarm level setting.

All the channels are scanned each second. Data can be recorded, on desire, between 1 sec and 10 minutes intervals. For each chamber are recorded: time, depth (pressure), two temperatures, oxygen and carbon dioxide concentrations. A saturation diving could last 10 days so the amount of data could be very large: 14 records/second for 10 days. This is why the primary data is recorded in 3 binary files, corresponding to each chamber.

For the reading of the data it was written a program which converts the three binary files into an Access Database, with three tables, and/or three Excel files. In this way data can be analyzed with a good mathematical support. The result is shown in Fig.3 and 4.

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Fig.3 Presentation form for data transfer program

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Fig.4 Access database and Excel Sheet created for data presentation

3. Conclusions

The opportunity and utility of the PC-based data acquisition systems are no longer in doubt. Implementation of such a system in the Hyperbaric Laboratory could represent the first step for a complete automation of the diving's control.

Due to the flexibility of these systems, offered by the modular configuration and the ease of writing dedicated software programs, the range of applications can be extended to the measurement of other process parameters.

Records of all the physical parameters involved in a diving process are extremely useful for further analyses and studies.

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Fluid Suction into the Conduit of Flow-Meter Calibration Stands

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Abstract: One presents the theoretical and experimental research concerning the steady and with axial symmetry suction flow of the inviscid fluid in a conduit, as a scientifically support for the Romanian Patent nr.103332/1998 entitled *Method and device for air calibration of gas flow-meters* experimented in the Laboratory of *Flow-meters, calibration Stands and scale Conversion for different Liquids and Gases* in POLITEHNICA University. In the specific boundary conditions one studies the numerical integration of Euler nonlinear equations, available for small velocities of the fluid at the long distances and in the pipeline entrance.

By a convenient variable change, and for a certain value of the angle $\theta = \pi/2$ on the symmetrical direction of flow, one succeeds to transform the nonlinear system of partial differential equations written in spherical coordinates into the Cartesian coordinates, propre to the pipe boundary conditions.

Keywords: Flow-meters, calibration stands, numerical integration, Euler's equation system.

1. Introduction. The method of the constant velocity section (CVS)

As a result of a varied laboratory activity carried on a long period, concerning the testing on water and air of different liquid and gas flow-meters and the construction of their stands for calibration and scale conversion for other liquids or gases [1]-[12], an idea has occurred us to built an *flow-meter gauge* [13], more precise than the one's existent by its constructive simplity and the reduced number of physical parameters, measured in static and in any environmental conditions, which in the same time not introduces neither energy loss and require a very simple formula for the calculation of the flow-rate.

Using this device we could verify experimentally with an accuracy under 1% the indications of the standard devices with diaphragms, installed in the correct metrological conditions.

2. Permanent suction flow of the compressible and viscous fluid into a circular pipe-line

The axi-symmetrical character of the suction flow of a compressible and viscous fluid in a circular pipe-line permits even the two-dimensional solving of the gas adiabatic evolution (fig. 1). In this case the equation system consists of:

2.1. Equation system with partial differentials

- *energy equations* till to the and after the entrance of the fluid in the conduit and more far until to the flow-meter **D** for calibration,

For the cylindrical co-ordinates, proper to the studied problem of compressible and inviscid fluid suction in a circular pipe-line $\left(\frac{\partial}{\partial t} = \frac{\partial}{\partial \theta} = V_{\theta} = 0\right)$, and denoting simply with *V* – the radial and *W* – the axial fluid velocity components, the Euler system of equations with partial derivatives [14] consists of non-linear equations of motion

$$\rho (VV'_{\rm R} + WV'_{\rm Z}) + P'_{\rm R} = 0, \qquad (1)$$

$$\rho(VW'_{\rm R} + WW'_{\rm Z}) + P'_{\rm Z} = 0, \qquad (2)$$

- *mass conservation equation* of inviscid and compressible fluid

$$\rho \left(\frac{V}{R} + V'_{R} + W'_{Z} \right) + V \rho'_{R} + W \rho'_{Z} = 0, \qquad (3)$$

and - *state equation of the gas*, in its supposed isothermic evolution in the case of small variation of pressure and density

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$$P = K_i \cdot \rho, \tag{4}$$

in which the constant K_i have the expression

$$K_{i} = \frac{P_{0}}{\rho_{0}} = \Re T_{0}.$$
 (5)

2.2 Undimensionalization of the partial differential equation system

For a more generality of the numerical solution, we shall introduce the dimensionless magnitudes in the equation system, choosing as characteristic parameters:

- $R_{\rm C}$ the radius of the suction pipe-line,

- $W_{\rm m}$ the mean velocity, which transport the volumic flow-rate $V' = \pi R_{\rm C}^2 W_{\rm m}$ in the pipe-line,

- P_0 the pressure in the suction gas chamber at large distance of the conduit entrance,

- ρ_0 the gas density in the suction room at the temperature T_0 and relative humidity U%.

With the new dimensionless variables and functions: $r = R / R_C$, $z = Z / R_C$, $v = V / W_m$, $w = W / W_m$, $p = P / P_0$, $\delta = \rho / \rho_0$, (6) the equation system in dimensionless magnitudes becomes:

$$\delta(\mathbf{v}\cdot\mathbf{v}_{\mathrm{r}}'+\mathbf{w}\cdot\mathbf{v}_{\mathrm{z}}')+\mathrm{Eu}\cdot\mathbf{p}_{\mathrm{r}}'=0\,,\tag{1}$$

$$\delta(\mathbf{v} \cdot \mathbf{w}_{\mathrm{r}}' + \mathbf{w} \cdot \mathbf{w}_{\mathrm{z}}') + \mathrm{Eu} \cdot p_{\mathrm{z}}' = 0, \qquad (2^{\prime})$$

$$\delta\left(\frac{v}{r} + v_{\rm r}' + w_{\rm Z}'\right) + v \cdot \delta_{\rm r}' + w \cdot \delta_{\rm Z}' = 0, \qquad (3')$$

$$p = \delta$$
, (4')

in which we denoted by **Eu** the Euler's number, which characterises the intensity of flow.

2.3 The elimination of the functions, whose values on the boundary are not known

By introducing the partial derivatives of p from the relation (4') in the equations of movement and the partial derivatives δ'_r and δ'_z from these in the equation of mass conservation, we obtain a single equation for the both two components of fluid velocity

$$\operatorname{Eu}\left(\frac{v}{r} + v_{r}' + w_{z}'\right) = v^{2}v_{r}' + vw(v_{z}' + w_{r}') + w^{2}w_{z}', \quad (7)$$

In the aim to eliminate from the calculation programme the difficulties concerning the different

values of the ideal fluid velocity on the exterior and interior of the pipe-line wall respectively, we shall introduce the stream function by the relations:

$$w = \psi'_r$$
 and $v = -\psi'_z$, (8)

in which case, the equation with partial derivatives became:

$$\frac{\mathrm{Eu}}{r}\psi'_{z} + \psi''_{rz}\left(\psi'_{r}{}^{2} - \psi'_{z}{}^{2}\right) + \psi'_{r}\psi'_{z}\left(\psi''_{z}{}^{2} - \psi''_{r}{}^{2}\right) = 0$$
(9)

Introducing in this non-linear equation the partial derivatives expressions, deduced by simple algebric calculus from the developpings in finite Taylor's series to the 2th order of the stream function ψ [15] in the knots of the quadratic network from figure 1, we obtain for instant the associated algebric relation

$$\psi_0 = \psi_3 + \frac{r}{4\mathrm{Eu}\chi^3} \,. \tag{10}$$

$$\cdot \left\{ \frac{1}{4} (\psi_5 + \psi_7 - \psi_6 - \psi_8) [(\psi_1 - \psi_3)^2 - (\psi_2 - \psi_4)^2] \\ - (\psi_1 - \psi_3) (\psi_2 - \psi_4) (\psi_1 + \psi_3 - \psi_2 - \psi_4) \right\}$$

With this occasion we shall observe that, for the numerical solution stability reason, we explicated the value of stream function in the knot 0 from the alone linear terminus of the equation.

2.4 The specific boundary conditions of the studied problem

For the numerical integration we consider (fig.2) the following boundary conditions:

- in the axis of the conduit r = 0 we take for the stream function the value $\psi(z,0) = 0$,

- on the pipe-line wall $r_C = 1$ and $z \ge 0$ we consider for the stream function the value

 ψ (z,1) = 1, in the same time with $v'_r = \psi''_{rz} = 0$ and also, for the ideal fluid, the shearing stress

$$\tau_{zr} = w'_r + v'_z = w'_r = \frac{\psi''_r}{r} - \frac{\psi'_r}{r^2} = 0, \qquad (11)$$

from which we have in the pipe $\psi(r) = (r^2+r)/2$

- at the end of the conduit we consider the parallelism of the stream lines i.e. $\psi_1 = \psi_0, \psi_5 = \psi_2, \psi_8 = \psi_4$.

- among the specific boundary conditions of the stated problem there is the *reflexion* applied to the pipe wall,

- on the outside of the domain on the rectangular boundary we consider the uniform velocity



Fig.1. The coordinate axis, a stream line, the velocity components and the grid knots numbering



Fig.2. The computational field, the boundary conditions and the grid knot numbering

3. Experimental research

The experimental research are focused on the determination of static pressure distributions along the entrance zone in a circular pipe-line, for different flow-rates (fig.3), with the aim to establish the very suitable place of the pressure intake for our measuring device.

moving away from the entrance cross-section, following, for the sake of the result accuracy, a helicoidal curve of place distribution in which the pressure intakes are mounted and putting down for every point the values of the measured for the different flow-rates, in order to establish the best flowrates place of the pressure intake.



Fig.3. Pressure distributions

3.1 Experimental installation

The experimental stand (fig. 4) consists in two pipe-line section, between which one mounted in the standardized metrological conditions the flow-meter for calibration or a diaphragm respectively for flowrate measuring to verify our device.

At the pipe extremity of the fluid entrance in the first conduit, equipped with a row of static pressure intakes, we mounted our patented device and at the out of the second pipe is mounted a flow regulator and a high pressure exhauster, foreseen at its outlet with a valve for the fine adjustment of the flow-rate. The pressure measuring continues along the pipe,

for different flow-rates

3.2. Measuring apparatus

The used measuring apparatus consists in: a barometer for the absolute pressure measuring of the air from suction chamber (accuracy 0,5 mb), a thermometer for air temperature measuring in laboratory chamber (accuracy 0,02°C), a hygrometer for measuring of air relative humidity in the laboratory room (accuracy 1%), manometers with liquid for the relative pressure measuring (accuracy 0,5 mmwc).

The high number of rotation of the blower and the flow without vortex separations into the chamber and the conduit assure the permanence of the gas flow without flow-rate and pressure pulsations.

The great volume of the laboratory chamber,



quantities and their measuring accuracy.

Fig.4. General view of the experimental stand

4. Conclusions

After the experimental measurements made in the laboratory of *Flowmeters, their calibration stands and conversion scale for other fluid as water and air* we can say that the described method is extremely precise, requiring only four static measurements: the barometric pressure, the temperature and the humidity in the gas chamber and also the relative static pressure inside the pipe. The patented device is simple, it can be achieved without any special technological efforts, which is why we are willing to offer it to the interested firms and metrological laboratories.

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Compressible Fluids in Hyperbaric Interventions

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Abstract:

The breathing medium for a diver may be air, pure oxygen, or a synthetic gas mixture made of oxygen and diluent gases. The comfort of the diver depends on the gas flow rate delivered by the breathing apparatus and on the resistance to breathing introduced by this apparatus.

There are presented some investigations on how the mass flow rate through a valve varies when the gas mixture changes. This study is only a part of the researches on the external breathing resistance added by an open circuit scuba (more specific, a two stages demand regulator). The results may be used in the engineering design of all types of breathing apparatus.

Keywords: Compressible fluids, nozzle, underwater breathing.

1. Underwater Breathing Media

Man needs a proper breathing medium in order to spend more time underwater, to work more efficiently and to return safe and sound to the surface. A proper medium refers to the composition of the respiratory gas and also to its physical parameters: pressure, temperature, density. Diver's respiration underwater must be protected by a breathing apparatus. The comfort of the diver depends on the gas flow rate delivered by the apparatus and on the resistance to breathing introduced by this apparatus.

The most usual breathing medium is compressed air. Air is a natural gas mixture composed of nitrogen, oxygen, carbon dioxide and trace amounts of other gases. Ambient air doesn't meet the requirements for use as diver's medium. It must be filtered and compressed in special breathing air compressor to avoid contaminants and to assure quality. In spite of all these conditions, air is the less expensive breathing medium. Air divings are limited by the narcotic effect of nitrogen which become obvious at depth greater than 50m.

The oxygen is used as a breathing gas for divers only in closed circuit scuba. Pure oxygen required for breathing is usually obtained by cryogenic separation from air. This is a complex ISSN-12223-7221 and expensive process. Oxygen divings are limited to 7 m depth. This limitation dues to the oxygen toxicity when its partial pressure exceeds 0.5 atm. Partial pressures between 0,5 and 1,2 atm can be tolerated but only for short periods of time. Pure oxygen is used only by combat divers, specially trained.

Oxygen may be mixed with nitrogen, helium, hydrogen or other inert gases for use as a breathing medium. Mixed gas breathing media are controlled mixtures of oxygen with:

one diluent gas, such as NITROX and HELIOX; two diluent gases, such as TRIMIX.

Nitrogen, the most comon diluent, is cautiously used with respect to its property to produce narcosis.

Also, the decompression time is longer when nitrogen concentration of a mixture increases.

Helium has no narcotic effects on the diver over a large range of depth. But, at depth over $150\div180m$ serious nervous syndrom may occur on the diver.

Nitrox contains a higher concentration of oxygen than air. Therefore, nitrox dives require less decompression time than air dives at the same depth and duration. The disadvantage is that the diving depth is limited by the partial pressure of oxygen rather than the nitrogen narcosis.

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Heliox mixtures are used as breathing media in dives over 70m.

Whatever would be the gas mixture, the required

oxygen percentage is determined by the maximum depth of the dive.

2. Underwater Breathing Apparatus

The above breathing media are used with main kinds of underwater breathing three apparatus:

- closed-circuit breathing apparatus, used • with pure oxygen or with synthetic gas mixtures;
- semiclosed circuit breathing apparatus, used with synthetic gas mixtures;
- open circuit breathing apparatus, used either with air or synthetic gas mixtures.

The respiratory gas is stored under pressure into cylinders and delivered to the diver by means of the breathing apparatus. The cylinders may be weared by the diver, in the case of scuba diving, or carried by a ship or a submmersible, in the case of umbilical diving.

The most comon phenomenon, used in the engineering design of these items, is the pressure deplete when the breathing gas flows through a nozzle or through a valve. A valve may be approached to a nozzle or even to a Laval duct, at which the minimum cross area is variable in time (when the valve opens or closes).

The study presented in this paper refers to the investigations on how the mass flow rate through a valve varies when the gas mixture changes. This study is only a part of the researches on the external breathing resistance added by an open circuit scuba (more specific, a two stages demand regulator).

The external resistance to breathing, R, is the ratio between the volume flow rate, V, delivered by an apparatus and the pressure differential Δp created by the respiratory action of the diver's lungs as the metering signal:

$$R = \frac{V}{\Delta p},$$

$$\begin{bmatrix} l/\min \cdot cmH_2O \end{bmatrix}, \begin{bmatrix} m^3/s \cdot cmH_2O \end{bmatrix} (1)$$

Regardless the constructive style in which the two stages demand regualtors are available, they are designed to reduce the gas in the cylinders to ambient pressure in two stages:

-the first stage reduces the high pressure to a medium value (usually 9 bar above the ambient pressure);

-the second or the demand stage reduces the medium pressure to the ambient value.

The main advantage of this apparatus is that the respiratory gas is supplied to the demand stage at an approximately constant pressure (at a constant depth), thus allowing a reduction in breathing resistance. As the depth changes or the cylinder pressure decreases, only small fluctuations in the breathing resistance may occur. The fact that the demand valve works against a controlled medium pressure results in a reduced breathing resistance.

Referring to the demand stage, we may consider the gas expansion through the valve a constant-enthropy process, thus we may use the Saint Venant equation for the mass flow rate:

$$\dot{m} = \alpha \sigma \sqrt{\frac{2k}{k-l} p_m \rho_m \left(\frac{p_e}{p_m}\right)^2} \left[l - \left(\frac{p_e}{p_m}\right)^{\frac{k-l}{k}} \right], (2)$$

m - the mass flow rate, $\begin{bmatrix} kg \\ s \end{bmatrix}$; where

pe-the environmental pressure, $\begin{vmatrix} N/\\m^2 \end{vmatrix}$

 $p_{\rm m}$ -medium pressure in the hose, $\left| \frac{N}{m^2} \right|$

 ρ_m -gas density coresponding to the medium

pressure,
$$\begin{vmatrix} kg \\ m^3 \end{vmatrix}$$

 α -the discharge coefficient;

k- the adiabatic coefficient of the respiratory gas;

 σ -the section area of the second stage valve, $[m^2];$

The mechanism is conceived to work in critical conditions of flow, that means the ratio between the down stream and the up stream pressure is lower then the critical one, β_c .[2] The critical ratio β_c for the second stage is:

$$\beta_c = \frac{p_{cr}}{p_m} = \left(\frac{2}{k+I}\right)^{k/k-I} \tag{3}$$

In critical conditions, the mass flow rate takes the maximum value [1] (for a constant opening section of the valve, σ):

$$\dot{m} = \alpha \sigma \left(\frac{2}{k+l}\right)^{\frac{l}{2}\frac{k+l}{k-l}} \sqrt{k} \frac{p_m}{\sqrt{RT_m}}$$

(4)

where T_m -the temperature in the hose, [K];

R-the gas constant, $\begin{bmatrix} J \\ kg.K \end{bmatrix}$;

The relationships (2), (3) and (4) were written for the second stage because it is the one that establishes the mass flow rate, in accordance with the depth and the diver's demand. The equation of continuity results in an equal mass flow rate through the first stage, when the motion is steady. The mass flow rate depends only on the upstream conditions of motion, when the area σ is constant (the inspiration amplitude is constant). It also depends on the nature of the gas (the constants k and R)

Otherwise, we can express the mass flow rate according to the density of the delivered gas in local conditions of pressure, assuming a constant enthropy evolution of the gas:

$$\dot{m} = \rho_e \dot{V} \tag{5}$$

The two relations (4) and (5) show that the volume

flow rate V depends on the nature of the breathing gas and on the pressure p_m established upstream the valve, if we consider that the stagnation temperature, T_m , and the area, σ , are constant whatever would be the depth.

3.The Flow Rate Variation according to the Nature of the Breathing Gas

The characteristics of some usual gas breathing mixtures were calculated taking into account the participation by volume of each component. The values are presented in Table 1. Air is considered a mixture of 21% oxygen and 79% nitrogen (by volume).

Characteristics	Components,	Molecular	Density	Gas	Adiabatic
	(participation by volume)	Mass,	(at p=0,98bar;	Constant,	Coefficient
		а	T=288 K)	R	, k
		kg/	[kg/]	$\begin{bmatrix} J/. \\ \end{bmatrix}$	[-]
Gas Mixtures		[[] / <i>kmol</i>]	$\left[\right] m^{3}$	$kg \cdot K^{-1}$	
Air	$r_{O_2} = 0,21; r_{N_2} = 0,79$	28,85	1.182	287,13	1,402
Nitrox	$r_{O_2} = 0,325; r_{N_2} = 0,675$	29,308	1.201	283,687	1,398
	$r_{O_2} = 0,4; r_{N_2} = 0,6$	29,606	1.213	280,831	1,398
	$r_{O_2} = 0,5; r_{N_2} = 0,5$	30,005	1.229	277,097	1,397
Heliox	$r_{O_2} = 0,325; r_{He} = 0,675$	13,101	0.537	634,631	1,541
	$r_{O_2} = 0,4; r_{He} = 0,6$	15,200	0.623	546,993	1,519
	$r_{O_2} = 0,5; r_{He} = 0,5$	18,000	0.737	461,905	1,493

Table 1 The characteristics of some usual gas breathing mixtures. They were calculated taking into account the participation by volume of each component

The mass flow rate variation is shown in Fig. 1. The values are calculated for the same

temperature and the same medium pressure, corresponding to the 51 m depth. It is notable that the

mass flow rate is nearly constant for air and nitrox mixtures.

The difference is considerable between nitrox and heliox mixtures. The mass flow rate

reduces with up to 30% in the case of heliox $32,5\%O_2$.



Fig.1 The mass flow rate of some usual breathing gas mixtures, calculated for the same upstream conditions of the demand valve, at 51m depth

The variation of the mass flow rate of different gas mixtures versus the depth of immersion is shown in Fig.2. It may be noticed that the difference between the mass flow rate of nitrox and heliox mixtures at surface increases with the depth of immersion. There are differences between the mass flow rate of heliox mixtures of different volume participations of the components.



Fig.2.The mass flow rate variation vs depth of immersion

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Following the curves represented in Fig.3., the volume flow rate, expressed in local condition of pressure, decreases as the depth of immersion increases. The lower volume flow rate, the greater resistance to breathing added by the breathing apparatus at the same pressure differential Δp created by the diver during inhalation. It also can be noticed that the volume flow rate is greater for the heliox mixtures than for the nitrox mixtures.





4.Experimental researches

us to use air in the investigation of the resistance variation. There was developed a series of simulated divings, with air, at various depth of immersion.

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0.8 0.7 51m depth Exhaled Flow Rate of Carbon Dioxide, [l/min] 0.6 0.5 0.4 surface 0.3 **1** G 0.2 5 10 25 30 15 20 Ventilatory Flow Rate, [l/min]

Fig.4 Breathing effort increase with depth of immersion. Experimental curves of exhaled carbon dioxide variation

The observation that the resistance to breathing takes the maximal values for air and nitrox mixtures allows 253

The apparatuses resistance to breathing was measured at surface, in laboratory conditions according to the relation (1) and then it was surveyed the breathing effort of the divers at different depths.

The breathing effort was considered proportional to the volume flow rate of carbon dioxide exhaled by the diver [4].

The curves in figure 4 represent the variation of the volume flow rate of exhaled carbon dioxide versus the ventilatory volume flow rate of air. It is obvious the difference between the breathing effort at 51m depth and the effort at surface. At the same ventilatory flow rate, the exhaled carbon dioxide flow rate increases up to 3.5 times. The gas volumes were measured while the divers were at rest.

It also must be noticed that the ventilatory flow rate decrease at the depth as an adjustment of the human respiration to the underwater medium.

5.Conclusions

The breathing gas flow rate through a valve or a series of two valves depends on the nature of gas. mixture. Whatever would be the type of underwater breathing apparatus it adds an extra resistance to breathing in hyperbaric condition. This resistance increases with the depth of immersion, as the volume flow rate decreases.

Air resistance is nearly the same as the resistance of the nitrox mixtures but greater than the resistance of the heliox mixtures. Thus, air is enough relevant for the experimental study of the breathing resistance evolution in hyperbaric environment.

The theoretical and experimental results of the above study offer an immage on how the resistance of a certain apparatus, measured at surface, with air, in normobaric condition of laboratory, willvary in hyperbaric conditions even if underwater it uses another gas mixture.

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The Controlled Orientation Method of Warm Air Rising Currents at the Multi-Storey Construction Façade

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Abstract: The work presents a small budget, easy applied method concerning the exact and controlled orientation of thin warm air layers, that describe a rising motion of washing the facade of block constructions. The proposed method removes the discomfort of unpleasant smell transport contained by the above mentioned rising currents, realizing an agreeable olfactory comfort, based on a small budget, for every storey of the construction.

Keywords: buoyancy force, upward motions, flow, thermal conduction, fluid density.

1. Introduction

The people who live in multi-storey blocks, especially those from the upper storey are often olfactory disturbed by the vicious warm air thin layers which practically "wash" that part of the building facade where the kitchens are placed from the first to the last storey.

Distributing and even annoying: toxic burned gases, smoke, steam of different types, food smells, etc. are often contained in the warm air evacuated through the open windows of the kitchens. As the it was shown, through it's nature the air is a poliphasic a fluid and contains, besides oxygen and nitrogen a series of other components of gas or solid nature.

The work presents two simple, easy to apply modalities which may lead to the reduction, in great part of the mentioned negative effects or even the elimination of this effects and to the augmentation comfort of multi-storey constructions lodgers.

2. Theoretical considerations

Consider a fluid whose density ρ is dependent on the temperature *t*, on the concentration *C* of some chemical component, and on static pressure p; that is, $\rho = \rho(t, C, p)$.

In a quiescent ambient environment of such a fluid in a gravitational field of strength g, a local ISSN-12223-7221

region of lower density produces, an upward buoyancy force \overline{B} , written as a vector.

This force will result in the motion of the fluid. The situation is seen in the figure 1, for a density field, which may be described in space coordinates x and y.





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This body volume V_u is equal with the unit. Here ρ is the instantaneous local density. In this body volume V_u the pressure gradients is $\frac{dp}{dx} = -\rho \cdot g$. The hydrostatic gradient in the

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ambient environment is $\frac{dp_a}{dx} = -\rho_a \cdot g$. This difference in pressure gradient drives the motion through the unit buoyancy force \overline{B} can be calculated as $\overline{B}(x, y) = \overline{g}(\rho_a - \rho)$. Thus, \overline{B} is the difference between two body forces. In this way a positive fluid motion on the vertical results.

This force \overline{B} is positive, and there fore the resulting flow, is upward. The magnitude of this buoyancy force depends on the local temperature and concentration levels. The local buoyancy force then is calculated from the local gravitational force at any point $-\rho \cdot g$. Also the buoyancy force \overline{B} is obtained by subtracting from $-\rho \cdot g$ some average or representative body force $-\overline{g} \cdot \rho_r$. Thus, $\overline{B} = \overline{g}(\rho_r - \rho)$. For such a flow, the local reference density at different elevation, ρ_r , is calculated from the linear temperature distribution, for pure thermal conduction.

For example, the flow will be vertical if confined by or attached to vertical surfaces. The quantities to be determined for any flow, are usually the heat transfer rate and the velocity field. The heat transfer coefficient h from a surface to a fluid is defined in terms of the heat flow rate Q:

$$Q = h \cdot A(t_0 - t_\infty), \qquad (1)$$

 t_0 ; t_{∞} - the temperatures are those of the surface and the ambient medium;

A - is the surface area.

The heat transfer or convection coefficient results as a Nusselt number:

$$Nu = \frac{h \cdot L}{k},\tag{2}$$

L - is the characteristic dimension of the surface;

k - is the fluid thermal conductivity.

The velocity u generated by an external flow may be estimated by equating the kinetic energy produced per unit volume:

$$\frac{\rho \cdot u^2}{2} \cong gL \cdot \Delta \rho \,, \tag{3}$$

where

$$\Delta \rho = \rho_r - \rho = \rho_\infty - \rho \,, \tag{4}$$

and:

 ρ_∞ - is the quiescent ambient medium density;

 ρ - is the local density.

The equations of motion induced by thermal transport are:

$$\frac{D\rho}{D\tau} = -\rho\nabla\overline{V} \text{ or } -\frac{\partial\rho}{\partial\tau} = \rho\nabla\cdot\overline{V} + \overline{V}\cdot\nabla\rho, \qquad (5)$$

$$\rho \frac{D\overline{V}}{D\tau} = \rho \cdot \overline{g} - \nabla p + \mu \nabla^2 \overline{V} + \frac{1}{3} \mu \nabla \left(\nabla \cdot \overline{V} \right), \qquad (6)$$

$$\rho \cdot c_p \cdot \frac{Dt}{D\tau} = \nabla \cdot k \nabla t + \beta \cdot T \cdot \frac{Dp}{D\tau} + \mu \Phi + q^{\prime\prime\prime} .$$
(7)

In the equations the following terms arise:

 $\overline{V} = \overline{V}(u, v, w)$ - is the velocity;

t and T - the conventional and absolute temperatures;

- ∇p the gradient of static pressure;
- $\rho \cdot \overline{g}$ the local body force;
- μ the dynamic viscosity;

 ρ - is the local fluid density;

 c_n - is the specific heat;

 β - the coefficient of thermal expansion;

k - is the thermal conductivity;

 $\mu\Phi$ - is the viscous dissipation energy effect;

q''' - is the volumetric energy generation rate.

The viscous dissipation term, $\mu \Phi$, is the volumetric rate of flow energy dissipation into thermal energy.
The dissipation function is:

$$\Phi = 2 \left[\left(\frac{\partial u}{\partial x} \right)^2 + \left(\frac{\partial v}{\partial y} \right)^2 + \left(\frac{\partial w}{\partial z} \right)^2 \right] + \left(\frac{\partial v}{\partial x} + \frac{\partial u}{\partial y} \right)^2 + \left(\frac{\partial w}{\partial y} + \frac{\partial v}{\partial z} \right)^2 + \left(\frac{\partial u}{\partial z} + \frac{\partial w}{\partial x} \right)^2 - \frac{2}{3} \left(\nabla \cdot \overline{V} \right)^2 \quad (8)$$

Thermal effect arising in Eq. (7) from the pressure field, from distributed sources q''', and from $\mu\Phi$ are often very small. The difference between the body force in Eq. (6), $\overline{g} \cdot \rho - \nabla p$, is frequently merely $\overline{g}(\rho - \rho_{\infty})$, to a good approximation, as will be seeing the next equation:

$$\nabla \cdot \overline{V} = \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0 , \qquad (9)$$

$$\rho\left(u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y}\right) = \mu\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + g(\rho_{\infty} - \rho)(10)$$
$$\rho\left(u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y}\right) = \mu\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right), \quad (11)$$

$$\rho \cdot c_p \left(u \frac{\partial t}{\partial x} + v \frac{\partial t}{\partial y} \right) = k \left(\frac{\partial^2 t}{\partial x^2} + \frac{\partial^2 t}{\partial y^2} \right). \quad (12)$$

The distributions: $u(x, y), v(x, y), \rho(x, y) = \rho(t)$, and t(x, y) are expressed in terms of x and y.

 $\rho_{\scriptscriptstyle \infty}$ - is the density in the quiescent ambient medium .

This presented relations are valid at the vertical surfaces of the multi-storey buildings that are heated by the sun, the fluid motion being a free convection. The developing of this kind of motion can be followed in figure 2. This represents the steady laminar transport adjacent to a flat vertical surface maintained at a uniform temperature t_0 .

In this case:

 $t_\infty\,$ - is the quiescent ambient temperature;

u(x, y) - is the velocity distribution;

t(x,y) - is the temperature distribution; where $t_0 > t_{\infty}$.



Fig. 2. Vertical thermally induced flow

For the vertical flat surface, for flows induced by combined thermal and mass diffusion can by used the following equations:

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} = 0, \qquad (13)$$

$$u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} = v\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) - \frac{1}{\rho}\frac{\partial p_m}{\partial x} + g\beta(t - t_{\infty}) + g\beta^*(C - C_{\infty}), \qquad (14)$$



 $u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} = v\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) - \frac{1}{\rho}\frac{\partial p_m}{\partial y}$ (15)

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Fig. 3. Inversed channels of warm air flows conductions; types of inversed channels for warm air flows orientation. a,b - combined channels between plane and cylindrical surfaces c – channel combination of plane surfaces.



Fig. 4. Capture panels for the warm air layers.

$$u\frac{\partial t}{\partial x} + v\frac{\partial t}{\partial y} = \alpha \left(\frac{\partial^2 t}{\partial x^2} + \frac{\partial^2 t}{\partial y^2}\right) - \frac{q'''}{\rho c_p}, \quad (16)$$

$$u\frac{\partial C}{\partial x} + v\frac{\partial C}{\partial y} = D\left(\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2}\right) + c'''. \quad (17)$$

In this way, the constant transport are μ , k, D. *D* - is the diffusion coefficient; μ - the dynamic viscosity;

k - is the thermal conductivity;

The other coefficients arise:

C - represents the species concentration imposed

at x=0, y=0; C_{∞} - is the species concentration in the quiescent ambient medium;

t - is the local temperature;

 t_{∞} - is the quiescent ambient temperature;

 $\boldsymbol{\alpha}$ - is the thermal diffusity coefficient;

q''' - is the volumetric energy coefficient;

 p_m - is the pressure ambient medium;

c''' - is the local rate of production of species C per unit volume;

 c_p - is the specific heat;

 β ; β^* - is the expansion coefficient:

$$\beta = -\frac{l}{\rho} \left(\frac{\partial \rho}{\partial t} \right)_{p,C},\tag{18}$$

and

$$\beta^* = -\frac{l}{\rho} \left(\frac{\partial \rho}{\partial C} \right)_{p,t} \,. \tag{19}$$

3. Practical proposed solutions

Aiming the elimination of the olfactory discomfort at the superior storey, two solutions are proposed for the controlled orientation of the vicious warm air flows, resulted especially from the kitchens of the block constructions.

In this way, in figure 3, is presented the orientation of vicious air flows using the inverse channels, bended towards the vertical "channels" realized with the orientation panels assembled perpendicular on the block facade, in the zone of the kitchen windows. Another solution that requires the utilization of capture panels for pollution warm air layers, made of light construction material panels attached to the vertical

"channels" from the previous solution, is presented next in the figure 4. Both solutions assure the level increase comfort at every storey of a multi-storey block.

4. Conclusions

The proposed solutions concerning the architecture of multi-storey constructions are pretty simple and can by realized using regular construction materials.

The olfactory comfort level can be increased by using this kind of methods, with simple means and with reduced supplementary investments.

This solutions can be applied, with usual technologies used in the construction domain.

The presented solutions do not require supplementary energy consumption concerning the vicious warm air flows suction.

Through this methods, the fresh air is assured, with simple means, in the multi-storey blocks from the urban zone.

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The Influence of The Hydraulic Resistance of the Air Chamber Branch on the Hydraulic Shock Extreme Pressures in a Pumping Installation

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Abstract:

The paper presents part of the results, obtained by numeric calculus, referring to the influence of the hydraulic resistance of the branch, which links the water chamber, as a protection means, and the protected discharge pipe, on the extreme pressures that occur in a pumping station during the hydraulic shock in the case of energetic damage.

Part of the results are shown either in tables or under graphical shape.

There are also presented the conclusions of the researches.

Keywords: Pumping installation, hydraulic shock, means of protection, water chamber, extreme pressures.

1. Introduction

The unsteady movement of water in under pressure hydraulic systems may be considered a regular situation, if we take into account the frequency of its occurance and development.

In the case of a rapidly variable unsteady movement (hydraulic shock) either overpressures that may exceed a few times (even tens) the actuating pressure in steady regime, or pressure drops under the limit of cavity may occur. It's for this reason that the hydraulic shock calculus is required, namely the determination of the minimal and maximal pressures in different sections of the installation, needed for sizing and checking an under pressure hydraulic system threatened by such a phenomenon.

Usually, the important hydraulic installations are protected from the negative effects of the hydraulic shock by means of equipments such as: surge tank, air chamber, water chamber, relief valves, air valve etc.

The worse situation for a pumping installation (the strongest hydraulic shock) arise in the case of energetic damage. The accidental electrical break is an unusual way to turn off the ISSN-12223-7221

pumping installation, which generates a strong hydraulic shock. Thus, in the most cases, the protection devices are required.

Regardless the type of the device: surge tank, air chamber, water chamber, the specialty literature [1], [2] reveals the importance of the hydraulic resistance of the branch between the protection device and the protected pipe, on the size of the protection device and on the extreme values of the pressure.

The authors crew intended to study, by numeric calculus, the influence of the hydraulic resistance of the branch on the extreme values of the pressure variation, in the case of a pumping installation protected by a certain air chamber, priorly sized.

2. The Pumping Installation Schema

The hydraulic shock calculus was made for the pumping installation outlined in fig.1. The significance of the symbols and the numeric values used in this calculus are the following:

 $Q_p = 1.25 \, m^3 / s$ -the discharge in steady regime;

 $H_G = 60m$ -the pumping geodesic head;

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L = 1200m -the discharge pipe's length; D = 0.80m -the discharge pipe diameter; $\delta = 10mm$ -the thickness of the pipe wall; $\Delta x = 150m$ -the calculus section length;

 $n_p = 1500 \frac{rot}{min}$ the rotational speed;

pipe;

n = 0.016 the coefficient of inner rugosity of the

 $I = \frac{G \cdot D_p^2}{4g} = 250 Nms^2$ - the moment of inertia of the

rotational parts of the pump;

G -the weight of the rotational parts;

 D_p -the average diameter of the rotational parts.



Fig.1 The pumping installation schema

2. The Features of the Air Chamber

 $V_t = 30m^3$ -the total volume of the air chamber;

 $V_a = 10m^3$ -the air volume in the air chamber;

l=20m-the branch's length;

d=350mm the diameter of the branch

 $R = M \cdot Q_H \cdot |Q_H|$ -the hydraulic resistance of the branch:

M-the hydraulic resistance modulus;

 Q_H -the discharge of water change for the air chamber;

It was taken into account the case of a symetrical resistance of the branch with respect to the sense of the water movement through the branch.

3.The Calculus Model

The present performances of the electronic computers allow the numeric integration of the hydraulic shock equations in a wide range of limit conditions, as they can be met in practice.

The characteristics method is the most currently used for the hydraulic shock calculus by numeric simulation. This is the method that has been used for the calculation in this paper.

In the case of a simple current calculus knot, the equations are:

$$v_{j,i+I} = \frac{1}{2} \left[v_{j-I,i} + v_{j+I,i} - \frac{g}{c} \left(H_{j-I,i} + H_{j+I,i} \right) - \frac{\lambda \cdot \Delta t}{2D} \left(v_{j-I,i} \cdot \left| v_{j-I,i} \right| + v_{j+I,i} \cdot \left| v_{j+I,i} \right| \right) \right]$$
(1)

$$H_{j,i+1} = \frac{1}{2} \left[H_{j-1,i} + H_{j+1,i} + \frac{c}{g} \left(v_{j-1,i} - v_{j+1,i} \right) - \frac{c \cdot \lambda \cdot \Delta t}{2g \cdot D} \left(v_{j-1,i} \cdot \left| v_{j-1,i} \right| - v_{j+1,i} \cdot \left| v_{j+1,i} \right| \right) \right]$$
(2)

where: *v* –the mean velocity; *H*-the total piezometric head;

c- the celerity;

 Δt -the time step; λ -Darcy's coefficient; *j* -the index of the calculus section;

i –the index of the moment of time.



Fig.2 Calculus knot with air chamber

In the case of a knot where is placed an air chamber (Fig.2), beside the equations (1) and (2), there can be written the following specific equations:

$$H_{j,i+1} = Y_{j,i+1} + \frac{p_{j,i+1}}{\gamma}$$
(3)

$$p_{j,i+1} \cdot V_{j,i+1}^n = p_{j,i} \cdot V_{j,i}^n \tag{4}$$

$$V_{j,i+1} = V_{j,i} + \Omega(Y) \cdot \left(v_{r,i+1} + v_{r,i} \right) \cdot \frac{\Delta t}{2}$$
(5)

where: p -the air cushion pressure; V -the air cushion volume; $\Omega(Y)$ –the area of the horizontal section at Y hight v_r -the ascending speed of water in the air chamber; $n = l \div l, 4$, the politropic coefficient;

In order to take into account the hydraulic energy loss in the branch, it may be used the relation:

$$H_{j,i+1} - H_{d,i+1} = M \cdot Q_H \cdot |Q_H|$$
(6)
where:

 H_d -the piezometric head at the entrance/exit of the air chamber.

4.Results

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Subsequent to the calculus program runing, there was obtained the pressure variation in the calculus sections of the installation for different values imposed to the hydraulic resistance modulus. more significant calculus sections: section 2, where is placed the air chamber and section 4, where the discharge pipe changes its longitudinal slope.

In the table 1 there are selectively shown the values for minimal and maximal pressures for two

No	Hydraulic resistance modulus	Section 2		Section 4	
	$\mathbf{M}, \left(m^{-5} \cdot s^2\right)$	$\frac{p_{min}}{\gamma}$,(m)	$,\frac{p_{max}}{\gamma},(m)$	$\frac{p_{min}}{\gamma}$, (m)	$\frac{p_{max}}{\gamma}$, (m)
1	0.0001	-10	232.12	-10	197.43
2	0.0001	26.81	100.14	9.36	63.72
3	0.001	26.81	100.14	9.36	63.72
4	0.01	26.83	100.11	9.36	63.7
5	0.1	26.95	99.83	9.39	63.5
6	0.25	27.14	99.36	9.42	63.17
7	0.5	27.41	98.63	9.49	62.65
8	0.75	27.66	97.92	9.56	62.13
9	1	27.88	97.23	9.62	61.63
10	2	28.1	94.71	9.84	59.78
11	3	27.78	92.45	10.02	58.15
12	4	27.43	90.74	10.22	58.15
13	5	27.05	88.66	10.41	55.3
14	6	26.65	87.05	10.41	54.01
15	8	25.59	84.26	10.98	52.01
16	10	25.3	81.92	11.37	49.59
17	12	24.3	79.91	11.35	48.78
18	15	23.09	77.51	10.33	46.88
19	18	21.72	75.51	7.56	45.35
20	21	18.99	73.85	4.83	44.08
21	25	15.22	72.03	1.4	42.67
22	30	12.85	70.22	-2.51	41.27
23	35	6.86	68.78	-6.14	40.15
24	40	3.3	67.6	-9.23	39.26
25	45	-0.12	66.76	-10	38.63
26	50	-5.04	65.8	-10	37.91
27	60	-10	64.47	-10	37.03
28	70	-10	64.45	-10	36.39
29	80	-10	64.45	-10	36.13

Table 1 The extreme pressures in sections 2 and 4

On the basis of the numerical results, there were ploted the diagrams of the extreme pressures variation in the two specified sections as fuctions of the hydraulic resistance modulus of the branch (Fig.3 and Fig.4).



Fig.3 The variation of the minimal pressure as a function of the hydraulic resistance of the branch



Fig.4 The variation of the maximal pressure as a function of the hydraulic resistance of the branch

5.Conclusions

The analysis of the results leads to the following conclusions:

- in the absence of any protection device, the extreme pressures reach dangerous values; even cavity may arise;
- the effect of the air chamber is considerable: it diminishes the over pressures and rises the minimal pressures to positive values, thus cavity is eliminated;
- the effect of air chamber on the minimal pressures is more important as the hydraulic resistance of the branch is lower;
- the effect of the hydraulic resistance of the branch is lower on overpressures than on depressions;
- when the hydraulic resistance takes high values, the air chamber controls no more the minimal pressures and cavity may occur;

• there is necessary to investigate the effect of some branches with the hydraulic resistance dependent on the sense of water movement through them.

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Regards about Pressure Pegulators used in Hyperbarism

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Abstract:

There are three types of the regulators of the pressure for a good working of the breathing-evacuation circuit:

1. the breathing regulator (second stages)

2. the dump-valve

3. the limiter of the pressure

Keywords: Regulators, pressure, second stages, dump-valve, limiter of the pressure.

1. Introduction

Hyperbaric precinct is a room that resists at the pressure, where it does simulated diving.

A hyperbaric room includes more includes more pneumatic installations as 'Breathing and dumping circuit'. (see Fig. 1)



Fig.1.The breathing-dumping circuit 1.Valve; 2.One-way valve; 3.Distributing frame; 4.Coupling Hannsen; 5.MP hose 6.Mask with second stage; 7.Wrinklehose; 8.Dump valve; 9.Limiter of the presure

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2. Apparatus presentation

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The pressuring regulators used to equip this installation are:

- 1. the breathing regulator (second stage)
- 2. the dump-valve
- 3. the limiter of the pressure

1. The second stage is a pressuring regulator which reduces the pressure of the breathing gas from the medium pressure (MP) level at manifold of the command rack to the ambient pressure into the precinct. This is an individually component of the diver mask ensemble.

The system is with downstream popppet, as shown in Fig. 2.

2. The dump-valve is located on the

pneumatic circuit, behind the second stage and it is an individually apparatus too. It determinates the controlled exit of the expired gas at a small pressure than the regulator's exciting pressure. The dump-valve has that only destination.

Dump-valve mechanism (see Fig. 3)

From the second stage, the gas comes in the dump-valve at the p_h , pressure, it push and lifts the diaphragm 4, which acts the levers system 5. The levers pulls the compensated poppet 3, which is opening, and it permits the controlled exciting of the gas, until the expiration is null and the p_h pressure balances the diaphragm into the room 2. Now the diaphragm is acting to the system it is shutting the poppet on the seat 6.





Fig2. Second stage regulator

1.Piston; 2.Inhalation precinct; 3.Diaphragme; 4.Pange button; 5.Enviroment communication precinct 6.Levers system; 7.Exhaust valve; 8.Mouth piece; 9.Seat

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Gh. Grad and T.Stanciu al. / Ovidius University Annals of Constructions 3, 4, 267-270 (2002)269individually apparatus too. It determinates theDump-valvemechanism(seeFig. 3)controlled exit of the expired gas at a small pressurethan the regulator's exciting pressure. The dump-valve has that only destination.YesYes



Fig3.The dump valve 1.Body; 2.Piston; 3.Second pappet; 4.Spring; 5.First pappet



Fig4. Limiter of the pressure mechanism 1.Body; 2.Piston; 3.Second pappet; 4.Spring; 5.First pappet

From the second stage, the gas comes in the dump-valve at the p_h , pressure, it push and lifts the diaphragm 4, which acts the levers system 5. The levers pulls the compensated poppet 3, which is opening, and it permits the controlled exciting of the gas, until the expiration is null and the p_h pressure balances the diaphragm into the room 2. Now the diaphragm is acting to the system it is shutting the poppet on the seat 6.

Breathing regulator mechanism

The breathing gas comes from manifold at $p_h + 8 \div 10 bars$. The inspiration of the driver by means of the 8 mouthpiece, creates Δ_p depression which moves the 3 diaphragm. As well because the 5 room is in contact with the ambient, the p_h pressure moves the diaphragm too. That is moved by the $p_h + \Delta_p$, and it acts the 6 levers system, which pulls the 1 poppet. Than, the gas comes, until the diaphragm is balanced, when it obtains the p_h pressure in the 2 precinct.

After balancing, the levers push the poppet until that shuts on the 9 seat. The exhaust valve 7 is opening and the expired gas projects. Can obtain a supplementary flow by pressing the purge button 4.

While the dump-valve works, it can't support than a pressure difference $\Delta_p \in (0,1.5)$ bar, where:

 $\Delta_{\rm p}=p_{\rm h}-p_0\left(1\right)$

 p_h =room pressure ; p_0 = atmospheric pressure

This requires the limitation of the apparatus use, at the 50m depth.

4.The limiter of the pressure

Limiter of the pressure mechanism (see Fig. 4)

The limiter of the pressure is piloted by the room pressure p_h , which acts the 2 piston. The piston shuts on the second poppet 3 and it balances the expiration hoses.

In time of the expiration through the dump-valve, the gas is evacuated in the limiter on p_i pressure. Them it projects from the room at p_0 pressure.

In this case, the differential pressure of the dump valve Δ_p , is:

 $\Delta_{\rm p}=p_{\rm h}-p_{\rm i}\left(2\right)$

 p_i = intermediary pressure into the second stage's exit and the limiter's entrance

3. Checking the good work of the unit second stage limiter :

The balance equation of the system is:

$$p_h \times (A_0 - A_1) = F_a + p_i \times (A_0 - A_1)(3)$$

 $p_i = p_h - \frac{F_a}{A_0 - A_1}(4)$
 $p_h - p_i = \frac{F_a}{A_0 - A_1}(5)$

 A_0 =surface of the principal poppet A_1 = surface of the second poppet

 $A_0 - A_1 = 251 \text{mm}^2$

 $F_a =$ force of the spring

The force of the spring varies from $F_amax = 86N$, at assembly position, to $F_amin = 62N$, at blocking position.

Calculating:

$$\Delta_p \max = \frac{F_a \max}{A_0 - A_1} = \frac{86}{251} = 3,43bar(6)$$
$$\Delta_p \min = \frac{F_a \min}{A_0 - A_1} = \frac{62}{251} = 2,47bar(7)$$

Results $\Delta_{p} \in (2,5 \div 3,5)$ bars

The dump-valve is designed to resist at $\Delta_p \in (0,1\div5)$ bars

The limiter of the pressure assures a good work to the dump-valve for a $p_h > 6bars$ pressure.

4. Conclusions

The second stage and the dump-valve have he same working principle. The dump-valve is opposite to the second stage, about of our action. Apparatus obligatory equip any hyperbaric room..

The limiter of the pressure is in fact, a one-way valve (which doesn't permise to a gas to pass just in one-way), piloted by the p_h precinct pressure.

The limiter equips just the hyperbaric rooms that are pressured at $p_h > 6bari$.

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Wave Movement as Technical Application to Potential Plane Movements

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Abstract: In marine hydraulics, an important part is played by wave movement which from the mathematical point of view has been dealt with in a satisfactory way till now.

Confronted with the theoretical failures of the mathematicians, the water supply engineers were required to substitute the real phenomenon form nature, the physical models of ondulatory water surface movements and with their help to solve the specific problems which the study and design of the marine environment structure *Keywords*: marine hydraulics, waves equation, mathematical simulation, marine environment structure.

1. Introduction

Marine hydraulics that includes water movement as waves, is part of the theoretical applied hydraulics, which is not being taught, in the higher educational establishments and practically this part of hydraulics is known by a very few number of specialists.

Working independently one from the other, Necrosov, in 1921 and Civita, in 1925, deduced the exact movement equation in case of finite deep water. In 1926, Struik did the same thing in the case of finite deep water. But neither them nor the mathematicians that came after them could find the specific solutions of the established equations.

Under these conditions, the engineers felt obliged to adopt a theoretical model by means of which to solve the technical issues imposed especially by the structures being under the stress of the waves. What is in fact of concern from the engineering point of view is the specific wave power calculation formula that represents the energy, which operates in clock unit, per linear meter from the shore or from the structure when they receive the wave action from a frontal direction.

The hypotheses the adopted calculation scheme is based on, are the following:

• Main wave characteristics (height, period, wave length) although in reality they vary in time,

they were considered to be constant for a very short time interval;

• Although the movement is spatial, it was reduced in plane, considering that it is identical in lateral sections;

• The waves were considered stabilized in order to neglect the energy exchange through the free surface as well energy dispersion both inside and on the bottom;

• Slack water depth was considered constant in a certain point on the sea surface where the calculation takes place;

• The water was considered incompressible and lacking viscousity.

All these simplifying hypotheses have proved by means of studies carried out in lab and natural environment that the model does not depart intolerably from the real phenomenon.

2. Diagram Basis

It is known that mechanical energy of the waves consists of the kinetic energy of the particles moving from the zero level on the bottom and of the potential energy of the particles between the free water surface and zero level. This energy is calculated over a wave length (λ) over a 1 m width (front) and over basin depth equal to H (Fig. 1)



Fig. 1 Volume in which energy calculation takes place

It is a known fact that from theoretical mechanics [1], certain forces (\overline{F}) can be equated as gradient, that is a scalar function U(x, y, z) can be found, that is uniform and differentiable in a certain domain D, whose partial derivatives

 $\frac{\partial U}{\partial x}$; $\frac{\partial U}{\partial y}$; $\frac{\partial U}{\partial z}$ should represent the reference axes

force projection module, namely axes that have the unit vectors \vec{i} , \vec{j} , \vec{k} .

That is:

$$\overline{F} = gradeU = \nabla U = \frac{\partial U}{\partial x}\overline{i} + \frac{\partial U}{\partial y}\overline{j} + \frac{\partial U}{\partial z}\overline{k}$$

It is also known that these forces that can be equated as gradient are called conservative forces. It has been convened to say that the conservative forces allow a potential function or that they derive from a potential.

By analogy with the force potential U, the notion of velocity potential was introduced in hydrodynamics, that is the movement of a fluid can be defined through the existence of a scalar function φ whose partial derivatives $\frac{\partial \varphi}{\partial x}; \frac{\partial \varphi}{\partial y}; \frac{\partial \varphi}{\partial z}$ should express $\overline{V}(V_x, V_y, V_z)$

velocity vector projection module on the axes of reference, namely

$$\overline{V} = \frac{\partial \varphi}{\partial x}\,\overline{i} + \frac{\partial \varphi}{\partial y}\,\overline{j} + \frac{\partial \varphi}{\partial z}\,\overline{k}$$

As known, function φ is called velocity potential.

Since the environment in which waves move is a continuously one, i.e. there are no spaces lacking matter in the moving fluid mass, it is possible to write a continuity equation. This equation has to express the fact through an arbitrary surface which closes a constant volume, the fluid mass which goes (m_i) into a certain time interval and the one that exists in the closed space (m_a) equals the one that exists (m_e) in the respective time interval and the remaining one in the closed space (m_r) . Therefore:

$$m_i + m_a = m_e + m_r$$

By adopting the elementary paralelipipedon dx, dy, dz and velocities v_x, v_y, v_z of the masses that enter through the paralelipipedon sides in the positive direction

$$v_x + \frac{\partial v_x}{\partial x} dx, v_y + \frac{\partial v_y}{\partial y} dy, v_z + \frac{\partial v_z}{\partial z} dz$$
 of the axes

of reference and velocities of the masses going out of the paralelipipedon opposite sides, the continuity equation can be easily reached (under the hypothesis that the fluid is homogenous and incompresible):

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} + \frac{\partial v_z}{\partial z} = 0$$

Replacing

$$v_x = \frac{\partial \varphi}{\partial x}, v_y = \frac{\partial \varphi}{\partial y}, v_z = \frac{\partial \varphi}{\partial z}$$

It results:

$$\frac{\partial}{\partial x} \left(\frac{\partial \varphi}{\partial x} \right) + \frac{\partial}{\partial y} \left(\frac{\partial \varphi}{\partial y} \right) + \frac{\partial}{\partial z} \left(\frac{\partial \varphi}{\partial z} \right) = 0 \text{ or}$$
$$\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial y^2} + \frac{\partial^2 \varphi}{\partial z^2} = 0$$

By using the "Laplacean" notion

$$\left(\Delta = \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}\right)$$
 it results: $\Delta \varphi = 0$

3. Deduction of function φ and of wave water particle path

If the movement is considered plane, function φ has to check the reduced continuity equation:

$$\frac{\partial^2 \varphi}{\partial x^2} + \frac{\partial^2 \varphi}{\partial z^2} = 0 \tag{1}$$

In order to identify function φ it is possible to try an expression of the form $\varphi = Z \sin(Kx - \omega t)$ in which Z is in its turn an unknown function. We shall impose that Z should depend only by variable $z (k \text{ and } \omega)$ being auxiliary constants).

By introducing this first expression of φ in (1) it results:

$$\frac{\partial^2}{\partial x^2} [Zsin(Kx - \omega t)] + \frac{\partial^2}{\partial z^2} [Zsin(Kx - \omega t)] = 0$$
$$-ZK^2 + sin(Kx - \omega t) + \frac{\partial^2 Z}{\partial z^2} sin(K - \omega t) = 0$$

That is : $Z'' - k^2 Z = 0$

This equation allows determination of unknown function Z = Z(z) if we treat it as a linear equation with constant coefficients. The roots of the unknown equation being k and -k it results:

$$Z = Ae^{kz} + Be^{-kz}$$

and therefore

$$\varphi = \left(Ae^{kz} + Be^{-kz}\right)\sin(kx - \omega t)$$

Resorting to the physical significance of function φ we can impose the condition that the velocity vertical component be null at the bottom (that is at depth H). Therefore:

$$v_z = \frac{\partial \varphi}{\partial z} = 0$$
 or
 $\frac{\partial \varphi}{\partial z} = \left(KAe^{kz} - KBe^{-kz}\right)sin(kx - \omega t)$

making z = -H and:

$$\left(KAe^{-kH} - KBe^{-kH}\right)\sin(kx - \varpi t) = 0$$

For $sin(kx - \omega t) \neq 0$ results

$$Ae^{-kH} = Be^{kH} = constant = \frac{C}{2}$$

Thus it is inferred $A = \frac{C}{2e^{kH}}$ and $B = \frac{C}{2e^{kH}}$ therefore

$$\varphi = C \left[\frac{e^{k(z+H)} + e^{-k(z+H)}}{2} \right] \sin(kx - \omega t)$$

or

$$\varphi = Cchk(z+H)sin(kx-\omega t)$$

According to the two axes O_x and O_z , the wave water particle velocity components will have the expressions:

$$\begin{cases} v_x = \frac{\partial \varphi}{\partial x} = CKchk(z+H)cos(k \ x - \omega \ t) \\ v_z = \frac{\partial \varphi}{\partial z} = CKshk(z+H)sin(k \ x - \omega \ t) \end{cases}$$

For a certain coordinate point (x_0, y_0) in the movement field, these velocity components of the water particles in the wave are nothing but the derivatives in relation to time, of the current path coordinates expressed in terms of parameter (t).

Consequently, by integrating the expressions representing velocity components according to the two coordinate axes it is possible to obtain the particle path parametric equations. Therefore:

$$\begin{cases} X = v_{x} \\ \dot{Z} = v_{z} \end{cases}^{\text{or}} \\ \begin{cases} \dot{X} = CKchk(z_{0} + H)cos(k x_{0} - \omega t) \\ \dot{Z} = CKshk(z_{0} + H)sin(k x_{0} - \omega t) \end{cases}$$

This infers:

c •

$$\begin{cases} X = \int CKchk(z_0 + H)cos(k x_0 - \omega t)dt = \\ = -\frac{CK}{\omega}chk(z_0 + H)sin(k x_0 - \omega t) + C_1 \\ Z = \int CKshk(z_0 + H)sin(k x_0 - \omega t)dt = \\ = \frac{CK}{\omega}shk(z_0 + H)cos(k x_0 - \omega t) + C_2 \end{cases}$$

$$\begin{cases} \frac{X - C_1}{-\frac{CK}{\omega} chk(z_{0+H})} = sin(k x_0 - \omega t) \\ \frac{Z - C_2}{\frac{CK}{\omega} shk(z_0 + H)} = cos(k x_0 - \omega t) \end{cases}$$

By squaring each of these congruencies and by adding up member by member, it is possible to obtain:

$$\frac{(x-C_1)^2}{\frac{C^2K^2}{\omega^2}ch^2k(z_0+H)} + \frac{(z-C_2)^2}{\frac{C^2K^2}{\omega^2}sh^2k(z_0+H)} = I$$

It is noticeable that each particle path is an ellipse in the vertical plane of the water along the wave propagation direction.

Since x, that is z for t = 0 must coincide with x, that is z for t = T (the movement being periodical), it results $\omega = \frac{2\pi}{T}$

If it is opposed the condition that at surface $(z_0 = 0)$ the small semiaxis of the ellipse must have the value $h/_2$ (half of the wave height) it results:

$$C = \pi h / (KT \sin KH)$$

Therefore:

$$\begin{cases} v_x = \frac{\pi h}{TshKH} chk(z+H)cos(kx-\omega t) \\ v_z = \frac{\pi h}{TshKH} shk(z+H)sin(kx-\omega t) \end{cases}$$

For the actual calculation of the energy contained in the wave, it is necessary to determine the undulated wave surface profile equation.

Because any fluid particle movement can be expressed through a relation among the stress forces, massic forces and inertia forces, the general movement equations are reached [2]:

$$\begin{cases} \rho F_x dxdydz = \frac{\partial p_x}{\partial x} dxdydz + \rho a_x dxdydz \\ \rho F_y dxdydz = \frac{\partial p_y}{\partial y} dxdydz + \rho a_y dxdydz \\ \rho F_z dxdydz = \frac{\partial p_z}{\partial z} dxdydz + \rho a_z dxdydz \end{cases}$$

In which the massic forces were designated by $\overline{F}(F_x, F_y, F_z)$, accelerations were designated by $\overline{a}(a_x, a_y, a_z)$, while the by $\overline{p}(p_x.p_y, p_z)$ normal stress (these were applied in the gravity center of each face of the elementary paralelipipedon dx, dy, dz).

Considering that the specific massic forces (F) derive from potential -U, one can obtain:

$$-d\left(U+\frac{p}{\rho}+\frac{v^2}{2}\right) = F$$
(2)

Because the velocity derives in the case of the waves from the potential function φ , equation (2) can be reduced to the form:

$$U + \frac{p}{\rho} + \frac{v^2}{2} + \frac{\partial \varphi}{\partial t} = 0$$

Replacing U = -gz and neglecting the term p/ρ (because pressure is equal to atmospheric pressure at wave surface) it results for the free wave water surface (η) :

$$\eta = A\cos(kx - \omega t) \text{ in which}$$
$$A = -\frac{l}{g} \frac{\pi \lambda h}{T^2 shkH} chkH$$

In case of waves, the water surface vertical variation velocity can be equal to the normal component of the particle velocity in the same place:

$$\frac{-\partial \eta}{\partial t} = \left(\frac{\partial \varphi}{\partial z}\right)_{z=0}$$

which means:

$$\frac{-\partial \eta}{\partial t} = -\omega A \sin(kx - \omega t)$$

Period (T) results from this equation:

$$T = \sqrt{\frac{2 \pi \lambda}{gthkH}}$$

Replacing :
$$k = \frac{2\pi}{\lambda}$$

$$T = \sqrt{\frac{2 \pi \lambda}{gth \frac{2 \pi H}{\lambda}}}$$

or

$$\lambda = \frac{gT^2}{2\pi} th \frac{2\pi H}{\lambda}$$

4. Energy and Power Calculation Relations

Using these elements, it is possible to calculate the specific energy (e_c) for 1m^2 of sea surface and specific potential energy (e_p) [3]

$$e_{c} = \frac{1}{\lambda} \int_{0}^{\lambda} \int_{-H}^{0} \frac{\rho v^{2}}{2} \cdot l \cdot dx dz = \frac{\gamma h^{2}}{16}$$
$$e_{p} = \frac{1}{\lambda} \int_{0}^{\lambda} \int_{0}^{\eta} \rho gz \cdot l \cdot dx dz = \frac{\gamma h^{2}}{16}$$

Totalizing the two energies, the total mechanical energy of the waves (p) will result:

$$e = e_c + e_p = \frac{\gamma h^2}{8}$$

In order to calculate power (P), the hydrodynamic pressure (F_p) and velocity component along the propagation direction will be used:

$$P = \frac{1}{T} \int_{0}^{T} \int_{-H}^{0} F_p v_x (1 \cdot dz) dt =$$
$$= \frac{\gamma h^2}{16} \cdot \frac{\lambda}{T} \left[1 + \frac{4\pi H_{\lambda}}{sh(4\pi H_{\lambda})} \right]$$

If the unit of measure IS are used:

$$\gamma$$
 = specific sea water weight in N/m³;

h = wave height (m);

 λ = wavelength (m);

T = wave period (s);

H =water depth (m);

wave power results for each linear shore meter in W/m.

In practice, sometimes a specific water weight (γ) is measured in tf/m³ and power (P) per front unit is measured in kW/m, the other values remaining to be measured in the same units of measure. In this case:

$$P = 9.81 \cdot \frac{1}{2} \left[1 + \frac{4\pi H_{\lambda}}{sh(4\pi H_{\lambda})} \right] \frac{\lambda}{T} \cdot \frac{\gamma h^2}{8}$$

Because this expression is too complicated, certain simplifications were tested by replacing

parameter $\lambda = \frac{gT^2}{2\pi} thkH$ and considering that for $4\pi \frac{H}{2}$

great depths ratio $\frac{4\pi H_{\lambda}}{sh(4\pi H_{\lambda})}$ tend towards zero.

Under these conditions:

$$P = \frac{9.81\gamma g}{32\pi} h^2 T \qquad [kW/m]$$

Adopting usual value 9.81 m/s^2 for gravitational acceleration (g), approximate value for 1tf/m³ for specific water power (γ) and value 3.14 for π , a coefficient results in front of product $h^2 T$ that draws it closer to the unit

 h^2T that draws it closer to the unit.

Therefore for the prompt engineering calculations, the simplified relation[4]can be used: $P = L^2 T$

$$P \approx h^2 T$$
 [kW/m]

in which h represents wave height (m) and T wave period (s).

6. Conclusion

In the absence of an exact mathematical solution of wave equation it is possible to approach the sinusoidal wave model as technical application to potential plane movements. The main element in this case is represented by determining the function called velocity potential, which allows calculation in each and every point and moment of the characteristics of the movement of the water particles making up a wave.

The hydraulic engineers should not stop their calculations at determining the pressures exerted by waves on structures. These calculations have to be conducted till assessing the wave specific power, since this assessment is the only one which can explain what is the overall situation of a structure that is stressed by waves.

Because quite often when the wave subsides it carries away fine solid particles from under a protection structure, the assessment of the section should not be neglected since it may represent the cause of the movement of the supporting structures that take place in time, after finalisation of the normal settlements specific to gravitational constructions propped on the sea bottom.

The model presented offers all these facilities.

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Numerical Simulation Method for Unsteady Water Flow in the Underpressure Systems of Pumped Storages

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Abstract : This work contains a simulation procedure for the unsteady flow at a hydroelectric power station with pumping. Knowing the initial conditions in the downstream reservoir, in the water storage basin (pumped), and also the working time period for the reversing turbine, by modeling there are obtained the level variations in both upstream and downstream reservoirs, and the power changing for the reversing turbine. This method is used in order to improve the working program at the hydroelectric power station with pumped storage.

Keywords: pumped water storage, reversing turbine, unsteady flow, numerical simulation.

1. General considerations

This work presents a study for the unsteady flow in hydroelectric systems with reverse pumping. For the numerical simulation of the water supply network a competitive computer software (EPANET2) is used [1].

The main idea is to realize a system with interconnecting pipes between two basins: one for the water storage reservoir - pumped water storage disposed at a high elevation, and the second being the downstream compensation storage.

For the water flowing simulation during the power producing period there is considered a by-pass ductwork at each reversing turbine group, with automatic valves, which are opened and have the propriety to maintain a constant flow towards downstream.

While the reversing turbine works as a pump, at each reversing turbine group there is used a pump with following parameters: discharge and efficiency. By pumping, a water volume is transported from the downstream storage reservoir towards the upstream storage reservoir.

Both storage reservoirs are simulated in computer program as two tanks were water is storage depending on the water levels from the storage reservoirs (effective highs).

2. Unsteady flow simulation in pressured pipe systems

The method used by EPANET2 is based on computing the discharge flows from the continuity equations and the head-loss estimation in the pressured pipe system depending on time (the wellknown gradient method).

The head-loss on a pipeline section with the "*i*" and "*j*" junctions, is:

$$H_{i} - H_{j} = h_{g} = rQ_{ij}^{n} + mQ_{ij}^{2}$$
 (1.1)

where: H - junction head

h-head-loss

r - local resistance coefficient

- Q flow
- n flow exponent
- m local head-loss coefficient

The resistance coefficients are given by the friction coefficient formulae. The head-loss for a pump is:

$$\mathbf{h}_{g} = -\left(\mathbf{h}_{0} - \mathbf{r}\left(\frac{\mathbf{Q}_{ij}}{\omega}\right)^{2}\right)$$
(1.2)

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where: h_0 – pumped loss

 ω – pumped rotation velocity

The junction head-loss has to satisfy the continuity conditions:

$$\sum_{j} Q_{ij} - D_{i} = 0 \quad i = 1,...,N$$
 (1.3)

where: D_i - concentrated flow in the "i" junction, positive at entrance.

For the start moment, the estimated discharge flows through each pipe, obtained by the gradient method, have to satisfy the junction continuity conditions. At each iteration, the new head-loss values can be find using the following matrix relation:

$$\mathbf{AH} = \mathbf{F}$$
(1.4)
where: A – Jacobian quadric matrix

H – unknown head-loss vector.

The Jacobian quadric matrix crossover terms must satisfy the following condition:

$$\mathbf{A}_{ii} = \sum_{i} \mathbf{p}_{ij} \tag{1.5}$$

The other matrix terms have to satisfy the following condition:

$$\mathbf{A}_{\mathbf{ij}} = -\mathbf{p}_{\mathbf{ij}} \tag{1.6}$$

where:

 p_{ij} – reciprocal derivative of local head-loss on a pipeline section with "i" and "j" junctions, in the shape of:

$$\mathbf{p}_{ij} = \frac{1}{nr(\mathbf{Q}_{ij})^{n-1} + 2m\mathbf{Q}_{ij}}$$
(1.7)

Considering one pump the expression (1.7) becomes:

$$\mathbf{p}_{ij} = \frac{1}{\mathbf{n}\omega^2 \mathbf{r} \left(\frac{\mathbf{Q}_{ij}}{\omega}\right)^{n-1}}$$
(1.8)

The terms of the F vector in the relation (1.4) can have a following expression:

$$\mathbf{F}_{i} = \left(\sum_{j} \mathbf{Q}_{ij} - \mathbf{D}_{i}\right) + \sum_{j} \mathbf{y}_{ij} + \sum_{f} \mathbf{p}_{if} \mathbf{H}_{f}$$
(1.9)

Each right hand side term consists of a net balancing flow in a junction, a flow correction factor



Fig. 1 The Tarnita - Hydropower System

and a factor representing the connection to a fixed junction. The following relation gives the flow correction factor:

$$\mathbf{y}_{ij} = \mathbf{p}_{ij} \left(\mathbf{r} \left| \mathbf{Q}_{ij} \right|^n + \mathbf{m} \left| \mathbf{Q}_{ij} \right|^2 \right) \operatorname{sgn} \left(\mathbf{Q}_{ij} \right) \quad (1.10)$$

After new heads are computed by solving the system of equation (1.4), the new discharge flows are found from:

$$Q_{ij} = Q_{ij} - (y_{ij} - p_{ij}(H_i - H_j))$$
 (1.11)

The linear head-loss coefficients are found from Hadgen-Poiseuille equation if Re<2000 and from Colebrook-White if Re>4000.

3. Numerical simulation of the unsteady flow at Tarnita – Hydropower System

The Tarnita – Hydropower System, simulated in this study, is composed by:

the upstream pumped water reservoir (T_4) ,

the head-race tunnel an interior diameter D=6.00 m, the hydroelectric power station with four reversal turbine groups,

two surge tanks (T_6, T_{35}) on the connection tunnels with the tailrace,

four by-pass pipes with valves for flow adjustment for the power production phase,

four pumps with known function parameters, the head-race tunnel, the downstream reservoir.



Fig 2 Details of Tarnita – Hydropower System; up to left: upstream reservoir; down to left: downstream reservoir; on the right Hydroelectric power station.

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Figure no.1 presents the Tarnita Hydropower System. The figure no.2 shows in detail the water storage reservoirs and the hydropower station.

Initial imposed conditions:

- Tarnita downstream reservoir: $N_{max} = 657 \text{ m}$ in elevation with respect to the see level, $N_{min} = 560 \text{ m}$, reference level = 550 m, $D_i = 8.77 \text{ m}$, initial level = 650 m, C_1 – volume change curve;
- Lapustesti upstream reservoir: $N_{max} = 1086$ m, $N_{min} = 1056$ m, reference level = 1046 m, $D_i = 6.20$ m, initial level = 1071 m, C_2 – volume change curve.
- Surge tanks R_6 and R_{35} : $N_{max} = 547.15$ m, $N_{min} = 492$ m, reference level = 446.20 m, $D_i=6.20$ m.

• Hydropower station with four reversing turbine groups with a stated discharge of 115800 m³/h and a water pumping head of 570 m. It works as a hydropower station between 06-09 h and 19-22 h, and as a pumping station between 09-19 h and 22-06 h. The analyzing period of time is of 20 days (472 hours).

4. Results of the numerical simulation at Tarnita – Hydropower System with reversing turbine

After the numerical simulation of the Tarnita – Hydropower System the discharge flow, the velocity and the head behavior with time was obtained for all system junctions over a period of 472 hours.

Figure no.3 presents the discharge flow values and the hydraulic heads in junctions for day five, at



Fig.3 Distribution of the flows and hydraulic load on section of pipelines and in junctions, on day five at 08:00 A.M.

08:00 A.M. when the hydropower station produces electric energy and at 23.00 P.M. when it is used as a pumping station.

Analyzing the discharge flow values given by figure no.3 it is noticed that the head-race discharge flow is 1163200 m³/h, at each group the flow being 290800 m³/h, and that the tailrace discharge flow is $581600 \text{ m}^3/\text{h}$.

When the station works as a pump, the value of the head-race discharge flow is $-599367.40 \text{ m}^3/\text{h}$ (the negative sign corresponds to a flow from "j" towards "i"). Regarding the four groups ($R_1 \div R_4$), the discharge flow values are:

 $-149475 \text{ m}^{3}/\text{h}$ (R₁), $-149784.70 \text{ m}^{3}/\text{h}$ (R₂),

-149977.90 m³/h. (R_3), -150129.80 m³/h (R_4). The corresponding tailrace discharge flow value

is -299259.70 m³/h.

It is noticed that by choosing decreasing values for the connection tunnel diameter: 3.48 m, 2.83 m, 2.00 m, and by assuming that the entering and outgoing diameters for the ramification pipe (2.00 m and 4.00 m) were properly settled, the resulted values of the discharge flows are almost equal.

The water level variation in the Tarnita downstream reservoir is presented in figure no.5.a, and the water level in Lapustesti upstream reservoir determined by pumping is given by figure no.5.b.

It can be noticed that the water level in Tarnita reservoir decreases in 20 days from the initial value of 650 m to 628 m, while the water level from Lapustesti raises from the initial value of 1071 m to 1081 m.



Fig. 4 Distribution of the flows and hydraulic load on section of pipelines and in junctions, on day five at 23:00 P.M.





Fig.5 a. Level varied in downstream reservoir; 5 b. Level varied in upstream reservoir

The increase and decrease in water levels for both reservoirs is favorable due to the fact that it determines an increase the gross head at the hydropower station (max. 32 m) and consequently determines a raise in power. A large decrease in water level for the Tarnita reservoir involves a temporary stop of pumps in order to restore the proper water level.

5. Conclusions

The main idea of this work is to use an ingenious method for numerical simulation of the unsteady flow in hydropower systems with reversing turbines, in order to improve their efficiency.

The best working procedure of the considered hydropower system supposes also to separate the

working periods of time at 24 hours, by coupling or disengaging one or more reversing turbine groups. The water level at the end of the analyzing period of time is close to the initial values.

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Calculating Methods in the Study of Marine Currents

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Abstract The paper presents the most important theoretical methods which were used, for a long time, in the study of marine currents. The emphasize is on the importance of this theoretical methods after many rough results were obtained, by using certain numerical approximation of equations of motion and by processing the measurement results.

Keywords: Marine currents, geostrophic currents, thermohaline circulation

1. Introduction

The marine currents, caused by the motion of marine and occeanic masses, constitute a very complex physical phenomenon. The global study of the marine currents falls within the oceanography field, on the other hand the solution of different theoretical and applicative problems of this phenomenon requires special studies from the hydrodynamics, thermodinamics and physiques fields.

From the hydraulic point of view, the study of marine currents leads to the solutions of the Navier-Stokes equations under special conditions of variation of water salinity and temperature.

The concerns in the study of the currents emerged in the second half of the XIX century, but the most important results were obtained in the XX century, when such oceanographers as Ekman, Sverdrup, Rossby, Stommel, Munk and many others issued real theories for the solution of the problem.

The mathematical methods used by them necessitated, however, wide range simplifications as well as taking into account only some of the factors only which contribute to creating the phenomenon, so that the results obtained generally have generally, a high degree of approximation.

Over the last years, the development of calculating technology, the increase of the capacity of accumulation and of data processing, allowed a spectacular approach to the problem. So, the data obtained with the assistance of satellites were

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reached, using a complex network of electronic computers to study the motion of oceans water at planetary level, in tight connection with the meteorological phenomena. By using numerical methods, especially the finite volume method, they worked on a network summing up 20 000 000 nodes, which resulted in obtaining highly valuable results regarding the water motion of oceans at planetary level [1].

2. General Equations of Motion

Generally, in oceanography, a coordinate system Oxyz is used after the standard convention in geophysical fluid: Ox to the Est, Oy to the North and Oz is up. The state of fluid is known if at each point and every time the following are known: the pressure p(x, y, z, t); the density $\rho(x, y, z, t)$ which is supposed as known by means of the equation of state; the fluid velocity \overline{v} with components u,v,w – all three functions of x,y,z,t; the exterior forces \overline{F} , having the components X, Y, Z – functions of x,y,z,t; the deviation of salinity from the average value-

S(x, y, z, t); the deviation of temperature of the average value- T(x, y, z, t).

The fluid movement is described by means of Navier-Stokes equations of motion. As the movement of oceanic waters is turbulent, the virtual stresses

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were introduced, known as Reynolds stresses, which allowed the use of two dynamic molecular viscosity factors, one for the horizontal direction (μ_h) the other for the vertical direction (μ_v) [2].

The Navier-Stokes equations were put in the form:

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + v \frac{\partial u}{\partial y} + w \frac{\partial u}{\partial z} = X - \frac{1}{\rho} \frac{\partial p}{\partial x} + \frac{\mu_h}{\rho} \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) + \frac{\mu_v}{\rho} \frac{\partial^2 u}{\partial z^2}$$

$$\frac{\partial v}{\partial t} + u \frac{\partial v}{\partial x} + v \frac{\partial v}{\partial y} + w \frac{\partial v}{\partial z} = Y - \frac{1}{\rho} \frac{\partial p}{\partial y} + \frac{\mu_h}{\rho} \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) + \frac{\mu_v}{\rho} \frac{\partial^2 v}{\partial z^2}$$

$$\frac{\partial w}{\partial t} + u \frac{\partial w}{\partial x} + v \frac{\partial w}{\partial y} + w \frac{\partial w}{\partial z} = Z - \frac{1}{\rho} \frac{\partial p}{\partial z} + \frac{\mu_h}{\rho} \left(\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2} \right) + \frac{\mu_v}{\rho} \frac{\partial^2 w}{\partial z^2}$$
(1)

Generally, the above system is processed by the use of some hypotheses of the form:

- the only exterior forces are the Coriolis force (an apparent force produced by the Earth rotation that changes the direction of motion), with non-null components in the directions of axes Ox and Oy and the gravity, the dominant force, with a nonnull component only in the direction of axis Oz (the wind driving force appears in the boundary conditions);

- the vertical velocity component is much smaller than the other two components from the horizontal plane which allowes that within the system (1) this to may be neglected.

The continuity equation is added to the system (1) which, in the case of sea water, we can write as :

$$div\overline{v} = 0 \Leftrightarrow \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0$$
(2)

The other two extremely important components which characterize and determine the oceanic motions are the salinity S and the temperature T. Starting from the necessity of preserving the salt quantity in the presence of turbulent diffusion, by means of some diffusion coefficients, one equation of salinity and one for temperature were obtained [3]. As the density is a function of S and T, the variation being given by the equation of state, there was

obtained the complete system that ties the seven unknown quantities (the three compnonents of velocities, pressure, density, salinity and temperature) witch characterize the motions of the water in seas and oceans.

$$\mu_{h}\Delta u + \mu_{v} \frac{\partial^{2} u}{\partial z^{2}} + fv = \frac{1}{\rho_{0}} \frac{\partial p}{\partial x}$$

$$\mu_{h}\Delta v + \mu_{v} \frac{\partial^{2} v}{\partial z^{2}} - fu = \frac{1}{\rho_{0}} \frac{\partial p}{\partial y}$$

$$\rho g = \frac{\partial p}{\partial z}; \frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0$$

$$\mu_{h}^{S}\Delta S + \mu_{v}^{S} \frac{\partial^{2} S}{\partial z^{2}} = \gamma_{S} w$$

$$\mu_{h}^{T}\Delta T + \mu_{v}^{T} \frac{\partial^{2} T}{\partial z^{2}} = \gamma_{T} w$$

$$\rho = \alpha_{T} T + \alpha_{S} S$$
(3)

The most general form of boundary conditions for the system of differential equations may appear (3) is:

$$u = v = 0; S = f_1; T = f_2$$

for Γ - border of the domain,

$$\frac{\partial S}{\partial z} = f_3; \frac{\partial T}{\partial z} = f_4; w = 0$$

for
$$z = 0$$
, at the sea surface (4)
 $\frac{\partial S}{\partial z} = 0; \frac{\partial T}{\partial z} = 0; w = 0$

for z = H, H- the depth of the sea.

In system (3) and in the conditions (4) besides Coriolis coefficient $f = 2\omega \sin \phi$, where ω is the angular velocity of Earth and ϕ - the latitude, a series of coefficients and functions appear, whose significance we are not dealing with in the present work.

3. Calculation Methods

The system of differential equations (3) cannot be integrated by exact methods. That is why either its simplification was tried, arriving at the reducion of the number of approximate solutions using numerical methods or reaching approximate solutions by using numerical methods. We further present the most important methods which were used, in the course of time, for the solution of the system (3) which means, in fact, the study of the oceanic motions and, therefore, of oceanic and marine currents.

a) The dynamic method: A first method to study the marine currents is the dynamic method, established in 1898 by the Norwegian Bjerknes, in the field of meteorology. It onlytakes into account the distribution of pressures and the Coriolis force, neglecting the viscosity forces. So a current is studied that was named "geostrophic current", similar to the geostrophic wind, which is evaluated on the basis of the measurements made at different hydrological stations. The dynamic method allows the calculation of the average velocity of the current, supposing the permanent motion, without mixture, without friction, without other exterior forces apart from the weight and the Coriolis force, starting only from the repartition of densities. Marking with D the geopotential or dynamic depth of the isobar, the two horizontal velocity components were obtained:

$$u = -\frac{1}{2\omega\sin\phi} \frac{\partial D}{\partial y}$$

$$v = \frac{1}{2\omega\sin\phi} \frac{\partial D}{\partial x}$$
(5)

b) Ekman 's Theory: At the beginning of the XX century, Ekman studied the currents called "drift currents", caused by the wind. The sea was supposed flat (ϕ -const) of constant density, with constant viscosity coefficients, over which a wind of constant speed and direction blows, so that the water motion becomes permanent. The exterior forces that appear are the superficial driving wind force which appears only in the boundary conditions and the Coriolis force. The vertical component of the velocity is considered as being null. At sea, in uniform field of velocities, we have a permanent motion regime. Consequently, the first and second degrees derivatives of the velocity in the horizontal directions are null, from which results that u and v are functions only of z. The equations of motion are written under the form:

$$\mu_{\nu} \frac{d^{2}u}{dz^{2}} + 2\omega\nu\rho\sin\phi = 0$$

$$\mu_{\nu} \frac{d^{2}v}{dz^{2}} - 2\omega\mu\rho\sin\phi = 0$$
(6)

The boundary conditions which are associated with this system are:

$$\mu_{v} \left(\frac{du}{dz} \right)_{z=0} = 0 \qquad \mu_{v} \left(\frac{dv}{dz} \right)_{z=0} = \tau_{y} \tag{7}$$

where it was supposed that the wind driving force, of size τ , is tangent to the surface, parallel to Oy.

From the system of equations (6), accompanied by the conditions (7), Gougeheim and Saint-Guilly have obtained accurate solutions, for the cases of the sea of infinite depth, as well as for that of the sea of finite depth, using the theory of functions of complex variable [2].

c) The method of the slope currents: If it is supposed that out at seas there is a permanent slope of the sea surface, and the sea is considered as homogeneous, in the absence of the wind, the so called slope currents are found. It is supposed that the pressure gradient at any depth has the same size and the same direction, which means that, in the absence of friction, the motion is the same at any depth, equivalent to the fact that the relative velocities of the different water strata are null. Considering the descending slope in direction of axis Oy and marking with η the level of the free surface, the system of equations of motion is obtained:

$$\mu_{h} \frac{d^{2}u}{dz^{2}} + 2\omega \sin \phi \rho v = 0$$

$$\mu_{h} \frac{d^{2}v}{dz^{2}} - 2\omega \sin \phi \rho u = g \frac{\partial \eta}{\partial y}$$
(8)

system which was solved with the assistance of the functions of complex variable, in the assumption that the free surface is known, which is only possible by knowing the expression of the wind driving force [2].

d)Sverdrup's Theory: The real motions are, generally, much more complex, which makes that the approximations given by the methods presented 1)-3) to describe only partially the complicated phenomenon of oceanic motions.

Sverdrup (1947) performed a synthesis between the currents, in the presence of a given distribution of the density in the viscous fluid and a wind deduced from the climatological conditions [4]. He inaugurated the method of the integration of the wind velocity between a reference surface and the sea surface. It started from the equations of motions, where the rectangular terms of the acceleration and the lateral efforts were neglected, and the motion was supposed as permanent:

$$\frac{\partial p}{\partial x} = 2\omega \sin \phi . \rho . v + \mu_v \frac{\partial^2 u}{\partial z^2}$$

$$\frac{\partial p}{\partial y} = -2\omega \sin \phi . \rho . u + \mu_v \frac{\partial^2 v}{\partial z^2}$$
(9)

The equations were integrated vertically with the assistance of P function, defined by the relations:

$$\frac{\partial P}{\partial x} = \int_{-d}^{0} \frac{\partial p}{\partial x} dz \qquad \frac{\partial P}{\partial y} = \int_{-d}^{0} \frac{\partial p}{\partial y} dz \tag{10}$$

where d is the depth where the slopes of izobar surfaces are cancelled.

The P function, defined by (10), may be calculated on the basis of the measurements made on the field of temperature and salinity at different hydrological stations. The functions which measure the mass transport in parallel with the horizontal axes Ox and Oy were also introduced:

$$M_{x} = \int_{-d}^{0} \rho u dz \qquad M_{y} = \int_{-d}^{0} \rho v dz \qquad (11)$$

Sverdrup obtained, only subject of the elements that characterize the wind driving force, expressions for the P function and for the two functions defined by (11). The results obtained describe the motion, not by the velocity components, but by the mass flow that moves in the directions of the horizontal coordinate axes. But they require the knowledge of the wind driving force. Formulas that allow the calculation of the partial derivations of P, subject of the components of the wind driving force were obtained. As the function may also be calculated on the basis of the hydrological measurements, the comparison of the two sets of results allow the evaluation of the above presented calculation method and known under the name of Sverdrup's synthesis.

e) Stommel's Theory: Stommel utilized the integration of equations of vertical motion assuming the ocean as homogenous and rectangular, of dimensions λ and b [2]. For the wind stress he chose

a distribution of the type
$$-F\cos\frac{\pi y}{h}$$
 and he

introduced a friction linear term, proportional to the velocity and opposited to this one. The motion was considered also as permanent and again the rectangular terms of the acceleration were neglected. By means of the determination of this function a differntial equation with partial derivatives of second order was found:

$$\Delta \varphi + \alpha \, \frac{\partial \varphi}{\partial x} = \gamma \sin \frac{\pi y}{b} \tag{12}$$

The equation was integrated in the case the ocean's shores, assumed as rectangular, may be considered as being lines of current, that are lines by which the flow is null(boundary conditions).

f) Munk's Synthesis: Munk presented a study of the circulation induced by the wind, taking into acount the lateral tangent efforts, the statistical wind, the mass transport and the variation of Coriolis coefficient with the latitude. He neglected the non-linear terms of the acceleration, he abandoned the homogenous ocean of Ekman and Stommel, resuming the P function of Sverdrup. The equations of motion from which he started were, this time, more complete and they were written under the form:

$$\frac{\partial p}{\partial x} - f\rho v - \mu_h \left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \right) - \mu_v \frac{\partial^2 u}{\partial z^2} = 0$$

$$\frac{\partial p}{\partial y} + f\rho u - \mu_h \left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \right) - \mu_v \frac{\partial^2 v}{\partial z^2} = 0$$
(13)

By elimination of the pressure and the introduction of the streamfunction ψ a differential equation was also reached with second degree partial derivatives:

$$\left(\nu_{h}\Delta^{4} - \beta \frac{\partial}{\partial x}\right)\psi = -rot\overline{\tau}$$
⁽¹⁴⁾

where v_h is the horizontal kinematic molecular viscosity, $\overline{\tau}$ is the vector with components τ_x, τ_y (the wind driving force) and Δ^4 is the differential operator :

$$\Delta^4 = \frac{\partial^4}{\partial x^4} + 2\frac{\partial^4}{\partial x^2 \partial y^2} + \frac{\partial^4}{\partial y^4}.$$

g) Numerical Methods : In the study of marine currents the two known numerical methodes are used : the method of finite differences and that of finite volumes. The use of these methods offers many advantages. They simulate the motion in real basins and take account of the influence of viscosity and the non-linear terms in the equations of motion, being

able of the observations obtainted by the ships, buoys, drifters and satellites.

Generally, flat models for stationary motion are used. The matter of the integral circulation in the oceans under the wind action constitutes a classic model and the solution of the problem supposes the use of the limit stratum theory, especially in the initial schemes, which must take into account the bondary conditions. The application of the numerical methods for the solution of the system (3) necessitate running through two stages: a) the approximation of the operators from the differential equations system (or integral-differential) and their simplification and b) the approximation of equations system subject the time.

The complexity of the system (3) the use of some approximations. The real motion of oceans water is more complex. The system was obtained, already, by using the hydrostatic approximation and the Boussinesq approximation. Most of the time, an integration of the equations of vertical movement is made and the average plane movement is studied. Highly complicated is proved to be the problem of the initiation of the model. For every model the velocity and the density at the initial moment must be known.

Currently used methods often set off from the results obtained and comprised in Levitus Atlases or from the results obtained with highly simplified methods. The problem is brought to the form>

$$L\phi = F \tag{15}$$

The used methods can be preserved in conservative methods and dissipative methods. The conservative methods preserve different aspects, on the analogy with the basis power relations characteristic of the differential model, necessary to the modelling of long stading tied mainly to the energy preservation. The so called power models are obtained, which refer to the conservation of kinetic energy in the mechanical work done by wind speed, by the border forces or by the loss of energy due to the fluid dissipation at the side borders.

The dissipative models are used in the solution of the local character problems and of those where mainly the viscosity is taken into account. The deficiency of the dissipative schemes consists in that the power result being distorted, the energy exchange with the surrounding strata of the ocean must be taken into account. It was established that the study of the problem in the surrounding strata may be done in coarse networks, of poor location in space [3]. Richardson introduced a method, on the basis of a dissipative scheme, as a combination of numerical solutions obtained by a succession of networks of different densities [5]. The calculation methods for the study of drift currents, in the case of the plane stationary motion present a special interest [6].

4. Conclusions

Marine currents, determined by the movement of the marine and oceanic masses, represent a very complex physichal phenomenon. The global study of marine currents is a part of the ocenographydomain, but the solutions to the many theoretical and practical problems of this phenomenon need special studies into the hydrodynamic's, thermodynamic's and physic's domains.

Concerns about the study of the marine currents existed since the last century, but the development of calculating technology allowed a very important approach to the problem, for understanding this natural complex phenomenon.

The calculating methods here described emphasize the limits of analytical methods as well as the importance of numerical calculating methods for more precise results.

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Mathematical Form of an Installations of Producing and Using Compressed Air

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Abstract: The air under pressure, is efficiently used in the mechanization and the automatization of the civil buildings works, in processes of the advanced treatment of water for waking the waters drinking and their combing out, etc. The mathematical modelation of the air sources under pressure, offer the possibility of correct dimensioning of air network depending on the user. The proposed method is helpful for the unliniar equations systems from the pneumatical automatic systems. The results way be applied in the whole domain of work.

Keywords: pneumatical systems, compressed air, and controller.

1. Introduction

The pneumatic actions is one of the most efficient ways of the mechanization and automatization in building works, of the modern equipment and tools. That is why we need to know the mathematical forms with which are made evident designing principles with scientific bases of the pneumatical systems.

The air under pressure is used in industrial processes to operate multi-purpose machines, to start naval engines, in automatic systems in technological processes and to operate pneumatic robots. The pneumatic system are used in building low-powered machines, low - mecanizated mechanisms and tools and regulating systems, to realize some binary functions etc. These systems use the energy of the compressed air that can be:

- Potential energy because of the static pressure increasing of the air; the systems which use the potential energy are named also pneumatic systems;
- Kinetic energy represent air moving energy; the systems that use this energy are named pneumodinamic systems, being less used, especially at the engines with high speed turbines [1].

The main ranges of application of the pneumatic actions are: building of low mechanized tools (pneumatic hammers, punches etc.), building of the

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break arrangements of the heavy vehicles; moulding equipment in casting bays; holding of device at machines-tools; aggregates for handling in the building materials industry, paper industry, light industry, food industry, etc.

The main advantages of the pneumatical action systems are [4]: weight lower than that of mechanic or electronic ones (used in hand tools domain); they admit overloading till the complete outage with no mishaps, low accident risks good functioning in dust laden air etc.; they don't influence the average through dribbling great possibilities of setting, multisupplying, possibilities to supply the energy, to realise some driving binary functions; these systems are also recommanded to realise some equipments of actioning or switch plants (such as pneumohydraulic, electro- pneumohydraulic ones etc.) [5].

The main disadvantages of the pneumatic action systems owe to the physical nature of the processes of producing supplying and using pneumatic energy, which make the compressed air to be one of the most expensive energy agents used in technique. There may appear energy less when compressing (about 28 %), internal energy less in cooling tank (about 19 %), internal energy less in adiabatic expansion, in the engine (about 36 %). So the total average less are about 83 % and a using energy of about 17 %.

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2. The installation presentation

The installation presented in Fig.1 is made up of the following main elements:



Fig.1. The proposed installation scheme.

SP – pressure source (compressor);	TP – pressure translator;
A – accumulator (tank);	FA – air filter;
DP – proportionate distributor;	RG – regulating cock;
PC – controller (electronic regulator);	M – manometer.

- the pressure source (compressor) – SP- gives air under pressure p_o having the feed Q_i ; when entering

- the compressor there is an air filter –FA- to retain the impurities;

- the proportionate distributor –DP- regulates the feed from the tank;

- the accumulator (tank) –A- maintains the air at the pressure *p*, pressure regulated by a controller of pressure (electronic regulator) –PC- that reads the information from the translator of pressure –TP- and action upon the feed using the proportionate distributor; when getting out of the accumulator, the evacuation feed will be controlled through a regulating cock.

The pressure in the system is regulated depending on the consumers' request.

The electronic regulator mentains in accumulator the pressure p at the value p_i . When $p > p_i$, the electronic regulator orders the closure of the proportionate distributor.

3. The mathematical form of the proposed installation

The mathematical form begins from the continuity [2]:

$$\frac{V \cdot M}{R \cdot T} \frac{dp}{dt} = \text{Qi-Qe} \tag{1}$$

Spelling,

$$\mathbf{p} = \mathbf{p}_{o} + \frac{RT}{VM} \int_{0}^{t} (Qi - Qe)dt$$
⁽²⁾

$$\Delta p = \frac{RT}{vM} \int_{0}^{t} (Qi - Qe)dt$$
(3)

The relations (2) and (3) show that the accumulator seems to be an integrating element for the in-feed. But the distributing elements and the pressure differences between them influences the feeds.

Supposing a turbulent flow, we can write the relations (4) [3], [5], [6]:

Qi = k_e
$$\sqrt{(po-p)\rho_i}$$
 [Kg/s] and $\rho_i = \frac{M}{RT} \cdot p$

Qe = k_v
$$\sqrt{(p - p_e)\rho_e}$$
 [Kg/s] and $\rho_i = \frac{M}{RT} \cdot p_e$
(4)

The symbols are:

- T the temperature of the fluid;
- M the air molecular weight;
- p_o- the source pressure;
- p the accumulator pressure;
- p_e the pressure in the system ;
- ρ_i , ρ_e the density of the air when enterring and getting out;
- R the universal.gas constant;
- Qi,Qe the air input feed and output feed;
- V- the volume of the accumulator;
- k_v,k_e the control coefficient of the distributor and of the cock;

The densities are expressed using the air parameters.

Using (4) in (1), we obtain an unliniar differential equation:
$$\frac{VM}{RT}\frac{dp}{dt} = kv \cdot \sqrt{\frac{M}{RT}p(p_0 - p)} - ke\sqrt{\frac{M}{RT}p_e(p - p_e)}$$
(5)

Deviding the relation (5) to $\sqrt{\frac{M}{RT}}$, we obtain:

$$\sqrt{\frac{V^2 M}{RT}} \frac{dp}{dt} = kv \cdot \sqrt{p(p_0 - p)} - ke\sqrt{p_e(p - p_e)}$$
(6)

If noting with $a = \frac{1}{M_a} \sqrt{\frac{RT_a}{M_{aer}}}$, so,

$$\frac{dp}{dt} = a \left[kv \sqrt{p(p_0 - p)} - ke \sqrt{p_e(p - p_e)} \right]$$
(7)

normally, the equation (7) muste made be liniar near a point, but integrating and using the function "Odesolve" from the "Mathcad" program, it could be integrated and presented graphical.

4. Numerical application

The transformation: an air source having a compressor and a bottle for supplying and a regulator to set the pressure.

It is asked to study the pressure variation [6].

T = 293 K;

$$M = 16 \text{ kg/mol};$$

$$V = 4.200 \text{ m}^{3};$$

$$R = 8315 \text{ J/mol.K};$$

$$p_{o} = 5 \text{ bar}; \quad p_{i} = 3 \text{ bar}; \quad p_{e} = 1 \text{ bar};$$

$$1 \text{ bar} = 10^{5} \text{ Pa};$$

$$Q_{i} = 750 \text{ l/h}; \quad Q_{e} = 50 \text{ l/h};$$

$$\rho_{i} = M \text{ x } p_{i} / \text{ R x } \text{ T}_{a}; \quad \rho_{i} = 1.970 \text{ kg/m}^{3};$$

$$\rho_{e} = M \text{ x } p_{e} / \text{ R x } \text{ T}_{a}; \quad \rho_{e} = 0.657 \text{ kg/m}^{3};$$

$$k_{v} = Q_{i} / [(p_{o} - p_{i}) \rho_{i}]^{1/2}; \quad k_{v} = 3.319 \text{ x} 10^{-4} \text{ m}^{2};$$

$$k_{e} = Q_{e} / [(p_{o} - p_{i}) \rho_{e}]^{1/2}; \quad k_{e} = 3.319 \text{ x} 10^{-4} \text{ m}^{2};$$

$$q = (1/V) \text{ x} (\text{RxT/M})^{1/2}; \quad q = 92.909 \text{ 1/m}^{2}\text{ s};$$

$$f(t,p) = k_{v} [p(p_{o}-p)]^{1/2} - k_{e} [p_{e} (p-p_{o})]^{1/2};$$

$$t_{o} = 0; \quad y_{o} = p_{o}; \quad t_{i} = 30;$$
Given>
$$y'(t) = a \text{ x} f(t, y(t)); \quad y(t_{o}) = y_{o}; \quad p = \text{ Odesolve}(t, t_{i})$$



Fig.2. The variation diagram pressure - time.



Fig.3. The variation diagram feed – pressure.

5. Conclusions

• Using the simulation, the equation (6) was solved, equation that expressed the variation of the pressure in the accumulator depending on consumption;

• The hydraulic sources may be correctly sized and we obtain hydraulic equipments with lowweight;

• The maximum amplitudes are made evident, amplitudes which, when becorning linear, they are flattened;

• The results are applied in working and they are real in a single point.

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The Energetical and Piezometrical Lines for a Variable Section Pipeline (Conical Pipeline)

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Abstract: This paper presents how it must treat the problem concerning the energetical and piezometrical lines for a variable section pipeline (conical pipeline). Were determined the equations which permit to trace these lines. The end of this paper contains a numerical example.

Keywords: conical pipeline, energetical line, piezometrical line, pressure, hydraulic charge, conical angle.

1. Introduction

The most used pipeline is the simple pipeline. In this case, the energetical line (Le) is parallel to the piezometrical line (Lp).

Because the section of the conical pipeline

increases, the velocity decreases and the term $\frac{\alpha v^2}{2g}$,

which represents the distance between the two lines, decrease continuously.

The justification of this work's elaboration is that, in the literature, this problem is not treated.

2. Mathematical model

For a pipeline with the constant discharge and section, by using the Chèzy's formula, we can write:

$$J = \frac{v^2}{C^2 R} = \frac{Q^2}{C^2 A^2 R} = \frac{h_r}{l} \qquad(2)$$

$$h_r = MQ^2$$

where: J represents the hydraulic slope, v - velocity, C - Chèzy's coefficient of the hydraulic resistance, A - section area, R - the hydraulic radium, h_r - the energy loss, M - the hydraulic characteristic, Q - the discharge, l - the pipeline's length.

From relation (1), we can write the relation for $\ensuremath{\mathsf{M}}$

$$M = \frac{l}{C^2 A^2 R} \tag{2}$$

By replacing the each term of relation (2) with

the relations
$$C = \frac{1}{n}R^{y}$$
, $A = \frac{\pi D^{2}}{4}$ si $R = \frac{D}{4}$ result:

$$M = \frac{l}{\frac{1}{n^2} \left(\frac{D}{4}\right)^{2y} \frac{\pi^2 D^4}{16} \frac{D}{4}} = \frac{4^{2y+3} n^2 l}{\pi^2 D^{2y+5}}$$
(3)

By compare to the case of the simple pipeline, which has M and Q constant along the pipeline, in the case of the conical pipeline, the parameter M_x depends on the section x.

For an infinitesimal length element dx of a conical pipeline, the relation calculates dhr is:

$$dh_r = dM_x Q^2 \tag{4}$$

The differential of the M_x in the section x is:

$$dM_x = \frac{dx}{C_x^2 A_x^2 R_x} \tag{5}$$

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By replacing $R_x = \frac{D_x}{4}$ and by using the approach relation of Pavlovski for the Chèzy's coefficient $C_x = \frac{1}{n}R_x^y = \frac{1}{n}\left(\frac{D_x}{4}\right)^y$, the relation (5) becomes:

$$dM_x = \frac{64n^2}{\pi^2} \frac{4^{2y}}{D_x^{5+2y}} dx$$
(6)

Integrating the relation between the limits 0 and x, the medium value of the hydraulic characteristic \overline{M}_x is:

$$\overline{M}_{x} = \frac{4^{2y+3}n^{2}}{\pi^{2}} \int_{0}^{x} \frac{d\xi}{D_{x}^{5+2y}}$$
(7)

The relation does the diameter in the section x:

$$D_x = D_0 + 2xtg\theta \tag{8}$$

where D_0 is initial diameter of the pipeline, in the entering section.

By solving the integral, the relation (7) becomes:

$$\overline{M}_{x} = \frac{4^{2y+3}n^{2}}{\pi^{2}} \frac{1}{2(4+2y)tg\theta} \left[D_{0}^{-4-2y} - (D_{0} + 2xtg\theta)^{-4-2y} \right] =$$
(9)
$$= \frac{4^{2(y+1)}n^{2}}{\pi^{2}(2+y)tg\theta} \frac{1}{D_{0}^{4+2y}} \left[1 - \left(\frac{D_{0} + 2xtg\theta}{D_{0}} \right)^{-4-2y} \right]$$

If $\theta \to 0$ (the case of the simple pipeline) and x=L, it obtains the undetermination $\frac{0}{0}$. The limit is solving by Hôpital method:

$$\lim_{\theta \to 0} \frac{1 - \left(\frac{D_0 + 2Ltg\theta}{D_0}\right)^{-4 - 2y}}{tg\theta} = \frac{(4 + 2y)\frac{2L}{D_0}}{1} = 4l\frac{(2 + y)}{D_0}(10)$$

Replacing the relation (10) in (9), we find again the relation (3) from the simple pipeline.

We note
$$c_1 = \frac{4^{2(y+1)}n^2}{\pi^2(2+y)tg\theta}\frac{1}{D_0^{4+2y}}$$
 the

relation (9) becomes:

$$\overline{M}_{x} = c_{1} \left[1 - \left(\frac{D_{0} + 2xtg\theta}{D_{0}} \right)^{-4-2y} \right]$$
(11)

The energy loss $h_{r_{0-x}}$ is done by the relation:

$$h_{r_{0-x}} = \overline{M}_{x}Q^{2} = c_{1}\left[1 - \left(\frac{D_{x}}{D_{0}}\right)^{-4-2y}\right]Q^{2}$$

$$h_{r_{0-x}} = c_1 \left[1 - \left(\frac{D_0 + 2xtg\theta}{D_0} \right)^{-4-2y} \right] Q^2 = c_1 \left[1 - \left(1 + \frac{2xtg\theta}{D_0} \right)^{-4-2y} \right] Q^2$$
(12)

For x=l the energy loss is:

$$h_{r_{0-x}} = c_1 \left[1 - \left(\frac{D_0 + 2ltg\theta}{D_0} \right)^{-4-2y} \right] Q^2 = c_1 \left[1 - \left(1 + \frac{2ltg\theta}{D_0} \right)^{-4-2y} \right] Q^2$$
(13)

To obtain the relations necessary to trace the energetical line, we apply the Bernoulli's relation between the initial section 0 and the section with x abscise:

$$\frac{v_x^2}{2g} + \frac{p_x}{\gamma} + z_x = \frac{v_0^2}{2g} + \frac{p_0}{\gamma} + z_0 - h_{r_{0-x}}$$
(14)

Because $z_x=z_0$ (we consider an horizontal pipeline), we obtain:

$$\frac{p_x}{\gamma} = \frac{p_0}{\gamma} + \frac{v_0^2}{2g} - \frac{v_x^2}{2g} - \overline{M}_x Q^2$$
(15)

From relation (15) we can determine the calculus formula for the pressure in the section x:

$$p_{x} = p_{0} + \frac{\rho}{2} \left(v_{0}^{2} - v_{x}^{2} \right) - c_{1} \left[1 - \left(\frac{D_{x}}{D_{0}} \right)^{-4-2y} \right] Q^{2} (16)$$

Replacing the continuity relation $Q = v_x A_x = v_0 A_0$, the relation (16) becomes:

$$p_{x} = p_{0} + \frac{8\rho}{\pi^{2}} \frac{Q^{2}}{D_{0}^{4}} \left[1 - \left(\frac{D_{0}}{D_{x}} \right)^{4} \right] - c_{1} \left[1 - \left(\frac{D_{x}}{D_{0}} \right)^{-4-2y} \right] Q^{2}$$
(17)

If we note
$$c_2 = \frac{8\rho}{\pi^2 D_0^4}$$
, results:

$$p_{x} = p_{0} + \left\{c_{2} - c_{1} - \left(\frac{D_{x}}{D_{0}}\right)^{-4} \left[c_{2} - c_{1} \left(\frac{D_{x}}{D_{0}}\right)^{-2y}\right]\right\} Q^{2}$$
(18)

The relation for trace the piezometrical line is:

$$\frac{p_x}{\gamma} = \frac{p_0}{\gamma} + \frac{1}{\gamma} \{ c_2 - c_1 - \left(\frac{D_x}{D_0} \right)^{-4} \left[c_2 - c_1 \left(\frac{D_x}{D_0} \right)^{-2y} \right] \} Q^2$$
(19)

In the section x=L, the relation (19) becomes:

$$\frac{p_{L}}{\gamma} = \frac{p_{0}}{\gamma} + \frac{1}{\gamma} \{c_{2} - c_{1} - \left(\frac{D_{0} + 2Ltg\theta}{D_{0}}\right)^{-4} \left[c_{2} - c_{1}\left(\frac{D_{0} + 2Ltg\theta}{D_{0}}\right)^{-2y}\right] \} Q^{2}$$
(20)

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We obtain the relation necessary to trace the energetical line by adding to relation (19), the kinetically term $\frac{\alpha v^2}{2g}$:

$$H_{x} = \frac{p_{0}}{\gamma} + \frac{1}{\gamma} \left\{ 2c_{2} - c_{1} - \left(\frac{D_{x}}{D_{0}}\right)^{-4} \left[c_{2} - c_{1}\left(\frac{D_{x}}{D_{0}}\right)^{-2y}\right] \right\} Q^{2}$$
(21)

and in the section x=L:

$$H_{L} = \frac{p_{0}}{\gamma} + \frac{1}{\gamma} \left\{ 2c_{2} - c_{1} - \left(22\right) \right\}$$
$$\left(\frac{D_{0} + 2Ltg\theta}{D_{0}}\right)^{-4} \left[c_{2} - c_{1} \left(\frac{D_{0} + 2Ltg\theta}{D_{0}}\right)^{-2y} \right] Q^{2}$$

These relations are valid for the divergent pipeline and for the convergent pipeline.

3. Numerical example

We consider a conical pipeline with the initial diameter $D_0=100$ mm, the final diameter $D_f=300$ mm, for divergent pipeline and initial diameter $D_f=100$ mm, for convergent pipeline. The pipeline length is L=2m, the discharge Q=0,1m³/s, the initial pressure $p_0=60000$ N/m².



Fig. 1 The energetical and piezometrical lines for the divergent pipeline



Fig. 2 The energetical and piezometrical lines for the convergent

4. Conclusions

From these graphics result the conclusion that the energetical curve form is approach the same for both cases: divergent pipeline and convergent pipeline. Regarding the piezometrical line, we observe that in the first part of the divergent pipeline is produced an quick increase of pressure, the increase rate decrease continually according as we remove from the initial section (fig.1).

For the convergent pipeline, the pressure variation produces inverse like in the divergent pipeline, that means that the decrease rate of the pressure is slowly in the first part of the pipeline, increasing according as we remove from the initial section (fig.2).

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Utile Volumes of Pumped Storage Reservoir

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Abstract : This paper present the determination of theoretical useful capacity for pumped storage reservoir in three cases: a). when pumping is done with full installed power;

b). when in working days and in the night after a working day function with a part (n) from pumps power;

c). when in working days function with a part (n) from pumps power; for daily and weekly compensation.

It is presented a comparison among three cases, pointing the principal hydropower parameters of pumped storage which are involve in this calculations: fill up time for upper reservoir, installed discharge, pumped flow, turbine flow, absorption power.

Keywords: Energetic system, head curves, daily-weekly compensation, and cycle of pumping-turbinating, pump-turbine installation, hydropower parameters of pumped storage.

1. General considerations

In order to determine theoretical util volume of upper reservoir from a pumped storage, having a pump-turbine installation where establish several calculation formulas. The formulas for pumping with daily compensation (V_{uz}) and weekly compensation with one or two non-working days (V_{us1} and V_{us2}) are establish depending by impute parameters on the head curve of energetic system (α , t_T, t_P, t_P) and principal parameters of pumped storage (Q_T, Q_P, H, P_T, P_P).

Values and variation of util volume, depending on hydropower parameters of power station and impute parameters on the head curve, are presented on tables and diagrams. Depending on pumping cycle, we analyzed same aspects regarding principal hydropower parameters of a pumped storage.

As we know pumped storage are installations in which electrical energy is produced by turbinating a water quantity which partial or total, is pumped from lower reservoir to upper reservoir. Such installations are capable to accumulate energy in medium consumption hours and to cover the top consumption hours of the energetic system. For storage and pumping installations we separate three type of schemas:

- "Pure" pumped storage power stations (pumped storage), in which all turbinate water volume came from lower reservoir, from where is pumped up. pumped storage is a power consumer, for 1 kWh produced it consume by pumping 1.30-1.35 kWh.

- "Mix" pumped storage power stations (power system - pumped storage), in which the turbinate water volume is obtain from pumping but also gravitational.

- "Open circuit" pumped storage power stations, as pumping stations which pump collected water though secondary upstream waterway situated at lower levels than principal storage. Such storage which represent a way to increase medium discharge though pumping, in order to realize power station storage schema based on discharge and head concentration, make possible to increase the utility degree of hydropower potential on river. Such types of storage was made at Lotru-Ciunget power system (3 pumping station with $P_i=62$ MW; Stations Petrimanu, Jidoaia, Balindru), Sebes-Sugag power system and Galceag (pumping station Cugir with $P_i=6$ MW) and Dragan-Remetii power system (pumping station Secu with $P_i=4.3$ MW).

From the energetic point of view pumped storage could be power stations with daily, weekly and

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seasonal pumping depending on fill up time of upstream and downstream reservoirs.

pumped storage with daily pumping cycle, where pumping is made in minimum consumption hours and turbinate in maximum consumption hours every day, have reservoirs which fill up in at least 5-7 hours.

pumped storage with weekly pumping cycle, where pumping is made in each day in minimum consumption hours but even in minimum consumption hours from non-working days and the water is turbinate in maximum consumption hours of working days. These category of pumped storage need bigger volume than daily cycle (minimum 24-26 hours) but increase minimum head on nonworking days improving in these wav thermoelectric power station and nuclear power station exploitation.

Pumped storage with seasonable pumping cycle, where the pumping is made in high water period of the year, when there is energy from power station and the water is turbinate in low water period and high power consumption. These types of pumped storage have big capacity reservoirs, which need a fill up period of several months. These pumped storage allowed to storage a part of pumped water during the summer period and to turbinate it in winter.

2. Reservoirs

Pumped storage reservoirs determine the size and distribution in time, as well as the ways of usage of the power stations in national energetic system.

Upper reservoir capacity is compose from theoretical useful water capacity, dead volume, water volume reserved for short time damage interventions, wave control storage volume, flood water storage volume, etc.

Though theoretical useful water capacity we understand the water capacity needed for pumping in order that, the power plant to participate at the coverage of top consumption from energetic system. Its value depends of installed power of pumped storage head, work regime of pumped storage, type of storage (daily, weekly or seasonal regularization), type of pumped storage (mix or pure) etc. In next part are establish formulas for determination of theoretical volume for upper reservoir of pumped storage with daily cycle (V_{uz}) and weekly cycle (V_{us1} , with one non-working day V_{us2} , with two non-working days) equipped with pump-turbine systems.

For utile volume is necessary to add to theoretical volume, the water volume lost in derivation system and in lower reservoir though infiltration and evaporation from storage etc. Also at the upper reservoir volume it is add a volume over theoretical volume, which allowed a normal function of pumped storage for $\frac{1}{2}$ -1 hour as reserve in case of a short time damage in the system.

Dead volume of the upper reservoir represents the water quantities, which remain in the reservoir, after all useful volume of the storage is finished.

The dead volume is chose from calculation for non allowed the destruction of bottom coverage of the lake with ice, even from calculation for non- allowed washed velocity in lake at the end of function cycle in pumped storage with non consolidated bed. About ice, it had to be providing a reserve of 1.00 m over the upper part of depth to not-allow the ice getting in upstream waterway. For bottom protection is provided "ice reserve". Generally is provided as 10% from utile volume of the lake as ice reserve which can be used in non-frozen period as damage reserve, increase the reserve intended in these purpose for energetically system.

Determination of characteristic parameters for reservoirs and constructive solutions for storage are made on the base of technique-economics fundaments though the comparison for the proper basin in the area.

3. Determination of theoretical utile volume

<u>Case A:</u> Pumping is made with full installed power.

The following relations between the parameters for electromagnetic head curve was obtain in paper 1 from the conditions that pumping volume during a week is equal to turbinate volume.

$$t_T = \alpha \cdot \left(\frac{r}{r'} \cdot t_P + t'_P \right) \tag{1}$$

where:

 $\label{eq:transform} \begin{array}{l} t_T - turbinate \ period \ in \ working \ days \ (hours/day) \\ t_{P} - \ turbinate \ period \ in \ non \ working \ weekend \ (hours/day) \end{array}$

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 t_{P} turbinate period in the night between a nonworking day and the working days (hours/day)

r - number of non working days in a week when the water is pump.

r' - number of working days in a week when the water is pumped.

 α - pump and turbine flow ratio $(Q_{p}\!/Q_{t})$ named reversibility coefficient

Pumping and turbinating times are related to the head curve shape, that could justify or not the installation of pumped storage power station. The influence of pump-turbine installation is manifested also on utile volume.

In table 1 and 2 are represented values for t_T depending on α , t_{P} , t_P and compensation degree of pumped storage using relation (1).

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DAILY COMPENSATION

 $t_T = \alpha \cdot t'_P$ (hourd/day)

Table	<i>3</i> I.		
$t'_{P} \alpha$	1	5	10
0.6	0.6	3.0	6.0
0.7	0.7	3.5	7.0
0.8	0.8	4.0	8.0

WEEKLY COMPENSATION t_T (hours / day)

Table 2 5 10 α t'_P/t_P 1* 1* 2* 1* 2* 2* 7.9 8 1.4 2.5 3.8 4.9 6.8 0.6 12 1.8 3.5 4.2 5.9 7.2 8.9 16 4.4 4.6 6.8 9.8 2.2 7.6 20 2.6 5.4 5.0 7.8 8.0 10.8 1.7 2.9 4.5 5.7 8.0 9.2 8 0.7 12 2.1 4.1 4.9 6.9 8.4 10.4 7.2 5.1 5.4 8.9 11.4 16 2.6 5.9 20 3.1 6.3 9.1 9.4 12.6 8 1.9 3.3 5.1 6.5 9.1 10.5 0.8 12 7.9 9.4 11.9 2.4 4.7 5.6 16 3.0 5.9 6.2 8.3 10.2 13.1 20 10.4 3.5 7.2 6.7 10.7 14.4

In table 1 and 2 are made the fallow notations: 1^* - one non-working day, 2^* - two non-working days

For daily compensation at α =0.7 and \dot{t}_P = 5...10 hours/day correspond t_P = 3.5....7 hours/day. Four weekly compensation at α =0.7 and t_P = 8...20 hours/day and t'_P = 5...10 hours/day correspond t_T = 4.5...9.4 hours/day for one non-working day and t_T = 5.7...12.6 hours/day for two non-working days.

In paper [1] was obtain the utile volume for weekly and daily compensation ratio:

$$\frac{V_{us.r}}{V_{uz}} = \frac{r! (r \cdot t_P + t'_P)}{r \cdot t_P + r! \cdot t'_P}$$
(2)

where:

 V_{uz} - utile volume for daily compensation (m³);

 V_{usr} - utile volume for weekly compensation, with r non-working days (m³).

Replacing t_p form relation (1) in relation (2) obtain:

$$\frac{V_{us.r}}{V_{uz}} = \alpha \cdot (r'-1) \cdot \frac{r}{r'} \cdot \frac{t_P}{t_T} + 1$$
(3)

or using relations:

 $V_{uz} = 3600 \cdot Q_T; \quad t_T = 3600 \cdot Q_P; \quad t'_P = 3600 \cdot \alpha \cdot Q_T \cdot t'_P (4)$ relation (3) became :

$$V_{us,r} = \left[t_T + \alpha \cdot (r' - 1) \cdot \frac{r}{r'} \cdot t_P \right] \cdot 3600 \cdot Q_T$$
(5)

or:

$$V_{us,r} = \left[\frac{t_T}{\alpha} + (r'-1) \cdot \frac{r}{r'} \cdot t_P\right] \cdot 3600 \cdot Q_P \tag{6}$$

For r=1 and r'=6 in case of weekly compensation with one non-working day:

$$V_{us.1} = \left[t_T + \frac{5}{6} \cdot \alpha \cdot t_P \right] \cdot 3600 \cdot Q_T \tag{7}$$

or:

$$V_{us.1} = \left[\frac{t_T}{\alpha} + \frac{5}{6} \cdot t_P\right] \cdot 3600 \cdot Q_P \tag{8}$$

For r=2 and r'=5 in case of weekly compensation with two non-working days:

$$V_{us.2} = \left[t_T + \frac{8}{5} \cdot \alpha \cdot t_P \right] \cdot 3600 \cdot Q_T \tag{9}$$

or:

$$V_{us.2} = \left[\frac{t_T}{\alpha} + \frac{8}{5} \cdot t_P\right] \cdot 3600 \cdot Q_P \tag{10}$$

In table 3 related to $\alpha = (0.6...0.8)$, $t_T = (2...10 \text{ hours / day})$, $t_p = (10...18 \text{ hours / day})$ and $Q_T = (10...150 \text{ m}^3 \text{ / s})$ are presented values for utile volume V_{us1} and V_{us2} (millions m³).

											I able	э.							
								τ	Jtile v	olume	e in m	illions	s m ³						
t (turb.) 2				6					10										
t (pomp.)		10			14		18		10		14		18		10		14		8
alpha	debit	1*	2*	1*	2*	1*	2*	1*	2*	1*	2*	1*	2*	1*	2*	1*	2*	1*	2*
0,6	10	0,3	0,4	0,3	0,6	0,4	0,7	0,4	0,6	0,5	0,7	0,5	0,8	0,5	0,7	0,6	0,8	0,7	1,0
	50	1,3	2,1	1,6	2,8	2,0	3,5	2,0	2,8	2,3	3,5	2,7	4,2	2,7	3,5	3,1	4,2	3,4	4,9
	100	2,5	4,2	3,2	5,6	4,0	6,9	4,0	5,6	4,7	7,0	5,4	8,4	5,4	7,1	6,1	8,4	6,8	9,8
	150	3,8	6,3	4,9	8,3	5,9	10,4	5,9	8,4	7,0	10,5	8,1	12,6	8,1	10,6	9,2	12,7	10,3	14,7
0,7	10	0,3	0,5	0,4	0,6	0,5	0,8	0,4	0,5	0,5	0,6	0,6	0,8	0,6	0,8	0,7	0,9	0,7	1,1
	50	1,4	2,4	1,8	3,2	2,3	4,0	2,1	2,4	2,6	3,2	3,0	4,0	2,9	3,8	3,3	4,6	3,7	5,4
	100	2,8	4,8	3,7	6,4	4,5	8,0	4,3	4,8	5,1	6,4	5,9	8,0	5,7	7,6	6,5	9,2	7,4	10,9
	150	4,2	7,1	5,5	9,5	6,8	12,0	6,4	7,1	7,7	9,5	8,9	12,0	8,6	11,4	9,8	13,9	11,1	16,3
0,8	10	0,3	0,5	0,4	0,7	0,5	0,9	0,5	0,7	0,6	0,9	0,6	1,0	0,6	0,8	0,7	1,0	0,8	1,2
	50	1,6	2,7	2,0	3,6	2,5	4,5	2,3	3,4	2,8	4,3	3,2	5,2	3,0	4,1	3,5	5,0	4,0	5,9
	100	3,1	5,3	4,1	7,2	5,0	9,0	4,6	6,8	5,5	8,6	6,5	10,5	6,0	8,2	7,0	10,1	7,9	11,9
	150	4,7	8,0	6,1	10,8	7,6	13,5	6,8	10,2	8,3	12,9	9,7	15,7	9,0	12,3	10,4	15,1	11,9	17,8

Volumes $V_{us.1}$ and $V_{us.2}$ depending on reversible coefficients increasing α (for domain 0.6...1.0), for $Q_T = 100 \text{ m}^3/s$ at $t_p = 14$ hour / day and $t_T = 10$ hours / day increase from 6.1 mil. m³, respectively 8.4 mil. m³, at 7.8 respectively 11.7 mil. m³, approximate 1.3...1.4 times. For $\alpha = 0.7$, $t_p = 14$ hour / day , $Q_T = 100 \text{ m}^3/s$,

 $V_{us.1}$ increase from 3.7 mil. m³ to 6.5 mil. m³, and

 $V_{us.2}$ from 6.4 mil. m³ to 9.3 mil. m³, an t_T increasing from 2 to 10 hours/day. At r = 0:

$$V_{us,0} = V_{uz} = 3600 \cdot t'_p \cdot \alpha \cdot Q_T = 3600 \cdot t_T \cdot Q_T \tag{11}$$

In table 4 are presented values of these volumes depending on α , Q_T and t'_P .

For same t'_P, V_{uz} varying depending on α , double themselves for an increase of α from 0.6 to 1.0. For Q_T

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= 100 m³/s and α = 0.7 t'_P variation from 2 to 10 hours/day, V_{uz} grow proportional to t'_P from 0.5 mil. m³ to 2.5 mil. m³ (5 times). From relation (4), (7) and (9) replacing Q value

equal to:

$$Q_T = \frac{P_T}{9.81 \cdot \eta_T \cdot \eta_G \cdot \eta_{hT} \cdot H}$$
(12)

obtain:

$$V_{uz} = t_T \frac{3600}{9.81 \cdot \eta_T \cdot \eta_G \cdot \eta_{hT}} \frac{P_T}{H} = 425 \cdot t_T \cdot \frac{P_T}{H} \qquad (m^3)(13)$$

or:

$$V_{uz} = 0.425 \cdot t_T \cdot \frac{P_T}{H} \qquad \left(mil. \ m^3 \right) \tag{14}$$

and:

$$V_{us.1} = \left[t_T + \frac{5}{6} \cdot \alpha \cdot t_P\right] \cdot 0.425 \cdot \frac{P_T}{H} = K_{us.1} \cdot \frac{P_T}{H} \qquad (mil. m^3)$$
(15)
$$V_{us.2} = \left[t_T + \frac{8}{5} \cdot \alpha \cdot t_P\right] \cdot 0.425 \cdot \frac{P_T}{H} = K_{us.2} \cdot \frac{P_T}{H} \qquad (mil. m^3)$$
(16)

were:
$$K_{us.1} = 0.425 \cdot \left[1 + \frac{5}{6} \cdot \alpha \cdot \frac{t_P}{t_T}\right] \cdot t_T$$
 (17)
and: $K_{us.2} = 0.425 \cdot \left[1 + \frac{8}{5} \cdot \alpha \cdot \frac{t_P}{t_T}\right] \cdot t_T$ (18)

in formula (12) we have:

 Q_T - turbinate flow (m³/s);

- H medium head (m);
- η_T turbine efficiency;
- η_G generator efficiency ;

 η_{hT} - hydraulic efficiency of turbine derivation;

 P_T - installed power of turbine in kW in formula (12) and MW in the anther relations starting with (14).

For daily compensation at $t_T = 5$ hours / day and weekly compensation

with one non-working day (with $t_T = 5 \text{ hours } / \text{day}$ and $t_P = 14 \text{ hours } / \text{day}$) and with two nonworking days ($t_T = 7 \text{ hours } / \text{day}$ and $t_P = 14 \text{ hours } / \text{day}$) it is obtain:

$$V_{uz} = 2.125 \cdot \frac{P_T}{H} \qquad \left(mil. \ m^3 \right) \tag{19}$$

$$V_{us.1} = 5.60 \cdot \frac{P_T}{H} \qquad \left(mil. \ m^3 \right) \tag{20}$$

$$V_{us,2} = 9.64 \cdot \frac{P_T}{H} \qquad (mil. \ m^3) \tag{21}$$

From relations (19-21) result: $\frac{V_{us.1}}{V_{uz}} = 2.7$, $\frac{V_{us.2}}{V_{uz}} = 4.6$,

$$\frac{V_{us.2}}{V_{us.1}} = 1.7$$
.

For an installed power $P_T=500$ MW depending on H(m), it is obtain the follow volume (mil. m³) from table 4.

Table 4.							
H(m)	100	300	500	700	900		
V_{uz}	10.6	3.5	2.1	1.5	1.2		
V _{us.1}	28.0	9.3	5.6	4.0	3.1		
V _{us.2}	48.2	16.1	9.6	6.7	5.4		

On the base of this data result utile volume variation diagrams depending on head, for the storage of pumped storage whit Pi=500 MW which are presented in figure 1. These diagrams could serve for approximate estimation of theoretical utile volumes in storage lakes, in the phase of preliminary study.

Formulas for determination of $\frac{V_{us,1}}{V_{uz}}$, $\frac{V_{us,2}}{V_{uz}}$,

 $\frac{V_{us.2}}{V_{us.1}}$ could be obtain from relations (4),(7), (9),

depending on α , t_P and t_T :

0

$$\frac{V_{us,1}}{V_{uz}} = \left[1 + \frac{5}{6} \cdot \alpha \cdot \frac{t_P}{t_T}\right]$$
(22)

$$\frac{V_{us.2}}{V_{uz}} = \left[1 + \frac{8}{5} \cdot \alpha \cdot \frac{t_P}{t_T}\right]$$
(23)

$$\frac{V_{us,2}}{V_{us,1}} = \frac{t_T + \frac{\delta}{5} \cdot \alpha \cdot t_P}{t_T + \frac{5}{6} \cdot \alpha \cdot t_P}$$
(24)



Fig.1. Utile volume variation diagrams depending on head, for the storage of pumped storage whit Pi=500 MW

If we consider relation (11) and (12) we obtain formulas for these ratio, depending on α , t_p t_p and t_T :

$$\frac{V_{us.1}}{V_{uz}} = \left[\alpha \cdot \frac{t_P + t_P}{t_T} \right]$$

$$\frac{V_{us.2}}{V_{uz}} = \left[\alpha \cdot \frac{2 \cdot t_P + t_P}{t_T} \right]$$

$$\frac{V_{us.2}}{V_{us.1}} = \frac{2 \cdot t_P + t_P}{t_P + t_P}$$
(25)
(26)
(27)

In paper [2] are presented diagrams for these ratio depending on α , t_p and t_T . From these diagrams we observe that $V_{us\,2}$ is bigger then one non-working day $V_{us\,1}$ with maximum 80% for value of ratio $\frac{t_p}{t_T} = 5$. α 's value importance is small in this ratio. In both compensation cases result that, in order to obtain a smaller storage, is indicated that α to have smaller values, 0.7-0.8 $(\frac{1}{\alpha} = \frac{Q_T}{Q_p} = 1.25 - 1.40)$.

<u>3.2. Case B</u>, when in working days and in the night after one non-working day it functions with a part (n) from pump power.

In this case from condition that pumped volume during one week to be equal with turbinate volume, result:

$$(r \cdot t_P + t_P') \cdot Q_p + (r' - 1) \cdot t_P' \frac{Q_P}{n} = r' \cdot t_T \cdot Q_T$$
 (28)

From this relation result:

$$t_T = \alpha \cdot \left(\frac{r}{r'} \cdot t_P + \frac{n+r'-1}{n \cdot r'} t_P \right)$$
(29)

or:

$$t'_{P} = n \frac{r' \cdot t_{T} - \alpha \cdot r \cdot t_{P}}{\alpha (n + r' - 1)}$$
(30)

In formula
$$\frac{V_{us,r}}{V_{uz}} = \frac{r'(r \cdot t_P + t_P')}{r \cdot t_P + r' \cdot t_P'}$$
, replacing the

value of t'_p result:

$$\frac{V_{us.r}}{V_{uz}} = \frac{n \cdot r' \cdot r' t_T + \alpha \cdot r \cdot r' (r'-1) \cdot t_P}{n \cdot r' \cdot r \cdot t_T - \alpha \cdot r (n-1) (r'-1) \cdot t_P}$$
(31)

<u>For n=2</u>, power station function with half of pump power, relations (35-37) become:

$$t_T = \alpha \cdot \left(\frac{r}{r'} \cdot t_P + \frac{1+r'}{2 \cdot r'} t_P \right)$$
(32)

$$t'_{P} = 2 \left(\frac{r' t_{T}}{\alpha (1+r')} - \frac{r \cdot t_{P}}{(1+r')} \right)$$
(33)
$$\frac{V_{us,r}}{V} = \frac{2 \cdot r' r' t_{T} + \alpha \cdot r \cdot r' (r'-1) \cdot t_{P}}{2 \cdot r' r' t_{T} + \alpha \cdot r \cdot r' (r'-1) \cdot t_{P}}$$
(34)

 $\frac{V_{uz}}{V_{uz}} = \frac{1}{2 \cdot r' \cdot r' t_T - \alpha \cdot r \cdot (r'-1) \cdot t_P}$ (34) For n=1, power station function with full

pump power, relations (29-31) become equal with those resulted in cap. 3.1(1) and (3).

<u>3.3. Case C</u>, when in working days it function with a part (n) from pump power

In this case from condition that pump volume during one week to be equal with turbinate volume result:

$$(r \cdot t_{P}) \cdot Q_{p} + (r') \cdot t_{P} \frac{Q_{P}}{n} = r' \cdot t_{T} \cdot Q_{T}$$
(35)
From this relation $t_{T} = \alpha \cdot \left(\frac{r}{r'} \cdot t_{P} + \frac{t'_{P}}{n}\right)$ (36) or
 $t'_{P} = n \left(\frac{t_{T}}{\alpha} - \frac{r \cdot t_{P}}{r'}\right)$ (37). In formula $\frac{V_{us,r}}{V_{uz}}$ (relation
22) from paper [1] replacing value of t'_{p} result:

$$\frac{V_{us.r}}{V_{uz}} = \frac{n \cdot r' t_T + \alpha \cdot r \cdot (r' - n) \cdot t_P}{n \cdot r' \cdot t_T - \alpha \cdot r \cdot (1 - n) \cdot t_P}$$
(38)

<u>For n=2</u>, power station function with half pump power, relations (36 and 37) become:

$$t_T = \alpha \cdot \left(\frac{r}{r'} \cdot t_P + \frac{t'_P}{2}\right) \tag{39}$$

$$\dot{t_P} = 2 \cdot \left(\frac{t_T}{\alpha} - \frac{r \cdot t_P}{r'} \right)$$
(40)

Replacing in formula $\frac{V_{us,r}}{V_{uz}}$ value of t_p , it obtain:

$$\frac{V_{us.r}}{V_{uz}} = \frac{2 \cdot r' t_T + \alpha \cdot r \cdot (r'-2) \cdot t_P}{2 \cdot r' \cdot t_T - \alpha \cdot r \cdot t_P}$$
(41)

<u>For n=1</u>, power station function with full pump power, relations (36-38) become equal with those from cap. 3.1 (1) and (3).

4. Comparison among this three cases

Comparative values for this three cases A, B, C with $t_T = 5$ hours / day , $t_P = 14$ hours / day and $\alpha = 0.715$, H = 500 m, $V_{uz} = 2 mil. m^3$ and n=2 could be seen table 8:

Table 8.						
CAZUL	А	В	С			
r = 1, r' = 6						
t_T (hours/day)	5	5	5			
t_P (hours/day)	4.66	7.99	9.32			
$V_{us.1}/V_{uz}$	2.67	2.13	2.00			
V_{uz} (mil. m ³)	2.00	2.00	2.00			
$V_{us.1}$ (mil. m ³)	5.34	4.26	4.00			
r = 2, r' = 5						
t_T (hours/day)	7.0	6.0	6.0			
t_P (hours/day)	4.2	4.6	5.5			
$V_{us.2}/V_{uz}$	3.29	3.19	3.00			
V_{uz} (mil. m ³)	2.8	2.4	2.4			
$V_{us.1}$ (mil. m ³)	9.2	7.66	7.2			
$V_{us.2}/V_{us.1}$	1.72	1.80	1.80			

From this comparative dates from the table result:

- for weekly compensation, with one nonworking day, for same turbinate period $t_T = 5 \text{ hours}/\text{day}$, result different values for t_p' in this three cases, respective 4.66 hours/day (case A), 7.99 hours/day (case B) and 9.32 hours/day (case C). Util volume ratio (daily and weekly compensation) decreases from 2.67 (case A) to 2.00 (case C). Result that for cases B and C when from energetic system necessity, can function for pumping just half installed power, utile volume are smaller represented approximate .75% from utile volume needed in case A.

For weekly compensation, with two nonworking days utile volume needed are with approximately 70-80% bigger than one non-working day variant. In cases B and C volumes are smaller (approximately 0.83, respectively approximately 0.78 from case A). In this case t_T value increase from 5 hours/day to 6-7 hours/day and t'_p value remain constant 4-5.5 hours/day.

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Researches on the Additional Forces at the Lifting Wing Emission through the Run Board

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Abstract: The aerodynamics follows the behavior of the lifting wing in the actual conditions. This aspect takes into account the report between the lifting force and the advancement resistance force. This, it is followed the making of a lifting force, as big as possible, and of a minimum advancement resistance.

Keywords: coefficient increase, coefficient decrease, jet force, aerodynamic profile

1 Introduction

One of the present concerns in the field of the fluid mechanics is the lifting capacity increase of the aerodynamic wing. The lifting capacity, defined as the report between the lifting coefficient C_y and the resistance coefficient C_x to the advancement, can be increased by one of the following methods:

 $-C_y$ coefficient increase, by C_z coefficient constant maintenance;

 $-C_z$ coefficient decrease, by C_y coefficient constant maintenance;

The analysis of the characteristic curves $C_y = f(\alpha)$ and $C_x = f(\alpha)$ for each profile apart (α being the incidence angle), emphasizes the followings:

– the increase of the coefficient C_y determines also the increase of the coefficient C_x ;

– the decrease of the coefficient C_x determines also the decrease of the coefficient C_y .

In the work [1], [2], [3], [4] there have been analyzed, in a theoretical and experimental way, a method to increase the lifting capacity. The method consists of the launching of a fluid jet with controlled energy through a slit made in the run board of the wing. The analysis of the got data emphasized, a considerable increase of the lifting capacity, this: it appears an increase of the lifting coefficient simultaneously with the decrease of the advancement coefficient; this gets negative values, for a field of the incidence angles $\alpha \in [0 \div 14^{\circ}]$.

The negative values of this coefficient show the fact that; the resistance force becomes a force with propelling character.

2. Contents

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The aim of the work is the establishment, on an experimental way, of the dynamic components of the jet force developed by the emitted jet through the run board of the lifting wing on the lifting force and on the advancement resistance force.

It is noted:

 $P_{\text{oy}}-$ the lifting force for the wing, without fluid emission

 P_{ox} – the advancement resistance force for the wing, without fluid emission

 $P_{\rm y}$ – the lifting force for the wing, with fluid emission

 P_x – the advancement resistance force for the wing, with fluid emission

 F_y – the vertical component of the jet force F, developed by the jet, witch is perpendicular on the speed direction W of the general current.

 F_x – the horizontal component of the jet force F, developed by the jet and witch is collinear with the speed direction W, of the general current. See fig. 1.

In order to get the speeds V of the emitted jet through the run board, as big as possible, in comparison with the speed W of the general current, this speed has been limited to the value of: W = 2- m/s. The incidence angles α , which are taken into consideration, are: 0°, 2°; 4°; 6°; 8°; 10°; 12°; 14°; 16°; 18°; 20°; 22°; 24°; 26°.

The chosen aerodynamic profile, on whose basis the lifting wing is built, is of Gö 593 type. The experimental tests have been made in the aerodynamic tunnel, at the Mechanics of Fluids faculty; the tunnel has the working chamber section of 0.8 m^2 . The geometrical sizes of the tested aerodynamic wing, are: cord c = 0.1 m; span b=0,1m.

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The lifting wing, which is paced in the tunnel working chamber, is connected to the measure scales, by which it can be simultaneously measured the lifting force and the advancement resistance force. The fluid emission from the run board is generated by means of a compressed air steamroller. The control of the admitted airflow in the palette and evacuated through the run board, is made by, means of a rot meters battery. The slit from the run board has the height h = 0,001 m and the length L = b = 0,1 m. The medium speeds of the fluid jet, measured to the continuity equation Q = AV, have the values of: 0 m/s; 10 m/s; 20 m/s; 30 m/s; 40 m/s; 50 m/s; 60 m/s.

After the experimental tests, there have been determined the following sizes: P_{oy} ; P_{ox} ; P_y ; P_x , which sizes have been defined previously.

The analysis of these forces emphasizes the fact that the emitted jet through the slit from the run board, determines the appearance of some forces witch modify appreciably both the lifting force as well as the advancement resistant force. So, the jet force with a direct action on the lifting is:

$$Fy = Py - Poy$$
 (1)
and the jet force, which operates on the advancement
resistance, is:

$$Fx = Pox - Px \tag{2}$$

After the interaction between the general speed current and the jet emitted through the run board, the last one is deviated with angle β in front of the lifting wing card direction, where:



Diagram 1 a







Diagram 2 a



Diagram 2 b



Diagram 3 a



Diagram 3 b

3. Conclusions therefore:

The results obtained after the experimental determination are show in the following diagrams.

Diagram 1.a. shows the variation of the vertical force $F_v = F_v(\alpha)$ at the constant speed V, of the jet. The diagram analysis puts into evidence the following conclusions:

- indifferently of the medium speed V value, the correspondent F_v increases at the some time with the increase of the incidence angle α ;

- the correspondent F_v increases at the some time with the increase of the medium speed V.

The diagram 1.b, shows the variation of the vertical force $F_v = F_v(V)$, at a constant incidence angle a. The diagram analysis shows that, the component F_v increases together with the increase of the jet speed V.

The diagram 2.a. shows the variation of the horizontal component $F_x = F_x(\alpha)$ at a constant speed. The diagram analysis shows that, this component of the jet force decreases together with the increase of the incidence angle.

The big values are got for small incidence angles.

The diagram 2.b. shows the variation of the component $F_x = F_x(V)$ for $\alpha = \text{constant}$. The diagram analysis puts into evidence the fact that, the value of this force increases together with the increase of the medium speed V of the jet emitted through the run board.

The diagram 3.a. shows the variation of the deviation angle for the jet β , in front of the lifting wing card direction. The diagram analysis shows that, at the speed V = constant, the β angle increases together with the α incidence increase.

The diagram 3.b. shows the variation of the derivation angle $\beta = \beta(V)$ for $\alpha = \text{constant}$. It is found out, from the diagram analysis, the decrease of the angle β together with the increase of the jet medium speed V.

Regarding the deviation angle β and its connection with the vertical component F_v : it is established, from fig. 1, that:

$$Fy = F \sin (\beta + \alpha) (4)$$

$$\beta + \alpha = \arcsin \frac{Fy}{F} \Rightarrow \beta = \arcsin \frac{Fy}{F} - \alpha$$
 (5)

If the angle $\beta > 180^{\circ}$, it results sin $\beta < 0$, so $F_y < 0$; therefore, the lifting force P_y decreases. This aspect has been pointed out in the work [5], but without to be explained from the theoretical point of view.

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Referring to the Limits of Incident Current Energy Utilization Factor on the Horizontal Shaft wind Turbines

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Abstract: Beginning with the flowing modeling by means of the moving components of the horizontal shaft wind turbines applying the turbulent flow theoretical concept and considering the papers published by BETZ and SABININ, there are new theoretical concepts presented regarding the limits of incident current energy utilization factor on the horizontal shaft wind turbines, by considering the effect of upstream and downstream flow velocities induced by the wind turbine rotor.

Rezumat: Pornind de la modelarea curgerii prin elementele active ale turbinelor eoliene cu arbore orizontal utilizând teoria curgerii turbionare și luând în considerație lucrările publicate de BETZ și SABININ, se prezintă noi considerente teoretice privind limitele coeficientului de utilizare a energiei curentului incident asupra turbinei cu arbore orizontal, prin luarea în considerare a efectului vitezelor induse în aval și amonte, de rotor.

Keywords: wind turbine rotor, turbulent flow, current energy utilization factor.

General

Following by means of the moving components of the wind turbines have been approached in the specialized technical literature since the latter half of the past century, in studies conducted by RANKINE [1] AND FROUDE [2], subsequently resumed by BETZ [3] and SABINI [4] during the period in witch there was laid the basis of the aerodynamic study of general flowing in horizontal shaft wind turbines.

The present paper, centered around the theories developed by BETZ [3] and SABINI [4], deals with the theoretic aspects of the study of flowing in the horizontal shaft wind rotor. Pointing out the continuity between these theories, the paper further presents some aspects related to the development of the turbulent flow theory as proposed by SABINI [4].

Possible solutions of the flowing equations are presented, by considering the effect of the rotor – induced upstream and downstream flow velocities, as well as the effect of the variation of

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the rotor – induced downstream velocities according to the horizontal shaft turbine rotor operating regime, in order to determine the correlation of the maximum efficiency regime versus the variation of the velocities induced behind the wind turbine.

1. Elementary theory of the horizontal shaft wind turbines

The aerodynamic model proposed by BETZ [3] in order to study wind energy catching by means of propeller – type devices, considers the fact domain, made up by the moving air current.

This aerodynamic model considers the total air pressure discontinuity accruing in the upstream – and downstream sections in the immediate vicinity of the rotor.

To this end in view, the following hypotheses are admitted: (1) flowing and the fluid are Newtonian; (2) upstream – and downstream

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flowing are non-rotational and the induced velocities are not considered; (3) the

upstream and downstream flowing far from the rotor is uniform along the current tube, and exclusively axial; (4) there are no pressure discontinuities registered between the rotor and the far-off downstream; (5) the pressure discontinuity occurs only the immediate sections upstream and downstream the rotor.

By using the concept of air current utilization factor, also called wind turbine power factor, marked C_p – then the initial power may be write as:



Fig. 1.1 – Aerodynamically model proposed by BETZ

Thus, using relationship (1.1) for power factor C_p the following relation can be derived:

Where
$$n = \frac{V_n}{V_0}$$

while
$$V_n = \frac{1}{2}(V_{11} + V_{12})$$
 (1.2)

If $V_0 = V_{II}$, it is found that power factor C_p becomes dimensionless and depends on the air current braking factor marked

$$a = I - \frac{V_m}{V 0}$$
 also called Betz braking factor.

In order to determine the values of braking factor by means of which the maximum values for dimensionless power factor C_p is obtained, the first

differential of C_p expression, as given by relation (1.3), shall be cancelled, obtaining thus:

$$\frac{dC}{da} = 4(1 - 4a + 3a^2) = 0$$
(1.3)

The condition $\frac{aC}{da} = 0$ leads to the

equation:

$$3a^2 - 4a + 1 = 0 \tag{1.4}$$

The rotor operation in turbine regime corresponds to the solution to equation $a = \frac{1}{2}$ when

$$V_2 = (1 - 2a), V_0 = (1 - \frac{2}{3}), V_0 = 0.33$$

Which means that air velocity after crossing the catching system still maintains about 33% of the initial velocity V_0 .

By solving out equation (1.4), there results a maximum values of the dimensionless power factor of $C_p = 0.593$.

There results, therefore, that an ideal rotor with an infinite number of blades placed in uniform current can catch at the most 60% of the incident air current energy.

The elementary theory of the wind catching system points out the following aspects: the power of catching from an air current of incident velocity V_0 is directly proportional to the rotor incidence area A, the cubic initial velocity V_0 downstream far off the section where the catching system is located, the power factor C_p and the air braking capacity of the incident air current energy also called *air current braking factor*

$$a = I - \frac{V_m}{V0}$$
 defined by Betz [4].

Power factor C_p reaches its maximum values when the braking factor is $a = \frac{1}{3}$.

A more through analysis of the wind energy catching mechanism is to be found in the following chapter by using the turbulent flow theoretical concept and applying the impulse theory to the aerodynamic study of the horizontal shaft wind turbine.

2. The impulse theory as applied to the horizontal shaft wind turbine dimensioning

The application of the impulse theory to the horizontal shaft wind turbine dimensioning has been taken over from the airplane propeller elastic theory

which, together with the classical whirlwind theory has led to highly efficient aerodynamic solutions.

One of the anterior of this calculation and dimensioning method was G. K. Sabinin [4] who had his studies in this field published starting from 1923.

2.1. Hypotheses

The application of the impulse theory to the calculation and dimensioning of the propeller – type horizontal shaft turbines is based on the following main hypotheses:

• the air jet crosses the rotor at an even velocity throughout the axial cross section;

• the rotor lets the air pass through the blades without determining a local velocity discontinuity and has an infinite number of blades;

• the presence of the rotor brings about a pressure variation between upstream and downstream, in a fluid domain delimited downstream the catching system by a cylindrical surface on which an infinite number of whose winding is the cylindrical surface corresponding to section A-A, Fig 2.1, downstream, in its immediate vicinity;

• the current tube delimited by the solenoid surface does not allow for the air does not allow for the air exchange between its inside and outside, the air current passing through the rotor being considered isolated from the ambient;

• the current non-uniformity increases at the rotor outlet, which leads to turbulent energy losses and therefore to a lower efficiency of catching the wind energy, while the current twisting dissipated in alternating whirlpools caused by the instability of the flow downstream the rotor;

• air pressure in the 0-0 cross-section in Fig.2.1 is assumed to be equal to the atmospheric one;

• in cross-section 1-1, pressure rises to values $p_1 < p_0$, yet further increasing while aiming asymptotically towards values p_0 , far-off downstream;

• in the sections downstream the wind turbine rotor, pressure variation is neglected, considering that the centrifugal forces determined by the remanencing flow speed twisting after rotor crossing are small as compared to the forces caused by the axial impulse in the same section;

• the pressure difference downstream and upstream the catching system leads to the occurrence of an axial force upon the rotor "Fa"; as a result of the adequately built rotor geometry there appears a tangential component "Fr" in the rotation plan, leading to the occurrence of the catching system useful moment. Obviously, wind catching systems should be build so that the rotor impulse tangential component should be as big as possible.

Figure 2.1 schematically shows the air current shape upstream and downstream the rotor according to the above hypotheses, as well as the diagram of the modality the whirlwind solenoid surface is formed, delimiting the current tube downstream the rotor; upstream the rotor, namely in the 0-0 section in Fig. 2.1, current velocity V_0 is equal to the far-off upstream velocity (infinite upstream); in the catching system rotor section 1-1, and as getting nearer and nearer to this section, the current axial velocity drops to values $V_{11} = V_0 - V_2$, where V_1 and V_2 are velocities induced by the cylindrical turbulent layer generated by the whirlwinds on top of the rotor blades forming up a current tube.

As in the wind turbines rotor there appears a rotating moment in the section 1-1, that leads to the occurrence of the induced rotating impulse, counterclockwise to the catching system rotation, as a result of the whirlwinds induced by the blades having as winding the current tube cylindrical surface as per the diagram in Fig. 2.1. There results therefore that downstream the rotor, the air current rotates at a velocity equal to the velocity V_2 in a close enough section, where the whirlwind tubes do not vicinity, the current peripheral velocity is considered not very much different from that, so that $V_2 \sim V_{2\infty}$, where V_2 is the rotating velocity at the rotor outlet.



Fig 2.1 The diagram of the formation of whirlwind solenoid surface downstream the wind turbine

2.2. Determining of the peripheral force component and of the axial force component acting upon the catching system blade

Isolating an annular area of radius, dr thick, off the air current, the axial component dFa as well as the rotation peripheral component dFr (Fig. 2.3) can be determined in the corresponding annular section in the rotor plan, Fig. 2.1. These components lead to the occurrence of the interactions in the rotor construction elements (blades).

By applying the impulse theorem and taking into account that, in keeping with second principle of mechanics, action is equal to reaction, there can be determined the reactions occurring in the rotor blades, along the axial and tangential directions, respectively, where: dFa – the elementary axial force, and dFr – the elementary tangential force.

There results the relation:

$$dm = 2\pi \cdot r \cdot p \cdot Vl \cdot dr \tag{2.1.}$$

Where:

dm – is the unitary moss air flow crossing the annular section in the rotor, Fig. 2.1

 $V_1 = V_0 - V_1$; V_1 is the induced velocity before the rotor, caused by the blades.



Fig. 2.2 The triangles of the velocities and forces acting upon the horizontal shaft rotor blade

By applying to the air mass delimited by the two concentric cylindrical areas of radius r and r+dr, as shown in Fig. 2.1, the Euler theorem for the motional quantity by taking into the expression of the elementary force caused by the action of the elementary air mass enclosed between the two cylindrical areas, upon the blades.

$$dFa = V_2 \, dm \tag{2.2}$$

Where:

 $dm = 2\pi \cdot r \cdot p \cdot Vl \cdot dr$, as peer relation (2.1)

In a similar way, in keeping with theorem of the motional quantity moment, the elementary torque created by the tangential rotational force occurring on the elementary surface of the blades enclosed between the two cylindrical areas of radius r and r+dr, there results the relation:

$$dFr = u2dm \tag{2.3}$$

 V_{l} , u_{l} , w_{l} , as well the corresponding velocities in a section downstream the rotor are written with the following relations:

 $V_2 = \sigma V_l$

$$u_2 = \tau \ u_1 \tag{2.4}$$

$$w_2 = kw_1 = \sqrt{v_2^2 + u_2^2} \tag{2.5}$$

Where w_1 and w_2 are relatives velocities;

By replacing in relations (2.2) and (2.3) the value dm in the relation (2.10) there results:

$$dFa = 2\pi \zeta \iota v_1 u_1 r \cdot dr \tag{2.6}$$

$$dFa = 2\pi \iota \varsigma v_1 u_1 r \cdot dr \tag{2.7}$$

There have been obtained thus expressions for the axial elementary thrust force dFa and the tangential elementary force dFr which, as resulted from Fig. 2.2 are the components of the elementary aerodynamically force dFy along the two reference directions, namely the direction of the rotor-incident wind velocity and the tangential direction in the rotor section.

On the other hand, the elementary bearing force dFa for the infinite aerodynamic profile as isolated has the following expression:

$$DFy = C_y \frac{\varsigma w_l^2}{2} b \cdot dr$$
(2.8)

where :

Cy – the aerodynamic power factor for infinite profile,

b – profile chord ;

 w_1 – relative wind current velocity incident on the profile (Fig. 2.2)

In its turn, the aerodynamic elementary force on profile Fp is decomposed along the perpendicular direction of relative velocity w_1 having component Fr, and along the direction of head resistance force aligned with relative velocity w_1 so that for the blade element of length dr there results:

$$dFp = C_x \frac{\varsigma W_1^2}{2} b \cdot dr \tag{2.9}$$

$$dFr = C_x \frac{\mathcal{G}W_1^2}{2} b \cdot dr \tag{2.10}$$

where: Cz and Cx are bearing and head resistance factors, respectively, of the aerodynamic profile.

Based on relations (2.9) and (2.10), as well as on the notations in Fig. 2.2 by also noting the profile aerodynamic characteristic by $\mu = \frac{C_X}{C_Z}$

the resulting relation is:

$$dFa = \frac{\varsigma w_l^2}{2} i b C_Z(\cos\beta + \mu \sin\beta) dr \qquad (2.11)$$

$$dFr = \frac{\varsigma w_l^2}{2} i b C_Z \ (\sin\beta + \mu \cos\beta) \ dr \qquad (2.12)$$

where in keeping with the notations in Fig. 2.2 there have been noted as follows : i – number of rotor blades

$$\sin\beta = \frac{V_0 - V_1}{W_1} \tag{2.13}$$

$$\cos\beta = \frac{\omega r + V_1}{W_1} \tag{2.14}$$

$$tg\beta = \frac{V_0 - V_1}{\omega R + U_1} \tag{2.15}$$

Based on relations (2.6); (2.7); (2.11) and (2.12) there results :

$$2\pi\rho \quad V_{I}v_{I}r \cdot dr = \frac{\varsigma W_{I}^{2}}{2}ib(\cos\beta + \sin\beta)dr$$
(2.16)

$$2\pi\rho V_1 v_1 r \cdot dr = \frac{\rho W_1^2}{2} ibC_z (\sin\beta + \mu\cos\beta) dr$$
(2.17)

Considering the relations (2.4) and (2.5) by summing up the squares of the relations (2.16) and (2.17) their results:

$$\left(\frac{v_1 k w_1}{w_1}\right)^2 = \left(\frac{i b C_z}{4 \pi r}\right)^2 (1 + \mu^2)$$
(2.18)

$$C_Y = C_Z \sqrt{1 + \mu^2}$$

so:

$$\frac{v_1 k w_1}{w_1} = \frac{ib}{4\pi r} C_y$$
(2.19)

The relation (2.19) express the interdependence between the rotor geometrical parameters and the air current parameters in the very immediate vicinity upstream the rotor.

By replacing $\sin\beta$ and $\cos\beta$ in the relations (2.11) and (2.12) with the definition relations as per Fig. 2.3

$$sin\beta = \frac{v_0 - v_1}{w_1}$$
 and $cos\beta = \frac{\omega r + u_1}{w_1}$ (2.20)

and using relations (2.4); (2.5); (2.6) si (2.7), there results:

$$\frac{v^2}{u^2} = \frac{(v_0 - v_1)\mu + (\omega r + u_1)}{(v_0 - v_1) - \mu(\omega r + u_1)}$$
(2.21)

The relation (2.11) represents the ratio of the axial induced velocity to the peripheral velocity component us. This ratio is variable along the blade from hub to apex.

The relations (2.19) and (2.21) allow for the correlation of the aerodynamic and construction parameters of the horizontal shaft wind turbine rotor, and may be considered basic relations to the horizontal shaft catching systems theory.

To analytically determine the aerodynamic factors aerodynamics k, ζ and σ in the relations (2.4) and (2.5) it is further proceeded to solve out the problem of the operation of the real rotor with a finite number of blades in an ideal current.

Out of the equilibrium condition of the forces acting upon the blade elementary section, as shown in Fig. 2.2 and Fig. 2.3 by using section o-o and 1-1 and writing down the energy equation, there results:

$$\frac{1}{2}V_0 \cdot dm = V_1 dFa - u_1 dFr \frac{1}{2} \left(v_2^2 + u_2^2 \right) dm \quad (2.22)$$

Considering that $v_1 = v_0 - v_1$ and $v_2 = v_0 - v_2$ as well as relations (2.1);(2.2) and (2.3) for *dFr* there results :

$$v_2(v_1 - \frac{v_2}{2}) = u_2(\frac{u_2}{2} - u_1)$$
 (2.23)

Relation (2.23) is fulfilled provided what is in between the brackets is equal to zero. Out of this condition their result:

$$v_2 = \frac{v_2}{2}$$
 and $u_1 = \frac{u_2}{2}$ (2.24)

Relation (2.26) represents a particular solution resulted from the condition of canceling relation (2.13).

The general case corresponds to the condition when relation (2.13) parameters are equal to one another, yet different from zero, which is expressed by the relations:

$$V_2(V_1 - \frac{V_2}{2}) \neq 0$$
 (2.25)

$$U_2(\frac{U_2}{2} - U_1) \neq 0 \tag{2.26}$$

That is conduce to two further conditions:

$$v_2 \neq 0 \to v_1 - \frac{v_2}{2} \neq 0$$
 (2.27)

$$u_2 \neq 0 \to \frac{u_2}{2} - u_1 \neq 0$$
 (2.28)

Relation 2.25 shall be written under the form:

$$v_2^2 - 2v_1v_2 + u_2^2 - 2u_1u_{2=0}$$
(2.29)

In the case of induced velocities as well, there can be admitted a module velocity λ_{i1} defined as the ratio between the induced peripheral velocity u_1 and the axial velocity v_i as follows:

$$\lambda_{i1} = \frac{u_1}{v_1} \text{ si } \lambda_{i2} = \frac{u_2}{v_{21}}$$
 (2.30)

By using the definition of the induced velocity after the rotor, relation 2.23 can be written:

$$\frac{dv_2}{d\lambda_{i2}} = -2\frac{u_1\lambda_{i2}^2 + 2v_1\lambda_{i2} - u_1}{\left(1 + \lambda_{i2}\right)^2}$$
(2.33)

Canceling of differential
$$\frac{dv_2}{d\lambda_{i2}}$$
 leads to:

$$u_1 \lambda_{i2}^2 + 2v_1 \lambda_{i2} - u_1 = 0 \tag{2.34}$$

By solving out the equation in relation with λ_{i2} their results:

$$\lambda_{i2} = \frac{v_1 \pm \sqrt{v_1^2 + u_1^2}}{u_1^2}$$
(2.35)

Their result, therefore, based on relation. (2.32) the expressions for $v_{2 \text{ max}}$ and $v_{2 \text{ min}}$

$$v_{i2\max} = v_1 + \sqrt{v_1^2 + u_1^2}$$
 (2.36)

Considering $\lambda_{i1} = \frac{u_1}{v_1}$ there results:

$$v_{i2 \max} = v_1 (1 + \sqrt{1 + \lambda_{i1}^2})$$
 (2.37)

there results accordingly :

$$v_{i2 \max} = v_1 (1 - \sqrt{1 + \lambda_{i1}^2})$$
 (2.38)

$$2(v_1 + u_1\lambda_{i2})\frac{1}{v_2} = 1 + \lambda_{i2}^2$$
(2.31)

From which the following expression for v_2 may be inferred:

$$v_2 = \frac{2(v_1 + u_1\lambda_{i2})}{1 + \lambda_{i2}^2}$$
(2.32)

Relation (2.32) expresses the variation of the axial flow velocity induced downstream the rotor versus the induced flow velocities upstream the rotor ω_1 and VI as well as the module of the flow velocities induced downstream the rotor.

By canceling the first order differential of relation. (2.32) in relation with λ_{i2} the extreme values of the axial flow velocity induced downstream the rotor, V2 can be determined as follows:

Figure (2.3) and (2.4) show velocity variation by varying v_2u_2 versus the induced velocity module.

Out of the analysis Fig.2.4 and Fig.2.5 there results that assuming both $v_1 < \text{and } v_1 > u_1$ there is a maximum of the induced velocities $v_2 > 2v_1$

We specify that the solution given by Sabinin (4) namely $v_2 = 2v_1$ is valid in the case of a module velocity $\lambda_{i2} > 1$

The axial induced velocity v_2 is coming down asymptotically at the same time with the increase of the wind catching system – induced downstream flow velocity module λ_{i2} aiming to values $v_2 = 0$

By explaining the rotor – induced downstream flow axial velocity u_2 in relation (2.29), there results :

$$v_2 = \frac{2(u_1\lambda_{i2} + v_1)\lambda_{i2}}{\lambda_{i2} + 1}$$
(2.39)

The function defined by relation (2.39) is studied graphically in figures 2.6 and 2.7, choosing induced velocity u_2 as ordinate and abscissa λ_{i2} . The intersection with the abscissa corresponds to the condition at the limit $u_2 = 0$ so that, out of relation (2.39) there results :

$$2(u_1\lambda_{i2} + v_1)\lambda_{i2} = 0 (2.40)$$

Equation 2.40 admits of two solutions:

$$\lambda'_{i2} = -\frac{\nu_1}{u_1} = -\lambda_{i1} \tag{2.41}$$

This solution corresponds to the case in which the flow velocities induced downstream the rotor are equal, yet opposed to the upstream induced ones.

The second solution :

$$\lambda_{i2}^{"} = 0$$
 and therefore $u_2 = 0$ (2.42)

corresponds to the case in which the induced velocities after the rotor are null, and therefore to the rotor braked condition.

Out of the graphical representation of relation (2.39) there results that induced velocity u_2 aims asymptotically towards a minimum values $u_2 = 2u_1$ a solution given gy Sabinin[4]. To be noticed that in the range $\lambda_{i2} > 0$ there is a maximum for the peripheral induced velocity $u_2 = u_1 + \sqrt{u_1^2 + v_1^2}$.

Out of the calculation of the asymptotic values their results, considering the limit of the function given by relation. (2.39) for $\lambda_{i2} \rightarrow \infty$:

$$\lim_{\lambda_{12\to\infty}} u_2 = \lim_{\lambda_{12\to\infty}} \frac{2(u_1\lambda_{i2} + v_1)\lambda_{i2}}{\lambda_{i2}^2 + I} =$$
$$= \lim_{\substack{l \\ \frac{1}{\lambda_{i2}\to 0}}} \frac{2u_1 + 2v_1 \cdot \frac{l}{\lambda_{i2}}}{1 + \frac{l}{\lambda_{i2}^2}} = 2u_1$$
(2.43)

Therefore, there results that the minimum value towards which velocities induced after the rotor aim when $\lambda_{i2} \rightarrow \infty$ is $2u_1$ that is the double of the flow velocities induced upstream the wind turbine rotor.

Value λ_{i2a} is a further determined, with $u_2 = 2u_1$. Into relation (2.39) there results:

$$u_2 = \frac{u_2 \lambda_{i2a} + 2v_1 \lambda_{ia2}}{\lambda_{i2a}}$$
(2.44)

getting to the solution :

$$\lambda_{i2a} = \frac{u_1}{v_1} = \lambda_{i1} \tag{2.45}$$

There results therefore that the minimum values of the velocities induced after the rotor u_2 are obtained for that case in which the module of the flow velocities induced upstream the rotor.

In order to determine the extremes of the variation function of the flow velocity v_2 induced after the rotor, as defined by relation (2.39) the first differential should be cancelled. There results in this case:

$$\frac{2\left(v_1\lambda_{i2}^2 + 2u_1\lambda_{i2} - v_1\right)}{\left(\lambda_{i2}^2 + 1\right)^2} = 0$$
(2.46)

which corresponds to the condition :

$$v_1 \lambda_{i2}^2 - 2u_1 \lambda_{i2} - v_1 = 0 \tag{2.47}$$

from which there result two solutions for relation (2.47) namely:

$$\lambda_{i2}^{x} = \frac{u_{1} - \sqrt{v_{1}^{2} + u_{1}^{2}}}{v_{1}}$$
(2.48)

$$\lambda_{i2}^{xx} = \frac{u_1^{'} - \sqrt{v_1^2 + u_1^2}}{v_1}$$
(2.49)

There results therefore, that induced velocity u_2 has a maximum value:

$$u_{2 max} = v_2 \frac{u_1 + \sqrt{v_1^2 + u_1^2}}{v_1} = v_2 \left(\lambda_{i1} + \sqrt{1 + \lambda_{i1}^2}\right)$$
(2.50)

and a minimum value:

$$u_{2\min} = v_2 \frac{u_1 - \sqrt{v_1^2 + u_1^2}}{v_1} = v_2 \left(\lambda_{i1} - \sqrt{1 + \lambda_{i1}^2}\right)$$
(2.51)

It is considered that the dependency relation of the induced velocity given by relation (2.39) verifies the condition at the limit.

For $u_1 = 0 \Rightarrow u_2 = 0$, the curve in Fig. 2.6 and Fig. 2.7 passes through the origin. For values of the module velocity induced after the rotor, in the range $0 < \lambda_{i2} < \lambda_{i1}$ the induced rotational velocity after the rotor $u_2 < u_1$. Thus, the solution given by Sabinin[4] is corrected through relation (2.26.)



Fig. 2.3. Variation of peripheral induced velocity u_2 versus the module of induces velocities λ_{i2}

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Fig. 2.4 The variation of peripheral induced velocity u_2 versus the module of induced velocities λ_{i2}

3. Conclusions

The developed theory regarding the flowing in the horizontal shaft wind rotor leads to values that are considerably higher than the maximum utilization efficiency because of the factor Cpmax = 0.593 determined by means of the elementary theory of the mechanical developed by Sabinin. This specify limit can have a much higher value, that is Cpmax=0.66.

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Energetical Criteria in the Analysis of The Requests for Upgrading the Water Supplying

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Abstract: The correct and efficacy solution of the water distribution systems development problem in to the populate areas, in the increasing of power tariffs and the limitations of the resources conditions, cannot be achieved without a systemic approach. This work intends to outline the principles and methods to be used for analyzing the shortcomings of water supplying networks and then deciding the upgrading measures to be taken.

Keywords: Water supplying, upgrading, analysis, energetical criteria.

1. Introduction

Implementing new demanded functions (in terms of water needs) in populated areas implies an adequate resource consumption, chiefly power consumption. Power is necessary to vehiculate great water volumes, which materialize the goal of redistribution in time and space of existing resources, according to the populace's needs (individual use, industry, public services).

Taking in account the increasing of power tariffs and the limitations of the resources, a correct solving of the issue of developing water distribution networks in populated areas cannot be achieved without a systemic approach. This approach has to start from the long-term monitorised strategically objectives which treat the water supply as an infrastructural basic system which operation (which is conditioning the evolution of the whole social and spatial frame in which it is fitting) has to be materialized through a minimal/rational resource consumption.

Considering this and the current technical textbooks this work intends to outline the principles and methods to be used for analyzing the short-comings of water supplying networks and then deciding the upgrading measures to be taken.

2. General considerations

Developed during the years according to the populaces' social and economic needs and according to the existing water resources, the water networks are complex systems which, apart the mains, are including several storage tanks, main pumping/repumping stations and auxiliary pumping stations.

Depending on their phased evolution, the hydrotechical diagrams have, over several districts, several types of ring shaped networks. These networks are usually organized on height, depending on relief's profile and are also interconnected the way to vehiculate flows towards an area or another, gravitary or by pumping. Concerning the housings, the water supply towards tower blocks of flats is made in a centralized manner and through auxiliary pumps (installed, usually, inside the heat distribu-tion stations).

Priorities, in the different phases of construction of the existing water supply networks, lead to their mainly extensive development, which was exclusively trying to attempt the goal to satisfy the water needs, in terms of volumes. This policy was neglecting the quality management for the supplied services and, nevertheless, the monitoring of the economical efficiency of these services.

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Referring to this aspect we must remark that in the existing water supply systems there is an acute lack of user metering. Globally there is a lack of measure and control devices in the main network's knots, equipment that should allow an optimal operating of the systems in various stressing conditions. Also the global lack of informatics do not allow the simulations of various operating situations as to establish the optimal operational vector, adapted to each concrete situation apart.

These conditions are inducing, when operating water supply systems in great urban areas, the following shortcomings:

- global water consumptions sensibly superior to the standard figures which are the basic parameters for sizing the systems' elements (mains, compensation tanks, pumping stations);
- great hourly fluctuations of the water demand (greater than the one which is usual for this type of systems);
- great pressure fluctuations, with great "falls" at peak hours, which disturb the supply conditions at the directly connected users and also at these fed by centralized sub-systems with auxiliary pumps;
- preferential flows, in total disproportion with the sources' capacities, appearing especially in the low areas (consequence of the energetical unbalance which appears when tanks and/or networks belonging to different areas located at different heights are connected without using pressure regulators, flow limiting or a proper monitoring of the flows for great consumers located in the low areas)

All these are inducing the following effects:

- low quality of the water supply conditions, especially because low pressures at the user and even because water cutoffs in certain areas (from 5 to 6 hours daily);
- high specific power consumptions because the pumps' low efficiencies (the low efficiencies are induced by worn out plant but also its operation in inadequate regimes created by the above-mentioned reasons)

When those which operate a water system are hurting them against the problems above-mentioned the upgrading must start by reinforcing the supplying network, thoroughly analyzing all pumps in the hydrotechnical diagram and then adequately equipping them the way to enhance the energetical and economical parameters of the system.

These, which are actually designing upgra-dings, are usually proceeding to successive testing, by choosing different reinforced diameters on certain mains and then checking the operational parameters in the new conditions. This process ends when the hydraulic acceptable solution is found, without special concerns for the energetical and economical efficiency of the system.

This type of "parametrical" study, when the mains and the pumps are re-sized at each test over and over (without guaranteeing the optimal solution in terms of economy) is to be avoided. This is possible by thoroughly analyzing all shortcomings and, then, formulating the optimizing issue which has the goal to minimize the global operational costs for the upgraded system.

3. Premises, assumptions, methods

Conceiving, designing and rationally operating an optimal system means a creative solving of all shortcomings, in an environment free of all empirics and without the attempts which are typical for the "test and error" solving methods. Also, the experts have to give up the "already checked" solutions (proper to reproductive solving) and adopt a systematical approach, which shall be consistently followed within all phases of the "analytical study" of the system.

The behavior of the considered system is determined by its real and intrinsic characteristics, by the "leading vector" which actions at a certain moment and by the actions of disturbing factors which act upon it within the time lapse for which analysis is done.

In the case of a service supplying system (as the water supply industry is) its behavior crucially influences the quality of the supplied service, and, within a significant period of time (e.g. one year), determines the consumption of basic resources, chiefly power.

When the hydraulic energy is supplied by pumps, the operating regime is determined, at a given moment, by the equivalent characteristic (at that moment) of the system represented by the network and the active users and by the equivalent characteristic (at the same moment) of the pumping systems. The resource consumptions depend on the features of the various operating conditions, which appear during operating but also by their change frequency within the analyzed period of time.

The equivalent characteristic of the networkactive users system is given by the hydraulic and geometric features of the constitutive elements of the network (on the route on which water flows) and also by the number of active users, their location within the network and their functional particularities.

Agreeing that each set of active users, defined by number, location within the network and functional characteristics defines an "operational configuration" of the network it is to observe the fact that to each of those corresponds a very well determined

The way to be sure that the quality that the users request is achieved, the actual operating conditions must be situated in an area of loads defined (for each operating configuration) in a probabilistic manner, between a minimal load which does not ensure the service load of users (the percentage of supplying of adequate services being 0) and a maximal load which ensures the service load for all users (which means a correct service to 100% of users).

The great number of users/potential users connected to the network, together with the diversity of their functional features and the multitude of possibilities in terms of location at one moment within the network, determines, at least theoretically, an infinity of possible operating configurations for the network-users system, towards which shall correspond an infinity of (H-Q) characteristics on which can be defined the loads domains which ensure optimal service with a total probability between 0 and 100% of the total of active users.

Considering this it seems rational that the operation of the users-network system to be defined (within the (Q, H) plan) by the wrapping load characteristics $(H-Q)^{P}$, obtained as the geometrical place of the points which, on various equivalent characteristics, ensure the same total probability of adequate supply to users (P), the way as it is principially shown in Fig. 1.



Fig. 1 Operational characteristics of the network-users system

The equivalent characteristic of a pumping plant is determined by its structure, the functional features of the component elements and the combined pumps configuration that operates at one moment at which defines "the operating configuration of the pumping plant".

Each operating configuration of the pumping plant has a well-determined (H~Q) cha-racterristic; each operating condition is charac-terized by a certain global pumping efficiency and a certain specifically power consumption.

The way to have an efficient pumping plant in terms of power and economy it is manda-tory that current operating conditions (on each operating configuration) to be situated on a load domain defined by the minimal and the maximal loads that can be ensured with an efficiency superior to an accepted figure e. g. 90% of the maximal efficiency of the used

pumps. The relatively small number of pumps that are included in a pumping plant and the fact that usually only two types of pumps are used (a number of main pumps and besides 2 or 3 "small" pumps which, on the same load domain, are processing flows being half the flows processed by the main pumps) determine a finite number of possible operating configurations, located between the (H-Q) characteristic corresponding to a separate operating of a "small" pump and the characteristic corresponding to the parallel operating of all pumps within the pumping plant (Fig.2).



Fig. 2 Operational characteristics of the pumping

Operational and energetical optimization of a water supply system has as goal to set the necessary structure of the mains and pumping plants, as well as to define the operational algorithm which should ensure all range of flows demanded by the users (within the capacity's limits and at an optimal service pressure), even in the most disadvantaged network areas, in the conditions in which the specifically power consumptions has to be minimized.

The way to achieve this goal is shown, within the (Q-H) plan, by the location of the wrapping characteristics corresponding to the desired quality level of the service (assessed by the global probability of total fulfillment of the service load) $(H\sim Q)^P$, within the limits of the efficient pumping of the pumping plant (Fig. 3).



Fig.3 Optimal operating of the network-pumping plant system (in terms of hydraulicity and power consumption)

The set of parameters that intervene within the process of conceiving a supply network, defines, within the project, a theoretical field for the system's operation, which sets its designed capacity. This capacity is chiefly determined by decisional parameters, which are set after a selection process, which is not always explicit, particularly in terms of service's quality (P), technical conception of the network (routes, equipment density) and forecasts of the needs.

The way in which things get concretized and especially the users' behavior lead to the defining to a real domain of the actual operation of the system, domain which indicates the possible values for the supplied service's quality and the corresponding specifically power consumptions.

During operating water supply systems, sometimes situations occur when the real operating domain does not match to the theoretical domain defined within the users that have to be supplied. This leads to a "fall" of the network, shortcomings within the process of fulfilling the users needs, and which have to be abated generally by reinforcing the network and/or enhancing the sources.

It is possible to complete these actions by conceiving a new system able to satisfy the current needs and also the future ones, selecting new decisional parameters and resizing the network in the new configuration. Also it is possible to try to make clear and explicitate the selection of the decisional parameters on the basis of the conclusions obtained from the assessment of the existing system and its shortcomings.

4. Analysing shortcomings

Shortcomings notified during operating water supply systems may have various causes.

Reporting us to the items shown in Section 2 these can be organized in two classes:

- inadequate definition of the theoretical operating domain (improper selection of the decisional parameters, determined by hindrances met during the estimation of the future behavior of the users);
- an unavoidable evolution of the supplied system, meaning an increase of demands, following an increase of the number of users and their specifically needs, evolution that

eventually leads to an alteration of the real operational domain.

Practically, shortcomings of the water systems can be also caused by exceptional situations/phenomena, which occur during short periods and with a low frequency. These are named conjectural shortcomings and do not make the scope of this work that treats only the quasi-permanent shortcomings.

Concerning their location within the system persistent shortcomings are classed as it follows:

- lacks of capacity of the adduction-supplying networks which has as impact the impossibility to ensure the standard service quality on the users' connections (poor quality of the service "downstream"); these shortcomings are subjectively perceived by the users, the system operator being eventually notified about their location and extent (usually through peers);
- lacks of capacity of water storages/capacity of supply of networks (these shortcomigs are first notified by the system operator.

"Downstream" shortcomings (at the user's connection)

The user, especially as low service pressure, which leads to low flows, especially during peak hours, observes these shortcomings. Generally it is hard to know precisely the causes for these shortcomings. The real causes can be pinpointed only by checks made on the field, simulating the behavior of the user-network system, during which the flow in the system is measured connection by connection most of the cases these connections are belonging to the area where the pressure lacks are notified.

Calculations are made with dedicated software which usually perform the following:

- setting the parameters of the hydraulic model the way to recreate the real system (in this phase it is possible to check the data of the network's operators, data which are not always properly recorded)
- objectively knowledge of the network's operation, separating areas with the most numerous shortcomings, quantitatively characterized by the "connection" situations in which the ratio "real value/nominal value" for the supplied flow is less than 1 (respectively of the service pressures p/p_n)

This connection assessment brings information about the networks "health condition" and about the shortcomings that may occur, but no information about the corrective measures to be undertaken. Their nature can be detected by investigation means that lead to a global description of the network (Fig. 4):



Fig. 4 Analysing shortcomings by confronting the three components of the water supply system





Fig. 5 Analysis of the of the necessary volumes for daytime compensation

- the load characteristics H~Q)^P, corresponding to the various service qualities ensured by the network
- the hyetograph of the real distributed flows in the studied network

The causes of the network's shortcomings can be assessed by confronting the characteristics of the

components of the system (supply plant, network, connections, users) with a probabilistic-cally behavior, shown in the same plan (**Q**,**H**) **4** (**Q**,**f**).

The specific features of the graphs drawn for the studied system are reflecting the situations in which shortcomings are occurring and also their frequency, also suggesting the directions in which can be searched the solution for reinforcing/ upgrading the network:

- acting upon the incidence of flows, by modifying the users' behavior (decreasing peak consumptions with low frequency)
- decreasing within the (Q~H) plan the network's incidental characteristics through its reinforcement
- increasing within the (Q~H) plan the characteristics of frontal plant by accordingly modifying the network's supplying systems
- combinations of the three classes of means, the way to outline the optimal upgrading solutions

"Upstream" shortcomings

These shortcomings are directly detected by the water supply system's operator and represent in fact a lack of water volumes to be distributed and this because of:

- the size of annual water reserves (hydrological and river management issue, solvable with water resources management programs within the respective hydrographical basin)
- the water input volumes (constant on 24 hrs, constant on a less 24 hrs period, variable on daytime)
- the extent of water demand (very fluctuant, depending on the nature/class of users and the daily fluctuation of the used water flows)

The analysis of the shortcomings of this class is to be made by confronting (on the same graph) the integral/cumulated curves of the water input with those of the simultaneous demands, curves that also allow the calculus of the necessary daytime compensation volume.

Situations to be studied apart:

- the water input is globally sufficient, the shortcomings being caused by a modification of the input and/or demands, compared to the project,
- fact generated by an alteration of the characteristics of frontal plant, of the users' and of their behavior
- the input becomes globally insufficient, being necessary the supply of an extra-volume (ΔW) and this because of: (1) an adequate modification of the pumping techniques; (2)

increasing the stored water volume; (3) increasing the diameter of the gravitary adduction main, within a conception to be completed from case to case. Within this analysis it is mandatory to check if the shortcoming was not caused by an exceptional situation of very low frequency (it is to be considered that frequently there is a correlation between the decrease of water inputs and the increase of water demands, correlation that complicates the analysis)

5. Quantifying the goals of the reinforcement

These goals are to be settled on the basis of the shortcomings detected during the operation of the water system and taking into account the evolution of the water demand within a forecast range with an extension appropriate to the magnitude of the upgrading.

The various phases of the analysis allows to set the water transportation capacities on various routes of the network and also the compensation and/or reserve volumes that have to be ensured, within a process which has to also consider the users' behavior which can be laxer as long as the networks is not saturated.

Generally the calculus parameters are very influenced by the way the system's operation is forecasted: on request with a certain operating quality; with daily variation of the flows consumed by the great industrial consumers or with agreement upon a certain restriction supply schedule; at least in certain areas. It is obvious that these different types of operating are requesting sensibly different annual expenditure and investments.

6. Conclusions

Phases of reinforcing water networks

The conception used to reinforce a water supply system is the result of a thorough analysis of the operating shortcomings and have as goals the following:

- modifying the supplying/pumping technologies (pumps number/types; pumping plant operating condition adjustments)
- modifying compensation capacities needed in the network
- the proper reinforcing of the network

The way to ensure an optimal upgrading of the deficient network, this process is to be completed after
an analytical study of the system, study which includes the following phases that can over-lap one each other or even can be re-started:

- analyzing the shortcomings of the current network and the forecasted users' water demands
- quantifying all upgrading technical solutions in the given conditions
- checking proposed solutions by analyzing the new network obtained after the designed modifications

An efficient reinforcement program for water networks cannot be conceived apart the attempt to optimize the reinforcement, a design activity completed in the 2nd phase of the process: quantifying solutions.

A good methodology for optimizing the reinforcement of water networks based on the analytical study of the links between the parameters that characterize its operation, the geometric and structural parameters and the investment's and operation's costs in the new conditions, are elemnts that dictate the approach for ellaborating the solution, decreases the necessary working time and guarantees the selection of the optimal ways to abate the detected shortcomings.

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Experimental Researches upon the Functional Characteristics of the Valves. VI. Dn80 Steel Centered Butterfly Valve

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Abstract: In this paper the experimental installation designed to determine the functional characteristics of the flow control devices, the relationships and the testing methodology are introduced. The testing methodology concerns some important aspects like: the total pressure drop of the testing pipe, including the pressure drop due to the pipe wall friction between the static upstream and downstream pressure taps, the pressure drop across the valve, the discharge coefficient, the inherent characteristic of the valve, the local hydraulic losses coefficient. Also, the experimental results obtained for the Dn80 steel centered butterfly valve are presented.

Keywords: steel centered butterfly valve, discharge coefficient, inherent characteristic.

1. Introduction

The experimental determination of the functional characteristics of the valves supposes that an efficient experimental strategy has to be developed, being directed toward: the analysis of the main schemes of the experimental installation and the determination of the relationships for the local hydraulic losses and the discharge coefficients [1]; the design of the traditional measurement system for the determination of the inherent characteristic of the valve [2]; the design of some constructive high-performance solutions based on the virtual instrumentation technologies (provided by the National Instruments-NI) [3], [4], [5].

In this paper the experimental installation, the relationships and the testing methodology are introduced. Also, the experimental results obtained for the Dn80 steel centered butterfly valve are presented.

2. Testing Methodology

In order to determine the inherent characteristic of the valve, generally, two strategies can be developed.

• The direct method. The inherent characteristic of the valve can be obtained directly, through the determination of the $k_v(\theta)$ function for a constant Reynolds number. Here the k_v is the discharge coefficient, θ being the valve opening.

• The indirect method. The inherent characteristic of the valve is obtained indirectly, by two steps. First, the $k_v(Re)$ function for a constant value of the valve opening (θ =const.) is determined. Second, the $k_v(\theta)$ function for a certain Reynolds number is obtained.

Due to the difficulties involving in the control process of the flow working conditions for different valve openings, the second method is used preferentially.

The most important steps of the experimental strategy, prior to the testing process of the valve are:

• The definition of the discharge coefficient:

$$K_{\nu} = Q \sqrt{\frac{\rho}{\rho_0} \cdot \frac{\Delta p_0}{\Delta p_R}} \quad [\text{m}^3/\text{h}]$$
(1)

in which $Q[m^3/h]$ is the flow of a fluid with the density ρ [kg/m³] which, when passing through the valve, produces a pressure drop equal to Δp_R [Pa] for reference conditions $\Delta p_0 = 1$ bar (10⁵ Pa) and $\rho_0 = 1$ kg/dm³ (10³ kg/m³).

• The definition of the inherent characteristic:

$$\frac{k_{\nu}}{k_{\nu 100}} = f\left(\frac{\theta}{\theta_{100}}\right) \tag{2}$$

in which k_{ν} and $k_{\nu 100}$ represent the discharge coefficients corresponding to any angular valve opening θ , and to the nominal valve opening θ_{100} , respectively, θ/θ_{100} represent the dimensionless valve opening parameter, Fig. 1.



Fig. 1. Scematic figure of the centered butterfly valve.

- The definition of the principle scheme of the measuring section [1].
- The definition of the experimental installation, Fig. 2 [2].
- The definition of the fluid flow:

$$Q = \alpha_c \frac{\pi d^2}{4} \sqrt{\frac{2}{\rho_a} \Delta p_d}$$
(3)

in which $\alpha_c = \alpha k_{\alpha}$ is the corrected flow coefficient; $\rho_a [kg/m^3]$ is the density of the working fluid; $\Delta p_d [Pa]$ is the pressure drop measured on the upstream and downstream sides of the diaphragm.

• The definition of the differential pressure drop across the diaphragm:

$$\Delta p_d = \Delta h_{dm} (\gamma_m - \gamma_a) \tag{4}$$

where Δh_{dm} is determined by means of the U-shaped differential manometer UI, which uses mercury as

manometer liquid, Fig. 2.

• The definition of the lengthwise hydraulic losses coefficient:

$$\lambda = 2g \frac{D_n^3}{Lv^2} \frac{\Delta h_l}{Re^2} \tag{5}$$

in which $\text{Re}=VD_n/v$ is the Reynolds number corresponding to the flow with the velocity V of the liquid with the cinematic viscosity v, through the pipe with the diameter D_n .

- The definition of the total pressure drop of the valve including the pressure drop due to the pipe wall friction between the static upstream and downstream pressure taps:
 - for one single manometer, for example the U-shaped manometer U2, which measure the level change Δh_{v1m} (γ_m and γ_a [N/m³] are the specific weight of the manometer liquid (mercury), and the working fluid, respectively.

$$\Delta p_{12}^1 = \Delta h_{v1m} (\gamma_m - \gamma_a) \tag{6}$$

• for two serial coupled manometers, U2 and U3.

$$\Delta p_{12}^2 = 2 \cdot \Delta h_{v1m} (\gamma_m - \gamma_a) \tag{7}$$

• for small pressure measurement, case in which a reversed U-shaped manometer U6 is used by adequate handling of isolation valves R_{i5} , R_{i6} , R_{i7} and R_{i8} .

$$\Delta p_{12} = \Delta h_{va} \cdot \gamma_a \tag{8}$$

• The definition of the linear loss of head on the sections upstream and downstream of the valve to be tested:

$$\Delta h_{vlin} = \lambda \left(Re \right) \frac{L_{v1} + L_{v2}}{D_n} \frac{V^2}{2g} \tag{9}$$



Fig. 2. The experimental installation.

V-valve; *D*-flow-meter with diaphragm; *P1*, *P2*-presure plugs for the determination of the linear loss of head; *TT*-temperature transducer; *Pv1*, *Pv2*-pressure plugs for the determination of the pressure drop across the valve; *Mv1*, *Mv2*- manometers; *R1*, *R2*-admission valve; *R3*-discharge valve, *R4*-emptying valve; *Ra1*, *Ra2*, *Ra3*-aeration valves; *Ri1*, *Ri2*, *Ri3*, *Ri4*, *Ri5*, *Ri6*, *Ri7*, *Ri8*-isolation valves; *U1*, *U2*, *U3*-U-shaped manometers with mercury; *U4*, *U5*, *U6*-reversed U-shaped manometers with water; *R*-weighing tank; *C*-scale.

• The definition of the pressure drop due to the pipe wall friction between the static upstream and downstream pressure taps:

$$\Delta p_{vlin} = \gamma_a \cdot \Delta h_{vlin} \tag{10}$$

• The definition of the pressure drop of the valve:

$$\Delta p_R = \Delta p_{12} - \Delta p_{vlin} \tag{11}$$

• The definition of the local hydraulic losses coefficient of the valve:

$$\zeta_R = \frac{n}{\left(\alpha\beta^2\right)^2} \frac{\Delta h_{v1m}}{\Delta h_{dm}} - \lambda \left(Re\right) \frac{L_{v1} + L_{v2}}{D_n} \tag{12}$$

in which *n* represent the number of the manometers used to determine the Δh_{v1m} level change.

The testing methodology consists in:

- The determination of the flow working conditions. For different flows in the testing pipe, the level change Δh_{dm} on the U-shaped differential manometer U1 is measured. The differential pressure drop Δp_d on the two sides of the diaphragm is calculated with Eq. (4). The flow Q through testing pipe is calculated with Eq. (3). The velocity V of the working fluid is obtained with the continuity equation for incompressible flows. Finally, the flow working conditions is determined by calculating the Reynolds number.
- The determination of the pressure drop due to the pipe wall friction. For different flows in the testing pipe the level change Δh_l in measured with the reversed U-shaped manometer U5. The dependence λ (Re) is obtained with Eq. (5).
- The determination of the total pressure drop of the valve including the pressure drop due to the pipe wall friction between the static upstream and downstream pressure taps. For different flows in the working pipe and for a certain value of the valve opening θ the change level Δh_{vlm} is measured with the U-shaped manometer U2, or the change level Δh_{va} with the reversed Ushaped manometer U6 (for small pressure drop).

The total pressure drop of the testing valve Δp_{12} is calculated with Eq. (6), Eq. (7) or Eq. (8). The measurement process is repeated for different values of the angular valve opening θ .

- The determination of the pressure drop across the valve. First, the pressure drop due to the pipe wall friction between the static upstream and downstream pressure taps is calculated with Eq. (10), where the linear loss of head on the sections upstream and downstream of the valve to be tested Δh_{vlin} is obtained with Eq. (9). Second, the pressure drop of the valve is calculated with Eq. (11). Third, the dependence Δp_R (*Re*) is obtained, for different values of the dimensionless valve opening parameter θ/θ_{100} .
- The determination of the discharge coefficient. Equation (1) is used. The dependence $K_{\nu}(Re)$ is obtained for different values of the dimensionless opening valve parameter θ/θ_{100} .
- The determination of the inherent characteristic. Equation (2) is used. It has to be indicated the value of the Reynolds number $(Re=10^5)$ for which the inherent characteristic of the value has been obtained.
- The determination of the local hydraulic losses coefficient of the valve. Equation (22) is used. The dependence $\zeta_R(Re)$ is presented for different values of the dimensionless opening valve parameter θ/θ_{100} .

3. Experimental Results

The dependence of the pressure drop of the valve $\Delta p_R [\text{N/m}^2]$ in function of the Reynolds number is presented in Fig 3 for different values of the dimensionless valve opening parameter $\theta/\theta_{100} = [90 \ 80 \ 70 \ 60 \ 50 \ 40]/90 = [1 \ 0.88 \ 0.77 \ 0.66 \ 0.55 \ 0.44].$

The inherent characteristic of the valve $k_v/k_{v100}(\theta/\theta_{100})$ for *Re*=10⁵ is presented in Fig. 4.

The dependence of the local hydraulic losses coefficient of the valve ζ_R in function of the dimensionless opening valve parameter θ/θ_{100} for $Re=10^5$ is presented in Fig. 5



Fig. 3. The pressure drop across the value $\Delta p_R(\text{Re})$, θ =const.



Fig. 4. The inherent characteristic $k_v/k_{v100}(\theta/\theta_{100})$, $Re=10^5$.



Fig. 5. The local hydraulic losses coefficient $\zeta_R(\theta/\theta_{100})$, $Re=10^5$.

4. Conclusion

The strategy developed in this paper for valves testing process consist in an indirectly determination of the inherent characteristic: the $k_v(Re)$ function for a constant value of the valve opening (θ =const.) is determined by experiment and then the inherent characteristic $k_v(\theta)$ function for a certain Reynolds number is obtained by calculation.

The experimental installation developed for this testing procedures is well equipped with all the measurement devices to perform measurements in a wide range of the flow working conditions. Unfortunately, for the reason of the increasing of the pressure drop across the valve (Fig. 3), only the range $\theta/\theta_{100} = 1\div0.44$ has been investigated.

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Experimental Researches Upon the Functional Characteristics of the Valves. VII. Dn 80 Steel Ball Valve

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Abstract. In this paper the experimental results (the dependence of the pressure drop across the valve $\Delta p_R [\text{N/m}^2]$ as a function of the Reynolds number for different values of the dimensionless valve opening parameter θ/θ_{100} ; the inherent characteristic of the valve $k_v/k_{v100} (\theta/\theta_{100})$, for $Re=10^5$; the dependence of the local hydraulic loss coefficient of the valve ζ_R as a function of the dimensionless opening valve parameter θ/θ_{100} for $Re=10^5$) that have been obtained for the Dn80 steel ball valve will be presented. Also, the calculation code that has been developed in MATLAB programming language for the experimental data analysis will be introduced.

Keywords: steel ball valve, discharge coefficient, inherent characteristic.

1. Introduction

The testing procedures, including the description of the experimental installation, the relationship for all the parameters to be measured and to be calculated for the determination of the inherent characteristic of the valves have been introduced in [1], [2] and [3].

Also, some experimental results of a Dn80 steel centered butterfly valve testing have been presented in [3]. The strategy developed for testing process consists in an indirectly two steps determination of the inherent characteristic of the valve.

First, the $k_v(Re)$ function for a constant value of the valve opening (θ =const.) is determined by the experiment. Second, the inherent characteristic $k_v(\theta)$ function for a certain Reynolds number (in this case the reference value of the Reynolds number Re=10⁵ has been used) is obtained by performing some calculation routines.

In this paper the calculation code that has been developed in MATLAB programming language for the experimental data analysis will be introduced.

Also the experimental results obtained for a Dn80 steel ball valve, which is schematic represented in Fig. 1, will be presented.



Fig. 1. Schematic figure of the ball valve.

2. MATLAB Calculation Codes

To effect the analysis of experimental data (the calculations and the graphic representations), in a first stage, codes of calculation have been developed in the MATLAB programming environment.

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This programming environment has not only been select for the native facility to manipulate matrix (there is also a new facility that allows to work with the multidimensional structures of data) but also for an impressive library of functions (functions to effect various specific operations for the analysis of experimental data: data preprocessing, basic data analysis functions, regression and curve fitting, Fourier analysis and FFT, functions for basic and specialized plotting, for graph formatting, for 3-D visualization).

The resolution of the problem of analysis of experimental data for the point of view that concerns the code of calculation developed in the MATLAB programming language has some particularities, among what the most important concern:

- The great number of experimental data is the motive for what different calculation codes have been developed, each one for every experimental data set. Thus, for the steel ball valve the primary experimental data have been saved in eight data files. Once that the version R11 of the MATLAB programming language has been released in 1999, also the possibility to work with the multidimensional structures of data has been developed. In this particular case, the experimental data have been allocated to a multidimensional array, named DATA. To do this the principal problem has been the different number of data in every data file. This problem has been resolved by re-dimensioning of every data files using the method to introduce some new experimental data that are equal with the last data in every data file, until that homogeneous dimensional data will be obtained.
- Some particularities of the different types of flow control devices that have been studied in this research, for example the maximum value of the opening valve angle for which have been made measurements with the experimental installation, makes to think about developing more codes of calculation, each one for every type of flow control device that have been tested. Therefore, for the case of the steel ball valve the principal calculation code is *csf.m.*
- The linear loss of head can be obtained making a preliminary measurement and it doesn't depend of the opening angle of the valve. This thing makes rationally the decision to develop a secondary calculation code for the determination of the

 λ (*Re*) function, which finally, will transferred this dependence into a data file on the hard disk. Therefore, the secondary calculation code is *lambda.m* and the data file with the λ (*Re*) dependence will be *LAMBDA.dat*. The principal calculation code contains a module of calculation destined to the determination of the linear loss of head that reads the data saved in the *LAMBDA.dat* data file; thus, the secondary calculation code being performed only one time.

2.1. The secondary code *lambda.m*

%THE SECONDARY CODE FOR THE CALCULATION OF THE DISTRIBUTED HYDRAULIC LOSSES COEFFICIENT LAMBDA pack:clear:clc: %INPUT DATA %Pipe diameter. The equivalent roughness coefficient of the pipe. The density of mercury. The acceleration of gravity. The atmospheric pressure. D=0.0795;ke=0.2*10^-3;rhom=13596; g=9.81; patm=101325; %Pipe length for calculation of the distributed hydraulic losses L=4;Lv=8*D; %Flow coefficient and the diameter of the diaphragm. alpha=0.61856;d=0.0371; %Working temperature. t=14; %PREPROCESSING DATA. %Calculation of the density and the kinematic viscosity of the water at the working temperature %Data file reading: RHO=dlmread('RHO.dat','\t');rho=table1(RHO,t);eta $0=17.887*10^{(-4)};eta=eta0/(1+0.0337*t+$ $0.000221 * t^2$;miu=eta/rho; %CALCULATION OF THE DISTRIBUTED HYDRAULIC LOSSES COEFFICIENT LAMBDA. %Data file reading: LIN=dlmread('LIN.dat','\t'); hd1l=LIN(:,1);hd2l=LIN(:,2);dhdl=(hd2l-hd1l)/1000; hl1=LIN(:,3);hl2=LIN(:,4); dhl=(hl1-hl2)/1000; %Calculation of the pressure drop of the two sides of the diaphragm: dpdl=(rhom-rho)*g*dhdl; %Calculation of the flow:

Ql=alpha*pi*d^2/4*sqrt(2/rho*dpdl);

%Calculation of the velocity: Vl=Ql/(pi*D^2/4); %Calculation of the Reynolds number: Rl=Vl*D/miu; %Calculation of the distributed hydraulic losses coefficient: lambdal=dhl*D/L*2*g./Vl.^2; %CURVE FITTING. gr=5; fl=polyval(polyfit(Rl,lambdal,gr),Rl); %PLOTTING THE DEPENDENCE lambda(Re): plot(Rl,lambdal,'ko',Rl,fl,'k-');grid;xlabel('Re');ylabel('\lambda'); %Data file writing in LAMBDA.dat file. dlmwrite('LAMBDA.dat',[Rl fl],'\t');

Observations

- The data that refer to the dependence of the density of the water as a function of the temperature are found in the data file *RHO.dat*.
- The data that refer to the measured parameters, which it is needs for the determination of the dependence λ (*Re*) are found in the data file *LIN.dat*.

2.2. The primary code *csf.m*

%THE PRIMARY CODE FOR THE DETERMINA-TION OF THE INHERENT CHARACTERISTIC OF THE STEEL BALL VALVES pack:clear:clc: %INPUT DATA. %Pipe diameter. The ecquivalent roughness coefficient of the pipe. The density of mercury. The acceleration of gravity. The atmospheric pressure. D=0.0795;rhom=13596;g=9.81;patm=101325; %Pipe length for calculation of the distributed hydraulic losses. L=4;Lv=8*D; %Flow coefficient and the diameter of the diaphragm. alpha=0.61856;d=0.0371; %Working temperature. t=14; %Reference values for density and pressure. rhor=1000;dpr=10^5; %Reference value for the Reynolds number. Re=10^5: %Reference values for the opening valve position.

%Reference values for the opening valve position. UR= $[0\ 10\ 20\ 30\ 40\ 50\ 60\ 70\ 80\ 90];$ %PREPROCESSING DATA. %Calculation of the density and the kinematic viscosity of the water at the working temperature %Data file reading: RHO=dlmread('RHO.dat','\t'); rho=table1(RHO,t); eta0=17.887*10^(-4);eta=eta0/(1+0.0337*t+ 0.000221 *t^2);miu=eta/rho; %CALCULATION OF THE DISTRIBUTED HYDRAULIC LOSSES COEFFICIENT LAMBDA. %Data file reading: RL=dlmread('LAMBDA.dat','\t'); Rl=RL(:,1);fl=RL(:,2);%CALCULATION OF THE FUNCTIONAL PARAMETERS OF THE VALVE. %Working values for the opening valve position. U=[90 80 70 60 50 40 30 20]; P=max(size(U)); %Data file reading: D90=dlmread('D90sf.dat','\t'); D80=dlmread('D80sf.dat','\t'); D70=dlmread('D70sf.dat','\t'); $D60 = dlmread('D60sf.dat', '\t');$ D50=dlmread ('D50sf.dat','\t'); D40=dlmread('D40sf.dat','\t'); D30=dlmread('D30sf.dat','\t'); D20=dlmread('D20sf.dat','\t'); %Multidimensional data array generation DATA(:,:,1)=D90;DATA(:,:,2)=D80; DATA(:,:,3)=D70;DATA(:,:,4)=D60; DATA(:,:,5)=D50;DATA(:,:,6)=D40; DATA(:,:,7)=D30; DATA(:,:,8)=D20 for k=1:P DCC=DATA(:,:,k); hd1(:,k)=DCC(:,1);hd2(:,k)=DCC(:,2); dhd=(hd2-hd1)/1000;dpd=(rhom- rho)*g*dhd; hv1(:,k)=DCC(:,3);hv2(:,k)=DCC(:,4); dhv=(hv1-hv2)/1000; %Calculation of the pressure drop across the valve is made in different way function of the type of the measurement device: U-shaped with mercury or reversed U-shaped with water. if dhv(:,k) > 0dpvt(:,k)=rho*g*dhv(:,k);

else

dpvt(:,k)=(rhom-rho)*g*(-dhv(:,k)); end

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%Calculation of flow, velocity and Reynolds number Q=alpha*pi*d^2/4*sqrt(2/rho*dpd);V=Q/(pi*D^2/4);

R=V*D/miu;m=max(size(R)); %Calculation of the distributed hydraulic losses is made for the specific values of the Reynolds number using the linear interpolation. for j=1:m if R(j,k) > max(Rl)lambda(j,k)=fl(length(fl)-1); elseif R(j,k)<min(Rl) lambda(j,k)=fl(1);else lambda(j,k)=interp1(Rl,fl,R(j,k)); end end %Calculation of the pressure drop across the valve. $dpvl(:,k)=lambda(:,k)*Lv*rho/(2*D).*V(:,k).^2;dpv($:,k)=dpvt(:,k)-dpvl(:,k);%Calculation of the local hydraulic losses coefficient of the valve. $zeta(:,k)=dpv(:,k)./(rho*V(:,k).^{2/2});$ %Calculation of the discharge coefficient of the valve kv. kv(:,k)=Q(:,k)*3600.*sqrt(rho/rhor*dpr./dpv(:,k));%CURVE FITTING. gr=3; fR=linspace(min(R(:,k)),max(R(:,k))); fzeta=polyval(polyfit(R(:,k),zeta(:,k),gr),fR); fkv=polyval(polyfit(R(:,k),kv(:,k),gr),fR); ZETA(k)=interp1(fR,fzeta,Re); KV(k)=interp1(fR,fkv,Re); %PLOTTING zeta(Re) AND kv(Re). figure plot(R(:,k),zeta(:,k),'ko',fR,fzeta,'k-'); grid; xlabel('Re');vlabel('\zeta'); title(['\theta/\theta100='num2str(U(k)/max(UR))]): figure plot(R(:,k),kv(:,k),'ks',fR,fkv,'k-');grid; xlabel('Re');ylabel('kv'); title(['\theta/\theta100=' num2str(U(k)/max(UR))]); end %PLOTTING THE INHERENT CHARACTERISTIC OF THE VALVE. %kv(theta), Re=const. figure plot(UR/max(UR),0,U/max(UR),KV/max(KV),'k-o'); grid; ylabel('kv/kv100'); xlabel(' Full close \theta/\theta100 Full open \rightarrow '); title(['Steel ball valve. Re= ' num2str(Re)]); %zeta(theta), Re=const. figure plot(UR/max(UR),0,U/max(UR),ZETA,'k-o');grid;

xlabel(' Full close \theta/\theta100 Full open \rightarrow); vlabel('zeta'); title(['Steel ball valve. Re=' num2str(Re)]); %PLOTTING THE PRESSURE DROP ACROSS THE VALVE. %dpR(Re), theta=const. figure plot(R(:,1),dpv(:,1),'k-o',R(:,2),dpv(:,2),'k-x', R(:,3),dpv(:,3),'k-<',R(:,4),dpv(:,4),'k->');hold on; plot(R(:,5),dpv(:,5),'k-*',R(:,6),dpv(:,6),'k-s', R(:,7),dpv(:,7),k-d');grid;xlabel('Re'); title('Steel ball valve');vlabel('dpR'); legend(num2str(U(1)/max(UR)),num2str(U(2)/max(UR)),num2str(U(3)/max(UR)),num2str(U(4)/max(U R)),num2str(U(5)/max(UR)),num2str(U(6)/max(UR))),num2str(U(7)/max(UR)),0);

Observations

- The data that refer to the dependence of the density of the water about the temperature are found in the data file *RHO.dat*.
- The data that refer to the dependence λ (*Re*) are found in data file *LAMBDA.dat*.
- The primary data that refer to the measured parameters are found in eight data files: D90sf.dat, D80sf.dat, D70sf.dat, D60sf.dat, D50sf.dat, D40sf.dat, D30sf.dat and D20sf.dat. These files refer to the opening valve angles $\theta=90^{\circ}$, 80° , 70° , 60° , 50° , 40° , 30° and 20° . The data have been allocated to a 3-dimensional homogeneous DATA array.
- The calculation of the pressure drop across the valve is made in different way depending of the type of the measurement device: U-shaped with mercury or reversed U-shaped with water. A conditional *if* statement has been used.
- The calculation of the distributed losses coefficient is made for the specific Reynolds number reading the data file *LAMBDA.dat* and performing a data linear interpolation with the *interp1* MATLAB build-in function.
- In order to minimize the measurement errors, the graphical representation of the experimental results consists in curve fitting with 3rd degree fitting polynomials, which will approximate the data for the discharge coefficient but also for the local hydraulic losses coefficient of the valve.

The current data set is divided in a finer grid with 100 points. The *polyfit* build-in MATLAB function has been used.

• The determination of the inherent characteristic but also of the local hydraulic losses coefficient are made by linear interpolation, for a specific value of the Reynolds number: $Re=10^5$. The dependence of the pressure drop across the valve as a function of the Reynolds number for all the working opening valve positions has been represented. The dimensionless valve opening parameter θ/θ_{100} has been preferred.

3. Experimental Results

The dependence of the distributed loss of head coefficient about the Reynolds number is presented in Fig. 2. It is noted that from about $Re\approx 10^5$ the distributed losses coefficient is approximately constant, $\lambda \approx 0.0262$.

Thus, in some situations in which the effective Reynolds number in the working pipe is greater than $1,5 \ 10^5$ (the maximum value for which have been

made measurements, Fig. 2), will be adopted the value 0,0262 for the distributed hydraulic losses coefficient. In order to perform this verification in the secondary calculation code *lambda.m* a conditional *if* statement has been used.

The dependence of the pressure drop of the valve $\Delta p_R [\text{N/m}^2]$ as a function of the Reynolds number is presented in Fig. 3 for different values of the dimensionless valve opening parameter $\theta/\theta_{100} = [90\ 80\ 70\ 60\ 50\ 40\ 30\ 20]/90 = [1\ 0.88\ 0.77\ 0.66\ 0.55\ 0.44\ 0.33\ 0.22].$

The discharge coefficient of the valve k_v (Re) for full open position ($\theta/\theta_{100} = 1$) is presented in Fig. 4. The inherent characteristic of the valve $k_v/k_{v100} (\theta/\theta_{100})$ for $Re=10^5$ is presented in Fig. 5.

The local hydraulic losses coefficient $\zeta_R(\text{Re})$ for full open position ($\theta/\theta_{100} = 1$) is presented in Fig. 6. The dependence of the local hydraulic losses coefficient of the valve ζ_R as a function of the dimensionless opening valve parameter θ/θ_{100} for $Re=10^5$ is presented in Fig. 7.



Fig. 2. The distributed loss of head coefficient $\lambda(Re)$.



Fig. 4. The discharge coefficient of the valve $k_{\nu}(\text{Re})$ for full open position.



Fig. 6. The local hydraulic loss coefficient $\zeta_R(\text{Re})$ for full open position.



Fig. 7. The local hydraulic loss coefficient $\zeta_R(\theta/\theta_{100})$, $Re=10^5$.

4. Conclusion

Figure 2 shows if there is it needs to have a constant value for the distributed losses coefficient λ (according to the general recommendations for this kind of experimental investigations), then the testing process must have done at flow working conditions characterized by Reynolds number greater than $Re\approx10^5$. In this case the value of the distributed losses coefficient is $\lambda\approx0,0262$.

This is the reason for which the graphical representations of the inherent characteristic, as well as of the local hydraulic loss coefficient that have been shown in this paper have been made for a reference value 10^5 of the Reynolds number.

According to the indirectly method developed for these experimental researches, in the first stage the $k_v(Re)$ and $\zeta_R(Re)$ functions for constant values of the valve opening (for example Fig. 4 and Fig. 6, $\theta/\theta_{100} = 1$) have been determined. In the second stage of the testing procedures, based on the primary experimental results obtained in the first stage, the inherent characteristic $k_v/k_{v100}(\theta/\theta_{100})$ and the local hydraulic loss coefficient $\zeta_R(\theta/\theta_{100})$ for the reference value of the Reynolds number Re=10⁵ have been obtained by performing calculation routines and, finally, have been represented in Fig. 5, Fig. 7.

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The Pseudoplastic Liquid Film Flow. The Prandtl-Eyring Model (I)

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Abstract: The non-Newtonian liquid film flow along a flat plane has been analyzed using the Prandtl-Eyring rheologycal model with two parameters. Both the velocity distribution within falling liquid film and the average and surface velocity expressions, without approximations are presented.

Keywords: non-Newtonian liquids, thin liquid film flow.

1. Introduction

Numerous fluids – macroscopic, homogeneous systems, shows deviations from the Newtonian behaviour, as the flow units are non-isodimensional and, under the shearing forces action, they undergo orientations. At these fluids, tension-velocity of shearing dependence is no more linear, and viscosity depends on the parameters of the stress [1].

In the flow problems, a special interest is accorded to non-Newtonian liquids which have only viscosity, and to viscoelastic liquids with a negligible elastic component.

2. Theoretical

For the thin liquid film flow, the most available rheological model, [2], was the Ostwald and de Waele (the power law) one, which characterize the pseudoplastic liquids. This model connects the shearing tension, $\tau [kg \cdot m^{-1} \cdot s^{-1}]$ with shearing velocity, $\dot{\chi} = du/dy [s^{-1}]$ by the following expression:

$$\tau = K \cdot \dot{\gamma}^n \tag{1}$$

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where $u [m \cdot s^{-1}]$ is the velocity upon the flow direction, *x*, but *y* is the other co-ordinate [*m*].

Beside its simplicity, another advantage of the model is to have only two constants: K, representing the liquid consistence, and n, the flow index (non-dimensional, sub-unital).

A drawback of the power law is the fact that one of the parameters, K, has inconvenient dimensions, $[kg \cdot m^{-1} \cdot s^{n-2}]$, depending mainly of the other parameter's value, n. Also, at the null shear ing velocity, the model give $K=\infty$, [3].

On the basis of liquid's kinetic theory, elaborated by Eyring, for the simple shearing case, one can utilise the equation: [5]

$$\tau = -K_1 \arg sh \frac{\dot{\gamma}}{K_2} \tag{2}$$

or, the equivalent, but more difficult to operate mathematically,

$$\tau = -K_1 \ln \left[\frac{\dot{\gamma}}{K_2} + \left(\frac{\dot{\gamma}^2}{K_2^2} + 1 \right)^{\frac{1}{2}} \right]$$
(3)

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where K_1 and K_2 are the material constants that are expressed in s^{-1} and $N.m^{-2}$, respectively.

The ease of work with a rheologycal model consists rather in facile possibility of practical determination, than in the small number of its constants. Here is the mode of calculation for the rheologycal constants, K_1 and K_2 . [4]

- K_1 and K_2 from the equation (2) are determined as follows:
 - from the τ γ diagram one determine the rheo-slopes γ₁(τ₁) and γ₂(τ₂ = 2τ₁);
 one write the ratio:

$$k = \frac{\dot{\gamma}_2}{2\dot{\gamma}_1} = \frac{sh\left(\frac{\tau_2}{K_2}\right)}{2sh\left(\frac{\tau_1}{K_2}\right)} = \frac{sh\left(\frac{2\tau_1}{K_2}\right)}{2sh\left(\frac{\tau_1}{K_2}\right)} = \frac{2sh\left(\frac{\tau_1}{K_2}\right) \cdot ch\left(\frac{\tau_1}{K_2}\right)}{2sh\left(\frac{\tau_1}{K_2}\right)} = ch\left(\frac{\tau_1}{K_2}\right) \tag{4}$$

- on the basis of previous relations:

$$K_{1} = \frac{\dot{\gamma}}{sh\left(\frac{\tau_{1}}{K_{2}}\right)};$$

$$K_{2} = \frac{\tau_{1}}{\arg ch\left(\frac{\dot{\gamma}_{2}}{2\dot{\gamma}_{1}}\right)} = \frac{\tau_{1}}{\arg chk}$$
(5)

Taking into account the expressions of hyperboloic functions, we obtain:

$$K_{1} = \frac{\dot{\gamma}_{1}}{\sqrt{k^{2} - 1}};$$

$$K_{2} = \frac{\tau_{1}}{\ln(k + \sqrt{k^{2} - 1})}$$
(6)

The Prandtl-Eyring model contains also two parameters of material: K_1 and K_2 .

The thin liquid film flow along an inclined plane, unwarmed, in the absence of surface waves, is governed by the following motion equation:

$$\frac{du}{dy} = K_2 sh \frac{\rho g_x \delta}{K_1} ch \frac{\rho g_x y}{K_1} - K_2 ch \frac{\rho g_x \delta}{K_1} sh \frac{\rho g_x y}{K_1}$$

$$\frac{1}{\rho}\frac{\partial\tau}{\partial y} + g_x = 0 \tag{7}$$

where, ρ is the fluid density, $[kg \cdot m^{-3}]$ and g_x – the gravitational acceleration component upon the motion direction, $[m \cdot s^{-2}]$.

We can integrate the equation (7), taking into account that at the free surface of the film, $(y=\delta)$, the shearing tension is null, the velocity geting a maximum value, hence du/dy=0.

Then, we have:

$$\tau = \rho g_x (\delta - y) \tag{8}$$

and, through the equation (2),

$$-K_1 \arg sh \frac{\dot{\gamma}}{K_2} = -\rho g_x \left(\delta - y\right) \tag{9}$$

or, further,

$$\frac{1}{K_2}\frac{du}{dy} = sh\frac{\rho g_x(\delta - y)}{K_1}$$
(10)

Developing to integration,

(11)

C.Zainescu,C.Ciobanu,C.Badea/ Ovidius University Annals of Constructions **3**,**4** 343-346(2001) 345 and, using the second boundary condition: the velocity at the wall is null, *i.e.*, u=0 at $y=\delta$, we within falling liquid film:

$$u = \frac{K_1 K_2}{\rho g_x} \left[ch \frac{\rho g_x \delta}{K} - ch \frac{\rho g_x (\delta - y)}{K_1} \right]$$
(12)

Substituting $y=\delta$ into Eq. (12), the surface velocity, u_s , will be:

and the average velocity, u_M can be obtined by solving its defining equation, *i.e.*:

$$u_{s} = \frac{K_{1}K_{2}}{\rho g_{x}} \left(ch \frac{\rho g_{x}\delta}{K_{1}} - 1 \right)$$
(13)
$$u_{M} = \frac{1}{\delta} \int_{0}^{\delta} u dy = \frac{K_{1}K_{2}}{\rho g_{x}} \left(ch \frac{\rho g_{x}\delta}{K_{1}} - \frac{K_{1}}{\rho g_{x}\delta} sh \frac{\rho g_{x}\delta}{K_{1}} \right)$$
(14)

It is evident that the expression of velocity, given by equation (12) is more complicated than that obtained by using the power law. Performing some lengthy calculation, we can applay the equation (12) in the case of regular waves laminar flow, too. This extension will be the object for subsequent papers. Expliciting rheologycal equation, (2), by developing in series the hyperbolic function and keeping the first two terms, one arrived at the model Steiger-Ory-Rabinowisch, [1].

$$\dot{\gamma} = K_3 \tau + K_4 \tau^3 \tag{15}$$

an equation which contains also two material parameters: K_3 and K_4

Many viscous non-Newtonian fluids, independently of time, have a Newtonian behaviour at both the small and the very large shearing velocity, since there viscosity has two boundary values. Such a behaviour is described by the empirical model Powell-Eyring with three parameters, [6]:

$$\tau = K_s \dot{\gamma} + K_6 \arg sh \frac{\dot{\gamma}}{K_7} \tag{16}$$

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The use of rheological models, (15) and (16) in calculations is limited, because of limited algebraical fomulae. Any such an extension don't been encountered in the case of the thin liquid film flow.

3. Conclusions

The majority of liquids encountered in practice have a different bihaviour than a Newtonian one. Moreover, the thin film technics is like a useful instrument. Consequently, it is necessary to know its resources.

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Evaluation des débits perdus et des débits captés du nouveau drain Timisesti par un modèle mathématique

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Résumé : Les drains imparfaits peuvent avoir pertes importantes d'eau. Ces pertes et la non-réalisation des paramètres caractéristiques conduisent à l'augmentation des coûts d'exploitation des drains. Pour éviter cette situation on doit actionner sur les paramètres qui influence a l'augmentation des débits collectés dans les drains. Dans ce papier sont présente des simulation numériques concernant l'évaluation des débits produites et débits captes sur le nouveau drain de Timisesti ayant comme but l'estimation des débits et des mesures qui doit emmener.

Mot clés: Drain imparfait, eau souterraine, débits captes.

1. Introduction

La zone de Timisesti se trouve dans la partie Est de la Roumanie, dans la région de Moldova. Le système aquatique Neamt-Moldova représente la source principale d'eau de 400.000 habitants. Ayant les caractéristiques générales des sources d'eau souterraine suivant: ancien drain, nouveau drain et le font de captage Zvoranesti (figure 1). La source d'eau souterraine Timisesti assure approximativement 40% du débit du système d'alimentation en eau potable de la ville de Iasi.



Fig. 1. Sources d'eau souterraine Timisesti (Mocanu V. et al., 1987)

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Le nouveau drain était projeté en 1970 et exécutait entre 1971-1973. Il est placé en continuité vers l'amont de l'ancien drain sur la terrasse inférieure de la rivière Moldova. Le drain est de type galerie visitables d'une longueur de 4050 m et une section constante, visitable par 15 galeries de visite $C_1 - C_{15}$ et un puits collecteur à l'extrémité aval. La profondeur de pose du drain est résultée, fonction de la cote de roche imperméable et du niveau d'eau dans la rivière Moldova, entre 10 et 15 m. Ayant une pente continue. Selon la situation existante dans le terrain, a résulté un drain positionner dans l'épaisseur de la couche, donc un drain imparfait (figure 2). Le radier du drain est approximativement au même niveau avec les eaux dans la rivière Moldova, mais l'alimentation du drain se fait uniquement sur la partie qui donne sur la terrasse. L'eau souterraine captée est transportée gravitationelement dans le puits collecteur de l'aval, où il existe un déversoir large. Le drain est projeté pour un débit de 1200 l/s.



Fig. 2. Coupe hydrogéologique du nouveau drain (Mocanu V. et al., 1987)

Dans les dernières 12 – 15 années d'exploitation, les débits moyens mensuels et annuels d'eau captée à Timisesti ont été très différentes. D'une manière les mesures des débits fournis du nouveau drain ont mis en évidence leurs dépendance face au régime des précipitations, ce qui a généré la nécessité d'effectuer des études et des recherches supplémentaires.

La réduction des débits captés dans le nouveau drain est de premièrement au fait que le cours d'eau Ozana facteur important de l'alimentation de l'aquifère a séché, et deuxièmement il a été supposé qu'une importante quantité d'eau du débit infiltré se perd sous le drain (figure 3).

Pour estimer et évaluer la variation des débits perdus sous le drain ainsi que les débits captés par le drain il a été effectué un calcul par un modèle mathématique.



Fig. 3. Situation actuelle

2. Modèle mathématique

Le modèle de calcul a adopté l'utilisation des méthodes numériques pour l'analyse du régime d'écoulement des eaux souterraines. Parmi les méthodes numériques a été opté pour la méthode des éléments finis, qui présente des possibilités supérieures dans la discritisation des domaines de l'écoulement souterrain, et les divers facilités dans la modélisation des solutions constructives de l'intervention et la protection.

L'équation générale de l'écoulement de l'eau souterraine, impliquant la recharge est :

$$\frac{\partial}{\partial x} \left(K_x h \frac{\partial}{\partial x} \right) + \frac{\partial}{\partial y} \left(K_y h \frac{\partial}{\partial y} \right) = S_y \frac{\partial h}{\partial t} - R \tag{1}$$

où x, y et t: variables indépendantes spatiales (m) et temporaires (jour) du régime d'écoulement; h(x,y,t): hauteur d'eau souterraine; K_x , K_y : conductivité hydraulique (m/j) dans les principaux axes; S_y : coefficient d'emmagasinement; R: recharge (m/j). Les calculs d'écoulement d'eau souterraine ont été effectués en régime permanent.

Pour l'évaluation des débits captés et des débits perdus sous le drain par un modèle mathématique, il a été choisi un domaine d'une longueur de 1000 m et une épaisseur de couche d'environ 12 m (figure 4). Ce domaine représente une coupe transversale du nouveau drain et il a été discritisé en réseau d'éléments finis ayant 330 noeuds et 290 éléments.

Pour le calage du modèle, le domaine étudié a été divisé en 2 zones de perméablités différentes, tenant compte des données hydrogéologique. Les perméabilités du terrain ont été attribuées à chaque élément selon la zone.

pour les éléments se trouvant dans la zone 1, $K_x = 300, K_y = 300 \text{ (m/j)}$

pour les éléments se trouvant dans la zone 2, $K_x = 625, K_y = 625 \text{ (m/j)}.$



Fig. 4. Coupe hydrogéologique de la rivière Moldova

3. Variantes analysées et résultats

Il a été pris en considération cinq variantes de simulation : une variante avec un drain parfait donc la distance d'ouverture entre le radier du drain et le lit imperméable (t = 0), en suite une variante pour t = 1 m et respectivement pour t = 2, 3, 4 m.

Dans la figure 5, il est présenté la variation du débit perdu sous le drain pour chaque variante de calcul, du cas d'un drain parfait jusqu'à une ouverture de t = 4 m.

Dans la figure 6, il est présenté la variation du débit capté du drain pour les cas de variante de drain parfait et une ouverture sous le drain de t = 1, 2, 3, 4 m

En continuité la figure 7 montre le pourcentage des débits perdus sous le drain en fonction de la

distance d'ouverture entre le radier du drain et le lit imperméable.



Fig. 5. Débits perdus sous le drain fonction de l'ouverture sous le drain.



Fig. 6. Débits captés par le drain fonction de l'ouverture sous le drain.



Fig. 7. Pourcentage des débits perdus sous le drain fonction de l'ouverture sous le drain.

4. Conclusion

De ces calculs, on remarque que le débit perdu sous le drain est proportionnel avec l'ouverture (t)entre le radier du drain et le lit imperméable, par contre le débit capté par le drain est inversement proportionnel avec l'ouverture (t).

Plus l'ouverture (t) augmente plus le débit perdu sous le drain augmente, et plus le débit capté par le drain diminue. On remarquant de la figure 7 que pour le cas de Timisesti les pertes sous le drain en général sont estimées à 5 % et dans certain secteur peuvent augmenter jusqu'à 12 % fonction du niveau de la couche imperméable.

Parmi les mesures adopte a fin de dimunie les pertes de debits sous le drain imparfait, on recomande la realisation d'un ecran d'etanchement.

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Results of A Few Researches in A Coastal Hydromorphological Polygon at Mangalia

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Abstract. In the aim of knowing of the coastal hydromorphological processes from the arrangement seaboard sectors with work protections of the beaches, through care of authors in the 1993-1995 years National Institute of Meteorology and Hydrology has organised a hydrological research program in a polygon constituted from two protection beach enclaves in front of Mangalia town perimeter.

It presents the hydrographical plan of the hydromorphological polygon with indicating the positions of the observation and measurements points and the research program in the two-hydrotehnical enclaves.

From the processing of accumulated data fond from nature researches have resulted a series of characteristics of the regime of the causative dynamic factors (wind, waves, currents and levels) from coastal zone and of their hydromorphological effects produced in inside and vicinity of hydrotehnical enclaves (waves, currents, sediments and their granulometry)

Results from Mangalia it aids to others obtained from north and central part of the Romanian coast at Black Sea, constituting elements, which completes the entry data in mathematical modelling of the hydromorphological coastal processes and hydraulic substantiation of the technical solutions of adopted for beach protection and conservation.

Key words: wind, levels, waves, currents, sediments, salinity, temperature and hydromorphological processes.

1. Introduction: working object.

Through the problems which requests at the projection and realising of the hydrotehnical arrangement for coastal protection and conservation, the knowing of the natural hydromorphological processes constitutes a main objective.

This main objective must to answer to expressing in adequate forms the interaction between the marine causative dynamical factors and the produced effects on the coast. Taking in consideration the characteristics of this interaction it can take the adjudication and to choose adequate hydraulic solutions for the realisation of coastal hydrotehnical arrangements with optimal effects. Of since up to now in activity of human community on the Globe this requirement was realised in a little cases. Neither on the world plan and neither on the

national plan were not observed distinct results of the hydrotehnical works realised in coastal protection and conservation aids. On the national plan stays as proof the thin results and disabled of a few hydrotehnical works in south part of the Romanian seaboard at Black Sea. A main cause of these results is the thin knowing of the coastal hydromorphological processes mechanism. In the more cases this inconvenient was due missing of the nature information, but and ignorance of the hydraulic of the hydromorphological processes. It is mentioned between others:

• The missing of the knowing of the digging up and manner raising in suspended state of waves of the sediment from marine bottom.

- The missing of an adequate mathematical model that to simulate the cross transport on the coast of the sediments.
- The missing of the knowing of the source provenance's of the marine sediments and of the participation potential to coastal morphological processes.

In this context of the knowing requirements of the coastal morphological processes from the Black Sea Romanian seaboard and on the base of the anterior own experience [4,5], authors have initiated in the frame of the research hydrological plan of the National Institute of Meteorology and Hydrology (NIMH) on the 1993-1995 years, the realization in a beach sector with hydrotehnical arrangements for protection and conservation of the beaches from the front of the town Mangalia of an hydro morphological program of observation and measurement.

With modest technical-material means and in spite of a logistic adversities of realization of the research program, it succeeds to accumulate an experimental data fond regarding interaction between marine dynamical factories and sediments in two twin beds coastal sectors with hydrotehnical arrangements.

In continuance are presented the experiment with the program and work method as the main results obtained from the primary processing of the hydromorphological observations and measurement performed in the two hydrotehnical arrangements.

2. Experiment: program, method and data fond.

Congruent with since that time of the technicalpossibilities NIMH, material of the hydromorphological experience from Mangalia first it has consisted from the perform of hydrographical plan in the rectangular system coordinate Gauss-Kruger (Fig.1 and 2) of the hydromorphological According polygon. to Figure 1 the hydromorphological polygon is situated in the north part of the port Mangalia and is composed from two basins A and B between in a system of three dikes with forked extremities (Fig.2). The length of the dikes is about 120 m with distances between them about 450 m in the basin A and 600 m in the basin B. The first basin A is quasi-closed towards at Sea with a parallel dike with shore situated on the line of the

extremities of perpendicular dikes. The embankment towards at Sea was protected with stabilopods. At the date when was performed the hydrographical plan of the two basins the all polygon was enveloped at Sea of isobath of 2.5 m depth. The research program in the hydromorphological polygon has provided the implementation of the following operations:

- Daily observations and measurements at three terms (hours 07, 13 and 19) on the waves, currents, water levels, air and water temperatures in the coastal point Mangalia situated at north dike root of the Mangalia avant-port (indicated in B5 from Fig.2).
- Daily observations and measurements morning at hour 07 on the surface currents in the points A1, A3, A4, A5, B2, B3, and B4 indicated through circles in Figure 2, A and B.
- Daily harvests of water samples at the three terms from the pints A4 and B3 in view of determination of the suspended sediments concentration and water salty.

Daily observations and measurements at three terms in coastal point Mangalia were performed of the hydrometrical observer in agree with working rules in vigor [3]. In the points A1-A5 and B1-B5 the daily observations and measurements at morning hour 07 were made by ing. Elena Dobrin.

The all data fond accumulated from observations and measurements was adequate processed by authors and is contents in tables 1-5. Below it is exposed the main results and interpretations obtained from studding respective tables.

3. Results and interpretations.

On the world plan are known the results of a lot coastal hydromorphological experiments performed in physical-geographical conditions total different of those on the Romanian coast. Too the conditions in which were performed the hydromorphological experiments on the Black Sea coasts of Russian, Ukraine and Bulgaria substantial differs of those of Romania. But and on the Romanian coast are physical-geographical inequalities between the north and the south part of the seaboard. Specific for the Romanian coast zone is the big influx of continental sediments transported of Danube river in Black Sea. According to local physical-geographical conditions this continental affluence of sediments is conducts towards south in the long of the Romanian coast under dominant actions of the marine dynamic factories (waves and currents) from north-west sector. A big influence in the coastal sediment transit in the long of Romanian littoral exerts the coast contours and the big coastal hydrotehnical arranges from zone. In those which follows will be exposed only the research results realized on the Romanian coast in hydromorphological polygon Mangalia in 1993-1995 years.



Fig.1. Plan of situation of coast zone Mangalia indicating with intermittent line the perimeter of location of the hydromorphological polygon.

3.1. Characteristic of the marine hydromorphlogical regime.

Because the hydromorphological processes from polygon were depended of marine hydrometeorological regime in coast zone Mangalia it presents a short character of this regime from research years 1993-1995.





positions of observation, measurements and water samples points.

• Winds (Tab.1).

In average the winds blows about 80 % from year with dominant directions from N, about 14% and V, about 27 %. The average velocity was of about 4.4 m/s with the maxim extreme of 17 m/s on the NE direction.

• Waves (Tab.2).

Generally waves regime has followed the winds regime. The average annual duration of the waves were about 75 %. The average height was about of 0.8 m with maxim extreme of 7.25 m. The average period was about of 4.5 s with extreme of 8 s. According to natural observations [2] the field assurance of the measurement wave height in coast point Mangalia was about 4.5 %.

• Currents.

Practically were permanently a marine water movement parallel with seaboard line. In average about 83 % from year the water movement was tended towards south with a surface velocity about 17 cm/s with maxim extreme elder of 40 cm/s. The north water movement was an average velocity about 2 cm/s with maxim extreme elder 12 cm/s.

• Levels.

Average of marine level at Constanta in research years was about 14 cm with mean monthly oscillations between 26 cm in May month 1995 and 7 cm in September month 1993.

• Suspended sediments.

The average of surface concentration of suspended sediments was about 98 g/m³ with average monthly oscillation between 120 and 70 g/m³. Comparative with average concentration of suspended sediments transported of Danube current the sediment load of water coastal current is enough of big. It results that the waves in coastal zone have semnificative effects of rising in suspense of the bottom sediments.

• Water salinity.

Average of surface water salinity was of 15.5 ‰ with average monthly oscillations between 19.3 ‰ in November and 10.6 ‰ in May. Extremes values of the salinity are in correspondence with sweetening influence of marine waters which exerts water sweet of Danube river.

• Water temperature.

Annual average of water temperature was of 10.8 0 C with average monthly oscillations between 20.9 0 C in august and 2.6 0 C in February.

3.2. Hydromorphological characteristics from hydrotehnical arrangements.

For hydromorphological characterisation from interior of hydrotehnical arranges it presents the results of the daily observations and measurements performed in indicated points with black circles in the arrangements A and B from Figure 2.

• Waves (Tab. 3).

From the three observed wave elements (period, length and height) in both hydrotehnical arrangements was observed that the average period of waves has had approaching values (4-5 seconds) in all observation points. This rapprochement of values it dues of fact that the mean period of the wave is the singly wave element which it conserves up to seaboard.

Big inequalies were observed between the measurement height of the waves from exterior and interior of the two arrangements. Average of the wave heights from exterior (A1, A2, B1 and B2) was about 1m and in interior points (A3, A4, A5, A6, B2, B3 and B4) this has had values between 0.32 and 0.77 m. It results that under the refraction and diffraction phenomena produced in the interior of the hydrotehnical arrangements the heights of the penetrated waves it strong decreases in function of the positions of the observation points and their depths.

Lengths of the waves were in tight dependence of the wave period and water depth in observed points, their magnitudes varying between 5.9 and 13.9 in the interior arrangement and 10.6 and 21 m in their exterior.

Currents (Tab. 4).

Presentation from Table 4 of the current data reports to their parallel components with seaboard line definitional as direction in report of the perpendicular to seaboard line in the observation points.

From the examination of Table 4 and of Figure 2 results diverse sense and velocities of the currents in observed points from the interior of hydrotehnical arrangements.

In the central part of the seaboard lines from both hydrotehnical arranges the water circulation was conducted towards south with an average velocity of 12.5 cm/s in point A4 and with 18.9 cm/s in pint B3.

In extreme corners of the arranges from north and from south the water circulation was conducted towards exterior at sea with average velocities between 3 and 7.5 cm/s.

• Suspended sediments (Tab. 5).

Vis a vis of coastal daily observation point from Mangalia (B) situated in exterior of the hydrotehnical arranges, in interior measurement points of the hydromorphological polygon were found more concentration of suspended loads.

Thus the mean concentration of suspended sediments in the points A4 and B3 were about 112 and respective 120 g/m³ vis a vis of 98 g/m³ in the point B5. The average of monthly values of the sediment concentration has varied between 87 and 160 g/m³, being more in months of spring and autumn.

• Sediment granules on the beach.

Sand of Mangalia beach has a mineral combined composition from quartz and shell with sizes between 2 and 0.08 mm (median diameter $d_{50\%}$ being of about 0.4 mm).

Comparing the recent data with those from past [2,5] it found a time modification of granule and mineral structure of the sand given in following table.

Weight percent (%) of particles less than diameter d (mm) and of the shells from Mangalia beach sand.

	Dia	neter a	(11111)		
1.600	0.315	0.250	0.125	d50%	CaO3(%)
100	78.0	60.0	3.00	0.216	60.0
95	41.6	17.9	2.00	0.368	-
89	3.45	2.90	0.14	0.640	100.0
	1.600 100 95 89	Dian 1.600 0.315 100 78.0 95 41.6 89 3.45	1.6000.3150.25010078.060.09541.617.9893.452.90	1.6000.3150.2500.12510078.060.03.009541.617.92.00893.452.900.14	1.600 0.315 0.250 0.125 d _{50%} 100 78.0 60.0 3.00 0.216 95 41.6 17.9 2.00 0.368 89 3.45 2.90 0.14 0.640

It results from table an increase in time of size of the particles of sand and a shell sand content due of the anthrop complex influences.

The main anthropic factor which has conducted to the process of time modification of the granules and mineral nature of sand beach is the decrease of affluence and coastal transit of continental sediments provided from Danube river influx. This decrease is due to so reduction of the Danube river influx sand how and obstruction of the coastal transit to south of the big marine port structure realized on the Romania coast in the last four decades. The marine ports Midia, Constanta and Mangalia constitutes barricades in the coastal transit way to south of the sediments. At the root and in the long of north dikes for protection of these ports it deposes big quantities of sand sediments which would must to transit to south for to participation to natural morphological balance of the beaches.

It adds to these negative processes the sediment continental influx at big depths of the Sulina canal mouth and the depose in the prolongation of the Sacalin island of a considerable quantity of continental sand sediments provided from Danube mouths.

4. Conclusions.

Without to consider that all data fund accumulated from hydromorphological measurements and observations in Mangalia polygon was complete used, from data processing have resulted a few important conclusions exposed in continuance.

4.1. Under direct action of winds and waves there is in the long of seaboard a coastal current with resultant direction to south.

4.2. The water coastal current transports an important sediment load in suspended state.

4.3. Under the influence of the waves which penetrates in the interior of coastal hydrotehnical arrangements of beach protection and conservation has place a specific circulation of the water mass loaded with suspended sediments.

4.4. The refraction and diffraction effects of the waves in the interior of the hydrotehnical arrangement conducts to decrease of the wave heights.

4.5. Under naturally and anthrop influences has place a modification process of the mineral structure and granule composition of the coastal marine sediments. It decreases the quartz sand

content of continental provenance and increases the shell sand of marine provenance. In the same time has place an increase of the particle size of the coastal sediments. **4.6.** For the profound knowing a mechanism of interaction between marine dynamical factories and coastal sediments it requests the continuance of assurance a adequate monitoring in the specific sectors from the long of the Romanian seaboard at Black Sea.

4.7. The Romania seaboard at Black Sea has enters in a accelerated process of the physical degradation due of the influences produced of the big hydrotehnical arranges realized in hydrographical basin of Danube and on the Romanian coast.

On the Romanian coasts the made works how are the dikes from Sulina mouth canal expanded at Sea up to isobaths of 7.2 m depth and maritime ports expanded at Sea up to isobaths of 11 m depth (Midia and Mangalia) and respective up to 23 m (Constanta) have produced a big changes of the coastal water and sediment circulation with negative irreversible hydromophological effects. These changes have conducted to generalization of the erosion processes

In the limits of the Danube Delta coast between the Musura bay and Midia Cap annual disappears in mean more 50 ha of dried seaboard. In these conditions the efforts for the completing with sand of the erosion beaches have considerable increased.

Too it observes a lot of deterioration hydrotehnical arranges for conservation and protection of the beaches produced due of the morphological deficit balance which was generalized on the Romanian coast.

4.8. The measures which must take for interception of erosion processes of seaboard requests the reconsideration of the look mode of the defense problems and conservation with hydrotehnical works of the marine coast.

• In the firs row it is necessary the raising of the knowing quality of the hydromorphological phenomena and their simulation through mathematical models.

- In the second row it is necessary the studying and the selection of technical solutions in diverse hydrometeorological scenarios and hydrotehnical arrangements.
- At the base of the projects elaboration must stay the hydromophological substantiation.

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Table 1	. Ave	age of	annua	al freq	uency	(%) of	E wir	nd dir	ections	on the	veloci	ties steps					
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13-15	0.047	0.183	3 -	(0.023	-	-	•	-	-	0.2	53					
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17-22	-	-	-	-	-	-	-	•	-	-	-						
22-28	-	-	-	-	-	-	-	•	-	-	-						
>28	-	-	-	-	-	-	-	•	-	-	-						
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.2550	-	.27	1.80	12.23	4.00	1.33	.30	-		-	19.93	72.75					
.5075	-	-	.53	8.00	9.47	4.50	1.97	/ _		-	24.47	52.82					
.75-1	-	-	-	.23	6.90	1.50	.33	-		-	8.96	28.35					
1.00-1.25	5.03	.03	-	.20	2.00	2.23	1.50) –		-	5.99	19.39					
1.25-1.5	-	-	-	-	.77	.63	.10	-		-	1.50	13.40					
1.50-1.75	5 -	-	-	-	1.70	2.00	1.13	.03		-	4.86	11.90					
1.75-2	-	-	-	-	2.77	-	.23	.03		-	3.03	7.04					
2.00-2.25	5 -	-	-	-	.40	.90	.80	.07		-	2.17	4.01					
2.25-2.5	-	-	-	-	.03	.03	-	.03		-	.09	1.84					
2.50-2.75	5 -	-	-	-	.03	.40	.43	.07		-	.93	1.75					
2.75-3	-	-	-	-	-	-	.03	-		-	.03	.82					
3.00-3.25	5 -	-	-	-	-	.30	.37	.03		-	.70	.79					
3.25-3.5	-	-	-	-	-	-	.03	-		-	.03	.09					
3.50-3.75	5 -	-	-	-	-	-	.03	-		-	.03	.06					
3.75-4	-	-	-	-	-	-	-	-		-	-	.03					
6.75-7	-	-	-	-	-	-	-	-		-	-	.03					
7.00-7.25	5 -	-	-	-	-	.03	-	-		-	.03	.03					
Total	.13	1.43	2.46	21.93	28.07	13.85	7.25	.26		-	75.3	8					
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	75.38	75.25	73.82	71.36	49.43	21.36	7.51	.26		-							
	Ca	11m=24	. 62	1													
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Annual average of the wanes periods tav=4.47 s.

Table 3. Characteristic values from 1993-1995 years of the wave elements in observation and measuremet points from hydromorphological polygon Mangalia

Points	Periode(s)	Lenght(m)	Height(m)	Depth
	Max. Ave. Min	Max. Ave. Min.	Max. Ave. Min	(m)
A1	7.40 5.07 2.60	27.0 138 32	1.50 0.77 0.20	0.8
A2	7.50 5.03 2.50	29.0 13.7 3.4	3.00 1.47 0.20	0.8
A3	6.00 4.60 2.60	20.0 11.2 6.0	- 0.37 -	0.8
A4	7.30 4.50 2.40	25.0 10.3 3.2	1.13 0.42 0,20	0.7
A5	5.50 3.37 2.50	14.7 5.9 2.7	1.20 0.33 0.20	0.5
A6	7.40 4.90 2.50	20.0 12.2 3.6	1.00 0.37 0.20	0.8
B1	7.50 4.90 2.30	30.0 10.6 2.2	3.00 1.17 0.30	0.8
B2	5.90 4.83 2.10	17.8 11.9 3.0	0.70 0.32 0.25	0.8
B3	7.50 5.17 2.60	27.5 13.9 2.6	1.80 0.77 0.25	0.8
B4	7.30 4.70 2.50	30.0 11.5 2.1	1.00 0.50 0.20	0.8
B5	8.00 4.47 1.00	43.0 21.0 2.0	7.25 0.84 0.20	1.7

 Table 4. Annual average of duration (%) and the average velocity (cm/s) of the parallel components with the seaboards line of the observed resultant surface currents from the hydromorphological Mangalia polygon (1993-1995 years)

Characteristics								
	A1	A3	A4	A5 -	B2	B3	B4	B5
Duration(%) Average	25.0	-36.0	24.2	34.4	-31.6	21.6	1.6	83.3
velocity(m/s)	7.1	-5.6	12.5	6.2	-7.5	18.9	3.0	17.5

Note: Positive and negative values regards the currents conducted to right and respective to left of the perpendicular to seaboard line in observed points.

Tabel 5. Monthly average values of the surface concentration (g/m^3) of suspended sediments in the Mangalia polygon in 1993-1995 years.

					An	nual n	nonths						
Points	1	2	3	4	5	6	7	8	9	10	11	12	Annual
B5	71	93	98	73	95	98	112	104	108	103	120	108	98.4
A4	87	110	114	89	108	111	117	127	122	125	122	116	112.3
B3	89	116	118	90	113	118	121	149	160	123	128	121	120.5

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The Safety of Offshore Structures

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Abstract: The crude oil exploitation equipment requires a special sustaining metallic structure. In Romania, a country with a long tradition in crude oil exploitation, the height of the drilling equipment rises from 40 to 100 m. In the case of offshore exploitations, the sustaining structure has to be built on a marine platform, which can be fixed or mobile. Although the use of steel or reinforced concrete structures was not a new problem in the structural computation, the offshore conditions involved on them were a challenge to most of those involved in structural analysis and design. An important component of the novelties in the field is the safety analysis due to the environmental conditions. The study presents some new aspects of the structural safety analysis.

Keywords: Offshore structures, stress spectrum, structural analysis.

1. Introduction

The structural safety problems concerning the design and building of offshore structures (Fig.1) are very complex taking into account a lot of aleatory variables (waves caused by wind and earthquake, execution faults, corrosion, fatigue effect, etc.) [1].

The ecological accidents from the marine environment have generated extremely strict internal and international standards to avoid new pollution of the World Ocean extended to over 70% of the globe surface.

In this context, to design and build the offshore structures from the Black Sea continental zone, the Romanian specialists did their best at the level of those years knowledge (1978-1990).

The severe exploitation conditions from the marine environment and also the wrong structural analysis approach were the source of some unwanted accidents - The Sleipner offshore platform from the Northern Sea [2], (Fig.2, 3), The steel offshore structure in the brasilian territorial waters, etc.

The present paper tries to bring a modest contribution regarding the structural analysis required by the safety approach of offshore structure [3].



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Fig.3

2. Some aspects of structural analysis

2.1. Computational models

Generally, actual modelling uses the Finite Element Method for the structural analysis. BEAM, SHELL, SOLID finite elements are the most used.

The modelling of bar connections is made using restraints and hinges (Fig.4).

Usually, the rag bolt modelling ignores the bolt length and his diameter and the bolt nut dimensions. Figure 6 presents a modelling scheme that compared with the experimental results emphasize correctly these aspects.

An extremely delicate problem turned up at lot of offshore structures with trussed cantilever type jacket: the fixing welding of leaning bars into the vertical bars presents cracks (Fig.7, a).



The theoretical and experimental analysis show the constraint sensible influence on stress status (Fig.5).

From the authors experience follows that the differences between the models that are not taking into account the superposition and the uncovered zones and the correctly performed modelling are 3-5% for bending moments and 6-8% for deflections. The explanation consist of the fact that bar 2-3 is double restrained, with a big axial force N that produces restraint moments with hyperbolic variation. These moments can be emphasized only using an extremely dense meshing and a higher order calculus (Fig.7b,c). A similar phenomenon produced the roof's crash of the "Kongresshalle" monumental building from Berlin in the eighties, 23 years after its construction.




Fig.6



2.2. Problems concerning the fatigue verification [4] ÷ [12]

The tension gaps are determined after an elastic analysis of the structure, usually from loads caused by waves. The waves action is a dynamic one, similar to the seismic action, but with an almost permanent character. The loads caused by waves are obtained from hydrodynamic analysis taking into account the Froude-Krylov components, the additional hydrodynamic mass, etc. The methodology used for fatigue verification are based on linear cumulating of damages, using different standards:

- DnV Det norske Veritas (Norway),
- GL German Llyod (Germany),
- ECCS European Steel Convention Standards,
- API American Petroleum Institute (USA).

The authors recommend the permanent monitoring of a number of vital joints for the structures and a permanent comparison with a Wohler curve, opening the possibility of an immediate action when the stress spectrum approaches the curve.

This procedure allows the measurement of corrosion contribution at offshore structures.

The sea-waterproof tensometric stamps (already in sale at reasonable prices) allow the monitoring without special difficulties.

The numerical calculus of stress spectrum is made using the "Reservoir" method.

3. Conclusions

- The offshore platform crashes caused by structural analysis faults, even when high performance programs were used, contribute to the warning of engineers in the way of the acceptance only of those results confirmed by a structural analysis, simplified but not simplistic.

- A recently tracked down situation shows that some parts of the most popular programs are not applicable to solve some particular cases.

- The tackling of fatigue problems has to be realized with permanent monitoring and comparison of stress spectrums with a synthetic Wohler curve.

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Sea Highway with Multiples Utilities

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Abstract: Developing the *multiple utilities idea* for the hydroenergetic arrangements, introduced in our country by the famous scientist, professor and engineer *Dorin Pavel*, one presents in this paper a special hydroenergetic building, placed in front of the Black Sea shore at the isobathes between 4 and 10 meters, which realizes the following utilities: captation of nonpolluted and inexhaustible wave energy which on the little Romanian littoral has the same size order of that of interior river potential energy, shore protection against the wave destructive action in the conditions of diminishing the Danube aluvionar transport as fact of the hydroenergetic dams and of dispositions to struggle the soil erosions, captation of nonpolluted and inexhaustible energy of wind by laying a row of wind turbine along the sea highway, high speed sea link by means of the slider ships on air cushion in this zone without waves between the littoral and the energy captation devices, terrestrial high speed transport on this sea highway parallel to the littoral, transport of containers, gaseous or liquid hydrocarbons by the pipe-lines mounted on the highway structure.

Keywords: Highway, wave energy, wind energy, shore protection, terrestrial, sea and hydrocarbons transport.

1. Introduction

Developing the *multiple utilities idea* for the hydroenergetic arrangements, introduced in our country by the famous scientist, professor and engineer *Dorin Pavel* [1], I present in this paper a special hydroenergetic building (Fig.1) placed in front of the Black Sea shore at the isobathes between 4 and 10 meters, which realize the following utilities:

- the captation of nonpolluted and inexhaustible wave energy, which on the little Romanian littoral of about 200 km has the same size order of that of interior river potential energy [2] - [6]/1/,

- the shore and beach protection against the wave destructive action [7] in the conditions of diminishing of Danube aluvionar transport, as fact of the hydroenergetic built dams and of the general dispositions to struggle the soil erosions /2/,

- the captation of nonpolluted and inexhaustible energy of wind, by laying a row of wind turbine along the sea highway [8] - [11]/1//2/,

- the high speed sea link by means of the slider ships on air cushion in this zone without waves, between the littoral and the energy convertors devices. - terrestrial high speed transport on this sea highway parallel to the litorral [6] /2/,

- in the end, the transportul of containers, gaseous or liquid hydrocarbons by the pipe-lines mounted on the highway construction /2/.

For such a project there are many beneficiaries in Romania as: Ministries of Industry and Trade, of Transport and Environmental Protection, of Health, Turism and Sports and in the foreign countries as: Ukraina, Moldova Republic, Bulgaria, Greece, Turkey and other West European countries.

2. On the advantageous utilities of the sea highway

The building of a highway along the Romanian sea shore should achieve in our meridian zone a multiple link between the North and South of Europa, favourable to the sustainable development of human society and with a reduced impact on the environment /2/.

The high speed transport of persons, goods and materials would solve the following problems:

2.1. elimination of terrestrial highway road transport difficulties, as:

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- energy consumption by crossing the mountains and hills,

- direction deviations, consumers of time by velocity diminishing,

- atmosphere polluation with toxic gases, aerosols and noises,

- risk increase of terrestrial road accidents

- the fact that a terrestrial highways represent an uncrossed barrier in the growth areal of the regional fauna.

2.2. *elimination of naval* (cheaper as the terrestrial) high speed transport difficulties, as:

- the immersed keel ships are very important consumers of time, due to their reduced going speed,

- extremely great energy consumption of immersed keel ships at high speeds,

2.3. *elimination of the cheaper transport* by pipe-line difficulties, for hydrocarbons, ground materials or containers, as:

- the level or direction differences may lead at the cloging of the pipes and their untimely wear,

- the non-corrosive coating of pipes or their burial in the earth,

- the possibilities to steal their content.

3. Protection of the sea shore, beach and biosphere reservation Danube Delta

The conservation of these natural presents have extremely good implications regarding the balnear therapy, the ecological tourism, the sports, the aquatic fauna and flora developing of a great economic value for this region.

All the Romanian litoral have been formed and maintained as a resultant between the alluvion material flow brought by the Danube and her tributary rivers (more wild or more quieted by the presence of hydroelectric power dams), corroborated with the works of torrent arrangements and soil protection to erosion, crumbling or land-slide and the activity of alluvionar transport along the coast under the wave empire driven from North-East by the energetic resultant of winds, prevailing in this region.

The research concerning the aluvionar transport on the Danube at the entrance in Romanian territory, undertaken beginning still 1840 year [3], shows a continuous diminishing of the aluvionar discharge, due to the above mentioned works, what make that in the present the placement of the expensive dams, transversal on the coast in the aim to keep the aluvionar material and to enlarge the beach surfaces, to be not efficient, the shore deterioration taking place on the average with about 1-2 meters each year.

For the conservation of these grounds, with a special role in the sustainable development of Romania, I propose not a mad struggle against the wave destructive energy [7], but her catchment and conversion in electricity, realizing in this kind the shore and beach protection with advantageous implications concerning the balneotherapy, tourism, sports, aquatic fauna and flora [4] [6] /2/.

At the same time, the sea highway realizes a high speed terrestrial and naval North-South link with a reduced consumption of fuel and diminished pollution, as well as a favourable transport of materials by pipe-line /2/.

4. Electrical clean energy production

The waves and wind are two nonpolluting and renewable energy sources, which can assure the sustainable development in Romania [6] /2/.

In our country and in the frame of the Society for Promotion of Renewable, Inexhaustible and New Energies – SPERIN, one had made a series of theoretical [9][16]/6//8/ and experimental research [10]-[15]/3/-/5//7//9/ concerning the wave and wind characteristics on the shore of Black Sea and the devices which can assure a conversion with good efficiency of these energies.

Also, by a lot of technical and scientifical contracts /presented down/ we have designed the building and the equipment for a Black Sea pilot station to convert with a good efficiency the wave static and dynamic energy. In this manner one assures with renewable and nonpolluting energies of wave and wind the Dobrudja and biosphere reservation Danube Delta, both these regions being poor in other energy sources and menaced by an intensive shore erosion, ensuring thus their sustainable development.

A very important two problems, which we are studying in the present, both theoretically and experimentally in the Laboratory of *New technologies of energy conversion* of the Politehnica University in Bucharest, are: the interaction between the wave and the oscillating flap valve as catcher under the influence of reflecting waves on its back, and the



optimum blade profile and its settling angle for a wind turbine.

Fig. 1 Cross section in the sea highway with multiple utilities

4. Concluzions

The building of a such sea highway is possible concerning the theoretical level of the present knowledgements as also from technical point of view, because the Benoto's installation for the pillar plantation is very efficient.

The linear character of the sea highway presents also an other important advantage, we can built it as we have the money and we obtain the new theoretical improvements and practical experiences.

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The Coast Engineering Problems on the Romanian Black Sea Littoral

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Abstract: At present, the entire Romanian shore of the Black Sea is intensely and continously damaged by sea erosion, on about 60-70 % of the shore length. The shoreline has moved onshore, on variable distances from an area to another, with values of up to tens meters, existing the risk to damage even the littoral bands. The erosion intensification is caused by the descentilibrium created by natural and artificial causes, between the quantity of available sediment in the littoral area, and the energy of waves and sea currents.

In the last 5 decades, more than 50 breakwaters for beach protection have been realized in the area between Mangalia and Midia Head, especially in the areas of touristic importance, refering to the development of the littoral cities. Analysing these works, conclusions can be derived in order to improve their efficiency. For the littoral protection it is necessary to elaborate a program for stopping the shore erosions.

Keywords: Danube Delta, Vama Veche, Midia Head, erosion

1. General presentation:

The Romanian littoral of the Black Sea extends from the delta of Chilia branch, at North, to the border with Bulgaria (Vama Veche) at South, having a length of about 240 Km.

At national level, the administration of the Romanian littoral is regulated by the Law of Environment Protection (nr. 137/1995), the Waters Law (nr. 107/1996) and the Law for the setting up of the "Danube Delta" Biosphere Reservation (nr. 82/1993).

From geomorphologic and genethic point of view two areas can be distinguished (Fig. 1):

- The area at North of Midia Head, afferent to the Danube Delta and the lagunar Complex Razim – Sinoe of 165 km length. This area is represented by the shore areas with low altitudes, overtaken by the sea waves at storms and are constituted of sand of Danube origine and of organogene material (shell fragments);

- The area at South of Midia Head, up to Vama Veche, of 75 km length, with subunit Midia Head - Singol Head and, respectively, Singol Head - Vama Veche, characterized by the existence of

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cliffs, broken off here and there by the littoral bands that separate the sea from the littoral lakes. The formation source of these littoral bands and of the beaches is prevalent organogene and in a smaller measure by the cliffs erosion, transport from offshore, erosion of the rock from the sea bottom and of the protection constructions.

2. Situation of the present morphologic evolution

At present, the entire Romanian shore of the Black Sea is intensely and continously damaged by sea erosion, on about 60-70 % of the shore length. In the area of the "Danube Delta" Biosphere Reservation in the last 35 years the beach lost over 2400 ha (about 80 ha/year), while the acummulations were of only 200 ha (about 7 ha/year). The shoreline has moved back on variable distances from an area to another, with values between 180m and 300m, the maximum value, in certain areas, being more than 400m. In some areas, where the littoral band width is smaller, the sea completely covers the shore. Sometimes breaches are

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formed, which connect with the littoral lakes, affecting this way the ecosystem specific to the respective lake (Fig.2).

Between Sulina and Sf. Gheorghe branches, the shoreline has moved back on certain areas with rates of more than 20 m/year, the shoreline of Sahalin Island with about 10-15 m/year, in the Ciotica-Perişor area with 5 - 10 m/year, while in the Periteasca-Portița-Periboina area with 5 - 20 m/year.

In the Midia-Vama Veche area, the mobility of the shoreline records a different evolution due to the submarin calacarous platform. Consequently, the



Fig.1. Erosions on the Romanian littoral

destructive effects are smaller, but they have an irreversible character. That's why protection works have been realised in order to stabilize certain shoreline areas. Other areas are already in danger: in the last 20 years, the Eforie beach, between Belona Lake and the sea, decreased with about 40m, the Northern side of Neptun beach with 24m, while Venus-Saturn beach with 36 m.



LIGHTHOUSE SULINA Fig. 2. Littoral beach erozion a-Sulina branch south area; b-Perisor area; c-chituc area; d-Mamaia area; e-Eforie area; f-Neptun area; g-Sulina and Sf.Gheorghe area;

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At Mamaia beach, because of the lack of sedimentary material brought by the sea, an intense erosion process has developed, leading to the decrease of beach width with about 15-70 m. In oder to attenuate this phenomenon, works of breakwaters and sand nourishment have been

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1871

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LANDMARKS AND DISTANCES BETWEEN THEM

1

DISTANCES UP TO WATER LINE MEASURED AT LANDMARKS

ARM

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achieved. As an example, in the Park Hotel area, between 1975-1978, urgent measures have been taken in order to protect the touristic arrangements.

ARM

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1871 1910

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1957 1962

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LEGEND

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The sea action causes important slidings and damages of the cliffs, being estimate that the shoreline will move onshore with about 0.5 m/year.



Fig. 3. Danube Delta 1881/2002

3. The erosion causes

374

The negative evolution of the Romanian littoral, because of erosion intensification, is caused by the desechilibrium created by natural and artificial causes, between the quantity of available sediment in the littoral area, and the energy of waves and sea currents.

- modifiactions produced in the Danube Delta in the last century, which have changed the conditions of alluvia transport towards the sea and especially the rectification of Sulina branch in 1861 and its dredging in order to assure the navigable depths (Fig. 3). Two breakwaters to protect the channel at the outlet in the sea have been realised, reaching gradually a length of 7.5 km. This way, the alluvia are discharged offshore and cannot return totally in the littoral circuit. On the other hand, the breakwaters stop the circulation towards South of the alluvia transported on Chilia branch and on the other rivers from North. (Fig. 4); - discharge of the material dredged in order to maintain the navigable depths at Sulina bar, at about 2-3 maritime miles offshore, at depths where the waves and the currents cannot distribute the sand along the shore;

- the protection breakwaters of Midia, Constantza and Mangalia harbors, realised in the last decades, which intercept and direct offshore the already reduced alluvia flow, transported by currents and waves alongshore. The erosions increase, especially in the areas at south of the ports (Fig. 1);

- utilization of beach sand as construction material.

This lack of equilibrium was caused mainly by the human activities and especially by the :

- decrease of the overall supply with alluvia from the Danube, with about 20 %, due to the works against erosion, to the dams and to the flows taking-over on the main way and on the affluents;





Fig. 4 Evolution of the 2.0m izobat in Baia Musura

The sea erosion processes are also encouraged by the climate global changes, by the modifications of the sea level and by the intensification of the sea total energy (waves and currents) (Fig. 5).



4. Protection works already achieved

In the last 50 years, a series of beach protection breakwaters have been realized, especially of touristic importance, refering to the development of the littoral cities. Between Mangalia and Capul Midia more than 50 such works have been constructed, based on many site and laboratory studies, with that time knowledges (Fig. 6).

In the complex context of the natural factors, these works haven't had the expected results, being necessary to complete them with additional works. In other situations, well located works do not benefit by the natural sand sourse and it is necessary to supply the beach with sand. Due to the oldness of some breakwaters, or to an inadequate execution, these works are now in different stages of damage. In the area from Midia Head, a series of offshore breakwaters have been realized in order to protect the littoral band.

Also, in Constantza area, longitudinal offshore breakwaters have been rezlized to protect the city cliff base.

All these breakwaters need the repair and completion works in order to increase their efficiency.



Fig. 6. Beach protection works

5. Conclusions

The problem regarding the damaging processes by shore erossion is considered by many states as a problem of national importance. The beaches erossion leads to losses of territory, but especially compromises the littoral ecosystems and the social-economic value of the touristic coast area, generating important damages to the national economy and a great loss for the future generations.

It is necessary to elaborate a well coordinated and scientifically based unique plan of research, design and execution of a protection and rehabilitation system for the Black Sea Romanian littoral. Any delay in the approach of these problems will lead to extremly serious problems, which will require increased financial efforts.

In this aim, it is necessary to promote a Law regarding the "Integral management of the coast area ", based on which a "National program of protection and integrated management of the coast area".

Till the elaboration of the above-mentioned law, it is firstly necessary that a "Program for fighting against littoral erosion" to be elaborated and approved by the Government Decision. This program will have the following objectives:

- estimation of the evolution tendencies, on geomorphologic components, of the littoral area, causes quantification and risk estimation;

- identification and substantiation of the design norms, as well as of the measures, methods and technologies of shore protection and rehabilitation;

- execution of the litoral area rehabilitation works;

- assurance of the legislative frame for coast area protection and integral rehabilitation, including the mechanisms to finance the littoral area management activities.

Solution for Buildings' Foundation on Lands Reclaimed from the Sea

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Abstract: Rezumat: Teritoriul portului Constanta Sud a fost câștigat asupra marii, prin umpluturi provenite, in principal, din excavațiile canalului Dunăre – Marea Neagra.

Studiile geotehnice au arătat ca umplutura poate fi caracterizata ca neomogena, constând dintr-o masa de materiale coezive cu incluziuni grosiere de diferite dimensiuni.

La stabilirea soluției constructive s-a ținut seama de mărimea solicitărilor si cerințele impuse de sistemul constructiv si tehnologic, utilajele de manipulare și, precum and de caracteristicile fizice ale terenului de fundare. S-au folosit doua soluții de fundare : fundare indirecta cu radier pe coloane și fundare directa pe teren îmbunătățit prin realizarea unei compactări dinamice.

Abstract: South Constantza Port territory has been reclaimed from the sea, by fillings proceeded mainly from the excavations of Danube – Black Sea Canal.

Geotechnical studies showed that the filling could be characterized as inhomogeneous, consisting in a mass of cohesive materials with coarse inclusions of different sizes.

To establish the constructive solution, the load magnitudes and the requirements imposed by the technological and constructive system, the handling equipment, as well as the physical characteristics of the foundation ground, have been taken into account.

Two foundation solutions have been used: Indirect foundation, raft foundation on piles and Direct foundation on improved ground by dynamical compaction

Keywords: reclaimed territory, indirect foundation, direct foundation.

1. Introduction

Constantza Port will develop as an important cargo storage and distribution centre, inclusive of transit cargo. Grain and container traffic represent a main component of the port activity. For this purpose, a grain terminal with 10 cells of 10,000 t capacity each and a container terminal with a storage surface of 2.5 ha have been built in port's area. Solutions of indirect and direct foundation, adapted to the geological and technological conditions, have been applied for these terminals.

2. Geotechnical conditions on site

The grain terminal was built on Pier I South following a modern concept, which would provide

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best flexibility to the handling and transfer operations. The terminal has a storage capacity of 100.000 tones and it has river, railway, road and sea transport facilities.

The container terminal was built in Enclosure 1B of Constantza Free Zone, on the land obtained by unifying plots "P" and "S". As shown in Figure 1, it is located between the southern breakwater and Pier II South. It has a surface of 2.5 ha and is provided with a platform for containers storage, a maintenance and repair workshop, an administration building, a transformer point, lighting poles and a rail track for containers' loading / unloading in / from rail cars.

The location of Pier IS and of the entire operative area of Constantza South Port was reclaimed from sea. In such areas, the sea used to be 11-12 m deep.

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Fig. 1 Location of the two terminals

The sea bottom consisted of a layer of plastically soft blackish mud, grey sand, brown clay or silt, plastically consistent to soft. This formation is 1.60-6.50 m thick. It is followed by a layer of reddish-brown, plastically stiff, residual clays resulted from limestone's alteration, which is 0.90-4.30 m thick. The bedrock consists of white Sarmatian limestone, with a relief descending from level -16.0 to -26.0, and in the southern part, the level down to which geotechnical bore-holes were executed is under -40 mMN.

Over these natural formations, fillings were executed in pier's body up to level +2.50 m and, in port's southern part, up to level +5.00 - +6.00 m. Such fillings were achieved by dumping and dynamic compaction, which influenced only the 5-6 m thick surface layer.

Most of the earth fills have been executed during the last 15 - 20 years, by placing materials preponderantly underwater, in enclosures bordered by quays and dykes. Therefore, the fillings have hoop outlines, and their side deformations are practically limited.

The clayey nature of the filling materials, the type of such clays, which are active in relation with water, capable of swelling, with low permeability, and which diminish while they are consolidated, are the key elements controlling the deformation process and its development in time.

Fills' placement using different technologies has resulted in a differentiation of their compaction state in relation with the depth, most of them being still unconsolidated, as it was revealed by their consistency state and the laboratory and in situ tests.

As the surface layer of the fillings was placed from the land and then it was compacted, it features a high compaction state, which is reflected by specific geotechnical indexes and which range them among the super-consolidated soils. Generally, the natural ground materials (sea bottom) placed on the degraded or intact limestone layer are saturated clays, with low permeability, and under-consolidated.

The presence of some interlayers of permeable materials (sand, gravel) in certain areas of fills' body could facilitate the drainage of some clayey packages and accelerate their distortion process.

The above, as well as other specific aspects have been revealed by complex geotechnical and laboratory studies. Laboratory tests' diagram is shown in the figure below, Fig. 2.:



Fig. 2. Laboratory tests on undisturbed samples

These findings, which were also assessed by previous studies, confirm and explain the in situ situation of pier IS and the other areas of Constantza Port that were similarly executed. During operation, in time (10 - 20 years), settling movements up to 20 - 30 cm have been noted there.

On the other hand, the edometric modules obtained in lab on clayey materials with some inclusions (concretions, sand, gravel) range these soils among the very or highly compressible ones. Therefore, calculations relying on such values would lead to the estimation of certain important settlings, under their own weights, as well as under additional loads.

There should be mentioned that the presence in the clayey mass, in different proportions, of some fragments of rocks, limestone boulders, coarse elements characterised by high deformation modules which classify them as practically incompressible materials, modify the compressibility characteristics, educing its tendency to deformation.

The geotechnical studies have shown that the filling can be characterised as heterogeneous, consisting of a mass of cohesive materials with coarse inclusions of various sizes. The common characteristics of these fillings are:

• the loose state of the clayey materials, in relation with the time passed since their placement and their present geological load (the super-

consolidation ratio of these materials decreases with the depth, down to values of approx. 0.4)

• the high compressibility of the clayey materials.

When assessing the constructive solution for foundation in these grounds reclaimed from sea, which are described above, the extent of the loads and the requirements imposed by the constructive and technological system, the handling plants as well as the physical characteristics of the foundation ground should be considered.

Two foundation solutions have been employed:

 \succ indirect foundation with raft placed on columns;

 \succ direct foundation on ground improved by compaction.

3. Grain terminal on pier IS

3.1. Terminal's structure

This terminal has the following main sectors (Fig. 3):



Fig. 3. Grain terminal of 100.000 t on Pier 1S. General layout

- receiving system on water, railway and road, including unloading, transfer and drying plants, vats and large underground tunnels, rolling tracks, trestle bridges, etc.;

- storage capacities consisting of 10 metal cells of 14,000 m³ each, as well as other smaller cells for wet grains;

- shipping system, consisting of conveyers, head frames, elevators and continuous operation plants for loading sea ships with capacities up to 65.000 tdw;

- related works, including the technicaladministrative buildings sheltering the automatic control system of the entire terminal activity.

Terminal's main component is represented by the 10 storage cells having a diameter of 27.43 m, a height of 27.1 m and a maximal total weight of 12.000 t.

3.2. In situ tests

Beside the ground and laboratory surveys, two categories of in situ tests have been performed:

- checking the behaviour of the port platform by overloading it with 100 KPa and 150 KPa on a circular surface having a diameter of 20 m 45 sq.m respectively;

- checking the bearing capacity of a drilled column having $1.08\ m$ on diameter an $30\ m$ on length

For testing the column, there was applied the solution where the vertical force was taken over by 8 radial tie rods and was transmitted to the column by the reinforced concrete head on which the presses were placed (Fig. 4). Each tie rod consisted of bundles of 24 wires, ϕ 7 mm, with a length of 52.0 m.

The bearing capacity calculated on the columns was of 420 t for those placed in clay, with diameters of 1.08 m and lengths of 30 m, 940 t for columns of the same diameter but supported on limestone and 920 t for columns placed in clay, with diameters of 1.50 m and lengths of 38 m.



Test column.

CAPITEL DE INCARCARE







b Fig. 4. Terminal of 100.000t

3.3. Foundation solution

Considering the stresses and the requirements of the constructive and technological system in relation with settlements, the adopted solution was a general raft founded on drilled columns. The raft is circular, having a diameter of 28 m and a thickness of 1.1 m, having a stiffening girder on its outline. On cells' similar axis, there is a conveyer tunnel, which is 3.5 m high and 3.2 m wide.

Two types of plants were utilised for columns' execution. Such plants perform drilling diameters of 1.08 m and 1.50 m. columns' number and distribution under the rafts were determined in relation with these diameters and the type of drilled pile: bearing on the top or floating.

Hence, three types of rafts have resulted (Fig. 5):

- 1^{st} type, 4 rafts including 26 columns ϕ 1.08 m, with lengths of approx. 20 m, supported on limestone;

- 2^{nd} type, 3 rafts including 38 floating columns ϕ 1.08 m, with lengths of 38 m;

- 3^{rd} type, 3 rafts including 20 floating columns ϕ 1.50 m, with lengths of 38 m.



Fig. 5.a. Terminal of 100.000 t. Longitudinal section



Fig. 5. b. Terminal of 100.000 t. Cross section

For constructive and technological reasons, rafts' geometry has been maintained, with the type of reinforcement varying in relation with the foundation system. The way of columns' layout resulted from a detailed study referring to strains' state in the raft, which also aimed to assure a uniform stress on the columns, to optimise the solution on the whole by considering the effect of the raft, columns and ground working together.



Fig. 6 Raft of a storage cell

For founding the silo cells, 278 columns have been drilled. By adding the columns necessary for other constructions, the total number of columns is of approx. 300.

4. Container terminal inside the 1B enclosure

4.1. Terminal's structure

For activity's development, the following works have been provided:

• Front for containers unloading / loading from / on the ship;

• Storage platform for full and empty containers and for refrigerated containers;

• Railway connection and loading-unloading track for containers coming or leaving on railway;

• Administrative building and additional buildings;

• Repair workshop for containers and platform plants, and warehouse for checking containers' cargo;

• Transformer point and distribution networks, and lighting installation on the storage platform;

• Drinking and fire-fighting water supply networks, Domestic water sewage networks; Rainwater sewage network;

• Fuel supply station for the platform plants;

4.2. Foundation solution

One main element of the Terminal is the storage platform itself. It was designed and sized so as to allow containers' storage on 3 levels.

Hence, the platform has a foundation consisting of natural aggregates and crushed stone, with a total thickness of 80 cm, reinforced with 2(two - lower and upper) geogrid nets of the SECUGRID type and a 12 cm thick cover of concrete blocks (see fig. 7). The foundation ground was improved by roller compaction. The superstructure consisting of small concrete interlocking blocks will be mounted on a 10 cm thick sand layer.

Platform's vertical planning was achieved by taking into consideration the present benchmarks of the ground on the platform, the possibility of gravitational discharge of rainwater, the existing works near the platform, etc

As terminal's foundation site is located on a land reclaimed from sea by means of fillings, it is necessary to improve the geotechnical characteristics of such land. For improving the foundation ground, it was compacted by a roller of 10-12 t. Before compaction, the land surface was stripped on a depth of approx. 15-20 cm., then the soil was excavated on an average depth of 1.0 - 1.5 m.

As mentioned above, platform's foundation consists of the following layers: (from bottom, upwards):

- geogrid net of the SECUGRID type;
- ballast layer, 40 cm thick after compaction;

- geogrid net of the SECUGRID type;
- crushed stone layer, 40 cm thick after compaction;
- geotextile material of the SECUTEX type;
- sand layer, 10 cm thick.

The purpose of geogrid's utilisation was to reduce the general, but particularly the differential settlements of the foundation ground, when it is unevenly loaded. Considering the specific settlements' values, the foundations should transmit to the ground the lowest pressures possible, and the active area should frame within the reduced settling layers, by placing the foundation level as high as possible.

Also, due to geogrids' utilisation, the foundation thickness decreased from 2.00 m to 0.80 m, resulting in important savings and and shortening of the completion time.



Fig. 7 Container terminal general layout

The geogrids acting as reinforcements for platform's foundation should meet the following requirements:

• Description: Bi-strained geogrids, made from colourless, monolithic polyester straps with welded knots;

- Supply sizes; rolls of 100m;
- o Material : polyester PET

 $\circ~$ Tensile breaking strength, as per SR EN ISO 10319 : 40 kN/m ;

 \circ Breaking elongation, as per SR EN ISO 10319 : <8% ;

 $\circ~2\%$ elongation strength, as per SR EN ISO 10319 : >15 kN/m ;

 $_{\odot}$ 5% elongation strength, as per SR EN ISO 10319 : >30 kN/m .

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The geosynthetic material, the geotextiles, act as an separation element between platform's foundation and its pavement.

It should meet the following requirements:

o Minimum surface mass, as per STAS 6142-73 : 250 g/mp;

• Material thickness, as per STAS 6139-86 : 2.5 mm;

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- Material type: unwoven fabric;
- Fabrication way: mechanical;
- Tensile strangth, as per SR EN ISO 10319 :

10 kN/m longitudinally and 15 kN/m transversally;



Fig 8 Current cross-section through the platform

• Breaking elongation, as per SR EN ISO 10319 : 60% longitudinally and 40% transversally;

- Punching strength, as per C 227/88:2780 N;
- Punching deformation, as per C 227/88 : 35%;
- Pores diameter, as per C 227/88 : 0,09 mm ;

 \circ Normal permeability on plan, as per C 227/88 : 2.9x10-3 m/s ;

 \circ Permeability in plan, as per C 227/88 : 1,7x10-2 m/s ;

5. Conclusions

The adoption of optimised foundation solutions for the grain silos and the container terminals of Constantza Port required a thorough analysis of the geological conditions, the technological, formation and operating solutions, the execution modes as well as the performance of calculations that would consider the structure, foundation system and the ground working together.

The proficiency acquired by the completion of these works is further employed for achieving new terminals in the sea ports, for traffic of containerised, or solid and liquid bulk cargo.

Tehnical Solution for Marine Erosion Control

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Abstract: All the countries that are conterminous with seas or oceans are confronted with erosion phenomena caused especially by the sea's movement as waves. The Romanian Black Sea shore is no exception. The costal erosion progresses year by year, the sea-fronts collapse over wide areas, the beaches become narrower, while in certain areas they have disappeared all together, and the buildings that are very close to the sea are loosing their stability, as they are aggressed at their foundation.

Being concerned with this issue AQUAPROIECT has reached the conclusion that the radical solution to control marine erosion is represented by dikes, which absorb wave energy, and not with those which disperse it.

Keywords: Marine erosion, coastal protection, floating dike.

1. Introduction

A program of pursuing the seashore erosions and costal constructions aggressed at foundation was carried out in Romania between 1980-1990. The efforts made then for stopping or at least mitigating this undesired phenomenon proved to be inefficient.

After 1990, the Institute of Research and Design for Water Management (I.C.P.G.A.) and later, its continuer, AQUAPROIECT S.A., resumed this issue with the support of the Ministry of Research and Technology through a more careful and better orchestrated monitoring of the seashore processes.

Having ascertained the absence of certain data to reflect the way in which the mechanical action of the waves destroy the shore during sea storms, if has been deemed necessary to achieve first of all an automatic wave parameter measurement station capable to operate continuously, day and night, over a period of several years. Only by knowing the mounting sea velocity, wave power intensity on each linear meter, action time, the destructions they induce through shore or land beneath base of structure erosion, can the phenomenon be understood. This is how AQUAPROIECT Station in Mangalia was set up.

There are years in which the shore storm are seldom and sometimes of low intensity. Other times,

they are violent bringing about extensive damages. That is way, the period of following up the sea in a certain location has to last several years, so as not to have the surprise that dimensioning a structure at the waves recorded over a short period of time, the respective structure might be destroyed the next year. The minimum observation period for the Black Sea is considered of 7 years, which does not mean that a maximum wave, measured in this time interval cannot be extended in the next years. But this likelihood is lower. The longer the measuring period, the greater the structure safety based on in-field information.

The local factor plays a special role as well. The values measured in a certain spot cannot be extrapolated over a too longer distance. That is way more wave parameter measuring stations are needed and they should be located at maximum 40-50 km distance one from the others.

Data centralization even during their acquisition may offer a global picture of the whole zone undergoing monitoring.

Having sufficient data related to the cause parameter principle that in the case of marine erosion is represented by the specific wave power,

the designer will know the maximum stress needed to dimension the future costal protection structure.

The modern technique offers nowadays extremely good performances in tracking and measuring the parameters of a phenomenon.

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AQUAPROIECT Station in Mangalia, through the use of an American analogical board is capable to obtain 50,000 pieces of information/second. By means of latest generation electronic computers, these automatically processed data provide the beneficiaries with the safety and precision they require.

2. New marine shore protection concept

In the last phases of preparing II World War, more exactly for the Normandy landing, floating dikes were used in order to achieve an artificial shelter for the ships that were to transport troops and combat material.

In the first landing moments, the floating dikes met the aim for which they had been built. However, in a short while there was a storm in the English Channel and the mooring elements underwent tremendous efforts which maintained fixed the caissons. These mooring elements breaking, left drifting great parts of the structure which caused a lot of damages, hitting the barges full with troops. As part of the caissons were sinking, the remaining ones were sunk by cannons so as not to endanger the military operation that had been started.

The conclusion drawn form this failure was that the real difficulty of the floating dikes lies in caisson reinforcement. Following thorough studies performed in Great Britain after the war it was ascertained that the width of the structure can reduce mooring efforts.

From the tests carried out on models whose width varied between 1m and 10m, it resulted that the performance is sensitive to the relation between the sea wave length and opposing solid surface.

The first turning to good account of these new studies was done when designing a wooden pontoon connected to the shore, at a shipyard servicing several platforms meant for the oil fields in the North Sea. Glasgow tourist harbor represented another touring to good account of the new studies [1].

It is worth mentioning that this harbor was achieved on floating caissons made of prestressed concrete, filled with polystyrene. The caissons were tested for behaviour on scale models during 4 billion cycles tests at highest stress, especially at skewness. In October 1977 a storm blowing from critical direction caused breakages in the dike line, but with no disturbing consequences for the sheltered ships.

It is to be noticed that between 1960-1980 when England was doing such experiments, there were a great number of new solutions in the world for protecting the marine shore [2].

Thus, the Land harbor in the British Columbia was designed as a floating breakwater, which in cross section looks like letter A (fig.1)



Fig. 1 Land Solution-1. floating; 2. melting wooden board

In the main, the dikes are up of treated wooden frameworks propped at both extremities on 0.97m diameter cylindrical floats. The screen by which the dike effect is brought about is made up of moisture-proof treated wooden plates, located in the middle of the framework, along the whole dike length. The frameworks have a 7.62m opening and 1.20m height. The screen below the floating level penetrates 4.29m into the water. Consequently, the floating barriers opposing the wave measure 5.49m on the vertical line. The floats are metallic and have polystyrene caps. The dike consists of chain – connected sections, while used tires were utilized as shock absorption pieces.

This type of dike was used at depths between 15 to 21m. Anchoring was performed by means of chains tied to bulky plain concrete blocks propped on the basin bottom. Each section is provided with 4 anchors, two in front and two at the back.

Another floating dike design is known as "Alaska Solution". This solution was tested in the following locations: Teneyki (1972), Uittier (1972), Sitka (1973). It consists of a floating barrier made up of frames lying on water.

The floating support has on the horizontal alignment the form of a rectangular frame with a crossmember in the middle (Fig. 2)



Fig. 2 Alaska Solution 1. neoprene fitting; 2. tirants; 3. wooden board; 4 light conrete; 5 polystirene

The horizontal values are poor: frame length is of 18.29m while is of 6.4m (which is the dike width as well). The floating element is of expanded polystyrene, arranged both perimetrically and in cross-cut, on a 1.32m thickness and a 0m91m width.

The resistance structure is made of reinforced concrete. In fact, $4.6m \ge 5.0m \ge 0.9m$ parallelepiped elements with polystyrene fill were achieved, which were then grouped and fixed by means of tie-nods.

On site, the sections are closely connected one to the other with chains. A rectangular rubber plate is placed between the sections. A wooden packing was provided between the rectangular rubber and the concrete element. The reinforced concrete wall which protects the polystyrene is 10cm thick. Consequently the total height of the floating element is of 1.52m, but of which 41cm remain above the floating plane. In front and in the back there are treated wooden plates.

This type of floating dike was used at depths of about 9m. The dike is anchored with chains to concrete blocks propped on the sea bottom. Four anchors were provided for each section.

Another floating dike design is the one known as the "Friday – Prot Solution".

In the main, this is a structure consisting of two parts, an upper one and lower one. The upper part is made up longitudinally and transversally arranged wooden beams, while the lower part is represented by floats. Each float is $3m \ge 1.5m \ge 1.5m$ and it is made

of polyolefine. The upper part is propped on 4 rows of floats, joined together two by two (Fig. 3).



Fig. 3. Friday - Port Solution 1 1. pontoon catching element; **2.** outside longitudinal bar; **3**. deck; **4**. inside longitudinal bar

The work is made up of section. The sections can describe different shapes on the horizontal line. In the case of Friday – Port, the shape is similar to letter "L", the short side making the connection with the shore.

The location depth varied between 9 and 13.3m. The anchors were provided for from 15 to 15m, the link being achieved each on their depth made up of 3 pieces (with chains at extremities, while between them having nylon cables). Their pinning in the lower part was carried out by means of piling on the bottom of the sea.

Another floating dike solution which gave very good result was used at protecting Orchard port (in the State of Washington).

The solution consist in achieving 6.4m long, 3.66m wide and 0.91m tall caissons made of a reinforced concrete shell that covers a Styrofoam nucleus. This nucleus ensures the floating of the whole structure (Fig. 4).



Fig. 4 Port - Ochard Solution 1 1. light concrete; 2. Styrofoam nucleus; 3. edge bar

The reinforced concrete walls of the floating cells had thicknesses that varied between 8cm and 15cm.

New other floating dike solution may by also be mentioned:

Embarcadero Solution, Holmes – Port, Thomales – Bay or Wave labyrinth, and so on.

3. Aquaproiect solution for marine erosion control

By analyzing the poor effect of the classical solutions applied till now of protecting the Romanian Black Sea shore and taking into consideration that the sea bottom in Romania is unstable up to considerable distances form the sea coast, it has deemed necessary to change the old protection conception based on massive, gravitational structures and to adopt a floating solution.

This has to :

1. Hold back the waves out at sea by means of structures placed somewhat in parallel with the shore;

2. Direct the affluent alluvia behind the retaining front.

If it is to be taken into account the confining requirement which the Bosphorus strait imposes and if we wish that any ship passing from the Mediterranean Sea, through the Marmora Sea into the Black Sea would be able to enter the Romanian ports too, it is necessary first of all to ensure the minimum depth of the Bosphorus and of the Romanian harbor enclosures.

The proposed structure being a floating one the regeneration of the water in the proximity of the seashore will not be hindered and thus the balneary treatment will not have to suffer.

From the tourist point of view the great distance from the shore of the protection structure will not pollute the natural landscape (since these structures will not even be seen with the naked eye from the beach). Moreover, the tourist activity will be much extended on the water at the shelter of the floating dike. This water surface can be used for bathing, swimming, nautical sports and so on, during the whole season, without interruption caused by storm waves. It will be possible to build or to set-up floating hotels and restaurants, offshore sports bases, artificial islands on this same location (even in the dikes).

As a first conclusion, it should be mentioned that Romania favours the seashore protection device to be made up of floating caissons very well connected, suitably ballasted and anchored out at sea towards the 27 isobathic contour line.

The floating caisson becomes thus the main component part of the protection dike.

The hydraulic structure called floating dike can be achieved in practice by aligning out at sea identical caissons made of water-repellent reinforced concrete.

Since the caisson is placed in the dike along the direction of the dominant wave propagations, the direction generally coinciding perpendicularly to the protection line, the caisson length becomes the dike width.

A value can be adopted for the width of the dike which should take into account the experience in the field of other countries, on the one hand (particularly the tests carried out by the United Kingdom), and, on the other hand, the correlation with the landscape, called for by the aim pursued, which is to take over that part of the wave energy that is especially of concern.

The caisson being however a floating one, it should have a stability of its own, besides the group stability.

This requires that its properties should be observed according to the navigation rules.

4. Energy Capturing Floating Dike

Since the main cause of marine erosion is represented by the energy-carrying waves, the question has been raised whether this energy could not be captured out at sea before it acts against the shore, thus it being taken out of the system. Romania was the first country that approached this technical issue, making it known within several scientific events, obtaining therefore the first patent as well [3].

In the new approach to the marine erosion control solution, the floating dike meant for coastal protection will have to be designed so as to be able to equipped with hydraulic machines for wave energy capturing.

Studies conducted by AQUAPROIECT specialists together with the university staff in the Technical University of Civil Engineering of Bucharest, have drawn the conclusion that for the irregular waves, such as those in the Black Sea area it is possible to try the solution with float with flywheels and rotary generator [4]

The principle lying at the basis of this equipment has received the name "Captive Floating Principle" because the wave raises a "free" float and when the latter reaches the highest travel point, a one-way device holds it back, therefore it no longer descends on the wave, but, as it is uncovered by water, it operates by its own weight, setting into operation the main flywheel by means of a mechanism.

The basic diagram of this hydraulic machine is given it in fig.5

The tests carried out in the Hydraulic Lab of the Technical University of Civil Engineering of Bucharest, pointed out a 31% hydraulic capturing efficiency. This efficiency was confirmed also by the Hydropower Studies and Design Institute (I.S.P.H.) through its own applied hydraulics laboratory [5].

In case the caission, the main component part of the floating dike is designed to meet the sea energy capturing function as well, it can be developed as a real wave micro-hydroelectric power plant.

Fig. 6 presents the draft of such an energy capturing floating dike, featuring a hotel with a restaurant built on its upper part. The caission inside can be used also for storage space, maintenance workshops, theatre halls, etc. The top of deck can be provided with landing and taking-off runways for the passenger or freight-carrying aircrafts.

The sheltered part of the lower deck can be provided here and there with mooring quays for the ships coming from the high sea, either through the specially developed dike entrances or from the coast.

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Fig. 5 Floating collector with rotating generator unit schematic diagram

1 float; 2 float arm; 3 connection arm; 4 rocking lever; 5 free pulley (ratchet disk); 6 gear coupling; 7 frequency multiplier; 8 flywheel I; 9 fluid coupling; 10 flywheel II; 11 resilient coupling; 12 rotating generator unit.



Fig. 6 Floating dike section meant for recreational purposes

Conclusions

In order to protect the sea shore and the harbor aquators the solutions of energy dissipating dikes are used throughout the world, they being either directly propped on the sea bottom or by means of pillars, or they are floating dikes.

Since the main cause of sea shore erosion and the damages brought about to the protection structures is represented by wave hydraulics energy, Romania has studied the possibility to absorb these out at sea energies in order to be taken out of the system before intercepting the shore or the structures.

Taking into consideration the geological sea bottom constitution in the vecinity of the Romanian coasts, AQUAPROIECT proposes the solution of energy capturing floating dikes. This solution consists of reinforced concrete caissons equipped with hydraulic machines for wave energy capturing. The recomended equipment is the one of the "captured" float tested in the Hydraulics Lab of the Technical University of Civil Engineering of Bucharest, it being specific to the Black Sea irregular waves.

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Completion of Sheltering Breakwaters of Constantza Port Precinct

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Abstract: The development of Constantza Port has been achieved in stages. The first stage included the port designed by Anghel Saligny and the extension, up to the limit that represents today the North Constantza Port. The second stage had started in 1972 and represents the extension of the North Constantza Port towards South, being called South Constantza Port.

Two breakwaters protect the precinct of Constantza Port, respectively the northern breakwater and the southern breakwater. The total length of the two breakwaters is of 12950 m. The Constantza Port breakwaters are of gravity type, with slopes. They have been sized at the action of significant waves with the assurance of 2 % in time.

Taking into account the continuous degradation of the breakwaters under the wave and current action, which could endangered also the port inner arrangements, the National Company – Maritime Ports Administration Constantza (NC - MPAC) decided to finalize these breakwaters.

Keywords: Constantza Port; breakwater; stabylopodes; berm;.

1. General

Constantza Port is the biggest port in the Black Sea basin and one of the biggest ports in Europe. It is composed of two large precincts, called North Constantza Port and South Constantza Port.

The development of Constantza Port has been achieved in stages, starting in 1896 and it included two important stages. The first stage included the port designed by Anghel Saligny and the extension, up to the limit that represents today the North Constantza Port. The second stage had started in 1972 and represents the extension of the North Constantza Port towards South. This extension was called South Constantza Port and includes in its precinct the outlet of the Danube - Black Sea Canal.

Each development stage had been started with the achievement of the sheltering breakwaters. The aim of these breakwaters was to create a sheltered precinct in which the works for the inner arrangement could be executed, to assure the ships access and safe stationing, including on bad weather, and finally to assure the fluent goods traffic.

In the present configuration, two breakwaters protect the precinct of Constantza Port, respectively the northern breakwater, with a length of 8000 m, and the southern breakwater, with a length of 5560 m. The total length of the two breakwaters is of 12950 m (Fig. 1). The Constantza Port breakwaters are of gravity type, with slopes. They have been sized at the action of significant waves with the assurance of 2 % in time.



Fig. 1. General layout of Constantza Port

The foundation level varies, the maximum level being of -26.0 m. The maximum level of the armour is of + 9.50 m for the northern breakwater and of +9.00 m for the southern breakwater. The resulted height of the construction is up to 35 m.

In principle, the Constantza Port breakwaters are composed of a quarry run core, protected with natural blocks, in accordance to the reverse filter criterion, and an armour of prefab concrete blocks with weight between 4.5 t/piece and 25 t/piece, leaning on a berm of antifers of 15 t.

At the top, the breakwater section is protected by a concrete slab with thickness between 0.40 m and 1.0 m and a protection wall (Fig.2).

2. Technical condition of the breakwaters

The execution of the South Constantza Port breakwaters had started in 1972 and constantly

continued till 1990, when due to the new economical conditions, the works stopped. Because of the specific technology for these works achievement, at works stopping the breakwaters haven't been finalized, remaining at different execution stages on important lengths.

Being incompletely achieved, the breakwater section suffered important damages under the waves continuous action. These damages consisted in (Fig. 3):

- destruction of the berm that sustain the stabilopodes armour;

- slide and dislocation of the stabilopodes armour;

degradation of the protection layers of natural blocks;
carrying away the breakwater core, causing breaches in the breakwater;

- degradation of the crown.



Fig. 2. Typical cross section ok breakwater





Fig. 3. Failure modes for a rubble mound breakwater

Moreover, at the storm from 4 - 5 January 1995, two ships of 20,000 tdw from the port exterior roadstead derived, striking against the northern breakwater of South Constantza Port. Because of the repeated impact between the ships and the breakwater, two breaches have been produced into the breakwater. One of the breaches is at km 2+500 and the other at km 3+500. The ships sink just near the breakwater base.

Taking into account the continuous degradation of the breakwaters under the wave and current action, which could endangered also the port inner arrangements, the National Company -Maritime Ports Administration Constantza (NC -MPAC) decided to finalize these breakwaters. Considering the total investment cost of 70 millions Euro, a repayable loan of 70 millions Euro has been contracted from the European Investment Bank, and a grant of 17.5 millions Euro from PHARE. The difference of 17.5 millions Euro will be covered from the budget.

The project include the completion works for the southern breakwater on a length of 3400 m and for the northern breakwater on a length of 4850 m, corresponding to South Constantza Port. Subsequently, due to the financial economies and to the reduction of execution period, the works have been extended. The repairs of another 2160m of the southern breakwater and of 2540 m of the northern breakwater have been included.

3. Works foreseen in the documentation

For the works execution a technical project has been elaborated. It included more stages, namely: - technical expertise of projects previously elaborated for Constantza Port breakwaters; -elaboration of the proper technical project; -elaboration of the tender documents for the selection of the Contractor;

-works supervision (Consultance).

Supplementary, the European Investment Bank required the technical expertise of the solutions adopted by IPTANA SA, by a foreign Consultant, respectively the Dutch company Frederic R. Harris B.V. The expertise confirmed the correctness of breakwater sizing and of the projects elaborated for breakwater execution.

In the technical project, the profiles from the previous projects have been adapted and resized to the actual condition of the breakwaters. The breakwater situation has been established by visual investigations and topographic and hydrographic surveys, correlated with the complex hydro-morphological conditions of the site.

The technical project foresaw the following works:

- restoration of the berm sustaining the stabilopodes armour;

-restoration of the breakwater core in the areas where it has been damaged;

-restoration of the protection layers of natural blocks;

-completion of the stabilopodes armour in order to realize the necessary density;

-execution of the stabilopodes armour in the areas where it doesn't exist;

-execution of the crown slab and of the protection wall; -rehabilitation of the existing crown slab and protection wall.

The adopted technical solutions assure the durability and the performance of the protection works for Constantza port precinct.

4. Works description (Fig. 4, 5and 6)

4.1. Restoration of the berm sustaining the stabilopodes armour

The investigations with frogmen and the measurements had showed that on certain areas the berm does not exist, and where it exists, it is incomplete.

Where it sustains the armour of stabilopodes of 4.5 t/piece, the berm is composed of natural blocks of 4 - 7 t/piece and where it sustains the armour of stabilopodes of 25/piece, the berm is composed of concrete antifers of 15t/piece.

The solution for berm foundation has two variants, depending on the ground nature:

- if the foundation ground has good geotechnical characteristics or stone is already laid down, the berm is directly founded on the existing ground;
- if the foundation ground is sandy, the berm is founded on a geotextile mattress, having separation and filtration role, on which a rubble stone layer of 2.0 m is laid down.

The geotextile has been placed by means of classic floating equipment, correspondingly adapted. The rubble stone layer has been placed with floating cranes, by direct dumping from barges of 500 - 1500 t capacities.

The natural blocks and the antifers have been placed by means of floating cranes.

4.2. Restoration of the breakwater core

Works for breakwater core restoration have been executed on the north – south section of the southern breakwater and on the northern breakwater between km 3+500 and km 4+850.

At the restoration of the breakwater core, rubble stone up to 500 kg/piece has been used. The stone has been transported from the quarry by carrier vehicles and placed by direct dumping and pushing with the bulldozer.

Before starting the works, all natural blocks and concrete prefab elements carried away by waves inside the section core have been retrieved from the respective areas.

4.3. Restoration of the protection layers of natural blocks

The sizes of the breakwater core protection layers depend on the design wave and take into account the reverse filter principle. On the external slope there are two layers, the first of blocks of 0.5 - 2.0 t/piece, 1.50 m thick, and the second of blocks of 2.0 - 4.0 t/piece, 2.30 m thick.

On the inner side, where there isn't a stabilopodes armour, the natural blocks layer has a thickness of 2.30m and the blocks have weight between 2.0 - 4.0 t/piece. The effect of waves overtopping the breakwater crown has been also taken into account.



Fig. 4. North breakwater - Cross section





Fig. 5. South breakwater - Cross section



Fig. 6. South breakwater - Cross section

4.4. Completion of the stabilopodes armour in order to realize the necessary density

Most of the stabilopodes armour surface did not have the necessary density of 20 pieces/ 100 m^2 . There were stabilopodes mounted on only one layer or if there were the two layers, the distance between them did not comply with the project.

The stabilopodes of 4.5 t/piece and 25 t/piece have been mounted in order to assure the density in

the layer and to realize the second layer. During these operations, works for existing stabilopodes dismounting and remounting on new positions have been necessary.

The stabilopodes have been mounted by means of big capacity boom cats of 1500 tm and floating cranes, equipped with G.P.S. system for the precision of subaquatic works (Fig. 7).



Fig. 7. Cranes, equipped with G.P.S

4.5. Execution of the stabilopodes armour in the areas where it doesn't exist

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These areas are in general at the back of the protection wall and towards the breakwaters heads. The stabilopodes have been mounted in two layers, with a special disposal in the crest area in order to obtain a pleasant appearance.

4.6. Execution of the crown slab and of the protection wall

The crown slab has a width of 7.0 m and thickness between 0.50 m and 1.0 m, depending on the loads at which the breakwater is submitted in the respective area. The protection wall has a height between 1.0 m and 2.0 m and a thickness of 1.0 m.

In order to prevent the carrying away of the fine material under the breakwater crown during wave action, a continuous concrete toe of 1.0 m x 11.0 m has been foreseen under the slab and the protection wall.

The crown slab has been poured in blocks of $6.0 \times 6.0 \text{ m}$, with expansion joints at 60 m distances.

Taking into account the behaviour mode of the breakwaters heads, where settlements bigger than on

the rest of the breakwaters took place, the concrete slab on the last 15.0 m has been replaced with natural blocks of 2.0 - 4.0 t/piece. The protection wall has a "L" shape and it has been poured without horizontal joints, in transoms of 6.0m length. At distances of about 150 m, stairs have been executed in the protection wall for the access to the crown, with a view to assure the personnel access in order to perform the visual observations and measurements necessary for the behaviour survey.

4.7. Rehabilitation of the existing crown slab and protection wall

Before 1990, the crown slab and the protection wall were executed on about 3500 m on the northern breakwater. They had many damages due both to the action of hydrological and meteorological factors and to the means of transportation. The restoration works consisted in pouring a new slab of minimum 25 cm thickness over the existing slab. A protection layer of epoxidic mortar has been applied on the protection wall face. These works have been executed in general with powered means, then the joints have been cut, observing the position of the existing joints.

5. Works execution phasing



Fig. 8. Comparision between programmed and real progress of works

Taking into account the big quantities of works and the necessity to work in open sea, which limits the execution favorable period, a period of 28 months has been foreseen for the works achievement. Due to the technical and technological measures foreseen in the project and applied on site, this period has been reduced with 5 months. This had direct effects on the work economy, because execution during winter season, which would led to material carrying away and inefficient use of equipment and workers, has been avoided (Fig. 8).

6. Financial aspects

In the frame of execution Details, the real situation has been analyzed at intervals of 5 - 10 m, which led to economies reported to the works cost established in the initial project.

This allowed the reasonable use of the stone blocks and the prefab elements, leading to economies of about 5 millions Euro. This way, the respective amount has been used for the repair of a supplementary length of 4690 m of breakwater.

7. Bill of quantities

For the completion of Constantza Port breakwaters, about 1,278,700 t of quarry products and about 393,300 cm of concrete in prefab elements and in crown have been used, this way:

- rubble stone	0–500 kg	357,500 m ³	(679,250 t)
- natural blocks	0.5-2.0 t/pcs.	$74,400 \text{ m}^3$	(130,200 t)
 natural blocks 	2.0-4.0 t/pcs.	213.700 m ³	(373,975 t)
- natural blocks	4.0-7.0 t/pcs	47.200 m ³	(80.240 t)
- crushed stone		9.600 m^3	(15.000 t)
- geotextile		26,960 m ²	(- , ,
- concrete antifers	15 t/pcs.	16.223 pcs.	
- stabilopodes	4.5 t/pcs.	17,555 pcs.	
- stabilopodes	25 t/pcs.	20,023 pcs.	
- concrete in the		50,480 m ³	
crown		,	
- injection of		6,650 m ²	
concrete in the			
inner face of the			
existing			
protection wall			
• C • • • • • • • •			

8 Conclusions

By the completion of the sheltering breakwaters, the damages evolution have been stopped and the use conditions of Constantza Port precinct have been considerably improved, respectively the safety degree for the access and operation of ships. It is now possible to safely complete other works, which are in execution.

The period in which the port can be safely used increased. Conditions have been created to operate certain goods when ships are anchored. The volume of the inner arrangement protections is reduced.

The resulted economies are more than 6.0 millions Euro in the first year. It is no more necessary to achieve protection and subdivision works inside the port, which would cost about 50 millions Euro. The economical effects of breakwater completion make this investment entirely efficient.

The Modelling Effect on the Offshore Structural Response

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Abstract: In offshore structures reliability analyses, random variables are taking into account. These needs special care also in design, construction and exploitation phases. The major doubts are associated with sea long-term statistics, structural modelling and fatigue degradation.

Keywords: model, DOF, stress

1. Introduction

The propose of this work is the study of two offshore frame structure models for establish of the errors who appear in the structural response.

For the first model elements are rigid connected and in the second are jointed. The random sea elevation is simulated using the Black Sea spectrum model and for the water particle speed and acceleration is used Airy theory. The hydrodynamic forces on the structure are given by Morison – O'Brian equation taking the inertia and drag coefficients as constants - $C_M = 1.5$; $C_D = 0.6$.

The structural response is obtained by spectral analyze. The local stresses are computed by time integrated method (using a time step of 0.02s).

2. Structural model. The offshore structure idealization

The self-lifting offshore platforms are made by steel tubular elements jointed together for create a axial frame. The metallic offshore structure is a dynamic system having an infinite number of degrees of freedom.

The fatigue collapses, being a localized phenomenon, imply that structural detail have to be carefully considerate. The structure and his joints have to be model to give an accurate time history stress answer. In the present work the structure was model as a spatial frame although the great number of the dynamic DOFs made difficult the analysis. This option is given by the propose to have realistic results, a plane model loosing the spatial character of the stress distribution, and in equivalent beam approximation only axial stress being evaluate. This kind of modelling satisfies displacements study but inaccurate for a fatigue study in welded joints.

Two structural models are considered. In the structural **model I** the still structure is idealized as a spatial frame with rigid connected elements. The DOFs are considered horizontal and vertical translations and node rotations.

The structural mass of the elements is considered uniform divided on the element's length.



Connection between elements for the model I

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For the **model II** the structure is idealized as a frame with tubular elements flexible connected. The DOFs are given by horizontal and vertical translations. Also the structural proprieties of the elements are considered in idealized structure.



Connections between elements for model II

3. Structure description

The structure is modeled with the help of FEAT finite element program, as a spatial frame with tubular elements, the size of the elements being similar with Gloria's.

Section		Elements dimensions (mm) and the number for a platform column							
No.	Length	Verticals	Horizontals	Diagonals	Middle elements				
1a	8890	φ914,40x44,45 3pieces		φ406,40x40,48 3 pieces					
1b	9754	φ914,40x44,45 3pieces	φ457,20x31,75 12 pieces	φ339,73x15,88 24 pieces	φ219,08x8,17 12 pieces				
2	14631	φ914,40x41,28 3pieces	φ457,20x31,75 12 pieces	¢339,73x15,88 24 pieces	φ219,08x8,17 12 pieces				
3,4,5	14631	φ914,40x34,93 3pieces	φ457,20x31,75 12 pieces	φ273,05x11,91 24 pieces	φ219,08x8,17 12 pieces				
6	14631	φ914,40x38,10 3pieces	φ457,20x31,75 12 pieces	φ273,05x13,75 24 pieces	φ219,08x8,17 12 pieces				
7	13970	φ914,40x44,45 3pieces	φ457,20x31,75 12 pieces	φ273,05x13,75 24 pieces	φ219,08x8,17 12 pieces				
8	14910	φ914,40x44,45 3pieces	φ457,20x31,75 12 pieces	φ273,05x13,75 24 pieces	φ219,08x8,17 12 pieces				

The platform height is 120m and the depth of the water in site is 50m. Eight welding connected sections make the structure columns.

The structure of each section is done by a beam system, three verticals connected by horizontal elements. The plane stiffness is made by diagonal and middle elements. At the first section level a structural tank is equipped with ballast and penetration in foundation rock role.



Section of the column

4. The structural response evaluation

The structural analyze is done by Newmark method. For allowing discriminated loads function of wavelength evaluation, the calculus is separate for each column, the effects being superposed for each time step.

For the increase of precision, the calculus is done with directional spectral models, defined for cardinal and intercardinal directions.

Two critical points are taking into account: at the splash zone N1 and at the mud line N2.

It is of great importance the vertical element behavior, especially the connection with the horizontal elements, the maximum degradation being observed in these areas.

Stress and displacements are presented for a 7m high wave train on E-W direction.

Model I - Total deformations H=7m										
A1A2B1B2	A3B3	D1D2C1C2	D3C3							
80.568 90)046 78.584.560 53.062 7377062 7377062 7377062 73770 62.403.255 148.012 61.483 73.610 84.696 24.656 27.900 19.043 20.404 13.025 13.825 13.455 148.012 61.486 24.656 27.900 19.043 20.404 13.025 13.455 14.835 14.835 14.835 14.835 14.835 14.835 14.835 14.835 14.835 14.835 14.855 14	79,406 79,406 66,064 60,064 7,506 7,506 7,506 7,506 7,506 7,510 7,257 60,064 7,257 7,25	57.725 54.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007 55.007	48.860 49.267 36.393 36.393 36.393 36.165 29.903 9.621 82.013 83.422 30.649 82.141 33.518 24.345 16.264 16							

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	Model I - Maximum stress H=7m [daN/cm ²]											
A1A2	2B1B2	A	3B3	D1D2	C1C2	D3	C3					
3.4993e3158 e-05	3.3413e.5112 e-05	4.200e-05	4.192e-05	3.8402e0157e-05	3.6112x3254e-05	4.200e-05	4.192e-05					
-1.906 -1.906 -1.473	1.7331.737 -[.909] -2.477	1.710	1.706	1.7361.730 -1.904 -2.473	-1.7271.732 -1.901 -2.468	1.710	1.706					
-3.352 -2.964 1.673	-3 352 - 4 21 31 3	-3.248	-3.247 10.450	-3.340 -2.815 1.767	-1 340 - 1 966	-3.248	-3.247 10.450					
-1₽892 -1₽892 73.749	-15.262 B.269 -15.262 B.269 -15.921	17.814 -21.075	-20.632	-112295 -112295	170 369 -14 930 8.444 27 2.949	-21.075	-20.632					
-92 <mark>0044</mark> 82.898	-96.862 75.053	48.779	70.498	-9710793 74990	-92.189 82.803	48.779	70.498					
-5 4.8271 .176	-90.05 55 (423	-89.960	:68:249	-5 3.338 ,778	-89.8928.643	-89.960	:98:293					
-91.553	-91.456	-88.346	-88.734	-91,316	-91.405	-88.346	-88.734					
-68.429 -69.934 -69.934	-90.65 <mark>0</mark> .945 -6 8 .485	-89.798		:89.099 -90 ₋₃₁₁	-90.424 -90.424	-89.798						
-8 9 3567 47 -77.119	-89.210 -75908	-89. 613 .74.941	-98.949 -75.431	-8 4.461 41 -76.232 -76.373	-89.640 -77 44 14 ⁶⁴	-89.678 .74 <u>.941</u>	- 98.949 -75.431					
.73.940 72.416	73.788 67.692	71.676	72.280	-74.531 - 82.45 8 -69.229	-74.690 - 89.289 -60.367	-71.676	-72.280					
-72.037 -72.037	68.410 171.487 63.365	65.674	-66.259 58.680	72.6184.312	64, 6 3,3090	57.787	58.689					
03.3400.977 Z .73.9837.229 59.303.629	67.0913.535 53.1008.147	Z -62.465	-53.219 -42.108	Z 74,9668.064	68. 279 5.356 -55. 40 8.543	× v 40.690	63.219 42.108					

Model I - Minimum stress H=7m [daN/cm ²]										
A1A2B1B2 A3B3				Ι	D1D2C1C2	D.	3C3			
-3.8192adD6e-	05 -3.6023-8254e-05	3.4346-05	-3.437+-05	-4.2693	9889 e-05 -3.9644-880 e-05	-3.672e-05	-3.665e-05			
2.6612.664 -2.867,750	2.6672.657 -2.74 6 1.863	.2.838 2.838	2.834	2.6612 -2.862,750	.666 2.665 -2.754.872	-2.862	2.644 -2.866			
-6. 4110 ,429	-10.509 651	-19. 818	-19.893	-6.6600.523	-10.446 419	-19. %30	-19.336			
-19.462 ^{1,585} -19344 3 9 92	-3.50 <u>f</u> 1,790 -20009999	.74 ST	-18 334	-19.777 ^{1,492} -19 ³⁰⁰³ 490	-3.56 <mark>9</mark> , 446 -359,391 <mark>- 376</mark> , 376	-47.588	-#8.619			
-122.563 -99.224	-110,767 -97.152	-230.775	-210.419	-109.547 -96.553	-12 - 398 -99.006	-210,980	-231.257			
-90009 -66.610	-98.880 -68.030	-96.993	-97.003	-98049 -64.956	-98.425 -66.540	-97.777	-97.767			
-72.493 -73.6 600 260 -9 .2 113	-71,180 -95,3300,372 -95,179	-95.210 -95.038	-94.951 -94.760	-70.947 -72.0 39 (115 -94,939	-70,262 -95,0372,318 -94,874	-95.475 -95.293	-95.754 -95.581			
.71, 41 9.712 -81.4 3 138335 80.904	-93.7 281 371 -93.7 281 371 -79.6 2 6	.93.719 -80.276	.03.300	-70 8 -80 3 38 701	9.721 .79.187 <u>1.351</u> .93.73811411	-92.913 -92.913	-79.764 -93.375			
78.5604.752	74.58778,991	83 678	¥2.038	73 78	15 242 85 6 729 859	85.363	76 909			
-96.384 .492 Z 84.850 957	80.6826.779 75.7884.482	2 86.060 2 81.839	83.772 81.083	Z 36.40	82.481 82.3548.896 16980 77.3936.776 188302 97.54812.432	Z 78.981	82,535 79,758 92,686			
	FA2.30038.402	X Y 95.405	895.006	ХҮГ		X Y Parts	102.000			



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The stress global distribution on sea states and cardinal directions is presented, the red color

corresponding to the N1 level and the blue color corresponding to N2 level.



5. Conclusions

In Romanian shelf, the 7th offshore structures made by Galati Naval Yard after a design project of American company "The Offshore Company" take into account the Mexico Golf conditions, obvious different with the Black Sea environment. Also the design initial data are different for the real location of the structures (90m depth of water, 12m maximum wave height, 45m/s wind speed)..

In present work the real environmental conditions for Lebada location is used (priority the water depth is 50m so the area where the platform deck is teach the columns is obvious much lower so the projected reinforcements are useless).

The structural response is much affected by modelling. A complex spatial modeling, with the respect of the geometric characteristics and the realistic environmental conditions, the separate loading of each column gives a height calculus precision. Although, the use of the spectral method give a better adaptation of the theoretic model at the real situation.

Evan it was expected a decrease of the stress in flexible connected elements model, it is observed rather a stress redistribution due to the great rigidity of the spatial frame.

It is also possible as for some wave length and loading directions higher sea states stress to be lower then middle sea states stress. For E-W direction is obvious this phenomenon due to the differential loading of the platform columns function of the wave length.

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Wind Forces Evaluation on Offshore Structures

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Abstract: Acțiunea dinamică a vântului coroborată cu încărcările datorate valurilor constituie acțiunile principale la care o structură de tip jaket sau platformă autoridicătoare trebuie să facă față pe parcursul întregii durate de exploatare. Dacă pentru o construcție volumică, cu suprafețe mari, aceste încărcări sunt relativ simplu de evaluat, pentru structură tip grindă cu zăbrele problema devine evident mai complicată. Un alt aspect necesar unei bune evaluări a încărcărilor este stabilirea factorului de turbulență respectiv aportul componentei pulsatorii, de rafală, în încărcarea totală.

Keywords: wind speed, pressure

1. Introduction

Together with the waves action, the wind loads have a great importance in offshore structures design, those two actions being superpose. Even in fatigue calculus the influence of the wave is preponderant, the wind loads can produce a height response of a flexible structure in storm cases.

The wind loads problem is attack taking into account the size and the type of the expose surfaces, the time variability of the element dimensions, speed and acceleration of the air mass.

In what regards the winds speed, intensity and direction it has to consider the time and spatial variability and the statistic characteristics varieties with the high.

The load evaluation has to consider also the extreme storms – which generate maximum stress – and moderate storms with great appearance probability.

Both wind components, stationary and fluctuant are studied. For the stationary component two speed profiles are usually used: logarithmic speed distribution and power distribution.

2. Theoretical background

Logarithmic speed distribution is based on consideration of uniform rubbing stress on some hundreds meters above the sea surface, respectively the linear increase of the speed.

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$$U(z) = \frac{u^*}{k} \ln\left(\frac{z+z_0}{z_0}\right) \tag{1}$$

where: $k \approx 0.4$ von Karman constant

 z_0 – the rugged characteristic high connected with the wave high

 u^* - friction speed proportional with the square rout of the rub stress on the water surface.

Power distribution low is empirical and can be retain as:

$$U(z) = U_R \left(\frac{z}{z_R}\right)^{\frac{1}{n}}$$
(2)

where $n = 6 \div 8,5$ for the open sea conditions

Davenport recommends other relation on wind speed variation with the high:

$$U(z) = U_R \sqrt{0.93 + 0.007z}$$
(3)

The fluctuant component, squall type, U(t), is considered a stationary time process, normal and independent of z quote. The spectral density of this process will be written as:

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$$S_{U^{1}}(n)dn = 4kU_{R}^{2}\frac{x}{\left(1+x^{2}\right)^{4/3}}$$
(4)

where: n – wave number of squall components $x = \frac{1220n/U}{r}$ parameter on which

 $x = 1220n/U_R$ – parameter on which is express the spectrum shape

 $k = (0,005 \div 0,05)$ – coefficient depending of the basis field rusty

Squalls dispersion is obtained by:

$$\sigma_{U^{1}}^{2} = \int_{0}^{\infty} S_{U^{1}}(n) dn = 4k U_{R}^{2} \int_{0}^{\infty} \frac{x}{(1+x^{2})^{4/3}} dn$$
 (5)

The wind turbulence intensity at z high is defined as the ratio between root mean square of the squall series and the mean speed at that high.

$$I(z) = \frac{\sqrt{\sigma_{U^1}^2}}{U(z)} \tag{6}$$

The ISO TC67/SC7 standard recommends

$$I(z) = 0.06(1+0.043U_R) \left(\frac{z}{10}\right)^{-0.22}$$
(7)

and the value of the storm effective wind:

$$U(z,T) = U(z)[1-0,41I(z)]\ln\frac{T}{T_0}$$
(8)

where T_0 mean period.

3. Wind forces on the offshore structures

Wind forces on the offshore structures depend on the following elements: wind speed, the shape and the expose surface of the element, element's high above the sea level and the position relate to the direction for where the wind blows. For cylindrical elements is recommended:

$$F_{\nu} = \frac{C}{16} U^2(z) \sin^2 \alpha A \tag{9}$$

where: F_v – wind force orthogonal to the element's surface

 α - the angle between wind direction and element's surface

A – surface of the element

$$C = C_{\infty} \left(0.5 + 0.1 \frac{L}{D} \right)$$
 - shape coefficient

Other calculus relations recommended by A.P.I. respectively ABS is:

$$F_{v} = 0,0473U^{2}(z)C_{s}A \tag{10}$$

$$F_{v} = 0,0623U^{2}(z)C_{h}C_{s}A$$
(11)

where: C_h – high coefficient

 C_s – shape coefficient (0,5 for cylindrical surfaces)

A - expose surface

Wind pressure on thin cylindrical elements is taken by the formula:

$$p(t) = \frac{1}{2} \rho_{aer} C_D [U(z) - U^1(t)]$$
(12)

where: $\rho_{air} = 1.3 \text{ kg/m}^3$ air density

 C_D – strength coefficient depending of Reynolds number

4. Interaction between wind and waves

When wind blows on a "frozen" sea with a regulate wave profile, current lines will have the shape shown in following figure:

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Fig. 1 Wave profile and wind current lines shape

The air flow is laminar and it will get up on the wave wind side. On the under wind wave side a vortex will appear where air tangential flow will be reverse. The vortex is limited by two points where

Wind speed 5m/s



Taking into account the wave shape, the pressure profile can be written:

$$p(x) = C_1 \sin \theta_1 \rho_{aer} U_R^2 \frac{\eta}{\lambda} + C_1 \cos \theta_1 \rho_{aer} U_R^2 \frac{1}{2\pi} \frac{d\eta}{dx} (13)$$

where: ρ_{aer} – air density

 λ - wave length

 U_R – wind speed at 10 m above the water surface

 θ - phases angle



Wind speed 17 m/s







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Wind speed 20m/s



Fig. 1 Wind turbulence, speed and force distribution with high

4. Conclusions

The turbulence calculus functions of squall dispersion in four study cases and the obtained of the wind calculus speed and pressure aloud a correct evaluation of the pressure value on the structural elements.

It can be observed that the formula which determine the greater increases with the high is the power distribution low (C2), in the same time the logarithmic distribution low have (C1) and ISO TC67/SC7 (C4) has a slower variation especially for values superior to the recording high $z_R = 10m$. The most uniform variation is given by Davenport formula (C3), on which it is not observed the rapid decrease of the speed for highest inferior to 10m.

The force distribution with the high graphics shape determinate with the API formula is similar with the speed graphics being practical dependent on the square of the last.

It could be evaluate from the obtained results that for highs superior to 10m, in open sea conditions, the real increases of the wind speed and the corespondent forces are better model by slow variation expressions, logarithmic distribution and ISO TC67/SC7, while for values lower to the recording high, in storm conditions, the power distribution low, with a sharp decrease is mach realistic

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Structural Analysis for a Harbor Mole under Tidal Dynamic Action

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Abstract. The paper encompass the procedure to determine the strains and stresses in a mole built by huge concrete blocks. The calculated force applied upon the mole is variable and is due to tidal. The paper describes the building of nodal model of blocks, taking into account the discontinuity between them.

Keywords: mole wall; gap elements, tidal, nonlinear analysis, strain, stresses

1.General considerations

The paper encompass the procedure to determine the strains and stresses in a mole built by huge concrete blocks. [the new module, Constanța-1961 year].



Fig. 1 The seaside zone

From all the charges acting upon a mole wall, there were taken into account those coming from: self-weight, hydrostatic pressure upstream and downstream, ascensional water force, the landfill side pressure and the tidal.

The mole wall is situated in the medium depth zone, were the movement apears on eliptical orbits, and the influence of the depth begin to be present (fig.1). The tidal elements:

- height $h_0 = 2.0m$;
- modulation lengths $\lambda_0 = 30m$;
- depth of the water in front of the mole wall H=10m;
- wall height $H_c = 12.5m$.

In the medium depth zones $[\lambda_0/2 > H \ge H_{cr}]$, for the vertical mole wall, it was considered the most unfavorable loading hypothesis, namely the appearance of a stady tide near the mole wall ["clopotiş" – an unfortunate interferal of the reflected tide with the incident tide].

In this case, the elements of calculating tidal becomes:

• tidal height:
$$h = 2 h_0 \sqrt[4]{\frac{H_0}{H}} = 4.40m$$

• modulation lengths:
$$\lambda = \frac{\lambda_0}{\sqrt{\frac{H_0}{H}}} = 27.0m$$
;

• propelling rate:

$$c = \sqrt{\frac{g\lambda}{2\pi}} \sqrt{th \frac{2\pi H}{\lambda}} = 6.43 \, m/s \quad ;$$

• oscilating rate:
$$au = \frac{\lambda}{c} = 4.212s$$
 ;

• the tidal heights at the time "t":

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$$\eta_{(t)} = \frac{h}{2} \sin \frac{2\pi H}{\tau} t \qquad (m)$$

• the uper heighting of the water level versus the stady level:

$$\Delta h_{(t)} = \frac{\pi}{\lambda} cth \frac{2\pi H}{\lambda} \eta_{(t)}^2 \qquad (m)$$

$$p_{z_4} = \gamma \{ \left[\frac{H-z}{H} (\eta_{(t)} + \Delta h) - \frac{\eta_{(t)}}{ch \frac{2\pi H}{\lambda}} \right] + \frac{\eta_{(t)}}{ch \frac{2\pi H}{\lambda}} \}$$

 $[[(\eta_{(t)} + \Delta h)], H]$

with values "+z" on the domain:

The loading time depending diagrams are presented in fig.2, and the pressure values are determinated for



Fig. 2 The hydrodynamic pressure diagram upon a vertical parament at a given time "t"

the two domains:

• The domain
$$0 < t < \tau /2$$
:
 $p_{z_1} = \gamma \left[(\eta_{(t)} + \Delta h) + z \right]$

with values "-z" on the domain: $[0, -(\eta_{(t)} + \Delta h)]$

$$p_{z_2} = \gamma \left[\frac{H-z}{H} (\eta_{(t)} + \Delta h) - \frac{z}{ch \frac{2\pi H}{\lambda}} \eta_{(t)} \right]$$

with values "+z" on the domain: [0,+H]

• The domain $\tau / 2 < t < \tau$:

$$p_{z_3}=-\gamma \ z$$

with values "+z" on the domain:

$$[0, -|(\eta_{(t)} + \Delta h)|]$$

2. The numeral modelling. Mole wall under dynamic tidal action.

The numeral modelling (plane) reffers to a gravitational vertical mole wall, realized from massif concrete blocks (cross section by the new 1961 year modul, in Constanta city), having the geometry presented in fig.3.

The concrete structure, the foundation soil and the landfill with rocks were discrete into plane four nodes polygonal elements. (PLANE-182 having the side dimension of 0.25m), and for the contact zone between the concrete and the foundation terrain were been used elements GAP (CONTACT-12, point by Gh. Lazar et E. Fulop / Ovidius University Annals of Constructions 3, 413-418 (2002)point type, the GAP element at the closing may be
charged with compression and frictional forces).For the considered domain, the
action due to tidal was determinated

For the considered domain, the hydro-dynamic water action due to tidal was determinated in a time period of 8.424 s ($T_{analiz\bar{a}} = 2*\tau$).



Fig.3 Gravitational vertical mole wall, realized from massif concrete blocks (cross section by the new 1961 year modul, in Constanta city)

The characteristics of the 5 zones of different materials (in the dry zone, and underwater respectif) ant the GAP element type constants are presented in the table nr.1.

Table nr.1 The material properties and constantes for the GAP-contact element.

Elem.		PLAN	GAP- CONTACT-12				
Zone	E daN/m 2	μ	ρ Kg/m ³	f	k _n daN/m ²	k _s daN/m ²	
Ι	2.1E+9	0.17	140	0.55			
II	3.8E+8	0.35	0.00	0.55			
III	2.1E+9	0.17	240	0.65	2.1E+9	1.26E+9	
IV	3.8E+6	0.27	95	0.60			
V	3.8E+6	0.27	195	0.70			

For each time step (Δt =0.117s) of the analized period (T_{analize}), the concentrated forces acting on the upstream vertical mole wall parament were determinated with the previous refered relationships (cap.1).

It was been considered an elastic behaviour of the materials and a nonlinear analysis was made in a transient regymen. By numerical simulation there were obtained the strain and stresses at each step of time Δt .

In fig.4 are presented the stresses on x direction (S_x) at the time $0.25\tau{=}1.03s$ when the hydrodynamic action is at the maximum (on uprising – the domain ($0 < t < \tau/2$) and the current pressure is given by $p_{z1}, \ p_{z2}$), respectiv at the time $0.75\tau{=}3.159s$ (fig.5) when the tidal is at minimum (at decreasing – the domain ($\tau/2 < t < \tau$) and the current pressure is given by p_{z3}, p_{z4}).

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One can observe that the unfavorable situation is in the interval of decreasing of the level oscilating, where the stress values on x direction (S_x) is maximum in the domain (-12053 daN/m² to 3350daN/m²), versus the level oscilating rising, were the maximum values are in the domain of (-4795 daN/m² to 566daN/m²).

Simingly in fig.6 and in fig.7 are presented the



Fig.4 S_x effort at the time t=1.053 s

the stresses on y axis (S_y) at the same time steps.

We can observe that the vertical efforts S_y are the gratest in the tidal decreasing (-27626 daN/m² to 252 daN/m²), versus the uprising interval where the values are in the domain (-5200 daN/m² to 215 daN/m²).



Fig.6 S_y effort at the time t=1.053 s

The level oscilation generates important horizontal forces which disturb the mole wall from its static equilibrium position and made it oscillant.

From the calculations on this system, was determined the dominant self oscilation period which is $T_s=0.25s$.



Fig.5 S_x effort at the time t=3.159 s

Comparing the fundamental oscilating period $T_s=0.25s$ of the mole wall with the tide oscilation period $\tau_0=4.212s$, we can presume that the resonance phenomena cannot occur.



Fig.7 S_v effort at the time 1 t=3.159 s

In the contact zones of concrete blocks, important are the sharing efforts S_{xy} , which are presented in fig.8 and in Fig.9.



Fig.8 S_{xy} effort at the time t=1.053 s

The S_{xy} effort in the decrease level oscilation interval takes values in the variation domain (-10779 daN/m² to 1627 daN/m²), versus the increasing interval where the values are smaller, in the domain (-1016 daN/m² to 2038 daN/m²).



Fig. 10 The total deformation u_{sum} at the time t=1.053 s

The time variation $(T_{analize} = 8.424 \text{ s})$ of the a S_x and S_y efforts is presented in fig.12 for some characteristical nodes, respectiv the variation of shearing efforts S_{xy} in Fig.13.

The deformating distribution on x axis (u_x) are presented for the two situations in fig.10 and in fig.11.It is to observe that the maxim value is in downstream of the mole wall, in the landfill zone.



Fig.9 S_{xy} effort at the time t=3.159 s

The deformation values are maxim in the top zone more exactly in the concrete block zone, with a value of approx.. 0.0000546 m (fig.10) and in the landfill zone (upper right corner) with a maximum value of 0.0001358 m (fig.11).



Fig.11 The total deformation usum at the time t=3.159 s

It is to remarck that the oscilating variation of the S_x , S_y efforts have a maximum value of $\pm 400 da N/m^2$, and the S_{xy} efforts have a maximum value of approx. $\pm 200 da N/m^2$.



Fig.12 Time variation of efforts S_x and S_y

Time variation of deformations u_x şi u_y is presented in fig.14 for some characteristical nodes on top and at the base of the mole.



Fig.13 Time variation of Sxy efforts

3. Conclusions

From the analysis results that the oscilation of water level implies an oscilation of the structure, but there is no danger for an resonance oscilation, the self oscilating period for the structure (T=0.25s) is far less than the self period of of the disturbing forces (τ =4.212 s).



Fig.14 Time variation of deformations u_x and u_y

Concerning the stresses wich apear in the structure in the landfill and in the foundation soil, for the tidal characteristics presented in cap.1, the most unfortunated situation is that before the appearance of the tidal. In this situation on the structure act too the dominant loadings given by: self weights, the hydrostatic water preassure, the ascensional forces, and the side landfill preassure.

On increasing the tidal items, the horizontal forces ampliffies whithuot a corresponding ampliffying of the shear stresses, situation which can destabilize the gap zones of the blocks inducing their displacements upstream or downstream.

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The Hydraulic Expertise of the High Water Discharge Structure in Earthen Dams

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Abstract: The hydrotechnics systems, in the time can change the function they has been designed and built. The economic changes in Romania after 1989-triggered alteration of the main function of some of the storage basins. The analysis of a dam operation safety including other hydrotechnics buildings involves a technical expertise of the hydraulic and structural type. This work presents some results obtained by the authors regarding the hydraulic and structural expertise of the high water discharge structure of the Varsolt dam, Salaj County. The hydrotechnic construction presents substantial discrepancies of the constructive and hydraulic parameters in comparison to those initially projected. Al the same time, the paper presents the methodology used in hydraulic verification of discharge opening. The results revealed new safety parameters of the high water discharge structure and suggested modernization solutions.

Keywords: Hydraulic expertise, dam, hydraulic regime, discharge structure, chanel, risk factors.

1. Introduction

Hydrotechnic systems can in time modify some of the functions for which they were created and implemented. In this case, the hydrotechnic system is subjected to new demands, which were not taken into consideration while designing.

These modifications affect the safety of exploiting by introducing supplementary risk factors. The hydraulic expertise allows for an analysis of the way in which hydrotechnic constructions behave in the new exploitation circumstances. What results are the characteristics of the new hydraulic regime and, at the same time, an insight into the safety measures required while exploiting.

The Virsolt Hydrotechnic System is located on the Crasna River in northern Romania and lies 96 Km away the border with the Republic of Hungary. The accumulation was deigned with the purpose of reducing flood waves with a probability of overflowing of 1% and 0.1%, irrigation, fish farming and supplying with water a town.

The hydrographic basin corresponding to the accumulation measures 310 Km^2 . The hydrotechnic system started to be exploited in the in the year 1979. In the later years the hydrotechnic system underwent several structural and functional modifications.

The storage basin is achieved through a frontal dam with a heterogeneous structure executed with local materials. The construction parameters of the dam are: maximum height – 14.0 m, width at the crest – 5.0 m, length – 2,160 m, average width of the road territory - 80.0 m etc.

The storage basins specific volumes are: on the total -46,2 million m³, accumulated -39,86 million m³ at the maximum level of exploitation (p = 1% and elevation 243,94 m.l.B.S. – meter level Black Sea), of reducing flood waves 24,60 million m³ etc.

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The calculus discharges are: $Q_{1\%} = 330$ m³/s – design discharge and $Q_{0.1\%} = 516$ m³/s – verification discharge. The debit absorbed from the basin for the water supply is Q = 0.75 m³/s.

An analysis of this type was carried out in regard to the high water discharge structure of the of the Varsolt dam.

2. The constructive and hydraulic characteristics of the high water discharge structure.

The discharge structure is situated on the left slope of the Varsolt storage. The hydrotechnic construction was sized to accommodate a debit $Q_{1\%} \cong 58.30 \text{ m}^3/\text{s}$ and verified for $Q_{0,1\%} \cong 236.0 \text{ m}^3/\text{s}$ (special verified discharge $Q_{0,1\%} \cong 250.50 \text{ m}^3/\text{s}$). The discharge structure is structured in six channel transoms in trapezoid section: the channel of approach with a spillway and a dissipator, the connector channel, the outlet channel, the rapid channel, the wave trap and the downstream apron, the outlet channel connected to the Crasna River. The full length is 411 m and the bottom width varies between 42.0 m and 20.0 m.

The channel of approach to the discharge opening is equipped with a Creager type spillway, with a length of 45 m and a height of 2.0 m. The spillway crest is situated at the 242.00 mdM level. After the spillway, a wave trap with two rows of teeth spread over a length of 10.0 m has been executed. The connector channel has the following parameters L = 20.0 mand I = 0.22 %, while the bottom with diminishes from 42.00 m to 20.00 m. The outlet channel has a length of 166.0 m, he bottom width is 20.0 m and the slope gradient is 0.22 %. The rapid channel has a length of 74.0 m, the bottom width is 20.0 m and the slope gradient I = 15.0 %. Following the rapid channel, over a length of 23.0 m the wave trap has been positioned, followed by a rizberma of 23.0 m in length, executed out of rubble stone.

The discharge opening is connected to the Crasna River through a connecting channel with a length of 80.0 m.

The perimeter of the channels, with the incline slope of 1:1.5, is lined with monolith concrete slabs with a width of 20 cm.

The bottom of the rapid channel as well as the incline are protected by a reinforced concrete facing with of 20 cm, equipped with expansion joints distanced at 6.00 m. The reinforced concrete facing protects the channel 2.50 m in depth.

3. Results and discussions.

After 27 years exploitation, the water constructional works (the dam, the discharge structure, the intake tower) shows a satisfactory functional status in what regards the structural integrity, its hydraulic response and its mechanical stability.

After the year 1989, the functions of the hydrotechnic system were modified. Presently, the achieved functions are: reducing the flood waves on the Crasna River, water supplying of two towns, fish farming, tourist area etc. Changing the initial function has determined consequently the designing and execution of several modifications in the dam's structure, as well as in that of the intake tower and execution of a supplementary intake tower.

The new conditions of exploitation have determined the maintaining of a raised retention level and the intense usage of the hydromechanic devices within the intake tower. At the same time, the hydrotechnic system has repeatedly undergone actions of an endogenous nature. These actions have led to a diminishing state of safety in exploiting the hydrotechnic system.

While exploiting the accumulation some important deficiencies have been noted, as a result of incomplete implementation of the execution project, of the interaction between the construction and the incorporating medium, of the action of the endogenous and exogenous factors etc.

According to the data which is measured and assessed on site as well according to the data resulted form the UCC study, a series of constructive and functional parameters for the high water discharge structure of the Varsolt dam were found as modified in comparison to those inserted in the execution project.

The most important modifications were:

- an increase in the longitudinal slopes of the constructive transoms of the discharge structure; the outlet channel presents a current slope greatly increased, respectively 1,27 % compared to that of 0.22 % as designed in the projects;

- the transversal section presents modifications of the incline slope for both banks in various analyzed sections; however, the parameters remain relatively similar to the project value (1;1,5);

the calculus hydraulic and hydrological data have been modified as a result of the process of bringing the hydrologic study up to date given the changes which occurred in the hydrographic basin.

A physical depreciation of the reinforced concrete facing in various degrees has been recorded on transoms I, II and III of the discharge structure. The concrete slabs presents fissures and are longitudinally craked while the expansion joints are damaged in a proportion of 40 - 60 %. The surface of the slabs is depreciated especially at the bottom of the channel, due to the chemical erosion of the concrete as well as to the freeze-defreeze phenomenon.

As a result of the analysis on site and of the new hydrological data it has become obvious that the hydraulic regime has suffered modifications in comparison to that initially calculated by the draftsman. The hydraulic regime is thus not respected given also the change in the calculus debits as a result of the modifications of the sorage function.

The hydraulic verification has been carried out with an automated calculus program with the current situation of the geometrical and hydraulic parameter of the discharge structure. The calculus debits varied between 50 and 350 m³/s, with the maximum discharge debit of $Q_{0,1\%} \cong .25950$ m³/s. The rugosity coefficients were n = 0.015 ... 0.016.

The modification of the slope, as well as the calculus debits determined the modification of the state of motion on the transoms and implicitly of the position of the sections and the control depths. The new hydraulic regime is characterized by an above critical state of motion on transmom II and III, uphill of the rapid channel.

The initial projected state of motion on these transoms was under critical. In this case, both the position of the control sections and the values of the control depths are modified. As a results the hydrodynamic stability of the discharge construction is negatively influenced. In the new functioning circumstances, the velocity recorded for the calculus debits on the connector and outlet transoms tend to reach high values (2.97 m/s ... 7.35 m/s) in comparison to those inserted in the execution project.

The depths of the water for the new verification debits are within the limits of the current flow section. For transom III the hydraulic characteristics are:

- for debit $Q_{0,1\%} = 58.3 \text{ m}^3/\text{s}$, results $h = 0.93 \dots 0.59 \text{ m}$, curve type 2 on an above critical state of motion;

- for debit $Q_{0,1\%} = 250.50 \text{ m}^3/\text{s}$, results $h = 2.36 \dots 1.52 \text{ m}$, curve type 2 on an above critical state of motion;

- for debit Q \cong 350.0 m³/s, results h = 2.91 ... 1.91 m, curve type 2 on an above critical state of motion.

The verification of the functioning of the spillway and the dissipator basin in heigh debit situations indicates a non-overflowing operation of the hydrotechnic construction, with influences on the stability of the hydraulic regime

The verificaton was carried out both for an aerated flow regime as well as for incident cavitation phenomenon. On the discharge/outlet transom there is no aerated flow regime produced.

Following the expertise carried out on five earthen dams within the second and third categories of importance, equipped with discharge structure a specific analysis method for discharge construction has resulted. The hydraulic expertise of the high water discharge structure should follow the following phases:

- the verification of the hydraulic and constructive parameters on site and in the execution project with the topographic layout of the longitudinal profile; - the comparison between the executed longitudinal profile and the projected longitudinal profile and the defining of the factors which influence the hydraulic regime; - the defining of the dimensioning and verification debits in accordance to the category of importance of the dam as well as the redrafting of the hydrologic study for the collecting surface of the storage;

- the verification of the hydraulic regime of the discharge structure for the initial operation conditions and for those modified;

- highlighting the loosening phenomenon and of the risk factors while defining the solutions for upgrading and modernizing required.

4. Conclusion

A partial or total modification of the initial functions of hydrotechnic system induces a series of new actions that can affect the safety in exploiting the system by inducing supplementary risk factors.

The analysis of the safety in exploiting the system is carried out through a hydraulically and structural type technical expertise which shows the systems behavior under new influences. In the case, the supplementary safety measures are also emphasized as well as measures for the technical rehabilitation and modernization of the hydrotechnic system.

The hydraulic verifications of the high water discharge structure of dams should be carriedout in accordance to a satndard type method which should be brought upt o date with the current technique and research in the field of hydrotechnics.

The analysis of the hydraulic regime should be carried out on the current longitudinal

profil of the discharge opening, taking into consideration the discrepaciens in the execution as well as the repair works carried out.

The hydraulic verification of the high water discharge structure of the Varsolt dam highlighted the substantial modification of the hydraulic regime on the channel transoms upstream of the rapid channel. The state of motion on the connector channel and the discharge channel is above critical, with high velocities, which velocities, which may induce risk factors in the flow of large debits.

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The reconsideration of Protection Engineering Solutions Implemented to Mamaia Beach

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Abstract: This work present certain field experiment's data, the integrat monitoring of bilded coastal zone and reconsideration of engineering solutions implemented for the protection and reabilitation of Mamaia beach. In the same time, the topics outlined represents the main branch of the initial analysis stage with a view to implement accessible protection and rehabilitation solution of the eroded littoral beaches, thus being the approach to data collection and evaluation, prior to modelling and considering the design-selection of the protection solutions. The work is made up of a case study including the preliminary analysis of the conditions and environment resources of the Mamaia beach southern area, including an experimental and analytical approach which is also determinative for the subsequent strategic projection and evaluation stages (i.e. impact, costs/benefits and scenario analyses) coming after the modelling phase.

Keywords: sediment budget, sediment transport rate, beach fill

1. Introduction:

Coastal zone dynamics, shore and sea zone is on a background of man made intervention, under the direct or/and indirect action of specific natural factors, from which the most important are considered wind, waves and marine currents.

The importance ensues of knowing the characteristics of the natural phenomena induced from this factors are crucial for the evaluation of the sediment transport process and the explanation of geomorphologic changes met at a certain moment.

The topic of the project includes the research extention for the contact sea-land litoral strip undering the erosion effect proces, manifesting itself with very strong intensities in the area Perla H. – Parc H.after the offshore breakwaters extention of the Midia harbor. At that time, the implementation of certain protection measures was required at very short dead-lines, thus making almost impossible the grounding research, according to the necessities and the concrete situations in the area. Acordingly, the protection systems implemented in Mamaia responded to the forecasts in different proportions. Thus, once the hydrotechnical system was implemented for beach ISSN-12223-7221 protection, I.N.C.D.M. has initiated detailed research for the quantification of coastal protection system effects, entailing some corrections for optimisation.

The main objective of this study is the evaluation of sedimentary transport processes, namely the determination of sediments longshore transport rates preliminary analysis of multi annual characteristic environmental conditions, and of the energetic coefficients on wave directions, which will be used for sediment longshore transport, using the integral methods(developed by C.E.R.C. USA) and field measurements, and also the preliminary analysis of complementary methods for protection and rehabilitation of southern sector of Mamaia Beach.

2. Experimental research regarding sediment transport; methods for protection and rehabilitation of eroded beach of Mamaia resort southern area

Study Zone Description

2.1. The Romanian littoral.

Stretching over 244 Km (between Musura arm and Vama Veche), the Romanian littoral represents 6% from the total length of the Black Sea shore. The landscape is format of low altitude shores, beaches (80%) and relatively high shores, cliffs (20%). From

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topologic point of view, includes both natural shore (beaches and cliffs - circa. 84%) and "built" shore, circa. 16% (harbors and hydrotechnic construction for protection).

The northern sector (Musura - Midia) with deltas, lagoons, and limans is formed form fluviomarine accumulation, recent shell sands, extended as beaches and low littoral belts, generally less than 2m.

The southern sector (Midia - Vama Veche) shows structural morphological characteristics of Dobroge., manmade relief, different from the northern sector form Midia. The shore is an alveolar type, with cliffs, mostly active, with heights reaching circa. 35m. The beaches formed at the base of the cliffs, are relatively stabile and have smaller dimensions. In front of all old mouth rivers or in front of old bays small beaches developed on fluvio-marine limans (Techirghiol, Costinesti, Tatlageac) or marine lagoons (Tasaul, Siutghiol, Comorova, Ieazerul Mangaliei).

The undertaken studies show that the Romanian shore is in a serious stage of erosion (circa. 60-70% of the shore line).

Between 1990-200, in the southern built shore, a change in the shoreline was recorded with a magnitude between +10m and -15 m, in which the erosive effect had a frequency of circa. 76%, which shows the fact the hard protection solutions didn't reach their initial goal.

2.2 The southern area of Mamaia beach

Positioned at the border sector between northern and southern littoral, Mamaia beach seen in the last 40 years an evolution that reflect the intensification and generalization of the erosion phenomenon, which is part of the general trend of the whole Romanian littoral, due to the stringent lack of sediment supply.

The causes of costal erosion are very well known, but both the natural (sea level rise, subsidence process, climatic changes) and manmade phenomena (building on dams on the Danube and rivers, offshore jetties on Sulina arm, extension of the harbor breakwaters at Constanta and Midia) can not be changed in the present.

Because of the reduction and deviation of longshore sediment transports the erosion developed an alarming character between 1966-1988, especially in the southern part of the littoral belt of Mamaia, endangering even the stability of Hotel Park's Olympic swimming pool. Thus, in order to stop and stabilize the local erosion phenomena solutions to construct a breakwater shore connected (hockey stick type) and an offshore breakwater dissipative system, were adopted between 1977-1979 and respectively 1988-1990 (Annex 1). The execution of the groin correlated with artificial nourishment done by dragging the sand from the lake Tabacarie (circa. 27,000 mc of sand), followed by the one of the longitudinal dissipative offshore breakwaters, also correlated with the artificial beach nourishment by dragging the sand form lake Siutghiol (circa. 500,000 ms of sand) resulted in salving of the beach areas previously lost(see photos)

In the Navodari - Mamaia sector (Cape Singol) base on the analysis of the balance of beach areas resulted the following general conclusions regarding the stage and the change trends of afferent beaches: the loss through erosion of 14.4ha and accretion of 9.7ha during the last 18 years - the overall damage being more important then in the previous, in which the protection was started.

2.3. The statistic characteristics of the winds and waves

The wind climate

The direction and speed of the wind were studied, from a statistic point of view, based on the daily recordings done in Constanta zone (Fig. 1). The wind frequency and the average multi annual speed on the main eight directions were calculated between 1971-1994 (Annual studies IRCM 1971-1994).



Fig. 1 Wind frequency distribution on the main eight directions

SE

s

SV

Е



Fig. 2 The variation of marine activity parameters.

Depending on the height value, the predominant waves were the ones lower than 1m. The average wave height was 0.8m, with an average period of 5 sec. - the most waves being the ones coming from NE and SE. The predominant direction is from east (circa 30% from the total waves) fact that shows a low energetic contribution of the hydrodynamic factors.

0

Ν

ΝE

2.4 Characteristics of the sediments in the southern area of Mamaia Bay

The verity of sediments is determined by the origin of the supply sources and that can be terrigeneous and organogeneous. Recently sediments can be added from littoral lakes (Tabacarie, Siutghiol) which were used to renourish Mamaia beach. Generally these sediments can be found mixed in various proportions, depending on certain factors, amongst the most important being the marine environmental conditions and the distance to the source of origin.

Spatial distribution of sediments with similar grainsize characteristics can be seen under different occurrences, the orientation which being relatively variable along a parallel direction with the shore, yet mirroring the transportation effect of longshore currents.

V

ΝV

In the coastal area, due to the quasi permanent mobility of the sediments, and as a result of the erosion / accretion processes, the sediments can be broken down as follows:

• coarse fractions are to be met in the vicinity of the shoreline;

• between the breaker lines and the vicinity of the shoreline the accumulation occurs as well as the disintegration of sedimentary material fragments, and in the wavebreak area the separation occurs of the medium - sized fractions;

• beyond the offshore limit of the surf zone the sand volume of the surface layer decreases, being replaced by silts and clay.

The main sources to take part in the formation of marine sediments in the Mamaia shallow zone can be grouped as follows:

• terrigeneous material, made of fine-grained mineral sand grey-coloured and originating from the Danube solid material and recently the sediments in the littoral lakes for the vicinity area of the shoreline;

• organogeneous sand, generally medium - sized, formed through the degradation of the mineral protective layer of marine organisms (shells, etc.)

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Grain size determination of marine sediments

Aspects are presented of the sediment texture of Mamaia beach southern area, as a result of the analyses of superficial layer samples of emerge and submerge beach of the H Perla sector (IPJ 3 P) analyses conducted during 1998-2000. The results of data analysis(sown in table no.1) were grouped in eight areas corresponding to the three characteristic segments of the beach profile (shore Protection Manual, 1984), typical of the three coastal areas: foreshore, inshore (1-5 m depth) and the offshore (5-10 m depth). The characterization of the sediments will subsequently be done according to the grainsize classes and the average diameter. *Sediment classes.* The spatial distribution of the fractions along dimensional segments vary from coarse to fine sediments as follows(in mm):

- B/P-G coarse pebble = 256-64;
- B/P-M medium-sized pebble = 64-4;
- B/P/F fine-grained pebble = 4-2;
- NFG very coarse sand = 2-1;
- NG coarse sand = 0,5-0,25;
- NM medium-sized sand 0,5-0,25;
- NF fine-grained sand 0,25-0,125;
- NFF very fine sand = 0,125-0,0625;
- SG course-grained silt = 0,0625-0,031;
- Mz median particle diameter.

Table .	1									
Class Section	B/P- G	B/P- M	B/P/F	NFG	NG	NM	NF	NFF	SG	MZ
Emerge beach	0	0	0	0.2	4.3	43.5	48	3.95	0.05	2.02(φ)/0.25mm
Swash Zone	0.5	7.5	3	1	4	30	51	2.95	0.05	1.49(φ)/0.47mm
Beach face	1	2	9	19	33	16	7	12.95	0.05	064(φ)/0.64mm
0.5m isobath	0	0.3	0.4	0.8	3.5	15	70	9.95	0.05	2.38(φ)/o.19mm
1m isobath	0.05	1	1	2	10	17	44	24.95	0.05	2.26(φ)/0.21mm
2m isobath	0	0	0.3	0.7	0.5	1.5	17	77	3	3.14(φ)/0.11mm
4m isobath	0	0	0.5	0.2	0.3	1	17	79	3	3.06(φ)/0.12mm
6m isobath	1	2	2	2	2	1	13	76	76	3.06(φ)/0.11mm

The area of the subversive beach, generally delineated by the depth of 1,0-6,0 meters across the entire studied sector, was characterized by great frequencies of the particles of a dimension corresponding to very fine / fine grained sand (0,25-0,0625) mm. The very fine-grained characteristic (0,125-0,0625 mm) is enhanced of the sediments nag particles, frequently going over 75% of the sediment.

The median particle diameter. Parameter of central trend distribution, show the medium kinetic energy(speed) of transport and sedimentation agent ($Mz\phi=\phi16+\phi50+\phi84/3$, Folk&Ward 1957); also it

is used in beach fill design. Generally, is revealed the gradual decreasing of medium size with the depth increasing.

2.5. The influence of the hydro-technical system over the emerge and submerge beach

The influence of the local particularities which consist in the isolation from the general littoral transport through the extension of both ports (Midia and Constanta) and also in construction of the protection hydro technical system of Mamaia beach resulted in a complex local model of development of the costal hydrodynamic process the effects being positive and negative as well, secondary. The research done through direct and indirect methods, based on data obtained form bathymetric survey and sediment samples a new deterioration of geomorphologic stage of Mamaia beach. In consequence, the space-temporal mobility of shoreline in the perimeter of southern sector of Mamaia Bay, along the area between the marks IPJ 2 and IPJ 3P, presented great variations in the last decade course(see photo). Thus:

• The mark IPJ 3P. The beach segment in front of Perla Hotel registered the greatest fluctuations of shoreline changes determinate by the man's actions. The artificial nourishment were induced the increasing of beach area, so in '89-'90 interval, the shoreline was extended seaward with annual rhythms included between 12,7 m/year(90/89) and 68,3 m/year(89/88). The strongest erosions were recorded in 1993 – 1995 interval, with changing rhythms between 13- 15 m/year.

• The mark IPJ 2. The beach situated in front of Dacia Hotel is under first offshore breaker's shadow. Also, this beach sector was influenced by the artificial nourishments, in '89-'90 interval, the shoreline was extended seaward with annual rhythms included between 8.9 m/year(91/90) and 36.5 m/year(89/88). In the second interval, '91- 98', have been predominate the erosive movements , with a multi annual rhythm of -3.7 m/year.

In the present, even if this sector at the finish of could season recorded the greatest decreasing of sediments stocks, the final stage at the finals of geomorphologic cycle is maintained in a relative equilibrium, due low environmental conditions in the last two years – there is no big storm recorded. Therefore, the variation of sedimentary stocks between 1992 and 2001, for the IPJ 3P 's section, have a maxim negative value, of -2.4 mc/ml.

The survey of submerged beaches surfaces in southern sector, constitute on the one hand the result of outside-zone sediment deficit, and on the other hand the result of local redistribution induced by coastal structures placement.

Therefore, the areas of submerse beach for this sector constitutes on one side the deficit of sediment supply from the outside that area and on the other side of local redistribution, generated by the protection system. The comparative analysis of the bathymetric maps: 1988 (prior to the protection measures) - 1998, leads to the following conclusions:

• The shore line, with few exceptions between marks IPJ 6 and IPJ 4 is still maintained in the accretion stage;

• The 1m, 2m and 3m isobaths are still in the accretion stage or relative balance;

• The 4m isobath is in the erosion stage between the offshore breakwaters A, B, C and D and in erosionaccretion stage between the offshore breakwaters E and F;

• The 5m isobath is in generalized erosion stage, with lower rhythm in the area of the last breakwaters, E and F.

The comparative analysis of bathymetric measurements from 1988, 1991, 2000 and 2001 has show that in the central segment landward from longitudinal breakwaters its were stabilized the "tombolo" formations with different orientations, determined by surf zone hydrodynamic conditions. In the same zone afferent the marks IPJ 2 and IPJ 3P, the losses of sediments from submerged beach, calculated until the 6m isobath for the 1990-2001 interval, have presented values between 288.89 mc/ml and 45.66 mc/ml

2.6. The calculus of energetic coefficients and longitudinal sediment transport rates on the wave directions

The sediment budget within a coastal zone compartment, such as is the sector under study, include the long-shore and cross-shore sediment transport rate , Q_1 si Q_n , aeolian transport rate, Q_e , sediments discharge from rivers and/or channels, Q_r and/or Q_c , eroded sediment rate form cliffs and coastal structures - the sum of all components Q determinate the erosion/accretion rate, in the other worlds, sediment volumes variations $V(\Sigma Q_i=dV/dt)$. Since for considered sector aeolian transport rate, Q_{e_c} is controlled by reed fence disposed, and solid suspension discharge, Q_{c_c} are relative small, in the following will be considered the sediment transport induced by marine hydrodynamics factors.

The littoral drift or the longshore sediment transport is in strong correlation with energetic aspects which characterizes the shallow water zone. In this section estimates of the sediment transport will be done based on longitudinal energetic flux consideration. This is an integral, qualitative method of study. This offers information regarding the sediment transport due to the longitudinal component of the energetic flux of wave, starting from the energetic flux conservation hypothesis, which is propagated in shallow water, used in small amplitude wave theory.

The general equation of long-shore transport rate calculus is:

$$Q_l = \int_0^h cv dz$$

where: c – sediment concentration;

v - longitudinal speed's component at z vertical coordinate ;

h – water depth.

For small tide shore and a small influence of bathymetric gradient on surf circulation is applicable C.E.R.C. formula for immersed weight of long-shore sediment transport rate:

 $Q_l = K E c_{g,d} \sin \theta_d \cos \theta_d = 0.025 C \sin(2 \theta_d)$ where:

 $-E = 1/8 \rho g H_d^2$ - wave energy at breaker line, - $K \approx 0.77$ (coefficient); - $c_{g,d} = n_g c_d = (gh_d)^{0.5}$ – wave group celerity at breaker line;

- c_o - wave group celerity at offshore,

- $c_o = T(g/2\pi);$
- H_d wave high at breaker line;

- h_d – water depth at breaker line;

- $\theta_{d/o}$ – wave angle at breaker line/offshore, sin $\theta_d = (c_d/c_0)\sin\theta_0$;

and C is the energetic coefficient, obtained after the following relation:

$$C = \frac{H_{d}^{2}L}{T} \left[1 + \frac{4\pi h}{L} \frac{1}{sh \frac{4\pi h}{L}} \right]$$

applied to data regarding wave parameter measure Constanta (Table 1), where:

- $H_{d/o}$ – wave high at breaker line/offshore,

$$H_{d} = \gamma \left[\frac{H_{0}^{2} c_{0} \cos \theta_{0}}{1.8 g^{0.5} \gamma^{2}} \right];$$

 γ - breaker coefficient, $\gamma = H_d/h_d \cong 0.8$;

L – wave length;

T – wave period.

d - depth at the measurement point.

Table 2. The average value of waves parameters over directions (%)

Dir Param.	Ν	NNE	NE	ENE	Е	ESE	SE
H (m)	0.74	0.75	0.73	1	1.2	0.8	0.6
T (s)	3.9	3.8	3.6	4.5	5.3	4.7	3
L (m)	19	18	17	26	34	28	16
F(%)	2	5	13	18	29	12	2

Based on energetic coefficient evaluation, in multi-annual marine activity conditions, it is observed that the waves coming form the north were predominant in determining the direction of the littoral deviation, followed by the ones coming from the east. (Table no.3). Considering the incident angle between waves and shore line ($\theta_0 = 22.5$ degrees for the study sector) it can be observed the maintaining of sediment transport resultant oriented longshore towards south.

Direction	Ν	NNE	NE	ENE	Е	ESE	SE		
С	2.7151	2.6990	2.5393	6.2224	10.933	4.1975	1.9317		
Q(mc/s)	0.0248	0.0387	0.0231	0.0493	0.0137	0.0132	0.0026		
Q situ(mc/s)				0.0307	-0.0022		0.0073		
0.030754									
-0.00222									
0.007344									

Table no 3. Energetic coefficient values, C, over waves directions

The calculus of the coefficient values shows that the most important sediment transport is from the northern sector. A significant value was obtained for values from the east direction. Overall, considering the coefficients determined by the waves from the other directions resulted the value 11.07 for the report between the long-shore and cross-shore sediment transport.

In addition, the physic signification of negative transport rate obtained in the third is that of one rip current identified as a concentrated jets from the surf zone to offshore zone seaward.

2.7 Evaluation of sediment transport rate

Marine circulation characteristics afferent to Mamaia beach southern area in the present are different from natural shore, unprotected by coastal structures. The diffraction and decreasing of wave highs through longitudinal breakwaters fetched a attenuation of marine hydrodynamic processes, and implicit a complicated sediment transport capacity.

In previously mentioned conditions it is very important to know the currents climate, specially near seabed, because the sediment transport in the marine environment is the result of wave and currents interaction, and essentially depends on local components of speed's flow. In consequence it is required to know the direction and size of speeds and bed sheer stress in the marine area of study.

The uniform stationary(turbulent) flow is described through the equation of movement:

 $\tau_z \Delta x \Delta y = \rho g (h - z) \Delta x \Delta y \sin\beta,$

where : τ_z –, bed shear stress, $\tau_z = \rho vv(du/dz) - \rho u`w`;$ x,y,z – coordinates; u – medium speed of fluid;

u`, w`- instantaneous speed components;

v - kinematic viscosity coefficient;

ρ - fluid density;

 β - slope angle.

As a result, data regarding the speed and flow directions in vicinity of seabed, registered with current-meter in specific points in relation with components of coastal structures system(see annex no.1) show a no-stationary low climate(different even between two station of the same breakwater). Thus, the values of currents speed have been obtained between 3.6 and 13.1 cm/sec

Data collection was done over three experiment sessions in order to obtain a better extension of bed-load transport in the surf and offshore zone of Mamaia beach southern area(see Annex nr.1) in tree different wave climate:

I – moderate swell \rightarrow EEN, H=0.6-0.75m;

II – smooth wind wave \rightarrow SE, H=0.25-0.4m;

III-intense wind wave \rightarrow NE, H=1.3-1.5m.

The choose of representative sampling sites and the number of measurements at each locations have been done after some general requirements, such as wave climate, depth and topography of bottom, placement of artificial obstacles, sampling duration and project resources.

The duration to prevail one sampling was estimated as 0.5-1 hour, depending on volume of nylon bag and blocking of the nylon material by sediment particles.

The dry mass of samples was obtained by quantitative determination and weigh with analytical balance.

The methodology of data analysis has included the local bed load transport rate determination - under assumption made in the calculation: that the transport

of material in suspension was to be included, this is because of high suspended sediment concentrations present near the seabed.

The local sediment transport is related by following relation:

$$s = s_s + s_t = U_s C_s + s_t = u_s c_s + u_s c_s + s_t$$

where:

s_{s/t}- time averaged sediment transport: suspension load/bed load;

U_s-instantaneous speed of sediment particle;

C_s – instantaneous concentration;

 $u_s u_s - time$ averaged speed, respectively pulsate speed at depth z;

cs cs -time averaged concentration, respectively pulsate concentration at depth z;

Helley-Smith sampler were selected due to use in bed-load sediment transport widely measurements, together with current meter(see photos).

Bed-load sediment transport rata(in Kg/m s) can be determinate by :

$$s_t = \frac{\alpha (1-p) \rho_s V_s}{bT} = \frac{\alpha G_s}{bT}$$

where:

 α = calibration factor =0.5÷1.5 in function by particle size of sediment;

 ρ_s =density of sediment particle ($\cong 2650 \text{kg/m}^3$);

 $p = porosity factor(\cong 0.4);$

 $V_s = immersed volume (m^3);$

 $G_s = dry$ weight of sediment catch (Kg);

b = intake opening width (=0.0762m);

T = sampling duration.

The values for the rate determinate in situ have been included between 0086÷0.05409 mc/s. The results regarding the median values obtained are presented in figure below; the variations between calculate and measured rate are evident.

2.8 Solutions for rehabilitation of Mamaia beach

The rehabilitation of Mamaia beach southern area, in the actual context of resort emplacement and the past experience of implementation, is necessary to include the reconsideration the artificial beach nourishment as a potential form, generally accepted, for shore protection.

Even if, at this moment there is no method known to stop beach erosion, the beach fill solution until a optima width respectively a new equilibrium profile, temporary protect the shore against waves and currents action, though it not change the causes of erosion. Thus, the beach renourishment as a soft technical work, will be considered, in a fesability manner, a potential desirable form of shore protection, due to the main advantages: the envisaged outcome can be obtained immediately, together with the material supply, and also the lack of negative side effects of the neighbouring areas.

According to Shore Protection Manual ('84), the required volume for nourishing of a beach linear meter is:

V / length unity = (b+h) ywhere: b = beach berm high;

h = depth of closer; $h = 1.5 H_{S0.137}$;

 $y = beach length(y_{min} = 30m),$ 21

and

$$y = \frac{s(h/A)^{3/2}}{(h+b)}$$

in with :

s = median sea level storm; s(max)=1.05m

A = parameter, related to median size diameter,

 $A = 0.21 \text{ Mz}^{-0.48}$ or $A = 0.067 \text{ w}^{-0.44}$, and $w = 14 \text{ Mz}^{-1.1}$ is following speed of particle.

For shore with oblique wave attack have been determinate the following coefficients(Fufrboter):

- $K_3 = Vn/Ve$ - rapport between required volume and erosion compensation volume. As ideal is $K_3 = 1$; $K_3=0.7-2.5$ and is >in 50% of projects.

 $-K_4 = V_{e, before} / V_{e, after fill} = rapport between erosion$ volume before and after beach nourishment;

 $K_4 = 0.3-2 < 1$, for 80%. Supplementary for compensation is necessary a quantity increased of sand with 20 - 50%.

The estimations for Mamaia southern area, IPJ 2 - IPJ 3, are:

-the averaged high of beach berm: 2.2 m;

-the median diameter: $Mz = (0.21 \div 0.35)$ mm; -the closer depth: $h \approx 2.5$ m;

-maximum storm level: s = 1.0m; -parameter A = 0.09 - 0.21

It will obtain: $y(min) = [1.0 \ (2.5 \ / \ 0.09)^{3/2} \ / (2.5 \ + \ 2.2)] \approx 30 \ m$, and total minim volume required: $V_{t\ n} \approx (2.2 \ + \ 2.5) \ 30 \approx 135 \ mc/ml$, circa. 270 mc/ml for $K_3 = 1$.

For verification were used GENESIS program, on base of following assumptions:

-the beach fill have the same median size particle with that of native;

-the profile of artificial have equilibrium profile features correspondent to particle size – mathematic decrypted by: $h = A x^{m}$, where x – horizontal distance;

-the berm high of beach nourished is the same with that of natural.



For section case IPJ 2, presented in figure no. 4, and :y = 35 m, b = 2.1 m, h = 2.5 m, A = 0.09, required minim compatible sand volume is minim de material in perimeter afferent of section

IPJ 2(figure nr.4) , will be 169.75 mc/ml(circa. 340 mc/ml for $K_3). \label{eq:k3}$



Fig.4 Comparative profiles - section IPJ 2(H.Dacia)

3. Results and discussions

The results show that the southern area of the Mamaia beach is strongly influenced by hydro technical works, breakwaters, which induced important changes of the beach profile.

Currents have a different regime if compared to natural, unprotected shores. Diffraction and

attenuation of the waves height, due to the breakwaters parallel to the shore, decrease the sediment transport. Currents speed and direction recorded near the bottom emphasize a no uniform, quasi-stationary regime. Therefore, medium transport rates determined "in situ" are different by those calculated by integral methods. The beach fill is a proper solution in this case, of the Mamaia beach southern area, due to position of the protection hydrotechnical system. For a better protection, some urgent measurements are required, involving the principle "working with nature", as a recommended solution to ensure beach equilibrium. Clear regulations are required as well, to prevent anthropogenic influence that might alter the natural mechanisms of beach rehabilitation and an unique action plan of the institutions involved in shore protection and beach management.

4. Conclusions

Both natural and manmade factors exerts an important influence on the hydrodynamic climate of Mamaia Bay, maintained in present the erosion processes. The single solution possible to realize in actual conditions is the artificial beach nourishment, which could compensate the rates of sediment loss.

The futures researches will be concentred on the mathematical modelling , as a scientific and economic tool in the study of beach protection works. On the other hand, as a main tool of the Romanian integrate coastal management, a program for surveillance and management of the beach natural resources will allow a better utilization of the sand, as generated by natural populations of molluscs and existing sedimentary deposits. It is an immediate priority, adoption of clear regulations and solutions, based on a coherent management plan, regarding the use and protection of the beach natural resources.

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Hydrodynamic Loads on Offshore Platform Structures

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Abstract: This paper presents an approach to the problem of wind, wave, and current loads acting on the structure of the fixed offshore platforms, using experimental and theoretical methods. A mathematical model, for computation of wind forces on the above-water superstructure of the offshore platforms, and for computation of wave and current forces on infrastructure, in particularly simplified assumptions, is briefly presented. The paper presents also the experimental results obtained in the wind and wave flume for a case study: the fixed offshore platform PFC-III, designed for the Romanian zone of Black Sea continental shelf. A scale model 1:40 was used for the measurement of wind forces acting on platform superstructure, and a scale model 1:20 was used for wave force tests. Some results of tests are presented in graphical form. Considering the obtained results, the paper stresses the need to carry out both theoretical and experimental studies, for the design, in safe and optimal conditions, of the offshore platforms.

Keywords: Offshore platform, wind load, wave load, hydraulic model, drag coefficient, inertia coefficient.

1. Introduction

Special efforts, concerning the design and construction of offshore platforms for drilling, production, and storage, are necessary for the extraction of hydrocarbons in continental shelf of the Black Sea. Adequate designing should be done in order to obtain a workable and economical offshore platform to perform the given function. The accurate evaluation of hydrodynamic forces (wind, wave and current load) on the structures of platforms is very important for a proper design of these platforms.

Determining the hydrodynamic loads is a problem with practical importance and with large amplitude of applications. A vast literature of this domain exists at present, including both theoretical and experimental significant results. However, the diversity of the oceanological and geological conditions enforces to continue the development of the hydraulic research for offshore platforms, with the purpose to realise a minimum risk, taking into consideration the disastrous consequences on the environment due to an eventual accident.

In the last 25 years, within the Hydraulics Department of the Research and Engineering

Institute for Environment (the former Hydraulic Engineering Research Institute Bucharest) many theoretical and experimental studies were carried out for the determination of hydrodynamic loads on offshore platforms. Some results of these studies are presented in the paper. The platforms were built on the Romanian zone of the Black Sea continental shelf, at depths between 30 and 90 m.

Considering the obtained results, the paper stresses the need to carry out both theoretical and experimental studies. In this way, it is possible to evaluate correctly the wind, wave and current loads for the design, in safe and optimal conditions, of the offshore platform structures.

2. Specific studies carried out

Within the Hydraulics Department of the Hydraulic Engineering Research Institute (ICH Bucharest) - at present Research and Engineering Institute for Environment - theoretical and experimental studies, for the determination of environmental loads (wind, wave and current load) on offshore platform structures, were carried out.

2.1. Model-testing equipment

Usually, the simulation of the environmental factors acting on the offshore platforms, in storm conditions, is made, using scale models, in the wind and wave flumes specially equipped. These flumes joint the specific characteristics of wave tanks and wind tunnels. In this way, it is possible to determine not only separated wind and wave loads but also the global loads on the structure of platform due to combined action of wind and waves.

In order to facilitate the research activities in this field, an experimental base has been provided in ICH Bucharest, with the equipment necessary for the tests as well as with the measuring and data processing devices. Two wind and wave flumes were equipped, having the following characteristics:

a. The little flume:

- Dimensions: 60.00 m length, 2.00 m width, 2.00 m height, 0.80 m maximum water depth.
- Wave generator: mechanical type, which carries out uniform waves with maximum 0.30 m height and with periods between 0.5 and 3.0 s.
- Wind generator: a battery of three air-blowers in open circuit. The air discharge is 240000 m³/h at a pressure of 60 mm water column. Maximum wind speed is 20.2 m/s. In addition, the wind generator can produce waves with maximum 0.20 m height and with periods up to 1.5 s.
- b. The big flume:
 - Dimensions: 65.00 m length, 7.00 m width, 4.70 m height, 4.00 m maximum water depth.
 - Wave generator: mechanical type, which carries out uniform waves with maximum 0.60 m height and with periods between 0.7 and 5.0 s.
 - Wind generator: maximum air discharge is 700000 m³/h and maximum wind speed is 20.0 m/s. Also, the wind generator can produce waves with maximum 0.30 m height and with periods up to 2.5 s.

For measuring the forces and moments on the model, specific transducers were used; these transducers were integrated in measurement chains. Therefore, for the measurement of the aerodynamic forces on cylindrical elements, the experimental assemblage includes two bi-directional force transducers, which join the cylinder that the wind load is measuring, and a frame that allows desired inclination of cylinder axis. The measuring equipment (first generation) consists of two force transducers (resistive type), an electronic bridge, and an oscillograph with photosensitive paper. For measuring the wind resultant force overall platform superstructure, the model is hung by wires, in wind tunnel. The horizontal force caused by the wind is measured, in this case, with three resistive transducers.

For measuring the hydrodynamic forces produced by waves, were used triaxial tensiometric transducers. These transducers were integrated in a measurement chain of second generation. The forces and wave heights were recorded simultaneously on an ultraviolet recorder. For some of the tests the forces and wave profile were also recorded on a magnetic tape recorder and data processed on a computer.

2.2. A mathematical model for determination of the hydrodynamic loads

In the Hydraulic Engineering Research Institute from Bucharest a mathematical model for determination of the forces on offshore platform structures, due to the wind, wave and marine current, was developed. With this model, it is possible to determine the wind, wave, and current load on members and on joints of platform structures, even for very complicate shapes of "template" platforms. The model integrates the elementary hydrodynamic forces calculated for every tubular member of the structure.

Offshore platforms for drilling, production, and storage of hydrocarbons can be classified in fixed platforms and mobile platforms. The fixed platform types are the following:

- template-type platform, consisting of a jacket or welded tubular space frame which is designed to serve as a template for pile driving, and as lateral bracing for the piles;
- tower platform, which has relatively few large diameter legs, so that the tower may be floated in location and placed in position by selective flooding;
- gravity platform, with massive infrastructure (usually reinforced concrete).

The most indicated fixed platforms, for meteorological and hydrological conditions of Black

Sea, are the template-type platforms. The selfcontained template platform consists of a large multileveled deck structure supported by long piles driven deep into the sea floor. The template (the jacket) is a three-dimension welded frame of tubular members and is used as a guide for driving piles through the hollow legs of the jacket.

For the structure elements of simple geometric shape, wind force can be calculated using semiempirical formula as:

$$F_w = \frac{1}{2}\rho C_D U^2 A \tag{1}$$

where ρ is the air density, C_D is a drag coefficient (aerodynamic coefficient), U is the wind speed, and A is the projection of the surface of the frame on a plane perpendicular to the wind direction [1].

It is necessary to know the wind speed vertical distribution U(z) and the value of the drag coefficient C_D in formula (1). This coefficient is determining by experimental methods; its value depends on: the flow Reynolds number, the shape of the cross-section of the frame subjected to the wind, the roughness of the body, and the influence of near objects (shielding effects).

Generally, the wind intensity is variably in time and in space. On length scales typical of even large offshore structures, statistical wind properties (mean speed and standard deviation of speed, for example) taken over durations of the order of one hour do not vary horizontally, but do change with elevation (profile). Within long durations, there will be shorter durations with higher mean speeds (gusts). Therefore, a wind speed value is only meaningful if qualified by its elevation above sea level and duration. A reference value U_0 (the "standard wind") is the 1 hour mean wind speed at an altitude of 10 m above sea level.

The wind loads on the structure of an offshore platform are dynamic in nature. In the case of the fixed offshore platforms like template platforms (the most indicated for meteorological and hydrological conditions of West Black Sea), wind loads are representing up to 10% of the whole environmental loads, but its importance is not negligible for the static and dynamic analysis of these platforms. A dynamic analysis is required when the predominant frequencies of the wind energy spectrum are nearly to natural frequencies of platform. Such analyses may require knowledge of the wind turbulence intensity, spectra, and spatial coherence.

The mathematical model, based on integrating in equation (1) for each simple structural element, allows to determine wind loads on members and on joints of platform superstructures, in any case of orientation in space for these members.

Concerning the action of waves, there are two major wave-induced forces exerted on structures. The drag force F_D is due to frictional and form drag; its magnitude depends on shape, roughness of the object, Reynolds number of the flow, and intensity of turbulence in the flow. The inertia force F_I is due to water-particle acceleration. It is assumed in practical application that the total wave forces acting on a structure can be obtained by linearly superimposing the drag and inertia forces. This is the basis of the well-known Morison, Johnson and O'Brien formula [2], [3], which gives the wave force vector dF acting on the cylinder ds, as:

$$dF = dF_D + dF_I = \left(\frac{1}{2}C_D D\rho u \mid u \mid + \frac{\pi}{4}C_M D^2 \rho \dot{u}\right) ds \quad (2)$$

where ρ is the mass density of water, u is the horizontal water-particle velocity, \dot{u} is the horizontal water-particle acceleration, and D is the diameter of the cylindrical element. The hydrodynamic coefficients (the drag coefficient C_D and the inertia coefficient C_M) are usually determined by experimental methods.

Integrating in Morison's formula (2), the resultant wave force on the entire cylinder is obtained. The mathematical model takes into account the variation of water-particle velocities and accelerations with depth. The values of these accelerations and velocities are computed by using the Stokes fifth-order approximation wave theory [2], [4]. Finally, for every joint of the structure, the total load results by algebraically summing the components of the wave force on all the bars, which are intersecting in this joint.

The mathematical model is materialised by a computer program, which is composed by a main segment and nine subroutines. The loads (wave forces) are written into an output file, which may be processed later, for the static and dynamic analysis of platform structure.
2.3. Offshore platforms studied

Many theoretical and experimental studies were carried out, since 1975, for the offshore platforms built on the Romanian zone of the Black Sea continental shelf. The most important offshore platforms, theoretical and experimental studied in ICH - Bucharest, were:

Name	Characteristics		Location	Total	Total
	Legs	Wells	depth	height	weight
Sinoe Vadu I	4	6	-37 m	52 m	4780 kN
Sinoe Vadu III	6	12	-51 m	66 m	8840 kN
PFC III	8	18	-52 m	70 m	15300 kN
PFS Sinoe	4	6	-39 m	54 m	4950 kN
PFS + A	10	16	-46 m	65 m	10600 kN
PFS 2	6	6	-50 m	68 m	8530 kN
PFS 3	8	18	-50 m	68 m	11850 kN
PFS + 4M	10	18	-52 m	70 m	15650 kN

It is important to specify that no significant incidents and accidents were recorded for these platforms.

3. A case study: PFC III platform

The fixed offshore platform PFC III was designed for extraction and storage of hydrocarbons, supporting 18 wells. This platform is located in west zone of the Black Sea, at 52 m depth. Its infrastructure is composed by an eight-leg jacket; total weight is 15300 kN, and total height is 70 m. The maximum diameter of the legs is 1200 mm.

3.1. Experimental results

Model tests, for fixed platform PFC III, were carried out in the wind and wave flume of the ICH-Bucharest, including measurements for the determination of wind load on the entire platform superstructure [6], [7], [8]. The model is a 1:40 linear scale representation of PFC-III superstructure. The multileveled decks, the legs, the main modules (production and living), and the derrick, were modeled (Fig. 1). Two configurations of superstructure were examined:

without derrick tower (projected area of the model being 0.30 m²);

• with derrick tower (projected area of the model being 0.39 m²).



Fig. 1. The model of the PFC-III superstructure in wind tunnel

The measurement results are presented in Fig. 2: the drag coefficient C_D as function of Reynolds number. The relatively large variation of the drag coefficient C_D may be also caused by specific conditions of model tests (vibrations due to model hanging by wires).



Fig. 2. Measurement results - drag coefficient C_D as

function of Reynolds number

Using a scale model 1:20, a series of laboratory tests on wave forces, acting on fixed offshore platform PFC III, were conducted (the model of this structure in wave tank is shown in Fig. 3).



Fig. 3. The model of the PFC-III structure in wave tank

Both horizontal and vertical components of wave forces were measured, taking the effect of the buoyancy variations on the vertical components into consideration. The correlation between the maximum wave forces and maximum wave heights obtained in our model tests is presented in Fig. 4. The horizontal components of wave forces, which are shown in Fig. 4, increase parabollically against the wave height.

Figure 5 shows a typical example of the relation between wave force coefficients and Reynolds number, which was calculated from the maximum orbital water-particle velocity at still-water level and the diameter of the jacket legs:

$$Re = U_{max} D/\nu \tag{3}$$

Generally, this relation is roughly the same for all the directions of wave propagation tested on model, the inertia coefficient C_M being constant at about 1.5 - 1.8 and the drag coefficient C_D tending to decrease as Reynolds number increases.



Fig. 4. Correlation between maximum horizontal wave force and wave height



Fig. 5. Wave force coefficients as function of Reynolds number

A better correlation was obtained between wave force coefficients and Keulegan-Carpenter number [5], defined as:

$$K_C = U_{max} T/D \tag{4}$$

where *T* is the wave period. This correlation, presented in Fig. 6, shown that inertia coefficient C_M is relatively constant and the drag coefficient C_D tends to peak at a Keulegan-Carpenter number about 25.



Fig. 6. Wave force coefficients as function of Keulegan-Carpenter number

The theoretical and experimental results presented herein correspond to regular wave conditions in the range of 5.0 to 14.0 s period and 6.0 to 24.0 m wave height [9], [10].

3.2. Theoretical and experimental results applied to platform design

- For static and dynamic analysis of platform structure, the mathematical model has provided wind, wave, and current loads in all joints of the structure. Directly, by experimental investigations on hydraulic models, the global force and the overturning moment were determined, for different values of the design wave.
- For fatigue analysis of platform structure, the mathematical model has provided wave loads (at different values of wave height and period) in all joints of the structure. These loads were utilised as input data for the spectral model of the fatigue analysis.

4. Conclusions

The paper presents some results concerning the theoretical and experimental studies carried out, since 1975, for the offshore platforms built on the Romanian zone of the Black Sea continental shelf.

A mathematical model, for computation of wind forces on the above-water superstructure of the offshore platforms, and for computation of wave and current forces on infrastructure, in particularly simplified assumptions, is briefly presented. This model can compute the environmental loads on every structural member and on joints of platform structure. The paper presents also the experimental results obtained in the wind and wave flume for a case study: the fixed offshore platform PFC-III, designed for the Romanian zone of Black Sea continental shelf. Some results of tests are presented in graphical form.

Considering the obtained results, the paper stresses the need to carry out both theoretical and experimental studies. In this way, it is possible to evaluate correctly the wind, wave, and current loads for the design, in safe and optimal conditions, of the offshore platforms.

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Works for Waste Management Improvement in Constantza Port

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Abstract: The activities in Constantza Port generate problems regarding environment pollution. It is necessary to achieve a sanitary system for waste management that will assure the compliance with Romanian legislation regarding environment protection, as well as with the provisions of International Convention MARPOL regarding the prevention of pollution by ships. This system will consist in an installation to incinerate potential infested waste, a sanitary landfill to dump solid waste, a treatment plant for wastewater containing hydrocarbon and a specialised ship to collect waste from the port basins.

Keywords: Waste management, sanitary landfill, percolate water, hazardous waste

1. General presentation:

An important and diverse traffic of goods is operated in Constantza Port. These goods consist of grains, chemical products and fertilizers, mineral products, ferrous and non-ferrous ore, coal and coke, crude oil and oil products, cement, general wares, goods in containers, etc.

The amounts and the types of port waste to be collected, treated and stored are directly related to the port activities, goods traffic, and ships transiting the port. About 30,000 tons of solid waste and 450,000 m3 of liquid waste are annually registered in the port.

In the present situation, the facilities of waste collection, treatment and storage, as well as the organisation of these activities, do not comply with Romanian Legislation regarding environment protection, neither with the provisions of the International Convention MARPOL regarding pollution by ships

The bottom of the existing waste dump is not protected against the penetration of pollutant liquid and therefore against the pollution of the soil and underground water. The potentially infested waste collected from ships is neutralised and free burned inside a special precinct of the existing waste dump, polluting this way the atmosphere and the underground.

Technological residual water, rainwater from the platforms of oil and chemical products warehouses and ballast and washing water from oil tankers are collected, stocked and treated only by gravitational separation in the stocking tanks and in the existing settling basin. After the separation of the part with hydrocarbon, the residual water is discharged in the sea. The pollutants contained in this water have values exceeding the values admitted by the regulations for water discharged in the sea.

2. Proposed solutions

With a view to the requirements of the International Convention for Prevention of Marine Environment Pollution due to Ships – MARPOL 73/78, the Strategic Plan for Waste Management in Constantza Port has been drawn up in 1998, as a part of the waste management improvement project, financed by the Dutch Ministry of Economical Affairs.





Fig. 1. Ecological system organization

From the analysis of waste types, waste quantities, the legislative regulations and the observed shortcomings, the Study proposed the improvement of port waste management and the completion of the existing facilities, as follows:

• Domestic waste resulted from ships, hospital waste from the town hospitals and port hospitals and solid waste collected from the water surface will be incinerated in an incinerator, according to the MARPOL Regulations.

• Domestic waste collected from the port platform, technological waste from port companies, street waste and hazardous waste will be stored in an ecological landfill. Hazardous waste will be stored under special conditions.

• Dirty ballast water, liquid waste collected from the water surface and water resulted from oil tanks washing will be treated in a wastewater treatment station, where modernization and completion with extra steps of physical-chemical and biological treatment will be realized.

• The collection of bilge water, wastewater from ships and liquid waste from the basin water surface will be done in a special collecting ship.

Taking into account that the ecological system will be earliest achieved in 2001 - 2002 and that this one has to assure the requirements on the following 10 years, the components of the ecological system have been dimensioned considering a traffic increase, respective an increase of the collected waste(Fig.1).

3. Incineration Installation

The Study proposes the incineration of potential infested waste. The environment protection rules forbid the storage of these waste categories.

Incinerating is a procedure of thermic treatment of high temperature by which waste is transformed in gases and non-combustible solid residues. As principle, an incineration installation works with two steps of combustion, at a temperature of about 1000 °C, in separate chambers equipped with burners, secondary air fans and installations for the control and the automatic adjustment of the burning.

Resulted flue gas will be passed through a secondary combustion chamber where potential pollutants from the gas are totally destroyed. The flue gas cleaning system consists in: bag filter, secondary

quench unit, wet scrubber unit, induced draught fan and stack.

By the construction parameters and the use technology of the incinerator, the emissions of pollutant substances resulted from the waste incineration will comply with the admitted limits established by the Decision no. 462 of MWFEP.

The ash is evacuated through a grill with rolls in special containers that will be transported at the landfill.

Optionally the incinerator can be equipped with a boiler for recovering and use the heat resulted from the flue gases.

After analysing different offers, the conclusion was that a small incinerator operating 7000 hrs/year in shifts continuously is preferred (lower costs) above an incinerator with a higher capacity operating 2000 hrs/year in 1 shift.

For the weighing and the monitoring of the waste amounts arrived inside the precinct of the ecological system the present documentation proposes to mount a weight-bridge of 40 tons for the waste transport means.

Concrete platforms were foreseen for the handling of waste to be incinerated, waste not having direct contact with the ground.

The platforms for the incinerator and the sanitary landfill are grouped together so that occupied area will be minimalized.

The surfaces that will be realised of concrete are dimensioned taking into account the waste amounts, the transport and work equipment and the necessary spaces for the working of the Ecological system. The occupied surface will be of about 6425m^2 .

The entire precinct will be enclosed with a fence of panels and concrete pillars. On the outline a vegetal curtain of brush will be realised in order to mask the system precinct and to protect the neighbourhoods.

The constructions and utility networks necessary for the incineration installation working are alsa foreseen (installations and electrical, water supply and sewerage networks).

4. Landfill

Description of works for the landfill

The new landfill capacity assures the waste storage for 5 years and the landfill occupies a surface of 32.000 m^2 from the port territory.



Fig. 2. Incinerator working scheme

A waste quantity of 33.200 tons/year is forecasted to be stored.

For the realisation of the ecological landfill the following main works are necessary:

• Waterproofing of the waste storage bottom in order to avoid ground- and groundwater contamination;

• The entire land surface affected by the landfill achievement will be waterproof with a clay layer of 50 cm thickness, which will be realised at the foreseen levels and slopes. Over the clay layer the surface will be protected with a geomembrane of high density polyethylene, with a thickness of 2 mm, protected by a geotextile.

• Collection and treatment of percolate water (rainwater transiting the waste landfill and water resulted from the waste fermentation process).

On the waterproofed surface a draining layer of monogranular gravel (16 - 32 mm) will be laid on. Its achievement will be correlated to the works necessary to the realisation of drains for the collection and removal of the percolate water. The perimetric drains will be realised at the same time with the dams foreseen for limiting the waste landfill. The drains of ϕ 200 mm (ϕ 300 mm for the collector drain) will be realised of perforated tubes of high density polyethylene.

The slopes of the waterproof and of the draining layer as well as the slopes and the sections of the drains assure the collection and the transport of the percolate water towards the settling basin with oil separator, which is placed outside the ecological landfill territory. The settled water is directed by pumping to the port sewerage.

• Collection and evacuation of gases resulted from the anaerobic processes of the organic substances contained in waste.

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If the necessary humidity is maintained, pursuant to the reactions that take place result mainly methane gas and carbon dioxide, which lead to the danger of explosion in the landfill zone. Over the last soil covering a sand draining layer of 30 cm thickness is foreseen for gas collection.

• Landfill covering in order to avoid the contact with waste and the air contamination in the landfill zone.



Fig.3 . Landfill working scheme

Over the gas draining layer a waterproof clay layer of min. 40 cm thickness, which will be continued also on the landfill slopes, is foreseen. The clay layer will have variable thickness and it will create the slope of 1,5 % necessary for the flowing of rain water from the ecological landfill.

Over the clay layer the entire landfill surface will be covered with a vegetal soil layer, which will be grassed. The works for the landfill covering will be realised according as the landfill storage height is completed.

• The collection of the rainwater falling on the landfill will be realised by a perimetric gutter, disposed along the landfill. The rainwater will be directed by pumping in the port sewerage towards the treatment station.

The waste is discharged in the landfill, levelled and compacted. The perimetric dams limit the storage surface. The waste is disposed in successive horizontal layers and platforms for the access of transport means are created. After each waste layer of about 2,00 m thickness a soil layer of about 25 cm thickness will be laid.

Hazardous waste storage near the landfill

For the storage of hazardous waste of chemical nature the Study proposes to realise a special reinforced concrete store.

In this concrete store, the barrels or containers filled with the hazardous waste will be placed.

The store is divided in several subsections, so that the stored waste can be registered and each subsection can be covered shortly after disposal.



Fig. 4. Cross section of the landfill

Works at the existing dumpsite

As soon as the new sanitary landfill is in operation, the existing dumpsite must be closed. The

works for closing this dumpsite in an environmental-safe way are:

- gathering the random waste and compacting;

- equalizing;

- covering top and sides with geotextile;

- clay layer;
- rainwater collection system;

- placing of monitoring filters for groundwater.

5. Wastewater treatment plant

Technological and rain wastewater result from the activity of oil and chemical products charging – discharging. In order to prevent the environment pollution this wastewater has to be treated so that the effluent discharged in the sea complies with the values in the norms regarding the overall and specific pollutants.

Another activity producing residual water is represented by the ballast discharging and the washing of oil tankers.

The wastewater amounts forecasted for the year 2010, for which the constructions, the installations and the equipment of the treatment flow have been dimensioned, are:

- ballast and washing water	240.000 m ³
- technological water	180.000 m^3
- rain water	<u>100.000 m³</u>
Total	520.000 m^3

At present the residual water is stocked in two tanks of 10.000 m^3 each one. It is gravitationally separated in the existent settling basin, which has an useful volume of 1.400 m^3 and works with four of the five compartments, one of them being spare.

After the settling of the residual water and the collection of the superficial layer with hydrocarbon, the water is discharged in the port basin. Before the outlet in the sea the effluent pass once more through a settling basin where superficial residues are manually collected.

The values of the overall and specific pollutants that characterize the pollution degree of the water evacuated from the existent installations are over the values in the norm NTPA 001/1997:

- CCOCr 1,3 1,8 times
- CBO5 4-30 times
- particles in suspensions 2 70 times
- extractable substances 3 14 times
- phenol compounds 12 30 times

• ammonium 13 – 100 times The proposed treatment includes:

- upgrading of the existing settling facility;

- the use of a range of coagulation/flocculation and conditioning chemicals, DAF and biological treatment (anaerobic and aerobic).

Choice criteria are process efficiency, aspects of operation and maintenance, versatility of the treatment, environmental impact assessment, cost aspects, local circumstances and availability of skilled personnel.

Regarding environmental impact, the impact of the use of the wide variety of considered chemicals (coagulants and conditioners) and also the resultant sludge production has to be considered. The treatment aims at achieving a positive environmental effect, however, different technologies result in different sludge production patterns.

In order to establish good performance of the chosen technology it is necessary to perform experiments at micropilot (pilot) level in real conditions.

The proposed physico-chemical treatment (coagulation/flocculation, conditioning and DAF) and biological treatment (anaerobic and aerobic) for the treatment of the specific wastewater streams of the oil terminal in Constanza Romania can generally be considered as one of the appropriate treatment concepts for this type of waters.

Technological flow description

1. *Physical – chemical step*

The first stage consists in maintaining and to improving the gravitational separation existing flow. The tanks of residual water stocking will be equipped with the foreseen automations in order to control the level and to adjust the discharged flow.

In order to improve the separator's functioning, it is proposed to add cationic polielectrolyte in its distribution pit. The dosing of the 0,1 % solution of cationic polielectrolyte will be done related to the flow, by means of an ultrasonic flow-meter coupled to the dosing pump.

The mixing with polielectrolyte will be realised with an electric mixer with prop, mounted on the distribution pit.

The second stage of the physical – chemical step proposes the following technological flow:

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- The effluent of the existing separator API will be taken over by pumping to the repression pipe, ϕ 1000 mm.

- On the repression pipe 10 % solution of sulphuric acid and 10 % solution of coagulant are automatically dosed.

- In the compartment of rapid mixing of the flotation installation (104) water will be treated with solution 5 % of Ca(OH)2 (depending on pH) and with solution 0,1 % of anionic polielectrolyte (depending on flow).

The respective solutions will be prepared in the reagent chamber.

After it was mixed with reagents the water passes in the slowly mixing compartment in which the flocculence take place. Then in the flotation vessels the aeration, respectively the flotation takes place. A part of the effluent of the flotation vessels is recirculated by means of a pressurising vessel.

The solid particles driven at surface will be collected and evacuated in the sludge basin from the chamber of sludge concentration.

After flotation the effluent gravitationally directed in an aspiration basin will be pumped in the stocking tank equipped with translators for maximum and minimum level. The tank sets off the flow fluctuations assuring a constant flow for the biological step.

2. Biological step

The Study proposes for the biological treatment step an anaerobic/aerobic combined system. The system is dimensioned to a flow (60 m³/h) assuring the treatment of the annually amount of residual water of 520.000 m³/year.

The physical-chemical treated water stocked in a tank will be pumped in the zone destined to the biological step, in the two immerse trickling filters.

The pipe connecting the distinct locations of the two steps will cross over the existing oil pipes and the road from the breakwater crown and then will be mounted underground at 5,00 m along the road.

The effluent of the trickling filters will gravitationally flow in the aeration basins, with active sludge.

The necessary air for the treating aerobic process is furnished by the blower mounted in the

blower chamber. Dissolved oxygen detectors especially foreseen adjust the air flow.



Fig. 4. Wastewater treatment scheme

From the aeration basins water will freely flow in the secondary settling basins, in which active sludge separation takes place (free flow in the aspiration basin). From here the treated water will be collected in the collection/monitoring basin and discharged into the port basin.

The sludge will be pumped (through a pipe Dn 50 mm underground mounted along the road) towards the physical-chemical step in the sludge collecting basin and then in the sludge concentration chamber from where will be transported by truck for final disposal.

The resulted sludge amount is estimated at 1,5 - 2 m3/day dry substance. The water will be re-directed in the flotation installation.

6. Collecting ship

The port administration deals with the collection of residual water from the ships from the port, of residual water from the port basin surface including oil residues from accidental discharges.

For the collection activities the Technical Ship Branch has:

- 3 collecting ships;

- 1 ship for taking over (stocking);

- 1 ship for residual water storage;

- about 2000 ml of anti – polluting barrage for reducing the accidental polluted zones.

3 skimmers for collection from the polluted zones The technical means of collection are common for:

- bilge water from ships;

- domestic wastewater from shipboards;

- liquid and solid residue from the port basin surface.

The quantity of residual water to be collected is estimated at about 7000 m3/year. The three technical ships for the residual water collection are old of 11-19 years and have limited technical performances. Because of periodical works of repair and maintenance only 2 of the 3 ships are active and the collection of the estimated quantity of residual water is not assured.

The collection capacity of these ships is negatively influenced by the big distances they have to cross, because of the large surface of the port basin. Thus, the basins from the Constantza port southern zone are at about 10 km from the collecting ship parking place. The effective time of collection is reduced at 50 %. (In future it is necessary to optimise the emplacement of the storageprocessing capacities).

The existing ships owned by the port administration have not the possibility to clean the quays and the breakwater slopes, which represent a permanent source of pollution for the water in the adjacent basins.

For the mentioned reasons, the project proposes to acquire a specialised ship that will help to improve the collection activity of wastewater from ships and of residues from the basin surface.

This ship has a stocking capacity greater than the one of the existing ships and it can perform multiple operations:

- Collection and delivery to the shore of residues from the water surface;

- Collection and delivery to the shore of bilge water from ships;

- Cleaning of the quay faces and breakwater slopes Interventions in case of accidental pollution; Interventions in case of fire;

Repair of the Gangway in the Area of Mamaia's Casino

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Abstract: In front of the Casino from Mamaia, a gangway has been built at the same time with this one, around the year 1935. The gangway is provided at one end with a square shaped pavilion, covered with a dome, supported by four pillars. A toboggan and a landing place with stairs descending towards the sea have been realized each side of the gangway, near the pavilion. The entire construction is of reinforced concrete.

Taking into account the touristic interest of this objective, to maintain the solution it is necessary to urgently achieve repair works. The repair solutions have to correspond to the damage (degradation) state and to assure the construction safety and function.

The investigation results lead to a better knowledge of the construction structural conception, as well as of its present technical condition. This allowed establishing the most appropriate constructive and technological solutions for the rehabilitation of the gangway and for the assurance of its durability.

Keywords: concrete deterioration, design philosophy, rehabilitation.

1. Introduction

The engineering constructions must give both resistance and stability and also durability. Maritime hydro-technical constructions must resist to the water's physical - mechanical destructive action, to the biological aggressiveness and to the action of waves, ices and alluvia.

The seawater is a mixture of different salts, especially chlorides (NaCl, MgCl₂) and sulfates (MgSO₄, CASO₄, K₂SO₄), as well as calcium carbonate CaSO₃ etc. The salts content, salinity, is expressed on grams at 1 liter of water and varies between $30 \div 35.5$ for oceans, to $10 \div 20$ for inland seas. In river inflow areas, the salinity decreases. Water's noxiousness increases in the area of the industrial water discharge.

Seawater action, depending on the character and aggressiveness degree to the construction materials, is different on the structure height. Four areas can be distinguished: an area up to the water level, that is never wet, an area at the level of action of waves or water level variations, an area down to the level of the 2^{nd} area; and area under the sea bottom where the soil is aqueous. The main materials used in the port hydrotechnical constructions are the wood, the metal, the concrete, the stone and the plastic.

2. Designing principles

The port hydro-technical constructions are different from other engineering constructions, especially by the following aspects:

- execution of the most important part of the structure is executed under the water level and sometimes in the difficult conditions caused by waves, currents, etc.;

- large prefab elements are often used, placed by specialized floating equipment;

- the constructions are submitted to both vertical and horizontal loads, having important values and dynamic character;

the construction materials are submitted to the aggressive action of the aquatic environment and especially to the maritime environment.

All these aspects give to the port hydro-technical constructions some specific features regarding the constructive solutions, the execution technology,

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calculation methods, materials, their technical conditions and also the importance class.

Also, we have to mention the technical difficulties and the increased price of the reconstruction or reparation works in case of under water part damage. That' why it is necessary to adopt solutions that have a durability in time as long as possible.

Generally, for civil works the lifetime is of 50 years, while for port structures it is much longer. The problem is that a cheap solution can touch a long life if considerable investments are made for maintenance works. On the other hand, a durable work can have a short life without investments in maintenance works. In conclusion, from point of view of the port structure durability, the main principle that govern the designing is the economic one. But, the designer cannot see very far into the future.

Generally, to chose between a durable project with small or absent maintenance works and a less durable one with big reparation and maintenance works, is not a very difficult problem. The reason is that the initial costs are much smaller than the further necessary costs. This aspect is explained in figure 1.



Fig. 1 Costs necessary to assure the hydrotechnical works durability

Depending on the reinforcement corrosion, four phases are distinguished:

• Stage A: designing and execution (when the durability aspect is not considered);

• Stage B: the beginning stage (corrosion did not appear, but the phenomena of carbonation and chlorides penetration exist);

• Stage C: local corrosion (corrosion has begun but is only local);

• Stage D: general corrosion (developed in the entire structure).

Costs necessary to fulfill the durability level in each consecutive stage are 5 times higher than those from the previous stage.

Each currency unit spent in the designing and execution stage is equal to 5 units in the stage B, to 25 units in the stage C and to 125 in the stage D.

In this way, a special attention for the designing and construction stage is enough to prevent important investments in the subsequent life stages of the structures, especially of the maritime ones.

In order to be able to determine how much attention must be paid, during the designing, to the durability aspect, the different damaging possibilities have to be known. Further on, the damages for concrete and reinforced concrete structures are in detail discussed.

3. Maritime environment action on the concrete for hydro-technical works

The corrosion effect is maxim in the area of water variable level. In the thin elements of reinforced concrete, damage is mainly produced due to the reinforcement corrosion under the effect of humidity penetrating through the fissures. The reinforcements' volume increases by rusting, leading to the detachment of the covering layer. Concrete degradation is caused by a chemical process produced between the seawater and concrete components, by the frost-thaw effect, by the mechanical action of waves and by the biological factors.

Damages that can appear can be grouped as follows:

- the reinforcements corrosion by carbonation or chlorides penetration;

- the concrete degradation by penetration of sulfates and by the free calcium hydration, causing the concrete swelling and aging, by the alkali-silicon reactions leading to gel formation, erosion.

Carbonation:

The reinforcement bars are protected against rust by the concrete high alkalinity. Nevertheless, the CO₂ penetration from the air can transform the Ca(OH)₂ in CaCO₃, leading to the decrease of the medium alkalinity and also of the pH values from $12 \div 13$ to $8 \div 9$. This process is called carbonation. If the carbonated front reaches the reinforcement bars, where enough moisture or oxygen has also penetrated, an electrochemical reaction appears, which leads to reinforcement oxidation.

The formed rust occupies a volume of about 6 times bigger than the metal from which it proceed, breaking this way the relatively thin layer of covering concrete.

Chlorides penetration:

The chlorides have a similar effect. The chlorides ions can penetrate by the water absorption due to capillarity or by diffusion. As long as the concrete structure pores are not saturated, the absorption by capillary attraction prevails. According as the saturation degree increases, the diffusion will increase too. The structure humidity degree has an important role regarding the chloride penetration degree.

The reinforcement corrosion is generated by chloride penetration. A chlorides level of 0.05 % of the dry concrete weight indicates a distinct corrosion risk, but negligible, while a level of 0.25 % indicates an important risk. This penetration begins to reach critical degrees after $5 \div 10$ years and develops on thickness larger than the reinforcement layer, even up to 240 mm.

Dislocation of the covering layer is produced at an corrosion level equal to 0.1 c/ ϕ (in mm), "c" representing the thickness of the covering layer, and " ϕ " – the bar diameter (figure 2).



Fig.2. Corrosion of a metallic bar

It means that the corrosion thickness must be limited to few millimeter tenths. At a corrosion thickness of 0.2 mm, the propagation period is equal to 2 years in the wave action area, 10 years in the water level variation area and 200 years in the submerse area (after the initiation period).

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The fissures influence on the corrosion rate is small or negligible if the fissures width remains under 0.4 mm. Different norms foresee that the fissure width has to be limited at 0.1 mm in case of areas with agressivness, at 0.05 mm if the agressivness is intense. The initiation period considerably decrease with about 2-3 times, in case of fissured concrete raported to the unfissured one.

The lifetime of the reinforced concrete structures in an environment containing chlorides or carbon dioxide can be divided in 2 periods, depending on the determination rate (figure 3).



Fig. 3. Determination of structures lifetime

The initiation period represents the period in which the carbonation or chloride penetration cover and degrade the reinforcement. At this period end, the electrochemical corrosion appears, depending on the oxygen content, on the electrical resistivity and on the temperature. This is the beginning of damages (the propagation period). Th experience shows that above the water level, the oxygen is not a limitative factor, so that the corrosion rate depends on the electric resistivity and on the temperature. In the sea environment, the high level of the concrete humidity produces a small resistivity. Therefore, the corrosion is more influenced by the temperature. The structure lifetime is in other words defined as the period in which the corrosion level is limited to an acceptable level.

In case these phenomena are noticed, in order to repair the constructions, it is necessary to remove the carbonated concrete layer, to check it by samples, directly applying phenolphthalein, to clean the rust from the reinforcement and to apply on it a film of substances based on polymers, cement and corrosion stoppers (about 150 g/ml for steel bars of 8 - 10 mm diameter) and then to apply a layer of mortar with controlled contraction, resistant at sulfates. Resins can also be used in order to prime the adherence at the new concrete pouring over the old one $(0.5 - 2 \text{ kg/m}^2)$.

Sulfates penetration

Besides the other salts, the seawater contains calcium sulfate and magnesium sulfate, which react with the cement components. The cement contains tricalcium aluminate, as well as free calcium and aluminium. As a result of the reaction the substance tringit (3CaO Al₂O₃ 3CaSO₄ 32H₂O) can be formed. The reaction is accompanied by a strong expansion, having as effect the destruction of the hydrated paste. The mortar or the concrete looses its resistance, then, by swelling, detachment and decomposition, it completely degrades. Thus, the reinforcements loose their protection and then are fast oxidized. Damage develops in time in the concrete mass taking place the so-called aging phenomenon. This phenomenon can be recognized by the loss of concrete cohesion and the apparition of a white powder. The risk of this type of damage appears to the concrete made of Portland cement with a higher content of tricalcium aluminate.

That is the reason for which it is recommended that the furnace or Portland cement have a reduced level of tricalcium aluminum.

In practice, the degradation with sulphate is less severe than in the laboratory, as a result of the inhibitor effect of the chlorides in the seawater.

Alkali – silicon reactions

These reactions lead to an expansive phenomenon, like in case of the tringit formation, with the difference that the destruction is not preceded by the cement structure weakening, but by the formation of a gel in the aggregates. Under the influence of the high content of alkali, the silicon gel is formed by the reaction with the silicon oxide from the aggregates (Na₂ SiO₃ nH₂O or K₂ SiO₃ nH₂)), leading to the fissures apparition.

This form of damage can be mitigated if in the wet environment there are alkaline in excess, like sodium or potassium. For the cement used at the maritime works, it is necessary to limit the weight content of the different components, as following:

Portland cement without additions

- $SO_3 < 2.5$ %; MgO < 3 %; $Al_2O_3 < 8$ % $(Al_2O_3, 2CaO) < 10$ %;

- sulphur content: < 0.2 %;

the index will be also checked:

$SiO_2 + Al_2O_3 > 0.31 + 2(Al_1O_1)$	
$CaO + MgO$ $(31 + 2(M_2O_3))$	

- the tricalcium content: 0.5 %

Portland cement with light ashes:

- the ash content will be between 10 % and 20 % and will be of 5 - 10 % for high resistance cement:

- $SO_3 < 3\%$; MgO < 4%; Al₂O₃ < 14%;

- the sulphur content: 0.5 %;

- $2.65 \text{ Al}_2\text{O}_3 - 1.69 \text{ F}_2\text{O}_3 < 16 \%$

 \succ the blast-furnace slag cement:

- the magnesium content: MgO < 2 %;

- the sulphur content: < 2%;

- the slag content: < 2%;

It is also necessary to obtain a fine grinding of $3500 - 4000 \text{ cm}^2/\text{g}$ and a hydration heat elimination of 50 - 70 cal/g.

The best binder for the maritime works is the one that contains less aluminium and free calcium. Therefore, the possibility to reduce the free calcium, by adding substances that can fix this calcium, has been analyzed. Examples of such addition are: ash of steam power plant, classic pozzolana, shingle or crushed brick (ferruginous aluminum silicate). These additions must be grind fine and have a double advantage, to fix the calcium and to increase the concrete capacity, but generally the cement hardening is slower.

The cements used for the maritime works are the following:

- artificial Portland cement with or without secondary components; this category includes the cements with light ashes;

- the metallurgic, furnace cement and clinker cement; at the last one, the reduction of the hardening time is less dangerous at the maritime works than at the works on land.

The cement with fast hardening time will be used only for obturations.

The aluminum cement (or the furnace cement) is not usually used, because it generates great drawbacks in the works quality.

Using the right cement is not enough in order to warrant the adequate quality of the mortar or concrete for maritime works. The concrete compactness and its composition are the determinant factors for the work's durability. It is necessary to stop the penetration of seawater into the concrete, so the compactness must be maximum, which impose a right vibration.

The usual composition of the mortar and concrete is the following:

- mortar: cement $-400 - 600 \text{ kg per m}^3 \text{ of dry sand};$

sand -2/3 of the whole > 2.5 mm;

mm:

- 1/3 of the whole > 5.0

- reinforced concrete:

cement -350-400 kg at 1 m³ of concrete;

aggregates	- 800 kg (l);
	- 400 kg (1).

The maximum size of the aggregates can be up to 50 mm or more for the mass concrete, but must be reduced to 25 mm for the high reinforced concrete.

The cement dosage has a great importance and conditions the concrete resistance and contraction. A high dosage imposes a larger quantity of mixture water, leading to the increase of concrete contraction, making worse the fissures, and consequently to the increase of the concrete degradation and reinforcement corrosion risk.

An important measure for the durability of the reinforced or pre-stressed concrete is to assure reinforcement covering layer of minimum 5 cm.

The mixture water must be clean, without organic substances and hydrocarbons, which, among other drawbacks, lead to the diminution of the adherence to reinforcements.

Certain normatives foresee the possibility to use the seawater, but they limit the quantity of dissolved salts to 28 g/l, for the reinforced and prestressed concrete and to 30 g/l for the simple concrete. At the last one it is taken into account that the sodium chloride accelerates the hardening. It will be avoided to use seawater for the facing concrete, because of the salt efflorescence.

Concreting of the dense reinforcement areas needs plastifier usage. The plastifier dosage is determined only by lab tests, because additives can be applied in order to slow down the concrete hardening, without affecting the final resistance that is often increased, and the capacity is bigger.

Additives can also be used in order to accelerate the concrete hardening, for concrete poured in water or rough water. The corrosive action of certain promoters, especially the calcium chloride, on the reinforcement of reinforced or pre-stressed concrete will be taken into account. The CaCl₂ dosage shall not be more than 1 % of the cement weight. For safety, on the bars, cement grout without additives can be applied, before concrete placement. It is also recommended to avoid using the additives that contain Cl⁻ ions in the pre-stressed concrete. These precautions that are available for the ground works, become essential for the maritime works where the risk of reinforcement corrosion is higher.

In order to obtain high performance concrete, the additives usage is recommended, contributing to the reduction of mixture water quantities for the same concrete workability and, consequently, to the increase of concrete durability. Superplastifiers additives based on sulphonated polymers or on acryilic polymers are considered.

Frost – thow:

The volume of the water from the concrete element pores, increases by freezing, leading to the concrete surface degradation, which encourages the chemical corrosion.

In the ices flowing areas, the effect of concrete surface erosion also appears.

Erosion:

It is an important damage form of the port structures submitted to mechanic abrasion under the action of waves and currents carrying out the sand.

In order to give to the concrete a higher durability in the sea environment, the concrete must be compact and resistant to frost cleftness. At concrete preparation, aggregates composed of 3-5 fractions will be used in order to reduce the void volume. Water/cement ratio will be under 0.5 and the cone settlement of 2-4 cm for simple concrete and up to 6-8 cm for strong reinforced elements. The concrete class will be between C 12/15 - C 20/25, while for the per-stressed elements of C 25/30 - C 32/40. The concrete quality is directly influenced by the mode of preparation and placement, which have to comply with the specific technical prescriptions.

It is recommended to adopt structures as simple as possible, with possibilities of natural ventilation and with less beams, because their crosings represents points of evaporation concentration. The plate foundations are advantageous from this point of view. Under the effect of loads in the reinforced concrete elements must not appear fissures. The fissures span has to be limited to 0.1 mm or even 0.5 mm where the exposure to an aggressive environment is severe and other protection measures against corrosion are not foreseen. If the atmosphere is not aggressive, a fissure span of 0.2 mm can be allowed. In the areas with small salinity, up to 2 ‰, the seawater does not present aggressivity to the concrete. An important provision for the good behavior of the reinforced or pre-stressed concrete works is to assure the covering of the reinforcement with 5 cm of concrete, using concrete or plastic spacers.

4. Gangway rehabilitation in the areaof Mamaia's Casino

Around 1935, in front of Mamaia Casino a gangway has been buit at the same time with this one. The gangway has a length of 105 m and a width of 4.0 m, having at its end a square shaped pavillion, of 404 m^2 surface, covered with a dome, supported by four pillars. A toboggan and a landing place with stairs descending towards the sea have been realized each side of the gangway, near the pavilion. The entire construction is of reinforced concrete.

Till now, maintenance works have been executed, having in general modest results.

The produced damages are not only on the surface, but they have affected the construction resistance, so that besides the ugly aspect, the gangway could become a real danger for the tourists.

Istoric presentation:

Due to the construction age and to its relatively small importance, notices in specialty papers do not include details of its structure, the following data being found:

- "Urbanismul" magazine no. 11-12/1937: "For modernisation purposes, there have been arranged a splendid park with a Casino restaurant and terraces, a great number of cabins provided with all comfort and elegance, and, as a main visiting objective, an almost 130 m long gangway, which includes two toboggans for children, two landing stages for motorboats and boats, and a terrace-bar. The work was designed to face any weather and to bear a people load of 600 kg/m². A special electric lighting installation, with internal catalysing, offers a good lighting during night time."

- Emil Prager – Reinforced concrete in Romania (1979): "The Casino is a simple construction, dedicated to services and cabins, having a ground floor only, with a large front and a lateral tower, which has a height corresponding to three floors. This construction has separate foundations shaped as reinforced concrete plates, directly executed in the beach sand.

Both constructions (this includes the International Hotel too) had a good behaviour during the earthquake of 1940, although it is known that earthquakes' effect on constructions founded on sand is sensitively accentuated.

Concomitantly with these works, there was executed a gangway with a pavilion and a springboard, located on sea, in front of the Casino. It is all made of reinforced concrete, founded on reinforced concrete piles. The architecture projects were prepared by arch. Gheorghe Cantacuzino, and the concrete ones by Eng. Vermecher. The execution was carried out by Eng. Aurel Ioanovici".

Completed studies

Considering the age of this construction (about 65 years), the available data regarding its structure are few and, for filling them up, some investigations have been necessary. Such investigations consisted of complex site, laboratory and office surveys.

The results of these investigations have led to a better knowledge of construction's structural conception and of its present technical state, with a

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view to assess the most appropriate constructive solutions for recovering its durability.

The following studies have been performed: - Site surveys:

- topo-hydrographic studies and surveys for assessing work's sizes and the features of the submersed beach in that location;
- geotechnical surveys, for assessing the foundation levels and the shore characteristics;
- visual inspections and photographs, for assessing the technical state of the construction, as a whole and in detail;
- sampling taking of concrete cores from different elements of the structure;
- water samples for determining its aggressiveness degree.
 - Laboratory surveys:
- tests on concrete samples;
- tests on reinforcing bars;
- soil tests;
- chemical analysis on the concrete's and sea water's composition.
 - Office studies:
- documentation on work's history, in order to assess construction's characteristics and conception;

- analysis of waves and currents characteristics;
- analysis of beach morphological evolution on the site;
- analysis of construction's interaction with the waves and alluvial currents;
- study on the physical-chemical processes occurred at the structural elements' level and integrated in the sea aquatic environment;
- analysis of elements' bearing capacity in various hypotheses related to loads and damages.

Work survey

As there are no drawings from the initial project of the construction, it was necessary to perform a survey in order to define its geometry. The results of this survey are shown in Fig. 4.

The construction is 130 m long, with a 110 m long gangway and a pavilion of 20 m.

The gangway is formed by a series of four decks with widths of 4.05 m and lengths of: 32.0, 30.0, 30.0 and 12.95 m. Each deck consists of three frames, the corbels being beams with heights of 0.60 m and widths of 0.35 m, connected at the upper side by crossbars with heights of 0.35 m and widths of 0.22 m and a 0.15 m thick top slab. In the locations where beams are embedded in the posts, each beam is provided with a 0.20 m high haunch, on a length of 0.60m.



Fig. 4 Gangway view

Beams are supported on the foundation blocks by means of posts having heights of 0.95 m between haunches' bottom side and the upper level of the foundation blocks. On cross-section, the sizes of the intermediary posts are 0.35 x 0.35 m and those of the end ones are 0.35 x 0.63 m. At the end toward the beach, the first deck has a cantilever of 2.0 m, to which the access stairs are connected.

On both sides of the third deck, there are two perpendicular reinforced concrete toboggans, with the lateral ends supported on foundation blocks of 2.45 x

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2.50 m. Toboggan's strength structure is a vault with a span of 6.88 m and a camber of 1.98 m. This vault has a constant cross-section of 0.33×2.38 m.

On both sides of the fourth deck, which connects the gangway with the pavilion, there are two perpendicular stairs providing access from sea. The external ends of these stairs are supported on two foundation blocks of 3.00×1.30 m. The stairs are 2.30 m wide between the external beams.

<u>The pavilion</u> is a square construction, with a side of 20.10 m, having a reinforced concrete plate of 0.15 m placed on a rectangular system of reinforced concrete beams. Such beams are 0.80 m wide and 1.00 - 0.50 m high. They are supported by four foundation blocks of 4.0 x 4.0 m. The pavilion is covered by a 0.15 m thick reinforced concrete plate with four slopes, on a system of rectangular and radial beams, with widths of 0.55 m and varying heights. The plate is covered with ceramic tiles.

Both the gangway and the pavilion used to be bordered by a metallic handrail.

The surface of pavilion's platform and of the gangway used to be covered with mosaic slabs.

Initially, the construction was lighted and used to have a water installation for toboggans' utilisation.

There should be mentioned the sewage ducts that penetrate, on the southern side, under the second deck and are then supported by the foundation blocks, under the gangway, having their outlet into the sea, in front of the pavilion.

Inventory of damages

All the damages have been photographed in February and March 2001.

Along the years, due to the marine climate and waves' mechanical action, the structural elements of the pavilion and the gangway suffered some damages, that were remedied until 1980 by repeatedly applying plaster coats within the current maintenance works. Two plaster coats can be easily noticed (Fig. 5.).



Fig. 5 Damaged beam and concrete slab Important damages also affected the strength structure (Fig. 6, 7).



Fig. 6 Landing stage stairs



Fig. 7 Supporting post

Rehabilitation solutions

In order to recover the existing works and to maintain their viability, there have been provided both the consolidation and repair of the damaged structure elements and the anti-corrosive protection of surfaces.

Relying on the analysis of construction's (the Casino gangway) technical state, some technical solutions have been proposed for its rehabilitation. Such solutions have taken into account the following principles:

- the entire gangway and pavilion slab will be removed, and then it will be reconstructed;
- the end posts of gangway's deck will be removed; they will be then reconstructed as simple supports;
- all beams and posts will be strengthened, replacing the damaged reinforcements;
- a protection will be provided for the pavilion, in order to reduce waves' action on the construction;
- additional plates for distributing the load transmitted by posts on foundation blocks' surface will be provided;

- there will be used materials that would assure the adherence to the old concrete and the protection of the steel and concrete elements against sea aggressiveness;
- there will be provided handrails and lighting poles made from galvanised steel sections;
- the surface of the gangway and pavilion plate will be plated with ceramic tiles and adhesives resisting to sea aggressiveness;
- toboggans' water supply system will be recovered.

Working technology

The working stages and the constructive solutions will be as follows:

Construction of a protection dyke for the pavilion, on its Northern, Eastern and Southern sides. The dyke will be made of stone blocks of 2-4 t/pc., with slopes and a 5.0 m wide berm.

Dyke's top will be 5.0 m wide and it will be raised at level +0.50 m.

The dyke will embed the existing block protection, as well as the broken beams that fell into the sea (see Fig. 8).



Fig. 8 Cross-section of the protection dam

D Removal of tiles from the dome.

□ Strengthening the pavilion dome and its supporting posts, consisting of: removal of degraded concrete; removal and replacement of degraded reinforcements; applying the adhesive for sticking the new concrete to the old one; pouring the fill-up concrete or mortar; applying the waterproofing on dome's surface; films on concrete surfaces for protection against marine aggressiveness. □ Demolition of pavilion's platform plate, starting from East to West. The broken pieces could be let to fall into the sea, between the stone blocks of the existing protection.

□ Execution, while demolishing, of some scaffolds suspended under the existing beams, from which beams' strengthening could be carried out.

□ Strengthening pavilion's beams, consisting of: removal of degraded concrete; replacement of damaged reinforcements; applying the adhesive on the previously cleaned up surfaces; pouring the concrete or applying the mortar for beams' strengthening; films on surfaces for protection against marine aggressiveness.

□ Removal of scaffolds and mounting the formwork for pavilion's platform plate between the beams.

□ Mounting new reinforcements and pouring concrete into the plate.

□ Removal of the waste water ducts.

□ Strengthening the intermediary posts of gangway's sections, consisting of: pouring reinforced concrete slabs on the foundation blocks; mounting of reinforcement cases around the posts; applying the adhesive on the cleaned

old concrete surface; concreting posts' coating, with an increased cross-section; protection film on the concrete surfaces.

□ Strengthening the end posts (see Fig. 9), on which the beams of gangway's sections (decks) are supported, consisting of: mounting metallic or working supports, for retaining deck's ends; demolishing the existing posts that are seriously damaged; recovery of beams' supporting ends, attaching to them neoprene supporting plates; pouring reinforced concrete slabs on the foundation blocks, for a more uniform distribution of the stresses transmitted by the posts and for blocks' protection; pouring the new reinforced concrete posts.



Fig 9. Strengthening works for supporting posts and beams

□ Reconstruction of the reinforced concrete toboggans and stairs.

□ Demolishing the gangway deck plate, making sure that the removed material does not fall into the sea.

□ Suspending scaffolds under the beams, for their strengthening.

□ Beams' strengthening, with the same operations as provided for pavilion's beams.

 \square Removal of scaffolds and mounting the formwork for deck's plate between the beams.

□ Mounting the reinforcements end the embedded metal pieces related to handrails and pouring of concrete in deck's plate.

 \Box Applying the waterproofing on the plate surface.

□ Removal of formwork and applying the anticorrosive protection on plate's bottom surface. □ Plating the gangway and pavilion surface with sandstone tiles, with adhesives resisting to marine environments.

□ Mounting the handrail and the lighting and water supply installations.

Materials used for the repair works

The strengthening of the various gangway and pavilion elements has to start by the removal of the previous plaster coats, demolishing the degraded concrete, followed by the operation of preparing the existing concrete surfaces and the consolidation works themselves, which consist of: recovery of reinforcements and concrete sections, and concrete's anti-corrosive protection. The materials to be used have to be compatible and to assure the quality of the repair works. For recovering gangway's strength structure, it is necessary to employ special materials, concrete and mortar types with the following characteristics:

- they should be waterproof and resistant to chemical aggressiveness of chlorides and sulphates;

- they should have controlled contractions so as to avoid the occurrence of micro-fissures that favour sea water infiltration in the concrete elements, leading to their destruction by leaching of the soluble salts, corrosion of reinforcements, etc.

For avoiding the potential discontinuities between the old and the new concrete, it is necessary that the new concrete would adhere to the old one and work with it for assuring the cross-section and taking over the stresses. For this purpose, there has been provided the appliance of an epoxy resin adhesive on the entire surface (horizontal and vertical) of the elements to be strengthened. On the surface that was previously washed by water jet until concrete saturation, a pressurised air jet will be applied in order to remove the bogging water. Then a continuous layer of epoxy adhesive will be applied, by brushing, on the entire surface. This layer will be 1.5 mm thick (2 kg/m²).

Technical data on the epoxy adhesive:

- □ consistency in the paste to be applied fluid paste
- □ paste Brookfield viscosity 3500 cPa
- \Box time of : 60 min la +23°C, 30 min la +30°C
- □ setting time: 7 hours at $+10^{\circ}$ C, 5 hours at $+23^{\circ}$ C, 3 hours at $+30^{\circ}$ C
- \Box minimal appliance temperature: +5°C
- □ final strength is reached in max. 15 days after placement
- □ concrete adhesive strength: 3.0 N/mm² (on concrete cores)
- \Box reinforcements shearing adherence : 18 N/mm²
- \Box direct tensile adherence to reinforcements: 18 N/mm²
- □ compression strength: 50 N/mm² after 7 days; 70 N/mm² after 28 days;
- □ compression shear modulus: after 7 days 2300 N/mm²; after 28 days 2900 N/mm²
- □ the supplier will also specify the appliance instructions.

In case concrete sections thicker than 150 mm are recovered, the concrete to be poured should not be contractile, impermeable and it should have compression and tensile strengths higher than those of the old concrete. It should also be resistant to water's corrosive action and to repeated frost-thaw actions.

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The non-contractile concrete will be made from super-fluid and expandable cements with low heat elimination during hardening. There can be used imported materials having these qualities or Romanian materials. In SR 3011 – 96, cements of the SR I type are recommended for this type of concrete. There is also utilised a softening agent that should be compatible with the epoxy adhesive utilised for treating the surfaces.

In order to determine the utilised concrete and cement class the following will be considered, as provided by Code NE 012-99: the surfaces are directly exposed to a massive aggressiveness, with an "intense" aggressiveness degree and construction's exposure class to the environment conditions is 4 b 2.

Hence, the minimal requirements for assuring concrete's durability in relation with the 4 b 2 exposure class are:

- o minimal concrete class C 25/30
- o minimal imperviousness degree P12
- minimal frostcleftness degree G150
- o frost-thaw resistant aggregates
- entrained air volume max 4.5%
- W/C ratio max 0.40
- o Cement type SR I class 32.5

The sections that are 10 - 150 mm thick will be recovered in three stages, the works being executed consecutively, at small time intervals:

- \blacktriangleright preparation of the concrete surface;
- \succ applying the priming layer;
- > applying the mortar protection layer;

In order to assure the imperviousness, an epoxyamine or epoxy-polyamine priming layer will be obligatorily applied before the protection layer.

The sections that are 10 - 150 mm thick will be recovered with fluid and expandable special cement mortars, which contain special additives and synthetic fibres. Aggregates' grading and quantity should be carefully selected, in relation with the thickness of the mortar layer to be applied. For instance in case of a thickness under 100 mm, there should be utilised aggregates with grain sizes of 3 - 10 mm, in a proportion of 50% with the special cement that is utilised.

At dome's soffit, it is recommended to use a cement mortar with thixotropic consistency, with high strength and adherence qualities, so as not to use formworks for layers up to 30 mm thick. It could be also applied by spraying.

The existing reinforcements not to be replaced should be cleaned from rust and then protected by special polymer-based substances. Such substances have to assure reinforcements' adherence to the protection cement mortar and to be corrosion inhibitors. It is recommended to apply them by means of a paintbrush.

Concrete's anti-corrosive protection will be assured by applying the protection film layer.

It could be a bi-component cement mortar (cement and acrylic polymer) with high elasticity and imperviousness characteristics, resistant to the action of chlorides and sulphates. It should be applied in layers minimum 2 mm thick, on the entire surface of the concrete elements.

5. Conclusions

The marine hydrotechnical construction is permanently subject to the action of very intense environmental factors – physical factors (sea waves, temperature variations, relatively high humidity) and chemical factors (high concentration of magnesium, sulphur and other salts) – which have led to the degradation of most of its parts.

As for the further investments, it is necessary to pay more attention during the design and construction stages, so as to prevent the need of major investments during the future operation stages of the marine structures.

About a Method of Determination of the Solution of the General Equation of the Water Vertical Movement in Unsaturated Porous Media

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Abstract Considering the fundamental equations of the water movement in the porous media (the Darcy's low) and the mass continuity equation law, was obtained the general movement equation in unsaturated porous media, the hydraulic conductivity and the suction according to the soil humidity. To solve the general equation, the expressions of these functions were determined experimentally. It is determined the form of the humidity function only of spatial coordinate y, that is for a moment t fixed, for given the curve where the humidity values are situated.

Keywords: humidity function, hydraulic conductivity, suction, water vertical movement.

1. Introduction

The equation of the water movement in the porous unsaturated media, on vertical direction, was obtained using the two fundamental equations of the water movement in porous media - the mass continuity equation and the movement equation - and it is [1]:

$$\frac{\partial w}{\partial t} = -k_w \cdot \frac{\partial h_s}{\partial h_w} \cdot \frac{\partial^2 w}{\partial z^2} - \left(k_w \cdot \frac{d^2 h_s}{dw^2} + \frac{dk_w}{dw} \cdot \frac{dh_s}{dw}\right) \cdot \left(\frac{\partial w}{\partial z}\right)^2 + \frac{\partial k_w}{\partial w} \cdot \frac{\partial w}{\partial z}$$
(1)

where:

w [%] is the humidity of the porous media; k_w [cm/min] is the hydraulic conductivity in unsaturated media; h_s [mbar] is the suction

2. Experimental conditions and results

For the experimental determination it was chosen a particular porous medium: the soil. Because the soil and the water are the decisive conditions for the growth and development of the vegetation, the research can be used in the land reclamation, domain that studies the optimization of these conditions.

The experiments were made in laboratory, in conditions closed to the natural ones:

- the soil samples, with natural structure, were laiddown in metallic recipients with the dimensions: D=40 cm, H=60 cm, which make the results good for the description of the phenomena of water circulation in the soil, on so - called "effective depth" - the depth on which the plants develop them principal mass of the radicular system;

- the measurements of the humidity variation and the suction began with the soil samples which had low humidity the water necessary for the humidity growth was realized by of the overhead irrigation;

- the domain of the suction variation was chosen between 0 and 1200 mbar, because in this interval the water is accessible to the plants, in the soil.

The measurement of the humidity values at the different moments was made using the moisture meter BWZ - Lanze. This is a proceeding of great accuracy, whose readings aren't influenced by the soil salinization, by the pH or by the soil temperature.

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Fig. 1- The moisture meter BWZ - Lanze

To determine the suction values it was used the tensiometers methods.



Fig. 2-The tensiometers for the determination of the soil suction

To determine the variation of the hydraulic conductivity in rapport with the humidity was used an original method, proposed by the authors [2], which continue some studies from the technical literature [3], [4].

The analysis of the obtained data on three soil textural types gave:

- the conductivity equation, function of humidity:

$$k_w = a + b \cdot w + c \cdot w^2 \tag{2}$$

- the suction equation, function of humidity :

$$h_s = \delta + \alpha \cdot w \tag{3}$$

- *the humidity equation*, function of time:

$$w = \gamma + \beta \cdot t \tag{4}$$

3. About the equation of the water movement in unsaturated soil

At a given moment, *t*, it was considerate the set of equations:

$$\begin{cases} k_{w} = a + b \cdot w + c \cdot w^{2} \\ h_{s} = \delta + \alpha \cdot w \\ w = \gamma + \beta \cdot t \end{cases}$$
(5)

If it is denote by:

$$w' = \frac{\partial w}{\partial z} \tag{6}$$

then (1) becomes:

$$\beta = -(a + b \cdot w + c \cdot w^{2}) \cdot \alpha \cdot w'' - (b + 2 \cdot c) \cdot \alpha \cdot (w')^{2} + (b + 2 \cdot c \cdot w)w'$$
(7)

First, it will be solved the homogenous equation:

$$-(a+b\cdot w+c\cdot w^{2})\cdot \alpha \cdot w'' - -(b+2\cdot c)\cdot \alpha \cdot (w')^{2} + (b+2\cdot c\cdot w)w'=0$$
(8)

Denoting by:

$$p = w' \tag{9}$$

it results: $w'' = p \cdot \frac{dp}{dw}$ and the equation (8) implies:

$$-(a+b\cdot w+c\cdot w^{2})\cdot \alpha \cdot p\frac{dp}{dw} - (10)$$
$$-(b+2\cdot c\cdot w)\cdot \alpha \cdot p^{2} + (b+2\cdot c\cdot w)\cdot p = 0$$

Then $p = 0 \Leftrightarrow w = const$ (doesn't convene) or

$$-(a + b \cdot w + c \cdot w^{2}) \cdot \alpha \cdot \frac{dp}{dw} - (11)$$
$$-(b + 2 \cdot c \cdot w) \cdot \alpha \cdot p + (b + 2 \cdot c \cdot w) \cdot p = 0$$

• If
$$-(a+b\cdot w+c\cdot w^2)\cdot \alpha \neq 0$$

(The condition $\alpha \neq 0$ is satisfied because h_s is not a constant function) then:

$$\frac{dp}{dw} + \frac{(b+2\cdot c\cdot w)}{a+b\cdot w+c\cdot w^2} \cdot p - \frac{(b+2\cdot c\cdot w)}{(a+b\cdot w+c\cdot w^2)\cdot \alpha} = 0$$
(12)

The general solution of this equation is:

$$p = \frac{1}{\alpha} \cdot \frac{k_1 + bw + cw^2}{a + bw + cw^2}$$
(13)

From (6) and (12) it is obtained:

$$z = \alpha \cdot \int_{w_0}^{w} \frac{a + bw + cw^2}{k_1 + bw + cw^2} dw$$
(14)

Where $w = w_0(t,0)$ is the minimum value of w (%), at the fixed time, t (min).

It is calculated:

$$I = \int \frac{a + bw + cw^2}{k_1 + bw + cw^2} dw$$

$$I = \int \frac{a + bw + cw^2}{k_1 + bw + cw^2} dw = w +$$
$$+ \frac{a - k_1}{c} \cdot \int \frac{1}{\left(w + \frac{b}{2c}\right) + \frac{4 \cdot k_1 \cdot c - b^2}{4 \cdot c^2}} dw$$
(15)

• If
$$4 \cdot k_1 \cdot c - b^2 \rangle 0$$
, then:

$$I = w + \frac{2(a-k_1)}{\sqrt{4 \cdot k_1 \cdot c - b^2}} \cdot \operatorname{arctg} \frac{2 \cdot c \cdot w + b}{4 \cdot k_1 \cdot c - b^2} + k_2$$
(16)

• If
$$4 \cdot k_1 \cdot c - b^2 \langle 0 \rangle$$
, then:

$$I = w + \frac{c}{\sqrt{b^2 - 4 \cdot k_1 \cdot c}} \cdot \frac{1}{2 \cdot c \cdot w + b - \sqrt{b^2 - 4 \cdot k_1 \cdot c}} + k_2$$

$$\cdot \ln \frac{2 \cdot c \cdot w + b - \sqrt{b^2 - 4 \cdot k_1 \cdot c}}{2 \cdot c \cdot w + b + \sqrt{b^2 - 4 \cdot k_1 \cdot c}} + k_2$$
(17)

In both cases, using (14), it can be determined z, function of w and the equation (8) is solved; k_2 can be chosen verifying (7).

Now, it can be determined the solutions for every particular case treated in the paragraph 2.

It rests to vary the time *t*, which was fixed, for the moment.



Fig. 3- The curve of the humidity function at the fixed moment t

4. Conclusions

Solving the equation (17), it was established the moisture variation in time, on the soil profile irrigated by aspersion.

Continuing the researches it is possible to determine the general form of w, function of t and z.

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Hydraulic and Operating Characteristics of Laterals For Sprinkling Irrigation Made up Using Zinc Plated Steel Pipes

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Abstract: The paper presents hydraulic and operating characteristics of laterals made up using zinc plated steel pipes. Laboratory and field experiences have been undertaken for a model, the results being used for improving the characteristics of the prototype installation. These laterals can be used for rehabilitation of irrigation arrangements in Romania.

Keywords: Sprinkling irrigation, distribution pipe, hydraulics, head loss.

1. Introduction

Romania has over 3.1 millions hectars arranged for irrigation, the 17th country in the world with respect irrigation projects and it is cited for irrigation using ratio 31% for 1995, the 9th country in the world, that means 960,000 irrigated hectars [2, after FAO 1996 Production Yearbook].

Government documents showed that in 2001, technically has been possible the watering of 500,000 ha, taking into account the existing mobile equipment and future efforts will be done for using irrigation up to the values mentioned by FAO.

The drastic decrease of watered areas is due to the financial situation of small land owners (irrigation water is expensive for them) and to the insufficiency of laterals quantity for watering.

Irrigation in Romania needs an increasing number of laterals, new equipment has to satisfy technical needs for modernizing watering and efficient operation of big land properties, but, at the same time, to satisfy the financial possibilities of small land owners, too.

New, modern and efficient equipment has to be imported and assimilated, but domestic industry has to be stimulated to produce different kinds of watering equipment.

Such an equipment, corresponding to the needs and to the budget of small land owners, is

proposed by S.C. TEPRO S.A. and S.C. PHOENIX S.A. Iaşi – the lateral for sprinkling irrigation made up using zinc plated steel pipes, with mechanic coupling joints, TEPHX 01.

This equipment, as model, has been experimentally checked by laboratory and field measurements, referring their hydraulic and operating characteristics

2. Lateral description

The irrigation equipment TEPHX 01 is for sprinkling watering of crops, having as components: transport and distribution pipes, coupling valve to hydrants, bent, T-piece, racords of sprinklers, long and short own pipes of sprinklers, tripods, end plugs, special inverting joints (fig. 1.). The equipment is made up using zinc plated zinc pipes of Dn 100, and joints of 6*m* in length pipes by mechanical coupling, tightened by rubber rings. Under sprinkler racords for stability of sprinklers there are some support pieces. ASJ 1-M sprinkler is used, but it can be changed with other type.

For all equipments zinc plated steel is used instead of aluminium and brass (excepting sprinklers). The pipe wall thickness is 0,9 *mm*, the weight of a pipe with coupling unions being up to 20 kg. Hot zinc plating, in a thickness up to 4μ , has a good quality.

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3. Hydraulic and operating characteristics of zinc plated steel lateral TEPHX 01

This sprinkling equipment corresponds to irrigation projects with pipe networks, having distribution pipes at 396 m, 612 m and 792 m distance apart, or for operating with thermic pumping units.

Laboratory experiments established: major head loss; friction factor λ , hydraulic equivalent roughness k_e ; coefficients of minor head loss ζ at coupling unions, coupling valve to hydrants, ramifications to sprinklers (with and without special coupling pieces), bent, T-pieces and lost water at coupling unions. The hydraulic gradient of the lateral, with 22 sprinklers at 18 *m* equidistant apart, has been drawn by computation at the elevation of the pipe of the lateral and the level of spinklers; then the pluviometry in shemes 18 x 18 *m* and 18 x 24 *m*. Field experiments permitted the determination of the real hydraulic gradient of the lateral in operation and the time of changing its working position.

Experiments satisfied precision of laboratory measurements, each type of measurements being repeated until obtaining the imposed precision.

3.1. Major head loss of zinc plated steel pipes Dn 100

Using for major head loss a power relation form:

$$j = aQ^b \tag{1}$$

by statistic calculus of the 86 pairs of measurements, resulted:

$$j = 2,850 \cdot 10^{-4} \cdot Q^{1,7592} \tag{2}$$

where Q is expressed in l/s. The standard error obtained is $r^2 = 0.9855$ for the flow interval, $Q \in [0.5;18] l/s$.

Transforming Eq. (2), the friction factor λ has been obtained:

$$\lambda = 0,3147 \,\mathrm{Re}^{-0,2408} \tag{3}$$

$$\lambda = 0.01206 (\text{Re} \cdot \nu)^{-0.2408}$$
(4)

The comparison of Eq. (3) and (4) with Blasius's formula shows that zinc plated steel pipes are smooth pipes.

Using the Colebrook-White generalised equation for the friction factor λ :

$$\frac{1}{\sqrt{\lambda}} = -2 \lg \left(\frac{2.51}{\operatorname{Re}\sqrt{\lambda}} + \frac{k_e}{3.71 \cdot D} \right)$$
(5)

for experimental values (Re, λ) the hydraulically equivalent roughness k_e has been determined. For the practical interval of the flow, mentioned before, resulted $k_e = 0.0614 mm$.

The results obtained for hydraulic gradient *j* and friction factor λ in figures 2 and 3 are presented.





Fig.4. Coefficient of hydraulic resistence of mechanic coupling unions of the lateral

3.2. Minor head loss of mechanic coupling unions of the lareral

The minor head loss of mechanic coupling unions of the lateral result as a difference of total head loss (major and minor) and major head loss. The total head loss has been determined for pipe line formed using conveying pipes only and using distribution pipes only (connections to sprinklers closed).

The results of measurements, separatelly processed for the two types of pipes permitted the computation of minor head loss of coupling unions and their coefficients of hydraulic resistence. Values obtained for the two types of pipes are nearly the same.

Finally, the formula for the coefficient of hydraulic resistence of coupling unions has been obtained:

$$\zeta = 61,797 \cdot \text{Re}^{-0,694} \tag{6}$$

with coefficient of determination, $r^2 = 0,986$, and it is presented in Figure 4.

This coefficient of hydraulic resistance is considerably influenced by Re number (ζ =0,07...0,014), because of the fluid flow is lessmechanic coupling unions has values of $l_e = 0,265...0,074 \, m$ for Re = $10^4...1,8 \cdot 10^5$, respectively $Q = 1...18 \, l/s$. Using in hydraulic calculus an average value for this coefficient $\zeta = 0,0215 \pm 0,0046$, the introduced errors will be under 0,5%, the minor head loss on this armature having small values. If the head loss at this armature is neglected, the total errors for the lateral calculated are below 2,5%.

3.3. Minor head loss for mechanic coupling union and ramification of sprinklers

Sprinklers ramification and mechanic coupling unions are armatures that influence reciprocally and a global coefficient of hydraulic resistance is determined for them (Fig.5).



Fig.5. Scheme for expressing coefficient of hydraulic resistence for ramifications of sprinklers

In hydraulic calculus of laterals is important the head loss on these two singularities: on line, expressed by ζ_{cr} and for ramification to sprinklers, ζ_{ra} . Both present particularities due to energy redistribution at ramifications.

a) The coefficient of hydraulic resistence on the main line, ζ_{cr} , is expressed as a function of the downstream kinetic head on the lateral, having physical signification the head loss for a number of sprinklers n=2, 3, ..., i. Values of the coefficient depends on geometric characteristics of the armature and on the ratio of the upstream and derived discharge, $\zeta_{cr}=f(Q_{am}/Q_{asp})$. Geometric characteristics of the ramification are identical for sprinklers that equip the lateral, so that, the coefficient of hydraulic resistence will depend only on position of the sprinkler on the lateral. Fig. 6 shows negative values of the coefficient, more important for sprinklers 2...6, then an asimthotical tendency to abscisse.

The coefficient of hydraulic resistence may be approximate by a hyperbolical equation:

$$\zeta = -\frac{0,4402}{Q_{am}/Q_{asp}} + 0,0260 \tag{7}$$





Increasing pressure downstream ramifications due to negativ values of coefficient of hydraulic resistence are small, $h_r = -(3...9)$ mm water column, so, pressure distribution on lateral is less influenced.

b) The coefficient of hydraulic resistance on sprinkler ramification, ζ_{ra} , is expressed with respect average velocity on the ramification. Its values depends on type of ramification and ratio of flow Q_{am}/Q_{asp} (fig. 5).

Its values in two variants has been determined:

without a special coupling sprinklers;

- using a special rapid coupling of sprinklers, having a rubber ball for closing.

*b*₁. <u>Coupling sprinklers by screwing</u> the coefficient of hydraulic resistance of the minor head loss has values of $\zeta_{ra} = 2,2...3,7$, depending on the ratio



 b_2 . Using a special raccord piece for sprinklers ramification, reduction of the cross-section and the rubber ball have a great influence on the flow perturbation, the coefficient of hydraulic resistance will be increased up to:

$$\zeta_{ra} = 15,083 \pm 0,0174 \tag{8}$$

The probable relativ error of measurements is $\delta\zeta = \pm 1,15\%$. Minor head loss increases 4,7...6,8 times, the supplementary head loss in absolute values, being of $h_r = 1,4 \, mCA$.

3.4. Minor head loss on bents, T-pieces and connections of laterals to hydrants

The bent, T-piece and connection to the hydrant are unique singularities of laterals, the bent and T-piece being used only for given schemes in operation of laterals. The connection to hydrant has a great importance, it produces minor head loss up to 1,90 mWC (5-6% of the service pressure). This head loss increases the price of irrigation water by energy consumption at pumping.

a) Coefficient of hydraulic resistance on bent, ζ_c , contains influence of two mechanic coupling unions and it depends on Re number. It is approximate by:

$$\zeta_c = 14,561 \,\mathrm{Re}^{-0,196} \tag{9}$$



Fig. 7. Coefficient of minor head loss on ramification of sprinklers (simple T-piece 100/100/25)

Its curvature is R = 4D and roughness like for pipes, that explains the great influence of Re number on it (Fig.8).



Fig.8. Coefficient of hydraulic resistance for the bent of the lateral TEPHX 01

b) Coefficient of hydraulic resistance of T-piece contains the influence of two mecchanical coupling unions too, and it is expressed with respect to Q_{am}/Q_{av} . The characteristic curve $\zeta_t = f(Q_{am}/Q_{av})$ has a hyperbolical shape (Fig. 9), being described by Eq.(10):

$$\zeta_t = \frac{3,2382}{Q_{am} / Q_{av}} - 0,7234 \tag{10}$$



Fig.9. Coefficient of hydraulic resistance of T-piece 100/100/100 *mm* of the lateral TEPHX 01

c) Coefficient of hydraulic resistance of coupling laterals to the hydrants is a global one, containing influence of the valve, bent and a mechanical coupling union. It perturbs substantial the flow, for that its coefficient is practically independent of Re number for usual discharges of 6...25 l/s.

Its value is:

$$\zeta_b = 7,046 \pm 0,0221 \tag{11}$$

having probable errors $\delta \zeta_b = \pm 0.314\%$.

Modernizing valve and form of the bent of the coupling piece, the head loss can be reduced.

4. Conclusions

The sprinkling lateral TEPHX 01, made up using zinc plated steel pipes, represents a viable solution of rehabilitation irrigation arrangements in Romania.

Its hydraulic and operating characteristics indicate a good quality of these laterals, comparable with other types of laterals.

Their own weight, with respect aluminium laterals, are greater, but their price is a serious advantage for them.

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Alternate Water Resources for Irrigation in Deficit Conditions (Storage and Catchements beneath the River Bed)

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Abstract: In actual social – agricultural conditions, frequently, the huge irrigation systems isn't used integrally, either by shortage of water by farmers, or a long – distance between farms and water supply, either of brought periods, when the river (the water source) is partially exhausted. In that situation, is being used a local (punctual) water source. One of them alternate water source is underground water accumulation, beneath the riverbed. In those papers, is shortly presented the achievement conditions of that solution, and his constructive – functional parameters.

Keywords: great irrigation systems, storage, catchement, underground water accumulation, riverbed.

1. Introduction. General issues concerning the necessity for creating the local and alternate water resources for irrigation

About 3 million hectares are arranged in Romania for irrigation, in various degrees of improvement – functionality – finalization. This important volume of work is achieved in order to strike against the disastrous effects of droughts, like the ones registered in 1865-1872, 1899-1904, 1946 (when "the hunger trains" are still present in the memory of the older people), and in the last 2-3 years (1998-2001, partial territorial).

The major part of these projects are focused on well established water courses, like Danube or inner rivers, some of them having cascaded reservoirs.

In some cases, the necessity of achieve local – punctual water withdrawal for irrigation system's command area comes out, due to restricted requirements from water users or to financial technical or organizations problems.

Frequently the farmers realize their own water supply (wells, drills, small reservoirs, or waste waters from farms or groups of farms: streets, small villages or villages, etc.). A peculiar

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local situation exists, when, small rivers or rivulets, even dried up for periods of time, may have important underground flows, which may become valuable by subterranean dams or reservoirs.

The present communication has in view these types of situations.

2. Subterranean reservoirs and catchements (beneath the river bed)

Such a solution, comprising an underground dam and draining gallery, is met in Ian riverbed, Hui-Xian County, China. The riverbed has a high degree of torrentially $(Q = 0 \div 1400 \text{ m}^3/\text{s})$, a large stony minor riverbed, with important flows through the deposits of stones.

The existing conditions in the Siret – Buzău area, with the Putna and Șușița rivers, could become a good model for a similar solution (Fig. 2).

Figure 1 shows the solution of subterranean blocking and catching of waters in the above mentioned conditions, in horizontal and vertical

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sections, when the rivers are drying for long periods and beneath the river bed are the important flows through the stony deposits.

The main component of the arrangement is represented by the underground (beneath the river bed) dam, placed in transverse position with respect to the water flowing, having a V shape with (un) equal wings, embedded in the borders of the river meadow and the head oriented downstream.

The underground dam, with collecting gallery on the whole length, has the upstream face with a filtering (made of rough rock) and an impenetrable downstream face (made of rock with cement mortar).

The access to the gallery is made through a well with a central placement in the riverbed with the upper elevation greater than the maximum level of the river waters.

From the collecting gallery, the water is directed downstream by a pipe placed also underground on the thread of the riverbed; at a favorable quota the pipe forks, to deliver the water through two channels placed on both sides, to irrigate by gravitation the downstream lands.





Fig.1. Underground dam with catchement gallery

- Figure no. 2 presents an example of the water source arrangement, with gravitational conveyance to the consumer (irrigation, water supply, etc.)
- The above-presented arrangement is suited in the following local conditions:
- water course with permanent or non permanent surface waters;
- hilly areas on the water course;
- large enough underground water supply;
- alluvial bed with large or middle granulation;
- geomorphologic stability;
- strong borders of the river, equal level or high terrace;
- preferred relatively parallel with the longitudinal axis inclination of strata.



Fig.2. Project Example

Some advantages are significant in the underground reservoir situation, when compared to a surface reservoir with over ground dam and the classical methods of water table enrichment:

- the creation of the underground water storage, beneath the river bed;
- the enrichment of the water table in the influenced area;
- the rational management of the generally unused underground waters;
- all the advantages associated with the surface water reservoirs, but avoiding the water losses by means of evaporation, pressure head loss, the unused terrain affected by the project, avoiding the silting of the reservoir;
- the placement is not dependent of the shape and width of the river;
- significant reduction of materials in comparison with a dam placed in the river bed;
- usage of local materials, with no special requirements, for the building of the dam;
- possibility of usage of prefabricated parts;
- greater stability of the dam, buried in a transverse trench;
- functioning of the water catchment even for reduces levels of the surface water or dried riverbed;

- avoidance of the silting of the river in the section of the works;
- avoidance of the affluence downstream the works, due to fact that the regular flow in the river bed is not affected;
- removing of difficult, clumsy works for the discharge the energy;
- gravitational catch and conveyance of the water to the consumers, by simple intake and transport structures;
- a simple, a reliable control of functioning and ease to manage;
- the possibility of simple hydraulic automation of the entire structure (with upstream command and restricted downstream);
- economy of agricultural terrain or for other landmasses.

3. Conditions for flow and flow interception. Dimensional elements

There are several practical situations, a specific design being necessary for each peculiar case.

The *horizontal* type, with *perfect* (I) drains (galleries) of underground water catchement is considered, if the structures are build on impermeable substratum, or *imperfect* (II), if the structures are build in upper position with respect to the impermeable
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substratum. The catchement is with frontal action, supplied from one side with water (in the direction of water flow), intercepting type, due to fact the elements of caption have a transverse position with the respect to the direction of the underground flow.

There sometimes exists the possibility of supply water from the terrace, but generally the

flows are small, and, when the direction of the water flow is approximately parallel with the main stream beneath the waterbed, this component is neglected (the case of regular levels in the river).

The connection between the water table in the meadow – terrace area and the water level in the river may be represented as in Fig. 3:



Fig. 3 The correlation terrace-river a) For high levels (flood); b) For low levels (drought)

a) For high flows in the river, the terrace is also supplied with water, and the excess of water may take a roundabout way through (beneath) the terrace, continuing its way downstream (3, a);

b) For low flows (or under drought conditions) in the river (3, b), the water table from the terrace and beneath the waterbed concentrates in the section where the structure is placed, and effect of enrichment of water (subterranean reservoir) beneath the river bed.

The analysis of the water movement spectra in the subterranean reservoir towards the interceptive drains (the two galleries) shows that a Q_c discharge flowing on the stratum front with $B = L_c$ width with a total length of the gallery $I_{real} < \alpha_{calcul}$ may be collected (Fig. 4).





3. 1. The dimensions of the intake. Principally, the dimensions of the intake with intercepting galleries placed in the front side, fed from one side (upstream bench) is done by establishing: the total length of the gallery (L_c) , the unevenness in the galleries (h_c) , the dimensions of the transverse

section and of the longitudinal slope of the galleries.

The computation scheme for the case shown above, with the gallery placed on impervious bed having a medium slope (0,003 < I < 0,01), in the most detrimental case – dried river (Fig. 3, b) and perfect drain (*I*), is as follows (Fig. 5):





➤ The specific discharge of the gallery:

$$q = H \cdot k \cdot I = h \cdot k \cdot J \quad \begin{pmatrix} l \\ sm \end{pmatrix} \tag{1}$$

> The length L_c of the intercepting gallery (total, for both branches):

$$L_{c} = K_{s} \cdot \begin{bmatrix} Q_{c} \\ (H_{min} \cdot K \cdot i) \end{bmatrix} (m)$$
⁽²⁾

The unevenness of the water at the input of gallery: *case I* (and I > 0,003):

$$h_0 = \sqrt{H^2 - [(2q \cdot X_{max})/K]} \quad (m)$$
 (3)

For the *case II*, when the gallery is an imperfect drain, the problem is more complex and experimental tests and proofs is necessary.

When the underground reservoir is full and the level of waters in the river is high (maximum), the drain is considered to collect the water from a subterranean basin under pressure (the basin effect occurs). In this case neither the slope (I = 0) nor the parallel (terrace) flow has any influence against the intake.

- The transverse section of the gallery is established from constructive hydraulic considerations; the most frequent shape is that a horseshoe. The hydraulic calculus is made for a 0,5 ÷ 0,6 filling degree for the free level flow in the gallery.
- The longitudinal slope of the gallery (for each branch directed to the collecting well) may be derived from the flow of the collected water with free level, within the admissible water celerity. We shall consider the increased turbulence of the water due to the existence of the access orifices (windows) and the reversed filter too, by considering a roughness coefficient greater than one usually considered for concrete channels.

The allowable minimum slope shall be 0,001, from constructive considerations.

3.2. The dimensions of the collecting well.

Considering *the dimensions of the collecting well* means to establish the height, diameter and width of the wall:

> The height (H_p) is established considering the position of the foundation plate (h_f) in case I or

case II and the maximum water levels (h_{max}) in the surface water flow (Fig. 6):



Fig. 6 Establishment of collection well height

$$H_p = h_f + h_{\max} + h_{sig} \quad (m) \tag{4}$$

 H_{sig} shall be equal with 1 m.

The diameter will have minimum 1,5 m, depending on the height and the usage (inspecting) conditions; the usual ratio for caissons shall be used;

The width of the wall, depending on the height, shall be obtained from the resistance conditions.

3. 3. The gallery and the conveyance channel do not pose special dimensioning problems, considering the gravitational flow; the maximum total discharge and the minimum longitudinal slope of the 0,1 % (considering the altitude of the consumers).

For the proper functioning of the intake, the catchement windows will be equipped with a gravel filter in 2-3 layers, with increasing grain size distribution towards the sieve of window. The bottom of the inspecting hole shall be lower with at least 50 cm than the bottom of the gallery, to allow the restraint of the sand casually drawn the catchement and ease of cleaning.

To increase the safety in the exploitation, each branch of the gallery will be provided with the shutters in the end placed towards of the collecting well. To control the functioning of the structure, the well will be provided with water flow and level measuring devices. The protective shell of the structure will be a spillway threshold, to ensure against ice in the wintertime; the profiled shape will not overtop the stable bottom of the riverbed with more than 50 cm.

In *case II* (deep impervious layer), considering the local conditions, waterproofing methods may be applied under the bottom of the dam (injections, tightness veils, etc.).

The components of the structure will be finally adopted only after the study of the interesting section according to available STAS for subterranean works.

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The Structure and The Calculation of The Constant Level Dams With The Hydraulic Automatizing Functioning, Used on The Irrigating Channels

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Abstract: This work presents some types of the constant level dam, used on the irrigating channels as regulating installations in the conditions of the hydraulic automatizing of the water flows, the way in which these are structured and some elements for their calculating and their dimensioning.

Keywords: constant level dam, regulating installations, hydraulic automatizing, the water flow.

1. Introduction

The constant level dams are those installations that are installed on the irrigating channels with the goal of rising the water level from them, for securing the irrigation of larger surfaces.

The constant level dams by the way in which the bief of the adjustment is done, they are structured in two groups: dams that automatically adjusts the level from upstream are of AMIL kind; dams that automatically adjusts the level from downstream are of AVIS kind, that control the channels sections and of AVIO kind that control the bottom orifices.

The working principle in the conditions of the hydraulic automatizing, presumes the existence of a floating device, which is a component of the dam, that discerns the level and trasmits to another element which has the function of obturating, the command to close or open the section or the orifices.

The choice of a certain kind of dam will be done taking into account the goal that has to be achieved, respectively the level that has to be adjusted [1].

2. The structure of the constant level dams

The constant level dams (Fig.1) are made up of three parts:

a) the dams deck or the dams platform, with the goal of closing the section, made up of a thick iron plate;

b) the floating device and the ballast chamber, having the role of discerning the wanted level, throught them the automatically hydraulic adjustment is done. The floating device and the ballast chamber are linked to the platform throught the help of some arms in the case of AVIS and AVIO dams, and in the case of AMIL dams the floating device is disposed on the platform in the upstream bief;



Fig. 1. The AVIS type dam, general view

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- c) the visiting and inspection foot bridge is made
- d) up of striated iron plate;
- e) the floating's device cheek at the AVIS and AVIO dam with the role of protecting the functioning of the floating device;
- f) the lateral supports placed within the wals of the channel;
- g) the dam's tree is the resistance element through which the forces that act upon the dam are equilibrated and through whose extremites the fastening of the supports is done;
- h) the fastening dam's pieces are on the lateral walls of the irrigating channel;
- i) the fastening cross piece embedded on the obturated channel's bottom on which the dam is staying.

3. The calculation of the constant level dams

The dams calculation is begun with the predimensioning of the components presented above, with the support of the prescriptions done by the institutes where those dams [2] are projected taking into account the obturated section, the

upstream level, respectively the the downstream level of the water in the channel. After the predimensioning there are established the loadings, there are calculated the stresses and verified the resistance elements.

The dams deck or the platform is dimensioned in function of the water's pressure which acts upon it (Fig.2) with the relation (1) that establishes the thickness "t" [3]:

$$t = a \sqrt{\frac{p}{\sigma_a}} \qquad [\text{cm}] \tag{1}$$

where:

a – the short side of the panel that is dimensioned, measured in centimeters;

p – the water's pressure in the middle of the panel, measured in daN/cm²;

 σ_a - the admisible resistance of the used steel, measured in daN/cm².

The dam's arms are verified at compression and bending [2] considering them embended in the tansversal tree of the dam (Fig.3).



Fig. 2. The diagram of the pressure of that water that acts upon the dam



Fig. 3. The main loadings of the resistance structure

The compression force F_X is the resultant of water's pressure on the surface of the deck (A_V) and is condsidered to act in the centre of the weight of the surface at h_G from the base (Fig.2.).

$$F_x = \gamma_{ap\check{a}} h_G A_v \quad [\text{daN}] \tag{2}$$

The bending is produced by the weight of the deck G_1 and the one of the arms G_3 , in relation with the dam's tree, forces that overall are equilibrated by the Arhimedic force (F_a) , the floating device weight (G_2) and of the ballast (G_4) from Fig. 3.

The bending moment is:

$$M_i = (G_1 + G_3)R \quad [daNcm] \tag{3}$$

 $F_x/2$ l_1 $F_x/2$ l_x l_x H_x H_y H_y The verifying of the arms resistance is done with the following relation:

$$\sigma = \frac{F_x}{2S} + \frac{M_i}{2W} \le \sigma_a \quad [\text{daN/cm}^2]$$
(4)

where:

S – the net area of the arms section, in cm²;

W – the net resistance modulus of the arms section, in cm³.

The dam's tree is resistance piece that assures the taking over of the horizontal and vertical loadings, beeing bended in the two planes (Fig. 4.).



Fig. 4. The static scheme of the dam's tree

The bending in the horizontal plane is the effect of the resultant of the water's pressure upon the deck (F_x) , the bending moment is:

$$M_{ioriz} = \frac{F_x}{2} \cdot \frac{l_2 - l_1}{2} \quad [\text{daN-cm}]$$
(5)

The bending in the vertical plane is produced by the deck's and the arms' weight, and the bending moment is:

$$M_{vert} = \frac{G}{2} \cdot \frac{l_2 - l_1}{2} \quad [\text{daN-cm}] \tag{6}$$

The maximum unitary normal stresses, taking into account the bending from both planes is:

$$\sigma_{i} = \frac{M_{i \text{ oriz}}}{W_{i}} + \frac{M_{vert}}{W_{i}} \le l, l\sigma_{a} \text{ [daN/cm^{2}]}$$
(7)

The other component parts of the dam do not need special calculation, at their construction one takes into account the constructive conditions, the shares of limit gauge and the periodical maintenance.

4. Conclusion

The static and resistance calculation of the constant level dams, practically, it is done through the asimilation of their components with simple elements, that can be easy, simply and efficiently solved, without the need of calculating programs, that do not always come in handy to the projector.

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The Impact of the Complex Land Reclamation Systems upon the Dynamic of the Salts from Filipoiu – Brăila's Great Island

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Abstract: In this work there are presented the results of the 20 years' studies, in the complex of embankment – draining – drainage and irrigation Filipoiu, upon the dynamic of the salt contain of the soil cover. Filipoiu system lies on the north part of the Brăila's Great Island (I.M.B.) and it constitutes, as well from economically point of view, as of the effect of application of the works with land reclamation character, a reference area for the entirely I.M.B. territory. The studies were started in 1970 and consisted of the annual determination of the value of The Total Content of Soluble Salts (CTSS) in a total number of 19 points of the system. The fact that the application of the works with land reclamation work upon CTSS of the soil cover. In the work, it is widely presented this impact. The most important conclusion which could be taken down is that in the most part of the Filipoiu system surface it was set up a season desalt – unreversible favorable for practice, without restriction of the land reclamation works after the application of the whole categories of land reclamation needed works.

Keywords: Irrigations, drainage, dikings, salts

1. Introduction

Filipoiu's land reclamation system is a part of the complex arrangement Braila's Great Island (I.M.B.). The system occupies a surface of 7084 ha, it is situated in the north part of I.M.B. and it contains works of draining, drainage and irrigation realized in different stages:

- diking – in 1964;

- draining – between 1965 – 1969;

- drainage and irrigations – between 1976 and 1983.

The type of hydrotechnical schema with complex function adopted in this system, as in the entire area of I.M.B. was with separate networks for draining – drainage, respectively irrigations.

The realized surfaces are, on work categories:

- draining 4240 ha;
- drainage 2061 ha;
- irrigations 7084 ha.

The entire irrigated surface of 7084 ha is protected by a dam of 23,5 km long.

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Filipoiu's draining system takes in 188 km of canals from which 12 km represent only Filipoiu's collecting canal, the main canals are of 65 km and the secondary canals are of 111 km. The evacuation of the excess water, from Filipoiu system, is done with the help of S.P. Reversible Filipoiu (Q=4,99 m³/s, H=3,6 m).

The drainage in Filipoiu system is constituted from 289km of absorbent drains, which unload the waters in main and secondary canals, and sometimes in closed collectors.

The technical parameters of the drains are:

the distance between drains, 30 - 40 m;

- the posing depth, 1.1 1.4 m;
- the drainage material, corrugated PVC $\Phi 65 80$ mm and ceramics $\Phi 70$ mm.

Filipoiu's irrigation system is deserved by 14 km of adduction canals, planked with large slabs (2500x1000x6 mm) and small slabs (50x50x8 mm) from reinforced concrete.

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The source is represented by Danube and the collecting of the water is secured by SPR Filipoiu.

The distribution network of the system is formed by joined up pipes at stations of putting under pressure. The total length of the pipe networks is of 156 km and the putting under pressure is secured by 5 stations of putting under pressure (SPP). Each SPP deserved a certain area which surface varies between 1177 and 1754 ha.

The distribution of the water on the entire surface of the system is done by aspersion with installation of IIAM type.

2. The performed measurements.

Immediate after the whole draining of I.M.B., in 1965, IPACH established a soil – cultivating detail on the entirely enclosure. By means of the established study it resulted the following (classification after Burringh [1]):

- 98.16% from the total sorted surface (\approx 70675 ha) was occupied with desalt soils, having CTSS on a 0 and 2mmho/cm (0-150 mg/100 g sol) profile;

- 0,3% from the total surface (\approx 16 ha) was occupied with weak desalt soils, with CTSS between 2 and 8 mmho/cm (150-350 mg/100 g soil);

- 0,14% from the total surface (\approx 101 ha) was occupied with medium salts, with CTSS between 8 and 15 mmho/cm (350-650 mg/100 g);

- 1,4% from the total surface (\approx 1008 ha) was occupied with strong salts soils having CTSS > 15 mmho/cm (>650 mg/100 g soil);

The situation was even better on the

territory of Filipoiu land reclamation system: the entire surface was occupied with desalt soils in the year 1965.

The studies were started upon the evolution of the salt content, of the soil covering, in 1970 and it continued until 1988. The points, from which the soil samples have been cropped, were traced in the proximity of the hydrogeologic wells from which were cropped the water samples for the evolution pursuit of the phreatic water desalt. In this way, it existed 19 points of measure for Filipoiu system. The soil sample was cropped, on horizons, from 25 in 25 cm, until a depth of 1,50 mIt was determined for each soil sample in the laboratory Total Content in Soluble

Salts (TCSS), It was possible the establish of punctual situation by the means of the measurements and of the processing (at a certain moment) and in dynamics, of the soil salts from Filipoiu system.

3. Obtained results

The detailed studies, made upon the dynamic of the salts over those 18 years of measurements, have been processed for the making evidence of the evolution of CTSS, on horizons. By this meaning, about the layer comprised between 0 and 25 cm, it was able to observe the following (fig.1):





- after draining, until the integral establish of the draining system, because of the drying climate, characteristic for the area, on the essential feature of the absence of washing processes, determined by the floods, it was produced a continuous accumulation of the soluble salts from soils. This fact done as in 1970 entire surface to be effectuated by a reduced process of desalting.

- in the next stage of exploitation in diking, drained and drainage stage (1971-1976), on the essential feature of the abundent years in rainfall (1971, 1972, 1973) and of an important infiltrations on Danube (1973, 1975) it was observed a process of reducing the quantities of the salt from the soil, for the year 1977, 37% from the surface turned back in the group of desalt soils and the rest (63%) was represented by the weak salt soils. In this desalt process, a major roll played the drained surfaces charged with salts which it realized the interception and evacuation of the percolation water.

- the introduction of the irrigation (in 1977) intensified the washing process of the salts from the current profile. After 11 years of irrigation, in the spring of 1988, the surface distribution on salts grades indicates (fig.1) a extension of the desalt soils of about 66%. As a remark, we can say that in 1988 have started to be identify medium salt soils, also in Filipoiu system. These (about 45 ha) were found in low amplitude areas, being characterized through a badly natural drainage and without any possibilities of discharge in the canals draining network.

The situation, under progressive report was almost identically with the one described previous, just the mode of surface distribution on salt grades was different, for the soil layer placed between 0 and 1 m. For exemplifying and comparison, in fig. 2 it is presented the situation of the salting in soils, for a depth of 0-1 m, during the spring of 1988, in Filipoiu land reclaiming system. It may be observed , also in this case, that the occupied surface of desalt soils is almost the same (~61%). The measures showed the presence of some surfaces covered with light and medium salt soils, in case of the soil profile 0-1 m also.

The nature of salting of the soils in Filipoiu system was identified as being prevailing sulphatic.

By watching the evolution of salting the soils, on geomorphologycal areas from the system, it results the following observations:

- on the top of the ridge, the variations were more increased, because of the intense change and permanent of salts between phreatic water (supplied or drained by Danube) and soil. As it was shown before, in this area variation amplitude of phreatic levels is maximum and this fact contributes in the the soil profile;

- in the intermediate area, the variation in time of the salting of the soil are more reduced. It



Fig.2. The surfaces distribution on salting grades in Filipoiu system, 1998, spring (the depth of the soil profile 0–1m)

appeared areas with high capacity of salts accumulation, because of low permeability and of descending circulation's (percolation), on profile, negligible.

- in the depression area, the salts variation were more reduced in time, but, because of the absence of natural and/or artificial drainage, there existed the highest potential of secondary salting of the soil profile in this area.

4. Conclusions

The date's analysis regarding the salting working conditions of the soils, in relation with the most important natural factors (rainfalls, evapo-perspiration, elevation of Danube waters) and antropic factors (irrigations, draining and drainage) pointed out the following:

there exists a season, annual and multiannual variation, as well quantitative as qualitative of the charging degree with salts of the soil cover. In principle, the dry winters and autumn, when the phreatic levels are low, releases an accumulation process of the salts, most in the case of the superficial horizon of the soil. During summer and autumn, under the action of the intense irrigations and precipitation's, it is produced a reduction of the total quantity of salts from the soil profile.

- the application of irrigation in the area modified substantially, in a positive way, the salting balance sheet of the soils, so that in 1988, as well in 0-25 cm profile as in 0-1m (fig.1;2) one, the desalt soils occupied over 60% from its surface, after what the execution of the diking caused an increase, on the entire surface, of the total content of the soluble salts from the area.

Kovada marks out 4 types of hydrosalt conditions [65] by analyzing the evolution process of the soils under the influence of the irrigation and drainage works:

- regime of season desalting - unreversible, characteristic for the soils in which CTSS increase permanently, producing and intensifying the phenomenon of secondary desalting;

- regime of season salting – reversible, characteristic for the soils in which conditions of desalting are found, but CTSS remains relatively constant;

- regime of season desalting – reversible, characteristic for the soils in which desalting conditions are found, but CTSS remain relatively constant;

- regime of season desalting – unreversible, characteristic for the soils in which CTSS decrease permanently. This last type of regime is characteristic for the areas in which there are applied antropic measures of desalting are applied.

- If it is analyzed the specific situation of Filipoiu system from I.M.B., in function of Kovda classification, we may show that, for the year 1988, there was fixed a desalting regime which is also season – reversible, in the area.

In conclusion, the processing and interpretation of the collected dates through almost 28 years, by the means of the dynamics of soil salting from Filipoiu system, in functionality condition of the land reclamation works, gives us the right to make the following observations:

- the application of the land reclamation works is of nature to raise the bid a desalting regime season reversible in the area;

- the absence of some autochthonous saliferic sources, the presence of alluvial parental materials, with good and very good permeability's, it is produces that, the installations of the desalting regime season – reversible is done at a reduced portion of CTSS on the entire soil profile;

- locally, most in the depression areas, of the plashes, brooks or pond bottom, natural deficient drainage may determine the appearance of accented phenomenon of salting. Here, it is absolutely necessary the accompany of the land reclamation arrangement with works of leveling, breaking up and as much as there exists any tendency of salting growing of irrigation with washing role.

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Variation of the Ground Water Table Due to Drainage Works and Environment

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Abstract: This paper analyses the variation of the ground water table in the Aranca surface-drainage system in the department of Timiş. Research on the hydrostatic ground water table started in 1975 and continued for 20 years. The data presented in the paper have been collected from 74 observation wells, which informed on the variation of the ground water table, from 4 rain-gauge points, which recorded the rainfalls, and from 8 pedological profiles, which observed the soil moisture evolution.

Keywords: Hydrostatic ground water table, ground waters.

1. Introduction

There are three hydrogeological units that can be observed in the Low Plain of Banat :

- the hydrogeological unit of the Pliocene-Pleistocene formations;

- the hydrogeological unit of the Pleistocene-Holocene formations;

- the hydrogeological unit of the Pleistocene superior Holocene alluvial formation.

The Pleistocene formations contain low mean aquifers, consisting of sands, and situated in the area of Sânnicolau Mare and Periam, at 8-350 m.

In the Aranca Plain, the aquifer is radical convergent, having an east-west water-course direction, and being supplied by rainfalls and by subterrane inflow from the Vinga Plain. The aquifer is 1-3 m and 3-5 m, deep its piezometric level amplitude being of 0,5-1 m.

2. Methodology

The research concerning the hydrostatic ground water table started in 1975 on an area of 96,735 ha and was carried on until 1995, on an area of 60,120 ha.

Twice a month, the hydrostatic ground water table was checked.

The number of the observation wells increased in time. In 1989, there were 173 observation wells, 109 belonging to IEELIF and 64

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to CNA. Since 1990 the variation of the ground water table was controlled by 74 observation wells, 41 of them belonging to RAIF and 33 to RA. Of the 41 observation wells of RAIF, only 20 were working, 8 being considered as characteristic.

These observation wells were distributed in a dispersed manner according to the repartition of the close to the surface-ground water table areas.

There were also 4 rain-gauge points, which were used to record the rainfalls. They were situated at SP Aranca, SP Galaţca, at the system headquarters-Sânnicolau Mare and at Periam.

3. Results

The rainfalls recorded at the rain-gauge points are presented in Table 1.

Comparing the rainfalls between 1981 and 1995, one can notice that during the interval autumn-winter, they decreased from 301,2 l/mp in 1981 and 1982 to 148,0 l/mp in 1983 and 1984. By 1985 and 1986, they increased to 258,5 l/mp. The slightest decrease – 100,8 l/mp – took place in 1988 and 1989. By 1994 and 1995, the rainfalls values varied rather slowly, between 201,2-242,2 l/mp.

In what concerns the rainfalls between 1981 and 1995, during the interval spring-summer, one can notice that the rainfalls were slightly equal between 1982 and 1983. In 1984, there was a slight increase to 299,4 l/mp followed in 1985, by a decrease to 280,7 l/mp. In 1986, there was a sudden increase to 332,2

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l/mp, followed in 1987 and 1988, by a decrease to 284,2 l/mp and to 251,8 l/mp, respectively.

Year		Rainfalls (average per system)l/mp
	1 X –31 III	1 IV – 30 IX
1981/82	301,2	-
1982/83	158,2	264,7
1983/84	148,0	246,4
1984/85	220,1	299,4
1985/86	258,5	280,7
1986/87	187,6	332,2
1987/88	241,5	284,2
1988/89	100,8	251,8
1989/90	205,2	377,6
1990/91	242,2	205,5
1991/92	201,8	353,2
1992/93	238,1	295,5
1993/94	228,3	219,7
1994/95	230,5	239,1
1995/96	-	248,3

Table 1. Rainfalls recorded between 1981-1995

In the summer of 1989, a new rainfalls increase to 377,6 l/mp was recorded. A new rainfalls decrease to 205,5 l/mp took place in 1990.

The year 1991, considered as a rainy year, brought a rainfall increase to 353,2 l/mp. Until 1995, there was a rainfall decrease, the years 1993 and 1994 being considered as droughty.

Concomitant with the rainfalls, the variation of the ground water table (table 2), during the interval autumn-winter and spring-summer, was recorded in cm.

During the interval autumn-winter, a decrease in the ground water table from 195 cm (1977/78) to 213 cm (1980/81) can be noticed in Table 2.

Table 2. Ground water depth in the Aranca system (mean values)

Year	Autumn-winter	Spring-summer
1977/78	195	188
1978/79	201	196
1979/80	209	187
1980/81	213	190
1981/82	152	170
1982/83	218	243
1983/84	265	245
1984/85	228	198
1985/86	229	187
1986/87	221	182

					Yea	r						
Drilling	Level	1987	1988	1989	1990	1991	1992	1993	1994			
_		1988	1989	1990	1991	1992	1993	1994	1995			
	mean					324	407	432	436			
P11	max.		on v	vater		325	412	439	442			
	min.				323	403	427	430				
	mean	344	338	327	308							
P12	max.	394	339	340	314	on water						
	min.	313	338	302	301							
	mean	214	215	245	200	159	212	216	218			
P13	max.	241	219	256	204	164	220	219	221			
	min.	171	212	233	195	155	214	211	213			
	mean	132	173	189	193	245	318	273	276			
P14	max.	142	174	256	197	246	320	279	282			
	min.	116	171	167	190	243	317	266	269			
	mean	223	272	223	288	252	314	312	314			
P17	max.	233	279	248	291	256	316	313	315			
	min.	213	269	208	284	249	312	309	311			

Table 3. Water ground depth (cm, meanvalues), autumn-winter

Comparing to the 152 cm ground water table in 1981 and 1982, the ground water table decreased continually, reaching 265 cm in 1983 and 1984, then increasing continually to 221 cm in 1986 and 1987.

During the interval spring-summer, the ground water table varied between 188 cm (1977/78) and 190 cm (1980/81). From 1981 and 1982 until 1983 and 1984 it decreased from 170 cm to 245 cm, and increased to 182 cm in 1986 and 1987.

The records of the hydrostatic level in some characteristic observation wells during the intervals autumn-winter and spring-summer between 1987 and 1995, are presented in Tables 3 and 4.

A significant number of drillings (P15, P16, P18) show the absence of water. This situation appeared in 1988 and 1989.

A significant descent of the ground water tables 3 and 4 in the whole Aranca system can be determined.

According to the data concerning the rainfalls data, recorded at the rain-gauge points of the combined irrigation and drainage system, this ground water table descent is considered to be determined by the rainfalls decrease, by a droughty period of time starting with 1988 and 1989, and by the evaporation losses.

In the 70 s, in tight connection with the rainfalls and the variation of the ground water table, the distribution of ground water areas having different minimum depths and these areas percentage were taken into account (Table 5).

The percentage of ground water areas of minimum depth between 1970-1995 is presented in Table 6.

In 1983 and 1984 as well as in 1985 and 1986, the ground water areas of 0 - 1 m accounted for only 4% of the system's area (Tables 5).

The ground water areas of 1 - 2 m depth are of 26-27% during the interval autumn-winter and of 38-48% the interval spring-summer.

The ground water areas of 2-3 m accounted for 67-70% during the interval autumn-winter and for 45-58 % during the interval spring-summer.

Starting with the years 1986 and 1987, the ground water areas of 0 - 1 m disappeared.

During the interval autumn-winter, the ground water areas of 1 - 2 m accounted for only 22 %, in 1986 and 1987 for 8% and in 1988 and 1989 for 6%. The ground water areas of 2 - 3 m increased annually, from 78% in 1986 and 1987 to 92% in 1987 and 1988, and to 94% in 1988 and 1989.

Drilling	Level					Year					
		1987	1988	1989	1990	1991	1992	1993	1994	1995	
P11	mean	161					344	408	442	446	
	max.	178		on v	vater		347	410	444	448	
	min.	110					341	407	440	444	
P12	mean		325	353	309	287					
	max.	-	337	356	350	292		on w	vater		
	min.		314	349	288	282					
P13	mean		173	243	266	152	186	226	220	223	
	max.	-	189	246	276	159	191	228	223	226	
	min.		163	240	251	145	181	223	217	220	
P14	mean		117	211	191	211	291	301	256	258	
	max.	-	161	214	214	214	294	306	259	261	
	min.		62	206	160	208	287	296	252	254	
P17	mean		247	268	241	254	262	302	294	298	
	max.	-	292	273	274	264	265	306	296	300	
	min.		212	261	213	244	258	298	291	295	
Total	mean	182	237	298	252	226	271	309	303	306	
system	max.	224	353	381	350	292	347	410	444	448	
	min.	85	62	206	160	145	181	223	217	220	

Table 4. Water ground depth (cm, mean values), spring-summer

During the interval spring-summer, the ground water areas of 1 - 2 m represented 35% in 1986 and 1988, while in 1989, they disappeared.

In 1986 and 1988, the ground water areas of 2-3 m represent 65% and expand to 100% in 1989.

During the interval autumn – winter in 1989 and 1990, the ground water areas of 1 - 2 m occupyed 23%, in 1990 and 1991, they decreased to 14% to increase to 27% in 1991 and 1992. In what concerns the ground water areas of 2-3 m, they increased from 77% in 1989 and 1990 to 86% in 1990 and 1991 to finally decrease to 34% in 1991 and 1992. This decrease was also due to the fact that, in 1991 and 1992, ground water areas of 3-4 m on 39% began to appear.

During the interval spring-summer in 1989 and 1990, only ground water areas of 2-3 m were recorded, which thus represented 100% of the drained area.

Comparing to the years 1989 and 1990 when ground water areas of 1-2 m were not recorded, in

1991, this type of area increased to 33%. In return, ground water areas of 2-3 m decreased from 100% in 1990 to 67% in 1991 and There was a slight difference between the quantity of the rainfalls during the interval autumn-winter and those of the interval spring-summer (1991/1992).

As a consequence, the ground water depths during the interval autumn-winter and spring-summer were equal.

Between 1992 and 1995, there were no ground water area from 0-1 m to 1-2 m.

During the interval autumn-winter the ground waterarea of 2-3 m decreased from 31% in 1992 and 1993 to 19% in 1993 and 1994 and to 18% in 1994 and 1995. The ground water area of 3-4 m

decreased from 32% in 1993 to 14% in 1994 and to 13% in 1995. In exchenge , at the same time the ground water area of over 4 m was increasing, from 37% in 1993 to 67% in 1994 and to 69% in 1995.

The same ground water depths were maintained during the interval spring-summer.

	Table 5. Per	centage of gr	ound water area	as of minimum de	epth (%)	
Year	Period		Gr	ound water depth	, m	
		0 - 1	1 - 2	2 - 3	3 - 4	> 4
1970/71	1 IV-30 IX	67,0	23,0	8,0	2,0	-
	1 X- 31 III	2,0	46,0	48,0	4,0	-
1971/72	1 IV-30 IX	46,0	41,0	13,0	-	-
	1 X- 31 III	1,0	48,0	34,0	17,0	-
1972/73	1 IV-30 IX	0,5	29,5	53,0	17,0	-
	1 X- 31 III	0,2	26,0	58,0	15,8	-
1974/75	1 IV-30 IX	-	14,0	64,0	22,0	-
	1 X- 31 III	2,0	37,0	48,0	13,0	-
1976/77	1 IV-30 IX	1,0	50,0	38,0	11,0	-
	1 X- 31 III	1,0	57,0	35,0	7,0	-
1977/78	1 IV-30 IX	1,0	50,0	37,0	12,0	-
	1 X- 31 III	1,0	41,0	46,0	12,0	-
1980/81	1 IV-30 IX	1,0	41,0	41,0	17,0	-
	1 X- 31 III	1,0	32,0	43,0	24,0	-
1983/84	1 IV-30 IX	4,0	29,0	67,0	-	-
	1 X- 31 III	3,0	38,0	58,1	-	-
1985/86	1 IV-30 IX	3,9	26,0	70,1	-	-
	1 X- 31 III	2,7	48,8	48,5	-	-
1986/87	1 IV-30 IX	-	22,0	78,0	-	-
	1 X- 31 III	-	34,6	65,4	-	-
1987/88	1 IV-30 IX	-	8,0	92,0	-	-
	1 X- 31 III	-	35,0	65,0	-	-
1988/89	1 IV-30 IX	-	6,0	94,0	-	-
	1 X- 31 III	-	-	100,0	-	-
1989/90	1 IV-30 IX	-	23,0	77,0	-	-
	1 X- 31 III	-	-	100,0	-	-
1990/91	1 IV-30 IX	-	14,0	86,0	-	-
	1 X- 31 III	_	33,0	67,0	-	-
1991/92	1 IV-30 IX	-	27,0	34,0	39,0	-
	1 X- 31 III	-	27,0	34,0	39,0	-
1992/93	1 IV-30 IX	-	-	31,0	32,0	37,0
	1 X- 31 III	-	-	31,0	32,0	37,0
1993/94	1 IV-30 IX	-	-	19,0	14,0	67,0
	1 X- 31 III	-	-	21,0	13,0	66,0
1994/95	1 IV-30 IX	-	-	18,0	13,0	69,0
	1 X- 31 III	-	-	18,0	13,0	69,0

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4. Conclusions

In the Aranca system, the ground water deepening after 1987/88 and especially in 1991 when ground water areas of 3-4 m and even over 4 m were dominant is obvions.

The close to the surface-ground water areas,

of 0-1 m, totally disappeared after 1986/87.

After 1992/93, the ground water areas of 1-2 m, also disappeared.

This phenomenon can be seen as a negative one, as an excessive surface-drainage having led to the loss of the ground water supply in the last years.

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Programming Water in Irrigation System Based on Forecast of Droughts Used ANN's Approach

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Abstract: Programming water is considered to be a significant operation for irrigation systems management. In aim to combat the damages produced by droughts through irrigation for Eastern Romania, in this paper has been proposed a method for early warning of irrigation systems based on forecast drought (quantitative characterized by the Humidity Index (*Iu*), proposed by Soroceanu) in epicenter. For this purpose has been developed an artificial neural network (ANN) considering monthly average values of precipitation and evapotranspiration, measured in Galati. Supplemental data processing (preprocessing) were added for the training set.

Keywords: forecast of watering, drought, artificial neural networks.

1. Introduction

In Eastern part of Romania, irrigation projects within catching areas of Prut and Siret rivers, are placed on flood plains, hilly areas and terraces. Pumping water for table lands and hills claim high energy consumption. In these conditions irrigation water is expensive and determines water application based on economic criteria of profitableness and it maximizing.

Forecast of watering, respectively of the instant of water application and amounts of watering rates are necessary for economic efficiency of irrigation.

Previous study [2] was presented elements that help translating characteristics of droughts in criteria of forecasting irrigation.

Damages due droughts depend by some characteristics as: intensity, duration, frequency and the affected area. Intensity of drought is the main factor, of it depend the others [1].

The paper has the objective to use forecast variation of drought intensity in next month of vegetation season using ANN for programming water in irrigation systems.

2. Researches and their results

2.1. Humidity Index

Intensity index of the drought used is proposed by N. Soroceanu [5], defined as a non-dimensional ratio of infiltrate part of the rainfall and water consumption from soil at potential evapotranspiration by the relationship:

$$Iu = \frac{a \cdot q + Q}{etn} \tag{1}$$

where: q is the sum of rainfalls during accumulation period of soil humidity (month XI ... II); a - storing coefficient of the water during cold season (XI ... II); Q - summed precipitations from March 1 up to date of calculus of Iu; etp - summed evapotranspiration from March 1 up to the calculus date. Values of adepend on month of which end Iu is computed: 0.6 for March and April; 0.5 for May; 0.4 for June; 0.3 for July; 0.2 for August; 0.1 for September and 0 for October.

Droughts are characterized with respect Iu as follows:

$Iu = 1 \dots 0.75$	slow drought;
$Iu = 0.75 \dots 0.5$	moderate drought

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$Iu = 0.5 \dots 0.25$	severe drought
<i>Iu</i> < 0.25	extreme drought

2.2. Features on ANN

Recently, ANNs have been applied extensively in many prediction tasks, mainly in modeling complex and/or nonlinear phenomena.

Artificial neural networks (ANNs) are a form of artificial intelligence (AI), which in their architecture attempt to simulate the biological structure of the human brain and nervous system [3,4]. ANNs consist of a number of artificial neurons (variously known as "processing elements", "PEs", "Nodes" or "Units"). An individual PE receives its inputs from many other processing elements via weighted input connections. These weighted inputs are summed and passed through a transfer function to produce a single activation level for the processing element, which is the node output. A typical structure of artificial neural networks consists of many processing elements that are arranged in layers: an input layer, an output layer, and one or more layers in-between, called intermediate or hidden layers (Fig 1).



Fig. 1. ANN structure with one hidden layer

Each processing element in a specific layer is interconnected to all the processing elements in the next layer via weighted connections. The scalar weights determine the strength of the connection between interconnected neurons. A zero weight refers to no connection between two neurons and a negative weight refers to a prohibitive relationship. In the training phase, at the input of the ANN a set of input vectors are presented; passing through the network of these vectors produce the output values, that are compared with the corresponding target values. The training process leads to the adjustment of weights and biases by minimizing a cost function; for example a *SSE* function (sum squared error).

2.3. The architecture of ANN

The necessary information for drought phenomena modeling tacking in account monthly average values of precipitation and evapotranspiration, measured at Galati for a period of time along 16 years (1981-1996).

The data set was split into two files, the first of this contains 13 years (1981-1993), used to train the ANN, and the second file contains 3 years (1994-1996), that has been used to test and evaluate the accuracy of the predictions of the trained ANN.

Supplementary data processing (preprocessing) has been added for the training set, by trend subtract. A linear trend has been considered:

$$x_d = x - (a + b \cdot t) \tag{2}$$

where: x_d is detrend value; x - registered value; a, b - coefficients (for precipitation a=43.62 and b=0.027 respectively for evapotranspiration a=71.43 and b=0.016).

The structure adopted and tested for the ANN has two hidden layers, with 35 and 20 neurons respectively, 7 inputs and one output (forecast monthly value of Iu).

For the input vectors the structure presented in table 1 was adopted.

The PC-based software package Matlab 5.1 is used in this paper to simulate the artificial neural network operation.

For the training phase, the *initff* and *trainlm* routines were used, for the network initialization and training, respectively. The *trainlm* routine implements a Levenberg Marquant back-propagation with momentum algorithm. The adopted error threshold for the cost criteria was SSE = 0.02 (fixed implicitly by the routine).

The ANN validation tacking in account the relationship between real and obtained results (using the *simuff* routine), representing the forecast of

monthly *Iu* values for the years 1994, 1995, 1996. Results are presented in fig.2.

The predictors' accuracy was evaluated according to the r^2 values (r^2 =0.982).

	Table 1. Inputs of ANN									
Current member of	Symbol	Description								
1	a _i	Coefficient corresponded month (established by Soroceanu)								
2 4	etpd _i	Actual detrend value (current month) and 2 previous detrend values								
		of monthly mean values of evapotranspiration for the significant								
		period (from March 1 to October 31)								
5 7	ppd _i	Actual detrend value (current month) and 2 detrend previous values								
		of monthly mean values of precipitation (for the same period)								



Fig.2. Correlations of obtained and real values of *Iu*

2.4. Forecasting method of watering at distance

For irrigation projects at a distance, in middle and Northern counties of Moldavia, spreading drought from epicenter to these is shown in table 2.

 Table 2. Duration of spreading drought (in days)
 from epicenter Galati with respect its intensity

Intensity	slow	moderate	heavy	extreme
Barlad	6-8	8-9	9-11	11-13
Husi	6-8	8-16	16-21	21-22
Bacau	25-30	28-48	48-57	57-61
Iasi	37-40	40-57	57-72	72-188

Operations follow the bellow:

a) Forecasting *Iu* in epicenter for the next month;

b) Forecast the approximately date when this value of Iu will be registered in zone of irrigation system.

c) For forecasted *Iu*, by correlation *Iu* - lost of crop, will be forecast the relative lost of crop.

d) Using correlation Iu - utile water reserve of the soil, Ru the real water application rate will be determined.

e) Taking in account water application rate, expenditure of watering will be compute (product of unit cost of 1 m^3 water and m).

Comparison of the cost of the water application rate and value of the forecasted lost crop will decide if that watering during that decade (or week) is profitable or not.

4. Conclusions

In case of irrigated areas situated in Eastern part of Romania specific energetic consumption for irrigation has high values.

For programming watering in irrigation systems has been developed a methodology based on forecast variation of drought intensity in epicenter.

In that aim a back-propagation ANN was used for the monthly prognosis of Iu. Trends subtract for the training set was used for preprocessing data. The results show that ANNs are efficient and flexible tools in forecast of *Iu*.

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Laboratory Method Proposed for the Determination of the Hydraulic Conductivity for the Unsaturated Soil with Medium Texture

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Abstract: The hydraulic conductivity *k* represents a very important parameter for the practice of irrigations, because intercede in the calculation of the watering technique elements: the watering norm, the watering length, the watering qualitative indexes and so on. In order, to determine it, the literature of specialty recommends either methods based on measurements made directly on the fluid, or tests purposefully prepared in labs [1], [2], or empirical calculation relations. This paper presents a method used to calculate the hydraulic conductivity when the soil has a medium texture, on the active moisture domain ($C_c - P_{min}$), in which the plants normally breed and develop.

Keywords: irrigation, unsaturated soil, filtration coefficient, hydraulic conductivity

1. Introduction

The laboratory method proposed here for the determination of the hydraulic conductivity of the unsaturated soil (named also hydroconductivity), is based on the observation that this parameter, which characterize the possibility of water to circulate through the soil whose humidity at that moment is smaller than the humidity of saturation, due to the resultant of the forces that has affects on the water particles contained in the soil at that moment. The intensity of each force (gravitational, capillary, etc.) depends on the momentary humidity of the soil. So, there is a very strong dependence between the momentary humidity of the soil, the physicomechanical characteristics and the resultant of the forces that action upon the water contained in the soil, all of these establishing the size of the hydraulic conductivity.

2. Experimental conditions and results

In the elaboration of this method, we started from the observation of migration phenomenon on the soil, profile when this is supplied with the water volumes necessary to the growing of humidity to its saturation value, water supply made from its surface [3]. An artificial rain, like liquid rains or ISSN-12223-7221 rain effected the administration of these volumes of water by sprinklering in case of irrigations. The rain intensity was mentained constant during the raining, its value being smaller than the value of the soil filtration speed, to avoid the breakage of the texture and the structure that appear at the unexpected expulsion of air from the defective spaces or the water bogging at the surface. The experiments were made on natural soil samples, knowing that physico-mechanical properties of soil change at the same time with the changing of its setting.

The measurements were made using a stand of working like in Fig.1.



Fig.1-Experimental stand for the determination of the hydroconductivity

For the problem studied there were picked some soil samples witch are included in the medium textural class (sample 1: 30% clay, 35% dust, 35% sand;

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sample 2: 24,9% clay, 32,1% dust, 43% sand; sample 3: 15% clay, 42,5% dust, 42,5% sand).

The notations used have the following meaning:

• A = soil sample; B = metallic container; T = tensiometers; C = opposed filter; W= moisture meter;

- 0-0 = section at the surface of the soil sample;
- 1 1 = section to measure;
- 1' 1' = section where the humidity front can be found when the section 1-1 has the saturation humidity

The necessary operations for the experimental determinations are the following:

• in the experimental stand from Fig. 1, each sample of those three soil textural species experimented are disposed, one by one; the initial humidity w_o was measured;

• the measurements instruments for humidity and suction, are fixed in the section 1-1, section situated at the distance L by the section at the surface, 0-0

• using the wetting installation, the water supply starts, at the moment t_o ;

• the moment t_1 is established by timekeeping, when the humidity front reaches the section 1-1 and then, the moment t_s when, in this section, the value of the soil humidity is maximum; this value is about the saturation value (94 ÷ 96%), and is constant during the whole period of water supplying;

• the measurements are repeated with all the soil samples, using different values of the initial humidity.

The results of the experiments are presented in the tables 1, 2 and 3.

The meaning of the terms in the tables 8 - 10 is the following:

 w_{oi} = the humidity when the experiment "*i*" took place;

 t_o = the moment when the wetting in the section 0 - 0 started;

 t_1 = the moment when the humidity front reaches the section 1 - 1;

 t_s = the moment when the humidity of the soil in section 1 - 1 reaches the maximum value (about the saturation value);

L = the distance between the sections 0 - 0 and 1 - 1;

 $h_{s i}$ = the initial suction of the soil (the same on the whole surface of the sample);

 h_{sf} = the suction of the soil in the section 1 - 1 at the moment t_s ;

 v_F = speed of the advance of the humidity front, calculated with the relationship:

$$v_F = L / (t_I - t_o) \tag{1}$$

where ΔL = the way passed by the humidity front in time intervals (t'_l-t_l) :

$$\Delta L = v_F \left(t_s - t_I \right) \tag{2}$$

where k_{wi} = the hydraulic conductivity of the unsaturated soil having the humidity w_{oi}

The relation used to calculated the hydraulic conductivity of the unsaturated soil is [3]:

$$k_{w} = \frac{v_{F} \cdot \Delta L}{\Delta L - \left(h_{sf} - h_{si}\right)} \tag{3}$$

Nr. crt.	w _{oi} [%]	t _o	t_{I}	<i>t</i> ₁ - <i>t</i> _o [min]	t _s	t_s - t_1 [min]	L [cm]	-h _{si} [mbar]	- <i>h</i> _{sf} [mbar]	$\frac{v_F}{\left[\frac{cm}{min}\right]}$	∆L [cm]	$\begin{bmatrix} k_{wi} \\ cm \\ min \end{bmatrix}$
1	20	7 ⁵⁵	10 ²²	147	10 ⁵⁹	37	25	750	60	0.170	6.29	1.53×10^{-3}
2	30	14^{28}	16^{36}	128	17^{09}	33	25	580	60	0.195	6.44	2.38×10^{-3}
3	40	16 ⁵⁷	1855	108	18^{56}	31	25	460	60	0.231	7.17	4.05×10^{-3}
4	50	10^{50}	12^{36}	96	12^{55}	29	25	360	60	0.260	7.55	6.38×10^{-3}

Table 1. Experimental results concerning the hydroconductivity; sample 1

Nr. crt.	w _{oi} [%]	t _o	t_{I}	<i>t₁-t_o</i> [min]	t_s	<i>t_s-t₁</i> [min]	L [cm]	-h _{si} [mbar]	- <i>h_{sf}</i> [mbar]	$\frac{v_F}{\left[\frac{cm}{min}\right]}$	∆L [cm]	$\begin{bmatrix} k_{wi} \\ \hline cm \\ \hline min \end{bmatrix}$
1	10	9 ³³	12^{37}	184	13 ¹⁴	37	25	1100	60	0.135	5,02	0.74×10^{-3}
2	20	15^{45}	17^{59}	134	18^{31}	32	25	650	60	0.186	5,97	1.86x10 ⁻³
3	40	10^{20}	12^{00}	100	12^{27}	27	25	400	60	0.250	6,75	4.76×10^{-3}
4	50	8 ³⁵	9 ⁵⁹	84	10^{24}	25	25	330	60	0.297	7.44	7.98x10 ⁻³

Table 2. Experimental results concerning the hydroconductivity; sample 2

Table 3. Experimental results concerning the hydroconductivity; sample 3

Nr. crt.	W _{oi} [%]	t _o	t_I	<i>t</i> ₁ - <i>t</i> _o [min]	t _s	t_s - t_1 [min]	L [cm]	- <i>h</i> _{si} [mbar]	- <i>h</i> _{sf} [mbar]	$\frac{v_F}{\left[\frac{cm}{min}\right]}$	∆L [cm]	$\begin{bmatrix} k_{wi} \\ cm \\ min \end{bmatrix}$
1	10	8^{20}	11^{18}	178	11 ⁵³	35	25	950	60	0.140	4.91	0.76×10^{-3}
2	20	9^{00}	11^{00}	120	11^{25}	25	25	570	60	0.217	5.21	2.01×10^{-3}
3	40	10^{45}	12^{15}	90	12^{34}	19	25	310	60	0.277	5.28	5.72×10^{-3}
4	50	10^{30}	11^{50}	80	12^{07}	17	25	240	60	0.312	5.31	9.01×10^{-3}

3. The processing of the experimental results.

The processing of the experimental results was made using the least squares method (Table 5, 6, 7), resulting hydroconductivity, k_w , with the humidity w, (Fig. 2, 3, 4) and the analytic expressions of these functions, such as: $k_w = m w^2 - n w + s$ (4) The value of the coefficients *m*, *n*, respectively *s*, are different for each sample, and they are placed in the Table 4, where the humidity is expressed in

[cm³/cm³], and hydroconductivity in [cm/min].

1 ås:

Sample	т	п	S	$k_w = mxw^2 - nxw + s$
1	37·10 ⁻³	-9.68·10 ⁻³	1.968·10 ⁻³	$k_w = (37 \cdot w^2 - 9.68 \cdot w + 1.968) \cdot 10^{-3}$
2	$35.2 \cdot 10^{-3}$	-3.73·10 ⁻³	$0.912 \cdot 10^{-3}$	$k_w = (35.2 \cdot w^2 - 3.73 \cdot w + 0.912) \cdot 10^{-3}$
3	$34 \cdot 10^{-3}$	$-0.19 \cdot 10^{-3}$	$0.522 \cdot 10^{-3}$	$k_w = (34 \cdot w^2 - 0.19 \cdot w + 0.522) \cdot 10^{-3}$

Table 4. The values of the coefficients m, n and s for the three soil samples

Table 5. The analytic processing of the experimental results concerning hydroconductivity-sample 1

w	w^2	w^3	w ⁴	k_w	$k_w \cdot w$	$k_w \cdot w^2$	k _{w-e}	e	<i>e</i> [%]
0.2	0.04	0.008	0.0016	1.53	0.306	0.0612	1.522	0.008	-0.52
0.3	0.09	0.027	0.0081	2.38	0.714	0.2142	2.404	-0.024	1.0
0.4	0.16	0.064	0.0256	4.05	1.62	0.648	4.026	0.024	-0.6
0.5	0.25	0.125	0.0625	6.38	3.19	1.595	6.388	-0.008	0.12
1.4	0.54	0.224	0.0978	14.34	5.83	2.5184	14.34	-	-

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Total $d = 8 \cdot 10^{-4}$	$s = 1.978 \cdot 10^{-3}$	$k_{w} = \frac{k_{w} - k_{w-e}}{100}$
$d_1 = 15,824 \cdot 10^{-4}$	$n = -9.68 \cdot 10^{-3}$	$\mathbf{e}_{[\%]} = \frac{1}{k_w}$ 100
$d_2 = -77,44 \cdot 10^{-4}$	$m = 37 \cdot 10^{-3}$	$k_w = [37 \cdot w^2 - 9.68 \cdot w + 1.978] \cdot 10^{-3}$
$d_3 = 296 \cdot 10^{-4}$	MAD = 0.016	



Fig. 2. The variation curve of hydroconductivity - humidity in time, $k_w = f(w)$; sample 1

 Table 6. The analytic processing of the experimental results concerning hydroconductivity-sample 2

	w	w^2	w^3	w^4	k_w	$k_w \cdot w$	$k_w \cdot w^2$	k _{w-e}	е	e[%]
	0.1	0.01	0.001	0.0001	0.748	0.0748	0.0075	0.7943	0.03	4.8
	0.2	0.04	0.008	0.0016	1.86	0.372	0.0744	1.7746	-0.09	-4.8
	0.4	0.16	0.064	0.0256	4.766	1.9064	0.7626	5.0514	0.285	5.9
	0.5	0.25	0.125	0.0625	7.99	3.995	1.9975	7.5943	-0.395	-0.61
Total	1.2	0.46	0.198	0.0898	15.364	6.3482	2.8419	15.161	-	_

$$d = 0.00036$$

$$d_1 = 0.00032832$$
 $n = -3.73 \cdot 10^{-3}$
 $d_2 = -0.0013428$ $m = 35.2 \cdot 10^{-3}$

 $s = 0.912 \cdot 10^{-3}$

$$e_{[\%]} = \frac{k_w - k_{w-e}}{k_w} \cdot 100$$

 $d_3 = 0.012672$ MAD = 0.2145

$$k_w = [35.2 \cdot w^2 - 3.73 \cdot w + 0.912] \cdot 10^{-3}$$



Fig. 3. The variation curve of hydroconductivity - humidity in time, $k_w = f(w)$; sample 2

Table 7 The analytic r	processing of the	evnerimental re	culte concerning	hydroconductivity	· comple 3
radic 7. The analytic p	Juccosing of the	caperinental re	suits concerning i	invario contauctivity	, sample 3

w	w^2	w^3	w^4	k_w	$k_w w$	$k_w w^2$	k _{w-e}	е	e[%]
0.1	0.01	0.001	0.0001	0.76	0.076	0.0076	1.04773	-0.288	-5.2
0.2	0.04	0.008	0.0016	2.01	0.402	0.0804	1.61912	0.3909	-4.4
0.4	0.16	0.064	0.0256	5.72	2.288	0.9152	5.67988	0.0401	-0.7
0.5	0.25	0.125	0.0625	9.01	4.505	2.2525	9.16925	-0.159	1.7
1.2	0.46	0.198	0.0898	17.5	7.271	3.2557	17.516	-	-

Total	d = 0.00036	$s = 0.522 \cdot 10^{-3}$
$d_{I} = 0.0$	0018792	$n = -0.19 \cdot 10^{-3}$

 $m = 34 \cdot 10^{-43}$ MAD = 0.11981

 $d_2 = -6.84 \text{E}^{-05}$ $d_3 = 0.01224$

$$e_{[\%]} = \frac{k_w - k_{w-e}}{k_w} \cdot 100$$

$$k_w = [34 \cdot w^2 - 0.19 \cdot w + 0.522] \cdot 10^{-3}$$



Fig. 4 The variation curve of hydroconductivity - humidity in time, $k_w = f(w)$; sample 3

4. Conclusions

• The method proposed for the determination of hydraulic conductivity of the unsaturated soil with medium texture, reveals that this parameter has a parabolic variation (4). Its maximum value (when the soil is saturated) is named filtration coefficient k_{sat} ;

• Knowing the variation hydroconductivity white the soil humidity, we can correctly choose a very important wetting technical element, its duration;

• This method can by apply especially to the irrigation systems design, for they're working there is another expedite method [4].

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The General Solution of the Suction Variation Equation within the Active Moisture Domain

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Abstract: Considering the general solution of the water vertical movement in active soil stratum, the moisture variation equation and the hydraulic conductivity variation equation, experimentally determined, there had been obtained the suction variation equation in active soil stratum, within the active moisture domain.

Keywords: Hydraulic conductivity, suction, water vertical movement.

1. Introduction

The general solution of the water vertical movement in active soil stratum supplied with water from its surface, is:

$$\frac{\partial w}{\partial t} = -k_w \frac{dh_s}{dw} \frac{\partial^2 w}{\partial h^2} - \left(k_w \cdot \frac{d^2 h_s}{dw^2} + \frac{dk_w}{dw} \frac{dh_s}{dw}\right) \cdot \left(\frac{\partial w}{\partial h}\right)^2 - \frac{dk_w}{dw} \frac{\partial w}{\partial h}$$
(1)

Where:

w = the soil moisture [L³/L³]; k_w = the hydraulic conductivity of unsaturated soil [L/T];

 h_s = the suction [L];

t = the time [T];

h = the depth [L];

By making many experiments there have been established the analytical expressions of the functions that shows the soil moisture variation (both on the profile depth and in time) and the variation of the hydraulic conductivity of the unsaturated soil with its moisture: $w = w_0 + \alpha t - \beta h;$

Respectively $k_w = m w^2 - n w + s$ (2)

Where: α , β , m, n, s = numerical coefficient.

Regarding the suction variation – the sum of forces potential that operate on the soil water – the results of the experimental determinations show that this one can't be overwritten on non of the usual curves that can be determined with ordinary methods.

For this reason, the problem has been changed: instead of considering, in equation no.1 with partial derivatives, the moisture as unknown function and the other functions as known, in the following, this equation was considered as an ordinary differential equation, having the suction, h_s , as unknown function, the other ones being experimentally determined.

2. Experimental conditions and results

Introducing the expressions of the functions w=f(t, h) and $k_w = f(w)$ (experimentally determined) into relation no.1, after derivation, we obtain:

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$$\alpha = -(mw^2 - nw + s) \cdot 0 \cdot \frac{dh_s}{dw} -$$
$$-\beta^2 (2mw - n) \frac{dh_s}{dw} -$$
$$-\beta^2 (mw^2 - nw + s) \frac{d^2h_s}{dw^2} + \beta \cdot (2mw - n)$$

...

(3)

that is:

$$\beta^{2} (mw^{2} - nw + s) \frac{d^{2}h_{s}}{dw^{2}} + \beta^{2} (2mw - n) \frac{dh_{s}}{dw} + \alpha - \beta (2mw - n) = 0$$
(4)

$$\frac{dh_s}{dw} = p \quad \Rightarrow \quad \frac{d^2h_s}{dw^2} = \frac{d}{dw}\left(\frac{dh_s}{dw}\right) = \frac{dp}{dw} \tag{5}$$

the equation no.4 becomes:

$$\frac{dp}{dw} + \frac{2mw-n}{mw^2 - nw + s}p + \frac{\alpha - \beta(2mw-n)}{\beta^2(mw^2 - nw + s)} = 0$$
(6)

The equation no.6 is an inhomogeneous equation; first, we solve the homogeneous equation, that is:

$$\frac{dp}{dw} + \frac{2mw - n}{mw^2 - nw + s}p = 0 \iff$$
$$\Leftrightarrow \frac{dp}{p} = -\frac{2mw - n}{mw^2 - nw + s}dw$$
(7)

The general solution of equation no.7 is obtained by integration:

$$ln\frac{|p|}{C} = -ln|mw^2 - nw + s|,$$

as a result of:

$$p = \frac{C_I}{mw^2 - nw + s} \tag{8}$$

The solution from above has been obtained knowing that, from physical point of view, we always have

$$p = \frac{dh_s}{dw} \ge 0;$$
 and $(mw^2 - nw + s) = k_w \ge 0$

In order to solve the equation no.6 it will be used the constants variation method, that is the determination of a particular solution as this one:

$$p = \frac{C_I\{w\}}{mw^2 - nw + s}.$$

From equations no.6 and 8 we get:

$$\frac{C_{I}'(w)(mw^{2} - nw + s) - C_{I}(w)(2mw - n)}{(mw^{2} - nw + s)^{2}} + \frac{C_{I}(w)(2mw - n)}{(mw^{2} - nw + s)^{2}} + \frac{\alpha - (2mw - n)\beta}{\beta^{2}(mw^{2} - nw + s)} = 0$$
(9)

and, after we do the necessary reductions, we obtain by integration the value of the parameter $C_1(w)$:

$$C_{I}(w) = \frac{m}{\beta}w^{2} - \frac{(n\beta + \alpha)}{\beta^{2}}w + \kappa$$
(10)

So, the particular solution of equation no. 6 is:

$$p = \frac{\frac{m}{\beta}w^2 - \frac{n\beta + \alpha}{\beta^2}w + \kappa}{mw^2 - nw + s} \Leftrightarrow$$
$$\Rightarrow p = \frac{\beta^2\kappa + \beta mw^2 - (\alpha + \beta n)w}{\beta^2(mw^2 - nw + s)}$$

(11)

Knowing relation (5), we obtain:

$$dh_{s} = \left(\frac{\beta mw^{2} - (\alpha + \beta n)w + \beta^{2}\kappa}{\beta^{2}(mw^{2} - nw + s)}\right)dw$$
(13)

The function expression $h_s = f(w)$ is obtained by integration of the equation no. 13:

$$\beta^2 h_s = \beta w - \int \frac{\alpha w + \beta s - \beta^2 \kappa}{m w^2 - n w + s} dw$$
(14)

The integral from the previous relation is solved in a different way, taking into account whether the determinant of the quadratic equation, $\Delta = n^2 - 4ms$, is positive or negative.

 $\frac{dh_s}{dw} = \left(\frac{\beta^2 \kappa + \beta mw^2 - (\alpha + \beta n)w}{\beta^2 (mw^2 - nw + s)}\right) \Leftrightarrow$ $\Leftrightarrow dh_s = \left(\frac{\beta^2 \kappa + \beta mw^2 - (\alpha + \beta n)w}{\beta^2 (mw^2 - nw + s)}\right) dw$

Case 1: $\Delta > 0$

$$h_{s} = \frac{w}{\beta} - \frac{\alpha}{2m\beta^{2}} ln \left| m w^{2} - n w + s \right| + \frac{1}{m\beta^{2}} \left(\beta s - \beta^{2} \kappa + \frac{\alpha n}{2m} \right) \frac{2}{\sqrt{4\frac{s}{m} - \frac{n^{2}}{m^{2}}}} \operatorname{arctg}\left[\frac{2\left(w - \frac{n}{2m}\right)}{\sqrt{4\frac{s}{m} - \frac{n^{2}}{m^{2}}}} \right] + K$$

$$(15)$$

Case 2: $\Delta < 0$

$$h_{s} = \frac{w}{\beta} - \frac{\alpha}{2m\beta^{2}} \ln\left|mw^{2} - nw + s\right| + \frac{2m\beta s - 2m\beta^{2}\kappa^{2} + n\alpha}{2m^{2}\beta^{2}} \frac{1}{2\frac{\sqrt{n^{2} - 4ms}}{2m}} \ln\left|\frac{w - \frac{n}{2m} - \frac{\sqrt{n^{2} - 4ms}}{2m}}{w - \frac{n}{2m} + \frac{\sqrt{n^{2} - 4ms}}{2m}}\right| + K$$
(16)

3. Conclusions

From a mathematical point of view, the equation no.14 has two solutions: 16 respectively 18. But from the physical point of view, we must choose solution 18 that is adequate to the situation

in which the adequate determinant of the expression that shows the hydraulic conductivity, that is

(12)

 $\Delta = n^2 - 4ms$, is negative. In this case, the expression has complex roots, and so this parameter will not cancel and will keep a constant sign.

So, the general solution of the suction variation equation within the active moisture domain is:

$$h_{s} = \frac{1}{\beta} \left[w - \frac{\alpha}{2m\beta} \cdot \ln\left(mw^{2} - nw + s\right) - \frac{1}{m} \left(s - \beta\kappa + \frac{\alpha n}{2m\beta}\right) \frac{2}{\sqrt{\frac{4s}{m} - \frac{n^{2}}{m^{2}}}} \operatorname{arctg} \frac{2\left(w - \frac{n}{2m}\right)}{\sqrt{\frac{4s}{m} - \frac{n^{2}}{m^{2}}}} \right] + K$$

$$(17)$$

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Study of the Infiltration's Below the Defense Dike of the Big Island of Brăila with Defect's and Conditions of Rehabilitation

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Abstract: The hydrological regime of the Danube takes a strong influence about hydrological balance on precinct. By mean multiannual flow, the level of river is superior by medium level of the groundwater, permanently supplied by infiltration, secured by a specific lithological configuration of the river alluvial deposit. On time course, in most representatives hydrological times, is makes investigations of the dike behavior, and the opens or subsurface drainage in edges of the precinct. On some dike's sectors, on the great water times, it shows most spectaculous, but most dangerous phenomena: the gryphons. In these paper is being analyze underground hydraulic phenomena, specific for edges of precinct, as the infiltration and gryphons, his effects, and cautions needed for limiting and controlling it.

Keywords: hydrological regime, underground water, infiltration, drainage, gryphons

1. Introduction - Short presentation of the defense measurements (dike and drain nets) of the Big Island of Braila

For putting into value of the productive potential of the land in Big Island of Braila, there were designed and built , starting with 1964, complex diking works, drain and later on irrigation. So, it was built a circular dike with a length of 150.5 km, with a medium height of 4 m, crown width - 4 m, outer ramp - 1/3-1/4, inner ramp - 1/4-1/2, axial convexity - 0.25-0.30, being moved an earth moving volume of abt. 10 millions of cubic meters, 65 cubic meters per dike meter and 152 cubic meters per of square ha of land (Fig. 1).

The studies and researches concerning the dike and the neighborhood area attested altitudes of the foundation soil comprised between 2.5 downstream and 8.5 upstream, as well as the existence of a an uneven relief, favoring the interception of the raised underground water and the manifestation of the water excess. The dike was built completely in 1964 at an assurance of 10 %, with checking for assurance of 3 %, later on being performed works of over-heightening at an assurance of 1 %. The dike goes through Marasu with its villages Plopi, Magura, Zatna, Bandoiu,

Tacau and Frecatei with its villages Stoienesti,

Cistia Agaua and Titcov. The total surface defended by diking is 72173 ha, the rest of the island surface, remained abt. 4700 ha, representing dike area with forestry designation.

In the situation of Danube high floods, in the vicinity of the dike, even in the areas more defended by this, with lower lands, frequently occurred surfaces affected by water excess and flood on quite large surfaces – abt. 10 % from the total diked area, mainly in the dike area – abt.40 % of the surface.

For collecting an discharge of the waters from the premises that are coming from Danube infiltration, underground water, rain, especially in the ex lakes area, of dike neighborhood area, it was built in the period 1967-1972 a draining arrangement consisting of 7 independent draining systems witch surfaces is between 8000and 15000 ha (Fig. 2). The total length of the draining channels is 1363 km, medium 2 m depth, the total amount discharged Qd=48.39 m³/s (for a hydro-module of 0.92 l/s and ha) and a discharged water amount of 205 m³. Water discharge is made in exterior into the two Danube branches, Macin and Valciu, by 7 reversible pumping stations located at the dike, 6 inner re-pumping stations and 17 dranage stations (Table 1).

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2. Study/watching of the infiltration phenomena: occurrence area, directions, amount and effects.

From the analysis of the hydro-geological observations made at the 145 drills, it was found

out that the underground water levels are mainly influenced by Danube levels and less by the climax factors (rain and evapo-transpiration).

Table 1. Pump	oing stations cl	naracteristics for	or drain – disc	harge in Big	g Island of Brăi	la

Crt.	Name of the	Machines in	Machines in	Number of	Pump	Power of the	Debit per
Nr.	system and station	use - 1990	use - 1998	machines	type	motor - kwh-	pump - mc/h
0	1	2	3	4	5	6	7
1	SPR Filipoiu	3xP30M	3xP30M	1.2.3.4	P30M	200	5292
	1	1xM20M	1xM20M 4xD26M	, , , ,	M20M D26M	/5	2117
2	SPR Gemenele	$1 \times DV5/47$	$1 \times DV5/47$	1,2,3,4,5	DV5/47	40	9323
2		1x P30M	1x P30M	1.2.2	P30M	200	5292
3	SPR Malcanu dig	2xM20M	2xM20M	1,2,3	M20M	75	2117
4	SPR Salcia Dig	3xP30M	3xP30M	1,2,3,4	P30M	200	5292
		1XM20M 3xP36M	1XM20M 3xD36M		M20M P36M	/5	9330
5	SPR Titcov dig	1xDV6/70	$1 \times DV6/70$	1,2,3,4	DV6/70	100	4320
		4xP30M	4xP30M		P30M	200	52.92
6	SPR Bălaia dig	2xM20M	2xM20M	1,2,3,4,5,6	M20M	75	2117
7	SPd Aurelu	5xDV450	5xDV450	1,2,3,4,5	DB450	55	2025
0	SDd Majaanu Int	1xDV6/70	1xDV6/70	1 2 2	DV6/70	100	3300
0	SPu Maicanu Int.	2xDV450	2xDV450	1,2,3	DV450	75	2025
9	SPd Titcov Int.	4xDV6/70	4xDV6/70	1,2,3,4	DV6/70	100	3300
10	SPd Bălaia Int.	4xDV6/70	4xDV6/70	1.2.3.4.5.6	DV6/70	100	4020
		2xDV5/4/	2xDV5/47		DV5/47	40	1440
11	SPd Salcia int.	4xDV6//0	4xDV6//0	1,2,3,4	DV6/70	100	4020
12	SPd Zăton dig	5xDV6/70	5xDV6/70	1,2,3,4,5	DV6/70	100	4020
13	SPd Zăton int.	4xDV6/70	4xDV6/70	1,2,3,4	DV6/70	100	4020
14	SPd 1	4xDV5/47	4xDV5/47	1,2,3,4	DV5/47	37	1448
15	SPd 2	3xDV6/30	3xDV6/30	1,2,3	DV6/30	30	891
16	SPd 3	3xDV6/30	3xDV6/30	1,2,3	DV6/30	18,5	627
17	SPd 4	4xDV5/47	4xDV5/47	1,2,3,4,5	DV5/47	40	1440
18	SPd 5	5xDV5/47	5xDV5/47	1,2,3,4,5	DV5/47	40	1440
19	SPd 6	4xDV6/30	4xDV6/30	1,2,3,4	DV6/30	18,5	677
20	SPd 7	3xDV6/30	3xDV6/30	1,2,3	DV6/30	30	677
21	SPd 8	2xDV6/30	2xDV6/30	1,2	DV6/30	22	677
22	SPd 9	2xDV6/30	2xDV6/30	1,2,	DV6/30	18,5	
23	SPd 10	2xDV6/30	2xDV6/30	1,2	DV6/30	18,5	
24	SPd 11	4xDV6/30	4xDV6/30	1,2,3,4	DV6/30	18,5	
25	SPd 12	4xDV6/30	4xDV6/30	1,2,3,4	DV6/30	30	
26	SPd 13	5xDV6/30	5xDV6/30	1,2,3,4,5	DV6/30	22	
27	SPd 16	4xDV6/30	4xDV6/30	1,2,3,4	DV6/30	18,5	
28	SPd 17	4xDV5/47	4xDV5/47	1,2,3,4	DV5/47	40	1440
29	SPd 18	5xDV5/47	5xDV5/47	1,2,3,4,5	DV5/47	40	1440
30	SPd 19	3xMV253	3xMV253	1,2,3	MV253	11	



Fig. 1. Peripheral dike sector division and drains systems in Big Island of Braila

During the high floods, due to the high water column, in front of the dike ramp, strong water infiltration occurred, both through dike body, as well as the alluvium deposits that constitute dike's foundation.

The lithology of the bottom area, favoring the water circulation in a different way. So, starting from the upper stratums, with the dusty argils alternances (K=0.94-1.94 m/day), dusty argils sands (K=3-8.5m/day), to the deeper stratums, with fine and medium sands (K=3-8.5 m/day), the highest permeability is reached in the most profound stratums with stones (K=1.5-9 m/day).

What it has to be signaled is the net difference of the litholgical conformation of the neighborhood lands to the two Danube branches, Macin and



Fig. 2. Complex hydro-improving arrangement, Danube meadows drain-Big Island of Braila premises

Valciu, with much more high permeability of the alluvium deposits to the first, that also ensuring extremely high water infiltration fronts.

Danube hydro-geological condition while high flood in May 1997, at a monthly medium height of 6.25, reported to the land configuration in dike area, the Danube levels were over the medium surface in dike area with 1-1.25 m, and compared to the lowest surfaces – the lakes and stream in the Danube flood land connecting its branches bottom - with 2.75-4 m.

Further to the researches, the following came out:

- the maximum amplitude in the hydrogeological drills follows generally the variation of Danube levels, with a drainage in time around 5 days near the dike, with the tendency of increasing inside the premises; reaching at 15-30 days at distances of over 5 km;

- the maximum amplitude at the underground water level variation near the dike was generally over 2 m, exceeding in the South area of the premise with up to 3 m, compared to the area in the center of the island which was below 2 m, and locally even below 1 m;

- the hydraulic gradients of the underground water varies between 0.27 % in the dike sectors in upstream and 1.2 % in the dike sectors in downstream.

The measure in which the Danube levels variation influences the underground water one was decided by correlating of the two elements, resulting that in dike area the correlation coefficient is generally over 0.95, indicating their very closed connection.

In order to point out the orographic characteristics of the different sectors in dike area, it was delimited a peripheral strip 1 km width, divided in sectors of 10 km long each (Fig. 3).

The underground water depth less than 2 m were met in dike area at Macin branch (in the sectors II, VI, VII, VIII, IX) on surfaces with the most reduced height and characterized by the presence of some more permeable deep stratums.

It was found out that in the dike area, for 1 m variation of the Danube level corresponds a variation of the underground water levels of over 0.30 m (in the sectors I, II, V, VI and XI) and locally even over 0.40 m. For compensation, it is mentioned that towards the interior of the premises, at over 2 km away off the dike, for 1 m variation of the Danube level corresponds a variation of the underground water levels of 0.20-0.30 m.





In what it is concerned the influence of the rains on the underground water levels variation in the dike area, it comes out that this depends on the rain amount fallen down and the period when it occurred and if they are overlapping on the influence of Danube high waters, which remains, anyway, the main influence factor. From the researches made, it results the rain effect on the increasing of the underground water levels in the dike area is more relevant at amounts of over 70-80 mm, if they were recorded in the cold

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season of the year, under the conditions in which the soil profile is wet by previous rains, also correlated with underground water levels more raised, it is re-felt the influence of the rains fallen down on the accumulation in the underground water stratum.

Branch	Dike le	ngth (km)	Length(m)	Width	Surface	
Branch	from	up to	Length(III)	(m)	(ha)	
	2200	5200	3000	230	69	
	21800	24000	2400	180	43	
	35200	36200	1000	200	20	
Valain	40300	43300	3000	140	42	
vaiciu	40000	49000	3000	150	45	
	55500	56800	1000	240	31	
	62200	64000	1800	170	31	
	total	15500	15500	Х	281	
	3000	8000	5000	350	175	
	11000	12500	1500	120	18	
	44000	45500	1400	60	6	
	47000	48000	1000	120	12	
	49800	50800	1000	200	20	
Macin	64000	67000	3000	150	45	
	69400	70800	1400	80	11	
	74400	77800	3400	220	75	
	79000	81800	2800	250	70	
	84200	85200	1000	90	9	
	Total		21500	Х	441	
,	Total dike area		37000	Х	722	

Table 2. Surfaces with water excess in the dike area (May 1997)

The temporary water excess in the Danube infiltration, mainly in the peripheral areas to the dike at distances comprised between 50-100 m up to 200-300 m, but in certain cases when the high flood duration is longer, the high water Danube levels, the litholgical particulars of the ground affected by the excess, the infiltration occurs at

longer distances, at abt. 800-1000 m inside the island.

The surface affected by the water excess in the dike area summarizes 722 ha, mainly on Macin branch, where 441 ha are summarized, compared to 281 on Valciu branch, this confirming the land differential between the two branches from litholgical point of view.
The infiltration intensity in the river while high flood was determined by the observations made in the draining experiment perimeter Gemenele during the high flood in 1997. This perimeter has a surface of 220 ha, the absorbent drains are made of PVC with \emptyset 50-80 mm, with discharge in open collectors. As it results from table 2, the observations made in period 15 April-30 May 1997 attests specific leakage varying between 0.20-0.30 l/s ha and 0.60-0.90 l/s ha, values quite big taking into consideration that the high flood in 1977 is relative small compared to those in 1965 and 1970, taking also into account the specific flow of the arrangement that if 0.7 l/s ha.

The successive high floods, in a number of three (Fig. 4), in autumn-winter, spring and summer determined the keeping of the raised underground water levels, especially at the upstream third of the premises, on a period of 5-6 months.

Under those circumstances, the heavy and often rains in relatively short periods were not any longer stored, the internal drainage being reduced, that led to their accumulation in the superficial stratums of the soils, determining water excess on a surface, in that year of abt. 5565 ha. It is important to underline the fact that actually the operational parameters of the drain arrangements do not correspond any longer to the designed ones, the drain stations flow being dimininuished by the pumping aggregates worn down, and the geometrical and hydraulic parameters of the drain channels are not appropriate due to the degradation produced during exploitation.

3. Griffon's problem in the peripheral area of Big Island of Braila premises and mainly of Maicanu area

As it is known, the defense dikes safety largely depends on the quality of the foundation ground. Strictly referring to I.M.B. defense dike, this one is located on dusty-argils of abt. 10 cm, followed by 25-30 cm fine and rough sand, so during the high waters, due to high water column, infiltration occurs both through dike body, as well as and mainly below its body that modifies the foundation equilibrium condition. At the beginning water eyelets on the inside ramp at different distances, and later on craters on the neibourghood



Figure 4. The situation of griffons development during Danube high flood in Big Island of Braila

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areas to the dike and in the drain channels limiting it, which move fine particles.

In the Big Island of Braila, on the 154 km of defense dike, there were found along the years in

the periods with high waters, an amount of 66 griffons, in different evolution stages, with diameters comprised between 2-3 cm and 40-60 cm (Table 3, Fig. 4).

Item no.	Defense sector	Griffons pos. km-	Number of griffons	Distance off dike		
0.	1.	2.	3.	4.		
1.	Maicanu	33+100	8	50-80		
		44+700	10	50-100		
		46+550	2	50-100		
		46+700	12	50-100		
		47+600	2	50-100		
		49+550	1	50-100		
		49+600	2	50-100		
		49+700	2	50-100		
		47+300	12	50-100		
2.	Gemenele	67+250	5	50-150		
3.	Filipoiu	81+250	2	50-60		
4.	Marasu	5+000	3	50-70		
	Macin branh	8+000	3	50-70		
5.	Bandoiu	21+500	1	50-100		
		21+700	1	50-100		
	Total griffons		66			

Table 3- The situation of the griffons foundduring the high waters in 1997

Analyzing the island chart before diking, it is found out that most of the griffons are found in the area of the ex stream in the Danube flood land connecting its branches with drainage in Danube, or at the end of ex lakes near to their shore, where the litholgical configuration is specific to those lands.

These were signaled mainly in the channels that overtake the infiltration through dike, which were built together with the dike at distances of 50-100 m off the ramp foot, but also on some drain channels perpendicular to dike.

In the high waters period in the interval 1994-1997, observations were made regarding griffons' location and evolution on the infiltration overtaken channel Ci 285 - km 50-51 on Macin branch, built ar 92 m off dike ramp foot, where 23 griffons were found in different evolution stages (Fig. 5).

What is significant regarding the danger degree and the intensity of this phenomenon is precocciousness of their starting even fro defense stage no. 1, when the high flood wter level is at the ramp foot was only 12 cm.

From the study of this phenomenon there were found part of the evaluative stages signaled in Kelner specialized books, namely: waving, boiling area, clear spring, griffon spring, individual griffon, group of griffons, without being signaled the hydraulic breaking phenomenon. As location of the occurrence in the channel section, there have been found griffons in channel ramp center, base, as well as springs which, generally, produce its sliding (Fig. 6).



Fig. 5 - Griffons in ramp and in channel Ci 285



Fig. 6 - Ramp slided due to the springs from infiltration

Depending on their size, on the degree of moving the particles (turbidity), the preventive measures are applied:

- preventive methods regarding their location and the reinforcement of the access area of the exterior water by filling the holes neighbors to the dike on the length on which griffons are shown;

- fighting methods, consisting of collecting the water leakage in concrete tubes of \emptyset 1 m for smaller griffons, the water column compensating the ascendant currents of the griffon, or lesting with soil bags or lest pillow with \emptyset 1.5-2.5 m for the bigger griffons.

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Bilateral Adjustment Engineering, by Reversible Systems, of The Water Humidity In soil in Big Island of Braila North – Est Area

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Abstract: In actual stage in the land reclamation operation of the Big Island of Brăila, in dyked – drained – irrigated regime, was meets an situations of exceeding quantity of the earth water from a locally specific sources and conditions.

Is well – known that in meadow soils the farming was benefits by more groundwater's, from a small depth. On the time course, the research themes was be oriented on the harmonizing the draining and the (sub) irrigation functions, on the soils with underground waters on the small depth in north – east zone of the precinct, this is a possibility a water consumption rationalize by decreasing electrical power needed to the pumping station.

In these papers are presented the results of the researches in that direction, and our conclusions.

Keywords: land reclamation, reversible systems, the energy consumption optimization

1. Introduction.

The study of the bilateral adjustment of the hydro condition of the soils is for Romania a major task; and for the hydro-improving specialists who will understand this matter, the unique chance, that, based on what they did up to now, to realize in the future either the inter- penetration between the irrigation and drain nets, now separate, or by implantation on the actual nets – especially drains – of the complementary objective – the irrigation. In this case, the actual irrigation technique, as well as the drain one, should suffer modifications and adaptations of the various kinds.

2. The concept of reversibility and mix operation of different hydrotechnical arrangement. Examples from Romania and other countries.

2.1 Gostinu – Greaca – Arges system, where I.C.I.T.I.D. Baneasa-Giurgiu organized two experiment fields.

2.1.1 The experiment field F. 101 (self-draining well) which serves a 4 ha area, drained by P.V.C. drains laid at 1.2 m underground, with

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distances of 20 m between the rows. The absorbent drains discharge the waters in azbo-cement

collecting drains with ND 250-300 laid at é m underground. The self-dischargeable well has 25 m depth and a stabilized flow of 2 l/s. The well was used for collecting the underground water that in under pressure (further to Danube levels) and its discharge in the collecting drain by means of a manhole, from where it then goes in the absorbent drains net. From the absorbent drains, the water is infiltrated in the soil and it supplies the underground layer that raises up to the desired height.

2.1.2 Listeava experiment field – has a surface of 30 ha. The experiment scheme contains ceramic tubes drains. the drains are supplied from channel Ce 19, which, by closing the two dams located on it, the drain is stopped, keeping a 0.8-1 m height of underground water level, and by means of the capillary raising, crop sub-irrigation is assured.

2.2. Dăbuleni-Potelu-Corabia system

Dabuleni-Potelu-Corabia system located in Danube meadows has a local surface, arranged with drain wells that allow to use the drain water for the irrigation of crops. The drilled wells of the drain channels sides are equipped with pumping systems and they are branched to the drain channels and to the irrigation antennae.

2.3. Bechet-Dabuleni system

Bechet-Dăbuleni system with a surface of 5646 ha, located in Danube meadows uses the waters from drain for crop irrigation. The drain waters are collected

by the exhaust channels net and directed in the area with small heights Măgura-Gisteni. Here, the drain waters are overtaken by a pumping station and discharged into a tilled channels, and from here, during the dry periods, are used for irrigation, and in the raining periods are directed to Danube.

2.4. Miercurea Ciuc experiment field

Miercurea Ciuc experiment field with a surface of 3000 ha, located in Ciuc and Gheorghieni depression in the meadows of Olt and Mures rivers, is found on peat soils which were drained. These, during summer are over-drained, but by closing the dams, it is allowed to raise the water level in them, stopping the over-drainage, thus assuring a good water supply of the plants.

Researches of the operation of the drain systems in reversible and/or mix condition were also made at:

- Scimeni system on 480ha, by C. Haret and colleagues;

- Dichiseni-Dervent system ona surface of 600 ha, by I. Nicolaescu;

- Big Island of Braila complex hydro-improving arrangement, by I.Visinescu and I. Dan;

- Lacu Sarat experiment field in Braila plain, by V. Zamfirache and H. Ioanitoaia;

- Radauti drain system, by St. Marcu and I. Stanciu.

The method was also used by agricultors, arbitrarily, who due to the lack of money used the water from the drain net to irrigate the crops.

Studies and researches on the exposed theme were made also in other countries in which there is

a certain tradition in realizing and exploitation of the reversible and/or systems with unique net.

2.5. Studies and researches made in Italy

In 1970 the tubular drain penetrated in Emilia-Roman area and immediately, on this surface of some thousands of hectares, the specialists adopted the idea to adjust the humidity in both senses, using the same net – the drain net.

2.6. Studies and researches made in Lithuania

Along the time, in Lithuania were made, in order to study the operation and the efficiency of the mix and reversible systems with unique net, more experiment fields. Their surface varied between 3 and 18 ha, and the hydraulic conductivness between 0.02 and 5.7 m/day (so a very wide range of natural situations). The arrangements variants adopted were remarked by the distance between drains – 10, 15, 20, 25, 30 and 40 m, the location deepness – 0.8-1.2 m and the length of the drains with reversible function.

2.7. Studies and researches made in Russia

The experiment perimeter around Moscow, on a surface of 18 ha, has a general slant relatively small (1-2 ‰), it is smooth and comprises soils of alluvium types, argillaceous-aluminous (physique argil 35-45 %) with soft acid reaction (5.6-6), medium gleized, the hydraulical conductivness at surface (0-30 cm) comprised between 1 and 3 m/day, and profoundly (30-60 cm) around 1.3 m/day.

The ceramic drains, with 50 mm diameter, were located at distances of 10, 20 and 30 m and 0.9 and 1.1 m deepness, having a length of 120 m.

3. The hydrotechnical scheme of drain and irrigation system in Big Island of Braila with the experimental reversible perimeter in North–East area.

Brăila Lake natural unit, with a surface of abt. 91500 ha, was arranged starting with 1964, by ample works of amelioration consisting of defending works against high waters, drain and later on irrigation. The diking works consisted of building a circular dike 150.5 km long, with a medium height of 4 m, with a slope between $\frac{1}{2}$ and $\frac{1}{4}$, with an initial assurance of 10 %, with checking for assurance 3 %, later on being made works of overheightening at an assurance of 1%.

The total surface defended by the diking works is 72173 ha, the rest of the territory being comprised between dike and Danube wharf.

In order to collect and evacuate the excess of water from the premises, it was performed in 1967-1972 a drain arrangement consisting of 7 independent drain systems which surface varies between 8000 and 15000 ha. The total length of the drain channels, consisting of CC, CP, CS, Ct, Ci summarizes 1363 ha, medium depth 2 m, the total discharged flow Qd=48.39 m³/s – for a hydromodule of 0.62 l/s,

through if flowing yearly an water amount of over 205 millions cubic meters.

Water discharge is made at exterior in emissaries, Macin and Valciu branches – Danube, by 7 pumping stations to dike, out of which 6 stations are reversible (drain-irrigation), all of them having a summarized flow 49.5 m³/s with flows per stations comprised between 3 and 10 m³/s. In order to overtake the inner waters from the lower areas, 6 re-pumping stations were built, with an installed flow of 33.1 m³/s. The flow difference compared to 49.5 m³/s at the base stations, is accumulated in the collecting channels, gravitationally.

Having in view the fact that the studied perimeter is located in the dry steppe area, characterized by air medium temperatures of 11.0° C, rain average of 447 mm potential evapotranspiration of 705 mm and water climax deficit of 345 mm, it was reached the conclusion to also arrange this territory from the point of view of irrigation

In this climax context, irrigation arrangements by aspersion were built, with centralized stations for putting under pressure and buried pipe net on a surface of 67620 ha.

The irrigation works were correlated with the already made drain works, having in view the specific pedologic frame in the premises, with hydromorfe soils and with salinity potential, which has also the advantage that they can supply the plants of the crop from underground water found at small depth in some area.

In order to establish of a technology of how to use the drain for direct the underground water level in optimum positions for water supply the crops, a special experiment field was made, in the North-Est

area of the premises, named Gemenele – see fig. 1.

Its surface has 30 ha, it is provided with horizontal drainage, with drain lines at 40 m distances, located at a medium depth of 1 m, with P.V.C. tubes with \emptyset 80 mm, geo-textile filter and with drains discharge in collectors open by drainage.

4. Water quality and levels dynamics, with the utilization of the conditions and periods that ensure the reversibility/sub-irrigation.

For watching the hydrologic condition of the drain collecting channels there were mounted alignment stakes, for watching the hydrologic condition there were mounted tubular soundings, and for keeping and directioning the water in the field there were mounted dams with adjustable flowing tube or cofferdams. The harmony of the drain functions also below irrigation of the drain arrangements were assured by the operation of SPD4 pumping station that serves the sub-system as follows: in the exceeding periods the drainage are assured freely through the drain collectors, following that after to be made stocks at the beginning of the dry periods and to stop the operation of the drain station, the underground water flow from the irrigated neighborhood premises assuring the raising of the underground water level. The optimum of underground water levels for the corn crop was decided pending on the humidity of the soils and on the crop behavior, it not being different on the climax character of the year (dry, wet) and on the monthly difference of the plant consumption connected to the earthing stage – (Table 1, Table 2).

Table 1 Optimum depths of the underground water for the corn crop during the studied period (1995-1998)

Month	Optimum depth of underground $(water - cm)$							
	Dry year	Wet year						
June	60-80	80-100						
July	70-90	90-110						
August	80-100	100-120						

		Soil humidity during period:							
Plac e	Layer	5.VII		10.VIII		31.VIII		16.IX	
	(cm)	Under ground water % depth (m)		Under ground water depth (m)	%	Under ground water depth (m)	%	Under ground water depth (m)	%
	0-25	23.3			24.8		25.6		25.0
	25-50	23.9			23.9		23.7		23.6
S1	50-75	0.70 29.4		0.83	31.7	0.81	32.4	0.93	30.8
	75-100	30.5			31.5		31.5		31.0
	100-125	30.6			33.6		33.6		34.8

 Table 2. The dynamics of soil humidity in the harmony experiment of drain and irrigation on the soils with underground water at low depth North-East area-Gemenele-1996

The salt content in the underground water, watched during the 4 years experiments, has mostly values below 1.0 g/l, being proved this way that there are not any essential modifications of the mineral residuals, generally the underground water being within the sweet waters group – (Table 3). The soluble salts content in soil, watched during the experiment period is reduced, with values comprised between 63 and 156 mg/100 g soil. The dynamic observations attest that no essential

modifications occur in the soil solution. This salty frame is mainly determined by the specific hydro-salt condition of the premises, characterized by the lack of some salt sources and hydrological balance having only sweet water penetration (rains, water from Danube, irrigation water from Danube). Also, another favorable factor constitutes the high permeability of the soils, favoring a good circulation of waters, and in such a way washing the salts in profile.

		Soil humidity during period:							
Place	Layer	5.VII			10.VIII				
	(cm)	Under ground	g/l	mg/100 g	Under ground	g/l	mg/100 g soil		
	. ,	water depth (m)	23.3	SOIL	water depth (m)	U	5 5		
	0-25	0.81		109		0.594	94		
	25-50			94			78		
S1	50-75		0.77	125	0.08		109		
	75-100		2	78	0.98	0.394	63		
	100-125			78			64		
	125-150			-			63		

Table 3. The content in soluble salts from the underground water and soil during the experiment of using the water from the drainage net at the sub-irrigation North-East area-Gemenele-1996

It was attested by the researches performed that, under the irrigation applied by the reversible using of the existing drain arrangements, there is no danger of secondary salinity of the soil, at least for short and medium term.

On the other side, the existence of the irrigation arrangements on the grounds arranged with drain works, constitutes a possibility of reaching a hydro-salinity balance in the perspective of some unfavorable salt evolutions on long term, using periodically the irrigation as preventive solution.

5. Reversibility impact in the North – East area on the soil, plants and ecological conditions.

The reversible using of the a drain arrangement for irrigating the crops by sub-irrigation implies the fulfilling of some morphological conditions (low grounds, uneven planes <30-50 cm), of soil (big permeability, low salt content in soil and underground water) and of

arrangement, which, devices and systems to assure a safe control of water depth.



Fig. 1. Hydrotechnycal scheme of the drain system Gemenele

In the case of the alluvium soils in Big Island of Braila in the North-East area, having a raised permeability and fertility by harmonizing the drain and irrigation-sub-irrigation functions of the drain arrangements, the following effects are realized:

- keeping of some optimum underground water levels under the condition of supply from Danube (while high waters) and in such a way the drain function manifestation;

- keeping of the water from heavy rain in the collecting net and its use for sub-irrigation of the crops;

- the water supply of the drain collects by interventions on the arrangement (controlling the drain stations operation condition) for supply the underground water.

The value of optimum supply depth in the underground water for the corn crop under the soil conditions in the studied area raises during the vegetation period from 0.60-0.80 m in June to 1.0-1.20m in August.

Under the conditions of some not salty soils, with a good permeability and and underground water lightly mineralized <1-1.5 g/l soluble salts, after a period of reversible use of the drain arrangement, *for 4 years salt accumulations were not noticed.*

In order to prevent some bad degradation processes of the soils, periodically it is useful to apply irrigation with Danube water through the existing irrigation net, with drains drilling without dikes.

In order to establish of a technology of how to use the drain for direct the underground water level in

optimum positions for water supply the crops, a special experiment field was made, in the North-Est area of the premises, named Gemenele (Fig. 1)

In surface has 30ha, it is provided with horizontal drainage, with drain lines at 40 m distance, located at a medium depth of 1 m, with P.V.C. tubes with ϕ 80 mm, geo-textile filter and with drains discharge in collectors open by drainage.

The agricultural cultures in the premises have a significant water support from the underground water, representing abt. 30-80 % from the total water necessary amount, differing according to: the underground water depth, soil, physiological development stage. In the period 1986-1990, the researches made in the lizimeter field showed the quantum of the water benefits under the conditions of the location of the underground water at optimum depth (90-120 cm) as follows: 2730 m³/ha for corn, 2180 m³/ha for grain, 3000 m³/ha for Soya and 3690 m³/ha for sunflower (yearly average values).

On the surfaces on which the underground water, after lifting up, does not reach optimum depths necessary to the development of the crops, it is necessary to apply 1-2 local irrigation, with irrigation norms reduced to 400 m³/ha.

6. Propositions for re-building the technology of the hydrotechnical arrangement.

By the experiment made, there came out the reversible use elements of the drain arrangements, imposing geo-morphological, hydro-geological, pedological and arrangement

In order to re-technology the hydro-technical arrangement on all surfaces suitable for subirrigation, it is compulsory to perform some works, such as:

a) For short term:

• to perform remedy works, for bringing to normal operation parameters of the drain channels, of the drains and their belonging hydro-technical constructions, repair of the drain outlets at drainage;

• to perform, in steps of some completion works for the drain net in the critical areas concerning the water excess;

• to perform sole diking works for the drain collectors by tubular small bridges equipped with dams or water level adjusting devices, at distances from 300-500 m to 1000-1500 m.

b) On medium and long term:

• to equip the drain pumping stations with high capacity pumps;

• to redesign and complete the drain channels net on the whole arranged surface of the premises;

• to equip the arrangements with hydro-mechanical systems for optimizing the drainage condition through the collecting-discharge net.

The economical effect of the technology is shown by reducing with at least 50 % of the labor force for moving the irrigation installations, there are made irrigation by aspersion on higher surfaces, realizing in such a way economies of the irrigation water of abt. 60-80 % from the irrigation norm, including the electrical energy that would be consumed for its pumping.

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The Technological Transfer towards Users of the Research Results Obtained in the Field of Land Reclamation

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Abstract: In order to increase the efficiency of Romanian irrigation systems, there are presented some technical solutions tested by author and his co-workers, having in view the following objectives: (i) the reduction of water losses through infiltration and exfiltration from open canal networks; (2) the improvement of water distribution and gauging over canal and pipe networks; (3) the improvement of systems management by updating and taking advantage of the databases resulted from previous study and research activities carried out at local level.

Keywords: Water transport efficiency, water measurement systems, data processing programs

1. Foreword

Agricultural technical information transmitted to and by farmers and other categories of users by extension and consulting system can be classified into two classes:

-general proper agricultural information;

-agricultural information obviously correlated with new scientific achievements (innovating technologies).

Proper agricultural information, aimed to improve current production practices, farms management/or products marketing and processing, can be used without being obliged to acquire a certain technology.

Generally, innovating technologies are to be found as inputs in farms production and facilitate to obtain some superior performance in various fields. They have different places in the frame of public and private goods system. In case of modern technologies concerning the construction of agricultural equipment, irrigation equipment, agrochemistry, new vegetal and animal hybrids, veterinary medicine etc., agricultural information is included at the chapter innovation. These technologies are classified as private goods because of their high degree of exclusiveness. The nature of some of these technologies and the use of some legal protection mechanisms grant them a high degree of exclusiveness and consequently they bring profits to the producers and distributors.

Being private goods, the delivery companies are motivated to send them together with the necessary technical information to all interested clients in the best technical and social conditions.

Agricultural consulting play an intermediary part in the information flow between the specialized research institutes and farmers or various categories of users. Thus, the results of research are offered to the people interested in an easy form in accordance with the specific features of local conditions. At the same time, the interception of requirements perceived by users at local level is necessary, in order to introduce them into the research plan of themes for future. The main reference points of consulting activity consist in:

- identification, division into fragments and analysis of target groups;

- participant analysis of problems, existing difficulties and restrictions;

- consulting messages formulation;
- selection and use for consulting methods;
- consulting activities planning;

- pursuit and estimation for activities developed.

The specific consulting for land reclamation sector can have the most diversified aspects beginning with the assistance in the setting up of associations for water users and the training of their members up to the punctual carrying out of some mini-projects for irrigations, surface and surface drainage soil erosion control or the melioration of the lands affected by various degrading processes. The beneficiaries of consulting and technologic transfer activities can be either farmers, designing or other units carrying out exploitation for land reclamation works [1].

In the chapters below there are examples of several specific technologies for land reclamation sector which were finalized by the author and his collaborators as a result of the research carried out in ICITID Baneasa-Giurgiu and transferred to some different categories of users.

2. Method

Starting with the gradual putting into operation of the large land reclamation systems of 1970, the research program "Exploitation of irrigation and drainage arrangements" was initiated by the Research and Engineering Institute for Irrigation and Drainage, Baneasa-Giurgiu (ICITID), working under the co-ordination of the Academy of Agricultural and Forestry Sciences.

The main aim of the research program was to establish the means for a rational use of water, in the context of cutting down energy consumption and preventing negative effects in the evolution of the level and chemical composition of both phreatic water and soils from the irrigation and drainage systems in use.

Apart from the observation made regarding the evolution in time of the level and chemical composition of phreatic water within irrigation and drainage systems, this research program has in view other aspects regarding the causes and effects of these changes as well as the measures which should be taken in order to prevent the negative effects with implications in the evolution of soils and the performances of systems.

The program undertook a complex study starting from the establishment of the water needs for the plants and continuing with all aspects concerning the most efficient use of water during its exploitation. However, the research activities are interdependent, having in view the close relationship which exists between the quantities of water needed by plants, the means by which they are transported through the irrigation systems, the consequences of water losses for the hydrogeological balance and, furthermore, the way in which the changes in the level and chemical composition of phreatic water are reflected in the management of the irrigation systems and the evolution of the soils. The research activities also have in view aspects regarding the evolution of meteorological factors, such as temperature and rainfall mainly, the quantities of water penetrated into the systems.

The effectuated experiments concerning the functionality of the new types of hydro-insulating materials, measuring devices and adjusting equipment consisted in laboratory tests, checks under polygon and experimental stand conditions and final checks under current exploitation conditions.

The laboratory analyses included checking physico-chemical characteristics, the behavior in the presence of aggressive chemical media, as well as under conditions of accelerated aging of the various types of hydro-insulating foils and elastoplastic putty.

The experimental stand tests were performed in order to verify the measuring precision of some flowmeters under conditions of functioning with conventionally pure water or with various concentrations of unsettled alluvial matter.

Checks were also performed on small scale models of some new types of equipment for water adjusting and distribution through open canal networks. These checks included the determination of charge losses, water flow variations in function of the hydraulic charge, as well as the ascertainment of other relations between exploitation elements of these pieces of equipment.

The main elements monitored through research activities at experimental polygon level were related to the efficiency of various adhesives and of the "warm" welding technology for hydro-insulating foils, as well as of various primers in the case of joint proofing elastoplastic putties for concreted canals.

To the determinations of water losses of experimental canals, there were added in several stages various other observations concerning the behavior of the new hydroinsulating materials under current exploitation conditions [2].

3. Results and discussions

3.1 Technical solutions for reduction of water losses through open canal networks

Although based on a modern conception, the Romanian irrigation arrangements, covering a surface

of about 3 million ha, suffer from some functional

shortcomings which can be overcome by applying constructively complex technical solutions of refitting and retechnologization and, at the same time, adapting them to the new structure of land ownership.

The biggest problems are posed by the open canal networks only partially lined, or with the line at different degradation level. Internal research activities in this field have shown that the transport efficiency varies presently between 40 and 70% on canals which are not lined (as a function of soil texture and phreatic water depth) and between 70 and 90% on lined canals (as a function of lining technology and lining degradation degree).

In order to reduce water losses through infiltration/exfiltration and to increase the transport efficiency over irrigation canal networks, there were created and tested new types of lining materials from the category of hydroinsulating foils and tightening mastics.

This research also had in view the possibilities to take advantage of some secondary products of the plastics and synthetic rubber industries.

Taking into account the physical and chemical characteristics of the foils used for lining irrigation canals, two new types of foil were created:

- HRS type, made of secondary polymeric resources, with a width of 1.0 mm;

- IF type, made of regenerated rubber, with a width of 1.2 mm.

From the same category of materials but using different compositions, there were created two new types of elastoputty for tightening the degraded joints of concrete linings:

- MP1 type, shaped girdle with a diameter of 10 mm, made of secondary polymeric resources;

- R8 type, shaped girdle with a diameter of 10 mm, made of regenerated rubber.

The new materials were tested under laboratory, experimental polygon and actual exploitation conditions.

In the case of irrigation canal lining with the new hydroinsulating foils ballasted with concrete slabs, there was obtained a reduction of water losses through infiltration of about 90% by comparison with the nonlined canals variant. By using the new elasto-plastic mastics as tightening materials for the degraded joints of concrete linings, there was obtained a reduction of water losses through exfiltration of about 75%. Based on the good results obtained from the testing, both categories of products were homologated and patented [3, 4, 5].

As a result of the best findings obtained from experiments, both categories of products were homologated and licensed (OSIM-license No. 87772/1985) being transferred to the designing sector represented by ISPIF Bucuresti, as new technologies by projects of technologic engineering, in order to be included to the projects of execution for the new arrangements or by the rehabilitation of the existing ones.

3.2 Technical solutions for improvement of water distribution and countering over canal and pipe networks

Another category of shortcomings is generated by the improper functioning or partial lack of automatic equipment targeted for network level and flow control. In certain cases, this leads to a deficient distribution of water to beneficiaries.

In what regards the water level adjusting equipment, very good results were obtained with a new type of hydraulic level regulator (RHL) conceived by the SNIF Tulcea (OSIM-license No. 79362/1982), both as prototype and series product. By comparison with the Neyrpic regulating valve, the new regulator exhibits numerous functional advantages, due to the 3 programmable input magnitudes: downstream constant level, maximum admitted upstream level and minimum admitted upstream level respectively. This allows adductors equipped with RHL to function during low water consumption periods with adequately lowered hydrodynamic levels, realizing thus a better correlation between water demand and supply.

Auto-adjustable flexible dams of various sizes were also designed, built and tested (OSIM-license No. 91040/1986). They were employed with good results both for water detainment in irrigation canals and for preventing functional water losses, as well as for drainage canals in order to use the water for irrigation during low humidity periods.

A third category of dams tested and approved for series multiplication due to the good results that were obtained, are floating body dams. These type of dam, designed to maintain a constant upstream level, has the advantage that it can be moved by floating to any section of the canal, then fixed to the crowning with metallic cables, effectively realizing thus a new division of the respective canal sector.

In choosing criteria of the experimental device for water measuring, selection there were considered both the requirements of information automatic control and teletransmission and the necessity of gauging water amounts delivered to users. Presently, the later exhibits a more diversified structure as a result of applying the new laws concerning land property. For example, the private sector which holds about 90% of the irrigated surface, comprises both individual farmers and farmers associations that own variable size land surfaces. In consequence, the land size surface of an agricultural exploitation was one of considered criteria.

Other criteria considered in this selection were those specific to water measurement in the irrigation practice, the most important being:

- the wide range of outflows to be measured;

- the high water turbidity;

- the lack of an electric power source (in some cases) for the supplying equipment;

- the special measures taken for device protection due to isolated working conditions.

Having in view the experiments made under operation conditions, from the many methods and devices of measuring water outflows, those which can be applied in the irrigation systems were recommended, taking into account the specific factors mentioned above.

In the case of open canals the obtained data reflect the network structure, differing in what regards the size and the conveyed water flow and their function within the network.

Calibrated hydrotechnical structures have been recommended and are usually used for water supplying canals (Q=20–80m³/s) and main distribution canals (Q=5–15 m³/s), where the outflows can be indirectly calculated knowing the geometric canal elements and measuring the water level variation or water velocity. The reading, recording and teletransmission of data are made with the hydrometric gauge, limnigraphs or level transducers respectively. The measurement

accuracy of these pieces of equipment varied between 5 and 10%.

A better but more expensive solution is represented by direct measuring devices like electromagnetic or ultrasound outflow gauges produced by some foreign companies and tested in some particular cases.

For sector distribution canals (Q= $0.3-1.0 \text{ m}^3/\text{s}$) which provide users with direct water supply for land sizes between 40 and 100 ha, there are usually used distributor modules outflow gauges, Parshall canals or weir boxes. In some cases, the usage of limnigraphs or level transducers as additional equipment improves the water countering and management.

In what regards the pressure pipes network, the range of measuring devices is more diversified and of higher accuracy.

More difficult problems are posed by pressure pipes of main pumping or repumping stations due to the pipe diameters (800-2000 mm) and high outflows (5-50 m³/s). Most common are the lateral throttling or Venturi devices associated with differential manometers or pressure transducers.

In the case of pressure pipes from the plot pumping stations (D=400-600 mm, and Q=0.8-3 m^3/s) with servicing areas varying between 500 and 3000 ha, the experimented and recommended gauges are among the most up-to-date with future perspectives of including them in an automated control circuit for the above mentioned stations. These are discharge transducers provided with turbine and inductive magnetic sensor, electromagnetic and ultrasound detectors, associated with their own calculation and integration electronic block.

The measurement accuracy of these devices ranges between 2.5 and 4%, the larger errors being recorded for higher turbidity water.

The last two categories of pipes, tertiary pipes and hydrants are important from the point of view of measuring devices, as these have to service with priority the private sector, the users that own average and small land areas.

Thus tertiary pipes with diameters varying between 200 and 300 mm and outflows ranging from 0.08 to 0.15 m3/s have service areas varying between 40 and 100 hectares. The hydrants are of 100 or 150 mm with outflow magnitudes of 0.015 and 0.03 m^3 /s,

ensuring the irrigation of areas varying between 0.5 and some tens of hectares.

Water meters with Woltman turbines have been successfully tested and are recommended for these two types of pipes. An important aspect which must be taken into consideration in the operation of these meters is the necessity for periodical checking and cleaning of the incorporated filters in order to avoid the blocking or defective functioning of the counting device [6, 7, 8, 9].

The experimental results obtained regarding the equipment for water levels and flows adjusting and control were homologated and transferred to the specialized designing unit by projects of technologic engineering. At the same time, this applicable research represents the subject of some economic contracts concluded with the districtual ex-enterprises for the execution and exploitation of land reclamation works (IEELIF) subordinated to the ex-General Direction of Land Reclamation and Agricultural Constructions (DGEIFCA).

3.3 Using of database programs related to open canal and pipe networks management

In order to monitor the water losses, to establish the transport efficiency over adductiondistribution networks and to keep track of the functionality of water adjustment and metering equipment, there was created a package of compatible computer programs using databases.

The first of these programs (INFORAND) contains the hydrotechnical networks characteristics at the level of each administrative system, as well as the unitary water losses established by previous research, as a function of the nature of lining (in the case of waterproofed canals), respectively soil texture and phreatic water depth (in the case of non-waterproofed canals). The program allows the computation of transport efficiencies over each network sector as a function of conveyed water volumes.

The function of the second program (INFOREG) is to store and keep track of data concerning hydraulic equipment for water adjustment and distribution in the open canals network. The equipment was classified in function of the acquisition order nature (upstream order, downstream order, mixed order), the program including in its graphical section general views, representative sections and functional diagrams for each type of equipment.

The third program (INFODEB) implements the database concerning the irrigation systems exploitation flows. Using graphic and tabular files as the previous program, this program allows the creation and actualization of the database containing water measurement means available for open canal networks and pipes under pressure, as well as the utilization of the stored information for various necessities, starting from the type of the water transport hydraulic system available at the respective arrangement and the category of beneficiaries serviced by the various terminals of the network.

The description of the operational way and the functions of those 3 mentioned informatics programs (where later 4 other programs were added) was transferred to the technical leading board of the ex-Autonomous Administration for Land Reclamation (the present National Society of Land Reclamation) in order to be informed and aware regarding the necessity of introducing these programs in its territorial branches.

The respective programs were popularized by a couple of articles published in the review "Hydrotechnics" being at the same time the subject of some scientific communications in more internal and international conferences [10].

4. Conclusions

The utilization of the solutions proposed in order to reduce water losses through infiltration/exfiltration and distribution from open canal networks leads to significant economies of water and pumping energy, with direct implications for the cost of water delivered to beneficiaries. The proposed measures are also beneficial for environment protection, avoiding the negative effects generated by water losses from canals on the phreatic water evolution and the soil production capacity.

The utilization of the aforementioned database programs in the exploitation of irrigation arrangements increases the operators' control and intervention capacity, thus improving substantially the arrangements management.

The modernization of irrigation systems, especially the reduction of water losses of canals and

the reorganization of exploitation by gradually putting the interior arrangements under the administration of farmers associations, as well as the water gauging at parcel level, are necessary conditions for developing an efficient irrigated agriculture.

The direct transfer of technologies resulting from finished applicable research can be done efficiently and advantageously by contracts with the economic agents interested in their promoting.

From the most common consulting methods that can be successfully used, demonstrative polygons with different irrigation equipment correlated with the type of crop and with watered surface, namely surface and subsurface drainage or soil erosion control.

Training and practical demonstrations with groups of interested users can be organized. The activities of individual technical assistance or in groups considering the specific problems mentioned by the participants, organized either at the headquarters of the research units or at the place of users are an equally importance.

The most rapid and efficient method to promote the research news and protect them at the same time (products, equipment, technologies) remains their spread by booklets or articles for popularization, and if it is possible, together with promotional presentation by radio and television.

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Considerations Concerning the Characteristics of Permeability of the Podzolic soil in Voinesti Catchment

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Abstract: The nature of the soil and the substratum is important for the determination of the proportions between the drainage of surface and the underground drainage. The substratum of the catchment of Voinesti is constituted by brown podzolic soil. We are interested here in the description of the soil concerning it's hydrological functioning knowing that the passage or not of the water in various compartments of the soil determine hydrological answer of the catchment.

Keywords: hydraulic conductivity, suction, drainage

1. Introduction

The nature of the soil and the substratum is important for the determination of the proportions between the drainage of surface and the underground drainage. The substratum of the catchment of Voinesti (situated in west extremity of the Under - Carpathian of Curvature) is constituted by brown podzolic soil.

We are interested here in the description of the soil concerning it's hydrological functioning knowing that the passage or not of the water in various compartments of the soil determine hydrological answer of the catchment.

It was executed two soil profiles, one in experimental station and another one at 400 m distance from the station.

Because the demarcation on the soil in horizons is difficult, we took samples for the following depths: 0-30 cm, 30-60 cm and 60-100. We sample three samples for every depth.

The purpose of this site investigation is to obtain data as regards the physical and hydraulic characteristics of the soil (the hydraulic conductivity in saturated soil, the grain size analysis, specific gravity, etc.). The determination was made in the Physical Soil Laboratory of the Ovidius University of Constantza and the results can be seen in the following paragraphs.

2. Laboratory tests

The quantity of the soil used to make the grain size analysis is between 100 to 200 g.

The grain size analysis was made by the hydrometer analysis method. The results of this test for one sample took from station profile by horizon 0-30 cm depth is presented in Fig.1.

Using the ternary diagram we determined the type of soil. For this example we obtained the following fraction: 28 % clay, silt 21 % and 51 % sand. The type of this soil is sandy clayey loess (Stefan P., 86). More interesting is to study the variation of the grain-size distribution on the soil profile. The Fig. 2 and the Fig. 3 present the grain-size distribution variation for both pedological profiles. In the grain-size distribution the clay and the sand have the greatest percentage. For the pedological profile opened in the zone of the experimental station (Fig.2) we observe an important variation of fractions percentage.

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The upper layer of the profile, up to the 60-cm depth, the contents of clay is 28 - 29 %, but in the bottom of the profile we observe an accumulation of clay. The content of clay has here the value of 57 %. The content of silt varies between 9 % and 34 % and the content of sand is from 9 % to 62 %.

Moreover the soil which is up to the 30 cm can be situated (according to the textural classification used in Romania) in the category of the soil (LAS) sandy clayey loess. The soil found between 30 and 60 cm is a loam clayey loess (LAL) and the soil from 60 to 100 cm enters the in the category clay soil (A).





The textural characteristic factor is superior to two what means one abrupt passage between both last horizons, specific for the passage enters the horizon E (deluvial) and Bt (alluvial or textural horizon).

The horizon Bt presents a less permeability what provokes a temporary stagnation of the water in the profile and determines the phenomenon of pseudo-gleyied.

Over against, the profile realized at the South-Est of the experimental station presents homogeneity of the composition grain-size distribution (Fig. 3). The contents of clay vary between 23 % and 28 % and that of silt is about 21%. Dominating in this profile, is the sand that has value comprising between 51 and 57 %. The soil of all profile depth is in category of the sandy clayey loess (LAS) and it is very difficult to identify the major pedological horizons.

Apparently this situation is due to the landscape phenomena which were it produces in last ten years. The clay is from 2-3m of depth, but we did not make boring until this depth.

The specificity of the solid phase of a soil ensues from its composition mineralogical and expresses them by the specific gravity ρ_s .

The values of the real gravity vary between 2,65 and 2,70 g/cm3 (for the profile 1) and from 2,59 to 2,61 g/cm3 for the second profile (Figure 4).

The dry density varies between 1,39 and 1,53 g/cm3 for the profile 1 and for the second profile this greatness is about 1,57 g/cm3. In that case, the porosity

of the soil vary between 42 % and 48 % by following the type of the soil for the first profile and ii is constant of 39 % for the second profile.

The natural density varies between 1,70 and 1,80 g/cm3 for the profile 1 and for the second profile this greatness varies from 1,74 to 1,85 g/cm3.





Determination of the hydraulic conductivity of saturation was made for every type of horizon of soil for undisturbed cylindrical samples, with 2 cm high and 5,6 cm in diameter. For this test we used a permeameter with constant head without suction. The sketch of this permeameter is presented in Fig.5. The sample is introduced into the permeameter by assuring the waterproofs. The air is removed of the soil using a flux of the water from bottom to the top. The presence of the water in the piezometer means the end of evacuation of the air of the soil. Once the air of the soil evacuated we measure the volume of the water evacuated by the surplus of the reservoir, the time in seconds and the difference between both piezometers with a precision of a millimeter. We repeat the measurements to us let us obtain constant relative values for the value of the infiltrated volume (the difference between two successive values should be under 10 %). The specific conductivity with k_{sat} saturation is calculated with the following equation:

$$k_{sat} = \frac{v}{i}, \quad v = \frac{V}{A \cdot T}, \quad i = \frac{h}{l}$$

where: V represent the volume of the water (cm³) evacuated meanwhile of time T (s) k_{sat} is the hydraulic conductivity of saturation (cm/s), A is the cross section of sample (cm²), h is the difference between both piezometer tubes (cm), 1 (cm) represents the height of sample



Fig. 5

1, 6 – tubes; 2, 3, 7 – overflow; 4 – oulet reservoir; 5, 8, 15,16 - valves; 9 – inlet reservoir; 10, 11 – piezometer tubes; 12 –permeameter; 13 – porous plate; 14 – soil sample

The value of the specific conductivity calculated is corrected with a coefficient if the reference temperature does not take the value of 20° C.

The following curves of infiltration in the time shows that the infiltrated volumes grow in a linear way in time (Fig. 6). For the profile 1 we observe three curves different attached to three found horizons; the value of the lowering (going down) infiltrated volumes as the contents of clay increase in the profile. On the other hand, for the second profile, the values of the infiltrated volume are in the distance 10-20 cm³ by the hour and the

curves of infiltration are close one of the other one. The values of the specific conductivity in the saturation are carried in the Figure 7.

By following the relation specific conductivity of saturation - depth we observe two stages for the profile 1 (in the station): a fast diminution of the value of the specific conductivity of saturation what means a fast drainage followed of one flattening of the curve corresponding to a lent drainage.

For the second profile the value of the specific conductivity of saturation varies among 5,29e-06 and 2,76e-06 cm/s by following the depth. The values correspond to a soil with lent drainage.







Fig. 7

We tried to find an analytical expression to express the relation between specific conductivity with saturation and depth for both profiles of the soil. Two types of equations are who can express this relation $(y=k_{sab}, x=depth)$:

$$y = \frac{1}{a+b\cdot x + c\cdot x^2}$$

$$y = a \cdot x^b$$

The first equation is a model in three parameters, which offers a good correlation (coefficient of correlation equal to 1). The standard deviation is close in zero. The second equation is a model in two parameters. The coefficient of correlation varies 0,999 (for the profile 1) in 0,989 (for the profile 2) and the 530 Considerations concerning the characteristics ... / Ovidius University Annals of Constructions 3, 4, 525-530 (2002)

standard deviation is also close in zero (0,0000001). We retained for our example the second model by taking into account that the models in two parameters are stable.

The parameters are obtained by means of the square test Chi. Two iteration are necessary to obtain the parameters which takes the values following ones (Figure 13 the series in red respective blue circle)

- a=0,0006301 et b=-1,2532865 for the profile 1
- a=1,41057^e-05 et b=-0,359201 for the profile 2

3. Conclusions

Dynamic of the drainage in unsaturated zone will be different in two categories:

• the working of height part catchment is influenced by the Bt (alluvial) horizon which principal characteristic is water accumulation. These characteristic combinations with the stratum of clay confer at this soil type a interesting comportment: a rapid drainage follow by a lent drainage and the alimentation possibilities with water for height stratum.

• the working of lower part catchment is influenced by the landscape phenomena. Concerning of that, the horizons can't be different. The characteristics of soil are homogeneous on profile.

We suppose that the overland flow is more important like others component in the catchment dynamic.

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Contributions to the Formation of the Data Base for Projects Regarding the Agricultural Sustainable Development

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Abstract: The programs regarding the rural development should be elaborated by using a data base which should contain information about the volume of principally agricultural production obtained in optimal technological conditions and about the volume of the total useful productions of the corps. In the work are presented cartograms regarding the maximum volume of the agricultural production from lands in south Romania, where works of irrigations and drainage were applied.

Keywords: durable development, cartograms, principally production, total useful production

The development as general social progress is perceived by the modern world as a durable process through which an optimal level of today's people's life should be assured, without affecting the possibility for the next generation to have their needs satisfied at an adequate level.

1. General principles of the durable development

The slow-moving character of the durable development makes that the effects of this process appear after a relatively long time from their implementation. That is why efforts should be made for a quickly start of them. For a durable rural development some general principles should be applied, as follows: - integrated approach of the activities, as per principle of concentration and differentiation, in the favor of the region with the biggest needs

- the diversification of the economical and social activities by sustaining the development of the private sector

- application of subsidiary principle based on which the decision comes closely to the its object in order to obtain the maximum effects

- application of the simplification principle for assuring a better coherence of activities

- financing the programs from local resources and rural credits

The general objective these principles should be applied to are presented in Fig. 1



Fig.. 1: General objectives of the rural development

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2. The durable development of agriculture

In the commune language the "rural" and "agrarian" terms are confounded, being considered synonyms. The rural term has a large signification and includes also agrarian elements. Even if the most important element of the rural term are the agrarian aspects and the largest space of it is occupied by agrarian events, the two terms should not be confounded.

A durable agriculture supposes the protection of the environment and of the natural resources simultaneously with the maintenance of the production potential, without destroying the other species; the possibility to perform a profitable agriculture for the farmers; the assurance of the people with sufficient food both in quantity and quality.

principle The technical solutions to materialize these concepts depend on agricultural exploitation technologies, which should be both ecological and profitable. Although these two conditions seem to exclude each other, they could be harmonized as the modern concepts of utilization of improvement techniques describe. The land improvements have a decisive role. By their adequate application is maintained the balance of water and salts in soil, is protected the surface water against physical and chemical degradation and are created favorable conditions for maintaining the natural potential of production.

The practical application of those aspects depends on the geographical area for which the program for sustainable development is designed.

The settlement step by step of the problems take into consideration the local or regional development plans, because the economical processes are estimated as a production chain, from raw materials to the end products.

A project for sustainable rural development is drawn up only by knowing the conditions regarding the natural environment, the recommendation for the crops structure and also the condition of totally valorification of the principally and secondary results of the production, obtained for each type of crop.

The natural conditions of some of the classical agricultural regions in Romania, influenced by the global climatic modifications, impose as necessary the regulation of water regime in soil, through combined works of irrigation and drainage, making the sustainable agricultural development dependent of the adequate utilization of land improvements works.

3. Forecast of the agricultural production on the fields where land improvements works were performed

The biological effectiveness of the agricultural crops obtained on the fields where irrigation and drainage works were performed as part of a durable agriculture program, impose the expression of the total (principally and secondary) useful production in an unitary manner (e.g. as "organic substance").

In the Romania's south part, the total useful production (average values), in optimal technological conditions is presented in the table no. 1:



Fig. 2. Forecast of the corn production on irrigated field



Fig. 3. Forecast of corn production on drainaged field



Fig. 4. Forecast of the sugar beat on irrigated field



Fig. 5. Forecast of the sugar beet on drainaged field



Fig. 6. Forecast of the soy bean production on irrigated field



Fig. 7 Forecast of the soy bean production on drainaged field

	1		1						
		Production	Land						
No.	Crop	category	without	with	with				
	- · F		improvements	irrigation	drainage				
			improvements	Inigation	urannage				
		PP	4.3	7.0	4.9				
1	Corn	тр	12.2	21.2	15.0				
		IP	15.2	21.5	13.0				
_	Winter	РР	2.6	3.5	3.7				
2	wheat				11.0				
	wheat	ТР	7.5	10.4					
		РР	1.7	2.9	1.9				
3	Soy bean								
-		ТР	4.0	6.8	4.5				
		РР	5.7	11.9	7.3				
4	Sugar beet								
	~	ТР	7.2	14.8	9.1				
5	Sun flower	DD	1.8	28	2.0				
5	Sui nowei	11	1.0	2.0	2.0				
6	Potatoes	РР	1.5	7.4	-				
7	Alfalfa, hay	РР	2.8	42.6	-				

 Table 1. Total average useful production (t/ha)

 on the irrigated fields from south Romania

PP - principally production result

TP - total production result

The most important economical value resulted from agricultural exploitation of the land is obtained from principally product, depending on which the volume of the secondary production is also determined.

Based on these facts we elaborated the cartogram for the level of the principally production obtained in technological conditions specific for the agricultural lands from south Romania (Table 1. position no. 2-7)

4. Conclusions and recommendations

The most modern conception about the evolution of humane society express itself by the term of "durable development" based on some general principles as: integration, diversification, subsidiary achievement mode, and simplification.

In the rural regions, where agricultural activities are prevalent features, working out of the programs for rural development imposes the arrangement of a data base that makes possible the estimation of the principally agricultural production and also the evaluation of the total useful production, in order to be valorificated in accordance with the integration principle.

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Aspects Concerning the Mathematical Approach of the Hidrogeological Dynamics in the Danube Meadows

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Abstract: The exploiting surface or underground drainage works, can change its technical characteristics after a long period of time, as a result of the subsidence process, clogging, settlement of the hydrotechnical works. For the new topohydrogeological conditions, the functional parameters of the elements of the hydrotecnical pattern should be modified, in order to achieve the modernization and rehabilitation actions.

Keywords: mathematical approach, phreatic water, boundary elements.

1. Introduction

This paper presents a mathematical calculation pattern for which the phreatic water depth is determined, underlining the surfaces with unfavorable evolution which should be first ones to be modernized, as well as the differentiation of the drainage parameter, taking into account the soil type and the depth of the phreatic water.

Solving the problems regarding the water movement inside saturated porous areas can be achieved using the hydrogeological pattern. Mathematical or experimental methods can be used in order to achieve this:

- the finite elements method;
- the finite differences method;
- the boundary elements method.

2. Theoretical consideration

The finite element method is an approximate mathematical method, solving the area problems. It is imperative to know how accurate is the approximation and how it can be improved in order to match the correct solution perfectly. Error evaluation is a very difficult problem, because the right solution to these complex problems is unknown. The approximation relation of the charge on element's domain is: $H(x, y, z) = \sum_{i=1}^{n} N_i H_i$ ⁽¹⁾

where N_i is the approximate function and H_i is the value of the element's junctions.

The finite differences method is based on lowering the movement domain in its junction element, for the function h in the proximity of a junction element, for a porous homogeneous isotropic area, gives the following relation:

$$h_o = \frac{1}{4} \left(\sum_{i=1}^{4} h_i + \frac{\delta_{\varepsilon}^2}{k} \right)$$
(2)

where h_o , h_i , i=1, 2, 3, 4, are the junction of the net 1, 2, 3, 4.

This method has some disadvantages: the necessity of lowering the whole domain of the movement, the necessity of development of some patterns in detail, when there are some singular points (drains, wells) inside the studied domain.

The boundary elements method consists of determining the piezometric parameter in any point of the area h(z). For this, the boundary of the domain is lowered in "n" linear elements: fragments j, j+1 (Fig.1). Also the outline ε rainfalls, irrigation, evaporation – is lowered in "m" linear elements.

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It is admitted that on section AB, BC, AD, of the frontier areas where we have the values of the piezometric parameter there are n_1 elements. For these elements we have n_1 equations as follows:

$$\sum_{j=l}^{n} a_{ij} \frac{\Psi_j}{kTS_o} + c = \frac{h_{oj}}{S_o} - \frac{\varepsilon^*}{kTS_o} \sum_{j=l}^{n} c_{jk}$$
(3)

where: $j=1,2,3,4...n_1$, h_{oj} - the values of the piezometric parameter on the element (j, j+1) wich approximates the real value h given on the outline section, (m), T is the layer transmisivity (m²/day), S_o - is the reference level difference, chosen as equal as the drainage parameter, C is the undetermined constant.

On the section CD of the outline is admitted that there are $n_2=n-n_1$ elements, where the normal speed values are given. On these, there can be written n_2 ecuations as following:

$$\sum_{j=l}^{n} b_{ij} \frac{\Psi_j}{kTS_o} - \frac{l}{2} \frac{\Psi_j}{kTS_o} = v_{nj} - \frac{\varepsilon^*}{kTS_o} \sum_{i=l}^{n} d_{jk} \quad (4)$$

where: v_{nj} is the normal speed on the elements *j*, *j*+*1*, wich can be determined with the relation:

$$v_{nj} = \frac{q_j}{kTS_0} \tag{4}$$

in which: q_1 is the water flow on the length boundary, on the element (j, j+1), kT is the hydraulic transmisivity, S_0 is the level difference reference (m).

To all these equation there can be added the final equation of all the artificial sources Ψ_j and the real ε , as following:

$$\sum_{j=l}^{n} \frac{\Psi_{j}}{kTS_{o}} l_{j,j+l} + \frac{\varepsilon^{*}}{kTS_{o}} A = o$$
(5)

where A is the sum of the interior underdomain $\widetilde{D} \subset D$ on which the water flow ε^* is distributed, $l_{j,j+1}$ is the length of the boundary element (j, j+1) of the underdomain \widetilde{D} . Solving the system of equations (3), (4), (5) the result is :

$$\frac{\Psi_{ij}}{kTS_o} \tag{6}$$

To solve the system of equation is made of the above equations, it is necessary to know the patterns of

the parameters a_{ij}, b_{ij}, c_{ij} and the conditions of the limit on the boundary domain h_{oj} and v_{nj}

3. The prognosis for the piezometric parametres of the phreatic water in Dnube Meadows.

To eleaborate the calculation programme, the drained and dyked are Gostinu – Greaca – Arges was lowered in 26 linear elements and the interior in 71 cellular elements with a length of 2 km.

The drained and dyked area Dabuleni – Potelu – Corabia was lowered in 21 linear elements and the interior in 97 cellular elements with a length of 2 km.

The system of equations made of relations (3), (4), (5) can be solved using a calculation program.

4. Results and interpretation

From the dates of the calculation programs, the piezometric parameters, the area parameters of the net junctions is obtained the depth of the phreatic water.

Knowing the coordinates in a general system X, Y, Z of the characteristic points from the junction net was obtained the bidimensional pattern (2D) using the *Surfer Utilitary Programme* with applications in the hydrogeological domain.

To exemplify, we present the obtained results for the process simulation with a water flow of 11/s/km and 21/s/km at terrace supplying.

For the area Gostinu - Greaca - Arges

- the supply with a water flow of 11/s/km from the terrace, the piezometric raised parameters are in the Southern part of the are, and in the Northen part there is a 3m difference (Fig.2)



5 .The limit conditions on the area boundary, characteristic for the simulation process area.

The area Gostinu - Greaca - Arges is delimited by the Danube on AB section, and the mountain on CD section, and the specific water flow is q. The chosen infiltration water flow was $q_1=11/s/km$, 21/s/km, 51/s/km, 101/s/km (the infiltration from the terrace or the Danube on 1 km area).

- the specific water depth of Danube level;
- the hydraulic conductivity K=1m/day;
- the hydraulic transmisivity $T=5m^2/day$;
- the level difference reference $S_0=0,6m$.

- the supply with a water flow fromm2l/s/km from terrace, there is a lowering of the 12 m piezometric parameter area and the Southern part the surface occupied is 14 m (Fig.3)



For the *Dabuleni – Potelu – Corabia*, were used the same values of the water flow of 11/s/km and 21/s/km for the terrace supply.

- -the supply with a water flow of 11/s/km we

notice that the piezometric parameters have maximal values in the extern part. In the central part there is a uniformity of parameter.



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Mathematical Model for the Investigation of Moisture Distribution through Soil Profile

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Abstract: This paper presents a mathematical model, which features the distribution of moisture through the soil profile. Mention is being made of the following: numerical solving of motion general equation for porous unsaturated media and a computer program with numerical.

Keywords: mathematical modeling, moisture, numerical solving, porous unsaturated media, soil.

1. Introduction

The behaviour of various substances in the soil (nutrients or polluants) is closely linked to the distribution of moisture through the soil profile. Undertaking impact studies presuppous a good knowledge of the soil moisture θ , al different moment *t*, for a porous unsaturated soil.

The mathematical model consists of the following: continuity equation, motion equations, boundary conditions (initial and of contour).

2. Mathematical model

The continuity equation for a prous incompressible medium [Musy A.] is:

$$divq = -\frac{\partial\theta}{\partial t} \tag{1}$$

where q is flow density,

 θ - volumetric moisture.

$$- \operatorname{div} q = \frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z}$$
(2)

where q_x , q_y si q_z are components of vector q related to the coordinates Ox, Oy and Oz.

The motion equation is given by Darcy's equation, generalized [Musy A.]:

$$q = -k(\theta) \cdot gradH(\theta) \tag{3}$$

where $k(\theta)$ is hydraulic conductivity tensor, $H(\theta)$ total potential of me

$$H(\theta) = -$$
 total potential of movement.
 $H(\theta) = z + h(\theta)$ (4)

where z = elevation and $h(\theta) = S(\theta)$ = suction (piezometric pressure head).

When the soil is orthotropic and isotropic, the tensor $k(\theta)$ components become:

$$k_{xy}(\theta) = k_{yx}(\theta) = k_{xz}(\theta) = k_{zx}(\theta) = k_{yz}(\theta) = k_{yz}(\theta) = k_{xx}(\theta) = k_{yy}(\theta) = 0;$$

$$k_{xx}(\theta) = k_{yy}(\theta) = k_{zz}(\theta) = k(\theta).$$
(5)

The general equation of motion in porous unsaturated media is obtained eliminating the flow q density between equations 1 and 2 and taking into account such relations as:

$$\frac{\partial h}{\partial x} = \frac{dh}{d\theta} \cdot \frac{\partial \theta}{\partial x} = -\frac{D(\theta)}{k(\theta)} \cdot \frac{\partial \theta}{\partial x}$$
(6)

where $D(\theta)$ is the apparent diffusivity of the soil.

$$D(\theta) = -k(\theta) \cdot \frac{dh}{d\theta}$$
(7)
This yields:

$$\frac{\partial \theta}{\partial t} = -div [D(\theta) \cdot grad\theta] + \frac{dk(\theta)}{d\theta} \cdot \frac{\partial \theta}{\partial z}$$
(8)

The law $h(\theta)$, we considered the following function, in section:

$$\lg h = \begin{cases} \lambda_1 \cdot \lg \theta + \mu_1, pentru \theta \in [\theta_r, \theta_{if}] \\ \lambda_2 \cdot \lg \theta + \mu_2, pentru \theta \in [\theta_{if}, \theta_s] \end{cases}$$
(9)

where θ_r , θ_s are residual moisture in the withering point, saturated moisture, respectively;

 θ_{if} – moisture corresponding point for $\lg h = \varphi(\theta)$ curve.

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The hydraulic conductivity k, function of moisture θ , based on Brook and Corey equation [Musy A.] as approximated in steps:

$$k(\theta) = \begin{cases} k_s \cdot \left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)^{3 + \frac{2}{\lambda_1}}, pentru \, \theta \in \left[\theta_r, \theta_{if}\right] \\ k_s \cdot \left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)^{3 + \frac{2}{\lambda_2}}, pentru \, \theta \in \left[\theta_{if}, \theta_s\right] \end{cases}$$
(10)

where k_s is hydraulik conductivity in saturated state.

The diffusivitty, function of moisture, is expressed by the following relation:

$$D(\theta) = \begin{cases} -k_{s} \cdot \lambda_{1} \cdot 10^{\mu_{1}} \cdot \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{3 + \frac{2}{\lambda_{1}}}, \\ pentru \theta \in [\theta_{r}, \theta_{if}] \\ -k_{s} \cdot \lambda_{2} \cdot 10^{\mu_{2}} \cdot \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{3 + \frac{2}{\lambda_{2}}}, \\ pentru \theta \in [\theta_{if}, \theta_{s}] \end{cases}$$
(11)

The general equation of motion in porous, unsaturated media becomes:

$$\frac{\partial \theta}{\partial t} = -\frac{\partial}{\partial z} \left[D(\theta) \cdot \frac{\partial \theta}{\partial z} \right] + \frac{dk(\theta)}{d\theta} \cdot \frac{\partial \theta}{\partial z}$$
(12)

or deriving $D(\theta) \cdot \frac{\partial \theta}{\partial z}$ and taking into account that $\partial D(\theta) = dD(\theta) = \partial \theta$

$$\frac{\partial D(\theta)}{\partial z} = \frac{dD(\theta)}{d\theta} \cdot \frac{\partial \theta}{\partial z}, \text{ finally yields:}$$

$$\frac{\partial \theta}{\partial t} - \frac{dk(\theta)}{d\theta} \cdot \frac{\partial \theta}{\partial z} + D(\theta) \cdot \frac{\partial^2 \theta}{\partial z^2} +$$

$$+ \frac{dD(\theta)}{d\theta} \cdot \left(\frac{\partial \theta}{\partial z}\right)^2 = 0$$
(13)

In relation (13), the derivatives $\frac{dk(\theta)}{d\theta}$ and

 $\frac{dD(\theta)}{d\theta}$ represents the results of deriving the relations

(10) and (11). The equation with partial derivatives (13) is a 2^{nd} order, parabolic, non-linear one, with coefficients depending on the independent variables *t* and *z* and the dependent variable θ .

The particular solution of equation (13) is given by the conditions:

a) initial, $\theta(0, z) = \Phi(z)$, where $\Phi(z)$ is a known function;

b) boundary, of the Fourier, nonhomogeneous type: - at the bottom of the soil under investigation (z = 0)

$$\alpha_0 \cdot \frac{\partial \theta(t,0)}{\partial z} + \beta_0 \cdot \theta(t,0) = \gamma_0 \tag{14}$$

- at the soil surface level (z = L)

$$\alpha_L \frac{\partial \theta(t,0)}{\partial z} + \beta_L \cdot \theta(t,0) = \gamma_L$$
(15)

where the coefficients $\alpha_0, \alpha_L, \beta_0, \beta_L, \gamma_0, \gamma_L$ are known functions.

3. Numerical solving of the general equation of motion in porous unsaturated media

The equation of the second order, parabolic, nonlinear, with partial derivatives (13) has the following form:

$$a_1 \cdot \frac{\partial u}{\partial t} + a_2 \cdot \frac{\partial u}{\partial x} + a_3 \cdot \frac{\partial^2 u}{\partial x^2} + a_4 \cdot \left(\frac{\partial u}{\partial x}\right)^2 = b \qquad (16)$$

Comparing equations (13) and (16) for the following correspondence between variables $t \Leftrightarrow t$, $z \Leftrightarrow x$, $\theta \Leftrightarrow u$, provides the concrete expressions of the parabolic equation coefficients:

$$a_{1} = 1; a_{2} = -\frac{dk(\theta)}{d\theta}; a_{3} = D(\theta);$$

$$a_{4} = \frac{dD(\theta)}{d\theta}; b = 0$$
(17)

The concrete expressions for the equation (13) coefficients derivatives are the following:

$$\frac{\partial a_{1}}{\partial \theta} = 0$$
(18)
$$\frac{\partial a_{2}}{\partial \theta} = \begin{cases}
\frac{k_{s}}{(\theta_{s} - \theta_{r})^{2}} \cdot \sigma(\lambda_{1}) \cdot \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{1 + \frac{2}{\lambda_{1}}}, \\
pentru \theta \in [\theta_{r}, \theta_{if}] \\
\frac{k_{s}}{(\theta_{s} - \theta_{r})^{2}} \cdot \sigma(\lambda_{2}) \cdot \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{1 + \frac{2}{\lambda_{2}}}, \\
pentru \theta \in [\theta_{if}, \theta_{s}]
\end{cases}$$
(19)
$$\frac{\partial a_{3}}{\partial \theta} = a_{4}$$
(20)

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$$\frac{\partial a_{4}}{\partial \theta} = \begin{cases} \frac{k_{s}}{(\theta_{s} - \theta_{r})^{2}} \cdot \lambda_{1} \cdot (\lambda_{1} - 1) \cdot 10^{\mu_{1}} \cdot \theta^{\lambda_{1} - 3} \cdot \\ \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{\frac{\lambda_{1} + 2}{\lambda_{1}}} \begin{bmatrix} \varphi(\lambda_{1}) \cdot \psi(\lambda_{1}) \cdot \theta^{2} - \\ 2 \cdot \psi(\lambda_{1}) \cdot \theta_{r} \cdot \theta + \\ (\lambda_{1} - 1) \cdot \theta_{r}^{2} \end{bmatrix}, \\ pentru \theta \in [\theta_{r}, \theta_{if}] \\ \frac{k_{s}}{(\theta_{s} - \theta_{r})^{2}} \cdot \lambda_{2} \cdot (\lambda_{2} - 1) \cdot 10^{\mu_{2}} \cdot \theta^{\lambda_{2} - 3} \cdot \\ \left(\frac{\theta - \theta_{r}}{\theta_{s} - \theta_{r}}\right)^{\frac{\lambda_{2} + 2}{\lambda_{2}}} \begin{bmatrix} \varphi(\lambda_{2}) \cdot \psi(\lambda_{2}) \cdot \theta^{2} - \\ 2\psi(\lambda_{2}) \cdot \theta_{r} \cdot \theta + \\ (\lambda_{2} - 1) \cdot \theta_{r}^{2} \end{bmatrix}, \\ pentru \theta \in [\theta_{if}, \theta_{s}] \end{cases}$$
(21)
$$\frac{\partial b}{\partial \theta} = 0$$
(21)

where
$$\psi(\lambda) = \frac{\lambda^2 + \lambda + 2}{\lambda}$$
 (22)

We intend to analyze the variation of moisture $\theta = \theta(t,z)$ on the depth L_a of the active soil layer. The origin of z axis was taken at the level withering point $\theta = CO$, situated at depth L. At surface level of soil z = L, water applications of intensity I and duration T_u are provided and real evapotranspiration is of intensity E.

At the initial moment $t_o = 0$, one a soil depth L_a , the minimum moisture level ($\theta = P_{min}$) was reached, with linear decrease of moisture up to the withering point.

In the above mentioned situation, the functions intervening in boundary conditions can be explained as:

$$\Phi(z) = \begin{cases} CO + \frac{P_{\min} - CO}{L - L_a} \cdot z, z \in [0, L - L_a] \\ P_{\min}, z \in [L - L_a, L] \end{cases}$$
(23)
$$\alpha = 0 \quad \beta = 1 \quad \gamma = CO$$

$$\alpha_{o} = 0, \beta_{o} = 1, \gamma_{o} = CO$$

$$\alpha_{L} = 1, \beta_{L} = 0, \gamma_{L} = \frac{E(t) - I(t) + k[\theta(t, L)]}{D[\theta(t, L)]}$$
(24)

$$I(t) = \begin{cases} I_0, pentru * t \in [t_0, t_0 + T_u] \\ 0, pentru * t \ge T_u \end{cases}$$
(25)

The functions derivatives, intervening in the boundary conditions, have the following values and/or expressions:

$$\frac{\partial \alpha_0}{\partial \theta} = \frac{\partial \beta_0}{\partial \theta} = \frac{\partial \gamma_0}{\partial \theta} = 0$$
(26)

$$\frac{\partial \alpha_L}{\partial \theta} = \frac{\partial \beta_L}{\partial \theta} = 0 \tag{27}$$

$$\frac{\partial \gamma_L}{\partial \theta} = \frac{1}{D[\theta(t,L)]} \cdot \frac{dk[\theta(t,L)]}{d\theta} - \frac{E(t) - I(t) + k[\theta(t,L)]}{D^2[\theta(t,L)]} \cdot \frac{dD[\theta(t,L)]}{d\theta}$$
(28)

4. Computer program UMID Z1.BAS

The numerical solving of the second order, nonlinear, parabolic, with partial derivatives (16) was achieved with the program UMID Z1.BAS, written in GW-BASIC, of the following structure:

I. Input data in 5 subgroups, as follows:

I.1 General data.

I.2 Constants, specific to the problem under study.

I.3 Constants of functions expressing equation (13) - solving conditions.

I.4 Functions expressing initial conditions.

I.5 Functions expressing coefficients of boundary conditions.

E. Output data.

The program is modulated on two levels. The 1st order module (main program) consists in the following four types of sequences:

I^o Input date introduction with sequences I^o.1 Reading I1 data. General data.

I^o.2 Reading input data I.2 - I.5 with a adequate 2^{nd} order subprogram.

 $I^{o}.3$ Input data display – print – save sequence I.2 - I.5 addressing two adequate 2^{nd} order subprograms.

A.2 The display/print C1 data sequence addresses an adequate 2^{nd} order subprogram.

I°.3 Define constants and auxiliary functions.

I^o.4 Define initial condition functions.

I°.5 Define boundary condition functions.

The calculus sequences follow:

C.1 Check compatibility of boundary conditions.

C.2 Calculus at moment t = 0.

C.3 Calculus at moment $t = j^*A_k$.

The 2nd order programs are used: parabolic equation coefficients evaluating; two matrices multiplying; solving linear equation systems heaving tridiagonal coefficient matrix; 3rd order subprogram.

The 2nd order program are used:

1.2 For input data displaying and/or printing and/or saving.

 $2_{.2}$ For calculating (with display and/or optional printing sequence).

The only 3rd order module is the subprogram used for the evaluation of characteristic function.

5. Numerical example of using UMID Z1.BAS program

Table 1 presents the suction curve for the investigated soil. The statistical processing of these data gave the coefficients shown in Table 2.

printing Due tu the low value saturated hydraulik conductivity, the variation of moisture an a layer of active soil, 2 cm depth, is presents. An artificial rain with an intensity 3,5 mm/hour is aplied at the soil surface. Moisture values for 0 < t < 4,5 hour and 1,75 on. < z < 2,0 cm are shown in Table 3.

	rable 1. Suction – moisture relation in the investigated son										
θ (%)	2,3	5	10	15	20	25	30	35	40	45	49,5
S(pF)	7,0	5,5	4,4	3,7	3,3	3,0	2,7	2,4	1,9	1,2	0,0
	Table 2 Hydraulic conductivity k_s and regression curve coefficients (9)										
ł	$K_{\rm s} ({\rm mm/h}) \qquad \theta_{\rm if} (\%)$		<i>P</i> _{if} (%)	λ_{1}		$\mu_1 \qquad \lambda_2$		λ_2	μ_2		
	3,50		30,0	-3,504654	4 -1	,136867	-1	1,64527	-:	-5,075329	
	Table 3 Moisture values										
2	Z (cm)	T (ore)									
	0		0,2	0,2 0,3			1,2		4,5		
	1,75	37,	5298 37,5298		3	37,5298		37,5298		37,5296	
	1,80	38,	38,2830 38,28)	38,2831		38,2834		38,2812	
	1,85	39,	39,0363)	39,0354		39,0263		38,8820	
	1,90	39,	,7896 39,834		7	39,8812		34,1045		34,1067	
	1,95	40,	5428	39,5709)	39,3666		38,6565		38,2061	
	2,00	41,2961		49,500		49,500		49,500		49,500	

6. Conclusions

Knowing the moisture distribution through the soil profile is fundamental in all research works which aim at studynig both the behaviour of various substances in soil (nutrients or polluants, either chemical or organic) and some physical, chemical or biologic processes in soil.

Mathematical modelling enables creating a prognosis of soil qualities at the right time and with reduced financial means.

As the soil – water – substances system is extremely complex, its accurate modelling is almost impossible to achieve. The various models, propoused by the literature, tray to satisfy bouth the representativity and the efficiency criteria of modelling. Essentially, all these are grounded on the principle of preserving water and substances in the soil, the characteristics of each model being defined by the types of water and substances motion. The more accurately they present the soil processes, the greater the namber of modelling parameters.

This mathematical model and the computer program we created satisfy the requrements of quick and efficient surveys of the influence of moisture on the soil quality, while needing more in-depth approaches and finished.

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Impermeable and Non-Corrosive Coating System for Concrete Elements by Treated its with Composite Polymer Putties

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Abstract: The composite polymer putties are modern materials obtained by mixing up one sintetical thermosetting resins: polyester or epoxide, with a filling material which gives to it high physico-chemical and mechanical qualities. When the mass of filling material is cement, the resulting putty is perfectly compatible with the concrete and/or mortar from the structures on applied it.

If the surface which must be treated is subjected to dynamic actions, vibrations or shocks, the glass fiber reinforced putty with one or two tissue layers, gives to the composite materials the needed mechanical resistance and elasticity.

The technological system proposed is a simple and quick one, getting in work after 48 hours from it application.

Keywords: thermosetting polymers, glass fibers tissue, composite putty, waterproof, corrosion-resisting.

1. Introduction

Classical concretes and mortars are attacked by variety corrosion agents from environment or from the buildings conditions of exploitation.

The Nicolina – Iaşi balneary complex contains: the clinical Hospital of recovery, the balneary cure Polyclinic, the Beach and a Hotel, having as natural tratment factors the mud and sulphur waters from this side of the town. Because of specific odour of free hydrogen sulphide liberation, the complex is "shrouded" by the stink specific sulphide compounds, and the natural factors used for medical attendances manifest an intense corrosive action upon the constructions.

The complex was equipped with two collecting tanks for process water and mud, resulted from treatment rooms but, after a long time of working these have required overhauling. For this reason, the complex was equipped with a pre-treatment of residual sulphur waters, wich takes in the ecologization and neutralization system of existing corrosive agents, conception of a mixed group experts from the "Gh. Asachi" Technical University of Iaşi, [1], so that the resulted waters to may be flowed into the town canalization and the air not be anymore polluted.

One of the work executed in this pre-treatment station consists in the finishing of the walls of the collecting residual waters tanks by the application of polymeric composite putty, [2], which replaces the antiacid gritstone plating suggested for initial design.

2. The polymer putty preparation

The polymer composite putty belongs the category of modern construction materials. characterized by a variety of properties wich gives them, for real, the name of "polyfunctional material": impermeabillity at liquids and gases, anticorrosive protection at a large assortment of polluting agents and chemical substance, superior mechanical and elastic characteristics, comparable with those of the special concrets of high resistance, agreeable appearance due to contain in the exterior layer of a color oxide etc. The obtaining and application technologies of these materials are simple, and the coming into work is very quick, 24 hours from the hardening of material.

The polymer putty is obtained by adding in the mass of a thermosetting resin, polyester or epoxide type, of a filling material, inert from chemical point of view with the resin, such as: cement, kaolin, silica, fine sand (0-1 sort) etc., in different proportions having into account the necessary workability for the application.

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The preparation of the polymer putty with cement admixture has the advantage of a perfect compatibility with concrete or mortar surfaces on wich are applied.

The material hardening reaction is few exotherm and the binder material gets in by absorbtion and mpregnates the old concrete surface

on a depth of 0,5 to 1 mm, realising on this zone, a composite of "polyme concrete" tipe. This is the

boundary material between the classical concrete or mortar and the modern polymer composite.

For establish the prescription of the polimer putty, [3], wich followed to be used for tratement the walls of that two tanks, were preparated eight kind of polymer putty, of different compositions, shown in the Table 1. Were used gray cement for construction and two kinds of thermosetting resins: polyester, NESTRAPOL 220 and epoxide, ROPOXID R 500.

	Componeți:							
Tipe of mixture	Thermoset	tting resin	Accelerator		Harde	ening	Cement	
	cm ³	gramme	cm ³	gramme	cm ³	gramme	cm ³	gramme
NESTRAPOL 220: a)	100	100	1,5	1,5	3,0	3,0	100	130
b)	100	100	1,5	1,5	3,0	3,0	150	195
c)	100	100	1,5	1,5	3,0	3,0	200	260
d)	100	100	1,5	1,5	3,0	3,0	300	360
ROPOXID R 500: e)	100	100	-	-	10	11,2	100	130
f)	100	100	-	-	10	11,2	200	260
g)	100	100	-	-	12,5	14	150	195
h)	100	100	-	-	12,5	14	300	360

Table 1. Prescriptions for polymer putties

The samples were realized and kept in laboratory conditions: interieur temperature 20 $^{\circ}$ C, interieure air relative humidity 55 %.

The workability of polymer putty depends on material quantity of filling material introduced in the resin mass, of the environment's conditions of preparation and the surface's state on wich is applied. So, in the prescribe's conditions of laboratory:

- the mixture **a**) is like a viscous liquid, black coloured, which can be applied on old and dry concrete surfaces with the spatula;

- the mixture **e**) is a less viscous liquid than **a**), black coloured, which can be applied by aid of a spatula or with a natural, hard hair brush;

- the mixtures **b**), **c**), **d**), **f**), **g**) and **h**) have greater quantities of cement and they look like a black paste which can only be applied with spatula.

The variation of quatities of hardening agents - accelerator, hardening - and the filling material

influence relative few, the *workability time* of the putty. For all prepared charges of putty, was ascertained that after 30 ... 60 minutes from mixing of components begins to easily increase the

of components, begins to easily increase the temperature, marking the beginning of *pre-gelatination* phase. The exothermic peak of hardening reaction takes place after 90 ... 120 minutes, the maximum achieved temperature being of 28 ... 30 °C, function of the hardening quantity. Once releasing, the hardening reaction quickly takes place, in aproximate two hours, in case of polyester resin and 20 ... 35 minutes, in case of epoxide resin.

The cement introduced in the polymer resin mass gives *tixotropic properties* –do not drop out when is applied on inclined or vertical surfaces-, in thin layers, on maximum 2 mm.

The polymer putty so obtained can be successful applied at: covering the concrete/mortar surfaces, as adhesive at joints for prefabricated elements, for the repairs of deteriorated or segregate concrete elements etc.

3. Conditions of realising the specimens from polymer putty

The specimens are test pieces having standard shape and dimensions, destined to give, by testing at different loads, information's about material's qualities, [3].

Depending on the resulted material's character, plastic or viscous, the specimens have different shapes and dimensions. So,

- the minimum dimension of specimen's side must be 40 mm;

- for prisms, their length, usually, are $(3 \dots 5) \times base's side$.

A sample of polymer putty is made of minimum three specimens.

The moulds in which the polymer putties are poured are metallic, dismountable, rigid, impervious, made of inert material in reaction with the polymer resin. The maximum permissible deviations of the moulds are $\pm 0,1$ mm for lengths; $\pm 0,15^{\circ}$ for angles.

At the specimens manufacturing was used a metallic battery with 12 pieces, fig. 1: 6 cubes with 40 mm and 6 prisms of $40 \times 40 \times 120$ mm. As a preliminary, the mould was cleaned, wiped with diluter and then covered with a substance for removing the pieces from the mould. After filling up the mould is finished, the material's surface must be smoothed.



Fig. 1. The battery used for pouring the specimens from polymer putty.

The polymer putty doesn't need compaction, being sufficient the troweling of the material in the mould.

Striking of the specimens was made at different periouds of time, after observing the hardening of the material and the specimens were kept in laboratory conditions in which they were poured.

4. Method of testing the specimens from polymer putty and phisico-mechanical characteristics of the material

On the hardened polymer putty – cubes and prisms – were made the following daterminations:

- the aparent specific weight;
- compression resistance on cubes;
- bending tensor resistance.

The tests were determined in "Gh. Asachi" Technical University of Iaşi, "POLYTECH" laboratories, equiped with necessary, as part of a design-research contract [4].

4.1. The determination of the aparent specific weight For determining this characteristic were used cubes, proceeding as follows:

- was determinated the volume, **v**, for each cubespecimen, measuring those the three dimensions, every dimension being the arithmetic mean of two paralel sides laying in a diagonal plane;

- was weighted every specimens thus obtaining the weight **G**, in grams;

- the aparent specific weight was calculated with the formula:

$$\gamma_a = \frac{G}{v} \quad (g/cm^3) \tag{1}$$

As a result was considered the arithmetic mean of the obtained values, written in the Table 2, for the two types of studied polymer putties.

Table 2.	The appar	rent spe	cific w	eight o	f the
	pol	ymer pu	ıtty		

Nb. of	The specific weight, γ_a , in g/cm ³ , for putty:		
specimen	poliester	epoxide	
1	1,79	1,62	
2	1,80	1,60	
3	1,78	1,61	
4	1,78	1,60	
5	1,82	1,59	
6	1,80	1,65	
Mean	1,792	1.612	

4.2. The determination of breaking stress in case of axial compression

The breaking stress in axial compression is an important criterion for appreciating the polymer putty's quality. The testing consists in introducing the cubic specimens between plates of a testing machine and applying with a progressing load, on one direction, with the loading velocity of maximum 3 daN/cm²/s, till breaking.

While testing were respected the following recommendations:

- the press's plates must be perfectly cleaned;

- the specimen must be correctly orientated towards the direction of applying the compression load;

- the specimen must be centered with the press's plate's axis, using its diagonals; the centering error must not pass over 1/100;

- using the specimens with perfect symmetry – parallel ending faces and perpendicular to the generatrice-;

- uniformly loading to secure the specimen's breaking in a time over 30 sec.

Breaking stress at axially compression, σ_{rc} , is described by:

$$\sigma_{rc} = \frac{P_r}{A} \quad (\text{daN/cm}^2) \tag{2}$$

where: P_r is the breaking stress applied on the specimen, in daN; A-the aria of the specimen's cross-section, in cm², perpendicular to the action's direction of the compression force.

The Table 3 shows the breaking stresses for that two types of polymer putties.

Table 3. Breaking stress in axial compression on cubes of polymer putty

Nb. of specimen	Breaking stress in axial compression σ_{rc} , in daN/cm ² , for the:			
-	poliester	epoxide		
1	1361,1	1349,3		
2	1362,0	1280,7		
3	1372,0	1265,6		
4	1362,3	1156,1		
5	1428,8	1348,9		
6	1387,0	1273,4		
Mean	1379	1279		

4.3. The determination of breaking stress in bending tension

Bending testing of the polymer putty is one of the indirect methods used for determining the breaking stress in tension of the material. This test means subjecting to bending a prismatic specimen, that was kept till testing moment in laboratory conditions.

The testing was made as in the Fig. 2. The subjecting device in bending is made of two semirounded supports that can oscillate around their centers, in a specimens's cross-section plane, for secure the uniform loading, without specimen's torsion and of a loading support.



Fig. 2. The loading diagram in bending of the polymer putty specimens.

Breaking stress in bending can be calculated with one of the relations:

- corresponding to RILEM:

$$\sigma_{ti} = \frac{4.5P}{l^2} \quad (\text{daN/cm}^2) \tag{3}$$

that presumes a triangular distribution on the whole height of the section, both in tensile zone and compressed zone;

- corresponding to Romanian standard:

$$\sigma_{ti} = \frac{2,63P}{l^2} \text{ (daN/cm^2)}$$
(4)

where is considered triangular distribution on the compressed zone and a rectangular one (elastic-plastic) in the tensile zone.

Taking into account the polymer resin's elastoplastic character, it was considered that the 4th relation reflects the behaviour of the polymer putty and as it follows the breaking stresses of specimens were established with this.

The results are written in the Table 4.

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bending for polymer putty					
	Breaking stresses in tensile from				
Nb. of	bend	ing, σ_{ti} ,			
specimen	in daN/cm	n ² , for putty:			
	poliester	epoxide			
1	431,7	321,4			
2	455,5	273,0			
3	398,7	316,5			
4	408,4	345,6			
5	379,3	311,9			
6	396,8	338,2			
7	410,7	373,0			
8	422,5	327,5			
9	409,9	301,0			
10	440,0	294,8			
Mean	415.35	320.29			

Table 4. Breaking stresses in tensile from

5. The reinforcement of the polymer putty

The applying place of the polymer putty have imposed researching the tensile characteristics improvement of the material, that leaded its reinforcement with glass fibers, for obtaining a new construction material, which is in the group of the materials reinforced with fibers, farther on named *"composite polymer putty"*.

The testing method for this new material, [5], is different from that of putties and is stipulated in international standards. Thus, the specimens are plates with a variable number of reinforceing layers.

For compression, these have the shape and dimensions from Fig. 3, with the testing device from Fig. 4, and for bending are used different dimensions plates, according to the thickness of the specimens.



Fig. 4. Compression testing device of specimens.

sleeve

Following there are given, for one layer of glass fiber tissue FER 3 reinforced composite polymeric putty, the modulus of the breaking stresses in axial compression and bending, Table 5. The values were determined for a lot of 10 specimens.

Table 5. Breaking stresses for composite polymer putty in axial compression and bending

		Breaking stresses, in daN/cm ² , at:					
Nb.of	compi	ression	bending				
specim.		for polyme	er putty type:				
	polyester	epoxide	polyester	epoxide			
1	2872	2632	1683	1441			
2	2097	1988	1837	1472			
3	2989	2014	1788	1513			
4	3127	2125	1844	1824			
5	3089	2418	1710	1506			
6	2666	1994	1805	1409			
7	2690	2005	1613	1608			
8	2821	2553	1936	1816			
9	2636	2397	1677	1843			
10	2127	1979	1734	1750			
Mean	2711	2211	1763	1618			

The quality contribution of reinforceing the polymer putty with glass fiber tissue for its mechanical characteristics is spectacular.

While manufacturing and applying the specimens of this material on the building site, was stated that:

- the epoxide resin is more fluid than the polyester one, so that the cement mixing is more easily made;

- in this resin can be incorporated a bigger quantity of filling material;

- the exothermy is the same for both types of thermosetting resins;

- the adherence of polyester putty on the old concrete/mortar surface is better in case of using the polyester resin;

- at the composite polyester putty strikingly appears the tixotropical effect of the mixture, while the composite epoxide putty has the tendency of leakage free flowing on vertical surfaces;

- the adherence of the polymer putty at the reinforcement layer is influenced by the type of glueing with which is trated the glass fiber tissue. So, between the polyester resin and reinforcement the compatibility is perfect;

- athe hardening of the material on the walls of those two tanks, the aspects is of natural stone;

- the protection to corrosive agents of the polymer putty is due to the phisico-mechanical properties of the resins.

6. Conclusions

Wall's surfaces treatment, in the collecting residual sulphurous waters tanks, from the prepurifying of the Nicolina - Iasi Balneary Complex, was realized with a modern material, belonging the group of glass fiber reinforced composites.

The applying technology of the finishing is simple and consists of the following actions:

- cleaning and repairing of the concrete or mortar surface by grinding and then puttying;

- application by filling of the first layer of polyester putty, with a thickness of 1 ... 2 mm;

- application on the putty surface, recent and, unhardened, of a glass fiber tissue layer. The reinforcement is partially impregnated with resin;

- keeping the hardening time of the polyester putty, aprox. 6 hours;

- application of the second layer of polyester putty that is recommended to e more plateful in resin that the first, for well impregnating the glass fiber tissue.. The thickness of this layer doesn't exceed 1,5 mm.

After 24 hours from the application of the last layer of polyester resin, the tanks could be set working.

Using the composite polyester putty presents the big advantage of a low price, almost twice lower than in case of using the epoxide resin.

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Correlation between Composition and Properties for Alkali Activated Blastfurnace Slag Binders

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Abstract: This paper presents some results of tests carried out on mechanical strengths of alkali activated blastfurnace slag. X-ray diffraction was used in order to bring some information about the nature of mineralogical components and the amount of glassy phase. Using chemical composition, usual activity indices for slags have been calculated and then compared with corresponding mechanical strengths.

Keywords: Blastfurnace slag, alkali activation, binder.

1. Introduction

The alkali activation of blasfurnace slag has been proposed by Glukhovsky since 1957, but the investigation and testing of this type of binders have been continuing to develop, in the last period being industrially produced cements with strengths over 100 MPa (Ukraine) or usually ones, like finish F- cement[1-3].

In our country research has been made on the use of sodium silicate as activator [4].

This type of binders consists of an aluminosilicate component, usually grounded at specific area greater than 2500 cm^2/g , activators and additives.

The aluminosilicate component is usually blastfurnace slag with or without fly ash addition.

Vitreous slags are prefered; more than that, between the main oxidic components some relations are usually required [5]:

$$1.2 < SiO_2 / Al_2O_3 < 4$$
 (1)

$$0.8 < CaO/(Al_2O_3 + SiO_2) < 1.25$$
 (2)

Blastfurnace slag reacts very slow with water and hardening properties are rarely present, only in strongly basic ones.

Blastfurnace slag activation allows hydration and hardening processes to develop with a

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convenable rate and is achieved by mechanical, chemical and thermal means.

Mechanical activation is achieved by grounding blastfurnace slag, chemical activation means the addition of some substances which greatly enhance the rate of reaction between slag and water; thermal activation is achieved by increasing temperature.

Alkaline hydroxides, carbonates, aluminates and silicates are used like chemical activators.

The aim of this paper is to point out the role of the binder composition, from the point of view of slag chemical and phasal composition and of the activator amount, on to its physico-chemical properties.

2. Materials and methods

It were used blastfurnace slags from Galați - GI, GII, GIII, Hunedoara - H and Călan - C, with main oxidic compositions given in Table 1.

These slags were grounded to Blaine specific area of 3000 cm²/g; for GII specific areas of 4000 (GII4) and 5000 (GII5) cm²/g were achieved.

Diffraction tests XRD using X-rays were carried out on grounded slag, in order to apreciate the main mineralogical and phasal composition.

The slags were activated with 3, 5, 7, 9% NaOH w/w ratio to binder. Using these components, sand, as described in SR EN 196-1 and water, it were prepared cubic samples 20x20x20mm from resulting mortar.

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14010 1. 011011								
Slag type	SiO ₂ (%)	Al ₂ O ₃ (%)	Fe_2O_3 (%)	CaO (%)	MgO (%)			
GI	40,61	10,82	2,18	37,58	3,97			
GII	32,57	16,82	1,48	40,19	4,74			
GIII	44,28	10,74	2,18	36,42	3,77			
С	28,71	10,59	4,34	40,65	7,52			
Н	34,50	10,55	2,28	42,10	5,75			

Table 1. Chemical composition of the used slags

Water/binder ratio was determined in order to ensure the same workability for all mortars; workability has been estimated using the method described in Romanian Norms C 248-93.

Based on the results obtained in this stage, standardised tests on water requirement for preparing standard consistency paste, setting time as described in SR EN 196-3, compressive strenghts, flexural strengths as described in SR EN 196-1 and drying shrinkage has been carried out.

3. Results and discussions

The XRD analysis made on each grounded slag (Fig.1) bring some information about the nature of mineralogical components and the amount of glassy phase.



The slag contains besides glassy phase, some crystalline compounds: quartz (3,34; 4,26; 1,82A), rankinit (2,66A) and melilites (2,86; 3,67; 3,05; 2,44 A).

The amount of glassy phase is quite different in analysed slags, GI type seeming to have a much greater proportion of crystalline phase. The results obtained on cubic microsamples show an optimum range of 5-7% w/w NaOH, for all types of slag, closed to those reported by others [6] (see fig.2).



Fig.2. Compression strength vs.time for GII (4000 cm^2/g) type slag activated with NaOH

An 6% w/w NaOH ratio was chosen for the rest of the experiment.

The results of the tests on water requirement for preparing standard consistency paste (WSCP) and setting time are shown in Table 2. Compressive strengths increase with the specific area of the slag (see fig.3).



Fig.3. Compressive strength vs. time for different specific areas of GII slag activated with 7%w/w NaOH

The values of water requirement for preparing standard consistency paste are lower than corresponding values for Portland cement. This feature will permit to reduce water/cement ratio in tests on mechanical strengths

Table.2.	Water requirement	for preparing standard	l consistency paste
----------	-------------------	------------------------	---------------------

and setting ti	and setting time for different types of slags, activated with 6% NaOH								
Slag		GI	GII 3	GIII	С	Н	GII 4	GII 5	
WSCP (%)		20,5	24	25,5	24	26	24,5	25	
Setting	initial	130	55	95	25	45	50	40	
time (min)	final	210	90	140	40	130	80	70	

Water requirement for preparing standard consistency paste presents a very low increase with the raise in specific area of the binder.

Initial setting time have values over 45 minutes (minimum accepted value for Portland cement), except C slag.

The values for initial and final setting time may be correlated with the basicity and the phasal composition of the slag. GI type slag has the greatest amount of glassy phase and a quite low basicity, which determine an increase of setting time.

Finer grounding allows hydration process to have a higher rate of developing and consecquently, setting time becomes smaller: initial setting time



Fig.4. Compressive strength vs. time for 6% w/w NaOh activated blastfurnace slag grounded at specific area 3000 cm²/g

It can be seen in these figures that mechanical strengths of alkali activated slags are influenced by compositional factors.

Finer grounding lead to an increse of compressive strength, especially at higher value of specific area. In this case compressive strengths can reach about 30 MPa after 28 days of hardening, value which represents the lower acceptable limit for Portland cement.

The development of hardening properties is quite fast, after 2 days of hardening compressive strength becomes more than half of that corresponding to 28 days value.

In order to relate mechanical strengths to the activity of the slag, some indices from literature could be defined:

decrease from 55 min. for GII 3 at 40 min. for GII5.

The evolution of compressive strength of prismatic samples of 40x40x160mm, prepared from mortar with 1:3 binder/sand ratio and 0.45 water/cement ratio is shown in fig.4 and Fig.5.



Fig.5. Compressive strength vs. time for 6% w/w NaOH activated blastfurnace slag GII grounded at different specific area 3- 3000cm²/g; 4- 4000cm²/g; 5 - 5000cm²/g

-basicity index I_b , with values greater than 1 for basic slags; when I_b is raising the hydraulic activity of the slag increases[7].

$$I_{b} = (\%CaO + \%MgO + \%Al_{2}O_{3})/\%SiO_{2}$$
(3)

-activity index I_a [5]:

$$I_a = \% SiO_2 / Al_2O_3 \tag{4}$$

-quality index F [8]:

$$F = (\%CaO + 0.5\%MgO + \%Al_2O_3)/(\%SiO_2)$$
(5)

The numerator consists in a sum of compounds with good influence on hydraulic activity, while those with bad influence are situated at denominator. -quality index i [5]:

$$i = (\%CaO - 1.1\%Al_2O_3)/(SiO_2 - 0.6\%Al_2O_3)$$
 (6)

This index shows the saturation of the calcium pres silicates; from CaO and SiO_2 the oxides bond in

 C_2AS has been released. Very active slags have i >1.6, the active ones 1.3<i<1.6, and usable ones i >1.

Values of the indices for the studied slags are presented in Table 3

Table 3. Indices of activity for studied slags and corresponding 28 days-compressive strength R_{c28} for 6%w/w NaOH activation

Slag	Ia	I _b	F	i	R_{c28} (MPa)			
GI	3,25	1,29	1,24	0,75	13,8			
GII	1,94	1,90	1,82	0,96	19,4			
GIII	4,12	1,15	1,11	0,65	13,4			
С	2,71	2,05	1,92	1,30	20,8			
Н	3,27	1,90	1,61	1,08	15,8			

Acording to I_b , all studied slags are basic; this index divide these slags in to groups: one of low basicity - GI and GIII and one with higher basicity - GII, H, C.

Acording eq. (1) GIII has a very poor activity index $(I_a>4)$.

Acording quality index i, only C and H slags are usable for binders.

From F index point of view C, GII and H slags will show acceptable hardening properties.

It can be seen that all presented indices offer some qualitative information about mechanical strength of the binder.

It can be assumed that higher value of I_b , F and i indices means higher value for compressive strength, but this assumption could be valid only for almost the same degree of crystallinity of the slags.

The crystalline phase of the slag have a different chemical composition than slag and have no hydraulic properties [8].

Grounded industrial slag, slowly heated to 1200°C and then slowly cooled to room temperature, in order to obtain mainly crystalline phase, show no hardening properties after one month, when activated with 6% NaOH.

For a better characterisation of the hydraulic activity of the slag, test should be carried out in order to establish the quantity of vitreous phase and its chemical composition.

The evolution of flexural strength for prismatic samples of 40x40x160mm is shown in Fig.6.

The values of the bending strength at 28 days are greater than 4 MPa even for the less active slags

(GIII, GI); the active ones show values near 6 MPa, comparable with those corresponding to Portland cement.

Drying shrinkage of standard mortars, at 120 days determined as described in STAS 2634-80 have values between 0,56 and 1,63 ‰, depending on the type of slag and raising with increasing specific area.



Fig.6. Flexural strength vs. time for 6% w/w NaOH activated blastfurnace slag grounded at a specific area of 3000 cm²/g

4. Conclusions

The results obtained on cubic microsamples show an optimum range of 5-7% w/w NaOH, for all types of slag.

The mechanical strengths of alkali activated slags are influenced by compositional factors. Finer grounding lead to an increse of compressive strength, especially at higher value of specific area.

Compressive strengths of about 30 MPa can be reached after 28 days of hardening.

Usual activity indices offer only some qualitative information about mechanical strength of the binder, which have to be correlated with phasal composition too.

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Behaviour of an industrial building made of monolith reinforced concrete at disruptive effect of fire

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Abstract: Fire action is quantized as particular and essential action of constructions generating the concrete involution phenomenon (explosion, depletion of bearing force etc.) with special problems for structural rehabilitation. In this respect, the current specifications and norms comprise prevention and abatement requirements but however, builder specialists regularly encounter such kind of less common and less controlled problems. In the paper are presented some aspects concerning concrete explosion mechanisms producing damages and early collapse of reinforced concrete elements.

Keywords: Industrial Building, Fireproofness, Concrete Explosion, Spalling

1. Introduction

Following a technical accident that took place in a hall located in the pulp and paper mill Chiscani, the strength structure of the hall was subject to the disruptive effect of fire.

The fire flared between operational elevation marks 9 and 15 affecting the building strength structure. The building, covering the industrial process was built in 1962 and its structure was made of monolith reinforced concrete frames with 9-m openings and 12 and 15 m-pile bents. The built area is 1750 sq.m and the developed area is 3500 sq.m. The plane conformation of the structure is shown in fig. 1.



Fig. 1 The plane conformation of the structure ISSN-12223-7221

It is mentioned that due to technological processes under way, the building has gradually been corroded.

2. Noticed damages of structural elements

As a result of the fire, damages noticed have been summarized on structural elements thus:

- Frame poles:
- smoke and soot deposits;
- exfoliated plastering;
- edge plastering;
- microcracks;
- exposures of strength reinforcement;
- coating with exfoliated concrete;

- change of pink concrete toward black-gray concrete;

- breakings at the middle of pole height (I4 - breakings to 45°, A5, A7, I7, etc.);

- reinforcement buckling (pole G4);
- stressed pole (pole G4);

- poles, bottom broken at around 0.5 m from the floor level.

- Main beams:
- soot deposits;
 - cracked, exfoliated plasterings;

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- cracks in beam structure, mixed effects of corrosion and fire;

- exposures of strength reinforcement;
- breaking of clamp bottom arms.
- Secondary beams and plates:
- soot and smoke deposits;
- exfoliated plasterings;

- exfoliated edges, exfoliated concrete coating;

- plate cracks and microcracks;

- beams in axle 4, 5, C, E have 3 cm-deflections.

3. Structural elements behaviour analysis

Tests have been carried-out non-disruptivelly and also, by extraction of core-samples that have been tested at compression. Tests performed highlighted that temperatures have changed in time (mainly between 2-10 axles), limited variations being recorded for the remaining hall pile bents. Areas with temperatures up to 100°C have been characterized by moisture and when increasing temperature up to 250°C, release of physically combined water and early stage concerning release of chemically combined water have been noticed.

The temperature threshold of 400-500°C have been characterized by breaking of chemical bonds and destruction of cement block structure ("explosion" phenomenon). Due to the temperature rise to 600-700°C (pole zone), concrete did not exist as such any longer, most aggregates loosened, concrete crushed and elements failed.

At the same time, a different temperature distribution on vertical has been noticed (rate of fire travel vertically is approximately 4 times higher than horizontally). During the cooling process, microcracking has continued. Cement block was shrinking while temperature was increasing and aggregates were expanding.

Cement block shrinkages during cooling explain decrease of concrete strengths and even collapse of elements.

Concrete tensile strength is decreasing while temperature is increasing.

Concrete compression strength has though a different variation: it increases up to temperatures of 300°C and then it drops.

Reviewing the behaviour of structural elements on zones and temperatures achieved and recorded at concrete surface, the following has been noticed: up to 300° C, concrete had a normal behaviour; up to 400° C, significant deformations were noticed; up to 500° C, drops of bearing force were recorded because above this temperature, decreases in the σ - ϵ chart development appeared.

4. Characteristics of reinforced concrete elements' collapse during fire

As known, the fireproofness use up mechanism of reinforced concrete elements offers the next specific features:

a) "The concrete explosion". This is about an extremely violent ripping, in fact a spalling. It is established that after fire, all concrete's area presents spallings of varied shapes and depths (concrete's sections from backside of the floor at elevation mark +15.00 were detached, the detached beams' edges uncovers the reinforcement).

b) The damage of stability by the concrete explosion in the maximum area efforts.

The test of areas liabled to the fire certifies the fact that the explosion breaks out in the first 20th minutes.

It was not able to find out the correlation between the humidity of concrete and the explosion itself.

Also it seems that the fire affects the monolith plate of reinforced concrete, like the enclosed span which didn't support the concrete explosion.



Fig. 2 Zones with expelled concrete in fire

As far as the behaviour of reinforcements at high temperatures is concerned, it became relevant after 400° C.



Fig. 3 Zone adjoining to those with expelled concrete

5. Plate elements and thin ribs stress analysis

As known, plate unit stresses caused by high temperatures appear in fact, on 2 perpendicular directions. By their combination, significant stresses appear and consequently, a significant loading of surface compression on two directions at the plate internal fibre and two direction tensile in the middle zone of plate height is noticed.



Fig. 4

Here are some guide marks of the explosion mechanism producing the early collapse of plate elements and thin ribs without warning:

a) In a rib subject to fire on both directions of plate elements and thin ribs, temperature varied in section in line with sketch shown in figure 5. Compression loads are identical on the two sides and proportional with the temperature curve.





b) Unit loads are proportional with temperature drop between outside and inside and not with effective section temperatures. In other words, the higher temperature gradient is, the higher resulted stresses are.

As in case of plates, stresses at ribs and beams are noticed on both directions. On the rib bottom side, compression stresses are noticed with a certain temperature distribution (on two perpendicular axis, on rib direction, on rib cross).

These unit loads are lower than those in the middle zone.





6. Concluding remarks

Conclusion achieved as a result of tests carriedout on types of elements and characteristic zones has led to the fact that, on zones subject to operation actions simultaneously with fire action, the degree of vulnerability to concrete "explosion" has increased considerably.

After about 30 minutes, temperature increasingly rises and section unit loads change. Experimental results done by INCERC have emphasized that compression as value drops but it develops in the plate depth.

When concrete fibre subject to fire is over 500-600°C, compression strength and elastic modulus decrease significantly and the compressed zone is plasticized. No explosive breakages are produced due to the brittle nature of concrete.



Fig. 7

These detachments in advanced stages without an explosive nature are generally favored by reinforcement that under high temperatures and an advanced cracking condition, it is subject to two phenomena, both favoring concrete detachment in this zone:

- a significant reinforcement expansion than concrete - though heat expansion factors are approximately the same - due to a much higher conduction and radiation transfer factor both of external medium as well as cracked concrete sides;

- a fast steel corrosion because of high temperatures, iron oxide (FeC) is built up with considerable smoke increases.



Fig. 8





Fire action is quantized as particular and essential action of constructions generating the concrete involution phenomenon (explosion, depletion of bearing force etc.) with special problems for structural rehabilitation. In this respect, the current specifications and norms comprise prevention and abatement requirements but however, builder specialists regularly encounter such kind of less common and less controlled problems.

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A Study of High Performance Concretes to be used in Different Constructions

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Abstract: Starting from the idea that lately in the world, the high performance concrete (HPC) applications became very important and there is a lack of Romanian norms, we have studied the importance of various composition factors in increasing the classical concrete performances.

In order to obtain HPC at a convenient price it was used a currently produced Romanian cement with a little higher dosage, different admixtures (either Romanian or imported) and an addition of silica fume (SF).

In this paper there are presented some concrete compositions that have been studied and the evolution of their physical and mechanical properties. Regarding the study results they lead to two conclusions:

-It is possible to produce HPC in Romania, using currently made materials;

-The properties of prepared and studied HPC make them interesting for applications in different constructions.

Keywords: high performance concretes, admixtures

1. Introduction

High performance concretes (HPC) are those concretes that besides the highly mechanical strengths possess an extremely increased durability, and, at the same time, the other performances are superior as compared with the usual concretes performances.

The use of HPC has been reflected in a rapid increase of constructions in the countries with developed industry, i.e. mostly the constructions that require not only mechanical strengths but also durability (high civil engineering, bridges, harbour works, marine drilling rigs, etc).

We are of the opinion that the use of such concretes in different constructions could provide increased safety or – at the same safety coefficient – there could be achieved important savings due to a superior suppleness of the construction elements (construction weight decreases, foundations become reduced, transport expenses diminish).

HPC show a much faster hardening, being able to reach only after 3 days mechanical strengths that the usual concretes attain after 28 days.

We consider that the current use of these concretes abroad and in our country in the future will avoid some of the classic concretes faults, having in view their principal advantages: -Reduce weight of construction elements with 20-30% by reducing the section at the same load carrying or an increased load carrying capacity of the construction element when keeping the same net section.

-Reduce by 10-20% cement consumption that means the use of high quality cements and reduction of concrete amount necessary for that work.

-Reduce the necessary reinforcing by 8-20%

-Make more valuable the aggregate quality

-Enlarge the concrete applications towards new construction elements, the use for military engineers works included.

For the HPC study and production, I applied the following strategy:

-Studying the possibilities of improving the concretes performances by the intervention on each concrete component or being correlated.

-Studying comparatively the concrete produced with components (admixtures) domestic or imported.

-Using in the formula of HPC of some Romanian cements – cheaper – in order to replace some special high performance cements, according to certain existing instructions (yet not standardized) [9,10].

When making studies on concretes, special consideration was given to enhancing performances, by complex additivation and introducing as addition of silica fume –SF- (in Romania it is obtained as a sub

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product from alumina manufacturing, through electrostatic filtration of the furnace gases).

2. The principal materials used

Aggregates : river sand $0\div7$ mm and granite chipping of $3\div8$ and $8\div16$ mm ; high performance concrete theory requires aggregates of a lesser maximum diameter than the usual concretes. The chosen grading of the aggregate [4] is presented in table I.

Ta	bl	e	I
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Screen mash diameter	0.2	1	3	7/8	16
Passage %	4	20	24	53	98

Cement type was BS 12/78, unitary Portland cement produced by SC Lafarge Romcim SA Medgidia, grinding fineness 3127 cm²/g and Rc $_{28} = 47.8 \text{ N/mm}^2$.

Water used responded to standards in force.

Silica fume (SF) used as an addition has $85 \div 98$ % SiO₂ and a particle fineness of about 20 m²/g.

Admixtures used and their percentage in relation to the cement dosage (c) are shown in table II which emphasizes some concrete compositions we dealt with (Rheobuild – France, Durasar – Canada, the other admixtures are Romanian).

3. Designing the concrete composition

We started from the prerequisite of obtaining Bc 65 (over C50/60) class concrete (65 N/mm²). The compositions of the concretes are presented in table II.

Sample	Cement		Aggregate kg	$/m^3$	Water	w/c	admixture	SF
code	dosage	0/7	3/8	8/16	l/m ³	w/c+	%cement	% cem.
	kg/m ³	river	crushed	crushed		+SF		kg/m ³
А	600	391.5	469.8	704.7	210	0.35	-	-
В	600	391.5	469.8	704.7	210	<u>0.35</u>	-	<u>8%</u>
						0.324		48
С	600	391.5	469.8	704.7	210	0.35	DURASAR	<u>8%</u>
						0.324	0.4% s.u.	48
D	600	391.5	469.8	704.7	138	0.23	FLUBET	<u>8%</u>
						0.213	2% sol.	48
Е	600	391.5	469.8	704.7	210	<u>0.35</u>	VIMC11	<u>8%</u>
						0.324	1.2% sol.	48
F	600	391.5	469.8	704.7	150	0.25	LSC	<u>8%</u>
						0.216	1.75% sol.	48
G	600	391.5	469.8	704.7	150	0.25	LAS+LV	<u>8%</u>
						0.216	1.33% sol.	48
Н	600	391.5	469.8	704.7	150	0.25	RHEOBUILD	<u>8%</u>
						0.216	1% sol.	48
Ι	600	391.5	469.8	704.7	155	0.258	ADCOM	8%
						0.238	2% sol.	48
J	600	391.5	469.8	704.7	231	0.385	LSA+HNO ₃	8%
						0.356	1.8% sol.	48

Table II

4. Fresh concretes properties

Properties taken into consideration were workability (settling) and density. Final results are presented in table III. Control sample is considered code B concrete.

l able III			
Sample	Settlings (cm) / workability	Appar	ent density
code		kg/m ³	% control sample
А	4.7 / L2-L3	2431	100.5
В	0.5 / L1	2418	100.0
С	1.0 / L1	2509	103.8
D	0.3 / L1	2491	103.0
Е	0.5 / L1	2491	103.0
F	1.5 / L1	2473	102.3
G	0.5 / L1	2455	101.5
Н	0.2 / L1	2505	103.8
Ι	0.5 /L1	2500	103.4
J	5.2 / L2-L3	2345	97.0
		0 0	

Concretes were produced on a constant workability basis. Densities variation as compared with the sample test with w/c 0.35 is between +1% \div - 5% (respectively + 38 kg/m³ \div -117 kg/m³).

5. Hardened concretes properties

It was studied the apparent densities, porosities, strength evolution (destructively) and dynamic elasticity module evolution (nondestructively and by calculation). The purpose was each time to make observations of the role and influence of composition factors on concretes high strengths.

The results in table IV show that the admixtures created slight density increases (as well as for fresh concrete with the same code) compared with the control sample, except for the concrete with LSA+HNO₃, where the apparent density of the concrete (fresh and hardened) is lesser than the control sample, possibly due to the slightly increased w/c ratio (0.385 in contrast with 0.35).

The SF addition worked favorably on porosity, reducing it with 16% even for concretes with no admixtures.

Table IV					
Sample	Apparer	nt density	Apparent porosity		
code	kg/m ³	% control sample	%	% control sample	
А	2406	101,5	6,16	116	
B*	2364	100	5,31	100	
С	2464	104,2	3,57	67,2	
D	2464	104,2	3,39	63,8	
Е	2464	104,2	3,32	62,5	
F	2373	100,4	3,03	56,9	
G	2436	103,1	3,11	58,6	
Н	2482	105	2,05	38,6	
Ι	2427	102,7	3,10	58,4	
J	2300	97,3	5,82	109,6	

• Control sample

Within the limits of w/c ratios used, the variation of hardened concretes densities in contrast with the w/c=0.35 control sample is of no significance (- $2.7\% \div +5\%$). As compared with the

fresh concretes with the same code, the mass losses chiefly caused by water evaporation, in 28 days, are presented in the table V.

T	al	bl	e	V
T	a	UI	E.	v

w/c	0.385	0.35	0.258	0.25	0.23
Total of mass losses kg/m ³	45	39	53	49	27
Total water in fresh concrete l/m ³	231	210	155	150	138
Retained water % cement	31	28	17	17	18.5

At the same w/c ratio, admixtures reduce the apparent porosity, which leads to concrete performances improvement. A w/c ratio increase conducts to apparent porosity increase but its reduction decreases porosity. The most important porosity reduction was obtained when w/c = 0.25.

For studying the concretes mechanical properties we considered the w/c ratio and admixtures influence on the value and evolution of tensile – bending strength (Rti) and compression strength within 90 days as well as the value of dynamic elasticity modules (Ed).

We executed destructive and non – destructive trials to evaluate the concretes quality and by calculating some of the above - mentioned characteristics.

We considered control samples the non - additivated concretes (code A) and the non - additivated concretes with 8% SF (code B - basic control sample).

When performing destructive trials, we found that the breakage occurred largely in aggregate.

The table VI shows the evolution of concretes' strength up to 90 days (on a destructive determination).

Table VI

	R_{ti} (extension-flexure strength)						R _c (compression strength)					
Sample	3 d	ays	28 0	lays	90 c	lays	3 d	ays	28 0	lays	90 d	lays
code		%		%		%		%		%		%
	N/mm ²	control	N/mm ²	control	N/mm ²	control	N/mm ²	control	N/mm ²	control	N/mm ²	control
		sample		sample		sample		sample		sample		sample
Α	5.1	92.7	6.3	103.3	6.8	128.3	34.8	83.9	58.3	87.9	61.8	84.1
В	5.5	100.0	6.1	100.0	5.3	100.0	41.5	100.0	66.3	100.0	73.5	100.0
С	8.1	147.3	8.1	132.8	8.8	166.0	56.3	135.7	80.5	121.4	88.0	119.7
D	7.3	132.7	8.3	136.1	9.2	173.6	49.0	142.2	77.3	116.6	85.5	116.3
Е	7.4	134.6	8.2	134.4	6.6	124.5	57.7	139.0	73.3	110.6	87.5	119.0
F	7.1	129.1	7.2	118.0	7.9	149.1	51.5	124.1	81.8	123.4	84.0	114.3
G	6.8	123.6	7.2	118.0	8.1	152.8	65.5	157.8	83.3	125.6	93.8	127.6
Н	8.6	156.4	7.4	121.3	9.6	181.1	59.5	143.4	74.0	111.6	101.5	138.1
Ι	7.3	132.7	9.1	147.5	8.8	166.0	53.5	128.9	76.8	115.8	95.5	129.9
J	5.6	101.8	6.5	101.6	6.4	120.8	23.7	57.1	53.0	79.9	60.5	82.3

For non – destructive trials there has been used an ultrasonic apparatus (N 4207). We could evaluate the quality of the concretes studied on the basis of ultrasonic pulse speed, by means of which A.M.Neville [7] classifies the concretes into some quality classes (table VII).

After some determinations made on 55 cm test pieces we obtained the values of longitudinal pulse speed (m/s) – in table VIII – for the studied concretes quality Having the ultrasonic determinations (at 90 days) on test pieces (10 cm),

we calculated the dynamic module (Ed, N/mm^2). In table IX some results are presented.

1	ľa	b	le	V	Ш	

Concrete quality	Longitudinal pulse speed (m/s)
Excellent	> 4500
Good	3500 - 4500
Uncertain	3000 - 3500
Poor	2000 - 3000
Very poor	< 2000

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Tabl		VI	II
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Tuble VIII		-				-	
Sample	Time (10 ⁻⁶	Speed	Concrete	Sample	Time (10 ⁻⁶	Speed	Concrete
code	s)	(m/s)	quality	code	s)	(m/s)	quality
А	118	4661	Excellent	F	119	4622	Excellent
В	122	4508	Excellent	G	118	4661	Excellent
С	118	4661	Excellent	Н	117	4701	Excellent
D	117	4701	Excellent	Ι	119	4622	Excellent
E	119	4622	Excellent	J	127	4331	Good

Table IX

Sample	Т	Time (10 ⁻⁶ s	5)	\mathbf{v}_1	t ₀₁	v ₀₁	ρ	Ed
code	t ₁	t ₂	t ₃	(m/s)	$(10^{-6} s)$	(m/s)	(kg/m^3)	(10^3 N/mm^2)
А	20.5	29.3	36.0	4878	22.6	4425	2400	46.994
В	20.3	30.3	36.3	4926	24.3	4115	2364	40.030
С	20.6	29.3	35.6	4854	23.0	4348	2464	46.582
D	20.3	29.0	35.4	4926	22.6	4425	2464	48.247
E	21.0	29.5	35.8	4762	23.2	4310	2464	45.772
F	20.6	29.6	35.7	4854	23.5	4255	2373	42.963
G	20.8	28.9	34.6	4808	23.2	4310	2436	45.251
Н	20.7	29.1	35.0	4831	23.2	4310	2482	46.106
Ι	20.4	29.6	35.4	4902	23.8	4202	2427	42.853
J	23.6	32.4	38.7	4237	26.1	3831	2300	33.756

Table X

Sample code	Ed determined non- destructively $(10^3 N/m^2)$	Ed calculated according to [8] (10^3 N/mm^2)
	(10 N/mm)	(10 10/11111)
А	46.994	44.011
В	40.030	46.005
С	46.582	48.266
D	48.247	47.891
E	45.772	48.192
F	42.963	47.662
G	45.251	49.118
Н	46.106	50.209
Ι	42.853	49.363
J	33.756	43.779

For comparison, dynamic elasticity module have been calculated according to CP 110/72 [8] data, the results are presented together with the values obtained from non-destructive determination in table X. For calculation we used R_c^{90} .

It may be observed that the calculated Ed values are higher than the experimental ones.

6. Conclusions

The composition of 0/16 mm aggregate concretes has been established to obtain the concrete Bc 65 (over C 50/60 - 65 N/mm²). In order to reach the class, taking into consideration the cement type used, we worked with an increased dosage over the optimum one.

The experimental results proved, once more, that without superior performance cement, the class of the

concrete could not be obtained even with the use of high dosages.

The use of SF for non – additivated concretes gives an increase of strength but not sufficient to obtain high performance concretes when a high performance cement is not used.

The use of water reducing admixtures and implicitly strength increasers allows the preparation of high performance concretes using a common cement (a slightly increased dosage) and high quality aggregates.

In order to obtain HPC under the specific circumstances of the researches carried out for this work, there should be worked on as little as possible

w/c ratios, this leading to a reduced workability. Using such concretes requires a strong clustering.

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The evolution of strengths in time, as a measure of concretes hardening, is different for additivated concretes in contrast with the non - additivated ones. Concretes with no admixtures, after 28 days, do not show remarkable strength increase, while the concrete with admixtures continue the hardening.

On the basis of these results, I consider that, for the additivated concretes, which should manifest high performances, the strength appraisal will be after 90 days and not 28 days as for usual ones.

High elasticity modules, increasing with the strength and, consequently, with the admixtures presence, are still representing a shortcoming of HPC.

For achieving high strengths, there should be observed all the requirements concerning material quality, composition and working technologies..

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Decorative Concrete

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Abstract: The paper represents a synthesis of the aspects concerning the decorative concrete. It deals with three of the most important problems of the decorative concrete, i.e. the change of the colour, surface treatment and degradation caused by dirtiness. The following factors, which determine the change of the colour of the concrete, are analyzed: the W/C ratio; type of casing; the porosity of the casing, inadequate vibration, striking time, the hardening and using conditions. The treatments by which there can be obtained the decorative concrete, having the aspect of sandstone or marble, are presented, as follows: washing, acid treating, mechanical treatment (polishing, buffing, sanding, stone granulating, milling). The paper ends with the analysis of the factors which determine the colouring (dirtiness) of decorative concrete, the ways of preventing and remedyng them.

Keywords: Decorative concrete, the composition of the concrete, the shade, the sandstone.

1. Introduction

Concerning the decorative concrete, the shade of the concrete is very important. If, after hydration the white cement remains white, the grey cement could be of different shades sometimes very diverse and often unforseeable.

The homogeneity of the "grey" shade is influenced by the following factors:

- a) the shade variation of the dry cement; the cements of the same class and resulted from the same source are not always uniform and homogeneous from the point of view of the gray shade;
- b) the variation of W/C ratio a defference of 0.05 is enough to make the shade of gray lighter (when W/C increases) or to make darker the shade of gray (when W/C decreases);
- c) the variation of the casing porosity could influence the darkening of the colour of the cement – because of the decrease of W/C ratio at the contact surface between the casing and concrete;
- d) the inadequate vibration determines the phenomenon of deposit of the grains depending on the diameter, on the surfaces, remaining only the cement paste;

- e) the time interval at which the striking is made; if the interval is short, the shade is light gray, if the interval is long the shade is dark gray;
- f) consequently, the drying conditions after striking influence the dry and warm air determines a light colour concrete and the very dry air but very wet causes a dark colour.
 Both in the case of drying conditions and striking

interval, the cause of the gray shade variation is represented by the warmth released at the hydration of cement.

2. Dismantled Concrete

The surface of the concrete sometimes called the "shell" is, in general, made of hydrated ciment paste and it is in direct contact with the casing. The colour of this exterior layer is determined by the type of white or gray cement used, by the very fine mineral pigments possible incorporated in the ciment paste (< 5%) and because of the very fine sand particle ($\phi < 0.1$ mm).

The thickness of this layer is very small (some mm) – Fig.1a. It is exposed to the rain of a more and more acid character, and to the abrasion phenomenon, too. As a result of these processes, the surface layer can disappear occuring the aggregate grains (Fig.1b.)



Fig.1. a) The surface of the concrete; b) The surface of the concrete after the rain action

In particular conditions the colour of the aggregate interferes as a factor which affects the shade of the surface, on a long termen. Therefore, it is recommended to take this factor into consideration when the composition is established.

3. General measures for obtaining a shade of gray as uniform as possible

Some of the most important general measures are those comected with the composition and casing.

The composition of the concrete influences the shades of gray by its components. In order to obtain an as uniform as possible shade of the concrete the value of the W/C component should be kept constant. The aggregate is another important factor, but the sand is more important. At the same time, the consistency of the concrete must be chosen so that it should resist to the compression it is subjected to during the use. At the same time the consistency must be fluid enough in order to be able to penetrate, inclusively, into the very narrow casings and not to influence the position of the reinforcements.

The stability of the ciment paste is insured through by a fine enough composed from cement, very fine sand and other components (lime, granular filler and quick ash).

The casing used must be lubricated with oil; they must have a uniform porosity and all the joints to be closed so that to avoid the leakage and grout loss. If these conditions are fulfilled the colour of the concrete is uniform after striking without being noticed tracks of casings.

4. The analysis of the incidences due to the heat transfer towards the exterior

During the hydration of the cement there is released an impressive quantity of lime, up to 250g/kg of cement, in the case of Portland cement. This lime, being in the mass of the concrete, can remain or migrate towards the surface in the form of a solution, in the water which is in the capilarity of the fresh concrete.

According to the type of the capilaries (wide or narrow) and to the external atmospheric conditions (a very dry, wet or raing atmosphere) the migration of the lime towards the exterior is variable and it modifies the aspect concerning the brilliance, luminosity even the staining of the surfaces.

If the striking time is short the capilarities are still very wide (Fig.2) real boulevards for lime) and lime can easily reach the exterior, obtaining brilliant surfaces.



Fig.2 The migration of the lime in the capillarity (strinking time is short)

If the striking time is longer (the concrete is already hardened) the capilary are narrow obtaining surfaces of dark shades (Fig.3).



Fig.3. The migration of the lime in the capillarity (strinking time is longer)

In the sunny periods, the lime is abundantly drained towards the surface because of the leakage loss from the capilary (Fig.4.) determining a surface of a light colour.

In the case of the forced drying due to the sun and wind, the front of the water evaporation remains retired from the surface (Fig.5) and the lime does not arrive at the surface and consequently, the surface is not bright, it has not a light colour.



Fig.4. The surface of the concrete after the sun and wind action



Fig..5. The forced action of the sun and wind of the surface of the concrete

In the case of a very wet atmosphere, the phenomenon of evaporation stops (Fig.6) and the capilary further close at the same time with the hardening of the concrete. The obtained surface has a dark colour, without brilliancy.



Fig.6. The surface of the concrete in the case of a very wet atmosphere

During the rains or because of the condense in the phase of green concrete the lime is on the surface and then it deposits, after drying and it carbonates in the air to form white traks (Fig.7).



Fig.7. The surface of the green concrete after the rain

5. Treatments applied to concrete

After some treatments applied to concrete, its appearant face can be modified so that not to be like the provions one, any more. These trreatments refer to the washing, acid treating, grinding, sand stone granulating and cutting.

The seen face has a different aspect according to the type of treating.

Concrete cleaned on the surface. In the casing and on the free face of the concrete, these is applied a product with a delay effect (Fig.8A). After striking the nonhydrated film is removed from the surface (Fig.C) by means of a powerful water jet. Depending on the intensity of the water jet, there will occur sand grains (light washing – Fig.D) and aggregate grains (energical, powerful washing – Fig.E) on the surface of the concrete.



Fig.8. Concrete cleated on the surface

Acid treated concrete – the concrete elements are soaked in an acid bath which attacks the whole surface or they are locally pulverized with an acid layer (acid pickling) (Fig.9)

The pickling (attack of the acid) doesn't take place deeply because of the salt deposit of reaction on the concrete surface. By washing, this deposit is easily removed. The pickling is limited to the occurance of the sand grains.

If the sand is of a silicious nature, the seen face is the same as that in Fig.9.C, but if it is of calcareous nature it is like in Fig.9.D.



Fig.9. Acid treated concrete

Concrete that is like sandstone and marble

On a thickness of 1-2 mm, the surface of the hardened concrete is subjected to a coarse buffing and then to a finer one, in order to remove all the scratches. There is obtained a concrete similar with the sandstone (Fig.10.A), trerefore, it is sometimes called "grease".



Fig.10 Concrete that is like sandstone and marble

In order to obturate the holes or the cracks an operation of cementation is made (Fig.10.B). after

the hardening of this completion layer, the concrete will be subjected to one or two ever finer buffings obtaining a polished concrete (Fig.10.C). If the buffing is made with even finer grains the concrete obtained has a satinet even brilliant aspect similar with the marble. The brillianey is due to the polished grains; in the rest, the surface has an "opaque" aspect. By resin treating an artificial brilliancy and its durability can be obtained, even in the case of the grains sensible to the acid rains.

The sand concrete trated by an abrasive pulverization

The surface on the concrete is worn out (through abrasion) by iron filings projected by a jet of compressed air. The method permits an instantaneous control of the obtained effect. According to the time of operation and the harduess of the filing the sand grains or the bigger ones (Fig.11.B) will be visible (Fig.11.A). The surface will always be opaque.



Fig.11. The sand concrete treated by an abrasive pulverization

Sandblasted concrete subjected to some tratment with special equipment

These treatments are, in general, ocasional using pneumatic hammers. By stone granulating (Fig.12) the surface of the concrete is crushed by the teeth of a pneumatic "chisel" with its head having the form of a honey comb. With the help of a Picamer the surface is destroyed. By cutting with a diamond cutter the surface of the concrete breaks in any form and at any depth.

All these tratments represent solutions of finishing the concrete surface using semi-automatic means (manual pneumatic tools) or 100% automatic means.

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Fig.12. Sandblasted concrete subjected to some tratement with special equipment

6. Protection of the decorative concrete against dirtiness

The geometry, porosity and the environmental factors play an important part in maintaining the façade clean.

The horizontal or slightly sloped surfaces have tendency to accumulate the dust from the atmosphere. Soon, under the effect of the water and wind, these zones will become ideal environments for the development of the microorganisms (Fig.13).





A high porosity of the face which maintains the humidity like a glue has the same effect.

Figura.14.a presents the effects and results caused by rain over a very porous surface and with a reduced enough slope.



Fig.14. The surface with a high porosity



Figura 14.b slows the way of getting dirty the same façade but less porous. The dirty zone is lower thus the façade is less affected.

Fig.15. The surface with a low porosity

The following measures of protection against the dirtiness of the facades are indicated:

- the removal of the water which falls over the horizontal or slightly sloped surfaces either towards the exterior (Fig.15.a) by constructing a larmier or towards the interior (Fig.15b) through a treshold and a coating of the wall with an adverse slope to evacuate the water;
- the resin printing in order to reduce the possibility of depositing the dirtiness.

7.Conclusions

For million years mankind has made natural cements called "conglomerates". The sand and gravel grains carried by the sea existant everywhere have covered the surface of the Earth, progressively. The water, which crossed the grains, carried away the fine mineral particles and the soluble binders most times siliceous. This mortar has comprised and covered the grain network and, in the process of hardening it changed into a more ar less solid and regulated monolith mass.

The tectonic movements broke up these tough layers in elements of different types: small (like the piece in Fig.16.a found in a mountain torent from Austria) and very big like the block and plates from the natural pits (Fig.16.b). The blocks represented the raw material used by people of neolotic for dwelling buildings. These monuments have resisted all the bad weather for over 4000 years and they are still standing. But what happened with the concrete made by man ever since the first centuries, before the Romans? The cement was used as a binder and the mortar was abundantly, being made so that the mixture be very plastic (quasi-liquid) in order to fill in all the voids between in the grains. This is the explanation for the attached images (Fig.17a and Fig.17.b) in comparison with the natural stones, richer in coarse grains.

The homogeneity of the concrete (quality indicator) is the result of this composition, very rich in mortar. The concrete made by man is, for sure, more regular and, at the same time, more resistant than that produced by the nature. Will it be durable, too ? The Roman concrete represents the proof and the favourable answer to this question.



Fig.16. The block from the natural pits



Fig.17. The concrete made by man

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Utilization of Geosynthetic Materials in Port and Coastal Works

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Abstract: Geosynthetic materials are more and more utilised within the port and coastal works too. Depending on their utilisation, these materials are intended to filter, separate, drain, strengthen, etc. Hence, the geosynthetic materials have been included in various works as:

- execution of the berm of Constantza Port breakwater;

- reinforcement of the crushed stone bed of the container storage platform, in the Constantza Free Zone;

- consolidation of the foundation ground of the 4000 t silos located on the bank of the Danube-Black Sea canal;

- temporary beach protection in the area of the PETROMAR Villas, in Eforie Sud.

Keywords: Geogrid, geosynthetics, geotextile, breakwater, berm, beach

1. Introduction

Generally, the port works are built on lands reclaimed from sea by fillings mainly executed under sea level.

The consolidation type of such materials is of the order of decades.

Generally, the coastal works are carried out for beach and cliff protection against erosion generated by waves and streams.

The foundation type of port constructions depends upon the nature of such works, the foundation ground characteristics, the loads transmitted to the foundation ground by the buildings, the permissible settlements from the technological point of view, etc.

Hence, we can find port structures that are indirectly founded by means of piles, columns, *Kelly* type walls, as well as port structures that are directly founded on the foundation ground or on a rockfill bed. Selecting the foundation type is a very important task, as it has an influence on construction's time behaviour, and on works' cost too.

Indirect foundation is a reliable solution, without settlements in time, but it is very expensive too. Direct foundation implies the acceptance of some settling movement that would not disturb the operation. This less expensive solution requires a much shorter completion time.

The advent of geosynthetic materials and their utilisation in construction works, including the port and coastal ones, support the adoption of the direct foundation system.

Under these circumstances, geosynthetic materials have been utilised within some construction works in Constantza Port: breakwater berm in Constantza South Port, container storage platforms in the Free Zone, foundations of the Grain Terminal located on the left bank of the Danube-Black Sea Canal (Fig. 1).



Fig. 1. General layout of Constantza Port

2. Breakwater in Constantza South Port

2.1. Breakwater structure

The breakvater of Constantza South Port are port hydraulic constructions of the gravitational type, with slopes, and they consist of a quarry-run rough stone core, protected by natural blocks coats and an armour of precast concrete elements of the "stabilopod" type.

At its upper side, the section is protected by a concrete slab that also plays the role of traffic road, especially for works' maintenance, bat also for vehicles access on the port territory.

The bottom side of the stabilopodes armour is supported on a berm of hollow gravity blocks mounted on a rockfill mattress (Fig. 2).

2.2. Geotechnical conditions on the site

The foundation ground on breakwater location is the sea bottom. It consists of a layer of plastically soft blackish mud, grey sands or brown silts, that are plastically consistent to soft, followed by a layer of red-brown clays resulted from limestone's alteration (bedrock), which is $1.0 \div 4.5$ m thick.

The bedrock consists of white Sarmatian limestone with a relief going down from -6.00 m (at the Southern dam bottom) to approx. -40.0 m (toward the waterway mouth – dams' ends).

2.3. Foundation solution for the berm

Considering the de characteristics of the foundation ground, a 2.0 m thick rough stone mattress placed on a geotextile support was provided for mounting the hollow gravity blocks.

The geotextile replaces the filtering layer with a thickness of approx. 1.0 m that would have been necessary, in order to prevent the rough stone sinking into the mud layer.

2.4. Time behaviour

During the site inspections and the measurements that were performed, there was not noticed any sliding of the stabilopodes armour, which proves that the berm has not suffered any settling movements, and therefore the geotextile has fulfilled its task.

2.5. Economical effects

Geotextile's utilisation instead of a stone and gravel filter leads to a sensitive shortening of the completion time and to a reduction of work's costs.





Fig. 2. Breakwater in Constantza Port - Cross section

3. Containers storage platform in Constantza Free Zone

3.1. Structure of the storage platform

The containers storage platform consists of the platform itself, dedicated to the storage of full, empty and refrigerated containers, a repair workshop for containers and handling plants, warehouses for containerised goods' control, railway connection, social-administrative building, lighting installations, utility networks, etc.

The total surface of the platform is approx. 27000 m^2 , allowing a traffic of approx. 30 000 TEU/year (Fig. 3).

The storage platform consists of a ballast and crushed stone foundation reinforced with geogrids and a pavement of small, interlocking concrete slabs 12 cm thick, mounted on a 10 cm thick sand layer.

3.2. Geotechnical conditions on the site

The site where the container storage platform is provided is a land reclaimed from sea by means of fillings with material resulted from the excavations for the Danube – Black Sea Canal. The filling layer thickness is $10.0\div11.0$ m and it is approx. 17 years old.

In order to determine the foundation ground characteristics, there have been performed geotechnical drillings and laboratory tests.

The filling material mainly consists of reddish clays with inter-layers of brown, green, grey, brickred clay, with degraded limestone fragments having sizes varying from $1\div 2$ cm to $15\div 20$ cm and sometimes more than that.

Under the filling there was found a blackish



Fig. 3. General layout of Container Terminal in Constantza Port - Free Zone

clayey mud layer, having a thickness of 20÷30 cm.

Under the mud layer there is a limestone base consisting of Sarmatian limestone, whose surface is degraded on a depth of $30\div50$ cm.

The upper part of the filling layer, on a thickness of approx. 1,0 m is cracked and loose, due to the frost – thaw phenomenon.

The permissible pressure of the foundation ground, at the lower level of platform's foundation, is of 100 KPa.

3.3. Foundation solutions

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The foundation solutions for the platform and its related buildings have been selected depending on the geotechnical characteristics of the foundation ground, the operating loads and the permissible settlements.

Hence, for all the buildings and the platform, direct foundation has been selected.

The buildings are founded on a crushed stone bed. At buildings with rigid structures, (social administrative group) measures for improving the foundation ground characteristics, by dynamic compaction, have also been taken.

Platform's foundation will consist of a ballast layer – 40 cm thick after compaction, placed on a geotextile layer of the SECUTEX 251 GRK4 type and a geogrid layer of the SECUGRID 40/40 Q6 type.

Another SECUGRID 40/40 Q6 geogrid is provided at the upper side of the ballast layer. It will be covered by a crushed stone layer, which will have a thickness of 40 cm after compaction. The total thickness of platform's foundation will be 80 cm (Fig. 4).



Fig.4. Container storage platform-Cross section

The technical characteristics of the SECUGRID 40/40 Q6 geogrid are as follows:

- Longitudinal breaking strength 40 KN/m
- Transversal breaking strength 40 KN/m
- Longitudinal breaking elongation $\leq 8 \%$
- Transversal breaking elongation ≤ 8 %
- 2% elongation strength \geq 15 KN/m

- 5% elongation strength	\geq 50 KN/m
- Mesh sizes	32 x 32 mm

For utilising the gosynthetic material, a separation of the ballast layer form the natural ground is obtained, preventing this way the soil grains migration within the ballast layer.

The reinforcement by two geogrid nets leads to a reduction of the general settlements, by reducing the pressures upon the foundation ground and, most of all, it leads to a reduction of the differential settlements generated by the uneven loads on the platform.

Eliminating the uneven settlements and reducing the general ones, results in an improvement of the container-handling plants' behaviour during operation.

4. Grain terminal on the bank of the Danube – Balck Sea Canal, in Agigea

4.1. Terminal's structure

The terminal has a simultaneous storage capacity of 15 000 t and it includes:

- road and railway receiving system;
- storage cells;
- water and road shipping system;
- conveyor system for commodities transfer, etc. (Fig. 5).

The 4 storage cells have a diameter of 18.39 m and a height of 22.0 m.

4.2. Geotechnical conditions on the site

The storage cells are located at approx. 120 m from the Northern bank of the Danube-Black Sea Canal, on a land reclaimed from sea, by filling the former Agigea lake with material resulted from canal's excavation (Fig. 6).

The geotechnical study prepared for assessing the foundation ground characteristics has revealed the presence of an approx. 6.0 m thick layer consisting of remains of construction materials embedded in a mass of reddish silty clay, yellow and grey clays with carbonates, gravel and limestone boulders in variable proportions.

These materials with medium-high plasticity, high humidity up to saturation and high porosity are compressible, and the consolidation process is not completed yet.

Filling's compressibility characteristics are somehow acceptable for low vertical pressures. Hence,





Fig. 5. Grain Terminal - General layout



Fig. 6. Grain Terminal - Cross section

The other layers consist of clays from 6.0 m to 12.0 m, clayey silts from 12.0 m to 20.0 m and silty clays from 20.0 m to 28.0 m, these materials having high plasticity.

As foundation ground, these soils are considered to be "liable to deformation", both under the future constructions' weight and under their own weight.

4.3. Foundation solution

Considering the details given in paragraph 4.2., two alternatives have been analysed as regards cells' foundation:

- indirect foundation by means of drilled piles, embedded in the limestone layer. This alternative avoids settlements, but the costs are very high;

- direct foundation; taking into account the process of consolidation in time of the filling and the ground beneath it, such solution could generate settlements under the construction. In case certain reasonable settlements are accepted from the technological point of view, this solution is less expensive and easier to achieve.

The solution of direct foundation was employed for founding the silo cells.

They are founded on a reinforced concrete foot-ring, with an overall height of 2.37 m. The cells are shaped as a bottomless cylinder, the grains transmitting most of their weight directly to the ground inside the ring, and to the rest of the foundation by friction against the wall.

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The results of calculations have revealed settlements up to 20 cm, in a full silo hypothesis.

In order to achieve the storage cells, the following works have been carried out:

- removal of the top layer and of the inappropriate soil pockets and their replacement with ballast;

- execution, over the ballast layer, of a 1.35 m thick crushed stone bed reinforced with geogrids. There were used SS 30 geogrids summing 3400 m², with the following characteristics:

- Longitudinal breaking strength 30 KN/m
- Transversal breaking strength 30 KN/m
- 2% elongation strength 10.5 KN/m
- 4% elongation strength 21 KN/m

- Weight

 0.3 Kg/m^2

The crushed stone bed was reinforced by means of three geogrid layers, at intervals of 25 cm. The crushed stone sized of $15\div25$ cm that was placed between these geogrid layers, together with meshes' size, of 30 cm, assured a better wedging and coworking of the three layers and the cutting of the geogrid net by the large stones was avoided. A minimal thickness of 2.5 m resulted for the foundation of non-cohesive material, which takes over the stresses transmitted by cells and assures a sensitive reduction of the general and relative settlements between cells (Fig. 7).

The purpose of geogrids' utilisation was to reduce the general settlements, but most of all, the differential settlements of the foundation ground generated by its uneven loading.

Geogrids' utilisation has increased with about 25% the capacity of foundations to take over the stresses, and has sensitively improved constructions' operating behaviour.



Fig.7. Silo foundation - Cross section

4.4. Monitoring the time behaviour

For observing the time behaviour of the four cells, measurements have been performed on their foundations' settling movement.

The settlements recorded after the first 4 months since commissioning, had values between 150 and 180 mm (except for cell no. 4, where the settlements were of only 50 mm).

After another year, there was found that settlements had increased with max. 20 mm, which proves that they practically died out (Fig. 8).



Fig.8. Settlements monitoring



Fig.8. Settlements monitoring

5. Temporary beach protection in the area of the Petromar villas, in Eforie Sud

5.1 Situation on site

The PETROMAR villas are located on the littoral belt between Eforie Nord and Eforie Sud. Under waves action, the beach in front of the villas was eroded, especially towards the Southern limit on the area where the villas were built-up. The beach erosion has maximal values in front of the PETROMAR Villas enclosure, bringing into danger these buildings, a part of the fencing being already destroyed on the Southern side.

Relying on recent surveys and observations, there could be assessed that the entire littoral belt in front of Techirghiol Lake, between the Eforie Sud Preventorium and Belona Lake, is undergoing a permanent erosion process.

The erosion rate of advance give an alarming perspective, bringing into danger the safety of the buildings located on the littoral belt, not only the PETROMAR Villa, but other objectives too: hotels, children camps, balneary resorts, rest homes, the tourism and the hydropathic activities in that region and even the national road DN 39 and the railway between Constantza and Mangalia.

This erosion phenomenon has become more active during the last 10 years, when the natural sand input is very low, the main supply coming from South.

5.2 Protection solution foreseen

The analysis performed for beach protection purposes covered the beach portion between the Southern limit of the PETROMAR Villas enclosure and the tourism port of Eforie Nord.

The beach protection works can not start very soon, due to problems of authorisation, financing and detailed design, and to the relatively high cost of investment. Therefore, before the start of the stormy season, in the most affected area South of the PETROMAR Villas, a local protection solution was executed, in order to prevent the damaging of the existing buildings, until the final works are completed.

In principle, the temporary protection solution consists of geotextile bags filled with sand and placed in a trench along the enclosure.

On the trench bottom and up to the limit of of the PETROMAR Villas enclosure, an un-woven geotextile of the Ha Te E650 type was placed, acting as a separator and a filter, whose sizes are (12.3×17.3) m.

The technical characteristics of the un-woven geotextile of the Ha Te E 650 type are:

- material utilised - polypropylene (PP)

- weight $[g/m^2]$ 650
- material thickness [mm] 6
- Tensile strength [KN/m]:

- longitudinal	- 13	
- transversal	- 22	
- elongation [%]		
- longitudinal	- 110	
- transversal	- 90	

- punching strength [N] ->1200

- permeability – K coefficient $[m/s] - 6.8 \times 10^{-3}$

- material strength (crushed stone test) [kg]:

- when the stone falls from 1 m, a stone weight of 120 kg should not produce material's penetration;

- when the stone falls from 2 m, a stone weight of 60 kg should not produce material's penetration.

In order to create a continuous mattress, it was sewn with a special sewing machine, using a polystyrene fibre.

Over this geotextil, a mattress consisting of geotextile bags was placed. Bags' manufacturing sizes were of (1.30×2.90) m. The bags were filled with sand brought from the Danube, and after being filled, their thickness was of 0.30 m, and each bag weighed 1,500 kg.

In order to protect the enclosure of the PETROMAR Villas, a guard wall was built in the area where the fence was destroyed by storms. This wall

also consists of geotextile bags filled with sand, placed on top of the mattress, up to level +3.25 m.

The bags are made from B 400 K 5 geotextile, initially sewn on three sides. The bags were filled up to 80% of their capacity, which gives them the ability to mould on the contact surface and one against the other. The bags have an edge of 4 cm all around, so that they can be sewn one to the other on the site.

These bags were interconnected by sewing on all directions with polystyrene fibres, using a special sewing machine. This way, the connected bags formed a mattress that would prevent the washing of the sand in this exposed area.

The technical characteristics of the geotextile bags of the Ha Te B 400 K 5 type are as follows:

- 3.36

- . material utilised polypropylene (PP)
- weight [kg/bag]

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- material weight [g/m³] - 400

- material thickness under a load of 2 kPa [mm]- 4

- tensile strength [KN/m]:

 longitudinal 	- 17
- transversal	- 35
elongation [%]	
- longitudinal	- 90
- transversal	- 60

- punching strength [N] - 3600

- permeability –K coefficient under a load of 2 kPa $[m/s] - 3 \times 10^{-3}$

In order to protect the bags placed in the traffic area, they have been covered with sand resulted from excavations, the initial profile of the beach being recovered (Fig. 9).

The adopted solution is an elastic solution, which takes over the deformations without endangering the arrangement itself and which can be easily removed too.



Fig. 9. Cross section - Geosintetic breakwater solution

Reconstruction Of Reinforced Concrete Floors With Modern Techniques. Analysis Of The Compatibility Between Epoxy Surfaces And Reinforced Concrete Pavements

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Abstract: This paper presents the behaviour of reinforced concrete pavements situated in an aircrafts hangar. The pavement, built in 1974, have in present much degradations. A solution recommended for repair the pavement is the system Technifloor with epoxy surfaces. The behaviour was modelated with FEM.

Keywords:

Reinforced concrete pavement, epoxy surfaces, system Technifloor, FEM.

1. Introduction

TECHNIFLOOR QUARTZ COLOR FIN is a widely used floor covering, wich is recommended for:

- electronic, micro-electronic, hichtech, mechanical and automobile industries
- pharmaceutical and chemical industries , bio-chemical laboratories
- food production, tobacco industry, dairy farm, slaughter houses and kitchens
- malls and supermarkets
- public areas:including hospitals, schools, exhibition halls, stations and airports

Characteristics

- its mechanical resistance is well over that of a reinforced concrete
- excelent resistance to abrasion
- it is extremely resistant to chemical aggresion thanks to its transparent top coat
- it is impermeable to water, but designed to enable water moisture from the sub-base to evaporate
- the coating is laid over large areas without joins, which improves security and hygiene. However,

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structural joins of the building must be treated appropriately.

non-slip surface

TECHNIFLOOR QUARTZ COLOR FIN it is extremely decorative which makes it an ideal choice for places frequented by the public. Ist is available in a wide range of colors.

TECHNIFLOOR QUARTZ COLOR FIN contains solvent free epoxy resin, hardener, coloured and natural quartz sand of various sizes.

The technical characteristics for Technifloor system are:

- Compressive strenght 80N/mm2
- Flexial strenght 27N/mm²
- Bond strenght > 3 N/mm^2
- Elasticity module (compression test) 15000N/mm²

All this properties of the TECHNIFLOOR make possible to use the system to repair the floor in a hangar of Otopeni Airport .Because the reinforced floor was made in 1974, it is necessary to analyse the behaviour of each plot loaded in different ways.

2. Theoretical aspects

If do not exists analytic solutions for problem of efforts and strains distribution, specially as a result of complicate boundary conditions, it is applied the analysis with finite elements.Without details, in this method it is purpose that any continuum for that exist compatibility between efforts state and deformations state can be discompose in a finite number of elements © 2000 Ovidius University Press
with little dimensions which state efforts could be known easily. Each element is analysed respecting compatibility conditions and the continuum structure it is remade by join the elements.

Deformations and displacements of each plate was made with calculus program FEAT 2000. The bondary conditions for analysed plate were deffined like program's manual It was deffined in this conditions, a foundation with following rigidities: $C_{1=}40MPa/m$ and $C_{2=}12Mpa \cdot m$. The results are presented for calculus group where were introduced all types of loadings. The calculus is nonlinear and use Newton-Rapfson method, with five incrementation steps.

With this research and evaluation method was realised the modeling of concrete floor behaviour under a continuous epoxidic surface.

3. Information about structure

The calculus was made for a reinforced concrete plate with following dimensions and characteristics of the materials:

- thickness-34cm
- width-3,5m

- lenght-12m
- concrete-B_c 22,5
- elasticity modulus E=27000Mpa
- Poisson coefficient μ =0,15
- Specific weight for concrete $\gamma = 2.5t/m^3$
- Thermic expansion coefficient 10⁻⁵

4. Calculus ipothesis

The structure was calculated using following ipothesis:

1.loadings coming from proper weight:-uniform distributed force on all plate surface $q=8.5 \text{ kN/m}^3$,

2.
loading coming from Boeing 747 landing train, with weight by wheel P=183,25 kN,

3.temperature variation $\Delta t=10^{\circ}$ C,

The most important loading coming from proper weight and transmited by landing train to the plate it is applied in following points:

- in center of the plate
- in one corner of the plate
- in two points situated at 11m distance on longitudinal axis of the plate



Fig. 1. Landing train scheme

5.Modeling results

First case of loading: the force is applied in center of the plate



Fig. 2. Deformed shape and finite element mesh







Fig. 5. Contact stresses

All diagrams show following conclusions:

- the displacement take values less 1mm
- the maximum values of the displacements are registered on thelimits, at the corners and at the middle of big side.
- The stresse in concrete pavement are less that concrete resistance

- At contact surfaces before plate and ballast foundation the values of the stresses are maximum in the same points where the displacements have maximum values on Z axis.

Second case of loading

The force is applied on the corner of the plate



Fig. 6. Deformed shape and finite element mesh





Fig. 9. Contact strasses

All diagrams show following conclusions: -the displacement take values less 4,2 mm.The behaviour modelinganalyse the extreme cases when the application point of the force obtained from effective loadings is on the corner of the plate. Practically, this force are transmited from a wheel mark with 1264,416 cm² surface.That means that the force are distributed on the corners of four neighbouring plates and the real displacement will have a value near 1 mm for each plate. - The stresse in concrete pavement are less that concrete resistance

- At contact surfaces before plate and ballast foundation the values of the stresses are maximum in the same points where the displacements have maximum values on Z axis.

The third case of loading: the force is applied in two points situated at 11m distance on longitudinal axis of the plate.



Fig. 11. Z axis displacement



All diagrams show following conclusions: -the displacement take values less 1,7mm

- the maximum values of the displacements are registered on thelimits, at the corners and at the middle of big side.

- The stresse in concrete pavement are less that concrete resistance
- At contact surfaces before plate and ballast foundation the values of the stresses are maximum in the same points where the displacements have maximum values on Z axis.

6. Conclusions

Based on presented calculus, it is observed that:

- the ansemble structure-foundation soil work good under all loadings and is not necessary to redimenssion it.
- The effects of desalkaline phenomenom observed on 10-20 mm depth in the concrete, must be stoped for not affect, in time, the mechanical behaviour of the plates.

Is proposed following solutions to repair the reinforced concrete floor:

-milling the concrete

-moulding a C20/25 mark concrete layer with 1,5-2,0 cm width

- assembling metalic joins Bameco type
- assembling circulating grills
- moulding an epoxidic layer with 2-3 mm widdth, on all the concrete surface and polish it
- applyng TECHNIFLOOR QUARTZ COLOR FIN system
- it is not reccomended to reinforced the dilatation joins with metal bars OB ϕ 14 because it change the kind of efforts transmission from the plate and the behaviour of the interaction plate-soil foundation.

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The Non-Linear Galloping Response of a One-dimensional Structure

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Abstract A non-linear two-degree-of-freedom model is develop to describe and predict galloping behavior of a single iced, electrical transmission line. Interactions are accomodated between a line's plunge and swing in the along-wind direction. Equations of motion are presented in nondimensional form and contains neliniar terms contributed by the aerodynamic forces. Because the equations of motion are weakly non-linear, the Multiple-Time-Scales Method is employed so that the governing equations can be manipulated to obtain explicit expressions for the periodic and quasiperiodic solutions as well as their stability conditions.

Keywords: Galloping, non-linear equation, periodic solution.

1. Introduction

Galloping is a low - frequency, high-amplitude oscillation that can occur on an iced electrical transmission line in a steady-side wind. Oscillations of the galloping type are caused by the aerodynamic instability of the cross - section of the body so that the motion generates forces, which increases its amplitudes. The inability to prevent or control galloping can lead to severe disruptions in the electrical power supply.

In this paper, the linear coupled vertical-horizontal equations derived by Jones [1] are extended by introduction a set of nonlinear terms contributed by the aerodynamic forces.

2. Description of model and equations of motion

Assume a one-dimensional body moving with

horizontal and vertical velocities z and y, respectively, in a horizontal wind field $V_{\infty}\left(y, z \ll V_{\infty}\right)$. The dot denotes the derivative

with respect to time. The angle of attack α of the body to the relative wind V_{rel} is shown in figure 1. By convention, the angle of attack is measured clockwise from the wind direction when the wind is blowing from the left to right. In terms of the true

wind and the body velocities, the relative wind V_{rel} and angle of attack α is given by :

$$V_{rel} = \sqrt{\left(V_{\infty} - \dot{z}\right)^2 + \left(\dot{y}\right)^2}$$

$$\alpha = \arctan \frac{\dot{y}}{V_{\infty} - \dot{z}}$$
(1)

The equations of motion in the horizontal and vertical directions for a unit length element are

$$\ddot{mz} + c_{z} \dot{z} + k_{z} z = \frac{1}{2} \rho_{a} dV_{rel}^{2} [C_{D} \cos \alpha + C_{L} \sin \alpha]$$

$$\ddot{my} + c_{y} \dot{y} + k_{y} y = \frac{1}{2} \rho_{a} dV_{rel}^{2} [-C_{D} \sin \alpha + C_{L} \cos \alpha]$$

(2)

where *m* is mass per unit length of the body (including ice coating), c_z , c_y are the structural damping coefficients (on *z* and *y* directions, respectively), k_z , k_y are the stiffness coefficients (on unit length and *z* and *y* directions, respectively), ρ the air density, *d* a suitable reference length of cross section and C_D , C_L the dimensionless drag and lift coefficients.

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In deriving Eq. (2) it was considered that aerodynamic force \vec{F}_a is given by his components



Fig.1: Basic model for galloping

- drag force
$$F_D = C_D \cdot \frac{1}{2} \rho_a \, d \, l V_{rel}^2$$
 (3a)

- lift force
$$F_L = C_L \cdot \frac{1}{2} \rho_a d l V_{rel}^2$$
 (3b)

By using the Taylor series expansion about small quantities $\frac{y}{V_{\infty}}, \frac{z}{V_{\infty}}$ of relative wind and lift and drag coefficients and non-dimensional quantities

$$\widetilde{y} = \frac{y}{d}, \ \widetilde{z} = \frac{z}{d}, \ \tau = \omega_y t, \ U = \frac{V_{\infty}}{\omega_y d},$$

$$\eta = \frac{\rho_a d^2}{2m}, \ \xi_y = \frac{c_y}{2m\omega_y}, \ \xi_z = \frac{c_z}{2m\omega_z}$$
(4)

we obtain the following non-dimensional equations of motion :

$$\begin{split} &\frac{d^{2}\tilde{z}}{d\tau^{2}} + \tilde{z} = \tilde{A}_{11}\frac{d\tilde{z}}{d\tau} + \tilde{B}_{11}\frac{d\tilde{y}}{d\tau} + \tilde{A}_{21}\left(\frac{d\tilde{z}}{d\tau}\right)^{2} + \\ &+ \tilde{B}_{21}\left(\frac{d\tilde{y}}{d\tau}\right)^{2} + \tilde{C}_{21}\frac{d\tilde{z}}{d\tau}\frac{d\tilde{y}}{d\tau} + \tilde{A}_{31}\left(\frac{d\tilde{z}}{d\tau}\right)^{3} + \\ &+ \tilde{D}_{12}\left(\frac{d\tilde{z}}{d\tau}\right)^{2}\left(\frac{d\tilde{y}}{d\tau}\right) + \tilde{E}_{12}\left(\frac{d\tilde{z}}{d\tau}\right)\left(\frac{d\tilde{y}}{d\tau}\right)^{2} + \tilde{B}_{31}\left(\frac{d\tilde{y}}{d\tau}\right)^{3} \\ &\frac{d^{2}\tilde{y}}{d\tau^{2}} + r^{2}\tilde{y} = \tilde{A}_{12}\frac{d\tilde{z}}{d\tau} + \tilde{B}_{12}\frac{d\tilde{y}}{d\tau} + \tilde{A}_{22}\left(\frac{d\tilde{z}}{d\tau}\right)^{2} + \\ &+ \tilde{B}_{22}\left(\frac{d\tilde{y}}{d\tau}\right)^{2} + \tilde{C}_{21}\frac{d\tilde{z}}{d\tau}\frac{d\tilde{y}}{d\tau} + \tilde{A}_{32}\left(\frac{d\tilde{z}}{d\tau}\right)^{3} + \\ &+ \tilde{D}_{21}\left(\frac{d\tilde{z}}{d\tau}\right)^{2}\left(\frac{d\tilde{y}}{d\tau}\right) + \tilde{E}_{21}\left(\frac{d\tilde{z}}{d\tau}\right)\left(\frac{d\tilde{y}}{d\tau}\right)^{2} + \tilde{B}_{32}\left(\frac{d\tilde{y}}{d\tau}\right)^{3} \end{split}$$

Non-dimensional coefficients used in Eq. (5) are given in Appendix.

The first step in determining the feasibility of galloping is to investigate whether the initial equilibrium solution of the linear form of Eq. (5) is unstable. If this is happened, the next step is to investigate possible new solutions bifurcating from the equilibrium solution and to determine their stability. Such of solutions can be computed by employing numerical time integration or by using complementary techniques such as time averaging, intrinsic harmonic balancing etc [2].

New bifurcation equations governing a motion can then be derived in terms of the amplitudes and phases of the motion.

3. Determining the periodic and quasi-periodic solutions by means of the Multiple-Time-Scales Method

Suppose that solution $\tilde{y} = \tilde{z} = 0$ becomes unstable when wind speed is varied such that one of the critical points is encountered. New equilibrium states or dynamic motions may emerge from the critical point. We are investigated only the periodic or quasi-periodic motions.

The autonomous system (5) is weakly nonlinear so we are used the Multiple-Time-Scales Method [3] for derive the bifurcation equations governing the motion. Two time scales are used, namely the normal time $t_1 = \tau$ and slow time $t_2 = \varepsilon \tau$, so that the obtained solutions are valid at time scale $\tau = O(\varepsilon^2)$.

 $\varepsilon \ll 1$ is the ratio between the damping and aerodynamic forces, on a side, and the stiffness and inertia forces, on the other side.

Letting

$$\tilde{A}_{ij} = \varepsilon \hat{A}_{ij}, \tilde{B}_{ij} = \varepsilon \hat{B}_{ij}, \tilde{C}_{ij} = \varepsilon \hat{C}_{ij}, \tilde{D}_{ij} = \varepsilon \hat{D}_{ij}, \tilde{E}_{ij} = \varepsilon \hat{E}_{ij}$$
(6)

and choosing the following solutions for system (5)

$$\widetilde{z}(t_{1},t_{2},\varepsilon) = Z_{0}(t_{1},t_{2}) + \varepsilon Z_{1}(t_{1},t_{2}) + \dots
\widetilde{y}(t_{1},t_{2},\varepsilon) = Y_{0}(t_{1},t_{2}) + \varepsilon Y_{1}(t_{1},t_{2}) + \dots$$
(7)

by equating coefficients of the same powers of $\boldsymbol{\mathcal{E}}$, we obtain :

$$\varepsilon^{0} : \frac{\partial^{2} Z_{0}}{\partial t_{1}^{2}} + Z_{0} = 0 \quad , \quad \frac{\partial^{2} Y_{0}}{\partial t_{1}^{2}} + r^{2} Y_{1} = 0$$
 (8)

$$\begin{split} \varepsilon^{1} &: \frac{\partial^{2} Z_{1}}{\partial t_{1}^{2}} + Z_{1} = -2 \frac{\partial^{2} Z_{0}}{\partial t_{1} \partial t_{2}} + \hat{A}_{11} \frac{\partial Z_{0}}{\partial t_{1}} + \hat{B}_{11} \frac{\partial Y_{0}}{\partial t_{1}} + \\ &+ \hat{A}_{21} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right)^{2} + \hat{B}_{21} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{2} + \hat{C}_{12} \frac{\partial Z_{0}}{\partial t_{1}} \frac{\partial Y_{0}}{\partial t_{1}} + \hat{A}_{31} \left(\frac{\partial Z_{0}}{\partial t} \right)^{3} + \\ &+ \hat{D}_{12} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right)^{2} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right) + \hat{E}_{12} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right) \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{2} + \hat{B}_{31} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{3} \\ &\frac{\partial^{2} Y_{1}}{\partial t_{1}^{2}} + r^{2} Y_{1} = -2 \frac{\partial^{2} Y_{0}}{\partial t_{1} \partial t_{2}} + \hat{A}_{12} \frac{\partial Z_{0}}{\partial t_{1}} + \hat{B}_{12} \frac{\partial Y_{0}}{\partial t_{1}} + \\ &+ \hat{A}_{22} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right)^{2} + \hat{B}_{22} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{2} + \hat{C}_{21} \frac{\partial Z_{0}}{\partial t_{1}} \frac{\partial Y_{0}}{\partial t_{1}} + \hat{A}_{32} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right)^{3} + \\ &+ \hat{D}_{21} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right)^{2} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right) + \hat{E}_{21} \left(\frac{\partial Z_{0}}{\partial t_{1}} \right) \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{2} + \hat{B}_{32} \left(\frac{\partial Y_{0}}{\partial t_{1}} \right)^{3} \end{split}$$

and so on.

The equations (8) give :

$$Z_{0}(t_{1},t_{2}) = a_{z}(t_{2})\exp(it_{1}) + \overline{a}_{z}(t_{2})\exp(-it_{1})$$

$$Y_{0}(t_{1},t_{2}) = a_{y}(t_{2})\exp(it_{1}) + \overline{a}_{y}(t_{2})\exp(-it_{1})$$
(10)

where a_z, a_y are complex functions and over bar denotes the complex conjugates.

Galloping may be classified as either internal non-resonant or internal resonant depending upon the ratios of the natural frequencies ω_y and ω_z .

Non-resonant case
$$\left(r = \frac{\omega_y}{\omega_z} \neq \frac{m}{n}, m, n \in N^*\right)$$

By using the solutions (10) and Eq. (9), eliminating the secular terms and introducing the polar forms

$$a_{z}(t_{2}) = \frac{1}{2} \rho_{z}(t_{2}) \exp(i \phi_{z}(t_{2}))$$

$$a_{y}(t_{2}) = \frac{1}{2} \rho_{y}(t_{2}) \exp(i \phi_{y}(t_{2}))$$
(11)

on obtains the perturbation equations

$$\rho'_{z} = \rho_{z} \cdot \left(\frac{\hat{A}_{11}}{2} + \frac{3\hat{A}_{31}}{8} \rho_{z}^{2} + \frac{\hat{E}_{12} r^{2}}{8} \rho_{y}^{2} \right)$$

$$\rho'_{y} = \rho_{y} \cdot \left(\frac{\hat{B}_{12}}{2} + \frac{\hat{D}_{21}}{4} \rho_{z}^{2} + \frac{3\hat{B}_{32} r^{2}}{8} \rho_{y}^{2} \right)$$

$$\phi'_{z} = \phi'_{y} = 0$$
(12)
(12)
(13)

where the prime denotes derivative with respect to time t_2 .

The Eq. (13) show us that the frequencies of motions in the horizontal and vertical directions remain unmodified. The steady state amplitudes $\overline{\rho}_z$ and $\overline{\rho}_y$ and the stability of the steady state solutions may be determined from equations (12). Using (7), (10) and (11), it is possible to go back to the normal non-dimensional time τ by combining them and to determine the first order approximation for the solution of equations of motion

$$\widetilde{z}(\tau) = \rho_{z}(\tau)\cos(\tau + \varphi_{0}), \widetilde{y}(\tau) = \rho_{y}(\tau)\cos(\tau + \psi_{0})$$
(14)

where φ_0 and ψ_0 are constants (they result from the initial conditions of motions).

Resonant case $r = \frac{\omega_y}{\omega_z} = 1$

Proceeding like in non- resonant case and noting $\delta = \phi_y - \phi_z$ on obtain the following set of governing equations from the initial equilibrium solution

$$\rho'_{z} = \frac{\hat{A}_{11}}{2} \rho_{z} + \frac{\hat{B}_{11}}{2} \rho_{y} \cos \delta + \\ + \left[\frac{3\hat{A}_{31}}{8} \rho_{z}^{2} + \frac{\hat{E}_{12}}{8} \rho_{y}^{2} (\cos 2\delta + 2) \right] \rho_{z} + \\ + \left(\frac{3\hat{D}_{12}}{8} \rho_{z}^{2} \cos \delta + \frac{3\hat{B}_{31}}{8} \rho_{y}^{2} \cos \delta \right) \rho_{y}$$

$$\rho_{y}^{'} = \frac{\hat{A}_{12}}{2} \rho_{z} \cos \delta + \frac{\hat{B}_{12}}{2} \rho_{y} + \left[\frac{3\hat{A}_{32}}{8} \rho_{z}^{2} \cos \delta + \frac{3\hat{E}_{21}}{8} \rho_{y}^{2} \right] \rho_{z} + \left[\frac{\hat{D}_{21}}{8} \rho_{z}^{2} (\cos 2\delta + 2) + \frac{3\hat{B}_{32}}{8} \rho_{y}^{2} \right] \rho_{y}$$

$$\rho_{z} \rho_{y} \delta_{z}^{'} = -\left[\frac{\hat{A}_{12}}{2} \rho_{z}^{2} + \frac{\hat{B}_{11}}{2} \rho_{y}^{2} + \frac{\hat{D}_{12} + \hat{B}_{21}}{8} \rho_{y}^{2} \rho_{z}^{2} + \frac{3\hat{A}_{32}}{2} \rho_{z}^{4} + \frac{3\hat{B}_{31}}{2} \rho_{y}^{4} \right] \sin \delta - \left(\frac{\hat{D}_{21}}{8} \rho_{z}^{2} + \frac{\hat{E}_{12}}{8} \rho_{y}^{2} \right) \rho_{z} \rho_{y} \sin 2\delta$$

$$(15)$$

4. Conclusion

The paper presents a non-linear model with two-degree-of-freedom which can be used for study the galloping response of a one-dimensional structure like electrical transmission line. Interactions are accomodated between a line's plunge and swing in the along-wind direction. The differential equations of motion are derived in nondimensional form and contains non-linear contributed by the aerodynamic forces. They are weakly non-linear so we are used the Multiple-Time-Scales method for determine the governing equations which describe the periodic or quasiperiodic motions when the static equilibrium position became unstable.

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Appendix

$$\begin{split} & \text{Coefficients used in equations (5) are :} \\ & \widetilde{A}_{11} = -2 \Big(\eta U C_D^0 + \xi_y \Big), \hat{B}_{11} = -\eta U \Big(\frac{d C_D}{d \alpha} - C_L^0 \Big), \\ & \widetilde{A}_{21} = \frac{\eta}{2} C_D^0, \widetilde{B}_{21} = \frac{\eta}{2} \Big(\frac{d^2 C_D}{d \alpha^2} + C_D^0 - 2 \frac{d C_L}{d \alpha} \Big), \\ & \widetilde{C}_{12} = \eta \Big(\frac{d C_D}{d \alpha} - C_L^0 \Big), \\ & \widetilde{A}_{31} = -\frac{\eta}{2U} C_D^0, \\ & \widetilde{D}_{12} = \frac{\eta}{2U} \Big(C_L^0 - \frac{d C_D}{d \alpha} \Big), \\ & \widetilde{E}_{12} = -\eta U \Big(\frac{d C_L}{d \alpha} + C_D^0 \Big), \\ & \widetilde{A}_{32} = \frac{\eta}{2U} C_L^0, \\ & \widetilde{B}_{31} = -\frac{\eta}{6U} \Big(\frac{d^3 C_D}{d \alpha^3} + 3 \frac{d C_D}{d \alpha} - 3 \frac{d^2 C_L}{d \alpha^2} - 3 C_L^0 \Big), \\ & \widetilde{A}_{12} = -2 \Big(\eta U C_L^0 + \xi_z \Big) \\ & \widetilde{A}_{22} = \frac{\eta}{2} \Big(\frac{d^2 C_L}{d \alpha^2} + C_L^0 + 2 \frac{d C_D}{d \alpha} \Big), \\ & \widetilde{C}_{21} = \eta \Big(\frac{d^2 C_L}{d \alpha^2} + C_L^0 + 2 \frac{d C_D}{d \alpha} \Big), \\ & \widetilde{D}_{21} = -\frac{\eta}{2U} \Big(C_D^0 + \frac{d C_L}{d \alpha} \Big), \\ & \widetilde{E}_{21} = -\frac{\eta}{U} \Big(\frac{d^2 C_L}{d \alpha^2} + C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha} + 3 C_D^0 \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_D}{d \alpha^2} + 3 \frac{d C_L}{d \alpha^2} + 3 \frac{d C_L}{d \alpha^2} + 3 \frac{d C_L}{d \alpha^2} \Big), \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_L}{d \alpha^2} \Big) \\ & \widetilde{B}_{32} = -\frac{\eta}{6U} \Big(\frac{d^3 C_L}{d \alpha^3} + 3 \frac{d^2 C_L}{d \alpha^2} + 3 \frac{d^$$

All the derivatives are computed in $\alpha = 0$.

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Wavelets – Differential Equations Coupling Applications on Studying the Bending Stressed Elements

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Abstract: The paper presents the outcomes of using new methods for numeric calculus with wavelets functions in mathematical models for static or dynamic stressed elements. Examples by case studies are presented in order to determine the deformations and the eigenmodes got by classical methods or by serial developments with Haar functions. A brief theoretical presentation of Haar type wavelet functions is inserted at the beginning.

Keywords: wavelets, bending, displacement, eigenmodes.

1. Introduction

The research on bending stressed elements is being done by using differential equations models. The use of wavelets function [4] in numeric solving of that type of problems represents a new issue that is to enlightened by simple examples that reveal the possibility of using them at more difficulty level problems.

Let's consider the differential equation:

$$v^{\text{m}}(x) = \frac{1}{EI} f^{\text{m}}(x) \tag{1}$$

where f(x) represents the function of the bending moment through a current section of the domain or the load intensity reduced by the rigidity of the section, *EI*.

Imposing certain boundary restraints, an analytical expression of the deformed configuration can be determined by integration. Hence, the rotations or efforts can be determined and then the rigidity and strength will be found.

Eigenmodes of vibration for variable loading in time can also be determined.

The analytical expression of the middle fiber configuration [3] for the mathematical model (1) represented trough physical model from the picture no. (1) obtained by direct integration is:

$$v(x) = \frac{p(x)l^2}{4EI}x^2 - \frac{p(x)l}{6EI}x^3 + \frac{p(x)}{24EI}x^4$$

Finite or boundary element metods can be used as classical numerical solving methods.[3] Furthere on, a method to determine approximative solutions by developments in Haar type wavelets serial functions is presented. [5] etc.

2. Haar Wavelets. Elementary function (1) integration.

In 1910 [1] A.Haar introduced an original example of orthogonal function system by which he managed to build faster transformations than those made by Walsh's or Fourier's ideas. Just after 1990 relevant applications of Haar's ideas where developed with deepering researches on the applications of some wavelets type functions in image processing and numeric calculus.

An eight Haar functions system and their integrals are presented in fig.no.1. (after [2]).

For a 4-level system [2], the integrals of Haar wavelet functions can be expres in a matrix like form as follows:

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$$\int_{0}^{1} f_{0}(x)dx = x \cong \frac{1}{8} [1\ 3\ 5\ 7]$$
 (2)

$$\int_{0}^{1} f_{1}(x) = \begin{cases} x \ 0 \le x < \frac{1}{2} \\ 1 - x \ \frac{1}{2} \le x < 1 \end{cases} \cong \frac{1}{8} \begin{bmatrix} 1 \ 3 \ 3 \ 1 \end{bmatrix}$$
(3)

$$\int_{0}^{1} f_{2}(x) = \begin{cases} x & 0 \le x < \frac{1}{4} \\ \frac{1}{2} - x & \frac{1}{4} \le x < \frac{1}{2} \end{cases} \cong \frac{1}{8} \begin{bmatrix} 1 & 1 & 0 & 0 \end{bmatrix} \quad (4)$$

$$\int_{0}^{1} f_{3}(x) = \begin{cases} x - \frac{1}{2} & \frac{1}{2} \le x < \frac{3}{4} \\ 1 - x & \frac{3}{4} \le x < 1 \end{cases} \cong \frac{1}{8} \begin{bmatrix} 0 & 0 & 1 & 1 \end{bmatrix} \quad (5)$$

A function that is square integrable on interval (0,1) (there for $\int_{0}^{1} f^{2}(x) dx$ exists and is finite) can be developed in Haar wavelets series as follows:

$$f(x) = c_0 f_0(x) + c_1 f_1(x) + c_2 f_2(x) + \dots$$
(6)

where

$$c_{i} = 2^{j} \int_{0}^{1} f(x) f_{i}(x) dx$$
(7)

Let's consider the following matriceal notation for integrals (2)-(5) :

$$\int_{0}^{1} H_{4}(x) dx \cong \frac{1}{8} \begin{bmatrix} 1 & 3 & 5 & 7 \\ 1 & 3 & 3 & 1 \\ 1 & 1 & 0 & 0 \\ 0 & 0 & 1 & 1 \end{bmatrix}$$
(8)

Hence, it easy to prouve that:

$$\int_{0}^{1} H_{4}(x)dx = P_{4}H_{4}(x)$$
(9)

where

$$P_{4} = \frac{1}{2*4} \begin{bmatrix} -4 & -2 & -1 & -1 \\ 2 & 0 & -1 & 1 \\ 1/2 & 1/2 & 0 & 0 \\ 1/2 & -1/2 & 0 & 0 \end{bmatrix}$$
(10)

A generally form for matrix P_m , with $m = 2^j$, $j \in N$ can be obtained using an iterative process:

$$P_{m} = \frac{1}{2m} \begin{bmatrix} 2mP_{m/2} & -H_{m/2} \\ H_{m/2}^{-1} & 0 \end{bmatrix}$$
(11)

Particularily, the matrix *P8* will have the form:

$$P_8 = \frac{1}{16} \begin{bmatrix} 16P_4 & -H_4 \\ H_{m/2}^{-1} & 0 \end{bmatrix}$$
(12)

2. Integration of a differential ordinary equation by wavelets Haar .

Let's choose an element, beam type, being loaded and lean on like in Fig. no.1:



Initial data :

L=3.00m, E= $2.1*10^8$ KN/m², I= $0.6667*10^{-6}$ m⁴, p=2kN/m.

Lets's consider, for this exemple, the following differential mathematical model:

$$v'''(x) + p_v(x)/(EI) = 0$$
 (13)

To solve this differential equation with a development in Haar series, the next stages of integration are undertaken to be passed through:

$$v^{\overline{m}} = cH(x) \tag{14.1}$$

$$v^{''} = cPH(x) + A \tag{14.2}$$

$$v'' = cP^2H(x) + Ax + B$$
 (14.3)

$$v' = cP^{3}H(x) + Ax^{2}/2 + Bx + C$$
 (14.4)

$$v = cP^{4}H(x) + Ax^{3}/6 + Bx^{2}/2 + Cx + D \quad (14.5)$$

where c is the vector of the development coefficient in Haar series.

Integration constants are obtained from restrains conditions. Then let's consider:

$$x = 0 \rightarrow v(0) = 0 \rightarrow D = 0 \tag{15.1}$$

$$x = 0 \rightarrow v'(0) = 0 \rightarrow C = 0$$
 (15.2)

The next limit conditions are:

$$x = l = 1 \rightarrow v''(1) = 0$$
 (15.3)

$$x = l = 1 \rightarrow v''(1) = 0$$
 (15.4)

If x=1, the equations (14.1) - (14.5) become:

$$v'''(1) = cH(1) \tag{16.1}$$

$$v'''(1) = cPH(1) + A \tag{16.2}$$

$$v''(1) = cP^2H(1) + A + B$$
(16.3)

$$v'(1) = cP^{3}H(1) + A/2 + B + C$$
 (16.4)

$$v(1) = cP^{4}H(1) + A/6 + B/2 + C + D \quad (16.5)$$

Introducing (15.3) in (16.3) will obtain:

$$cP^2H(1) + A + B = 0 \tag{17.1}$$

Then (15.4) turns into:

$$cPH(1) + A = 0$$
 (17.2)

From (15.1) and (15.2) it results D=C=0. Reducing and clustering in (17.1) and (17.2) we get:

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$$A + B + cP^2 H(1) = 0 \tag{18}$$

$$A + cPH(1) = 0 \tag{19}$$

Introducing (15.1), (15.3), (15.5) in differential equation (13) is aquired:

$$cH(x) + p_y/(EI) = o \tag{20}$$

Grouping equations (20), (18) and (19) it results the algebraic linear system (21) of m+2equations with unknowns : the *m* components of *c* vector of Haar development of coefficients and the two integration constants previously left unknown *A*,*B*. $cH(x) = -p_{\nu}/(EI) \tag{21.1}$

$$cP^2H(1) + A + B = 0 \tag{21.2}$$

$$cPH(1) + A = 0 \tag{21.3}$$

The values of vector v and its first four derivatives will be determinated using (14) after the algebraic system solving.

3. Graphical representations of exact and Haar approximative solutions given by (14).

Figure 3 represents the exact value of the variation law of load function [3] as well as the Haar approximation (14.1).

Pictures 4 and 5 present the variation law of the effort – shear force and bending moment – analitical determined by Haar wavelets.

Pictures 6 şi 7 represent, in paralel, the variation law of the cross-section turning and displacement functions on the longitudinal axis of the beam for the sharp and approximate variants with Haar wavelets.



Fig. 3







Tables 1 and 2 contain numerical values of exact solution in 16 points (V.exact) and aproximative solution (V.Haar) as well as the absolute error (Err.abs.%). It has been found minimal

errors (>=2/1000) beginning from 81 point (at a discretization of 128 nodes). The approximative solution may be more enhanced using an Haar discretization order greater then 7.

Tabel.1											
Nr.crt.	1	9	17	25	33	41	49	57			
V.exact	0.	-0.0012	-0.0044	-0.0093	-0.0157	-0.0234	-0.0321	-0.0417			
V.Haar	0.	-0.0012	-0.0044	-0.0093	-0.0157	-0.0233	-0.0320	-0.0416			
Err.abs%	0.0038	0.0186	0.0372	0.0573	0.0792	0.1029	0.1286	0.1564			

Table.2.												
Nr.crt.	65	73	81	89	97	105	113	121				
V.exact	-0.0520	-0.0629	-0.0742	-0.0858	-0.0977	-0.1097	-0.1218	-0.1340				
V.Haar	-0.0519	-0.0627	-0.0740	-0.0856	-0.0974	-0.1093	-0.1213	-0.1333				
Err.abs.%	0.1865	0.2190	0.2541	0.2918	0.3324	0.3760	0.4226	0.4723				

Next we represent the first three eigenmodes of vibration obtained by analitical solution as wel as the approximative one obtained by development Haar series. It is obvious a cvasi-coincidence of the same order trends. Diferences are of 10^{-3} order.



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4. Conclusions.

Solving the differential equations through approximative methods based on developments in waveles function serials allows us to get higher precision solutions. An example analitically solved out and , then in order to emphasize close results, a wavelet transformation was presented. The result's accuracy and algorithm's high speed are ecouraging us in order to make some research towards defomable solid mechanics approaces with a higer degree of complexity for wich there are no analytic solutions or they are difficult to be reached.

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About the Using of the Transfer Matrix at the Calculus of the Orthotropic Cylindrical Tanks

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Abstract: The determination of the stress and strain state in the structure of a tank with variable geometry and loading conditions, requires a large amount of computation. For isotropic tanks the transfer matrix analyze technique was developed. In what follows, this technique is extended for the orthotropic tanks. The transfer matrix analyze technique allows to order the calculus process and facilitates the programation for the automatic computation.

Keywords: Transfer matrix, cylindrical tanks, material orthotropy.

1. Introduction

The vertical cylindrical tanks are frequently made from steel or reinforced concrete or prestressed concrete. In some industries, such as chemical, food, petrochemical ones and so on, there are used also tanks or containers made from composite materials, usually reinforced with different fibers on one or two directions. The reinforcing ratios, much different on the two directions, such as the prestressed forces, are different at the tanks made of prestressed concrete, involve also different elastic characteristics on the principal stress directions two (material orthotropy).

In the case of relatively heigh reinforces tanks, the wall thicknesses is linear variable achivied, increasingly with the pressure exerted by the liquid. At the metal tanks, but also for those from reinforced or composite materials tanks, the wall thickness is achivied frequently stepwise variable, being maximum at where the liquid pressure has the greates value. Some computation assumptions take into account partially filled tank, but in the case of prestressing its effect in the circumferential direction are as circular uniform forces and on the generatrix direction the effects from prestressing apear as concentrated forces at extremities.

In design is imposing an exhaustive analysis of the stress state corresponding to a bending theory

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for a more adequate reinforcing of reinforced concrete tanks or reinforced composite onse, in order to achiev joining and stability checking in the case of the metal or composite materials tanks.

The determination of the stress state in the structure of a tank with variable geometry and loading conditions requires a large amount of computation. In the works [1], [2], the transfer matrix analyse technique was developed for isotropic tanks. This technique allows to order the calculus process and facilitates the programation for the automatic computation.

In what follows, this technique is extended at the orthotropic tanks. There are taking into account, simultaneously or successively, the following design parameters (Fig. 1):



Fig.1.The design parameters of the tanks
the liquid pressure in case of the partially filled tank;
the circumferential prestressing forces; stepwise wall thickness variation.

It is analysed the general case, where the base and the top part efforts interact. It is used the Cauchy and the initial parameters methods, in order to © 2000 Ovidius University Press determinate particular solutions on loaded interval. By means of the transfer matrix, is determinate the state vector at different levels. The fundamental state vector, localized frequently at the base of the wall, is determinate from the supporting conditions at the top edge. There are considered various supporting manner, both at the base and the top edge.

2. Displacements and internal forces in the tank walls

 $\frac{d^4 w_i}{d\xi_i^4} + 4w_i = s_i^4 \frac{q_i}{D_{xi}}$

(1)

It is considered orthotropic cylindrical tanks having walls with constant or variable thickness, with geometrical, elastic, supporting and loading symmetry against the vertical axis. On the current section i, where the geometrical parameters R_i and h_i , also the elastic properties E_{xi} , $E_{\theta i}$, v_{xi} , $v_{\theta i}$, are constant, we can use the equation [3], [4]: where: w_i - is the radial displacement from the bending of the wall; $\xi_i = x_i/s_i$ - is the reduced length;

 q_i – is the distributed radial loading; s_i – is the decreasing coefficient; D_{xi} – is the bending rigidity in the generatrix direction:

$$D_{xi} = \frac{E_{xi}h_i^3}{12(1 - v_{xi}v_{\theta_i})}$$
(2)

The s_i diminution coefficient is the reversal of the damping coefficient βi :

$$s_{i} = \frac{1}{\beta_{i}} = \frac{1}{\sqrt[4]{\frac{3(1 - v_{xi}v_{\theta_{i}})}{R_{i}^{2}h_{i}^{2}} \frac{E_{\theta_{i}}}{E_{xi}}}}}$$
(3)

The displacements w_i , ϕ_i and the internal forces M_{xi} , Q_{xi} , $N_{\theta i}$, in the points of the "it" segment, are expressed with the next relations:

$$w_{i} = w_{j-l}f_{li} + s_{i}\phi_{j-l}f_{2i} - \frac{s_{i}^{2}M_{x,j-l}}{D_{xi}}f_{3i} - \frac{s_{i}^{3}Q_{x,j-l}}{D_{xi}}f_{4i} + w_{pi}$$

$$\phi_{i} = -\frac{4}{s_{i}}w_{j-l}f_{4i} + \phi_{j-l}f_{li} - \frac{s_{i}M_{x,j-l}}{D_{xi}}f_{2i} - \frac{s_{i}^{2}Q_{x,j-l}}{D_{xi}}f_{3i} + w_{pi}^{'}$$

$$M_{xi} = \frac{4}{s_{i}^{2}}D_{xi}w_{j-l}f_{3i} + \frac{4}{s_{i}}D_{xi}\phi_{j-l}f_{4i} + M_{x,j-l}f_{li} + s_{i}Q_{x,j-l}f_{2i} - D_{xi}w_{pi}^{''}$$

$$Q_{xi} = \frac{4}{s_{i}^{3}}D_{xi}w_{j-l}f_{2i} + \frac{4}{s_{i}^{2}}D_{xi}\phi_{j-l}f_{3i} - \frac{4}{s_{i}}M_{x,j-l}f_{4i} + Q_{x,j-l}f_{li} - D_{xi}w_{pi}^{'''}$$

$$N_{\theta_{i}} = -\frac{E_{\theta_{i}}h_{i}}{R_{i}}w_{i}$$
(4)

 w_{j-l} , ϕ_{j-l} , $M_{x,j-l}$, $Q_{x,j-l}$ – are the displacements and the internal forces in the extremity j-1 of the i tronson, considered of the origin (Fig. 2), and f_{li} , f_{2i} , f_{3i} , f_{4i} are the Krilov functions:

$$f_{li} = ch\xi_i \cos\xi_i$$

$$f_{2i} = \frac{1}{2} (ch\xi_i \sin\xi_i + sh\xi_i \cos\xi_i)$$

$$f_{3i} = \frac{1}{2} sh\xi_i \sin\xi_i$$

$$f_{4i} = \frac{1}{4} (ch\xi_i \sin\xi_i - sh\xi_i \cos\xi_i)$$

(5)

 w_{pi} is the particular solution of the differential equation (1) and it can be determined using the Cauchy method:

M. Vrabie and N. Ungureanu / Ovidius University Annals of Constructions **3**, **4**, 605-610 (2002) 607 $w_{pi} = \frac{S_i^4}{D_{xi}} \int_0^{\xi_i} f_{4i}(\xi_i - \eta) q_i(\eta) d\eta$ (6)



Fig.2. Geometric and loading characteristics of the divided tank

The hydrostatic pressure on the interval i, whose extremity are j-1 and j, varies trapezoidally and it can be considered as the sum of a loading with constant intensity $q_j = \gamma(H_L - x_j)$, and a triangular one with maximum intensity $\gamma(x_j - x_{j-1})$. From the relation (6), we can obtain:

$$w_{pi} = \frac{s_{i}^{4} (H_{L} - x_{j})}{4D_{xi}} [I - f_{li}(\xi_{i})] + \frac{s_{i}^{5} \gamma}{4D_{xi}} \cdot (7)$$

$$\cdot [\lambda_{i} - \xi_{i} - \lambda_{i} f_{li}(\xi_{i}) + f_{2i}(\xi_{i})]$$

where $\lambda_i = H_i / s_i$ (H_i is the lenght of the interval).

The first three derivatives of the particular solution for the loading with trapezoidal variation are (Eq. 8):

$$w'_{pi} = \frac{s_i^3 (H_L - x_j) \gamma}{D_{xi}} f_{4i}(\xi_i) + \frac{s_i^4 \gamma}{4D_{xi}} [f_{1i}(\xi_i) + 4\lambda_i f_{4i}(\xi_i) - I]$$
(8)

$$w_{pi}'' = \frac{s_i^2 (H_L - x_j) \gamma}{D_{xi}} f_{3i}(\xi_i) + \frac{s_i^3 \gamma}{D_{xi}} [\lambda_i f_{3i}(\xi_i) - f_{4i}(\xi_i)]$$
$$w_{pi}''' = \frac{s_i (H_L - x_j) \gamma}{D_{xi}} f_{2i}(\xi_i) + \frac{s_i^2 \gamma}{D_{xi}} [\lambda_i f_{2i}(\xi_i) - f_{3i}(\xi_i)]$$

The effects of a concentrated couple, respectively of a concentrated force in the direction of the generatrix or a radially uniform distributed force on the circumference, are introduced by analogy with those from the origin. The concentrated forces can proceed particularly from the circumferential or longitudinal prestressing in the case of the tank from prestressed concrete. The particular solutions in the case of loading with a couple M and a force P are:

$$w_{pi} = -\frac{s_i^2 M}{D_{xi}} f_{3i} (\xi_i - \lambda_m) - \frac{s_i^3 P}{D_{xi}} f_{4i} (\xi_i - \lambda_p)$$
(9)

with $\lambda_m = l_m/s_i$, $\lambda_p = l_p/s_i$, where l_m and l_p are lenghts wich are distributed M and P.

3. Results and interpretation

3.1. The determination of the state vector using transfer matrix

The static parameters w, ϕ , M_x , Q_x and the unit are considered as components of the vector {S}, named *state vector*, for wich a current section from the interval "i", are written in line:

$$\{S\}_i = \{w; \phi; M_x; Q_x; I\}^T$$
(10)

The state vector from the extremities j-1 and j of the interval "i" are written:

$$\{S_{j-l}\}_{i} = \{w_{j-l}; \phi_{j-l}; M_{x,j-l}; Q_{x,j-l}; l\}^{T}$$

$$\{S_{j}\}_{i} = \{w_{j}; \phi_{j}; M_{x,j}; Q_{x,j}; l\}^{T}$$

$$(11)$$

For express the state vector $\{S_j\}_i$ depending on $\{S_{j-1}\}_i$ the relations (4), in which $\xi_i = H_i/s_i = \lambda_i$ are written in the matrix shape: $\{S_j\}_i = [T]_{j-1}^j \{S_{j-1}\}_i$ (12)

where $[T]_{j-1}^{j}$ is the *segment matrix* or *the interval matrix* for the continuity interval (j-1, j) [1], [2], [4]:

$$[T]_{j-l}^{j} = \begin{bmatrix} f_{li}(\lambda_{i}) & s_{i}f_{2i}(\lambda_{i}) & -\frac{s_{i}^{2}}{D_{xi}}f_{3i}(\lambda_{i}) & -\frac{s_{i}^{3}}{D_{xi}}f_{4i}(\lambda_{i}) & w_{pi}(\lambda_{i}) \\ -\frac{4}{s_{i}}f_{4i}(\lambda_{i}) & f_{li}(\lambda_{i}) & -\frac{s_{i}}{D_{xi}}f_{2i}(\lambda_{i}) & -\frac{s_{i}^{2}}{D_{xi}}f_{3i}(\lambda_{i}) & w_{pi}'(\lambda_{i}) \\ \frac{4D_{xi}}{s_{i}^{2}}f_{3i}(\lambda_{i}) & \frac{4D_{xi}}{s_{i}}f_{4i}(\lambda_{i}) & f_{li}(\lambda_{i}) & s_{i}f_{2i}(\lambda_{i}) & -D_{xi}w_{pi}'' \\ \frac{4D_{xi}}{s_{i}^{3}}f_{2i}(\lambda_{i}) & \frac{4D_{xi}}{s_{i}^{2}}f_{3i}(\lambda_{i}) & -\frac{4}{s_{i}}f_{4i}(\lambda_{i}) & f_{li}(\lambda_{i}) & -D_{xi}w_{pi}'' \\ 0 & 0 & 0 & 1 \end{bmatrix}$$
(13)

The axial force $N_{\theta,j}$ is determined knowing w_j .

The last column of the matrix $[T]_{j-1}^{j}$ depends on the loading and for the hydrostatic pressure is determinated from the relations (7), (8), in which ξ_i = λ_i . If the interval (j-1, j) is unloaded, than the last column is zero, excepting the last term, which remains 1.

At the passing over a concentrated couple or a radial concentrated force, which are uniform on the circumference direction and, possible, over an axial force, which is distributed also uniformly on the circumference, are used *crossing matrix*. Such a relation, which connect the state vectors from the adjacent sections k and k-1, is:

$$\{S_k\}_i = [T]_{k-l}^k \{S_{k-l}\}_i$$
(14)

or detailed, when the crossing matrix has all three loading cases:

$$\begin{cases} w_k \\ \phi_k \\ M_{x,k} \\ Q_{x,k} \\ 1 \end{cases} = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & M \\ 0 & 0 & 0 & 1 & 0 & -P \\ 0 & 0 & 0 & 0 & 1 & N \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \begin{cases} w_{k-1} \\ \phi_{k-1} \\ M_{x,k-1} \\ Q_{x,k-1} \\ N_{x,k-1} \\ 1 \end{bmatrix}$$
(15)

The couple M has been considered clockwise, the force P having the radial direction on the towards centre and the axial force N in the direction of x axis, therefore of the tension at the wall. Obviously, some of these actions can to be absent.

The change of stepwise thickness impose an adequate segmentation, which put into evidence differents stiffnesses. In a series of relations between the succesive state vectors it is achivid requrance relationships. The state vector from the section "j" is:

$$\left\{S_{j}\right\}_{i} = [T]_{j-1}^{j} [T]_{j-2}^{j-1} \cdots [T]_{i}^{2} [T]_{0}^{j} \left\{S_{0}\right\}_{i}$$
(16)

If the index j and i have certain values, the state vector is expressed in any section. In "n" extremity the vector is:

$$\{S_n\}_n = \leftarrow \prod_{j=0}^n [T]_{j-1}^j \{S_0\}_I = [T]_0^n \{S_0\}_I$$
(17)

where $\leftarrow \prod_{j=0}^{n}$ is an orderly product from the right

to the left of the transfer matrix; $[T]_0^n$ is the transfer matrix between the two extremities of the tank.

3.2. Boundary conditions

In order to determinate the state vector $\{S_0\}_1$, means to set the boundary (supporting) conditions at the top part of the tank. The relation (17) express the connection between the state vector from the section situated on the base and the superior part of the tank, the last term has the boundary conditions. The relation (17) becomes:

$$w_{n} = t_{11}w_{0} + t_{12}\phi_{0} + t_{13}M_{x0} + t_{14}Q_{x0} + t_{10}$$

$$\phi_{n} = t_{21}w_{0} + t_{22}\phi_{0} + t_{23}M_{x0} + t_{24}Q_{x0} + t_{20}$$

$$M_{xn} = t_{31}w_{0} + t_{32}\phi_{0} + t_{33}M_{x0} + t_{34}Q_{x0} + t_{30}$$

$$Q_{xn} = t_{41}w_{0} + t_{42}\phi_{0} + t_{43}M_{x0} + t_{44}Q_{x0} + t_{40}$$
(18)

The "t" coefficients are the terms of the multiplication matrix $[T]_0^n$. For the tanks which has the walls fixed at the base, $w_0 = 0$, $\phi_0 = 0$, and for the wall which is hinged at the base $w_0 = 0$, $M_{x0} = 0$. The equations (18) for the two cases are:

$$w_{n} = t_{13}M_{x0} + t_{14}Q_{x0} + t_{10}$$

$$\phi_{n} = t_{23}M_{x0} + t_{24}Q_{x0} + t_{20}$$

$$M_{xn} = t_{33}M_{x0} + t_{34}Q_{x0} + t_{30}$$

$$Q_{xn} = t_{43}M_{x0} + t_{44}Q_{x0} + t_{40}$$

(19 a)

$$w_{n} = t_{12}\phi_{0} + t_{14}Q_{x0} + t_{10}$$

$$\phi_{n} = t_{22}\phi_{0} + t_{24}Q_{x0} + t_{20}$$

$$M_{xn} = t_{32}\phi_{0} + t_{34}Q_{x0} + t_{30}$$

$$Q_{xn} = t_{42}\phi_{0} + t_{44}Q_{x0} + t_{40}$$

(19 b)

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In what follows, if the wall is free at the top edge, we have $M_{xn} = 0$, $Q_{xn} = 0$ and, from the Eq. (19a) we determine M_{x0} , Q_{x0} and, from Eq. (19b) we can calculate ϕ_0 , Q_{x0} , therefore the vector $\{S_0\}_1$ is completely determined.

In the case of an elastically supporting at the base of the wall, which has spring ties for displacement and for rotation, M_{x0} and Q_{x0} can be expressed in terms of ϕ_0 and respectively w_0 :

$$M_{x0} = k_{\phi} \phi_0, \quad Q_{x0} = k_w w_0 \tag{20}$$

where k_{ϕ} and k_{w} are the rotation, respectively displacement stiffneses of the springs, simulating the elastical connection wall-foundation. In the case of the free tank at the superior part, we can obtain the following relations in w_{0} and ϕ_{0} :

After the determination of the fundamental state vector from the supporting condition at the superior part, we can express with the help of the transfer matrix, the state vectors at the different levels (16). The developed analysis technique arranges the computation process and facilitates the programation for the automatic calculus.

3.3. Calculus example

For the proposed of this procedure it is considered a cylindrical tank with the radius $R_i = R$ =15m and the height H = 8m, made of prestressed concrete that is considered:

a) isotropic material with $E_x = E_\theta = E = 300000$ daN/cm², $v_x = v_\theta = v = 0.2$;

b) orthotropic material with $E_x = 240000$ daN/cm², E_{θ} = 300000 daN/cm², $v_x = 0.16$, $v_{\theta} = 0.2$.

The tank is partially filled with water ($\gamma = 10$ kN/m^3); the liquid has the height from the base H_L =6m. The wall of the tank has a thickness h_1 = 25cm, for a height $H_1 = 3m$ from the base, untill the superior part the thickness is 15 cm. The wall of the tank, fixed at the base and free at the superior part, is divided in three segments for the calculus (Fig. 3a).



Fig. 3. – a) Geometric and loading conditions; b) Internal forces diagrams (M_x and Q_x)

It is calculated the segment matrix $[T]_{0}^{l}, [T]_{1}^{2}, [T]_{2}^{3}$ and than, the multiplication matrix $[T]_{0}^{3} = [T]_{2}^{3} [T]_{1}^{2} [T]_{0}^{1}$, by which is written the state vector from the free end $\{S_3\}_3 = [T]_0^3 \{S_0\}_1$.

Using the boundary conditions $M_{x3} = 0$, Q_{x3}=0, is determinated the components of the fundamental state vector:

a) M_{x0} =-4965.712 daN, Q_{x0} =75.112 daN/cm;

b) M_{x0} =-4481.515 daN, Q_{x0} =71.664 daN/cm;

The state vectors from the sections 1, 2 in the two cases of material are:

 ${S_1}_1 = {0.113; 2.0358E - 04; 628.69; -1.451; 6780; 1}^T$ $\{S_2\}_2 = \{0.018; -3.1177E - 04; -196.97; -0.167; 54; 1\}_1^2$

in the case a) and b) the following

$$\{S_1\}_1 = \{0.117; 1.8086E - 04; 516.54; -1.289; 5834; 1\}^T \\ \{S_1\}_2 = \{0.017; -2.9735E - 04; -169.88; 0.046; 51; 1\}^T$$

$$\{S_2\}_2 = \{0.017; -2.9735E - 04; -169.88; 0.046; 51; 1\}$$

The plotting of the internal forces on the interval (1) (where they have significant values), is presented in Fig. 3b

4. Conclusions

1. The determination of the stress state in the structure of a isotropic tank with variable geometry and loading conditions implies a large number of design parameters. This number of parameters is bigger for the orthotropic tanks.

2. Analyze technique with transfer matrix has some advantages:

- allow to take into account the various loading conditions, variable geometry of the tank walls and various boundary conditions;

- allow to order the calculus process and facilitates the programation for the automatic computation.

3. The results obtained from the calculus example show that, in the case "b" (the tank made from orthotropic material), the values of the internal forces is smaller than in the case "a" – with 10% for M_x and 5% for Q_x in section 0 at the base of the tank wall.

4. The values of the radial displacements w is bigger in the case "b" that in the case "a" (with 3,5% in section 1, at 3m from the base of the tank).

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The Effect of Multiple Properties of Aluminium Sulphate in Surface Water Treatment

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Abstract. The present paper discusses aspects specific of the Bucecea Water Treatment Plant, focusing on the use of the aluminium sulphate in order to eliminate colloids and water colour. The present study has also investigated the hypothesis according to which the aluminium sulphate might be a risk factor as far as the Alzheimer's disease is concerned. The quantitative and qualitative peculiarities of this water supply have led to these studies which have been carried out since 1993.

Abstract. Articolul sintetizează studiile efectuate în perioada 1993-2001, la stația de tratare a apei Bucecea, privind optimizarea procesului de utilizare a sulfatului de aluminiu pentru eliminarea coloizilor și culorii din apă. Studiul a investigat, de asemenea, ipoteza privind implicarea aluminiului rezidual în apa tratată, în apariția bolii Alzheimer.

Keywords: coagulation, aluminium sulphate, Alzheimer's disease.

1. Introduction

One of the water sources of Botosani – a town situated in the north-eastern part of Romania – is Bucecea Lake, located on the Siret River; its discharge is $0.7 \text{ m}^3 \cdot \text{s}^{-1}$.

The small and medium water treatment plants have to cope with two antagonistic problems: on the one hand, the quality of the water treated has to be the same as that of the water supplied by the big waterworks; on the other, this operation has to be carried out by using a simple technology and its costs have to be low, because of the low discharges treated. This explains the necessity of identifying cheap, simple but very efficient treating methods. The raw water discharge, the characteristics of the lake water and the financial aspects of this operation led to the initiation of the present study, which has been carried out since 1993. The physico-chemical parameters of the water in Bucecea Lake, as well as the factors influencing the coagulation-flocculation process and its dynamics have been thoroughly investigated by the authors of the present experiment.

2. The Influence of the Physico-Chemical Characteristics of the Natural Water on the Coagulation-Flocculation Process

The Botosani water treatment plant has been using chemical treatment in the clarification and disinfection phases. In order to improve the clarification technology, one has to take into account the characteristics of the lake water, which have been a direct and very strong influence upon each and every treatment stage: the surface tension, high dielectrical constant, the electrical conductivity, the bond energy of the water molecule, the dissolution (hydrating attraction), the solubility of solids, the water ionizing capacity, water participation in redox reactions, the turbidity, the colour.

The substances present in natural water may have *three dispersion states*:

a) molecular dispersions; in this case, the diffusion velocity of the molecules dissolved in the liquid agrees with Fick's law;

b) colloidal dispersions where the superficial forces are predominant as compared to the mass forces.

c) gravimetric dispersions – the two phases tend to separate, according to Stokes' law.

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3. The Colloids Stability and the Necessity of Coagulation

The natural water colloids have an organic origin (proteins, polypeptides, humic compounds etc.) and a mineral one (aluminium silicates, clays). Generally, these are hydrophobic particles, characterized by a negative charge. Colloids are subject to two types of forces: – van der Waals attraction forces; they depends on both the shape and structure of the colloids and on the characteristics of the environment; – repulsive electrostatic forces, which depend on the superficial charge of the colloidal particle. The stability of the colloidal dispersions depends on the equilibrium of these forces.

3.1. The Colloidal Stability Explained by the Double Electrical Layer Theory, by the Chemical Theory and by Other Theories

a) In 1924, Stern stated a theory of *the colloidal stability*, considering the particle surrounded by two layers. The first layer (adherent or adsorbtion layer) of 0.1 μ m thick contains water molecules strongly bound due to the colloidal particle. The electrical potential of this layer decreases rapidly. The second layer is a diffuse area in which counter-ions are attracted by the colloidal particle – this colloidal particle keeps moving and some of the ions of the diffuse area are lost; consequently, the ensemble remains charged [1]. The electrical potential decreases slowly and its value at the limit which separates the two layers is known as Zeta potential (*Z*):

$$Z = \frac{km\mu}{D} \tag{1}$$

where: $k = 4\pi$ - 6π is a factor depending on the nature and dimensions of the colloid; m = electrophoretic mobility of the colloidal particle (m²·s⁻¹·V⁻¹); for natural water at 25°C, $m \approx 10^{-6}$; D=water dielectrical constant (F·m⁻¹); μ = water dynamic viscosity (kg·m⁻¹·s⁻¹).

In natural water, there are fine particles of clay and quartz and some organic substances, which have a negative charge and are not connected with water molecules. The neutralization of theese charges, named "coagulation", is conditioned by the presence of the electrolytes of polyvalent metals or other colloids having an opposite charge, as well as substances modifying the OH⁻ and H⁺ contents (*p*H).

b) The chemical model is based on the fact that the energy of the covalent bonds is 20-50 times higher than the energy of the electrostatic attraction [1, 2, 3]; it also states that the initial charge of the colloid is due to the ionization of the polar chemical groups existing at its surface. The neutralization of this charge will be determined by covalent reaction between these ionized groups placed on the particle surface and the polyvalent metallic ions of the coagulant. This theory shows that the simultaneous precipitation of the metallic hydroxides and the "bridging coagulation" represent the fundamental phenomena of the coagulation process [1].

c) The adsorbtion, the bridging effect and the net (or sweep) effect are other theories which explain the destabilization and aggregation phenomena (Fig. 1) [1].

4. The Colloids Sedimentation Phases. The Utilization of the Aluminium Sulphate in the Coagulation-Flocculation Process

The coagulation process comprises several successive or simultaneous stages which could be listed as follows: the hydrolysis, the coagulation and the flocculation. The flocculation has two transport phases: perikinetic and orthokinetic flocculation.

The choice of a certain flocculant has to take into consideration Schultze-Hardy's rule (1882), according to which the number of ions necessary for the electrostatic coagulation is inversely proportional to their charge: $n_i = (q_i)^{-6}$.

According to the classical theory, the salts of the strong acids hydrolyze with weak bases in order to create hydroxides slightly soluble: $M^{n_+} + nH_2O \leftrightarrow M(OH)_n \downarrow + n H^+$, where M^{n_+} stands for the cation of the salt used as coagulant. The aluminium sulphate satisfies all these requirements.

Practical experiments have shown that $Al_2(SO_4)_3$ also improved other water characteristics. The behaviour of this reagent in water indicates that in concentrated solutions having a low *p*H, the aluminium sulphate can be

found under the form of ions: $Al_2(SO_4)_3 \rightarrow 2Al^{3+} + 3(SO_4)^{2-}$, but if the same reagent is placed in natural water (which is a diluted solution), the Al^{3+} ion hydrolyzes under the form of polynuclear ionic groups. These groups contain di- and trivalent ions (other than OH⁻) and up to 8 Al atoms.

The forming of such species depends on the H⁺ and OH⁻ ions concentration (the water pH) and the raw water chemical composition. Many OH⁻ ions participate in this process, so that there appears a buffer effect provoked by the water bicarbonates, which finally reduces alkalinity dose, the amount of unprecipitated residual Al₂(SO₄)₃ in the treated water The *p*H influences the aluminium sulphate and the dynamics of the coagulation process.

The organic colloids and especially those responsible for water colour require 2-3 pH stages, according to their nature and the non-aggressive pH necessary for treated water. The correlation between the colloid origin and pH influences the duration of the coagulation.

The $Al_2(SO_4)_3$ has a positive influence on other treatment processes too; it accelerates the flocculation of the fine mineral suspensions, even at low doses; it also favours colour removal.

The water pH decreases towards the coagulation end, so that the residual $Al_2(SO_4)_3$ will interfere with chlorination in a favourable way (the bactericidal effect of the chlorine is stimulated by a low pH).

Moreover, the experiments have indicated that an insignificant addition of Cl_2 before the coagulation favours the oxidation of certain organic compounds; their transformation into inorganic substances will facilitate the flocculation process.

5. Results and Conclusions

The systematic studies allow us to interpret the results and some conclusions. The analysis of all the parameters indicates that some of them have a weak influence on the adjustement of the reagent doses, while others have a more important role during the operation.

a) The water source is characterized by a *high stability of the pH* and this facilitates the adjustement of the coagulant doses.

Nevertheless, two stages can be identified within the analysed period: 1993 - 1996 when the *p*H varies between 7.3 (Feb. 1993) and 6.7 (Sept. 1994); 1996 - 2000, a stage characterized by a general and constant growth of the *p*H value of the Bucecea Lake water (Fig. 2 a); its maximum value was 8.4 (June – July 1998; Sept. 1999), whereas the minimum level was 7.5 (July 1996).

The water source alkalinization begun in 1996 is produced by the discharge of some effluents insufficiently treated.

For the utilization of rich commercial substances (13% in 1996 and 16.8% nowadays), an operational decision was taken in April 1999.

b) The *raw water temperature* increases and decreases seasonally (from 2.4 °C in Feb. 1998 to 24.6 °C in July 1998 and Aug. 1999); this slow process makes possible the identification of the optimum reagent doses.



Fig. 1. Aggregation mechanism: a - adsorbtion; b - bridging adsorbtion; c - sweep coagulation

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in July).

NTU).

The difference between extreme temperatures wasn't very important until 1995 (the minimum temperature was 8.2 °C in Feb. 1994 and the maximum value was 21.9 °C, in August) (Fig. 2 b); the excessively continental character of the local climate has been predominant since 1995.

This explains the considerable differences between the mean summer temperatures and the mean winter values (in 1998, the minimum



a.- *p*H; b.- temperature; 1.- maximum; 2.- minimum

c) The *turbidity periodicity* of the raw water also represents an important characteristic (Fig. 3).

During the period November – April, its mean multi-annual value is rather low (7.06 NTU); this value rapidly increases up to 32.17 NTU during the period April – November. In July 1995 and May 1998 it even reached 150 NTU, and 130 NTU in April 1996.

During the period 1993 – 1996, the mean annual turbidity evolved between 3.9 NTU and 20.41 NTU.

After 1996, this value evolved considerably; the minimum value went up to 9.24, and the maximum one up to 45.92.

This phenomenon happened because of the abundant precipitations, which affected the Siret drainage area, after 1996.

We should also mention the *accidental colouring of water*; this problem couldn't be solved by a specific economical treatment.

Consequently, the coagulant doses were adjusted so that they might adapt to the variations

of turbidity, temperature, pH, colour and so as to combine efficiently with the final chlorination process (Fig. 3).

was 2.4 °C in February and the maximum 24.6 °C

did not require the use of some flocculation aids,

because the turbidity values were also low, excepting Dec. 1996 (5.5 °C; the sulphate dose = 14 NTU), Feb.

1998 (2.4 °C - 18 NTU), Feb. 2000 (5.3 °C - 31

The low raw water temperature after 1995

The results of these measurements were positive: during the cold periods, the coagulant consumption was extremely low.

The previous stocks made up for the high coagulant consumption during high turbidity periods: 134 g·m⁻³ in June 1994; 90 g·m⁻³ in July 1995 and May 1998; thus the whole annual operational cycle saved 10-15% of the total amount of reagent.

The rises in the treated water turbidity (7.9 NTU in March 1999; 6.5 in July 1995; 5.5 in Feb. 2000) do not go beyond the limit of the EEC and USPHS-AWWA standards.

Excepting these accidental rises, the final treated water respects Romanian quality standards as well as international norms, due to a very economical operational management.



Fig. 3. The adjustement of the coagulant doses

6. Aluminium sulphate and Alzheimer's disease

Due to various studies which have recently emphasized the role of the aluminium as a potential risk factor for the *Alzheimer's disease*, Romanian psychiatrists reassess the senile dementia diagnosis nowadays.

During the analysed period, no person suffering from Alzheimer's disease was registered, according to official data for the Botosani district.

The amount of aluminium absorbed by the organism from the drinking water treated with $Al_2(SO_4)_3$ is not able to favour the appearance of this disease.

Alzheimer's disease is provoked only by a very high blood concentration of aluminium; this

fact indicates that aluminium might have other origins too (i. e. vessel, aluminium foils, dialysis).

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Some Aspects Regarding Biological Nutrients Removal From Waste Water Treatment plant of Iasi

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Abstract: This work presents some results obtained by the authors in the field about biological nutrients removal, especially nitrogen and phosphorus, using nitrification and denitrification processes. The experimental results are presented comparatively on the industrial equipment and pilot equipment.

Keywords: Biological nitrification and denitrification precesses, biological kinetics, pilot equipment

1. Introduction

Every community produces both liquid and solid wastes. The liquid portion-wastewater-is essentialy the water supply of the community after it has been fouled by a variety of use.

Untreated wastewater usually contains numerous pathogenic or disease - causing microorganisms that dwell in the human intestinal tract or that may be present in certain industrial wastes. It also contains nutrients, which can stimulate the growth of aquatic plants, and it may contain toxic compounds. For these reasons, the immedate and nuisance-free removal of wastewater from its sources of generation, followed by treatment and disposal is not only desirable but also necessary in an industrialized society.

At the present time, unit operations and processes are grouped together to provide what is known as primary, secondary and tertiary (or advanced) treatment. In primary treatment, physical operations, such as screening and sedimentation, are used to remove the floating and settleable solids found in wastewater.

In secondary treatment, biological and chemical processes are used to remove most of the organic mater. In tertiary treatment, additional combinations of unit operation and processes are used to remove other constituents, such as nitrogen and phosphorus, which are not removed by secondary treatment.

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2. Biological nitrogen removal

Biological nitrogen removal requires a two - step process. In the first step ammonia is oxidized to nitrate (nitrification) and various process configurations are then employed to provide the nitrate as an electron acceptor for biological respiration so that it can be reduced to molecular nitrogen (denitrification). Fundamental considerations will be presented first for the nitrification step, followed by similar review for the denitrification step in nitrogen removal.[4]

2.1. Biological nitrification

2.1.1. Oxidation and synthesis relationships

The energy - yielding two - steps oxidatuion of ammonia to nitrate is generally accepted to be as follows [1]: Nitrosomonas

$$2NH_4 + 3O_2 \rightarrow 2NO_2^- + 4H^+ + 2H_2O$$
 (1)

Nitrobacter

$$2\mathrm{NO}_2^- + \mathrm{O}_2 \to 2\mathrm{NO}_3^- \tag{2}$$

Total reaction : $NH_4^+ + 2O_2 \rightarrow NO_3^- + 2H_+ + H_2O$ (3)

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The eqution was developed using cell yield coefficients of 0.15 g/g NH_4 -N oxidized and 0.02 g/g NO_2 -N oxidized.

 $NH_4^+ + 1,83O_2 + 1,98HCO_3^- \rightarrow 0,21C_5H_7O_2N + 0,98NO_3^- + 1,041H_2O + 1,88H_2CO_3$ (4)

Cell yields for Nitrosomonas are generally heigher than those of Nitrobacter.

2.1.2. Biological kinetics

The generally accepted design approach for nitrification system is to determine the aeration tank volume byselecting an appropriate SRT that, will provide the desired effluent ammonia nitrogen concentration at the important operating conditions expected, which include temperature, disolved oxygen concentration and pH. The design SRT is related to the expected specific growth rate of the nitrifying bacteria using Monod kinetic expression and usually includes a safety factor to account for the need to have a greater inventory of nitrifying bacteria to handle peak ammonia loads. The safety factor should be equal to the peak to average ammonia loads to maintain the effluent ammonia concentration at or below the selected design value [2]:

$$SRT_{d} = S.F.(SRT)$$
(5)

where :

 SRT_d - design solids retention time, d S.F. - design safety factor = peak/average $NH_3\mathchar`-N$ load

SRT - required solids retention time, d

The aeration tank volume for a completely mixed activated sludge system can be calculated as follows using the design SRT :

$$V = [Y_{nH}(S_0 - S) + X_I + Y_{nN}(N_{OX})]Q(SRT_d)/X$$
(6)

where :

V - aeration tank volume, m³; Q - influent flow;

- X aeration tank mixed liquor concentration, mg/l;
- S_0 influent BOD₅, mg/l;
- S efluent BOD₅, mg/l;
- Nox ammonia in influent flow oxidized, mg/l
- Y_{nH} net yield of heterotrophic organisms at
 - design SRT including endogenous decay, g TSS/g BOD₅ removed;
- Y_{nN} net yield of nitrifying bacteria, gTSS/gN oxidized;
- X₁ influent non-biodegradable inert solids concentration, mg/l.

The required SRT is equal to the reciprocal of the net specific growth of the nitrifying organisms :

$$SRT = \frac{l}{\mu_n - K_{nd}} \tag{7}$$

where :

- μ_n nitrifying bacteria specific growth rate, g new cells/g cell-d;
- K_{nd} endogenous decay rate of the nitrifying bacteria, g cells decayed/g cells-d

Since information is not available to substantiale a decay coefficient value, it is ignored and the required SRT is reciprocal of the specific growth rate determined by Monod kinetics :

$$\mu_n = \frac{(\mu_n, \max)N}{K_n + N} \tag{8}$$

where :

 μ_n , max – maximum specific growth rate of nitrifiers, g cell produced/g cell-d; K_n – half saturation coefficient, mg/l; N – ammonia nitrogen concentration, mg/l.

In active nitrifying systems the nitrite concentration is usually very low compared to the ammonium concentration, due to higher nitrite oxidation rates by Nitrobacter compared to ammonia oxidation rates by Nitrosomonas.Thus, the kinetics are based on ammonia-nitrogen utilisation rates or the activity of the Nitrosomonas bacteria. Note that during the start up of nitrification system, the nitrite concentration will be higher until the Nitrobacter population increases to equilibrium.

2.2. Biological denitrification

Two models of nitrate reduction can occur in biological system : assimilating and dissimilating or denitrification. Assimilating nitrate reduction involves the reduction of nitrate to ammonia for use in cell synthesis. It occurs independently of oxygen tension and when ammonium nitrogen is not available. On the other hand, dissimilating nitrate reduction or denitrification is coupled to the respirating electron chain and involves the reduction of nitrate to nitrite to nitric oxide to nitrous oxide to nitrogen [5]:

$$NO_3^- \rightarrow NO_2^- \rightarrow NO \rightarrow N_2O \rightarrow N_2$$
 (9)

Denitrification is considered to be an anoxic process, occurring in the absence of oxygen, and requires an organic electron donor.

2.2.1. Oxidation-reduction reactions

An appreciation of the oxidation-reduction reactions and biological yield coefficients is needed to determine substrate requirements. The following shows oxidation-reduction reactions proposed for various substrate used for denitrification in wastewater treatment. Methanol is shown first, as it has been the widely studied substrate and most commonly used substrate when an external carbon source has been employed.[7]

Methanol

$$5CH_3OH + 6NO_3^- \rightarrow 3N_2 + 5CO_2 + 7H_2O + 6OH^-$$
 (10)

Acetic acid $5CH_3COOH+NO_3 \rightarrow 4N_2+10CO_2+6H_2O+68H^-$ (11)

Methane

$$5CH_4 + 8NO_3 \rightarrow 4N_2 + 5CO_2 + 6H_2O + 8OH^2$$
(12)

In all the above heterotrophic denitrification reactions, one equivalent of alkalinity is produced per equivalent of nitrate-nitrogen reduced. The interes in biological kinetic rates for denitrification usually applies to three common types of nitrogen removal system design :

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- a) system with anoxic zones preceding aeration, where the influent wastewater provides the carbon source to create a biological demand for nitrate;
- b) a post-aeration reactor where the endogenous respiration oxygen demand of the bacteria is used for nitrate reduction;
- c) a post-nitrification reactor system with a separate clarifier where methanol is added to grow bacteria and to create a demand for the nitrate electron acceptor.

Other system, such as intermitent aeration or system that add methanol to a post-anoxic zone, may be used, and modifications of the denitrification rate expressions to follow will be required.[6]

3. Biological removal of phosphates

For biological P removal it was necessary that the organisms pass through an anaerobic stage in the absence of both nitrates and dissolved oxygen.

It identified *Acinetobacter* as the organism responsible for accumulating excess phosphates in their cells when they have short-chain volatile fatty acids, especially acetates, as feed stock.

The acetates taken up were stored as poly- β -hydroxybuterate (PHB) inside the cells until they reached the aeration zone where the PHB was methabolized under aerobic conditions, providing energy for the uptake of all available ortophosphate.[3]

4. Experimental results

There have been performed experimental studies on a pilot equipment, with a 1-2 1/20 sec. Flow, connected to the industrial treatment flux of wastewater treatment plant of Iasi.

The scheme of the equipment is presented in figure 1.

The tables 1 and 2 present the concentration and quantity variations of the nutrients in the experimented technologies



Figure 1 - The scheme of the experimented technological flux with advanced primary and secondary step

Phosphorus removal

where: 1- principal pumping station; 2- waste water influent; 3- flowmeter ($O_2=0,2 \text{ mg/l}$); 4- vertical desilter ($O_2=0 \text{ mg/l}$); 5- preaeration basin ($O_2=2 \text{ mg/l}$); 6- fate removal tank ($O_2=1...2 \text{ mg/l}$); 7- primary settling tank ($O_2=0...0,3 \text{ mg/l}$); 8- anoxic basin ($O_2=0 \text{ mg/l}$); 9- activ sludge aeration basin ($O_2=5 \text{ mg/l}$); 10-final settling tank ($O_2=4 \text{ mg/l}$); 11- treated waste water; AE- breating-air equipment; a- anaerob activ sludge in exces; b- aerob activ sludge in exces; c- recycling aerob activ sludge.

Within the experiment there has been used only a circuit on the advanced primary and secondary step, and the experimental cicle has been performed paralel with that of the wastewater treatment plant. By comparison the following final results of the efluent flow caracteristics have been obtain, and are show in figure 2.
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Table 1. The concentration variations of the nutrients in the experimented technologies

								0	
Parameters –	INF	LUENT (1	ng/l)	EFI	LUENT (n	ng/l)	EFFI	CIENCI	E (%)
experiment type	min.	max	med.	min.	max	Med.	min.	max	med.
			COD	– Mn					
a	-	-	-	62	131	102	-	-	-
b – ind. exp.	142	266	229,4	34	56	45,2	69	85	79,77
c – pilot exp.	142	266	229,4	11	16,6	14,49	89	95	93,35
			NI	\mathbf{H}_{4}^{+}					
a	-	-	-	15	29	24	-	-	-
b – ind. exp.	15,36	39,93	24,03	15	26,62	17,01	1	58,1	33,17
c – pilot exp.	15,36	39,93	24,03	2,08	5,8	3,17	71	92	86,5
$\mathbf{P}_{\text{total}}$									
a	-	-	-	2,6	4,1	3,1	-	-	-
b – ind. exp.	2,63	4,24	3,21	1,9	3,08	2,43	2,2	36	24,25
c – pilot exp.	2,63	4,24	3,21	0,06	0,2	0,1	92,8	98,2	96,69

a – situation in the wastewater treatment plant of Iasi;

b – experiment at industrial scale on the 50 percent of the flow;

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c – experiment at pilot scale in paralel with b point.

Doromotors	Parameters – experiment typeINFLUENTTREATED			EFLUENT	
experiment type			Total	Adm. max.	Over adm. max
		COD-Mn			
a	83,24	46,23	37,04	5,44	31,60
b – ind. exp.	83,24	66,69	16,55	5,44	11,11
c – pilot exp.	83,24	77,98	5,26	5,44	-0,18
	NH4 ⁺				
a	8,72	0,01	8,71	1,08	7,63
b – ind. exp.	8,72	2,55	6,17	1,08	5,09
c – pilot exp.	8,72	7,57	1,15	1,08	0,07
P _{total}					
a	1,16	0,04	1,12	0,03	1,09
b – ind. exp.	1,16	0,29	0,87	0,03	1,84
c – pilot exp.	1,16	1,13	0,03	0,03	-

	Table 2. The q	uantity variations	of the nutrients	s in the expe	rimented techno	logies [to/day]
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The Accomplishment of the most Propitious Control of one Water Supply Systems

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Abstract: At present the water supply system of our country has the following drawbachs: discontinuous supply and functioning with a great energy consumption. The modernization of one water supply systems imposes the study of the functioning of one distribution network, the pump stations and one reservoirs, starting from a global outlook that should include both the present situation and the financial administration of each element as well as the exploitation, development and best relationship between them. This paper presents the possibility of the accomplishment of the most propitious control in a particular study.

Key words: water supply system, most propitious control, efficiency

1. Introduction

As well as other public utilities, the management of the water supply system should accomplish or lead to an efficient functioning of the systems, regardless of their complexity and at low costs. In order to get the best benefices of the system there should be adopted a global outlook gathering all the different parts of an integral system.

The existant technology succeeded in offering data for measuring and control. Still, the computer software as algorytms are not developed enough so that they could accomplish the control of the technological process this research in this field becomes compulsony.

2. Accomplishment y one most propitious control

The structure of the water supply systems an be represented as a figure composed of sub/systems mainly interconnected with the help of reservoirs and pump stations. In case of such systems, thanks to an operative management and control the surveyance and coordination of the production sub- systems is attained this they work in best conditions, provided the level of the reservoirs is appropriate and the economic and technological sides are good, this ensuring the delivery dates to be respected. Considering the big volume of production even small waste of water and better consum the energy. ISSN-12223-7221 changes in the functioning of the system leads to great effects.

The coordination of these processes can be considered to be a global improvement having a distinct level of management following the hierarchy after the levels of control, regulation and improvement of the single processes [2].

Fig. 1 unites a series of modeling elements in order to obtain the computer surveyance. This the mentioned chart includes the system model, the analysis model and the consumption schemes of the networks for 24 hours.

The network appears as a circular one, with a reservoir and a pump station

sending the information to the models through a monitored system.

The measured data, usually represented by the levels in the reservoirs, the consumption in knots and the characteristics of the pumps are transmitted to the model of the system and the model of the consumption chart. They should take into consideration the influence of the day in the week, of the hour, temperature and season. The initial entrance in the models of the system aims to accomplish a similitude to its state. Once this similitude established, several control alternatives can be studied, present or future control of the system in order to reduce the

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3. Particular study

In order to analyse the functioning of a system there should be known the position of the network, the diameters and the materials of the channels, the correct topography of the area, the parameters describing the network as well as the water consumptions established are the basis of statistics with the aid of charts and of reduced measurements in the network this it is accomplished a better surveyance of the functioning of the system [1].

In the case of a distribution system with 2 reservoirs (chart 2) the obtained results are shown in chart 1 and 2 : the number of channels, the way of the flow, the flow through channels, the water speed, the levels of the water (starting from the ones in the available pressure.



Fig. 1: 1- Reservoir; 2 – Pump station; 3 – Distribution Network; 4 – Model consumption chart; 5 – Model system; 6 – Analysis model; 7 – Several control hypothesis; 8 – Order; 9 – selected order.



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Chart 1

Knot	Height	Piezometer	Available	Consumption
	(m)	Measure (m)	Pressure(m)	$(m^{3}/)$
1	150	165.9011	15.9011	0.0080
2	140	164.0835	24.0835	0.0100
3	135	164.6851	29.6851	0.0070
4	137	165.4780	28.4700	0.0050
5	142	164.4453	22.4453	0.0015
6	162	167.0000	5.0000	-
7	155	166.2000	11.2000	-

Chart 2

Channel	Flow	Speed	Length	Diametre
	(m^{3}/s)	(m/s)	(m)	(m)
T1	0.0249339	0.7936713	300	0.200
T2	0.00937489	0.7639303	300	0.125
T3	- 0.0004430	0.2256186	300	0.050
T4	0.00750927	0.6119069	200	0.125
T5	-0.00025748	0.1310931	500	0.050
T6	0.02021410	0.6434350	300	0.200
T7	0.00767369	0.6253035	350	0.125
T8	0.00761275	0.6204000	250	0.125

4. Conclusions

The presented model has the following advantages:

- more precise modeling of the consumption in the networks

- accomplishment of improvement studies having more precise results

- simulating the functioning of the system under certain imposed conditions

- accomplishment of some forecast on consumed flows that should give the basic information for expansion, planning and getting a better control

- accomplishment of a better exploitation through a most propitious control.

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Disinfection of the Water in Swimming Pools

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Abstract: The existence of some meningitis cases and some other illness caused by infections from the water in the swimming pools is the reason why the disinfection of the water in the swimming pools has become a serious matter.

This paper presents some laboratory studies on the reduction of the number of bacteria in the water of the swimming pools, using different oxidizers.

The obtained results establish what oxidizers can be used, in what quantities and the minimal doses that could exist in the water so that the swimmers should not get contaminated.

Key words: disinfection, swimming pools, oxidizers

1. Introduction

The risk of getting contaminated in the water of the swimming pools starts from a simple irritation of the mucous membrane and could lead even to death. The appearance of source meningitis cases and some other illness in or country, due to the contamination in the swimming pools is the reason only this it has become a serious problem.

The disinfection has a hygienic purpose as well as an aesthetic one. The hygienic purpose aimes to stop the transmission of catching diseases. The second purpose is to avoid the growth of microscopic algae in the water, giving the green colour of the water. In order to avoid contamination the water should contain a permanent disinfectant.

There are known several disinfection procedures: with chlorine, with chlorine components, with bromine, with ozone, with ultraviolet rays and other electro-physical procedures.

The disinfectant agent that should be used must have the following characteristics or be able to:

- destroy in a shout time the embryos existing in the water
 - have a high stability
 - be easy to control

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- do not produce harmful side effects
- do not alter the qualities of the water in the pool the disinfection of the swimming pools is usually done using chlorine.

2. The disinfection with chlorine and chlorine components Laboratory studies

The chlorine components used for the disinfection of the water in the swimming pool are gaseous chlorine, sodium chlorine, calcium chlorine, lime chlorite and other.

The dissolution of chlorine in water leads to several reactions. First, the chlorine acid HclO splits giving the ion:

 $HClO + H_2O = ClO^- + H_3O^+$

The oxidant and microbian power of the chlorine in water is given by the cell of the chlorine acid further producing the oxygene in natural state: $HCIO + H_2O = H_3O^+ + CI^- + O$

The chlorine acid is the form of the chlorine in water having a disinfective power, called"active chlorine".

The destruction of the microorganisms with chlorine follows several stages:

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- the chlorisne acid formed in water enters the cell
- the chlorine works on the protoplasmatic proteins, annihilating the metabolism of the glucose and the enzymes
- the natural oxygene freed by the HCLO cell work on the groups of free

sulf – hydrogen (SH) of the enzymes and alter them completely.

For a better disinfection with chlorine the water should have a neuter pH between 6.9 - 7.7.

In order to establish the chlorine doses there should be found the print shown in fig.1.



Fig.1 Finding the point for a sample of water from Moldova Swimming Pool - Iasi

The point describes the inflexion of the curve shawing the chlorine dose existing or remaining in the water. In order to efficiently disinfect the water, the chlorine dose should be superior to one break point. Chlorine, apart from destroying the microorganism in the pool, has a permanent activity (Fig.2).



Fig.2 Chlorine subsiding in Moldova Swimming Pool

3. Conclusions:

The laboratory analysis and the practice from Moldova Swimming Pool prove that a minimum of $0,6 - 0,8 \text{ mg/dm}^3$ of chlorine is enough in order to maintain the water in the pool within the limits allowed from a bacterian point of view.

An excess of chlorine neutralizes the cells, the water becomes disagrable and the smell unpleasant. The swimmers suffer from initations and the swell of chlorine is too strong.

The laboratory study to reduce the microorganisms in the water is not always decisive because the embargos are net emitted in their natural state lent

included in different organic substances, protecting against the antiseptic.

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Fuzzy Control of Urban Drainage Systems

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Abstract: This paper presents a fuzzy logic approach of existing urban drainage practices. Fuzzy logic is closer by human thinking and by natural language, so is easier to use in many fields. The main concepts in fuzzy logic are summary presented. Fuzzy control system is robust, flexible and easily accepted because it included the expert knowdlege. It can be a useful suplement to other conventional optimization techniques, or a replacement. It offers to the operator the possibility to participate directly in the system control, combining the results of the modern optimization techniques with the experience and knowledge accumulated in time by experts. Thus, the control of urban drainage system can be well solved by implementing an intelligent control system, based on available information and on expert's experience.

Keywords: fuzzy logic, urban storm drainage, intelligent control system.

1. Introduction

Conventional design of urban drainage has focused on conveyance of runoff flows and neglected water quality considerations. Urban development increases the runoff volume and the risk of flooding might become worse over the years. Localized flood events do still occur due to high local storm intensities, subsequent hydraulic problems and the lack of on-site storage. Temporary sewer blockages due to maintenance work increase this risk. Also, transient flows into treatment plants can cause disruption of the treatment process. In the age of advanced technology it is thought that protection against all types of disasters should be provided.

During the past several years, fuzzy control has proved to be the active research in the applications of fuzzy set theory, especially in the processes which do not lend themselves to control by conventional methods because of a lack of quantitative data regarding the input-output relations. The way of using fuzzy logic in urban drainage controlling brings some essential advantages like the use of algorithms in the case of defining affirmations by (logical) expressions.

Most of successful applications of fuzzy systems have been in control systems with few variables, because most of the pioneers were control engineers and a simple control loop regulates most consumer products. The fuzzy system works best when the rules linking outputs to inputs can be accurately specified. In the application of fuzzy set there are still a lot of problems to solve such as: how to obtain membership values, how to assign weights, how to handle continuous variables [1].

The fuzzy logic implementation in urban drainage systems is a remarkable solution due to the following characteristics: the vagueness of data, the linguistic description of model and the possibility of solving the system, even the information is incomplete or contradictory. The system is robust in that some rules can be left out or can contain errors without seriously compromising performance.

Dealing with the complexity of urban drainage systems, the uncertainty about future flows, and future energy prices, developing optimal pump operating rules is not a simple task. Finding the optimal way to operate the pumps could be an active area of research for many years.

2. Fuzzy Logic Programming

Fuzzy Set Theory is the transposition of the concept that imprecise and/or subjective statements cannot be handled using precision actions. Using this idea, linguistic variables as well as imprecise statements may be described as sets on which mathematical operations can be defined. Owing to the feasibility to transform linguistic, imprecise and subjective expressions into a countable quantity, fuzzy set theory is suitable to help a decision maker, who has to deal with such expressions. Furthermore, when the known relationships are vague and qualitative, a fuzzy logic based controller can be constructed to implement the known heuristics. Thus in such a controller, the variables are equated to nonfuzzy universes giving the possible range of measurement of action magnitudes. These variables, however, take on linguistic values which are expressed as fuzzy subsets of the universe.

A fuzzy subset A of an universe of discourse U is characterized by a membership function $\mu_A: U \rightarrow [0,1]$ which associates with each element y of U a number $\mu_A(y)$ in the interval [0,1] which represents the grade of membership of y in A. The support of A is the set of points in U at which $\mu_A(y)$ is positive [2].

The key ideas are that fuzzy logic allows for something to have a "belongingness" degree to a set or category, numerically described by a membership number between 0 and 1. Fuzzy membership functions can take many forms, as shown in fig. 1, but simple straight-line functions are often preferred. Triangular functions with equal base widths, are the simplest possible and these are often selected for practical applications.

For control purposes, fuzzy categories can be used to set up rules of the following form: "If the value of variable x_1 is *large* and variable x_2 is *medium*, then the result y is *small*". It is claimed that such rules more closely resemble the way we think than explicit mathematical rules do. Fuzzy logic programming can be used in two main ways: as a way of trying to model the behavior of a human expert, and as a way of relating a set of outputs to a set of inputs in a "model-free" way – the fuzzy inference method. To model the thinking of a human expert, input variables are specified by category, such as *large*, and fuzzy rules are developed on the basis of the expert's knowledge and experience. In the fuzzy inference method, sets of input data along with the corresponding outputs are provided to the fuzzy system, and it "learns" how to transform a set of inputs to the corresponding set of outputs through a fuzzy associative map.

Setting up a fuzzy rule base is very simple if there already is a function or algorithm for computing the output from the input variables. In this case values of the output y are computed for all combinations of input variables. The more usual method is to use sample data and derive the rule base from sets of input and output data, by the fuzzy inference method.

Checking the outputs from some systems, it was found that results were significantly better when the rule base was derived directly from the function than when it was obtained by the fuzzy inference method. However, after some trials, it was found that the fuzzy inference method could be improved by giving greater weights to inputs with larger membership functions when setting up the rule base.

Fuzzy control algorithms are constituted of many control rules that will form the rule base of a fuzzy logic controller. To calculate these statements means to evaluate a succession of Min and Max expressions. By choosing the maximal value among these statements the final decision is made. Even in fuzzy control statements, only one value should be chosen to guarantee the steady output. Though fuzzy algorithms are based on fuzzy sets of ambiguous concepts, fuzzy control algorithms must provide only one determined value when a fixed input is applied to a fuzzy control system.

The fuzzy logic controller comprises four main components: a fuzzification interface (FI), a knowledge base (KB), a decision - making logic (DML), and a defuzzification interface (DI) (fig. 1).

The fuzzification interface involves the following functions:

• measures the values of input variables;

• performs a scale mapping that transfers the range of values of input variables into corresponding universes of discourse;

• performs the function of fuzzification that converts input data into suitable linguistic values which may be viewed as labels of fuzzy sets.

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Fig.1 The fuzzy logic controller structure

The knowledge base comprises knowledge of the application domain and the attendant control goals. It consists of a database and a "linguistic (fuzzy) control rule base", which have the following functions:

- the database provides necessary definitions, which are used to define linguistic control rules and fuzzy data manipulation in a fuzzy logic controller;
- the rule base characterizes the control goals and control policy of the domain experts by means of a set of linguistic control rules.

The decision-making logic is the kernel of a fuzzy logic controller. It has the capability of simulating human decision-making based on fuzzy concepts and of inferring fuzzy control actions employing fuzzy implication and the rules of inference in fuzzy logic. The defuzzification interface performs the following functions:

- a scale mapping, which converts the range of values of output variables into corresponding universes of discourse;
- defuzzification, which yields a non fuzzy control action from an inferred fuzzy control action.

3. Fuzzy Control of the Urban Drainage System

The fuzzy logic controller implementation is a remarkable solution due to the following characteristics: the vagueness of data, the linguistic description of model and the possibility of solving the system, even the information is incomplete or contradictory [3].

FCDS (Fuzzy Control Design System) is a collection of powerful software tools, which permits the control system designers to test fuzzy logic specific mechanisms. This development product is made up of several interfaces, functions, procedures, and algorithms that implement particular technics, which are very useful in the configuration process of control applications having as support the fuzzy set theory and the associated concepts.

Using specialized simulation software, the critical points in the system can be found. In these points, in the next step will be placed measuring and output elements.

Fuzzy logic control of a sewer system was tackle with FCDS and with corresponding module WNC-Water Network Control [4]. A friendly graphic interface permits the description of the network topology and some defining parameters (fig. 2). Constructive elements, like inputs in the system, measurements ramifications. pipes, points. overflows, gates and pumps can be used. One selected element is placed in the defined field from the configuration of the system. The specific features of the physical elements corresponding to graphical elements are introduced in "fuzzy controller's structure editor" window. Different types of elements offered by designer are used to build the network. A name and parameters referring to maximum and minimum carried flow are assigned to each object.





Fig. 2 The interface for network configuration

The state of the process is presented in "Control panel" window (fig.3). The operator panel displays the state at different moments, enabling START, STOP, and time recording facilities. Thus, the process signals and the commands transferred to the controller could subsequent be memorized and revised. The control panel could be displayed from the simulation module too, thus we can supervise the process evolution, command elaboration and transmission.

The simulation interface uses a database containing the network structure, control panel, rulebase and the inference engine. For the moment, the commands are executed by an operator that use the software, in manual regime only. The possibility of automatic regulation will be considered in future developments of this software pack.

Fuzzy logic has been used in a number of water resource applications but generally as a refinement to conventional optimization techniques in which the usual "crisp" or "hard" objective and some or all of the constraints are replaced by fuzzy ones. Daily operation of urban drainage system deals with different situations concerning pump functioning. Thus, in storm periods pumps are used to reduce overflows. Also, in order to minimize investment costs when the terrain height varies, pumps can raise the water in pipes, avoiding expensive and difficult deep excavations. In other cases pumps evacuates the excess of rainfall water, which exceed the capacity of sewer toward treatment plant.



Fig. 3 The control panel window

Pumping inflow volumes collected during storm conditions and a time period after, can limit overflows and urban flooding. Knowing that maximum velocity, respectively maximum flow in a pipe is corresponding to a fill degree of 80% and the control rules are similar to:

If (condition = true)

Then (consequence) control rules for this particular case can be founded.

Fuzzy control algorithm for the above described type of structure is:

If In1 is L THEN Out1 is L

where In1 represents measured water flow or level in pipe, and output Out1 is pump command. In this specific case membership function for variable In1 is *Small, Small Medium, Medium, Medium, Medium Large, Large.* Membership function for variable Out 1 is *Small, Medium, Large.*

When two pumps are working to evacuate water on different pipes, the following situations can be identified: • If measured water level (or flow) does exceed the value corresponding to maximum flow, then only one pump should operate

• When water level is closer to full, both pumps should operate

In case of normal operation, without storm conditions, gravitational flow in measurement point is not greater then allowed value and pumps are stopped.

Considering the membership function for input variable In2: VS, S, SM, M, ML, L, VL (*Very Small, Small, Small Medium, Medium, Medium Large, Large, Very Large*) and for output variables: S, M, L (*Small, Medium, Large*), the rules for pump operating in storm conditions are similar to following:

If In2 is VL THEN Out2 is L and Out3 is L If In2 is L THEN Out2 is L and Out3 is S If In2 is M THEN Out3 is S and Out2 is S Sometimes pumps are used for raising the

water in cases of ground with variations in level. Pump operating can be controlled depending on measured water level in a specific point, so that in dry weather periods the pump is stopped. If measured water level at the bottom of the slope is smaller than a value established by design engineer, than pump should not working.

The pump control rules are as follows:

If In4 is M THEN Out4 is L

If In4 is L THEN Out4 is L

where In4 is measured water level, and Out4 is the command of the pump. The membership function used for input and output variables is: S, M, L (*Small, Medium, Large*).

Described fuzzy control logic can be successfully used to command pumps in a pumping station. Let's consider a pumping station with four pumps (fig.3) controlled depending on measured water level. In1 is input variable and Out1, Out2, Out3 and Out4 are output variables corresponding to the four commands of pumps.

If the membership function of input variable In1 is Very Small, Small, Small Medium, Medium, Medium Large, Large, Very Large and the membership function of output variables is Small, Medium, Large, then the control rules are:

If In1 is M THEN Out1 is L

If In1 is ML THEN Out1 is L and Out2 is M

If In1 is L THEN Out1 is L and Out2 is L and Out3 is M $% \mathcal{M}$

If In1 is VL THEN Out1 is L and Out2 is L and Out3 is L and Out4 is L

Thus, when measured water level is higher, the excess of pluvial waters will be evacuated by many pumps, according to specified rules [1].

4. Concluzii

Fuzzy logic is closer by human thinking and by natural language, so is easier to use in many fields. Thus fuzzy logic controller provides an algorithm that can transform the control linguistic strategy, based on an expert knowledge into an automatic strategy. Fuzzy logic controller has better results comparing to conventional controllers, especially in complex systems with uncertain or inexact information. In this context fuzzy logic offers a flexible and promising alternative, easily accepted by operators due to expert knowledge incorporated.

The WNC module is an off-line tool for prepanning phase of real-time control projects, for demonstration purposes of real events and visualization of control possibilities, for education of professionals and students and for personnel training. An optimal control of sewage networks can be simulated and trained by WNC. The user can recognize regularities of the system behaviour and bottlenecks in the system dimensioning. A control of the whole system is based upon the control of local components, which is enabled by measurements or forecast of the rainfall and estimate of the available storage capacities.

The determination of an optimum set of set points is advantageous for a prospective support of critical components, i.e. components of too low capacity, or components which should not be used for discharging. An active control of the system improves its performance. A qualitative forecast is capable to increase this gain. By an extension of the forecast horizon up to the flowtime the whole overflow and flooding could be optimally reduced.

Objectives related to implementation of a distributed control system for urban drainage are to reduce the pollution of the receiving waters from domestic and industrial sewage and to reduce the frequency of flooding caused by malfunctioning of the drainage systems. Finding a cost effective way to realize constant inflow in the treatment plant, meeting in the same time all the objectives proposed is the desired task.

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Mathematical Model for Evaluation of the Flow of a Surface Water Source

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Abstract: The present material describes a methodology and procedure for estimating the low flows when flow records are available. An illustrating example, with detailed steps, is also presented.

Keywords: Low flow, surface water, probability density, and distribution function, extreme-value.

1. Introduction

When designing a water supply system from a surface water source, it shall be investigated the possibility of achieving a scheme of direct intake from the natural river. Obviously the cheapest solution is the direct intake because it does not need to build-up a dam or an embankment for regulating the river. The source development works are not only expensive, but also have great social and environmental impact, such as the necessity of relocating the communities and changing the ecology and environment for natural habitats. For the water supply scheme that catches the flow directly from a natural river, the designer must always determine, with the highest possibly accuracy, the degree of confidence of the water source for the low flow periods, because the intake, the water treatment plant and the distribution systems have to be sized according to the availability of raw water.

2. Mathematical Model

2.1. Estimated low flow

If the water source consists of a not regulated river, the degree of confidence in the availability of the water flow is a function depending on the low flow characteristics. The main three characteristics of the low flow the designer has to deal with are: the period, the magnitude and the frequency of recurrence.

The permissible period of the low flow will reflect the user's tolerance towards the water deficit periods. The low flow magnitude for the specified period will determine the water quantity available for the user. The specified frequency of recurrence of the low flow reflects the associated risk to the failure of the water supply and depends on the social and economical importance of the water supply scheme for the community.

The scope of analysing the low flow frequency is to obtain a graph of frequencies for a certain period (let's say D-days) of low flow. The graph is obtained by adjusting a theoretical frequency to the data recorded for a D-days low flow using analytical or graphic methods. For estimating the degree of confidence for a direct flow used for water supply, the analysis relies on a 7 days low flow. Once estimated, the frequency curve of the low flow can be used for generating the 7 days low flows for any requested reference period.

2.2. Probability density function and distribution function

Consider a graph of the probability density function and the correspondent distribution function.



Fig. 1. Probability density function graph



Fig. 2. Distribution function graph

f(x) represents the probability density function which gives the probability or the relative frequency of recurrence of x. Within the analysis of low flow frequency, x may represent the recurrence of a 7-days (or of a D-days) low flow, which is a random variable denoted by X. F(x) is the corresponding distribution function. The relationship between the two functions is:

$$f(x) = \frac{dF(x)}{dx} \tag{1}$$

or
$$F(x) = \int_{-\infty}^{x} f(x) dx$$
 (2)

It has to be specified that F(x) is a non-decreasing and continuous to the right function.

Also,
$$F(+\infty) = \int_{-\infty}^{+\infty} f(x) dx = 1$$
 (3)

The probability that X is less than x' is:

$$P(X \le x^{\prime}) = F(x^{\prime}) = \int_{-\infty}^{x^{\prime}} f(x) dx$$
 (4)

The probability that X is more than x' is:

$$P(X \ge x') = I - P(X \le x')$$

or
$$P(X \ge x') = I - F(x')$$
 (5)

Also, the probability that X lies between x' and x" is:

$$P(x' \le X \le x'') = P(X \le x'') - P(X \le x') = F(x'') - F(x')$$
(6)

Close related to this is the concept of recurrence interval or return period (T years). The return period of a low flow, q, is the average interval of time (in years) on which a low flow less or equal to q is expected to occur.

2.3. Measured flow

For an intake where daily average flow records are available, annual 7-days low flows can be extracted and denoted by x_1, x_2, \ldots, x_n , where n is the total number of years for which low flows records are available. The following statistical values can be easily calculated:

$$\overline{x} = \frac{1}{n} \cdot \sum_{i=1}^{n} x_i \tag{7}$$

$$s^{2} = \frac{1}{n} \cdot \sum_{i=1}^{n} (x_{i} - \overline{x})^{2}$$
(8)

$$\overline{z} = \frac{1}{n} \cdot \sum_{i=1}^{n} \ln x_i \tag{9}$$

$$s_z^2 = \frac{1}{n} \cdot \sum_{i=1}^n (\ln x_i - \overline{z})^2$$
(10)

Where	$\frac{1}{x}$	is the arithmetic average of the				
		7-days low flows,				
	S	is the standard deviation,				
	s^2	is the variance,				
	7	is the arithmetic average of the				
	~	7-days low flows natural				
		logarithms,				
and	Sz	is the standard deviation of the 7-				
		days low flows natural				

2.4. Method for estimating low flows for measured intakes

logarithms

The following procedure can be used for estimating the 7-days low flow corresponding to a given T-years period, having a record of the observed low flows:

Step 1. Model choosing

A theoretical distribution on which the actual data fit is chosen from the following usual distributions:

- 1. Log-normal Distribution (GIBRAT),
- 2. Exponential Distribution (FULLER-COUTAGNE),
- 3. Gamma Distribution (PEARSON TYPE III),
- 4. Extreme Value Distribution, or
- 5. Gamma Distribution of the annual 7-days low flows natural logarithms (LOG-PEARSON TYPE III).

Step 2. Model estimating

Model parameters estimating, i.e. establishing the most appropriate theoretical model from those mentioned above, using the method of moments for getting the estimates.

Step 3. Model testing

Is the theoretical model chosen at step 2 good enough? That can be verified by one of the two following correlation tests:

- Chi-squared test (χ^2)

- Kolmogorov-Smirnov test.

If the test result is <u>negative</u>, one must go back to step 1, i.e. another theoretical distribution model is chosen. If the test result is positive, one may pass to step 4.

Step 4. Model applying

Estimating the 7-days low flows for the reference period T.

Observation:

Applications have generally proved that the general extreme-value distribution (type III), (known also as EV3 distribution) represents a satisfactory model for analysing the low flows. Therefore, this distribution should be chosen for the first attempt.

2.5. Theoretical presentation of the general extreme-value distribution (EV 3)

2.5.a. General properties

The type III extreme-value distribution (EV3) with the distribution function:

$$(\text{for } -\infty < x < +\infty, \beta > 0 \text{ and } \gamma > 0)$$

$$F(x) = \begin{cases} exp(-(1 - \gamma(\frac{x - \alpha}{\beta}))^{1/\gamma}) & \text{for } x < \alpha + \beta/\gamma \\ 1 & \text{for } x > \alpha + \beta/\gamma \end{cases}$$
(11)

EV3 distribution moments

The mean is given by:

$$E(x) = \alpha + \frac{\beta}{\gamma} - \frac{\beta}{\gamma} \cdot \Gamma(1+\gamma)$$
(12)

The variance is given by:

$$\operatorname{var} \mathbf{X} = \alpha + \frac{\beta}{\gamma} - \frac{\beta}{\gamma} \cdot \Gamma(1 + \gamma)$$
(13)

Coefficient of skewness, defined as:

$$g = \frac{E(X - E(X))^2}{(varX)^{3/2}}$$
(14)

g becomes a function depending only on γ , so that g<1,14 for EV3, as illustrated in the following figure:



Fig. 3. Skewness coefficient g as a function of shape parameter γ

The next table simplifies the calculation:

Table 1. Some data regarding the extreme-value distributions

γ	Γ(1+γ)	Γ(1+2γ)-	g
		$\Gamma(1+\gamma)$	
-0,05	1,031453	0.004727	1.532000
-0,10	1.068622	0.022272	1.903000
-0,15	1.112482	0.060426	2.532000
-0,20	1.164225	0.133763	3.535000
-0,25	1.225413	0.270803	5.605000
0,00	1.000000	0.000000	1.140000
0,05	-0.97350	0.003650	0.911500
0,10	-0.95135	0.013093	0.623041
0,15	-0.93304	0.026906	0.436171
0,20	-0.91816	0.044242	0.255755
0,25	-0.90640	0.064659	0.086610

2.5.b. Estimating parameters using the method of moments

Because the coefficient g of skewness is a function depending only on γ , in order to obtain the $\hat{\gamma}$ estimator for the γ parameter, the selection

coefficient of skewness is calculated first. Skewness g as a function of γ is given in figure 6.1. Then, the replacing γ with $\hat{\gamma}$ in eq. (12) and (13) will give $\hat{\alpha}$ and $\hat{\beta}$ estimators.

2.6. Correlation tests

2.6 a. Chi-Squared Correlation Test

We define k classes inside of which the observations must lie:

$$\begin{array}{c} \text{class 1} & x_0 \leq . < x_1 \\ \text{class 2} & x_1 \leq . < x_2 \\ & \cdot \\ & \cdot \\ & \cdot \\ & \cdot \\ \text{class k} & x_{k\text{-}1} \leq . < x_k \end{array}$$

The selection of classes is more or less arbitrary (as shown below). Then, we consider the statistical test T, a random variable defined as:

$$T = \sum_{j=1}^{k} \frac{(F_j - n \cdot p_j)^2}{n \cdot p_j}$$
(15)

where k=total number of classes

n=total number of observations

F_i=observed frequency of class j

 p_j =probability of observations to lie in class j and where np are the theoretical frequencies of class j assuming that the observations follow the parent distribution.

In accordance with the selection theory the statistical test T approximately follows a Chi-squared (v) distribution, the degrees of freedom, v, being given by the relation:

$$v = k-1-m \tag{16}$$

where m is the number of estimated theoretical parameters from the actual data (these parameters must be estimated for calculating the theoretical frequencies np_j). For this approximation to be valid, must exist at least 5 theoretical frequencies, that is $np_i \ge 5$.

In our example we have only one performing of the statistical test T, given as:

$$t = \sum_{j=1}^{k} \frac{\left(f_j - n \cdot p_j\right)^2}{n \cdot p_j} \tag{17}$$

As the value of t is closer to zero, as is the correlation between the observations and the theoretical model higher. To be noted that also f_i, in the formula above, is considered a performance of F_i. Because the statistic T is considered chi-square distributed, we have that:

 $P(T \leq \chi^2_{\nu,\alpha}) = \alpha$

(with a given α of 0,9 or 0,95 in normal cases).

Therefore, the actual observed value of the statistical test, t, is less than $\chi^2_{\nu,\alpha}$ in 100 α % of selections, when the hypothesis ("observations follow the considered theoretical distribution") is true, in our case 100 α % being that 90% or 95%. Two possibilities might occur:

1st possibility
$$t = \sum_{j=1}^{k} \frac{(f_j - n \cdot p_j)^2}{n \cdot p_j} \le \chi^2_{\nu,\alpha}$$

In this case we can say that the hypothesis is *accepted* at a significance level of $100(1-\alpha)$ %. That does not mean that the hypothesis is true. It only means that within the selection data there is no proof that the hypothesis is false.

2^{nd} possibility $t > \chi^2_{\nu,\alpha}$,

in which case the hypothesis is rejected at a significance level of $100(1-\alpha)$ %. The fact that we find that the t performance of T is more than $\chi^2_{\nu,\alpha}$ may of course be due of the fact that our hypothesis was wrong. Or it may mean that our selection is exactly one of the $100(1-\alpha)\%$ samples for which t proves to be more than $\chi^2_{\nu,\alpha}$, although the hypothesis was true.

Because the interval $100(1-\alpha)$ is small, the *decision* of rejecting the hypothesis is taken at a significance level of $100(1-\alpha)$ %. If assuming this risk is not wanted, that means the rejecting of a true hypothesis, then α should be taken equal to 1, that would mean that $\chi^2_{\nu,\alpha}$ becomes $+\infty$ and then one would never come to the conclusion that a theoretical model fits a given sample data set.

3. Illustrating example for estimating the minimum 7-day flows for given return periods, having a set of data records

3.1. Basic data

For estimating the 7-days low flows for given return periods we used twenty six (26) years of flow records for the river X.

Table 2. Record of the 7-days minimum flows for river X

01 11	
Year	7-days minimum flow (m ³ /s)
1961	19,05
1962	17,47
1963	12,42
1964	24,26
1965	18,77
1966	26,14
1967	19,49
1968	18,45
1969	26,99
1970	20,69
1971	21,50
1972	19,38
1973	26,76
1974	22,00
1975	24,32
1976	22,23
1977	21,56
1978	14,24
1979	16,71
1980	17,58
1981	14,44
1982	13,30
1983	18,93
1984	14,30
1985	18,13
1986	14.70

From the Table 2 the following statistical values can be calculated:

$$\overline{x} = \frac{1}{n} \cdot \sum_{i=1}^{n} x_i = 19,38 \text{ m}^3/\text{s},$$

where x_i is the annual minimum 7-days flow.

$$s^{2} = \frac{1}{n} \cdot \sum_{i=1}^{n} (x_{i} - \overline{x})^{2} = (4,0824 \text{ m}^{3}/\text{s})^{2} = 16,66 \text{ (m}^{3}/\text{s})^{2}$$

$$\overline{z} = \frac{1}{n} \cdot \sum_{i=1}^{n} \ln x_i = 2,9415$$

$$s_x^2 = \frac{1}{n} \cdot \sum_{i=1}^n (\ln x_i - \overline{z})^2 = (0,2142)^2 = 0,0459$$

summarized in a table:

$\frac{-}{x}$	s^2	- z	s_x^2
19.,38	16,6662	2,9415	0,0459

Skewness coefficient g, defined as

$$g = \frac{E(\frac{1}{n} \cdot \sum_{i=1}^{n} (x_i - \overline{x})^3)}{s^3}$$

with: $E(\frac{1}{n} \cdot \sum_{i=1}^{n} (x_i - \overline{x})^3) = a_1 - 3a_2a_1 + 2a_1^3$ where $a_j = \frac{1}{n} \cdot \sum_{i=1}^{n} x_i^j$ for j=1,2,3

It results that:

$$a_{1} = \frac{1}{n} \cdot \sum_{i=1}^{n} x_{i} = \overline{x} = 19,38m^{3} / s$$
$$a_{2} = \frac{1}{n} \cdot \sum_{i=1}^{n} x_{i}^{2} = 392,15$$

$$a_3 = \frac{1}{n} \cdot \sum_{i=1}^n x_i^3 = 8258,80$$

 $a_3 - 3a_2a_1 + 2a_1^3 = 16,85$

$$s^{3} = (4,0824 \text{ m}^{3}/\text{s})^{3}$$

 $\Rightarrow g = (\frac{16,85}{64,04}) = 0,2476$

$$\hat{\upsilon} = (\frac{2}{g})^2 = 65$$

3.2. Extreme-value general distribution

3.2.a. Estimating parameters

Skewness, g, as calculated, is 0,2476. In accordance with Fig. 3 and Table 1, it results that the type 3 extreme-value distribution with v>0 must fit actual data.

Moment estimators are given by Eq. (12) and (13).

From Table 2, with g=0,2476, it results:

$$v =$$
0,20475 $\Gamma(1+v) =$ -0,91704 $\Gamma(1+2v)$ - $\Gamma(1+v) =$ 0,04618

$$\bar{x} = 19,38 = \hat{\alpha} + \frac{\hat{\beta}}{0,20475} + \frac{\hat{\beta}}{0,20475}(-0,91704)$$
$$s^{2} = 16,6662 = (\frac{\hat{\beta}}{0,20475})^{2}(0,04618)$$

$$\Rightarrow \quad \hat{\alpha} = 17,804 \text{ m}^{3}/\text{s}$$
$$\hat{\beta} = 3,8897 \text{ m}^{3}/\text{s}$$

3.2.b. Estimating the minimum flow

The 7-days minimum flow with return period 50 years (q_{50}) results from Eq. (11).

$$F(q_{50})=P(X \le q_{50})=1/50=0,02$$

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$$exp(-(1-0,20475(\frac{q_{50}-17,804}{3,8897}))^{\frac{1}{0,20475}})=0,02$$

 \Rightarrow q₅₀=11,68 m³/s

The 7-days minimum flow with return period 30 years (q_{30}) is:

 $F(q_{30})=1/30=0,0333$ $\Rightarrow q_{30}=12,39 \text{ m}^3/\text{s}$

3.2.c. Graphical solution

Annual minimum 7-days flows are ordered (from the highest value to the lowest) and the unbiased positions to be plotted on the graph are calculated as shown in Table 3. The ordered data are plotted on a log-Gumbel extreme-value graph paper. The plot position formula is:

 $P = \frac{i - 0.44}{n + 0.12}$ where P = overflow probability i = order number n = selection size (in this case n=26)

A best-fit straight line is drawn. The 7-days minimum flows with return periods of 50 and 30 years resulting from the graph are:

 $q_{50}=10.8 \text{ m}^3/\text{s}$ $q_{30}=11.8 \text{ m}^3/\text{s}$

Table 3. Data and plot positions represented on a graph for EV3 distribution

graph for L v 5 distribution					
Ι	7-days minimum	Plot positions			
	annual flows (m^3/s)	i-0,44			
	Xi	n+0,12			
1	26,29	0,0214			
2	26,76	0,0597			
3	26,14	0,0980			
4	24,32	0,1363			
5	24,26	0,1746			
6	22,23	0,2129			
7	22,00	0,2511			
8	21,56	0,2894			
9	21,50	0,3277			

10	20,69	0,3660
11	19,49	0,4043
12	19,38	0,4426
13	19,05	0,4809
14	18,93	0,5191
15	18,77	0,5574
16	18,45	0,5957
17	18,13	0,6340
18	17,58	0,6723
19	17,47	0,7106
20	16,71	0,7489
21	14,70	0,7871
22	14,44	0,8254
23	14,30	0,8637
24	14,24	0,9020
25	13,30	0,9403
26	12,42	0,9786

3.3. Testing the type III extreme-value distribution (EV3)

 $(\alpha = 17,804, \beta = 3,8897, \nu = 0,20475)$

5 classes have been selected, the estimated data being shown in Table 4.

distribution							
Class	Abs. freq. f _j	Th. freq. np _j	$\frac{(f_j - np_j)^2}{np_j}$				
. ≤15,80	6	5,08	0,1666				
15,80<.≤18,10	3	5,22	0,9441				
18,10< . ≤20,20	7	5,19	0,6312				
20,20<.≤22,80	5	5,26	0,0129				
22,80< . ≤36,80	5	5,24	0,0110				
Total	26	25 99	1 7658				

Table 4. Chi-square correlation test for EV3

np_j values are calculated thus:

 $p_1 = P(X \le 15,80) = F(15,80) =$

 $= \exp(-(1-0.05264(15,80-17,804))^{4.884})$ (in accordance with Eq. (11)) = 0.1955

 \Rightarrow np₁=26 x 0,1955=5,08

Also,

 $p_2 = P(15, 8 < X \le 18, 1) =$

$$= P(X \le 18,1) - P(X \le 15,8) =$$

=F(18,1)-F(15,8)=
=0,3961-0,1955=
=0,2006
np₂ =26 x 0,2006=5,22

v=k-1-m=5-1-3=1

 \Rightarrow v=1, that is statistical test T follows a chisquare distribution with one degree of freedom.

From Table 4, t=1,7658

 \Rightarrow t=1,7658< $\chi^2_{1:0.9}$ (=2,71)

So the EV3 distribution is accepted at a significance level of 10%.

4. Conclusions

- EV3 distribution is acceptable for the given set of data at a significance level of 10%.
- 7-days minimum flows with the given return periods are:

7-days minimum	7-days minimum flows
flows analytically	graphically resulted
resulted from EV3	from EV3 distribution
distribution	
11,68 m ³ /s	$10,80 \text{ m}^3/\text{s}$
$12.39 \text{ m}^{3}/\text{s}$	$11.80 \text{ m}^{3}/\text{s}$

• Type III extreme-value distribution (EV3) is normally a satisfying model for the analysis of low flows. The graphical solution allows a quick but raw check because the method of fitting the "best" straight line by eye is subjective and the results obtained on the same graph by different persons may differ sometimes by over 20%. In our case, the 7days minimum flows with 50 and 30 years return periods obtained by the graphical method are smaller then the calculated values with 7,5 and respective 4,8%. The graphical method is expected to offer reasonable results when there are not enough data for statistical analysis. As a basic rule, for estimating the low flow with a return period of 50 years, 20 years of data records are necessary.

• The graphical solution of the EV3 distribution may give a quick but approximate estimate for the 7days (or D-days) low flows with given return periods. This method is usually used when not enough data (let's say less then 20 years of records) are available for a statistical analysis.

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A General Strategy to Control Water Losses from Water Supply System is a Necessity

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Abstract:

Water supply system is essential for localities good live. Unfortunately existing systems are old and water losses are important, in our country about 20-50%. To rehabilitate this system, especially distribution network is necessary a lot of money. A study case is developed. Using water tariff is possible to rehabilitate the distribution system in economical conditions.

Key words: water supply system, water losses, network rehabilitation

1. Introduction

Water supply system must supply the consumers respecting three simultaneous conditions: water must have drinking water quality, the supply must be continuously and the cost must be moderate (not so high that consumers couldn't pay it, not so cheap that consumers try to spread it). Because the consumer pays all water, he is interest that this water is rational utilized. A specific consumptions like 150...200 l/hab/day is normal.

But, like each element of constructions, water supply system age. If it is not continuously rehabilitated, step-by-step, its deterioration will be great and water lost can be so important that management cost can be exaggerated, quality of water can be worse and possible, dangerous for use. Unfortunately, water supply systems, in our country, are in a difficult situation - leakage being 25...50%. That means poor water sources utilization (we are not so reach in water in the sources) and spread of a very important quantity of energy (more and more expensive); about 20...60% of water delivered to the population cost is energy cost. So a global strategy to control this leakage is necessary, fundamental one for the normal suppliers development.

2. Aspects of financing water supply systems

About the technical leakage subject were made a lot of discussions /1,2,3,4,5/. The general conclusions were: the pressure of water and dimension of orifices are most important factors influencing the leakage measure; to control the leakage needs a lot of money.

Because distribution system is the most developed water supply object and it is working in a very difficult conditions (pipes support a continuously fluctuation pressure, traffic vibration, etc), is about normal that great part of leakage is produced in the localities. So a normal effort must be done to control the network.

According to some information supplied from Germany, to obtain a fantastic percent of 5% leakage, spent over 60% of amount of about 30 USD/capita, year to control this network.

When the leakage is so important, over 20%, an energic method "to repair" the network is important to be developed. Unfortunately, because the available money is so little, (we have no money - is the general and usual formula of a great part of supplies from our country), a very easy method is used "to mend" the great discovered leakage. Easy to repair, but unfortunately a lot of money is possible to be spent and the final results can be about the same. Really this method has only one advantage: if all spend some money are calculated is possible to establish, time to time cost of repairing and produce an important support to make a decision: when the cost of repairing is more important than a new pipe cost,

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pipe must be changed (or rehabilitated by relining method). In the worse situation, is possible to raise water tariff to cover the cost of leakage.In each method the consumers must finally pay. In this moment is important to demonstrate that for rehabilitation chosen method, the cost is minimum and the difficulties during this works are acceptable.

Development and good management of water supply system are complicated problems: that means <u>investment effort</u> – not so easily to do in the economy heaving a low resources, <u>social aspects</u> – to have good water it is a right of each people, <u>environmental aspects</u> – rational water sources utilization, control of water quality etc. /7,8,9/.

So far undertakers are not so easily to decide something.

To have a dimension of the finance effort a short and very simple case study will be shown. Basically elements:

- a locality heaving 100.000 people
- specific consumption 300 l/cap/day (with normal leakage 10%, accepted); all quantity of water is necessary for consumer – 270 l/cap/day
- distribution system length (pipe) 300 Km
- average pipe's diameter 200 mm
- real leakage 40%
- the percent of energy in the water tariff 40%
- investment in a new pipe, DN = 200 mm, 2 mil lei/m
- pipe's life 50 years
- to invest is possible to loan without interest (very important but not realistic)
- now, all water distributed is recovered, by tariff (really this percent is about 40...90%).
- The leakage is uniform proportional with length of pipe; so if 30% of length of pipes will be changed the leakage will be in the "normal" limit – 10%
- The water tariff and energy cost will be constant for years (it could be fain)

Basically values:

- water to supply the system
 - $30,000 \text{ m}^3/\text{day} \cong 10,95 \text{ mil m}^3/\text{year}$ water for population – "normal"
 - $0.9*10.45 = 9.85 \text{ mil m}^3/\text{year}$

"real" $0.6*10.45 = 6.27 \text{ mil m}^3/\text{year}$

- volume of water losses "abnormal" $0.3*10.95 = 3.28 \text{ mil m}^3/\text{year}$
- yearly water "value" by tariff 10.000 lei/ m³ * 10.95 = 109.5*10⁹ lei/year
- yearly of water losses value "out of norm" 10.000 lei/ m³ * 3.3 = 33*10⁹ lei/year
 - yearly lost energy cost
- $0.4*33*10^9$ lei/year
- investment for a new pipe (30% of all pipes length)
 - new pipe's length 0.3*300 Km = 90 Km
 cost of new pipes = 10⁶ lei/m * 90

 $Km = 180*10^9$ lei / an

-yearly cost (cost of amortization)

 $= (1/50) * 180 = 3.6 * 10^9$ lei/an

Partial conclusions

- we can compare: leakage "cost" 33*10⁹ lei/year - investment for a new
- pipe 3.6 * 10⁹ lei/year - leakage energy cost 13.2

*10⁹ lei/year

- reduction of abnormal losses, 33 mld lei/year "will cover" the investment cost for a new pipe, 180 mld lei, in about 5.5 years; a very profitable solution <u>Remark 1</u>

Following the results of pressure test, of pipe from PEID, realized in the MUDP II program, is possible to noticed that leakage is very small, if the realization is right / Manual of leakage control – 2000/. For example: pipe heaving 200 mm diameter, tested at 15 bars, after one hour the pressure was reduced with 0.3 bars; water leakage, calculated by normal formula from EN 805 is about 0.0021 m³/h*Km or about 18.4 m³/Km*year. If we compare with actual leakage in the studied case (3.3 mil m³/year/90 Km = 36.670 m³/year*Km) we can notice that the leakage reduction is about total, 99.9 %.

Remark 2

By standard EN 805, during pressure proofing test, water lost must be maximum.

$$\frac{\Delta V}{V} = 1.2 * A_p (\frac{1}{E_p} + \frac{D}{E * E_a})$$

where:

ΔV = maximum water lost volume, measured by water introduced in the tronson to obtain the initial test value of pressure, liters

= volume of water contained in the V tested pipe tronson

= static pressure diminuation in one Δp hour, kPa

= water's elasticity modulus, KPa Ea

Ec material's pipe elasticity modulus, KPa

> = pipe's interior diameter, m D

Ε = pipe's wall thickening, m

1.2 = correction factor (residual air in the pipe)

3. Money from rehabilitation, from where?

Generally and very brutally speaking, there are three sources:

loan from an external bank; loan value will be investment value plus interest rate (great in this moment)

- allocation from budget, difficult to obtain, low value but without interest rate;
- internal resources; tariff improving for a short duration; when a good some was realized is possible to use this for rehabilitation starting; is something like MRD founds realized by Regias involved in MUDP II programmed; is a better solution because a minimum cost is involved and finally all borrowed money must be paid by some consumers, by tariff.

To follow what happens we can continue the previous study case. The values are contained in the table 1, in the next hypothesis:

- the loan without interest rate (interest rate is zero; in another case the calculation can be done)
- investment duration is 5 years
- total investment will be 180 mld. Lei _
- water tariff is the same 10.000 lei/m³
- leakage reduced is proportional with rehabilitation of pipe's length
- adapted solution is to change all 120 Km who produced 40% leakage; the calculation is realized by a new pipe realized in free tranch (another solution can be adopted)

Indicator	Measure	year						
	units	1	2	3	4	5	6	7
Annual investment	10 ⁹ lei	36	36	36	36	36	-	-
Water losses	$10^{6} \text{ m}^{3}/\text{an}$	8.3	2.64	1.98	1.32	0.66	-	-
Water losses cost	10 ⁹ lei /an	33 x	26.4	19.8	13.2	6.6	-	-
Recovered water	$10^{6} \text{ m}^{3}/\text{an}$	-	0.66	1.32	1.98	2.64	3.3	3.3
Recovered water cost	10 ⁹ lei/an	33 (6x)	6.6	13.2	19.8	26.8	33	33
Total recovered water $cost(7x)$	10 ⁹ lei/an	69	62.4	55.8	49.2	42.6	-	-
Water delivered (2x)	$10^{6} \text{ m}^{3}/\text{an}$	4.4	5.06	5.76	6.38	7.04	7.7	
Delivered water tariff	lei/ m ³	15.680	12.330	9.755	7711	6050	6000	6050
								(3x)
Some needs over normal water	10 ⁹ lei/an	25	11.8	-	-	-	-	-
tariff								
Supplementary income								
- for each m^3 (4x)	lei / m ³	-	-	245	2290	3950	4000	4000
- for all network (5x)	$10^9 { m m}^3/{ m an}$	-	-	1.4	14.6	25.2	28	28
Remarks:		3x	-	without	cost o	f netwo	rk nor	mal

Table	1:	Fluctuation	funds	values	during	the	network	reha	bilitat	tion

Remarks:

- any quantity of water is not recovered but Х consumers must pay water losses (33 mld.lei/year) - 4.4 mld.m3 /year obtained from network 2x

supervising - differences between tariff and real cost 4x

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plus recovered water

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5x - the sums that can be utilized to start a new work of water supply general rehabilitation, development and modernization

6x - water losses cost

7x - water losses cost plus yearly investment Other remarks:

- We can notice that except the first 2 years, when consumers must pay investment plus water losses cost, the solution is advantageous, if we judge this values year by year

- We can see that after investment ending is possible to reduce water tariff or is possible to reinvest the sums obtained

- Is possible to discover a minimum effort of investment that after first 2 years the spare sums can be invested in the next year, like a chain belt

- If possible to adopt a mixed technical solution (a part of pipes in free tranch, a part from tranchless, a part from relining etc).

4. General conclusions

a – Water losses management must be a continuously preoccupation; so a general strategy must be developed in a specific manner because each water supply system is an unique one.

 $b-\mbox{The}$ advantage of the general strategy of leakage management:

- a rational way to save water resources
- a good relationship between consumers and supplier
- a measure of environment protection
- is an economical measure one; saved money can be used to improve technical and economical performances of another water supply objects, personnel training, automation

c – Because the strategy needs a large amount of money is necessary a strategy to:

- choose a better solution to find out the money
- choose a better solution for pipe's material and building realization
- prove by real values that the solution is economical one

d – To choose an economical solution is necessary:

- better knowledge about real performances of the system

- a good quantification of performances indicators; the real cost of rapid remediation to fight against leakage is essential /6/

e – Because a lot of parameters (about all of them) are not constant values (like in the previous example) two elements are important:

- a tendency estimation of values evolution (a lot of them very difficult to be estimated – for example invested rate, inflation rate, energy cost, etc)
- a strategy periodical reevaluation (new methods and technologies can be developed, a new stage of undertaker charge must be established, etc).

f - Romanian Regia must renouvelate their network to bring in a very short time water losses in the normal limits – about 10% instead of 20...50% existing now. That means to renouvelate about 20-30.000 Km pipes in the next 10-20 years.

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Business Plan – The Instrument of Substantiation of the Investing Decision

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The Business Plan represents the main source of information and analysis for the creator, in its way for creating its own firm, the basic tool in substantiating the investment decision. Its purpose is to guide with the object of finding the shortest way, the most efficient and the cheapest.

Keywords: The idea of business, market, management, firm, profitableness .

1. Creator and business ideea

A firm creating project, is in fact a HUMAN/PROJECT couple:

- the project, involve a number of constitutive elements, coherently interrelated;

- the human, must have coherent competences and qualities in relation with the project and adapted to its necessities: for example, we cannot imagine a hairdresser creating a construction firm without being afraid for his future.

1.1. Creator Profile

- The creator must:
- be a tenacious person,
- be prepared to fight whenever the hope exists,
- have spirit of initiative,
- assume responsabilities up to the succes,
- to show enthusiasm and capability to communicate easily even when he is not in good moods,
- to make quick decisions, in uncertaine situations, in insecure conditions, when he will be obliged to give proof of adaptability and flair.

1.2. Business Idea

The idea of creating a firm appears and stands

In this case creating is more complex because beside the difficulties of a new firms, must be added

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out after some direct observations about the market's needs for products and services, based on preceding The idea of creating a firm appears and stands out after some direct observations about the market's needs for products and services, based on preceding studies regarding the future fabrication technologies evolution, based on informations regarding the needs, the nature and the importance of the future customers. The creator will be aware that an idea, without a market or conceived to answer to some unsatisfied needs will not be viable.

A. How Do We Get an Idea? What Are the Risks?

A.1. Create begining from a known activity

Everyone, based on the professional or extraprofessional experience, has a competence. The demand is enough for permiting to a new firm to find its position within the market. The product or the service is known. The creation of the firm occures in a domain within the creator feels fine. The risk is related on the creator competence, his credibility in the eyes of the costumers and the suppliers.

A.2. Create applying old technologies to new activities

those related with the introduction of a new product on the market.

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652 <u>Business Plan</u> – The Instrument... / Ovidius University Annals Construction 3,4, 651-656 (2002)... A.3. Create begining from an inovation

That means departing from a new product resulted from a scientific research or as result of the market's demand.

B. Where Do We Found the Idea of Business?

It results from the observation about:

- the work environment,
- everyday life,
- economic life,

- capitalizing the others ideas (the ideas belonging to others).

C. How We Pass from an Idea to a Realistic Project?

C.1. Accurately defining the idea:

- What does it want to sell, product or service?
- To whom does he want to sell? (clients)
- C.2. Collecting the information:
- Related to what he wants to sell: product or service
- Related to whom he wants to sell: clients
- C.3. The specialists advices:

Because they look from exterior they are more realistic than the creator.

C.4. The conditioning analysis

The conditionings of any kind:

- money,
- project size,
- management,
- family,
- competition,
- health, etc., can they be overcome?
- C.5. Is the idea realistic?
 - A list will be made containing:
- the motivations,
- the formation,
- experience,
- competence,

and the coherence with the idea will be verified. *C.6. Outlineing the project:*

- What is sold and to whom?
- What type of firm will be created?
- Which will be its activity, its vocation?
- Will it need personal stuff, equipments?
- What strategy will follow?
- Are there competences?

2. The market study

2.1. Construction Market in Romania

A. The Beneficiaries Nature

Beneficiaries can be:

- a) State's Organisms Townhalls, County Departments of Roads and Bridges, National Roads Administration;
- b) Joint firms;
- c) Private firms, private persons;
- d) Other private firms in regims of subcontracting.

They could be structured, for example, in accordance with the business figure like following:

B. Information about the Competition

Vital in informations for the creator are:

- present situation of the competition;
- number of competitors existing in the influence zone of the future firm;
- rithm of appearance and disappearance of the concurent firms in the zone;
- the price policy;
- the diversification plan;
- the development strategies;
- promotional policy;
- the conception about the servicies and the production;
- the financing;
- the managers paying.



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 Firm's Is strongly influenced by: advertising; firm's stuff behaviour; manager's behavidiplomacy in business; respecting the appoint the works, respecting the prices; displaying a self – rest aspect, showing respecting the partners. 	; our and ed terms for quality and spected firm ct for the e business	Account and financial administration quality	 The creator, as well as the future manager must have capabilities: finding financing sources; atracting funds from the beneficiaries; getting information regarding the prices of the concurence; constant controling the execution. 	
 C. Information about the Suppliers Their reliability will be esimate if with: competitiveness; Products and services performances, which are The creator information about: • modern work teg and technologies 	in accordance will get chniques s;	Human resources administration	 Is closely related to: worker's competence; foremen competence; the climat inside the firm; stuff stimulation: salary, prestige, security, status. 	
 determined in accordance with: price; quality; rapidity of the carrying out. performant equi materials quali complying with construction organization's q competitivenes performers. 	pments; ty and the the norms site uality of the	Methods for atracting the beneficiaries	 firm's image <= quality <= seriousness; previsioned prices smaller then those of the concurence; publicity policy; the policy of crediting the beneficiaries. 	
 promptitude; services qualitytheir relatively imported the firms; the importance of the firm for them; their own conditions of buying, selling; the strategic behaviour; the situation toward the concurrents. 2.2. Trumps and weaknesses of the Against the Concurrence	3. , production, <i>Future Firm</i>	Financial foresee Management	 Fing The manager's portrait: solid professional knowledge; competence, capability to organize, control, animate the colaborators; personality, will, desire, morality; the aptitude to find and 	
Kev factors of succes Creators Trum	select good workers, in accordance with the labour			

Key factors of succes Creators Trumps

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market within the firm's activity's zone.

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3.1. Financing Necessary

A. Creating Expenditures

Constitution expenditures, creating formalities expenditures, as well as organize physical activity expenditures will be estimated, and they will be considered immobilisations to be amortized in time.

Physical Organize of Activity

A repartition of the responsabilities is made and it will be settle the structure of the personal stuff needed:

- qualification specific to the future activity;

- efectives considered to be necessary;
- posibilities of using temporary stuff;

- cost of searching, selecting and forming the productiv personal;

- level of salaries.

B. Expenditures with Immobilizations

Uncorporal immobilizations can be: patents, trade funds, etc.

Corporal immobilizations:

- Buildings:
 - the residence of the future firm;
 - site organizing objects needed;
 - calculation of the necessary surfaces.
- Teraines:
 - production base organization;

- necessary surfaces calculation;
- extending posibilities.
- Machines, equipment, bureaux materials:
 - nature and number of the necessary
 - equipments and vehicle;
 - delivery terms;
 - transportation costs.

C. Other Expenditures

Financing an exploitation cycle, the need for circulating fund respectively, NCF:

NCF = (Stocks + Credits given to the clients) - debts given to the suppliers

3.2. Mode of Financing

The sources for financing the investments: own capital, OCP:

$$OCP = SC + FR$$

- where: SC = social capitalFR = other financial resources
 - permanent capital, PC:

PCP = OCP + Co

where: OCP = own capital Co = initial credit



5. Cash – Flow value updating methode "NUV"

The Net Updated Value is a notion which results from a very simple judgement, to compare the result of a firm creating project with its cost, analogy made in faze 0.

Net Updated Value is calculated as a difference between the sum of the present values of the cash – flows calculated using the updating rate

"u", conveniently chosen, for the 5 years previsioned and its cost:

$$\begin{split} NUV &= C_1 \; (1{+}u)^{-1} + C_2 \; (1{+}u)^{-2} + \; + C_5 \; (1{+}u)^{-5} - I_0 \\\\ NUV &= \sum C_p \; (1{+}u)^{-p} - I_0 \; , \quad p = (1,2,\; ...\;,5) \end{split}$$

- if NUV > 0, the initial funds are returned and get an excedant. The firm creating project can be accepted.
- if NUV < 0,it will not returned the initial funds invested, taking into account the time needed for getting them.

The firm creating project is unfavourable.

The "rentability" is a notion which must be disociated from the notion of "profit" of the traditional economic analysis. Essentially it must be understood as an analogy between: obtained results / used means.

The starting point of determining the responsability is represented by the technical "image" of the economic activity, expresed by the notion of productivity: outputs / inputs.

The productivity, even if it is expressing the need to create wealth, to issue something in addition, remaines however an element of analysis pure technical. Nevertheless, the economics have become monetary, consequently the economic and financial activity of the firm will be measured not in tones but in monetary units. The physical tide of inputs and outputs will be valorized by a system of prices. The monetary value of the definitive resources will be the global income of the firm, and the monetary value of the utilizationwill give a measure of the expenditures.

The rentability will be easily revealed by the excedend between the ultimate resources and the ultimate debts.

In other way the analyse of the rentability consists in using the cash – flow notion. The economical activity of the firm, within a system of given prices reduces itself to inputs and outputs of cash. One of the methode of estimating the rentability is to compare the "cash – in flows" with the expenditures flows, "cash out flows", the difference between the two of them being the cash excedent or "cash - flow".

6. Creating the firm

The creation of a firm is first of all a mobilization of resources, people, energy, competence, know – how and finances for issuing a profit. Very often the creator has no clear view of the importance of the juridical aspect in his enterprising.

Selecting the juridical structure for the begining, gives a colour of tomorow, therefor, for avoiding cloudy days the project must be examined in all its aspects, for supplying the project with a proper and adapted structure wich will give the creator the means for his action and ambition.

The *personal criteria* of the social status, responsability, power, desire of self achievement as well as the *pure economical imperatives* must be examined.

"Win and then organize" is a caricatural advise for a firm creator. For sure, he will not be succesfull if his action will not be in accordance with a predesigned scenario, represented by the "Business Plan", wich, as we have underlined is a detailed analyse of the business and serves as a documentation for the objectives, strategies and tactics of the firm.

His achievement does not however guarantees the succes.

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Respiratory Apparatus for Emergency Situations

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Abstract: This work develops a very interesting subject for the people who are involved in underwater activities. It is about a respiratory apparatus used in insecure conditions indoor of underwater vehicles and installations (submarines, small submersibles, hyperbaric chambers and caissons). The operation principal is based on the carbon dioxide adsorption by a soda lime filter calculated to assure enough time of efficiency to not jeopardize the men condition and to avoid the effects of intoxication with this dangerous gas. The apparatus was designed in our laboratory and its construction was carrying out entirely with local materials.

Keywords: respiratory system, soda lime filter, and carbon dioxide absorption.

1. Introduction

This respiratory apparatus was designed to equip the underwater vehicles and specialized installations that the diving teams are using in their particulars missions.

It is known as "security apparatus" and it is working in normobaric conditions (atmospheric pressure) and also in hyperbaric conditions (the working pressure that is directly dependent on the depth). The soda lime filter that the apparatus is fitted retain the carbon dioxide from the ambient air and it is working at equal pressure by removing the cover which protect the soda lime to be inactivated by the moisture absorption.

After using it, the cover is refitted on its place. The most important feature of this filter consists in its operation autonomy that was very carefully projected considering the large number of factors, which are involved. It was considering a minimum autonomy to allow the appropriate length of emergency procedures necessary to surfacing.

After surfacing securely the charge of filtering substances used during these operations will be

changed at each apparatus and so, these will be prepared for other unforeseen situations, when the ambient regeneration systems aren't work.

2. Established operation conditions

On the underwater vehicles, as well as on the chambers or caissons needs to be carrying out certain typical conditions for the security of the human crew.

The most important in this case is to maintain the gas concentration in the internal atmosphere between the normal limits specific to the respiratory air. That is to be carrying out by the own regeneration systems of these vehicles. But in the hazardous situations when the regeneration systems fail, the personnel have to use their additional respiratory apparatus that assure them the possibility to surface securely. It is well known the toxicity of the carbon dioxide accumulation in the human tissues. That for is very important to maintain the carbon dioxide concentration (or partial pressure = P_pCO_2) under the extreme limits of the human tolerance, which are as it follows:

- at $Pp_{CO2} = 6$ mb: unlimited, optimal exposure;

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- at $Pp_{CO2} = 10$ mb: unlimited, maximum and hygienically exposure in chambers;
- at Pp_{CO2} = 20 mb: 8-24 h (the max. value allowed in diving activities);
- at $Pp_{CO2} = 30$ mb: 2-8 h;
- at $Pp_{CO2} = 40$ mb: max. 20 mb (the 30 mb and 40 mb values are allowed in diving when the exposures are short and accidental);
- at $Pp_{CO2} = 50$ mb: max. 30 min. (allowed in emergency cases in chambers, bells, other vehicles or underwater installations).

Other particulars established conditions are:

- The gas flow, which can pass trough the filter of the apparatus, is imposed by the medium value of the human respiratory flow on repose, Q=6/min. and also by the medium value of the respiratory flow on medium effort, which is between (20-30) l/min.;
- The operation pressure: the internal atmosphere of the vehicles and installation named above is very well controlled, so that it can work both at the atmospheric pressure (1 bar) and also at the pressure corespondent to the work depth.
- The range of the operation temperature, which is increasing in proportion as the depth increasing

must be: $(-4^{\circ}C \div +30^{\circ}C)$;

- The partial pressure of the oxygen is strongly recommended to have the values between the

lowest limit of hypoxia ($Ppo_2 = 400mb$) and the maximum limit of CO_2 toxicity ($Ppo_2 = 700mb$);

- The appropriate moisture concentration in the respiratory air must be not greater than the range of values 60° H- 80° H, to not affect the CO₂ absorption capacity by the soda lime filter.

3. Materials and other equipment components

The complete assembly has the following components made of specific materials allowed in diving, materials that in direct contact with oxygen under pressure are not flammable or explosive:

- facial masque (moulded rubber);
- inhale-exhale valves system (moulded rubber);
- valves support (polyamide);
- special gofer hose (rubber);
- cylindrical body (polyethylene);
- filter element (polyamide);
- charge of filtering active substances;
- two circular filter of special paper or textile material;
- protection plug.

The apparatus is shown in the Fig.1, below:



Fig.1

4. Results and interpretations

The main operating parameters of the respiratory apparatus that we have achieved are the following:

- Operation autonomy......4 6 hours;
- Rate of CO₂ absorption......20% weight;
- Tightness at overpressure.....max.35 cm H₂O;

- Materials that the components are made of, respect the prescriptions for the operation with oxygen under pressure.

There is some operation restrictions to assure the correct function of the apparatus and the security of the human personnel:

- CO₂ partial pressure.....max.10mb (max. allowed in diving);
- Relative air humidity.....70 80⁰H;
- Operating temperature..... $(-4^{0}C \div +30^{0}C)$;

5. Conclusions

Considering the high risk of the underwater work, the sealed precincts of the underwater vehicles and installations are fitted with a number of these respiratory apparatus equal to the number of the crew members and the personnel are to be instructed for using them in critical situations described above.

Each apparatus is to have its own list of periodic checks and operating instructions (starting with transport, storage, preparation for use, periodic control and change if necessary of filtering charge, filter checks, etc.).

The transportation to the work location it is made using special transportation bags made of resistant materials to avoid the moisture and the high temperatures, which can inactivate the filtering substances.

The storage is also a very important aspect and has to be done carefully, in dry, closed spaces, out of direct sunlight and cold. There are a number of elements that must be replaced periodically such as

O-rings, the charge of soda lime and active carbon, rubber valves, and paper (textile) filters.

The diving crews will use this apparatus as an additional device in case of emergency to succeed in completing securely theirs tasks.

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6. Thanks

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Water Transfer modelling in a High Valley Equipped with Hydropower Plants: Rio Zongo Valley (Bolivia)

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Abstract: Pour modéliser les écoulements dans une vallée de montagne aménagée pour la production d'hydroélectricité, une double démarche est proposée. Il s'agit, dans un premier temps, de mettre en œuvre un schéma de surface développé par Météo-France afin de représenter la redistribution des précipitations neigeuses et pluvieuses. Dans un deuxième temps un modèle de transfert décrivant la superposition d'un réseau hydrologique naturel avec un réseau hydraulique complexe est développé en utilisant les techniques de la dynamique des systèmes. Le résultat obtenu est très encourageant et permet d'envisager une utilisation opérationnelle d'une telle méthode par les opérateurs de l'eau en haute montagne.

Keywords: mountain hydrology, modelling, hydropower, system dynamics, Andes

1. Introduction

L'Institut Français de Recherche pour le Développement (IRD) conduit depuis 1991 des recherches dans la Cordillère des Andes (Bolivie, Chili, Equateur et Pérou) afin de mieux comprendre l'impact de la variabilité climatique sur les ressources en eau de très haute montagne et d'en évaluer les conséquences sur leurs usages. C'est dans ce cadre qu'une étude fine a été entreprise en collaboration avec la *Bolivian Power Company* (Cobee) afin de modéliser les transferts d'eau dans une haute vallée équipée de plusieurs centrales hydroélectriques en cascade, ainsi que d'un réseau complexe de retenues et de canaux de captage [1].

La vallée du Rio Zongo se situe à une trentaine de kilomètres au nord de la capitale de la Bolivie, La Paz. Elle descend depuis les sommets du Huayna Potosi (6088 m), du Condoriri (5648 m) et du Charquini (5392 m) et rejoint les cours d'eau plaine amazonienne. de la Dix usines hydroélectriques sont installées en cascade totalisant une puissance totale de 174,6 MW (Fig. 1). La présente étude s'intéresse au dispositif constitué par les trois usines supérieures au dessus de l'altitude de 3500 m.

Les ressources en eau de cette vallée proviennent de trois origines dont les processus et ISSN-12223-7221

la dynamique diffèrent : les glaciers qui couvrent les principaux sommets et sont tous situés à une altitude supérieure à 4900m, les précipitations neigeuses et les précipitations pluvieuses. La variabilité climatique et tout particulièrement le réchauffement global observé ces dernières années ont une influence considérable sur la disponibilité de cette ressource [2].

Il était intéressant de modéliser les écoulements superficiels de cette vallée à la fois pour des raisons méthodologiques (configuration complexe de processus de production et combinaison de transferts naturels et artificiels), mais aussi à des fins opérationnelles (construction d'un outil permettant à l'opérateur hydroélectrique de mieux évaluer les conséquences d'une évolution future soit du climat, soit des aménagements).[3]

2. Tools and method

L'application de modélisation a été faite sur la partie supérieure de la vallée limitée à la section de Llaullini (Fig.1), 2 km à l'aval de l'usine hydroélectrique de

Botijlaca. Le bassin versant correspondant couvre une surface de 95 km² et contient trois usines, plusieurs retenues contrôlables et tout un réseau de canaux et de prises d'eau (Fig.2).

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Fig. 2. Bassin versant de Llaullini

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La modélisation pluie / débit a été faite en deux étapes :

- Une première étape d'évaluation de la redistribution sur les versants non englacés à l'aide du schéma de surface ISBA développé par Météo-France [4, 5]. La variabilité climatique en très haute montagne nécessite en effet une prise en compte fine des paramètres climatiques (températures, rayonnement).
- Une deuxième étape pour assurer le transfert des écoulements dans le réseau hydrographique naturel et artificiel en tenant compte des opérations hydrauliques. Un modèle de flux utilisant les méthodes de la dynamique des systèmes [6] et plus particulièrement le langage Vensim ® [7] a été spécialement développé.

Les écoulements glaciaires n'ont pas été modélisés dans le cadre de cette étude et on a utilisé les observations directes de l'écoulement, mesurées sur deux des principaux ensembles glaciaires. Pour les glaciers non observés, on s'est contenté d'une extrapolation proportionnellement aux surfaces couvertes.

La mise en œuvre de ces modélisations, et tout particulièrement celle du schéma de surface ISBA, a nécessité l'établissement d'une cartographie précise au sein d'un système d'information géographique. C'est l'outil Arc View ® qui a été retenu pour croiser les cartes de relief (découpé en tranches d'altitudes de 300 m) et des types de surfaces (regroupées en quatre catégories : glaciers, rochers, formations de pente et formations de fond de vallée) (Fig.3).

Le modèle ISBA a été appliqué pour chacune des surfaces élémentaires obtenues par ce croisement. Ces surfaces sont ensuite combinées entre elles pour constituer des entrées dans le système de transfert (Fig. 4).



Fig. 3: Croisement de la carte des tranches d'altitude avec la carte des types de surface par requête spatiale sous Arcview®. Le résultat du croisement d'une surface avec une tranche d'altitude dans un sous-bassin, définit une unité de surface. Celles-ci sont représentées par niveaux de gris différents dans chaque sous-bassin. Les surfaces glaciaires n'ont pas été prises en compte



Fig. 4. Organisation schématique des écoulements dans le système hydroélectrique. Les lignes en pointillés représentent les écoulements en rivière et les traits pleins les écoulements en canalisation à surface libre ou en conduites forcées. Les entrées notées en script sont les sous-bassins sur lesquels ISBA a été appliqué. Les entrées en gras sont des bassins dont la production d'écoulement est directement introduite dans le modèle à partir des observations.

3. Results and discussion

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Pour l'application, nous avons utilisé un jeu de données d'observations hydrologiques et climatologiques acquis par les auteurs sur le bassin effectué au cours de la période allant du 1^{er} septembre 1999 au 1^{er} février 2001. Cette période de 17 mois couvre une année hydrologique complète avec une saison d'été humide d'octobre à mai, suivie d'une période sèche d'hiver de juin à septembre, ainsi que le début de la saison suivante.

Toutes les simulations ont été réalisées avec un pas de temps journalier.

Dans une première étape, nous avons appliqué seulement le schéma de surface ISBA en composant les débits observés à l'issue de chacune des surfaces élémentaires pour obtenir le débit total à l'exutoire du bassin (Llaullini) (Fig. 5).



Fig. 5: Débits simulés sur les 7 sous-bassins naturels du bassin de Llaullini, cumulés pour comparaison avec les débits observés à son exutoire.

On remarque que pendant les saisons humides les ordres de grandeurs de débits simulés sont très proches des observations. Au contraire, en début de saison des pluies certaines crues sont surestimées et tout au long de la saison sèche, on observe un décalage systématique. Ces écarts sont dus aux opérations réalisées par le gestionnaire hydroélectrique dont les manœuvres n'ont pas été prises en compte dans le modèle de production. En effet, pendant la saison humide qui correspond à l'été en région tropicale, d'une part la demande énergétique est diminuée, et, d'autre part, la ressource en eau utilisable par les ouvrages est suffisante. Au contraire en saison sèche, pour couvrir la demande énergétique, le gestionnaire utilise l'eau stockée dans les retenues soit en cours de saison humide, soit en provenance des réservoirs que constituent les glaciers.

La **Fig.6** présente le résultat obtenu après la deuxième étape en modélisant les transferts dans le

système hydrologique comme dans le système hydraulique.

On confirme bien que les écarts observés précédemment sont comblés et que la prise en compte des opérations hydrauliques de stockage et de déstockage des retenues donnent un résultat global assez proche de la réalité.

Il subsiste néanmoins des différences :

• Les déficiences observées pendant la période qui précède la saison des pluies montrent que l'intégration des écoulements dans le système hydroélectrique peut être améliorée. Il s'agit principalement de mieux prendre en compte les stockages de moyenne durée dans les petites retenues.

L'écart observé au cours de la saison sèche est probablement du à une sous-estimation des débits glaciaires au cours de l'extrapolation des données des glaciers observés aux glaciers non observés dont l'exposition est différente et plus favorable à la fonte.



4. Conclusion

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La méthode présentée ici donne des résultats très encourageants et elle intéresse fortement les opérateurs qui utilisent à des fins énergétiques ou d'alimentation en eau potable dans les régions de haute montagne. Elle reste toutefois à ce stade un outil scientifique et peut difficilement, en l'état, être utilisé comme système d'aide à la décision. En effet, elle nécessite une excellente connaissance des données climatiques et une cartographie très fine, informations qui ne sont le plus souvent pas disponibles dans ce type de milieux. On pense actuellement à l'utilisation d'une modélisation plus grossière pour la production. Enfin, pour que cette approche soit totalement *autonome*, un modèle d'écoulement glaciaire devra y être couplé.

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Legislative Frame and The Impact Estimation On The Environment Due To The Transport Movments

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Abstract: As the society aware the importance to apply the principles to prevent and to stop, near to the technical solutions some legislative solutions having as aim the elaboration, till the impact produced by a project both on the natural environment and on the economic, social and political factors. The legislative analysis specific to the environment safety takes into evidence some legislative compulsions referring to the project elaboration and promotion.

Some project has an impact on the environment, its development depending on many factors. The impact assessment on the environment is a complex process, that needs the dependence analyze of the environment factors to the human one and also the economic and the politic factors, as well as the quantification of the positive and negative elements, including on the long term.

The transport way, component of the infrastructure necessary to a durable development, can have a minimum impact on the environment are increased by the technical solutions, development of the adequate reglementations and allocation of the financial resources necessary to the ecology problems answer.

Keywords:

1. Introduction:

Appeared as an initiative of the science men, after 1950, the environment safety problem begins to represent important inters both for the political and finical circles and, in the same manner, to make the matter of some international conventions.

Any project has an impact on the environment, its development depending on many factors. The impact assessment on the environment is a complex process, that needs the dependence analyze of the environment factors to the human one and also the economic and the politic factors, as well as the quantification of the positive and negative elements, including on the long term.

Starting from the precaution principle, general accepted in the European Union member states, as result of the pollution level decreasing of the North Sea, at the E.U. was adopted by the Council Directive nr. 85/337/EEC on the assessment of the effects of certain public and private projects on the

environment, directive that was amended by the Council Directive 97/11/CEE.

In our country, the Environment Protection Law nr. 137/1995, foreseen that for the works mentioned in the annex II at the law to elaborate the studies on the environment impact assessment.

2. Impact on the environment of the transport ways

A general scheme of the environment pollution due to the transport, explained for the road transport (figure 1), takes into evidence the direct or indirect action of the environment on the 3 components of this transport category:

- the transport way;
- the traffic;

- the merchandises that are transported, and as result of an accident can affect the environment factors.

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Fig.1. Scheme of pollution caused by road and road transport

Pollution can appear during the transport ways execution time or due to the traffic and affects direct the main environment factors: water, air, soils (fig.2). By those vectors, or direct, the other factors the fauna and fauna, can be affected but the human factor and the historic, cultural and archeological patrimony. Analyzing the pollution implications, observe that, finally, those effects can be noticed on the economic, social and politic factors. On this aspect, any pollution needs a usual financial effort in order to apply the de-pollution measures or the other compensatory solutions, or goes to an economic lost, in case of the traffic interdiction to the pollutant vehicles. In the social plan, the air, water or the soil pollution, goes at illness to extremely hard consequences, as those transmitted by waterway. In the politic plan, the effects can be both on the internal or international plan, as it is the pollution to the border crossing effect.



ROAD TRANSPORT

GENERAL DIAGRAM OF POLLUTION

Fig.2. General diagram of pollution regarding road transport

3. The environment impact assessment of the transport ways

As the society notice the importance of the prevention and precaution principles execution, near the technical solutions, some legislative reglementations, having as aim the warming from the designing impact produced by a project both on the natural environment and on the economic, social and political factors

In this way, in Romania, the Law of the Environment Protection gives the general frame regarding the environment protection. Regarding the development of this law, the laws regarding the environment factors protection were involved, as well as laws regarding the ratification of the international conventions in this field, as well as some Governmental decisions and the ministerial orders. We cannot put a stress on the detailed analysis of those law previsions. Afterwards, we put a stress just on those articles that foreseen the necessity of the environment impact assessment of some projects. In this way, in the chapter II of the Environment Protection Law and in the Order of Minister the Water, Forests and the Environment Protection nr. 125/ 1996, previsions regarding the economic and social activities reglementation on the environment impact. The 1st section of the 2nd chapter establishes the authorization procedure from the environment point of view

The environment impact assessment takes part from this procedure and has as aim the following:

- Establishment of the environment initial stage, before starting the work execution;

- Evaluation of the pollutant sources and the impact produce on the environment;

- Determination of the increasing opportunities of the impact or finding the alternative solutions in order to touch the purposed aim;

- Avoiding to generate the ecology unbalances due to the adopted solutions.



Fig .3 Legislative Frame

In order to execute such estimation is necessary to dispose by some information regarding the following :

- the initial stage of the environment factors;
- the site of the future work or activity;
- the people sites in the location area;
- the natural and cultural patrimony of this area;
- the social economic aspects;
- the legislation specific to the environment protection. From the analyze of those data and basing on the project technical specifications, it is possible to assess the impact to which it could have on the mentioned area taking into account by the legislative previsions,

but in the equal measure the compensatory/ diminution/ the impact increasing measures.

The legislative compulsions referring to the project elaboration and promotion can be grouped as following :

- recommendations in order to obtain the approvals and the agreements (e.g. : the water management approval, the urbanism certificate and the environment permit);

- the minimum admissible limits of the main quality indicators of pollutants eliminated in the air, water and on soil;

- restrictions given by the wastes management;

- restrictions regarding the protected areas, as well as the natural parks and reservations, the archeology sites, the moisted areas;

- the restrictions imposed both by the international conventions regarding the environment factors and by the European Agreement regarding the international transport of the dangerous goods.

Regarding the Order of the Minister of Waters, Forests and the Environment Protection nr. 125/1996 in order to approve the Procedures of the economic activities improvements, in order to

obtain the environment agreement is necessary to elaborate an Environment Impact Assessment Study. This Study must also include the compensatory/ increasing/ elimination measures of the negative impact on the environment. In case of the road transport ways, these measures can be as the mentioned in the figure 3. In this way, the precaution principles and the preventions principle are apllied from the designing stage.

4. Conclusions

Any project has an impact on the environment, its development depending by several factors.

The environment impact assessment is a complex process, needing the following:

- analyzing the dependency of the human, economic and social factors on the environment;

- quantification of the positive and negative elements, on the long term.

The road ways are a necessity. Those also joint and separate, in the same time, lives and collectivities, having a positive impact on the human, social and politic factors.

The transport ways, components of the infrastructures necessary of a durable development, having a minimum impact on the environment, if the negative effects are increased by :

- the technical solutions;

- the adequate reglementation promotion;

- allocation of the financial solutions necessary to solve the ecology problems.

From the environment protection point of view, is necessary to find the most adequate solutions in order to increase at minimum the negative impacts and the landscape integration. Although, involves the action given by the economic, politic and social factors this problem is possible to execute.

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Fig.4 Measures for compensation/reduction/elimination of project's negative impact on the environment

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Mathematical Modeling of the Same Physical, Chemical and Biological Parameters of the Bahlui Catchement Retention

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Abstract: This work presents some considerations about the mathematical modeling of the physical, chemical and biological parameters of the artificial lakes. The results of the mathematical modeling consist in the diurnal evolution of the basic parameters which characterize the tropical stade of the lakes: the nutrient, algae biomass and zooplancton.

Keywords: Aquatic systems, nutrient, algae, zooplancton, mathematical modeling, eutrophication.

1. Introduction

Water quality models of the retention are some differences that river quality models. The notions, the parameters are more similarities but must be considered the follows aspects: the retention receive important organically substances which due reduction of the oxygen; the lakes have a great retention time comparatively with rivers the growth of the inorganic nutrients; response time is greater that the rivers; the evolution gradient of the quality parameters is made principally by the vertical axis and less by the longitudinal axis; BOD-OD relation is slightly important comparatively with the stratification. algae evolution and eutrophycation process [4].

The principal factors for the quality water of the lakes are: the quality of the influent; physicalchemical process in the period of the storage of the substances; biological growth and the role in the modification and transformation of the substances [2].

2. Some aspect regarding the water quality of the Podu-Iloaiei retention

For the Podu-Iloaiei retention, the principal pollution source is the river Bahluet which supply

the retention. This river are the quality deteriorate by the poor treatment of the waste water plant from the Plant of Târgu Frumos city (downstream Treatment Plant-Târgu Frumos) and by the zootechnical farms (the cross-section Războieni).

The nutritive potential of the retention, due the development of the algae population in each warm season. The principal factor in this sense is the great concentration of the water in the Phosphorus.

Physical-chemical analysis characterize the water after organically and total Phosphorus in the D categories, after Magnesium and Iron II-nd and after all indicators in the I-st category. The CCOMn-DO rapport had the 3,697 mg/dm³ value.

Quantitative, the phytoplanctone has average density between 2,1 mil. cell/dm³ and 7,6 mil. cell/dm³. The phytoplanctonic biomass due by the microscopically algae has average values between 10,4 mg/dm³ and 15,3 mg/dm³ with the dominant form: *euglenoficee, cloroficee and blue algae*. The

zooplancton associations present a great organisms number: rotifers, copepods, cladocers with a density between $34 \text{ ex}/\text{ dm}^3$ and $365 \text{ ex}/\text{ dm}^3$. The microbial charge has greater values, principally in the upstream of the lake (from a average values 290.000 to 2.750.000 total coliform bacteria/dm³).

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Quality in diagton	T T . */	Characteristically values			
Quality thatcator	Unu	Minimal	Average	Maximal	
1	2	3	4	5	
рН		7,1	7,8	8,2	
DO	mg/dm ³	4	7,9	11	
DO at saturation	%	44	80	93	
DOB	mg/dm ³	12	20,5	29	
DOC-Mn	mg/dm ³	21,2	29	41	
Total minimal nitrogen	mg/dm ³	3	3,8	5	
Nitrogen	mg/dm ³	5,4	9,7	14	
Ammonium	mg/dm ³	0,5	1,8	3,1	
Total Phosphorus	mg/dm ³	0,03	0,26	0,54	
Phosphate	mg/dm ³	0,11	0,87	1,62	
Suspension materia	mg/dm ³	44	65	104	
Fixed Residue	mg/dm ³	520	650	780	
Calcium	Calcium mg/dm ³		74	90	
Magnesium	fagnesium mg/dm ³		65	78	
Bicarbonates	Sicarbonates mg/dm ³		340	398	
Total Iron	mg/dm ³	0,2	0,65	1,1	
Total durity	degree G	16,8	19,4	24	
Phitoplancton D	1000 cell/dm ³	1500	5410	9320	
В	mg/dm ³	5,8		26	
G		euglenoficea, blue algae cloroficea			
Zooplancton D	1000 ex/dm^3	12	1232	2453	
В	mg/dm ³	0,03	2,44	5,52	
G		rotifer, copepods, cladocers			
Total coliform bacteria	$\frac{1000}{\text{ex/dm}^3}$	21	290	160900	

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Table 1- Water quality characterization of the Podu Iloaiei lake (1997-2001)

Source: C.N. Apele Romane - Iasi (where: D-density; G-dominant group; B-biomass; G-german degre)

3. Presentation of the model

The model can to determine the evolution in time of the primary nutrient (the Phosphorus, of the algae biomass (at the intermediary product) and of the zooplanctone (final consumer). Initial concentration of this parameters, averages values, are (in conformity with the C.N. "Apele Romane" - Iasi data): $P_0 = 0.26 \text{ mgP/dm}^3$; $A_0 = 15.3 \text{ mg}$ Chla/dm³; $Z_0 = 0.244 \text{ mgC/dm}^3$ [1].

The carbon-a chlorophyll rapport in the phitoplancton has appreciate at a_{ca} =40 mgC/mgChla.

Algae velocity consumption of the zooplanctone is $C_{za}=1,7 \text{ mgC/m}^3$ day with a efficiency factor $\epsilon=0,6$. The rapport *phosphorus-chlorophyll a* may be appreciate at the value

 $a_{pa}=1$ mgP/mgChla (the figure 1). The coefficient of the disappearance by the respiration, excretion and mortality for zooplanctone may be considered $k_{dz}=0,1$ day⁻¹. For the net velocity of the algae development we can admit a Monod kinetic, limited by the phosphorus abatement:

$$k_{ca} = k_{max} \frac{P}{k_{SP} + P} \tag{1}$$

The maximal value $k_{max} = 0.3 \text{ day}^{-1}$ and semisaturation constant is $k_{SP} = 0.002 \text{ mgP/dm}^3$.

The interactions nutrient-phitoplanctonzooplancton may be represented about the following equations:

$$\frac{dA}{dt} = k_{max} \frac{P}{k_{SP} + P} A - C_{za} \cdot Z \cdot A \tag{2}$$

$$\frac{dZ}{dt} = (a_{ca} \cdot \varepsilon \cdot C_{za}) \cdot Z \cdot A - k_{dz} \cdot Z$$
(3)

$$\frac{dP}{dt} = a_{pa}(1-\varepsilon)C_{za} \cdot Z \cdot A + a_{pc} \cdot k_{dz} \cdot Z -$$

$$- a_{pa} \cdot k_{max} \frac{P}{k_{SP} + P} A + P_i - P_e$$
(4)

P_{inflow}

where: A is the biomass of the phitoplancton in Chlorophyll a mass/dm³; Z- the biomass of the zooplanctone exprimated in organic carbonic mass/dm³ and P is the phosphorus concentration of the lake (mgP/dm³); P_{inflow} , $P_{outflow}$, the phosphorus concentrations of the inflow respectively of the outflow from lake (mgP/dm³).



Figure 1 - The schematic interactions of the water aquatic body [1]

The growth of the algae mass is influenced and by the temperature, light intensity and by the presence of the nitrogen as secondary nutrient. The algae velocity of consumption by zooplanctone may be represented by the available of the algae mass. Also, the number of the zooplanctone may be influenced by a superior member of the pyramid biomass (fish etc.), by the natural disappearance, by settlement etc.[5].

The inflow of the nutrient in the lake measured upstream (cross-section Razboeni-Bahluet), is considered constant with a value $P_{inflow}=37,3$ mgP/dm³, and the outflow, also

constant, $P_{outflow}=0,267 \text{ mgP/dm}^3$ (the cross-section Podu-Iloaiei Bahlui).

The numerical resolution of the equations may be effectuated with the Runge-Kuta IV order method.

The result confirm the following conclusions:

- the biomass are the maximal values after 6 days;

- the zooplancton are the maximal concentration at 5 days after the maximum of the phytoplancton. The plants are a maximal evolution due the organically pollution from the zoo-technical farms and from the effluent poor in the Phosphorus;

- the great time of retention for the nutrient (Phosphorus and nitrogen) [3].

5. Acknowledgements

We want to tank at C.N. "Apele Romane"-Iasi for the quality data put at disposition.

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Water Quality Management of the Bahlui River Using the Mike 11-3.01 Model

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Abstract: This work present some considerations about the mathematical modelling used in the field of water quality management of the surface waters. The results are based on the utilisation of the MIKE 11-3.01 model for the dispersion of the pollutants in the Bahlui River.

Keywords: Water pollution, mathematical modelling, pollutant dispersion, water quality management

1. Introduction

The water is a natural limited resource. The water pollution given very hard problems in the water management of resources.

In the zones with strictly climate regime the pollution can to affect seriously this resources. The utilisation of mathematical modelling can give some solution in this sense [3].

2. The general presentation of the MIKE 11-3.01 model

The MIKE 11-3.01 model was elaborated by the Danish Hydraulic Institute. The model was used for the water quality simulation of the pollutant dispersion of the Bahlui river.

The advection-dispersion module (AD-WQ) use the velocity result from hydrodynamic module (HD). The validation of the model is made using the hydrological module NAM (Nedbør-Afstrømnings-Modele) which describe the land phase of the water [3].

The AD-WQ module is based by the onedimensional equation of the conservation of mass of dissolved or suspended material. The module r requires output from HD module, in time and space, in terms of discharge and water level, cross-sectional area and hydraulic radius. The solution of equation is obtained with an implicit scheme in finite differences. The equation has the following form [2]:

$$\frac{\partial(AC)}{\partial t} + \frac{\partial(QC)}{\partial x} - \frac{\partial}{\partial x} \left(AC \frac{\partial C}{\partial x} \right) = -AkC + C_2 q \qquad (1)$$

where:

C is the concentration of the substance, [mg/l];

D- the dispersion coefficient, $[m^2/s]$;

- A cross-sectional area, $[m^2]$;
- *k* linear decay coefficient, $[s^{-1}]$;
- C_2 source concentration, [mg/l];
- q- lateral inflow, $[m^2/s]$;
- *x* space co-ordinate, [m];
- *t* time co-ordinate, [s].

The equation describe two transport mechanisms: advective transport with the mean flow

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and dispersive transport due to concentrations gradient.

10,00 Km

3. The computational hypothesis

The mean assumptions are the following:

- the considered substance is completely mixed over the cross-section and the pollution source is considered to mix instantaneously over the cross-section;
- the substance is conservative or subject to a first order reaction;
- the Fick's diffusion law (the dispersive transport is proportional to the concentration gradient)
 [2].

The advection-dispersion equation is solved with a finite difference scheme. The mass balance for the parameters are calculated for all points of the computational scheme and the computational grid is automatically generated. The concentration points are placed midway between neighbouring points and structures [2].

The computational hypothesis are the following:

- a flooding period (1-2 March 1995);
- a pollution with nitrates compounds over the tributary Voineşti (down stream at confluence between Voineşti and Bahlui rivers (10 km); the 0,00 km are considered the confluence between Bahlui and Bahlueţ rivers at Podu-Iloaiei) with a maxima concentration 15,00 mg/l at 7 A.M.-1 March; the flow of the source of the pollution 0,395 m³/s at 7 A.M. - 1 March;
- the dispersion coefficient are considered 40 m²/s.

The results are presented in the Figure 1, 2, 3, 4













Fig. 3. The results of the simulation with MIKE 11 AD-WQ module at the 30 km by the confluence cross-section



Fig. 4. The results of the simulation with MIKE 11 AD-WQ module at the 40 km by the confluence cross-section

4. Conclusions

The mathematical modelling used in the field of water quality management of the surface waters demonstrate the factors which have an effect at the dispersion capacity.

The advection-dispersion equation is solved with a finite difference scheme.

The equations describe two transport mechanisms: advective transport with the mean flow and dispersive transport due to concentrations gradient.

The results presented in the Figure 1 lead the following conclusions:

- at the confluence cross-section of the Voineşti and Bahlui rivers (10 km), the maxima concentration are 11 mg/l at 11 A.M. time (1March);
- at the 20 km the maxima concentration is 10,25 mg/l at 3 P.M. time (1 March);
- at the 20 km, 30 km, and respectively 35 km the maxima concentrations are 10,00 mg/l at 3 P.M.-1 March time, 5,9 mg/l at 10 P.M.-1 March and 5,15 mg/l at 1 A.M.-2 March;

• the results confirm a good dispersion capacity of the Bahlui river in the conditions of a supplementary flow (in the flooding periods or in the conditions with a supplementary flow pour out from a lake);

• in the period with a small flows this capacities are diminued due the small velocity and flows [4].

5. Acknowledgements

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Contributions on Establishing The Maximum Debits Necessary In The Designing Of The Water Management Works

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Abstract: The aim of this paper is to establish the maximum, minimum and average discharges, which are necessary within the design of water management works. Before processing the discharge data obtained from measurements, computer programs have been developed to enrich the arrays of data by generating chronological values with a steady pace of one year or one month respectively.

Keywords: debits, probability, generating of chronological values.

1. Introduction

The hydrological elements of the flow system are given by the base of first rank in solving the water administration problems and therefore they must be determined with much accuracy.

The flow system of natural water courses defined through solid and liquid debits, flow speed, intensities and character of the high floods is very complex, being influenced by many factors such as: biological, geological, geografical, climatic factors.

This flow system gives a influence over the activity of water administration especially through liquid debit.

The knowledge of debits variation on a previous period, let us make estimations about debits that follows to be recorded, debits suitable in designing the water administration work.

Statistical mathematics cannot specify the appearance moment of hydrological phenomena, but enable the establishing with a satisfactory accuracy of the frequencies of these phenomena. The method which present guarantee in establishing the debits, consists of their observing and measuring on a very long period, of the interpretation of these data and their analysing. It's not enough the knowledge of the debits in practice but also their variation in different significant points of the hydrographical basin, therefore into a hydrographical basin must exist or had to be organised a hydrometrical stations network in much more significant points.

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In computing and exploiting the watre administration works, there are necessary the debits: maximum, minimum and medium.

The maximum debits are important especially in dimensioning and the exploiting of the water administration works, which must resist the maximum requirements.

The minimum debits are used in the designing and the exploiting of the water management works, taking into account that the water utilization should function at normal parameters in the most difficult conditions during the small debits water period.

The average debits indicate the rich content of water source studied.

2. The computing of the debits necessary in the water management works

For a hydrographical basin the designing of these debits it is made in the following places: the hydrometrical posts and stations from the basin; upstream and downstream of each confluence; the plug points; the restitution points; the accumulating and derivation points, and also points with special importance.

These computing points are grouped in three categories, if taking into account the used computing methodology, that is:

- A) points where there are observations and measurements on a period of at least 20 years;
- B) points with recordings below 20 years (even 3-5 years);

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C) points without hydrological measurements and observations.

A) In that points where there are observations and measurements on a period of at least 20 years, the hydrological data are analysed as to result yearly tables which contains daily average debits, monthly average debits, yearly average debits and the ten-days average debits (from april to september period); in the meantime one stands out: the daily minimum debits in each month, the ten-days minimum debit from the vegetation period of each year and the daily minimum debit in each year.

For the hydrological data necessary in the activity of water management, there are considered as basic debits - the daily average debits with the help of which one calculates the minimum debits with different probabilities, for irrigations are used the ten-days minimum debits.

For the computing of daily minimum debits we allow their classification in summer minimum debits and winter minimum debits.

The minimum debits concerned are computing for the utilization character and refers to certain periods during the year; thus, when a industrial unit which uses the water must be ensured 95% in the summer period, that is we will calculate the daily minimum debit in this period with a 95% probability.

From each year of measurements and recorded direct observations, it is chosen the summer minimum debit (april-october) and the winter minimum debit (november - march) composing different tables. In the same way it could be composed the tables with weekly, monthly and ten-days debits according to the utilization character.

Using data from these tables made for each subperiod we calculate maximum debits with the predicted probability, using the empirical curves of probability and the theoretical curves of probability fited over the empirical ones (according to the statistical mathematics methodology) [1,2].

If we want a high accuracy, before analysing the debits resulted through measurements, it enlarges the data sequence by generation of chronological values taking the following example.

One knows from statistics the fact that the producing of an event is described by means of a numerical value sequence associated to the variable, that characterizes the event and the probability of realizing such values. For instance the water's leaking by the cross-section of a water's stream with a certain yearly maximum flow constitutes an event characterized by the measure yearly maximum debit.

The probability of an event it is considered from the classical point of view, as the ratio between the favorable number of results for producing that fenomenon and the total number of posible results.

But in hidrology we do not know the probability of such an event because of not knowing those two mentioned measures. Instead of this, we can count the favorable results from the set of all the proposed results, till the considered moment and make their ratio, obtaining the relative frequency of producing an event.

When the total number of the produced results is big enough, the frequency is approaching very much to the probability.

In the case of hydrological sequences of data, in the majority of situations, one knows a very little number of terms because one disposes for a very few number of yearly observations.

Thus, when computing the probability of producing a hidrological event appears the necessity of enlarging the data sequences. All of those are also required in order to obtain more informations on the studied phenomenon which is suitable in designing, execution and exploiting the hydrological works.

a. The generating of the chronological values with constant step (one year)

In order to generate such values we are taking into account of a basic sequence composed of known hydrological data (obtained by chronological measurements).

The basic sequence and the obtained one which is composed by the basic sequence terms concatenated with the generated termes, must have the statistical features (the arithmetic mean, the variance, the mean square deviation, etc.), approximately equals.

We consider the studied hydrological phenomenon in probabilistic terms, e.g. a random process.

The basic relation used in generating new chronological values derives from the *Markov*'s chains theory and it has the following form [3, 4]:

$$Z_{i+j} = \overline{Z}_i + \rho \left(Z_{i+j-1} - \overline{Z}_i \right) + + \gamma_{i+j} \cdot \sigma Z_i \sqrt{1 - \rho^2}$$
(1)

where $i = \overline{1, N}$; $j = \overline{1, M}$; \overline{Z}_i - the arithmetic mean; ρ - the serial autocorrelation coefficient of order 1; $\sigma \cdot Z_i$ - the mean square deviation of the sequence; γ_{i+j} - the random variable of Pearson IIIrd tipe, Z_{i+j-1} - the previous generated term in the sequence; Z_{i+j} - the current generated term in the sequence.

In what follows we give description of the designed program:

Input data: N – the number of given values $\overline{Z}_i = \overline{1, N}$; M – the number of values to be generated by program (in fact, the program generates M+1 values); Z – vector, which components are the given values Z_i , $i = \overline{1, N}$ and which will contain the next generated values Z_j , $j = \overline{N+1, M+1}$.

Output data::

$$Z_j, j = \overline{N+1, M+1} \tag{2}$$

Requested subrutines:

- **PRIM** (N, A, J) where N signifies an integer number, A is a vector A(N) and J is used as an index for the storage of generated prime numbers from the interval [1, N]. Into the program we took in PRIM subroutine N = 5000, thus it will be generated 671 prime numbers (if we need a larger N, we took for instance N = 10000 and it will be automatically generated 1231 prime numbers, situated in the interval [1, 10000]).

We need the prime numbers for entering in ALEATOR subroutine.

- **ALEATOR** (X0, F, S4) where X0 is the starting value (a prime number issued from PRIM subroutine), F is a vector with 12 components which contains generated random numbers with an uniform repartition on [0,1] and

$$S4 = \sum_{i=1}^{12} F(i) \tag{3}$$

b. The generating of the chronological values with constant step (one month)

In order to generate such values we are taking into account a known sequence of chronological data, obtained by monthly measurements.

The chronological values matrix (the index line represent the observation year and the column index – the month j, $j = \overline{1,12}$ and the total matrix, resulted by juxtapositioning of the given matrix with the matrix of the generated values must have the statistical features (the arithmetical mean, the variance, the mean square deviation etc.) closely as values. One considers the studied hydrological phenomenon as a random process.

The basic relations used in generating chronological values with constant step (of one month) derives from the *Markov*'s chains theory [4].

In the program we have made the following notations and relations:

 the vector S, which components are the total of the chronological values corresponding to a month, taking N given years;

$$S(j) = \sum_{i=1}^{N} Z_i, \text{ pentru } j = \overline{1,11}$$
(4)
$$S(12) = c + \sum_{i=1}^{N-1} Z_{i,12}$$
(5)

where C is the value for the twelfth month, corresponding to the previous year of the first given year;

- the vector Z, which components contain the arithmetical means of the values $S(j), j = \overline{1,12}$:

$$\overline{Z}_{j} = \frac{\sum_{j=1}^{N} S(j)}{N}, \text{ for } j = \overline{1,12}$$
(6)

- the vectors S1,S2, S3 which contain as well the mean square deviations, the total of the value products for closed months and the total of the value squares for a given month:

$$S1(j) = \sum_{i=1}^{N} \left[Z_{i,j} - \overline{Z}_{j} \right]^{2}, \text{ for } j = \overline{1,11}$$
(7)
$$S1(12) = \left[C - \overline{Z}_{12} \right]^{2} + \sum_{i=1}^{N-1} \left[Z_{i,12} - \overline{Z}_{12} \right]^{2}$$
(8)
$$S2(i) = \sum_{i=1}^{N} Z_{i,12} - \overline{Z}_{i,12} - \overline{Z}_{i,12} - \overline{Z}_{i,12} = \overline{1,11}$$
(9)

$$S2(j) = \sum_{i=1}^{N} Z_{i,j} \cdot Z_{i,j+1}, \text{ for } j = 1,11$$
(9)

$$S2(12) = C \cdot Z_{1,1} + \sum_{i=2}^{N} Z_{i,1} \cdot Z_{i-1,12}$$
(10)

$$S3(j) = \sum_{i=1}^{N} Z_{i,j}^{2}$$
, for $j = \overline{1,12}$ (11)

- the vector σ , which contains the mean square deviations for the values of each month:

$$\sigma_j = \sqrt{\frac{S1(j)}{N-1}}, \text{ for } j = \overline{1,12}$$
 (12)

- $\delta_{j,j-1}$ is the serial autocorrelation coefficient between the data of the months j and j-1, given by:

$$\rho_{j,j-1} = \frac{\frac{1}{N-1} \left[S2(j) - (N-1)\overline{Z}_{j} \overline{Z}_{j-1} - S(j)\overline{Z}_{j-1} - S(j-1)\overline{Z}_{j} \right]}{\sqrt{\frac{1}{N-1} S1(j)} \sqrt{\frac{1}{N-1} S1(j-1)}}, \quad j = \overline{1,12}$$
(13)

 $\rho_{1,0}$ is a serial autocorrelation coefficient between the data of the january month and the previous december months:

$$\rho_{1,0} = \frac{\frac{1}{N-2} \left[S2(1) - (N-1)\overline{Z}_1 \overline{Z}_{12} - \overline{Z}_{12} S(1) - S(12)\overline{Z}_1 \right]}{\sqrt{\frac{S1(1)}{N-1}} \sqrt{\frac{S1(12)}{N-1}}}$$
(14)

- the vector γ , the random variable of Pearson IIIrd type, which components are calculated according to the relation:

$$\gamma_{j} = \frac{2}{CS_{j}} \left[1 + \frac{CS_{j}(S4 - 6)}{6} - \frac{CS_{j}^{2}}{36} \right]^{3} - \frac{2}{CS_{j}}, \text{ for } j = \overline{1,12}$$
(15)

where: $S4 = \sum_{i=1}^{12} F(i)$ is the total of the pseudo-

random numbers calculated in the subroutine of the random numbers from the program, starting with a given prime number. We notice that the F(i) frequencies are uniformly distributed on the

$$Z_{i,j} = \overline{Z}_{j} + \rho_{i,j-1} \Big[Z_{i,j-1} - \overline{Z}_{j-1} \Big] \frac{\sigma_{j}}{\sigma_{j-1}} + \gamma_{j} \sqrt{1 - \rho_{j,j-1}^{2}}, \text{ for } j = \overline{2,12} \text{ and } i = \overline{N+1,2N} \quad (16)$$

$$Z_{i,1} = \overline{Z}_{1} + \rho_{1,0} \left[\overline{Z}_{i-1,1} - \overline{Z}_{12} \right] \frac{\sigma_{1}}{\sigma_{12}} + \gamma_{1} \sqrt{1 - \rho_{1,0}^{2}}$$
(17)

for i = N + 1,2N (for january month).

In the program are used outside the program two subroutines especially for: generating the prime numbers in the interval [1,N]; the generating of pseudo-rendom numbers uniformly distributed in the interval [0,1]; the generating of prime numbers in the interval [1,N].

Their description is similar to the one given at the chronological values with constant step (one year).

B) In the points with few recordings, the minimum and average debits with diverse probabilities are setting by the extension of recorded data on the basis of their correlation with the data from the points where there are recorded measurements and observations on a long term.

For a better accuracy one looks for the data correlation between the closest points and having also the physical-geographic parameters alike.

If in the hydrographical basin there is only one hydrometrical station S_0 with recordings on long terms and other stations S_1 , S_2 ,... S_n with few recordings, it won't correlate each other with S_0 but it will be correlating only the closest station and alike with S_0 (for instance S_1); afterwards S_2 is correlatind with S_1 if it is much closer and much similar with this than with S_0 ; after this proceeding the correlation continues.

The correlation can occur between the yearly, monthly and ten days debits, but the correlation

interval [0,1] and CS_j are the coefficient calculated during the program.

- the chronological data are generating according to the following relations:

between the daily debits have uncertain connection and it is recommended to give up on them.

The analytic correlations are recommended to be applied for very short sequences (5 years minimum) with correlation coefficient with absolute values of at least 0,8 or for short sequences (10 years minimum) with correlation coefficient of at least 0,7; the phisicogeographic factors must be alike for those two posts with correlated sequences. These analytical correlations are verified by drawing the regression lines y = f(x) and x = f(y) and the bisectrix of the angle between them and it is chosen the most suitable (if the angle between those two regression lines is too big then the suitable line is the bisectrix of this angle).

The graphical correlation are more expeditious and thus it eliminate the computing errors and they can be used for setting the relations between those two calculating points, when it can't be applied the analytical correlations. The graphical expression of correlation can be straight or curve.

If in the studied hydrographical basin there are not points with long recording sequences, we can make analogy with another hydrographical basin where there are measurements on a long period of time and this basin is alike the first in terms of physical-geographic factors (the most important are the rains and the basin's area); in this case we are making the correlation between the debits and the rains for the basin with long sequences of recordings and this correlation it is used for the studied basin.

C) In the points without direct observations and measurements, the interesed debits are established using computing empirical formula, formulas which give approximate results.

These empirical formulas can be applied only for phisico-geographical conditions (the rains, the temperature, the deficit of atmospherical humidity, the pedobotanical conditions, the relief etc.) similar to the ones where they were established. The empirical formulas provide results much closer to the reality, if they have more parameters (the correlation coefficient which express the phisicogeographical factors' influence over the leaking).

For the leakings average values which usually follows the correlation between the specific debit and the basic factors (rains, the basin' area etc.), the empirical formula are less laborious than the minimum or maximum values, because they are taking into account a smaller number of leaking's factors.

If we have two hydrographical basins where the phisico-geographical factors are alike and one disposes of observations and measurements into a basin, then one recomends the using of analogies instead of empirical formulas; otherwise one recomends interpolations. The debits concerned in the activity of water management could be determined also with the help of the maps with isolines made for our country.

3. Conclusion

a) The enlarging data sequences by generating chronological values with constant step (one year or one month) for which were realised computing programs by the authors, it is useful also for the calculation of the probability of producing a hydrological event as for obtaining some more informations on the studied phenomenon, necessary in designing, execution and exploiting the hydrological works.

b) The extension of the recorded data on the basis of their correlation with the data from the points where there are long term recorded observations and measurements, must be done until the sequence reach at least 20-25 values, afterwards one calculates through statistical methods the ten-days, monthly and yearly average debits with computing probabilities.

c) In place of empirical formulas one recomends the using of analogy if there are hydrographical basins in which the phisico-geographical factors are alike, in different situations one recomends interpolations.

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Physical Modeling of the Failure of the External Dikes at Decantation Ponds

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Abstract: The paper presents the results regarding hydraulic modeling of the failure of a dike made from real material discharged in a decantation pond. The material has been carried from Copper Exploitation Rosia Poieni, placed in Western part of Romania. The results have been used to mathematical modeling of the failure of a dike at decanmation ponds from mining industries.

Keywords: decantation pond, hydraulic model, material dischrged on decantation ponds

1. Introduction

The importance of studies regarding the failure of decantation ponds from nonferrous mining industries has been demonstarted, unfortunatelly, by the accidents produced in the last years (in 2001 at Baia Mare and Baia Borsa).

These kind of accidents produce a significant environmental impact and the life of humans and animals can be affected as well.

2. Experiences on a physical hydraulic model

The experiences carried out on a small scale physical model have followed the study of the movement of water with solid material on a channel.

A model has been built in the hydraulic research hall for the visualization of the behavior of the solid material from the decantation pond Valea Sesii at Rosia Poieni Mining Exploitation. The main goal has been the qualitative aspects of phenomena obtained in the case of the failure of the dike built with material from the decantation ponds. Due to of the difficulty of modelling the real valley, a symplifyed model has been used.

The study material is in situ material. It was possible to dig and carry the material from the beach because it was dry, with humidity 15% and withd 200 m.

The physical characteristics of the solid material are the following: the component sand is predominant with the diameter of the particle 0,2 mm and a small clay content (0,7%).

The characteristics of the physical model are the following (photo no.1):

-the channel shape is halfround, made from iron pipe, the diameter 182 mm;

-the channel has been painted in white colour for a better observation of phenomena:

-the variable slope of 2%, simlary to the mean natural slope of the Sesii creek;

-the recovery of the downstream carried material during the experiences.





The starting point for the experiences has been the same for all experiences: the dike shape has been the same and compactated and water saturated first and after that the water discharge has been increased till the failure of the dike and the transport of the solid material.

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The biginning of the experiences have been the dike failure at the channel slope 2%, without the water discharge measurements.

The dike section used to all experiences are: height 15 cm, batter slopes 1:1, the crest width crest 20 cm (photo. no. 2).





All the failures have been asymmetrical to different slopes.

Experiences have been made with channel slopes 2%, 1%, and 0.5% for which a graph Q= f(h) has been determinated (graph 1).



In all experiences, after the failure of the dike the solid materal is carried downsteram like bottom transport almost. the channel. It has been determind the solid discharge transported during and after the failure of the dike and the transport time of all the solid material (experiences 2,3, photo 3 and table 1)

To all slopes the experiences followed the failure of the dike, deposition and transport of the solid material downsteram till the complete empting of

No. of the experience	slope	h(cm)	B(cm)	Q(l/s)	$Q_t(l/s)$	$Q_s(l/s)$	Observations
2	2%	1,05	11,6	0,2			
		1,8	17	1,5			
		2,5	20	2,6			
		3	14,5	3,4	2,12	0,35	Before failure
3	1%	0,9	12,5	0,86			Dike
		2,3	18	0,68			
		3	21	1,5			
				1,6	1,5	0,11	
					1,8	0,22	
4	0,5%	1	13	0,07			Dike
		2	18	0,055			
		2,9	20,5	0,9			
		4,58		2	2,42	0,166	
5	1,5%	1,1	12	0,176			Dike
		1,9	17	1,26			
		2	17	1,58			
		2,5	19	2,25			
					4,87	0,375	
					3,13	0,33	
6	2%		8	0,061	Dirty	Clean	Certain shape
					water	water	
			10	0,26	Less		5' transport
					dirty		
				ļ	water		
				ļ	0,26	0,05	
			15,2	1,04	1,09	0,016	
7	1%		23	2,3			Certain shape

Table no. 1

			0,4	0,01	1 h transport
			2,2	0,07	
	3,8	21	2,4	0,013	



Fig. 3

When the slope of the channel is small, the solid material deposition is faster than to higher slope and the transport is similary to the alluvions transport in the rivers.

The dunes formation and the segregation of the solid material on the bottom of the channel has been observed (photo no.4).



Fig. 4

The transport time of the material has been from 15 minutes to the slope 1% and 1 hour to the slope 0,5%.

Since the clay content is reduced, the material has non newtonian behaviour in the deposition zone, in the decantation pond and on the channel only.

The water speed during the experiences ranged within 0,4 m/s and 0,5 m/s.

The roughness calculated from experiences has been 0,0025 to the slope 2% and 0,021 to the slope 1%.

3. Conclusions

The downstream transport of the solid material has been specially in suspension in the failure area of the dike where the slopes are higher and the failure produced valley-ward snow. This phenomena have been due to the fine sand content.

Some parts of the solid material has been carried downstream by jumps, at the slope 0.5% almost, when the time to carry all the material has been longer.

The results of experiences on hydraulic model have been used for mathematical modelling of the failure of dikes at decantation ponds from mining industries..

Aspects Concerning the Mathematical Approach of the Danube Meadow's Drainage and Dyking Surfaces

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Abstract: The every day dates are a part of the most different domains: Geography, Sociology, Economics, Environmental Studies. The quick answers to the questions about these domains can be obtained when the respective dates are stored in electronic memory. To explore this of our life we use electronic tools, especially built to process, present and interact with useful information. One of these tools is the Geographic Informational System.

Keywords: Geographic Informational System, drained and dyke surface, ecological conditions.

1. General

The Geographic Informational System can be defined as a collection of hardware, software and projected procedures created in order to collect, administrate, manipulate, pattern and display the utilized dates in order to solve the complex problems of planning and administration.

A G.I.S. has four functions. You can get the wanted resultats with these functions, and they ore:

- 1.Data introduction and checking
- 2.Data storage and administration
- 3.Data analysis pattern
- 4.Results visualizing and printing

2. Data introduction

This stage can be achieved in many ways, and depends on the nature and quality of dates, as well as the nature of the application. The graphic dates can be introduced, using the following procedure:

- from measuring ground devices, wich offer digital information;
- digitalizing some existing plans or maps;
- scanning and vectorizing some existing plans or maps.

The alphanumeric dates are introduced using the following procedure:

- from other databasis, through direct analysis or conversion;

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- from primary documents, helped by a classical administration system of databasis.

1. Data storage and administration

Knowing the fact that every mao is obtained by combining the part themes, the structure based on thematical data parts appears natural. Even the primary datas, that make the base achievement of mape can have different sources. The plan of a system for a land improvement includes the following elements:

- lots;
- the open draine net;
- the roads;
- the electric supply net.

2. Data analysis pattern

The data format is not important, the most important is the way that the analysis is made.

The analysis and pattern of the data begin with topology, wich explains the relationships between the geographical elements from a layer. Creating the topology is a base characteristics of the Geographical Informational System and one of the main features that separate them from the Computerized Blueprinting.

After creating the topology, you can go with analysing and patterning the spacial dates. Generally, ther are four types of analysis, in a Geographical Informational System: - the proper sapcial analysis, wich includes a polygon superposition, connections, etc;

- length measurments, surfaces area, volume based on datas in a raster form or vectorial form;

- statistical analysis;
- report making.

3. Results vizualizing and printing

The datas and the final results can be visualized in a combined or simple from maps, pictures and graphics.

You van notice the easiness regardind the use abd rapidity to obtain graphical and alphanumerical quality precise results.

To give an example, we presents the mathematical approach of the drained and dyked surface, Dabuleni – Potelu – Corabia (fig.1). The purpose is to modernize and reabilitate the drainage works from Danube Meadows in ecological conditions.

Dabuleni – Potelu – Corabia drained and dyked surface stretches out on over 40 km along the terraces of the Roumanian Plain.

From the climatic p.o.v.it receives yearly an average of 520 mm rainfall, at the average temperature of $11,1^{\circ}$ C; potential evapotranspiration sums up yearly 720 mm (forest steppe zone).

On the part of the zone, after damming, it was settled a close correlation between the Danube levels and the phreatic ones, which increases at medium and high levels of the river, being drained at low levels. The rainfalls feed the phreatic bearing during the humid periods; other sources are scarce.

During 1964 – 1977 there were aschieved drainage works, with a canal network for a specific flow of 0,851/s/ha, and pumping station for 0,951/s/ha. Since 1978 a new step was taken: irrigation works for 24. 745 ha and the extension of underground drainage on 9 500 ha (depth 1 - 1,4 m, distances between lines 20 - 40 m), ensuring thuse a good control of the phreatic water.

In this zone, detailed studies on the evolution of the pedogenesis process of the former flooded soils and on hydrosaline regime evolution of soils have been carried out. These evolutions were favourable, the zone soils reaching a hight fertility; because their evolution was rapid, they need agrochemical surveys each 3-4 years; it was also established the optimum phreatic level from the water supplying of soils and the reduction of irrigation requirement.

Other important conclusion has been also drawn concerning thre design and execution of hydroameliorative works and the hydroameliorative and agricultural operation of the zone.

The purpose is to modernize and reabilitate the drainage works from Danube Meadows in ecological conditions.

Due to the complexity of problems concerning every surface we must analyse and solve the drainage and dyking works and the regularization ones, the use of some specific areas for pisciculture and irrigation in order to combine properly all these characteristics works we can use some of the elements from hydrotechnical works sketch (drains, pumping stations) and the further development of the area.

A mathematical pattern is presented wich is the base of surveiling the humidity affected areas, in order to establish a economical, social and biological solution, to diminish the produced negative effect.

4. Conclusion

The centenial experience concerning the improvement of the inundable region of the Lower Danube by hidroameliorative works has demonstrated that there have always been found proper, suitable and efficient solutions foe ech problem. The dammings, drainage and irrigation are linked into natural succession along some stages whose duration depends on the natural difficulties and of the consistency of the applied technology. The stage by application of these interventions and especially at the best of times has proved of most technical and economical impoetance.

Another cathegoty of measure meant to contribute highly to the remving of yhe cause wich may lead to negative effects in what the hydrological structure and the evolution of the soils is concerned are those wich refer to the water losses throught infiltrations from canals and those from the watering are concerned, the measures of the functioning rules and ending with the good oganization of the work) in the case of water

losses through infiltrations from canales, action must be taken with the help of technical measures (the tight coating of canal mainly) wich undobtedly trigger investments wich should be stipulated in the plans, and afterwards carried out.



Fig. 1 The mathematical approach of Dabuleni



Fig. 2 The orthographic approach of Dabuleni
The materialisation of this most important conditions and attentive control of the hydrological

The data of this control continuously feed the agricultural technologies applied of this hydroameliorative background, both in the current operatin and in the prospect of the consevation and improvement of the productive potential of the Danube Plain. In order to ensure this control it was adopted a verified scientific methodology , in use in the current activity of hydroameliorative operational units. and hydrosaline evolution of the territories included in the dammed zone

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Drainage Solutions for Increasing the Stability of Versants

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Abstract: This paper presents the advantages and disadvantages between boreholes and drains in reducing the groundwater level for increasing the stability of the versants.

Keywords: Keywords: water level, drains, drilling.

1.Introduction

One of the most important measurements applied increases of the stability of versants is reducing the groundwater level.

In theory, a drainage system can be realized with drainage shafts or drains.

The drainage shaft solution is indicated in the case of permeable layers with high depth and high permeability, to increase the influence ray of boreholes.

The results are high flow density, and there is the risk of sanding. The advantage of this solution is the relatively small area of execution.

The drain solution leads to small specific flows and to a better control of water levels, in which case this solution is recommended.

The drain execution can be made perpendicular or parallel with the level line of the versant.

The perpendicular drains on the level curves can be made in open digging, by drains at the base of versant or by horizontal controlled drilling.

For showing the hydraulic effect of some perpendicular drains we present, briefly, the hydraulic studies made for the versant in the Țicău zone of Iași district. These studies have been made in 1974 - 1978, when the only possibility to complete it was the use of the drains in the versant technology, which could not have a great length. Even so, qualitatively, the conclusions are the same, maintained in the situation when, by horizontal controlled drilling could be realized

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perpendicular drains on the level lines of greater lengths (250-300 m).

2. The analyzing of the design solutions

In case of the Copou hill in Iasi, were studied a number of solutions on the schemata shown in Fig. 1: the levels were considered constants at the base of versant (a_2 line) and in terrace (a_1 line), the length of a drain is marked with d (with values between 0 and 20 m), the distance between the drains is marked with L (with values between 8, 14 and 20 m), and the versant was considered unclogged or clogged (with different permeability).



Fig. 1. Drainage schema.

A solution like this raises two problems:

 If the L distance raises and the length of the d drain is not properly chosen, the groundwater levels in the section 2-2 (between drains) don't decrease enough to ensure the stability of versant;

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- The loading of the drain is not uniform, the upper head of the drain takes a flow density much greater than the part from the versant;

To examine the first problem, we present (in Fig. 2) the minimum distance from the versant where it is located the echipotential of 10%. In the option A2 (L=20 m, d=5 m) the drains begin to be too short and too far so that the equipotential of 10% didn't change much its position comparing with the situation without drainage (A1). Obviously, the distance a_{10} grows in the solutions in which the length of a drain grows and the distance decreases. In Fig. 4 is represented the fraction d/L for different options. If this fraction is bigger than 1, the length of the drains perpendiculars of the versant will be much bigger than the length of a single drain parallel with the versant. The drainage effect is represented by the distance from the shore were it is the equipotential of 50% (a_{50}) reported at the option without drainage.



Fig. 2. Minimum distance from the shore were it is the equipotential of 10%.

The second problem was studied by tracing the hydrodynamic specter, out of which we obtained the flow over 1 linear meter of drain. The A_5 option (L=20m, d = 20 m) it was obtained the dimensionless distribution shown in Fig. 4.



Fig. 3. Comparing the effect of perpendicular drains on the shore and the situation without drainage (d = 0).



Fig. 4. The specific flow distribution in A₅. option.

If the flow density at the upper head is marked as a unit, we can observe that by three-quarters of the d length, the flow density is under 24,4% and at the lower head of the drain it reaches 8,4%. From here it raises the idea of shorter drains of 5 - 10 m, made of

drained secant piles, at greater distances from the base of the versant.

Making perpendicular drains on the versant, of greater lengths (over 200 m), must be studied, remembering the observations below.

In the hydraulic studies for the Copou versant of Iaşi or for the Danube shore in Calafat harbor, were also analyzed some other problems, like the obliqueness of groundwater flow on short parallel drains, perpendiculars on the shore. The studies were pointing that the charge of drains was unequal in these situations.

For Iaşi and Calafat, the necessary time for quasi-steady levels was appreciated at 23 - 28 days, considering the lowering levels and the permeabilities.

Parallel drains with the versant can be made only by the horizontal drilling who presents more advantages, both from a hydraulic point of view and field execution.

From a hydraulic point of view, this solution allows a better control of water levels and a uniform flow of it along the versant. They also have a better response on grounds of lower permeability, because there is no problem regarding the formatting of influence areas.

Also, the charge is uniform along the drain, with a lower flow density, and a lower danger of sanding.

The clogging degree of the versant is not important, because there are not areas of springing.

From a constructive point of view, the technology is very good and consists in using a digging tool with a form of a rock drill which advances horizontally by rotating and dislocating the ground with the aid of special fluid jets at high pressure. This fluid also represents the lubricated fluid. The guidance from the surface, with the

support of the electromagnetic wave units and a computer, allows avoiding the obstacles and

completes the wanted line. The drilling device is introduced in a pontoon, it follows the wanted trace and it comes out through another pontoon, without interfering with the activities in the area.

GEOROM has two installations that can plug in filters over 200...360 m distance, at maximum depths of 15...30 m, with diameters of 250...500 mm, and a guidance precision of $\pm 5 \dots \pm 20$ cm.

This technology was applied in lots of situations with good results, confirmed by the lowering of registered water levels.

An example of applying this technology was SIDEX Galați, the Cătuşa shore lake.

The hydro geologic characteristics are quite complicate. The heterogeneous terrain has hydrostatic levels of different depths and conductivity coefficients of lower values: 0,02 m/day ... 0,5 m/day.

At the surface there are siding-lines and three unfreezing tunnels where wagons with frozen minerals are standing during the winter. Water supply networks are relatively important in the area, and at the surface there are open channels for the evacuation of water terrace. The shape of these installations is not good because there are not watertight.

There were chosen more representative profiles from the layers point of view and there were examined several layout options of two or three drains parallel to the level lines of the field.

In Fig. 5 after Table 1, there are shown the specific flows that can be obtained in different options, for each drain.



For each option there has been traced the free surface of the groundwater. There have been examined options for the conductivity coefficient k=0,01-0,1-0,3-0,5 m/day and with surface supply $\epsilon=0,002-0,007$ m³/day, covering the possible options. It has been used a program with finite elements for discontinue motion, but maintaining the same

conditions for a proper time (between 100...10000) has lead to obtaining the flows and surfaces for stationary motion.

In Fig. 5 it is shown one of the options. We can notice the dishevelment time.

|--|

Studied section	Conductivity coefficient	2 drains option. The flow of the drain (m ³ /day/lm)		3 drains option. The flow of the drain (m ³ /day/lm)		
	(m/day)	D1	D3	D1	D2	D3
T2A	heterogeneous	0,825	0,792	0,710	0,235	0,480
	k = 0,5	1,076	1,217	1,927	0,156	1,042
	k = 0,3	0,814	0,795	1,287	0,187	0,669
	k = 0,1	0,271	0,089	0,468	0,006	0,102
T3A	Heterogeneous	1,532	1,251	1,408	0,245	1,147
	k = 0,5	1,860	1,682	1,843	0,033	1,667
	k = 0,3	1,291	1,307	1,197	0,202	1,214
	k = 0,1	0,462	0,458	0,440	0,055	0,434
T4A	heterogeneous	1,483	1,014	1,322	0,288	0,897
	k = 0,5	1,565	1,082	1,485	0,145	1,021
	k = 0,3	1,237	0,953	1,042	0,353	0,805
	k = 0,1	0,437	0,320	0,386	0,094	0,281

3. Conclusions

The results of the hydraulic calculation have been confirmed on the field. The drains have been executed in the recommended locations and they have drained flows anticipated by the predictions. In the same way, the level lowering of the underground water has had the same magnitude order as the one predicted by hydraulic study.

In heterogeneous ground conditions with sudden parameter changes between the closest areas and with the reduced permeability's of the ground, the only standing solution is the one of the drains parallel to the level lines of the versant, and the only applicable technology in acceptable conditions is the one of the horizontal controlled drilling.

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The Rehabilitation of the Underground Water System at PETROMIDIA S.A.

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Abstract: PETROMIDIA platform it is placed on the seaside line that lays between the Black Sea (water level \pm 0.00maBS), Navodari lake (the calculation level of the water + 2.50 maBS) and Corbu lake (the calculation level of the water + 1.50 maBS). Water losses and oil products from the plant network are superposed over the underground water current so that, in order to avoid sea pollution, has been projected and realized a protection system of 7 km. of imperfect perimetral screen and of 22 km. of drains inaugurated at the same time as the plant, in 1977-1980.

During the 20 years function of the drainage, the pollution and levels of the underground water have been monitorised through a piezometer network, and in 1996 have been reactualised field studies and have been several proposals for the completion of the drainage.

Keywords: water level, drainnage zistem

1. The effects of the drainage system on underground waters at Petromidia

The underground water solution has:

- An imperfect perimetral screen necessary for the separation of the industrial platform from the rest of the territory, for the retention of the oil products (immiscible or with volume weight smaller than that of the water), and for the reduction of the leaked out overflows from the Navodari and Corbu lakes, or towards the Black Sea.

- The perimetral drain intercepts the underwater current, collecting the oil products retained by the screen

- The internal draining insures the normal functioning of the networks and plants, without water and pollution levels increased requiring special waterproofing

The project conditions have been:

- the draining system will maintain the water levels at values under + 1.50 maBS;

- the drains will not entrap salted water which can affect the procedures in the purifying water plant;

- the drains will follow the scanning field of the street;

The drains have been designed to be made of stoneware with perforations in the upper midst of the circumference, with observation and keeping manhole with diameters between 200-500 mm;

The considered flows for establishing the drains dimension have been the following:

- the leaked out flow from the Navodari (60 l/s km) and Corbu lakes (30 l/s km);

the leaked out flow from rainfall $(7 \times 10^{-2} \text{ l/s ha});$

- the losses from the leaked out field networks: 1% of the emerging flow through short technological networks; 2% of the emerging flow through long technological networks ; and 10% of the household networks;

This way, the total of the drained flow has been approximated at 1000 m³/h, resulting a uniformly distributed flow of 1.2×10^{-4} l/s m².

On the plant, the calculations have shown that for the drains positioning at values between ± 0.00 and +0.50 maBS (the drain slope) can be maintained in under water drains under the value 1...1.5 ‰ if the drains are situated at a maximum distance of 200m between them.

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Have been obtained 22 km of drains (including the perimetral drain) held in 8 drain networks, each of them having its own pump

station (added 2 pump stations on the perimetral draining) the water is delivered under pressure towards the puryfing station (fig. 1).



Fig. 1. Drainage networks.

During the execution of the drainage certain changes and faults have came out, some of them objective and others unjustified.

Out of these are mentioned:

- the changing of some distances (lake- screendrain) with the consequence of modifying the leaked out flows
- the changing of the reversed filters or the improper assembling of it;
- the changing of some leak out courses due to the impossibility of performing certain under crossing;
- the changing of some draining pipes diameters out of administrative reasons;
- the making of the perimetral screen with lower depths than those designed on some pipe section, specially in the stone area towards the Black Sea;

- Beside the observations made along the execution, there have been formulated detailed

instructions for the reception and functioning of the drainage, containing the documentation, the washing of the drains, testing with increasing flows following the underwater levels and the clearing of the drained water, testing of the pump stations.

There have been elaborated instructions for the operating of the draining system, containing the program and measurements parameters, the formatting and rigging up of the intervention team, the interruptions and fault functions that can show up.

For financially reasons, all these recommendations haven't been respected, and the drains were unable to function for a long time.

The plant's employees have made only underwater and oil pollution measurements.

In 1995-1996 it has been established as necessary the checking up of the drainage, considering the high underwater levels.

We observed by field study that when measuring several drain section pipes was dysfunctional.

The effect of this state it is illustrated by the hydroisohypse map. This way we can observe the following:

- on a large surface the underwater levels are over passing the cote +1,50maBS, projected as a maximum in the design of the plant.
- *The underwater arches* are signaled in water economy areas in petro - chemistry and oil products tanks from Refinery, because of the losses from networks and installations
- These arches are assessing flow directions very different from the natural course
- At the surface of the underwater is has been formed a level of oil products, more or less continuous, with maximum width of 0.85 m, covering all most the entire surface of the Refinery
- the upper and lower screen piezometer pairs show that the oil products pollution hasn't over passed the screen because the perimetral drain has been functioning.

There have been determined the permeability coefficients, by experimental pumping, with values of 2.5...13 m/day (isolated 17 m/day), with a medium value of 6 m/day.

Field study showed the necessity of the rehabilitating of the draining system at PETROMIDIA. Also, for the north- east area it is necessary the expansion of the draining network, because it is estimated that this location should be used.

On real field conditions, especially because of the very high density of networks and plants, drains executed with controlled horizontal draining can't make the rehability solution.

The supplementary drains can't be made at different cotes, than the existing ones. The functions of such systems, with different coat drains, raises loading up problems of the drains, analyzed further.

2. Hydraulic calculation for the increasing of the drains networks

There have been used two models: vertical plan and horizontal plan.

The differential equations used are for the discontinues movement, with specific supply ε from the surface.

It has been estimated the formatting of the free surface of the water in time but also the situation continuous movement, maintaining the limit conditions unchanged for a long time.

The utilized schemata in finite differences are explicit, with stability conditions for space and time.

There have been analyzed a number of 27 options with the following variables:

- the permeability coefficients k=0.3 6 10 m/day
- the water cotes in drain 2 0.5 maBS; 0.0 maBS;
 + 0.5 maBS;
- the losses percent of the total discharge (80000 mc/h) of 1 2 3 %, distributed on a surface of 200 ha, lead to the special supply flow from the surface ε=0.0096 0.0192 0.0288 mc/day mp. The constant elements have been:
- water cote in drain 1: + 0.5 maBS
- the cote of the permeable bed: 13 maBS;
- Field porosity: n = 0.2;
- Drains distances: 200 m.

With out a details description of the results, there would be presented the conclusions of these cases.

The conditions for continues movement are realized n several days: for example, for k = 3 m/day losses percent of 3%, the free surface is forming during 10 days (fig. 2).

Between two parallel drains, at different cotes, for different permeability coefficients and with variable percent of losses, the maximum levels between the drains are presented on figure 3, with the following conclusions:

the water cote in drain 2 influences the maximum levels in rather small limits (for example, for k = 10 m/day, the raising of the drain 2 with 1 m represents the increase of maximum levels with 32 cm, and this is the biggest of the signaled raises in the 27 analyzed options)





- the losses percent of the total flow influences the maximum levels between the drains, specially if the permeability is low (for example, for k = 3 m/day, the increase of the losses percent from 1% to 3% produces supplementary raise of underwater levels with 0.9m)

- the + 1.50 maBS cote of the underwater is not over past if the losses percent doesn't over pass 2%; this cote is over past only for a 3% losses for lower permeability
- This calculations show that the 200 m distance chosen between the drains is very well chosen even for big losses percent from the total flow (2-3%);
- the local situations, where are concentrated very big flows must be examined so that the

drains should over take immediately the water losses and to be dimensioned in order to carry on this losses.

Working with calculations data shows up the unequal loading of the parallel drains at different cotes. The flow report can reach 4 for a loss of 1% and a permeability of 10 m/day and it is of 1.6 for $\varepsilon = 3\%$ and k = 10 m/day (fig. 4).



Fig. 4. The flow rapport between parallel drains.

With out detailed calculations, the conclusions are that the supplementary drains would be dimensioned so that can carry on this unequal loading.

On the horizontal plan model it has been considered a rectangle with sides of 200×400 m, in which on the sides N and W the drains are at the cote + 0.5 maBS, and on the sides S and E the drains are at the cote - 0.5 maBS.

For example, in one of the options (k = 6 m/day and ε = 0.0096 mc/day mp), the registered flows are: on the N side (of 400 m) it is registered 22% of the total flow, on the W side (of 200 m) – 5%, on the S side (of 400 m) – 52% and on the E side (of 200 m) – 21%.

In conclusions, on N and W sides, with drain cote + 0.5 maBS, is registered only 27% of the

flow, and the other sides, at drain cotes -0.5 maBS, is registered the rest of 73 %.

This calculations have been used to underline the principal solutions, further presented and will be taken into consideration when establishing the design level of the work.

3. Solutions for optimizing the drain functioning at PETROMIDIA

Before executing supplementary drains we propose the fallowing measures:

- the unsilt of the drains which are not currently functional, on networks, starting from the upper pump station, through scooping of the manhole, insuring all this time the continuous functioning the pump station

- by keeping on function the pump station it will be fallowed it's effect on the underwater level in the area and it would be checked out whether it is or not sand in the manhole; in this way we can conclude over the efficiency of the drain functioning and it can't be underlined the areas with drain fallowing and blocking
- the rehabilitation of pump station, replacing the machines so that the efficiency and the liability should be improved, and the starting or stopping on different levels to function normally.
- Considering the large density of pipes and cables on each drain, the incomplete or modified documents, it is recommended the investigation of the proposed traces for the geo radar drains.

The execution of supplementary drains it is necessary around the north lakes, or on D10 in the Refinery.

The supplementing or replacing solutions for the drainage system must be established according to the conclusions after applying the measurement mentioned below.

These solutions take into consideration the design criteria mentioned in chapter I, which must be respected further in the design section.

After the informations of this moment, in the completion solution of the drainage system would be executed on a first base 4875 m of drain, and on the second base another 675 m of drain, using 8 draining shaft.

In this solution, the cost control is smaller, but the vertical emplacement and the effective

dimensioning of the tubes should take into consideration the above and the hydraulic calculation for parallel drains at different cotes, with different charges.

In the replacing of the non functional drains solution, there would be executed on a first base 11365m of drain, in the second base another 3425m of drain, by using 11+2 draining shaft. This solution is more expensive and it raises certain problems regarding the drain tube diameter, which would have to unload large flows.

In conclusion, the recommended solution for PETROMIDIA contains the unsilt and rehabilitation measures of the existing works, mentioned at the beginning of this chapter, it implies a period of the evaluation of the obtained results. After that it would be indicated to start the design and execution of one of the completion solutions or replacement mentioned above.

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Appreciation's regarding the Ecological Rehabilitation of Waste Dumps for Agricultural Purposes

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Abstract: Agricultural valorification of waste dumps is performed using a specific technology, applied in two phases. After the mining and technical one, comes the biological valorification phase, when the using of certain crops imposes a special technology established by experimental ways.

Keywords: waste dump, corn, wheat, rye, sunflower

1. Introduction

From the total surface of the Gorj County (560.174 ha), the agricultural fields represent 250.254 ha (about 46%) from which 118.364 ha arable land. An important part of this surface (about 77.000 ha) is represented by not-formed soils. That is why the most important objective of the agriculture in the region is the using of new surfaces for agricultural purposes. A way to realize this goal is the improvement of the waste dump.

2. Waste improvement for their agricultural use

The wastes are made of a mixture of nonfertile rocks, with heterogeneous physical and chemical particularities, which make the works for their improvement very hard and impose a clearly differentiation of the technical solutions. The barren gangue material deposits into the quarry in the order of the exploited soil layers. A selective deposition of the barren gangue is difficult to realize, both technical and economical.

In the first 2-3 years, the materials deposited into waste dump are submitted to an intensive subsidence process, which determine the formation of creeps, that confer to the land a deficient external drainage. Because of this, each year must be performed surfacing works. Only after the relatively stabilization of the land's elevations will be applied above-ground with fertile soil.

Depending on the waste dump's cliff these lands have to be used for: agricultural corps (for cliffs of 10%), pasture (for cliffs of 10%-25%) and for forestry base (for cliffs more than 25%)

3. Results obtained from experiences with different agricultural corps

Researches performed on the Garla Rovinari waste dump from the experimental land of ICPA, present the following results.

For *corn crops* were studied 3 versions: on unfertilized land (V1), on fertilized land with N100 P80 K80 (V2) and on fertilized land with N200 P160 K160. The obtained yield values are resumed into the Table 1:

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lable I - C	Table 1 - Corn production (first year)							
Version	Production (kg%/ha)	%	Differences kg/ha	The meaning of differences				
V1 – reference	2197	100	mt					
V2 N100 P80 K80	3003	136	806	XXX				
V3 N200 P160 K160	3157	143	906	XXX				
DL 5%			263					
DL 1%			288					
DL 0,1%			324					

Table 1 - Corn production (first year)

The production increments obtained through fertilization are clearly significant relatively to the reference value, having limits between 806-960 kg/ha.

Production differences are rising also do to the weather conditions favorable for corn crops, the improvements of the physical and chemical characteristics of the soil during the regrowing years

The waste dump material, very heterogeneous, resulted from the brown coal exploitation from coal field Rovinari is poor on nitrogen and has an average contain of phosphorus and potassium; it could be used by its growing with corn crops, after application of bigger doses as for normal soils. It must be mentioned that the corn crop should be cultivated again every 4th year. In the second year the productions obtained from the three versions are illustrated into the Table 2

It can be observed significant production increments in the version 3 compared with the reference production, due to the using of chemical fertilizer, which also determine a production increments compared with the version 2. This fact could be explained by the better utilization of the fertilizers by the plants.

Table 2 - Statistical analysis of corn production (second year)

Version	Production (kg%/ha)	%	Differences (+/-) kg./ha	The meaning of differences
V1 – reference	15	100	mt	
V2 N100 P80 K80	1777	129	477	XXX
V3 N200 P160 K160	1863	136	563	XXX
DL 5%			266	
DL 1%				
DL 0,1%			277	

In the case of *winter wheat*, 29 Fundulea variety, sowed after corn crop,were studied 6 fertilization variants, as presented into the Table 3.

Table 3 - Statistical analysis of the variants of winter wheat production

Version	Production (kg%/ha)	%	Difference (+/-) (kg/ha)	The meaning of differences
V1 – reference	171	100	mt	
V2 20 t/ha manure $+$ 20 cm soil	2244	131	520	XXX
V3 40 t/ha manure $+$ 40 cm soil	2376	130	660	XXX
V4 N100 P60 K60	2112	123	396	XXX
V5 N200 P120 K120	2508	146	792	XXX
V6 N300 P180 K160	2640	153	924	XXX
V7 N100 P60 K60 + 40 t/ha	2772	161	1056	XXX
manure				
DL 5%			147	
DL 1%			201	
DL 0,1%			204	

The optimal doses are of 40 tones/ha manure, N100 P60 K60. Although the versions 2 and 3 were covered with 20 cm (V2) respectively 40 cm (V3) of vegetal soil, the productions are lower compared with the versions V6 and V7. Because of the high

cost of the vegetal soil transport to the surface unity, it is not recommended their covering.

The corn is a winter grain which finds favorable conditions for growing in the related area, if also the land designed for its growing corresponds (Table 4).

Table 4 - Experimental results obtained for corn production

Version	Production (kg%/ha)	%	Differences in kg/ha	The meaning of differences
V1 – reference	1222	100	mt	
V2 N100 P80	1352	110	130	XXX
K80	1895	155	673	XXX
V3 N200 P160				
DL 5%			113	
DL 1%			126	
DL 0,1%			260	

The corn, which grows on such lands, gives very good productions when the doses of active substance are much bigger than the ones for normal lands. The production of version 3 (1895 kg/ha) is clearly bigger than the reference value, due to the big fertilizer doses. In the version 2 was obtained a production of 1352 kg/ha (more than the reference value), but much lower than the production obtained in version 3.

Rye. The rye was sowed after the corn crop being the only crop which has results in the first year of its sowing on these fields.

Analyzing the table no. 5, it is to observe the significant differences, in all the cases, compared with the reference value, depending on the applied treatment. It to be observed that the biggest productions were realized in versions 6 and 7 (related to the reference value).

Table 5 - Analysis of the variation of rye crop

Version	Production	%	Difference (+/-) (kg/ha)	The meaning of
	(kg%/ha)			differences
V1 – reference	1600	100	mt	
V2 20 t/ha manure + 20 cm soil	1800	112	200	XXX
V3 40 t/ha manure $+$ 40 cm soil	2100	131	500	XXX
V4 N100 P60 K60	2000	125	400	XXX
V5 N200 P120 K120	2400	150	800	XXX
V6 N300 P180 K160	2500	156	900	XXX
V7 N100 P60 K60 + 40 t/ha manure	2700	168	1100	XXX
DL 5%			174	
DL 1%			244	
DL 0,1%			345	

Sun flower. The variety of sun flower used for experiences was Record, recommended for all the region, which was cultivated after corn.

By interpretation of the data regarding the productions of sun flower cultivated on the waste dumps could be appreciated that the significant production increments were obtained in the variants where average doses of nitrogen, phosphorus and potassium were applied. The conclusion is that the cultivation of the sun flower gave good results on these fields, with the condition of bigger doses of chemical fertilizers. 710 Appreciation's regarding the Ecological ... / Ovidius University Annals of Constructions **3**, **4**, 707-710 (2002)

Version	Production (kg%/ha)	%	Difference (+/-) (kg/ha)	The meaning of differences
V1 -not fertilized	850	100	mt	
V2 N100 P80 K80	1100	129	250	XXX
V3 N200 P160 K160	1430	160	580	XXX
DL 5%			79	
DL 1%			102	
DL 0,1%			132	

Table 6 - Synthesis of the results of experimental productions of sun flower crop

4. Conclusions and recommendations

The experimental results obtained in normal weather conditions show that the ecological recovery of sterile waste dumps for agricultural purposes is possible.

Related to the agricultural valorification of the sterile waste dumps, the following recommendations have to be made:

- during the first 2 years the productions of almost all experimental crops are very low or even zero. The explanation for it could be the unselective excavation of covering material existing over the coal. Beginning with the 3rd year the productions are significant, especially for wheat and corn crops; - is recommended that in the first 2-3 years all the secondary production to be incorporated into soil. This have catalyzed role for organic material formation necessary for soil layer,

- which is used for the growing and the development of plants.

- the weeded crops are recommended to be cultivated/experienced only beginning with the 2nd year since the material from waste dump was layered;

- the doses with organic and chemical fertilizers should be double or even triple than the ones used for normal lands in the area;

- the surfaces took over for recultivation must be maintained 5-6 years till they will be used into the agricultural production circuit; - the thickness of the covering soil layer must be of 40 cm for cereals crops and 1,20 cm for wine- and fruit growing ;

- in each cultivation year is recommended to perform leveling works with the bulldozer, because these lands are submit to compressions and unevennesses appear where the water stagnates;

- after the period of 5-6 years of biological cultivation is necessary to be made an agricultural and chemical mat plotting, in order to establish the fertility of those surfaces, based on which both the fertility group and their further utilization is established.

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Soil survey and Land Classification for Land Improvement

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Abstract: With the help of the necessary field investigation, an aerial photo interpretation map maz be transformed into a soil map, each soil unit having its own problems as for as drainage is concorned.

Keywords: soil, survey, photograph, land, improvement.

1. General aspects

Traditionally, the task of the engineer was to provide a system for conveying water from a source and distributing it equitably over an agricultural area. At a later stage, and quite recently in some countries, the appearance of saline and waterlogged ground demonstrated the fact that the removal of unwanted water is as important as the irrigation water supply itself, and so the engineer is now required to design the complementary supply and drainage systems.

This takes the from of apreinvestment or feasibility study in which the basic features are:

-A survey of resources of land and water

-An investigation into the present state of agriculture

-Outline designs for requisite engineering works -An economic

2. Soil surveys

The purpose of a soil survey is to define soil types, physical and chemical properties, drainage characteristics, and agricultural potential of land within the project area.

The soil survey can be based on aerial photographs which are interpreted by matching definable patterns on the photographs with soil characteristics as determined by vegetation, with soil characteristics as determined by samples from auger holes and trial pits. Field test sites are located by inspection of the photographs to give the

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maximum amount of information at the minimum expense.

Soil data related to agriculture, irrigation and drainage are presented in tables, charts, diagrams, and maps with explanatory notes, and appear as an appendix to the main report. The engineering properties of soil are normally set down in a separate section dealing with site investigations for structures.

For a well-balanced drainage plan, the following soil data will be required:

a).Hydromorphic soil properties

These include active gley phenomena, which give a reasonable idea of the fluctuations in groundwater levels;

b) Permeability of the soil profile for water.

-The following distinctions can be made:

-The property for the topsoil is the *infiltration rate*.

-For any Bt horizons, the property of interest is *unsaturated or saturated conductivity*

-Drainability of subsoil and deep substrata also in relation to the occurrence of any impermeable layers. Generally these deep substrata are saturated and the property of interest is the hydraulic conductivity.

c) Moisture storage in the soil

The amount of moisture should be done for each important layer or horizon.

d) Data on salinity and alkalinity

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-Degree of present soil salinity and alkalinity. -Estimation of possible salinization and alkalinization

-Estimation of the possibilities of desalinisation and de-alkalinization resulting from changed irrigation and drainage conditions.

e) Degree of ripening of the soil.

The degree of a soil's physical ripening is expressed in N-values for each of its layers. From this, one can calculate the future subsidence that will result from drainage.

f) Geographical distribution of soil mapping units.

If the soils are highly contrasting this can cause problems. Yet a given parcel of land will receive the same management even if the soil changes from one corner to the other.

g) External factors

Flood hazard on low-lying land, topography, vegetation, climate and effluent seepage of water or springs are external factors.

3. The use aerial photographs

A stereoscopic study of the aerial photograph may supply many valuable data on area's topography, hydrology, morphology and vegetation.

An aerial photo interpretation map can then be compiled on the basis of these data. This interpretation map is not a soil map, but it shows areas that probably has the same composition.

With the help of the necessary field investigation, the interpretation map may be transformed into a soil map, each soil unit having its own problems as for as drainage is concerned. The aerial photographs make it possible to save time and money, because the fieldwork can be reduced and carried out in the most effective way

For soil survey purposes, it is possible to use an aerial photograph or mosaic as a "map". For

topographical and design work, this is impossible, as the central projection results in deviations from the actual distances, which is of minor concern for soil survey work, but very important for the topographical and design work. A special photogrammetric procedure will be necessary to rectify these deviations.

For hydrological purpose, the aerial photographs give valuable information concerning the character (dry and wet areas) and approximate acreage of a catchment area to be drained.

Interpretation of the type of vegetation has to be done very carefully. Sometimes the vegetation dominates everything (Papyrus) and no differences can be observed on the aerial photo between mineral and organic deposits nor between firm and muddy areas.

The photographs used should therefore be as recent as possible.

4. Hydropedological data collected in specific surveys

The existing soil maps do not supply the necessary design data for a number of pedological aspects of a drainage plan. This is due in part to the fact that soil maps do not furnish specific data for drainage projects. On the other hand, it is a failing of many soil maps that various pedological properties are not given more exactly either in the map or in an accompanying report.

The existing soil maps imply the availability of important data, but it will always be necessary to collect supplementary details, especially for planning and designing purposes.



Figure 1. a. A part of the design drawing of the general layout of canals and drains



Figure 1.b. an aerial photograph of the same area.

Table 1.		
Required planning data	Relevant soil characteristics	Possibility of deduction from general soil map
Localization and estimation of waterlogging	-Gley sympous -Ground water fluctuation. Available moisture	-Cannot be fully deduced
Subsidence	-Compressibility and volume weight of the different soil layers	-Cannot be deduced
Desirable ditch water levels and drainage depth. Density of the drainage system	-Available moisture -Permeability -Irreversibile drying out	-Can only be partly deduced -Cannot be deduced (obser-vation depth limited) -Cannot be fully deduced
Water-storing capacity	-Tension-free pore volume, volume above phreatic surface	-Cannot be fully deduced
Stability of excavation. Permissible talud gradient	-Angle of internal friction, volume weight, cohesion, quicksand.	-Cannot be deduced (obser-vation depth limited)

The existing soil maps show observation to a depth of 120 to 150 cm.

This is not sufficient for the planning of main and subsidiary drainage.

For hydrological calculations it is sometimes necessary to know the composition of the subsoil to depths of 50 to 100 m.

5. Soil profile characteristics to be determined

a).Hydromorphic properties

The occurrence of waterlogging can be determined by means of soil profile characteristics: the presence of gley phenomena is a good indicator. Thus, a survey can show, fairly reliably, areas, with serious waterlogging or no waterlogging at all.

The depth at which reduction symptoms are observed is an indication of the degree of

waterlogging. From this information, conclusions can be drawn as to the required drainage measures.

The degree of waterlogging and the intensity of the required drainage measures have to be considered in conjunction with crops and climatological conditions.

The gley symptoms are caused by oxidation and reduction of iron compounds in the soil.

Hydromorphic characteristics, other than gley phenomena, are:

-Increasing humus content of the top soils, sometimes with peat.

-Unripened sub-soils mottling and manganese concretions in a grey El horizon and in the B horizon.

-Salinity increasing to words the soil surface in arid climates.

On the other hand fossil reduction symtoms may remain visible in soils, depending on the density of the soil, for centuries. This may lead to wrong conclusions.

b) Permeability.

Three aspects are distinguished; -the vertical permeability of the topsoil. -the horizontal permeability of the topsoil. -the transmissibility of the deeper subsoil.

c) Transmissibility.

The augerhol method allows the permeability to be determined but for the deeper subsoil we have to make use geohydrological observations to depths of about 100 meters.

d) Available moisture.

The amount of available moisture in the rootzone (the difference between field capacity and wilting point) is of vital importance.

e) Salinity and alkali problems.f) Ripening (physical and chemical).

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Studies and Researches Related to Retrieving Technological Waste Water

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Abstract: I hereby present a particularly important aspect from the operation plants, which produce drinking water, namely the retrieval of technological water. The retrieval of this water plays a very important role with a view to reducing the impact upon environment. Following the experiments carried out on pilot facility a series of advantages came out, such as: the significant reduction of turbidity, respectively the content in suspension, limitation of organic and biological overloading, mitigation of water colour value. This procedure tends to be compulsory in the future for all plants that produce water.

Keywords: drinking water, operation plants, retrieval of technological water.

1. Introduction

At present, environmental protection is a major problem of humanity. Man in its activity used nature resources and rendered them valuable, and this has always affected the condition of environmental factors. In order to develop and to assure their perpetuance, human societies created technologies for the activities that work (process) natural resources making environment artificial.

As regards water we can say that it contains very many substances, some of them with favorable action, the other ones having an unfavorable action upon human health. As we know it is extremely important to find out its effect before the human body gets sick.

Water meant for human consumption comes from underground surface resources which most of the times need previous treatment. Water treatment with a view to render it drinkable, may be defined as the process by which it is ensured the correction of the water quality from the respective sources, in order to meet the quality requirements imposed by the consumer.

The most affected environmental factors, by the treatment activity are:

- human settlements, by the quality of assigned (distributed) drinking water;
- the air of the respective area, by influencing the activity of water treatment

(disinfecting) and providing the station with thermal energy (thermal power station);

• surface water, by the quality of water resulted from technological processes (decanting, clarification, filtering) discharged into environment;

The paper presents studies undertaken in order to reduce impact upon environment and of specific consumption by retrieval of technological wasted water resulted from the treatment branch of Apa Rosu Plant from SC APA NOVA Bucharest SA.

2. Studies of literature

Studies and researches were also developed in other countries and as a result there were elaborated projects and technologies for retrieval and recycling of waste technological water within treatment Stations. The most recent technologies are based on fundamental researches carried out by AWWA Research Foundation between 1991-1993, which conducted experiments on 24 American plants. It is for the first time at world level when such a systemic research is organized its conclusions been materialized in an AWWA guide.

From the studies of literature data we can state that every plant has a unique scenario related to the modalities of water retrieval from filters washing and mud from decanters, thus: •Kanawha Valley (West Virginia Plant) recycles only water from filter 718Studies and researches related to retrieving.../ Ovidius University Annals of Constructions 3,4, 717-724 (2002)

washing, the mud from lamellar decanters that is discharged into the sewerage;

- in case of the plants: "Swimming River" (New Jersey) and "New Castle" (Pennsylvania) there is retrieved both water from filters washing and those resulted from the mud of decanting thus achieving only the mixture supernatant from the 2 water categories;
- Water from filters washing is primarily decanted and the mud resulted is mixed with mud from decanters and with the supernatant from filtering press in gravitational thickeners or lagoons; the supernatant from these, is mixed with the supernatant from primary water decantation from filters washing and it is recycled with the water from the source by treatment operations;
- The plant, which produces drinkable water -(Moshannon valley (Pennsylvania) presents the only technological scheme where mud from the decanting stage (clarification devices with absorption on support) are directly mixed with waters coming from filters washing; waters resulted are primarily treated after which the supernatant is recycled and the mud is dehydrated on drying beds the draining water from drying beds is introduced again into the primary decanter of the recycled water.

A special study was undertaken at the water plant of Bangor and Moshannon (Pennsylvania) in order to determine the influence of biological characteristics of waters

coming from filters washing, upon the treatment operations.

It was pointed out that waters coming from filters washing contain a great amount of

Giardia and Cryptosporidium cysts as compared to water from the source. Waters resulted from filters washing at Moshannon plant, had a level of Giardia and Cryptosporidium cysts of 150 units/l, while at Bangor plant the number of cysts were of 8-14 units/l; the concentration of cysts for raw water (natural water) of the 2 plants was of 0.05 -3cysts/l; even after the primary sedimentation of waters coming from filters washing, the supernatant contained a larger larvae quantity than raw water. In these situations the study requires an efficient sedimentation of waters coming from filters

washing. There were obtained favorable results with efficiencies over 85-90% using polymers in dosages (proportions) of 0.5-0.8 mg/l.

2.1 Study carried out for Rosu Water Plant

In order to evaluate the impact upon environment, produced by technological waters of Rosu Water Plant there were taken into account both the quality and quantity of the respective waters.

The scheme for treating the water from Rosu Water Plant is a classical one: pumping of raw waterdecantation-filtering - disinfection (decontamination), used in most of water plants from Romania.

a) Decanters: The station has 6 suspension decanters (decanter with pulsation system), with sludge recycling, dimensioned at 1 cubic meter each.. At present these decanters are operated with over 20% as compared to the flow of dimensioning. The system for collecting and evacuation of technological sludge is made up of :

- bridge raclor, which ensures mud collection and discharge towards a central canal of trapezoidal shape;
- the canal for collecting mud, with base located at 120 degrees one from the other from which start hydraulic circuits branches for mud discharging (diameters of 200 mm in the I-st stage respectively 250 mm in the II-nd stage);
- system for automation, for opening and closing of hydraulic circuits. This system is made up of the limits for closing and opening activated by the scraper system.

b) Rapid sand filters: in the Rosu Water Plant is achieved with 30 rapid filters, with steady flow and compensation of load losses, dimensioned for a flow of 800 cubic meters/h each. Technological losses in this section come from filters washing. Filters washing is achieved in 2 stages: the first stage, with air and water in counter-flow, and the second, only with water, by means of 3 pumps, located in the basement of filtering station.

3. Aspects about technological waters from Rosu Water Plant

3.1 Generality

Technological waters from Rosu Water Plant represent a mixture made up of waters coming from filters washing, sludge discharge from decanters and

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losses derived from filters and decanter. This waters are characterized, in principal, through a high suspensions content (table 1).

	Suspensions		
	content-	Average	Raw
Dov	technological	dosage	water
Day	water-daily	$Al_2(SO_4)_3$	turbidity
	average values	(mg/l)	(NTU)
	(mg/l)		
13-Sep	740	40	58
14-Sep	1816	37	55
15-Sep	1010	35	50
16-Sep	1593	36-46	76
17-Sep	1352	45	90
27-Sep	2100	30	42
28-Sep	2056	30	50
29-Sep	744	30	38
30-Sep	1204	30	32
1-Oct	2039	25	38
25-Oct	1516	50	103
26-Oct	1316	50	88
27-Oct	1245	50	82
28-Oct	2636	47	88
29-Oct	1287	45	82

Table 1. Suspensions content – technological water

The values from table 1, interpreted graphic, make evident 3 zones:

 plateau of the graph around at 600 mg/l, during double filters washing;

values over 1500 mg/l – technological water from decanters.

Right now is estimated to 5% technological losses total value in Rosu Water Plant.

3.2 Qualitative aspects

Technological waters presented a fluctuating turbidity owing to filters washing, respectively sludge discharge from decanters conditions. Average value turbidity is 1120 NTU, with the maximum 9800 NTU, owing to low average value pH- 7.1.

Suspensions content is change depending turbidity; having average value 1680 mg/l, with a maximum 7000 mg/l. At this value the maximum values of oxidizing organic matters is 380 mg $KMnO_4/l$, and average is 111.8 mg $KMnO_4/l$.

Heavy metals concentration in tehnological waste waters had overrun from STAS 4706-88 (table 2):

Table2. Average values of	heavy metals
concentration	

	Cd^{2+}	Cu^{2+}	$\mathrm{Fe}_{\mathrm{tot}}$	иМ	Ni^{2+}	Pb^{2+}	Zn^{2+}
Technological waste waters values (mg/l)	SLD	0.02	0.32	0.12	SLD	SLD	0.03

S.L.D.= under detectable limite (0.032 for Cd^{2+} , 0.063 for Ni^{2+} , 0.100 for Pb^{2+}).

Aluminum content values in homogenized samples was between 0 and 0.07 mg/l.

Dizolvated oxigen values was comparable with those obtain for the influent of Rosu Water Plant.

The compartment with the hightest development in aquatic biocenosis is the phytoplankton, represented by vegetal microscopically organisms, the diatoms algues having the hightest effective (Phyllum Bacillariophyta); the predominance are Fragilaria and Melosira, with special sanitary semnification because favourable its potential to multiply very intense in conditions for development its species and that can modify physical and organoleptical characteristics of water; it's important for mention that for all these cycles, the two genus have the high-test effectives in comparison with the others plankton organism.

The zooplankton is represented by species of phyllum Ciliata and Rotifera with the populations very reduced in comparison with the phytoplankton.

The majority of identified organisms in influent's water presents latent life, without have the characteristics of organisms that usually there Are in fresh waters (free movement, evident differentiation of the contour of cellular membrane by comparison with the particles being in the water, specific color etc.), because the processes of flocculation that the sludge suffered and the processes of elimination at the filter's washing. Except dead organisms, many organisms doesn't present usual dimensions or are partial degraded, the filamentous organisms (genus Melosira) are fragmented, the algues usually pigmented are decolorated etc.



Figure 1. Duration curves - suspensions content

4. Experimental conditions

The study assumed the treatment of technological waters on a facility for retrival made up on:

- basins of sludge natural sedimentation,
- chamber of rapid response,
- chamber of slow response,
- lamellar decanter and quick filter (fig.2).

The main phys-chemical and biological indicators of the above mentioned waters are given in table 3.

The parameters of the facility for retrieval of technological waters were:

 $- Q = 1.25 \text{ dm}^3/\text{s}$

- Time for natural sedimentation: t=1.9 hours for a basin;

- Hydraulic loading for a basin with natural sedimentation: I_{h} = 2 m 3 /s,m $^{2};$

Hydraulic gradient – quick response chamber: G = 400 s - 1;

Table 3 Physical-chemical and biological indicators of technological waters

Crt. No.	Analyzed parameters	Values	Admissible values STAS 4706-88
		064	Cal. I
1	(NTU)	264 - 9105	-
2	pH (pH units)	6.7 - 7.2	6.5 - 8.5
3	Oxidizing organic matters (mg KmnO ₄ /l)	47.2 - 143.6	10
4	Colour (units Pt – Co)	2 - 5	No colour
5	Content of suspensions (mg/l)	794 - 2336	-
6	Total iron (mg/l)	0.010 - 0.032	0.3
7	Manganese (mg/l)	0.018 - 0.200	0.1
8	Calcium (mg/l)	32.06 - 68.00	150 200 300
9	Plankton (no./l)	$2x10^{6}$ - $20x10^{6}$	-

Note: Most of plankton organisms are characterized by latent life, without having the characteristics of organisms which are usually found in shallow (surface) waters (free motion, obvious differentiation of the contour of cellular membrane as compared to particles from water, specific color etc), due to coagulation and flocculation processes and to the elimination processes achieved by filters washing.

Technological water retrieval installation experiments made in 3 cycles:



Figure 2 The scheme of the facility for technological water retrieval

5. Experimental results

5.1 Physic-chemical aspects

Concerning experimental results wen the observation :

- as a result natural sedimentation the turbidity values was in 10% cases, in cycle I, in 80% cases in cycle II, and in 60% cases, in cycle III, under 100 NTU (figure 3);in 80% cases the technological water turbidity values was over 1500 NTU.
- the value of pH for decanted, respectively filtering water was between 6.5 7.35 (figure 4).
- cycle I reactive agents is aluminum sulfate 5% in combination with anionic polymer AN 910 PWG – 0.05%;
- in which concern efficiency retaining of oxidable organic matters content (CCO-Mn), these different in those three cycles experimentally in function so organic loading initial of technological waters; average values of oxidable organic matters content of technological waters was 121 mg KMnO₄/l in cycle I, 140 mg KMnO₄/l in cycle II and 78 mg KMnO₄/l in cycle III (figure 5), the total efficiency of retaining on filtering phase was 84 - 95% (figure 6);

the content of matters in suspensions of technological waters was reduced in proportion as 94% after natural sedimentation phase and in totality (99%) after decanted phase (figure 7).

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cycle II – reactive agent is AVR product (iron polysulfate) – 10%;

 cycle III - reactive agents is aluminum sulfate – 5% in combination with anionic polymer AN 910 PWG – 0.05% (for pH adjustment value is used Ca(OH)₂ – 1%.

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view is the phase of decantion for cycle I, respectively that of filtration for cycle II and III (efficiency per phase); even if then number of

Figure 3. Duration curves of turbidity values on treatment phase



Figure 4. Duration curves of pH values on decanted, respectively filtering phases

5.2 Biological aspects

The results of biological determinations in the experiments showed the followed aspects:

- is evident a progressive reducing of the number of planktonic organisms on the treatment technological process;
- the pre-decantation reduce the planktonical effectives with 57 61%; but, the most the phase efficiency treatment phase in this point

organisms at the enter of installation is doubled for cycle II, in comparison with cycle I, total efficiency of planktonic reducing at the exit of installation presents identical high values (98.81% for cycle II, in comparison with 97.97% for cycle I, respectively 97.01% for cycle III), in the others words, the performances maintain even in cause of an input doubled of the planktonic

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organisms number; the maximum total fficiencies were obtained at the last treatment phase – the filtration;



Figure 5. Average values of CCO-Mn on treatment phase



Figure 6. Total efficiency of retaining CCO-Mn on treatment phase



Figure 7. Efficiency of retaining the matters in

suspensions on treatment phase

- the values of the efficiencies raported at the influent for the predecantation (57-61%) decrease at 89-92% for the decantation and respectively at 97-98.8% for the filtration on the sand (table 5).

Table 5. The reducing efficiencies of the plankton at the phases of treatment

	Cycle I		Cycle II		Cycle III	
	Efficiency /phase	Total efficiency	Efficiency /phase	Total efficiency	Efficiency /phase	Total efficiency
Pre	56.8	56.8	61.1	61.1	59.9	59.9
decantation						
Decantation	80.6	91.6	71.8	89.0	75.5	90.2
Filtering	75.7	98.0	72.7	97.0	87.9	98.8

5.3 Proposals concerning technological water volumes decrease deficiency

Because the decanters exploitation flow is 20% greater than designed flow, is find some problems in decanters exploitation:

- doesn't ensure recycled sludge from decanter;
- the water flowing speed from decanters is greater.

This difficulties involved decanters exploitation, which admit of the compromise to evacuated decantation discharge sludge immediately after producing, fact which generated a large volumes of technological water. For decreasing technological water volumes is necessary to use the flocculation adjuvant.

Concerning of the filtering phase:

- flowing water from granular materials doesn't make uniform on the filtering surface, because wrongly washing filters;
- washing filters is effected with losing granular materials because of washing equipment conditions (pumps and blowers);
- because of water aggressiveness draining doesn't provided;
- quality filtering water is not satisfying; fraction sand is presented in filtering water tank from Rosu Water Plant and Bucharest.

All of the inconveniences are generated technological losses in filtrating and decanting phases, fact which leaded to 5% losses.

Thus is necessary to improved:

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- changing of the granular materials and improvement draining process;
- changing washing equipment and fitting.

6. Conclusions

Concerning researches about technological water retrieval from Rosu Water Plant followed:

- reduction technological waters volumes
- technological water treatment to utilization in technological scheme for treating or/and supplied as industrial water.

Retrieval of waters coming from filters washing and of the sludge from decanters represents a particularly important aspect, witch will be compulsory in the future for every plant producing drinkable water. This is required because the necessary quantity of fresh and quality water increases although water resources remain the same so we have to find out solutions in order to assure the quantity requested by consumers. In the situations when the percentage of technological water lost by a water plant, exceed 7%, be method of retrieving this water becomes even economical, since the costs involved are more reduced that the procurement of water from the source.

This problem must be looked from qualitative and quantitative point of view.

Quantitative aspects are referring to reduction technological waters volumes. In case of Rosu Water Plant it make through:

- utilization flocculation adjuvant in decanting phase;
- improvement of sludge discharge / recycle;
- replacement of granular materials in filtering phase;
- improvement of draining system;
- replacement of washing equipment and fitting;

From qualitative point of view technological water that is processed in the retrieval facility is integrated with its most indicators, in STAS 1342/91, fact which points out the efficiency of such a facility.

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Studies and researches related to the use of iron chloride in the processes of making water drinkable

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Abstract:

The ever increasing requirements in the field of treating water in order to become drinkable, at the same time with the continuous degradation of water sources, involves the process of finding solutions regarding treatment technologies which should lead to a water quality so that this should meet the consumers' requirements. To this purpose, the S.C APA NOVA BUCHAREST S.A.'s concern were directed towards the optimization of treatment processes by finding alternatives to the use of the "classical" cake of alum.

The present paper makes known the studies and researches carried out for Arges water source, with a view to optimizing the coagulation-flocculation processes, irrespective of the season and on water quality. There were developed experiments at pilot scale in parallel, on two technological facilities (plants) with the same technological parameters, using the iron chloride, cake of alum and cake of alum combined with an anionic polymer.

1. Introduction

Coagulation represents the physic-chemical process of modifying (correcting) the turbidity by formation of particles, which may be retained and, respectively removed.

Coagulation-flocculation processes of colloidal particles are developed in three stages [1]:

- *Neutralization of electrical load* of colloidal particles by addition of coagulating agent; in this stage an energetic blend is recommended;
- *Formation of micro-flocs* (micro-flakes) by particles collision, due to Brownian movement (peri-kinetic stage)
- *Formation of macro-flocs* as their size increases (ortho-kinetic stage), these make a movement to deposit themselves.
- The criterion which leads to the most favorable results in estimating the efficiency of coagulation-flocculation, taking into account water "quality", reagents quality and the hydro-dynamic conditions, is given by the product- G x C x T (gradient, concentration, reaction time), experimentally determined for each water category.

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The usual values of speed gradient [2] are of the order 400-1000 s in the coagulation stage, respectively 100 s in the flocculation stage. In practice it is used the speed gradient corresponding to the turbulent regime [2] which is calculated with the following equation:

$$G = \sqrt{\frac{P}{V\eta}} \tag{1}$$

Where: G – average gradient of speed (s);

P – real dissipated power (W);

 η - dynamic viscosity (kg/ms);

V – volume of reaction chamber (m³).

Modern clarification is mainly based on retaining mechanisms in the suspension layer. Gregory [3] establishes the relationship which rules (monitors) the process of retaining flocculated colloidal suspensions in the suspension layer.

$$\Phi = \text{UsC} \tag{2}$$

Where: Φ - flow (mass rate of sedimentation);

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Us - sedimentation speed of suspensions; C - concentration of particles in suspension.

If it is achieved a suspension clarification in a tube with the height and diameter great enough, and if it is measured the height of suspension layer (layer-decanted water interface), depending on time, a curve is obtained (fig.1) which presents the following distinct stages:



Figure 1. Phases of suspension

- A-B area; the separation surface is not clearly delimited; this is the stage of coalescence;
- B-C area; a straight line which represents the constant speed of sedimentation v₀ (the slope of straight line); speed v₀ is, for a tube of given dimension, depending on the initial concentration of solid matters and on the flocculation characteristics of suspensions; if the initial concentration Co increases, the sedimentation speed decreases;
- starting from D, the flakes (flocs) are agglomerated and exert a compression upon lower layers.

Among the factors that influence the coagulation-flocculation processes we mention:

- (1) dosage of coagulating agent;
- (2) pH;

(3) - concentration of colloidal substances (turbidity);

(4) - anions and cations from the solution;

- (5) TOC and COD (or color);
- (6) the effects of blending (mixing);

(7) - electrophoretic mobility or zeta potential(8) - temperature.

2. Experimental conditions

The studies and researches assumed the carrying out both of coagulation-flocculation tests, and experiments on pilot technological ways – based on the comparative testing of iron chloride, of aluminum sulfate respectively.

2.1 Technological aspects

Experiments at pilot scale were gradually developed, in parallel, on two technological ways having the same technological parameters (I = 4m/hm – for decanting devices and respectively V = 5 m/h for filters).

The first experimental stage assumed the following:

- simulation of technological processes from Rosu water plant by using on way 1 the aluminum sulfate in solutions of concentrations ranging between 11 and 17%, the dosages being maintained at the same values as in Rosu plant;
- using the iron chloride on way 2 the values of
- dosages being variable depending on the quality indicators of clarified, respectively filtered water.

In the second stage it was tried to model the technological processes of Rosu Water Plant in case when it will be used the cake of alum in combination with an anionic polymer AN 910, respectively AVR. Thus, for way 1 it was used as coagulating agent the cake of alum(Al So) in combination with an anionic polymer AN910, respectively AVR, and for way 2 - iron chloride (FeCl₃).

The concentrations of coagulating reactive agents on the 2 ways were: 16-17% for $Al_2(SO_4)_3$; 0.05% AN 910;10% AVR and 3% for FeCl₃.

2.2 Characteristics of the source

Experiments took place in the cold season, months of January-February, water plants encountering great difficulties in treating water in order to render it drinkable in this season.

From physic-chemical point of view water was mainly characterized by:

temperatures between 3 and 7 Celsius degrees;

- values of turbidity of 20-45 NTU, respectively of the pH : between 7.8 and 8.5;
- content of suspensions included within the limits: 25-54 mg/l;
- chemical oxygen consumption with the average value of 7.91 mg KMnO₄/l, the maximum being 9.66 mg KMnO₄/l

The biological determinations achieved for the water from Arges river during the experimental period pointed out the following:

- plankton abundance had values between 472000 and 1431667 plankton organisms/liter. Small values were recorded when water temperature was lower (3-4 Celsius degree), and great values for the increasing water temperature (7-8 Celsius degree);
- the biocenotical structure of plankton revealed a particular complexity; together with diatoms (Bacillariophyta branch), in phitoplankton were present species of green algae (Chlorophita), blue algae (Cianophita), euglenophits and golden algae (Crysophyta);
- the prevailing species of phitoplankton are Navicula and Stephanodiscus the latter species is in a progressive increasing at the same time with water temperature increase;

As regards water quality from microbiologic point of view, STAS 4706-88 norms as an indicator of quality for category I waters subject to the process of changing them into drinkable waters, only the total coliforms indicator.

During the development of experiments there was registered an excess of unprocessed water, the values not being accepted for surface water meant for the process of treatment in order to render water, drinkable, thus being able to create problems from qualitative point of view both in the way, and in the distribution network.

3. Experimental results

Studies and researches assumed the evaluation of iron chloride behavior in comparison with the cake of alum in the technological processes and establishment of utilization opportunity in close correlation with operating costs.

Experiments at pilot scale were developed gradually, benefiting from 2 technological lines with the same technological parameters and had

mainly aimed at improving the conditions for developing coagulation-flocculation processes in clear and cold water.

3.1 Experimental results - stage I

In this experimental stage on way I it was used aluminum sulfate with concentrations between 11-17%, and on way 2 the iron chloride with concentration of 3%. The experiments pointed out the following:

- for the clarification stage on way 1, water turbidity had an average value of 3.3 NTU, with values between 1.52-10.5 NTU, and on way 2 it was registered an average value of 3.55 NTU, with values between 1.29-10.12 NTU; By increasing the dosage of iron chloride from 4 to 11.37 mg Fe/1 (thus achieving the equivalence with the dosage of aluminum sulfate). The values of decanted water turbidity on way 2 were under 3 NTU; for the filtering stage way 1- it was obtained an average
- value of turbidity of 0.51 NTU, as compared to the one on the way 2 which was 0.29 NTU; the variations of reactive agents dosages, respectively of decanted water turbidity are shown in fig. 2 and 3;
- average value of water pH on way 1, were 7.24/decanting, respectively 7.06/filtering;
- oxidizing organic matters (CCO-Mn) were retained with comparable efficiencies, however greater in case iron chloride is used: 26.8-47.8%, as compared with the use of aluminum sulfate 26.7-47.2%;
- the values of indicator that shows the content of total iron, for the filtered water/way 2, did not exceed the limit admissible by STAS 1349-91, at the same time with the change of reactive agent dosage, there are registered some increases of total iron concentration for the decanted water (fig. 4); the concentration of residual aluminum did not exceed the maximum admissible value for drinkable water, thus maintaining itself within the range of 0.01-0.04 mg Al³⁺/l;
- on both technological ways the biological charge (loading) decreases progressively. The best results are obtained in the last day of experiments (fig. 5), although the biological loading in raw water is the greatest in the last day;



Figure 2 Variation of aluminum sulfate dosage, respectively decanted water turbidity



Figure 3 Variation of iron chloride dosage, respectively decanted water turbidity

 high efficiencies per stage, as well as total efficiencies of reducing the organism number/l, are obtained for decanting device 2 as compared to the decanting device 1, but the filter F1 presents the maximum efficiencies; the results of bacteriologic analyses ("total coliforms" and "fecal coliforms") pointed out the better performance of way 1 for treatment, as compared to way 2 towards the end of experimental stage (table 2).



Figure 4 Variation of total iron content, respectively chloride iron dosage



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 Table 2. The results of bacteriological analyses on technological ways

	Day						
	20.02.2001		21.02.2001		23.02.2001		
Water test	CT/ 100 cm ³	CF/ 100 cm ³	CT/ 100 cm ³	CF/ 100 cm ³	CT/ 100 cm ³	CF/ 100 cm ³	
В	6300	700	9180	542	9400	1090	
D1	17	5	16	5	2	0	
F1	5	0	6	0	0	0	
D2	26	18	22	14	141	22	
F2	0	0	0	0	49	7	

3.2 Experimental results - stage II

In this stage it was tested by comparison the cake of alum with concentrations: 16-17% in combination with 0.1 mg/l anionic polymer AN 910 0.05% (stage I), respectively AVR 10% (stage II) on way 1 with iron chloride with concentration of solution: 3% on way 2.

Experiments evidenced the following:

- on way 1, the turbidity of decanted water had values between 1.36-15.1 NTU, with an average value of 3.53 NTU, in the conditions when the dosages of the cake of alum were kept relatively constant; great values being obtained at the beginning of experiment; for filtered water, there were registered values between 0.17-0.73 NTU, with an average of 0.35 NTU;
- on way 2, for the decanting stage (clarification stage), the values of turbidity varied between 1.81-13.80 NTU, registering an average of 3.20 NTU; it is necessary to be mentioned that the values over 5 NTU were obtained at the beginning of experiment, when the suspension layer was in the process of formation, and the dosage of iron chloride was 4 mg Fe³⁺/l; this one being maintained around the same value during the experiment; the average values of the turbidity of filtered water from way 2 is similar to that of filtered water way 1;
- the dosage of Al₂(SO₄)₃ was maintained from value point of view within 12-14 mg/l interval, and the dosage of FeCl₃ was integrated between 3.5-4.0 mg Fe³⁺/l (fig.6), thus, obtaining average pH values which were

comparable, for the 2 technological ways: 7.16-7.22 for the decanted water and respectively 7.24-7.31 for the filtered water;

- as regards the content of organic matter, there were found out comparable efficiencies of retaining on the clarification stage, but however, better in case of using the combination: aluminum sulfate + polymer as compared to the dosage of iron chloride utilization (3%), respectively AVR (10%);
- the determinations achieved for the two indicators – "total and fecal coliforms" – evidenced the fact that the greater number of "total coliforms" in stage II determines their increase on the way. There is a difference percepted especially at the levels of clarification devices between the two stages; after filtering, this difference between the two stages is reduced for the filter of way 2 (33% in stage I and 49% in stage II);
- as regards the content of total iron, values were obtained between 0.004-0.023 mg/l for filtered water – way 2 and content of residual aluminum for filtered water – way 1, was 0; these values are (integrated) within the limits imposed by STAS 1342-91;
- On both ways, the numerical weight of plankton progressively decreases (fig.7); the initial (raw water) being greater in stage II, naturally incurs higher values of the number of organism/l at clarification.

4. Conclusions

The conclusions drawn from these researches point out the alternatives, which exist as regards the development of coagulation – flocculation – clarification process. Experiments were developed in parallel, on two models, which had the same source of raw water and the same technological parameters, fact which enabled a realistic evaluation of the advantages – disadvantages of each tested product.

The development conditions, from the qualitative point of supply water, were of the kind that often incur problems in the production of drinkable water (clear and cold water, with corroding characteristic).



Figure 6 Variation of iron chloride dosage, respectively of decanted water turbidity



Figure 7 Variation of the number of plankton organisms by treatment stages

It is mentioned that there were modeled technological processes from ROSU WATER PLANT, using the same values of concentration, of reactive agent dosage and of hydraulic loading of clarification devices; this fact may constitute the starting basic point for a possible implementation at industrial scale. From physic-chemical point of view, the determinations carried out on water samples collected on the treatment steps, byobserving the technological time, pointed out the following aspects:

- in case iron chloride was used, the values of residual turbidity were smaller in all experiment stages, as compared to the values obtained when the other reactive agents for coagulationflocculation were used. This aspect was best pointed out in the first stage of experiment, when the temperature of raw water registered the lowest values $(3-5 \ ^{0}C)$;
- an important aspect related to the use of FeCl₃ was that of STAS 1342-91, the use of Ca(OH)₂ being not necessary for adjustment;
- due to the quality of influent the duration of a filtration cycle considerably increased (practically it was doubled), fact that involves a incurs saving, mainly, as regards the consumption of electrical energy;
- as regards the retaining efficiency of oxidizing organic matters (CCO-Mn), it is values were comparative for each experimental stage, however an increase was pointed out in the case of using the cake of Al₂(SO₄)₃ 16% combined with the polymer AN 910 0.05% in stage I;
- the value of residual aluminum were in 80% of cases, equal to 0, the maximum value did not exceed 0.03 mg Al³⁺/l; and the content of total iron had values which do not exceed the limits provided by the quality normative of drinkable water;
- during experiments, for both technological ways, the effluents showed a strong corroding characteristic (the value of aggressiveness index Ryznar being > 9), determined both by the source of raw water, and by the reactive agents used.

From biological and microbiological point of view we can assert the following:

- the results of biological and microbiological determination evidenced in the first stage of experiments that we obtain better results in reducing plankton and microorganisms when we use the iron chloride as coagulation-flocculation agent, as compared to the aluminum sulfate; but, bacteriological indicators show a better efficiency in eliminating microorganisms for decanted water when using the aluminum sulfate (way 1);
- in comparison with the use of the aluminum sulfate, combined with polymer AN 910, in phase II, stage I, for the iron chloride there are obtained lower efficiencies in retaining plankton organisms and bacteria;
- when using the iron chloride in comparison with AVR product, the biological indicators on

decanted water and the microbiological indicators show a better rate of retaining for way 1 on which AVR was used, as compared to iron chloride; but the lowest values of these indicators are registered when using the iron chloride as coagulationflocculation agent.

In order to optimize the exploitation of water producing plants, besides the technological and qualitative aspects, one should also take into account the economic aspect. Thus, in the present case, starting from the prices for acquiring the reactive agents used and taking into consideration the dosages applied, there is pointed out a net difference which is in favorable of using FeCl₃ (table 3):

Table 3 – Economic aspects of using iron chloride in the processes of water treatment

Experimental phase	Reactive agents used		Cost of reactive agent / m ³ of treated water (lei)	
Ι	Al ₂ (SO ₄) ₃ 14-17%	FeCl ₃ 3%	78.80	31.34
II	Al ₂ (SO ₄) ₃ -16% + AN910- 0.05%	FeCl ₃ 3%	82.17	31.34

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Polyaluminium Chloride – an Coagulation Agent Used within Effluent Treatment from Papermaking Process Based on Waste Paper Fibres

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Abstract The use of the waste paper in the paper and board manufacturing process, and the increase of water reuse rate reduce fresh water comsumption and increase the level of contamination. Waste water containe suspended, disolved and coloidal materials, both organic or mineral.Solids represent the main part most of this materials, but some colloidal or dissolved is present in solution. This paper emphasize the possibility to diminish the content of total suspended solids (TSS) and chemical oxigen demand (COD) in waste water. Polyaluminium chloride in substitution for aluminium sulphate was tested as coagulation agent for the treatment of the effluent from paper and board manufacturing process.

Keywords: wastewater, flocculation agents, effluent.

1. General

In the papermaking process, water is an essential resource. Water is not discharged after used, almost all is reuse a within the paper mill.

Average discharge of water from Western European paper mills are now less than 20 cubic metres per tone of paper. This is a substantial reduction from the 1970's figure of approximately 100 cubic metres per tone [1].

Table 1 -	Product	specification

Nr.	Product group	Specific water
crt.		demand, m ³ /ton
1.	Massive board	0-5
2.	Corrugated	5 - 10
3.	News print	15
4.	Fine paper	15 - 20
5.	Tissue (waste paper)	15 - 25

Now, "Zero Liquid Effluent" is a concept that implies to accomplish within the papermaking process a liquid effluent reduction to a level in at wich the environmental impact is minimized. The average specific water consumption is of approximately 20 cubic metre per ton product, including a water reuse rate of about 30 (table 1).

When the waste paper is used in the manufacturing processes, the water discharge of increase and effluent treatment become more difficult.

Clarification of waste water depends on the characteristics of the solid and colloidal matter that must be removed.

Chemical coagulants used in our country are: aluminium sulphate, and other bi or trivalent salts, such as iron sulphate, iron chloride, lime, etc [2].

This paper showed competitivity of polyaluminium chloride in comparison with the mineral agents, such as aluminium sulphate.

2. Experiments

We have tested cleaning process of waste water from the paper and board manufacturing, with polyaluminium chloride (ALPOCLAR 200). From AUSIMONT Sp A, Italy.

The main characteristics of product are:

- chemical name Polyaluminium chloride
- chemical formula Al_x(OH)_yCl_z(SO₄)_k

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Tipical properties:

- appearance yellow liquid
- Al_2O_3 , % w/w 17-18
- •density (20 deg C), $g/cm^3 1,37-1,41$
- viscosity (20 deg C), cP 70-90
- pH (20 deg C) 0,5-1,0

In this experiment at laboratory level, we used effluent from a paper and board mill that used 100% wastepaper fibres and coagulation products, polyaluminium chloride and aluminium sulphate.

Efficiency of coagulation agents was determined by total suspended solids (TSS) and chemical oxxigen demand (COD) content, using PAC and AS.

The first effluent was from paper manufacturing, and second from board.

Quantities of coagulation agents was 100 - 500 ppm for AS and 30 - 50 ppm for PAC.

Results and discussion

The test results are graphically outlined in table 2, 3.

From these figures, we can see a decreasing level of the total suspended solids content (TSS) over than 98%, using of 400 - 500 ppm aluminium sulphate (AS) and 40 - 50 ppm polialuminium chloride (PAC).

The decreasing of the chemical oxigen demand level (COD) is established also for the same coagulation agents.

A decrease of addition levels over 60% percent was obtained for 400 ppm AS and 50 ppm PAC.

In this case, the ratio of optimal quantities of the coagulation agents is AS : PAC = 10 : 1.

Table2- Results in the treating with AS

Indicators	AS, ppm				
	0	100	300	400	500
TSS, mg/l	2690	412	40	26	22
COD, mg/l	484	363	240	180	191

Table3 – Results in the treating with PAC

Indicators	AS, ppm				
	0	30	40	50	100
TSS, mg/l	2690	288	39	18	2022
COD, mg/l	484	282	235	175	182

Conclussion

• Taking into account the initial load of the effluent, we have obtained the decrease of the total suspended solids content up to 20 mg/l.

• Over optimal quantities the coagulation agents, that acts as protective agent, stopping formation and setling large flocs.

• Polyaluminium chloride is more efficaceous than AS, the ratio of optimal quantity is AS : PAC = 10 : 1 for same decrease of effluent load.

• Together with the decrease of TSS, there is a decrease of COD of about 60% percents, both by chemical reaction of coagulation agents with organical compounds, and adsorbtion of some organical compound at the largest flocs.

• Polyaluminium chloride can be used in the effluent treatmentfrom the writting printing paper manufacturing and also in the pseudo-neutral sizing.

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