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SECTION I

COMPUTATIONAL METHODS IN CIVIL ENGINEERING

STRUCTURAL ENGINEERING

SEISMIC ENGINEERING

Ovidius University Annals Series: Civil Engineering, Issue15, October 2013

Study on a Seismic Isolated Building Using Two Different Isolation Systems

Gabriel Dănilă

***Abstract* – Seismic protection of buildings using the base isolation method has become a technique increasingly used especially in countries with strong seismicity, due to the efficiency which had been proved after major earthquakes.**

The paper presents a comparative study between two seismic isolation systems. The first system is composed of high damping rubber bearings, linear motion guides and nonlinear fluid viscous damper (HDRB+LMG+NFVD) and the second system is composed of friction pendulum bearings with one sliding surface (FPB). The study showed that the seismic isolation system composed of HDRB+LMG+NFVD had higher relative displacements, smaller absolute acceleration, higher base shear force and higher dissipated energy than the seismic isolation system composed of FPS.

***Keywords* – friction pendulum bearing, high damping rubber bearing, linear motion system, nonlinear fluid viscous damper.**

1. Introduction

The seismic isolated buildings subjected to the major earthquakes of the end of the XXth century exhibits a good behaviour without major damages. In Romania, the first seismic isolated building was Victor Slăvescu building from Bucharest, whose rehabilitation was completed in 2010. Thus, no data are available concerning the behaviour of seismic isolated buildings to the earthquakes from Vrancea source.

This study make a comparison between two different seismic isolation systems in terms of relative displacements, absolute accelerations, base shear forces and dissipated energies, considering the seismic action from Vrancea source. The comparison was made on a reinforced concrete building having the height regime of ground floor and eight storeys.

2. Description of the Building and of the Seismic Isolation Systems

The analysed structure is a dual structure, with reinforced concrete shear walls and frames, for which was considered the Bucharest location. The height regime is of ground floor and eight storeys, with storey height of 2.8m. The building has three spans - the central one of 3m and marginal ones of 7m – and four bays of 8m. The resistance to the lateral forces is provided by the reinforced concrete shear walls, placed on both direction of the building and reinforced concrete frames. The wall thickness on *x* direction is 35cm and on *y* direction is 30cm, being constant on the entire height of the building. The columns are made of square section of 70cmx70cm, without reduction of section with height. The longitudinal beams are made of T cross-section with the web thickness of 35cm, the height of 70cm, the flange thickness of 16cm and flange width of 100cm. The transversal

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beams are also made of T cross-section with the web thickness of 30cm, the height of 60cm, the flange thickness of 16cm and flange width of 100cm. The thickness of the reinforced concrete slabs was taken 16cm.

At the level of the isolation plane it was considered a reinforced concrete slab of 16cm thickness, having the main role of distributing the horizontal forces to the isolation system.
The reinforced concrete slab is supported on longitudinal and transversal beams of T cross-section with the web thickness of 100cm, the height of 60cm, the flange thickness of 16cm and the flange width of 170cm. This beams has greater cross-sections to avoid plastic hinges occurance and to ensure a good connection with the isolation devices.

The analysed building was seismically isolated using two different seismic isolation systems. The first system (HDRB+LMG+ NFVD) is composed of fifteen high damping rubber bearings (one bearing under each column), ten linear motion guides (two under each reinforced concrete wall) and twelve nonlinear fluid viscous damper (six on the longitudinal direction and six on the transversal direction of the building).

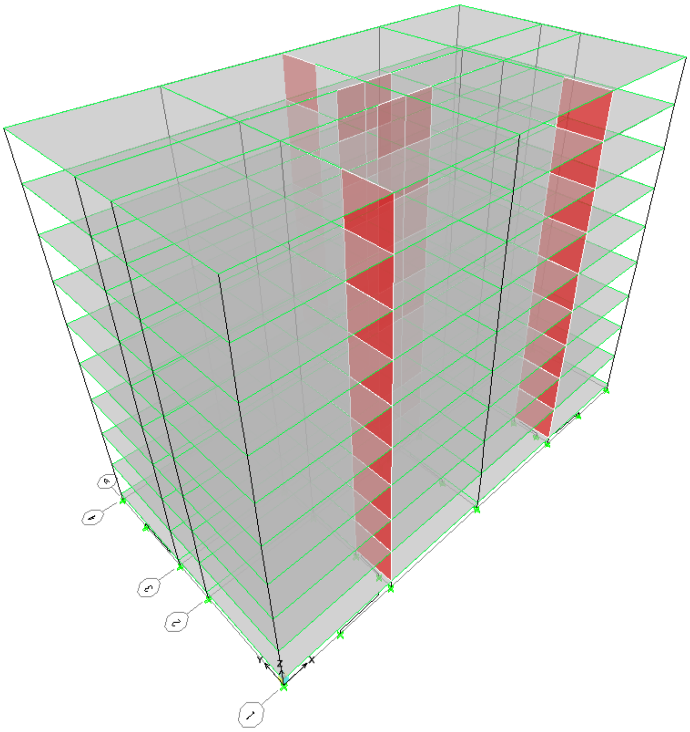
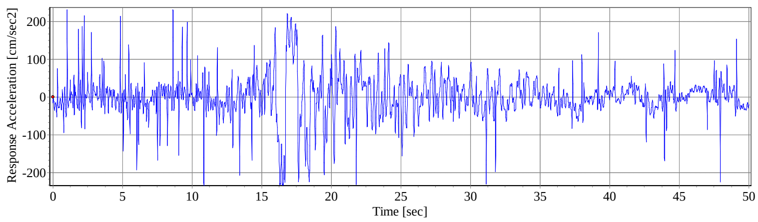


Fig. 1. General view of the analysed building

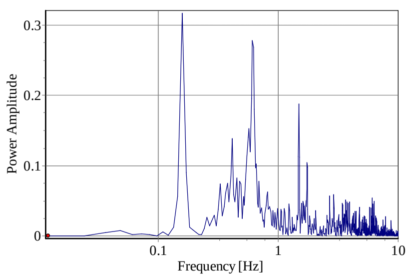
The second system (FPB) is composed of twenty five friction pendulum bearings; one under each column and two FPBs under each reinforced concrete wall. The friction pendulum bearings have a single sliding surface.

3. THE SEISMIC ACTION

The seismic action is described by six artificial accelerograms compatible with the design spectrum for Bucharest and one accelerogram, recorded on INCERC-Bucharest site, corresponding to the N-S component of the March 4, 1977 earthquake.
The recorded accelerogram was scaled, according to P100-1/2006 [10] seismic code, to the maximum ground acceleration of 0.24g, corresponding to the design ground acceleration for Bucharest, having the mean recurrence interval of 100 years. The artificial accelerograms were generated by means of SeismoArtif [12] computer program using two procedures.



a)



b)

Fig. 2. a) Generated accelerogram: March 4, 1977; b) Power spectral density

Three accelerograms were generated starting from the recorded accelerograms, on the INCERC-Bucharest site, of the March 4, 1977; August 30, 1986 and May 30, 1990 earthquakes, N-S component. The frequency content of the three recorded accelerograms was adjusted, using Fourier transformation method, to fit the target spectrum (design spectrum from P100-1/2006 seismic code, corresponding to Bucharest city). The maximum ground acceleration was considered 0.24g.
The other three accelerograms were generated using random processes by correction in frequency domain. It were applied the envelope shapes Saragoni&Hart, Compound and Exponential.

4. STRUCTURAL ANALYSIS AND DESIGN OF THE ISOLATED BUILDING

The preliminary design of the isolation systems was performed considering the analysed structure a system with one dynamic degree of freedom. The structure was isolate at a vibration period, $T_{is}=3.5s$, taking into account a damping ratio of the isolation systems, $\xi_{ef}=28\%$.
The displacement demand of the isolation systems, d_{dc}, to the design earthquake was determined using (1).

$$d_{dc} = \left(\frac{T_{is}}{2\pi}\right)^2 \cdot a_g^d \cdot \beta\left(T_{is}\right) \cdot \eta = \left(\frac{3.5s}{2\pi}\right)^2 \cdot 0.24 \cdot 9.81\frac{m}{s^2} \cdot 0.718 \cdot 0.55 = 0.289m \tag{1}$$

where: a_g^d is the ground acceleration corresponding to the design earthquake; $\beta(T_{is})$ is the normalised spectral ordinate, corresponding to the vibration period, T_{is} and η is the damping correction factor.

The damping constant, C_{nvd}, of the nonlinear fluid viscous dampers was determined by using (2).

$$C_{nvd} = \frac{2\pi \cdot G_{SC} \cdot \xi_{nvd}}{n_{nvd} \cdot \left(\frac{2\pi}{T_{is}}\right)^{\alpha-2} \cdot d_{dc} \cdot \lambda \cdot g} = \frac{2\pi \cdot 65668.16kN \cdot 12\%}{6 \cdot \left(\frac{2\pi}{3.5s}\right)^{0.4-2} \cdot 0.289m \cdot 3.62 \cdot 9.81 m/s^2} = 281.4kN\frac{s^{0.4}}{m^{0.4}} \tag{2}$$

where: G_{SC} is the total weight of the building in the special combination of loads; ξ_{nvd} is the damping ratio of the nonlinear fluid viscous dampers; n_{nvd} is the number of the nonlinear fluid viscous dampers on x and y direction, respectively; λ is the coefficient of the nonlinear fluid viscous dampers and g is the ground acceleration.

The effective horizontal stiffness, k_{ef}^{lmg}, of the linear motion guides was determined by using (3).

$$\begin{aligned} F_f^{lmg} &= \mu_f^{lmg} \cdot N_{lmg} = 0.007 \cdot 2006.26kN = 14.04kN \\ k_{ef}^{lmg} &= n_{lmg} \cdot \frac{F_f^{lmg}}{d_{dc}} = 10 \cdot \frac{14.04kN}{0.289m} = 486.3\frac{kN}{m} \end{aligned} \tag{3}$$

where: μ_f^{lmg} is the friction coefficient of the linear motion guides; N_{lmg} is the axial force on a linear motion guide; n_{lmg} is the number of linear motion guides and F_f^{lmg} is the friction force developed by a linear motion guide.

The effective horizontal stiffness, k_{ef}^{1hdrb}, of a high damping rubber bearing was determined by using (4).

$$k_{ef}^{hdrb} = \left(\frac{2\pi}{T_{is}}\right)^2 \cdot \frac{G_{sc}}{g} - k_{ef}^{lmg} = \left(\frac{2\pi}{3.5s}\right)^2 \cdot \frac{65668.16kN}{9.81m/s^2} - 486.3\frac{kN}{m} = 21086.6\frac{kN}{m}$$

$$k_{ef}^{1hdrb} = \frac{k_{ef}^{hdrb}}{n_{hdrb}} = \frac{21086.6\,kN/m}{15} = 1405.8\frac{kN}{m} \tag{4}$$

where: k_{ef}^{hdrb} is the effective horizontal stiffness of the all high damping rubber bearings and n_{hdrb} is the number of high damping rubber bearings.

The effective horizontal stiffness, k_{ef}^{1fpb}, of the friction pendulum bearings with one surface of sliding was determined using (5).

$$R_{fpb} = \frac{1}{\left(\frac{2\pi}{T_{is}}\right)^2 \cdot \frac{1}{g} - \frac{\mu_f^{fpb}}{d_{dc}}} = \frac{1}{\left(\frac{2\pi}{3.5s}\right)^2 \cdot \frac{1}{9.81 m/s^2} - \frac{0.042}{0.289m}} = 5.46m$$

$$k_{ef}^{fpb} = \frac{G_{SC}}{R_{fpb}} + \frac{\mu_f^{fpb} \cdot G_{SC}}{d_{dc}} = \frac{65668.16kN}{5.46m} + \frac{0.042 \cdot 65668.16kN}{0.289m} = 21570.6\frac{kN}{m} \tag{5}$$

where: R_{fpb} is the radius of curvature of the friction pendulum bearing with one sliding surface and μ_f^{fpb} is the friction coefficient of the friction pendulum bearing.

The linear static analysis was performed using ETABS v9.2.0 [3] computer program, considering the stiffness of the elements reduced with fifty percent due to the concrete cracking.
Modelling of the high damping rubber bearings was made using the link type element *Isolator 1*, which was put in parallel with a *Gap* element to take into account the different behaviour in tension and compression. For the nonlinear fluid viscous damper was used a *Damper* element. The linear motion guides and the friction pendulum bearings with one surface of sliding were modeled using *Isolator 2* link type element.
The horizontal seismic forces, f_i, applied to the each level of the analysed structure, were determined using (6).

$$f_i = m_i \cdot S_a\left(T_{ef}, \xi_{ef}\right) = m_i \cdot \frac{S_e\left(T_{ef}, \xi_{ef}\right)}{q} \tag{6}$$

where: m_i is the mass of each storey; $S_a(T_{ef},\xi_{ef})$ is the design spectral acceleration corresponding to the effective period of vibration, T_{ef}, in the fundamental mode of vibration of the analysed structure, and to the effective damping, ξ_{ef} ; $S_e(T_{ef},\xi_{ef})$ is the elastic spectral acceleration corresponding to the effective period of vibration, T_{ef}, in the fundamental mode of vibration of the analysed structure, and to the effective damping, ξ_{ef}; q is the behaviour factor taken as *1.5*.

The structural elements were designed like low-dissipative elements, adopting the ductility class L, according to SR EN 1998-1:2004 [6]. According to the recommendations of SR EN 1998-1:2004 and P100-1/2006, it is not necessary to meet the requirements of the capacity method and global or local ductility. The reinforcement steel used for the longitudinal bars was S355 and for the stirrups was S235. The concrete class used was C20/25.

5. NONLINEAR TIME-HISTORY ANALYSIS AND COMPARATIVE REZULTS

The nonlinear dynamic analysis of the isolated structure was performed using SAP2000 v15.1.0 [4] computer program, considering the structural elements and the isolation systems with nonlinear behaviour.

The nonlinear behavior of beams and columns was modeled with plastic hinges at the elements ends (concentrated plasticity model) of *M3* type and *PM2M3*, respectively.
The shear walls were modeled with *shell layered-nonlinear* elements, with nonlinear behaviour in both bending with axial force and shear force. For the concrete from the boundary elements of the shear wall was used a model with constant confinement - Mander 1988 model [8] - and for the reinforcement was used the model automatically generated by the program with yielding plateau and post-elastic hardening. The strengths of the materials were considered with mean values.
The devices which form the isolation system HDRB+LMG+NVD were modeled in the following manner: the high damping rubber bearings were modeled using the link type element *Rubber Isolator*, which was put in parallel with a *Gap* element to take into account the different behaviour in tension and compression; the linear motion guides were modelled using *Friction Isolator* link type element and the nonlinear fluid viscous damper were modelled using *Damper* element. In the **Table 1** are given the parameters of the devices which compose the HDRB+LMG +NVD isolation system, used in the nonlinear time-history analysis.

Table 1. Parameters of HDRB+LMG+NVD isolation system used in nonlinear dynamic analysis.

Direction	HDRB			LMG					NVD		
	k_e	f_y	k_p/k_e	k_e	μ_f(slow)	μ_f(fast)	ξ	R	k_e	C	α
	[kN/m]	[kN]	[-]	[kN/m]	[-]	[-]	[m/s]	[m]	[kN/m]	[kNs/m]	[-]
U1	3010000	-	-	1121000	-	-	-	-	243600	281.37	0.4
U2	4444	140	0.233	14044	0.007	0.0075	100	0	-	-	-
U3	4444	140	0.233	14044	0.007	0.0075	100	0	-	-	-

where: k_e is the elastic stiffness; k_p is the post-elastic stiffness; f_y is the yielding strength; μ_f is the friction coefficient; ξ is the rate parameter; R is the radius of curvature; C is the damping coefficient and α is the velocity exponent.

The devices which form the isolation system FPS were modeled using *Friction Isolator* link type element. In the **Table 2** are given the parameters of the devices which compose the FPS isolation system, used in the nonlinear dynamic analysis.

Table 2. Parameters of FPS isolation system used in nonlinear dynamic analysis.

Direction	FPS				
	k_e	μ_f (slow)	μ_f (fast)	ξ	R
	[kN/m]	[-]	[-]	[m/s]	[m]
U1	3000000	-	-	-	-
U2	80000	0.042	0.042	100	5.46
U3	80000	0.042	0.042	100	5.46

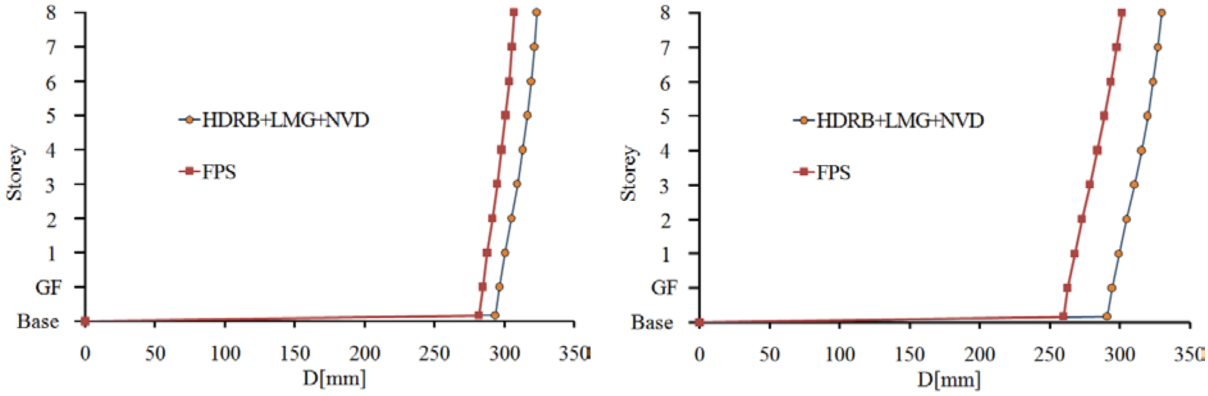
where: k_e is the elastic stiffness; μ_f is the friction coefficient; ξ is the rate parameter and R is the radius of curvature.

The seismic action was considered simultaneously in the three directions of the building, respecting the provisions of paragraph 4.5.3.6.2 (4) from P100-1/2006 seismic code.

The elastic damping was taken into account by using Rayleigh damping, considering the damping ratio of 3% for the vibration modes between $0.2T_1$ and $1.5T_1$ (T_1 is the period of vibration in the fundamental mode).

The response of the isolated structure is highlighted for each seismic action described in Chapter 3 and for each horizontal direction of the structure.

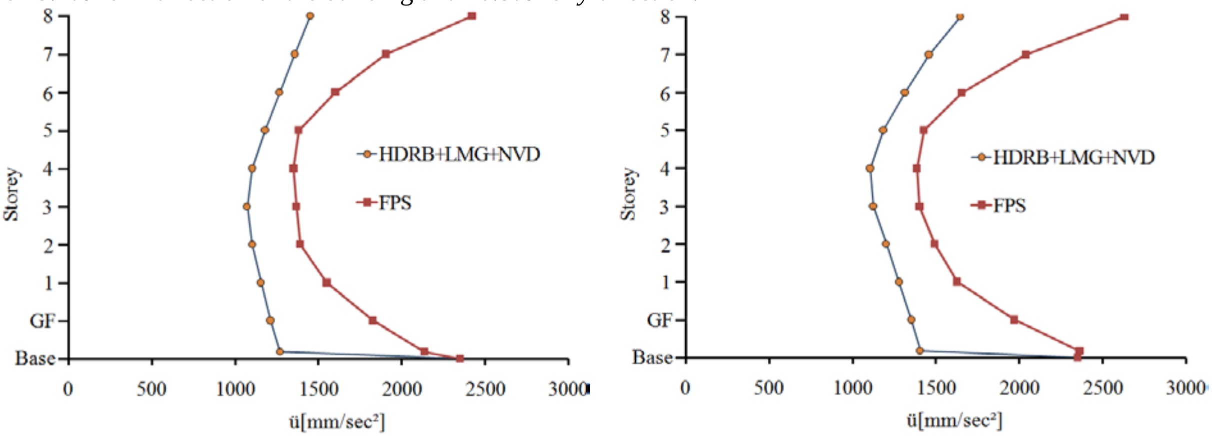
The mean relative displacements of the two isolation systems are given in the **Fig. 3.** In both horizontal directions of the structure, minimum displacements are obtained with FPS isolation system. The percentage difference between the two isolation systems, at the level of the isolation plane, is 4% for *x* direction of the building and 10.7% for *y* direction.



a) b)

Fig. 3. The mean relative displacements of the structure, isolated with HDRB+LMG+NVD and FPS system: a) *x* direction; b) *y* direction

In same design cases it is necessary to limit the accelerations in the structure to protect a certain valuable content. Thus, were made comparisons in terms of accelerations both at the level of the isolation plane and at each floor level of the structure. **Figure 4** presents the mean absolute accelerations of the two isolation systems**.** In both horizontal directions of the structure, minimum accelerations are obtained with HDRB+LMG+NVD isolation system. The percentage difference between the two isolation systems, at the level of the isolation plane, is 43.2% for *x* direction of the building and 40.5% for *y* direction.



a) b)

Fig. 4. The mean absolute accelerations of the structure, isolated with HDRB+LMG+NVD and FPS system: a) *x* direction; b) *y* direction

In the conventional design the energy induced by an earthquake is dissipated through post-elastic deformations of the structural elements. Through base isolation the dynamic properties of the structure are changed, so that the energy induced by an earthquake is greatly diminished and it is dissipated, for the most part, by the isolation system. To analyse the energy dissipated by the two isolation system, the hysteretic curves was integrated through the entire duration of the seismic actions described in the Chapter 3.

In **Fig. 5** is presented the mean energy induced by the seismic actions and dissipated through various mechanisms. There was used the following notations:

- E_i: the energy induced by the seismic actions;
- E_{is}: the energy dissipated by the isolation system;
- E_s: the energy dissipated by the structure through post-elastic deformations and elastic damping;
- E_k: the kinetic energy;
- E_p: the potential energy.

In order to have a fair indicator of the energy dissipated by the isolation systems and by the structure, this must be reported to the energy induced by seismic actions. Thus, for the *x* direction of the building, the structure isolated with HDRB+LMG+NVD system, dissipates 85.7% of the energy induced by the seismic actions through isolation system and 10% through post-elastic deformations and elastic damping and the structure isolated with FPS system, dissipates 84.6% of the energy induced by the seismic actions through isolation system and 11.5% through post-elastic deformations and elastic. For the *y* direction of the building, the structure isolated with HDRB+LMG+NVD system, dissipates 85.2% of the energy induced by the seismic actions through isolation system and 10.3% through post-elastic deformations and elastic damping and the structure isolated with FPS system, dissipates 83.7% of the energy induced by the seismic actions through isolation system and 12.5% through post-elastic deformations and elastic.

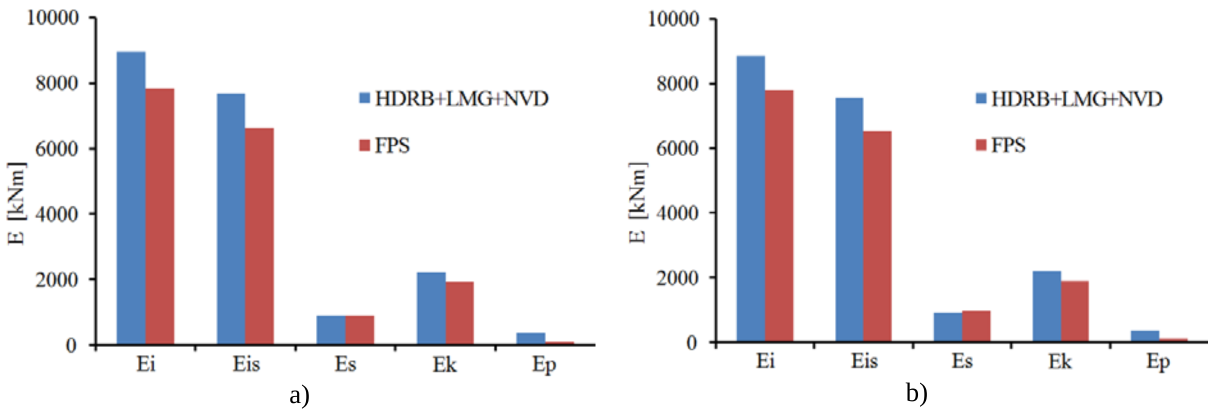


Fig. 5. The mean energies of the two isolation system: a) *x* direction; b) *y* direction

The base shear force is a key parameter in characterizing the seismic response of structures and is used to design them. In **Fig. 6** is presented the mean base shear force for the two isolation systems.

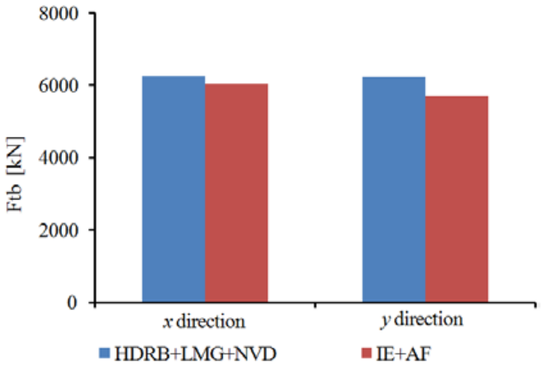


Fig. 6. The mean base shear force

For the both horizontal directions of the building, the maximum base shear force is obtained for the structure isolated with HDRB+LMG+NVD system. The percentage difference between the two isolation systems is of 3.6% on *x* direction and of 8.4% on *y* direction.

6. Conclusions

The performed study examines the seismic performance of two different isolation systems, considering a vibration period of the isolated structure of 3.5s and a damping ratio of 28%. It was analysed the response in displacements, accelerations, energy dissipated and base shear force of the structure and isolation systems to the seismic actions from Vrancea source. The minimum displacements and minimum base shear forces are obtained with FPS isolation system. With the HDRB+LMG+NVD isolation system is recorded the minimum accelerations, both at the level of the isolation plane and at each level of the building. The HDRB+LMG+NVD isolation system dissipates more energy than the FPS isolation system.

Both isolation systems have advantages and disadvantages. Depending on the design requirements it can be used a system or another; for example, if it is required the limitation of the storey accelerations the HDRB+LMG+ NVD isolation system is more suitable.

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Ductility of reinforced concrete structures

Daniel Purdea

Abstract – **The article emphasizes the overall ductility requirements for the structures in the seismic regions, the different types of ductility and especially the ductility of the reinforced concrete structures. It shows the correspondence between the different types of ductility and the code requirements for the seismic forces, as well as the main factors that affect the ductility of the elements.**

Keywords – **Reinforced concrete, ductility, earthquake design.**

1. INTRODUCTION

The ductility problem of reinforced concrete elements is known although a good period of time and successfully applied in practice remains current one. This is evident by the importance given to the subject matter of numerous studies and seismic design codes. Successful use of the concept of ductility of reinforced concrete elements and structures in general construction enabled competitive in terms of cost-benefit. Below are the main features of ductility of reinforced concrete.

2. GENERAL REQUIREMENTS, COMPLIANCE AND DEFINITION

General Requirements Compliance

According P100/2006-1 norm governing the calculation of the shares Sesma construction in Romania, we can emphasize the following basic requirements:

- requirement for safety of life, which requires a sufficient safety margin to the level of strain at which collapse occurs
- requirement to limit degradation, it means that the structure will be able to take seismic forces greater than those for which it is designed without suffering significant damage, the repair of which is lower than the cost of the structure

To be implemented the aforementioned requirements various methods to dissipate energy accumulated due to the seismic force have been analyzed so that they are technically feasible and economically. This can be done by making controlled dissipative zones from initial concept, by introducing the dissipative elements (diagonal dampers) or through advanced base isolation procedures.

Calculation of seismic force

The seismic force acting on a structural assembly is considered taken with the two basic components elastic and a plastic one.

For the computation of the seismic force the first is calculated the elastic component which is directly proportional to the acceleration recorded on the position and weight of the structure.

In the specific case of seismic force calculation for designing a resilient structure is affected by a factor subunit, ψ, which takes into account the ability of the structure to dissipate energy through inelastic deformations

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$F = F_{el}$ x ψ. Considering the above we can speak of two methods for assessing the inelastic response of a structure:

a) non-linear static analysis of the response of the structure

b) by analyzing the non-linear dynamic response of the structure

One can write the equation of dynamic equilibrium for the general case of a structure in case of non-linear static analysis: $[K_T]\{\Delta x\} = \{F\}$ (1).

In the case of a non-linear dynamic analysis of a structure equation (1) then becomes: $[M]\{\Delta\ddot{x}\} + [C]\{\Delta\dot{x}\} + [K_T]\{\Delta x\} = -[M]\{\Delta\ddot{u}\}$ (2).

According to the Romanian norm P100/2006-1 equivalent static base shear corresponding fundamental mode for each principal horizontal direction considered in the calculation of the building shall be determined as follows:

F = γI Sd (T1) m λ, where

Sd (T1) - neat design response spectrum corresponding fundamental period T1

T1 - own fundamental vibration period of the building in the plane containing the horizontal direction considered

I calculated the total mass of the building as the sum of the masses of level

γI - import-exposure factctorul construction of Section 4.4.5

λ - correction factor that takes into account the contribution of its fundamental mode effective modal mass associated with it, whose values are λ = 0.85 if T1 ≤ TC and the building has more than two levels and λ = 1.0 in other situations. It is thus seen that the value of the basic seismic force is less than the total force resulting from seismic action. The difference between the two values is taken up by deformation of structural elements.

Deformation capacity of the structural elements can be quantified by means of ductility. table titles above the tables.

Definition of ductility

Ductility is the ability of a structural element or system to develop plastic deformation, while maintaining resilience. Depending on the view taken, local or global, and depending on how the definition of ductility results different numerical values.

The ductility factor of the structure defined by

$$\mu = \frac{\Delta_{\max}}{\Delta_y} \qquad (3)$$

where Δ_{max} is the maximum impose displacement in a point measured during an earthquake measured at a point, and Δ_y is the maximum displacement measured at that paoint when the first plastic hinge has appeared.

Ductility specific strain

For the definition of the specific strain ductility of the material is used for the characteristic curve (Fig. 1).

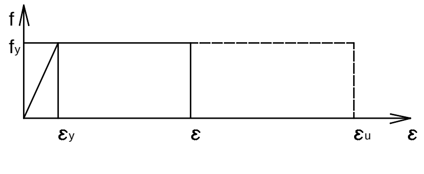

Fig. 1

Thus we can write the relationship $\mu_{\varepsilon} = \frac{\varepsilon}{\varepsilon_y}$ (4),

the number μ_{ε} shows how many times the charactetisc strain *(ε)* of the material is bigger than the one coresponding to the beginning of the yielding deformation of the material). Specific deformation must not exceed the computed limt strain ε_u. The ductility limit is defined as the ratio between $\varepsilon_u/\varepsilon_y$. Although the ductility of the material is the basic ductile behavior of structural elements and systems, it is not used in structural calculations. In the case of structural elements capacity design certain conditions are impossed, not exceed certain limit values.

The curvature ductility is defined by the angle of rotation of the two sections between which the distance is equal to unity ("curvature") (Fig. 2). Under the action of a bending moment increased to a certain value of curvature $ø_y$ (corresponding to $\varepsilon_s=\varepsilon_y$) plastic deformation starts.

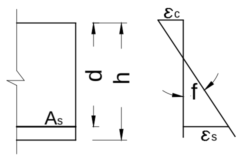

Fig. 2

Till the section failure, usually due to crushing of compressed concrete if reinforcement ratio is not too high, the angle of rotation increases very much. The curvature ductility is defined by the following relationship:

$$\mu_{\phi} = \frac{\phi}{\phi_y} \tag{5}$$

The value $\mu_ø$ shows how many times the effective curvature $ø$ overpasses the yelding curvature $ø_y$. Nevertheless the effective curvature can not exceed the limit curvature $ø_u$. From the ratio of the two values $ø_u/ø_y$ results the limit ductility. This depends largely on the section, the yelding limit of the reinforcement and the concrete copressive propertiess. In (Fig. 3) it shows the influence of transverse and longitudinal reinforcement on the ductility limit curve under pure bending efforts.

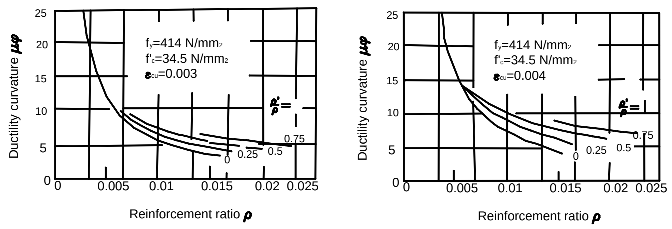

Fig. 3

This shows that a large percentage of reinforcement ρ leads to a lower curvature ductility limit and the large deformation capacity of steel is only partually used. For the case of beams with a current percentage of reinforcement $\rho \leq 1{,}5\%$ şi $0{,}5<\rho'/\rho< 1{,}0$ is possible to obtain curvature ductility up to $\mu_ø = 10$. The ductility limit of curvature is reduced by the presence of axial compression forces.

Thus, (Fig. 4) it can be seen that an axial force equal to $P = 0{,}15f'_cA_g$ for $\rho' = \rho$ may reduce the ductility of the $\mu_ø = 4$.

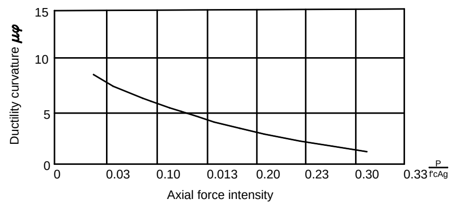

Fig. 4

The rotation ductility is defined using the angle of rotation of the plastic hinge

$$\mu_\theta = \frac{\theta}{\theta_y} \tag{6}$$

where $\theta > \theta_y$, μ_θ value indicates how many times the rotation angle of the deformed joint, is greater than the one from the beginning of the plastic deformation θ_y. Rotation θ can not exceed limit rotation θ_u taken in to account. From the ratio of these two values is defined the rotational ductility limit θ_u/θ_y. Rotation ductility is used first o all for structures to wich we accept local plastic hinges, in which case the size of θ is directly calculated by the rotation angle.

Displacement ductility is an important property of a system or a structural element of the structural system that defines the whole ductility. The displacement ductility μ_Δ is defined as the ratio of the displacements:

$$\mu_\Delta = \frac{\Delta}{\Delta_y} \tag{5}$$

where $\Delta > \Delta_y$,, μ_Δ value shows the number how many times the displacement of the system or of the structural element is greater than the one before of plastic deformation begins. Displacement can not exceed the limit displacement Δ_u. In this way one can define the displacement ductility limit Δ_u/Δ_y. For the multy-storey buildings construction Δ is generally the horizontal displacement at the top of structural sistem. The displacement Δ_y is regarded as a reference value and does not correspond to the formation of the first plastic hinge within thestructure. While for some elements, such as for example the columns in the base, these joints can occur very early, for structural elements of the upper floors is possible that cracks formation is not yet completed. It can be said that each newly formed plastic hinge contributes to global transition from elastic behavior to plastic behavior overall. It follows that for a lower value of Δy for example, will lead to higher computational ductility for the same rotation of the plastic hinge. When comparing the results one will have to pay particular attention to defining the starting value Δy coresponding to the formation of plastic hinges of generally idealized system. It can be said that the ductility is always defined as a ration beeing a mean relative measure.

For very flexible frame structures with high ductility, located in seismic areas one should limit the relative displacement level in order to limit the degradation of structural elements and not diminish the strength of the elements because of the P-Δ effect. Also an important factor is the choice of the structural system, frames, structural walls or mixed.

Checking of the cappable ductility

In the case of a structure the gerneral ductility expressed by a displacement ductility μ_Δ express its overall behavior characteristics. The value of this ductility is derived from inelastic deformations of all plastic zones that develop throughout the entire structure. Ductility of such zones expressed by the rotation ductility factor μ_θ or curvature ductility factor $\mu_ø$ has generally another order of magnitude than the ductility factor expressed by lateral displacement system μ_Δ. The relationship between local ductility and the ductility of the overall system is determined by the deformations accompanying the development of a plastic mechanism. The size of the equivalent seismic force based on the estimation of sistem ductility. Whithin the situations when it is required by the general design the development of potential plastic hinges the local ductility requeste are crucial.

3. DUCTILITY OF REINFORCED CONCRETE STRUCTURES

As shown above the ductility provizions are aimed to satisfey the requirements of structural movement during a strong earthquake, $\Delta_{max} \leq \Delta_{cap}$.

Ductility provizions of reinforced concrete elements, of structural elements in general are required if the elastic response capacity of elements or the entire system is less than the lateral forces applied to the structure.

From the above relations one can see that the force to which the structure is considered to have an elastic behavior of the structure is lower than the total force applied, F = ψ F_{el}. The elastic force reduction factor ψ ($\psi = \frac{1}{q}$, according to **[4]**) is directly related to the ductility of the structure. The lower ψ values the more sever are the ductility provisions for the structure. The general analysis of the phenomenon of structural ductility shows that the value of ψ is different from the real coefficient of the elastic force reduction ψ_{real}. The differences are due to safety factors used in the definition of the resistance material and the approximations used in computations. Thus it can be concluded that it can not be accurately known and the maximum value of the actual ψ_{real} of a structure during a major earthquake. It can be said that in general the increase of capable strains impose the following types of measures:

- designing the plastic hinges within the elements that have large deformation capacyties
- ensure a sufficiently large value Δ_u, within elements with potential plastic hinges, so to avoid the risk of premature rupture (in this case been an element ductility, one should use local ductility, the rotation ductility μ_θ and the curvature ductility $\mu_ø$)

  The following comments are general issues of the ductility measures described above:
- avoid shear failure Q ≤ Qcap, this requires the calculation of Q with the associated shear of the plastic mechanism and in the calculation of Qcap concrete strength is neglected.
- degradation of adhesion between the reinforcing bar and concrete in its anchor area, the area represented in the general structure nodes. To improve adhesion using rolled bars and the increase of the anchorage length, or anchorage special measures
- sectional rotation capacity in the plastic area is given by: $\Phi_u = \frac{\varepsilon_{bu}}{\bar{x}}$ (6)

or $\Phi_u = \frac{\varepsilon_{su}}{h_0 - \bar{x}}$ (7),

according to (Fig. 5). Maximum limit of *x* is determined by the relation $\xi \leq \xi_{lim}$ (8).

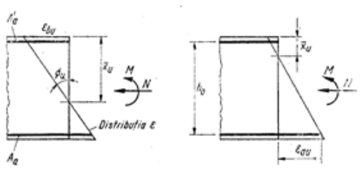

Fig 5

- by the general concept of the structure, aiming the developemnt of the plastic hinges at the ends of the beams (Fig. 6)

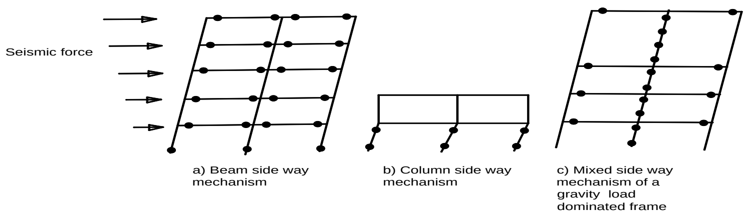

Fig. 6

4. CONCLUSIONS

Design of structures located in areas with high seimicity requires knowledge and mastery of the concept of structural ductility from the structural design engineer. This is needed from both technicall and scientificall point of view, as well as economically. Ductility defined as the ratio of the value recorded at a given moment and the value at the time of reinforcement yeld start can be of many types. Some types of ductility refer to elements individually while others treat the structure as a whole. Practical applicability depends on the type of structure and its compliance review in particular. One major thing that can influence the ductility of reinforced concrete elements is the transverse and longitudinal reinforcement ratio, and the presence or absence of axial force in the element considered. By analyzing the elements ductility in particular and the one of the structure in general can be determined the energy dissipation capacity by plastic deformation.

Even when the elements individual ductility is high care should be taken to limit it in order to prevent major damage to a structure after an earthquake greater or smaller. In the U.S. it is considered acceptable that the amount o money needed to be spent for the rehabilitation and strengthening of a structure shoul be a maximum of 60% of the value of a new construction, above this percentage the intervention is not considered cost effective enymore.

5. ACKNOWLEDGMENTS

The author thanks Mr. prof.Phd.eng. Augustin Popaescu and Mr.eng. Mircea Mironescu for the help and support given for research of elements and structures of reinforced concrete and prestressed concrete.

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Calculation and structural layout of reinforced concrete structures with flat slabs located in seismic area

Daniel Purdea

Abstract – **Design of reinforced concrete flat slab structures located in areas of high seismicity is one of date, because of modern requirements layout structures. Such structure requires certain features compared to classical or dual structurese. The article presents the current state of research in our country and other countries regarding the layout of these types of structures, and a briefly cover of how to calculate the flat slabs.**

Keywords – **flat slabs, ductility, seismic design, structural conformation.**

1. INTRODUCTION

Flat slabs floors are an alternative to the clasical space frames structures or dual structures and are used internationally and nationally for a long time. This system emerged from structural economic reasons such as: fast execution, reducing the height level, easy installation of equipment and facilities, etc.

In our country the first code for calculation of flat slabs STAS 3434 **[4]** was published in 1952. However, this code only refers to the calculation of flat slab or slabs with a drop panel for gravity load. The seismic component seismic computation is covered neither for the slab nor for the structure layout. This paper aim is to make a review of the current codes for the calculation of flat slabs for structures located in seismic areas.

2. SPECIFIC STRUCTURAL DESIGN REQUIREMENTS FOR STRUCTURES WITH FLAT SLABS LOCATED IN SEISMIC AREAS AND COMPUTATION METHODS OF FLAT SLABS

The use of flat slabs for buildings located in areas of high seismicity is possible as long as they comply with certain requirements of structural conformation. These special rules were imposed by their susceptibility to seismic actions. Among them we can mention the following provisions:

-Layout the structure so that it can provide a level of security and stability in accordance with the codes requirements.

-The existence of structural walls for buildings with more than one level.

- Provision of adequate energy dissipating system for the structure (**[1]** and **[9])**.

The reduction factor of the elastic force of the structure should be q = 2.5, if the sistem flat slab-column should rezist itself to the seismic action.

For structures located in the areas where $a_g \geq 0.16g$ besides the presence of structural walls is also necessary to provide perimeter beams that together with the columns are contributing to energy dissipation by

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plastic mechanisms. This is advisable because often the sufficient layout of structural walls on both directions is not possible because of architectural or functional reasons.

The slab should have a sufficient thickness so as to ensure the transmission of efforts between the vertical elements.

Slabs will be checked for the serviceability limit state for fundamental loads (ie self weight, weight of the upper floor slab, other transient loads during construction, etc.) as well as for the effect of the seismic actions **[9]**.

As mentioned before several mechanisms of energy dissipation can be used in concrete structures, the links between structural elements, or through the ends of vertical structural elements (plastic hinges in the walls or pillars).

As with any concrete structure in the case of the flat slabs must be checked the mechanism of formation of plastic hinges to be one that does not jeopardize the stability and does not lead to formation mechanisms.

An illustrating example for such structures is the formation of story mechanisms so-called "soft story" **[1]**. This is due to differences of rigidity between the slab and columns. In the case of seismic actions columns may break due to high rigidity of the plate and that the plastic hinge can not be obtain with tehe same capacity, as in the case of conventional beam to column connection. This can be avoided by introducing structural walls in combination with columns so that the force is taken mainly by the walls.

Formation of plastic hinges in the slab-column node is not impossible, but this should be avoided due to the risk of developing progressive collapse phenomenon, or kept under sever control in the design stage. It is also recommended that at the lower part of the slab a minimum amount of reinforcement is disposed, “hanging” reinforcement which prevents the gradual clolaps (pane cake phenomena).

It is recommended that for the flat slab structures horizontal seismic forces are taken by structural walls. They should be designed as far as possible so that potential plastic deformation to form at the column ends, hat requires on a slab-column joints the fullfilment of the conditions condition:

$$\sum M_{Rd,s} \geq \gamma_{Rd} \sum M_{Rd,c} \tag{1}$$

$\Sigma M_{Rd,s}$ - sum of the capable design moments of the slab strip entering the node;

$\Sigma M_{Rd,c}$ - the sum of the design values of capable moments of columns, minimum values will be considered corresponding to possible variation of axial forces in the load combination that includes seismic action;

γ_{Rd} - unevenness factor (overstrength factor due to the consolidation of the steel, which will be considered 1.30 for ductility class high (H) and 1.20 for medium ductility class (M)). For this reason the capable moment o the column should be checked so that its value is less than the capable moment of the slab. Thus one calculates reinforcement plate at the top and bottom depending on the requirements of the load fundamental group. With the computed reinforcement we are able to determine the maximum bending moment of the slab section. Maximum bending moment of the slab so calculated shall be entered into relationship above for the checking of vertical reinforcement in columns. Note that it will use the reduced longitudinal modulus for concrete beams and columns that take into account the degradation of concrete in repeated seismic actions **[7]**.

Another specific problem of these types of structures is the punching of the slab under the action of gravitational loads. In the case of classical frame structures, this problem does not occur because the connection in done between the column and the beam, the slab beeing on the beam itself.

If a slab has direct discharge on a column it leads to a concentration of efforts. Thus in the plate, the area around the column radial cracks are formed, the line that closes this area beeing called the control perimeter (Fig. 1).

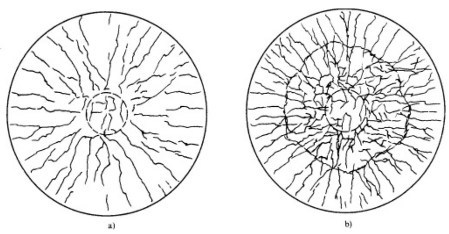


Fig. 1

According to Romanian code STAS 10107/0-1990 **[5]** control perimeter calculation is performed by the following formula:

for a rectangular column, $U_{cr} = 2(a + b + 2h_p)$ (2)

for a round column , $U_{cr} = \pi(d + h_p)$ (3)

where:

a and *b* are the dimensions of the two sides of a rectangular column

h_p is the slab thickness

d is the round column diameter

The norm Eurocode 2 **[6]** defines this area as in (Fig. 2) denoted by u_1.

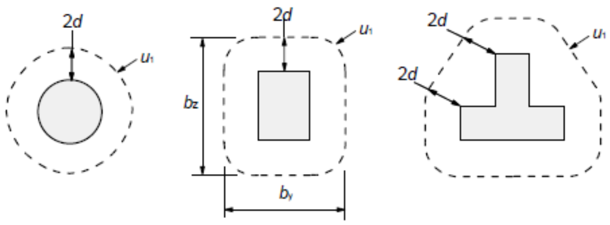
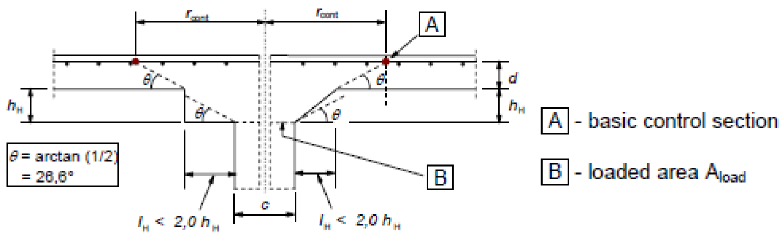
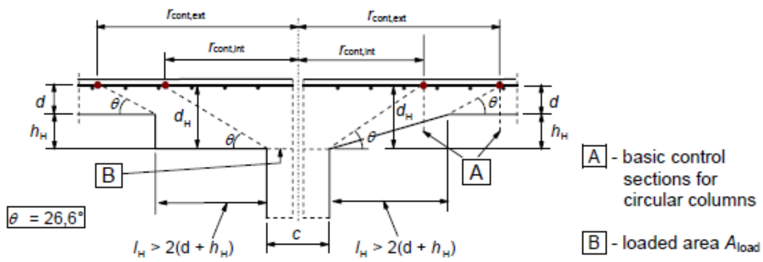


Fig.2

However European standard [6] of the control performance requires that the punching check should be made at a distance less than 2d where concentrated forces are acting on the slab inside the control perimeter. If the column is provided with a drop pannel (of concrete or metal profiles embedded in the slab) control area will be adjusted accordingly, see (Fig. 3).



a) rectangular column with $l_H < 2,0\ h_h$



b) circular column H $l_H > 2(d+h_H)$

Fig.3

Check of the capable shear force is done according to the Romanian standard (STAS 10107/0-1990) as follows:

for slabs without transverse reinforcement in the boundary of the columns $Q \le 0.75U_{cr}h_0R_t$ (4)

for slabs with transverse reinforcement in the boundary of the columns

$$Q \le 0.50U_{cr}h_0R_t + \sum A_{av}R_{at} + \sum A_{ai}R_{at}\sin\alpha \le 1.2U_{cr}h_0R_t \quad (5)$$

where:

ΣA_{av} is the sum of the areas of vertical reinforcement crossing theoretical shear surface (inclined faces of the truncated pyramid (Fig. 4);

ΣA_{ai} is the sum of the inclined reiforcement at an angle α to the plane of the slab, which crosses the theoretical shear surface.

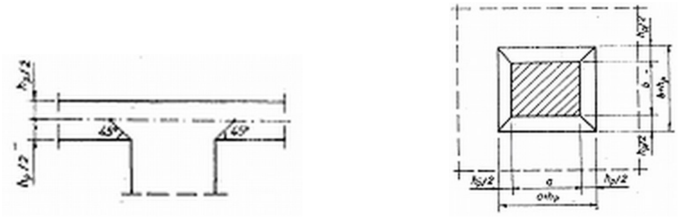


Fig.4

To calculate this type of slab three methods are available, of which the first two are most often used. These two allow for simplified calculations of moments and keep in mind the redistribution of moments between strips and field support.

Coefficient method known as the direct method uses tabulated coefficients depending on the bearing type and edges ratio of the plate. This method is very good and gives results very close to reality, but it can not be used for calculating slabs with prestressed reinforcement or posttensionată because coefficients were determined based on laboratory tests run on cast in situ concrete slabs **[10]**.

Equivalent frames method comprising in the division of the structure into the equivalent continuous frames, centered on the columns and which extend on both sides

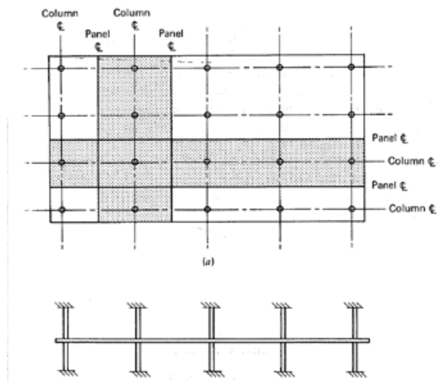


Fig 5

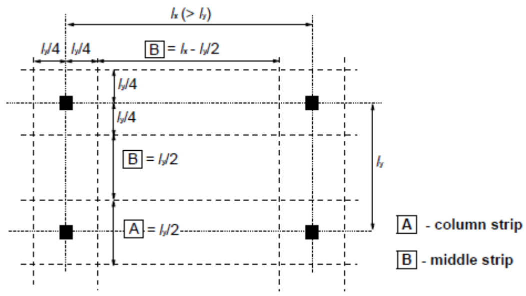


Fig. 6

The European Standard Eurocode 2 proposes the distribution of moments between areas and field support, but so that their sum remains equal to the maximum time (Fig. 6).

Table 1. Distribution of moments according to EC2

	Negative moments	Positive moments
Support strip	60-80%	50-70%
Middle strip	40-20%	50-30%
Notă: The summ of the moments both possitive and negative must be 100%		

In [10] are proposed the following distribution coefficients for the moments strips and field support for prestressed reinforcement plates as recommended by ACI comette 1976 and 1980.

Table 2. Distribution moments in [10]

No. of spans	Support strip	Middle strip
1	55-60%	45-40%
2 ≤	65-75%	35-25%

The current version of ACI 318-2008 code gives the following distribution for an interiour slab.

Table 3. Distribution moments as ACI 318-2008 for an inner slab

Support strip	Middle strip
65%	0,35%

To calculate the distribution of positive and negative points for an edge plate **[7]** proposed several values depending on the type of slab and its boundary support conditions, with the perimetral beam, without the perimetral beam, perimetral walls, etc.

To calculate the bending moments in pretesioned or posttensioned slabs the equivalent frame method can be used, considering all dead or super dead loads, live loads or transient loads multiplied by safety factor as required by code and secondary moments due to tensioning reinforcement effects value by a factor of 1.0.

A practical method for calculating the reinforcement neede and the predimensioning of the element is the equivalent load method. This method assumes that there are equal and opposite forces due to the tension, so that the element is in balance.

In case of slabs this method can successfully be used. Tension reinforcement can be arranged so that the load of its own weight to be canceled by the force of tension, so it can be considered that all dead weight and permanent loads lead to an effort equals 0 in the concrete element. Later it does not take only the live load through the reinforcement partially tensioned or by classic reinforcement or through both. The calculation method can be applied both to the method coefficients, as well as equivalent frame method.

It was suggested that failure can be calculated using tensioned slabs slab with yelding line theory. However, this theory can not be applied unless plastic behaviour is allowed that would cause large sections rotations with important efforts of the effect of tension reinforcement. Experiments have shown that this is true, but there is currently no sufficient data and therefore slabs with prestressed reinforcement should be computed in the elastic stage [8].

The finite element method is a method that can be used for any type of floor. The disadvantage of this method is that the majority of computer programs in fact do not allow automatic re-distribution of moments. This may result in a higher consumption of reinforcement than in the previous used methods, especially when the size and the distribution of the reinforcement are done automatically by a computer program. However, this method is particularly useful when the slabs have shapes which do not comply with the usual calculation of which is covered by the first two methods.

3. CONCLUSIONS

Flat slabs are a viable alternative to current requirements in architecture and functionality of buildings. Although most of the times the consumption of material (concrete and reinforcement) may be higher than for conventional structures rapidity of execution, reduced height floors and ease of installation of equipment and installations often recommends that for the current conditions in large urban areas and beyond.

For buildings subjected to seismic actions, flat slabs calculation is more complex than just gravity subjected slab. So, one must take into account the general requirements for energy dissipation through plastic hinges and dispative devices for structures located in seismic areas, in addition to the particularities of the floor slab.

Computing an ideal model would be one in which all seismic force should be taken by the vertical elements that would remain in the elastic range for a seismic force calculated by a reduction factor q = 1, this is very expensive, almost unprofitable. Accepted behavior factor is q = 2.5 to 3.0 for buildings with an average ductility and q = 3.5 ÷ 4.0 for buildings with high ductility. Another feature for seismic zones with higher seismicity ag ≥ 0.16 g represents the need for structural walls for buildings with more than one floor (some authors recommend for buildings over 3 levels and including) for taking seismic forces, reducing the displacement and 2nd order effects.

Calculation of flat slabs floors can be done by three main methods; the finite element method is the most conservative because it does not take into acount the redistribution of moments. For flat slab with post tenssioned reinforcement calculation can be done using the finite element method or frame equivalent.

Structures with flat slabs can be used in areas with medium and high seismicity under certain specific provisions, recommending a push-over modeling of the structure.

4. ACKNOWLEDGMENTS

The author thanks Mr. Prof. PhD. Eng. Augustin Popăescu, Mr. Eng. Mircea Mironescu for the support and help in the research and elements of structures of reinforced concrete and prestressed concrete.

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Prestressing Effects on Strands

Augustin Pintea, Traian Onet

***Abstract* - The manufacturing process of mono-block pre-stressed concrete beams, with pre-stressed reinforcement, include separate operations: pre-stressing strands. During the operation of strands pre-stressing, the following effects occur: stretching, gliding in jams and couplings, strains and ruptures in jams and couplings. The author has conducted research on these effects in order to develop a mathematical model calculation which takes into account all the elements involved in the calculation of strands elongations. The research was conducted in the laboratory of the LuppGmbH of Cristian- Sibiu, prefabricated and pre-stressed concrete Factory. The research results were applied in the manufacturing process of mono-block pre-stressed concrete beams reinforced with pre-tensioned reinforcement, of this plant.**

***Keyword* - coupling, elongation, pre-stressed concrete, pre-stressing, strand.**

1. INTRODUCTION

The manufacture of prefabricated beams of mono-block pre-stressed concrete, with pre-stressed reinforcement in the factory, involves strict control on technological phases, so at the end, the prefabricated element corresponds to the product standard and the project. A very important phase that should to be given special attention to is the reinforcement (strands) pre-stressing.

During the pre-stressing operation particularly dangerous situations may arise if all stages of technological process are not respected or followed, situations that can cause serious workplace accidents and also the degradation of the respective item, going up to compromise it.

In the research conducted by the author, several studies have been made, on the effects of pre-stressing the strands, elaborating the model for calculating elongation, gliding and deformations in jams and couplings, strands breaking models from various causes. The research results have provided valuable information on the quality of strands from various manufacturers, and the measures that were to be taken to eliminate the causes of the breakage.

2. DESCRIPTION OF THE EXPERIMENT

1. Strands elongation

In the stand execution technology, of the mono-block pre-stressed concrete beams, with pre-tensioned reinforcement, the reinforcement pre-tensioning is made individually, with monofilament pressing machine on the active end, where the normal open blockage props and at the passive end the reinforcement is anchored in normal open blockages. In accordance with the Code of Practice for the execution of prefabricated concrete elements, reinforced concrete and pre-stressed concrete: NE 013 - 2002, the designer must specify:

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a) – size of the control force, taking into account the actual size of tension losses;
b) – size of the control elongation of the pre-stressed reinforcement for the control unitary effort, considering the established elastic modulus of reinforcement and the related losses.

Reinforcement pre-stressing is typically performed in a single step in the effort established by controlling the pump gauge group. For measuring elongation, tensioning is made in two steps, the first step up to ≈ 40% of the final effort and the second step, up to the final effort.

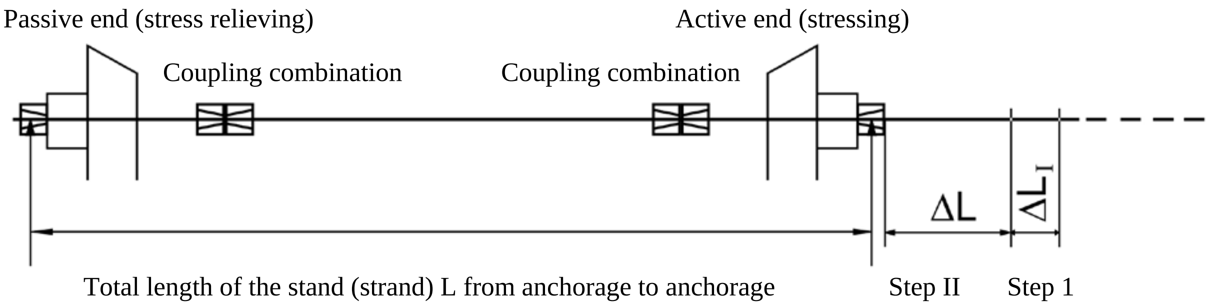

Fig. 1. The calculation of control elongation

For the elongation calculation at the 2nd step, please see (1):

$$\Delta L = \frac{\Delta F[kN]}{Ap[mm^2]*Ep[kN/mm^2]}*Lst[cm] = ...[cm] \quad (1)$$

where:
ΔL – the theoretical elongation calculated at the 2nd step;
ΔF – the difference between the force level at the 2nd step and the force level at the 1st step;
A_p - the cross-sectional area of the strand;
E_p - the elasticity modulus of the pre-stressed reinforcement (strand
L_{st} - the stand length between the blockages of the both ends

The elongation was calculated for the total length of the strand, including the portions up to the clamping anchorages in pressing machines. The elongation control measurement was performed at the second stage.

2. Gliding and local deformations in anchorages and couplings

On the prefabricated elements with pre-stressed, rectilinear, provisionally anchored on the pre-stressing abutments stand, reinforcement local gliding occur both in the blockage elements and in the combination couplings of the strands with the extensions. Consequently, the strands undergo a shortening strain deformation. In order to calculate the deformation of strands shortening, please see (2): [1]

$$\varepsilon p = \frac{\Sigma\lambda 1 + \Sigma\lambda 2}{Lp} \quad (2)$$

where :
$\Sigma \lambda_1$ - sum of gliding in anchorages at both ends;
$\Sigma \lambda_2$ - sum of gliding in anchorages in combination couplings of strands with the extensions;
L_p - pre-stressed reinforcement length between anchorages;

To calculate the voltage loss due in pre-stressed reinforcement due to gliding in blockages and couplings, please see (3): [2]

$$\Delta\sigma\lambda = \varepsilon p * Ep = \frac{\Sigma\lambda 1 + \Sigma\lambda 2}{Lp} * Ep \tag{3}$$

where:

E_p - elasticity module of the pre-stressed reinforcement;

This voltage loss due to gliding in blockages and couplings are compensated by the correction of tensioning force with their related value (**Table. 1**).

The speciality literature does not take into account the gliding in combination couplings of strands with the extensions, strands are considered itself as a single piece, equal in length with the stand, including anchorage length at both ends, regardless of the length of the element.

In practice, from economic reasons, things are different. Considering the fact that the length of the stand is ≈ 51, 00 m, measured, inter-ax blockages, and the frequent length of pre-stressed concrete beams is 30, 00 m < (exceptionally 36, 00 m), the difference in length of strands, of ≈ 21, 00 m, would constitute a significant loss from the economic point of view, if this portion would not be reused in the entire series of beams of the same length. For this reason the strands length in the reinforcement carcass, are made equal to the length of the beam, to which is added the required length of the combination with the extensions.

3. Strands breakings in blocks and couplings.

The breakage of one or more strands during the tensioning operations is a phenomenon that generates in addition to financial losses and delays in the program execution, the delivery and the installation.

One of the common causes that produce strand breakage is *non-centering the tensioning force in the strands axis* either due to incorrect positioning of the stressing beams from the two ends of the stressing stand, on the two directions, either due to exceeding the bearing capacity of the abutments stressing stand level, which leads to the progressive loss of their verticality and implicitly changing the strands default position in relation with the theoretical axis of the element and of the stressing beams.

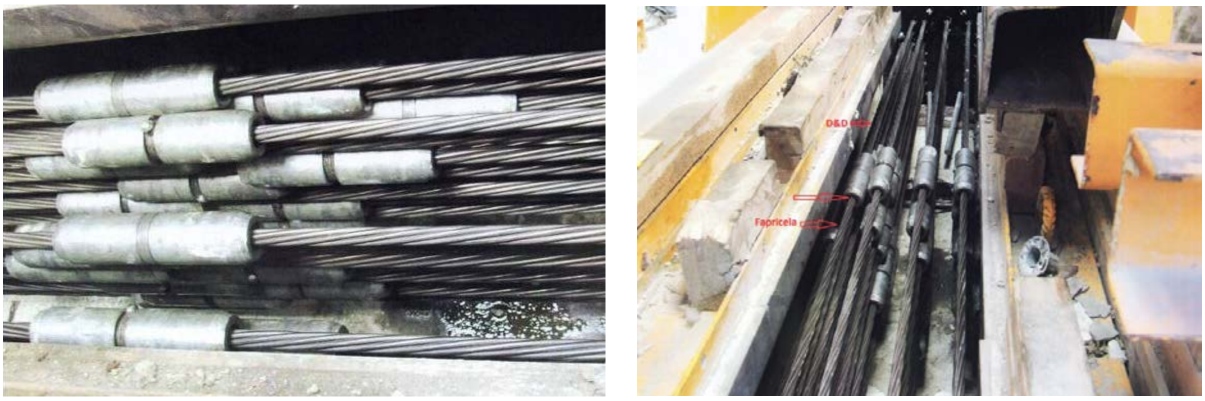

Fig. 2. Coupling combination of strands with different winding directions left / right

A *first effect* when combining different strands with winding directions (left / right), is the reduction of mutual forces present in the moments, the tension force being smaller than the one calculated.

A *second effect* is related to the unplaiting effect of the strand with winding towards right during the stressing operation, pressure clamps loosening of joint couplings and the possible release of the strands from clamps, with very dangerous effects on the personnel's security.

In order to prevent degradation of the strand surface, at clamping in the pressing machine's jaw with which they have carried out the tests, we used aluminum inserts.

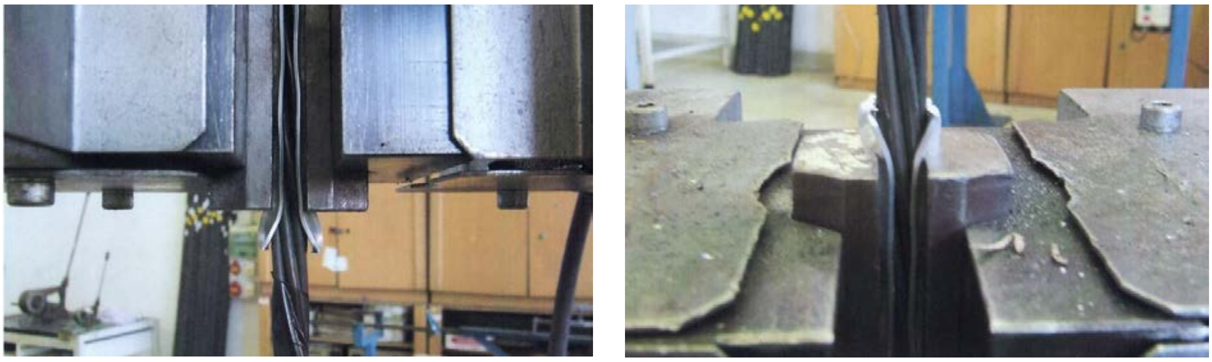

Fig. 3. Using aluminum inserts in the clamping jaws pressing machine

Fig. 4. The normal breaking mode of strands through constriction.

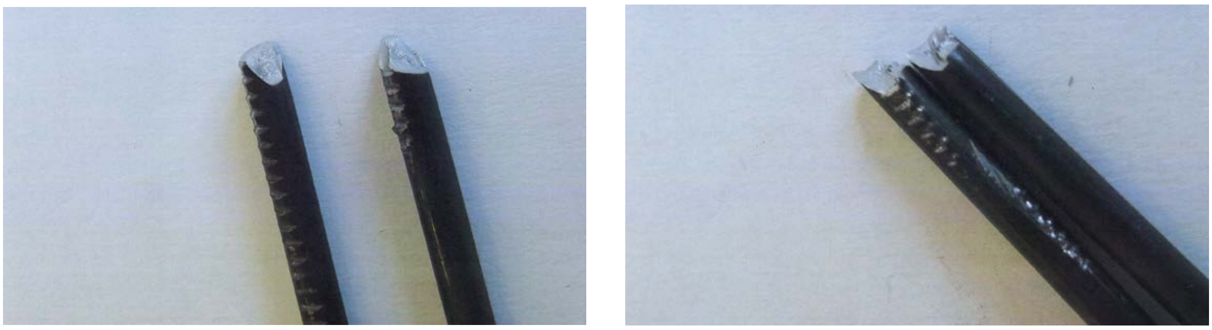

Fig. 5. The breaking mode of the wires damaged by the anchorage in the blockages clamps.

To see the effect of the repeated stressing on strands as extensions, it led to unplaiting the strand and to the separate test on component wires.

It was found that in the case of wires located outside the strand, which suffered surface degradation and cross-sectional reduction, due to repeated clamping, the average stressing resistance was lower than in undamaged wires located inside the strand. At the same time, it was found that wire breakage does not occur normally, through constriction, but in different ways by certain angles close to 45 °.

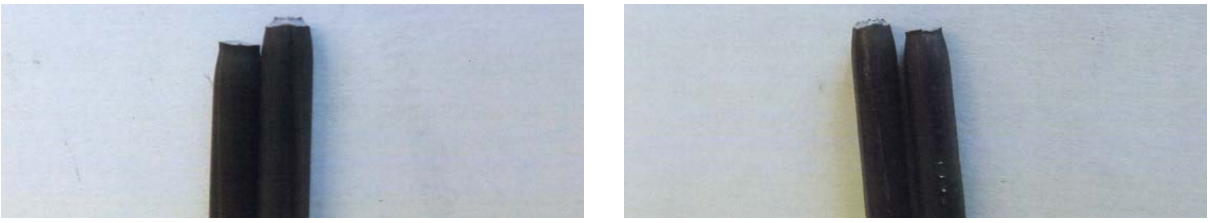

Fig. 6. Breaking mode of the undamaged wires situated inside the strand.

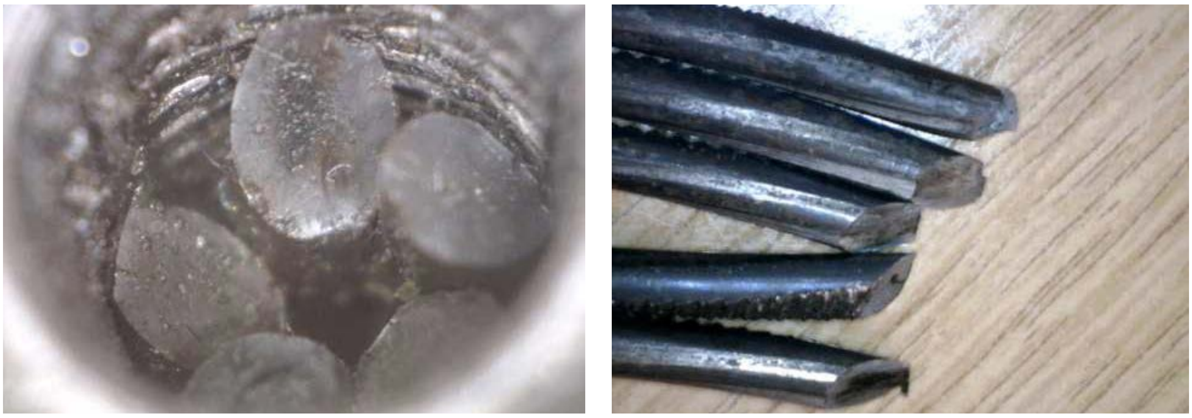

Fig. 7. Breakage at 45º angle of coupling strand, due to the use of small clamps.

Fig. 8. Wire surface damages caused by clamping.

Original outer wire diameter is 4.25 mm. The measured diameter is 3.8 mm. The clamps depth is of 4, 5 mm.

Fig. 9. Used clamps type K 32-28 (PAUL Catalogue).

Fig. 10. Wire breakage with contraction and normal constriction.

Fig. 11. Premature wire breakage, at 176,3 KN, caused by the deep teeth of the clamping**.**

3. RESULTS AND SIGNIFICANCES

Table. 1. Strand elongation calculation at step 2

Strand elongation calculation at step 2					
Project:	Orastie Highway – Sibiu Bridge 9 KM 7+510				
Element no.:	Gr.36,00 m				
Length of the stressing stand			L=	5092	cm
Type of the stressing steel			YS7	1660/1860	
Elasticity module of the pre-stressing steel – strands			E=	196.6	KN/mmp
Cross-sectional area of the strand			A=	100	mmp
Maximum admitted stressing force		144	KN	339	bar
The stressing force provided in the project	F=	136	KN	320	bar
Stressing force / pressure at stressing step 1	F1=	67	KN	159	bar
Stressing force / pressure at stressing step 2	F2=	72	KN	172	bar
The blockage number of the combination and anchorage elements				6	buc
Blockage gliding per combination/anchorage element				1.5	mm
gliding ∑ of the combination / anchorage elements			s=	9	mm
Loss of stressing force afferent to gliding in combinations and anchorages				3.5	KN
Pre-stressing force corrected with the force afferent to Gliding in combinations and anchorages	FP	139.5	KN	328	bar
Elongation calculated at the 2nd step of stressing $\Delta L=\Delta F$ [KN] / (A[mm²] x E [KN/mm²]) x L[cm]			ΔL=	18.77	cm
Length of piston stroke of the stressing pressing machine	20	cm	of which usable	19	cm
Stressing pressure increase for the 2nd stressing step				8.1	bar
The new stressing force/pressure		139.5	KN	328	bar

Before elongation calculation is being performed the correct pre-stressing force with the values linked to gliding in jams and couplings is effectively measured.

For calculation efficiency the author has designed and implemented a mathematical model that takes into account all criteria and all the factors involved in calculating the elongation of strands.

At a stand length of 50.92 m stand (measured inter-ax blockages at both ends), total elongation, calculated with equation (1) is 36.15 cm (corresponding to a stressing force of 140 kN / strand) , which will require two strokes of the piston , so two stressing steps.

4. CONCLUSIONS

The research results have provided valuable information on the quality of strands from various manufacturers, and the measures that were to be taken to eliminate the causes of breakage.

The breaking form study showed the following conclusions:

- breakings at an angle of 45 °, show that during the stressing process, there is a bending force caused by the couplings clamps that are touching. These breakings, similar, are found when testing strands at uni-axial deflected traction at an angle of 20 º.
- also it has been found that the use of inadequate stressing clamps, too small in relation to the strand diameter, produced a higher pressure on the strand and thus a greater damage on the strand surface caused by stressing clamp’s teeth.

These two combined effects, the deep damages caused by clamp’s teeth and the small bending force caused by the touching of combination couplings are sufficient to cause premature, abnormal, rupture of the strand, abnormal, at an angle of 45 º.

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Contributions on Realization Techniques for Vaulted Floor Decks

Bucur Dan Pericleanu, Mihaela Drăgoi

***Abstract* – The paper aims to develop the technical concept of brick vaults floor decks realization in the context of technical execution and of the materials that have characterized each historical stage in south-eastern Romania. Study methodology adopted in this research included stages like bibliographical documentation, information gathering and experimental research in situ by integrated studies of buildings that have resulted in inspections reports, surveys, nondestructive test reports and photographic surveys. One of the desired results of this study is the development of a technology for weaving bricks for vaulted floors with large thicknesses characteristic to patrimony buildings and to propose a model of execution of scaffoldings necessary for execution and restoration works of existing vaulted floors.**

***Keywords* – brick, heritage buildings, traditional techniques, vault.**

1. Introduction

Vaulted floors are load bearing structures that provide vertical partitioning of buildings, providing support for architectural floor assemblies and also for acoustic protection. These types of floor were made until the nineteenth century from various materials such as brick, stones, steel and wood. They had the role of supporting the load-bearing structure elements, situation encountered especially at vaults reinforced by arches. The geometric and mechanical concept of their structural composition, the materials and technologies used, constitute major milestones very well contoured for localization in time and space of vaulted floors.

The interaction of the structural element – floors – and the environment emerge into a development of physical and chemical transformation processes manifested by the appearance and development of visible or deductible phenomena. This type of phenomena we are referring refers to: deformation, cracking, breaking, vibrations or humidity. Behaviour properties can refer either to the material or the technology and can have a negative impact to both structure and function of the building. Behavioural properties are set in a historical process and their quantification is achieved as knowledge of technical progress and execution technologies increases.

The concept of authenticity is a concept of the architectural heritage of increasingly emphasized in studies and meetings of international committees that define this concept highlighting several directions, namely, design authenticity, genuineness and authenticity of materials, and traditional execution techniques, especially traditional techniques. We believe such a study that seeks to discover and explore traditional techniques regarding the execution of vaulted floors in our country is an issue of great importance.

In this paper, the research team has set as main objective the development of realization techniques of brick vaulted floors. In the first phase of the research we investigated some brick vaulted floors belonging to historical buildings in the south-eastern Romania. The authors conducted a detailed research highlighting the unique

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features that characterize the heritage buildings in the context of the requirements stipulated by Law no. 422/2001. Research carried out has led to remarkable and valuable results in terms of historical evolution of constructive solutions characteristic for vaulted floors, an evolution that this paper shows in terms of geometry of floors, constituent materials, technologies used [3]; the research is including the XVIII, XIX and XX century. The results of this investigations provides well-defined marks for locating major structural systems for floors in time and space in conjunction with the results of their behavior over time and the factors that contributed to the emergence of some forms of degradation.

Research activity, by the way it was structured, enabled the possibility of knowledge of floors structural composition made through time for a wide range of buildings, knowledge of constituent materials used for this floors, knowledge of traditional technologies and structural compliance solutions. Parallel to this, the research activity conducted was directed to identify the factors that lead to degradation of floors structural components, the analysis of calculation models applicable to current constructive solutions, identification of these slabs discharge conditions, methods and techniques to improve the mechanical behavior of floors in heritage buildings.

2. STUDY OF TECHNIQUES USED FOR VAULTED FLOORS

Currently vaulted floors are made of brick masonry and have a limited use (in traditional buildings, underground passages or basements with specific architectural plastic, wineries, etc.) because of the major disadvantages:

- large consumption of materials (bricks or blocks) and workforce which leads to a higher cost;
- big thick floor;
- big weight;
- execution of vaults require framing and supports of wood or metal;
- vaults require massive walls capable of taking over thrusts efforts, which increases material consumption and therefore the costs;
- a vaulted form ceiling is a disadvantage in terms of functional use (there are unused volumes in operation, additional costs for heating these volumes etc).

Floors of masonry vaults in the study area are made of brick, natural stone, mixed masonry. After their shape, the vaults can be simple curvature vaults (cylindrical vaults) or double curvature vaults (spatial vaults). Brick vaults can have a constant thickness of half a brick (1/2C) or one brick (1C) (C corresponding to the length of a brick) or an uneven thickness, thinner in the center and thicker towards the birth area [2], where the efforts are bigger (Fig. 1). The execution of brick masonry vaults is based in principle on the bricks weaving technique (Fig. 2).

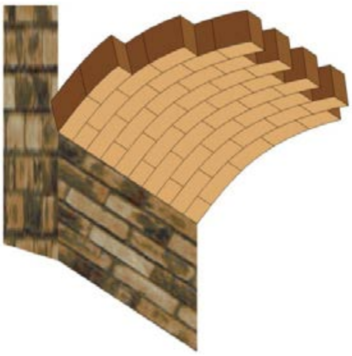

Fig. 1– The beginning of weaving for a cylindrical vault (author)

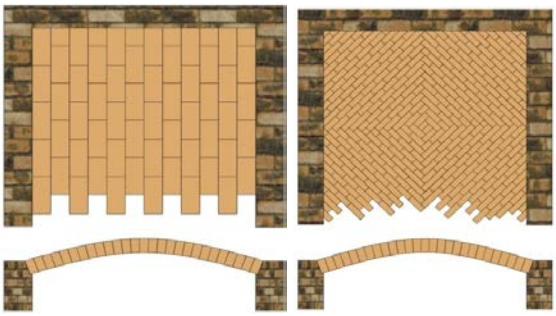

Fig. 2 – Ways of weaving of bricks for cylindrical vaults (author)

Thickness of the vaults with large openings is achieved by weaving brick masonry on several rows with different types of bricks [4] so as to make weaving also between the vertical rows of the vault (Fig. 3).

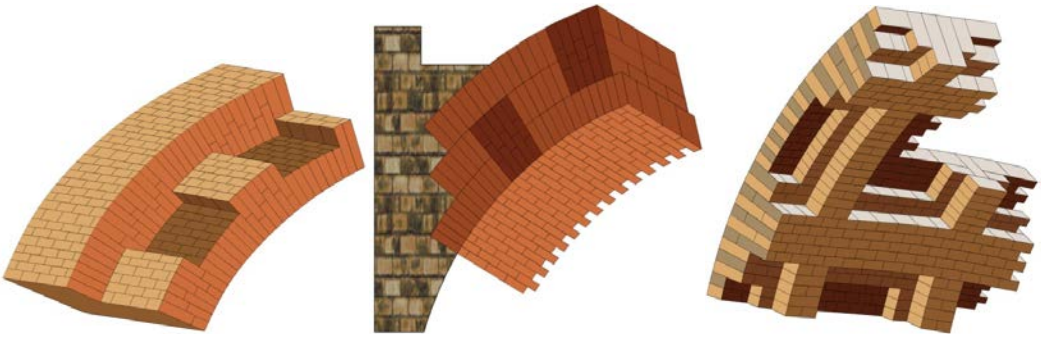

Fig. 3 Weaving systems for masonry vaults with large openings (author)

Vaults with large openings (5 ... 10m) have some stiffener masonry arches support placed at 3 ... 4 m. The vaults have as support the arches or the masonry walls. When the wall thickness is less than 2 bricks, the vaults propping on the wall is done by means of consoles (Fig. 4). For taking the thrusts transmitted by vaults to exterior walls, they are provided with buttresses and sometimes with apparent tie rods or buried (Fig. 5). For a better distribution of efforts it is recommended to make strong concrete small beams (belts) in the vault birth area.

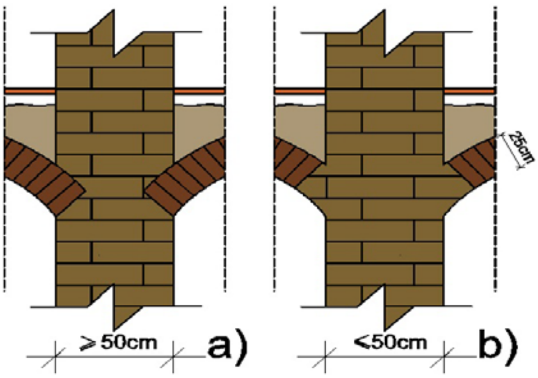

Fig. 4 – Vault propping on the walls (author) : a - with thickness greater than or at least equal to 50 cm, b - with thickness less than 50 cm.

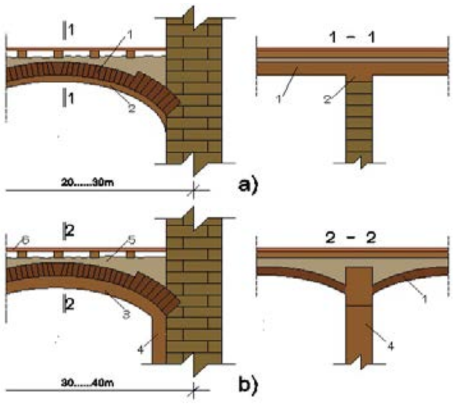

Fig. 5 – Floors masonry vaults for large openings (author) : a - with reinforcement arches; b - supported on masonry arches and pilasters;1 - vault;2 - the stiffener arch;3 - masonry arch;4 - pillar:5 - filling:6 - floor.

“Boltisoare” type floors or mixed floors (Fig. 6), are made of metallic profiles that represent the main load-bearing elements which transmit the efforts to the masonry walls. This type of floor has been approved in eclecticism and is made of metallic profiles section type I spaced approximately 1m between them on which lean against small cylindrical brick vaults with reduced curvature [1]. The small cylindrical brick vaults are considered secondary elements subjected to axial effort and metal beams are considered the main elements subjected to bending. Metal profiles in I support flattened vaults, the vault arrow is between 2-3 cm and allow creating flat surfaces plastered on the inside lower face, achieving a plan ceiling under that sequence of vaults [4].

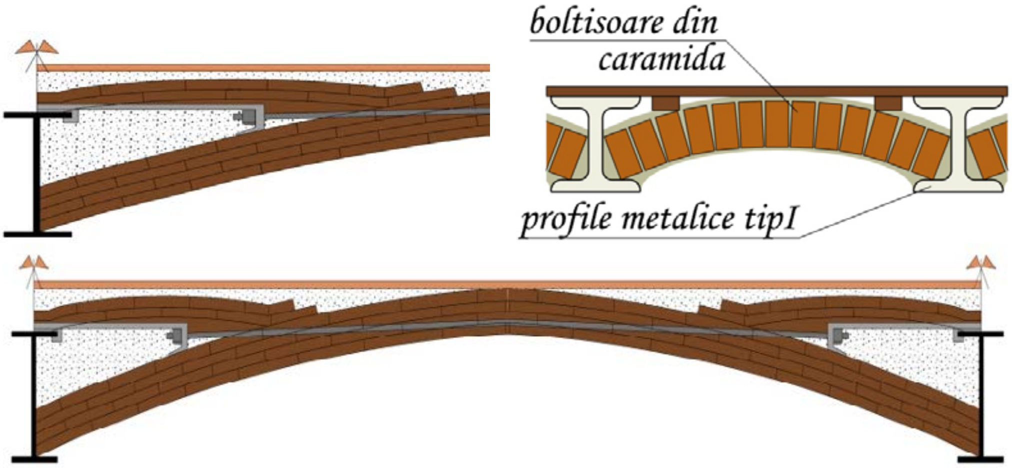

Fig. 6 – “Boltisoare” floors of masonry vaults for large openings – (author)

In execution technology, construction and restoration operation begins by making the working scaffolding under the slab. Then the top part of the vaulted floor is unfold in the degradated / damaged areas and the filling material is removed. Next, the damaged masonry of the small vault between two metal beams is being removed. After this, the area is cleaned with a wire brush and the masonry that remains, who will come in contact with new masonry, is washed with water. Then, the specific scaffolding for the floor is being done (“romanatul”) which will be needed for the execution of the new masonry vault and that will be supported by wooden props.

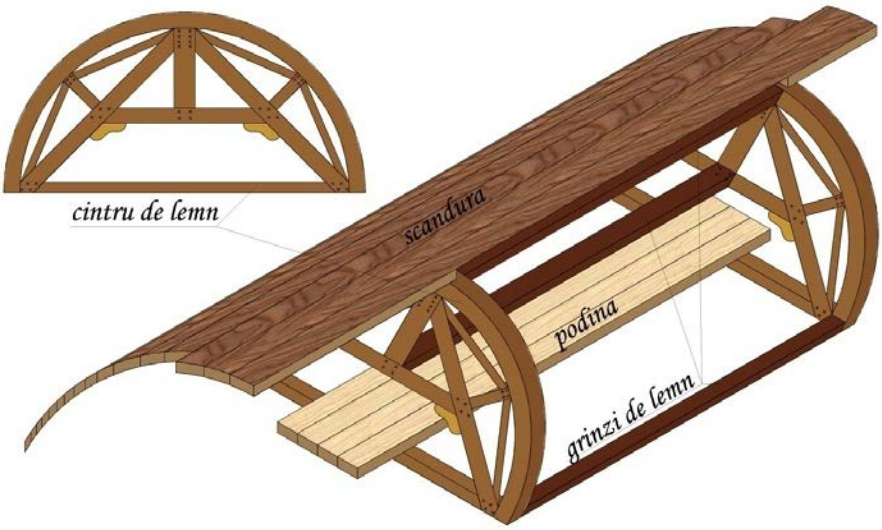

Fig.7 Shuttering cylindrical masonry vaults with wood scaffolding – (author)

Fig.8 Shuttering with wood scaffolding– (author)

The new masonry is done using normal bricks, if possible with the same size as the ones in the vault. The curve of the scaffolding (Fig. 7, 10, 11) has the same shape as the vault that is being kept in a restoration work. After the strengthening of the mortar in the vault masonry the scaffolding is unfold and the interior and exterior surface of the vault is plastered (Fig. 8). The scaffoldings were raised and held in place in situ by different systems using purlins, jacks or bags and boxes filled with sand as it is shown in Fig. 9.

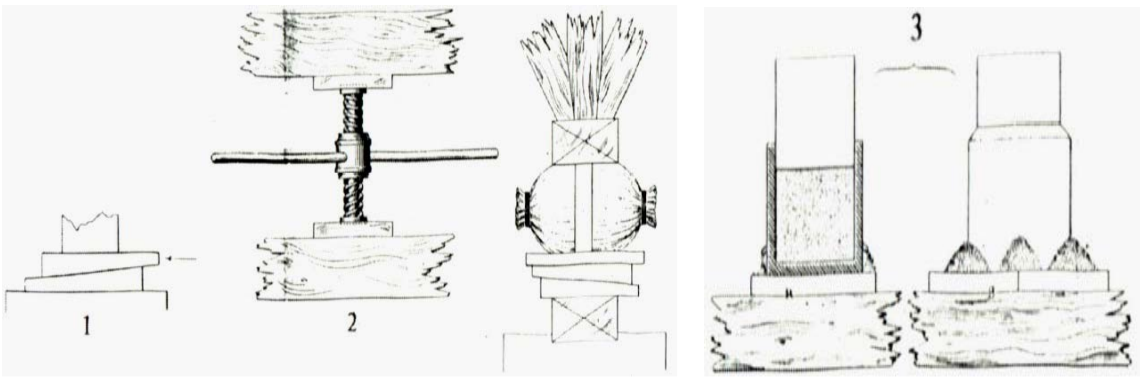

Fig. 9 Lifting and fastening systems for vaults scaffolding:

1) with purlins, 2) by jacks, 3) with sand bags and boxes

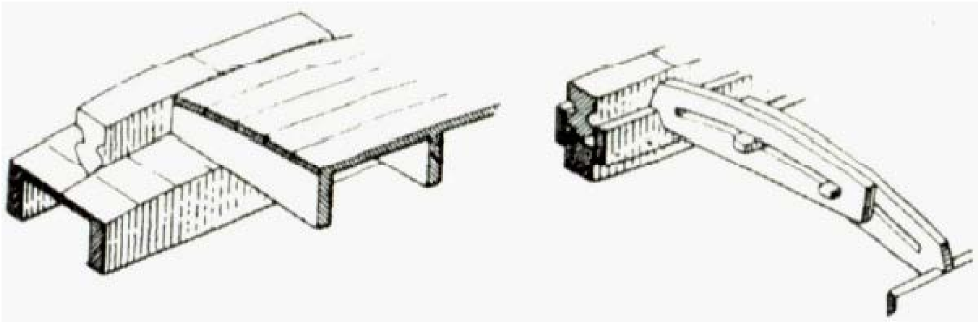

Fig. 10 Scaffolding systems for Gothic ribbed vaults

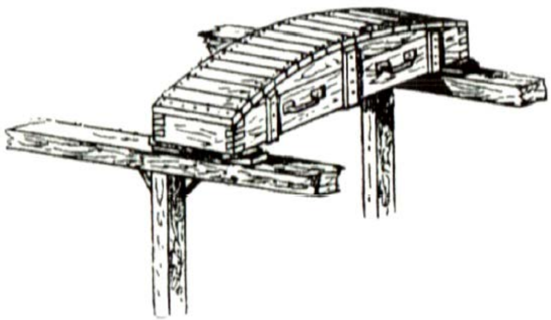

Fig. 11 Scaffolding systems for Moller vaults

3. CONCLUSIONS

Research activity, by the way it was structured, enabled the possibility of knowledge of floors structural composition made through time for a wide range of buildings, knowledge of constituent materials used for this floors, knowledge of traditional technologies and structural compliance solutions. The research conducted has revealed also some difficulties in quantifying realistic calculation parameters such as synthetic analysis of test methodology for constituent materials, highlighting the low homogeneity, non-uniformity compared to new structures, important variation of material properties and other more. Research results allow us to establish in the next phase some mathematical models for determining the calculation of floors closer to its real behaviour. Research results enabled the acquisition of an impressive amount of casuistic which were classified by the authors based on some analysis criteria.

4. ACKNOWLEDGMENTS

The authors would like to thank professors: João Gomes Ferreira, Fernando Branco, Ana Paula Pinto and Grãmescu Ana Maria.

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Integrity analysis for multistorey buildings

Krzysztof Cichocki, Mariusz Ruchwa

***Abstract* – This study concerns the integrity analysis performed for multistorey buildings subjected to exceptional loads (explosions, impacts). Main problems of numerical analysis, definition of discrete model of the structure and numerical description of the impact lads have been defined and discussed. Theoretical studies concerning the risk assessment criteria were also taken into consideration in order to find the objective criteria of risk acceptance. The numerical analysis of selected example of multistorey building subjected to exceptional load has been performed using the Finite Element Method. The obtained results have been used to write the final conclusions.**

***Keywords* – numerical simulation, exceptional loads, structural integrity, reinforced concrete, Finite Element Method.**

1. Introduction

In order to define the integrity of the structure, understood as its ability to maintain the partial functionality after the damage of certain structural elements, it is necessary to perform the detailed analysis of the damage propagation it the structure subjected to exceptional loads. Static replacement methods performed for the structure with deleted selected bearing structural elements allow only for qualitative evaluation of the structural response for such kind of loads. Wave- like character of this phenomenon and transient dynamic response of the considered structure may differ significantly from the results obtained for replacement static analysis. This is especially important for the loads of high intensity (explosions of large charges, etc) which produce the damages in elements relatively far from the center of an explosion. In authors' opinion, the simplified static analysis performed for structures with removed selected bearing elements insufficiently describes the overall dynamic behavior of the structure, and not allow for realistic evaluation of integrity and resistance assessment for exceptional loads taken into consideration.

The studies on laboratory tests and numerical analyses [1] show that the damage of material of the structure is influenced by two characteristic factors:

- direct action of the load in a form of local impact or a blast wave propagating in structure environment (air, water, soil, etc.);
- secondary propagation of stress wave in the material, its interaction with external loads, reflection on structural boundaries.

This complex mechanism of generation of material damages can be simulated numerically only with the use of explicit integration analyses.

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Integrity and robustness of structures (in particular multistorey buildings) has been studied in the framework of international research programme COST TU0601 „Robustness of Structures" where the criteria for evaluation of structural integrity and resistance for local damages have been defined. The entire study was a continuation of research carried out by JCSS (Joint Committee of Structural Safety), with special attention focused on the problems of so called „progressive collapse", i.e. failure of the entire structure produced by its local damage. First author of this paper was a member of the working group WG1, dedicated to the theoretical bases for acceptance criteria of structural robustness for exceptional loads.

The very important problem is to define the set of unequivocal criteria for evaluation of structural integrity, reduced if possible to only several parameters. The proposal for selection of such parameters was presented in the publication of Joint Committee of Structural Safety [2] and in final report of the action COST TU0601 [3]. They allow for qualitative evaluation of structural integrity, but depend on too many parameters assumed arbitrary, in particular concerning the evaluation of the effects in the case of structural failure. In authors' opinion only the numerical simulations of considered problems give the realistic description of dynamic response of the structure for exceptional loads.

Former studies carried out by authors show the decisive influence of assumed material model on results produced by the algorithm based on Finite Element Method [1,4]. In a case of impact loads, the assumed material model should take into account also the rate of deformation, initiation and development of damages, etc. Another important issue is the adequate description of loads generated by impact or explosion. For this last one, it is important to take into account its variation in time and space, interaction with deformable structure, reflection on structural elements, etc. Possible simplifications and adequate assumption were described in details in [1].

2. CRITERIA FOR EVALUATION OF STRUCTURAL INTEGRITY

The formulation of unequivocal criteria for evaluation of structural integrity is difficult in a case of various kind of exceptional loads, due to the numerous scenarios of possible load configuration. Practically, adequate simplifications are assumed in order to reduce the number of analyzed cases to those assumed as the most representative. Nevertheless this does not exhaust the possible combinations of load intensity, area of application, mode of damage propagation in structural elements, as well as the reciprocal interaction on damaged elements with the other part of the structure.

Numerical simulations allows for relatively simple analysis of dynamic response of the structure for many cases of the load. The main problem is to define the extent of acceptable structural damages and losses due to these damages. The overall rules for evaluation of acceptable losses are described in [3], but their application in any singular case needs the assumptions concerning the values of individual parameters defining the risk acceptance levels.

Problematic aspects of evaluation of the threat level and risk acceptance were discussed in elaboration JCSS [2] based on the LQI (Life Quality Index) parameter. In this way in this single parameter many possible direct and indirect results of structural failure can be included, also those developing in time. In this document the uniform terminology was presented, as well as bibliography and numerical examples of application of LQI parameter in practice.

In authors' opinion the tendency to establish the single criterion for the risk acceptance is understandable. For obvious reasons it is impossible to build the structure totally resistant for any kind of exceptional loads, due to economical and technical reasons. On the other hand it is necessary to ensure the adequate safety levels for important structures, enabling to maintain their integrity in a defined range of exceptional loads. This problem was discussed in details in [8], where the classical methods of risk assessment were used together with the methods of structural robustness evaluation.

3. EXAMPLE OF NUMERICAL ANALYSIS

The analyzed structure is four-storey garage realized as reinforced concrete space frame. Dimensions: 70 m x 56 m, total area 15 680 m2 (Fig. 1), material: concrete C45/C55, steel of class A. Columns with cross section 0,5 m x 0,5 m, mesh of columns 8,0 m x 10,0 m, thickness of the reinforced concrete floor slabs 0.4 m.

Entire structure has a double symmetry in regard to two perpendicular surfaces. In order to reduce the dimensions of discrete numerical model the symmetry of load has been assumed. Due to this assumption it was possible to analyze ¼ of the entire garage (Fig. 2). The detailed analysis has been performed for the part of the structure situated in the middle part of the garage, in its lowest level, close to the applied dynamic load (Fig. 3).

Discrete model was built and its analysis was performed using a commercial Finite Element Method system ABAQUS [5]. Discretization of concrete elements was carried out using solid (brick) 8 node elements, reinforcement of steel bars was modeled with truss element of 2 nodes. For all elements the reduced integration was assumed.

In the analysis the nonlinear elastoplastic material model for concrete with damage has been implemented (this model is described in [1, 6]), based on continuum damage plasticity proposed by Rabotnov [7]. For reinforcing steel the elastoplastic material model with isotropic hardening was applied. In both models the rate dependency was also taken into consideration.

Two variants of load were assumed. First one concerns the sudden replacement of central column (as the result of an impact), second one is the load produced by explosion in the vicinity of central column of the first storey.

Analyses were carried out in two steps: first one concerns gravity and utility (5 kN/m2) loads, second one is the assumed exceptional load. First step of calculations has been performed as a quasi-static analysis, second one used explicit dynamic analysis (integration with central difference method).

For both analyzed variants, the exceptional load was simulated in the second step of calculations. For the first variant of load, a sudden replacement of the column was modeled as the replacement of boundary joints at the bottom of the element. Second variant concerning a load produced by an explosion was realized using authors' own procedure in order to calculate the field of pressure variable in time and space. The charge of 30 TNT was assumed for this analysis [1]. All calculations were carried out using ABAQUS/Explicit [5].

For comparison reasons several variants of explosions were initially studied: various weights of the charge, various localization on the column. The dependence of results of a numerical analysis on these factors was studied, specially the range on resulting damages for bearing elements.

In the analysis of results, special attention was paid for the course of entire phenomenon, development of deformations, damages and the development of energies in a function of time. Selected results are presented in figures 4 ÷ 9.

Fig. 1. Model of entire garage

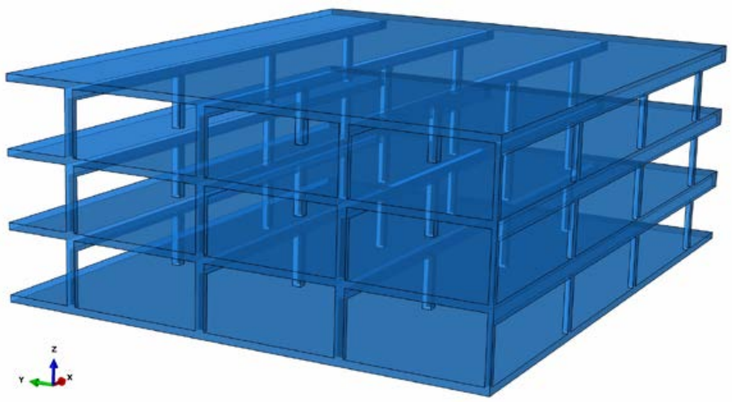

Fig. 2. Simplified model (1/4 of entire structure)

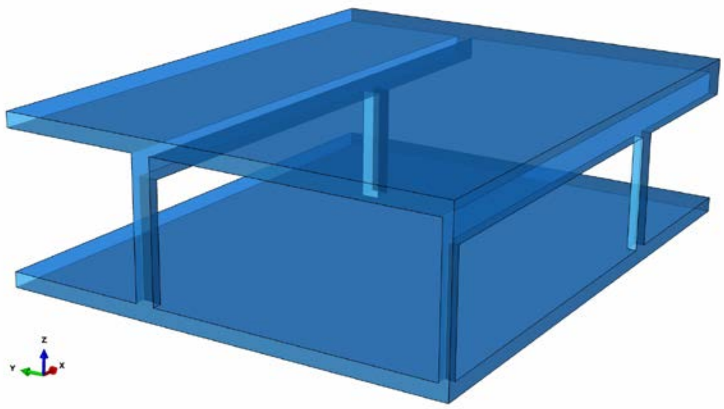

Fig. 3. Analysed part of the model

Figures 4 and 5 show the function of kinetic energy (KE), energy dissipated on permanent deformation (PD) , energy dissipated due to development of brittle cracking in concrete (DD). In both figures the results are presented starting from the beginning of the second step of the load (exceptional load). Diagrams show the faster development of damages for the structure subjected to an abrupt replacement of the column. In this variant most important changes occur in time $0 \div 0.12$ s, than the process stabilizes. For the variant of load concerning explosion, such changes are localized in time $0 \div 0.25$ s, two times longer than in the former variant.

Figures 6 and 8 present the distribution of material damages in a concrete for the variant of load concerning the replacement of the column, in two views (top and bottom), figures 7 and 9 concerns the same variables obtained for the blast load. Damages ($0 \leq \text{DAMAGET} \leq 1$) are marked in colors, from blue to red. For the case of column replacement damages are less distributed but more intensive, for the second variant of load damages are more distributed on the structure.

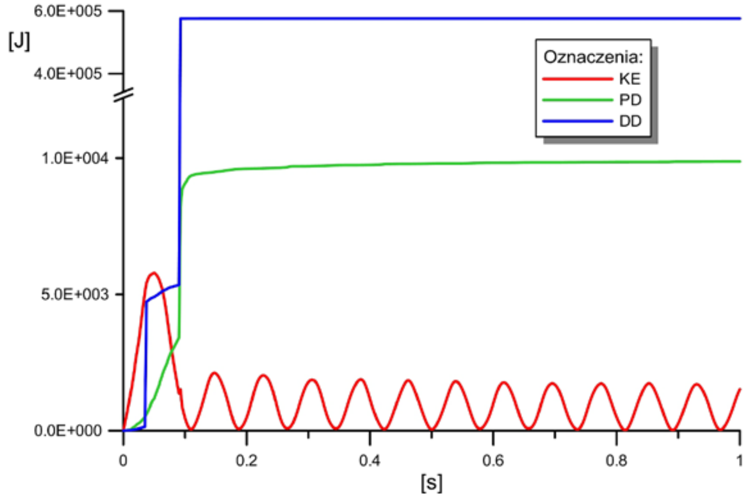

Fig. 4. Energies for the variant with column replacement

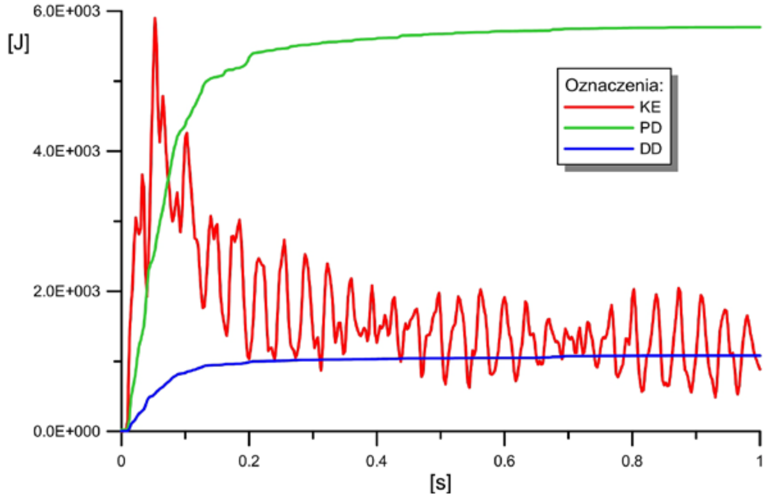

Fig. 5. Energies for the variant with blast load

KE – kinetic energy, PD – energy dissipated on permanent deformation, DD – energy dissipated on brittle cracking

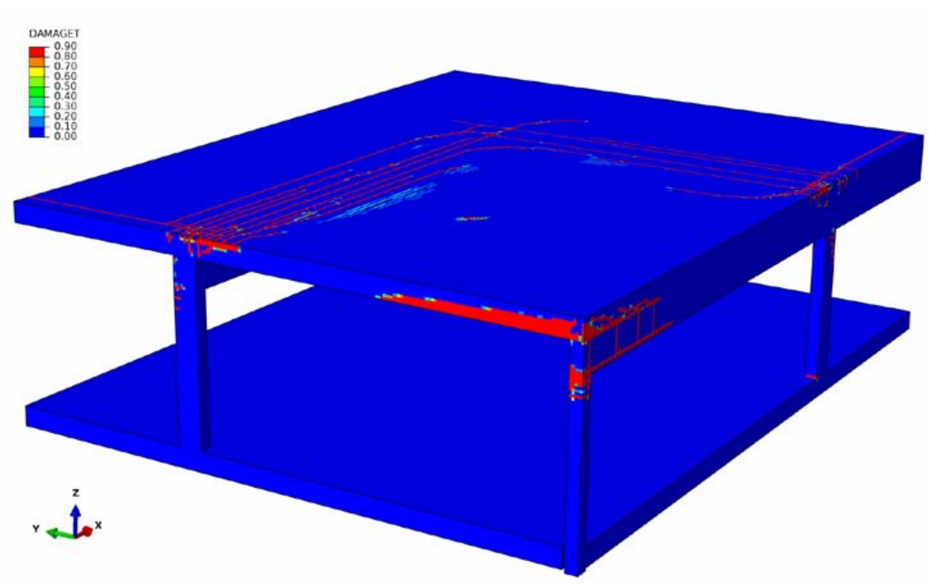

Fig. 6. Distribution of structural damages. Replaced column.

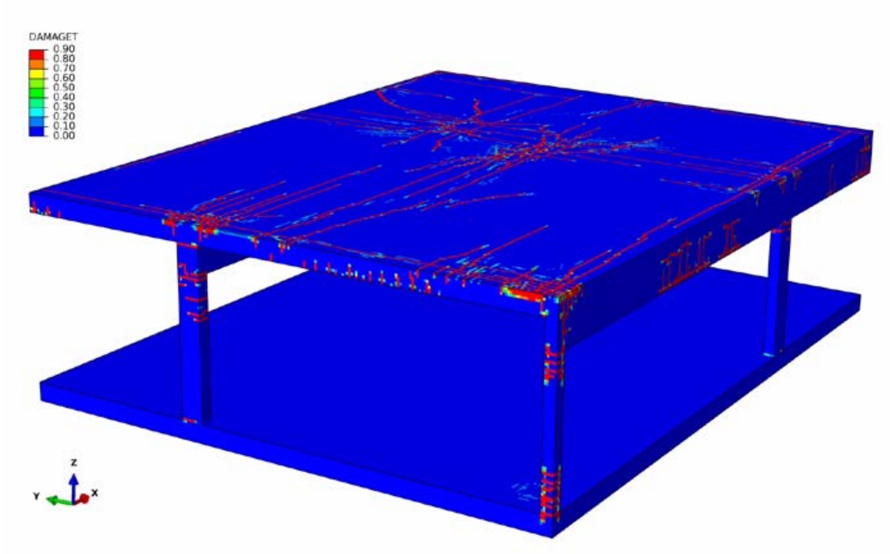

Fig. 7. Distribution of structural damages. Blast load.

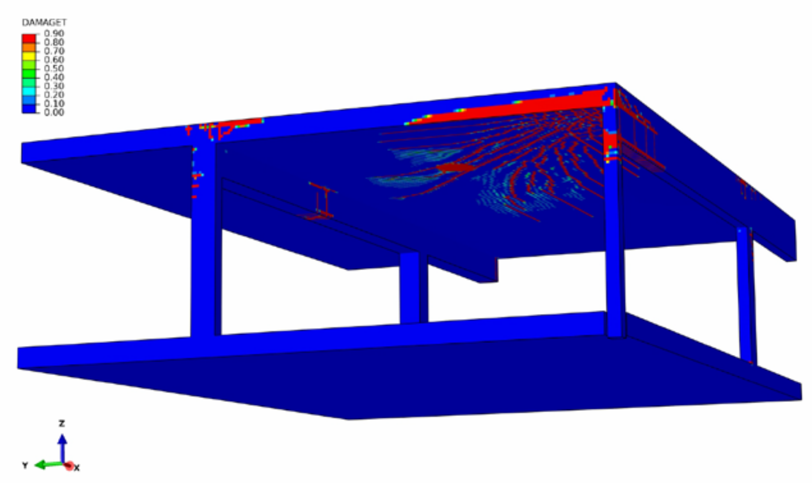

Fig. 8. Distribution of structural damages. Replaced column.

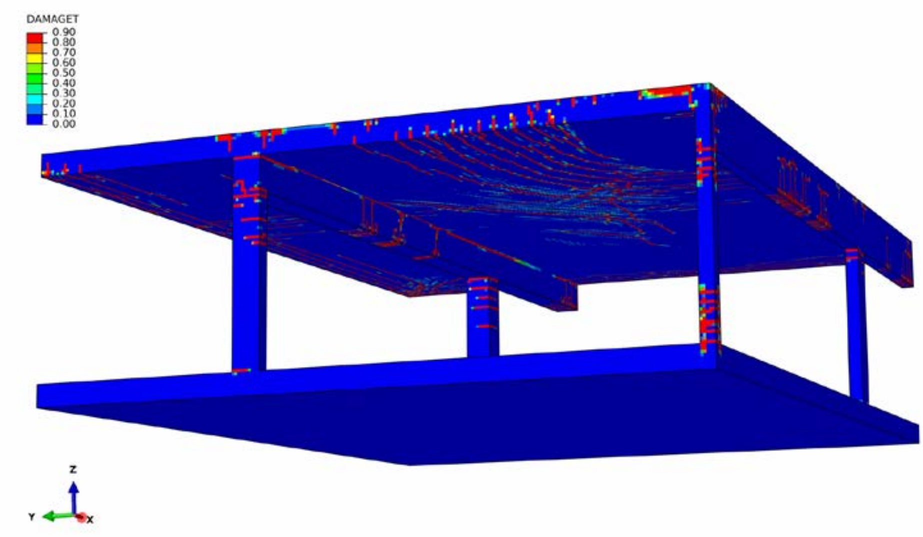

Fig. 9. Distribution of structural damages. Blast load.

4. CONCLUSIONS

Presented results of finite element model calculations show the problems concerning the evaluation of integrity for structures of multistorey buildings. Generally, in analyses of such problems it is necessary to take into account many features of complex problem, including geometric characteristics, material behavior from initial linear elastic response until its full damage, as well as description of exceptional load (appropriate regarding the real acting of the load on the structure).

Thereby the integrity evaluation for structures subjected to exceptional loads (impacts or explosions) comes down to the nonlinear dynamic analysis carried out by explicit procedures, concerning large or very large numerical discrete model. This results in significant costs (expressed in time or expenditures) not always possible to accept.

Although the intensity of loads assumed in numerical example does not lead to the loss of structural integrity of the building, obtained solution gives certain information concerning the development of damages in time and space. Comparison of results of analyses carried out for two different variants of load (removal of the column, load due to explosion – the amount of explosive charge was assumed to obtain the similar effects of loads for both variants) shows the significant differences in the process of damages development for the structure. Major differences concern the time of progress of damages, their spatial distribution and intensity. Maximum values of dissipated energy are higher for the case with removed column, but effects of explosion give the expanded system of damages also in a zone relatively far from the center of explosion. This may cause the situation of progressive collapse of the part of structure, or even its total collapse.

It should be noted that the attempts of integrity evaluation for structures, which are based on assumption of certain equivalent loads may lead to incorrect results and conclusions. To avoid this situation, the numerical analysis can allow for correct evaluation of effects, but in a cease of exceptional loads produced by impact or explosion it in necessary to assume various scenario of loads. The results of analyses confirm the controversy of certain JCSS [2] recommendations or methodology described in TU0601 [3] reports.

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The effect of the hydrodynamic loads of waves regarding on seismic vulnerability of harbor structures with vertical walls

Mirela Popa

***Abstract* – The assessment of vulnerability for harbor structures should take into account some particularities such: the appropriateness of the calculation criteria in relation with the type and nature of hazard; modification of the characteristics for the granular materials from the body of harbor structures and from foundations due to cyclical loading induced by waves; different modeling of loading of waves, depending on the type of wave, breaking or non-breaking. In this context I aimed that in the process of evaluation of seismic vulnerability to keep account of the changes induced by considering the dynamic wave action. Modeling of wave action had in mind in particular the impulse type actions. This type of load can be met in the case of breakwater with vertical walls, if it fulfilled the breaking criteria The analysis found that neglecting a state of stresses and strains changed as a result of interaction with the marine environment may lead to underestimated values of seismic response in terms of displacements.**

***Keywords* –** breaking wave, maximum displacement, residual displacements, seismic vulnerability

1. INTRODUCTION

In the last years the safety of life has become an increasingly important challenge in all the areas of the human activity. In civil engineering this objective is aimed from two viewpoints: new construction of whose design is based on performance objectives and old construction made by the design codes to be outdated, but having a long lifetime.

The superior performance objectives defined by the newest design codes require satisfying the exigencies of performance levels associated with a seismic hazard level with an increasing reference return period of the seismic action.

The assessment of vulnerability is a necessary activity, important for assessing the risk failure of the extreme events but also as part of a plan for the management of maintenance for port facilities. It should be noted, that for the harbor structures, considering their specifics and prevailing loads, besides the seismic vulnerability it is advisable to study the vulnerability at extreme phenomena such as those due the sea state or of the impact of the vessels.

Seismic vulnerability of the structures can be expressed in many forms, such as vulnerability functions, damage probability arrays, or fragility curves, and using various methods. The seismic vulnerability of buildings is the subject of much research, but on the harbor structures vulnerability are few references. The seismic vulnerability expresses the state of damage that can occur to the structure for a certain level of seismic input. The assessment of state of damage requires the calculations of seismic response related the failure mechanisms

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specific to the type of structure. The main types of port structures can be distinguished as follows: rubble mound, monolithic, tied wall. Failure mechanisms for each of these categories are presented in detail in the paper [1].

Of these, structures with vertical impermeable walls show a growing interest because the interaction with the waves has several forms: slowly varying wave forces generated by nonbreaking waves and short-duration impulsive wave forces caused by breaking waves directly on the structure. The wave loads leading to stress variations in the soils that can get to degradation in soil strength generated by the pore pressure build-up. For this reason, the objective of this work is to present the changes in the seismic response of a breakwater with vertical walls if it takes into account irreversible deformation and decreasing bearing capacity of foundation due to hydrodynamic loads of impulsive waves.

2. THE DYNAMIC NATURE OF LOADS GENERATED BY BREAKING WAVES

Wave forces on structures are difficult to estimate, being influenced by both the wave conditions and geometry of the structure. Even if the waves' action has a dynamic character, taking into consideration that the fundamental period of the defending structures (0,2…2s) is significantly smaller than the period of the regular waves, the response of structures is deduced usually by the static application of the wave loading.

For non-breaking waves, the pressure at the wall has a slight variation in time. This load is called pulsating or quasi-static because the period is much longer than natural period oscillation of the structure. Hence the wave load can be considered as a static load but a careful analysis is required if the wall is placed on fine soils. In this case the pore pressure may increase, leading to a significant weakening of the soil.

If the waves that break in a plunging mode and form an almost vertical front just prior to the contact with the wall, then a very high pressure is generated. This pressure has extremely short durations. A negligible amount of air is entrapped and result a very large single peak force continued by very small force oscillations. The pressure peak has duration of the order of hundredths of a second [11].

The analyses presented in this paper were considered that during the life of breakwater it is possible to fulfill the breaking criteria that it's leading to this type of impulsive loads. Modeling of these actions was made as presented in [10]

A slowly quasi static part, denoted P1, is applied on the entire front of quay wall, and an impulsive load P2 acts on the top of quay wall, thus modeling the wave slamming effect. For short duration impulsive wave forces generated by the waves breaking directly on the structure I showed in [9] as the dynamic amplification factor depend o the time increasing and time decreasing for triangular shock. In numerical analyses carried out I have opted that the time increasing and time decreasing will be both 0.2s.

Most vertical harbor structures are placed on the rubble foundation and under the base of caisson a water flow takes place through the rubble and it produces an upward pressure. This upward pressure reduces the vertical stabilizing forces. Therefore, a correct modelling of dynamic actions generated by the waves should consider both horizontal wave action and upward pressure for which it is proposed the linear distribution Fig. 1.

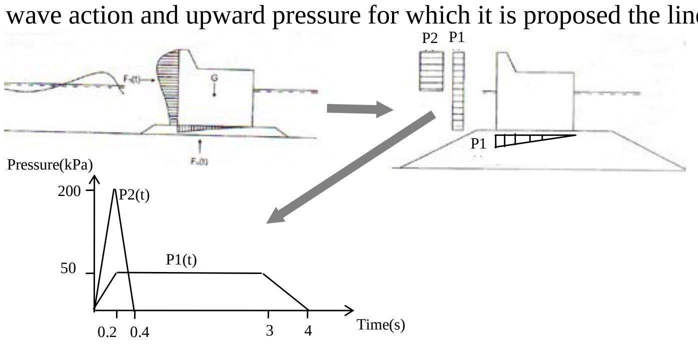


Fig. 1 The hydrodynamic pressures due breaking waves

In this paper, the upward pressure will be modeled only by the hydrodynamic pressure component, P1. I kept in mind and the hydrostatic pressure component by modifying the density material for the submerged part of the structure.

3. THE HARBOR STRUCTURES VULNERABILITY

Many studies have focused on the study of the vulnerability of buildings, but there are few references about the harbor structures vulnerability.

Seismic vulnerability of the structures can be expressed in many forms, such as vulnerability functions, damage probability arrays, or fragility curves, and using various methods. The seismic vulnerability expresses the state of damage that can occur to the building for a certain level of seismic input.

For the buildings, but also for other structures, assessment of seismic vulnerability requires completion of a connection between the data about the structure's ability to respond to horizontal loads – seismic capacity - with data about the intensity of seismic event – seismic input as shows in Fig.2.

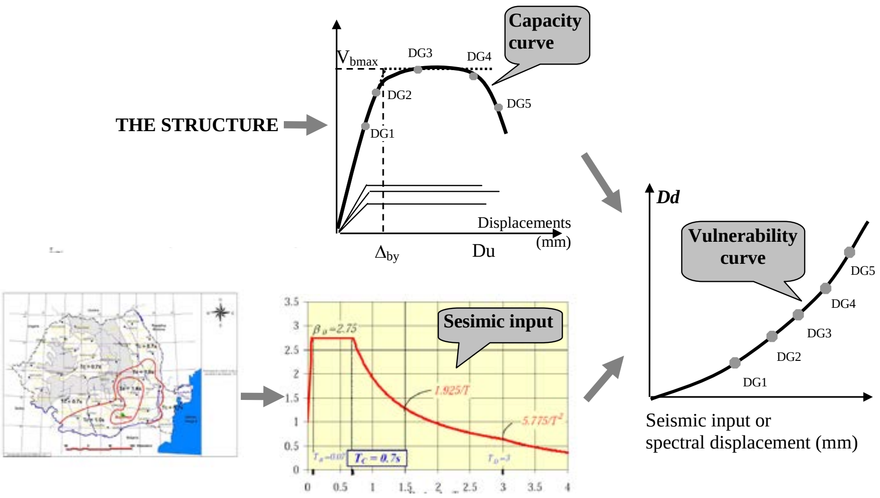


Fig. 2 The general concept of seismic vulnerability

A description of seismic action, used especially in seismic design is one that is based on the synthesis of seismic response for structures with single dynamic degree of freedom for a certain level of damping, in the form of response spectra and design spectra. Evaluation of response port structures using response spectra requires the following approach: identifying a system equivalent with a single dynamic degree of freedom or application modal superposition which is limited to the structures with elastic behaviour. Because it wanted consideration of nonlinear character of granular media response it has not been used this way for seismic input description.

It was chosen as a description of seismic input to make using the accelerograms. They used two accelerograms: Vrancea (Romania) 4 March 1977 and Friuli (Italy) 6 May 1976.

To express the degree of vulnerability, the damage shall be expressed, depending on the specific failure mechanism. For example, European Macroseismic Scale (EMS) provides five levels of seismic damage and for each one of them there is a qualitative description of loss for masonry buildings and another for reinforced concrete buildings. To express degrees of vulnerability is needed to establish numerical criteria corresponding to the level of losses incurred by the structure.

There are only few references in which it approaches the problem of identifying the appropriate numeric criteria levels of degradation for existing port structures.

Some damage criteria are specified in design guidelines for port structures, especially for mooring structures. These are expressed in terms of allowable displacements and stress level and are representing the required performances of a structure against design earthquake motions, for particular types of mooring structures [4], [5], [6].

Table 1 Quantitative description of degree of damage for gravity quay, Uwabe, 1983 [4], [5]

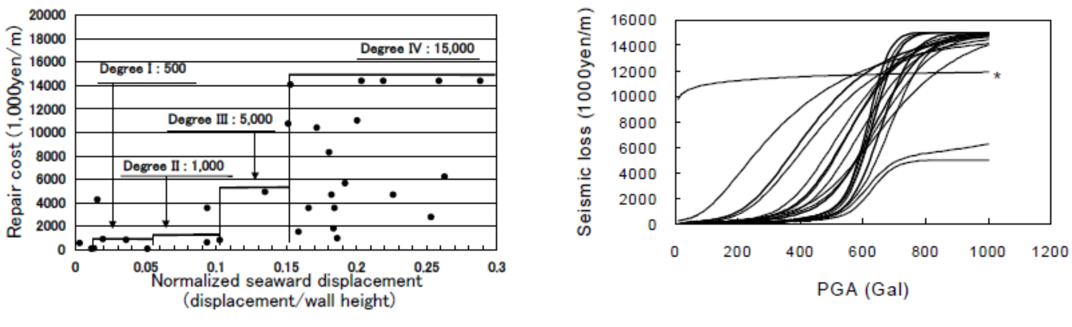
Degree of damage	Gravity quay wall		Sheet pile quay wall	
	Maximum residual displacement at top of wall (cm)	Average permanent displacement at top of wall (cm)	Maximum residual displacement at top of sheet pile (cm)	Permanent displacement at top of sheet pile (cm)
0	0	0	0	0
1	<25	<25	<30	<10
2	25 - 70	25 - 40	30 - 100	10 - 60
3	70 - 200	40 - 200	100 - 200	60 - 120
4	>200	>200	>200	>120

Table 2 Quantitative description of degree of damage for gravity quay, [6]

Extent of damage	Deformation rate (δ/H)	Tilting towards the sea
Degree I	<3%	<3 deg
Degree II	3-10%	3-5 deg
Degree III	>10%	5-8 deg

It must be noted that the values exemplified above represents the acceptable level of damage for new structures in performance based design that requires achieving a performance objective and there is not the seismic capacity for a given seismic input as it is required to define the vulnerability.

In the works devoted to assessment of the seismic risk for Japanese ports the damage level is defined based on restoration cost [8], as shows in Fig.3.



Damage level criteria and seismic loss

Seismic loss evaluation for 20 situations

Fig. 3 The Seismic losses in risk analysis (PGA is peak ground acceleration, 100 Gal=1m/s^2) [8]

The assessment of the port structures vulnerability is rather complicated and involves the following activities:

- identifying the capacity to take horizontal seismic forces for various types of structures;
- the quantitative and qualitative description of degrees of damage in terms of displacements, stress.

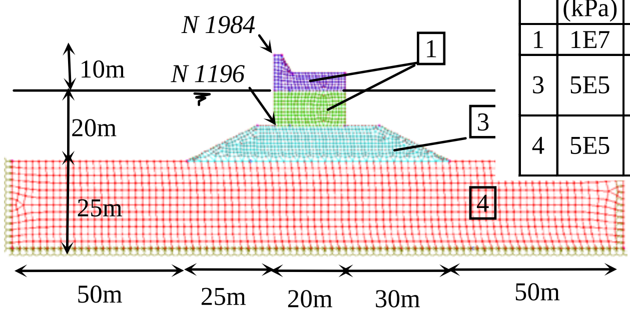
As a first phase of study, in this paper it is showed how to port structures capacity to take seismic horizontal forces is diminished if it is considered the irreversible deformation due the interaction with breaking waves.

Identification of damage will be done in terms of displacements: the maximum horizontal translation at the top of the crown; the maximum rotation of the seawall.

3. COMPUTATIONAL MODEL AND RESULTS

Evaluation of seismic response for analyzed breakwater It was acquired by nonlinear dynamic analysis using the software with finite element COSMOSM/2.6.

Computational model represents a caisson breakwater placed on rubble mound foundation with the geometry similar to that presented in [10]. For caisson was considered an equivalent material modeled with finite elements with linear behavior. For granular materials were considered non-linear model Drucker Prager. Fig. 4 shows a sketch of the calculation model used in this paper.



	E (kPa)	ρ (t/m^3)	ν	C (kPa)	ϕ (deg)	Finite element PLANE 2D, plain strain
1	1E7	2	0.20			linear elastic
3	5E5	2	0.25	1	45	elastic-perfectly plastic Drucker-Prager
4	5E5	2	0.25	1	35	elastic-perfectly plastic Drucker-Prager

Fig. 4 The mesh for computational model

This type of analysis requires the definition of time curves time that multiply the pressures P1 si P2 and the seismic base acceleration. For such an analysis, the gravitational loads must not be neglected. It is therefore necessary to define a curve time for gravitational acceleration. The curves time are reproduced in Fig. 5.

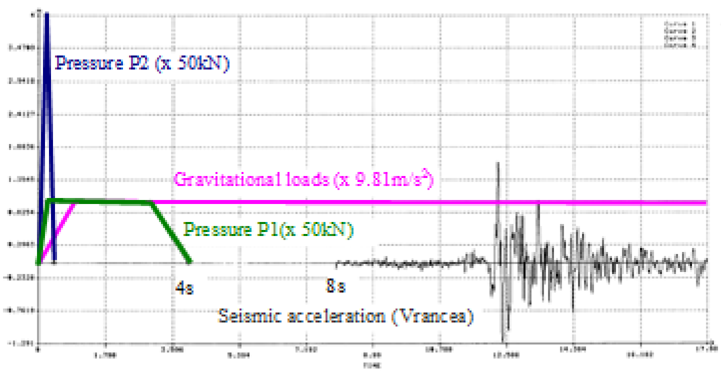


Fig. 5 Curves time for different action

Seismic response assessment was made for the following cases:

1. The seismic action is applied at time t = 0 and is described by one of the above mentioned accelerograms-Vrancea or Friuli
2. At time t=0 it is applied the horizontal pressures P1 and P2 as the model shown in Fig. 1 and at time t=8s it is apply the seismic action.
3. The effect of hydrodynamic waves considering hydrodynamic upward pressure which is multiply with the same time curve as the horizontal pressure P1.

The time history response displacements for nodes 1984 and 1196 for the cases A, B and C for Friuli earthquake is shown in Fig.6,7,8. Synthesis of the obtained results from the numerical tests is shown in Table3.

Table 3 Results from numerical tests

cases	Vrancea accelerogram a_g =0.16g	Vrancea accelerogram a_g =0.32g	Friuli accelerogram a_g=0.50g
Maximum crown displacements (node 1984 in cm)			
A	1.7	3.3	7.1
B	7.8	9.1	10.8
C	11.3	12.6	14.3
Relative displacements at crown versus bottom (node 1984 – node 1196 in cm) // Tilting sea wall (in degree)			
A	0.85 // 0.02	1.51 // 0.04	4.1 // 0.12
B	5.36 // 0.15	5.85 // 0.16	6.1 // 0.17
C	8.36 // 0.23	9.27 // 0.26	9.6 // 0.27

The response time-history shows that breaking waves effect can be expressed as a residual displacement of about 6 cm, if we are considering only the effect of horizontal hydrodynamic pressure. Consideration of upward hydrodynamic pressure leads to increase residual displacements of about 9 cm.

It also should be noted that the increasing magnitude of shock or changing its specific times will result an increasing of shock response in the terms of displacements. For this work it considered the report P2/P1 = 4, but in paper [10] is proposed also a second version with P2/P1 = 8

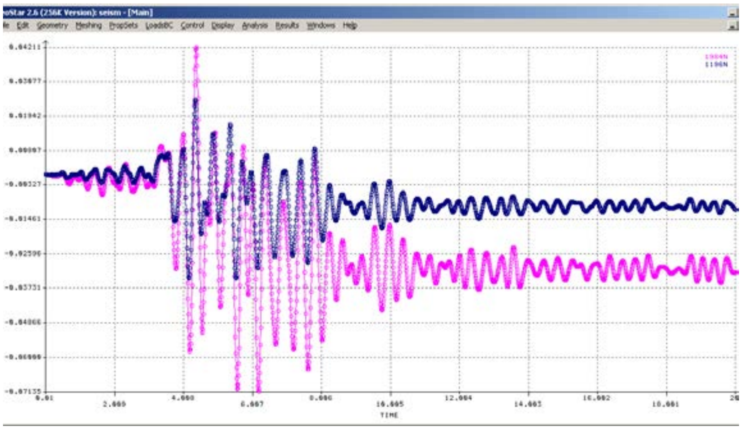


Fig. 6 Response time history - displacements for nodes 1984 and 1196 node (in m) – Case A (Friuli seism)

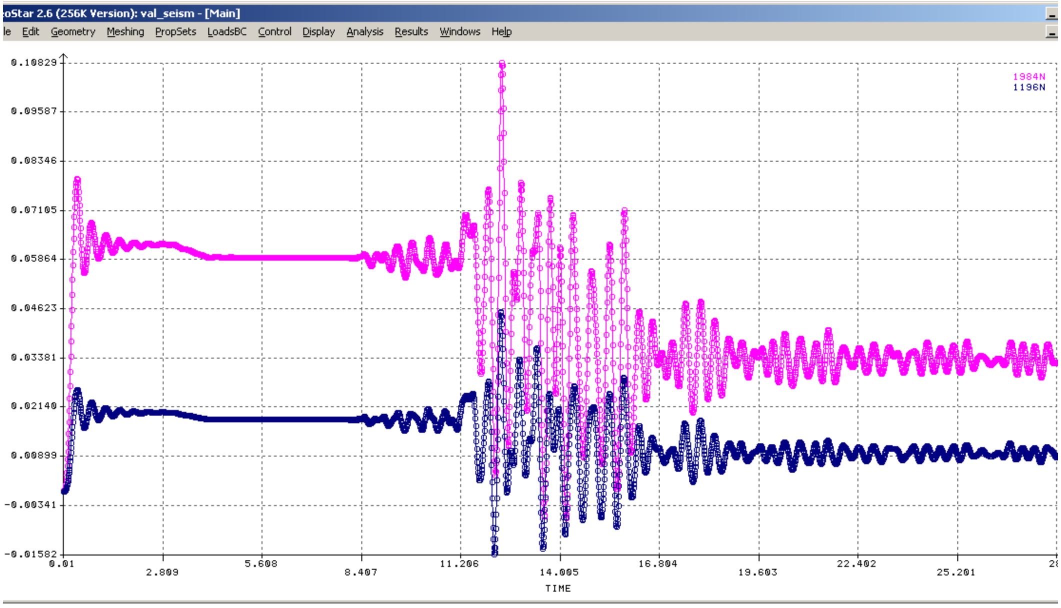


Fig. 7 Response time history - displacements for nodes 1984 and 1196 node (in m) – Case B (Friuli seism)

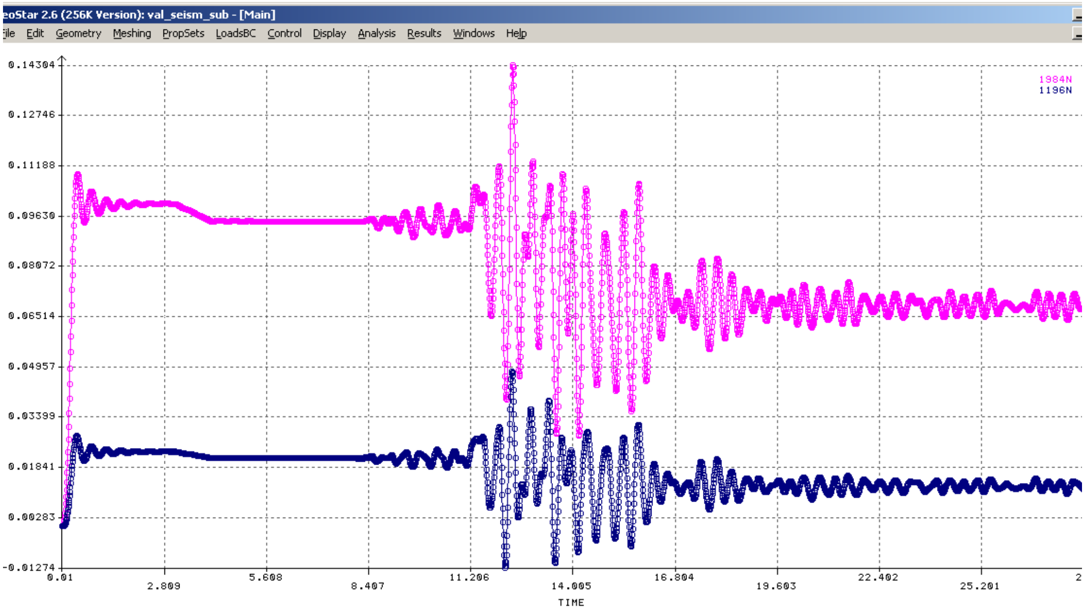


Fig. 8 Response time history - displacements for nodes 1984 and 1196 node (in m) – Case C (Friuli seism)

4. CONCLUSIONS

The identifying a quantitative response characteristic of port structures such as breakwaters, which will be significant for the assessment of the degree of damage, is a difficult task if one takes into account the diversity of constructive solutions and the multitude of mechanisms used for each type of solution. In this context I aimed that in the process of evaluation of seismic vulnerability to keep account of the changes induced by taking in account the dynamic wave action. Modeling of wave action had in mind in particular the impulse type actions, which are generated by the breaking waves on vertical wall.

The analysis found that neglecting a state of stresses and strains changed as a result of interaction with the marine environment may lead to underestimated values of seismic response in terms of displacements. These breaking waves that do not produce large damage and does not require intervention on the structure will lead to accumulation of residual displacements that will add to the displacements produced by earthquake.

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SECTION II

MANAGEMENT IN CIVIL ENGINEERING

CONSTRUCTION MATERIALS AND TECHNOLOGY

Structural analysis of construction industry and firm's strategy: A study of Bulgarian construction industry

Aneta Marichova

***Abstract* - Since the economic change in Bulgaria, the construction industry has played an important role in terms of the economic, social and culture development country. The industry generated 10-12% of GDP, 10% of value add and 11% of the workforce. Although, now the prospect for the Bulgarian construction industry is very promising, many small and local firms have serious difficult, such as poor performance and low competitiveness. For construction firms in Bulgaria, the change in environment is the critical starting point to gain or keep competitive advantage. The managers can identify and understand the opportunities and threats and define successful strategy in long run of strengths and weakness of their companies. This paper make analysis conceptual model: "Market structure - Firms strategy - Competitive advantage" to an able a construction firms to development a long run strategy that generated competitive advantage.**

***Keywords* - Bulgaria, construction industry, structural analysis, competitive strategy and competitive advantage**

1. INTRODUCTION

Construction is a major industry in each country. In EU27 the sector accounted average 10-12% of GDP, 10% of value added and 11% of the workforce [1]. Development this sector is a factor, but a consequence of the general economic activity. In last years the change in world economy define the change in construction industry. These change include globalization, development information technology and telecommunication, increasing government control and regulation. The dynamic environment affects competitive position, strategy and competitive advantage and creates opportunity for firm's development, but and threats for firm's position. The development international market - economic growth, increasing income per capita, urbanization, increase demand construction product - building and non-building construction. The increasing demand can not satisfied by local companies in terms technology and management skills. Therefore, the construction firms must development activity, organization, capabilities and relations with domestic or international companies. The analysis construction market and construction product must give the next objective specific characteristics:

1) The construction object is connection with land, but land is very scarcity resource.

2) The construction product is location on the place, which established.

3) Construction firms use very much resources - human, capital, financial, material and this define high value finish product.

4) The specific construction is a result specific product and process. Construction product include characteristic of goods and services. Therefore, construction firms have complicated structure. The construction process includes investor, executive firm, under executive firms, clients, suppliers. The aim everyone is profit and long run growth or survival. The long run relations decrease risk, losses and increasing profit and advantage.

5) General construction include three market segments- construction real estate, commercial building and civil engineering, with different, specific product, demand, and supply, diverse costumers and competitors.

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6) Structural business statistics (SBS)[2] EU27 separately recognized construction activities (NACE, section F), to include the construction of building, construction installation, building completion, civil engineering and other specialized construction which can be combined in general construction, which is the subject of statistics.

Construction activity is particularly cyclical, influenced by business and consumer confidence, interest rates and government programs. During the recent financial and economic crisis, activity in the construction sector declined greatly and need of effective strategy decisions and policy. Today, construction in Bulgaria produced on level 2006 year. The main question for every firm is: Do have perspectives for development and what strategies ensure competitive advantage? The answer this question includes: analysis market structure, market position every firm, competition analysis and firm strategy that realize competition advantage. Every strategy can be combined firm's mission, aims, possibility, resources, and elements environment - competitors, costumers, suppliers, contractors and subcontractors. Therefore, structure industry defines competitive strategy and advantage. The strategy must ensure protection and/or growth market share, firm's position and its approach to survival and success.

Traditionally for market analysis, use two distinct yet related models. One method of analysis is the markets theoretical framework "Structure-Conduct-Performance" paradigm [3] presupposed only a limited number of key differences between firms: size, differentiation of the product and vertical integration. Key differences between firms related to market share and industrial concentration. High concentration may be a natural result of the market mechanism if there is no freedom to enter the market, if there is a threat of new rivals and if the level of minimal optimal scale firm is high. Development technology, innovation product or process, growth effective and productivity in different industry are factors, which change conduct firms and method for analysis. Therefore, another theoretical model is Porter's Five Forces (Bargaining power of suppliers, Industry competitors, Bargaining power of buyers, Threat of entry, and Threat of substitutes)[4]. Each element of five forces determines industry profitability as a result present and future change and possibility for competitive advantage and realization profit above average in long run. Five forces determine firm position, protection and possibility for competitive advantage - cost leader or differentiation.

The intensity of rivalry influenced by the following industry characteristic: exit barriers, industry concentration, fixed cost/value add, industry growth, product differences, switching cost, brand identify, diversity of rivals.

The threat of substitutes influenced of the next factors: switching cost, buyer inclination to substitute, price performance trade-off of substitutes.

The threat of entry influenced of the next factors: absolute cost advantage, learning curve, access to input, economies of scale, capital requirements, brand identify, switching cost, access to distribution, proprietary products and government policy.

The bargaining power of suppliers influenced of next factors: supplier concentration importance to volume to suppliers, differentiation of inputs, impact of inputs on cost or differentiation, switching cost of firms in industry, presence of substitute inputs, and threat of integration.

The bargaining power of buyers influenced of next factors: buyer volume, buyer information, buyer concentration vs. industry, brands identify, price sensitivity, threat of integration, product differentiation.

As a result of the combined two models, we construct the model: "Market structure - Firms strategy - Competitive advantage" and will analyse:

1) The strategy factors for competitive advantage - dynamic demand and supply, market structure and government policy.

2) The strategy sources for competitive advantage.

3) The strategic decision of the construction firms.

2. DEVELOPMENT CONSTRUCTION INDUSTRY IN BULGARIA AND FIVE DRIVING FORCES

The dynamic development private construction in Bulgaria beginning with economic change and especially input currently board in 1997 year. In period 1997-2007 year, the construction growth with 15-20% average annual [5]. In January 2008 year, construction win peak, without externally financial pressure. During all this year, the firms finished financial insure project. Only real estate increase with 30% towards 2006 year and 17% towards 2007 years [6]. The construction market is market with long run between invest, starting project and realize object, for that reason in 2007-2008 continue increase investment in building. However, this fact defined in next period decrease prices, big unfinished buildings and difficult survival firms.

The world financial crisis reduced the banks credit and increase requirement for investor. In construction, the recession began in 2009 years. The real estate market decrease with 15 % and 8.9 % decrease the whole market [7]. Many construction firms change his work in infrastructure and this segment increase with 6 % towards 2008 year. In 2010 years, the foreign investors in construction decrease with 70 % and construction decreasing with 40 %. In 2011 year, the industry decreasing with new 10 % and in 2012 year still 8% [8].

1) The intensity of rivalry in Bulgaria construction industry:

In Bulgaria, the construction firms are many number and relative very small size. In 2009 year, their numbers is over 23 thousand, and now are over 17 thousand [9].This is 6.6 % of whole firms in Bulgarian economy. The small construction firms (with less 10 persons employed) are 95 % of all firms. They generated 23% of net revenue and played particularly important role of general activity and employment in country. The medium-sized (with less 250 persons employed) and large enterprises are 5 %, but generated 45 % of net revenue of sales. Only 2% of these firms generated over 50 % of net revenue of sales)[10].

Average employment in construction market is 10.4 % of whole employment, but now decrease with 40% and unemployment is higher than average for country. The construction accounted for more 25-28 % of GDP in peak years, and now only 5.1%. In Bulgaria construction generated an estimated 10.8 % of total value added (value added represents the difference between the value of what is produced and intermediate consumption) and now this share decrease too-5.9 %) [11]. Average personnel costs (defined as personnel cost divided by the number of employees) are EURO 3.6 thousand and wage, adjusted labour productivity (the value added divided by the number of persons employed) is EURO 9.0 thousands. Interesting fact is comparison this indicators with average personnel costs and labour productivity in EU-27. In EU-27 average personnel costs are EURO 31 thousand and labour productivity is EURO 40 thousand [12]. In Bulgaria the wage, adjusted labour productivity ratio is 279% and in EU-27 this indicator for construction is average 129 % and this to put a question for motivation labour and stimulus for development [13]. In construction, the profit (the relation between the gross operating surplus and net revenue of sales-turnover) and gross operating rent (defined as the size gross operating surplus relative to turnover) is higher - 15%, comparison average value (13%) in EU-27. The investment in Bulgaria is higher in peak years and strong decreasing. In period 2000- 2008 years annual investment are EURO 900 million, but in 2011-2012 year only EURO 15 million. The average propensity to investment (the size gross investment relative to value added) is 12-13% annual and decreasing under 5% annual [14]. In every country, government and public sector are major investors in civil engineering market segment. This is importance factor for dynamic development industry and ensure high employment, income, growth demand and realize multiplication effect. But in Bulgaria the government and public sector debt construction firms over EURO 60 million and as a result the debt between construction firms is over EURO 10 billion and unfinished building is over EURO 10 billion too)[15].

The development every market structure is function of development demand and supply. General market conditions - demand and supply, faced by an industry are often important factors in the choice of conduct by firms and for the ability of firms to generate profits and meet expected performance goals.

The demand of construction product depend of price product, price elasticity of demand, income, income elasticity of demand, cross-price elasticity, expect and preferences consumers, the policy government and banks. The construction market includes three segments and respective the different demand. The demand, real estate segment defines of all households. In Bulgaria in period 2004-2009 years demand increase always and purchases increasing independent of increasing price. This is a result of increasing income, credit active bank and optimistic expect households that have the preferences for new, modern houses. The demand of commercial building defines of national and foreign investor and of expectations of future returns of investment and the demand of segment civil engineering define of government and public sector. The basic indicator for construction demand is publishing licence for building and value public order. Today the number and value two indicators are decreasing drastic.

The supply construction firms is a function of price product, production cost, development technology, credit policy, mobility and adaptation firms on one segment towards other. Now in construction use divers kinds of modification in the nature of contracts, the structure of deals and the other way in which deals are cut. This is reflection on the change competition in industry. However, this line of competition development is destructive. Expenditure, productivity, profitability, rate of investment is very much different in small and large firm. Financial indicators in some lieder firms keep stability and increasing activity in new national and foreign markets, realize new quality project.

The cyclical fluctuation has significant effect on the profitability and competition in construction industry. In this competition situation and the pessimistic future of the companies will not make the managers optimistic.

2) Product characteristic:

The construction firms supply a wide range of activities - commercial, industrial building and diverse transport infrastructure that is a result of economy and urban development. The high specialization functions of construction firms make very difficult define their product and in fact, they produce both goods and services. A wide range of activities and complexity the final products given very mach opportunities for different competitive business strategies.

In the market real estate building, the product need comparatively of low technology and differentiation product and intensive price competition, because the choice by the buyers is based largely on the price. In the market commercial building, the construction firms realize large project, use high technology and managers skills. They supply the differentiation product and can create high value add for investors and customers. In the construction industry work diverse competitors, which have different organization, specialization, management, human and capital resources, vertical relations with construction materials manufacturing. This diverse means, that firms may have different development goals, different priorities and different strategies. This fact, make very difficult analysis the behaviour competitors and predict future change.

3) Exit barriers and barriers to entry:

For many small construction firms, like consultant or project management in Bulgaria, the barriers to exit is low, because they have low fixed cost. However, for large construction firms, their specialized assets and high fixed cost as well as other factors make the exit barriers higher. This factor, define the higher profitability on the whole industry, comparison average standard.

Barriers to entry in construction market are not high. The important strategy barriers to entry new competitors are reputation, history, tradition and corporate image the firms. The development globalisation, more and more foreign firms will enter into Bulgarian construction market. These large firms have competitive advantage, that as a result of using high technology, managerial skills, human and capital recourse, economies of scale.

4) Bargaining power of buyers:

The buyers in construction have different characteristic from other industries.

- The buyers and the owners of the project initiate the project and realization object and ensure work and revenue for construction firms. This gives a lot of bargaining power to the buyers over construction firms.
- Every project that the construction firms realize defines opportunities for increase profit or survive. In long run, this factor change firm position on the market and possibility for competitive advantage.
- In construction industry are so many small firms, whose technical and management ability are similar, and that makes the buyers have a lot of choice.
- The buyers have no switch cost and they can just decide the construction firm, based on their, own strategy.
- The choice buyers define of quality of the project after it is finished. As a result, the experience and reputation of companies are important source of competitiveness. The increasing technical, procedural complexity of the projects, requires the construction firms active participation in all phases of the project.

5) The bargaining power of suppliers:

The building materials manufacture characterized by the large, dominance firms and oligopoly structure. That makes the construction firms, especially small firms in a disadvantaged position. Therefore, the vertical integration is objective factor for decrease transaction cost and stability production in construction market.

We can conclude: In Bulgaria, construction market characterize with many number of sellers and relative small size, very little degree differentiation product, which are all characteristic construction in every country and low barriers for entry. The construction firms have not activity vertical relations with suppliers. Many large firms have complicated structure as a result vertical integration. Characteristic three market indicators shows model perfect competition in construction market, but analysis market measures change this picture. Structural analysis use coefficient of concentration - $C_{4,}$ C_8 and $C_{10,}$ which measure the share of total revenue of top four, eight and ten largest firms in the total revenue construction industry in Bulgaria. The Herfindahl-Hirschman Index of concentration (HHI) represents the sum of the squares of shares of total revenue of each particular firm in the total revenue of the industry. In this research, the HH Index calculated on the shares of total revenue of "Top 20" largest firms in the total revenue of construction industry, whereas other firms assumed to have equal market shares. The level of concentration evaluates of values of concentration ratios $C_{4,}$ C_8 and C_{10} and the HH Index. The economy theory defines: if $0<CR_4<40$ and $0<HHI<1000$, the market structure can interpretation as effective competition or monopolistic competition and tendency development towards oligopoly. The effective monopolistic competition and oligopoly have increasing potential for profit and advantage and define as workable competition [16]. Definition effective monopolistic competition and oligopoly have increasing potential for profit and advantage. In period 2007-2011 years the financial reports firms [17] and authors calculate show increasing value coefficient of concentration and Harfindal-Harshman Index: CR_4 [6.9-25.5], CR_8 [7.17-38.5], CR_{10} [7.7- 40.3] and HHI [9.1-226.2]. This heightening of level concentration has combined with a slowing the demand, besides the process realize more hurriedly comparison with general decreasing construction work and total revenue. In "Top 20", the first five firms working in civil engineering segment and have whole 25.5% market share, which always increase in this period. The firms, which included in "Top 20" and working real estate and commercial building market segment have whole 8.6% market share. Therefore, market indicators shows, that in real estate and commercial building segment, the market structure is effective monopolistic competition and in the civil engineering market structure is oligopoly.

3. FIRMS STRATEGY AND COMPETITIVE ADVANTAGE

The firm's conduct and strategy is a function of market structure. The analysis defines strategy factors for competitive advantage. The managers can identify and understand the factors that establish opportunities and threats and choose successful strategy in long run of their companies.

The opportunities and threats to development a long run strategy that generated competitive advantage:

The perspectives development construction is result of development demand, social and economic in every country. Analysis segment real estate in Bulgaria shows deficit modern building with high quality and very little green, stable construction. Development commercial building market will result of increasing economy and business activity. This market is market quality “green building”, which decreasing firm’s cost, as a result of learning curve and economies of scale, therefore construction firm can supply quality and cheap product on the real estate market in next period. The characteristic of civil engineering market segment is low development and scarce investment, because the perspectives are very much. In this segment, importance factor for development is development public-private partner (PPP) and assimilate money of Europe programs.

The analysis construction market structure shows strong fragmentariness market. The reasons are next:

1) Low barriers for entry and high barriers for exist (law requirement and control construction)
2) Absence economies of scale
3) Transfer firm’s resources of one to other segment
4) Strong influence economy cycle
5) Increase role of buyers and suppliers
6) Increase construction cost

7) Increase cost for individual, different project

8) Increase cost for differentiation product and advertising

9) Increase different demand and preferences consumers

10) Increase regulation and control construction work

The fragmentary market can decrease with the next actions of firms:

1) Realize economies of scale

2) Diversification or specialization firms

3) Horizontal and vertical mergers and decreasing transaction cost

4) Entry strategy investor in Bulgarian construction market

5) Vertical integration relations

6) The choice successful scope and realize competitive advantage

The construction firms are many of number, many small and which work in diverse market segments. Every segment has specific and this define different opportunity for success and risk. Risk influenced and defined of next factors:

1) Big deficit of the professional management skills in the small and medium-size firms.

2) Capital resources are scarcity, but it is a factor of effective using and realized economies of scale and increasing firm capacity. Financial investors have a big influence on the industry. They have become more powerful and asked for more participation.

3) Increase prices rows, materials and as a result - the total firms cost. This is very higher barriers for small construction firms, because construction used very much resource.

4) Increase the land prices, as a result increase the building demand. On the suppliers side, the land is scarce and owners to get more of the profit out of each transaction.

5) Increase the unstable financial revenue and reduced investment, therefore and the work and the profit construction firms.

6) Increasing share damping prices and offer project with price bellow average variable cost and marginal cost. This firm's policy define survival problem.

7) The buyers (brokers and households) are consolidated and have accentuated the structural problems.

8) Increase government requirements and standard in EU-27 about "green" building and increasing cost for environment.

The fragmentariness market defines many number and small size market segments and very much competitors on every segment and necessary of horizontal and vertical integration. The entry strategy investor in Bulgarian construction market is importance factors for decrease fragmentariness market, stability firm's position, decrease influence the fluctuation business cycle and suitable development construction. The strategy investor can ensure higher value added trough experts, information, capital resources for national and international expansive, economies of scale and big building project. The investor can ensure professional management and strong control firm's work.

The success strategy of construction firm is a result of effective organization all process - architects and constructors, investors, executive firm, subcontractors, clients, relation suppliers, control performance object. The construction firms must development effective vertical relations on the principles effective logistic, just in time and supply chain management and reduced transaction cost. This is chance to realize economies of scale, as a result of increase productivity and stability financial revenue, development and use innovations methods in construction.

We can conclude: In segment real estate and commercial building with effective monopolistic competition successful strategy is strategy product position. This strategy realize with elements marketing mix - product, price, place and promotion. The aim firms, is protection and increase market share with differentiation product, that firm produce for target market [18]. In this segment, the strategic sources for advantage are: firm's strategy, innovation in green building, the change consumer behaviour, increasing qualification labour, use quality rows, materials, technology and activity government policy. The every firm must comparison his individual project with the next good competitor and estimate possibility for realization horizontal and vertical differentiation product, effect advertising, prices, level quality uses resources and firm's specialization. In this comparison analysis, every firm can define his competition advantage - differentiation product or differentiation cost. This decision ensures stability and increase position firms, as a result of established loyal clients, trough production and supplying higher add value comparison other competitors.

In segment civil engineering with oligopoly, successful firm's strategy is strategy position. A firm choice its boundaries based on its ability to create value within a positioning. This strategy realize with decision for mergers and increase market power, entry into a new market, entry into a foreign market, provide a missed attribute, diversification, issues industry capacity, innovation in new product, or cooperative strategic behaviour[19]. Strategy position firms ensure increase return to scale and realize lower average cost in user technology. Therefore, the firm must realize advantage in differentiation cost.

In segment civil engineering the strategic sources for advantage are: firm's strategy, innovation in green building, development public-private partners, the change demand, increase qualification labour, use quality rows, materials, technology and activity government law policy. The every firm can comparison his individual project with the project next best competitor and evolution possibility realization increase the horizontal and vertical borders and market share, the return to scale, uses higher quality resources, that is factors for increase firms capacity, effective vertical relation and diversification. In this comparison analysis, every firm can define his competition advantage - differentiation product or differentiation cost - ability to establish and supply quality construction product with higher economic effective and price for suitable best technology. This decision ensures stability and increase firm's position as a result, supply higher add value, comparison competitors. Realization competition advantage of every firm ensures growth profit and market share, increasing evolution (estimation) consumers and all society for working firm and whole construction market.

The choice a strategy, which can ensure the competitive advantage, is the choice diversification or specialization firms. The choice diversification has a result the economies of scale and cost differentiation. The choice specialization, define advantage in differentiation product according to specific target market and clients. The result of two ways is increase productivity in each firm and long run advantage. As a result of competitive advantage, the firms can realize the aim - they get higher profit comparison average in market.

Very difficult problem is problem choice scope [20]: 1) area scope, 2) vertical scope, 3) product scope- what produce - goods or architecture, project service, 4) market scope - choice target market segment, when firm supply and success competition. The choice competition advantage is the choice scope. The choice cost differentiation is a function of wide scope in all aspects construction work. Vertical organization firms permit realization economy of scale and decrease cost and prices, but the organization problem, problems of management and control, can increase transaction firm's cost and therefore “X-efficiency”. In the future, this problem defines disintegration process. The large company must differentiation and advantage in diverse work, but not in all together. The choice production differentiation is result specialization firm in only one type project for specific buyers and concrete localization. This specialization, as result of process markets segmentation define client’s profile, estimation competitors and divers firm’s offer for choice target market and product position. This strategy is expensive, but practice shows, that ensures higher effective and return/profit above average in market. Every success daily is function of analysis market, estimate behaviour clients, competitors and firm’s experience in chosen type project and possibility realization quality and better comparison the next competitor.

4. CONCLUSION

The success strategy includes the plans, programs, actions and reaction towards expectation change competitive condition and protect advantage. The future development the construction industry is a result of fundamental, structural change, but not only cyclical change. The change in competition, buyers, suppliers, investment banking shift all construction industry. The future development demand, technology, supply, vertical relations and government requirements for stable construction, behaviour and decision investors define new competition advantage for firms. The construction firms must use success combination of the assets and capability and decrease negative tendency and influence five forces:

Every firm can decrease market power buyer's trough active partners, established loyal clients, change behaviour towards characteristic product, not-price competition and can create more value for consumers. The firm can decrease market power suppliers with active and long run relation, differentiation contracts with more value. The blocking entry usually is a result of realize economies of scale and possibility supply lower price, make powerful brand, which identify and diverse firm and his product of really and potential competitors. The development innovation and patents are high barriers for new competitors.

The threat of substitute, every firm can lessen with higher switch cost for buyers, higher requirements for contractors and subcontractors and strong control over realization the every project. The success strategy, that ensure competitive advantage in cost lieder or differentiation product is consolidate firms position, minimization risk and maximization profit. The development horizontal and vertical mergers, strategic alliances create market power and decrease intensity in industry. In this way, the firms will stability more independence of change demand, technology and business cycle - peak and crisis. The key success is always analysis market and information about demand and change preferences consumers, action and reaction competitors apply strategy "tit for tat". The firm must envisage possible future industry scenarios and identify the alternative for development. This analyse trace the path firms: recognize distinctive capabilities that include intangible assets - secrets of value, established business network, brands, management skills, engineering competency, innovation which is not easy to copy or development now. As a result of success combination and use this capabilities provide unique capabilities and opportunity provide unique value and receive the gains from providing this unique value, that is necessary to maximise unique opportunities.

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Competitive strategy and competitive advantage in the civil engineering construction industry

Aneta Marichova

***Abstract* - The market (industry) structure in the construction is different from those found manufacturing sectors. The construction industry is traditionally characterized by a large number of small firms, with very few barriers to entry and dispersive market structure. The problem "firm's strategy and advantage" is very much problem for construction firm. Although the prospect for the construction industry are now very promising, many small and local firms have serious difficulty, such as poor performance and low competitiveness. There two reasons behind this problem: the environment is not favorable and the firms haven't strategy to improve advantage and performance. This research aim to analyze an able construction firms to identify market structure, evaluate the potential for profitability (with an industry analysis), define the firm's strategic position within the industry and its business practices, which ensure sustained competitive advantage (the firm's ability to outperform competitors in the industry).**

***Keywords* - Construction industry, market structure, oligopoly, strategy position, competitive strategy, sustained competitive advantage**

1. INTRODUCTION

In the construction, the market (industry) structure are different from those found manufacturing sectors. Manufacturing is mostly dominate by a concentration of very large companies that are able to utilize capital-intensive technology, produce very much production and realize economies of scale. In contrast, the construction industry is traditionally characterize by a large number of small firms, with very few barriers to entry, a disperse market structure and a relatively low level of fixed cost and variable profit, that influenced by business and consumer confidence, interest rates, government programs and cyclical economy activity.

The problem "firm's strategy and advantage" is very much problem for construction firm. Although the prospect for the construction industry are now very promising, many small and local firms have serious difficulty, such as poor performance and low competitiveness. There two reasons behind this problem: the environment is not favorable and the firms haven't strategy to improve advantage and performance. Although some large construction firms used strategy and realize competitive advantage. The important problem for construction firms is choice strategy that ensured competitive advantage. The little construction firm has strategy. The analysis shows, "they just did deals; they were relatively opportunistic. They could always make a case for why they should do a particular type of deal, even though they had never done a similar one before. So what is it that will allow you to outperform your industry? People didn't think in terms of competitive advantages that they could sustain" [1]. This behaviour is profitability in short run, but in long run, the firm lose your specific profile, image, market position. This strategy is non-success, when demand decrease. In next years, if the demand will begin increase, the firms must ready supply new quality, green product and use new technology.

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The firm’s aim is realization higher profit comparison average in market as a result competitive advantage. Therefore, what strategy will ensure competitive advantage?

Strategy is a word with many meanings and all of them are relevant for firms which established strategy their business. K. Andrews define strategy as the pattern of decisions in a company, that determines and reveals its objectives, purposes or goals, produces the principal, policies and plans for achieving those goals [2]. M. Porter defined competitive strategy, as a broad formula for haw a business is going to compete, what its goals should be and what policies will be needed to carry out those goals [3]. Bruce Henderson define relationship between strategy and competitive advantage. Competitive advantage is function of strategy, make different firms and ensure loyal consumers and stability market position [4]. Competitive advantage is what enables a business organize to thrive. It is objective of strategy and combination of elements in the business model, that enables a business to better satisfy the needs in its environment and ensure higher economic rent. In analysis the firm's strategy position must make difference between competitive advantage and sustained competitive advantage. A firm has competitive advantage when it is implementing a value creating strategy not implementing by any current or potential competitors. A firm has sustained competitive advantage it is implementing a value creating strategy, that not implementing by any current and potential competitors and other firms can't duplicate the benefits of this strategy.

The successful strategy of the firm is a result from both analysis - analysis industry structure and strategic position within the industry. Structural analysis define the level concentration which is importance structure variable for every market and explain different competition and profitability. The strategy is a set of objectives, policies, and plans that, taken together, define the scope of the enterprise. Every strategy can be combined firm's mission, aims, possibility, resources, and elements firm's environment - competitors, costumers, suppliers, contractors and subcontractors. Therefore, the industry structure defines competitive strategy and advantage. The strategy must ensure protection and/or growth market share, firm's position and its approach to survival and success in health and successful industry.

In many research construction strategy management and competitive advantage is simply define as organization performance. Tradition the strategy management has limited place and development in construction comparison other industry. Some large construction firms use strategy and realize competitive advantage as a result of competencies, reputation, innovation and firm's specific resources. Often the aim construction firms is only decrease transaction cost, or development human resources and quality education and stability financial position[5]. But the sustained competitive advantage is not connection with revenue and profit. Sustained competitive advantage must be a result of two factors: possibility firm's identification and analysis competitive forces and theirs change and possibility capabilities firm's, their successful combination, transforming assets, when change environment. Therefore, the construction strategy management must analysis the industrial specific factors - national construction industry, level of concentration and competition, barriers to entry, vertical relations, mergers, management skills and project specialization and competitive advantage defines as a function of social, market, technology and intangible assets and capabilities [6].

2. ANALYSIS MARKET STRUCTURE THAT TYPIFY THE CONSTRUCTION INDUSTRY

The every market structure is function of development demand and supply. General market conditions - demand and supply, faced by an industry are often important factors in the choice of conduct by firms and for the ability of firms to generate profits and meet expected performance goals. Traditionally market structure determination of three object indicators - number firms of sellers and number firms of buyers, differentiation product, barriers to entry and position firm in vertical relations and two market measures - Coefficient of concentration and Harfindal-Harshman Index [7]:

1) The number of sellers in an industry is important indicator because a small number of sellers with relatively large market shares may have market power. However, this market power may be offset, if the number of buyers are small compared to the number of sellers. A small number of buyers may be able to demand lower prices.

2) An important indicator for classifying market structure is whether the product is homogeneous, differentiated, or unique. A product is homogeneous if every firm in the industry sells exactly the same product. Homogeneous products can be found in perfect competition and pure oligopoly. A product is different, if a firm in a competitive industry can increase price without losing all of its sales (which means the individual firm faces a downward sloping demand curve). Product can differentiate on any element of the marketing mix (the 4 P's) which consists of price, product, promotion, and place. A product is differentiation, only if the consumer perceives and values the differentiating feature of the product. The degree of differentiation, and thus the individual firm's market power, depends upon the degree of substitutability for the product. Differentiation can achieve with advertising and promotion, service contracts and warranties, style, and many more ways. Advertising itself can be a barrier to entry.

3) Barriers to entry ensure competitive advantage for incumbent firms as a result asymmetry of information [8]. Barriers to entry includes: natural barriers (economies of scale, economies of scope, absolute cost advantages, capital costs, etc.), strategic barriers (actions taken by firms such as product differentiation and increasing the cost of entry), legal barriers (patents, licenses, laws and regulations, etc.) [9].

Market concentration measures are be use to classify how competitive an industry is. Concentration measures help us to understand how much market share is concentrated in the hands of a small number of firms. An industry characterized by low concentration will have a large number of firms with small market shares. An industry characterized by high concentration will have a small number of firms with relatively high market shares. Industries with high concentrations are more likely to have market power, i.e. the ability to set price. The market measures include:

1) Coefficient of concentration - C_4, C_8 and C_{10} that present the sum of the market share of top four, eight and ten largest firms in the total revenue construction industry.

2) The Harfindal-Hirschman Index of concentration (HHI) that present the sum of the squares of shares of total revenue of each particular firm in the total revenue of the industry.

The level of concentration evaluated on the basis values of coefficient of concentration (C_4, C_8, C_{10}) and the HH Index. The analysis the concentration construction market must according to two criteria: the level of concentration and shifts in concentration. The economy theory defines: if $0<CR_4<40$ and $0<HHI<1000$, the market structure must interpretation as effective competition or monopolistic competition and tendency development towards oligopoly. The effective monopolistic competition and oligopoly have increase potential for profit and advantage and define as workable competition [10]. Therefore, the increase the industry concentration in monopolistic competition, oligopoly and monopoly is important factor for increase the market power and profitability in every industry.

The important reason to classify industries as to market structure is that the number of firms in an industry plays a role in determining, whether firms explicitly take other firms action into account. In monopolistic competition, the multitude of firms makes it unlikely that they explicitly take into account rival firms' responses to their decisions. In oligopoly, with fewer firms, each firm explicitly engages in strategic decision - taking explicit account of a rival's expected response to a decision being made. Concentration measures limited, by how the researcher defines the relevant market because import competition is important, new competitors can enter a market and competitiveness has more than one dimension. Very often, inter-industry competition (indirect substitutes) is just as important is intra-industry competition (direct substitutes) [11]. However, the importance of analysis market structure is the understanding of the implications of the market structure for the competitive strategies of firms in the market. Competitive strategies must ensure to maximize the long-run profits of firms. Thus, an important question is whether the market structure will change over time. Oligopoly and monopoly market structures can sustained over time only if there are barriers to entry.

Construction market characterize with many number of sellers and relative small size, very little degree differentiation product, which is all characteristic construction in every country and low barriers for entry. The construction firms haven't activity vertical relations with suppliers. Many large firms have complicated structure as a result vertical integration. Characteristic three market indicators show model perfect competition in construction market, but analysis market measures change this picture. Analysis date construction industry in

EU-27 shows positive connection between two indicators of level concentration in period 2004-2011 year [12]. In last years, the coefficient of concentration and Harfindal-Harshman Index increase. The date and their change shows shift towards increasing market share lieder firms and level of concentration in market. This heightening of level concentration has combined in last years with a slowing the demand, besides the process realize more hurriedly comparison with general decreasing construction work and total revenue. Traditionally, the first five firms working in civil engineering segment and have large market share, which increasing always in this period. The firms, which working real estate and commercial building market segment have relative smaller market share. Therefore, market indicators show that in segment real estate and commercial building market structure is effective monopolistic competition and in civil engineering segment market structure is oligopoly, and therefore more market power of the firms and possibility for control and enforce higher price. Definition about effective monopolistic competition and oligopoly suppose increasing potential for profit and advantage. The important characteristic the competitive strategies of firms in an oligopoly is mutually inter dependent [13]. When a firm in an oligopoly chooses its strategy, it must take the potential reactions of its competitors into account. The outcome of a decision for one firm depends upon the decisions of its competitors. Thus, firms in an oligopoly are strategically interdependent. In this condition, the firms must choose strategy of strategic positioning or product positioning. A firm's strategic positioning delineates its boundaries, both vertical and horizontal. A firm's product positioning delineates the attributes that its product will offer. The firm should find a fit position in their market from which they can best defend competitive forces or influence them and realize competitive advantage - cost leader or differentiation.

3. COMPETITIVE STRATEGY AND COMPETITIVE ADVANTAGE IN CIVIL ENGINEERING CONSTRUCTION INDUSTRY

In civil engineering construction market work the large construction firms with specialized assets and finish product, high fixed cost, higher the exit barriers and barriers to entry, that as a result of using high technology, managerial skills, human and capital recourse, economies of scale. These factors define the higher profitability on the whole industry comparison average standard. The important strategic barriers to entry new competitors are reputation, history, tradition and corporate image the firms.

In this condition, in segment civil engineering with oligopoly successful firm’s strategy is strategy position. The strategy position can change horizontal and vertical boundaries and every firm can change his market share with realization economy of scale and/or strategy and structure barriers to entry. In the oligopoly, each firm has enough power to avoid being a price taker, but they are still subject to a sufficient amount of competition to know the market is not under their control. Therefore, firms in oligopoly markets will price their product according to how they think competitors will react. This defines dilemma of not knowing, whether to compete or co-operate and uncertainty oligopoly. If firms co-operate as a group, they will make more profit, but if one firm deviates from agreement and become aggressive competitive, it will make more profit too. When firms agree to co-operate to raise profit it is collusion. Collusion is common practice in construction industry. Many construction firms follow some form of oligopoly behavior. This cooperative strategy include actions that competitors take together in order to reassure each other that they are not cheating on a cooperatively agreed-to price. The extreme of cooperative strategic behaviour is a cartel. In a cartel, firms explicitly coordinate their decisions. The agreements and collusions however are not necessarily formal. The practice shows two types of agreements: "explicit agreements" and "quasi agreements"[14]. Explicit agreement is covert and secret collusion, but quasi, implicit agreements we can describe as a "spontaneous co-ordination". All firms should take the price determination from the most dominant firm in the market. Implicit cooperative behaviour is now more commonly referred to as tacit collusion and usually not illegal. In this case, a firm simply observes the actions of its competitors and makes its own decisions accordingly. The basic aim this decision is estimate the relative strength of two forces - compete with rivals to gain as large a share of the possibility potential profit or co-operate with rivals to maximization joint profit. Quasi agreements and to joint profit maximization behavior are common practice in a specific civil engineering construction market. The two forces, which concurrent operate are diverse in diverse industry are result of influence the following five market characteristics:

1) In this market works very few big firms. Every firm understands that one of them cannot gain sales and profit follow retaliation. Therefore, some agreement to co-ordination may be reach all policy.

2) The construction firms produce similar products and as a result for firms is difficult gain specific advantage in the market. In particular, the elements of each firm marketing mix will look very much the same. Products will be priced similarly, promotional expenditures will be similar, product development will appear to be on a parallel track between firms, and firms will distribute their products through similar channels. In this condition, the firms prefer some form of tacit collusion.

3) The civil engineering construction market usually has one or two dominant firms. However, this fact is not mark dominance position and no-loyalty behavior leaders. In this situation, the collusion and strategy union - open or secret, ensure possibility to win auction and realization the object, which increase the public benefit. The other small firms have only successful strategy - "follow leader". The aim is stability industry as whole and market position leader and blocking entry new rivals. Therefore is very high strategy barrier in this market.

4) The construction firms have similar average cost and this exclude price competition. The competition can realize on other forms co-ordination and tacit collusions - agreed to higher price and joint profit maximization.

5) The new competitors face significant barriers to entry. In theory, the high profit is the attractive for new competitors and as a result increasing supply and reduced the prices and profit. However, this cannot realize in the construction market. Economies of scale, huge capital, high investment and specialized inputs are usually required to enter new firms in civil engineering market. Established firms may have loyal subcontractors, customers and this is very high barrier for new firms. A few old-firms may own or control the supply of raw materials required in the production and often the public sector may operate with only a few firms in this market. The further barrier to entry provided by limited pricing and a price-low discourage entry into the industry. These actions sacrifice short run profit, but to maximize long run profits and existing firm's position.

The leader firms can block the entry in this profitable market. The leader firm is ordinary the firm with lower cost and as a result it is define lower price and realize higher revenue and profit. The strategy position of leader ensures increasing return to scale and realize lower average cost in user technology. Therefore, the firm must realize advantage in differentiation cost. In this situation, the other small firms, which can use only successful policy "follow leader" difficult survival.

Often the collusions between construction firms are about rigging activities and cover rigged prices. Cover pricing is practice where one contractor asks another contractor of the same tender a price, which will be above this tender by the interested. As a result the tendering contractor impression of the level competition and inflated the price for construction work. Often the contractors agree and decide who will win a contract and what at price. In other cases, the firms may "pay off" those, who agree not to tender a bid, but ensure similar favors for future contracts. In practice the construction firms often use suspicious bidding, with which in advance define winner in auction, viz. [15]:

1) The firm's offer are very same or unusual different.

2) One firm participant in auction discussions with other, demands information or announces secret agreement.

3) The firm's offer contains less detail than expected.

4) The leader firm decides no-participate and the auction wins the next firm.

5) The offers separate firms are very different comparison requirement auction.

6) The firms offer suspicious bids without connection production cost.

7) The winner offer is very low and after the winner firm declines to sign agreement.

8) The successful firm often concludes higher bidder with suppliers and subcontractors that is basic reason for actualization the main agreement. In next period the winner firm beginning actualization agreement and increasing the price.

9) One firm wins with very high offer and this offer is unique. The winner firm has in advance bides with other firms. In this case the participants play as a cartel and drops on the entry of a new or infrequent bidder. In a result the construction firms ensure higher profit and big market share.

In construction has low barriers to entry and these implicit cartels or co-ordination between firms is usually short duration. Some authors explain this aggressive policy of construction firms as a result the next specific construction industry: Construction firms produce identical service and operate in a market that offers relative low profit margins. Therefore, the firms respond with agree to price, condition bids and participation in informal agreements to manage their work load efficiently. The comparison with the companies in other industry shows their possibility manage throughput by adjusting their level of output, whereas construction firms on tender lists have to meet client's requests. In other industry, the firms work at full capacity and realize high profit, but a construction firms operate at near full capacity it might tender a cover price simply to remain on a list of recognized contractors for future projects. The firm's aim in other market realize of manager's decides, but in construction market the firm's aim realize of legal or illegal agreement and higher auction price. Therefore, in construction market the implicit cartel ensure stability and gain profit, which cover the increasing cost [16]. The general characteristic construction market is many thousands of small competitive firms and big fragmentariness by regions, market segments and preference buyers, but in many of these markets have a local monopoly or a local oligopoly with price leader. This is reasons in the construction industry, the firms allege in some form of the agreements.

The analysis of construction market concludes that this market is contestable and that high profit and market share are unlikely to persist [17]. In the last years, the contestability of construction market becomes questionable. At the bottom end of the market, registration schemes have made it more difficult for construction firm to enter the industry, at the top end an increasing trend for partners arrangements has also reduced the number of firms able to compete. Therefore, as barriers to entry emerge, competition level is becoming less apparent in construction industry. The government is important subject who must ensure sustainable development, competition in free market and resource efficiency. Competition policy attempts to restrict unethical business behavior that acts against the public interest. The government, therefore prefer that firms do not make agreements to restrict competition and control prices.

The public sector is a major player in the construction market and local construction firms and government departments are therefore encourage of the benefits of competitive tendering. The general aim is ensure optimal combination of using resources, use effective technology and increase social benefit [18]. In practice, the competition in the supply chain defines as a healthy, but the agreements within the supply chain to restrict competition and control price is unhealthy. Often, the public sector encourages tacit agreements and collusions. The actions firms, which aim is increase horizontal and vertical boundaries and as a result market share is support of restrictive acts of governments sector, who is major investor and organization auction public offer. The public investor defines discriminating and subjective requirements for firms-participant in auction and chooses always "ours firm". In construction market, has enough cases, which evidences restriction, aggressive policy of oligopoly firms, but and restrictive policy of public sector. Because of all this, increasing and stability the monopoly position one-two firms in construction market. In crisis and reduce the construction work as a whole, the competition between construction firms for gain auction is very strong and therefore no-legal practice is very mach too. The secret or tacit, quasi or implicit cartel in construction market support of public sector ensure maximization profit and market share for firms, but this policy is not stability in long time. The dominant firm's strategy is break requirements and agreement. As a result, the next step is revenge and price war. Therefore, the law of public offer is non-effective and its time is over. The practice in number country shows that the market defects and problems can decide with development Public Private Partners (PPP). One main reason for development PPP is regional development Central and East Europe (CEE) and Europe market.

The PPP ensure necessary funding, investment for public infrastructure in period, when the government and public sector budgets have very scare resources [19]. The budgets are very limited, which enforce more rational and effective using public resources. As a result the government must reorientation of policy "gain actives" to policy" get services". The PPP is long run agreements between government and private sector for funding, management, construction and reconstruction the big infrastructure objects, that have very big social significance. The PPP ensure more effective, minimization the cost, the risk and high quality finish object. The successful PPP is possibility for the society to get more value added of public resources that - value of money. This can realize trough full, strict control and comparison analysis cost and benefit of project. The government must make estimate, which include quality and quantity analysis potential project and this estimate definable Net Present Value. The Net Present Value comparison with standard estimates definable Comparison Public Cost. Comparison Public Cost includes all cost and risk, if the object realize of government. If the Net Present Value is bigger from Comparison Public Cost, the object must realize of private sector. The difference between Net Present Value and Comparison Public Cost define Value of money or value added, which will can to get the society. PPP are effectively because use private experience, effective scare resources and short period for realization idea. The realize value add of money is a result of innovation and optimal resources combination trough whole project cycle.

The main problem in organization PPP is whole approach to process, which includes project, building, to invest, exploitation and risk. This mark that the public sector participate in all process - construction, building, funding, exploit the finish infrastructure object and take on the risk together with private sector. The government take decision as a result of comparison no only value of building, but total cost (explicit and implicit cost) of realization object and the service.

The government's pays begin after the private partners supply finish product, therefore the government have not financial liability in stage construction and building. The private partners take on the all risk of funding and realization object, but the public partner (government) take on the risk of ensure demand service and necessary revenue that can cover the cost investors. The payments of public sector can to be:

1) Direct payments - this are taxis, paid only of consumers.

2) The secret taxis, which paid of all members in the country.

3) The government payments for all period concessions.

For create and development PPP in CEE must realize the next steps:

1) The government must establish the laws, which regulation the function PPP. The government must work requirement and standard for start PPP.

2) The government must create mechanism, which strictly define and estimation Net Present Value and Comparison Public Cost.

3) Development work mechanism for ensure the government's payments.

4) Define possibility the government can take on a risk, where the demand service is difference of expectation and ensure necessary payments for private partners, independent of lower demand or back.

5) The development PPP requires active participation banks and other financial institutions.

6) Stimulate private sector to integration and active participation in successful PPP as a result of combination innovation, private experience, business sense with increase need of modern, effective infrastructure and other public goods, which ensure higher private profit, but and higher profit, benefit for society.

In imperfect market condition and necessary governments regulations, the success strategy for construction firm is a result of effective organization all process - architects and constructors, investors, executive firms, subcontractors, clients, relation suppliers, control performance object, realize economies of scale as a result increasing productivity and stability financial revenue, development and use innovation methods in construction, effective vertical relations. The result of this ways is increasing productivity in each firm and long run sustained competitive advantage.

4. CONCLUSION

The construction market analysis shows that in segment real estate and commercial building market structure is effective monopolistic competition and in civil engineering segment market structure is oligopoly, and therefore more market power of the firms and possibility for control and enforce higher price. In this condition, the firms must choose strategy of strategic positioning or product positioning. In segment civil engineering with oligopoly successful firm’s strategy is strategy position. The strategy position can change horizontal and vertical boundaries and every firm can change his market share with realization economies of scale and/or structural and strategic barriers to entry. The important characteristic the competitive strategies of firms in an oligopoly is mutually inter dependent. When a firm in an oligopoly chooses its strategy, it must take the potential reactions of its competitors into account. The outcome of a decision for one firm depends upon the decisions of its competitors. Thus, firms in an oligopoly are strategically interdependent. Because of that, collusion is common practice in construction industry. Many construction firms follow some form of oligopoly behavior. This cooperative strategy include actions that competitors take together in order to reassure each other that they are not cheating on a cooperatively agreed-to price.

In oligopoly market structure, the leader firms can block the entry in this profitable market. The leader firm is ordinary the firm with lower cost and as a result it is define lower price and realize higher revenue and profit. Strategic position leader firm ensure increasing return to scale and realize lower average cost in user technology. Therefore, the firm must realize advantage in differentiation cost. In this situation, the other small firms, which can use only successful policy "follow leader" difficult survival.

Collusive agreements and oligopoly civil engineering market lead to higher level of profit. The higher profit associated with imperfect market and government is keen to foster the resource efficiency associated with competitive market. Resource efficiency is particularly relevant for government a sustainable development. Therefore, the governments prefer that firms do not made arrangement to restrict competition and control prices. The construction industry and competition in this market needs to be carefully monitoring and effective competition policy. Important, the competition policy prevents the development anti-competitive behaviour and ensure to achieving high production and technology level, decrease welfare losses and increase economy growth.

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Technological Damage of Concrete Reinforced by Polypropylene Fiber

N. V. Pushkar, Hassein Juhad Salman Al-Amery, Sabir Yousif Bakir

***Abstract* – This article researches the influence of polypropylene fiber on the formation of technologic damage of fiber-reinforced concrete and its deformation properties. Products of different shapes were prepared of normal-weight concrete and of fiber-reinforced concrete. In determining of the extent of their technologic damage, attention was paid to the grid of surface cracks. Areas were selected on the samples within which the length of the cracks was measured. Then, the technologic damage coefficients were defined for the conventional concrete and for the fiber-reinforced concrete. The comparison showed that the cubes and arches made of fiber-reinforced concrete have coefficients greater than similar products made of conventional concrete; the prisms have a bit lower coefficients. When comparing the relative compression deformation, it was established that fiber-reinforced concrete having greater technologic damage in the form of cracks is more deformable than normal-weight concrete.**

Keywords* – *Concrete, cracks, damage, fiber.

1. INTRODUCTION

From the ancient times, it is known that if you add straw, cane or sheep wool to the building clay then the wall will become more durable and will have fewer cracks. So the adobe was created – a distant predecessor of the modern fiber reinforced concrete. Fiber reinforced concrete is still a new, but, without any doubt, very promising material.

Fiber reinforced concrete is a variety of fine grain concrete with addition of fiber. Glass, synthetic or steel fibers 5 to 150 mm long and about 0.2 mm to 1.0 mm in diameter may be used as fiber [1], [2]. As a result, we get fiber reinforcement, which gives the concrete several unique properties in comparison with ordinary concrete: high impact strength, tensile and shear strength, as well as freeze-thaw durability and waterproofing, which allows separating it into an independent and very valuable group of construction materials with their own unique structure and properties. Polypropylene fiber improves concrete's specifications during the initial period of concrete strength development, while steel does it after concrete strength development, i.e. it has a forcing function.

Fiber reinforced concrete, just as ordinary concrete, is a composite material, i.e. it consists of two material and has properties which the original materials lack: cement-concrete matrix with even distribution of oriented or randomly located fibers of different origin.

Inside fiber reinforced concrete, fibers take the tensile stress, which increases its bending strength and compression resistance. Polypropylene fiber has a perfect hydration, it controls the even distribution of water in the concrete structure. Thus, the internal load is reduced and, as a result, the crack resistance and the impact strength concrete are increased [3].

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Polypropylene fiber is resistant to alkalies and the majority of chemical agents, which makes it a chemical resistant material; which increases the durability of concrete surface; which increases concrete's waterproofing – thanks to blocking of concrete's capillars by fibers, due to which corrosion of steel fittings is reduced and resistance of concrete to low temperatures is increased; upon destruction of concrete under load, the fragments are not separated, but remain interconnected by polypropylene fiber. Fiber reinforces corners and ends of concrete structures, thus, avoiding spalls, reduces shrinkage and, accordingly, formation of cracks in the course of the first hours of concrete's hardening, reduces the probability of damaging the structures when dismantling forms.

2. Experiment Description

According to [4], [5], one of the advantages of concrete reinforced with polypropylene fiber is reduction of microplastic shrinkage and formation of cracks in the process of concrete's hardening. This advantage is very important, as the initial (technological) cracks arising during the concrete's hardening and disturbing the structural integrity, affect the structure's further work [6]. In view of the above, we define the purpose of the research as studying of polypropylene fiber's effect to the formation of concrete's technological damage.

In order to achieve the purpose set up, samples of 2 series were made in the lab of Reinforced Concrete and Stone Structures Department under Odessa State Academy of Building and Architecture. Samples of series A made of ordinary heavy concrete included 25 cubes with an edge of 10 cm long, 15 prisms sized 10×10×40 cm and 6 reinforced concrete arches with a constant rectangular cross section sized b×h=5×7 cm, with a span of L=210 cm, with a rise of f =42 cm. Samples of series B made of concrete with addition of polypropylene fiber included 24 cubes, 15 prisms and 5 arches of similar sizes.

For the production of concrete, granite crushed stone of 5…10 mm grading was used as coarse aggregate and river sand of 1.8 fineness modulus and Mark 400 cement produced by Odessa Cement Plant were used as fine aggregate. Composition of concrete per 1 m3: crushed stone – 1200 kg, sand – 600 kg, cement – 320 kg, water – 160 l. In the process of production of the samples of series B, polypropylene fiber (previously soaked in water for 10 min.) was added in the amount of 900 g per 1 m^3: of concrete to the concrete mix after all the components had been added, 5 min. prior to the end of mixing.

During our research of technological damage of the test samples, we paid our attention to the grid of surface cracks appearing in the process of concrete hardening. In order to assess the technological damage better, the development of cracks was made after the samples were 300 ... 320 days old. Water solutions of tannin [7] (**Fig. 1**) were used for the development of cracks. All test samples were kept in the solution for 20 ... 30 minutes; after the soaking they were dried in the lab for two days.

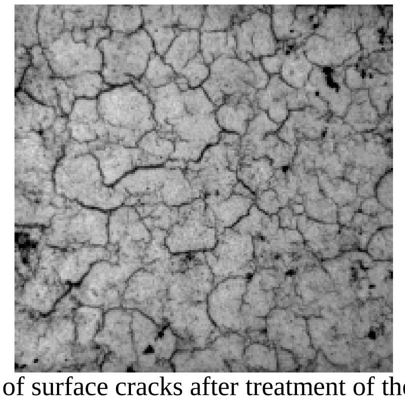

Fig. 1. Characteristic pattern of surface cracks after treatment of the sample with tannin solution

In order to determine damage coefficients of the test samples, the technique proposed by V. S. Dorofeyev and V. N. Vyrovoy was used.

This technique is based on measuring the lengths of the developed surface cracks using curvimeter and determining the damage coefficient as a ratio of the total length of surface cracks L to the sample area S on which the measurements were made: C_{AD} (coefficient of areal damage) = L/S (cm/cm^2) (**Fig. 2**). The physical meaning of this coefficient is the assessment of a crack's specific length per surface unit [7].

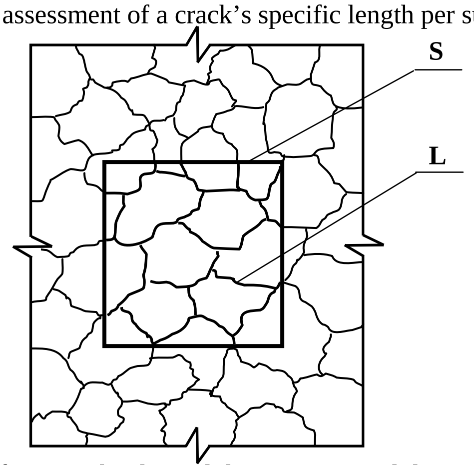

Fig. 2. Defining technological damage via areal damage coefficient

In this research, the assessment of technological damage was made according to the described technique, however the length of the cracks was not measured with curvimeter, but using AutoCAD [8]. Reference [8] showed that using AutoCAD for the measurement of surface technological cracks allows: eliminating the human factor during the work with curvimeter, getting more reliable data on technological damage, saving images with an ability to double check the data, if necessary.

The technique of measurement of the lengths of technological crack with AutoCAD was as follows. The selected sections with cracks on the labeled test samples were shot using Nikon D90 professional camera, with a maximum frame size of 4288×2848 pixels, then the images were imported into the AutoCAD graphical environment and scaled to the full size. Then we measured the actual projection of the cracks' length in the horizontal plane by repeating the visible contour of cracks using the polyline tool. The obtained redundant contours grid was divided into straight sections and was added to the calculation of the total length of the lines using a command set up by an additional AutoLISP application written in LISP programming language, so the length of the surface cracks L was determined accurate to 0.01 mm. The zoom-in area of the image was previously limited, which allowed us comparing the results objectively.

For the cubes, the cracks length L was measured on the sections sized: S = 5 × 5 cm^2, for the prisms: S = =10 × 10 cm^2, for the arches: S = 7 × 7 cm^2.

3. Results and significances

Average values of the obtained damage coefficients for samples made of ordinary heavy concrete and concrete with addition of polypropylene fiber are shown in **Table 1**.

Table. 1. Average values of technological damage coefficients of the test samples.

Type of concrete	Average damage coefficients C_{AD}, cm/cm^2		
	cubes	prisms	arches
Heavy-weight concrete	4.66	3.06	3.86
Heavy-weight concrete, reinforced with polypropylene fiber	5.23	2.93	4.34

Having taken as a benchmark the arithmetic mean value of the damage coefficients of heavy concrete samples, we get the following results: formation of surface cracks in the fiber reinforced concrete cubes increased by 12.2%; in the prisms - decreases by 4%; in the arches - increases by 12.4% [9].

Now we will give you a review of deforming properties of heavy-weight concrete and fiber reinforced concrete. As similar values of damage coefficients of these materials were obtained in the prisms, we will examine deformation of the prisms upon compression. The prisms were tested according to the standard method with phased loading by $0.1F_u$, steps; deformations in the process of loading were measured by a dial gauge graduated in 0.001 mm and a 200 mm base fixed on two opposite faces of the prism.

After the processing of the indicators data, the values of deformation of the prisms at each stage of loading were obtained. Concrete, as an elastoplastic material, has curvilinear shape diagrams. In general, the nature of deformation of the prisms made of concrete and fiber reinforced concrete is the same: near-linear relation $\sigma_c - \varepsilon_c$ with stresses up to $0.3f_{cm, prism}$ (elastic concrete behavior) and curvilinear relation - with stresses above $0.3f_{cm, prism}$ (elastoplastic concrete behavior).

For a more detailed study of compressive deformation of the prisms made of concrete and fiber reinforced concrete, depending on their grade of damage, 3 stress levels were selected: I – upon loads of 6 MPa ($\sigma < 0.3f_{cm, prism}$), II – 18 MPa ($\sigma < 0.5f_{cm, prism}$) and III – 30 MPa ($\sigma < 0.8f_{cm, prism}$). For each of these levels corresponding deformations were selected.

When compared to ordinary heavy-weight concrete in prisms made of heavy-weight concrete with addition of polypropylene fiber, the values of relative compressive deformations are bigger than the ones of ordinary heavy-weight concrete: with stress level I - by 20%, with stress level II – by 15%, with stress level III – by 11%.

4. CONCLUSIONS

This article researches the influence of polypropylene fiberglass on the formation of technological damaged concrete. It presence in the structure of concrete increases the quantity of surface technological cracks in the cubes and arches on average by 12%, in the prisms produced a slight decrease in the technological damages by 4%. Surface cracks are a reflection of internal damages in concrete structure. Since the cracks are structures defect, materials with a large quantity of cracks will be more deform, which was obtained by comparing the relative of compression deformation in normal-weight concrete and the concrete reinforced by polypropylene-fiber. Despite of the fiber-concrete has several advantages over normal-weight concrete if compared, and its scope is very wide, therefore it’s necessary to further study the properties of fiber-reinforced concrete for samples in other shapes and sizes (dimensions), as well as the volume of concrete has influences in formation of technological damages.

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Cement Mortars Based on Sand Partially Replaced by Waste Ceramic Fume

Katzer J. and Domski J.

Abstract – **This paper presents the research programme focused on utilising waste ceramic fume in production of cement mortars. Waste sand (a by-product of hydroclassification process) of natural origin was harnessed as a main type of aggregate. The aggregate was partially replaced by waste ceramic fume. The ceramic fume was obtained as a by-product during production of coarse "post demolition" aggregate. Properties of both fresh mixes (consistency) and hardened mortars (density, compressive strength and flexural strength) were tested and analysed. The achieved results prove that mortars in question can be used to fabricate elements characterized by less demanding mechanical characteristics.**

Keywords – **aggregate replacement, ceramic fume, composite, fine aggregate, mortar, waste.**

1. Introduction

Cement based composites are the most popular building materials in the world [14]. Fine and coarse aggregates constitute around 70% of each cement composite volume. Global annual production of ordinary concrete and other cement based materials is growing every year and reaching 7 billion cubic meters. It means that about 3 tonnes of aggregates are used per person every year. This mass scale production of cement based composites creates multiple environment problems. One of the largest environmental issues associated with concrete production is growing difficulty to obtain natural aggregates in sustainable way. This situation led cement composite manufacturers to search for feasible alternatives, such as harnessing waste aggregate. So far there were performed some quite successful attempts to employ coarse waste aggregate for concrete production [6, 15, 16, 20]. Using waste coarse aggregate (usually obtained through crushing and milling concrete and ceramic debris), addresses multiple ecological and technological issues associated with disposing the waste, recycling of construction and demolition waste and conserving available resources of natural aggregate. The only disadvantage of this process is waste fume obtained during crushing debris [13]. The Authors decided to find ecologically efficient [17] and sustainable solution for utilizing the waste fume in cement composite production. This paper presents the results of a research study where the technological viability of creating cement mortar based on natural sand partially replaced by waste ceramic fume was analysed. The Authors decided to conduct mortars tests using ordinary procedures described by European codes and standards. This approach would enable to compare the performance of tested non-conventional composites with traditional mortars (utilized by construction industry on mass scale on daily basis). The main aim of the research programme was to prove

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technical and practical civil engineering serviceability of tested mortars and assess their potential as ordinary mortars' substitute.

2. USED MATERIALS AND MIX DESIGN

Ceramic and concrete debris obtained from the demolition of degraded structures keeps on growing in Europe with an increasing trend. Only in 15 European countries, the amount of construction and demolition waste, produced annually, is equal to around 180 million tonnes [6, 15, 16, 20]. Although the reutilization of ceramic debris has been practiced, the amount of waste reused this way is still negligible and in majority of cases incorporates only coarse fractions. For the purposes of this research study ceramic fume (CF) obtained as a by-product during manufacturing of coarse "post demolition" aggregate was utilized. Sieve analysis of the fume in question was conducted. A full grading characteristic of the fume is presented in **Fig.1**. This fume was used to partially replace post-glacial sand of hydroclassification origin. This sand was thoroughly tested in previous research programmes and was described in detail in numerous publications [4,8,9,10,11]. As a binder Portland cement CEM II/B-V 42.5N (conformable EN 197-1:2011) was used in all prepared mortar mixtures. Tap water (conformable EN 1008:2002) constituted the last main component of all mixes in question. All mixes were modified by a superlasticizer containing silica fume (Betocrete-406 FM). Admixture dosage was equal to 1.1%. The influence of the superplasticizer on properties of different fine aggregate cement composites was thoroughly described in numerous previous publications [8,9,10,11].

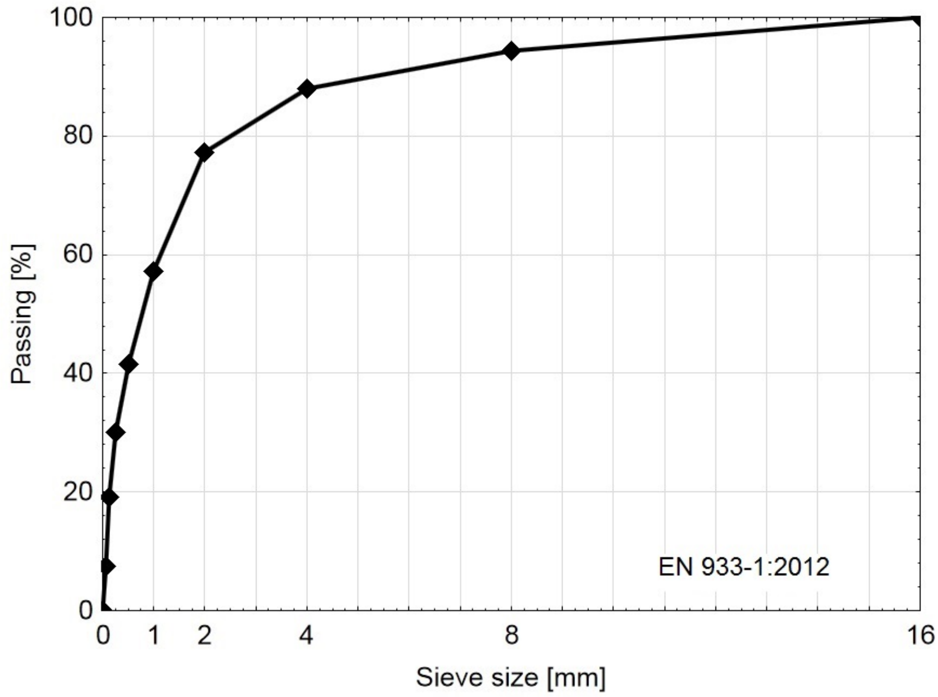


Fig. 1. Grading curve of used, waste ceramic fume

3. RESEARCH PROGRAMME

Three groups of mortars were tested and analysed during the research programme. The groups of mortars were characterized by different *w/c* ratio (ranging from 0.50 to 0.60) and different amount of sand replaced by

CF (from 0% to 50% by volume). The mix composition of the three basic mortars with no CF are presented in **Table 1.**

Table. 1. Mortar mix composition

w/c	Cement [g]	Water [g]	Aggregate [g]	Admixture [g]
0.50	450	225.0	1350	5
0.55	450	247.5	1290	5
0.60	450	270.0	1230	5

At the beginning of the research programme the consistency of fresh mortars and density of hardened mortars (ρ) were tested. During the second part of the study the compressive (f_c) and flexural (f_f) strength were tested. The compressive test was conducted on the surface of 40mm·40mm=1600mm^2. The flexural test was conducted on freely supported prisms (span 100mm) and loaded in the middle with single force. The results of the tests were statistically processed, and values bearing the gross error were assessed on the basis of Grabbs criterion. A table of random numbers was employed to choose the sequence of the realization of specific experiments to assure the objectivity of the experiments. All mathematical relations were established with the help of STATISTICA software. Achieved results were presented with the help of bubble charts with prediction intervals (probability 95%). Bubble charts are a variation of the scatter plot where the data points are basically replaced with bubbles. Such an approach to data analysis enables a comparison of the entities displayed on a bubble chart in terms of their size as well as their relative positions with respect to each numeric axis. In the discussed research programme horizontal axis represents the volume of fine aggregate exchanged by CF and vertical axis represents tested physical or mechanical property (e.g. density or strength). The position of the plot is an indicator of those two distinct numeric values and the area of the plot depends on the magnitude of the third numeric feature which in this case is cardinality of results population. This approach to the analysis of properties of mortars was successfully utilized in the previous research programme dealing with very fine ceramic fume [8],[9],[10],[11].

All mortar mixes in question were prepared with the help of a standard mortar mixer and standard mixing procedure (slow speed mixing - 140 rpm for the first 30s, then high speed mixing - 285 rpm). Specimens were in a form of prisms 40mm·40mm·160mm. There were prepared 9 specimens out of each mortar mix. Curing procedure was performed in two stages. The first stage of curing was to keep the specimens in their moulds covered with polyethylene sheets for the initial 24h (the specimens were then removed from their moulds). The second stage of curing was realized by storing specimens in a water tank (temperature of water 21^{o}C ± 1 ^{o}C). The mixing/casting sequence and curing procedure of tested mortars were described in detail in the previous publication [9] where fine CF was harnessed to partially replace cement [21].

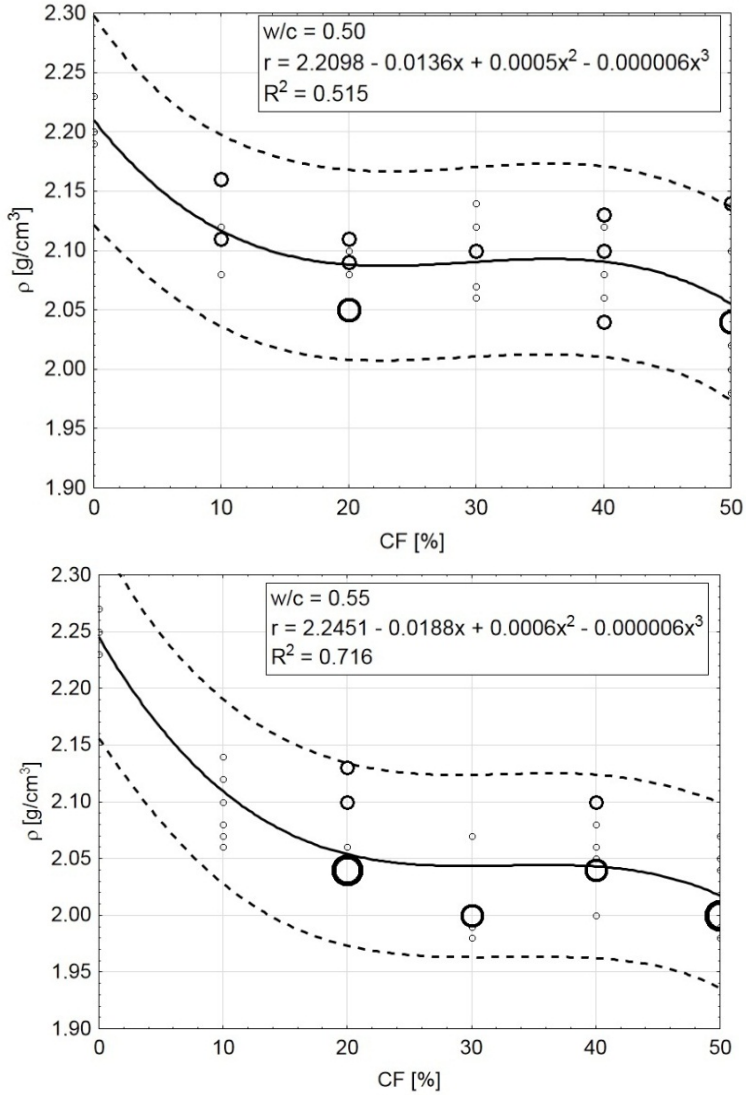
4. Results and Discussion

The results of the first stage of the research programme are presented in Tab.2. and Fig.2. In Tab.2 the results of consistency of tested mortars are summarized. The test was conducted using a flow table according to EN 1015-3:1999. There are three different charts in Fig.2. (each corresponding to mortars with a different *w/c* ratio) showing density of cement mortars.

Table. 2. Consistency of tested mortars *d* [mm]

w/c	CF [%]					
	0	10	20	30	40	50
0.50	185	170	150	150	130	105
0.55	165	210	210	210	200	185
0.60	200	220	235	230	225	195

While analysing Fig.2. one should keep in mind that ceramic is characterized by density around 1.8 g/cm^3 and density of used natural aggregate is equal to 2.65 g/cm^3. The more sand is replaced by CF, the smaller overall density of the mortar. All three relations are cubic and characterized by very reasonable correlations (R^2 > 0.50). The best correlation (R^2 = 0.716) was achieved for mortars characterized by w/c = 0.55. Density varies from 2.25 g/cm^3 (for mortar without CF and *w/c*=0.55) to 2.02 g/cm^3 (for mortar with 50% CF and *w/c*=0.55).



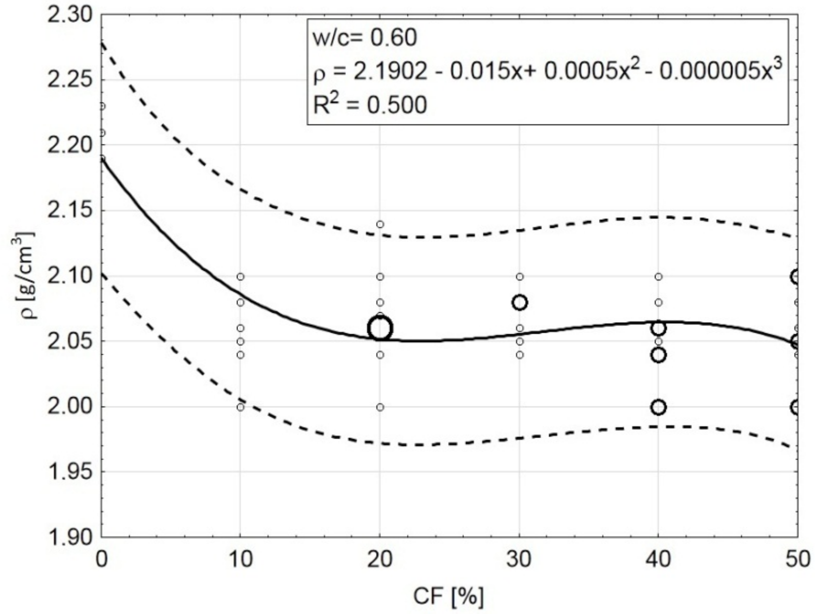
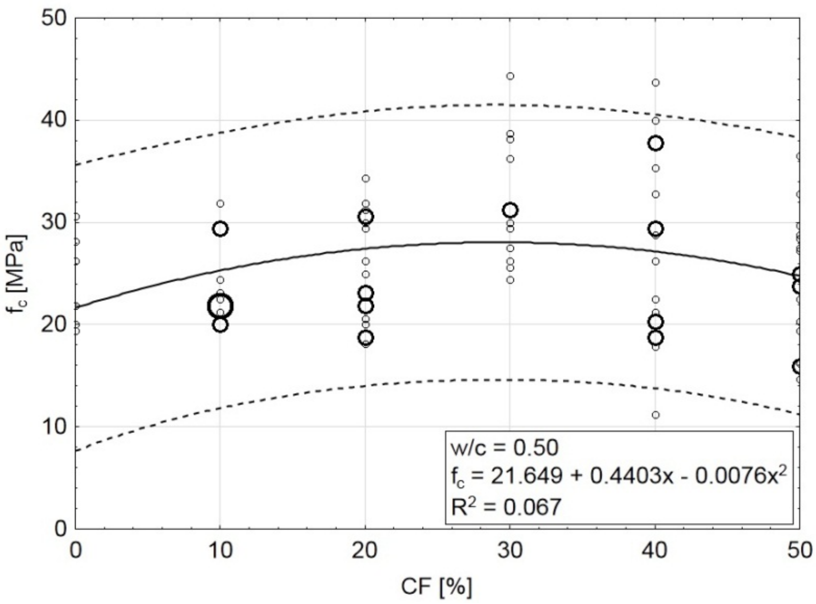


Fig.2. Density of tested mortars



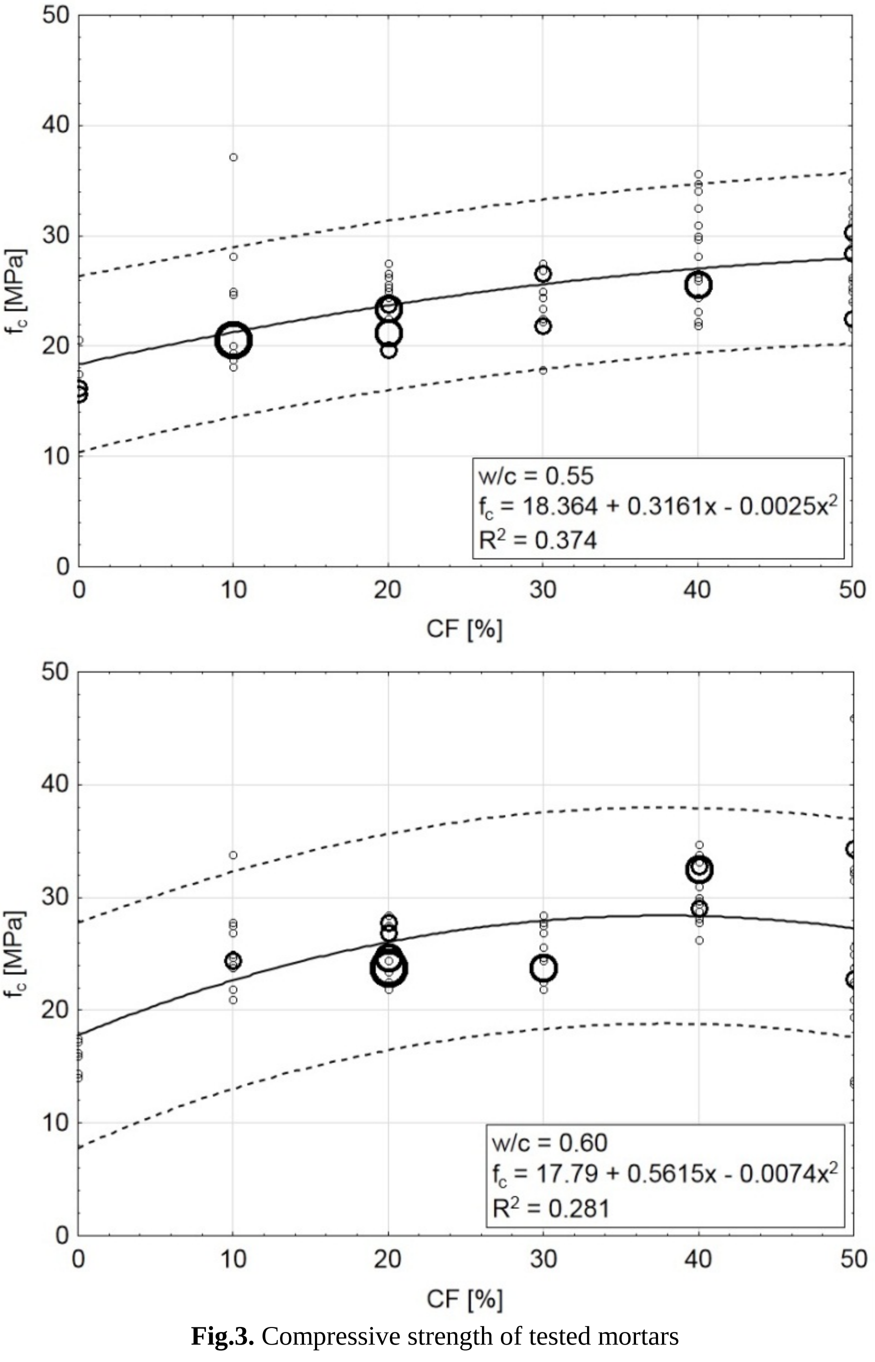
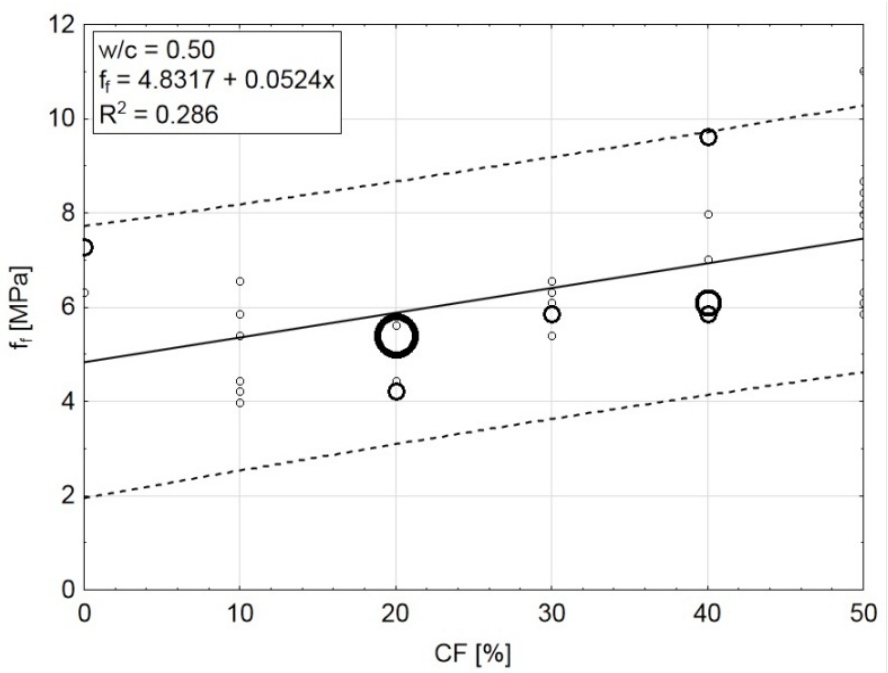
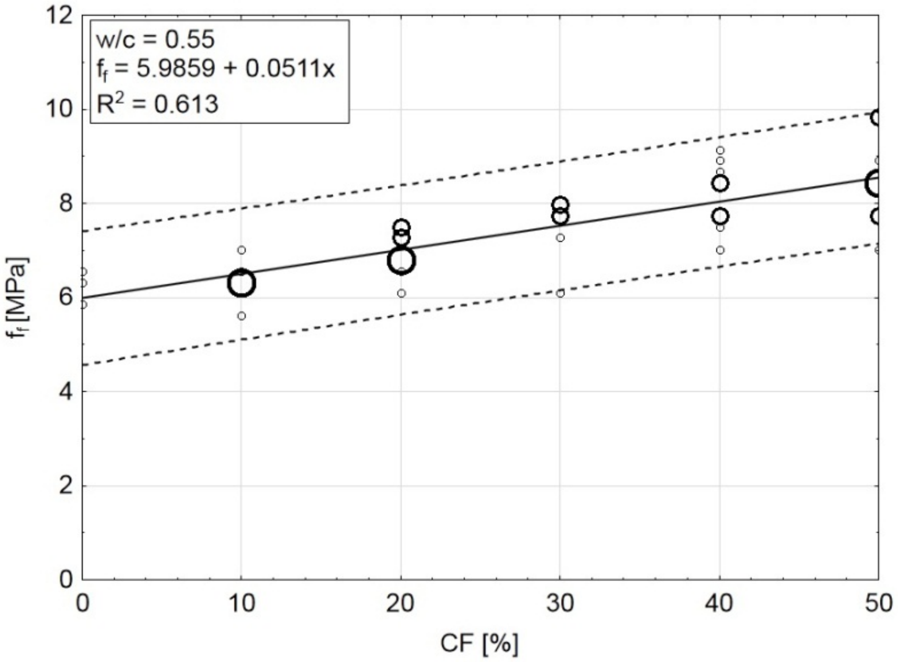


Fig.3. Compressive strength of tested mortars


w/c = 0.50
f_f = 4.8317 + 0.0524x
R^2 = 0.286
f_f [MPa]
CF [%]


w/c = 0.55
f_f = 5.9859 + 0.0511x
R^2 = 0.613
f_f [MPa]
CF [%]

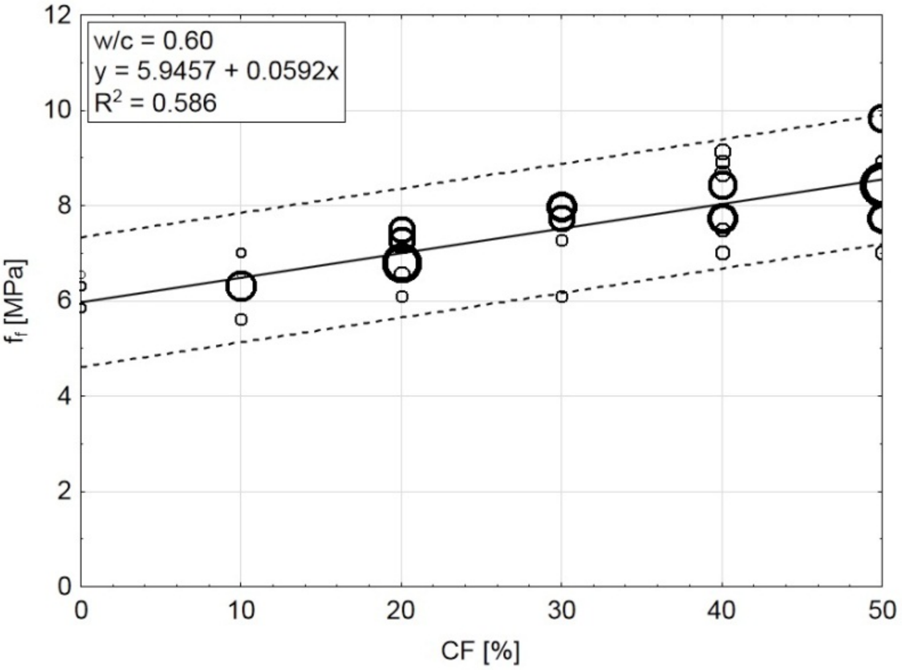


Fig.4. Flexural strength of tested mortars

The results of the second stage of the research programme are presented in Fig. 3 and 4. Results of tests of compressive strength are presented in Fig.3 in a form of three different bubble charts. The figure shows relations of compressive strength for varied *w/c* ratio. All relations are polynomial (quadratic). In general compressive strength of tested mortars was increasing (for all considered *w/c*) alongside the increasing volume of sand replaced by CF. The “worst” results were achieved for mortars with w/c = 0.50. In this case the prediction interval is the widest one, the correlation of the fitted mathematical function is weak (R^2 = 0.067) and prediction interval is very wide (around ±14MPa). Moreover, the increase of compressive strength is achieved only for CF addition from 0% to 30%. Results for mortars characterized by *w/c* = 0.60 are described by a similar relation. Only the prediction interval is narrower (around ±10MPa). The most promising results of compressive strength were achieved in case of mortars characterized by *w/c* = 0.55. In this case the correlation of the fitted mathematical function is strongest (R^2 = 0.374) and the prediction interval is narrowest (around ±8MPa). The compressive strength of these mortars varies from 18.36MPa for mortar with no CF to 27.91MPa for mortar with maximum CF addition of 50%.

Results of tests of flexural strength are presented in Fig.4. All fitted functions are linear and plotted on three bubble charts (each for a different w/c ratio). Mortars with *w/c* = 0.50 are characterized by the worst flexural strength performance. Overall strength is smallest throughout the tested range of CF addition. The correlation of the fitted function is much weaker than in two other cases and the prediction interval is widest (over ±2MPa). The best results were achieved by mortars with *w/c* = 0.55. The flexural strength ranges from 5.98MPa for mortar with no CF to 8.54MPa for mortar with maximum CF addition of 50%. The correlation of the fitted function is the best among discussed flexural strength relations (R^2 = 0.613) and the achieved prediction interval is the narrowest and equal to ±1.5MPa.

5. CONCLUSIONS

The results achieved during the research programme prove that mortars based on natural aggregate partially replaced by CF can be used to cast elements characterized by less demanding mechanical characteristics. It seems that such a production would to be possible and feasible at the same time. Taking into account the tested properties, mortars characterized by *w/c* = 0.55 are the most promising. Both strength characteristics of these mortars are “stable” and with the narrowest prediction interval. Both strength relation are also described by equations with the best correlation (comparing to other tested mortars).

This study work forms the basis for further research on mortars made with fine aggregate partially replaced by CF. Testing shrinkage (as the porous nature of the CF should significantly decrease it) and larger scale specimens, would be the most desired area of scientific interest. The analysis of mortars’ microstructure and comparing it with microstructure of ordinary mortars should be the key element giving the answer to strength performance of mortars in question [7]. Propagation of damage and behaviour under dynamic loading would be also very interesting to test and model due to porous structure of CF, possibly significantly influencing properties of discussed mortars [2,3].

Growing problems with obtaining natural aggregates and environmental awareness put pressure on construction industry to improve its efficiency in harnessing waste and local materials [1,5,12,13,22]. CF is probably the best waste material to start industrial scale production of sustainable cement composites. Combining two waste aggregates (as in the discussed research programme) allows to achieve a very low carbon footprint of one cubic meter of cement composite.

It would be also interesting to pursuit other technological possibilities like adding fibres (including waste fibres) to discussed mortars and testing their self-compacting ability [19].

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Studies and Research on Physical and Mechanical Parameters of Building Stone Used in Fortresses in Dobrogea. Case study: Histria Fortress

Mihaela Drăgoi, Bucur Dan Pericleanu

***Abstract* – This paper presents a historical and technical study on the physical and mechanical characteristics of stone used in the construction of Histria fortress. The main objective of the analysis performed by the authors is to establish structural compatibility parameters needed to promote new technologies in structural restoration. The research is extended also to other heritage buildings from Constanta County.**

***Keywords* – heritage buildings, physical parameters, stone.**

1. INTRODUCTION

Conservation and restoration of heritage buildings is a complex activity, both in terms of preserving historical substance and intervention measures that must be undertaken to ensure structural safety and functionality. A special category is represented by natural stone structures, for the frequent use of stones is characteristic to the oldest stone building housing and worship places from some parts of the country. A first step in this activity is represented by the correct identification of the degradation process of the stones that are themselves of great complexity and depend on physical, chemical and mineralogical characteristics specific to each type of stone, but is connected with nature, intensity and frequency of aggressive factors and their possible interactions. In this context, the paper proposes that based on studies, theoretical and experimental research, to bring contributions concerning the determination of physical and mechanical parameters of the material used in the fortresses of Dobrogea and thereby contribute to the development of models, techniques and solutions compatible conservation and restoration of natural stone elements and construction.

2. HISTORICAL AND TECHNICAL STUDY ON NATURAL STONE

The material analyzed in this research paper was taken from Histria fortress in Dobrogea area. Histria fortress is one of the oldest city on Romanian territory and was the first Greek colony on the West shore of the Black Sea. According to the historic research performed and by Eusebius writtings, it was proved that the colony has been founded in the middle of the 7th century B.C. by Milet colonists. The city developed constantly for the next 1300 years on different historical periods, starting with the Greek period up to the Roman – Byzantine period. The historical study identifies three important periods for the fortress development: the Greek period (7th

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– 1st century B.C.), the Roman period (1^{st} century B.C. – 4^{th} century a.D.) and the Roman – Byzantine period (4th – 5th century a.D.) [1].

During the Greek period, the city had two very distinctive parts: the civil area and the acropolis, each of those parts surrounded by its own area wall. The acropolis was a sacred area, very important for the city religious life, constituted as temples of Zeus and Aphrodite. The civil settlement area was located on the west side of the acropolis and was surrounded since the archaic epoch by a stone site.

During the next period the city adhered to the Athens Maritime League which lead to the development of trading and allowed the city to stamp its own coin in the middle of the 5th century B.C. Along the different periods from 6^{th} to 4^{th} century B.C. the citadel has been destroyed several times but every time it has been rebuilt more prosper as it was proven by the archaeology vestiges on site. Towards the middle 3rd century, the city suffered another violent destruction period from the carpo-Gothic attacks and soon after another reconstruction period followed as proven by the site wall construction. This wall restricted drastically the city to an area of 7 hectares.

The Roman – Byzantine epoch was not a glamorous one, but it has been documented very precisely by archaeological research the fact that people have continued living inside as well as outside of the site wall area. Most of the monuments that can be seen today date from this period, for example five Christian basilicas have been discovered.

There were periods that left their mark on the fortress like the Avar – Slavic invasions (6^{th} – 7^{th} century a.D.) which determined its inhabitants to desert the city. The ruins of the fortress start being investigated in 1914, as they were an important source for understanding the history of Dobrogea.

We pointed out some of the important historically data to emphasize again the importance of this monument in terms of uniqueness, of historical value, but also that these data gives us some important information about events that had consequences and the compliance structure, and the constituent materials of the resistance structure of the city.

A characteristic of Dobrogea area is the use as construction materials of limestone (with compressive strength ranging from 120 to 550 daN/cm^2) and sandstones (with compressive strength between 65 and 1380 daN/cm^2). Both sedimentary rocks - limestone and sandstone - are easy to cut compared to other stones, which has made it quite often used in historical masonry buildings. On Romanian territory are found many masonry buildings made from siliceous sandstones and calcareous sandstones, the first ones with higher compressive strength. In general, stone as a building material is of local origin, rarely building stones taken from other quarries that give superior qualities of this material are used in heritage buildings. Traditional stone masonry can have different dimensions; stones are usually bounded with lime and sand mortars.

In the investigated area, the oldest historical building fund include mainly churches and monasteries, followed by residential buildings; as a consequence of local seismic conditions, as well as the historical conditions, the buildings, even the monumental ones, are low and rigid, with a balanced development of volumes, horizontally especially.

In general, the stones used for the structure resistance of buildings in the investigated area come from very varied mineralogical composition, and as a consequence their mechanical characteristics and behavior over time is different. In the work of consolidation, conservation, rehabilitation of these types of constructions, it is necessary to identify the mechanical characteristics to be able to place the stones into one of the categories of rocks, with certain physical, chemical and mechanical characteristics, in many cases it cannot be identified the origin of stones, in which situation sampling and laboratory analysis is required. At the same time there are situations in which the material used belongs to closed quarries or construction are made with reused materials, in which situation it is mandatory to evaluate the bearing capacity and thus the degree of reliability necessary; investigations for establishing the mechanical characteristics to be considered in structural calculation are also required.

3. STUDY METHODOLOGY

Starting from the above mentioned items in order to considerate the appropriate intervention measures to strengthen and / or conservation of buildings made of natural stone, it is necessary to identify the type of stone used, the mortar, the vulnerabilities. Thus, it is necessary to carry out observations, analyzes and investigations aiming at:

- approximate identification resulting from a visual inspection of color, texture, stone structure, which allows classification in a certain category of rocks;

- precise determination of various characteristics when sampling and laboratory analysis is necessary.

Currently there is no database in Romania for more accurate identification of these materials, their properties depending on the origin / extraction area.

Internationally and nationally there are defined rules, procedures, standards and recommendations for material characterization (especially stone and mortar) - and we refer here to those of RILEM [2; 3] and NORMAL [4], analysis of various published papers that allows verifying frequently several other procedures, but nevertheless, in terms of the conservation materials are not compatible with those normally used to characterize a new material. There are inconsistencies in terms of size and minimum number of samples that can be used in a study on the conservation of material because in this domain, in this vast area, the number and dimensions of the samples are imposed by the specificity of the study required and the amount of existing material. All of the above and studies on this topic in the literature justify adapting these rules, procedures and recommendations for each situation from their original form, although in some cases they come to represent merely a starting point in defining the methods used to do research.

The first major phase which precedes design of rehabilitation of historic masonry structures will reveal the detailed inspection and degradation of building stone. The second major step is the instrumental investigations, respectively physical, petrographic and biological tests. For degraded masonry is required to identify and establish the natural stone type and the composition of mortar link. The determination should be if possible, nondestructive, and otherwise to rely on samples with small dimensions (samples taken from the existing structure).

Investigation criteria of stone masonry can be classified as follows:

- Influence of sizes and shapes of masonry blocks on its resistance;
- Influence of materials used to built the stone masonry;
- Influence of execution technology on its resistance;
- Influence of key demands faced by the construction elements of stone masonry;
- Influence of external factors and rheological phenomena on physical and mechanical properties of stone masonry;
- Influence of masonry degradation on mechanical resistance of masonry structural elements.

These criteria were exemplified as materials characterization of natural stone masonry and the presentation of factors that affect it is done.

Evaluating the progress of degradation we can summarize it according to this scheme:

Constituent materials	Building history	
		Alteration phenomena
Sampling	Environment	
↓	↓	↓
Analysis	Climate	Tests
	Microclimate	Sampling
Study of new materials	Aggressive agents	Analysis
↓	↓	↓

Analysis and interpretation of results

↓

Evaluation of possible treatments

4. ON SITE AND LABORATORY TESTING – CASE STUDY

Characterization of stone types that were studied was made in terms of physical, mechanical, colorimetric and petrographic characterization and study methodologies used correspond to a set of tests commonly used in studies in the conservation of stone. To characterize the physical, mechanical and colorimetric properties international RILEM recommendations for laboratory testing stone were applied. Petrographic characterization was performed by microscopic observation with the objective to identify and characterize the minerals present.

To characterize the physical, mechanical and colorimetric properties the samples were dried to constant mass in an ventilated oven at (60 ± 5) ° C – the drying temperature recommended by RILEM and chosen instead of a higher one to avoid damaging organic materials used for the treatment of stone - and then cooled to the ambient temperature in a desiccator. Constant mass is achieved when the difference between two successive weightings at intervals of 24 hours is not greater than 0.1% of the sample mass, determined with an accuracy of 0.01%. After systematic attempts made with different varieties of stones the study found that constant mass is reached after 96 hours, and this time was considered as the reference for drying the tested samples.

All material characterization tests were conducted in the laboratory of Superior Technical Institute - Technical University of Lisbon, Portugal and in the mechanical testing laboratory from the Faculty of Civil Engineering, "Ovidius" University from Constanta, Romania.

Physical characterization of stones [2] is made by tests that define:

- Internal structure of the analyzed material:
  - Porosity accessible to water;
  - Real density and bulk density;
  - Maximum percentage of water absorption;
- Properties related to the presence and movement of water:
  - Saturation coefficient;
  - Water absorption by capillarity coefficient;
  - Water absorption under low pressure (Karsten tube method and box method);
  - Conductivity coefficient of water vapor;
  - Evaporation curve;
- Internal cohesion:
  - Compressive strength;
- Mechanical properties of the surface:
  - Surface hardness measured by recoil;
- Durability of the material:
  - Could not be determined;
- Miscellaneous characteristics:
  - Exterior aspects of stones;
  - Sampling.

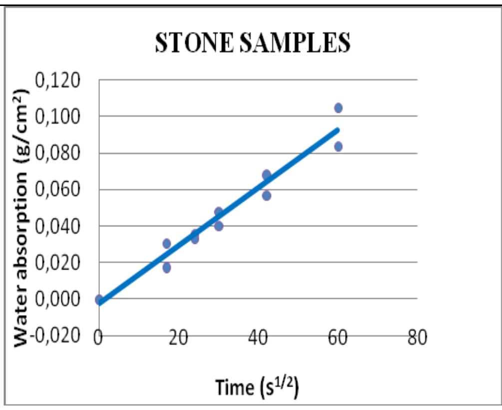
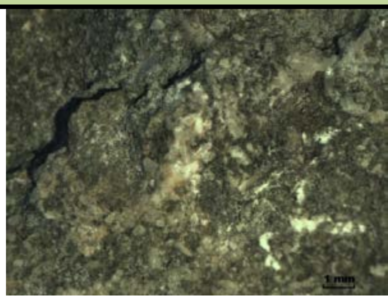
Mechanical characterization of varieties of stones was performed by using tests that allow assessing internal cohesion, for example the compressive strength. Other properties, such as the propagation velocity of ultrasound gives us indirect information on the mechanical properties.

To achieve mechanical characterization the following tests were used:

- propagation velocity of ultrasound;
- compressive strength;
- resistance by micro-drilling technique.

Colorimetric characterization of the material was performed in a quantitative form through the use of a colorimeter.

Petrographic characterization (in small specimens extracted from degraded areas) has established the nature of building stones and biological tests indicated the nature of live agents affecting masonry.

MATERIAL INTERNAL STRUCTURE		
Porosity accesible to water [5]	3%	 **Fig. 1**. Porosity tests
Real density [6]	27 kN/m^3	
Bulk density [6]	26 kN/m^3	
Maximum percentage of water absorption	1,1%	
PROPERTIES RELATED TO THE MOVEMENT OF WATER		
Coefficient of water absorption by capillarity [7]	0,5843 $Kg/m^2h^{1/2}$	
Absorption of water under low pressure [8]		 **Fig. 2.** Results for absorption of water under low pressure tests
INTERNAL COHESION		
Compressive strength [9]	28,77 MPa	
MECHANICAL SURFACE PROPERTIES		
Surface hardness measured by rebound tester [10]	N = 49	N – average recoil index
EXTERIOR ASPECT		
Exterior aspect of the surface of stones		 **Fig. 3.** Microscopic optical analysis

CHROMATICS

Chromatic [11] -

CIELAB system with coordinates L *, a * and b *, and complementary chrome values, color saturation,C *, which is obtained using the formula:

$$C^* = \sqrt{a^{*2} + b^{*2}} \; .$$

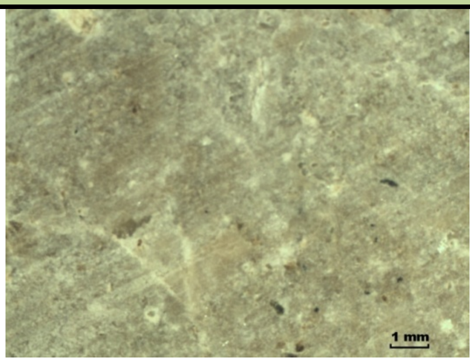


Fig. 4. Sample surface for colour characterization

L*	a*	b*	C*
73,74	3,32	6,88	7,65

MECHANICAL CHARACTERIZATION

Propagation velocity of ultrasound

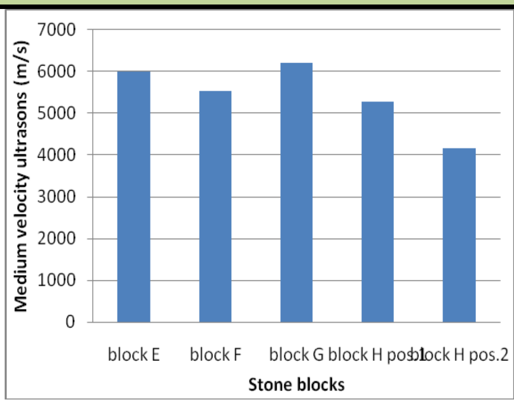


Fig. 5. Propagation velocity of ultrasound results

Fig. 6. Propagation velocity of ultrasound testing

5. RESULTS INTERPRETATION

We can quantify the results of research conducted by the research team as follows:

• for buildings made of limestone in the areas of Dobrogea was found that in case the walls were not protected against the elements by finishing systems or films, the compressive strength of stone fell an average of (15-18)% due to its high porosity, freeze-thaw phenomena that led to micro-cracking of the surface in contact with the environment;

• durability of stone masonry structures is influenced by two factors: strengthening of mortar layer between stones and stone technology distribution. The execution technology has proven to be relevant in assessing the bearing capacity of structural walls;

• quality of the mortar is influencing the behavior in time, especially on unprotected structures, motivated by the fact that atmospheric humidity combined with temperature variations destroy the adhesion between particles of sand and other constituent particles, mortar becoming friable, crumbly, unable to contribute properly to the resistance of the composite material - stone masonry;

• atmospheric factors, in many cases, have contributed to exfoliate the top layer of the limestone;

• in case the stone masonry is being positioned in swampy areas and the type of stone is limestone without cement addition, it was found that the volume changes, possibly due to alternative phenomena moisture - drying.

The research results provide a knowledge base to quantify the characteristic resistance of materials and structural safety assessment that benefits a building. Studies and analyzes conducted in this phase of research have contributed to the knowledge base regarding the analysis of constituent materials of natural stone structures; the analysis focused on both the mechanical properties of the original material, but also how these properties have changed over time due to factors like atmospheric phenomena, frost - thaw rheological phenomena, physical and chemical factors that have altered the materials that are characteristic to Histria fortress and Dobrogea area.

On existing building we are working with old constituent material whose decreased mechanical property generates reduced characteristic strength in the overall structural conformation and a low bearing capacity. To these aspects we add a significant factor in not few situations that often complicate the restoration works, and that is human negligence and ignorance regarding consolidation works.

Structural restoration work is an integrated approach which necessarily must lead to the preservation of the historical substance of the built monument, preservation of the traditional technology used, the insertion of materials that are compatible with the constituent materials. These issues are still a component of great importance to the knowledge of mechanical parameters of materials.

It is difficult to establish an idealization and a specific intervention solution to be accepted in a given situation for a monument, because each historical monument has a unique value and each must be examined and evaluated accordingly. So there is no preset recipe that can be applied. But what it can be done through existing research studies and case studies from the literature is a procedure to be followed in the analysis of such a monument. We can achieve to implement a working methodology for the study, evaluation and analysis of a heritage building.

6. ACKNOWLEDGMENTS

The authors would like to thank professors: Ana Paula Pinto, João Gomes Ferreira, Fernando Branco and Grãmescu Ana Maria.

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SECTION III

COASTAL ENGINEERING

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Probabilistic analysis methods for breakwater stability

Mari-Isabella Stan

***Abstract* – Estimating the probability of failure of a port construction defense is an important task for an engineer. The purpose of this article is to introduce the most common structural reliability analysis techniques. First-order reliability method (FORM) is considered to be one of the most reliable methods for calculating structural safety. This article presents a probabilistic analysis of stability failure mode for mantle of port construction protective defense (breakwaters) at the request of the waves, using the computer program NESSUS.**

***Keywords* – analysis, breakwater, First-order reliability method (FORM), probabilistic.**

1. INTRODUCTION

Probabilistic structural analysis can be seen as an extension of deterministic structural analysis, which is the art of formulating a mathematical model within which one can ask and get answer to the question: "How is a structure behaving when its material properties, geometric properties and actions all are uniquely given?"[1]

In civil engineering, there are several methods to estimate the reliability of structures for a given limit state. These structural reliability design methods are classified into three categories according to the probabilistic concepts used, for example, Level 1, Level 2 and Level 3 methods. Each method gives results in different forms, but all of them can be expressed in terms of probability of failure. Assessment of structural reliability is always linked to the structural response defined as failure mode.

First-order reliability method (FORM) is a Level 2 method and is considered to be one of the most reliable methods for calculating structural reliability [6]. FORM method is used to analyze the structural stochastic and can include any amount of probabilistic information. In this method the failure surface is approximated by a tangent hyperplane.

FORM method is based on linearization function g at the most probably point of designing (MPP) in the space u (**Fig. 1**). The polynomial is of 1st degree and is obtained by expressing the Taylor's series of the function g:

$$g_1(u) = a_0 + \sum_{i=1}^{n} a_i \left(u_i - u_i^*\right) \tag{1}$$

where: terms a_i (i=0, 1, 2, ... n) are constant and the u_i terms are uncorrelated random variables. If we apply the three-step procedure for determining the Hasofer-Lind reliability index, we would obtain the following expression for β [2]:

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$$\beta = \frac{a_0 + \sum_{i=1}^{n} a_i u_i}{\sqrt{\sum_{i=1}^{n} \left(a_i \sigma_{u_i}\right)^2}} \qquad (2)$$

β is the reliability index and is a measure of the probability of failure. This is the inverse of the variation coefficient and is the distance (in standard deviation units) from the most probable value of g (in this case mean) at the surface of failure, $g = 0$.

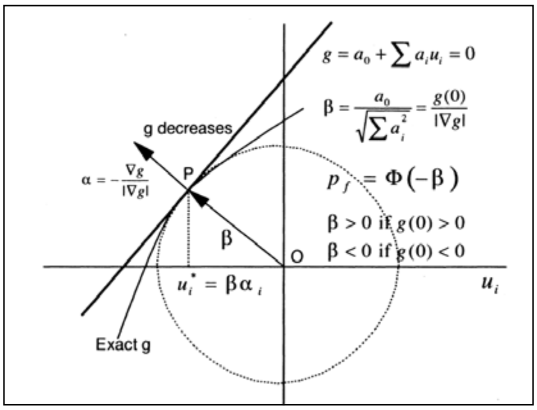

Fig. 1. First-order reliability method (FORM)

2. APPLICATION OF FORM METHOD FOR EVALUATING THE STRUCTURAL RELIABILITY

In case of a breakwater the result of wave's requests depends, on the one part, of breakwater composition (weight of individual blocks shell, their density, slope, permeability of the nucleus and mantles, position of berms, etc.) and of the other part, of hydraulic parameters (wave height, direction, period, storm duration, water depth, sea bottom evolution). The relationship between these parameters for the dimensioning weight concrete blocks is given by Hudson.

In the case study I proposed a stochastic method for dimensioning of protection mantle weight concrete blocks of breakwater filled with stone and foundation height of 35m to -24m in Constanta [4].

All failure modes are described by a formula, and the interaction between different modes of failure must be known. For instance, it is considered a single failure mode "the weight of mantle concrete blocks or stone that covers the part of a structure is driven by the force of the waves" [3], described by the formula:

$$W = \frac{\gamma_r H_s^3}{K_D (S_r - 1)^3 ctg\alpha} \qquad (3)$$

where:

W – minimum weight for concrete blocks or stone;

γ_r – the unit weight of concrete blocks or stone in the air (tf/m^2);

S_r – the specific weight of concrete blocks or stone under the water level;

α – the angle between the slope and the horizontal plane;

K_D – stability coefficient determined by the rate of clench failure rate of the mantle and its products, coefficient with the significance of destruction (movement of blocks);

H_S – the significant height of the wave.

Using simplifying assumptions in specifying behavior of the structure under the action of wave's requests is necessary due to the given limitations of numerical methods used.

One of the most important parameters is the height of the wave H_s. This is a parameter that varies in time and is best modeled as a stochastic process. Therefore in the above equation I considered H_s a parameter of the load, and all the others variables are the parameters of resistance.

Starting from formula (3) I expressed the equation in the form failure function (or the function of performance):

$$g = W \cdot K_D (S_r - 1)^3 \cdot ctg\,\alpha / \gamma_r - H_S \begin{cases} < 0 & FAILURE \\ = 0 & LIMIT\,STATE \\ > 0 & SAFETY \end{cases} \qquad (4)$$

I considered that all input parameters involved are interpreted as random variables, u_i, the exception being K_D, that signifies failure - namely a certain level of damage, chosen by the structural designer.

I performed the stochastic analysis using the computer program Nessus [8] and the calculation method used is the method FORM (Level 2). This computer program is used for performance probabilistic analysis.

3. Results and significances

Following the development of computational procedure for structural reliability based on First-order reliability method (FORM) results in the following form:

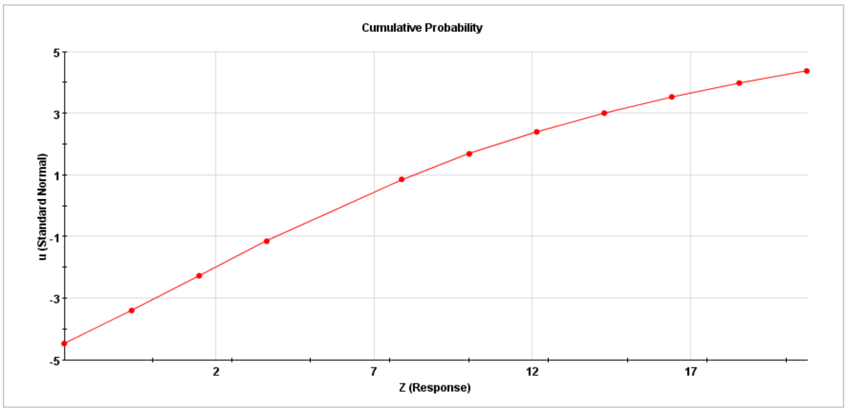

Fig. 2. Representation of failure function

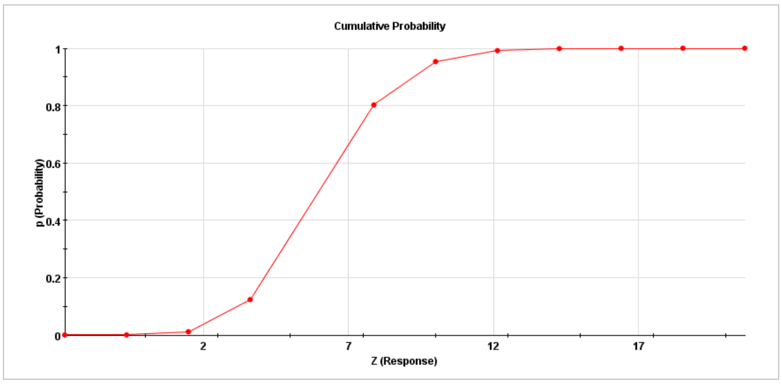

Fig. 3. Representation of cumulative probability

As a result of the computer program, I obtained the graphical representation of probabilistic sensitivity factors (**Fig. 4**) and probabilistic importance factors (**Fig. 5**) of the interpretation follows that different sign for parameters signifying resistance to those who signify loads affect probability failure in different directions: when H_s (wave height) increases, increases the probability of failure structure, and the other parameters increase, the probability of failure decreases.

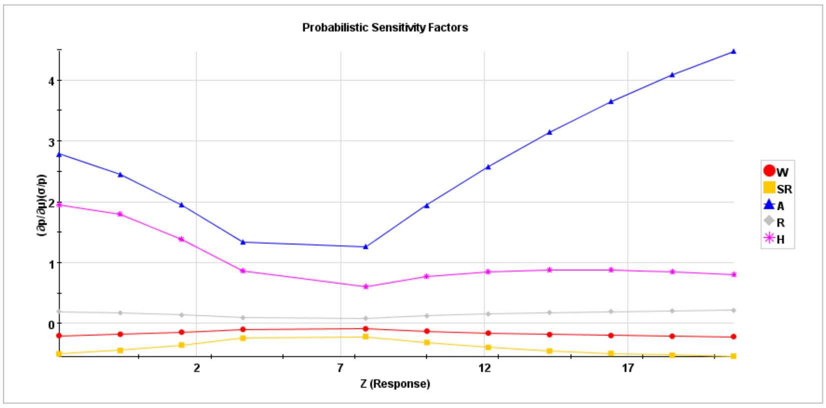

Fig. 4. Probabilistic sensitivity factors

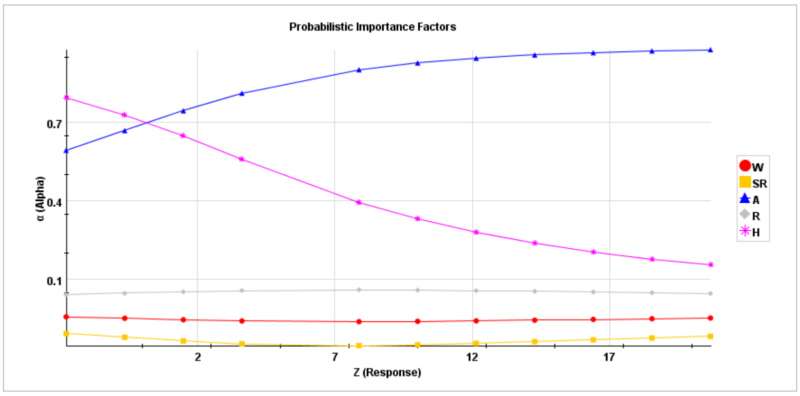

Fig. 5. Probabilistic importance factors

For different values of the failure probability results from the computer program, I obtained the graphical representation of probabilistic importance factors for different probabilities of failure (**Fig. 6**), which observes that the importance of wave height is constant and the weight of concrete or stone blocks also has the same importance regardless of their position on the failure probability the interval of application. If I impose g = 0 (limit state) I obtained from the calculations that the probability of failure is **0.1093041247E-02.**

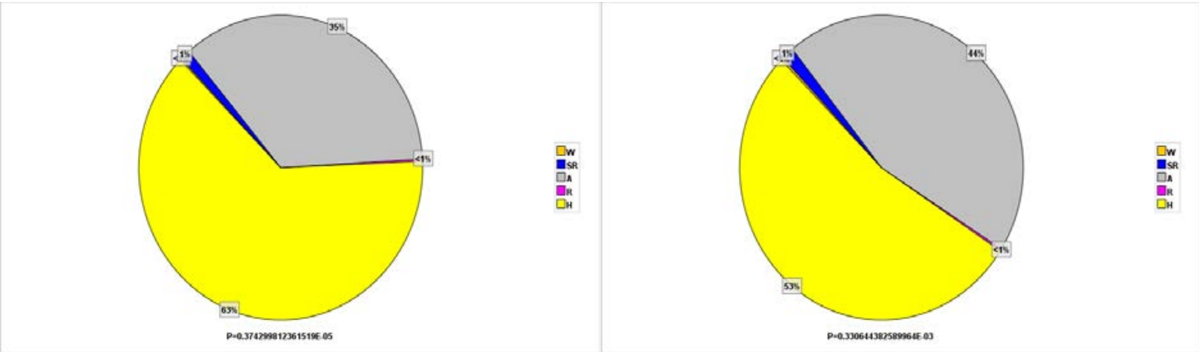

Fig. 6. Probabilistic importance factors for different probabilities of failure

By means of the computer program, I performed the graphical representation of the sensitivity levels (**Fig. 7**) for different levels.

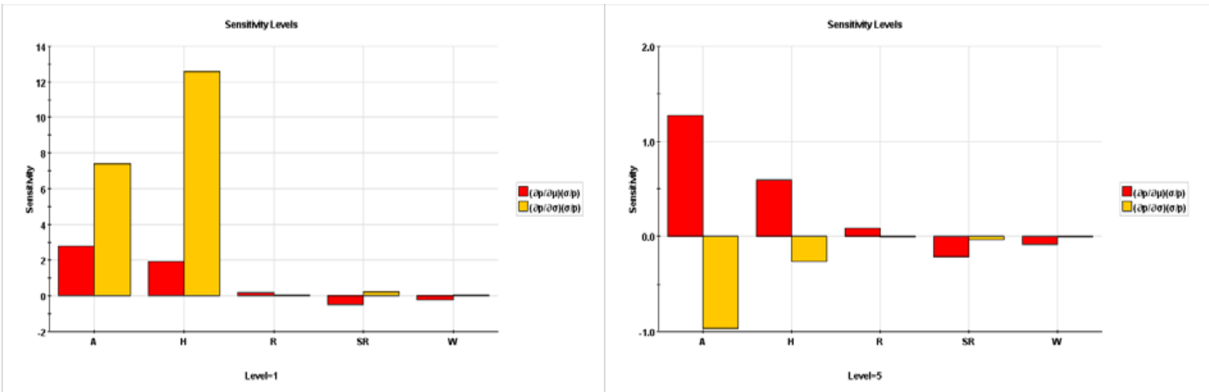

Fig. 7. Sensitivity levels

Another result obtained by computer program is the graphical representation of the importance levels (**Fig. 8**) from interpretation of which is found that throughout the interval of analysis [0,1] the importance of mantle weight concrete blocks or stone is constant, which suggests a careful tracking of materials that are put into practice.

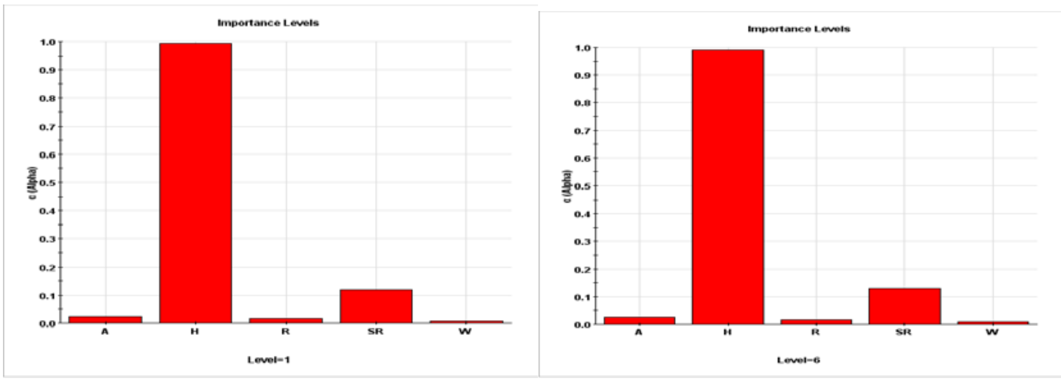

Fig. 8. Fig. Importance levels

4. Conclusions

Absolute reliability of a structure cannot be guaranteed because of the uncertainty and the variability of future requests properties of materials used.

The methods introduced by structural reliability theory provide tools for calculation of probabilities of failure of the structure or part of it.

First-order reliability method (FORM) is an attractive technique for stochastic modeling for the following reasons: directly generates a probability estimate associated with a particular uncertain event, can be used with model numerical solutions, directly produce sensitivity information and at the same time is effective even for events of small probability.

Results obtained with the FORM method using the software program NESSUS lead to the identification of advantages in response calculations of harbor defense construction, namely:

- deterministic type approach, accepting the general idea of the risk of failure of a structure during its life and considering the probabilistic aspect of the factors involved in evaluating behavior time, allows evaluating the economicity of work;
- more complete characterization of action factors, a better measure of cumulation effects in time, and the possibility of risk assessment and quantitative estimation, anticipated, possible damage;
- opportunity to act fairly and efficiently in the choice of technical solution by solving the optimization problem which involves the acceptance of possible mechanisms of breakwater failure under wave action.

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Longshore Sediment Transport Evaluation for the Mamaia Coast

R.Ciortan and K. Mezouar

Abstract – This research is a contribution to a collective study for wave nearshore transformation and preliminary evaluations for longshore sediment transport quantities and rates of the Mamaia north coast. In this study waves were transformed from the offshore area to the nearshore area, and longshore sediment transport quantities and rates were evaluated by applying and adjusting some of the available modern formulae. This was done by calibrating with the available reference data, as given in the literature. Thus, the applicability of such formulae for that area can be checked. Three bulk-type modern formulae, (CERC), Kamphuis, and Van Rijn, for longshore transport evaluation are applied in the study area. The reference targets used for comparison are based on the literature. Through calibration with the reference targets and among themselves, the study comes up with some correction factors for both CERC and Van Rijn formulae to give quite realistic evaluations for that area. These extended/corrected formulae can also be applied in places with similar conditions to the Romanian coast.

***Keywords* – sediment transport, Mamaia, Kamphius, Van Rijn .**

1. INTRODUCTION

Total long shore sediment transport (LST) rate and its cross-shore distribution in the surf zone are essential to many coastal engineering and science studies. Practical engineering applications such as predicting beach response in the vicinity of coastal structures, beach-fill evolution and renourishment requirements, and sedimentation rates in navigation channels all require accurate predictions of LST rates. Present predictive tools have been developed based primarily on field studies; however, obtaining high-quality data in the field is difficult [14]. It has long been recognized from a large number of field and laboratory experiments that variation of longshore sediment transport is strongly controlled by long shore currents generated by waves breaking at an angle with the shoreline [6] [8]. The scope of predictive formulas is largely empirical and reflects results based on field studies from around the world [8] [4] [2] [9] [11].

Researchers have found that sediment concentration and transport at the breaker line is strongly influenced by breaker type and thus wave energy [7] [10] [12]. Field techniques for measuring total and suspended long shore sediment transport include sediment tracer, impoundment and streamer traps. Here we employ the impoundment technique for comparison with three predictive long shore transport models.

The near-shore sediment transport system of Bejaia Beach is examined using 10 monthly beach surveys. We describe the dominant spatial and temporal patterns of sediment transport and volume variability and evaluate three commonly used Longshore Sediment Transport (LST) formulas: (CERC, 1984; KAMPHIUS, 1991; and Van Rijn, 2004). We find the KAMPHIUS (1991) model fits observations of longshore sediment

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transport best while the CERC (1984) and Van Rijn (2004) models are prone to overestimate the observed longshore transport by roughly an order of magnitude.

2. ENVIRONMENTAL SETTING

The studied area (Mamaia North- Navodari) beach is situated in the south eastern extremity of Romania, near Constanta city, on a narrow sand bar, 250 - 350 m wide, between the Black Sea and Siutghiol. Mamaia is the largest touristic seaside resort of Romania, stretching 5 Km from north to south. It is formed by sandy material that originates from the Danube. Mamaia beach is facing east and is a natural low sandy beach characterised by gentle sloping underwater profile down to - 6 m. The beach consists of alluvial sediments (brought into the Black Sea by the Danube and transported to the beaches by combined wave action and the north to south flowing current along the Romanian coast) and biogenic shells sediments. The sand is fine and has a grey light colour. The site under study is located on the Northern of Mamia beach to Media harbour between the geographical coordinates 44 20'N – 28 38'15"E and 44 15'N – 28 37'30"E. Shoreline length of the region is nearly 05 km. General alignment of the shoreline is north to south.(see figure 1)

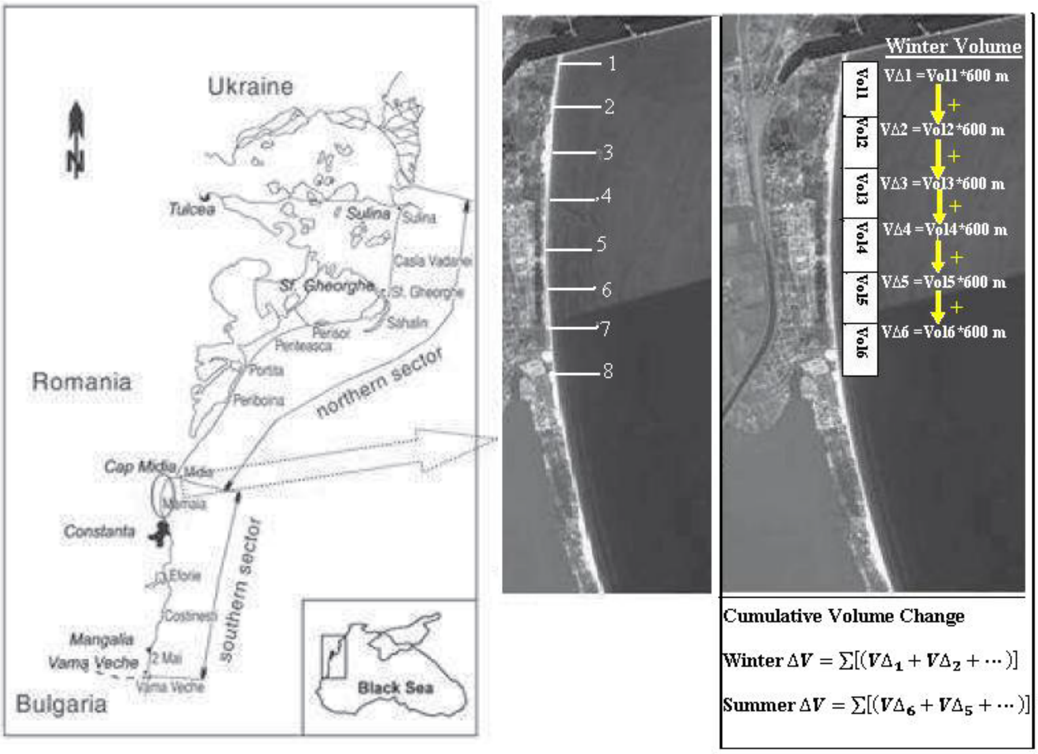

Fig. 1. : Studied area (Mamaia North- Navodari), Romania.
Cumulative profile volume change. Cumulative alongshore volume change derived from profile volumes (vol_1 to vol_6). Winter cumulative volume change calculated from profile vol_1 to vol_6, while summer is calculated from vol_6 to vol_1.

3. METHODOLOGY

BEACH PROFILE: Observations of beach profile changes were collected at a series of 8 shore-normal beach profiles transects situated along the length of the study area. Ten monthly surveys were performed from April to December, 2006. Beach profiles and volumes were measured using a streamer traps. The sediment traps were deployed along two shoreperpendicular transects with a spacing of about 100 m to investigate the alongshore variability in longshore transport rate. Measurements of longshore sediment transport were carried out at several locations across the zone. Shore-normal profiles extended over the sub-aerial and sub-aqueous portions of the beach with measurements at approximately 3m intervals or at each significant change in slope or bottom type. Surveys were conducted randomly with respect to swell conditions and typically extended approximately 100 m offshore into water depths of 5 to 7 m.

STREAMER TRAPS: Longshore transport rate rates were estimated using streamer traps similar to the original design of Kraus (1987 [9]). The sediment traps consisted of a vertical array of five individual streamer traps with 63 µm mesh size sieve cloth that collected sand-size particles at different elevations above the bed, up to a height of approximately 1m. Calculations of the sediment flux from sand traps were carried out according to the procedure of Rosati and Kraus [9].

$$\mathrm{F} = \mathrm{h}\sum_{\mathrm{i}=1}^{\mathrm{N}} \mathrm{F(i)} + \sum_{\mathrm{i}=1}^{\mathrm{N}-1} \mathrm{a(i)} \times \mathrm{FE(i)} \tag{1}$$

Where F is the depth integrated flux in $kg.s^{-1}.m^{-1}$, h is the height of the streamer opening in meters, F(i) is the sediment flux at a streamer I, a (i) is the distance between neighboring streamers, FE(i) is the sediment flux between neighboring streamers and N is the total number of streamers.

DEPTH OF CLOSURE: The depth of closure *hc* was calculated by the following formula by Hallermeier [5]:

$$\mathrm{h_c} = 2.23\mathrm{H_e} - 68.5\left(\frac{\mathrm{H_e^2}}{\mathrm{gT_e^2}}\right) \tag{2}$$

where *He* denotes the significant wave height to be exceeded for 12 hours per year. The wave climate off Constan a yields *He* = 5.0 m and *Te* = 9.1 s, and the closure depth is calculated as *hc* = 9.3 m. For the coast south of Eforie, the closure depth has been set at *hc* = 7.1 m in consideration of larger grain size than the sand in Mamaia Beach.

FORMULAE USED IN EVALUATION: As mentioned previously, three bulk-type formulae are used for the required purpose of littoral drift evaluations. The details of these formulae are as follows:

CERC Formula [2] [3]: This is the most widely used formula in coastal engineering practice all over the world. It was originally developed by the U.S. Army Corps of Engineers in 1984. It relates the immersed weight (I) of the longshore sediment transport rates to wave energy flux. It works based on the proportionality principle of both the volume of transported fine sediments (Q_{lst}) and the beach longshore wave power per unit length, as given in Equations (3) and (4).

$$\mathrm{I} = \mathrm{K} \times \mathrm{E} \times \mathrm{C_{g,br}} \times \sin\theta_{\mathrm{br}} \times \cos\theta_{\mathrm{br}} \tag{3}$$

$$\mathrm{E} = \frac{1}{8} \times \rho_{\omega} \times \mathrm{g} \times \left(\mathrm{H_{rms,br}}\right)^2 \tag{4}$$

where I = longshore transport rate (immersed weight [N/s]); K = coefficient for the CERC (0.39), the value that is derived from the original field study carried out by Komar and Inman using tracers; E = wave energy at breaker line [N/m]); $H_{rms,br}$ = root mean squared wave height at breaker line (m); $C_{g,br}$ = nbr X cbr = wave group celerity at breaker line (m/s); θ_{br} = wave angle at breaker line (between wave crest line and coastline, or between wave propagation direction and shore normal direction [degree]); C_{br} = phase velocity of the waves at the breaker line (g X h_{br})0.5 (m/s): h_{br} = water depth at the breaker line (m); n_{br} = coefficient at breaker line; Hs = the significant wave height (1.414 X Hrms); ρ_w = 1030 kg/m^3 (preliminary guiding value); and P = the porosity factor (0.40). Equations (5) and (6) follow.

$$Q_{t,vol} = 0.023 \times \left(H_{s,br}\right)^2 \times n_{br} \times c_{br} \times \sin(2\theta_{br}) \quad (5)$$

$$Q_{t,vol} = 1/\left((1-P)(\rho_s - \rho_\omega) \times g\right) \quad (6)$$

where $Q_{t,vol}$ = longshore sediment transport by volume, the sediment transport by dry mass, (m^3/s, including pores); $H_{s,br}$ = significant wave height at the breaker line (m); ρ_w = density of water (kg/m^3); ρ_s = sediment density (kg/m^3); and g = acceleration of gravity (m/s^2). Applying $n_{br} \approx 1$, $c_{br} \approx (g\ h_{br})^{0.5}$, and ($br = H_{br}/h_{br}$), Equations (7) and (8) follow:

$$Q_{t,vol} = 0.023 \times g^{0.5} \times (\gamma_{br})^{-0.5} \times \left(H_{s,br}\right)^{2.5} \times \sin(2\theta_{br}) \quad (7)$$

$$Q_{t,mass} = (1-P)^2 \times {\rho_s}^2 \times Q_{t,vol} \times 0.023 \times g^{0.5} \times (\gamma_{br})^{-0.5} \times \left(H_{s,br}\right)^{2.5} \times \sin(2\theta_{br}) \quad (8)$$

where $Q_{t,\text{mass}}$ = longshore sand transport (kg/s, dry mass) and $H_{s,\text{br}}$ = significant wave height at breakerline (m).

Kamphuis (1991) Formula: Kamphuis (1991) developed a relationship for estimating longshore sediment transport (LST) rates based primarily on physical model experiments. The equation, which Kamphuis (2001) found to be applicable to both field and model data, is expressed as:

$$Q_y = 0.0013\ \frac{\rho H_{sb}^3}{T_p} \times m_b^{0.75} \times \left(\frac{H_{sb}}{d_{50}}\right)^{-1.25} \times \left(\frac{H_{sb}}{L_0}\right)^{0.25} \times \sin^{0.6}(2\theta_b) \quad (9)$$

in which Q_y is the transport rate of underwater mass in kg/s, T_p is the peak wave period, m_b is the beach slope from the breaker line to the shoreline, and d_{50} is the median grain size. Kamphuis (2001) [6] uses the same equation, but redefines the beach slope as the slope that causes breaking, *i.e.*, the slope over one or two wavelengths offshore of the breaker line. However, the slope offshore of breaking in the LSTF is somewhat artificial because of the physical model limits. Therefore in the present study, m_b is defined as the slope from the breaker line to the shoreline as defined by Kamphuis (1991).

Equation (9) is appealing because it includes wave period and slope, which influences wave breaking. Additionally, grain size diameter, a relevant factor in incipient sediment motion, is included although d_{50} is used as a stirring function rather than threshold of motion in Kamphuis (1991) [6].

Van Rijn Formula [13]: This is a simple bulk longshore transport formula, developed based on the parameterization of computed transport rates of modeling work and measured transport rates in the surf zone of various beaches. Its initial determined trendline is valid for very fine particles, especially sand with a median diameter (d_{50}) between 150 and 500 μm and bed slopes between 0.02 and 0.1. The data set used in the formula development is too small to detect any effect on grain size and/or the bed slope. The longshore transport rate is described as a combination of the wave-related stirring action parameters and the wave-driven longshore current velocity ($V_{\text{wave,L}}$) in the approximate middle of the surf zone area. This approach can be used to include the tidal-driven longshore current velocity if any. The obtained results have been used to determine a simplified formula for the longshore fine sediments/sand transport including all the expected effects [13]. It reads as given in Equation (10), and the definitions of ($V_{\text{eff,L}}$) and ($V_{\text{wave,L}}$) are as given in Equations (11) and (12), respectively.

$$Q_{t,mass} = 42 \times K_{swell} \times K_{grain} \times K_{slope} \times \left(H_{s,br}\right)^{2.5} \times V_{eff,L} \quad (10)$$

$$V_{eff,L} = \left[\left(V_{wave,L}\right)^2 + \left(V_{tide,L}\right)^2\right]^{0.5} \quad (11)$$

where $V_{\text{eff,L}}$ = effective longshore velocity at the middle surf zone (m/s) for tidal velocity and wave-induced velocity in the same direction (minus sign for opposing conditions).

$$V_{wave,L} = 0.3 \times \left(g \times H_{s,br}\right)^{0.5} \times \sin\ (2\theta_{br}) \quad (12)$$

where $V_{wave,L}$ = wave-induced longshore velocity in the middle surf zone (including wind effects [m/s]); $V_{tide,L}$ = longshore velocity in middle of the surf zone due to tidal forcing (0 for non tidal cases as the conditions in the Mediterranean Sea; K_{swell} =(T_{swell}/T_{ref}) = swell correction factor for swell waves < 2 m; T_{ref} = reference wave period = 6 s; K_{swell} =1 for wind waves; K_{grain} =($d_{50,ref}/d_{50}$) = particle size correction factor with d_{50} in mm ($d_{50,ref}$ = 0.20 mm), with $K_{grain,min}$ = 0.1 for d_{50} > 2 mm; K_{slope} = $(\tan\beta/\tan\beta_{ref})^{0.5}$ = bed slope correction factor; $K_{slope,max}$ = 1.25, $K_{slope,min}$ = 0.75; tan β = actual bed slope; $\tan_{\beta\ ref}$ = 0.01 ; the overall profile slope is defined as the slope between the waterline and the 8-m depth contour; tan β = (8/*B*) where *B*= distance between waterline and location of 8-m depth contour seaward of outer breaker bar (m). The beach slope of the inner surf zone slope cannot be used in the slope correction factor. For a zero tidal velocity ($V_{tide,L}$ = 0 m/s), Equation (3) results, as presented in Equation (13).

$$Q_{t,mass} = 40 \times K_{swell} \times K_{grain} \times K_{slope} \times \left(H_{s,br}\right)^3 \times \sin\ (2\theta_{br}) \quad (13)$$

After adjusting this proposed empirical formula, it can be considered suitable for longshore sediment transport evaluation in the study area. This area has very fine silty sand particles, as will be discussed in detail based on the literature studies. No limitations due to artificial constructions existence or similar are proposed with this formula.

4. RESULTS AND SIGNIFICANCES

Table 1. Beach profiles volume by location

profile	Maximum volume (m^3/m)	Minimum volume (m^3/m)	Volume range (m^3/m)	Mean volume (m^3/m)	Mean volume rate change (m^3/m/month)	Net volume change (m^3/m)
1	121,97	95,83	26,14	98,23	-0,33	-2,33
2	119,21	103,54	15,67	105,55	-0,64	-5,65
3	137,61	113,15	24,46	95,64	0,96	10,43
4	127,66	110,53	17,13	105,17	-5,60	-37,54
5	162,17	109,23	52,95	132,34	4,43	23,11
6	175,22	122,13	53,09	123,00	6,00	41,90
7	191,14	143,12	48,02	134,87	-1,33	-13,77
8	187,36	139,00	48,36	152.30	-3,75	-31,59
Mean	152,79	117,07	35,73	118,39	-0,04	-15,44

Beach volumes are calculated as the volume under the profile extending from the landward edge of the subaerial beach to the first occurrence of submerged hard substrate often just seaward of the toe of the beach. The profiles extend from the landward edge of the dune system beyond the beach toe to the edge of the reef slope. We calculate the spatial cumulative beach volume alongshore based on each sectional volume (profile volume per alongshore unit of beach). The section volumes are in turn multiplied by the alongshore distance between each profile to account for the monthly volume change for each section of beach. In order to integrate over the entire area and reduce the effect of seasonal outliers, we calculate the cumulative net sum alongshore as a proxy for longshore transport rates (figure 1). Three main sources of error are identified in the uncertainty analysis for profile area volume. Where: Volume Uncertainty (*VU*).

$$VU = [(\text{meander error})^2 + (\text{basement error})^2 + (\text{ cross-shore error})^2]^{1/2}$$

Measurement error is considered negligible as the profiling technique used has centimeter accuracy. The landward margin of the profile is fixed and thus induces no uncertainty. Meander error (± 300 m^3), is associated with variation in the seaward margin of the sub-aerial profile as observed in foreshore meanders at the shoreline. Meander error is calculated by taking the mean observed meander width (± 10 m) times the alongshore wavelength of the meander (50 m) times the profile area (1 m). Basement error (± 2 m * 300 m * 1 m = ± 600 m^3), caused by variable relief of the basement strata, (which constitutes the lower boundary of the profile volume) is assumed to be horizontal landward from the first occurrence of hard basement. Cross-shore error (± 900 m^3), calculated from the seasonal profile net volume difference between profile 5 and 9, represents sediment lost outside the profile due to cross-shore transport. Using the additive error process described above, volume uncertainty for observed net annual volume change is estimated to be ± 800 m^3/month and is reported as the mean percentage of each monthly cumulative volume.

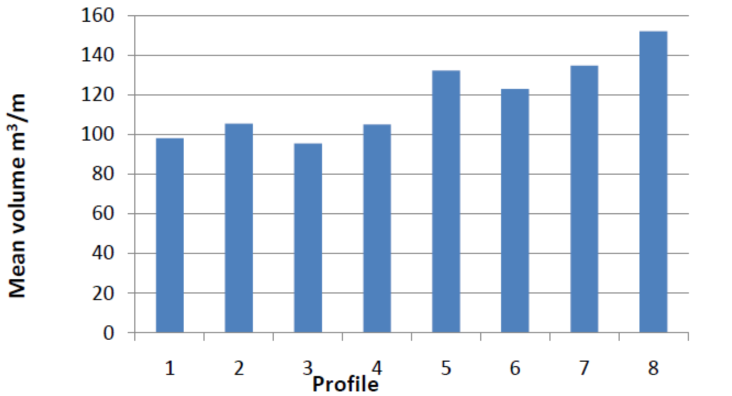

Fig. 2: Mean profile volume by location.

Surveyed beach profiles at Mamaia reveal a clear cyclic pattern of erosion and accretion due to seasonal wave forcing. While the profile volumes are not highly variable alongshore we see that the mean volume, volume range and net volume are in the southern portion of the study area (profiles 5–8) (Table 1). Beach profiles exhibit little to no transport of sediment offshore but they do show a change in beach face profile volume, which suggests longshore transport is acting upon the profile. The distribution of profile volume change reveals the dynamic nature of the southern portion of the study area. Most profiles reveal a strong seasonal signal with net erosion in the winter summer and accretion in the summer. A closer look at the dynamics of the profile volume shows that the net volume for all profiles is highly variable over the 10 month period with the peak summer and winter months showing the largest net loss or gain from the mean. In addition to seasonal trends, we find an alternating pattern of erosion and accretion alongshore. The alternating nature of the profile state switches alongshore with one profile contributing sediment to the neighboring profile seasonally.

The main aim for reviewing some of the previous studies related to longshore transport governing environmental parameters, quantities, and rates is to test and calibrate the predictive capability of the modified/corrected formulae in the study area. Generally, division of data into the calibration should be done randomly. When identifying the data sets for the purpose of being suitable for calibration of the modified longshore sediment transport formulae, accuracy and reliability of the measurements are considered primary criteria for inclusion. Generally, the data sets consist mainly of wave properties, sediment governing characteristics, and the longshore transport rates at a group of locations ($H_{s,br}$ [m], T [s], d_{50} [mm], θ_{br} [degree], and $Q_{measured}$ [m^3/s]). Tide is considered insignificant in the study area and the black Sea on general bases. In the following discussion, a summary for the available records related to some historical studies are presented. The considered hydrodynamic parameters are as follows: the significant wave height value equals 1.8 m associated to the given water depth, the wave period was assumed to be 7 s, the average water depth equals 8 m, the average longshore current velocity equals 0.2 to 0.3 m/s, and the considered median grain size (d_{50}) equals 150* 10^{-6} m at the associated water depth, 8 m. The study highlighted the importance of taking the effects of the blowing storms on the study area into consideration. Their importance is based on their severe effects in a short time compared to the existing normal conditions during the year.

The Mamaia nearshore generally exhibits a reflective beach state with plunging to surging waves as described by WRIGHT and SHORT [11].This beach state favors coarser sediments and/or longer period swells and generally displays a steep narrow beach with a well-defined toe at the base of the foreshore. The strong swash and coarse sediment often form sub-aerial beach cusps. The presence of a distinct, migrating pattern of erosion and accretion suggests neighboring profiles exchange sediment seasonally and supports the theory that longshore transport is controlled by seasonal wave energy.

The approach angle has a direct influence on the direction and magnitude of the LST rate and is one of the primary influences of seasonal transport of sediment in the study area.Evaluations of the annual longshore transport quantities (*Qt* , m^3/y including pores) and rates (kg/s) are carried out based on CERC, Kamphuis, and Van Rijn formulae. This research finds these formulae provide quite realistic estimates and simple application.

For good calibration, the ideal average trend line between both calibration values and calculated ones should have an orientation very close to 335^o sau 45^o, with more liability for the evaluated values. Total average annual littoral drift target value for the calibration equals 2.30 * 10^5 m^3/y. The seasonal littoral drift distributed over the dominant offshore wave propagation directions is summarized in Table 1.

The dominant offshore wave propagation directions for causing the littoral drift along the coast are SE and SSE. They cause total annual average quantities equal to 120,000 m^3/y and 96,000 m^3/y, respectively, or 52% and 42% of the total annual littoral drift in that area. For the offshore wave propagation direction coming from NE, it gives a smaller contribution, with 12,000 m^3/y or approximately 6% of the total annual ones. The offshore wave propagation direction coming from NNE occurs in both summer and spring. It causes an inverse littoral drift (with associated direction toward south, from noth to south) with a very small quantity compared to the ones caused by the other offshore wave propagation directions, 17,000 m^3/y or approximately 2% of the total. Similar conditions are existed for NE offshore wave propagation direction, with 3,000 m^3/y or approximately 1% of the total.

LST models are very sensitive to incident wave angle and height therefore detailed wave modeling or field measurements of wave conditions are essential for accurate results. Based on modeling carried out in the model (CERC, 1991) we find a mean summer incident swell angle of 45 ^{o}, and a mean winter incident swell angle of 335 ^{o}. Modeled wave angles and heights roughly match observed wave characteristics from field observations. The approach angle has a direct influence on the direction and magnitude of the LST rate and is one of the primary influences of seasonal transport of sediment in the study area.

In using the LST models and formulas, it is important to recognize each model's strength and weakness and utilize the formulas collectively as an interpretive tool rather than an absolute gauge of LST. Each model should be used in conjunction with at least one other formula in order to confirm the gross and net transport direction and secondly as a rough estimate of LST magnitude. We find this method works very well in this study and yields consistent results on the direction of transport. All the models we employed agree on the direction of seasonal gross and net annual LST even though they vary widely on the magnitude.

6. CONCLUSIONS

Surveyed beach profiles reveal a strong seasonal variability with net accretion in the summer and erosion in the winter. An alongshore-alternating pattern of erosion and accretion is identified from our beach profile surveys, manifested as large alongshore meanders.

Observations of gross seasonal sediment volume change reveal a nearly balanced longshore sediment transport system with the gross sediment loss at profile 9 accounted for by a nearly equivalent gain at profile 4 and confirms the presence of a strong seasonal longshore transport mechanism. We attribute the longshore transport of sediment from seasonal wave forcing with minimal cross-shore displacement. In general, south swells tend to decrease the total beach volume while north swell tends to induce volume recovery.

Three modern bulk formulae, which are CERC, Kamphuis, and Van Rijn are used to evaluate the longshore transport quantities and rates for the study area on the northern Mamaia coast on the Black Sea. The conditions for their satisfactory application are explored and found in the study area.

The Kamphuis formula provides quite acceptable evaluations for the annual littoral drift quantities (m3/y) and also their associated rates (kg/s) on both the seasonal and offshore wave propagation directional bases. No correction factor is needed for the Kamphuis formula since its evaluations can be considered quite realistic and satisfactory for the study area compared to the available target values.

For the original CERC and Van Rijn formulae, the evaluated quantities are considered quite high. Suitable correction factors were determined to make their evaluations quite realistic and acceptable in terms of agreement with the available target values from literature. These correction factors (Cor_{CERC} = 0.20 and $Cor_{Van\ Rijn}$ = 0.30) are applied and give satisfactory preliminary evaluations for the littoral drift quantities for the study area. Applying these correction factors, the evaluated seasonal longshore transport quantities based on offshore waves propagating coming from North, NNE, NE, and SE (m^3/y) are considered quite close, reasonable, and acceptable compared to the available historical reference ones. To calibrate these evaluations against each other, the relationship between both the evaluated annual longshore transport quantities (m^3/y including pours) and the associated offshore significant wave heights (values used in evaluations) are carried out on both directional and seasonal bases. The three modern bulk formulae after the provided corrections give quite acceptable, realistic, and close results. From the evaluated results, it can be concluded that the evaluated longshore sediment transport quantities and rates are very sensitive to any little variation in the incident wave angle (offshore wave propagation orientation to the north). Less sensitivity is recognized on these evaluations for wave period ones.

More sensitivity analyses by using field measurements are required for the longshore transport quantities in the study area. This confirms this study's main conclusion, that the Kamphuis formula is the best available formula for evaluation in that area. The modified Van Rijn formula comes after, and CERC comes at the last priority for evaluation accuracy, and so applicability for that area compared to available the target values.

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Hydro-technical Constructions in the Romanian Coastal Zone

Gelmambet Sunai and Omer Ichinur

***Abstract* – This work is part of a background study on the current state of NW coastal zone of the Black Sea Basin, consisting in the establishment of a reference database in this area, concerning hydro-technical constructions and existing coastal protection. The interest zone, analyzed in this paper, represents Romania's entire coastline stretching from the Musura Bay, at north (border with Ukraine) and Vama Veche, at south (border with Bulgaria). The current state of the Romanian coast was influenced by the coastal protection measures, protective breakwaters harbors, development of industrial capacities in the coastal zone, the construction of reservoirs and dams in the rivers basins. To stop the erosion of beaches and the stability loss of cliffs, there are urgent necessary the rehabilitation works and the improvement of existing coastal protection.**

***Keywords* – coastal zone, hydro-technical constructions, coastal protection, erosion.**

1. INTRODUCTION

The objective of this paper is a substantiating study on the current state of NW coastal zone of the Black Sea Basin, consisting in the establishment of a reference database in the zone concerning hydro-technical constructions and existing coastal protection.

The interest zone, analyzed in this paper, represents Romania's throughout coastline which extends from the Gulf Musura at north (border with Ukraine) and Vama Veche at south (border with Bulgaria). The relief of this zone is characterized by the shores of low altitude - beaches (80%) and relatively high shores - cliffs (20%).

From the geomorphologic point of view, Romanian coast is divided in two main zones:

- Nordic area between the Musura Bay and Cape Midia - Năvodari, having about 165 km length.
- Southern area between Cape Midia - Năvodari and Vama Veche having about 80 km length.

The Romanian Black Sea's coastline extends over a length of 245 km between the Musura arm to the north and Vama Veche to the south (6% of the total length of the Black Sea’s coastline).

From the typological point of view, includes both: the natural shoreline, about 84% (beaches and cliffs) and the shore-built, about 16% (ports, hydro-technical constructions of protection).

Romanian coast has two aspects:

- the shoreline - down (in northern area - almost 165 km length), from Chilia Arm's to the Midia Cape. Including Delta and Razim - Sinoe Lagoon, is characterized by sandy beaches, low altitudes and less steep submarine slopes.
- the shoreline - high (in southern area - 80 km length) from Cape Midia to Vama Veche, is covered mostly by cliffs with various heights, between 3 and 35 meters, short sections of sandy beaches at the mouths of rivers

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and harbors (Midia, Constanta, Mangalia) and submarine slopes steeper than those from the northern area. Along the southern zone, the unstable soft rock cliffs are likely to collapse in landslides.

2. THE HYDRO-TECHNICAL CONSTRUCTIONS AND COASTAL PROTECTION EXISTING IN THE SOUTHERN AREA OF THE COASTLINE

VAMA-VECHE: There are no hydro-technical constructions and coastal protection elements (fig.1).

This sub-sector extends to Romanian border with Bulgaria. The sub-sector includes cliffs and sandy beach. The beaches are submitted to erosion and the cliffs are susceptible to landslides.

Fig. 1. Cliff erosion in Vama Veche

Fig. 2. Beach and cliff erosion in 2 May

2MAY: There is a breakwater made by rocks and reinforced concretes, which is routinely used as a wharf by local fishermen. The status dams are degraded. Otherwise, there are no coastal protections (fig.2)

The impact of construction on the environment: In front of the second beach from 2 May "pocket", the alluvial transport direction is suddenly changed, the influence of breakwaters is reduced and the longitudinal alluvial transport resumes the southern general route. The fishermen's dike from 2 May, which is a break-wave type, has an influence located on these movements, but the southern net regime of the alluvial transport continues to the Romanian territory.

Compared to the maps from 1960 to 1979, there are major changes in the shoreline appearance of this sub-zone. Due to the breakwater from Mangalia harbor, there is a strong erosion of the beaches and the waves reaching at the cliff produces the caving-in, endangering the nearby buildings. In the northern end of this sub-sector, due to the Mangalia's breakwater influence, the beach is relatively stable. The other parts are affected by a net tendency of erosion, accompanied by the loss of beach sediments. The cliffs are likely to collapsing through erosion and landslides. There are proposed the rehabilitation works of existing breakwater and the protection of the bottom cliff, where is necessary, with measures to reduce the impact on the habitats.

MANGALIA: There are rock fill breakwaters protected by concrete blocks which delimits semicircular the beach. The shape of the northern breakwater is T and of the southern part is Y. In the north part, the beach narrows and is protected by rock fill. The breakwaters of the Mangalia harbor and the tourist harbor have the nucleus and the mantle made with the rock fill protection and in the sea shall they are protected by concrete blocks shell. At the top of the breakwaters and along their length, there is a concrete coping.

The impact of construction on the environment: There is erosion at the north of the beach and deposition at the south, as a result of the breakwaters which separate the beach. For this reason, at the north of the beach was realized a rock fill protection (fig. 4.) The breakwaters of the Mangalia harbor have affected the transport along the shoreline and the general circulation of sediments, some of these being transported to the offshore.

Mangalia's harbor influences the cliffs and beaches erosion between 2 May and Vama Veche village. At 2 May and in the central part, where are narrow golf beaches, less than 10 meters in the summer / the calm season period, the shoreline retreats during the storms are near the base of the cliff. The exception is the northern section of the 2 May's beach, which is protected by the southern dike of Mangalia.

Fig. 3. Mangalia dams

Fig. 4. Mangalia Beach

Like other harbors situated in this section of the coast, the Mangalia's harbor is an impermeable boundary for sediment transport. Immediately at the south of the break-wave dike is a small strip of beach stable or slightly increasing for a few tens of meters length. This represents a small sedimentary basin as a result of a sheltering effect and it has an impact on seaward wave generated by the break-wave dike from Mangalia's harbor. There are proposed the repair works and improvement of existing protections. There is an artificial sanding of beach, 20 m width between breakwaters, in front of President Hotel.

Fig. 5. Dike from the south part of the Saturn beach

Fig. 6. Saturn's Dikes

SATURN: The beach consists of four coves protected by five rock fill dikes in T shape which are protected with concrete blocks. This area of the coast, between Cerna Hotel and Mangalia, is characterized by a number of well-dammed intervals supported by modified dikes break wave and artificial promontories structures.

The impact of construction on the environment: The dynamic of beach is almost entirely controlled by the human intervention. The beaches are narrow and have been protected by dikes against erosion. There is a slight degradation of the dikes. There are proposed rehabilitation works and improving the existing protections.

JUPITER-CAP AURORA-VENUS: At Jupiter, Cap Aurora and Venus, there are a number of bays closed and protected by modified breakwaters and structures of artificial promontories with occasional reefs offshore. From place to place, there are stone clothing and breakwaters behind beaches generally narrow. The dikes are realized from rock fill and protected with concrete blocks. The Cap Aurora shoreline (between Jupiter and Venus) comprises a series of rock-dikes and a submersed breakwater. The result was the creation of a series of artificial coves.

The impact of construction on the environment: The dynamic of beach is almost entirely controlled by the human intervention. In general, the beaches are narrow less in the south of the resort Venus where is found a large beach. Beaches suffer slight erosion and the structure protections have degradation.

There are proposed rehabilitation works and improvement of some breakwaters, removal of some existing structures for to widen bays, except the two break-wave dike situated in front of Carmen Hotel. For the protection and the sanding beach, there are necessary rehabilitation works, improvement and construction of new structures. Also there are necessary to repair and to improve the two break wave dike in front of Carmen Hotel.

Fig. 7. Rock fill Dike from Cap Aurora

Fig. 8. Venus Dike from rock fill with concrete blocks

NEPTUN: The coastal protection system, between Olympus and Mangalia, consisting of break wave dikes, rock dyke, breakwaters, produces also changes in water circulation templates and sediments. Throughout this sector, between Tatlageac Fishery and Silvia Hotel, a succession of beaches which are controlled and maintained artificially exists. These beaches are sustained by break-wave dikes structure modified and artificial promontories, with occasional offshore reefs. The shoreline sector Olimp - Neptun is characterized by the existence of six detached breakwaters and a submersed breakwater parallel to the shore. The breakwaters show traces of degradation.

The impact of construction on the environment: The dynamic of beach is almost entirely controlled by the human intervention. In general, the beaches are narrow and are eroding due to lack of new sediments and the structures which fails. There are proposed the rehabilitation and improvement of breakwaters, removal of some existing structures for wide bays, construction of new protection structures and beach sanding.

Fig. 9. Neptun's Dike

Fig. 10. The beach and the dikes from Olimp

OLIMP: The Olimp Resort's beach is small consisting of bays protected of three rock fill breakwaters. The dykes exhibit degradation, requiring repairs and strengthening.

The impact of construction on the environment: The protection dikes proved efficient protecting the beach against erosion. There are proposed removal works of some existing structures for wide bays, the rehabilitation, improvement and construction of new structures for protection and sanding the beach.

23 AUGUST: There are no hydro-technical constructions and no coastal protection elements.

The impact of construction on the environment: There are only very narrow beaches at the base of the cliffs. The cliffs from soft loess are, therefore, vulnerable to the waves attack and are prone to landslides under the wave's action (fig.11).

COSTINESTI: New dikes break-wave protecting the entrance / outlet of the lake. There is one rock fill breakwater protected with concrete blocks in the south of the resort.

The impact of construction on the environment: Beach it widens toward south and currently is stable, generally throughout central section and is eroding at south of the new constructed breaking wave dikes. These dikes appear to be interrupted the natural coastal transport regime. There are started the consolidations works of the cliffs and the narrow beach areas are protected with stone. The unprotected areas are prone to landslides. There are proposed the rehabilitation works, the improvement and construction of new protection structures on medium term and sanding artificial beaches on long term.

Fig. 11. 23 August's unprotected cliff

Fig. 12. Dike of rock fill protected with concrete blocks in Costineşti

TUZLA: New works protection and protective to seawall with reinforcement stone. Throughout this sub-sector, between Fishery, South Eforie and Tuzla Cape, the cliffs are protected by new protection works. In front of the cliffs the beaches are very narrow and are protected by stones. The stones protection works are in progress and will be completed in 2013. (fig. 13).

The impact of construction on the environment: Throughout this sector the cliffs have been prone to landslides. The new protection works will help to reduce the erosion of these cliffs, but will not prevent further the narrowing of the beaches due to the lack of the new sediments and the various coastal structures upstream that interrupt any drift throughout shoreline.

Fig. 13. The protection works with stone-Tuzla

Fig. 14. Dike in South Eforie with the new costal protection

SOUTH EFORIE: There are a number of narrow sandy bays between the modified breakwaters and the structures of artificial promontories from stones and concrete blocks. In general, the beaches are eroding due to the reduced sediments supply and the protection structures which doesn't resist, that they are no longer efficient to keep the beach. They are also prone to erosion during the storms. In eroded beach areas are started the consolidation and the protection of these cliffs with stones .

The impact of construction on the environment: The coastal protection system in front of South Eforie also led to the disturbance of the general transport directions and rates; however, it no longer produces effects because the protection system is worn morally and technically. In most places, at the base of the cliff the beach does not exist, except for some small sections which lasts between the coastal structures. Even here, the protection structures do not resist, which means that they are not efficient and cannot maintain the beach. Therefore, the net trend is erosion, cliffs landslides are likely. There are proposed rehabilitation works, the improvement and construction of new protection structures with sanding beach.

NORTH EFORIE: One modified rock dyke from rocks in the northern end. There is natural erosion at the south of the platform with little or no beach. Farther south beach is narrow, sustained by a series of five rock dyke which are made in 1956-1960 to extend the beach. The cliff is protected with clothing stone from place to place and submersed breakwaters. At the southern end a marina was built in 1986 (Yacht Club Europe) and helps maintain a relatively wide beaches and in good condition, in the upstream of coastline stream. There are break-wave structures submersed off the coast, over the southern half of the front.

The impact of construction on the environment: The construction of the Constanta and South-Agigea's Harbour had a negative impact on the North Eforie sector through the beach erosion and the dikes built for protection, the stop of the beach erosion were ineffective. The marina had a strong impact on the behind of the beach, leading to a significant enlargement, but in the south of the marina is observed an erosion area. There are proposed rehabilitation works, the improvement and construction of the new protection structures with sanding of the beach.

Fig. 15. North Eforie’s Dikes

Fig. 16. Agigea beach- The North Dike

AGIGEA: There are the Agigea harbor protection dam and dykes of protection of the beach. The port's protective dike is made of rock fill and is protected with concrete blocks. Protection breakwater in the form of T from the south of the beach is achieved of rock fill with concrete slab and shows local degradations.

The impact of construction on the environment: Due to the protection breakwater of the Agigea harbor the beach is protected only partially being exposed in the south, where the T-shaped dike protects not enough beaches. No significant changes observed the beach.

TOMIS NORD - TOMIS SUD: This sector stretches between Fishery, Renaissance Street, Tomis Harbour, Constanta Harbour and is characterized by cliffs in front of which there is a narrow beach. The shoreline consists of a series of bays sustained by promontories created from the break wave dikes. Are present also at the southern end submersed reefs and breakwaters in Tomis Port. This sector also includes the tourism beach in Constanta ("Modern" beach), miniport Tomis and entirely artificial coast at south of Miniport , to the Port of Constanta. The sector is bounded by a long breakwater to the north and there is a new consolidation work with stone, includes a small beach or no beach, south of the breakwater in the shape of "L" in the northern end of the front. Further to the south, the beaches are backed by wide promontories backed by steep ends created of rock fill dikes with enlarged ends in the T-shape, achieved of stone and reinforced concrete, foreseen occasional with stone clothing to protect the foot cliff.

The impact of construction on the environment: The coastline in the front of Constanta is exposed mainly north-eastern and eastern waves, but and the southeast waves play an important role. All coastal protection works, locally influences also alluvial transport. The high dike, which forms the northern border of this sector, and land surface reused significantly, had discontinued the transport of material along the shoreline, resulting to increase of the beach erosion. These structures, together with the coast line to the south to have resulted in a small bay. Further south, shore is maintained artificially by a series of heavy coastal structures that have significantly affected the natural dynamics of the coast. At the southern end of the unit, there are unprotected steep cliffs. The cliffs are composed of poorly consolidated sediments and are therefore unstable natural and prone to landslides, although south of the L-shaped dike and north of the bay, the new constructions provide their protection localized. There are proposed protection works combined with rehabilitation against erosion and increasing dimensions of beach by artificial sanding.

Fig. 17. Hydrotechnical structures in the Tomis Nord area-[1]

Fig. 18. Break-wave structures offshore in Mamaia-[1]

MAMAIA: There is coastal protection structures of six break wave dikes (fig.18) located in the central area and in the southern part of the Mamaia resort. The dikes are made from rock fill and protected with concrete blocks.

The impact of construction on the environment: In the central and south part of the Mamaia resort, behind to those six breakwaters, the shoreline exhibits special characteristics induced of the sediment transport in behind of the coastal structures. This sector is stretching between Hotel Rex and Fishery and is protected by a series of detached breakwaters. The coastal protection works influence the local alluvial transport and help maintain beaches along this front. However, breakwaters seem to have been ineffective in preventing of the local erosion and are also in poor condition. The beach behind the interspacing between breakwaters is narrowing, and the beaches are characterized by a general erosion phenomenon. The dikes system (reef) seems that concentrate the erosion to the north and south of the structures, resulting a narrowing of the beaches in front the empty spaces between dikes. The dikes haven't been effective in preventing the local erosion and are also in a very bad state. The beach is therefore the primary means of protection for assets and lake. There is also a beautiful view pontoon with "foot-passenger Bridge" to the south. There are proposed construction works of new dikes to the beach stability/ break wave dikes of stone and performing works sanding the beach, on the 60 m in width.

3. CONCLUSIONS

The present study is part of the first stage of the project PN-II-PT-PCCA-2011-3 (ECOMAGIS) "**IMPLEMENTATION OF A COMPLEX GEOGRAPHIC INFORMATIC SYSTEM FOR ECOSYSTEM-BASED MANAGEMENT, THROUGH INTEGRATED MONITORING AND ASSESSMENT OF THE BIOCOENOSIS STATUS AND ITS EVOLUTION TRENDS IN A FAST CHANGING ENVIRONMENT AT THE ROMANIAN COASTAL ZONE OF THE BLACK SEA".**

There are a number of human activities that have influenced the Romanian coast, such as coastal protection measures, protective breakwaters harbor, development of industrial capacities in the coastal zone, the introduction in ecosystem of not indigenous species, and the construction of reservoirs and dams in the river basin. The costal interventions have affected deposits sediment. Breakwaters harbors Midia, Constanta and Mangalia in turn affect the transport along shoreline and the general circulation of sediments, parts of these being transported to offshore. Midia Port has influenced especially Mamaia beach and partially north beaches of Tomis marina. The Constanta Port has a major influence on the sea walls and beaches of North Eforie and South Eforie, while Mangalia Port affects the sea walls and the beach erosion of 2 May and Vama Veche.

Following the completion of the Master Plan "Coastal protection and rehabilitation. Technical Assistance for Priority Projects. Major field of intervention 2: Axis 5" Reduction of coastal erosion " of A.N.R Romanian Waters prepared by Halcrow Romania S.R.L. in December 2011 and the presentation of the report "COASTAL PROTECTION PLAN FOR THE SOUTHERN ROMANIAN BLACK SEA SHORE" elaborated by The JAPAN INTERNATIONAL COOPERATION AGENCY ECOH CORPORATION in March 2006 the South Mamaia and North Eforie areas shall be given the priority on the rehabilitation work and improvement of existing coastal protection. Following this study it is observed that besides the mentioned areas as priority areas are also serious affected South Eforie and 2 May. For this reason should be considered also rehabilitation work and improvement of existing coastal protection.

4. ACKNOWLEDGMENTS

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Waves regime in Romanian Coastal Zone

Omer Ichinur and Gelmambet Sunai

***Abstract* – One of the important factors contributing to coastal erosion is the wave. Romania's coastline has a length of approximate 247 km. This paper presents an analysis of measured data on waves, these measurements being made in 2003-2010 by NIMRD (National Institute for Marine Research and Development "Grigore Antipa" Constanta). In the studied period, the frequency on direction of the wave is the highest value SE and NE. Wave frequency is higher in cold months (October to March), the highest value recorded in January (45.7%). We present also an analysis of data from the waves measured in 2010, setting the average values of the main parameters of the waves.**

***Keywords* – waves regime, the coastal area, frequency of the waves.**

1. INTRODUCTION

One of the important factors that lead to changes in the Black Sea shore is the wave dynamics. The waves are movements of surface water with linear propagation, which causes the entire mass of water, propagated radials. The hydrodynamic energy wave induces significant effects, the erosion or the deposition of solid materials. The interest area, analyzed in this paper, represents Romania's entire coastline stretching from the Musura Bay at north (border with Ukraine) and Vama Veche at south (border with Bulgaria).

Fig. 1. Studied area ([1])

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From geomorphologic point of view, Romanian seaside is divided into two main areas (fig. 1):

1. ***The Northern Zone***, between the Musura Bay and Midia - Năvodari Cape, having about 165 km length;
2. ***The Southern Zone***, between Midia - Năvodari Cape and Vama Veche, having about 82 km length.

2. MEASURED DATA ANALYSIS OF WAVES IN THE COASTAL ZONE

To study the regime of waves in the coastal zone, there were processed 6144 data obtained from measurements made in 2003-2010 by National Institute for Marine Research and Development "Grigore Antipa" Constanta (NIMRD). The monthly distribution measurements per year are shown in Fig. 2.

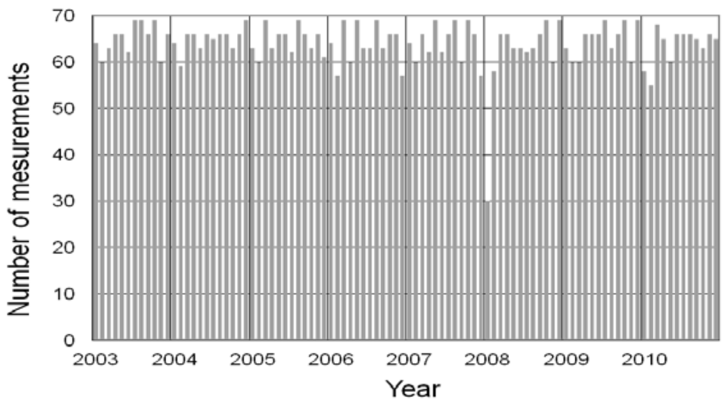

Fig. 2. The monthly number of measurements of wave parameters (2003 - 2010)

It is represented the annual average and monthly frequency of wave, expressed as a percentage per cent, the average being calculated for the 2003-2010 study period (Fig. 3 and fig. 4).

From fig.3., it results that the wave frequency is higher in cold months (October to March), the highest value was recorded in January (45.7%).

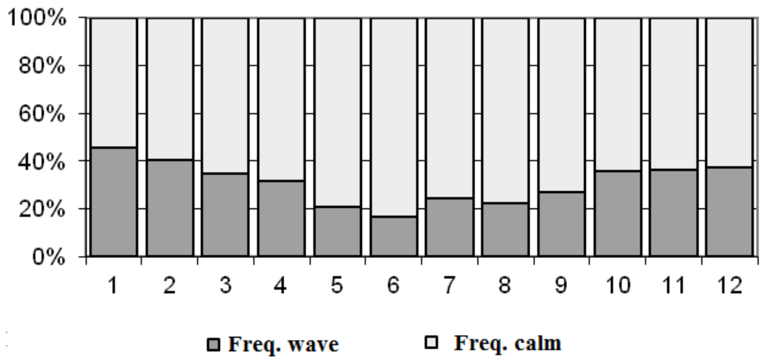

Fig. 3. Average monthly frequency of the waves (2003 – 2010)

It is noted that in 2010 were recorded the highest amount of wind waves and swell frequency (39.1%) and lowest in 2008 (18.8%).

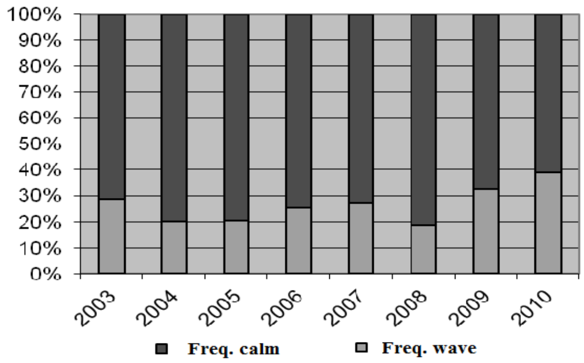

Fig. 4. The average annual frequency of the waves (2003 – 2010)

From the characteristic dynamics of coastal processes such as circulation currents, pollution dispersion etc., the wave direction is an essential factor. The only method that might associate these measurements and direction is the correlation with wind data. Thus, in 2003-2010, the frequency of the wave direction has the highest value SE and NE.

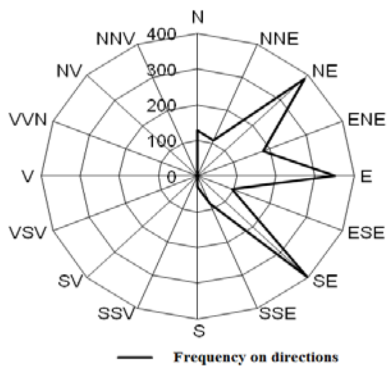

Fig. 5. Frequency wave directions (2003 - 2010)

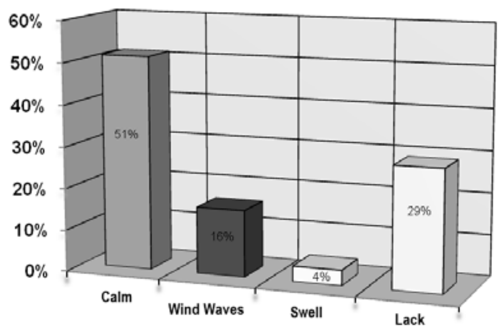

Fig. 6. Annual average frequency of the waves (2003 - 2010)

In 2010, it had the highest frequency waves from the North (73%), both in winter (30%) and in summer (43%).

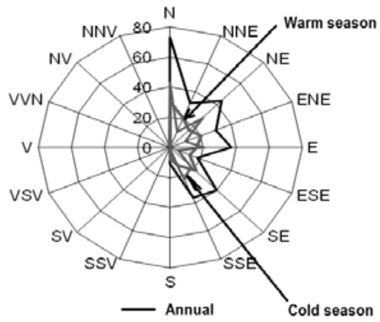

Fig. 7. Frequency wave directions (2010)

From the 1069 observations in 2010, 67% are calmly situations and missing data, and 33% heavy sea (wind waves and swell, Fig. 8.).

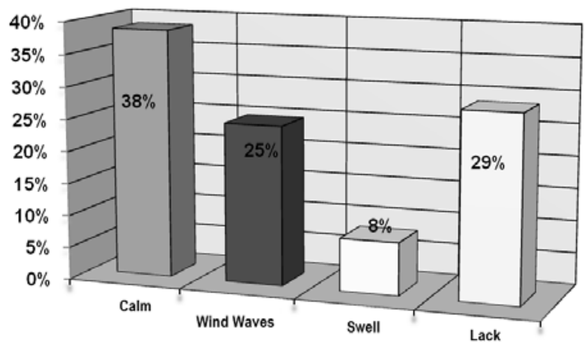

Fig. 8. The average frequency of the waves (2010)

In summer, 69% are calmly situations and missing data, and 31% heavy sea, while in winter were recorded 65% calmly situations and missing data, and 35% nervous sea.

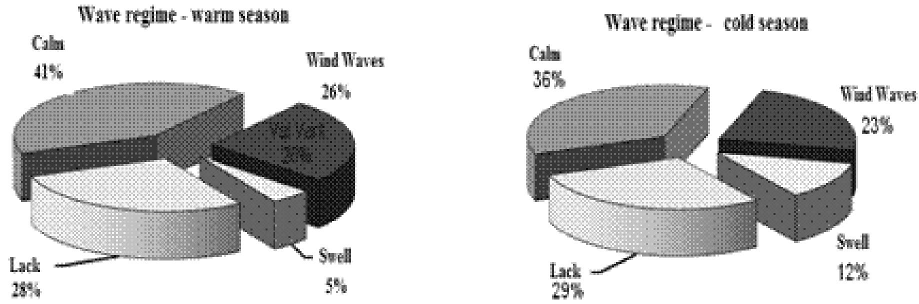

Fig. 9. The average frequency of the waves in the warm season and cool season (2010)

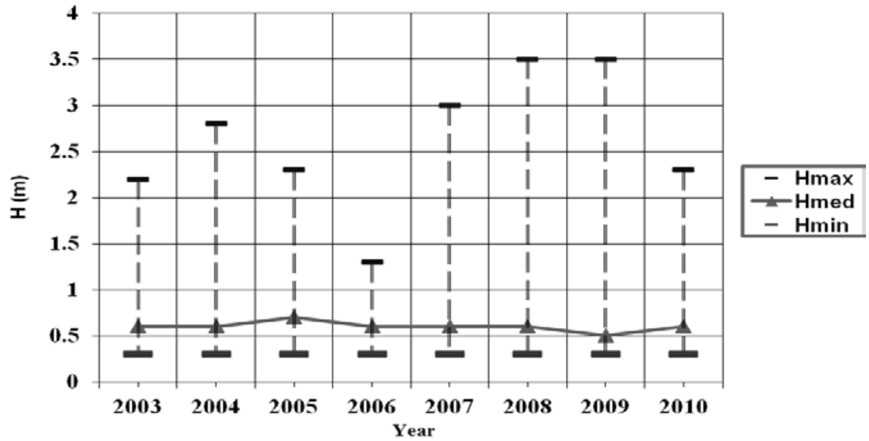

Fig. 10. Maximum and minimum annual heights and the medians (2003-2010)

The statistical analysis of wave height (Fig. 10.), in 2003-2010 periods, shows the following:

– maximum wave height of 3.5 m was recorded in 2008 and 2009;
– minimum wave height of 0.3 m was recorded in all years of the period studied;
– Multi median value is 0.62 m.

Regarding the wave parameters in 2010, it is found that the mean values were:

– wind wave height of 0.6 m and 1 m for summer winter;
– swell wave height of 0.6 m to 0.9 m for the hot season and winter;
– the period wind waves 4s for 4.6 s summer and winter;
– surges between 6.2 s and 6.9 s for the hot season for the cold season;
– wavelength of the waves of 6 m to 8.4 m for the warm season and cold season
– wavelength of surges 13.5 m in summer and 14.75 m in winter.

CONCLUSIONS

The dynamic of waves is a very important factor with significant effects on the erosion or the deposition of solid materials; therefore it leads at the shoreline changes,

We presented an analysis of measured data on waves, these measurements being made in 2003-2010 by NIMRD (National Institute for Marine Research and Development “Grigore Antipa” Constanta).

By the analysis these wave measurements, it was found that:

– The frequency wave direction has the highest value in SE and NE direction;
– The wave frequency is higher in cold months (October to March);
– The highest value was recorded in January (45.7%);
– The maximum wave height was recorded in 2008 and 2009 (3.5 m) and the minimum height of the waves was recorded in all the years of the study period (0.3 m).

It was realized also an analysis of the wave regime in 2010 and we concluded that 67% are calmly situations and missing data, and 33% are heavy sea situations.

4. ACKNOWLEDGMENTS

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SECTION IV

INTEGRATE WATER MANAGEMENT

ENVIRONMENT PROTECTION

A New Life for the Old Pumping Stations?

Petar I. Filkov, Georgi N. Nachev, Gergina D. Mihaylova, Georgi D. Tonchev

***Abstract* – The political and economic changes in Bulgaria after 1989 seriously affected the agriculture – reduction of arable land, damaging of irrigation devices and structures, including pumping stations (PS), etc. The maintenance companies spend part of their insufficient financial resources to keep the PS from damages and robberies. In the last years usage of renewable energy sources becomes more and more popular, so the interest for building new Water Power Plants increases. The idea for establishing joint ventures between the State maintenance companies and private companies for reconstruction of some of the existing main PS to Power Plants rose. The variant for turning a PS to a Pumped Storage Power Plant is examined in the article and a case study is presented.**

***Keywords* – pumping station reconstruction, pumped storage power plant.**

1. INTRODUCTION

Since the political and economic changes in Bulgaria in 1989 the agriculture has been in serious crisis. Due to refusal of irrigation, lots of irrigation structures became nonoperational. The most serious problems were with Pumping Stations (PS), as the State maintenance companies have too big expenditures for their maintenance. Considering the interest of the private companies in renewable energy sources for building, the idea of reconstruction of the existing PS to Water Power Plants rose. Establishing a joint venture between the state and the private companies would help the former side to get rid of the useless expenditures, and the latter – to reduce the investments.

The Main irrigation PS, situated by the river, can be reconstructed in two ways: as “Run-of-the-River” Power Plants (RRPP), as presented on **Fig. 1a**, or as Pumped Storage Power Plants (PSPP) , shown on **Fig. 1b**.

The first way is much easier and faster for realization, much more profitable, so it is a preferable from the economic point of view. The new RRPP will operate at low head and big discharge, so great installed power of the plant can be achieved. The investments will not be so significant, as far as the barrage, the inlet structure and the building exist. Only the costs for the new equipment, the reconstruction of the PS building and the construction of a new outlet structure for the Power Plant should be taken into account. The constraints for such transformation of the existing PS are the size of the building and its position in the site. The disadvantage is that the PS cannot be used again for its original purpose.

The second way of reconstruction is much more complicated, lots of circumstances should be taken into account, but it could be profitable in some cases. The new PSPP will operate at approximately the same head and discharge as original PS, so installed power is roughly set in advance. The pressure pipe (penstock) and the

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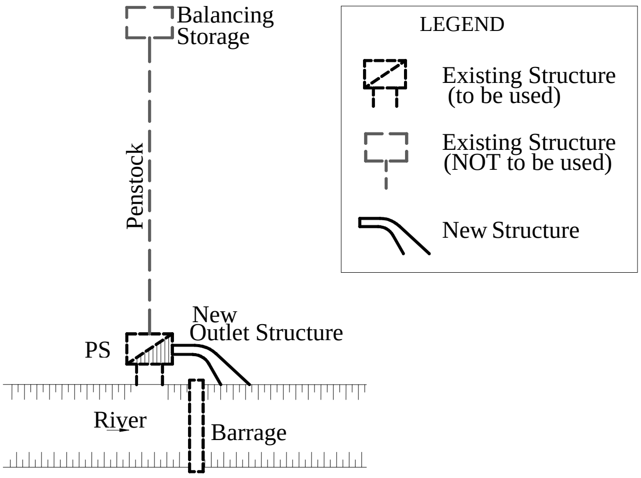
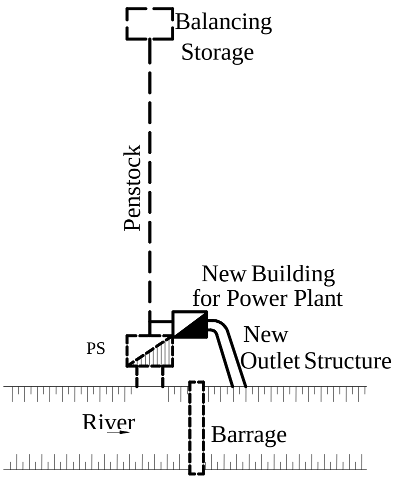
Balancing Storage exist, so no investments for them are needed, but they set additional constraints for the reconstruction, regarding the discharge and the treated volume of water per day. The investments depend also on the existing pumping equipment – if it is usable or not, if the pumps can be used as turbines and if they can be coupled both with motors and generators, etc. The biggest advantage of such a way of reconstruction is that in the future the PS could be used for its original purpose, or to have combined operation – as a PSPP and as a PS.



(a) Reconstruction to Run-of-the-River Power Plant



(b) Reconstruction to Pumping Storage Power Plant

Fig. 1. Options for reconstruction of an existing PS to Power Plant

Turning a PS to a RRPP does not provoke much of interest, from the designers and scientific point of view, because it resembles to developing a new project with not so many constraints. Much more interesting is the reconstruction of a PS to a PSPP.

2. THEORETICAL ASPECTS OF THE PROBLEM

The treated amount of water by the PSPP per day W_d is provided by the PS and it is estimated as:

$$W_d = 3600 Q_{PS} t_{PS}\text{, m}^3\text{/d,} \tag{1}$$

where Q_{PS} is the PS discharge, m^3/s;

t_{PS} is the PS time of operation, h/d.

Considering that the PS should operate only when the energy price is the cheapest, the maximum treated volume will be estimated assuming t_{PS} to be equal to the night tariff period duration t_n.

The first constraint for the reconstruction is the available Balancing Storage capacity V_{BS}. If V_{BS} is smaller than W_d additional investments should be made for enlargement of the storage. In some cases it is not possible to enlarge this storage, so t_{PS} should be shortened.

The second constraint regards the turbines discharge Q_T and it is derived by (1):

$$Q_T = Q_{PS} \frac{t_{PS}}{t_T}\text{, m}^3\text{/s,} \tag{2}$$

where t_T is the time of operation of the turbines, h/d.

There are three variants for estimation of Q_T. Variant A is to have equal discharges of the pumps and the turbines. But, considering that the night tariff period duration t_n in Bulgaria is 8 h/d and the peak tariff periods t_p are 6 h/d total, two alternatives are possible. The first one is turbines to operate additional 2 h/d during the daylight tariff period. In this case the maximum value of W_d is achieved and additional investments may be needed, but if the energy selling price is the same for the peak and the daylight tariff periods, the economic effect can be positive. The second alternative for variant A is to shorten t_{PS} and in such a way to decrease the volume W_d. Variant B is to assume $t_T = t_p$ and to estimate Q_T by (2). This will be the maximum value of Q_T, so the velocity in the existing penstock should be checked. Variant C is to accept Q_T between Q_{PS} and the maximum value and to estimate t_{PS} using (2). The most preferable variant, considering investments, is to have no replacement of the penstock and no or minimum enlargement of the balancing storage. But from the point of view of the incomes it is better to have maximum W_d. These two tendencies are reciprocal, so optimization estimations are necessary.

The daily income from energy production I_E can be estimated as follows [1], [2]:

$$I_E = 0{,}002725 W_d H_T \eta_T \eta_G s_P\text{, €/d,} \tag{3}$$

where H_T is the actual turbines head, m;

η_T and η_G are the turbines and generator efficiencies;

s_p is the energy selling price during operation period, €/kWh.

Analogically the daily energy expenses for PS are [3]:

$$E_E = \frac{0{,}002725 W_d H_P}{\eta_P \eta_M} c_P\text{, €/d,} \tag{4}$$

where H_P is the actual pumps head, m;

η_P and η_M are the pump and motor efficiencies;

c_p is the energy purchase price during operation period, €/kWh.

The energy production revenue estimated for 360 days per year operation of the PSPP is:

$$R = I_E - E_E = \frac{0{,}981 W_d H_P c_P}{\eta_P \eta_M}\left(\frac{s_P}{c_P}.\frac{H_T}{H_P}\eta_P \eta_M \eta_T \eta_G - 1\right), \text{ €/year,} \quad (5)$$

or

$$R = \frac{0{,}981 W_d H_P c_P}{\eta_P \eta_M}\left(e_{PSPP} - 1\right), \text{ €/year,} \quad (6)$$

where e_{PSPP} can be named “total efficiency of the Pumping Storage Power Plant” and it is defined as:

$$e_{PSPP} = e_F e_E \,, \quad (7)$$

as e_F and e_E are so called “financial efficiency” and “energetic efficiency”, defined as follows:

$$e_F = \frac{s_P}{c_P}\,, \quad (8)$$

and

$$e_E = \frac{H_T}{H_P}\eta_C \,. \quad (9)$$

where η_C is the combined mechanical efficiency of the PS and the Power Plant:

$$\eta_C = \eta_P \eta_M \eta_T \eta_G \,. \quad (10)$$

Obviously, to have positive revenue, it is necessary to have:

$$e_{PSPP} > 1\,, \quad (11)$$

or

$$e_E > \frac{1}{e_F}\,, \quad (12)$$

The financial efficiency e_F cannot be changed, because the prices are governed by the state. This efficiency is “positive”, i.e. greater than one, because the selling price for peak periods is greater than purchase price for the night period. For Bulgaria this ratio, without VAT, is approximately e_F = 1,9. Taking into account that the combined mechanical efficiency η_C varies between 0,60 and 0,68, the general condition for a positive economic effect, derived from (9) and (11) is:

$$\frac{H_T}{H_P} > 0{,}77 \div 0{,}88\,. \quad (13)$$

If assumed that the geodetic head H_G is the same both for the pumps and the turbines, and the head losses h_L in pumping regime are the same as those in turbine regime, the condition (13) is transformed to:

$$h_L < (6\% \div 12\%) H_G \,. \tag{14}$$

3. CASE STUDY

The PS "Sofronievo 1" is situated near Sofronievo village, on the right bank of Ogosta River, Northwest Bulgaria. It was designed to deliver water for irrigation of approximately 2800 ha south of Misia Town. For the needs of water intake a barrage was built across the Ogosta River. The PS was designed to deliver 1,72 m^3/s, at total head of 74,0 m and was equipped with 4 split case pumps with vertical axis, situated below the lower water level. The diameter of the penstock is 1000 mm and it is in good condition, as well as the balancing storage with volume of 30000 m^3. The main parameters of the existing PS are given in the third column of **Table 1**. The PS has not been in operation for more than 15 years.

Table. 1. Pumping Station "Sofronievo 1" Main Parameters (Current State)

Parameter	Units	Value
Upper Water Level	m a.s.l.	103,0
Lower Water Level	m a.s.l.	34,0
Geodetic Head - H_G	m	69,0
PS Design Discharge - Q_{PS}	m^3/s	1,72
Number of Units	-	4
Pump Discharge Q_{PA}	m^3/s	0,43
Pump Actual Head H_P	m^3/s	74,0
Pump Motor Nominal Power	kW	500
Balancing Storage Capacity V_{BS}	m^3	30000
Balancing Storage Depth H_{BS}	m	2,0

The initial idea was to reconstruct the PS "Sofronievo 1" as a Run-of-the-River PSPP. The displacement between the water levels before and after the barrage was more than 2,5 m. This idea was abandoned, because before it was stated out loud, the Danube River Basin Directorate had issued a license for building a Run-of-the-River Power Plant 1,8 km downstream the barrage of the PS. The admissible water level for the new PP was 2,6 m higher than the elevation of the spillway crest of the barrage, i.e. it would be submerged.

It was decided the possibility for reconstruction of the PS to a PSPP to be considered. The PS building, penstock and the Balancing Storage were in relatively good repair, but the rehabilitation of these structures and of the pumping equipment were necessary.

The PSPP is to be equipped with one Francis turbine with a design discharge of 2,0 m^3/s, placed in a new building next to the existing one. A diversion from the existing penstock should be provided and a new outlet structure should be constructed.

The Balancing storage should be enlarged to 43200 m^3, which is equal to the treated daily amount of water. The maximum water level in the Balancing Storage should be raised with 0,40 m, and the bottom should be lowered by 0,5 m. A scheme of the specific water levels and actual heads is shown on **Fig. 2**. The main parameters for both pumping and turbine regimes of operation are presented in **Table 2**.

It is accepted that the Upper Water Level for both regimes is equal to the average water level in the Balancing Storage. The difference between Lower Water Levels is due to head losses in suction sump and in the outlet structure.

As it is seen from the **Table 2**, because of the decrease of the geodetic head H_G, the PS discharge increases from 1,72 m^3/s to 1,80 m^3/s. In pumping regime the head losses of 6,2 m are 9,4% from the pumping H_G and for the turbine regime the head losses of 6,4 m are 9,8% from the respective H_G, so the condition (14) is fulfilled in both cases, but it is close to the upper (unfavorable) limit of the given range. It should be noted, that the selling energy price s_p of 0,08318 €/kWh used in **Table 2** is a constant for all tariff periods, according to the rules of the State Energy and Water Regulatory Commission [4].

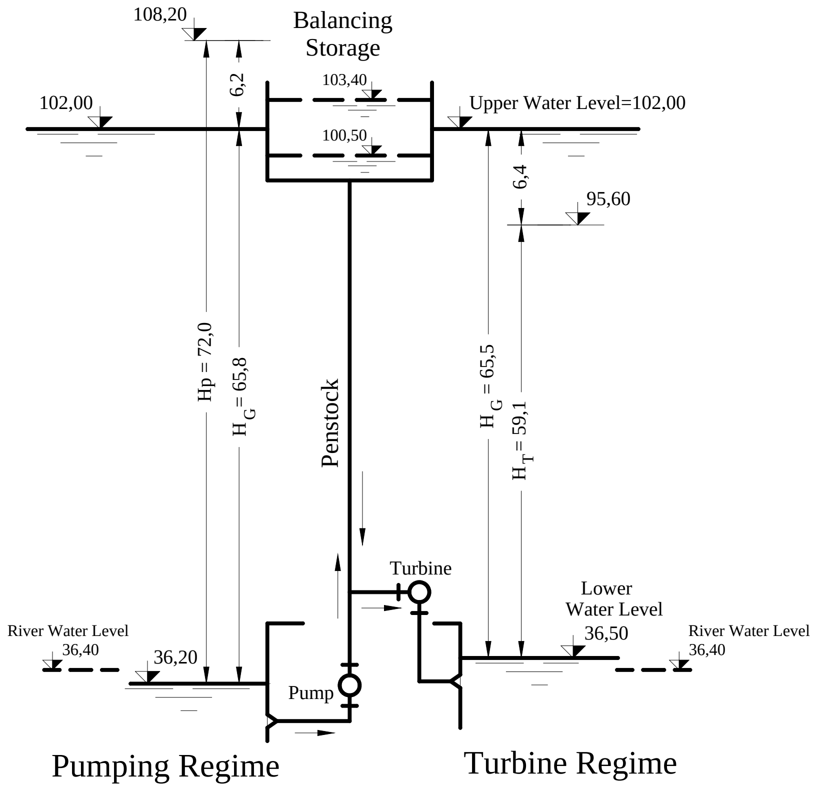


Fig. 2. Scheme of Water Levels and Actual Heads

Table. 2. PSPP "Sofronievo 1" Main Parameters (Preliminary Design)

Parameter	Units	Pumping Regime	Turbine Regime
Balancing Storage Capacity V_{BS}	m^3	43200	43200
Balancing Storage Depth H_{BS}	m	2,9	2,9
Upper Water Level	m a.s.l.	102,0	102,0
Lower Water Level	m a.s.l.	36,2	36,5
Geodetic Head H_G	m	65,8	65,5
PS / Turbine Discharge Q_{PS} / Q_T	m^3/s	1,88	2,00
Pump / Turbine Discharge Q_P / Q_T	m^3/s	0,47	2,00
Actual Head H_P	m^3/s	72,0	59,1
Pump / Turbine Efficiency η_P / η_T	-	0,84	0,90
Motor / Generator Efficiency η_M / η_G	-	0,93	0,95
Operation Time per day	h/d	6,38	6
Energy prices	€/kWh	0,04387	0,08318

It is important to state that the pump and motor efficiencies are only presumable, based on the data in catalogues. Because of the long period of non-operation these values could differ significantly from the catalogue ones.

It was not possible to perform the validation of these efficiencies by means of the test pumping, because there were technical difficulties to put the pumps in operation.

The project efficiencies, defined by (7), (8), (9) and (10), and also the energy production revenue, estimated by (6) are presented in **Table 3**.

Table. 3. PSPP “Sofronievo 1” Efficiencies and Energy Production Revenue (Preliminary Design)

Parameter	Units	Value
Combined Mechanical Efficiency η_C	-	0,668
Energetic Efficiency e_E	-	0,548
Financial Efficiency e_F	-	1,896
Total Efficiency of the PSPP e_{PSPP}	-	1,039
Treated Daily Volume W_d	m^3	43200
Energy Production Revenue R	€/year	6684,5

As it is evident by **Table 3**, there is positive revenue from the energy production, but its absolute value is very small. It is not sufficient to cover even the expenditures for staff salaries, so nothing left to return the investments.

Considering (6), (7), (8) and (9) the increase of the revenue is possible in three ways: by increasing the treated daily volume W_d; by increasing the energetic efficiency e_E, or by increasing the financial efficiency e_F.

The first way is the easiest one, but it is not productive, because it will lead to increase of the Balancing Storage Volume, i.e. to additional investments.

The efficiency e_E can be increased, but very slightly. The mechanical efficiency η_C could not be changed, so the ratio between the actual heads should be modified. Considering the restriction for the geodetic heads and the fact that the penstock should not be replaced, the only solution is to decrease the head losses in ducts and valves in the PS and the PSPP. They are approximately 1,8 m in both regimes and can be reduced to 1,3 m, but only by providing bigger ducts and valves. The reduction will lead to an increase of e_E to 0,580, i.e. with 5,8%. Anyway, such a reduction will raise the revenue to 17020 €/year, which is an increase of more than 2,5 times. It can be concluded that a very small increasing of the efficiency e_E produces a “jump” of the revenue. Anyway, because of the geodetic and other physical restrictions, no big improvement of the efficiency e_E can be done.

The greatest effect will be achieved if a special selling price s_p for the peak demand energy is negotiated, so to raise e_F. If e_F is increased by 10%, i.e. to have e_F = 2,086, the revenue will leap with 267% to 24530 €/year. Considering the investments and the expenditures, a good return period of the enterprise could be achieved only if the e_F reaches value of 3,0. In such case the revenue will become 110379 €/year. Nevertheless, the selling price s_p for the peak demand energy is defined by the State Agency on the base of certain rules and nowadays no exclusions or special cases for private PSPP are provided. Moreover, the current situation in Bulgaria regarding the energy prices is very delicate, so no rise of the s_p could be expected.

According to the results presented in **Table 3**, the project for the reconstruction of PS “Sofronievo 1” to PSPP is considered as unprofitable.

4. CONCLUSIONS

Turning an existing Main PS with Balancing Storage to PSPP could be profitable only if lots of conditions are fulfilled:

1. The PS site should be close to the river and suitable for additional building for the Power Plant Machine Room.

2. The pumping equipment should be in good repair.

3. The head losses both in pumping and in turbine regimes should be less than 10% of the respective geodetic heads.

4. The Balancing Storage should have large volume, so minimum investments for enlargement to be necessary.

5. The energetic efficiency e_E should have value greater than 0,5.

6. The financial efficiency e_F should be at least equal to 3,0, which means that the energy selling price s_p should be at least three times bigger than the energy purchase price c_p.

Considering the current economic conditions, the most important conclusion is that the reconstruction of the PS to PSPP is not profitable. So the new life of the old PS seems unrealizable.

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Role of the Human Resources in the Management of the Drainage Systems

Neli H. Banishka

Abstract – **Examining the state of the drainage systems in Bulgaria, their maintenance and exploitation on the level "branch" indicates, that the corporation operating the hydromeliorative systems have not qualified experts, who are necessary for the effective management. The aim of the present report is based on defined functions and tasks to suggest necessary members of qualified human resources, which are based on suitable designed organizational structure and division of the responsibilities between them to ensure the effective management of drainage systems on the level " "Branch", "Hydrotechnical region" and "Hydrotechnical section"**

Keywords **– human resources, drainage systems, effective management.**

1. Introduction

Drainage systems in Bulgaria are designed and built from 80 - years of the last century. Then, they made only repair and reconstruction works of damaged and most endangered elements of sections and equipments [3]. At present, much of this drainage infrastructure is destroyed, and the rest didn't work effectively, which requires urgent measures to complete the reconstruction and modernization of the same infrastructure. The change is urgently with a view to ensure a sustainable management of the surfaces and groundwaters, environmental protection and protection from the harmful effects of the waters.

The data of the "Irrigation Systems" JSC to 2012 year show, that in our country there are in operation the following objects [4]:

- Protective dikes of the Danube River – 295 km
- Correction of rivers in the country – 3240 km
- Protective dikes of rivers in the country – 385 km
- Retention dams – 14 number
- Drainage pumping stations – 90 number
- Drainage fields - 1434 ha. In 32% of these fields, drainage is done pumping, others – gravity
- Main drainage channels – 2334 km
- Cumulative channels network – 11192 km
- Sheltered drainage system – on 365 ha

All these equipments form a group irrigation facilities. Their task is to protect agricultural lands, populated places and different economic buildings from the harmful effects of the waters. These water systems and equipments are public state property and the financing of the activities under their protection, maintenance and operation is subject to the investment policy of the country, conducted by the Ministry of Agriculture and Food in the Republic of Bulgaria (Under the Water Act of 28 January, 2000).

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The problem of the lack of the effective management of Irrigation and Drainage fund by the company comes from the fact, that from 2011 to now, the funding for the maintenance and operation of the drainage infrastructure by the Ministry stopped. Partial funding is made only after occurring of the disaster by the Commission for disaster to General Directorate Fire Safety and Civil Protection.

At that time, the company operates and maintains drainage infrastructure is state trade corporation "Irrigation Systems"JSC. It performs its functions through 10 branches, located on the territory of our country.

For this purpose, this organization uses a certain number of specialists and employees, the number and members depends on the size, structure and complexity of the drainage systems.

Organizational structure of "Irrigation Systems" JSC is presented in **Fig. 1.**

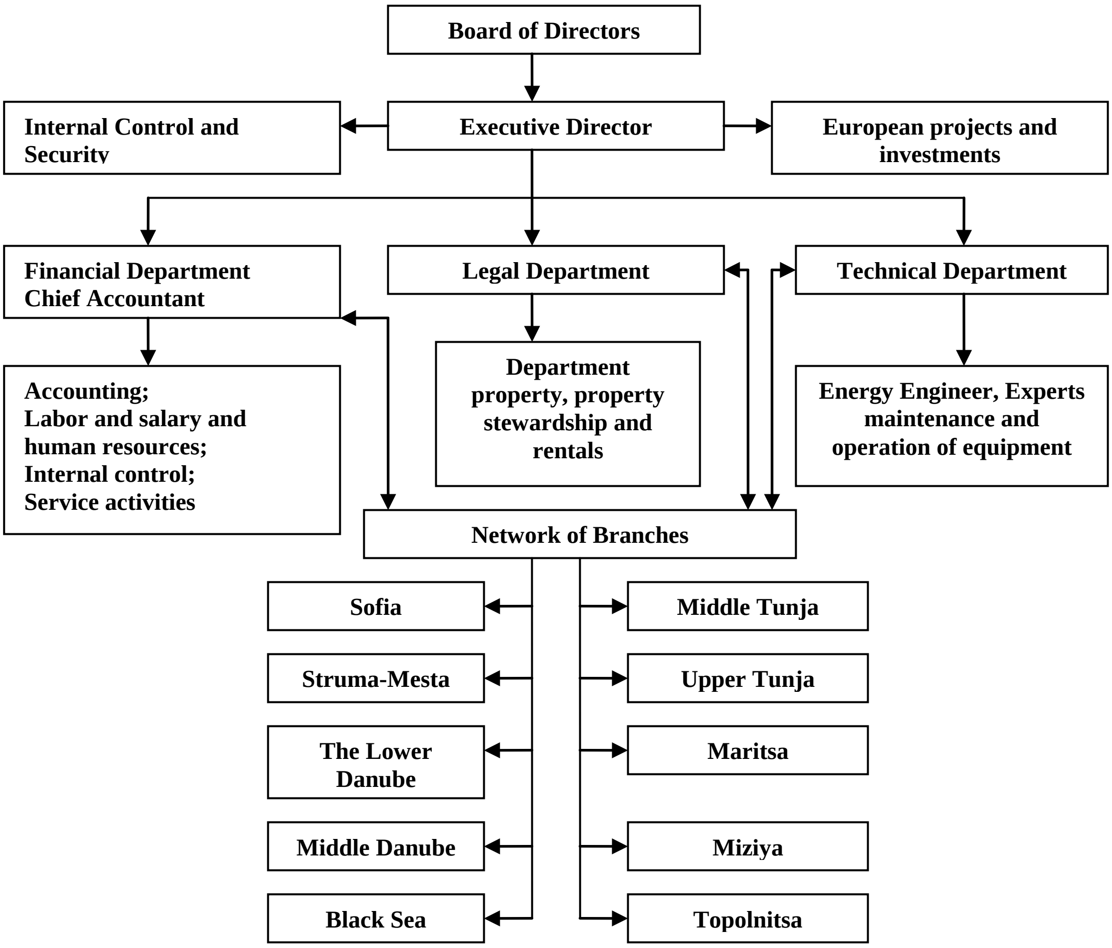

Fig.1 Organizational structure of "Irrigation Systems" JSC

This organizational structure is linear-functional , ie the total resource management and investment of the funds is a responsibility of the line managers / heads of departments / and process management to achieve the purposes assigned to the functioning sectors / departments and branches /[1]. Branches, in turn, are divided into "hydrotechnical region", and they are of "hydrotechnical sections" according to the size of the parts and this equipments, which are located on them.

The average number of the employees in the structure of “Irrigation Systems” JSC in mid-2012 was 2083 people and in the drainage infrastructure they are only about 400 people.

This research on human resources shows, that the company which operates the drainage systems in our country, feels the needs of qualified staff at the level "Branch", "Hydrotechnical region" and "Hydrotechnical section". For example in the department "Construction, repair and Locations to protect against harmful effects of water" to "Maritza branch", work the following staff: Head Department / 1 university graduates / organizer, construction and repair / 1 with specialized secondary education / and specialist / 1 with specialized secondary education /. The "hydrotechnical region Topolnitsa", which is belong to "branch Maritza", works only manager and cashier host, and in "hydrotechnical section Trud", for drainage facilities within the area meets a maintenance worker with primary education. This set of professionals is extremely insufficient for the management, operation and support of the drainage infrastructure in this "Branch".

Namely this requires structuring of the main activities and formulation of the main tasks and functions of the team of experts, creating an organizational structure matrix for allocation of the responsibilities.

2. MAIN TASKS AND FUNCTIONS OF HUMAN RESOURCES

The main tasks of human resources related with the technical operation and management of the drainage systems and equipments are consist of [2]:

- Ensuring normal operation of all elements and facilities of the drainage systems
- Security and maintaining in good condition of all elements of the systems
- Technical improvement of the systems and modernization of the drainage systems and facilities
- Establishing an appropriate water regime in the soil root zone for development of agricultural crops
- Create and maintain the conditions necessary for productive use of agricultural equipment and vehicles in the drainage areas

The main functions of human resources associated with the operation and management of the of drainage systems are consist of :

- **Planning** – it is consists in collecting and organizing information for preparation of a work program for short and long term periods of time. Planning takes place of the management level of the operating company and should meet the stated aim of efficient operation and maintenance of drainage infrastructure.
- **Organization** - establishes the organizational structure and terms of interaction between experts of various management levels to achieve the aim planning and carrying out all activities and measures in technical exploitation of the systems
- **Control and regulation** - procedures with information carriers through which is monitored for unfavorable variations in the work program and that would impede the effective management and operation of the drainage infrastructure
- **Stimulation** - creates conditions for the effective discharge of the assigned tasks and aims. It should be applied at all levels of human resources management
- **Reporting** - flow of information at different levels of government, registering the implementation of concrete tasks. Reporting is performed in a certain time interval. The information obtained should be recorded on defined medium and saved

The basic structure of the company, operates and maintain the drainage systems at the level of "branch" should be consistend mainly of:

- administratively management including administrative and financial and accounting services, activity planning, contractual relations, cadastre, etc.
- data team, which deal primarily with the collection, processing and use of information for the drainage areas of systematic observations on the operation of drainage facilities, the regime of surface and

groundwater, drainage flow, water regime of the soil, meliorative condition of areas, equipments management, regulation of water regime drained areas, etc.

- measurements group to conduct observations and measurements needed the information team
- group for construction and repair
- group of transportation and installation
- emergency group

3. NECESSARY HUMAN RESOURCES IN THE MANAGEMENT OF THE DRAINAGE SYSTEMS AT LEVEL "HYDROTECHNICS REGION"

To be well balanced team for operation and maintenance of drainage systems at the level of "Hydrotechnics region" is necessary to have at least the following members:

- **Head of region** - an expert in higher education in "irrigation and drainage engineering," which organized a team of experts, manage the exploitation and maintenance of drainage systems and communicate on a higher level / **chief engineer** /;
- **Engineers - improver** - experts with higher education in "Irrigation and Drainage Engineering", with extensive experience in the operation and maintenance of irrigation and drainage systems and equipments.

The number of engineers is determined and depending on the size and type of drainage system and the facilities. It is possible to include engineers for the operation of dikes, discharges and water abstraction facilities and adjacent parts of the water recipients and water sources, operating and regulating drainage networks and equipment, for the operation of the irrigation network and its facilities, maintenance and operation of pumping stations, of the information activities and more.

- **Agronomist** - expert in higher education in "Agronomy", who has experience in the cultivation and irrigation of agricultural crops on land with built drainage systems
- **Technical staff**: **technician improver, trackman - drain, a district hydrometric, observer staff pumping stations, masters and maintenance workers, security guards, machine operators and others,** who has the necessary education and skills so as to facilitate the activities of engineers improver.

4. ORGANIZATIONAL STRUCTURE

Organizational structure presents organizational interdependence among the team members for the operation and maintenance of drainage systems at all levels of management. Schematic of an organizational structure at the level of "Hydrotechnics region " is presented in **Fig. 2**

The main factors which should be taken into account when designing the organizational structure are:

- Specialized division of labor, ie It is necessary each activity maintenance and operation of drainage systems to be implemented by the corresponding expert
- Amount of the work performed

During the design of the organizational structure, it could be take into account that that each "hydrotechnical region” is different in size, facilities, maintenance and management decisions, which is why there can not be a uniform scheme of organizational structure.

The organizational structure clearly shows communication channels simultaneously in vertical and horizontal level. It is important to follow the principle that information between the different experts is performed only if there is direct relation to the organizational structure.

Effective planning of communication channels in the design of such structure will contribute to the achieve of the most appropriate ways to transfer information between the human resources associated with the operation and maintenance of drainage systems at the level of " hydrotechnical region ". It is essential the planning of the ways to get the information, ie at what time and in what period to transmit and receive operating and extraordinary information. It is necessary to describe the measures which must be taken when you do not perform the given deadlines and periods.

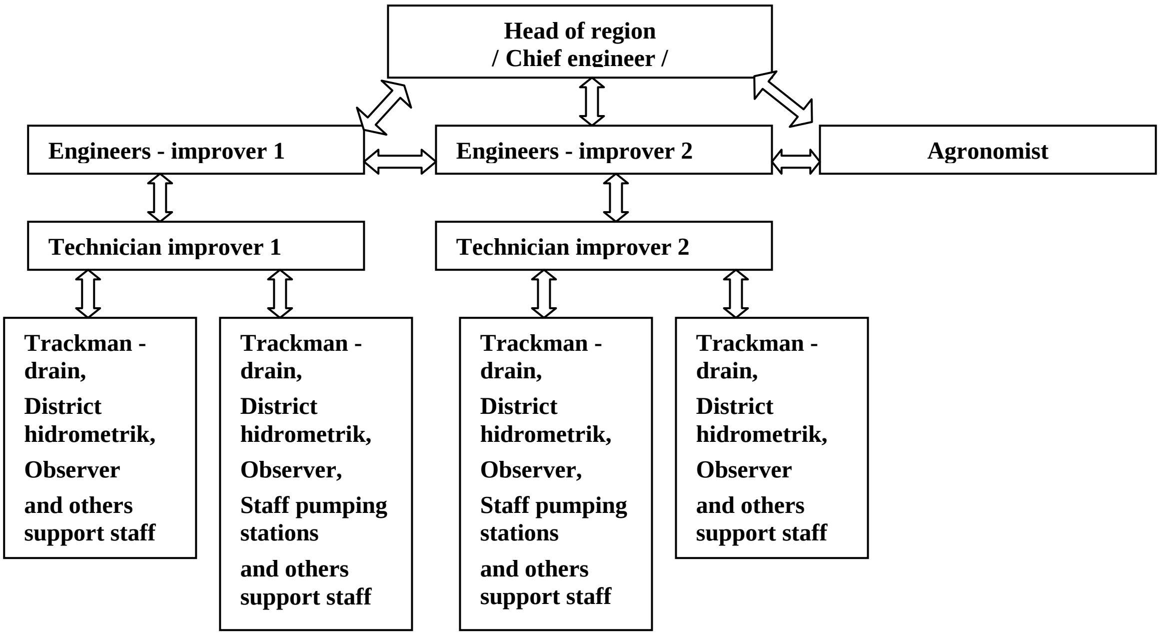

Fig.2. Organizational structure at the level of "Hydrotechnics region”

Table. 1. Exemplary matrix for the allocation of responsibilities

Human resources / Main tasks	Chief engineer	Engineers improver	Agronomist	Technician improver	Trackman - drain	District hidrometrik	Observer	Staff pumping stations	masters and maintenance workers
Ensuring normal operation of all elements and facilities of the drainage systems	✓	✓		✓				✓	
Security and maintaining in good condition of all elements of the systems	✓	✓		✓	✓			✓	✓
Technical improvement of the systems and modernization of the drainage systems and facilities	✓	✓		✓					✓
Establishing an appropriate water regime in the soil root zone for development of agricultural crops		✓	✓	✓	✓	✓	✓		
Create and maintain the conditions necessary for productive use of agricultural equipment and vehicles in the drainage areas		✓	✓	✓	✓		✓		✓

5. MATRIX FOR DISTRIBUTION OF RESPONSIBILITIES

Matrix of the distribution of responsibilities between experts in the team for operation and maintenance of drainage systems at the level of " hydrotechnical region " is a required element, regardless of the size and facilities that serve it. To the development of the matrix list of experts, included in the organizational structure of the drainage systems level " hydrotechnical region " and a list of the main activities, which need to be fulfilled for the correct maintenance and operation of the drainage infrastructure, are needed

Exemplary matrix for the allocation of responsibilities for the maintenance and operation of a drainage system and its facilities is given in **Table 1**.

In the horizontal direction are shown human resources, associated management, operation and maintenance of the drainage systems. In the vertical direction are written the basic tasks to be carried out. For every expert is marked in an appropriate manner any of the main tasks is responsible.

6. CONCLUSION

The establishment of effective and sustainable competitive agricultural production is impossible without recovery and restructuring of irrigation and drainage engineering in Bulgaria in the coming years. In this regard, the Ministry of Agriculture and Food in the final stage of negotiations with the World Bank to develop a strategy for irrigation and drainage engineering, which will be financed by the Technical Assistance Programme for Rural Development. On the other hand as a priority in the new programming period 2014 - 2020 y. is expected to include activities aimed at the reconstruction and modernization of existing hydromeliorative structure.

When this occurs in the direction of drainage infrastructure, the policy of the Ministry of Agriculture and Food will have focus its efforts on designing effective organizational structure of the operating company of the various management levels. The selection of highly qualified human resources with their clearly defined tasks and allocation of responsibilities will lead to effective management, operation and maintenance of existing drainage infrastructure in Bulgaria.

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Analysis of evapotranspiration deficit on the East Slovakian Lowland environment

M. Gomboš, B. Kandra, D. Pavelková, I. Pálešová

***Abstract*: In this paper an analysis of long-term data of actual and potential evapotranspiration is presented. Data needed for analysis were made of daily values of evapotranspiration during the growing seasons (April-September) between the years 1970–2012. Daily values were obtained by calculation on the mathematical model called Global. Calculations were made for conditions of the central region of the East Slovakian Lowland. Descriptive statistics, trend analysis and probabilistic processing in the form of crossing lines were used. Precipitation, as one of the basic components of water balance in the country, was also incorporated into the analyses. Long-term trends of evaporation, precipitation and evapotranspiration deficit were identified.In 43-years long dataset was found that in the last 15 years actual and potential evapotranspiration increased significantly.**

***Keywords* – actual evapotranspiration potential evapotranspiration, evapotranspiration deficit, precipitation**

1. INTRODUCTION

Evaporation of water in nature from the physical point of view means the process of conversion of water into the steam. Due to the high value of the latent heat of evaporation (2450kJ kg^{-1}at 20^{o} C evaporating surface) it is an energy-difficult process. In nature it is a crucial regulator of energy flows. Evaporation from soil and plants is called evapotranspiration. Amount of evapotranspiration in time is influenced by energy required for a change of water into vapor and amount of water in this environment. It means that evapotranspiration has seasonal character. Crucial amount of water obviously evaporates in the vegetative period of year, from April to September. The maximum possible evapotranspiration under the given meteorological conditions is called potential evapotranspiration (ET_0). Its size is the result of energy balance on the active surface. Actually evaporated water from the soil and plants is called actual evapotranspiration (ET_a). It depends from positive energy balance, turbulent exchange between stands and atmosphere, the water supply in soil profile and from the ability of plants to regulate their water intake and outtake. In case of sufficient amount of water the actual evapotranspiration is the same to potential. If $ET_0> ET_a$, it indicates the water deficit in the root zone of a soil profile and the beginning of soil profile drying [3]. The ongoing climatic changes are reflected in the redistribution of precipitation during the year and in the increased air temperatures. In other words it means

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redistribution of water and energy in hydrological cycle. These changes are reflected in changes in evaporation and subsequent changes in water balance of environment.

The aim of this paper is to analyze long-term trends of evaporation, precipitationand evapotranspiration deficit in the study area by use of mathematical statistics methods and to explore the impacts of climate change on evapotranspiration. The analysis was realized in conditions of East Slovakian Lowland in the area called Milhostov.

2. EXPERIMENT DESCRIPTION

The process of evaporation was examined in the central part of Eastern-Slovakian Lowland in the area of Milhostov (ϕ = 48°40'11,08"; λ = 21°44'18,02"; 100 m). The location is characterized by medium-heavy gleyed soils with clay particles of 18–39 % (Figure 1). From the climatic point of view the area of East Slovakian Lowland is located in the area between oceanic and terrestrial climate. One of the basic features of this climate is great temporal variability of weather and therefore all of the meteorological elements. In the long-term average summer culminates in July, which has the highest average air temperature (19,1 – 20,2°C). A broader vegetation period with air temperatures above 5°C begins in the last decade of March and ends at the end of first decade of November and lasts for 229 to 236 days. Temperature sum of temperatures equal or higher than 5°C is 3174 – 3445°C. A closer vegetation period with air temperatures above 10°C starts on 15th–20th April and ends on 8th –12th October. It average durations172 – 181days and temperature sums reach 2745 – 3028°C.The summer period with air temperatures above 15°C starts on average on 14th – 25th May and ends on 9th – 16th September. It lasts 108 – 126 days and temperature sums reach 1932 – 2334°C.

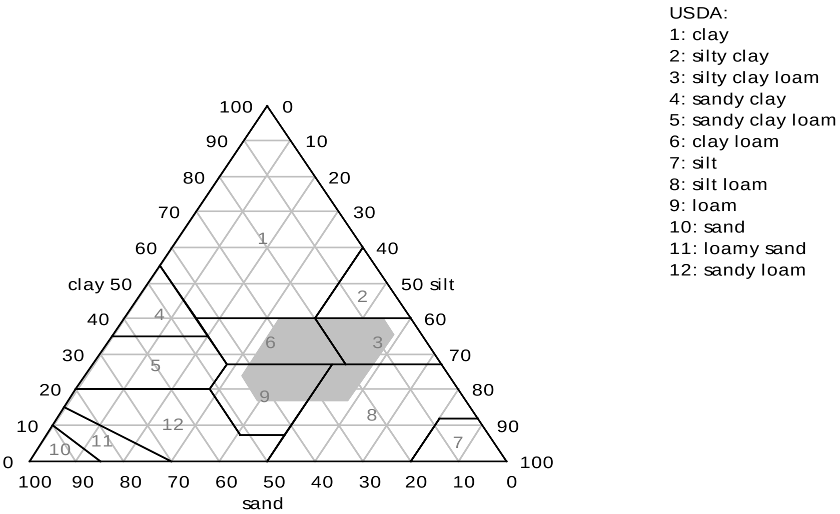

Figure 1. Texture of the examined soil profile according to the triangular classification diagram USDA (sand 0,05–2,0mm, clay <0,002mm, silt 0,002–0,05mm)

In the area, there is located an agro-ecological climatic and research station. This station provided all the data concerning hydro-meteorological elements and plant cover characteristics required for numerical simulation on the mathematical model GLOBAL. The model GLOBAL is a simulation mathematical model of soil water transfer which enables the calculation of moisture potential distribution or soil moisture in real time [Majerčák, Novák, 1994]. Potential evapotranspiration ET_0 is calculated according to FAO, by Penman`s method of–Monteith [1]. For determining the actual transpiration or evaporation intensities the method developed on IH SAS was used. According to this method the evapotranspiration structure depends on the value of leaf area index (LAI). Intensity of potential evaporation E_{e0} is calculated from the value of potential evapotranspiration ET_0 using the formula:

$$E_{e0} = ET_0 \times \exp(-m_1 \times LAI) \quad (1)$$

The value of empirical coefficient (m_1 = 0,463) was gained by field measurements in the wheat plant cover. Calculation of the actual evapotranspiration intensities and its structure is based on the knowledge of potential evapotranspiration ET_0 and the relationship between $\frac{E_{e0}}{ET_0}$and moisture of the soil profile, i.e.:

$$ET_r = \frac{E_{e0}}{ET_0} = f(\theta) \quad (2)$$

Used calculation method is based on the assumption that mean value of soil moisture in the root zone incase of calculation transpiration or value of moisture of the upper layer of soil profile in case of evaporation calculation, in which evaporation starts to decrease (critical value θ_k), depends on the intensity of evaporation. The higher the evaporation intensity is, the higher is the value of θ_k, in which evaporation starts to decrease. This method was verified using the model GLOBAL and there was shown conformity of the calculated values with the values gained by field measurements in real conditions.

By use of this model, daily amounts of potential and actual evapotranspiration from the period of years 1970–2012 were calculated. Subsequently, monthly amounts of ET_0 and ET_a and amount for whole vegetation periods were calculated from mentioned daily values.In addition evapotranspiration deficit (ET_D) was calculated according to the formula:

$$ET_D = ET_0 - ET_a \quad (3)$$

For this analysis were used methods of descriptive statistics, trend analysis and probabilistic processing in the form of crossing lines. For capturing of broader context, there is precipitationas one of the basic components of the water balance in country incorporated in the analysis.

3. Results and significances

Table 1shows the basic statistical characteristics of average long-term daily, monthly and growing season amounts of ET_0 and $ET_a$and average long-term amounts of precipitation during growing seasons. From the above table the descriptive characteristics (position, shape and variability) are evident. From the long-term view there is actual evapotranspiration in average60% from potential and 83% from precipitation in the study area during the growing seasons. However, it should be noted that no influence of winter water storage was captured in unsaturated zone and in groundwater [7].

Potential evapotranspiration during the growing season from the long-term view 1,4-times exceeded precipitation. The highest value of ET_0during the growing season was 675 mm (2009) and the highest value of $ET_a$was 468 mm (2010). The maximum monthly value of $ET_0$was 160 mm (7/1995) and $ET_a$was 112 mm (6/2005). Average long-term daily value of $ET_0$was 2,9 mm and $ET_a$was 1,7 mm. Absolutely maximum daily value of $ET_0$was 9,32 mm (22.7.2007) and ET_a 5,57 mm (16.6.2005).

In terms of variability, values of $ET_0$are more stable to the average values than values of ET_a. Variability of ET_a relative to the mean values approximates to the variability of precipitation.

Table 1. Descriptive statistical characteristic for average long-term daily, monthly and growing season amounts of ET_0 and ET_a and average long-term amounts of precipitation during vegetation periods in 1970–2012.

Statistical characteristics	average daily total 1970 - 2012		average monthly total 1970 - 2012		growing seasons 1970 - 2012		precipitation 1970 - 2012
	ET0	*Eta*	*ET0*	*Eta*	*ET0*	*Eta*	P
Mean	2,9	1,7	87,1	52,1	522,8	312,8	375,1
Standard Error	0,02	0,01	1,53	1,27	11,72	10,52	12,91
Median	2,77	1,64	86,68	50,96	526,42	292,84	366,10
Standard Deviation	1,37	1,02	24,62	20,44	76,83	68,97	84,67
Sample Variance	1,88	1,03	606,16	417,59	5903,22	4756,33	7168,21
Kurtosis	-0,09	-0,08	-0,24	-0,21	0,58	-0,51	2,29
Skewness	0,27	0,45	0,26	0,30	-0,35	0,64	1,04
Range	9,29	5,57	131,94	106,34	372,87	266,20	444,40
Minimum	0,02	0,00	28,29	5,28	302,70	202,29	226,80
Maximum	9,32	5,57	160,23	111,61	675,57	468,48	671,20
Coefficient of variation	0,48	0,59	0,28	0,39	0,15	0,22	0,23
Sum	22482	13450	22482	13450	22482	13450	16131
Count	7869	7869	258	258	43	43	43
Confidence Level(95,0%)	0,03	0,02	3,02	2,51	23,65	21,22	26,06

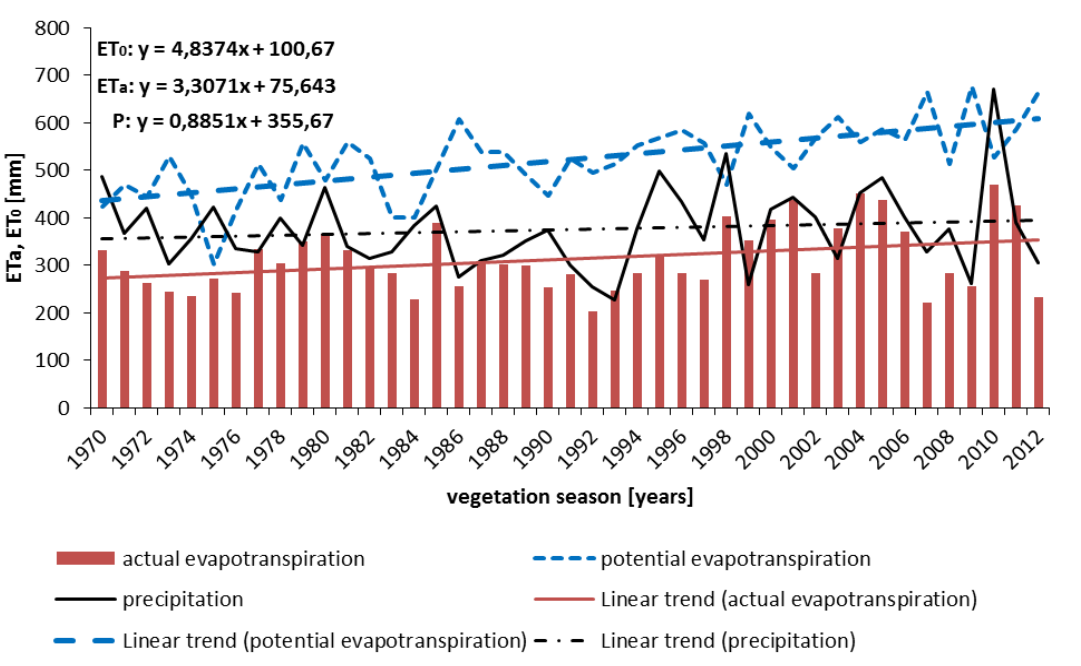

Figure 2 Development of vegetation amounts of ET_0, ET_a, P and their linear trends during the period of years 1970 – 2007

Figure 2 shows courses of potential evapotranspiration ET_0 during growing seasons, actual evapotranspiration ET_a, precipitation P and their linear trends for the period of years 1970–2012. Precipitation in investigation period was stable. Their trend in growing seasons showed just small increase (0,89 mm/year). We could identify an increasing trend of ET_0 and ET_avalues on the Figure 2. The difference between ET_0 and ET_ahas increased. Potential evapotranspiration had greater increase according to ET_a. Increased values of ET_0 and ET_a can be observed mainly in the last 15 years of investigated period. During this period were identified the absolute highest and also smallest values of ET_0 and $ET_a$for the whole observed period of years 1970–2012. Increase in difference between potential and actual evapotranspiration, i.e. increase of evapotranspiration deficit indicates drying out of the investigated area. Therefore the development of evapotranspiration deficit $ET_D$is the subject of further analysis (3).

The values of evapotranspiration deficit are shown in Figure 3. Long-term average value of ET_D during growing season was210 mm. Long-term linear trend ET_D had an increased character. The three absolutely highest values of ET_D has been achieved in the last seven years, 2007 (443 mm), 2012 (432 mm) a 2009 (419 mm). In these years, the second absolutely lowest evapotranspiration deficit has been achieved, 2010 (58 mm). Analysis with the use of 5-year moving average showed, that ET_D course had 2 local minimums and 2 local maximums. First local minimum started in the year 1978 (1974–1978) on the value of 145 mm. Since 1979,the average 5-years ET_D in growing season has been increasing until 1997 (275 mm). In the following period, ET_D values were decreasing to the year 2001. In 2001–2005 the 5-years moving average oscillated around a value of 175 mm. In 2006–2009 the moving average increased to 287 mm. In the next three years decreased to 260 mm.

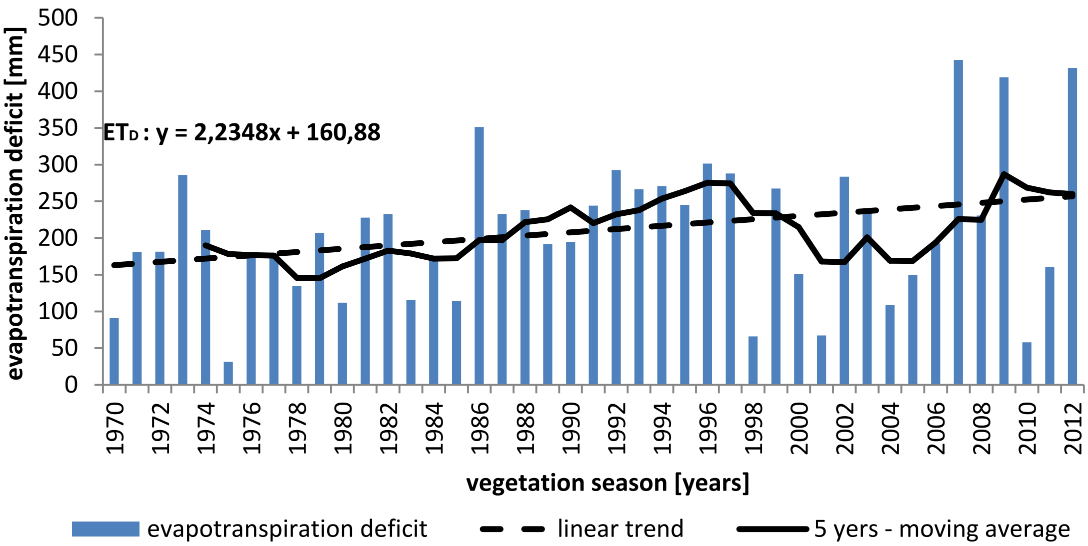

Figure 3. Vegetation amounts of evapotranspiration deficit ET_D during the period of years 1970 – 2012, five-years moving average and linear trend of development

From the course of ET_D on Figure 3 it is possible to identify the period 1998–2012, when the variability and periodicity of extreme values ET_D increased. For quantification of this phenomenon the years 1970–1997 and 1998–2012 were compared. Basic statistical characteristics of both series and also of whole period are shown in Table 2. The table indicates that it is possible to expect statistically significant differences in variability of both series.

Table 2. Summary statistics of evapotranspiration deficits ET_{D1} during growing seasons in 1970-1997, ET_{D2}in 1998-2012 andET_{D}during whole period of years 1970–2012

Statistical characteristics	ET_{D1} (1970–1997)	ET_{D2} (1998–2012)	ET_{D} (1970–2012)
Count	28	15	43
Average	205,974	217,629	210,0397
Standard deviation	72,9008	130,981	95,74275
Coeff. of variation	35,3932%	60,1854%	45,58317%
Minimum	31,28	57,94	31,27809
Maximum	351,36	442,69	442,6904
Range	320,08	384,75	411,4123
Stnd. skewness	-0,742744	0,970255	0,486919
Stnd. kurtosis	0,0152726	-0,554601	0,348867

The comparison of average values shows, that 95,0% confidence interval for mean of ETD1: 205,974 +/- 28,268, i.e. [177,706; 234,242], 95,0% confidence interval for mean of ETD2: 217,629 +/- 72,535, i.e. [145,094; 290,164] and 95,0% confidence interval for the difference between the means - assuming equal variances:- 11,6554 +/- 62,5106, i.e. [-74,166; 50,8552]. This comparison shows that in terms of averages there are no statistically significant differences between examined periods at the level of 95 % confidence.

The comparison of standard deviations of the selected series with 95,0% confidence interval showed, that standard deviation of ETD1 is [57,6368; 99,2279], standard deviation of ETD2 is [95,8947; 206,57] and ratio of Variances is [0,112333; 0,741868]. From this comparison and from Table 2 follows that there are statistically significant differences between the examined periods.

The question is what caused the change in variability of evapotranspiration deficit in growing season in period 1998–2012. One possible explanation is the ongoing climate changes which became evident in the examined area about 15 years ago by intense evaporation of water in nature. This explanation could be confirmed in the following years by deepening of the disparity between actual and potential evapotranspiration and increasing of the trend of evapotranspiration deficit of recent 15 years. If the statistical characteristics and trends of ET_{D} do not returned to the range of values from the period of years 1970–1997, then we could apparently talk about impact of climate changes on evaporation in the examined area.

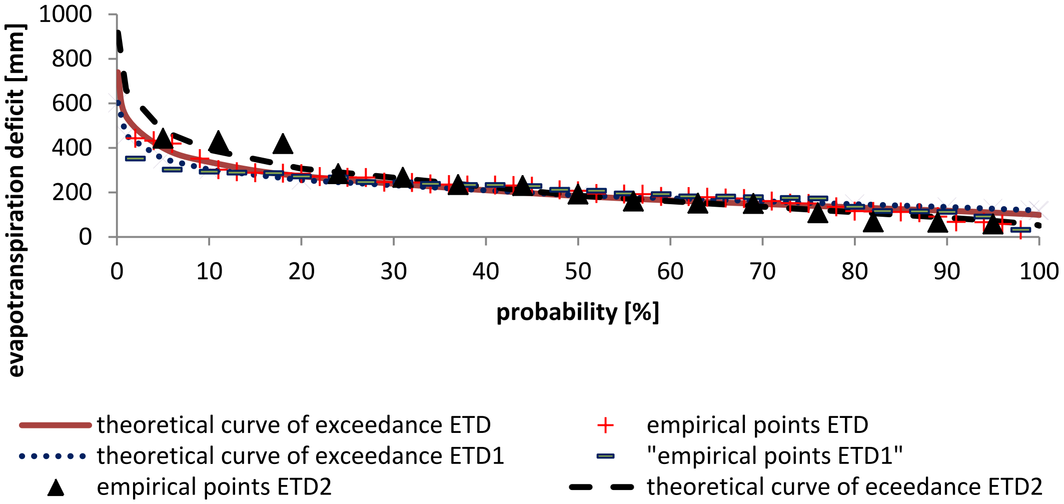

Figure 4 Empirical points and theoretical curves of exceedance calculated forET_{D}, ET_{D1} and ET_{D2}

Amounts of evapotranspiration deficits during growing seasons have been also studied by probability, with the use of empirical and theoretical lines of exceedance and achievement. The parameters of theoretical lines were calculated by the method of moments by the use of binomial distribution of variability. Whole period of ET_D (1970–2012) and periods1970–1997 (ET_{D1}) and 1998–2012 (ET_{D2}) were processed like that. The results are shown in Figure 4. From the theoretical lines of exceedance and also from their empirical points shown in Figure 4it is clear that the exceedance lines of evapotranspiration deficits are in periods 1970–2012 and 1970–1997 almost identical. The line of exceedance for the period 1998–2012 had a steeper course. The above course indicates higher variability of the evapotranspiration deficits in that period. For better illustration there are theoretical values of exceedance and achievement of ET_D, ET_{D1} and ET_{D2} for the probability of 1%, 50% and 95 % shown in Table 3.

Table 3. Theoretical values of evapotranspiration deficits for different probability of occurrence

probability	**ET_{D1} (1970–1997) [mm]**	**ET_{D2} (1998–2012) [mm]**	**ET_D (1970–2012) [mm]**
1%	455	660	541
50%	187	185	184
95%	127	72	109

With regard to the previously mentioned three absolutely highest deficits achieved in the last seven years, it follows that probability of occurrence of these deficits in growing seasons are 3,8%, 4,1% and 4,5%. This corresponds to the period of repetition every 26 years, 24 years and 22 years.

4. CONCLUSIONS

The paper analysed the development of long-term data series of evapotranspiration deficits during the growing seasons. The period of years 1970–2012 was evaluated. The baseline data needed for analysis were daily amounts of potential and actual evapotranspiration. The values of ET_0 and ET_a were calculated on a mathematical model GLOBAL with one-day step. Modelling has been carried out in conditions of locality Milhostov on the East Slovakian Lowland. It was possible to assemble large-scale hydro-meteorological, hydrological and phenological databases and hydrophysical characteristics needed for the numerical simulation of evaporation of water in that area.

The basic characteristics of descriptive statistics were calculated and long-term trends of ET_0,ET_a and evapotranspiration deficit ET_Dwere identified. Development of these trends (ET_0 and ET_a) had increasing characterand their difference (i.e. ET_D) grew mainly in the last years. This shows the increasing intensity of the drying of investigated area. It has been also shown that during the last 15 years of the examined period had significantly greater variability of ET_D while the average was maintained. Increased variability was reflected by increased occurrence of extreme values. It was also demonstrated by empirical and theoretical line of exceedance or achievement of random variable.

One possible explanation of the identified temporal changes in variability of evapotranspiration deficit is the effect of climate change. These results are still incomplete. The processes of water evaporation in connection to the water supply in the unsaturated soil zone and groundwater level are the subject of further research on theEast Slovakian Lowland.

5. Acknowledgments

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Impact of climate change on the Trifolium alexandrinum crop irrigated by treated wastewater in Tunisia

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***Abstract* - The public irrigated area "Cebala" located in the lower valley of Medjerda was built to enhance exploitations and increase efficiency by exploiting important potential treated wastewater. These waters used for irrigation are often saline (3 to 4 g/l), so we can expect loss of crop-types income and risk of soil salinity, if we do not take special measures. As part of the global framework to conduct sustainable agriculture and environmental preservation, we will determine the optimum technical and economic reuse treated wastewater, which preserves soil fertility and prevents soil salinity in the long term. We surveyed 83 farms and isolated six types of farms belonging to six distinct soil types. We adopted a biophysical model "CropSyst" that allows long-term simulation Trifolium alexandrinum yields and soil salinity. We generated climate data for 30 years using real data and "ClimGen" program in "Cropsyst". We also generated others climate data for the same period using the same program and adopting two scenarios of temperature increase respectively TO one and two degrees Celsius to study the climate change effects on crop yields.**

Keywords - Biophysical, climatic change, environment, modelling, salinity, simulation, sustainability.

1. INTRODUCTION

The gradual land loss is a result of climatic and anthropogenic factors (El Mokthar et al, 2012). Consequently, 6.4% of the land is affected by salinity; 2 to 3 million hectares of irrigated land are lost every year (Mermoud, 2006). Increase production could be obtained by the irrigation development, which provides 40% of the world's food production and increase yield by 2 to 4 (IRD, 2012). The water then undergoes the same pressure as the earth. Availability per capita per year was 7122 in 2000, it will be only 4800 m3 in 2025 and 3400 m3 in 2050 (Dugot, 2001). This global issue of shortage food and the factors that determine: availability of land, water and soil salinity apply to Tunisia. Water reserves of the country are particularly low: 460 m3/an/habitant (Zaara, 2008), against 1700 m3 recommended to meet the need (and Rekacewicz Diop, 2003), and soil loss is estimated at 20,000 hectares per year. The urgency is soil conservation and increased water reserves. Water desalination (Lattemann and Höpner 2008; Zaara, Tata-Ducru 2008 and 2009), the treated wastewater are the most cited way (FAO-stat, 2012, UN-WWDR4 2012). Tunisian government has invested a lot in the treated wastewater (169 million m3 collected). However, only 35 million m3 against a target of 100 million m3 were used for irrigation (Neubert and Ben Abdallah, 2003).

Furthermore, the development of irrigation is a national priority. But more than 70% of land is irrigated with water salinity greater than 1.5 g / l. Land degradation resulted by salinization and water logging. 30% of irrigated land is affected. Some irrigated areas have been completely abandoned (CIHEAM, 2005). The public area of "Cebala" located in the lower valley of the Medjerda was built to intensify farm production by treated wastewater. However, intensification rate has not exceeded 48% since 1995/96. Often the treated wastewater reused for irrigation has a load of salt in 3 to 4 g / l. We should therefore expect income losses and risk of excess soil salination (Neubert and Ben Abdallah, 2003).

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This work is therefore part of the overall framework of sustainable agriculture and environmental preservation (Vermersch, 2001). It aims to determine the technical and economic optimum use of wastewater, which, preserves soil fertility and prevents salinization in the long term. So we adopted a biophysical model, "CropSyst" (Stöckle et al, 2003). That after calibration and validation, allows long-term simulations of crop yields as a function of salinity soil, and allows studying the possibility of eventual replacement of the installed cultures or increased utilization of irrigation water. Our simulation over a period of 30 years applied of Trifolium alexandrinum crop.\

2. EXPERIMENT DESCRIPTION

We collected soil and climate data in the area of Cebala from studies and work already done on this site (INRGREF, 1998). These data were related to the minimum and maximum temperature, rainfall, wind speed, the minimum and maximum solar radiation for 26 years from 1983 to 2008, relative humidity, salinity and soil texture, quality water used (3 to 4 g salt/ l) and crops grown. Other information collected are concerned the historic farms, their identifications, their production systems and irrigation, installed cultures, the size of livestock. The number of farms surveyed is 83 ranging in size from 1-170 ha. They all have the characteristic of being irrigated with treated wastewater.

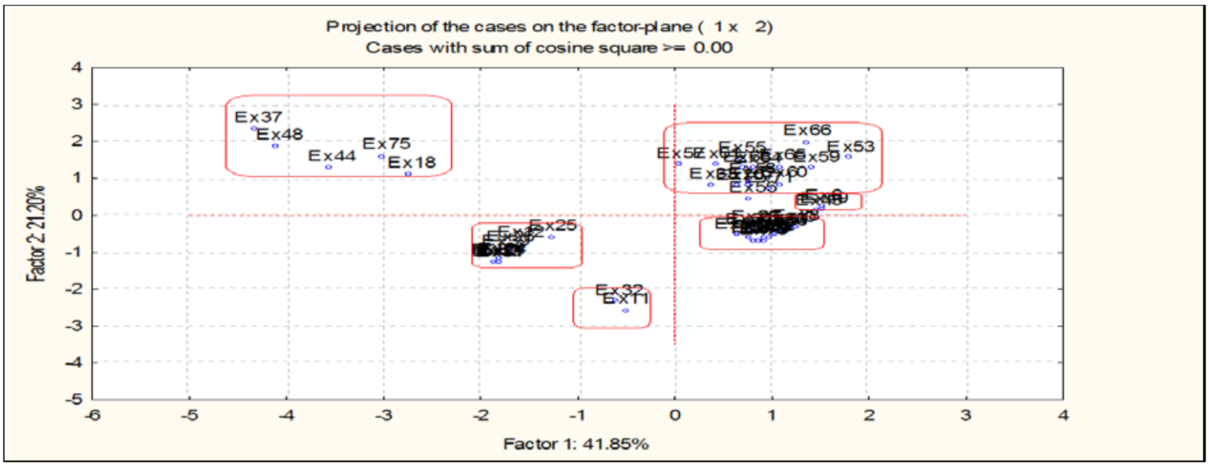

Fig. 1. Exploitations Classification by principal component analysis

We used a statistical tool to a principal component analysis "ACP" (Philippeau, 1986), to reproduce a composite image (Perrot et al, 1993). We tried to describe poles of aggregation as clearly differentiated as possible. This analysis was used to identify classes of farmers along axes trend (Hanafi et al., 2007). It allowed to isolate six separate exploitation groups (fig. 1), capitalizing the maximum elements of similarity. We selected six typical farms (EXP6, EXP11, EXP20, EXP26, EXP48 and EXP64) belonging to different soil type.With other crops, *T.alexandrinum* is practiced by four type's farms (EXP6, EXP11, EXP20 and EXP64). We are interested of *T. alexandrinum* crop because it is widely used for livestock alimentation in Tunisia. We have built model basis to establish long-term simulations for all farms type. In this study, we chose "CropSyst" model. It has been used for the evaluation of different agricultural production systems. It is capable to simulate many cultures, for several consecutive years and establishes the relationship between input-output regarding crop productions, to estimate the results of various cultivation techniques, to measure the degradation of natural resources, to evaluate the impact of different agricultural policies. It has been used and validated in arid regions, particularly in the lower valley of Medjerda in Tunisia (Mlaouhi 2002; Belhouchette 2004; Abbes 2005et Mlaouhi et al, 2012). Indeed, we entered the location data, climate data of the region, the soil texture and salinity for all farms selected, physiological *T. alexandrinum* crop parameters, crop rotation, amount of water irrigation and fertilizer applied by farmers. These data are needed to interact with each other, embedded in "Cropsyst" modules (Module "location and climate," Module "ground" Module "salinity" Module "culture"). The crop growth through several phases of development constitutes a cycle dependent on genetic, soil and environmental factors. During each of these phases limiting factors (water deficit, salinity, nitrogen nutrition etc ....) involved as reducing functions according the law minimum established by Justus von Liebig in 1840. This act resulted from no substitutability between agricultural inputs (water, salinity, nitrogen doses etc.) whatever the dose administered (Ackello-Ogutu et al, 1985; Frank et al, 1990; Berck and Helfand, 1990; Paris 1992; Berck et al, 2000). Assumption of non-substitutability between inputs, for example, the Von Liebig production function, representing water (E) and nitrogen (N), β and U, constants to be determined, takes the follow form:

$$Y_i = \min [Y^*, (\beta_1 + \beta_2 E_i + u_{Ei}), (\beta_3 + \beta_4 N_i + u_{Ni})] \tag{1}$$

This function implies that the plant responds in a linear fashion only the most limiting factor. After a level of water (E *) and nitrogen (N *) supply, the crop does not respond to additional contributions and the maximum yield (Y *) is reached (Belhouchette, 2004 and Abbes, 2005). Later, some agronomists, believing in a certain degree of substitutability between inputs at low application rates, have developed other functional:

- Von Liebig non-linéaire: $Y_i = \min [Y^*(1 - k_E e^{-\beta_E E_i}), Y^* (1 - k_N e^{-\beta_N N_i})] + u_i$ (2)
- Mitscherlich-Baule : $Y_i = Y^* (1- k_E e^{-\beta_E E_i}) \times (1- k_N e^{-\beta_N N_i})] + u_i$ (3)

"Liebig" kind Functions assume that we already know the mathematical form of the function (quadratic, logarithmic, exponential...). But, in reality, beyond two factors, it is very difficult to find the most appropriate form of the response function. So, in this work we need functions with more complex than those mentioned above forms. The standard econometric approach, stemming from the convergence of statistical inference and economic theory has limitations in addressing the complex relationship between agriculture and the natural environment and the economic environment (Boussard, 1987 Flichman, 1997). These limitations have led to adopt another approach more original, using production functions engineer obtained by simulations through the biophysical model "Cropsyst". This model operates at a daily time and the law of limiting factor intervenes to determine the biomass (yield). This potential biomass is daily subjected to water, salt and nitrogen stress. The biophysical model is governed by laws and equations that have not linear from. For this reason, it is used to determine the most appropriate form of *T. alexandrinum* function response considering its surrounding conditions. Yields, soil salinity and their changes over time are analyzed. Based on actual location data, structure and climate data of 26 years from 1983 to 2008, using a sub "ClimGen" program "Cropsyst" we generated climate data for thirty years (2011 to 2040), to study the crop response functions in long-term. Similarly, based on the fact that climate change, combined with other factors surrounding cultures, could change the *T. alexandrinum* conditions in the future, we generated using the same subroutine, climate data for 30 years. We adopted two scenarios of temperature increase respectively to one (1 ° C) and two degrees Celsius (2 ° C), for studying the climate change effects on crop yields. Following investigation and data collection related to *T. alexandrinum* crop and farm conditions surrounding, we applied a model for each exploitation type. Simulations were used for long-term keeping the same cultivation techniques adopted by farmers, and using climate data generated from 2011 to 2040 with and without climate change. First, we study the response of *T. alexandrinum* crop in long-term and climate change impacts. Secondly we adopted scenarios to increase the initial level irrigations doses respectively to 20%, 40%, 60% and 100% to determine the effects on yields and soil salinity.

3. RESULTS AND SIGNIFICANCES

The results are related to *T. alexandrinum* yields to four farms during thirty years (2011-2040). Table 2 summarizes the average yields *T. alexandrinum* per decade regardless of climate change. The average *T. alexandrinum* yields related and soil salinity are summarized in tables 3 to 4, and shown in figures 2 to 4.

Table 2. Average *T. alexandrinum* yields (t/ha) to four farms per decade regardless of climate change

Decades/Exploitations	EXP 6	EXP 11	EXP 20	EXP 64
Average yield/ first decade	59.9	57.5	62.9	57.7
Average yield /second decade	58.4	56.4	61.6	54.8
Average yield/ third decade	55.2	53.5	58.6	52.4

EXP : Exploitation

We note that yields *T. alexandrinum* (Table 2) will suffer long-term declines. During the second decade, average *T. alexandrinum* yields for all farms EXP6, EXP11, EXP20 and EXP64, will suffer declines of 1.7 tons, 1.2 tons, 1.5 tons and 2.5 tons over the first decade. During the third decade drop will be respectively 3 tons, 3 tons, 3.2 tons and 3.4 tons. Over the first decade the respective total losses will be 4.7 tons, 4.1 tons, 4.7 tons and 5.9 tons

Table 3. Average *T. alexandrinum* yields (t/ha) per decade with possible temperature increase to 1° C

Decades/Exploitations	**EXP 6**	**EXP 11**	**EXP 20**	**EXP 64**
Average yield/ first decade	52.3	52.1	50.3	50.0
Average yield /second decade	49.5	51.9	49.6	48.1
Average yield/ third decade	49.2	48.7	49.0	44.3

With a possible increase in temperature of 1 degree Celsius, the average yield (Table 3) will decline during the second decade. They will be reduced respectively by 2.8 tons, 0.3 tons, 0.7 tons and 1.9 tons over the first decade. During the third decade declines will be respectively 0.2 tons, 3.1 tons, 0.5 tons and 3.9 tons. The total losses will be respectively 3.1 tons, 3.41 tons, 1.3 tons and 5.7 tons over the first decade.

Table 4. Average *T. alexandrinum* yields (t/ha) per decade with possible temperature increase to 2°C

Decades/Exploitations	**EXP 6**	**EXP 11**	**EXP 20**	**EXP 64**
Average yield/ first decade	48.3	49.1	46.1	46.3
Average yield /second decade	45.3	48.2	45.7	44.6
Average yield/ third decade	44.0	46.1	44.0	43.0

With a possible increase in temperature of 2 degrees Celsius in the future, the average yield (Table 4), will decline during the second decade. These reductions will be 3 tons, 1 ton, 0.4 ton and 1.8 tons over the first decade. During the third decade declines will be 1.3 tons, 2.1 tons, 1.8 tons and 1.5 tons. The total losses are respectively 4.3 tons, 3.0 tons, 2.1 tons and 3.3 tons over the first decade.

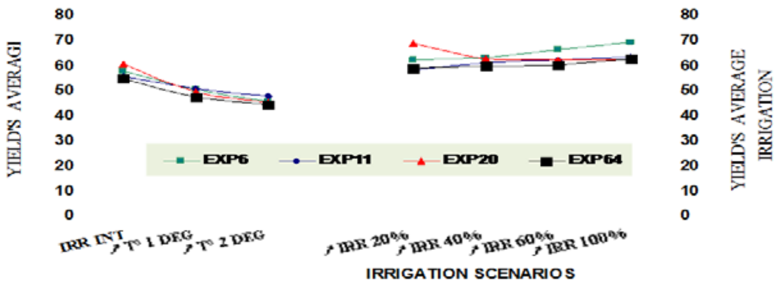

Fig. 2. Average *T. alexandrinum* yields (t/ha)

By doubling the amount of irrigation water for the three farms (EXP6, EXP11, EXP64) the average yields (Fig. 2) will increase until reaching respectively 69.5 tons, 63.7 tons and 62.8 tons., the average yield For the farm EXP6 will increase to 69.1 tons by increasing to 20% the amount of water applied. Improvements average yields are 11.8 tons for the farm EXP6, 8 tons for the farm EXP11, 8.2 tons for the farm EXP20 and 7.9 tons for the farm EXP64.

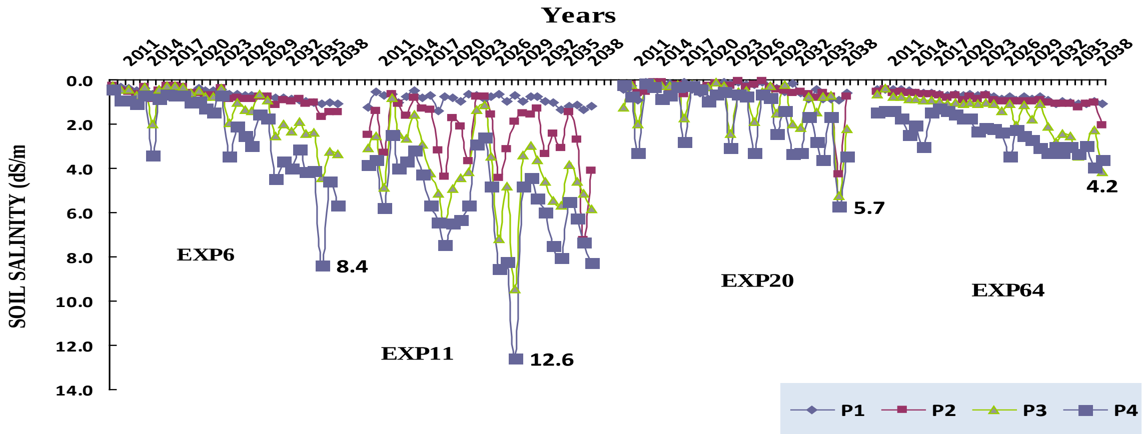

Fig. 3. Evolution of the soil salinity (dS/m) during thirty years (2011 to 2040)

“Cropsyst” model has a salinity module, interfering with other modules of climate and culture. It requires that we initially introduce the basic data on soil characteristics of each farm including soil salinity profile ranging from 0.10 to 1.5 m. Profiles are 4, namely: Profile 1 (P1), Profile 2 (P2), Profile 3 (P3) and Profile 4 (P4). The results for soil salinity expressed in dS / m, are given by soil profile. Figure 3 shows the evolution of the simulated soil salinity in the long term. The amount water initially used by farms, soil salinity (Fig. 3) will increase and farms. It would reach respectively 8.4,dS/m, 12.6 dS/m, 5 .7 dS/m, in profiles P4 for the farms EXP6, EXP11, EXP20, and 4.2 dS/m in the profile P3 for the farm EXP 64.

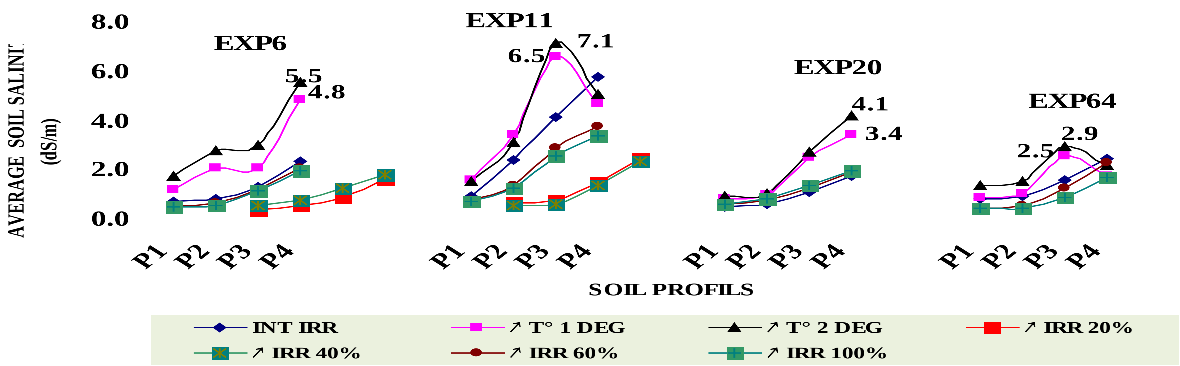

Fig. 4. Evolution of average soil salinity (dS/m)

The average soil salinity are included between 0.7 and 2.3 dS/m in the profile soil (SO22) of the farm EXP6, between 0.9 and 5.53 dS/m in profile soil (SO26) of the farm EXP11, between 0.4 and 1.73 dS / m for the profile soil (SO11) of the farm EXP20 and between 0.8 and 2.53 dS / m for the profile soil (SO19) of the farm EXP64. They will increase in response to climate change. Increasing temperature of 1 degree Celsius (Fig. 4) will increase average soils salinity for all farms. It could respectively reach 4.8 dS / m, 6.5 dS / m, 3.4 dS / m and 2.5 dS / m. This increase will be accentuated if the temperature is increased by 2 degrees Celsius. The average soil salinity could reach respectively 5.5 dS/m, 7.1 dS/m, 4.1 dS/m and 2.9 dS/m.

4. DISCUSSIONS

Simulations *T. alexandrinum* yields for thirty years for all farms (6 EXP, EXP 11, EXP 20, EXP 64) show that the average yields will suffer long-term declines that could reach respectively 7.9% 7.1% 7.4% AND 10.2% per hectare. Decreases in yields will be accentuated when the temperature is increased by one degree Celsius. They spend respectively 12.8%, 8.7%, 18.5%, and 13.6%. They will be accelerating, with temperature increase to 2 degrees C. They could respectively reach 20.6%, 14.3%, 25.7% and 18.8%. These changes are related to farming techniques, the soil types and berseem behavior with environmental conditions. These results corroborate previous studies which have shown that climate change will have drastic consequences on agriculture (Gerald et al., 2009). Large studies have shown that the tropical regions will most suffer from the negative climate change consequences (Reilly and Hohmann, 1993). In the tropics regions with dry seasons, projections show decreasing crop yields, even for small local temperature increases of 1 to 2 ° C (Seguin, 2010). It appears unlikely to escape to increase temperature to 2 or 3 ° C in the century end, and to predict inevitable consequence of adaptation (Howden et al., 2007). Soil salinity will grow in the future. However, salt will leaching depth beyond the P4 profile by convective transport (Lahlou et al, 2000) water percolating, with an additional irrigation. The average soil salinity will be reduced between 0.06 dS / m and 4.5 dS/m. This variation is related to the behavior of each soil according to its structure (Badraoui et al, 1998), the environmental conditions and the volume of initial water applied. Results show a significant impact on T. alexandrinum yields, causing reduced about 26%. So, we could conclude that the crop model is applicable in the region studied (Belhouchette, 2004 Abbes, 2005; Mlaouhi, 2012).

5. CONCLUSIONS

T. alexandrinum simulations practiced in irrigated area located in lower valley of the Medjerda, northeast of Tunisia, showed that Cropsyst model simulates quite accurately soil and climatic conditions of the region, in Tunisian contexts. Results show that soil salinity will increase; unlike the yields will decline in the future and more with climate change. The irrigation scenarios adopted increase yields and reduce soil salinity. So, the farms EXP11, EXP20, EXP64 should double the amount of water initially applied. For the farm EXP6 the amount of water applied initially should be increased by 20%. Indeed, additional doses of water allow a better consideration of the environment and reduce the impact of salinity on the natural environment and better value the treated wastewater

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Detailed Proposal to Adopt Decision Support Systems (DSS) for Integrated Water Management in Romania

Mary-Jeanne Adler

Abstract – **The basic DSS components for integrated water management proposed by WATMAN Project application, include: sensors and data processing hardware and software that measure and transmit the state of the system, the data base, a forecast system that estimates future states, "Dispecer Ape", a dispatcher system to disseminate information on system conditions, calculation results and evaluation against an expert system, analytical processing components, including flood, spill and water supply condition evaluation models; and the expert software system and associated database to organize information characterizing flood, spill, and water abstraction emergencies and execute routine tasks in hazard recognition and notification. The presented WATMAN concept is applied in the detailed software designing by e-LAC Project – „Pro-active operation of cascade reservoirs in extreme conditions (floods and droughts) using a Comprehensive Decision Support Systems (CDSS). Case study: Jijia catchment implementation".**

Keywords – **Arges reservoirs cascade, e-LAC Project, flood attenuation, water distribution optimization, WATMAN Decision Support System.**

1. Introduction

A Decision Support System (DSS) for water management is an integrated, interactive computer system, consisting of analytical instruments and information management capacities, designed to support decision makers whom it helps solve relatively broad and unstructured water management problems. In this context, the decision makers are the planners, managers and operators of water resource systems in charge of solving water related problems or meeting water supply demands.

The integrated water management in Romania is described in the National Environmental Action Plan which stipulates a balance between water use and nature protection. Related water management decisions concern: providing good water quality supply to the population, protecting water source quality, reconstructing riverside ecosystems and reducing flood risks.

ANAR (National Administration Apele Romane) has the following main responsibilities in which the WATMAN DSS (WATMAN Project final report, 2008-2010) will be of major help:

- Administration and operation of the National Water Management infrastructure;
- Surface and ground water resource management and development;
- Development of River Basin Management Plans;
- Developing hydrological, hydro-geological and water management yearbooks, reports, case studies and rules, monographs, impact studies, environmental audits;
- Hydrological and water resource monitoring, including the development of diagnostics and forecasts;

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- Set up and management of the National hydrological, hydro-geological and water management Database;
- Warning and prevention, control and remediation of flood effects through hydro-technical works; and
- Warning and preventing flood and spill effects.

The National Institute of Hydrology and Water Management (INHGA) is the technical and research body of ANAR. INHGA is involved in the following activities for which the WATMAN DSS will be of major assistance:

- Develop and improve data collection, transmission, and quality control, modeling systems for the preparation and dissemination of flood related forecasts;
- Develop and improve flood forecasting models;
- Prepare hydrological measurement schedules of flood characteristics; and
- Warn, forecast and inform on floods.

INHGA provides the specialist knowledge for hydrological data collection and processing and river and flood modeling. ANAR provides specialist support for water quantity and quality management and transmission of adequate hydrological information to the public and to private agents.

In developing the WATMAN DSS three main water management decision areas have been considered:

- Water allocation, regulation and quality – in regard to water supply and quality for municipalities, agriculture, industry, power and environmental protection;
- Flood management – in regard to reservoir operation, derivation of flood flows, hydraulics in channels and facility operation; and
- Spills – in regard to unintentional introduction of chemical or biological agents into the waters.

Decision making regimes tend to differ in these areas due to availability of time for decision making (days or months in the first case and hours in the second). Each of these areas and how the WATMAN DSS addresses them will be discussed in the following sections.

2. WATMAN DSS CONCEPT

The water management decision making process follows a cyclic pattern of data collection and processing; problem analysis; use of analysis and specialist decision support ort plan formulation assistance; and, finally, implementation of the established decisions (see **Fig. 1**). DSS for water management consists of three main interactively integrated sub-systems:

(1) user interface;

(2) set of models integrated with the interface and database; and

(3) information management sub-system (i.e. a database).

The WATMAN decision support system –DSS – concept [6] was planned to provide unavailable information for well documented water management decision making in Romania. It was designed for use by the engineers and researchers of the National Administration Apele Romane (ANAR) and to be operated, maintained and repaired by the ANAR personnel and their consultants. It uses real time data collected by ANAR, and historic data validated by the National Institute of Hydrology and Water Management (INHGA). Both types of data are stored in the databases created and managed by ANAR and INHGA.

The WATMAN DSS was integrated with the products and services provided by water management projects in Romania, including the Destructive Waters Abatement and Control (DESWAT) and Integrated Meteorological Surveillance Forecast and Alert System (SIMIN) Projects. DSS is designed to allow integration and applications with older software developed and successfully used by INHGA. The WATMAN DSS is designed for rapid rollout, but the design strategy allows for phased extension and development, which is planned under the e-LAC Project designing.

Figure 2 illustrates the proposed WATMAN DSS concept — main components and connections for data and information flow between components. The purpose of this DSS is to provide timely information to the decision makers in regard to river conditions; so that actions may be taken to minimize the adverse effects of excess or dearth of water or low quality water

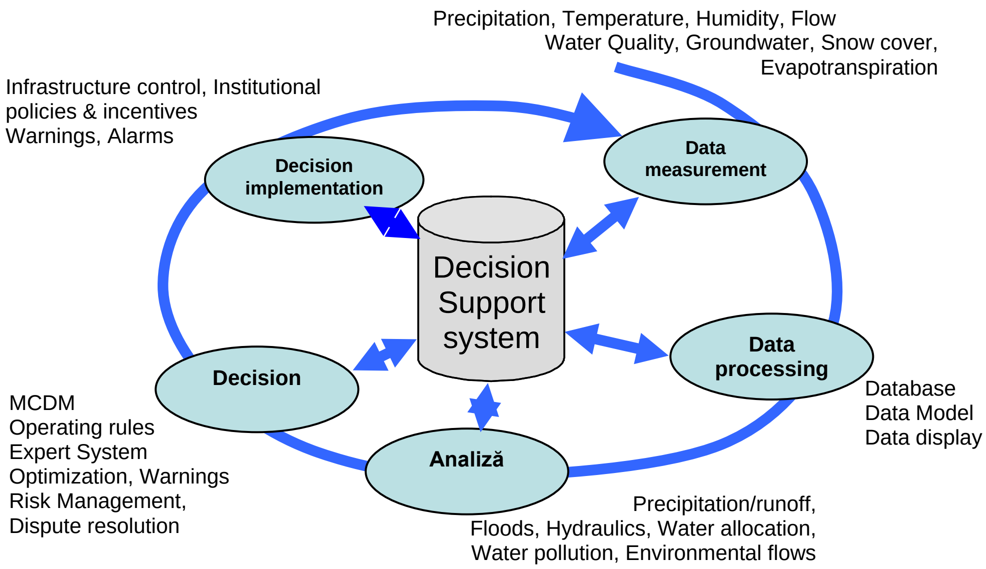

Fig. 1. Decision Support System phases

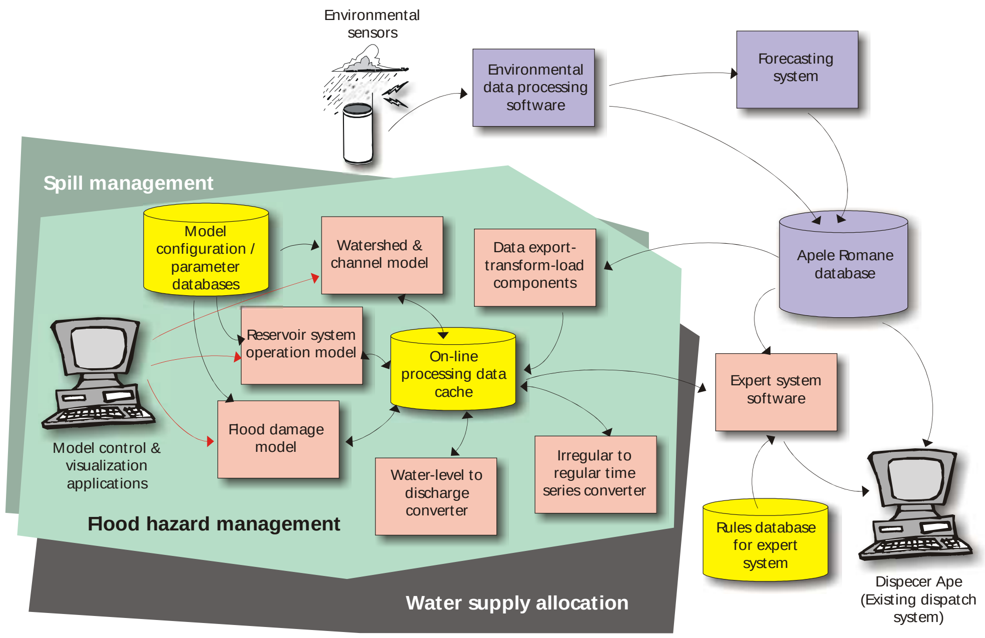

Fig 2. Decision Support System description

2.1. Water allocation, regulation and quality

The WATMAN DSS is designed to assist system managers and operators handling a multitude of water allocation, regulation and quality problems, including:

(1) River basin management – reservoir operation for various purposes;

(2) Non-point surface pollution – development of plans for the use of chemicals in agriculture or the protection of vulnerable water bodies, watercourses and aquifers and

(3) Groundwater management – planning and management of groundwater uses and aquifer protection.

The following describes three modules included in the WATMAN DSS:

(1) Alloc – a water allocation and distribution model by

(2) Mike-Basin – a multifunctional river basin management model, including surface water quality and links to groundwater abstractions and

(3) GMS – a comprehensive groundwater modeling system including aspects of supply quality.

- Alloc – the Alloc model is designed to be used by water system managers and operators in planning monthly and quarterly operation of reservoirs. It is a general model that may be adapted to any water management system, irrespective of form, number of reservoirs or water uses.

The water management systems are represented in the model as arc-node grids (see **Fig. 3**) where system operation becomes a problem of finding flows in the networks according to prioritized objectives and a variety of restrictions. The nodes and orientated arcs, and their characteristics (upper and lower limit and associated costs) represent the direct analog of the physical and operational characteristics of the system. Arcs represent the rivers, channels or pipes, water demands or other requirements, reservoir volumes, etc. The nodes are intersections of at least two arcs and represent reservoirs, river confluences, water inflows or wastewater discharge points, etc. Water sources, consumer demand and infrastructure capacity are expressed as restrictions with the upper and lower limits represented by arcs. Operations, i.e. storage/emptying of water into /from the reservoirs or derivation from one part of the system to another in order to meet demand and the economic benefit /loss ratio for the user are defined by the objective function and expressed as costs associated to the arcs

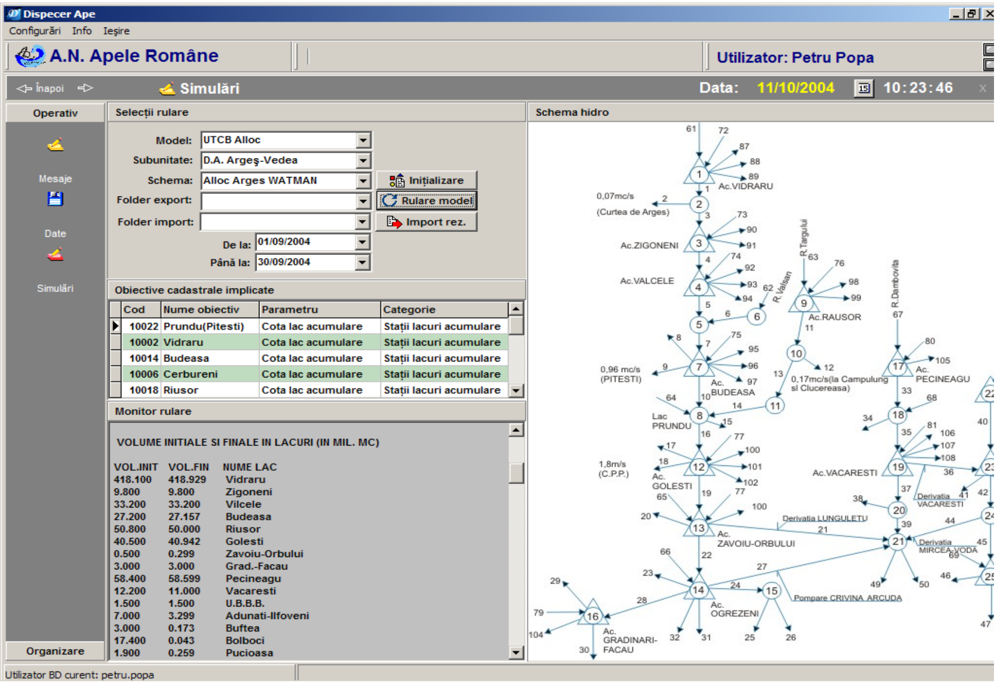

Fig. 3. Alloc Model, intregration in Dispecer Ape interface

Models formulated in Alloc are solved by using an optimization procedure, linear programming of integers using the Out-of-Kilter algorithm. Alloc is recommended for the WATMAN DSS, with a copy installed at each of the 11 basin offices, at the national ANAR office and at INHGA. For implementation, the model must be made compatible to the ANAR database and Dispatch/ Dispecer software.

- Mike-Basin – Mike-Basin is coupled to the Geographic Information System ArcGIS of hydrological modeling to analyze water availability, water demand, multifunctional reservoir operation, transfer/derivation structures and possible environmental restrictions in a river basin. Mike-Basin uses a quasi-stabile mass model with grid representation of hydrological simulations and derivation of river flows, where the arcs represent flow sections and the nodes represent confluences, derivations, reservoirs or water users (similar to Alloc). ArcGIS is used to display and edit network elements. It can model water quality simulations assuming advective transport and degradation. Groundwater aquifers may be represented as linear reservoirs.

The basic input data for Mike-Basin consist of data time series for runoff /drainage in the basin for each tributary, reservoir characteristics and operating rules, met data time series and data on water demand and rights (for irrigations, municipal and industrial water supply and power generation) and information describing return flows. The used may define derivation and abstraction priorities for multiple reservoirs and priorities for water allocation to multiple users. Reservoir operation policies may be expressed by rule curves defining the preferred storage volume, water levels and discharges at any time as a function of the existing storage volume, time of the year, water demand and possible inflows.

Water quality modeling using Mike-Basin is based on constant uniform flow in every river sector and a mass balance that takes into account constituent input, advective transport and reaction in the respective sector. It assumes complete mix downstream of each source and at confluence points. Non-point pollution sources are also treated by the model as well as direct loads from point sources. The model can calculate the following water quality parameters in rivers: oxygen biochemical demand, dissolved oxygen, ammonium, nitrates, total nitrogen and total phosphor. Loads from non-point sources are represented by the use of a surface loading method that takes account of nitrogen and phosphor loads from small settlements, livestock farming and agricultural land assuming certain unit loads for each category.

- GMS – The Groundwater Modeling System (GMS) is an integrated set of instruments for groundwater modeling, including site characterization, model development, calibration, post-processing and visualization. For the WATMAN DSS, we recommend the following GMS package (or equivalent): Map, Grid, Modflow, MT3D, Hardware lock, Subsurface Characterization, and Geostatistics. A copy of this software will be installed at each of the 11 basin officies at national headquarters ANAR and at INHGA. Model application in the 11 basin offices of ANAR will require acquisition of a significant amount of data and model calibration. The ANAR personnel at the national and basin level will have to be trained in using the model.

The GMS package will include Modflow, a three-dimensional, finite difference groundwater model of modular structure. Modflow simulates constant and inconstant flows in an irregular flow system where the aquifers may be closed, open, or a combination of open and closed. Flows from external stress, such as well flows, recharging by area, evapo-transpiration, leak flows and riverbed flows may also be simulated. Hydraulic conductivities or transmissivities for any layer may differ in space and may be anisotropic (restricted by aligning the main directions to the grid axes), and the storage coefficient may be heterogeneous. Heads and specified flow barriers may also be simulated, as well as a head dependent flow throughout the outer limit of the model, that allows water supply to a limit block on the area represented in the model at a proportionate flow to the current head differential between a water "source" outside the modeled area and the limit block.

Besides Modflow, the GMS package will include MT3D, a 3D numerical model simulating dissolved material transport de across complex hydrogeological systems. MT3D takes into account advection in complex fields under constant or transitory flow conditions, anisotropic dispersion, first order break down and reaction products and linear and nonlinear absorption. MT3D is based on a modular structure that allows simulation of the independent or combined transport components. MT3D has a direct interface with Modflow for head solutions and supports all the Modflow hydrological characteristics and of discretisation.

2.2. Flood Management

Flood control requires larger models with smaller time spans than most other models of water resource management. Flood flows typically occur over very short intervals of time (hours to days or weeks). Calculating flood volumes as a result of flood wave propagation across a basin requires bi-dimensional instead of uni-dimensional modeling, which requires the collection of detailed terrain and elevation data along the riverbeds. The following describes three modules included in the WATMAN DSS: (1) Vidra – hydrological model for runoff and precipitations, (2) ResSim – operational model for reservoirs and (3) Unda – hydraulic model for river channels and Vidra Model, simulating hydrological in-puts for Unda Model

Vidra is a runoff and precipitation model at basin where the basin is divided into sub-basins based on a topological runoff formation and integration scheme; for each sub-basin snowmelt water is calculate with the help of a degree-day method; precipitation and thaw water is calculated on average for every sub-basin; the depth of effective precipitation (runoff) is determined with the help of a conceptual model of infiltrations for each sub-basin. A consistent hydrographic procedure is applied in order to obtain the hydrographs of discharge flows by sub-basin, which are then integrated into the nodes representing the topological outline of the basin. This will continue tracing the flow to the point of exit from the basin.

The Vidra model was calibrated and tested for almost all the basins in Romania using of regionalization relations. An application of the flood forecast model for the Argeş basin is shown in **Fig. 4**.

The Vidra (or equivalent) model is recommended for use in the WATMAN DSS. For full implementation of the model, it will have to be made compatible with the ANAR database and the Dispatch software. Model application in the 11 basin offices of ANAR will require acquisition of a significant amount of data and model calibration. The ANAR personnel at the national and basin level will have to be trained in using the model.

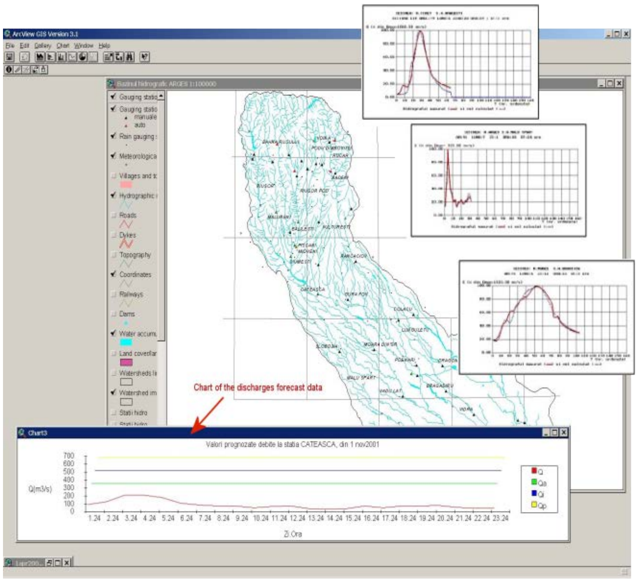

Fig. 4. VIDRA Model into DSS interface

ResSim – Reservoir System Simulation (ResSim) was created by the US Army Corps of Engineers – Hydrological Engineering Center. Res-Sim has a graphics interface with the operator (GUI) and uses the HEC data storage system (HEC-DSS is used to simulate reservoir operation including all the reservoir characteristics and hydrological channel derivation downstream). The model allows user defined alternatives and simultaneous simulation thereof to compare results – **Fig. 5**. Grid elements include reservoirs, derivation sectors, inflows and connections. In ResSim, the basins include creeks, projects (i.e. reservoirs and dikes), landmark location, impact zones, location of time series and hydrological and hydraulic data for the respective area. The schematic elements of ResSim allow basin representation, reservoir network and data simulation in visual and geo-referential context that interacts with the associated data. The reservoirs are complex elements consisting of a pond, a dam and one or several discharge channels. Criteria for reservoir discharge decisions, an operational set, derive from a set of discrete zones and rules. The zones divide the reservoir based on level and contain a set of rules describing the objectives and restrictions that need to be met when water levels in the reservoir are in the respective zone. Alternatives are developed for result comparison with the help of schematic models (physical properties), operational sets, inflows and/or initial conditions. To help review the simulation results, ResSim contains model graphs, a variety of summary reports HEC-DSSVue. ResSim does not treat water quality, ecological, recreational inflows, etc. The only aspect it deals with is power generation as a characteristic of the reservoir.

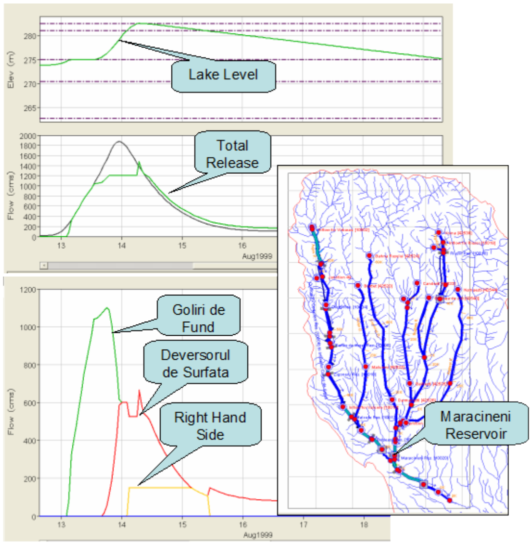

Fig 5. ResSim integration for reservoir management

For the WATMAN DSS, the ResSim model (or equivalent) was recommended. A copy of this software is planned to be installed at each of the 11 basin offices, at the national ANAR office and at INHGA. For full implementation of the model, it will have to be made compatible with the ANAR database and the Dispatch software. Model application in the 11 basin offices of ANAR will require acquisition of a significant amount of data and model calibration. For the pilot basin Rausor application, Unda Model was calibrated for flood propagation. Unda is a model that provides uni-dimensional simulations of inconsistent, free surface flows. It is based on a Saint Venant set of equations, numerically integrated by means of finite differences on a rectangular grid in the X-T plane, into an implicit scheme of equation linearization. The initial condition is considered to be constant flow. An example of how model results may be used to forecast various flood waves in the Argeş Basin is shown in **Fig. 6**.

For the WATMAN the use of the Unda – INHGA Hidraulic Model (or equivalent) model was also recommended. Model application in the national Forecasting Centre of INHGA and the 11 basin Hydrological Services offices of ANAR will require acquisition of a significant amount of data and model calibration. The INHGA will help for customizing the models latform and ANAR personnel at the basin level will have to be trained in using the this model platform.

2.3 Considerations in model implementation

Significant implementation efforts will be required of the ANAR specialists and staff in developing and successfully rolling out the WATMAN DSS and the modules of its components. Many modules will need to be significantly adapted for compatibility with the dispatch software and ANAR database. Deployment at basin level will require development of precise models for all the basins in the country, a significant task entailing considerable efforts in data collection, calibration and validation. To follow the process, eLAC Project [2] was planned, for developping a full application for one component of flood management; this application will be made in Jijia pilot project, where is available a detailed DTM, data of maximum discharges and flood characteristics data, and flooded areas in different hypothesis. For a successful development, and extension of the system substantial resources have to be allocated to the design of WATMAN DSS development, adaptation, implementation and training for Jijia Basin and Prut Basin Water Administration.

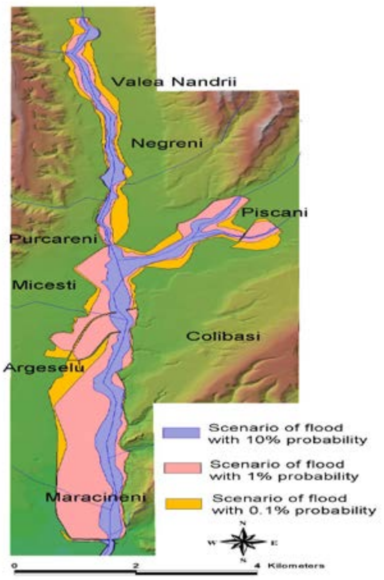

Fig. 6 UNDA Model – flood hazard mapping

3. WATER MANAGEMENT DSS TECHNOLOGY

As shown above, a decision support system consists of several components, in particular: an interface, a model base and a database. This section will describe the interface and database.

3.1 DSS Interface

WATMAN DSS will be based on the current display and reporting software Apele Române Dispatch (Dispecer Ape) now used by the dispatcher system of ANAR. Dispecer Ape is a client-server distributed application, based on operational branching flow environmental data from Level 4 (sensor level) to Level 1 (national level). The application was designed to perform 2 main functions: data flow management and operational messages surveillance and control. The dataflow consists of hydrological, reservoir and water quality observations (see **Fig. 8**). The operational messages consist of 'system state' reports (regarding meteorological, hydrological, hydro-technical structure, water supply, water quality, information system and operation statement data), explicit warnings /alarms, events, reports on the effect or damage caused by floods, etc. (see **Fig. 9**); and serve as a framework for the application (reservoir operation, monitoring and hydrological and water quality forecasting modules) and studies. This interface is planned to be further improved for a successful development and implementation of the WATMAN DSS by eLAC Project. Significant further development will also be needed to integrate the system components within this project and for interfacing with the DESWAT project. For further designning a multimodel platform is planned to be developped during the e-LAC Project implementation, as well as to be designed the multi-model optimization procedure.

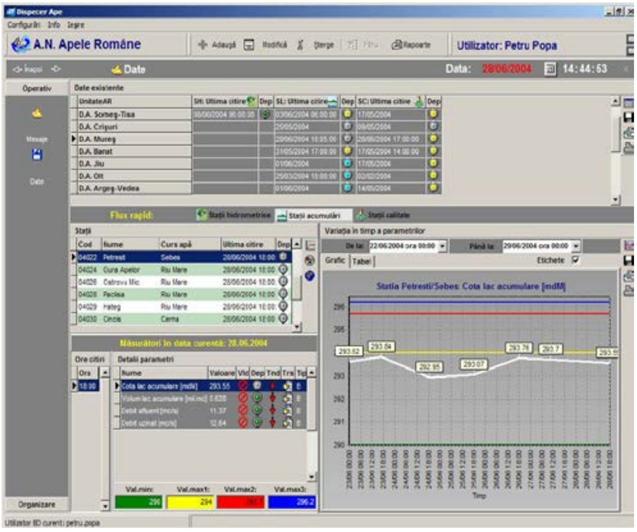

Fig. 8. Dispecer Ape Data Collection Interface

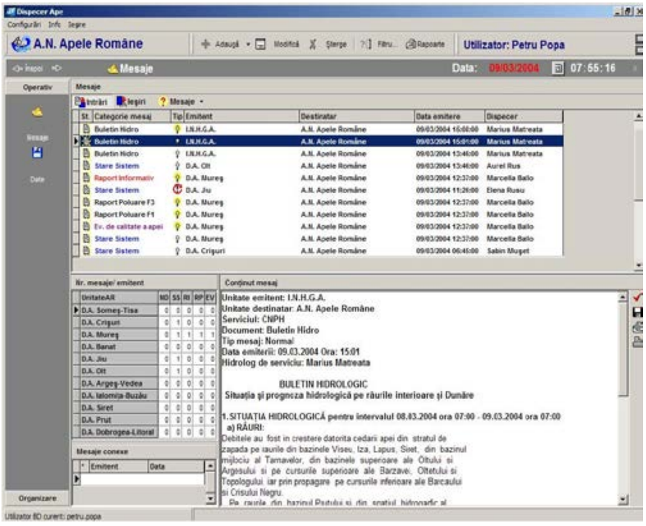

Fig. 9. Dispecer Ape Interface for messages and operational information

Designed and developed on a Microsoft SQL-Server platform, Dispecer Ape is also a data feeding system, based on a characteristic of data duplication on servers. Information is directed up and down through this branching system, to meet specific needs.

Data acquisition follows this sense within the network level1 and 2 (Hydrological Stations). Once validated and processed, historic and forecast data follow the uncertain path and reach Level 3 (Hydrological Basin Services), data collection to national level – ANAR - INHGA.

3.2 DSS Database and Expert System

Operational database – Operational (real time) databases exist at all the dispatcher levels: National (including INHGA), Regional and Local (SGA). The National Dispatcher office is the only one to contain data from all the regions. Regional office databases contain only data for the respective region. A new, SQL database is being developed at the National Dispatcher office, to deal with all the necessary data for ANAR. The database is designed to use the ArcHydro data model for hydrological and modeling data. Currently, ANAR owns a SQL database of economic information, used as a model for the hydrological database.

All the relevant data from the water management system are stored in the ANAR database. Stored data include numbers, words and images describing the historic and current state of the river basins, riverbeds, water control installations and weather — as well as the future state, as the forecasting system is configured to handle water flow, water levels and others in the same database. This database is the cornerstone of the WATMAN DSS, which mainly relies on the database to establish source conditions for analytical programs.

Historic database - INHGA is preparing the historic hydrological and meteorological database at the national level, based on validated data received in real time from on-site measurements. This is in ORACLE format. The database is regularly (monthly) updated with validated data. This database generates annual reports of daily data.

Nowadays, database systems provide comprehensive data storage, accessing, display and handling solutions, essential for decision making processes. Two common handling and storage systems or instruments are the relational database, relating tabular information so as to enable application of relational algebra rules and the *geographic database* (or geographic information system-GIS), relating information on fundamental spatial features such as points, lines and polygons. GIS not only brings the spatial dimension into the traditional water resource database, but, more importantly, allows better integration of various social, economic and environmental factors related to the planning and management of water resources used in the decision making process. A really useful water management DSS requires a data model with geometrical representation and spatial reference and open architecture, to facilitate GIS and model integration. In the pilot basin for WATMAN concept application, was used ArcHydro. ArcHydro defines a data structuring by class (e.g. basins, cross sections, monitoring points and time series) in a way that reflects the underlying physical basin; ArcHydro defines relations between data, so as to identify which point within the basin represents discharge or find the time series of records for the location of a monitoring point and may operate using the time series data and data visualization. ArcHydro facilitated close coupling of water resource models and GIS by establishing connectivity between the hydrological characteristics in the landscape that may be used to trace water flow between two physical elements of the model. The ArcHydro toolkit also helped in calculating certain attributes useful for the models, by attribute accumulation routines, relations, network associations, or direct calculation of parameters such as distance from a point in the network to the discharge of the river system. Water resource system modeling may be done by data sharing between ArcHydro and an independent model attached to ArcHydro with by means of a dynamic link library. ArcHydro is fully compatible with the RiverSpill and MikeBasin models, recommended for the WATMAN DSS.

For the WATMAN DSS was recommended extensive development of the GIS capacities within the ANAR system, including by developing the ArcHydro datasets for each basin. In particular, WATMAN recommended ArcGIS implementation at Level-1 National Dispatcher, Level-2 Regional Dispatchers and Rapid Response Centers. To derive full benefit from the use of GIS in the DSS, the ANAR database is compatible with the GIS data and software, developed under WIMS Project. This could be easy integrated under the Expert System. Consisting of a user defined set of rules and data interacting via an inference engine, an expert or knowledge based system may derive or deduct new facts or data from the existing data and conditions – **Fig. 10** [2].

Aviso Watch was the expert system engine used for the pilot basin, which database associated to rules was included in the DSS. The objectives of the expert system are threefold:

- *Librarian function* Among other tasks, the expert system analyzes input environmental data and compares them to predefined trigger values indicating a hazard condition. The trigger values are stored in the expert system database under the if-then rules thereof. This requires consistent organization and interpretation of information for hazard recognition

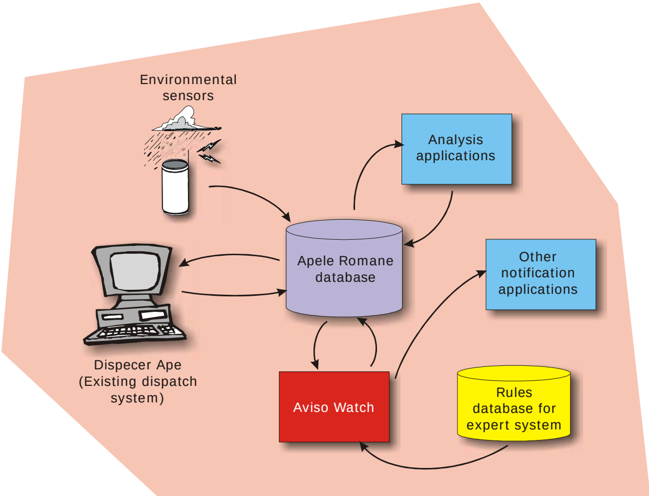

Fig. 10. Aviso Watch configured for ANAR

- *Consultant function*. The expert system provides a professional experience sharing mechanism between water and emergency managers, as such experience is formally requested and memorized in the rule content. For example, if the experience of an experienced reservoir operator shows that 2 vanes should be open when reservoir water level reaches X meters and inflow rises at a speed of Y m^3/s per hour, then the necessary recommendations may be recorded and memorized as a rule in the expert system.
- *Assistant function*, taking over routine (possibly boring) tasks, e.g. examining every water level observation and notifying the dispatcher when the level reaches a value of 0.5 m below reservoir complete fill. This frees the true experts for more complicated tasks.

As seen in **Fig. 10**, the expert system uses the environmental database as data source, as was presented in previous paragraph. The expert system included the monitoring rules for the future inputs forecasted for the reservoir and future reservoir storage levels resulting from simulations [4, 8, 9]. Thus the dispatcher is notified via Dispecer Ape on conditions that call for special attention or action. The expert system may take many forms, but a rule-based system such as the flood warning system developed and rolled out in the USA by David Ford Consulting Engineers Inc. [2] is also recommended for the WATMAN DSS [7] (see application examples on http:// www.avi.so and http://ptwc.weather.gov/). That system, now known as Aviso Watch, includes a rule editor, an inference engine to "trigger" the rules, its own dispatch application (which will be replaced here by Dispecer Ape), flexible connections to a variety of database structures and a flexible database for knowledge (rules). Such rules were described in some other works as [1], [3], [5], [6].

4. CONCLUSIONS

The value of a water management decision support system (DSS) largely derives from its capacity to provide the results of informed analysis in a timely, consistent and easy to understand manner. Timely provision of information has a very critical role in the DSS application of emergency management. For example, correct application of a flood warning DSS may bring significant tangible benefits if sufficient warning time is provided for the lifting or removal of goods from the flood path. Similarly, the DSS may protect lives if the information provided leaves enough time for the exposed population to reach higher ground.

Hazard recognition computer systems are a special category of DSS. Such a knowledge-based system uses economic rules for the specification of facts related to conditions qualified as threatening and the identification of actions to be taken when the threats have been identified. The rules are incorporated in what information technology specialists call economic management rules or rule database. Considering the above mentioned aspects, the new e-LAC Project envisages the elaboration of a Comprehensive Decision Support System (CDSS) composed of an Expert System (ES), a Supervisory Control and Data Acquisition (SCADA) system, together with an Advanced Control Technology (ACT) system assisting the authorities to prevent the disasters in both cases (flood and drought). WATMAN Project DSS recommendations will be implemented and adequately developed in a dedicated application for Jijia Basin; here, connection with the operational database and hydrological forecasting model, including reservoir application will be developed in partnership with University Babes Bolyai of Cluj, IPA Cluj, University of Civil Engineering, Bucharest, the National Institute of Hydrology and Water Management Bucharest and S.C. Duk-Tech S.R.L, the future integrator for the IT products designed under the partnership of ELAC Project., as: development of mathematical models, of CDSS and an Advanced Control Technology (ACT) applied to the Jijia river sub-catchments, CDSS validation for the optimal use of water reservoirs along the Jijia river and provide the promotion of CDSS application and training of beneficiaries – Prut Basin Water Authority. The new e-LAC partnership hope by this new project to increase the international visibility of the Romanian scientific research in the field of water resources management and its social-economic impact at national and international level.

5. ACKNOWLEDGMENTS

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The Indoor Air Pollution as the Environmental Factor on Quality of Life

I. Škultétyová

Abstract – **There are many activities focusing on life quality improvement of population in Slovak republic, which show the negative effects of ingredients and factors of environment on human health. The study on indoor air pollution is based on data collection and on a common methodology to implement the measurement and evaluation of selected chemical, physical and biological factors in the school environment. It also assesses the impact of external factors affecting air quality in this area. The first stage in this study was the realization of a comprehensive research on exposure monitoring of pollutants in the indoor environment of buildings. The goal of the paper is to extend the current range of knowledge in the field of improving the life quality, on the basis of experimental results obtained in Slovak schools.**

Keywords – **Environment, Human Health, Indoor Air Quality, Quality of Life.**

1. Introduction

Researchers across multiple disciplines have taken an interest in quality of life issues. World Health Organization (WHO) defines quality of life as "individual´s perception of their position in life in the context of the culture and value systems in which they live and in relation to their goals, expectations, standards and concerns" [1]. It is broadly defined as "life satisfaction", it is a comprehensive concept influenced in a complex manner by the individual's health, psychological state, degree of independence, social interrelationships, personal beliefs and their interdependence to noticeable features of their environment.

Environmental conditions have a considerable effect on people's sense of life contentment. The quality of the water, air, food and usual living conditions, together with the climatic elements of wind, sun and rain all contributed to the well-being of persons. Various chemicals are emitted into the air from both natural and anthropogenic sources. Organisation for Economic Co-operation and Development (OECD) provides a summary of evidence and quantitative estimates of the impact of the long-term effects of risk pollution on mortality [2]. The OECD forecasts of number of premature deaths from selected environmental risk are given in **Fig. 1**. The **Table 1** presents percentage of urban residents with dissatisfaction of the quality of air in their city or local area by socioeconomic characteristic.

Health problems associated with the built environment and intolerance towards harmful pollutants brought about laws which aimed to provide better, safer built environments. These facts are now slowly appearing in home and work environments in order to get health standards to an improved quality. Modern day health problems associated with indoor environment, in particular those connected to indoor air quality, lead to four

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fundamental developments such as the increase of the period of time spent in indoor environments; the enhanced dependence on artificial products; energy conservation methods; and advances in medicine.

Table 1 Percentage of urban respondents with dissatisfaction of the quality of air in their city or local area by socioeconomic characteristic

Socioeconomic characteristics		**Percentage of urban residents [%]**
Gender	Men	34
	Women	36
Age	< 25	35
	25-44	37
	15-64	35
	65+	29
Education	Elementary	32
	Secondary	34
	Tertiary	37
Health status	Satisfied	33
	Non satisfied	42

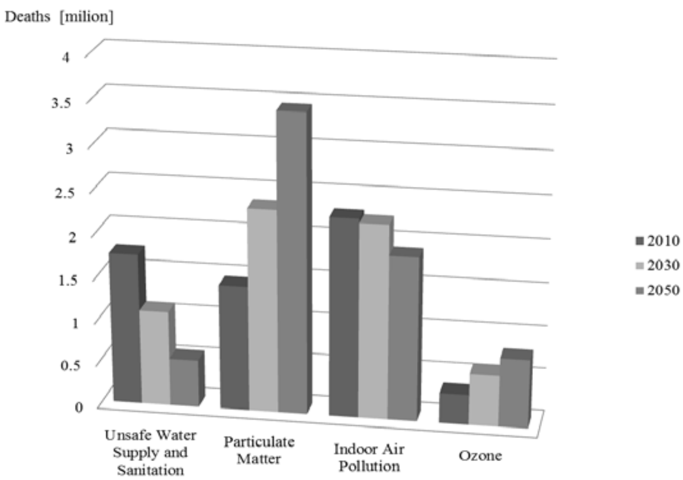

Fig. 1. Global premature deaths from selected environmental risks (2010 to 2050) [2]

Well-being and good atmosphere, as well as proper development of students are closely related to healthy school environment. In particular, it is the indoor air quality, which affects the physical, chemical and biological environmental factors. Therefore, it is necessary to know the properties of the environment around us, whether at home, at work or at school. What is the composition of the objects around us, which is the composition of air in the room in which we find ourselves? Room air is the most important factor in the quality of indoor air. Air pollution may reduce welfare and human health. The knowledge is very important for the formation of a conscious approach to health care and the environment.

The air has some specific characteristics. Clean air does not contain any solid, liquid or gaseous pollutants, and in fact it does not occur under natural conditions, because of the dynamic changes constantly taking place between the atmosphere, land surface, hydrosphere, biosphere, etc. Indoor air affects the health of humans, causing health problems such as headaches, depression, insomnia. Sources of pollution are various and numerous. The most vulnerable group is paediatric population, because they react more sensitive to environmental factors.

Average daily time allocation shows that spent time is composed of 4% - outdoors, 6% - in vehicle. 40% - in the work, 50% - in the residence. Most people spend 85-90% of their days in indoor environment, either at homes, at workplaces or at schools, universities, shopping centres and vehicles (the urban inhabitants and in the cold climate areas). The period of time an average individual spends indoors has become considerably longer due to a few compounding factors (change of the rural to urban lifestyles, free time home activities, sports and shopping in buildings etc.) which increase the indoor human activities.

Building materials, furniture, paint, surface materials have changed. Flats have lower ceilings than in the past, therefore the concentration of hazardous substances in the air is greater. We often use gas for heating. More and more sources of harmful substances that are released into the air are in homes and in classrooms.

The air quality of the indoor environment has the capacity to affect human comfort in many different ways, depending on the contaminant. Each of the contaminants has different effects on the human body. The indoor air quality (IAQ) may have a significant influence on health (reduced lung function, worse asthma, significant occurrence of chronic bronchitis, nonfatal heart attacks, irregular heartbeat, and early death among people with heart or lung disease) and welfare and comfort of occupants, which may impact the performance and productivity [3].

2. METHODOLOGY

Study Area. Analysis of factors influencing the (IAQ) was performed in the context of examination of relationship between contaminant exposure in the school indoor air and health condition of children under the international project “Project SEARCH – School Environment and Respiratory Health of Children” [4]. This project is based on the European Action Plan on Environment and Health, the implementation of the EU strategy on environment and health and Children European Health Action Plan for Europe (CEHAPE), which was signed by 52 countries. It comprises the main constituents: environmental monitoring, human health assessment and energy consumption evaluation.

The first stage in the environmental monitoring, human health evaluation and energy consumption assessment was the school selection, such as:

• 10 schools/ country;
• About 100 children/ school;
• Children age ranked between 8 and 11;
• School selection was done according to the building type (new/old or light/traditional construction); and polluted/clean environment.

There are lots of cities, schools, classes and children involved in the project in different countries, as it is shown in the **Table 2**.

Table 2 Number of cities, schools, classes and children involved in the project in different countries [5]

State	**Number of**			
	Cities	**Schools**	**Classes**	**Children**
Albania	3	10	35	1020
Bosnia	3	10	40	-
Hungary	4	10	43	747
Serbia	7	10	44	735
Slovakia	2	10	40	908
Total	19	50	202	3410

Data. The sampling in SEARCH project was carried out in the breathing zone of the selected set of children and in open outdoors air in the area near the windows of the classrooms, in the heating period. Sampling was implemented by using the same Instrument Technology and consistent passive dosimeters at all schools. There are given the measured variable parameters of IAQ in the **Table 3**.

Table 3 The measured parameters of indoor air quality (IAQ)

Parameters	**The method of the measurement**	**Duration of the sampling**
CO_2, T, H	IAQ – monitoring during lessons	1 class/day
PM_{10}	HAZ DUST – PM_{10} – monitoring during lessons	1 class/day
NO_2, BTEX, HCHO	Passive dosimeters	3 – 5 days

The estimation of exposure is based on the results of IAQ measurements in the primary schools and the results of outdoor air quality measurements. The survey and the evaluation are then fulfilled by means of experimental measurement of exposure levels, information gathered by questionnaires and lung function assessment.

The exposure level measurement. The concentration of the considered pollutants (CO; CO2; PM10; benzene, toluene, ethyl benzene and xylenes - BTEX and formaldehyde - HCHO), as well as relative humidity (RH) and temperature (T), will be recorded during the heating season either inside or outside the schools in the participating countries aiming to establish children's exposure levels.

The information gathered by questionnaires. The following questionnaires were taken into account:

- School questionnaire: collects data about the school environment (type of the building, neighbourhood, heating methods, maintenance etc.). The team which conducts the measurements fills in the questionnaire (one for each school).
- Classroom questionnaire: collects data about classroom features (number and type of windows, floor and wall coverings, ventilation system, number of pupils in the classroom, furniture etc.). This form must be filled in as well by the team conducting the measurements (one per classroom).
- Parents' questionnaire: this is an anonymous questionnaire which gathers data on each child's health status (past and present), the home environment (heating system, building type, floor and wall coverings, living density, smoking habit and other lifestyle factors, and the family's socioeconomic status). This form must be filled in by the parents (one for each participating child). If the parents refuse to consider the questionnaire, the decision is their own and should be respected.
- Comfort questionnaire: collects data on the children's perception of comfort in the learning environment. One form must be completed by each participating child.
- Energy questionnaire: collects data about the school building and its energy performance. The energy consumption questionnaire is merged with additional information given by data loggers (10 days surveillance, three data loggers inside the school, and only two data loggers outdoors).

Spirometer*.* The child's and his/her parents approval is mandatory to be obtained prior to the lung function test (spirometer). The tests were carried out in a quiet spot in the school, during the morning.

3. RESULTS

In Slovakia, the project was implemented in six elementary schools in Bratislava and 4 primary schools in Banská Bystrica by staff of the Public Health Authority of the Slovak Republic [5].

The chemical composition of indoor air

The maximum allowable concentration of fine particulate matter PM_{10}, which represent particles of any substances that are less than or equal to 10 micrometers diameter, 50 $\mu g.m^{-3}.day^{-1}$ by the Regulation of Slovakia Ministry of Health No.259/2008 Collection of Laws was exceeded in 8 out of 10 schools monitored (see table 4) [6]. Average daily concentrations of fine particulate matter PM_{10} in indoor environments of the monitored schools ranged from 55 $\mu g.m^{-3}$ in Hungary to 100 $\mu g.m^{-3}$ in Bosnia. Average concentrations of NO_2 ranged from 12 $\mu g.m^{-3}$ in Albania to 22 $\mu g.m^{-3}$ in Serbia. The average concentration of NO_2 in Slovak schools ranged from

4,4 µg.m^{-3} to 30,8 µg.m^{-3}. The maximum allowable concentration (200 µg.m^{-3} NO_2) under legislative provisions concerning IAQ is not exceeded in either of the monitored schools (see **Table 4**).

Table 4 The parameters of indoor air quality in Slovak schools

Schools	**Temperature [^{o}C]**	**Relative humidity [%]**	**PM_{10} [µg.m^{-3}/24 our]**	**NO_2 [µg.m^{-3}/4 days]**	**CO_2 ppm**	**Benzene [µg.m^{-3}/24 our]**
Jesenského	23	34	37	22,4	1304	2,2
Majerníkova	23	34	42	6,8	1196	1,5
Novohradská	23	39	84	16,9	1767	9,6
Podzáhradná	23	36	104	14,1	1643	14,3
Vazovova	21	35	84	30,8	1439	2,6
Veternicova	22	38	63	4,4	1300	4,2
Bakošova	21	40	63	18,4	1212	2,4
Ďumbierska	22	45	67	10,6	1547	3,3
Tatranská	21	29	122	8,6	1631	5,3
Trieda SNP	21	46	142	12,2	2109	4,7
Maximum *			**50**	**200**		No safe level of exposure

*Maximum allowable concentration in SR

The impact of external factors on internal environment

In order to detect the impact of the quality outdoor air quality to IAQ of monitored schools held measurement monitored contaminants of the outside air near the objects tracked schools. To evaluate the microclimate parameters of schools indoor environment have been objectified in air temperature and the relative humidity (**Table 4**). In all the Slovak primary schools were provided satisfactory climatic conditions according to the Regulation of Ministry of Health no. 259/2008 Collection of Laws.

The questionnaires

The number of children involved in the study with questionnaires in the Slovak Republic, is given in **Fig 2.**

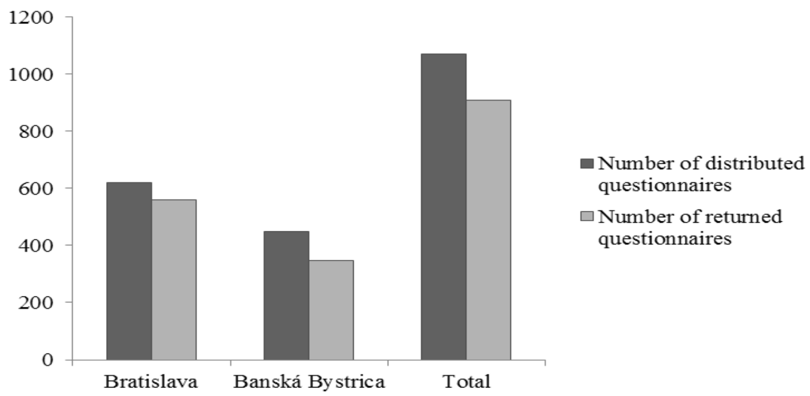

Fig. 2. Number of distributed and returned questionnaires in the study in Slovak Republic

4. CONCLUSION

There is a growing concern about IAQ in schools, in many countries. Indoor air quality is one of the most important environmental problems. Schools constitute a particular indoor environment because children represent a special susceptible group of the population. Indoor and outdoor concentrations of contaminants such as VOCs, carbonyls, NO_2, PM_{10}, OC, EC, carbonates, microbiological components and comfort parameters (CO_2, CO, temperature and RH) were measured in elementary schools. Thermal comfort is a key component of quality of indoor environments. Elements such as lack of heating systems, missing of an appropriate ventilation, high humidity levels, and inadequate performing building envelopes result in poor thermal comfort. If these factors are not appropriately addressed, schools become an environment inwhich either teachers or students must adapt to poor comfort levels. Such an inadequate working environment can be distracting or even harmful to students and teachers, and may reduce their productivity. The recommendations of Public Health Authority of the Slovak Republic [5] emerging from the study with respect to the protection of human health from risks caused by a number of chemicals usually present in indoor air are the following:

- Ensure the compliance of each type of instructional space with the minimum space - the capacity of theoretical lecture room (1,65 m^2 per 1 student in clear height of the room 3,3 m) and specialized study rooms -computer room, laboratory (2 m^2 per 1 student in clear height of the room 3,3 m).
- Ensure a regular routine for sufficient ventilation of classe space (Operating Manual for School).
- Ensure regular maintenance of school premises (Operating Manual for School).
- Establish clear rules of the school environment cleaning system, using of disinfection products, checking of their performances.
- Ensure satisfactory facilities for children dressing, for placement of shoes and clothing used in the external environment.
- Aviod either renovation work or painting during school operation.
- Minimize the use of water-resistant paint in the school environment.
- Replace furniture, floor coverings, and wall painting so that there is sufficient time for effective ventilation of the space - min. 2 weeks.
- Choose quality furniture and floor coverings with low emission classes of volatile organic compounds.
- Provide the school space with appropriate vegetation that regulates IAQ in schools and has no allergenic effects.
- Aviod the use of local heaters.
- Check the safety of operation of boiler, flue gas system.
- Use appropriate building materials for the construction of school facilities.

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Rehabilitation and Efficiency Solutions For Palas – Constanta Water Treatment Plant

Ion Oprea

***Abstract* – These are proposals aimed at the rehabilitation of the Palas water treatment plant, so that it can provide quality drinking water at flow rates higher than those currently treated and allow future expansion of consumption, according to the share given to the surface sources in the drinking water distribution systems from Constanta.**

In developing the proposals for the rehabilitation of the Palas Constanta water treatment plant, it was also considered the technology of water treatment for similar situations in other cities in Romania, where the proposed solution has been a success.

***Keywords* – coagulation, disinfection, filtration, flocculation, quality, technological flow, treatment plant, water.**

1. The Current Schema of the Palas – Constanta Water Treatment Llant and Related Processes

At the surface water source at Galesu is a chlorination plant which performs a precloration of raw water before being pumped into the Palas plant. From the source of surface water, water is transported to the plant by two pipes. The pipes are made of steel and PREMO and have the role of downloading their content in the mixing and distribution tank of the plant. In the case of the Palas Constanta Water Treatment Plant, the water resulted from the surface water source at Galesu is mixed, after having been treated, with the water from groundwater sources, before being pumped to consumers. The mixing tank for the raw water with reagens and of water distribution at decanters presents a malfunction in the system of mixture reagent. The mixing basin is continued by pipes that distribute water to the four decanters using the gravitational forces. The existing decanters are pulse-powered, although they do not function and have never functioned work as such. At the top of the water decanters, the water is collected through a system of pipes with holes that works like an overflow, spilling it into concrete channels that direct the decanted water into a manhole. From this manhole the water flows using the gravity force through a pipe from each decanter to the water filters station. The filtering station consists of four batteries each of 5 filters and is located in a warehouse type building. Water is coming from the decanter pipes to a set of channels in the upper part of the filters. For these channels the water is introduced into the tanks of sand filters. From the water filters, water get through pipes in the gallery below filters. In the filters room a hydrotransport-system is installed for sand to be replaced in the tanks. the 6000mc storage tank that collects the filtered water is under the gallery of pipes and valves, below the filters room (in the basement). The blowers convey the air blown towards the gallery below the filters, where the air flow rate is divided to the bottom of the filters through a set of pipes. Washing the filters is done with pressurized air produced by the blower station and the resulting water is evacuated through the existing sewerage network. The building of the chlorination station includes: chlorine storage; chlorine dosing equipment room; neutralizing substances room; protection materials deposit. The existing chlorination station is the one that disinfects water before being sent to consumers, both for

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treated surface water and for the water coming from underground sources. Pump station no. 4 (PS4) is located in a building adjacent to the filters station, in the basement.

The station takes the filtered water from the adjacent storage tank of 6000 mc and pumps it to the storage tank of 10000 mc in the station. In the pumping station PS4 there is also a compressor acting on the pneumatic valves [1].

2. Efficiency Solutions for the Processes in the Palas Constanta Water Treatment Plant

These proposals aim at the rehabilitation of Palas water treatment plant so that it can provide quality drinking water at a flow rate which can cover those currently treated and allow future expansion of consumption, according to the share given to the surface sources in the the drinking water distribution systems from Constanta.

The station will operate at full capacity during the entire rehabilitation process.

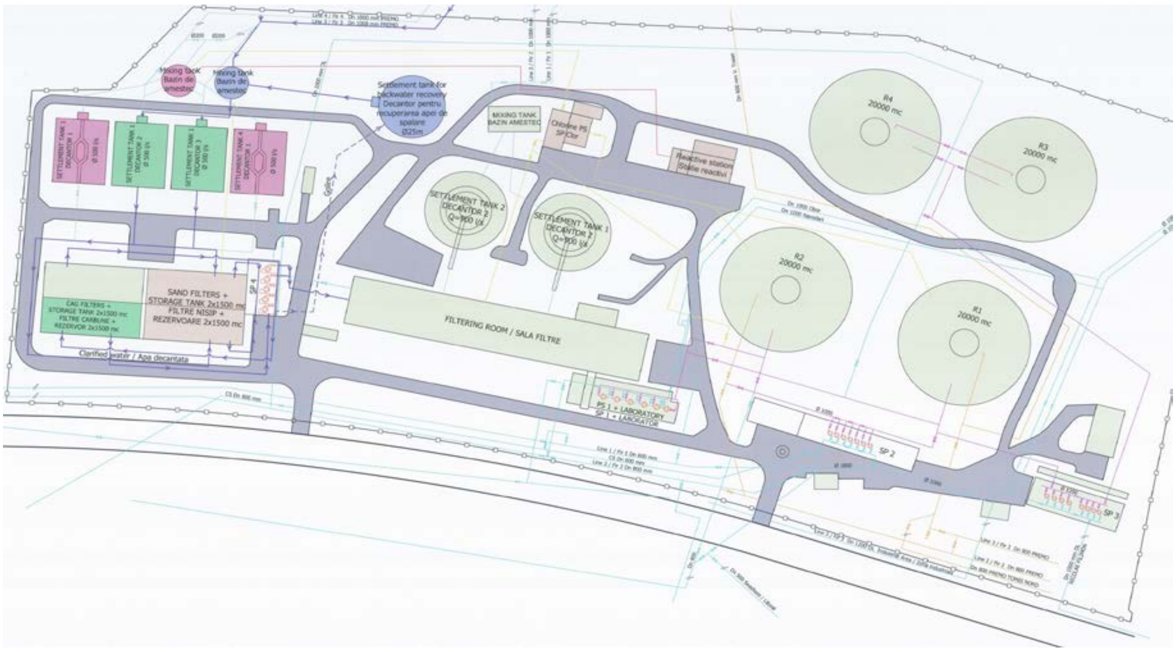

Fig. 1. Situation plan of Palas Constanta water treatment plant after rehabilitation [1]

Along the Danube Channel - Black Sea, the water flow is slowed down because of locks so that the channel works as a huge intermittent decantor, with two consequences:

- the slurry are massively reduced, with a turbidity ranging between 1.6 ÷ 5.5 NTU;
- the occurrence of a taste and odor characteristic of pond water.

These elements caused by the exploitation of the treatment plant have imposed a change in the current technological scheme of operation, namely:

a) the introduction of a sodium hypochlorite disinfection at Galesu source, (instead of chlorine gas treatment) - with on the spot production;

b) setting a new water distribution room at the entrance of Palas plant;

c) quitting the existing suspension decanters that weren’t needed anymore becuse of the massive reduction of suspensions; the decanters will be built in basins of rapid and slow reaction, by their proper hydraulic profiling; steerers will be set in these tooms and they will ensure variable reaction gradients depending on the raw water characteristics;

d) the introduction of a new automated storage and dosage of reagents to be introduced through the supply pipes before distribution room; the rehabilitation of the reagent station and the replacement of all the pipes that carry the reagents; in the new technological scheme, the permanent usage of an efficient reactive is intended, together with an adjuvant in the cold seasons and the changing two decantors into reaction basins and suspension flocculation basins, so that the water filtering should become more efficient;

e) Filters: introducing granular activated charcoal filters in the scheme, as a finishing stage, with the main objective of improving the organoleptic qualities of the water; the filter pipe gallery will be completely rehabilitated and all components will be replaced with other materials resistant to corrosion; the remaining sand filters will be fully rehabilitated; the command and control systems of the filters will be replaced, including their electrical and automatic parts, the control boards, the automatic or electrical valves, the level surveillance systems, the air ducts,strainers; the filters room will also be rehabilitated by restoring indoor and outdoor painting and plaster, waterproofing the roof and injecting its joints, replacing the longitudinal skylight on the roof, the gutters and the downspouts, by rehabilitating the heating system, replacing the metal covers and railings and by adequate partitioning of the filters room;

f) The blowers will be placed on a concrete support and the construction which currently protects them from the weather changes will be replaced by a new lightweight soundproof building;

g) the rehabilitation of the pumping station no. 4 (PS4) including soundproofing the entries, the restoration of outfoor and indoor plasters and painting restoration and the rehabilitation of the joinery; the pumping groups and hydromechanical equipment, the indoor pipes, the valves, directional control valve, and so on, will be rehabilitated; the compressor for the pneumatic valves equipment serving the filters will be replaced, including the faucets and air ducts;

h) the construction of a radial decanter for the takeover of the washing water from the filters, including a pumping system for the deposited sludge and the cleaned water;

i) the rehabilitation and supplementation of the networks on the premises, including the water transport pipes which are not made of acceptable materials and new water transport pipes to make the connection between the various elements of new technology; also the reagents pipes, the compressed air ducts and the blown air pipes will be rehabilitated; it will be installed a properly designed drainage system to take the sludge produced in the new radial decanter, in the slow and fastreaction rooms, and also the other sewage flows from administrative buildings;

j) the plant producing sodium hypochlorite, for more safety durign the exploitation, within the Palas treatment plant, with injection in all the reservoirs leading to consumers; the disinfection will treat the entire flow of the Palas treatment plant, both that from the Galesu surface source and those coming from underground sources;

k) the introduction of sensors – with transmission to the measurement equipment finally to the automation elements and the introduction of a SCADA system with a common equipment system;

l) the rehabilitation of the pathways on the premisses, which will be damaged by the rehabilitation processes – especially because of the pipes and also because of the construction processes on other objects [1].

3. Process Flow Adjustment in the Palas Eater Treatment Plant to the New Treatment Concept

In developing the rehabilitation proposals for the Palas Constanta treatment plant, it was considered the technology of water treatment for similar situations in other cities in Romania, where the suggested solution has been successfully used.

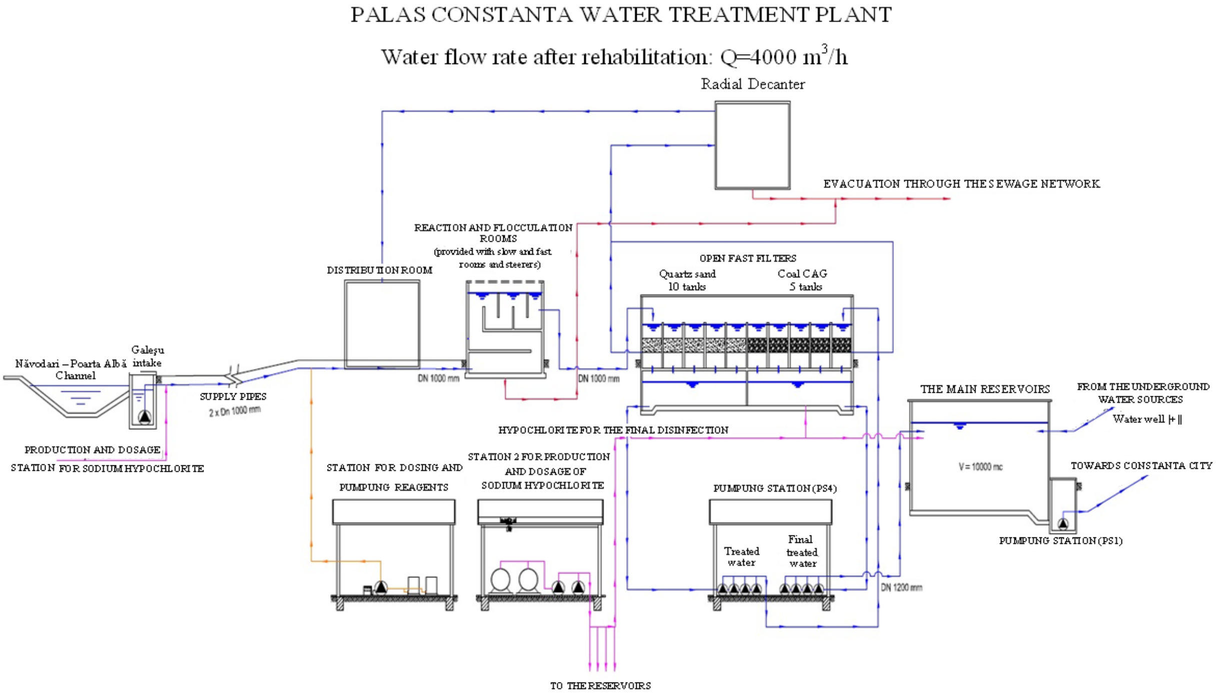

Fig. 2. Process Flow of Palas Constanta treatment plant after rehabilitation [1]

a) The prechlorination station from Galesu source

A disinfection equipment with sodium hypochlorite will be introduced at the Galesu surface souce, with production on the spot, for placing the substance in the discharge piping toward the Palas treatment plant.

The disinfection equipment will be installed in place of the existing chlorination station. The minimum capacity of the equipment will be of 250 kg Cl /day.

Galesu source chlorination system will serve the entire flow coming from this source.

The prechlorination from the Galesu source will consist of:

- electrolyzer (sodium hypochlorite production, brine reservoir, storage reservoir, dosage, dispensing pump, online monitoring system free chlorine/pH);
- dispensing pumps that will be provided in the system (1+1) - one set for each injection location and connecting pipes;
- free residual chlorine sensors to be mounted in the Galesu source on the supplying pipes towards the Palas plant;
- the teletransmission of data to the SCADA control room from Palas plant.

For reasons of safety and ease of use during exploitation, the sodium hypochlorite solution produced system should be 0.7 % ÷ 1%.

b) The distribution room

It is proposed that the existing mixing tank to be preserved and a distribution room to be built. This should be designed for a total flow of 4000 m³/ h. It will be located at a hight allowing alimentation using the gravity force continuing the process flow. Through two new PEHD pipes the reagent will be inserted in the two pipes of water transportation, upstream from the distribution room.

c) Reaction tanks

The settling stage should be skipped, given the reduced amount of suspensions of raw water. These suspensions may be retained directly in the sand filters without affecting their functioning, respectively without reducing the time between two washes, rated at a wash a day. The coagulation-flocculation stage will take place in the presence of a coagulant and a polyelectrolyte. Tests have shown that the aluminum sulfate behaves very well also as a coagulant, but any other efficient reagents can be used. The addition of polyelectrolyte as coagulation adjuvant will take place only when necessary (at low temperatures of raw water). We recommend an anionic polymer. The coagulant and polyelectrolyte doses will be determined by laboratory tests (jar - test) whenever the quality of raw water changes. The process will take place in the slow and fact reaction rooms, properly equipped. The settling stage is not recommended, because the raw water is generally very clear, and the type of decantors existent in Palas plant don't correspond to the quality of the source water and it can't form in the suspension layer, thus the stage beeing inefficient.The existing decanters will be turned into slow and fast reaction rooms, through proper design and equipping and the use of mechanical steerers, which could ensure variable reaction gradients, according to the quality of the raw water. The coagulation-flocculation stage is absolutely necessary because it is necessary to significantly reduce the natural organic matter and thus, the risk of THM formation .

The proposed washing system includes all the necessary components for its proper functioning, including pipelines, the pump, a control box, the power supply and any other item necessary for its proper functioning. The pumps will be installed where the compartments of the reaction tanks are placed. The pumps will be providedwith lifting mechanisms and access points . The fluids resulting from washing and collected at the base of the filters will be transported in the sewage station of the plant.

The volume of tank : Vu = 2 x 25.00 x 8.40 x 5.50 = 2310 m^3.

d) Reagents treatment plant

A coagulation reagent type aluminum sulphate will be used and in the case of high turbidity, the basic aluminium polychloride provided as a solution, which has proven effective in treatment facilities for water supply of other large towns that also take their water from surface sources.

The dosage will be done automatically according to the turbidity and the laboratory analysis. Using the proposed reagent will be permanent for a reduced dose prescribed by the plant laboratory. The storage and dosing facilities for the reagents will also be provided in a way as to be flexible - in order to allow the use of alternative substances besides aluminum sulfate, namely basic aluminum polychloride.

Two sets of dispensing pumps will be provided in the system (1+1) , one set for each supplying pipe and for each type of dispensed substance (reagent and polyelectrolyte) follows:

- 2 x (1+1) reagent dispensing pumps;
- 2 x (1+1) polyelectrolyte dispensing pumps.

e) The filtering station

The speed of filtration for the flow rate of 4000 m^3/h is 12.90 m/h (below the usual speed), and the time span to get through the filtered layer is 4.65 minutes. Washing the filters with granular activated carbon (G.A.C.) was provided with filtered water at a flow rate of 12 l/s and m^2.

The insertion in the technological scheme of granular activated carbon filters involves a double pumping of the filtered water in the sand filters and stored in the two compartments of the storage tank of 6000 m^3 placed below these filters. These two compartments of the reservoir will supply the washing pumps of both (sand and G.A.C.), and the other two compartments of the tank of 6000 m^3 placed below the active carbon filters will be used for the contact with the sodium hypochloryte present in the final disinfection stage.

Of the 20 existing filters of 62 m^2 each, it is recommended the rehabilitation of the hydraulic, mechanical, electrical and control facilities (including theconsoles) for 10 filters with quartz sand, designed for a filtering speed of 6.45 m/h and also for 5 G.A.C. filters, while the other 5 are going to be preserved for a later stage.

Their capacity will be: Q = 31 m^2 x 2 x 10 items x 6.45 m/h = 4000 m^3/h.

Filters will work at a constant level with variable flow rate.

It has been arranged a full replacement of the filter layer by using a type of quartz sand with measured grain-size of 1÷1.5 mm, with the thickness of the sand filter layer of 1.10 m and with 0.8 m water coverage.

To improve the organoleptic qualities of water (taste and odor), it is necessary to provide an additional filtering of the water through a bed of granular activated carbon. This filtering will be done in independent filtering tanks with granular activated carbon and they will be placed on the process flow after the sand filters.

Five C.A.G. filters have been provided, with a total area of 5 x 2 x 31 m^2 = 310 m^2 with the height of the carbon filter layer of 1.0 m and a water layer of 0.80 m. At the washing stage, the total height between draining bottom and the collecting troughs for the wash water is 2.32 m, allowing a growth of 0.52 m.

It will replaced the hydromechanical, electrical, control and of automation facilities for filters both with sand and with C.A.G.

f) The blower station

Two new blowerswill be provided in the system 1+1 with Q = 3350 m^3/h, P=0.7 bar.

All data about the blowers functioning (on- off) will be transmitted to the SCADA control room by the pumping station PS4. Blowers can be operated also from the SCADA control room.

The lightweight building currently covering the filters will be removed and replaced with a waterproof and soundproof roof and with a soundproof perimeter wall, so that the noise level to fall significantly.

g) The pumping station no. 4 (PS4)

The pumping station PS4 adjacent to the filters will be modified as it follows:

- The pumping P4 and P5 are kept (GRUNDFOS) and prepared to supply water for Năvodari town; the 3 existing electropumps for the water in the reservoirs will be replaced by 5 high efficiency units, each having the following characteristics: Q= 2000 m^3 h, H=10m CA;
- The worn-out washing pumps will be replaced by 4 units (2+1+1 R) with high efficiency levels (at least 70%) to wash the sand and activated carbon filters, each having the following characteristics: Q = 900 m^3 /h h=10 m CA;
- The existing compressor will be replaced by a new one with a power of 2.2 kW to supply compressed air into the pneumatic valves;
- The mechanical and hydraulic facilities, the valves, the directional control valves and full-length connection will be rehabilitated.

h) The radial decanter

A new radial decanter will be built. It will take the washing water from the sand and activated carbon filters.

The decanter will have the following minimum dimensions:

- Depth 1.8 m;
- Diameter 25 m.

At the base of the radial decanter a base will be built. Here, the slidge will be collected and then pumped to the sewage network by submersible pumps.

Also, pumps will be installed for the settled water, which will be pumped toward the distribution room.

i) Networks on the premises

All the transport lines between the items will be replaced. These pipes will be made of the following materials: PAFSIN, PPR, PEHD si PVC-KG (the sewage pipes). There have been provided: water trasnport pipelines, air rtansport pipelines, reagents transport pipelines, sodium hypochlorite transport pipelines and sewage pipes.

All pipes will have valves and associated manholes.

The water transport pipes between the items will be sized for the flow rate of 4000 m^3/h.

j) The station prodicing sodium hypochloryte within the Palas plant.

The station producing sodium hypochloryte from the Palas plant will function in the old building of the chlorination station and it will treat the entire water flow, both the flow from the surface and underground sources. The minimum capacity of the equipment will be of 100 kg Cl/day.

The station producing sodium hypochloryte from the Palas plant will consist of:

- electrolyzer (sodium hypochloryte production, brine reservoir, storage reservoir, dispensing pump and online monitoring system free chlorine/PH);
- dispensing pumps that will be provided in the system (1+1) - one set for each injection location and connection pipes;
- free residual chlorine sensors will be installed on pipes leading out to consumers;

For reasons of safety and ease of use during exploitation, the sodium hypochlorite solution produced by the system should be of 0.7 % ÷ 1%.

k) SCADA System

SCADA (Supervisory Control and Data Acquisition) is a technology that allows you to receive the information by an operator located away from the equipment with the ability to transmit a set of instructions to them.

The SCADA system allows both monitoring of the equipment and operating on it.

In the case of the Palas plant, the following will be provided: the control room for the Galesu source (which will collect all data on the prechloration station, sensors and flow rate sensors and it will transmit them to the SCADA control room for the respective treatment line), the control room of the treatment line (which will collect the data from the Galesu control room and those from the items which are going to be rehabilitated and built on the treatment line and they will be transmitted to the general control room of the Palas plant) and the general control room of the Palas plant (which will collect all the data transmitted by the control room of the treatment line (including the data from the Galesu source) and all the data from other items belonging to the Palas plant, through the sensors or/and the measurement equipment already installed.

The SCADA system will continuously measure and count the technological parameters of the station input and output, and also the electrical parameters and power consumption. The SCADA system will be designed in such a way that the entire system, in both the general and the treatment line control rooms to work effectively and include any necessary component for its proper functioning.

Both the general and the treatment line control rooms will be able to function independently, both for data collection and control.

All pumping, disinfection and filtration stations will be equipped with modern automatic equipment so that they can transmit the data at great distance to the general control room of the Palas plant. They will function manually or automatically, depending on various parameters. The automatic control will be done from the local control panel or from the control room.

l) The alleys on the premisses

All the existing paths on the Palas premises will be rehabilitated [1].

4. CONCLUSIONS

All things considered, it is recommended the following technological scheme for water treatment at the Palas Constanta Water Treatment Plant:

- **Pre-oxidation with chlorine** (1.5 - 2 mg /l) at Galesu intake point, providing a contact duration of at least 1 hour to inactivate the larvae of Dreissena Polymorphic before the water gets to the treatment plant; currently the chlorine injection is performed in the raw water pump discharge outlet at Galesu, which loads the headrace; the current technical solution can be kept, considering that the headrace works as a contact basin; the contact duration provided by the headrace is of 1.5 - 2.0 hours, enough for the complete inactivation of larvae or adult molluscs of *Dreissena Polymorpha* ;
- **Coagulation - flocculation** in the presence of a coagulant and a polyectrolyte. Tests have shown that the aluminum sulfate behaves very well also as a coagulant, but any other efficient reagents can be used. The addition of polyelectrolyte as coagulation adjuvant will take place only when necessary (at low temperatures

of raw water). The process will take place in the slow and fact reaction rooms, properly equipped. The settling stage is not recommended because the raw water is generally clear, and the type of decanter existing in the Palas treatment plant can’t form a suspension layer because the quality of the raw water doesn’t correspond. It is recommended that two of the existing decanters to be turned into reaction tanks (rooms) of fast and slow reaction through appropriate designed items (replacing the existing indoor facilities- pipes and distribution channels- by lightweight partition walls, forming corridors and water circulation baffles on three levels, which should result in decreasing speeds corresponding to the reaction times, in order to create and develop flocs). The coagulation- flocculation stage is absolutely necessary also because the water becomes biostable by reducing the total organic carbon content and the period in which the water keeps its qualities increases. Without coagulation- flocculation, the suspensions existing in the water can not be retained, basically going through the rapid sand filters;

- **Direct filtration** - rapid sand filters. It is recommended the technological rehabilitation and that of the corresponding structure for the rapid sand filters station within Palas treatment plant and providing it with modern equipment in terms of automation and process control. It should be noted that if polyelectrolyte is to be used as adjuvant for coagulation, it may lead to more frequent clogging of filters and their need for frequent washing. In this respect, it is recommended to use polyelectrolyte only in special situations, namely when the coagulant is inefficient (low temperatures, high turbidity, etc.), and for limited periods of time;
- **Post-oxidation with ozone -** this stage has the role to oxidise the orhanic compounds, especially those who are precursors to the formation of is to retain oxiding organic compounds from ozone; the post-oxidation with ozone will also achieve a significant improvement in water taste and smell as well as an increase in treated water bio-stability;
- **Adsorption by granular activated carbon filtration -** this stage is meant to retain oxidized ozone organic compounds; the GAC filtration stage is dependent on the presence and effectiveness of prior treatment processes. (coagulation-flocculation, rapid/fast filtration on sand and post-oxidation with ozone); the granular activated carbon will be selected to match the retention of organic compounds;
- **Disinfection with chlorine** (0.5 mg/l) to ensure the dose of marking in accordance with Law 458/2002 [4] on drinking water quality with subsequent completions and additions [2], [3].

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Urban Regeneration in Protected Areas – Solution for Sustainable Development of Cities in Romania

Diana Ţenea, Mari-Isabella Stan and Dragoş Vintilă

Abstract – **The European Union supports urban regeneration through the Structural Funds, to which participating Member States can accede the European programs in the field of urban incidence. The issue of sustainable urban development and urban regeneration requires the development of public policy by the name of urban development and urban regeneration. One of the intervention areas of urban regeneration in Romania is local culture and identity through the protection and restoration of cultural heritage preservation of national historical and architectural and local urban values. Protected areas that hold true architecture "jewels" can be harnessed in terms of cultural, historical and architectural point of view. The paper tries to answer the question: if integrated interventions in protected areas of central cities are a success or a failure for public authorities?**

***Keywords* – projects, protected areas, public administration, sustainable development, urban regeneration.**

1. INTRODUCTION

Since the ’80 documents and data are key issues of sustainable urban development and urban regeneration, but starting with 2001, at the *European Council in Goteborg*, the European Union has an integrated development strategy with the objectives of growth and competitiveness for sustainable development.

Key means set by the *Leipzig Charter on Sustainable European Cities* [4]: use an integrated approach to urban development policies and focus on disadvantaged urban areas, as well as the *"Declaration of Toledo"* [5] which aims at developing smart and sustainable urban areas, specifically integrated urban regeneration leading to implementation of Europe 2020 *„A European strategy for smart, sustainable and inclusive growth"*.

According to the *"Declaration of Toledo"* integrated urban regeneration is "*conceived as a planned process that must transcend the boundaries and approaches which have been usually the norm until now, in order to address the city as a functioning whole and its parts as components of the whole urban organism, with the objective of fully developing and balancing the complexity and diversity of social, economic and urban structures, stimulating at the same time greater environmental eco-efficiency*".

Urban regeneration promoted by the European Union designates a set of principles for action aimed to contribute to the sustainable development of cities, such as [6]:

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1) Urban regeneration means the public authority intervention in distressed urban areas, which are located in an urban setting but fail to set out in a continuous process of degradation – it could be residential areas, industrial sites or decommissioned military or historical centers.

2) Urban regeneration requires an integrated approach to all sectors of intervention for urban operations that are made for sustainable development.

3) Urban regeneration requires good local governance in the sense of transparency in making political decision, partnership between public and private, participation of the population on urban regeneration projects which will have an impact.

4) Urban regeneration needs of all local actors the ability to integrate with the local European policies in the field of urbanism. The European Union supports urban regeneration through the Structural Funds, to which participating countries can accede the European programs in the field of urban planning implications.

2. THE PUBLIC ADMINISTRATION ROLE AS URBAN ACTOR

Considering the urban dimension and future challenges faced by European cities, as well as the need for European funds absorption, after Romania joined the European Union concerns of central public administration authorities to harmonize existing legislation with European law have led to the need to develop policies named public *urban regeneration* and *sustainable urban development*.

The role of central public administration is to establish strategic documents on sustainable and integrated territorial development in the medium and long term of Romania. In this respect, was developed*"The Strategic Concept of Territorial Development Romania 2030"* in order to highlight the integrated territorial perspective, ways to exploit the national potential, to recover disparities from European countries, to boost balanced development of Romania and strengthen the role of Romania as a member state of the European Union and as an active player in the Central and Eastern Europe [7].

The concept of sustainable urban development is a complex activity that requires a set of complementary approaches and appropriate strategies incorporating aspects such as: attractiveness of cities, support for innovation, job creation, development disparities within cities, governance and funding of urban regeneration.

One way of European urban action is integrated urban regeneration project representing *"a collective project that correlates multiple aspects of urban development and different scales of intervention"*. This project is developed and redefined by a process which involves: traditional urban actors - public administration, but also the new actors - representatives of citizens, professional organizations, national and international investors. The integrated project is the result of negotiations among all participants urban actors [6].

From the definition of urban regeneration that is *"bringing to life the urban areas with cooperative effort of municipalities, owners and other involved actors in order to improve living conditions, increase environmental quality and social climate and strengthen the local economy"* [8] it can be said that the urban project implies a new mode of urban management of the city, involving all urban actors that lead to sustainable urban development.

Urban actors involved in urban regeneration projects are defined in various ways: *initiators*, *customers*, *users*, *funder*s, *target groups*, *operators*, *partners* and so on, as well as how an urban actor involved or affected by the changes made to a project.

The partnership is a solution adopted by public administration to resolve urban problems therefore communication and negotiation is needed between urban actors. Rules of partnership between actors within an integrated urban regeneration project are [6]:

- The mutual recognition of the need to work in a team - public administration, professionals, citizens;
- Partnership is an arena where urban actors facing views on city development, discuss and decide priorities for action, meet and share resources;
- A partnership is a long process that requires patience to build trusted relationships, to understand the differences between the various partners.

The role of the public sector is to create urban policy, to initiate urban regeneration practices and to ensure the continuity of long-term projects.

The role of the private sector can be as partner in elaborating and implementing the project of urban regeneration. The role of civil society is essential in that area a long time known intervention and can contribute through participation in problem definition and identity of the neighborhood.

Development of towns depends on the emergence and operation of local development coalitions that consist of private enterprise networks and representatives of local administration, between which is performed formal or informal arrangements for development [2].

3. EXAMPLES OF "GOOD PRACTICE"– URBAN REGENERATION PROJECTS IN ROMANIA

One of the intervention areas of urban regeneration in Romania is local culture and identity [6] through the protection and restoration of cultural heritage preservation of national historical and architectural and local urban values.

The legal framework on protected areas in Romania is regulated by Law no. 5/2000 and in Article 1, the protected areas are defined as *"the natural or built zones, geographically delimited and / or topographic, which comprise natural heritage values and / or cultural and are declared as such to achieve specific conservation objectives values heritage "*[13].

In the category of protected areas and historical monuments that are part of the national cultural heritage according to Law no. 422/2001 are *"real estate, construction and land situated on Romanian territory or across borders, Romanian state property, in compliance with the State in which is significant for the history, culture and national and universal civilization"* [14].

The protected area is land surfaces with the buildings thereon, where applicable, situated within or outside city limits, delineated by spatial planning plans or advised urban plans and approved according to law, which establishes rules for interventions constructed or protected natural areas.

The protection of historical monuments are areas of land with associated buildings around the historic monument delimited by urban plans, advised and approved according to law, which establishes regulations for the execution of works in the area.

Protected areas that hold true architecture "jewels" can be harnessed in terms of cultural, historical, architectural point of view. By establishing a program of urban regeneration by local public administration, the charm of historic areas is not lost and can become the pride of the city's historic center.

In the following we present some urban regeneration projects, as examples of “good practice”, involving local public administrations from Romania, through the establishment of programs for the enhancement of the built heritage and its rehabilitation, as follows:

1. *Rehabilitation of the historic center East Route, Southern route, North route. Vauban Fortification - Alba Iulia, access roads, lighting and street furniture specific* [9]. The project provides cultural modernization of the infrastructure - related historical Citadel from Alba Iulia, and the restoration, protection, heritage tourism potential of Vauban Citadel.

Fig. 1. Alba Iulia - A large urban garden movement

Fig. 1. Alba Iulia - A large urban garden movement (continued)

2. *Integrated plan for revitalization of the protected area of Arad 2012* [10]. It proposes the principal measures and projects to revitalize the historic area, with items of correlation for the provision of structural funds financing in the next horizon, 2014-2020.

Fig. 2. Arad - Integrated plan for revitalization of the protected area

3. *Zonal Urban Plan for the historic center of Sighisoara - Sit inscribed in the World Heritage List* [11]. This general objective is to facilitate a process of integrated urban development by maintaining the historic character and identity of the historic center - a center that has kept most of unspoiled urban spatial organization and construction quality composing it - and by proposals for conversion or adaptation of buildings and open spaces that require such sustainably operations.

Fig. 3. Historic Centre of Sighisoara

4. *Concept Integrated Rehabilitation Measures for Economic Revitalization Prudence and Historical Districts in Timisoara* [12]. This document elaborated by the Timisoara City Hall and the German Society for Technical Cooperation (GTZ) unifies key areas for action interdisciplinary urban regeneration in a unitary concept, who is considering the prospects for achieving short, medium and long term.

Preservation of historic buildings and architectural cultural heritage concomitantly, and improving living conditions and housing, is a central challenge for the Timisoara City Hall.

Fig. 4. The historic center of Timisoara

The arrangements for local public administration intervention in this area are technical, namely: ranking stages buildings in protected areas (historical monuments); identifying investment objectives (buildings and infrastructure, construction); systematizing data on technical fiches, prioritizing investments and proposals for projects with identifying sources of financing.

On the other part, failure of local public administration act could lead to: degradation of protected areas and urban identity loss; decreased quality of life of protected areas, including the possible diseases or accidents; leaving / abandoning buildings in protected areas especially buildings owned by several owners (condominiums) and transforming the historical areas in unhealthy and dangerous social areas.

From the examples of "good practice" presented it can be observed that involvement and empowerment of local authorities with the establishment obligations and responsibilities for the quality of life of the community members through culture, total ensuring transparency of programs and projects aimed at the restoration and enhancement of cultural heritage are necessary for the success of interventions in protected areas in the context of integrated urban development.

4. CONCLUSIONS

Starting from the question: if the interventions integrated into protected central areas of cities are a success or a failure for public authorities can draw the following conclusions:

- A local public administration may define the limits of its competence policy with exact addressability;
- Interventions must be founded on solid project based over technical expertise;
- The absence of intervention of a local public administration may result in damage the protected area and its transformation into a serious social problem due to the impossibility of residents to rehabilitate buildings with their own financial resources;
- Continuous degradation lead to decreased the quality of life of the protected area and leaving those areas as being unattractive;
- Prices buildings in close proximity decrease due to "contamination", lowering businesses and entrepreneurs retire to safer areas;

- The charm of historic areas is lost and, from the pride of the city, the historic center may become the biggest problem both from socially and in terms of local public administration capacity to institute a program of urban regeneration;
- Integrated urban regeneration projects attract serious solutions, funds, investors and the general interest to learn and apply the experience already gained by others.

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SECTION V

WATER SUPPLY
SEWAGE

Aspects of water deferrization and demanganisation

Iancu Paulina, Tudor Vlad-Cristian, Dracea Dragos

***Abstract* – Groundwater has an important role for drinking water provide. In many cases extracted water is loaded with high concentrations of iron and manganese. These substances have exceeded concentrations also in surface water sources, requiring special processes for treatment. This paper presents the treatment of drinking water from water sources with high loads of iron and manganese.**

***Keywords* – deferrization, precipitation, water treatment**

1. Introduction

Recent studies have determined that most of the major rivers in the country have iron and manganese content above permissible limits. Water with high content of iron and manganese are found in the deep aquifers also. These waters to be used as drinking water require special processes to perform water deferrization-demanganisation.

Iron and manganese are found in underground water sources in molecular dispersions which represent discrete and stable systems formed by simple molecules in thermodynamic equilibrium, represented by solutions of divalent compounds. Dispersion particles below 1 µm pass through filters and cannot be withheld from the waters. This is due to lack of oxygen, the present of free carbon dioxide, hydrogen sulfide, pH <6.8 and an important role is the low level of water mineralization.

2. The mechanisms of deferrization and demanganisation phenomenon

Physico-chemical properties of water containing iron and manganese

The key factors that condition the oxidation-reduction processes, surface phenomena (adsorption of iron and manganese in molecular and colloidal dispersions on sand grains) electrokinetic phenomena, coagulation processes involved in the deferrization including temperature, pH, concentration of Fe, Mn, CO_2, O_2, water mineralization, organic content, the number and nature of iron bacteria, electric potential for oxidation and electrokinetic potential of dispersions.[2]

Iron and manganese compounds can be found in dispersion form: molecular systems composed of discrete stable particles <1mµ that do not filed, colloidal systems composed of heterogeneous biphasic with size 1 - 100 mµ which are retained in ultrafiltered, simply with size > 100 mµ which precipitate and are retained in filters.

Molecular dispersions are the compounds of divalent Fe and Mn, and the resulting colloids and suspensions is the result of passing them to a higher state by oxidation and coagulation.

Physico-chemical processes within deferrization and demanganisation

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In water deferrization and demanganisation steps occur oxidation processes, filtration and coagulation processes in special cases.

Oxidation process is intended to correct the overall balance of iron and manganese compounds from water so Fe and Mn can be retained by filtration. Process of oxidation is carried out by introducing a certain amount of oxygen into water and remove free CO_2. By aeration/oxidation process and free CO_2 removed, the pH is corrected and the electric potential for oxidation-reduction reaches Eh = 0.10 - 0.14 V.

The coagulation process is intended to correct the dispersion electrokinetic potential so that colloidal particles can be retained by decantation.[3]

The filter is designed to retain Fe and Mn keeping values of concentrations in treated water limits.

Concentrations of iron and manganese compounds allowed in drinking water:
According to the norms and standards of drinking water, acceptable concentration of iron and manganese in drinking water are Fe < 0.2 mg / l and Mn < 0.05 mg / l.

3. RESULTS AND SIGNIFICANCES

A. Study Case 1: Treatment plant with water from underground sources

The station has a water flow Q_{med} = 460 m³ / h provide from 27 wells with depths between 30-45 m average concentration of iron in the raw water is 2 mg / l Fe. **Table 1**

Water from the wells is pumped to a header pipe where is injected chlorine for pre-chlorination and soda to increase the pH for supporting deferrization process. Deferrization process begins in the oxidation tank where raw water with dissolved iron will oxidize in the presence of oxygen and form iron hydroxide. The amount of air required for oxidation and precipitation of iron will be maintained in the oxidation tank about 5 mgO_2 / l.

Value of 5 mg/l residual oxygen is generally accepted. Oxidation tank is fitted with diffusers that uniformly distribute air from two blowers that provide an average flow of 71.45 m3 air / h.

For the oxidation of iron by aeration pH = 7.5 ÷ 8.0. In this case pH = 7.2 and is corrected with NaOH introduced before oxidation tank maintaining pH = 8.

Theoretical is necessar 0.1432 mg/l of oxygen per 1 mg iron/l.[4]

Maintaining a proper amounts of oxygen is necessary for various reasons, provide a "buffer" of oxygen to react to sudden increases iron, the resulting water tastes better, air needed to maintain oxygen tank mixing facilitates iron to react quickly and efficiently with oxygen.

The following reaction describes the oxidation of iron in the presence of oxygen:

$$4\,Fe(HCO_3)_2 + O_2 + 2H_2O \rightarrow 4\,Fe(OH)_3 + 8CO_2$$

At the end of the oxidation processes of iron, water is loaded with iron hydroxide particles, colloids and other substances and suspensions. Removing water loading is performed in coagulation and flocculation step.

Table 1: Water quality at source

Characteristics	Water from source	CMA*	CMA
pH	7.4	6.5-9.5	6.5-8
Turbidity (NTU)	2.75	≤5	< 1
Conductivity (μS/cm)	1474	2500	<2500
Total hardness (º d)	18.48	> 5	> 5
Ammonia (mg/l)	0.41	0.5	< 0.3
Iron (mg/l)	1.5-2.3	0.20	< 0.1
Color (units of Pt/co)	10	< 25	< 25

*CMA = maximum concentration permitted under law 458/2002
CMA = maximum admissible concentration water plant require

Iron oxidation is not instantaneous. For this reason the station have specific reaction times follow to such a process. Sizing of reaction tanks is optimal, oxidation tank with a retention volume of 125 m^3, guarantees a reaction time of 17 minutes at maximum output. Mixing tanks are sized to a total volume of 14 m^3, which guarantees a minimum of 2 minutes rapid reaction. Flocculation tanks are sized to a volume of 50 m^3, guaranteeing the slow reaction time of 8 minutes.

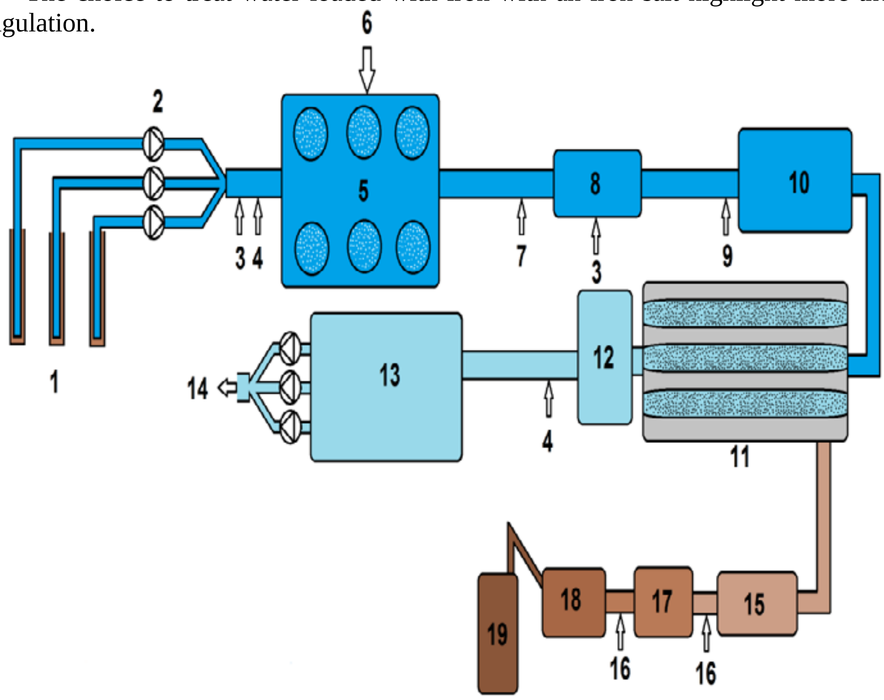
Once oxidized, iron from the aeration chamber should be removed from the water. Processes that continue deferrization are coagulation and flocculation. Coagulation occurs in mixing tanks where $FeCl_3$ (ferric chloride) is introduced to produce the phenomenon of coagulation and NaOH (soda) for pH correction.

The next stage is represented by flocculation tanks, where an anionic polyelectrolyte is dosed to perform in slow mixing, flocculation of particles produced in previous processes. **Fig 1**

Coagulation is a very important step in the process of drinking water treatment, by chemical reaction of destabilization, colloids and matter in suspension suspended solids in water, forming settleable flakes or microflakes easily removed by filtration.[4]

To achieve a maximum efficiency a laboratory tests is needed to determine the optimal dose of coagulant.

The choice to treat water loaded with iron with an iron salt highlight more the phenomenon of chemical coagulation.



1. Wells drilled
2. Pumping station
3. NaOH dosage
4. Chlorine dosage
5. Oxidation tank
6. Air dosage
7. $FeCl_3$ dosage
8. Mixing tank
9. Anionic polymer dosage
10. Flocculation tank
11. Filters
12. Filtered water storage
13. Treated water storage
14. Treated water output
15. Float recovery storage
16. Cationic polymer dosage
17. Sludge thickener
18. Sludge dewatering
19. Dried sludge storage

Fig. 1 Technological scheme of groundwater treatment plant

To establish the optimal dosage is important to take in consideration pH and water loading. Before the coagulation step is provided pH = 8 and a water charge with iron hydroxide suspensions and colloids.

From laboratory tests showed that the coagulation/flocculation took place suitable for a certain amount of $FeCl_3$.

The last step of this process is the water treatment filtration. This is done in three sand filters, which provide water to the finish like removing agglomerates formed in the chemical step.

Filtered water is collected in deposits where final disinfection with chlorine occurs.

The station is provided also with a line for sludge resulting from the treatment process. Loaded sludge is thickened and dehydrated in special installations where cationic polyelectrolyte is dosed, with role of bringing dry matter at values specified in standards.[4]

Laboratory tests were made jar-test type, in 1 liter glasses, simulating the processes of the plant, respecting the response time of each stage. (quick mixing 2 minutes, slow mixing 8 minutes and decanting 12 minutes). **Table 2**

Table 2: Raw water characteristics Jar-test characteristics

Objective:	Treatment plant		**Time**	**RPM**
Raw water:	Deepwater drilling	Quick mixing	2 min	80
Turbidity:	2.75 NTU	Slow mixing	8 min	40
Iron:	2 mg/l	Decanting	12 min	-
pH:	8			
Color:	10 units Pt/Co			

Table 3: Laboratory Jar-test results

Probe nr.	MU	1	2	3	4	5
$FeCl_3$ dosage	µl/l	2	4	6	8	10
Polymer dosage	ml/l	0.7	0.7	0.7	0.7	0.7
NaOH dosage	µl/l	1	1.5	2	2.5	3
pH		7.21	7.18	7.15	7.13	7.09
Turbidity	NTU	1.2	0.9	0.7	0.82	0.95
Fe	mg/l	0.26	0.15	0.08	0.06	0.05
Color	units Pt/Co	11	14	18	24	27

Considering the data obtained in the laboratory, it was determined an optimum coagulant dose of 6 µl/l, a flocculant dose of 0.7 ml / l and pH correction with soda at a dose of 2 µl/l, resulting in a concentration of Fe in treated water <0.1 mg / l. **Table 3**

B. Study Case 2: Treatment plant with water from accumulation lake

Station treat a flow of 1000 m3/h. Raw water is taken from accumulation lake, pumped to the raw water storage tank of which goes to the decanters. In the connecting pipe is inserted chlorine dioxide and polyaluminum chloride as coagulant. At the station chlorine dioxide is produced from sodium chlorite and hydrochloric acid, it is held in a special facility in preparation of chlorine dioxide unit.

The role of chlorine dioxide is for pre-disinfection water, but it also produces light effects on water quality, the first visible effect is that the water color due to its higher content of iron becomes reddish yellow. Chlorine dioxide produce a small change in water pH.

Dosage of coagulant has an important role in this system, is a liquid reactive acid based on aluminum salt, made up of chains of Al, OH and chlorine, and having a degree of polymerization reaction.

Table 4: Water quality at source

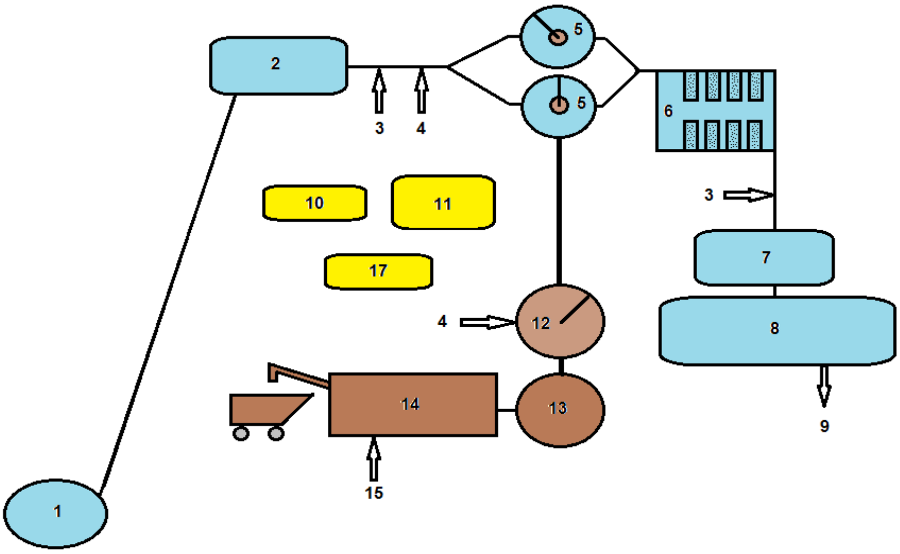
Characteristics	Water from source	CMA*	CMA
pH	7.4	6.5-9.5	6.5-8
Turbidity (NTU)	7.5	≤5	<1
Iron (mg/l)	0.30-0.90	0.10	< 0.1
Color (units of Pt/co)	12	< 25	< 10
Aluminum (mg/l)	0.02	0.2	< 0.2

*CMA = maximum concentration permitted under law 458/2002

CMA = maximum admissible concentration water plant require

Reaction times of the plant are large and designed so water to cross all the plant in approximately 10 hours, this is an advantage for physico-chemical reactions of coagulation and filtration.

Water enters in decanter first through reaction cone where is maintain a optimum amount of sludge with coagulant content that help to realize an complete chemical reaction. After cone reaction, decantation occurs followed by pre-filters in lamellar modules, and the water is directed to filters. If the water is chemically treated and decanter works well, the water reaches the filters close to 1 NTU turbidity, even below this value. **Fig.2**



1. Accumulation lake
2. Raw water tank
3. Chlorine dioxide dosage
4. Polyaluminum dosage
5. Decanters
6. Filters
7. Treated water reaction tank
8. Treated water storage
9. Treated water output
10. Chlorine dioxide preparation unit
11. Polyaluminum dosing unit
12. Float recovery storage
13. Sludge thickener
14. Sludge dewatering
15. Polymer dosage
16. Dried sludge storage
17. SCADA unit.

Fig. 2 Technological scheme of surface water treatment plant

The 8 sand filters are designed to retain existing small impurities and maintaining filtered water turbidity below 1 NTU.

After filtration the water is directed in the treated water reaction tank for final disinfection, the treated water are going to be stored in treated water storage whence water is directed to the final consumers.

The process to eliminate the iron from water starts with Dosage of chlorine dioxide for its oxidation.

The reagent used is polyaluminum chloride. Al (OH) x (Cl) (3-x)

Poly Aluminum is effective as a coagulant for water treatment and drinking water treatment plants. It is based on multivalent aluminum, allowing the use of small quantities, but with a high efficiency. This leads to the use of lower doses and therefore reducing the volume of sludge and chemical requirements for pH adjustment. Also enhances the removal of suspended solids and phosphorus compared to traditional coagulants.

OH ions existing in coagulant precipitate iron from water and forms iron hydroxide that is found in micro-flocks and flocks coagulant reacted. [1]

For such a phenomenon is recommended high basicity coagulant, with many chemical chains of OH to precipitate iron from the water. A check of the efficiency of coagulant can be seen in the treated water reaction tank after the final disinfection with chlorine dioxide if color factor is not exceeded.

To determine the optimal dosage a laboratory test were carried out. **Table 6**

Table 5: Raw water characteristics Jar-test characteristic

Objective:	Treatment plant		Time	RPM
Raw water:	Accumulation lake	Quick mixing	5 min	80
Turbidity:	6.5 NTU	Slow mixing	15 min	40
Iron:	0.6 mg/l	Decanting	30 min	-
pH:	7.4			
Color	12 units Pt/Co			

Table 6: Laboratory Jar-test results

Probe nr.	MU	1	2	3	4	5
Polyaluminum chloride	µl/l	4	6	8	10	12
pH		7.24	7.21	7.19	7.15	7.12
Turbidity	NTU	1.8	1.3	0.72	0.68	0.61
Fe	mg/l	0.32	0.16	0.07	0.06	0.05
Al	mg/l	0.23	0.19	0.12	0.18	0.21
Color	units Pt/Co	15	13	8	7	12

According to data obtained in the laboratory an optimal dose of coagulant has been established at 8 µl/l with a iron reduction to 0.07 mg/l. **Table 6**

4. Conclusions

The problem of raw water loaded in iron and manganese is overall in the country, and therefore must find optimal solutions according to the characteristics of the area, the space allocated to a water plant, other parameters in water composition, economic criteria, etc.

The two examples chosen have different ways of treating water loaded with iron, both with good yields.

We can distinguish two effective ways and purpose coagulation reagent:

As regards treatment plant supplied from underground wells using ferric chloride which only serves to coagulate suspended matter forming micro-flocks further removed in the process, chosen variant is interesting due to the elimination of iron from raw water with a iron salt.

In the latter case the water plant is supplied from a surface lake and where is used coagulant based on aluminum, polyaluminum chloride, which due its composition, rich in hydroxide ions actually help to precipitate iron and formation iron hydroxide that also is coagulated to form micro-flocks and settle flocks.

In both cases was successful to eliminate high concentrations of iron from the raw water.

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Evaluation of structural conditions of sewer system city Holíč as a basis for the preparation of rehabilitation plan

Monika Bronišová, Dušan Rusnák

Abstract – **Topical subject in the field of sewerage is not only the construction of new sewer systems as well as their rehabilitation. These sewer systems are constructed from different materials with different length of lifetime, wear rate or hydraulic capacity. It is therefore necessary to develop a rehabilitation plan, which defines the need to restore the sewer system or object on the network and selects the most problematic place suitable for rehabilitation. All this preceded by a thorough assessment of the sewer system. This article will be devoted to a specific location Holíč, will be presented the partial results, which are preparatory documentation in elaborating the plan of rehabilitation.**

Keywords – **sewer system, plan of rehabilitation, method of rehabilitation**

1. Introduction

Into awareness to the general public in recent years gets the essence and importance of sewerage municipal for everyday life. This is a consequence of increased attention towards ensuring the protection of the environment by gradually enforcing stricter and better of government regulations. Implementation of the legislation is for sewerage operators increasingly challenging. This is due to aging operated facilities and equipment under conditions of worsening economic situation. Another reason is the tightening pollution limits permitted under the requirements of the EU, which requires constant improvement of operation of sewerage and wastewater treatment plants (WWTP).

Significant development of sewerage was recorded in the 60´s (WWTP) and 70´s sewer system (SS) of the last century, when the pace of construction of sewers achieved by Pašek [1] over 150 km sewer system annually.

A major problem of existing sewer networks is their quality execution of which are often closely related to a high proportion of infiltrated water. In the 90s began to be current building condition assessment of sewer networks, where a relatively large proportion of operating the sewer system receives gradually to limit its life, which was by Pašek [1] estimated at 60 years (generally, no material distinction).

Sewer systems and rainwater systems in cities around the world suffer from insufficient capacity, construction defects and dilapidation of the pipe. The consequences are structural damage and local flooding leading to an influx of water into the basement, traffic disturbances, surface erosion and erosion streets, pollution of local water resources.

The reason problems are common effects of aging infrastructure, urbanization and climate change.

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The current state of sewerage

According to [2] used to 31.12.2010 in SR in houses connected to public sewerage 3 281 723 inhabitants, which was 60,38% (of the total population of the Slovak Republic), of which to the public sewerage with WWTP 3 202 927 inhabitants. In 2010, the total length of sewers in Slovakia amounted to 10 750,6 km.

Obligations of owners' public sewerage

Law Nr. 394/2009 collection of law [3], which identifies the owner of public sewerage (PS) ensure its development in terms of development plan public sewerage in SR [4] and a minimum of 10 years to develop a rehabilitation plan for sewerage, together with its financial security. A detail of the contents of the rehabilitation plan clarifies Decree Ministry of the Environment 262/2010 coll. of law [5], which entered into force on 15 June 2010. This decree provides content of rehabilitation plan public sewerage and procedure for the elaborating. At present are rehabilitation plans in the stage of preparation, their implementation must begin no later than 30.06.2015.

Procedure for establishing the rehabilitation plan public sewerage based on an assessment compliance of the current state of existing objects and equipment PS, with technical and specific requirements of Slovak technical standards based on the analysis of their structural state, capacity and the impact of these objects and equipment of public sewerage on the environment.

Rehabilitation plan for public sewerage and process for their preparing

According to the Decree no. 262/2010 coll. of law [5] public sewerage develops a recovery plan includes:
(a) evaluate existing information on the status of sewers, objects and equipment of public sewerage and update them based on their research capacity, assess their condition and their impact on the environment; in the assessment should be based mainly on existing operational documentation specific public sewerage, which should include be the focus of the actual status,
(b) determining the causes of technical and specific weakness condition of objects and equipment based on the outcome of a survey of capacity, identified structural deficiencies and assessing the impact on the environment,
(c) selection of the most favourable option for the rehabilitation in term of the technical, economic and environmental.

The basic condition for the inclusion of objects and equipment in the rehabilitation plan and prioritize the needs of rehabilitation is to assess the technical condition public sewerage according these basic indicators (IN): (a) their age, (b) failure, (c) the state of capacity utilization and (d) compliance with applicable laws and the requirements specified in permits for water construction.

Objects and equipment public sewerage after assessment of their technical condition are classified in the basic classes: (a) C1 - satisfactory value IN - does not require any action under the rehabilitation, (b) C2 - average IN - do not require immediate solution and is potential consider with rehabilitation, (c) C3 - critical value IN - require implementation of measures to address the existing situation and is necessary to plan rehabilitation, (d) C4 - poor value IN - indicates that the object and device require necessarily the rehabilitation because they are threatened its basic function and represents an increased risk.

Table 1. The matrix values quality classes indicators for calculate the wear rate

Class of indicators	**Age of object**	**Failure rate**	**State of usage existing capacity**	**Compliance with legislation/permission**
Unit	**Years**	**nf [1)]/km year nf/object year**	**%**	**-**
C1	1	1	1	1
C2	2	2	2	2
C3	3	3	3	3
C4	4	4	4	4

1) "nf" is number of failures

After inclusion of objects and equipment to public sewerage base classes, according to Decree no. 262/2010 coll. of law [5] calculated the rate of wear and tear, which expresses the urgent need for rehabilitation.

Table 2. The category the wear rate objects and equipment

Category the rate of ware of property	**Description prioritization (urgency) of rehabilitation**	**The range of values ware rate of property for the given category**
WR - 1. category	satisfactory degree value wear rate of property, which does not require any action under the rehabilitation	1 - 16 but none of the indicators, in addition to age, shall not be included in C4
WR - 2. category	satisfactory value, which does not require any action under the rehabilitation (potential consider with rehabilitation)	17 - 36 but none of the indicators, in addition to age, shall not be included in C4
WR - 3. category	critical values that require the implementation of measures to address the existing status (should to plan the rehabilitation)	37 - 144
WR - 4. category	undesirable condition of existing property, which requires a priority rehabilitation, because they are threatened its basic function and represents an increased risk	145 - 256

According Turček et al. [6] and Klepsatel and Raclavský [7] can be **causes of failures** of sewerage pipes and sewers divided the following:

(a) naturally aging material, (b) infiltration water to or from sewer, (c) usage inferior building material, (d) bad quality of work, (e) external effects.

Those negative factors act on sewers usually in combination, thus multiplying their effect and accelerates deterioration their status.

Signs that indicate rehabilitation needs are:

(a) collapse, (b) flooding of land and roads (especially when it comes to foul sewer), (c) increased pollution in streams, (d) blockage, (e) high infiltration.

2. DESCRIPTION OF THE THEORETICAL METHOD AND CHOSEN AREA

Methods of rehabilitation of sewerage pipes and sewers

Term rehabilitation is defined in the STN 75 0160 [8] that any measure for renewal or improve the functional capabilities of the existing sewer system or system of sewerage pipes. Linked to it *maintenance*, which represents activity for the operation of the sewer system or sewerage pipe, which slows the progress of physical wear and extending their functional ability. Rehabilitation its extent and essence includes repair, renovation or replacement. *Repair* is to remove local structural damage sewer / pipe or object. *Renovation* called for measures to improve the functional capacity of the existing sewer system or system sewerage pipes wholly or partly keeping the original structure. *Replacement* represents replace existing sewer / pipeline install a new sewer / pipe without original structure.

The situation of the area of interest

City Holíč located in Western Slovakia region Záhorie, 88 km north of the capital city Bratislava, in the northern part of the Trnava region, 72 km northwest from the county town Trnava. Administratively, it is included in the district Skalica, from district town is 7 km to the southwest. Area cadastral area is 3482 ha. The average population density is 333 inhabitants / km^2. Status of the population to 27th November 2010 was 11 071 [9].

Brief description of the public sewerage city Holíč and its most important objects

Public sewerage started to be built in the 50's and 60's of the 20th century. Sewer system Holíč is built as a gravitational with overflows chambers with and individual rainwater sewers connected directly to the recipient, combined for waste water, industrial and rainwater.

Combined sewer is located in steep terrain, sometimes with a smaller slope. As recipients of waste water are local flows - especially stream Kyštor – WWTP [10].

3. RESULTS AND SIGNIFICANCES

Assessment of the condition of technical indicators by age

Every single object on a public sewer has a different average lifetime, see table 3. Consideration of objects according to their age is the first step in the rehabilitating planning. On the basis of data about public sewerage we can calculate the estimated average lifetime value of the property. After calculating this value, we integrated the individual objects into classes and assigned them to the appropriate values. Such an assessment we made for sewer systems as well as overflow chambers.

As can be seen in table 4 and table 5 for evaluation lifetime of sewer system had been calculated value in the class C1 for each given year of installation. That means satisfactory value object of age that does not require any measure under rehabilitation.

Evaluating the state of overflow chambers, had been calculated value in the class C2, average age of value object, which do not require immediate solution (consider potential with rehabilitation), see table 6 and table 7.

Sewer system

Table 3. The average lifetime objects of public sewerage [5]

Object of public sewerage	Average lifetime
Sewer system, sewerage connection	80 years
Gatherers	90 - 120 years
WWTP, pumping stations, overflow chambers, outfalls objects	60 years – building objects
	15 - 25 years – machines and equipment
	6 - 10 years – measurement and control

Table 4. Calculated estimated average value of lifetime sewer system

Year of installation pipe	Estimated average value of lifetime	Class	Value
1985	0,34	C1	1
2005	0,09	C1	1
2006	0,08	C1	1
2009	0,04	C1	1
2010	0,03	C1	1

Based on this calculation we subsequently identify the rate wear in evaluating the state of objects according to age. Both the calculated rating falls into category 1, satisfactory degree wear rate value of property, which does not require any action under the rehabilitation.

Table 5. Classification of objects and equipment into classes according to age sewer systems [5]

Estimated average age	Class	Characterization (description) condition of property	Value
till ≤ 0,40 values of lifetime given for object	C1	Satisfactory value age of object, that doesn't require any measure under rehabilitation	1

Overflow chambers

Table 6. Calculated estimated average value of lifetime overflow chambers

Year of installation	Estimated average value of lifetime	Class	Value
1985	0,45	C2	2

Table 7. Classification of objects and equipment into classes according to age overflow chambers [5]

Estimated average age	Class	Characterization (description) condition of property	Value
From > 0,40 till ≤ 0,70 values of lifetime given for object	C2	Average value age of object, which don't require immediate solution (potential consider with rehabilitation)	2

Assessment of the state of usage existing capacity

The most important phase is the assessment of existing capacity of most sewers, but especially the major collectors ended in overflow chambers, which was necessary to evaluate comprehensively for the whole system. This assessment was done first using by program SewaCad and to compare the results, but in particular for the assessment of balance parameters of overflow chambers will done by program MIKE URBAN too.

For this calculation was used program SewaCad. By using added information about the sewer system, we calculated the overloaded sections. As can be seen in fig. 1 in some places overloading reached value more than 500%. This is caused by rainfall in sewer and old performance of sewer, as this involves a combined sewer system. Due to lack of information about side sewer needed to enter data in the assessment of existing capacity by program SewaCad we calculated only the main gatherers.

In this case the evaluated sewer system was classified into several classes with different values. On scale from 0-100 % is necessary exactly to find out what capacity is used and then assign the class and value. In other scale is not important, because the existing capacity is overloaded and we can assign them the last class C4.The design section means that the oriented real slope is incorrect and the sewer goes uphill. The program then proposed a new section with correct slope for the evaluation the sewer system.

Table 8. Classification of objects and equipment into classes according to state usage of existing capacity [5]

State of usage existing capacity	Class	Characterization (description) condition of property	Value
Existing capacity can be used in full entirety - over 90%	C1	satisfactory state of capacity, which does not require any action under the rehabilitation	1
Existing capacity can be used in entirety from 80% to 90%	C2	average state of capacity, which does not require any action under the rehabilitation (potential consider with rehabilitation)	2
Existing capacity can be used in entirety from 60% to 80%	C3	critical state of capacity that require the implementation of measures to address the existing status (should to plan the rehabilitation)	3
< than 60% existing capacity can be used	C4	undesirable state of capacity, which requires a priority rehabilitation, because they are threatened its basic function and represents an increased risk	4

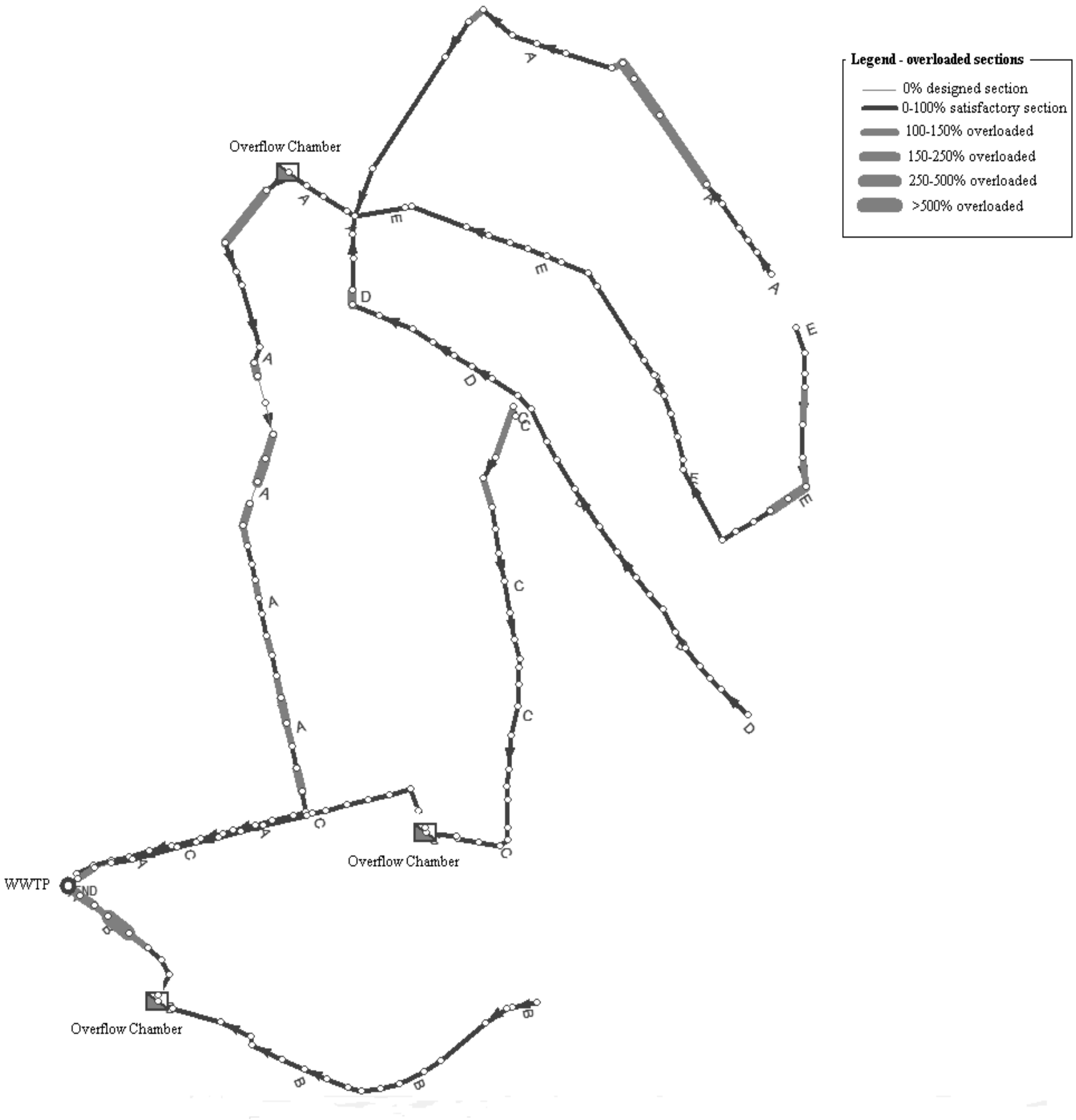

Figure 1 Assessment of the state of usage existing capacity city Holíč by using SewaCad

Assessment of sewer system in accordance with legislation/permission

The third element entering in the development of the rehabilitation plan is a set of data reflecting the impact of the overflow chambers for recipient (table 1 - Compliance with legislation / permission). It is important to evaluate flow during the rain, which begins with overflow through spillway edge overflow chamber to lightened sewer. Also the evaluation of compliance the limit values prescribed dilution ratio lightened water discharged waste water from overflow chambers to the recipient.

All WWTPs must comply the limits from EU for discharging waste water to the river after purification, including from overflow chambers. It means that all lightened water from overflow chambers should be assigned in the class C1 (satisfactory condition, fulfils all the requirements of the legislation at maximum operating conditions, which does not require any action under the rehabilitation) or the class C2 (average state of satisfying the requirements of the legislation under normal operating conditions, which does not require any action under the rehabilitation but potential consider with rehabilitation).

4. Conclusions

Assessment of existing information about the status of sewers, objects and devices of public sewerage and updating them based on a survey of their capacity, assessment their condition and their impact on the environment is the most important part in the development of a rehabilitation plan.

Last step which is necessary to do is the evaluation of failure based data obtained from the operator the public sewerage. By operator upcoming TV - monitoring of network we obtain information about the actual status of construction sewers and objects of network and we will use them for applying coding system of monitoring results according to EN 13508-2 [11]. This TV - monitoring and coding system can help us to choose the next critical section, where wasn’t according to the other technical indicators required the rehabilitation.

After calculating all the necessary parameters for the classification objects and equipment of public sewerage to base classes we evaluate the wear rate of the individual sections and objects on the network, draw up rehabilitation plan and propose the appropriate method of rehabilitation.

5. Acknowledgments

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Water balance of municipal solid waste landfills

Kristína Galbová

Abstract – **The landfill is a dynamic object, in which occurs to mineralization of wastes and their correlative reactions. Stored wastes undergo through many physical, chemical and biological changes. The rate of these processes increases with temperature and moisture up to a certain upper limit. The result of these activities is leachate and landfill gas. Predicting quantity of leachate is a critical parameter in the design of landfill construction. Created amount of leachate influences the way how to capture, collect and process the leachate.**

In resolving that issue, it´s important to design optimal water regime of landfill order to ensure the elimination of potential environmental pollution.

Keywords – **municipal solid waste landfill, leachate, water balance, modeling of landfill hydrology, leachate production.**

1. Introduction

Waste management in the Slovak Republic is based on the dominant landfill, currently landfilled more than 82% of municipal waste produced. The hydrometeorological conditions in the area of the landfill and its surroundings are of high importance as they affect the hydrogeological status of the area, leachate production and subsequently the risk of contamination [6].

Extended section of the Zohor Landfill, located 26 km northwest of the Slovakian capital, Bratislava has been in operation since 2011 and its capacity is 240.000 m^3, which represents a total area of 1.42 hectares. Enabled annual amount of stored waste is 200.000 tonnes per year and it's projected the landfill closure in the year 2018. Construction commenced in 2011, the site received about 200.000 tonnes of municipal solid waste in its first year of operation. This paper summarises the simulation of the water balance of the landfill.

2. Leachate generation

Leachate is the percolation of precipitation, surface drainage and irrigation water into the landfill including the biological and chemical reaction of waste being disposed at the landfill. Leachate formation is an indicative of increased moisture content, which is associated with enhancing biodegradation in landfills [3]. Leachate generation can be determined directly by collecting leachate production from landfill site that has leachate collection system.

Generally, water balance of landfill is used to estimate leachate formation. The water balance components include water inflow, water outflow and water store within landfill.

Water inflow such as water entering from above which mainly is precipitation, water entering in solid waste and cover materials from which moisture is inherent in materials. Water outflow such as water leaving

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from the bottom is called leachate, water consumed in the formation of landfill gas and water lost as water vapour.

3. QUANTITY OF LEACHATE

The amount of leachate generated from landfills over long time periods (e.g. years) can be predicted quite well using available water balance models (e.g. HELP [9]). The water balance components are presented in Fig. 1.

Their quantity is influenced by the intensity of rainfall, the surface runoff, evaporation and the ability to accumulate the landfill body water. They calculate leachate discharge equal to the difference between precipitation and the sum of actual evapotranspiration, runoff, and water storage within the waste body, where by the later one is determined on the concept of field capacity (no water flow until the soil or waste reaches certain water content). However, the variation of the leachate discharge rate over time is much more difficult to describe, since it requires an understanding of the water flow processes inside the landfill [5].

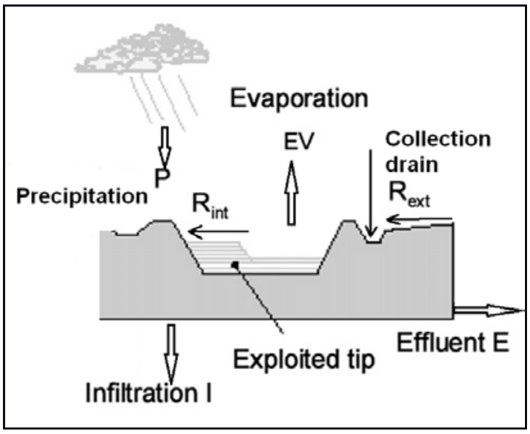

Fig. 1. Water cycle in a MSW landfill

Despite the need for obtaining information on the quantity of leachate from the landfill, they have so far not been sufficiently addressed. Process water balance for the landfill is more complex than the water balance of natural origin, because in addition to natural conditions affect the quantity water and bio-chemical processes inside a landfill [5].

Informative Annex A - STN 83 8101 Landfilling, waste leachate from landfills describes the calculation of the amount of landfill leachate. It states that quantity leachate can be determined by water balance of landfill, which is generally expressed as follows:

$$Q = Z - P - V + W \pm U + R \quad (1)$$

where: Q - is the quantity leachate collected for landfill sealing system;
Z - rainfall;
P - surface runoff;
V - evaporation;
W - moisture content of the waste deposited;
U - water consumed by or underlying the on-going march of the landfill;
R - retention capacity of landfill.

Getting quality input data is determined by several factors: the complexity of relationships in a landfill, the availability of necessary information, financial capabilities and etc. To balance modelling of existing landfills is appropriate equation to calculate the quantities of leachate (1) modified [4].

4. SIMULATION OF THE WATER BALANCES OF THE MSW LANDFILL

The site was proposed as a modern landfill which meets the standard requirements of according to the generally binding regulations. It was designed to receive a mix of non-hazardous commercial and domestic municipal solid waste.

It is situated in a warm climate and the average annual rainfall reached 702 mm. Total precipitations during the growing season are from 400 to 500 mm and in winter season from 200 to 300 mm. Monthly rainfall typically ranges from 30 - 100 mm, with distinctive occasional monsoons in June and July. The average year temperature is of 9.0 to 9.9 °C and an average sunshine duration is of 156.4 hours the solar radiation.

Estimates of leachate generation rates were simulated with varying heights of deposited waste for a certain year of operation of the landfill using the HELP model in conjunction with a simple water balance model. The assessment of leachate generation was for the 1.42 ha.

Hydrologic Evaluation of Landfill Performance Model

Determining water management of landfills need to understand leachate formation, factors influence leachate production, including model application. Hydrologic Evaluation of Landfill Performance (HELP) model is a tool to estimate of water balance for municipal solid waste landfill. Use of HELP model is recommended by the U.S. Environmental Protection Agency (US EPA) and required by most states for evaluating closure design of hazardous landfill non-hazardous waste management facilities.

Model application, the HELP model is classified as quasi-two dimensional because several one-dimensional models (percolation vertically, drainage and surface runoff horizontally) are coupled [1]. The model accepts weather data, soil and design data and uses solution techniques for water balance analysis. Generally, landfill system consists of the various combinations of vegetation, cover soils, waste cells, lateral drainage layers, low permeability barrier soils and synthetic geomembrane liners. The model facilitates rapid estimation of the amounts of runoff, evapotranspiration, drainage, and leachate collection and liner leakage.

HELP model still has limitation of application. For example, the model has limits on the arrangement of layers in the landfill profile. The physical characteristics of landfill are constant over the modelling period.

The model was applied for estimating the leachate generation in landfill and its response on rainfall variation. The HELP model version 3.07 was selected for simulation water balance of open MSW landfill. Significant inputs were collected and assessed for model procedure.

Data collection and analysis

• Weather data was recorded by Meteorological Station Kuchyňa. Seven years (2006 - 2012) daily rainfall, temperature, solar radiation, etc. were collected. The other significant data for HELP model was obtained from the previous experimental data, literature review and available data from model.

• Soil and landfill design data were followed the specific design and operation of open MSW landfill. Using the default data from model was considered.

Modelling procedure

A comprehensive review on HELP model and its application was performed. The procedure followed the HELP model user's guide Version 3 [7].The water balance of MSW landfill was modelled.

Since the model does not include the climatic data of the considered site Zohor were manually inserted into the model seven annual data on rainfall, temperature and solar radiation.

The simulation was composed profile of MSW landfills. Profile is composed of six layers, wherein the first layer is municipal waste, the amount of which varied during the seven years of operation of the landfill.

During the first year the height of the waste deposited reached 2.41 meters, in the second year, the amount of waste 4.82 m, in the third year of 7.23 meters, in the fourth year of 9.68 meters, the fifth year of 12.05 meters, in the sixth in the seventh year of 14.46 m and 16.87 m. The second to fourth layer is a drainage system and the fifth and sixth layer consists of sealing landfills.

In the simulations were used climatic data for 7-years from the meteorological station military airport Kuchyňa.

5. RESULTS

Leachate generation is not constant and it depends on the initial moisture content, decomposition of solid waste, and the influence of climate. Fig. 2 presents the relationship between rainfall and cumulative leachate generation from landfill during the year 2009. This figure presents rainy season and dry season.

Rainy season was started from June to mid-September 2009. High amount of leachate was generated which mainly based on initial moisture content of MSW, decomposition of waste and precipitation in this period. The leachate from landfill was significantly increased on July 2009 which had high intensive rainfall with long duration. The rainfall was rapidly infiltrated into landfill.

During the dry period was very less or no rainfall. Thus, in this period, the leachate was produced in small amount. The cumulative leachate generation from landfill was slowly increased.

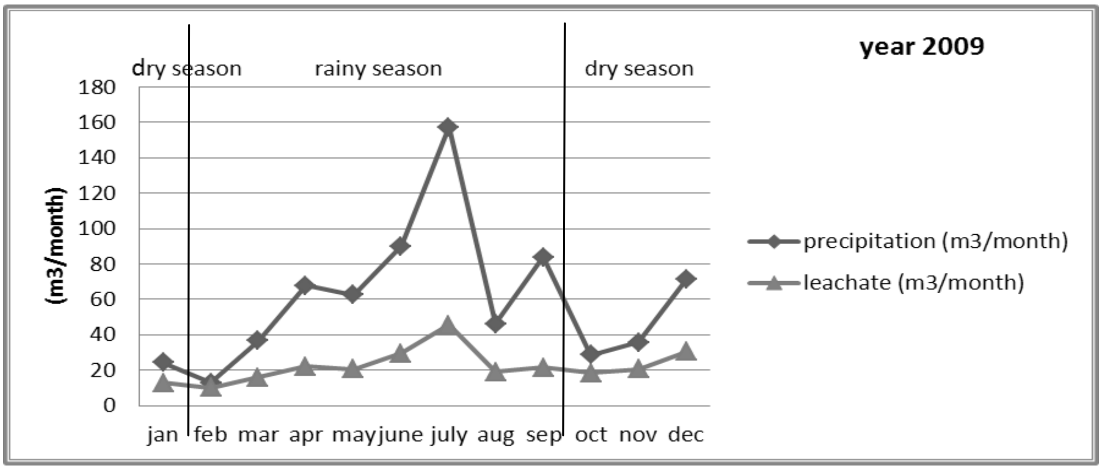

Fig. 2 Values of precipitation and leachate for the fourth year of operation of the landfill.

Water management at landfill sites is an important issue. The open landfill simulation showed that the highest cumulative leachate generation during monsoon and leachate ceased out during the dry period due to heavy loss of moisture by evaporation.

After input all data, HELP model simulated the results such as daily, monthly and annual output of precipitation, runoff, evapotranspiration, lateral drainage collected and percolation/leakage through layer.

Fig. 2 shows the output of the simulation based on the monthly average data of seven years (2006-2012). The model output illustrated that the cumulative leachate generation and the evaporation were 66 % of annual rainfall, respectively.

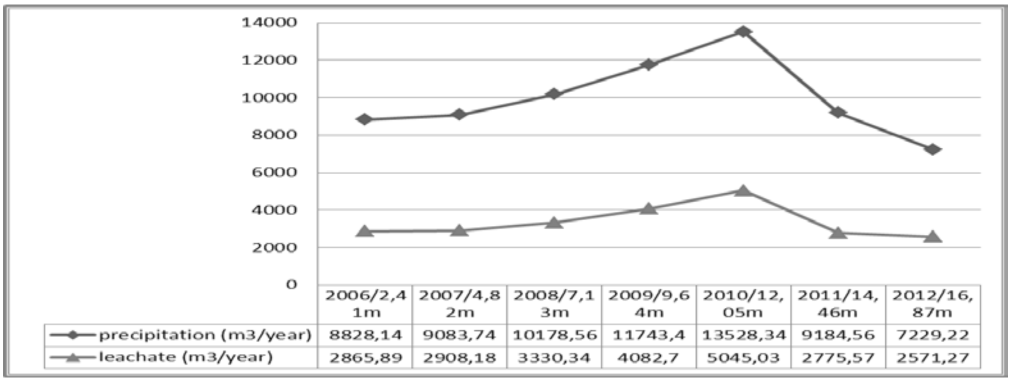

Fig. 3 Production of leachate during operating years

The change in water storage in landfill was varied depended upon the infiltrating rain, evaporation and leachate generation components.

Use the HELP model was performed complex simulation behaviour of the landfills. Because the HELP model does not take account changes in physical and chemical properties of the municipal waste thus the production of landfill gas, it was necessary to modify the model results. Creation of landfill gas affects the production of leachate, therefore it is necessary to take into account the amount of water consumed during his formation. For the calculation of production of landfill gas may use different equations. For the calculation of the production of landfill gas was chosen equation by E-plus-US-EPA (1997):

$$Q_{T,x} = k \text{ x } R_x \text{ x } Lo \text{ x } e^{-k(T-x)} \qquad (2)$$

where: QT, x - methane production in the current year (T) for the waste Rx,

x - year start storing,

Rx - the amount of waste disposed of in the year x in tons,

T - the current year

Lo - 2.0 m^3 per ton of waste

k - factor of degrade, 0.04 for wet environment [2].

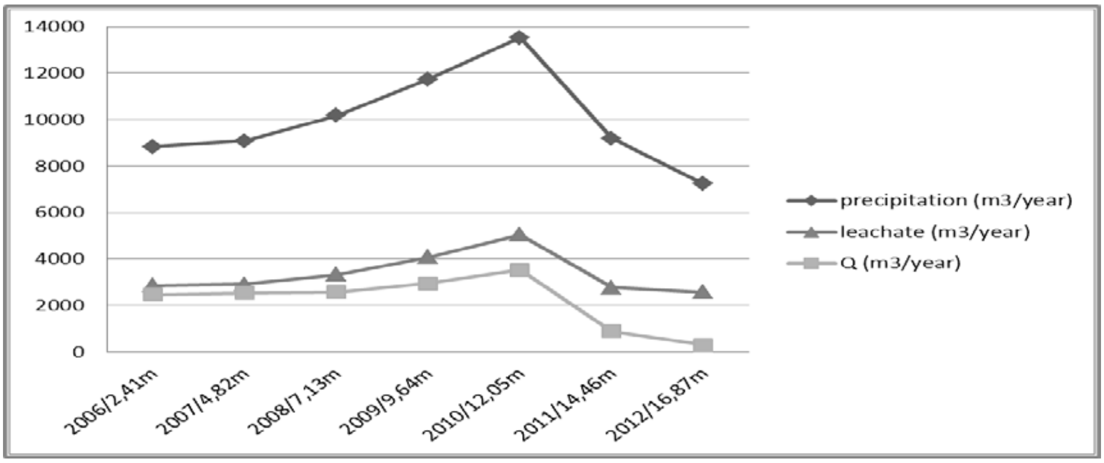

Fig. 4 Production of leachate during operating years after modification

Fig.4 shows the ratio of rainfall during the seven years of landfill operations, the amount of leachate and Q represents the total amount of leachate after the modification. At higher volume of disposed waste are consumed a larger amount of water consumed in the formation of landfill gas. After taking account production of landfill gas, the amount of leachate decreased by 35.42%.

6. CONCLUSIONS

In Slovakia the open landfill approach is the most predominant waste disposal option creating considerable environmental problems. With the accelerated generation of waste caused by an ever increasing population, urbanisation and industrialisation, the problem has become one of the primary urban environmental issues.

This study aims to investigate water management in the open landfill. Water management consisted of storage and evaporation of leachate. The application of Hydrologic Evaluation of Landfill Performance (HELP) model for water management was determined.

The results of simulation water balance of open MSW landfill from HELP model indicate that the leachate generation was the main component of water balance (around 33.79 % of total precipitation) which needs to manage for open MSW landfill.

Use the HELP model was performed complex simulation of water balance of the landfills. Because the HELP model does not take account the production of landfill gas, it was necessary to modify the model results. The amount of leachate was reduced, which may affect of water management of landfill. Simulation of the leachate production can be predicted quantities for few decades.

7. ACKNOWLEDGMENTS

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Risk analysis and risk management in water supply system sector

Pavol Nemeš, Jarmila Božiková

***Abstract* - Ever new emerging diseases and pathogens, aging infrastructures or deliberate contaminations introduce the threat for quality of water supplies. These dangerous events have been important for decide of implementation risk analysis and risk management as first step for create water safety plans (WSP) in developing countries. The text presents existing Framework and national guidelines together with illustration a basic principle for risk managing in water supply from catchment to consumer. The main goal of project is create a methodology of hazard identification and risk analysis of water supply systems for Slovak Republic conditions.**

***Keywords* – risk analysis, water supply, water safety plans**

1. Introduction

The main goal at water distribution is a quality-water delivery, which is hygienically safe and satisfy requirements of consumers. The owners of infrastructures should have obligation to protect the system against qualitative (chemical, biological) and quantitative (lack of water) dangers. Also we don't forget a flood situations or still increasing periods of drought, which have a dangerous impact for water supply in many countries. All these risks with unexpected or intentional interventions mean threats from outside, state authorities and infrastructures owners compelled to higher attention by water distribution issue. The importance of water safety and theory of risk in water supply has increased after the terrorist attack on the WTC in USA on 11. September 2001.

Acceptation of water safety plans

Even if many systems of water distribution are delivering quality water without any Water Safety Plans (WSP), implementation and acceptation of such plans bring many advantages. The main benefits are systematic and detail risk assessment, to know their priority and adoption of regulations measures. In addition, a WSP provides for an organized and structured system to minimize the chance of failure through oversight or lapse of management and for contingency plans to respond to system failures or unforeseen hazardous events [6]. Water Safety Plans are also as part of WHO 3rd guidelines of drinking-water quality.

2. Risk analysis

Risk analysis is main part of risk management, but it doesn't exist a valid definition to describe the risk. We can define the risk as comprehensive function of hazard event connected with social, technological and environmental or another system. The risk is possible to precise according to two main notions which we can find in any current definition:

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- occurrence of undesired events,
- probability (frequency), when undesired consequences will occur.

The risk analysis means the use of all information about system to evaluate and estimate the hazard for lives, health, environment and property. For the purposes of the water supply system (WSS) risk analysis, we have accepted and developed a definition of risk which can be expressed as follows:

$$R = P \times C \tag{1}$$

where P stands for probability of occurrence of undesired event and C stands for consequences of the event.
The risk analysis serve as precautionary access by risk solution, when the individual scenarios are searched, analyzed and further evaluated with consideration on the population damage [5].

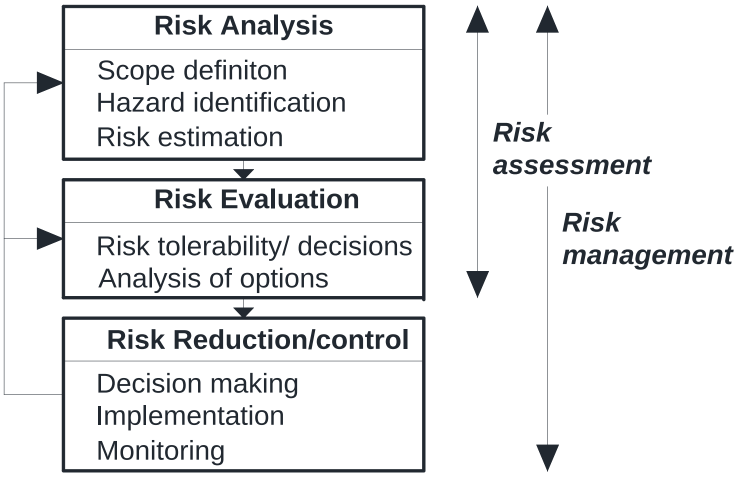

Fig. 1 Risk assessment and management [1]

Risk structuring

Hazards may arise in the whole water supply system (WSS), therefore for the purpose of risk analysis the WSS is divided into four subsystems. Also hazards are divided according to their origin into three basic categories. The structuring of risk is shown in Fig. 2.

WSS Parts	**Origin of hazard**	**Consequences**
Water source	Natural Hazards	Health
Water treatment	Manmade threads	Economic
Water distribution	Technical and technological hazards	Socio-economic
Service connections and plumbing		Environmental

Fig. 2 Risk Structuring [5]

3. WATER SUPPLY SYSTEM DESCRIPTION AND IDENTIFICATION OF HAZARD

The first task by creation of risk analysis is to fully describe the water supply. This description covers the whole water supply system from the catchment area to the tap including water resources, treatment technologies etc. It is supposed that the operator as the enduser choses those parts, which exist in his water supply system. Selected parts should contain information like dimensions, age, functionality, operating conditions, familiar problems, etc.

As we can see on the figure 3, the water supply system consist out of four main systems and every main system can be subsequently divided into subsystems.

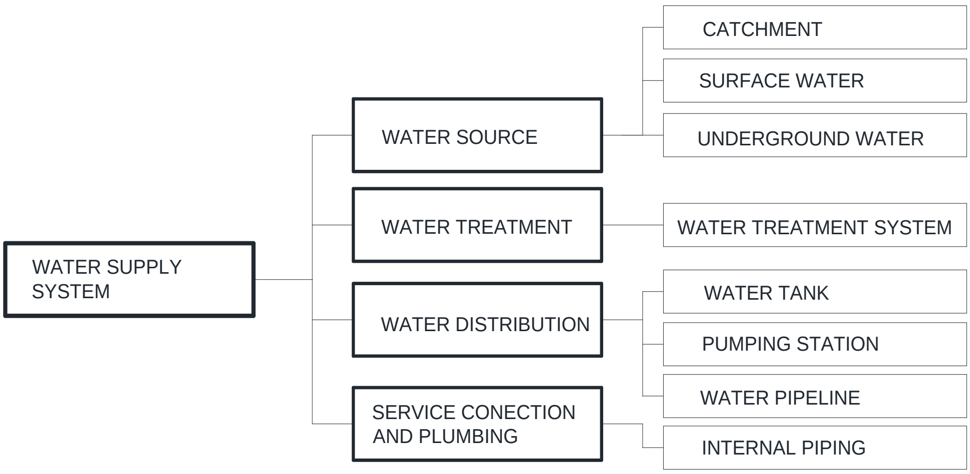

Fig. 3 Example to describe water supply system.

The hazard identification as a next step by creation of risk analysis requires that all potentials hazards, such as chemical, biological, physical and radiological, have been located and analyzed. For reason of easy identification of hazards was created the Techneau Hazard Data Base (THDB). The database has to be user-friendly and at the same time complete for providing sufficient information. The presented database aims to cover both aspects. The water supply system is subdivided into 12 sub-systems, of which 10 are physical sub-systems representing the installations, one is a non-physical subsystem representing organizational aspect and one is a sub-system representing future hazards [4].

Table. 1 Illustration of the use of TECHNEAU hazard database (THDB). [4]

System	**Example hazardous event from THDB - to be used as checklist**
Source/Catchmen	- Industrial discharges of chemicals (continuous discharge, no accident). THDB 1.1.1 - Contamination of catchment zone: Industrial discharge of biological matter. THDB 1.1.2 - Contamination of catchment zone: Emissions during accidents (fire or explosions) e.g. industrial accidents or forest fire. THDB 1.1.3
Water treatment plant	- Insufficient flocculation. Improper coagulant mixing and/or flocculation; inappropriate flocculant or flocculation agent; improper pH control. THDB 6.4.3.
Distribution and plumping	- Contamination introduced during repairs. Poor hygiene during repairs. THDB 8.1.2.

3. THE RISK ANALYSIS METHODS

Nowadays there are many methods how to manage risk analysis of technical systems. Existing methods can be divided into the “ex-ante methods” (methods before the event) and “ex-pos methods” (opposite to the actual method).

Our selected method should have following criteria:

- scientific defensible and suitable for existing system,
- provide a results in formula, which improves the understanding of the nature of risk and the way how regulate it,
- easy use for different professional users with possibility of repeat and verify.

The main issues in selection of the method is how to estimate the values of C and P, the lack of data, uncertainty of employed methodology of risk analysis and proper interpretation, etc. This issues can be solved by using of frequency instead of mathematical probability of occurrence of undesired event and also by employing the FMEA/FMECA methodology [2]. FMEA uses categorization of probability of occurrence, severity of consequences and all other potential inputs into categories. For instance, categories of frequency of occurrence may be as follows: almost certain – likely – moderate – unlikely – rare. The category is then represented by its point-score only, e.g. almost certain 5, rare 1. Each analyzed element is to be assigned into one of the categories. This is done based on some chosen factors or indicators and, of course, based on limits of categories. Limits are set up by expert with sound knowledge of the system. This is a very efficient approach especially in the situation where hard data is missing and the analysis has to be based on “soft” data. [5] [2]

For our study will be used The CRA (Coarse Risk Analysis) method. This method is used in small water supplies and seems to be a suitable tool for risk identification and estimation.

Table. 2 Definition for likelihood used for risk estimation.

Level	Descriptor	Description
Likelihood		
A	Almost certain	Once a day
B	Likely	Once per week
C	Moderate	Once per month
D	Unlikely	Once per year
E	Rare	Once every 5 years

Table. 3 Definition for consequence used for risk estimation.

Level	Descriptor	Description
Consequence/impact		
1	Insignificant	No detectable impact
2	Minor	Minor aesthetic impact causing dissatisfaction but not likely to lead to use of alternative less safe sources
3	Moderate	Major aesthetic impact possibly resulting in use of alternative but unsafe water sources
4	Major	Morbidity expected from consuming water
5	Catastrophic	Mortality expected from consuming water

CRA is relative simple for use and may be completed or replaced by a more sophisticated method, for example FMEA, but for the initial risk assessment process it represents a reasonable approach due to professional, technical and financial requirements [2] [3]. As initial material was used the material from European project Techneau "Risk assessment and Risk management".

Table. 4 Risk analysis matrix

Likelihood	Consequences				
	Insignificant	Minor	Moderate	Major	Catastrophic
A (Almost certain)	**H**	**H**	**E**	**E**	**E**
B (Likely)	**M**	**H**	**H**	**E**	**E**
C (Moderate)	**L**	**M**	**H**	**E**	**E**
D (Unlikely)	**L**	**L**	**M**	**H**	**E**
E (Rare)	**L**	**L**	**M**	**H**	**H**

E – Extreme risk, immediate action required; H – High risk, management attention needed; M – Moderate risk, management responsibility must be specified; L – Low risk, manage by routine procedures.

4. CONCLUSIONS

Operation of systems based on the safety analysis and rentability of risk, is currently used in nuclear sector, aviation or energetic industry as the primary task. The obtained knowledge from sectors, that are listed above and from other distribution systems are lately used in water supply sector, after the previous modification. Estimate and risk analysis is becoming the basic background for working out the strategies of risk reduction and also possible consequences threats for individual water supply companies. In the water industry, it is the theory of risk that is being used for example, for creating flood maps.

However, currently there exist no legislation regulations in Slovak Republic, which would oblige the water supply companies to make the plans for water safety assurance. But the assumption of new direction dealing with drinking water is expected in horizon of some next years among the European states. This direction will replace the current used one 98/83/EC. In the coming years it is planned that the risk analysis and risk management principles will be legally introduced and obligatorily used by water utilities in the process of drinking water abstraction, production and distribution in all EU countries.

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Increasing the Efficiency of the Coagulation and Flocculation Processes in Water Treatment Plants

Ion Oprea

Abstract – **In order to establish the technological optimum operating parameters and the types of reagents for the water treatment plant Palas Constanta, experimental determinations were performed in laboratory to establish the optimum coagulation-flocculation reagents.**

Laboratory research was conducted in order to establish the necessity of using coagulation-flocculation reagents, their effect on the treated water and the recommended dosage of reagents that produce maximum rinsing effect in a period of time as short as possible.

Keywords – **coagulation, flocculation, quality, treatment, water.**

1. INTRODUCTION

Particles in natural water vary in origin, size and concentration. They can come from atmospheric sources (clays, pathogens, asbestos fibers) from the ground or can be the result of chemical and biological processes that occur in raw water (algae, $CaCO_3$ precipitate). The size of the particles can vary by several levels, from a few tens of nanometers (viruses) to several hundred micrometers (microplankton). These particles can be efficiently removed from the water if the processes of coagulation, sedimentation and filtration were properly designed and built. **Figure 1** presents the size of the most common water particles in comparison to the size of the pores of different filters [1].

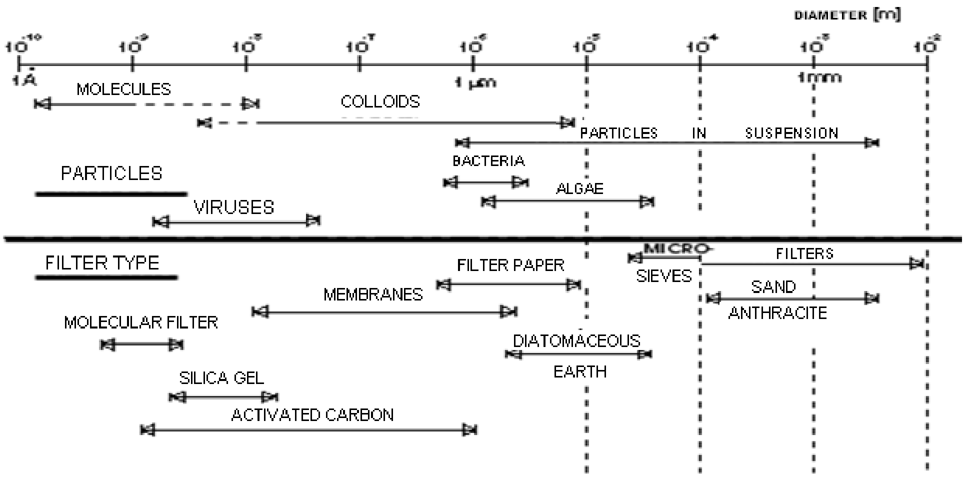

Fig. 1. Dimensions for types of particles and types of filters [1].

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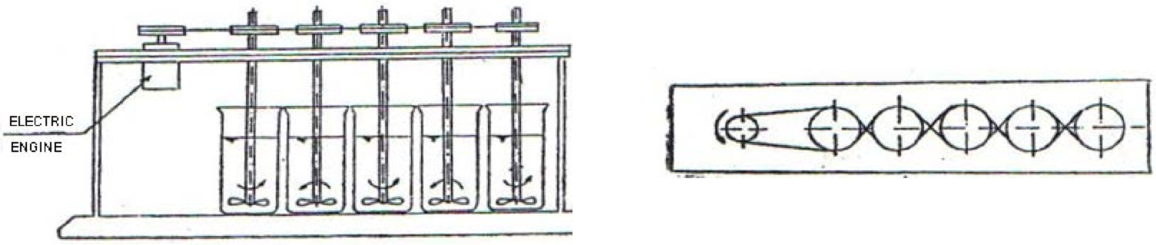
2. Dosing of Reagent in Treatment Plants

An accurate dosage of reagents is meant to establish the minimum quantities of reagent producing maximum rinsing effect in as short a period of time as possible. The Jar-test procedure, considered to be the closest to reality, uses a device (**Fig. 2**) which performs systematic trials with different doses of coagulant [2].



a–front view b–top view

Fig. 2. Determinating the accurate dosage of coagulant-JAR-TEST procedure [2]-[4].

Treated water is introduced in 5-6 containers of $1dm^3$. Each container is given a dose of reagent, close to the upper and lower dose fixed by the statistics resulted from water treatment practice. They are mixed in two stages, fast then slow, aiming at finding the effect of each dose through visual observation and accurate determinations.. The company I.WS.A - U.S.A. recommends: 1 minute at 60 to 80 rev/min.; 15 minutes at 30rev/min; after 20 minutes, precise determination of turbidity, color and organic substances; after 30 minutes, its filtration and determination of turbidity and organic matter [2]-[4].

An extended practice on water treatment indicates the need for a safety factor to be introduced for doses used through the industrial equipment, compared to those produced in a laboratory, according to the relation:

$$D_{apl} = C_{sig} \times D_{opt} \tag{1}$$

Values recommended for the safety coefficient: $C_{sig} = 1.2 \div 1.5$ depending on: the purity of reagents in industrial plant equipment; the efficiency of dosing -injection equipment, the efficiency of stirring, mixing and reaction systems; the climatic conditions in which the industrial equipment is operating (water temperature, air temperature) [2]-[4].

Flocculation tests should be performed:for each significant variation of the following parameters of the raw water: temperature, pH, turbidity and mineralization, chemical oxygen demand (in case of accidental pollution); for every significant change in the treatment chain or operating conditions of the plant; when analytical controls emphasize a degradation of the quality of the decant or treated water: when the values continuously exceed the required limits determined by laboratory analysis; exceeding the values of required parameters of water such as: aluminum content, iron, manganese, microbiological features. [2], [5].

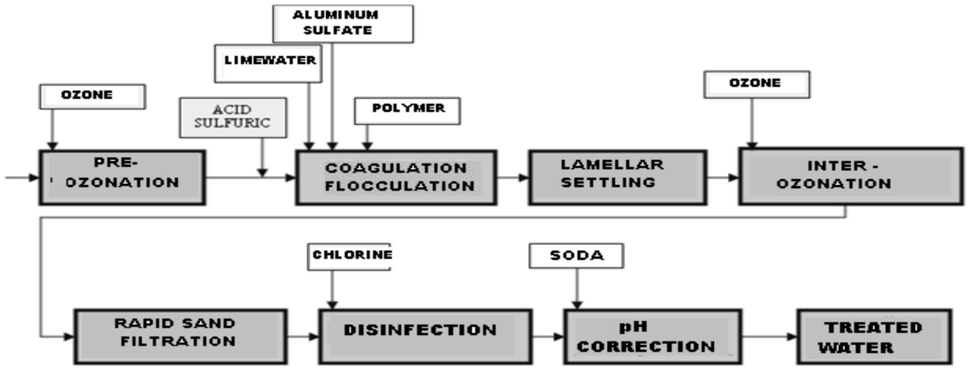


Fig. 3. The diagram of a water treatment plant which describes how to determine the dosage of the reagent. [2], [5], [6]

Depending on the raw water quality parameters, especially turbidity and temperature, after performing a jar test the following are noted:

- The dose of aluminum sulfate, the reagent used for the coagulation of colloidal particles in the raw water (unsettled very small particles which are in balance in the stream of water and which give the water turbidity);
- The dose of sulfuric acid or whitewash, some reagents used to adjust the pH of the water during coagulation;
- The dose of polymer, which is an auxiliary of aluminum sulfate.

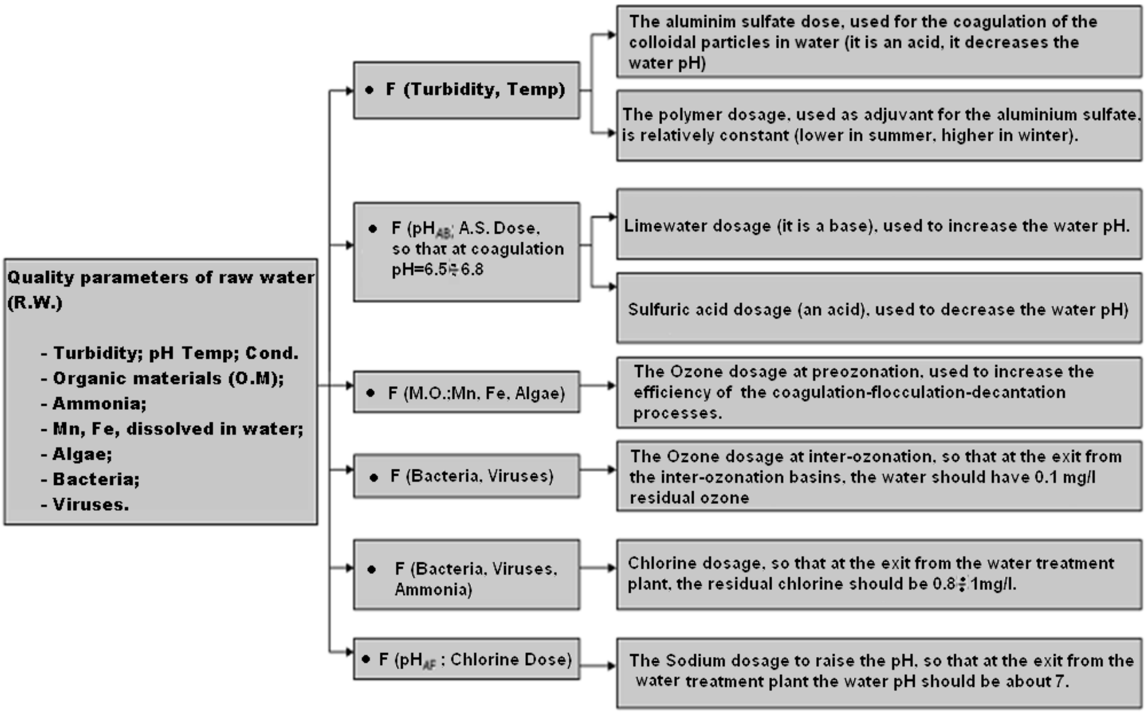


Fig. 4. The diagram of the way the principle of the reagent dosage is established [2], [5], [6]

The aluminum sulfate is an acid and, in addition to the role of coagulating colloidal particles in water, it also decreases the pH of the water.

The pH value for which the maximum efficiency of coagulation is possible is within the limits of 6.5 ÷ 6.8. In order for this process to take place in best conditions, the pH value has to be between these limits at the coagulation time [2].

The polymer is an adjuvant for the aluminum sulfate, having the purpose of increasing the efficiency of the decantation process, by linking between them more flocs formed during coagulation, thereby increasing the mass of the newly formed flocs, thus, the conditions for a more rapid decantation.The doses of polymer are relatively constant, higher in winter and lower in summer, due to the temperature of water (the efficiency of mixing the polymer with the water is lower in winter and higher in summer). The dosage of ozone in the pre-ozonation stage is determined by the loads of iron, manganese and organic matter in the raw water. This facilitates the coagulation-flocculation mechanism by destroying the organic film surrounding the colloidal particles which prevent the attachment of the cations of the coagulants. So the pre- ozonation stage helps to increase the efficiency of the coagulation - flocculation – decantation processes. The dose of ozone for inter-ozonation stage is set so that, after this stage, the residual ozone in the water shold be around 0.1 mg / l, in this way ensuring the water disinfection. The dosage of chlorine is such that at the exit from the water treatment plant the residual chlorine in water is 0.5 mg / l [2], [7], [8].

The last reagent injected is sodium hydroxide, this one having the role of establishing calco-carbonic balance and rising the pH of water. The dosage of soda is set so that at the exit from the treatment plant, the pH should be at about 7, in order to prevent the clogging of pipes due to calcar deposits and the corrosion of the distribution pipes made of steel. One important thing to operate the water treatment plants is the fitting of online analyzers for water quality for both water entering the treatment plant and for each stage of the process and for the water leaving the treatment plant and being able to see graphically the evolution in time of these parameters. Thus, after modifying a dose of reagent, one should be able to see the effects this change and, by this, to adjust the dosage if necessary. Also, for a better exploitation of the water treatment plants, it is good that at least one analyzer for on-line monitoring of quality of water supply to exist upstream (about 2÷4 away from the treatment station) [2].

3. Laboratory Studies on Some Processes of Coagulation-Flocculation

Laboratory studies have been conducted in order to establish the necessity of using coagulation-flocculation reagents, the recommended doses and their effect on the treated water.

Law 458/2002 [7] regarding the quality of drinking water provides turbidity values of 5 NTU, but where disinfection with chlorine is done it is mandatory that the treated water turbidity to be more than 1 NTU. This is related to the fact that natural organic matter (NOM) in the water reacts with chlorine and can produce trihalomethanes (THM), which increase the risk of cancer.

NOM removal from natural waters without using coagulation-flocculation reagents is extremely low and therefore it is considered necessary testing coagulation-flocculation reagents in the process of raw water treatment.

Thus, several coagulation– flocculation tests have been done with the following reagents:

A. Aluminium sulfate;
B. Basic Aluminium Polychloride: MOPAC, PAX 18;
C. Ferric Chloride.

A. Coagulation Tests with Aluminium Sulphate

The results of the coagulation tests with aluminium sulphate at the natural pH of the water are shown in **Table 1**.

Table 1. Coagulation test with aluminum sulfate

Sample	1	2	3	4	5	6
Aluminiun sulphate dosage (mg/l)	10	15	20	25	30	35
Residual turbidity (NTU) – after 15 min. sedimentation	2.82	0.87	0.52	0.44	0.44	0.76
pH (pH units)	8.27	8.18	8.07	8.01	7.92	7.86
Oxidability (mg $KMnO_4$/l)		13.9		12.62		12.62
Total organic carbon (mg C/l)		6.41		5.3		5.01

It is obvious a reduction of turbidity values of up to 0.44 NTU at a dose of aluminum sulphate of 25-30mg/l.

The concentration of organic substances corresponding to this dose was of 12.62 mg KMnO4 /l (15.7% reduction).

We carried out a coagulation test at pH = 6.7. Its results are presented below (**Table 2**).

Table 2. Coagulation test with aluminum sulfate to pH = 6.7

Sample	1	2	3	4	5	6
Aluminiun sulphate dosage (mg/l)	10	15	20	25	30	35
Residual turbidity (NTU) – after 15 min. sedimentation	0.49	0.37	0.29	0.23	0.37	0.4
pH (pH units)	6.68	6.66	6.64	6.63	6.61	6.58
Oxidability (mg $KMnO_4$/l)		12.64		8.84		8.53
Total organic carbon (mg C/l)		5.7		5.12		4.9

Note the reduction in turbidity to value less than 1 NTU from aluminiun sulphate doses of 10 mg/l.

In terms of organic load reduction, coagulation at low pH led to a better retention. The following figures (**Fig. 5**) show in a comparative way the variation of concentration of organic substances versus the concentration of total organic carbon.

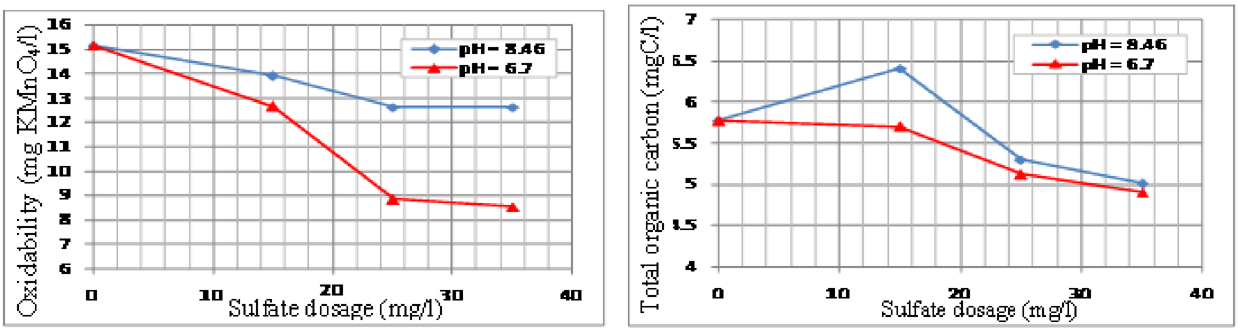


Fig. 5. Organic load variation depending on the dosage of aluminum sulfate - coagulation at natural pH of the water and at pH = 6.7

Note that lowering of the pH before coagulation reduces the oxidizable organic substances with an efficiency of 41.69% at a dose of 25 mg / l compared to 16.75% for the same dose with coagulation at the natural water pH. Also, the reduction efficiency of the total organic carbon concentration was higher in the case of pH reduction before coagulation.

B.1 Coagulation Test with Aluminium Polychloride – MOPAC

A coagulation test has been made with MOPAC, which is a basic aluminum polychloride used in water treatment processes. MOPAC coagulation test results are shown in **Table 3**.

Table 3. Coagulation test with MOPAC

Sample	1	2	3	4	5	6
MOPAC dosage (mg/l)	5	10	15	20	25	30
Residual turbidity (NTU) – after 15 min. sedimentation	0.41	0.38	0.38	0.38	0.3	0.35
pH (pH units)	8.38	8.37	8.31	8.28	8.24	8.22
Oxidability (mg $KMnO_4$/l)	12		11.69		11.69	
Total organic carbon (mg C/l)	5.55		3.98		3.92	

Note the reduction in turbidity to values of 0.3 NTU for a MOPAC dose of 25 mg/l.

The concentration of organic substances corresponding to this dose was 11.69 mg $KMnO_4$ / l compared to 15.16 mg $KMnO_4$ /l for raw water, while the total organic carbon concentration was 3.92 mg C/l compared to 5.77 mg C/l for raw water (32% reduction).

B.2 Coagulation Test with Aluminium Polychloride - PAX 18

A coagulation test has been made with the PAX 18, which is a basic aluminum polychloride used in water treatment processes. The results of coagulation tests with PAX 18 are shown in **Table 4**.

Table 4. Test coagulation with PAX 18

Sample	1	2	3	4	5	6
PAX 18 dosage (mg/l)	5	10	15	20	25	30
Residual turbidity (NTU) – after 15 min. sedimentation	0.67	0.49	0.39	0.31	0.31	0.26
pH (pH units)	8.01	7.98	7.93	7.92	7.88	7.84
Oxidability (mg $KMnO_4$/l)		10.11		9.48		9.48
Total organic carbon (mg C/l)		5.12		4.71		4.11

When using coagulation reagent PAX 18 turbidity values decreased to 0.26 NTU for a dose of 30 mg/l PAX 18 compared to 7.36 NTU for raw water. The concentration of organic substances corresponding to this dose was 9.48 mg $KMnO_4$/l dompared to 15.16 mg /l $KMnO_4$ for raw water (reduction efficiency of 37%).

C. Coagulation Test with of Ferric Chloride

The results of the coagulation test with ferric chloride are shown in **Table 5**.

Table 5. Coagulation test with ferric chloride

Sample	1	2	3	4	5	6
Ferric chloride dosage (mg Fe^{3+}/l)	1	2	3	4	5	6
Residual turbidity (NTU) – after 15 min. sedimentation	0.75	0.59	0.54	0.46	0.49	1.66
pH (pH units)	7.96	7.96	7.88	7.83	7.75	7.71
Oxidability (mg $KMnO_4$/l)		10.74		10.11		9.79
Total organic carbon (mg C/l)		5.69		5		5.76

Water turbidity decreases to values below 1 NTU when using ferric chloride, but it should be noted that the supernatant remains slightly colored.

4. CONCLUSIONS

In order to establish the optimal technology of the operating parameters as well as the types and doses of reagents for the Palas Constanta Water Treatment Plant, laboratory experimental determinations have been made to find the optimal coagulation-flocculation reagents.

To determine the quality of the raw and the treated water in the Palas Constanta Water Treatment Plant, complex analyzes have been performed in the laboratory of the Technical University of Construction in Bucharest, in the Sanitary Engineering and Water Protection Department.

The analysis of the results of raw and treated water quality has revealed the following :

- raw water which supplies the Palas Constanta Water Treatment Plant is relatively clear, but it has a relatively high organic load, likely to produce disinfection byproducts (trihalomethanes);
- from a biological point of view , the analyzed raw water samples, taken both from the Danube - Black Sea (Galesu section) and the entrance in Palas Constanta Water Treatment Plant, belong to the category of quality “oligosaprobe-β-mezosaprobe”, i.e. water with low to moderate contamination levels, with high levels of dissolved oxygen and without polisaprobe forms. Water shows a small number of bacteria and high numbers of insects and insect larvae, and the saprobic index value corresponds to the value determined for a "good" ecological status; the concentration values of chlorophyll type "a" place the water in the quality class I - II which corresponds to a "very good" - "good" ecological status [11];
- the measurements done on *Dreissena polymorpha* indicate a significant numerical abundance of both young and adult species;
- the water treated in the Palas Constanta plant shows the turbidity exceeding in some cases the amount required by Law 458/2002 [7] for drinking water produced from surface sources (1 NTU); in terms of organic load, had the concentration of oxidizing organic matter with potassium permanganate in the treated water was of 11-12 mg $KMnO_4$/l and respectively a total organic carbon concentration of 3.5 - 6.2 mg C/l. According to the specialized literature, higher concentrations of total organic carbon in treated water (>2mg/l) leads to bacterial growth potential in the water distribution network, i.e. with trihalomethane formation potential when using high doses of chlorine for disinfection.

Worldwide research (Chapra, Canale and Amy, 1997) [9], have shown a strong correlation between the potential formation of trihalomethanes and total organic carbon concentration in the raw water. Thus, it has been established following relation [9]:

$$THMFP = 43{,}78\,TOC^{1{,}248} \tag{2}$$

Where: THMFP (μg /l) - trihalomethanes formation potential; TOC (mg /l) - the concentration of total organic carbon. According to this formula, for an average concentration of 8 mg C/l in raw water, trihalomethanes formation potential is of 586.58 μg/l , if the chlorine dose and contact duration are adequate.

The reduction of organic load after sand filtration stage was performed at concentrations of 3 mg/l, which leads to a trihalomethanes formation potential of 172.47 μg/l compared to of 100 μg/l, the maximum allowed by Law 458 / 2002 [7]), which shows significant risk of cancer formation.

Therefore, in order to avoid the formation of these compounds and to retain the compound formed at the source in the pre-oxidation stage, the post-oxidation with ozone and the adsorption on active carbon are necessary.

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SECTION VI

HYDRAULIC STRUCTURES

HYDRAULICS AND FLUID MECHANICS

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Methodology for Fish Passes Design

Lea Čubanová

Abstract – **Fish passes are constructions for ichthyofauna, which enable passing of the gradient created by water structure and restore stream continuity for fish migration. They include technical (vertical slot pass, pool pass, Denil pass) and also natural types (rocky chutes, ramps, biocorridors) of construction. Article is presenting a procedure for fish pass design on water structures. This procedure can be summarized in three parts – the data, the design and the verification.**

Keywords – design parameters, fish pass, methodology, water level and discharge regime

1. INTRODUCTION

Constructed water structures and hydropower plants almost certainly create a barrier for the ichthyofauna migration. This is the reason for decrease of fish population downstream as well as upstream the river. Therefore the part of every water structure must be also fish pass. Today, it is possible to say that in general there is a trend of the building of the natural fish passes eventually the hybrid types, which are partly designed as technical, but have elements of natural character. Some standards exist for the design of all hydraulic structures. In our country, the fish passes do not have such a standard, therefore their hydraulic design is difficult for the designers and mainly this design is not acceptable for the ichthyologists during the environmental impact assessment.

2. DESCRIPTION OF THE METHODOLOGY

The methodology for the fish pass design combines into one complex the technical knowledge, hydraulic calculations and requirements for the occurring ichthyofauna. It describes the order of the single steps of the design and simulation to achieve the optimal fish pass design, which is hydraulically functional and also attractive for the migratory ichthyofauna. The paper mentions the interconnection of the individual steps, which affect the next design procedure [1].

Input data are the most important for the beginning of the preliminary design. From these data, the designer obtains relevant information and values for the fish pass design, which are necessary, because it is not possible to use standardized fish pass schemes, but every design needs an individual approach. This idea results from the uniqueness of every water structure, as well as from the type and zone of the stream and existed fish species. The data can be divided into the three categories [1]:

- water management data (they are based on the parameters of the designed water structure):
  - *gradient* (H) – it represents the difference between upstream and downstream operating water level of the water structure. Many water structures are built for the utilization of the hydropower potential and therefore they do not keep the water surfaces on the constant level, but these levels are changing in the range between

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maximal and minimal operational water level defined in the operational manual. Therefore it is necessary to find all real combinations of the upstream and downstream water levels (the possible gradient values).

- *discharge* in the fish pass (Q) – this parameter determines the operator of the water structure and the hydropower plant according to the water structure's character and the recommended minimal biological discharges. It is recommended to supply the fish pass with a discharge, which is approximately equal to 1 % – 6 % of the mean discharge in the stream (Q_a). The fish pass must be designed that it is functional for at least 300 days per year [2].

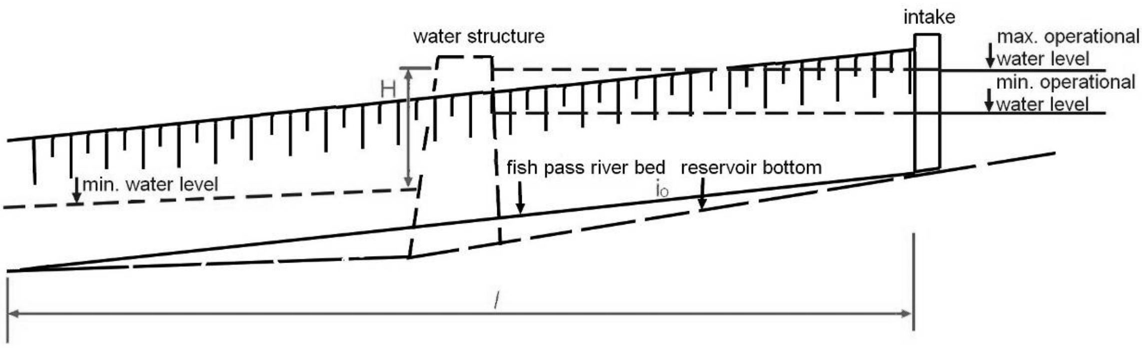

Fig. 1. Longitudinal section of the water structure with fish pass

- geomorphological data – the terrain data, obtained by the survey and geodetic measurements of the area of the planed water structure. Besides, it is appropriate to use an available map data, which will serve especially for the design of the future course of the fish pass. In this step it is necessary to know the area size, which will be used for the planed fish pass realization. From the area of interest , the following data is needed to be found out:
  - *geology* – based on its analysis it is possible to design a stable cross section, especially due to the bank slope.
  - *topography* of the original stream – the designer should make a detailed survey of the stream and its vicinity, he should notice the stream's routing, fish refuges, boulder elements as well as the existing fish fauna. It is appropriate to determine the cross sections by the geodetic measurements and to find out the discharge and velocities in the stream for achieving the relevant data. The designer obtains a review of the depths, longitudinal slope, water level differences on the sills, roughness coefficients and arc radiuses.
- ichthyological data – the fish pass serves for the migration of fish species in both directions, so it has to respond to their needs and requirements. The designer needs following data for the fish pass project from the ichthyologists [1], [2]:
  - the stream zone, or the type of the fishing ground (trout, grayling, barbel, bream),
  - the existing ichthyofauna species (trout, bullhead, grayling, dace, barbel, undermouth, pike),
  - the values of the minimal and maximal depth suitable for the existing ichthyofauna (trout streams: y = 30 – 50 cm, other than trout streams: y = 50 – 80 cm),
  - the maximal acceptable water level difference or the jump height of the existing species (trout streams: Δy_{max} = 20 cm, other than trout streams: Δy_{max} = 15 cm),
  - the maximal cross sectional velocity (v_{max} = 1,5 - 2 m.s^{-1} for trout, dace, grayling, nase, and common huchen, v_{max} = 1,2 – 1,5 m.s^{-1} for gudgeon, stone loach, minnow),
  - the starting distance of the fish, because of the jumps (the recommended minimal length of pool is 2 – 5 m depending on the fish region),
  - the migration period,
  - the fish abundance (abundance (pcs.ha^{-1}), ichthyomass (kg.ha^{-1})).

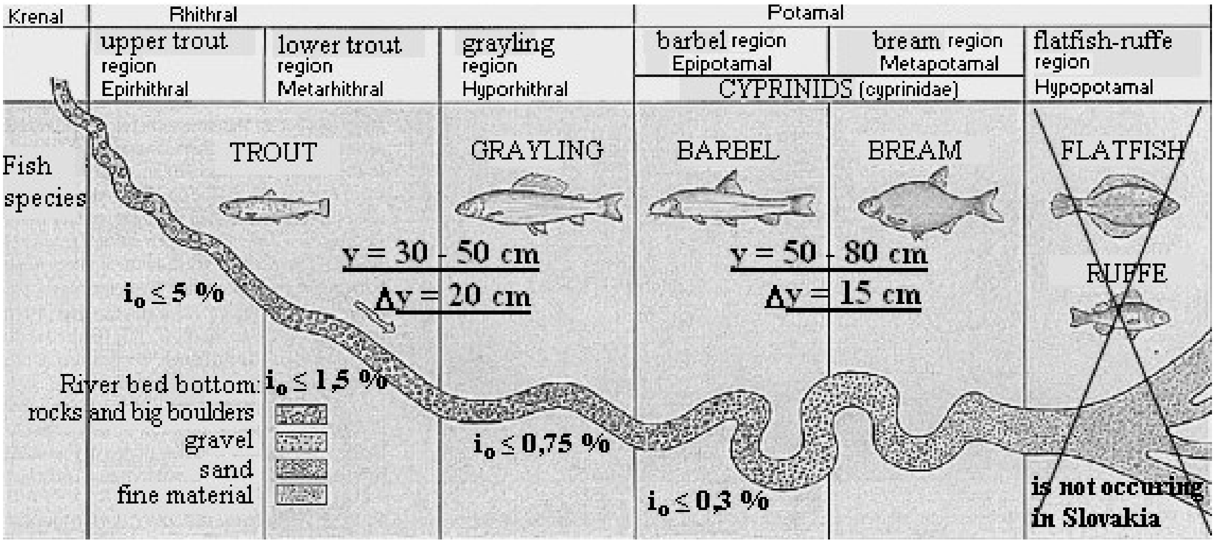

Fig. 2. Stream zonation according to the fish species

Table. 1. Limit values for the fish fauna (ramps and biocorridors) [3]

Stream zone	Max. water level difference Δh_{max} (m)	Max. middle velocity in the pool v_B (m.s^{-1})	Max. middle velocity in the migration corridor v_{WK} (m.s^{-1})	Main flow by Q_a v_{min} (m.s^{-1})	v_{max} (m.s^{-1})
Epi-Rhithral	0,20	0,5	1,0	0,3	2,0
Meta-Rhithral	0,18	0,5	1,0	0,3	1,9
Hypo-Rhithral	0,15	0,5	0,9	0,3	1,7
Epi-Potamal	0,13	0,5	0,8	0,3	1,6
Meta-Potamal	0,10	0,5	0,7	0,3	1,4
Hypo-Potamal	0,09	0,5	0,6	0,3	1,3

Table. 2. Limit design parameters of the pools and slots in the fish pass depending on the fish species [3]

Fish species	Pool dimensions by Q_{30} (clear dimensions)				Min. slot width for at least 1 slot on the sill		Approximate values for the smallest discharge in the fish pass	
	Min. depth under sill h_u (m)	Min. slot height $t_{s,min}$ (m)	Min. clear length L (m)	Min. clear width b (m)	Technical types s (m)	Natural types s (m)	Technical types $Q_{FAA,min}$ (m^3.s^{-1})	Natural types $Q_{FAA,min}$ (m^3.s^{-1})
Trout	0,4	0,2	1,5 – 1,9	1,0 – 1,2	0,15	0,2 – 0,4	0,1	0,2
Grayling, Bream, Chub	0,45	0,2	2,0	1,4	0,17 – 0,3	0,4 – 0,6	0,15 – 0,25	0,35
Barbel, Bream, Pike-Perch, Pike, Danubian Salmon	0,5	0,3	2,8 – 4,0	1,8 – 3,0	0,3 – 0,6	0,6	0,4 – 1,0	0,5 – 0,55
Sterlet	0,8 – 1,0		5,0	3,0	0,8	0,8	0,7 – 1,5	1,2 – 2,0

Table. 3. Recommended dimensions for pool passes [4]

Fish species to be considered	Pool dimensions in m			Dimensions of submerged orifices in m		Dimensions of notches in m		Discharge through fish pass	Max. difference in water level
	Length l_b	Width b	Depth h	Width b_s	Height h_s	Width b_a	Height h_a	$m^3.s^{-1}$	Δh in m
Sturgeon	5 – 6	2,5 – 3	1,5 – 2	1,5	1	--	--	2,5	0,2
Salmon, Sea trout, Huchen	2,5 – 3	1,6 – 2	0,8 – 1,0	0,4 – 0,5	0,3 – 0,4	0,3	0,3	0,2 – 0,5	0,2
Grayling, Chub, Bream, others	1,4 – 2	1,0 – 1,5	0,6 – 0,8	0,25 – 0,35	0,25 – 0,35	0,25	0,25	0,08 – 0,2	0,2
upper trout zone	> 1,0	> 0,8	> 0,6	0,2	0,2	0,2	0,2	0,05 – 0,1	0,2

Design of the fish pass contains the course design, or the fish pass placement, the cross section design along the whole designed length and the design of the cross section barriers (sills, boulder elements). The basis of the fish pass course design is the water level difference (gradient), longitudinal river bed slope and area, on which is possible to build the fish pass. This area also influences the course length of the designed fish pass. These three parameters are interconnected, whereas the gradient passing is needed. The design begins upstream with a continuous joining to the downstream and upstream water surface.

The course design begins from the maximal recommended longitudinal river bed slope, which is given either by ichthyologists or there are some recommendations for the slope depending on the fish region or the stream zone. But definitely the area around the water structure is limited. It may happen that the achievement of the required slope values is impossible, neither by the fish pass course prolongation by its enormous curving. In this case it is possible to reduce a big longitudinal slope by the drop ditches, cross section structures or boulder elements. The recommended shape of the cross section of the fish pass river bed is trapezoidal or dish-shaped, because they provide a larger water surface then a rectangle cross section.

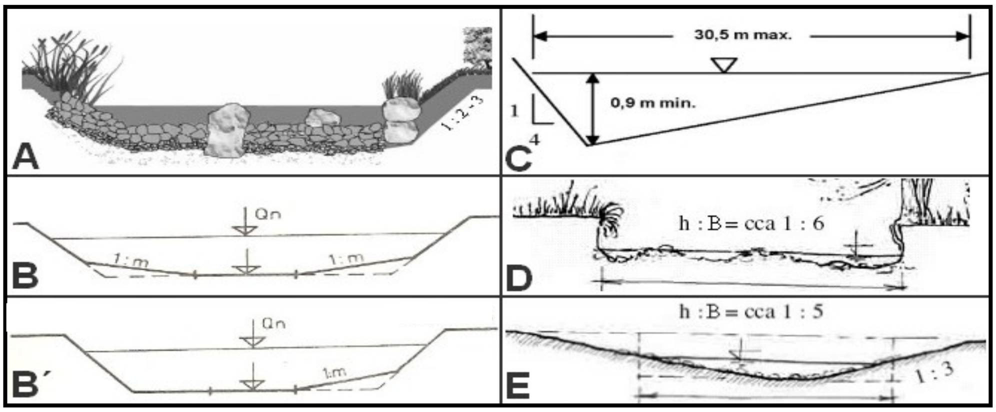

Fig. 3. Recommended cross section shapes for the fish pass

The size of the cross section, and also its width, is determined by the maximal design discharge rate, the expected quantity of migratory fish fauna and the area, on which it is possible to build the fish pass. The most efficient way is to combine both types to obtain an appropriate streamline routing along the whole length of the fish pass. If we decide for a rectangle cross section, then it is necessary to involve also barriers with slot(s) into the cross section. So a pool fish pass is created.

The design of single stones and boulder sills, which positively affect the depths and velocities in the fish pass, is the next step. The boulder elements in the fish pass river bed have various functions, e.g. the depth increasing, velocity influence. It is possible to use them locally, whereby they might be varying with the sections without any elements. In such a way, enough refuge possibilities for the fish fauna in the pools behind the boulder elements are created, the effect of the water surface stirring is achieved. All these sections enable a continuous two-way fish migration.

Single stones represent efficient means which allow the increase of depth without discharge change. Their big advantage is their natural look and easy installation into river bed, as well as their simple refilling in the case of need. These stones positively influence water level and velocity regime in fish passes. Their design is very positive not only from the hydraulic, but also ichthyological point of view. Single stones, which are designed for achievement of some hydraulic parameters, have following definition: boulders individually embedded into cross sections of the stream (they do not create a continual body), in specific spacing, on specific length, they represent local increase of macroroughness, which will cause increase of water level and thus they influence depths and velocities.

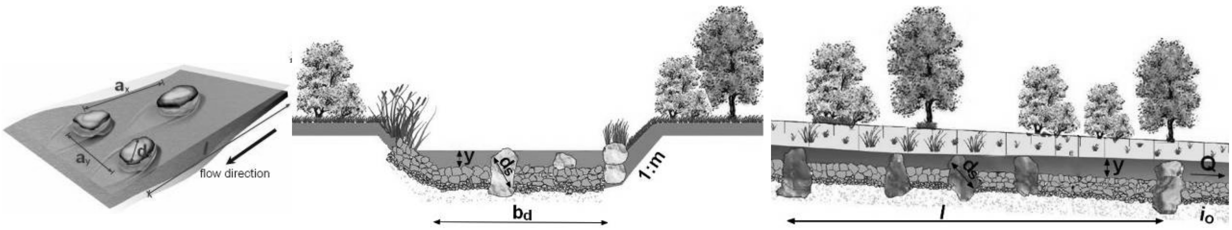

Fig. 4. Single stones in the river bed

Boulder sills are constructed across the river bed. They are created from different boulder sizes, placed vertically and horizontally. Vertical position creates a weir and horizontal position forms spill. Several boulder sills situated one after another create a pool structure of the fish pass, which is very suitable for creation of fish rest areas (refuge possibilities) and also it is possible to achieve continual flow of the stream by suitable configuration of the boulder sills.

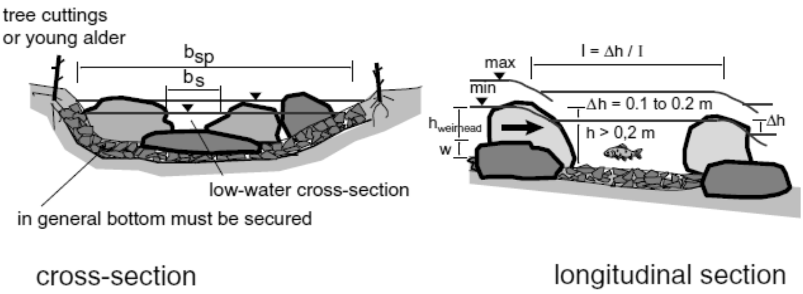

Fig. 5. Boulder sills [4]

Verification is the last step. The designed fish pass with all elements (slope changes, cross section sills with slots, boulders, local extension, etc.) has to be modeled with mathematical software. This modeling verifies the achievement of the required parameters, especially the depths and velocities thus to calculate the water level and velocity regime of the fish pass for various discharges in the range of the fish pass operational manual. The most appropriate tool for the modeling of such rugged river bed is two-dimensional software (e. g. River 2D), which allows the circumfluence of the barriers. However, for the verification’s needs, a one-dimensional model (e. g. HEC-RAS) is faster and more available.

The U. S. Army Corps of Engineers´ River Analysis System (HEC-RAS) is software that allows you to perform one-dimensional steady and unsteady flow river hydraulics calculations, sediment transport-mobile bed modelling and water temperature analysis. Steady Flow Water Surface component of the modelling system is intended for calculating water surface profiles for steady gradually varied flow. The steady flow component is capable of modelling subcritical, supercritical and mixed flow regime water surface profiles. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning´s equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The momentum equation is utilized in situations where the water surface profile is rapidly varied. These situations include mixed flow regime calculations (i. e., hydraulic jump), hydraulics of bridges and evaluating profiles at river confluences (stream junctions). The effects of various obstructions (bridges, culverts, dams, weirs, etc.) may be considered in the computations [5].

River2D (Depth Averaged Model) solves the basic mass conservation equation and two (horizontal) components of momentum conservation. Outputs from the model are two (horizontal) velocity components and a depth at each point or node. Velocity distributions in the vertical are assumed to be uniform and pressure distributions are assumed to be hydrostatic. Important three-dimensional effects, such as secondary flows in curved channels, are not included. The River2D model is a two-dimensional, depth averaged hydrodynamic and fish habitat model developed specifically for use in natural streams and rivers. It is a Finite Element model, based on a conservative Petrov-Galerkin upwinding formulation. It features subcritical-supercritical and wet-dry area solution capabilities. Ice covers with variable thickness and discontinuous ice covers can be modelled. It is intended for use on natural streams and rivers and has special features for accommodating supercritical /subcritical flow transitions, ice covers, and variable wetted area. It is basically a transient model but provides for an accelerated convergence to steady-state conditions. The fish habitat module is based on the PHABSIM weighted usable area approach, adapted for a triangular irregular network geometrical description. The hydrodynamic component of the River2D model is based on the two-dimensional, depth averaged St. Venant Equations expressed in conservative form. These three equations represent the conservation of water mass and of the two components of the momentum vector. The dependent variables actually solved for are the depth and discharge intensities in the two respective coordinate directions [6].

3. RESULTS AND SIGNIFICANCES

Based on the above described steps the graphical methodology was created (Fig. 6.). It is useful tool for the designers of the water structures because they do not have such knowledge about fish species and their behaviour. Following single steps of the methodology will lead to the successful fish pass design. The design procedure is iterative, it means that the designed fish pass is modified after simulation until the required parameters (especially the depth and the velocity) are achieved. Of course every water structure is unique, so the design parameters are different and especially the realization conditions. Therefore every fish pass design requires its own individual approach, with the consideration of the recommendations which are involved in this procedure.

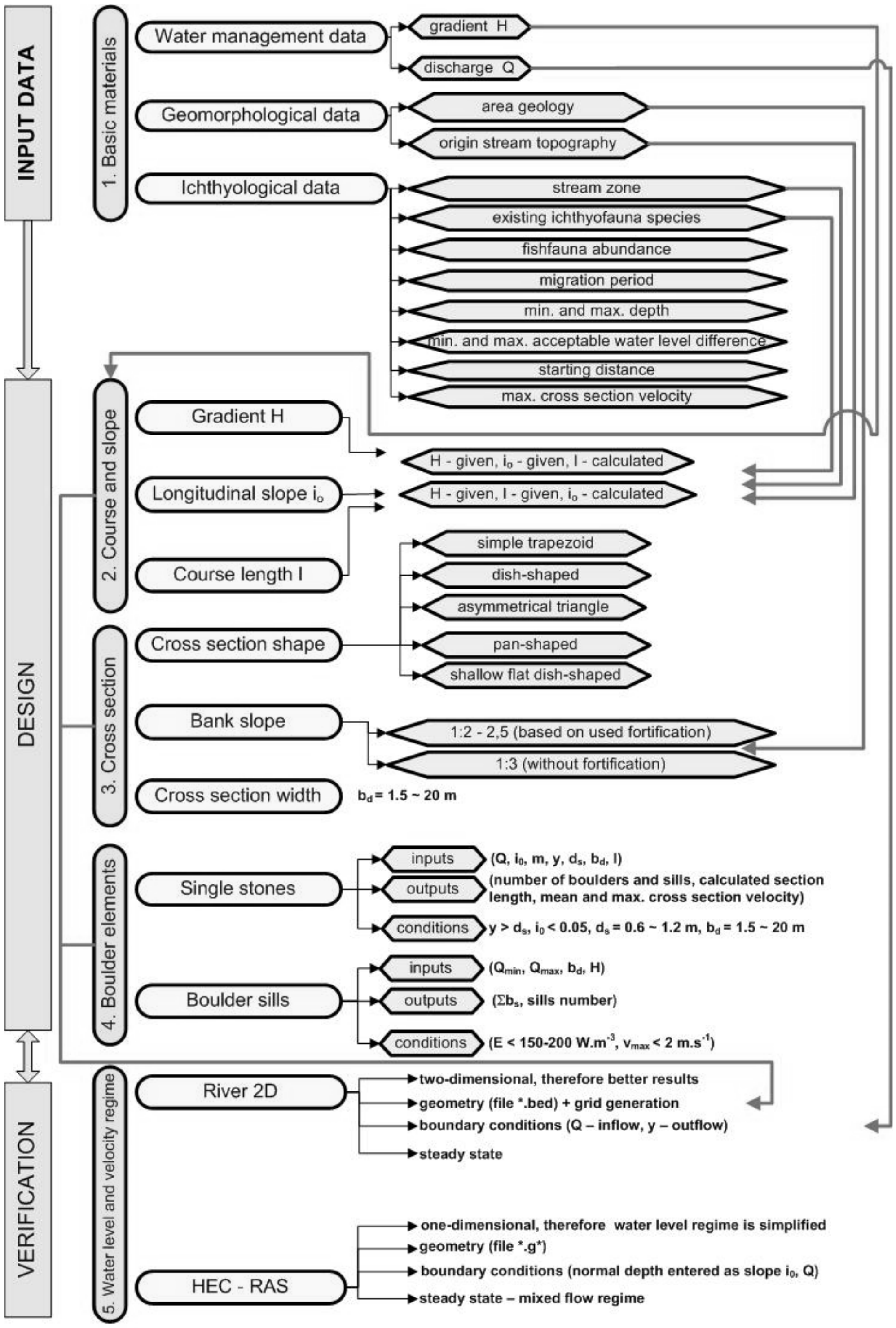

Fig. 6. Graphical methodology for the fish passes design [1]

4. CONCLUSIONS

The summary of described knowledge and information leads to a logical system, which is showed on the scheme of the fish pass design methodology on the water structures (Fig. 6). The data create direct inputs for the design. After the first suggestion of such a fish pass, the verification follows. If the boundary requirements (input data) are not fulfilled, it is necessary to return back to the design module. A successful fish pass design is achieved by the keeping of the order of the single methodology steps.

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A Relationship for Calculation of Waves Hydraulic Shock (Water Hammer) Speed, Closer to the Reality Phenomenon

I. Omer and M. Florea

***Abstract* –The presence of air in free (dissolved) state in the system has a major influence on the variations of pressure in a hydraulic pressure system where the hydraulic shock develops. This air changes essentially the water-air mixture elasticity and hence the rapidity and amplitude variations of pressure waves. The paper presents the results of experimental research on the variation of density of water-air mixture in the volume of air and the pressure in the system. We determined a relationship for the calculation of wave speed, taking into account these elements and presented the results of comparative calculations using current calculation model and that of the proposed model. The proposed calculation removes a part of currently accepted approximations and brings calculation results closer to the reality of the phenomenon. The use of computer makes it possible to envisage wave speed as a variable parameter and recalculate the value each step of the calculation speed.**

***Keywords* – waves hydraulic shock (water hammer) phenomenon, pressure hydraulic system.**

1. Introduction

Given the importance of the hydraulic shock phenomenon (water hammer) that appears in pressure systems, there is an extensive bibliography relating to this subject.

In Romania, there are important achievements in the study of this phenomenon, particularly collective translation of the university centers such as Bucharest, Timisoara, Iasi and Constanta.

The first phase of this paper refers to the influence of free air, which is always found to a greater or lower extent in the water flowing in a pressure pipe from a water supply system.

The presence of free air in hydraulic pressure is inevitable and is due to:

- disengagement of air dissolved in water, when the system pressure decreases.
- incomplete elimination of air during the operation of water filling the plant, especially when the longitudinal profile of the pipe has slope change points.
- vacuum air vortex that may occur due to the river intake pumping station or due to the leaks in the suction pipe.
- the operation of air valves, which ensures the introduction of air in the facility during the transient phenomena in order to prevent cavitations.
- accidental or deliberate intrusion of air in the pipes due to depletion of water volume of the surge tanks or air chamber, used as means of protection.

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The free air (undissolved) in the hydraulic pressure systems may have both beneficial and adverse effects on the evolution of transitional phenomena (especially when it accumulates as bags).

This air changes the propagation velocity of the waves that appear in the phenomenon of hydraulic shock in water-air two-phase mixture, as well as it changes the conditions of cavitations, the maximum and minimum pressures, and generally it changes the development of a transitional phenomenon, as opposed to the case when the water contains no free air. Since the compressibility of air is much higher than that of water (atmospheric pressure), then the lower the pressure variation generated by a valve handle is, the higher the air content is.

Given Jukovski's formula which expresses the factors on which the pressure variation of the valve depends when a manoeuvre is performed in an infinitely small time, it is proved, shown that:

- the air has little influence on the flow velocity of water and on the density of liquid, that is why in the usual engineering calculations this influence is neglected;
- the speed (the propagation velocity of pressure changes) has an influence that should not be ignored because the water-air two-phase mixture has a considerably higher compressibility than water without air, thus resulting a significant reduction of speed.

Given the complexity of the phenomenon, for a model of computation it is necessary to use simplifying assumptions in addition to cases in which dissolved air is neglected:

a) the percentage of free air, noted α and expressed as a ratio between the volume of air and of liquid-gas two-phase mixture, has small values $\alpha \leq 6\%$ typical). Since for a given amount of gas, its volume varies with pressure, the volume of gas considered in this report corresponds to normal atmospheric pressure.

b) free air is dispersed into very small fractions and distributed almost evenly in the liquid. This means that the air in the liquid, considered in small percentages, is not affected by the homogeneity and isotropy properties of the liquid.

c) during the development of the phenomenon, the mass of liquid and free air mass remains constant, in other words no changes of state liquid gas are generated.

d) the variations of temperature that might occur in the phenomenon of hydraulic shock. are neglected

2. INFLUENCE OF FREE AIR AND PRESSURE ON THE DENSITY VARIATION

The experimental determination of the density variation and of two-phase mixture of water and water - air, depending on pressure and air content, was achieved by means of laboratory facilities designed by the research team members.

The principle of the determination method is shown schematically in Fig. 1. The laboratory facility is shown in Fig. 2 (photo 1 ÷ 2).

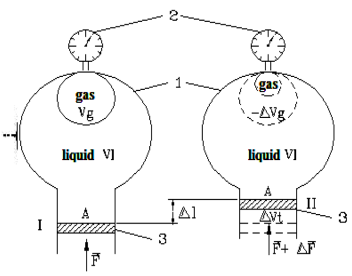

Fig. 1. Schematic diagram of experimental facility: 1 - container un-deforming; 2 - pressure measurement system; 3 - A cross-section sealed piston

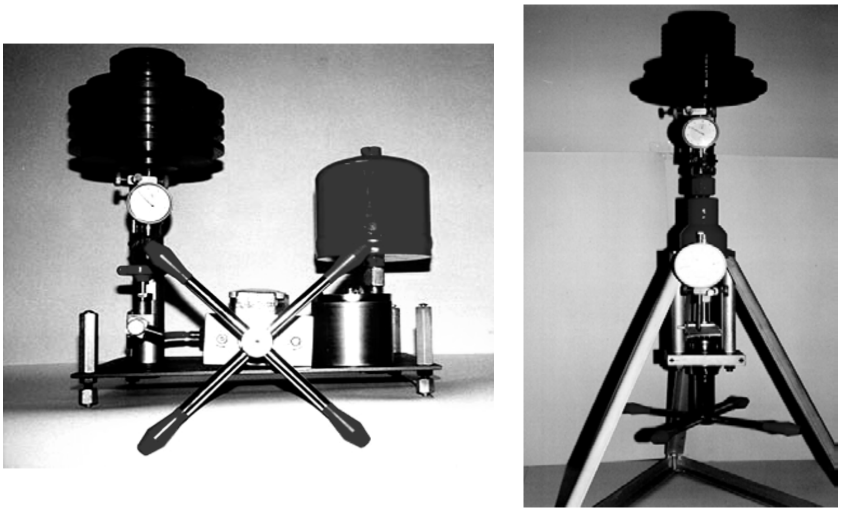

Fig. 2. Pilot plant

The procedure description is:

- the piston (3) (Fig. 1) provides a pressure rise in the container with Δp. Variation of pressure should be measured with great precision. We measured accurately the piston stroke Δl made to increase the pressure. The total change of the container volume can thus be determined due to pressure changes.
- different levels of pressure p_i are achieved through a plate piston cylinder assembly mounted on top of the plant. Weights of different values can be placed on the piston pad, precisely determined, which ensures the indicated accuracy of pressure measurements. The position of the piston is spotted using a micrometer callipers with dial indicator with a precision of 0.01 mm.
- At the bottom of the plant there is another cylinder-piston assembly, which determines the changes in volume ΔV_t, corresponding to different pressures p_i.

Movements of the piston from the initial position Δl_i are measured using another micrometer calliper with dial indicator with a precision of 0.01 mm. The cross-section of the second piston is precisely determined. Lower piston moves by means of a screw mechanism. To make the experimental measurements, the used liquid is distilled water and the air as gas. Measurements were carried out to determine the variation in density with no air pressure for both water without un-dissolved air and water-air two-phase mixture, for different percentages of un-dissolved air, varying between 1%, 2%, 4% and 6%.

It appears that if the water doesn't contain free air (un-dissolved), the density of water has a little, a minor increase with the increase of pressure (up to 4.5 ‰ for a pressure rise of 100 bars).
If the water contains un-dissolved air (max 6% by volume), the change of density by pressure becomes important (relative variation increases up to ten times).

The density increases more pronounced by pressure in low pressure domain and this increase is reduced at high pressure and aspires asymptotic towards the value of density from the case of dissolved air absence.

Table 1. The experimental values for the variation of water density

p (bars)	**ρ (kg/m³)**	**ρ_{am1} (kg/m³)**	**ρ_{am2} (kg/m³)**	**ρ_{am4} (kg/m³)**	**ρ_{am6} (kg/m³)**
2.53	1000.08	995.98	991.66	983.10	963.59
3.56	1000.13	997.04	993.49	986.85	972.13
4.58	1000.18	997.65	994.50	988.88	976.94
6.62	1000.28	998.38	995.69	991.17	982.20
7.65	1000.33	998.64	996.09	991.91	983.83
8.67	1000.38	998.85	996.42	992.50	985.11
9.69	1000.43	999.03	996.69	992.99	986.13
10.71	1000.48	999.18	996.93	993.40	986.98
11.73	1000.53	999.32	997.13	993.75	987.69
13.27	1000.61	999.51	997.40	994.19	988.54
15.31	1000.71	999.72	997.69	994.67	989.42
17.36	1000.81	999.90	997.94	995.06	990.12
19.40	1000.91	1000.06	998.16	995.40	990.70
21.45	1001.01	1000.21	998.36	995.69	991.18
24.51	1001.16	1000.42	998.62	996.05	991.76
28.60	1001.36	1000.67	998.92	996.46	992.37
32.69	1001.56	1000.90	999.19	996.81	992.87
36.78	1001.76	1001.11	999.44	997.11	993.28
40.87	1001.96	1001.32	999.67	997.39	993.64

3. MATHEMATICAL MODEL

The mathematical model which will be presented below is based on the principle of mass conservation and Jukovski's formula $\Delta p = -\rho c \Delta v$ that determines the relationship between the variation of water flow velocity in a pipe and the resulting pressure variation.

$$c_i = \frac{\Delta p_i}{\rho_{i-1} c_i \left(\frac{\Delta \rho_i}{\rho_{i-1}} + \frac{\Delta A_i}{A_{i-1}} \right)} \rightarrow c_i = \sqrt{\frac{\Delta p_i}{\rho_{i-1} \left(\frac{\Delta \rho_i}{\rho_{i-1}} + \frac{\Delta A_i}{A_{i-1}} \right)}} \tag{1}$$

Since this calculation was performed on the steps of pressure variation, the variations at the time "i" were determined by the values of the time "i-1".

We further seek to express the relative variations $\frac{\Delta\rho_i}{\rho_{i-1}}$ and $\frac{\Delta A_i}{A_{i-1}}$ considering the variation of pressure which generates such variations.

For the relative change $\frac{\Delta\rho_i}{\rho_{i-1}}$ a second degree polynomial correlation was considered:

$$\frac{\Delta\rho_i}{\rho_{i-1}} = a\left(\frac{\Delta p_i}{p_{i-1}}\right)^2 + b\frac{\Delta p_i}{p_{i-1}} + c \tag{2}$$

The free term is zero, because it is obvious that a zero pressure variation leads to a null density variation.

Based on the results of [8], by the method of least squares, we obtained the values of a and b constants for different percentages of air (table 2):

Table 2. Values of a and b constants for different percentages of air

α	a	b
0,01	0,021	$5,0968\cdot10^{-4}$
0,02	0,044	$-6,926\cdot10^{-4}$
0,04	0,099	$-3,757\cdot10^{-3}$
0,06	0,22	$-9,892\cdot10^{-3}$

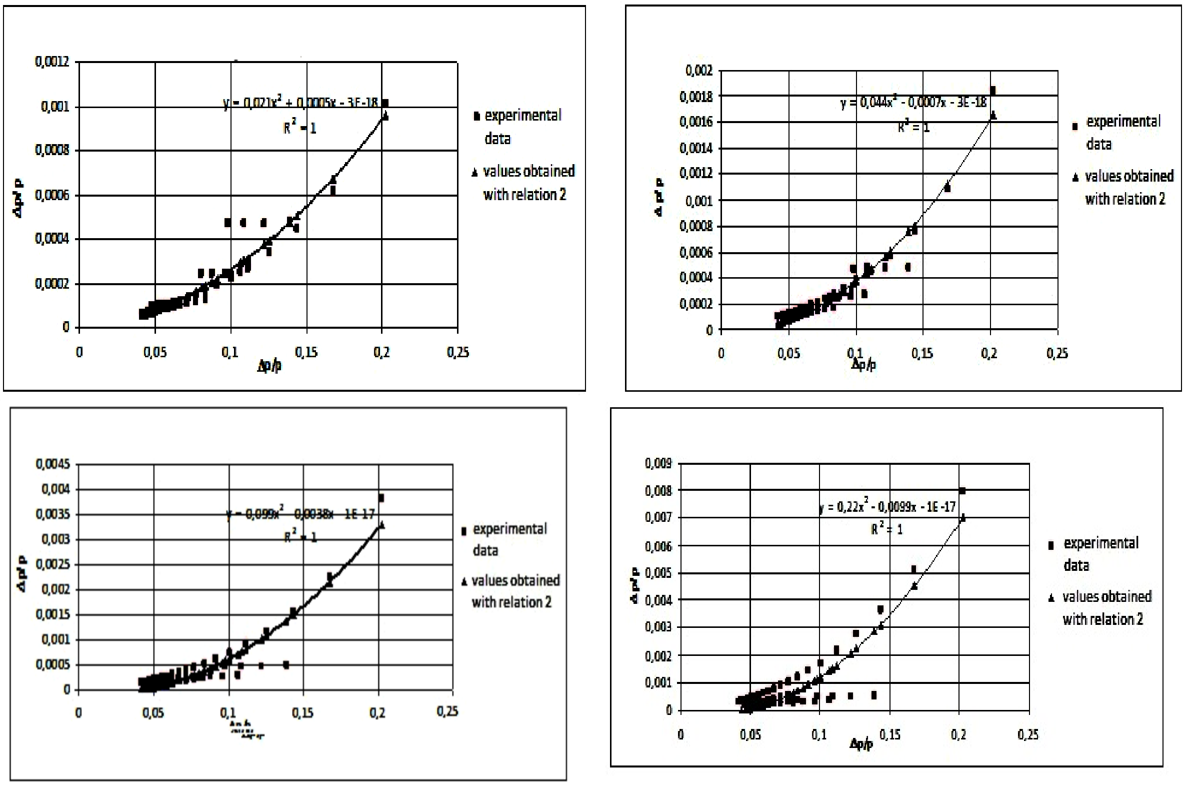

Fig. 3. Variation of Δp/p function of Δρ/ρ

For relative variation of cross section $\frac{\Delta \mathrm{A_i}}{\mathrm{A_{i-1}}}$, applying the usual formula in the specialized literature, we obtained the formula:

$$\frac{\Delta A_i}{A_{i-1}} = \frac{D_{i-1}\Delta p_i}{\delta E_c} \tag{3}$$

In addition to the classical theory, we considered this relative variation of pressure and the variation of the diameter.

It is known that:

$$\frac{\Delta A_i}{A_{i-1}} = 2\frac{\Delta D_i}{D_{i-1}} = \frac{D_{i-1}\Delta p_i}{\delta E_c} \tag{4}$$

Hence $\Delta D_i = \frac{D_{i-1}^2 \Delta p_i}{2\delta E_c}$ (5)

Knowing the diameter variation, we can calculate the diameter to the next step:

$$D_i = D_{i-1} + \Delta D_i \tag{6}$$

Introducing the obtained results in the formula for the relative variations of density and section we obtained:

$$c_i = \sqrt{\frac{\frac{p_{i-1}}{\rho_{i-1}}}{a\frac{\Delta p_i}{p_{i-1}} + b\frac{D_{i-1}p_{i-1}}{\delta Ec}}} \tag{7}$$

Based on these values of speed, a logarithmic correlation was proposed to allow the speed calculation depending on pressure:

$$c = m\ln\left(\frac{p}{p_{at}}\right) - n \tag{8}$$

It was found that the parameters m and n depend on the air percentage, the numerical results being presented in Table 3.

Table 3 The parameters m and n function of the air percentage

α	**m**	**n**
0,01	309,33	230,54
0,02	287,45	197,17
0,04	255,43	142,06
0,06	210,31	73,032

Further correlations m (α) and n (α) were determined:

$$m(\alpha) = -1934{,}8 \cdot \alpha + 328{,}51$$
$$n(\alpha) = -3103{,}7 \cdot \alpha + 261{,}57 \tag{9}$$

Thus the correlation for the prompt becomes:

$$c = (-1934{,}8 \cdot \alpha + 328{,}51)\ln\left(\frac{p}{p_{at}}\right) + 3103{,}7 \cdot \alpha - 261{,}57 \qquad (10)$$

In formula (10) we have considered the report $\frac{D}{\delta}$=200.

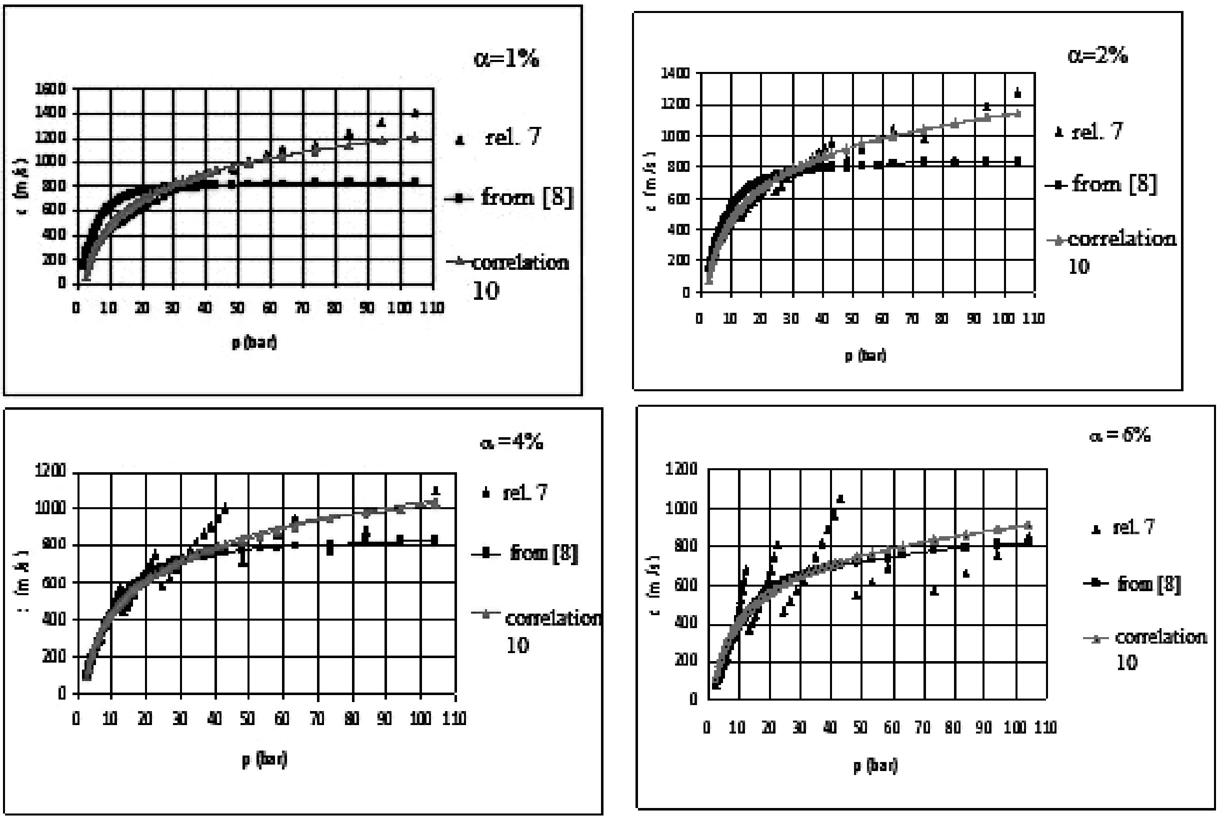

Fig. 4. Values of propagation speed in relation to pressure

4. CONCLUSIONS

Based on experimental results contained in PhD theses dealing with the influence of air in pressure pipes on the evolution of the phenomenon of hydraulic shock (water hammer), we obtained a new structure for the calculus of waves propagation speed depending on the pressure in the pipeline and the percentage of free air. We also obtained a logarithmic formula of waves propagation speed for steel pipeline and for a given value of the report $\frac{\delta}{D}$.

NOTATIONS

ρ- density of water, ρ_{am} - density of water-air two-phase mixture, c – propagation speed of waves, D –diameter of pipe, δ - thickness of pipeline, A – cross-section area of pipeline, E_c – elastic modulus of pipeline, α - percentages of free air in water, a, b, m, n – coefficients, p_{at} – air pressure, Δp - variation of pressure, $\Delta \rho$ - variation of density, ΔA - variation of cross section area, ΔD - variation of diameter.

5. ACKNOWLEDGMENTS

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Investigation on the Hydraulic Shock Response of a Modernized Pumping Station

C. Şt. Niţescu and G. Iordache

Abstract - **The continuous decrease of the irrigated surface in Dobroudja, Romania turned the old pumping stations of high discharge, into oversized, inefficient systems. The rehabilitation and modernization of old pumping stations aim a good compliance of the water discharge with the stakeholders' new demand. The paper depicts a variant for the rehabilitation of an irrigation water supply pumping station of 46,1 m3/s discharge at 55m total head. Pumps replacement has to be done as to meet the small discharge values requested especially in the beginning and in the end of watering season. The adjustment of the old structures involves new extreme pressure values in the discharge duct during hydraulic shock. The paper depicts the way the best variant for the new configuration of the pumping system is chosen on the basis of a hydraulic analysis of either steady or unsteady water flow. The improvement of the discharge duct response to the hydraulic shock is investigated by numerical simulation and the most appropriate means of protection are chosen.**

Keywords – **Hydraulic pressure system, irrigation water supply, pumping station, water hammer.**

1. INTRODUCTION

The existing irrigation system in Dobroudja County, Romania, has been built to deliver water to a large surface of agricultural land. One of the most representative examples is Sinoe pumping station (PS) which was conceived, three decades ago, to supply water to an area of 60,000ha. This PS delivers a discharge of 46,1 m3/s at 55m total head.

The pumping installation consists of 8 vertically mounted pumps, each one delivering a discharge of 5.76 m3/s. Despite the dry climate in the region, with an annual average precipitation 200mm, the water demand for irrigation declined very much lately, due to the high cost of pumped water. Besides, the irrigated agricultural area decreased very much. As a consequence, the capacity of one single pump is much greater than the water demand in the beginning of the watering season. Therefore, Sinoe PS which is equipped with constant speed pumps cannot adapt the discharge to low water demands.

There are many variants for the modernization of the PS that can be conceived, but the choice has to be made on the basis of a sound hydraulic analysis.

The aim of our study is to establish an optimum variant for the protection from water hammer of Sinoe PS. It is known that the modernization solution has to meet a series of requirements: a good compliance between the pumping system and the water demand, a high efficiency and a low energetic consumption and reliability during hydraulic shock [1].

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2. COMPARATIVE DESCRIPTION OF THE OLD PUMPING STATION AND THE PROPOSED ONE

Sinoe PS takes water from the Golovita Lake, part of the Razim complex, and delivers it into a channel of the Sinoe irrigation system. The PS has 8 identical pumps which are connected in 2 groups of 4 pumps each, operating in parallel. Each of the two groups has a discharge duct of 800m in length and 2.8m in diameter. Experimental investigations on the pumps operation showed a decline of the flow rate and of the efficiency for all the pumps in a group. Furthermore, one of the pumps in a group has been broken.

The main features of the existing pumping station are given in **Table 1**.

Tab. 1 Duty points for the existing pumping station [2]

Pumps	Head	Discharge	Power	Efficiency
[-]	[m]	[m^3/h]	[kw]	[%]
P1	51.90	16,132	3,920	58.2
P2	50.87	18,468	4,025	63.6
P4	52.90	16,664	3,960	60.6
P1+P2	52.90	32,814	7,586	62.4
P2+P3	53.70	33,196	7,850	61.9
P1+P3	52.90	30,744	7,465	59.5
P1+P2+P3	55.29	47,214	11,370	62.6

The data in the **Table 1** indicate the smallest discharge of 16,132 m^3/h delivered with an efficiency of only 58.2%. The efficiency is low for all the duty points. Therefore, the replacement of all the four pumps in one group was considered. The variant we propose for the old pumps replacement offers more possibilities to adjust the discharge in accordance with the stakeholders' water demand. It comprises the following hydraulic pumps:

- One high discharge pump (HDP), of 20,250 m^3/h at 51.20m head;
- Three identical low discharge pumps (LDP), of 4,068 m^3/h at 50.20m head.

Along with the replacement of the pumps, the replacement of the old metal made discharge duct with a new one of 1.8m in diameter, made of reinforced polyester is considered. It will consistently improve the operation of the new hydraulic system by decreasing the friction coefficient of the conduit [1],[3].

Tab. 2 Duty points for the proposed pumping station

Pumps	Head	Discharge	Power	Efficiency
[-]	[m]	[m^3/h]	[kw]	[%]
1HDP	51.21	20,250	3,400	87.5
1LDP	50.25	4,068	720	77.5
1HDP+1LDP	51.60	23,760	4,035	85.4
1HDP+2 LDP	52.00	27,000	4,710	84
1HDP+3 LDP	52.50	30,528	5,395	82.2
2 LDP	50.37	8,136	1,442	77.4
3 LDP	50.56	12,132	2,145	77.7

Despite the constant operation speed of the new pumps, this combination would cover a wider range of flow rate values. The hydraulic analysis, conducted for the steady flow of water throughout the pumping installation, gives the main duty points in the case of individual or parallel operation of the pumps in a group [1]. These duty points can be seen in **Table 2**.

3. INVESTIGATIONS ON THE NEW PUMPING STATION RESPONSE TO A HYDRAULIC SHOCK

The replacement of the pumps changed the hydraulic parameters of the pumping system, therefore an investigation of the PS response to hydraulic shock variation is compulsory.

Numerical simulation is an easy and reliable way to determine extreme pressure values during hydraulic shock, to find out the most vulnerable cross sections of the conduit and also to choose the most effective protection means. The numerical simulation was carried out assuming power failure and sudden stop of the running pumps in all the cases the pumping system may operate. Investigation on pressure oscillation during the pumps start was also carried out.

The computer programme used for simulation is a non commercial one, developed to solve the equation system that governs the hydraulic shock by the use of characteristic method and the finite differences technique, depicted in detail in [3], [4], [5]. The most important assumptions this programme considers are: water is a single liquid phase, water is a barotropic, compressible liquid, water movement is one-dimensional, friction losses may be calculated with the same formula either the regime is steady or unsteady [3], [4].

The new discharge duct is made of reinforced polyester and the diameter is reduced to 1.8m according to the new smaller flow rate.

The investigations were developed considering the installation is unprotected and then, by turn, the installation is protected by different means such as different closing laws of the check valve on the discharge duct (in one or in two stages), surge tank or air chamber mounted on different sections of the conduit.

We will further refer only to the results regarding the closing law as the single means of protection from water hammer.

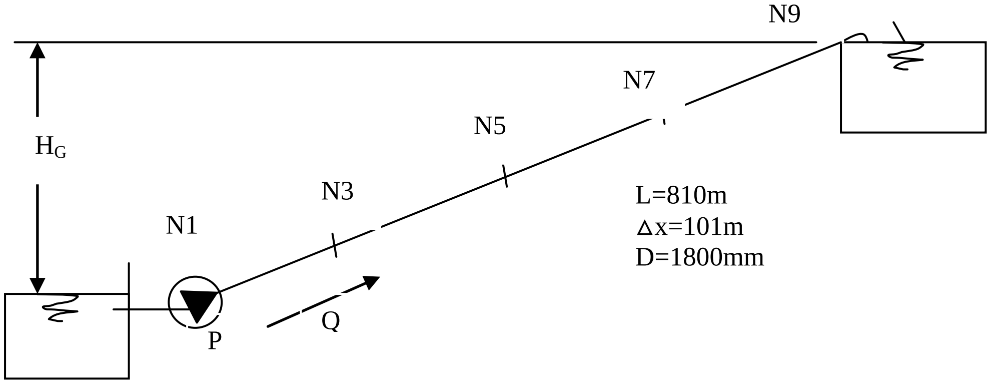

Fig. 1 Layout of Sinoe PS and calculus nodes on the discharge duct for a group of pumps

4. DISCUSSION OF THE RESULTS

We will present and compare below the results of two similar numerical simulations: one was carried out for the existing Sinoe PS and published in [3] and the other carried out for the new Sinoe PS, considering the replaced pumps, but the same layout of the station. The length and the number of sections of the discharge duct are the same. Numerical simulation was developed by the use of the same computer programme.

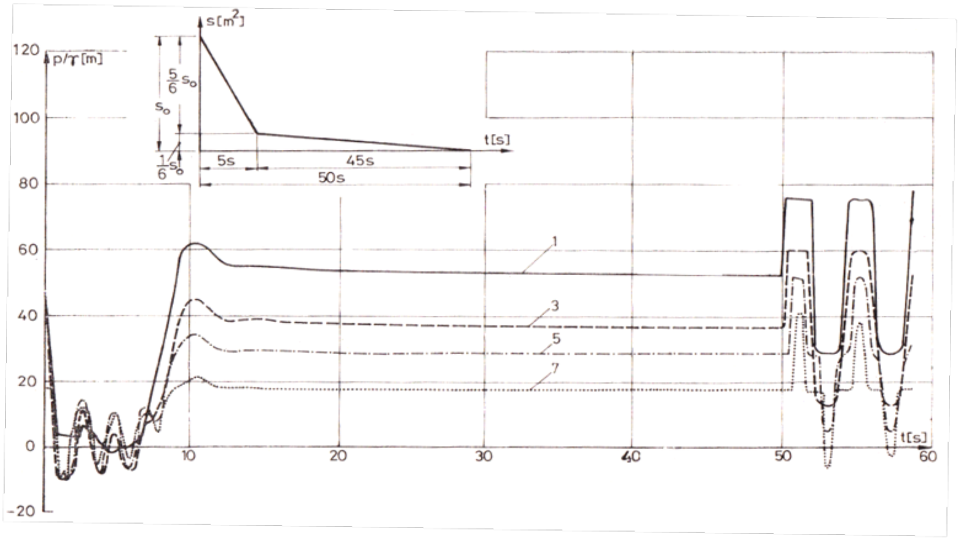

Fig. 2 Pressure oscillation in the discharge duct of the existing Sinoe PS [3]
1, 3, 5, 7 calculation node

The analysis of the existing Sinoe PS response to hydraulic shock is presented in [3] . This analysis was conducted for a series of different closing laws for the check valve and assuming no other protection devices are mounted on the discharge duct. The metal made discharge duct is of 2800mm in diameter and 810m in length. It was divided in 8 sections of approximately 100m each.

Figure 2 shows pressure variation along the discharge duct, considering a two stage closing law for the check valve on the discharge duct, when the first stage consists of 5/6 of the cross section closed during 5 s and total closing in 50s. Dangerous negative pressure values are reached in the first moments of valve closing, along the entire duct. Cavitation occurs. After the total closing of the valve, dangerous negative pressure occurs only in the last sections of the duct. Protection devices in the last part of the discharge duct are mandatory in order to increase inner pressure.

The results regarding pressure oscillation in the new proposed variant of Sinoe PS during water hammer will be presented in the same calculus nodes (node N1-with pump, nodes N3, N5 and N7), so that one may compare the two variants of the PS. The closing law is also in two stages: 5/6 of the cross section closed in 8s and a shorter duration of total closing, of 30s. This law was chosen as the best option. Numerical simulation was carried out for all the operation possibilities of the pumps in the station. Only part of the results are presented below, namely the three cases when power failure sudden occurs when the PS operates at maximal flow rate and at two intermediate flow rates.

In the first case, when all the four pumps are running (1HDP+3LDP) small negative pressure values are obtained and they last for short periods of time. The lower value of pressure is of -3.87mwc and it is recorded in node N3; the other negative pressure values along the duct are higher; pressure in node N7 is still negative -1.44 mwc. We may conclude that cavitation doesn’t occur in any section of the duct and furthermore, negative pressure is not a threaten to the duct.

Maximal pressure value is 65.68 mwc, in node N1, close to the pump. The oscillation frequency is half the frequency in the existing PS variant presented in [3].

In the second case, when three pumps are running (1HDP+2LDP) **Fig. 4**, maximal pressure is 5% lower than in the first case. Minimal pressure values are positive along the whole duct.

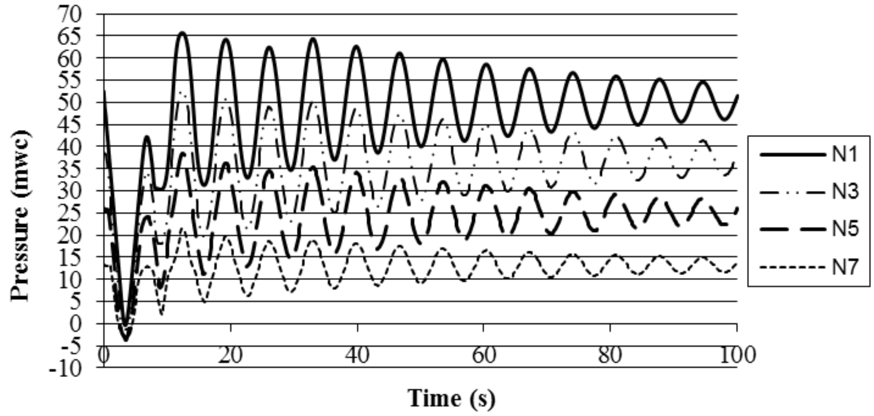

Fig. 3. Pressure oscillation during hydraulic shock.
Case of power failure during the operation 1HDP+3 LDP
N1, N3, N5, N7 calculus nodes

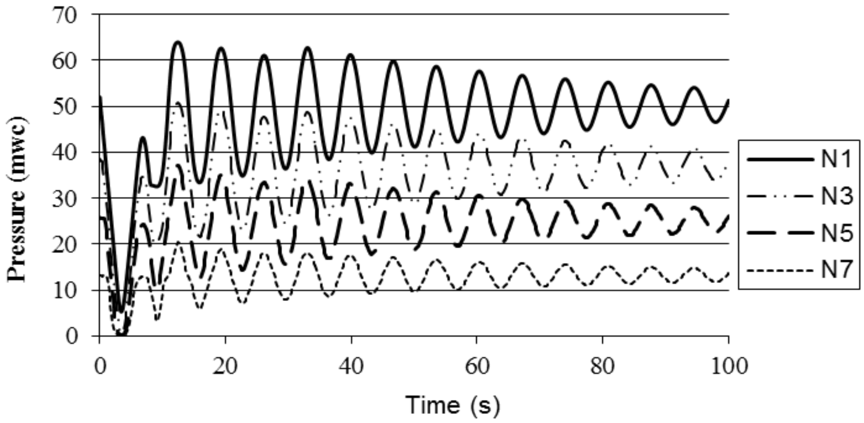

Fig. 4. Pressure oscillation during hydraulic shock.
Case of power failure during the operation 1HDP+2 LDP
N1, N3, N5, N7 calculus nodes

In the third case, when only the HDP is running and suddenly stops, minimal pressures are also positive Fig. 5, ranging between 16.23 mwc in node N1 and 3.42 mwc in node N7, during the valve closing. Small amplitude values and a quick dumping of the pressure oscillation (in about 150 s) may be noticed in this case.

Conducting a comparative analysis between the pressure oscillation during hydraulic shock in the old discharge duct (for a discharge of 23 m^3/s, Fig.2) and in the proposed discharge duct (for a discharge of 8.48 m^3/s, Fig.3), we may conclude that, in the absence of any additional protection device:

-maximal pressure in the proposed variant in node N1 is 13% lower; in the other nodes, maximal pressure value is also reduced with 12,5% up to 48.2%;

-maximal pressure lasts for shorter periods of time (about half the period in the old variant);

-minimal pressure values are higher in the proposed variant: the lower pressure is -3.87mwc instead of cavitation which occurred in the old discharge duct.

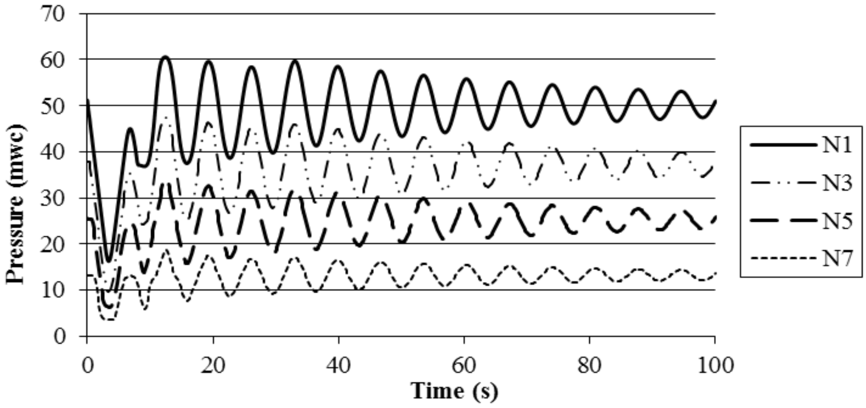

Fig. 5. Pressure oscillation during hydraulic shock.
Case of power failure during the operation 1HDP
N1, N3, N5, N7 calculus nodes

5. CONCLUSIONS

The modernisation of the Sinoe PS involved the replacement of the old pumps with new ones, in an appropriate variant to achieve the actual smaller water demand. These changes in the PS imposed a new study on the system's response to the hydraulic shock.

The study involved numerical simulation of water hammer phenomenon in different cases of protection of the discharge duct. It showed that the two stages closing law of the check valve on the discharge duct (5/6 of the cross section closed after 8s and total closing in 30s) is optimal for the protection of the discharge duct of the pumping group. This closing law may itself ensure the protection of the system from hydraulic shock. There is no need of additional protection devices such as air chamber or surge tank.The old PS needed additional devices.

The decrease of the discharge led to a better compliance of the pumping system with the nowadays water demand, to an improvement of the energetic efficiency of the installation and to a better response of the pumping system to hydraulic shock, in each case of pumps operation.

Numerical simulation is a very reliable tool for the water hammer response analyse. Such an investigation has to be conducted not only for the new designed PS but also for the modernized ones, especially when the new configuration of the hydraulic system involves major changes.

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SECTION VII

HYDROLOGY AND HYDROGEOLOGY

ENVIRONMENT PROTECTION

A Mathematical Model Used to Simulate Floods on The Racu Brook

Daniela Sârbu

Abstract - **Floods are common natural phenomena that cause hazards to man and his activities. Being the most common natural disaster, floods are responsible for the highest economic losses each year, a death count of tens of thousands of people, and affect in different forms and levels of severity the lives of hundreds of thousands of other people. The increasingly destructive recurrence of these phenomena stressed the need for an integrated and sustainable approach to flood risk management in drainage basins, in order to allow reducing the severity of potential damage in the future.**

This paper presents a mathematical model created with the help of the HEC-RAS software, in order to generate flooding limits for different flow rates on the Racu Brook, tributary of the Upper Olt River (Harghita County). These allow the determining of floodable areas by overlapping the borders generated on different maps or orthophotos.

Keywords **- flow, flood, flood maps, mathematical model**

1. INTRODUCTION

Floods are natural phenomena that are part of Earth's hydrological cycle and have the greatest impact on life and on the environment. Over time, floods caused human deaths, destruction of infrastructure, crops, environmental degradation. The past decade marked an unprecedented increase in incidence and magnitude of flood damage, in all its forms of manifestation.

In Romania, dozens of annual floods occur with greater incidence in the Carpathians and Sub-Carpathians medium altitudes and at low plains. Spring floods are caused regularly by melting snow (the accumulation of which is ensured by sub-zero temperatures in winter), completed by spring rains. In early summer, floods occur all throughout the country due to heavy rains.

In order to determine areas at risk of flooding the "Plans for prevention, protection and mitigation of floods" were drafted, pursuant to Government Decision no. 1309/27.10.2005. These plans entail the mapping of floodplains areas with different exceedance probabilities and the development of measures for reducing or stopping the effects thereof. [1]

2. THE MATHEMATICAL MODEL USED TO DETERMINE THE FLOODING LIMITS OF THE ANALYSED AREA

Mathematical modelling of specific hydraulic phenomena consists primarily in the precise rendering of the stream channels geometry on the basis of topographical surveys and of digital terrain model made at the site of the river embankment works, both in the channel and in the overbank area. Subsequent hydraulic calculations aim to determine the current capacity of drainage, of flooding curves for maximum flow with different annual exceedance probabilities, as well as the maximum levels. The mathematical model in this case was performed using the HEC-RAS software.

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HEC-RAS is a product of U.S. Army Corps of Engineers, Hydrologic Engineering Center, one of the most known and used software packages for the analysis of hydrographical systems in the world.

The model can perform the calculation of maximum levels of free surface curve of non-permanent and permanent moving water, in uniform or gradually varied hydraulic state, for natural or improved rivers (according to the works included in the river development schemes or designs) with single thread channel, but also for dendritic and ring type channels. The elevation of free surface of water is computed from one profile to another, solving the energy equation in an iterative routine called the standard step method. The energy equation is written as follows [3-5]:

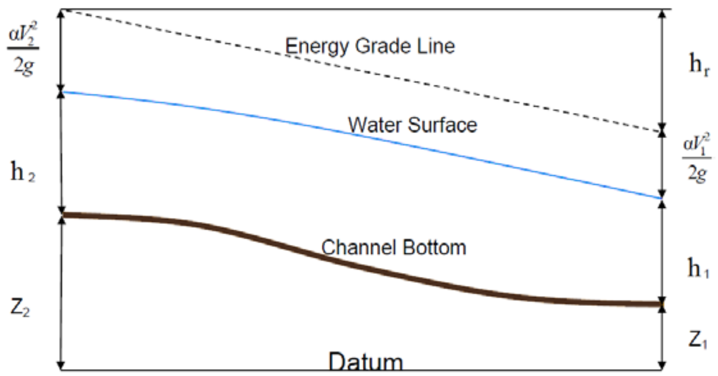

Fig. 1. Cross sectional representation of the energy equation

$$z_2 + h_2 + \frac{\alpha_2 v_2^2}{2g} = z_1 + h_1 + \frac{\alpha_1 v_1^2}{2g} + h_r \tag{1}$$

where:

h_1, h_2 - depth of the water in the downstream and upstream cross section;

z_1, z_2 - bottom elevation;

v_1, v_2 - mean velocity over the cross section;

α_1, α_2 - the Coriolis coefficient;

g - the gravitational acceleration;

h_r - head losses.

The head loss between two sections is composed of actual head loss and loss/recovery of kinetic energy due to the flow contraction or expansion.

The ratio for the total head losses is:

$$h_r = L\,I + c\left|\frac{\alpha_2 v_2^2}{2g} - \frac{\alpha_1 v_1^2}{2g}\right| \tag{2}$$

where: L - weighted sector length;

I - the hydraulic gradient between the two sections;

c - coefficient of loss/recovery through expansion or contraction.

The working method used in HEC-RAS is to divide the discharge in the stream channel using the *n* values shown in the cross section, as a basis for quantization. The flow is calculated for each subdivision (left floodplain, channel and right floodplain) with the following relation from Manning's equation.

$$Q = KI^{1/2}, \quad K = CAR^{1/2}, \quad C = \frac{Q}{A\sqrt{RI}} \tag{3}$$

where: K - flow module;
n - roughness for subdivision;
A - area for subdivision;
R - hydraulic radius for subdivision.

To obtain the flow module for that particular section, the program totals the flow modules for the right and left floodplains and for the channel.

The elevation of free surface of water in a cross section is determined by iteratively solving the energy equation and the head loss ratio.

3. DESCRIPTION OF THE MODELLING PROCESS

In this work we aimed to establish the flooding limits on the Racu Brook, a left tributary of the Olt River. The river basin area of the brook (VIII.1.14) is 126 km^2, located entirely in Harghita County and covers 0.94% of the Upper Olt total basin area.

Racu Brook transits the localities of Siculeni, Racu, Văcăreşti, Mihăileni, Nădejdea and Livezi to which it has caused, over time, great damages during floods. Setting the flooding limits for the Racu Brook was made by mathematical modeling with the HEC-RAS software, and the data were pre-processed and post-processed using Global Mapper. The background data used to determine the levels of free surfaces of water for maximum flow corresponding to different exceedance probabilities are of two types:

- topographical data that refer to: the entire channel and floodplain cross-sections, detailed plans and drawings of the engineering structures (bridges, weirs, culverts etc.), digital terrain model (DTM) and/or site plans, georeferenced imaging;

- hydrological data that refer to: maximum flow values under the current drain, corresponding to the exceedance probabilities of 0.1%, 0.2%, 0.5%, 1%, 5%, 10% and 80% in all sections concerned, information on land use, vegetation coverage and the type of soil in the channel and floodplains [2].

Initially, the topographic data were entered into Global Mapper (**Fig. 2**) where they were processed and exported in .csv format and then imported into HEC-RAS.

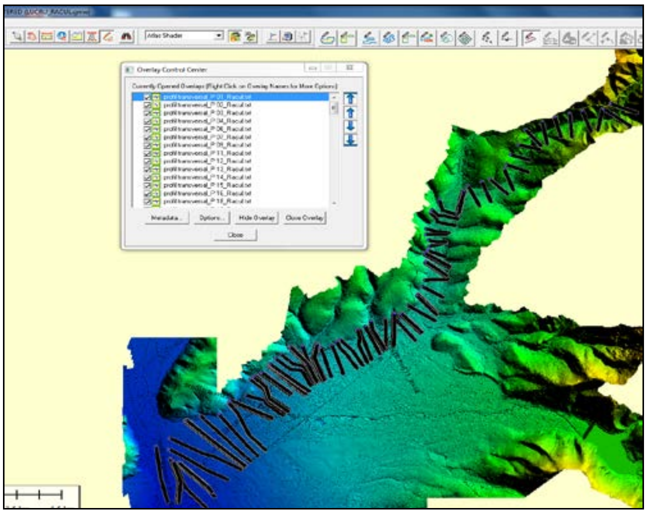

Fig. 2. Data loaded into Global Mapper

Geometric data inserted in the mathematical model developed with HEC-RAS for the Racu Brook flow simulation consisted of 49 sufficiently large cross sections on both sides of the banks, surveys of the hydraulic structures and weirs (bridges and footbridges - 17, weirs - 4) the distance between them, the position of the banks, and head loss coefficients and channel roughness coefficients.

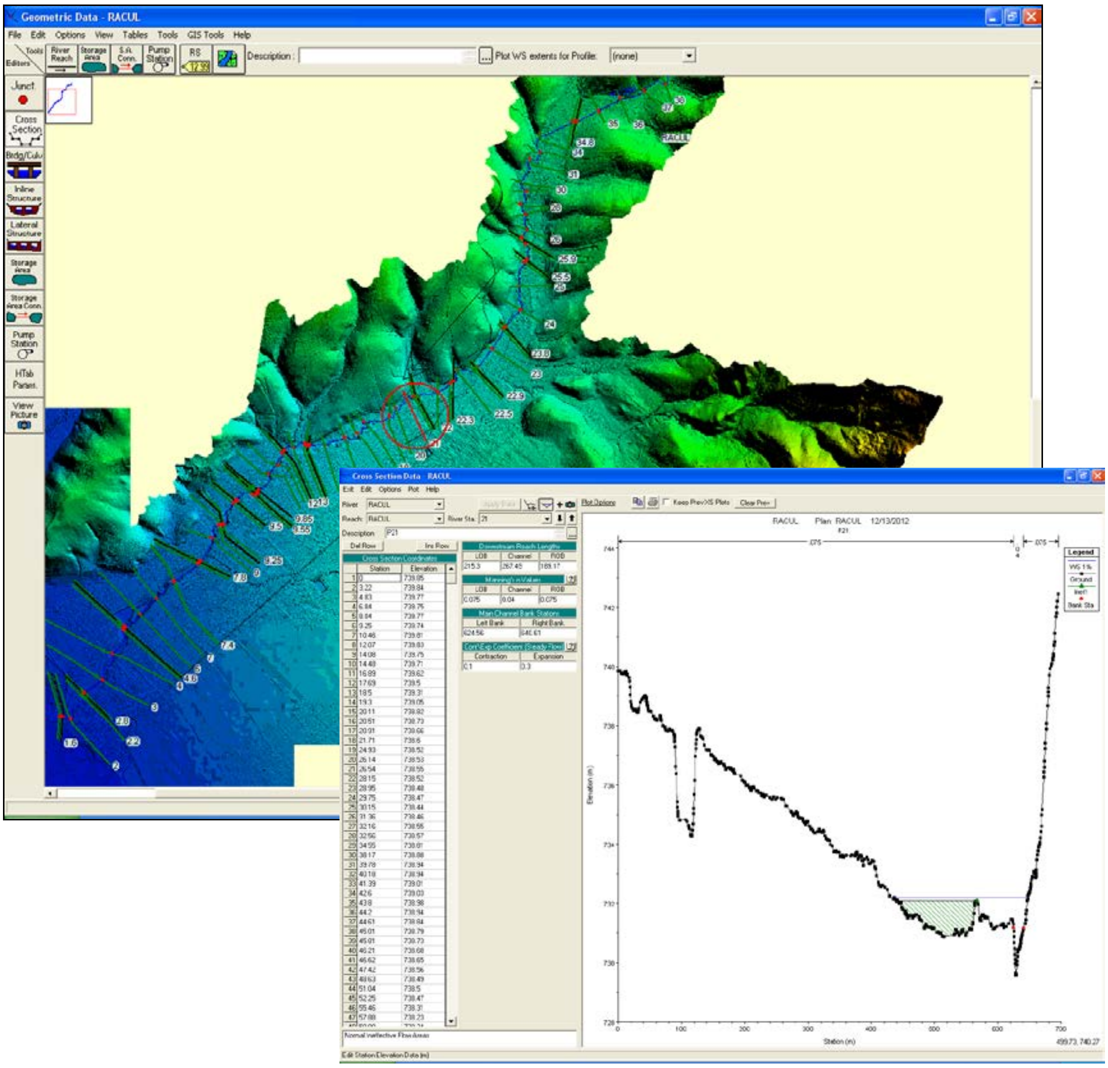

Fig. 3. Geometric data of the terrain (Racu Brook)

Once all the necessary data for the cross sections have been entered, the constructions on the channel course are described (bridges). The effect of bridges on the flow capacity level of the flood is calculated generally in three distinct areas: one immediately downstream of the bridge section where there is an expansion of the flow, one in the actual section of the bridge and one upstream of the bridge, where there is a contraction of the flow before it enters the free section of that structure. As hydrological data, we use the maximum flow capacities for different exceedance probabilities extracted from the hydrological study drown-up and verified by the "National Institute of Hydrology and Water Management". **Figure 4** illustrates the longitudinal profile of the river on which are shown the levels corresponding to the 1% and 0.2% flow capacities, the thalweg line, as well as the perpendicular structures along the channel (bridges, footbridges).

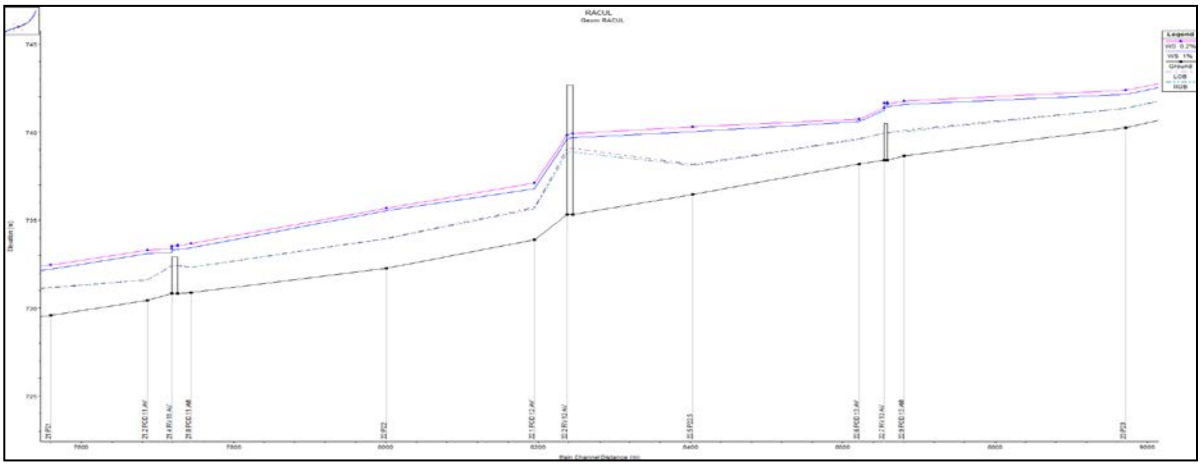

Fig. 4. Longitudinal profile illustrating the levels of 1% and 0.2%, respectively, the thalweg and the hydraulic structures

With the help of RAS Mapper extension, the flooding limits are generated (**Fig. 5**).

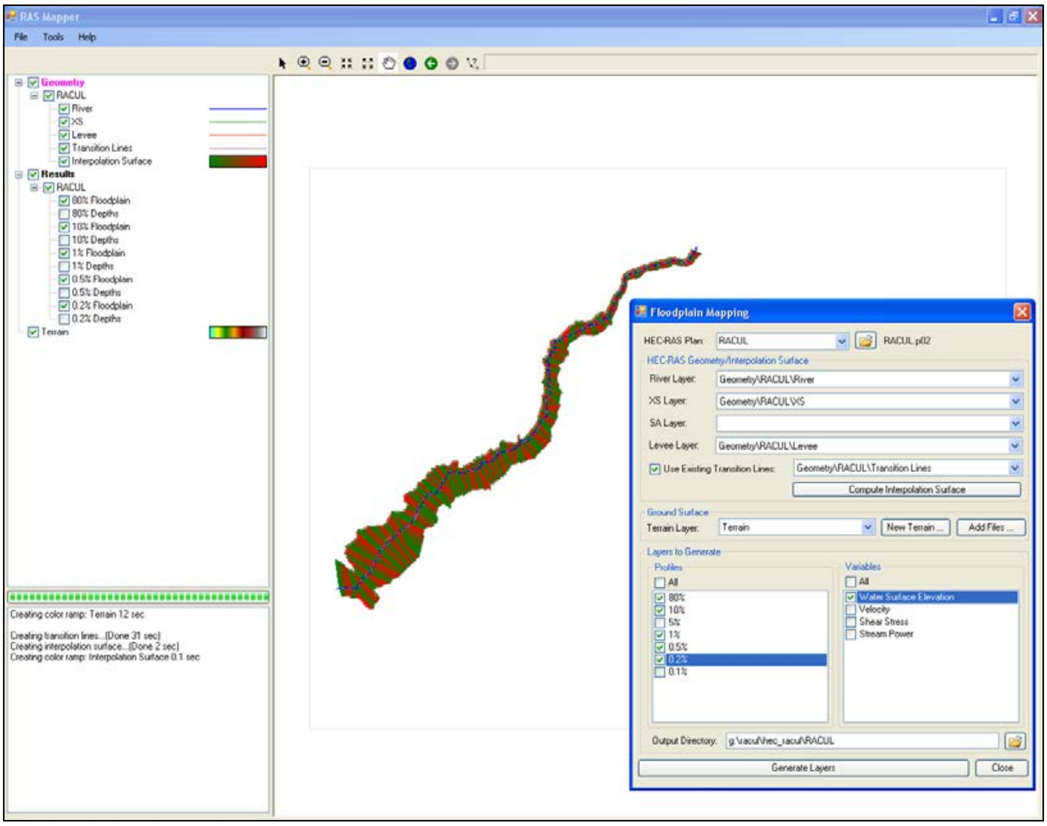

Fig. 5. The generation of limits

4. RESULTS OBTAINED FROM MATHEMATICAL MODELLING

The results consist of: elevation of the free surface of water, average velocity, depths and are they are presented graphically and tabulated.

The mapping of the flooding limits is done by intersecting the plan of the free surface of water with the numerical model of the land. The polygon representing the floodable areas can be saved as a shapefile or other formats and can be overlapped over any plan.

Figure 6 illustrates the flood limits at 1% and 0.2% superimposed on an orthophotomap, opposite of Văcăreşti and Mihăileni localities.

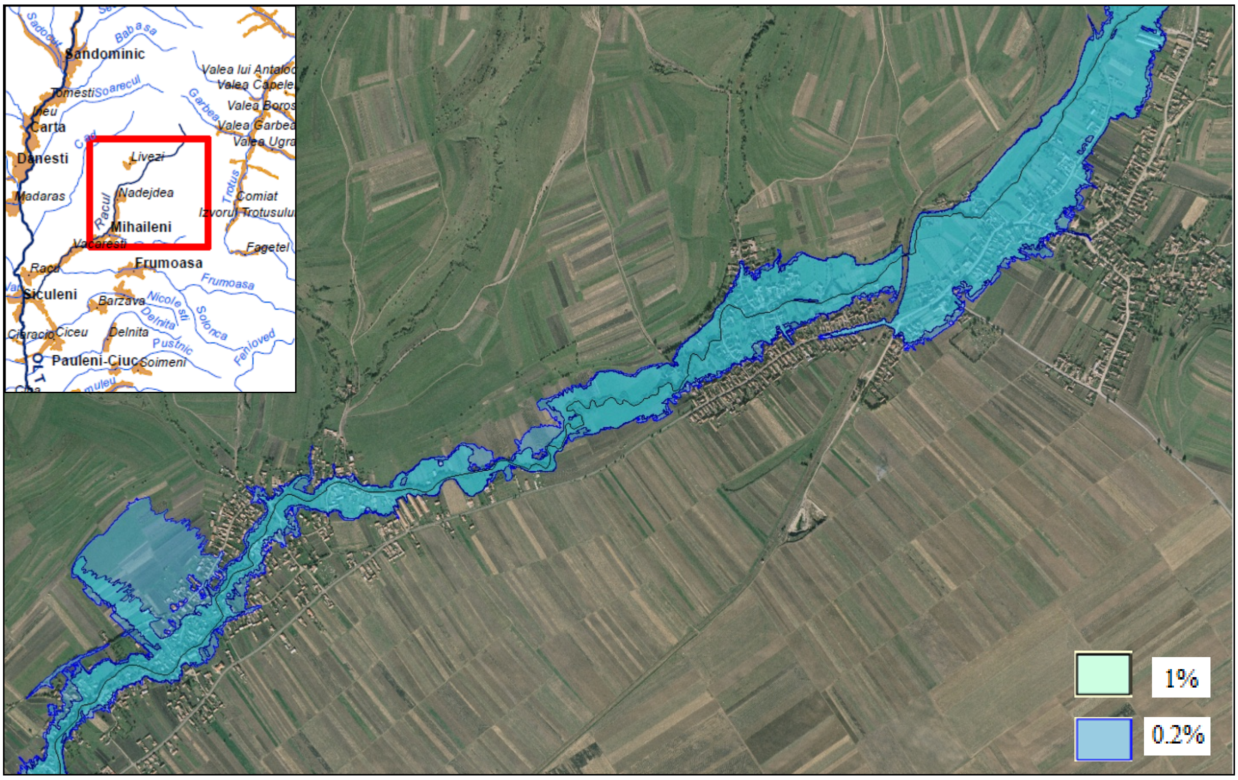

Fig. 6. The flooding limits (1% and 0.2%) in Mihăileni and Văcăreşti

After overlapping the flooding limits, generated by RAS-Mapper and processed in Global Mapper, the orthophotomap shows that both villages located on the river course, as well as an important area of agricultural land, are prone to flooding.

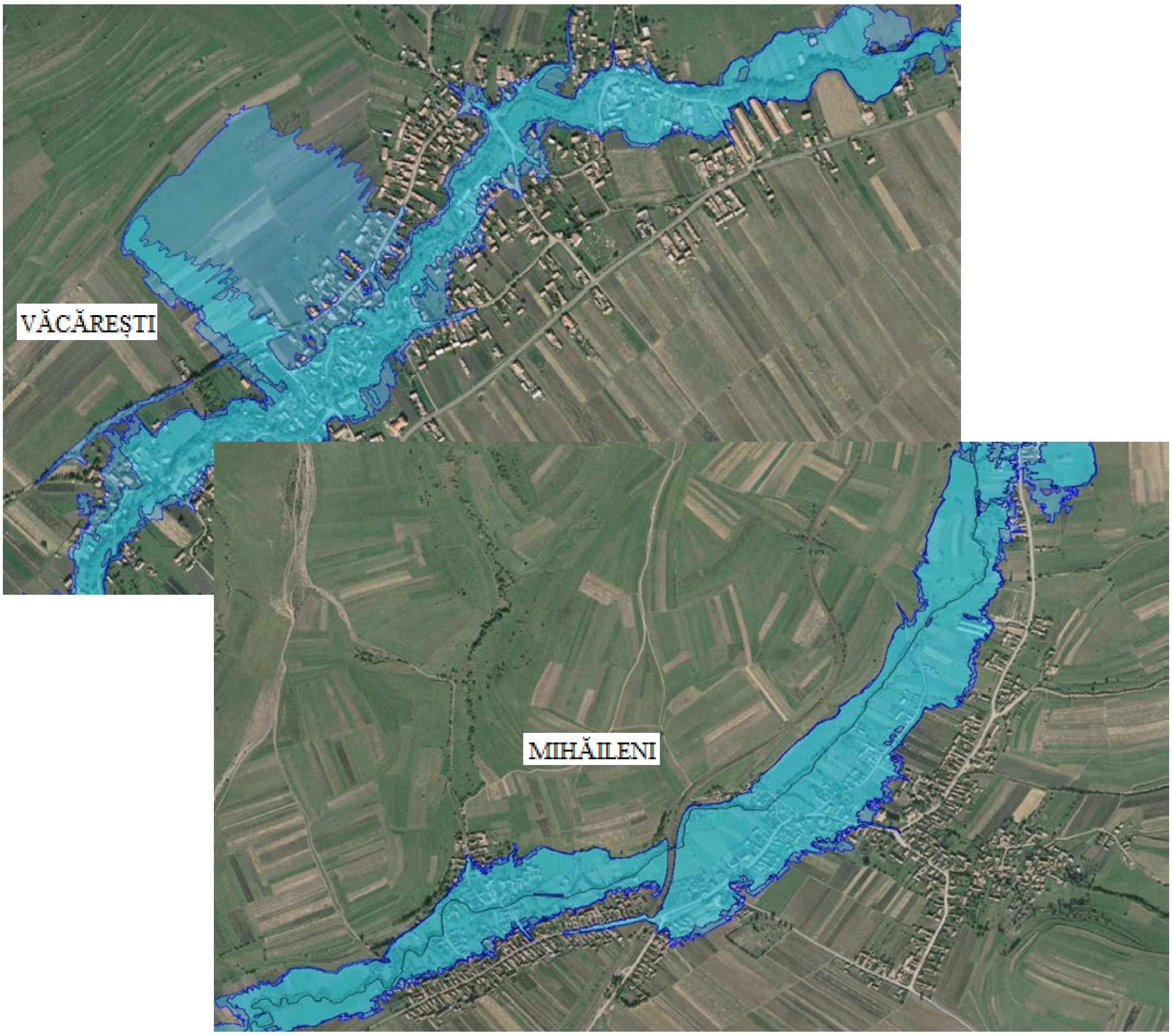

Fig. 7. Detailed view - flooding limits (1% and 0.2%)

Such flooding maps are necessary in order to reduce the risk of natural disasters affecting the population, by implementing preventive measures in the most vulnerable areas and by developing insurance plans, and can be of great interest to contractors, designers and engineers.

4. CONCLUSIONS

The mathematical model developed using the HEC-RAS software was used to generate flooding maps in the area crossed by the Racu Brook, based on which the flood protection system can be improved. It can also be used to alert local authorities and the population in case of floods and may help to set up a program for public information and participation in the decision-making process.

Such maps are generated within the "Plan for prevention, protection and mitigation of floods in the Olt drainage basin" which aims to: identify areas where there is a risk of flooding, flood hazard regionalization, determining vulnerability to flooding in areas with a high flood risk, establishing the flooding causes and describing the anthropogenic factors contributing to the worsening of the flooding phenomenon, estimation of trends in the occurrence of future floods, the presentation of measures and actions needed to reduce the flooding risks, planning various categories of works to protect infrastructure and areas which may be affected by any possible flood.

5. Acknowledgments

The author would like to thank S.C. AQUAPROIECT S.A. for the technical data support, without which mathematical modelling could not be performed and Professor Virgil Petrescu, for academic support.

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Models for the study of hydrochemical processes and radiochemical river and marine

Constantin Borcia

***Abstract* – The paper presents some software and some hydro and radiochemical conditions of their application for the Danube and the Black Sea coastal zone, where conditions are not met, of course that the software will not be applied. The main expected potential impact of applying this software refers to deepen the knowledge of physical processes and ecological modeling these processes in the Danube River and the Romanian coastal zone, and the creation of a fund of data and database documentation. The results will be used in other regional or sectoral studies on the evolution and dynamics of physical-ecological river and maritime.**

***Keywords* – hydro and radiochemical processes, model, software, the Danube, the Black Sea coast.**

1. INTRODUCTION

Hydrochemical processes and radiochemical processes are important since they define the fluvial and marine biotopes and they depend on many characteristics associated biocenosis. It is clear that these processes have caused in part natural, partly artificial. Over the Danube and the Black Sea coastline are several sources that generate such processes both natural and anthropogenic sources. Integrated study covering both defining characteristics and radiochemical hydrochemical and hydrological and morphological characteristics of these processes for the Danube River and the lower sector of the Black Sea coastal zone is important because of the potential risks of chemical and radioactive pollution. To study the behavior of various pollutants, particularly of radionuclides in aquatic and marine waterways, various institutions and companies have developed models and related software. They can be classified into explicit radiochemical models that are specifically designed for radiochemical studies exclusively and models implied or collateral, that are not specially designed for studies of this type, they are models and specific software or hydrological or ecological, but studying processes occurring in the environment that can be adapted radiochemical possibly for radiochemical studies. Some models and software are supplied a fee, others are free. A simplified schematic is shown in Figure 1 [2], [3]. Among the models and software RIVTOX model river and model Explicit SHETRAN - UK and among the models are implied or collateral DESERT, V2SDI, GENSCN, AQEM and PAEQUAN.

2. DESCRIPTION OF THE THEORETICAL MODELS

Fluvial Models

There are a variety of models and software which, in principle, may be classified as follows: 1) models and complex software (integrated) - consist of several interconnected specific modules (hydrologic

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modules / hydraulic modules, chemical, radiochemical, biochemical, etc.) - For example AQUATOX 2-1 DESERT, RIVTOX, SHETRAN-UK, etc. The input and output data from the model are represented by a variety of parameters (hydrologic / hydraulic, climatic, chemical, etc.).

2) models and specialized software - generally consist of one specific module (hydrological, hydraulic, chemical, radiochemical, etc.) - For example BIOCHLORE 1.0 VS2DI, HydroChem, TDPF 1.0 (TopoDrive / ParticleFlow), Smad / GVPROF, etc. The input and output of the model are the specific parameters (either hydrologic / hydraulic or chemical or intended for the study of a particular pollutant, etc.).

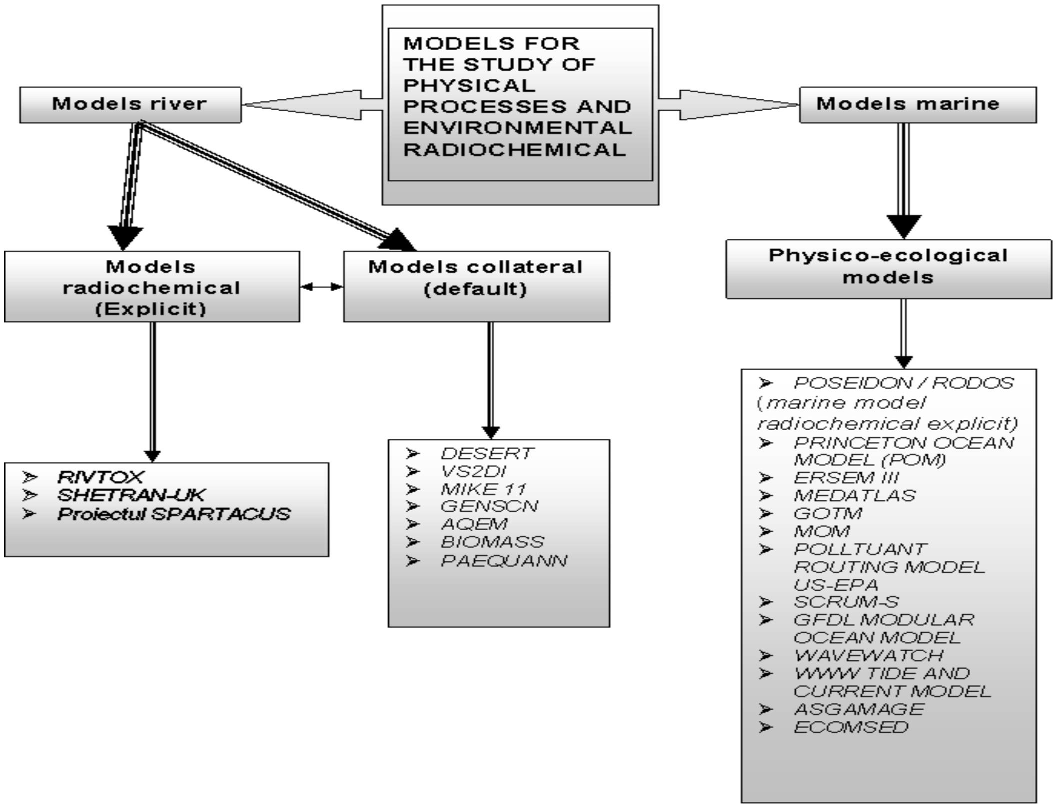

Fig. 1 Simplified scheme on river and marine models and software

- The AQUATOX 2-1

  AQUATOX (Richard Park, Jonathan Clough-AQUATOX MODEL) [4] is a simulation model for aquatic systems. Predicts effects of various pollutants such as nutrients and organic matter in ecosystems, including fish, invertebrates and aquatic plants. Simulates the transfer of biomass energy and chemicals from one compartment of an ecosystem to another. The model is calculated once each of the major chemical and biological processes are known therefore basic mechanical model.
- The DESERT - Decision Support System for Evaluating River Basin Strategies [8].

  Model / software consists of several units: unit viewer - performs spatial distribution of objects; hydraulic unit - hydraulic calculation caracterisicilor River (River) - depth, cross-sectional area and the propagation time required for simulating water quality; unit water quality simulation - simulation of water quality for river systems is generally achieved by solving a set of equations, ordinary and partial differential equations, which describe the relevant physical and chemical processes.

- BIOCHLOR 1.0 (United States Environmental Protection Agency Office of Research and Development Washington DC 20460 EPA/600/R-00/008 January 2000) [15].
  It is a visualization model for simulating natural attenuation remedy the organic solvent containing dissolved chlorine (cloroethan). This software programmed in MS Excel, is based on solvent transport model based on the analysis (equations) Domenico, one-dimensional advection is simulated, three-dimensional dispersion, linear adsorption, and biotransformation via reductive dechlorination
- The RIVTOX model (Institute of Mathematical Machines and System Problems (IMMSP) Glushkova Prospect 42, Kiev, 03187, Ukraine) [14].
  One-dimensional model of RIVTOX, was developed to simulate the transport of radionuclides in the network of rivers (rivers).
- The SHETRAN - UK (JC Bathurst, A. Purnama NERC Reosurce Water Systems Research Unit, Department of Civil Engineering, University of Newcastle upon Tyne, NEI 7 RU, UK). Refers to contaminants in sediment transport modeling. SHETRAN - UK introduce a system for modeling the transport of water, sediment and contaminant basin scale with some applications [1], [6].
- VS2DI [7] - Model and graphical software for simulating fluid flow and energy transport solutions for various saturated porous media (U.S. Geological Survey, Water-Resources Investigations Report, 99-4130, Lakewood, CO, 2000) is a software package VS2DI graphics to simulate fluid flow and transport in variable saturated porous medium 1 or 2 dimensions using Cartesian coordinate systems or radial.
- HydroChem (HydroChem Manual - rockware.com, First Edition: January, 1997)

It is a model and software designed for tabular and graphical realization of ion concentration dissolved in water [16].

- TDPF 1.0 - TopoDrive - ParticleFlow (U.S. Geological Survey (USGS), 2001) [17].
  There are two models for simulation and visualization of groundwater flow and transport of fluid particles in two dimensions, the stationary state.
- SMADA - GVPROF (trapezoidal calculation program developed by Dr. Ron D. Eaglin, University of Central Florida) [5].

Input data for this model calculation are: base length, slope, Manning coefficient, depth, flow rate. Output data (for uniform flow and uniform flow respectively - gradually varied) critical depth, normal depth - and for normal depth - flow area, wetted perimeter.

Marine complex model for the diagnosis of physical coastal ecology features Romanian Black Sea

To apply the complex model (ie POM coupling patterns and ERSEM III, [9], [10], [11], [18]) for Romanian coastline of the Black Sea) have to meet several conditions, the first condition, the adaptation to this area of complex model. This was done through the development of two models, namely BREG - regional model for the entire area of the Black Sea and BSHELF - zonal model for Romanian coastline; this is because, the coastal area suffers influences from Black Sea water object considered as a whole.

These models resulted directly from Princeton Ocean Model Model (POM) and ERSEM III and were written in the programming language (code) FORTRAN 77 and runs on Linux Mandrake OS 9. Adapted programs were conducted by R. Sorgente and M.Zavatarelli from ICM Sardinia (BREG, BSHELF) and by M. Vichi, from INGV Bologna (ERSEM III) [12], [13], [19].

Data for the 12-month atmospheric climatic parameters were made available by the ERA-European center for medium range weather forecast 6 hourly Re-Analysis (European Centre for Environment weather forecast orderly analyzed the frequency of 6 hours). This set of data (1 ° x1 °) contains the components x, y interaction of wind (wind stress), air temperature, relative humidity, cloud cover (cloud), and the amplitude of wind speed (averaged from January 1. 1979- December 31. 1993).

The physical parameters considered in modeling are: elevation (profile) - ETA-tem temperature, salinity, sal, QUP radiated heat flux, total heat flow - qtot, the wind (wind stress) on the x - WSU, the wind (wind stress) in the direction Y - WSV, evaporation - emp, the pressure on the x - Ubara, the pressure in the direction Y - Vbar, the speed in the direction x - u, y-direction speed v - speed in the direction z - w. Environmental parameters considered: basic structures: C - carbon, N - nitrate, P - phosphate O - oxygen, I - solar radiation, M - depth (m), R - reduction equivalents; pelagic structures, benthic structures; variables green transport.

The programs run in a specific sequence, after the advance was made data distribution and mediation for four seasons (winter, spring, summer, autumn).

Main programs included (specify, however, that many programs have subincluse and associated routines).

1. **GRID program.** This program forms the network (grid) bathymetry and vertical distribution of sigma level of POM (and by extension, for BREG-BHSELF). It is necessary first to specify the size of the domain.

2. **MEDATLAS program**. This program reads the files and interpolates MedAtlas high-resolution bi-linear interpolation model.

3. **ecmwf** program. The program reads the wind speed and direction after 12-month climatological data interpolation provided by ERA (Jan. ... Dec.), a bi-linear interpolation model. Upward heat flux = heat flux + sensible heat flux longwave (big waves) + heat flux of evaporation. All flows (solar, ascending, evaporation) are considered positive. It included the model domain size and the program 'gridcom.h'.

4. **POM** program. (Black Sea regional model). Includes sub 'comblk98.h', 'obdata.h', 'sbdata.h', with fewer moving steps. Under this program, a series of calculations are performed (vertical integral calculation viscosity term horizontal. Barotropice calculation of the average velocity multiplied by the depth, average time step, the calculation of the surface profile, the calculation speed barotropics, etc.).

Associated subroutines main obdata. f (data handling subroutine in the program POM) BCOND (IDX) (subroutine distributes data POM conditions (initial and edge) to: integrating vertical velocity, total velocity, temperature and salinity, vertical speed, energy kinetic) run_diag.sh (variable label may be a password, for example toto.sh) (subroutine command to run programs) init0 (Includes' comblk98.h ', and has zero initializes all common blocks' comblk98.h 'and main programs).

Figure 2 presents a scheme for defining complex model structure POM / BREG / BSHELF and ERSEM III.

Input data: longitude, latitude, meteorological parameters (air temperature, humidity, cloud cover, wind speed amplitude, solar radiation), evaporation, precipitation, depth (level), water temperature, salinity, ecological parameters: the concentration of carbon nitrate, phosphate, oxygen, corresponding to reduce) the concentration of chlorophyll concentration of pelagic and benthic organisms.

The model complex: BREG / BSHELF / ERSEM III
Programs (specific for four seasons) grid.f, bathy.ctl, gridcom.h, medatlas.sh, ecmwf.sh, comblk.h, pom.f, pom.sh, pom.txt, pom.ctl, sigtoz. f, HIROPE 1 D, Bornholm 2

Output data : - the diagnostic, prognostic mode - representing seasonal (winter, spring, summer, autumn) for the parameters: elevation (profile) - eta, temperature - tem, salinity - sal, radiant heat flow - qup , the total heat flow - qtot, wind action on the x (wsu), evaporation - emp, pressure field on the x (ubar), y (vbar) velocity field in the x (u), y (v), z (w)., ecological parameters - evaluation variables (pelagic, benthic, oxygen, nutrient recovery benthic)

Fig. 2 Representation of complex synthetic model BREG / BSHELF / ERSEM III

3. RESULTS AND SIGNIFICANCE

Application conditions for models and software

To apply the models / software previously described are required a number of conditions, among which may indicate the following.

If AQUATOX 2-1 first be assured monitoring data required by the model on the multitude of toxic substances (acrolein, alachlor, atrazine, chlordane, etc.) of aquatic organisms (animals - Amphipoda, Cisco, Thricorithode, and so on, plant - Chladophora, Dinophlagellats, Hydril, and so on,) as well as other parameters such as temperature, pH, BOD 5, the intensity of the wind, evaporation, water movement, etc. Second calibration must be done using reliable data for a range of toxic substances and aquatic biota. Third, bear in mind that the simulation performed with a certain delay to the real processes and require interpretation rather laborious.

If DESERT, the main problems that arise are: adapting the model for river water system (as it is configured especially for rivers) and then select the network nodes, grid system and the choice of pollution sources and types of pollutants.

In the case of biochemical main problems which arise are related to the monitoring and calibration (for data backup data hydraulic and organic compounds containing chlorine).

Some characteristics - conditions of application of the model marine complex

1. Programs running under Linux Mandrake 9 run time for each season being about four hours during growing more so as the number of data is larger.
2. Regarding the edge boundary conditions.

For the internal mode calculations is very important to define the boundary conditions. At this stage not specified with sufficient rigor edge boundary conditions for the directions north, east, south, the Romanian coastal zone, it constitutes itself as one of the conditions of applicability (Figure 3) [20].

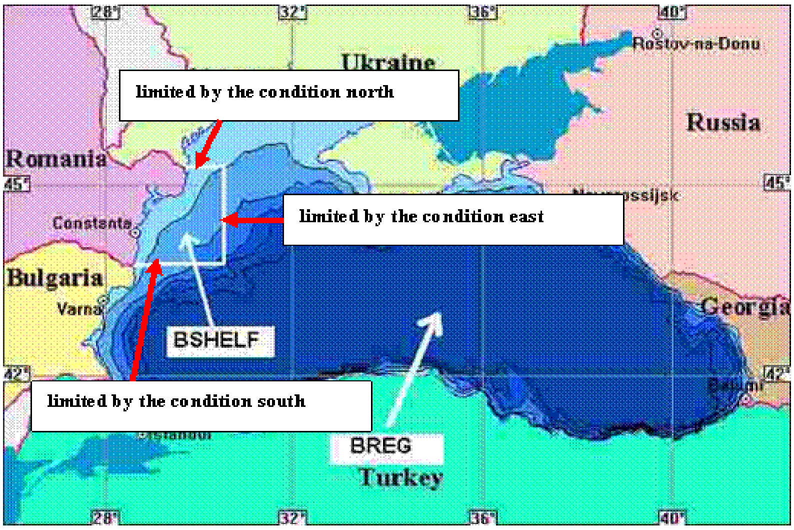

Fig. 3 Edge demarcation boundary conditions for the application BREG / BHSELF the Romanian coastline of the Black Sea

3. Another condition for the applicability of the model is the existence of complex software implementation (input) continuous flow monitoring data [20].

The data used were provided by ERA (ERA-European center for medium range weather forecast 6 hourly Re-Analysis) is implemented in a separate program strictly included in the software package BREG / BHSELF. Using IM = 105, JM and KM = 92 = 24, then running the diagnostic mode and viewing the program grads are obtained, for example, the following graphical results [20].

- Speed barotropic sea surface (Figure 4):

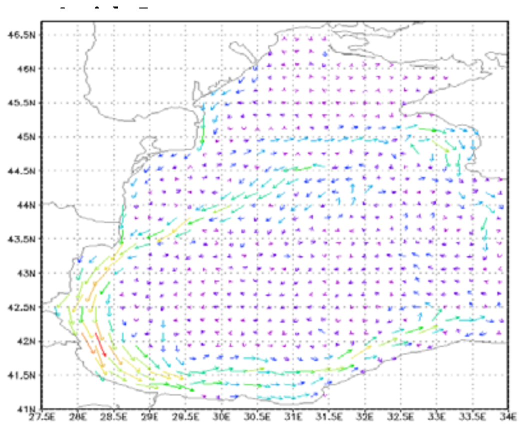

Fig. 4 Speed barotropic sea surface in winter

- Salinity at the depth of 50 m (Figure 5):

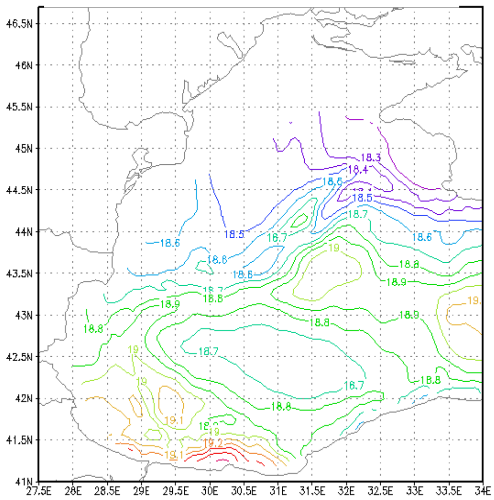

Fig. 5 Distribution of salinity (psu) at a depth of 50 m in winter

- Water temperature (^{0} C) at a depth of 5 m (Figure 6):

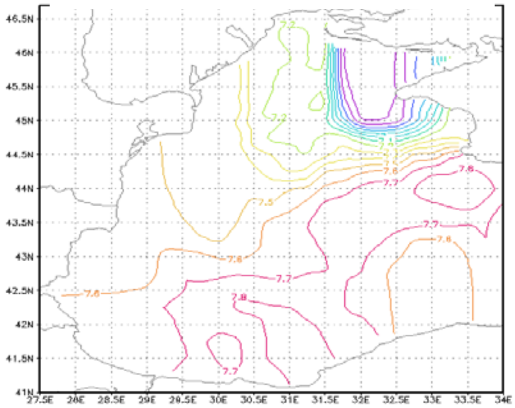

Fig. 6 Water temperature (^{0} C) at a depth of 5 m in winter

4. CONCLUSIONS

1. In principle, there are three possibilities for the application of models (software) eco-hydrological (hydro / radiochemical) where the Danube:

- Or it may take one or two complex models (integrated) (AQUATOX, MIKE 20, dessert, etc.) that involve a great deal of work with the possibility of errors larger or smaller during operation with these models;

- Or it can adopt a set of specialized models (a pack of models such as HEC-RAS hydrological parameters, VS2DI, etc., Hydro HydroChem or biochemical parameters for AQEM biological, geochemical, and radiochemical parameters PHREEQC, etc.) and working with this success, in this case there is a possibility distribution as well as intervention in the process work if the errors may occur, the difficulty lies in assembling and interpreting the results;

- Or it may take a combined way - ie a model (software) and a group of complex models (software) specialized or complementary.

2. Among the conditions of application of the model marine complex, include: setting the edge boundary conditions is a prerequisite, it means that the establishment of equations that must be imposed in order to define the applicability of the model BSHELF. For the Romanian coastal zone should be set three edge boundary conditions, ie limit the northern, eastern and southern. At this stage these conditions are not established. Another condition of application is the development of a program for the implementation of continuous flow monitoring data, existing data is entered via a separate species, including various data files provided by ERA (European center for medium range weather forecast 6 - hourly Re-Analysis), included in this software package. Also be assured monitoring data required by the model, currently not fully met this requirement (eg, for data streams heat evaporation or marine or benthic population). A condition of applicability of collateral is to ensure adequate support for storage and processing of monitoring continuous flow.

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Extreme Value Analysis of The Barlad River Time Series

Silvia Chelcea, Monica Ionita

Abstract – **The purpose of this study is to determine the most suitable extreme value distribution for the monthly discharge time series of Barlad river (measured at Barlad gauge) for the period 1957 - 2005. To achieve this objective, Peak over Threshold approach associated with the General Pareto Distribution. For performing parameter estimation the Maximum Likelihood Estimation (MLE) method was used.**

The results have shown that, the Pareto distribution provides the most reasonable model for our river for spring and summer months, meanwhile the normal or beta distribution fits best the winter and autumn months. Different thresholds have been tested for the daily discharges. The Pareto distribution, a heavy tailed distribution, yields a relative high probability of having large values. We have also estimated the return levels associated to different return periods (2 years, 5 years, 10 years, 50 years and 100 years) and the 95% confidence level for each return level.

Keywords **– atmospheric circulation, general Pareto distribution, return levels.**

1. INTRODUCTION

Throughout the world, interest in the research focussing on extremes (e.g. temperature, precipitation) and their variation has increased considerably in the last couple of years [3]-[5].

Hydrological extremes, such as floods and droughts, have always been a major concern. Although a lot of achievements have been made for understanding and predicting these types of extremes, flood and droughts continue to produce a huge amount of material losses and also human casualties.

Since the 1990s, several extreme floods have occurred on European rivers [1]. In august 2002 Elbe and Danube rivers have flooded their basins [10]-[11] and the year 2005 was an exceptional year for the rivers situated on the Romania territory. The water levels, especially for the rivers situated in the north-western part of the country, have reached historical values [9].

Information about maximum flood values in terms of water level or stream flow is crucial for reservoir design and management, construction of dams for flood protection and area planning. In order to asses potential flood damages, one needs to have information about hydrological variables, the use of floodplains on the catchment areas and the degree of exposure of human related activities to these extremes.

In flood frequency analysis, methods such as Generalized Extreme Value (GEV) and Peak over Threshold (POT) have been used. Hydrological time series do not necessarily fulfil the basic requirements of extreme value theory. They are a result of complex processes (snow melt, precipitation, soil humidity, etc) and a univariate distribution with two or three parameters is not necessarily a good approximation of their complex nature [7]. It is important to select extremes in the data so that GEV or POT distributions to fit these extreme data. The POT

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approach is assumed to be more precise than Block Maxima (approach with one-year block size), associated with GEV, because the annual maxima are not always true extremes.

The main objective of this study is to find a suitable distribution class for flood frequency analysis for the Barlad river. The specific tasks of the analysis are to perform POT analysis for Barlad discharge time series, to analyse the shape of the distribution tail using quantile-quantile plots and to determine the return level for various return periods.

2. DATA AND METHODS

2.1 DATA

Barlad catchment area (**Fig. 1**) is situated in the eastern part of the country. Barlad river, which is a tributary to Siret River, is considered to be one of the biggest rivers of the country, having an area of 7220 km^2, 207 km length and an average altitude of 374 m.

The whole Barlad river basin is situated in the Romanian province. The climate of the region is continental and is under the great influence of the Atlantic air masses that attenuate the continental nature of the climate, increasing its humidity. The small quantities of precipitation that fall in this basin determine a semi-permanent runoff regime.

In this study we have used the daily discharge values measured at Barlad gauge, for the period 1957-2005. From the daily discharge values we have selected the daily values for every specific month of the year.

In order to see which are the weather patterns associated with some of the extreme events in our time series we have looked at the weather maps corresponding to the occurrence of one of the most intense flood event, which took place between 15-16 July 1969. The highest discharge values, in our time series, have been measured in these two days.

As large scale atmospheric variables we used: a) daily geopotential height measured at 500mb level (G500); b) daily sea level pressure (SLP) and c) Precipitable Water Content (PW). These data sets were taken from NCEP/NCAR reanalysis data [6].

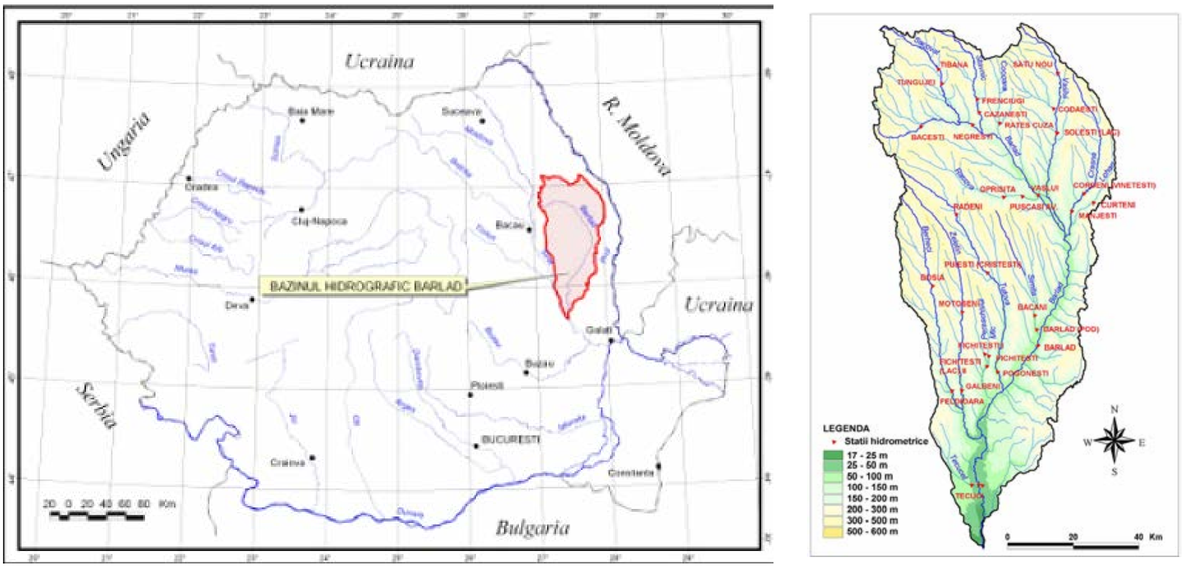


Fig. 1. Barlad catchment area

2.2 METHODS

2.2.1. PEAK OVER THRESHOLD

Analysis of the distribution of extremes is an important diagnostic tool for studying the occurrence of rare events [2], [8]. The extreme value theory (EVT) aims at studying the statistics of extreme phenomena. To study the distribution of extreme values, two methods can be applied: Generalized Extreme Values (GEV) method and Peak over Threshold (POT) method. In this study, we will focus just on POT method. The POT method describes the probability density function of a variable when it exceeds a high threshold *u*. Distribution of the values selected using POT should have an approximate generalised Pareto distribution (GPD), for a high sufficient threshold. The advantage of GPD, when compared with GEV, is that GPD estimate also takes into account all the observations that exceed a certain threshold. In contrast, only block maxima (e.g. the highest or the lowest values in a year) are considered by the GEV.

For a large enough threshold *u*, the distribution function X-*u*, conditional on X>*u*, is approximately:

$$F(y;\sigma^*,\xi) = 1 - \left[1 + \frac{\xi y}{\sigma^*}\right]^{-1/\xi} \tag{1}$$

$$\text{with: } \begin{cases} 1 + \dfrac{\xi y}{\sigma^*} > 0 & \xi \neq 0 \\ 1 - e^{-y/\sigma^*} & \xi = 0 \end{cases} \tag{2}$$

where σ^* is the scale parameter and ξ is the shape parameter.

Depending on the value of ξ we can have three possible cases:

- $\xi=0$, a light-tailed (or exponential) distribution
- $\xi>0$, a heavy-tailed (or Pareto) distribution
- $\xi<0$, a bounded (or beta) distribution

2.2.2. RETURN LEVELS

Typically, when considering extreme values of a random variable, one is interested in the return level of an extreme event, defined as the value, Z_p, such that there is a probability p that Z_p is exceeded in any given year, or alternatively, the level that is expected to be exceeded on average every once in a 1/*p* years (1/*p* is often referred as the return period). For example, if the 100-year return level for discharge at a given station is found to be 20.5 m^3/s, then the probability of discharge exceeding 20.5 m^3/s in any given year is 1/100=0.01. The return level z_p is define as:

$$Z_p = u + \frac{\sigma}{\xi}\left[\left(n_y p \zeta_u\right)^{\xi} - 1\right] \tag{3}$$

where n_y is the number of observations per year and ζ_u is the probability of an individual observation to exceed the threshold *u*.

The advantage of the return level is that for instance, one could obtain 200-year return levels of water stages with only 150 years of data.

3. RESULTS AND SIGNIFICANCES

3.1. APPLICATION OF THE GPD TO BARLAD MONTLY RIVER DISCHARGE

The GPD distribution uses more information than a model based only on block maxima (e.g. the highest or the lowest value in the year). For this reason we fitted the GPD distribution to our times series. This method contrasts with block maxima approach through the characterisation of an observation as an extreme if it exceeds a high threshold. A threshold which is too low is likely to violate the asymptotic basis of the model, leading to biases; while a threshold which is too high will generate just few excesses with which the model can be estimated, leading to a high variance [2]. One of the most difficult parts of the POT approach is to identify a suitable the threshold. We have focused on two methods to identify a suitable threshold:

- An exploratory technique carried out prior model estimation (mean residual plot). The idea of the method is to identify the threshold up to where the line is almost linear, taking into account the 95% confidence bounds (**Fig. 2** – first graph).
- The assessment of the stability of parameter estimates based on the fitting of the models across a range of different thresholds (**Fig. 2** – last two graphs).

Tacking into account that we have applied this method for each month of the year, we will show the procedure of threshold identification, just for one month.

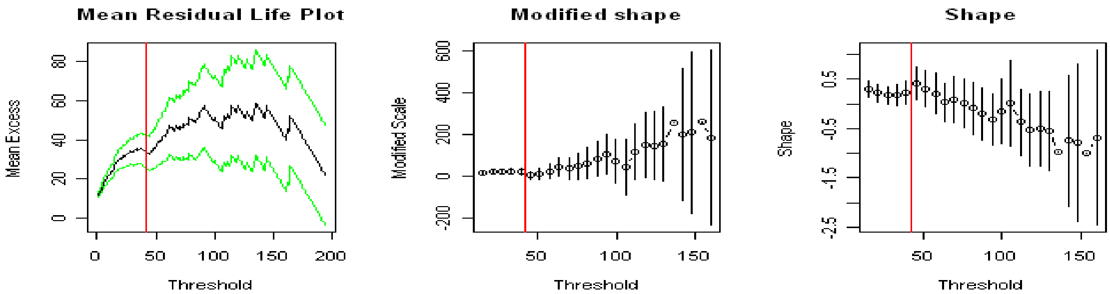


Fig. 2. Mean residual plot of April daily discharge thresholds vs. mean excess discharge – first graph; GPD fits for a range of 50 thresholds from 0 to 150 m^3/sec for April daily discharge

Using the methods described above the thresholds identified for each month is shown in **Table 1**.

Table 1. The threshold values corresponding to each month

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Threshold (m^3/s)	10	13	40	42	30	16	25	20	16	15	12	10

The shape parameter estimated by GPD, using the thresholds identified above, is given in **Fig. 3**. From the values of the shape parameter it can be noticed that the months March-April and June-August tend towards a heavy-tailed (or Pareto) distribution ($\xi>0$), meanwhile the rest of the months have either a normal distribution ($\xi\approx0$) or a bounded distribution ($\xi<0$).

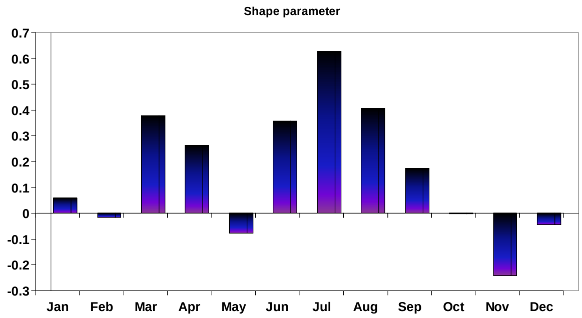
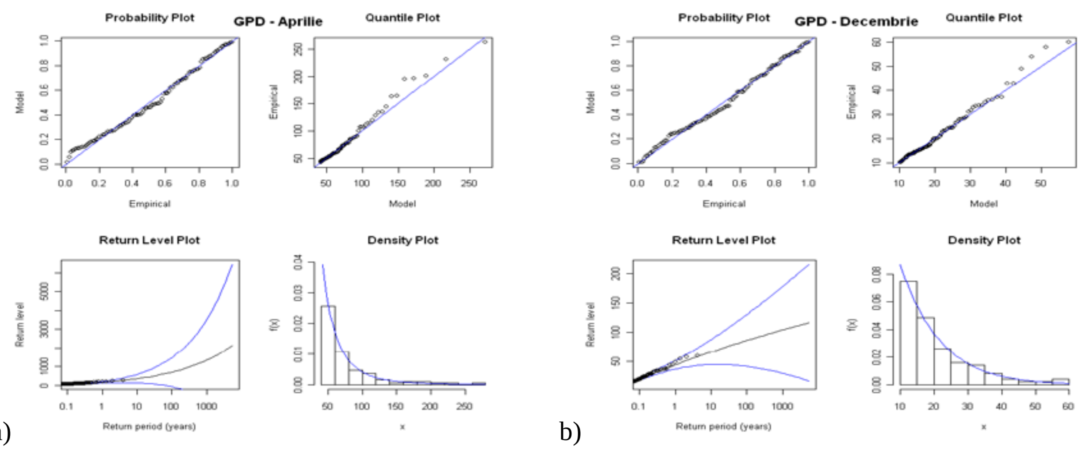


Fig. 3. Monthly distribution of the shape parameter

A physical explanation for the heavy-tailed distribution in spring might be related to the fact that there is an increased discharge during these months due to the melting of the snow in the catchment area and the availability of the humidity in the soil. For the summer months, the heavy-tailed distribution might be related to the heavy precipitation events, specific to the summer months.

To test if GPD fits well our data sets, the diagnostic plots for April (a heavy-tailed distribution) and December (a light-tailed distribution) are represented in **Fig. 4a**, respectively **Fig. 4b**. In both cases the probability and quantile-quantile (Q-Q) plots are approximately linear, indicating that the assumed form of distribution is reasonable. Although not shown here, the Q-Q plots are approximately linear for the all months, suggesting that the GPD is a suitable distribution for our data set.



a) b)

Fig. 4. Diagnostic plots for the GPD fit to: a) April daily discharge and b) December daily discharge

3.2. RETURN LEVELS

Once the best model for out time series has been determined, the interest is in deriving the return levels, corresponding to different return periods. The return level is the level exceeded on average every once in T years. **Table 2** gives the estimates of the return level for the best fitting model for T = 2, 5, 10, 50 and 100 years. We also computed the associated 95% confidence intervals.

Discharge values between 50 and 200 m^3/s have return periods ranging from 5 to 100 years, for the winter and autumn months. Taking into account that for these months the shape parameter is light tailed, the error band is not too large for return periods above 50 years.

For the spring and summer months, discharge values reaching even 1000 m^3/s have return periods between 20 to 100 years. These high values, compared to winter and autumn moths, are determined by the value of the shape parameter. A positive shape parameter (heavy-tailed distribution) induces larger biases for the return periods, especially the ones above 10 years.

Table 2. Estimates of the return level for different return periods. Between the brackets are the values for the 95% confidence level.

Month	**Return levels**				
	2 years (95 % CI)	**5 years (95 % CI)**	**10 years (95 % CI)**	**50 years (95 % CI)**	**100 years (95 % CI)**
January	33.98 (28.61, 44. 8)	49.99 (33.41, 57.139)	57.35 (43.29, 82.72)	75.65 (51.12, 110.81)	84.07 (53.916, 123.74)
February	79.87 (68.23, 102.47)	93.10 (76.85, 128.18)	102.98 (82.59, 133.40)	125.49 (93.70, 163.52)	134.99 (97.67, 276.24)
March	228.71 (163.41, 334.64)	333.57 (209.78, 498.35)	440.99 (248.17, 666.06)	831.98 (387.45, 1276.51)	1089.38 (500.83, 1678.38)
April	215.75 (164.06, 300.28)	289.56 (200.44, 410.59)	359.33 (228.72, 513.71)	579.09 (317.79, 840.39)	706.65 (383.29, 1030.01)
May	122.86 (106.07, 167.71)	141.58 (118.28, 195.47)	154.87 (125.69, 215.22)	183.24 (138.13, 257.25)	194.42 (141.92, 273.83)
June	205.50 (147.33, 272.45)	297.31 (197.92, 396.69)	389.69 (257.67, 521.70)	716.81 (469.22, 964.39)	926.92 (605.11, 1248.73)
July	212.50 (128.62, 347.48)	379.98 (187.30, 635.51)	588.72 (242.92, 994.51)	1622.11 (472.43, 2771.78)	2508.08 (720.64, 4295.52)
August	151.33 (101.39, 251.65)	235.21 (137.93, 399.6)	323.07 (167.62, 554.59)	654.09 (241.86, 1138.47)	878.75 (276.01, 1534.75)
September	80.86 (60.48, 129.98)	108.92 (76.04, 179.31)	133.36 (86.99, 222.25)	202.92 (108.42, 344.56)	239.54 (115.88, 408.84)
October	87.55 (68.02, 155.67)	110.97 (82.99, 201,.08)	128.65 (91.67, 235.35)	169.57 (104.54, 314.68)	187.13 (107.79, 348.73)
November	37.35 (32.56, 53.27)	42.43 (36.73, 66.68)	45.59 (39.27, 72.37)	51.20 (38.89, 82.43)	53.02 (40.14, 85.70)
December	50.91 (43.42, 68.61)	59.65 (49.25, 81.13)	66.04 (52.97, 90.29)	80.17 (59.62, 110.53)	85.95 (61.79, 118.82)

3.3 CASE STUDY: 15-16.07.1969 FLOOD EPISODE

The highest discharge values, for the period analyzed in this study, were recorded between 15-16 July 1969. The peak was on 16th of July, when the recorded discharge has reached a value of 317 m^3/s.

The synoptic situation, associated with this extreme case, is presented in **Fig. 5**. On the first column is the evolution of the geopotential height, measured at 500 mb level, starting from 12.07.1969 00:00 until 16.07.1969 18:00. We choose to show the maps every 12 hours, due to the fact that this event was short and is important to see the evolution of the weather patterns using a very small time step.

As it can be noticed form the maps on the first and third column (G500 and SLP) over the eastern part of the country, a low pressure system persisted during these days. This low pressure system is much more evident in the SLP field. Throughout this low pressure centred on the eastern part of the country, it was possible the advection of cold air from the north which caused, in contact with the warm air coming from south-east, an intense atmospheric instability over these regions. These short periods of atmospheric instability, are usually associated with heavy rain, strong winds and flash floods.

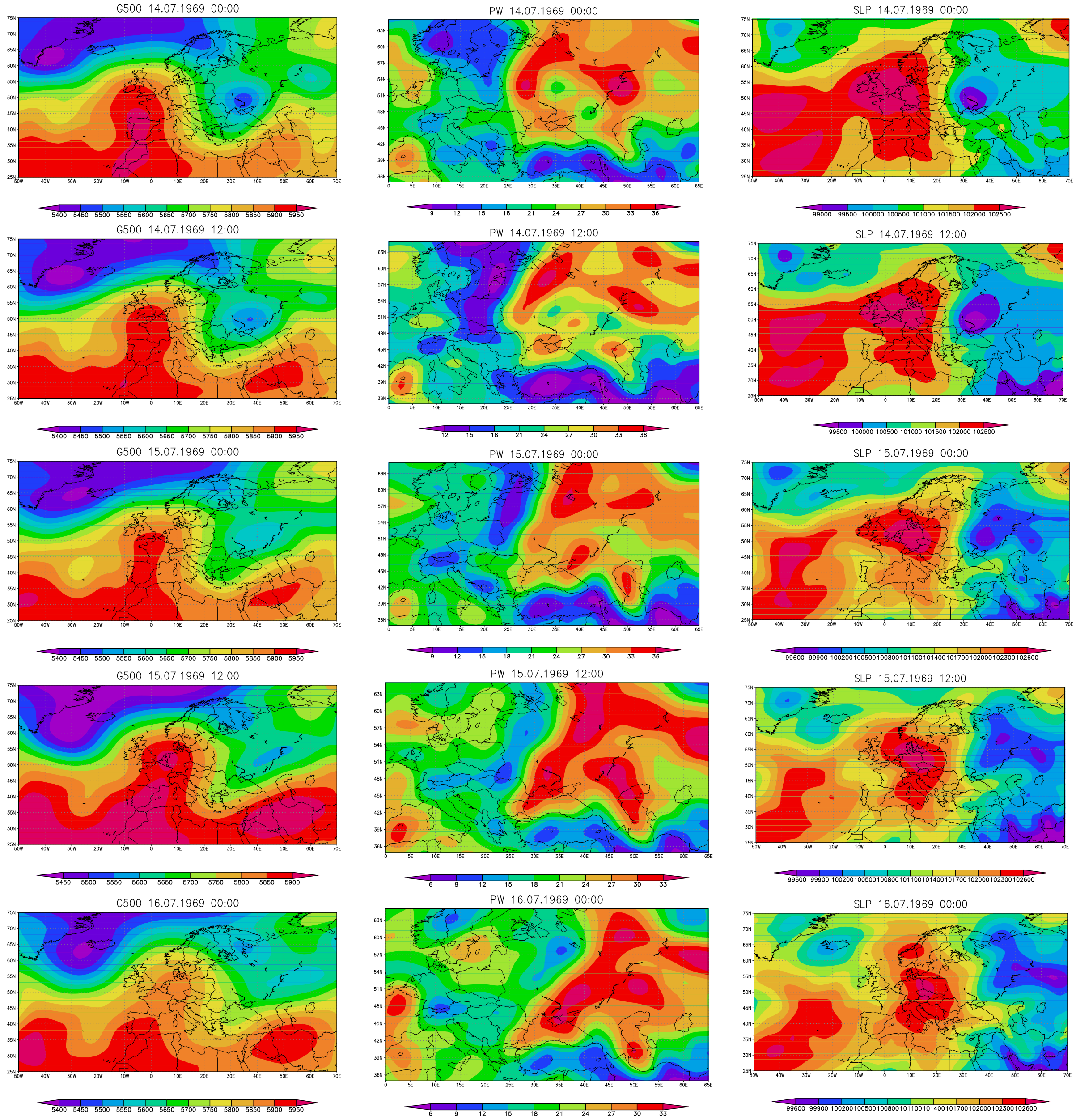


Fig. 5. The synoptic charts (G500, PW and SLP) associated with the extreme flood event which took place between 15-16 July 1969

Extreme precipitation should be explainable also in terms of enhanced moisture availability and/or conditions that enhance condensations of water vapours. Precipitable water provides a measure of the first term, being the total water vapour content in a vertical column of the atmosphere. Condensation of the precipitable water content typically requires uplift provided by different mechanism, but the most important one is cyclonic circulation, like in our case. Due to the low pressure system, a zone of intense convergence has developed over the eastern part of the country, inducing an uplift of the warm air, which in contact with the cold air that was brought from the north, created a very unstable atmosphere.

4. CONCLUSIONS

From the statistical analysis, in terms of POT method, of extreme daily discharge for Barlad river, it can be concluded that:

- Testing different methods, the exceeding threshold for the daily discharge, corresponding to every month of the year, was found to vary between 10 m^3/s for winter months and 40 m^3/s for spring months.
- Spring and summer months tend to have heavy-tailed distribution, meanwhile winter and autumn months have either a normal or a light-tailed distribution.
- POT method fits well our data set, the quantitle-quantile plots being approximately linear for all months.
- Discharge values between 50 and 200 m^3/s have return periods ranging from 5 to 100 years, for winter and autumn months.
- For spring and summer, discharge values reaching even 1000 m^3/s, have return periods between 20 and 100 years.
- The synoptic situation associated with an extreme event, which took place between 15-16 July 1969, indicated that this event was due to a low pressure system centred over the eastern part of the country. Due to this low pressure system, a zone of intense convergence has developed, inducing an uplift of the warm air, which in contact with the cold air that was brought from the north, created a very unstable atmosphere and enhanced precipitation.

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Facilitation of fish migration upstream the Centre Bridge on the Crisul Repede River

Razvan Voicu, Ecaterina Luca and Liliana Voicu

***Abstract* – The subject approached in the current paper represents a European theme of great topicality and interest regarding the restoration of the water courses affected by the hydro-morphological pressures created by the presence of transversal works. The paper presents a case study for that are proposed practical solutions for the restoration of longitudinal connectivity of Crisul Repede River and ensuring fish migration over the weir Centru Bridge, placed on Crisul Repede River near Oradea Town Hall, in the context of the problems specific to the area. The proposed solutions provide the building of migration system consisting of some modules that are meant to facilitate the migratory fish access to the upstream habitats.**

***Keywords* – Crisul Repede, fish migration, weir.**

1. INTRODUCTION

This article approaches new issues related to the possibility to facilitate fish migration by proposing an engineering solution to restore longitudinal connectivity of Crisul Repede River on the reach: Bonor confluence - Hungarian border (RW3.1.44_B7 water body). This water body has been designated as heavily modified due to the presence of a large number of transversal sealing devices (dams, outlet and bottom sills). There are 15 transversal obstacles transform the flow regime of water body in a slowly system on the about 30% of its length. In this area the average density of sills are 0.8 sills/km, and some sections exceeding the value of 0.4 sills/km [4].

After careful analysis of the situation in the study area (documentation, expedition, discussions with specialists from Crisuri WBA and analysis of water quality) the Centru Bridge weir placed on Crisul Repede River was selected as a case study. For this is proposed practical solutions for longitudinal connectivity restoration of Crisul Repede River in order to ensure fish migration upstream and downstream the Centru Bridge weir. The systems proposed for fish migration will be built on the left bank of the Crisul Repede River.

The paper has a practical value because offer solutions for a case study, that was included in the Measures Plan which follow to be implemented by 2015 (Annex 9.17.a of the WBMP).

1.1. STUDY AREA

The selected case study (Centru bridge weir) is placed on the water body *Crişul Repede River → Bonor – border* (RW3.1.44_B7) – extends from the confluence between the Crişul Repede River and the Bonor Creek up

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to the border with Hungary, being located mostly in Oradea. It is bordered by sites of Community Importance: *ROSCI0104 the Lower meadow of the Crişul Repede River* (area = 844.5 ha) placed downstream the Oradea City and *ROSCI0050 – the Crişul Repede River upstream of Oradea* (area = 2006.3 ha) [5], [6], placed upstream the Oradea City; both sites are important for hosting rich fish fauna and some rare species of invertebrates (**Fig. 1**).

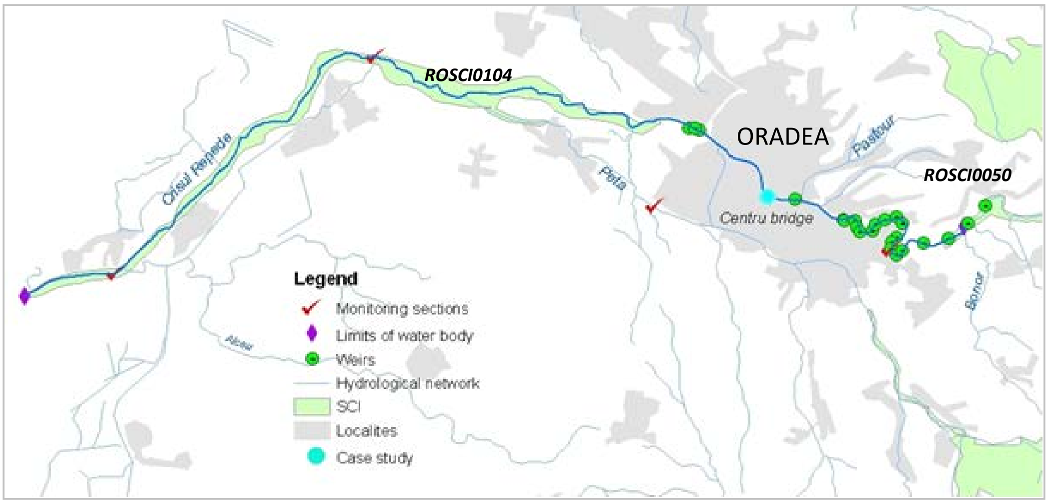

Fig 1. The analyzed case study placement

According to the Bănărescu (1964) the study area is part of the *nase fish zone.* The main migratory fish species characteristic to this fish zone are: the common nase (*Chondrostoma nasus*), the barbel (*Barbus Barbus*) and the zarte (*Vimba vimba*), to which other migratory fish species characteristic to the neighboring fishing zones can be added: the Danube gudgeon (*Gobio uranoscopus*), the burbot (*Lota lota*) and the sterlet (*Acipenser ruthenus*). Species such as the common carp (*Cyprinus carpio*), the common bream (*Abramis brama*) and the ide (*Leuciscus idus*) can penetrate more rarely, from the carp area into the slow water areas; the asp (*Aspius aspius*) can penetrate faster water areas [Bănărescu, 1964].

The current monitoring of the ichthyofauna in the Criș Basin began in 2005 and has been developed in stages, starting with the main watercourses, then considering each tributary. Analysis of fish fauna monitoring results achieved in 2007-2011 by WBA Crișuri specialists on 4 monitoring sections (**Fig. 1**) showed that there were reported twelve fish species, of which three migratory: the common nase (*Chondrostoma nasus*), the barbel (*Barbus Barbus*) and the common bream (*Abramis brama*) [2].

The comparative analysis of historical reference data related to those obtained during the monitoring period of ichthyofauna show that in the study area belonging to the common nase area there have not been reported any migratory fish species such as the zarte (*Vimba vimba*), the burbot (*Lota lota*) and the sterlet (*Acipenser ruthenus*), mentioned by Bănărescu. Among the most common fish species in the monitoring section *upstream Oradea*: the common nase (*Chondrostoma nasus*), the chub (*Leuciscus cephalus*) and the Prussian carp (*Carassius auratus gibelio).*

The targeted migratory fish species are the common nase (*Chondrostoma nasus*), the barbel (*Barbus barbus*) and the common bream (*Abramis brama*), which are part of the family Cyprinidae.

2. RESULTS AND DISCUSSIONS

This paper ofer a solutions to ensure fish migration upstream and downstream the Centru Bridge weir selected as a case study.

The Centru Bridge is located on the Crișul Repede River, near the City Hall of Oradea, being known as the Ferdinand Bridge or the Hall Bridge (**Fig. 2**). In this place the Crișul Repede River bed is 50m, and has 22.3m^3/s flow rate and 4 m/s water speed [2], [8].

The analyzed weir is 1.5 m high and has a drop of 1 m; it consists of a spillway sill (50m width), a stilling basin and a fixed risberm totally submerged under water (**Fig. 2**).

downstream of the Centre Bridge   downstream of the weir

Fig. 2. Centru Bridge weir

2.1. PROPOSED SOLUTION

As the first way to provide for migration of fish fauna upstream of the analyzed weir we propose to achieve a migration system as a canal consists of modules, built on the left bank of the river. The modules of this canal will be drilled into the concrete beam at the base of the concrete abutment (support wall) of the left bank of the river, following the river flow gradient (**Fig. 3**).

The *Module 1* of the migration canal will be rectangular parallelepipedic and will be diagonally drilled into the concrete beam at the base of the left bank abutment (**Fig. 3**), made in steps descending the slope downstream.

The module will be positioned so that water blade size in the migration canal to be of 50 cm in order to ensure good migration conditions of those migratory species targeted, according to special environmental requirements. *Module 2* of the migration canal will be further drilled, parallel to the left bank, into the next step of the beam (**Fig. 3**).

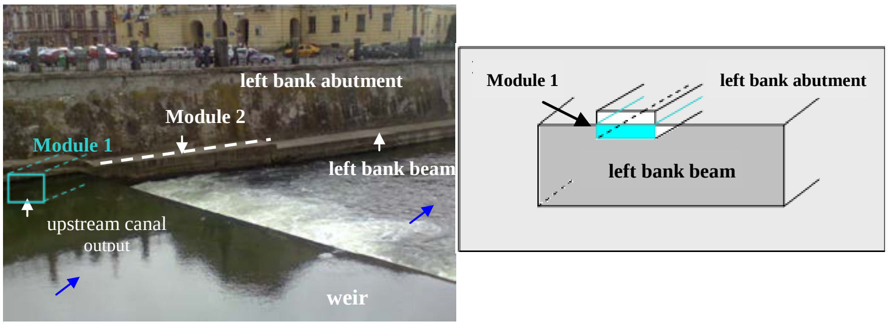

Fig. 3. Scheme on the Module 1 and Module 2 of the fish migration canal

The two modules of the migration canal will be joined at an angle of about 120° so as to avoid contact with the fish and keep the same slope for both modules in order to not disturb fish migration (**Fig. 4**).

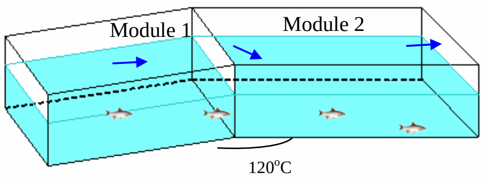

Fig. 4. Joint between Module 1 and Module 2 of migration canal

At the upstream end of *Module 1*, a distance of about 1 m in front of it, a protective metal grid to prevent cannel from blocking will be fixed. The grid may be rectangular or circular (**Fig. 5**) and it will be fastened by means of some bars embedded in the concrete beam, at 10 to 15 cm underwater. Fish access will be carried out both under the grid and laterally. The canal (Module 1 and Module 2) into the concrete beam must be illuminated so that fish can migrate. Thus this canal will be provided with some drills of about 15cm diameters at every meter. At the surface, these drills will be covered by transparent and resistant glass. In these conditions the fish can migrate smoothly through this canal upstream and downstream.

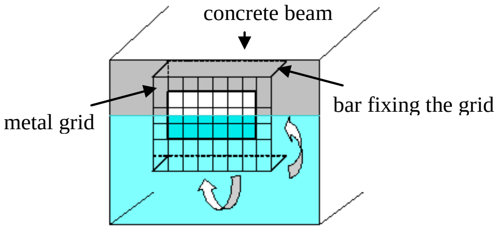

Fig. 5. Scheme of positioning of the metal grid concrete beam

At the end of the *Module 2* a rectangular resting basin for fish resting, having the following dimensions 1m x 1,5 m, will be located; it will make the connection to *Module 3* of the migration system (**Fig. 6**).

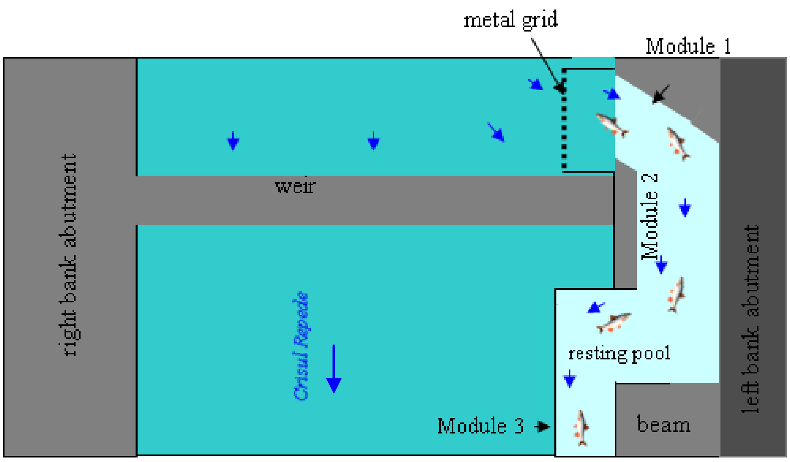

Fig. 6. Indicative scheme on the resting basin for fish

Further, there are two possibilities (a, b) to continue the migration canal of fish fauna.

a) The first possibility consists of attaching a metallic rectangular canal (*Module 3*) to the left bank, then it will slowly go down to the river bed, following the same slope as for the previous modules and it also will be closer to the natural slope of the Crișul Repede River (**Fig. 7**).

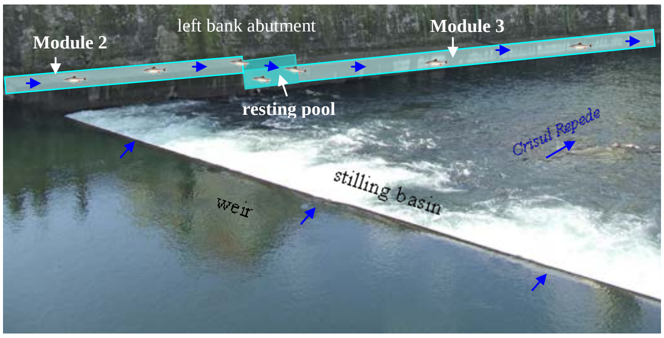

Fig. 7. Indicative scheme on positioning the Module 3 of the migration canal downstream of the weir

Module 3 will be fixed to the right bank beam by the means of some metal dowels arranged on two rows (**Fig. 7**). In order to protect the fish and the migration canal modules, the whole migration system will be covered with a protective grille (**Fig. 8**).

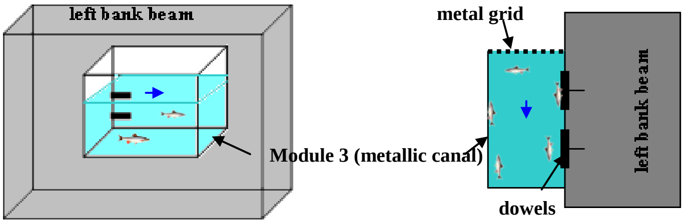

Fig. 8. Orientative scheme on fastening the fish migration canal

b) The second possibility would be represented by drilling, for the next step, into the left bank beam for the *Module 3* of the migration canal, which will have a length of about 1m in order to avoid the turbulence created downstream of the stilling basin (**Fig. 9 a**). Being connected to a rectangular basin for water level rising and then attached to the beam, the downstream end of *Module 3* will form an angle of 90°C. At its upper end the basin will be provided with two windows on the upstream and lateral sides, which will function as overflow outlets. On the downstream wall of the basin will be provide an access window for fish to facilitate there upstream migration. To avoid erosion of the basin walls, the windows will be provided with two ducts made of durable plastic (gutter like) to drain the water excess (**Fig. 9 b**). The upper surface of the basin is planned to be from transparent glass, resistance to impact. To ensure a better lighting, this pool can be made partially or entirely of glass impact resistance.

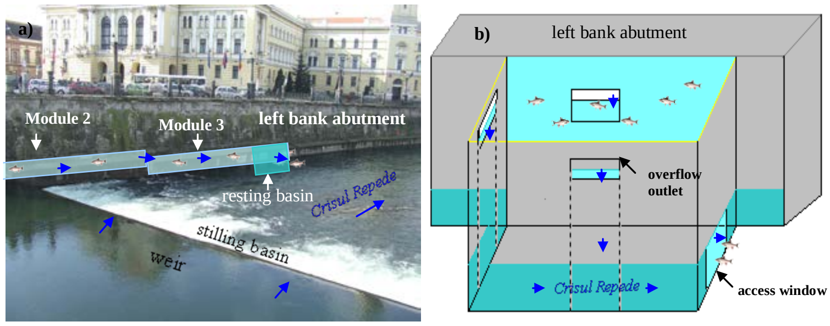

Fig. 9. Positioning of the concrete basin for water level rising

During the winter the access to the migration system can be closed by the means of a metal door set against the upstream end of the *Module 1* of the migration canal. The metal door is provided with an upstream opening and it will be fixed to the left bank beam using hinges whilst the closure will be performed through various methods (using lock, screw, spring steel, etc.).

In order to ensure a constant flow and water velocity into the migration canal, in the dry summer period it can be provide with small wood slots, placed at every meter (**Fig. 10**). Also, on the bottom of the 1st and 2nd modules and resting pool it can be realized a close to natural bed from gravel.

Ex. of canal with small wood slots

Gravel bed of the migration canal

Fig. 10. Examples of canal bed designed

3. Conclusions

The proposed migration systems can be made from sheet metal or carbon fiber that are less expensive and can be quickly assembling, providing considerable advantages (less price of final construction and reduced execution time).

Constructing of the proposed migration system will offer some advantages such as environmental, economic and social benefits:

- facilitate the access of migratory fish to upstream habitats;
- restoration of the longitudinal connectivity of the river;
- improving of water quality and ecological status of water body;
- improving recreational and angling opportunities.

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Mathematical Modelling in Transport of Pollutants

Ana-Maria Laura Petruţa, Paula Iancu and Adriana Pienaru

***Abstract* – This paper studies the matter of pollutants transport and dispersion in water and submits mathematical models in polyphasic liquids and porous media. Dispersion, diffusion equations, the Navier-Stokes in Reynolds form equations, the Saint-Venant and continuity equations are used. These models established the basis of computing programs, useful for evaluating the pollutants' transport in dynamic and static regime.**

***Keywords* – Navier-Stokes equations, pollutant, polyphasic liquid, porous media.**

1. INTRODUCTION

Water movement in porous media and the movement of polyphasic liquid can be expressed by complicated equations and difficult to resolve. For a given case, these equations are solved through mathematical and physical modeling. The models used in the study of water flow and in the contaminant transport is an approximation of the real system, developed under conditions and by means that can ensure the validity of the theoretical results obtained.

The simplifications are introduced as a set of assumptions which expresses our understanding regarding the nature of the system and its behavior. As a result, there will be no single model for a given system. Each set of assumptions will lead to a different model. Choosing the most appropriate model, for a given case, depends on the objectives of the investigation and the available resources (time, budget, etc.). The basic simplifying assumption involves finding the unknown function in a finite number of points of the analyzed field, in order to adequately approximate the unknown function of the field.

The mathematical modeling of a process is described through a system of equations and appendices items necessary for identifying and obtaining the solution of the specific case. In this meaning, the physical reality is abstracted through mathematical description.

The necessity of using modeling is the result of several reasons, including: the mathematical model is a conceptual study method, used to formulate hypotheses and theories; allows the automatic control and the optimization of the process after validation; in research, it allows the development of knowledge about the field; in design, the model allows the study of the impact of parameters on the receptors and streamlines the performance of the purification processes [3]. Mathematical modeling includes several steps, such as: process analysis, establishing the purpose of the mathematical model, building this model, choosing the method of resolving and determining the solution, validating the results and calibrating the constants. [3]

The analysis of the process requires a detailed study of the problem to correctly identify physical, chemical and biological phenomena, the parameters, and the correlations between them. Thus, it is necessary to know the

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basic principles of pollutants' transport in porous media and polyphasic liquid and define the theory which governs those processes.

The correlation model of the features aims to specify the relationship between the parameters and the essential characteristics of the process. By solving these equations (mathematical form of the model) the degree of compliance will be verified in relation to reality and is possible to change the mathematical expressions.

The theoretical foundation has the same goal to analyze the phenomena which appear in the studied process and to underlie the mathematical relationships into the explanatory model.

The purpose of the model is to identify the evolution process, to set the time scale which models the process and, finally, to specify the desired accuracy, so that the accuracy of the results will be determined.

The model construction represents the transposed theory into mathematical symbols and represents the analysis of the equations' systems structure. The method used to solve equations and systems of equations generally presents many levels of solving: a) the analytical solving of equations through exact integration, b) numerical integration using the computer, by obtaining solutions with limited validity in the case specifically studied.

The optimal mathematical model includes all of the important aspects of process dynamics, considering all possible variations of the parameters in order to identify the optimal areas. It must have a reasonable number of equations that can be solved analytically or numerically, and not least, to have comparable results with reality.

2. EQUATIONS OF THE POLLUTANTS TRANSPORT

The transport of the pollutants through porous media and in the polyphasic liquids is achieved by convection (or advection), respectively, the movement of elements in the solution, as a result of the ground water flow and surface water and through diffusion, meaning the penetration of a corpus into another corpus mass which is in contact with, due to the local difference of concentration as described in space and time**.** [1]

2.1. The pollutants motion equation for polyphasic liquids, used in modelling

Polyphasic mixture is composed of N stages (constituents). [3]

In constitutive laws elaboration, in many motion applications in polyphasic liquids, an incompressible liquid (ρ = constant), homogeneous and isotropic is accounted. The interactions between the constituent phases of the polyphasic mixture are chemical, biochemical or biological, and they manifest through mass transfer and internal energy.

The characteristic sizes of polyphase fluids are: instantaneous sizes, i.e. sizes attached to each phase of the mixture: speed, viscosity, pressure, tensor power, the relative density of the constituent, pulsing sizes (local), which express the relationship between the phases of the mixture and average sizes that characterize the turbulence regime of the polyphasic liquid. [3]

The physical model of the pollutants dispersion has as a concept the equations of fluid dynamics and the first and the second principle of thermodynamics, which complements the fluid movement with mathematical equations which include the pressure, the temperature and the volume of the fluid.

One of the fundamental laws of fluid motion, which is quantitatively described through mathematical countenance, is the law of mass conservation, by which we obtain the continuity equation:

$\frac{1}{\rho}\frac{d\rho}{dt} + div(\vec{v}) = 0$, taking into account the Taylor series progress (1)

ρ – specific fluid volume; t – unit of time; v – the flow rate of the fluid.

The second equation governing the transport and pollutants dispersion are the Navier-Stokes equations, which characterize the turbulent regime of polyphasic fluid flow. Consider the case of a viscous, homogeneous and isotropic fluid, (explained in a normal plan to the x-axis).

$$X - \frac{1}{\rho}\frac{\partial p}{\partial x} + \upsilon \Delta v_x = \frac{\partial v_x}{\partial t} + v_x \frac{\partial v_x}{\partial x} + v_y \frac{\partial v_x}{\partial y} + v_z \frac{\partial v_x}{\partial z} \quad (2)$$

υ – kinematic viscosity coefficient; p – pressure

Because of the turbulence, tensors of the unitary effort occur, so called tensors of stresses of the apparent friction. The total efforts result of the sum of tensors stresses due to viscosity (μ) and unit effort of the apparent friction (τ):

$$p_{xx} = -\overline{p} - \overline{pv^2{}_x}; \tau_{xy} = \mu\left(\frac{\partial v_x}{\partial y} + \frac{\partial v_y}{\partial x}\right) - \overline{pv'_x v'_y} \quad (3)$$

Physical modeling of an effluent pollutant evolution in a liquid environment is the result of driving forces action from the point of discharge up to the dispersion and its complete dilution.

The discharge of a pollutant into the stream produces a jet, whose confluence depends mainly of the velocity ratio and of the specific mass ratio of the pollutants and of the stream. The characteristic regions of the pollutant jet are [1]:

- the zone of entry, is the point of discharge and operates as long as the source of pollution exists; the pollutant bends in the direction and towards the emissary’s flow orientation;
- the transition zone, where the pollutant’s energy is degraded by increasing flow path;
- the dispersion zone; the pollutant loses all its energy and its velocity is equal to the water velocity in the stream.

The orientation of the pollutant jet from the discharge area opposite to the direction of water flow into the stream favors a faster mixture of the pollutant with the receiving environment. Thus the jet area from the discharge into the stream, corresponds to a fluid mass with which it is continuously or intermittently associated with its intrinsical amount of movement that can be generated by Archimedes forces as a result of the difference in concentration.

The size that characterizes both the Archimedes lift forces and the velocity of the fluid is the number of Froude densimeter:

$$Fr^2 = \frac{u^2}{gd\frac{\Delta\rho}{\rho_a}} \quad (4)$$

u – the feature of the current average speed;

d - characteristic geometric size of the entry section of the pollutant into the emissary; g - gravity acceleration.

$\frac{\Delta\rho}{\rho_a}$ - the relative difference between the effluent and the receiving environment;

If: Fr $\to \infty$ - lift forces are initially zero and will remain zero in the absence of a external thermal gradient.

Fr = 0 - inertial forces are null, resulting an purely portable system, called wedge polluting.

$0 \leq$ Fr $\leq \infty$ - lift forces and inertial forces have the same size order.

Solving the complexity of the convective dispersion phenomenon to the gradual movement, non-permanent, in the flow section (ω = ω (x, t)) and in the current velocity (u = u (x, t)) is determined by the Saint-Venant equations.

This water movement is governed by the equation of continuity (5) and the equation of dynamics (6), solving these equations requires finite difference method, considering Q, z, c, ω derived for the unknown function.

$$B\frac{\partial z}{\partial t}+\frac{\partial Q}{\partial x}=0 \tag{5}$$

$$\frac{\partial z}{\partial x}+\frac{1}{g\omega}\frac{\partial Q}{\partial t}+\frac{2Q}{g\omega^2}\frac{\partial Q}{\partial x}-\frac{Q^2}{g\omega^3}\frac{\partial \omega}{\partial x}+\frac{Q^2}{K^2}=0 \tag{6}$$

Q - stream flow; z - free surface area; B - bed width; ω - flow section; K- module flow.

Due to properties changes between the pollutant which enters in contact with a fluid, the transport property of the pollutant is the concentration, which is equivalent to the disturbing property into that fluid.

If it is assumed that this pollutant concentration is much smaller compared to the density, it is shown that:

$\frac{\partial C}{\partial t}+div(Cv)=div(D\mu gradC)+F_c$, general equation of dispersion phenomena. (7)

The mathematical expression of pollutant transport equation is characterized by the convection - diffusion equation described by the variation of pollutant concentration C, in time and space. The dispersion has the ultimate role to mix the multiphase fluids and it closes where the pollutant concentration uniforms.

$$\frac{\partial C}{\partial t}+v_x\frac{\partial C}{\partial x}+v_y\frac{\partial C}{\partial y}+v_z\frac{\partial C}{\partial z}=\frac{\partial}{\partial x}(D_x\frac{\partial C}{\partial x})+\frac{\partial}{\partial y}(D_y\frac{\partial C}{\partial y})+\frac{\partial}{\partial z}(D_z\frac{\partial C}{\partial z})+S(x,y,z,t,)\pm S_{\text{intern}} \tag{8}$$

C – the concentration, that is the mass of the pollutant per unit volume of water; D_x, D_y, D_z – the coefficients of diffusion in the x, y, and z directions; v_x, v_y, v_z - the components of the river velocity in the x, y, and z directions; t – time;

S(x, y, z, t) - external sources, functions of space and time; S_{intern} - internal sources whose evolution is influenced by C.

The initial condition, t = 0, c = c_0(x) and boundary conditions in the general case, x = 0, c = c_1(t) and x = l, c = c_2(t).

The unidimensional equation in longitudinal direction which describes the surface water quality for a permanent movement, neglecting the diffusion, can be written as:

$$v_x\frac{dC}{dx}=-kC \tag{9}$$

k - is a coefficient of the first order velocity.

The simplified equation who neglects the diffusion and the convection, meaning the dispersion, can be written in the following shape:

$$\frac{dC}{dt}=-kC \tag{10}$$

Since the dispersion equation (8) is more difficult to solve, the diffusion equation can be solved, as it is of the parabolic type, we obtain the solution of the pollutant plume, which differs according to the dispersion parameters and the dispersion relative sizes that appear in Gauss's bell.

This is a Gaussian distribution of the pollutants' concentration in space with time - variant deviation.

$$\chi_i=\frac{x_i-x_{0i}}{\sqrt{\sigma_i}} \tag{11}$$

$\sigma=\sqrt{2D_t}$ - standard deviation of the x variable.

In approaching the diffusion equation the following method is proposed- the separation of variables and the comparison of the function obtained by solving the differential system attached to the Fourier integral equation.

$$C(x,t) = \frac{Q}{\omega v}\frac{1}{\sqrt{2\pi D_t}}e^{-\frac{(x-x_0)^2}{4Dt}} \tag{12}$$

It represents the solution of the pollutants' concentration in a section ω of finite length.

2.2. The equation of pollutants' motion in porous media used in modeling

The most important processes in pollutant transport are diffusion, advection and sorption.

The transport by advection is characterized by water flowing into the ground, controlled by hydraulic conductivity and the hydraulic gradient, expressed by Darcy's law:

$v = -k_H \frac{dh}{dl}$, homogeneous flow, constant section and ρ - constant fluid (13)

v – the rate of infiltration, Darcy; k_H – hydraulic conductivity; h – hydraulic load; l – the length of the current line in porous media;

$\frac{dh}{dl}$ - hydraulic gradient.

The simplest form of linear model of sorption process is: S= K_DC, where S – concentration of the pollutant in the adsorption phase, K_D – the sorption distribution coefficient and C – the pollutant concentration in liquid phase.

According to the law of mass and flow conservation, the general equation that describes the transport of pollutants through porous media with variable saturation and that takes account of the diffusion, convection and sorption processes and chemical decomposition is represented by the dispersion equation:

$$\theta\frac{\partial C}{\partial t} + \rho\frac{\partial S}{\partial t} = div(\theta DgradC) - VgradC - \lambda(\theta C + \rho bS) + QC_{in} - QC \tag{14}$$

C – pollutant concentration; ρ - the volumetric density of porous media; S – the pollutant surface in convection phase; D – dispersion coefficient tensor; V – Darcy velocity vector; λ - constant of pollutants' decomposition; Q –entered flow; C_{in} – pollutant concentration in the injected fluid.

The portion of the domain contour (B_d) is imposed by the limits of pollutant concentrations, called the Dirichlet conditions which are defined as:

C = C_d(x_d, y_d, z_d, t) on B_d limit. (15)

The flux of pollutant limits imposed are the limits where the infiltration takes place, so Cauchy conditions are imposed:

n(VC-θDgradC) = q_c(x_d, y_d, z_d, t) on B_d limit. (16)

C_d – concentration required; x_d, y_d, z_d – coordinates of the contour; n – unit vector normal to the boundary; VC – advection flux; θDgradC – dispersion flux given by concentration gradient; q_c – flux required.

3. MATHEMATICAL MODELS TO STUDY THE MOVEMENT OF POLLUTANTS

The processes representation through mathematical models gained momentum along with the development of informational systems. In order to elaborate the mathematical model of the pollutants' motion in polyphasic liquids equation, the SMS (Surface-water Modeling System) and the SWMM (Storm Water Management Model) systems were conceived.

The RMA4 module (of the SMS soft) uses the Navier – Stokes equation system for turbulent motion in slow regime. It starts from the premise that the vertical distribution of the concentration is uniform and data regarding the water depth (the level of the water), flow velocity, flow and the local pollution source are introduced. The analysis of the pollutant's dispersion was performed in dynamic regime, which allows the estimation of the pollutant's evolution at different time intervals along the free surface.

The SWMM (Storm Water Management Model) calculates the pollutant loading in a sewer network through the TRANSPORT module (it use the Saint-Venant equations for the gradually unidimensional variated motion of the stratified currents). The program models the basic pollutants' dispersion: the entry data, meaning hydraulic information about nodes and pipelines (the nodes coordinates, radier quote in nodes, nodes flow, length, diameter, the section and collector gradient) and information regarding the injected pollutant concentration.

The exit data appear as tabels or graphics and represent the time variation of the pollutant concentration in nodes and pipelines, including the flow orientation of the pollutant between the afferent nodes of the pipeline.

In the analysis of the pollutants transport in porous saturated / unsaturated media, the CONCENTR program uses a hydric aproximation, meaning a regressive scheme of particle movement. The basic data are: the nodes coordinates, the physical-chemical properties of the pollutants, the initial and marginal conditions. The exported information represent the mass flow of pollutant in various types of limits and concentration.

4. CONCLUSIONS

The mathematical models reflect the static and/or dynamic behavior of the process regarding the identification of the optimal functional areas.

The pollutants transport in polyphasic liquids and porous media is characterized by the convection and diffusion of pollutant constituents. Porous media are also influenced by sorption either absorption and/or adsorption, either by the organic carbon fraction within the earth, or by adsorption on the surface of the mineral particles in the earth.

Three distinct computing models were presented, through which information for a single situation is processed. Due to the computing algorithm, there are differences in the input and output of basic data and in the imposed conditions for result achievement.

This pollutant transport matter will be approached in future by making a case study of dynamic behavior of pollutants in prismatic channels.

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Analysis of a wastewater settling tank using CFD methods

Michal Holubec

***Abstract* – The paper is focused on CFD modeling (Computational fluid dynamics) of a primary settling tank at the WWTP in municipality of Humenné. The primary sedimentation tanks are structures of mechanical treatment, before biological activation. The settling tanks are of rectangular shape, which is a common sight at WWTPs in Slovakia. To achieve the most effective operation concerning the proper technological processes and cost saving, we need to find the best solution for hydraulic conditions in these facilities. The CFD offers solutions without the need of building a physical model.**

***Keywords* – CFD, Settling tank, WWTP,**

1. INTRODUCTION

Present Slovakia waste water treatment plants (WWTP) treat the waste waters from about 60% of inhabitants, which are connected to the public sewer system. Many of the WWTP's use more than 40 years old equipment and need reconstruction. The hydraulic design of these reconstructed structures is mostly based on empirical or semi-empirical methods. Using CFD methods, we were able to analyze one such reconstructed structure. Very often we can find sub-optimal design, mostly on rectangular tanks, which means less effective operation of the treatment process and higher operational expenses. The analysis consisted of simulation using ANSYS CFD software supported by in situ velocity measurements.

Based on our research and experiences from WWTP operators in Slovakia we can declare that there are a lot of WWTP's which have hydraulic problems in their technological processes, even after reconstruction. This paper focuses on CFD analysis of a wastewater settling tank located at a WWTP in the municipality of Humenné, Eastern Slovakia.

The wastewater treatment plant is designed to process wastewater from 96700 equivalent residents and works in two stages – mechanical treatment and biological treatment. The wastewater is transported via a combined sewer system, with maximum inflow of 1050 l.s-1 (1922 l.s-1 during storm events). Storm water tanks and the storm water treatment process line are in place to deal with storm water inflow. This analysis focuses on the primary settling tank at Humenné WWTP. It is a rectangular tank with horizontal flow. The wastewater is transported from the grit chambers through a concrete conduit, perpendicular to the flow direction in the tank. The water flows from the inflow chamber through ten T-shaped steel pipes DN300, which are located on the front wall of the tank, 1622 mm above the tank bed. There is also a small, 600 mm wide floodgate in the same wall. There are 4 holes for collecting the sludge at the bottom of the tank in the inlet zone. The tank is 36 meters long and 12 meters wide, the maximal height of water surface is 3,72 meters, according to the project documentation. The volume of the tank is divided by a 3,55 meters tall and 300 mm thick concrete wall which is situated in the middle of the tank. The tank is equipped with a moving bridge with sludge scrapers. The sludge is hauled into the holes at the inlet zone and pumped out of the tank with sludge pumps. The treated water flows out of the tank through a weir at the far end. The schematics of the tank are shown in Figure 1.

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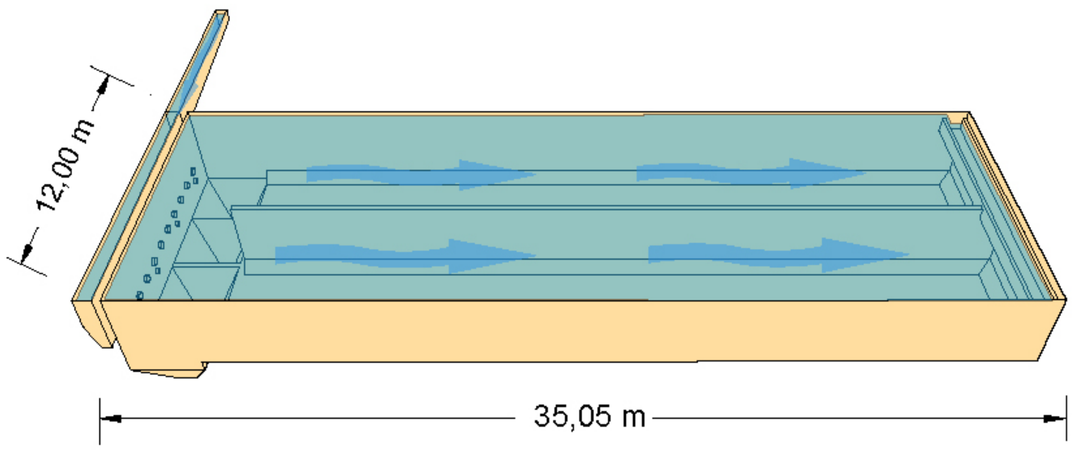

Figure 1 Schematics of the settling tank

The settling tank was simulated in ANSYS Fluent using the default energy equation model and standard K-omega viscous model. The 3D geometry was created in AutoCAD, based on project documentation and measurements and then imported to ICEM CFD, in which the computational mesh was created. Several simplifications were employed for defining the settling tank to conserve time and computational capacity. The surface of the tank was defined as a zero shear stress wall boundary condition as opposed to free surface simulation, which simplifies the surface of the tank to a simple plane with no friction. This means that the surface remains at constant height during the simulations, similar to normal operating conditions in the tank. The height of the surface plane was determined based on field measurements at the site, and so was the flow rate/velocity at the inlet boundary. The weir was simulated as an outflow boundary condition (there is no backflow calculated on this boundary). There were two types of walls with different roughness coefficients to simulate differences between the concrete tank and the steel pipes. Settling tank was simulated with „pure water", meaning without sediment transportation. K-omega turbulence model was used for its precision with low Reynolds number turbulent flows. Four alternative configurations of the settling tank were simulated, to help analyze the flow patterns in the tank. These alternatives are described in Table 1. CFD POST application was used for processing of the results and creating the output images.

Table 1 Description of simulation alternatives

1.1	Simulation of real settling tank
1.2	Simulation of the tank with closed floodgate
1.3	Simulation of the tank with closed floodgate and without the separation wall
1.4	Simulation of the tank with closed floodgate with the separation wall elevated above the water surface

2. METHODS AND RESULTS

The first step in the process of CFD simulation is to create the mesh which will be used for the computations. The mesh for this problem was created in ICEM CFD, based on geometry imported from AutoCAD (Figure 3). All the meshes for the various alternatives are composed exclusively of triangles and tetrahedrons, details can be seen in Table 2.

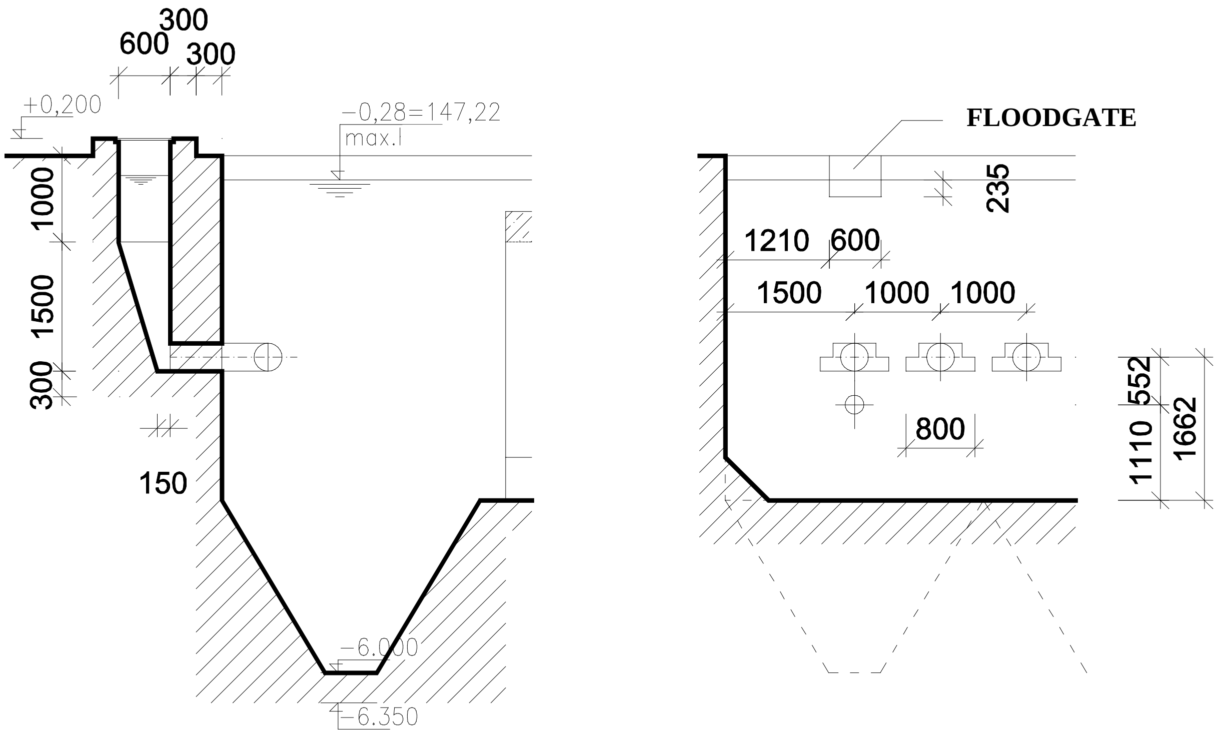

Figure 2 Inlet chamber and inlet pipes

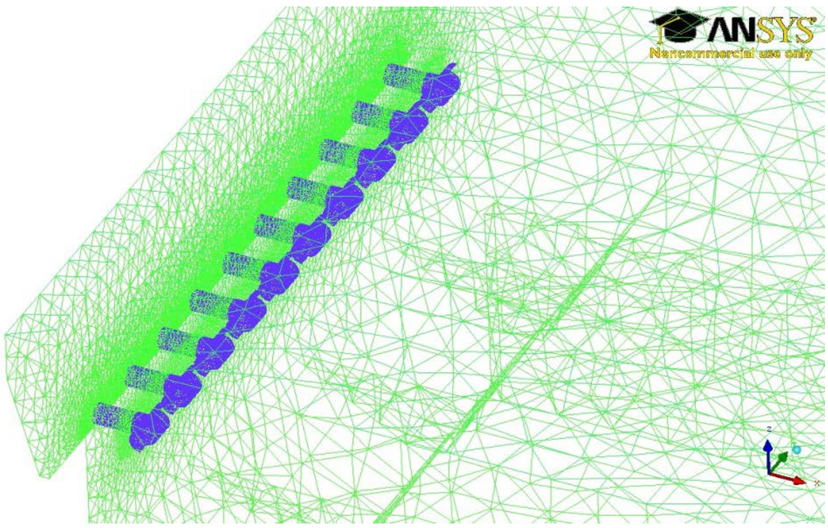

Figure 3 Example mesh, pictured is the inlet zone for alternative 1.1

Table 2 Compositions of meshes for the alternatives

	No. Of elements	No. Of nodes	Triangle	Tetrahedron
1.1	1243764	214403	73524	1163370
1.2	1212514	208965	70880	1134929
1.3	1210318	208615	70800	1132816
1.4	1213098	209019	69762	1136935

After generation, the mesh is loaded into the FLUENT software. The boundary conditions were defined according to Table 1.

Table 3 Boundary conditions

Boundary condition	Specification
velocity inlet	v_{st}=0,276 m.s-1 for the floodgate v_r=0,05 m.s-1 for inlet pipes
outflow	-
wall (surface)	Zero shear stress
wall (concrete)	Standard wall function
wall (pipes)	Standard wall function
fluid	Defined as h2o<l>, T=298°K

The result of the simulation is a lot of information and simulated data, such as velocity, turbulent kinetic energy, pressure, turbulent viscosity, etc.

The results of simulations were evaluated based on three main types of graphical output:

- Velocity streamlines which help us identify some of the hydraulic phenomena in the tank. Allow a better understanding of the flow patterns in the tank.
- Speed contours, which show the velocity field in the tank at user-defined locations.
- Isosurface - connecting points with the same value in the volume of the tank.

The parameter of limit velocity (v_H) was derived for the purpose of evaluation of the different variations of the tank. This velocity is the horizontal component of the velocity vector in the longitudinal direction (in relation to the reservoir). Is derived from the calculation of residence time t [h] proposed by the Slovak standard STN 75 6401. According to the formula:

$$v_H = \frac{L_1}{t} \tag{1}$$

$$z_L = \frac{34{,}9}{3} = 11{,}63\ m.h^{-1} = 0{,}0032\ m.s^{-1} \tag{2}$$

where L_1 is the effective length of the tank and t=3 hours is the residence time for minimal inflow, proposed by the standards. Isosurfaces of this parameter were used with the graphical analysis of the tank.

Simulation 1.1

This is a simulation of the real settling tank, according to project documentation and on-site measurements. The shape of the inlet pipes coupled with the perpendicular direction of the inflow channel directs the flow in the inlet zone to the right (Figure 4, Figure 5). As a result, we can see higher velocities in the sludge holes on the right-hand side, which could potentially lead to re-suspension of the settled sediments (Figure 6). In general, the velocities in the right-hand side of the tank are higher, which can be seen in Figure 7. There is a visible surface current, running from the floodgate directly to the outlet weir which could have a serious negative impact on function of the tank.

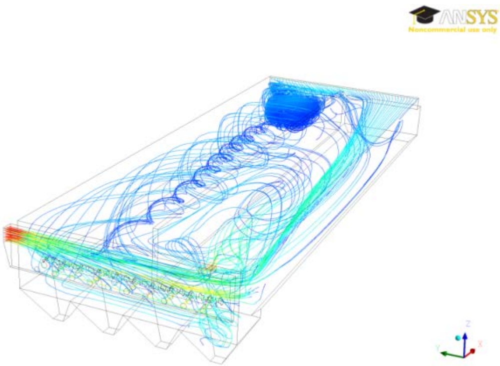

Figure 4 Velocity streamlines (simulation 1.1)

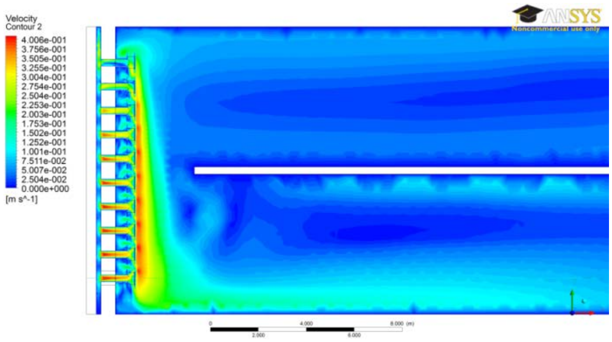

Figure 5 Velocity contours, top view (simulation 1.1)

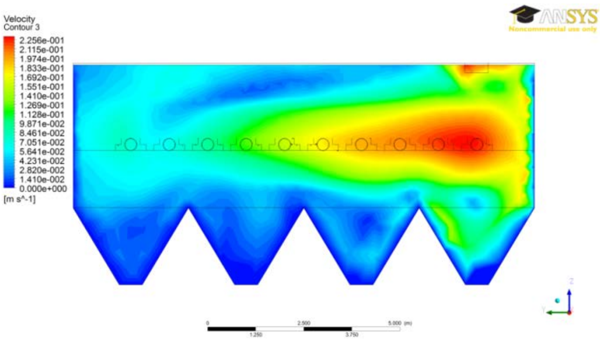

Figure 6 Velocity contours, view of the sludge holes (simulation 1.1)

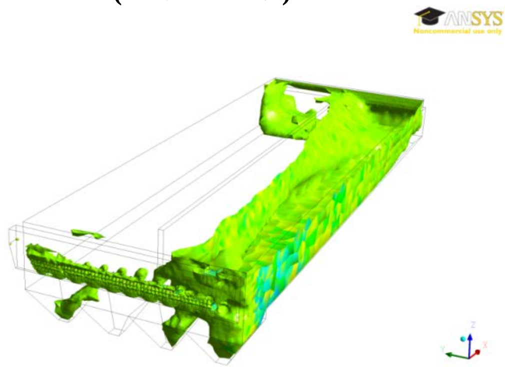

Figure 7 Isosurface for the v_H parameter (simulation 1.1)

Simulation 1.2

This simulation is almost identical to 1.1, the only difference is the closed floodgate. The flow patterns are similar to 1.1 (Figure 8, Figure 9) including the velocities in the sludge holes. The flow is slower at the surface thanks to the closed floodgate.

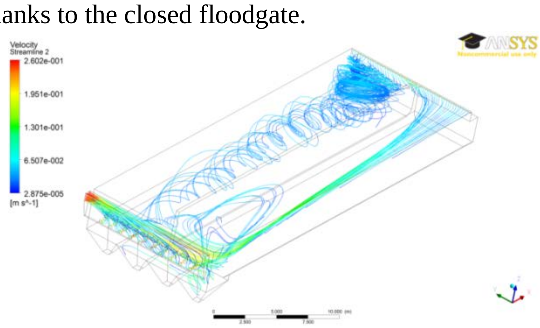

Figure 8 Velocity streamlines (simulation 1.2)

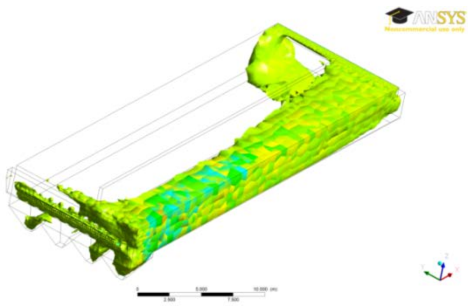

Figure 9 Isosurface for the v_H parameter (simulation 1.2)

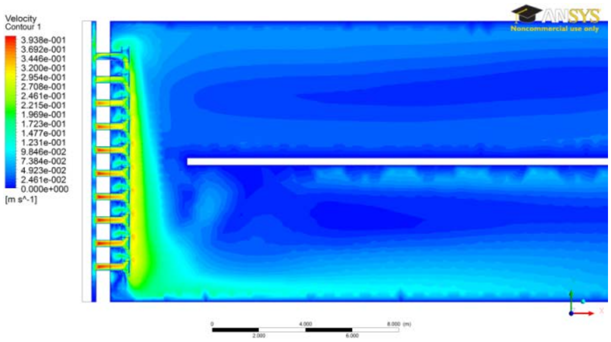

Figure 10 Velocity contours, top view (simulation 1.2)

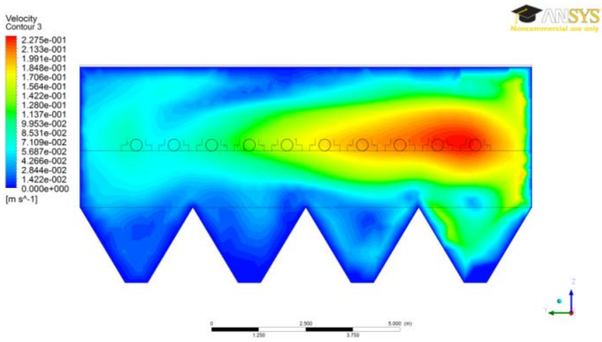

Figure 11 Velocity contours, view of the sludge holes (simulation 1.2)

Simulation 1.3

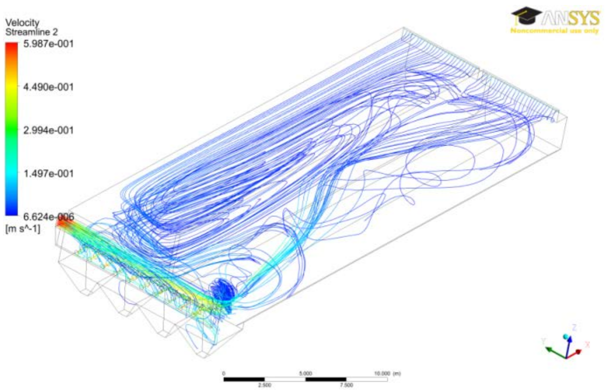

Figure 12 Velocity streamlines (simulation 1.3)

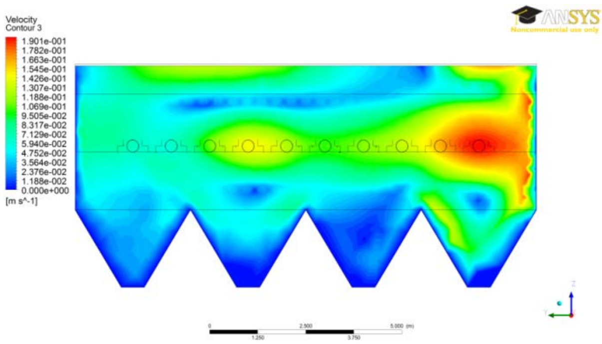

Figure 13 Velocity contours, view of the sludge holes (simulation 1.3)

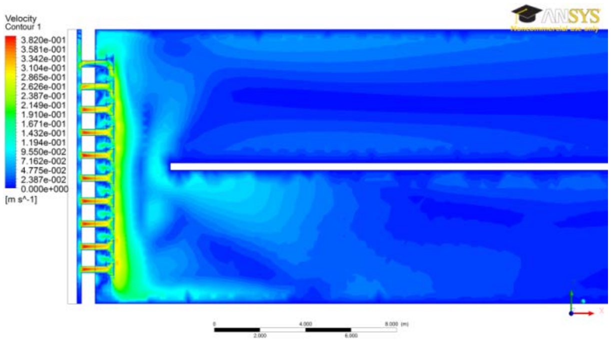

Figure 14 Velocity contours, top view (simulation 1.3)

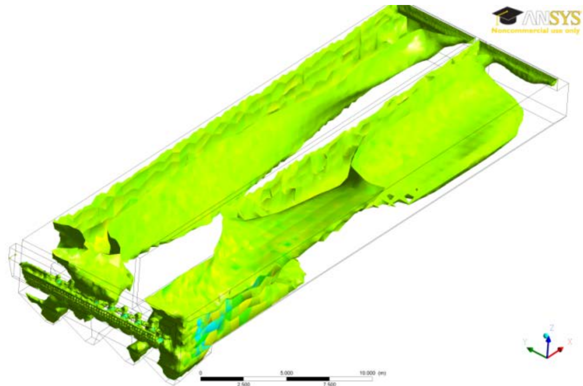

Figure 15 Isosurface for the v_H parameter (simulation 1.3)

This variant simulates extending the separation wall above the surface. As a result, the flow pattern has slightly changed and overall, velocities in the tank grew (Figure 14). The problem in the sludge hole still persists (Figure 13)

Simulation 1.4

Variant 1.4 simulates the tank without the separation wall in the middle. Removing the wall creates a right-hand rotating flow system (Figure 16) due to the shape of the inlet pipes and the perpendicular inflow channel. The limit velocity (v_H) has been exceeded in a large part of the tank as seen in Figure 19.

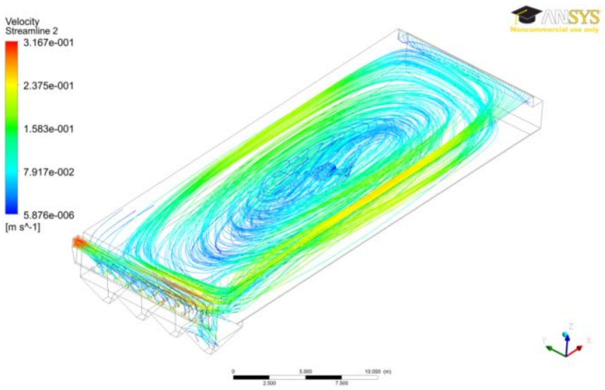

Figure 16 Velocity streamlines (simulation 1.4)

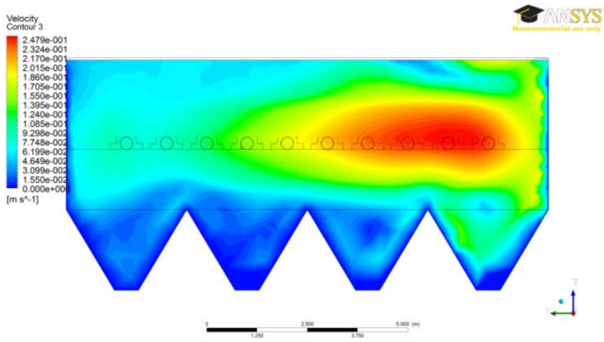

Figure 17 Velocity contours, view of the sludge holes (simulation 1.4)

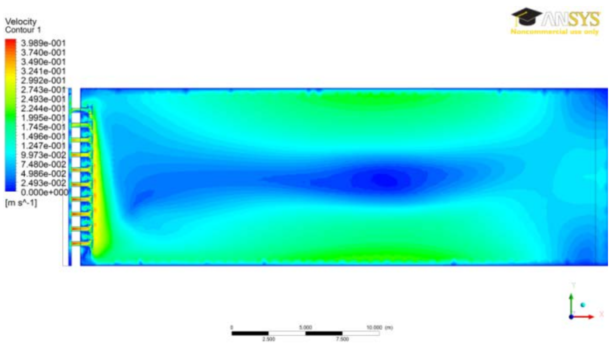

Figure 18 Velocity contours, top view (simulation 1.4)

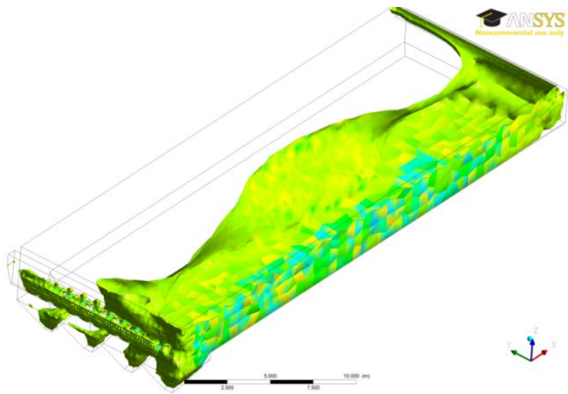

Figure 19 Isosurface for the v_H parameter (simulation 1.4)

3. CONCLUSIONS

All four simulations point to the direction of inflow as the strongest negative influence on the flow in the tank. The biggest problem – increased velocities in the sludge holes on the right side of the tank – is caused by the perpendicular inflow of wastewater into the tank from the left side. The velocities in the sludge holes were a problem in all of the four scenarios, regardless of the changes. These results indicate possible hydraulic problems, which could have a negative influence on overall performance of the tank.

However, another simulation series with particle transport and sedimentation is needed to get conclusive results. As a side note, based on the outcomes, we definitely advise the operator to operate the tank with closed floodgate, as this may lead to short-circuit currents and low settling efficiency.

This paper focuses on the simulation of hydraulics of wastewater treatment settling tank. The settling tank in WWTP Humenné was selected based on the survey of operators of wastewater treatment plants. The WWTP was recently reconstructed and therefor new documentation was available. The work also touches on the subject field measurements of discharge in settling tanks. The work focuses on modeling of various alternatives of the tanks geometry, in order to find the most effective configuration for its operation. ANSYS FLUENT CFD model was utilized for all simulations.

4. Acknowledgments

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5. References

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