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Parametric Analysis of a one Degree of Freedom System Subjected to Seismic Motions

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Rezumat: S-a considerat oportună realizarea unui studiu parametric comparativ cu răspunsul unui sistem cu un grad de libertate supus unor excitații seismice şi armonice. Excitațiile sunt reprezentate printr-un număr de opt accelerograme specifice unor cutremure majore din țară şi din străinătate, cu intensă activitate tectonică şi sunt reprezentative zonelor intens locuite. În urmă cu mulți ani, a început derularea acestui program de cercetare în cadrul Stației de Cercetări Seismice INCERC Iaşi, în colaborare cu Facultatea de Construcții de la Universitatea Tehnică din Iaşi. O atenție deosebită s-a acordat comportării histeretice, evoluției disipării de energie. Răspunsul inelastic al structurii este influențat de trei modele, patru perioade de oscilație, două tipuri de amortizări (vâscoasă şi de tip Penzien), trei valori ale procentului din amortizarea critică, de rigiditatea ramurii post elastice ascendente.

Abstract: A research program intended to investigate the dynamic and seismic nonlinear inelastic behavior, useful for reinforced concrete structures acted beyond the elastic limits has been started a long time ago, in the Building Research Centre from Jassy. A special attention has been devoted to the hysteretic behavior, to the evolution of energy dissipation in different steps of the timehistory. The analysis has been made taking into account the model configuration, the slope ratios of loading and unloading branches of the hysteretic loops, the combined viscous and the hysteretic Penzien type damping factors, different seismic actions, and other parameters. The analysis has been carried out on an equivalent inelastic one degree of freedom system with variable characteristics. The paper is restricted only to some aspects concerning the damping force variation and spectral values response.

Keywords: *dynamic and seismic actions***,** *inelastic response, hysteretic behavior, damping factors.*

1. Introduction

In order to select the main parameters which are relevant in the time–history response of a structure subjected to a set of eight earthquake–like motions and harmonic actions, the authors have considered an equivalent yielding one degree of freedom system provided both with viscous Voigt damping and also with a proportional with displacement Penzien type of dissipation. In this way, a combined damping force, proportional with the velocity and with the displacement, in different rates, could be taken into account. In an attempt to estimate the major factors interfering in the inelastic response of the mono–massic structure and their influence on the peak values of the dynamic and seismic inelastic response, the study has been restricted to a few number of natural periods and damping values, slope ratios of the loading and unloading branches, inelastic configurations of the hysterezis loops, coefficient of post-elastic solicitation, and other relevant parameters. Nevertheless, the range of the considered parameters proved to be in good agreement with the most frequently encountered reinforced concrete buildings subjected to seismic excitations stressed beyond the elastic limits. The simulated excitations comprise accelerograms of some major earthquakes recorded in our country and abroad, which are specific for areas revealing strong tectonic activity. For comparison, a sinusoidal excitation with a reference *2c/s* frequency, which was modeled in order to obtain other values of the circular frequency by changing the time scale, has been also included.

2. Research Programme

In order to investigate the influence of the type of seismic motion, natural period, damping of the one degree of freedom system, also the influence of shape

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and other characteristic elements of the hysteretic loop, a number of 400 runs have been undertaken trying to draw some conclusions about the inelastic response of the single degree of freedom equivalent system throughout the time-history of the dynamic action. The seismic motions with their main characteristics are listed in Table 1, inclusively the harmonic sinusoidal action. The duration, the time step, the number of discrete points and the peak accelerations of the earthquake motions are also presented in that table.

Tabel 1 The seismic motions

No. crt.	Naming	Description of Earthquake	Duration (s)	No. of points	Δt (s)	$\left\|a_0^{max}\right\|$ (cm/s²)
0	**1**	**2**	**3**	**4**	**5**	**6**
1	**BucNS**	**Bucharest, March 4, 1977, component NS**	**40.1**	**4010**	**0.01**	**194.927**
2	BucEW	Bucharest, March 4, 1977, component EW	7.8	780	0.01	163.087
3	**El Centro**	**El Centro, May 18, 1940, S00E**	**53.76**	**2688**	**0.02**	**341.7**
4	Mexico	Mexico City, Sept. 19, 1985, SCT Station, S60E	120	9006	0.02	167.918
5	Sin	Harmonic Action with 2 Hz frequency	10	1000	0.01	200
6	BNS	Bârlad, Aug. 30, 1986, component NS	7.8	780	0.01	165

The runs have been undertaken with a program elaborated especially for this purpose. This program succeeded to simulate the seismic and non–seismic response of the response spectra in elastic and inelastic range considering a number of three hysteretic models: bilinear elastic–perfectly plastic behavior, bilinear hysteretic Clough (Q_{hyst}) model with degrading behavior $\alpha \neq 0$ (fig. 1), bilinear non–degrading hysteretic Clough model (fig. 2) $\alpha=0$, elastic model without plastic deformations. Since the number of parameters defining the inelastic structural response in the time history of the motion is rather large, in the present analysis only a restricted number of parameters, which are relevant in the study, have been considered [2, 3].

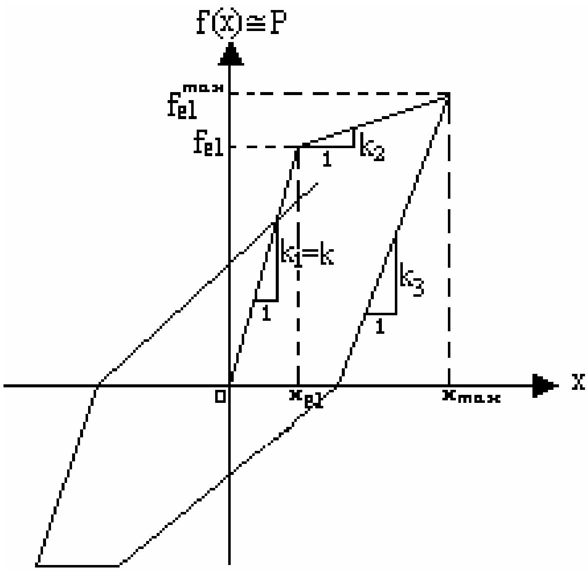

Fig 1 The bilinear Q_{hyst} model with degradation

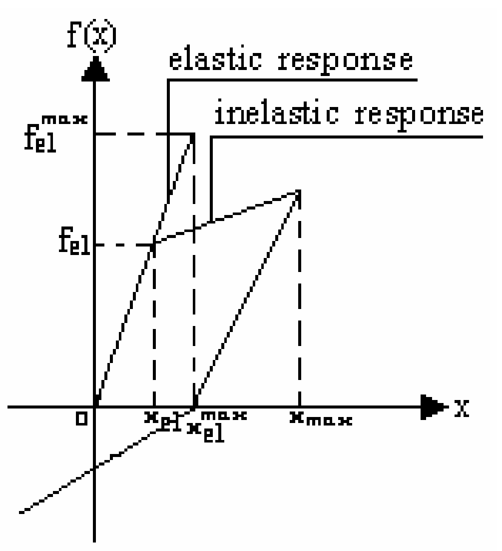

Fig. 2 The bilinear non degrading inelastic Clough model

Most part of the tested structures have been cast-in-place and prefabricated reinforced concrete structures, scaled to 1/2 and 1/5 size, but also some of them have been constructed full–scale. Incidentally, metal structural models have been also investigated, finally concluding that results are in good agreement with the analytic response in displacements, velocities and accelerations. We mention that, in the last case, the use of Ramberg–

Osgood two parametric curves lead to a close coincidence.

The selected variable parameters adopted in the study are the following:

1. A number of four natural periods of the one degree of freedom oscillator in the *(0.5÷2.0)s* range

2. A number of two types of damping ratios: the ''classical" equivalent viscous damping fraction proportional with velocity (Voigt damping component) and another damping component, proportional with the displacement of Penzien type, which have been considered in different proportion
3. The slope or stiffness of the post-elastic loading (ascending) branch of the hysteretic relation
4. The slope of the unloading or descending branch at load reversal
5. The configuration of the elastic-plastic model, of Clough and Q_{hyst} type. In such a way, the inelastic models simulate the non-degrading and also the degrading behavior
6. The harmonic SIN excitation in the range from *0.10* up to *2.0s*.

In most of the undertaken runs, for every type of model and earthquake, some constant parameters have been maintained constant, such as: the mass value, the design seismic peak acceleration, the time increment, the limit elastic force (which must to be smaller than the reference elastic force, the initial velocity $v_0=0$ and the elastic solicitation factor:

$$\mu_0 = \frac{f_{el}}{m \cdot a_0^{max}} \tag{1}$$

For the same seismic or harmonic action, the value of parameter μ_0 expressing the ratio between the limit elastic force and the maximum elastic seismic force is constant. The resulting variable response elements are following: the Fourier spectrum, the maximum velocity, the natural period T, the maximum elastic force, the post–elastic excursion factor $\mu_1<1$, the maximum spectral values of displacement, velocity, acceleration, the variation of damping force and of the restoring force during the time–history of the motion. The limit elastic force has been arbitrarily adopted, as a study case, equal to *1000daN*. As a rule, the smaller is the ratio μ_0, the greater will result the degree of degradation of the structure, the more advanced will be the level of damage. On the contrary, if this parameter is greater, we are situated in the neighborhood of the elastic behavior, corresponding to purely viscous damping, without the damping component, which is proportional with

the displacement. The values of linear spectral response are not depending upon the slope of the ascending branch k_2. The incremental time is:

$$\Delta t \le \frac{T}{50} \tag{2}$$

The numerical analysis is made for *1000* points. The sine force has been simulated in the program by a sinusoidal reference frequency of 2c/s which can be converted also to other input frequencies. For this purpose, different values of the incremental time Δt are taken in the analysis, according to discretization goals in order to reach a good accuracy of the numerical analysis. As it is known, applying the d'Alembert principle, the dynamic equilibrium for the one degree of freedom system with viscous damping is given by:

$$m\ddot{x}(t) + c\dot{x}(t) + k\,x(t) = -ma_0 \tag{3}$$

In the case of the system featuring inelastic behavior, after reaching the elasticity limits, the function *f(x)* becomes a variable complex non–linear parameter along the time–history of the motion, as a consequence hysteretic loops are changing their loading and unloading curves. This type of behavior is usually called in the case of seismic actions, low cycle fatigue. Equation (3) can be written under the form:

$$m(\ddot{x} + a_0) + fa + f(x) = 0 \tag{4}$$

where: *fa* is the damping force, which can be, as in our case, a combination between the Voigt viscous component and the Penzien type damping proportional with the displacement, taking into account inelastic hysteretic characteristics.

Equation (4) can be also written as equation (5), called the characteristic equation for the inelastic behavior with viscous damping:

$$ma + \frac{\nu kt}{\pi} v + f(x) = 0 \tag{5}$$

The expression adequate for the present study, in the case of the one-degree of freedom system has the form:

$$ma + \frac{\nu_1 kt}{\pi} v + \pi \nu_2 k |x| \operatorname{sgn} v + f(x) = 0 \tag{6}$$

where: $\operatorname{sgn} v = \begin{cases} 1, & for\ v > 0 \\ 0, & for\ v = 0 \\ -1, & for\ v < 0 \end{cases}$ (7)

Therefore we obtain:

$$\operatorname{sgn} v = \frac{v}{|v|} = \frac{\dot{x}(t)}{|\dot{x}|} \tag{8}$$

The previous relations characterize the inelastic system having combined damping namely:

- viscous Voigt damping;
- damping force of the second type, which is proportional with displacement and in phase with the velocity.

In the solution of the equation system, interferes the amplitude Fourier spectrum. If the seismic motion has a duration from $\tau=0$ up to $\tau=t_n$, the amplitude Fourier spectrum is given by the relationship:

$$|F(\omega)| = \sqrt{\frac{2E(t_n, \omega)}{m}} \tag{9}$$

The ductility factor associated to an oscillation cycle is:

$$\mu_i = \frac{x_{\max}}{x_{el}} \tag{10}$$

As for the stiffness decrease, Clough and other authors have assumed for the slope k_3 at load reversal the relationship:

$$k_3 = k_1 \left(\frac{x_{el}}{x_{\max}} \right)^{\alpha} \tag{11}$$

where, $\alpha = (0.4 \div 0.5)$.

In the present analysis, this index has been considered equal to 0.4, valid for usual concrete grade. If α=0 we reach to the bilinear elastic–perfectly plastic model, where $k_3 = k_1$. In that case, load reversal takes place after a line parallel with the linear ascending (loading) line, according to Gerstner assumption.

$$\mu_0 = 1.026 \tag{12}$$

3. Some Results of the Parametric Analysis

Let consider the El Centro Seismic Motion, S00E component and the Bucharest Seismic Motion, NS component (Table 1). If we adopt a Q_{hyst} inelastic behavior, the results of the simulation are those given in Table 2 and Table 3.

Together with the increase of the oscillation period the rigidity lowers presenting the same values

as in the previous case (Table 2). The maximum elastic forces falls in relation to the period increase *T=(0.5÷2.0) s* and an increase is registered for the absorption $\nu_1=15,9\%$.. In parallel with the appearance of the absorption $\nu_1=\nu_2=2.5\%$, the same descending tendency is registered in relation to the period, but the values are superior when $\nu_1=5\%$, $\nu_2=0\%$ (Fig. 3).

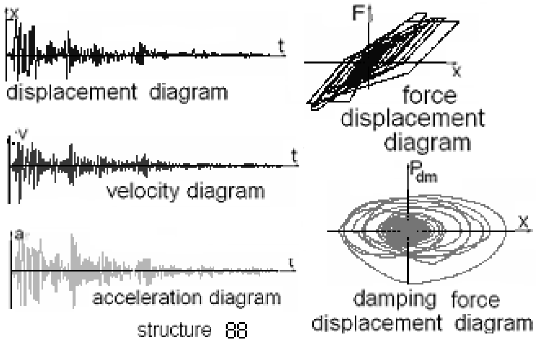

Fig. 3 Inelastic timehistory for different damping rates El Centro seismic motion

The maximum movement of $x_{el}^{\max}$ present alternative increase and descending in the analyzed field and is influenced by the value of the critical absorption factors ν_1 and ν_2. According to the rise of the period the absolute values of the movement specters *Sd* increase and of the *Sv* speed, but no significant variations are noticed according to the type and value of the absorption factors. The diagrams of absorption forces depend upon the absorption ν_1 and present similar shape, differentiated for the absorption structures of type *1* (balls with great initial forays), as well as the absorption structures ν_2 (type *2* papillon shape the linear contour Fig. 4).

The degree of μ_1 post elastic stress, generally reaches supra unitary values, showing that the structure works from the very beginning above the elastic limits, performing post elastic forays and generating an imminent collapse. The degree of μ_0 stress is the relation between the elastic force limit value and the one of the basic seismic force having the value of *0.585*. Taking into account that in certain cases $\mu_1>1$ in a comparative survey the limit elastic force should be developed, that is

redimensioning of the structure subject of a strong earthquake.

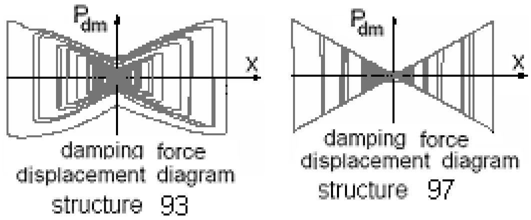

Fig. 4. Inelastic timehistory for different damping rates El Centro seismic motion

Let consider the Bucharest Seismic Motion, NS component. If we adopt a Q_{hyst} inelastic behavior, the results of the simulation are those given in Table 3. We can see, for instance, that if natural period is increasing, in the interval *(0.5÷2)s*, the stifness k_2 of postelastic range is significantly decreasing. The maximum elastic forces become higher in the range *T=(0.5÷1)s* and lower in the range *T=(1.5÷2)s*, irrespective of the damping. The maximum recorded accelerations when running the NS component present rather small fluctuations in terms of the period, but are significantly different in terms of the two types of damping, i.e.

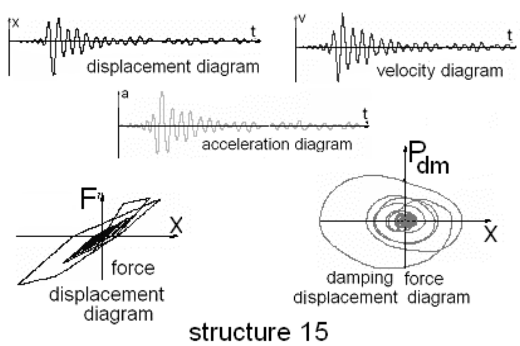

Fig. 5. Inelastic timehistory for different damping rates Bucharest, NS component seismic motion

From the above given tables, we can see that damping force graphics f_{am} are dependent of, ν_1 revealing a characteristic shape for the model with viscous Voigt damping only, in that case of *5%,* when the second type of damping is missing, the structure displaying large amplitude excursions in the post–elastic domain. In this case, the configuration of the resulting curves is like clew irregular loops, for 1÷5, 14÷17 and others. (Fig. 5)

In the case of the runs 6÷13 and 18÷21, when damping is a combination between viscous and Penzien type components, excepting structures 10÷13 at which damping is purely of the second type, the influence of the damping proportional to the displacement amplitude component is put into evidence by a damping force variation of "butterfly" or "papillion" (Fig. 6) shape, having a pinching in the zone of small displacement amplitudes.

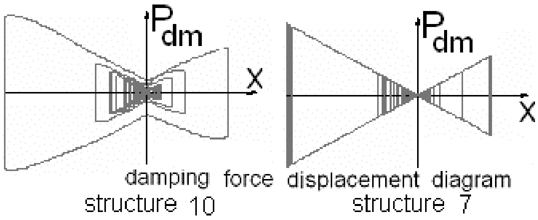

Fig. 6. Inelastic timehistory for different damping rates Bucharest, NS component seismic motion

The ''bow–tie" and/or the ''butterfly" shape of the curves during the time–hystory of the motion depends upon the damping 1 and damping 2 rates, as it can be seen in the NS graphics, after running the programs with the considered parameters.

The same statements are valid for other inelastic model types (bilinear yielding Clough model with $k_3=k_1$, elastic–perfectly plastic with $k_2=0$, etc.), which have been ruled in the study. Depending upon the amount of the considered dampings (viscous or hysteretic Penzien ones), the considered types of damping. The clew type is specific for viscous damping, the butterfly shape to a combination of inelastic viscous and hysteretic Penzien energy dissipation and the bow–tie configuration with straight line shape corresponds to the pure Penzien type of damping force.

Tabel. 2 The El Centro seismic motions with their main characteristics

Nr. crt.	$\sqrt{2E(t)/m}$ (cm/s)	SF	T (s)	k (daN)	ν_1 (%)	ν_2 (%)	c_1 (daN s/cm)	c_2 (daN/cm)	k_2 (%)	x_{el}^{max} (cm)	f_{el}^{max} (daN)	μ_1	Sd(T) (cm)	Sv(T) (cm/s)	F_{am}^{max} (daN)	a_{max} (cm/s^2)	Pd_m (daN)
0	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
1	77.74	63.283	0.5	7895.7	5	0	62.832	0	0	2.393	18891.1	0.529	-7.482	-29.16	-1832.14	246.51	-10713.1
2	106.67	63.158	1.0	1973.9	5	0	31.416	0	0	-14.205	-28039.6	0.357	-25.185	-70.88	-2226.61	250.92	-10681.5
3	301.17	28.915	1.5	877.3	5	0	20.944	0	0	30.859	27072.2	0.369	-32.945	-88.9	-1861.87	237.97	-10225.2
4	174.22	112.414	2	493.5	5	0	15.708	0	0	39.518	19501.4	0.513	-37.907	-105.99	-1664.93	229.74	-10205.8
5	106.67	63.158	1	1973.9	15.9	0	99.903	0	0	-10.549	-20822.5	0.480	-13.896	-50.58	-5053.15	282.78	-10522.9
6	77.74	63.283	0.5	7895.7	2.5	2.5	31.416	620.126	0	2.258	17827.3	0.561	-5.414	-27.85	-3471.17	286.39	-10848.1
7	106.67	63.158	1.0	1973.9	2.5	2.5	15.708	155.031	0	-13.812	-27264.7	0.367	-22.322	-68.63	-3662.92	285.86	-10627.0
8	301.17	287.915	1.5	877.3	2.5	2.5	10.472	68.903	0	30.544	26796.3	0.373	-32.8	-90.47	-2525.93	262.18	-10585.1
9	174.22	112.414	2	493.5	2.5	2.5	7.854	38.758	0	39.963	19720.9	0.507	-38.318	-108.69	-1721.04	244.29	-10493.4
10	77.74	63.283	0.5	7895.7	0	5	0	1240.25	0	-2.18	-17214.9	0.581	-4.534	-27.74	5623.85	331.21	-10941.3
11	106.67	63.158	1.0	1973.9	0	5	0	310.063	0	-13.598	-26841.5	0.373	-20.709	-67.91	6421.24	338.33	-10500.3
12	301.17	287.915	1.5	877.3	0	5	0	137.806	0	-30.381	-26653.4	0.375	-31.872	-90.1	469.1	296.04	-10410.5
13	174.22	112.414	2	493.5	0	5	0	77.516	0	40.458	19965.2	0.501	-38.463	-110.0	2981.49	264.54	-10245.5
14	77.74	63.283	0.5	7895.7	5	0	62.832	0	15	2.393	18891.1	0.529	-4.134	-21.87	-1374.3	284.92	-14056.4
15	106.67	63.158	1.0	1973.9	5	0	31.416	0	15	-14.205	-28039.6	0.357	-22.457	-73.3	2302.91	325.88	-15916.7
16	301.17	287.915	1.5	877.3	5	0	20.944	0	15	30.859	27072.2	0.369	-33.709	93.89	-1966.34	278.4	-13346.0
17	174.22	112.414	2	493.5	5	0	15.708	0	15	39.518	19501.4	0.513	-38.494	-108.08	-1697.76	237.43	-11393.6
18	77.74	63.283	0.5	7895.7	2.5	2.5	31.416	620.126	15	2.258	17827.3	0.561	-3.751	-24.08	-242.01	324.49	-13858.3
19	106.67	63.158	1.0	1973.9	2.5	2.5	15.708	155.031	15	-13.812	-27264.7	0.367	-20.333	-71.07	3349.47	369.93	-15272.9
20	301.17	287.915	1.5	877.3	2.5	2.5	10.472	68.903	15	30.544	26796.3	0.373	-33.083	-94.84	-2575.76	314.74	-13352.6
21	174.22	112.414	2	493.5	2.5	2.5	7.854	38.758	15	39.963	19720.9	0.507	-38.381	-108.69	-1721.04	244.29	-10493.4
22	77.74	63.283	0.5	7895.7	0	5	0	1240.25	15	-2.18	-17214.9	0.581	-3.848	-28.12	4772.21	389.44	-14719.6
23	106.67	63.158	1.0	1973.9	0	5	0	310.063	15	-13.598	-26841.5	0.373	-19.042	-70.3	-5904.25	413.60	-14775.8
24	301.17	287.915	1.5	877.3	0	5	0	137.806	15	-30.381	-26653.4	0.375	-33.072	-95.19	-4557.47	352.06	-13045.6
25	174.22	112.414	2	493.5	0	5	0	77.516	15	40.458	19965.2	0.501	-39.144	-112.32	-3034.26	291.31	-11531.2
26	77.74	63.283	0.5	7895.7	5	0	62.832	0	30	2.393	18891.1	0.529	-2.988	-17.61	-1106.19	296.53	-14724.8
27	106.67	63.158	1.0	1973.9	5	0	31.416	0	30	-14.205	-28039.6	0.357	-20.339	84.51	2655.1	402.45	-19900.3
28	301.17	287.915	1.5	877.3	5	0	20.944	0	30	30.859	27072.2	0.369	-33.742	99.10	2075.58	331.29	-16226.4
29	174.22	112.414	2	493.5	5	0	15.708	0	30	39.518	19501.4	0.513	-38.826	-110.15	-1730.25	262.13	-12852.4

Tabel. 3 The Bucharest, NS component seismic motions with their main characteristics

Nr. crt.	$\sqrt{2E(t)/m}$ (cm/s)	SF	T (s)	k (daN)	ν_1 (%)	ν_2 (%)	c_1 (daN s/cm)	c_2 (daN/cm)	k_2 (%)	x_{el}^{max} (cm)	f_{el}^{max} (daN)	μ_1	Sd(T) (cm)	Sv(T) (cm/s)	F_{am}^{max} (daN)	a_{max} (cm/s^2)	Pd_m (daN)
0	**2**	**1**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
88	939.62	184.841	0.5	7895.7	5	0	62.832	0	0	5.133	40527.4	0.197	5.458	-50.57	-3177.32	-256.38	9857.2
89	1440.6	833.23	1.0	1973.9	5	0	31.416	0	0	-12.782	-25230.8	0.317	9.794	-56.93	-1788.62	-223.3	10148.9
90	947.79	900.986	1.5	877.3	5	0	20.944	0	0	-10.599	-9298.10	0.860	-10.654	46.78	979.70	175.23	-8356.9
91	1167.8	1028.49	2	493.5	5	0	15.708	0	0	-17.658	-8714.0	0.918	-17.701	-60.21	-945.79	168.63	-8107.5
92	1440.6	833.23	1	7895.7	15.9	0	99.903	0	0	6.605	13037.8	0.614	6.73	-51.85	5179.73	-240.28	9726.9
93	939.62	184.841	0.5	7895.7	2.5	2.5	31.416	620.126	0	4.913	38793.8	0.206	-4.781	-51.61	-3306.77	300.41	-11713.7
94	1440.6	833.23	1.0	1973.9	2.5	2.5	15.708	155.031	0	-12.655	-24979.5	0.320	9.065	-54.07	1580.23	-205.4	8689.8
95	947.79	900.986	1.5	877.3	2.5	2.5	10.472	68.903	0	-10.365	-9092.8	0.880	-10.41	46.78	-842.48	179.6	-8143
96	1167.8	1028.49	2	493.5	2.5	2.5	7.854	38.758	0	-17.093	-8435.0	0.948	-17.102	-61.51	-790.04	177.79	-8135.2
97	939.62	184.841	0.5	7895.7	0	5	0	1240.25	0	-4.781	-37752.7	0.212	-4.524	-49.18	5611.22	333.36	-11127.2
98	1440.6	833.23	1.0	1973.9	0	5	0	310.063	0	-12.512	-24698.1	0.324	9.112	-54.65	2825.25	-231.61	8755.5
99	947.79	900.986	1.5	877.3	0	5	0	137.806	0	-10.094	-8855.6	0.903	-10.113	47.04	-1393.65	191.69	-8190.9
100	1167.8	1028.49	2	493.5	0	5	0	77.516	0	-16.60	-8191.6	0.977	-16.6	-61.13	-1286.78	187.46	-8086.4
101	939.62	184.841	0.5	7895.7	5	0	62.832	0	15	5.133	40527.4	0.197	6.119	-58.08	-3649.17	355.36	-16939.8
102	1440.6	833.23	1.0	1973.9	5	0	31.416	0	15	-12.782	-25230.8	0.317	9.978	-57.8	-1815.89	-236.39	11442.2
103	947.79	900.986	1.5	877.3	5	0	20.944	0	15	-10.599	-9298.1	0.860	-10.643	46.78	979.7	175.23	-8504.0
104	1167.8	1028.49	2	493.5	5	0	15.708	0	15	-17.658	-8714.0	0.918	-17.695	-61.44	-965.09	168.63	-8201.2
105	939.62	184.841	0.5	7895.7	2.5	2.5	31.416	620.126	15	4.913	38793.8	0.206	-5.794	-55.08	-4079.95	443.7	-18427.1
106	1440.6	833.23	1.0	1973.9	2.5	2.5	15.708	155.031	15	-12.655	-24979.5	0.320	9.361	-55.61	1639.26	-243.59	10654.2
107	947.79	900.986	1.5	877.3	2.5	2.5	10.472	68.903	15	-10.365	-9092.8	0.880	-10.402	46.78	-842.48	181.29	-8290.3
108	1167.8	1028.49	2	493.5	2.5	2.5	7.854	38.758	15	-17.093	-8434.0	0.948	-17.101	-61.72	-790.04	177.83	-8180.7
109	939.62	184.841	0.5	7895.7	0	5	0	1240.25	15	-4.781	-37752.7	0.212	-6.196	-53.65	7684.78	511.38	-17980.3
110	1440.6	833.23	1.0	1973.9	0	5	0	310.063	15	-12.512	-24698.1	0.324	9.39	-56.28	2911.61	-276.07	10891.9
111	947.79	900.986	1.5	877.3	0	5	0	137.806	15	-10.094	-8855.6	0.903	-10.11	47.04	-1393.25	193.72	-8292.7
112	1167.8	1028.49	2	493.5	0	5	0	77.516	15	-16.60	-8191.6	0.977	-16.6	-61.16	-1286.78	187.78	-8102.2
113	939.62	184.841	0.5	7895.7	5	0	62.832	0	30	5.133	40527.4	0.197	7.977	-61.32	-3852.99	584.61	26333.1
114	1440.6	833.23	1.0	1973.9	5	0	31.416	0	30	-12.782	-25230.8	0.317	-11.224	-60.54	-1901.98	292.84	-14376.9
115	947.79	900.986	1.5	877.3	5	0	20.944	0	30	-10.599	-9298.1	0.860	-10.633	46.78	979.7	176.2	-8648.2
116	1167.8	1028.49	2	493.5	5	0	15.708	0	30	-17.658	-8714	0.918	-17.668	-62.15	-976.25	168.66	-8293.9

4. Conclusive Remarks

According to the briefly presented considerations, some general conclusions can be drawn namely:

1. The undertaken study has revealed some aspects of the influence of the major parameters interfering in the inelastic timehistory response using an equivalent one degree of freedom system subjected to a set of seismic motions and to a sine excitation with circular frequencies in the usual range of the actual structures.

2. Restricting to the configuration of the damping force versus displacement relationship, the influence of the damping is obvious and defines the shape of the obtained diagrams. The butterfly or the bow versus tie shape is visible in all diagrams excepting the case when only classical viscous damping is considered. If the slope of the bilinear inelastic curve changes, the damping force representation is independent of this parameter and also of the natural period, when using Q_{hyst} model.

3. In the case when damping rates of the two types are equal, the functions of the damping force display a curvilinear symmetric shape; in the case of pure Penzien type of damping, the outlines of the diagrams are straight lines.

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Damage and Decay in Masonry Structures

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Rezumat: Lucrarea prezintă cauzele principale care conduc la apariția avariilor în construcțiile cu pereți structurali din zidărie.

Abstract: This paper presents the principals causes, which lead to occur of damage in masonry structures.

Keywords: *damage, decay, cracks, masonry.*

1. Introduction

The term damage is used to describe a situation in which a structure has lost some or all of its bearing capacity, a condition that can lead to failure and collapse. Damage is usually marked by cracks, crushing, crumbling, breaking away of elements, permanent deformations, out-of-plumbness, etc., and is related to mechanical actions.

Decay or deterioration is an alteration of the material that usually leads to a reduction in resistance, increased brittleness, porosity and loss of material that usually begins from the outside and works inward; it is mainly related to physical or chemical actions.

2. The origins of damage and decay

The origin of damage and deterioration can be attributed to one or more of the following factors:

a) Lack of due care in the original design

The safety of a construction can never be an absolute certainty, since it is affected by the uncertainty of evaluating the various phenomena and characteristics and therefore depends on probabilities: its strength and the actions to which it may be subjected may have more unfavourable values than those envisaged in its design. Safety coefficients laid down in building codes or introduced by architects and engineers are designed to meet these uncertainties.

Thus the probability that some event, as for example an earthquake, may have a more serious effect on a structure with regard to say damage or collapse than those envisaged in the design depends on the degree of caution used when the safety levels were established at the design stage. However safety has a cost and even modern codes accept a certain probability, albeit very low, those exceptional situations may give rise to crisis conditions.

b) Lack of scientific knowledge

A proper understanding of the main phenomena that increase the probability of critical situations occurring can only be acquired through scientific knowledge. In the case of ancient structures, the lack of scientific knowledge was usually compensated by practical experience and by constructing the structural elements with more than adequate dimensions. When innovative structural concepts arose, numerous collapses, and much damage had to occur before the builders acquired the necessary experience and arrived at a successful structural solution.

c) Use of constructions beyond their life expectancy

Every construction is designed with a more or less consciously predetermined life expectancy; this concept is implicit in the formulation of modern codes, in which "safety coefficients" and the corresponding probability that critical situations will arise are fixed in relation to a certain life expectancy for the structure. This may vary from decades to centuries, depending on the more or less rapid evolution of factors such as functionality, convenience, maintenance costs, etc.

Ancient buildings have often survived well beyond their life expectancies, so that damage, and

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especially weathering and decay, can be considered normal phenomena.

d) Errors and imperfections in the original design

Progress in the field of construction has been achieved more by intuition and experience than by scientific knowledge and therefore safety was ensured by repeating similar shapes and dimensions. The development of forms, shapes, and proportions was based on earlier models and experiences that had proved less prone to damage and collapse, but even that route did not totally avoid all risk.

e) Introduction of new and unknown factors

Constructions may have been designed for conditions different from those that effectively occur. Environmental changes (speeding up the process of deterioration of materials), variations in the use of structures (increased loads, etc.), structural alterations (extensions, partial demolition, etc.), earthworks (excavations, variations in water levels, embankments, fillings, galleries) and earthquake in areas not originally considered seismic are among the principal causes of damage and decay.

3. Lowering of the strength: stress ratio

Whatever the causes of these critical situations, they always ultimately result in a reduction in the strength: stress ratio, or in others words, the reductions of the bearing capacity and/or the increase in the effects of the actions involved. There are three factors at work here: the kind of action, the characteristics of materials, and the type of structure.

3.1. The actions

To understand the possible increase in action effects more clearly it is useful to group them as follows:

a) Mechanical actions

These are static and dynamic and their increase usually leads to increased stresses.

Static actions, these are of two kinds:

- ***Direct actions (applied forces)***

These are always present and consist of applied loads, such as dead loads (dead weight, permanent loads) and live loads (furniture, people, etc.). These actions can be increased by number of factors, including extensions of the building, introduction of new materials giving may to added weight, changes in the use of a structure, etc.

- ***Indirect actions (imposed deformations)***

These are related to deformation or strain imposed on structures, such as soil settlement, thermal variation, viscosity, or shrinkage of materials, etc.

These actions produce forces only when the deformation are contained or not free to develop; to avoid or reduce the generation of such forces, joints should be created to allow certain movement.

The most important of all indirect actions is soil settlement, which may even start to create new and significant stresses as soon as a construction is complete. Temperature is also an important factor in the creation of stresses and, consequently, cracks. The gradient between outer surfaces and the internal body can particularly produce superficial cracks that will accelerate decay. Daily and seasonal variations of temperature constitute cyclic actions that often result in cumulative damage.

- ***Dynamic actions***

The main result of these actions (earthquake, vibrating machinery, urban traffic, explosions, etc.) is acceleration applied to the structure.

The intensity of the forces produced is related not only to the intensity of the acceleration, but also to its frequency content and its relation with the natural frequencies of the structure and the capacity of the structure to dissipate energy.

Dynamic actions were rarely correctly taken into account in the original design of ancient buildings so that they often represent unexpected phenomena.

b) Physic-chemical actions

The presence of specific chemical agents like pollution, etc. can accelerate the natural process of weathering, leading to detrimental changes in materials and finally to a reduction of the strength.

3.2. The materials

The resistance of materials may be reduced as a consequence of weathering and deterioration due to physic-chemical actions.

Decay, which is the detrimental change of a material's characteristics, is linked to natural environmental conditions such as humidity, rain, frost, deposits of soil, presence of water and temperature extremes; factors such as traffic, vibrations, pollution or lack of maintenance may accelerate natural processes.

Decay may be of a chemical, physical or biological nature and as well as being linked to general environmental factors, it is also related to the characteristics of the material's used (chemical composition, microstructure, etc.), and to the protection offered to buildings by roofs, drainpipes, plastering, etc.

a) Moisture and the crystallisation of salts

One of the principal causes of decay is the penetration of moisture inside masonry. This moisture can be generated:

- by the condensation of water vapour present in the air, bringing with in the pollution contained in the atmosphere. The condensation increases as humidity rises and when a surface is colder than the dew point that is the temperature at which vapour condenses (we all know how drops of water form on the outside cold surface of glass of ice-cold water);
- by rain leaking through roofs and other surfaces;
- by ground water absorbed through the foundations and walls by capillary action: the extent of this absorption depends on the porosity of the building materials, evaporation and temperature. The nature of the soil, the type of foundation, the depth of the water table and its cyclical variations affect this phenomenon.

Each material requires some moisture content for its conservation. An excess of moisture helps metals corrosion and the development of fungi and bacteria. Excessive dryness increases shrinkage, and materials such as mortar become brittle or powdery.

Fluctuation and oscillations in moisture content are the most dangerous factors, since, as we shall see, these can activate soluble salts; any movement in the air and, consequently, fluctuations in moisture content. All materials are, to some degree, affected by this phenomenon: in a cave where the humidity, albeit high, remains constant, materials and even frescoes may be preserved in good condition.

The main problem created by moisture relates to soluble salts, such as sulphates, nitrates, chlorides, etc. In historic buildings the situation is complicated by the presence of not one single salt, but to complex mixtures of salts, often with different origins.

Salts may be present naturally in a construction (in the materials used in buildings) or may be introduced, originally in a very diluted form, by water absorbing them form the soil (where they are also deposited by humans and animals in the form of organic and other waste), from the atmosphere or from the surface of materials (where they are deposited by wind or animals).

Relative humidity and temperature affect the cyclical dissolving and re-crystallisation of salts, but it is evaporation, which is the principal factor; the evaporation cycle acts as follows:

- soluble salts are dispersed throughout the material by moisture;
- when water evaporates, a process enhanced by wind and temperature variations, salts are crystallised, internal stresses are produced and efflorescence (patches of the re-crystallised salts) appear on the surface. If the moisture spreads upwards, these phenomena will be concentrated in a band parallel to the foundation;
- the original volume of the material is increased by the presence of these crystals and a process of disintegration starts near the surface.

b) Air

Another cause of decay is air. The principal components of air are hydrogen (78%) and oxygen (21%). Pollution is caused mainly by dust and gases such as carbon dioxide (always present albeit in minute quantities) and sulphur dioxide.

Pollution corrodes both stone, especially on the surface (particularly visible in sculptures).

The wind, besides aiding surface evaporation, causes mechanical abrasion.

c) Soil deposits and biological and botanical factors

Biological pollution is caused by deposits of soil-contained moulds and insects, while the mechanical action of roots, especially of large plants, also causes cracks.

The acids contained in the excrement of birds, particularly pigeons, produce physic-chemical corrosion, while the excrement itself is a source of bacteria that can act as a fertiliser for vegetation.

The most effective means of preventing these changes is regular and careful maintenance.

d) Water

Water interacts with constructions mainly in the form of rain, snow, ice, and ground water and, as we have seen, contributes to the moisture content of masonry in different ways.

Rain, and snow, whether flowing over the surface or penetrating deeply into the structure, generates changes in porous materials such as masonry, plaster, wood. Acid-containing water attacks igneous rocks and sandstone, producing clayey substances prone to scaling.

Continuous mechanical action of water also contributes to attack of rocks and sandstone.

Ice is another cause of decay, especially when water penetrates existing cracks, which are then gradually widened and deepened by frosting. This mechanical action can lead to serious damage.

Ground water, fed by rains and rivers, is subject to periodical variations in levels, so that moisture is drawn up through the foundations by capillary action.

Finally, water allows development of moulds, bacteria and algae, whose damaging effects are well known.

e) Temperature

Temperature also affects the resistance of materials to physic-chemical decay. Daily and seasonal alteration in air temperature and the corresponding cycles of swelling and contraction may create or increase previous cracks which can trigger or accelerate deterioration, by aiding the penetration of polluting agents and the mechanical action or ice.

f) Human intervention

Incorrect use cleaning and maintenance using harsh techniques, such as brushing, hammering and blasting can weaken surfaces, thus facilitating decay.

3.3. The structural behaviour

The structure is that part of a building which, on the one hand, transforms actions into stresses and, on the other hand, provides the strength to sustain the construction.

Structural behaviour depends mainly on the characteristics of the materials used, the shape and dimensions of the structure, the connections between different elements and the boundary conditions, and also on the actions.

Damage, which is the detrimental change in the structural behaviour, is caused by the increase of the mechanical action or by the reduction of the structural efficiency: either natural phenomena or human intervention can, if not carefully controlled, have negative effects on buildings.

These alterations may take the following forms:

- Previous soil settlements (indirect actions), earthquakes (dynamic actions), etc., that have weakened connections and created permanent deformation, etc.
- Increasing, or extension of the dimensions of window and door openings, etc. (reduction of the structural efficiency), can induce cracks, deformations, crushing effects, etc.
- Elimination of structural elements: the removal of walls, slabs, or staircases can alter the mutual support of structural elements, causing cracks or local damage; the monolithic arrangement of the whole building may also be compromised.
- Removal of or alteration to the supports in structures producing thrust: the removal or addition of an arch, a vault, etc., can alter the equilibrium of horizontal forces.
- Additional loading: the construction of elements such as roof extensions or attics can alter structural behaviour, as well as increase loads.
- Excavations, galleries, the demolitions of nearby buildings or the construction of embankments or new buildings. All these alterations modify the loads on the soil and can induce settlements and eventually alter boundary conditions.

4. Conclusions

To prevent occur of damage in masonry structures it is important to know exactly the causes which lead to occur the damage and the effect of this in masonry structures.

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Numerical Simulation of a Concrete-Made Structural Wall

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Rezumat: Modelul experimentat este alcătuit din doi pereți structurali solidarizați prin intermediul unor rigle de cuplare. Peretele realizat la scară redusă nu a cuprins şi porțiunea de conlucrare între pereții transversali a şi pereții longitudinali b. Acoperitor, abordarea a fost extinsă şi la peretele detaşat destinat simulării numerice.

Abstract: The experimental model consists of two structural walls stiffened by some coupling braces. The wall (model realized to a reduced scale) not contains the co-operation zone between the transversal walls (a) and the longitudinal wall (b). This departure also was extended to the detached wall, made for the numerical simulation.

Keywords: *finite element modelling, numerical simulation, concrete wall.*

1. Introduction

The ensemble (realised at the 1/5 scale) simulate a part from a 4 levels structure, one transversal wall, dense walls structure, type honeycomb. The model ensemble was realised in horizontal position, in a single phase, avoiding the levelling technological joints, which could disturb the model inducing some discontinuities.

In order to experimentation, the model was set-up in accordance to the real structure, taking into account:

- description of the structural ensemble and of the non-structural elements;
- the permanent, temporary variable and exceptional loads;
- description of the real structural wall – the calculus of the loads at the detached wall;
- passing from the real wall to the model (for the numerical simulation);
- finite element modelling;
- numerical simulation of the model;
- resulting data analyse and proposals in order to consolidate the reinforced concrete self-carrying wall;

The composition, intersections through the transversal walls, the perimetral closing elements and also the concordance between the experiment to a reduced scale and numerical simulation (presented in this paper) lead us to detach the wall shown in the Figure 1.

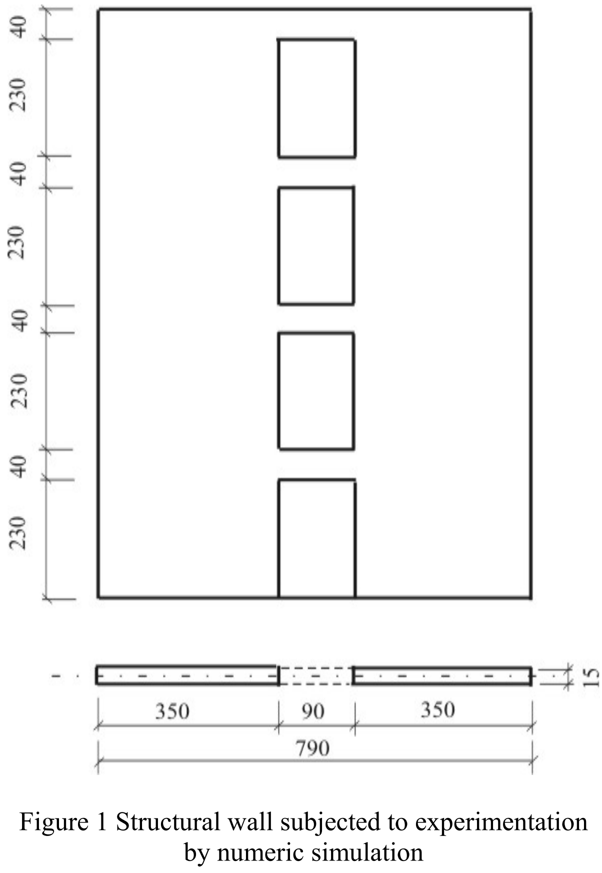

Figure 1 Structural wall subjected to experimentation by numeric simulation

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The scale reduced wall not contains the co-operation zone between the transversal walls (a) and the longitudinal wall (b). This departure also was extended to the detached wall, made for the numerical simulation.

The experimental model consists of two structural walls stiffened by some coupling braces.

The conventional seismic force was applied in ten steps by hydraulic pressing machine.

2. Passing from the real wall to the model (for the numerical simulation)

This intermediate stage is necessary in order to optimise the numerical simulation. The stage objectives are:

- number limitation of the finite elements types;
- number limitation of the volumes and distinctive surfaces, on each type of finite element;

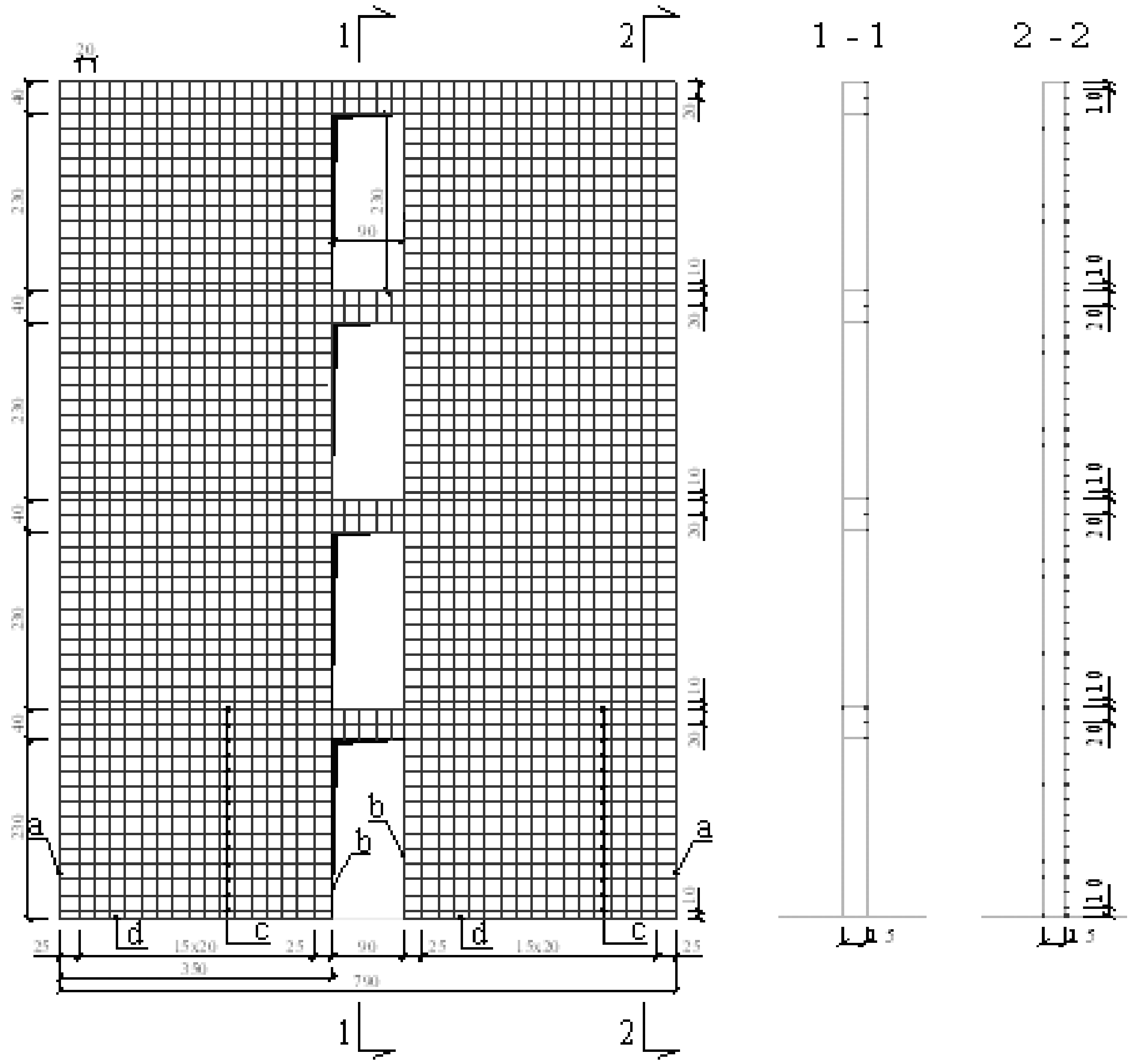

Fig. 2 The structural wall modelling in order to numerical simulate

- the creation of the possibility to pass to the finite element of those which are the same from the exterior geometrically point of view.
- the creation of the possibility to attach, in a simple mode, of the finite element which will be part of the consolidation.

Taking into account these objectives it was passed from the detached wall (shown in Figure 1) to the model, shown in Figure 2.

The interventions made in order to obtain the desired model are:

- the "a" vertical bars displacements (5 cm), final position being on the vertical perimeter of the self-carrying wall;
- the "b" vertical bars displacements (5 cm), final position being on the vertical perimeter of the wall's emptiness;
- retaking the "c" horizontal bars positions, from the base to all levels – in this manner results the identically reinforcement for each level and, also, to each coupling brace;
- the "c" reinforcement meshes positioning to the wall's "faces", like how doesn't exist a concrete covering of the reinforcement, in order to define the "beam" type finite elements, in the intersection of the "brick" type concrete element.

3. Remarks

The numerical modelling was realised with ALGOR computer program, in elastic domain. It was used "brick" type elements, for the wall, and "beam" type elements, for the reinforcements.

The wall modelling it was realised so that the edges positioning of the finite elements to coincide to the longitudinal and transversal reinforcement position; knowing that the real behaviour of the wall and its consolidation method, must be create the easy possibility to attach the finite elements which will compose the bars ensemble – in this sense on the d_1 and d_2 portions the lines grid were thickened from 20 cm to 10 cm. It result a variable step, with "bricks" by 20 x 20 x 15 cm in the current field of the wall and by 20 x 10 x 15 cm on the d_1 and d_2 zones.

A special attention was given to the discretization of the coupling braces, because in these zones are expected great oscillations of the stresses. So that, the discretization was sophisticated, using "bricks" by 5 x 5 x 7,5 cm in the current field and 2,5 x 2,5 x 7,5 at the edges of the coupling spar.

So, it was obtained a moderate ratio between "bricks" flanks which could generate a good precision. The domain connections were assured by blocking the nodes displacements from the base, on all three directions.

The vertical loads was distributed and attached at the nodes from the floor level and the horizontal loads were defined like "pressures" on the lateral finite elements from the floors levels.

It result a model composed from the 4790 "brick" type element for the concrete from wall and 8092 "beam" type elements in the wall's mesh.

The precision test, based on the equivalent stress continuity Von Mises, showed the modelling accuracy.

The wall was loaded with all horizontal and vertical loads. It result principal maximum bearing stresses greater then bearing strength in the zone of the coupling braces and in the left inferior part of each strut.

The main minimum stresses overtake (on restrain zones, in the coupling braces zone) then the bending strength.

The analyse was continued starting from the following hypothesis:

- the zones in which the bending strengths of the concrete are overtaken were declared without concrete (because of the concrete fissuring);
- because of the alternant stresses from real mode, these zones was disposed together with their symmetrically portions.

The wall, prepared like this, was subjected to the loads ensemble corresponding to the special grouping, and it was result the main maximum stresses in the damaged structure.

It can be observed that the symmetry appliance - caused by the alternant loads – leads to the struts collapse from the base and, in consequence, to the wall collapse.

Although, the hypothesis that are the basement of this reasoning are under covered, its can be accepted and, even, preferred to a laborious analyse which can reveal the stresses which are in the neighbourhood of each fissure.

The performed analysis was point out the fact that the appliance of the concrete removing hypothesis is more credible if this operation is done step by step and does not correspond with the maximum combination of forces. This conclusion is considered if we taken into consideration the big values, greater then the tensile strengths of the maximum stresses.

The acceptance of the successive loading cycles, that point-out the formed zones from that the concrete is removed, is a procedure that better approximates the reality:

Cycle "0"

The wall was loaded with integer vertical load and successively, in ten stages, with horizontal loads bigger step by step. By following the principal maximum tensions and its symmetrically disposing it result like significant step the fourth iteration and, corresponding to it, was removed the fissured concrete.

Cycle "1"

The wall with corresponding degradations of the cycle "0" the fourth iteration, with removed concrete from the coupling braces was subjected to the load with all horizontal charge bigger and bigger until to the maximum load.

Following the maximum principal tensions from the struts, the coupling braces exhausted its bearing capacity and, it symmetrically disposing resulted like a significant step the seventh iteration and corresponding the strong fissured concrete was removed.

Cycle "2"

The wall with degradation corresponding to cycle "0", with the removed concrete from the coupling braces and with degradation from the cycle "1" – the seventh iteration, with some part from the first two level strut's concrete removed, was subjected to the total vertical load and successive, in 4 steps, with the difference horizontal load bigger and bigger until to the maximum load exceed.

In this mode is realize the maximum principal stresses stage that exceed tensile strength and to the minimum principal stresses stage bigger than the strength at compression, that combined and symmetrically lead to the total lose of the bearing capacity.

It must be remark that even to an alternant stress the concrete from the fissured zones (from the removed zones), don't work in the same time to the tensile, the concluded hypothesis are covered.

The covering degree is given by the difference between proposed behaviour, in the covered mode, by removing the concrete from the fissured and compressed zones, and the wall behaviour when, in the compressed and fissured zones it is considered that the degraded concrete work having a reduced elasticity modulus according to its fissured stage.

In order to point-out this was simulate to the wall, where the fissured concrete zones where arise tensile stresses, are removed and its symmetric, fissured and compressed zones, was considered having a reduced elasticity modulus 0,2 E_b.

The covered degree was defined like the ratio between surfaces, at the most stressed wall level, on which was exceeded the tensile stress in the case of elimination of the concrete zones by symmetrically and the idem surfaces defined for the case with compressed fissured zones considered having elasticity modulus 0,2 E_b = E".

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Viable Structures – Stone Bridges Erected by Steven the Great

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Rezumat: În lucrare se prezintă podurile de piatră construite în timpul domniei lui Ştefan cel Mare, domnitorul Ţării Moldovei între 1457 - 1504. După o scurtă trecere în revistă a drumurilor care străbăteau atunci Moldova, se fac referiri la podurile de la Borzeşti (Negoieşti), Cîrjoaia şi Zlodica, conform surselor bibliografice existente. Fotografiile au fost efectuate în luna mai 2004.

Abstract: The paper presents the stone bridges erected under the reign of Steven the Great, the ruler prince of Moldavia between 1457 and 1504. After a short review of the roads that went that time across Moldavia, the paper refers to the bridges from Borzeşti (Negoieşti), Cîrjoaia and Zlodica with respect to the existing bibliographic sources. The photos are made in May 2004.

Keywords: *bridge, vault, road, stone, reliability, viability.*

1. Introduction. About the "big highroad" and other roads of Steven's Moldova

In the spring of 1457, with support from Vlad the Impeller, Steven, the son of Bogdan Voda, was coming "on the big highroad connecting the Walachian Buzău with the Moldavian Bacău, road for commerce and road good for military"[1]. Close to the village named Doljeşti (a commune in present Neamţ County), on April 12th, Steven defeats the resistance opposed by Petru Aron's peoples and goes on his way to Bucovina. Teoctist, the Metropolitan, comes to encounter him and, on the field from Direptate (a plain on Siret River close to Suceava), "he anointed him for ruling" [2].

The following period, the entire defense system of Moldavia was consolidated and completed. The fortress of Roman (first mentioned into a document from Roman I Muşat), constructed before in wood and earth, was now reconstructed in stone masonry. The stronghold of Suceava become, after successive transformations, a magnificent and complicate palace – castle. At the castle of Neamţ, erected by Petru I Muşat, probably between 1382-1387, new fortifications were built. By the order of Steven the Great, 800 masons with 17000 aids built the fortress of Chilia in the summer of 1479.

The trade roads that were passing through Moldavia were of great significance both for country's and neighbors' economy. Transilvania's road network was primary oriented on east, south-east and south. Development of Braşov, Sibiu and Bistriţa towns is largely due to the commerce with Moldavia and Walachia. Steven the Great renewed the privileges of the nobles from Braşov, Bistriţa and Lyov immediately after accessing the throne.

The most important road, named the "big highroad" [3], [4], was going from North to South on the right bank of Siret River. This road is attested in documents in the following points: Cernauţi, Siret (ferry - "moving bridge" on Siret River), Grameşti, Suceava (custom office) Horodniceni, Mogoşeşti (custom office), Roman (custom office), Dragomireşti (in 1408 – ford on Moldova river, in 1458 ferry ("moving bridge"), is the oldest testimony of a ferry in Moldavia), Bacău (custom office), Livezi (ford on Tazlău River).

Other roads crossing Moldavia [5] were: Iaşi – Vaslui – Bârlad – Tecuci – Galaţi; Focşani – Adjud – Bacău – Roman – Târgu Frumos – Botoşani – Mihăileni with branches: Târgu Frumos - Iaşi and Roma – Fălticeni – Suceava; Iaşi – Dorohoi – Mamorniţa – Cernăuţi and Iaşi – Huşi – Folteşti – Galaţi.

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At the beginning, these roads were set as natural roads on the fields and along the valleys avoiding the road artworks. The ruler got involved only in case of war. Construction and maintenance of bridges was leaved to the private initiative in exchange for the right to collect levy.

The deep rivers were crossed in winter on ice bridges and on fords in summer when the level of water was lower. Therefore one can find a lot of villages on the territory of Moldavia with names like: *Vaduri* (Fords), *Vădeni* (people at the ford), *Vadul lui ...* (the ford of …). Sometimes these fords were signalized on the banks of the rivers with big knolls.

About fixed bridges, one can find in boundary documents a lot of words (*podeț* – culvert, *podişcă* – deck, *podişor* – small bridge, bridgy), but rarely bridge (*pod*) and often sitting bridge (*pod stătătoriu*) and settled bridge (*pod statornic*).

Among fix bridges mentioned in documents [3], [4] one can remember: the bridge of Dragomir Brinişterul over Negru brook at Trifeşti (Neamț County) – August 4th 1400; the bridge of Gârlanici over Bârcoveț – 1420; stone bridge over Şomuzul Mare river at Mihăileşti (Suceava County) – April 18th 1444 and March 12th 1488; the bridge of Hasniş over Runcului Brook at Berchişeşti (Suceava County) – September 13th 1473; the bridge of Vețea over Negru brook at Vlădiceni (Neamț County) – October 15th 1488.

From the times of Steven the Great several stone bridges lasted until present days. Among them we nominate: the bridge from Borzeşti (Negoieşti), the bridge from Cîrjoaia, the bridge from Zlodica etc.

2. The bridge from Borzeşti (Negoieşti)

The bridge from Borzeşti (the place where Steven was born, locality incorporated now in the municipality of Oneşti), name introduced in literature by Academy member C.I. Istrate [6] in 1903 ("… is about the bridge, or better the culvert, existing near Borzeşti"), is placed actually on the territory of the Negoieşti village, now a part of Steven the Great commune (Bacău County). Therefore, we think that is better to connect the name of the bridge with the village of Negoieşti. The bridge from Negoieşti, Fig.1 is erected on the road Adjud–Târgu Ocna–Oituz over the Steven the Great brook.

Fig. 1. The bridge at Borzeşti (Negoieşti), general view

About this bridge prof. Ion Ionescu wrote in 1934 at the celebration of Polytechnic Society [7]. We quote from this paper: "… bridge in stone, with a vault of 5.8 m large, executed with two rings of boulders with mortar of fat lime, with tilted inner joints and shaped visible stones …".

It is also known, from the paper of Academy member C.I. Istrate, that in the time of the ruler Mihail Sturza a small monument was erected for "notify the construction of the bridge". The monument was latter destroyed by the constructors of the railway Adjud – Oneşti (1881-1885). The engineer George Sion, Head of 7th circumscription of roads and bridges (institution administrating the national roads from Bacau, Putna, Tecuci and Covurlui counties until 1908) referring to this bridge specifies: "in 1873 the construction of the road Adjud – Oneşti – Frontier which after its completion was maintained by the state as national road ; … among the existing bridges we found at km. 24+810, starting from Adjud, a vaulted stone bridge erected by Steven the Great,…, The bridge was built in raw stone with fat mortar, the vault constructed with round boulders … The highroad was first positioned at the first gusset, 5.30 m above the water having a parapet of 1.20 m height between the two gussets , but in time the way raised and reach the second condition 6.50 m above the water and the bridge has no parapet."

The bridge was repaired between 1902 and 1903 by engineer George Sion who built two check dams, replaced the missing boulder in the vault, which waslater daubed in cement mortar. The bridge is no longer in use even in good condition.

3. The Bridge at Cîrjoaia

According to the documents, close to Cotnari, in the village of Cîrjoaia, on the county road 281C that connects the localities Blăgeşti (near Paşcani) – Hărmăneştii Vechi – Boldeşti – Stroieşti – Coasta Măgurii – Cîrjoaia – Cotnari, there is a stone bridge built in the times of Steven the Great, fig.2.

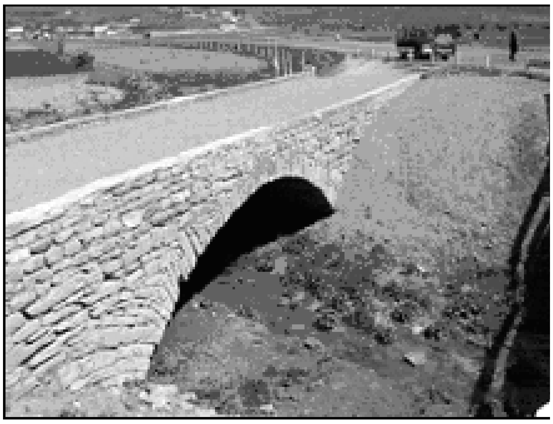

Fig.2. The Bridge at Cîrjoaia, general view

The bridge at Cîrjoaia is constructed over the Măgura River, from shaped stone blocks, as a 7 meters elliptic vault. The stone was probably brought from the Bărbăteşti quarry – bassarabian oolitic limestone [8] or from the deposit of Deleni-Fierbătoare – pseudo-oolitic sandstone and calciferous sandstone.

Referring to this bridge, the Department of "Inner Causes" addressed in 1832 the Magistrate's Service from Hîrlău pointing that reparations – consolidation should be conducted with great care such that "the truth of its being and its antiquity" not to be changed. This **document** might be considered as the first **instructions in our country** directing the way a work of art would be restored.

In another document [9], it is specified that "searching in place it was found that, in fact, the bridge is 42 meters long, with 4 vaults built in shaped stone with a balustrade on which there is an inscription". In time, it was buried in silt, only one opening remaining at sight.

In the year 1847 the bridge is restored and the balustrade, which initially was made from flagstones bound together with lead riveted iron, was replaced with massive stone blocks on two of which the following inscription was enchiseled:

> "This bridge, constructed by the most blessed to be remembered prince Steven the Great, was renewed in the year 1847 under the commandment of his most highness the prince Mihail Grigorie Sturza Vaivode by the sedulity of logothet Iordache the Minister of Interior, on state's exchequer expenses."

In the eighth decade of the twentieth century, from the initiative of Local Roads Direction of the Iaşi County, the bridge was reconsolidated, and presently it is in very good condition but is no longer in operation. Into a close location a brand new modern bridge was erected which surely will last for about 50-60 years!

4. The Bridge at Zlodica

On the territory of locality Zlodica, a village in Cotnari Commune, on the communal roads DC 156, there is a massive bridge built in shaped stone, Fig.3., which is mentioned in a document from 1806 found in Iaşi State Archives [10] from which we quote: "At Zlodica toward Cotnari there is Steven the Great's bridge in stone, great bridge covered upon above with stone blocks. The bridge had an ancient inscription from which only the place remained. Its bases are now buried in mud and its only vault open for the drainage of the waters of the brook that springs under the Hill of Pigs nearby and from freshets in rainy times, will soon be totally covered if no urgent restoration measures will be pursued" [11].

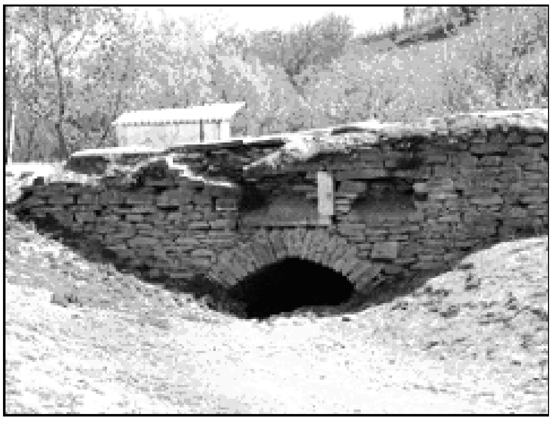

Fig.3. The bridge at Zlodica, general view

There is also, in Cotnari, a church about which the catholic inhabitants keep a tradition saying that it was built by Steven the Great for a sweetheart of his, Cătălina, which was a catholic. In front of the church passes the road named “of the Great Steven”. This road connects Cotnari with the old road toward Hârlău [11].

Near the bridge at Zlodica, in the yard of a villager, we still can see the ruins of a rathskeller of remarkable dimensions also built in shaped stone. The historical documents speak about a stone cellar of an ancient inn near by the Steven the Great’s stone bridge from Zlodica [11].

5. Conclusions

Through this paper we want to bring an homage to the illustrious prince ruler Steven the Great and Saint in the moment when we commemorate the 500th year of his passing to the world of saints.

Because the bridges we presented before are for more than 500 years in operation, one may conclude that this type of construction – the vault in stone – represents a viable construction due to its proven high reliability.

In the same time, for the rivers in our country, with relatively narrow scours, erection of this kind of bridges, using modern techniques and technologies, would represent a solution for many of the problems existent on county and communal roads.

From bibliographic study we found that with Steven the Great reign is also connected with other bridge that existed in Moldavia:

a) The bridge over Jijia River close to Ştefăneşti in Botoşani County.;
b) Stone bridge around Dorohoi Town, attributed by Alexandru Odobescu to logothet Ioan Tăutu (boyar at ruler’s court);
c) Stone bridge over Şomuzul Mare River close to its confluence with Siret River [12]. Also, the existence of this bridge is noticed in 1464 in connection with “the village of Mihăileşti on Şomuz … near the stone bridge upper Bahrineşti” [12].

We also must notice the fact that always when the ruler prince was building a church in a locality, a bridge was erected nearby:

a) the church and the bridge from Borzeşti (the bridge at Negoieşti);
b) the church and the bridge from Cotnari (the bridge at Cîrjoaia).

This way to construct was inherited in Moldavia until modern times. As an example, at the beginning of the 20th century the following constructions were built:

a) the church and the bridge from Tansa (in Iaşi County);
b) the church from Mălini and the stone masonry culverts on “*drumul talianului*” (Italian’s road) in Suceava County.

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Energy Dissipation Systems Used in Seismic Isolation

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Rezumat: Lucrarea reprezintă o sinteză a sistemelor de disipare a energiei utilizate în izolarea seismică. Sunt prezentate dispozitivele disipatoare (histeretice de energie bazate pe deformarea plastică a metalului, bazate pe extrudare, bazate pe fricțiune, vâscoelastice, cu fluid vâscos), sisteme acordate (disipatori cu masa acordată - Tuned Mass Damper – TMD, disipatori cu lichid acordat (TLD), disipatori cu coloane de lichid acordat (TLCD), materiale inteligente (aliaje cu « memoria formei », materiale piezoelectrice, materiale electroreologice (ER), fluidele magnetoreologice).

Abstract: The paper presents a synthesis of the energy dissipation systems used in seismic isolation. It is an overview of the dissipation devices (systems that use hysteresis dissipators based on the nonlinear deformation of the metal,systems based on extrusion, friction devices, viscoelastic dampers , viscous fluid dampers) , tuned systems (Tuned Mass Damper – TMD, Tuned Liquid Damper -TLD, Tuned Liquid Column Damper –TLCD), intelligent materials(shape memory alloys), piezo-electric materials, Electro-Rheological materials (ER) and Magneto-Rheological fluids(MR)).

Keywords: Dissipation, hysteresis, viscoelastic dampers, intelligent materials, nonlinear deformation.

1. Introduction

Most of the existing constructions were designed and built according to the old codes : P13-63 ; P13-70. After the 4th of March 1977 earthquake the P100-92 was elaborated, which brought a lot of changes to the designing spectrums. From this point of view, the old buildings are over-passed by the actual codes.

To fix this deficiency, structural strengthening is dued, but also the introduction of energy dissipation devices is needed.

During the most powerful earthquake we cannot avoid the structure getting into the nonlinear field, but with exagerated expenses, to be able to insure the strength capacity of the whole structure.

Considering that the force of an earthquake is an exceptional one that may appear or not during the lifetime of a structure, these exagerated expenses cannot be justified.

In present, in the structural design, the accent is layed upon,the insurance of the structure's ductility, in a way that the amount of energy dissipated trough nonlinear deformations is increased a lot, especially in the first moments of the earthquake.

In this way, the method tries to determine the birth of nonlinear joint mechanisms in the areas with the best ductility (the ends of the beams).

Strong after-effect deformations from the nonlinear joints can cause significant damages, sometimes followed by the collapse of most nonstructural elements.

These damages bring a lot problems : their collapsing is a real threat to the human lives of the inhabitants of the buildings – often the collapse of the nonstructural elements produces more victims than of the building's total distruction.

In case such a designing strategy is adopted, the structural system can be compromised, which rises huge expenses for reconstruction or demolition.

So, the designing method of the strength capacity supposes the acceptation of important damages, with all the risks and expenses that come along, with the purpose of increasing the energy dissipation capacity trough nonlinear deformations, avoiding the structural collapse.

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The alternative solution for this designing strategy is the increasing of the dissipation capacity trough introduction of energy dissipaters.

In this case, for the eartquakes that are weaker than the strongest possible earthquake, the structure would be capable of dissipating enough energy troughout the damping capacity of the dissipater in a way that the structure will not get into the nonlinear field, without producing important damages in the nonstructural elements.

This strategy would eliminate the weaknesses of the first method by reducing the relative displacements of the floors trough the increasing of damping.

2. Energy dissipation devices

2.1 Hysteresis dissipators based on the nonlinear deformation of the metal

The metal dissipators appeared as a result of the research made in 1972 by Kelly and 1975 by Skinner, considerated to be one of the most efficient mechanisms.

Trough deformation of the metal from the device, beyond the yield point energy dissipation occurs.

The absorption element can work under bending forces, compression, forge drawing, compressive buckling or any combination of two or more stresses.

The plasticity of the metal materials is their capacity of deforming in a plastic manner,which means it can change it's shape and primary dimensions permanently, under exterior loads, without destroying their internal structure.

The metal is a soft iron , but can be also lead or other alloys, called " intelligent materials ".

The most familiar dissipators based on the nonlinear deformation of the metal are the ones with one-direction action: (with plates and steel bands subjected to bending, torsioned-bended bars, devices with band steel winding, devices with plastic joints, devices with deformed bars on sheaves), devices with bended plates , devices with flatten tube, devices with the exterior deformation of the tube, devices with forming tube and plate and « W » Frame devices.

2.2 Dissipation devices based on extrusion

The extrusion process is in fact a plastic deformation dued to a force which determines the metal to flow trough a hole.

Depending on the direction of the deformation and on the direction of the extrusion force, the extrusion process can be direct (when the two have the same direction) or indirect (when the two have opposite directions).

The extrusion process can be defined as deformation of a multicristal metal according to it's elongation and to it's big number of failures in each corn.

Research showed that after an certain amount of time, at high temperatures, the metal can temper to the primary condition – without damages dued to the plastic deformation, as a result of recristallisation, restoration and increasing processes.

The devices in which solid materials are being subjected to the extrusion process absorb energy trough deformation inside the crystals using the slip process.

Lead was identified even since 1970 as a metal that can be used in the extrusion process.

In an ambient temperature the lead is getting restored and recristallised very quikly which means that the necessary extrusion force has the same value for each cycle.

The first dissipation device based on extrusion was invented by W.H.Robinson in April of 1971 and it consists of the forced pass of a metal troughout a hole, changing it's shape.

In 1976 Robinson and Greenback proposed and built some dissipation devices based on extrusion.

The most familiar dissipators are the ones based on lead's extrusion with a constricted cylinder and the dissipators with a bellied axle.

Among the latest achievements in this field is the research of N. Masaki and S.Suzuki,who propose building a damper based on the extrusion of un-vulcanized rubber.(Figure 1).

O.Furuya and S. Fujita propose a semi-active damper using the extrusion of intelligent materials together with a new ideea, building a multi-step damping system.(Figure 2).

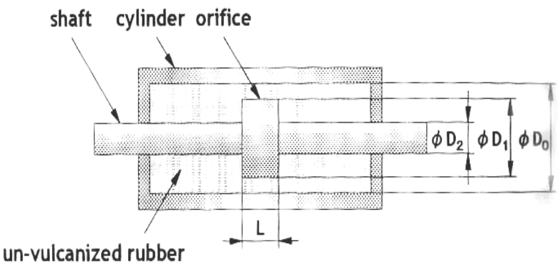

Fig.1.Damper based on the extrusion of un-vulcanized rubber

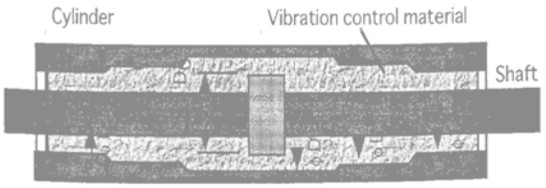

Fig.2. Semi-active damper using the extrusion of intelligent materials together with a new ideea, building a multi-step damping system.

2.3 Dissipation devices based on friction

Inspired by phenomenons met in nature, these dissipators based on the friction between the contact surfaces have found use in different fields. The heat and deformation of the contact surfaces, which generate the energy dissipation are the effect of internal friction. The friction damper concept was introduced by Emile Mors in 1899 and J.M.M. Truffault who built the first device used for cars, made of two disks that have a leather layer in between.

These devices use dry friction, the energy dissipation is made trough a tangent force which usually appears in sliding friction between two solid bodies with dry surfaces .

Research reached the conclusion that surfaces don't' have to be perfect plane, on the contrary they should be coarse. In high aggressive environments, the corrosion can become a problem .

E.V.Hartford elaborated in 1906 a much more accurate friction device, that can be adjustable. Damping devices based on friction are being used to improve the seismic behavier of new structures but also for the consolidation of the old ones. In Canada , Pall dampers (LSB- Limited Slip Bolt Joint, 1980, "X" shape in 1982) are being used and Sumitomo (with axial action in 1990 and ERD – Energy Dissipating Restraints with Nims friction connection in 1993) dampers in Japan.

2.4 Viscoelastic dissipators

The materials used for viscoelastic dampers are conjoint polymer or glass materials. When constrained to deformations produced by a shearing force, this materials are able to dissipate energy. In 1969 there have been installed 10.000 viscoelastic dampers in each tower of the World Trade Center building in New-York City, from to 10th floor up to the 110th floor. The dampers worked together with the cannular steel structure to reduce the wind vibrations under the human perception level.

Viscoelastic dissipators were the work of Mahmoodi, made of two steel plates that have a elastomer fixed in between. The elastomers are usually used in structural applications because they are able to dissipate energy when constrained to deformations produced by a shearing force. Together with natural rubber, artificial elastomers are also used, like: chloroprene, artificial rubber .

Viscoelastic dissipators are being placed where the relative displacements of the floors have an important value. If their position is not the best one, their number has to be increased. The dissipators increase the viscous damping and the lateral stiffnes, their efficiency growing for the steel and reinforced concrete structures .

2.5. Viscous fluid dissipators

Even since 1976 Harris and Crede have created the viscous fluid dissipator concept using it for vibrations and shocks attenuation, first as car's dampers.

The energy dissipation is made transforming the mechanical energy into heat dued to a viscous fluid driven by a piston. The most familiar are the Taylor device, Jarret device and GERB device.

For the materials that are nor fluids neither solids, the energy dissipation is produced both ways. In case of a fluid device ,an increased piston stroke determines a higher cuantity of energy dissipated on extrusion volume unit, and the speeds of the fluid at the entrance hole will be higher.

The energy dissipation is produced when the fluid passes trough the exit holes with small diameter.

The energy is then absorbed using vorticity and then rising temperature. Another posibility is the deforming viscous fluid which is set into an open container.

J.Tagami, H.Koshida and T.Sugiyama elaborated a semi-active switching oil damper applied to an actual 11-story steel building.

A.Nishitani, J.Nitta and S.Yamaguchi studied the semi-active structural control using oil dampers, application technology and ways to place them inside the building.

This damper has a electromagnetic valve. Changing valve's voltage, the damper modifies it's characteristics (Figure 3).

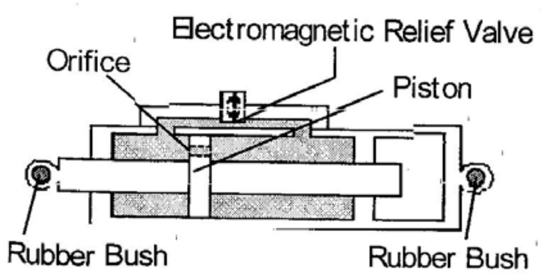

Fig.3.The damper with a electromagnetic valve

M.Kondou and Y.Tsuyaki use an oil damper for the basis of the structure, together with rubber bearing (Figure 4).

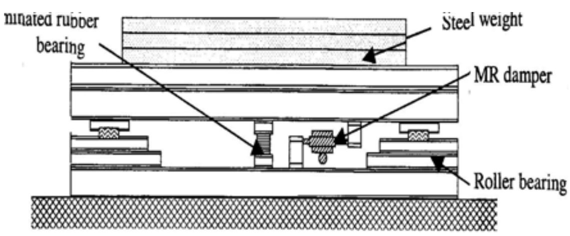

Fig.4.The oil damper for the basis of the structure, together with rubber bearing

3. Tuned systems

3.1 Tuned Mass Damper – TMD

Tuned mass dampers reduce the quantity of the energy an earthquake inserts into the structure because of the transfer towards the tuned mass dampers of a certain amonut of structural vibration energy.

Tuned systems are auxiliary devices attached to the structures in the purpose of reducing dynamic vibrations caused by wind or earthquakes. The name of this devices "tuned systems" comes from he fact that their fundamental frequency has almost the same value as the structure's frequency.

Tuned mass dampers reduce the amount of energy inserted into the structure and the ductility requirements of structural elements.

If the TMD mass is smaller than the structural mass the device has to be tuned in a way that the auxiliary mass will not determine the entrance of the structure into resonance with the excitation's frequency.

If the fundamental frequency of the structure is smaller than excitation's frequency, the TMD will be given a smaller frequency than the exterior action's in order to avoid resonance. If the fundamental frequency of the structure is higher than the exterior action's, the TMD will be given a higher frequency than the excitation's frequency.

An ordinary TMD is made out of an additional mass which has a relative movement, different from the structure's and is connected to the structure trough a spring, it also has a viscous device.

3.2 Tuned Liquid Damper –TLD

The TLD are made of stiff tanks filled with liquid. The absorbed energy is dissipated using the liquid's viscosity, the waterbraking on the tank's walls and the damping caused by "floating shells". The fluid works as a relative moving mass, the tempering force being determinated by gravity.

TLD compared with TMD has a number of advantages, like: the possibility of reducing structural movement in two directions at the same time, it's no necessary to have large structural displacements to make the efficiency of the damper an optimum one; the liquid mass is smaller than of the TMD's (steel,concrete,lead) needing a reduced space to obtain the same damping.

3.3 Tuned Liquid Column Damper –TLCD

The TLCDs are made of containers (with liquid tube shape) attached to the structure. The energy dissipation is dued to the liquid movement from the tube trough a hole. The damper's frequency is tuned according to the changing of the liquid column's length. The damping can be increased by modifying the hole's opening.

4. Intelligent materials

A especial chapter in vibrations control is the one that concerns the "intelligent materials" - shape memory alloys, piezo-electric materials, electro-rheological materials and magneto-rheological fluids.

4.1 Shape Memory Alloys

The "shape memory "effect was first used in 1932 on an gold-cadmium alloy that had a hysteresis behaviour. This Shape Memory Alloy – SMA effect was defined in 1951 as the capacity of an alloy to accept reversible transformations between the two familiar crystalline state :the phase with high temperatures of the alloy si the phase with low temperatures of the alloy. In this category we find alloys based on copper, nickel-titanium, some ferroalloy, ceramic alloys or polymer alloys.

4.2 Piezoelectric materials

The electromechanic property that connects the elastic and magnetic fields, called piezoelasticity was discovered in 1880 by Curie. A piezoelectric material generates an electric power as result of the mechanical actions he is subjected to.

This has a direct piezoelectric effect being useful for sensors from the control chain. If the material is applied an electric load, a mechanical state of deformations and tensions is induced, obtaining a conversion pizoelectric effect, useful for control forces generation. The piezoelectric elements cover a large frequency range, reliabil, light being used for control systems and structural elements: beams, plates, or for the thin cloths structures.

4.3 Electrorheological materials (ER)

The electrorheological materials are known for their capacity to have reversible growths of the yield strength when subjected to an electric field .The ER are contralable fluids in the presence of an electric field.

Considering the structural control this materials were used even since 1949 by Winslow. In 1980's once the anhydric ER fluids were discovered appeared a lot of studies concerning the energy dissipation devices with ER fluid, first being the control valve and then the dampers, the cyber equipment's and vibration control dampers.

The ER fluids are used in the control valves to produce moment transfer or for the opening and closing operation.

The ER fluids used for dissipators and dampers devices have the control upon a gradient pressure –the flowing way .

4.4 Magnetorheological fluids (MR)

Part of the controlable fluids are also the magnetorheological fluids (MR), able to transform in miliseconds from fluids with a linear viscous yield into semisolids with a controlable yield point as long as they are surrounded by a magnetic field.

This ideea belongs to Iacob Rainbow since 1948.It is used in many countries like: Italy, China (where Ou Jinping, Wang Gang use semi-active MR fluid dampers for the marine platforms where the TLDs didn't work).

The MR are suspensions of magnetizing particle very, very small (3-5 mycrons) in a viscous fluid with engine oil's consistency. Being in a magnetic field the ferros particles receive a dipole moment associated to the external field, which determines their laying in parallel line with field line direction, making the suspension particles solid, stoping the fluid's flowing (20-40)% of the MR fluid's volume are solid ferros particles in suspension in mineral oil, syntethic oil or water.

S.Morishian, T.Shiraishi, N. Nakaya applied magnetorheological fluid dampers, a high quality material , which has characteristics that can be changed trough a magnetic field. (Figure5)

In Macedonia, at the Institute of Earthquake in Skopje, Zoran Rakicevic and Dimitar Jurokovski studied the ways the dampers can be placed in buildings at different height levels.

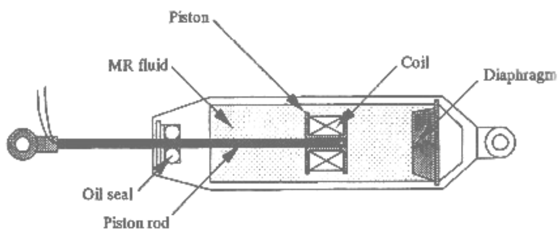

Fig.5. The magnetorheological fluid dampers

5. Conclusion

The present synthesis clearly shows that there are a lot of energy dissipation systems available today, applicable to seismic engineering, some working on their own, some combined with bearings to reduce the earthquake's effects. Depending on every building's characteristics an optimum dissipation system will be chosen.

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Buckling Resistance of the Compression Members With Open cross – Section (Part II)

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Rezumat: Această lucrare este partea a doua a lucrarii cu titlul "Buckling resistance of the compression members with open cross – section (Part I)".În cadrul unei aplicaţii numerice este analizată stabilitatea unei bare comprimate cu secţiune monosimetrică, profil deschis şi se determină capacitatea portantă la compresiune centrică în conformitate cu standardele române şi cu EC 3.

Abstract: This paper is second part of the paper entitled "Buckling resistance of the compression members with open cross – section (Part I)" by P. Moga, T. Orghidan, Şt. Guţiu. In the first part, was presented the analysis regarding the compression members, combined flexural and torsional buckling and the checking methodology according to technical references and Romanian norms as well as to the EUROCODE 3 norm.
The paper presents a numerical example for a compression member with a mono-symmetrical open cross section and some conclusions concerning members design.

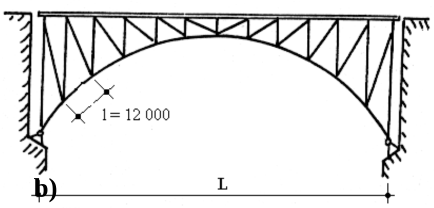
Keywords: Eurocode 3, flexural - torsional buckling, compression member.

1. Design dates

An arch section, part of a bridge superstructure (Fig. 1) is analyzed.

The section has a length of 12 m, a mono-symmetrical cross-section as it is shown in Fig. 3.

The structure is made up by OL 37.3k steel grade ($\sigma_a = 1600$ daN/cm^2 and $\sigma_c = 2400$ daN/cm^2) and it is considered $\mu_x = \mu_y = \mu_\omega = 1$.



a) **b)**

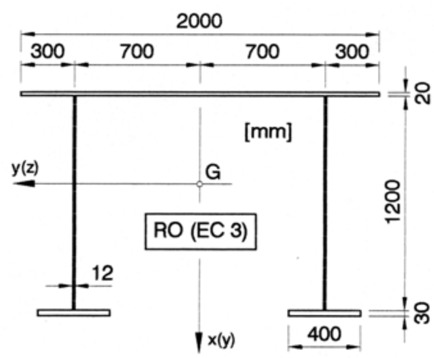


Fig. 1

2.Buckling checking of the compression member

2.1. Cross-section properties

Evaluating the gross cross – section centroid, shear centre (Fig. 2) result the cross – section characteristics presented in Table 1.

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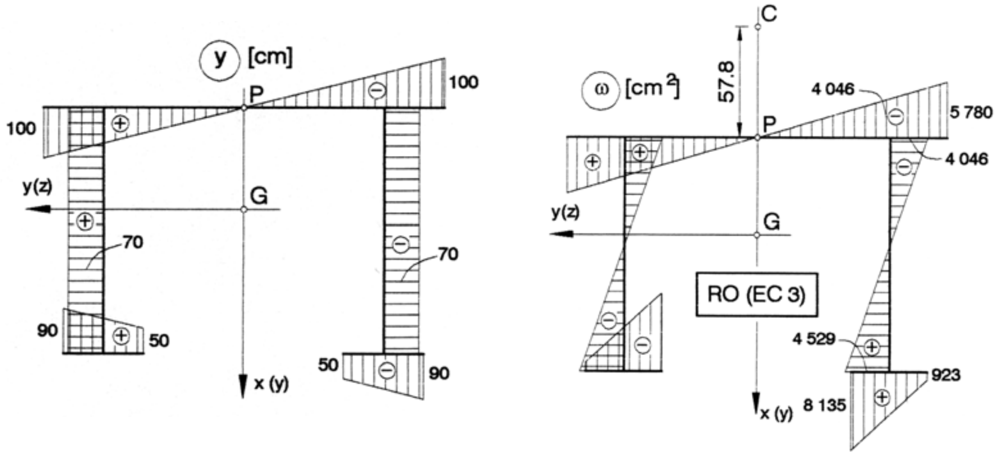

Fig. 2

Table 1

EC 3	A_g	y_G	y_C	I_y	I_z	J	C_ω	i_y	i_z	i_0^2
RO	**A**	x_G	x_C	I_x	I_y	I_t	I_ω	i_x	i_y	i_c^2
U.M.	cm^2	Cm	cm	X 10^6 cm^4	X 10^6 cm^4	cm^4	X 10^{10} cm^6	cm	cm	cm^2
	928	51	57,8	3,953	2,642	1392	1,2258	65,26	53,36	18 944

2.2. Load bearing capacity according to Romanian norms

The critical buckling forces will be:

$P_{x.cr} = 56{,}8 \cdot 10^6$ daN;

$P_{y.cr} = 38{,}0 \cdot 10^6$ daN;

$P_\omega = 9{,}35 \cdot 10^6$ daN,

where the intermediate parameters have been used:

$$i_c^2 = \frac{I_x + I_y + A \cdot x_c^2}{A} = 18\,944 \text{ cm}^2;$$

$$I_t = \frac{1}{3}\left(200 \cdot 2^3 + 2 \cdot 120 \cdot 1{,}2^3 + 2 \cdot 40 \cdot 3^3\right) = 1392 \text{ cm}^4$$

Solving equation (9) from [8] it results: critical forces P_1 and P_2 and critical stress $\sigma_1 = \sigma_{cr}$:

$P_1 = 8{,}43 \cdot 10^6$ daN $< P_{y.cr}$, $P_2 = 168 \cdot 10^6$ daN

$$\sigma_1 = \frac{P_1}{A} = \frac{8{,}43 \cdot 10^6}{928} = 9\,084 \text{ daN/cm}^2 = \sigma_{cr}.$$

Under the form (13) from [8]:

$$\sigma_{cr}^* = 2\,400\left(1 - 0{,}25 \cdot \frac{2\,400}{9\,084}\right) = 2\,241 \text{daN/cm}^2$$

it results $\lambda_{tr} = \sqrt{\dfrac{\pi^2 E}{\sigma_{cr}^*}} = 96$

and $\varphi_{tr} = 0{,}553$ (curve b – SR 1911-98).

The load bearing of member will be:

$$N_{cap}^{i-r} = \varphi_{tr} \cdot A \cdot \sigma_a = 0{,}553 \cdot 928 \cdot 1\,600 = 821 \cdot 10^3 \text{ daN}$$

2.3. Buckling resistance of member according to EC 3

Effective cross section

Top flange effective area

The top flange can be divided into two simply supported elements (cantilevers) and an internal doubly supported plate.

The cantilevers under uniform compression (positive compression), see Figure 3:

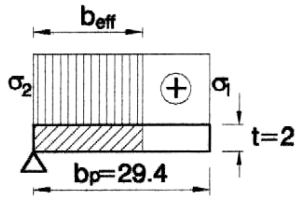

Fig. 3

$$\sigma_2 = \sigma_1 \Rightarrow \psi = \frac{\sigma_2}{\sigma_1} = +1$$

The buckling coefficient for the flange element is:

$$k_\sigma = 0{,}43$$

$$\bar{\lambda}_p = 1{,}052 \frac{b_p}{t} \sqrt{\frac{\sigma_{comp}}{E \cdot k_\sigma}} =$$

$$= 1{,}052 \cdot \frac{29{,}4}{2{,}0} \sqrt{\frac{2400}{2{,}1 \cdot 10^6 \cdot 0{,}43}} = 0{,}79 > 0{,}673$$

It results

$$\rho = \left(1 - \frac{0{,}22}{\bar{\lambda}_p} \right) \cdot \frac{1}{\bar{\lambda}_p} = \left(1 - \frac{0{,}22}{0{,}79} \right) \cdot \frac{1}{0{,}79} = 0{,}91$$

The flange effective width:

$$b_{eff} = \rho \cdot b_p = 0{,}91 \cdot 29{,}4 = 27{,}0\,\text{cm}.$$

The internal part of the flange between webs, Figure 4, a doubly supported plate under uniform compression:

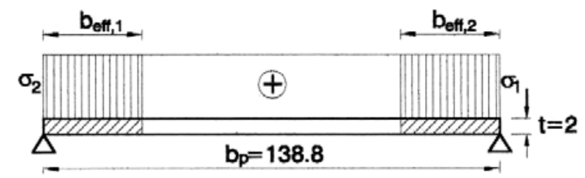

Fig. 4

$$\sigma_2 = \sigma_1 \Rightarrow \psi = \frac{\sigma_2}{\sigma_1} = +1$$

The buckling coefficient for the flange element is:

$$k_\sigma = 4\,.$$

$$\bar{\lambda}_p = 1{,}052 \cdot \frac{138{,}8}{2{,}0} \sqrt{\frac{2400}{2{,}1 \cdot 10^6 \cdot 4}} = 1{,}23 > 0{,}673$$

It results:

$$\rho = \left(1 - \frac{0{,}22}{\bar{\lambda}_p} \right) \cdot \frac{1}{\bar{\lambda}_p} = \left(1 - \frac{0{,}22}{1{,}23} \right) \cdot \frac{1}{1{,}23} = 0{,}665\,.$$

The effective flange width:

$$b_{eff} = \rho \cdot b_p = 0{,}665 \cdot 138{,}8 = 92{,}4\,\text{cm}\,,$$

$$b_{eff.1} = b_{eff.2} = b_{eff} / 2 = 46{,}2\,\text{cm}\,.$$

Bottom flange

Each flange in made up by two cantilevers (simply supported elements), Figure 5:

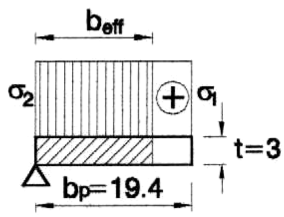

Fig. 5

$$\sigma_2 = \sigma_1 \Rightarrow \psi = \frac{\sigma_2}{\sigma_1} = +1$$

The buckling coefficient for flange is $k_\sigma = 0{,}43$

$$\overline{\lambda}_p = 1{,}052 \cdot \frac{19{,}4}{3{,}0}\sqrt{\frac{2400}{2{,}1 \cdot 10^6 \cdot 0{,}43}} = 0{,}35 < 0{,}673$$

It results $\rho = 1$

The bottom flange effective width:

$$b_{eff} = \rho \cdot b_p = 1 \cdot 19{,}4 = 19{,}4\,\text{cm}$$

Web effective area

The web is a doubly supported element under uniform compression as it is presented in Figure 6:

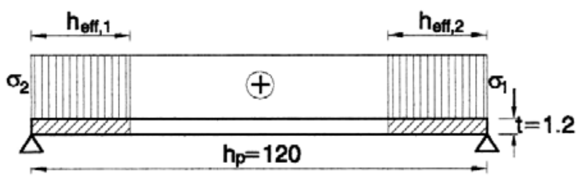

Fig. 6

$$\sigma_2 = \sigma_1 \Rightarrow \psi = \frac{\sigma_2}{\sigma_1} = +1$$

$$k_\sigma = 4$$

$$\overline{\lambda}_p = 1{,}052 \cdot \frac{120}{1{,}2}\sqrt{\frac{2400}{2{,}1 \cdot 10^6 \cdot 4}} = 1{,}78 > 0{,}673$$

It results $\rho = \left(1 - \frac{0{,}22}{1{,}78}\right) \cdot \frac{1}{1{,}78} = 0{,}49$;

$h_{eff} = 0{,}49 \cdot 120 = 58{,}8\,\text{cm}$;

$h_{eff.1} = h_{eff.2} = h_{eff} / 2 = 29{,}4\,\text{cm}$

The shift of effective cross-section neutral axis with respect to the gross cross-section is ignored. It might succeed in practical design and allow an evaluation of member bearing capacity under concentric compression only. This will yield a slightly overestimated value of member capacity, however it is a practical method to evaluate load bearing capacity.

The effective cross-section of the member is presented in Figure 7.

The effective cross section area will be:

$$A_{eff} = 2(74{,}4 \cdot 2 + 40 \cdot 3 + 2 \cdot 29{,}4 \cdot 1{,}2) = 678{,}7\,\text{cm}^2$$

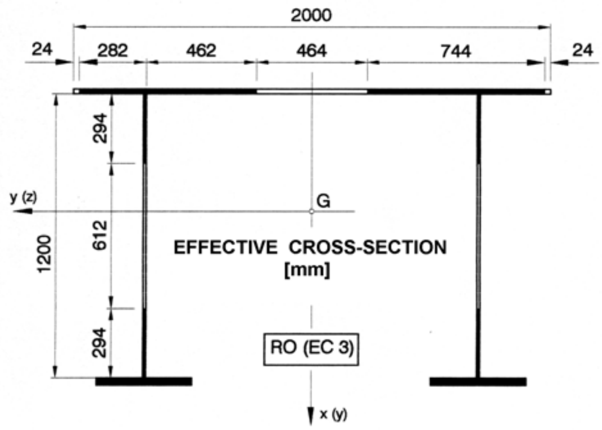

Fig. 7

Loading bearing capacity of the member under concentric compression

Flexural buckling of concentrically loaded compression member

Since the cross-section is mono-symmetrical and $I_z < I_y$, the risk of flexural buckling exists, the member design resistance being:

$$N^F_{b.Rd} = \chi \cdot A_{eff} \cdot f_y \cdot \frac{1}{\gamma_{M1}}$$

where the safety factor $\gamma_{M1} = 1{,}1$.

The reduction coefficient χ should be computed using $\alpha = 0{,}49$ (“c” buckling curve).

$$\overline{\lambda} = \sqrt{\frac{\beta_A \cdot A \cdot f_y}{N_{cr}}} = \frac{\lambda}{\lambda_1} \cdot \sqrt{\beta_A}\ ;$$

$$\lambda = \frac{l}{i_y} = \frac{1200}{53{,}36} = 22{,}49$$

The member buckling length for flexural buckling, conservatively taken as equal to member actual length, is $l = 1200$ cm.

$$i_g = \min \left\lfloor i_z ; i_y \right\rfloor = 53{,}36 \text{ cm}$$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = \pi\sqrt{\frac{2{,}1 \cdot 10^6}{2400}} = 93 ;$$

$$\beta_A = \frac{A_{eff}}{A_g} = \frac{678{,}7}{928} = 0{,}73$$

$$\overline{\lambda} = \frac{22{,}49}{93}\sqrt{0{,}73} = 0{,}206$$

$$\phi = 0{,}5\left[1 + \alpha\left(\overline{\lambda} - 0{,}2\right) + \overline{\lambda}^2\right] =$$
$$= 0{,}5\left[1 + 0{,}49(0{,}206 - 0{,}2) + 0{,}206^2\right] = 0{,}52$$

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \overline{\lambda}^2}} = \frac{1}{0{,}52 + \sqrt{0{,}52^2 - 0{,}206^2}} = 1$$

It results:

$$N^F_{b.Rd} = 1 \cdot 678{,}7 \cdot 2400 \cdot \frac{1}{1{,}1} = 1\,481 \cdot 10^3 \text{ daN} .$$

Flexural-torsional buckling of concentrically loaded compression member

Mono-symmetrical cross-section members are generally subjected to flexural-torsional buckling.

Member corresponding strength should be determined using a similar formula to the former case:

$$N^{FT}_{b.Rd} = \chi \cdot A_{eff} \cdot f_y \cdot \frac{1}{\gamma_{M1}}$$

The only significant differences appear when calculating χ reduction coefficient, i.e.:

$$\overline{\lambda} = \sqrt{\frac{f_y}{\sigma_k}} \cdot \sqrt{\beta_A} \; ; \quad \sigma_k = \min[\sigma_T ; \sigma_{FT}]$$

where:

$$\sigma_T = \frac{1}{A_g \cdot i_0^{\,2}}\left(G \cdot I_T + \frac{\pi^2 \cdot E \cdot C_w}{L_T^{\,2}}\right) =$$
$$= \frac{P_\omega}{A_g} = \frac{9{,}35 \cdot 10^6}{928} = 10\,075 \text{ daN/cm}^2$$

$$\sigma_{Ey} = \frac{\pi^2 \cdot E}{\lambda_y^{\,2}} = \frac{\pi^2 \cdot E}{(1200 / 65{,}26)^2} = 61\,207 \text{ daN/cm}^2.$$

The critical stress for flexural-torsional buckling is:

$$\sigma_{FT} = \frac{1}{2\beta}\left[\left(\sigma_{Ey} + \sigma_T\right) - \sqrt{\left(\sigma_{Ey} + \sigma_T\right)^2 - 4\beta\sigma_{Ey} \cdot \sigma_T}\right]$$
$$= \frac{P_1}{A_g} = 9\,084 \text{ daN/cm}^2$$

where: $\beta = 1 - \left(\frac{y_0}{i_0}\right)^2 = 0{,}375$.

It results:

$$\sigma_k = \min[\sigma_T ; \sigma_{FT}] = 9\,084 \text{ daN/cm}^2$$

The reduced slenderness for flexural-torsional buckling $\overline{\lambda}$, ϕ and the reduction coefficient χ will be:

$$\overline{\lambda} = \sqrt{\frac{2400}{9084}} \cdot \sqrt{0{,}73} = 0{,}44$$
$$\phi = 0{,}5\left[1 + 0{,}34(0{,}44 - 0{,}2) + 0{,}44^2\right] = 0{,}64$$

$$\chi = \frac{1}{0{,}64 + \sqrt{0{,}64^2 - 0{,}44^2}} = 0{,}90 < 1$$

The load bearing capacity, against member flexural-torsional buckling will be:

$$N_{b.Rd}^{FT} = 0{,}90 \cdot 678{,}7 \cdot 2400 \cdot \frac{1}{1{,}1} = 1333 \cdot 10^3 \text{ daN}$$

Member buckling resistance in centric compression will be:

$$N_{b.Rd} = \min\left[N_{b.Rd}^{F}; N_{b.Rd}^{FT}\right] =$$
$$= \min\left[1481 \cdot 10^3; 1333 \cdot 10^3\right] = 1333 \cdot 10^3 \text{ daN}$$

In conclusion the member failure occurs via flexural-torsional buckling.

3. Conclusions

According to EC 3 the load bearing capacity of the compression member is evaluated by taking into account the effective cross-section area and even more accurately considering the bending moment caused by the shift of effective cross-section neutral axis with respect to gross cross-section.

The obtained results through the two presented methods can be compared by the ratio $\chi \cdot A_{eff} / \varphi \cdot A_{br}$ which in this example has a value of 1,19.

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Considerations in the Design of Passive, Active and Semi-active Systems of the Structures under Seismic Actions

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Rezumat: Articolul prezintă criterii de proiectare a strategiilor de control pasive, active si semi-active luând in considerare balanțul de energie pentru întregul sistem (mecanismul de control şi structura). Dacă sistemele de control pasiv sunt folosite la mărirea amortizării, rigidității si rezistenței structurale, celelalte tehnici de control folosesc forte controlate pentru a introduce, disipa, sau ambele, energie intr-o structura, prin dispozitive specifice suplimentate de senzori, controler (ex. calculator) si procese de informare in timp real pentru a opera. Pentru rezultatele analitice va fi considerat modelul cu grinzi infinit rigide al unei structuri cu un singur grad de liberate, supus la componenta NS a acțiunii seismice El Centro.

Abstract: The paper presents design criterions of passive, active or semi-active control strategies taking into account the balance energy for the whole system (control mechanism and structure). If passive control systems are used for enhancing the structural damping, stiffness or strength, the other control techniques employ controllable forces to add or to dissipate energy in a structure, or both, due to the specific devices integrated with sensors, controllers and real-time processes to operate. For analytical results, there will be considered the shear-type model of a single-degree-of freedom structure under the NS component recorded at El Centro.

Keywords: *control system, structure, balance energy, energy dissipation.*

1. Introduction

During the past decades many techniques have been proven to develop successful physical, analytical, numerical and experimental models in predicting the dynamic behaviour of the civil engineering systems that are subjected to excitations. Therefore, in recent years, it has been paid a considerable attention to new concepts of structural control including passive, active and semi-active techniques. Passive, semi-active or stable active control reduces the energy demand for a structure. It's well to know that the effect of a lower energy demand through the use of active or semi-active control can be met by a passive control [3]. The analytical studies with respect to the energy balance of the model are carried out to mitigate the energy dissipation demand for the primary structural elements in the following four cases: 1) the non-linear behaviour of the primary structural system; 2) a passive fluid damper attached at the primary structural system; 3) an active system that applied control forces on the structural mass; 4) a semi-active fluid damper attached at the primary structural system.

Design decisions in order to choose a right control strategy regarding the reduction of energy requirement and the decreasing of the displacement response for a SDOF structure are discussed.

2. Energy Balance

Generally, the work done by the external forces acting on a system is equal to the sum of the mechanical energy stored temporarily in the structure (kinetic and potential energy) and the energy transformed to another form, through either energy dissipation or absorption mechanisms. The energy balance equation takes the following form:

$$E_{input} = E_k + E_{el} + E_{pl} + E_{da} + E_{sup} \qquad (1)$$

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where,

E_{input} - energy inputted in the structure by ground excitation;

$$E_{input} = \int_0^t f_{input}(\tau)dx(\tau) = \\ = \int_0^t -ma_g(\tau)\frac{dx(\tau)}{d\tau} = -m\int_0^t a_g(\tau)v(\tau)d\tau \tag{2}$$

E_k - kinetic energy stored in the mass that is equal to the work done by the force f_m on the mass;

Considering the Newton's second law given by

$$f(\tau) = ma(\tau) \tag{3}$$

the relationship becomes

$$E_k = \int_0^t f_m(\tau)dx(\tau) = \int_0^t f_m(\tau)\frac{dx(\tau)}{d\tau}d\tau = \\ = \int_0^t f_m(\tau)v(\tau)d\tau = m\int_0^t a(\tau)v(\tau)d\tau = \frac{m}{2}v(\tau)^2 \tag{4}$$

E_p - potential energy stored in the spring that is equal to the work done by the force f_p on the spring;

$$E_p = E_{el} + E_{pl} = \int_0^t f_p(\tau)dx(\tau) = \\ = \int_0^t f_p(\tau)v(\tau)d\tau \tag{5}$$

where,

E_{el} - energy due to elastic deformations;

$$E_{el} = \int_0^t kx(\tau)v(\tau)d\tau = \frac{k}{2}x(\tau)^2 \tag{6}$$

E_{pl} - energy due to plastic deformations;

E_{da} - energy dissipated due to the viscous damping of the structure;

$$E_{da} = \int_0^t f_d(\tau)dx(\tau) = \int_0^t cv(\tau)v(\tau)d\tau = \\ = c\int_0^t v(\tau)^2 d\tau \tag{7}$$

E_{sup}- energy dissipated or added by the supplemental passive, semi-active or active system.

The passive and semi-active control provide energy storage and energy dissipation. The magnitude of active control force adds the energy in structure. The actuator force is selected to oppose the motion and does negative work on the mass. Considering an active force f_{act} proportional to the displacement and the velocity, the relationship is given by

$$f_{act} = -k_{act}x(\tau) - c_{act}v(\tau) \tag{8}$$

The active control based on the equation can be interpreted as introducing virtual stiffness and damping. In case of linear SDOF the energy balance equation becomes:

$$-m\int_0^t a_g(\tau)v(\tau)d\tau + \int_0^t f_{act}v(\tau)d\tau = \\ = \frac{m}{2}v(\tau)^2 + \frac{k}{2}x(\tau)^2 + c\int_0^t v(\tau)^2 d\tau \tag{9}$$

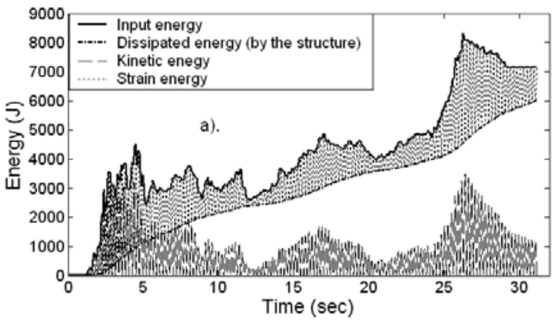
When f_{act} is taken to have the same sign as v, active control decreases the energy input to the system since the integral is always negative.

3. Case Study of a SDOF Structural Model

Let's consider a SDOF model in order to make energy comparisons among passive, semi-active and active control strategies. The corresponding dynamic characteristics are as follow: m=10000Kg; k=2000000N/m; c=2815Ns/m; ω=14.14rad/s; T=0.44s; f=2.25Hz; ξ=0.01. The system is subjected to El Centro seismic record.

It's known that damping reduces the strain energy and the system response, especially in the vicinity of the resonance. Figures 1(a) and 1(b) illustrate the influence of viscous damping for energy balance response in two cases, when the system is under El Centro's earthquake. For low viscous damping case ($\xi = 0.005$), the energy dissipated per cycle is small. For a greater damping value ($\xi = 0.01$) the dissipated energy increases, and the stored energy is reduced.

The approach using energy dissipation mechanisms is to transfer as much energy as possible from the primary structural members to secondary damping or ductile elements. Let's consider now that the stiffness elements of the structure are designed as energy dissipation devices. In this case, a hysteresis loop corresponds to energy dissipation and is formed by the cyclic inelastic deformation path (Fig. 2(b)).



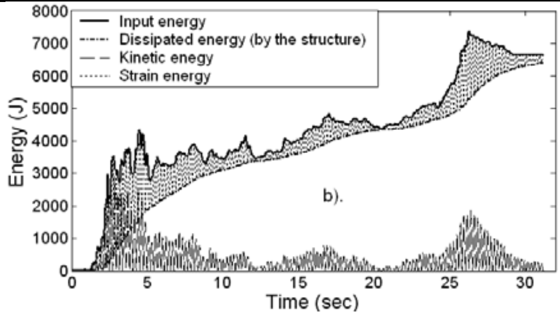
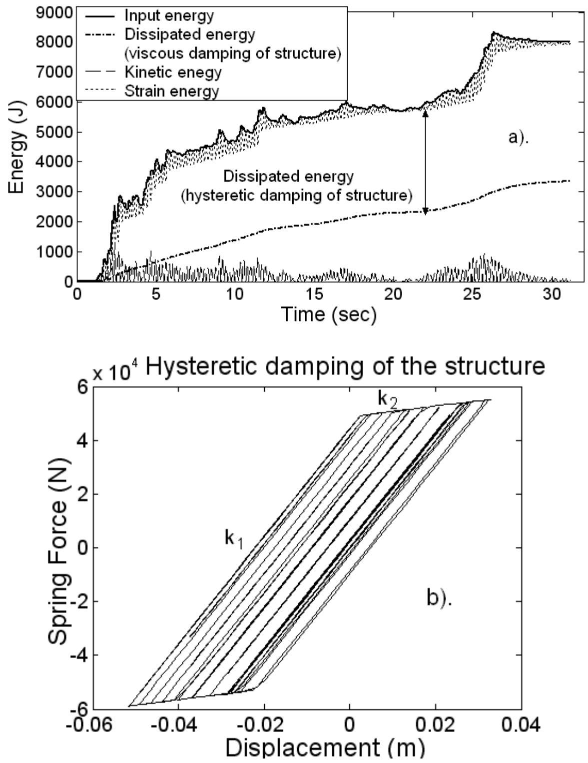


Fig. 1. Energy balance, Uncontrolled system, a) $\xi = 0.005$ and b) $\xi = 0.01$

Figures 2(a) and 2(c) show the energy build up and displacement response respectively, taking into account the following conditions for the non-linear SDOF system: k_1=k, k_2=0.1k and yield displacement u_y=2.7cm. It's seen that the energy dissipated by the hysteretic damping of the model is approximately 50% and also the displacements decrease. The inconvenient are that after more inelastic cycles the structure could fail and also the increasing of the input energy.



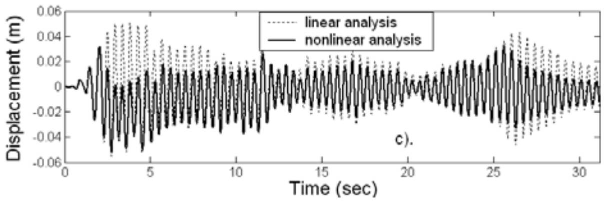
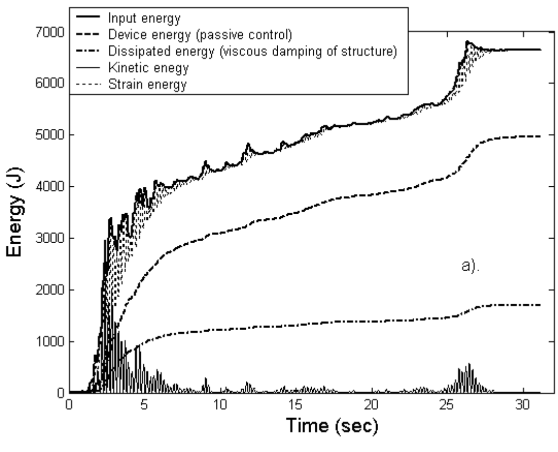


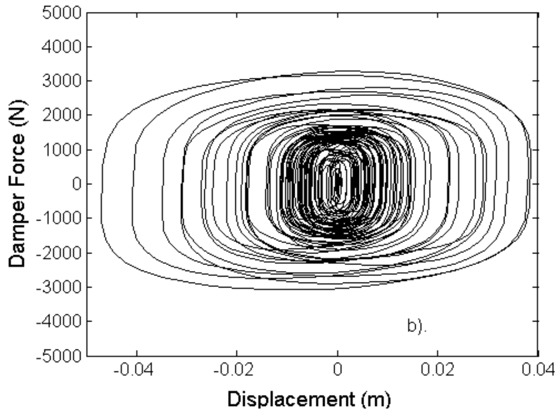
Fig. 2.
a) Energy balance;
b) Force-Displacement (hysteretic damping of structure);
c) Comparison of SDOF displacement response between linear analysis case ($\xi = 0.01$) and non-linear analysis case

Damping devices installed at discrete locations in structures supplement their natural energy dissipation capabilities. Results are show in Figures 3(a), 3(b) and 3(c) supposing that the structure works in elastic domain and it's equipped with a passive fluid damper. The force in the fluid damper is expressed as

$$f = C|\dot{x}|^{\alpha} \operatorname{sgn}(\dot{x}) \tag{10}$$

where $\dot{x}$ =velocity of the piston rod, C = 5000 constant and α = 0.5 coefficient in the range of approximately 0.5 to 2.0.





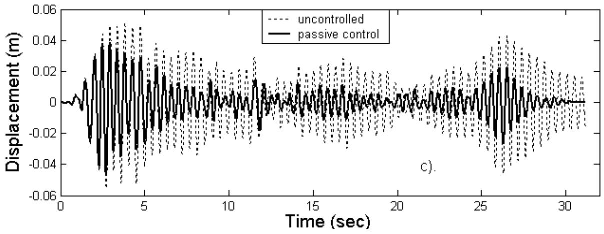


Fig. 3.
a) Energy balance
b) Force-Displacement (passive fluid damper);
c) Comparison of SDOF displacement response between linear analysis case ($\xi = 0.01$) and controlled case with passive fluid damper

In this case the demand of energy absorption capacity on the main structural members and the response of the system is reduced. Thus, the possibility of structural damage is minimized. When the maximum response of the system is achieved, the peak of input energy is decreased approximately with 25%.

For active control purpose, a SIMULINK model was developed for SDOF response. The model is based on physical balance of forces and energies (Fig. 4). Two cases are analyzed for the design of active control system. Both cases take into account only the velocity and for the case A is used a gain *g*=20000 Ns/m and in the case B, g=65000 Ns/m. The obtained gains are based on quadratic optimal control. Results are shown in Figure 5, for both cases, when the model is subjected to El Centro's earthquake. In both cases the results are better than passive case.

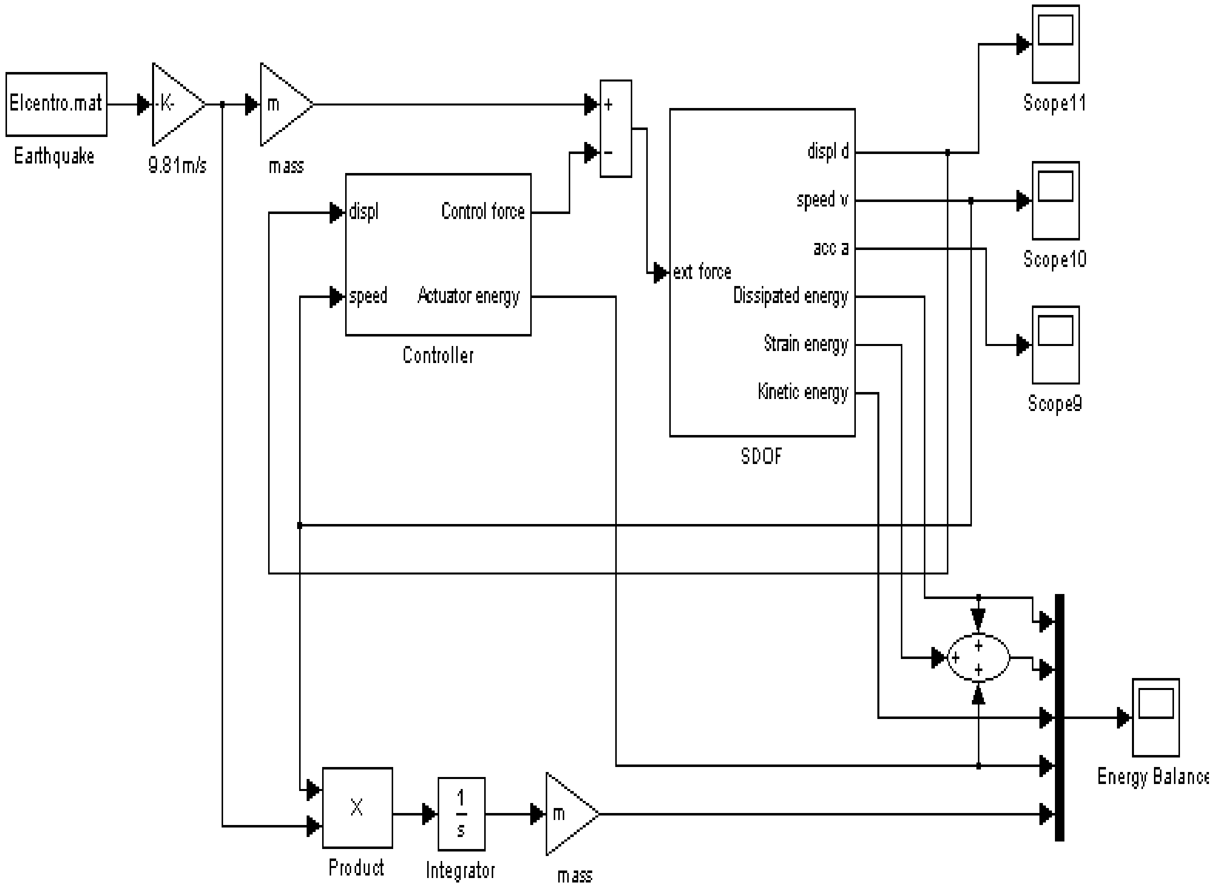
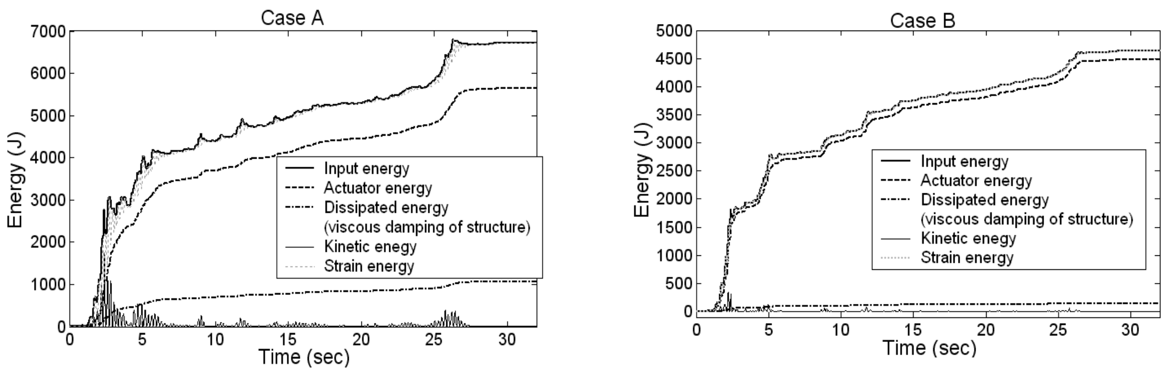


Fig. 4. Simulink model of SDOF structural system



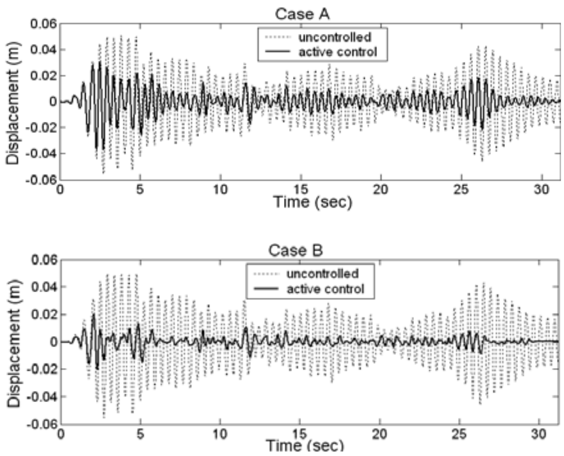


Fig. 5. Energy Balance: case A and case B; Comparison of SDOF displacement response between uncontrolled case ($\xi = 0.01$) and controlled case with active system

In the case A, the actuator must be able to produce a maximum force of 9000 N. If we want to have a greater decreasing of the displacements, the input energy and the stored energy, the case B is preferable. Thus, the requirement of the power source increases and the actuator will have to be able to produce a maximum force of 20000 N.

Semi-active control device have, in fact, the behaviour of passive mechanisms but they require a small amount of power energy, for instance a battery, to change the stiffness or damping of the device. In most design cases of semi-active techniques the control forces track the forces that could be applied by an active control [2]. Therefore, a semi-active control device could achieve the performance of an active control system [1].

There is considered a variable orifice device in order to control the damping force f. On-off clipping control is used in order to design the semi-active strategy. There are two steps in order to design a clipping control for any type of semi-active device. First, an active control law computes a desired control force f_c. In this case any type of active strategies can be chosen (optimal control, integral force feedback control, H_2 or H_∞ control) because the second step is independent from it. Next, secondary clipping controller tries to make the semi-active device to replicate the force that the active device would apply on the structure. For on-off clipping control, the following rule is usually used: If the magnitude of the force f produced by the device is smaller than the magnitude of the desired optimal force and the two forces have the same sign, the voltage applied to the current driver is increased to the maximum level so as to increase the force produced by the damper to match the desired control force. Otherwise, the commanded voltage is set to zero. The command signal is described by the following formula:

$$U = U_{max} H\left[\left(f_c - f\right) f\right] \qquad (11)$$

where U is the command signal, $H[.]$ is the Heaviside step function, U_{max} is the maximum voltage applicable on the semi-active device to obtain the maximum damping and f and f_c are the measured and required control forces. The damper is controlled by two functions capable to describe the passive damping force versus velocity paths. For numerical simulations, when the commanded voltage is set to zero, the force is expressed by formula (10) but the constant C has the value of 1000.

The results in active control case A can be compared with the results in semi-active control case (Fig. 6). It's preferable to choose the use of the semi-active control damper because of the close results regarding the displacement response, stored energy and energy dissipation (viscous damping of structure). This mechanism requires external power energy, but given by a battery, whereas the active technique cannot function without a big external source of energy. The semi-active damping device removes energy from response, like in passive case and therefore cannot cause the response to become unstable. In case of active system there is the potential for introducing instability in the system and great costs to implement such technologies.

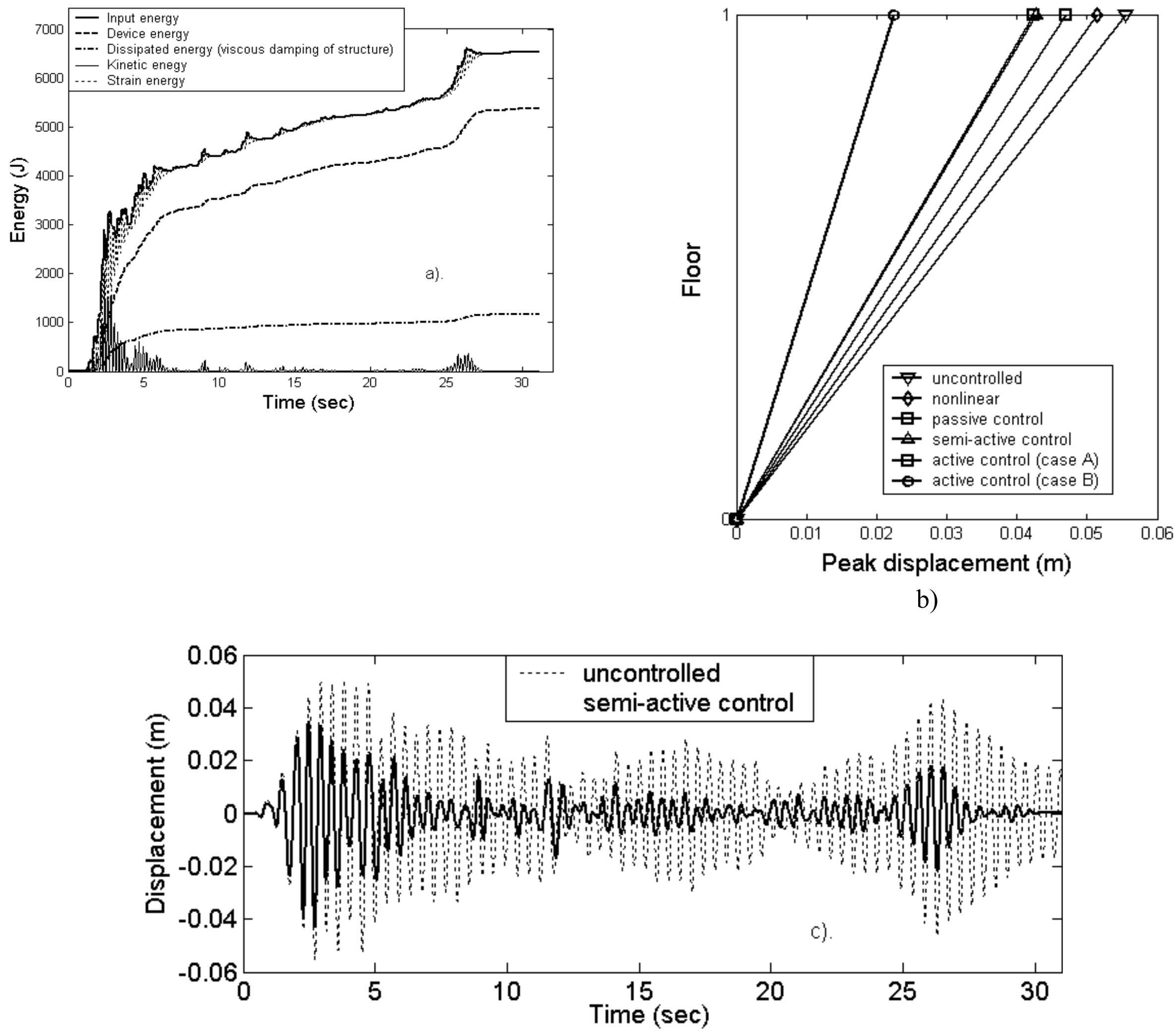


Fig. 6.
a) Energy balance: semi-active case;
b) Comparison between uncontrolled case; nonlinear case; passive case; active case A; active case B and semi-active case;
c) Comparison of SDOF displacement response between uncontrolled case ($\xi = 0.01$) and controlled case with semi-active fluid damper

4. Conclusions

From point of energy balance view, the control strategy for a particular structure subjected to a dynamic excitation, involves a number of decisions as follows:

1. What is the admissible level with respect to the structure for the energy storage and the energy dissipated by viscous damping (E_{da})?

2. Which strategies should be implemented? Passive, active or semi-active system or a combination among these.

3. The type and mechanical properties of passive energy dissipation devices, which are appropriate for structural system and their locations.

4. If semi-active or active control mechanisms are used instead of passive devices or to supplement them, the type and capacity of control forces need to be specified and compared for certain cases. Also, there must be decided on the locations for the semi-active and active systems.

After the choice of a control mechanism, an experimental campaign must be conduct in order to find its real behaviour. If unexpected characteristics of the device are seen, the control law must be modified in accordance. Once the control law is done and the mechanical properties of the device are known, the numerical simulations of the whole system (structure and control devices) must be performed.

The fundamental step is to conduct a specimen test with all the mechanical, electrical and structural limitations or problems that could be met in practical case. The final step will be the implementation of the control strategy in real structure.

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Influence of Stiffness to Torsion in Beams Networks

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Rezumat: Lucrarea pune in evidenta influenta rigiditatii la torsiune a elementelor din beton armat, printr-un studiu parametric a doua structuri simple, la care torsiunea are un caracter dominant. Acestea s-au studiat in trei ipoteze: elementele structurii nefisurate, elementele structurii fisurate si cu rigiditate redusa numai la torsiune si, a treia ipoteza, elementele structurii fisurate si cu rigiditate redusa atat la incovoiere cat si la torsiune. S-a aratat importanta considerarii corecte in calcule a rigiditatilor la incovoiere si torsiune, ca rigiditati reduse, in urma fisurarii. Neluarea in considerare a acestor rigiditati indeparteaza valorile calculate de cele reale.

Abstract: The study emphasizes the influence of stiffness to torsion of the reinforced concrete elements using a parametrical study of two simple structures where torsion is predominant. There were three hypothesizes: structure's elements non cleaved, structure's elements cleaved and reduced stiffness only to torsion, structure's elements cleaved and reduced stiffness to torsion and bending too. It has been ascertained the importance considering stiffness to torsion and bending after cleaving like reduced stiffness for a right reckon. Not considering these stiffness values gives different results comparing to reality.

Keywords: *torsion, stiffness, reinforced concrete.*

1.Introduction

We choose 2 simple structures, respectively 2 possible beams structures to emphasize the influence of stiffness to torsion of reinforced concrete elements.

2.Material and method

Example 1: The monolithic reinforced concrete floor (6m X 6m) with main and secondary beams was modeled and reckoned using SAP 05 software. The static scheme of the structure and the elements dimensions in the figure 1 will be showed.

Main beams 13,14,15 and 14,16,18 of the modeled structure will be the most stressed to torsion. The bended elements which were studied were the structures beams.

There are 3 reckoning hypothesis for this structure:

Hypothesis 1 – non cleft network's elements

Hypothesis 2 – cleft elements with decreased stiffness only to torsion

Decreasing stiffness was made step by step from 0 to 100%, respectively covering the 1 to 0 interval for I/I ratio, where ItII is the inertial moment for elements section of the beam network, in second stadium, and ItI for the primary stadium.

Hypothesis 3 – elements of the beams network are cleaved, and decreasing of stiffness is due to bending (50%) and torsion (0 to 100%) both. In figures 2 and 3 we present the bending moment and torsion moment diagrams for the interested beams of the network. Diagrams were superposed giving possibilities to compare the structure.

In figure 1 you can see the increasing of the maximum bending moment M. You can observe that if we decrease the stiffness to bending for the cleaved structure, than the bending moment value is standing somewhere between his value for the I hypothesis (non cleft structure) and II hypothesis (reducing stiffness to torsion).

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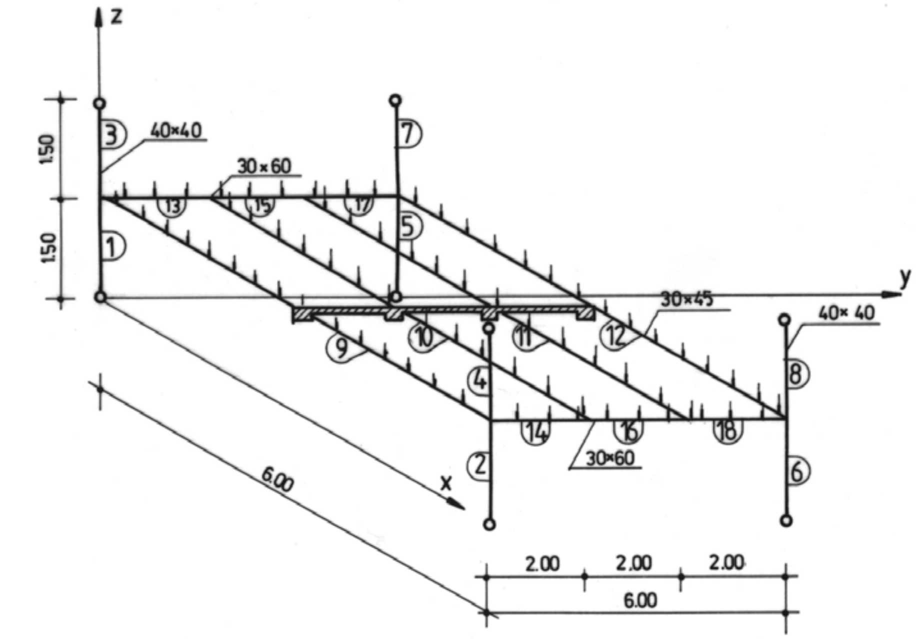


Figure 1. Static scheme of a beams network

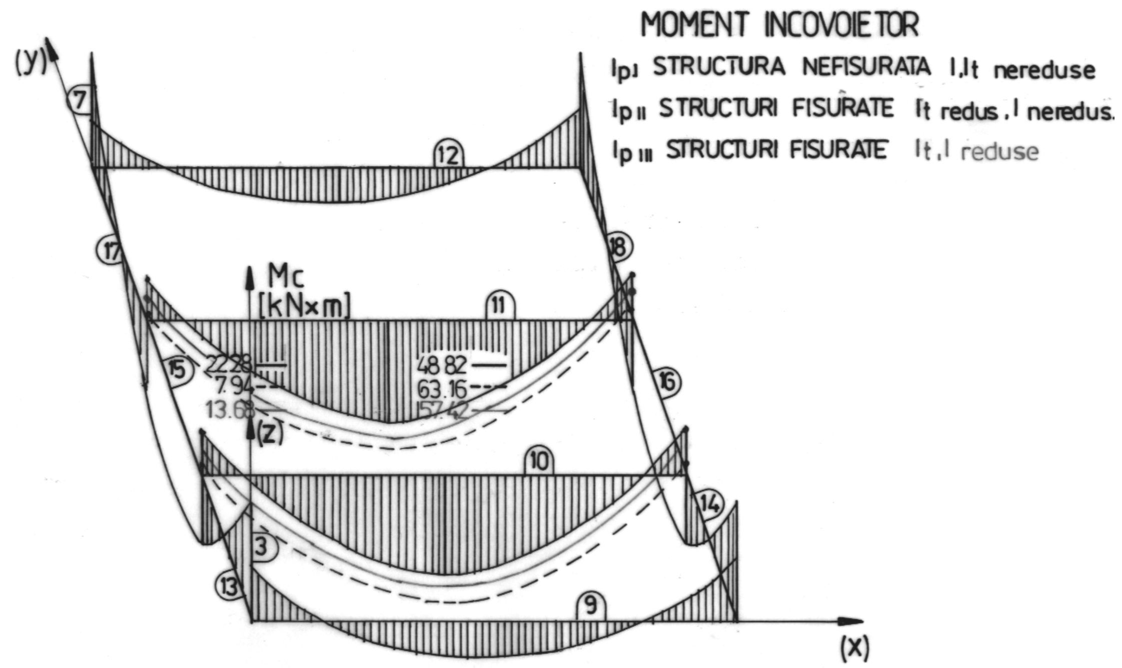


Figure 2. Bending moment diagram

Not the same way is the behavior of the structure's elements according the possibility of torsion moment taking over (fig.3).

The torsion moment value Mt, for III hypothesis is standing between the values of I and II hypothesizes with specification that inferior value is about torsion moment in cleaved structure hypothesis, and the higher value is about torsion moment of the non cleaved structure.

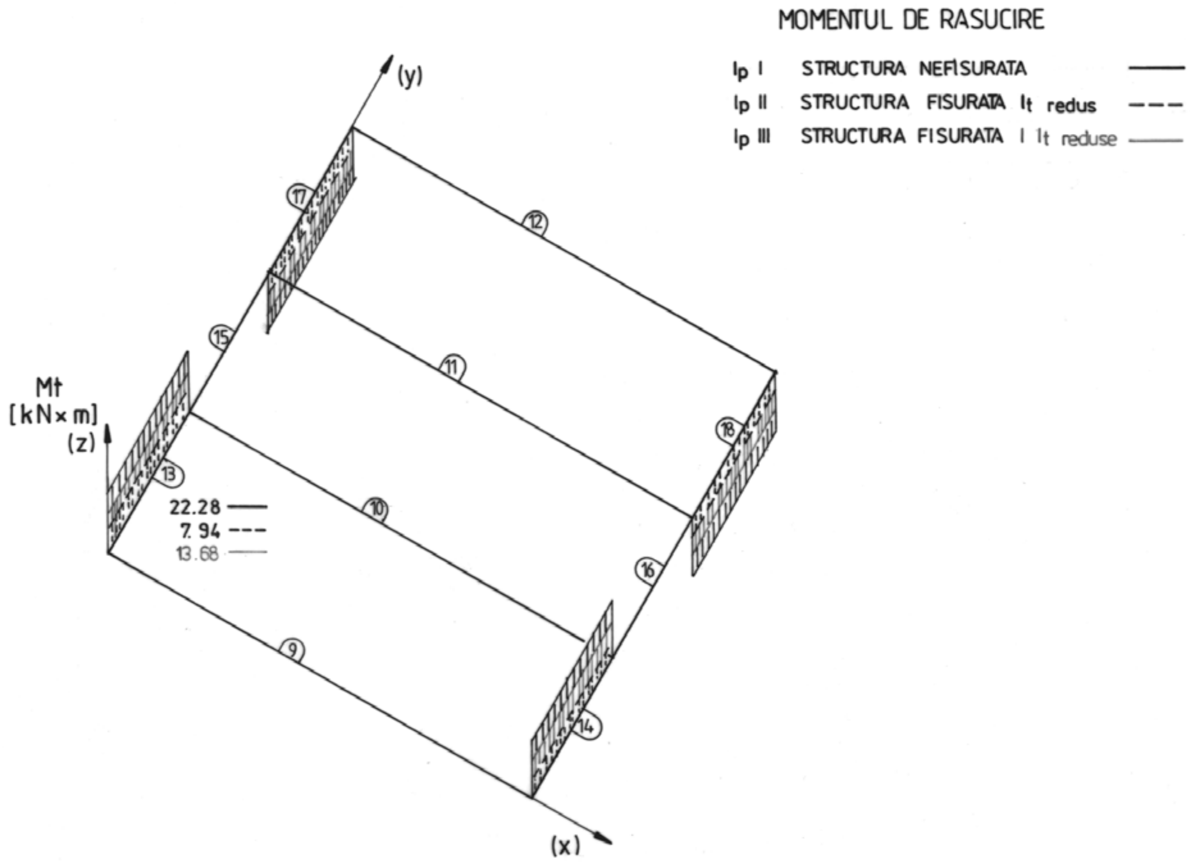


Figure 3. Torsion moment diagram

Figure 4 shows the comparative study for the both situations, when stiffness to bending decreased or not, showing dependence to torsion moment Mt through Itii/Iti ratio (element no.13, fig 4a) and dependence to bending moment Mc through Itii/Iti ratio (element no.10, fig.4b) when this ratio is from 0 to 1.

Results

This study emphasizes that:

- torsion moment decrease with reducing stiffness to torsion and increase with reducing stiffness to bending (36,44%)
- bending moment increase with reducing stiffness to torsion (45,43%) and decrease with reducing stiffness to bending (19,9%)

Example 2: We were modeling and reckoning, with SAP 05 software, a monolithic reinforced concrete floor with irregular spans (6m X 9m). The static scheme of the beams network and the elements dimensions are showed in the figure 5.

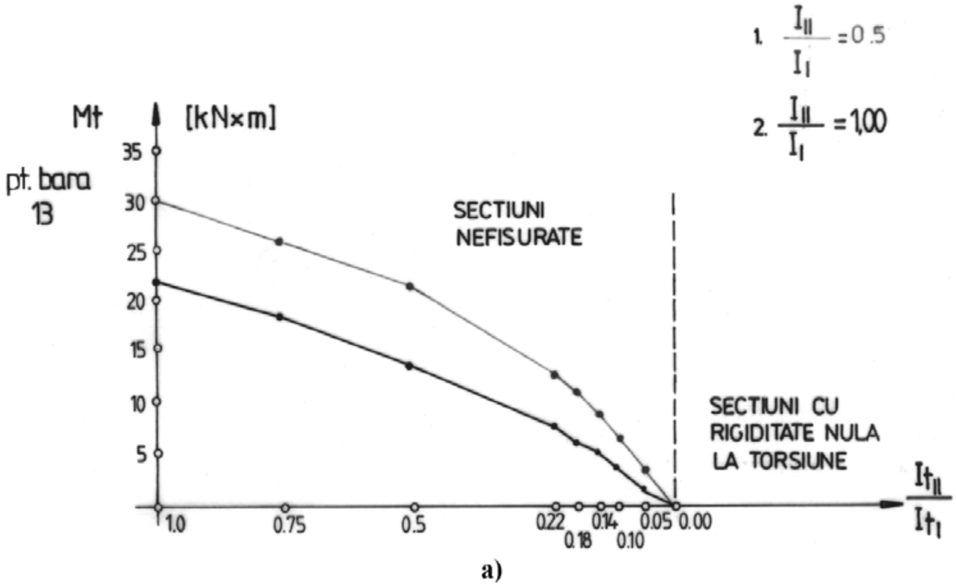


Fig. 4. a) Torsion moment dependence of $\frac{I^{II}}{I^{I}}$ ratio

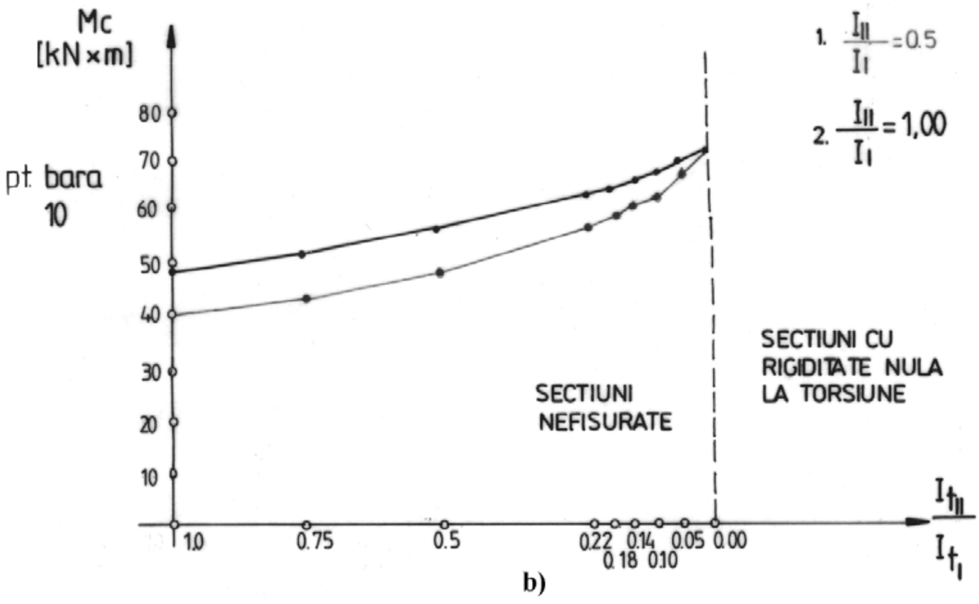


Fig. 4. b) Bending moment dependence of $\frac{I^{II}}{I^{I}}$ ratio

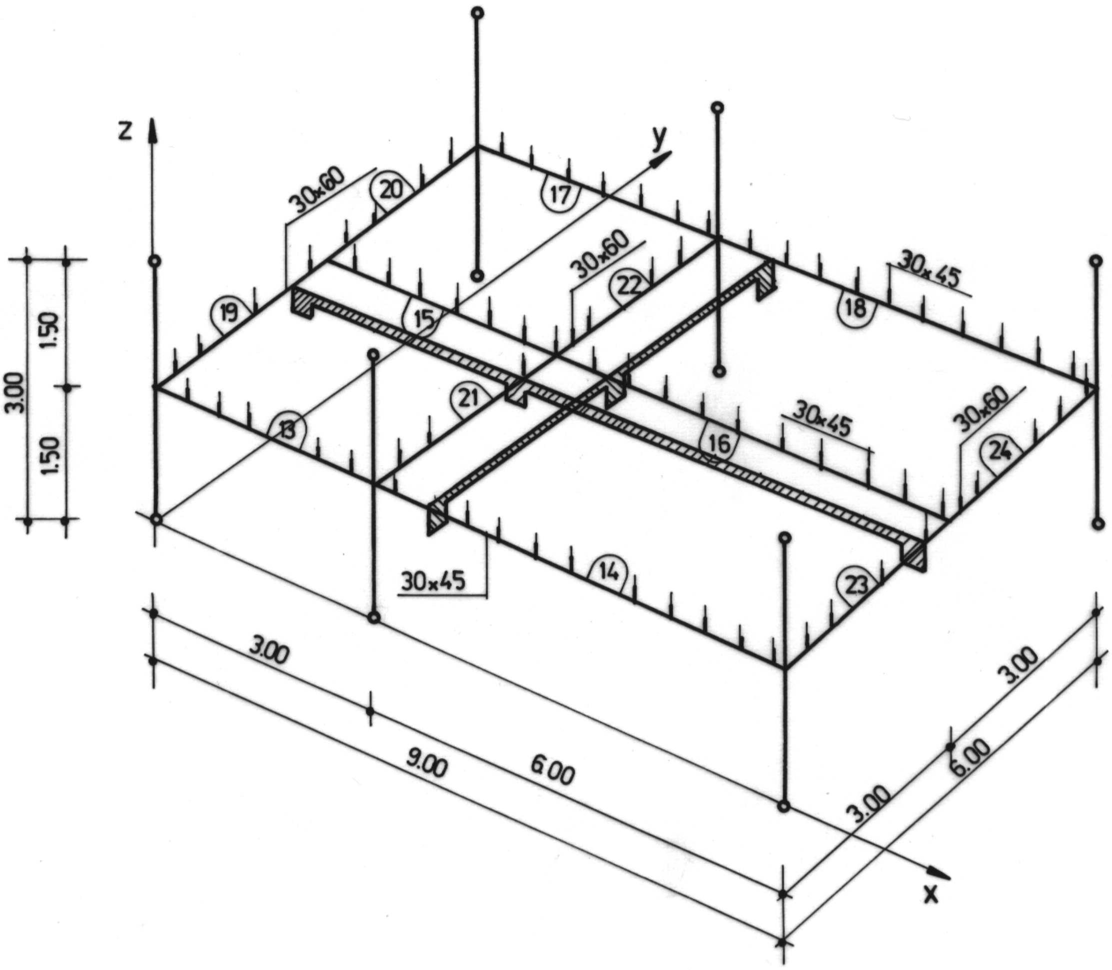


Figure 5. Static scheme of a beams network

We consider, like before, 3 reckoning hypothesis:

- Hypothesis I: structure's elements are non cleaved and "work" in elastic domain
- Hypothesis II: structure's elements are cleaved, accepting the reduce of stiffness just to torsion
- Hypothesis III: structure's elements are cleaved, reducing of stiffness due to torsion and taking over bending moments

We observe a reducing with appreciatively 40% of quality in case of torsion moments of the rotated structure's elements (figure 6), and we show bending moments diagram for the modeled structure, in these 3 reckon hypothesizes.

According to previous example in figure 7 we emphasize dependence of the bending moment and torsion moment by Itii/Iti ratio, in those two situations: when stiffness to bending is reduced (Iii/Ii = 0,5) ; when stiffness to bending is constant (Iii/Ii = 1,0) and stiffness to torsion step by step reduce it from 0 to 100% (figures 8a,8b).

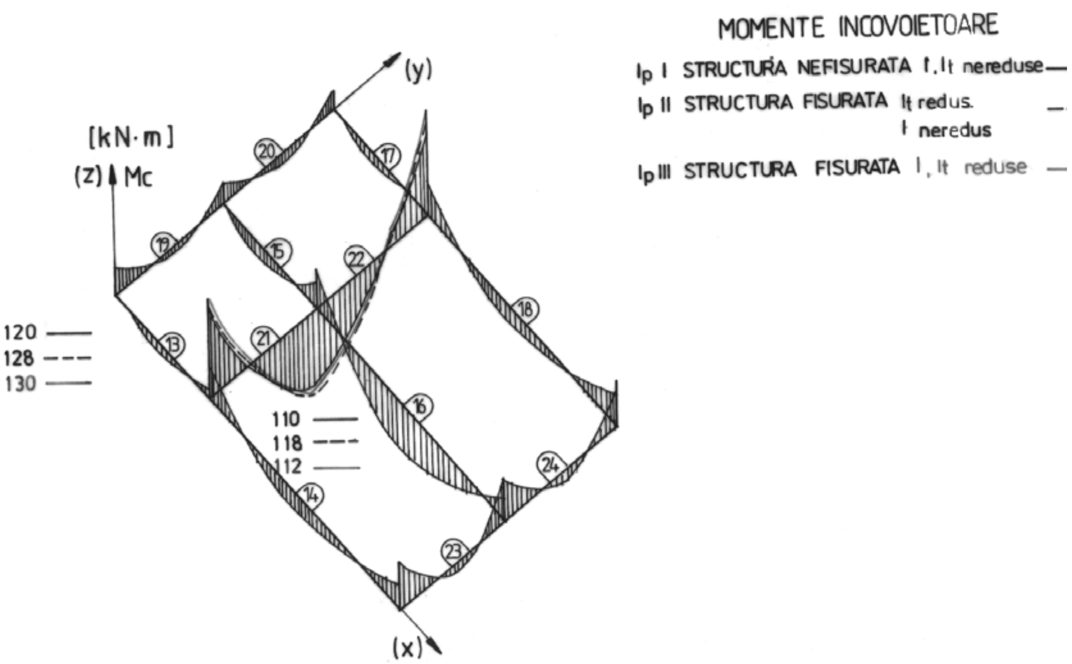


Figure 6.Bending moments diagram

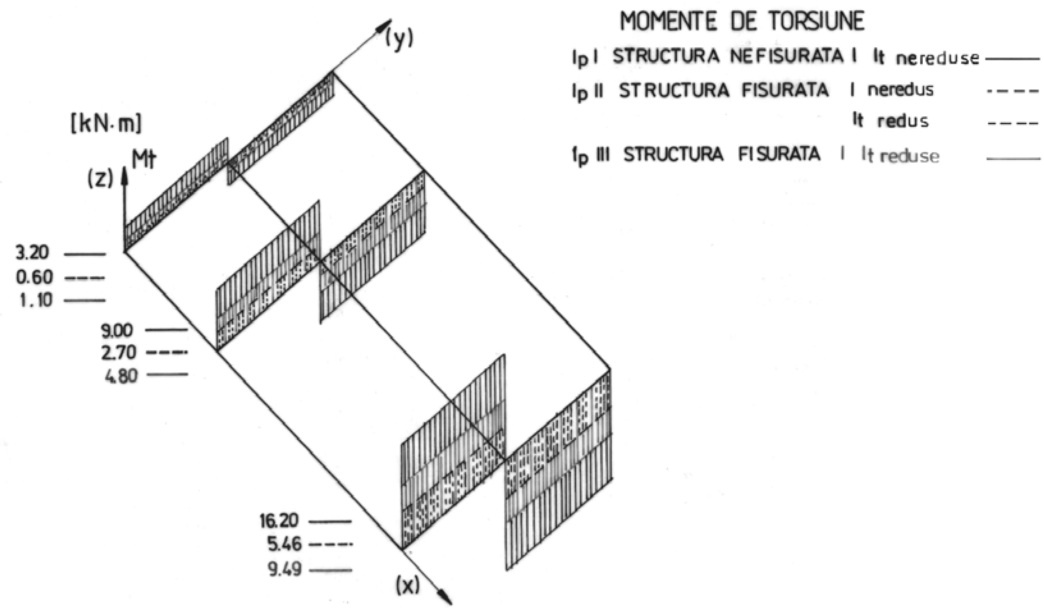
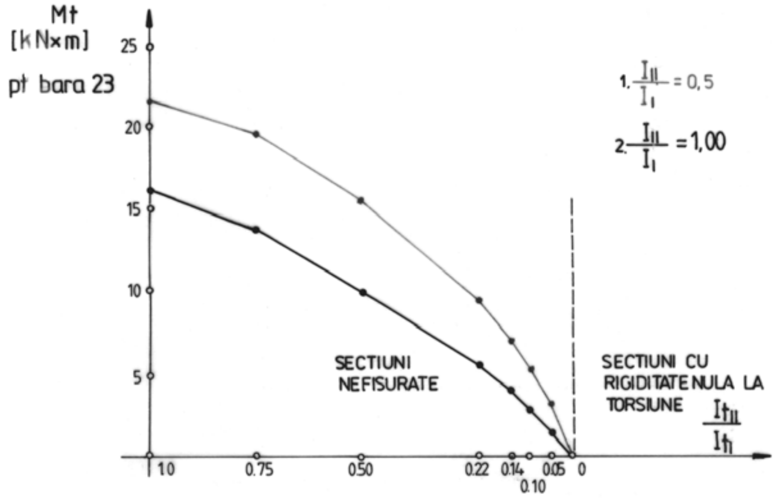


Figure 7. Torsional moments diagram

Results:

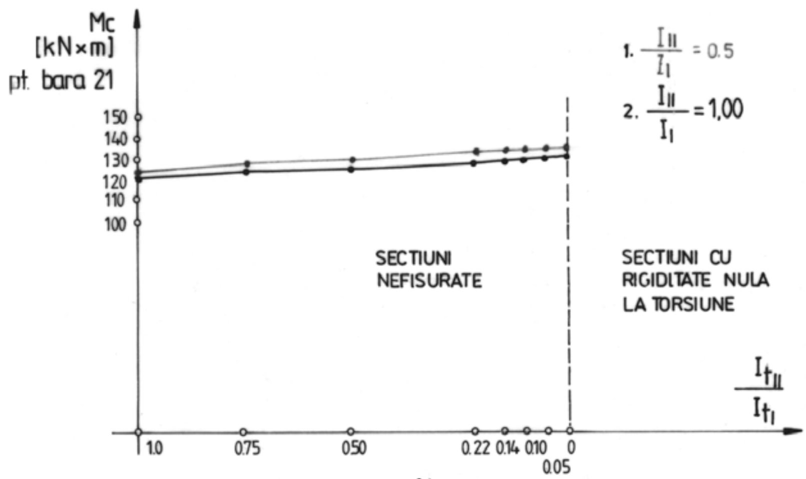
Finally we can affirm that:

- torsion moment decrease with reducing stiffness to torsion and increase with reducing stiffness to bending (36,41%)
- bending moment increase with reducing stiffness to torsion (8,33%) and increase with reducing stiffness to bending (an insignificant 0,3% increasing)



a)

Fig. 8 a) Torsion moment dependence of $\frac{I^{II}}{I^{I}}$ ratio



b)

Fig. 8 b) Bending moment dependence of $\frac{I^{II}}{I^{I}}$ ratio

3.Conclusions

It has been ascertained the importance considering stiffness to torsion and bending after cleaving like reduced stiffness for a right reckon. Not considering these stiffness values gives different results comparing to reality.

4.References:

[1] Bob Li, Dănetiu Gh . (1979) - "*Rigiditatea la torsiune a elementelor din beton armat supuse acțiunii combinate a încovoierii și torsiunii*" – referat științific – ICCPDC Timișoara

[2] Lüchinger P., Thürlimann B. (1972) - *Schub und Torsion.Bulletin d'Information CEB*, No 86

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The Stiffness of Elements Stressed to Torsion and Combined Torsion with Bending Action

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Rezumat: Prezenţa solicitării compuse torsiune-încovoiere intr-un număr tot mai mare de structuri moderne de rezistenţă, determină tot mai mulţi cercetători să abordeze studiul interacţiunii celor două solicitări. Pe baza constatărilor teoretice şi experimentale s-a stabilit că la proiectarea structurilor din beton armat solicitate concomitent la acţiunea încovoierii şi răsucirii, neglijarea rigidităţii reduse la torsiune, duce la diferenţe între comportarea reală şi calculul acestora. Lucrarea de faţă face un studiu parametric al rigidităţii la torsiune, venind în întâmpinarea proiectanţilor cu un tabel conţinând raportul între rigiditatea unei secţiuni dreptunghiulare de beton armat supusă la torsiune, înainte de fisurare şi după fisurare.

Abstract: In our days, a lot of modern strength structures are stressed to the torsion-bending dual action, so many scientists were studding the interactions between these two. According to theory and experimental it has been ascertained that in designing of reinforced concrete structures stressed same time to bending and torsion, ignoring low stiffness to torsion gives us differences between real behavior and reckoned result. The main study is a parametric study of stiffness to torsion, and places a chart to structural engineers disposal which contents the ratio of stiffness of a rectangular reinforced concrete section stressed to torsion before and after cleaving.

Keywords: torsion-bending, reinforced concrete structures, stiffness.

1. The Stiffness of Elements Stressed to Torsion

In the case when the main unitary effort of stretching σ satisfied the relation: σ < R stiffness to torsion can be evaluate due to elasticity theory, using the transversal elasticity modulus: G = 0,4 E

CEB recommendations are to use a low value transversal modulus:

G = 0,7G ≈ 0,3 E

Relation of specific torsion is:

$$\frac{d\theta}{dx} = \frac{M_t}{GI_t} = \frac{M_t}{K_t^I} \qquad (1)$$

To evaluate the stiffness to torsion modulus K, it is reckoning the inertial moment I using the following relations:

- for close thin wall sections

$$I_t = \frac{4A_{bs}^2}{\sum \frac{h}{\delta}} = \frac{4A_{bs}^2\delta}{u_s} \qquad (2)$$

- for open thin wall sections

$$I_t = \frac{1}{3}\sum h\delta^3 \qquad (3)$$

The relations of CEB-FIP and DIN standards for K stiffness to torsion reckoning are:

K = G . I = α . E . I (4)

I = inertial moment to torsion
G = transversal elasticity modulus
E = elasticity modulus
α = subunitary coefficient experimentally determined using concrete quality, transversal section form and loading ratio

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The DIN 4224 reckoning formula for stiffness to torsion is:

$$\left(G \cdot I_t\right)_{II} = 0{,}15\left(1+\mu\right) \cdot \frac{1}{1+\dfrac{M_b}{M_{bu}}}\left(G \cdot I_t\right)_I \quad (5)$$

for $\mu \geq 0{,}5\%$, where
$(G\,I_t)_I, (G\,I_t)_{II}$ = stiffness to torsion I and II

μ = longitudinal reinforcement percent
Mb = bending moment corespondent to maximum torsion moment
Mbu = cleaving moment of the section in case of pure torsion

2. Stiffness after cleaving:

After cleaving stiffness modulus is defined as ratio between the torsion moment and specific turning at the beginning of reinforcement flowing. Comparing to stiffness to bending, the stiffness to torsion goes down when cleaving starts.

In determinate static structures the change of stiffness ratio generates an efforts redistribution. In CEB 1977 standards, the reinforcement influence can be consider by:

$$K_t^{II} = \frac{E_s \cdot A_{ef}^2}{\dfrac{u_{ef} \cdot s}{2A_s} + 1{,}5\dfrac{E_s}{E_e} \cdot \dfrac{u_{ef}}{h_{ef}}} \quad (6)$$

Kt^{II} stiffness to torsion II stadium
E_s steel elasticity modulus
E_e concrete elasticity modulus
A_s hoop transversal section
s distance between hoops

For the reckoning we consider a bin section equivalent to a thin wall section named effective section and which is defined by medium polygonal contour u_{ef} and wall thickness h_{ef}.

In 1973 Thurliman and Luchinger shows that determination of the torsion rigidity modulus in cleaving stadium makes using the 45° latticed beam model.

Experimental researches shows that the rigidity modulus is strongly determined by the total reinforcement percentage and not by the reinforcement perimeter distribution. Also it consider that the compressive diagonals are stiff.

The torsion rigidity modulus is obtained by equalizing the virtual mechanical work of the external and internal forces: Lint. + Lext. = 0

Considering:

$$L_{ext} = \bar{M}_t\, d\theta; \qquad L_{\text{int}} = -\int \bar{\sigma}\varepsilon dV$$

$$K_t^{II} = \frac{A_{bs}^2 A_s E_a}{u_s s_e}\left(1+\psi\right) \quad (7)$$

$$\psi = \frac{\sum A_{al} s_e}{A_s u_s} \quad (8)$$

We obtain the second stadium torsion rigidity modulus:

$$K_t^{II} = \frac{A_{bs}^2 A_s E_a}{u_s s_e}\left(1+\psi\right) \quad (9)$$

$$\psi = \frac{\sum A_{al} s_e}{A_s u_s} \quad (10)$$

If we have an irregular hoops distribution, relation (7) became:

$$K_t^{II} = \frac{4A_{bs}^3 E_a}{u_s^2} \frac{\mu_e \mu_{al}}{\mu_e + \mu_{al}} \quad (11)$$

$$\mu_e = \frac{\sum\left(A_s \dfrac{h_s}{a_e}\right)}{A_{bs}} \quad (12)$$

$$\mu_{al} = \frac{\sum A_{al}}{A_{bs}} \quad (13)$$

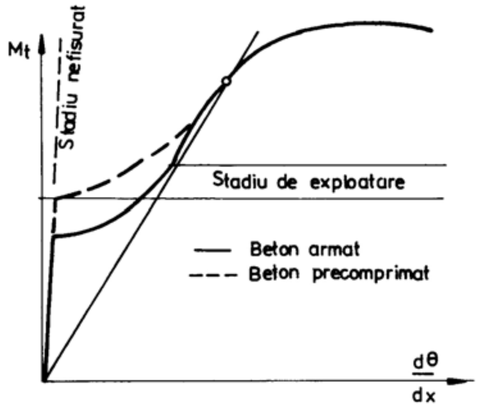

Fig. .1 Underlining exploitation stage undera torsion stresses

Kt^{II} , stiffness to torsion in II stadium

A_{bs} , area of the beam model

u, area contour

μ_e , hoops reinforcement percentage

a_e , distance between hoops

μ_{al} , longitudinal reinforcement percentage

L.Bob et al. has been ascertained that the best results between theory and experimental are obtained using the (4) formula. Figure 2 shows the experimental values of α coefficient in function of Mt/m ratio.

In conclusion, Bob et al. proposal for reckoning of reinforced concrete elements stressed to bending with torsion is:

- for I stadium:

$$K_t^{\ I} = 0{,}3 \cdot E_b \cdot I_t \qquad (14)$$

- for II stadium:

$$K_t^{\ II} = \alpha \cdot E_b \cdot I_t \qquad (15)$$

and the values for α are:

- for $\psi \leq 0{,}2$ $\alpha = 0{,}08$
- for $\psi \geq 0{,}8$ $\alpha = 0{,}05$

- for intermediate values an interpolation will be made

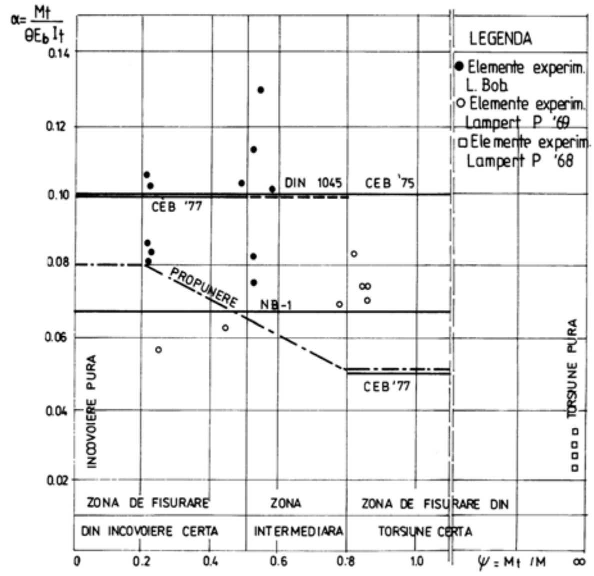

Fig. 2 Experimental values of α coefficient

3. Parametrical studies of stiffness to torsion:

For the cleaved and uncleaved states it is using the ratio

$$\frac{K^{II}}{K^{I}} = k \cdot \frac{Ae \cdot (1+\psi)}{u_s \cdot s_e} \cdot \frac{E_a}{E_b} \qquad (16)$$

where K is a coefficient

$$k = 2{,}313 \cdot \frac{1}{\alpha} \cdot \frac{h}{b}\left(1 - \frac{1}{6} \cdot \frac{b}{h}\right)^2 \qquad (17)$$

So, stiffness to torsion depends to awesome parameters which are:

- section shape

$$\frac{K^{II}}{K^{I}} = f\left(\frac{h}{b}\right)$$

As = 0,79 cmp
S = 15 cm
Ae = 36 cmp
Eb = 210.000 daN/cmp
u = 180 cm

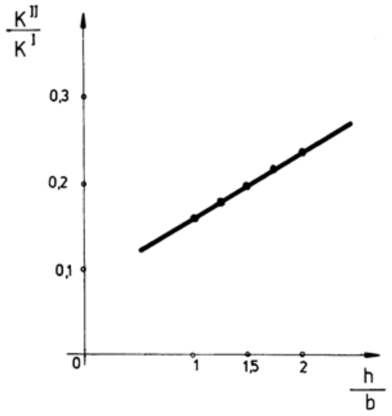

Fig.3 Section shape influence on $\frac{K^{II}}{K^{I}}$ ration

The maximum reduction is obtained for square section.

- concrete quality, defined through the concrete elasticity modulus

$$\frac{K^{II}}{K^{I}} = f(E_b)$$

As = 0,79 cmp
S = 15 cm
Ae = 36 cmp
Eb = 210.000 daN/cmp
u = 180 cm
h/b = 1,5 Ψ k = 13,99

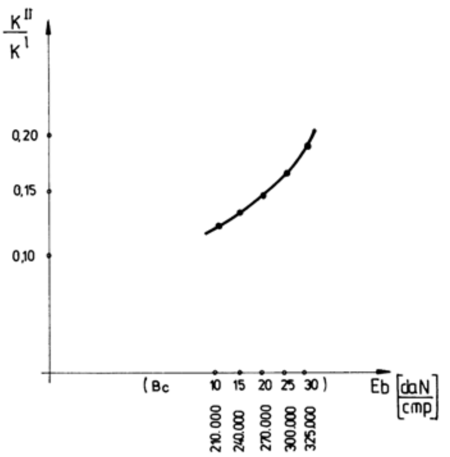

Fig. 4. Concrete quality influence on $\frac{K^{II}}{K^{I}}$ ratio

So, the concrete elasticity modulus is considerable influenced by the stiffness to torsion in second stadium.

- transverse reinforcement-hoops space

$$\frac{K^{II}}{K^{I}} = f(s), \text{(fig.5) and } \frac{K^{II}}{K^{I}} = f(A_s) \text{ (fig. 6)}$$

s = hoops space
As = hoops area

The hoops space considerable influenced the stiffness to torsion in second stadium.

There is a linear increase between KII/KI ratio and hoops area.

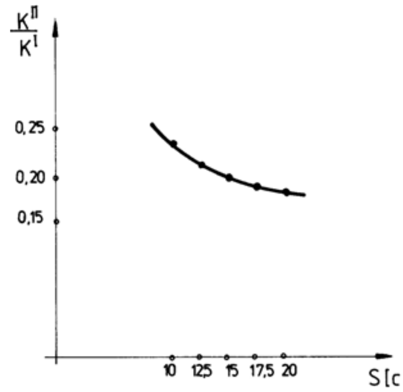

Fig.5 Transverse reinforcement influence – hoops space on $\frac{K^{II}}{K^{I}}$ ratio

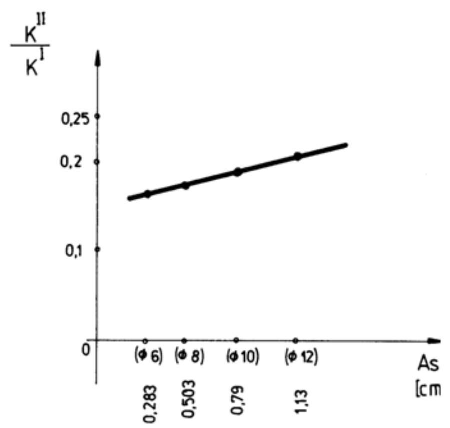

Fig. 6 Transverse reinforcement influence – hoops section on $\frac{K^{II}}{K^{I}}$ ratio

- longitudinal reinforcement (percentage p%)

$$\frac{K^{II}}{K^{I}} = f\left(p\%\right)$$

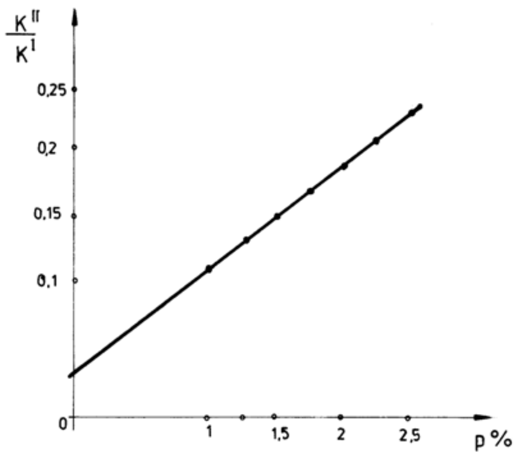

Fig.7 Transverse reinforcement influence – longitudinal reinforcement ratio on $\frac{K^{II}}{K^{I}}$

According to these, we can say that the reinforcement type of the element by the hoops space, hoops section, and longitudinal reinforcement, is very significant in determination to decreasing of stiffness to torsion. The influence of longitudinal reinforcement disposal is more important than hoops disposal.

Using formula 14 and modifying parameters it was obtained the tabel which contains KII / KI ratio for a concrete reinforced section in function of section sides ratio, diameter and distance of transverse reinforcement, longitudinal reinforcement percentage, and concrete quality.

4. References:

[1] Li, Dănetiu Gh . (1979) - *"Rigiditatea la torsiune a elementelor din beton armat supuse acțiunii combinate a încovoierii şi torsiunii"* – referat ştiințific – ICCPDC Timişoara

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[5] ***- "*Effort Tranchant-Torsion"* Final Draff. Bulletin d'Information CEB Nr.92, Juin 1973

Tab.1.KII / KI for concrete quality: C 12/15 (Bc 15), (B 200)

E_b $\left[\frac{daN}{cmp}\right]$	$\frac{h}{b}$	Q [mm] etrieri	S [cm]	p_0						
				1,0	1,25	1,50	1,75	2,0	2,25	2,5
240.000	1,00	6	10	0,1	0,122	0,143	0,164	0,185	0,206	0,227
			15	0,095	0,116	0,137	0,158	0,179	0,200	0,222
			20	0,092	0,113	0,134	0,155	0,176	0,197	0,218
		8	10	0,113	0,134	0,155	0,176	0,197	0,218	0,239
			15	0,103	0,125	0,146	0,167	0,188	0,209	0,230
			20	0,0988	0,119	0,141	0,162	0,183	0,204	0,225
		10	10	0,129	0,151	0,172	0,193	0,214	0,235	0,256
			15	0,114	0,135	0,156	0,177	0,198	0,219	0,241
			20	0,107	0,128	0,149	0,170	0,191	0,212	0,233
		12	10	0,149	0,170	0,191	0,213	0,234	0,255	0,276
			15	0,128	0,149	0,170	0,191	0,212	0,233	0,254
			20	0,117	0,138	0,159	0,180	0,201	0,222	0,243

E_b $\left[\frac{daN}{cmp}\right]$	$\frac{h}{b}$	Q [mm] etrieri	S [cm]	p_0						
				1,0	1,25	1,50	1,75	2,0	2,25	2,5
240.000	1,50	6	10	0,124	0,149	0,175	0,201	0,227	0,253	0,279
			15	0,117	0,143	0,169	0,194	0,220	0,246	0,272
			20	0,114	0,139	0,165	0,191	0,217	0,243	0,269
		8	10	0,139	0,165	0,191	0,217	0,242	0,269	0,294
			15	0,127	0,153	0,179	0,205	0,231	0,257	0,282
			20	0,121	0,147	0,173	0,199	0,225	0,251	0,276
		10	10	0,159	0,185	0,211	0,237	0,263	0,288	0,314
			15	0,140	0,167	0,192	0,218	0,244	0,269	0,296
			20	0,131	0,157	0,183	0,209	0,235	0,261	0,286
		12	10	0,184	0,210	0,236	0,261	0,287	0,313	0,339
			15	0,157	0,183	0,209	0,235	0,260	0,286	0,312
			20	0,144	0,169	0,195	0,221	0,247	0,273	0,299
	2,00	6	10	0,102	0,123	0,144	0,164	0,185	0,206	0,226
			15	0,096	0,117	0,137	0,158	0,178	0,199	0,220
			20	0,093	0,113	0,134	0,155	0,175	0,196	0,217
		8	10	0,118	0,139	0,160	0,180	0,201	0,221	0,242
			15	0,106	0,127	0,147	0,168	0,189	0,209	0,230
			20	0,100	0,121	0,142	0,162	0,183	0,204	0,224
		10	10	0,137	0,158	0,180	0,200	0,220	0,241	0,262
			15	0,119	0,140	0,161	0,181	0,202	0,223	0,243
			20	0,110	0,131	0,151	0,172	0,193	0,213	0,234
		12	10	0,162	0,182	0,203	0,224	2,244	0,265	0,286
			15	0,135	0,156	0,177	0,197	0,218	0,239	0,260
			20	0,122	0,143	0,163	0,184	0,205	0,226	0,246

Energy Concentrator for Ducted Wind Turbines Assisted by the Coanda's Effect

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Rezumat: În lucrare este prezentată soluția constructivă a unui concentrator de energie cu aripă inelară, asistat de efectul Coandă, pentru turbinele de vânt axiale, întubate, de mici dimensiuni. Pornind de la studii originale referitoare la ejectoarele Coandă interioare, se demonstrează prin calcule că ejecția Coandă asociată eolienelor întubate conduce la o creştere importantă a debitului de aer prin turbine, respectiv la creşterea performanțelor funcționale ale acestora: puterea fluidică incidentă, puterea mecanică, coeficientul de putere, randamentul. Pentru realizarea inducției fluidice se valorifică energii disponibile, reziduale, secundare.

Abstract: In this paper is presented the technical solution of an energy concentrator for small wind turbines in annular wing, assisted by the Coanda's effect. Starting from some original studies regarding the Coanda's inner vent-ejectors, it is demonstrated that the Coanda's ejection associated at ducted propeller has as result a significant increasing of flow rate through turbines, respectively, the increasing of main functional characteristics of these: incident fluidic power, mechanical power, factor of power, efficiency. There are used secondary, recoverable sources of fluidic energy.

Keywords: *ducted wind turbine, Coanda' effect, functional characteristics, recoverable sources.*

1. Introduction

One of the main factors which are characterised the functioning of an wind turbines is the coefficient of power C_P, defined by the Equation:

$$C_P = \frac{M \cdot \Omega}{\frac{1}{2}\rho_{air} v_{air}^{\ 3} A_\pi} \tag{1}$$

where:

- M is the torque at the shaft of turbine;
- Ω is the speed of the shaft of turbine;
- ρ_{air} is the density of the air (wind);
- v_{air} is the velocity of the air (wind);
- A_π is aria of the disk of propeller.

Defined with the Eq. (1), C_P represent the degree of conversion of the incident fluidic power into mechanical power. The Sandia Laboratories use a formula with the velocity at the end of the blade $\Omega \cdot r_B$:

$$K_P = \frac{M \cdot \Omega}{\frac{1}{2}\rho_{air}\left(\Omega \cdot r_B\right)^3 A_\pi} \tag{2}$$

where:

r_B is the radius of blade;

According with Betz's theory, the maximum (ideal) value of C_P (or K_P) is:

$$C_{P\,max} = \frac{16}{27} \quad \text{for} \quad \frac{v_R}{v_w} = \frac{2}{3} \quad \text{and} \quad \frac{v_{ds}}{v_w} = \frac{1}{3} \tag{3}$$

The main influence on reducing of nominal power is represented by the decreasing of flow rate through wind turbine (by a factor $(2/3)$, while the influence of an incomplete absorption is smaller (by a factor $\left[1-(1/3)^2\right] = 8/9$).

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2. Main section

The conception of a ducted wind turbine, into an annular wing, has as essential advantage a significant increasing of the flow rate through turbine. A supplementary advantage is represented by the improving of the efficiency due to the elimination of the loss at the end of the blades, when the gap between propeller and annular wing is small enough.

Figure 1 depicts the axial flow through such of ducted wind turbine in an annular wing having a profiled section, as an airfoil with the lower side to exterior. The turbine is placed in the minimal section of the wing.

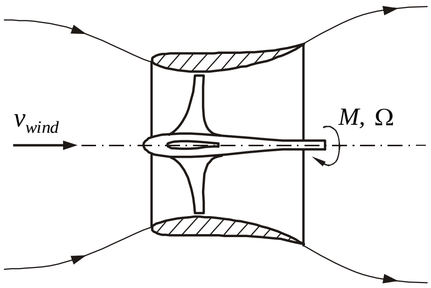

Fig. 1. Ducted wind turbine

The active factor is velocity of wind, which on the upper side of the airfoil, in the interior of the wing, is accelerated and the resulted depression provoke a fluidic induction having as results some limitations of the performances of wind turbine.

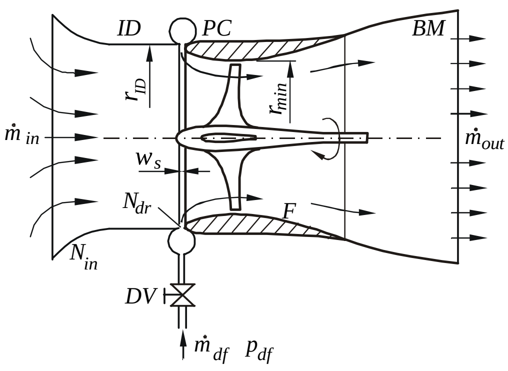

Fig. 2. Ducted wind turbine with Coanda's interior vent-ejector

The performances (incident fluidic power, mechanical power, factor of power, efficiency, carrying factor u) can be increased if at the flow through the annular profiled wing is associated the effect generated with the aid of a Coanda's interior vent-ejector, as in Fig. 2. The following notation were made:

BM is the exhaust bell-mouth;
DV is the drosel valve of delivery duct;
F is the flap (annular wing) by min. radius r_{min};
ID is the inlet duct by radius r_{ID};
N_{dr} is the driving nozzle by width of slot w_s;
N_{in} is the inlet nozzle;
PC is the annular pressure chamber;
$\dot{m}_{df}$ is the mass flow rate through driving nozzle;
$\dot{m}_{in}$ is the mass flow rate through inlet nozzle;
$\dot{m}_{out}$ is the mass flow rate through exhaust bell-moth.

It is appreciating that the performances of the ducted wind turbine is significant increased, also in the case of using a multi-stage driving nozzles witch are launching overlapped thin fluid jets, see Fig. 3. Practically, in this way it is realised an active control of the flow through annular wing (having a multi-element airfoil in his longitudinal section) at the level of boundary layer.

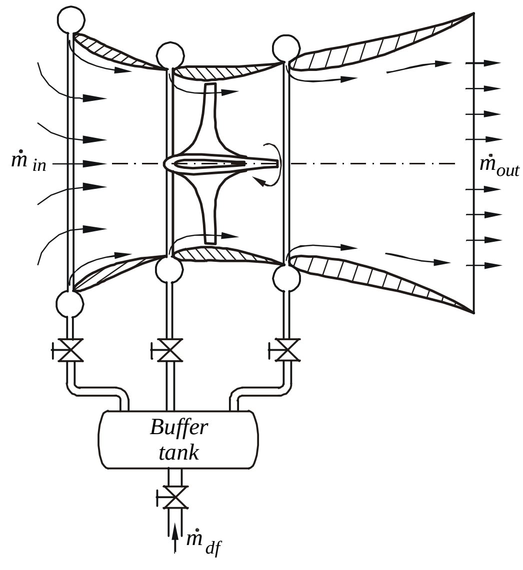

Fig. 3. Ducted wind turbine with Multi-stage Coanda's interior vent-ejectors

Regarding the Coanda's effect, the authors have imagined, designed, realized, experimented and published numerous achievements of ejectors (vent-ejectors, exhausters) assisted by Coandă's effect, this being generated on curve depressive surfaces (flaps), the driving fluid being launched through thin slots.

The applications are mostly industrials, as ventilation and air conditioning, pneumatic transport a.o., taking advantage of the available secondary fluidic energies, recoverable: compressed air, burned gases, vapors etc [1, 2, 3].

Regarding Fig. 2 and Fig. 3, the supply with driving fluid, having the mass flow rate $\dot{m}_{df}$ and gauge pressure p_{df}, is done from an external source (buffer tank) through a delivery duct provided with a drosel valve DV, an annular pressure chamber PC and a radial drain by width w_s. The driving fluid is launched radial with velocity v_0 through driving nozzle, tangential to the annular flap, at the leading edge of this.

The radial driving fluid sheet, thin, with high kinetic energy, launched tangential to the circular flap, deviates and attaches itself to the surface of the flap, in the Coanda's way, following the downstream interior of the ejector body to the exhaust duct. In this way, in the interior of inlet duct is created a depression and a supplementary flow rate is absorbed from the exterior ($\dot{m}_{in}$ is increased), having finally a mixture of fluids, with mass flow rate $\dot{m}_{out}$.

3. Theoretical approaching

The mass flow rate of driving fluid, what is launched through the slot of annular driving nozzle, for adiabatic expansible fluids with exponent k (adiabatic constant), having as initial conditions p_{0df} (absolute pressure) and density ρ_{0df} can be computed with the Eq. (4).

The value of the pressure coefficient on the surface of the annular flap can be evaluated with the Eq (6).

For the computation of vacuum pressure in the min. section of the ejector is used the Eq (7):

$$\dot{m}_{df} = \mu_{df} A_{Ndr} \rho_{df} \sqrt{2\frac{k}{k-1}\frac{p_{0df}}{\rho_{0df}}} \cdot \sqrt{\varepsilon^{\frac{k-1}{1}}} \quad [kg/s] \tag{4}$$

where:

μ_{df} is the coefficient of flow rate; can be computed with the Eq. (5);

A_{Ndr} is the active area of the driving nozzle:

$$A_{Ndrv} = 2\pi r_{ID} w_s \quad [m^2]\ ;$$

ε is the expansion factor.

$$\mu = \frac{1}{\sqrt{\alpha + \sum\zeta}} \quad [-] \tag{5}$$

where:

α is the Coriolis' coefficient;

$\sum\zeta$ represent the sum of local coefficients of head loss on delivery duct.

$$c_P = \frac{p_v}{p^* - p_0} \quad [-] \tag{6}$$

where:

p_v is the static, gauge pressure on flap;

p^* is total pressure;

p_0 is the atmospheric pressure (references pressure);

$$p_{v\,min} = \frac{\rho_{air} v_{te}^2}{2} \quad [N/m^2] \tag{7}$$

where:

v_{te} is the velocity of the air at trailing edge of the flap;

For the computation of the average vacuum pressure $p_{v\,ID}$, what determined the suction of carrying fluid in inlet duct, the following original Equation was obtained:

$$p_{v\,ID} = \frac{\left(\frac{r_{min}}{r_{ID}}\right)^2}{1 + \varphi_{ID}^2} p_{v\,min} \quad [N/m^2] \tag{8}$$

where:

φ_{ID} is the factor of velocity of the inlet duct.

The volumetric rate of flow of the fluid in inlet duct is computed with the Eq. (9), the mass flow rate being $\dot{m}_{in} = \rho_{air} Q_{in}$ [kg/s] .

$$Q_{in} = \mu_{ID} A_{ID} \sqrt{\frac{2}{\rho_{air}} p_{v\,ID}} \quad [m^3/s] \tag{9}$$

where:

μ_{ID} is the factor of the flow rate of the inlet duct;

The carrying (induction, mixing) factor u is:

$$u = \frac{\dot{m}_{in}}{\dot{m}_{df}} = \frac{\dot{m}_{out}}{\dot{m}_{df}} - 1 \quad [-] \tag{10}$$

where:

$$\dot{m}_{out} = \dot{m}_{in} + \dot{m}_{df} \quad [-] \tag{11}$$

4. Numerical example concerning presented prototype

It is considered an axial wind turbine having the exterior diameter $2r_{wt} = 1\,m$ ($A_{wt} = 0.785$ m^2). For a wind velocity of $v_{air} = 10\,m/s$, and $\rho_{air} = 1.2\,kg/m^3$, the incident fluidic power is:

$$N_i = (\rho_{air} v_{air}^2 / 2) A_{wt} v_{air} = 0.471\,[kW]\,.$$

Also, is considered an annular wing having the radius $r_{aw} = 0.52\,m$ ($A_{aw} = 0.85\ m^2$), characterised by the following coefficients: $\mu_{in} = = \varphi_{in} = 0.97$. For the driving nozzle, the width of slot was $w_s = 0.3\,mm$ ($A_{Ndr} = 0.98 \cdot 10^3\ m^2$) and $\mu_{Ndr} = 0.63$.

As driving fluid was used air, at different pressures (controllable), in a range of $p_{df} = (150 \div 1000)\,N/m^2$. The leading edge of the curved flap is, in section, a quarter of circle with the radius by $20\,mm$, used by authors in the numerous studies regarding the Coanda's effect.

The results of the calculus are presented graphically in Fig. 4. The obtained mass flow rate of the driving fluid is $\dot{m}_{df} = (0.7 \div 1.8)\,kg/s$. The flow rate of the air suctioned from atmosphere was obtained in range of $Q_{in} = (20 \div 67)\,m^3/min$, what is represented an average velocity of suction in range of $v_{in} = (0.4 \div 1.34)\,m/s$ and an increasing of the velocity of the air through wind turbine with $\Delta v = (4 \div 13.4)\%$ (considering $v_{air} = 10\,m/s$). Results an increasing (theoretical) of the torque at the shaft of turbine $\Delta M = (8 \div 29)\%$ and of the incident fluidic power $\Delta N_i = (12 \div 46)\%$. Practically, there are need some corrections due to the real increasing of equivalent hydraulic resistance through turbine.

We are recommending the using of such devices in the case of the small wind turbines, whit a moderate consume of driving fluid, placed in industrial areas, where is installations which are working under pressure.

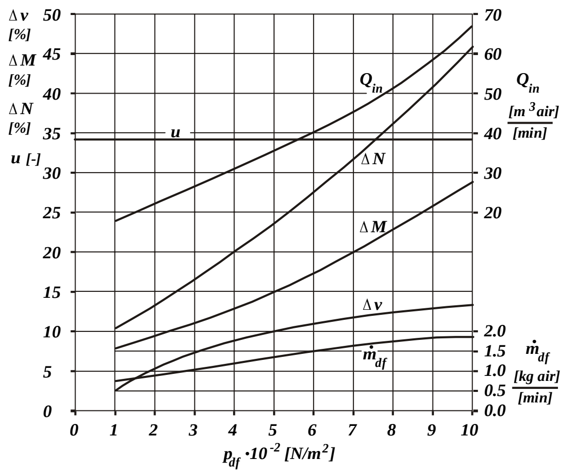

Fig. 4. The variations of the studied indicators of efficiency for wind turbines

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Air-Gas Dynamic Exhauster

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Rezumat: În lucrare sunt prezentate rezultatele obținute în urma unor teste efectuate în scopul determinării caracteristicilor funcționale ale unui exhaustor aero-gazo dinamic, conceput special pentru evacuarea fumului şi a gazelor de ardere produse de sistemele casnice de încălzire cu flacără deschisă. Funcționează pe principiul turbinei de vânt, putând fi folosit de asemenea şi la ventilarea spațiilor închise, camere, hambare, chiar hale industriale fără mediu toxic. Este simplu de construit şi funcționează utilizând energia eoliană, regenerabilă.

Abstract: In this paper are presented the results obtained in some tests effectuated in order to obtain the functional characteristics of an air-gas dynamic exhauster designed especially for the exhausting of the smoke and the burnt gasses produced by the household heating systems with open flame. It functioning as a wind turbine and can be used also for ventilation of the closed chamber, barns, and even halls without toxic environment. It is simple to build and is put in action by the energy of wind, a renewable one.

Keywords: *air-gas dynamic exhauster, functional characteristics, renewable energy.*

1. Introduction

In this paper are presented the results obtained in some tests effectuated in order to obtain the functional characteristics of the air-gas dynamic exhauster, Smoky [2].

Fig. 1

According to specifications of the producer, this fan, see Fig. 1, is designed especially for the exhausting of the smoke and the burnt gasses produced by the household heating systems with open flame. Also, it can be use for ventilation of the closed chamber, barns, and even halls without toxic environment. It is simple to build and is put in action by the energy of wind, a renewable and green one.

It can be made in more shapes and geometrical variants, the tested one having a spherical shape of active part and the following main characteristics:

- *diameter of active part:* $D_{WT} = 265\ mm$;
- *high of the active part:* $H_{WT} = 200\ mm$;
- *inner diameter of the : suction duct.* $d_{SD} = 170\ mm$;

The previous mentioned functional characteristic were concerned the following:

- *establishing of the dependence between the velocity of the air* v_{air} *and the flow rate of the carrying fluid through the suction duct* Q_{cf} ;
- *computation of the values of the average pressure at the level of the inlet section of the fan;*
- *establishing of the minimal velocity of the air for what the fan is becoming functional.*

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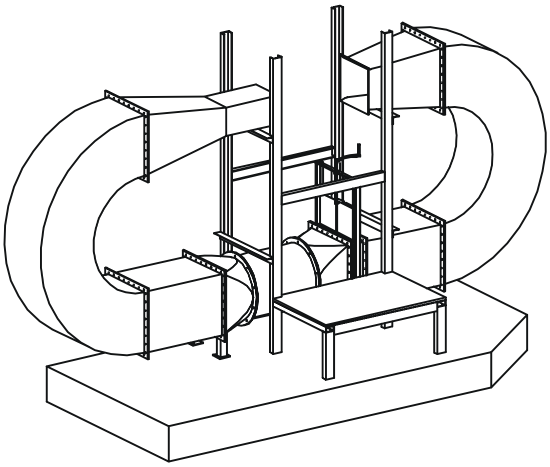

Fig. 2. Axonometric view of the used wind tunnelwith a detail of the test chamber with wind turbine

2. Experimental installation and measurement instruments

For measurements it was used one of the low speed wind tunnels of the Thermo & Fluid Mechanics department from Transilvania University of Brasov, see Fig. 2. It is a closed circuit wind tunnel with open test section and has the following characteristics:

- dimensions of test chamber: $b_{TC} \times h_{TC} \times l_{TC} = (0.32 \times 0.23 \times 0.85)\, m^3$
- range of the velocities in the test chamber: $v_{air} = (1.5 \div 45.0)\, m/s$

The wind turbine was placed in the middle of the test chamber and fixed on the frame structure of the wind tunnel. The suction duct has the length $l_{SD} = 1\, m$. For measurement of the velocity of carrying fluid on suction duct was used a thermal velocity probe TESTO 0635.1549 with the following technical data:

- *range of the measured velocity:* $v_{TVP} = (0.00 \div 10.00)\, m/s$;
- *range of the measured temperatures:* $t_{TVP} = (-20.0 \div 70.0)\, °C$;
- *exterior diameter:* $d_{TVP} = 4\, mm$;

For measuring of air velocity in the test chamber was used a vane probe TESTO 0635.9540. The measuring principal of this is based on the conversion of rotary motion of the windmill into electrical signal, digitally displayed on a TESTO 491 apparatus (see Fig. 3).

The vane probe measures simultaneously both velocity ant temperature and has the followings main characteristics:

- *range of measured velocities:* $v_{VP} = (0.4 \div 60)\, m/s$;
- *range of measured temperatures:* $t_{VP} = (-30 \div 140)\, °C$.
- *exterior diameter:* $d_{VP} = 16\, mm$;

3. Experimental procedure

For some air flow states in the test chamber, which are obtained with the aid of the control sluice of the wind tunnel, the following parameters were measured:

- *velocity of the air in the test chamber:* v_{air} ;
- *temperature of the air in the test chamber:* t_{TC} *(for each flow state separately);*
- *local velocities of the air through the suction duct:* v_i *(according with the procedure below presented).*

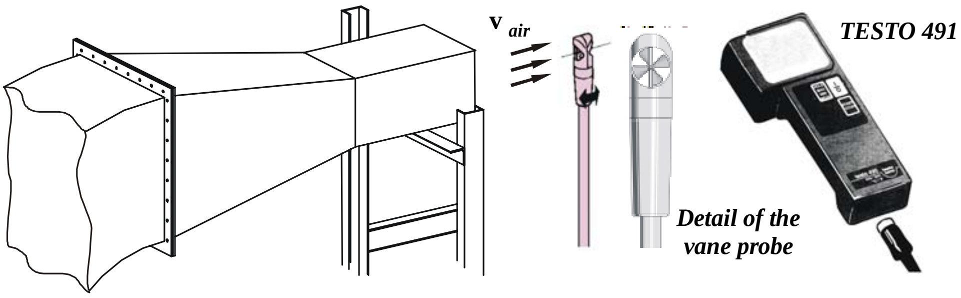

Fig. 3. Measurement instruments

Also, there were measured the atmospheric parameters t_{air} (T_{air}) and p_{air} at the time of performed tests: $p_{air} = 708\ mmHg$ **;** $T_{air} = 22\ K$.

With the aim to determine the average velocity of the carrying fluid v_{cf} , respectively the flow rate Q_{cf} , the cross section S of the suction duct was divided in n annular equal areas S_i . In our case was considered $n = 12$. The magnitude of the average velocity can be determined as arithmetical average of the local velocities v_i , which are measuring in the centres of annular areas, with to the Eq. (1):

$$v_{cf} = \frac{1}{S}\int_S v \cdot dS = \frac{1}{nS_i}\sum_{i=1}^{n} v_i S_i = \frac{1}{n}\sum_{i=1}^{n} v_i \qquad (1)$$

With this value of average velocity and knowing the interior diameter of the duct, the volumetric flow rate is:

$$Q_{cf} = \frac{\pi \cdot d_{SD}^2}{4} \cdot v_{cf} \qquad (2)$$

The result obtained after three open-closing cycles of the control sluice of the wind tunnel are presented in the tables and as graphical form, for a range of velocity $v_{air} = (1.8 \div 30)\ m/s$.

4. Results and Conclusions

The pressure of carrying fluid at the level of the inlet section of the fan was computed using the specifications of [1]. In this order the following Eq. (3) was used:

$$p_{cf} = -\frac{\rho_{air} v_{cf}^2}{2}\left(1 + f_{SD}\frac{l_{SD}}{d_{SD}} + \zeta_{i\,SD}\right) \qquad (3)$$

where:
ρ_{air} is the density of the carrying fluid, Eq. (4);
f_{SD} is the pipe friction factor, Eq. (5);
$\zeta_{i\,SD} = 1$ is the local factor of head loss at the inlet.

$$\rho_{air} = \rho_{0\,air}\frac{p_{air}}{p_{0\,air}}\frac{T_{0\,air}}{T_{air}} \qquad (4)$$

where $p_{0\,air}$, $T_{0\,air}$ and $\rho_{0\,air}$ $\rho_{0\,air}$ are the parameters of the atmosphere in the physical state.

$$f_{SD} \cong 0.11 \cdot \left(\overline{\Delta}_{SD} + \frac{68}{Re}\right)^{0.25} \qquad (5)$$

where:
$\overline{\Delta}_{SD}$ is the relative roughness ($\Delta = 0.085\ mm$);
Re is the Reynolds number;

Figure 4 depicts the variation of the volumetric flow rate of the carrying fluid Q_{cf} versus the velocity of the air in the test chamber of the wind tunnel.

This is representing only the flow rate due to the action of the wind turbine. In the real cases of the heating systems at this flow rate is added the one resulted through natural draught.

Preliminary tests made on a real exhausting system of the burnt gases show an increasing of the flow rate with 8% in case of using an exhauster as presented one, at a velocity of the wind by $2\ m/s$.

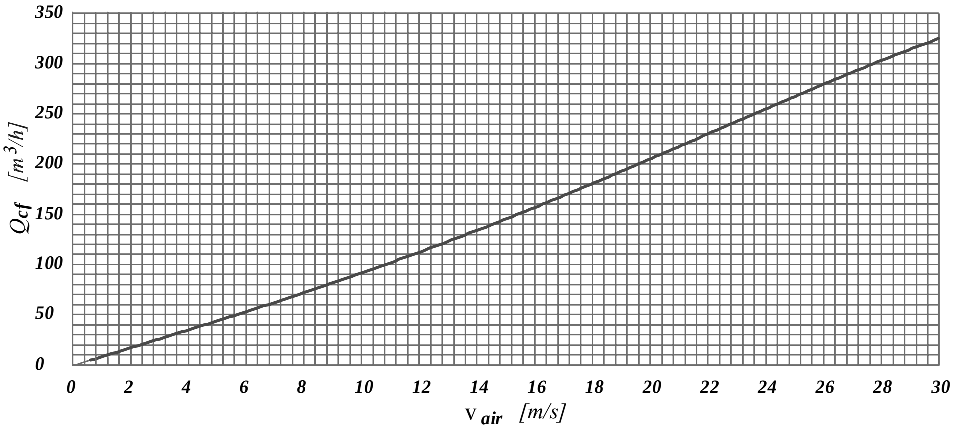

Fig. 4. The variation of Q_{cf} versus v_{air}

The minimal value of the wind velocity for what the fan is become functional was $1.8\, m/s$ (during the tests effectuated in laboratory with wind tunnel).

The movement of the exhauster provoke a depression (see Table 1) at the level of the inlet section and in this way are avoid potential critical situations of the reversed draught of heating system (see Fig. 5.a).

Table 1

No		1	2	3	4	5	6	7	8	9	10
v_{aer}	*[m/s]*	1.9	4.0	5.95	8.05	10.15	12.05	16.0	20.2	24.0	29.5
p_{cf}	*[mm H_2O]*	-0.004	-0.024	-0.052	-0.093	-0.177	-0.240	-0.425	-0.830	-1.183	-1.879

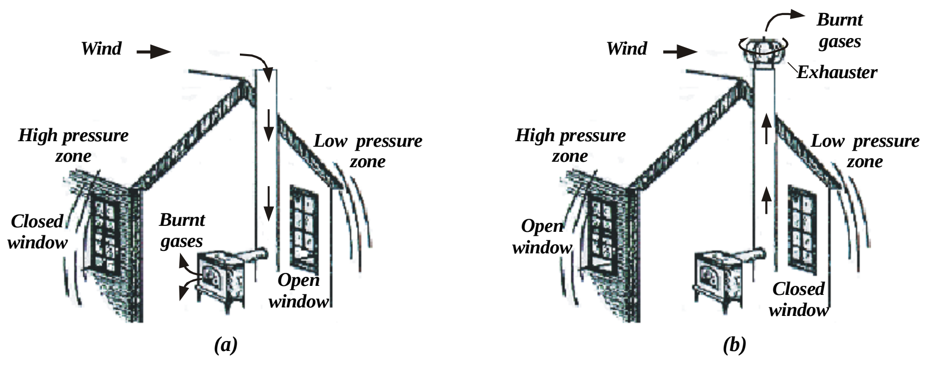

Fig. 5. Reversed draught situation avoid with exhauster

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Roughness - An Important Factor for Hydraulic Calculus on Under Pressure Pipes and on Free Surface Canals

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Rezumat: Calculul hidraulic al conductelor sub presiune şi al canalelor cu suprafaţă liberă necesită cunoaşterea rugozităţii suprafeţei rigide aflate în contact cu lichidul în mişcare. Rugozitatea poate fi exprimată fie prin rugozitatea absolută echivalentă, fie prin coeficientul de rugozitate de tip Manning. In literatura de specialitate nu există relaţii de echivalenţă între cele două moduri de exprimare a rugozităţii. Lucrarea de faţă prezintă o metodă teoretică pentru determinarea echivalenţei între cele două tipuri de rugozităţi.

Abstract: Hydraulic calculus on under pressure pipes and on free surface canals requires the value for the contact surface roughness. Roughness can be expressed by the equivalent absolute roughness or by Manning roughness coefficient. The specialty literature does not contain equivalence relationships between the two ways for expressing roughness.

Keywords: Absolute roughness, Manning roughness coefficient, velocity distribution.

1. Introduction

Usually, the hydraulics based on the fluids mechanics theory solves problems related to pressures and flowrates calculations in different sections of a hydraulic system.

Hydraulic calculations for hydraulic galleries, hydroelectrical power plants pipes, pipes in industrial and domestic water supply systems requires the roughness value for the interior surface of the involved pipes or canals.

The roughness is an intrinsic characteristic for the rigid surfaces. It can be expressed by the height of the surface asperities (absolute roughness), by the absolute equivalent roughness k for which the head loss has the same value like the absolute roughness or by using the roughness coefficient n of Manning type.

In the current practice there are used the absolute equivalent roughness and the Manning type roughness. The computing formulas are differentiated in function of the two ways of expressing roughness.

From a hydraulic point of view, the roughness is important only in the case of the turbulent flow on a hydraulic rough surface.

In the specialty literature there are no equivalence relationships between the two ways of expressing roughness.

Based on the results presented in [1], in this paper is analysed the influence of the shape and the size of the cross section area of the canal or of the pipe.

2. Velocity distribution in a turbulent flow

2.1. Turbulent flow with a free surface

The turbulent flow in the vicinity of the bed can be completely determined if the following items are known: the geometry of the bed roughness which is given by absolute size (height) k of the elements forming the bed roughness, the physical properties ρ and μ of the fluid, the mechanical state of the fluid motion which is given by the shear velocity V_*.

Hence any mechanical quantity related to the flow in the vicinity of the bed must be a certain function of the following four independent quantities: ρ, μ, V_*, k.

The ratio of the size k of the roughness to the thickness δ_0 of the viscous sublayer can be regarded

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as a quantitative property of the flow in the vicinity of the bed and therefore it must be given by:

$$\frac{k}{\delta_0} = \varphi(\rho, \mu, V_*, k) \qquad (1)$$

Using the dimensional analysis, the Eq.(1) reduces to:

$$\frac{k}{\delta_0} = \varphi_\delta\left(\frac{V_* k}{\nu}\right) = \text{const}\frac{V_* k}{\nu} \qquad (2)$$

The proportionalities :

$$\frac{k}{\delta_0} \approx \frac{V_* k}{\nu} \qquad (3)$$

or:

$$\delta_0 \approx \frac{\nu}{V_*} \qquad (4)$$

are valid if the influence of k can be neglected with regard to the formation of the thickness δ_0 of the viscous sublayer (when $k < \delta_0$). If $k \approx \delta_0$, the proportionality given by Eq.(3) or Eq. (4) might or might not be valid and it is not possible to make an assessment on this situation theoretically.

The velocity distribution in a turbulent flow with free surface is given by:

$$\frac{v}{V_*} = \frac{1}{\chi}\ln\frac{y}{k} + B \qquad (5)$$

where:

$$B = \frac{1}{\chi}\ln\frac{V_* k}{\nu} + 5.5 \text{, if } y > \delta_0 > k$$

$$B = 8.5 \text{ if } k < y < \delta_0$$

The experimental relation between B and $\log\left(\frac{V_* k}{\nu}\right)$ was obtained by Nikuradse [2].

In the zone where $\delta_0 > k$ the turbulent flow is called *turbulent flow over a hydraulically smooth boundary* and where $k > \delta_0$ the flow is called *turbulent flow over a hydraulically rough boundary*.

2.2. Velocity distribution in turbulent flow over a hydraulically rough boundary

In this case the quantity B is constant, $B = 8.5$ and velocity distribution is given by the following expression:

$$\frac{v}{V_*} = \frac{1}{\chi}\ln\left(30\frac{y}{k}\right) \qquad (6)$$

in which $\chi = 0.4$ is the von Karman universal constant.

The maximum velocity is obtained for $y = h$:

$$\frac{v_{max}}{V_*} = 2.5\ln\left(30\frac{h}{k}\right) \qquad (7)$$

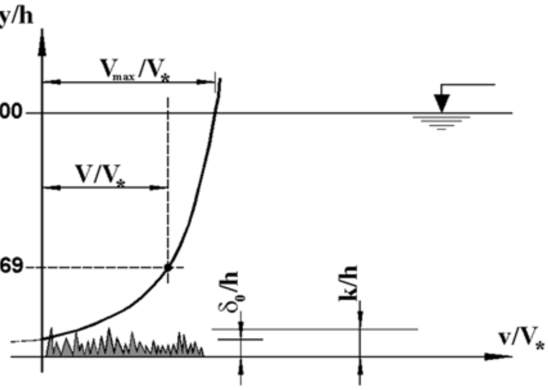

Fig.1. Velocity distribution in a turbulent flow over a hydraulically rough boundary.

Using Eq.(6) in the expression of the average velocity:

$$V = \frac{1}{h-k}\int_k^h v\,dy$$

and treating k/h as an infinitesimal of the first order, the following relation is obtained:

$$V = V_*\left(\frac{v_{max}}{V_*} - 2.5\right). \qquad (8)$$

Equations (6) and (8) are valid for:

$$\delta_0 \leq 2.327k \quad (9)$$

The dimensionless velocity diagram of this case is shown in Fig.1.

2.3. The velocity distribution in under pressure pipes

In circular under pressure pipes the movement is axially symmetrical and the pipe axis can be assumed as a line contained in a free surface. In this case, the velocity distribution is given by Eq. (6) where $y = r$, that means:

$$\frac{v}{V_*} = 2.5\ln\left(30\frac{r}{k}\right) \quad (10)$$

Maximum velocity is obtained for $r = r_0$:

$$\frac{v_{max}}{V_*} = 2.5\ln\left(30\frac{r_0}{k}\right). \quad (11)$$

Average velocity results by integrating:

$$V = \frac{1}{r_0 - k}\int_k^{r_0} v dr .$$

The resulted expression is:

$$V = V_*\left(\frac{v_{max}}{V_*} - 2.5\right) \quad (12)$$

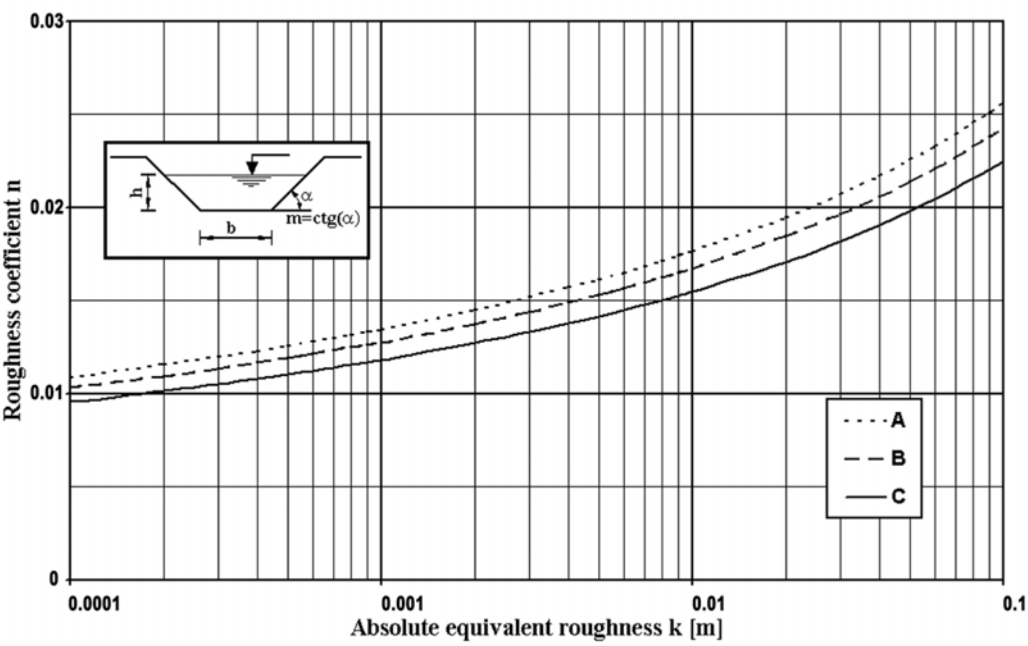

Fig. 2. The equivalence between absolute equivalent roughness k and Manning roughness:
A: rectangular section shape; b = 10.00 m; h0 = 1.50 m; m = 0.00 m
B: trapezoidal section shape; b = 2.00 m; h0 = 1.50 m; m = 1.00 m
C: triangular section shape; b = 0.00 m; h0 = 1.50 m; m = 1.00

3. The equivalence between absolute roughness and Manning roughness

For a uniform flow the flowrate is expressed by the continuity equation:

$$Q = VA \quad (13)$$

where V is the average velocity
A is the cross section area.

The average velocity can be computed either by the relations (8) or (12) or by Chézy expression:

$$V = C\sqrt{RI} \tag{14}$$

where C is the Chézy coefficient,
R is the hydraulic radius
I is the hydraulic slope.

The Chézy coefficient can be computed with the formula proposed by Manning:

$$C = \frac{1}{n} R^{1/6} \tag{15}$$

where n is a coefficient that expresses globally the roughness. It is known as Manning resistance coefficient or Manning roughness.

Regardless of the used relation for computing the average velocity, the flowrate value must be the same.

From relations (13), (14) and (15) it can be obtained:

$$n = \frac{R^{2/3} A \sqrt{I}}{Q}. \tag{16}$$

For the equation (16) the flowrate can be computed using Eq. (8) or (12).

4. Results

By using the equations (8), (13), (14), (15) and (16) and also by using a computer software, there have been established some equivalence relations between the absolute roughness k and the Manning roughness.

This correspondence between the two types of roughness for a few different transversal section shapes is shown in Fig.2.

For rectangular cross sections, in Fig.3 is presented the influence of the cross section magnitude concerning the roughness. The curves are drawn for normal depth $h_0 = 1.50\,m$ and for width b variable. It comes out that for small value of the ratio b/h_0 the influence of the cross section magnitude concerning the roughness is important and for large values of the ratio b/h_0 this influence is insignificant.

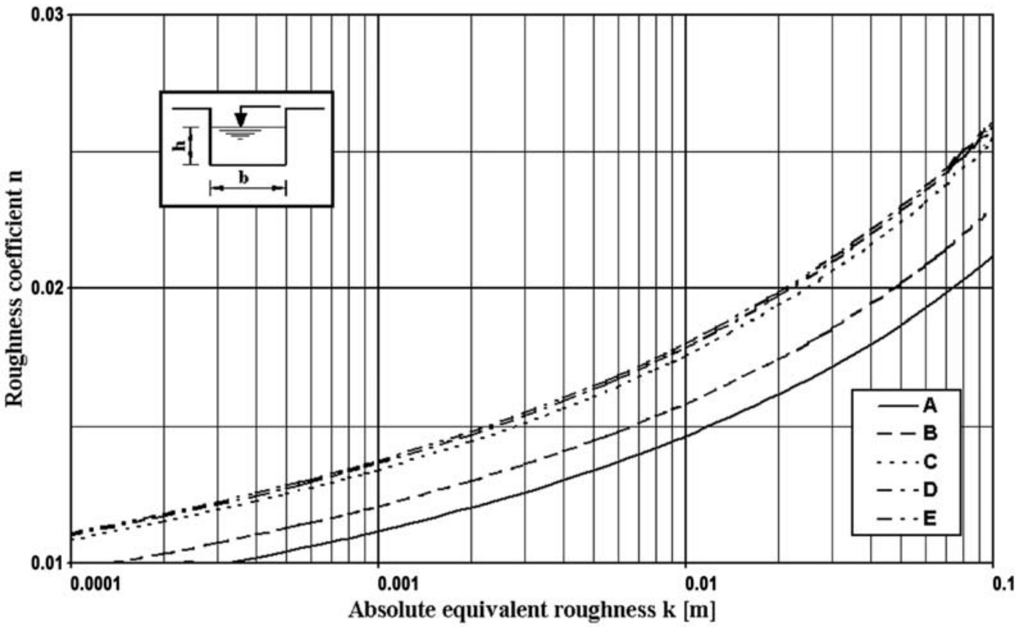

Fig. 3. The influence of the size of the transversal section area on roughness for rectangular section shapes
A: b/h_0=0.67; B: b/h_0=1.33; C: b/h_0=6; D: b/h_0=10.00; E: b/h_0=13.33

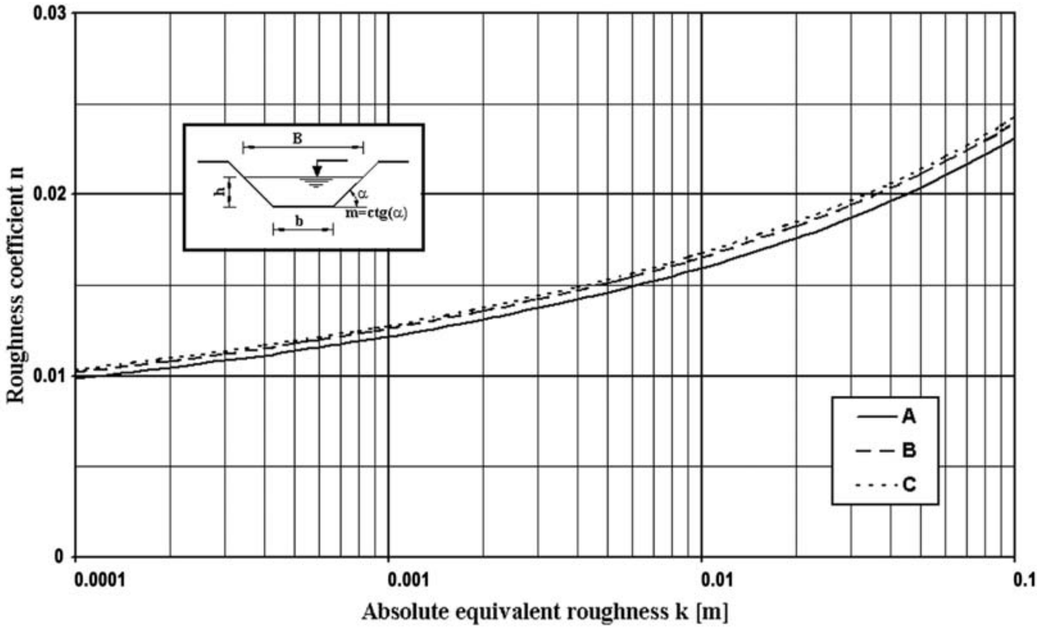

Fig. 4. The influence of the size of the transversal section area on roughness for trapezoidal section shapes
A: b = 0.50 m; m = 1.00; $B/h_0 = 2.33$
B: b = 1.50 m; m = 1.00; $B/h_0 = 3.00$
C: b = 2.00 m; m = 1.00; $B/h_0 = 3.33$

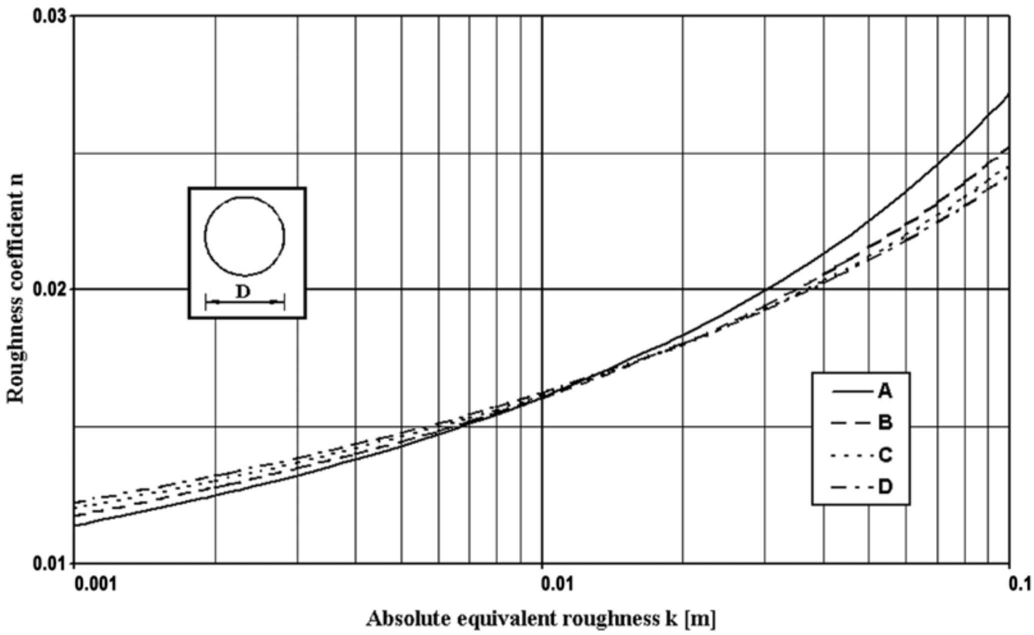

Fig. 5. The influence of the size of the transversal section area on roughness for circular section shapes

A: D = 0.50 m; B: D = 1.00 m; C: D = 1.50 m; D: D = 2.00 m

Figure 4 shows the influence of the cross section magnitude for the trapezoid cross sections shape. The normal depth $h_0 = 1.50\,m$, $m = tg\alpha = 1.00$ and the bottom width b is variable.

Figure 5 refers to circular cross sections shape.

5. Concluding remarks

According to the presented facts, the following conclusions can be drawn:

The speciality literature does not contain informations concerning a direct relationship between the equivalent absolute roughness and the Manning type roughness.

Using the theoretical formulas which gives the vertical velocity distribution in correlation with the average velocity expression given by Chézy, it can be deduced an equivalence relationship between the two types of roughness.

The results obtained with the presented method and the data in the specialty literature are very similar.

The roughness is an intrinsic characteristic of a rigid surface. However, the presented computations demonstrate that the Manning type roughness depends on the shape and the size of the transversal section area of the flow.

This dependence can be explained by the following reasons:

- The expression of the Chézy coefficient - Eq.(15)- proposed by Manning is obtained by empirical methods.
- The equations (6) and (10) which gives the vertical velocity distribution are not perfect because for $y = 0$ (at the bottom level) the velocity value is $v = -\infty$.

An accurate evaluation of the head losses means an accurate estimation of the energetical consumptions.

Therefore, because in the current engineering practice new materials are used to build canals or pipe networks, is necessary to perform studies and experimental tests for a accurate evaluation of the rigid surface roughness.

List of symbols

$$B = f\left[\frac{V_* k}{\nu}\right]$$

b - cross section width at bottom
C - Chézy coefficient
h_0 - normal depth
I - hydraulic slope
k - absolute equivalent roughness
m - lateral slope
n - Manning roughness coefficient
r_0 - geometric radius of the pipe
R - hydraulic radius
$V_* = \sqrt{r_0/\rho}$ - shear velocity
χ - Von Karman universal constant
μ - dynamic viscosity
ν - kinematic viscosity
ρ - density of the liquid
τ_0 - shear stress

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One Extension of Bernoulli's Equation in the Non-Stationary Movement of the Ideal Fluids

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Rezumat: În lucrare se deduce o ecuație pentru mişcarea nepermanentă unidimensională a lichidelor ideale, denumită ecuația energiei mecanice, care reprezintă de fapt o extindere a ecuației lui Bernoulli. Deducerea ecuației se face pe două căi: pe de o parte prin efectuarea derivatei substanțiale a energiei în metoda de reprezentare spațială a mişcării (Euler) şi pe de altă parte scriind în două moduri ecuația de bilanț a energiei (analog cu modul în care se deduce ecuația de continuitate.

Abstract: In this paper we deduced an equation for the one dimensional non stationary movement of the ideal liquid, named the mechanical energy's equation, which represents in fact an extension of the Bernoulli's equation. The deduction of this equation is made by two ways: on the one side by effectuating the substantial derivate of energy in the spatial representation method of movement (motion) and on the other side writing in two ways the energy survey equation (analog to the deduced way of the continuity equation).

Keywords: *energy, Bernoulli's equation, permanent and nonpermanent states.*

1. Introduction

It's well known the Bernoulli's equation [1], [2], [3], [4] in the non-permanent movement of the ideal fluids, which express the fact that certain equation is constant along the current line:

$$\frac{1}{g}\frac{\partial \vec{u}}{\partial t}d\vec{l} + d\vec{l} \cdot gradH = 0 \Rightarrow$$

$$\frac{1}{g}\int_{Lcrt}\frac{\partial \vec{u}}{\partial t}d\vec{l} + H = \tag{1}$$

$$\frac{1}{g}\int_{Lcrt}\frac{\partial \vec{u}}{\partial t}d\vec{l} + \frac{u^2}{2g} + \frac{p}{\gamma} + z = C(Lcrt, t)$$

the constant depending by the considered current line and the respective moment.

If in the permanent state (stationary state), in which the locale derivate is null, the things are clear, in the non-permanent state the things are not the same.

It's also known the energetically interpretation of this equation's terms, that's the reason for what, in some papers [3], this equation is named the energy equation.

We observe that in the infinitesimal domain, the current line element coincide with the trajectory's element, the both having the same direction with the velocity vector. In general, the current line doesn't coincide with the fluid particle trajectory, it happens only in the case of the permanent movement or the one-dimensional non-permanent movements.

The proposed extension for the Bernoulli's equation is based on the conservation principle of energy, valuable in the case of the conservative forces the ideal fluids and on the Jukovski theory about water hammer.

2. The equation of energy

Corresponding to the conservation principle of energy, the specific energy (on the weight unit) whom possess the fluid particle in the gravitate field must be constant as long as the particle moves on the trajectory.

In the material representation method (Lagrange) of the movement, this writes:

$$H = \text{constant} \rightarrow \frac{dH}{dt} = 0 \tag{2}$$

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While, in the spatial representation method (Euler) of the movement, this writes:

$$\frac{\partial H}{\partial t} + \vec{u} gradH = 0 \tag{3}$$

Usually, this equation is obtained using the condition of dynamic equilibrum (Newton's law).

It's interesting to remark that we can obtain the same equation by another way, based on the conservation principle of energy, in a similar way like the continuity equation. We exemplify this further.

Thus, considering a rectangle prism with Δx, Δy, Δz sides, it results that the liquid mass energy which exists into this prism is done by the formula:

$$E = H\rho g \Delta x \Delta y \Delta z \tag{4}$$

We write the energy variation in two ways in the time interval Δt:

a) $$\Delta E = \frac{\partial E}{\partial t}\Delta t = \frac{\partial(\rho H)}{\partial t} g\Delta x \Delta y \Delta z \Delta t \tag{5}$$

b) We write the energy variation like the difference between the input energy and the output energy on the three directions in the same time:

$$\begin{aligned}\Delta E &= -\left[\frac{\partial(\rho\vec{u}H)}{\partial x} + \frac{\partial(\rho\vec{u}H)}{\partial y} + \frac{\partial(\rho\vec{u}H)}{\partial z}\right] g\Delta x\Delta y\Delta z\Delta t = \\ &= -div(\rho\vec{u}H) g\Delta x\Delta y\Delta z\Delta t\end{aligned} \tag{6}$$

Equaling the two expressions, after simplification with $g\Delta x\Delta y\Delta z\Delta t$ and passing to the limit for Δx, Δy, Δz, Δt tending to zero we obtain the equation:

$$\frac{\partial(\rho H)}{\partial t} + div(\rho\vec{u}H) = 0 \tag{7}$$

We consider now the continuity equation which expresses the mass conservation (which is obtained by an analogue mode):

$$\frac{\partial \rho}{\partial t} + div(\rho\vec{u}) = 0 \tag{8}$$

If we develop the equation (7) and we consider equation (8), we find again the equation (3), which justify the name, mechanical energy equation:

$$\begin{aligned}&\frac{\vec{u}}{g}\frac{\partial\vec{u}}{\partial t} + \frac{\partial(p/\gamma)}{\partial t} + \vec{u} gradH = 0 \\ &\Leftrightarrow \vec{u}\left(\frac{1}{g}\frac{\partial\vec{u}}{\partial t} + gradH\right) + \frac{\partial(p/\gamma)}{\partial t} = 0\end{aligned} \tag{9}$$

Applying the Jukovski's theory for the non-stationary flow of the liquids in the under pressure pipes, which establish the relation between the pressure local variation and velocity local variation (the both propagate always with the elastic wave propagation velocity), we obtain [4]:

$$\mathrm{dp} = \pm\rho \mathrm{c}\mathrm{dv} \Rightarrow \frac{\partial \mathrm{p}}{\partial \mathrm{t}} = \pm\rho \mathrm{c}\frac{\partial \mathrm{u}}{\partial \mathrm{t}} \tag{10}$$

The energy equation can be writing:

$$\frac{u}{g}\left(1 \pm \frac{c}{u}\right)\frac{\partial u}{\partial t} + \vec{u} gradH = 0 \tag{11}$$

Amplifying the relation with the infinitesimal dt, the energy relation get the following form:

$$\begin{aligned}&\frac{1}{g}\left(1 \pm \frac{c}{u}\right)\frac{\partial u}{\partial t} dl + d\vec{l}\ gradH = 0 \\ &\leftrightarrow \frac{1}{g}\int_{Lcrt}\left(1 \pm \frac{c}{u}\right)\frac{\partial u}{\partial t} dl + H = C(Lcrt, t)\end{aligned} \tag{12}$$

We observe that the obtained equation represents an Bernoulli's equation extension, containing one supplementary term.

It results that this equation is a union of Euler-Bernoulli's theory and Jukovski's one, containing the both of them like particularly cases corresponding to the lent variable and rapid variable movements.

3. Implications on the one-dimensional problems solution in the non - stationary movements

Indeed, in the lent variable movements, in which a significant variation of the velocity, the waves from the under pressure pipe will covered this for many times and in the both senses, so that:

– for the progressive waves

$$\frac{1}{g}\left(1+\frac{c}{u}\right)\frac{\partial u}{\partial t}dl + d\vec{l}\ gradH = 0 \qquad (13)$$

– for the regressive waves

$$\frac{1}{g}\left(1-\frac{c}{u}\right)\frac{\partial u}{\partial t}dl + d\vec{l}\ gradH = 0 \qquad (14)$$

is admissible to consider the average expression, that is classical form of Bernoulli's equation:

$$\frac{1}{g}\frac{\partial u}{\partial t}dl + d\vec{l}\ gradH = 0 \qquad (15)$$

In the rapid variable movements, is known the four equations system of the water hammer [6], where we considered the velocity uniform distributed on the cross section of the pipe:

$$\begin{cases} dx = \pm c dt \\ \pm\dfrac{c}{g}\dfrac{dV}{dt} + \dfrac{dH}{dt} = 0 \end{cases} \qquad (16)$$

The plus sign correspond to the progressive wave (which itself propagates in the same sense with the velocity) and minus sign to the regressive wave (which itself propagates in the contrary sense with the velocity).

In the rapid variable movements we discard to the unity which appears in the extend equation, thus that we start from the equation:

$$\pm\frac{c}{g}\frac{\partial V}{\partial t} + V\frac{\partial H}{\partial x} = 0 \qquad (17)$$

At this equation we add the other differential equation of the water hammer:

$$\frac{\partial H}{\partial t} + \frac{c^2}{g}\frac{\partial V}{\partial x} = 0 \qquad (18)$$

By addition of these equations results:

– for the progressive wave

$$\frac{\partial H}{\partial t} + V\frac{\partial H}{\partial x} + \frac{c}{g}\left(\frac{\partial V}{\partial t} + c\frac{\partial V}{\partial x}\right) = 0 \qquad (19)$$

– for the regressive waves

$$\frac{\partial H}{\partial t} + V\frac{\partial H}{\partial x} - \frac{c}{g}\left(\frac{\partial V}{\partial t} - c\frac{\partial V}{\partial x}\right) = 0 \qquad (20)$$

Taking into consideration that the energy displaces in the same time with the fluid particle, with the V velocity considered to the energy equation deduction, results that in the spatial representation method of movement:

$$\frac{\partial \mathrm{H}}{\partial \mathrm{t}} + \mathrm{V}\frac{\partial \mathrm{H}}{\partial \mathrm{x}} = \frac{\mathrm{dH}}{\mathrm{dt}} \qquad (21)$$

Also, the variation of velocity propagates under the wave form with the velocity of the elastic waves (celerity) $\pm c$ so that:

– for the progressive wave

$$\frac{\partial V}{\partial t} + c\frac{\partial V}{\partial x} = \frac{dV}{dt} \qquad (22)$$

– for the regressive wave

$$\frac{\partial V}{\partial t} - c\frac{\partial V}{\partial x} = \frac{dV}{dt} \qquad (23)$$

We obtain the following four equations system, which can be bring to the initial system by algebraic calculus:

$$\begin{cases} \pm\dfrac{c}{g}\dfrac{dV}{dt} + \dfrac{dH}{dt} = 0 \\ dx = \pm c dt \end{cases} \qquad (24)$$

The calculus formulas and the numerical methods used in problems, concerning to the mass oscillations in the surge tank of the hydro-energetically system like the water hammer problems remain valuable. They are not affected by the proposed extended form of the

Bernoulli's equation, because the extended equation contains the particularly cases for the lent variation and rapid variation problems, previously treated.

4. Implications on the one-dimensional problems solution in the free surface systems

In the most cases, in the free surface systems, the velocity of elastic waves (celerity) has the same order with the medium velocity. The flow state is named lent or subcritical when the celerity is inferior to the medium velocity. In the contrary case, the flow state is named rapid or supercritical. [6].

The limit case which separates the two states is named critical state.

Results that in the critical and supercritical states the waves can be propagate downstream (so exists the progressive wave) and they can not propagate in upstream. In this case the extended Bernoulli's equation (the equation of energy) becomes in the nonstationary state:

$$\frac{v^2}{2g} + \frac{cv}{g} + \frac{p}{\gamma} + z = \text{constant at certain moment} \quad (25)$$

We observe that comparatively with the classical Bernoulli's equation, it appears a supplementary term.

For c the usual formula is:

$$c = \sqrt{gh} \quad (26)$$

Substituting $\frac{p}{\gamma}$ by h, we obtain the following form:

$$\frac{v^2}{2g} + v\sqrt{\frac{h}{g}} + h + z_p = \text{constant} \quad (27)$$

(z_p – bench mark of the bed)

In uniform flow (h = const., v = const.) this is reduced to the classical Bernoulli's equation.

5. Conclusions

In this paper we deduced an equation for the one dimensional non stationary movement of the ideal liquid, named the mechanical energy's equation, which represents in fact an extension of the Bernoulli's equation. Moreover, this equation takes into account the energy that the fluid particle transports due to the fact that it is in a wave field.

The deduction of this equation is made by two ways: on the one side by effectuating the substantial derivate of energy in the spatial representation method of movement (motion) and on the other side writing in two ways the energy survey equation (analog to the deduced way of the continuity equation.

Analyzing the implications on the problems solution, we conclude that in the under pressure systems (the propagation velocity of elastic waves is much bigger than the velocity), don't appear changes of the actual methods for the lent and rapid variation movements. But in the free surface systems, where the velocity of elastic waves (celerity) has the same order with the medium velocity, appears such modification. In the case of the critical state, the term representing the kinetically energy becomes double by compare with the classical Bernoulli's equation. In the future, this problem must get thoroughly into more studies.

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Alternative Solutions for Protection against Waterhammer of Hydropower Plants

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Rezumat: În comunicare este analizată posibilitatea utilizării rezervoarelor de aer ca alternativă la protecția la şoc hidraulic a uzinelor hidroelectrice, în comparație cu soluția clasică utilizată în mod curent - castelele de echilibru. Sunt prezentate rezultatele unor studii hidraulice efectuate pentru proiectarea a două uzine hidroelectrice din România, care, în prezent, sunt în funcțiune.

Abstract: The paper analyses the possibility of using the Air Chamber alternative for the protection against waterhammer of Hydropower Plants (HP) in comparison with the classical solution with Surge Tank, which is currently utilised. For this analysis the results of a case hydraulic study have been used. This study was carried out for the design of two Small Hydro in Romania that are now in operation.

Keywords: Hydropower Plant, Air Chamber, Surge Tank.

1. Introduction

As it is known, the protection solutions of hydropower plants against waterhammer are either of the type of an surge tank, or of type air chamber [2,4,7,12].

Type air chamber solutions are used especially to pumping stations and exceptionally to hydropower plants. In this last case they are also called "closed surge tanks". In the literature many examples of important hydropower plants are mentioned, equipped with air chambers, which operate in security conditions [1,3,10,11].

In comparison with the surge tanks, which operate completely secure, the air chambers require some very strong operating rules, which, when not respected, may produce catastrophes (as examples: breaks of valves, pipes, pressure headraces etc), with very serious consequences.

We point out that the principal conditions in operating the air chambers require to keep the values of the volume of the air cushion in steady flow in the special circumstances of the corresponding hydropower plants.

This may be one of the main reasons for using more seldom the air chambers as a protection solution for hydropower plants.

On the other hand, in the current hydrotechnical practice of designing hydropower plants, are often met situations in which the use of a solution of the surge tank type would lead to works of a great difficulty, or will require very high costs (underground hydrotechnical works of very great volumes, relatively very high concrete tanks, etc).

In such situations is normal to analyse, and, if it is convenient, to adopt the air chamber type solution.

Until now, in Romania, no hydroelectric plant was provided with air chamber type system. The first hydraulic studies in Romania in view of designing hydropower plants (HP) were performed at the Hydraulic Engineering Research Institute in Bucharest (Institutul de Cercetari Hidrotehnice -Bucuresti), [8].

The hydraulic transient computations for hydropower plants and pumping stations are presented in several known papers: Chaudhry [2], Jaeger [4], Popescu [5,6,7], Wylie [12], hence we shall not insist on them.

In this paper we analyse the possibility of applying the protection solution against waterhammer to waterpower plants with air chambers [9], starting from a hydraulic study for designing SH Valenii de Munte and SH Izvoarele, in Romania (performed by Institute of Hydroelectric Studies and Design: ISPH Bucharest).

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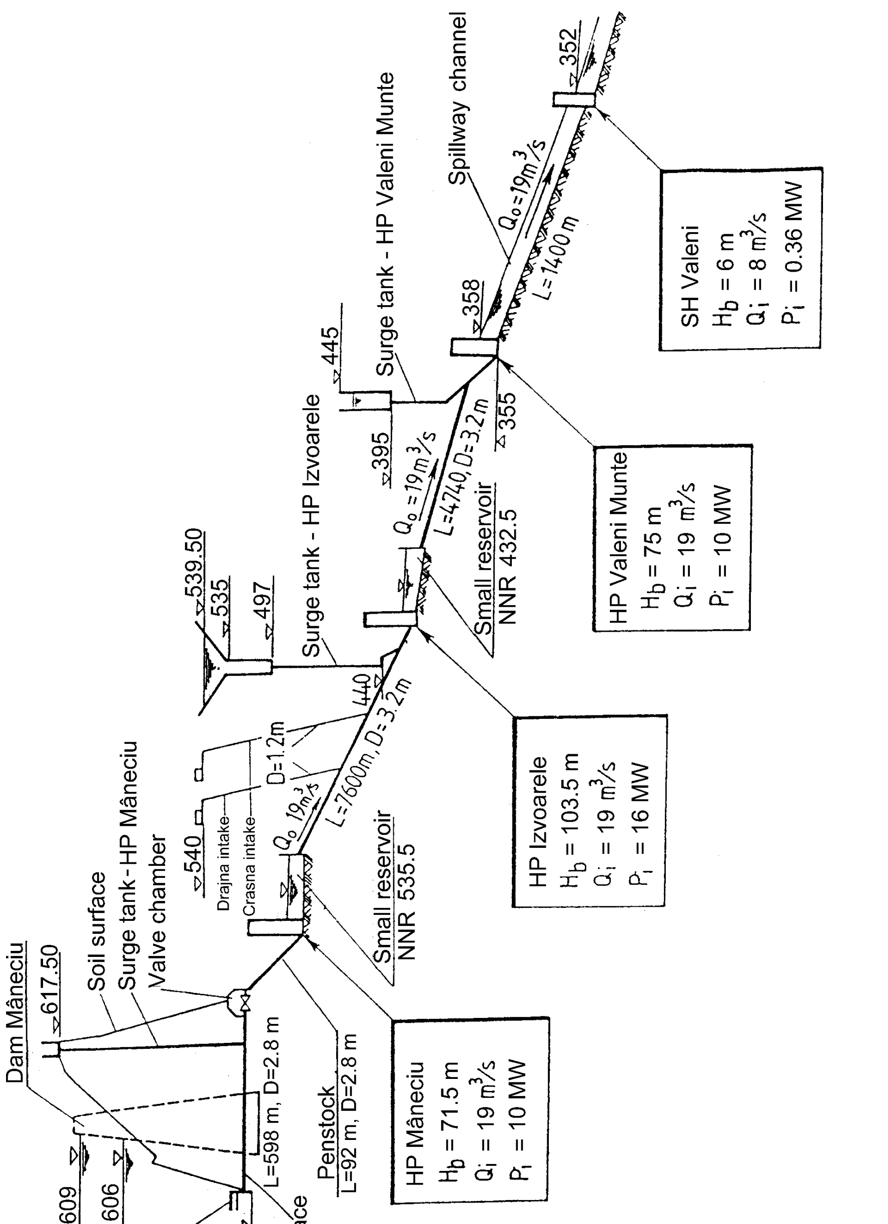

Fig. 1. Hydroelectric scheme of Maneciu: HP Valenii de Munte and HP Izvoarele (synoptical profile)

Taking into account the installed powers of the hydropower plants Valenii de Munte and Izvoarele of 10 MW, 16 MW respectively, in this paper we used the term "Small Hydro" (SH).

2. Some technical characteristics of Hydropower Plants Valenii de Munte and Izvoarele

Hydropower plants Valenii de Munte and Izvoarele are among a sequence of plants situated on the Teleajen River downstream of the Maneciu dam and reservoir, designed on similar hydraulic schemes.

The Maneciu dam has 78 m height, 60 millions cubic meters reservoir capacity, and multiple purposes: hydroelectricity, drinking and industrial water supply, irrigation, flood control etc.

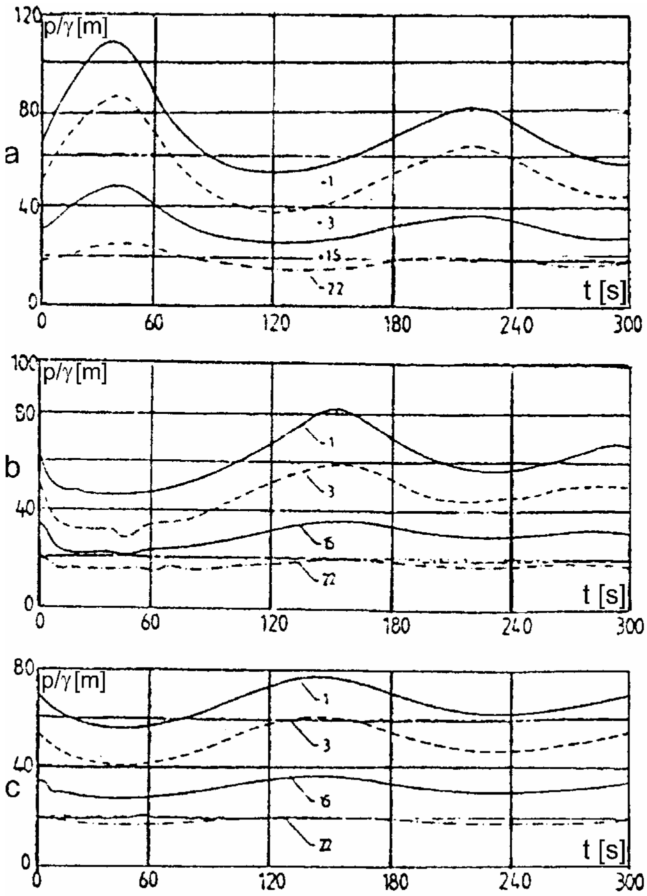

Fig. 2. HP Valenii de Munte (variant B).

Results of the computations in the pipe for the following actions at the central:
a) stopping the central when working with two turbines;
b) starting the central with two turbines;
c) starting the central with one turbine.

2.1. Hydropower Plant Valenii de Munte

Among the general technical characteristics of the plant we mention (figure 1):

- General characteristics: total hydraulic head H=75 m., steady state discharge Q=19 mc/s, installed power P=10 MW;
- Metallic pressure pipe situated on soil surface, along the Teleajen river, has a total length L=4740 m, diameter D=3200 mm;
- Underground surge tank, situated on the pressure pipe, of diameter D= 3200 mm and above soil upper chamber of height H=50 m and diameter D =7.50m;
- The plant is equipped with two hydraulic turbines of Francis type, of power 6.5 MW each, steady state head of 65 m. Each energetic group has butterfly vans calculated to support 11 atm.

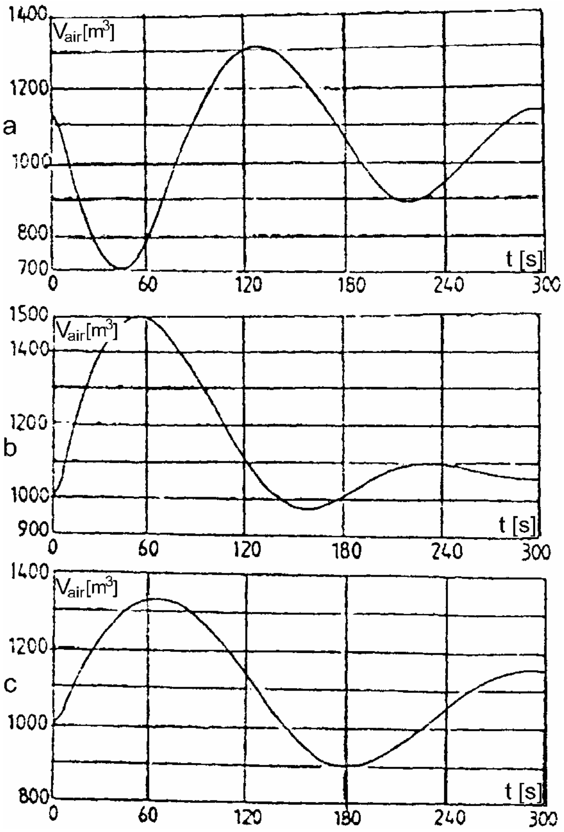

Fig. 3. HP Valenii de Munte (variant B):
Time variation diagrams of the volume of the air cushion for the mentioned actions in fig. 2.

2.2. Hydropower plant Izvoarele

Among the general technical characteristics of the plant we mention:

- General characteristics: total hydraulic head H=103 m, steady state discharge Q=19 mc/s, installed power P=16 MW;
- Metallic pressure pipe situated on soil surface, along the Teleajen river, has a total length L=7600 m, diameter D=3200 mm;
- Underground surge tank, situated on the pressure pipe, consisting of two parts of diameters D=1800 mm, 6300 mm respectively, and above chamber at soil surface.
- Secondary intake shafts Drajna and Crasna are directly connected to the pressure pipe by pipes of diameter D=1200 mm;
- The plant is equipped with two hydraulic turbines of Francis type, of calculus power of 8 MW each, steady state head of 95 m and, as the Valenii de Munte plant, with butterfly vans.

3. Alternative solution of protection against waterhammer of those hydropower plants

3.1. Hydropower Plant Valenii de Munte

For this plant were analysed the following variants of protection against waterhammer:

A. Underground surge tank on the pressure pipe;
B. Air chamber situated near the central;
C. Mixed solution with surge tanks and air chamber.

We present the results of hydraulic computations for the variants B and C.

In order to obtain the optimal protection solution of type air chambers placed near the central, ten constructive variants were calculated and analysed, for different values of the characteristic parameters: volume of the air cushion at the steady state pressure (V_{air}), the hydraulic resistances of the connecting pipes of the reservoirs with the pressure pipe and the total volumes of the air chamber (V_{tot}).

The hydraulic computations were performed for the actions imposed by the operation, namely: either the total stopping of the plant when both turbines were working, or the successive starting of the two turbines. The computations put into evidence the fact that the complete starting of the central, simultaneously of the two turbines, is a very dangerous action, existing the possibility of penetrating of some air volumes in the pressure pipe, hence it must be eliminated from the current operation of the plant. For starting the central it is recommended to start successively the two turbines, at a time interval of 10 minutes.

The results of the hydraulic computations (variant B) are presented for the optimal functional and technico-economical variant and are graphically illustrated in figures 2 and 3.

We mention that a mixed type protection solution (variant C) was also analysed, with air chamber and surge tank (H=35 m and D=4 m) placed at 1300 m distance downstream from the water reservoir, the results of hydraulic computations being presented in figures 4 and 5.

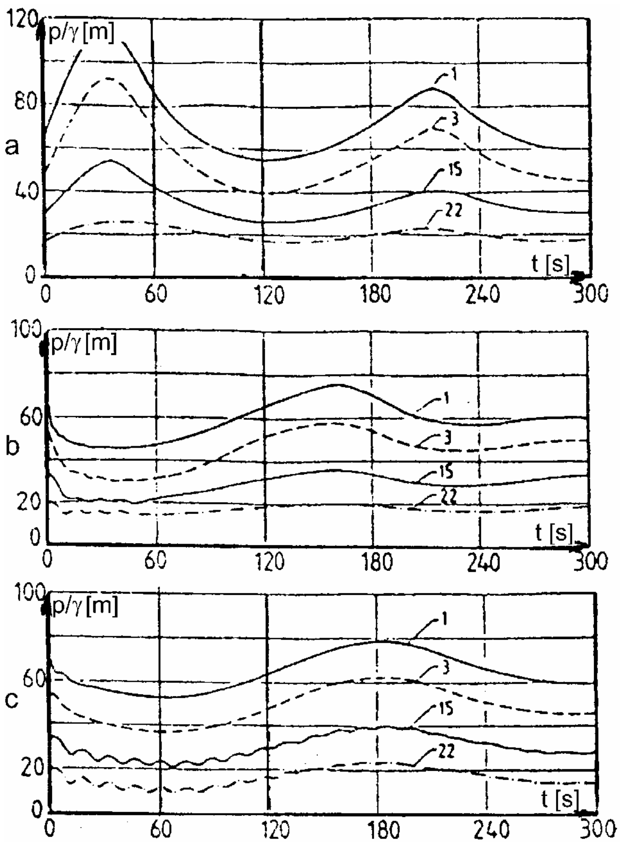

Fig. 4. HP Valenii de Munte (variant C):
Results of the computations in the pipe for the mentioned actions in fig. 2.

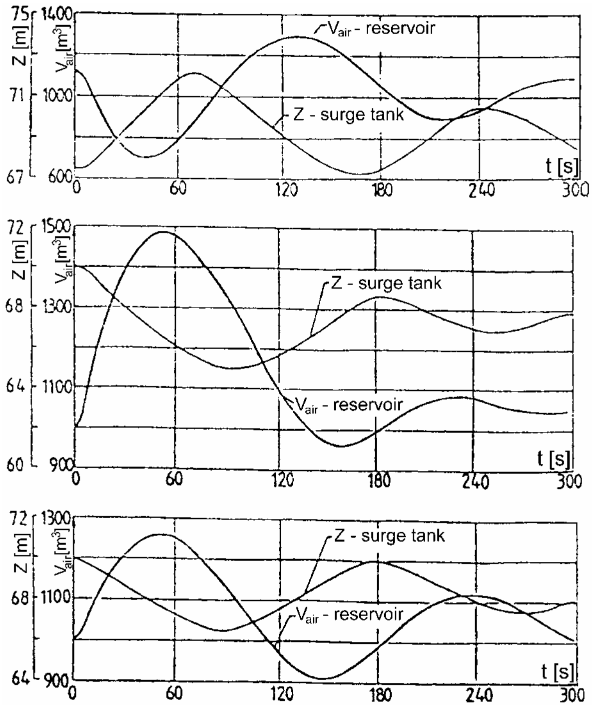

Fig. 5. HP Valenii de Munte (variant C)
Time variation diagrams of the volume of the air cushion for the mentioned actions in fig. 2.

Comparing the results of the computations one finds that the influence of the surge tank on the total volume of the air reservoirs and on the maximal and minimal extreme pressures in the pressure pipe is insignificant.

26 sections of calculation (Δx=190 m) on the pressure pipe were chosen for performing the hydraulic computation by numerical methods, the section number 1 being chosen in the turbine section.

For the protection system against waterhammer, of type air chamber placed near the central (variant B), the following geometric and hydraulic characteristics resulted:

-total volume of the air chambers: 1600 mc;

-total volume of the air cushion at steady state pressure (when the plant is working at maximal flow, Q= 19 mc/s): 1000 - 1100 mc;

-numbers of the link pipes for air chambers-pressure: 6 or 8; corresponding pipe diameters: 600 mm, or 550 mm;

-distance of the position to the central: 20 - 120 m.

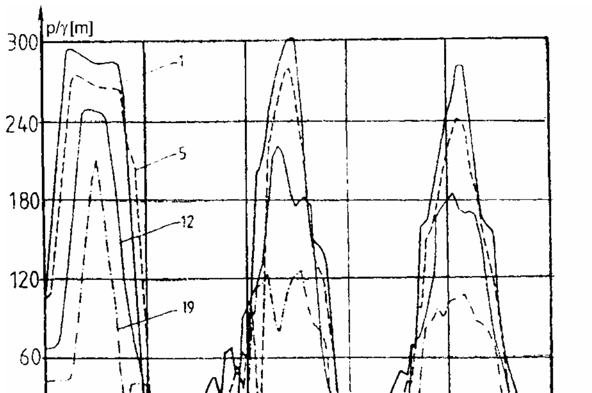

Fig. 6. HP Izvoarele. The results of the computations of the pressures in the pipe, without protection against waterhammer, when stopping the plant, with two turbines working.

We mention the fact that the solution "air chamber near the central" is practically realized by placing several air chambers, of smaller volumes, which have he same principal characteristics: total volume and air cushion volume at steady state operation.

3.2. Hydropower Plant Izvoarele

For this plant were analysed the following variants of protection against waterhammer:
A. Underground surge tank on the pressure pipe;
B. Air chamber situated near the central;
C. Underground surge tank at a distance from the pressure pipe.

We present the results of hydraulic computations for the variant C.

In this variant the underground surge tank has a link pipe (L = 527 m, D = 2800 mm) with the pressure pipe.

28 sections of calculation (Δx= 280 m) on the pressure pipe were chosen for performing the hydraulic computation by numerical methods, the section number 1 being chosen in the turbine section.

The results of the computations are presented in the figures 6 - 8 for the action of totally stopping of the central when working with two turbines.

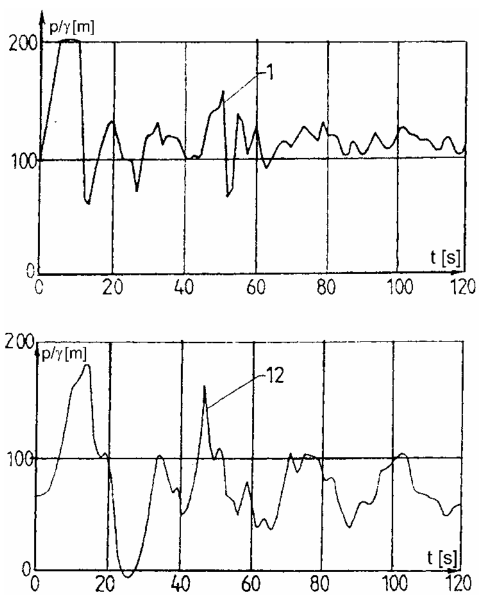

Fig. 7. HP Izvoarele (variant C). The results of the computations of the pressures in the pipe when stopping as in fig. 6.

In the variant without protection system against waterhammer, the hydraulic computations put into evidence the apparition of maximal pressures in the pressure pipe of 30 atm (fig. 6).

The computation program used, based on the model of a compressible fluid, allowed us to take into account the influence of the link pipe of the tank, of a relative great length, on the variable pressures and flows in the system.

From the results of the hydraulic computations graphically presented in fig. 7 we see that the unsteady state of the water in the system preserved its rapidly variable character and that the surge tank with a long linking pipe led to the decrease of the maximal pressure only until the threshold of 20 atm.

We emphasize that in this variant of protection with surge tank, the maximal values of the pressure are unsatisfactory, due to the possibility of appearing maximal pressures of 20.

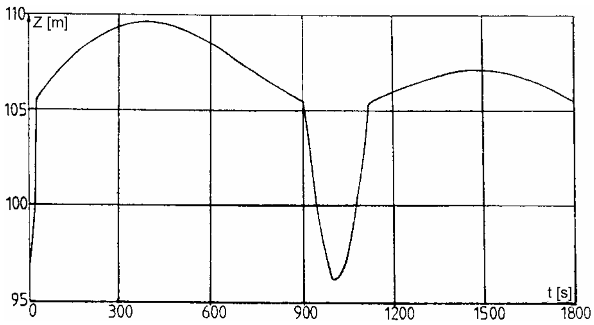

Fig. 8. HP Izvoarele (variant C). The results of the computation of the oscillations of the water level in surge tank when stopping as in fig. 6.

atm, and also to the probability of the presence in some areas of the pressure pipe of negative pressures, even of the cavitation pressure.

In figure 8 is presented the time variation curve of the water level in the surge tank, showing the maximal levels in the tank and the mass oscillation character of the movement in the system during a longer time period.

The result of the computations, presented above, show that such a solution is not technically acceptable, since it determines very large pressures in the system.

4. Final remarks

In designing the hydropower plants Valenii de Munte and Izvoarele it was chosen the protection against water hammer of surge tank type (variant A), although this solution implied costs 15-20 times greater in comparison with the air chamber one (variant B).

The surge tanks were placed in an area geologically stable without danger of landslide, but the length of the pressure pipes increased with 400-600 m.

The two hydropower plants were erected and are operating now.

However, the protection solution with air chamber against waterhammer must be considered by the design engineers of the hydropower plants, as a solution to be courageously introduced at the today level of techniques in this domain.

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Supporting & Protecting Buildings on the Bank Slopes of Danube-Black Sea Canal

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Rezumat: Pe traseul Canalului Dunăre – Marea Neagră s-au executat lucrări de consolidare a taluzurilor în argile roşii şi de protecţie a taluzurilor în crete şi calcare. În afara acestora, s-au mai realizat impermeabilizări cu loess şi pământ vegetal, precum şi lucrări de captare, conducere şi evacuare a apelor de infiltraţie şi de suprafaţă. În prezentul articol sunt prezentate în principal, construcţiile pentru consolidarea şi protecţia taluzelor.

Abstract: On Danube – Black Sea Canal's routing, rainforcement workings have been made for the red clays bankslopes and protective workings have been made for the slopes of chalks and limestones. Except these, vegetal dirt & loess impermeabilizations have also been accomplished and catchment, driving & evacuation of infiltration & superficial water, too. This article is about buildings for the reinforcing and protecting the slopings.

Keywords: *Consolidation works, protection works.*

1. Geomorphological Conditions

Along Danube Canal – Black Sea two different morphological sections can be well seen: the first one – Carasu Valley, tributary to Danube and the second one – the peak top area, the elevated plane that interfere between Danube's pond and the seawater.

Carasu Valley alluvial plain is a subsidence area with a low altitude measuring between 4-5 m at Cernavodă – Saligny and 12 – 15 m at Poarta Albă – Basarabi. On the elevated plane area, (Basarabi – Agigea, the surface's levels touch 60 - 70 m on the biggest part on the canal's length.

On Carasu Valley's area the bank slopes' sustaining and reinforcement workings and the phreatic waters and surface waters taking collecting and evacuating have a relatively reduced development; they are concentrated on the bay from Cernavodă, on the hill's mouths from the 18th km, the 23rd km (entry at Medgidia), the 30th km and on Basarabi's crossing. The workings for the bank slopes' protecting, sustaining and reinforcement have a special extension because of the ditch's deep and the length of overpassing 20 km on the section between Basarabi and Agigea.

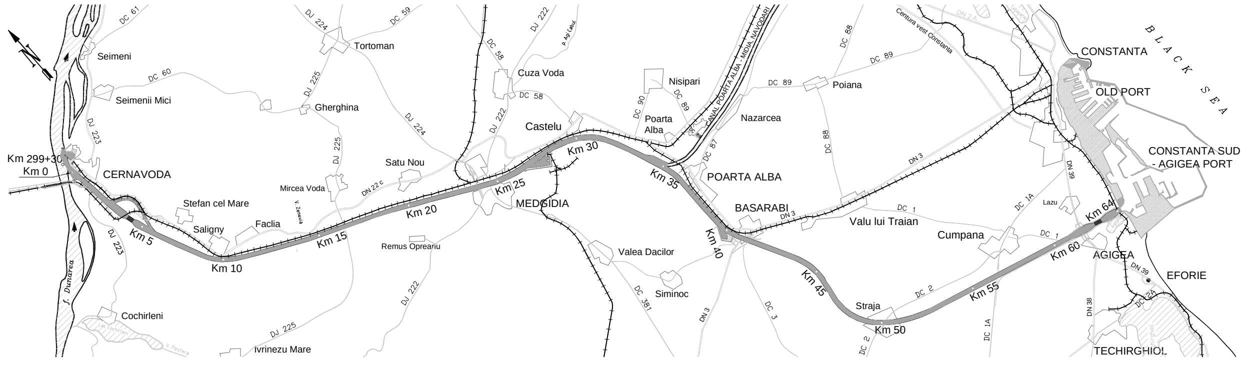

Fig.1.General Plan Danube-Black Sea Canal

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This ditch's excavations emphasized, red clays, green clays, and sands, chalks deposits, limestone's and limes boulders, under a pack of loess whose thickness is not less than 7-8 m.

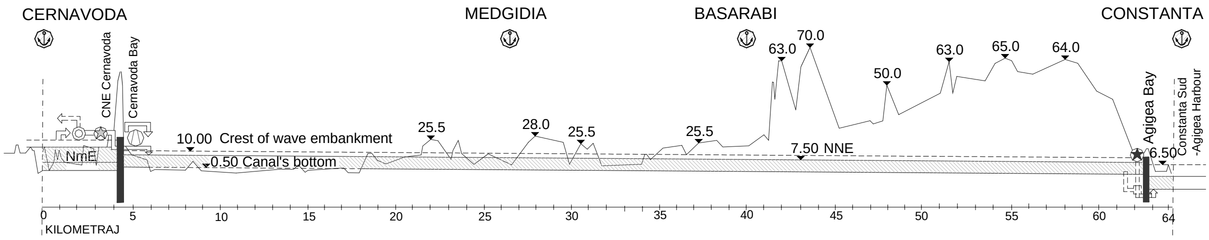

Fig.2.Longitudinal profile Danube-Black Sea Canal

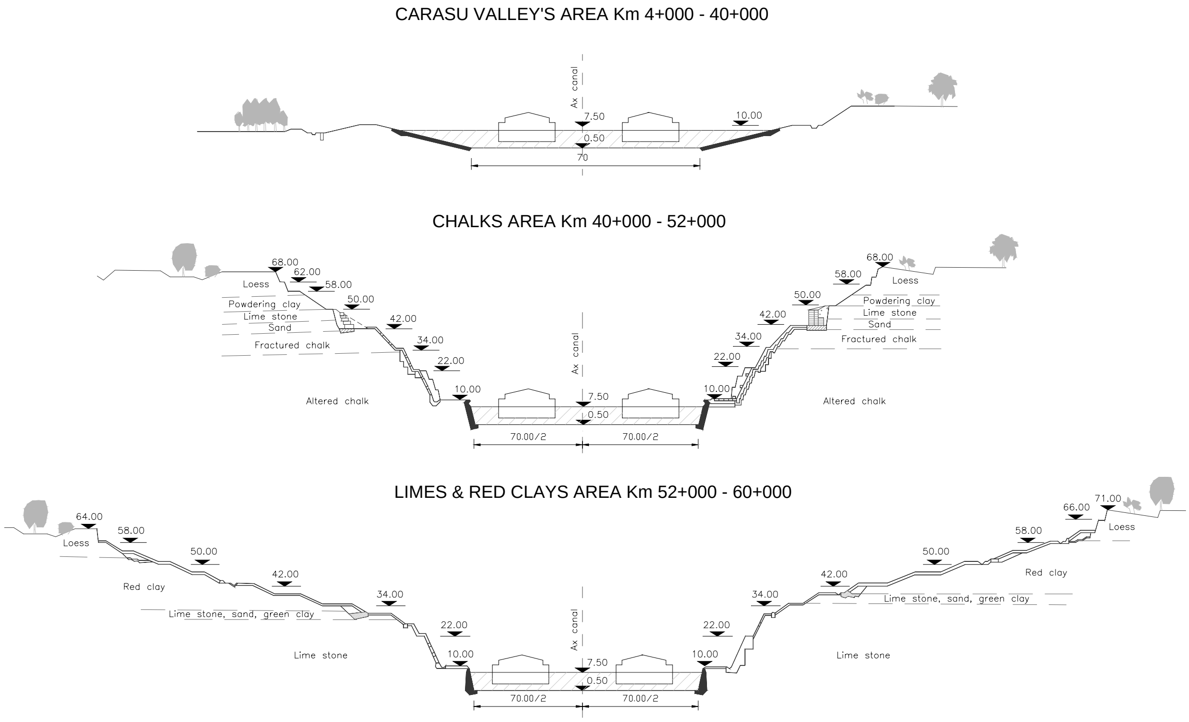

Fig.3. Characteristical profiles on Danube-Black Sea Canal

In order to assure the stability and integrity of the ditch's bank slope in the area of the elevated plane, a multitude of protecting sustaining and reinforcement workings have been used, and they are briefly presented as it follows.

2. Reinforcement Workings

On the canal's elevated plane section, its ditch has intercepted the geological complex made of structural red clays, with thick touching measure between 8 and 35 meters. These clays' acting bring up some difficult problems; knowing the fact that the clays are quite sensitive when it's about atmospherically agents they are affected by contractions and crozings raported to dry air and by heaving reported to water and by dipping, it gets very unpleasant physical – mechanical features, especially concerning the shearing resistance that happens to show a pronounced increasing.

The inforcement workings executed or being executed so that the bank slopes in red clays wouldbe stabilized is based on insolated reinforcements made of concrete, simple ones, with a resistance prism between them, insolated reinforcements on drilled pillars, lockers made of concrete steel.

The type of the workings has been chosen according to the local geomorphological conditions.

2.1. Insolated reinforcements made of concrete with rubble stone walls between them

This type of reinforcement working of the bank slopes with reinforcements made of concrete and stone wall, has been executed at the red clays' base, where their thickness overpasses 15 m, and on the inferior level they block the water, fact that can damage the whole bank slope's stability. The working is made of simple concrete reinforcements executed at interax distances variable between 8 – 11 m, with the elevation's height of 8 m. The space between reinforcements it is executed a wall made of rubble stone on a grating of foundation made of simple concrete having a slope for water discharge placed at the center. For water discharge behind the sustaining there are used concrete tubes both in the reinforcements' section and in the wall's grating of foundation. The reinforcements have been founded inside the level made of red clays

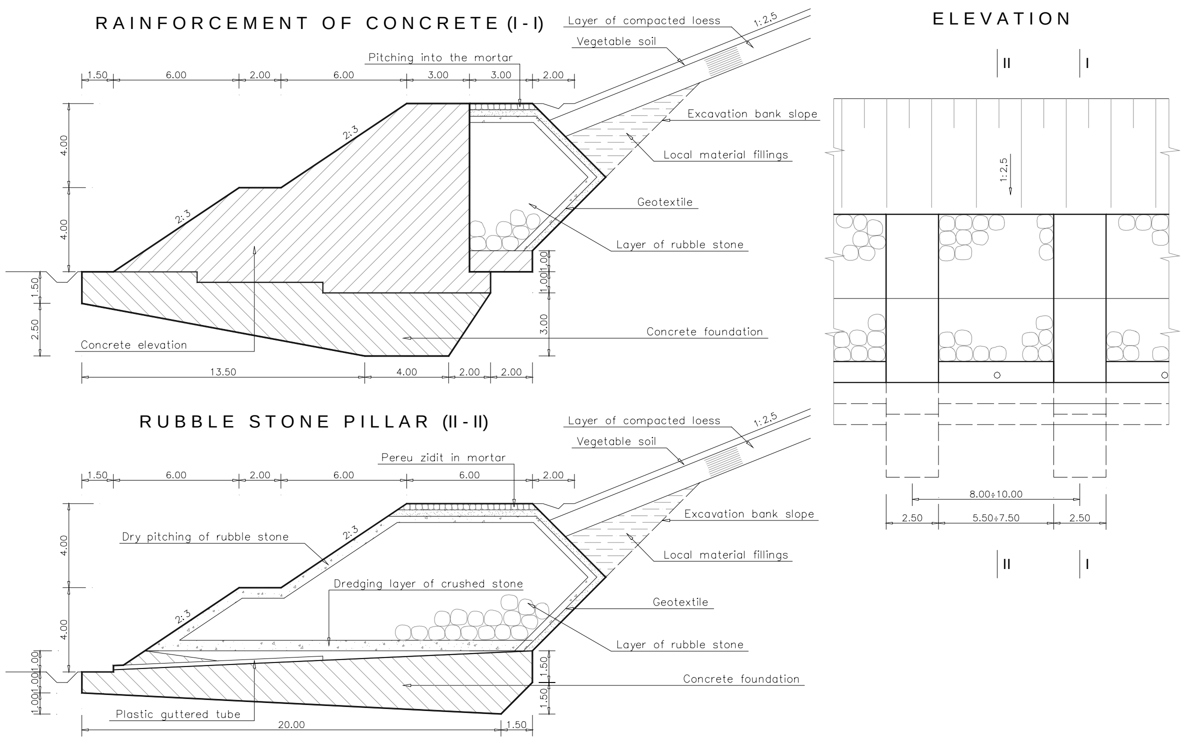

Fig.4.Reinforcement of concrete

2.2. Reinforcements on the drilled pillars

In the 57+400 – 57+900 km area left shore, workings have been executed for the bank slopes in red clays to be consolidated, with a slope of some reinforcements on drill pillars.

The constructive executed solutions consists in 16 reinforcements founded on concrete steel pillars ϕ

108 cm, placed at a 12 cm distance between them, at level +38,0 mrMB (superior level of the reinforcements' grating of the foundations).

Here's a reinforcement structure:

- three isolated vertical pillars (level +38 at level +24) executed in a vertical direction at the distance of 1.83 m between them;
- a pillar leanen to 14^o (level +38 at level +19.50) placed downstream (to canal's axe) from the vertical ones.

The pillars are fixed in boulders in the mass of clays, green clays, and limes altering with sands. At the end of the pillars a grating of the foundation made of concrete steel have been executed, having the length of 7.50 m, the extension of 1.30 m, and the height of 3 m.

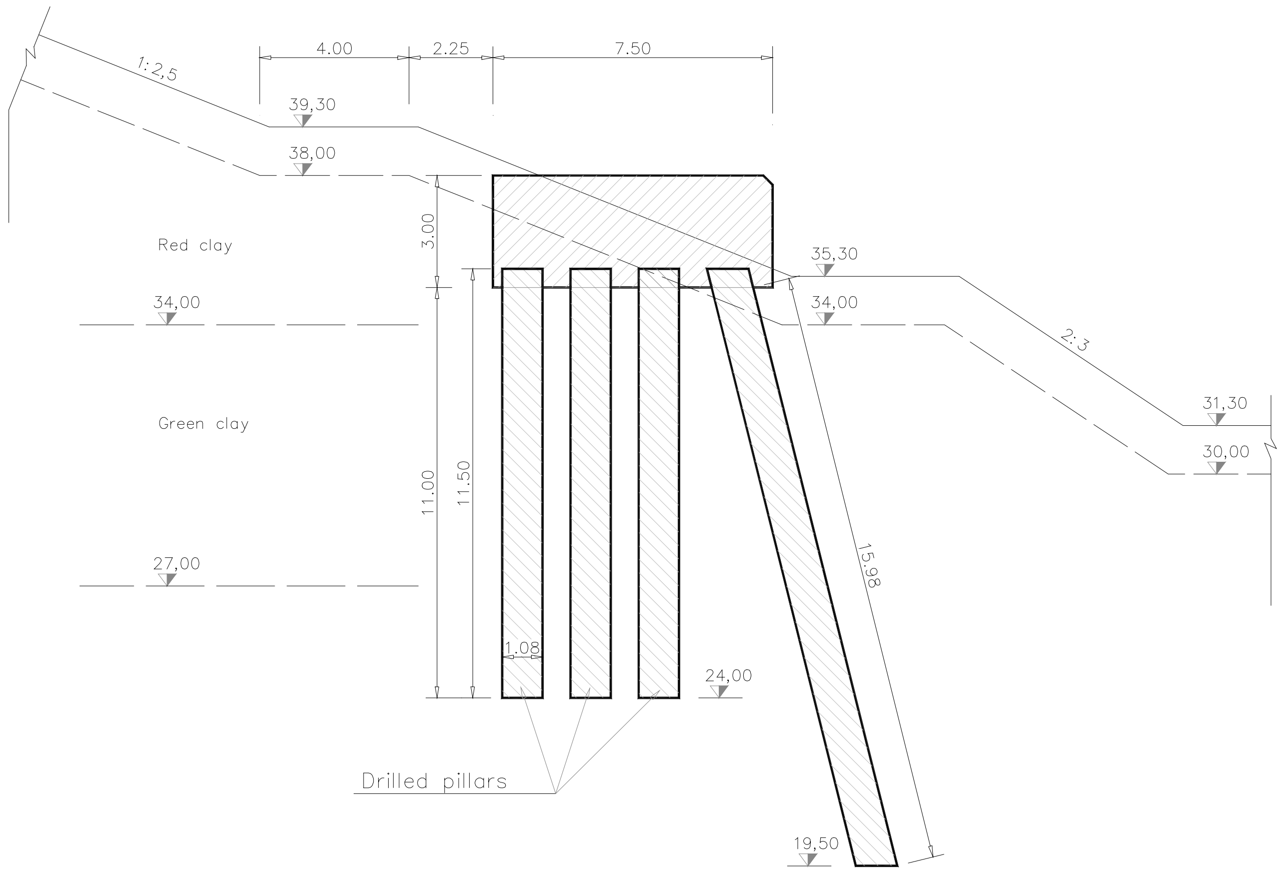

Fig.5.Reinforcement of concrete steel on pillars

2.3. Concrete steel lockers

The reinforcement's solutions regarding the unstable bank slopes with lockers have been applied behind some already existent reinforcement working with reinforcements and it has been chosen so that the pushing of the earth behind the pushed reinforcement would be taken over.

It has to be added the fact that the pushing has been produced on the groundsills and these do not show important movements and their structure is not affected.

The solution reflects the fact that it doesn't supposes open digging, so there's no risk of slides over the digging. The lockers show the advantage of dredging the infiltration water through small filters introduced in the sliding mass between them and through drains made of monogranulled concrete executed behind the lockers.

The reinforcement workings behind the reinforcements consist in the execution of a system formed of eight lockers opened with counterforts, made of concrete steel, spread over a lay of green lay and limestone. The lockers measure in plan 7x14 m^2 and deeps of 15 m, arranged at 10.50 m interact and they present two tubular counterforts each downstream with the empty section of 1.00 x 1.00 m^2 having the role of collecting the water between lockers.

The collected water driving along the reinforced waterfront is assured in every locker's area by the collector canal, the continuity between lockers being accomplished by steel collecting tubes introduced in the unclean land mass between lockers by some presses. The lockers' spaces sunk at the design's level finally fill up with concrete and the two spaces for evacuation of each locker (caught between the vertical walls) fill with dense land all over their height.

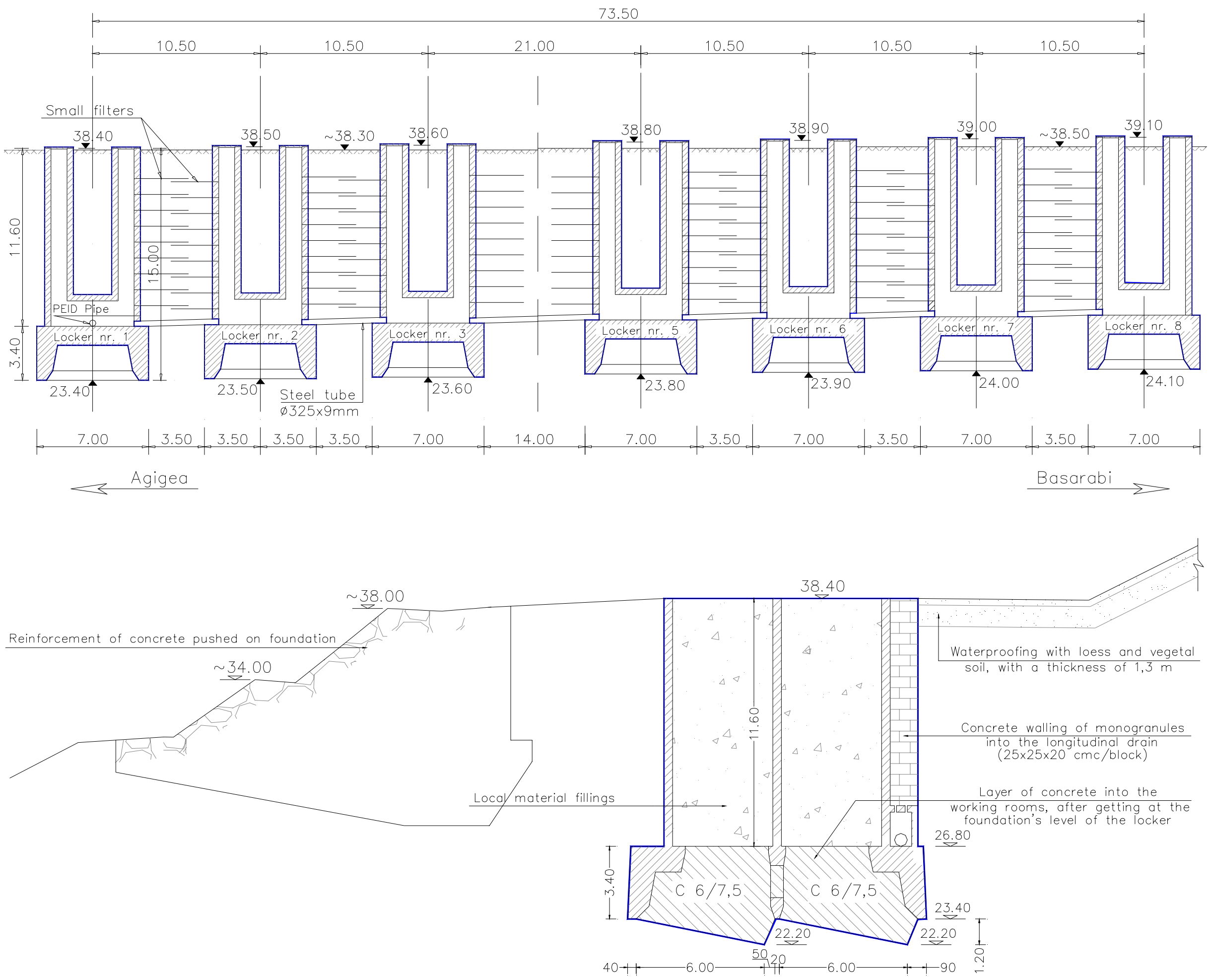

Fig.6.Lockers of concrete steel

3. Protection Workings

These workings have been made on a large scale, between +10 and +22 levels – rmBaltic Sea, on the canal's platform area, having a double role: of protection and sustaining in some reduced pushing's case. They've been applied not only for stopping the chalks' exfoliation's effect but for preventing the crashing of the bank slopes formed of thin sands, limestone's and fractured chalks, with water exfiltrations. The solution aims the protection with caissons and frames made of concrete steel filled with rubble stone; here's a detailed description.

3.1. The protection and the sustaining of the bank slopes with prefab caissons

These protections have been executed and they are still being executed on the bank slopes, usually formed of chalks, with variable gradients (1:1 ÷ 4:1), in order to reduce the atmospherical agents' effect that cause the exfoliating process of the chalks and the bank slopes' crashing into weak concrete sands from where the water exfiltration starts.

The caissons placed in check and filled with rubble stone, so that the protected bank slope does not obey the yearly cycles regarding the freezing - unfreezing process.

On the sections with bank slopes having a high alteration level, strongly fractured, with a favorable orientation for the blocks' break down, the wall of caissons is reinforced with concrete being continuously poured, adherent to rock and with the role of preventing the blocks'falling through of the bank slope.

The reinforcements have an extension of 1,2 ÷ 1,5 m and they have been placed (at distances) with spaces between them according to the massif's fractures.

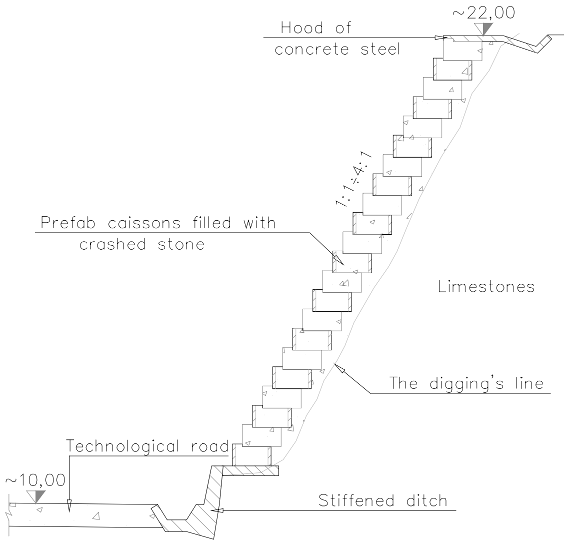

Fig.7.Bank slope protection with prefab caissons

Fig.8.Bank slope of limes protected with prefab caissons

3.2. The bank slopes' protection and sustaining with prefab elements type frame

This type of working has been preponderantly executed at +10 and +22 level (rm)Baltic Sea, too, having the same purpose as the protection with caissons, but in the situation regarding some bank slopes with an advanced level of degradation and fracture and in the case regarding some interlayers thicker than weekly cemented sands, water carrier.

For the protection's execution the prefab elements have been set by overlapping, and during their setting the filling with rubble stone have been executed from the prefab's interior and behind it.

The prefab presents two angle sidebars and two crossbars each of them having two supporting bars. The frames have been settled with their joints on the same vertical, or in check, with staggered joints.

The foundation has been made isolately in front of each sustaining crossbar.

The waters discharge takes place through the stiffened channel executed continously all over the area. At the wall's base inside the frames, they've used a concrete coping having a slope to the stiffened refuse spout under the wall.

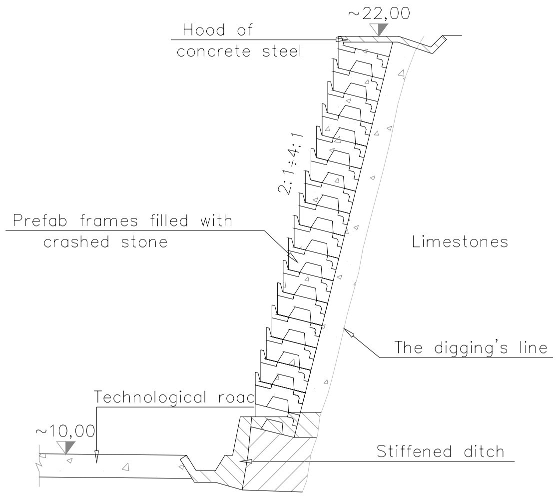

Fig.9.Bank slope protection with prefab frames

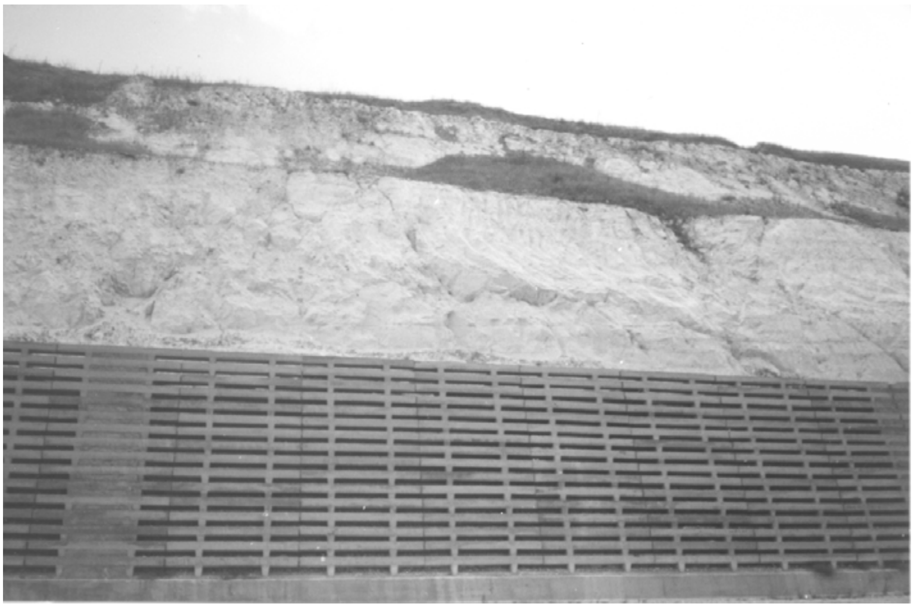

Fig.10.Bank slope of limestones protected with prefab frames

3.3. Working for water capturing, driving evacuation

The bank slopes protecting and reinforcement workings' described above, have been completed with workings for the water captating, driving and evacuation: drains, draining bands, draining masks ditches, channels, dischargers but also protecting working with loess and vegetal land of the bank slopes in red clays.

The drains (fig.11) have been executed in a crossing way on the bank slopes with downgrades of 1:1÷1:2,5 only where there is water inflow; these drains have reduced extensions (1,5 m) and they also have discharge end. The draining element is formed of crushed stone laid on an elastical invert made of plastic films and o lay of sand that replaces the rigid invert of concrete. The drain's hood is formed of compacted loess spreaded on a lay of sand.

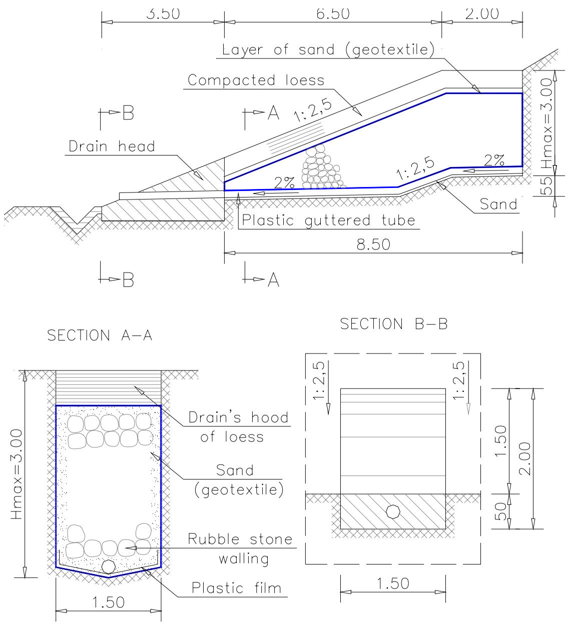

Fig.11.Drain

The draining masks (fig.12) are usually in the superior part of the red clays, when exfiltrations show up on bank slopes. The draining mask has a body of rubble stone covered with a lay of loess and vegetal land.

The system regarding the discharge of captated waters has been conceived like a prism made of rubble stone and weepers of plastic tubs, with a loess invert stabilized with a cement in the case of the discharge of waters into the surfaces channels or with an invert made of plastic in the case of the discharge of waters into the drains.

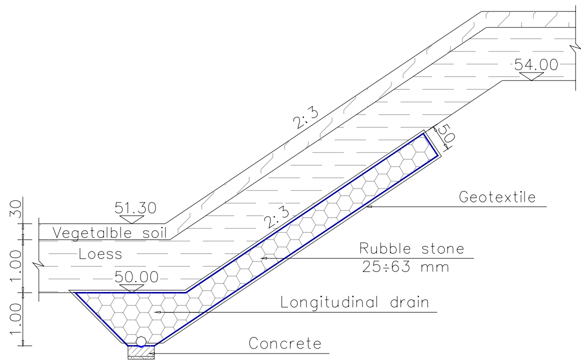

Fig.12.Draining mask

Some of the captating workings of the infiltration waters also have the robe of reinforcements as they have the shape of dredging bands and gabions at the bank slope's base.

4. Conclusions

It's been 20 years since the Canal functions and it is important sustained the fact that the executed workings show not negative effects. The adopted solutions have been correct. The stability problems discovered in time showed up because of the integral unexecution of the designed workings or because of the inappropriate conditions.

The workings' correct execution, solutions adopted according to the local conditions, their way of being kept in appropriate conditions, show only positive effects in time and obtaining the aimed target.

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Marine Structures with Thin Reinforced Concrete

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Rezumat: In domeniul inginerei civile marine structurile din beton armat se realizează de obicei din elemnte masive. Având în vedere că în ultimii ani s-au înregistrat progrese deosebite în tehnologia betonului obținând-se betoane de înaltă rezistență şi performanță precum şi betoane autovibrate, a devenit posibilă schimbarea opticii privind egalitatea structuri masive ≡ structuri masive. Lucrarea de față prezintă soluții din beton armat pentru scopurile ingineriei civile masive

Abstract: In the domain of marine civil engineering, the reinforced concrete structure ar usually realized of massive elements. Taking in account that in the last years great progresses were registered in the concrete technology, obtaining high resistance and performance concretes, than self-compacted concretes, changing the optic regarding the equality marine structures-massive structures became possible. This paper presents new solutions concerning the utilization of thin structures made of reinforced concrete for aims the propose of marine civil engineering.

Keywords: *marine structures, cable, thin reinforced concrete structures.*

1. Introduction

The structures utilized in marine civil engineering can be characterized usually by special massivity.

These massivity is justified by two reasons. On one hand it covers the problems of taking the actions on which these structures are loaded [1]:

- the waves action, which is manifested as a permanent seismic force,
- wind action,
- seism,
- the geological modifications of marine relief, etc.

On the other hand, it covers the theoretical incertitudines.

The reinforced concrete thin structures, even they are utilized from the *30*' years, until now they didn't find application in the marine domain.

The small trying of application of these structures for sheep's structures becomes possible because of the activity of the well-known Italian engineer Pier Luigi NERVI, in *1943*. The sheep's made by NERVI, had at the beginning *40* mm thickness for the walls, who later becomes *12* mm.

The application of the concrete in the sheep's structures domain, haven't known any development for reason of the technological difficulties and of the high dead load.

A reconsideration of these problems becomes possible since *1985* years, whit the extension and utilization of the high resistance and performance concrete and the discovery of the self-compacted concrete [2], [3].

In the context of this conceptions, in this paper the authors approach the utilization of thin concrete structures in the marine civil engineering domain.

The presented solution (being in the patents stade) are remarqued not only by what the realization of these structures suppose, whit a higher durability than the conventional structures, but they have as a strong point the simplification of in situ works, and the reduction of the execution time.

2. Coffer Dams

Dams are generally constructions executed from local materials, usually from clay soil.

Also, the existing soils for the execution of these works don't fulfill the impermeability asked conditions.

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As well, the execution of dams whit clay nucleus isn't always possible, because the lack of the appropriates material in the area, or because the high costs in the case of the clay soil transport from big distances.

The damages of the soil dams appears generally because the direct action of the water, by the slope degradation, or because the thining of the dams because the infiltrations or griffin, which antrenates the fine materials.

Face to solutions with retaining walls today are utilized also variants whit reinforced prefabricated concrete, which have a considerable thick due to the pushing of the soil and water (Fig 1).

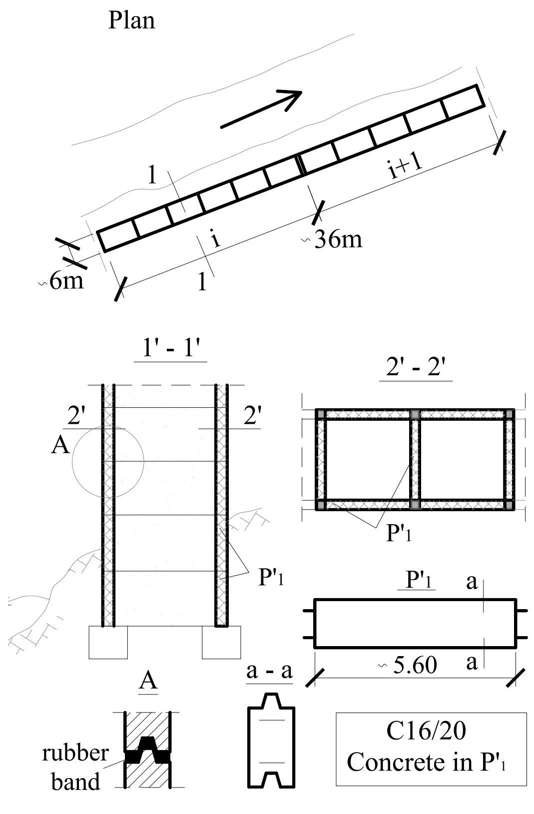


Fig.1

In figure 2 is presented a variants made of high performance concrete [3] – having thin walls (*10-12* cm) and a structural shape of lentil in plane.

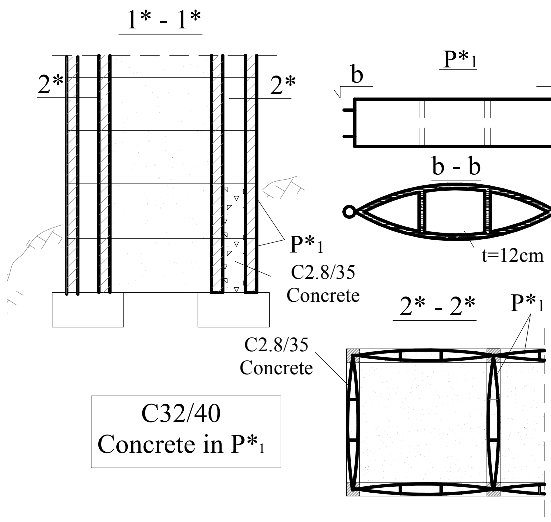


Fig.2

By this solution at least two structurales advantages are obtained:

- the reduction of the concrete consumption taking in account the big length of the dams.
- the easy montage for the reduction of the dead load of the elements.

The dimension of this structures depends only on transportation and monage consideration, because there dead load became insignificant higher whit the increase of the span.

The internal efforts have values whit few order condition smaller than the conventional structures, because of the spatial way of working.

3. Shore Arrangement

For shore arrangement usually vertical walls or dipping walls are utilized.

Figure 3 presents a new solving for the tubes protection.

The protection blocks realized in the shape of shell made of ferrocement, whit cells (*4-5*), are bring in situ by floating (Fig.4).

The montage and their provisory ancoration doesn't implies special utilages. By the introduction of the self compacting concrete in the cells, a controlated immersion of these elements is proceeded.

In figure 5 asimilar solving is given, for vertical protection blocks. The stability of the work is assumed by the weight of self compacted concrete.

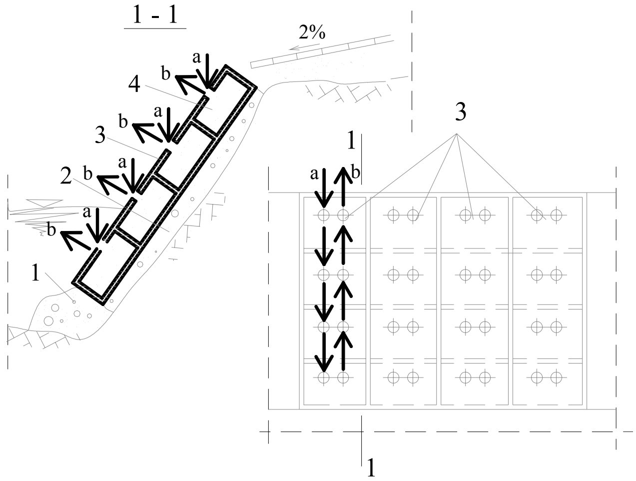


Fig.3

Legend:
1. Foundation
2. Egalization Concrete
3. Ferrocement Shell
4. Self Compacting Concrete
5. Hole for concrete introduction
6. Hole for air elimination

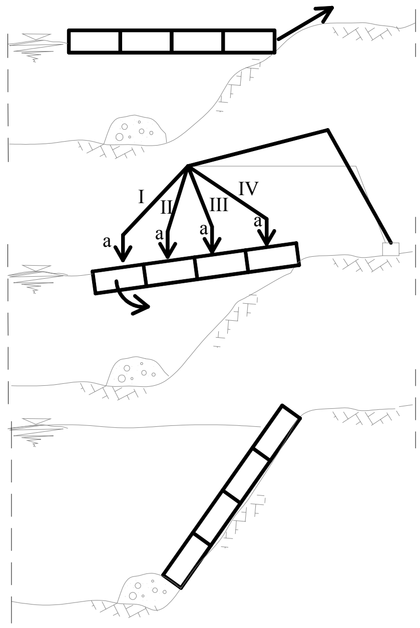


Fig.4

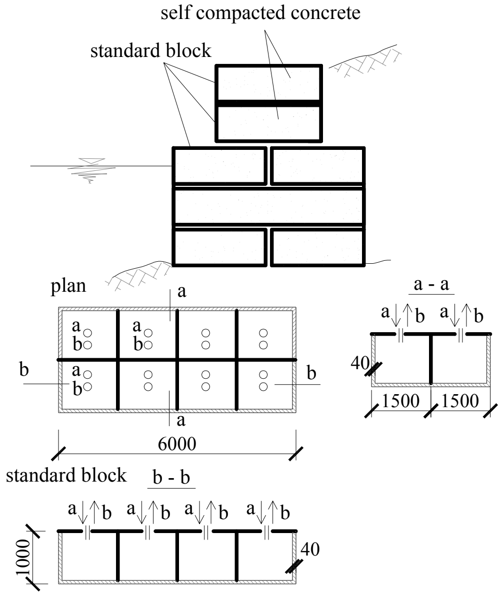


Fig.5

The solution from figure 6 is for permeable keys and dams, whit room of oscilation of the water level. The novelty of the solution is in the realization of the holes zone in the shape of prefabricated section whit conics tubs of dissipation.

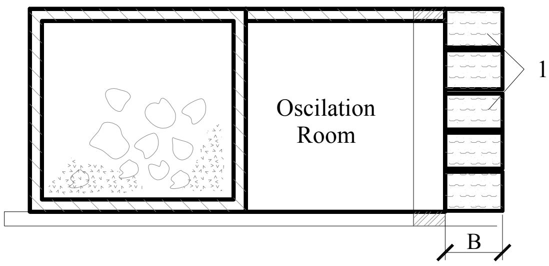
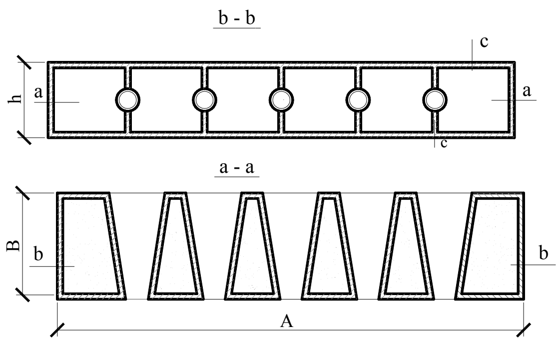


Fig.6

Legend:
1.Disipation block

The realization of the disipation block is prezented in figure 7.



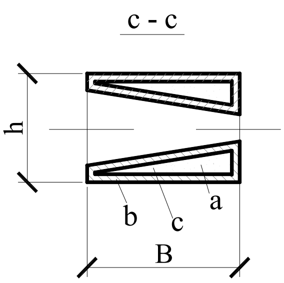


Fig.7

Legend:
a. Self Compacted Concrete
b. Ferrocement Shell
Ferrocement Conic Tub

4. The Protection Of The Pipes On The Sea Bottom

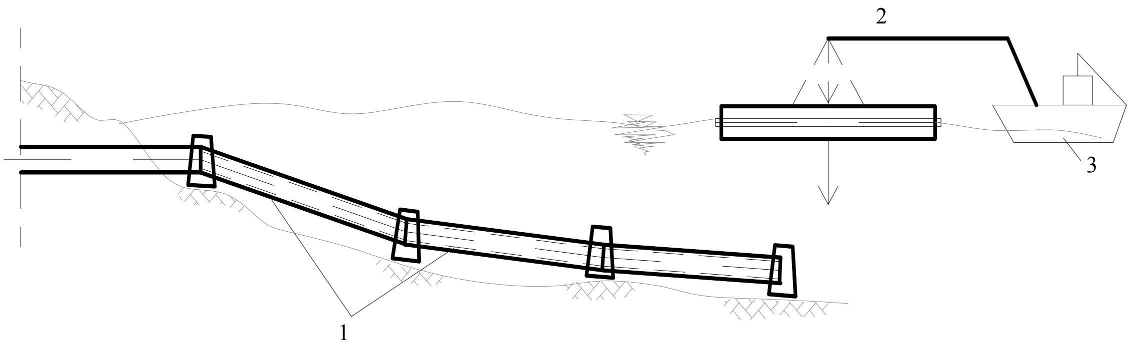
The pipes put on the sea bottom are exposed to waves solicitations, to shocks from the sheep's, to movements due to marine relief.

The problem of the protection of these pipes is usually solved by the flushing in trench whit the high of flushing in trench established by certainnes considerations as:

- the value of the Archimede's force for the full and empty pipe,
- possible likefiation of the sand,
- the value of the hydrodynamic forces from waves,
- the possibilities of changing of the marine relief,
- earthquake.

The technology of flushing of the pipes implies special utilages, as for digging as for ballasting and has high costs.

In figure 8 is presented a new solution for the protection and stabilization of the pipes on the sea bottom utilizing paralelepipedics elements, made in the shape of a ferrocement shell.



Legend:
1. Block with concrete protection for marine tub
2. Flushing pipe
3. Flotting instalation

Fig.8

Pipe realization for concrete protection in ferrocement shell is prezented in figure 9.

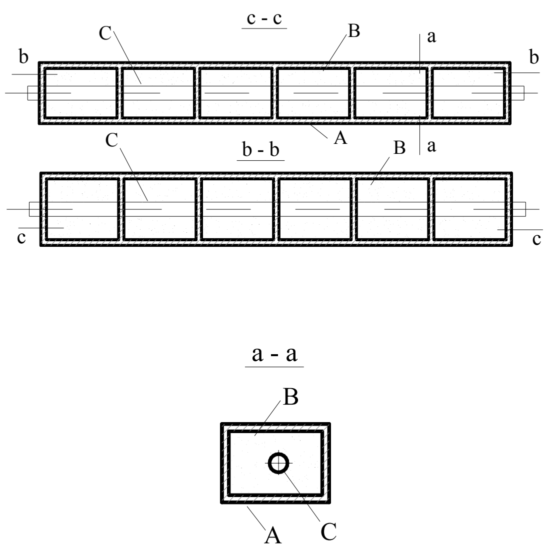


Fig.9

Legend:
A. Ferrocement Shell
B. Self Compacted Concrete
Marine pipe

5. Conclusions

In the paper the authors try to bring, by the new solutions presented, a small contribution in the reevaluation of marine structures with thin concrete elements.

The proposed solutions, eliminates the necessity of being of foreign licenses, whit high costs.

The conceives structures follow the revolutionary changing appeared in the concrete technology domain, by the current realization of the high resistance and performance-concrete, as well as the self compacting concrete.

Using high resistance concrete due the financial advantages, that derives from the reduce of concrete, steel, etc., apper special advantages on the structural line of quality. This fact is bounded to the dens structure of this material having a direct consequence to a special durability that reflects in

the costs bounded to maintenance no speaking of the ecological advantages that derives from here.

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New Terminals in Constantza Port

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Rezumat: Pentru a creşte atractivitatea portului Constanța este prevăzută realizarea de noi terminale, retehnologizarea celor existente şi creşterea vitezei de efectuare a tranzacțiilor comerciale. Astfel s-a realizat un nou terminal de containere şi se afla în diverse stadii de proiect cel de soia, cereale, bunkeraj şi pasageri. De asemenea, societățile de operare îşi modernizează fluxurile tehnologice. Se urmăreşte de asemenea creşterea vitezei de efectuare a tranzacțiilor comerciale prin oferirea utilizatorilor a unui centru de afaceri.

Noile terminale şi amenajări necesită un volum important de lucrări care vor conduce la îmbunătățirea serviciilor şi în final la creşterea atractivității portului Constanța.

Abstract: In order to increase the attractiveness of Constantza port, the construction of new terminals, the upgrading of the existing ones and the increase of the commercial transactions' speed are considered. Thus, a new container terminal has been realized and the terminals of soy, cereals, bunkering and passengers are in different stages of project. Also, the operating companies update their technological flows. The increase of the commercial transactions' speed will be achieved by a business center offered to users.

The new terminals and arrangements require an important volume of works that will lead to the improvement of services and in the end to the increase of Constantza port attractiveness.

Keywords: *Container terminal, terminals of soy, cereals, bunkering , passengers.*

1. General

In our country conditions, the development of naval transports is largely encouraged by the existence of more than 200 km littoral at the Black Sea and of the Danube. Through the Mediterranean Sea, the Black Sea is in connection with all the world's oceans. The Danube represents the Pan-European corridor no. 7 and is the main waterway for the Central Europe countries.

Taking into account that the technical and economical parameters of the water transport are in general better than the ones of the other transport means, it is foreseen that the percentage of the fluvial and maritime transports reported to the total transports volume will increase.

Correspondingly to the water transport activity, important constructions and port arrangements, as well as capacities for ships construction and repair, have been realized in the last decades.

The total traffic capacity in the Romanian ports at the Black Sea, on the Danube and waterways is of about 135 mil. tons/year. It is concentrated in 3 maritime ports, 4 fluvial – maritime ports, 29 ports on Danube and 5 ports on waterways (fig.1).

Regarding the activity developed in present in the Romanian ports, it comes out that, as a result of the essential changes in the country's economy that took place after 1990, the import of raw material and the export of processed products have considerably decreased. Consequently, the port traffic decreased, so that the capacities, the destination of port areas and the setting up of other capacities have to be re-considered.

2. New terminals

In perspective, the ports in our country will be mostly developed as zones of logistic activity, corresponding to the rhythm of the country's economy and to the new tendencies of activity diversification, in the context of Romania's integration in the E.U.

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The relations of transport on water and on land, the available depths and surfaces make easier the goods circulation between Central Europe countries and the ones from the Black Sea basin, from Middle and Far East. As a result, Constantza port became a center of storage, distribution and transport of goods.

The development and organization of new terminals that will lead to a more efficient port activity, has been considered.

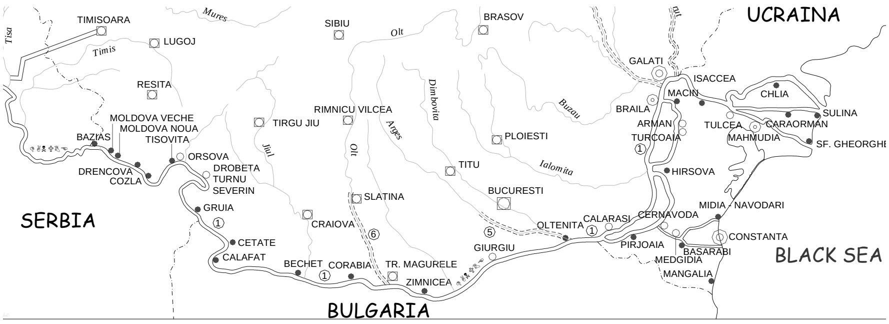

Fig. 1. Romanian river and sea ports

Thus, the new container terminal with an annual capacity of more than 300,000 TEU has been recently realized (fig.2).

Last generation shiptainers up to 5,000 TEU can berth here, and the railway, road and fluvial access is assured. The terminal has storage platforms and the afferent accesses, as well as freighting station, workshop, administrative building, access area, etc.

The surface arranged in this first stage is of about 100,000 m^2, and berth width -16,50m depth is 620m long. The terminal can be developed up to 800,000 TEU.

The main equipment consists of 3 quay gantry cranes, 9 rubber tyred gantry carnes , 2 rail mounted gantry cranes, forklifts of 38 t capacity, 30 trailers. About 240 persons will be involved in this activity.

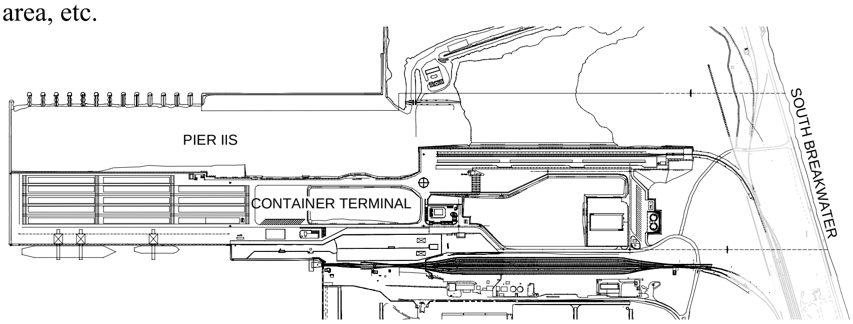

Fig. 2 New Container Terminal in Constantza port

A soy terminal is going to be realized in Constantza port, with the perspective also to process soy to better use the port territory surface.

The arranged surface will be of 50,000 s.q.m. In the first stage the terminal will have storehouses, silos, loading-unloading installations and the afferent infrastructure (fig. 3).

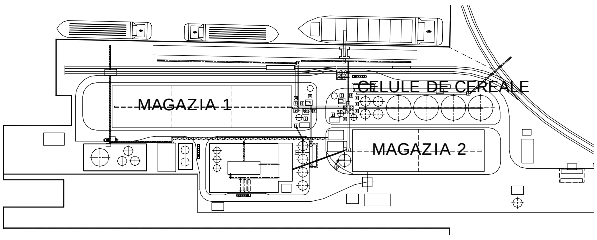

Fig. 3. Soy terminal

The terminal will be connected by a belt conveyor to the cereal terminal situated at 1000 m. This way, the flexibility of the cereal terminal will increase, using more efficient its silos of 10,000 t capacity together with the soy terminal's silos of 6,000 t.

The road and railway access will be assured by connection to the port ways.

The capacity of this terminal will be min. 200,000 t.

For the arrangement of a new cereal terminal that will take over the necessary traffic, pier III S is considered (fig. 4).

The cereal traffic will increase in Constantza port, reaching 7 mil. tons/year. In this purpose, a pier will be arranged and silos of about 4000 t will be constructed on the platform. These will be realized in stages and will have road and railway connections. Cereals' drying installations will be also necessary.

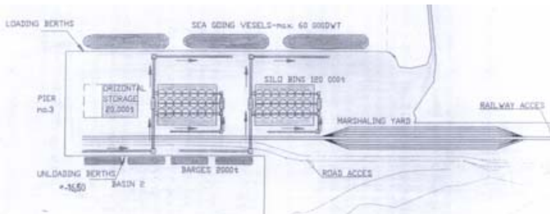

Fig. 4 New cereals terminal on pier IIIS

For ships' supply with fuel, a bunkering terminal will be arranged on about 3 hectares in the western zone of the port. The terminal will have fuel tanks of 1000 – 3000 t capacity, buildings and installations for delivery – reception at a specialized berth (fig.5).

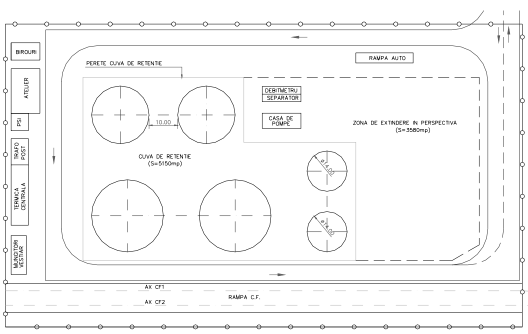

Fig. 5 Bunkering terminal

The service will be assured by ships' supply with a specialized ship, with road vehicles for small quantities, or directly, depending on the draught. The foreseen annual traffic will be of 150,000 t.

The nautical tourism for cruises will increase at the sea and a new passengers terminal will be constructed, including a building of 3,000 s.q.m. with all the necessary facilities (fig.6)

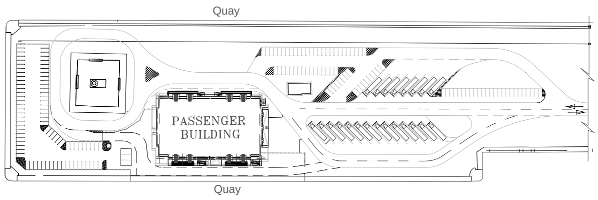

Fig. 6 Passengers terminal

Ships with draught of up to 12 m, each one transporting about 2,000 persons, will be able to accost at the berths. The arrangement of this terminal will be done at an updated berth of the old port.

3. Improvement of the operation technologies

The new terminals and arrangements will have productive equipment that will lead to the minimum time of ships' stationing in the port. The equipment with superannuated physic and moral state will be replaced with new ones

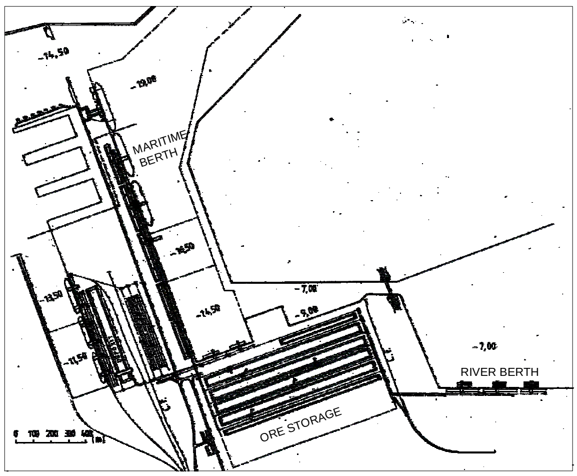

Fig. 7 Ore Terminals

For example, the company COMVEX SA, having as main activity the unloading of ore, coke, coal and bauxite ships, will be updated by the construction of a new conveyor, arrangements in the store and for the railway (fig.7)

At the berths with smaller activity, other activities will be also developed. Thus, at berth 83 of ore loading-unloading, operated by MINMETAL, liquid fertilizers will be also handled.

4. Financial aspects

The increase of the commercial transactions' speed will be had in view, by the contribution of the banks that will lease buildings in ports. Thus, in Constantza port a business center will be set up in the area of the Port Administration (fig.8). It will include functions offered to the users, information center, logistic support services, and headquarters for the firms involved in the port activity.

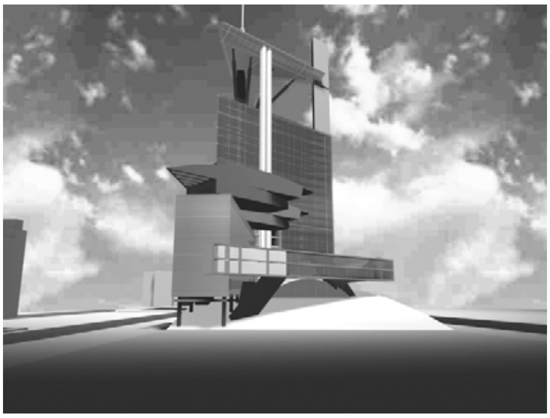

Fig. 5 Business center

5. Impact on the environment

All the activities in the port will be developed in the conditions of an adequate environment protection. The exploitation will be organized, execution and maintenance works will be executed and water supply and sewerage installations will be realized. Measures for waste collection, including from ships, and their treatment, will be also taken. The port will be equipped with special installations and type DEPOL ships for the better environment protection.

6. Constructions necessary for the new terminals

The achievement of the mentioned terminals requires a special preoccupation regarding the systematization, for their integration in the port ensemble, as location and accesses. It is also necessary to conceive an adequate technology, which respond to the imposed specific requirements. Further on, the spaces for storage, the buildings, the platforms, the installations, the road and railway accesses and the afferent berth have to be designed and executed.

Site studies, respective topographic and bathimetric surveys and geotechnical study, have to be previously performed.

7. Conclusions

A port terminal is a complex work, both by its role and by the constructions that have to be realized. If it is necessary, a possible extension has to be considered. In general, the principle “the port wait for the ship” has to be applied, in order to result exploitation expenses as small as possible on the whole. The new terminals and arrangements will lead to the increase of the competitiveness in the port, to the improvement of the services and in the end to the increase of Constantza port attractiveness.

Practical Structural Dynamics of Marine Cables

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Rezumat: În cablurile marine apar vibrații cu amplitudini foarte mari. Aceste vibrații, pe lângă faptul că produc o stare de nesiguranță, adesea reprezintă şi cauze ale unor avarii grave ce se produc datorită ruperii cablurilor din solicitarea la oboseală. În lucrare autorii propun o metodă practică de analiză dinamică a structurilor marine cu cabluri. Pentru combaterea vibrațiilor exagerate sunt prezentate soluții prectice, care utilizează amortizoarele cu acțiune punctiformă. Lucrarea prezintă şi un procedeu pentru montajul structurilor marine cu cabluri utilizând teoria vibrațiilor.

Abstract: Most of the times in the marine cables appears vibration with high amplitudes.These vibrations, beyond the fact that they produce an incertitude state, frequently represents the causes of serious damages that are caused thanks to the fact that they are tear to pieces because of their great solicitation in time.In this paper the authors propose a practical method of analytical dynamic in marine cables structure. In order to struggle these exaggerate vibrations there are presented practical solutions, that use punctual damper in action.. The paper presents also a proceeding for the marine structure with cable mounting using the vibration theory.

Keywords: *marine structure, marine cable, pendulary platform, floatable platform.*

1. Introduction

It is known that in marine structures (Fig.1-4) appear important vibrations from the waves and current action, solicitations that are manifesting like a permanent earthquake [1], [2], [3].

In these phenomenon context, we find out that in cables and submarine pipes cases assimilated with cables, the vibration amplitudes are with some higher decree size than structural vibration amplitude [4], [5].

World's studies were started after the apparition of some serious problems at the offshore platforms in:

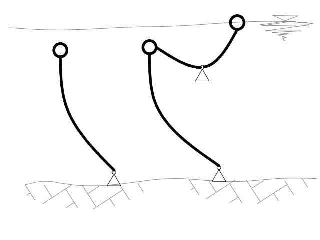
- IMMINGHAM, Great Britain, 1969,
- COGNAC, Mexico Golf, 1980,
- MERCURY, the Channel Le MAN, 1979-1981,where because of the great amplitude vibrations took place give disposals because of the tiredness solicitation (although the axial forces were representing almost 30-35% from the axial breaking off).

In general, marine currents cause these cable vibrations and also by the base structural vibrations. Through the base structural vibration the cable's catching points are setting in action. At these displaces of the fixing points, the cables are reacting sensitively [6], [7].

From these considerations, in this paper is presenting the practical dynamic analyses, in order to intercept these kind of phenomenon, their disapprove and a practical procedure for the marine cable's montage using vibration theory.

Suspended submarine structures

a. Monocable



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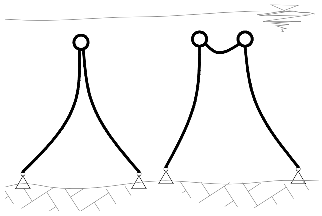
b. Double cable



c. Triple cable

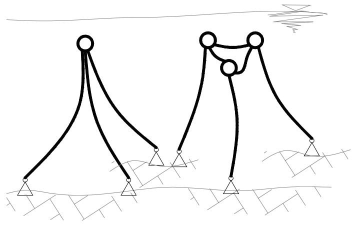


Fig.1

Pendulary Offshore platform suspended with cables.

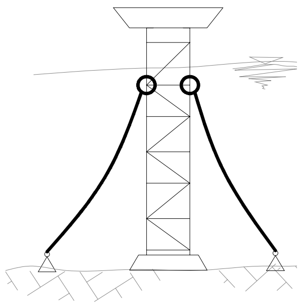
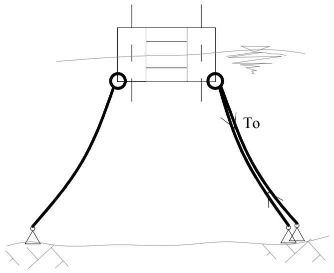


Fig.2

Floatable Offshore platforms

a. With inclined cables



b. With verical cables

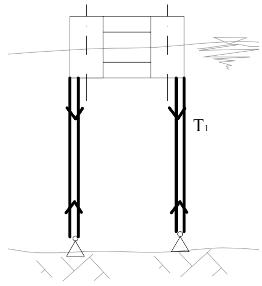
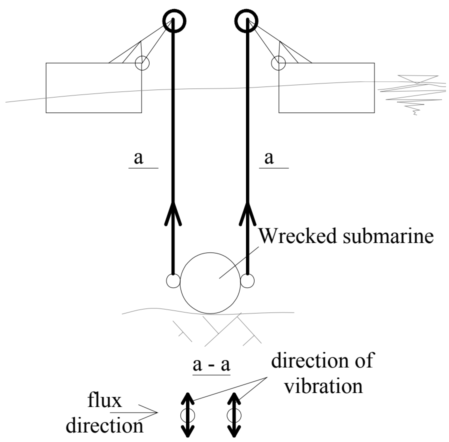


Fig.3

Uplift Cables



2. Practical Dynamic Analyses

In order to follow in time the phenomenon movement it can be used instead of d'Alembert principal, Hamilton principal thus

$$\int_{t_1}^{t_2} (\delta E + \delta L_e + \delta L_i)dt = 0 \tag{1}$$

where:

-E is the kinetic energy,
-L_e is mechanical work of exterior forces,
-L_i is mechanical work of interior forces.

The displacements, kinetic energy, mechanical work of exterior and interior forces can be expressed depending on generalization coordinates.

If in the equation [1] there are supposed accomplished the conditions for the transition in an external form and it is introduced Lagrange's function, $L^t = E - \prod_i$, where $\prod_i$ is the internal elastic potential, then the Hamilton's principal is written under the form

$$\int_{t_1}^{t_2} \delta L' dt + \int_{t_1}^{t_2} (\delta L_e + \delta \Delta L_i) \mathrm{d}t = 0 \qquad (2)$$

in which

$$\delta \Delta L_i = \delta L_i - \delta \prod_i \qquad (3)$$

If it is note with P_i the exterior generalized forces, with D_i amortization generalized forces and with Q_i generalized displacement, than:

$$\delta L_e = P_i \delta q_i \qquad (4)$$

$$\delta \Delta L_i = D_i \delta q_i \qquad (5)$$

After the equation displacement [3], [4], [5] in equation [2] and using the variation calculation, there are obtained movement equations of Lagrange:

$$\frac{d}{dt} = \left(\frac{\delta L'}{\nu \dot{q}_i} \right) + \frac{\delta L'}{\nu q_i} = P_i + D_i \quad (i = 1, n) \qquad (6)$$

Lagrange's equation [6], in studies of unlined dynamic problems are forming a system of unlined difference equations on degree 2.

Linearization can be made with incremental techniques, replacing the initial unlined problem with a succession of lined problems.

Although, the equation system differentiation with partial deriving decree 2, obtained after all due to the incremental formulation, is lined, the exact solution it is possible only in particular cases.

Discreet modeling or numerical, eliminate these inconvenience, being practically possible the solutions of certain configuration structures.

The calculation can be lead using finite element method.

The cables are considered made of an ensemble of unidimentional straight finite elements, without the rigidity at inflection, having straight to the nodule three degrees of liberty. The relation effort – deformation is considered to be lined.

The basic structure can be moderate with double bars as well as with finite elements of isoperimetric membrane type.

Double articulated bars are moderating also with finite element of cable, but with the possibility

To assume compression forces also. In order to analyze dynamic stability and the values solicitation are very important amortization properties taking into account, as well for cables as for the base structure.

Through amortization it is consumed energy having as a direct consequence the diminution of the amplitudes. It is well known that at the rigid steel structures and at the cables structural amortization is less, so significant remains the hydrodynamic amortization.

As a measure of amortization it is used logarithm decrement

$$\delta = ln \frac{A_i}{A_{i+1}} \qquad (7)$$

where A_i and A_{i+1} are two successive amplitudes.

If amortization is little, than between logarithm decrement δ and the amortization ratio λ can be written as a relation of this form

$$\lambda = \frac{\delta}{2\pi} \qquad (8)$$

Taking into account that oscillations are made in marine medium, beside of the proper inertia of cables structure interfere also the water inertia that is surrounding the cables.

So it is forming a pressures oppose to acceleration. These pressure forces can be considerate as a product between a mass and masses acceleration.

The supplementary mass we'll call it hydrodynamic mass. It isn't the mass of a certain physic margin, but a formal model of calculation. In this way the pressure due to acceleration is

$$P_{ai} = m * a_i \quad (i = 1, 2, 3) \qquad (9)$$

where :

-m^* is hydrodynamic mass,

-a_i is the acceleration after global direction.

Hydrodynamic mass depends of the size and form of the pendulous corps and also of the water density. For practical calculations, in the case of the cable structure, hydrodynamic mass can be considerate

$$m^* \cong \frac{1}{3} D^3 \rho_a \tag{10}$$

where:

-D is cable diameter,

- ρ_a is water density.

For the base structure, with rigidity at inflection, hydrodynamic mass can be expressed alike.

$$m^* = C_H D^3 \rho_a \tag{11}$$

where:

-C_H is a constant that depends of the corps shape (it is determinate, experimental on models),

-D is the imaginary circle diameter in that which it can be enroll the oscillate corps.

Experimental it was observed that amortization in marine medium is proportional with the ratio between hydrodynamic mass and oscillation mass of the system, thus it can be written

$$\delta = \frac{1}{2} c_a \frac{m^*}{m} \tag{12}$$

with $\boldsymbol{c_a}$ representing a determine coefficient through the integration of pressure curbs and amortization.

In practice the calculation can be realized with the automatic calculation program SUM 01 [9], [10].

In the program, the actual variant, it is considered only the elastic lined behavior of the material. In order to resolve balance problems it is used iteration of NEWTON –RAPHSON types, that are independent of the type of finite element used.

The integration of the moving equations are made through the NEWMARK and WILSON methods.

The structure of the program utilizes the operating idea with a unique vector and the data transmissions through common blocks. The necessary memory depends of the size of this vector.

Program testing was made with the example assumed from the [12] paper and has as a purpose the check up on unlined free vibration with or without amortization (Fig.5).

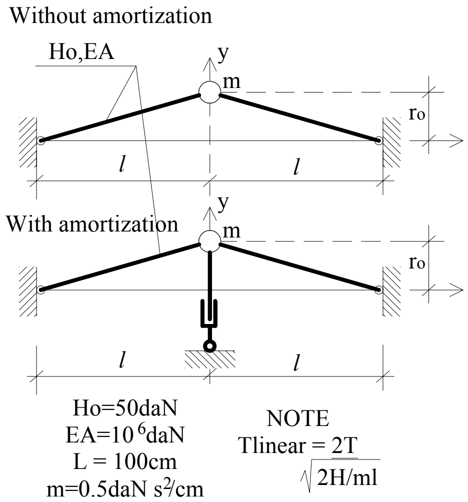
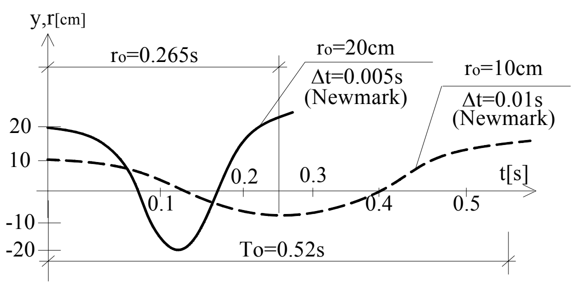


Fig.5

For the calculation it was utilized NEWMARK's as well as WILSON's integration procedure.



In figure 6 we observe a big difference between the period calculated with formula from lined domain (T = 4.456s) and that calculated with the SUM program in lined domain.

Also it can be seen that in order to realize a better precision, time step must be taken very small

[$(\frac{1}{25} \div \frac{1}{100})\, T_o$, being the approximate value of the fundamental period].

So, for the 0.005*s* ($\approx \frac{T_o}{53}$) time step it was obtained a perfect correspondence with the published results in paper [12].

Taking into consideration the vibration dispute with exaggerated amplitudes it is proposed the stipulation of same shock absorbers with a punctual action (Fig.7, 8).

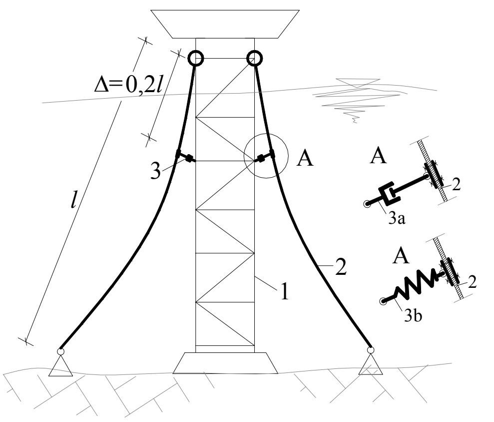


Fig.7

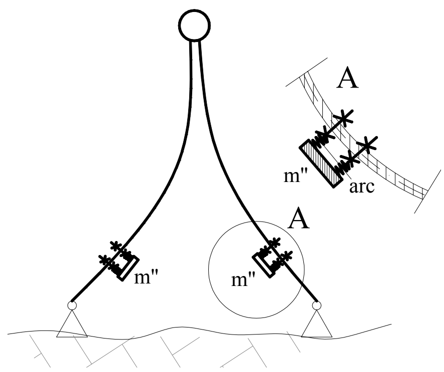


Fig.8

For suspended submarine structures, where doesn’t exist fixe points, it is recommendable to utilize anti shocks with or without absorbent (Fig.9).

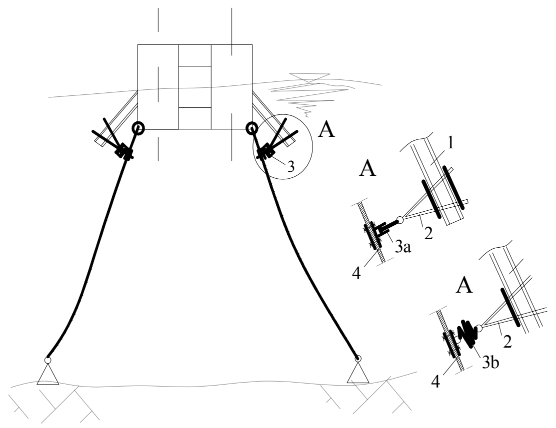


Fig.9

3. Utilization Of Structural Dynamics For The Settlement Of Marine Cables Geometry [13], [14]

Respecting the portant cables geometry at the marine structures is a fundamental problem of the execution, taking into account that the tension in the cable is tight up directly with it’s arrow. On the other hand the forces with what the anchors are also tight up with the arrow of the bearer cable.

The proposed method, used by the authors at the montage of suspended bridges, can be easily used even in the case of marine cables [14].

If it is considered the charged only with its own weight (Fig.10) and with the provoked vibration state by a dissipate force of type viscous, it can be written

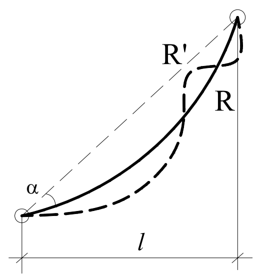


Fig.10

$$H_o = \frac{4WS_o^2 n_o^2}{gt^2 \cos^2 \alpha} \qquad (13)$$

where (Fig.10) were noted with:

- α - the angle between the horizontal and the line that connects the fixing points of the cable,
- H_o - the horizontal projection of the effort inside the cable,
- W - the weight on line meter for the cable,
- g - the gravitation acceleration,
- S_o - the cable's length,
- n_o - oscillation's number measured in the time interval t,
- t - the time period considerate for the oscillation measure.

In this way, if it is provoked o local perturbation of the cable and it is measured with a detector of the oscillation numbers in a interval of time (usually 1 minute), then through the relation [13] we have geometry control.

4. Conclusions

This paper proposes a practical methodology of dynamic analyses of marine structures with cables.

Beyond the base theoretic aspects there are presented also measures of disapproval of these vibrations using shock absorbents with arc and waterworks with punctiform action.

An interesting aspect presented in the paper is referring at the utilization of vibration theory for the marine structure with cables.

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Effectiveness of Hard and Soft Coastal Protection Solutions Implemented on the Southern Romanian Shore

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Rezumat: În sudul României în zona de coastă, între Capul Midia şi Vama Veche (la 75 Km de granița cu Bulgaria) este, în general, un țărm cu faleze şi refief neregulat datorat plăcii calcaroase Sarmațiane.

În județul Constanța, țărmul reprezintă 41,45 % din litoral, 14,63 % ocupat de porturi şi 26,82 % lucrări de protecție a coastei. Lucrările hidrotehnice realizate au urmărit protecția țărmului atât în zona falezelor cât şi a plajelor şi lagunelor. Protecția plajei a fost realizată prin intermediul implementării soluțiilor de apărarea a costei (diguri de larg etc). În această problemă, în ultimii 55 de ani s-au executat lucrări de apărare a coastei, începând cu perioada 1936 – 1940 şi continuând cu stadiile de dezvoltare (1956 – 1960, 1967 – 1970, 1981 – 1985 şi 1989 – 1990) până în 1991, când această activitate s-a diminuat puternic. Deasemenea, zona țărmului acestui județ este puternic influențată de lucrările hidrotehnice interioare şi de cele din larg, care au fost implementate într-un timp foarte scurt, astfel fiind aproape imposibilă o cerectare ştiințifică foarte atentă conform necesităților şi situațiilor concrete din această zonă. Datorită acestor motive, soluțiile de protecție implementate au răspuns necesităților într-o oarecare măsură. Astfel odată implementate lucrările pentru protecția plajelor, s-au inițiat cercetări detaliate pentru cuantificarea efectelor acestor lucrări, impunându-se câteva corecții pentru optimizare.

Abstract: The Southern unit of the Romanian coastal zone, between Midia headline and Vama Veche (at the Bulgarian border - about 75 Km long) is generally composed of relatively high shore with cliffs and irregular bottom relief due to the hard Sarmatian limestone plate.

In Constanta County, the shore in "built" regime represents 41,45 % of the shoreline, 14,63 % for ports, and 26,82 % for coastal protection works. Hydrotehnical works realized were follow the protection of high shore with cliffs as well as low shore, with beaches and lagoons. The protection of beach was realized through implementation of hard (breakwaters, groins…) and soft (beach fill, read fences…) coastal defense solutions. In this matter, at the last 55th years were executed coastal defense works starting from 1936 – 1940 period and continuing on development stages (in 1956 – 1960, 1967 – 1970, 1981 – 1985 and 1989 – 1990) till 1991, when this activity were largely diminished. Also, the shore area of this county is strongly influenced by inland hydro-technical works, off-shore and shore-connected breakwaters, which at the time of the implementation were required at very short deadlines, thus making almost impossible the grounding research, according to the necessities and the concrete situations in the area. From these reasons, the protection systems implemented responded to the forecasts in different proportions. Thus, once the coastal works were implemented for beach protection, it has initiated detailed research for the quantification of coastal protection systems effects, entailing some corrections for optimisation.

Keywords: *coastal structures, erosion, accretion, vulnerable areas.*

1. Introduction

The evolution of each shore sector is the result of the balance between losses and supplies of sedimentary material. In the last decades, this balance was negative for large sectors of the Romanian Black Sea shore, as a consequence of new conditions distressing the pre-existent natural environment.

Between the impacts of human interventions which occur in the littoral zone, the coastal engineering

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aspects take a special place, for their socio-economics implications, as well as for their impact on the coastal ecosystem. Thus, due to the extension of the navigation jetties (Sulina Branch jetties seaward prolonged more than 9 km), and port breakwaters having as effect the deviation of

coastal sediments drift through distancing/separation of their discharging point in the sea, together with the sequential development of the coastal protection construction (groins, etc.), the shoreline supports an important pressure, which it determines more or less accentuated its dynamics.

Under these disturbing factors during the last years the Romanian shore was strongly affected; the elaborated studies shown that the erosion process was extended on about 60-70% of shore length, getting a critical character in several sectors.

For the temporary correction of erosional process, the most frequently adopted protection solutions were the hard type once, implemented within complex arrangement schemes of protection systems along the Romanian littoral.

Erosion processes affect the Southern area of the Romanian littoral, but the rates of coastline regression do not reach the same values as within the category applied for Northern unit, which consists of sandy complex barrier beaches with strong longshore sediment drift systems. The effects in this case are much smaller, the environmental transformations are smaller, the economic losses should be much lower, except for the zones with very intense touristic activity. The effect of this misbalance added to the rise of the sea level brings about a very active erosion process of the coastline. Anthropogenic pressure of different littoral structures is also very important strengthening the destructive processes along the shoreline.

The design of coastal protection works was pursued the segmentation in alveolar unities of southern Romanian shore by natural and artificial formations, formed of natural promontories and hydrotehnical constructions of protection, in order to block the longshore sediment transport. As a result, the beaches of this sector are relative stable, and shown in the last years a small trend of increase of the surfaces. Between 1990-2003, in the southern built shore, a change in the shoreline was recorded with a magnitude between +16m and -24 m, in which the erosive effect had a frequency of circa 76%, which shows the fact that the hard protection solutions didn't reach their initial goal. Also, because of economic considerations, the existent coastal structures are every so often extremely damaged. From this reason, now it is crucial to be outlined a beach management plan, focused on adequate use and maintenance of the effective coastal structures coupled with beach nourishments, based on a synthetic revisiting of hard and soft henceforth-implemented solutions for coastal protection and rehabilitation.

In this regard, the existent hard coastal protection schemes, has been coupled with soft protection works (artificial beach nourishment, artificial plantations for stabilization of sediments dunes, temporary revetments for the protection of vulnerable sectors against destructive action of severe storm waves, and the settlement of reed fences for protection against deflation).

2. Material and methods

To achieve the shoreline evolution assessment, the emerged beach modifications of the southern Romanian shore sector between Eforie and Mangalia, in the last decade, were considered. The changing trends were determined on measurements basis of about 50 sections of emerged protected beaches surveillance. The determination of coastal processes magnitude was realized by grouping of sea-land interface variation rates/rhythms, in the following classes of intensity and evolution sense: SE – strong erosion, ME – medium erosion, LE – low erosion, RS - relative stability, LA – low accretion, MA – medium accretion, SA – strong accretion.

The post-monitoring of hydrotehnical works for coastal protection is an essential element in the exploitation, maintenance and performance evaluation of these structures. The periodic inspection or post-exceptional event (earthquake, storms…), permits the design and execution deficiencies, which can be used to the future optimizations and maintenance coast reduction.

The evaluation of the coastal structures effectiveness takes in account the functional aspect of construction, which is related to fulfilling of it purpose (waves energy reduction, or/and sediment transport blockage…), there are more or less included in obtaining of desired geomorphological effects in the settlement areas, or respectively, in its erosion control, and also the aspect of structure performance to the waves and currents action, which is related to preservation of constructive characteristics at designed parameters, with the conservation of the structure stability/safety without degradation and material losing.

The evaluation criterions of the structures degradation stages, were establish in function of the structural degradation deepness, in the emerged and submerged part of structure, related to multiple deterioration ways/mechanisms. Thus, it was done a classification of structure deterioration in tree stages of deterioration: good stage (0-30% damages), medium stage of deterioration (30-60% damages), accentuated stage of deterioration (60% damages to structure failure).

3. Results and discutions

3.1. Context of coastal changes occurring along the Romanian Black Sea shore

The 244 km length of the Romanian shore represents 6% of the total length of the Black Sea shore. The relief of this shore consist in low shore, beaches: (80%) and relative high shore: cliffs (20%). From the geographical point of view, it includes natural shores (beaches and cliffs - about 84%) as well as “built” shores, about 16% (ports, coastal protection constructions).

The geomorphological characteristics zones of the southern Romanian shore, extended from Cap Midia to Vama Veche (Bulgarian border), with an appreciatively length of 74 km, are specific to a relative high shore, with cliffs of 35 m maximum high, mainly active. The beaches formed at the cliffs basis are relative stable and small dimensions. This sector, having different structural and morphological relief compared with Northern Sector, is mainly affected by the marine abrasion. Due to its geological structure, especially of the hard substratum of the calcareous plate, and to specific hydro-meteorological conditions as well, the shore had suffered intense irreversible modifications in several sectors. Also, the beach of « Eforie Sud » International Camp retreated with more than 40 m (1981 - 1992), the northern part of Neptun beach with 24 m (1981 - 1992), and Venus – Saturn beach with 36 m (1983 - 1992). Although a large part of this sector was protected by hydrotecnical constructions, these had not the expected effect in the shore stabilization. The redundant implementation of these protection systems was started between 1936 and 1940 and continued gradually, by socio-economical development stages, (between 1956 – 1960, 1967 – 1970, 1981 – 1985 and 1989 – 1990) until 1991, when such shore protection, maintenance activities and coastal development works considerably diminished.

Thus, along the southern littoral, in a period of about 50 years, closely related to the development of land improvement works and the extension of navigation works, followed by progressive intensification of erosion processes, some coastal protection systems were designed and executed. All these shore protection and coastal development works cumulating about 13,5 Km, represented 49 shore-connected breakwaters, 24 longitudinal breakwaters (16 of them submerged), and more than 17,5 Km seawalls and revetments. Also, about 14 Km of cliff were consolidated

Between 1981 and 2002, the evaluation of southern coastal processes (erosion/stability/accretion) accomplished through assembling the sea-land interface rates/rhythms of modification in intensity and evolution sense classes [VSE - very strong erosion (<-35 m), SE - strong erosion (-35÷-25,1m), ME - medium erosion (-25÷-15,1m), LE – low erosion (-15÷-5,1m), RS - relative stability (-5÷5 m), LA – low accretion (5,1÷15m), MA – medium accretion (15,1÷25m), SA – strong accretion (25,1÷35m), VSA – very strong accretion (>35m)] had shown a more equilibrated situation of beach evolution in this zone.

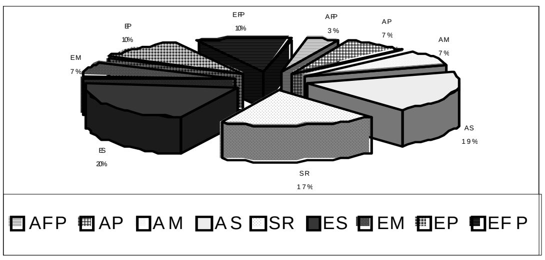

Fig.no.1. Predominance of littoral erosive processes in the shore sector
Mamaia – Vama Veche (1981 – 2002)

Due to erosive process intensification, the execution of certain protection measures was required at very short deadlines and a necessary fundamental research was almost impossible, according to the necessities and the existent concrete situations in the areas. The temporal behaviour and morphological effects over the nearest/ adjacent zones responded to the forecasts in different proportions, and the acquired results after detailed research for the quantification of induced geomorphologic modifications, entailed some corrections for optimisation.

For the limitation of erosional effects several surveillance, modelling, design implementation and monitoring actions of the hydrotechnical structures for coastal protection were gradual undertaken.

Starting with 2000, recent actions, carried on with Environment Ministry support, were included in the development of the coastal erosion survey programme; it consists in data collecting and stocking, through the performing of specific measurements, in order to realize a complex date base, as an informational support with the view of the coastal protection and rehabilitation solutions design and implementation.

Therefore, taking into account the need for the protection of the built shore, as to both natural and anthropogenic factors, several coastal protection measures can be used.

Present worldwide applicable trends for shore protection are non-intervening ones; the action is valid for short time and limited to monitoring, and the implementation of the "hard" (structural) and "soft" (non-structural) solutions, on short and medium terms. In practice, many coastal protection systems incorporate aspects of both sorts of solutions.

For the northern, as well as for southern Romanian littoral a coastal management in agreement with coastal dynamics is encouraged, much more than a traditional/classical one (which would fight against sea action); thus the soft solutions, which use the natural processes in coastal protection, are very proper; among these, the artificial nourishment of beaches is a largely used method, in the conditions of the proper localized sedimentary sources, lacking premise at the Romanian littoral.

3.2. The evaluation of the shore changes in protected sectors

The present state of the developed southern shore is reflected by the shoreline modification, determined by annual measurements.

Cape Singol – Cape Constanta sector

The entire sector was adjusted with hard shore protection structures, in several stages, starting with interwar period 1936 – 1940 until 1990. The shore segmentation in several alveolar units, was gradually achieved, delimitate by mixed formations, consisting of natural promontories and hydrotecnical protection constructions. To increase the sand retaining capacity of the adopted scheme, alveoli were protected, in concave zones, by permeable longitudinal breakwaters, up to about circa 300 m seaward. These breakwaters became submerged, through subsidence with time. At present, these beaches are relatively stabile, with a small accumulation trend. The groins settled in this sector are 37,5 % in good stage and 62.5% in medium and accentuated stage of degradation.

Eforie Nord – Eforie Sud sector

This sector is characterized by the presence of protected beaches at the basis of consolidated and embanked cliffs. Between 1951 and 1960, five groins for beach extension were located, taking in consideration the absence of systematic measurements and observations, and that longitudinal sediment transport is intense enough for filling assurance. Later on, once with Constantza South port extension, new protection measures were enforced, determined by the intensification of erosive processes, concretised in the execution of submerged permeable longitudinal breakwaters (1985–1986) in the supplement of the groins. These groins together with a harbour for small tourism boats, realized in front of Belona Hotel, induced a big accumulation of massive sedimentary deposits. The groins of the sector are in all tree considered degradation stages, in equal proportion; also here existing the most affected structures of the littoral.

The beaches from Eforie Nord perimeter maintained by the gradual extension of a protection system registered normal/equilibrated changes in the last geomorphologic cycle (2002-2003). As to 1990, the beach width had a retreat of 85% from all supervised sections, with a maximum of –14 m (IRCM 19 - southern Acapulco).

Eforie Sud p*erimeter*

The distance to the water-face, comparatively with 1990, indicates a moderate erosion stage, with shoreline retreats between –8,7 and –0,6m, even if, between 2002 and 2003 was registered a trend of beach surfaces recovering, with advancements of the shoreline, between 1,2 ÷ 8,3 m/year. Due to groins degradation a decrease of beach quality was remarked, through the increase of the number and the dimensions of shell and stone deposits, which in the spring covers about 65 % of beach surface.

Olimp – Jupiter sector

This sector has as main characteristics, as well as the entire shore zone south of Constantza, due to its physical and geographical structure, namely a discontinuous sedimentary transport. Comparatively with 1990, the shoreline registered a moderately generalised retreat. Also,

Olimp beach was settled by artificial filling, protected by four groins. The beach there is in a relative stability stage, but comparatively with 1990, all alveolar/pocket beaches are 60% retreated. Also, the beach had disappeared in southern areas, for over 70 m lengths, the disposal of a rubble-mound revetment being required. In this zone suspended artificial beach surfaces can be developed.

Jupiter beach, created in the same time with the resort by the construction of five groins, is at present together with Cape-Aurora beach, one of the poorest beaches, and also its groins are extremely damaged. In this zone suspended artificial beach surfaces can be moreover developed.

Photo. No. 1 Top of groin collapsed
(groin 3, Jupiter – 08.08.2002)

Aurora – Venus sector.

Aurora beach On this beach, the artificial nourishments are yearly realized in the spring, in order to compensate the sediment deficit from the could season. Comparatively with 1990, the shoreline retreat was of 5,4 to 11,5 m, which enforces a more careful maintenance and exploitation.

Venus beach is a relative stable beach. The beach width decreases southward and disappears on

circa 300 m, between groins settled in front of Lidia and Silvia Hotels (where a rubble-mound revetment was disposed, and a seawall against wave attack).

Saturn s*ector* is situated on a natural promontory, protected by five groins, and presents a relative geomorphologic stability.

Mangalia s*ector* was recent developed, between 1989 and 1990 and in 2002, with four groins, disposed on a calcareous platform and under strong present erosion. In the northern part of first two alveols there is no beach in front of a seawall. Due to erosion a about 100 m length rubble-mound revetment of lengths blocked retreat/extension shoreline processes. The multi-annual rates of beach modification were reduced. The afferent groins are in good conservation stage, due to the singular actions for the structures maintenance occurred in all littoral.

4. Conclusions

In the southern sector with beaches protected by groins between 1990 and 2004 shoreline modifications were remarked with changing magnitudes embraced between + 16m (IRCM 31 north Olimp) and -24m (northern residential zone of Neptun), in which the erosive effect prevailed about 70%; this reflects a partial efficiency of the protection solution implemented in this sector. Also, due to absence of maintenance actions, from the constructive point of view, the groins state emphasized 45 % good stage, 25 % medium stage of altering and 30 % accentuated stage of deterioration.

In near future, the adoption and the optimisation of the conservation-rehabilitation measures for the southern sector will have to include the better management of local transported sandy sediments, recovering of beach surfaces, with sedimentary deficit, from Eforie, Olimp – Mangalia tourism resorts (the marine or land sedimentary sources for artificial nourishment of these beaches have to be identified), a better protection of emerged beaches in the could season, and also the consolidation and the improvement of efficient coastal protection constructions schemes.

Further on, several maintenance and optimisation actions will be required at mostly hard structures for the coastal protection. Also, for the growth of tourism capacity through increasing of beach surfaces, the solution of artificial suspended beach construction can be considered, on alignments where the beach was extremely retreated due to local geo-hydrodynamic conditions, and geographical features respectively.

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The Prognosis Evolution of the Riverbanks Morphology and the Elaboration of the Solutions for Preventing the Silting of the Storage Lake Chirita

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Rezumat: Lucrarea îşi propune să analizeze zona de contact dintre pârâul Şapte Oameni şi lacul de acumulare Chiriţa din bazinul hidrografic al râului Bahlui. Acest lac este utilizat pentru stocarea apei transportate prin conducta de aducţiune din râul Prut fiind apoi preluată de staţia de tratare Dancu şi distribuită utilizatorilor de apă din oraşul Iaşi. Protecţia acestui lac împotriva fenomenelor de reducere a capacităţii utile este foarte importantă fiind tratată problematica transportului de sedimente în lac din bazinul aferent prin aspecte legate de prognoză şi masuri de prevenite a colmatării lacului.

Abstract: This work presents the situation of the contact zone between Şapte Oameni brook and storage lake Chiriţa placed in the basin river Bahlui. This storage lake is used like reservoir for the transported water by the supply network from Prut river beeing treated in the treatment Station Dancu and distributed for drinking and industrial water utilisator of Iaşi city. The protection of this storage lake is very important regarding the sillting processes which can induce the reduction of the available storage. Are treated the prognosis and preventing measures about the sillting process of the storage lake Chiriţa.

Keywords: storage lake, water supply, river morphology, sillting proces, available storage, preventing measures.

1. Introduction

The storage lake Chiriţa is placed in the basin river Bahlui and is used like buffer-reservoir for the transported water by the supply network from Prut river. The protection of this storage lake is very important regarding the silting processes, which can induce the reduction of the available storage.

2. The Prognosis of the Silting and Erosion of the Bank River using the Integration Polynom Lagrange

The prognosis of the morphological changes of a river cross-section may be realised using the procedures of integration of function by interpolation.

The most comune formula for interpolation is the Lagrange function. We purpose that the interval [a,b] are specified n values of the argument, $x_1,x_2,\ldots,x_n$ and the coresponding values of the f(x) function [3]:

$$f(x_i)=y_i,\ \ i=1,2,...,n \tag{1}$$

We must build a polynom $L_m(x)$ which give in specificated points x_i the same values like f(x) function.

$$L_m(x_i)=y_i,\ \ i=1,2,...,n \tag{2}$$

In the first step is build a polynom $p_i(x)$

$$p_i(x_j)=\partial_{ij}=\begin{cases}1, \text{dacã } j=i\\ 0, \text{dacã } j\neq i\end{cases} \tag{3}$$

Because the polynom, $p_i(x)$, is annulled in (n-1) points $x_1,\ldots,x_{i-1},x_{i+1},\ldots,x_n$ give the expression:

$$p_i(x)=C_i(x-x_1)\ldots(x-x_{i-1})(x-x_{i+1})\ldots(x-x_n)=C_i\Pi(x) \tag{4}$$

where C_i is the coefficient constant, and

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$$\Pi_i(x) = \prod_{j \neq i} (x - x_j) \qquad (5)$$

If $x=x_i$ and $p_i(x_i)=1$ we obtain $C_i = 1/\Pi_i(x_i)$.

With this purposes is define a polynom $p(x_i)$ which satisfy the conditions:

$$p_i(x) = \frac{\prod_i (x)}{\prod_i (x_i)} \qquad (6)$$

In the particulary case n=2, we have two table points and the Lagrange formula is reduced at the linear equation $y=L_1(x)$, which pas by this two points:

$$y = \frac{x - x_2}{x_1 - x_2} y_1 + \frac{x - x_1}{x_2 - x_3} y_2 \qquad (7)$$

If n=3, we have three table points and the Lagrange formula is reduced at the parabolic equation $y=L_2(x)$ which pas by this three points.

The polynom $L_m(x)$ which satisfy the (3) conditions can be

$$L_m(x) = \sum_{i=1}^{n} p_i(x) y_i \qquad (8)$$

Because the $p_i(x)$ polynom are by (n-1) order and $L_m(x)$ is by the same order (n-1) and satisfy the (3) conditions

$$L_{n-1}(x_j) = \sum_{j=1}^{n} p_i(x_j) y_i = \sum_{i=1}^{n} \partial_{ij} y_i = y_j ,$$
(9)j=1,2,…,n

Making the necessary changes, result the Lagrange interpolation formula:

$$L_{n-1}(x) = \sum_{i=1}^{n} \frac{\prod_i (x)}{\prod_i (x_i)} y_i = \sum_{i=1}^{n} \frac{\prod_{j \neq i} (x - x_j)}{\prod_{j \neq i} (x_i - x_j)} y_i \qquad (10)$$

3. The Prognosis of the Morphology Changes in the Bank River Sapte Oameni

The brook Sapte Oameni is a tributary of the Chirita storage lake. This storage lake is used like temporary reservoir for the water supply system of the Iasi city and represents a very important element for this system.

For a long time exploitation with a corresponding available storage we must protect the storage against the silting. The brook Sapte Oameni represents a source for alluviums transport being situated in the contact zone between lake and brook. The evolution of the alluviums volumes, which can be transported in the lake, was analysated using surveying methods (profile P1, P2, P3 and P4) and a prognosis method using a calculus program based by the Lagrange polynom (figure 1) [2].

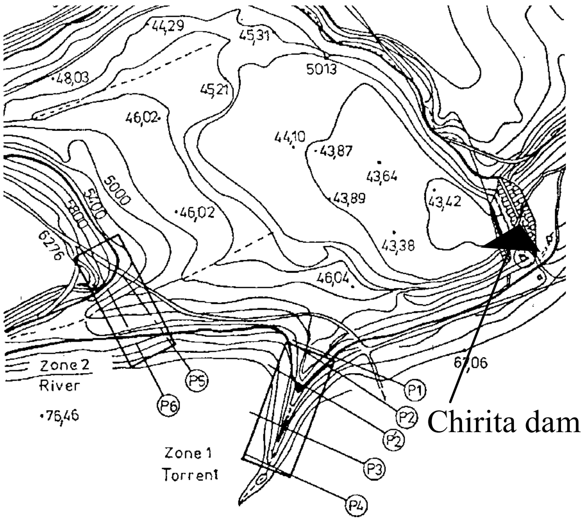

Figure 1 Investigated area

4. Protection Solution of the Chirita Storage Lake Against the Silting

The stabilization of the riverbanks is making for consolidation of the soil from basin river. We can utilise biological (reforestation and becoming overgrown the with grass) work and a combination between the transversal and the bank works rarely embankment with picket fence.

The transversal works by the torrential system, may have an optimal elevation, for to accomplish all basic function (consolidation of the thalweg, embankment support and maximal retention of the alluvial land), with an accent for torrential network transformation in a silting area.

The brook Şapte Oameni can be set up for limitation transport in storage lake Chirita using the news constructive solutions [1]

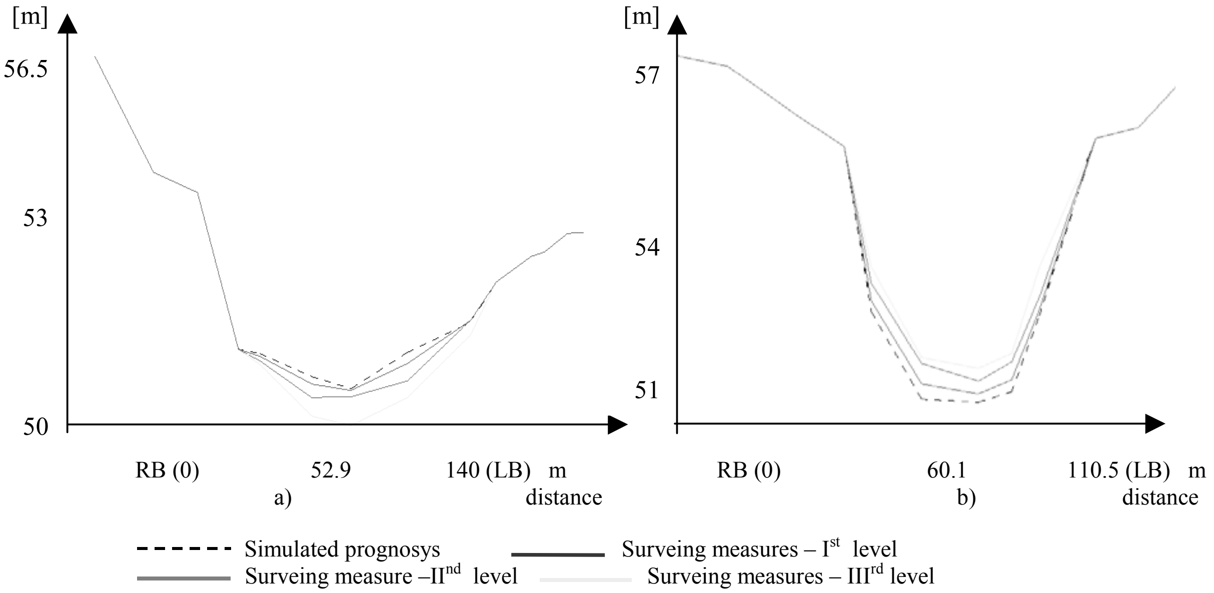

Figure 2 Transversal profile a) P1 and b) P2

4.1. Filtrate Barrage with Flexible Elements for Torrential Arrangement

This type of barrage is encadred in transversal works for torrential network and can be use for thalweg consolidation, embankment support and maximal alluvial land retention.

The transversal works are rigid, has a good elastic resistance and many advantage: economic work (less that classical work with 40…60%), small material consumption and energy (reduction 30…50 %), a high prefabrication degree and tipyification, can be executed in all area and utilise basic technologies.

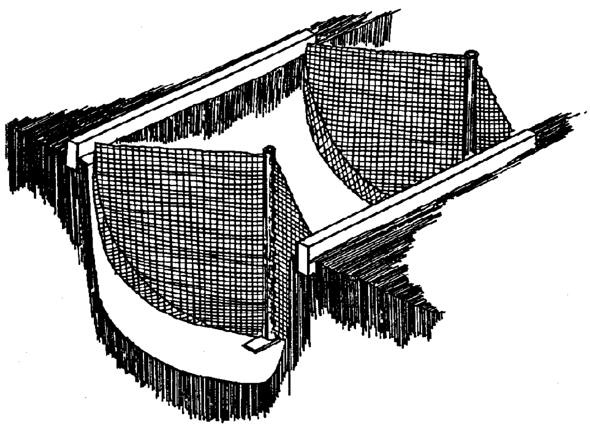

Figure 3 Perspective view of the barrage

Using this type of work the alluvial land deposition is formed din the both water race, in time the work becoming more resistant. For a good assurance we can introduce an underground bar, downstream at the work this area being a dissipation role [1].

5. Conclusions

The contact area between the lakes and the permanent river or torrential systems give more problems regarding the morphology of the bank river.

For the lake protection is important the limitation of the silting and reduction of the solid discharge from the contact area.

The prognosis of the evolution of the erosion process is important for giving the measures for storage lakes protection.

Using new constructive solutions to accomplish the torrential network we can protect the storage lakes against the silting and the reduction of the available storage.

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Romanian Shore Currents – Influence of Salinity

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Rezumat: În lucrare sunt prezentate câteva rezultate obținute în studiul influenței salinității asupra mişcării apelor din Marea Neagră. S-a ppornit de la simplificarea sistemului ecuațiilor de mişcare. În ecuațiile Navier-Stokes singurele forțe luate în calcul sunt greutatea şi forța Coriolis. Densitatea apei de mare este considerată ca o funcție liniară de salinitate. Sunt discutate câteva rezultate privind influența salinității asupra mişcării apelor Mării Negre.

Abstract: Some theoretical results are obtained in the study of the influence of salinity towards the Black Sea waters motion. The general system of equations of motion is transformed to become simpler. The only forces used for Navier-Stokes equations are gravity and Coriolis force. The density of marine water is considered a liniar function depending only on salinity. Some recent results obtained in the study regarding the influence of salinity for the Black Sea waters motion are discussed.

Keywords: equations of motion, density of marine water, salinity, marine currents.

1. Introduction

The Black Sea is a semiclosed ocean basin. It is named “miniatural ocean” because is complete with intermediate layer ventilation and deep water formation. The Black Sea has a large river dischange and a limited water change with the Mediterranean Sea. The result of this change is a extremely vertical stratification in such way as to this marine basin can be described as a two-layer sistem with a thin low salinity surface layer overlying a relatively uniform high salinity deep layer [1]. The exchange flow is controlled by the hydraulic processes in the Strait of Bosphorus as well as by the density differences between the Black Sea and Mediterranean Sea waters [2].

The Mediterranean water is a very important element for the Black Sea circulation. It seems that the Mediterranean plume would propagate to the north, because the bottom in the inflow area has a uniform north-south slope. But some observations led to the hypothesis that the penetration of Mediterranean water in the Black Sea is often blocked by a sill.

The study of Turkish oceanographers showed that Mediterranean Sea water entrains cold surface or near surface water and forms the Black Sea deep water. There is a strong startification in the Black Sea basin for salinity, identified by salinity values of 18 psu in the sea surface layer and 22.3 psu in the deep layer. Stanev demonstrated that the stability controls the water mass formation in the intermediate layers. The entrinment of The Bosphorus Strait water by sinkins the Mediterranean Sea Water and the lateral intrusions of the plume dominate the penetration of Mediterranean Sea Water in the deeper layers and control the internal mixing mechanism. The deep water in semi-enclosed seas is formed in restricted area [3]. In the Mediterranean Sea the formation fo the deep water occurs in localised regions. The problem of the formation of Black Sea deep water is a difficult one, because it is difficult to know the depth on whitch the salty water from the Mediterranean Sea penetrates and spreads in Black Sea. The startification of the Black Sea water is mainly haline. The Black Sea deep water has been formed due to the mixing of the Black Sea upper water and salty Mediterranean water of Marmara Sea suplied by the flow through the Bosphorus Strait.

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2.Equatins of motion

We are interested of the Black Sea water circulation. The study of marine currents on the Roumanian shore is a very important problem for science, economy, ecology. The Black Sea currents are the result of many reasons. The most important reason is the stress of wind. It seems that the second one is the variability of the density. Density is calculated from measurements of salinity, temperature, conductivity and pressure using the equation of state for the sea water.

Density of sea water is rarely measured. The equation of state is an equation relating density to temperature, salinity, and pressure. The equation of state become one of the equations of motion for the marine water circulation because it is a relation between density, pressure, salinity and temperature.

Generally the equation is derived by fitting curves through laboratory measurements of density as a function of temperature, pressure, and salinity, chlorinity, or conductivity. The International Equation of State (1980) published by the Joint Panel on Oceanographic Tables and Standards (1981) is now used. See also Millero and Poisson (1981) and Millero et al., (1980). The equation has an accuracy of 10 parts per million, which is 0.01 units of S (q) [4].

The equation of state of water consists of three polynomials with 41 constants (JPOTS, 1991).

The equation of state of the sea water used for the study of marine currents is reduced to the lowest terms, in a linear form:

$$\rho = \alpha_T T + \alpha_S S \qquad (1)$$

where ρ is the density of marine water,

T is the temperature,

S salinity ,

α_T, α_S are the temperature respectively salinity coefficients.

For the motion of Black Sea water the salinity is more important than the temperature. The density of Black Sea water depends on salinity more than temperature. The haline circulation in the Black Sea should be more intensive than the termal circulation in the world ocean since the Black Sea and Marmara Sea salinities are 18 and 36 respectively, eqiuvalent to a difference of about 120^0 if temperature if the linear state equation (1) for the density is used.

For this reason we can reduce equation (1) at terms:

$$\rho = \alpha_S S \qquad (2)$$

where S is the salinity ,

α_S the salinity coefficient.

The equations of motion are:

$$\begin{aligned}
&\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} - 2\omega\sin\varphi\ v = \\
&= -\frac{1}{\rho}\frac{\partial p}{\partial x} + \nu_h\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + \nu_v\frac{\partial^2 u}{\partial z^2} \\
&\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + 2\omega\sin\varphi\ u = \\
&= -\frac{1}{\rho}\frac{\partial p}{\partial y} + \nu_h\left(\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2}\right) + \nu_v\frac{\partial^2 v}{\partial z^2} \\
&g = -\frac{1}{\rho}\frac{\partial p}{\partial z}
\end{aligned} \qquad (3)$$

momentum equation in Cartesian coordinates where (u,v,w) are the components of velocity, p- pressure, $2\omega\sin\varphi = f$ is the Coriolis coefficient, ν_h , ν_v are the viscosity horizontal respectively vertical coefficient, g is acceleration of gravity.

The standard convention in geophysical fluid mechanics is x is to the east, y is to the north, and z is up. We use f-Plane what is a Cartesian coordinate system in which the Coriolis force is assumed constant, f constant. It is useful for describing flow in regions small compared with the radius of the Earth.

In the first part of this paper the component w of velocity is assumed zero, because is lower than the horizontal components of velocity. Within the ocean's interior away from the top and bottom layers, for horizontal distances exceeding a few tens of kilometers, and for times exceeding a few days, horizontal pressure gradients in the ocean almost exactly balance the Coriolis force. The nonlinear terms and the viscosity in the equations of motion are negligible. The result is a horizontal marine current. This balance is known as the geostrophic balance. We use the hydrostatic assumption.

The dominant forces acting in the vertical are the vertical pressure gradient and the weight of the water. The two balance within a few parts per million. Thus pressure at any point in the water column is due almost entirely to the weight of the water in the column above the point. The dominant forces in the horizontal are the pressure gradient and the Coriolis force. They balance within a few parts per thousand over large distances and times.

In this case the system of equatins of motion has the following form:

$$\begin{cases} -\dfrac{1}{\rho}\dfrac{\partial p}{\partial x}+2\omega v\sin\varphi=0 \\ -\dfrac{1}{\rho}\dfrac{\partial p}{\partial y}-2\omega u\sin\varphi=0 \\ -\dfrac{1}{\rho}\dfrac{\partial p}{\partial z}=g \end{cases} \tag{4}$$

3. Equations for salinity variation:

We are interested of influence of salinity, that is why we use the equation of salt diffusion what is of a generaly form:

$$\frac{\partial S}{\partial t}+div\,\vec{v}\;S+\gamma_S w=\mu_S\;\Delta S+ +\frac{\partial}{\partial z}\nu_S\frac{\partial S}{\partial z} \tag{5}$$

where γ_S, μ_S, ν_S are coefficients assumed frequently constant.

With all the assumptions made for our study the equation below is reduced to the lowest terms:

$$u\frac{\partial S}{\partial x}+v\frac{\partial S}{\partial y}=k\left(\frac{\partial^2 S}{\partial x^2}+\frac{\partial^2 S}{\partial y^2}\right) \tag{6}$$

In this case the system of equations of motion has the form:

$$\begin{cases} -\dfrac{1}{\rho}\dfrac{\partial p}{\partial x}+2\omega v\sin\varphi=0 \\ -\dfrac{1}{\rho}\dfrac{\partial p}{\partial y}-2\omega u\sin\varphi=0 \\ -\dfrac{1}{\rho}\dfrac{\partial p}{\partial z}=g \\ u\dfrac{\partial S}{\partial x}+v\dfrac{\partial S}{\partial y}=k\left(\dfrac{\partial^2 S}{\partial x^2}+\dfrac{\partial^2 S}{\partial y^2}\right) \\ \rho=\alpha_S S \end{cases} \tag{7}$$

Using the third and the fifth equations we assumed that:

$$p=-\alpha_S\;S\;g\;z \tag{8}$$

where α_S and g are assumed constant and

$$S=S\;(x,y) \tag{9}$$

This study is done for one fixed level given by z and we obtained tow dimensional equations only for the horizontal plane.

The first and the second equations allow to determine the two horizontal components of the velocity :

$$u=\frac{g\;z}{f\;S}\frac{\partial S}{\partial y} \tag{10a}$$

$$v=-\frac{g\;z}{f\;S}\frac{\partial S}{\partial x} \tag{10b}$$

where $f=2\omega\sin\varphi$ is the Coriolis coefficient.

We substitute this in the equation of salt diffusion. The equation obtained is the Laplace equation

$$\Delta\;S=0 \tag{11}$$

We assumed that the shape of basin is a rectangular one with a and b dimensions. We choose the plane coordinate system with the two axis *Ox, Oy* sides of rectangle. The boundary conditions for this equation (11) are:

the salinity is S_0 for all the bondary of basin excepting two thin regions, one on the *Ox* axis, where the salinity is

$$S_M = S_M(x)$$

for the Bosphorus Strait . S_M is larger then S_0 because there here is an increse in salinity occurs due to Marmara Sea water entering the basin from the Bosphorus. The other value of salinity on the Oy axis for the riverine input it is assumed to be

$$S_R = S_R(y)$$

smaller than S_0. The boundary conditions are:

$S = S_0$ for all the boundary of rectangle, excepting two segments

$$M_1 = \{(x,0) \,/\, x \in [x_1 , x_2]\}$$

where $S = S_M(x,y)$ and

$M_2 = \{(0,y) \,/\, y \in [y_1 , y_2]\}$ where

$S = S_R(x,y)$. $S = S_0$ for

$$\begin{aligned}&\{(x,0) \mid x \in [0, a] / [x_1 , x_2]\} \cup \\ &\{(x,b) \mid x \in [0, a]\} \cup \\ &\{(0,y) \mid y \in [0,b] / [y_1 , y_2]\} \cup \\ &\{(a,y) \mid y \in [0,b]\}\end{aligned} \tag{12}$$

Using the solution for the Dirichlet problem (11)-(12) [5] we obtained the solution:

$$\begin{aligned}S(x,y) = &\sum_{n=1}^{\infty}\left[a_n \frac{sh\frac{\pi n}{a} y}{sh\frac{\pi n}{a} b} + b_n \frac{sh\left[\frac{\pi n}{a}(b-y)\right]}{sh\frac{\pi n}{a} b}\right] \times \\ &\times \sin\frac{\pi n}{a} x + \\ &+\left[c_n \frac{sh\frac{\pi n}{b} x}{sh\frac{\pi n}{b} a} + d_n \frac{sh\left[\frac{\pi n}{b}(a-x)\right]}{sh\frac{\pi n}{b} a}\right] \times \\ &\times \sin\frac{\pi n}{b} y + A + Bx + Cy + Dxy\end{aligned} \tag{13}$$

where all the coefficients must be calculated depending on the boundary conditions. A very difficult problem is to choose the functions $S_M(x)$ and $S_R(y)$. Even if this functions are constants it is necessary to keep many terms for the expresion of *S* in (13). We can use

$$A = S_0 \ , \ B = C = D = 0 .$$

If *S* can be determinated the components of the velocity can be calculated using (10a) and (10b).

If the functions $S_M(x)$ and $S_R(y)$ can be introduced by

$$S_M(x) = S_0 + k_1 \sin\frac{\pi x}{a} \tag{14a}$$

$$S_R(y) = S_0 + k_2 \sin\frac{\pi y}{b} \tag{14b}$$

where S_0 is the significant value assumed constant of salinity in the boundary conditions.

Using the solution of Dirichlet problem the solution of (11) with boundary conditions gives by (12) is:

$$S(x,y) = A\frac{sh\frac{\pi}{b}(x-a)}{sh\frac{\pi}{b}a}\sin\frac{\pi}{b}y + B\frac{sh\frac{\pi}{a}(b-y)}{sh\frac{\pi}{a}b}sh\frac{\pi}{a}x + S_0 \qquad (15)$$

The two horizontal components of velocity for geostophic currents due of salinity variability are:

$$u = \frac{g\,z\,\pi}{f\,b}A\frac{sh\frac{\pi}{b}(x-a)}{sh\frac{\pi}{b}a}\cos\frac{\pi}{b}y - \frac{g\,z\,\pi}{f\,a}B\frac{ch\frac{\pi}{a}(b-y)}{sh\frac{\pi}{a}b}sh\frac{\pi}{a}x \qquad (16a)$$

$$v = -\frac{g\,z\,\pi}{f\,b}A\frac{ch\frac{\pi}{b}(x-a)}{sh\frac{\pi}{b}a}\sin\frac{\pi}{b}y - \frac{g\,z\,\pi}{f\,a}B\frac{sh\frac{\pi}{a}(b-y)}{sh\frac{\pi}{a}b}ch\frac{\pi}{a}x \qquad (16b)$$

In this way we can determinate the values of the velocity for marine currents, in a matter-of-fact way geostrofic currents due of salinity differences. The currents depends of z , the depth for that they are calculated.

We can obtain maps with marine currents for different depths, but, for do this, it si necessary to know very well the values of salinity, its variability with the depth and its horizontal distribution. For this purpose are necessary many measurements for the salinity values depending on depth.

The form of $S_M(x)$ and $S_R(y)$ functions can be introduced also by parabolical variation depending on values of salinity.

4. The vertical variability of salinity for the Black Sea basin:

For the 1991-1994 surveys in the Black Sea basin [6] the vertical profiles of salinity and temperature gradients show that the temperature gradient and the second derivatives varies in much broader range than the salinity gradient. The decrease in salinity was more uniform with depth than the decrease in temperature. Variations in salinity and temperature train variations in density structure what are very important for the Black Sea from a dynamical point of wiew.

The balance between vertical diffusion, vertical advection and temporal variations for salinity can be characterize by the following equation:

$$\frac{\partial S}{\partial t} + w\frac{\partial S}{\partial z} = k_S\,\frac{\partial^2 S}{\partial z^2} \qquad (17)$$

For the Black Sea basin the lateral injected waters affect the vertical component of velocity. It seems that the ratio of entrainment to Bosphorus inflow varies more in the upper pycnocline (layer of maximum vertical density gradient) then in the deep layer. The changes in salinity vertical gradients were many time neglected but it was observed that the variation of vertical velocity is a effect of $\frac{\partial S}{\partial z}$, $\frac{\partial S}{\partial T}$ variations.

The variations of vertical velocity are strongly connect with the variations of salinity, temperature, density. It is important to know the evolution of position of pycnocline for the chemical transformation of Black Sea, the largest body of anoxic waters rapid deterioration under the impact of increasing inputs.

The measurements show that the density gradient in the Black Sea is mostly due to salinity stratification [7]. This is the result of the convection process. The maximum depth of winter convection in the Black Sea is about 50 m [8]. The Black Sea is a poorly ventilated basin. Some authors suggest that the process of upwelling, downwelling and isopycnal turbulent mixing become significant factors which determine the vertical exchange. A process of duble diffusion in a diffusive regime play also an important role in the evolution of layers of Black Sea waters.

Near Cape Kaliakra and Danube canyon there are relatively intense and permanent regions of upwelling and downwelling. This fact determinates the high slope of isopycnical surfaces here and provides conditions for intensive alongisopycnical mixing between the surface coastal waters, near Cape Kaliakra and Danube canyon and the central downwelling area of anticyclonic gyres.

This process has an important influence on the water circulation along the Roumanian seaside. We must observe that the Roumanian seaside is a region with a great salinity gradient. The minimum of salinity values appears in the north of this region. It is very imoportant to choose the functions which give a good approximation for salinity boundary conditions.

5. Conclusions:

The present paper is a theoretical one. We discuss the influence of salinity in the dynamics of marine waters and different form of equations for the salinity variation: horizontal variation and vertical one. Some elements regarding the Black Sea waters are discussed.

We want to apply later the results for the study of marine currents for the Roumanian shore. For this we must study if it is posible to use for the distribution of salinity functions (14a) and (14b) and we need of many practical measurements for the values of salinity, vertical profiles of salinity, variation of salinity with depth for the boundary conditions.

The study of marine waters dynamics it is a very important present problem and we want to continue the theoretical and the experimental study of this problem.

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Touristic Ports on the Romanian Littoral

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Rezumat: Pentru adăpostirea şi staţionarea ambarcaţiunilor de agrement, a căror dezvoltare s-a făcut simţită pe litoralul românesc, de-a lungul timpului au fost proiectate şi realizate porturi turistice şi puncte de acostare. Lucrarea de faţă prezintă succint aceste porturi, precum şi pe cele în perspectivă, aflate în stadiul de studii sau proiecte. Sunt prezentate principiile de amenajare a unui astfel de port, care să includă toate facilităţile specifice.

Abstract: For the sheltering and stationing of pleasure ships, which developed on the Romanian littoral, touristic ports and berthing points have been designed and realized during time. The present work briefly describes these ports, as well as the ones in perspective, in stage of studies or projects. The arrangement principles for such a port are presented, including all the specific facilities.

Keywords: *sheltering breakwaters, berthing structures, design wave.*

1. Introduction

The Romanian littoral of the Black Sea stretches from the delta of Chilia branch at North up to the border with Bulgaria (Vama Veche) at South, having a length of about 240 km.

By its configuration and the stations' position, the Romanian littoral assured the conditions of the gradual development of the pleasure navigation and nautical sports. Consequently, touristic ports and berthing points have been designed and realized along the time for ships' sheltering.

The success of a touristic port does not lie only in the quality of the single structures, but is a matter, rather, of how efficiently these are managed. This in turn will depend largely on the extent and the adequacy of the region's infrastructure, and on the potential synergy with other social, cultural and economic activities in the area.

The coastal touristic ships and the pleasure boats will can sail along the Romanian littoral with calls from south to north at Mangalia, Neptun, Costineşti, Eforie (Belona), Constanţa (Tomis), Mamaia and even at Midia, where there are berthing possibilities or new points can be realized.

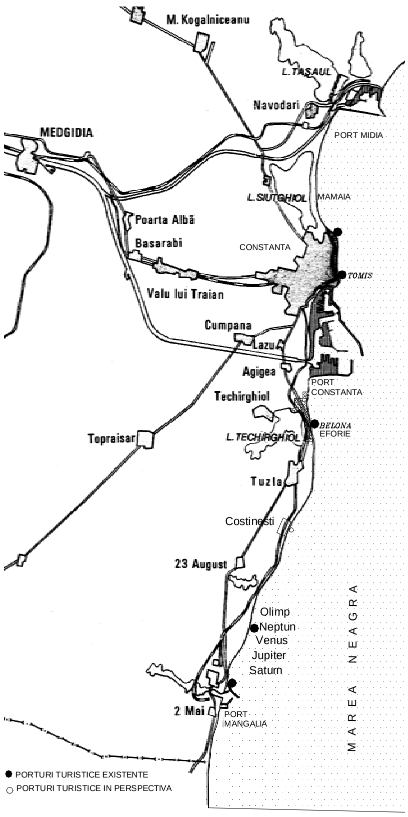

Fig. 1. Touristic ports on the Romanian littoral

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2. Principles of touristic ports arrangement

In case of a new touristic port achievement, the selection of its location has to take into account the following main aspects:

- To affect lesser as possible the adjacent beaches;
- To exist a convenient access and to keep the circulation along the beach;
- The distance to the potential clients to be minimum;
- To assure the natural depths for ships' access;
- To be possible to execute works in stages, in order to offer a number of ships' sheltering places according to the request;
- To exist the possibility of utilities connection.

The development of the pleasure navigation implies to gather a sufficient number of equipment that allow ships' shelter, call or wintering.

The port has to offer to the tourist, as it is possible:

- mooring devices, quays equipped with water and electric power;
- all the administrative services and comfort facilities;
- commercial spaces;
- facilities for waste reception;
- fuel supply

3. Design parameters

Ships' dimensions and number

To dimension a touristic port, respectively the basin and platforms surfaces, the landing stages, it is necessary to know the number of ships foreseen to frequent the port and the estimated percentage of ships with different sizes.

Unlike the commercial navigation, the pleasure navigation includes such a various range of ships types that it is impossible to establish precise classes of ships.

Ships with lengths smaller than 20 m are considered as pleasure ships. Besides these ones, there are cruise ships with lengths of about 25 m.

Table 1 presents the average dimensions of different types of ships:

Table 1. Dimensions of different types of ships

Total length	Sails and motor sails		Motor ships		Catamarans	
	Draught	Width	Draught	Width	Draught	Width
$L \leq 8$	1.5	2.8	1.0	3.3	0.6	4.8
$8 \leq L < 10$	1.6	3.1	1.1	3.6	0.8	5.0
$10 \leq L\ 12$	1.8	3.4	1.2	4.0	1.0	5.5
$12 \leq L < 15$	2.0	3.9	1.4	4.7	1.2	7.0
$15 \leq L < 18$	2.5	4.5	1.6	5.0	-	-
$18 \leq L < 25$	3.0	5.5	2.0	5.5	-	-
$L \geq 25$	4.5	7.0	2.5	7.0	-	-

Design wave

One of the most important design parameters is the wave for which the port sheltering breakwaters will be dimensioned.

Because of the shallow water at the site, the breakwaters are not submitted to the action of the greatest wave coming from offshore. This wave breaks far offshore and only the regenerated wave that can propagate to that small depth reaches the breakwater.

The most dangerous wave for which it is suitable to calculate the work is the wave that breaks when it is touching the structure.

The height of the significant wave (H1/3) with the assurance of 2 % (the storm once in 50 years) is used in the calculations.

However, any general rule can be applied. The selection of the design wave depends, besides on the depth at the site, on the local and meteorological conditions and especially on the bottom sea configuration, on its geological and geographical nature, on the coast shape, on winds regime.

Site studies and bathimetric, ocean and sediment measurements are also necessary for the design stages.

4. Necessary works

The main works to assure the operation of a

touristic port are:
- sheltering breakwaters;
- system for ships' launching and lifting;
- quays for cruise ships' berthing;
- structures for ships' berthing and mooring;
- platforms for vehicles parking, temporary storage of ships for repair or wintering, and for buildings;
- buildings for administration, harbour master's office and guard;
- fuel station;
- space for waste storage;
- fences;
- access road;
- electrical installations, water supply, sewerage;
- devices against fire.

Sheltering breakwaters

The best result expected from the sheltering works is to obtain a complete shelter inside the port, whatever the conditions outside the port would be.

The orientation and the shape of the breakwaters have to assure a good protection against waves and littoral transport and to facilitate ships' access in the port.

The breakwaters' advancement offshore will be determined also by the depth to be assured in the access channel.

The breakwaters' section will be determined as a classic section of a rubble mound breakwater.

The height of the breakwater crest is generally at 1 - 1.3 of the design wave. The breakwater can be overtopped by waves only in exceptional conditions.

The inner side has to be protected so that in these periods serious damages do not appear. The estimated cost for the repairs in case of damage is smaller than the investment cost for a breakwater that is never overtopped.

Access opening

From hydraulic point of view, the port's entrance represents a discontinuity in the protection constructions. It has to satisfy in parallel two contradictory requirements:
- to have a large enough width to assure the convenient access of ships even at storms, with moderate speeds (which allow their stop up to the quay);
- to be small enough to not encourage waves propagation and alluvia penetration in the port.

The dimension of the port entrance depends on the access of boats with sails, which have a specific route, and on the other ships types' drift.

Berthing structures

The stages for ships' landing can be fix, classic structures of reinforced concrete directly or indirectly founded, or floating, anchored, which can be placed and moved depending on requirements.

The distances between decks will be established depending on ships dimensions, in order to assure the space of maneuver for berthing.

The space for each boat will be limited by decks or bars of wood or metallic, fixed on the stages, at distances corresponding to ships' sizes.

Systems of launching to water and lifting on land

For ships' lifting on land different systems are used in the world: lifting platforms, fix or mobile cranes, monorails, forklifts, inclined plans.

Buildings

The buildings to be built in the port are: building for administration and harbour master's office, repair workshop, building for sanitary groups, guard cabin.

Utilities

The berthing places and the buildings will be supplied with electrical power for lighting and for repair devices, as well as with drinking water. The supply will be done by connection at the locality sources. The sewage of domestic wastewater from the tourists frequenting the port and from the administrative personnel will be also assured.

5. Existing touristic ports

The ports used in present for pleasure navigation are: Tomis, Belona, Neptun and South Mamaia. The existing ports have to be upgraded, in order to assure all the facilities specific to a "marina".

Although the most important touristic port on the Romanian littoral, Tomis port does not assure the safe access of ships and does not have in present all the necessary facilities. That's why a complex arrangement of this port has been designed.

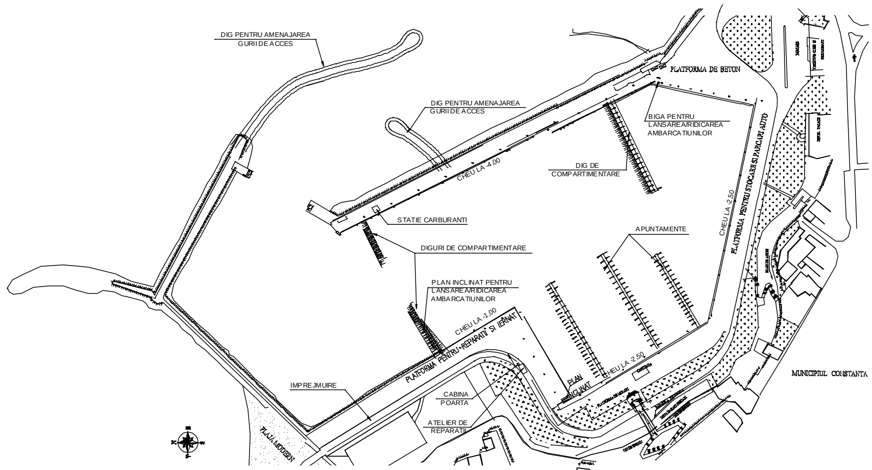

Fig. 2. Complex arrangement of the existing Tomis touristic port

Belona and South Mamaia had occurred as work ports for floating equipment's sheltering during the execution of beaches protection works, with the perspective of their transformation in touristic ports.

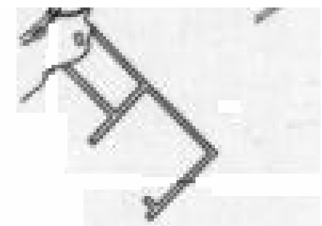

Fig.3. Schema of South Mamaia port

The main problem in Belona port is represented by the entrance's and basins' silting, leading to a difficult access in the port. The port is not adequately equipped.

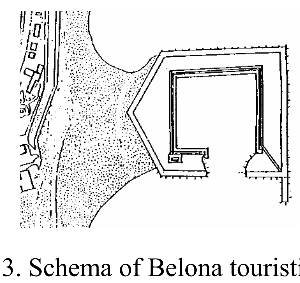

Fig. 3. Schema of Belona touristic port

In front of Neptun there is a berthing point, consisting of a dyke of T shape. The longitudinal branches have quays with vertical face for pleasure ships' berthing.

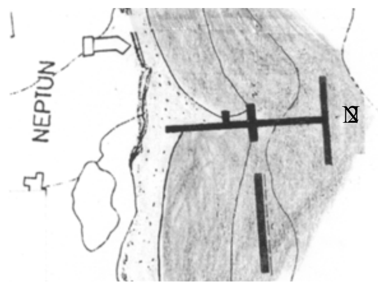

Fig. 4. Schema of Neptun berthing point

6. Touristic ports in perspective

The apparition of few pleasure ships in the Costinesti area, as well as the development of the touristic locality, lead to the necessity to analyze the possibilities of a pleasure port's achievement, correspondingly to this situation.

The project for the touristic port at Costinesti allows the works' execution in stages, so that for the beginning only a shelter for a small number of ships to be realized, with possibilities of ships' lifting on land.

The achievement of a new touristic port at Mamaia, in front of the Casino, is in design stage. It is intended to use the existing coastal protection longitudinal breakwater as sheltering breakwater.

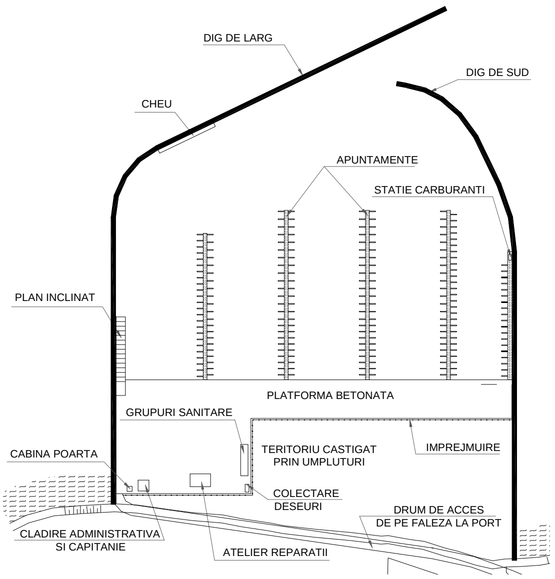

Fig. 5. Costinesti touristic port–in perspective

The future Mangalia touristic port is foreseen to be situated at north of the Mangalia commercial port, in the basin of the old Mangalia port. This will allow its development with all the facilities necessary to a good functioning, from both technical and economical point of view.

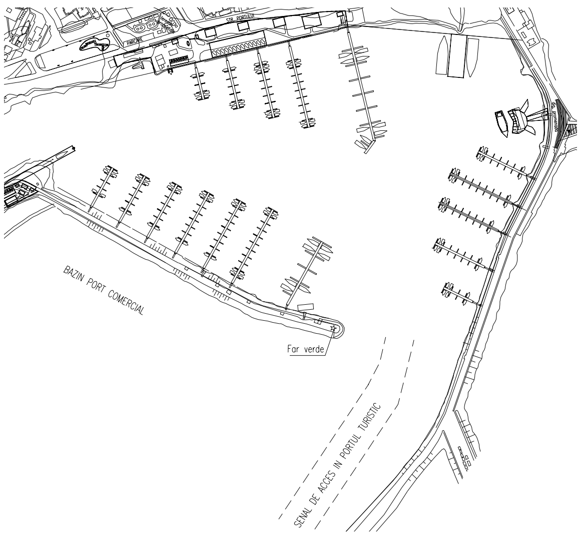

Fig. 6. Mangalia touristic port – in perspective

7. Economical considerations

The achievement in stages of a touristic port can test the proportion of the nautical pleasure activity in the area.

By itself, the achievement of a pleasure port is not an investment with profitableness assured by direct cashing. Its development will lead to incomes from other activities related to the tourism in the area.

Even if the request is apparently limited, after the construction of such a port, it can determine the request.

The direct incomes as a result of the touristc port's arrangement will proceed from the tariffs for different offered services: ships' berthing, lifting of ships on the platform and their launching to water, stationing on platform, assurance of electric power and drinking water, waste collection, use of spaces and equipment for linen washing and drying, showers and sanitary groups.

8. Environmental aspects

The port's construction will unavoidable have an impact on the environment. The impact is negative during execution, but in limited measure, and in general benefic during exploitation.

The development of a pleasure port does not raise problems from landscape point of view, because by its nature it gives color and attractiveness to an area.

Regarding the influence on the living beings, it has been found that the submerged constructions represent a fixing support for the marine organisms.

The advancement of the breakwaters towards offshore will lead to changes in the morphological evolution of the submerged beach, by currents modification and sediments movement. The breakwaters' location and orientation will be so chosen that the negative influence on cliffs and beaches regarding the erosion to be avoided. The port can constitute a protection for the area afferent to the port.

The sea water's pollution will be avoided by the assurance of the waste reception facilities and their adequate management.

9. Conclusions

Given the recent growth in recreational navigation in terms of both quantity (with large increase in number of boats) and quality (with ever-increasing boat sizes and destinations further and further afield), which implies the development of the tourism in the area, the achievement of new pleasure ports along the Romanian littoral and the arrangement of the existing ones seems to be necessary.

However, the investment for the construction of a new port being very important, and the number of pleasure ships being not yet large enough, the arrangement of the existing ports is considered suitable in a first stage. The existing ports South Mamaia, Tomis, Belona, Neptun and Mangalia are situated at convenient distances along the littoral so that the ships can stop, but they are in different stages of construction or equipping.

Thus, it is firstly necessary to improve the ships' access conditions, both from sheltering and depth point of view, as well as to assure a minimum water moving in the port basin, caused by the refraction and reflexion phenomena determined by the configuration of the port.

The attraction of the tourists largely depends on the ensurance of the facilities, both functional ones, as berthing structures, fuel supply stations, means of ships' launching and lifting, and recreational ones.

The purpose itself of these ports being to facilitate the entertainment in the nature, one of the most important aspects to be taken into account at the design and execution of such arrangements is the environment conservation.

Shores Protection against Erosions

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Rezumat: Ritmul de avansare si intinderea eroziunilor litorale dau o perspectiva ingrijoratoare, punand in pericol siguranta constructiilor existente.Lucrarea de fata prezinta succint unele solutii locale de protecție,aflate in stadiul de studii sau proiecte, ce constau din lucrări de reținere a materialului sedimentar, corelate cu aport artificial de nisip, sau din lucrari de protecție a bazei falezei cu un dig din anrocamente, poziționată la linia promontoriilor acesteia.

Abstract: Advancement and extension of the coast erosion lead to alarming perspective jeopardising the safety of the existing buildings. This paper briefly presents some local protection solutions, as studies or projects, consisting of sedimentary material retaining works, in conjunction with beach fill or with protection works for the cliff base by riprap dyke located in the promontory's line.

Keywords: *erosion, dykes, protection work.*

1. Introduction

Advancement and extension of the coast erosions lead to alarming perspective, jeopardising the safety of the existing buildings.

As a result of the erosion occurred on a wide stretch of the Romanian coast, having complex and irreversible causes, the breadth of shore got narrow very much, in certain areas, jeopardising some of the existing conditions, such Petromar`s villas, located in the southern side of Eforie North Town, the Esplanada pile of buildings, located in the shore area between the Venus Resort and the Cap Aurora Resort, as well as the shore of the Costinesti Resort, located in the northern side, to the south part of the Tuzla Cape.

Therefore, SC IPTANA SA has undertaken lately documentations concerning the shore or cliff protection, including the buildings, out of which this paper extracts for: the protection of the Eforie North and Venus shores as well as protection of the Costinesti cliff.

2. Natural Conditions

In general, the Black Sea shore, between the Constantza City and Vama Veche, consists of slanting cliffs, interrupted by coast belts and shores accompany them and divide the Black Sea lakes or lagoons, such as:

- The Techirghiol Valley with the Techirghiol Lake and the shore between the Eforie North and South;
- The Costinesti Valley with the Costinesti Lake (Mangiaounar) and the Costinesti shore;
- The Tatlageac Valley with the Tatlageac Lake and the shore between the lake and the sea, etc.

Within these coordinates, the geological coast's structure, mostly consisting of quaternary deposits (loess and clays) and of Sarmathian limestones, in a small proportion, resulted in some slanting cliffs with loess of right structure and in the action of the waves erosion at the cliffs base.

Where the Sarmathian limestones plate developed under the quaternary deposits raises over the sea level, more resistant to sea erosion points have been created, forming promontories more advanced to the sea (the Tuzla Cape, the Forum Costinesti Hotel, etc).

These promontories played also a role in the cliffs moulding by the influence they had in directing the sea currents.

In the areas of high cliff, the shores are generally narrow (5-10 m), consisting of sandy deposits resulted from the shells crushing, with small gain of detritic material, brought from the erosion of the sarmathian limestone in the base.

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Where a water layer formed up with drains to the cliff, at the loess base, landslides occur due to the quaternary clays, which seldom have a high plasticity.

The entire coast belt, between the Preventoriul in the Eforie South and the Belona Lake, is under the erosion process.

On the shore of the Coastinesti resort, to the north, in the southern of the Tuzla Cape, approximately 160 m from the shore, the shipwreck “Evanghelina” has been there since 1970, which due to its location has been a hinder against the waves, currents and consequently against the sedimentary carrying away and it might be a cause of the cliff erosion process. The cliff is eroded, endangering its steadiness, the safety of the parade and the buildings in the neighbourhood.

On one hand the morphologic regime of the coast depends on the structure and characteristics of the material of the underwater and shore bottom and on the other hand on the dynamic factors, such as wind, waves, currents and level variations of the sea.

A wide range of alternatives, shore protection general drafts, which joining together types of works have been assessed to obtain a maximum efficiency to meet the existing requirements so that the sedimentary material might be retained. They join the cross and the longitudinal dykes, of emerge of submerge type.

3. Protection Works

3.1 Protection Works of Petromar Villas

The Petromar villas are in the Constantza County, to the south borderline of the Eforie North Town (resort), along the coast belt that divides the Tasaul Lake from the Black Sea. In the southern side of these villas, there are the Solarium and the children summer champ.

Advancement of the coast erosions lead to alarming perspective, jeopardising the safety of the existing buildings on the seacoast (hotels, children champs, spa resorts, rest houses), the tourist and spa activity in the area and even in the national highway DN 39 as well as the activity of the railway connecting the Constantza City to the Mangalia Town.

The morphologic study shows that the general line of the shore has withdrawn approximately 20 m during the last 50 years.

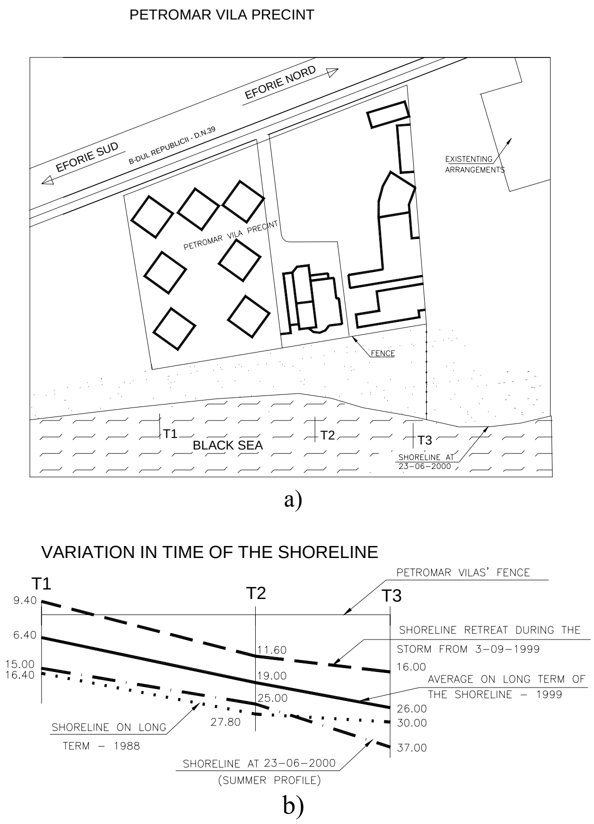


Fig.1 Layout of Petromar Villas (a).
Variation in time of the shoreline (b).

Provided that no shore protection measures are to be taken, it is estimated that the width will decrease approximately 1.0m/year and simultaneously its upper level will also decrease.

Under the action of the hydrodynamic regime, the shore has been moulded in many erosion and storage areas, which are the traffic cells along the shore.

First traffic cell emerged in front of the solarium, the shoreline penetrating further within the land, emphasising strong erosion. To the south of the dykes near the Champ, the shoreline is more advanced as against the northern section (Solarium), proving that due to the traffic coast sedimentation is from south towards north.

The next coast traffic cell came up in front of the pile of the Petromar villas, its maximum erosion -area being near the south fence and to the north it is bordered by the sewage pipes, which cross the sea.

Proposed protection works.

They are of boxes size, made up of emerged cross dykes extended by subdivision-submerged dykes, which front is closed y a submerged longitudinal dyke.

The cross dykes will keep the material carried by the longitudinal current, the longitudinal submerged dyke will keep the sand, which might be driven to the open sea during storms, maintaining the needed amount within the boxes.

The emerged cross dykes (breakwaters) will start from the shore going to the open sea up to approximately 2.50 m in depth.

In the cross section, the structure is as follows:

- Rubble stone core of 100-500kg/ piece, with slopes of 1:3. At the level +1.0m, the crown will be 4.5m;
- The rubble stone layers, adjacent to the crown, are of 100-500kg/ m3, 4.0 m width, laid on a geo-synthetic coarse of 600gr/ m2;
- Slopes and crown's protection shell is made of stone blocks of 2-4 t/piece of 2.30m thickness.

The submerged sub-divisional dykes will be made in the extension of the emerged cross dykes. They will be made of a rubble stone prism of 100-500kg/ piece laid on a geo-synthetic coarse of 600gr/ m^2. The level of the upper side of the crown will be of –1.50m and it breadth will be of 5.0m.

Submerged longitudinal dykes will be located at approximately 3.50m depth, parallel to the shore, being continuous and presenting the structure:

- A rubble stone layer of 100-500kg/ m^3 laid on a geo-synthetic coarse of 600gr/ m^2 ;
- A block stone prism of 0.5-2t/ piece, with its upper side of 8.0 m width, at the level of -1.50m

As a design sub-alternative, the solution to replace the coarse stone by the container- bags has been analysed.

The container-bags are designed to make the core of the breakwaters, replacing the coarse stone.

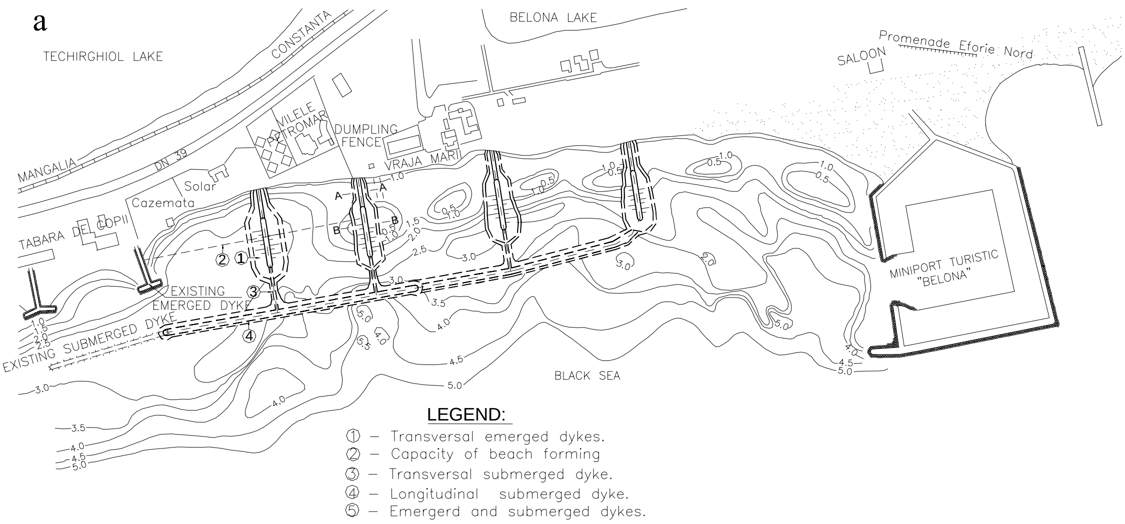
A container-bag is 1500 kg in weight, having the volume of 1 m^3.

The container-bags are made of unwoven geo-synthetic material, sawn on two sides. By choosing the best material, a good filtering protection will be obtained. The recommended filling material is sand or ballast. Filling will be done up to 80% of the amount, which allows a good mechanical handling and adaptation to the founding conditions, even if the soil is uneven.

To optimise the proposed solution, it is recommended that in the first stage the construction shall contain only the northern breakwater, the submerged dykes and the dummy sand gain. The respective area will be supervised from the view of the shore sanding up. According to this result, the second breakwater and cross-submerged dyke will be constructed. If it is noticed that the works are efficient and allow remaking and preservation of the shore, the second breakwater may be given up.

To speed up the storage phenomena and to remake the shore, dummy sanding up has also been designed. Clean and homogenous sand shall be used, which grading is similar to the existing one.

a



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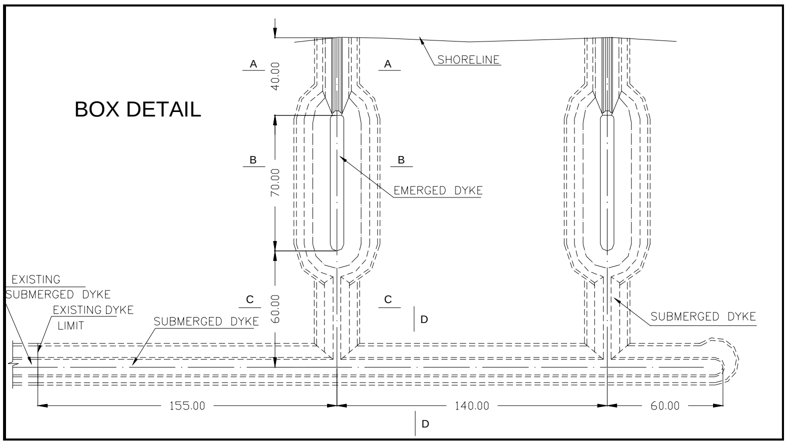

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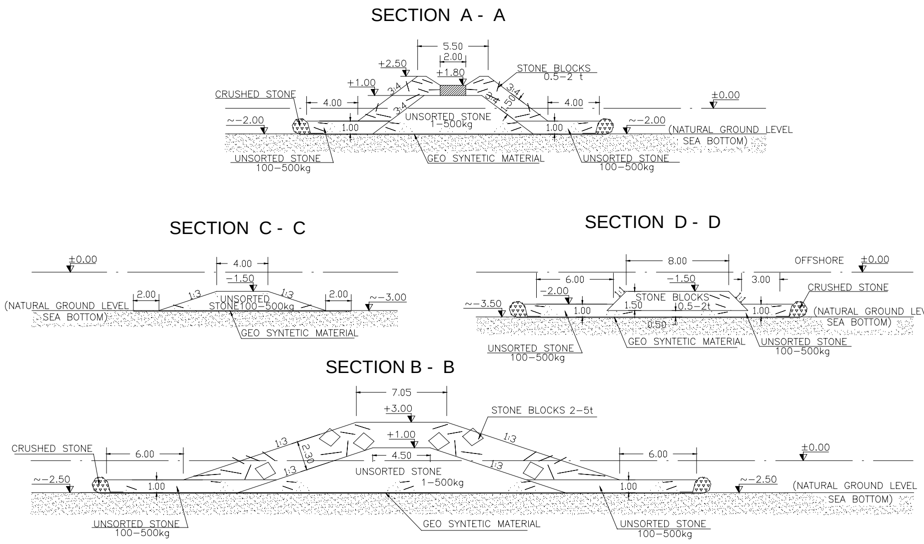


Fig.2 Layout (a). Case detail (b). Characteristic cross section (c).

The advantage of the boxes protection solution

This solution of submerged protection dykes, which forms up emerged dyke boxes, allows that the shore breadth increases evenly, permitting a more efficient utilisation. The water of the area continuously changed, therefore there is no dead-water area.

The landscape impact is minimum, occurring only on sides, due to the existing emerged dykes, which are short and leave the sea horizon free.

Traffic of small crafts can be done without restrictions; the submerged dykes are under the level of –1.50m.

3.2 Protection works of the Esplanada Pile of buildings and of the related shore, by improving of the morphological behaviour of the existing dykes system

The Esplanada pile of buildings is on the coast area, near to the north of the Mangalia Town, where during many stages, many spa resorts were built, which are of high attraction nowadays. The Espananda pile of buildings is located between the Venus resort and Aurora resort, between the protection dykes from the borders of the two resorts.

The natural conditions did not provide an adequate beach according to the accommodation places due to the coast poor in alluviums. Due to this reasons, at the same time as building of the resorts, a wide range of dykes was made to protect and obtain the suitable beach.

All resorts benefit from a very low transport of alluviums, explainable by emerging of the shoreline, with concave sections divided by promontories laid on resistant rocky submarine platforms. Thus, the protection works did not have the same efficiency everywhere. Sometimes, the erosion endangers some buildings on the shore or gets the beach narrower.

The area of the Esplanada pile of buildings contains both the dykes from the border of the two resorts Venus and Aurora and the shore between the dykes.

The first dyke towards south of the Aurora resort (A1), in front of the ONIX hotel, is built of three section, a longitudinal one (parallel to the shore), with three stagger sectors and two bent cross sections (connected to the shore), which make up the Greek letter π, in plane.

The last dyke to the north of the Venus resort (V3) is like the letter T.

The two dykes are placed at the end of a promontory, where the speeds of sea currents concentrate. Besides, the relatively large distance between the dykes' heads allows the high waves penetration into the cell. Under these circumstances, in the area of the Esplananda pile of buildings, during storms, the waves reach the cliff base, scouring the shore and bringing the red clay and the limestone up from the base.

The proposed protection works consist of:

- Submerged longitudinal dyke that will link the heads of the existing dykes;
- Dummy sanding up;
- Repair of existing dykes.

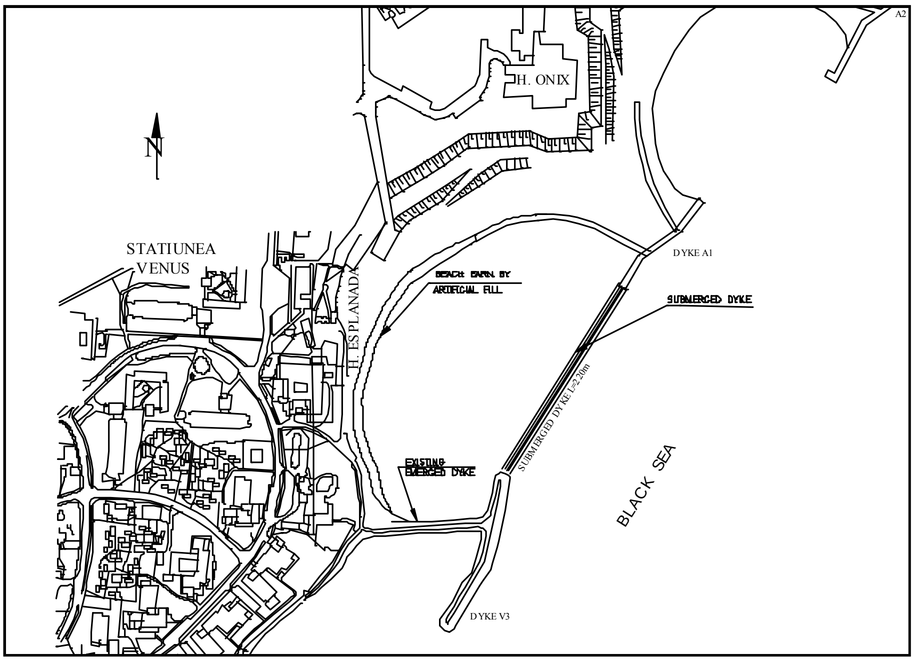
According to this design alternative, the trapezoidal section of the submerged dyke is made up of container-bags (previously described) filled with sand and ballast.

The crown of the dyke is of 4 m width, at the level of –1.5m. Its slopes are of 1:3. It is laid on a geo-textile mattress.

To remake the shore, sand dummy gain will be provided, since the poor sediments fund carried by the coast current cannot remake it in a natural way.

Storms and keep the needed amount within the box.

a



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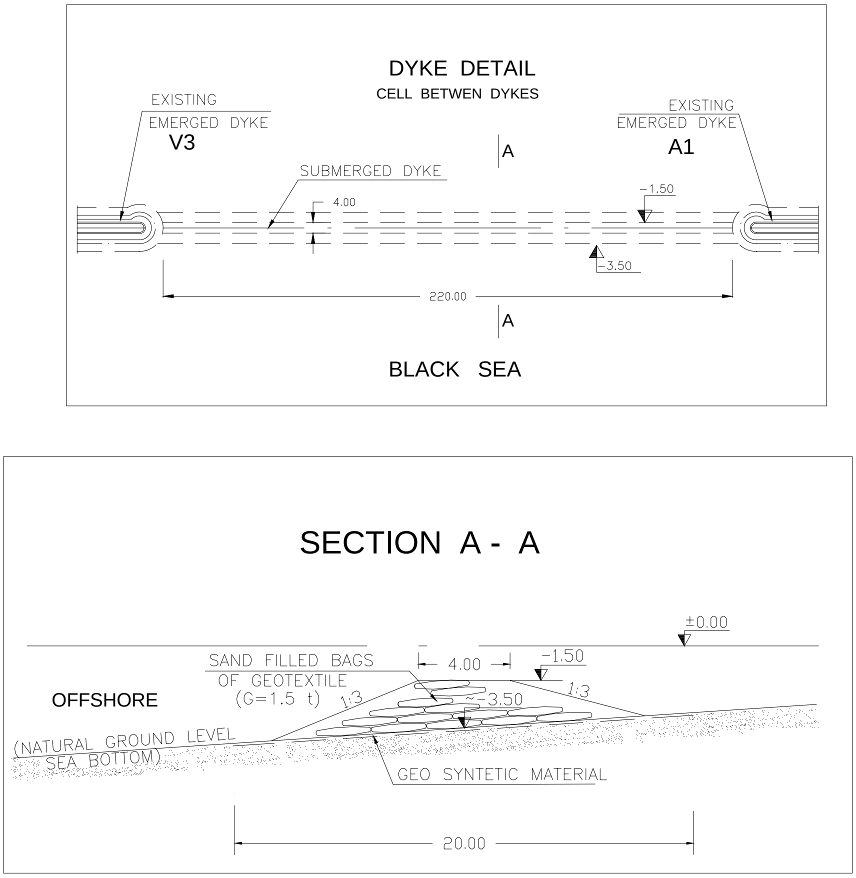

Fig.3 Layout (a). Dyke detail (b). Characteristic cross section (c)

3.3 Protection works of Costinesti cliff in the fishery area

The Costinesti Commune is made up of the Costinesti and Schitu Villages, placed on the Black Sea shore, 31 km to the south of the Constantza city, in the area of a ocean-sea firth.

The cliff of the Costinesti area has been shaped due to the sea scouring and abrasion. Its structure is determined by the geological structure of the soil laid over the sea level, by the relief's current morphology, by the waves and sea currents.

The shape of the shoreline is characterised by concave sections divided by promontories laid on resistant rocky submarine platforms.

Thus, the shore presents a sequence of sections, which morphological characteristics are very various. The variability is more or less emphasised depending on the type of the coast relief, the distance to the main sand sources, the size of sources stock, the direction, the intensity and rhythm of alluviums supply. In the shore's area, the material and energy within the coast system are under a permanent changing. Under the action of the cross and longitudinal currents entailed by the waves, the sand is alternatively carried along the shore or frontal to the bottom.

As a result, some shores have different characteristics concerning the constitutive sand grading size and the mineralogic nature, respectively the sand proportion due to the rocks crushing. There are two different section, as follows:

- the erosion coast with narrow shores and cliffs under parallel retreat. The stone cliff of the shore, partly structural, sustains a sand course thick enough to allow creation of The current cross dykes shall retain the material carried by the longitudinal current; the submerged longitudinal dyke will retain the sand that might be forwarded to the open sea during narrow shore at the shores base. The submerged profile is scarped, with structural shapes;

- the accumulative shore of barrier type (the coast belt that divides the Costinesti Lake form the Black Sea). The scoured material in the adjacent cliffs, the rocky bottom of the sea and the organic material (shells) represent the supply source.

Proposed protection works consist of protection of the shore base, placed at the promontories line, without reshaping.

Protection consists of:
- a block course of 500-1000 kg/ piece of approximately 70 cm thickness to spread the waves energy and protect the slopes;
- a rubble stone course of 10-150 kg/ piece, of 30 cm minimum thickness, and thicker in the areas of the cliff indentations will have a repair and filter role;
- a geotextile mattress of minimum 80g/s.m

The protection will start to be made from the level of –1.0 m up to +3.0m, at its base it will have a horizontal return, which will sustain the protection layers on the slope.

A gravel prism covered by a geotextile will do fixing of the protection's sustaining base.
- recover of the work with the extracted sand in order to remake the shore;

In order to make the prism at the base of dyke`s leg, an excavation will be performed, where the material resulted on the edge of excavation will be stored. The sand resulted after excavation will be stored in a different place as the loess resulted after excavation so that it might be used to remake the shore, after the dyke`s completion.

The stone prism will be made up to +/-0.00 m. This elevation permits to remake and maintain a minimum sand layer in the area.

Advantages of the protection solution

This protection solution provides a current shore section at the cliff's base, dispersion of the waves energy from the dyke over the shore, a flexible structure as well as dyke`s easy maintenance.

The landscape impact is low, the riprap protection dykes are at the base of the cliff.

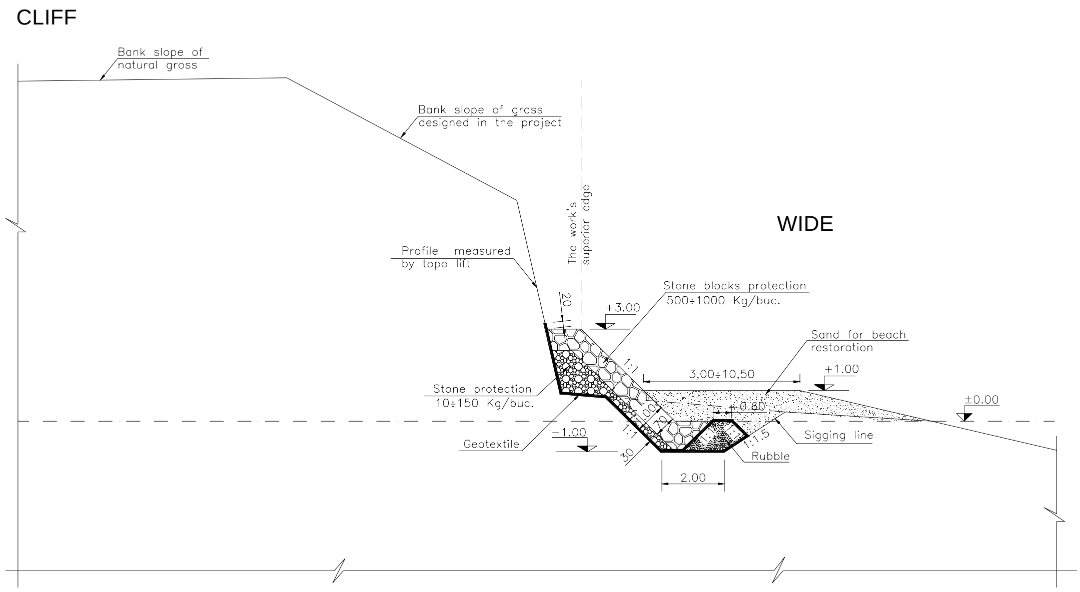

Fig.4. Characteristic section

4. Conclusions

The proposed solutions for protection of the Petromar villas and Esplanada pile of buildings provide works able to retain the sedimentary material, in conjunction with sand dummy gain so that the alluviums of the natural sedimentary transport be disturbed as less as possible.

The cross dykes shall retain the material carried by the longitudinal current; the submerged longitudinal dyke will retain the sand that might be driven to the open se during the storms, keeping the needed amount within the box.

In this area, the shore protection works are important both in respect to the area's protection and to the safety of the coast belt.

The proposed solution for the protection work of the Costinesti shore consists in protection of the cliff's base, placed at the promontories line, without reshaping.

The works material is not polluting and will be a base for renewal of the vegetation in the respective area.

By the proposed works, the sections subject to the most sever erosions where there are important investments and which are menaced by damage will be protected.

The works are under construction and their effect will be monitored so that this solution could be implemented also to other similar coast situations.

The entire coast belt is subject to erosion. The proposed works meet the area's systematisation planning, having as main target to make and maintain the shore as well as to provide the proper quality and tourist utilisation. The local protection works will not solve out the issue, on the whole, but they might be further added and changed according to the area's morphological progress.

Soils' Hydraulic Conductivity in Aranca Surface-Drainage System

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Rezumat: Conductivitatea hidraulică este o proprietate fizică a solurilor deosebit de importantă. Calcularea ei în sistemele de desecare este necesară atât pentru stabilirea corectă a distanţei între drenuri cât şi pentru stabilirea adâncimii de pozare a drenurilor.

Abstract: Hydraulic conductivity is a very important physical characteristic of the soils. That is why, in the surface-drainage systems, it is necessary to calculate it in order to correctly establish the distance between drains, as well as the depth of the tile-drain setting.

Keywords: Surface- drainage systems, hydraulic conductivity, auger hole.

1. Introduction

Aranca surface-drainage system is situated in the extreme western part of the country, pertaining – from a hydrographic point of view – to the Tisa – Mureş inferior drainage areal.

The system's total area varied throughout the time from 96,735 ha in 1975 to 123,329 ha in 1989, reaching 60,120 ha from 1990 to present, 54,242 ha being used as agricultural lands.

In the north it is bordered by the Hungarian border and by the Mureş River's left dike, in the south and south-west by the Yougoslavian border, and in the east and south-east by Aranca's department II.

It is important to determine the hydraulic conductivity in the surface – drainage systems soils in order to correctly establish the distance between drains as well as the depth of the tile-drain setting.

2. Work Procedure

The determination of the soil permeability was done in situ and by calculation.

To determine the soil permeability in situ, the „auger hole„ method was used.

Thus, 10 drillings of 10 cm diameter were drilled. Figure 1 shows the drilling' s notations for the homogeneous soil profile. The drilling's initial elements were measured after having established the hydrostatic ground water level.

Then, water was expected to emerge to the ground water level in the drilling. At that moment, a 20-40 cm water dislevelment was created in the drilling with the help of a float.

The actual determination means establishing the water emergence velocity in the drilling, at constant time intervals. At least 5 readings were necessary.

The relation between the hydraulic conductivity and the water emergence velocity in the drilling is the following :

$$K = C \, (\Delta y/\Delta t) \qquad (1)$$

For a homogeneous soil, having an impervious layer at a depth of $S \geq ½ H$, the relation :

$$K= [(4000 \, r^2)/ (H+ 20r) (2- y/H) \, y] \, \Delta y/\Delta t \qquad (2)$$

can be used.

Taking into account the measured and then calculated data, Ernst's nomograms were used to establish the in situ permeability coefficient for the under-ground water level soil.

Due to the fact that the hydraulic conductivity determined in situ through the auger-hole method characterises the soil found at the base, or even lower- in the subsoil – of th profile, it is necessary to

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determine the hydraulic conductivity of the upper part of the soil profile.

It was estimated, by calculation, according to the formula :

$$\log(Ks) = 9.56 - 0.81\log(\%\ dust) - 1.09\log(\%\ clay) - 4.64\ DA \quad (3)$$

in which : Ks is the saturated hydraulic coefficient (J.D. Jabro) (cm/h)
DA is the apparent density (g/cm^3)

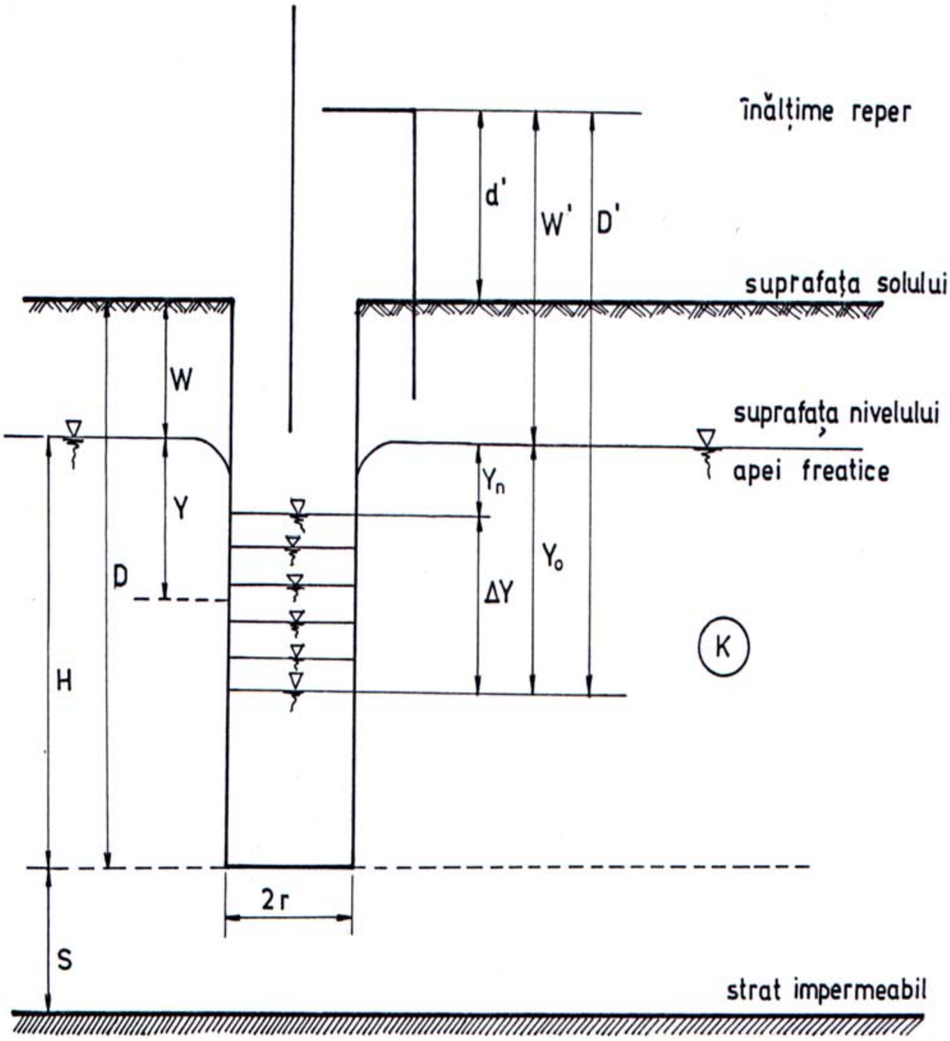

Fig. 1. Auger-hole method

3. Results

The results obtained using the „auger hole„ method are presented in Table 1.

The saturated hydraulic coefficient estimated by calculation for the upper part of the soil profile is presented in Table 2.

Table 1. Saturated Hydraulic Coefficient

Crt. N^o	Location	Depth (cm)	Ks		Observations
			cm/h	m/day	
1.	Gotlob	132-135	1.61	0.387	high
2.	Sânnicolau Mare	287-297	1.94	0.467	high
3.	Sânnicolau Mare	389-396	5.04	1.210	very high
4.	Tomnatic	93-98	10.41	2.500	very high
5.	Saravale	94-108	10.21	2.45	very high
6.	Lovrin	209-214	6.75	1.62	very high
7.	Sânpetru Mare	129-137	1.65	0.396	high
8.	Igriş	170-179	0.50	0.121	medium
9.	Periam	296-310	6.70	1.61	very high
10.	Periam Port	142-148	7.45	1.790	very high

Table 2. Estimated Saturated Hydraulic Coefficient

Crt. N^o	Location	Depth (cm)	Particle-size Distribution		DA (g/cm^3)	Ks		Observations
			dust (%)	clay (%)		cm/h	m/day	
1.	Gotlob CZ ti	15-20	20.9	30.2	1.26	10.76	2.58	very high
		35-40	20.1	29.7	1.30	7.45	1.78	very high
		55-60	20.2	27.6	1.27	10.96	2.63	very high
		75-80	21.3	28.5	1.19	24.47	5.87	very high
2.	Sânnicolau Mare CZ gz	15-20	17.7	30.6	1.37	3.85	0.92	high
		35-40	20.2	37.4	1.48	0.85	0.20	medium
		55-60	20.1	40.1	1.51	0.57	0.14	medium
		75-80	19.6	35.2	1.53	0.54	0.13	medium
3.	Sânnicolau Mare VS gz	15-20	27.0	47.3	1.53	0.30	0.07	medium
		35-40	29.5	47.0	1.50	0.31	0.07	medium
		55-60	29.3	45.3	1.59	0.30	0.07	medium
		75-80	26.6	54.0	1.46	0.56	0.13	medium
4.	Tomnatic CZ ti	15-20	27.3	40.2	1.51	0.45	0.11	medium
		35-40	25.8	39.6	1.35	2.63	0.63	high
		55-60	25.2	38.2	1.35	2.75	0.66	high
		75-80	29.0	36.1	1.35	2.63	0.63	high
5.	Saravale CZ	15-20	23.7	37.6	1.39	0.13	0.05	low
		35-40	26.0	40.4	1.41	1.35	0.32	high
		55-60	26.8	37.0	1.39	1.75	0.42	high
		75-80	26.8	37.0	1.35	2.75	0.66	high
6.	Lovrin CZ	15-20	13.0	24.7	1.45	2.61	0.63	high
		35-40	12.8	24.9	1.33	9.54	2.29	very high
		55-60	16.5	21.3	1.30	12.85	3.00	very high
		75-80	13.1	20.2	1.34	10.62	2.55	very high
7.	Sânpetru Mare CZ gz	15-20	16.1	32.5	1.60	0.32	0.08	medium
		35-40	17.5	38.9	1.55	0.43	0.10	medium
		55-60	19.0	39.8	1.60	0.23	0.06	medium
		75-80	22.8	34.3	1.56	0.36	0.09	medium

Table 2. Estimated Saturated Hydraulic Coefficient

Crt. N^o	Location	Depth (cm)	Particle-size Distribution		DA (g/cm^3)	Ks		Observations
			dust (%)	clay (%)		cm/h	m/day	
8.	Igriş EC gz-ac	15-20	23.1	31.6	1.31	5.33	1.32	very high
		35-40	29.6	45.8	1.49	0.44	0.10	medium
		55-60	28.4	39.0	1.51	0.44	0.10	medium
		75-80	21.5	31.0	1.51	0.72	0.17	medium
9.	Periam EC	15-20	11.6	19.2	1.52	1.79	0.43	high
		35-40	5.7	15.9	1.37	19.39	4.65	very high
		55-60	6.6	13.2	1.24	85.01	20.40	very high
		75-80	7.8	11.3	1.23	97.14	24.28	very high
10.	Periam Port AS gz-ac	15-20	11.2	26.6	1.36	0.58	0.14	medium
		35-40	11.9	24.0	1.49	1.88	0.45	high
		55-60	10.3	18.7	1.52	2.01	0.48	high
		75-80	3.6	13.3	1.55	5.03	1.21	very high

4. Conclusion

Analysing the data in the 2 tables, the following conclusions can be drawn :

- chernozems have a high and very high permeability on the whole soil profile, the saturated hydraulic coefficient decreasing to medium where there is gleization.
- brown soils (eutricambosoil) have a very high permeability, which decreases to medium where there is gleization and alkalinisation.
- alluvial soils in the Mureş Plain have a variable permeability, increasing from medium (in the first 20 cm) to high (between 35 and 60 cm) and to very high (to 150 cm).
- the vertosoils at Sânnicolau Mare have a medium (to the low) saturated hydraulic coefficient on the whole soil profile.
- soil permeability for water depths lower than 1.0-1.5 m is not extremely varied, but according to the lithological nature of the parenthal materials.
- soil profile permeability is a characteristic of the soil type and of the apparent density value.

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Romanian Lands: Concepts and Evaluation Methods Development

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Rezumat: În lucrare este prezentată o sinteză cu privire la evoluția concepțiilor de evaluare şi bonitare a terenurilor agricole.
De asemenea sunt prezentate concepțiile actuale care stau la baza bonitării terenurilor agricole precum şi unele aspecte cu privire la evaluarea lor.

Abstract: The article presents the land evaluation methods in our country.
The first papers where the first notions concerning land evaluation and classification appear are also mentioned.
The article also presents the current methodology applied in agricultural lands evaluation.

Keywords: Land Evaluation, Cadastre.

1.Romanian Lands: Concepts and Evaluation and Classifying Methods Development

In Romania, agricultural lands evaluation and classifying methods appeared and evolved under the influence of German concepts (a part of the cadastre was introduced by the German state – Austria – and some scientists were trained at the German school).

Since the country's territory belonged until 1918, to three different states with different statal administrations, the lands evaluation in Banat, Ardeal and Bucovina was applied with good practical results while in the rest of the country it remained only at the theoretical phase.

The first information about lands classification dates from the beginning of the XIXth century and can be found in a register in Ghimbav, Braşov County

The city of Braşov's lands are registered with topographical numbers and classified in a 1787 register called 'The Control Book'.

In 1850, in the 'Agricultural Census' register, the lands were categorized according to their uses.

The evaluation and classifying procedures in Transilvania and Banat were revised in accordance with the 1909 law when the net income for the arable lands as well as the general mean for the surface unit were calculated.

In Bucovina, cadastre was initiated on 18.08.1770, but a real change could be seen in 1820 when lands evaluation and classification were done using the same methods as in Transilvania.

The agricultural lands classification and evaluation in Transilvania, Banat and Bucovina were done using a 'general- combined system' because it combined the natural characteristics as well as the specific economic conditions of the soils.

As a result, the lands were evaluated classified into 8 evaluation classes.

In the Old Kingdom, the first information concerning the taxes for agricultural lands dates back to 1818. Still, no information is given about lands classification.

I.I. de la Brad was the first scientist who established the lands suitability for different cultures.

Once the Romanian Geological Institute was founded in 1906, wherean agrogeology division also functioned, the institution which had to deal with lands classification was also created.

Starting with the 1930s, the interest for the lands evaluation and classification in order to correctly tax them, is discussed at a national level.

In 1934, professor I.C. Drăgan Ph.D., evaluated the soil thickness (0-30 points), its physical state (3-15

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points), its nature (0-20 points), its chemical state (0-10 points). Thus, the land can receive a total of 75 points for its soil.

A.Vasiliu, Ph.D., evaluated the lands by using points to mark the main natural factors, which influence the growth and development of agricultural plants.

The soil was evaluated at 60 points according to its nature (1-20 points), its thickness (5-20 points) and its physical characteristics (0-10 points); the subsoil was also evaluated at a maximum of 30 points according to its nature (0-10 points), its thickness and its physical characteristics (0-10 points), resulting a maximum total of 90 points for the soil.

A.Vasiliu did not 'evaluate the climate separately. Aaccording to him, the climate can change, the soil, but only after a long period of time'.

Other researchers such as C.Chiriță and N. Cernescu elaborated a classifying system for the forest soils while I. Crişan (1959) discussed the soil fertility in the for mer region of Timişoara, according to naturalistic and pedological criteria.

In 1964, St. Cârstea proposed that the American classifying system should be adopted to our country's conditions.

In 1972, S.N. Maxim evaluated the lands used for growing potatoes by applying a synthetic classifying system.

The most important research and studies in this domain were done by the team supervised by D. Teaci, Ph.D., for 30 years.

They invented an evaluation system based on ecological and naturalistic systems.

D. Teaci, Ph.D., affirmed that 'special attention was given to synthetic indexes such as soil texture, edaphic volume and especially to climate' (1980:193).

The originality of the evaluation method proposed by D. Teaci, Ph.D., lies on categorizing the soil according to its uses and on analyzing a bigger number of characteristics of the factors which determine the soil production capacity.

The medium leached chernozem evolved on loess was considered to be the ideal soil for our country. It received 100 points and the environmental factors and conditions were given the maximum number of points as follows:

- 50 points for the soil and its intrinsec characteristics;
- 15 points for the relief conditions;
- 20 points for the climatic conditions;
- 15 points for the hydrological and hydrographical conditions.

As the terms of land evaluation and land classification are different notions, in what follows we shall present only the development of the evaluation concept of land evaluation.

2. Land Evaluation Nowadays

Agricultural lands evaluation represents the complex process of thoroughly knowing the growing conditions of plants and of determining the degree of suitability of these conditions for each use and culture by using a system of technical indexes and evaluation marks.

Thus, the evaluation determines how many times a land is better than another one by taking into account its fertility shown by its productions.

The Romanian agricultural lands evaluation methodology is a mathematical – heuristic model which combines the research of different land evaluation schools and national contributions (D. Teaci, 1960, 1975, 1980, ICPA Bucureşti 1987).

The natural land evaluation is done by taking into account some biophysical parameters, transformed into indicators of ecological characterization for soil and lands.

The indicators of ecological characterization comprise the soil, the relief, the groundwater, the climate, hydrology and pollution.

The natural land evaluation marks contain points from 1 to 100 and are established for each village, farm or cadastral plot for soil land units (TEO) and for the current use category of the plot.

For the arable lands, the natural land evaluation mark represents the land evaluation marks mean for the four cultures having maximal suitability for the specific cadastral plot.

If the four neighbouring TEO units differ, one from another in what concerns the natural land evaluation mark, they are reunited in one simple unit of natural land evaluation (UNB).

As for the individual plots, each property has to be differentiated so that plot evaluation mark shall the reflect the true composition of the plot.

If there are permanent land improvement works on the evaluated lands, these works will be considered as part of the UNB (simple unit of natural land evaluation), and after applying the specific coefficients, a current land evaluation mark for cultures is obtained by multiplying by 100 the result of the coefficients which participate directly in finding the land evaluation mark.

$$Y = (x_1 * x_2 * \ldots * x_n) * 100;$$

Y = land evaluation mark.

$x_1, x_2, \ldots, x_n$ = coefficients value.

If all the coefficients have value = 1, then the land evaluation mark is 100, but if one coefficient has value = 0 then, the land evaluation mark is 0.

A new element that has been introduced in the present methodology is the fact that the lowest land evaluation mark will be 1 except for the case when the annual medium temperature has a coefficient 0.

To estimate the general potential of a production unit (farm or plot), the medium mark is calculated. It is obtained by multiplying each land unit (UT or TEO) by its mark for the 24 situations (more precisely 27 situation; by calculating the land evaluation marks for the arable land, trees, vineyards as an arithmetical mean), comprising uses and different agricultural species, and the obtained result is divided to the surfaces mean (the total surface respectively).

$$NB_{MP} = \Sigma S_{UT} * NB_{UT} / \Sigma S_{UT};$$

in which:

ΣS_{UT} = total surface (in hectares per unit or per each farm or plot);
NB_{MP}=medium land evaluation mark;
NB_{UT} = land evaluation mark for each of the 24(27) situation of each land unit (UT or TEO).
S_{UT} = surface in (ha) per each land unit (UT or TEO).

The obtained medium land evaluation mark, gives general information on the agricultural lands suitability for different cultures and on their use in the production process.

The economic land evaluation is done for the cadastral plot and consists in evaluating the main conditions of infrastructure exploitation (in accordance with the law 16/1994).

The cadastral land evaluation represents the value of an agricultural land, established on the natural and economic land evaluation.

The cadastral land evaluation mark is represented by points between 1 and 100 and results from multiplying the natural land evaluation mark by the economic land evaluation coefficient.

3. Conclusions

- The diversity of the environmental conditions is reflected into the productive potential of the soil which is presented as a land evaluation mark;
- The natural land evaluation marks quantify the relation plant – environment, taking into account the land demands and the way the are used;
- The existence of one or more limitative factors determines the lands qualitative heterogeneity.
- The land evaluation result, presented as a land evaluation mark, and the sum of
- The points for each plot are on objective basis for the agricultural and forest lands evaluation.
- The value of the land depends on the economic conditions:
  - Internal (number, plot size and form, distance from the farm center and the state of the roads)
  - External (distance from market, railway station, harbor, production price, farm organization and management not being taken into account).

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Permeability of Silica Fume Hydraulic Concrete

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Rezumat: Studiile efectuate scot în evidență influența dozajului de microsilice asupra permeabilității la apă a betoanelor hidrotehnice. Au fost testate două clase de beton (BcH 15 şi BcH 20) realizate cu 0%, 10%, 20%, 30%, 40%,50% şi 60% micosilice ce a înlocuit aceeaşi proporție de ciment.

Rezultatele arată că microsilicea conduce la reducerea permeabilității la apă a betonului hidrotehnic. Dozajul optim de microsilice este de 20%. Pentru proporții mari de microsilice permeabilitatea la apă a betonului hidrotehnic creşte.

Abstract: The results of studies show the influence of silica fume dosage on the water permeability of hydraulic concretes. There were tested two concrete grade (BcH 15 and BcH 20) realized with 0%, 10%, 20%, 30%, 40%, 50% and 60% silica fume that replaced the same quantity of cement.

Results show that silica fume results in a reduction of water permeability of hydraulic concrete. The optimum dosage of silica fume is 20%. For bigger proportions of silica fume the water permeability of hydraulic concrete increases.

Keywords: *silica fume, permeability, hydraulic concrete.*

1. Introduction

In the case of hydraulic concretes the permeability is near the compressive strength and resistance to frost-thaw cycles one of the most characteristics.

Obtaining low permeability of concrete involves not only mix proportioning, but also attention to proper consolidation, protection and curing. Also for low-permeability concrete structure, the design must result in to minimize the size and the number of defects such as cracks and open joints (due to eg. creep, shrinkage, thermal gradient, loading). The relationship between permeability and capillary porosity is changed by introduction of supplementary cementing materials (SCM) such as fly ash, slag and silica fume. When concretes made with these materials are cured, large capillary pores left from original cement matrix become subdivided by secondary or later hydration of SCM into small pores, helping provide discontinuity, and permeability is reduced [1].

From literature it appears that the use of silica fume for the productio of high performance and/or chemical resistant concrete has gained popularity in last years [2]. This is due mainly to high reactivity and very fine particle siye of silica fume (30-100 times finer than cement. Silica fume has basically three roles in concrete paste: it reacts with free lime, which results from hydration of cement; it fills in pores for better interparticle arrangement and it may improve aggregate-paste bonding [3].

Due to variety of information in the literature regarding the permeability of silica fume concrete, it was felt necessary to investigate this for hydraulic concrete prepared with different percentages of silica fume (0%, 10%, 20%, 30%, 40%,50% and 60%) for two grades of concrete: BcH 15 and BcH 20.

2. Experimental Program

The experimental work of this research was mainly concerned with the effect of defferent silica fume dosages on the permeability to water of hydraulic concrete.

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The tests were made according to STAS 3519-76. Sets of three samples were tested at each dosage of silica fume and concrete grade. Both grades of concrete included samples with 0, 10,20,30,40,50 and 60% silica fume by replacing the cement.

2.1 Materials

A type H II/A-S 42,5 SR 3011:1996 cement conforming to STAS SR 3011-96 was used.

The coarse aggregate used in concrete test samples was 40mm nominal maximum size and fine aggregate was used a natural sand with a 5 mm maximum size. The sorts were:0- 7 mm, 7- 16 mm and 16-40 mm. The aggregates satisfy the conditions given by PE 713/90 and Stas 1667/76.

The superplasticizer was type DISAN A (air reducer and air-entraining agent); it was used in proportion that realized an percentage of ocluded air of 2-4%.

The silica fume (SF) used was from Tulcea Plant and is a by-product resulted from the production of ferrosilicon alloys and it contains very fine particles of SiO_2 [4]. Its main characteristics are as follows:

- It contains 85%-98% silicon dioxide;
- Its mean particle size is range 0.1-0.2 μm;
- Its particles are spherical
- Its colour is green to white;
- Density is 2000kg/m^3;
- Density undensified 240 kg/m^3
- Specific surface is 22 000 cm^2/g.

The control concrete mixes had the following proportions for 1 m^3: group A (BcH 15): cement 275 kg; fine aggregate 912 kg; coarse aggregate 1161 kg; water 72,9 l; and superplasticizer type DISAN 2,75 l. For group B (BcH 20): cement 325 kg; fine aggregate 905 kg; coarse aggregate 1111 kg; water 105 l and superplasticizer type DISAN 3,25 l.

The specimen were cast in cubic moulds 200 x 200 x 200 mm, were kept in laboratory conditions for 24 hours and then demoulded specimens were kept in laboratory conditions at 20^{0}C for 90 days when they were tested. The tests were made in accordance with STAS 3519-76.

3. Results and discussions

The test results for all specimes and the medium values are presented in Table 1.

Table 1.Test results to water permeability

Sample	Concrete Grade	Days	Pressure	Water depth (cm) 1 2 3 Medium
1.0%SF	BcH15	90	8	17 18 15 16.6
2.10%SF	BcH15	90	8	4 3 4 3.6
3.20%SF	BcH15	90	8	3 1 3 2.3
4.30%SF	BcH15	90	8	8 10 11 9.6
5.40%SF	BcH15	90	8	18 10 11 13
6.50%SF	BcH15	90	8	12 10 6 9.3
7.60%SF	BcH15	90	8	17 12 8 15.6
8.0%SF	BcH20	90	8	16 17 17 16.6
9.10%SF	BcH20	90	8	3 7 5 5
10.20%SF	BcH20	90	8	4 2 1 2.33
11.30%SF	BcH20	90	8	4 5 1 3.3
12.40%SF	BcH20	90	8	3 1 4 2.6
13.50%SF	BcH20	90	8	8 12 5 8.3
14.60%SF	BcH20	90	8	10 18 7 15

In Figure 1 there are presented the graphical representation for the medium values of test results to water permeability made at 90 days.

The test results show that all silica fume concretes have lower permeability to water than the concrete without silica fume. For the first goup A,concrete grade BcH 15 with 275 kg/m^3 , the best results was obtained for 20% silica fume, followed by

10% silica fume; over 30% silica fume the results are aproaching to that of concrete without silica fume, for 60% for example. In the second group B, concrete grade with 325 kg/m^3, the best results was obtained for 20% silica fume, followed by 40% content, than 30% content and for 60% silica fume the results are near to that for concrete without silica fume. Between the two groups, in the case of concrete with more content of cement (325 kg/m^3) the results were better than in the case of concrete with a smaller quantity of cement (275 kg/m^3); for both grade the best results were obtained for a content of 20% silica fume. If the concrete has bigger cement dosage the values of silica fume replacement are also bigger, but they must be under 40%.

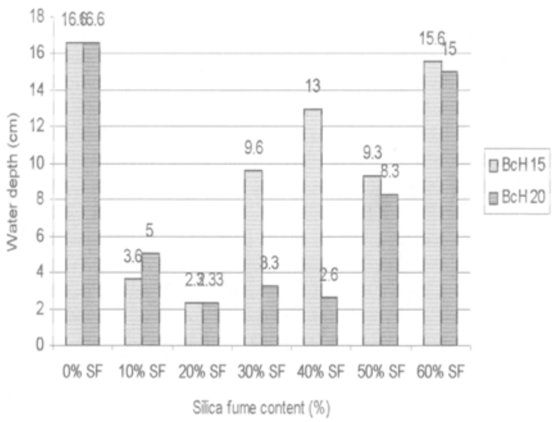

Fig. 1. Water permeability for 90 days cured specimen

For explaining the behavior of silica fume concrete to water permeability it must underline some aspects: silica fume is both reactive pozzolan and very effective filler [5].Its pozzolanic reactivity will result in the increase of concrete strength; the abundance of silica fume particles at higher levels (40%) leads to greater dispersal and hence increased reactio rates with the calcium hydroxide which, in turn produce additional gel. The filler effect of silica fume is another reeason for improving concrete properties. It has been found that silica fume in concrete essentially eliminates the pores betwwen 0.5 and 500 μm in size [5]. Silica fume densifies the microstructure of the concrete and reduces the size of capillary pores. All the above effects of silica fume are possible to accur simoultaneously. The extent to which one effect dominates the overall permeability is highly dependent on the test conditions. This may explain why there are conflicting results in the literature regarding this issue. Also, an increase in silics fume content above a critical level results in an increase in porosity and also the concrete with very high silica fume content had shown an increase of water permeability.

4. Conclusions

The general trend of the results suggests that concrete incorporating silica fume as replacement of cement exibits lower water permeability value than crresponding specimen without silica fume. The optimum content of silica fume for both concrete grades is between 10% and 20%, but is also function of the cement dosage. For 20% content of silica fume the reduction of water permeability is substantial (of about 85%), much lower than for concretes without silica fume. With the increasing of cement dosage, the content of silica fume can also increase for improving the water impermeability, but must be under 40%. For big contents of silica fume, over 50% the resistance to water permeability decreases, the results being near to that for usual concretes.

In the same time with tests on water permeability it must also determine the effect of silica fume on the strengths and other characteristics, because in many cases they can be minor.

Also, it must pay attention to the design detailing for minimization of cracks, proper mix design, adequate compaction and sufficient moist curing and concretes with low permeability will result. Further studies that include these aspects need to be undertaken.

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A Means of Accelerating the Concrete Setting with Unconventional Energy

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Rezumat: În dorința de a grăbi întărirea betonului, este binecunoscut faptul că procedeele termice utilizate în fabrici şi poligoane, reprezintă soluția cu cea mai mare eficacitate. Din păcate acestea se practică cu consumuri energetice mari care, în contextul actualelor prețuri, ridică nepermis de mult costul acestor produse. După o serie de experimentări efectuate la scară naturală, autorii propun în acest scop folosirea celei mai răspândite forme de energie neconvențională care este cea solară. Conversia ei în energie calorică se obține utilizând captatoare solare simple şi uşor de întreținut apelând la cunoscutul efect de seră. În lucrare se prezintă performanțele termice şi tehnice prin utilizarea acestor sisteme.

Abstract: In order to quicken the setting of the concrete, it is well known the fact that the heating procedures used in factories and testing fields is the most efficient solution. Unfortunately, these procedures require a large expenditure of energy that, due to its actual high prices, rise above measure the costs of these products. As a result of a series of experiments performed on a natural scale, the authors suggest that for this purpose the use of the best-known form of unconventional energy, that is the solar energy. Its conversion into caloric energy is acquired by using solar collectors that are simple and easy to maintain resorting to the well-known greenhouse effect. The paper deals with the thermal and technical performances obtained from using these systems.

Keywords: *thermal treatment, greenhouse effect, heliothermal treatment.*

1. Fabrication – a still reliable solution

The fabricated structures, which have been used only sporadically after 1990 are back on the market especially in some country areas characterized by big investments which require a quick execution of the constructions. At the same time, the factories and the construction units polygons go on with the manufacture of the already known fabricated elements like platform panels, rollers, drain covers, drain walls, tubes, gullies, girders, small capacity tank walls etc. The fabrication solution stands indeed due to its well-known advantages and it is unfortunate that many of the profile units, especially in Moldavia, have been totally or partially deallocated although they have proven their profitability before 1990. It is especially self-evident in this sense the fact that in the municipality of Iaşi none of the three biggest producers of fabricated elements in the socio-political unfortunate times was not able to conclude some well-known owner's order of performing 4.000 prestressed panels and walls elements for a mall type construction. The poor technical state of these profiles units as well as its partially modified functions has triggered the situation above described, which we consider to be rather embarrassing.

Unfortunately for the all-cast constructions as well as for the fabricated elements, their execution remains tributary to the natural setting rhythm of the concrete, which is slow. If for the constructions performed in the position from the project the environmental factors influence the way of evolution of this material's resistance, in the case of the fabricated materials industry the above-mentioned flaw is solved using the well-known thermal procedures.

2. The effect and consequences of the thermal treatment

The rise in the concrete's temperature above the conventionally accepted one leads, as it is well known, to the acceleration of the cement-water reactions, and implicitly to the substantial reduction of this material's setting time [1, 2]. The speed rises of course according to the temperature, but this one does not double every 10 ºC as it would result from the law of the masses

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action known from the chemical kinetics, the rise being of only 1,7 times. This explains through the physical phenomena, which become apparent when the temperature rises and which are common to all the heterogeneous reactions; the main ones refer to the differential expansion of the concrete components, to the screening effect of the cement granules, to the premature dehydration of the concrete and to the falls of temperature with the formation of the thermal resistance. All this has been extensively analyzed in specialized papers and treatises, implicitly leading to quantitative and qualitative discrepancies between the concrete with accelerated setting and the one that has benefited from a normal regime, evidently in favor of the first. From these effects, the most affected ones are resistance and durability.

Various procedures have been suggested to reduce the intensity and the consequences of the destructive manifestations, some of them being especially efficient, but which have never been implemented unfortunately on an industrial scale due to the concrete and fabricated elements producers' indolence.

Nevertheless, at least in the last decade, the great disadvantage of the reinforced concrete elements production in specialized units is the ever-higher costs of the conventional energies dedicated to the acceleration of this material, which is so vastly used in constructions. As the caloric energy in vapor form with low pressure is the most used thermal agent in the fabrication industry, it is enough to remind the fact that the price of a kilocalorie is today, on a national level, between 800.000 and 1.000.000 ROL. If in order to raise the temperature of concrete from 18 ºC to 60 ºC we need 2,76 kilocalories/m^3 one may easily deduce the economic consequence of the acceleration of the setting by thermal treatment of any type of fabricated element. Due to this fact, the restraint manifested versus the current use of these procedures is somehow justified.

One of the most efficient possibilities of improving this major disadvantage is to at least partially replace the thermal energy provided especially by the central-heating stations with unconventional energies, among which the solar one stands out due to its easy conversion to caloric energy by the well-known greenhouse effect.

3. The greenhouse effect in the fabrication industry

From the analysis of the dates provided by the Institute of Meteorology and Hydrology from Romania it is drawn the fact that our country enjoys a sunny rate on a sufficiently large period of time during the year, both in the warm season and in the cold one. Based on this characteristic, the authors conceived three types of simple and easy to maintain solar collectors, which have been described in detail in [3]. Their different dimensions allowed the heliothermal treatment of various reinforced concrete elements: plates, beams, pillars, curved elements etc. The same collecting principle was used; the prefabricated parts were covered with a polyethylene black sheet on which were applied or rolled covers provided with a polyester sheet with a very high transparency level. Between the two sheets, which are set at an optimum distance (18-22 cm), the solar energy creates a warm air cushion whose temperature is influenced by the degree and duration of the sunny time as well as by the warmth specific to any type of collector. When the direct sunny effect is over, the collectors are covered with heat insulating tarpaulins in order to maintain the accumulated heat. In order to achieve an acceptable setting level for he performance in safety conditions of the operations of striking, manipulating and storing, the heliothermal cycle may have a duration between 24 and 72 hours according to the season, the external temperature and especially the degree and duration of the sunny time.

One can easily establish the termination of this treatment cycle by automatically following the evolution of the concrete temperature with a recording mechanism. With this one can measure the maturity degree which, being correlated with the setting level of concrete according to the type of cement used, offers the possibility to establish the β parameter with sufficient precision. Sampling and testing of the control samples is no longer necessary but to prove the superiority of the non-destructive method of control of the setting level by correlating it with the maturity degree at variable temperatures.

We have to mention the fact that, during the first experiments we have performed on an external track of a polygon of prefabricated parts from Iaşi, we were greatly surprised by the thermal and technical performances of this procedure of accelerating the setting of concrete. Let alone the economical advantages. Beside this, many times, especially during spring and autumn, the heliothermal treatment of concrete can be deemed as moderate. Therefore, the above-mentioned destructive effects manifest themselves more discretely, and the resistance and durability characteristics of concrete are less affected.

4. Significant experiments

We will here present the results obtained from one of the most spectacular experiments we have performed a few years before at the Midia-Năvodari Production Basis of TAGC Ind Constanța on an external polygon arranges according to a project of the authors of the present work. The experiment has taken place in July in the conditions of permanent sunny time. The elements treated heliothermally were cover plates and curved prefabricates for small capacity tanks of C 16/20 concrete made with H II/A – S 32,5 cement. The casting began at 6.30 a.m. and while in every independent modulus of the collector the casting was finished, the prefabricated parts were covered in black foil and then the shutting covers provided with the polyester sheet of high transparency were being rolled.

Figure 1 presents the evolution of the temperatures (θ_b, θ_s, θ_{al}) during a 24 hour cycle. In the analyzed collector there have been also introduced six control cubes which have been tried after the end of the heliothermal cycle and at 28 days, the latter being kept in external track conditions.

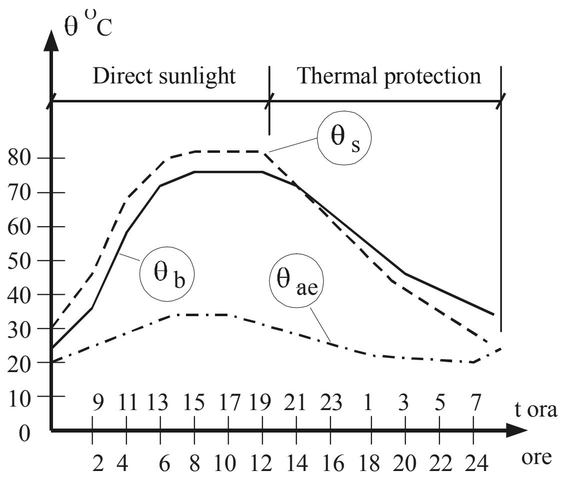

Fig. 1. Heliothermal treatment, Midia-Năvodari, July

θ_b – the temperature of concrete
θ_s – the temperature of the source (the warm air cushion)
θ_{ae} – the temperature of the external air
$M_\theta = 5560$ h°C
$\beta = 68,2$ %
$\beta^{ef} = 75,4$ %

The special climatic conditions lead to achieving a thermal maximum of the concrete of 78°C that was maintained for almost 4 hours. The fact that after 20.00 p.m., the thermal protection applied on the collector allowed for a relatively slow falling in the concrete's temperature is very important as at the end of the heliothermal cycle it has been registered a temperature of 43° C. Calculating the maturity degree M_θ, this resulted to be 5660 h° C and to it corresponds, based on the correlation M_β - M_θ – β a setting level of 68,2 %. testing the control samples has lead to an effective setting level of 75,4% calculated according to the results obtained on the cubes kept in standard time conditions of 28 days which means that the M_θ method is covering. [4]. We must say that the fact that at the standardized term the resistance losses were of only 7,9 % in comparison with the samples set in normal conditions of temperature and humidity, therefore this type of thermal treatment is less aggressive than the classical ones. We like to think that to obtain such a setting level in only 24 hours without any expenditure of conventional energy represents a performance that must be taken into consideration.

Without a graphic representation, we consider mentioning also an experiment concluded in Iaşi, at the beginning of November, when during two sunny days the air temperature was of 8-12 °C during the day, and 4 °C during the night. The sunny time was evidently reduced which determined the extension of the cycle to 49 hours, with the correspondent thermal night protection. The treated elements were platform semipanels from the same prepared concrete class only with Ii/A – S 32,5 R cement. It was obtained a thermal maximum of 48 °C for the first one and of 45 °C for the second one, during the night the temperature falling to 28 °C. The results are also special in this case if we take into account the climatic conditions in the month of November. The obtained setting level was of 52,7%, the effective one of 56,1%, and the resistance losses at the age of 28 days following this moderated heliothermal treatment have been of only 3,7% in comparison with the witness samples.

5. Conclusions

The successive energy crises in the history of the world as well as the unprecedented rise of the costs for all the forms of conventional energies, compels the construction industry to severe energetic restrictions and also to appeal to forms of unconventional energy whenever it is possible.

Among the known ones and characteristic to our country, the solar energy presents the greatest facility of conversion into thermal energy based on the well-known greenhouse effect.

As the performance of some types of structural elements of reinforced and precompressed concrete is still practiced in prefabricated variant, their execution on external tracks with the arrangement of some simple and easy to maintain solar collectors can completely eliminate the conventional expenditure of energy meant for the acceleration of the concrete setting.

The economical consequences of the heliothermal treatment are obvious, and the technical ones surprise by their performance. According to season, sunny time, the solar collector thermal insulation solution and the external air temperature, a cycle like this can go on for 24-72 hours, if we also take into account the expected setting level.

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Risk Factors in the Assessment of the Concrete Setting Level

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Rezumat: Se pune în evidență faptul că aprecierea sau determinarea nivelului de întărire al betonului prezintă o importanță deosebită pentru tehnologia structurilor realizate din acest material. Metodele clasice destructive şi nedistructive nu sunt întotdeauna operante, iar ultimele pot produce erori suficient de mari. Reluând conceptul de grad de maturizare al betonului, colectivul de cercetare de la Facultatea de Construcții din Iaşi, pe baza unui amplu program experimental, a stabilit corelarea dintre nivelul de întărire al betonului şi gradul de maturizare la temperaturi normale şi variabile pentru cele mai uzuale tipuri de ciment folosite în elementele structurale, stabilind astfel o metodă simplă şi eficace de determinare a acestui nivel.

Abstract: It is emphasized the fact that the assessment or measurement of the level of the concrete setting is of utmost importance for the technology of the structures made up of this material. The classical destructive and non-destructive methods are not always operational, and the latter can bring up sufficiently high errors. Back to the concept of the maturity of concrete, the research stuff of the Construction Faculty in Iaşi, relying on a vast experimental program, have established the correlation between the level of the concrete setting and the degree of maturity on normal and variable temperatures for the most customary types of cement used in structural elements, therefore establishing a simple and efficient method of measuring this level.

Keywords: *matury degree, heliothermia.*

1. Introduction

One of the major concerns related to the theory and practice of the reinforced prestressed concrete structures is that of establishing the level of concrete setting of the various composites made of this material. Assessing this level is of utmost importance for the monolithic elements and structures as well as for the fabricated ones from at least two points of view:

- On the one hand, from a technological point of view, which implies the risk free development of some specific technological phases, partially or totally removing of the proppings, manipulating and transporting the fabricated products, transferring the prestressing force in the case of the elements with initial tensions, etc.
- On the other hand, it refers to establishing the above - mentioned level at the age of 28 days in order to assess the quality of the material by class. This assessment is made exclusively by the destructive method and it does not raise difficult problems except for the fact that in practice the environmental conditions and the construction elements' form differ a lot from the standardized one as well as from the form of the samples used to this effect.

An aspect to be highlighted is the one referring to the age of the concrete at the moment of the assessment related to the standardized term of 28 days. After this age, the concrete resistance raises slowly following the sensitive lessening of the water – cement reactions, the raise being due especially to the reinforcement process of the crystalline concretions and the cement slab's ageing. Usually, the compression resistance of the concrete can grow with up to 10-15% compared to the one registered at the age of 28 days. Various researchers have quantified this growth by exponential relationships and especially by logarithmic ones. Anyway, knowing the setting level for the concrete after the standardized term is of less importance and it is necessary only in solving more special problems.

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Establishing or assessing the above-mentioned characteristic at terms before the age of 28 days ca mostly be determined in the reinforced concrete construction technology. Unfortunately this is a lot more difficult and it is affected by a lot of parameters, the destructive and non-destructive methods frequently being inoperative or inducing sufficiently big errors. The estimate relations suggested by various researchers relying on the compression resistance measured at the age of 7 days (in the laboratories of Prague, Czech Republic) as well as other calculation formulas present a lot of disadvantages leading to ambiguous results.

2. Classical methods of assessment

As it is well known, these methods can be destructive or non-destructive, the first ones not dealing with an assessment proper but with a more or less approximate estimation of this level. In the contemporary technique of the concrete composites, the non-destructive methods would present of course a certain comfortableness, if they did not induce rather big errors and if they did not require the knowledge of a lot of the concrete parameters that facilitate the establishment and interpretation of the results. Unfortunately, in practice, there appear situations in which a part of these parameters are unknown and cannot be established.

According to the Physics branch to which the method of investigation belongs, the non-destructive ones can be classified [1] as follows:

- Acoustic and ultra-acoustic methods (resonance methods, ultrasonic impulse methods and the surface waves method);
- Mechanical surface methods (of recoil, of print ant by shooting);
- Atomic methods using the penetrating radiations or neutrons;
- Electromagnetic methods by attenuation of the microwaves and the pachometer method;
- Mixed or combined methods that use the ultrasonic speed and the recoil, the speed and attenuation of ultrasounds, the speed of ultrasounds and the attenuation of the gamma rays.

Each one of these, especially those used for the assessment of the concrete resistance, have a certain application domain which can be sufficiently restrictive concerning the minimum class of the concrete, the age of the material, the areas of the element in which the assessment takes place, the depth of the investigation, etc. The accuracy of these estimates is usually situated in the interval ± 30 – 40%, except for the ultrasonic methods of impulse that give an accuracy of ± 10 – 20%, in case of the existence of a calibrating curve, the concrete structure, the maintenance data and possibly the samples or drill cores. The partial lack of these data raises the accuracy interval at 15 – 25%, and the total one finalizes it at ± 30 – 50%.

The control methods by destructive processes have been well known and used for a long time, practically since the birth of the first laboratories dedicated to the study of this material's properties. Trying the standardized samples until they broke (cubes and cylinders) helps the nowadays assessment of the concrete quality by class (at 28 days) and also the estimation of the material's setting level at various times, necessary for the development in safety conditions of some characteristic technological phases. For the first estimation, the samples must also be kept in standardized conditions, and the try out gives indication primarily on the quality of the concrete mix and not on the quality of the concrete in the structural elements where the setting took place in natural environmental conditions, at the most times far away from the standard ones. The other try outs, which take place at the age of 28 days, require a large amount of sampling; when the expected setting level has not been obtained, the destructive try out method stays without an object.

3. The setting level control by the maturity degree

Mostly after the year 1950 there comes out the notion of concrete maturity defined as a time and temperature function in which the setting of this material has taken place. Soon it was established the fact that there is a correlation between the concrete maturity degree and its setting level which can be assessed as significant. Consequently, this becomes a non-destructive control method which by its simplicity, by the apparatus used and the ease of the results' interpretation clearly surpasses the other methods.

Among the first and most important researchers stands out A.M. Neville [2] who defines the concrete maturity as a sum of the time – temperature products, measured in ºC an hour or by ºC a day. The temperature has as its reference point a value between – 12 ºC and – 10 ºC, explained by the fact that at temperatures below the water freezing point and until the reference point the concrete resistance raises insignificantly in time. Neville established that the concrete compression resistance, expressed in MN/m2, depending on the maturity logarithm gives results which are closer to reality when the initial temperature of the concrete is between 16º and 27ºC and a loss of humidity by drying does not take place in the considered amount of time.

Other researchers have tackled the concrete maturity problem, too. Thus, Papadakis [3] shows that heating the fresh concrete leads to an acceleration of the setting and the relation between maturity and the compression resistance is available only for a certain type of concrete, being influenced by the concrete type and mix, by the a/c ratio and it has to be determine by experiments. For the calculation of the maturity, the researcher adopts Saul's formula:

$$R- = \Sigma\ (Ti + 10)\ \Delta\ t(1)$$

in which Ti stands for the average temperature of the concrete expressed in the time period Δ t.

It is stated in the quoted book that better results concerning this concept are obtained for concrete temperatures that go beyond 10 ºC. Papadakis introduces the term of weighted maturity degree and comes with a personal calculation method, while the correlation with the setting level is done graphically.

4. Researches performed at the Constructions Faculty in Iassy

Many of the researches of the staff of The Construction Technologiess at the faculty of Iaşi have concentrated on the concrete' maturity degree and its correlation with the setting level [4,5]. Only the wish of not over extending the volume of the present work has determined us to quote just two of them.

According to this staff, the maturity degree represents the surface between the temperature variation curve and the y-coordinate of –10 ºC under which the chemical processes practically stop. (fig. 1):

$$Mi = (\theta i + 10)\ ti(2)$$

where θi stands for the concrete temperature considered invariable or average in ºC in a certain period of time ti measured in hours, and Mi – the actual maturity degree at this θi temperature, in h ºC.

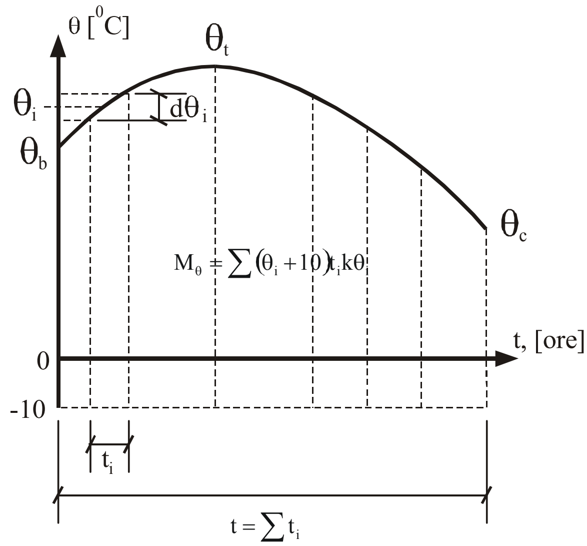

Fig. 1. The maturity degree diagram by heliothermia

When the concrete sets at the normal temperature of +20 ºC, in a period of 28 days, the total maturity degree M28 to which corresponds a setting level β = 100% has the value:

$$M28 = (20 + 10)28\ 24\ h/day = 20160\ hºC(3)$$

This remains constant for all types of concrete, while the maturity degrees Mβ corresponding to the various setting β degrees, at the same temperature of 20ºC, vary according to the concrete type, having thus as a result, for the same setting level, fluctuating ti durations.

Nevertheless, for the same type of concrete, at the stated temperature, there is a significant correlation between the setting β degree and the maturity Mβ degree presented in the table. [4,5].

For fluctuating temperatures, the maturity degree and the durations obtained for the realization of a β level implicitly vary. Thus, for $\theta \neq 20$ ºC the maturity degree is calculated with:

$$M\theta = \Sigma\ (\theta i + 10)ti\ k\theta i = \Sigma\ miti \quad (4)$$

where $k\theta i$ is the correlation coefficient of the maturity degree at a normal temperature and at the θi temperature, for the same setting level.

For the cements customarily used in the preparation of concrete destined to the structural elements, the values $k\theta i$ are also presented in a table by the mi parameter defined by the relation:

$$mi = (\theta i + 10)k\theta i \quad (5)$$

In order to establish the correlations β - Mβ and Mβ - Mθ, a vast experimental program has taken place in the profile branch of science laboratory which lead to extremely self-evident results that have been statistically interpreted.

5. Conclusions

- Establishing the setting level of concrete in some phases of its structural finishing is of utmost importance in the practice of monolithic and fabricated structures.
- The destructive methods require a lot of sampling, with a sufficiently high consumption of concrete, manual labour and energy, and once we have tried them, the evolution of the setting level until the age of 28 days remains unknown; in order to achieve more telling results these samples must be kept in "elemental conditions";
- In the case of the non-destructive methods which have become classical, these offer only a quick look over the characteristic brought forward, they require the knowledge of some parameters and may trigger great errors;
- The method of assessing the setting level by the maturity degree is a simple comfortable economic process, and according to the experiments' results it is easy to cover in comparison with the one offered by the mechanical try outs;
- The correlations β - Mβ - Mθ that we have already established have very important applications in the concrete technology: the control of the setting level, the assessment of the escape intensity of exothermia, the projection of the thermal conditions, etc.

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Verification of Plastic Strain Growths Based on Yin's Plasticity Condition for Rocks and Masonry Materials

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Rezumat: Creşterile sau ratele eforturilor plasice, în cazul legilor corespunzătoare de curgere plastică cu o anumită condiţie de plasticitate, nu sunt întotdeauna în concordanţă cu experienţele, iar rezultatele fizice nu sunt confirmate. Condiţia de plasticitate Yin descrie foarte bine starea de stress pentru câteva tipuri de gresii şi mostre prelevate din calota de sare brută. Analiza creşterilor efortului ne permite să spunem când poate fi aplicată această condiţie legilor de curgere în concordanţă cu teoria materialelor rigide şi elasto-plastice.

Abstract: Growths or rates of plastic strains in the case of associated plastic flow rule with a selected plasticity condition do not always accord with experience and the results are not always confirmed physically. Yin's plasticity condition describes quite well the state of stress for some sandstones and samples taken from rock-salt domes. The analysis of strain growths makes it possible to say whether this condition can be applied to associated flow rule in accordance with the theory of rigid or elasto-plastic materials.

Keywords: *Plasticity, associated flow rule, Yin's condition, plastic dilatation*

1. Introduction

Accepting the plasticity criterion makes it possible to allow for some permanent (irreversible) deformations in the elasto-plastic analysis, while in the model of a rigid body the values of plastic strains determine the total strains. In the model of an elasto-plastic body, the total strain growth is:

$$d\varepsilon_{ij} = d\varepsilon_{ij}{}^{e} + d\varepsilon_{ij}{}^{p} \tag{1}$$

where $d\varepsilon_{ij}$ growths stand for elastic strains, while $d\varepsilon_{ij}^{p}$ represent growths in plastic strains. The growths or rates of $d\varepsilon_{ij}$ and ε_{ij} strains can be calculated straight from the associated plastic flow rule for incompressible materials:

$$d\varepsilon_{ij}{}^{p} = d\lambda \frac{\delta F}{\delta \sigma_{ij}} \tag{2}$$

or

$$d\varepsilon_{ij}{}^{p} = \lambda \frac{\delta F}{\delta \sigma_{ij}} \tag{3}$$

This rule defines the relationship between the components of the tensor of plastic strain growths or the components of the tensor of plastic strain rates and the components of stress tensor in strain processes in perfectly plastic bodies [1].

This is the case when an assumption is made about identifying the function of plastic potential $G(\sigma_{ij})$ with the plasticity condition $F(\sigma_{ij})$. The deviator of plastic strain growth or the deviator of strain rate is orthogonal to the plane of the plasticity condition. In the case when the plasticity condition is different from the plastic potential, with reference only to growth notation, the plastic flow rule is related only to the existing plastic potential:

$$d\varepsilon_{ij}{}^{p} = d\lambda \frac{\delta G}{\delta \sigma_{ij}} \tag{4}$$

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Thus, here applies the rule in which the deviator of strain growth is co-linear with the gradient of plastic potential function $\partial G/\partial\sigma_{ij}$. Certainly, the gradient of this function is a vector of normals to the plane of plastic potential. Note that it is then impossible to define plastic strain growth in accordance with correlation (2) or plastic strain rate from formula (3).

Considering Yin's criterion as plasticity condition $F(\sigma_{ij})$ and assuming that $G(\sigma_{ij})=F(\sigma_{ij})$, one can define the flow rule associated with this plasticity function.

Yin's plasticity condition is as follows: (5)

$$F = J_2 + \frac{(I_1)^2}{6} - BI_1\sqrt[3]{I_3 + I_1 I_2} - A = 0 \tag{5}$$

or

$$\begin{aligned} F = &\frac{1}{6}\left[(\sigma_x-\sigma_y)^2+(\sigma_y-\sigma_z)^2+(\sigma_z-\sigma_{xy})^2+6\left(\tau_{xy}^2+\tau_{yz}^2+\tau_{zx}^2\right)\right]+ \\ &+\frac{1}{6}(\sigma_x+\sigma_y+\sigma_z)^2+B(\sigma_x+\sigma_y+\sigma_z)\cdot \\ &\cdot\sqrt[3]{\begin{matrix}\sigma_x\sigma_y\sigma_z+2\tau_{xy}\tau_{yz}\tau_{zx}-\left(\sigma_x\tau_{yz}^2+\sigma_y\tau_{zx}^2\sigma_z\tau_{xy}^2\right)+(\sigma_x+\sigma_y+\sigma_z) \\ \left(-\frac{1}{6}\left[(\sigma_x-\sigma_y)^2+(\sigma_y-\sigma_z)^2+(\sigma_z-\sigma_{xy})^2+6\left(\tau_{xy}^2+\tau_{yz}^2+\tau_{zx}^2\right)\right]\right)\end{matrix}} - A \end{aligned} \tag{6}$$

where: A and B stand for material constants, while I_1 and I_2 are the first and the third invariants of stress tensor, and J_2 is the second invariant of stress deviator.

2. Plastic Dilatation

Plastic dilatation *e,* in the case of associated plastic flow rule, is represented in the form of the following formula [4]:

$$de = d\varepsilon_x{}^p + d\varepsilon_y{}^p + d\varepsilon_z{}^p \tag{7}$$

Aftereer calculating, according to formula (2), strain growths in the directions x, y and z, then adding them up and ordering them, one obtains the following form of dilatation for Yin's condition:

$$de = d\lambda\frac{1}{\sqrt[3]{A_1^2}}\begin{pmatrix} 3BA_2+\sigma_x\sqrt[3]{A_1^2}+ \\ +_y\sqrt[3]{A_1^2}+\sigma_z\sqrt[3]{A_1^2}\end{pmatrix} \tag{8}$$

where A_1 is:

$$A_1 = 9\begin{pmatrix} 6\sigma_x\sigma_y\sigma_z+6\tau_{xy}\tau_{yz}\tau_{zx}-6\sigma_x\tau_{yz}^2-6\sigma_y\tau_{zx}^2-6\sigma_z\tau_{xy}^2-3\sigma_z\tau_{xy}^2-3\sigma_z\tau_{zx}^2-3\sigma_z\tau_{yz}^2- \\ 3\sigma_y\tau_{xy}^2-3\sigma_y\tau_{yz}^2-3\sigma_x\tau_{xy}^2-3\sigma_x\tau_{zx}^2-\sigma_{2x}^2-\sigma_y^2-\sigma_z^2 \end{pmatrix} \tag{9}$$

while A_2 equals:

$$\begin{aligned} A_2 =& -\sigma_y^2\sigma_z+13\sigma_x\tau_{zx}^2-\sigma_x\sigma_y^2-\sigma_x\sigma_z^2-\sigma_x\sigma_z^2-18\tau_{xy}\tau_{yz}\tau_{zx}+22\sigma_x\tau_{yz}^2+13\sigma_x\tau_{xy}^2+13\sigma_y\tau_{yz}^2+13\sigma_y\tau_{xy}^2+ \\ &22\sigma_y\tau_{zx}^2-\sigma_y\sigma_x^2+13\sigma_z\tau_{yz}^2+22\sigma_z\tau_{xy}^2+13\sigma_z\tau_{zx}^2-\sigma_z\sigma_x^2+4\sigma_x^3+4\sigma_y^3+4\sigma_z^3-24\sigma_x\sigma_y\sigma_z \end{aligned} \tag{10}$$

Since A_1 is in the second power under a cube root, the expression: $\sqrt[3]{A_1}^2$ will be always positive. $d\lambda$ is a non-negative number, so plastic dilatation will be greater than or equal to zero only in the case when:

$$3BA_2 + \sigma_x \sqrt[3]{A_1^2} + \sigma_x \sqrt[3]{A_1^2} + \sigma_x \sqrt[3]{A_1^2} \geq 0 \quad (11)$$

i.e. after the following transformation:

$$A_2 \leq \frac{1}{3B}\left(\sigma_x \sqrt[3]{A_1^2} + \sigma_y \sqrt[3]{A_1^2} + \sigma_z \sqrt[3]{A_1^2}\right) \quad (12)$$

After factoring the expression $\sqrt[3]{A_1}^2$ out, one obtains:

$$A_2 \leq \frac{\sqrt[3]{A_1^2}}{3B}\left(\sigma_x + \sigma_y + \sigma_z\right) \quad (13)$$

On the right side of inequality (13) there appears the sum of stresses $\sigma_x + \sigma_y + \sigma_z$, which is equal to the first invariant of stress tensor I_1 or the tripled value of the mean stress $\sigma_{śr}$ (also called hydrostatic pressure), so the above relationship can be written as the following formulas:

$$A_2 \leq \frac{\sqrt[3]{A_1^2}}{3B} I_1 \quad (14)$$

or

$$A_2 \leq \frac{\sqrt[3]{A_1^2}}{3B} \sigma_{śr} \quad (15)$$

The material constant B for rock-salt and for brick and mortar assumes positive values [2,3], so in the case of uniaxial tension, at $\sigma_x \neq 0$ and $\sigma_y = \sigma_2 = \tau_{xy} = \tau_{yz} = \tau_{zx} = 0$, plastic dilatation, according to formula (8), equals:

$$A_1 = -\sigma_x^3 \quad (16)$$

$$A_2 = 4\sigma_x^3$$
(17)

$$de = d\lambda \frac{1}{\sqrt[3]{\left(-\sigma_1^3\right)^2}}\left(12B\sigma_x^3 + \sigma_x \sqrt[3]{\left(-\sigma_1^3\right)^2}\right) \quad (18)$$
$$= d\lambda\sigma_x(12B + 1)$$

Checking the inequality from formula (13), after necessary transformations and simultaneously applying the equation $(-\sigma_x)^2 = (\sigma_x)^2$, one sees that:

$$A_2 \leq \frac{\sqrt[3]{\left(-\sigma_x^3\right)^2}}{3B}\sigma_x = \frac{\sigma_x^3}{3B} \quad (19),$$

so **B** must be less than or equal to 1/12. Looking closely at relationship (18), we can additionally observe that even when B=0, we have a positive value of plastic dilatation in the case of uniaxial tension for growths in the main strains associated with Yin's plasticity condition.

Positive plastic dilatation means growing volume at uniaxial tension, which is of course contrary to experience. The increase in the volume of the material can be confirmed by calculating strain values ε_y^p and ε_z^p for uniaxial tension straight from the definition of plastic flow rule associated with Yin's destruction criterion, according to relationship (2):

$$d\varepsilon_y^p = d\varepsilon_z^p = \frac{1}{3} d\lambda\sigma_x\left(2 + \sqrt[3]{3^2}B\right) \quad (20)$$

On the basis of formula (20), one can ascertain that for associated plastic flow rule with Yin's plasticity condition, in the state of uniaxial tension, there actually occurs material swelling, since the relationship is always positive for the material constant B≥0. The results that follow from the flow rule are unconfirmed physically. One must remember that in experimental analyses there appears a certain increase in the volume of the sample, but it is only due to the formation of new scratches and cracks [4].

It is possible to consider Yin's criterion in the space of the state of stress invariants. Starting from the form of the criterion found in literature [2]:

$$K_2 = A + BI_1\sqrt[3]{K_3} \quad (21)$$

and replacing K_2 and K_3 with corresponding relationships in the form of stress tensor invariants I_1, I_3 and the second invariant of stress deviator J_2, one obtains Yin's formula, consistent with formula (5).

After transposing part of the expression to the right side, the condition assumes the following form:

$$J_2 + \frac{I_1^2}{6} - A = BI_1 \sqrt[3]{I_3 + I_1 I_2} \qquad (22)$$

Then, after raising both sides of the equation to the third power, the formula becomes converted to the following relationship:

$$\begin{aligned} & J_2^3 + \frac{1}{2} J_2^2 I_1^2 - 3AJ_2^2 + \frac{1}{12} J_2 I_1^4 - \\ & - J_2 I_1^2 - J_2 I_1^2 A + 3J_2 A^2 + \\ & + \frac{1}{216} I_1^6 - \frac{1}{12} I_1^4 A + \frac{1}{2} I_1^2 A^2 - \\ & - A^3 = B^3 I_1^3 I_3 + BI_1^4 J_2 \end{aligned} \qquad (23)$$

The relationship (23) allows defining the third invariant of stress tensor I_3 as a function of the other two invariants I_1 and I_2:

$$I_3 = \frac{1}{BI_1^3} \left(\begin{aligned} & J_2^3 I_1^2 + \frac{1}{2} J_2^2 I_1^2 - 3AJ_2^2 + \\ & + \frac{1}{12} J_2 I_1^4 - J_2 I_1^2 A + 3J_2 A^2 + \\ & + 3J_2 A + \frac{1}{216} I_1^6 A^2 \frac{1}{12} I_1^4 A + \\ & + \frac{1}{2} I_1^2 A^2 - A^3 - BI_1^4 J_2 \end{aligned} \right) \qquad (24)$$

After simplifying the analysis to the plane expressed only by the invariants of stress tensor *(I_1, I_3)*, the functional relationship is reduced, at J_2=0, to the following formula:

$$I_3 = \frac{1}{BI_1^3} \left(\frac{1}{216} I_1^6 - \frac{1}{12} I_1^4 A + \frac{1}{2} I_1^2 A^2 - A^3 \right) \qquad (25)$$

It is a cubic function, which cannot assume the value equal to $I_1=0$. It means that in the no-load state, for which we have a stress tensor composed exclusively of zeros and marking the point located at the beginning of the coordinate system of the state of stress, the plasticity function in the form of (25) loses its physical sense.

This situation is illustrated graphically in Fig.1.

Fig. 1. Yin's plasticity condition on plane (I_1, I_3) at J_2= 0.

Yin's plasticity condition can be also presented on the plane of the state of stress (I_1, J_2). The form of such functional correlation, at I_3= 0, is illustrated in Fig. 2.

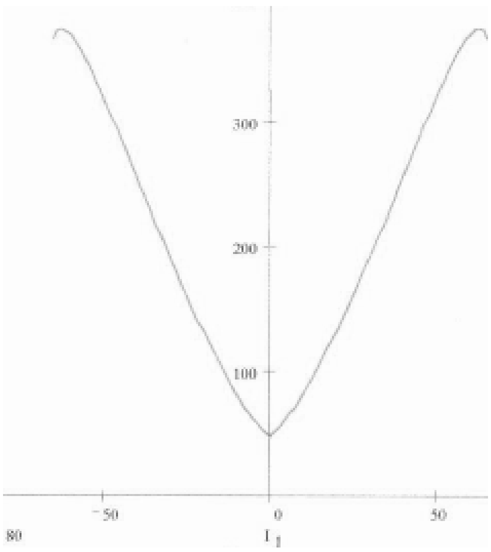

Fig. 2. Yin's plasticity condition on plane (I_1, J_2)

Yin's condition, on the plane defined by the third invariant of stress tensor I_3 and the second invariant of stress deviator J_2, is virtually independent of the first variable, since the functional correlation $J_2(I_3)$ is constant and its ordinate is equal to the material parameter A, irrespective of the assumed I_3 values.

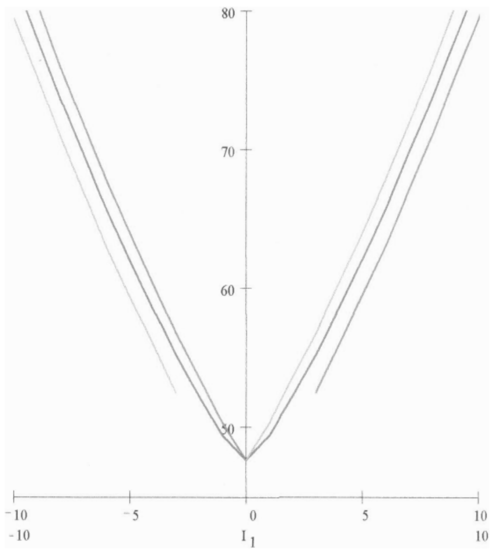

Fig. 3. Yin's plasticity condition on plane (I_1, J_2), depending on the value change of invariant I_3; $J_2(I_1) – I_3 = 0$, $J_{2b}(I_1) – I_3 = 100$

3. Conclusions

The obtained correlations make it possible to state unmistakably that for associated plastic flow rule with Yin's plasticity condition, in the state of uniaxial tension, there occurs an increase in the material volume that is physically unjustified and contradictory to experimental analyses.

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Hygrothermic Prediction of the Profiled Sheet Sandwich Panels of Polyurethane Rigid Foam

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Rezumat: Lucrarea prezintă un studiu privind analiza câmpurilor termic şi de presiuni, pentru două panouri tip sandviş, cu fețe din oțel profilat şi cu miez din spumă poliuretanică rigidă, de grosime diferită. Studiul higrotermic al acestor panouri implică stabilirea riscului de condensare în sezonul rece, prin determinarea câmpurilor presiunilor de saturație şi efective a vaporilor de apă. S-a constatat că mai eficiente, din punct de vedere higrotermic, sunt panourile cu miez gros, care îmbunătățesc confortul şi durabilitatea clădirii.

Abstract: The paper presents a study concerning the thermal and pressure fields considered for two sandwich panels of steel sheets and polyurethane core of variable thickness. The hygrothermic study of these panels involves the condensation risk evaluation during the cold season, by determining the fields of saturated and effective vapour pressures. One can observe that the thick core panel is more efficient from the hygrothermic point of view, improving the comfort and the stability of the building.

Keywords: *sandwich panels, field of saturated vapour pressure, condensation zone.*

1. Introduction

The sandwich panels carry out a good heat insulation and represent structures made up of two profiled sheets between which a polyurethane rigid foam is injected. These panels are used in civil and industrial constructions as fastening elements to the walls and the roofs. The hygrothermic study of these panels involves the condensation risk evaluation during the cold season, by determining the fields of saturated and effective vapour pressures.

2. Physical and Mechanical Properties

The sandwich panels are made up of two relatively light, but resistant sheets, between which there is a core of variable thickness. The sheets can be of stainless steel, of smooth (Al99.5) or corrugated (STUCCO) aluminium, of galvanized steel (SENDZIMIR) or of an Al-Mg alloy. The thickness of sheet varies between 0.4... 1.2 mm, function of the material type.

The polyurethane of rigid core has the following physical and mechanical characteristics:

- density 39±10% Kg/m^3;
- traction strength 0.14 ÷ 0.17 N/mm2;
- compression strength 0.14 ÷ 0.17 N/mm2;
- adherence at metal supports 0.2 N/mm^2;
- dimensional stability ± 0.9% at 80°C and ±2.5% at 100°C.

The thermal and acoustic characteristics refer to:

- the heat transfer coefficient λ=0.020W/m°K;
- the fire resistance: C1 and C2 combustibility class;
- the optimum waterproofing, 90% closed cells;
- the index of sound insulation, 25 dB at least.

3. The Analysis of the Thermal and Vapours Diffusion Field

The financial support for this scientific investigation was provided by the INCO-COPERNICUS program and "Gheorghe Asachi" Technical University of Iasi. Several studies concerning the analysis of thermal and vapour diffusion fields have been developed, by using the RDM computer software, because it can be easily observed that the mathematical models for the heat transfer by conduction and the vapour diffusion are of identical form [1,2,3].

In the case of a virtual condensation zone, it is necessary to establish the real dimensions of the phenomenon. This zone contains all the points of the field in which the partial pressure of water vapours is grater than the saturated vapour pressure, calculated taking into account the thermal field. In order to establish the real condensation zone, the following iterative methode has been used:

- an equality between the partial and the saturation pressure is imposed in the point where the difference between the partial pressure of the vapours and the saturated one is at its maximum;
- a new comparative analysis of the vapours partial pressure field and saturated pressures is carried out and then, the point of maximum difference is imposed;
- the analysis follows these steps until there are no more condensation points.

4. Case Study

Numerical simulation

The simulation procedure on two sandwich panels steel sheets and a rigid polyurethane foam core of 40 mm and 80 mm thickness, respectively, has been used.

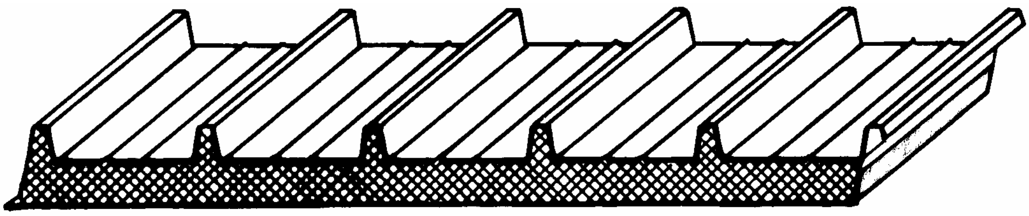

Fig. 1. Sandwich panel

The thermophysical characteristics are presented in the table below:

Table 1

Material	Thermal conductivity, λ (W/m/°C)	Factor of vapour permeability resistance, $1/K_D$
Stainless steel	58	∞
Polyurethane rigid foam	0.02	30

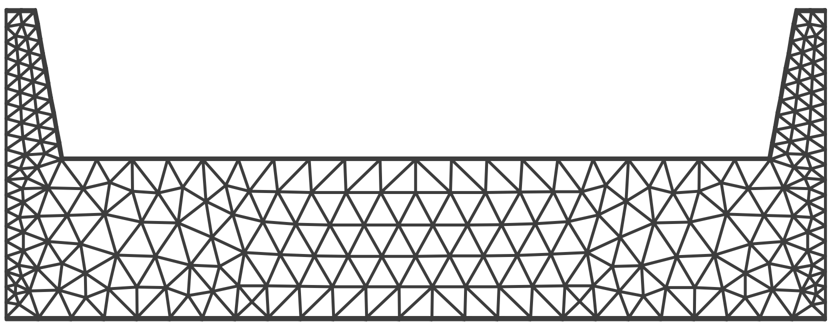

Fig. 2 Triangular mesh discretization

The first simulation concerns the thermal field analysis, by determining the temperature θ in each point of the mesh. The saturation water vapour pressure p_s is obtained by the relations (1):

$$P_s=610.5 \exists 2.718282^{(21.875 \exists \theta) / (265.5+\theta)}, \text{ for } \theta>0$$
$$P_s=610.5 \exists 2.718282^{(17.269 \exists \theta) / (273.3+\theta)}, \text{ for } \theta<0 \qquad (1)$$
$$610.5 \exists 2.718282^{(21.875 \exists \theta) / (265.5+\theta)},$$

and boundary conditions are:

α_I = 8kcal / m^{2}h°C, θ_i = 20°C, α_e =24 kcal / m^2°C and θ_e=-15°C, taking into account the convective heat transfer. The boundary conditions in the case of the diffusion field analysis are: p_i=1170 Pa, p_e=140 Pa, corresponding to the interior temperature θ_i=20°C, the exterior temperature, respectively θ_e= -15°C, the interior humidity φ_I=50% and the exterior one, φ_e=85%.

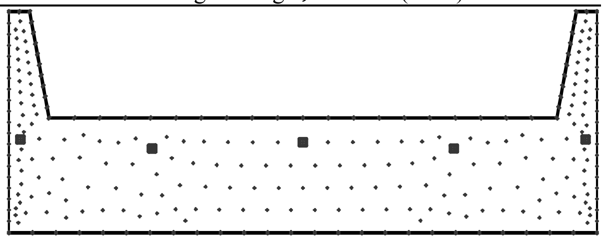

Fig. 3 Real condensation area

The real condensation area is represented in figure 3 and the values of equal pressure are plotted in figure 4.1 (d=40mm core thickness) and figure 4.2 (d=80mm core thickness). It can be seen that the condensation zone is much closer to the junction, in the case of the polyurethane rigid core of 40 mm thickness, which influences the durability of the core and the adherence of the component layers of the panel.

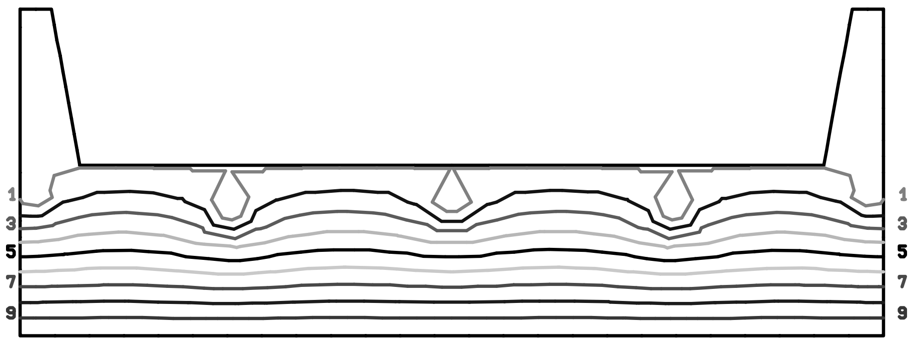

Fig. 4.1 Pressure field (core thicknes =40 mm)

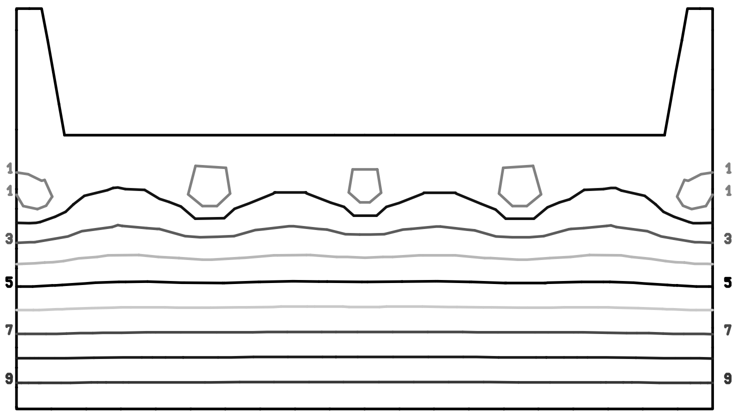

Fig. 4.2 Pressure field (core thickness = 80 mm)

4. Conclusions

The thermal and pressure fields are presented for two sandwich panels of steel sheets and polyurethane core of variable thickness. One can observe that the thick core panel is more efficient from the hygrothermic point of view, improving the comfort and the stability of the building.

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Contributions to the development and testing of a super mortar ferro-cement

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Rezumat: Lucrarea prezinta contributiile autorilor la realizarea si testarea unui ferociment cu super mortar. Prezentarea generala a ferocimentului ca material si a specificitatii supermortarului este urmata de detalierea modului de armare si de realizare efectiva a compozitului. In final se prezinta cateva caracteristici mecanice.

Abstract: The submitted paper presents the author's contributions in a super mortar ferro-cement development and testing. After a general view on ferro-cement as a material, the effective layout of the armature and composite itself are presented, together with some mechanical characteristics issued from the tests

Keywords: ferro-cement, super mortar, armature.

1. General

A ferro-cement is a composite material made mainly from a steel mesh armature in a mortar matrix. The armature for ferro-cement is usually a square welded mesh of small diameter steel wire (about 1-1.5 mm) and square size of about 15-20mm. The mesh layers are applied in sequence, either alone or in combination with thin steel reinforcement rods (4-6 mm) to form the armature, which is eventually compacted to ensure a minimal thickness.

The mortar matrix is usually made from Portland cement, silica sand and water, mixed all together. If we talk about a "super mortar", the Portland cement is substituted by a synthetic resin, usually polyester or epoxy.

The finished and faired armature is then plastered, usually by one side, the armature's high degree of compactness rendering impossible the use of a closed mold of any kind. After cure (about 28 days for ferro-cement or 14 days for ferro-cement with "super mortar") the result is a structure which can take any shape, is watertight and presents excellent mechanical properties.

The object of our study [1] was a ferro-cement with "super mortar", the components and their mixing ratios being entirely original.

Considering the high degree of novelty of the material, extensive experimental tests were performed, in order to explore as deep as possible it's physical and mechanical properties.

The interest for ferro-cement with super mortar or super ferro-cement is that it over passes some of the drawbacks of the Portland ferro-cement.

First, the cracking behavior is far better, the armed mortar possesing a higher first crack stress (the polyester resin being more plastic than the Portland cement). Also, the natural cracking of Portland mortar caused by the waste of the mixing water, is totally out of question.

Second, the cure of the mortar is by far reduced (14 days compared to 28 days) and more important, the plastering of the armature can be interrupted and continued whenever is necessary. The plastering with Portland cement is a delicate and stressing event, any delay or malfunction (electrical blackout, water shortage etc.) having as consequence the total loss of the structure beeing built.

Third, the natural porosity of the cured Portland mortar, porosity which permits that the sea water penetrates the outer surface until it reaches the armature, thus enhanceing the chances of galvanic corrosion, it totally absent. The super mortar is absolutely watertight and moreover, it has a very strong adhesion to the armature's rods.

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2.The experiment: Main steps in the super ferro-cement manufacturing process

***The armature*:** The choice we made was for a welded square seel wire mesh, the square being 19mm and the wire diameter 1.5mm. The wire mesh pannels (Fig.1) are assembled together in several layers in order to achieve the necessary scantlings. (Fig.2).

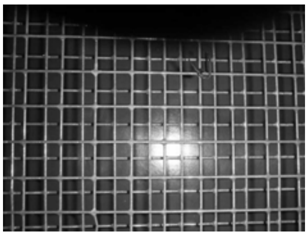

Fig.1. Typical mesh pannel

Fig.2. The assembling of the mesh pannels in succesive layers

The super mortar : The mortar matrix is composed of marble dust and a polyester resin (ENYDYNE D26-526 TAE), the mixing ratio beeing 1 part resin/1.6 parts marble dust (in volumes). The specifications for resin and marble dust are presented in Fig.3


CRAY VALLEY information technique

ENYDYNE® D 20-526 TAE
Résine polyester insaturé

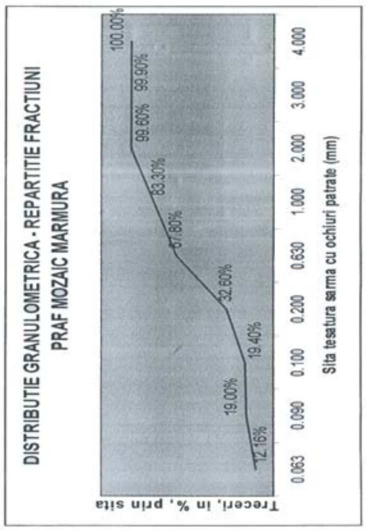

Fig.3. Technical specifications: (above -resin; below-marble dust)

The plastering: This step was performed manualy using an enclosable mold (Fig.4), after plastering the mold's top beeing pressed in order to achieve an uniform and minimal thickness (Fig.5).

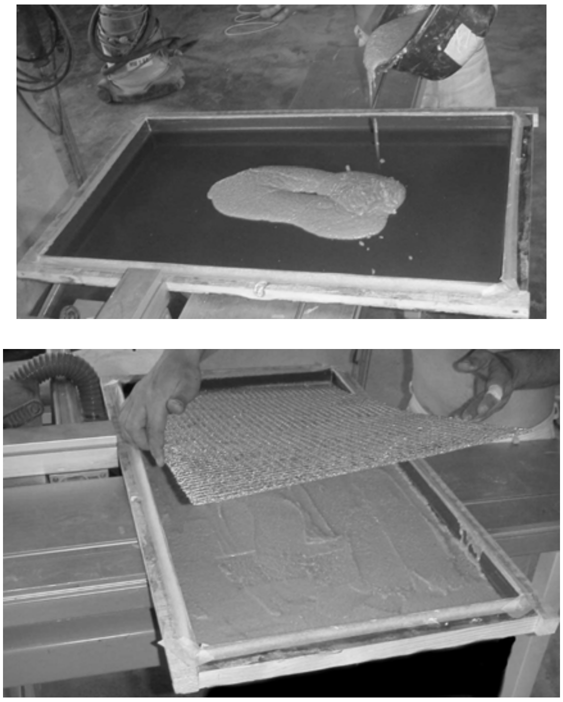

Fig.4. The plastering process
(above-the pouring of the mortar;
below-the insertion of the armature)

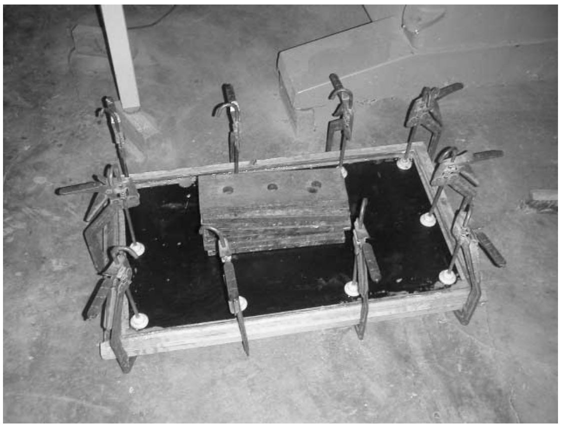

Fig.5. The pressing of the closed mold
during the cure

After the achievement of the resin's gel time, the mold's top is removed and the demolded composite is left for complete cure (about 14 days).

3.Results and interpretation: Some mechanical characteristics of the tested super ferro-cement

Among other tests, the bending behavior was also explored, using the loading scheme and the strain gauges location [2] showed in Fig.6. The tests were run on a tensometric adjustable test bed, device of an original conception and a domestic manufacture (Fig.7.). The test schedule included dedicated tests for the "super mortar" and for the composite itself.

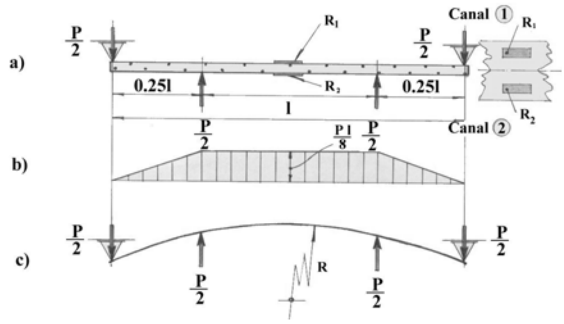

Fig.6. Loading scheme and strain gauges location

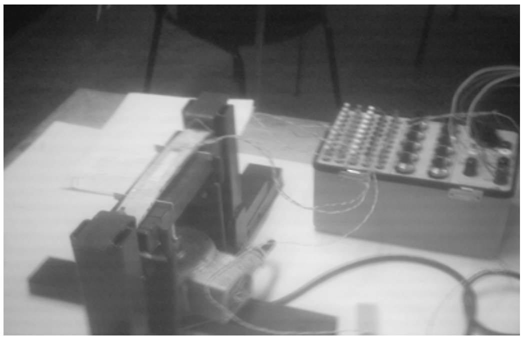

Fig.7. The tensometric adjustable test bed

The results were interesting and somehow surprising. First, the "super mortar" showed a typical bending behavior, found usually in the case of "consolidated" in the flexi-plastic field materials (two different but constant flexural modulus in succession Fig.8.).

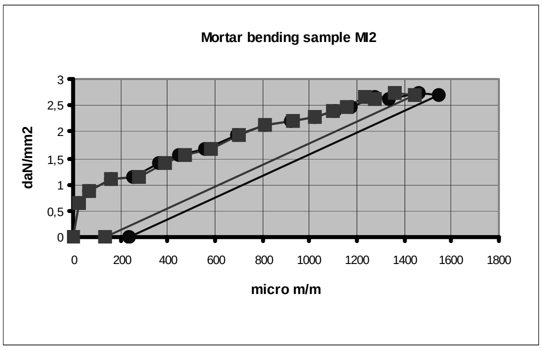

Fig.8. Stress / strain curve for mortar bending

Second, the composite itself proved a surprisingly bending (without major cracking) capacity in the plastic field, both the flexi and remnant deflections being of a very high value (Fig.9.). This behavior suggests a high shock absorbing capacity, a very desirable item for a boat's hull structure.

Fig.9. Composite sample showing considerable deflection without cracking

Third, the stress/strain curve for the composite itself approaches the same curve for the mortar alone, Fig.10 showing also an interesting behavior when the test sample is loaded and unloaded in succession.

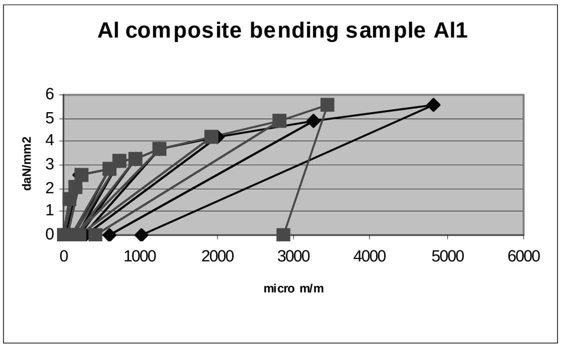

Fig.10. Stress / strain curve for composite bending loaded and unloaded in succession

4.Conclusions

The super ferro-cement as resulted is an interesting material, with good capabilities in the boat building branch and also in other fields were the light, thin and resilient structures are required.

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Nondestructive Testing Methods for Masonry Structure

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Rezumat: Metodele traditionale de evaluare a conditiei si a proprietatilor structurilor din zidarie sunt metode destructive. metodele destructive de evaluare sunt limitate deoarece mostrele indepartate pot provoca pagube estetice cat si culturale. Folosirea metodelor nedistructive poate furniza inginerului structurist, care evalueaza zidaria, informatii utile. Utilizarea metodelor nedistructive necesita folosirea a doua sau mai multe tehnici complementare.

Abstract: Traditional evaluation methods for the condition and properties of masonry features of buildings have been destructive testing. Destructive methods of evaluation are inherently limited becouse specimen removal may be aesthetically and structurally damaging. The use of nondestructive techniques can provide the structural engineer, who is charged with evaluating of the masonry, invaluable information. The utilisation of nondestructive techniques calls for use of two or more complimentary techniques.

Keywords: *nondestructive evaluation, flatjack.*

1. Rebound hammer

Measurement of masonry surface hardness is conducted using a specially designed impact hammer that measures the rebound of a mass from the tested surface to indicate masonry condition and unit material properties. The method is useful for preliminary nondestructive examinations for locating poorly constructed or deteriorated areas.

1.1 Equipment.

The Schmidt hammer consists of a hardened steel plunger with a spring-actuated mass. The unit is placed against the masonry surface and the spring is released, driving the mass against the masonry. The mass rebounds from the surface and the rebound distance are read off an arbitrary scale.

1.2Procedure.

Most surface hardness investigations will be conducted by laying out a grid-work and taking readings at regular intervals. It is important that test areas be chosen to capture the range of materials variations throughout the structure. The test requires 10 separate readings from single impacts in chosen area.

2. Probe penetration

Probe penetration tests are used extensively for indicating strength of in-place concrete. The technique is applicable to evaluation of masonry units and mortar and provides information on material property and condition.

2.1 Equipment

Probe penetration equipment is portable and compact, consisting of a pistol-type apparatus that directs a powder-actuated charge toward the probe.

2.2 Procedure

Test location is chosen carefully to represent the normal variation in material properties and condition throughout the structure being investigated. The device is pressed firmly against the masonry surface at the desired test location. The trigger is pulled, firing the explosive charge. Probes must remain embedded in the masonry after firing. The depth of penetration is measured using a micrometer. Test results are analyzed by measuring penetration depths.

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3. Pullout test

Existing and installed masonry anchors are tested to determine tension and shear capacity by applying loads with hydraulic jack.

3.1 Equipment

Pullout testing requires a hydraulic loading ram and pump, reaction frames and devices for measurements of applied loads and displacements.

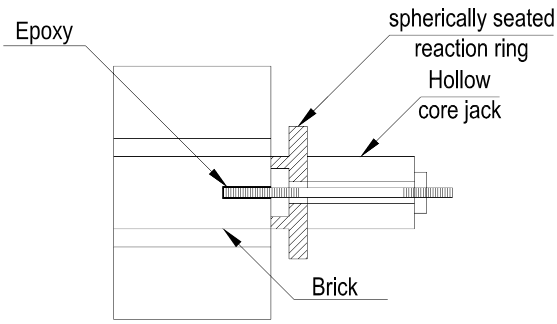

Fig. 1 Test aparatus for pullout of embedded anchors

3.2Procedure

Two different procedures are used:

- one for determination of proof loads for instaled anchors and ties;
- a second method is used for indicating masonry unit properties using pullout strength of epoxied anchors.

4. In-place push test

The in-place shear test, or push test, is used widely accepted method for determining masonry shear strength.

4.1 Equipment

The in-place push test is simple to conduct and requires only the most basic equipment:

- Rotary drill and masonry bits for mortar removal
- Calibrated hydraulic ram
- Pressure gage
- Hand pump

4.2 Procedure

At the chosen test location it is necessary to isolate the test joints. A single unit on onr side of the test unit is removed for placement of the hydraulic ram. The head joint on the opposite side of the unit also must be cleared. A hydraulic cylinder is inserted into the space and shimmed to center point of load application on the brick end.a steel bearing plate is placed between the jack and the test unit. The test method requires that the load at first movement of the test unit be recorded as failure load.

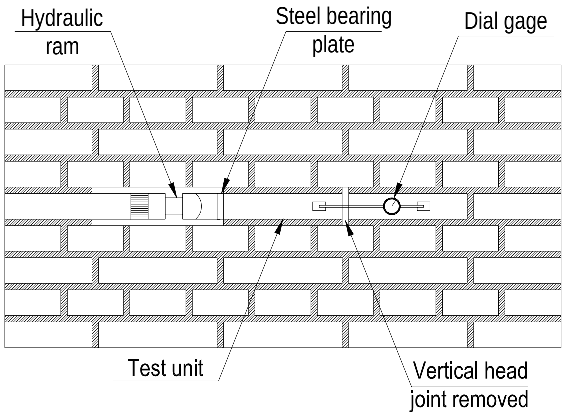

Fig 2 Test setup for the in-place shear test

5. In-place push test conducted with flatjacks to control vertical stress

The in-place push test has been related to overall masonry shear strength and provides the most rational basis for determination of design shear values currently available.

5.1 Equipment.

The in-place push test is simple to conduct and requires only the most basic equipment:

- Rotary drill and masonry bits for mortar removal
- Flatjack with appropiate dimension to fit ia a head joint for application of shear loads
- Deformation measurement dimension
- Two hydraulic pumps

5.2 Procedure.

The test equipment is instaled in the configuration shown in Figure 3, with two parallel flatjacks separated by five courses of brick masonry. The unit for shear testing should be located on middle course and centered between two flatjacks. This approach requires removal of only two head joints, one on either side of the test unit.
The initial loading cycle is conducted with zero pressure in the vertical stress flatjacks by increasing pressure in head joint flatjack. the load at the first movement of the test unit is recorded and privides the base shear strength.

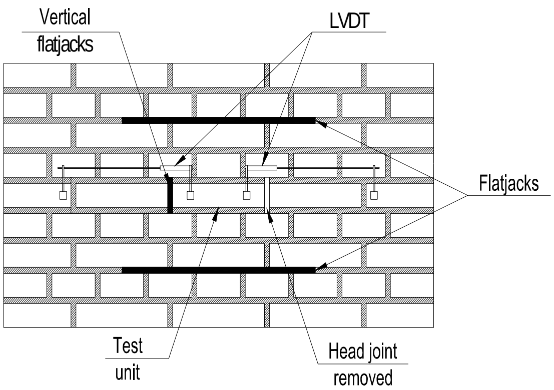

Fig 3 The modified in-place shear test

6. In-place stress test

The test is based on a simple principle of stress relief and measures the stress in the masonry from dead loads, any acting live loads, thermally induced stresses, or load transfer resulting from shrinkage of building frames.

6.1 Equipment:

- Rotary drill and masonry bits for mortar removal
- Flatjack
- Deformation measurement dimension
- Hydraulic pumps

A flatjack is at thin steel envelope sealed at the edges and fitted with ports, allowing internal pressurization using hydraulic equipment.

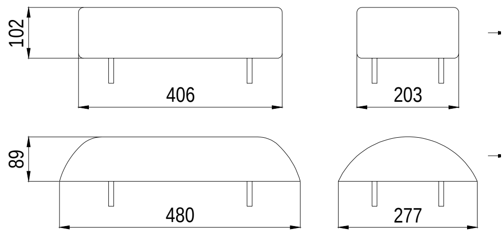

Fig 4 Types of flatjacks

6.2Procedure

The technique for the in-place stress test requires initial displacement measurements to be made before removing mortar from flatjack slot. In general, three to ten separate lines of gage points are used. Following mortar removal from the flatjack slot, another series of measurements are made. Flatjack is inserted and pressurized until the original separation between the gage points has been restored, the internal hydraulic pressure of the flatjack is recorded.

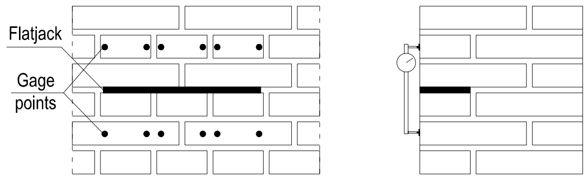

Fig. 5 In-place stress test

7. In-place deformability test

Two parallel flatjacks are used for the in place deformability test.

7.1 Equipment:

- Rotary drill and masonry bits for mortar removal
- Flatjacks
- Deformation measurement dimension
- Hydraulic pumps

7.2Procedure.

The in-place deformability test uses two flatjacks, separated vertically by several courses of masonry, to subject the masonry to compressive stresses. The flatjacks are pressurized, deformation measurements are taken and recorded for each stress level. If damage to the masonry is acceptable, the test may be carried to failure.

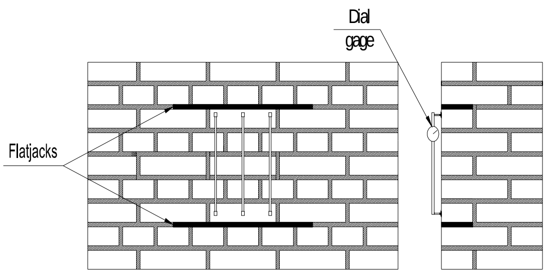

Fig. 5 In-place stress test

8. Pulce velocity

Measures the time of travel of a pulse or train of waves through masonry to determine the uniformity, to indicate changes in masonry properties, or to survey structures to estimate the severity and extent of deterioration, cracking, and voids.

9. Impact echo

Measures the depth of reflecting surface for a stress wave generated by mechanical impact. Detects and delineates internal discontinuities in masonry and, with interpretation, identifies the nature and orientation of discontinuities.

10. Infrared thermography.

Uses selective infrared frequencies to identify heat patterns characteristic of certain defects. Has been used to locate bands of mortar dropping, voids in solidmasonry walls, and location of wall ties.

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Moderns Methods for Rehabilitation and Consolidation of Bricks Buildings

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Rezumat: Cel mai frecvent material întâlnit la constructiile civile, este cărămida. De aceea edificiile construite din zidărie de cărămidă, constitue adesea un interes deosebit pentru constructori, mai ales pentru că acestea trebuie sa suporte tot mai multe transformări structurale, nemaicorespunzând cerințelor de rezistență, confort sau funcționalitate. Prevenirea degradării constructiilor trebuie făcută încă din faza de proiectare, continuând cu cea de exploatare. Umiditatea este una din principalele cauze ale dezagregării zidăriei, implicit a degradării ei. Se prezintă un sistem de restaurare-asanare a zidăriei, bazat pe injectare de materiale microsiliconice, care intra in structura peretelui. Alte cauze ale degradării construcțiilor sunt seismul, greşeli de proiectare sau execuție. Pentru consolidarea acestora se va prezenta o metodă modernă, care foloseşte materiale composite polimerice armate cu fibre de carbon, sticlă sau aramid, saturate cu răşini epoxidice.

Abstract: The most used materials in civil construction were the burnt clay bricks. The buildings are very old and so, they need to support permanent structural changes.
The prevention of buildings debasement is recommended from the designing stage, and must continue through the service period.
Humidity is one of the principal causes of the degradation and loosing properties of the masonry. We present a restore and sanitation system which is a modern procedure to a complete dampness remove.
Other causes of building's masonry degradation are: earthquakes, unequal settles, design and execution errors.
According to these there are new consolidation techniques, using polymeric composite materials reinforced with carbon, glass or aramid fibers, saturated with epoxidic resins.

Keywords: *masonry degradation, bricks, rehabilitation, polymeric composite materials*

1. Introduction:

Till now, in our country, the most used materials in civil construction were the burnt clay bricks. The buildings are very old and so, they need to support permanent structural changes due to:
- seismic noncompliance
- changing of function ability
- intervention of perturbations phenomenon
- necessity of new structural loadings reckoning

2. The importance of rehabilitation:

There are two fundamental aspects in maintenance and rehabilitation of the civil buildings that take our interest: to prevent and to heel.

Figure 1

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The prevention of buildings debasement is recommended from the designing stage, and must continue through the service period.

The necessity of consolidation and rehabilitation of a building appear when that building is not according to:

- evaluate the bearing capacity in current status
- estimate repairs to be made
- choose and detail the rehabilitation method according to a project

We can't establish perfect directives or rules about the opportunity of a building consolidation or demolition. Currently it rehabilitates. Anyway, there are five fundamental stages to repair a building:

- discover degradation
- determine causes which were producing these degradation
- evaluate the bearing capacity in current status
- estimate repairs to be made
- choose and detail the rehabilitation method according to a project

We must take notice beforehand about the building debasement and we have to know that the accessible parts are protected meanwhile the inaccessible parts are ignored. The most important and most difficult stage is to determine the causes of degradation. For this stage there are no rules or methods, each case has a particular diagnose. The building must be examined and studied with lot of patient to observe and discover the hidden or latent abnormalities.

Humidity is one of the principal causes of the degradation and loosing properties of the masonry (fig.2).

According to this, the degradation of masonry is happening from these points of view:

Figure 2.

In consequences, to a brick building we have to protect both structures, supra and infrastructure (fig.2).

3. Modern procedure of complete dampness remove:

Further, we present a restore and sanitation system which is a procedure to a complete dampness remove.

This procedure is used when prevention of falling in disrepair caused by disastrous effects of humidity could not be done. It is a heeling method.

Using this system's about doing a damp course or a protective coating which:

- stops the ascending of water due to materials capillarity
- blocks water on foundations level (figure 3)

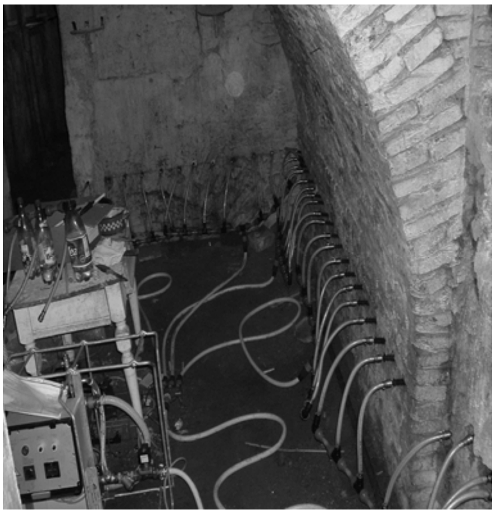

Figure 3.

You're able to maintain masonry in good conditions by injecting the Murisol solution. After you pickle the damp surface, you must apply an absorbing plaster. These plasters have the quality of absorbing humidity and minerals from the structure and permitting so called "breathing" of the masonry.

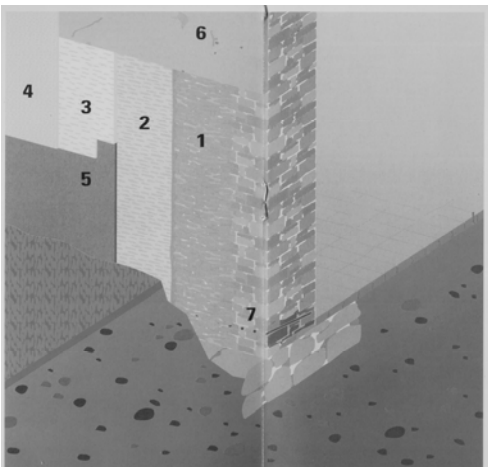

Figure 4.

Legend for figure 4:

1- Murisol VS plaster
2- Murisol GP
3- Murisol SP fein/weiss
4- Hydrophobic plaster
5- Murisol DS/BD1K/BD2K
6- Old plaster
7- MicroMurisol horizontal injection

Are made from micro silicone synthetic resins and belongs to masonry or stones structures.

Finally, we can use a silicone, silicate or mineral paint applied over the Murisol plaster.

Other causes of building's masonry degradation are:

- Earthquakes

Figure 5.

- unequal settles
- design and execution errors

Figure 6.

In these cases, first we must evaluate the situation and then we must achieve a consolidation project and choose an intervention method on the structure.

The expert and the structural engineer have two options; they can choose a classical method of rehabilitation which is cheap but needs long work time, or a modern procedure, new consolidation techniques, using polymeric composite materials reinforced with carbon, glass or aramid fibers, saturated with epoxidic resins. These materials presents like thin sheet, lamella or strips in case of carbodur and have the following properties:

- compression strength compare to steel
- specific weight 40 times less than steel

As a result of laboratory work the conclusion is that consolidated masonries by using polymeric composite materials are more strength than others and have very good ductility, with a very high energy absorbing capacity.

4. Modern procedure of consolidation:

Further we present the polymeric composite materials rehabilitation technique, which means:

- deep pickle of the surface
- even up all irregularities of the substrate followed by leveling with an adhesive coating
- apply an epoxidic resin seal or layer (fig.9).

Figure 7.

- inclusion in this layer of the carbodur strips, till the resins penetrate it, so the surface turn to white.
- in the case of malposition of the strips the surface completely turn to black (fig.7)

The advantages of using this system are:

- a very good proportion between material, time and work
- the structure, so the foundations too are not overloaded

Figure 8.

- flexible materials
- no casing needed
- can be used to a multiple constructive solutions
- confers high durability, corrosion and tiredness resistance
- social advantage due to not have to evacuate lodgers

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Factors Inducing the Damage Processes of Limestones

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Rezumat: In lucrarea sunt prezentați principalii factorii care determină procesele de alterare a calcarelor din elementele de construcție. În funcție de modul de acțiune, aceştia sunt intrinseci şi extrinseci. Factorii intrinseci sunt în funcție de textura şi structura rocii. Factorii extrinseci acționează din exterior asupra rocilor: temperatura, atmosfera, umiditatea. Analiza acestora este însoțită de exemplificări şi imagini ale calcarelor degradate din unele monumente realizate în Cluj-Napoca. Având în vedere urmările proceselor de alterare – degradare prezentate, lucrarea se încheie cu câteva măsuri şi soluții de prevenire a acestora.

Abstract: In this paper are presented the main factors, which determine the damage process of limestones in the construction elements. Depending on the way these factors act, they can be internal factors and external ones. The internal factors depend on the rock's texture and structure. The external factors act from the exterior on the rocks, such as temperature, atmosphere and humidity. Their analysis comes together with exemplifications and pictures of damaged limestones, from monuments in Cluj-Napoca. Considering all the consequences of the damaging processes presented, the paper ends with some measures and solutions to prevent them .

Keywords: *internal factors, external factors, damaged limestones.*

1. Introduction

Subjected to the stress of aggressive factors, natural stone changes its quality, either by losing color or by strength losses. The damaging action of aggressive factors is different from one rock to another depending on its mineralogical composition, on the forming history and also on its structure.

Therefore, solid rocks, made of minerals hardly altered resist very well to the aggressive action of the environment, while bullate rocks, made of altered minerals do not resist to the aggressive activity of the environment.

The aggressive factors that act over the natural stone can be physics, chemicals and biological factors. The most frequent physics factors that stress the natural stone are water, air, wind and temperature.

The destructive process caused by water can be explained in the following way: because of the cracks, pores or capillarity the water penetrates the mass of the stone and trough the volume increasing of the stone due to temperatures variations, axial stresses appear that lead to stone exfoliation.

Another way the water damages the stone is through the crystallization of salts solved in water, that lead also to exfoliation.

The chemical aggressive action is dued to the presence in the atmosphere of carbon dioxide and sulphur dioxide, compounds of acid character that attack the basic mineralogical components of the stone, creating sloved salts, washed by water. This phenomenon is called corrosion.

The aggressive action of microorganisms is dued to the deposits in layers of atmospheric powders on horizontal surfaces, which getting in contact with water makes the vegetal life real. This gives the layer an acid character, the loam acids are forming; these also attack the basic mineralogical components of the stone therefore the phenomenon of corrosion appears.

The factors that govern the alteration process of limestones can be internal or external ones.

The internal factors depend on the rock's texture and structure and also on its mineral compounds.

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The external factors act from the exterior on the rocks, such as temperature, atmosphere, humidity, geometrical shape and processing level.

2.Internal factors

2.1.The texture and structure of the rocks

The chemical, physics and mechanical characteristics of the rocks used as construction materials are being influenced by the microstructure.

The disposition of the minerals inside the rock respects a certain rule depending on the conditions in which the acumulation has been realized.

The eddy currents deposit in a non uniform way sedimentary material, causing changes of texture and microstructure. Precipitation from solutions forms inside the rock accumulations which collect impurities during transportation and deposit.

The effect of these processes is materialized in the limestone used for different monuments in a nest shape or in colored calcspar moduluses using iron hydroxides.

The currents that caused the transportation of mineral granules give the massive texture or horizontal layers uniformly disposed.

A tense location of deposing imprints criss-cross stratification. In the case the sedimentation plan is being changed a tilted stratification will result.

In the thickness of the layers that form the rock might appear mineralogical differences.

These differences cause modifications of the physical-mechanical characteristics.

Being in contact with environmental factors, the rock acts different. Figure 1 presents such a case: limestones with different texture and structure on various layers.

These limestone will have different degrees of degradations for each layer. The limestone belongs to the Saint Michael Cathedral [1] from Cluj-Napoca.

Disposing the rock in a different position than the primary orizontal stratification can cause degradation processes like the ones in Fig. 2, Reformed Church [1] from Cluj-Napoca, Kogalniceanu Street, the southern face.

Fig.1. Limestone with different texture and structure on various layers

The different degradation of the limestone is accelerated in the case the limestone blocks [2] are layed vertically.

Fig. 2. Degradation of limestone blocks vertically layed

Each litic system has its own evolution particularities. For instance, a building made of the same building materials has different degradation–alteration forms, such as: lobes, roundedness, material losses, exfoliations, breaking in stairs or flakes [3].

2.2. Porosity

Depending on the formation way, the pores can be primary (formed after the storage of the rock) and pores formed during the alteration process.

According to the diameter, the pores can be: micropores (< 0,002 μm), medium pores (0,002-0,05 μm) and macropores (> 0,05μm). The space filled by pores represents the medium where the alteration reactions take place and also where the dissolution and precipitation of solutions come in contact with minerals belonging to the rocks occurs.

The space filled with pores and cracks becomes the host of fluids and gases from the environment where the material is located.

The humidity is dependent on the porosity. Depending on the water occupation degree of pores we have dried rocks, humid rocks and saturated rocks [3]. The dried rocks have pores with insignificant water content.

The humidity causes the decrease of mechanical resistances, the circulation of solutions through capillary pores, the acceleration of the dissolution processes, the swelling of clayish rocks and other effects.

2.3. Capillarity

The spaces that communicate between them cause water ascent dued to capillary pressure. The diameter of capillaries is so much smaller as the the rock's granules are also very small. Inside the pores that have a diameter $\Phi < 0{,}1\mu m$, the psysics effects are predominant and inside the pores that have a diameter $\Phi > 0{,}1\mu m$, the chemical-physics effect are predominant.

The convex meniscus have the tendency to brake the liquid ascent. Together with water ascent, salts are been lifted on the whole height, with water variations and concentrations that vary in opposite direction than humidity.

The bottom end (with much more water) creates lower concentrations that grow with height, where the diameter of capillaries drops, the vaporization is getting higher and the humidity also drops [3]. The capillaries's short space favors quick volatility, which bring us to oversaturation. This is the reason why on the building's socle becomes visible a 0.5 m-1 m light color belt. Here is where the salts mark the superior end of capillaries.

The bottom area of the socle is being altered through chemical dissolution of the binding granules. The superior area is where the crystallization-dissolution processes take place and will end with the dislocation of the plaster.

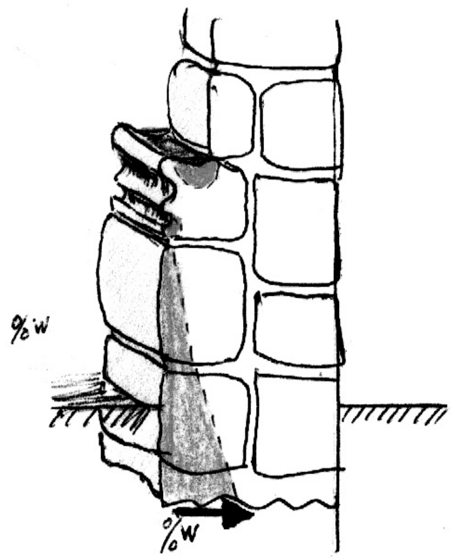

Fig. 3. The humidity distribution upon the socle's height

Inside the rocks with large pores, the water presents quick movements. In narrow pores, the water hardly moves causing water accumulation inside capillaries. This water supplies the large pores with salts, where is the center of crystallization from solutions or crystallization through frost.

The rocks with large pores have the crystallization front located on the surface. Therefore, material dislocations appear at the surface in small thicknesses.

The rock with many capillaries has the crystallization front placed inside the depth of material. The degradation process is manifested by the detachment of larger fragments (Fig.4.) during salts crystallization.

The percent of deterioration prodused compared with the rock's surface (stairs flakes) is predominate on the southern façade.

The experience showed that for the same amount of NaCl, the limestone with larger pores is less altered than the limestone filled with small diameter capillaries. (1996, Fitzner) [3].

Fig.4. Limestone with small diameter pores – massive dislocations from rock mass, at Saint Michael Cathedral, Cluj-Napoca

3. External factors

3.1. Temperature

Increasing the temperature with 10°C may accelerate the alteration processes up to (2-2.5) times. (1970, Millot). The temperature changes influence the linear dilatation of the minerals. When temperature rises, the hydrogene ion concentration increases in the water. At 0°C the water's pH is 7,5, at 50°C the water's pH becomes 6,5. (Moller, 1998).

Negative temperatures have a double effect:

-ice dilatation, which becomes the source of mechanical stresses that triggers the formation of cracks;

-freezing the salty liquors within pores and capillaries, producing the separation of NaCl from other salts;

The polimineral rocks become the nest of unequal mechanical stresses. Mechanical pressures caused by dilatation are in a range of hundreds and thousands of daN/cm^2. These pressures produce the cracking and splitting the grains and cement in which they are included.

The air currents are modeled by city architecture and controlled by tall buildings. These currents cause a preferential deposit of particles in certain areas, modifying the local temperature as well. (Ciulache, 1976).

3.2. Atmosphere

One of the external factors is the atmosphere. In the mountain air, the dust reaches a minim of 400-700 particles/m^3, on the field areas it reaches up to 130000 particles/m^3.

In the cities it can reach up to 470000 particles/m^3. The preservation of monuments is tightly tied to the presence of solube sulphur compounds. The limestones are the most vulnerable. The SO_3 chemical group imprints an acid- aggressive behavior, resulting a coarse surface. Dust particles will lay on this surface.

Thus, light colored sulphates will become gray-black colored, phenomenon called the black scab effect. Simultaneous, iron salts form, which diffuse in liquors and migrate through material pores. These minerals, gathered in pores and capillaries determine the appearance of eflorescences and the black scab.

In 1980 a study has been conducted which showed that 1000 cars send out daily 3,2 tone on CO_2 in the atmosphere, that may stay unchanged for many years [3]. Since industries appeared until now, 330 billion tones of CO_2 have been sent out in the atmosphere, changing its composition with at least 14%. (Ciulache, 1980).

The processed surfaces and carved details are chemically active. These are the areas of sulphate accumulation and forming of the black scab.

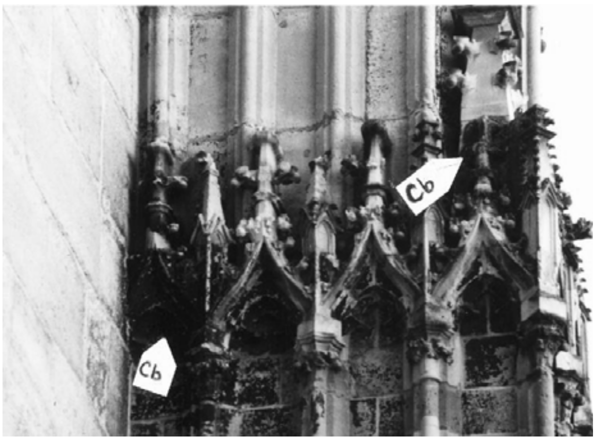

Fig. 5. Architectural ornamentation elements on which the black scab lays

3.3. Humidity

Humidity is the most important weathering factor. The places in which humidity may appear become chemical weathering and physical damage zones.

The humidity depends on the rock's structure, texture and geometrical shape, on the number of faces exposed to external factors, on the distance from the source and also on the orientation of the interest zones according to cardinal points.

3.4. Geometrical shape and Processing level

The bigger the rock's volume is, the processing level is reduced and the microcrackes are fewer. Therefore, the resistance to deterioration is higher.

As a result, the ashlar stone is more resistant than the cut-stone.

According to this reason, the sculptures are the most exposed to degradation. They have the highest processing level and so, a bigger number of microcracks.

The carved details become a braking source for air currents, determining a transition to whirly microcurrents, generating a longer lasting contact with air masses loaded with aggressive agents.

The microcracks and whirly currents accelerate the forming process (Fig. 6.) of the black scab.

Fig.6. The forming process of the black scab

The foundation's elevations base zones of the majority of old buildings are belt shaped-a semicylindrical belt bent towards the wall.

In these areas the rain water gathers and the salts and pollutant substances using capillary systems are being dissolved [4].

The buildings's orientation towards the cardinal points influences the storage of salts in the rock's pores, water gathering so all the degradation processes are being accelerated.

The southern facades under the influence of solar radiation are exposed to a more intense vaporization process, determining high salt concentrations, visible in efflorescences.

4. Conclusions

Any restoration work of a building starts with the analysis of the building's material state.

If the appraisal is shallow, after a few years the processes of plaster exfoliation, of salts's expansion as well as the appearance of the black scab on highly processed surfaces will appear.

The person incharge of the reconditioning of the building must have knowledge of history, architecture, archeology, art history and of course, civil engineering.

In order to reduce the effect of these aggressive agents, constructive measures are being proposed or stone treatments [5].

Some of the constructive measures are: the use of stones with appropriate properties, avoiding horizontal or slightly oblique surfaces, using hydroinsulations.

Protecting the stone by treating its surface, after the construction work is done, can be made by [2]:

- covering with a layer of organic chemical compounds, realizing a protective film, or plating the pores- in the presence of SO_2, a aqueous liquor of barium chloride;
- limestones are treated with fluates-magnesium silico fluoride $MgSiF_6$ and aluminium silico fluoride $Al_2(SiF_6)$;
- treating the surfaces with a salty liquor that reacts with the aggressive chemical agent neutralizing it.

The protective film obtained by painting with drying oils, diluted in a volatile solvent, so that the oil can reach the stone's pores, is resistant and impermeable, but the film alters the natural aspect of the stone.

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Techniques and Instruments Used in Construction Project Management – The Project Breakdown Structure

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Rezumat: Structura de descompunere a unui proiect de construcţie reprezintă o metodă de dezagregare, într-o manieră ierarhică a unui proiect în grupuri structurale foarte diferite ca natură şi complexitate, având la bază o serie de considerente cum ar fi: complexitatea proiectului, etapele de realizare ale acestuia, factorii implicaţi în realizarea lui, diversele variante tehnologice şi organizatorice de realizare, resursele folosite, cultura managerială a celor implicaţi în realizarea proiectului.

Abstract: The breakdown structure of a construction project is a hierarchical decomposition method of a project, in structural groups very different in nature and complexity, based on: project complexity, performance stages, the factors involved, various technological and organizing ways, resources, the managerial culture of the persons involved in the project.

Keywords: *project management, construction management.*

1. Introduction

While realizing a construction project, five main decomposition structures stand out as follows:
1.The decomposition structure oriented on the project's realization stages (SDE)
2.The decomposition structure organization oriented (SDO)
3.The decomposition structure resources oriented (SDR)
4.The decomposition structure constructive systems oriented (SDS)
5. The decomposition structure activities oriented (SDA).

2. Description

1. The decomposition structure oriented on the project's realization stages (SDE)

SDE suggests dividing the construction project in its realization stages by also highlighting their content (studies, documentations, projects, etc.) (Figure 1)

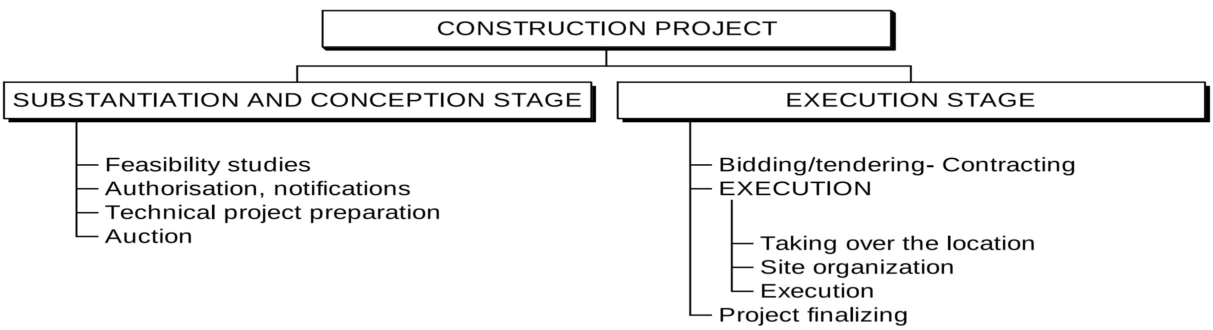

Figure 1. The decomposition structure oriented on realization stages

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2. The decomposition structure organization oriented (SDO)

This structure's purpose is identification of the groups and organizations involved in the realization of the construction project establishing at the same time the hierarchical relations and the responsibility for the realization of some parts of the project.

The main objective of SDO is to establish "who does what", "who is responsible" during the realization of a project.

Considering the characteristics of the construction projects and the factors involved in their realization four main types of organizational structures are derived as follows:

a. Operative organizational structure – that specifies the operative groups or the organizations involved in the realization of the project. A sample of this type of structure is presented in Fig. 2.

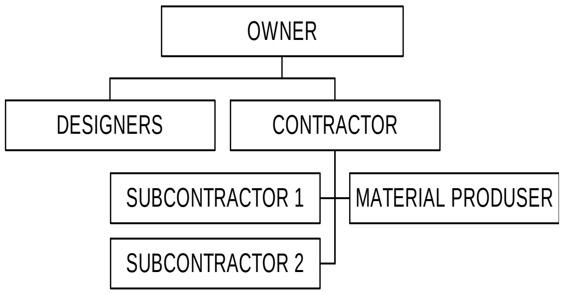
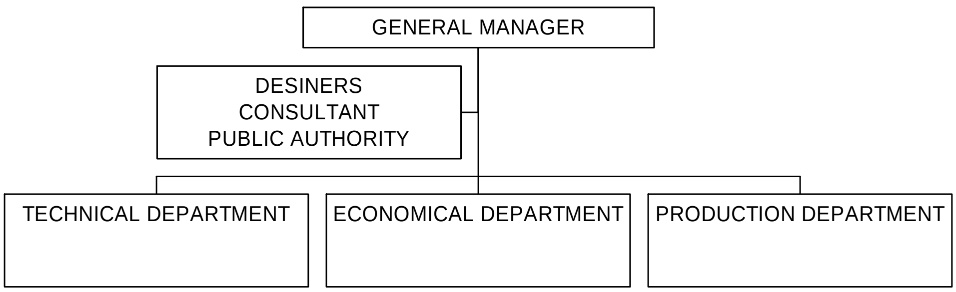
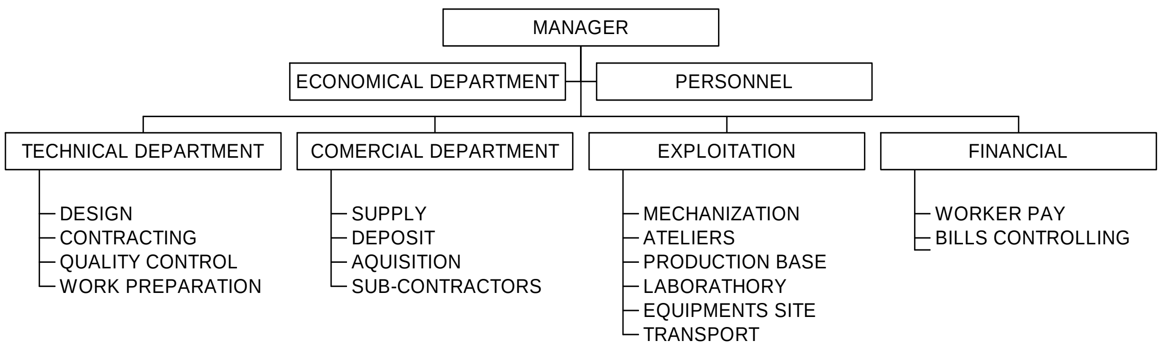


Fig. 2 Operative organizational structure

b. The intrafunctional organizational structure – presents the involvement and establishment of the hierarchical relationships in the departments of the organization involved in the realization of the project. Figure 3 presents an example of such a structure for two main organizations involved in the realization of the project:



a. The organization that benefits of the project – THE BENEFICIARY



b. The organization that applies the project – THE CONSTRUCTOR

Fig.3 The intrafunctional organizational structure *c. The organizational structure projects oriented* – has as main object establishing the responsibilities and the hierarchical relations of the representatives of all the organizations involved in the realization of the project. An example of such a structure is given in Figure 4.

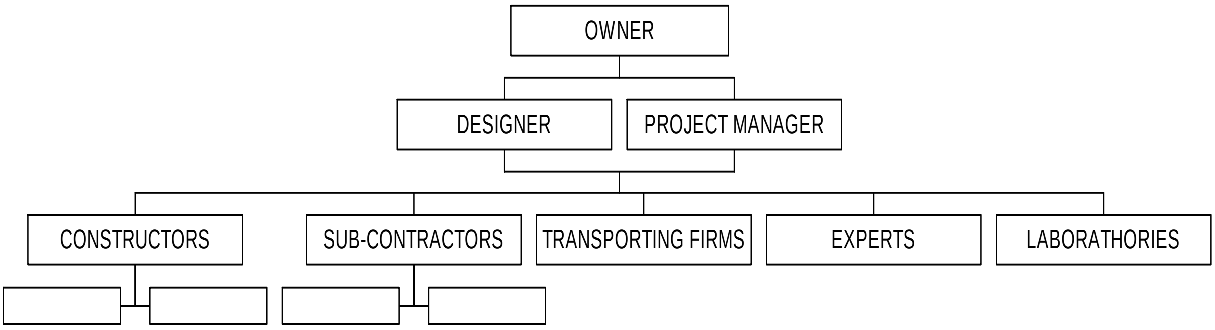


Fig. 4. The organizational structure project oriented

d. The hybrid organizational structure – is a mixed complex structure made of combinations of the three organizational structures from above.

3. The decomposition structure resources oriented (SDR)

This structure aims at identifying the main resources necessary for the realization of the project and their structuring according to nature and characteristics. We may establish therefore the following main resources categories involved in the realization of a project (Figure 6): human resources, material resources, technological equipments resources, financial resources, information resources.

The share and types of resources are influenced by a great amount of productive factors necessary for the realization of the project.

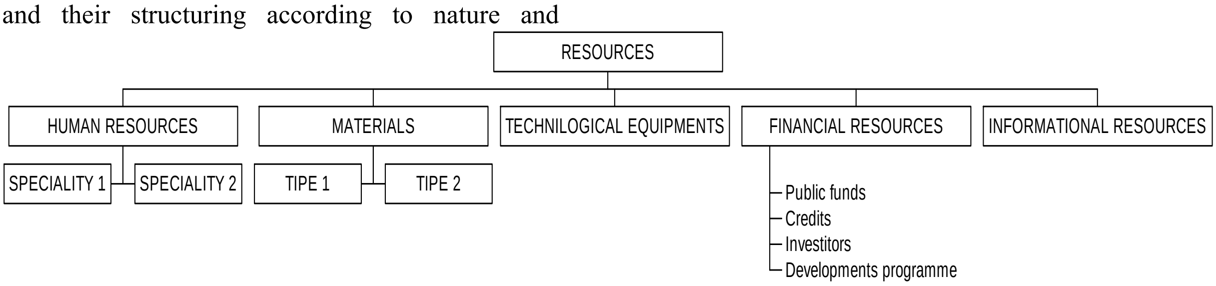


Fig. 6. The decomposition structure resources oriented

4. The decomposition structure constructive systems oriented (SDS)

This structure aims at dividing the project in its component systems (elements) in view of reaching a common chart for choosing, conceiving, projecting, execution different types of realization of the project as well as assigning the responsibilities for each type.

The dividing of levels into constructive systems can be presented as follows:

Level	Name	Content
1.	CONSTRUCTION OBJECT	Stands for that part of a construction project spatially defined with a certain function. The realization of every construction object is based on the technical project adherent to the object by which the technical parameters are established (form, dimensions and constructive structure). Examples: Building block, Heating station, Transformation point, etc.
2.	CONSTRUCTION ENSAMBLE	Stands for that part of a construction project established according to structural and functional reasons or according to stages in the realization of a component construction object (infrastructure, superstructure, appliances etc.)
3.	CONSTRUCTION ELEMENT	Stands for all the construction processes materialized in that part of a construction object that has a certain function (foundations, basement walls, platforms, closures, lifting walls, etc.)

SDS presents in a tree form, whose detailed levels stand for a clearer description of the objects, sets and elements of construction (Figure 7).

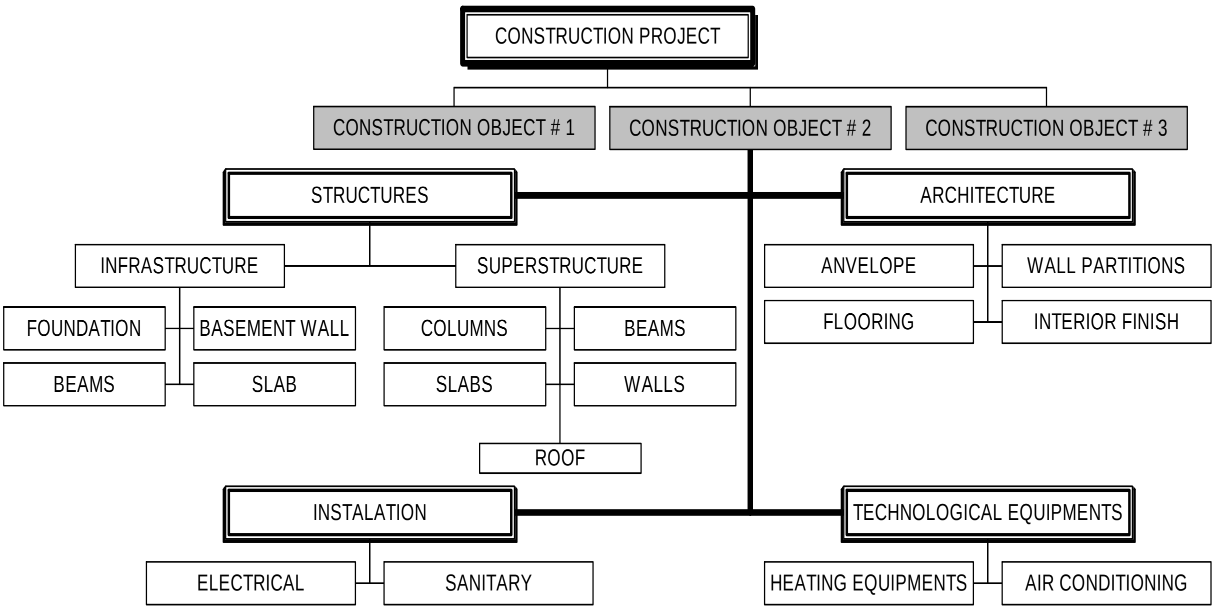


Fig. 7 The decomposition structure oriented on the constructive systems

5. The decomposition structure activity oriented (SDA)

SDA defines an arborescent net, which structurally presents the activities necessary for the realization of a construction project.

The activity (component of a construction project) represents a set of tasks, actions, construction processes established and linked according to technical, organizational, administrative, legislative reasons which use time and resources.

Ways of establishing the component activities of a construction project:

- the name of the activity should be clearly formulated (a maximum of 30 characters);

- the name of each activity must begin with an action (concrete casting in foundations, masonry execution, project elaboration in view of obtaining the construction licence);
- so that it may be quantified (amount of processes, realization time, costs, beginning time, finishing time);
- the categories and quantities of resources necessary for its realization should be easily established (no. of workers, quantities and types of material, times of equipment functioning, etc.);
- the organizational and technological methods of execution should be easily established;
- conditioning relations between the component actions and with other activities should be established;
- so it can be easily monitored, directed and assessed;
- to allow a clear description and representation of the component construction actions and processes as well as the correlation between them;
- encoding of activities should be made in a easily extendible size allowing for its division into subactivities, actions, construction processes, phases, events;
- some quality requirements should be established and secured;
- a person responsible (an accomplisher) should be associated to it;

Figure 8 presents a pattern of structuring the construction project on activities:

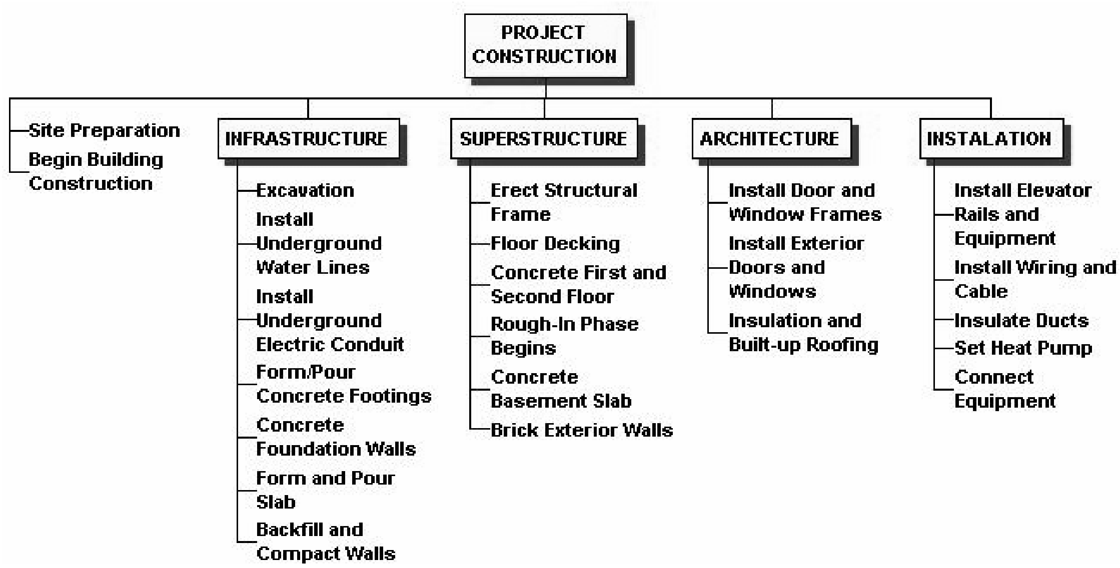


Fig. 8 The decomposition structure activity oriented

3. Technical instruments suggested by SDP

Two technical instruments are suggested by SDP development:

1. The technical record of the activity (TRA)
2. The technical flow-chart (TFC)

1. The technical record of the activity (TRA) is a technical instrument that derives from the SDP development which mainly aims at ensuring technical support in a structured and unitary manner for the information and the specific parameters for each activity or element mentioned in SDP.

The main types of information specific to an activity (element) from SDP in view of its realization and assessment can be thus structured:

- General information
- Technical-organizational information
- Information regarding quality
- Other information

2. The technical flow-chart (TFC) – is an instrument derived from the SDP development which mainly aims at representing the factors involved in realizing a construction project as well as their responsibilities on well defined hierarchical levels starting with the project as a whole and ending with making up the working teams for the realization of the actions specific to one activity.

Based on TFC the questions "who does what?" and "who is responsible and for what?" are answered.

Three ways of presenting the TFC may be highlighted starting with the components of the decomposition structure of the project:

a. TFC resulted from the presentation of the project's stages (SDE) – implies identification and assigning of responsibilities on the stages of realization of the project (Fig.9).

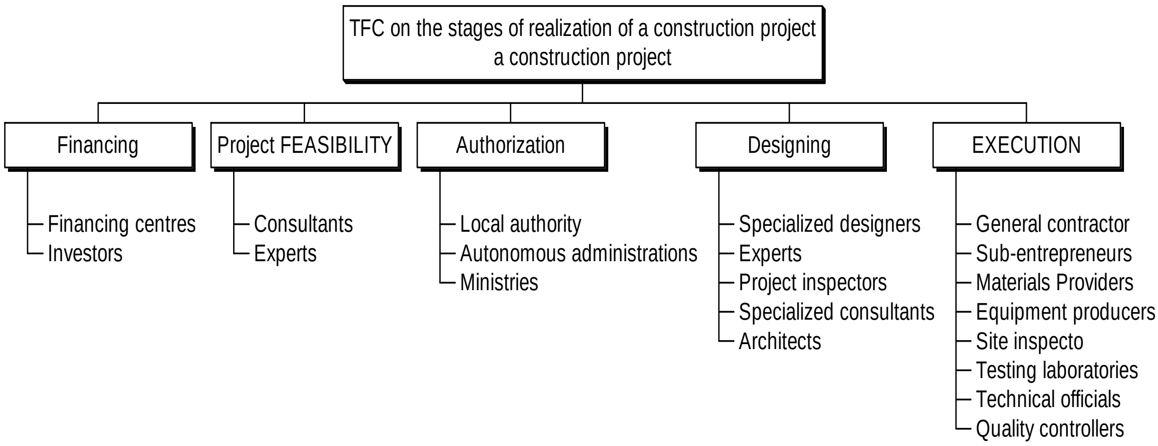


Fig. 9. TFC oriented on the realization stages of the project

2. TFC resulted from dividing the project into constructive systems (SDS) – implies identification of the factors involved in conceiving, projecting and realizing the component constructive systems (objects, construction sets, technological equipment, etc.).

3. TFC resulted from dividing the project into activities (SDA) – implies identification and assigning of responsibilities for the factors involved in the realization of the component activities of a construction project.

4. Conclusions

SDP suggests and develops a structured hierarchical approach of a construction project allowing for the visualizationof all its component parts; identifies the factors involved in the application of the project and the hierarchical relations between them; provides technical support for the conception of the way of development in time and space for the execution of the construction project; suggests and develops a structured support for the technical, economical, legislative information regarding quality in the realization of the project; suggests two instruments necessary for the development of a construction project: the technical activity record (TAR) and the technical flow-chart (TFC).

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Techniques and Instruments Used in Construction Project Management - Promac Methodology -

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Rezumat: Datorită transformărilor economice din țara noastră precum şi a necesității de adaptare rapidă la cerințele unei economii de piață, se observă o cerere imensă de informații şi mai ales de tehnici, instrumente, metodologii care să faciliteze şi să eficientizeze activitatea managerială din orice domeniu.

În cazul activității de construcție problema se pune şi mai acut datorită aspectelor specifice precum şi a sistemului tradițional de informare care este foarte rigid, formalistic şi birocratic, ceea ce îngreunează procesul de adaptare a specialiştilor la realitățile economice.

Abstract: Due to the economic changes in our country and the necessity of quick adaptation to the requirements of a market economy, a great need for information can be noticed, especially of techniques, instruments, methodologies which can facilitate and make management more efficient the in any field.

In the construction field the issue is even more crucial due to its specific aspects and the traditional informational system that is very strict, formal and bureaucratic, which makes the accommodation process of the specialists to the economic realities even more difficult.

Keywords: *project management, construction management.*

1. Introduction

Specific aspects of the construction projects:

- the realization of a construction project involves a great amount of resources that differ both in nature and quantity.

- a long time for execution which results from the great amount of construction processes as well as economic, technical and administrative actions;

- it involves a great number of natural and legal persons who play different parts (beneficiaries, consultants, producers and materials and equipment providers, technical experts, entrepreneurs, inspectors, etc.);

- each construction project has a unique character with special characteristics such as: destination, importance, functionality, method of usage, specific technical, economical and qualitative aspects, etc.

- the realization of a construction project is made in an unknown predictable future.

2. The visualized objectives

In this context, taking into account the above-presented aspects and the characteristics of the construction business, PROMAC methodology aims at achieving the following objectives:

- it presents the realization stages of a construction project and their contents

- it presents a series of techniques and instruments for:

➢ establishing the realization costs of a construction project
➢ establishing the realization time (for all constituent stages) and the component activities
➢ establishing the types and quantities of resources necessary for the realization of the project and the constituent processes
➢ establishing and realizing the expected quality level
➢ directing, coordinating and controlling the realization of the project.

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- it presents data and establishes the responsibilities of the factors involved in the realization of the project;
- it presents a structured level (on stages and activities) concerning the main parameters in the realization of the construction project (cost, duration, quality);
- it presents an information support in a unitary and structured manner on a series of technical, organizational, economic, legislative, qualitative, administrative, leading and controlling aspects;
- it suggests and develops a technical-mathematical and managerial pattern which could establish the best realization methods concerning cost, time, resources and quality requirements;
- it suggests and develops a series of realization programs for a construction project taking into account the realization stages and the main factors that take part in their carrying on, of which we mention:
  - The preliminary general program for the construction project realization (PGP)
  - The execution program for the construction project (PEP).

3. The pattern adopted by PROMAC methodology

The pattern adopted by PROMAC methodology suggests and develops asset of methods, procedures, technologies that coordinately specify the real actions, the human and material means, the various execution stages and methods in order to successfully realize a construction project in certain imposed conditions (a certain cost, a certain time and a certain quality level).

The pattern is structured on three functional blocks as follows:

I. THE PATTERNING BLOCK
II. THE ESTIMATE BLOCK
III. THE OPERATIONAL BLOCK

Figure 1 presents the general diagram for the pattern adopted and developed by PROMAC methodology.

I. THE PATTERNING BLOCK – outlines a series of specific information and elements connected to the general description of the construction project (setting the objects), its decomposition into activities which are very different in nature and complexity as well as the representation of the conditionings between them by graphic nets with the purpose of conceiving the best way of developing in time and space.

The content of the stages from the patterning block perspective:

STAGE 1. – General characteristics, structuring of objectives

It is the stage which expresses the beneficiary's requirements and needs related to the construction project. It is also during this stage that a series of aspects are being developed such as:

- General aspects – which present the project's coordinates: name, destination, place, importance scale, site, substantiation and necessity of realization theme;
- Technical aspects – by which the component construction objects are established and described (new buildings or already existing ones rehabilitated structurally, functionally, thermally, etc.); the dimensions and form of the objects, of the construction elements and of the installation equipment are established; information about the main materials and equipment employed, about their consequences and effects on the environment is given;
- Economical aspects – which give information on the financial sources necessary for the realization of the project like: personal financial sources (economic agents – individual or state-dependant, natural persons) and acquired financial sources (sums invested by natural or legal persons, bank credits, financing programs). A series of issues related to capital and material expenses connected to the realization of the project are also presented;
- Trading aspects – which hint at the nature and quality of the materials and equipments employed in the realization of the project;
- Managerial aspects – which hint at choosing and coordinating the factors involved in the project. Among the main factors ("actors") which take an active part in the realization of the project are: investors / beneficiaries, architects, engineers, technical experts, consultants, specialists, general constructor / entrepreneur, producers and providers of materials, technological equipment, working force, technicians, unions, insurance societies, local

institutions (The Local council Board, Construction Inspection), utilities administrations (water, sewers, gas, electric power, telephone, etc.), transportation firms, testing laboratories, entertainment institutions, inspections (environment, work protection, etc.).

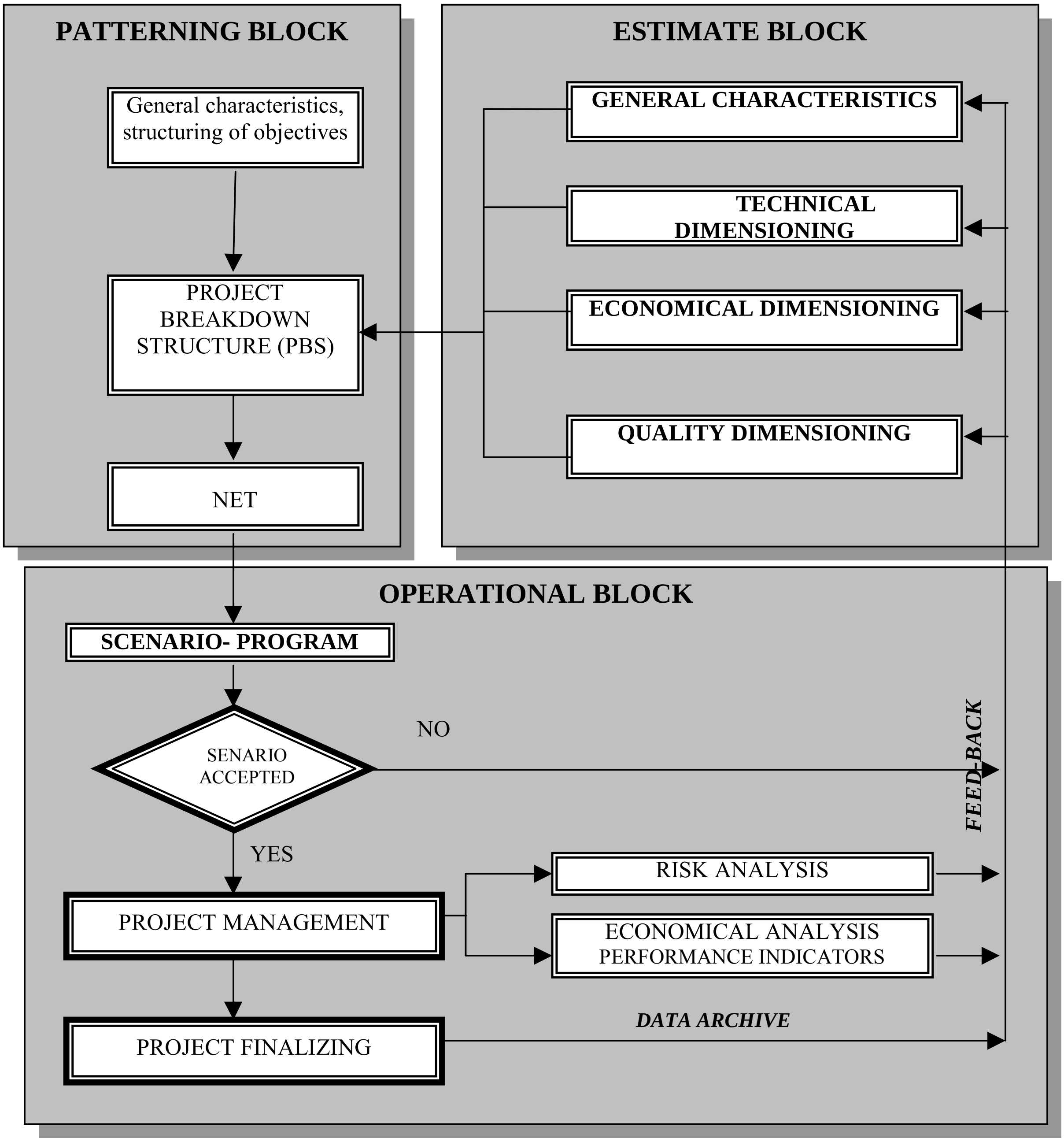

Fig.1 – Adopted model in PROMAC Methodology

STAGE 2 – The project breakdown structure (PBS)

The project breakdown structure is a disintegration hierarchical method of a project into structural groups which are very different in nature and complexity, based on a series of characteristics such as: complexity, realization stages, involved factors, various technological and organizational ways of realization, resources, the managerial education of the people involved in the realization of the project.

Thus, five main decomposition structures of a construction project can be highlighted as follows:

- The decomposition structure oriented on the realization stages of the project (DSS);
- The decomposition structure oriented on organization (DSO);
- The decomposition structure oriented on resources (DSR);
- The decomposition structure oriented on constructive systems (DSCS);
- The decomposition structure oriented on activities (DSA).

STAGE 3 – Net

The net can be described as being a graphical representation of the component actions of a construction project by establishing the correlations between them so that one may step in at the perfect time, with the necessary resources (materials, workmen, equipment) in view of their realization.

The net is the main programming and controlling instrument for the realization of a construction project by logical and conditioning relations between the component actions.

Among the widely known programming techniques are:

- CPM – Critical Path Method
- PERT – Program Evaluation and Review Technique
- PDM – Precedence Diagram Method
- GERT – Graphical Evaluation and Review Technique

The most widely used ones of these methods are: CPM (Critical Path Method) and PERT which appeared in1958-1959 in full engineering development age.

CPM (Critical Path Method) stands for a set of procedures for the programming and directing the execution of the projects based on the graph theory. In CPM, the representation of the development of the project execution is made as plane nets called graphs or net graphs and the mathematical substantiation is ensured by the theory of graphs.

The stages of the programming and directing the construction projects by CPM are presented in figure 2.

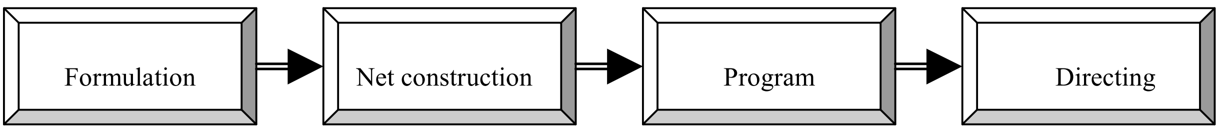

Fig.2 – Stages of project programming by CPM

II. ESTIMATE BLOCK – shows a series of instruments and techniques for settling the parameters and information related to:

- settling the technological ways of realization of the various component activities;

- settling the realization time of the component activities as well as of the whole project;

- settling the costs for the realization of the project and the component activities;

- settling the types and quantities of resources;

- settling and ensuring the desired quality level.

A construction project implies various ways of accomplishment, each being based on various technological, organizational, economical methods of realization. Each method is characterized by a series of information related to the technological content, the spatial structuring, realization time and cost, needed resources, a certain quality level, etc.

With the purpose of choosing the best method specific techniques and instruments are used which hold a series of specific parameters. These parameters present the information specific to each method of realization of the project as well as of the component activities both quantitatively (quantified) and qualitatively.

A parameter specific to a construction project may be defined as a measure (numerical expression), which is used to characterize and analyse a phenomenon, action or method of realization of a construction project.

Any parameter can be characterized by a structure and form of expression according to the nature of phenomena and processes to which it refers and can be represented in three ways:

- in a numerical form resulting from a quantification or determination process
- under the form of specifications (provisions) that refer to a real phenomenon or action
- under the form of approximate values or intervals (diversions, tolerances, etc.)

In order to better seize all the aspects related to the realization of a construction project, the following grouping of the specific parameters is suggested:

- Group I – Parameters with a general character
- Group II – Technical-organizational parameters
- Group III – Economical parameters
- Group IV – Quality parameters

Each parameter of the four groups presented above may refer to a component activity or to the construction project as a whole.

Taking into account the above-presented aspects, the estimate block suggests and develops a series of techniques, instruments and concepts in order to:

- dimension the technical-economical and quality parameters specific to the component activities of a construction project.
- efficiently structure and centralize the specific information for each activity
- calculate the processes and actions within an activity in order to establish the basic parameters: cost, time, quality requirements, resources consumption

Starting with the presented parameter groups, the estimate block is structured on four sections as follows:

- **SECTION I** – *General characteristics* – which presents the general parameters that lead to forming an image of the whole construction project.
- **SECTION II** - *Technical dimensioning* – is the section by which the technical-organizational parameters are established specific to the component activities of a construction project.
- **SECTION III** – *Economical dimensioning* – by which the economical parameters are established specific to the component activities of a construction project, especially those related to the total cost of the realization of the project.
- **SECTION IV** – *Quality dimensioning* – by which the parameters related to ensuring and controlling quality in the realization of the construction processes are established.

III. THE OPERATIONAL BLOCK – outlines information and specific elements related to programming and directing the execution of the construction project.

The content of the stages from the patterning block perspective:

STAGE 1 – SCENARIO – PROGRAM

It is a component stage of the Critical Path Method which deals with treating the nets through programming techniques.

The scenario is developed based on the resources and capacities that the entity involved in the execution of the construction project enjoys, having in view the restrictions and contractual conditions.

The realization of a construction project implies a large number of scenarios due to the various technological, organizational, administrative restrictions to the various technological methods of realization of the construction projects, the different possibilities of development in time and space, the different durations that may be adopted for the realization of the activities according to the resources that can be allocated.

The next steps need to be taken for the realization of this stage:

1. The calculation of the time parameters of the program;
2. The analysis of the maintenance within the time restrictions;

3. Integration and condensation of the general graphic;
4. The calendar transposal of the net graphic;
5. The analysis and program of the necessary resources for the realization of the project.

STAGE 2 – PROJECT MANAGEMENT

It is the stage in which the production apparatus, the production means and the working force are put into motion within the frame established by the realization program of the construction project.

The management of the project is put into practice through a set of techniques and instruments, which aim at realizing the component activities according to the dimensioned parameters and the realization of the whole project according to the established program implicitly.

The frame of the "Project management" stage

The following elements constitute the base for launching the "Project management" stage:

- The technical flow-chart of the activity (TFC) – by centralizing the information resulted from the dimensioning of activities (costs, realization time, necessary resources, quality requirements, technological and organizational compliance, etc.)
- The execution program of the project – by providing the information related to the starting point and ending point of activities, restrictions on the activities, working programs, resources program, quality program, etc.
- Logistics – by providing the information related to production capacities, supply, transportation, IT products, secretariat, documentary background, team, organizational adopted structures, etc.
- Communication – internal, external, delegations, negotiations

The construction projects managerial functions:

1. Command – consist in the initiation of the component actions
2. Coordination of activities
3. Control (quantitative, qualitative)
4. Assessment

The main instruments employed in project management:

1. The technical flow-chart (TFC)
2. The execution program of the project (EPP)
3. The project breakdown structure (PBS)
4. The activity technical sheet (ATS) through registration of the parameters in the realization stage of the project; drawing up the comparative methods; specifying performance indicators.
5. Risk analysis

Main elements of the activity coordination: expenses, time, quality, security, environment, and contractual obligations.

Management tasks: cost management, program management (work, supply, quality control, etc.), contract administration, inspection, communication, finalizing of actions (assessment, filing).

STAGE 3 – PROJECT FINALIZING

It is the stage that certifies the finalization of the project and the ending of contractual obligations to the factors involved in the realization of the project.

This stage begins with drawing up a reception report at the end of the works for the component construction objects and is available during the whole pledge of the works that is until the drawing up of a final reception report.

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Characteristics of Interfacial Transition Zone in Plain and Silica Fume Lightweight Concretes

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Rezumat: În studiul prezentat s-au investigat proprietățile din zona de tranziție interfacială a betoanelor cu greutate specifică mică realizate cu şi fără silice prin procedeele de observare ESEM/EDX. Agregatele artificiale cu greutate specifică mică vor fi fabricate prin legarea la rece a granulelor de ASTM Clasa C, cenuşă şi macrogranule de agregate uzate, ocupând 45% din totalul volumului de agregate din beton. Restul de 55% din mixtura betonului fiind nisip natural fin. În total sunt patru mixturi de betoane cu şi fără silice acestea înregistrând pentru o asigurare de 0,35 până la 0,55 w/c valori ale rezistențelor la compresiune în intervalul 23,2 – 39,1 MPa. Caracteristicile ITZ au fost studiate prin mărirea de 500 şi 2000 de ori a planului de beton pentru ITZ cu lățimile de 50 şi 100 µm pentru diverse rate de asigurare mai mici sau mai mari. În betonul cu silice respectivele zone sunt foarte dense şi compacte de aceea nu poate fi observată o separare clară.

Abstract: The study presented herein deals with the investigation of the interfacial transition zone (ITZ) properties in the lightweight concretes with and without silica fume by means of ESEM/EDX observations. Artificial lightweight aggregates were manufactured through cold-bonding pelletization of ASTM Class C fly ash and used as coarse aggregate at 45 % of the total aggregate volume in concrete. Natural sand fine aggregate covered the remaining 55 % in the mixture. A total of four concrete mixtures with and without silica fume was designed and cast at 0.35 and 0.55 w/c ratios providing a compressive strength range of 23.2 to 39.1 MPa. The characteristics of ITZ were observed at 500 and 2000 magnifications and it was observed that the plain concretes had ITZ widths of 50 and 100 µm for low and high w/c ratios, respectively, while this zone was very dense and compacted in the concretes with silica fume so that no clear separation could be observed.

Keywords: *EDX, ESEM, fly ash aggregate, interfacial transition zone, lightweight concrete*

1. Introduction

In any composite material the physical and chemical properties of the constituents and the interactions between them determine the behavior of the material. Concrete is a composite material with coarse and fine aggregate embedded in a cement paste matrix. Thus, the mechanical behavior of the concrete is greatly affected by both the aggregate and the cement paste as well as the interfacial zone between them. In order to have a satisfactory performance from a concrete the interfacial zone needs to be as dense as possible. In normal weight concrete, the concrete properties are mostly controlled by the interface as being the weakest link in this composite material [1]. However, it is believed that the aggregate is the weakest constituent controlling the material properties in lightweight concrete (LWC) [2]. Moreover, recent studies into the microstructure of such concretes suggested that the interaction of the lightweight aggregate (LWA) and the matrix may be quite different than that in NWC. In some studies it was suggested that there existed a pozzolanic reaction in this zone in LWC [3] while in others a negligible pozzolanicity was observed [4]. The objective of the present study is to examine the microstructure of the interfacial transition zone in plain and silica fume LWCs made with cold-bonded fly ash lightweight coarse aggregates.

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2. Experimental Study

2.1. Materials

An experimental study was conducted to investigate the microstructure of the interfacial transition zone in LWCs with and without silica fume. LWCs were produced with cold-bonded fly ash coarse aggregate, natural sand fine aggregate, superplasticizer, water, and portland cement. The mixtures were also replicated by adding 10 % silica fume by weight of cement.

In the production of both artificial lightweight aggregates and the lightweight concretes, a TS EN 197-1 CEM I portland cement with a Blaine fineness of 3499 cm^2/g was utilized. Silica fume used was in the form of commercial grade and had a specific gravity of 2.31 and a specific surface area of 20500 cm^2/g. LWAs were made through the pelletization of an ASTM Type C fly ash which came from Some Thermal Power Plant located in the Aegean part of Turkey. The specific gravity and the Blaine fineness of the fly ash were 2.56 and 3928 cm^2/g, respectively. **Table 1** shows the physical and chemical properties of cement, silica fume, and fly ash used in this study.

The pelletization was realized by means of cold-bonding process which includes the agglomeration of a dry powder mixture of 90 % fly ash and 10 % cement through moistening in a tilted pan. The whole process took about 20 min, in the first half of which the water was sprayed on to the fly ash-cement mixture in the revolving pan to make the green fresh pellets while the additional 10 min was devoted to the further stiffening of these fresh aggregates. Afterwards, they were put in sealed plastic bags and stored in a curing room for 28 days for the final hardening. Finally, the hardened aggregates were screened to desired sizes such that those passing 9.5 mm sieve and retaining on 4 mm sieve were selected as coarse aggregate.

2.2. Lightweight Concretes and Test Specimens

A total of four plain and silica fume concrete mixtures belonging to water/cement ratios (w/c) of 0.35 and 0.55 were designed and cast with cement contents of 550 and 400 kg/m^3, respectively. The silica fume was added at 10 % of cement by weight in the silica fume concrete mixtures. For each w/c ratio, the mixtures had a 45 % of LWA and a 55 % of natural sand by total aggregate volume. Actual mix propotions for 1 m^3 concrete are given in **Table 2**.

Table 1. Properties of cement, fly ash, and silica fume

Analysis Report		Cement	FlyAsh	Silica Fume
SiO_2 (%)		20.1	36.9	85.8
Al_2O_3 (%)		4.1	17.2	1.1
Fe_2O_3 (%)		4.3	4.8	0.9
CaO (%)		63.4	33.2	1.9
MgO (%)		1.2	1.4	2.6
SO_3 (%)		2.4	3.8	1.0
Na_2O (%)		-	0.3	0.3
K_2O (%)		-	1.8	4.1
Cl^- (%)		0.008	0.005	0.05
Insoluble Residue (%)		0.36	-	-
Loss of Ignition (%)		2.56	0.19	1.82
Free Lime (%)		1.51	-	-
Specific Weight		3.14	2.56	2.31
Fineness (%)	45 mm	11	23.3	27.7
	90 mm	0.3	9.93	4.5
	200 mm	-	2.74	1.2
Specific surface area		3499	3206	20500

The mixtures were designed to have an initial slump of 200 mm ± 20 mm which was achieved by using superplasticizer at varying amounts. Before being used in concrete casting, LWAs were first submerged in water for saturation and then laid on lower sized sieves for the seepage of excess water for about 30 min for each process to minimize the early slump loss owing to the high water absorption of the LWAs. The concrete casting sequence consisted of the dry mixing of saturated LWAs with cement and silica fume, when used, in a laboratory pan mixer. Then, the natural sand was incorporated, followed by the gradual addition of water with superplasticizer and mixing the constituents for about four min. Following the tests for slump and unit weight, the fresh concrete was poured into the steel molds in two layers each of which being compacted through vibration for couple of seconds.

Eventually, the concrete specimens were kept under polyethylene sheet for 24 hrs and then they were demoulded and submerged in water for 28 days till the time of testing. From a typical mixture three 100 mm cubes for compressive strength testing, three 100x200 mm cylinders for modulus of elasticity and splitting tensile strength, and one 100x100x500 mm prism to prepare the samples needed for the ESEM observation were obtained. The cubes and the cylinders were tested by means of a 2000 kN capacity Autotest machine at the age of 28 days. For the microstructural analysis, the concrete samples of about 10x10x10 mm were obtained from the prism by means of saw cut and then the interfacial zone between the LWA and the cement matrix was observed using a Philips XL30 ESEM-FEG/EDAX machine at 500 and 2000 magnifications without any surface preparation. Also, the energy dispersive spectrums (EDX) for each sample was acquired.

Table 2. Concrete mix proportions

Mix No	w/c	cement (kg)	Water (kg)	Silica Fume (kg)	SP* (kg)	LWA (kg)	Sand (kg)
M1	0.35	550	192.5	0	11	486.9	862.6
M2	0.35	550	192.5	55	11	465.9	825.4
M3	0.55	400	220	0	2	509.6	902.8
M4	0.55	400	220	40	2	494.3	875.7

Table 3. Test results

Concrete properties	Mixture no			
	M1	M2	M3	M4
Compressive strength (MPa)	36.9	39.1	23.2	25.3
Modulus of elasticity (GPa)	20.0	23.2	17.0	18.3
Split-tensile strength (MPa)	2.86	3.17	2.16	2.35

3. Test Results and Evaluation

3.1. Mechanical Properties

The test results of the concretes regarding the compressive and splitting tensile strengths, and modulus of elasticity are shown in **Table 3**. It was observed that the plain concretes had a compressive strength of 23.2 and 36.9 MPa while the compressive strength of the concretes with silica fume were 25.3 and 39.1 MPa at high and low w/c ratios, respectively.

3.2. Characteristics of Interfacial Transition Zone

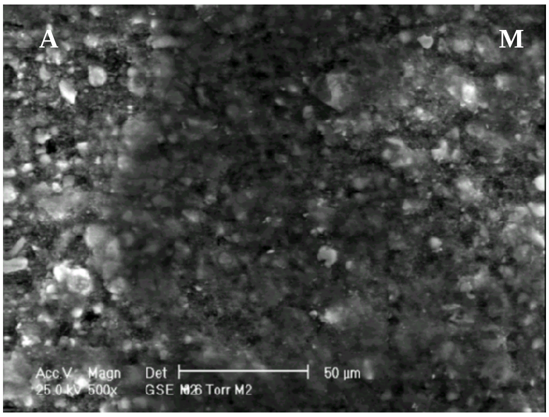
ESEM observation was conducted at 500 and 2000 magnifications on the interfacial transition zone (ITZ) between the LWA and the matrix phase. In all pictures the area to the left represents the matrix phase and to the right is the LWA. The ITZ views at 500 and 2000 magnifications accompanied by the corresponding EDX spectrum are depicted in **Figs. 1 and 2** for low and high w/c ratios, respectively. It was observed that at low magnification there appeared to be intimate contact between the cement paste and the LWA. Although some porous structure existed in the interface (darker region), there were abundance of hydration products. The very bright points were unhydrated cement and fly ash particles in the matrix and the aggregate, respectively. The thickness of the ITZ was not so clear that no clear separation was observed for the concretes of low w/c ratio as seen in **Fig. 1a**. However, the companion high w/c ratio concrete had more porous interfacial zone **(Fig. 2a).** The measured width of ITZ was approximately 50 and 100 μm for the former and the latter, respectively. The aggregate side of the ITZ was well observed at 2000 magnification as seen in **Figs. 1b and 2b** for low and high w/c ratios, respectively. A more compacted structure was achieved near the aggregate side. The hydration products close to the aggregate surface were very fine, dense, and nearly uniformly distributed in the concretes with low w/c ratio while there was plenty of pores in the ITZ with the increase in the w/c ratio. The higher Ca peak associated with the lower Si peak observed in **Fig. 2c** than those in **Fig. 1c** confirmed higher amount of the hydration products in the latter. The previous research [5] suggested that the LWA had been penetrated by the hydration products or cement grains through the open pores located in the aggregate shell. The oriented calcium hydroxide crystals that are normally present in the conventional NWC were not detected, even at high w/c ratio. This finding was also confirmed by Sarkar et al.[6] and Wesserman and Bentur [2].

Figures 3 and 4 demonstrate the characteristics of ITZ in silica fume concretes at 0.35 and 0.55 w/c ratios, respectively. It was clearly seen that the ITZ of the concretes containing silica fume was dense and compacted, irrespective of w/c ratio, as in the NWC [1]. The aggregate and the cement paste were interlocked so that no clear transition zone was detected. In addition to the prompt pozzolanic effect of the silica fume, its micro-filler influence gave rise to this dense structure. These figures also exhibited that there existed plenty of unhydrous cement grains in the concretes with low w/c ratio which being rarely observed with the increase in the w/c ratio. The EDX spectrums suggested that a higher rate of hydration was achieved with the addition of silica fume as being justified the remarkably higher Si peaks seen in the Figures 3c and 4c, irrespective of w/c ratio.

4. Conclusions

1. The width of ITZ was about 50 and 100 μm for the plain concretes with low and high w/c ratios, respectively.
2. A higher Si peak along with a lower Ca peak in the EDX spectrum of the concretes indicated greater amount hydration products for the concretes with silica fume and/or with low w/c ratio.
3. The oriented CH crystals that are normally existent in NWC were not detected in all concretes.

Use of silica fume provided a more compacted and denser structure so that no clear transition zone was observed in such concretes.



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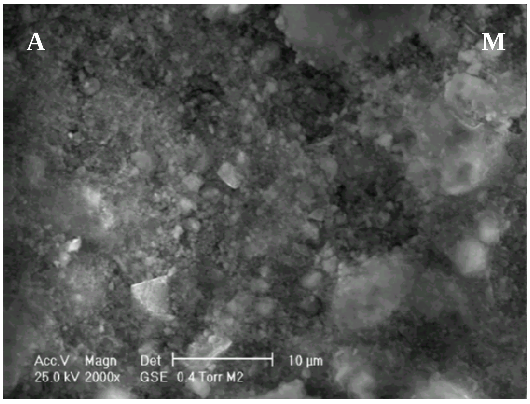

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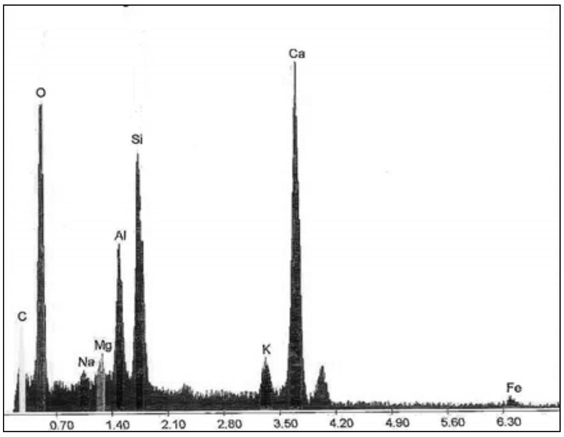
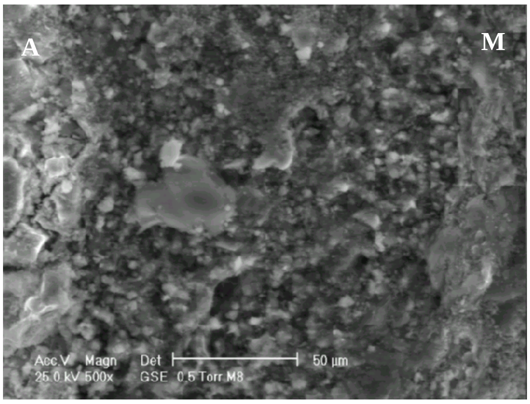


c)

Fig. 1. ITZ in the plain LWC at low w/c: a) x500 b) x2000, c) EDX spectrum



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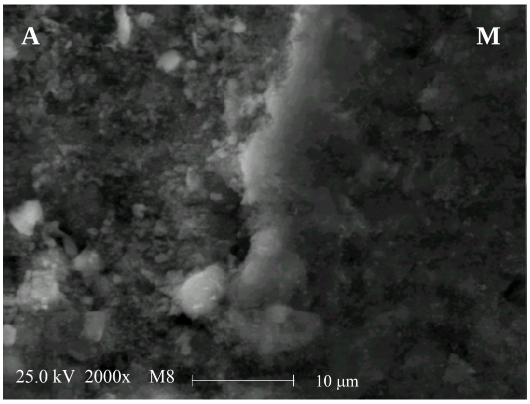

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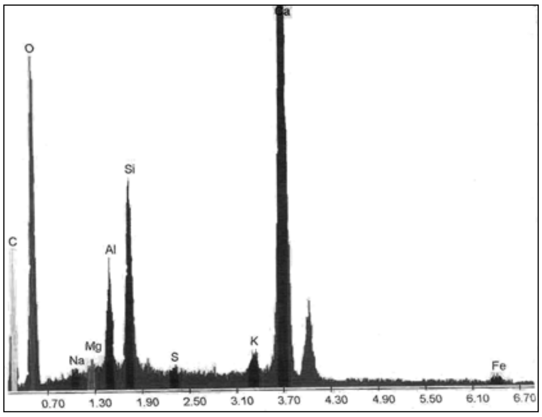
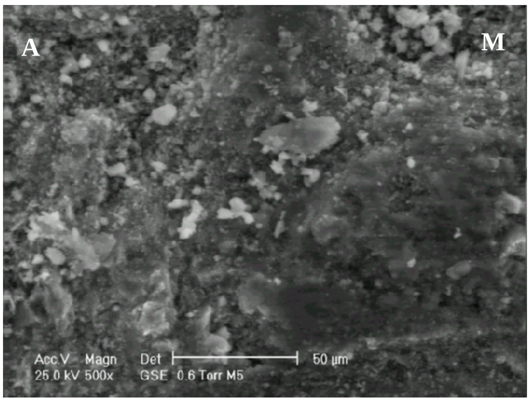


c)

Fig. 2. ITZ in the plain LWC at high w/c: a) x500 b) x2000, c) EDX spectrum



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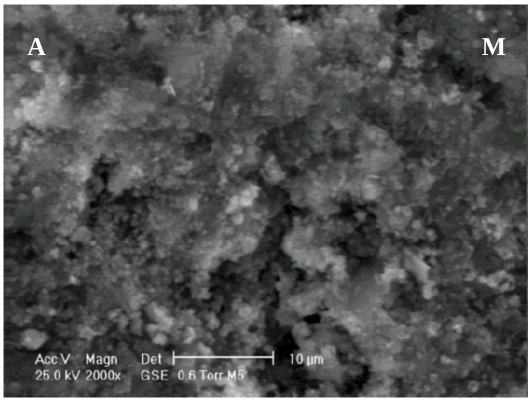

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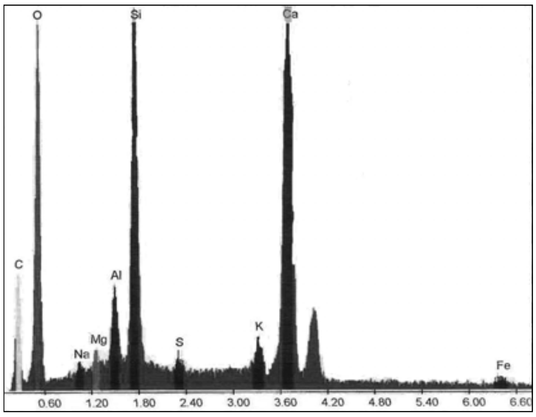
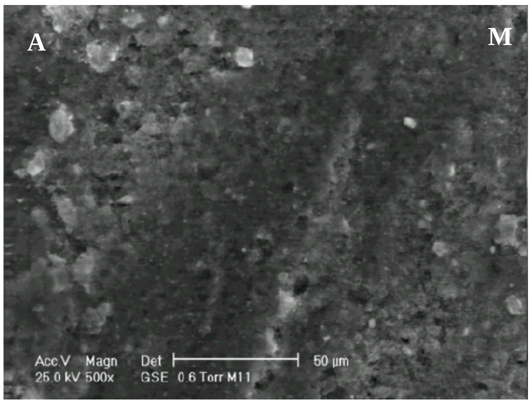


c)

Fig. 3. ITZ in the LWC with silica fume at low w/c: a) x500 b) x2000, c) EDX spectrum



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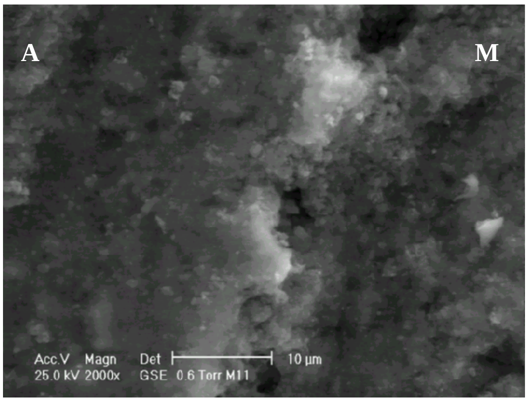

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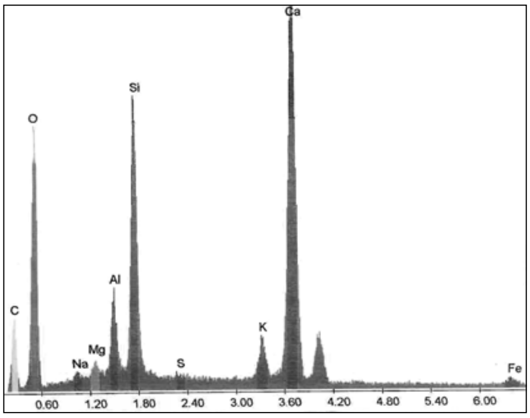

c)

Fig. 4. ITZ in the LWC with silica fume at high w/c: a) x500 b) x2000, c) EDX spectrum

5. Acknowledgements

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Evaluation of Bond Strength of Rebars in Concrete: Influence of Cement Type and Curing Condition

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Rezumat: În lucrare a fost studiată rezistența de legătură dintre armătură şi beton. A fost examinată rezistența de legătură în funcție de tipul de ciment, raportul apă-ciment, protejarea betonului până la întărire, expunerea la cloruri, îmbătrânirea betonului. În total sau realizat zece mixturi diferite cu ciment portland pentru patru amestecuri diferite şi două valori ale raportului apă-ciment de 0,45 şi 0,65. Testele au fost realizate pe probe de beton armat obținute cu trei regimuri de protejare a betonului până la întărire, controlată, necontrolată şi protejare umedă. Câteva probe au fost expuse la o soluție de clorură de sodiu cu concentrație de 4% după 28 de zile. Testul de comportare la smulgere s-a făcut la 28 de zile şi 180 de zile de la turnare pentru diferite regimuri de protejare şi expunere la cloruri. Rezultatele testelor indică o îmbunătățire a rezistenței de legătură când raportul apă-ciment scade şi se aplică o protejare corespunzătoare. Pentru amestecul de ciment în probe se obțin rezistențe de legătură relativ mari atunci când avem un raport apă-ciment mic şi o protejare corespunzătoare.

Abstract: In this paper, the bond strength of deformed rebars in concrete was studied. The effect of cement type, water-cement ratio, curing regime, chloride exposure, testing age on the bond behavior were examined. A total of ten different concrete mixtures having a plain portland and four different blended cements were designed at two water-cement ratios of 0.45 and 0.65. The reinforced concrete test specimens were subjected to three curing regimes, namely uncontrolled, controlled, and wet curing. Some of the specimens were also exposed to 4% sodium chloride solution after curing for 28 days. The pull-out test was conducted at the age of 28 and 180 days after being subjected to different curing regimes and exposed to chloride solution, respectively. The test results indicated that decreasing water-cement ratio and applying proper curing improved the bond properties. The blended cement concrete specimens exhibited relatively higher bond strength than the plain concrete specimens under the conditions of low water-cement ratio and proper curing.

Keywords: *bond strength, curing regime, plain and blended cement concretes, pull-out test*

1. Introduction

In the majority of reinforced concrete applications, steel bar continues to be the most effective and cost-efficient reinforcing material. Its strength and ductility make it well suited for reinforced concrete. The bond between concrete and reinforcing bars is one of the main traits of reinforced concrete structures. This is the case for any type of reinforcement. The critical design parameters, such as development length, depend directly on the bond between concrete and reinforcement. So do the member deflection, crack spacing and crack width [1]. The rebar coating systems (epoxy-coated steel reinforcing bars) is on the increase and it is becoming a common practice in some areas, especially those with corrosive environment [2,3]. However, the main disadvantage of using epoxy-coated bars is the reduction of bond strength between the rebars and concrete. Furthermore, the concern over bond strength is increasing with the use of high performance concrete and coated or uncoated reinforcing bars.

In this experimental study, the performance of deformed type reinforcing steel embedded into plain and blended cement concretes has been evaluated by means of pull-out test. In the concrete mixtures, a plain portland and four blended cements were utilized. After

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casting, the specimens were subjected to different curing regimes prior to testing. The influence of cement type, exposure condition, water-cement ratio, testing age was discussed.

2. Experimental Study

2.1. Materials

Portland cement (PÇ 42.5 & CEM I), two portland composite cements (PKÇ/A 42.5 R & CEM II/A-M and PKÇ/B 42.5 & CEM II/B-M), composite cement (KZÇ/A 42.5 & CEM V/A), and blast furnace slag cement (CÇ 42.5 & CEM III/A) were used as cementitious materials. Properties of these cements are given in **Table 1**. The coarse aggregate was a crushed limestone with a maximum particle size of 20 mm whereas the fine aggregate was a mix of natural sand and crushed limestone sand. A sulphonated naphthalene formaldehyde-based superplasticizer was used to get a workable fresh concrete. Deformed steel bars, 16 mm in diameter, were utilized as reinforcement.

2.2. Mixtures, Specimens, and Curing Conditions

In this study, two series of concretes were produced. The concretes in the first series had a water-cement ratio of 0.65 and a cement content of 300 kg/m^3. In the second series, the concretes were made with a water-cement ratio of 0.45 and a cement content of 400 kg/m^3. The first and second series concrete mixtures were denoted by (N) and (H), respectively. Five different cements were used in concretes of the both series. Concretes produced with PÇ 42.5, PKÇ/A 42.5 R, PKÇ/B 42.5, KZÇ/A 42.5, and CÇ 42.5 were denoted as B1, B2, B3, B4, and B5, respectively. Grading of the aggregate mixture was kept constant for all concretes. All concrete mixtures were designed to provide a slump of 17 ± 2 cm. The superplasticizer was added at the time of mixing. Mix proportions for the concretes are summarized in **Table 2**.

All concretes were mixed as per ASTM C192 in a pan mixer by first mixing the dry ingredients for one minute, and then adding the water and mixing for an additional three minutes. The reinforced concrete specimens for the pull-out test were 100x200 mm concrete cylinders in which a 16 mm diameter steel bar was centrally embedded. The embedment length was kept constant as 170 mm for all specimens. Rebars were cleaned with a wire brush to remove the rust from surface just before casting the reinforced concrete specimens. Eighteen reinforced concrete specimens were cast from each concrete mixture. The specimens were cast in three layers and compacted using a vibrating table. After casting, the moulded specimens were covered with a plastic sheet and left in the casting room for 24 hr. They were then demoulded and divided into three equal groups and cured under the three different conditions: 1) Uncontrolled curing (UC): specimens were air cured at uncontrolled temperature and relative humidity until the test age. 2) Controlled curing (CC): specimens were immersed in 20 ± 2 ^{o}C water for 7 days and then air cured in a room at 20 ± 1 ^{o}C and 50 ± 5% relative humidity until the test age. 3) Wet curing (WC): specimens were immersed in 20 ± 2 ^{o}C water until the test age. At the end of 28 days, two of six specimens from each curing condition were exposed to a 4% NaCl solution for 180 days. The test was conducted at the age of 28 and 180 days after the application of different curing regimes and exposure to chloride solution.

2.3. Test Method

The pullout tests were conducted following a procedure similar to ASTM C234 [4]. The photographic view of the experimental set up is given in **Fig. 1**. The rebars were pulled out of the concrete cylinders in a universal testing machine having a capacity of 600 kN and the maximum load and type of bond failure were recorded for each specimen. Average of two specimens was used for the determination of the maximum (or ultimate) bond strength.

3. Evaluation of Experimental Results

The variation of the bond strength with time and exposure condition, for the three types of curing, is shown in **Figs. 2-4**. The measured bond strengths were in the range of 4.4 to 8.7 MPa. It was generally observed that cement type, curing condition, testing age, and water-cement ratio of the mixtures were very

Table 1. Properties of cements used

Cement type						
Turkish & EN 197-1		PÇ 42.5 & CEM I	PKÇ/A 42.5R & CEM II/A-M	PKÇ/B 42.5 & CEM II/B-M	KZÇ/A 42.5 & CEM V/A	CÇ 42.5 & CEM III/A
Experimental code		B1	B2	B3	B4	B5
Results of chemical analysis		-	-	-	-	-
Silicone dioxide SiO_2 (%)		20.64	18.38	28.34	25.63	28.81
Aluminum oxide Al_2O_3 (%)		5.06	5.05	7.33	5.06	7.2
Ferric oxide Fe_2O_3 (%)		3.14	2.89	2.89	3.72	2.31
Calcium oxide CaO (%)		63.98	61.78	52.55	48	49.94
Magnesium oxide MgO (%)		1.2	1.36	2.09	-	4.44
Sulfur trioxide SO_3 (%)		2.38	2.34	2.88	2.3	2.41
Sodium oxide Na_2O (%)		0.31	0.28	0.21	-	0.15
Potassium oxide K_2O (%)		0.8	0.73	-	-	0.87
Chloride Cl^- (%)		0.035	0.036	-	0.01	0.027
Insoluble residue (%)		0.46	0.48	7.8	-	0.64
Loss of ignition (%)		1.72	6.44	1.16	-	2.4
Free lime (%)		1.41	1.44	0.35	-	0.83
Physical tests		-	-	-	-	-
Specific gravity		3.15	3.12	3.01	3.05	2.94
Vicat (hour:minute)	Start	02:28	02:28	02:40	02:32	02:40
	Stop	03:02	03:08	03:30	03:22	03:30
Le chatelier (mm)		2	2	1	1	1
Fineness (%)	45 µm	11.7	18.1	-	-	1.3
	90 µm	0.8	3	6.4	0.2	0.0
	200 µm	0.0	0.4	0.7	-	-
Specific surface (m^2/kg)		336	334	406	430	464
f_{cc} (2 day) (MPa)		27.5	23.7	23.1	20	13.3
f_{cc} (7 day) (MPa)		41.3	39	35.9	31	24.6
f_{cc} (28 day) (MPa)		51.4	46.2	51.2	45	-
Component fraction in cement		-	-	-	-	-
Clinker (%)		95.5	78.7	70.5	57.5	46.7
Blast furnace slag (%)		0	2.0	13.0	21.8	48.3
Limestone (%)		0	11.9	0	3.0	0
Natural pozzolans (%)		0	3.2	13.0	12.6	0
Gypsum (%)		4.5	4.2	3.5	5.1	5.0
Total (%)		100	100	100	100	100

Table 2. Mix proportioning of concrete in kg/m^3

Concrete Series	Code	Mix proportioning (kg/m^3)							
		W/C	Cement	Water	Coarse Aggregate		Fine Aggregate		SP*
					No I	No II	Sand	Crushed sand	
1	N-B1	0.65	308.1	200.3	558.3	616.0	537.6	191.8	0.77
	N-B2	0.65	302.8	196.8	547.8	604.4	527.5	188.2	0.76
	N-B3	0.65	304.7	198.1	548.6	605.4	528.3	188.5	0.76
	N-B4	0.65	306.5	199.2	552.9	610.1	532.4	189.9	0.77
	N-B5	0.65	306.4	199.2	549.7	606.5	529.3	188.8	0.77
2	H-B1	0.45	405.4	182.4	536.2	591.6	516.3	184.2	3.04
	H-B2	0.45	399.6	179.8	527.5	582.0	507.9	181.2	3.00
	H-B3	0.45	399.5	179.8	523.7	577.9	504.3	179.9	3.00
	H-B4	0.45	400.7	180.3	526.0	580.4	506.5	180.7	4.01
	H-B5	0.45	399.8	179.9	521.0	574.9	501.7	179.0	4.00

*SP=Superplasticizer

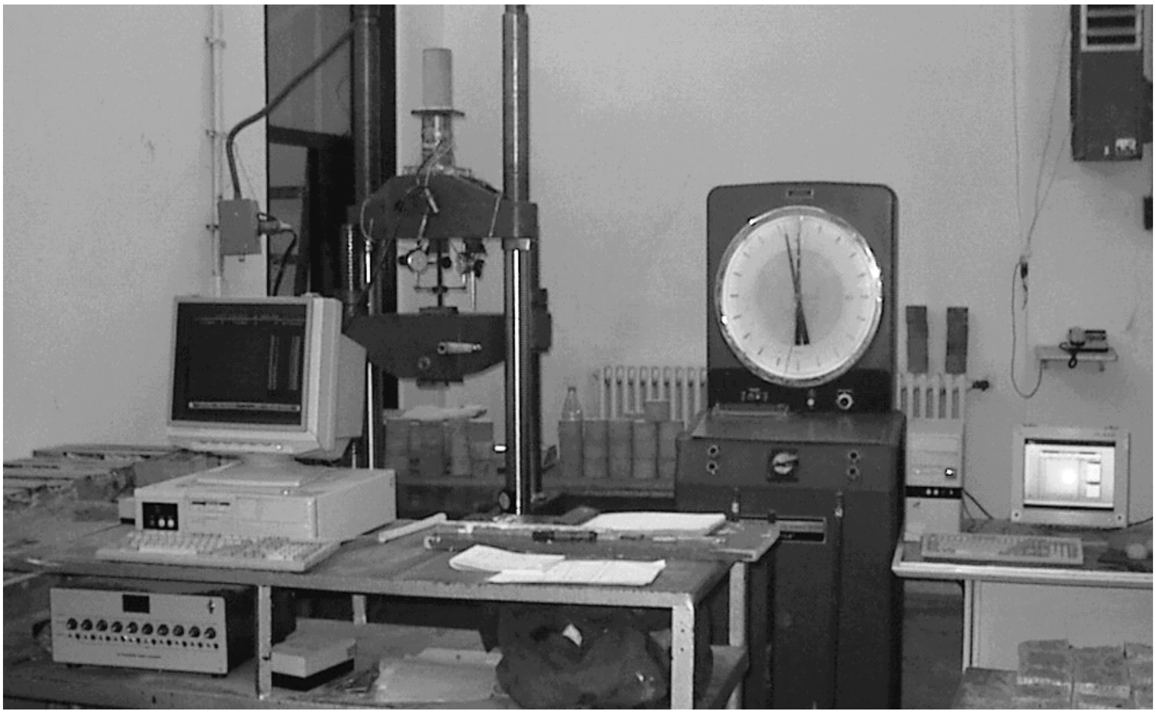

Fig. 1. Photographic view of experimental setup for pull-out test

effective on the bond strength. However, the effect of the water-cement ratio and curing condition were more significant in comparison to the others. Under the curing condition of UC (uncontrolled curing), decreasing the water-cement ratio from 0.65 to 0.45 resulted in an increment of 43% at 28 days and 36% at 180 days for the specimens with the plain portland cement concretes (B1) while those increments were in the range of 13 to 41% at 28 days and 17 to 36% at 180 days for the specimens made with the blended cement concretes (B2-B5). It was evident from these data that the blended cement concrete specimens exhibited lower rate of increment and had mostly lower bond strength as compared to the plain portland cement concrete specimens. This trend was changed for the specimens subjected to curing conditions of CC and WC (controlled and wet curing, respectively). Majority of the test results indicated that the specimens made with the blended cement concretes had higher increment rate and greater bond strength, especially at the low water-cement ratio, than those produced with the plain portland cement concrete. It was also noted that both plain and blended cement concrete specimens exhibited splitting type of failure during the experiment.

As seen from the figures, for all specimens, the lowest bond strength values were obtained when the bond specimens were kept continuously in air (UC). The results generally showed that the blended cement concrete specimens were influenced adversely by the poor curing condition. Moreover, no significant difference in the bond strength was observed between the specimens subjected to the curing conditions of CC and WC. The specimens subjected to the curing condition of CC yielded about 5-10% lower bond strength than those cured with respect to that of WC. Among the blended cement concrete specimens, in most of the cases, the specimens made with portland composite cement (B3) depicted higher bond strength, especially at low water-cement ratio.

The comparison of the test results of the specimens exposed to the chloride solution after the 28 day pre-curing period of UC, CC, and WC were also shown in **Figs. 2-4**. It was observed that, at the end of 180 days, test specimens exposed to the chloride solution after the pre-curing period of UC resulted in somewhat higher bond strength than those subjected to the curing condition of UC. The increase in the bond strength may be due to the hydration of the cement with time. Moreover, there was no considerable trend between the specimens exposed to the 180-day chloride exposure after the pre-curing periods of CC and WC and those subjected to the curing conditions of CC and WC, respectively. However, the variations in the bond strength of the reinforced concrete specimens were not significant (within about ±5%).

4. Conclusions

The bond strength of deformed rebars in concretes made with plain portland and blended cements was studied. Specimens were subjected to different exposure conditions for time durations up to 180 days. The results obtained from this experimental study can be summarized as follows:

1. Test results indicated that cement type, curing regime, testing age, and water-cement ratio of the mixtures had pronounced effect on the bond strength. However, the effect of the water-cement ratio and curing condition were more significant as compared to the others.

2. It was observed that decreasing w/c ratio from 0.65 to 0.45 resulted in upto %43 greater bond strength, depending mainly on cement type, curing condition, and testing age.

3. For all specimens, the lowest bond strength was obtained when the specimens were subjected to the poor curing condition (UC). However, the highest one was reached under the condition of wet curing (WC). No significant difference in the bond strength was observed between the specimens subjected to the curing conditions of CC and WC. It was also noted that test specimens exposed to chloride solution for 180 days after the 28 days curing period of UC resulted in to some extent higher bond strength than those subjected to the curing condition of UC. But the variations in the bond strength of the specimens exposed to the chloride solution for 180 days after the 28 days curing period of CC (or WC) were not significant.

4. Results also revealed that under the curing condition of UC, the blended cement concrete specimens exhibited lower rate of increment and had mostly lower bond strength as compared to the plain portland cement concrete specimens.

Conversely, this trend wasopposed for the specimens subjected to proper curing conditions

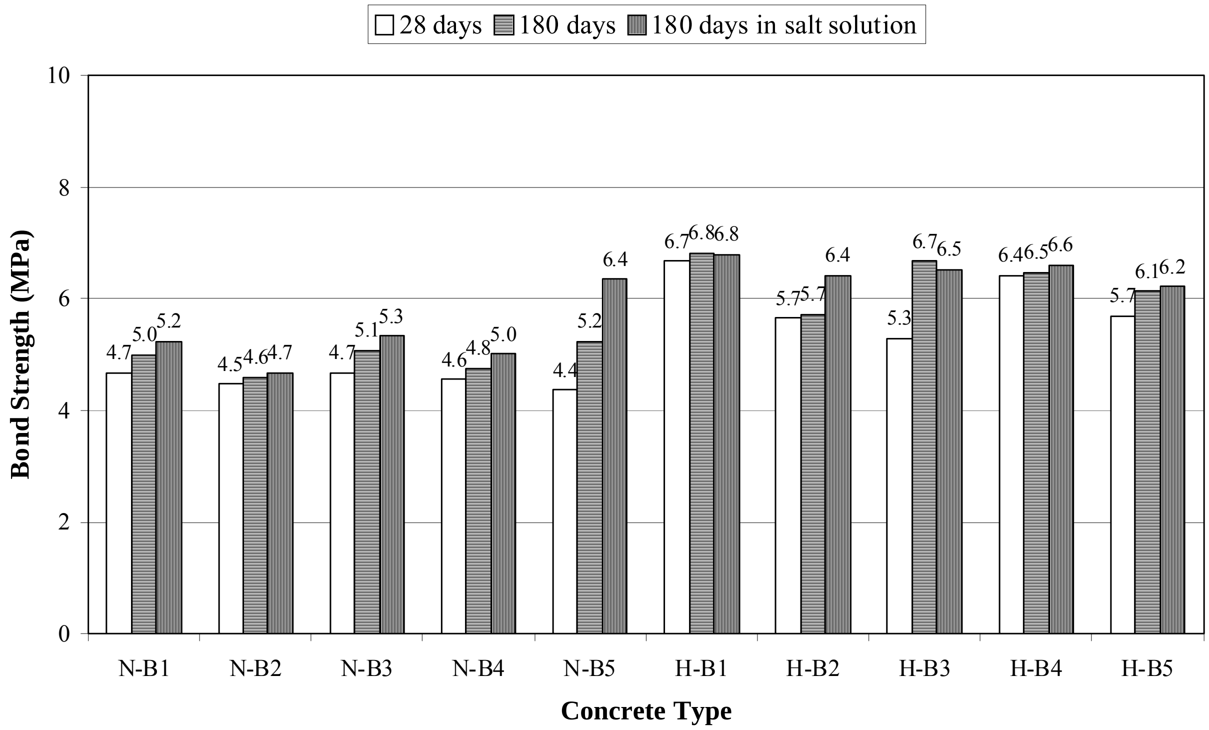

Fig. 2. Variation of bond strength of the specimens subjected to uncontrolled curing (UC) and chloride exposure after the 28 day of uncontrolled curing

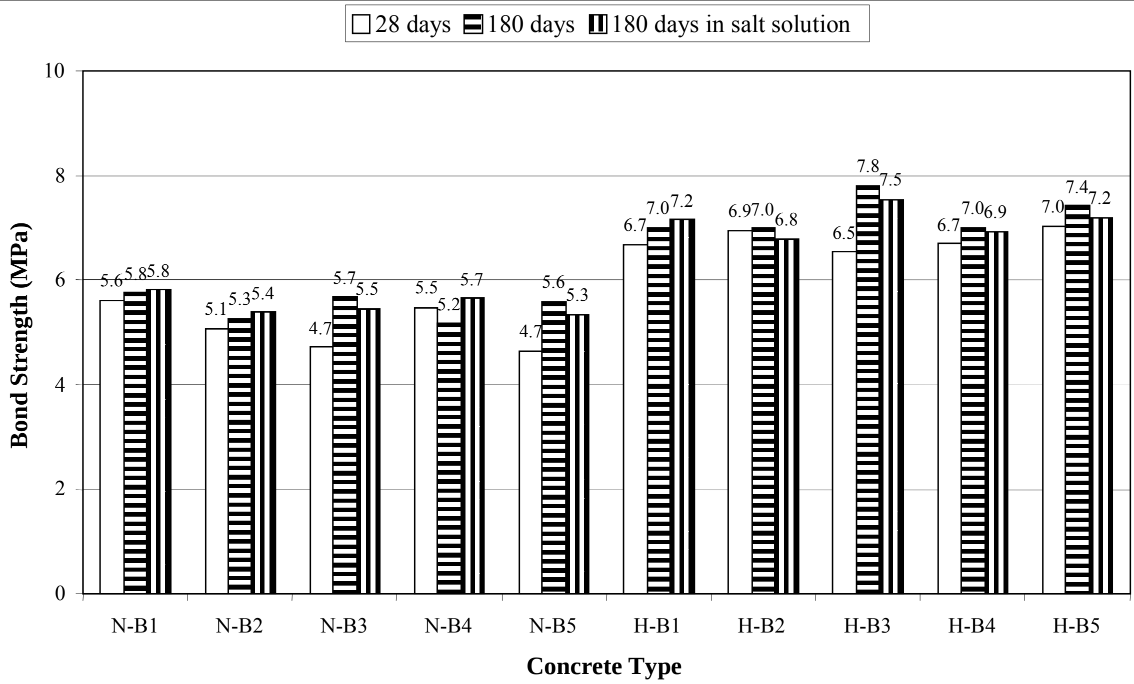

Fig. 3. Variation of bond strength of the specimens subjected to controlled curing (CC) and chloride exposure after the 28 day of controlled curing

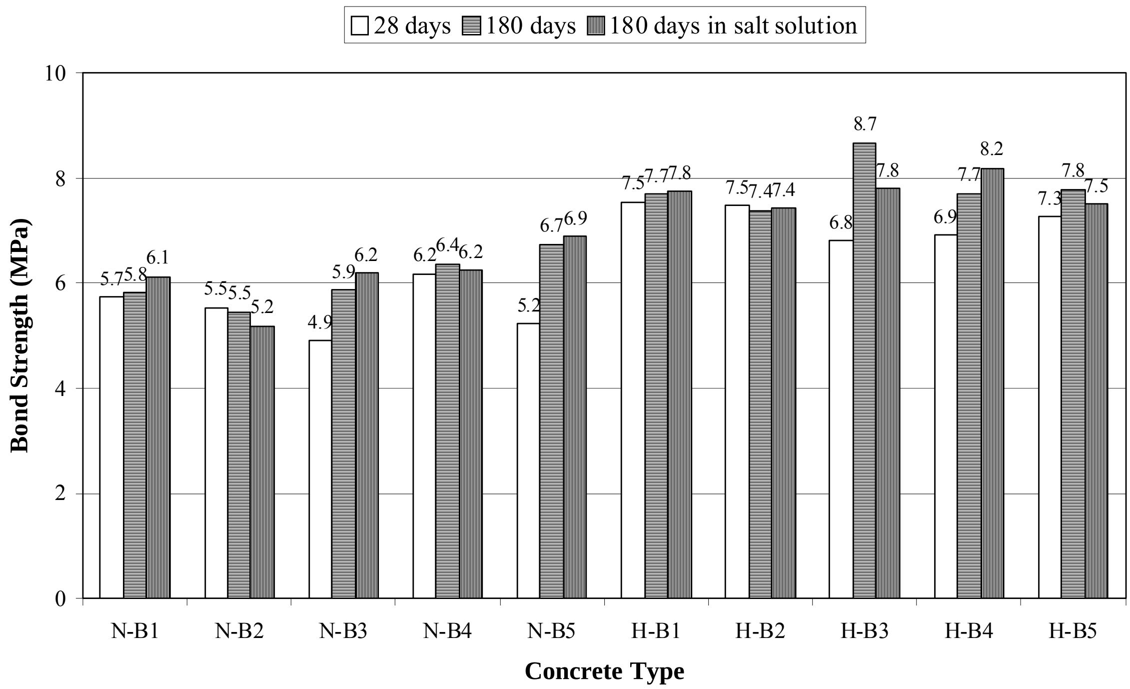

Fig. 4. Variation of bond strength of the specimens subjected to wet curing (WC) and chloride exposure after the 28 day of wet curing

5. Acknowledgements

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Concretes with Siloxane Additives for Hydrotechnical Works

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Rezumat: Betoanele cu aditivi siliconici obținuți din vulcanizarea la rece la a cauciucului siliconic reprezintă o soluție performantã pentru lucrările de construcții hidrotehnice. Ĩn lucrarea de fațã sunt prezentate rețete noi de betoane aditivate cu produşi siliconici, precum şi calitățile acestora comparativ cu alte soluții menționate în literatura de specialitate.

Abstract: Concretes with siliconical additives obtained by cold vulcanizing of siliconic rubber represent a performing solution for hydrotechnical works. The paper introduces new compositions for additivated concretes with siliconical products and their qualities in comparison with other solutions mentioned in specialty literature.

Keywords: *siliconical polymer, impregnated and polymerized concretes, waterproofing.*

1. Introduction

Polymerized and impregnated concrete results from common concrete, hardened under normal conditions by impregnation with a monomer which, in certain conditions of microclimate, polymerizes, so that the new composite product has higher characteristics than the plain concrete.

The use of natural polymers in manufacturing and use of build materials is quite an old idea. Thus, were used: natural protein polymers, natural rubber latex, butadiene or chloroprenic latex in order to increase the strength, freeze-thaw resistance, imperviousness of concrete used in hydrotechnic works.

For the last decade, the efforts spent in creating new types of concretes with polymers, improving their technical characteristic and enlarging the use range have continued in most countries, with a remarkable breakthrough, the polymerized and impregnated concrete [4].

The paper introduces new variants of polymerized and impregnated concrete of high efficiency as: mechanical strengths unaltered by severe (humidity and aggressiveness) environmental conditions, reduced consumption of polymeric solutions used as additives in water proofing – hydrophobing, law perviousness.

Concrete coating, setting and hardening are not influenced by ambient temperature, the polymerization occurring between -50^0C and 300^0C. The workability of the concrete with such additions can be adjusted by the amount of catalysts used alongside the polymer solution utilized as an additive.

2. Components of Polymerized and Impregnated Concretes:

2.1 Additives:

From the polymer group there have been used thermo-reactive polymers like synthetic rubbers, silico-organic compounds obtained by cold vulcanizing of siliconic rubber, respectively.

The siliconic elastomers types I, II and linear siloxanes [2], are soluble, polymerizable products the latter process taking place during concrete hardening. The products are manufactured in the "P. Poni" Macromolecular Chemical Research Institute, silicone department.

NASA has used products with similar characteristics in achieving the thermal protection of space shuttle playboards [2].The physical – mechanical properties of additives are centralized in Table 1.

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Table 1

Additive / Characteristics	Siliconic elastomers		Linear siloxanic compound
	I	II	
Viscosity at 25^0C, [CSt]	66.9…92.6	52.5…61.7	2.7…2.8
Creeping, [%]	465	80…170	
Creeping strength, [daN/cm^2]	15.10	10…15.50	
Hardness Shore, [Å]	21	27…30	
Refraction's coefficient at 20^0C			13.95…13.97
Humidity after 24 h			13.95…13.97

The structure of silico – organic polymers has the form:

```
   R    R    R
- O-Si-O-Si-O-Si-O-
   R    R    R
```

where, $R \equiv CH_3(OH_3COO)_2$.

The reticulation occurs in solution, in the presence of a catalyst, added in the matrix formed by cement, aggregate, water, after a minimum 3 minutes mixing.

From the chemical point of view, the influence of the silicones with Si-H reactive groups on the treating materials can be interpreted as a hydrolytic reaction due to the free OH groups of the support or the water film adhering to the aggregate / support surface in the presence of catalysts (alkali, metallic salts of organic acids, organo – metallic compounds) as in the subsequent condensation of the siloxanes thus formed.

During the first stage the reaction involves quickly, resulting in the splitting of Si-H bonds and the formation of siloxanes, which, in a second stage condense by two adjacent polymer chains, thus leading to reticulated polymers [1], [2].

In the same time, even OH groups of siloxanes have a reticular influence on H-siloxanes, so that a part of siloxanes will react with the new Si-H groups, which are still in excess. Using catalyst may accelerate this reaction.

The siloxanic polymers with reactive groups are able to reticulate on the surface of the support and form thin, flexible films, which are resistant to abrasion. These films prevent exterior water from penetrating, without being an obstacle to the interior water drainage. The permeability measurements show clearly that by treating with silicones, the pores' diameter in the mass of material is not diminished [3], [4], [5].

The influence of the analyzed additive on the cement concrete is reflected in the fresh and hardened concrete properties.

2.2 Concrete with Silico –organic Additives

The composition of concrete with additives, mentioned above, refers to the physical – chemical compatibility of the additive aggregate and binder graves, with a moderate content of fine, Portland cement in amount of 325 kg/m^3.

Workability

This quality, influenced by water cement ratio, was correlated to the presence of the additive, which reduces the ratio to a value of maximum 0.4.

Modifying the amount of catalyst in relation to the siliconic solution used can regulate the modification of workability, depending on the casting technology.

Thus, the modification of the catalyst percentage, related to the additive mass, varies between 1…5% resulting in a range of concrete with different workability, without any modification of the hardened concrete properties. This can be explained by the fact that an increase of the catalyst amount leads only to the decrease of reticulating speed of the polymer spatial grid.

By varying the amount of catalyst we can make it a setting and hardening accelerator or retarded.

Workability, determined according to technical codes, has values of 5…10 cm, depending on the type of concrete element, means of transport, technology.

The degree of compaction is of 1.0 … 1.04, values which agree with the technical regulations [3], [5].

The reduction of water cement ratio under 0.4 in concrete of a higher class than Bc_{25}, without any effect to the hardened concrete properties, proves that the additives are fluidizers.

Porosity

The microscopic analysis of hardened concrete sections shows pores with diameters of 0.2 … 2 mm, unbounded, uniformly distributed in the concrete mass. Additives are beneficial in this respect as contributing to the concrete imperviousness.

Thus, experimentally, the apparent porosity is 3.90% for liner siloxanic compounds and 6.10 and 7.70% for siliconic elastomers, respectively.

The apparent porosity is much lower than the minimum mentioned nowadays in the specialty literature, 12% for cement mortars RMM types, obtained in USA [1].

The *permeability coefficients* are $0.1 \cdot 10^{-5}$ cm/s for concrete with II additive and $0.32 \cdot 10^{-5}$ cm/s for concrete with linear siloxanic additive.

The *water absorption* in the mass of concretes specimens is maximum for the first, 48 hours from immersion (test made according the current technical codes) being caused by the "thirst" of concrete while, in the next 24 hours, it decreases under 0.2 cm as an effect of additives.

The witness specimens immersed in water, show, after 24 days, that along the perimeter, water could be traced in the pores, over a depth of 1 … 1.5 cm. Therefore, the additives behave like those in the group of hydrophobic tense – active additives.

The *mechanic tests* (compressive strength bucking strength) prove that additives do not determine a remarkable increase of mechanic strength as compared to the concrete without additives.

The *frost – thaw test* evidences after 100 cycles the mass loss is of the order of 0.01% and ratio of mechanic strengths after 28 days (preservation under standard conditions) and those obtained after 100 cycles of frost-thaw is higher than 0.85, indicating the quality of the concreted.

The *adherence* between the support layer and the pellicle of concrete with silico – organic additives is of order of $4 N/mm^2$.

The *dimensional variations* in the specimens of concrete with additives are consequence of silico-organic additives.

Thus:

a).when immersed in water (without hydrostatic pressure), the deformations are due to the swelling of the synthetic seed;

b). After 12 frost – thaw cycles, the variations are minimal, as the additives used are extremely resistant to extreme temperatures.

The dimensional variations are of the order of -0.9 … 2 mm/m.

3. Conclusions

By comparing the experimental results of concrete with silico-organic additives with those in the literature for concretes with inorganic or organic additives, some very important advantages can be emphasized namely:

a) possibility of reducing water amount in cement mixture;

b) possibility of adjusting the workability depending on the type of cast element and the casting technology;

c) increasing or decreasing of setting and hardening time;

d) reduction of pore volume entrained in the concrete mass;

e) reduction of capillary ascension;

f) reducing of capillary ascension coefficient and increasing of concrete imperviousness, respectively;

g) preserving of mechanical properties at frost-thaw cycles than for the concrete without additives o with other inorganic or organic additives;

h) using a small amount of additive against the cement mass;

i)increasing of resistance to aggressive environmental factors.

Having in view the above mentioned data, we can conclude that the concrete with siliconic elastomers of the I, II types and with siloxanic cycles give a complex, viable and profitable solution as they can be used in insuitable conditions: hydrotechnical constructions, civil and engineering buildings, and roads with concrete resistance and wear layers.

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Superficial Treatments to Rehabilitate Masonry and Concrete Structures

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Rezumat: Reabilitarea termicã a fondului construit existent reprezintã o preocupare importantã pentru constructorii zilelor noastre. Soluțiile practice propuse în lucrare se pot aplica elementelor afectate de excesul de umiditate realizate din zidãrie, respectiv din beton, şi constau în aplicarea unor filme de protecție cu caracter hidrofob remanent.

Abstract: Thermal rehabilitation of built constructions represents an important problem for nowadays constructors. Practical solutions indicated in the following paper can be applied on the constructional elements affected by humidity excess and consist in application on the support of some films having a permanently hydrophobia.

Keywords: *rehabilitation,, superficial treatment; topcoating systems, permanently hydrophobia.*

1. Introduction

Many historical sites with masonry structure were affected by natural corrosion. Stones containing $Ca(OH)_2$ and SiO_2 in their structure were supposed to support not only water' aggression, but also even biological aggression (blue and green alga make biological aggression). Both generate in the same time aesthetical damages and heat transfer problems.

Water's aggression mixed with air wastes and biological facts involve the protection problem. To limit the aggression some performant chemical products correlated with the masonry support were utilized and, in the same time, ensure an important protection against biological aggression without any modification of aesthetical aspect of constructional elements.

Macromolecular Institute "P. Poni, Jassy, has obtained products indicated for performant superficial treatments namely siloxanic cycles, polymer α –dyiole and the waste of type alkali – methyl-silalonate. These products, in solutions, can be applied on support having natural humidity or more; after application, 2…3 weeks, a fine film impregnated in the pored support had a hydrophobic effect.

By the other hand, the interest for thermal rehabilitation is a result of meeting the user's requirements, as a consequence of either the partial or total change of the buildings' initial destination or increasing the comfort degree.

The masonry and reinforced concrete structures are frequently affected by the unfavorable effects of humidity, because of the exterior microclimate and interior use of built spaces.

The avoidance of local condense effects may lead, however s the specialty literature mentions, to work which require a large volume of manpower, consumption of energo-intensive materials and do not lead to a partial or definitive removal of the condense phenomenon.

The practical solutions proposed present local chemical treatments, which, from the point of view of efficiency give favorable results and might constitute real solutions in the treatment of the walls.

2. Masonry- Historical Sites Rehabilitation

As *support* were analyzed masonries made by limestone blocks, grit stone blocks with Ca and gritstone with Si and marble blocks. As physical and thermal properties, are interesting for experiments: porosity, water's absorption and thermal conductivity

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(of support) [6], [7]. The technical properties of support are shown in Table 1.

Table 1

Blocks' property	Lime Stone	Grit Stone with Ca	Grit Stone with Si	Marble
Porosity [%]	0.5…13.5	5…28	5…28	0.8…1.13
Water absorption [%]	0.39…7.88	1.92…7.81	1.92…7.81	9.20…23.41
Heat conductivity [W/mK]	8.5…10.6	17	17	4.3

2.1 Superficial treatments

Superficial treatments were used in the following solutions: siloxanic polymers in toluene solutions, Androsil Z, Aseptina A – solution 3% in n-buthanol 50% and Hyamine 1622 [6], [7].

The solutions' consumption [g/cm^2], as function of supports' porosity [%] is shown in Table 2.

Table 2

Limestone type	Pore volume [%]	Consumption [g/cm^2]
Podeni	15.12	850.45
Viiştea	12.15	728.14
Başchior	5.01	204.45

Since our days applied treatments have in view to create a protection film or pores' collision.

After 60 cycles in laboratory conditions of accelerated olden process, by using the mentioned products, the hydrophobia effect remains active.

Notes:

- In the same reason were performed experiments with siliconic consolidated products as Androsil Z and Aseptina A, applied after cleaning support with a mechanic brush and water at high pressure, having the goal to increase the suction's ratio of support.

Applied on support, these products generate a film with anti-microbiological effect that increase the corrosion's resistance and ensure the maintenance of the aesthetic aspect of constructional elements.

- With the same effects, other kind of superficial treatments, can be performed by using poly-organically – siloxanes with a high degree of polycondensation, like toluene solution (15 and 2%) from the waste of siloxane cycles' unpolymerizated processes.

This kind of treatment was used on the old masonry of "Trei Ierarhi" Monastery in Jassy, with good results [7].

2.2 General Conclusions

In order to protect the aesthetical aspect of older masonry made by lime, grit stone or marble blocks, can be used, in solutions, high performance products like Aseptina A and Androsil Z.

These solutions, after application on the cleaned support, generate a pellicle film that have in the same time, hydrophobic effect and also, anti-microbiological effect.

The consumptions are smaller than the adsorption rate of each support.

3. Coating Systems for Masonry

3.1 Introduction

The coating systems [2], [7] are indicated for masonries in order to protect them.

In this purpose there are indicated mixtures of inorganic materials and various resins (epoxi, polyurethane, siloxanes, etc.) applied on support by spraying or roller. The thickness of protective film must be of 0.3 … 1.5 mm.

The durability of coating systems depends on the compound of the admixture especially on the type of catalyst, pigment, fine aggregates and their mixing ration.

The types of coating systems confer to the wall – coating system higher waterproofing and flexibility to follow the micro-movements of substrates.

The main analyzed systems are:

a) Admixtures of inorganic materials (Portland cement, gypsum plaster, sodium silicate with silicate sand.

b) Mixture of acrylic emulsion and polymer emulsion with silicate sand and emulsion of elastomers obtained by cold vulcanizing of siliconic rubber with catalyst (Sn octoate) with Portland cement and silicate sand.

c) Mixture of acrylic emulsion, polyvinyl emulsion or solvent type epoxi – polyurethane with silicate sand, limestone.

d) Mixture of acrylic chloroprene rubber, emulsion or solvent type polyurethane with calcium carbonate powder or silicate sand.

3.2 Performance Tests

The selection of systems had in view the following aspects:

a) All presented systems at the previous section are available now on the market.

b) All systems can lose in time their initial properties under the exploitation's conditions.

Performance tests were made for outdoor exposure and in laboratory.

From the first category it must be mentioned the following: ordinary exposure (30^0 and 90^0 facing South, vertical South and North) under successive artificially dew condensed conditions and under cyclic elongation and shrinkage condition specifically for coating films.

From the second category it must be mentioned the following test: at sunshine conditions created by exposure to carbon arc, fluorescent conditions created by exposure at artificially lamps and high temperatures.

3.3 Consequences

The interpretation performance test refers to the following important aspects: the main role of top coating, the effect of cyclic loading and the effect of temperature.

Thus, top coating with the total thickness of maximum 0.1 mm applied by rolling or spraying has the main role *to prevent the deterioration of the support at the complex action of water and ultraviolet radiation.* Experimentally, the performance of top coatings is variable for each type and external temperature condition.

The effect of cyclic loadings consists of elongation capability decreasing under laboratory conditions. Initial elongation was of 120%; after cyclic loading it was of 50%, the failure mode was both on top coating and on general system top coating-support.

The temperature effect is remarkable on reducing elongation capability of the top coating. This effect is moderated only when using main compound of mixture of linear siloxane cycles obtained by cold vulcanizing process of siliconic rubber.

Generally, the service of top coating systems is in range of 2…15 years depending both on technical properties of the support and type of used admixture products.

4. Experiments for Concrete Elements

In other experiments, author has utilized the chemical substances mentioned in this paper, respectively classic siliconic solution and siliconical elastomer [2], [3], [5].

4.1 Superficial treatments

For the beginning, the concrete support must be very well cleaned mechanically (affected by the condense effect), then a polymeric solution of siliconic elastomer obtained by the cold vulcanizing of siliconic rubber should be applied on the surface, followed by a cement mortar pellicle with siliconic additive of the same type (1% of silicatic cement Portland 32.5). After application, for seven days, the treated area must be maintained fewer than 95% moisture conditions.

The structure of the silico-organic polymers used is:

$$-\mathrm{O}-\underset{\mathrm{R}}{\overset{\mathrm{R}}{\mathrm{Si}}}-\mathrm{O}-, \text{ where } R \equiv CH_3(OH_3COO)_2 .$$

The siloxanic bond (Si-O) is remarkably resistant thermally and chemically which leads to maintaining the physical-chemical and mechanical properties in temperature range of – 50^0 … + 300^0C. The penetration of polymeric solutions in the support layer during the superficial chemical treatment leads to the consolidation of its crystalline grid by superposing the spatial grid of polymers. The polymer reticulation velocity is "regulated" by catalysts in whose presence siliconic polymers are used [2], [3].

The exterior / interior film, i.e., the cement mortar with additives, maintains its hydrophobia [4] enabling the support to "work" in service under balanced hygroscopic humidity conditions [7]. At the same time, the films, i.e. the mortars with additives, allow the mass transfer in the closure elements structure without being a barrier for water vapors.

The consumption of polymeric solution used in the superficial treatment is 0.013 g/cm^2 and 0.024 g/cm^2.

4.2 Conclusions

The covering of concrete walls with polymers determines an increase of the durability of these materials by permeability decrease ($1 \cdot 10^{-5}$ and $0.1 \cdot 10^{-5}$ cm/s), an increase of frost - thawing resistance, an increase of resistance to mechanical wear – important especially for façades exposed to combined action (wind - water), an improved behavior, when exposed to destructive chemical agents.

The superficial treatment can be applied on cleaned support without any other adhering layer, as they are highly adhesive.

Although the thermal characteristics of the support – finishing ensemble do not alter the condense, on the interior surface of the building element is removed as a consequence of the remanent hydrophobic effect of the polymer used which keeps a dray support [1].

An increase of the structural durability is obtained concomitantly with adequate maintenance / improvement of comfort conditions and aesthetic aspect applying superficial treatments with polymers.

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Aspects Regarding the Analysis of Shells Using a Hybrid-Strain Finite Element

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Rezumat: Articolul prezintă rezultatele obținute la modelarea plăcilor curbe cu elementul finit hibrid-deformații "HYBLAT-18", care lucrează în teoria plăcilor groase. Pentru crearea unei imagini cât mai exacte asupra acurateței elementului în cazul modelării acestor elemente structurale, sunt prezentate comparativ rezultatele analizei obținute cu ajutorul unui program nou creat, numit "HYFEM" [12], care este bazat pe acest element finit, cu cele obținute cu ajutorul a două programe comerciale consacrate "MSC-Nastran for Windows" v.4.5.1 şi "ROBOT Millennium" v.16.5.

Abstract: The paper presents the results obtained in modeling shells with a hybrid-strain finite element "HYBLAT-18", following the theory of thick plates. In order to create an image on the accuracy of the finite element, applied to model the behavior of those structural elements, the paper presents comparative results of the analysis with a new program, "HYFEM" [12], based on this finite element, and two acknowledged commercial programs "MSC – Nastran for Windows" v.4.5.1 respectively "ROBOT Millennium" v.16.5.

Keywords: *Finite elements analysis, Hybrid-strain approach, Shell, Thick plate.*

1. Introduction

This finite element, based on the proposals of J. D. Chiseler and A. Ghali [1], [2], [3], is formulated in detail in [9] and represented in Fig. 1.

The finite element, termed "HYBFLAT-18", has four nodes, situated within its median surface, and six nodal degrees of freedom, represented by translations (u, v şi w) on the global coordinate axes and rotations ($\theta_{x'}$, $\theta_{y'}$ şi $\theta_{z'}$), where x' şi y' represent a set of orthogonal axes within the median surface of the a plane element (thick plate or shell) that coincide with material's directions of orthotropy.

Coordinates x, y and z of a current point are interpolated through relationship:

$$\underset{(3\times1)}{\begin{Bmatrix} x \\ y \\ z \end{Bmatrix}} = \sum_{k=1}^{n} N_k \cdot \left[\begin{Bmatrix} x \\ y \\ z \end{Bmatrix}_k + t \cdot \frac{\zeta}{2} \cdot \begin{Bmatrix} l_3 \\ m_3 \\ n_3 \end{Bmatrix}_k \right] \tag{1}$$

where: k represents the node number, n is the number of nodes per element, $\{l_i, m_i, n_i\}$ for $i = 1, 2, 3$ are the direction cosines of local coordinate axes x', y' şi z' and t is the element thickness.

Bi-dimensional shape functions N_i of the izoparametric four nodes finite element are given by the formula:

$$N_i = \frac{1}{4} \cdot (1 + \xi\, \xi_i) \cdot (1 + \eta\, \eta_i), \text{ where } \xi_i, \eta_i = \pm 1 \tag{2}$$

Displacements $\{u\}$ on the element outline are interpolated according to nodal displacements $\{q\}$, through the isoparametrical interpolation functions matrix $[N]$, as follows:

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$$\begin{bmatrix} u \\ v \\ w \end{bmatrix} = \begin{bmatrix} \left. \begin{matrix} N_1 & \cdot & \cdot & -A_1 \cdot l_2 & A_1 \cdot l_1 & 0 \\ \cdot & N_1 & \cdot & -A_1 \cdot m_2 & A_1 \cdot m_1 & 0 \\ \cdot & \cdot & N_1 & -A_1 \cdot n_2 & A_1 \cdot n_1 & 0 \end{matrix} \right|_1 \cdots \left| \begin{matrix} N_4 & \cdot & \cdot & -A_4 \cdot l_2 & A_4 \cdot l_1 & 0 \\ \cdot & N_4 & \cdot & -A_4 \cdot m_2 & A_4 \cdot m_1 & 0 \\ \cdot & \cdot & N_4 & -A_4 \cdot n_2 & A_4 \cdot n_1 & 0 \end{matrix} \right|_4 \end{bmatrix} \cdot \begin{bmatrix} u_1 \\ v_1 \\ w_1 \\ \Theta_{x_1} \\ \Theta_{y_1} \\ \Theta_{z_1} \\ \hline \cdots \\ \hline u_4 \\ v_4 \\ w_4 \\ \Theta_{x_4} \\ \Theta_{y_4} \\ \Theta_{z_4} \end{bmatrix} \qquad (3)$$

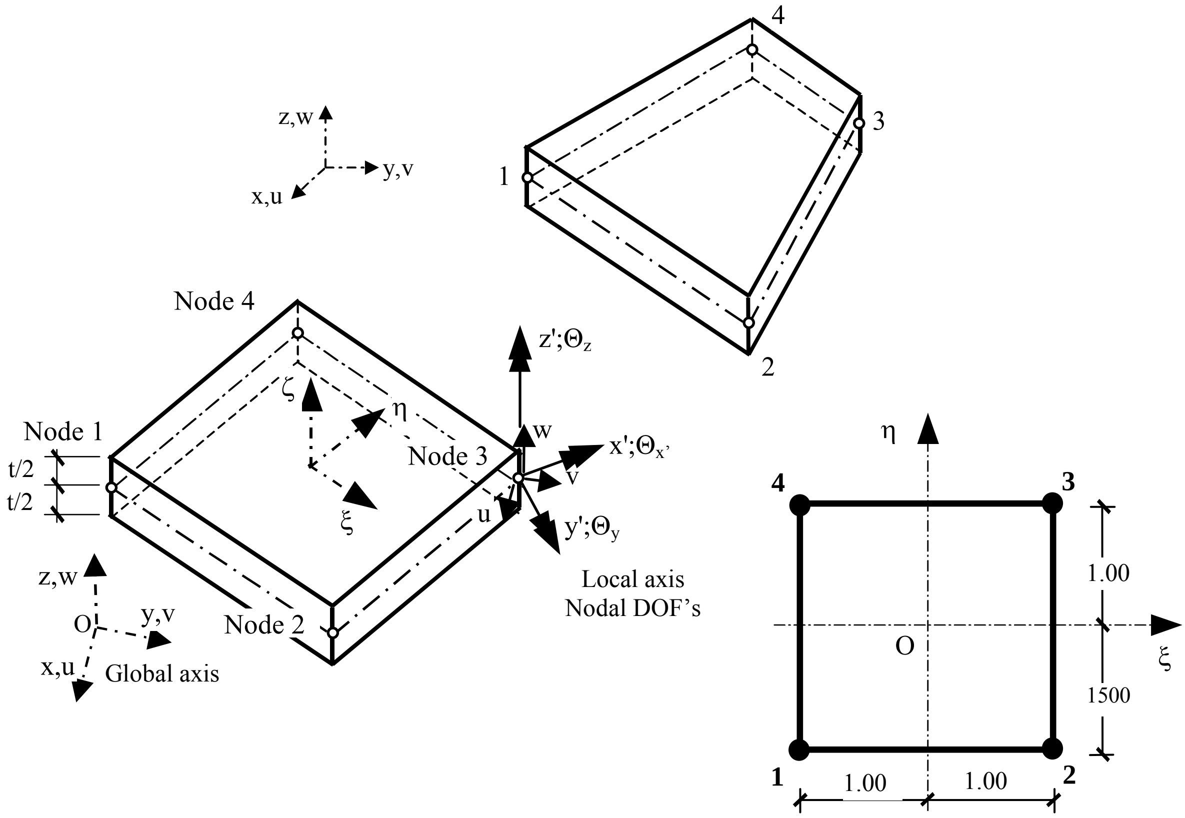


Fig. 1 Hybrid finite element for thick plates and shells, “HYBFLAT-18”

Strains $\{\varepsilon'\}$ are aproximated, starting from the parameters $[\alpha]$, as below:

$$\begin{bmatrix}\varepsilon_\xi\\ \varepsilon_\eta\\ \varepsilon_\zeta\\ \gamma_{\xi\eta}\\ \gamma_{\eta\zeta}\\ \gamma_{\zeta\xi}\end{bmatrix}=\begin{bmatrix}1&\eta&\zeta&\eta\cdot\zeta&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot\\ \cdot&\cdot&\cdot&\cdot&1&\xi&\zeta&\xi\cdot\zeta&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot\\ \cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&1&\xi&\eta&\xi\cdot\eta&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot\\ \cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&1&\zeta&\cdot&\cdot&\cdot&\cdot\\ \cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&1&\xi&\cdot&\cdot\\ \cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&\cdot&1&\eta\end{bmatrix}\cdot\begin{bmatrix}\alpha_1\\ \alpha_2\\ \alpha_3\\ \cdots\\ \alpha_{16}\\ \alpha_{17}\\ \alpha_{18}\end{bmatrix} \tag{4}$$

Material constitutive matrix is defined in local coordinates, with z' (sau ζ) considered as normal to element surface. For the case of an orthotropical material, having ξ şi η as directions of orthotropy, this matrix has the following form:

$$\underset{(6\times6)}{[D']}=\begin{bmatrix}e_{11}&e_{12}&e_{13}&0&0&0\\ e_{12}&e_{11}&e_{13}&0&0&0\\ e_{13}&e_{13}&e_{33}&0&0&0\\ 0&0&0&g_{13}&0&0\\ 0&0&0&0&g_{13}&0\\ 0&0&0&0&0&g_{33}\end{bmatrix} \tag{5}$$

The stiffness matrix of the finite element

$$\underset{(24\times24)}{[K]}=\underset{(24\times18)}{[G]}^T\cdot\underset{(18\times18)}{[K^*]}^{-1}\cdot\underset{(18\times24)}{[G]} \tag{6}$$

is obtained by using the following two matrices:

- overall stiffness matrix:

$$\underset{(18\times18)}{[K^*]}=\int_V\underset{(18\times6)}{[P]^T}\cdot\underset{(6\times6)}{[D]}\cdot\underset{(6\times18)}{[P]}\cdot dV \tag{7}$$

- transformation matrix:

$$\underset{(18\times24)}{[G]}=\int_V\underset{(18\times6)}{[P]^T}\cdot\underset{(6\times6)}{[D]}\cdot\left(\underset{(6\times3)}{[L]}\cdot\underset{(3\times24)}{[N]}\right)\cdot dV \tag{8}$$

After assambling the global stiffness matrix, it is possible to compute the global solution for the displacements vector of the entire structure $\{U\}$, using the classical equation of a finite element formulation

$$[K]\cdot\{U\}=\{F\} \tag{9}$$

where

$$\{F\}=\int_V\underset{(24x3)}{[N]^{\mathrm{T}}}\cdot\{b\}\cdot dV+\int_{S_t}\underset{(24\times3)}{[N]^{\mathrm{T}}}\cdot\{t\}\cdot ds \tag{10}$$

is the nodal loads vector. Than, the stresses on each element may be computed as:

$$\underset{(6\times1)}{\{\sigma\}}=\underset{(6\times6)}{[D]}\cdot\underset{(6\times18)}{[P]}\cdot\left(\underset{(24\times18)}{[G]}^T\cdot\underset{(18\times18)}{[K^*]}^{-1}\right)^T\cdot\underset{(24\times1)}{\{q\}} \tag{11}$$

At the end of this brief presentation of the hybrid - strain finite element „HYBFLAT-18", meant to the analysis of plane and curved plates in the theory of thick plates, we emphasise that the element insure the auto-equilibrium and inter-element continuity of the displacements, stresses and internal actions.

2. Numerical Results

In order to obtain a clear image on the efficiency and the accuracy of the solutions obtained with the hybrid-strain plane element "HYBFLAT-18" for the analysis of shells, two categories of numerical test were carried out. First, were performed some "classic" tests for shells modeling, followed by "user" – type tests, corresponding to the user modeling needs.

(a) "Classic" tests for shells

The single span cylindrical roof, supported on tympani with gravitational load (own – weight), represented in Fig. 2, was first proposed by A.C. Scordelis and K.S. Lo (1964) and, subsequently analyzed by other researchers in the finite elements field: H. Tottenham and C. Brebbia (1971), R Cook (1974), R.H. MacNeal and R.L. Hartner (1985), O.C. Zienkiewitz and R. Taylor (1991).

Description:

Length = 50 ft = 15,24 m;

Radius = 25 ft = 7,62 m;

Thickness = 3 inch = 7,62 cm;

Modulus of elasticity = $4{,}32\times10^{8}$ psf =

$$= 2{,}1092\times10^{7}\ \frac{\text{kN}}{\text{m}^{2}};$$

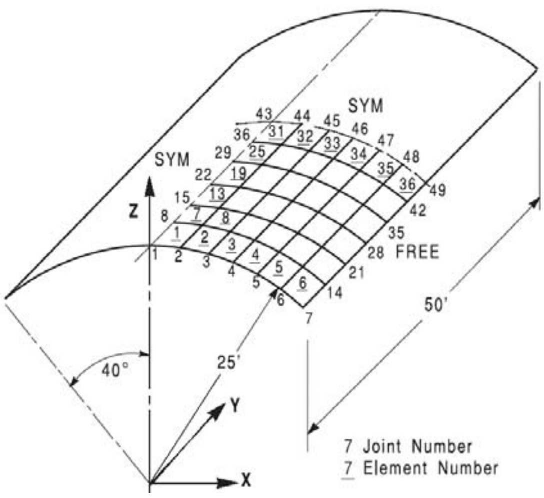
Poisson's ratio = 0;

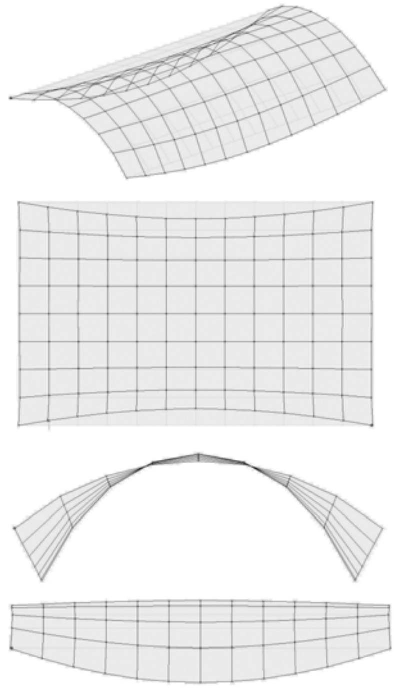
Gravity load = 90 psf $= 4{,}394\,\frac{\text{kN}}{\text{m}^{2}}$

(uniform on surface area);

Boundary condition:

simply supported on curved edges.





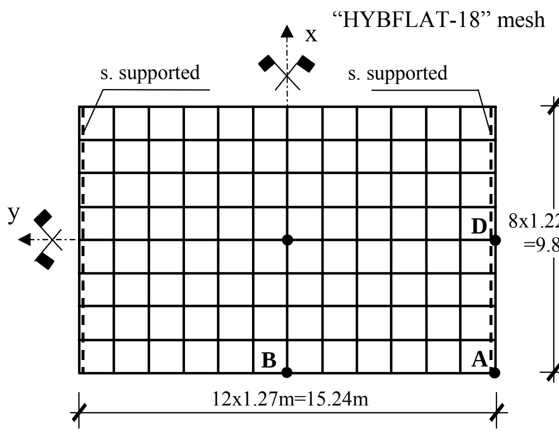


Fig. 2 The cylindrical SCORDELIS-LO roof

The results of the analysis are synthesized in Fig. 3. It can be noticed the finite element "HYBFLAT – 18" leads to convergent and good solutions with respect to the theoretical values and close to those delivered by "ROBOT Millennium" v.16.5, respectively "MSC Nastran" v.4.5.1, unanimously recognized as two of the most preferment commercial programs in the structural analysis

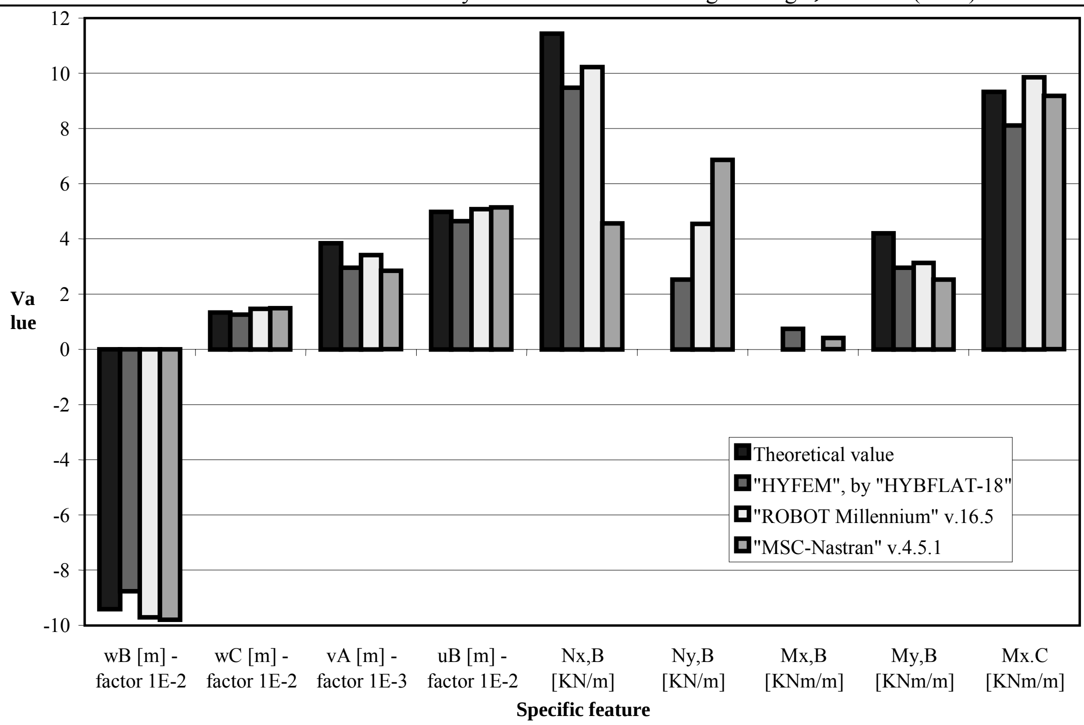
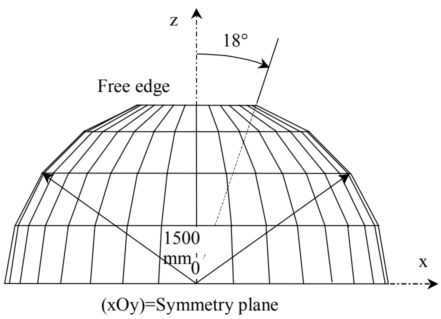


Fig. 3 Solutions for the cylindrical SCORDELIS-LO roof

The hemispherical shell loaded by local loads (Fig. 4), was proposed by R. H. MacNeal and R. L. Harter (1985) and subsequently analyzed, among others, by O. C. Zienkiewicz and R. Taylor (1991). The analyzed model represents a sphere, open at the upper and inferior level, subject to two symmetrical pairs of concentrated loads that induce stretching on one direction and compression on the corresponding orthogonal direction.

Description:
Radius =10,0 m;
Thickness = 4 cm;
Modulus of elasticity $= 6{,}825 \times 10^{7}\ \dfrac{\text{kN}}{\text{m}^{2}}$;
Poisson's ratio = 0,3;
Loading: concentrated forces as shown.



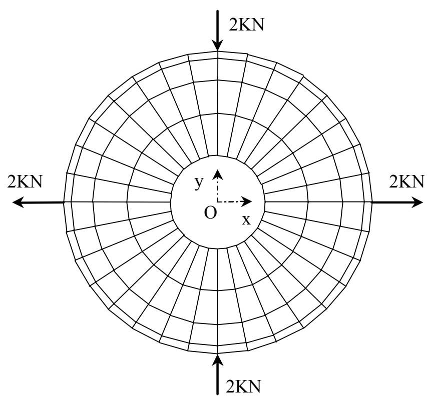
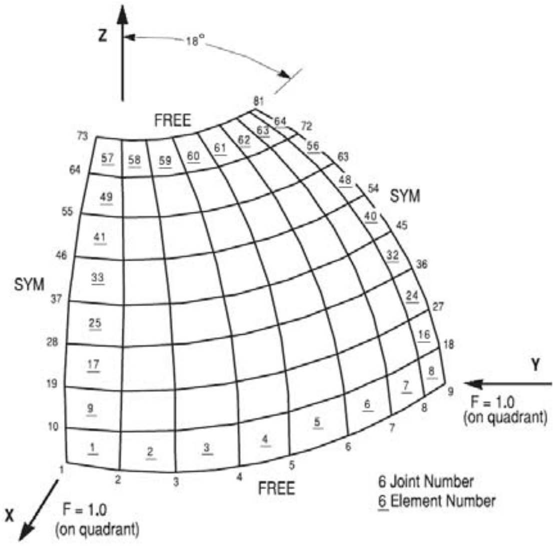


Fig. 4 The hemispherical shell MacNEAL-HARTER



The results of the analysis, synthesized in Fig. 5, show that the finite element "HYBFLAT-18" offer very good solutions for both discretization cases, comparable to those obtained with the programs "ROBOT Millennium" v.16.5 and "MSC-Nastran" v.4.5.1, well acknowledged in the finite elements analysis.

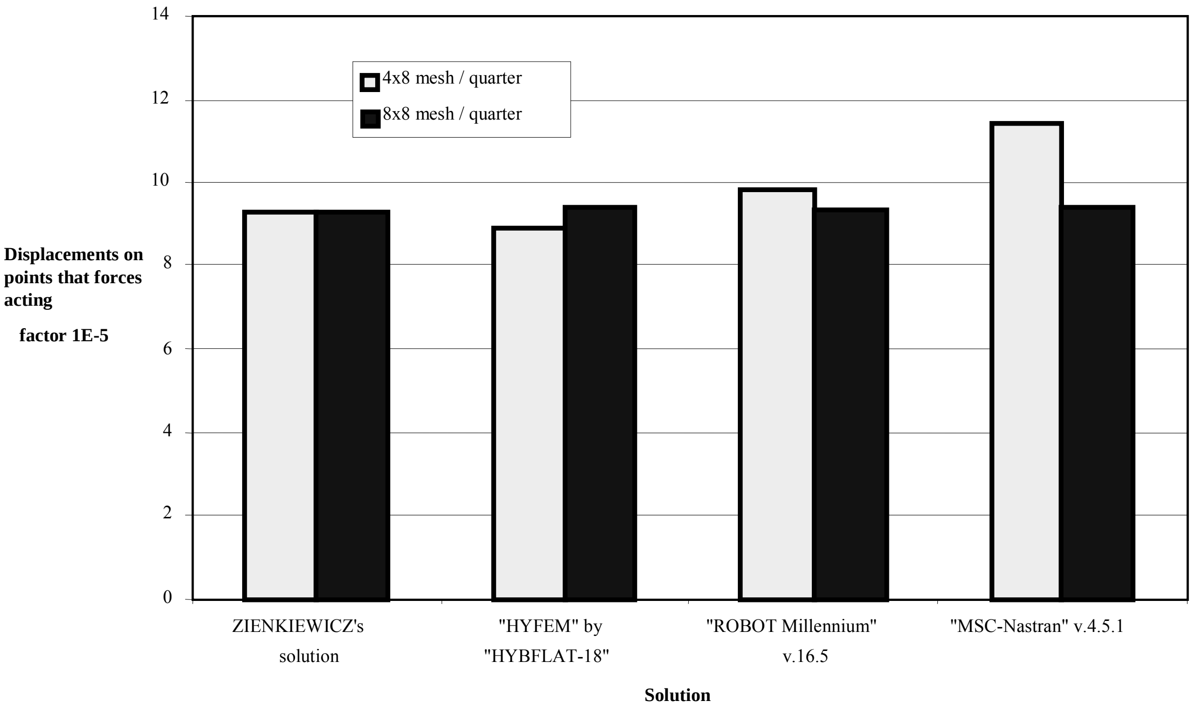
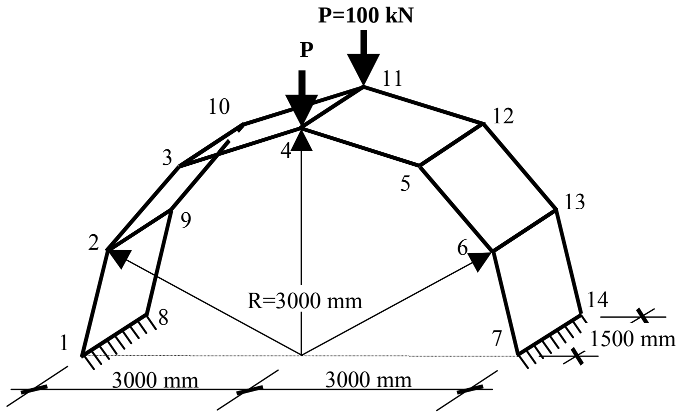


Fig. 5 Solutions for the hemispherical shell MacNEAL-HARTER

(b) Some "user's" tests for shells

One of the tests performed with the finite element "HYBFLAT-18" is described in Fig. 6.



Description:

Material: steel OL37;

Thickness: 30 to 100 mm (by 5 mm).

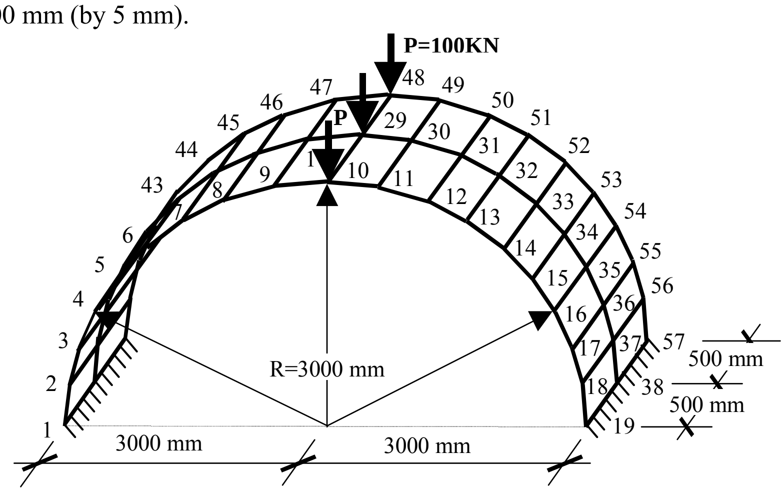


Fig. 6 Two "user's" tests for shells

3. Conclusions

The finite element "HYBFLAT-18", based on a hybrid – strain approach in the theory of thick plates, offers very good solutions not only for the classical shell problems but also for particular "user" problems. The results are comparable to those delivered by the program "ROBOT Millennium" v.16.5 and "MSC-Nastran" v.4.5.1, well acknowledged in the finite element analysis.

The hybrid – strain finite element "HYBFLAT-18" impose itself as an efficient and performant alternative in the finite element analysis of shells, approached in the theory of thick plates.

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Roof-surfaces Generated by Blending Interpolation

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Rezumat: Suprafețele de tip mixt (blending) ocupă un loc important în modelarea geometrică asistată de calculator, fiind frecvent utilizate în activități cum ar fi: proiectarea în construcții, arhitectură, design în industria auto, navală, aeronautică, etc. În lucrare este prezentat un procedeu de generare a unei familii de suprafețe de tip Hermite şi Birkhoff. Sunt exemplificate grafic câteva cazuri particulare ale acestor suprafețe.

Abstract: Blending surfaces are very important in geometric modeling and are widely used in different activities such as civil engineering, industries of automobiles, ships, airplanes, industrial design and other theoretical and practical situations. The paper presents a procedure for defining a family of Hermite and Birkhoff interpolation type. Particular cases of these surfaces are illustrated graphically.

Keywords: *Free-form surfaces, interpolation, Computer Aided Geometric Design (CAGD).*

1. Introduction:

The possibility of designing free-form surfaces with the aid of computers has led to new methods for defining surfaces of the following types: Bezier, B-spline [3], Shepard [9], blending (Coons and Gordon) [12], and others. In our paper we present a procedure for defining a family of interpolation surface of Hermite and Birkhoff type.

2. Surface with two support curves and tangent ribbons:

In this section we define a family of surfaces each one containing the same two opposite space curves, say (C_1) and (C_2) and different tangent ribbons (across-boundary derivatives), Figure 1.

The curves (C_1) and (C_2) are represented by the equations:

$$(C_1)\ \begin{cases} y = 0 \\ z = h_0(x) \end{cases} \text{ and } (C_2)\ \begin{cases} y = y_1(x) \\ z = h_1(x), \end{cases} \qquad (1)$$

$x \in [0,a]$,

For the given functions $h_0(x),\ h_1(x),\ m_0(x),\ m_1(x)$ and $y_1(x),\ x \in [0,a]$, let us find the surface (S), of

equation $z = f(x,y),\quad x \in [0,a],\quad y \in [0, y_1(x)]$, which satisfies the following conditions:

$$\begin{aligned} &f(x,0) = h_0(x) \\ &f(x, y_1(x)) = h_1(x) \\ &f'_y(x,0) = m_0(x) \\ &f''_{y^2}(x, y_1(x)) = m_1(x),\ x \in [0,a]. \end{aligned} \qquad (2)$$

The unique solution to the problem (2) is the third-degree Hermite interpolation polynomial with respect to y-variable. By direct calculus on obtains:

$$\begin{aligned} &f(x, y;\ y_1(x),\ h_0(x),\ h_1(x),\ m_0(x),\ m_1(x)) \\ &= \frac{[m_0(x)+m_1(x)]y_1(x) - 2[h_1(x)-h_0(x)]}{y_1^3(x)} y^3 + \\ &+ \frac{3[h_1(x)-h_0(x)] - y_1(x)[2m_0(x)+m_1(x)]}{y_1^2(x)} y^2 + \\ &+ m_0(x)y + h_0(x). \end{aligned} \qquad (3)$$

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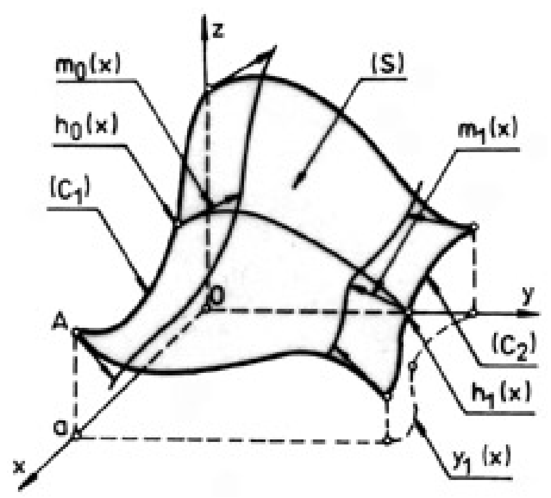

Fig.1. A surface with two support curves and tangent ribbons

In cardinal form [9], the polynomial f is:

$$
\begin{aligned}
& f\left(x,y;y_1(x),h_0(x),h_1(x),m_0(x),m_1(x)\right)= \\
& = H_3^0\left(y;y_1(x)\right)h_0(x)+H_3^1(y;y_1(x)m_0(x)+ \\
& + H_3^2\left(y,y_1(x)\right)m_1(x)+H_3^3\left(y;y_1(x)\right)h_1(x),
\end{aligned}
\tag{4}
$$

where the cardinal (blending) function

$H_3^i\left(y;y_1(x)\right), \quad i=\overline{0,3}$ are

$$
\begin{aligned}
& H_3^0\left(y,y_1(x)\right)=\frac{\left[y_1(x)-y\right]^2\left[2y+y_1(x)\right]}{y_1^3(x)} \\
& H_3^1\left(y,y_1(x)\right)=\frac{\left[y_1(x)-y\right]^2 y}{y_1^2(x)} \\
& H_3^2\left(y,y_1(x)\right)=\frac{\left[y-y_1(x)\right]y^2}{y_1^2(x)} \\
& H_3^3\left(y;y_1(x)\right)=\frac{y^2\left[3y_1(x)-2y\right]}{y_1^3(x)}, \\
& x\in[0,a], \quad y\in\left[0,y_1(x)\right].
\end{aligned}
\tag{5}
$$

The equation:

$$
\begin{aligned}
& z= f\left[x,y;y_1(x),h_0(x),h_1(x),h_1(x),m_0(x),m_1(x)\right] \\
& x\in[0,a], \\
& y\in\left[0,y_1(x)\right],
\end{aligned}
$$

where f is given by Eq. (3) or Eq. (4), represents a family of surfaces which depend on
$y_1(x), \ h_0(x), \ h_1(x), \ m_0(x) \quad and \quad m_1(x).$

The function $m_0(x)$ and $m_1(x)$ determine the shape of each surface.

Observation:

a) The equation of surface (S) and its symmetric with respect to xOz plane has the equation:

$$
\begin{aligned}
& z= f\left(x,|y|;y_1(x),h_0(x),h_1(x),m_0(x),m_1(x)\right), \\
& where \quad y\in[0,a], \quad |y|\le y_1(x).
\end{aligned}
\tag{6}
$$

b) A surface (S) and its symmetric with respect to yOz plane is represented by the equation:

$$
\begin{aligned}
& z= f\left(|x|,y;y_1(|x|),h_0(|x|),h_1(|x|),m_0(|x|),m_1(|x|)\right), \\
& where \quad |x|\le a, \quad 0\le y\le y_1(x)
\end{aligned}
\tag{7}
$$

c) The equation of surface (S) and its symmetric with respect to xOz and yOz planes is:

$$
\begin{aligned}
& z= f\left(|x|,|y|;y_1(|x|),h_0(|x|),h_1(|x|),m_0(|x|),m_1(|x|)\right), \\
& where \quad |x|\le a, \quad |y|\le y_1(x)
\end{aligned}
\tag{8}
$$

In figure 2 is represented the surface (S_A) from the family Eq.(8), where f is given by Eq.(3), corresponding to the following data:

$$
\begin{aligned}
& h_0(x)=\frac{4}{75}(x-15)(x-20)-\frac{4}{125}x(x-20)+\frac{4}{125}x(x-15) \\
& h_1(x)=1+\frac{1}{2}sin\frac{2\pi}{15}\left(x+\frac{3}{2}\right) \\
& m_0(x)=\frac{1}{6}cos\frac{\pi}{5}(x+1) \\
& m_1(x)=-\frac{1}{4} \\
& a=20, \quad y_1(x)=20.
\end{aligned}
$$

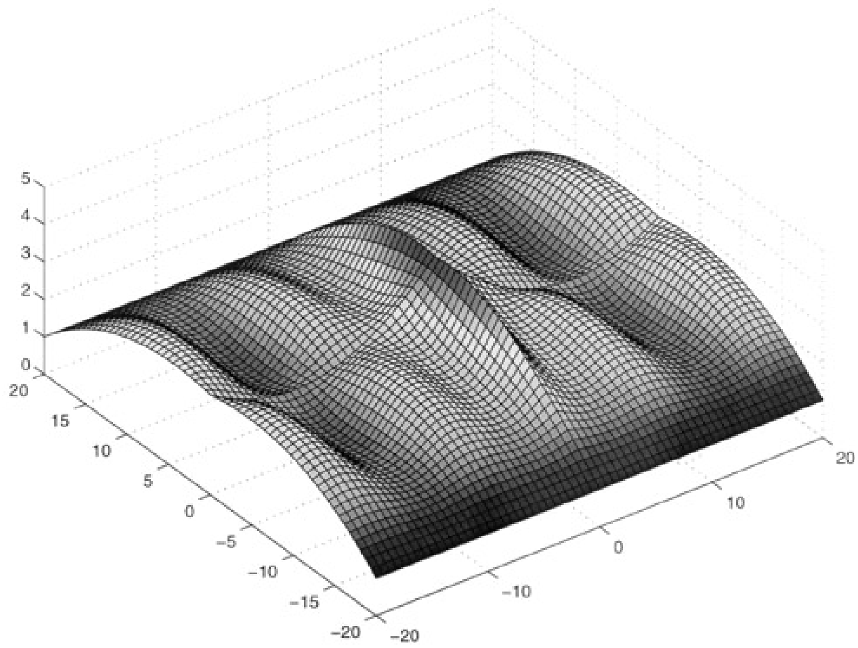

Fig. 2. The surface (S_A) from the family Eq. (8)

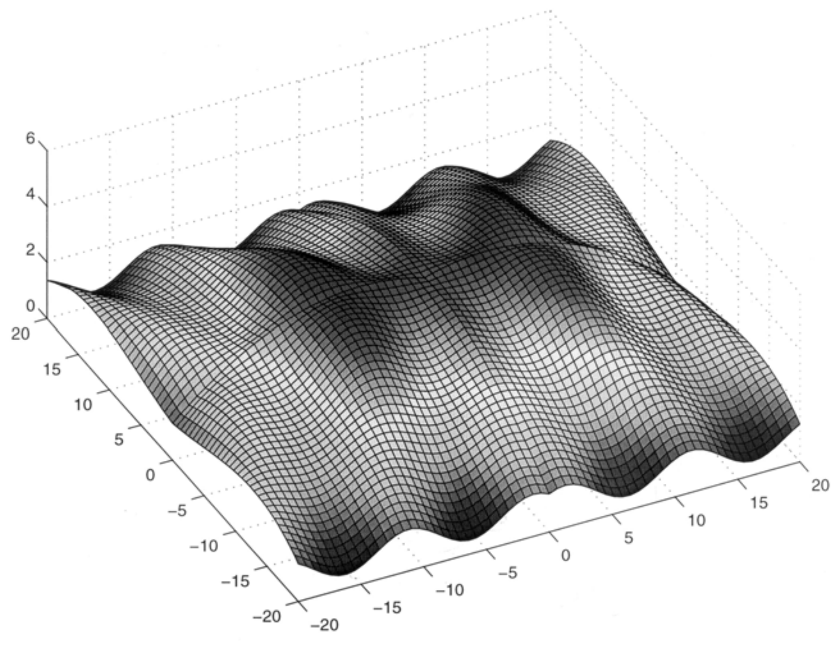

Fig. 3. The surface (S_B) from the family Eq.(8)

In Figure 3 is represented the surface (S_B) from the family Eq. (8), where f is given by Eq. (3), corresponding to the following data:

$$h_0(x) = \frac{(x-15)(x-20)}{75} - \frac{x(x-20)}{15}$$
$$h_1(x) = 1 + \frac{1}{2} sin \frac{\pi}{5}\left(x + \frac{3}{2}\right)$$
$$m_0(x) = \frac{1}{6} cos \frac{\pi}{5}(x+2)$$
$$m_1(x) = \frac{1}{4}$$
$$a = 20, \quad y_1(x) = 20$$

Figure 4 shows the surface (S_C) from the family Eq. (8), where f is given by Eq. (3), corresponding to the following data:

$$h_0(x) = \frac{-3x}{30} + 12$$
$$h_1(x) = 0$$
$$m_0(x) = \frac{1}{4} sin \frac{\pi}{5} x$$
$$m_1(x) = \frac{1}{6} cos \frac{\pi}{5}(x+2)$$
$$a = 20, \quad y_1(x) = 12.$$

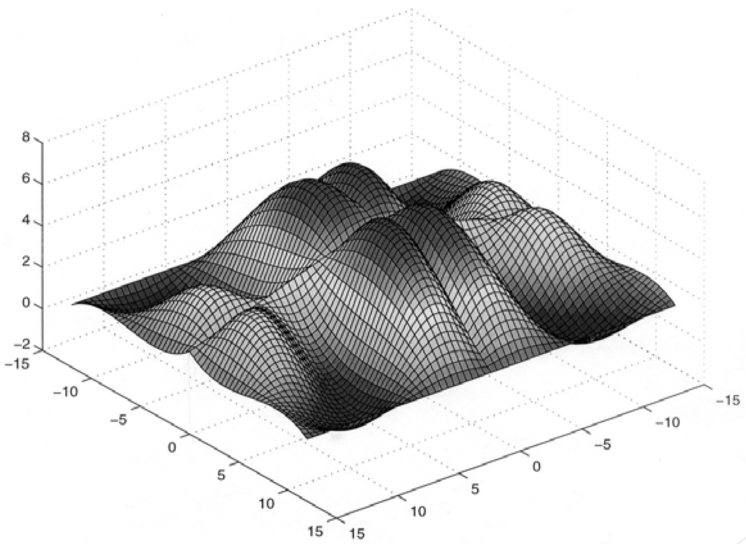

Fig. 4. The surface (S_C) from the family (8)

In Figure 5 is represented the surface (S_D) from the family Eq. (8), where f is given by Eq. (3), corresponding to the following data:

$$h_0(x) = -\frac{3x}{20} + 12$$
$$h_1(x) = 1$$
$$m_0(x) = -\frac{1}{2}$$
$$m_1(x) = -\frac{1}{4} sin \frac{\pi x}{5}$$
$$a = 20, \quad y_1(x) = 20.$$

Figure 6 shows the surface (S_E) from the family Eq. (8), where f is given by Eq. (3), corresponding to the following data:

$$h_0(x) = -\frac{3x}{20} + 12$$
$$h_1(x) = 1$$
$$m_0(x) = \frac{1}{6} sin \frac{\pi}{4} x$$
$$m_1(x) = \frac{1}{4}$$
$$a = 20, \quad y_1(x) = 20.$$

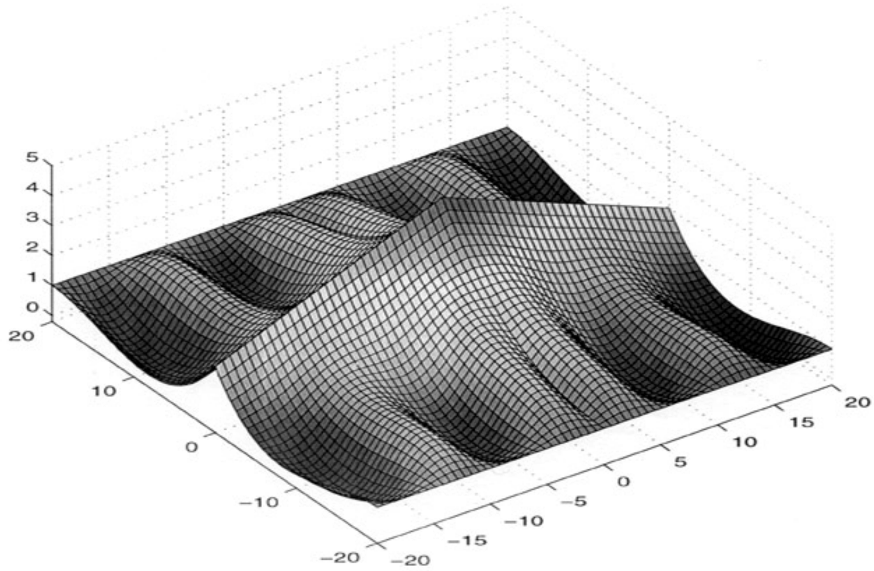

Fig. 5. The surface (S_D) from the family Eq. (8)

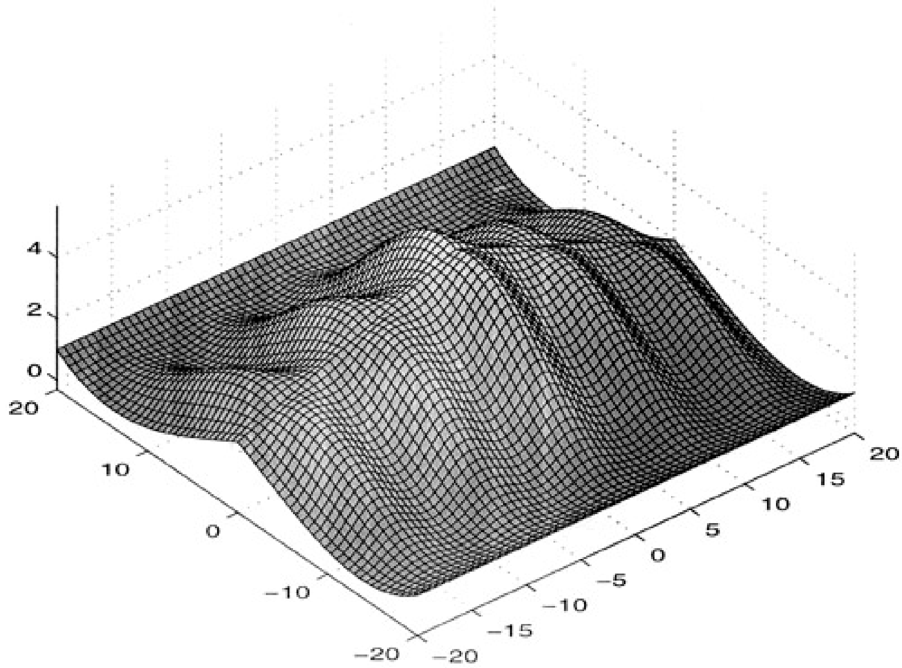

Fig. 6. The surface (S_E) from the family Eq. (8)

3. Conclusion

The field of conceiving and designing structures in constructions aims at finding some optimum constructive solution to achieve the highest performances with minimum expenses and materials.

The geometrical, structural and aesthetically characteristics of the studied surfaces made them useful in the study of the shell roofs.

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The Actual Stage of the Hygrothermal Design Automation of the Buildings

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Rezumat: În lucrare se prezintă principalele programe de calcul higrotermic al construcțiilor elaborate de autori, în perioada 1978 -2004, în urma unor colaborări pe linie de cercetare cu principalele firme de execuție, proiectare, cercetare şi învațământ în domeniul construcțiilor din țară.

Utilizarea intensivă a programelor elaborate, pentru un număr mare de aplicații practice, demonstrează necesitatea şi oportunitatea elaborării acestui valoros instrument de calcul pus la îndemâna specialiştilor higrotermicieni din țară şi de peste hotare. Problematica rezolvată şi performanțele obținute prin utilizarea acestor programe de calcul situează specialiştii higrotermicieni din România printre vârfurile cercetătorilor mondiali în acest domeniu de specialitate.

Abstract: The present paper presents the main hygrothermal calculation programs of the construction, elaborated by the authors between 1978-2004. They are the result of a research collaboration with the main companies of execution, design ,research abd education in the construction domain in our country.
The intensive use of the programs elaborated for a large number of practical applications,demonstrates the necessity and oportunity of elaborating this valuable calculation,instrument put at hygrothermal specialists' hands both in our country and abroad.The solved problems and the results obtained by using the calculation programs place the hygrothermal specialists in Romania among the tops of the world researchers in this domain of speciality.

Keywords: *building physics, thermal rehabilitation, thermoenergetics of buildings.*

1.Introduction

The major importance of the energetical criterion in analysing the efficiency of the conformation of the buildings, in plane and space, needs to put some new methods and technical data at the designers'and researchers' disposal. These methods and technical data should permit the taking into the consideration, even from the beginning of the design elaboration of the elements which lead both by hygrothermal measures and architectural composition, to the increase of the global thermal efficiency of the buildings, providing a rational thermal confort and a corresponding internal micro-climate.

In building design the major importance of the energetical criterion in ensuring the thermal confort as well as a corresponding internal microclimate does not requier additional justifications, representing a major factor for an optimum development of the activity within the building. In the conditions of the world energetical shortage the solving of the heat insulating of the constructions needs a rigurous knowledge of the energy losses through the external walls and elements of steam migration. The compensation of these losses by an additional heating or by subsequent hygrothermal rehabilitation of the buildings is too expensive. In this respect the correction of the thermal bridges and the optimization of a constructive structure of the high thermal permeability zones is necessary.

The knowing of the local effects and the elimination of their negative influence, even from the building design stage have very important economic consequences which are not to be demonstrated.
The determination with a satisfactory precision of the minimum and average resistance at the thermal transmission to the influence zone of the portions of an increased thermal permeability which can't be caught by the usual empirical experimental formulae, involves the knowledge of the plane or spatial field of

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temperature and water vapour difusion. This task of a complex analysis of the hygrothermal behaviour of the external walls were ascertained to the calculation programs written for macro mini and micro computers from the current endownment of the calculation centres. The programs for the hygrothermal calculation design of the buildings, represent a valuable calculation instrument at the hand of the constructors involved in the execution, desing, research and construction education activities. The elaborated calculation programs, established descrete values of the temperature and humidity field, in thermal-steady–state or non-steady-state regime in the knots of an orthogonal network with variable pitches, within which the physical and mathematical model of the analyzed element may be inscribed.

For the konwn contour conditions the calculation programs indicate the existence of the condense risk on the interior surface of the element, the quantity of condense in the mass of the element. These programs also calculate the energy losses through its opaque side. The programs have been theoretically proved and put into use in several variants, with utilizing handbooks, registered in the libraries at the national level under the code BN.INTER 900 in the period between May 1978-July 2000 at the requirement of the ICCPDC, INCERC, IPCT, MIS Bucharest and other interested users. By the attained performances, the programs of the hygrothermal calculation were at the level of the top requirements on a world plane, two of them having technical documentation in English, being delivered at export („CIMPPLAN" and „CIMPSAT" –1984 variant).

2. The List of the Main Programs for the Hygrothermal Design of the Buildings

In the following there are presented the names (terms)of the main hygrothermal calculation programs of the constructions and the year of the first version elaborated by the authors. They are members of the Physics of Constructions staff within the Civil Building department – Technical University of Cluj –Napoca.

2.1.Thermal Steady State Regime

2.1.1."CTEMP" program package for the analysis of the thermal steady state field in diverse thermal bridges, 1980;

2.1.2."CIMPPLAN" program package (software package) for the calculation and analysis of the temperature plane field in the thermal steady state regime for the external walls of complex structures, 1980;

2.1.3."CIMPCIL" program for the calculation of the temperature plane field in steady state conditions in cylindrical co-ordinates, 1981;

2.1.4."CIMPSAT" software package for the calculation and analysis of the thermal spatial field in thermal steady state regime for the shut-off devices of complex structures,1981.

2.2.Thermal Non-Steady State Regime

2.2.1."RENEST11" and "RENEST12" programs for the calculation of the unidirectional field of temperature in thermal non-steady state regime for construction elements with a multilayer structure, 1981;

2.2.2."RENESTL" program from the "RENEST" software package for the calculation of the unidirectional field in thermal non-steady state regime at constructions elements with a complex structure,1982;

2.2.3."RENESTP" program from the "RENEST" software package for the calculation and analysis of the temperature plane field in thermal non-steady state regime for the shut-off devices with a complex structure,1983;

2.2.4."RENESTS" program from the "RENEST" software package for the calculation and analysis of the temperature spatial field in thermal non-steady state regime for the external walls with a complex structures,1985;

2.2.5."TERMO" program for the calculation of heat storages and releases in the layers of structure members in thermal non-steady state regime for the shut-off devices with a complex structure, 1990;

2.2.6."RENTER" program for the calculation of the temperature uni-directional field in thermal non-steady state regime taking into consideration the sun energy through the external walls and windows,1991.

2.3.Condense Calculation

2.3.1.”CONDL” program from the ”CONDENS” software package for the calculation of the uni-directional difusion of steams through construction members having a complex structures, 1981;

2.3.2.”CONDP” program from the ”CONDENS” software package for the calculation of the plane difusion of steams through construction members having a complex structure, 1982;

2.3.3.”CONDS” program from the ”CONDENS” software package for the calculation of the spatial difusion of steams through construction members having a complex structure, 1983;

2.4. Non-Linear Hygrothermal Calculation (the variation of the thermal conductivity with the temperature and humidity)

2.4.1.”CAMPNEL” program from the ”CAMPNEL” software package for the calculation of the temperature unidirectional field in non-steady state regime for members having a complex structure, 1981;

2.4.2.”CAMPNEL” program from the ”CAMPNEL” software package for the calculation and analysis of the temperature non-linear plane field in non-steady state regime for the members with a complex structure, 1982;

2.4.3.”CAMPNELS” program from the ”CAMPNEL” software package for the calculation and analysis of the temperature non-linear spatial field in thermal non-steady state regime for members having a complex structure, 1982.

2.5. Thermal Optimization

2.5.1.”DITEEC” program for a thermo-economical optimization of the construction elements, 1980;

2.5.2.”OPTIMTEEC” program for the thermo-energetical optimization of the buildings, 1987.

2.6. Overall Thermotechnical Analysis of the Buildings

2.6.1.”GLOBAL” program for the evaluation of the overall coefficients of the heat losses for the whole building, 1992.

3.Performances

The calculation programs have been elaborated in three distinct stages:

-1977-1983 stage: the programs were elaborated for ”FELIX” computers in ”FORTRAN IV” and ”ASIRIS” programming language;

-1985-1990 stage: the programs were elaborated for the micro-computers from ”TPD”, ”M118”, ”CORAL”, ”INDEPENDENT” series in ”FORTRAN 77” programming language. These programs variants have conversational processors of introducting the data;

-1990-2000 stage: when the programs were implemented on PC compatible IBM computers.

The used Programming languages were ”FORTRAN 77”, ”PASCAL”, ”ASM” under ”DOS” operating system.

After the year 2000 the programming in ”DELPHI” language has been used. It permits a better use of the resources possesed by the new generation of computers.

The calculation programs are conceived for reading the input data in two ways: in the graphical conversational variant of introducing the data and in the variant of direct reading the input data from a file created with the help of a text editor. The obtained results may be numerically displayed in data tables and graphically by means of postprocessor conceived both for the screen display and for the printing of the graphical representations. We point out the fact that, the people who conceive computation (calculation) programs in the field of construction calculation, should consider the large number of the unknowns of the equation system generated by the mathematical models which describe the analysed physical phenomena. If for current applications the minimum number of the discretization network is about 30,000 points (knots), for complex applications the number of the knots of the calculation network increases to the order of hundreds of thousands. For PCs the variants of the performed programs permit a direct solving through the ”accurate” (exact) methods of the network with up to 12,000,000 knots and by superrelaxation, relation and iterative methods the number of the calculation points is theoretically unlimited.

The automatic calculation programs from the physics of buildings domain, elaborated since 1978, have a high degree of generalization fact that permitted their use in diverse technical applications required by many people (by those interested in those programs).

We mention the following:

- the field of temperature, the stress and strain state for the resistance structure of a building subjected to a fire or an explosion (resistance in time till the yield-crash);
- the field of temperature, the stress and strain state for bodies in motion(the piston of the heat engines);
- the calculation of the temperature state and plastic deformations of the materials subjected to low temperatures (Kryogeny);
-thermal problems connected to superconductivity;
-the calculation of the magnetic and electrostatic fields (metal coating on different surfaces chromium and nickel plating);
-temperature fields in ovens for thermal treatment applied to mechanical pieces;
-the effect of a instantaneous release of some powerful energy sources in contact with fixed pieces(explosion chambers) or moving pieces (powerful energy sources in contact with surfaces having different layered structure and different working degrees of the surfaces-shield);
-the study of the wetting-drying phenomena of the materials:drying ovens for wooden products;
-thermal protection of the cooling towers from Cernavoda nuclear power plant;
-thermo-technical design of the chimneys/smokestacks (over 50m).

4.Practical Applications

The thermo-energetical calculation programs have been applied in activities of technical expertise of the buildings, in research (civil, industrial and special buildings), designs of thermo-energetical rehabilitation of the buildings, the current thermo-energetical design of some new buildings, activity with students and specialists in heat transfer, elaboration of normatives,guides and calculation instructions in the above mentioned fields.

We present graphically the results obtained by using the following automatic calculation programs:

- Application of the programme CONDL (Fig.1-Fig.5).
- Application of the programme CIMPPLAN (Fig.6).
- Application of the programme CIMPSPAT (Fig.7-Fig.10) to a masonry wall.

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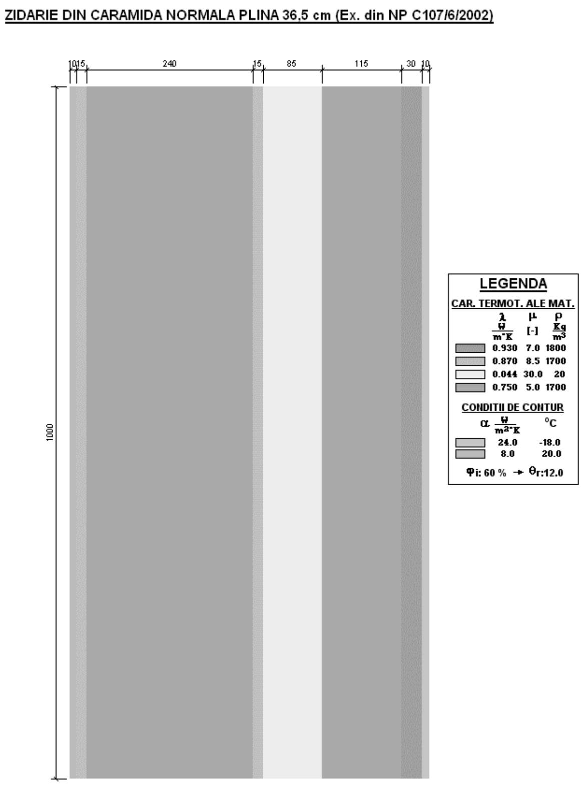

Figure 1 Termotechnical characteristics for the masonry wall made from bricks of 24 cm thick, thermal insulation from polystyrene of 8^5 cm thick and bricks of 11^5 cm thick, plastered on both sides.

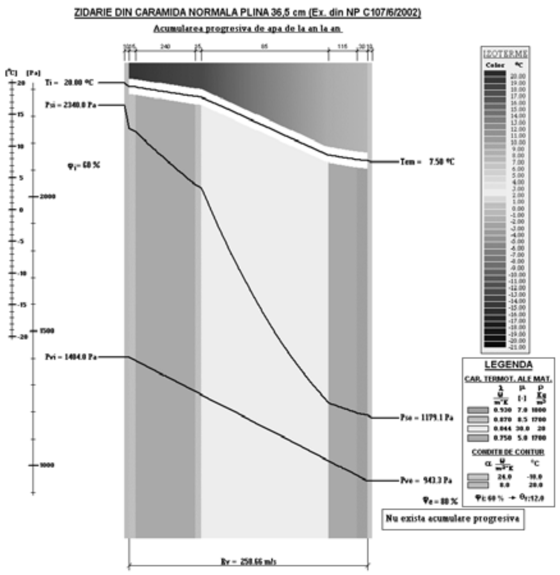

Figure 2 Non-accumulation of condense in the structure of the wall from year to year

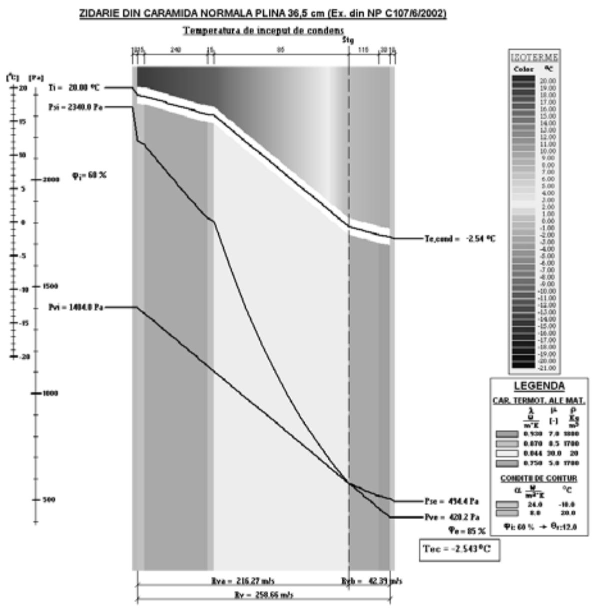

Figure 3 Beginning temperature of the condense

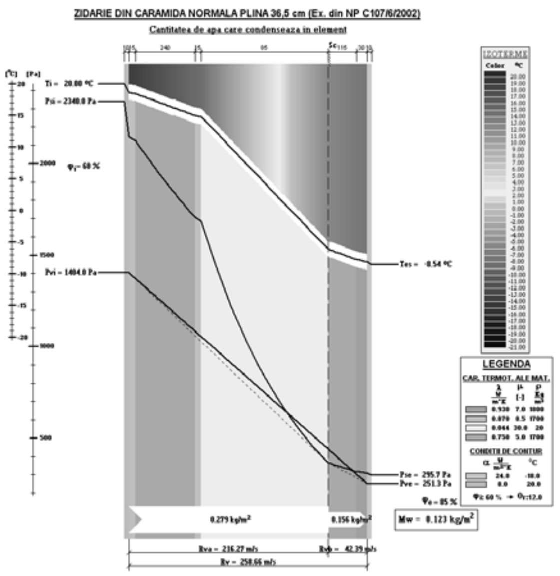

Figure 4 The quantity of condensed water in the cold period of the year

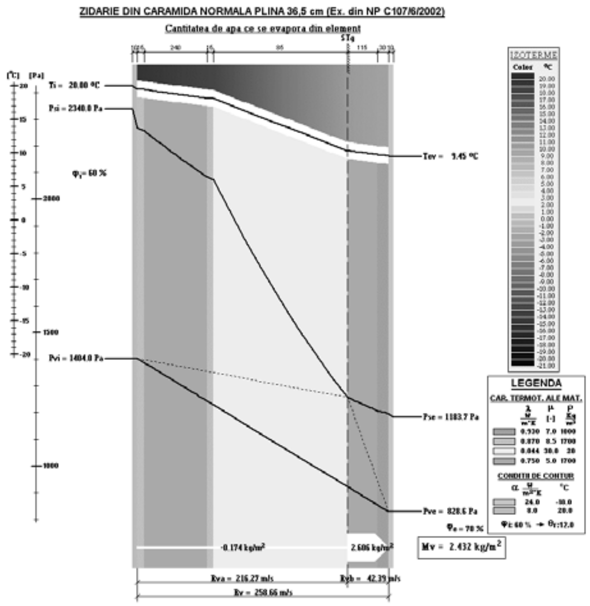

Figure 5 The quantity of evaporated water in the warm period of the year

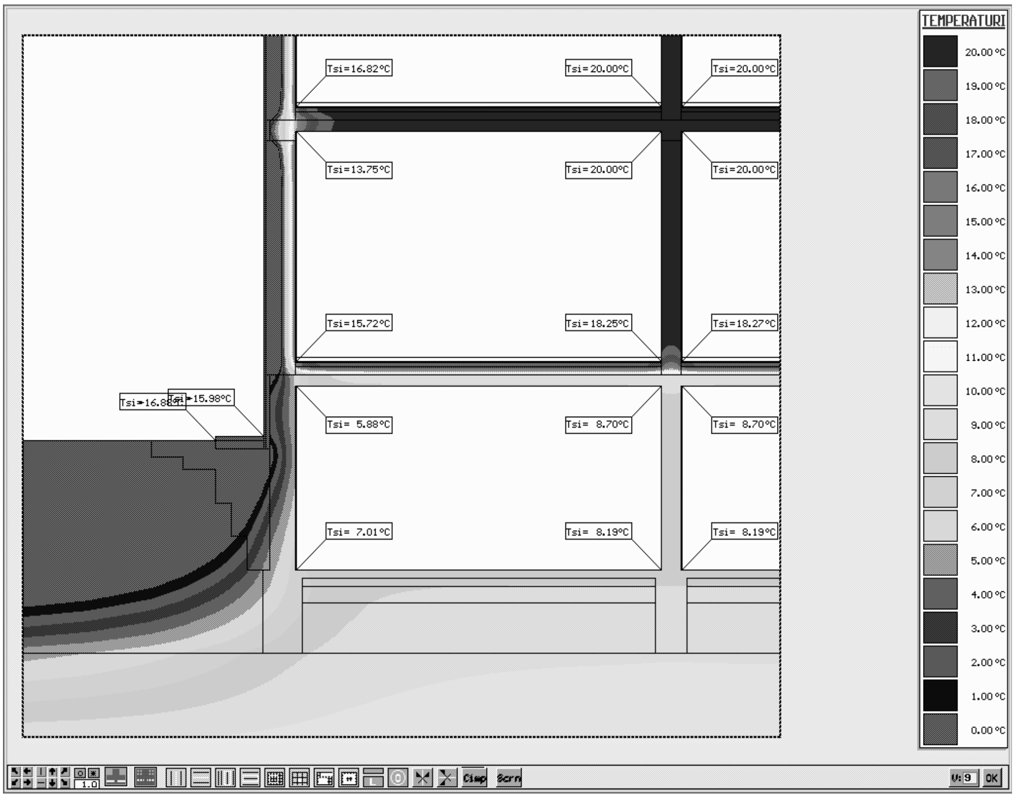

Figure 6 Vertical section through the building and the soil

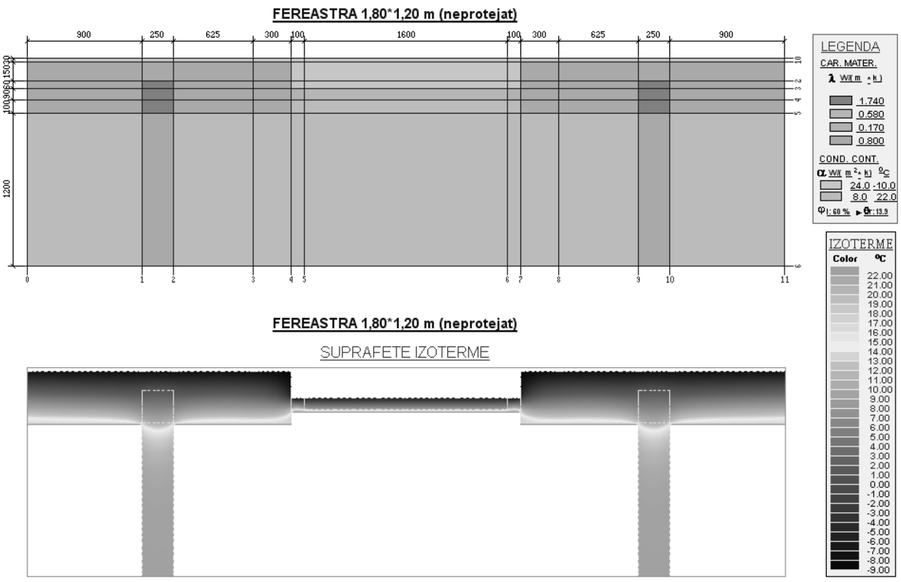

Figure 7 Horizontal section through the window

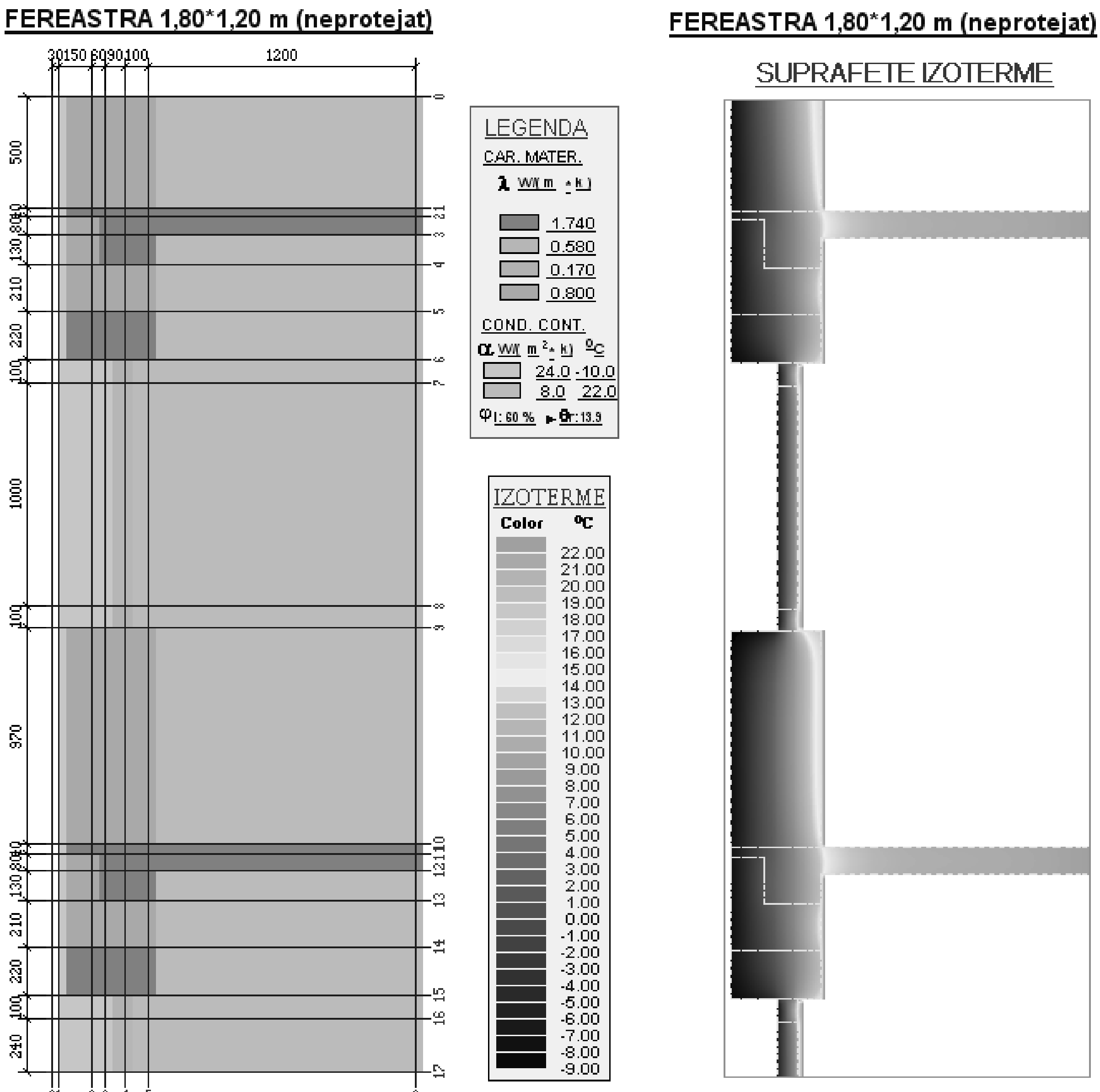

Figure 8 Vertical section through the window

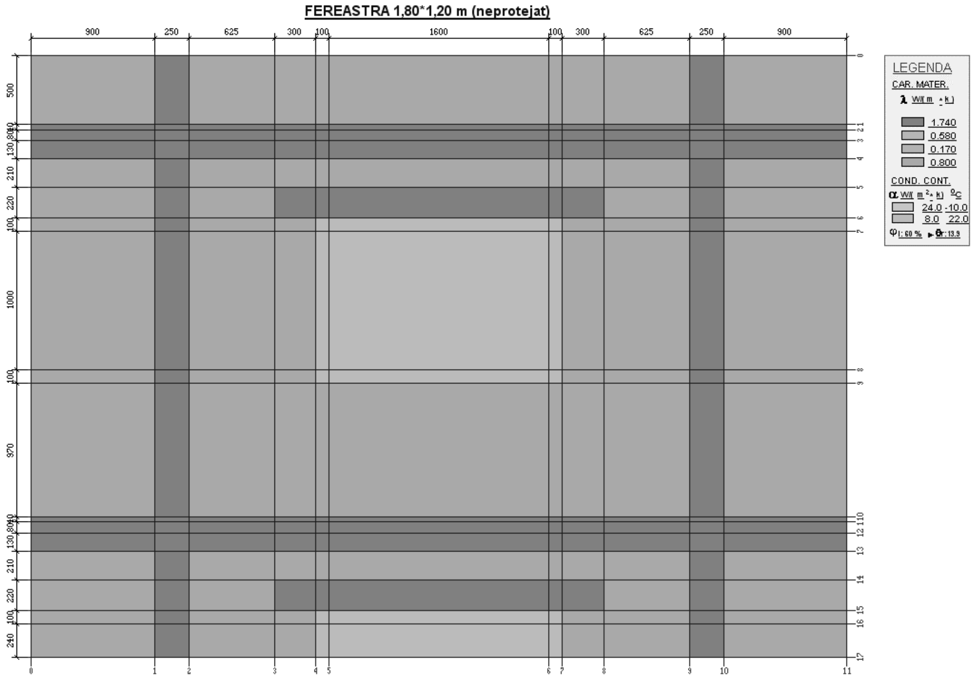

Figure 9 Sight of the interior surface of the wall

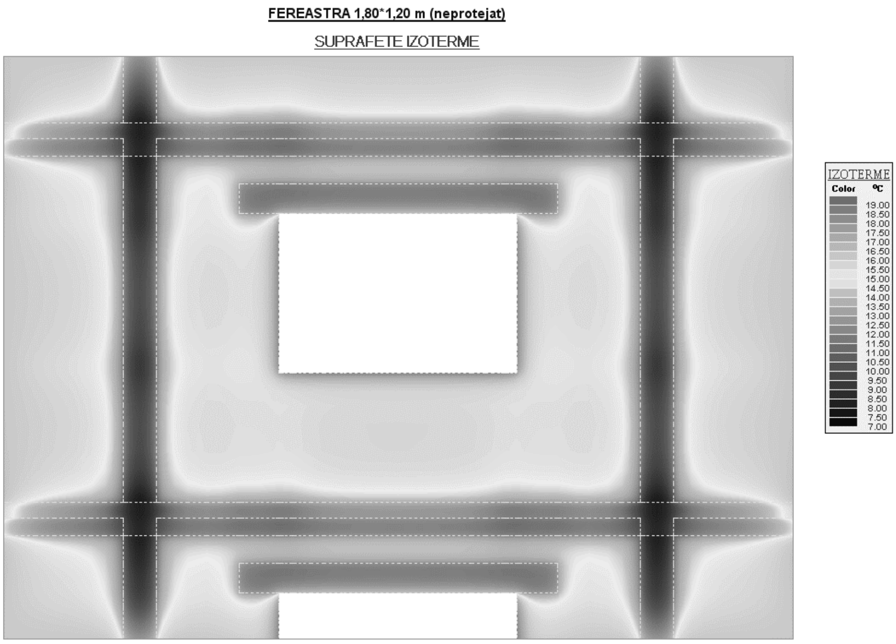

Figure 10 Isotherm surfaces on the interior face of the wall

Current Trends in Computer Aided Design for Land Reclamation

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Rezumat: Lucrarea prezintă dezvoltarea Sistemului Informatic de Proiectare Asistată de Calculator pentru imbunatatiri funciare prin crearea unor programe de calculator care rezolva cu ajutorul unor modele matematice specifice, problemele legate de: proiectarea asistată a canalelor de desecare, proiectarea şi optimizarea nivelării / modelării terenurilor agricole, proiectare asistată a amenajărilor de irigatii cu reţele de conducte, etc. Aplicaţiile software cu utilizare în îmbunătăţiri funciare dispun de facilităţi care permit automatizarea unor segmente importante ale activităţilor înglobate în procesul de proiectare cu ajutorul calculatorului în sistem PAC.

Abstract:

The paper presents the development of the Informational System of Computer-Aided Design for land reclamation domain by creating some computer programs which solve by the help of specific mathematical models, problems related to: the Computer-Aided Design of the drainage canals, the design and the optimization of the levelling/ modeling of agricultural terrain, Computer-Aided Design of the irrigation systems with pipe networks. The software application used in the land reclamation has facilities, which permit the automation of some important segments of the activities integrated in the design process by the help of PAC system.

Keywords: *computer aided design, land reclamation.*

1. Introduction:

Computer Aided Design (CAD) is not a novelty in engineering. In the early 60's, at the Massachusetts Institute of Technology, Ivan Sutherland, who some called "the father of computer graphics", invented CAD for the SAGE project (1963). In the 70's, 2D and 3D graphics progressed, aiding in the field of solid modeling and surface analysis. In tandem with the development of computer hardware (marked in the 80's by the appearance of PCs), CAD software evolved thoroughly in the area of solid modeling, through analysis and modeling of sections, computing geometric features – areas, volumes, and mass properties, inertia momentum. Numeric models, analysis models of stress and pressure and others were built in the frame of computer simulations. Animated graphics were also built and used for visualization and analysis of complex phenomena – structure dynamics, thermodynamic phenomena, biochemistry, and biology.

In the frame of this evolution, CAD entered several engineering fields; mechanical engineering had a major part by means of complex systems – CAD/CAM, CIM – Computer Aided Design/Computer Aided Manufacturing, Computer Integrated Manufacturing.

The development of CAD systems took place along with the engineering domains where they applied.

In Land Reclamation, the creation and development of SIPACIF - Information System for Computer Aided Design in Land Reclamation – at I.S.P.I.F. Bucureşti, in 1984 *(1)* was outstanding for our country. This CAD system, developed for minicomputers (Independent, Coral), aimed to solve, by means of several computer programs, a variety of frequent problems in Land Reclamation design, using adequate mathematic models, such as the design of drainage channels, the optimization of agricultural

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terrain leveling and modeling, the design of pipes networks for irrigation systems.

Further developments in CAD technology lead to a new approach, based on the use of PC's and the achievement and circulation of GIS and LIS (Geographic Information System, Land Information System) software.

Some of these applications and features are examined in this paper.

2. Design Process Characteristics. Land Reclamation Design Particularity

The concept of CAD means the use of computer hardware and software for designing goods *(2)*. The final goal of the design process is
one or several models with graphic representation and technical specifications, used in building the project.

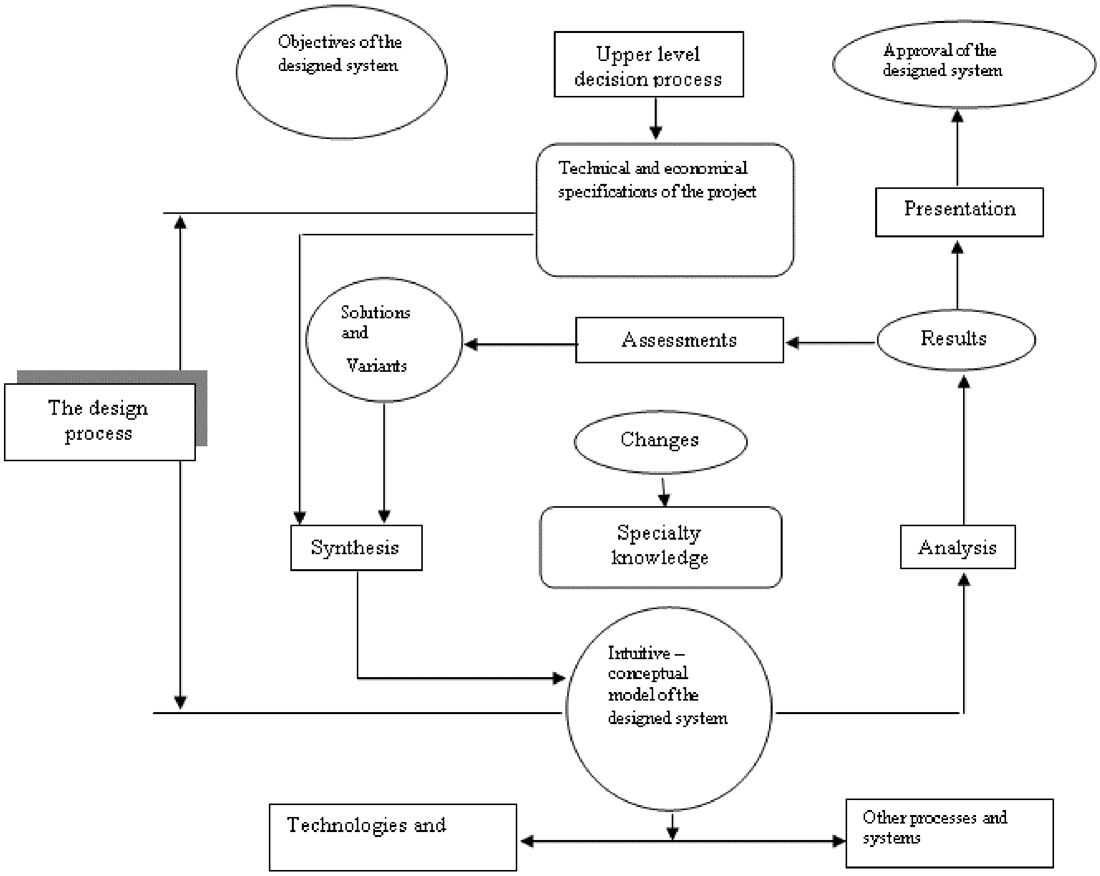

Fig. 1. The flowchart of the design process (after Encarnação, J., Schtechtendahl, E.G.)

According to the definition above, the systems analysis applied to the design process helps identify the activities and (sub-)proceedings, which can be automated with the help of the CAD system. A flow chart of this process is presented in Fig. 1. *(3)*
Figure 1 emphasizes some specific features of the design activity:

- Design is a step-by-step process. There are several phases in the process; they are distinct and may overlap as the implementation of the project advances. A phase requires the end of the previous one; however, depending of the result, relapsing of the previous phase may be done, entirely or in part.

- The feedback feature – In a systemic view, the design activity has, at various levels, the possibility of re-entrance in previous levels, influencing the overall output.

Highlighting these features reveals the major advantage of using a computerized system in the field of design: the computer-generated models resulting from various design hypotheses quickly enrich the design options, leading to an optimized final project.

The design for Land Reclamation shows specific features, in comparison to other engineering designs:

- *The economic and ecologic significance of Land Reclamation works*: These projects involve human intervention regarding the existing natural setting; this leads to the necessity of relating the project with other plans concerning the territory, building projects, agro-ecological zoning, etc.;
- *The complexity of Land Reclamation design*: the accomplishment of a project means getting through a long sequence of technological phases, requiring the assistance of several departments: land reclamation, hydrotechnics, hydraulics, civil design, etc.
- *The importance of field studies*: The Land Reclamation project is directly connected with the site of its achievement. The land surveys, along with the analysis of the local hydrologic, agrochemical, climate, and geologic conditions, accurately accomplished, provide the data and indices that ultimately establish the design development.
- *The prototyping problem*: The Land Reclamation designs are unique. So the design process is somehow a heuristic one. The designer has to use conventional technologies, standards and methods in a special way for each project; in this frame, a major role is play by the creative side of design, by conceiving proper technical solutions and design versions.

3. Current CAD systems usable in Land Reclamation design:

Along with the spread of PCs, the impact of information technology in all fields of activity – including design – grew significantly. The development of specialized software has a similar evolution; nowadays in the software market, a variety of application targets different fields.

Those include complex software packages that may be used in Land Reclamation design. Such products are created by ESRI - ArcInfo and associated software, Bentley - MicroStation and Autodesk, with Autodesk Civil Series. A short description of the former follows.

The **Autodesk Civil Series** (4) suite comprises of three major software packages, each one with its own functionality, but possibly interoperable. The suite allows a complex design team to cooperate through a local area network (LAN), using a centralized common database of the project. The access rights management controls the way in which project data are used, so that undesired data modifications do not happen, and the cooperation and data exchange between team members has maximum efficiency.

The main parts of the suite are:

- **Autodesk Survey** allows the acquisition of field survey data in electronic format – point coordinates and codes associated with point types provided by total stations. This data is automatically plotted and the associated codes allow the establishment of specific data structures on user-defined layers.
- **Autodesk Land Development** (LDD) includes a set of software tools dedicated to the generating, visualization and editing of digital plans and maps. The survey points are included in a COGO database, which has multiple facilities. The software allows the creation of 3D terrain models using the Delaunay triangulation method.

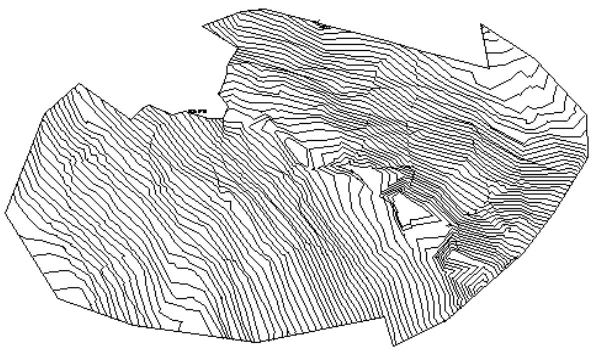

Fig. 2. Automately generated contour lines in LDD

The use of TIN, the contour and point processing (see Fig. 2), achieves functionality for defining and manipulating alignments, creating and managing parcels, calculating volumes by several methods, plot cross sections and generating cut/fill contours. *"The creation of an accurate terrain model … is the foundation for the rest of the design decisions throughout the project"* (5). The software has functionality for defining flow lines along surfaces and calculation of watershed areas and subareas.

- **Autodesk Civil Design** (CD). Together with LDD, CD embeds functionality for the design and drafting of roads and channels, and site development and grading. It allows the calculation of surface-water runoff with a variety of methods and dimensioning and analysis of pipes, weirs, and other structures, and optimizes the cut/fill balance.

4. Conclusions:

Conclusively, CAD is certainly a future tool in Land Reclamation design. Adopting CAD procedures, methods and systems within the design organizations involves a series of problems; some are highlighted in the following.

- The use of a CAD system assumes the acquaintance of the designer with corresponding hardware and software, and the necessity of some knowledge toward the system's "intimacy"(6). Clearing the designer of routine tasks, allowing him to use more time for the creative side of the design process, pays this effort off.
- Adopting the CAD methods means that the decision-makers must make a thorough analysis of its features, advantages, disadvantages and the assessment of the organizational, technical and financial consequences. Information technology is not cheap; its use implies maintenance, development and software/ hardware update costs, besides initial investments. The specialization of personnel using the CAD system is also needed. The evaluation of costs, training of the personnel, the conception of a new data and documents flow inside and outside the organization, have to be considered when the necessity, opportunity and objectives of a new workflow are judged. The assessment has to be done in correlation with the economic efficiency analysis, within a competitive market economy.
- Technical, institutional and legislative issues. The adoption of CAD methods must consider increasing data exchange with other organizations and institutions. This leads to specific concerns regarding creation and use of specific standards in Land Reclamation design, for data structures and data representations. Standardization of procedures and methodologies for hydraulic and structure design is also important in the CAD frame. Taking up such standards at a nation wide scale may represent, as in other countries, the object of certain legislative issues.

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The Estimation Model of Biological Solids Production Rate from Waste Water Treatment Plant

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Rezumat: Cunoaşterea cu precizie a cantităților de nămol produse de o stație de epurare reprezintă un prim pas pentru evaluarea sistemelor existente de tratarea a acestora şi implicit influența lor asupra capacității viitoare a stației respective.
În lucrare se prezintă factorii care influențează producția de nămol în sistemele biologice şi metodele pentru estimarea ritmului de producere a nămolului.
În acelaşi timp se prezintă un model matematic pentru estimarea ratei de creştere a producției de nămol şi definirea în final a ceea ce înseamnă ritm ridicat (de mare încărcare organică), ritm convențional (la o încărcare organică medie) şi ritm scăzut (încărcare organică mică) de creştere a acestei producții al proceselor de epurare biologică.

Abstract: The precise knowledge of the sludge quantities produced by a waste water treatment plant represents a first step in evaluating the existing systems of their treatment and implicitly their influence on the future capacity of the plant.
This work presents the factors which influences the production of sludge in biological systems and the methods to estimate the rate of sludge production.
In the same time there is presented a mathematical model of estimating the rate of sludge production and finally what high rate means refers to system accomplishing partial BOD_5, removal, conventional refers to systems accomplishing reliable BOD_5, removal and some nitrification, and low rate refers to systems reliably accomplishing complete nitrification.

Keywords: *Biological solids production, net observed yield, production rate estimate.*

1. Introduction

Inadequate solids processing systems are often the cause of unacceptable performance and limited treatment capacity at existing WWTPs. Solids processing limitations impact liquid process capacity in two ways: (1) solids processing limitations prevent sludge from being wasted from the liquid process as needed to maintain efficient treatment and (2) poor quality recycle streams from overloaded solids processing systems can, in turn, overload the liquid processing system.

The performance of such facilities can be improved, and their capacity increased, through modifications to their solids processing systems.

As with liquid processing technologies, new solids handling technologies are also installed to improve plant efficiency and reduce treatment costs [1].

Three types of solids processing technologies will be discussed in this chapter, as follows:

- sludge thickening;
- sludge dewatering;
- sludge stabilization.

Accurate prediction of sludge quantities is the necessary first step for evaluating existing solids processing systems and determining whether they limit overall plant treatment capacity. Inaccurate predictions of sludge quantities have resulted in inadequate solids processing systems at more than one WWTP.

2. Factors Affecting Biological System Sludge Production

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Many factors affect the solids production rate from biological wastewater treatment facilities. However, they may generally be classified as follows [2]:

(1) The mass of organic matter applied to the biological system, expressed as the mass of five-day BOD in kg/day;

(2) The mass of TSS applied to the biological system, expressed in kg/day;

(3) The effective solids residence time (SRT) of the biological process, expressed in days;

(4) The characteristics of the influent organic matter (BOD5), TSS, and the biological wastewater treatment system.

The mass of organic matter affects biological solids production directly since biological solids are generated directly from the applied organic matter. The mass of TSS affects biological solids production directly because a portion of the applied TSS are no biodegradable. Included in this are not only the influent fixed (or non-volatile) suspended solids (FSS), but also a portion of the volatile suspended solids (VSS) that are not biodegraded in the biological treatment system. Generally, 30 to 40 percent of the VSS in municipal wastewaters are not biodegraded in biological wastewater treatment systems.

The SRT affects biological solids production through its impact on the overall degradation of influent organic matter. Increased SRT allows increased endogenous respiration, leading to greater degradation of synthesized biological solids. Endogenous respiration results in the conversion of synthesized biological solids to carbon dioxide (CO_2) and water (H_2O). However, synthesized biological solids cannot be fully oxidized; a non-biodegradable residue remains. Thus, biological system solids production cannot be reduced to zero through extended SRT. SRT also impacts the degradation of influent particulate matter. At short SRT (generally less than about 3 days), little degradation of influent particulate organic matter occurs. At SRTs in excess of about 3 days, biodegradable particulate matter in the influent wastewater is degraded, leaving only the no biodegradable residue. These concepts are illustrated in Figure 1.

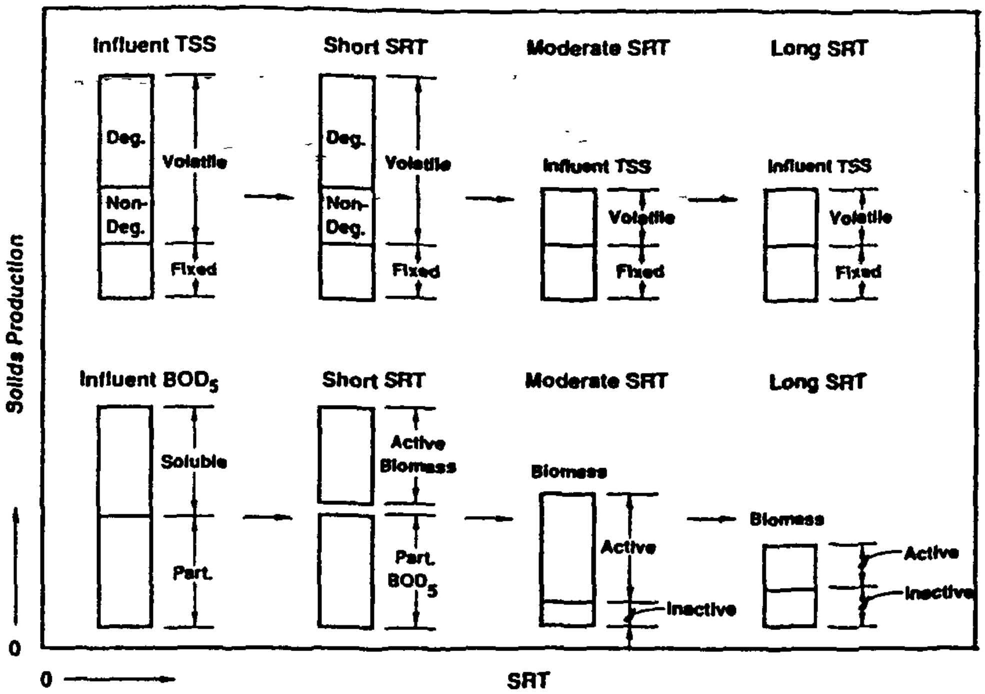

Figure 1 Effect of SRT on biological process solids production

Wastewater and biological system characteristics affect biological solids production directly. The degradability of the influent organic matter affects the conversion of organic matter to biological solids and its subsequent oxidation as a result of increased SRT. Likewise, biological system characteristics affect biological solids production and degradation rates [3].

3 Mathematical Model for Biological Solids Production

This model was selected because it addresses the four factors discussed above and because of the developing consensus in the engineering profession that it is an appropriate model for such uses.

The impact of influent fixed and no biodegradable suspended solids on biological solids production are predicted directly by model. Influent FSS and no biodegradable VSS are assumed to be conserved. Influent FSS and VSS concentrations and masses are calculated directly from the influent TSS using factors. The volatile content of the influent TSS is denoted f_v specification of this value is based either on direct measurements or on typical values that often range from 70 to 80 percent. The no biodegradable fraction of the influent VSS is denoted f_{NV}. Specification of this value is often based on values that have proven effective in practice; typically it ranges from 35 to 40 percent. This parameter can also be used as an adjustment parameter to improve the fit between measured and calculated sludge production rates.

Taken together, the impact of influent TSS on biological solids production rates is predicted as follows [4]:

$$X_I = \text{TSS}\ (1 - f_v + f_v f_{NV}.) \qquad (1)$$

where X_I is the concentration of no degradable TSS in the influent waste-water.

Active biomass concentration (X_B) is predicted using growth yield (Y_H) and decay (b_H) terms for heterotrophic organisms. The equation is:

$$X_B = \text{BOD5}\ Y_H/(1 + \text{SRT}\ b_H) \qquad (2)$$

The concentration of inactive biomass (X_P) is calculated as a fraction (f_I) of the biomass that has undergone endogenous respiration, as follows:

$$X_P = f_I, Y_H \text{BOD}_5 \text{SRT} b_H/(1 + \text{SRT}\ b_H) \qquad (3)$$

The total biological solids production is the sum of X_I, X_B, and X_P. Dividing by the influent BOD_5 gives the unit observed biological solids production rate, or net observed yield (Y_{OBS}), as follows:

$$Y_{OBS} = (X_I + X_B + X_P)/\text{BOD}_5 \qquad (4)$$

This model applies to SRT values sufficiently long so that essentially complete degradation of particulate matter is occurring, generally at SRT values greater than about 3 days. For systems operating at less than this value, a different model must be used. In general, little error occurs if endogenous respiration is neglected at such short SRTs. Consequently, biological solids production can be calculated as the influent solids that are not degraded plus the growth yield times the organic matter degraded. If no degradation of influent TSS is occurring, the relationship is:

$$\text{Biological Solids} = \text{TSS} + Y_H \text{SBOD}_5 \qquad (5)$$

where $SBOD_5$ is the influent soluble BOD_5. The net observed yield is calculated as:

$$Y_{OBS} = \text{TSS/BOD}_5 + Y_H \text{SBOD}_5/\text{BOD}_5 \qquad (6)$$

If partial degradation of the influent particulate matter is occurring, biological solids production can be calculated as:

$$\text{Biological Solids} = \text{TSS}\ [1 - f_v + f_v f_{NV} + (f_v - f_v f_{NV})(1 - f_D)] + Y_H\ [\text{SBOD}_5 + (\text{BOD}_5 - \text{SBOD}_5) f_D] \qquad (7)$$

where f_D is the fraction of influent biodegradable VSS that is degraded in the biological system. Dividing Equation (7) by BOD_5, the net observed yield is:

$$Y_{OBS} = \text{TSS/BOD}_5\ [1 - f_v + f_v f_{NV}\ (f_v - f_v f_{NV})(1 - f_D)] + Y_H[\text{SBOD}_5 + (\text{BOD}_5 - \text{SBOD}_5) f_D]/\text{BOD}_5 \qquad (8)$$

4. The Results and Conclusions

Figure 2 illustrates the impact of SRT and TSS/BOD_5 ratio on the net observed yield as predicted by this model. Values of the model parameters used to develop figure 2 are listed in table 1. To develop this figure, degradation of biodegradable particulate matter was assumed to be complete at an SRT of 3 days, and lesser degrees of degradation were assumed for lower SRTs as listed in table 1.

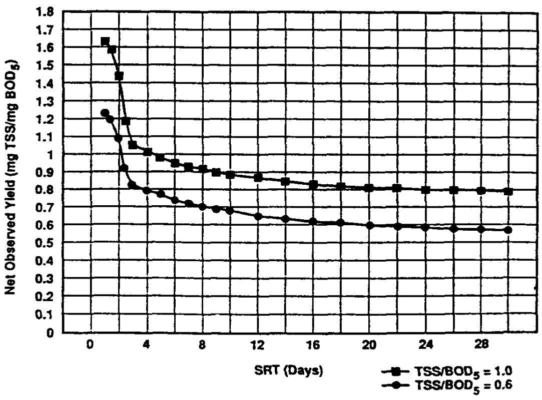

Figure 2 Impact of SRT and TSS/BOD_5 ratio on net observed yield

For both TSS/BOD_5 ratios considered, the net observed yield decreases as the SRT increases. The decrease is initially quite rapid, due first to the increasing biodegradation of influent particulate matter and then to the effects of endogenous respiration. However, it slows with increasing SRT as the proportion of active biomass decreases. The significant impact of the TSS/BOD_5 ratio is also noted.

The two cases were selected to represent a plant with primary clarification (TSS/BOD_5 ratio of 0.6) and a plant without primary clarification (TSS/ BOD_5 ratio of 1) [5].

Table 1 Parameter values used to develop figure 2

Parameter	Value
TSS Characteristics	
f_v, Volatile content	0.75
f_{NV}, No biodegradable fraction of VSS	0.40
Biological Characteristics	
Y_H, mgTSS/mgBOD_5	0.73
b_H, day^{-1}	0.23
f_i, No degradable residue	0.23
For SRT > 3 days, used Equation (6)	
For SRT ≤ 3 days, used Equation (8) with: SRT (days) 1.0, 1.5, 2.0, 2.5 f_D is 0, 0, 0.25, 0.75	

In some instances it is necessary to develop preliminary biological solids production rate estimates prior to more complete characterisation of the wastewater and biological system. In such circumstances, useful preliminary estimates can be developed using experience-based net observed yield values. Table 2 summarises such values. Values are presented for a biological system at WWTPs that do and do not possess primary clarifies. Values are also presented for high-rate, conventional, and low-rate biological treatment systems.

Table 2 Typical Net Observed Sludge Yield Values for Municipal Wastewater.

Biological Process	Net Observed Yield (mg TSS/mg BOD_5)	
	Without Primaries	With Primaries
High rate	1.0-1.6	0.8-1.3
Conventional rate	0.8-1.1	0.7-0.9
Low rate	0.7-0.9	0.5-0.7

For suspended growth biological wastewater treatment systems, these terms generally correspond to SRT values of 1 to 3 days, 4 to 8 days, and greater than 15 days, respectively. With experience, this terminology can also be applied to fixed film and other biological treatment processes.

The term high-rate generally refers to systems accomplishing partial BOD_5 removal, conventional refers to systems accomplishing reliable BOD_5 removal and some nitrification, while the term low-rate refers to systems reliably accomplishing nearly complete nitrification.

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The Hydraulic Calculation of the Urban Sewerage Network at Rain Waters

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Rezumat: In lucrare se prezinta un nou concept privind calculul hidraulic al retelelor de canalizare orasenesti la ape pluviale, in ipoteza functionarii acestora sub presiune, cu un program de calcul similar celui pentru retelele de apa, cu conditii specifice de margine si in nodurile retelei.
Noua metoda de calcul hidraulic al unei retele de canalizare scoate in evidenta faptul ca solutia de suplimentare a retelei existente, sau supradimensionarea unei retele noi (dimensionata conform metodologiei actuale), are un efect neglijabil asupra timpului de inundare, la ploi torentiale, a orasului.
In schimb, solutia cu bazine de retentie pe reteaua de canalizare, care sa preia temporar volume importante de apa pluviala din retea, la ploi torentiale, apare ca o solutie inginereasca cu eficienta foarte mare privind reducerea timpului de inundare a zonelor orasenesti canalizate.

Abstract: In the work there is presented a new concept concerning the hydraulic calculation of urban sewerage network at rain waters, in the hypothesis of its functioning under pressure, based on a calculation programmer similar to that one for water supply network, with specific conditions of margin and in the network nods.
The new hydraulic calculation method of a sewerage network reveals the fact that the existing network supplementing solution or over dimensioning a new sewerage network (dimensioned according to the present methodology), has a neglecting effect on the flood time of the town, at torrential rains.
On the other hand, solution concerning retaining basins on the sewerage network, to take up temporarily important volumes of rainwater from the network, at torrential rains, seems to be an engineering solution with high efficiency referring to reducing flood time of urban zones provided with sewerage network.

Keywords: *urban sewerage network, hydraulic calculation at rain waters.*

1.Introduction

The problems of the urban sewerage network hydraulic calculation at rain waters has a special place in the specialty, and in our country is regulated by the standards in force [4], [6].

According to the present practice in domain, the problem of dimensioning sewerage network at rain waters present 2 phases:

- Phase I – Determining calculation flows in different section of sewers and main collectors for rains of certain insurances, imposed by the standards in force;
- Phase II – Hydraulic dimensioning the sewers (main collectors) at full section (complete flow section) by simple hydraulic calculation methods, in witch there is admitted the hypothesis that hydraulic slope is equal to geodetically slope of the respective sewer. At torrential rains with higher insurances than those of calculation ones it is accepted putting under pressure of the sewerage network until the level of the ground and therefore flooding streets and arranged urban surface, for longer or shorter periods of time.

Regarding the phase I, referring to determining calculation flows, it is mentioned that present norms admit as calculation rain a conventional rain of standardized frequency, whose duration t_p is equal to the time t_s necessary for the water to concentrate on the fall places until the sewer and run the respective sewer until the calculation section.

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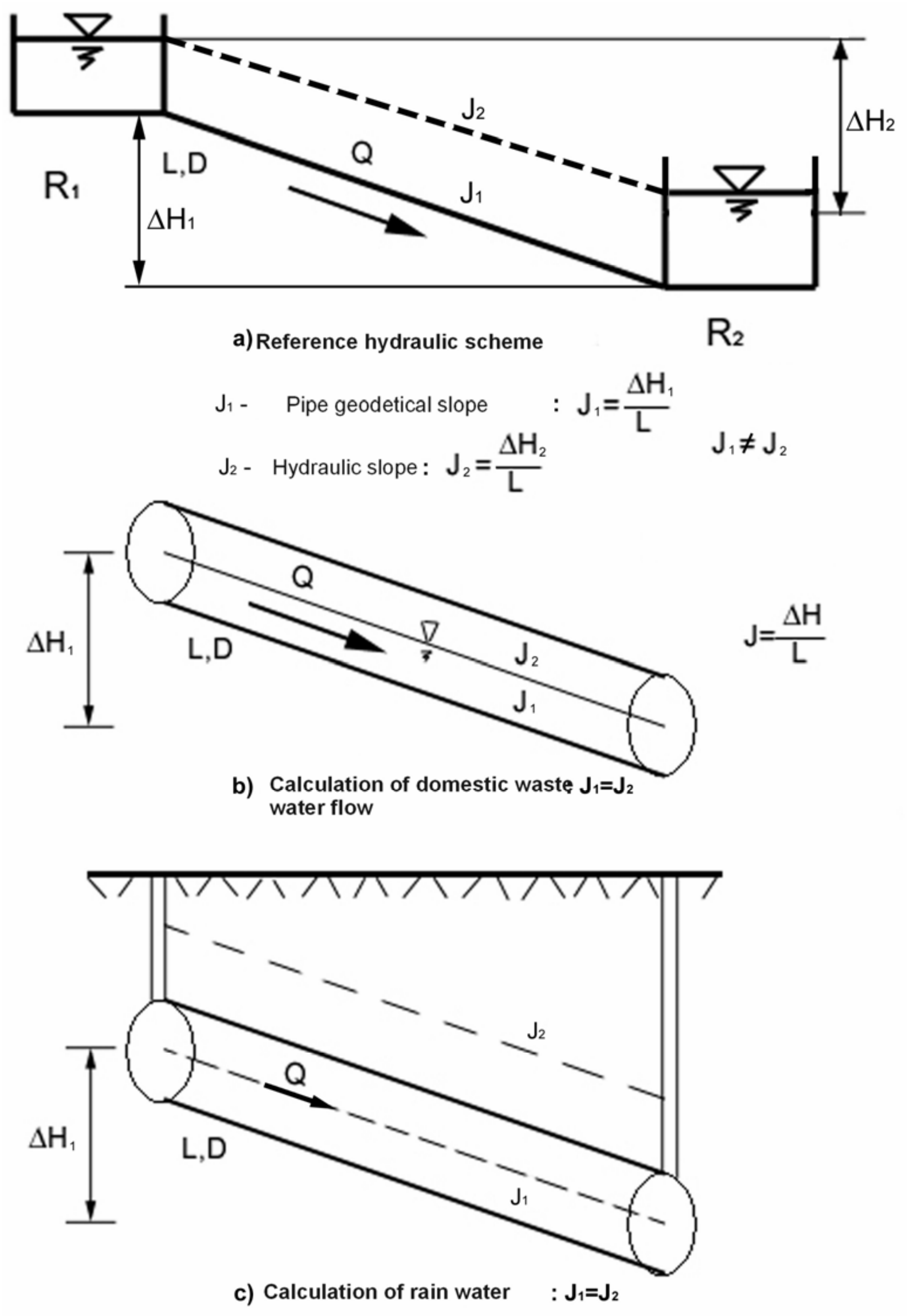

Fig.1 – Present hydraulic scheme for sewerage network calculation

As for phase II, admitting hypothesis of sewer calculation at full section (complete flow section), and therefore functioning under pressure, it is an ideal calculation hypothesis which in sewerage network functioning on very short period of time, maybe for some minutes (figure 1).

Consequently, present methods of hydraulic calculation of sewerage network at rain waters are acceptable for equal or small rains in comparison with the calculation rain.

When the effective rain exceed by duration or intensity the calculation rain, the streets and arranged urban areas are flooded and the engineering explanation encountered in such ituation, is that the sewerage network is subdimensioned.

In such situations the decision factors accept generally new investments concerning the extension of the sewerage network with other sewers that supplement the existing network partially or totally, in certain important urban areas.

1. Calculation method

The present work introduces a new concept in dimensioning and checking hydraulic functioning at rainwater's of the sewerage network, which changescompletely both the hydraulic calculation method and the vision on solution promoted in designing practice of this field.

In principle, the calculation methods proposed in communication, starts from idea that the phase I of calculation can remain unchangeable, but in the phase II it is admitted that the hydraulic calculation hypothesis, using effective hydraulic slopes (figure 2) in calculation of sewer between the two sections (nods), and the sewerage network will be calculated by methods of specific network calculations in general (figure 3, 4).

In this situation the water circulation in the network, for example between the two sections (nods) will be made after a hydraulic scheme of the type presented in the figure 2, [1].

For the sewerage network hydraulic calculation it will be adopted a scheme of the presented in the figure 4, similar to a water network under pressure (figure 3), in which the lows in network nods are replaced with fictional water basins of known volumes, corresponding with the collecting zone of the respective nod and reference rain type.

As one can remark in figure 3 and 4, the network margin conditions change so that the pumping station is replaced by discharging into the river.

The network calculation programmer permits calculating the evolution in time, in all network nods, both of flows and of water levels in the fictional water basins namely related to the street level [2], [5].

From a simple analysis of the new implemented calculation concept, two important sewerage network parameters are identified:

a) sewerage network total volume, written down Vol_t;

b) calculation rain total volume in the receiving basin, which arrives at the sewerage network, written down Vol_p.

Referring to the torrential rains of higher insurances than calculation rain, the first estimative calculation indicates that the ratio:

$$\alpha = \frac{Vol_t}{Vol_p}$$

has a very small value, or order 0,01 – 0,05.

Therefore the classical solution leading to the over dimensioning sewerage network has a neglecting effect on the flood time of the urban zones provided with sewerage network [3].

2.Conclusion

The new concept concerning the hydraulic calculation of urban sewerage network at rain waters imposes a new methodology of its hydraulic calculation and consequently the corresponding adaptation to the resent standard in domain.

The solution of supplementing an urban sewerage network by removing partially or totally of the town flood at rain water appears as a solution with a neglecting effect.

The solution with retaining basins to take up temporarily important volumes of water from the sewerage network at rain water appears as an engineering solution with very high efficiency, concerning, reducing flood time of urban zones provided with sewerage network.

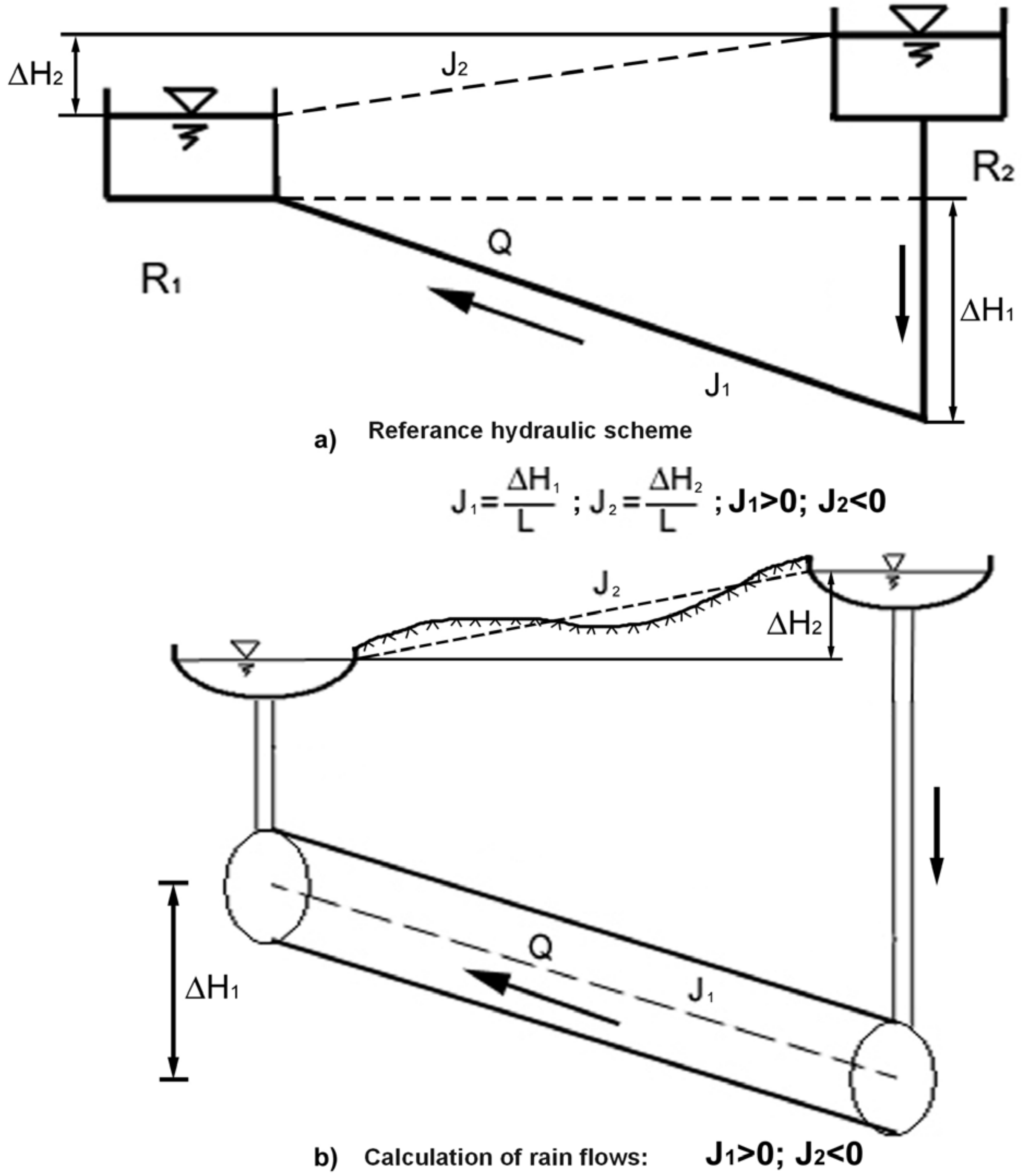

Fig. 2 – Hydraulic scheme proposed for sewerage network

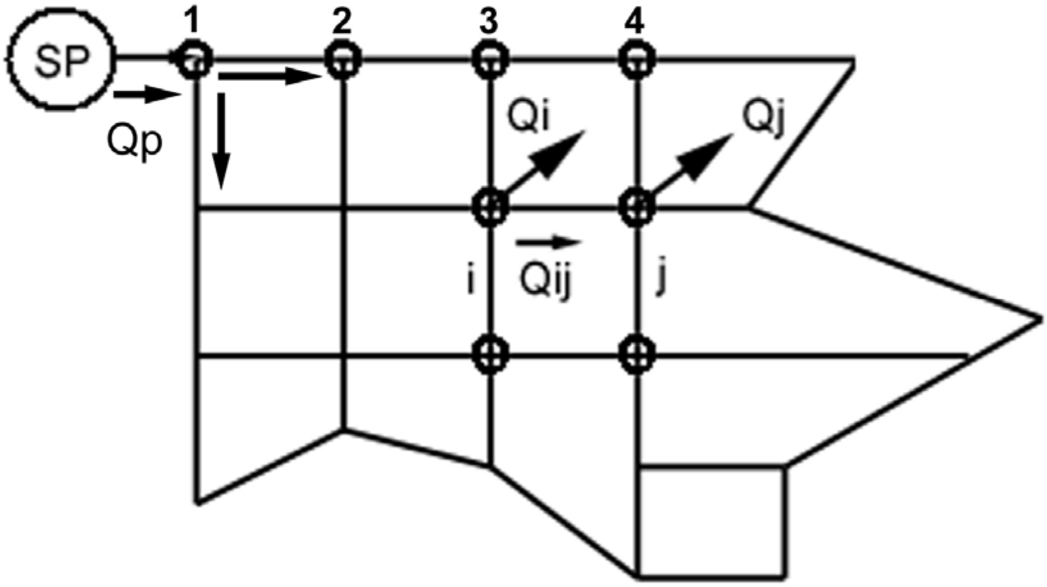

Fig. 3 – Scheme of an urban water network

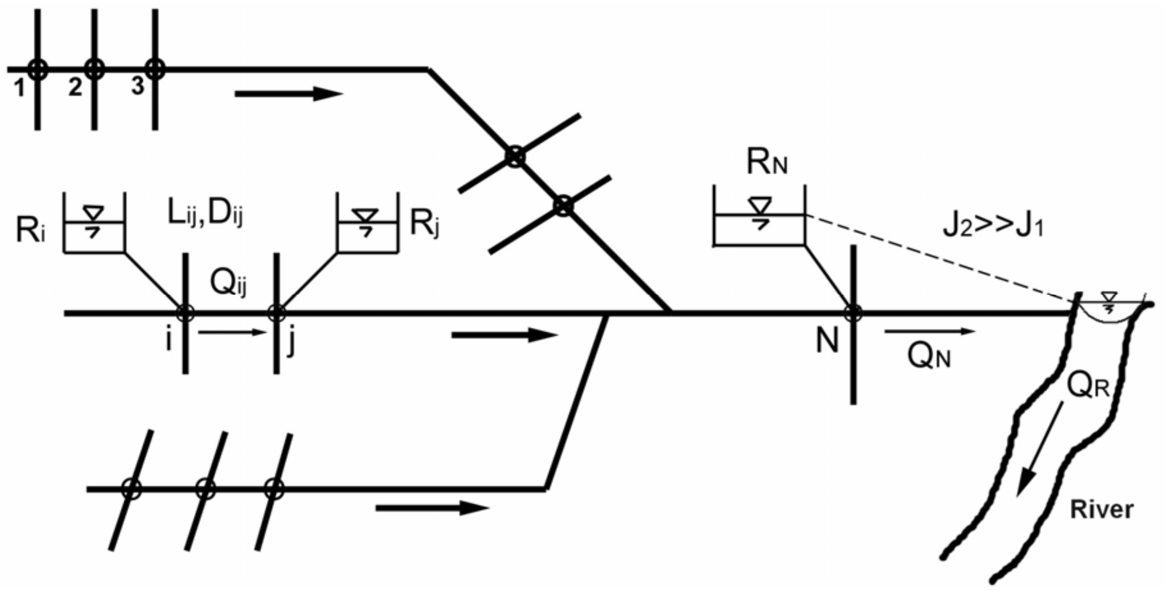

Fig. 4 – Scheme of an urban sewerage network

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Integrated Concepts for Reuse of Upgrated Wastewater

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Rezumat: Astazi, efluentii pot fi generati doar ca impact ecologic al apelor primite. Luand in considerare mijloacele de epurare a apelor uzate, pentru a fi deversate, pare o risipa nu numai din punct de vedere economic, dar si din punct de vedere al investitiei, pentru a fi reintrodus in ciclul natural, reutilizandu-se ulterior in alte scopuri. Dezvoltarea acestei noi resurse de apa este considerata dependenta de factori importanti cum ar fi deficientele pe termen lung in balanta apei sau frecventa si gravitatea secetei. Aceste probleme sunt relevante in Europa Sudica unde irigarea din agricultura conduce la un “stress” al apei suplimentar, la momentul actual reprezentand un factor care trebuie sa conduca la dezvoltarea reutilizarii apelor uzate.
In plus, implementarea acestui concept este dependenta de atitudinea generala fata de reutilizarea apelor uzate si detaliile viabilitatii economice a acesteia. Totusi, cele mai multe tari sunt abia la inceputul stabilirii posibilitatilor de planificare a reutilizarii apelor uzate, fiind putin cunoscut volumul potential de apa uzata care poate fi recuperat.

Abstract: Today, effluents can be generated that barely impact ecology of receiving waters. Considering the means spent to treat wastewater for discharge it almost seems – not only from the economic point of view – a waste of investment to release it into the natural water cycle from which it has to be extracted again for the other purposes. The extent to which this new water resource is tapped depends on “hard” facts like possible long – term deficiencies in the water balance or frequency and severity of droughts. These problems are more relevant in Southern Europe where agricultural irrigation causes additional water stress and actually this is where additional focus should be given to the development of wastewater reuse.
Furthermore, the implementation of wastewater reuse is highly dependent of the general attitude towards wastewater reuse and the details of its economic viability. But as most countries are just beginning to assess the possibilities of planned wastewater reuse, little is known about the potential volumes of wastewater, which could be reclaimed.

Keywords: *water cycle, water balance, wastewater reuse.*

1. Introduction:

Water is an essential and basic human need for urban, industrial and agricultural use. Unfortunately the worldwide concern for the depletion of global water sources is rising day by day. With the ever-increasing population and growing economy, demands for water are naturally continuously growing. Some of the countries of the Mediterranean area and Middle East area are experiencing a continuous growth in population, in standards of life and in tourist infrastructure and industrial development. These parameters have tended to increase water consumption and the stress on water supplies. Renewable water resources per capita have seen a reduction of up to about 80% in the last 10-15 years. A unique and viable opportunity to augment traditional water supplies provides water reclamation and reuse. Promising is the fact that since many years it is feasible to treat wastewater to a high quality. Hence, wastewater could be regarded as a resource that could be put to beneficial use rather than wasted; so in many parts of the world reclaimed water is used as a water resource and water reuse is called the greatest challenge of the 21st century.

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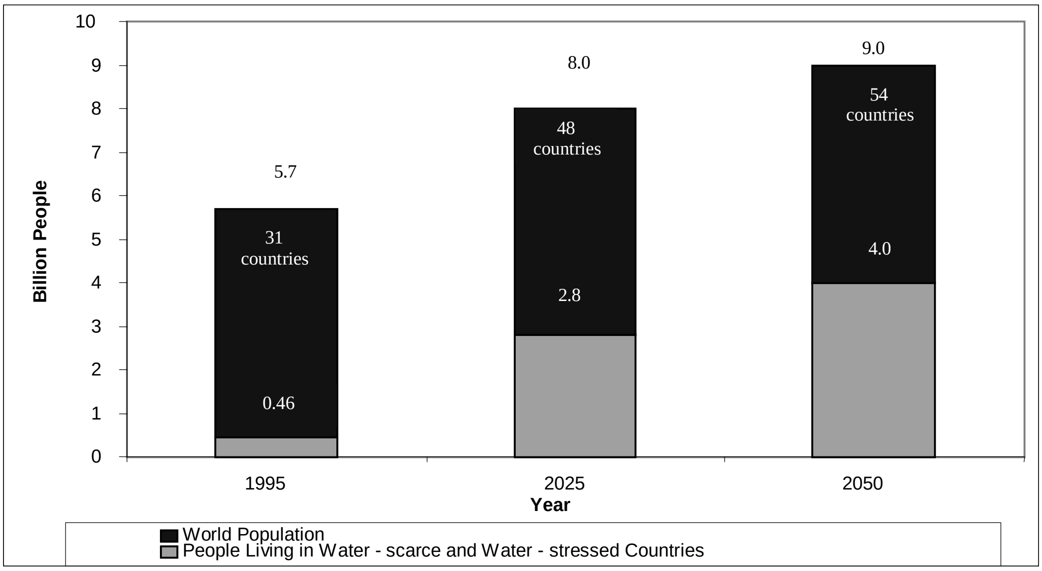

Figure 1 Population in water – scrce and water – stressed countries [1]

2. Water reuse: objectives

Water reuse accomplishes usually two fundamental functions: the treated effluent is used as a water resource for beneficial purpose and the effluent is kept out of streams, lakes, and beaches: thus reducing pollution of surface water and groundwater [2]. The wastewater should be treated using very effective wastewater treatment technologies that guarantee protecting public health and the environment. In many water-scarce regions the situation is worse because according to the ancient tradition, untreated wastewater has been reused without any treatment. This situation has to be changed; unplanned water reuse has to be replaced by integrated water resources management. This management provides a strong argument against this traditional practice and offers systematic progress towards safe and reliable sanitation practice. According to this the base of water reuse is built upon three principles [3]:

- providing reliable treatment of wastewater to meet strict water quality requirements for the intended reuse application;
- protecting public health,
- gaining public acceptance.

Of course, according to the intended water reuse applications govern the degree of wastewater treatment required and the reliability of wastewater treatment processing and operation. The most frequently application for the use of reclaimed water includes: agricultural irrigation, landscape irrigation, industrial recycling and reuse and groundwater recharge.

The principal objective of the reuse of wastewater is not only to provide additional quantities of water of good quality by accelerating the cycle of natural purification of water, but also to ensure the balance of this cycle and the protection of the surrounding medium. By definition, this reuse is a voluntary and planned action which aims at the production of the complementary quantities out of water for various uses in order to make up hydrous deficits. The future water resources management strategy must include at least:

- Efficient irrigation;
- Small scale solutions;
- Reuse o f municipal effluents for irrigation;
- Effluents recycling for industrial reuse;
- Effluents recycling for non – potable municipal reuse;
- Water conservation (demand manager);
- Application of adequate tariff policy;
- Application of adequate regulation;

3. Categories and examples of reuse water

During the planning and implementation of water reclamation and reuse, the reclaimed water application will usually govern the type of wastewater treatment needed to protect public health and the environment, and the degree of reliability required for each sequence of treatment process and operations. Water reuse applications, from a global perspective, have been development to replace or increase water resources for specific applications depending of course on local water use standards. Generally depending on water origin and treatment process; water reuse application can be divided in seven categories:

- **Agricultural irrigation** represents the largest current use of reclaimed water throughout the world. This reuse category offers significant future opportunities for water reuse in both industrialised countries and developing countries.
- **Landscape irrigation** is the second largest user of reclaimed water in industrialised countries and it includes the irrigation of parks; playgrounds; golf courses; freeway medians; landscaped areas around commercial, office, and industrial developments; and landscaped areas around residences. Many landscape irrigation projects involve dual distribution systems, which consist of one distribution network for potable water and a separate pipeline to transport reclaimed water.
- **Industrial activities** represent the third major use of reclaimed water, primarily for cooling and process needs. Cooling water creates the single largest industrial demand for water and as such is the predominant industrial water reuse either for cooling towers or cooling ponds. Industrial uses vary greatly and water quality requirements tend to be industry-specific.
- **Groundwater recharge** is the fourth largest application for water reuse, either via spreading basins or direct injection to groundwater aquifers. Groundwater recharge includes groundwater replenishment by assimilation and storage of reclaimed water in groundwater aquifers, or establishing hydraulic barriers against salt-water intrusion in coastal areas.
- **Recreational and environmental uses** constitute the fifth largest use of reclaimed water in industrialised countries and involve non-potable uses related to land-based water features such as the development of recreational lakes, marsh enhancement, and stream flow augmentation. Reclaimed water has been applied to wetlands for a variety of reasons including: habitat creation, restoration and/or enhancement, provision for additional treatment prior to discharge to receiving water, and provision for a wet weather disposal alternative for reclaimed water.
- **Non-potable urban uses** include fire protection, air conditioning, toilet flushing, construction water, and flushing of sanitary sewers. Typically, for economic reasons, these uses are incidental and depend on the proximity of the wastewater reclamation plant to the point of use. In addition, the economic advantages of urban uses can be enhanced by coupling with other ongoing reuse applications such as landscape irrigation.
- **Potable reuse** is another water reuse opportunity, which could occur either by blending in water supply storage reservoirs (indirect reuse) or, in the extreme, by direct input of highly treated wastewater into the water distribution system [3].

Water from recycling systems used in each one of the seven categories should fulfil four criteria: hygienic safety, aesthetics, environmental tolerance and technical and economical feasibility. This has occurred as a result of several factors (WHO, 2002):

- the increasing scarcity of alternative waters for irrigation, exacerbated by increasing urban demand for potable water supplies, and the growing recognition by water resource planners of the importance and value of wastewater reuse;
- the high cost of artificial fertilizers and the recognition of the value of nutrients in wastewater, which significantly increase crop yield;
- the demonstration that health risks and soil damage are minimal if the necessary precautions are taken;
- the high cost of advanced wastewater treatment plants;
- the sociocultural acceptance of the practice.

The majority of the projects of reuse of wastewater relate to **agricultural uses**. For this sector, the reuse of water improves the outputs of the cultures and brings financial benefit. In order to guarantee the protection of the public health, it is essential to set up standards and strict regulations, adapted to the specificity of the various cultures. There are two great groups of standards: recommendations of WHO (1989) and Californian regulation "Title 22" (1978). The principal objective is to eliminate the medical risks. Thus, for the irrigation without restriction, the microbiological pollution of wastewater must, according to WHO, to remain below 1,000 Faecal Coliforms (FC)/100 ml and less than 1 egg of helminthes/l. The Californian "Title 22" fixes more severe restrictions, even the total absence of germ-tests: less than 2.2 Total Coliforms total (TC)/100 ml. In certain countries, the standards are draconian for the plants intended for consumption. In the countries where the existing standards are very severe (Australia, the United States, certain countries of the Middle East), a secondary treatment is obligatory, and sometimes, in addition, a tertiary treatment.

The experiment of **Mexico City**, for exemple, seems the most significant project of reuse of the water on a world level (Jiménez-Cisneros, 1997). Almost 100% of non-treated wastewater of the Mexican capital (from 45 to 300 m^3/s in rainy weather) are reused for the irrigation of more than 85,000 ha of various agricultural cultures.

The second major use of reclaimed municipal wastewater is in **industrial activities**, primarily for cooling and process needs. Industrial uses vary greatly, and to provide adequate water quality, additional treatment is often required beyond conventional wastewater treatment. The industrial reuse of wastewater and internal recycling are from now on a technical and economic reality. For certain countries and types of industries, recycled water provides 85% of the total needs out of water. The sectors the largest consumers out of water are the power stations and nuclear (cooling water) and the paper mills. The quality of reused water is regulated and depends on the type of application or industrial production. The share of urban wastewater does not exceed 15% of the volume of the water reused in industry. In the **United States**, for example, the volume of wastewater reused in industry is approximately 790,000 m^3/day, including 68% for cooling (Lazarova, 1998).

4. Water Reuse Treatment. Water Quality. Environmental aspects and benefits of water recycling:

4.1 Water quality of the reclaimed water:

Wastewater reclamation and reuse have become significant elements in water resources planning and management, particularly in arid and semiarid regions. Proper and integrated planning of water reuse may provide sufficient flexibility to respond to short-term needs as well as to increase the long-term reliability of water supply. Moreover, water quality criteria, economic analyses and project management, in the context of water resources, are essential components in the implementation of any water reuse project. The essential foundation for successful implementation of such a project is the capability of producing water of a desired quality to provide adequate public health protection and meet the environmental and socio-economic goals than can be practically achieved at given time. (Abel-Jawad, 1999). The wastewater treatment and reclamation technology employed is always the key element of the water recycling scheme and a general overview on the treatment options has to be given as presupposition to discuss planning processes. Different methods can be employed to renovate effluents for their use for agricultural, industrial and domestic applications. Although potable quality of the reclaimed water can be obtained, direct potable application of the reclaimed water is not expected to highly increase mainly due to public acceptance.

Upgraded wastewater use in several application categories shall specify the following conditions:

- Water reuse shall be safe for its intended use and shall not jeopardise the safety of the product through the introduction of chemical, microbiological or physical contaminants in amounts that represent a health risk to the consumer;
- Water reuse should not adversely affect the product quality (flavour, colour, texture);
- Water reuse intended for incorporation into a food product should at least meet the microbiological and, as deemed necessary, chemical specification for potable water. In certain cases physical specifications may be appropriate;
- Reuse water shall be subjected to on going monitoring and testing to ensure its safety and quality.

- The water treatment system(s) chosen should be such that they will provide the level of reconditioning appropriate for the intended water reuse;
- Proper maintenance of water reconditioning systems is critical;
- Treatment of water must be undertaken with knowledge of the types of contaminants the water may have acquired from its previous use;
- Container cooling water should be sanitised (e.g., chlorine) because there is always the possibility that leakage could contaminate product.

4.2. Water reuse treatments:

The choice of the right wastewater treatment technologies is the most important step in planning a water reuse system because they are the important mean of decreasing or removing the environmental risk. The environmental risk is connected with the contamination that can be found in the upgraded wastewater. In general, the risk can be divided into a chemical and a microbiological one. The fundamental purpose of water treatment is to protect the consumer from pathogens and from impurities in the water that may be injurious to human health or offensive. Where appropriate, treatment should also remove impurities which, although not harmful to human health, may make the water unappealing, damage pipes, plant or other items with which the water may come into contact, or render operation more difficult or costly [4]. These purposes are achieved, by introducing successive barriers, such as coagulation, sedimentation, filtration and advanced treatments, to remove pathogens and impurities. The final barrier is often disinfection [4].

Table 1 provides an overview of water treatment technologies and their applications. Moreover, in the *Figure 2* main treatment trains for agricultural wastewater reuse are shown and in the *Table 2* recommended treatment schemes as a function of wastewater reuse applications are summarised.

Extensive treatments are cheaper ones than intensive ones but the final quality of the treated water is usually lower. It is for those reason that these types of treatments are more used in non-developed countries.

Anyway, for extensive treatments as for example lagoons and wetlands climate characteristics as for example type of winds and temperatures are very important issues.

Combining the various processes and operations it is now possible to produce high quality water from municipal wastewater for any reuse application. However, the feasibility of such wastewater reuse program will depend mainly on public acceptance and cost. To protect public health, in USA considerable efforts have been made to establish conditions and regulations that would allow safe use of reclaimed wastewater for irrigation. Reclaimed wastewater regulations for specific irrigation uses are based on the expected degree of human contact with the reclaimed and intended use of the irrigated crops. Additional safety measures that have been adopted for non-potable water reuse applications include:

- installation of separate storage and distribution systems for potable water;
- use of colour-coded tapes to distinguish potable and non-potable distribution piping;
- cross-connection and back-flow prevention devices;
- periodic use of tracer dyes to detect the occurrence of cross contamination in potable supply lines;
- irrigation during off-hours to further minimise the potential for human contact.

4.3. Environmental aspects and benefits of water recycling:

The environmental aspects are a very important part in a reuse project. With a water reuse project several benefits can be achieved.

For example, improvement of an specific area (for example an old rubble dump), increase of the ecological flow (improvement of marshes), increase of the animal and vegetal species or population, increase of the cultivated area and so on. Moreover these aspects, in a feasibility study another subjects as for example noises, odour problems, visual impacts, changes in soils composition, disposal of inadequately treated water and accidental spills of pollutants have to be considered.

Table 1. Overview of representative unit processes and operations used in water reclamation (Asano, 1998).

Process	**Description**	**Application**
Solid/liquid separation		
Sedimentation	Gravity sedimentation of particulate matter, chemical flocks, and precipitates from suspension by gravity settling.	Removal of particles from turbid water > 30 µm.
Filtration	Particle removal by passing water through sand or other porous medium.	Removal of particles from water those are larger than about 3 µm. Frequently used after sedimentation or coagulation/flocculation
Biological Treatment		
Aerobic biological Treatment	Biological metabolism by microorganisms in an aeration basin or biofilm process	Removal of dissolved and suspended organic matter from wastewater.
Oxidation pond	Ponds up to one metre in depth for mixing and sunlight penetration.	Reduction of suspended solids, BOD, pathogenic bacteria, and ammonia from wastewater.
Biological nutrient removal	Combination of aerobic, anoxic, and anaerobic processes to optimise conversion of organic and ammonia nitrogen to molecular nitrogen (N2) and removal of phosphorus.	Reduction of nutrient content of reclaimed water.
Waste stabilisation ponds	Pond system consisting of anaerobic, facultative and maturation ponds linked in series to increase retention time.	Reduction of suspended solids, BOD, pathogenic bacteria, and ammonia from wastewater. Facilitates water reuse for irrigation and aquaculture.
Disinfection	The inactivation of pathogenic organisms using oxidizing chemicals, ultraviolet light, caustic chemicals, heat, or physical separation processes (e.g. membranes).	Protection of public health by removal of pathogenic organisms.
Advanced treatment		
Activated Carbon	Process by which contaminants are physically adsorbed onto the surface of activated carbon.	Removal of hydrophobic organic compounds
Air stripping	Transfer of ammonia and other volatile components from water to air.	Removal of ammonia and some volatile organics from water
Ion exchange	Exchange of ions between an exchange resin and water using a flow through reactor.	**Effective for removal of cations such as calcium, magnesium, iron, ammonium, and anions such as nitrate**
Chemical coagulation and precipitation	Use of aluminium or iron salts, polyelectrolytes, and/or ozone to promote destabilization of colloidal particles from reclaimed water and precipitation of phosphorus.	Formation of phosphorus precipitates and flocculation of particles for removal by sedimentation and filtration.
Lime treatment	The use of lime to precipitate cations and metals from solution.	Used to reduce scale-forming potential of water, precipitate phosphorus, and modify pH.
Membrane filtration	Micro-, nano-, and ultrafiltration	Removal of particles and microorganisms from water.
Reverse osmosis	Membrane system to separate ions from solution based on reversing osmotic pressure differentials.	Removal of dissolved salts and minerals from solution; also effective for.

Table 2. Recommended treatment schemes as a function of wastewater reuse applications.

Type of reuse application	Extensive treatment	Intensive treatment
1. Irrigation of restricted crops	E.1. Stabilisation ponds in series or aerated lagoons; wetlands; infiltration -percolation	I.1. Secondary treatment by activated sludge or trickling filters with pr without disinfection
2. Irrigation of unrestricted crops, vegetables eaten raw	E.2. Idem as E.1. with polishing steps and storage reservoirs	I.2. Idem as I.1. with tertiary filtration and disinfection
3. Urban uses for irrigation of parks, sport fields, golf courses	E.3. Idem as E.2.	I.3. Idem as I.2. filtration in the case of unrestricted public access
4. Groundwater recharge for agricultural irrigation	E.4. Idem as E.2. completed by soil-aquifer treatment	I.4. Idem as I.2. with nutrient removal (when necessary)
5. Dual distribution for toilet flushing	E.5. Not applicable	I.5. Idem as I.3. with activated carbon (when necessary) or membrane bioreactors and disinfection
6. Indirect and direct potable use	E.6. Not applicable	I.6. Secondary, tertiary and quaternary treatment, including activated carbon, membrane filtration (included reverse osmosis) and advanced disinfection

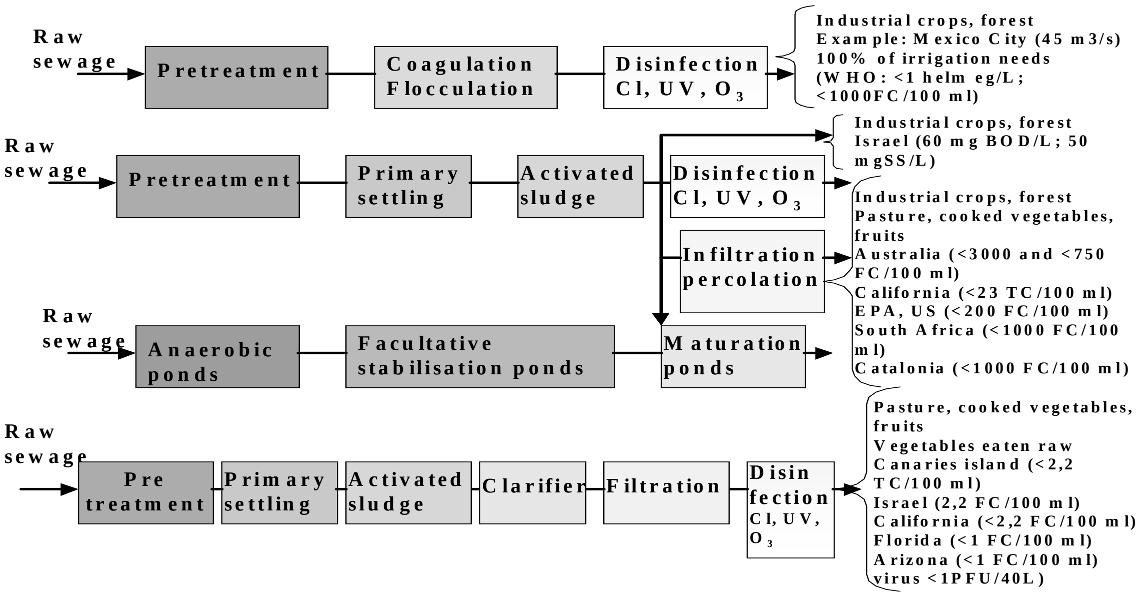

Figure 2. Main treatment trains for agricultural wastewater

In the case that reclaimed water is reused for crop irrigation some of the most important parameters to consider are Na, Ca, B and salinity as these compounds can decrease the agricultural productivity. On the other hand, nutrients concentration will be compounds that they must be very well controlled and monitorised as they can produce water eutrophization.

In this sense, the temperature, the turbidity (or the total suspended solids) and the dissolved oxygen concentration are another parameters to be studied.

Water recycling can decrease diversion of freshwater from sensitive ecosystems.

Plants, wildlife, and fish depend on sufficient water flows to their habitats to live and reproduce. The lack of adequate flow, as a result of diversion for agricultural, urban, and industrial purposes, can cause deterioration of water quality and ecosystem health. Water users can supplement their demands by using recycled water, which can free considerable amounts of water for the environment and increase flows to vital ecosystems.

Water recycling decreases discharge to sensitive water bodies.

In some cases, the impetus for water recycling comes not from a water supply need, but from a need to eliminate or decrease wastewater discharge to the ocean, an estuary, or a stream. For example, high volumes of treated wastewater discharged from the San Jose/Santa Clara Water Pollution Control Plant into the south of San Francisco Bay threatened the area's natural salt water marsh. In response, a $140 million recycling project was completed in 1997.

Recycled water may be used to create or enhance wetlands and riparian (stream) habitats.

Wetlands provide many benefits, which include wildlife and wildfowl habitat, water quality improvement, flood diminishment, and fisheries breeding grounds. For streams that have been impaired or dried from water diversion, water flow can be augmented with recycled water to sustain and improve the aquatic and wildlife habitat.

Water recycling can reduce and prevent pollution.

When pollutant discharges to oceans, rivers, and other water bodies are curtailed, the pollutant loads to these bodies are decreased. Moreover, in some cases, substances that can be pollutants when discharged to a body of water can be beneficially reused for irrigation. For example, recycled water may contain higher levels of nutrients, such as nitrogen, than potable water. Application of recycled water for agricultural and landscape irrigation can provide an additional source of nutrients and lessen the need to apply synthetic fertilisers.

5. Conclusions

One of major problems of world interest now exists is surroundings protection; in nearly all countries they take measures for limitation pollution, while one of these constituting itself (herself) worn-out waters recycling.

Worldwide, waters recycling is a technology in full expansion, basically associated agriculture (about 70%). Numerous technical solutions allow generally frame in existences norms and mainly instructions W. H. O. data irrigation on areas had been straitening and without reserves.

In countries, where fresh-water reserves are or should be confined of survival level, worn-out waters recovery can be ostensible a solution much more accessible so from financially point of view (extensive treatments they are highly elastic) as well as from technical point of view. Appear the extra costs, that in case of recycling waste waters in agriculture is not representing further of 30 % from amount cost of classical treatment to flow in natural emissaries.

Recycling technology conveniences and recovery of waster waters they are confessed by the numerous countries, these being entered often in national policies looking watery resources management.

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The Quality Characterization of the Surface Water of the Bahlui Hydrographical Basin

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Rezumat: În această lucrare se prezintă situația calității apelor de suprafață din bazinul hidrografic Bahlui, situat în Nord-Estul României într-o regiune cu resurse de apă deficitare, aici existând suficiente surse de poluare care au puternice implicații ecologice, nu numai asupra apelor. Sistemul de monitorizare a calității apelor este structurat în patru subsisteme: ape de suprafață, lacuri de acumulare, ape subterane şi ape uzate. Secțiunile de monitorizare de ordinul 1 şi 2 furnizează date cu frecvență lunară; pentru lacurile de acumulare, apele subterane şi sursele de poluare frecvența de monitorizare variază de la o lună la şase luni. Unul din instrumentele cele mai eficiente care pot gestiona calitatea apelor din acest bazin este modelarea matematică dar o altă soluție poate fi oferiră de extinderea sistemului de monitorizare pentru identificarea cât mai exactă a surselor de poluare şi de asemenea modificarea frecvenței de montorizare.

Abstract: This work presents the situation of the quality of surface water resources of the Bahlui river basin, plaed in the North-Eastern part of Romania in a region with scarce water resources. Many industrial and agricultural objectives, having serious ecological effects, are concentrated in this area of about 1,400 km^2. The monitoring system of the water resources is structured in 4 subsystems: surface water, reservoirs, phreatic water and wastewater sub-systems. On the rivers, monitoring sections of 1^{-st} and 2^{-nd} order provide data with a monthly frequency; for the reservoirs, phreatic water and pollution sources the frequency of sampling varies from 1 month to 6 months. The water quality management in the basin should be improved not only by using more efficient mathematical models, but also by extending the monitoring system in order to identify the unknown pollution sources and by rending more frequent the sampling prelevation.

Keywords: *water resources, pollution sources, water quality, monitoring system, water quality management.*

1. Introduction

The catchments area of Bahlui river basin (Figure 1), belonging entirely to the county of Iaşi, is about 1,917 km^2. The land is largely used for agriculture and cattle grazing. The population density and industrial activities are relatively high, mainly due to the city of Iaşi. Other concentration of population and economical activities are the towns of Târgu Frumos, Hârlău, Belceşti and Podu Iloaiei.

The length of the Bahlui river between its spring and the confluence with Jijia river is about 119 km. Bahlui river ends in the Jijia about 6 km before the latter flows into the Prut river, 390 km upstream from the confluence of the rivers Prut and Danube. Jijia river is channelized between the confluence with Bahlui river and the confluence with the Prut river. The old river bed called Jijia Veche unsually contains only a small amount of mostly stagnant water.

Bahlui river is, like Jijia and Prut, a rain fed river, with a relatively small discharge most of the year, and a few short periods of high waters, usually in early spring, when snow melts and most rainfall occurs. Most of the time about 75% of Bahlui river discharge of at the confluence with Jijia is represented by the effluent of Waste Water Treatment Plant (WWTP) of Iaşi town, entering Bahlui 3 km upstream from the confluence.

As much as 17 reservoirs are in operation in Bahlui basin. Dams construction took place between1965 and 1980. Initially the reservoirs were designed for flood control in downstream areas, especially for Iaşi town protection. The reservoirs are

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also used for urban and industrial water supply, commercial fishing, irrigation and recreation.

The mass balance study concerns Bahlui river downstream the reservoir Pârcovaci, a short stretch of Jijia river between the monitoring stations upstream and downstream the confluence with Bahlui river and the only monitored tributary of Bahlui - the strongly polluted Bahlueț brook.

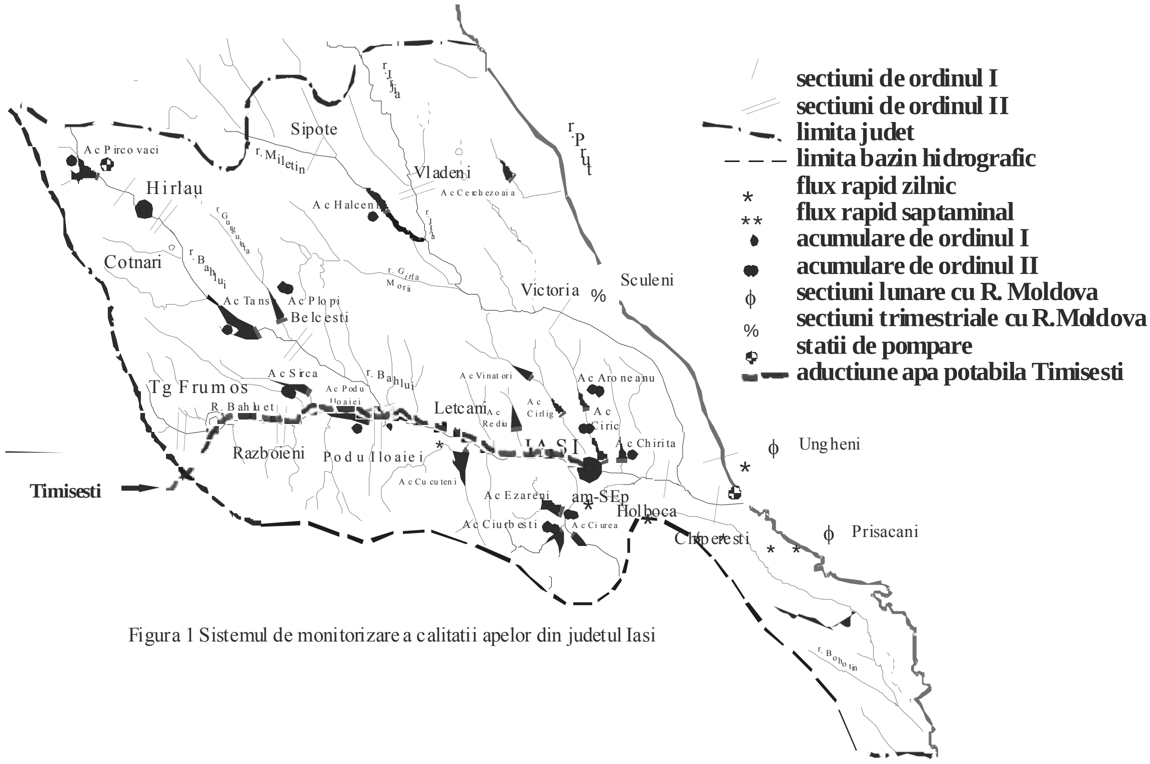

Figure 1 Iaşi County water quality monitoring system

2. Monitoring Water Quality Programme and Monitoring Stations

The surface water quality monitoring programme of the National Administration Romanian Waters (NARW) Iaşi Branch includes river monitoring stations of first and second level, with a measuring frequency of 12 times a year, respectively 4 times a year. At first level stations three samples are taken with a time step of 8 hours at 8.00 h, 16.00 h and 24.00 h; the average values of three analyses are reported. From monitoring stations of the second level just one sample is taken. The reservoirs are also given a first level or second level monitoring status. The first level reservoirs are monitored four times a year, while the second level reservoirs only twice near the dam. Furthermore, waste water discharges are monitored by NARW Iaşi Branch with a frequency of 2 to 12 times a year, depending on their potential or expected degree of pollution.

The quality of phreatic waters is monitored by drilled wells located in characteristic points of the river basin, in conection with the most important potential pollution sources.

2.1. Mass Balance

A mass balance compares the amount (named load) of a substance entering and leaving a certain part of a water system. The load of a substance in a stream is computed by multiplying the water discharge by the concentration of the substance. The time interval for which the discharge (m^3/s) is expressed becomes the time interval for the load (g/s) [1], [4]. The average loads of the river were computed for the years 1995, 1996 and 1997, corresponding to different hydrological regimes.

The instantaneous loads get a weighting factor proportional to the discharge (Figure 2). The average load L_{av} is [4]:

$$L_{av} = \sum_{i=1}^{n} (q_i \cdot c_i)/n \qquad (1)$$

where q_i is the measured discharge, c_i the concentration of the monitored parameter and n the number of the chemical measurements.

The average weighed-discharge $L_{av,Q}$ is computed as [4]:

$$L_{av,Q} = k\left(\frac{\sum_{i=1}^{n} c_i q_i}{\sum_{i=1}^{n} q_i}\right)\cdot \bar{q} \qquad (2)$$

where k is a conversion factor considering the drawing frequency and the units used for concentration and discharge; $\bar{q}$ is the average discharge computed on the basis of daily registrations.

The load decrease of the effluents from WWTP Iaşi in order to fulfill the quality requierements at the confluence of Bahlui river with Jijia (Chipereşti section) can be analyzed according two scenarious, considering that the pollution from the existent sources remains at the same level:

a) framing in the I[st] category at Chipereşti; this category cannot be achieved only by improving the operation at WWTP Iaşi, being necessary to diminish the pollution at all the other sources.

b) framing in the II[nd] category in the same section; the necessary concentration at Victoria (Jijia) and Bahlui upstream WWTP Iaşi is: 15 mg O2/l COD, 7 mg O2/l BOD5, 3 mg/l ammonium [1].

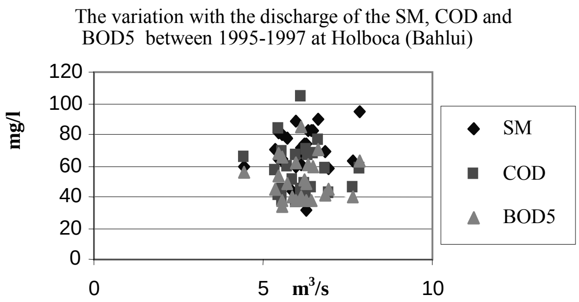

Figure 2 Values of the loads weighed with the discharge used for mass balance analysis

3. Chemical Characterisation of the Surface Waters

The main reaches of Bahlui and Bahlueţ rivers are characterised as following [1]:

- *River reach Bahlui spring – Hârlău town*. According to Romanian standard this river section is in I-II category for oxygen regime and special toxic parameters;
- *River reach Hârlău town – confluence of Jijia and Bahlui rivers.* The water quality is of bad quality for the whole length of the reach. Downstream Belceşti, Podu Iloaiei and Holboca the oxygen specific indicators indicate a degraded water due to human activities, zootechnical farms and other pollution sources;
- *River reach Bahlueţ springs – downstream Târgu Frumos,* according to oxygen regime and special toxics belongs to second water quality;
- *River reach downstream Târgu Frumos-confluence of Bahlueţ-Bahlui rivers* with a degraded water due to oxygen regime indicators in the section Războieni and upstream Podu Iloaiei;
- *Upstream WWTP Iaşi* the Bahlui river is degraded: average mean annual values of COD is 32.8-37.6 mg/l, ammonium 1.1-1.3 mg/l, phenols 0.001-0.009 mg/l;
- *Downstream of WWTP Iaşi* the Bahlui river is degraded: average mean annual value for COD 35.0-47.9 mg/l, ammonium 3.92-4.39 mg/l, phenols 0.001-0.022 mg/l and total iron: 0.9-1.2 mg/l.

Concerning the length of the water quality of the river reaches, out of the total of 160 km of the main rivers of the basin (Bahlui and Bahlueţ) only 44 km (in 1997) and 27 km (in 1998-2001) fit into the I[st] and II[nd] categories; the rest of 116 and 133 km respectively, fit into the III[rd] and D categories. In the year 2000 only 27 km in the upper part of Hârlău basin belonged to the I[st] quality, the rest being affected by pollution [1].

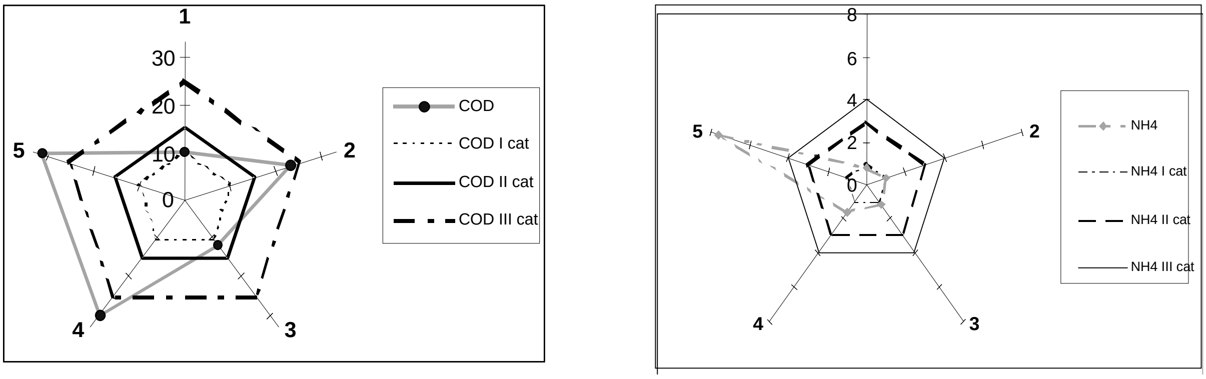

Figure 3 The evolution of the COD (mg/l) and NH4 (mg/l) indicators on the section river of Bahlui and Bahlueţ rivers in 2001 (1- Hârlău upstream, 2 – Hârlău downstream, 3- Belceşti, 4 – Podu Iloaiei, 5 – Holboca)

Table 1 Fitting the surveillance sections into quality categories and reference classes [1], [5], [6], [7]

No.	Water course	Monitoring section	Order	Oxygene Regime		Special Toxics		General	
				STAS 4706/88	EU Reference Classes	STAS 4706/88	EU Reference Classes	STAS 4706/88	EU Reference Classes
1	Bahlui	upstream Hârlău	I	II	II	II	II	II	II
2	Bahlui	downstream Hârlău	I	III	IV	D	IV	D	IV
3	Bahlui	downstream Belceşti	II	III	III	D	IV	D	IV
4	Bahlui	Podu Iloaiei	I	D	IV	D	II	D	IV
5	Bahlui	Holboca	I	D	IV	D	IV	D	IV
6	Bahlueţ	upstream Târgu Frumos	II	III	IV	III	III	III	IV
7	Bahlueţ	Războieni	II	D	IV	D	IV	D	IV
8	Bahlueţ	upstream Podu Iloaiei	II	D	IV	D	IV	D	IV

I - first category; II – second category; III – third category; D – degraded category.

On a general characterization by the Refference Classes of the European Community, by parity of reasons with STAS 4706-88, the parameter of the most defavoured parameter was applied (Table 1) [5].

The evolution of the COD and NH4 indicators on the section river of Bahlui (Belceşti, Podu Iloaiei and Holboca) and Bahlueţ (Hârlău) rivers in 2001 are show in the figure 3.The global characterization of the water quality in the reservoirs may be performed in agreement with the standards and considering the reservoirs as dyamic eco-systems and operating with average values of the quality parameters for a given period.

From the reservoirs of first level monitoring status studied between 1998-2002, considering the phosphourous concentration two of them, Chiriţa and Pârcovaci belong to the I[st] quality category, while the rest to the II[nd] and III[rd] category. The trend of the global quality shows that five reservoirs of the second level monitoring status have a negative evolution, four of them have a positive evolution, and three tend to preserve the present quality. From the analyzed parameters, 29 % had an improving trend, 37% a decreasing trend, and 34% preserve the present quality.

Generally, the reservoirs of second level monitoring status fit into the IIIrd and D category, which shows a bad water quality both from chemical

and biological point of view. The siltation of these lakes as well as the increased pollution during the past years led to the quality deterioration [1].

3.1. Podu Iloaiei Reservoir

Podu Iloaiei reservoir, built on Bahlueț river, has at the normal water level a surface of 2.1 km^2, and a volume of 2.5 million m^3. Some of the water users (fishery, irrigation) need a water of the II category; the lake has in the same time a touristic potential imposing thus for the swimming water the Ist category. However, the levels of COD and BOD5 fall in the categories III and degraded; the lake has too high bacteriological pollution and phytoplankton blooms, those of bluegreen algae being the most problematic. The oxygen levels registered in 1995 to 1997 were mostly in the Ist category (six value in the Ist category; one value in the IInd category).

Phosphorus concentrations measured in 1995 and 1996 exceeded the STAS standard value corresponding to category III, which was not the case with the values measured in 1997. Due to the limited number of measurements (2 in 1995, 3 in 1996 and 2 in 1997), the conclusion that the phosphorus load entering the lake decreased would be premature. Moreover, the lower values of 1997 year could be the effect of algae blooms, which can convert almost all dissolved phosphate (ortho-phosphate) into algae-bound phosphate, which is not included in phosphorus determination by NARW Iaşi. Water samples are filtered, removing thus algae and particulate matter to which phosphorus can bind.

Two values of ammonium measured in 1995, in samples taken in January and early March, indicated a degraded water, while the five values measured in the spring and summer of 1996 and 1997 were much lower, corresponding to the category I and II. Algae blooms lower also the concentration of nitrate, especially because the uptake of ammonium by algae cuts off the nitrate production by nitrification. All nitrate levels measured between 1995 to 1997 were outside the limits of the Ist category. Measuring of Kjeldahl-nitrogen in unfiltered samples could confirm whether the nitrogen is indeed taken up by algae [2].

Understanding the pollution mechanisms of Podu Iloaiei reservoir and possible measures to improve the water quality require: a monitoring station immediately upstream from reservoir; a higher monitoring frequency of the reservoir and accurate discharge registration; measurement of total phosphorus and Kjeldald-nitrogen in unfiltered samples [1], [2].

3.2. Tansa Reservoir

Tansa reservoir, built on Bahlui river, has at the normal water level a volume of 6.8 million m^3.

Because the water of Tansa reservoir is used for the production of drinking water, it should be in the Ist category of quality. According to the monitoring data (1995 to 1997), the water quality is better than that of Podu Iloaiei reservoir, but the organic matter content is too high, and from time to time the lake has high bacteriological pollution and algae blooms. BOD5 concentrations from 1995 to 1997 usually qualified the water as belonging to the categories II – III and occasionally as degraded water. COD corresponds sometimes to the Ist category of quality, but mostly to the categories II and III. Sometimes ammonium exceeded the limit of the Ist category. Phosphorus exceeded 0.1 mg/l limit during the measurements in the years 1995 and 1996, which was not the case in 1997. As noted above, phosphorus which is present in algae and other suspended matter is not included in the measured values. Oxygen and nitrate levels corresponded to the Ist category [1], [7].

4. Conclusions

The waters of the Bahlui hydrographic basin are mainly affected by the industrial and agricultural activities and by the waste water evacuation of the main urban localities.

The economic, the public administration units'activity and industrial fields are significantly involved in the pollution process, because by their emplacement mainly onto urban localities, are connected to the city sewage systems. This hides their direct impact on water courses, the polluting substances being mixed with the specific domestic residuum. The zootechnical farms from Iasi are part of this category, for instance.

The quality improvement of the Bahlui hydrographic basin can be accomplished by:
- improving the functional efficiency of the urban WWTPs;
- decreasing the industrial nature pollutants by assuring the proper functionality of the WWTPs at the economic agents connected to the public sewage system;
- replacing the hydraulic evacuation of the zootechnical dejection technologies with dry evacuation systems; so it results a lower volum of dejections and its nutritive potential can be capitalized;
- evaluating by monitorization the pollutant charges of the effluent creeks, inclusive of surveillance campaigns nearby the confluence;
- installing monitoring sections nearby upstream storage and completing the database by increasing the monitoring frequency and the quality of the data as well as the measure of the total phosphour and azote Kjeldahl on unfiltered section as well as of focus azote and phosphour from the suspension matter – in order to understand the phenomenon which affects the quality of the Podu Iloaiei and Tansa reservoirs;
- better monitoring of the heavy metals and micro-polluting organic matters, especially of the pesticides; in this respect, for some of the pollutants with a high level of toxicity, the sampling are restricted to some significant emplacements with a lower frequency (once a year, for instance);
- compiling a GIS environmental database which should identify the areas with the risk of affecting the water quality;
- concluding the application mechanism of the environment protection legislation and the financial measurments follow up of the redemptions for overloading the authorized values of the markers for the water utilities;
- ellaborating a policy of reducing the pollution having in mind the present situation, the immediate measurements and necessities and also the long term necessities;
- inventorying the discharge points and diffused sourced as a must on long term as priority.

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Aspects Regarding the Pollution and Protection of Underground Water Sources

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Rezumat: Sursele de apă subterane trebuie protejate astfel încât să-şi păstreze caracteristicile de apă potabilă. În acest sens, în jurul sursei de apă se instituie un perimetru de protecţie suficient de mare, astfel încât la o impurificare accidentală cu substanţe chimice sau organice biodegradabile, datorită procesului de autoepurare al solului, apa să-şi recapete caracteristicile de apă potabilă.

Abstract: The underground water sources must be protected so that the water will keep the potable water characteristics. In this respect, round about the water source will be set up a protection perimeter, big enough so that at an accidental pollution with chemical or organic biodegradable substances, because of the soil self-waste-treatment, the water will get back the potable water characteristics.

Keywords: underground water, catchment, polluter, protection zone.

1.Generalities

The prevention of underground water sources' pollution is a more chipper process then the activity of finding out, treatment and elimination of the effects of a possible pollution. In Romania, 80% of the rural area inhabitants use as drinking water the water from the phreatic layer from individual wells. The risk of using the phreatic layer, which is not protected against the chemical and bacteriological pollution, is big because the chemical pollution with chlorides and nitrates as well as the bacteriological pollution due to the lack of some measures and works for hygienically–sanitary protection can appear. The importance of this problem led to the elaboration of some normative acts of regulating and correct institution of water sources protection perimeters.

2.Some aspects of aquifers' pollution

The aquifers' pollution is characterised by the following aspects:

a) the process is very slow in the aquifer aeration zone, being registered vertical percolation speeds ≤ 1m/year;
b) the process is very stable, the aquifer remaining polluted because of the underground water dynamics;
c) Lent migration of pollution

The aquifer's pollution sources can be punctiform or diffuse sources situated at the ground surfaces in soil or subsoil. The main pollution sources of the underground water can be from the agricultural activities, waste landfills, septic fosses, as well as the illegal discharges of chemical or oil substances.

3.The transport equations of the polluters in porous mediums

The main processes in polluters transport are: diffusion, advection and sorption. The diffusion of a polluter in a stationary process case is described by the relation:

$$f = -D_m \frac{dc}{dz} \tag{1}$$

in which: f – the polluter flux transported on a surface unit and time unit;

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Dm – the molecular diffusion coefficient of the polluter in the considered medium;

C – polluter concentration in point of z coordinates at the time t,

dc/dz – the concentration gradient that constitutes the driving force of the diffusion process.

In a saturated porous medium, the polluters' diffusion is expressed by:

$$f = -nD_e \frac{dc}{dz} \tag{2}$$

in which: n – effective porosity of the porous medium,

D_e – the effective diffusion coefficient in porous medium.

The polluters transport through advection is the movement of the polluters together with the water. The water running in the soil is expressed by the Darcy law:

$$v = -k_H \frac{dh}{dl} \tag{3}$$

in which: v – infiltration speed;

k_H – hydraulic conductivity;

h – hydraulic load;

l – length of the line of flow in porous medium;

dh/dl –hydraulic gradient.

The hydraulic conductivity depends on the porous medium through its permeability, as well as on the fluid through density and viscosity.

The sorption process can be described through a linear model:

$$S = k_D C \tag{4}$$

in which: S represents the polluter concentration in liquid adsorbed phase,

k_D – the distribution coefficient of sorption,

C – the polluter concentration in liquid phase.

In this model, it supposes that exists proportionality between the polluters' concentrations from the solid and liquid medium.

The general equation that describes, according to the law regarding the preservation of mass and flux, the polluters transport through porous mediums with variable saturation level and that counts the chemical advection processes is:

$$\theta\frac{\delta C}{\delta t} + \rho\frac{\delta S}{\delta t} = div(\theta DgradC\ ) - VgradC - x(\theta C + \rho_b S) + QC_{in} - QC \tag{5}$$

in which:

θ – volumetric humidity,

C – polluter's concentration,

t – time,

ρ_b - volumetric density of the medium,

S – polluters' concentration in adsorbed phase,

D – dispersion coefficient tensor,

v – Darcy speeds' vector,

χ – polluter's decomposing constant,

Q – included flow,

C_{in} – polluter's concentration in the injected fluid.

Limits with imposed polluter's concentrations (conditions of Dirichlet type) are defined as follows:

$$C = C_d(x_d, y_d, z_d, t\ ) \text{ on } B_d \text{ limit} \tag{6}$$

in which: C_d – imposed concentration,

B_d – portion on the domain's outline with the condition of Dirichlet type,

x_d, y_d, z_d – outline coordinates.

Zones with clean water situated upstream of the pollutant sources can be approximated through this condition. Limits with imposed polluter flux

(Cauchy type conditions) are the limits from where infiltrations began. These infiltration's can appear from the levigation migration from the solid waste landfills or other impounded surfaces, applying of fertilisers or pesticides on agricultural surfaces, the effect of diluting the polluters resulted from pesticides or fertilisers due to rain or irrigation. Condition of limit with imposed flux is:

$$n(VC - Q\Delta gradC) = q_C(x_c, y_c, z_c, t) \text{ on } B_c \text{ limit.} \quad (7)$$

in which: n – vector of the normal line on limit,

VC –advective flux,

QΔgrad C – dispersive flux due to the concentration gradient,

q_c – imposed flux,

B_c – portion from the domain outline with the Cauchy type condition.

4.Methods of resolving the equations of polluters' transport through porous mediums

Methods of resolving the equations that governs the transport of polluters in porous mediums can be grouped in the following categories:

4.1 Analytical resolving is generally considered as being ideal for fast calculation. These solutions are obtained for simple, ideal problems.

The best-known analytical solution is calculated for a layer of infinite thickness, with polluting source at the surface, with the concentration c_0 constant in time. For the depth z and time t has the form:

$$c = J(z,t) \quad (8)$$

in which

$$J(z,t) = \frac{c_0}{2}\left[\exp\left(\frac{v_z}{2D_e}\right)\cdot \exp(-ab)\cdot erfc\left(\frac{a}{2\sqrt{t}} - b\sqrt{t}\right) + \exp(ab)\cdot erfc\left(\frac{a}{2\sqrt{t}} + b\sqrt{t}\right)\right] \quad (9)$$

where

$$a = z\sqrt{\frac{n+\rho_b K_d}{nD_e}}\ ;\ b = \sqrt{\frac{n}{2D_e(n+\rho_b K_d)}} \quad (10)$$

$v = v_z$ – underground water speed on vertical (after Darcy)

$D_e = D_{ez}$ – the effective diffusion coefficient in z direction (vertical)

n – the effective porosity of the soil

ρ_b – soil density in dry state

K_d – distribution coefficient of the linear sorption model.

The problem solution in the case of a layer with the finite thickness H, which has at the base an aquifer layer of thickness h, is:

$$c = -\left\{\sum_0^\infty \exp\left(-\frac{v}{D_e}[(p+1)H - z]\right)J[2(p+1)H - z, t] - \sum_0^\infty \exp\left(-\frac{v}{D_e}pH\right)J(2pH + z, t)\right\} \quad (11)$$

in which:

J(z,t) – is obtained from the equation (9) for

Z = 2(p+1)H – z and Z = 2pH +z

and the rest of terms have the meaning presented above.

It was observed that in practice the series converge fast and need only few terms.

Existing analytical solving can be used directly for polluters transport prognosis only for a limited domain of problems.

4.2 The finite layer method implies either a Laplace transformation (for the mono-dimensional case), or a Fourie transformation (for bi and tri-dimensional) of the equation that govern the transport of polluters through porous mediums, and then finding of some analytical solutions in the transformed space.

This method can be applied only for situations that can be schematised by horizontal layers, and the soil properties are the same in any point of the layer. As a consequence of this, the vertical and horizontal speeds can not vary with the horizontal plan position. Nevertheless, for many practical problems, reasonable estimations can be obtained in analysing the impact of the waste landfills on the underground water.

4.3 The margin element method is considered appropriate for solving the equations for advective – dispersive polluters transport through porous mediums. The main advantage against the finite layer method consists of the possibility of modelling more complicated geometries.

4.4 The finite differences and finite element methods have a wide application in analysing the polluters' transport in porous mediums. There is a multitude of researches in this sense. These methods can be used for:

- analysing the flow in soils for calculating the speeds' field;
- analysing the polluters' transport using the speeds' field previously determined.

With the finite element method can be modelled both complicated geometries and the non-linearities of the phenomenon. In the speciality' literature are published various algorithms for solving the equation of advective – dispersive polluters transport through porous mediums. Their use is not so simple. In order to obtain good results is important to correct choosing of the algorithm, of the finite elements network, and of the time steps.

All these methods of solving the equations that govern the polluters transport in porous mediums have their role in designing the waste landfills and barriers that hinds the underground water pollution.

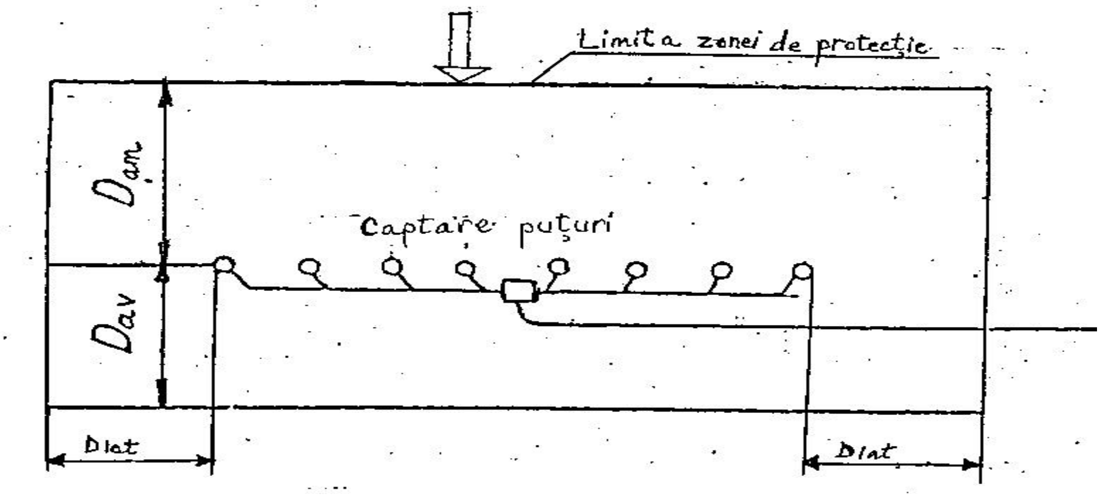
5.The sanitary protection of the underground catchments for drinking water

5.1 Protection zones for water sources

The water to be potable, some characteristics must comply with STAS 1342-91. As in the case of water from underground sources the water doesn't need safety disinfection, is needed to be assured the protection against pollution. In this sense, the catchment location is chosen so that to be upstream or lateral from the supplied locality. Exception from this makes the wells with depth over 100 m, because at those it is considered that the layers' succession above the catched aquifer assures sufficient protection. It was find out that in the case of a natural pollution with chemical or organic biodegradable substances, if the water run through a clean porous layer, for 20 days, due to the soil natural process of self-treatment, the water obtains the characteristics of the potable water. For this reason, the surface around the source corresponding to this period of time, is named severe regime zone, will be enclosed, will become overgrown with grass and will not be used fertilisers or pesticides. Around this zone, will be instituted a second zone, determined so that the water will cover the subsoil in minimum 50 days, named restriction zone, in which the laying out of any construction will be done only with the approval of the sanitary bodies.

5.2 Determination of the size of the sanitary protection zones

The size of the severe regime protection zone must be determined with accuracy, because the delimitated surface will be de-allocated from other uses. According to the Decree 1059-67, this zone cannot be smaller than 50 m upstream and 20 m downstream.



D_{am} = upstream protection distance; D_{av} = downstream protection distance; D_{lat} = lateral protection distance

Fig. 1 Sanitary protection zone for underground water catchments

For determining the geometrical elements of the sanitary protection zone, it's admitted a simplified calculation scheme, in which the water pass through the underground way from the sanitary protection zone limit to the catchment, in the normal filtration time, neglecting the time in which the polluted water pass through from the ground surface to the underground water layer. Through this is assured an increased level of calculation assuring. It is considered that the aquifer layer is uniform, homogeneous and isotope and the underground water movement is done according to Darcy law. Calculation of the sanitary protection distances is made through:

5.2.1 Calculation of the time T, in which a fluid particle cover the distance between the perimeter limit and the well.

By integration of the differential equation:

$$dt = \frac{dl}{\frac{v}{p}} \qquad (9)$$

in which: dl – a space element on the water particle trajectory, v – local speed in the considered point, p – porosity coefficient of the soil from the aquifer layer, results:

$$T = p\int_{D_1}^{D_2} \frac{dl}{v} \qquad (10)$$

in which: T – time in which a fluid particle covers a distance $D = D_2 - D_1$, in which D_2 and D_1 are the limits of this distance.

For a singular well in under pressure aquifer basin, the sanitary protection distance D_1, measured from the well axis, by solving the equation (10) between limits D_1 and r, results:

$$T = \frac{2\pi M}{Q}\left(\frac{D_1}{2} - \frac{r^2}{2}\right) \qquad (11)$$

If the term r^2 is neglected, results for the protection distance calculation in this case:

$$D_1 = \sqrt{\frac{QT}{\pi p M}} \qquad (12)$$

in which: Q – well's maximum flow,

T – time for the water for passing through the protection zone,

p – effective porosity coefficient,

M – thickness of the under pressure aquifer layer.

5.2.2 Equalisation of the water volume included inside the severe regime perimeter, with the water volume extracted from the well in time T, duration needed for water purifying.

The method can be used in some cases of the well in underground basin. Through this method results the equation:

$$QT = p\pi D_1^2 M \tag{13}$$

from which results the same expression for D_1 as in the precedent method.

For the phreatic water layers at which the movement is potential, it can be admitted:

$$M = H - \frac{s_0}{3} \tag{14}$$

resulting the expression of the upstream protection distance:

$$D_{am} = \sqrt{\frac{QT}{\pi p\left(H - \frac{s_0}{3}\right)}} \tag{15}$$

5.2.3 Analysis of the hydrodynamic spectrum of the movement and identification of the limits of the zones that brings the water to the wells.

This method is used for the catchments through wells in under pressure underground stream. The water movement to the catchment well results from the composing of two elementary plane movements, namely: axial-symmetric movement of the water from the layer to the well, negative source with the unitary flow Q/M and the parallel flow with the speed v_0.

For a singular well in under pressure underground stream, the movement spectrum is as presented in fig. 2

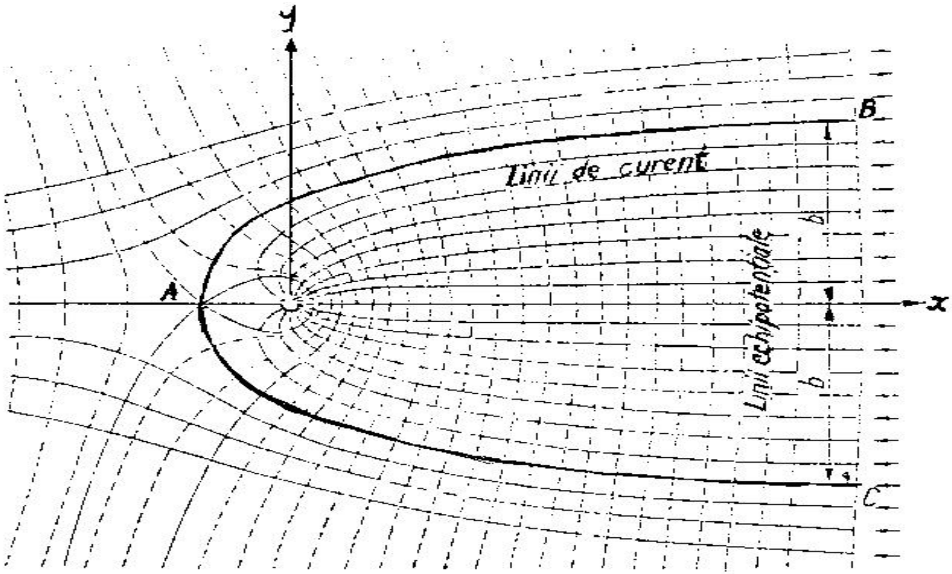


Fig. 2 Movement spectrum of water from an under pressure underground stream to a singular catchment well

The well is supplied with the flow Q/M from a distance:

$$2b = \frac{Q}{Mv_0} \tag{16}$$

As it can be noticed from fig. 2, the spectrum has a point a, which represents a point of null speed. The supply of the well with water is done only from the region marked by the stream line BAC, leading to the conclusion that the severe regime perimeter of the protection zone must be inside this curve. The limit values of the protection distances are determined through the crossing of the ox and oy axis with the BAC curve. It can be written:

$$(D_v)_{\lim} = \frac{b}{\pi} \text{ and } (D_l)_{\lim} = \frac{b}{2} \tag{17}$$

In order to determine the lateral distance, admitting the rectangular shape of the perimeter, it's considered in a covering way:

$$D_{lat} = b \tag{18}$$

5.2.4 Calculation of the sanitary protection distance for a line of wells

For a line of wells bored in an underground basin with ascending level, the sanitary protection distance D is determined with the diagram from the fig. 3, calculating the sanitary protection distance for a singular well D_1.

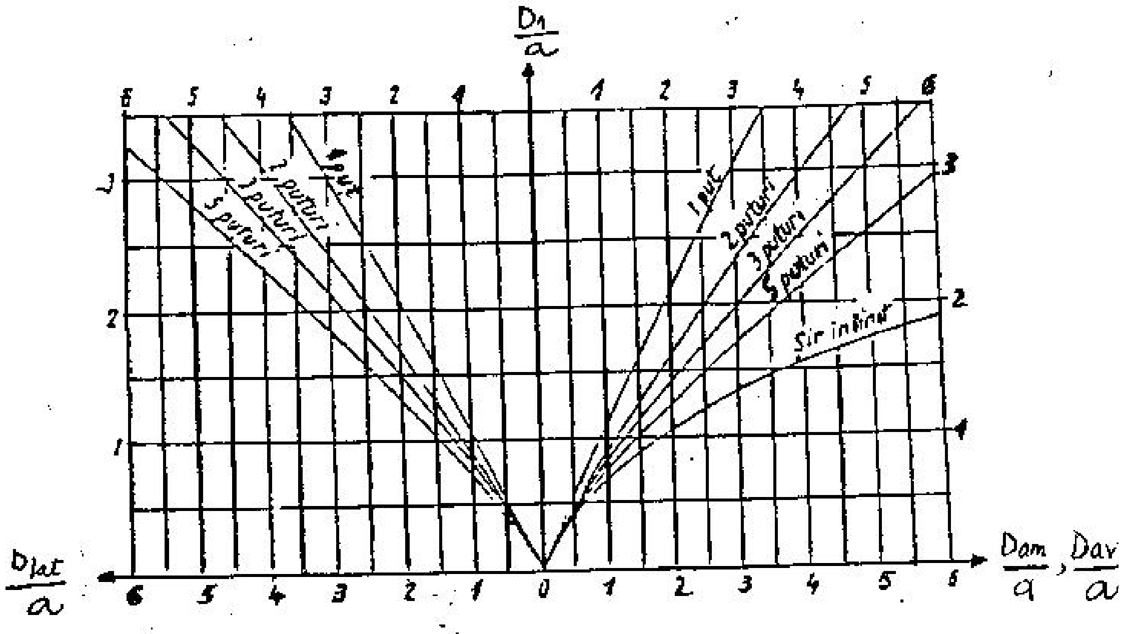


Fig. 3 Diagram for sanitary protection distances calculation for the wells' row in under pressure underground basin

For a wells' row in phreatic water layers, it's taken into account the fact that the depression surface is lower than at an isolated well, because the influent between the wells. For this reason is needed a covering calculation and it is considered:

$$D_{am} = \sqrt{\frac{QT}{\pi p\left(H - \frac{s_0}{2}\right)}} \quad (19)$$

6. CONCLUSIONS

In present conditions, approximately 80% of the rural inhabitants use for water supply water from the phreatic layer unprotected against chemical and bacteriological pollution. The main polluting sources are the infiltrations and the leakages from the septic fosses, stables, wastewater, as well as the use of an uncontrolled chemistry in agriculture.

The correct determination of the sanitary protection zones for the underground water catchments assures the prevention of possible pollutions of underground water sources, that is cheaper than the elimination and treatment of the pollution effects.

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The Project of Mining Exploitation in Rosia Montana and the Influence over the Environment

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Rezumat: Extinderea exploatării miniere de la Roşia Montana se doreşte a fi amplasată în Munții Apuseni şi se va realiza prin mărirea capacității exploatării existente.
După cum se ştie, multe organisme de stat şi neguvernamentale au alertat autoritățile şi publicul privind aspectele sociale şi de mediu care vor apare la realizarea proiectului, în timpul realizării şi funcționării minei şi după abandonarea acesteia.
Lucrarea prezintă amploarea proiectului, capacitățile de producție şi problemele care apar din punct de vedere social şi de mediu.

Abstract: The extinction of mining exploitation in Rosia Montana it is aims to be placed in the Western Mountains and will be realized by increasing the capacity of existent exploitation.
As it is known, many state organisms and NGO has presented awareness for authorities and public regarding the social and environmental aspects which will appeared during the construction, operation and closure of the project.
The paper presents the imenssity of the project, production capacities and social and environmental connected problems.

Keywords: mining exploitation, environment, impact assessment

1.Introduction

The gold and silver exploitation in this area has begun before the *conqueror* of Dacca by Romans.

The company Rosie Montană Gold Corporation S.A. with Resources Ltd was established in 1998 year for the realization of the huge intensive exploitation of existing gold and silver resources during 17 years. After these time the mine and all facilities will be abandoned.

In 2003 year at Alba Iulia has been place a meeting with the representatives of the two companies and decision makers from different fields. At the meeting was presented the project of the future exploitation.

The gold concentration in analyzed rocks is 1,7g/ t and for silver is 9,1g/t.

The annual production will be 17,107 kg gold and 80,869 kg silver.

The total land surface occupied with all works will 2,000 ha.

2.Plans

In the present, there are not sure projects and even environmental impact assessment study.

The two companies promise the realization of the project with the help of the best international experts in the field.

Plans which have been realized regard:

- movement of the population of the other places;

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- social and environmental protection actions;
- public consulting and information;
- cultural heritage management;
- regional development;
- management for decantation ponds;

- cyanides management;
- water management;
- water and erosion management;
- biodiversity conservation;
- intervention in cases of damages/accidents;
- closing the activity and environmental recovery;
- environmental and social monitoring;
- social and environmental management.

3. Project description

For geological researches and for determination the concentration of gold and silver in the rock were made many boreholes. In the beginning the Acetate and Cornice or will be exploited and next extinction will be at Orlea and Jig ores.

The total annual ore processed quantity will be 13 millions tones.

At present, exploitation the total annual quantity processed is 400,000 tones, in excavation and underground as well.

The activities before mining will involve the removal of forests, vegetation, fertile soil and sterile rock.

The timber from cuter forests will be sell to private societies and to the population.

The results of these activity will be keep for closing activity use.

The conventional exploitation methods consist in a surface excavation using explosives in boreholes, with a rate of 2-3 weekly, charging the rock in trucks of 150 tones capacity and the transport of the economically useful rock at the preparation plant for processing.

The sterile rock will be transported at disposal sites and to decantation pond for the construction of the dam.

The preparation plant will process with 36,000 tones/day and the sterile rock quantity will be 30,960 tones/day.

For gold and silver concentration at preparation plant will be made using cyanides.

The wastewater with cyanides content will be discharged at decantation pond.

The life cycle estimated for the four mines is: Cetate 10 years, Cirnic15 years, Orlea 7 years and Jig 6 years.

4. Aspects of exploitation influence over the environment

First at all, the landscape will be destroyed, of the forests, vegetation and a huge land surfaces affected.

The acid waters it is possible to occur because the ore has a big sulphur content. The acid waters appear in the moment when rains wash the sterile rocks disposal sites or the decantation pond which content acid material.

The protection measures for protect the environment in this case are the following:

-in the decantation pond the treatment of the discharged waters with lime;

-parietal dices around disposal site of sterile rock and the transport by pumping of acid waters in treatment before the discharge in the river.

All the measures must be taken of the beginning of exploitation.

But the most important problem environmental point of view is represented by cyanides use for ore processing.

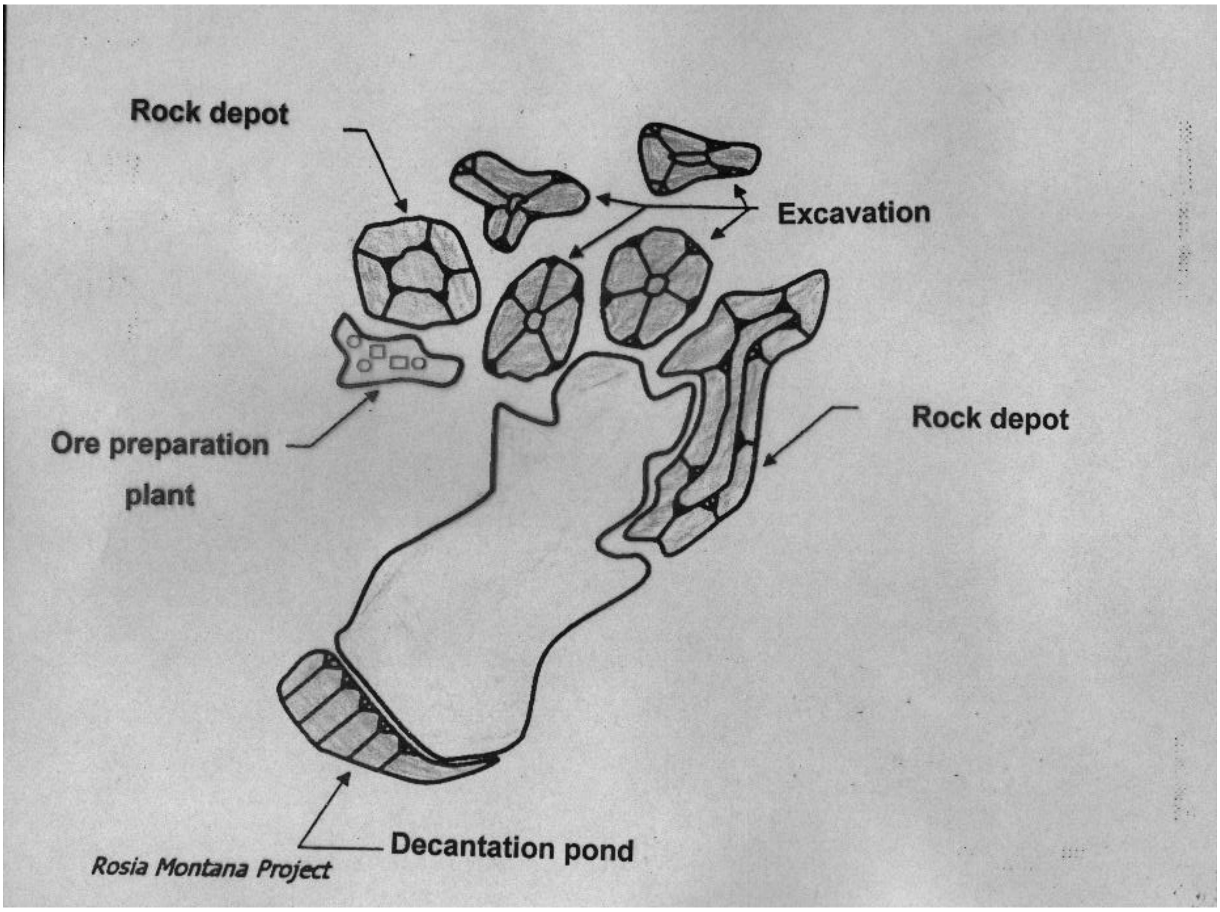
From the preparation plant waste water with cyanide content is discharged in the decantation pond.

In the contact with the air the result consists in different combination of cyan ion, but they are very stable and toxic as well.

The discharged water in creek from the decantation pond content cyanide as well.

It was necessary the elaboration and implementation of International Code for Cyanide Management, which is an issue from United Nation for Environmental Protection (UNEP). It is known the effect of Baia Mare accident in the year 2000, when the waste water with cyanides content was discharged over the dam of the decantation pond. The difference is than at Rosia Montana exploitation the decantation pond will bigger and it will be existing even after closure of mining activities.

At the study place the winters are very strong and flush floods occur every year many times and the possibility of accidents must be taken into account.



5. Conclusions

Following all problems presented is understanding the public awareness by mass media against the project realization.

Romanian Science Academy, associations of experts, NGO’s, romanian and foreign mass media, have taken negative attitude regarding this project.

The Mondale Bank from which are expected to give the financial founds for the project starts to made a very detailed analyze of the project and in the present the accord is not favorable.

The inhabitants in the affected area don't want to left the origin site sand the historical monument of old roman exploitation to be destroyed.

The alternative for the regional development is the ecotourism due to the potential of the Western Mountains and the specific hospitality of Romanian people.

6. Aknowledgments

Thank you to my colleague Paula Catana for tehnoredactation.

Present-Day Trends in the Ecologically-Responsive Design of Buildings

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Rezumat: Într-o perioadă de criză energetică, conceptul de clădire ecologică a cunoscut o ascensiune rapidă datorită avantajelor pe care le prezintă din punct de vedere al consumurilor energetice şi al microclimatului creat. Acestea urmăresc utilizarea energiilor ce țin de potențialul propriu al clădirii şi realizarea unui mediu cât mai aproape de cel natural.

Abstract: The notion of ecologically-responsive building has undergone a rapid development during this period of energy crisis due to the advantages offered in so far energy consumption and the microclimate developed is concerned. They point to the use of the energy related to the potential of the building itself and to creating an environment that is as close to the natural one as possible

Keywords: *Ecological building, bionic principles, passive low-energy, natural ventilation, ecosystems.*

1. Introductory notions

The concentration of the administrative and business activities in the very much required central urban areas has brought about the tendency of erecting more and more vertical buildings. However, this type of buildings has shown detrimental aspects, such as:

- these buildings are high consuming structures, with a negative energy balance;
- they are polluting waste producers to a large extent;
- they have created the suitable premises for the setting up and evolution of acari, disease generating bacteria – allergies, respiratory diseases, the sick building syndrome, the Monday fever, Legionella – or psychological disturbances, such as claustrophobia.

Statistics show that during the third millennium, intensive buildings shall be erected in the most of the world cities, mainly in Asia and America. These buildings be they either skyscrapers, or shopping malls or convention centres, will gather together large size human agglomerations. This trend is going to remain operational until other economic alternatives shall be invented or the planning administration shall be reformed or the rural-to-urban migration shall be stopped.

It is in this background that to set up a proper climate in the environment people carry out their activities in the largest part of their time becomes a necessity. Thus, tall buildings, social, cultural and administrative buildings should exhibit a perfect correlation between the building members and the heating/ventilation systems in order to eliminate discomfort issues: thermal stress, surface condense, radiation asymmetry, separating surfaces low air temperature, air purity etc.

These represent priorities as objectives for the architects and designers/engineers, a conundrum expecting solutions, irrespective of the type of building extension, i.e. vertically or horizontally. A conception that was present mainly ideologically up to the last decade of the 20th century and that has become a firm alternative to the existing buildings is that of constructions erected on the principles of ecological and bionic intelligence. This notion has developed in the economically and technically advanced countries in Europe, though at the beginning they were rather demonstrative in character. The design reached a maximum with the works of Sir Norman Foster, such as: the rehabilitation of the German Parliament in Berlin, the Commerzbank Tower in Frankfurt, the Citygate Eco Tower, 30 St. Mary-Axe London, the City Hall in More London etc., that will be referred to later on in the present paper.

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Mention should be made that Europe witnesses the concern of the builders to achieve neo-modernistic buildings, with unique architectural shapes, though no special emphasis is put on vertical developments. The challenge of the designers lies in the capacity of extrapolating beneficial outcomes already reached in these buildings and adapting them to taller structures. In this respect, some buildings that are part of some case-studies and that are suitable to the demands of ecological buildings shall be taken as reference points.

The ecologically-responsive buildings represent constructions that bear a minimal impact upon the natural environment, that integrate with the ecological systems of the sites forming the so-called ecosystems and that make use of the traditional energy and of the non-conventional forms of energy in an efficient manner. Thus, interactive buildings that perfectly integrate to the natural environment are reached.

This notion has reached the attention of the experts in civil engineering and of the law-makers of the country following a finding that says that 30% of the national energy consumption and a significant amount of CO_2 are produced in the building sector. In this way, some norms and regulations related to the thermal insulation, limitation of energy loss and rational energy consume have been implemented. Following the studies carried out by experts and due to the advances in the insulation of the building façade correlated to adequate equipment for buildings with no subsequent effect upon the heat comfort of the habitat took place. For instance, in the 70's a building used to consume about 300 kW/h/m^2, while in 1995 this was reduced to 100 kW/h/m^2 and it continued to be diminished so that in 2000 the need reached only 70 kW/h/m^2. A complex research of all the intervening and interacting factors that has included a comparison with living bodies led to a favourable and encouraging energy balance.

2. Strategies of erecting ecological buildings

Leaving aside the rationale, the strategies aimed at by the creators of the concept of ecologically-responsive building can be summed up in the following:

- the reconsideration of the bearing structures and of the intelligent façade and domotitic systems, optimising of the architectural shape and volume morphology;
- the orientation of the constructions so as to store and use passive solar energy;
- new technologies related to active solar energy that enable the conversion into heat and electricity – by means of façade integrated photovoltaic cells or by means of facades, all having high efficiency and multi-level use of energy by means of heat recuperative systems;
- air circulation and convection made by combining a passive and a dynamic system that could allow the reversibility according to season and external or internal climate factors, while generating natural ventilation systems;
- building some "green oases" to create an agreeable climate, close to nature, supplying the humidity necessary according to season and allowing a photosynthesis process that partially removes the CO_2 production;
- making use of natural materials (wood) that can be introduced back in the natural cycle, that can be reused or recycled;
- the proper partitions, by supplying atriums and using solar energy to optimise the lighting of the estate;
- the artificial lighting should be controlled and adjusted by a detector that measures the number of the building inhabitants.

The following will exemplify concretely the manner in which these principles have been put into practice in some buildings of the kind.

In the case of the *Commerzbank Building in Frankfurt* (erected in 1997), modular mobile windows with more degrees of freedom were inserted in order to establish an as natural as possible environment in the habitat area and to make use of the natural forces instead of mechanical energy. Thus, function of season, the direction of the outside winds and air currents, the appropriately oriented windows can produce a controlled gravitational ventilation of the areas of interest (Figures 1 and 2). It is obvious that natural ventilation coexists with and completes the climate.

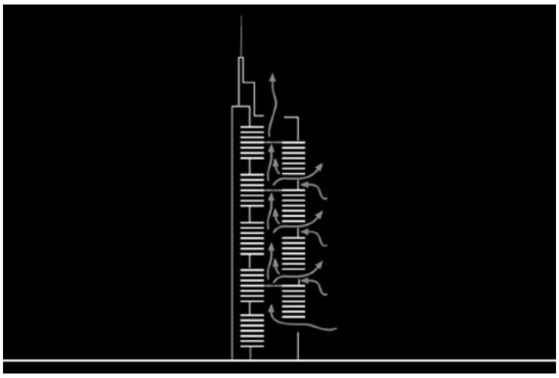

Fig.1. The natural ventilation of the building

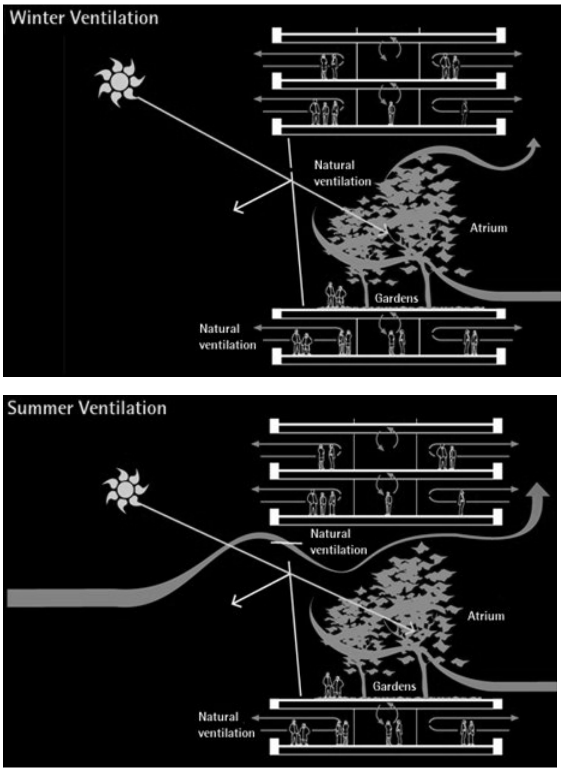

Fig.2. The air circulation during winter and summer conditions

The hot water produced in the air conditioning equipment is cooled down in an own cooling tower, that is in fact the highest level of the building and after pumping it from the bottom to the top, it weeps in thin flow towards the heat exchangers and then as a cold environment it is recirculated through the building floors. The nine gardens that cover four floors each are situated near the rooms dedicated to offices and they bring in natural life and light up to the most hidden corners. According to the British architect, these contribute to helping the people who generally live at a horizontal level at home to meet in a point on the vertical line of the tower while attracted by the green flowers: the Asian ones on the east wing, the mediterranean ones in the South , the North Americans in the west. The central area of the building is also naturally lighted by means of a glass and steel atrium.

In the case of the *London City Hall* (that was finished in 2002) a high performance façade was erected to contribute to the offices heating, cooling, and ventilation respectively. The most important part of the building cover in the offices area is a triply glazed cladding system provided with an externally ventilated cavity (Figure 3). This protection system diminishes the heat gain and the heat loss respectively, to very low levels.

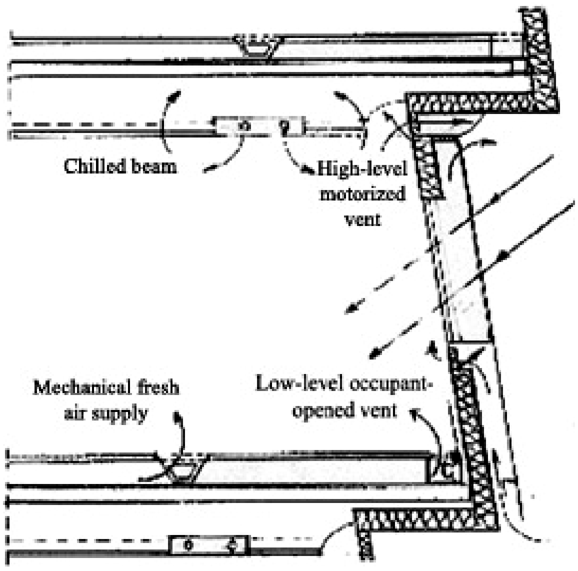

Figure 3. Detail of the façade ventilation system

Each panel is 1.5 m wide and exhibits a unique geometry due to the shape of the building. The inner panel combines an insulated spandrel with a doubly glazed member. The blinds are located in the cavity.

The outer member of the façade opens in such as way as to provide natural ventilation, weather permitting. Both the blinds and the façade can be controlled by the occupants of the building. Under the window units there are holes that can provide natural ventilation.

The fresh air is replaced at each floor by a plenum that is situated in the raised floor. Cooling is provided at each level by chilled beams that are incorporated in the ceilings, the beams being the place of cold water circulation. Cooling is supplied by groundwater extracted from deep boreholes that extend up to the chalk bedrock. In this manner, the rooms cooling is satisfied.

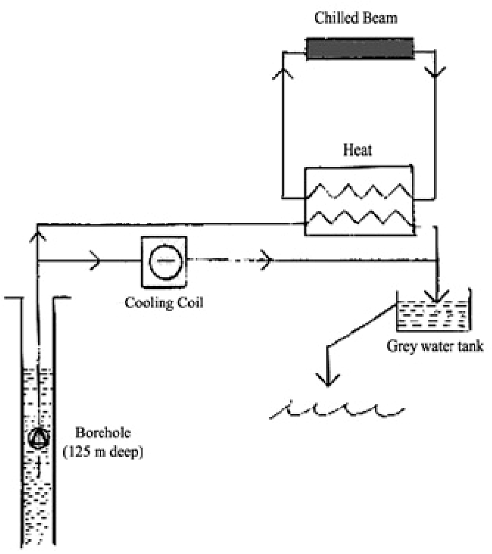

Figure 4. The diagram of the cooling system

The designers wanted to get an interactive building in which structural members and the ones providing thermal comfort should work together and form a unitary at the same time. As an example, an challenging solution was given to ensure the horizontal stiffness of the structure at the third level. The steel ductwork act as heating ducts at the same time, so that the lattice structure takes over the radiator role too, counteracting the cold air in the atrium.

The challenging shape of modified sphere of the building – that incorporates the largest volume with the smallest outer surface and a southward orientation of the building – to support its own shadow contributes also to the achievement of a record in so far the energy consumption is concerned where thermal comfort is provided. This record is of 25% of the consumption of a similar sized building.

The Citygate Eco Tower of London is supplied with wind turbines and voltaic cells incorporated in the facades to generate electricity.

These concrete examples can lead to the conclusion that each building has its own personality, that in spite of having the same objectives, is achieved by specific means related to the own potential of the site in which the constructions are situated. Following this analysis it can be stated that a blending between saving and energy is possible.

3. Conclusions:

The notion of ecologically-responsive building has gained more and more followers among engineers and public due to two basic reasons:

- the substantial reduction of the energy consumption and implicitly of the current maintenance costs;
- the setting up of a pleasant climate, close to the nature, as opposed to the artificial, public allergy and disease generator climate;

The building of such constructions requires a perfect co-operation between the engineers dealing with structural, functional and equipping aspects and also between technicians and customers, whose priorities, needs and social cultural standards are different. In a millennium in which the green movement has taken a totally motivated swing, the ecologically-responsive buildings will replace the high energy consuming and pollution producing buildings in order to reduce the negative effect upon the biosphere.

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Groundwater Resources Management

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Rezumat: Lucrarea se referă la managementul apei subterane din acvifer. Obiectivul managementului apei subterane, doar cu o mică aplicabilitate la apa de suprafață, este de a dezvolta strategii operaționale în idea de a suplimenta rezerva de apă de suprafață cu apă extrasă din subteran. Aceste strategii sunt necesare pentru asigurarea necesaului de apă din irigații şi alte activități, păstrând în permanenîă în vedere calitatea apei subterane.

Abstract: This study deals with the optimal management of groundwater in aquifer systems. The objective of groundwater management, with only a moderate availability of surface water, is to evolve appropriate operational strategies to supplement the surface water with the groundwater to meet the water demand for irrigation and other uses, while controlling the aquifer quality. Excessive groundwater extraction in some regions, particularly in the lower plains, may lead to significant water intrusion.

Keywords: groundwater, pollution, flow modelling, remediation of groundwater quality, medium contaminated.

1. Introduction

The water resources are a strategic, vital natural resources having interstate importance. Romania has an important resource of underground and ground waters, the significant stocks of which are in the rivers and snow massifs.

There is a significant amount of lakes and other natural reservoirs on the territory of our country, an important percent of lakes being located in high regions of tectonic origin.

A significant part of taken away waters is lost during their use. The reason consists in unsatisfactory technical condition of irrigation and water-distributive systems, wear of equipment, application of imperfect methods for watering, absence water-saving technologies. In the last years, the stable tendency of growth of unproductive losses of water is marked, and most of them are losses in irrigative branch.

Water is divided on household, industrial and agricultural use.

The greatest alarm causes technogenic pollution of water resources.

2. How to threat the quality of groundwater

Most groundwater originates from rainfall that has entered the earth. The drawing shows a typical situation with water-saturated soils (overburden aquifer) over a bedrock aquifer. In the overburden aquifer, water fills the void space between grains of the soil. Bedrock aquifers underlie the surface soils (overburden) and overburden aquifers. In the bedrock aquifers, water occurs in fractures and other voids in the bedrock.

Some types of bedrock such as sandstone may also have additional voids (intergranular voids) that are filled with groundwater.

As is the case for surface water, groundwater flows from higher elevations (or pressures) toward lower elevations (or lower pressures). Groundwater flow is usually toward a groundwater discharge area, as shown in the figure 1. The stream in the drawing represents a typical groundwater discharge area. Groundwater pressure, rather than elevation, controls the rate and direction of flow in confined (or artesian) aquifers. Those are aquifers that are isolated under impervious or poorly pervious strata (aquicludes and aquitards).

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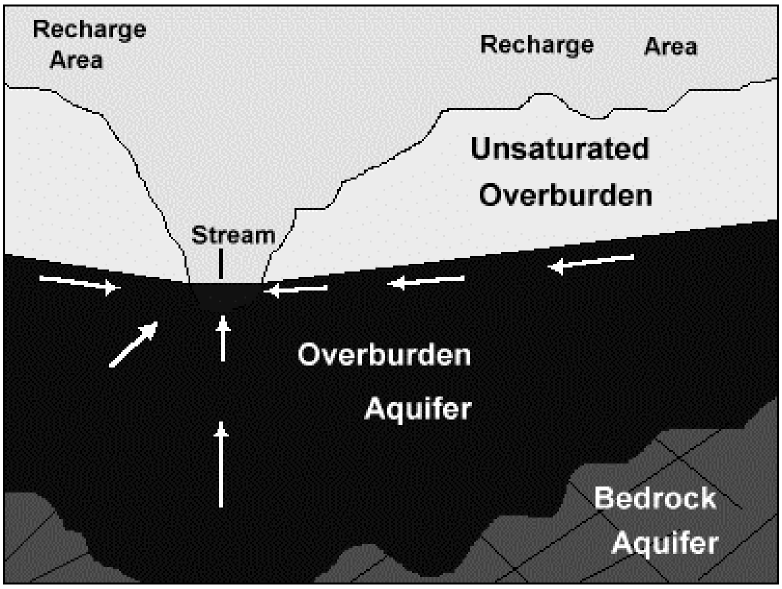

Fig.1: A typical groundwater discharge area.

Every year, an important quantity of various drains is removed in surface water objects of the country, and from them big quantities of wastewaters passes biological, physico-chemical or mechanical clearing. Every year, dangerously polluted wastewaters are faulted without clearing in open water reservoir and channels. The content of harmful substances in them exceed the established norms (bacteriological and physico-chemical norms).

An appearance of organic pollution, petroleum, phenols and other harmful substances in water objects is connected to inefficient clearing of urban drains, drains of the enterprises of meat, food and local industry, leather-processing and agricultural manufacture, motor transportation enterprises.

Besides, an important percent from a general number of various complexes of clearing structures are in an unsatisfactory technical condition and do not provide of effective clearing of waste.

All this together creates potential ecological danger for surface and underground of water of the country in future.

Waters of basins of the rivers are subject to pollution. An increased content of ammonium and nitrite nitrogen, compounds of zinc, petroleum, and organic and other harmful substances are systematically observed here.

An amount of drinking water (for household needs) and a large part of water for industrial needs are supplied from underground sources. A threat to quality of underground water goes on the part of pollution of the top part of water-bearing layers.

The basic reasons of pollution of underground waters are:

- location of objects of stock-breeding in zones of sanitary water protection, development of irrigated agriculture, a bad sanitary condition of populated regions, an absence of systems of water supply and water drain;
- location of numerous heaps and tail storages of waste of mining enterprises in intermountain troughs and hollows, flood plains of rivers, containing radioactive substances, salts of heavy metals, cyanogen-contained substances.

Examples of pollution of underground waters:

- increase of concentration of nitrates up to depths of 150 m in the region where water supply ensure the drinking water;

- pollution by nitrates and manganese because of outflow in the last of polluted industrial drains from tail storages of mining plants;
- increase of mineralisation and concentration of chlorates and sulphates in the regions of the gold extracting.

Pollution by nitrates, petroleum, and poison chemical substances are also marked. Thus, increase of objects of outbreaks of polluting substances in environment, unsatisfactory storage, processing, utilization of industrial and household wastes, a low culture of agricultural manufacture, have resulted in local pollution of open water reservoirs and underground waters of our country.

The subterranean realm is underground everywhere and most of us are totally oblivious to its existence. In order to know what exists below the surface of the earth, subsurface exploration of one type or another must be done. The figure 2 shows one type of subsurface exploration, test drilling. During drilling, samples of the earth may be brought to the surface for us to observe. After enough test drilling has been completed, the structure of the subsurface can be reconstructed. The borings can then be converted into wells by installing pipe with slots or holes, or a screen section to permit water to enter. Then the depth to water can be measured and groundwater samples can be collected for analysis to investigate flow direction, the composition of the water, and if it contains contaminants.

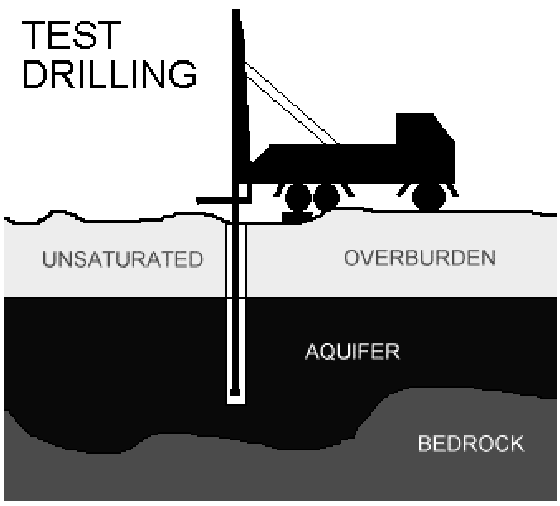

Fig.2: Test drilling (subsurface exploration).

There are other methods that may be used to explore the subterranean realm. They include excavation and mining, cavern entry and exploration, and indirect methods that may involve geophysical techniques.

3. Groundwater Monitoring and Flow Modelling

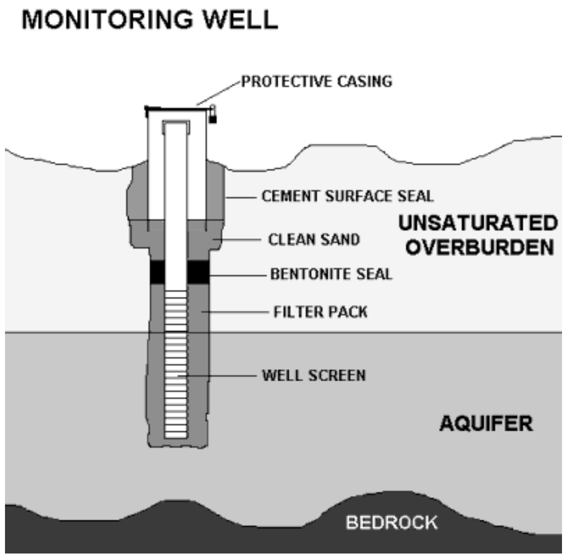

Fig.3: A shallow overburden aquifer monitoring well and some of its basic features.

Groundwater monitoring is performed in a number of situations with varying objectives. It involves measuring the physical and/or chemical properties of groundwater on a periodic basis.
Concentrations of contaminants are frequently monitored to determine if they are increasing, decreasing, or remaining in the same range. Monitoring is also performed at and in the vicinity of water supply sources to determine the quality of the water and trends of indicators of water quality.

Groundwater monitoring programs normally involve an array of monitoring or observation wells.

The figure 3 indicates a shallow overburden aquifer monitoring well and some of its basic features. A well of this type makes it possible to measure the groundwater elevation and permits water sampling to test the composition of the groundwater.

Groundwater flow modeling is generally used to define the quantity of groundwater available or direction of dissolved contaminant migration. It is also used to define the limits of a capture zone for a contamination recovery well (or well field), or for delineating a water well protection area (or recharge area) for a water supply. Groundwater scientists frequently use analytical means to model these situations using the classical mathematical formulas to estimate the effect on the groundwater surface. In order to estimate the long-term yield and water level draw down of a recovery or water supply well (or well field) they often use the classical formulas for making projections [1].

Modeling by manual calculations (analytical modeling) can be very time-consuming and, therefore expensive. That is why, groundwater scientists have prepared a number of computer groundwater modeling programs that allow for a more rapid and efficient assessment of groundwater flow under conditions that may involve the addition of simulated wells and/or simulated sources of recharge in an existing flow field.

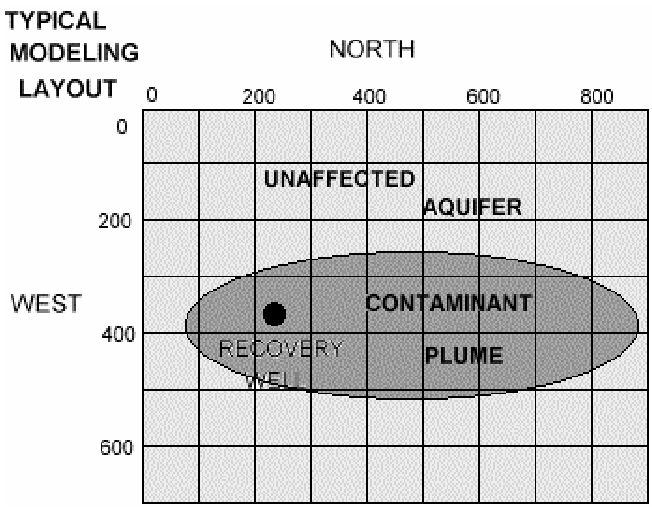

Fig.4: Computer groundwater modeling.

The models often generate contour maps that illustrate relevant data that are related to groundwater flow (fig.4). The computer output is usually plotted as groundwater elevation (or artesian pressure) maps for further analysis.

4. Groundwater Quality and Remediation

Groundwater quality reflects substances that are dissolved or suspended in the water. Suspended material is not transported far in most subsurface materials, but it is usually filtered out. In general, groundwater flow is very slow and depends on the permeability (water transmitting ability) of the subsurface materials, as well as the hydraulic gradient (slope of the water-table or pressure gradient for artesian conditions). The rate of groundwater flow is usually measured in meters per day or meters per year. In some situations where flow is slow it is measured in centimeters per year.

Groundwater usually contains higher concentrations of natural dissolved materials than surface water. The materials dissolved in the water usually reflect the composition and solubility of the earth materials (soil or rock) that the groundwater is in contact with and time that it has been in the subsurface. A number of the activities of man pose threats to water quality. Some of these activities include:

- Landfill solid waste disposal
- Liquid waste disposal basins
- Septic waste infiltration systems
- Highway deicing with chemicals (eg. salt)
- Gasoline service stations
- Petroleum bulk storage facilities
- Underground storage tanks

- Many industrial activities
- Livestock feed lots
- Urban storm water infiltration

A source of groundwater contamination can pollute millions of liters of groundwater in an underlying aquifer. The "industrial area" is presented as a typical source of contamination.

Usually, the groundwater contaminants are volatile organic chemicals that are used as solvents or degreasers in various industrial processes. The degree of contamination is indicated by the concentration of total volatile organics by contours. Degree of contamination is also illustrated by varying colors in the aquifer in figure 5.

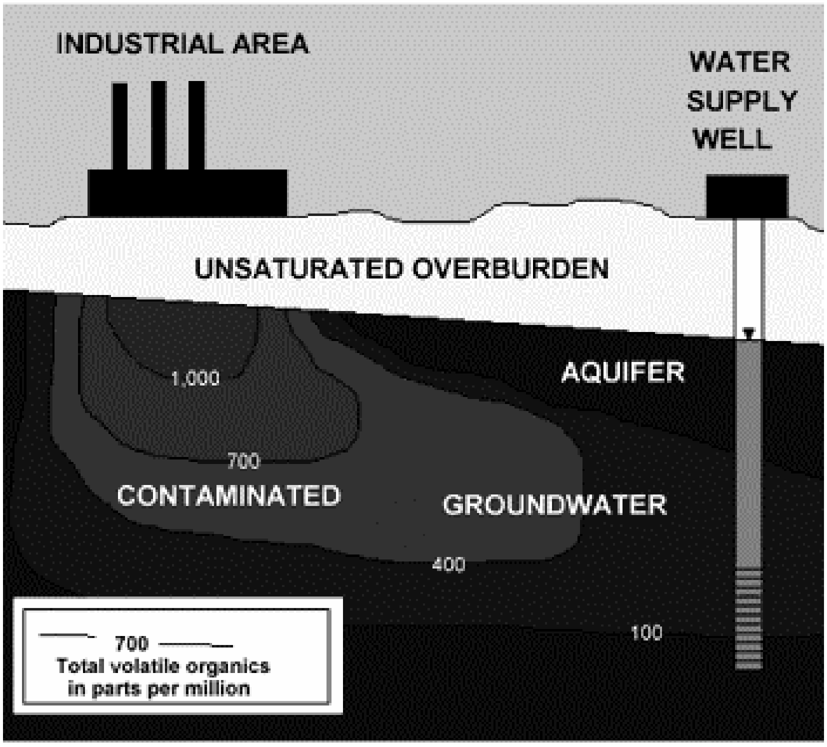

Fig.5: Water supply well.

Those closest to the industrial area have the highest concentrations, while those at a distance or up gradient have lower concentrations. The down gradient water supply well is being impacted by the contamination from the industrial area and may have to be shut down until the aquifer is cleaned up.

Groundwater scientists have used a number of different computer models to evaluate the transport of various dissolved organic and inorganic compounds in groundwater in a number of geologic situations.

One frequently encountered modeling situation involves the assessment of the distribution of hydrocarbon concentrations around a contaminant source to evaluate various remedial scenarios including removal of the source of contamination, concentration reductions resulting from consumption by microbes, effect of groundwater recovery wells, and recharge sources. The illustration shows a simple grid layout for solute transport modeling. Such models can be used to predict the time required for aquifer cleanup or for natural concentration reductions by existing processes in the subsurface.

The figure 6 shows a schematic drawing for a treatment system for removing petroleum (gasoline or oil) components from groundwater.

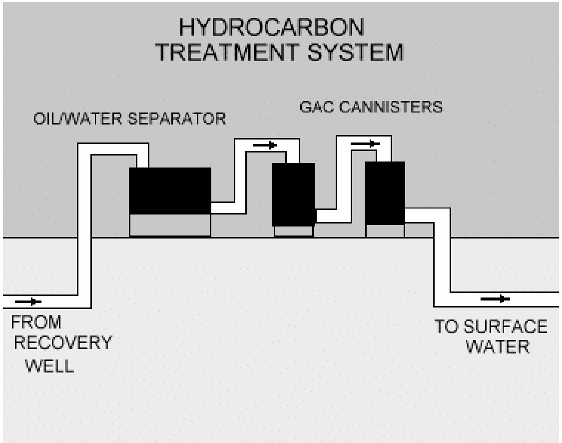

Fig.6: Treatment system for removing petroleum components from groundwater.

If groundwater contamination is identified on a site, and if contaminant concentrations are found above regulatory limits, remedial activities or feasibility studies must be done just to keep a site in compliance [2].

Such activities vary with the contaminant, medium that is contaminated, and surrounding environmental factors.

Common remedial methods include the following:

- Excavation and offsite removal.
- Excavation and onsite treatment.
- Groundwater "pump and treat".
- Soil vapor extraction.
- Passive recovery of non-aqueousphase liquids (NAPL).
- Enhanced bioremediation.
- Onsite encapsulation.
- In-situ onsite treatment.

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The Importance of Hydrogeological Analyses of Groundwater Behavior in the Slope Stability Analyses

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Rezumat: Analiza apei subterane a arătat că modificarea cantității de apă conduce la modificarea calității apei. Lucrarea analizează apa subterană din punct de vedere al aspectelor hidrogeologice, punând accent pe activitățile hidrogeologice şi pe metodele de determinare a cantității de apă subterană în acvifer. Aceste analize sunt punctul de pornire pentru modelarea geologică a stabilității versanților, pentru aplicarea unor măsuri optime de protecție.

Abstract: The analysis of groundwater quality showed that change in water quantity caused the variation of water quality. This paper analyzes groundwater system respecting to hydrogeological aspects and puts an emphasis on the hydrogeological activities and methodology of a reliable, quality determination of the groundwater regime in the landslide body as a basis for geotechnical modeling of slope stability, in order to prepare optimum measures of protection. According to the geological properties, aquifers may be simplified as a one-layer aquifer model. A two dimensional model is adopted to simulate the fluctuation of groundwater level.

Keywords: *groundwater, hydrogeological characteristics, aquifer model, fluctuation of groundwater level.*

1. Introduction

Groundwater management is a very important subject in the environmental field. Many unstable slopes with active or potentially active slides are limiting factors in urban planning, and urban, traffic and other development on the riverbank in Romania. Because of complex lithological structures, geotectonic, erosion and other current geodynamical processes in the area, a multidisciplinary approach to investigations is required and particularly needed for any local remedial or protection measures. The numerous papers on engineering-geological investigations on unstable slopes clearly show that we have not enough experience in the determination of groundwater table fluctuations, although its presence is one of the key active factors in slope stability [1].

Generally properties of groundwater depend on regional influence. Water quality and quantity are not only affected by the recharge and soil utilization, but also by the surrounding geological conditions that is considerable as the reservoir of groundwater [2].

This study analyzes the groundwater system respecting to hydrogeological respects.

For the purpose of determining geotechnical parameters for local land development, vineyard raising, construction of access communications, subsidiary buildings, an optimum irrigation system and drainage systems in the overhumid zones, a program of specific hydrogeological and engineering geological investigations, in the laboratory and in the field, must be first defined and then implemented [3]. The main approach is to first identify all ground instability bearing in mind planned use, and then to briefly refer to the groundwater regime in surrounding of the landslide.

In the final phase the ground stability must to be modeled using different groundwater tables and protection measures of the level of a detailed design were to be planned in order to arrange and develop the land.

2. Geological Conditions

Let's consider an alluvial deposit plain surrounded by high areas in three directions and by a

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lake in one direction. Though the deposit plain flows a river that finally enters into the sea [4].
Generally groundwater and hydrogeology of each area has its own regional characteristics. Different geological conditions and recharge rates induce different chemical component of groundwater. We consider that the groundwater in the alluvial deposit plain is diluted fossil water that formed from the pore water (lake water) in the sedimentation period. It is considerable that the recharge resource is seepage of rainwater and river water from high sites.

We consider that the area of our study lies under plantations of trees, vineyards and auxiliary buildings and communications and that the area extends at about 180 – 200 m above sea level.

In order to make a comprehensive review of the soil engineering geological and hydrogeological characteristics, a broader district must be investigated. Let's suppose that, with regard to geomorphology, the investigated area lies on a relatively gentle slope. The stretch below the southern boundary of the area, at the riverside (on which private housing projects and other buildings have been built) is much steeper and shows evident scars of active sliding (toe of the landslide).

The plain surface has extensive distribution of the very soft and sensitive clay. We suppose that we found that the thickness of clay layer is generally about 10 to 20 meters and somewhere with a maximum value of 30 meters. Underneath parts are the unconsolidated sedimentary layers of which was mainly mingled of sand and mud.

After a preliminary study of the basic geological documents, the following study and investigations must be first designed and then realized:

- Study of the available documents in order to define the programme and schedule of fieldwork.
- The fieldwork included:

a) Engineering geological ground mapping, an expert inspection of the buildings (detailed in the basement) and verification of the effects of the remedial measures by taking records of the existing deformations and planning measures for long term protection and preservation.
b) Detailed hydrogeological mapping, a questionnaire on the use and yield of all the existing wells, preparation of a cadastre of observation water structures. For the selected wells, tests must be carried out in order to identify filtering parameters of captured aquifers and set water intake rules, regulate the groundwater levels and specify ground remedial measures and requirements for future drainage and irrigation systems. At all registered water structures a systematic observation of the groundwater table fluctuations and daily observation of the amount of precipitation on the farm must be started.
c) The land was surveyed in order to ascertain the extent of soil movement over the past years.

- The laboratory work includes:

a) A detailed study of the past investigations, an engineering geological map, an engineering geological profile and detailed hydrogeological maps pinpointing hydro-contours of the observed ground water levels.

We suppose that the results of past investigations state that the ground in the building area is in a so-called tranquilized state and that any possible future disturbance of its stability may be associated with the sliding plane which lies in the first phreatic aquifer floor and that any extremely high groundwater rise in this aquifer can activate the whole landslide or some of its parts. For this reason additional hydrogeological investigations and quantification of groundwater regime and balance must be started in the wide surrounds of the investigated area.
b) Hydrogeological and hydrodynamic studies of groundwater fluctuations in the determined aquifer environments, particularly in the first phreatic aquifer and correlation with the observed daily amounts of "in situ" precipitation this becoming the base for a geotechnical analysis of soil stability.

A detailed analysis of the available documentation and additional hydrogeological investigations can pointed three characteristic aquifers (fig.2):

- The "first phreatic aquifer" located in the surface and subsurface complexes in loessoid sands. Replenishing conditions in the first aquifer are directly related with vertical balance parameters as well as with inflows from the background.
- The free level phreatic aquifer is formed in the complex of the so-called roof sands. Water is replenished at the expense of infiltration from the "first aquifer" in shallow subsurface complexes and particularly from the loess deposits in the background. The aquifers are drained, in addition to gravity percolation towards the river stream.

A sub-artesian aquifer is formed within the so-called floor sands. This aquifer is replenished with infiltrated water from the shallow aquifers in the wide investigated area. The aquifer is drained practically in the whole wide investigated area.

Continuous systematic observation of groundwater table in all three aquifers must be performed on water structures with simultaneous daily recording of the amount of precipitation. Groundwater observations must be so programmed that the retardation period and the infiltration rate particularly in the zone of the "first aquifer" to be determined with the highest possible reliability after any intense precipitation.

3. Aquifer Model and Simulation of the Fluctuation of Groundwater Level

Data from pumping in selected wells were analyzed on a full hydrogram, using original hydrodynamic software programme for pumping data processing and analyzing the values of the transitivity coefficient for the "first aquifer" and effective porosity. On the other hand, a preliminary hydrodynamic model must be formed to analyze primary and basic hydrogeological prerequisites for selecting a drainage system in order to monitor and control groundwater table on the basis of the identified filtration parameters of the "first aquifer".

There are different pumping purpose and amount [5]. If an important percent of pumping is for industry, the variation of pumping amount is comparatively small. If an important percent of pumping is for agriculture, the seasonal variation of pumping amount is large. During summer, increasing amount for irrigation water resulted in the reduction of aquifer pressure and the groundwater level draws down, then induced to the land subsidence. This kind of pumping is difficult to control. During winter, due to the decrease of water use, the water level returned. Even though the land subsidence became small and land elevation showed a return tendency in this period, the land could not be restored to the former state. From observed data, it is found that land subsidence is remarkable when pumping amount is large.

Extensive hydrogeological investigations must be conducted for a reliable separation of characteristic aquifer complexes down to the investigated depth, followed by a specific quantitative analysis of the "first aquifer" groundwater table. The aim is to determine conditions for possible active monitoring and control of the "first aquifer" groundwater table in particular for the purpose of preserving the existing conditional ground stability (by appropriate geotechnical measures of protection) on the one hand and design optimum and rational hydrotechnical amelioration measures for ground development and raising of new plantations in the area on the other hand.

A six-month analysis of fluctuations of the first aquifer groundwater table indicated risky sudden rising of the table after abundant precipitation. Due to this, a hydrogeological methodology was established for an approach to a concrete targeted geotechnical analysis of soil stability. With regard to earlier landslide engineering geological investigations and hydrogeological properties of the alluvial complex in the first aquifer, conditions were considered for preserving ground stability by maintaining groundwater table at the given optimum depth. A quantitative geotechnical analysis of slope stability must be done, first of all, in the narrow zone of the investigated area from which the toe of the landslide was excluded. The method of calculation depends on the type of the slide planes that are considered to the depths of first aquifer groundwater table.

In addition to the hydrogeological parameters of the existing water structures in the wide surrounds of the area, may appear a need to adopt values for the physical mechanical parameters of the lithological environment derived from preliminary investigations. The standard for an analysis of the impact of groundwater table fluctuations on stability is the limit state of the balance Fs=1 and the average observed groundwater levels in the period.

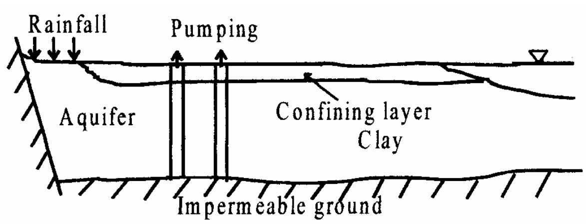

Figure 1: Aquifer model of the deposit plain

This paper simulated the fluctuation of groundwater level in order to master the groundwater variation due to pumping in quantity aspect. According to the geological property of the model, the aquifers are simplified as a one-layer aquifer model (fig.1). The upper impermeable clay layer is considered as a confining layer and the strata underneath is considered as a unit confined aquifer.

We consider that the aquifer thickness is much less than the extension in horizontal direction. That is why it is reasonable to neglect the groundwater flow in vertical direction. Based on the Darcy's Law and continuity principle, following differential equation was used to describe the groundwater flow:

$$T\ \delta^2 h/\delta x^2 + T\ \delta^2 h/\delta^2 = S\ \delta h/\delta t + Q(x,\ y,\ t,) \qquad (1)$$

where: T is the transmissibility of aquifer;

S is the storage coefficient;

Q is the net recharge;

h is the groundwater level.

Because three boundaries are high areas and the thickness of clay there is zero, it is reasonable to think the aquifer is unconfined in two of the boundaries. Then the high area is assumed a recharge source of groundwater. In addition, we consider that the groundwater levels there are about 0 meter and did not vary significantly with time. Therefore the boundary condition there is fixed in 0 meter. In the third boundary, we consider that the aquifer is connected to outside. Due to fluctuating flows, water level is fixed as initial value. The fourth boundary is the sea, and the groundwater level there is considered not affected by pumping, and the water level is fixed at sea level.

Finite differential method was adopted in the simulation and the comparison of calculated results with the observed record results shows that the one-layer two-dimensional plane model seems to fit well with actual observations of alluvial deposit plain. This model was able to extend the prediction of groundwater level fluctuation due to pumping.

4. Conclusions

The groundwater in alluvial deposit plain is seriously affected by over-pumping. Groundwater level varied seasonally, by pumping amount and precipitation.

One-layer two-dimensional plane model matched well with actual observations of alluvial deposit plain. It could be extended to predict the fluctuation of groundwater level, and give a reference to guide pumping.

Because of over-pumping in alluvial deposit plain, the groundwater in shallow aquifer and medium aquifer are polluted by water intrusion to a different degree. The pollution in the area near the lake is more serious than that in the inland area. It is remarkable as the draw down due to over - pumping in shallow aquifer and medium aquifer induced to old water seepage from deep aquifer.

The groundwater in such area may be affected more strongly by fossil water than lake water or river water. In order to utilize and protect groundwater resources, more detail observations and study about the groundwater system are necessary in the future.

On the basis of a stability analysis and with a view to all preliminary geological, hydrogeological and hydrodynamic observations the following conclusions and recommendations have been made [6]:

- With regard to the latent risk that the existing natural state of balance may be disturbed on the whole analyzed slope, any inappropriate big cutting of the ground, say for example for making terraces for vine rows, for local roads, for local building construction, then landfills on some sections or humus topsoil redistribution may contribute to disturbing the balance and may intensify the sliding process depending on the current groundwater table. More extensive changes in the slope morphology will imminently bring about changes in the natural groundwater table in the treated sections and on the whole grounds.
- Since lowering of the first aquifer groundwater table from the existing average values and its maintenance on that average value will lead to satisfactory stability of the analyzed slope (one of rational measures of protection) it will be indispensable to define by appropriate quantitative hydrodynamic analysis, the most optimum technical solution for ground drainage [7].

- In order to arrive at a reliable definition of the groundwater regime it will be necessary to continue observations for at least one hydrological year andcompare results with the vertical balance parameters. Simultaneously with groundwater table observations it will be necessary to measure both active and incidental physical movements of ground on the survey network and conduct an adequate periodical analysis of stability particularly checking the effects of extreme groundwater rises in spring.

- The ground stability in the narrow zone of the building area requires a specific detailed analysis of the existing drainage system around and under the houses. The groundwater table observations in the water bodies close to the building area may show that the drainage system is shallow and most probably sealed.

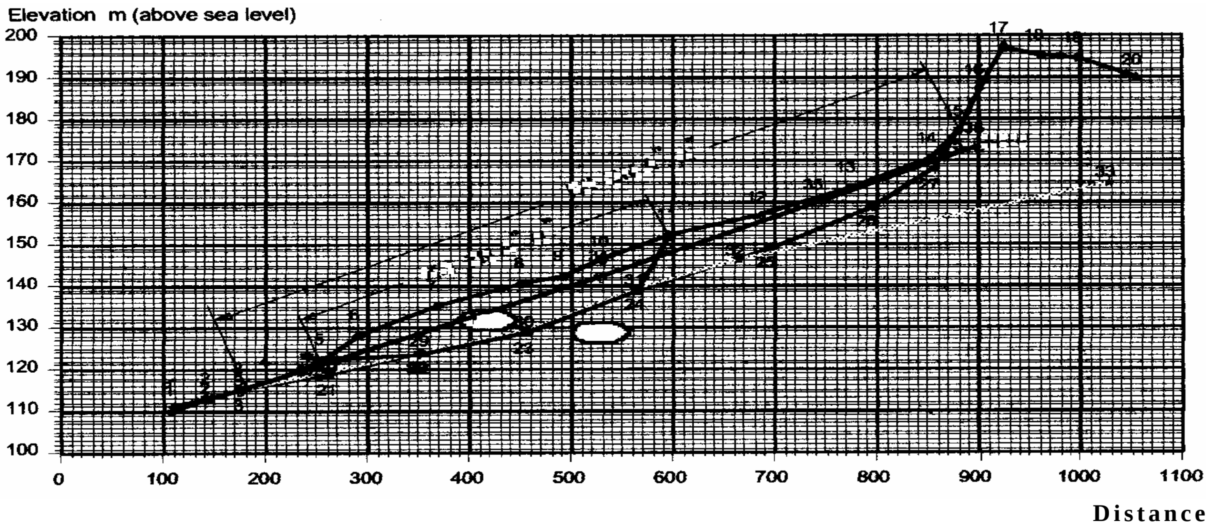

Figure 2: Analysis of the slope stability along two selected slide planes.

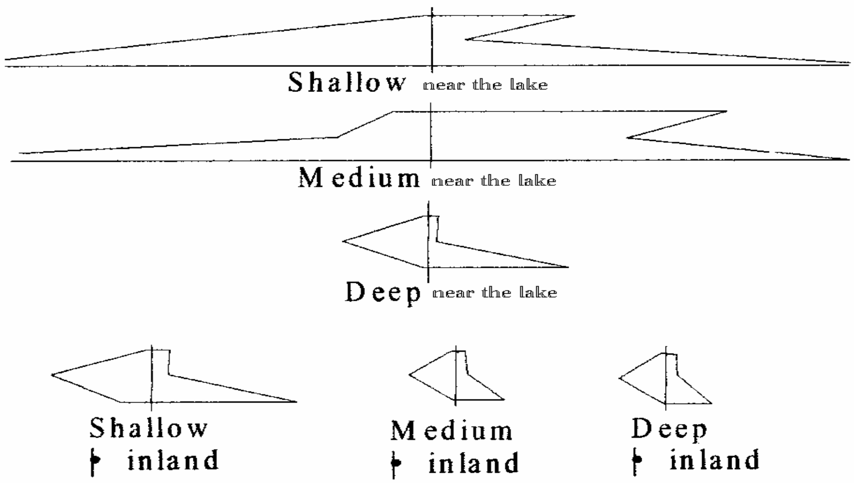

Figure 3: The representative diagrams of water samples in different depth and different locations.

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