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Seismics-2014

„ Seismic resistance and rehabilitation of buildings“

29-30 May 2014

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29-30 May 2014, Tbilisi, Georgia

DEVELOPMENT OF LOW-RISE ENERGY-EFFICIENT CONSTRUCTION IN UKRAINE

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Abstract: *The research presents the analysis of the factors influencing the positive cost-effectiveness of environmental low-rise building shows that affordable housing projects of this type must meet the following criteria as simplicity of design solutions, short installation time, good thermal characteristics.*

Keywords: energy-efficient construction, low-rise housing, local materials, eco-house

Introduction. Statement of the problem. Providing affordable and quality public housing that complies with the principles of contemporary world politics of sustainable development is an important social strategic task that is relevant for Ukraine. For solving this problem it is necessary to implement energy efficient techniques economically justifiable in mass construction with use of environmentally friendly materials in the design of low-rise residential buildings.

Analysis of publications. The question of study of alternative technology in design of low-rise buildings and sustainable housing design – are studied by local scientists [1], [2] and by many others. Study designs with ecological materials studied by foreign scholars [4] and others.

However, there is no work aimed at analyzing and studying the economic feasibility of using local ecological materials in low-rise construction in Ukraine.

The purpose of the article: the study and analysis of cost-effectiveness of using local ecological materials in the construction of affordable low-rise housing.

Statement of material. Environmental construction offers a wide range of materials, techniques and architectural solutions. The cost of this building is depends on the chosen variant

eco-house.

The main issues faced by the developer of ecological housing from local materials towards achieving economic efficiency of the project are:

• initial cost and time involved in developing **the project concept**, well -planned in the future it will ensure unnecessary losses during the construction and operation of housing;

• an important part - is **the choice of material**, which should provide the necessary structural strength, high thermal and economic performance of the project;

• it is advisable to use **simple design solutions** that save time and cost of installation works. Also very effective is the use of **energy-efficient engineering systems** and equipment, such , for example, as heat exchanger air. When choosing such systems should take into account their energy consumption and calculate the efficiency in each case [3];

• **flexibility of house to change** - easy with the necessary renovations or houses, with subsequent recycling of materials;

• Development of **phases of the project** will gradually integrate and implement more expensive energy-saving systems, after performing basic construction.

One of the important components of economic efficiency is the efficiency of capital investments. It is expressed by the ratio obtained effect to investments that caused this effect. In other words, it is an economic effect that per unit of investment, provided this effect.

The effectiveness of capital investments measured set of indices, which includes the overall effect of capital investment, rate of return, payback period, comparative effectiveness, and others. Indicators of economic efficiency of capital investments used to compare alternative investment projects and the selection of optimal project design is an important element of affordable low-rise housing.

For optimal results in finding the most favorable solution for the design of low-rise housing should take into account all major affordability criteria - environmental, social validity, and efficiency - both during construction and in the operation phase.

The combination of all these elements to select the most effective option for low-rise construction.

The resulting economic effect the use of technology in wood frame housing construction using local materials with reduced environmental affordable housing due to the following factors: short-term installation, ease of design elements, high thermal characteristics.

Cost-effectiveness of housing during it‘s operation can be achieved by reducing of the cost

of heating, i.e. reduction or reduction to zero specific heat loss of the building during the heating season.

When analyzing the cost-effectiveness must take into account all costs incurred in the life cycle of low-rise buildings - the engineering survey, design, construction (including conservation), maintenance (including running repairs), reconstruction , repair, demolition.

Particular attention in the design of energy-efficient affordable housing is given to the utilization of structures at the end of life, due to economic and environmental problems physically and demolition of obsolete housing existing today.

The cost of liquidation (demolition) of the building depends on the type of construction , type of material and its state , the degree of density of buildings in the considered area. For all these parameters using the low-rise building with wooden frame and wall insulation with local environmental materials is the most economical type of structures. Also the process of building this type of waste can be economically positive, thanks processing and briquetting wood waste and their subsequent use as a solid fuel used for heating and production of organic fertilizer.

Timber frame is the most common constructive bearing system that is used for the construction of housing type. The main question that arises - what material to use as insulation to fill the volume between pillars frame the exterior walls of the house. The concept of ecological construction is based on the use of local materials, which can significantly save on the cost of "green" homes.

To assess the economic feasibility of using local ecological materials in low-rise affordable housing the calculation of capital expenditures for the construction and elimination was made, as well as it was made calculation of operating costs for maintenance heating of ecological low-rise round separate building with total area of 91.8 m^2, with space of exterior walls 60 m^3. Architectural planning house features are shown in Fig. 1, a constructive solution external structure - Fig. 2.

The costs of construction, operation and demolition of buildings were calculated by the available rates for the current period (as of August 2013).

Capital expenditures (investment) for the construction of the building and execution by the i-th variant depends on the type of local material used as a filler of outer envelope. With the current for the summer 2013 average market prices of straw bales – 12 UAH. per bale - based on the size of the unit cost of 1 m3 of building materials wall with straw blocks is 47 UAH., on the dried stalk of straw cereals 480 UAH. 1t by considering light of adobe - the cost of building

materials in 1 m^3 wall of light is adobe 43 UAH ., hemp 1200 UAH. 1 t - cost of materials for construction of the wall 1 m^3 of filling the walls with hemp – 75 USD. The cost of operating vehicles , the cost of workers' wages have been calculated by ABK software complex rules and rates determine the cost of construction of wood-frame building with walls filled with wood-fiber material which could be related with pressed straw and hemp; and lightweight concrete which could be related with light adobe. Total cost of materials and estimated costs for salaries and operating machines in m2 building area is given in Table 2.

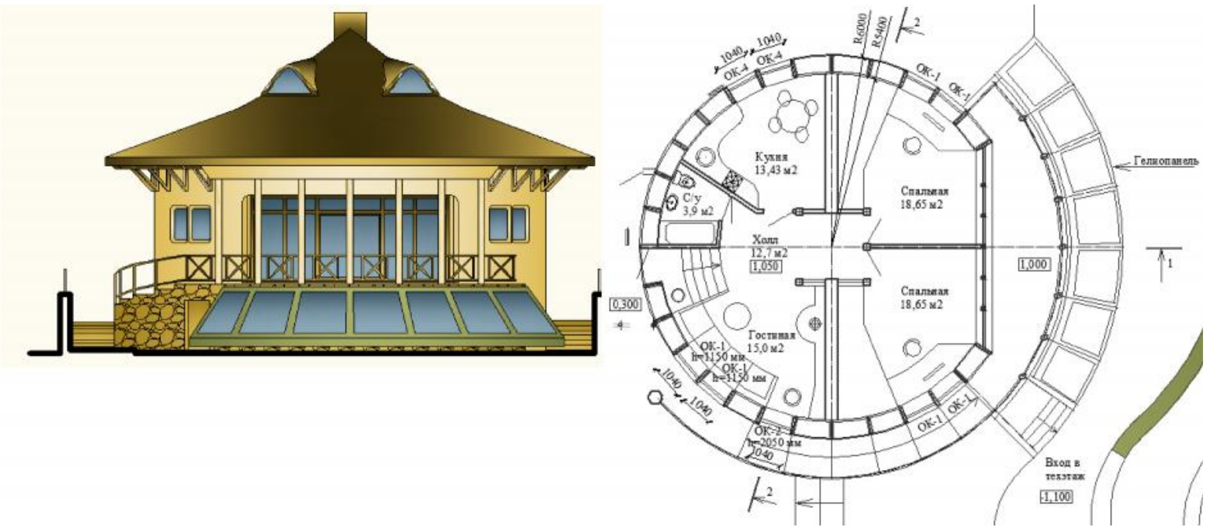

Fig.3. Architectural planning of eco-house

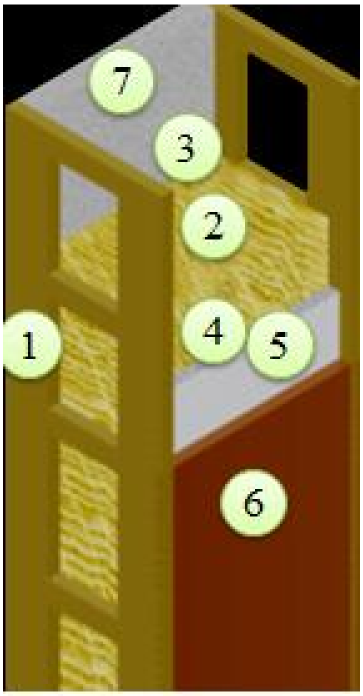

Fig.4. Constructive solutions of outer wall envelope :

1 - rack wooden frame element type "ladder" ,

2 - heater on environmental local materials (straw cereals , moderate or traditional adobe , blend compaction cutting hemp or reed , light hemp concrete), 500 mm;

3 - steam ,

4 - windscreen ,

5 – net,

6 - exterior decoration ,

7 – interior decoration

Capital expenditures for termination (demolition) construction of the building on the i-th variant was performed in the software AVK accordance with the rules and rates determine the cost of the elimination of a separate building with a wooden frame with preservation of suitable materials.

Annual operating costs of the building according to (1) consist of costs for heating buildings and carrying out repairs :

$$E_i = E_0 + E_r \tag{1}$$

where: E_o - the cost of home heating ,

E_r - cost of repairs of the building.

In district heating home heating costs are defined as follows:

$$E_0 = C_m \cdot Q_{god} \cdot k, \tag{2}$$

where C_m - price of heating;

Q_{god} - annual heat consumption for building during the heating period;

k - conversion factor from kW to Gcal.

The need for the introduction of transfer factor due to the fact that usually tariff for heat is installed in Gcal and building heat consumption is determined in kW.

Cost of 1 Gcal of heat for residential customers in Dnipropetrovsk region in August 2013 is 256 UAH.

Estimated annual costs of current and capital repairs are normalized according to [2] as follows:

$$E_r = E_{tek} + E_{kap}, \tag{3}$$

where: E_{tek} - the cost of maintenance;

E_{kap} - the cost of major repairs.

$$E_{tek} = 0.0075 K_i \tag{4}$$

$$E_{kap} = 0.017 K_i \tag{5}$$

According to (4), (5): K_i - the capital cost of construction of the house .

Given the fact that the repair work will be performed only for external walls , which is the same for all variants, capital expenditures for the current and capital repairs have been made for all variants – 69 UAH .

The calculation results are summarized to Table 1 and Table 2.

Table 1

Architectural and structural and thermal characteristics of environmental low-rise building with local materials

Variant of house	Material of wall insulation	Heat-conduction coefficient of insulation, W/(m • K)	Calculated values of specific losses of heat to heat your home during the heating period , kWh / m^2
A1	pressed straw cereals	0,05	107,8
A2	hemp	0.06	109
A3	lightweight adobe	0,084	111

Table 2

Characteristics of the economic efficiency of environmental affordable housing with the use of local materials

Variant of house	Capital costs for 1 m^2 of space		Annual operating cost of 1 m^2 of space	
	construction, UAH	liquidation, UAH	repairs, UAH	heating, UAH
A1	2835	368	69	21
A2	4050			22
A3	3645			23

Conclusions.

1. The analysis of the factors influencing the positive cost-effectiveness of environmental low-rise building shows that affordable housing projects of this type must meet the following criteria as simplicity of design solutions, short installation time, good thermal characteristics.

2. On the basis of the selected criteria was developed fundamental basic design solution outer wall enclosing construction of low-rise building with local environmental building materials.

3. The most economically feasible at present there during the construction of eco heater as the use of straw as a very cheap material.

4.Calculation fixed of costs of construction, operation and disposal of low-rise building with local environmental materials showed the economic feasibility of using this technology in the construction of affordable housing.

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29-30 May 2014, Tbilisi, Georgia

THE DECREASE OF SEISMIC FORCES FOR MULTISTORY REINFORCE CONCRETE SHEAR WALL-FRAME BUILDINGS WITH APPLICATION OF SEISMIC ISOLTAION

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Abstract: The efficiency of the application of seismic isolation laminated rubber bearings (SILRB) has been shown based on comparative analysis of multistory reinforce concrete shear wall-frame buildings are common used in the Republic of Armenia. Reinforce concrete shear wall-frame buildings with various stories have been considered in order to estimate the decrease of seismic forces in bearing structures. The dynamic analysis of systems has been carried out by application of software program based on the finite element method, with use of real accelerogram.

Key words: seismic isolators, earthquake action, accelerogram, reinforce concrete, shear wall-frame.

Seismic resistance of buildings and structures has primary importance for earthquake dangerous and densely populated regions, and the existence of multistory buildings increases the risk of numerous human losses. Currently, seismic isolation laminated rubber bearings provided effective damping of energy during the earthquakes, become widespread application in various countries to increase seismic resistance of buildings.

The construction of buildings and structures with the application of seismic isolation is mainly connected with the solution of the problems are related to decrease the earthquake loads. That is help to decrease the relative horizontal intermediate floors displacements (drifts), which in its turn decreases the amount of local destructions, economical losses, increases people's psychological comfort in case of different intensity earthquakes. All these factors have practical interest for potential investors and customers.

Seismic isolation is placed mostly between foundation and aboveground part of the building or construction dividing the bearing system into two parts. The lower part is connected with the foundation rigidly, whereas the upper part is located on the seismic isolators. Moreover,

two types of seismic isolation bearing systems are applied in Armenia according to structural solutions: the systems that are placed below and above the blind area of the building but were located not more than two floors above. The choice of the type of seismic isolation depends on the subsoil conditions as well as the functional purpose of the building.

In accordance with [1], seismic isolation is used for the buildings and structures with the main periods of natural vibrations are within 0.1 – 1.0 sec in case of the usual foundations (without seismic isolation) and not more than 3.0 sec for buildings with seismic isolation. According to the same building code [1] maximum number of stories for reinforce concrete buildings are located at the zone three where maximum estimated acceleration of the ground equals to 0.4g is limited to sixteen. Therefore, for this reason, multistory reinforce concrete shear wall-frame buildings with various stories (from 6 to 16) have been considered in the article.

About forty different buildings and structures were built in Armenia during last decades. Most of the seismic isolation laminated rubber bearings (fig. 1) used for them were made in Armenia.

a) b)

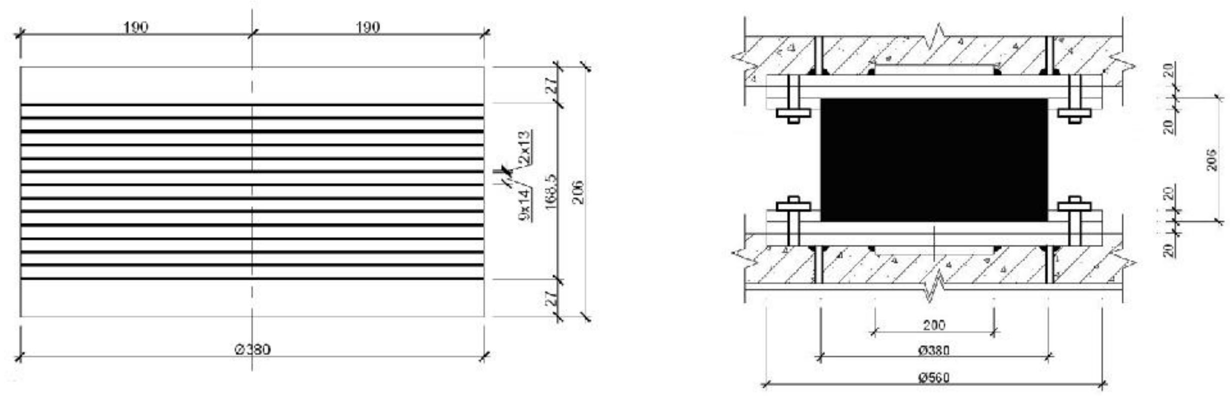

Fig.1. Seismic isolation laminated rubber bearings (SILRB) are applied in Armenia
a – general view with its dimensions, b – connection with reinforce concrete structures

Seismic isolation layered rubber bearings [2] are located at zero level of the buildings (between basement and ground floor) were used in the present article. Presented seismic isolators have the following geometrical and physical-mechanical characteristics: diameter is equal to 380 mm, the number of rubber layers is 14, the number of steel sheets is 13, thickness of metal sheets is 2 mm, thickness of rubber layers is 9 mm, height of seismic isolator is equal to 206 mm, horizontal stiffness of a seismic isolator is 0.81 kN/mm, vertical stiffness have not be less than

300 kN/mm, maximum permissible horizontal displacement is 280 mm, allowable axial (vertical) force is 1500 kN.

The number of seismic isolators for each building was determined based on the restrictions of horizontal displacement and normal force for seismic isolators. The quantity and the thickness of reinforce concrete diaphragms have been changed depending on the height of the building. Standard floor plans for 6 and 16 stories buildings are presented in fig. 2.

a) b)

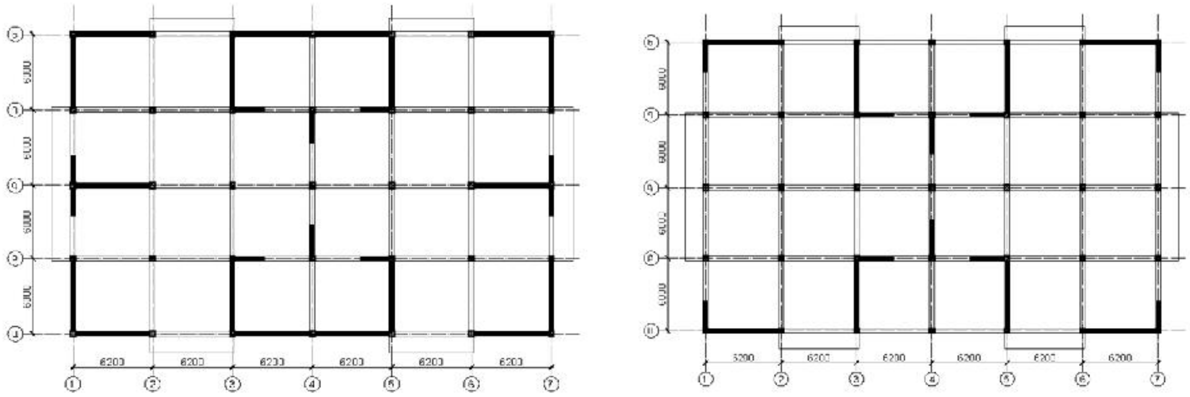

Fig. 2. Location schemes of reinforce concrete columns and shear walls

a – for 16 stories building, b – for 6 stories building

The location of seismic isolators in the plan and their quantity that is dependent on the stories of buildings are presented on fig.3 and table 1.

a) b)

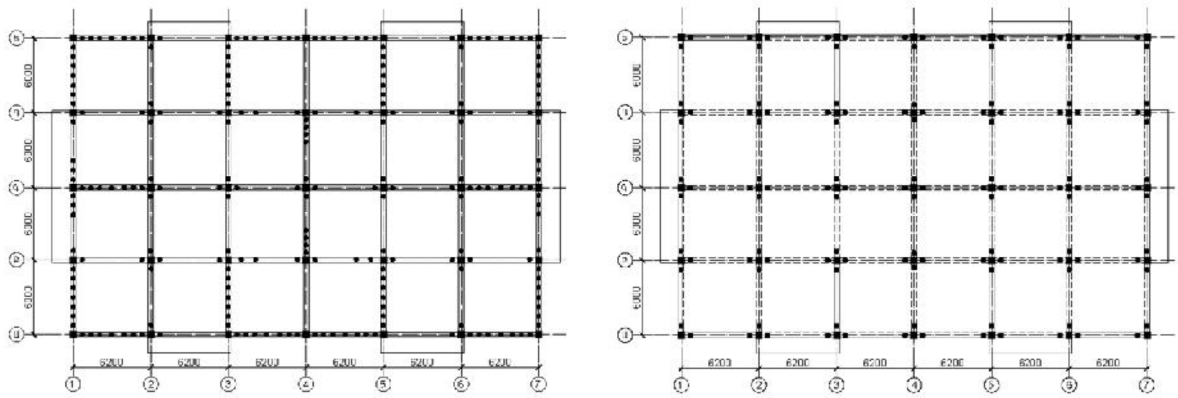

Fig 3. Location schemes of seismic isolation layered rubber bearings

a – for 16 stories building, b – for 6 stories building

The application of seismic isolators is quite effective [3, 4] for rigid buildings where the values of natural periods of vibration are small enough, because the building can be considered as a system with single degree of freedom (SDF), where the rigidness of the systems is determined by horizontal rigidness of seismic isolators. The effectiveness of seismic isolation layered rubber bearings are used in multistory buildings depends on the characteristics of the seismic isolators as well as the structures on the whole [5].

For the quantitative estimation, the effectiveness of seismic isolation layered rubber bearings in multistory reinforced concrete shear wall-frame buildings has been considered in terms of comparison between of the main parameters of the same buildings with and without seismic isolation.

Table1

Number of stories	6	9	11	14	16
Number of seismic isolation laminated rubber bearings	152	202	234	246	250

16, 14, 11, 9 and 6 storied buildings have been analyzed in the present article with and without seismic isolation layered rubber bearings. The calculations of the buildings have been carried out based on the current building standards of Armenia [1] as well as the application of accelerogram (fig. 4), where the acceleration of the ground reached 0.4g.

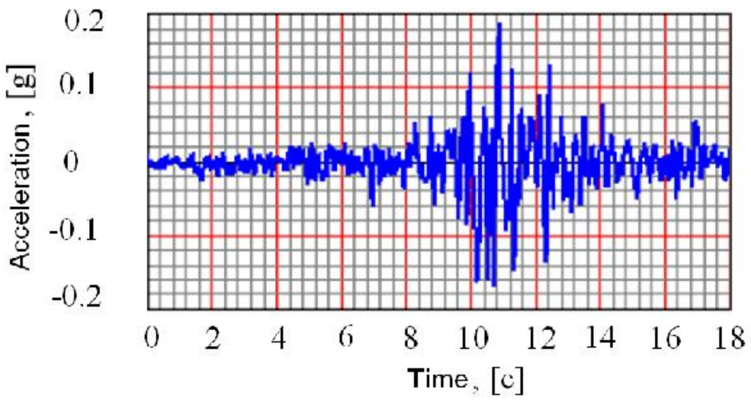

Fig. 4. Ghukasian record of Spitak earthquake (north-south)

The systems have been considered in the form of three-dimensional models. At the same time, calculations have been carried out for seismic zone 3 (A = 0.4), for soils of second category

(k_o = 1.0). The coefficient of allowable damages for reinforce concrete structures are located above the level of seismic isolation is accepted to be 0.4 and for the ones located below to be 0.8.

For all the cases with seismic isolation, the critical damping for seismic isolation layered rubber bearings is accepted to be equal 10%.

The results of the calculations are presented in the table 2, fig. 5 and fig. 6.

Table 2

The main characteristics of the buildings depend on considered direction

Characteristics and direction of the load		Periods of natural vibration and displacements of buildings and isolations depend on their stories									
		Building without seismic isolation					Building with seismic isolation				
		6	9	11	14	16	6	9	11	14	16
Period of natural vibration, sec	X	0.49	0.57	0.63	0.76	0.88	1.78	1.92	2.02	2.26	2.31
	Y	0.41	0.66	0.87	0.98	1.04	1.78	1.92	2.01	2.27	2.32
Maximum top displacement of buildings, mm	X	87.4	87.8	106	121	147	203	274	288	306	335
	Y	48.2	116	134	153	163	184	260	308	327	338
Maximum displacement of isolators, mm	X	-	-	-	-	-	171	205	193	201	223
	Y	-	-	-	-	-	156	198	209	213	221

The dependences of displacements are presented in fig. 5. Changes of inertial forces at different levels of systems depending on the number of stories are shown in fig. 6.

a) b)

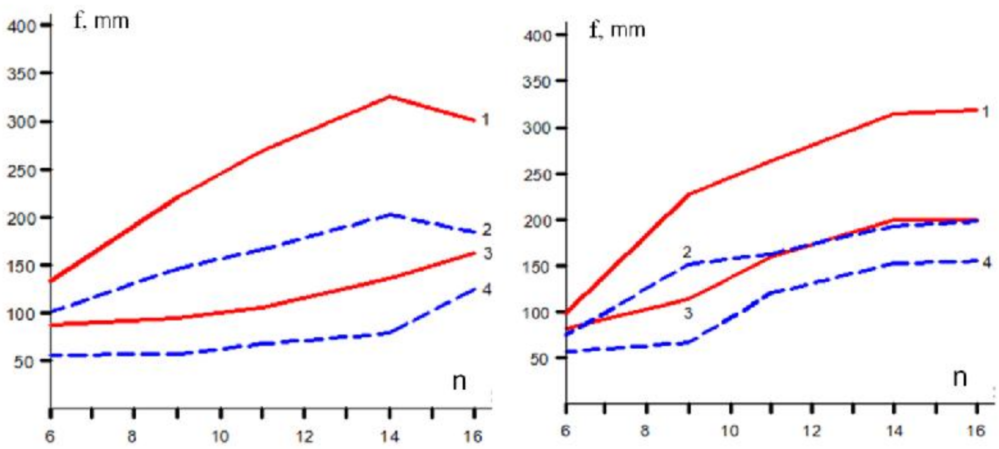

Fig 5. Displacement of the building depending on the number of stories

a – in the direction of X, b – in the direction of Y

1 – with seismic isolation by [1], 2 – with seismic isolation with application of accelerogram, 3 – without seismic isolations by [1], 4 – without seismic isolation with application of accelerogram.

According to the obtained results, the value of a seismic load on the level above the basement floor with application of seismic isolation in case of a 16 storied building is decreases by 42.5% (on Y axis) and by 52.9% (on X axis), in case of 6 storied building it is decreases correspondingly by 66.7% and 70.1%. For the rest of the buildings the value is in between. At the same time, it should be mentioned that received values for the buildings with 11, 14 and 16 stories are very close to each other.

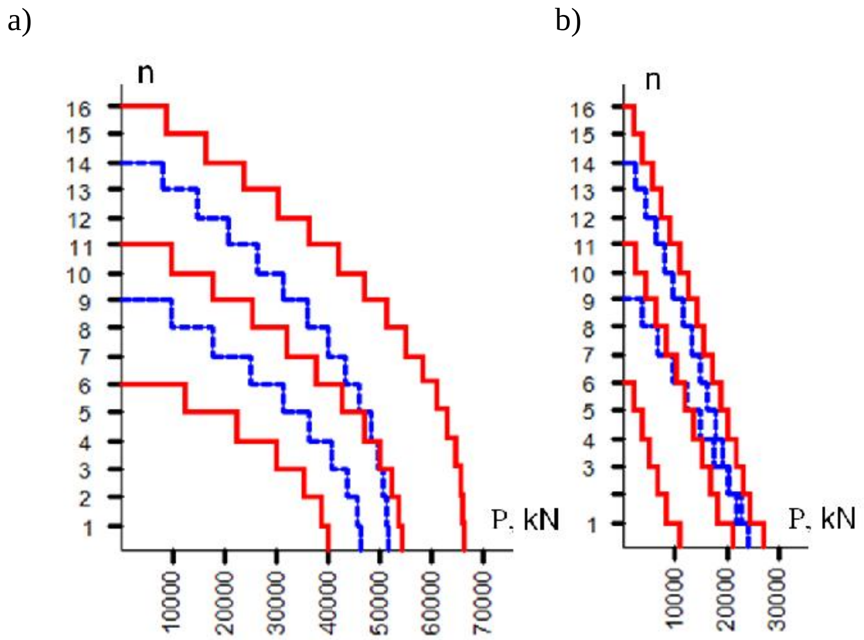

Fig. 6. The changes of lateral inertial forces depending on the stores of buildings
a – without seismic isolation, b – with seismic isolation

Considering the changes of reinforcement percentage for the edge column, it is became obvious, that the percentage of longitudinal reinforcement for 6 storied building, in case of the same sectional size, changes about 71%, and up to 53% for 16 storied one, see fig. 7.

As a conclusion it should be mentioned, that for multistory buildings, where the period of natural vibrations is between allowable limits the decrease of seismic load reaches up to 50%.

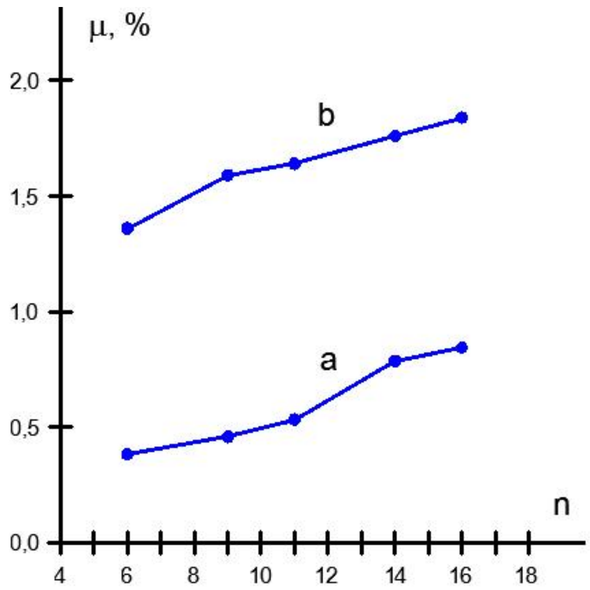

Fig. 7. Percentage of reinforcement for identical most heavily loaded column depending on the number of stories

a – with seismic isolation, b – without seismic isolation

Taking into account the prize of seismic isolation layered rubber bearings and also the necessity of periodic monitoring of their state and replacement in a certain period of time can be concluded that only additional economic calculations in each particular case may let us realistically to assess the efficiency of application of seismic isolators.

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29-30 May 2014, Tbilisi, Georgia

TO UNIFIED SYSTEM OF EQUATIONS OF CONTINUUM MECHANICS AND SOME MATHEMATICAL PROBLEMS IN SEISMOLOGY

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Abstract: *Proposed the unified mathematical models of continuum mechanics containing in particular Navier-Stokes, Euler Differential equations, systems of PDEs of Solid mechanics. In the second part the method of constructing 2D nonlinear models of von Kármán-Mindlin-Reissner type for binary mixture of porous, piezo and viscous elastic thin-walled structures with variable thickness is given. We will report that these problems have direct relations of some problems of seismology.*

1. Unified form for some nonlinear problems of continuum mechanics

Within the Newtonian mechanics and Noll's axiomatic we propose a unified dynamic system of pseudo-differential equations. Such form allows us to prove that the nonlinear phenomena observed in problems of solid mechanics can also be detected in Navier-Stokes type equations, and vice versa. For this case the basic system of PDE has the following form:

$$\circ\,\frac{D_\Gamma^2 u}{Dt^2} = f-(1-\Gamma)\nabla p+\nabla\left[(1+\nabla u)^2\right],\quad \frac{D_\Gamma^2 u}{Dt^2} = \begin{cases}\partial^2 u/\partial t^2, \Gamma=1\\ Dv/Dt, \Gamma=0\end{cases}, \tag{1.1}$$

where ° is a density, p is pressure, f is known volume forces, D/Dt is total or convective derivative, ² is stress tensor, $u=(u_1,u_2,u_3)^T$ and $v=(v_1,v_2,v_3)^T$ denote displacement and velocity vectors.

Newton's type law for viscous flow and Hooke's generalized law for solid structures may be recovered from the following expression:

$$^2 = \left[(1-\Gamma)\frac{\partial}{\partial t}+\Gamma\right]A_\Gamma \cdot £\ . \tag{1.2}$$

Here symmetric matrix A_Γ corresponds to fluid if $\Gamma = 0$ and to solid media if $\Gamma = 1$. The strain tensor is $\varepsilon = (\varepsilon_{11}, \varepsilon_{22}, \varepsilon_{33}, \varepsilon_{23}, \varepsilon_{13}, \varepsilon_{12})^{\mathrm{T}}$, where $2£_{ij} = \partial_i u_j + \partial_j u_i + u_{k,i} u_{k,j}$.

For conditions of conservation of mass or equations of continuity we have:

$$[(1-\Gamma)\partial_t + \Gamma]B_\Gamma[£] = 0, \tag{1.3}$$

where $B_0[°, £] = \partial_t ° + \nabla(° v)$. $B_1[£] = (B_{11}, B_{12}, B_{13}, B_{14}, B_{15}, B_{16})^T$ describes the St.Venant-Beltrami conditions.:

$$\begin{aligned} B_{1i}(£) &= £_{ii,kl} + £_{kl,ii} - £_{li,ki} - £_{ki,li} + C_{1i}(u), i,k,l = 1,2,3, k \neq l, i \neq k, i \neq l \\ B_{17-i}(£) &= £_{ii,kl} + £_{kl,ii} - £_{li,ki} - £_{ki,li} + C_{17-i}(u), i = 1,2,3, k = l = i+1, x_1 = x_4, \end{aligned} \tag{1.4}$$

$C_{ij}(u)$ are nonlinear differential forms of at most than 3 order. In classical case, when

$£_{\sim\sim} = u_{\sim,\sim} + 0.5u^2{}_{3,1}, £_{12} = 0.5(u_{1,2} + u_{2,1} + u_{3,1}u_{3,2})$, $C_{11}(u) = 2(u_{3,12})^2 - 2u_{3,11}u_{3,22}$ corresponds to well-known Monge-Ampere form: $[u, ¨] = \partial_{11}u\partial_{22}¨ - 2\partial_{12}u\partial_{12}¨ + \partial_{22}u\partial_{11}¨$.

For completeness to the general presentation (1.1)-(1.3) must be added the energy conservation (energy balance) equations which have evidently the same with (1.3) form. Now we consider this problem when continuum media is anisotropic elastic structures characterized by 13 independent constants.

If $\Gamma = 1$ and $\nabla(1+\nabla u)² = (\partial_j(²_{1j} + ²_{kj}u_{1,k}), \partial_j(²_{2j} + ²_{kj}u_{2,k}), \partial_j(²_{3j} + ²_{kj}u_{3,k}))^T$, from (1.1) follows system of nonlinear PDE of spatial theory of elasticity:

$$°\partial_{tt}u_i = -f_i + \partial_j(²_{ij} + ²_{kj}u_{i,k}). \tag{1.5}$$

See for example,[1).

Hooke's generalized law $(£ = A², A_1 \overset{def}{\equiv} A)$ has a form:

$$\begin{aligned} &£_{ii} := u_{i,i} = a_{ij}²_{jj} + a_{i6}²_{12}, £_{\sim 3} := 0.5(u_{\sim,3} + u_{3,\sim}) = a_{\sim+3,\sim+3}²_{\sim 3} + a_{\sim+3,6-\sim}²_{33-\sim}, \\ &£_{12} := 0.5(u_{1,2} + u_{2,1}) = a_{6i}²_{ii} + a_{66}²_{12}. \end{aligned}$$

Here A is the symmetric matrix and the compliance constant a_{ij} may be written in terms of engineering coefficients[2,3]

$$\begin{aligned} &a_{ii} = 1/E_i, a_{ij} = a_{ji} = -¬_{ij}/E_i = -¬_{ji}/E_j, a_{6-\sim,6-\sim} = 1/E_{3+\sim,3+\sim}, a_{66} = 1/E_{66}, \\ &a_{3+\sim,6-\sim} = -¬_{3+\sim,6-\sim}/E_{3+\sim}, a_{i6} = a_{6i} = -¬_{i6}/E_i = -¬_{6i}/E_6. \end{aligned}$$

Here according to {[3]], E_i are 6 " true modulus of rigidity",$¬_{ij}$ are 7 distributors of rigidity and the angles of turn absent ,or E_i are Yung's type modulus of elasticity,$¬_{ij}$ are the ratios of Poisson. For potential energy $(A²,²)$ we have the following inequality:

$$(A²,²) = \sum_{ij}(A²)_{ij}²_{ij} \geq \frac{1}{E_i}(1-¬_{i1}-¬_{i2}-¬_{i6})²_{ii}^{2} +$$

$$+\frac{1}{E_{6-\sim}}(1-¬_{3+\sim,6-\sim})²_{3-\sim 3}^{2} + \frac{1}{E_6}(1-¬_{6i}\,signi)²_{12}^{2}.$$

Thus $(A²,²)$ is positive definiteness, if

$$(¬_{i1}+¬_{i2}+¬_{i6}<1), i=1,2,3,6.$$

For an orthotropic case $¬_{3+\sim,6-\sim} = ¬_{i6} = 0$ (there are 10 independent coefficients) and above inequality is obviously satisfied as $¬_{ij} < 0.5, \forall i, j$.

Further, if $\Gamma = 0$, $\nabla u = 0$, $²_{ij} = -\frac{2}{3}«¢_{ij}divv + «(v_{i,j}+v_{j,i})$, $¢_{ij}$ is Kronecker delta, « is ***dynamical viscosity*** and a fluid is Newtonian liquid , then it's evident that from (1.1) follows:
Euler equations

$$°\left[\partial v_i/\partial t + v_j v_{i,j}\right] = -p_{,i} - ¢_{i3}g\ , i=1,2,3, \qquad (1.6)$$

For clearing, we remind that for $i=1$, from (1.6) follow:

$$°\left[\frac{\partial v_1}{\partial t} + v_1\partial_1 v_1 + v_2\partial_2 v_1 + v_3\partial_3 v_1\right] = -\partial_1 p, \partial_i v_j = \frac{\partial v_j}{\partial x_i},.¢_{13} = 0.$$

Navier – Stokes' PDEs

$$°\left[\partial v_i/\partial t + v_j v_{i,j}\right] = -p_{,i} - f_i + ²_{ij,j}. \qquad (1.7)$$

See, for example,[5]If $\Gamma = 0$ and $\nabla u \neq 0$, (1) represents Navier – Stokes type(new class) PDEs. *Our above elaborations are conformity of Newton's second law: $°\partial_t v = -f$ and different from it with concrete substance***.**

In case if on some continuum media acts also the electro-magnetic field (with PDEs (1.1)-(1.3)) using methodology of works it's necessity to consider Maxwell' dynamical system.
For example, for them we use a sufficient convenience form [4]:

$$£_{ijk}E_{j,k} + B_{j,t} = 0, £_{ijk}H_{j,k} - D_{i,t} = 0_i,\ \ B_{i,i} = 0, \quad B_{i,i} = 0, \qquad (1.8_{em})$$

where $£_{ijk}$ is antisymmetric unit tensor, E, H are tensions of an electric and a magnetic fields, D, B are an electrical and a magnetic induction vectors. Further, we also have:

$$E_{k,ki} - E_{i,kk} = -«_0 D_{i,tt}. \quad («_0 \text{ is a magnetic penetrance}). \qquad (1.9_{em})$$

Thus, the systems (1.1),(1.3),(1.8_{em}),(1.9_{em}), (1.2) (which must be corrected by state equations type of (1.45) p.24 [5]) and the corresponding initial and boundary value conditions are presenting mathematical models for problems, connected with an emission of an electromagnetic waves of piezo-electric and electrically conductive continuum media.

I underline that in these monographs [4] or other ones, were considering and investigating linear cases for corresponding thermo-dynamical problems and as the basic equation for electro-magnetic elastic plate and shell was using only Kirchhoff-Love' classical theory for isotropic media…

For completeness, in addition to (1.1)-(1.3), the energy conservation (energy balance) equalities, which have evidently the same form with (1.3), must be considered. From (1.1) Euler and Navier–Stokes' PDEs (see [1]) can be recovered if

$$\Gamma = 0, \ \nabla u = 0, \ ²_{ij} = -\frac{2}{3} «¢_{ij} div v + «(v_{i,j} + v_{j,i}).$$

If $\Gamma = 1$ and $\nabla(1+\nabla u)² = \left(\partial_j\left(²_{1j} + ²_{kj}u_{1,k}\right), \partial_j\left(²_{2j} + ²_{kj}u_{2,k}\right), \partial_j\left(²_{3j} + ²_{kj}u_{3,k}\right)\right)^T$ from (1.1) follows system of nonlinear PDEs of spatial theory of elasticity (see for example. [5]: If $\Gamma = 0$ and $\nabla u \neq 0$, (1.1) represents Navier – Stokes type (new class) PDEs.

2. Some mathematical problems of thin walled structures

One of the most principal objects in development of mechanics and mathematics is a system of nonlinear differential equations for elastic isotropic plate constructed by *von Kármán.* In 1978 *Truesdell* expressed a doubt: "Physical Soundness"of *von Kármán* system. This circumstance generated the problem of justification of *von Kármán* system. Afterwards this problem is studied by many authors, but with most attention it was investigated by Ciarlet. In particular, he wrote: "The *von Kármán* equations may be given a full justification by means of the leading term of a formal asymptotic expansion" ([6], p. 368). This result obviously is not sufficent for a justification of "Physical Soundness" of this system, because representations by asymptotic expansions is dissimilar and leading terms are only coefficients of power series without any "Physical Soundness."

Based on the [6], the method of constructing such anisotropic nonhomogeneous 2D nonlinear models of *von Kármán-Mindlin-Reissner (KMR)* type for binary mixtures; (poro/visco/piezo-electric/electrically conductive)elastic thin-walled structures with variable thickness is given, by means of which the terms become physically sound. The corresponding variables are quantities with certain physical meaning: averaged components of the displacement vector, bending and twisting moments, shearing forces, rotation of normals, surface efforts. The given method differs from the classical one by the fact that according to the classical method, one of the equations of *von Kármán* system represents one of *Saint-Venant*'s compatibility conditions, i.e. it's obtained on the basis of geometry and not taking into account the equilibrium equations.

At last we remark that in dynamical cases the corresponding system contains wave processes not only in the vertical, but also in the horizontal direction. The corresponding equations are [8]:

$$\left(D\Delta^2 + 2h^{\circ}\partial_{tt} - 2DE^{-1}(1+{}^{3})^{\circ}\partial_{tt}\Delta\right)w = \left(1 - \frac{h^2(1+2¥)(2-{}^{3})}{3(1-{}^{3})}\Delta\right)\left(g_3{}^{+} - g_3{}^{-}\right) +$$

$$2h\left(1 - \frac{2h^2(1+2¥)}{3(1-{}^{3})}\Delta\right)[w,\ddot{}\ ] + h\left(g_{\sim,\sim}{}^{+} - g_{\sim,\sim}{}^{-}\right) - \int_{-h}^{+h}\left(tf_{\sim,\sim} - \left(1 - \frac{1}{1-{}^{3}}\Delta\left(h^2 - t^2\right)\right)f_3\right)dt \ , \quad (2.1)$$

$$\left(\Delta^2 - \frac{1-¬^2}{E}{}^{\circ}\Delta\partial_{tt}\right)^{\cdot\cdot} = -\frac{E}{2}[w,w] + \frac{¬}{2}\left(\Delta - \frac{2^{\circ}}{E}\partial_{tt}\right)\left(g_3^{+} + g_3^{-}\right) + \frac{1+¬}{2h} f_{\sim,\sim} \quad (2.2)$$

From (2.1), (2.2) we get *von Kármán* system (in dynamical case too) if $¥ = -0.5$, ${}^{\circ} = g^{\pm} = f_{\sim} = 0$. Thus *von Kármán* classical system gives the possibility to use methods of Harmonic Analysis. Since the new dynamical terms are $\Delta\partial_{tt}{}^{\cdot\cdot}$ and also $\partial_{tt}\left(g_3^{+} - g_3^{-}\right)$, therefore the *KMR* type (2.1)-(2.2) systems describe new nonlinear wave processes. We remark that if the equations (2.1), (.2.2) are in final form it's evident that for them it is not possible to apply the Fourier Analysis technique. Because this system is nonlinear and both DEs contains dynamical members against to *von Kármán* equations in classical form.

In addition, an equation *corresponding to (2.2) by von Kármán, A. Föppl, Love, Lukasievicz, Tomoshenko, Donnel, Landau, Ciarlet, Antman et al.* were constructed by the condition $\varepsilon_{11,22} - 2\varepsilon_{12,12} + \varepsilon_{22,11} = -0.5[u_3, u_3]$ and Hooke's law (but without using the equilibrium equations!). As we prove in works [4] the form (2.2) follows immediately for more general cases, when thin-walled elastic structures are anisotropic and if we use Hooke's law, equilibrium equations with and nonlinear relations between strain tensor and displacement vector:

$$\varepsilon_{\alpha\beta} = 0.5\left(u_{\alpha,\beta} + u_{\beta,\alpha} + u_{3,\alpha}u_{3,\beta}\right).$$

Now we prove that (2.2) equations in dynamical case has the following form:

$$\left(-\frac{1-\nu^2}{E}\rho_1\Delta\partial_{tt}\right)\Phi = \frac{\nu}{2}\left(\Delta-\frac{2\rho_1}{E}\partial_{tt}\right)\left(g_3^+ + g_3^-\right)+\frac{1+\nu}{2h}f_{\alpha,\alpha}. \qquad (2.3)$$

Thus we must demonstrate that both way give the expression $\Delta^2\Phi - 0.5E[w,w]$

In fact, we constructed (2.2) by using the following expression (see [4]) :

$$\begin{cases} (\lambda^* + 2\mu)\Delta(\bar{\varepsilon}_{11}+\bar{\varepsilon}_{22}) = \\ \left(2\mu(3\lambda+2\mu)\right)^{-1}(\lambda+2\mu)(\lambda^*+2\mu)\Delta(\bar{\sigma}_{11}+\bar{\sigma}_{22}) + \cdots = \\ \mu\left((-1)^{\alpha+\beta}\partial_{3-\alpha}\partial_{3-\beta}\bar{u}_{3,\alpha}\bar{u}_{3,\beta}\right)+\cdots, \end{cases} \qquad (2.4)$$

where **dots** denote other different members from (2.2). Let us $\bar{\sigma}_{\alpha\beta} = (-1)^{\alpha+\beta}\partial_{3-\alpha}\partial_{3-\beta}\Phi$, then from preliminary equation follows (2.2) or: $\Delta^2\Phi = -0.5E[w,w]+\cdots$ From St.Venant-Beltrami compatibility conditions it is evident that

$$\Delta^2\Phi + 0.5E[w,w] \equiv 0.$$

The mathematical models considered in [7] , ch.I contain a new quantity, which describes an effect of boundary layer. Existence of this member not only explains a set of paradoxes in the two-dimensional elasticity theory (*Babushka, Lukasievicz, Mazia, Saponjan*), but also is very important for example for process of generating cracks and holes (details see in [3], ch.1, par. 3.3). Further, let us note that in works [8] equations of (2.2) type are constructed with respect to certain components of stress tensor by differentiation and summation of two differential equations. Also other equations of KMR type, which differ from (2.2) type equation, are equivalent to the system, where the order of each equation is not higher than two. For example, in the isotropic case, obviously, for coefficients we have [8]: $c_{\alpha\alpha} = \lambda^* + 2\mu$, $c_{66} = 2\mu$, $c_{12} = \lambda^*$, $c_{\alpha 6} = 0$, $\lambda^* = 2\lambda\mu(\lambda+2\mu)^{-1}$, λ and μ are the Lame coefficients. Then the system (2.1) of [4] is presented in a form:

$$\left(\text{a}^* + 2\text{«}\right)\partial_1{}^{2} + \text{«}\partial_2\text{µ} = \frac{1}{2h}\overline{f_1} + \text{«}\left(\partial_1\left(\overline{u_{3,2}}\right)^2 - \partial_2\left(\overline{u_{3,1}}\,\overline{u_{3,2}}\right)\right) - \frac{\text{a}}{2h\left(\text{a}+2\text{«}\right)}\left(\pm_{33,11}\right), \qquad (2.5a)$$

$$\left(\text{a}^* + 2\text{«}\right)\partial_2{}^{2} - \text{«}\partial_1\text{µ} = \frac{1}{2h}\overline{f_2} + \text{«}\left(\partial_2\left(\overline{u_{3,1}}\right)^2 - \partial_2\left(\overline{u_{3,1}}\,\overline{u_{3,2}}\right)\right) - \frac{\text{a}}{2h\left(\text{a}+2\text{«}\right)}\left(\pm_{33,22}\right), \qquad (2.5b)$$

where the functions: $^{2} = \overline{\text{£}_{\sim\sim}}$, $\text{µ} = \overline{u_{1,2}} - \overline{u_{2,1}}$ correspond to plane expansion and rotation respectively.

Thus, in the dynamical case the KMR type systems are (2.1) and (2.2). In the statical case from (2.5) immediately follows such relations:

$$\frac{\nu}{2}\Delta(g_3^+ + g_3^-) + \frac{1+\nu}{2h} f_{\alpha,\alpha} = 0.$$

The system (2.1)-(2.2) represents 2Dim mathematical model having clear practical meaning where it is possible to consider together the methods of Mathematical Analysis (in wide sense) and Optimal Control Theory of Bellman-Pontryagin.

Let us consider the main terms from (2.1):

$$D'\Delta[w,\text{¨}] = D'\left([\Delta w,\text{¨}] + [w,\Delta\text{¨}] + 2[\partial_{\sim} w, \partial_{\sim}\text{¨}]\right) \quad (\mathrm{D}' = 4\mathrm{h}^3(1+2\gamma)/3(1-\nu)),\ \ D\Delta^2 w.$$

By using for simplicity the typical relations as $\partial_{11}\text{¨} = \mp_{22}$, $\partial_{12}\text{¨} = -\mp_{12}$, $\partial_{22}\,\varphi = \overline{\sigma}_{11}$, the first term may be rewritten in the following form:

$$\Delta[w,\text{¨}] = \left(\mp_{11}\partial_{11}\Delta w + 2\mp_{12}\partial_{12}\Delta w + \mp_{22}\partial_{22}\Delta w\right) + \left(\partial_{11} w\Delta\mp_{11} + 2\partial_{12} w\Delta\pm_{12} + \partial_{22} w\Delta\pm_{22}\right)$$
$$+ 2\left(\mp_{11,\sim}\partial_{11} w_{,\sim} + 2\mp_{12,\sim}\partial_{12} w_{,\sim} + \mp_{22,\sim}\partial_{22} w_{,\sim}\right)$$

The calculation and analysis by these expressions of a symbolical determinant show that the characteristic form of the system (2.1) and (2.2) may be strongly elliptical, hyperbolical and parabolical or mixed type. Other than that, in statical cases the system may have shock waves solutions.

Let us consider (2.1) equation and underline the members:

$$D_1\Delta[w,\text{¨}] = D'\left([\Delta w,\text{¨}] + [w,\Delta\text{¨}] + 2[\partial_{\sim} w, \partial_{\sim}\text{¨}]\right),\ (D_1 = 4h^3(1+2\text{¥})/3(1-\neg)),\ D\Delta^2 w.$$

By using for simplicity the typical relations as $\partial_{11}\varphi = \overline{\sigma}_{12}$, $\partial_{12}\text{¨} = -\mp_{12}$, $\partial_{22}\,\varphi = \overline{\sigma}_{11}$, the first expression may be rewritten in the following form:

$$\Delta[w,\text{¨}] = \left(\mp_{11}\partial_{11}\Delta w + 2\mp_{12}\partial_{12}\Delta w + \mp_{22}\partial_{22}\Delta w\right) + \left(\partial_{11} w\Delta\mp_{11} + 2\partial_{12} w\Delta\pm_{12} + \partial_{22} w\Delta\pm_{22}\right)$$
$$+ 2\left(\mp_{11,\sim}\partial_{11} w_{,\sim} + 2\mp_{12,\sim}\partial_{12} w_{,\sim} + \mp_{22,\sim}\partial_{22} w_{,\sim}\right).$$

(2.6)

The calculate and analysis by these expressions of a symbolical determinant show that the characteristic form of systems type (2.1) and (2.2) may be positive, negative or zero numbers as well as an arbitrary continuous function of x, y. Here we must remark that $ED_1 = 4(1+2\text{¥})(1+\neg)D$, as so if $\{f\}$ denotes physical dimension of value f, it's evident $\{\Delta^2 w\} = \{\Delta[w,\text{¨}/E]\}$.

Thus, the first summand of (2.6) may be defining the nonlinear wave processes for static cases. The structure of the third summand obviously corresponds to 2D soliton type solutions of Corteveg- de Vries or Kadomtsev-Petviashvili kind.

Analogous three-dimensional nonlinear model for anisotropic binary mixtures are
presented in the our recently work .Here we generalize previously known model for poro–viscous-elastic binary mixtures. The constructed models together with certain independent scientific interest represent such form of spatial models, which allow not only to construct, but also to justify von KMR type systems as in the stationary, as well as in nonstationary cases. Under justification we mean assumption of “Physical Soundness” to these models in view of Truesdell-Ciarlet (see for example details in [6, ch.5]). As is known, even in case of isotropic elastic plate with constant thickness the subject of justification constituted an unsolved problem. The point is that von Kármán, Love, Timoshenko, Landau & Lifshits, Donell and others considered one of the compatibility conditions of Saint-Venant-Beltrami as one of the equations of the corresponding system of differential equations. This fact was verified also by Podio-Guidugli recently.

Further, let us note that in our works equations of (2.2), (2.5) type are constructed with respect to certain components of stress tensor by differentiation and summation of two differential equations. Also other equations of KMR type, which differ from (2.5) type equation, are equivalent to the system, where the order of each equation is not higher than two.

We remind that matrices of compliability - A and rigidity - B in the formulae (1.5) [3] contain no more than thirteen independent elastic constants, i.e., at any point of body Ω_h even if one plane of elastic symmetry spreads, which is parallel to the coordinate plane- oxy .Using of formulae (2.25) [3], we have:

$$\sigma_{\alpha\alpha} = c_{\alpha\beta}\varepsilon_{\beta\beta} + c_{\alpha\alpha}\varepsilon_{12} + b_{13}b_{33}^{-1}\sigma_{33}, \sigma_{\alpha 3} = c_{3+\alpha 3+\alpha}\varepsilon_{\alpha 3} + b_{45}\varepsilon_{\alpha 3-\alpha}, \sigma_{12} = c_{\alpha 6}\varepsilon_{\alpha\alpha} + c_{66}\varepsilon_{12} + b_{36}b_{33}^{-1}\sigma_{33}$$

where

$$c_{\alpha\alpha} = b_{\alpha\alpha} - b_{\alpha 3}^{2}b_{33}^{-1},\ c_{12} = c_{21} = b_{12} - b_{13}b_{23}b_{33}^{-1},\ c_{\alpha 6} = b_{\alpha 6} - b_{\alpha 3}b_{36}b_{33}^{-1},\ c_{66} = b_{66} - b_{36}^{2}b_{33}^{-1}.$$

Then, obviously, from equations (1) and (2) (when $\Gamma = 1$) follows:

$$2h\left[c_{\alpha\alpha}(\bar{u}_{\alpha,\alpha\alpha} + \frac{1}{2}\bar{A}_{\alpha\alpha,\alpha}) + c_{\alpha 3-\alpha}(\bar{u}_{3-\alpha,12} + \frac{1}{2}\bar{A}_{3-\alpha 3-\alpha,\alpha}) + \frac{1}{2}c_{\alpha 6}(\bar{u}_{\alpha,12} + \bar{u}_{3-\alpha,\alpha\alpha} + \bar{A}_{12,\alpha})\right]$$

$$+2h\left[c_{6\alpha}(\bar{u}_{\alpha,12} + \frac{1}{2}\bar{A}_{\alpha\alpha,3-\alpha}) + \frac{1}{2}c_{66}(\bar{u}_{\alpha,3-\alpha 3-\alpha} + \bar{u}_{3-\alpha,12} + \bar{A}_{12,3-\alpha})\right]$$

$$+b_{33}^{-1}(b_{\alpha 3}\partial_{\alpha} + b_{36}\partial_{3-\alpha})\int_{-h}^{h}\sigma_{33}dz = \int_{-h}^{h} f_{\alpha}dz - \partial_{\beta}(\sigma_{k\beta}u_{\alpha,k}) - (g_{\alpha}^{+} - g_{\alpha}^{-}).$$

Further, from these equations, using the method ch. I [3], we have:

$$2h\Big[(c_{\alpha\alpha}\partial_{\alpha\alpha}+\frac{3}{2}c_{\alpha 6}\partial_{12}+\frac{1}{2}c_{66}\partial_{3-\alpha 3-\alpha})\bar{u}_\alpha+(\frac{1}{2}c_{\alpha 6}\partial_{\alpha\alpha}+(c_{12}+\frac{1}{2}c_{66})\partial_{12}$$

$$+c_{3-\alpha 6}\partial_{3-\alpha 3-\alpha})\bar{u}_{3-\alpha}\Big]\;+h\Big[(c_{\alpha\alpha}\partial_\alpha+c_{\alpha 6}\partial_{3-\alpha 3-\alpha})(\bar{u}_{3,\alpha})^2+(c_{\alpha 6}\partial_\alpha+c_{66}\partial_{3-\alpha})\bar{u}_{3,1}\bar{u}_{3,2}$$

$$+(c_{12}\partial_\alpha+c_{3-\alpha 6}\partial_{3-\alpha})(\bar{u}_{3,3-\alpha})^2\Big]+b_{33}^{-1}(b_{\alpha 3}\partial_\alpha+b_{\alpha 6}\partial_{3-\alpha})\int_{-h}^{h}\sigma_{33}dz=\bar{f}_\alpha\,. \qquad (2.7)$$

Here

$$\bar{u}_\alpha=\frac{1}{2h}\int_{-h}^{h}u_\alpha(x,y,z)dz\,,\ \bar{u}_3=\frac{3}{2h^3}\int_{-h}^{h}(h^2-z^2)u_3(x,y,z)dz\,,$$

$$\bar{f}_\alpha=\int_{-h}^{h}(f_\alpha-\partial_\beta(\sigma_{k\beta}u_{\alpha,k}))dz-(g_\alpha^+-g_\alpha^-)-R_\alpha^{AN},$$

$$R_{\sim}^{AN}=R_N^{AN}[u_1,u_2,u_3-\bar{u}_3]=\int_{-h}^{h}[(c_{\sim\sim}\hat{o}_{\sim}+c_{\sim 6}\hat{o}_{3-\sim})(u_{k,\sim}^2-(\bar{u}_{3,\sim})^2)\,.$$

$$+(c_{\alpha\alpha}\partial_\alpha+c_{66}\partial_{3-\alpha})(u_{k,1}u_{k,2}-\bar{u}_{3,1}\bar{u}_{3,2})+(c_{12}\partial_\alpha+c_{3-\alpha 6}\partial_{3-\alpha})(u_{k,3-\alpha}^2-(\bar{u}_{3,3-\alpha})^2)]dz\,.$$

The system of equations (2.7), if we neglect the remainder terms R, for a linear case corresponds to the problem of defining generalized plane stress-stain state. For a nonlinear case from (2.7) it follows immediately one of the basic equations of the *von Kármán* system, corresponding to the Airy function if each equation is differentiated and summed (for details see below).

For an isotropic case, obviously, for coefficients we have $c_{\alpha\alpha}=\lambda^*+2\mu$, $c_{66}=2\mu$, $c_{12}=\lambda^*$, $c_{\alpha 6}=0$, $\lambda^*=2\lambda\mu(\lambda+2\mu)^{-1}$, λ and μ -are the Lame coefficients. Then the system (2.1) is presented in a form:

$$(a^*+2«)\hat{o}_1^2+«\hat{o}_2\mu=\frac{1}{2h}\bar{f}_1+«(\hat{o}_1(\bar{u}_{3,2})^2-\hat{o}_2(\bar{u}_{3,1}\bar{u}_{3,2}))-\frac{a}{2h(a+2«)}\int_{-h}^{h}\pm_{33,1}dz\,, \qquad (2.8)$$

$$-\mu\partial_1\omega+(\lambda^*+2\mu)\partial_2\tau=\frac{1}{2h}\bar{f}_2+\mu(\partial_2(\bar{u}_{3,1})^2-\partial_1(\bar{u}_{3,1}\bar{u}_{3,2}))-\frac{\lambda}{2h(\lambda+2\mu)}\int_{-h}^{h}\sigma_{33,2}dz,$$

where the functions: $\tau=\bar{\varepsilon}_{\alpha\alpha}$, $\omega=\bar{u}_{1,2}-\bar{u}_{2,1}$ correspond to plane expansion and rotation. The second equation with respect to the Airy function the von Kármán system, following from (12) has a form:

$$\left(\lambda^{*}+2\mu\right)\Delta\bar{\varepsilon}_{\alpha\alpha}=\left(\lambda^{*}+2\mu\right)\Delta\left(\frac{1}{2\mu}-\frac{1}{\mu\left(3\lambda+2\mu\right)}\right)(\bar{\sigma}_{11}+\bar{\sigma}_{22})=\mu(\partial_{11}(\bar{u}_{3,2})^{2}$$

$$-2\partial_{12}(\bar{u}_{3,1}\,\bar{u}_{3,2})+\partial_{22}(\bar{u}_{3,1})^{2})+\frac{1}{2h}\bar{f}_{\alpha,\alpha}+\frac{1}{2h}\left(\frac{\lambda\left(\lambda^{*}+2\mu\right)}{2\mu\left(3\lambda+2\mu\right)}-\frac{\lambda}{\lambda+2\mu}\right)\int_{-h}^{h}\Delta\sigma_{33}dz\,,$$

or

$$\Delta(\bar{\sigma}_{11}+\bar{\sigma}_{22})=-\frac{E}{2}[\bar{u}_{3},\bar{u}_{3}]+\frac{\nu}{2h}\int_{-h}^{h}\Delta\sigma_{33}dz+\frac{1+\nu}{2h}\bar{f}_{\alpha,\alpha}\,. \qquad (2.9)$$

If we introduce the Airy function by a well-known way:

$$\sigma_{\alpha\beta}=\left(-1\right)^{\alpha+\beta}\partial_{3-\alpha\,3-\beta}\Phi\ ,$$

from (2.8) it follows the second equation of the von Karman system

$$\Delta^{2}\Phi^{*}=-\frac{E}{2}[\bar{u}_{3},\bar{u}_{3}]+\frac{\nu}{2}\Delta\left(g_{3}^{+}+g_{3}^{-}\right)+\frac{1+\nu}{2h}\bar{f}_{\alpha,\alpha}\ , \qquad (2.10)$$

If we consider an orthotropic case, when $c_{\alpha 6}=0$. Then from (2.1), obviously, it follows

$$2h\left[c_{\alpha\alpha}\partial_{\alpha}\bar{\varepsilon}_{\alpha\alpha}+\left(c_{12}+c_{66}\right)\partial_{\alpha}\bar{\varepsilon}_{3-\alpha 3-\alpha}+\frac{1}{2}\left(-1\right)^{3-\alpha}c_{66}\partial_{3-\alpha}(\bar{u}_{1,2}-\bar{u}_{2,1})\right]$$

$$+hc_{66}\left[\partial_{3-\alpha}(\bar{u}_{3,1}\,\bar{u}_{3,2})-\partial_{\alpha}(\bar{u}_{3,2})^{2}\right]=\bar{f}_{\alpha}-b_{\alpha 3}b_{\alpha 3}^{-1}\int_{-h}^{h}\sigma_{33,\alpha}dz-R_{\alpha}^{AN}\,, \qquad (2.11)$$

where

$$\bar{\varepsilon}_{\alpha\alpha}=\frac{1}{2h}\int_{-h}^{h}\left(u_{\alpha,\alpha}+u_{k,\alpha}^{2}\right)dz\,.$$

When coefficients b and c satisfy the condition of generalized transversality ([7], p. 27), i.e. there are true the relations:

$$c_{11}=c_{22}=c_{12}+c_{66}\,,\ b_{13}=b_{23}\ ,$$

then from (2.11) immediately it follows:

$$c_{11}\partial_{1}\tau+\frac{1}{2}c_{66}\partial_{2}\omega=\frac{1}{2h}\bar{f}_{1}-b_{13}b_{33}^{-1}\frac{1}{2h}\int_{-h}^{h}\sigma_{33,1}dz-hc_{66}\left[\partial_{2}(\bar{u}_{3,1}\,\bar{u}_{3,2})-\partial_{1}(\bar{u}_{3,2})^{2}\right]-R_{1}^{AN}\ , \qquad (2.11a)$$

$$c_{11}\partial_{2}\tau-\frac{1}{2}c_{66}\partial_{1}\omega=\frac{1}{2h}\bar{f}_{2}-b_{23}b_{33}^{-1}\frac{1}{2h}\int_{-h}^{h}\sigma_{33,2}dz-hc_{66}\left[\partial_{1}(\bar{u}_{3,1}\,\bar{u}_{3,2})-\partial_{2}(\bar{u}_{3,2})^{2}\right]-R_{2}^{AN}\,.$$

The systems of differential equations (2.7-2.10) obviously, are a splitting of one, corresponds to the function Φ^{*} from the von Kármán equations and equivalent to it in case differentiability of

the functions $\bar{u}_\alpha$, which are averaged on a thickness of the plate of horizontal components of displacement vector.

Thus, obtained the system of differential equations (2.7-2.10) is constructed from the initial three-dimensional problem of the theory of elasticity (1.1) - (1.5) [7] with respect to the averaged on a thickness of the components of displacement vector-$\bar{u}$.

The another basic equation of the von Karman system corresponds for a linear case to a bending problem. For clarity and completeness we now give a presentation of the second basic relation in case, when Ω_h is an isotropic elastic plate of constant thickness (more general case, when an elastic plate of a variable thickness with finite displacement is anisotropic and non-homogeneous see [7], ch.1).

$$\frac{(1-\nu)D}{2}\Delta u_\alpha^* + \frac{(1+\nu)D}{2}\partial_\alpha u_{\beta,\beta}^* - \frac{3(1-\nu)D}{2h^2(1+2\gamma_\alpha)}(u_\alpha^* + \bar{u}_{3,\alpha}) = f_\alpha^* + R_{\alpha+2}[u_\alpha], \tag{2.12}$$

$$\frac{3(1-\nu)\mathrm{D}}{2\mathrm{h}^2(1+2\gamma_\alpha)}\left(\Delta\,\bar{\mathrm{u}}_3 + \mathrm{u}_{\alpha,\alpha}^*\right) = \mathrm{f}_3^* + \mathrm{R}_5[\bar{\mathrm{u}}_3]\,.$$

Obviously, the equations (2.7) (or (2.8)-(2.11)) and type of (2.12) without remainder terms present full system of KMR type differential equations with respect to functions $\bar{u}_i(x,y)$ and $u_\alpha^*(x,y)$.

We remark that the non-linear two-dimensional models for Reissner type DEs with layered effects for anisotropic elastic plates first were constructed in [7].

At last, for example, let us consider Kirchhoff (see, for instance,[9]) 1D with respect to spatial coordinate

model:

$$w_{tt} + w_{xxxx} - \left(\sim + \int_0^l \left(\frac{\partial w}{\partial x}\right)^2 dx\right)\frac{\partial^2 w}{\partial x^2} = f\ . \tag{2.13}$$

From (2.10) equation if

$$1 + 2¥ = \frac{3(1-¬)D}{h^2(2-¬)\Delta(g_3^+ - g_3^-)}\left(\sim + \int_0^l \left(\frac{\partial w}{\partial x}\right)^2 dx\right)\frac{\partial^2 w}{\partial x^2},$$

as $\Delta^2 w = \partial_1^4 w$ and suppose $2E^{-1}(1+¬)°\partial_{tt}\Delta w = 0$, immediately follows (17).

This example is typical for constructing nonlinear dynamical models of second order accuracy (considering by Ambartsumian, Antman, Ball, Lamb, Love, Pochhammer, Rayleigh, Timoshenko). See details in [7],ch.1,p.2.3

Acknowledgments

This report and an article were supported by the Georgia National Science Foundation (Applied programs, Grant N 30/28) and Iv. Javakhishvili Tbilisi State University Rector's foundation.

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29-30 May 2014, Tbilisi, Georgia

IMPROVEMENT OF BUILDINGS SEISMIC RESISTANCE BY APPLICATION OF SEISMIC INSULATION

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Abstract: *To the reduction of losses caused due earthquake damages by implementation of new structural system is dedicated this article. One of such system that has worldwide spread over the last 20-25 years, are seismic insulation systems,which are used in rigid structural systems, as well as in frame structures. In the article are considered the contribution of Georgian scientists and civil engineers in the developmentof earthquake-proof engineering in the former Soviet Union and in its member republics located inseismic areas. Are stated also the issues of retraining of construction industry specialists that are working in seismicareas.*

Keywords: seismic resistance, resonant - vibration theory, seismic isolation, Bologna, Lisbon process, hardness, rubber - metal insulators.

It is known that the construction of the inter-sector activity, is creating the foundations for the development of all branches of national economy, stipulates the engineering protection of environment, and is creating a residential housing, cultural - living and social conditions. The industrial construction is most important for stipulating the improvement of the country's potential and employment conditions.

The importance of civil engineers role has special interest in regions where earthquakes are frequent.

To this region belongs our country.

Draws attention to the fact that with the increasing in industrial potential are increased the covered by earthquakes areas.

In addition, is increasing the level of losses, because in the damaged due earthquake factories compared with previous is installed more expensive equipment, which is associated with technical progress. The houses are from more expensive construction materials, and the apartments are equipped with more modern and expensive furniture and equipment.

Hence the calculated in unit losses are bigger.

In the mountainous regions of Georgia, where the buildings are not constructed by modern technical regulations (codes), recently becomes more frequent not so strong, but the devastating earthquakes.

The losses are significantly large. So much that several governments yet nowadays did not perform their full liquidation.

Of course, they caused by earthquake damage is depending on the level of construction sciences, planners, civil engineers and industrial base specialists.

This level is defined due their qualifications. Engineer in whose hands is the life of people must be licensed at all levels.

According to our information, "in every state of United States of America, on whole territory of country, the law provides the licensing of Engineers to protect the population's health, welfare and safety conditions".

Similar requirements are in many countries.

Do not believe it, but the fact is that in our country in XXI century in the field of construction is abandoned the licensing and was issued a special law on it.

In this regard, I have to state one sad, but fact.

On December 7, 1988, the tragically 6.9-magnitude earthquake in our neighboring Armenia (Spitak, and all area of country), was killing approximately 25 thousand people and caused huge material damage.

On October 17, 1989 in California (Loma Prita - U.S.) the earthquake with same magnitude, were killing 62 human lives the loss is relatively small.

This example demonstrates it that qualified certified civil engineer construct more resistant buildings that less was damaged during the earthquakes, even tragic.

Where the great attention was paid to building design, application of scientific researches results, their implementation in practice, as well as protection at the construction of norms and regulations on quality control, the results of devastating earthquake are significantly reduced.

For seismic stability very important is to construct the buildings by up-to-date, scientifically proven methods, including their strength, stability and reliability. The country's leadership should be interested in this, because it concerns not only the civil engineering, but also all branches of the national economy: mechanical or electrical engineering, chemical or agriculture engineering, etc.

In many countries worldwide were established scientific, developing or other types institutions of construction industry that are aimed to protect human and material resources.

It is paradoxically, but in Georgia was closed for the last 9 years, a world-renowned research centers such as the Academy of Sciences of Kiriak Zavriev Institute of Structural Mechanics and Earthquake Engineering Institute, well-known in the Soviet Union famous zonal scientific - research and

design - experimental Institute. Actually was stopped the functioning of building materials and hydraulic engineering institutions. Were destroyed scientific - experimental bases.

Gentlemen, the resonant oscillation phenomenon is of great importance to the field of study of the impact of the earthquake.

The resonant - vibration theory, originally proposed by Japanese scientists N. Mononobe in 1920, thus laying the foundations for resonant phenomenon: at matches of ground frequency with building's own oscillation frequency arise the resonance. Exactly these frequencies of oscillations are extremely dangerous.

This theory was extended by K.S. Zavriev (1927), those Institute will no longer exist.

Though created by Georgian specialists new, progressive, efficient and earthquake proof engineering, were constructed a unified framework systems for industrial and civil buildings in seismically active all 11 Soviet republics, now independent states.

Let's remember constructed at that time a unique Enguri dam, Tbilisi TV tower, metro in Tbilisi, Moscow and other technical wonders.

In addition, by Georgian specialists have developed construction drawings albums in quality of 3 000 000 copies distributed in both the Union and former socialist countries. It was built during almost 25 years the earthquake structures that were designed for 7, 8 and 9-magnitude seismic impact and despite of large-scale participation caused by arbitrary seismic impact has not taken place significant damages.

With consideration of above mentioned, to the issue of civil engineer training, including their preparation accordingly of advanced level in Europe and possibly of whole worlds of achievement has the greatest importance.

Therefore, the European Council of Civil Engineers organization, an official member of which is the Georgian Society of Civil Engineers, in its 57-th General Assembly in Portugal unanimously decided to establish in Tbilisi civil engineers training and their professional development courses, while simultaneously will be in parallel taken into account the acquisition of English language.

The organization must serve the South Caucasus countries, as well as specialists in Eastern European countries.

Thus, the graduates of training courses will be awarded by a certificate which they will be eligible to work in any EU country.

Is prepared a proposal for such training implementation, and it is waiting for private companies and government organizations investments.

Will be noted that retraining idea is not a new for our specialists.

I would like to mention the work in the Georgian Technical University accordingly of Bologna Declaration principles, whose goal is the establishment of a common European higher education, training and research high-performance requirements.

The University also provides a process adopted by the Lisbon strategy, according to which in Europe should be the most competitive and knowledge-based economy.

We must recall also signed by Georgian parliament "The concept of the recognition of qualifications concerning higher education in the European Region" that gives the possibility to Georgian citizens, technical specialists, request a certificate of his qualifications or study periods recognition by the Convention signatory countries.

Gentlemen,

Look at how the Bologna and Lisbon processes are comparable with the solution we will establish with the principles of professional development!!!

As it is mentioned above, created by our country's scientists and engineers a lot of new structural system or the results of scientific research, we implemented in the Soviet Union and other countries construction practice. For proofing of this is enough the creation of scientific grounds and earthquake engineering unified framing elements.

Now the situation has changed. Now we have to import in Georgia technologies and research novelties, which was designed and implemented in different countries at a time when our scientists and civil engineers are practically deprived of the opportunity to work in the field of scientific research.

Still, after the leading scientific - research institutes were closed, when our qualified specialists are running to other countries, and others has changed the specialty, at least we consider that we have to regain the lost positions of Georgian specialists in the field of seismic resistance.

The neighboring countries are in forward to us, the Central Asia, United States and Japan has always been ahead of us.

Gentlemen, our main goal are focused on increasing the seismic resistance by application of seismic insulation facilities.

Japan has always been in the foreground in this field. Now the great achievements have United States, New Zealand, France, Russia, Kazakhstan, our neighboring Armenia.

Since we have lost quite a lot of time in the field of improvement of structures seismic resistance, our goal is to study the experience of technically developed countries achieved by using seismic isolation of structures and their effectiveness in improvement of stability.

Also is taken into consideration the state of the available scientific - experimental bases, where it is possible to study destructive force of the earthquake simulation in the natural ot reduced-scale models, nowadays it is destroyed and we are unable to conduct experiments.

It is known that the resulting of earthquakes in buildings are originated various sizes cracks. At subsequently repeated impacts the crack quantity increases, simultaneously decreases the building's stiffness and as result is decreased the value of seismic load.

But if we know that we can reduce a seismic impact by reducing of the building's stiffness, therefore, in our opinion in such way would be saved a building.

That fact prompted scientists to begin search of seismic protection systems.

Over the past 20-25 years has created implemented in practice methods: seismic isolation systems that are widely used in our country, as well as in foreign countries.

At repeated less than design impacts on building, the periodicity of that is more frequent, the damages are accumulated that reduces the design seismic resistance and makes it unable to withstand the earthquake impact.

This condition is detected by observing of facilities that are located in a zone of frequent earthquakes. They are reporting on the devastating affects to a lesser magnitude earthquake.

This condition occurs at the experimental vibrational tests.

From the above stated discussion we can made one conclusion: the calculated and designed accordingly of normative regulations structure, in which at the design was considered the appropriate price increases, are aimed at only one design earthquake or two, relatively low intensity earthquake perception, after that the building would be reinforced due constructive action or to be demolished, and construct new building.

Both are related to the large labor expenses, the financial amounts and time that is always creating a large problems, especially in residential construction.

As a result of these circumstances we have today due many scientists search we have a number of system that could save or dramatically reduce buildings damages.

In many being under seismic impact countries are using active and passive seismic protection structural schemes: seismic isolation and seismic damping.

In the active seismic protection actions are involved the additional energy sources and elements, due that is done to regulate these sources operation, but their realization requires considerable expenses in their design and operation. This circumstance excludes the wide application of active seismic protection in construction industry.

The seismic protection passive systems include seismic damping and seismic isolation facilities. Here the additional sources of energy are not used.

The seismic damping systems include dampers and dynamic attenuators. There the mechanical energy of vibration structure is transferred to other kinds of energy that leads to a oscillation damping or redistribution from protected structure up to fully damping.

In the seismic isolation systems is provided the reduction in mechanical energy that the structure receives from foundation, by the avoiding of building oscillation frequency from the impact of excessive frequency.

There are adapted and stationary seismic isolation systems.

In the adapted systems dynamic characteristics of building irreversibly change in the earthquake process, are "matching" the seismic effect.

In the stationary systems at earthquake process remains the dynamic characteristics.

In seismic protection systems most widely are distributed the seismic isolation foundation foundations, which are quite common in the former Soviet republics (now independent countries), as well as in earthquake proof engineering in foreign countries.

This type of seismic isolation foundations are used in France by "Chpie atignolle" and "Electritsite de France" firms.

The support that occupies the upper base slab is consisting from friction slabs that are reinfrced by neopren (elastomer) gaskets. On the bottom foundation slab are supported the concrete frames. The stiffness of supports in vertical plane near about 10 times is greater than stiffness in the horizontal direction.

The "Chpie atignolle" system represents a classic example of seismic isolation foundations, in that step-by-step are arranged the elastic and damping elements.

At influence of relatively small oscillations when the horizontal load on support surface is not exceed the friction force, the system behavior is in the linear field.

At stress increasing the friction force will be overcome and the upper basement slab will slide related to bottom slab. At the same time we have the opportunity in several times to reduce the load on the building and installed on it device.

Wide are spread so-called KF- type frames, due application of them constructed buildings undergoing the 8-magnitude intensity earthquake impact on MMSK-64 scale.

Accepted for the multi-story construction in seismic areas type of structural systems, including metal structures, is related first of all, to the searches of rational systems of buildings that will be able to protect the residents from strong earthquakes impact and respond to the suitable structural system of building’s functional destination.

Below we discuss on seismic protect ion compression supports that represents the rubber - metal and rubber - plastic elements. Such supports used in France, New Zealand, Japan, the United States, Italy and now in Armenia.

Despite on some structural differences, mainly their solutions are identical. This is an alternation of rubber and metal sheets that are placed between the supporting metal plates that have holes with arranged in them anchors. They are fixed to the supporting structures of building.

In some cases, in these joints are included also Teflon (fluoroplastic) gaskets.

In order to avoid from exceed deflection due loading form building’s own weight, loading of the structure, supports are performed as rigid in vertical and flexible in horizontal planes. In addition, in order

to provide flexible lateral motion, they have a small in few times stiffness in the horizontal plane in comparison with stiffness in the vertical plane.

Due to the elastic properties of rubber, the metal-rubber supports are characterized by high strength in compression, tension and torsion.

But for their production is required high-quality, expensive rubber.

In addition, due the compaction features of rubber in cold the rubber - metal supports changes its physical - mechanical (elastic) properties. Also, due the polymers "aging" phenomenon, when the polymers change their physical - mechanical properties, as some scholars point out, their life time is in the range from 20 up to 50 years. This is clearly insufficient for operation of such objects.

Due the dismatch in center of gravity and center of symmetry occurs the torsion phenomenon caused by the spatial, dynamic actions of seismic impact.

This phenomenon causes the overload of extremal seismic isolation supports. Therefore, it is necessary to implement some special compensatory measures to protect the building.

The manufacturing of rubber – metal multilayer frames is easy because of their simplicity. At design of buildings the characteristics of frames was easily selected by determination of gaskets thickness and number.

The application of seismic isolation supports rubber - metal elements significantly increases the damping of oscillation and almost 2 times reduces the reaction of building at seismic impact, 2-3 times are reduced its basic period of oscillations.

Below are presented constructive examples of several types of most widespread seismic protection compression supports.

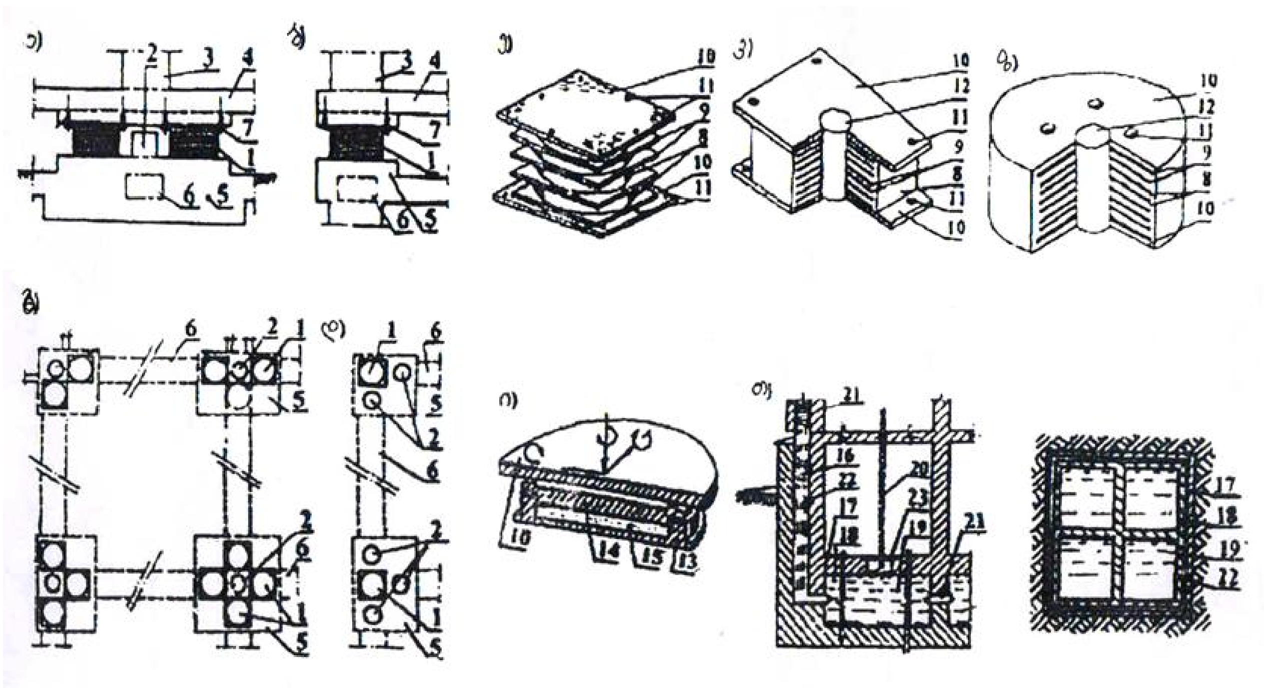
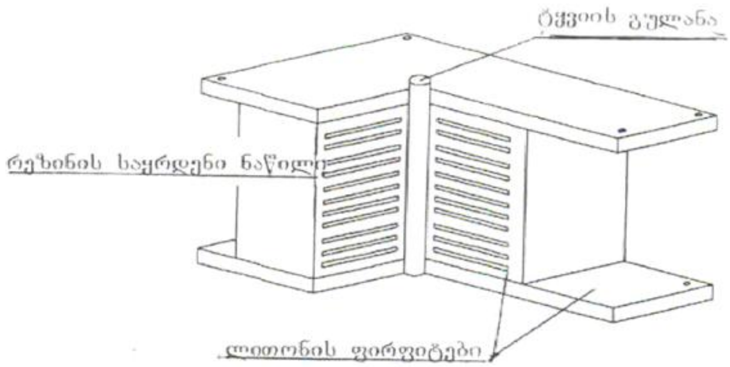


Fig. 1. seismic protection compression supports constructive examples rubber - metal and rubber - plastic base

Support6s arrangement layouts: a)on foundation slab, b)on arranged underground - columns capitals, c) and d) arranged on plan options: in different countries are used variants : a)France , f) in New Zealand , i) in Italy, g) in Japan, h) Copyright certificate on invention № 327296 : 1) support ; 2) alternating fence ; 3) column ; 4) covering slab , 5) foundation slab , 6) foundation beam , 7) anchor little screw ; 8) rubber layer 9) metal inner sheet, 10) metal retaining plate 11) building construction poles mounted holes , 12) lead or rubber Gulani , 13) metal building, 14) metal cover, 15) Pottery glazed rubber disc; 16) reinforced concrete foundation ; 17) hollow cups ; 18) elastic waterproof tank ; 19) liquid , the working environment, 20) vertical pipe; 21) controlled valve system ; 22) flexible gaskets; 23) observation window



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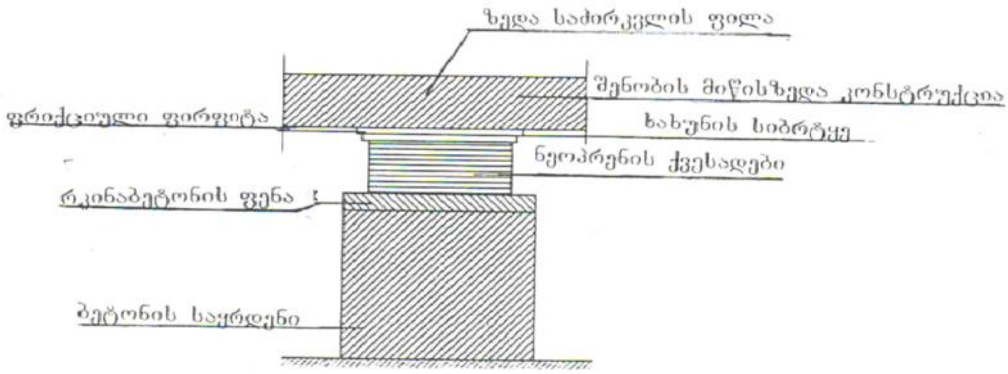

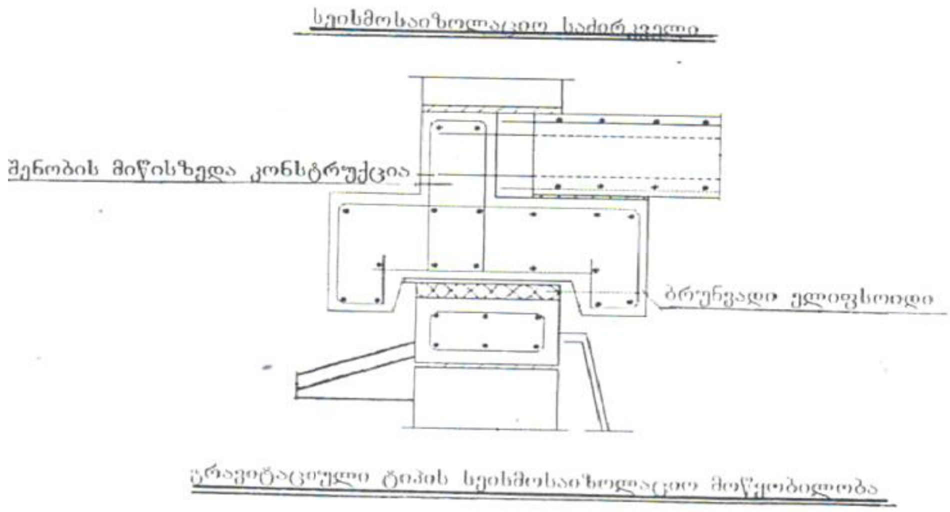

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29-30 May 2014, Tbilisi, Georgia

RESEARCH OF TRANSVERSE VIBRATIONS OF BUILDING AS DISCRETE-CONTINUAL SYSTEM WITH CONSIDERATION OF CAUSED BY SHOCK EFFECT PULSE IMPACTS (EARTHQUAKE, BLAST, ETC)

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Abstract: *In the paper are considered longitudinal and torsional oscillations at earthquakes as well as values of strains and forces in structures at seismic impact. Examples of two, five and sixteen-storey building design and measurements to reduce the seismic impact are considered. Are also considered the impact of viscosity coefficient change, as well as in inertia of rods, rods rotation angles and torque.*

Keywords: oscillations; inertia; torque; seismic resistance; viscosity.

INTRODUCTION

In the paper is considered the issue of study of influence of caused due earthquake seismic effect, as impact effect, on frame building as a discrete - continual system, the study of the influence.

Is stated the analysis of applied at structures calculation of the earthquake impact of design models and methods of analysis. Are mentioned out the contradictions, containing by them, and is concluded that at calculation of seismic impact it is necessary to take into account the impact effect.

As frame building design scheme was accepted the discrete-continual system, where the discrete masses are connected accordingly to Calvin - Vogt model by deformable longitudinal rods. In the paper are considered the influence of longitudinal and torsional oscillations in the impact effect conditions. The applied differential equations of oscillations, in contrast from

energy method, are obtained along the rod due velocities arbitrary type distribution (linear or any other dependence) without permissions.

The solution algorithm of oscillation simultaneous differential equations and program for arbitrary number of equations (i.e., number of storeys) is developed. On the example of specific building are studied: impact of rods masses with taking into account the inertia, viscosity coefficient and influence of certain discrete mass changes on building mode of deformation. The results are presented as diagrams of oscillations and were made the conclusions.

BASIC PART

In many countries worldwide in order to ensure the seismic resistance of buildings the development of methods of calculation and creation of normative documents was started in the first quarter of the twentieth century.

The main design models and methods are grounded on the old approach that implies a calculation on constant static load. Due this are stipulated existing up to day contradictions in the seismic resistance standards, although these contradictions are gradually moderated by various coefficients contradictions and corrections.

Basic methods for evaluation of seismic resistance and calculation of seismic load still are implies the methods of calculation on static loads, mainly horizontal that is considered as equivalent of seismic load.

Initially, these static loads were determined as the inertial forces that are equal to product of the building mass on seismic acceleration of the ground. This approach is called as a static method of calculation.

At the early fiftieth of twentieth century, in the U.S. and the Soviet Union norms almost simultaneously at determining of inertial forces instead of the ground acceleration have begun the consideration of construction of acceleration of the masses into account. These accelerations will be accepted by multiplying of ground accelerations on dynamic spectral coefficient that was varied according to the period of oscillation in the range of 0.6−0.8 (soft grounds) up to 2.5−3 (natural grounds).

The norms, codes and standards of all countries, relating to the calculation methods of structures on seismic loads, are based on the same principles. The difference is only in the details, or values of coefficients.

For seismic areas the calculation of most structures was carried out a comparison of acting, basically horizontal load that includes seismic loading, with the load carrying capacity of structure. The seismic resistance criteria shall be considered as satisfactory if the load carrying capacity is equal to the product of seismic load on safety ratio.

Actually in all countries seismic codes the seismic load is defined as the product of building mass and acceleration divided on the certain reducing ratio. This ratio that in the United States and in some other countries codes is called as reduction factor depending on structure's non-elastic behavior and was changing in accordance of structure's type in the range from 1.25 up to 6-8. If we compare it with values of relevant coefficient in our standards, we see that the reduction of loads occurs 3 - 4.5 times or 6-9 times less for responsible structures, i.e. the calculation of real building was carried out on several times less value of seismic loading that would have taken accordingly of accepted from the seismic zoning map acceleration [1].

At the same time a design mathematical model of building consider the structure as a linearly elastic system, and the analysis on strength and stability was carried out on static load action.

The reduction of seismic intensity value corresponds to the less on 1-2 magnitude of earthquake of decrease for each of construction sites.

Thus the recommended by norms calculation procedures in most cases provide the building's safety only at weak earthquake. While the standard's compilers and planners consider that due designed in such way building's safety is provided even at strong earthquake. This is ground on currently obtained following assumptions:

1. With consideration that strong earthquakes occurs relatively rarely, the localized damages, cracks and non-elastic deformations that do not cause the global destruction, are considered as necessary and acceptable [1, 2, 3, 4, 5, 6, 7].

2. The above mentioned methods of calculation are based on two hypothesis: a) the maximum horizontal strains with equivalent frequency of elastic and non-elastic systems are same [8]; b) non-elastic deformations defined grounded on the first hypothesis are acceptable and not dangerous in terms of total collapse.

The analysis of occurred in recent years strong earthquakea, such as San-Fernando, USA, 1971, Spitak, Armenia, 1988 [9], Kobe, Japan [10], Neftegorsk, 1995 [11] and others showed that anti-seismic calculation concepts and hypotheses that underlie them, can not be considered as proper and universal.

The first hypothesis that is related to the equality of elastic and non-elastic systems maximum horizontal displacements did not confirm by analysis of the seismic response spectrum that are constructed on the basis of strong earthquakes instrumental recorded accelerations. In some cases, the maximum horizontal seismic displacements were found to 2-3 times higher than the maximum displacement of elastic systems. For example, based on the 1985 Mexico City earthquake records due jointly constructed by Istanbul Technical University and the Kucherenko University Central Scientific Research Institute displacements diagram shows that the non-elastic displacements are hundred times greater than the and elastic displacements accordingly of first hypothesis [12, 13]. In the case of other significant earthquakes are same inadequacies.

Only this inadequacy is sufficient to ensure that the accepted on anti-seismic design norms calculation procedures will be considered as insufficiently grounded and despite the fact that in some cases are obtained satisfactory results, they principally are unable to provide seismic resistance of structures.

If the actual strains are exceeded up to ten or hundred times more than expected, is obvious that they are dangerous and with vertical load they would cause a total collapse.

As in noted above, the formal criteria of structure's seismic resistance represent the comparison of seismic load with load carrying capacity. Such criteria at static load are acceptable, but in alternating dynamic load case the exceeding design load does not mean the destruction. These criteria are accepted from the old approach of static load case except that now the acring force is horizontal.

The applying of inertial forces on the elastic system really is a step forward, but it can not explain the fundamental contradictions that take place in buildings of earthquake proofing design applying theory. These are:

1. By all countries specialists is accepted the concept of design in seismic areas, accordingly of that at maximum intensity earthquake are acceptable significant damages that does not cause in human deaths and the destruction of a unique spiritual and social values. At the same time the calculations are performed assuming that we do not have damages. The existence of small non-elastic deformation is accepted in some indirect ways.

2. In calculations is assumed that the destruction of structures occurs due horizontal seismic load (more precisely, due the horizontal component of seismic load).

In most cases, the destruction is caused due the seismic impact the damaged building by action of vertical forces. If we consider the influence of the vertical load component that causes

the longitudinal vibrations of building, will become more visible existing contradictions in the modern theory of seismic resistance.

The stated review gives the possibility to conclude that at earthquake there is an impact effect and it is necessary to consider its effect at frame buildings calculation.

Is stated the issue and its solution ways. Are analyzed basic principles and concepts of the traditional seismic calculation. On existing in them contradictions are mentioned. Is stated the concept accordingly of that the buildings are damaged not by caused by the earthquake harmonic oscillations but by impacts. Are referred the works in that was recorded the revealing of impact effect of earthquake. On collapse gravitational model is said that a consideration of vertical component makes more real the viability of this model. It is noted that the identification of possible areas of impact effect represents the surrounding area of tectonic destruction. On the way to resolve this issue is considered the possible impact effect in conditions of structure adequate consideration design scheme. As such scheme is accepted a discrete - continued scheme and is stated a brief overview of the current methods of calculation.

The building as discrete - continual system longitudinal vibration simultaneous differential equations is obtained. The discrete - continual system represents concentrated masses with interconnected deformable rods. The simultaneous differential equations stipulate the impact of rods masses on oscillation process. At the same time, in contrary of energy methods at obtaining of these system is not applying the arbitrary assumption of velocities distribution along the rod (linear or other dependence).

Is stated the building as a discrete - continual system torsional vibrations simultaneous differential equations as it is done for the longitudinal oscillations.

Are constructed the solutions of fluctuation simultaneous differential equations and is developed the corresponding program. The solutions are constructed due numeric way, by application of corresponding Runge - Kutta method the standard program. In order to apply this program is developed the step-by-step approximation method, in that for each mass, for that is written down the equation, the accelerations of adjacent masses in each step of approximation are considered as known from the previous approximation. The program has been developed for arbitrary number of masses. Are stated the initial conditions of task and the law of changes external of influences. For considered specific case are stated dependencies for five concentrated masses.

Are stated the calculation results and their analysis. The calculations are carried out for a having specific geometric dimensions and mechanical characteristics four-storey (with five concentrated mass) building. Is studied the impact of its rods masses on structure’s mode of deformation when the material is subject of Young or Calvin - Vogt rheological model. Also is investigated the influence of initial velocity and viscosity coefficient. The results are obtained as vibration diagrams and are made relevant conclusions.

CONCLUSIONS

1. At longitudinal and torsional oscillations the ignorance of rods masses inertia reduces the values of strains and forces approximately up to 9-10.
2. Arisen in structure forces and strains are proportional for initial velocity, but they are not proportional to the impact force when we have the initial velocity.
3. The displacements and forces are not changed in proportion to the viscosity coefficient. Its increasing again causes more intensive reduction of forces and strains than the process of its further increasing. In the considered case of longitudinal oscillations the taking into account of viscosity causes accordingly the reducing in masses from the first up to fifth from 13 up to 36%, and the forces from 19 up to 41 %.
4. In the case of torsional vibrations the maximum rotation angle we have for fifth mass, while maximum torque in the first rod, with taking into account the rods masses as well as without it.
5. With taking into account the viscosity maximum reduction of rotation angle (46 %) we have in the fifth mass, for bending moment in the adjacent rod (in 2.5 times).
6. By the reducing in first mass moment of inertia the maximum reduction in the angle of rotation we have for fifth mass (approximately 42 %) with taking into account a viscosity coefficient, as well as without it.
7. The impact of viscosity coefficient changes on rotation angles on the torsion case is maximal for a fifth mass (47 %). The impact of this change is more intense moments for torque and cause it’s several times reduce (for I masses - 3 times, for fifth masses - 5 times).
8. The influence of second mass moment of inertia to reduce the angle of rotation is insufficient and causes increase in all the masses angle of rotation near about 7%. As for the first rod force is reduced, while in others is increasing.

9. Therefore at longitudinal and torsional oscillations essential is influence of rod mass inertia, as well as are also more likely to consider the rheological properties that is necessary calculation.

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29-30 May 2014, Tbilisi, Georgia

STUDY OF NON-LINEAR OSCILLATION OF TOWER BUILDINGS CAUSED BY PULSE DISPLACEMENT OF GROUND WITH CONSIDERATION OF PHYSICAL NON-LINEARITY OF MATERIAL

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INTRODUCTION

As it is known the character of seismic impacts depends on distance from epicentral zone. The displacements of ground have the pulse character in immediate vicinity of epicentral zone. In the works [1, 2] are analyzed the cases of impact on buildings, located directly on breaks, as well as located in adjacent of tectonic break: records, made in Taiwan, in Kobe, Loma, Prieto, Nortrige and Waitiere (USA). Grounded on the analysis is mage conclusion that qualitative the intensity of oscillations up to 1.5÷2.5 times exceeds the intensity at 9 magnitude seismicity that indicated on special approach, including consideration of impact effect.

Also would be taken into account that in recommendations on design of tower buildings is indicated on necessity of definition and limiting of non-elastic deformations.

Thus, is arisen an issue on non-elastic behavior of structures at impact on level of base of tower building.

In the work is considered the issue of study of non-linear oscillation of frame buildings as discrete-continual systems, when structure is undergoing to impact of pulse displacements of ground and connecting the concentrated masses rods are working on shear accordingly of Prandtl scheme.

BASIC DEPENDENCIES

Lets consider the located on single axis oscillation of set of masses, caused due horizontal pulse YF displacement of ground.

Simultaneously let's consider that force S_k is applied in the center of gravity of mass. Therefore, the mass motion is stipulated only due force S_k that represents reaction of rod and depending from state of rod (elastic or plastic) would be depending on displacement linearly (in the case of elastic state) or non-linearly $S_k=F(y)$ (plastic state).

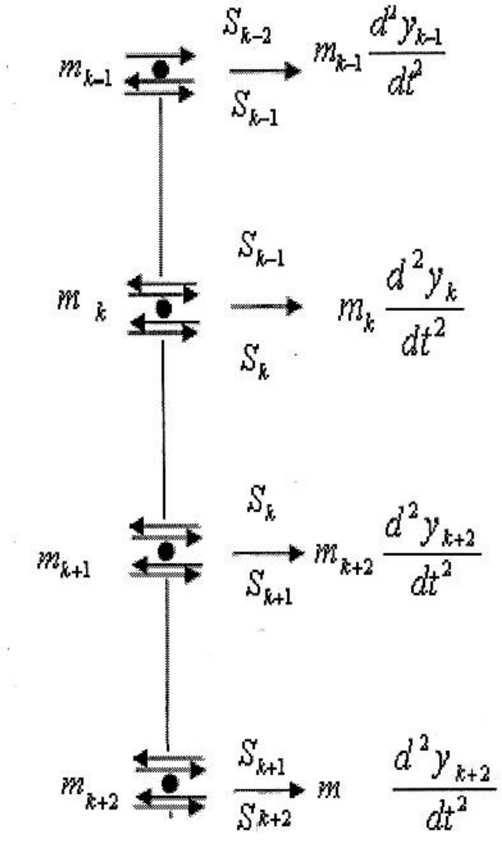

Fig. 1. Design state

Let's divide the oscillation time on small Δt sections and on each section let's consider S as constant $S_k=\Sigma S_i$ [3, 4]. Let's consider arbitrary mass m_k. The differential equations of its motion will be

$$m_k\frac{d^2y_k}{dt^2}+S_{k-1}-S_k=0. \tag{1}$$

If we represent each force as summand

$$S_k = \sum_{i=1}^{I-1} S_{k,i} \tag{2}$$

and carry out the integration of equation at zero initial conditions, we obtain

$$y_k = -\sum_{i=1}^{I} \frac{(t_I - t_{j-1})^2}{2m_k} S_{k-1,i} + \sum_{i=1}^{I} \frac{(t_I - t_{j-1})^2}{2m_k} S_{k,i}$$

As it was mentioned above, the rod is working only on shear, thus its elastic displacement with taking into account of shear deformation will be $\frac{S_k \ell}{GF}$, where G is the shear modulus, and F – is thereof rod cross-section. It is obviously that in oscillation process, when S_k reach the certain value $S_{\lim}$ the end of rod will have the maximal elastic displacement $\frac{S_{\lim} \ell}{GF}$ and any plastic displacement Δ_{pl}.

The condition of continuity of displacement between masses m_k and m_{k+1} for first interval of time Δt will be written down as:

$$-S_{k-1,1}\frac{\Delta t^2}{2m_k} + \left(\frac{\Delta t^2}{2m_k} + \frac{\Delta t^2}{2m_{k+1}} + \frac{l}{GF}\right)S_{k,1} - S_{k+1,1}\frac{\Delta t^2}{2m_{k+1}} = 0$$

For second:

$$-S_{k-1,2}\frac{\Delta t^2}{2m_k} - S_{k-1,1}\frac{(2\Delta t)^2}{2m_k} + \left(\frac{\Delta t^2}{2m_k} + \frac{\Delta t^2}{2m_{k+1}} + \frac{l}{GF}\right)S_{k,2} + \left[\frac{(2\Delta t)^2}{2m_k} + \frac{(2\Delta t)^2}{2m_{k+1}} + \frac{l}{GF}\right]S_{k,1} -$$
$$- S_{k+1,2}\frac{\Delta t^2}{2m_{k+1}} - S_{k+1,1}\frac{(2\Delta t)^2}{2m_{k+1}} = 0$$

For arbitrary i rod:

$$-S_{k-1,I}\frac{\Delta t^2}{2m_k} - \sum_{i=1}^{I-1} S_{k-1,i}\frac{(t_I - t_{i-1})^2}{2m_k} + S_{k,I}\left[\Delta t^2\left(\frac{1}{2m_k} + \frac{1}{2m_{k+1}}\right) + \frac{l}{GF}\right] +$$
$$+ \sum_{i=1}^{I-1} S_{k,I}\left[(t_I - t_{i-1})^2\left(\frac{1}{2m_k} + \frac{1}{2m_{k+1}}\right) + \frac{l}{GF}\right] - S_{k+1,I}\frac{\Delta t^2}{2m_{k+1}} - \sum_{i=1}^{I-1} S_{k+1,i}\frac{(t_I - t_{i-1})^2}{2m_{k+1}} = 0$$

This expression is right, if the rod is elastic. If the rod is plastic, then is added

$$\Delta_{k,pl} = \Delta_{k,l} + \sum_{i=1}^{I-1} \Delta_{k,i}$$

If we add them and transfer in the left part only terms, including $S_{k,I}$ and $\Delta_{k,i}$ we obtain

$$S_{k,I}\left[\Delta t^2\left(\frac{1}{2m_k}+\frac{1}{2m_{k+1}}\right)+\frac{l}{GF}\right]+\Delta_{k,I}=\sum_{i=1}^{I}S_{k-1,I}\frac{(t_I-t_{i-1})^2}{2m_k}-$$
$$-\sum_{i=1}^{I-1}S_{k,I}\left[(t_I-t_{i-1})^2\left(\frac{1}{2m_k}+\frac{1}{2m_{k+1}}\right)+\frac{l}{GF}\right]+\sum_{i=1}^{I-1}S_{k+1,i}\frac{(t_I-t_{i-1})^2}{2m_{k+1}}-\sum_{i=1}^{I-1}\Delta_{k,i}$$

With taking into account the following designations

$$tl_1(n)=\frac{(n\times\Delta t)^2}{2m_1}+\frac{(n\times\Delta t)^2}{2m_2}+\frac{l_1}{GF}\qquad tl_2(n)=\frac{(n\Delta t)^2}{2m_2}+\frac{l}{GF},$$

$$Dlm(n)=tm_2(n)\cdot S_{21}+tm_2(n-1)S_{22}+....+tm_2(2)S_{2,n-1}-tl_1(n)S_{11}-tl_1(n-1)S_{12}-....-tl_1(2)S_{1,n-1}$$

$$Tlm(n)=YF(n)+tm_2(n)S_{11}+tm_2(n-1)S_{12}+....+tm_2(2)S_{1,n-1}-tl_2(n)S_{21}-tl_2(n-1)S_{22}-....-tl_2(2)S_{2,}$$

$$tm_2(n)=\frac{(n\Delta t)^2}{2m_2},\qquad tm_1(n)=\frac{(n\Delta t)^2}{2m_1},\ \mathrm{FL=l/GF}$$

we obtain

$$S_{k,I}\left[tm_k(1)+tm_{k+1}(1)+FL\right]+\Delta_{k,I}=S_{k-1,1}tm_k(I)+S_{k-1,2}tm_k(I-1)+...+S_{k-1,I}tm_k(1)-$$
$$-S_{k,1}\left[tm_k(I)+tm_{k+1}(I)+FL\right]-S_{k,2}\left[tm_k(I-1)+tm_{k+1}(I-1)+FL\right]-...-$$
$$-S_{k,I-1}\left[tm_k(2)+tm_{k+1}(2)+FL\right]+S_{k+1,1}tm_k(I)+S_{k+1,2}tm_{k+1}(I-1)+...+$$
$$+S_{k+1,I}tm_{k+1}(1)-\sum_{i=1}^{I-1}\Delta_{k,i}$$

For equation written down for lower rod is added the displacement of ground YF.

The obtained simultaneous equations are solved by step-by-step approximation. For each step we gives the certain (zero) values to magnitudes, that will be defined in the right part, calculation the new values and compare with previous values; when the difference will be less than required accuracy, we transit on next step. Simultaneously for each step in each rod will checked, neither exceed the obtained force the ultimate values. If occurs the exceeding, the force gradient will be equal to zero and instead it will be defined the plastic deformation accordingly of above mentioned formulae.

RESULTS OF CALCULATIONS

The calculation are carried out for study of non-linear oscillation of building, when the load bearing structure on each stage represents the metal columns with interval 6 m in both directions. The columns consist from two U-beams № 20a that are covered by reinforced

concrete slab 20×20 m with thickness of 16 sm. The height of columns- 4 m. The yield point on shear τ_s=1600kg/sm², the area of U-beams cross-section – 800 sm². The mass of reinforced concrete slab makes up to 15 000 kg. The basis of building as result of earthquake is subjected to pulse displacement, $W=ate^{-\beta t}$, where a - is the initial velocity; $\frac{1}{}$ - is the time, during that the displacement reaches to maximum.

The calculations are carried out for two, five and sixteen-storey buildings.

In the case of two-storey building is studied the influence of change of mass values on magnitudes of elastic and plastic strains, as well as values of arising forces (Fig. 2 and 3).

In the case of five-storey building is studied the influence of repeated pulse (Fig. 4 and 5).

In the case of five-storey building is studied the influence of change of stiffness on elastic-plastic strains and the values of arising forces (Fig. 6 and 7). The Fig. 6 corresponds to building, the stiffness of that is constant on the total height. The Fig 7 corresponds to building, the stiffness of that is constant on the in the range of four storey's and is reducing in downward direction.

It should be mentioned that in the case of mi-mass system the oscillations of upper point starts from the point 0 on the axis of abscissa and the oscillation of lower point from the point, corresponding the value of 400. Similarly are compiled the diagrams in the case of five and sixteen points.

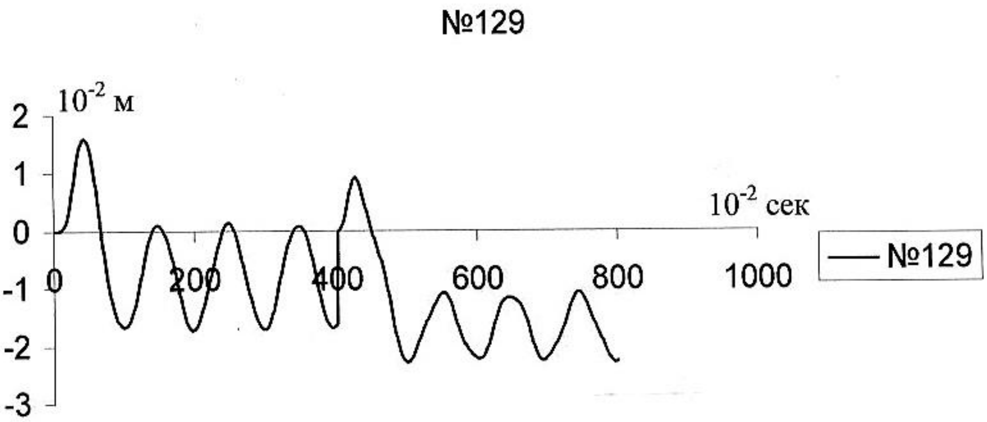

Fig. 2. Strains of elastic-plastic system

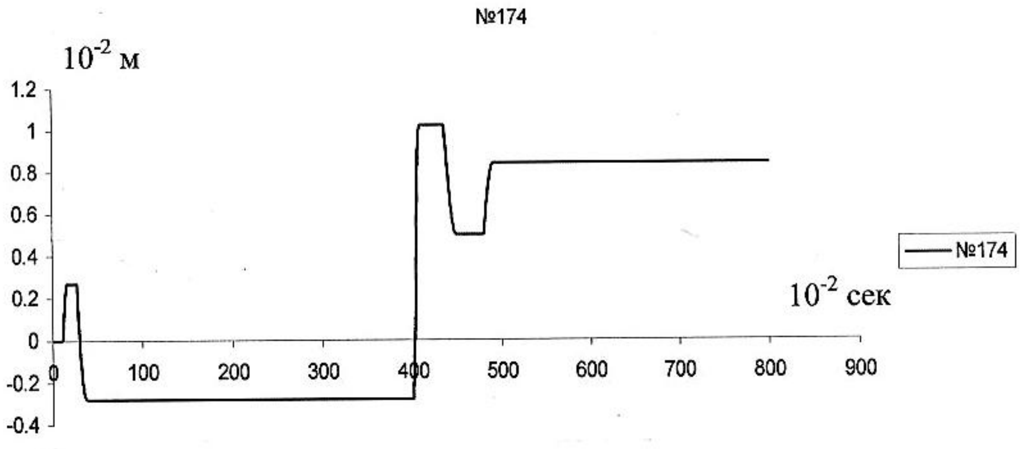

Fig. 3. Plastic strains in rods

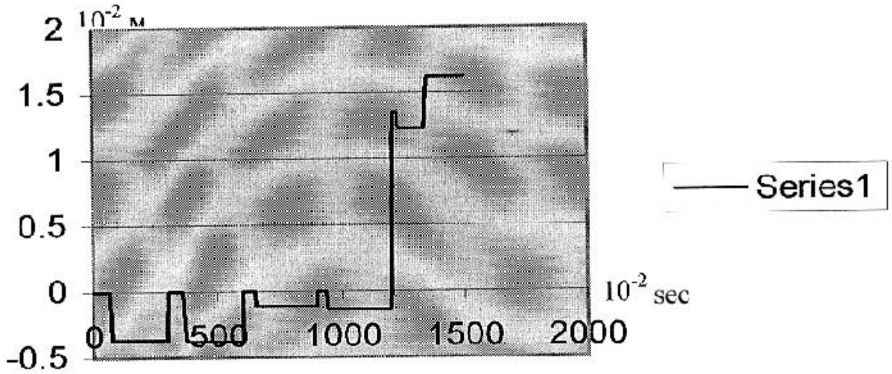

Fig. 4. Plastic strains in the case of five storey's

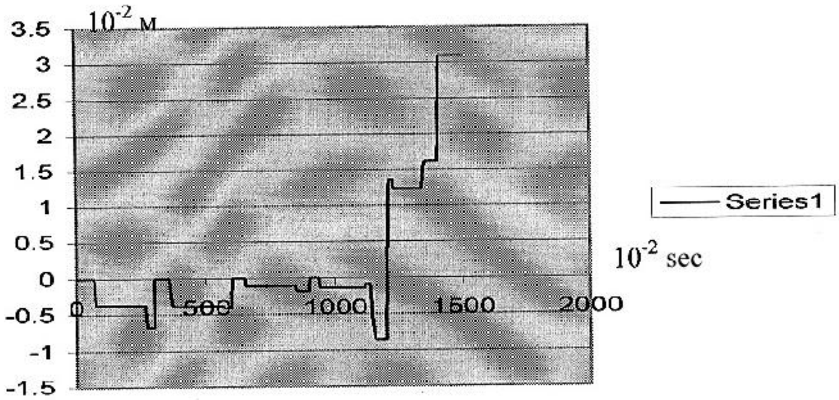

Fig. 5. Plastic strains at repeated pulse

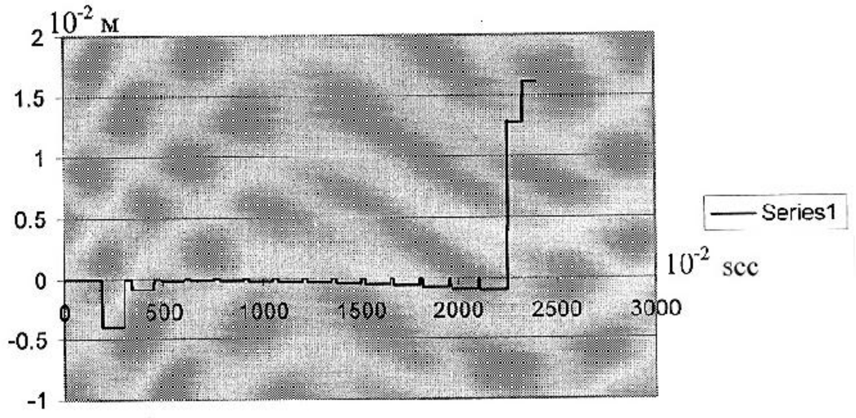

Fig. 6. Plastic strains at constant stiffness

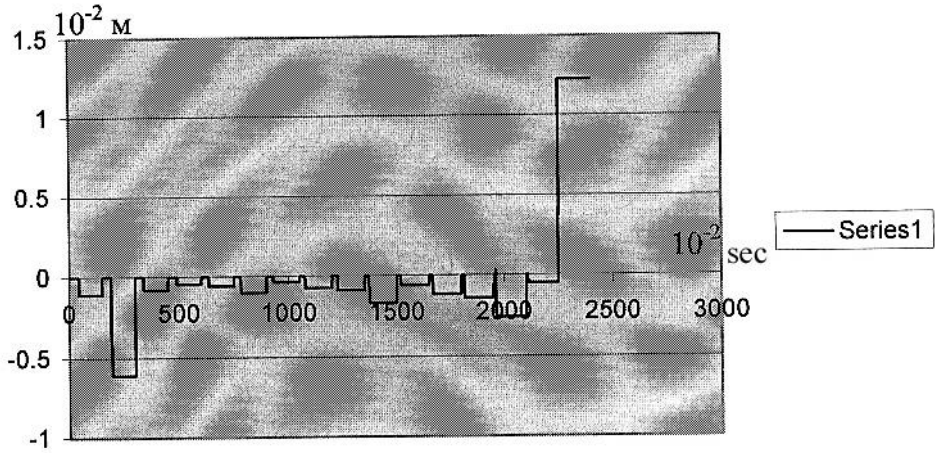

Fig. 7. Strains at variable stiffness

CONCLUSIONS

1. In the case of pulse impact the strains of elastic-plastic system is less than oscillations of elastic system (if there is not the plastic district) [5].

2. In the case of elastic-plastic system after termination of pulse impact the oscillations occurs related the residual plastic strain.

3. In the case of pulse impact the same acceleration would be caused due the various velocities, to that corresponds the values of strains and forces. From this on area, in adjacent to tectonic break, where takes place the impact as pulse, as seismic regionalization is advisable to consider not the acceleration, but the velocity.

4. In the case of reputed impact, depended on which moment of oscillation occur the repeated impact, the results will be varied. From physical laws, the force would not exceed the ultimate value but it would be significantly reduced. While the strains that are not limited, would be increased significantly.

5. In the case of bi-mass system the tree-times decreasing of lower mass almost don\t change the maximal strain of upper mass, but it reduce the plastic strain nearby in three times, relating of that is continuing the oscillation. The strain of lower mass is increasing on ≈30%, and plastic strain in reduced in half. In the case of three-times decreasing in upper mass, the strain almost twice is increasing for both masses. In the case of elastic oscillation the change of mass leads to significantly distinguished results. In the case of three times mass reducing the strain is reduced for upper point in ≈2.2 times, while for lower mass – in ≈2.4 times. The next oscillation of upper mass occurs with reduction in amplitude in three and more times, while for power mass - with reduction in amplitude in four times. In the case of reducing of upper mass in three times, the maximal amplitude at pulse action is reduced in three times for upper mass and in two times for lower mass. The next oscillation of upper mass occurs with reduction of amplitude in four times, while for lower mass – with reduction of amplitude in five times. The forces in upper rod are almost twice reduced, and in lower rod – in four times.

6. In the case of five-storey building, when α=50 and β=8, in all five rods takes place the plastic strains. The maximal strain is in lower rod (1.62 cm), in two upper rods it is significantly less (0.37 cm) and more less in the third and fourth rods(0.11 and 0.14 cm). The maximal strain in three lower rods is almost uniform (0.3 cm) and rather less in two upper rods (1.9 and1.8cm).
The comparison of these values with elastic oscillations gives the principally various results. The maximal strains are in two upper rods, and they almost twice exceed the elastic-plastic strains.

7. The influence of repeated impact is significant for elastic oscillations in comparison with elastic-plastic ones.

8. On the same impact were studied sixteen-storey buildings in the case of various distribution of stiffness: when the stiffness is always constant, it is constant in the range of four storeys and is changing on height due linearly and parabolic laws. In the process of pulse action the strain is more in the case of building with more stiffness.

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29-30 May 2014, Tbilisi, Georgia

RIKOTI TUNNEL OPERATIONAL PROBLEMS AND SEISMIC STABILITY

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1. Background

During construction of Rikoti transport tunnel treatment with excess of massif occurred. As was revealed by engineering research, between the outside perimeter of lining structure and surrounding massif huge gaps (gaps) were detected, including the ones along the whole length of arched, also on both sides of walls with various frequency.

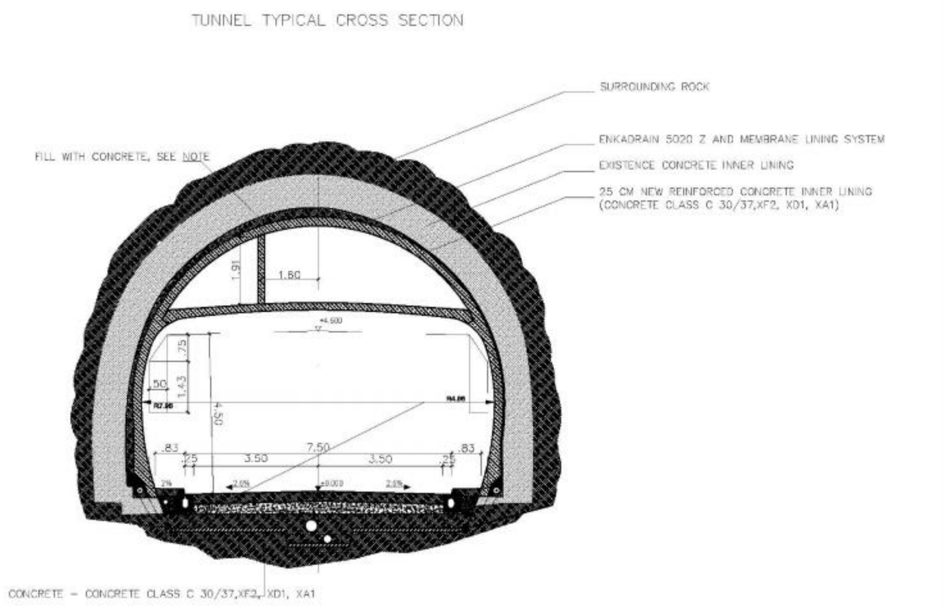

Fig. 1 Rehabilitation solution of tunnel by strengthening lining.

The existence of gaps between lining structure and massif created especially big problems in terms of both lining stability and subsequent development of geological fractures and cracks existing in massif. It should be noted that the seismicity of region reaches up to 8 points.

The rehabilitation engineering solution stipulated filling gaps behind lining and constructing a 25-cm reinforced belt on the inside (Fig. 1). In connection with processing the

rehabilitation option the analysis of the seismic stress state of system „lining-massif“ was performed.

2. Calculation model

Calculations were carried out with the use of a two-dimensional quasi-static finite-element model (FEM), where the following assumptions were taken [1,2].

1. Seismic waves:

-are elastic, harmonious and propagate in a flat front;

-are represented by superposition of flat waves spread in various directions.

In this connection for obtaining extreme stresses in lining the combination of two problems was considered: a) compression-stretching waves and b) long seismic shear waves.

In the first problem compressing stresses are defined by formulae:

$$\begin{aligned} \pm_x(\infty) &= -p \\ \pm_y(\infty) &= \P\, p \end{aligned} \tag{1}$$

In the second problem long shear stresses are defined by formula:

$$2^{\infty}_{xy} = -Q\ , \tag{2}$$

where:

$$p = \frac{1}{2®} A \cdot K_1 \cdot °_0 \cdot c_1 \cdot T_0\ ; \quad \P = \frac{\neg_0}{1-\neg_0}\ ; \quad Q = \frac{1}{2®} A \cdot K_1 \cdot °_0 \cdot c_2 \cdot T_0\ ;$$

A - a coefficient, the value of which depends on seismicity; K_1 - a coefficient, which takes into consideration local faults of structure; ρ_0 - strength of ground; $C_1$and C_2 – velocities of longitudinal and shear seismic waves propagation, respectively; T_0 – the primary period of ground vibration.

2. The length of shear waves exceeds the dimensions of cross-section of tunnel not less than three times. To satisfy the latter it is necessary that the following condition must be fulfilled according to the deformation characteristics of massif:

$$D \leq T_0 \cdot \sqrt{\frac{E_0 g}{2\rho_0(1+\nu)}} \tag{2.3}$$

where: Dis the largest diameter of tunnel; g – free fall acceleration; ρ_0, E_0, υ - strength of ground,mudulus of elasticity, and Poisson ratio, respectively. The indicated condition according to the characteristics of tunnel massif is satisfied with high margin.

3. The system „lining-massif" operates as linearly deformable till the time when there occurs a fracture of lining or contact between lining and massif. In this connection two calculation events were considered dealing with strength of the contact during the propagation of seismic waves in massif:
   - the contact maintains strength. In this case lining works together with massif also when there exist tensile stresses at contact(linear behaviour);
   - the contact fails to maintain strength at stretching $\sigma_r > [\sigma_r]$, (where σ_r tensile radial stress and $[\sigma_r]$ permissible tensile stress of contact -$[\sigma_r] = 200 \div 300$ kN/m^2). In this case in contact appears „tensile crack".
   - the contact fails to retain strength in shear. In this case radial stresses are insignificant, and there develop shear stresses of high value, in other words, it approaches the case of pure shear.

The first version is the condition of worse. At this time extreme value stresses (compressing and stretching) will develop in lining, and, accordingly, constructing will be performed with high margins.

During the second and third versions at the simultaneous propagation of compression-stretching and shear waves (the probability of such combination is likely), as a result of the development of extreme stresses a shear crack appears, which may increase even more extreme shears in lining structure.

Thereby during calculation for obtaining the most unfavorable condition of lining construction four calculation alternatives were considered, which took into account the simultaneous combination of compression-stretching and shear waves:

$$\begin{cases} \sigma_1 = \sigma_c + \sigma_{sh} \\ \sigma_2 = \sigma_c - \sigma_{sh} \\ \sigma_3 = -\sigma_t - \sigma_{sh} \\ \sigma_4 = -\sigma_t + \sigma_{sh} \end{cases} \qquad (2.4)$$

where, σ_1, σ_2 represent the combination of stresses at the simultaneous propagation of compression and shear, and σ_3, σ_4 - stretching and shear waves.

The assessment of stability of contact „lining-massif" and rock massif (of local fractures) was performed based on the strength conditions of material on tensile and shear stresses:

$$\begin{cases} \pm < [\pm] & (a) \\ ¦ = \dfrac{2_{\sim 0}}{2_{0}} > 1 & (b) \end{cases} \qquad (2.5)$$

where, σ, [σ] - calculating and permissible tensile stresses; η - margins of shear stresses; τ_α is a shear stress on a risk sliding surface; $2_{\sim 0}$ - an estimated resistance of rock material on shear defined by the dependence: $2_{\sim 0} = \pm \cdot tg\ddot{} + c$ r ($\ddot{}$; c are sliding parameters).

when, $¦ \leq 1$ a fracture occurs, and when $¦ > 1$ strength on sliding secured.

In the „lining -massif" model the contact is simulated with „tensile crack" and „shear crack" elements [3]. The „tensile crack" receive only compressing and shear stresses (satisfied conditions by 3.2) and does not operate on stretching; and „shear crack" admit shear displacement, when $¦ \leq 1$.

So, during calculation the gradual opening of contact –„lining-massif" and redistribution of stresses in this connection goes on in an iterative sequence until the condition (2.5) has been satisfied.

3. The analysis of calculation results

The rehabilitation of tunnel was considered according to two alternatives: a) the existing lining with filled gaps; b) strengthening lining - the existing lining with filled gaps and internal reinforced belt.

In the considered calculation cases the most unfavorable stressed condition was created at the simultaneous impact of compressing and shear waves directed basically vertically, partially horizontally. Below separate calculation results are presented reflecting the extreme stresses in tunnel lining at 8-point seismic impact, operating loads, including inertial forces of gaps filled with concrete.

a) The existing lining with filled gaps

It was accepted that during simultaneous impact of vertically directed compressing and sheare waves in the massif surrounding tunnel the area of fracture ($¦ \leq 1$)in the zone of arch reached 2.5m

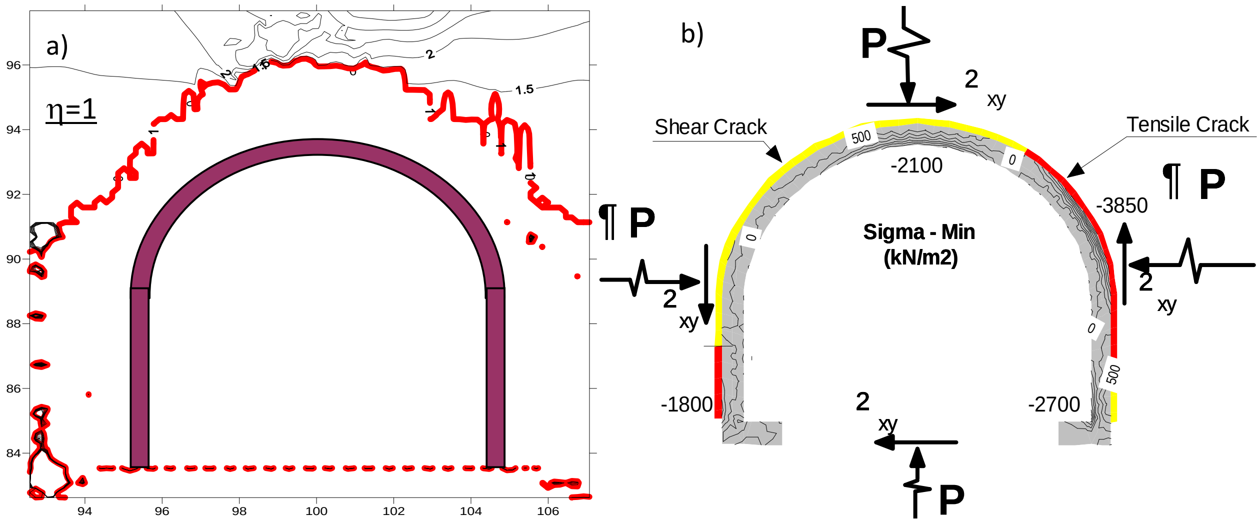

Fig. 2 a) Fracture zones around the tunnel; b) Minimal main stresses in the existing lining construction after filling the gaps.

(Fig. 2.a), while the maximum value of tensile stresses in arch lining reached 3850 kN/m2 (Fig. 2.b).

If we consider that the quality of construction is not sufficiently high (the quality of reinforcement is unknown), the tensile stresses developed in crown section and heel section may cause the creation of crack in lining structure and, accordingly, the loss of arch stability.

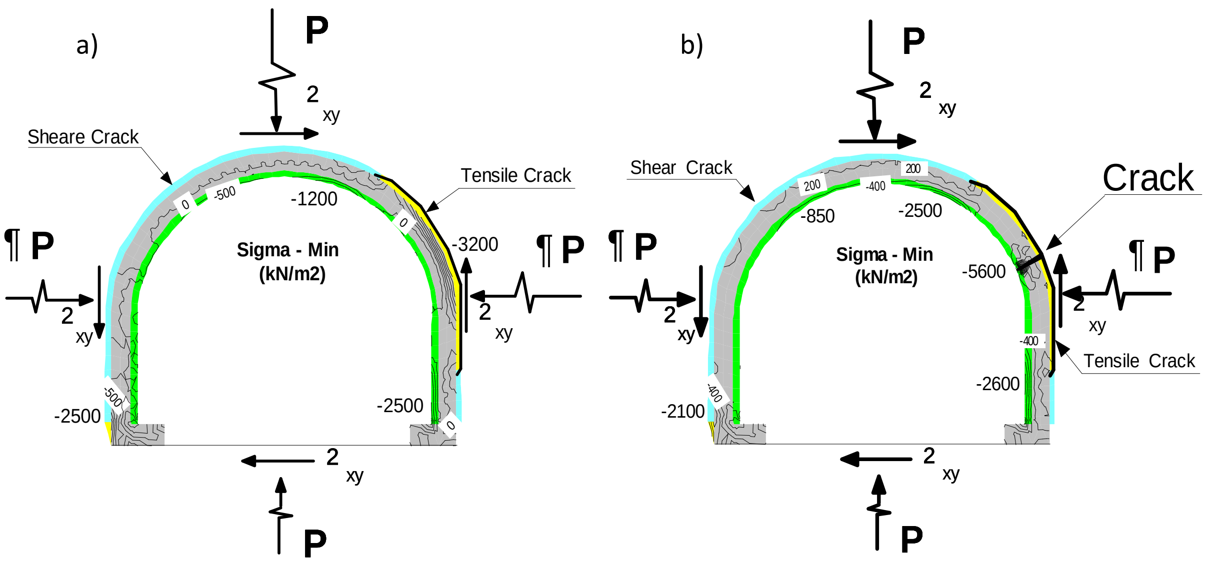

Fig. 3. Minimal main stresses in the strengthened lining construction at the passage of vertical compressing seismic waves.

b) The strengthening lining with filled gaps and internal reinforced belt

It is necessary to note, that during seismic waves spreading both in vertically and in horizontally directions the strength loss of contact occurs, by tensile stress ($\sigma_r = -23000\ kN/m^2$) shear stress conditions (fig. 1,2,3). So, „tensile crack“ will appeared in one part and „shear crack“ in another part of contact. The location of cracks (the side of lining, in which it will develop) will depend on the mark of shear waves.

Under the simultaneous impact of vertically directed compressing and shear waves the maximum tensile stress reaches: a) in existing lining $3200\ kN/m^2$; b) in the internal reinforced belt $1200\ kN/m^2$(fig. 3). According with this if appears crack in lining there will be a significant reduces of tensile stress and increase the tensile stress concentration ($5600\ kN/m^2$) in reinforced belt. In this case reinforced belt satisfied the condition of crack resistance (Fig. 3.2).

At the simultaneous impact of horizontally directed compressing and shear waves the above presented general regularity is retained (fig. 3.3a).

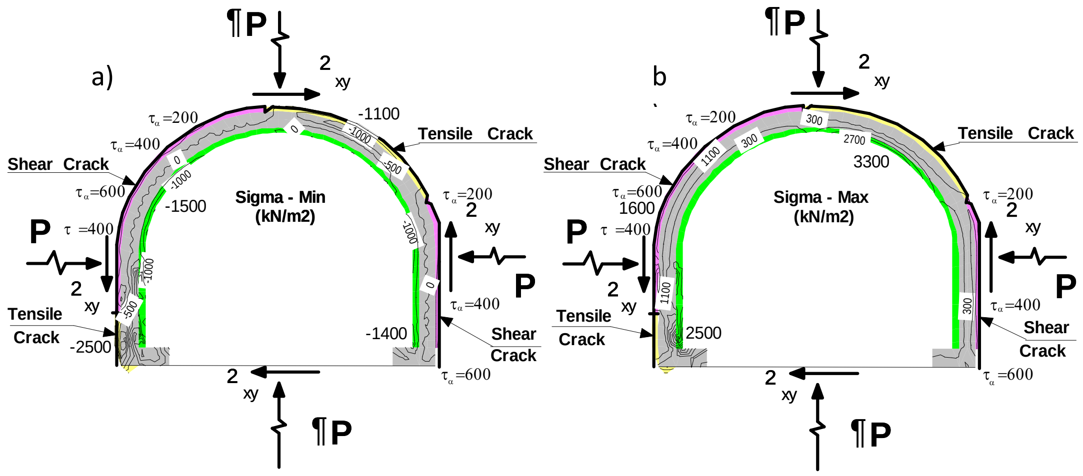

Fig. 4. Minimal main stresses in the strengthened lining construction at the passage of horizontal compressing seismic waves.

Conclusions

- During construction of tunnel serious violations were committed (the existence of huge gaps behind lining), which posed a great threat in terms of the loss of stability of existing lining construction. During seismic impact the fracture zones in massif, interial forces on lining and concentration of stresses in it increase. All the above statedstressed the urgency of taking rehabilitation engineering measures.

- Based on FEM a quasi-static model was developed of calculating the system „Lining-massif" on seismic stability (the combination of longitudinal and shear long seismic waves), which makes it possible to evaluate the joint action of lining and massif, including consideration o contact strength failure (emergence of tensile and shear cracks) and crack formation in massif and lining.
- In the calculation cases the most unfavorable seismically stressed situation is created at the simultaneous impact of compressing and shear waves directed vertically. The alternatives presented according to the above mentioned may be estimated as follows:
  - filling gaps behind tunnel is insufficient in terms of seismic stability. As a result of development of cracks in lining (accompanied by a fracture zone developed in rock massif) the loss of stability of construction is expected;
  - a lining with filled gaps and internal reinforced belt represents an effective rehabilitation alternative, which ensures strength and overall stability of lining construction.

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29-30 May 2014, Tbilisi, Georgia

RESULTS OF SURVEY OF PRESTRESSED CONCRETE BEAMS REINFORCED WITH BASALTPLASTIC BARS

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Abstract: *Structural materials reinforced with basaltic fibers represent particular interest for building industry. These materials (such as: basaltic-plastics, concrete reinforced with basaltic-plastic bar reinforcement, basaltic fibrous concrete etc.) combine high physical-chemical and mechanical significance with relatively small density. Creation new types of pipes and pipelines on the basis of basaltic fibers, first of all is connected with using of composite materials reinforced with fibers. Application of basaltic fibers as a reinforcing material is one of the perspective directions not only for nowadays but also for future as well.*

Keywords: basalt fiber, reinforced, concrete, pipe, pipeline, composite, material, compression, flexure, natural experiment, glass fibers.

1. Introduction

Georgia is rich with volcanic rocks, in which according to its amount, mineral and various physico-chemical content basalts are at first place. It is possible to make high strength pipes and bearing structural elements for different purposes by processing of it.

The unbroken glass fibers are used as a reinforcing agent in many countries of the world in production of composition materials and in light industry.

The development of their production is delayed mainly because of the increasing lack of raw materials (calcinated soda, boric acid, soleplate etc.) and because of the high technical requirements.

Some technological processes for receiving unbroken fibers from widely-distributed igneous basalt rocks, which require no additional processing, are worked out at present.

There are some quarries in Georgia, near Tbilisi (regions of Marneuli, Kaspi, Aspindza, Bakuriani etc.), where the raw materials contained rare and rare-earth metals can be found for

receiving high quality fibers. The volume of these materials is estimated at tens of millions cub. meters.

As a reinforcing material basaltic fibers are strong, stable for aggressive force and durable.

By using basaltic fibers, it is possible to create some source saving technologies for light industry, building and other fields (machine construction, aviation, ship-construction etc.).

Here is a list of qualities of basaltic fibers and products made of them:

- stableness for corrosion (12-5 times more than metals);
- frost- and heat-resistance (-265°C ÷ +900°C);
- almost incombustible (become charred when t = (1000 ÷ 1100)°C);
- non toxic;
- high durability showings: $\sigma = 1900 \div 2200$ MPa when the diameter of a fiber is 9 ÷ 12mcm;
- construction elements are sometimes 3-10 times stronger, than analogous traditional constructions made from steel and concrete;
- lightness (decrease the weight of construction elements 5-20 times);
- do not create hindrance for radio and television waves and are dielectrics;
- heightened water resistance;
- when are plated with polyethylene, meet hygienic requirements of food industry;
- when fibers are received as a mineral wool, meet the requirements to be raised to heat-insulating materials.

Structural elements made with using basaltic fibers is protected I have by patent №SU 1830405, SAKPATENTI: №P1755, №AP2162A, №AP2000, №NP 2930, №P5185, 003937 and №U493, by St. committee on inventions and Discoveries of the USSR and Georgia "**Sakpatenti**". These structures should be effectively used by construction of earthquake-resistant buildings, because these materials are very durable and light.

1. INTRODUCTION

In many regions of Georgia (Akhaltsikhe, Marneuli, Kaspi, Borjomi, Chiatura etc.) there are quarries of raw materials from which high-quality basalt fibers are obtained on the basis of which building materials, construction and consumer goods can be obtained. Basalt non-metallic building materials, construction and consumer goods obtained from these quarries have high initial strength and durability.

On the basis of crushed basalt processing the high basalt fiber (roving, filaments, basalt fiber etc.) can be obtained those having the following physical and mechanical characteristics:

1. - Tensile strength 18000-19000 kgf /cm^2;
2. - -265^0 - 650^{0}C in the temperature range does not alter the physical and mechanical characteristics;
3. - Corrosion resistance than iron and steel products 15 - 20 times higher;
4. - Compared to materials based on iron and steel, structural weight obtained on the basis of basalt fiber is 3.5-3.6 times lighter.
5. Modulus of elasticity of basalt fiber is within a range of 650000-850000 kgf/cm^2.

Popularity of basalt-plastic reinforcement (BPR) among construction companies is growing day by day. Nowadays, in a number of countries, including Georgia there are factories built for the production of fibers from basalt stone and production of BPR for the reinforcement of concrete structural elements is also planned. In this regard, we have conducted studies to widespread the use of BPR in the construction.

BPR has a number of advantages over steel reinforcement. The main advantages are:

1. **Lightness** - a significant advantage for all kinds of construction works. Low weight BPR not only facilitates its delivery (transport costs), unloading and installation, also due to the law weight of the structure the cost of the construction works and the total weight of the whole building or structure is also reduced, on the basis of which the costs of the foundation and other bearing structures are reduced.
2. **Reliability –** BPR reinforcement has more strength than steel reinforcement. This reinforcement has high ultimate (rapture) strength. It is capable of withstanding action of 1000 1200MPa. This indicator allows using the reinforcement in the construction of buildings and structures of various sizes and for various purposes.

3. **Non-conductor of electricity (dielectric)** - it allows boosting the quality of safety factor against fire, as well as eliminates the occurring of vagabond currents and accumulation of static voltage.
4. **The high rate of resistance to corrosion processes** - BPR is resistant to corrosion and virtually cannot be destroyed by corrosion.
5. **Low thermal conductivity** - the use of BPR allows reducing heat consumption, and energy through thermal bridges, respectively, which occur when a steel reinforcement is used. This quality is very important and should be taken into account in the construction of efficient structures in terms of heat energy consumption.

BPR reinforcement does not emit noxious substances, is non-corrosive, does not create the magnetic and electric fields and a Faraday cage; mobile phones, Wi-Fi and any other wireless devices can get better signals.

The usage of nonmetallic BPR reinforcement instead of metallic BPR reinforcement enables us to obtain not only non-metallic materials but it has a qualitatively new and higher performance that increases the lifetime of structures used in a corrosive environment and reduces metal consumption in structures, their weight and cost, and accordingly, the working time of construction. Compared with steel reinforcement, BPR have a significantly higher corrosion resistance, heat insulation and dielectric properties and are non-magnetic and radiolucent.

One of the constraints of the use of BPR for the reinforcement of concrete is the increased deformability of reinforced concrete structural elements of BPR, which led to the need for extensive experimental studies, including this study. BPR are produced in a pilot plant in Tbilisi, the production of basalt reinforcement and other profile products based on continuous basalt fiber. For the production of BPR basalt fiber was used, “Rustavi Plant production of basalt Parody of Marneul origin”. Diameter of BPR is 6.5mm, 1.0mm of sheath. Self-stressing cement produced by Pashiysky plant RF NC-20. Concrete grade is B22.5.

2. BASIC PART

The pre-stressed structures were derived from the expansion of the concrete. In the initial stage of design it is assumed that they are equal because at this stage sliding of the reinforcement does not occur. The compression force of the reinforcement in the expansion of concrete is defined from the condition of equilibrium in the transition to the stresses.

$$\sigma_0^6\ F_6 = \sigma_0^a\ F_a \tag{1a}$$

Here σ_0 with index $_0$ denotes the value of the stress immediately after the concrete hardening and the completion of the basic process of expansion. Moment values from the off-center of forces and pre-compression of structures were calculated using the formula 1:

$$M_0 = N_0 / \left(h/2 - a\right), \tag{1}$$

From the formula 1 compression forces can be obtained:

$$N_0 = \frac{M_0}{\left(h/2-a\right)} = \frac{M_T^{on} - M\xi}{\left(h/2-a\right)} \tag{2}$$

To study the problem of deformation beam sections are considered in pure bending zone. It is known that in pure bending, deformation of beams can be assumed as the result of the rotation (twisting) of plane sections relative to each other. As a result, rotation (twisting) of the sections, the lower layers is extended, and the upper layers are shortened and the change of curvature of the neutral layer can be expressed as follows: (Formula)

$$\frac{1}{\rho} = \frac{d\theta}{dz}, \tag{3}$$

As a consequence of the expansion at the level of the center section of tensile reinforcement, beam sections will increase in length Δa, and then we can write the expressions for defining the angles of rotation:

$$\theta_a = \frac{\varepsilon_{ac}\ell}{r}, \tag{4}$$

$$\theta_b = \frac{\varepsilon_{bcp}\ell}{r}, \tag{5}$$

Where: ε_{ac}- is the average elongation of BPR;

ε_{bcp} - is the average shrinkage of the concrete at the level of the neutral line;

r – is a distance from the neutral reinforcement.

The total elongation of BPR in tension, as well as the value of rotation of the sections from the formulas 4-5, with the use of computer modeling laid in [13, 14 and 15] can be written as:

$$\varepsilon_{ac} = \frac{2\theta_a\left(h_0 - X_{cp}\right)}{\ell} \tag{6}$$

For the curvature of the neutral layer from the center of rotation of the cross-sections have the following entry:

$$\frac{1}{\rho} = \frac{2\theta}{\ell} = \frac{\varepsilon_{ac}}{h_0 - X_{cp}} = \frac{\varepsilon_{bc}}{X_{cp}} = \\ = \frac{\varepsilon_{ac} + \varepsilon_{bc}}{h_0} = \frac{\varepsilon_{ac} + \varepsilon_{bc}}{r} \tag{7}$$

As it is known, [13, 14 and 15], middle curvature in the location with the broken discontinuity of material (1) can be written as:

$$\left(\frac{1}{\rho}\right)_{cp}^{\ell_n} = \frac{\Psi_a^{\ell_n}\varepsilon_b + \Psi_b^{\ell_n}\varepsilon_b}{h_0} \tag{8}$$

When measuring the average strain of BPR on a fixed base ℓ, the coefficient ψ_a taking into account the work of concrete in tension between cracks can be represented by the expression:

$$\psi_a = \frac{\int_0^{\ell} \varepsilon_{ax} dx}{\ell \varepsilon_a^{\ell}, \max} = \frac{\varepsilon_{ac}^{\ell}}{\varepsilon_a^{\ell}, \max} \tag{9}$$

Arm lever of the internal force couples in the section with and without crack differ little. To a first approximation, it is assumed that they are equal. The rotation angles of the displacements are as follows:

$$\theta_a^{on} = \frac{4Y_{max}}{\ell} \tag{10}$$

Where: $\gamma_{\max}$ – is the maximum value of deflection in the span,

l – is pure bending zone.

Determine the reduced moment of inertia of the BPR in the section between the cracks:

$$I_{am} = \frac{M\ell^2}{8E_a Y_{max}}; \tag{11}$$

Reduced moment of inertia of the BPR in the section with a crack:

$$I_{am} = F_{am} Z_{am}^2 \cong F_{am} \left(0.9h_0\right)^2, \tag{12}$$

Then:

$$F_{am} = \frac{I_{am}}{0.81h_c^2} \tag{13}$$

If only BPR works, then $I_{am} = 0.81h_0^2E_{am}$ and deflections become significantly larger. If reinforcement and concrete in tension work, then there occurs less deflection, which is equivalent at a constant $Z_{am}^2 \times 0.81 \times h_0^2 \times E_{am}$, increases the area of BPR F_a. Then the deformation of BPR F_{am} with equal deflections $Y_{max} \rightarrow \varepsilon$

$$\varepsilon_a = \frac{M}{Z_{am}F_{am}E_0}, \tag{14}$$

Where:

$$\varepsilon_{ac} = \psi_a\varepsilon_a; \quad z_{am} = 0.9h_0;$$
$$M = F_a\sigma_a 0.9h_0 = E_0F_a 0.9h_0 = F_a \frac{\varepsilon_{ac}}{\psi_a} 0.9h_0E_a \tag{15}$$

After conversion, we get:

$$\varepsilon_{ac} = \frac{M\psi_a}{E_aF_{ac}0.9h_0}. \tag{16}$$

Geometric dimensions, reinforcement diagram and the application of loads, as well as the results of the study of pre-stressed concrete beams reinforced with BPR are presented in the form of dependency graphs and presented in Fig. 2.3.

With the purpose to clarify the theoretical results and a detailed distribution of strain and stress, self-strained concrete beams reinforced with BPR and an analysis based on computer modeling using computational complex “LIRA CAD 2013” is conducted, taking into account the characteristics of the nonlinear deformation of materials. The results are shown in Fig. 4.

With the purpose of modeling, the structural elements have been applied universal modules from the library of “LIRA CAD 2013” corresponding to the following types of finite elements:

- 208 - a special physically-nonlinear two-node finite element for pretension (modeling BPR);
- 36 – Universal spatial (3D) eight-node isoparametric finite element (modeling concrete);
- Supports for beams: - pinned.

Loads: 1. Self-weight. 2. Pre-stressing. 3. The load applied to the beam on the steps in the one third of the span.

Data for the calculation based on the studies of test samples of structures with BPR periodic (ribbed) profile made by tightening methods in the experimental setup (in Tbilisi).

Prestressing force to each reinforcement bar was taken N_0 =-6tonf.

BPR “Stress-strain” diagram obtained with this method of production is substantially rectilinear to failure (see Fig. 1.). However, the data are experimental and subject to further

refinement, taking into consideration the content of the fiber. The essential fact is that the diameter affects the temporary resistance value of the BPR, thinner reinforcement, the greater its strength.

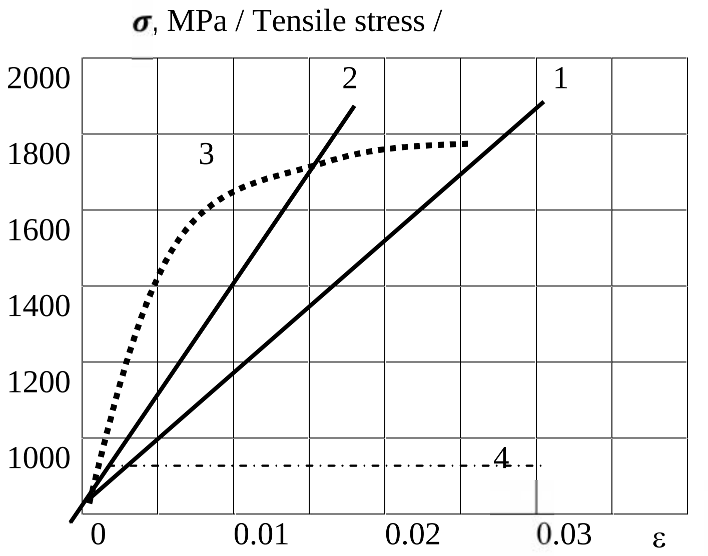


Fig. 1. The dependence of the relative elongation ε of the stressed $\boldsymbol{\sigma}$ for:

1. BPR;
2. Cold-drawn steel;
3. high-strength cable;
4. Steel with high yield stress.

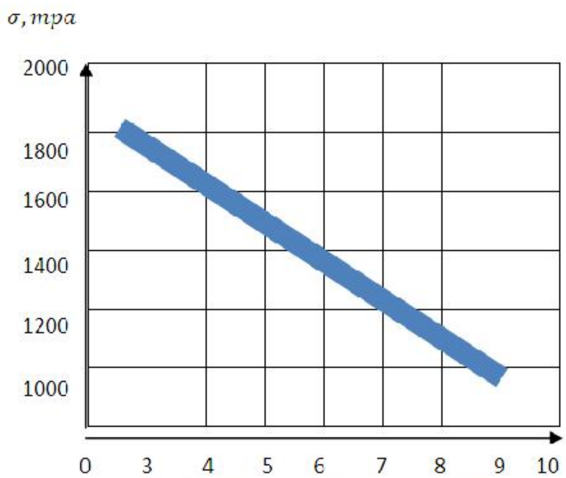


Fig. 3. Influence on Diameter of BPR
In Case Of Failure

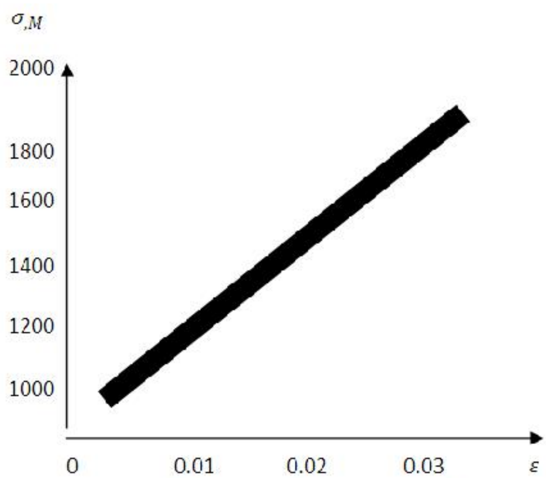


Fig. 4. Diagram of Tension Of BPA

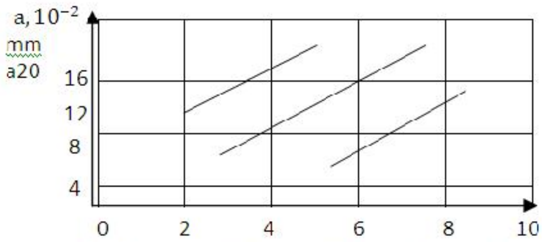


Fig. 5. Gap And Depth Of Thread Recoiled On BPR Bar
a Crush Depth; b gap of Recoiled Threads

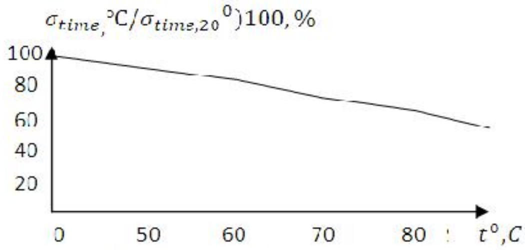


Fig.6. Influence of Temperature Of Vaporization Chamber On The Strength Of BPR

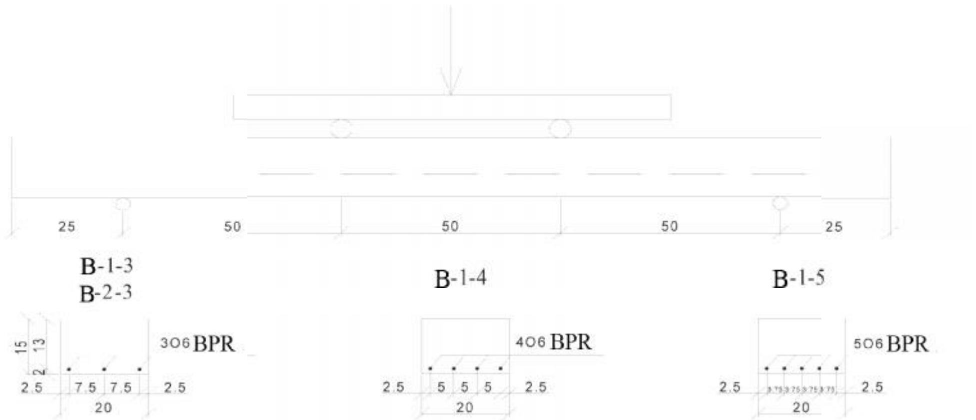


Fig. 7. Diagram of reinforcement and loading of prestressed beams

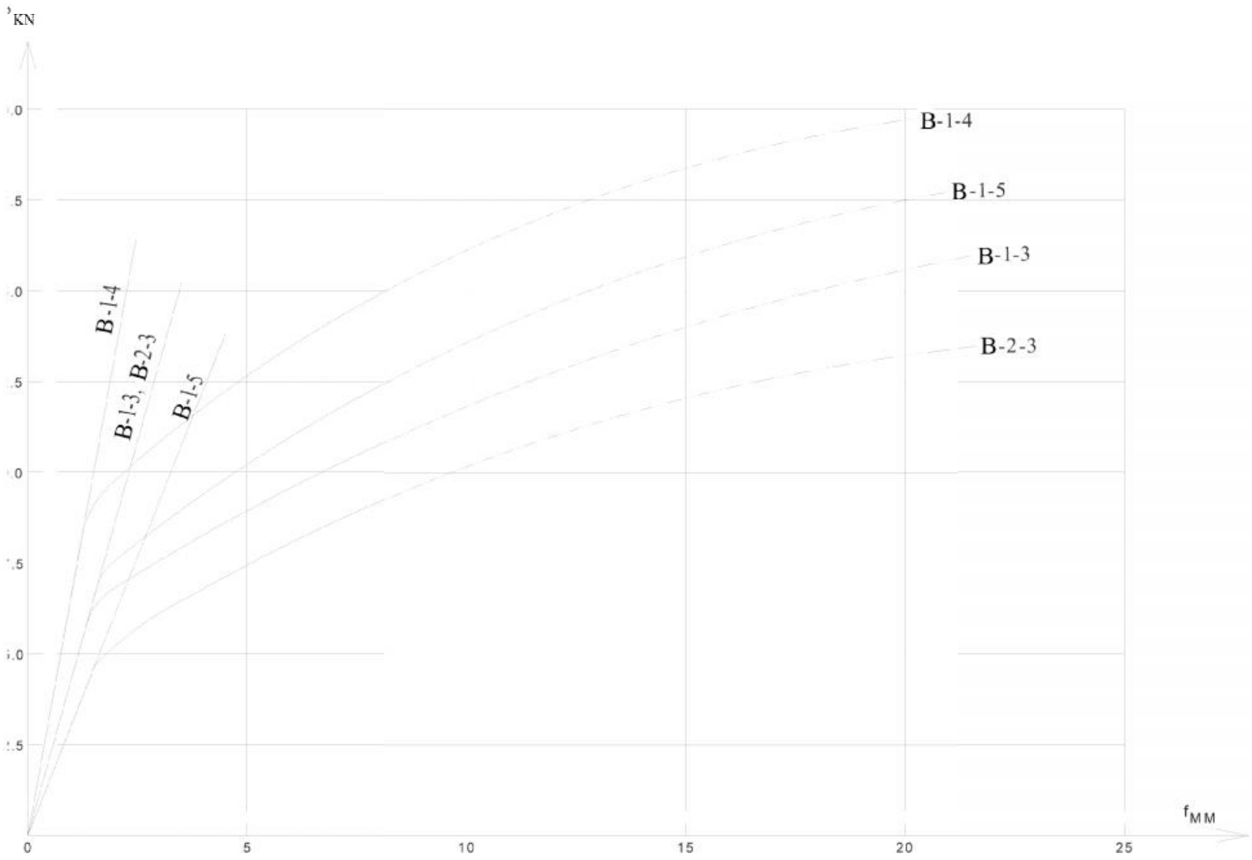
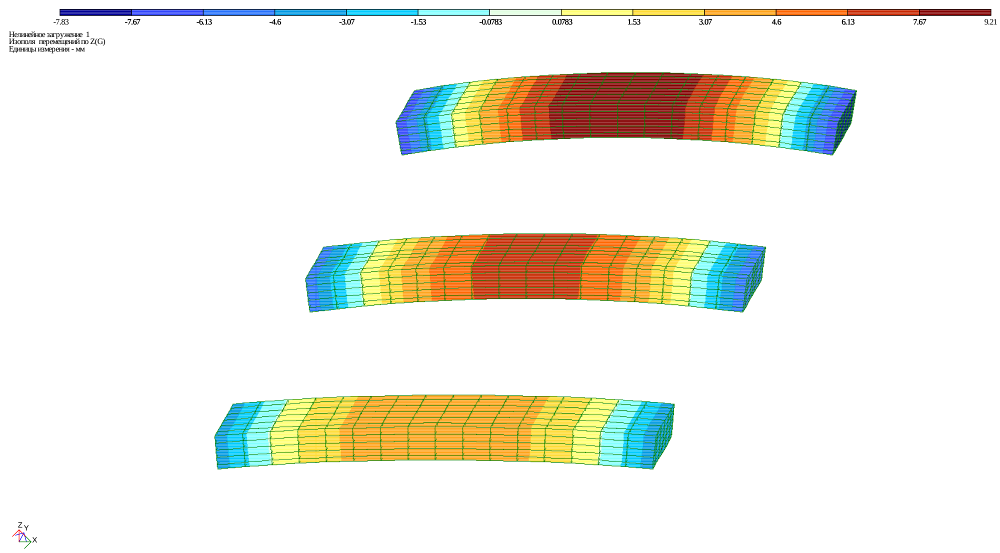


Fig. 8. Relation of load deformation of prestressed beams reinforced with basalt-plastic reinforcement



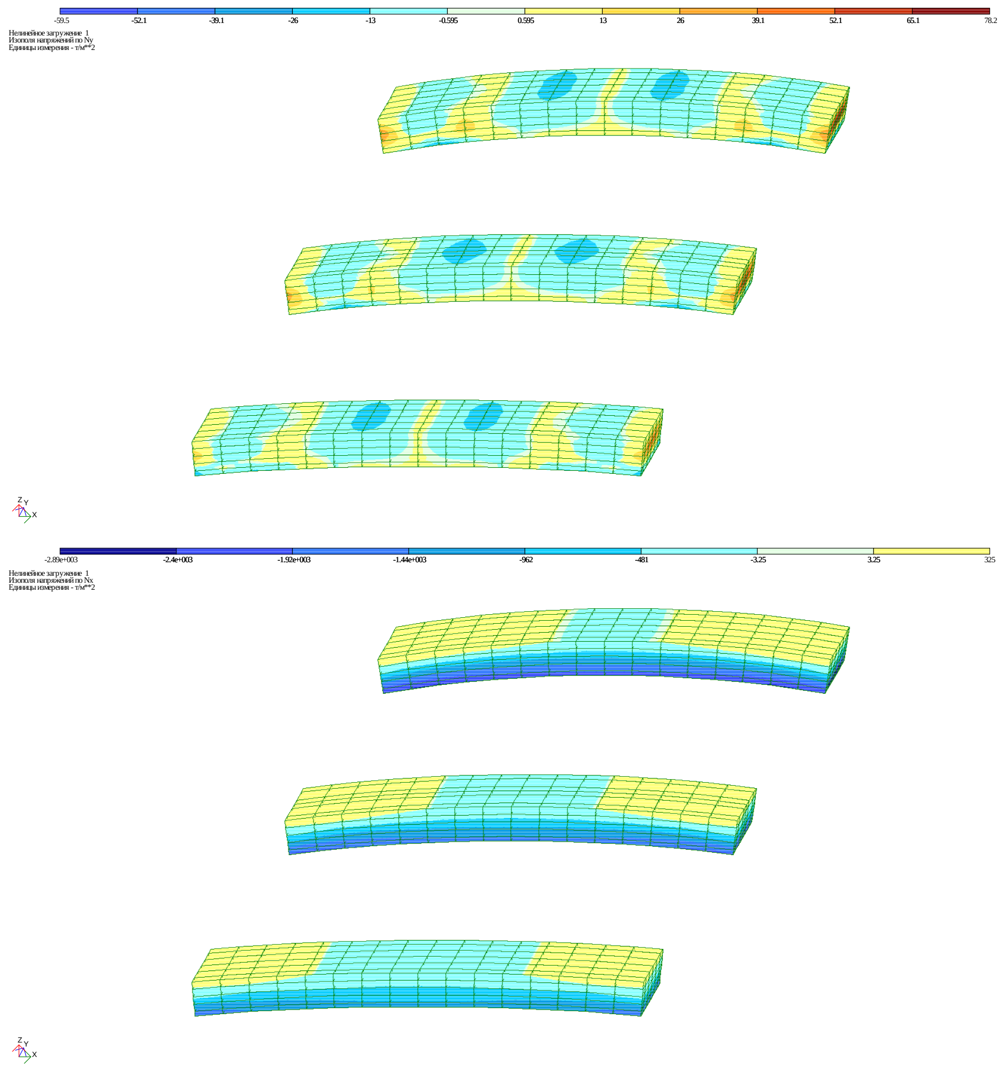


Fig. 9. Distribution of strains and stresses of prestressed concrete beams reinforced with BPR.

CONCLUSIONS:

1. To design prestressed reinforced structures of basalt-plastic reinforcement, proposed technology requires no special gripping devices that are inconvenient for service and also tend to damage the reinforcement at bar tightening zone. According to the proposed technology, transfer of tensile forces is due to the shear stresses that arise when expanding of stressing cement occurs and develops along the contact line reinforcement-concrete;
2. The results of the proposed methods and the methods of SNIP 11-21-85 and computer modeling to determine the stress-strain state of structures differ by an average of 15%;

3. The use of self-stressing cement or low-energy expansion and a minimum diameter of BPR (6.5mm) to design a pre-stressed structures should be considered ineffective, because destruction of the beams occurred before reaching the limit values of BPR;
4. A substantially rectilinear relationship σ-ε (stress-strain) of the BPR to tensile fracture retains prestressed position when cracking occurs and to the destruction of concrete beams, additionally, reduces the deformation parameters of construction elements;
5. High technical and deformation performance of concrete elements reinforced with basalt reinforcement determine the feasibility of using such structures for earthquake-resistant and special buildings and structures;
6. The numerous studies that are conducted prove that the sticking coefficient of concrete to BPR is less than the sticking coefficient of concrete to the steel reinforcement. Therefore, improving factor of the bonding of BPR with concrete should be given greater role and attention.

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29-30 May 2014, Tbilisi, Georgia

EXPERT ASSESSMENT OF CARRYING CAPACITY STIFFERING DIAPHRAGMS CARRYING CAPASITY UNDER SEISMIC IMPACTS

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Abstract: *The engineering methodology of estimation of the maximum state of reinforce-concrete diaphragms is considered with cracks at seismic influences.*

Keywords: Diaphragm, seismic impacts, cracking, under n-load cycles, reinforcement.

Diaphragms and enclosed stiffening cores are the main load-bearing elements, providing modern high-rise concrete buildings frameworks seismic strength. That's why the issue of their carrying capacity investigation was considered in many studies [1 - 10], etc. To estimate the carrying ability of structures under seismic impacts, time-consuming calculations based on the real accelerogramic records, physically nonlinear calculation methods [5, 7] and others (which yet allows to solve the dynamic multimass system in time, considering the physical nonlinearity for a very narrow range of tasks).

At the same time in the United States, Japan, Greece, Turkey, Moldova and other countries with areas of high seismicity and increased probability of earthquakes for expert assessment of ultimate strength stiffening diaphragms the engineering techniques based on extensive database of experimental results [1 -4, 9-10] are used. In Ukraine, with the new edition of DBN B.1.1 -12 : 2014 [6] acceptance, the requirements for 3D FEM analysis of buildings located in seismic regions increased with mandatory application of the seismic microzonation accelerograms and non-linear methods of structures carrying capacity estimation. Nevertheless, the methodology of direct time integration for dynamic systems [10] implemented in SP "LIRA"

has not yet been brought to the level of wide practical application, and SP “SCAD” is totally absent.

Thereby, the subsystem "Seismic strength diaphragms" implemented in EEH 2013 (Electronic Engineers’ Handbook) is proposed to supplement with a new method. The method is designed for expert assessment of the stiffening diaphragms and piers limiting shear strength under the seismic and cyclic loads with regard to its behavior on the second limit state (meaning that in methodology is was taken into consideration the cracking in the diaphragm, residual stiffness of concrete strips with factors of damage accumulation, pin effect and alternating loading. The initial required data: the geometrical parameters of walls, properties of reinforcement, working conditions ratio, load, etc. (Fig. 1).

Methodology. The “limit equilibrium” method is implemented in the subsystem in combination with empirical methods of determination of the reinforced concrete diaphragms with cracks ultimate strength under the cyclic and seismic loads: Uniform Building Code of the United States (according to UBC); Hernandez O.B., Zermeno M.E - WCCE, Istambul; Barda F., Hanson J., Corley G - American Concrete Institute, Detroit; Scientific and Technical University of Greece-Tassios T., Lefas J., Lulurgas S. - NTU Atens; Hirosawa M - Building Research Institute, Japan; ATS-3 (Temporary recommendations on earthquake-resistant buildings, USA) [1 - 10].

In this paper the attempt to obtain generalized relations for determining the strength of the stiffness diaphragms under the static, dynamic and cyclic loads, based on an analysis of the techniques above was made. It is well known, that the behavior of the reinforced concrete diaphragms under the seismic and cyclic loadings is significantly affected by reinforcement with the yield point. This allows to achieve greater plasticity, energy-dissipating capacity of the wall and increases the compressive and shear strength of the cracked concrete strip and leads to the appearing of so-called “plastic hinge”. Additionally, as shown in [1, 4, 9, 10], the spatial, contour (fringing) reinforcement and character behavior of the concrete seams (embedment) mainly predetermine the walls shear strength. So, it is proposed to take these factors into consideration.

The area of diaphragm carrying abilities in any combination of loads N_i, Q_i, M_i formed in graphical form (Fig.2) and allows the user to receive the safety factor using the implemented techniques.

Permissible types of loads and impacts on the diaphragm:

- N_i, Q_i, M_i, – special rated load combination (RLC);
- Estimated seismicity of the area - 6, 7, 8, 9 points;
- Number of the exposure cycles;
- Forces obtained through spectral method or accelerograms;
- work conditions ratios for the diaphragm, diaphragm seams (joints), concreting and other conditions ratios are recommended to set to account for cyclical loadings, reducing the shear resistance of technological seam, given the data [1, 3, 4]:

$K_{us\_a}=0.95-0.9$ subject to special seam treatment, otherwise $K_{us}=0.7$;

$K_{us\_b}= 0.95, 0.9, 0.8, 0.75$ at the estimated seismicity of 6, 7, 8, 9 points respectively;

$K_{us\_\partial}=0.9$ for hollowed and porous elements;

In the case of presence of several factors from above: generalized ratio K_{us} (¥$_{us}$) is definite as:

$$K_{us} = K_{us\_a} * K_{us\_b} * K_{us\_c} * K_{us\_\partial} * K_{us\_i}.$$

The basic design dependences of the diaphragms shift strength which are implemented in PS "Seismic strength of diaphragms" are shown in Table. 1.

Where: α_s - shear impact ratio, which can be determined as: $\alpha_s=Mi\backslash(Qi*L);$

f_c, f_r – concrete compressive, tensile strength, on 2nd in limit state respectively (MPa);

f_{sx} – yield strength of horizontal reinforcement;

f_{sy} – yield strength of vertical reinforcement;

ρ_h, ρ_v – ratios of horizontal, vertical reinforcement respectively;

$\rho_w =(\rho_h + \rho_v)/2$ – ratio of resulted volume reinforcement;

ρ_l – ratio of bordering reinforcement;

σ_o – average compression stress;

τ_u – ultimate shear strength of diaphragm;

μ - irregularity factor of tangential stresses;

f_b - shear area in the horizontal cross section;

N_{pr} - ultimate force accepted by the section with a crack before the formation of "plastic hinge";

N_{1c} - ultimate force accepted by the section at central compression;

$\Delta\tau_1$ – first loses of shear stress at the forces obtained of the linear analysis by accelerograms;

τ_{lin} - tensions derived through analysis of the linear accelerograms;

¥$_{us}$ - working conditions ratio for seam concreting.

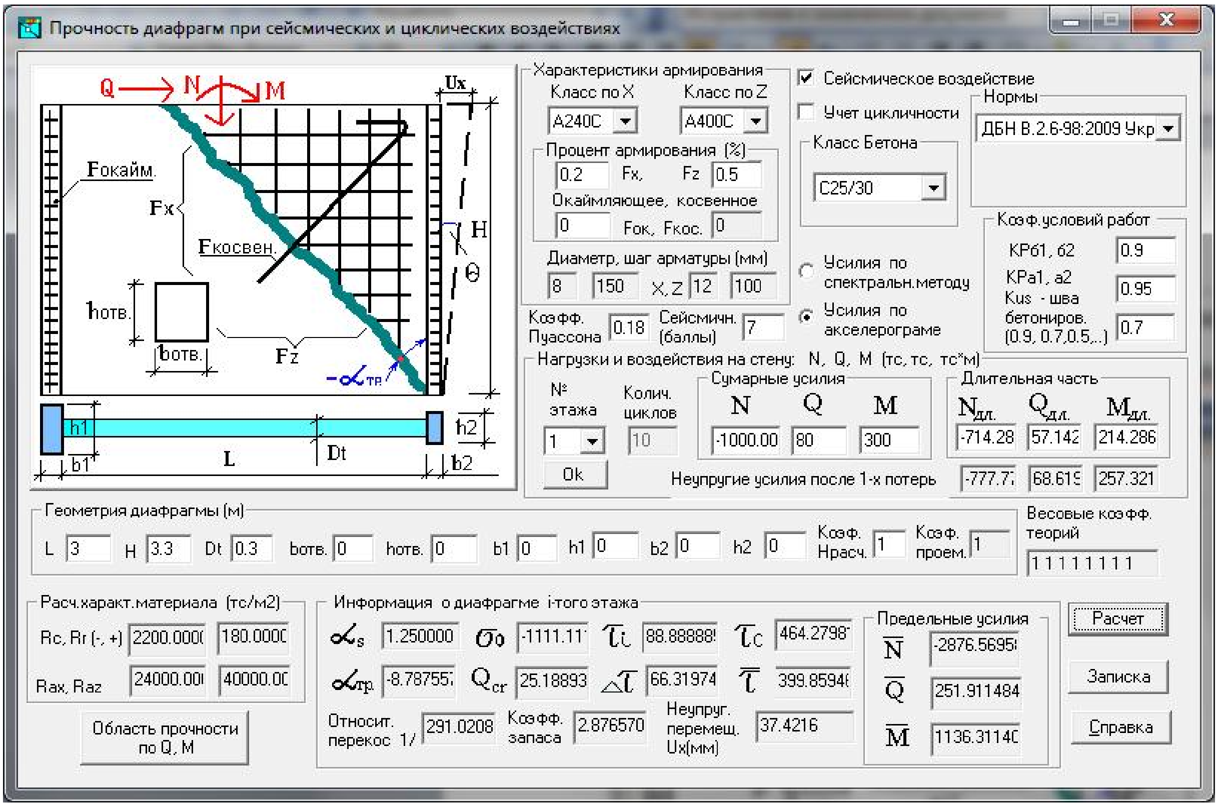


Fig. 1. General view of the working window of the SP «Seismic strength of diaphragms»

Table 1

Design dependences
Design dependences: $$\left(\frac{N_i}{N_{pr}}\right)^2 + \left(\frac{Q_i}{Q_{pr}}\right)^2 - 1 = 0; \qquad 2_u = 2_c + 2_s;$$ where: N_{pr} - ultimate normal force perceived by section; Q_{pr} - ultimate shearing force of perceived by section; τ_u - shear strength of reinforced concrete diaphragm with cracks; τ_c - strength of the concrete strips between the cracks, resistance to bending, shear and pin effect of cracked concrete;

τ_s – strength of reinforcement of the diaphragm with cracks;

$$npu\ a_s \geq \sqrt{2}, \sigma_0 \leq 2f_c/3;$$

$$\tau_c = \left(1.0 - 0.2a^2{}_s\right)\left[\sqrt{(f_c + 0.5\sigma_0)}\right] \geq 0.25\sqrt{f_c} + 0.1\sigma_0;$$

$$\tau_s = 0.1\mu_h f_{sx} + 0.9p_v f_{sy} + 0.01\mu_l f_{sy}; npu\ p_v \leq 0.005(0.5\%);\ \mu_l \leq 0.035(3.5\%).$$

otherwise: $npu\ a_s < \sqrt{2}$

$$\tau_c = 0,25\sqrt{f_c} + \frac{1}{2}\sigma_0 \geq 0.33\sqrt{f_c} + 0.1\sigma_0,\ npu\ \sigma_0 \leq 2/3\ f_c;$$

$$\tau_s = \mu_h\left(a_s - \frac{1}{3}\right)f_{sx} + \mu_v\left(\frac{3}{2} - a_s\right)f_{sy} + 0.05\mu_l f_{sy};\ npu\ 0.33 \geq a_s \leq \sqrt{2};$$

$$\tau_s = \mu_h\left(\frac{1}{3}\right)f_{sx} + \mu_v\left(\frac{2}{3}\right)f_{sy}\ npu\ a_s < 0.33;$$

$$\tau_u \leq 0.95\sqrt{f_c} + 0.1\sigma_0;$$

$$Q_{pr} = \gamma_b * \gamma_{us} * \tau_u * \mu * f_b,\ at\ \ N_i < N_{pr};$$

otherwise:

$$\bar{Q}_{pr} = Q_{pr}\ (N_{1c} - N_i)/(N_{1c} - N_{pr});\ M_{pr} = \gamma_{us}\ Q_{pr}\ (\bar{Q}_{pr})\alpha_s\ L;$$

$\bar{\tau}_u = \tau_u - \Delta\tau_2;$ with forces obtained of the spectral method;

$\tau_u = \tau_{lin} - (\Delta\tau_1 + \Delta\tau_2);$ with forces obtained from the accelerograms;

The restriction $\tau_c \geq 0.15 \sim 0.25\sqrt{f_c}$ of the strength of the concrete strip between the cracks is confirmed by studies [1, 2, 4], and others where it is shown that the strength of the concrete strip is cracked within $R_{n6} \approx 0.7f_c$ – if there is no fluidity of reinforcement, $R_{n6} \approx 0.4f_c$ – in case of reinforcement fluidity in one direction only, and $R_{n6} \approx 0.3f_c$ – in case of reinforcement fluidity at the two directions. The proposed dependences accepted that $R_{n6} = 0.2f_c$ - not less than the minimum value of the spectral method and $R_{n6} \approx 0.4f_c$ when calculation is made according to the accelerograms.

The main points on the strength area of the wall are shown in Figure 2 and calculated according to the dependencies on Table 1. The maximum shear strength of the wall - τ_u is

defined taking into account the tangential stresses degradation (dowel reaction) of the element with crack under n-load cycles – $\Delta\tau_2$ as: $\bar{2}_u = (\tau_{u1}+..+\tau_{ui} + \tau_{uc} - \tau_{u\_max} - \tau_{u\_min})/(i\text{-}1)\text{-} \Delta\tau_2,$

where:

τ_{uc} – average value of diaphragm strength by various methods;

$\tau_{u\_max}, \tau_{u\_min}$ – maximum and minimum strength of the wall according to the procedures τ_{ui};

τ_{u1}, τ_{ui} – wall strength according to the methodology which:

dependences from UBC - USA ; Hernandez O.B., Zermeno M.E – WCCE, Istambul; Barda F., Hanson J., Corley G - American Concrete Institute, Detroit; Tassios T., Lefas J., Lulurgas S. - NTU Athens; Hirosawa M. - Building Research Institute, Japan; ATC-3 (Temporary recommendations for designing earthquake-resistant buildings, USA) was implemented in.

$\Delta\tau_2$ - degradation reactions of the second losses can be defined:

$$\Delta\tau_2 = \beta_d \sqrt[4]{n} 2_i, \quad \beta = 1/(0.8 + 170\varepsilon 1)$$

where: β_d – degradation rate of the reaction in accordance with the [2, 15], as $\sim_s > 0.5$, is equal to $\beta_d = 0.1/\alpha_s$.

In accordance to the dependences of Tassios T., Lefas J., Lulurgas S. [1, 10] the average tangential tensions at the beginning of cracks formation are:

$$2_{cr} = 0.3(2.6\text{-} a_s)\,[(0.1+20p_w)\sqrt{f_c} + 2p_w^2\, 10^4], \text{ as: } 2.5 \le \sim_s > 1,$$

$$2_{cr} = 0.5(1.5\text{-} a_s^2)\,[(0.3+20p_w)\sqrt{f_c} + 3p_w^2\, 10^4], \text{ as: } \sim_s \le 1,$$

In accordance to the dependences of Tassios T. и др. [1, 10] the diaphragm skew angle Θ_u and the maximum deformation U_x when $\sim_s > 0.5$ и $2_u > 2_{cr}$ and taking into account the "plastic hinge" formation:

$$\Theta_u \approx 0.012 \sim_s (1\text{-}0.1\sqrt{\pm_o}); \qquad Ux = \zeta\,(tg\,(\Theta u)*H),$$

where ζ – hysteretic damping ratio in accordance with [1] which depends on the nature of the diaphragm destruction;

the flexural – $\zeta \approx 8\text{-}10$; flexural and shearing – $\zeta \approx 4\text{-}8$; shearing – $\zeta \approx 2\text{-}4$, otherwise nonlinear elastic deformation of wall is defined in the form (see Fig. 3):

$$U_x = U_x^Q + U_x^M = \left[\frac{Q_i H^3}{E_b^s \text{¢} L^3 / 4} + \frac{M_i H^2}{E_b^s \text{¢} L^3 / 6} \right],$$

where: E_b^s - secant elasticity modulus of concrete or concrete strip with a cracks in the direction of the principal compressive stress σ_2 can be determined from the nonlinear dependence on Eurocode-8 [4].

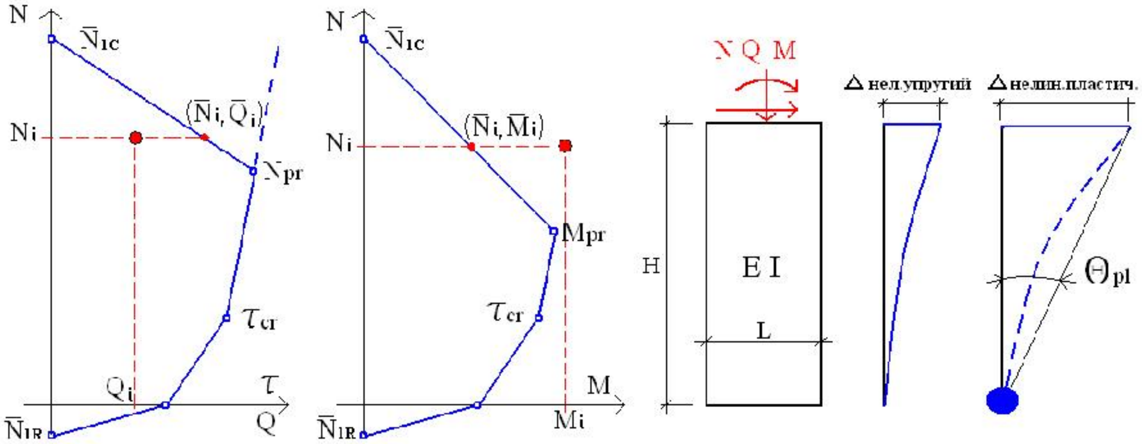
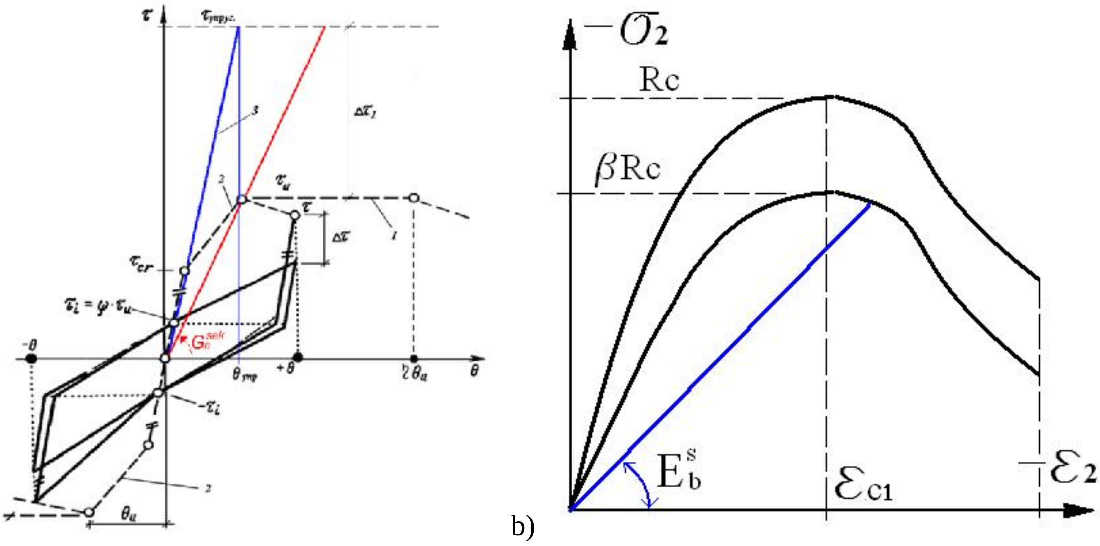


Fig.2 Area of wall strength for N ~ Q, N ~ M and nonlinear deformation of the diaphragm

Accounting of seismic loads cyclicality taken by the idealized model of hysteresis of reinforced concrete walls (see Fig. 3a).



a b)

Fig. 3. Idealized hysteresis model - a);
Secant elastic modulus of concrete strip with a crack - b):
1- Skeletal curve cyclic loading at the bending nature of the work;
2- Skeletal curve cyclic loading at the shear nature of the work;

τ_{cr} – tangential stress at the beginning of cracking;

β – softening ratio of concrete strip at "tension-compression" in accordance with [2];

The maximum compressive stress in the concrete strip considering ε_1, ε_2 - relative deformations of the concrete strip between cracks on the main sites taken as:

$$\sigma_{c2} = \beta R_c \left[\frac{\pounds_2}{\pounds_{C1}{}^*} - \left(\frac{\pounds_2}{\pounds_{C1}{}^*}\right)^2 \right]$$

Strength of the concrete strip between the cracks is verified by the relative length of the compressed zone - ζ_c of conditions:

$$\sigma_2 / \zeta_c \le \beta R_c, \ \zeta_c = \omega/(1+ \sigma_{at} / \sigma_{aw} * (1 - \omega/1.1)),$$

where:

ω – ratio curves completeness of uncracked concrete is defined in accordance with Eurocode 8 [2].

Estimated maximum vertical force N_{pr} can be determined taking into account the calculated eccentricity (for option of rectangular cross section without transverse walls):

$$N_{pr} = R_c * L * \delta * (1 - 2_{eo} \backslash L), \ \text{when} \ e_o < 0.475L,$$

$$\text{otherwise:} \ N_{pr} = 0.5R_c * x * \delta * (L - x\backslash 3)$$

where: δ – wall thickness, L- wall length x - length of uncracked concrete stripe:

$$x = 1.5 * L(1 - 2eo\backslash L), \ \text{when} \ e_o < 0.475L$$

$$\text{otherwise:} \ x = \left[\frac{N_i + {}^\circ{}_h f_{sy} A_s}{0.5 R_c * ¢} \right], \ \text{or:} \ x = L * \boldsymbol{\xi}_{min}$$

where:

φ – Ratio taken 0.95, 0.9, 0.8, 0.75 respectively at the estimated seismicity 6, 7, 8, 9 points respectively and forces obtained by the spectral method at forces derived from accelerograms $\varphi \ge 1.0$; $R_c = \ddot{} f_c$;

$\boldsymbol{\xi}_{min}$ – relative height of the concrete compressed zone according to EN 1992 Eurocode 2:

$$\xi_{min} = \omega/(1 + R_{sy} / \text{fsy} * (1 - \frac{\mu}{1.1})); \quad \omega = k_s - 0.008 fc ;$$

where: k_s= 0.85 or 0.8 for heavy or fine-grained concrete respectively.

The shorter length of uncracked concrete x is assumed into the calculation from those defined taking into account the flexural nature of the wall.

When calculating the walls and piers using efforts derived from accelerograms it is

important to get the tension descend in the elements with a crack in the primary stages of inelastic behavior development (crack opening, fluidity of reinforcement, etc.). These so-called "first losses"$\Delta\tau_1$ (Fig. 3a) with the rated forces according to the accelerograms are determined by linear current shear modulus G_n^{sek} in form of:

$$G_n^{sek} = \sim_n G_o / \sim_o ; \quad {}^{2}{}_u = {}^{-}{}_{упруг} G_n^{sek} ; \quad \Delta\tau_1 = \tau_{упруг} - \tau_u ; \quad {}^{-}{}_{упруг} = \tau_{упруг} / G_o ,$$

where: G_o - initial shear modulus;

$\sim_n$ – nonlinear ratio which is determined by the impact of shear in accordance with the recommendations of Yamaguchi [1] for the bending, shear and flexural-shear wall nature of the behaviour:

$$\sim_n^{изгиб} = 0.004 + 8.46 \sum_{Acol}^{Acol} + 7.88\, p_w - 0.021 \sim_s + 0.183 \frac{\pm_o}{f_c};$$

$$\sim_n^{изг-сдв} = 0.032 + 16.0\, p_w + 0.04 \sim_s + 0.35 \frac{\pm_o}{f_c};$$

$$\sim_n^{сдвиг} = 0.417 - 30.9\, p_w + 0.032 \sim_s - 0.139 \frac{¢}{h_{col}}$$

Strength area of the wall on N~Q, N~M in accordance with the implemented methodology is presented in graphical form (black shows a generalized calculated dependence and by yellow the proposed one (see Figure 4).

Table 2 demonstrates the experimental data from experiments provided by Cardenas A, Russell H. – for the beam-wall sample with dimensions of 1905x1905x76mm with the various schemes reinforcement, experiments at KPI and named by Lazo TSNIIE – sample beam-walls with dimensions of 2400x2700x100mm, experiments performed Paylay T., Pristley M. [1] - beam-wall sample with dimensions of 3000x2350x100мм.

Conclusions: Comparison of experimental data with the results of calculations confirmed that the technique implemented in PS "Seismic strength of diaphragms-2" provides lower confidence ±10% reliability interval and allows performing the expert evaluation of the strength of the diaphragms for various procedures.

Tensions in the elements of the diaphragm can be defined using the spectral method of calculation and accelerograms as well, and differs it from the PS "Seismic strength diaphragms" implemented in EEH-1, EEH-2 [7, 8]. First losses calculation methodology allows making an estimate of the carrying capacity of the diaphragm under the forces of accelerograms obtained taking into account the non-linear plastic walls behavior [18].

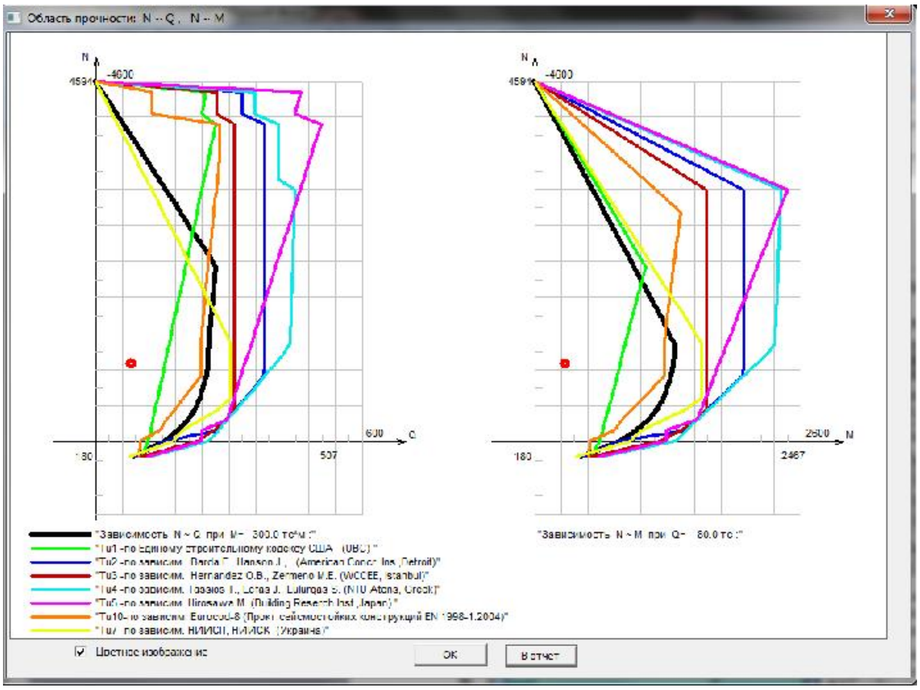


Figure 4. Area of the wall strength for N ~ Q, N ~ M by various techniques.

Comparison of the diaphragms carrying capacity Table 2

Sample identifier, geometry **L, H, D_t (cm)**	Loads: **N, Q,** (кН) **M** (кН*м)	Compressive, tensile strength of concrete, ***R_{bc}, R_{bt} (MPa)***	Horizontal vertical reinforce ***F_{ax}, F_{az} (%)***	Volume fringing reinforce ***F_{fr} (%)***	Experiensed shear strength ***Q_{ex}, (кN)***	Rated shear strength ***Q_{rat}, (кN)***	Rated limit torque ***M_{rat} (kNm)***	Designed safety factor	Error *(%)*
SW-7 191x191x8	-0.75, 518.7, 988.1	43.0, 4.3	0.13, 0.23	0.36	518.7	432.7	893.0	0.834	-16.6
SW-8 191x191x8	-0.75, 569.4, 1084.7	42.5, 3.9	0.13, 0.30	0.43	569.4	489.2	916.0	0.844	-15.6
SW-10 191x191x8	-0.75, 306.4, 583.7	40.3, 3.9	0.0, 0.165	0.165	306.4	321.8	579.0	0.993	+5.03
SW-13 191x191x8	-0.75, 631.6, 1084.7	43.4, 4.3	0.09, 0.30	0.39	631.6	540.8	1034.0	0.856	-14.4
KPI, Lazo-1 240x270x10	-1500, 650, 600	20.0, 1.8	0.55, 0.55	0.308	650.0	738.1	1974.0	1.135	+13.5
KPI, Lazo-2 240x270x10	-1000, 630, 400	20.0, 1.8	0.55, 0.55	0.308	630.0	628.1	1834.0	0.997	-1.0
ЦНИИЭП 240x270x10	-500.0, 400, 625.0	20.0, 1.7	0.5. 0.5	1.5	400.0	103.4	300.2	0.26	+29.7
Paylay T., Pristley M. 300x235x10	-1.6.0, 80, 216.0	27.2, 2.5	0.807, 1.608	1.5	80.0	71.8	193.9	0.897	-11.3

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29-30 May 2014, Tbilisi, Georgia

SIMULATION OF SEISMIC ACTION FOR TBILISI CITY WITH LOCAL SEISMOLOGICAL PARTICULARITIES AND SITE EFFECTS

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Abstract: *The utilization of time histories of earthquake ground motion has grown considerably in the field of earthquake engineering. It is very unlikely, however, that recordings of earthquake ground motion will be available for all sites and conditions of interest. Hence, there is a need for efficient methods for the simulation of strong ground motion for a given region.*

Due to lack of the real strong ground motion records the objective of this research is to develop a methodology for rapid generation of horizontal and vertical components of earthquake ground motion at any site for Tbilisi region (within 50 km). The model developed in this study provides simulation of ground motion over a wide range of magnitudes and distances at 8 earthquake source s zones of Tbilisi region.

The research includes two main topics: (i) the stochastic simulation of earthquake ground motion at a given site of the city of Tbilisi; (ii) the estimation of acceleration time histories at a given site using the direct method of engineering seismology considering soil conditions based on the theory of the reflected waves and (iii) calculation of horizontal and vertical elastic response spectra for main sites of Tbilisi territory.

The simulation procedure typically consists of multiplying deterministic modulating function with a stationary process of known power spectral density.

The obtained results in the terms of elastic response spectra can be widely applied in the practice of earthquake engineering in Georgia.

Keywords: Ground, Motion, Seismic, Stochastic, Wave, Response, Spectra

1. INTRODUCTION

In the practice of earthquake engineering an earthquake effect quantitatively is classified according to the seismic scale and by the building code. For this purpose in the seismic scale are used Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Peak Ground Displacement (PGD).

In the building code the seismic action usually is represented by an elastic ground acceleration response spectra and the ground acceleration time-histories.

It should be noted that, each earthquake represents individual process, which is generated under certain geographic and geological conditions, its destructive effect first of all depends on the seismic source magnitude and the epicentral distance. The elastic response spectrum shape or outlines of a spectral curves of dynamic coefficient depends on the earthquake generation mechanism and ground response in the site of interest. Therefore, the elastic response spectra defined according to the recorded accelerograms in different regions, differ from each other and reflect only local site conditions.

The time-histories dynamic analysis provides the evaluation of seismic demand of structures using the recorded and artificial or simulated accelerograms that give information on earthquake intensity, its frequency content and duration, i.e. it does not exclude time factor as it occurs in the response spectrum analysis.

Proceeding from the regulations on seismic action basic conception given in the EC8 (European Standard EN 1998-1:2004), selection of the elastic response spectrum shape in the country or part of the country is possible from the certain country National annexes that are worked out by local Authorities. In accordance with the recommendations suggested by EC-8 deep geological data of the construction site should be considered and the horizontal and vertical elastic response spectra should be computed taking into account the seismic sources and the earthquake magnitudes generated from them.

In general, lack of the strong earthquake records statistic package creates certain problems for the elastic response spectra and the dynamic coefficient spectral curves elaboration. Note that, an example of such problems is the capital of Georgia – Tbilisi.

For Tbilisi region (within 50 km) records of the strong earthquakes data are limited. During last 100 years at the territory of Tbilisi city about hundred weak earthquakes took place. Local strong earthquake occurred only on April 25, 2002, under the central part on the city with magnitude

M=4.5 and recorded on the bedrock peak horizontal acceleration of 0,11g, which was amplified to the range of 0.20 to 0.30g due to dynamic response of surface soil deposits.

It is evident that, on the basis of the weak and rare earthquakes real records formation of the seismic action specified regional model is impossible. In such conditions the most straightforward procedure is to generate ground motion time histories using of regional earthquake sources zones parameters and classification according to the soils seismological and geological properties spread at Tbilisi territory.

At the same time, according to the Georgian building code (Building Code, PN 01.01-09, 2009) Tbilsi is located in the seismic zone of intensity 8 degree by the MSK-64, with a maximum horizontal acceleration equals 0.17g and a return period of earthquakes 2500 years (2%/in 50 years). The spectral dynamic coefficient is determined for grounds of hard, medium and soft categories and without special investigations its maximum value for all three categories grounds is accepted equal to 2.5.

It should be noted, that from the earthquake source zones of Tbilisi region at the territory of the city are expected the earthquakes with magnitudes M=5.0–7.0 and corresponding seismic generated kinematics of shifting as reverse and dextral strike slip (Rekvava and Mdivani, 2011). It follows from this that it is necessary to define more precisely given in the National code the spectral curves of dynamic coefficient considering the data of Tbilisi region seismological and geological properties of the grounds spread at city territory. In such conditions it is important simulation of the expected regional spatial seismic action in the form of the acceleration time-histories and elaboration the elastic response spectra and the three components spectral curves of dynamic coefficient for various sites of Tbilisi.

Several models exist in the literature for numerical simulation of earthquake ground motion. The ground motion simulation models can be classified into two categories: geophysical models and engineering models, which estimate the ground motion in fundamentally different ways (Rezaeian and Der Kureghian, 2010; Rekvava and Mdivani, 2011).

The objective of this study is to develop a methodology for simulation ground motions and evaluation the elastic response spectra and the three components spectral curves of dynamic coefficient at any site for Tbilisi territory considering the regional seismological characteristics and geological conditions for the site of interest. The proposed approach includes three main topics: (i) the stochastic simulation of earthquake ground motion at a given site of the city of

Tbilisi; (ii) estimation of acceleration time histories at a given site using the direct method of engineering seismology takeing into account a soil properties based on the theory of the reflected waves (iii) calculation of the horizontal and vertical elastic response spectra and corresponding the spectral curves of dynamic coefficient for main sites of Tbilisi territory.

2. STOCHASTIC SIMULATION OF EARTHQUAKE GROUND MOTION

For simulation of possible seismic ground moutions on the territory of Tbilisi city in this paper is employed the discrete nonstationary Gaussian stochastic process represented as (Rekvava and Mdivani, 2010)

$$A_{gi}(t) = E_i(t)\ X_i(t), \qquad (i=1,2,3) \tag{2.1}$$

where $A_{gi}(t)$ determines of ground accelerationin in the direction of three principal orthogonal axses with zero cross correlation between of components; $E_i(t)$ is the deterministic normalized envelope function or modulating function; $X_i(t)$ represents a typical realization

of the stationary filtered white-noise process.

Normalized stationary random function with zero mean and unit-variance is characterized by $K(\tau)$ function of correlation as

$$K(\tau)=e^{-\alpha_j|\tau|}(\cos\omega_j\tau+\alpha_j/\omega_j\sin\omega_j|\tau|) \tag{2.2}$$

where α is correlation coefficient characterizing bandwidth of the process ; ω is circular process frequency; j represents a ordinal number of process .

The modulating function $E_i(t)$ is defined in terms of so-called Berlag impulse and with $|E_i(t)|_{max}=1$ is given by

$$E_i(t) = \varepsilon\, t \exp(1-\varepsilon t) \tag{2.3}$$

where ε controls the shape of the envelope function and determines the effective duration and process nonstationarity.

Generalizing the form in (2.1), the horizontal and vertical components of the process can be written as

$$A_{g1}(t) = \text{©}\pm_1 \text{£}t\exp(1-\text{£}t)x_1(t)$$

$$A_{g2}(t) = \text{¦}\pm_2 \text{£}t\exp(1-\text{£}t)x_2(t) \tag{2.4}$$

$$A_{g3}(t) = {}^{3}\pm_{3}\pounds t\exp(1-\pounds t)x_{3}(t)$$

where $\pm_i$ is a mean square value of acceleration in the direction of principal axses and denotes random process intensity that is defined by its variance; k, ¦ და ³ are corrective factors of the value of the horizontal and vertical components which are accordingly equal to 1.0, 0.85 and 0.7.

Thus, the formulation in (2.4) is completely determined with fixed values of omoinant frequency ω_j using three parameters: α, ε and σ which are depended on regional seismological and geological conditions or in the simple form on the earthquake magnitude, hypocentral distance, dominant frequency and ground characteristics at the site.

On the basis of propsed stochastic ground motion model formulated in (2.4) the software package ACCSIM (Rekvava and Mdivani, 2011) was developed, which allows to generate the multiple artificial accelerograms of the predicted earthquakes.

3. PARAMETERS ESTIMATION

Territory of Tbilisi city from the earthquake sources zones of Tbilisi region the two various expressions are applied (Javakhishvili et al.,1998):

for small earthquakes (M_s<6)

$$I_{Tb}=1.5M_s-3.4\lg R+3.1 \tag{3.1}$$

for strong earthquakes (M_s>=6)

$$I_{Tb} = 1.5M_s - 4.7\lg R + 4.0 \tag{3.2}$$

where M_s is surface-wave magnitude; $R=(\Delta^2+h^2)^{1/2}$ is hypocentral distance; Δ is epicentral distance; h -focal depth.

The resulting equation for larger horizontal values of peak horizontal acceleration is defined by (Smit et al., 2000)

$$\log PGA_{h1}=0.72+0.44M_s-\log R-0.00231K+0.28p \tag{3.3}$$

and

$$K = \sqrt{\Delta^2 + h^2 + 4.5^2} \tag{3.4}$$

where p is 0 for 50-percentile values and 1for 84-percentile.

Empirical relations between the surface-wave magnitude of the earthquake and a hypocentral distance derived for shallow-focus near-source earthquakes under an average soil site conditions are given by the following formula (Mikhailova and Aptikaev, 1996)

$$\lg T=0.15M_s+0.25\lg R+C_1+C_2\pm 0.2 \tag{3.5}$$

where C_1 is parameter of fault mechanism (C_1=-0.1 for reverse, C_1=0 for strike slip, C_1=0.1- for dextral strike slip); C_2–coefficient of influence not taken into consideration factors that is equal to 1.11;

Duration of the ground motion is computed by

$$\lg D = 0.15M_s + 0.50 \lg R + C_1 + C_2 + C_3 \pm 0.30 \tag{3.6}$$

where C_1 – is parameter of fault mechanism (C_1=-0.25 for reverse, C_1=0.0 for strike slip, C_1=-0.12 for dextral strike slip; C_2=-0.15 for hard ground; C_2=0 for medium ground; C_2=0.15 for soft ground; a mean value of ratio C_3 is equal to 1.3.

The calculated parameters for the borderline territory of Tbilisi city are listed in Table3.1. It should be noted that for computation of PGA has been used 84-percentile. On the basis of empirical data the more intensive horizontal component of PGA_{h1} is obtained 1.28 times greater than other one and the vertical component is 2/3 of the maximum horizontal component.

For 10 sites of Tbilisi city territory (350 square kilometers) were also determined (Fig.3.1) minimum hypocentral distances, PGA for 2% and 1% probabilities of being exceeded in 50 years, values of the dominant periods and duration of oscillation. As an illustration in Table 3.2 are given parameters generated from the high potential seismic generating zone №3 which is situated to the north-west of Tbilisi region. Note, that values of PGA in Table 3.2 correspond to 2% probability of being exceeded in 50 years and are by 15% less than computed for 1% probability of being exceeded in 50 years.

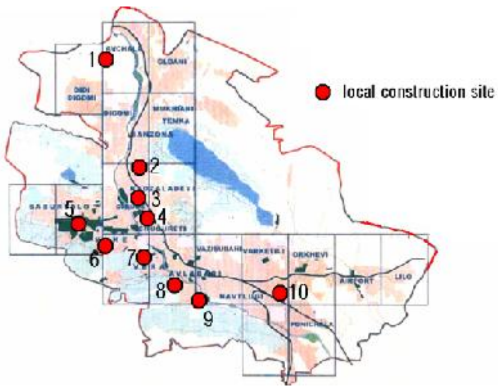

Fig. 3.1. Location of the sites on the territory of the city

The main parameter ω_j of the ground motion model has been determined based on the formula (3.5) using the expression:

$$\omega_j=2\pi/T_j \tag{3.7}$$

The value of the correlation degree characterizing parameter α was evaluated based on the analysis of the earthquakes records data (Rekvava and Mdivani, 2011) depending on ω and for 1(x), 2(y) and 3(z) components consists of

$$\alpha_{j1}=0.204\omega_j; \quad \alpha_{j2}=0.253\omega_j; \quad \alpha_{j3}=0.41\omega_j; \tag{3.8}$$

The mean square value of acceleration σ was accepted considering that

$$\sigma_i=PGA_i/3, \quad i=1,2,3 \tag{3.9}$$

The parameter ε is determined on the basis of the given duration of intensive iscillations above-mentioned records and is equal to

$$\varepsilon_j = 0.02\omega_j \tag{3. 10}$$

Thus calculated parameters are represented in Table 3.3, but mmean square values of the horizontal and vertical accelerations for earthquake generated from the high potential seismic generating zone №3 are given in Table 3.4.

Table 3.1. Quantitative Characteristics of the Predicted Ground Motion on the Borderline of Tbilisi City

Zone №	R (km)	I_{TB} (deg)	T (sec)	D (sec)	PGA_{h1} (m/sec^2)	PGA_{h2} (m/sec^2)	PGA_{h3} (m/sec^2)
From focus with M=5							
2	10.6	7	0.13	1.63	2.11	1.65	1.41
5	8.38	7	0.12	1.45	2.20	1.72	1.47
From focus with M=5.5							
7	10.7	8	0.15	2.06	2.38	1.86	1.59
From focus with M=6							
4	11.2	8	0.18	2.66	2.53	1.98	1.69
6	10.0	8	0.18	2.51	2.57	2.01	1.72
8	16.0	7	0.2	3.18	2.38	1.86	1.67
From focus with M=6.5							
1	29.4	7	0.28	5.42	2.32	1.81	1.55
From focus with M=7							
3	16.3	9	0.28	5.08	2.81	2.20	1.88

Table 3.2. *Quantitative Characteristics of the Predicted Ground Motion for the Concrete Sites of Tbilisi City*

Zone №	M	Parameters	Site №									
			1	2	3	4	5	6	7	8	9	10
3	7	R_{min}, km	15.6	20.0	21.2	23.3	18.1	21.6	25.1	26.8	30.0	33.7
		PGA_{h1}, (m/sec^2)	2.83	2.72	2.69	2.65	2.77	2.69	2.62	2.58	2.53	2.47
		PGA_{h2}, (m/sec^2)	2.21	2.13	2.10	2.07	2.16	2.10	2.04	2.02	1.98	1.93
		PGA_{h3}, (m/sec^2)	1.89	1.81	1.80	1.77	1.85	1.79	1.74	1.72	1.69	1.65
		T, sec	0.281	0.299	0.303	0.31	0.291	0.305	0.316	0.321	0.331	0.34
		D, sec	4.98	5.63	5.8	6.08	5.35	5.86	6.3	6.52	6.9	7.31

Table 3.3. Parameters for Genaration of Regional Synthetic Accelerograms

Zone 1	ω_j sec^{-1}	α_{j1} sec^{-1}	α_{j2} sec^{-1}	α_{j3} sec^{-1}	ε_j sec^{-1}	$\Delta t=0.04T_j$ sec
From focus with M=5						
2	48.3	9.85	12.1	19.8	0.97	0.005
5	52.3	10.67	13.1	21.4	1.05	0.0048
From focus with M=5.5						
7	41.8	8.53	10.5	17.1	0.84	0.006
From focus with M=6						
4	34.8	7.1	8.7	14.3	0.7	0.007
6	34.8	7.1	8.7	14.3	0.7	0.007
8	31.4	6.4	7.9	12.9	0.63	0.008
From focus with M=6.5						
1	22.4	4.5	5.6	9.2	0.45	0.011
From focus with M=7						
3	22.4	4.6	5.6	9.2	0.45	0.011

Table 3.4. Mean Square Values of Aaccelerations for Concrete Sites

Mean square value of acceleration m/sec^2	Probability of exceeding in 50 years	Site #									
		1	2	3	4	5	6	7	8	9	10
σ_1	2% /50	94	91	90	88	92	90	87	84	84	82
	1% /50	109	104	103	102	106	103	100	97	97	95
σ_2	2% /50	73	71	70	69	72	70	68	67	66	64
	1% /50	85	81	81	79	83	80	78	77	76	74
σ_3	2% /50	63	60	60	59	62	60	58	57	56	55
	1% /50	72	70	69	68	71	69	67	66	65	63

4. DETERMINATION OF MULTILAYER GROUND MOTION BASED ON THE THEORY OF REFLECTED WAVES

Method of the multiple reflected waves gives a possibility to determine for a certain concrete territory by geologic profile conformity to natural laws of seismic oscillation of the multilayer ground surface, under motion of rock bed as foundation according a law of given accelerogram.

For analytical drawing of accelerogram of the ground surface oscillation let consider, a wave picture at any time in the ground area, with different thickness and horizontal borderline. It is accepted an assumption that the ground is elastic and waves are propagated in the vertical direction (Fig. 4.1). In the form of seismic influence in this case is used recorded on the rockbed accelerogram from the database of ground motions with known earthquake (Rekvava and Mdivani, 2011).

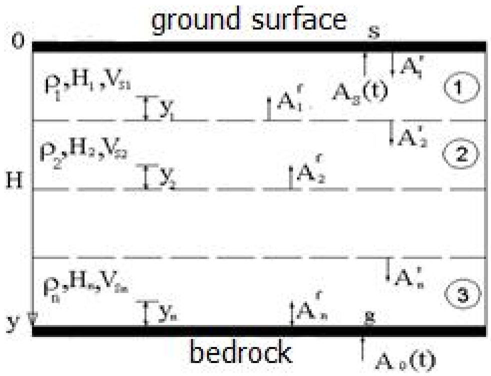

Figure 4.1. Design model of nonhomogeneous ground

In the Fig. 4.1 are accepted following designations: $A_i^f(t)$ is value of acceleration's wave function at the t time on the bottom level of the i-th layer; $A_i^r(t)$ is value of acceleration's wave function at the *t* time on the top level of the i-th layer; $A_0(t)$ is accelerogram on the level of hard rock.

For any i-th layer of ground the wave equation of shear oscillations can be written as (Napetvaridze, 1973):

$$\frac{\partial^2 A_i(t)}{\partial t^2} - V_{s\,i}^{\,2}\frac{\partial^2 A_i(t)}{\partial y^2} = 0 \qquad (4.1)$$

where A_i(t) is the acceleration of ground layer particles; t is the time; y represents the coordinate of ground particles in the vertical direction; V_s is the velocity of the shear wave propagation.

Solution of the equation (4.1) is given by

$$A_i^r(t) = \sim_{i-1,i} A_{i-1}^r\left(t - {}^{2}{}_{i-1}\right) + \quad{}_{i,i-1} A_i^f\left(t - {}^{2}{}_{i}\right) \qquad (4.2)$$

where $\sim_{i,i-1}$ is the factor of refraction under passing of wave from i-1-th to i-th layer; ${}_{i,i-1}$ the factor of wave reflection on the borderline between i and i-1 layers; ${}^{2}{}_{i}$ represents the time of wave passage in the i-th layer (${}^{2}{}_{i} = H_i / V_{si}$, where H_i and V_{si} are accordingly the thickness of ground layer and the velocity of the shear wave propagation in the i-th layer).

$\sim$ and factors are defined by

$$\sim_{i-1,1} = 2\,{}^{\circ}{}_{i-1} V_{s,i-1} / \left({}^{\circ}{}_{i-1} V_{s,i-1} + {}^{\circ}{}_{i} V_{si}\right) \qquad (4.3)$$

$${}_{i,i-1} = \left(V_{si}\,{}^{\circ}{}_{i} - {}^{\circ}{}_{i-1} V_{s,i-1}\right) / \left(V_{si}\,{}^{\circ}{}_{i} + V_{s,i-1}\,{}^{\circ}{}_{i-1}\right) \qquad (4.4)$$

where ${}^{\circ}{}_{i}$ is a density of i-th ground layer.

Hence, finally solution of the direct problem of engineering seismology can be represented by the recurrent relations as (Napetvaridze, 1973):

$$
\begin{aligned}
&A_1^r(t) = A_1^f(t - {}^2{}_1),\\
&A_1^f(t) = \sim_{2,1} A_2^f(t - {}^2{}_2) + \quad {}_{1,2} A_1^r(t - {}^2{}_1),\\
&A_2^r(t) = \sim_{2,1} A_1^r(t - {}^2{}_1) + \quad {}_{2,1} A_2^f(t - {}^2{}_2),\\
&A_2^f(t) = \sim_{3,2} A_3^f(t - {}^2{}_3) + \quad {}_{2,3} A_2^r(t - {}^2{}_2),\\
&A_i^r(t) = \sim_{i-1,i} A_{i-1}^r(t - {}^2{}_{i-1}) + \quad {}_{i,i-1} A_i^f(t - {}^2{}_i),\\
&A_i^f(t) = \sim_{i+1,i} A_{i+1}^r(t - {}^2{}_{i+1}) + \quad {}_{i,i+1} A_i^r(t - {}^2{}_i),\\
&A_n^r(t) = \sim_{n-1,n} A_{n-1}^r(t - {}^2{}_{n-1}) + \quad {}_{n,n-1} A_n^f(t - {}^2{}_n),\\
&A_n^f(t) = \sim_{n+1,n} A_0(t) + \quad {}_{n,n+1} A_n^r(t - {}^2{}_n).
\end{aligned}
\tag{4.5}
$$

Oscillation of the particles from the bottom of i-th layer on the level of y_i can be calculated according to

$$
A_i^{y_i}(t) = A_i^f(t - y_i / V_{si}) + A_i^r(t - (H_i - y_i)/V_{si}) \tag{4.6}
$$

Thus, the developed algorithm of solution the direct problem of engineering seismology is realized by the software package GAFART (Rekvava and Mdivani, 2011) .

5. SIMULATION RESULTS FOR TBILISI CITY SITES

The computer code ACCIM was used for generation of the horizontal and vertical components of synthetic accelerograms corresponding possible seismic source zones of Tbilisi region, given in Table 3.1. Discrete step of the simulated accelerograms was taken equal to 0.04T. When assessing the probabilistic mean elastic response spectra and the dynamic coefficient spectral curves for all sites, which are presented in Fig 3.1, the required number of realizations was reduced for each synthetic accelerogram up to 20 realizations. The most novel aspect of this extension is elaboration of proper the three-components dynamic coefficient spectral curves, which are computed for a 50 years exposure time and 2% and 1% probabilities of exceeding.

In Table. 5.1 are represented values of the probabilistic mean spectral dynamic coefficient of synthetic motion, generated from the source zones N3 and N6 for the sites situated in the centre of Tbilisi city.

Using the software GAFART was studied an influence of a typical earthquake and local geological conditions upon forming the elastic acceleration response spectra for the abovementioned sites. With that and in view from the data set (Rekvava and Mdivani, 2011) was selected five recorded on the bedrock accelerograms (EL Centro-1940, M = 6.7, Santa Barbara-

1980, Montenegro-1979, $FM = 7.0$, Friuli 1976, $M = 6.0$, and Tbilisi-2002, $M = 4.5$), which are different from each other by parameters of PGA, dominant period (T) and duration (D), but by the magnitude and epicentral distance are close to predicted earthquakes characteristics for Tbilisi region.

Table 5.1. Maximum Values of Dynamic Coefficient for 2%/50 and 1%/50 years

Zone	M	Component	Site											
			N1		N2		N5		N7		N8		N10	
			β_{max}		β_{max}		β_{max}		β_{max}		β_{max}		β_{max}	
			2%	1%	2%	1%	2%	1%	2%	1%	2%	1%	2%	1%
3	7	x	2.5	2.7	2.4	2.5	2.5	2.8	2.4	3.0	2.5	3.0	2.4	2.6
		y	1.8	2.2	2.0	2.4	2.6	3.0	2.2	2.6	2.5	2.5	2.5	2.7
		z	2.2	3.0	2.7	3.0	1.8	3.0	2.1	2.2	2.6	2.8	1.8	2.3
6	6	x	2.6	3.0	2.0	2.9	3.0	3.7	2.35	2.5	2.6	3.2	2.3	2.5
		y	2.5	2.7	2.0	2.5	2.5	3.0	2.2	2.5	2.5	3.0	2.3	2.5
		z	3.0	3.4	2.5	2.7	2.7	3.0	3.25	3.5	2.5	2.8	2.0	2.5

Considering the soil profile properties (thickness, density, wave velocity) of these sites, received from geological test, on the basis of the abovementioned recorded ground motions were calculated the three components of time-histories on the ground surface of the sites. Then at the final phase of analysis the three components of spectral curves of dynamic coefficient have been plotted. Table.5.2 displays the effect of local soil condition on the dynamic coefficient.

According to the obtained results was computed value of the relative seismic factor as ratio of the maximum accelerations on the ground surface and on the bedrock. The analysis shows that in the given soil properties of the sites under investigation the seismic factor is changed from 1.5 to 2.6.

Table 5.2. Maximum Values of Dynamic Coefficient from Different of Earthquqkes

N	Earthquake	Component	Site					
			N1	N2	N5	N7	N8	N10
			β_{max}	β_{max}	β_{max}	β_{max}	β_{max}	β_{max}
1	EL Centro	x	2.2	3.0	3.0	2.5	3.0	2.5
		y	2.2	3.0	3.0	2.5	3.0	2.5
		z	2.2	3.0	3.0	2.5	3.0	2.5
2	Santa Barbara	x	2.7	3.5	4.0	3.0	2.5	2.7
		y	2.7	3.5	4.0	3.0	2.5	2.5
		z	2.7	3.5	4.0	3.0	2.5	2.4
3	Montenegro	x	2.5	3.5	2.5	3.0	4.0	2.5
		y	2.0	3.0	3.0	3.7	3.0	2.5
		z	3.0	2.5	2.7	3.0	2.0	3.0
4	Friuli	x	2.4	3.0	3.5	3.0	3.0	2.8
		y	2.7	3.3	3.0	2.8	2.5	2.7
		z	1.8	3.0	2.3	2.3	2.5	2.5
5	Tbilisi	x	2.5	2.0	2.1	2.5	2.3	2.3
		y	2.5	2.2	3.1	2.0	2.5	2.5
		z	2.3	2.3	2.4	3.0	2.1	2.2

6. CONCLUSIONS

The complex approach of simulation ground motion and construction the horizontal and vertical elastic response spectra and corresponding dynamic coefficient spectral curves are proposed, which account for the location of the earthquake source zones and seismological and geological characteristics of the Tbilisi region territory and concrete construction sites.

Based on the empirical relations and characteristics of the earthquake source zones the values of mean accelerations of ground motion for 2%/ and 1% probabilities of being exceeded in 50 years expected in the sites of Tbilisi city has been determined and appropriate probabilistic horizontal and vertical elastic response spectra and spectral dynamic coefficients are calculated which can be used in seismic design and analysis of structures.

On the territory under examination for the concrete construction sites in result of experimental research the dynamic parameters of soil geological layers are determined and based on the theory of reflected waves the ground motion for the sites of Tbilisi city horizontal and vertical elastic response spectra and spectral dynamic coefficients are calculated and their corrected shapes considering the local sites conditions for II and III category of soil are constructed, which can be widely applied in the practice of earthquake engineering in Georgia.

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29-30 May 2014, Tbilisi, Georgia

LOSS ESTIMATION MODELING FOR EARTHQUAKE SCENARIOS

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Risks of natural hazards caused by natural disaster are closely related to the development process of society. Disasters pose hazard to sustainable development of the country. The high level of natural disasters in many countries makes necessary to work out the national programs and strategy. The main goal of these programs is to reduce the natural disasters risk and caused losses. This problem is of vital importance for Tbilisi as it is a big city and capital of Georgia.

The attempt has been approached within the framework of the project "EMME – Earthquake Model for Middle East Region: Hazard, Risk Assessment, Economic and Mitigation". The primary scientific objective of this project was to combine analysis of the contemporary elements at risk inventories, seismicity and vulnerability to assess seismic hazard and seismic risk for the capital of Georgia – Tbilisi.

Tbilisi lies at the eastern end of the Achara-Trialeti fold-thrust belt. The North Achara-Trialeti (ATN) thrust separates the Achara-Trialeti belt from the Georgian block to the north. The South Achara-Trialeti (ATS) thrust separates the Achara-Trialeti belt from the Artvin-Bolnisi block to the south. Tbilisi lies between the eastern terminations of these two thrusts. The city is characterized with rapid increase of population density, high speed of urbanization and vulnerable infrastructure, which increases seismic risk.

The Tbilisi area had been considered as a region of relatively low seismicity. Historical seismicity in this area has been recorded since the 13th century and all known strong ($Ms > 5$) earthquakes in this area are associated with the above mentioned faults. In 2002 Tbilisi earthquake with $Ms = 4.5$ took place that was the result of seismically active, newly discovered fault passing the capital territory.

In risk assessment one of the most important parameters is the inventory of elements at risk. In urban area, elements at risk are comprised of buildings, lifeline systems, population, socio-economic activities. Extensive and comprehensive collection of element at risk inventories is vital for estimation of losses to elements at risk during the earthquake. So the first step was creation of high quality building inventory map in GIS.

The second step was to obtain seismic site conditions and amplifications map for Tbilisi city. This aspect deserves major attention since it plays considerable role in the definition of the seismic impact to be considered in the design and retrofitting of structures. The most important parameter of soil maps of seismic site conditions, the shear wave velocity in the upper 30 m section of the ground (V_{S30}) on regional scales are relatively rare since they required substantial investment in geological and geotechnical data acquisition and interpretation. For this purpose all available large-scale maps, published archived materials, as well as unpublished materials from private archives was discussed. Available geotechnical information, as well as seismic data obtained during implementation of the project was collected and analyzed.

Multichannel seismic prospecting system: RAS-24 6 was used to carry out seismic prospecing work. Combined all obtained data together with obtained geo-engineering map were summarized and local map of seismic site conditions in 25 000 scale have been evaluated figure 1, 2.

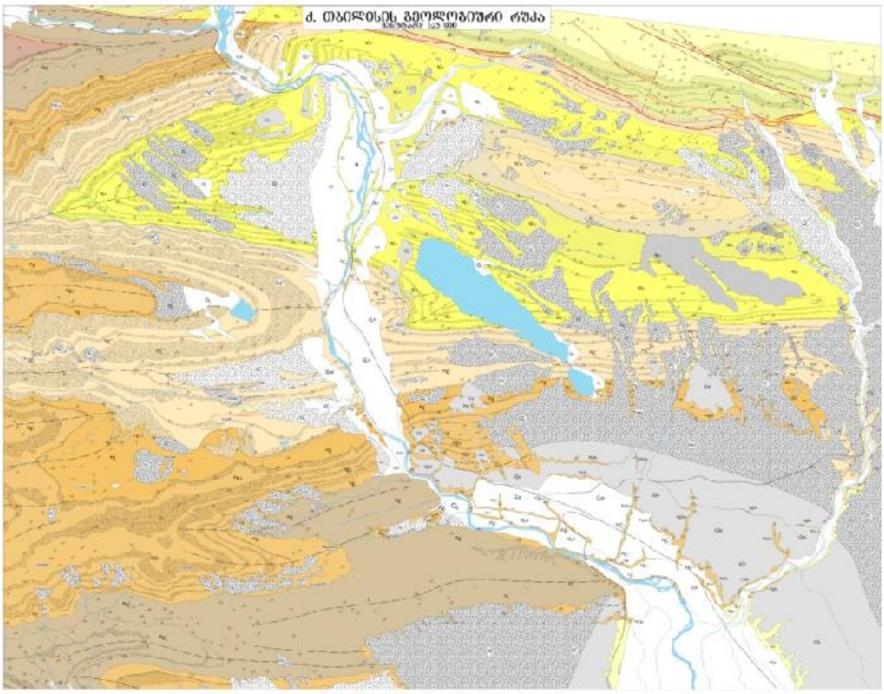


Fig. 1. Geo engineering map in 1: 25 000 scale

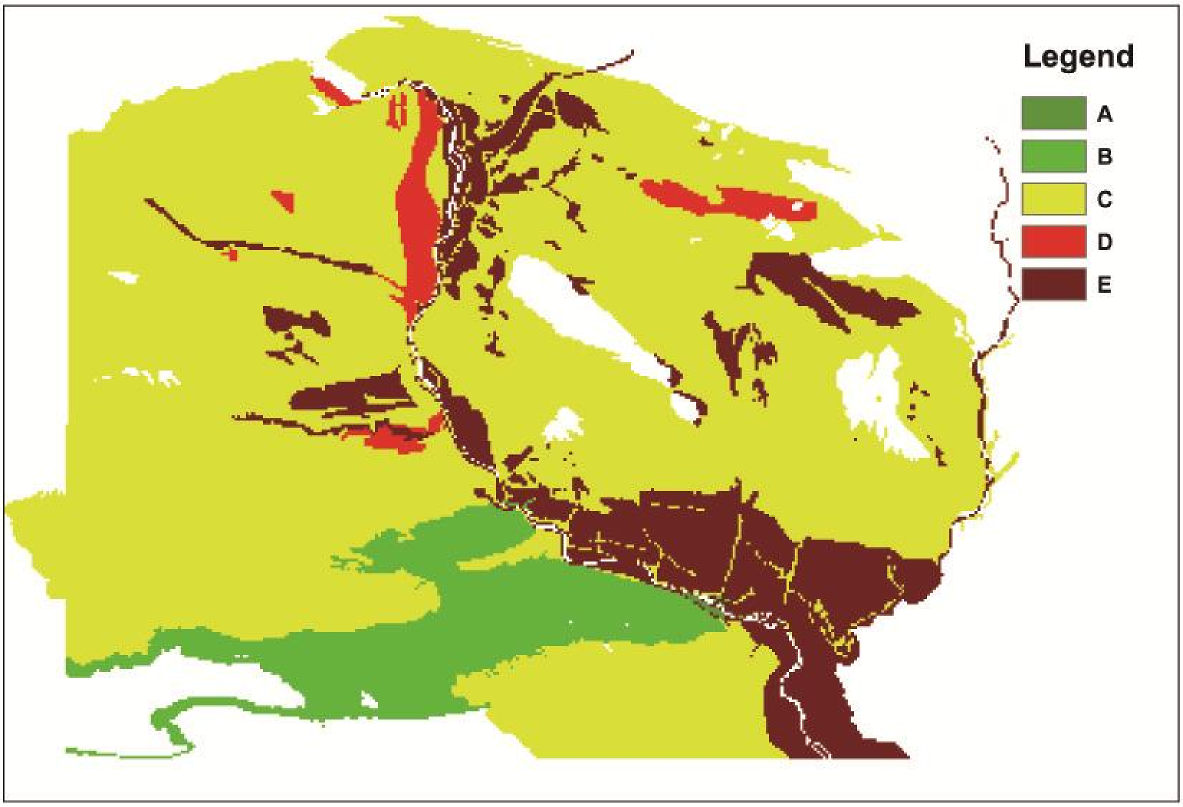
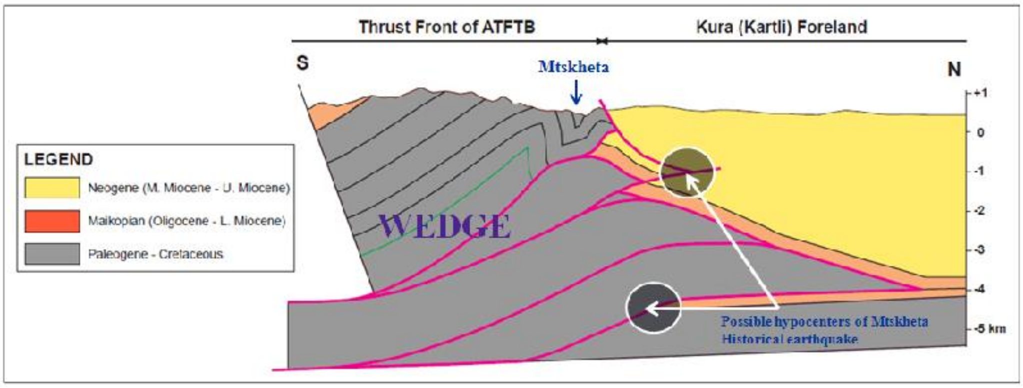


Fig. 2. Obtained Geo engineering map were reclassified by cite classification scheme in EC8

The third step was investigation of active structure of Tbilisi in large scale 1:25 000. 2 & 3D structural model of Tbilisi area show that the east Achara-Trialeti fold and thrust belt is active thin-skinned fold and thrust belt; Structure is represent by fault-related folds (fault-bend and fault-propagation), duplexes and backthrusts; Frontal part of eastern Achara-Trialeti are represent by triangle zone; Tbilisi earthquake may be related north-vergent thrust (F2 ramp fault plane – please see slide # 15) ; Historical Mtskheta earthquake may be related north-vergent thrust (F4 - wedge thrust) or back thrust (F5) Figure 3.



ATFB – Achara-Trialeti Fold and Thrust Belt

Mtskheta historical earthquake

Fig. 3. Active structure of Tbilisi in large scale 1:25 000

The next step was assessment of probabilistic seismic hazard (PSH) on the bases of selected GMPE models Cotton et al. (2006). For ranking and selecting candidate GMPEs using the data-driven testing procedure proposed by Scherbaum et al. (2004), Scherbaum et al. (2009) and Kale and Akkar (2012). Proceeding from this analysis, GMPEs (Akkar et al., 2013; Chiou and Youngs, 2008; Akkar and Cagnan, 2010; Zhao et al., 2006) were used with equal weights in the logic tree combination in the seismic hazard calculation (EMME, 2013) Figure 4, 5, 6.

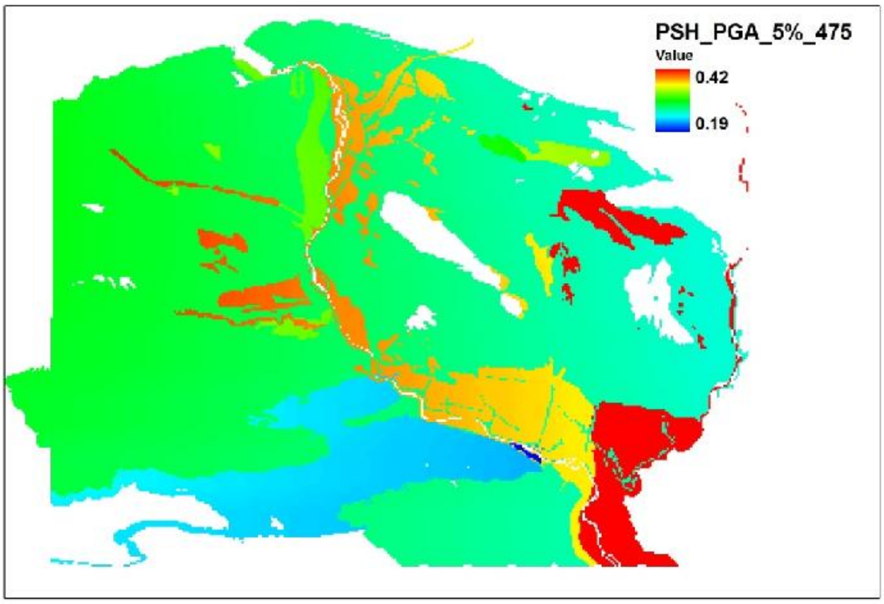


Fig. 4. PSH map for a recurrence period of 475 year

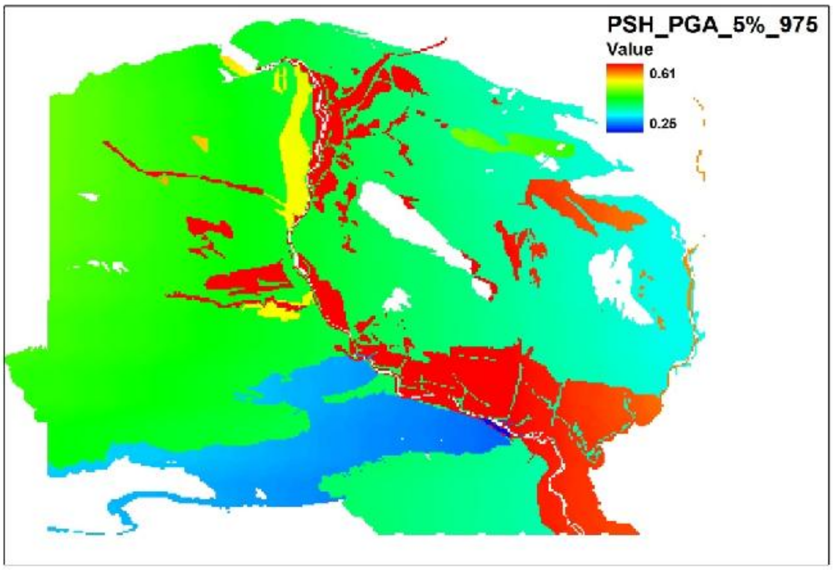


Fig. 5. PSH map for a recurrence period of 975 year

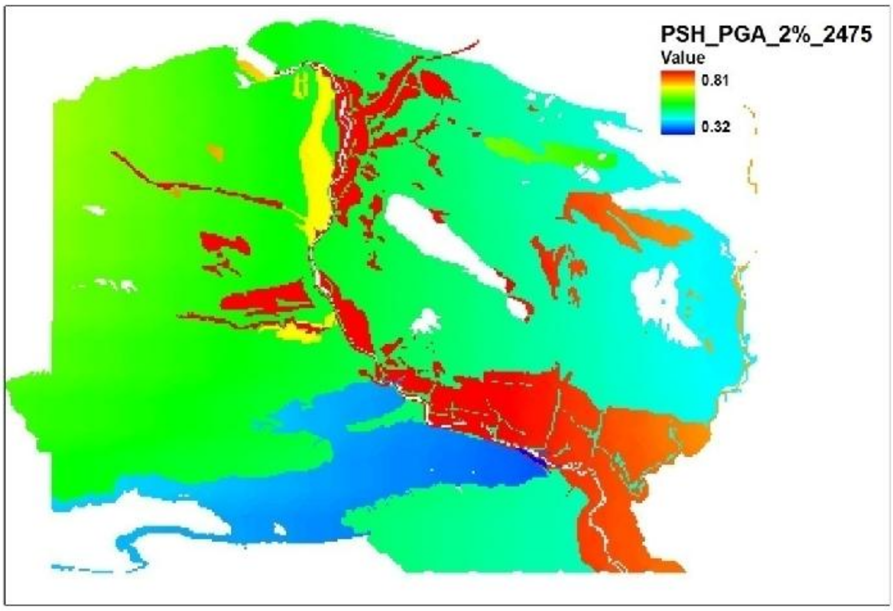


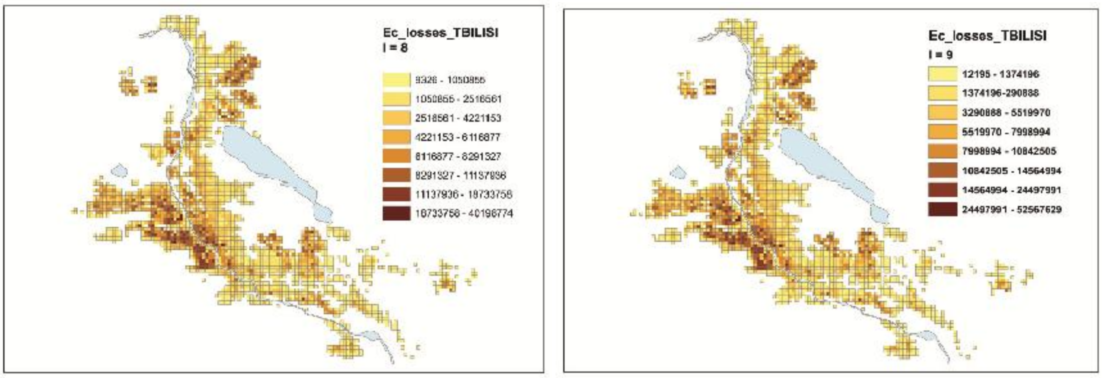
Fig. 6. PSH map for a recurrence period of 2475 year

On the bases of empirical data that was collected for Racha earthquake (*Ms* = 6.9) on 29 April of 1991 and Tbilisi earthquake (*Ms* = 4.5) on 25 April of 2002 some intensity based vulnerability study were completed. These records does not explain the building height. For this, according to the actual observable inventory and expert opinion the following assumptions are made: simple stone are consider as mid-rise M3M ; Pre code masonry rc floors are considered as low rise M6LPC; MC masonry rc floors are consider as mid-rise M6LMC. Also industrial types

of building large panel buildings that do not have analog in European buildings were investigated. The vulnerability for large block buildings were developed on the data bases of the same house in Irkutsk on the bases of experts' judgment.

Probabilistic seismic risk assessment in terms of structural damage and casualties were calculated for the Tbilisi city for 1 km grid cells using obtained results. This methodology gave prediction of damage and casualty for a given probability of recurrence, based on a probabilistic seismic hazard model, population distribution, inventory and vulnerability of buildings. ELER software (Hancilar et al., 2010) were used for this calculation. The approach used in damage estimation is to obtain a normally distributed cumulative damage probability for each building type. The damage probability distribution is a function of each building's vulnerability and ductility parameters (Lagomarsino and Giovinazzi, 2006).

Very important is to estimate the initial cost of building for assessment of economic losses. From this purpose the attempt was done and the algorithm of this estimation were prepared taking into account obtained the inventory. Build quality, reliability and durability are of special importance to corresponding state agencies and include different aesthetic, engineering, practical, social, technological and economical aspects. The necessity that all of these aspects satisfy existing normative requirements becomes evident as the building and structures come into exploitation. The long term usage of building is very complex. It relates to the reliability and durability of buildings. The long term usage and durability of a building is determined by the concept of depreciation. Depreciation of an entire building is calculated by summing the products of individual construction unit' depreciation rates and the corresponding value of these units within the building. This method of calculation is based on an assumption that depreciation is proportional to the building's (constructions) useful life. We used this methodology to create a matrix, which provides a way to evaluate the depreciation rates of buildings with different type and construction period and to determine their corresponding value. Finally economic losses were calculated for some possible scenario earthquakes. In Figure 7 a, b are presented Tbilisi earthquakes scenario calculation in terms of damage an economic losses.



a b

Fig. 7. Economic losses for some possible earthquakes for Tbilisi city:
a) *I* = 8 MSK; b) *I* = 9 MSK

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29-30 May 2014, Tbilisi, Georgia

SUPER-PLASTICIZERS IN TECHNOLOGY OF MANUFACTURING OF ENERGY-SAVING AERATED-SANDWICH ARTICLES

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Abstract: *Technology for production of light concrete aerated-sandwich articles from by single-stage moulding is worked out. Concretes are prepared on cement, as well as, on sand-lime binders. Introduction of super-plasticizing additives into concrete mixture makes easy vibration-lamination of concrete, widens range of used aerated binders, results in reduction of vibration time and water consumption and improves layer structure of concrete. Consequently, higher density and strength concrete with good thermal insulation properties and structural layer is obtained. Double-layer (sandwich) porous elements - prisms - of unite concrete mix prepared with single-stage moulding were investigated at long action of statically applied compressive load. The research showed that creep flow of layers of air-concrete as well as heavy-weight porous concrete are of diminishing character. Centering of double-layer elements for compressive load should be done by physical center.*

Keywords: Super-plasticizer, Vibration lamination, Aeration, Thermal Insulation, Sand-Lime Binder, Single-Stage Moulding Technology

Development of construction during recent years envisages most rational and economic use of available industrial capacities and material resources. In order to achieve that big attention is paid to improvement of modern technologies and materials. Use of sandwich concretes and reinforced concrete articles manufactured of local materials seems rather promising, as sandwich layers ensure both thermal-insulating and good load-bearing properties. Area of use of sandwich concretes practically is similar to area of use of single-layer (solid) concretes of same thickness. At the same time articles made of sandwich concretes significantly reduce operational costs related with thermal losses. In articles produced by multi-staged technology (with separate

moulding of layers) the contact between layers is often brakes during their use, resulting in additional costs.

Table 1 Composition of concrete mixes and their creeping, yield and lamination parameters

Mix composition	Mix creep, kJ cm	Yield according to Soutard D cm	Proportion of mix components, % of mass							Vibration Time for lamination, min	lamination
			Sand-lime binder	Sand	Gravel	Grinded sand	Aluminum powder	Water	SP KM-30		
I sup	20-22	17-18	28	13	24	17	0.04	17.82	0.14	1	Separated
I	17-20	16-17	26.84	12.46	23	16.3	0.038	21.36	0.14	1.5-2	Separated
II sup	20-21	17-18	24.5	15.5	28.5	16	0.035	16.41	0.125	1-1.5	Separated
II	14-16	17	23.1	15.2	17.9	14.22	0.033	19.54	-	1.5-2.5	Separated
III sup	20	17-19	20.86	18	33	13	0.03	15	0.11	1-1.5	Separated
III	12-16	17-19	20.14	17.37	31.82	12.54	0.029	18.09	-	1.5-2.5	Separated

Table 2 Composition of concrete mixes

Type of concrete	Ratio of thermal insulating and structural layers	Amount of materials (in kg) per 1 m^3								
		Binder		Sand	Gravel	Grinded sand	Water	Super-plasticizer KM-30 (0.5%)	Aluminum powder	Gypsum
		Cement	Sand-lime							
Air-silicate	1:2	-	486	425	787	300	359	2.43	0.72	3
	1:2	-	550	342	650	327	363	2.75	0.79	4
	2:1	-	540	259	480	340	367	2.8	0.84	4
Air-concrete	1:2	293	-	525	937	89	200	1.44	0.68	3
	1:2	281	-	394	703	133	194	1.41	0.63	4
	2:1	270	-	263	469	178	188	1.37	0.60	4

This paper considers technology of single-stage moulding of aerated-sandwich concrete articles. Articles consist of structural layer in form of aerated concrete that takes load bearing function and thermal-insulation layer made of aerated concrete. Such articles are produced by mixing heavy and light ingredients and moulding in forms. Heavy filler of mix after the vibration settles down and forms construction layer. During this process is squeezes the mortar out and forms the second – thermal-insulation layer. Use of plasticizing additives in this technology allows get cast concrete mix out of creep concrete mix that significantly facilitate the lamination process during vibration.

Research established lamination limits for concrete mix as well as impact of vibration at solidification on parameters of concrete. Use of plasticizing additives in concrete allows reduction of water consumption that changes the mix lamination limits. Besides, it causes increase of concrete strength.

By study of concrete mix lamination conditions we established compositions of aerated sandwich articles both on sand-lime and cement binders. During the experiments the amount of used water, vibration time and amount of plasticizing additive (MELMENT) were changed. Mix compositions are given in Table 1. As it can be seen from the table the use of super-plasticizer additive at 0.11-0.14% of binder mass provides lamination of mixture and formation of structural layer in 1-1.5 min during vibration. While for mixtures without additives this time is 1.5-2 min. At the same time water content in mix with additive is 20% lower and creep flow of concrete mix is significantly higher. Comparison of concrete mix strengths and mass parameters given in Table 1 shows that despite increase of strength values of concrete with additives (1.4-1,6 times for aerated layer and 1.12-1.15 times for solid structural layer), small increase of density is observed. It should be noted that reduction of moulding process duration time allows reduction of ware of equipment and significantly improves working conditions for operating personnel.

Final compositions of mixes per 1 m^3, amount of materials (in kg) that provide lamination of concrete mix with sand-lime and cement binders are given in Table 2. For ordinary concretes and concretes with sand-lime binders volumes of water and amounts of super-plasticizer and aluminum powder are varied that provides formation of thermal-insulator aerated concrete layer with pre-defined density. Average density of aerated concrete depends on yield of mixture and amount of used aerator component (aluminum powder). Creep of mix is checked by creep parameter (in cm) that depends on water/binder proportion in mixture. Optimal value of proportion varies in 0.4-0.6 range. Reduction of water/binder proportion facilitates increase of

aerated concrete strength. Use of super-plasticizers in aerated concrete mass increases yield 2.18 times or gives 20-25% reduction in water consumption. As it can be seen from data, the efficiency of super-plasticizers is higher for aerated concrete on cement binder than for air-silicate mix on sand-lime binder.

Use of super-plasticizers in aerated concrete mix at amount of 0.4-0.6% of binder mass is considered as optimal.

Besides, research was carried out under leadership of Prof. V. Loladze in order to study deformation properties of concrete layers of the aerated sandwich articles. For this purpose 10x10x40 cm concrete prisms were made with use of sand-lime binder and addition of super-plasticizer “MELMENT”. Studies were carried out on two directions:

- first direction – with separate moulding of structural and thermal-insulation layers after lamination of complete concrete mixture;
- second direction – with simultaneous moulding of sandwich samples with 1:1 proportion of structural and thermal-insulating layers.

Prepared samples after thermal-humid processing were kept in air-drying condition with 55-65% humidity of air and 20^0C temperature till acquiring 5-8% moist content. Average density of thermal-insulation layer was equal to 660 kg/m^3, while density of structural layer was 1670 kg/m^3. Compression strength of thermal-insulation layers of concrete samples was 4.5 MPa and compression strength of structural layer was 9.7 MPa.

For study of creeping deformation the spring-type power device was used. Head pressure limit and scale step was determined for each specific case. For both single-layer and sandwich samples pressure was applied at geometric center of cross section. Study of concrete creeping was carried out at the following relative values of compression: 0.3R for thermal-insulation layer, 0.5R for structural layer and 0.4R for sandwich sample (at 60±5% relative air humidity and $20^0 \pm 1^0$C temperature).

At creeping tests several samples from each set were wetted by air-gun during 30 min in order to imitate humid operation conditions like rain. Studies showed that deformations of separate layers of samples do not differ from known data on deformations of cellular and aerated concretes. Impact of wetting was insignificant for creeping curve of thermal-insulation layer and resulted in reduction of creeping deformation. After five days deformations started rising again and after twenty days their rise curve become almost parallel to non-wetted sample creeping deformation curve. Wetting of sample of structural layer caused more significant change of

creeping curve. In this case it seems that strong compression during wetting of porous part caused redistribution of internal tension from concrete mortar to heavy filler. Observed post-impact deformations after removing of pressure off samples show elasticity of both structural and thermal-insulation layers of aerated sandwich article at prolonged impact of compression loads.

Change of sandwich sample deformations in time is given on Figure showing that when applying loads to geometric centers of layers the irregular distribution of loads has significant impact on aerated sandwich article concrete layers. The over-load of thermal-insulation layer and redistribution of pressure from structural layer to thermal-insulation layer over time can be observed from Figure. This is indicated by deflections of concrete creepage deformation curves for structural layer that are characteristic for start of deformation stabilization at earlier period than for thermal-insulation period. It is possible that this change is facilitated by increase of physical eccentricity of longitudinal force impact that is caused by larger deformation of thermal-insulation layer.

Study showed also that wetting of sandwich article from one layer side affects both layer deformation changes. At that wetting effect is different for these layers, e.g. at wetting of thermal-insulation layer the reduction of deformation is observed while at wetting of structural layer deformation increases at first and then reduces that is caused by moisture relocation towards thermal-insulation layer.

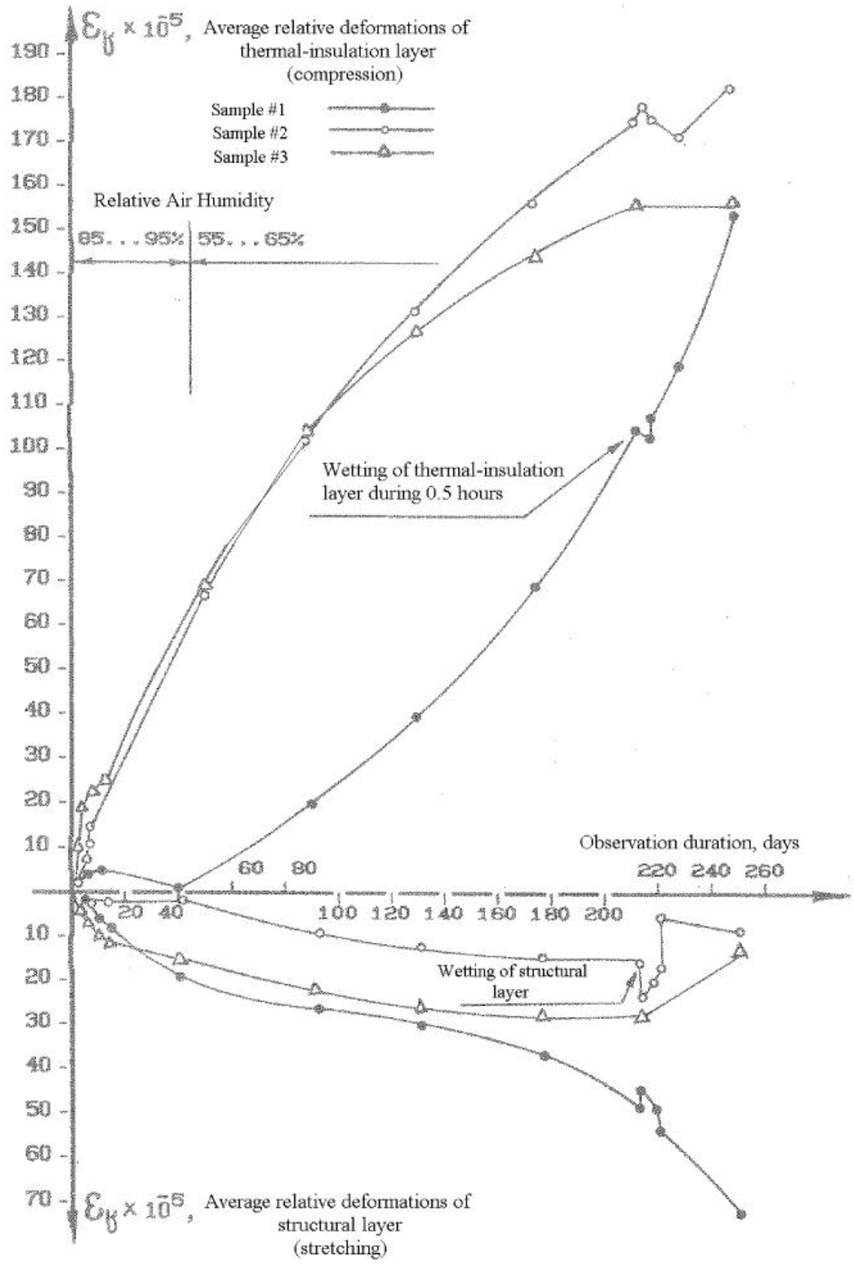

Fig. 1. Changes of creeping deformation for sandwich air-silicate samples at loading according to geometric center

Intensity of increase of creeping deformations of separate layers of sandwich samples may indicate on structural changes, e.g. formation of micro-fractures during stress redistribution at creeping, that causes non-stable deformation of layers. These studies indicate on necessity of centering of aerated sandwich article according to physical center. If we compare creeping deformations of single-layer samples made of thermal-insulation layer with creeping deformations of sandwich samples for the same period of observation, we can conclude that at

geometric centering of aerated sandwich sample compression value should be determined with account of parameters of thermal-insulation layer that are less strong compared to structural layer. This is confirmed by fact that relative compression 0.4R caused relatively intensive increase of creeping deformations of sandwich samples. This is confirmed also by the fact that on 220-th day deformation of thermal-insulation layer of sandwich samples is 1.8 times larger than deformation of thermal-insulation layer of single-layer samples (at relative compression 0.3R).

According to experimental results, at compression levels described above, the creep flow of aerated silicate concrete in moist environment is significantly lower than in dry environment.

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29-30 May 2014, Tbilisi, Georgia

MODELING ASPECTS OF WAVE GENERATION PROCESSES IN RESERVOIRS UNDER SEISMIC ACTION

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Abstract: *Waves in reservoirs arising during earthquakes due to the impact of seismotectonic displacement (STD) at its bottom can be of considerable period and amplitude. In order to avoid or reduce such seismic hazard as the dam protracted overtopping process caused by these seismogenic (tsunami-like) impulse waves, it is important to predict their main parameters in designing or operating hydro-projects/dams in mountain seismoactive zones to implement the emergency action plan (EAP) including a dam failure inundation mapping.*

Using the observation data the formulae for assessment the maximum amplitude of seismotectonic vertical displacement (upthrow) at the fault line and its length depending on the design magnitude or intensity of the HPP region are presented.

Numerical analysis based on the solutions of proper two-dimensional (2D) hydrodynamic problem obtained formerly by T. Gvelesiani, revealed a number of characteristics concerning the seismogenic wave's processes in reservoirs. Also the simplified relationship for short-term and practically exact prediction of the wave maximum amplitude (run-up height) at dam site is obtained.

Keywords: earthquake, reservoir, dam, seismotectonic displacement, seismogenic wave, wave maximum amplitude, wave profiles, emergency planning**,** magnitude.

INTRODUCTION

The primary objective of the problem under study is developing the refined techniques based on mathematical modeling for the explicit description of whole processes of high (extreme) wave's motion caused by earthquakes in mountain reservoirs including the reliable prediction procedures of environmental impact, exerted by that waves, in order to prevent or

eliminate potential hazards to adjacent regions (downstream) as well as to ensure adequate secure conditions for hydropower plants operation [8, 11, 13, 15].

The generation of large devastating waves in natural and artificial reservoirs of mountain regions may be caused mainly by the rapid landslide mass impact or seismotectonic deformations at the bottom of reservoirs [8, 11]. These waves, which may be called landslidegenic and seismogenic impulse waves, are similar to tsunami waves. However, in contrast to them, they possess a number of specific properties concerning their transformation, interference, etc. [8].

History knows many cases of generation of gigantic impulse waves in reservoirs, their overtopping of dam crest and transformation along up- and downstream resulting in tremendous fatalities and property damage. The catastrophe in the River Viont Valley (Italy, 1963) is but one example; a mass of rock about 250 mln.m3 moving at 25-30 m/s slid in the deep reservoir. The resultant overtopping wave of 70-90 m devastated the populated area of its downstream, causing heavy loss to both life and property [11]. Similarity, numerous instances of large impulse waves have been observed in many natural reservoirs (lakes, fiords, bays) all over the world (Alaska, Switzerland, Norway, Chile, USA, Japan, Peru, etc.) [3, 10, 11].

The reservoirs created by high dams have significantly changed the natural strain-stress state of their bank slopes, intensified the filtration, geological and seismotectonical processes, significantly increasing the probability of large landslide and earthquake phenomena [4, 11].

The above problem is of a particular interest for Georgia, the Caucasus region and for many other countries (such as Italy, Greece, Romania, Turkey, USA, Canada, China, India, The South America countries, etc.) which are characterized by high seismic and complicated geological conditions considering in addition, that a number of deep reservoirs and high dams are located there. Obviously, it is vital to predict the extreme waves environmental impact accurately enough to carry out emergency action plan including a dam failure inundation map, the proper engineering procedures and implement urgent measures to minimize or eliminate the potential hazards [13, 15].

On mathematical modelling of extreme impulse waves in reservoirs

In mathematical modeling of impulse wave processes in reservoirs, mainly the following two approximate theories are used:

- The small-amplitude wave (SAW) theory, where in a fluid is assumed to be ideal, incompressible, whereas motion to be potential (or non-rotational), governing equations and boundary conditions being linear;
- The shallow water (SW) theory, using the non-linear equations were in water depth is assumed small relative to a certain characteristic measurement (e.g. the wavelength or the radius of a free surface curve); the essential assumption, however, is that the component of water particle's acceleration along the vertical axis is negligible and therefore, the pressure yields the law of hydrostatics [2, 7, 11].

A large number of theoretical and experimental works were committed to the problem under study. Likely the first theoretical papers in which two-dimensional (2D) extreme waves were described (based on the SAW theory) have been published by Prof. T.Gvelesiani in the "Bulletin of the Georgian Academy of Science" (1969) and E. Noda (USA, 1970) [12]. Further investigations on mathematical modeling of these waves were developed in Georgian Institute of Power Engineering and Power Structure (GIPEPS) mainly based on the SAW theory [5, 6, 8, 11] and in other scientific centers of the USA, Russia, Norway, etc. using primarily the SW theory [3, 7, 9]. In this case, the shape of a reservoir can be partly taken into consideration; however, wave's 1D or 2D process simulation will be applicable in general, only for relatively shallow water basins. Accordingly, to SAW theory, the analytical solutions of 2D and 3D boundary wave problems have been obtained for the fluid domains (reservoirs) having essentially simple shape [8, 11].

As it was noted above the methods of wave investigation using the linear SAW theory possess in a number of cases a certain advantage in comparison with the nonlinear SW theory. In connection with it, we cite here the opinion of R. G. Dean and D. A. Dalrymple expressed in their book on advanced series on Ocean Engineering (World Science Publ. 1991): "It is surprising accuracy and easy for application that has maintained the popularity and the widespread usage of so-called linear or small amplitude wave theory. The advantages are that is easy to use, contrary to more complicated nonlinear theories, and lends itself to the superposition and other complicated manipulations. Moreover, linear wave theory is an effective stepping-stone to some nonlinear theories"

Here, it is noteworthy for add that the results of investigations developed in the GIPEPS (Manager of the works - Prof. T. Gvelesiani) based mainly on the SAW theory were employed in assessing of the ecological safety of the deep Sarez Lake (in Tajikistan) regarding the large

potential landslide hazard, as well as in design and construction under high seismicity condition of a number of high dams (among them are unique ones) in the different States and regions: Saiano-Shushenskaia (Power 6400 MW, Height 240 m) in Russia, Rogun (3600 MW, 335 m) in Tajikistan, Irganai and Miatli (in Dagestan), Getik (in Armenia), Jinvali (in Georgia), etc. Also, the research results have been included in a number of the State Standards and Building Codes (Norms) and Guidance's. The book [11] (the authors were the coworkers of the GIPEPS) have been published in Moscow by the famous publishing house "Atomenergoizdat" and it was likely the first book devoted to the problem of wave generation in reservoirs under earthquake effects.

Assessment of the seismotectonic deformation parameters in the reservoir region

For the analytical study of the rather complex problem of wave generation in reservoirs under seismic conditions requires, first of all, schematic representation of this phenomenon. This applies to both the water-filled area (the water basin) and the nature of boundary displacement (seismotectonic displacement) which may take place at the bottom of a reservoir. Such schematic representation is reasonable also since we do not know in advance the precise data relating to the geometrical and kinematical parameters (size and shape, velocity, duration and rate of change) of possible sesmotectonic residual deformation. In this case, the carrying out of computational tests using the proper analytical solution of a hydrodynamic problem for analysis of these parameter's variation effects on the final results will be an essential factor [8, 11].

The observation data show that the STD on the earth surface has the following main properties: the displacements are arose most probably along the existing seismogenic fault lines (which could be revealed by the seismological and geophysical investigations at the hydraulic project region) [1, 11]; the most intensive displacements (upthrow) occurring along the fault line reach their maximum amplitude near its central part (Fig.1). As we move away from the fault line the intensity of displacement decreases. Accordingly, in the different cases (Fig.1a,b) the possible shape of seismotectonic displacement along an existing seismic fault line may be approximated as a semi-sinusoid or rectangle (Fig.1a) or as two triangles with upthrow and downthrow side (Fig.1b, Fig.2).

Most commonly the possible shape of the STD in the plan can be approximated by an ellipse (Fig.2). Thus the area of deformation can be estimated as follows

$$S_s = \pi \frac{L}{2} b_s \qquad (1)$$

where b_s = length of semi-minor axis. L=seismogenic fault line length.

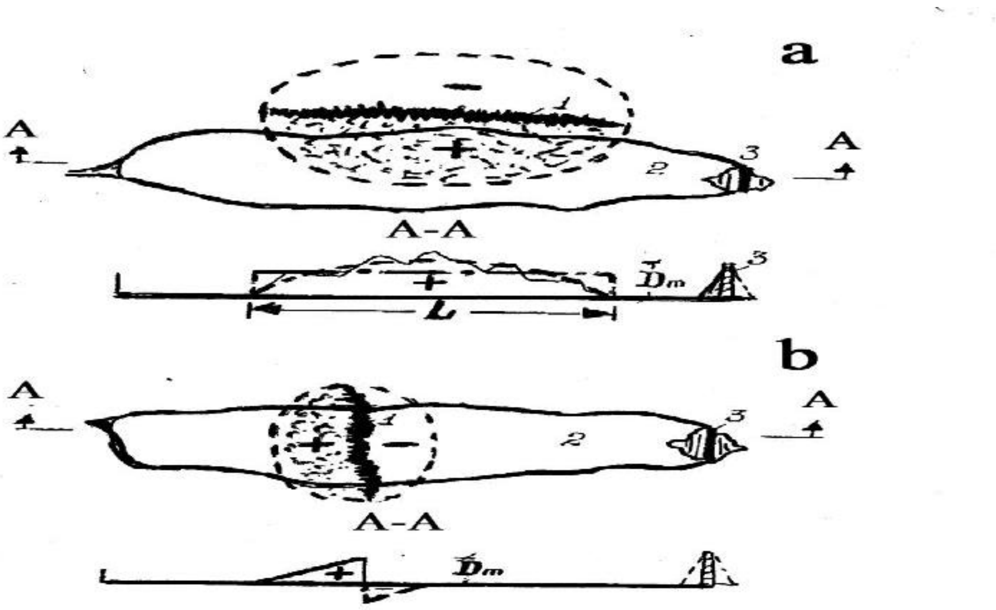

Fig.1 Seismotectonical deformation (STD) at reservoir area

1 - fault line, 2 - reservoir, 3 - dam

Plan Section

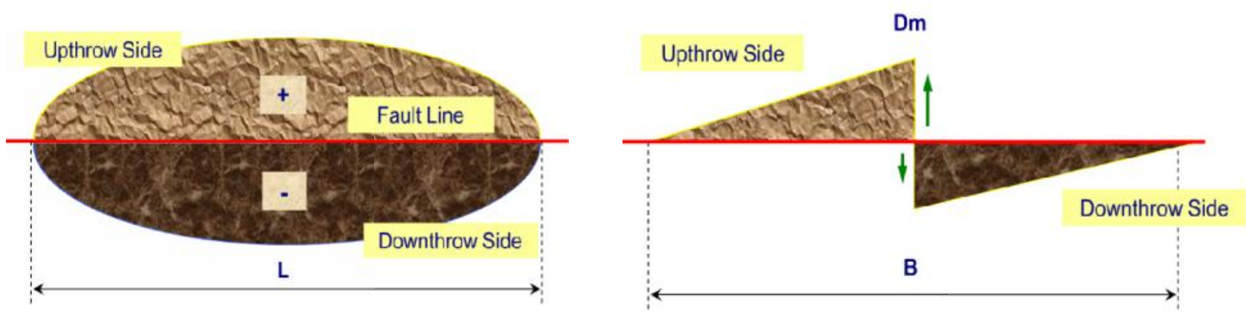

Fig.2 The STD possible shape approximated as an ellipse.

According to the recommendation given in [14], approximately $L/2b_s=2.5$ (when $M\leq7.1$). So the formula (1) in this case can be written as

$$t'_{s} = ® \frac{L^2}{10} \qquad (2)$$

Correct prediction of primary, seismotectonic deformation (STD) in a given seismic region is a difficult problem which still awaits solution [1]. At present, the prediction and definition of the deformation parameters is based primarily on monitoring and observation of geophisical data and their analysis. As a result of this analysis our recommendations (formulae) for estimation the main properties of seismotectonic (vertical and horizontal) displacement have been

represented in the Application to the State Constructional Norms and Rules (SNIP II-7-81), Moscow, Stroiizdat, 1982 (of the former USSR).

The formula to define the maximal value D_m, (m) of vertical displacement (upthrow) (Fig.2) at the fault line is expressed in the form of [8, 11]

$$\log D_m = 1.82\sqrt{M - 5.3} - 2.0 \tag{3}$$

or considering Shebalin's empirical formulas as

$$\log D_m = 1.82\sqrt{0.67 \cdot I + 2.33 \log H_E - 7.3} \tag{4}$$

where M and l = the magnitude and intensity (on the earth surface) of the earthquake, H_E= depth of the hypocenter or focus of earthquake (the relatively not deep earthquakes characterized usually by HE=10...15 km).

Moreover, we have obtained the formula for approximate forecasting of the fault line length, L, (km) as

$$\log L = 0.533M - 1.933 \qquad (when\ M > 6) \tag{5}$$

Based on both empirical data and the concept of "seismic moment" Academician I.V.Riznichenko has obtained relationship [14] similar to the formula (5) in the form of

$$\log L = 0.440M - 1.289 \tag{6}$$

Furthermore, for practical assessments, the following simplified formulas can be used

$$D_m = 2.35M - 14.1 \qquad (when\ 6.2 < M \leq 7.1) \tag{7}$$

$$\begin{aligned} L &= 40.0M - 220.0 \qquad (when\ 6.0 < M \leq 7.1) \\ L &= 142.4M - 957.1 \qquad (when\ M > 7.1) \end{aligned} \tag{8}$$

Some calculation results using formulas (3, 7) and (5, 6, 8) are presented in Tables 1 and 2.

Table 1. Estimation of value D_m

M	6,2	6,3	6,5	6,8	7,0	7,1
I	7,7	7,9	8,2	8,6	8,9	9,1
D_m (3)	0,53	0,66	1,00	1,69	2,36	2,77
D_m (7)	0,47	0,70	1,17	1,88	2,35	2,58

Table 2. Estimation of value L

M	L (5)	L (8)	L (6)
6,0	18,4	20,0	22,1
6,3	26,6	32,0	30,0
6,5	34,0	40,0	37,5
6,8	49,1	52,0	50,0
7,0	62,8	60,0	62,0
7,3	90,8	82,4	83,8

According to the solution of 2D problem [8, 11], the wave maximum run-up height (or water level rise) $\eta_{d,m}$ at the dam site (x=0) (Fig.3, Fig.4, Fig.5), can be expressed in a non-dimensional form as the function of the following parameters

$$\frac{\eta^*_{d,n}}{D^*_n} = \frac{1}{t^*_0} f\left(l^*, \frac{b}{l}, \frac{x_0}{l}, t^*_0\right), \qquad \left(x^* = \frac{x}{h} = 0\right) \tag{9}$$

where

$$\eta^*_{d,m} = \frac{\eta_{d,m}}{h}, \qquad l^* = \frac{l}{h}, \qquad t^* = t\sqrt{\frac{g}{h}}, \qquad t^*_0 = t_0\sqrt{\frac{g}{h}}, \qquad D^*_m = \frac{D_m}{h}$$

g = free fall acceleration; l = length and h=depth of the reservoir; t_0=duration of the seismotectonical displacement; $2b$= the extent of seismotectonic displacement (upthrow) at the reservoir bottom along x coordinate and x_0=corresponding coordinate of its center (Fig.3).

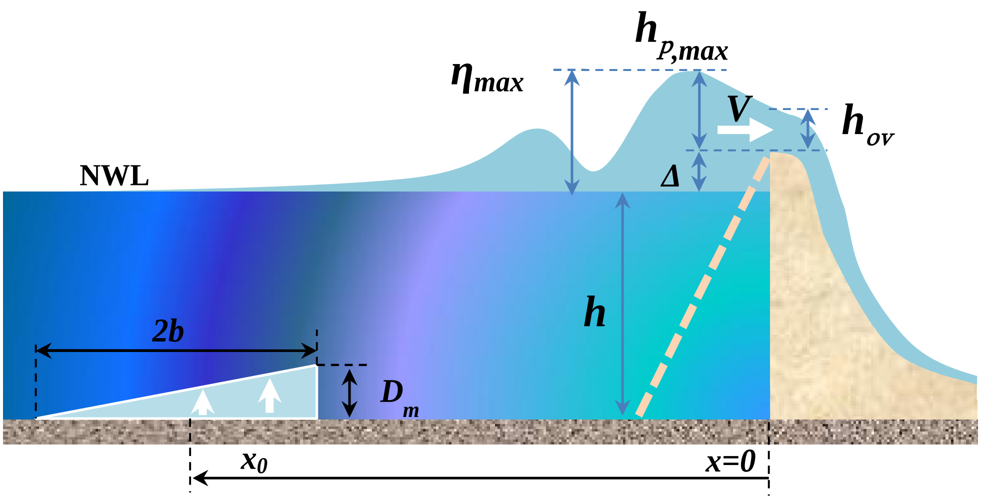

Fig.3 Design scheme of seismogenic (tsunami) wave generation due to the seismotectonical displacement at the bottom of reservoir

Some computed results using the above mentioned 2D wave problem solution are shown at figures presented below, thus Fig.4 presents seismogenic wave generation and propagation process in the reservoir (in non-dimensional form) and Fig.5 shows profiles of wave's run-up on the concrete and earth dams, when D_m=2.4m and M=7 (in dimensional form).

It's interesting to note that the maximum run-up height of seismogenic waves in cases considered (Fig.5) is equal to $\eta_{d,m}$=2.5m and 3.25m for concrete and embankment dams correspondingly. The results in case of embankment dam have been obtained on the basis of *our numerical solution* of the problem under study (program WNSL2D).

l^*=100; t^*_0=0.94; D^*_m=0.024; b/l=0.1; x_0/l=0.5; h=100m

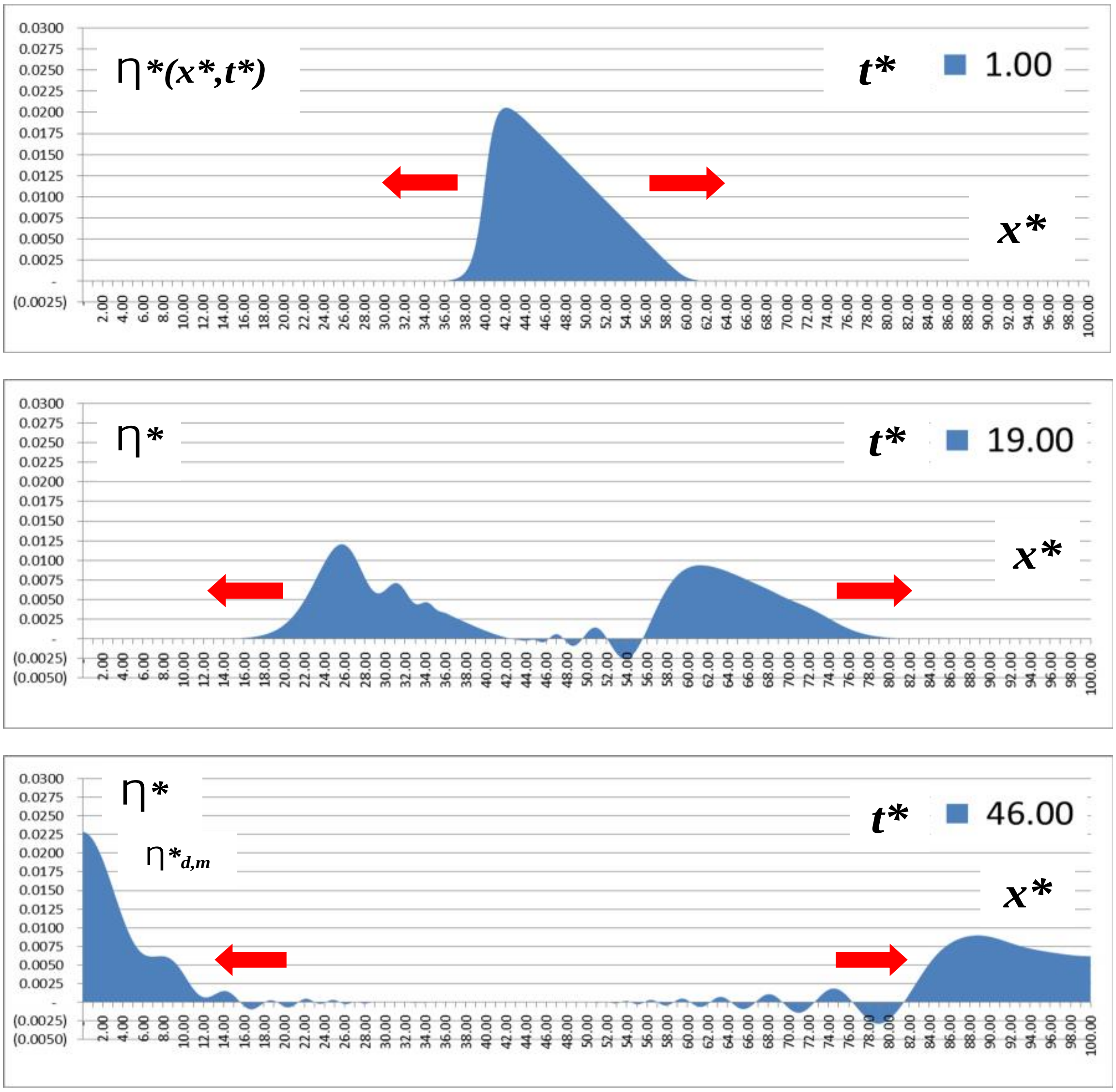

Fig.4 Seismogenic wave profiles for different time moments (including t*=46, for the maximum rise of the water level at dam site – x*=0).

It should be noted also that in case of the worst possible scenario the seismogenic (existing) fault line which is situated near the dam site (Fig.1b and Fig.2) can be activated during strong nearby earthquake, then the dam and the adjacent part of the bottom as the whole fault block may dip (settle). As a result, the height of the dam freeboard Δ (Fig.3) will decrease and therefore it leads to the greater hazard concerning the dam overtopping event and possible negative or catastrophic consequences at downstream.

l=10000m; h=100m; t_0=3s; D_m=2.4m; b/l=0.1; x_0/l=0.5

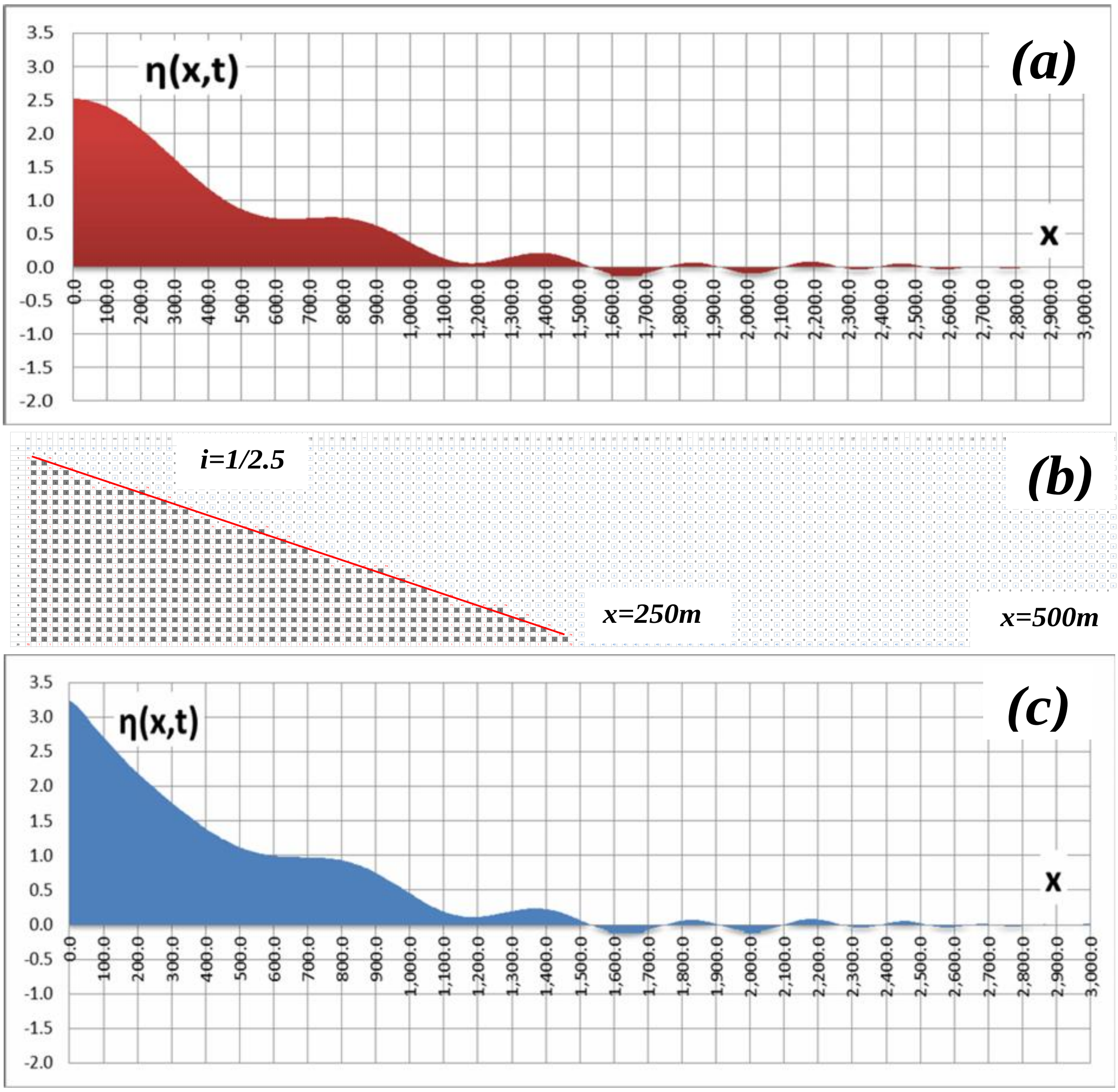

Fig.5 Seismogenic wave profiles at (a) concrete vertical pressure face and (c) and earth dam slope; (b) upstream slope of dam.

The analysis of the computation cycle results shows that the variation of parameters l^*, x_0/l and t_0^* (Fig.3) in practically interesting cases has comparatively insignificant effect on the wave amplitude - $\eta_{d,m}$. As a result of this analysis the following simplified formula for short-term and sufficiently exact prediction of the wave maximum run-up height on the concrete dam face was obtained:

$$\eta^*_{d,m} = D^*_m\left(0.97 + 0.54\frac{b}{l}\right), \text{ when } \frac{b}{l} \geq 0.1,\ l^* \geq 75,\ t^*_0 \leq 2.5 \qquad (10)$$

Conclusion

Extreme tsunami-type long-period waves in mountain reservoirs may be generated due to strong earthquakes (M>6.3...6.5). The dam protracted overtopping process by these seismogenic waves may cause the emergency situation at downstream: the sudden extreme flow in case of a concrete dam and the catastrophic wave as a result of an embankment dam failure (scouring). Numerical analysis based on the 2D hydrodynamic problem solution revealed the main properties of seismogenic waves processes in reservoirs and the proper parameters having an essential influence on the wave maximum amplitude (run-up height) at dam site, that allowed obtain the simplified formula for quick and practically exact prediction of this value. The method developed should be used in particular both for implementation of emergency planning issues in seismic design or operation of dams and to carry out of new proper guidelines and recommendations (to scale of international and regional application).

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29-30 May 2014, Tbilisi, Georgia

OSCILLATION PROPERTIES OF TSUNAMI TYPE WAVES DUE TO AN EARTHQUAKE IN RESERVOIRS

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Abstract: *The generation of long-period tsunami like impulse waves in a mountain reservoir of hydraulic works may be stipulated by a strong earthquake accompanying by seismotectonic (residual) deformations at the earth surface of the reservoir zone. The possible prolonged and repeated overtopping the embankment dam by these waves may cause the partial or complete scouring (failure) of the dam and catastrophic consequences at the downstream region.*

The analysis of the computed cycle results based on the proper 2D hydrodynamic boundary value problem solution for the reservoir represented schematically as the rectangle, allowed to predict the possible dam overtopping accident and to assess the parameters of the wave oscillation process at dam site such as: moments of the waves maximum peaks (run-up heights) occurrence at the dam face, time intervals between these peaks and the possible duration of the wave overtopping process. The new simplified formulas to assess these parameters are presented too.

Keywords: earthquake, reservoir, dam, seismotectonic displacement, seismogenic wave, wave oscillation period, wave profiles, dam overtopping, overtopping duration

Introduction

The analysis [1, 5] showed that the amplitude of waves generated in reservoir due to an earthquake or the ground seismic movement of vibration type (when a seismic wave is propagated along the bottom of reservoir) are in general, smaller than wind-generated waves.

On the other hand the generation of long-period tsunami like (seismogenic) waves in a mountain reservoir of hydraulic works may be stipulated by a strong earthquake accompanying by seismotectonic (residual) displacement (STD) at the earth surface of the reservoir zone. These

waves may be of considerable period and amplitude [3, 5]. Therefore the possible prolonged and repeated overtopping of the embankment dam by these waves may cause the partial or complete scouring (failure) of the dam and catastrophic consequences at the downstream region [4].

It should be noted that reservoirs with high dams, situated usually in mountain areas with steep slopes and shallow water in river mouth near the initial site of a reservoir (where the backwater curve approaches the normal depth), should not be regarded as channels of unlimited length [2]. At this site, both the increase of the height of the wave and its reflection (caused by the intensive decreasing a depth and width of the reservoir) are occurred. That leads to the water level oscillation with relatively long periods [4]. As a first approximation to take account of the reflection factor we shall consider the process of wave-formation in a reservoir of limited length and average depth [5]. So the reservoir can be represented as a rectangle (plane or 2D representation) or as a rectangular parallelepiped (3D representation); also it was assumed that the amplitude of residual displacement was negligible relatively to the reservoir height. Thus in the papers [2, 5] the author has solved (based on the small amplitude wave's theory) two and three dimensional (2D, 3D) problems on linear oscillation of an ideal incompressible fluid in a reservoir during seismic disturbance at its bottom as the vertical velocity of displacement representing an arbitrary function of time and coordinates.

Assessment of the parameters of water oscillation in reservoir under seismic action

According to the solution of the 2D problem of wave generation in reservoir due to the STD at its bottom [3, 5], the wave maximum run-up height (or water level rise) $\eta_{d,m}$ at the dam site (x=0) (Fig.1) can be expressed in a non-dimensional form as the function of the following parameters

$$\frac{\eta^*_{d,n}}{D^*_{n}} = \frac{1}{t^*_0} f\left(l^*, \frac{b}{l}, \frac{x_0}{l}, t^*_0\right) \tag{1}$$

where

$$\eta^*_{d,m} = \frac{\eta_{d,m}}{h}, \qquad l^* = \frac{l}{h}, \qquad t^* = t\sqrt{\frac{g}{h}}, \qquad t^*_0 = t_0\sqrt{\frac{g}{h}}, \qquad D^*_m = \frac{D_m}{h}$$

g=free fall acceleration, l=length and h=depth of the reservoir, t_0=duration of the seismotectonical displacement, D_m and $2b$=maximum amplitude and extent of seismotectonic displacement (upthrow) at the reservoir bottom along x coordinate, x_0=corresponding coordinate of its center (count from dam site).

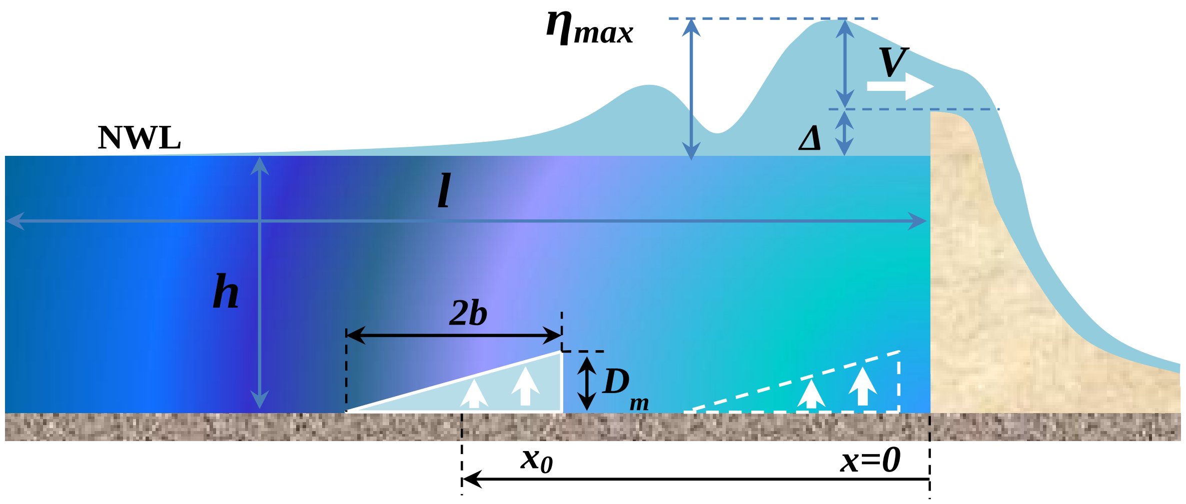

Fig.1 Design scheme for study the seismogenic wave oscillation in reservoir due to STD at its bottom

For the estimation of the possibility and character of a dam overtopping event by tsunami type waves when the location and sizes of the STD approximately are known in advance, we have to answer in particular, on the following basic questions:

a) what will be the maximum amplitude of waves at the water surface in a reservoir and in particular, at the dam site?

b) how often the maximum water level rises (peaks) will be occurred at the dam during the water oscillation in a reservoir?

c) what will be the duration of the maximum level rises at a dam site?

Below we'll touch upon shortly the aspects b) and c), as for the issue a) it have been considered in [3].

We will consider the case (Fig.1) when the earth surface rise (upthrow) occurs on one side of the fault (line) ignoring the deformations of the downthrown [3]. Obviously, it will be the extreme case with the most possible dam overtopping accidence with dangerous consequences at downstream.

It is known that the fundamental eigen (one node) period of water free oscillation (seiche) in a basin corresponding to the essential form (harmonic) is determined by Merian's formula [2]

$$T_m = \frac{2l}{\sqrt{gh}} = \frac{2l}{c} \tag{2}$$

where $C = \sqrt{gh}$ = velocity of a long wave motion.

The second form of a water basin eigen oscillation is equal to

$$T_1 = \frac{T_m}{2} = \frac{l}{C} \tag{3}$$

Let us consider two limiting cases of the wave generation in reservoir as a result of the STD.

The case a: if the STD takes place close proximity to the dam site of reservoir (Fig.1), than the wave generation process may be described in the following manner:

The first maximal wave (let us denote it as W1) arising near the dam site (x=0) interacts with the dam; then after reflection from it, the wave moves toward the left board of reservoir (x=l) and comes up to this site approximately within the time period

$$2\Delta t = \frac{l}{C}$$

The time interval of wave propagation in the opposite direction (till it reaches again the dam site) is estimated evidently also as l/C=2Δt.

Thus the period of water level oscillation at the dam site is amount to

$$4\Delta t = \frac{2l}{C} = \frac{2l}{\sqrt{gh}} = T_m$$

Therefore we obtain Merian's formulas and so the water level rising at the dam site will be occurred periodically with the interval, T_m.

The case b: the STD is happened in the middle part of the reservoir (x_0/l=0.5) (Fig.1)

At first the maximal wave which is generated at the water surface in reservoir is divided gradually in two waves, W1 and W2. The wave W1 begins to move towards the dam and another wave W2 - in the opposite direction - to the board [3]. In this case the value $\Delta t = l/2C$ is the interval for the wave W1 to reach the dam. Similarly for the wave W2 the duration of its propagation as far as the board is Δt. It is required the time period 2Δt for the wave W2 to reach the dam (after its reflation from the board). So the wave - W2 will be come up to the dam within 3Δt (after the STD start) or after the time interval 3Δt-Δt=2Δt (after the initial water level rise at the dam site).

Therefore the time interval between the first and the second wave peaks (wave's maximum rises) which are occurred at the dam is determined as

$$2\Delta t = \frac{T_m}{2} = \frac{l}{\sqrt{gh}} = T_1$$

Now the water level oscillation takes place with the frequency which is two timing more than in the case a.

Thus the *period of water oscillations* in reservoirs caused by the STD are depended on the parameter x_0/l. In particular, that period is approximately equal to T_M for the case when

$$\frac{x_0 - b}{l} = 0 \ or \ \frac{x_0}{l} = \frac{b}{l} \ \text{(STD occurs near thi site } x = 0\text{)}$$

When x_0/l=0.5 the period of oscillation is $T_1=T_m/2$.

Such character of the water oscillation may be seen by the graphs presented at Figure 2.

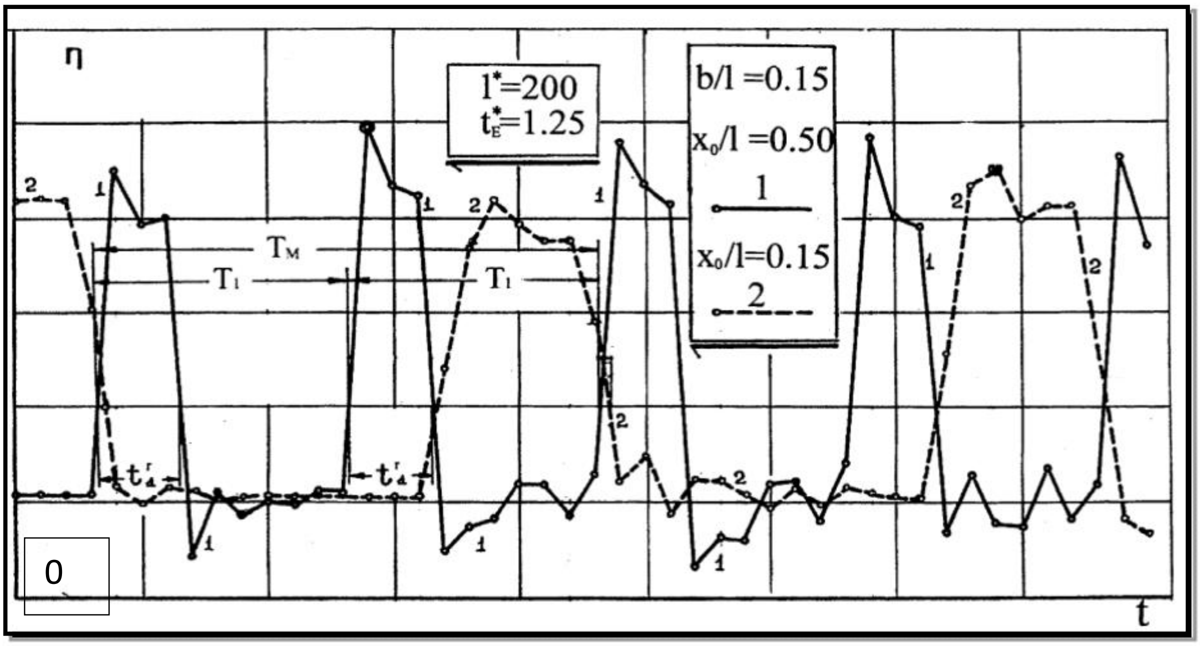

Fig. 2. Relationship between wave amplitude and time at dam site (x=0) for different x_0/1 values

The moments of the wave maximal peaks t_j^d (j=1,2,3...) *which are occurred at dam* may be determined by means of the following expressions obtained by T.Gvelesiani [2]:

- for the wave peaks with uneven numbering (j=1,3...)

$$t_j^d = \frac{(1-\rho_x)l}{c} + \left(\frac{j-1}{2}\right)\frac{2l}{c} \ or \ t_j^d = 2(1-\rho_x)\Delta t + 2(j-1)\Delta t \qquad (4)$$

- for the wave peaks with even numbering (j=2,4...)

$$t_j^d = \frac{(1-\rho_x)l}{C} + 2\frac{\rho_x l}{C} + \left(\frac{j}{2} - 1\right)4\Delta t$$

or (5)

$$t_j^d = 2(1-\rho_x)\Delta t + 4\rho_x\Delta t + 2(j-2)\Delta t$$

where

$$\rho_x = 1 - \frac{x_0}{l}; \;\; \Delta t = \frac{T_m}{4} = \frac{T_1}{2} = \frac{l}{2C}$$

The *time intervals ΔT^d between the occurrences of wave peaks* at the dam are determined as follows:

- between the first and the second peaks

$$\Delta T_{1-2}^d = 2\rho_x \frac{l}{C} = 4\rho_x \Delta t \tag{6}$$

- between the second and the third peaks

$$\Delta T_{2-3}^d = \frac{2l}{C} - 2\rho_x \frac{l}{C} = 4(1-\rho_x)\Delta t \tag{7}$$

For the other intervals we have

$$\Delta T_{3-4}^d = \Delta T_{1-2}^d, \qquad \Delta T_{4-5}^d = \Delta T_{2-3}^d, \qquad \Delta T_{5-6}^d = \Delta T_{1-2}^d$$

EXAMPLE: Determine the time moments corresponding to the occurrence of the wave first three peaks t_j^d and the intervals between these peaks ΔT^d. The location of the STD at the bottom of a reservoir is characterized as $\frac{x_0}{l} = 0.75$, $\rho_x = 0.25$

According to (4) and (5) we obtain

$$t_1^d = \Delta t + \frac{\Delta t}{2}; \;\; t_2^d = \frac{\Delta t}{2} + 2\Delta t; \;\; t_3^d = \frac{\Delta t}{2} + 5\Delta t$$

Using (6) and (7) the time intervals between the peaks we have

$$\Delta T_{1-2}^d = 4 \cdot 0.25\Delta t = \Delta t; \;\; \Delta T_{2-3}^d = 4 \cdot 0.75\Delta t = 3\Delta t$$

Thus in this case the interval between the second and the third peaks is three times more than between the first and the second ones.

As for the *duration of the wave possible dam overtopping process*, it may be expressed as the following function [2]

$$T_{ov} = f(\eta_{d,m}, t_p, \Delta) \tag{8}$$

where $\eta_{d,m}$= maximum wave amplitude (maximum water level rise) at dam (x=0), Δ = freeboard or height between the design (normal or maximum) water level and the elevation of the dam crest, t_p = average duration of the water level rise (above the dam crest) or "thickness of wave peak" at the Δ level (Fig.3, Fig.4).

In the case when the STD is located at the middle part of reservoir (Fig.3a) the lengths of wave profiles x_p between the dam site and the cross points (shown at Fig.4b, when $\Delta^* = \Delta/h = 0.006$) can be assessed as follows

$$x_p^* = \frac{x_p}{h} = 4 + 74.3\left(\frac{b}{l} - 0.05\right), when\ 0.05 \le \frac{b}{l} \le 0.4 \qquad (9)$$

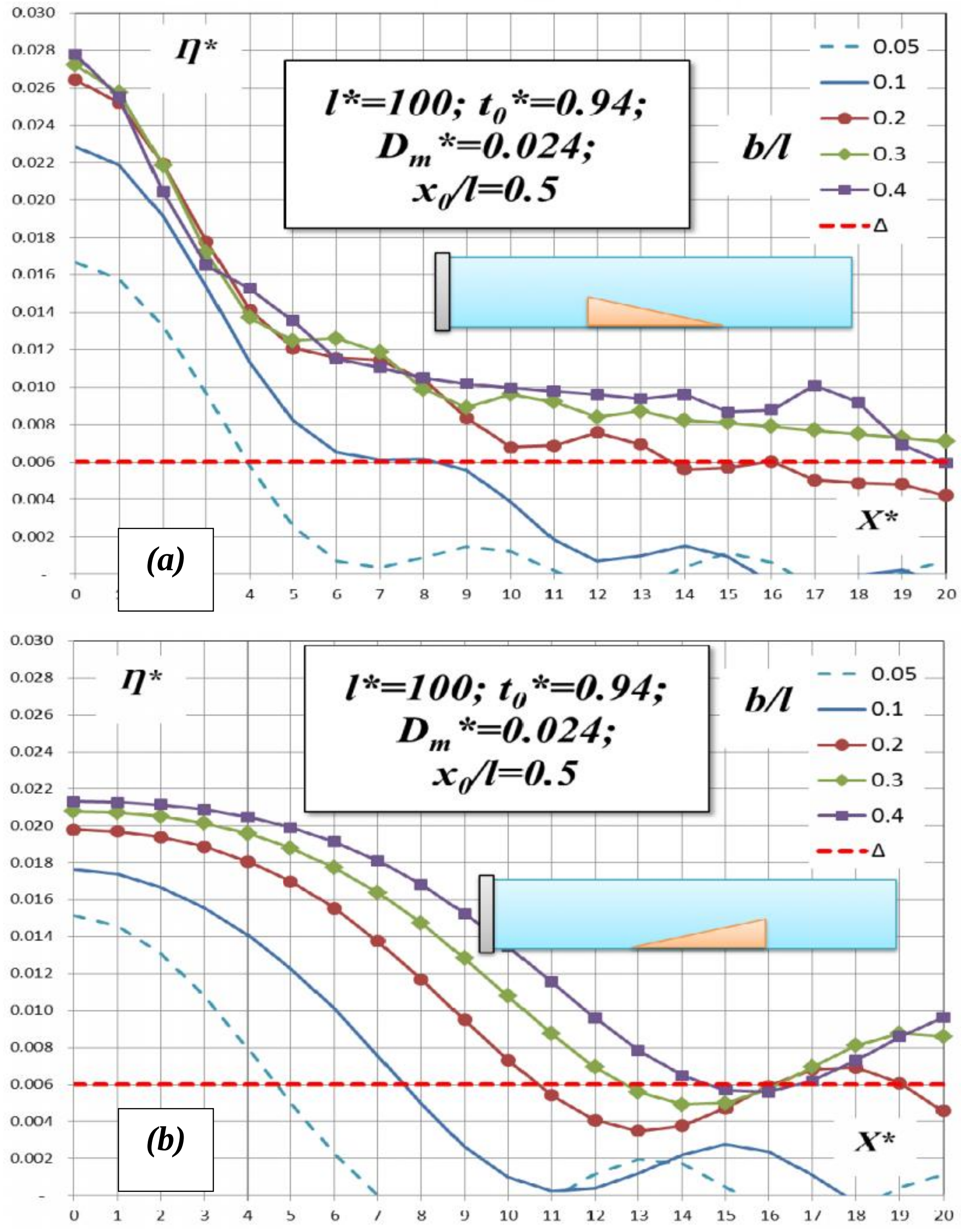

Fig.3 Profiles of tsunami waves corresponding to the wave maximum run-up height at the dam face caused by STD located at the middle part of reservoir for (a) and (b) cases.

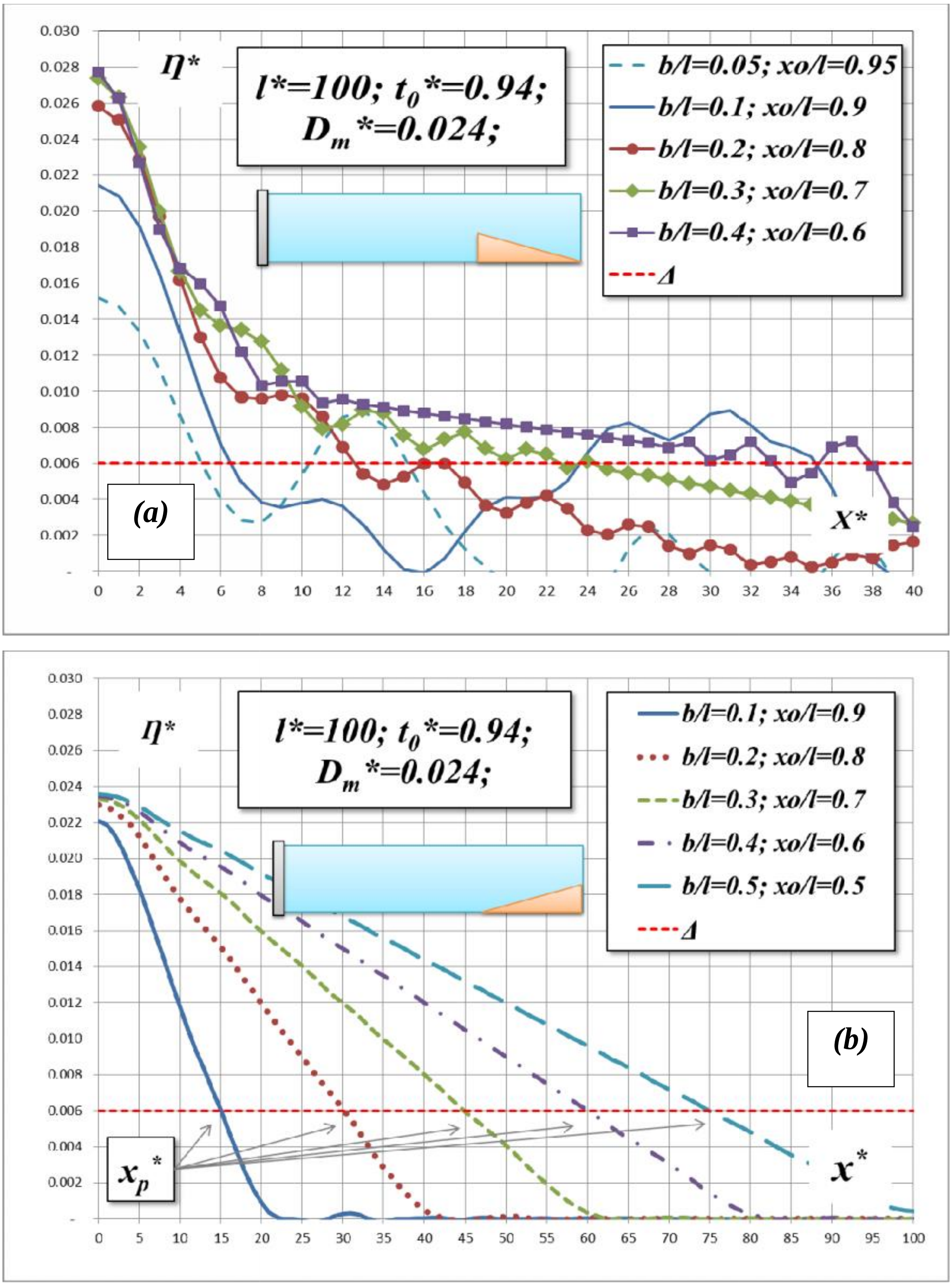

Fig.4 **Profiles of tsunami waves corresponding to the wave maximum run-up height at the dam face, caused by STD located at the board (opposite to dam) of reservoir for (a) and (b) cases.**

The duration of the water level rise t_p (corresponding to the value *Δ*) is approximately equal to the duration of the wave dam overtopping process T_{ov}

$$T_{ov} \approx t_p = \frac{2x_p}{C} = \frac{2x_p}{\sqrt{gh}} \quad (10)$$

In non-dimension form we have

$$T_{ov}^* \approx t_p^* = 2x_p^* \quad (11)$$

Where

$$T_{ov}^* = T_{ov}\sqrt{\frac{g}{h}}; \;\; t_p^* = t_p\sqrt{\frac{g}{h}} \quad (12)$$

Considering (9), (10) and (12) the duration of wave overtopping process is assessed by the following formula

$$T_{ov} \approx t_p = \sqrt{\frac{h}{g}}\left[8 + 148.6\left(\frac{b}{l} - 0.05\right)\right], when\ 0.05 \leq \frac{b}{l} \leq 0.4 \quad (13)$$

Analogously *in the case when the STD is located at the reservoir board (Fig.4,b)* the duration of dam overtopping is determined as follows

$$T_{ov} \approx t_p = \sqrt{\frac{h}{g}}\left[30 + 300\left(\frac{b}{l} - 0.1\right)\right], when\ 0.1 \leq \frac{b}{l} \leq 0.5 \quad (14)$$

It should be noted that formulas (13) and (14) can be used for the following criteria

$$t_0^* \leq 2 \text{ (or } h \geq 25m, t_0 \leq 3s) \quad \text{and} \quad l^* \geq 75 \quad (15)$$

The duration values of the dam overtopping process caused by seismogenic waves in case of deep reservoirs (h=75, 100 and 130m) assessed by formula (14) are presented in Table 1.

Table 1

b / l	0.1	0..2	0.3	04	0.5
T_{ov}, min h=75m	1.4	2.8	4.2	5.5	6.9
T_{ov}, min h=100m	1.6	3.2	4.8	6.4	8.0
T_{ov}, min h=130m	1.8	3.6	5.5	7.3	9.1

It should be noted that the specific overtopping water volume is less than 8-10% in comparison with the whole volume of the wave at dam site, which is assessed in particular as $w_{wave} \approx 0.5$ ¦ , $(2x_p)$ (in the case shown at Fig 4b) This factor explains why the maximum wave has ability to make appearance at dam and overtop it several times after reflections from the initial site of reservoir ("board"). Such 4-fold the concrete dam overtopping event and long-period water oscillations (seiches) have been observed at the Hebgen Lake (USA, Montana) during the earthquake of August 17, 1959 (M=7.1) (Steinbrugge K. V and Cloud W. K "Bull. Seism. Soc. Am" Vol. 52, 1962) [5].

Conclusion

The analytical solution of the proper 2D hydrodynamic (wave) problem have been obtained formerly by T.Gvelesiani using the potential motion waves theory (for the reservoir represented schematically as the rectangle) is used to describe mathematically the seismogenic waves oscillation character in the reservoir during an strong earthquake. The analysis of the computed results showed in particular, that the frequency and duration of the water level maximum rise at dam site are mainly depended on such element as: the extent of seismotectonic displacement zone at the reservoir bottom (along the longitudinal coordinate) and the distance between this zone and the dam site. Also the proper simplified relations to assess the abovementioned wave oscillation parameters are obtained.

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29-30 May 2014, Tbilisi, Georgia

ESTIMATION METRO INFLUENCE ON STRUCTURES ADJACENT BUILDINGS

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Abstract: The article is considered the impact of the dynamic vibration load of Metro on bearing capacity structures of nearby buildings. The goal of theoretical research is to establish the effect of the dynamic loads of Metro on the stressed state of bearing structures of buildings.

Key words: computer modeling, life cycle, vibration loads, structural engineering, information technology, strain

Urgency of the subject.

Often, due to lack of available space in the big cities the construction of residential and public buildings makes near the subway lines. The growth of all types of traffic flows, increasing of the speed and intensity of traffic make the necessity for obtaining the quality and quantity of estimates of the influence of transport vibration. Both in domestic and foreign literature appear periodically reports about negative effects of transport vibration, but it is usually not taken into account either in new construction or renovations in existing buildings and structures. The fact that the transport vibration does not lead to emergency situations and explains the practical absence of regulations governing its intensity in numerical estimates of the criteria of durability and reliability of objects. The questions of ensure the reliability of structures related to transport vibration, may soon become relevant, taking into account the total physical deterioration of existing buildings, especially monuments that will not demolished with the modernization of the historical center.

Direct application of traditional theoretical methods of solution of dynamic problems and methods of classical structural mechanics do not provide sustainable solutions that are suitable for practical use.

Numerical modeling of the impact on the complex building structures of random waves of a different nature is the actual problem for reliability and safety of construction projects during the

operational phase. Vibration created by subway train is the most intense and tangible for human. And the vibration is usually aggravated by the lower to the upper floors.

Area of research.

Foreign scholars pay great attention to the problem of construction nearby subway. Konstantinos Vogiatzis works [3] are dedicated to this problem. Scientists which examine the problems of subway influence, mainly investigate dynamic loads influence on soil and underground structures. In number of works of researchers C.O. Aksoy, T. Onargan [5], Mete Kun (Turkey) [7], Jiangfeng Liu, S. Imanzadeh (France), Taiyue Qi, Zhanrui Wu (China) [6], Mohammad S. Pakbaz, K.H. Bagherinia (Iran) [8] are proposed computer modeling variants of enclosed structures and underground installations for subway construction in different program complexes. Also different measures are proposed for decreasing destructive influence on soil at tunnels construction. All these works are dedicated to investigation of stress-strain state of underground structures and soil at subway construction. In Aijun YAO, Xuejia YANG, Lei DONG (China) [9] works is considered the variant of foundation protection from pernicious influence of subway lines, in particular from vibrations.

The scientists‘ minority pay attention to stress-strain state of bearing structures and foundations of ready-built buildings nearby subway lines. In Paul Simon Dimmock (Australia), Robert James Mair (UK) article [4] is carried out such investigation and performed the bearing structures analysis for low-rise buildings

Currently in traditional dynamics of building structures adopted the concept the natural oscillation frequencies of of building elements for matching lead to resonance phenomena. I would like to underline that this is not the resonance phenomena, and not even an error in the design and construction – just the building is designed based on the classical principles of structural mechanics, as the result, system can be at the critical stability.

One of the main tasks of the solution of this problem is the task of developing methods for the numerical simulation of the effects of vibration on buildings and structures that are close to the subway. In connection with the above, the task of assessing the safety of buildings, located near the traffic flow, it seems that an urgent task for the economy.

The Solution.

Reinforced concrete monolithic structure are more resistant to vibration under dynamic effects. Compared with buildings made of prefabricated concrete elements, they reduce vibration level of overlaps by 5-8 dB [2]. Such abatement is specified by peculiarities of dynamic work of

monolithic structures, which doesn't sustain resonance, but "softer" resonant phenomena. The most appropriate scheme of the building in this case is a column framework whose efficiency increases with increasing of thickness of the slabs and the reducing of column section. In the capacity of foundation is always recommended to use a solid monolithic concrete slab, which smooths the effect of inhomogeneities of subgrade and promotes the distribution of oscillations along the basement area and therefore, their reduction. Civic monolithic buildings can be located even in the immediate vicinity of the subway tunnels.

Creation of computer models is appropriate of big sense, that adequately describe the operation support systems of buildings under dynamic loads influence caused by subway.

The methods used.

Abstract computer model of the building is designed to study the behavior of the load-bearing elements of the building. It is located in 10 meters from the subway tunnel with shallow foundation slab and soil model, which simulated the movement of subway. Computer modeling was performed with the help of of LIRA SAPR software system. Numerical simulation example in this article allows multiple times and in a wide range of variable input parameters and conditions for the functioning of a complex system «aboveground part of the building - ground - soil - subway», replacing, thereby, experimental investigations of computational experiment. Such realization leads to a time-saving solution of a number of similar problems, and and allows draw corresponding conclusions on the stress-strained state of load-bearing elements, exposed to permanent effects of dynamic loads. By the author of this article conducted research of frame-monolithic building. The problem was solved in two-dimensional statement. In computation columns modeled by plates, girders modeled by rods with rigid joints, overlaps-rods with reduced stiffness, the soil was modeled by plates of variable stiffness.

System DYNAMICS + [1] is used in the program complex LIRA-SAPR considering the peculiarities of behavior of load-bearing elements under the influence of dynamic loads. The algorithm is based on dynamic loads put an iterative process, alternating phase set of static load on the structure, later setting the weights of the masses, later the task of dynamic loading on the structure. For modeling of dynamic loads of subway motion used sinusoidal load.

Given in the form $\boldsymbol{A\,sin(\omega \cdot t + \varphi)}$, (1)

where A – amplitude, ω – frequency, φ – phase shifting, set the beginning and finishing of load action;

In the corresponding entry field – amplitude P impact forces, frequency of influence in radians, phase shifting in degrees, as well as the beginning and finishing of exposure in seconds. Reflection of diagram z(i)=P*sin(ti) performed with the command of viewing.

To solve the problem of dynamic analysis of structures using two main methods:

- direct integration of the equations of motion;
- decomposition by own forms.

Method of decomposition by own forms can be applied only in the linear calculation, since the principle of superposition is not valid in the nonlinear theory. Direct integration methods are general and can be applied to solve all the problems of the dynamic analysis of structures.

In the dynamics in time used the direct integration of the equations of motion. The term "direct" means that the integration is not performed before any manipulation of equations [1].

The calculation is based on the dynamic impact on the solution of differential equations

$$M\ddot{\vec{u}}(t)+C\dot{\vec{u}}(t)+K\vec{u}(t)=\vec{q}(t), \tag{2}$$

where M, C, K – correspondingly matrixes of mass, damping and rigidity of system,

$\vec{u}(t), \dot{\vec{u}}(t), \ddot{\vec{u}}(t)$ – vectors of nodal displacements, velocity acceleration in time moment t,

$\vec{q}(t)$ – loading, correspond to time moment t.

It is considered that the initial velocities are zero $\dot{\vec{u}}(0)=0$, and the initial movements are derived from decisions of the first loading $\vec{u}(0)=\vec{u}_1$.

From the system (1) of ordinary differential equations with constant coefficients follows, that approximate the velocity, acceleration and displacement can be any finite-difference expressions of the displacement. For accelerations at the time moment t, using of central difference method, we can write:

$$\ddot{\vec{u}}(t)=\frac{\vec{u}(t+\Delta t)-2\vec{u}(t)+\vec{u}(t-\Delta t)}{\Delta t^2} \tag{3}$$

Mistake in calculation by the formula (2) is in order Δt^2, and to calculate the velocity and displacement with errors of the same order necessary to use next expression

$$\dot{\vec{u}}(t)=\frac{\vec{u}(t+\Delta t)-\vec{u}(t-\Delta t)}{2\Delta t}, \tag{4}$$

$$\vec{u}(t)=\frac{\vec{u}(t+\Delta t)+\vec{u}(t-\Delta t)}{2} \tag{5}$$

Substituting the expressions (2), (3) and (4) to expression (1) and determining the vector $(\bar{u}(t+\Delta t)+\bar{u}(t-\Delta t))$, receive the following system of equations:

$$\left[\frac{2M}{\Delta t^2}+\frac{C}{\Delta t}+K\right](\bar{u}(t+\Delta t)+\bar{u}(t-\Delta t))=2\left(q(t)+\frac{2M}{\Delta t^2}\bar{u}(t)+\frac{C}{\Delta t}\bar{u}(t-\Delta t)\right) \quad (6)$$

“New” displacements $\bar{u}(t+\Delta t)$ are determined by the are determined by the previously found displacements $\bar{u}(t)$ and $\bar{u}(t-\Delta t)$ by solving the system of equations (5). Such integration scheme called the implicit integration schemes. Such integration scheme is called the modified method of central differences. Expression (5) is the source for the decision of both linear and nonlinear problems of by direct dynamic calculation in software package LIRA-SAPR.

Solving the problem (1) consistent mass matrix is used, based on the same approximate the functions that was based the rigidity matrix. In this approach takes into account the rotational inertia - "torsion" elements of the mass are appeared.

In integrating the equations of motion in the PROCESSOR window displayed the kinetic energy diagram ($\frac{1}{2}\dot{u}_t^T M\dot{u}_t$), which allows you to estimate the nature of the integral transmission process. If further integration is not interest, it is possible to interrupt the process of integration, and browse the results to the point of interruption.

Vibration influence of subway on the stress-strain state of bearing structures of a buildings.

There are a large number of publications, authors of which have different approaches to the calculation of buildings and structures. However, not every method can reflect the real work of the support system of the building. Most of these methods involves the definition of the stress-strain state of load-bearing elements, based on the final design scheme of the building, fully loaded. Part of methods are focused on the characterization only separate parts of the building, which also leads to a distortion of the real work of the building. Therefore requires their further development and refinement. The purpose of the theoretical and experimental researches of many scientists now is to determine the contribution of the vibrations on stress-strain state elements supporting systems of buildings.

Neither in Ukraine nor in the CIS or in Euronorm no existing rules regulating allowable vibration levels for buildings and structures, caused by traffic. Vibration from the subway is evaluated by sanitary and hygienic requirements. You can see СП 23-105-2004 «Evaluation of vibration in the design, construction and operation of underground». But this document

describes some methods of assessment vibration impact according to the sanitary norms of vibration impact on people at night and day time. However, in the normative documentation not specified methods for evaluating the impact of a permanent dynamic loads on the bearing capacity of building structures, and the possibility of crack formation. In this case, a very important factor is the possibility of resonance vibrations in the building caused by external dynamic vibration of subway.

Investigation of own frequencies of building structures has great practical value in solving various dynamics problems, analysis of vibration propagation pathway of buildings is a complex engineering task, that can not be solved only by means of experimental or theoretical methods.

Depending on the design of solutions, efficiency and safety, as well as other conditions apply different methods of numerical modeling of vibration loads effect on multistory buildings. In all cases, modeling process is contained that the building is presented as system «aboveground part – ground – soil». At modeling must taking into account the properties of the soil, type of foundation, the type of structural scheme (arrangement of columns and stiffening diaphragm, stiffness of structural elements, the thickness of slabs, of reinforcement methods, the type and grade of concrete of the main load-bearing structures).

Soil condition has the greatest influence on the bearing capacity of the building under the influence of constant dynamic load. Under constantly acting vibrations the soil may settle, change its strength characteristics, it may be lowering of groundwater level, and in combination with constant vibration loads, such a state can lead to cracking in structural elements.

Also of great importance has age of the building, located in zone of vibration influence. For example, the impact of vibration on the newly constructed building can be completely negligible on the strength characteristics of structural elements. In this case if vibration is within sanitary norms, therefore vibration on structure of the building has almost no effect.

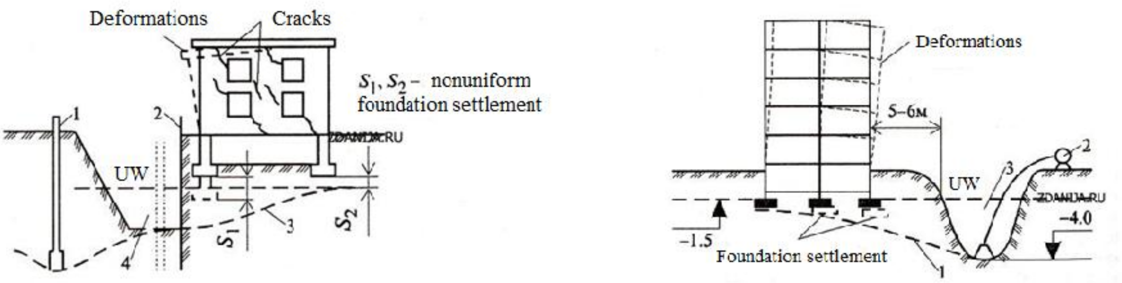
If the building has more than thirty years of service, may develop deformities due to various reasons, such as the natural aging of the material, soil settlement, fatigue damage, etc. (fig.1, a, b). In these cases, the vibration loads of shallow subway can play a decisive role under aggravation of building deformation, even lead to a breach of its reliability and safety.

In fig. 2, 3 an example of a computer modeling of dynamic effects of subway on the building under construction.

Abstract computer model of the building is designed to study the behavior of the load-bearing elements of the building. It is located in 10 meters from the subway tunnel with shallow

foundation slab and soil model, which simulated the movement of subway. Because the length of subway train is 140 meters that exceeds the length of the ordinary dwelling house or the length of the thermal block, numerical experiments consider only the two-dimensional problem.

During the motion of subway trains, there are several sources of vibrations. There are the engine work , work of compressor, brake system of the train [3]. Oscillations with a frequency of 35-50 Hz caused by vertical oscillations of unsprung mass of trains. Wheel pair can be viewed as a system with one degree of freedom, elasticity is the elasticity of the rail base. The own frequency of the system - 40 Hz. Oscillations with a frequency of 50-60 Hz appear due to the action of horizontal vibration. Movement of subway trains causes vibrations of building constructions with a frequency of 35-60 Hz and an amplitude of a micron to 1-3 mkm. Horizontal vibrations are predominant. Vertical oscillations have the same frequency content of two or three times smaller amplitude. The largest amplitudes of horizontal vibrations are observed in the level of the basement floor of the building. Here the amplitude of the walls of 2-2.5 times more than the amplitude of the first floor of staircase. Above the first floor vibration amplitude can be changed in the direction of decreasing, and in the direction of increasing.



a) b)

Fig.1, a, b. Deformation of the building under changes in the properties of soil

The primary cause of the oscillations is the contact interaction of the rolling stock wheels and rails. The main cause of the vibration excitation wheel-rail system is the presence of joints the way that leads to jump in at the turn of the wheel and uneven load in the transition from one wheel to the next rail link.

Computer modeling performed with application of the program complex LIRA-SAPR by method of integrating the dynamic loads. Presented in this work the numerical experiment repeatedly and in a wide range of variable input parameters and operating conditions of a complex system «aboveground part of the building - ground - soil - subway», replacing, thereby,

experimental investigations of computational experiment. In the software package calculation was performed using the subsystem DYNAMICS +. Was set vertical dynamic load along the Z axis with the amplitude of oscillations ω=35 radian, corresponding f=50 Hz, amount of taken into account waveforms – 100. Calculations are made with the integration step 0,1 c, integration time – 30 s.

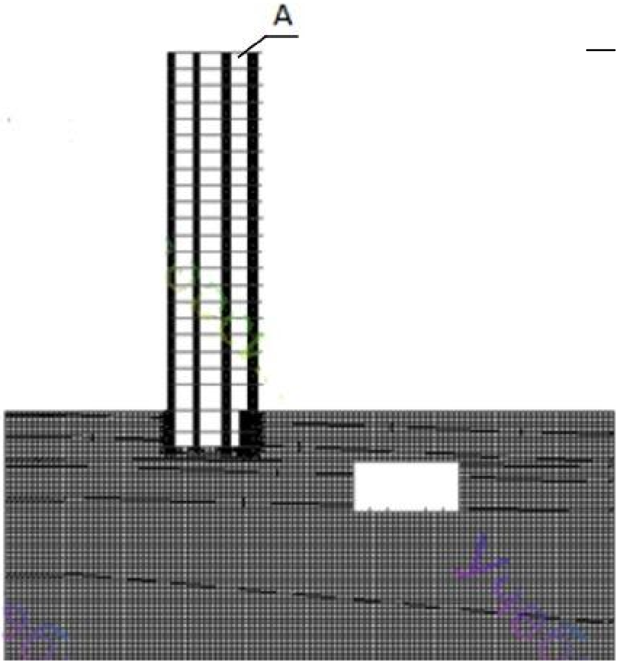


Fig. 2. Design scheme to dynamic effects;

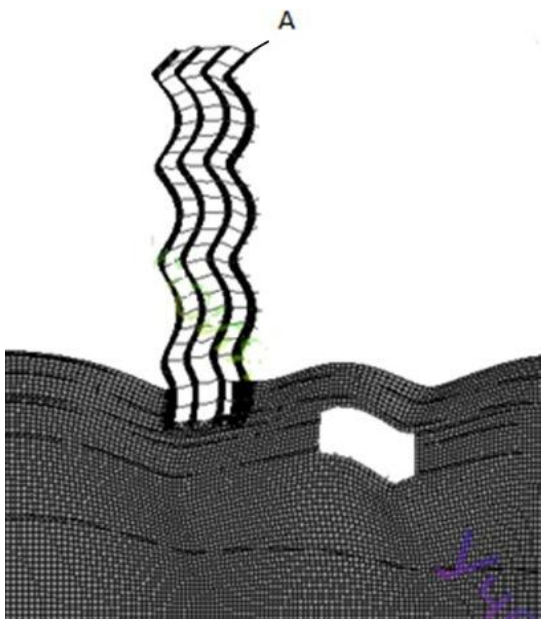


Fig. 3. **Buckling mode of the building, after the dynamic influence**

Figure 3 shows the form at t =16 c.

Figure 4, a and b are graphs of acceleration at the reference point A.

Table 1 gives a comparative analysis of the permissible vibration acceleration according to sanitary norms, standard ISO, and received results in the numerical experiment.

During researches was established:

- Additional precipitation bases depending on the types of soil, their condition and the vibration intensity reaches 50–200 мм, is usually uneven, and their development is comparable with the period of operation of the facility;
- Projects newly constructed within the existing transport vibrations of buildings and structures shall be made according to the damping properties of the soil of their bases to meet design loads and impact of transport modes;

- Should be noted practical absence of standards of acceptable levels of vibration of the soil and buildings caused by traffic. Navigate in this case on the sanitary standards should be very careful, As for human and building structures vibration in different frequency ranges have different degrees of dange. Equally, very problematic spreading of earthquake engineering on the transport vibration standards, are themselves in some cases are quite problematic.

Table. 1 Permissible root mean square deviation of acceleration along the axis Z

Designation of the normative document	$u \cdot 10^{-3}$, м/s^2 in octave bands with geometric mean frequency, Hz	
	31,5	56
CH2.2.4/2.1.8.566	7,0	12,75
ISO 2631-2	27,6	49,06
The experimental results	66,25	138,62

Conclusions.

1. A large number of experimental studies of the vibration influence on the lower structures of buildings and structures allows the use of existing mathematical models, and using based on them software systems, to conduct a series of numerical experiments.
   To determine the stress-strain state of the support system of the building and ensure its information support throughout the life cycle of the information necessary to create a model of the construction, which should be based on mathematical models that adequately reflect the spatial work bearing system at each stage of the life cycle. Unified information model allows: efficiently react to emergency situations; model the processes of development of certain of negative processes; evaluate the stress-strain state at any stage of the life cycle of building object.

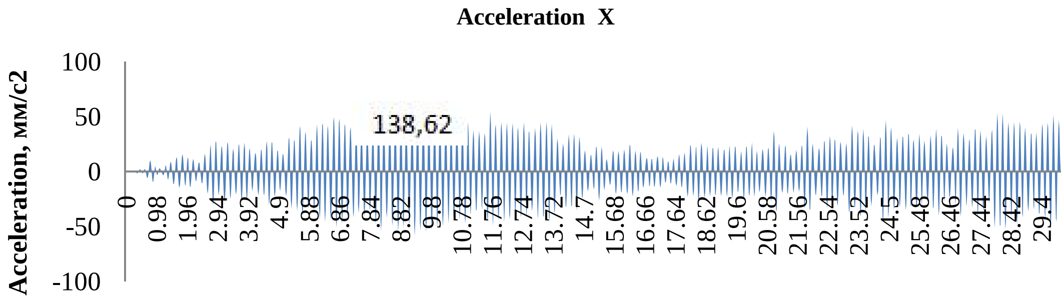


Fig. 4,a - acceleration along the X-axis at point A

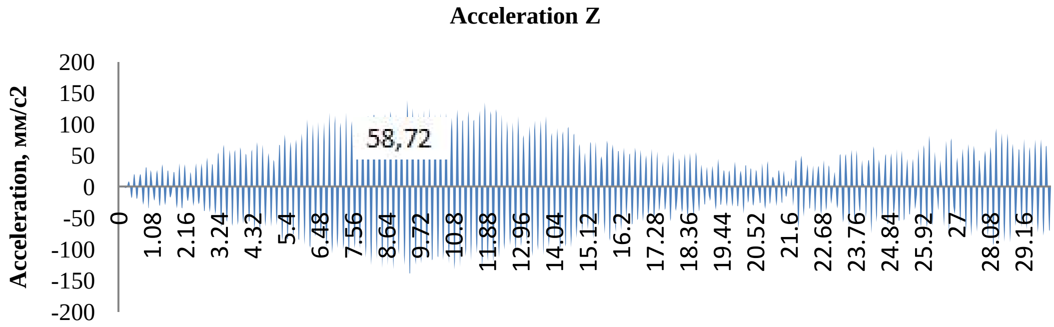


Fig. 4, b - acceleration along the Z-axis at point A

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29-30 May 2014, Tbilisi, Georgia

DESIGN AND CONSTRUCTION OF TOWER EARTHQUAKE-PROOF STEEL CANTILEVER FRAME SYSTEM

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Abstract: In the article are stated issues of design and construction of the unique Sarpi customs high-rise tower steel asymmetric load-bearing frame and mounted on then large-span 7 cantilevers, spatial structural system. The design dynamic model and basic issues of principal solution of constructed seismic resistance high-rise buildings high seismicity, wind loads and complex geological conditions are considered with corresponding analysis and recommendations.

Keywords: proof; cantilever; frame-bracing; stiffness diaphragm; deflection.

In the modern seismic resistance engineering practice in recent years more widely are applied building with complex architectural shapes and tectonics. Such buildings include: sport facilities, large-span halls, airports, high-rise buildings and other special architecture - urban buildings. This case demonstrates important role of scientist and engineer - designer which with architect are the co-authors of unique, complex architectural shape's and accordingly project's structural system. The special approach requires construction that is implemented in seismic country, where due the standards are accepted many design restrictions. The existing analysis methods on seismic impact in some cases can not fully reflect the reliable mode of deformation of the structural system and elements. The complexities are made also due wind loads and increasing aerodynamic coefficients, turbulence and consideration of other related factors.

At mentioned complex, asymmetric distribution of stiffness and mass on the buildings during earthquake are acting additional torsional forces that dramatically increases their spatial tensions and level of damages, in many cases makes up to the critical point of load carrying capacity. The present level of development of the theory and analysis methods and design practice shows that in non-traditional buildings some structural engineering decisions will be made due design requirements. For providing of buildings seismic resistance will be considered engineering thinking and practice. In projects also will be applied progressive systems of seismic protective, dynamic parameters regulation and other cost-effective solutions. In the design process of high-rise asymmetrical, complex structural scheme for their seismic safety it is recommended to conduct a so-called "digital experiment" or calculation of other structural variants and their analysis, with taking into account strength, deformation and dynamic parameters. The Sarpi customs tower height makes up to 39.9 m, in the plan 9,4×8,3 m, on load bearing frame asymmetrically are mounted 7 cantilever spatial girders with maximum length of 15.47 m (Fig. 1).

The steel load bearing frame of tower is made with 12 columns that are made by welded rectangular cross-section 8 columns with dimensions 500×500×12 mm and arranged on the outer contour 4 columns with cross-section of 1000×500×16 mm. On arranged on the outer contour columns from two sides are directly mounted the cantilever trusses. The selection of columns geometric parameters and rectangular cross-section will be defined basically due the requirements for stiffness and reliable behavior on torsion. The longitudinal girders of load bearing frame are executed by rolled I bar № 30.

The main tasks of engineer - designer was obtained due developed with consideration of architectural parameters design dynamic model that which would ensure the carrying capacity of the building, permissible deformations and according dynamic parameters for creation of given unusual architectural shape. The task is complicated due building height, limited dimensions in the plan, large-span non-symmetrical cantilevers and other geometric parameters. The load-bearing basic steel frame was accepted by frame-bracing design layout. In X-Y direction columns and longitudinal girders are connected to each other by frame units. For undergoing of horizontal seismic and wind loads in both directions are arranged vertical bracing (stiffness diaphragms).

Due mentioned constructive solution, it is possible to achieve spatial stiffness of system in the horizontal X-Y direction. The stiffness check is expressed by maximum displacement of frame that should not be greater than 1/500 of height. In our case, this requirement was kept with sufficient safety factor: f<1/500 H max 35 mm <1/500 × 39.900 = 80 mm. Where f - its design deflection, H – is the height of tower.

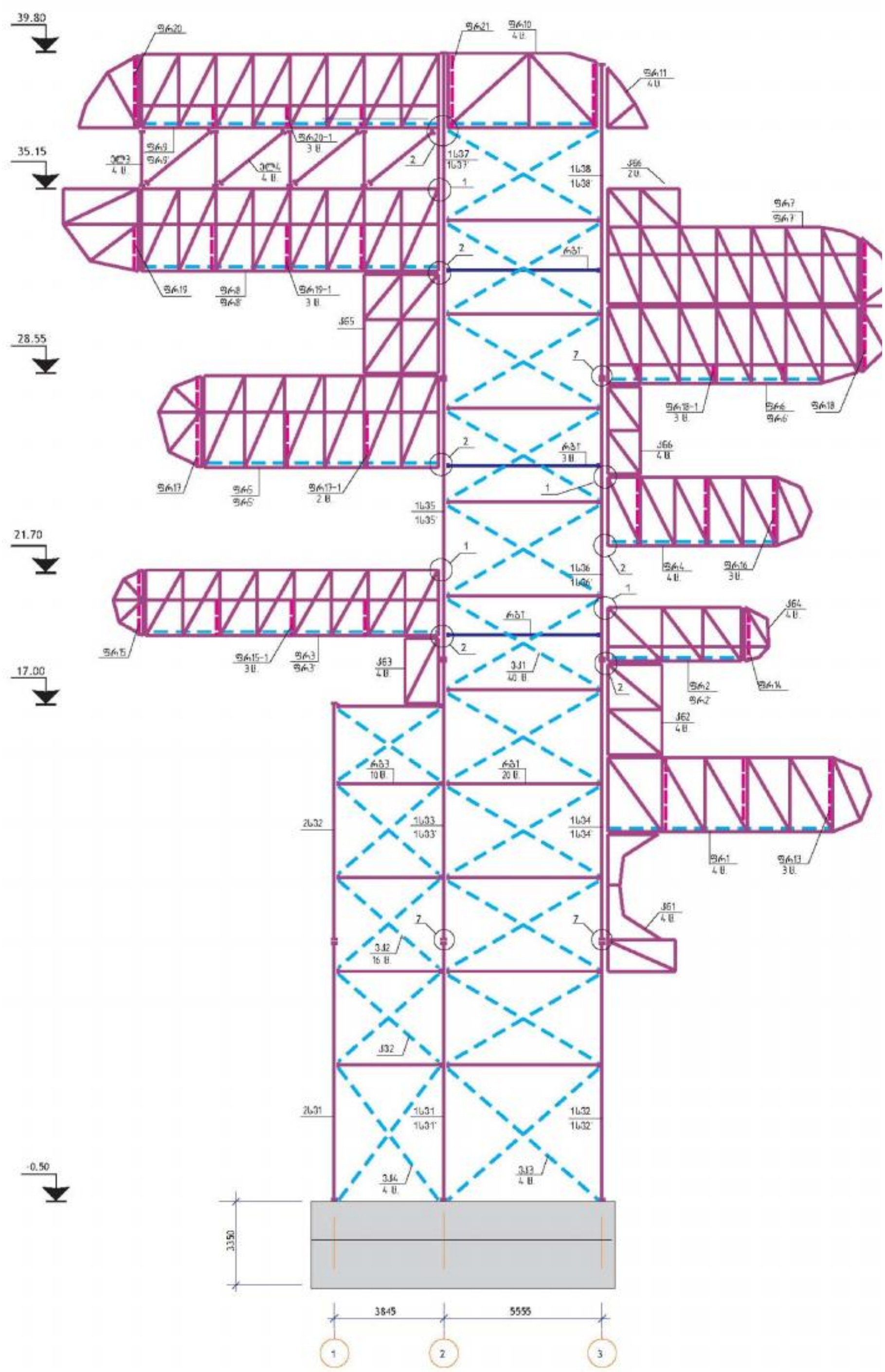

Fig. 1

The determination of mentioned deflection is conditional, because at static wind load are not considered its dynamic parameter options and frame filler and various elements stiffness and friction forces that is very important, also at impact of seismic forces. It would be mentioned that such conventional check guarantee that as a result of building design displacement will not be substantial damaged the frame filler and it will be practically available for operation. The consideration of these circumstances was very important at outer contour facing of Sarpi high-rise tower that is executed by daubing of 5-6 cm thickness cement – sand mortar on wire grid. The almost 3 years practice of operation shows that on mentioned daubed faces have not been observed originated due horizontal movement cracks. In some areas there are only minor wooly cracks.

At definition of earthquake-proof of high-rise buildings structural system's spatial stiffness a necessary condition is the engineering analysis of obtained by calculations the free oscillations period T sec that globally defines the actual stiffness and dynamic parameters of the system that is directly depended on the formation of the final seismic loads and in accordance with the load carrying capacity of the according bearing structures. The free oscillation period of tower makes up to T=1.59 sec and due practice, it is on fully accordance with the obtained in earthquake-proof high-rise buildings values that confirms the correctness of frame - bracing solution (in multi-storey buildings it is known due the practice that T=2-2.5 sec).

On the load bearing steel frame, on full height non-symmetrical from level 12.5 m up to level 39.9 meters are mounted 7 different sizes large-span cantilever spatial girders with maximum length of 15.47 m that with jointly with frame by application of horizontal and vertical connections makes spatial, structural system (Fig. 2, 3, 4). In girders as horizontal and vertical connections are applied the rolled profiles (angle bar, channel bar). The complex task of design were to create from spatial structural load-bearing large-span girders the cantilevers with stated architectural - geometric dimensions the spatial structural load bearing system that would satisfy the strength, stability and deformation conditions with taking into account the acting seismic, wind and other loads. The maximum shoulder of cantilever makes up to 15.47 meters. The normative allowable values of deflection will be defined in the end of cantilever: L/250X2 15.470/250X2 = 124 mm > 96 mm (design deflection), where L – is the length of cantilever. The maximum deflection of cantilever with safety factor of 23% is provided. In the rest of cantilevers the safety factor is significantly higher. In order to reduce deflections the cantilever's heads are raise upward up to 5-10 cm or we make structural building rise. The cantilevers covering's floors

are make from corrugated steel sheets with a thickness of 4 mm that in comparison with reinforced concrete thickness of 10 cm approximately in 5-6 times reduce constant load on 1m².

Fig. 2

Fig. 3

Due the seismic resistance requirements the non-symmetric large-span cantilever part of high-rise tower represents complex structural system, executed in the high seismicity and wind conditions. It should be noted that practically there are not existing experience of design, construction and operation of similar type buildings. This ensured that in the project was accepted reliable frame - bracing layout of frame. The cross-sections of steel structures of cantilevers and all load-bearing elements are selected on the basis of the elastic area of their behavior, or their design strain makes up to 60-65% from yield strength point. Also liberally are selected due the flexibility condition compressed elements and allowable deformations of basic structures. The aforesaid conditional decisions improve the reliability of tower steel structures for seismic resistance. In the frame elements and girders are applied the following steels: C245 GOST27772 88*, GOST94 (St.3ps 6, St.3ps). The load bearing frame of tower is made by automatic and semi-automatic welding method in Rustavi metalwork's factory, the erection on site is done by manual welding. Particular attention was paid on the factory and on site to the quality of welding seam's execution; their control was carried out using the ultrasonic method. The transportation of metal structures from the factory on construction site, from Rustavi up to Sarpi was executed by special motor transport trailers. The maximum length of columns and girders makes up to - 11.5 m. (weight is 4.7 t) and 12.8 m (weight up to 3.4 t). The transportation of mentioned elements on the passes and tunnels was carried out according to the recent rules.

Fig. 4

I would like to note that the chief engineer – designer of project should be involved in design, manufacture and erection during the total cycle, especially when dealing with the complex engineering high-risk facilities. At design of Sarpi custom complex, where are located the different height buildings, was the most important the definition of foundations levels in adjacent of the tower for avoiding the unforeseen impacts of loads and stresses on each other from neighboring foundations. It is known that the transition of seismic wave on structure starts from the foundation and its reliable work determines the total structure reaction on seismic impact, and therefore on the load carrying capacity. The foundation of tower was located at direct sea coast. His depth level and structure was determined due the given area engineering - geological report and carrying out reports. Grounded on the mentioned data have been assumed two-layered foundation slab construction from concrete and reinforced concrete, located in level −3.80 m. with design resistance of gravel ground R=4.0 kg/cm². The steel load-bearing frame columns are supported on reinforced concrete slab with height of 1500 mm that in turn supports on a concrete slab or pillow with the height of 1850 mm and dimensions 14X14 m. These two slabs are connected to each other on contour with the shoulder of 4 rod reinforced bar at 20 locations. the absolutely rigid disc with an overall height of 3350 mm, weighing about 1,500 tons that is supported on ground and fixed in it is created.

For more persuasiveness are carried 3 variants of calculation by joint scheme "foundation - tower"(i.e., the boundary conditions), or we conduct a digital experiment. In the assumed variants of calculation are considered the change of modulus of deformation of basic supporting ground characteristics: in the first variant is assumed E=15 MPa, or weak (virtual) ground, in the second in the real ground E=40 MPa on that is based on the foundation of tower and in the third variant was assumed non-deformable absolutely rigid foundation. The comparative analysis showed that the change in tower fixing stiffness and modulus of deformation of supporting ground practically is not affected on the spatial tension, deformation and the period of natural oscillations of system – T sec.

The spatial frame of tower with cantilevers is calculated on the personal computer using universal software complex "Lira-9.6", with loads basic and special action by SNiP 2.01.07-85* “Loadings and impacts”. In the project was accepted the following natural conditions: wind load - 70 kg/m², snow load- 50 kg/m², with seismicity magnitude 8. For more persuasiveness in 2013 the above mentioned tower was re-calculated by a new software package - Autodesk Robot Structural Analysis. The obtained results with 8-12 % accuracy match with "Lira-9.6" results.

Constructed in 2011 the Sarpi custom's high-rise tower by its architectural and engineering design belongs to unique engineering facilities. It was built in the coastal region with high seismicity and complex geological conditions.

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29-30 May 2014, Tbilisi, Georgia

PROBLEMS OF PRESERVATION-MODERNIZATION OF URBAN HOUSING STOCK AND RELIABILITY OF EXISTING BUILDING STRUCTURAL SYSTEMS IN CONDITIONS OF INCREASED SEISMICITY OF TERRITORY OF GEORGIA

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Abstract: *In the article is developed the idea of creation of carrying out automated information system that will be based on well-ordered set of documents and information technologies; on application of computers machinery and communication equipment due that will be realized the information process. Is proved that the efficiency and effectiveness of mentioned system would be provided due informatics, as well as due keeping the forensic tasks respondent principles that are related with information search for the information system, operation of information support system due that occurred functioning of these systems and creation of search - information fund.*

Keywords: expertise; engineering; automated; informational system; fund.

1. INTRODUCTION

The increasing the seismicity of the whole territory of Georgia by one magnitude, especially after the devastating earthquakes in Spitak, Racha - Upper Imereti and Shida Kartli in 1991, represent the late, but has been recognized from the very beginning as absolutely necessary and justified measure. However, for the working in earthquake engineering field professionals, this was a very extraordinary phenomenon, and they face not the short chain of very difficult problems. Firstly on the agenda is rise the task of residential and public buildings existing stock categorization to be maintained or modernized having architecture - historical importance of certain objects and avoid the possible loss of human lives caused by devastating

earthquakes, or big financial - economic losses. Not to mention the complexity of this task execution, after each building categorization would be resolved the issue, whether each particular building would withstand stronger on one magnitude seismic effect impact compared to the design one, on that it was designed and built. In other words, it should be resolved the building will be demolished, reconstructed, or it does not require any alteration. Obviously, this design – calculation task will be solved in close connection with architectural - planning complex issues. While the rational solution of calculation - selective tasks are directly related to the structural bearing systems reliability of existing buildings and seismic resistance problems. In particular, due the jointly application of buildings seismic resistance theoretical and practical methods would be determined the all possible reserves of existing buildings, and if they are not de enough to undergo increased on one magnitude seismic impact, it becomes necessary to develop a rational design solutions appropriate for achieving our goal. At this point, the revealing of possible reserves of existing buildings, as well as rational choice of their keeping - modernization design solution, always will be related with the selection of as accurately as possible and advantageous scheme of real behavior of buildings bearing structural systems during an earthquake. In addition, practically is stipulated a making of more effective solution if the calculation of structural system has been conducted due a deep analysis of their spatial and non-linear behavior on seismic impacts.

By recent normative documents on buildings seismic resistance is not defined any jointly method on buildings spatial and non-linear calculation on seismic loads. For buildings spatial and non-linear analysis on the territory of Georgia is presented the wide range of existing buildings load-bearing structural systems. The only by wall materials would be grouped the brick, stone, small block, block, large-panel buildings. The latter may, in turn, be divided as frameless, frame - panel (frame-bracing) and frame building with load bearing structure system. For each of these types of buildings the task of spatial and non-linear calculations, of course, includes a range of different issues, and the addition of each specific building, and simultaneously the each specific building, in addition, with only its characteristic features, and (therefore) makes accordingly pure engineering - structural problems.

Here, the set by us problems, with taking into account recent state, would be belonging also to defining what is the specific governmental policy for residential and civil engineering; how is demographic background and urbanization problems; how is urban planning policy in the capital city and industrial - transport facilities (what buildings are required for habitants and

architectural services in cities – multi storey, in plane and in height having non-regular shapes, or so-called a having a "box of matches" shape, or low-rise building, and so on). This and architectural - planning nature issues, of course, are far beyond our conference's topic, however, on this problem solution governmental approaches is highly dependent the future of some of the conference's participants direction - intensity of activity during certain period, because the country's government and financial - economic strategy for the fields will be the main determinant of how it will be developed by specialists the application field of improvement of building seismic resistance methods.

With this point of view would be some interest in the review of general nature of statistical materials of quantitative - qualitative state of existing in the territory of Georgia housing stock and small analysis that, however, makes conceptual conceptions on building keeping - modernization measures expected scales accordingly of bearing structural systems types.

In thickness Table 1 is stated the percentage of existing residential buildings accordingly of their exterior wall materials and years of construction cited from the Republic of Georgia socio - economic information committee's statistics (see [1]). Due the visual observation-analysis of existing in country buildings, and as a result of the analysis of above mentioned socio - economic information committee's stat (mainly in buildings durability terms), would be done the conclusion related with some of these conceptual problems. At the same time, in my opinion, first of all it is desirable to define some used below concepts.

Accordingly of determined by design or relevant normative document value of service life the existing residential buildings in the territory Georgia would be divided into two large groups - the capital and non-capital building. To the capital buildings would be belonged buildings that service life is equal to or greater than fifty years. Accordingly of such buildings bearing structural wall materials and systems they would be combined in four typical groups:

- Brick, stone, concrete (including in-situ reinforced concrete) and small block buildings;
- Block buildings;
- Large-panel buildings;
- Frame and frame - panel buildings.

All the rest of the building, whose walls were built of wood, mixed materials and other materials may be belonged to non-capital air-dried airbrick buildings.

Table 1

Distribution of residential houses by external wall materials and building year (in percents)

	All houses	Years of construction						
		Up to 1918	1918-1940	1941-1950	1951-1960	1961-1970	1971-1980	1981-1988
All houses	100	3.6	6.0	6.3	17.2	28.3	22.3	15.6
Including constructed from:								
Brick, stone, concrete, reinforced concrete, block	100	5.9	7.2	6.9	20.5	30.3	19.0	10.2
Panel	100	0.3	1.0	2.0	10.0	25.6	33.0	28.1
Timber	100	4.2	15.0	15.7	22.0	23.5	12.3	7.3
Combined materials	100	3.2	9.2	9.6	22.8	30.9	16.8	7.5
Air-dried brick	100	6.7	29.8	21.0	24.6	12.2	4.0	1.7
Other materials	100	5.2	8.2	12.0	29.0	24.7	16.2	7.7
All houses	100	100	100	100	100	100	100	100
Including constructed from:								
Brick, stone, concrete, reinforced concrete, block	47.1	77.2	56.2	51.5	55.7	50.6	39.3	30.6
Panel	33.6	3.1	5.9	11.1	19.3	30.6	48.3	60.4
Timber	7.5	8.8	18.5	18.6	9.5	6.2	4.0	3.5
Combined materials	11.2	10.0	17.1	17.1	14.7	12.3	8.3	5.4
Air-dried brick	0.4	0.7	2.1	1.4	0.6	0.2	-	-
Other materials	0.2	0.2	0.2	0.3	0.2	0.1	0.1	0.1

Existing a serial, as well as other types of capital building characterization in terms of their bearing structural systems and elements of modernization - reconstruction on the conceptual level, actually means the assessment of buildings typical groups according of normative acts. As for non-capital buildings, in point of structural view, they should not be reconstructed and after the termination of their service life completely will be demolished. Grounded on the analysis of statistical materials, we may say that their share in the total housing fund makes up to 12-20 %.

For the first group of capital buildings the structural modernization is necessary, when their roofing (most of the arranged on wooden beams) is significantly deformed and unable to perform the load-bearing wall spatial earthquake-proof system integration function, or when the buildings bearing vertical structural system itself cannot meet seismic standards (building does not have the seismic belt, has an unfavorable configuration (in terms of seismic resistance) in plan and on height), are non-reliable connected the walls and roofing, due the foundations uneven settlings or

by other reasons the bearing walls have a significant damages and so forth. The statistical analysis of the existing in country housing stock shows that the specific weight of such buildings makes up to 10-15% of the total housing stock.

The second group of capital buildings - the structural necessity of modernization - reconstruction of block buildings mainly is stipulated due increase of the seismicity of territory of Georgia, because of the designed - calculated on low seismicity eight-storey block houses would no longer meet current seismic standards [2] in increased seismic conditions. Therefore, these buildings will obligatory require reconstruction activities for improvement of seismic resistance. According to the statistics data such buildings share in country's housing stock makes up to 5-10 %.

As for large-panel (third group of capital buildings), frame and frame - panel buildings (fourth group of capital buildings), they meet the structural requirements of current regulatory acts on the increased seismic conditions, if their precast structures and elements connections are made in full compliance with design solutions. In addition, it should be noted that rejection of this type (or arbitrary) of buildings reconstruction-modernization (in increased seismic conditions) in each specific case must be preceded by the study - analysis of any adverse factors in the building configuration and seismic resistance including the spatial and non-linear calculations on the increased seismic impact. Such analysis and calculation would be performed in terms of seismic resistance unfavorable configuration of frame (pure frame) buildings, which probabilistic assessment of seismic resistance, as is well known, makes relatively low levels of reliability in comparison for large-panel and frame - panel buildings. It is clear the seriousness of the issue is more growing for multi-storey extremely responsible - frame buildings.

Accordingly of wall materials and structural systems are conventionally grouped related to buildings, at the end of stated - proposed reconstruction measurements lest generalized, once again I have say that for every specific building the necessity of reconstruction and modernization will be determined grounded on the building's load-bearing structural system and elements detailed study of the buildings and deep analysis of the building's seismic resistance. The necessary for the implementation of reconstruction work determination - selection of specific solutions and works would be based on comprehensive engineering calculations and technical - economic analysis.

Due the constructive point of view, the reconstruction-modernization of existing buildings, caused by the increasing on the seismicity of territory of Georgia, mainly will be implemented by extension or extension-superstructure. In both cases the crucial importance has such interconnection of the new and the old bearing systems, enabling them to reliably behavior jointly in any direction of new design seismic loads. The creation of joint spatial systems from new and old bearing structural systems in each specific case, as already was mentioned, would be preceded by development of variants and comparative analysis based on relevant accuracy of the spatial and nonlinear calculation. At carrying out these calculation would be applied developed by us methods, algorithms, as well as created on their basis CAM programs that provide the proposed, execution of requirements of system – variable stiffness units and equipped with elements of old and new bearing structures reliable behavior (see [3]).

General and a priori would only to say that even in the case of extension-superstructure as well as in the general seismic resistance construction, the great importance has durable and lightweight construction materials (for example, for the filling of external walls is effective the application of lightweight three-layer wall panels, etc.) that provides significant decreasing of expected seismic forces and seismic resistant structural elements materials consumption. In other words, in fact, due this yet are developed certain constructive fundamentals and prerequisites, causing with bearing structural systems change - modernized significantly would impact on building expected seismic reliability.

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29-30 May 2014, Tbilisi, Georgia

RELIABILITY OF ULTIMATE LOAD-CARRYING CAPACITY OF ARCHITECTURAL MONUMENTS LONG TERM IN SEISMICALLY ACTIVE REGIONS

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Georgia, a country with ancient history is rich with architectural monuments of antiquity and Christian period. In regards to climatic diversity, almost all microzones characteristic of Mediterranean Sea basin and therefore designs of architectural monuments are also extremely diverse. Nevertheless main architectural essence of construction is based on Greek-Roman engineering achievements. Basic bearing, vertical structure of Georgia buildings, i.e. a wall is a three layer masonry (two layers of stone with mortar in between) characteristic of Roman empire and Provinces. In Georgia, similarly to other places in Mediterranean Sea basin, stone shivers are inserted in mortar, which makes it much more durable compared to common lime-stone mortar. It seems that lots of attention is paid to usual masonry but in historical reality of Georgian architecture it is basic construction element and its comprehensive analysis shall enable us to understand contemporary stability and stiffness of monuments constructed decades of centuries ago.

In the 20-th century people in Georgia, and as far as I know in many other countries as well, have become aware of the threat, born by degradation of stability and therefore stiffness indicators of bearing constructions of historical monuments.

Some time ago it was considered that destruction of a building was a logical outcome of development of plastic deformations. Recently a controversial approach has appeared, which does not view destruction as an active thermo activated process. Destruction is taking place in a building, which certainly is under the strain from the moment when it is subject to common or functionally load. According to the above theory, which is based on stability of materials and dependence of stability on time factor, deformations and splitting should be determined by rate of deformation and splitting process rather than maximum stress level, which is quasi stable as a

result of plastic sliding. Here we need to mention fundamental feature of specific construction materials to change the volume in different degrees as a result of deformations. Records of alteration of material's volume, e.g. during compressing and stretching even in case of hypothetically ideal flexible material demonstrate, that there is no linear relation between stress and deformations.

Hence, recording of changes in the volume of flexible-plastic materials in the process of deformation makes it necessary to introduce sequence of finite set of deformations in order to apply analytical relations of variable parts of Poisson ratio, or formulas representing true stress levels etc.

Since we have accepted changes in volume of flexible-plastic material as a criterion for assessing the deformation process, this relation can be best reflected, from analytical point of view by Ludwic-Hationnk deformation logarithm functions: $£ : d£ = \frac{l_0}{l} d\overline{£}$, where l_o _ is initial size of elements to be studied, and l _ final.

Without presenting mathematical transformations, which have been carried out in Russian Federation, probable image has been identified taking into consideration changes in material's volume, which made it possible to determine concept of material's energy intensiveness. Material's energy intensiveness is defined by proportion of value of specific energies of material with their deformation within given range, based on correlations of stress diagrams for the same material.

As we see, characteristic feature of material's stability are variable values for one and the same construction. In this case important indicator of a building is storage ratio , which is accepted as a determinate value and equals to proportion of mathematical possibilities with stability and load, i.e.

$¸ = \overline{R} / \overline{Q}$.

Calculated stability and load can be received by multiplying their mathematical probability by relevant ratios:

$$R_p = K_o \overline{R} \text{ da } Q_p = K_n \overline{Q};$$

Material stability ratio is called uniformity ratio and are equal to

$$K_o=1-\square A_p$$

Value of this ratio is always

$$K<1$$

Load rato is called overload ratio:

$$K_n=1+\square A_q \text{ and always } K_n>1.$$

Safety indicator $\square$, which normally represents a number of standards, will be intercepted on the distribution curve at the right from the centre and normally $\square=3$, but its value may be specified according to the probability of destruction and shape of distribution curve.

For the category of buildings, which we are going to now discuss, we have excluded useful (operational) loads, but natural stresses: atmosphere (wind, snow) and most importantly, overstresses caused by earthquakes create extreme conditions for buildings in long term.

We need to pay special attention to this issue, since Georgia is a classical country from seismic activeness point of view, especially taking into consideration that during the recent decades Caucasus region and Georgia in particular has become arena for a number of destructive earthquakes

As is it known, powerful earthquakes are caused by tectonic processes taking place in the crust of the Earth, which are associated with disturbance of integrity of the crust in its certain sections. A wave, created as a result of such disturbance, experiences cycles of reflections and refractions because of chaotic placement of geological strata and therefore it reaches foundation of buildings in irregular manner. Accelerogrmas recorded during earthquakes represent different functions of time and due to their multiplicity belong to class of random functions. Generally, aggregate seismic accelerations represent non-stationary random process.

Therefore, calculation of buildings shall also be done applying theory of probability in order to establish correlations between random functions of stability and stress.

For flexible-plastic material a stem is taken which works in accordance with Prandtle diagram, i.e. it is subject to Hook's Law to the point of fluidity limit and after that under pressure of the same force it suffers constant deformation .i.e. the probable tensity is expressed by the following formula:

$\overline{\pm}(£) = £E\langle 1 - P_{GT}(£E)\rangle + £E\pm^2 P\pm^2(\pm^2)d\pm^2$

AAfter calculation of disperse of the received random function and after applying simplified signs $\overline{\pm}(£) = £_*(1 - P_0) + P_1$, when $£ \to \infty$ $P_o=1$ $P_1=G_t$ $P_2-P^2=G_{T.}$

With presented equation and their transformations we would like to prepare the audience to hear about the results of studies, which have been carried out in the agency for cultural heritage protection during recent years.

As it has been mentioned at the beginning of the paper, masonry of walls of Georgian churches are three layered – two side stone facing and mortar layer in between. During centuries the above buildings endured vertical stress and at the same time they have been through extreme overloads because of seismic events.

This self-survival has been achieved through aggregation of three layer masonry, which we are going to talk about, using examples of Georgian monuments of various periods.

First of all, we would like to stop your attention on our outstanding monument – Athen's Sioni.

Athen church was constructed in the 7-th century and is an extremely important monument of Georgian architecture in the sense that with Athen's Sioni, Mtskheta Jvari and Martvili Cathedral, system of cross-dome churches are introduced and established in Georgia, and it becomes a leading and canonized architectural system till the 19-the century. The church is constructed on stone substructure built on the left side of river Tana gorge and the foundation has been stable during the past centuries.

The church came into the centre of attention of foreign experts and restorers due to restoration of frescoes of 12-th and 13-th centuries and after that state of its bearing construction also was paid attention, since stability of the building became an issue and precondition for survival of frescoes.

German experts: professors Riner Barter and Ervis Emerling from Munick Technical University have contributed a lot into study of church stability; they have comprehensively studied static condition of the church and provided detailed characteristics and description of peculiarities of the masonry. They have also identified key sites of damages and described their causes. Scientific work has been further continued on the church and at the suggestion of Mr. Bartel and Mr. Emerling a relevant laboratory of Holtsminden Applied Science and Arts Institution , with then participation of Mr. Tao Guo and Mr. Hao Hong have tested samples taken

from Athen's Sioni and have received rather comprehensive and informative results in the context of technology and composition of mortar.

The received findings with the kind permission of the authors were used for our further researches and for replications on other Georgian churches.

As it has been mentioned German experts have carried out comprehensive study of thechurch construction and have implemented a number of cycles of static calculations, according to which the most overloaded sections of the church have been identified.

The highest indicators of concentration of tensity have been identified at the base of dome neck at the height of trumpet arches and in the aone of arches supporting the dome, which was explained with existence of buttresses of the dome.

However, visible damages have been identified in the contact area (interface) of the walls and upper foundation, which is not in full compliance with the outcomes of static calculations.

In any case the scientists have identified the- end- to end crack, which crosses south section of the church along the apsis conch and postophorium and has separated this section from the main building.

According to German scientists, the above deformation was caused by subsidence of substructure located at the base of the building; since this opinion was doubted by Georgian experts, a pit of approximately 8.0 m height was cut at south apsis (conch) close to the foundation, which showed that substructure had been laid by cobblestone and crushed rock on lime mortar and was based on rock. Visual study of test pit showed that artificial base, i.e. substructure, had no deformations and therefore subsidence of the foundation could not have been considered as a factor causing appearance of the crack.

Obviously, the German specialists failed to take into consideration the fact, that Ateni Sioni is located in the most active seismic zone of Georgian territory and even a slight earthquake has its impact on bearing structure. This fact explains the frequent damages of the walls adjacent to socle.

The work undertaken is outstanding for its results, as we already have the data on mortar chemical analysis, strength characteristics and a vital finding that proved the presence of diffusive processes in Georgian stonework.

The data obtained will be used while studying and analyzing the features of random function caused by seismic influence.

Furthermore, the German specialists made a significant discovery for the Georgian construction technique; it was surprising for the European specialists that despite the lack of written documents, Georgian constructors managed to maintain the mortar preparation technology that is conveyed from generation to generation orally.

German counterparts expressed keen interest in mortar technology and they studied the 16th century monument – Kiriskhevi Church in Kakheti region, which is located quite far from Ateni Sioni. The monument is outstanding for outbuildings of various centuries - 5th, 6th, 10th and the last one of the 16th century. The precise analysis revealed that Georgian architects strictly adhered to the mortar technique, from chemical and granulometric standpoint.

As far as the mortar structure and composition acquired such an importance, the work in this direction has continued to study the technologic aspect based on the data concerning the restoration works to other significant monuments as The Cathedral of the Dormation, or the Kutaisi Cathedral. The Cathedral was built in 1003 during the reign of King Bagrat III due to which it was called Bagrati Cathedral.

Bagrati Cathedral is a masterpiece of Georgian architecture. It is one of the largest cruciform domed churches, a dominant style in the then Georgian architecture. However, it significantly differs from the other two excellent monuments such as Mtskheta Svetitskhoveli and Alaverdi St. George Cathedrals. These and other cruciform domed churches of Georgia (Tsromi, Samtavisi, Betania, Kintvisi, Pudznari etc.) are inscribed in tetragon and the cruciform composition of the church is visible on the roof level, while Bagrati has a clear cross shape on the upper basement level and is in harmony with the excellent cathedras in South Georgia, namely Tao-Klarjeti (currently within the Turkish state borders) such as Oshki and Ishkhani. Therefore, Bagrati Cathedral is an important cultural heritage site as a symbol of the unity of artistic mentality of architects of the ancient Georgian provinces.

Up to the year 1692, the church had been stably fulfilling its function until its dome was exploded by Ottoman invaders while running away from Kutaisi. The next centuries were not successful for the Cathedral; in the 18th century, Imereti Kingdom experienced difficult interior and exterior Liberation struggles; in the 19th century, Georgia came under control of Russian Empire, whose Holy Synod tried to wipe the trace of Georgian Christianity off the face of the earth by its circulars and actions and therefore, it would not care about the restoration of ruined church. The 20th century – the epoch of Bolshevism - was outstanding for atheism and

blasphemy. Only in the 21^{st} century, after restoring the Georgian state, the full restoration of Bagrati Cathedral (on the 310^{th} anniversary of its disruption) has become possible.

By the decision of the International Organization of European Architects, in 2012 the Bagrati Cathedral as the best model of restoration project, together with one of the Venice monument was awarded gold medal .

Prior to the restoration works, we conducted a study of the strength of the remnant walls. Similar to Ateni Sioni, Bagrati Cathedral stonework consists of three layers and its strength characteristics continue the traditions we talked about while studying the Ateni Sioni stonework.

Prior to the restoration works, several studies have been conducted including the mortar strength. Over 300 years the walls have been without roof. After removing internal and outer facing, the exposed mortar-concrete layer was studied to define its strength in many points using sclerometric devices.

The measurements produced very interesting results: minimal value of mortar strength was 3.4 MPa in the Northern branch interior; maximum 10.0 MPa – on Southern façade. According to the 31 points measured, the average strength is 5.7 MPa, which should be considered a good result taking account of the fact that the mortar was exposed to atmospheric conditions for over centuries. Unfortunately, natural stones showed large decline in strength - 10.4÷50.8 MPa. The latter once again proves the high quality of Georgian mortar.

According to the standard tables for determining stonework grade, the leading component of the stonework strength is the specific strength of mortar. Strength of dry stonework for any figures of stone strength (25÷1000 kg/cm^2), does not exceed 10% of its strength.

Mortar strength in combination with stone strength, taking account of the ratio of the component strengths (mortar strength to stone strength), produces more interesting results, e.g. in terms of the stone strength: 500, 400, 300, 200, 150, 100 and mortar grades 100, 75, 50, the stonework strength is fluctuating; in particular, when mortar grade is 100 and 75, the stonework strength is within 70% and 23%, while in case the mortar grade is 50, it sharply changes from 60% to even 240%, which perhaps will become the topic of next study. Such a slight digression from the subject is to prove that mortar strength and deformability should be the key issue for further studies.

In this respect, I consider necessary to capture your attention on the late 19^{th} century building in Tbilisi, which was intended for synagogue. The building is outstanding for the fact that its main ritual part is encompassed in ferroconcrete carcass. The basement and double light

hall is inscribed between the coupled columns located in the angles of a right octagon. On the perimeter of these columns, a ferroconcrete 14 m diameter hemispheric dome is rested. Ferroconcrete carcass is surrounded by brick walls, which also serve as the support walls for overhung tiles of basement, second floor and dome support circle, in terms of the storeys.

From the standpoint of ferroconcrete usage, it is one of the first buildings in Georgia and certainly contains some gross errors due to the lack of relevant experience. First, the building is based on filled soil; second, the coupled columns are placed on concrete tiles without any connection (which was incredible on the stage of measurement works); and finally, weak fastening of ferroconcrete overhung tiles in brickwork. Owing to the above facts, during one of the earthquakes the base of two coupled south-western columns turned around its edge rib and as a result of such overload the building was seriously damaged.

In this case, we are interested in the state of brickwork, which was made of so called Russian brick (250X120X65), grade 75÷90kg/cm^2 , and mortar 50÷60kg/cm^2. It is worth noting that mortar is based on cement indicating that a traditional approach to the mortar preparation issues highly prized during the early and middle medieval period was actually lost in the 19th century.

I have touched upon this issue while discussing the background of Bagrati Cathedral, so I will not go into detail now.

Based on the studies and data obtained, we can safely say that modern approach to the processes of building collapse, is linked with the collapse of a strong body.

Namely, in this case it depends, for instance on the following: Ateni Sioni wall collapse, which in its turn is a combination of middle layer of mortar, and internal and external facings. That’s why we assume that collapse will occur during seismic overload, when the walls of southern or northern apsis and adjacent longitudinal walls (similar to the formation of a through-thickness crack taking place on southern façade 50cm high, i.e. within the two rows of horizontal quadras of coating) will not undergo final destructive Deformation.

Assume that the rate of structure collapse is directly proportional to the mass of the core and facing. We need to define wall mass change with time, if in initial time t=10 the mass is m_o=40x0.9x0.5x2.1=37.8t ,

the rate of structure collapse is determined by formula $l1m\frac{\Delta m}{\Delta t}=\frac{dm}{dt}$.

According to the task, $\frac{dm}{dt} = -km,$

where $k>0$ and is a Proportionality coefficient.

Differential equation with divisible variables has been obtained $\frac{dm}{dt} = -kdm,$ $m=Ce^{-kt}$.

in order to determine K, assume that during the t_o time, ~ % of mass is collapsing, then

$- kt = (1 - \frac{\sim}{100});$

In our case, K=0.000385;

That is why, it will be acceptable if the half volume is destroyed, i.e.

$\frac{m_0}{2} = m_0 e^{-0.000385}$

$T = \frac{l_n 2}{0.000385} =$ 1800 years

Approximately 1300 years passed since the constriction of Ateni Sioni and in case of 28% damage, the church lifetime is ≈500 years. The above calculations are almost idealized, as seismic overloads, i.e. declines from random loading functions are to be considered.

29-30 May 2014, Tbilisi, Georgia

ROAD STRUCTURES AND THEIR SEISMIC RESISTANCE

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In the world there are a lot of disaster facts that are caused by the earthquakes. The roads and road structures (bridges, pipes, reference walls, etc.) are very sensitive constructions to the earthquake shocks.

Man had experienced many natural disasters back to the dawn of human civilization. Because of ignorance, humans asked gods for help, performed ritual dances and sacrifices. In the process of civilization development human made a lot of findings, the science and techniques were developed. In XXI century people must have ability to forecast the natural and technical disasters, to avoid the great moral and financial losses.

There are many historical examples confirming the development of human cognition in study of impacts of seismic forces.

On Georgia territory, it is important to note the catastrophic consequences of earthquake, occurred in 1989 year in Spitak: loss of life, layed to waste multi-storeyed houses, financial loss. At the same time, after earthquakes that occurred in 1989 year in California and in 1992 year in Los-Angeles, victims and destruction were minimal, in spite of densely populated territory with skyscrapers. The comparison of mentioned above two incidents consequences, shows that prerequisite on which is based modern seismic resistance theory, and normative acts elaborated on this base, sufficiently determines earthquake impact on buildings, and based on this data qualitatively constructed buildings can withstand even the most devastating earthquakes. Testing of many destroyed and damaged or remained after earthquakes constructions, testifies that buildings lifetime and the issue of people safety, as far as other factors, mostly depends on scheme of load-carrying unit, node point design, quality of the construction and also depends on the state of the load-carrying unit during the earthquake.

For many decades Ltd. "Sakgzametsniereba" performs testing and examination works for bridges located on the open roads, to determinate their technical state and actual bearing capacity. Special attention is paid to bridges located on International and Internal State roads, among them to bridges of middle and big sizes.

During its activity Institute “Sakgzametsniereba” has tested more than 800 bridges, only during last 7 years 158 bridges were tested. During bridge examination process the special attention is paid to measures against the seismic forces and to their technical state. To reduce seismic forces influence, the appropriateness of bridge location must be analyzed (rocky foundation, slopes without landslides, etc.), besides this should be investigated if the measures against the seismic forces are in accordance with the map of seismic zoning of Georgia. After every strong shock the technical state of bridge should be tested. Must be mentioned that, in our practice, none bridge was destroyed under the seismic forces impact.

Based on the analysis of earthquakes occurred in recent years in Georgia (Racha) and in neighbor countries, particularly, in Armenia (Spitak), was performed new zoning of Georgia territory in order to increase the earthquake grade. On this basis, taking into account determined earthquake grade during the design stage of highways and road structures, significantly helps to avoid the incidence.

Georgia is mountainous region, so during design of highways and road structures the specific attention should be paid to seismic loads. According to many years experience, more attention (when focused on road topping impact) should be paid to the roads designed on landslide sections. Very common are temporary stabilized landslide sections, which start activity during the seismic shocks. At that time sub-soil mass starts moving, and often occurs the roadbed failure facts. Taking into account all mentioned above, during road design process on landslide areas, great attention should be paid to the anti-landslide measures, from its side it should provide the maximal mitigation of the negative impact from the seismic forces.

For the purpose to increase the stability and reliability of roads and bridges, Institute “Sakgzametsniereba” has elaborated the effective constructions of coast protecting and regulation buildings, most of them are adopted and has State patent.