

INFLUENCE OF DYNAMIC ANALYSIS METHODS ON SEISMIC RESPONSE OF A BUTTRESS DAM

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Abstract: The seismic analysis of a buttress dam with 73.50 m height is performed by the spectral analysis method and the direct time integration method. An accelerogram with 0.1g maximum acceleration was applied horizontally, in the upstream - downstream direction, at the bottom of the dam-foundation finite element mesh. The hydrodynamic effect of the reservoir was considered according to the added mass procedure (Westergaard relation). ABAQUS software was used to make the analyses. The same type of finite element C3D20R was used for the mesh of the dam body and of the foundation. The comparison of the results is made on the displacements, the stress state and the sliding stability on the dam-foundation contact in the full reservoir hypothesis. The comprehensive analysis concluded that both methods had provided close results for the considered case study. The spectral analysis method revealed itself to be more conservative compared to the direct time integration method.

Keywords: dams, earthquakes, dynamic analyses, stress

1. Introduction

Edward Wilson a well-known Professor of seismic analysis of structures from the University of California wrote (August, 28th 2013, www.edwilson.org) the following comments on seismic analysis methods: “Convince engineers that the Response Spectrum Method produces very poor results. Convince engineers that it is easy to conduct Linear Dynamic Response Analysis”.

The scope of this paper is to check this opinion in the case of concrete dams, structures known as having a complex seismic response because of the interaction of the structure with water from the reservoir and the foundation area [6], [7], [8].

The seismic response of the Gura Râului dam, a buttress dam with 73.50 m maximum height, is evaluated comparatively by means of the Response Spectrum Method and the Direct Time Integration of Motion Equations [1], [4].

The seismic action with a PGA of 0.1g consisted of a horizontal component of the accelerogram recorded at Focsani seismic station during the August 30th 1986 Vrancea earthquake. The horizontal component with a PGA of 0.271g (g-gravity) of the recorded accelerogram at Focsani INCERC seismic station during the August 30th 1986 Vrancea earthquake, scaled to 0.1g maximum acceleration was used to act on Gura Râului dam. The subsoil of the seismic station has a geologic profile close to that of the dam site. The accelerogram was applied at the bottom of the dam–foundation system finite element mesh horizontally, on the upstream-downstream direction.

The influence of the reservoir on the seismic response was taken into account by the added mass procedure. The added masses were computed according to the Westergaard relation.

The analysis was performed with ABAQUS software, the solid elements type C3D20R were used for the mesh of the dam–foundation system. The linear elastic behavior of the materials was accepted in all analyses [3].

The comparison of results in the dam seismic analysis for those two methods mentioned above is performed in displacements, stress state and sliding stability on the dam-foundation contact in the full reservoir hypothesis.

2. Short description of the Gura Râului dam

Gura Râului dam, commissioned in 1974, is a buttress dam with 73.50m maximum height (Fig.1). [2], [5].

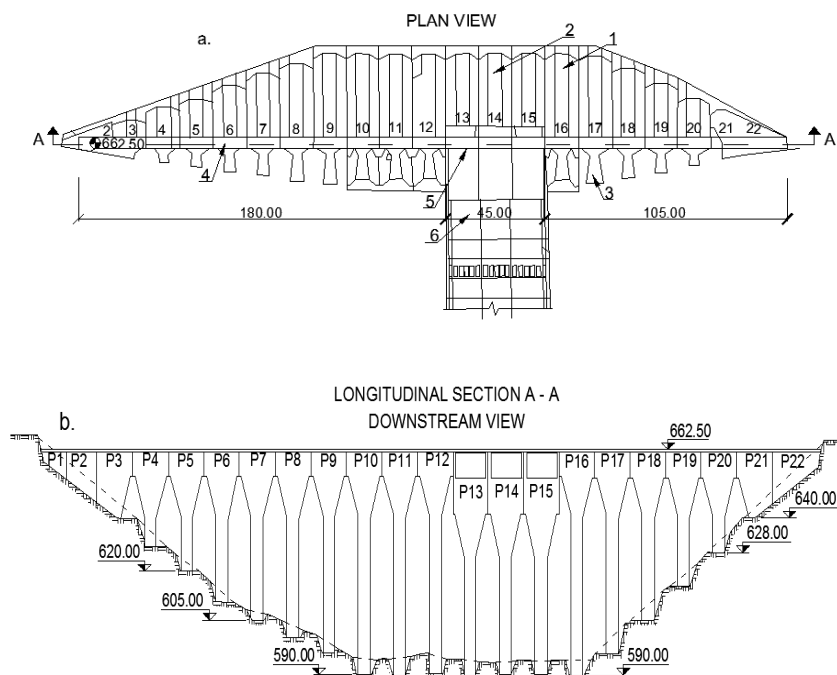


Fig. 1 - Gura Râului dam layout (a) and longitudinal section through the dam axis (b): 1 – non-overflow block, 2 – overflow block, 3 – buttress, 4 – crest, 5 – overflow span, 6 – stilling basin

The dam is located within the Sub-Carpathian Depression of Sibiu County. The rock from the dam foundation consists generally by gneiss with rare amphibolite intercalations and pegmatite intrusions. The friction coefficient to sliding on the dam-foundation contact was evaluated at 0.70.

Gura Râului development has multiple uses as follows: water supply of Sibiu city and neighboring localities (1440 l/s), production of hydroelectric energy (hydropower plant with $P_i=3700$ kW located downstream of the dam) and flood control [2].

The main characteristics of the development can be summarized as follows (Fig. 2):

- maximum height of the dam..... 73.50 m
- crest length..... 328.00 m
- crest width..... 6.20 m
- base width..... 57.40 m
- upstream and downstream slopes..... 1 : 0.57 and 1 : 0.28
- thickness of buttress (variable)..... 4.50...8.00 m
- concrete volume of the dam body..... 300500 m³
- capacity of discharge works..... 832 m³/s
- reservoir volume at NRL..... 15·106 m³

The dam blocks are with polygonal heads and buttresses with variable thickness in horizontal cross-section, increasing from the contact polygonal head-buttress to downstream. This innovative solution proposed by Priscu – Popovici [1], applied for the first time in this field led to significant savings in dam concrete volume evaluated at about 14% compared to the usual solution with buttresses having constant thickness.

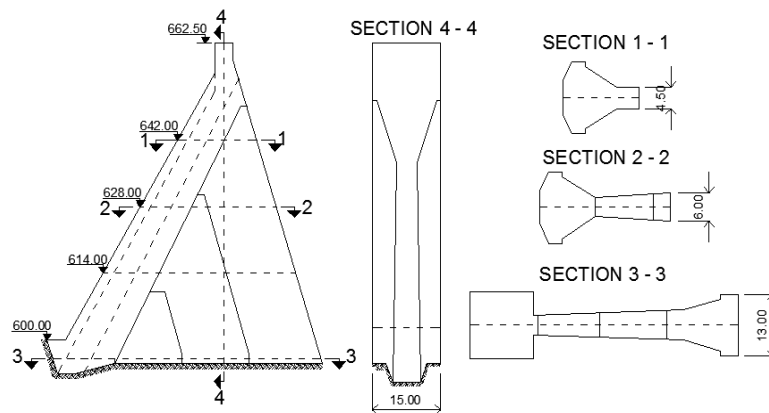


Fig. 2 - Representative cross sections through block no. 11

A number of 19 non-overflow blocks and 3 overflow blocks make up the retention. Water tightening between the blocks within the area of the polygonal head units was achieved by means of a 1.5 mm thick copper steel sheet doubled by a M35 type polyvinyl band.

The dam spillway consists of three free overflow openings located in central blocks. The total length of the surface spillway is 39 m with maximum discharge capacity of 800 m³/s. The bottom outlets consist of two pipes with 1000 mm diameter having together a total discharge capacity of 32 m³/s with the reservoir at Normal Retention Level (NRL).

The energy of the discharged water from the reservoir is dissipated in a battery consisting of two stilling basins located at the dam downstream heel and having 34+18=52 m total length.

The foundation sealing was carried out by a grout curtain consisting of two rows of injected drillings at 1.50 m span, with maximum depth of 45 m for the upstream row and 40 m for the downstream row. In order to discharge the uplift pressure acting into the dam foundation, downstream of the grout curtain a row of 30 m deep draining drillings displayed by two for each block was provided.

According to the Romanian national regulations, the dam is classified as the second class in terms of the economic importance and in the category B in terms of the collapsing risk. Thus, this dam requires a special monitoring during its lifetime.

In order to achieve this provision, the dam-foundation unitary system was equipped with several monitoring devices (4 direct pendulums, 2 inverted pendulums, 7 rock meters with 3 rods, piezometer drillings, hydrometers, telepress meters etc.). The monitoring activity, accompanied by visual inspections, ensures the proper surveillance of the dam behavior. It should be mentioned that the dam has behaved normally in terms of displacements, stresses, and seepage during the entire operation, which started in 1974.

3. Mathematical models and input data

The block no. 11 (Fig. 2) was selected to be computed to OBE seismic action.

In compliance with the Seismic Hazard Map of Romania the dam site is located in an area with $PGA = 0.20$ g (PGA – Peak Ground Acceleration) and period corner of 0.7 s.

Table 2–1 from NP 076–2013 sets that for dams classified in the II class as importance and B category as collapsing risk OBE (Operation Basis Earthquake) is 0.28 PGA but no less than

0.10g. Consequently, the dam was computed to the action of an earthquake with 0.10g maximum acceleration.

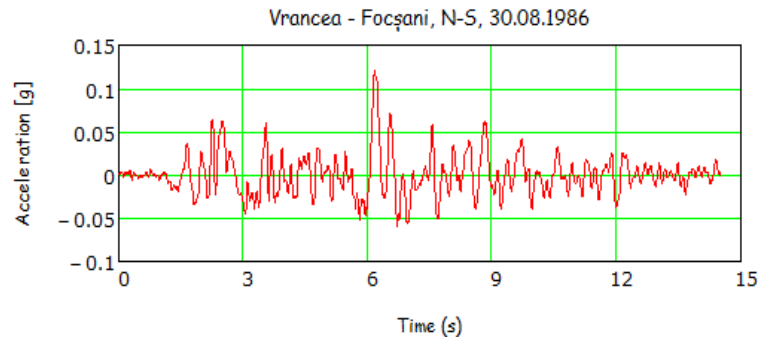


Fig. 3 - N-S component of recorded accelerogram at Focsani INCERC station during the 30th August 1986 Vrancea Earthquake

The seismic action consisting of N–S component of recorded accelerogram at Focsani INCERC station during the 30th August 1986 Vrancea Earthquake (Fig. 3) was applied at the bottom of the dam–foundation finite element mesh system, horizontally, upstream-downstream direction.

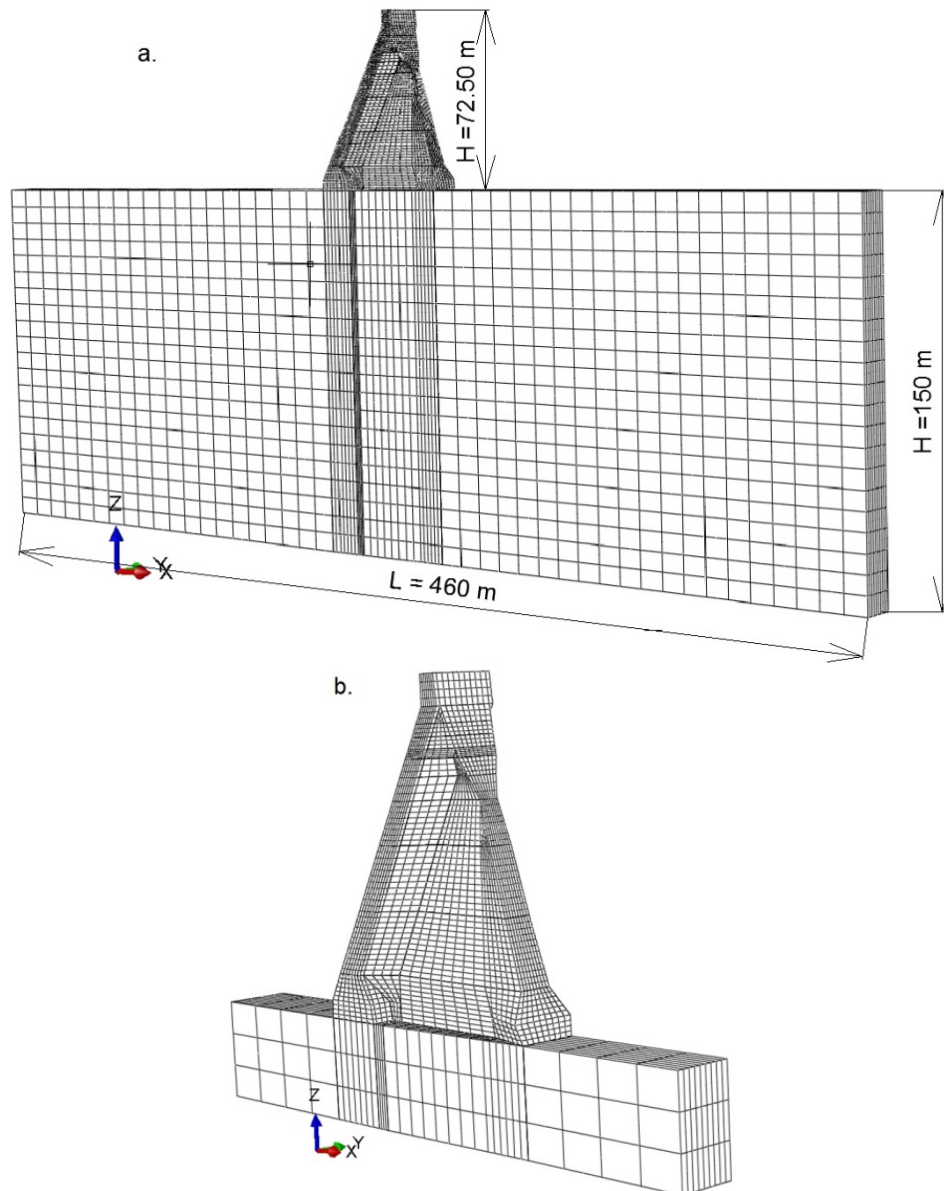


Fig. 4 - Finite element mesh of dam – foundation system: general (a) and dam detail (b)

The finite element mesh is illustrated in Figure 4. Quadratic hexagonal finite elements type C3D20R from ABAQUS software library were used to perform mesh. In Table 1 are given some data on the finite element mesh.

Table 1

Data on finite element mesh

Nodes – Elements Subsystem	Total number of nodes	Total number of finite elements
Dam	67118	12835
Foundation	73742	15260

C3D20R is a three-dimensional solid element with 20 nodes and 3 degrees of freedom in each node. It may be degenerated to an element with 9 nodes. This element is a general-purpose quadratic brick element, with reduced integration points (2 x 2 x 2 integration points). The reduced integration uses a lower-order integration to form the element stiffness but reduces running time, especially in three dimensions. The mass matrix and the distributed loadings use full integration.

The seismic response spectrum was computed from the accelerogram given in Figure 3, using a well-known relation with convolution integral:

$$|\delta(t)|_{max} = -\frac{1}{\omega} \left| \int_0^t \ddot{u}(\tau) e^{-v\omega(t-\tau)} \sin \omega(t-\tau) d\tau \right|_{max} \quad (1)$$

$$\text{Respectively} \quad S_d(v, \omega) = \frac{S_v(v, \omega)}{\omega} = \frac{S_a(v, \omega)}{\omega^2} \quad (2)$$

Where $|\delta(t)|_{max}$ is the maximum value (spectral) of the seismic response in displacements;

ω – circular eigenfrequency of the oscillator $\left(\frac{rad}{s}\right)$;

$\ddot{u}(\tau)$ – accelerogram of the earthquake;

v - oscillator fraction of the critical damping;

S_d, S_v, S_a – spectral values of the response in relative displacements, relative velocities and, absolute accelerations respectively.

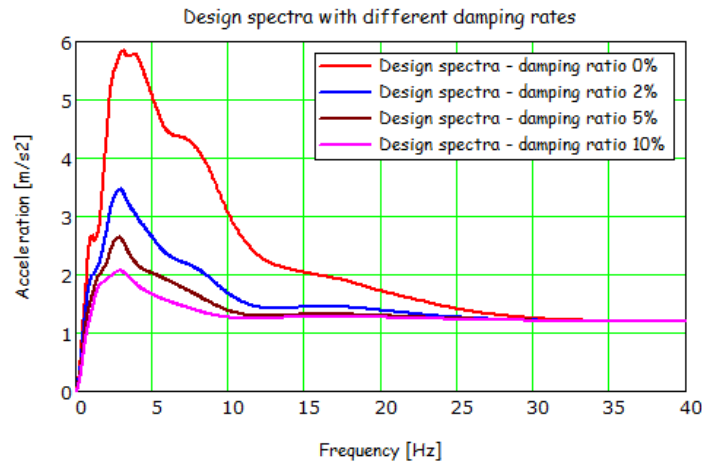


Fig. 5 - Design spectra computed from the NS component of the accelerogram presented in figure 3

Design spectra were obtained by smoothing the values from the seismic response spectra according to the rule of the least squares. The design spectra for v equal to 0.00, 0.02, 0.05, and 0.10 are presented in Figure 5. The seismic analysis of the dam was performed for $v = 0.05$. This value is recommended in literature, resulting from several in dam site dynamic investigations.

The material characteristics of the dam - foundation system are given in Table 2. The materials were considered homogeneous, isotropic and linear elastic behavior.

Material characteristics of the dam – foundation system

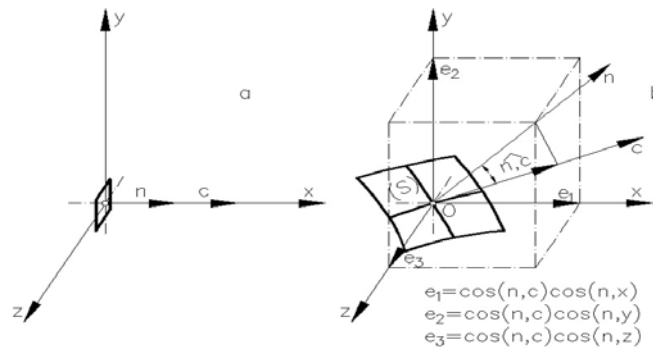
Properties \ Materials	Concrete	Rock foundation
Mass density (kg/m ³)	2400	-
Static Poisson coefficient	0.16	0.26
Dynamic Poisson coefficient	0.22	-
Static Young modulus (MPa)	24000	20000
Dynamic Young modulus (MPa)	32400	-

The effect of water in the seismic response was considered by the added mass procedure. The definition of the added mass $[M_h]$ acting on the normal direction in the point of application is as follows:

$$\{P_h(t)\} = - [M_h] \{\ddot{u} + \ddot{\delta}_r\}_n \quad (3)$$

where $\{P_h(t)\}$ is the hydrodynamic force in the reference point and $\{\ddot{u} + \ddot{\delta}_r\}_n$ is the total acceleration response to the normal direction at surface in the reference point.

This means that added mass as value corresponds with the hydrodynamic force generated by a unitary acceleration on the normal direction at surface in the reference point.

**Fig. 6 - Assessment of added masses**

In the general case when the directions of the normal to surface, of the earthquake and of the structure degrees of freedom are different, the added mass computed according to relation (3) must be projected successively on earthquake direction and on the structure degrees of freedom (Fig. 6). The dynamic equilibrium equations are written as follows:

$$[[M] + [M_h] [rcn] [rn, x, y, z]] \{\ddot{\delta}_r\} + [C] \{\dot{\delta}_r\} + [K] \{\delta_r\} = -[[M] + [M_h] [rcn] [rn, x, y, z]] \{r\} \ddot{u} \quad (4)$$

where $[rcn]$ has the dimensions equal with the number of the degrees of freedom of the system and contains on the diagonal the cosine directors between the normal to surface in the nodes of the mesh and earthquake direction;

$[rn, x, y, z]$ has dimensions corresponding to the number of the degrees of freedom of the system and contains on the diagonal the cosine directors between the normal to surface in the nodes of the mesh and directions of the system degrees of freedom.

The boundary conditions in the static and spectral analyses consisted in the displacement blockage in x, y, z directions for all nodes located on the lateral faces and the bottom of the foundation and in the y direction of the bank, bank lateral faces of the profile polygonal head.

In the direct time integration the accelerogram was applied at the bottom of the mesh on x direction. The displacements on y, z directions at the bottom of the mesh, y directions on the bank, bank lateral faces of the mesh (dam plate and foundation) and x, y, z directions on upstream, downstream lateral faces of the foundation respectively were blocked.

The time step (Δt) in direct time integration was constant and equal to 0.01 s. The response accelerations during a time step were computed using an implicit scheme by solving for dynamic response parameters at time $t + \Delta t$ based not only on values at t but also on these same quantities

at $t + \Delta t$. But because they are implicit, the nonlinear equations must be solved. The static loads and the seismic action were applied successively in distinct steps: dead load (step 1), hydrostatic pressure (step 2), uplift pressure under polygonal head (step 3) and seismic action (step 4).

In the direct time integration analysis, the damping matrix $[C]$ was evaluated as a function of the mass matrix $[M]$ and stiffness matrix $[K]$ in compliance with the linear Rayleigh relation:

$$[C] = \alpha[M] + \beta[K] \quad (5)$$

where α and β coefficients were evaluated for $\nu = 5\%$ in first two mode shapes in full reservoir hypothesis ($\alpha = 1.402$ and $\beta = 0.00158$).

4. Results in free vibration analysis

In Table 3 the first six circular eigenfrequencies (ω , rad/s) eigenfrequencies (f , Hz) and eigenperiods (T , s) of the dam profile are presented in both empty and full reservoir hypotheses.

Table 3

Free vibration characteristics

Mode number	Empty reservoir			Full reservoir (NRL level)		
	Eigenfrequency		Period	Eigenfrequency		Period
	ω (rad/s)	f (Hz)		ω (rad/s)	f (Hz)	
1	30.4	4.834	0.207	21.0	3.343	0.299
2	47.0	7.492	0.133	42.4	6.751	0.148
3	59.9	9.533	0.105	45.7	7.275	0.137
4	64.6	10.280	0.097	46.3	7.375	0.136
5	91.7	14.595	0.069	71.6	11.397	0.088
6	107.2	17.069	0.059	85.2	13.566	0.074

In Figure 7 are illustrated first six lowest mode shapes in full reservoir hypothesis.

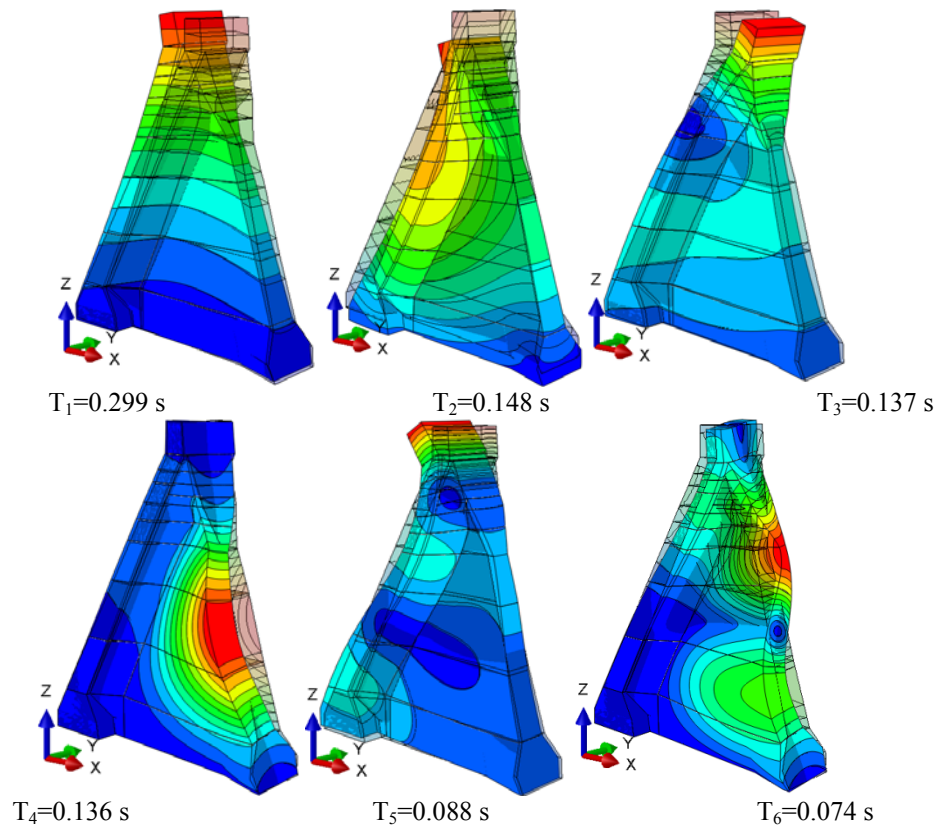


Fig. 7 - Gura Râului dam – first six lowest mode shapes in full reservoir hypothesis (reservoir level at NRL)

It should be noted that Gura Râului buttress dam is classified in the field of rigid structures. The fundamental dam mode is developed mainly on the horizontal upstream – downstream direction

(x axis). The second mode in the empty reservoir hypothesis and the fourth mode in the full reservoir hypothesis are developed horizontally along the dam crest (bank – bank) modeling the free vibration of the dam buttress as a plate (y direction). The reservoir effect led to the increase of the dam fundamental period with 44% compared to the empty reservoir (0.299 s versus 0.207 s), but the profile remains in the same domain of rigid structures.

The participation factors in six lowest mode shapes are given in Table 4. The participation factor for a mode k in direction i is a variable which indicates how strong the global x,y,z translation is about one of the three axes represented in the eigenvector of that mode. For instance, the fundamental mode is important on x axis (horizontal upstream – downstream direction)

Table 4

Eigenmodes and participation factor (empty and full reservoir)

Empty reservoir						Full reservoir					
Mode No.	Natural frequency	Eigenperiods	Participation factor			Mode No.	Natural frequency	Eigenperiods	Participation factor		
	[Hz]	[s]	x	y	z		[Hz]	[s]	x	y	z
1	4.834	0.207	2.15	0	-0.20	1	3.343	0.299	-7516	0	-719
2	7.492	0.133	0	2.05	0	2	6.751	0.148	1596	0	-8804
3	9.533	0.105	0.43	0	1.46	3	7.275	0.137	-4467	8	-2273
4	10.28	0.097	-1.67	0	1.39	4	7.375	0.136	-11	-3142	-7
5	14.60	0.069	0	-0.43	0	5	11.40	0.088	-2403	0	812
6	17.07	0.059	0	-0.93	0	6	13.57	0.074	1	-1040	1

5. Analysis Results by response spectrum method

Response spectrum analysis provides an inexpensive approach to estimating the peak response of a structure subjected to base motion: the simultaneous motion of all nodes fixed with boundary conditions.

The maximum dam displacements and stresses $|R_i|_{\max}$ were computed from “m” (m=25) representative peak modal responses using the square root of the sum of the squares (SRSS):

$$|R_i|_{\max} = (\sum_{k=1}^m |(R_i)_k^{\max}|^2)^{1/2} \quad (6)$$

where index i is referring to the degree of freedom and k to the eigenmode.

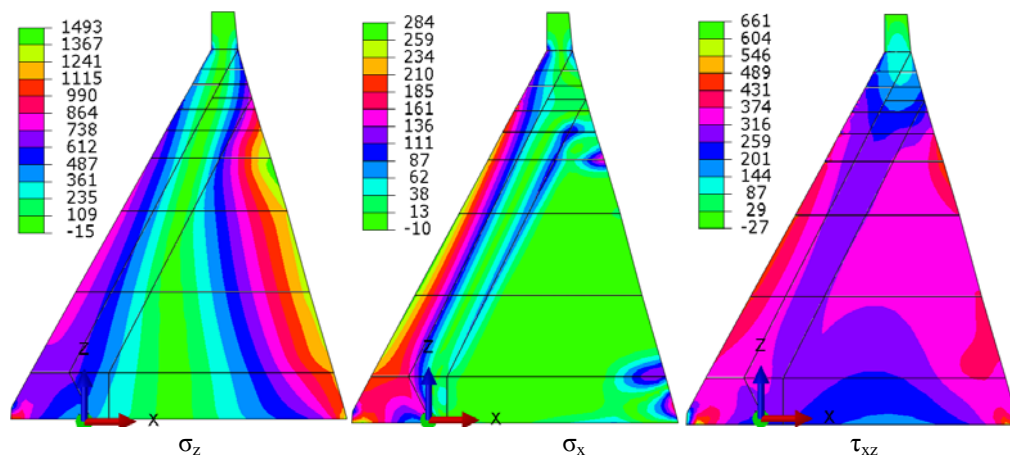


Fig. 8 - σ_z , σ_x and τ_{xz} contour lines in the central section of the profile resulted in spectral analysis, full reservoir (kPa)

In Figure 8 σ_z , σ_x and τ_{xz} contour lines are illustrated in kPa in the central section of the profile resulted in the spectral analysis, full reservoir hypothesis. The maximum vertical stress (σ_z) reaches maximum value of 1277 kPa at downstream face. The maximum shear stress reaches maximum value of 489 kPa at downstream dam heel. The alternative maximum displacement on upstream–downstream horizontal direction computed by the spectral analysis reaches ± 9 mm at crest. On the vertical direction it reaches ± 2 mm.

The variations in elevation of σ_z and τ_{xz} resulted in the spectral analysis, full reservoir case, at upstream and downstream faces of the profile are illustrated in Figure 9.

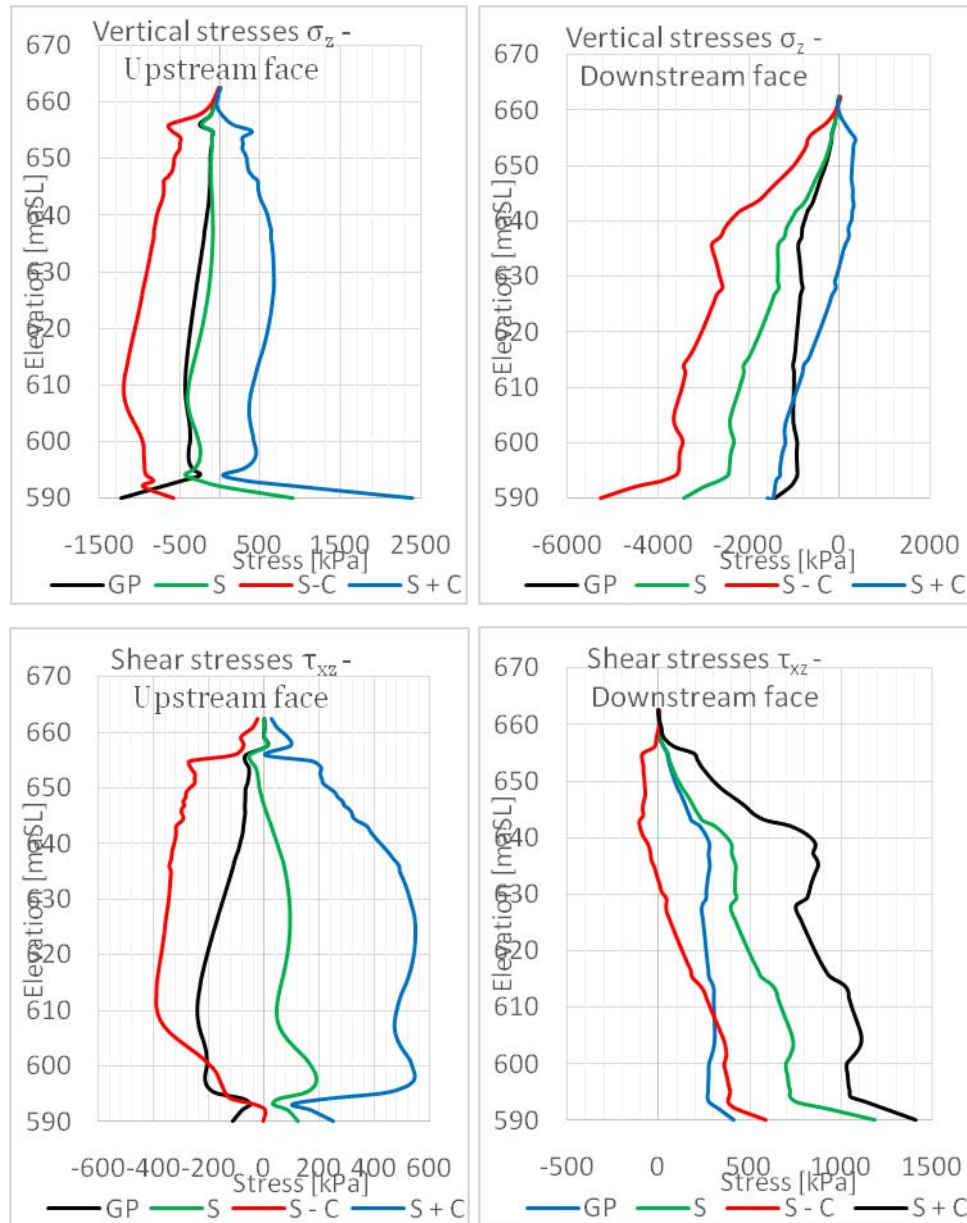


Fig. 9 - Variation of σ_z and τ_{xz} along the upstream and downstream faces of the dam profile, full reservoir case: GP – dead weight, S–dead weight + hydrostatic pressure including uplift, ± C–spectral analysis 0.1 g

It can be remarked that vertical stresses σ_z due to load combination dead weight + hydrostatic pressure including uplift \pm spectral stresses 0.1 g vary between (-1000...+600) kPa at upstream face and (-3900...+300) kPa at downstream face (+ tensile stress).

Shear stresses τ_{xz} directed from downstream to upstream in the dam body at the same load combination reach 500 kPa at upstream face and 1100 kPa at downstream face.

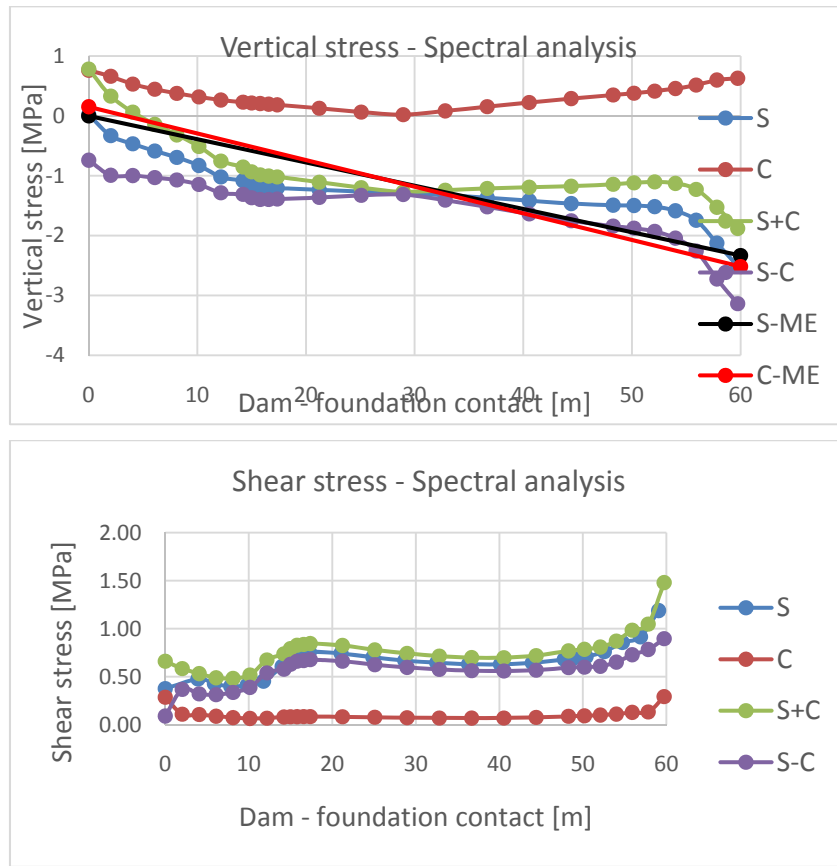


Fig. 10 - σ_z and τ_{xz} diagrams on dam – foundation contact computed in spectral analysis in different hypotheses in full reservoir case (S – dead weight + hydrostatic pressure + uplift pressure, C – spectral analysis, ME – elementary method computing stresses + inertia forces computed in compliance with spectral analysis)

Some representative diagram of σ_z and τ_{xz} at different hypothesis on dam – foundation contact in full reservoir case are given in Figure 10. The meaning of notations S, C, was already given, ME corresponds to elementary method computing stresses + inertia forces computed in compliance with spectral analysis. A satisfactory correlation should be noted between σ_z diagram at S-C combination obtained in the spectral analysis and the one obtained by the elementary method.

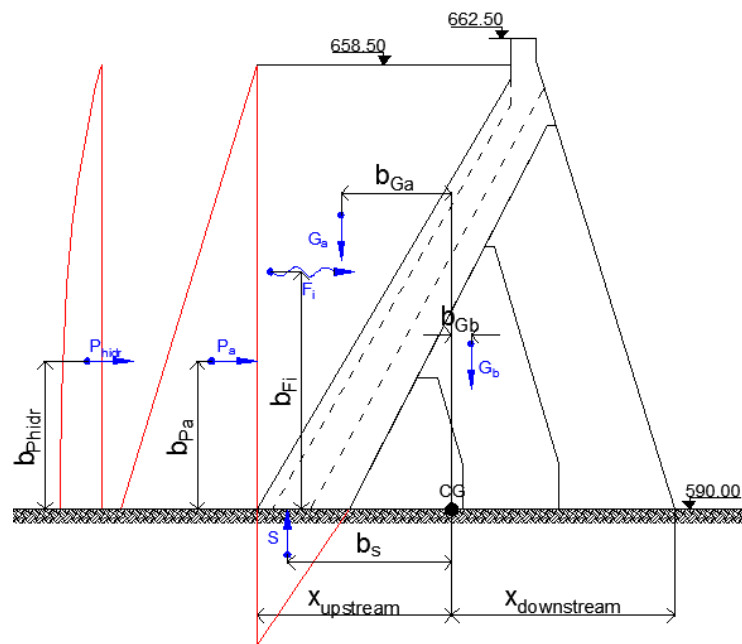


Fig. 11 - Scheme of loads taken into account in the analysis based on strength materials theory (elementary method computing stresses + inertia forces computed in compliance with spectral analysis)

The loads taken into account in the elementary method based on strength materials theory (eccentric compression calculus) are presented in Figure 11.

The stress state is in allowable limit, taking into consideration the dynamic character of the seismic stresses.

6. Analysis results by direct time integration

The main hypotheses accepted in the direct time integration method were already presented at point 3.

Some results obtained in this analysis are illustrated in Figures 12 and 13. The component of the baseline corrected accelerogram having a PGA of 0.1g was applied at the finite element mesh bottom on the horizontal upstream - downstream direction.

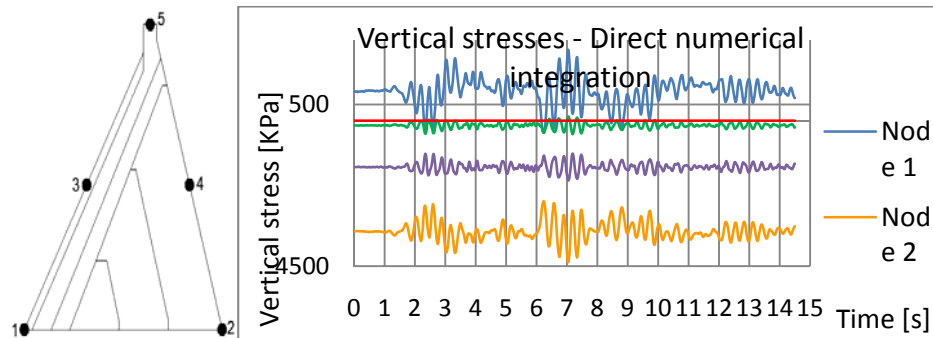


Fig. 12 - σ_z response time history computed in Direct Time Integration Method due to load's combination of dead weight+hydrostatic pressure+uplift pressure+accelerogram 0.1g

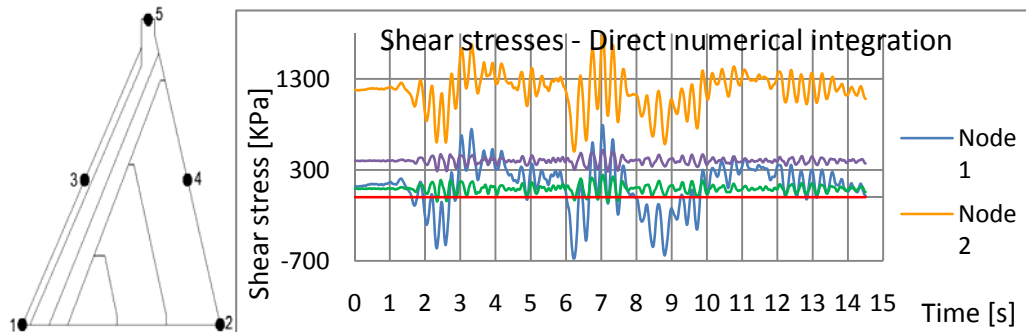
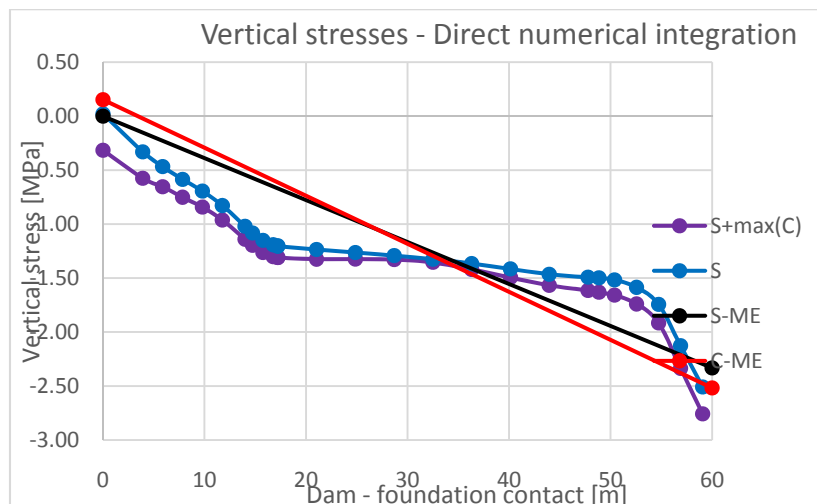


Fig. 13 - τ_{xz} response time history computed in the Direct Time Integration Method due to the load combination of dead weight+ hydrostatic pressure+uplift pressure+accelerogram 0.1g



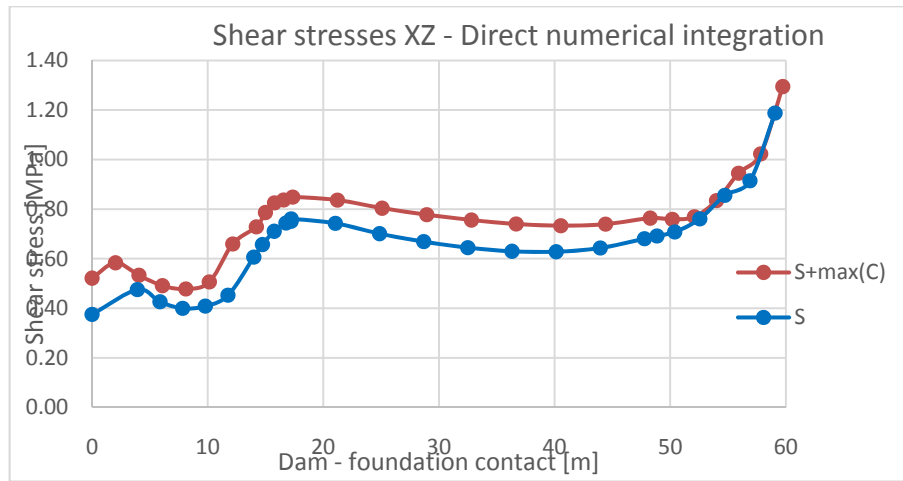


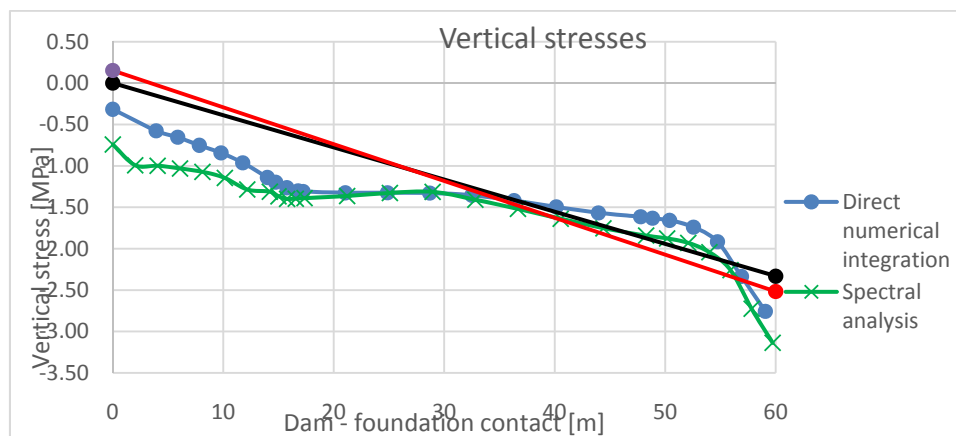
Fig. 14 - σ_z and τ_{xz} diagrams on dam – foundation contact computed in direct time integration at combination of dead weight+hydrostatic pressure+uplift pressure+accelerogram 0.1g compared with σ_z diagram computed by elementary method

In Figures 12 and 13 are presented the response time history in σ_z (vertical stress) and τ_{xz} (shear stress) in 5 points located in the profile central section due to dead load + hydrostatic pressure including uplift + accelerogram 0.1g maximum acceleration. The maximum σ_z reaches 378 kPa tension (node 1, upstream toe) and -2766 kPa compression (node 2, downstream toe). The maximum τ_{xz} directed to upstream reaches 1275 kPa (node 2, downstream toe).

The σ_z and τ_{xz} diagrams on the dam foundation contact computed in direct time integration at the combination of dead weight + hydrostatic pressure + uplift pressure + accelerogram 0.1g compared with the σ_z diagram computed by the elementary method computing stresses + inertia forces computed in compliance with the spectral analysis are given in Figure 14. The corresponding results obtained by both methods are in satisfactory correlations. Some tensile stresses reaching about 400 kPa resulted in the elementary method at upstream dam toe versus 378 kPa in the direct time integration method. Instead at the downstream dam toe the compression computed in the direct time integration method is higher than in the elementary method (-2766 kPa versus -2500 kPa).

7. Comparison of results and concluding remarks

In order to compare the results of the analyses carried out by the response spectrum method and the direct time integration method some representative results are given in Tables 5 and 6, respectively in Figure. 15.



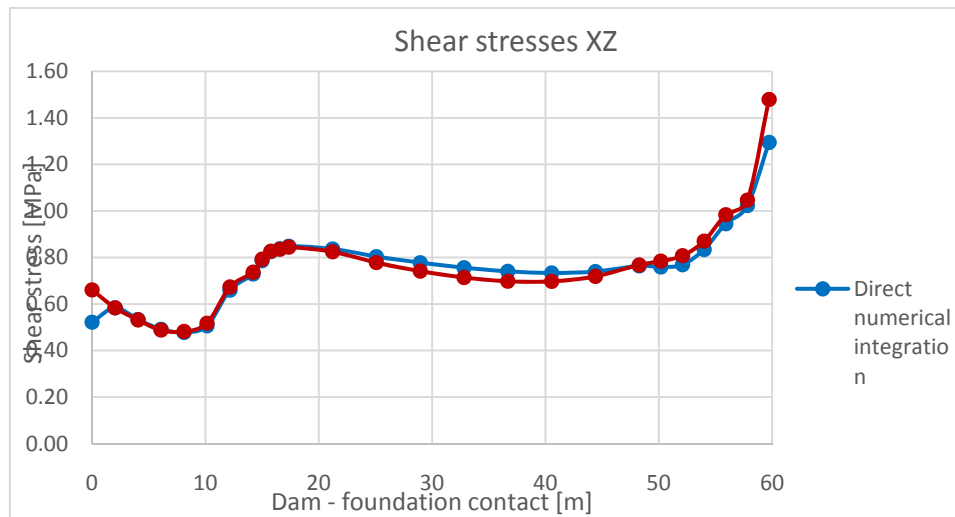


Fig. 15 - Comparative diagrams of σ_z and τ_{xz} on dam foundation contact, full reservoir case, computed by spectral analysis, direct time integration and elementary method.

Table 5

Vertical stress - σ_z

Node		1	2	3	4	5
Method	min	-316	-2766	-414	-1851	0
	max	378	-1865	129	-1021	1
Spectral analysis	min	-741	-3138	-1054	-3239	-10
	max	782	-1880	606	-614	15

Table 6

Shear stress - τ_{xz}

Node		1	2	3	4	5
Method	min	283	1088	-54	284	0
	max	461	1275	245	517	0
Spectral analysis	min	90	896	-373	174	-25
	max	661	1479	542	912	2

According to Table 5 the maximum σ_z compression from dead weight + full reservoir + 0.1g horizontal earthquake upstream – downstream combination is at the downstream dam toe (node 2) reaching -2766 kPa in the direct time integration and -3138 kPa in the spectral analysis (13% difference).

The maximum σ_z tension from the same load combination is at the upstream dam toe (node 1) reaching 378 kPa in the direct time integration and 782 kPa in the spectral analysis (106 % difference). The high difference between the local values may be explained through their small order of magnitude relative to other values that influenced the accuracy of the numerical computation.

According to Table 6 the maximum shear stress τ_{xz} appears at the downstream dam toe (node 2) reaching 1275 kPa in the direct time integration and 1479 kPa in the spectral analysis (16% difference).

Additionally from Figure 15 one can notice a good correspondence between σ_z and τ_{xz} stresses computed with the direct time integration and the spectral analysis methods along the dam – foundation contact. The values computed by the elementary method are also in satisfactory correlations with their corresponding values obtained in the dynamic analyses.

The comparison between the methods was extended also to the values of the sliding safety coefficient (k) on the dam – foundation contact in the full reservoir hypothesis considering the response to the earthquake action by the spectral method and, by the direct time integration respectively.

In comparison the k value resulted by the elementary method of calculus was also considered, in compliance with the relation:

$$k = f \frac{\iint_{FA} \sigma_v(x,y) dx dy}{\iint_{FA} \tau_{xz}(x,y) dx dy} = f \frac{\Sigma V}{\Sigma H} \quad (7)$$

where FA is the dam – foundation area;

σ_z - vertical stresses on the dam – foundation area calculated with the elementary method

τ_{xz} – shear stresses on the dam – foundation area calculated with the elementary method

ΣV - sum of the vertical loads acting on dam block

ΣH - sum of the horizontal loads acting on dam block

$f = 0.70$ friction coefficient to sliding on the dam – foundation contact.

The comparison results are summarized in Table 7.

Table 7

Comparison between the methods on the values of the sliding safety coefficient (k)

Methods and types of analyses	ΣV kN	ΣH kN	k -
Full reservoir – foundation on stresses evaluated by FEM $a_c = 0$	567010	307030	1.292
Full reservoir – foundation on stresses evaluated by elementary method $a_c = 0$	548490	345110	1.113
Direct time integration $a_c = 0.1g$	500610	353260	0.991
Spectral analysis $a_c = 0.1g$	430400	338900	0.889
Inertia forces evaluated by spectral analysis. Foundation stresses evaluated by elementary method $a_c = 0.1g$	548490	379930	1.010

According to the results presented in Table 6 the spectral analysis method is conservative compared with the direct time integration methods concerning the dam sliding stability on the foundation.

Based on the data presented above the followings concluding remarks may be formulated:

- The aim of the paper was to check the accuracy of the results in the dam seismic response provided by the spectral analysis taking into account some doubt on the performance of this method;
- The results in the seismic response of a buttress dam with 73.50 m maximum height computed by the spectral analysis method were compared with those obtained by the direct time integration. In the comparison corresponding results obtained by the elementary method of analysis were also included (linear distribution of normal stresses). The comparison included the stresses in the representative points of the dam body, dam foundation area and the safety coefficient to sliding on the dam – foundation contact.

- The conclusion is that both the spectral analysis method and the direct time integration method provided close results, acceptable for engineering use. Generally, the differences between the corresponding values are in the range 0...20%, but locally they can reach even 100% when their order of magnitude is very low relatively to the current values. This may be explained by the accuracy of the numerical computation. As it was expected, the results provided by the spectral analysis method are the conservative ones.

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