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EARTHQUAKE RESPONSE ANALYSIS OF A CONCRETE BARRAGE IN INTERACTION WITH THE SOIL

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ABSTRACT

The paper deals with the dynamic analysis of the geometrical non-linear response of the concrete dam and hydroelectric power station in interaction with the foundation soil, under the seismic conventional loads.

Structure has a complex response under external loading, both because of physical specifically behaviour and the change of its geometric shape. Here is presented a model for non-cracked concrete using the theory of plasticity, based on the strength theories and a model for cracked concrete using the theory of fracture mechanics.

The dam and the power station have been modelled with concrete elements using the von Mises theory and soil elements using Drucker - Prager theory.

The stresses and deformations were determined respecting the following hypothesis: - the gravity forces; - the gravity forces together with the static and hydrodynamic pressure of the water; - the previous hypothesis together with the seismic forces established respecting the Romanian standard for the seismic calculus. A rule of variation of the seismic forces in keeping with the fifth mode shapes of the structure was recomputed at each loading step and each iteration of the equilibrium.

It was established: - frequencies and modes shapes of structure; - stresses and maximal stresses; - deformation shapes and maximal deflection.

KEYWORDS

Earthquake; response, concrete; dam; interaction; soil.

STRUCTURE OF AN OVERFALL-DAM, IN INTERACTION WITH FOUNDATION SOIL

The dam is located in the middle of Danube minor bed of river and assures regulation of water levels in upstream, evacuation of ice and floats and together with the hydroelectric dam station and navigation locks is very important in discharging of floods. This case is a gravity dam made of concrete, having a maximal elevation of 60.60 m, a length of 441 m and is composed of 14 overflow fields, with 25 m openings, separated through piers of 7 m gauge. Openings are equipped with plane gates with clasp of 25.00 x 14.86

m^2, which can run between guide-ways. The niches have dimensions of 3.8 x 2.00 m and draw hydrostatic pressure off. The hydrostatic pressure represents 27,000 KN/pier. The dam has an upstream rigid fixed corbel, 5 m under the foundation footing. Transversal dam section is approximately triangle, with the proportion between base and elevation almost equal to one, corresponding to geotechnical conditions from location. Rocks of foundation in bed of river and shores belong to a crystalline, composed of paragnaises with lateral passes to quartz varieties more or less shistuosities. Friction coefficients between concrete and rock which have been accepted are around 0.5 - 0.6. To reduce uplifts, on foundation surface has been realised a watertight diaphragm, soil logs and drainage galleries. The body of the dam is divided in 16 m width blocks through constructions joints which are situated in overflow fields and are waterlighted with plastic sealing bands.

APPLICATION OF F.E.M. - FINITE ELEMENT METHOD IN DYNAMIC ANALYSIS

In stress state evaluation problem, inside the body of dam, F.E.M. has been applied owing to higher accuracy possibility given by dynamic analysis, taking into account a large effects number and hypothesis regarding both loading and material behaviour and interaction phenomenon. It has been established stress and strain state inside the body of dam and foundation soil using a transient non-linear numerical analysis, step by step, with F.E.M. - owing to the weight of the dam and the pressure of the water from the lake. In this purpose, the whole ensemble dam - soil has been meshed in isoparametric, patrulater finite elements, with four integration Gauss points, which reproduces a plane strain state around a point. The vertical and horizontal reinforcements have been concentrated on the plane finite elements contours and connected with their nodes. The reinforcements unidimensional TRUSS finite elements have unidimensional rigidity only. It have been used 633 concrete finite elements and respectively 405 soil finite elements - fig.1. Hydroelectric power station and dam have been modelled with elements in yielding von Mises theory, with hardening, and soil with non-linear Drucker - Prager's theory. The stiffness matrix of the structure has been recalculated to each loading step and respectively to each equilibrium iteration step.

Shape particularities, material types which have been used in different pours and border conditions have been taken into account to the mesh discretization. Depending on the significant stresses values inside foundation soil, it has been chosen, through trials, the mesh discretization limit inside depth of soil, to reproduce the half-plane conditions and the final result of these was the mesh discretization density. Extension of analysed area was 1.5 times greater than the elevation of the dam.

Taking into account that proportion between Young concrete and Young foundation modulus is not greater than 50, condition of null displacement to soil mesh limit didn't produced any numerical error which could affect solution. Both geometrical nonliniarities effects (displacements and great rotations with moderate strains) and physical nonliniarities, in T.L. - Total Lagrangean formulation, have been considered. In consequence, stresses and strains have been formulated using the II order Piola - Kirchhoff - PK2 stresses tensor, and respectively the Green - Lagrange - GL strains tensor (Bathe and Ramaswamy, 1979).

The analysis has been realised using a specialised original ANSEF software, which offers important hardware facilities in the WINDOWS version, having a double precision, 30,000 words common blank. This last one creates a task which will occupy 3.5 Mb from the memory of a PC 486 - DX/33 MHz during the run time, enough for a "step by step" non-linear analysis. The response of the system has been calculated using an incremental static equilibrium equations solution with Newmark integration scheme. The equilibrium equations from the displacements method has been solved taking into account the stiffness decreasing both to each loading step during one step iterations (Ieremia, 1991).

GENERAL EQUATIONS GOVERNING THE DYNAMIC PHENOMENON

In calculation of seismic response of "dam - soil" structure, having "n" DOF, modal superposition method has been applied, expressing static equilibrium equations through a "n" independent equations system where dynamic characteristics corresponding to each vibration mode can be found. Knowing all these, dynamic general equilibrium equations, expressed in modal co-ordinates (noncupled), becomes:

$$\left[\widetilde{M}\right]\{\ddot{\eta}(t)\}+\left[\widetilde{C}\right]\{\dot{\eta}(t)\}+\left[\widetilde{K}\right]\{\eta(t)\}=\left\{\widetilde{P}\right\}\ddot{\delta}_s \tag{1}$$

where: $\left[\widetilde{M}\right]=[\Phi]^T[M][\Phi]$; $\left[\widetilde{C}\right]=[\Phi]^T[C][\Phi]$; $\left[\widetilde{K}\right]=[\Phi]^T[K][\Phi]$; $\{\delta(t)\}=[\Phi]\{\eta(t)\}$; $\left\{\widetilde{P}\right\}=[\Phi]^T[M]\{1\}$;

[M] - concentrated masses matrix; [C] - viscous damping; [K] - elastic stiffness matrix; $\{\delta(t)\}$ - generalised displacements vector; $[\Phi]$ - modal matrix constituted of eigen vectors corresponding to each eigenvalues, $\ddot{\delta}_s$ - earthquake excitation at base. Masses and stiffness matrix are diagonal matrix, and damping matrix has been expressed as a proportional damping, on Rayleigh type - $[C]=\widetilde{\alpha}[M]+\widetilde{\beta}[K]$. $\{\widetilde{P}\}$ is vector of modal participation factor, characterising contribution of each vibration eigen mode to structure dynamic response. Validation condition of this dynamic analysis was that mobilised mass represents over 90% from total mass. When mesh discretization gauge has been established on analysis of eingenvalues spectra from different mesh levels has been taken into account (Ieremia, 1993).

ELASTO-PLASTIC BEHAVIOUR MODELS OF MATERIALS

Elasto-plastic model which have been used for describing the concrete and reinforcement behaviour, uses energetic breaking M. Huber criterion and respectively von Mises plasticity theory in concordance with Prandtl's "stress - strain" technical characteristic diagram, with hardening (Owen and Hinton, 1980).

In the non-linear numerical analysis using the Finite Element Method, it will be defined the yielding criterion at a "t" time moment (to a certain loading level), as an extended expression of the Coulomb condition.

$${}^{t}F=3\alpha\,{}^{t}\sigma_0+{}^{t}S-\sigma_y=3\alpha\,{}^{t}\sigma_0+\frac{1}{\sqrt{2}}\,{}^{t}s_{ij}-k=0, \qquad i,j=1,2,3 \tag{2}$$

where, σ_0 - mean stress; $\sigma_0=\frac{1}{3}\sigma_{ii}=\frac{1}{3}I_1$; S - intensity of tangential stresses; $S=(\frac{1}{2}s_{ij}s_{ij})^{\frac{1}{2}}=(-J_2)^{\frac{1}{2}}$; σ_y - material yielding strength; α and k - are specific parameters of the material; J_2 - the second invariant of deviatoric stress tensor $[T_\sigma^d]=(s_{ij})$, which characterises the shape of an infinitesimal element around a point.

$$J_2=-\frac{1}{6}\left[(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2+(\sigma_3-\sigma_1)^2\right]=\frac{1}{2}(s_{ij})(s_{ij}) \tag{3}$$

For the evaluation of the dynamic response of the system structure - soil, considering the non-linear properties of the soil, the Drucker - Prager criterion is obviously used. This criterion represents both an extension of the crack theory (maximal shear stress) of Coulomb and an approximation of the Mohr - Coulomb criterion; in this case, the yield condition of von Mises is completed with an additional member, representing the influence of the spherical stress tensor.

The yield criterion at the moment "t" has the following form (Ieremia, 1994):

$$F=(3\alpha\sigma_8+S)-\sigma_y=0 \tag{4}$$

where: σ_8 - is the normal octahedral stress; α, σ_y - are the material coefficients, depending on the soil cohesion (c) and on the internal friction angle (θ), as follows:

$$\sigma_8=\frac{1}{3}(\delta_{ij})(\sigma_{ij})$$

$$\sigma_c=\frac{6c\cdot\cos\theta}{\sqrt{3}(3-\sin\theta)} \tag{5}$$

$$\alpha=\frac{2\sin\theta}{\sqrt{3}(3-\sin\theta)}$$

The stiffness matrix for the post-elastic computation, depends on the deviatoric tensor.

$$[D_{e\text{-}p}]=\begin{bmatrix} K+\frac{4}{3}G-(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{11}+\beta_2) & K-\frac{2}{3}G-(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{22}+\beta_2) & -(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{12}) & K-\frac{2}{3}G-(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{33}+\beta_2) \\ K-\frac{2}{3}G-(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{22}+\beta_2) & K+\frac{4}{3}G-(\beta_1{}^t s_{22}+\beta_2)(\beta_1{}^t s_{22}+\beta_2) & -(\beta_1{}^t s_{22}+\beta_2)(\beta_1{}^t s_{12}) & K-\frac{2}{3}G-(\beta_1{}^t s_{22}+\beta_2)(\beta_1{}^t s_{33}+\beta_2) \\ -(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{12}) & -(\beta_1{}^t s_{22}+\beta_2)(\beta_1{}^t s_{12}) & G-(\beta_1{}^t s_{12})(\beta_1{}^t s_{12}) & -(\beta_1{}^t s_{12})(\beta_1{}^t s_{33}+\beta_2) \\ K-\frac{2}{3}G-(\beta_1{}^t s_{11}+\beta_2)(\beta_1{}^t s_{33}+\beta_2) & K-\frac{2}{3}G-(\beta_1{}^t s_{22}+\beta_2)(\beta_1{}^t s_{33}+\beta_2) & -(\beta_1{}^t s_{12})(\beta_1{}^t s_{33}+\beta_2) & K+\frac{4}{3}G-(\beta_1{}^t s_{33}+\beta_2)(\beta_1{}^t s_{33}+\beta_2) \end{bmatrix} \tag{6}$$

The coefficients β in the previous formulas are:

$$\beta_1=\frac{G}{{}^t S\left(G+9K\alpha^2\right)^{1/2}} \qquad \beta_2=\frac{3K\alpha}{\left(G+9K\alpha^2\right)^{1/2}} \tag{7}$$

where K represents the bulk modulus and G is the shear modulus.

The stress and deformation state was analysed with numerical Finite Element Method inside a concrete dam in interaction with soil, under seismic conventional loads, in concordance with design spectra.

GEOMETRICAL NON LINEARITY WITH T.L. - TOTAL LAGRANGEAN FORMULATION

Incremental equilibrium equation at "t+Δt" time moment, related using the total virtual mechanical work principle, with real forces in dynamic equilibrium and virtual displacements (Bathe, 1982), is:

$$\int_{(V)^0} {}^{t+\Delta t}_{0}S_{ij}\,\delta\,{}^{t+\Delta t}_{0}\varepsilon^{*}_{ij}\,{}^{0}dV={}^{t+\Delta t}L \tag{8}$$

where ${}^{t+\Delta t}_{0}S_{ij}$ - PK2 stresses tensor components, in configuration of the body at time "t+Δt", related to the body configuration at the initial time t=0;

$${}^{t+\Delta t}_{0}S_{ij}=\frac{{}^{0}S}{{}^{t+\Delta t}S}\,{}^{0}_{t+\Delta t}x_{i,s}\,{}^{t+\Delta t}\sigma_{s,r}\,{}^{0}_{t+\Delta t}x_{j,r} \tag{9}$$

$x_{i,s}$ - (i,s) element of deformation gradient;

$${}^{0}_{t+\Delta t}x_{i,s}=\frac{\partial^{0}x_i}{\partial^{t+\Delta t}X_s}=\left({}_{t+\Delta t}\nabla^{0}X^{T}\right)^{T} \tag{10}$$

${}^{t+\Delta t}_{0}\varepsilon^{*}_{ij}$ - GL strains tensor components in configuration at "t+Δt" time, related to the body shape at initial time moment, t=0;

$${}^{t+\Delta t}_{0}\varepsilon^{*}_{ij}=\frac{1}{2}\left({}^{t+\Delta t}_{0}u_{i,j}+{}^{t+\Delta t}_{0}u_{j,i}+{}^{t+\Delta t}_{0}u_{k,i}\,{}^{t+\Delta t}_{0}u_{k,j}\right) \tag{11}$$

where ${}^{t+\Delta t}_{0}u_{i,j}=\dfrac{\partial^{t+\Delta t}u_i}{\partial^{0}X_j}$.

${}^{t+\Delta t}L$ - total virtual work of external and internal (gravitational) forces, when body is submitted to a displacements variation at "t+Δt" time moment.

Three in-plane PK2 stresses contribute to the strain energy and four displacement gradients appear in the corresponding GL strains. The four displacement gradients are arranged as:

$$\{g\}^{T}=\left\langle \frac{\partial u_x}{\partial X}\ \ \frac{\partial u_y}{\partial X}\ \ \frac{\partial u_x}{\partial Y}\ \ \frac{\partial u_y}{\partial Y}\right\rangle \tag{12}$$

The strain measures chosen are the three components of the GL strains - $\left[T^{*}_{\varepsilon}\right]=\left(\varepsilon^{*}_{ij}\right);\ i,j=x,y$.

$$\varepsilon_x^* = \begin{Bmatrix}1\\0\\0\\0\end{Bmatrix}^T \{g\} + \frac{1}{2}\{g\}^T \begin{bmatrix}1&0&0&0\\0&1&0&0\\0&0&0&0\\0&0&0&0\end{bmatrix}\{g\}$$

$$\varepsilon_y^* = \begin{Bmatrix}0\\0\\0\\1\end{Bmatrix}^T \{g\} + \frac{1}{2}\{g\}^T \begin{bmatrix}0&0&0&0\\0&0&0&0\\0&0&1&0\\0&0&0&1\end{bmatrix}\{g\} \qquad (13)$$

$$\gamma_{xy}^* = \begin{Bmatrix}0\\1\\1\\0\end{Bmatrix}^T \{g\} + \frac{1}{2}\{g\}^T \begin{bmatrix}0&0&1&0\\0&0&0&1\\1&0&0&0\\0&1&0&0\end{bmatrix}\{g\}$$

For the incremental solution of equilibrium equation at "t+Δt" time moment, movement general equation around the state from "t" time will be linearized and using the generalised Newton - Raphson iteration, it will be obtained the equilibrium equation written under incremental shape for a finite element in deformed state, with implicit Gauss integration time.

RESULTS

It have been established the stress and strain state which appear inside the structure of the dam and of the hydroelectric dam station in interaction with foundation soil, which represents the result of an excitation at base produced by a vrancean earthquake like that from 4 March 1977, it have been taken into account as loading hypothesis the own weight of the structure together with static and dynamic water pressures. Seismic calculation forces has been considered in conformity with Romanian seismic calculation Normative P100-92, after a variation rule in concordance with first five vibration modes of the structure.

It was established:
- deformation shapes and maximal deflection - fig.2;
- frequencies and modes shapes of structure - fig.3;
- stresses and maximal stresses.

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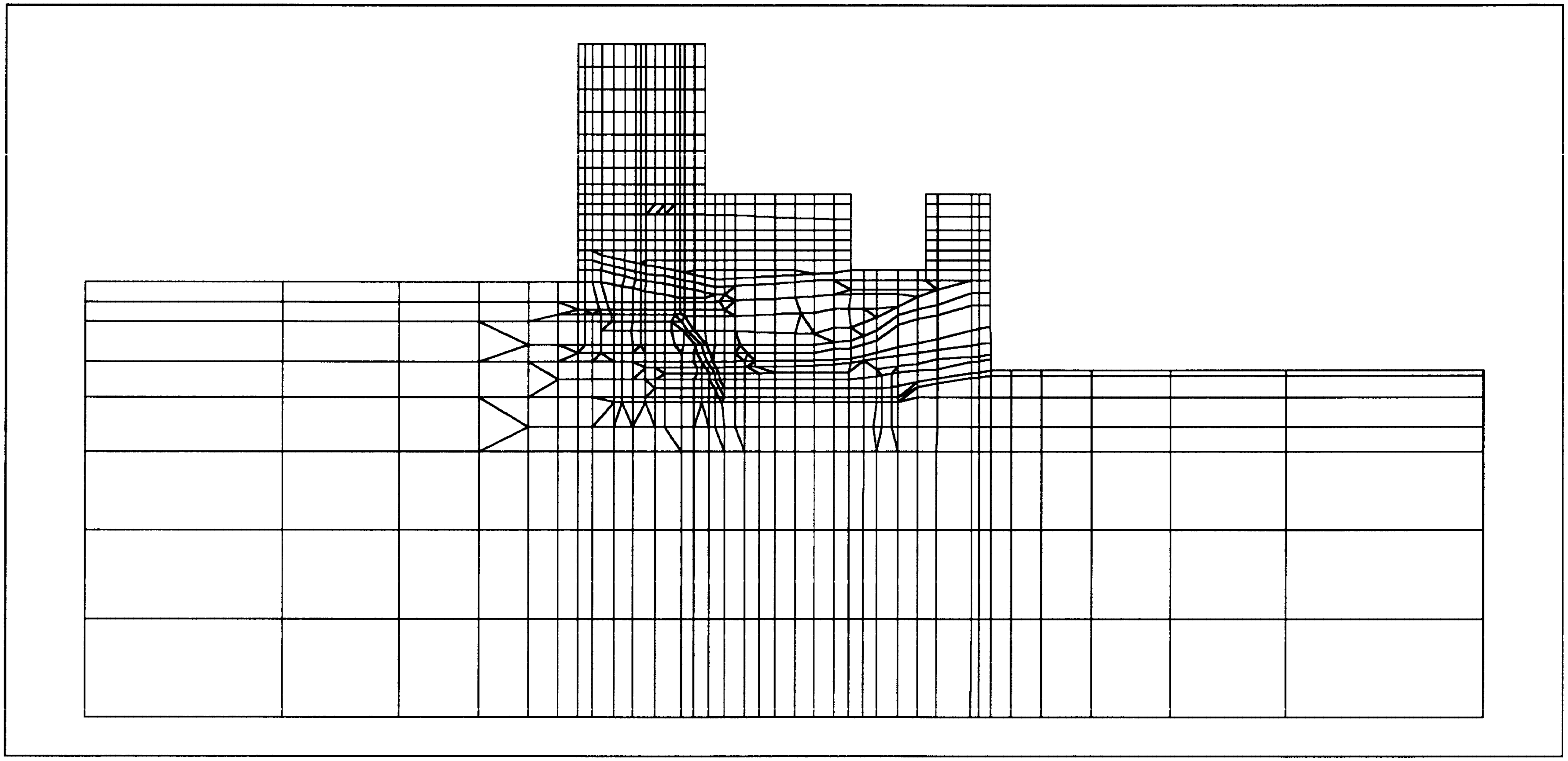

Fig.1 Finite element mesh
concrete : von Mises; soil Drucker-Prager
1038 finite elements; 1060 nodal points

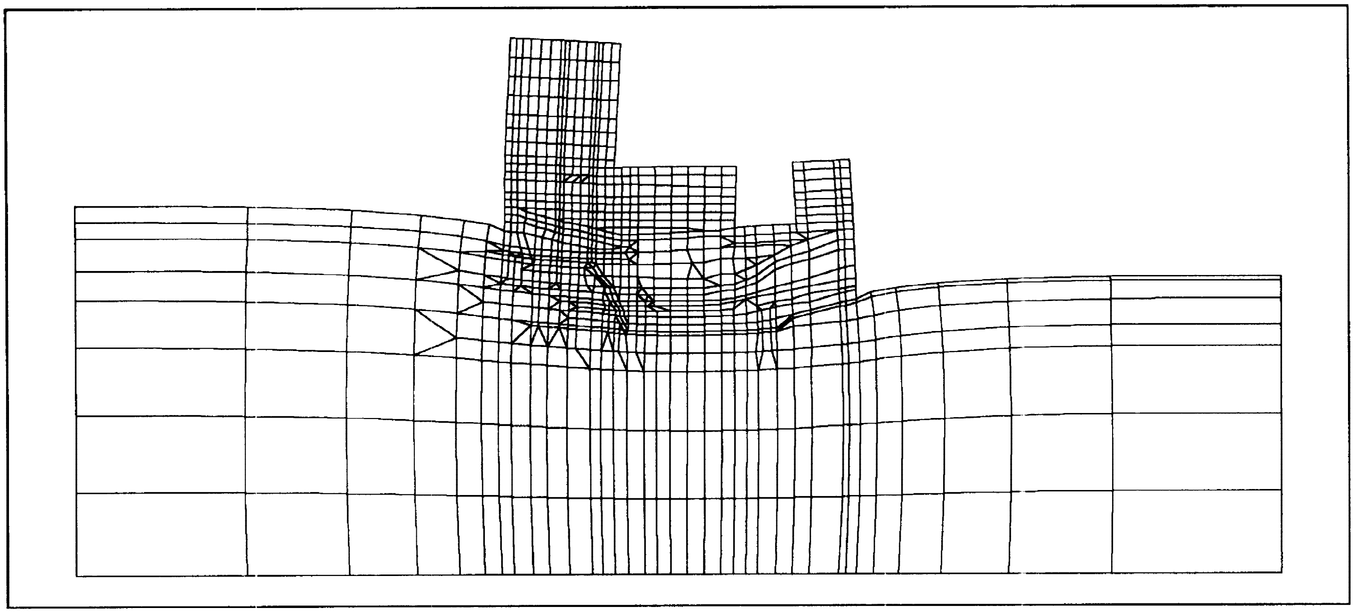

Fig.2a Strained Structure - loads:weight

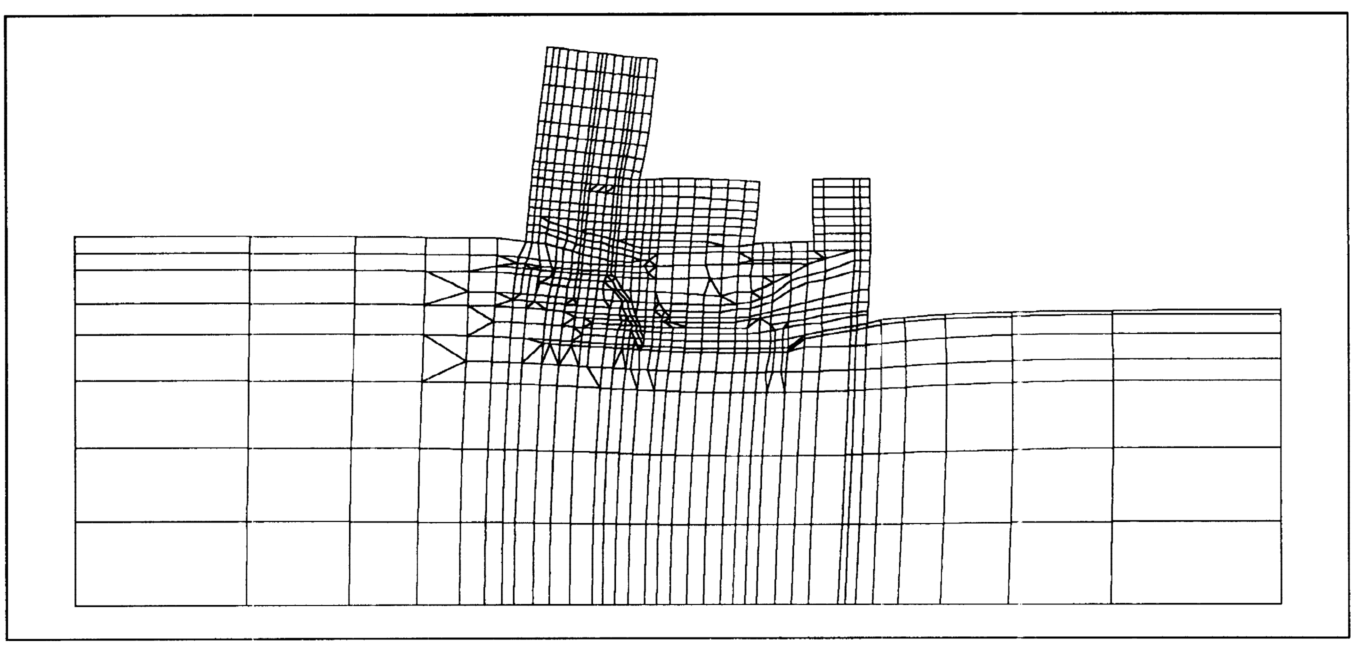

Fig.2b Strained Structure - loads : weight + water pressure

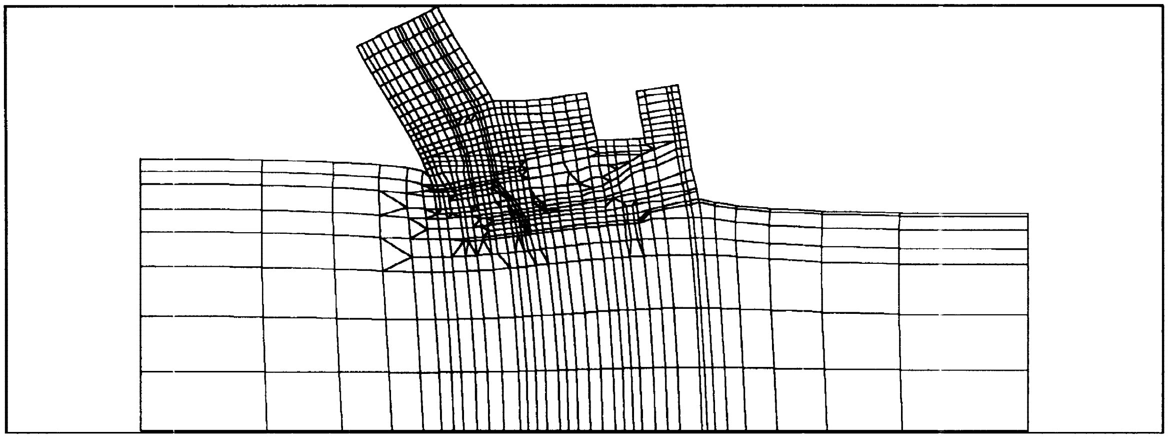

Fig.3a Frequencies and Mode Shapes no.1
f_1= 1.2 Hz ; ω_1=7.55 rad/s ; T_1 =0.83s

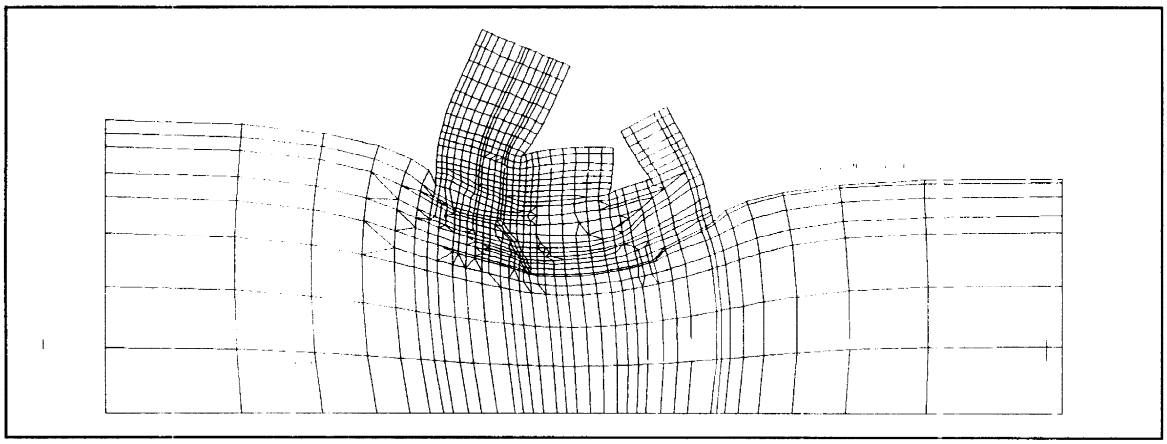

Fig.3b Frequencies and Mode Shapes no.2
f_2= 1.79 Hz ; ω_2=11.26 rad/s T_2= 0.56s

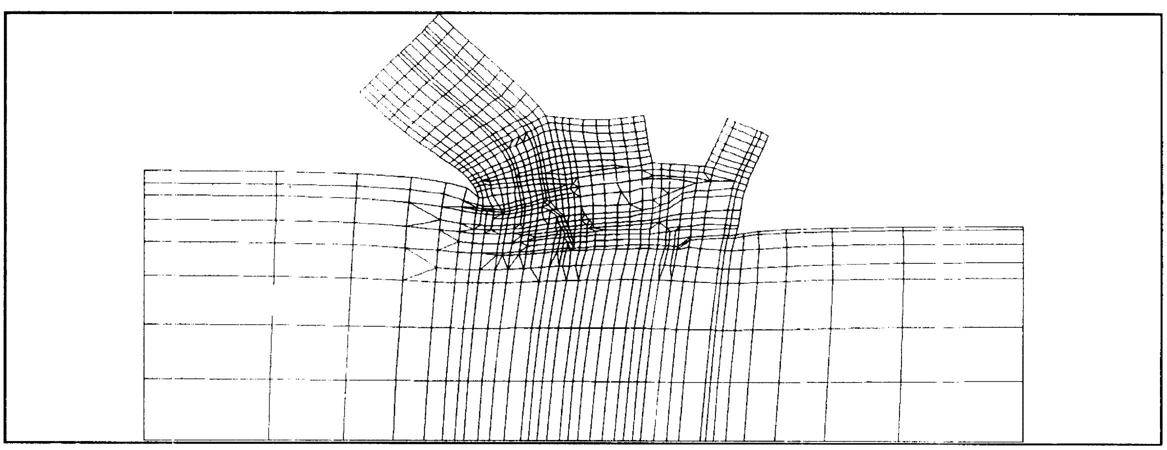

Fig.3c Frequencies and Mode Shapes no.3
f_3= 1.98 Hz ; ω_3=12.42 rad/s T_3= 0.51s