09 May 1984, 9:00 am - 12:00 pm

Behavior of Some Earth Dams on Liquefiable Soil

A. Popovici
Civil Engineering Institute of Bucharest, Romania

V. Perlea
Hydraulic Engineering Research Institute, Bucharest, Romania

L. Corda
Institute of Hydropower Studies and Design, Bucharest, Romania

Behavior of Some Earth Dams on Liquefiable Soil

A. Popovici
Reader, Civil Engineering Institute of Bucharest

V. Perlea
Senior Research Engineer, Hydraulic Engineering Research Institute, Bucharest

I. Corda
Senior Design Engineer, Institute of Hydropower Studies and Design, Bucharest

SYNOPSIS The 1977 March 4 Vrancea earthquake emphasized several zones with liquefiable materials on Romanian territory. Some earth dams of such zones, placed to over 200 km from the earthquake epicenter, were damaged. Important hydropower works are at present in different design or construction stages in such area, comprising long earth dams. The seismic analysis procedure applied to their design was based on the finite element method. Some characteristic cross - sections of the earth dams in different versions have been studied. The analysed sections had different shapes (with and without stabilizing benches downstream) and included different zoning of the materials (sand, fine sand and free draining materials). The analyses pointed out the importance of the drainage blanket at the base of the dam for the increase of the liquefaction strength capacity of the soil - structure system. Some improvement works in certain zones of the foundation soil resulted as being necessary.

INTRODUCTION

Hydroelectric facilities comprising the retention structures which required detailed studies on their seismic behavior, dealt with in the followings, are placed on foundation soil which has been proved to be liquefiable.

Hydroelectric developments on the lower reaches of Olt river, between Slatina town and the Danube river, marked by "A" on Fig. 1, include five typified hydraulic power plants, 53 MW each, equiped with reversible water turbines, employed both for electric power generating and for water pumping of all - round use. Each reservoir, 8...15 km in length, is surrounded by earth dams, totalizing 158 km along the "A" section in the whole. The foundation soil is composed of sand and gravel layers, 12...18 m thick, underlain by Levantine deposits, consisting of claystone with sandy layers down to about 300 m.

Hydroelectric developments in cascade on Siret river, are in foundation conditions similar to those specific to Olt river layouts. The site of these facilities, marked by "B" on Fig. 1, is located to only about 100 km from the Vrancea source of seismic energy release. Besides the Vrancea seismic source, some other local earthquake occurred, due to an extremely complex tectonics: a major fault along Siret river and many other seismically active faults which induced to this area a structure divided into blocks, permanently moving up and down. The local faults have been detected by gravimetric, magnetic and electrometric survey.

The body of analysed dams is made up of rolled fill, using a mixture of alluvial sand from the top layer and gravel from the subjacent layer. In order to reduce seepage through dam and foundation soil a concrete revetment is provided on the upstream slope, continued with a cut-off screen, down to the claystone layer. Under the upstream concrete revetment a gravel filter is placed, continued by a 2 m thick drainning blanket under the compacted fill. A typical cross section, with 2.5 : 1 upstream slope and 3 : 1 downstream slope, is presented in Fig. 2,a. The maximum height of the dams reaches about 30 m in each of reclamation schemes.

Some materials in the foundation soil had been proved to be liquefiable. Levees for flood protection in these areas suffered damages from the liquefaction of sand in foundation soil or in their body, during the strong earthquake of March 4, 1977. Therefore, the study of the influence of liquefaction occurrence in the foundation soil on the stability of dams with permanent retention function in the two hydroelectric facilities has been found necessary for providing an adequate degree of safety during earthquakes. As a result, cross section comprising berms (Fig. 2,b) or improving measures in the foundation soil (Fig. 2,c) ought to be analysed.

ASPECTS OF ROMANIA SEISMICITY

The seismic activity in Romania is governed by subcrustal intermediate earthquakes of Vrancea region, with some specific peculiarities almost unique in the world. These earthquake shakings are entirely different as against the majority of destructive earthquakes in the world, by its uncommon large affected area and unusual long dominant periods. According to known data, they are similar to only motions native under Hindukush Mountains (Central Asia) or under Bucaramanga, Columbia (South America) located however in less populated areas (Balan et al., 1982).

Among the strong earthquakes having the epicenter in Vrancea region, with a periodicity of 30...50 years, the latest produced in March 4, 1977; by his focus features (magnitude, source mechanism, area affected by important intensities), as by its consequences on structures, this earthquake is considered one of the most violent shaking recorded in Europe in the last decades. This earthquake was a multiple seismic event, consisting in a foreshock (point F in Fig. 1), other two shocks (points S_1 and S_2) and a final shock (point S_3, magnitude M = 7.2, focus depth H = 109 km). The final shock produced 19 seconds after the foreshock, at a horizontal distance of 62 km to S-W from it. The energy released by the earthquake was directed mainly towards NNE-SSW direction, activating some faults in this direction and so releasing additional quantities of energy accumulated on faults; in this way, the important effects of the earthquake shaking on earth's crust to large distance from the focus may be explained.

The distribution of maximum seismic intensities observed on Romanian territory during the March 4, 1977 earthquake, is shown in Fig. 1 and is specific also to other major Vrancean earthquakes. Balan et al. (1982) remark the followings with regard to this distribution:

- the multiple shock character of the earthquake and the focus migration towards South-West, determined prevalent severe effects in this direction, with elongation of isoseismal lines in NE-SW direction;

- the maximum intensity I = VIII observed in the epicentral zone, on a relatively restricted area, was recorded too in chief town Bucharest, situated to about 170 km from the epicenter;

- some "seismic islands" with I = VII - VIII have been noted inside the I = VII zone, along the direction of minimum attenuation of seismic intensity, inclusively in the locations of hydraulic works analysed in this paper;

- intensity I = VII has been noticed on a very large area (120,000 km^2), the largest part of it on Romanian territory and completely including the locations of analysed works.

The most complete record of the seismic motion in March 4, 1977 earthquake was obtained in Bucharest at Building Research Institute - INCERC (Fig. 3). The strong motion accelerograph was set up in the basement of a light one - story building, that is very probably not significantly influenced the earth motion. In tne building site, a silty loam layer of about 5 m in surface is followed by a sand deposit with claystone interlayers down to 128 m where a gravel thick layer may be considered as the bedrock; the dominant period of the alluvium results in this way about 1.3 seconds.

LIQUEFACTION PHENOMENA OCCURRED DURING THE MARCH 4, 1977 EARTHQUAKE

As a result of the earthquake, ground damage occurred in many locations that were concentrated in the river flood plains, due to saturated uncohesive soil liquefaction, manifested at surface by cracking and sand boiling. These locations were grouped in a zone (plotted by dashed lines in Fig. 1) in which the loading level and the soil features were favourable to liquefaction. In Fig. 1 one can find that liquefaction occurred to large distances from the epicentral zone, as far as 240 km both to NE and to SW, greater than observed in the case of other earthquakes of the same magnitude.

This finding, beside some data from technical literature concerning effects of other earthquakes, justified to suggest

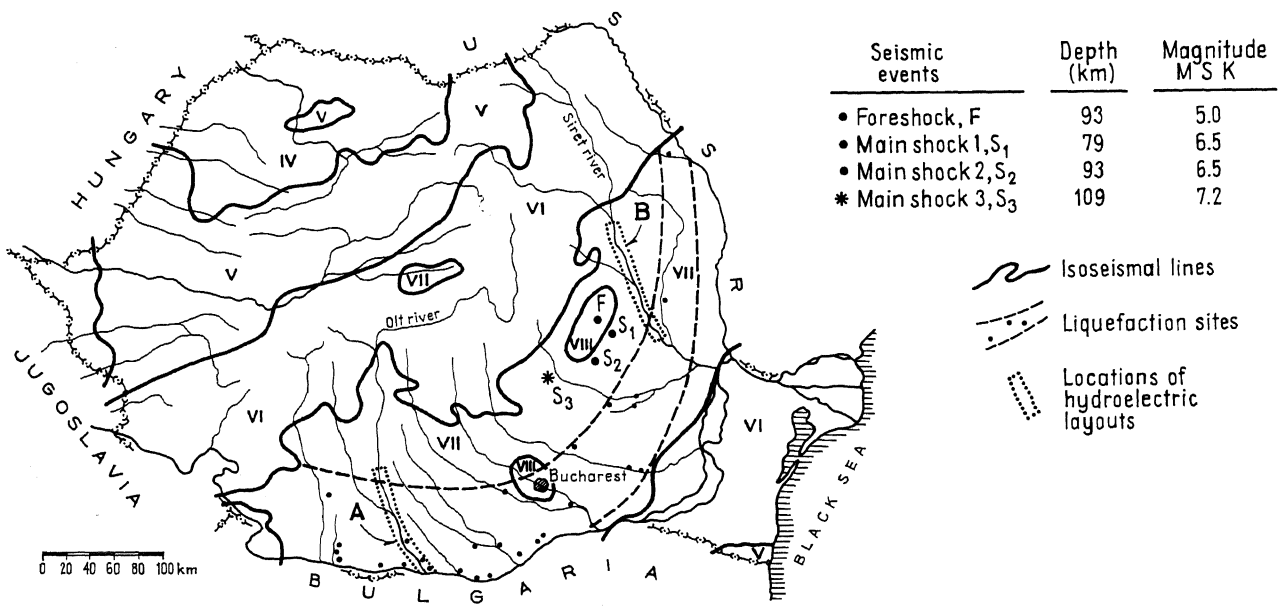


Fig. 1. Macroseismic map of March 4, 1977 earthquake on Romanian territory and sites where evidence of liquefaction was obvious (after Balan et al., 1982)

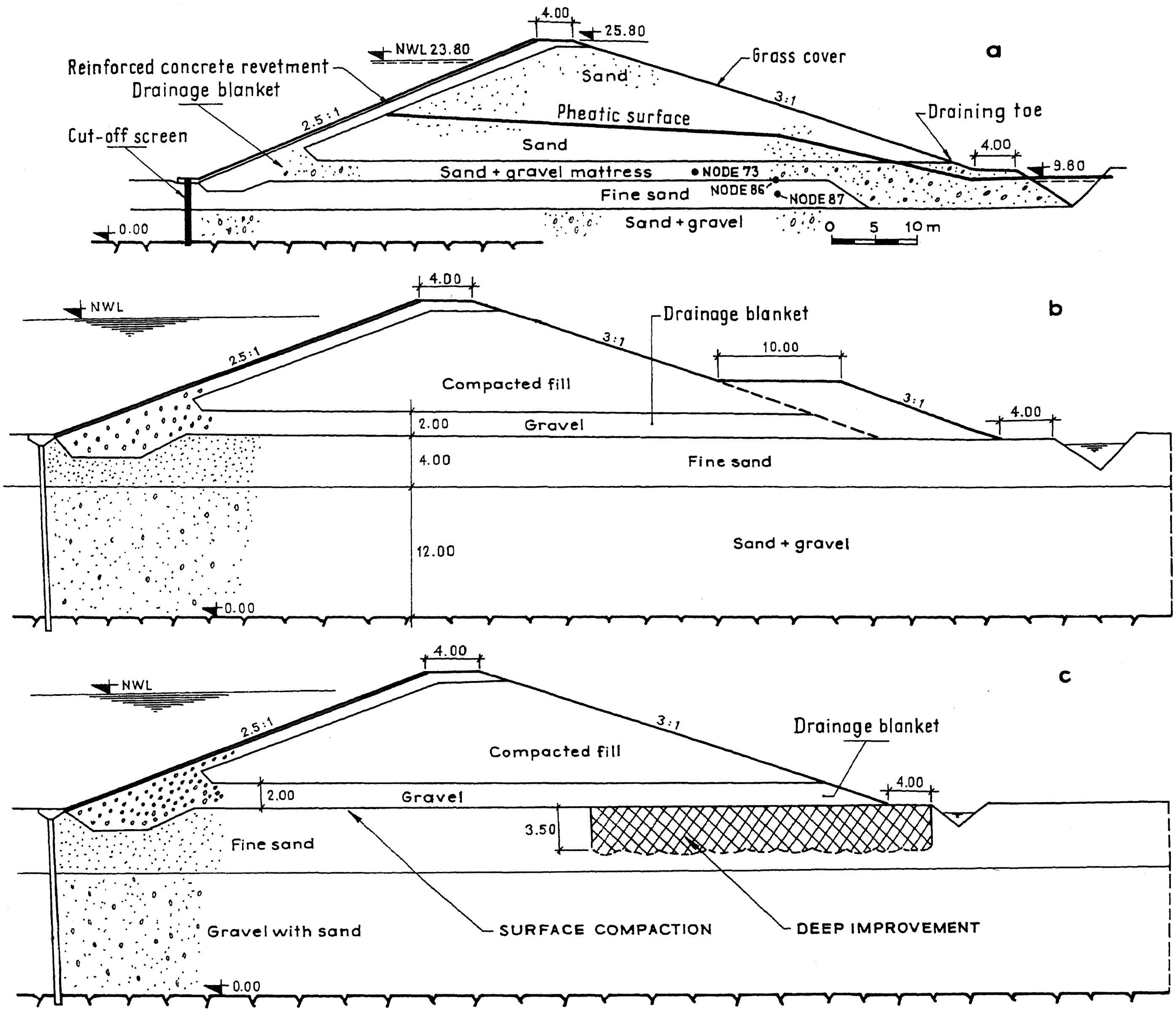


Fig. 2 . Typical cross sections of analysed embankment dams:
a - on thin layer of fine sand in foundation soil; b - on thick layer of fine sand in foundation soil; c - with improved foundation soil

a relationship between the earthquake magnitude (M) and the maximum epicentral distance to sites where liquefaction may be apparent at the ground surface (R) plotted with solid line in Fig. 4, and expressed by:

$$\lg R = 0.914\ M - 4.2 \qquad \text{or}$$

$$R = 3.57 \times 10^{0.914\ (M\ -\ 5.2)} \qquad (1)$$

This relationship (Perlea V. and Perlea M., 1984) is different from other relations previously established, but having a narrower application field.

The liquefaction occurring on March 4, 1977 brought damage to some 25 km of levees for flood protection, along Danube river and its tributaries, in the area limited by dashed lines in Fig. 1; among these, some 1.8 km were on the left bank of Olt river, in the area of hydraulic engineering works that will be discussed in the followings as concerning seismic stability. The damages incurred by liquefaction consisted mainly by longitudinal, seldom transversal, cracking of levee body on the crest, slopes or benches (Fig. 5), occasionally sand boiling at ground surface (Fig. 6). More rarely, and strictly local, slope sliding and settlement have been noticed.

This feature of the levee damage was directly assigned to horizontal displacements following liquefaction of sandy layers in foundation soil. The preferential longitudinal direction of cracks is a direct consequence of preserving

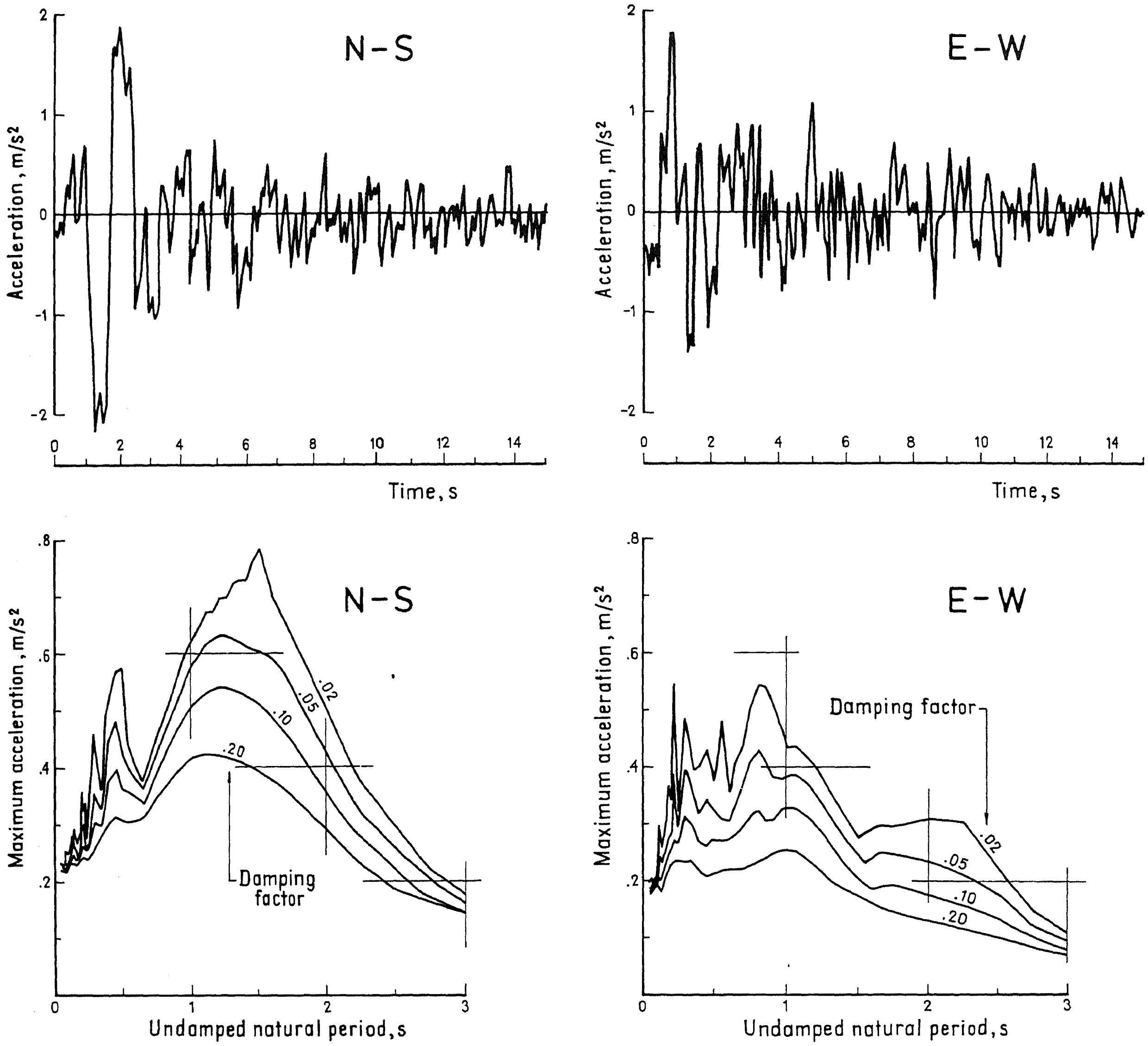


Fig. 3. Earthquake shaking Vrancea March 4, 1977 characteristics: corrected accelerograms and acceleration response spectra (Balan et al., 1982, after INCERC Bucharest recording)

the plain strain state both before and after liquefaction occurring, with insignificant change of vertical normal effective stress, but with essential alteration of the coefficient of earth pressure at rest, K_0, in liquefied zones (from 0.4...0.5 in the case of normally consolidated deposits, to 0.9...1.0 after liquefaction).

IN SITU AND LABORATORY TESTS

Quaternary deposits lying at shallow depths in the foundation soil may be classified in two distinct layers: the first, at surface, consisting of fine alluvium (sand, clayey silt, silty clay), 2.5...7.0 m thick; below this layer, there exists a second horizon consisting of coarse alluvium (sand with gravel, gravel with sand and cobbles), 4.0...10.0 m thick.

As bedrock, Levantine claystone from Upper Plyocene underlying Quaternary deposit to large depth may be considered.

In Fig.7, grain - size ranges of different soils encountered are shown. It is to be noted that fine sand in shallow layer may be classified as easy liquefiable an sand with gravel underneath, as liquefiable. Although in the dam body some materials liquefiable from the grain size distribution point of view are introduced, their liquefaction likelihood may be considerably reduced by a proper densifying.

By standard penetration testing (SPT) along retaining structures, it was concluded that only a part of soils which are liquefiable according to the grain - size distribution criterium are also liquefiable according to density state criterium. So, in the case of one of

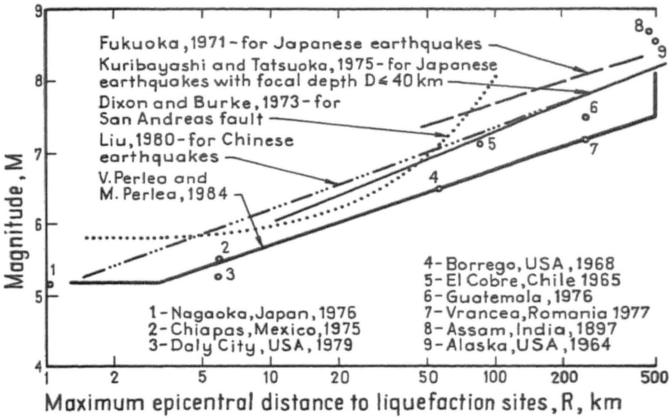


Fig. 4. Relationship between the maximum epicentral distance of apparent liquefied sites and earthquake magnitude

Fig. 5. Longitudinal cracks observed on levees

Fig. 6. Ejected sand near levees

schemes, only 37% of penetration sites revealed liquefiable deposits under seismic loading of intensity VII, in Modified Mercalli Scale (Fig. 8,a). It was found that this proportion varies for different types of soil: the percentage is 29% in the case of silty sand, 35% for fine and medium sand, and 67 % for coarse sand with gravel.

Investigation in detail of density state employed light dynamic (cone) probing. In a test site intended to evaluate the efficiency of improving methods of foundation soil, many such penetrations have been performed; these penetrations have shown a medium to loose density state of the shallow deposit, but also a very important non-uniformity in density state, even on such a small area (Fig. 8,b).

Undisturbed samples extracted at surface with core barrel or tests with the "Soiltest" densimeter proved in the same test site that in 46 % of cases the relative density is less than 60 %. As well as for the analysis of SPT on large territory, it was found that among fine alluvia these coarser ones are in a looser state than the finer ones; the corresponding percentages were 73 % for soils with mean diameter $D_{50} \geq 0.15$ mm, and 15 % for $D_{50} < 0.15$ mm (Fig. 8,c). Similar results were obtained in other test sites.

The relative high percentage of samples denser than necessary for easy liquefying during a VII intensity earthquake (in the case of site "A") led to the conclusion that fine alluvium deposits can not be a priori considered as liquefieble, but need investigations in detail along dams. It was stipulated to perform these investigations by light (cone) dynamic penetrometer.

The laboratory characterization of sand properties has been performed by triaxial compression tests on saples having 37.5 (or 51) mm diameter and 75 (or 97.5) mm height; coarser materials were tested on ∅ = 250 mm and h = 450 mm samples. The used stress paths in these

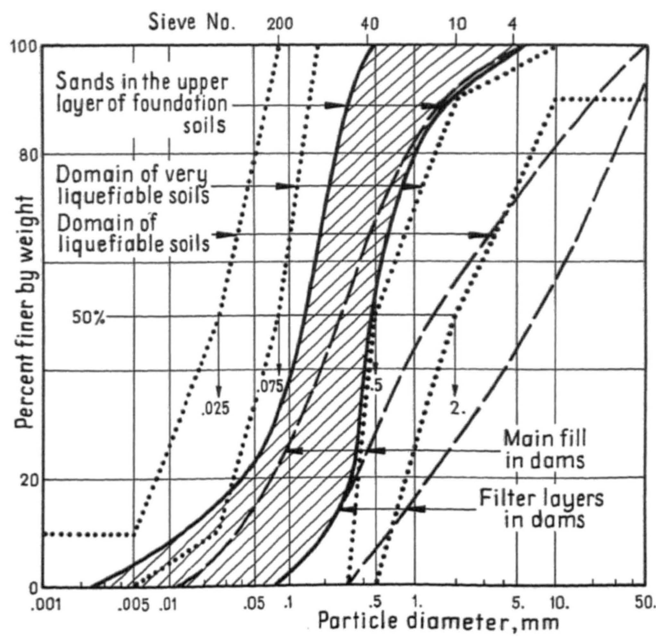


Fig. 7. Grain - size distribution of encountered soils and effect of grading on susceptibility to liquefaction

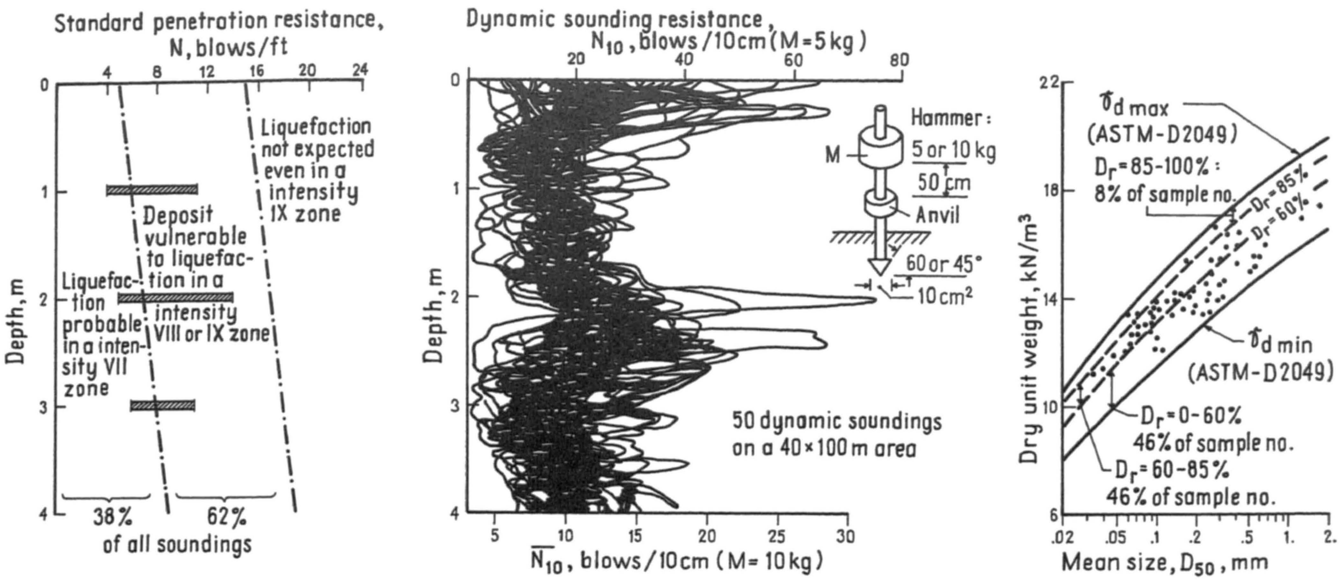


Fig. 8. Results concerning the density state of foundation soil:
a - Standard Penetration Tests; b - Dynamic (Cone) Penetration Tests; c - Soiltest Densimeter and undisturbed samples

laboratory investigations tried to model the best possible the loading sequence in the field, thus ensuring parameters required by a response analysis in nonlinear behavior hypothesis to be obtained.

Cyclic loading tests have been performed in a SOILTEST triaxial apparatus at the Building Research Institute, modified for two - way loading and pore - water pressure recording.

In Fig. 9 some curves used for evaluation of liquefaction potential, obtained by cyclic triaxial tests on undisturbed samples, are shown.

Laboratory tests performed in resonant column equipment and static triaxial apparatus, as well as field exploration for seismic wave velocities determination, allowed to obtain the variation of deformation and damping characteristics in a wide range of shear strains.

ANALYSIS PROCEDURES

The methodology of the earth dam seismic analysis applied in the present report is based on researches of H. B. Seed and his fellow - workers at University of California, Berkeley (Seed, 1979). According to these researches, the seismic analysis of earth dams on liquefiable soils is successively developed in the following stages:

- the evaluation of the initial static stress before the earthquake, in the foundation soil and dam body, by nonlinear analysis based on the finite element method; the soil behavior is modelled by elasto - plastic law (Drucker - Prager yield criterion) or nonlinear ones (Kondner stress - strain curves and Duncan - Seed method);

- the estimation of the design earthquake and of the maximum credible earthquake on the base of seismic history of the place and the geomechanics and seismic foundation soil characteristics;

- the approximation of the actual dynamic stress cycles, in the representative elements, by frequency and constant amplitude cycles; these constant cycles are useful for determining by laboratory testing the soil liquefaction strength curves;

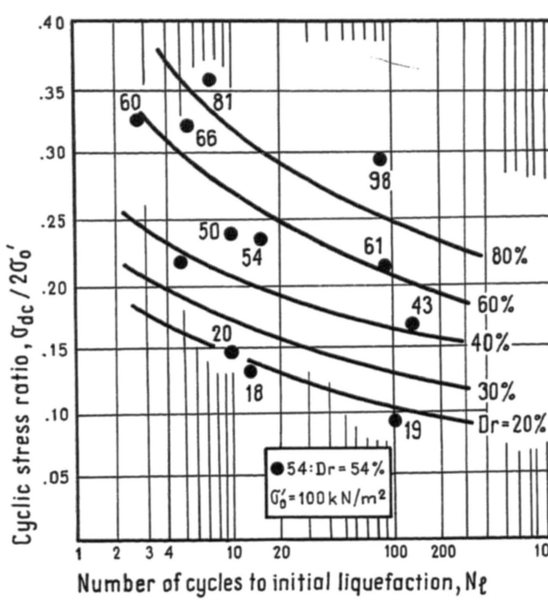


Fig. 9. Results of cyclic loading triaxial tests on undisturbed samples (tests performed by INCERC Bucharest)

the determination by laboratory and in situ tests of the ynamic properties of the dam body materials and of its oundation soils (shear modulus, damping factor, bulk odulus, Poisson's ratio, et al., as a function of strain);

the computation of the seismic response of the dam by ynamic nonlinear analysis using finite element method;

- the computation of pore water pressure inside the dam ody and the foundation soil during and following the earthquake; the analysis is based on the dynamic uniform stress equivalent cycles, the strength liquefaction curves, the permeability coefficients of the materials and the initial stress state before the earthquake;

- the estimation of potential deformations in the representative dam body elements by the comparison between the computed dynamic stress and the response curves determined by laboratory tests;

- the evaluation of permanent displacements and of upstream and downstream slope sliding stability in the dam cross section; this analysis is based on the potential deformations in the representative dam body elements; the drawing of conclusions upon the safety and the seismic behavior of the analysed dam.

The earth dams seismic behavior study requires a careful correlation between the laboratory and in situ testing results and the mathematical results using numerical methods (especially finite element method).

At present, several methods and computer programs in soil mechanics and structural analysis field are developed. Nonlinear finite element procedures are used evermore in static and dynamic stress and strain analysis. The most recent international conferences ICSMFE (Stockholm, 1981) and ICOLD (Rio de Janeiro, 1982) have underlined a large use all over the world of some computer programs for static and dynamic analysis of earth and rockfill dams, elaborated by the University of California at Berkeley. Such specialised computer programs for earth dams (LSBUILD, ISBILD, QUAD4, GADFLEA, DEFORM) or general computer programs for structures analysis (SAP5, NONSAP, FLUSH), adapted by the authors of this Report for Romanian FELIX computers, have been used in the analysis presented below.

A different feature in the seismic analysis is due to liquefiable fine sand forming the foundation soil. According to recent research results, the dense coarse - grained materials also change their deformability and strength characteristics under dynamic cyclic loadings. The generation of the excess pore water pressure during the earthquake, associated to more or less dissipation in function of the drainage conditions, has the greatest influence upon the above mentioned characteristics.

The study of pore water pressure variation in the earth dam - foundation system is based on the general consolidation equation by Terzaghi in which Seed et al. (1976) introduced a supplementary term due to dynamic loads under one - dimensional condition. The supplementary term, named source term, corresponds to the pore water pressure excess generated by the alternating shear stresses. The general relation has the following form:

$$\frac{1}{\gamma_w}\left[\frac{\partial}{\partial x}\left(k_x\frac{\partial u}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y\frac{\partial u}{\partial y}\right)+\frac{\partial}{\partial z}\left(k_z\frac{\partial u}{\partial z}\right)\right]=$$

$$= m_{v3}\left(\frac{\partial u}{\partial t}-\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}\right) \qquad (2)$$

where x, y, z is a threedimensional orthogonal system; u - total pore water pressure; k_x, k_y, k_z - permeability coefficients corresponding to x, y, and respectively z axis; γ_w - unit weight of water; m_{v3} - coefficient of volume change.

The source term $\frac{\partial u_g}{\partial N}\frac{\partial N}{\partial t}$, introduced by Seed et al. (1976), means that during an incremental time dt, if the total pore water pressure has a change du, there will be a supplementary increase of the pore water pressure $\frac{\partial u_g}{\partial N}dN$, because of the dN applied cycles of alternating shear stress; u_g is the pore water pressure generated by the alternating shear stress.

According to relation (2) the computer program GADFLEA has been elaborated by Booker, Rahman and Seed (1976) at University of California, Berkeley. The relation (2) is solved in the plane flow and radial symmetric flow cases by finite element method. The field is discretized by quadrilateral and triangular finite elements system.

The earth dam permanent seismic displacements are essentially influenced by initial dam body stress state and by seismic cyclic stresses. This happens because of the induced changes in the material characteristics (increases of pore water pressure associated with important or complete loss of the strength of the materials). The potential axial strains induced by seismic cyclic stresses in the dam materials, considered as individual elements, are determined by laboratory tests. The actual seismic strains of dam elements are established from the potential element strains and the strain compatibility condition of the whole system taking into account the connections between elements. According to these principles, the equivalent nodal seismic force method and the computer program DEFORM have been developed by Serf, Seed, Makdisi and Chang (1979) at University of California, Berkeley. The seismic action in this method is modelled by equivalent nodal static force sets calculated from the condition of equality between their effects and the strains resulting from dynamic analysis. The maximum seismic shear stresses ($\Delta\tau_{max}$) are considered as acting along horizontal planes and they can be directly calculated from deviatoric seismic stress ($\Delta\sigma_d$) (axis x is horizontal):

$$\Delta\tau_{max}=\Delta\tau_{xy}=\Delta\tau_{yx}=\frac{1}{2}\Delta\sigma_d \qquad (3)$$

The equivalent nodal seismic forces to any element are then calculated by the relations:

$$F_x = \Delta \tau_{max} \frac{(x_j - x_i) + (x_l - x_k)}{2}$$

$$F_y = \Delta \tau_{max} \frac{(y_l - y_i) + (y_k - y_j)}{2} \tag{4}$$

The calculation procedure for permanent seismic displacements, which has been developed in the computer program DEFORM, follows two stages. In the first stage, the initial dam stresses before earthquake are evaluated, using a step - by - step nonlinear procedure. In the second stage, the equivalent nodal seismic forces are statically applied on the structure and the total stresses and the permanent seismic displacements are calculated. Conclusions are formulated on this basis, about dam seismic behavior.

ANALYSIS RESULTS

The initial static stress state before the earthquake and dynamic response to recorded or simulated earthquakes have been calculated for several earth dam cross sections studied for the Olt hydropower works. Many of them will be founded on liquefiable soils. Fig. 10 shows, for example, the static and dynamic stress pattern for a typical cross section of Frunzaru dam (Fig. 2,c), due to gravity weight, hydrostatic pressure and Vrancea - Bucharest March 4, 1977 type earthquake motion, scaled to 0.15 g maximum acceleration, applied in sequence. The earthquake induces important increases of the shear stresses in the body of the dam.

The pore water pressure distribution has been evaluated for a cyclic uniform action corresponding to an earthquake of Richter magnitude M = 7.5. The pore pressure ratios ($r_u = u/\sigma'_o$, u - the pore water pressure, σ'_o - the initial mean effective stress for triaxial conditions or the initial vertical effective stress for simple shear conditions) for two typical cross sections of the same dam, just at the end of the earthquake, are illustrated on Fig. 11 (Popovici, Corda, Diacon and Enica, 1982). The comparative analyses of the liquefied zones corresponding to the two typical cross sections of the dam, showed a much larger area of these zones in the case of the dam section having stabilizing bench at the downstream slope toe (Fig. 11,b). This result is assigned to more difficult drainage conditions of the dam in the case of cross section with bench. The analysis of the liquefied zones development during earthquake, showed that benches have some local delay effect in arising of the mentioned zones, due to the supplementary vertical stresses induced by bench. Afterwards, the more difficult drainage conditions of the foundation soil under dams with benches, are however decisively upon the size of the liquefied zones.

In Fig. 12 the variation of σ'_v (vertical effective stress) and u_g (pore pressure induced by earthquake) during and after earthquake are presented for some nodes of the dam cross section (Fig. 2,a). The dashed lines representing σ'_v and u_g values correspond to a dam cross section with a 1.50 m thick drainage blanket; the full lines correspond

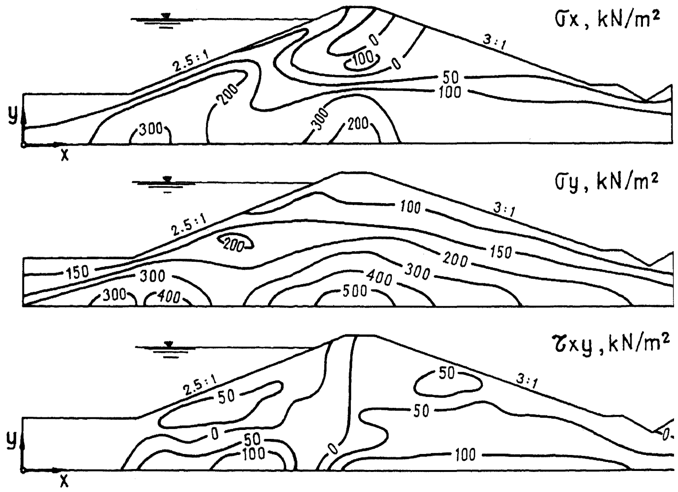


Fig. 10. Distribution of stresses induced by gravity load, hydrostatic pressure and seismic load

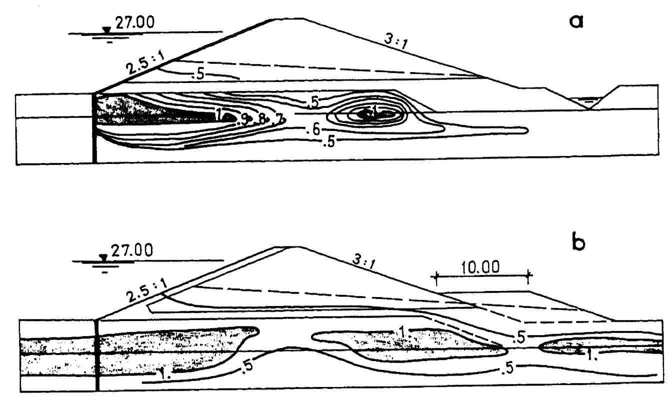


Fig. 11. Pore pressure ratio distribution at the end of eartl quake shaking: a - cross section without berm; b- cross section with berm

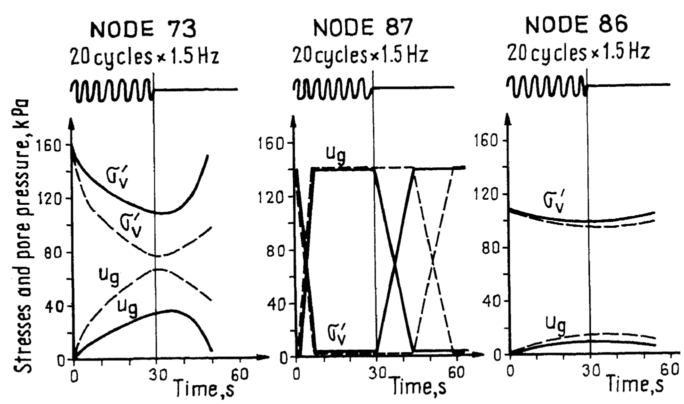


Fig. 12. The influence of the drainage blanket thickness (in the case of usual cross section shown in Fig. 2,a) on earthquake induced pore pressure (u_g) and on vertical effective stresses (σ'_v): dashed lines - 1.50 m thick drainage blanket; full lines - 2.00 m thick drainage blanket

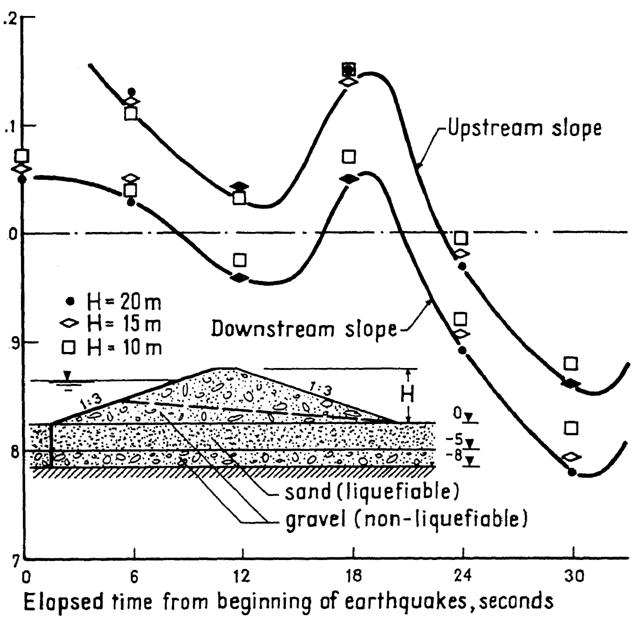
a 2.00 m thick drainage blanket. In the case of the uivalent earthquake having 20 cycles x 1.5 Hz, the graph ows that the first liquefied zones appear about 5 seconds er beginning of the earthquake. Important dissipation and distribution of the pore pressure in the dam body take ıce after the end of the earthquake. In this way, a small ne near the downstream toe remained in liquefied state) seconds after the end of the earthquake.

e increase of the drainage blanket thickness has an portant local effect on decreasing of liquefied zones, on aying of the liquefaction occurrence, and on accelerating the pore pressure dissipation. The increasing of the ınage blanket thickness has not a noticeable influence on e pressure generation and dissipation in the upstream t of the dam body.

e unfavourable effect of the downstream bench which af- ts the section drainage is noticeably (Popovici, Corda, .con and Enica, 1982).

ne limit equilibrium methods (Fellenius and Janbu) were lied for sliding stability analysis. The above mentioned lysis stages gave several elements for a thorough - ng sliding stability study taking into account the ef- tive mobilization of the shear strength on the failure ne and the variation of the pore water pressure during following the earthquake. In this way, the stability ety factors of the upstream and downstream dam slopes e important variations during and following the earthquake.

s variation is illustrated in Fig. 13 for a hypothetical ss section and based on a simplistic analysis scheme:



13. Time variation of stability factor, taking into account the double effect of pore water pressure rise: effective stress decrease and preventing shear waves to propagate through a fully liquefied layer

the pore pressure distribution has been determined for certain moments in the course of the 30 seconds of a harmonic motion, having 20 cycles of 0.65 τ_{max} maximum amplitude; for these moments, pseudostatic stability analyses have been performed, taking into account horizontal body forces corresponding to a seismic coefficient $k_s = 0.075$; when liquefaction prolonged in a continuous horizontal layer, the horizontal body forces above it have been considered vanishing.

The qualitative results of this simplified analysis have shown that: the first critical moment in dam stability appears 10-15 sec (6 - 10 equivalent cycles) after the motion beginning; the second, and the most dangerous, appears in the last third of this long motion, when the liquefied zones become generalized. In addition, the diagram shows that dam high has negligible influence on the stability safety, and that the upstream slope, protected by impervious revetment is more stable than the downstream one.

In dam design, only the first critical moment has been taken into account, as 20 equivalent cycles have been considered too conservative for the Vrancea type earthquake.

In support of this statement, a comparison of recommended representative number of cycles for an equivalent uniform cyclic loading and for a Vrancea type motion may be used. Some recommended values for a M = 7.5 earthquake and the maximum amplitude of an equivalent uniform cyclic loading $\bar{\tau} = 0.65\ \tau_{max}$, are: 32 (Haldar, 1981), 22 (Lee and Chan, 1972), 20 (Seed and Idriss, 1971), 15 (Seed and Idriss, 1982), 10 (Valera and Donovan, 1977). For the Vrancea type accelerogram (Fig. 3) scaled to 0.15 g, and D_r = 30 % soil strength curve (Fig. 9) 7 cycles only have been found representative according to Donovan's cumulative damage procedure.

The calculus permitted quantitative evaluation of pore water pressure variation during earthquake loading in case of the different shapes of the dam cross section and of some artificial measures for the improvement of the foundation soil properties. The existence of the loose fine sand, susceptible to liquefaction, in the upper part of the foundation soil has required the comparative analysis between total or partial removal of the liquefiable layers or the improvement of their geotechnical characteristics.

The cost of the above mentioned works has been very high because of the large involved surface size, and any decision has required well justified study and calculus.

In the conditions of some loose sand layers 3...6 m thick and placed under the ground water level, the vibro - compaction by vibratory rollers has proved its efficiency only to about 1 m deep under the ground level.

Surface compaction with different types of vibratory rollers beside some other surface and deep compaction procedures (heavy tamping, vibrating probes, vibro - flotation, gravel columns, preloading with a 5 m high fill) have been experimented on small areas at Draganesti-Olt hydropower works (Fig. 14). Vibro - rod method has proved efficient in layers situated at 1...6 m depth, but accompanied by

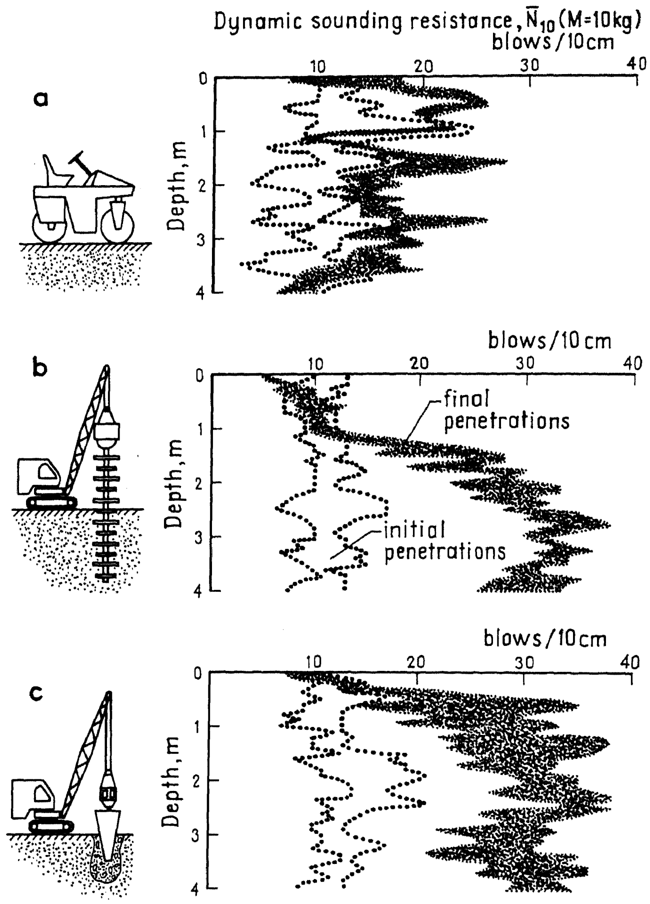


Fig. 14. Penetration logs in natural and improved soil: a - vibratory roller (12 tons, 8 passes); b - deep densification by vibro - rod and water injection; c - vibro - frustum of a pyramid with gravel addition

water injection only. Finally, an original technology has been chosen. It consists of vibro - pressing in the loose sand of a concrete block, 3...4 m long, having a frustum of pyramid shape, and then filling the realized hole with gravel compacted by means of the same block. The technology ensures the vibration and confining pressing of the sandy material, as well as the achievement here and there on the treated surface of some vertical drains for dissipation of pore water pressure (Paunescu et al., 1983)

Technical and economical analyses have been used for establishing the zones justified for the improvement of the geotechnical characteristics of the liquefiable layers. A da cross section on a ground having a liquefiable layer 4...7 m thick is illustrated in Fig. 2,c. The improvemen in depth of soil characteristics has been proved necessary and efficient to the downstream toe of the dam, on a 20...30 m strip width. On the rest of the site, a surface compaction with vibratory rollers was sufficient.

In the seismic response estimation, the loading of the foundation soil by fill carried out in successive 0.50 m layers with surface compaction has been taken into accour

The coarse gravel drainage blanket from the base of the dam, having the grain size distribution curve according tc Fig. 7 has been proved to be very efficient. This drainag blanket causes both the drainage of seepage water infiltrated through the upstream revetment and cut - off wall, and the quick dissipation of the pore water pressur generated by the earthquake.

The cross section type presented in Fig. 2,b has not bee implemented. The downstream stabilizing bench decreases the downstream general slope of the dam but very much delays the dissipation of the pore water pressure generat by the earthquake, negatively influencing the general seismic stability of the profile.

The cross section presented in Fig. 2,c is advisable in the cases when the thickness of the loose fine sand layer at the surface does not exceed 2...3 m. This cross

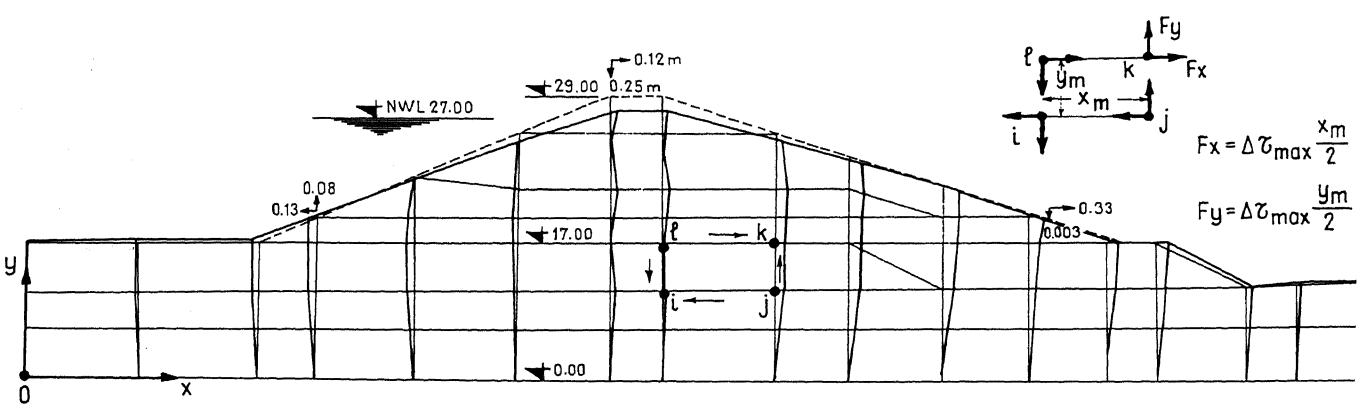


Fig. 15. Permanent seismic displacements computed for a 0.2 g horizontal shaking and a typical Olt (zone "A") cross section

:on has been applied to Draganesti - Olt hydropower :s.

15 illustrates permanent seismic displacements for the imum credible earthquake applied horizontally upstream - istream on a cross section type corresponding to Draga - i - Olt hydropower works. The foundation soil includes ose fine sand layer, 4 m thick, improved according to above mentioned procedure. Several analyses regarding permanent seismic displacements of different cross sec- types, permitted to predict a good seismic behavior of analysed profile. In all cases the maximum permanent lacement did not exceed 0.80 m (Popovici and Corda,).

extended study by numerical analysis, laboratory and itu tests, as well as application of the improvement edures to the foundation soil, are justified by the iomic importance of the analysed hydropower plants by the necessity of the satisfactory safety of the ple, localities and industries downstream.

ICLUSIONS

ual liquefaction cases of loose sand have been noticed he flood plains of Danube river and its tributaries fol- ing the March 4, 1977 Vrancea - Romania earthquake. se liquefactions have occurred to unusual large ances from the shaking source (as far as 240 km from main shock location), justifying the modification of vious established relationships to express the correla- between the maximum epicentral distance to sites re liquefaction may be apparent at the ground surface the earthquake magnitude.

liquefactions were concentrated in lowland areas with se alluvial deposits where the earthquake intensity was or more (on MSK or Modified Mercalli scales). The ge distances from epicenter to liquefaction sites was a ect consequence of the intensity distribution of the ancea earthquake, with great intensities in a very large a.

ne protection embankments against floods along the nube river and the low courses of its tributaries, luding Olt and Siret rivers, have been affected by uefaction of foundation soil. As a consequence, similar rks in these areas must be designed taking into account liquefaction likelihood.

e important reservoirs for different facilities (electric ergy, irrigation, water supply) have been built after 78 or are still in construction in the mentioned zones on uefiable soils. These reservoirs are transversally and igitudinally limited by earth dams having 30 m maximum ight.

ie seismic behavior of earth dams taking into considera- n their interaction with foundation soil has been analysed thoroughgoing in situ and laboratory studies.

ie forecast of earth dam behavior has been made by athematical models using advanced methodology and mputer technique. A chain of computer programs based on finite element method in the case of the nonlinear behavior of the materials has been used. These computer programs have been developed on the basis of some recent researches at the University of California, Berkeley, under the leadership of Prof. H. B. Seed and are frequently applied in many countries.

The thorough-going studies were justified by the economical importance of the works and by the need to ensure their sliding stability and the satisfactory strength of the earth dams placed on liquefiable soils in seismic zones.

Among tested improving methods for liquefiable layers stabilizing, an original technology using a vibrating probe and gravel adding have been found the most efficient. The improvement ought to be stipulated when liquefiable layer exceeded 5 m depth from the ground surface, and was limited to a 20...30 m wide strip under the downstream slope, near its toe.

In order to detect the loose zones and to settle the limits of liquefiable deposits, the light dynamic (cone) penetrometer has been found adequate. A correlation with local validity between dynamic (cone) penetration and standard penetration results has been used with this aim.

REFERENCES

Balan, S., et al. (1982). March 4, 1977 Earthquake in Romania (in Romanian), 516 pp., Ed. Academiei RSR, Bucharest, Romania.

Booker, J.R., M.S. Rahman and H.B. Seed (1976) "A Computer Program for the Analysis of Pore Pressure Generation and Dissipation During Cyclic or Earthquake Loading", Report No. EERC 76-24, University of California, Berkeley, U.S.A.

Dixon, S.J. and J.W. Burke (1973). "Liquefaction Case History", Journal of the Soil Mechanics and Found.Div., Proc. ASCE, vol.99, no. SM11, 921-937.

Dobry, R., D. Powell, F.Yokel and R.S. Ladd (1980). "Liquefaction Potential of Saturated Sand - The Stiffness Method", Proc., 7th World Conference on Earthquake Engrg., Istanbul, vol. 3, 25-32.

Flores - Berrones, R.J. and A.W. Dawson (1977) "A Liquefaction Case History, Chiapas, Mexico", Proc., 9th International Conference on Soil Mechanics and Found. Engrg., Tokyo, vol. 2, 237-240.

Haldar, A. (1981). "Uniform Cycles in Earthquakes: A Statistical Study", Int. Conf. on Recent Advances in Geotechnical Earthquake Engrg. and Soil Dynamics, St. Louis, ses. 2, 195 - 198.

Kuribayashi, E. and F. Tatsuoka (1975). "Brief Review of Liquefaction During Earthquakes in Japan", Soils and Foundations, Tokyo, vol. 15, no. 4, 81-92.

Kuribayashi, E. and F. Tatsuoka (1977). "History of Earthquake - Induced Soil Liquefaction in Japan", Bulletin of Public Works Research Institute, Tokyo, vol. 31, 1-26.

Lee, K.L. and K.Chan (1972). "Number of Equivalent Significant Cycles in Strong Motion Earthquakes", Proc. Int. Conf. on Microzonation for Safer Construction. Research and Application, Seattle, vol. 2, 609 - 627.

Liu, Y. (1980). "Methods for the Prediction of Sand Liquefaction", Proc., 7th World Conference on Earthquake Engrg., Istanbul, vol. 3, 105-112.

Paunescu, M. et. al. (1983). "Studies and Tests in Situ on Structures Performed on Soils Improved by Vibration", Proc., 8th European Conference on Soil Mechanics and Found. Engrg., Helsinki, vol. 1, 65-68.

Perlea, V. and M. Perlea (1984). Dynamic Stability of Sandy Soils (in Romanian, in press), Ed. Tehnica, Bucharest, Romania.

Popovici, A. and I. Corda (1983). "The Earth Dams Permanent Seismic Deformations Prognosis (in Romanian), Hidrotehnica, Bucharest, vol. 28, no. 8, 263-267.

Popovici, A., I. Corda, V. Diacon and N. Enica (1982). "Generation and Dissipation of Pore Pressure During Seismic Action on Earth Dams" (in Romanian), Hidrotehnica, Bucharest, vol. 27, no. 11, 322-325.

Seed, H.B. (1979). "Considerations in the Earthquake Resistant Design of Earth- and Rock - Fill Dam", Geotechnique, London, vol. 29, no. 3, 215-263.

Seed, H.B. and I.M. Idriss (1971). "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Found. Div., Proc. ASCE, vol. 97, no. SM9, 1249 - 1273.

Seed, H.B. and I.M. Idriss (1982). Ground Motions and Soil Liquefaction during Earthquakes, 134 pp., Earthquake Engineering Research Institute, Berkeley, California.

Serf, N., H.B. Seed, F.I. Makdisi and C.Y. Chang (1979). "Earthquake Induced Deformations of Earth Dams", Report No. EERC 76-4, University of California, Berkeley, U.S.A.

Valera, J. and N.Donovan (1977). "Soil Liquefaction Procedures - A Review", Journal of the Geot. Engrg. Div., Proc. ASCE, vol. 103, no GT6, 607 - 625.

Youd, T.L. (1977). "Brief Review of Liquefaction During Earthquakes in Japan. Discussion", Soils and Foundations, Tokyo, vol. 17, no. 1, 82-85.