17 Apr 2004, 10:30am - 12:30pm

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Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

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ABSTRACT

The complex hydrotechnical works achieved upon the lower basin of the Olt river, within the southern part of Romania, imposed the building of several large reservoirs situated within the geo-morphological unit of the above named river flood plane. During the last years, due to the recent earthquakes that affected Romania, the macro-earthquake zoning of the country was changed. In these conditions, the problem of studying the stability of dikes foundation ground from the liquefaction point of view has raised. The paper presents the survey done in site and laboratory tests performed in order to determine most accurately, the natural ground geotechnical and dynamic parameters as well as an original method in order to estimate the ground liquefaction potential. Finally, the general stability analyses of the assembled dike-foundation ground is presented in pseudo-static hypothesis with taking into account the geotechnical parameters expected in dynamic conditions.

INTRODUCTION

In the last two decades large hydrotechnical works have been achieved along the Olt River, in the southern part of Romania. These imposed the building of several reservoirs along the above mentioned river plane. (Fig. 1).

Fig. 1. Romania map with the studied location area

In Figure 2 it is presented the location for the three hydroelectric developments that are studied in this paper, Strejesti, and Slatina.

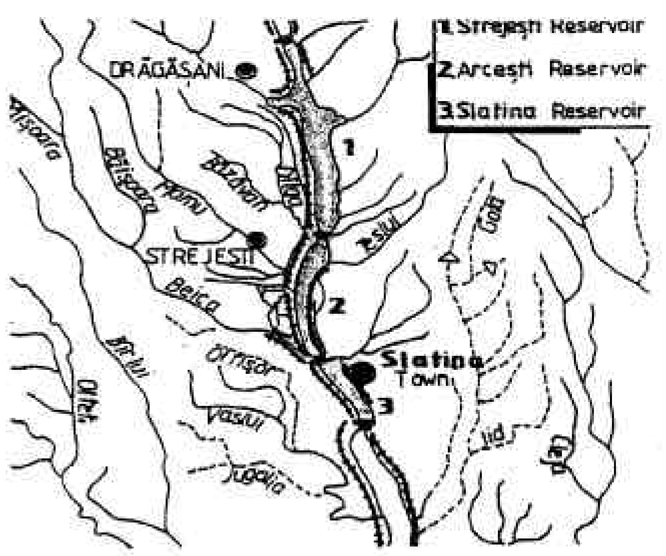

Fig. 2. Location of the studied reservoirs

The reservoirs have usable volumes greater than 15 million cubic meters. They have been achieved by building a concrete spillway dams upstream the river next to the power station. The closing of the right and of the left side have been done by means of two embankment dams.

The dikes have lengths of 8-12 km and are made of cohesive fill (ballast taken from the borrow pits: one part of sand and five parts of ballast). The body dikes will be “E” in the followings.

Site and laboratory tests were made in order to determine the physical and mechanical characteristics for the geological units of the foundation ground. The results are presented in the next chapter.
Due to the last earthquakes that affected Romania, the seismic partition zone changed, and so, the characteristic seismic acceleration for the studied location increased from 0.05g to 0.16g.

Therefore, a new problem was imposed: the foundation ground stability, taking into account the liquefaction potential in the new seismic conditions.

Next chapter summarises the geotechnical in site and laboratory tests results.

GEOTECHNICAL TESTS RESULTS

GEOTEC Company carried out on site and laboratory surveys between June–September 1997. The following representative lithological columns were obtained for the location of the three HPD presented above:
Strejesti HED
a) Covering formations
- A_1- Alluvial Deposits: fine-medium silty sands having thickness of 1÷2.5m;
- A_2 – Terrace Deposits: sands and gravel having thickness of 5÷8 m;
- A_3 – Flood Plains Deposits: silty clay having thickness of 0.5÷1.0m;

b) Bedrock
- B_3 – Sedimentary rocks: marly clay

Arcesti HED
a) Covering formations
- A_1- Alluvial Deposits: fine-medium silty sands having thickness of 1÷3,0 m;
- A_2 – Terrace Deposits: sands and gravel having thickness of 5÷10 m;

b) Bedrock
- B_{12} – Breccia Formations of Tectonic and Sedimentary Origin: marly fine-medium sands having thickness of 2-6 m;
- B_3 – Sedimentary rocks: marly clay

Slatina HED
a) Coverig formations
- A_2 – Terrace Deposits: sands and gravel having thickness of 3÷7 m;

b) Bedrock
- B_{1-1}-Breccia Formations of Tectonic and Sedimentary Origin: marly-silty clay sands having thickness of 1,0÷1,5 m;
- B_3: – Sedimentary rocks: marly clay;

Design values for the geotechnical parameters were established using the geotechnical physical and mechanical tests.

They were used in the stability and liquefaction potential valuation.

In tables 1, 2 and 3 are summarized the design values for the main geological units parameters.

Table 1. Strejesti HED- Geotechnical parameters

Layer	γ	γ_s	γ'	n	ϕ	c	I_D
	[kN/mc]			[%]	[^{o}]	[kPa]	-
E	20,5	27,0	11,0	-	34	-	0,71
A_1	18,0	26,5	9,7	42	22	20	0,70
A_2	19,0	26,5	10,7	35	35	-	0,66
A_3	19,0	26,5	-	-	14	40	-
B_3	20,0	27,2	10,0	-	14	60	-

Table 2. Arcesti HED - Geotechnical parameters

Layer	γ	γ_s	γ'	n	ϕ	c	I_D
	[kN/mc]			[%]	[^{o}]	[kPa]	-
E	20,5	27,0	11,0	-	34	-	0,71
A_1	18,0	26,5	9,7	42	22	20	0,70
A_2	19,0	26,5	10,7	35	35	-	0,66
B_{12}	18,3	26,5	9,9	40	32	14	0,70
B_3	20,0	27,2	10,0	-	14	60	-

Table 3. Slatina HED - Geotechnical parameters

Layer	γ	γ_s	γ'	n	ϕ	c	I_D
	[kN/mc]			[%]	[^{o}]	[kPa]	-
E	20,5	27,0	11,0	-	34	-	0,71
A_2	19,0	26,5	10,7	35	35	-	0,66
B_{1-1}	19,5	26,5	9,7	41	28	20	0,70
B_3	20,0	27,2	10,0	-	14	60	-

where:
γ is unit weight of the soil in natural condition
γ_s is unit weight of the skeleton
γ' is submerged unit weight
n is porosity
ϕ is internal friction angle
c is cohesion
I_D is relative density (D_R).

IDENTIFICATION OF LIQUEFYING ZONES

We have chosen simplified analytical methods in order to identify the liquefied zones under seismic loads. These semi-empirical methods rely on the comparison between the cycle loads effects on the soil samples tested in the lab and on the inn site seismic load effects upon the same geological unit.

The comparison can be done from unit stresses point of view (Seed & Idriss, 1977), but also from the specified angular strains point of view (Seed, 1971). The cycle specified strains evaluation is rather a difficult path to follow. It also means to estimate the effective value for shear modulus in dynamic conditions, G_d. So, under these conditions we chose the first possibility.

The procedure used needs to follow three different steps, that is:

a. to calculate the maximal unit tangential stresses, τ_{max}, induced by the earthquake to the ground;

b. to calculate liquefaction limit (resistance), $\tau_{max,l}$, in the ground at the same depths were τ_{max} had been calculated;
c. Liquefied zones identification corresponding to z depths in the ground, where according to Fig. 3:

$$(\tau_{max})_z \geq (\tau_{max,l})_z \qquad (1)$$

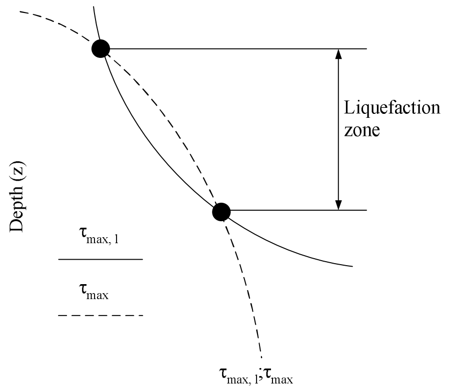

Fig. 3. Graphical determination of liquefaction zone.

The determination of the tangential unit stresses values induced by the earthquake at a certain depth into the ground theoretically can be done with the relation:

$$(\tau'_{max})_z = a_{max}\ \rho\ z \qquad (2)$$

Where a_{max} is maximal acceleration induced by the earthquake at the ground surface; ρ is soil density.

If we consider the soil flexibility, expressed by a coefficient, r_d, as long as the underground water presence, the tangential unit stress variation induced by the earthquake can practically be determined with the relation:

$$(\tau_{max})_z = [a_{max} / g]\ r_d\ [\gamma h_w + \gamma_{sat}\ (z - h_w)] \qquad (3)$$

Where: γ, γ_{sat} are the soil unit weights over and below the underground water from the depth h_w; g is the gravity acceleration; r_d is the coefficient, which can be calculated with linear relation (Iwasaki & Tokida, 1980):

$$r_d = 1 - 0.015z \qquad (4)$$

where z is the depth, expressed in meters.

Because the liquefaction limit (resistance), $\tau_{max,l}$ absolutely depends on the vertical unit stress, σ'_{vo} and on compaction degree I_D (Void,1981), we can determine $\tau_{max,l}$ at a certain depth, z, from the foundation ground, using the chart from Fig. 4 (Gibs & Holtz,1957) and the relation:

$$(\tau_{max,l})_z = (R_l)_z\ (\sigma'_{vo})_z \qquad (5)$$

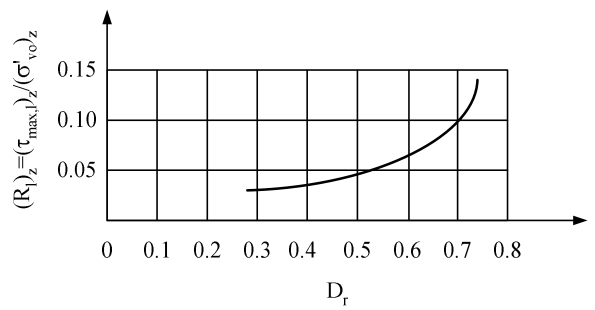

Fig. 4. Graphical calculus of the liquefaction limit stress value.

In the last step, after verifying of relation (1), the liquefaction zones into the dike foundation ground can be done.

The above-described methodology was applied for all dikes representative sections, corresponding to the three studied reservoirs. Different values of the ratio, a_{max}/g, until the maximal value, 0.16, were taken into account.

For example, on Fig. 5, one can see the calculus results related to a section situated on the left side (LSD) close to the dam of HED Arcesti.

By comparison between τ_{max} graph and $\tau_{max,\ l}$ graph result the liquefaction possibility, only inside of A2 layer (sand and gravel) for $[a_{max}/g]=0.16$ at depths above 9.0m.

Similar results were also obtained for HED Strejesti and Slatina.

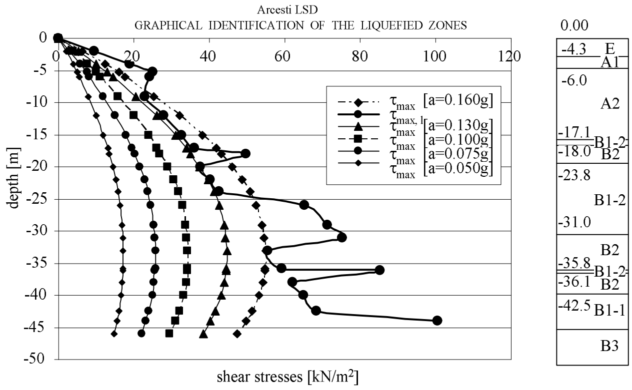

Fig. 5. HED Arcesti. Left side close to the dam. Graphical identification of the liquefaction zones.

STABILITY ANALYSIS UNDER DINAMIC CONDITIONS

Because it's negative effect on the soil shear, the liquefaction process can be the trigger mechanism of stability loss phenomena.

Consequently, for the critical dykes sections, from the point of view of the liquefaction potential, stability analysis under pseudo-static conditions were performed (Fig. 6). Some methods like Bishop, (B), Jambu, (J), Morgenstern-Price, (M-P) based on the limit equilibrium as main hypothesis, were used.

The stability calculus was performed also for different values of the a_{max}/g ratio (0.050, 0.075, 0.100, 0.130, and 0.160).

The decreasing of internal friction angle value was considered according to the bellow relation (Barkan, 1962):

$$tg\phi_d = tg\phi_\infty + (tg\phi - tg\phi_\infty)e^{-B_\phi a_{max}/g} \qquad (6)$$

where ϕ_∞ is the ϕ_d value when $a_{max} \rightarrow \infty$; B_ϕ is a coefficient depending on the total vertical stress value (see Fig. 7).

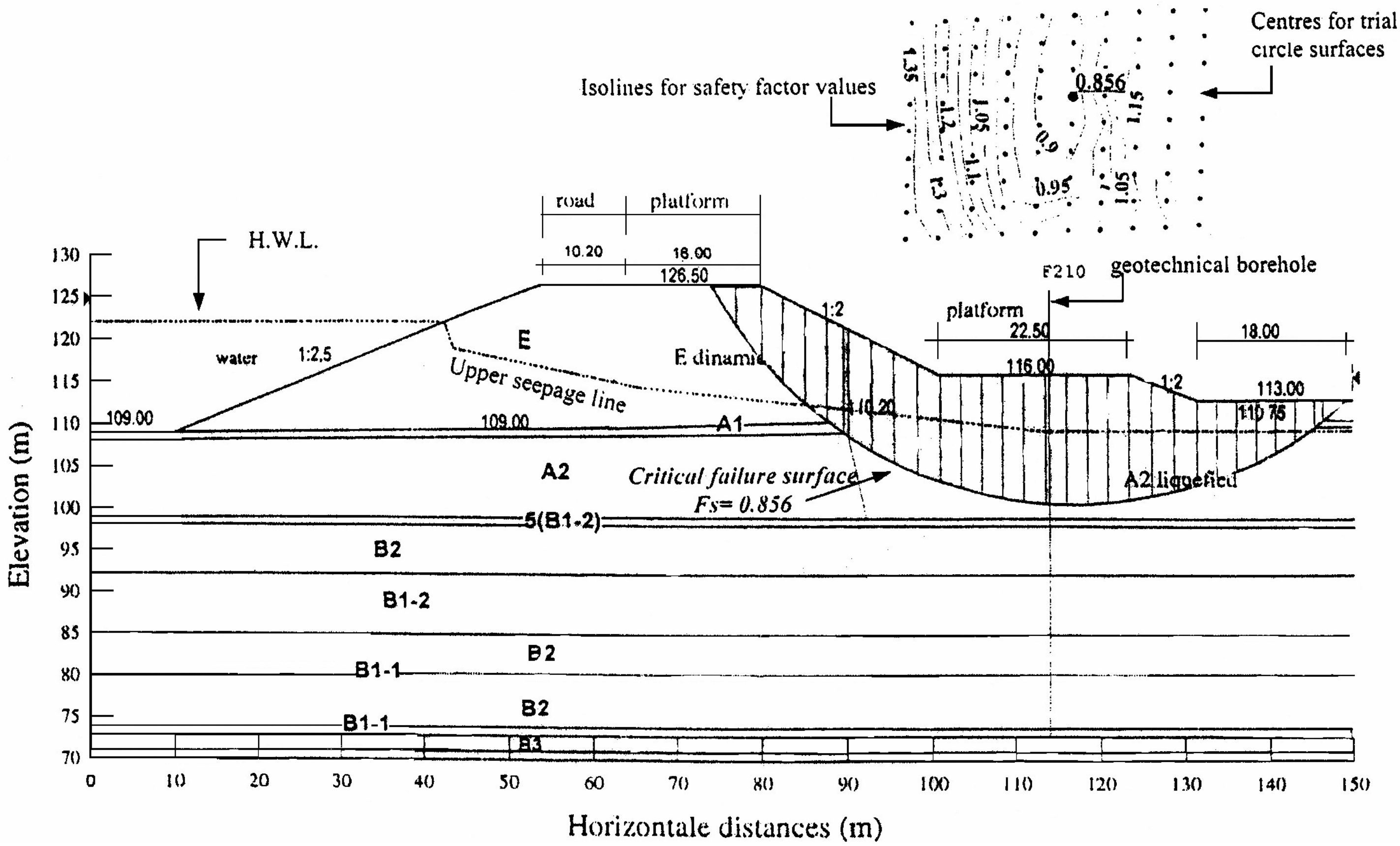

Fig. 6. HED Arcesti. Left side close to the dam. Stability analysis results.

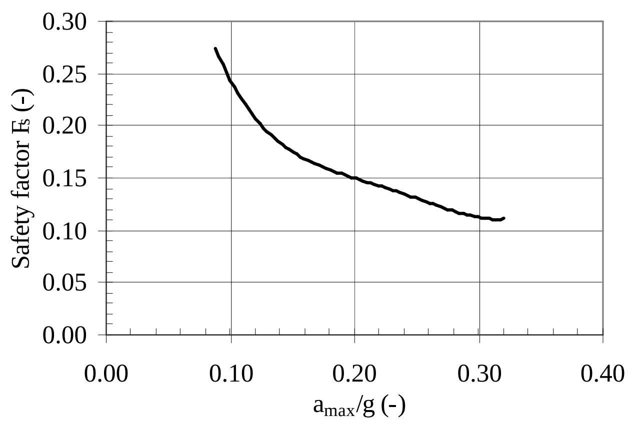

Fig. 7. B_f coefficient variation with s_v

In the case of the liquefying layer, like A2 under seismic acceleration greater than 0.13g, reduced value for the internal friction angle was used, namely ϕ=12º.

On the Fig. 8 one can see the critical section (HED Arcesti LSD), see also Fig. 6, with the critical failure surface position.

According to Fig. 8, the safety factor value, for different calculus method and seismic acceleration value are done in table 4.

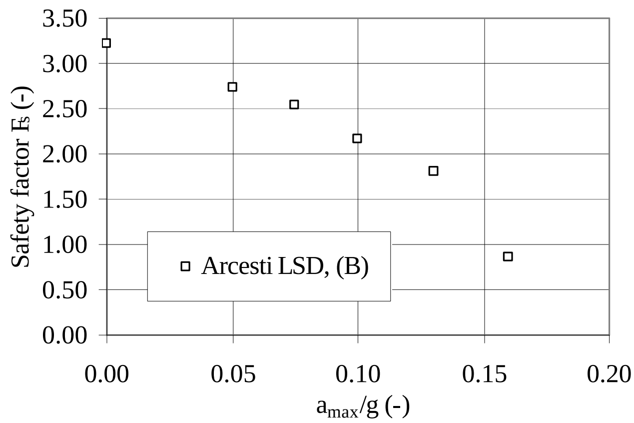

Fig. 8. Stability analysis for HED Arcesti LSD

Table 4. HED Arcesti stability factor values (Fs)

F	Arcesti LSD		
a/g	B	J	M-P
0	3.21	2.68	3.22
0.05	2.73	2.30	2.75
0.075	2.53	2.15	2.55
0.10	2.16	1.85	2.18
0.13	1.80	1.57	1.81
0.16	0.86	0.71	0.90

CONCLUSION

The results of a complex research work activity concerning the liquefaction potential for some hydrotechnical Romanian dikes are presented.

The critical sections corresponding to the three studied HED were identified from the point of view of the liquefaction potential, and stability analysis was performed.

According to the lower safety factor values obtained for the characteristic seismic acceleration, a_{max}=0.16g, the critical studied sections have to be reanalysed in the near future from stability point of view.

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