DYNAMIC ANALYSES OF AN EARTH DAM WITH DIFFERENT INITIAL STRESS STATE

R. Previtali[1], R. Pellegrini[2], F. Carnevale[2] & P. Chemello[3]

[1] ISMES, Seriate, Italy, riccardo.previtali@ismes.it

[2] ISMES, Seriate, Italy

[3] Enel Green Power, Rome, Italy

Abstract: *This paper reports of the outcomes of the seismic assessment of a homogeneous earth dam located in Northern Italy carried out according to the Italian dam regulation. The assumed behaviour of the soil materials of the dam and its foundation follows a non-associated Mohr-Coulomb elasto-plastic law. A nonlinear hysteretic damping model is also included to reproduce cyclic elastic strain energy dissipation. The nonlinear seismic analyses, performed into FLAC2D, consider seven different natural horizontal and vertical accelerograms compatible with the reference spectra defined through a site-specific probabilistic seismic hazard analysis. The main parameters of the geotechnical model of the dam-foundation system are derived from specific experimental campaign. Since the dam is founded on clayey, normally consolidated, deformable soils, the progressive settlements of the embankment are calibrated through a back analysis of the measured dam settlements to get the current degree of consolidation, the spatial distribution of the interstitial overpressures and the residual time of the primary consolidation process. Focus is made on the effects of the initial distribution of stresses and pore pressures within the dam body and its foundation on its seismic response.*

1. Foreword

The dam, about 90 years old, located in the Italian Alpine region, despite its modest highness (11.2m) guarantees 40 million cubic metres of water impounding because of its planimetric extension (its overall length is 1980m). This is the reason why it is a strategic existing dam according to the Italian Ministry of infrastructures and transport. The Italian Code for Dams (NTD14) and the Italian Building Code (NTC18) foresee in this case that seismic assessments be carried out by applying several natural earthquakes as time histories to the dam-foundation system, all ensuring spectral compatibility with the assigned hazard spectrum. To account for the expected strong shaking due to ultimate state earthquakes, the dam and its foundation have been modelled within a coupled non-linear approach by an elastic plastic constitutive model and by a nonlinear hysteretic damping model. A careful assessment of the initial condition of the dam and foundation system is therefore required.

Dams in service exhibit some long-term settlement. An almost linear relationship between settlement and the logarithm of time has been observed in many cases described in the literature of earth and rockfill dams (Hunter and Fell, 2003). Long term settlements are meant as a creep effect not involving changes of the in-situ stress.

Tedd et al. (1997) suggest that settlement index, as defined by Charles (Charles,1986), exceeding 2% of the initial crest dam height may indicate in earth dams the presence of deformation mechanisms other than creep, recommending a more in-depth analysis. The values calculated for the dam under investigation are indeed higher than the 0.02 threshold, leading, therefore, to consider processes other than creep. In this case, the measured settlements are considered as an effect of the still progressing primary consolidation of the foundation soil, a thick layer of loamy clay; accordingly, some overpressures are still in place and modify the initial effective stresses and ultimate strength to shearing induced by the seismic action.

2. The Dam

2.1. Materials

The earth dam, of the homogenous type, with a simple originary trapezoidal section, has been built in 1927 with materials coming mostly from the eastern zone of the reservoir and, to a lesser extent, from the excavation site of a discharge canal in the interior of the lake. At the most depressed point of the foundation, the embankment reached a height of 10.50 m (original elevation 388.00 m a.s.l.).

At the western side, the embankment rests, for a short length, on a lithoid substratum of calcareous reddish marly sediments. This formation, distinctly stratified, dips towards east, with a low inclination, Figure 1, resulting in increased thickness of the upper loamy-clay layer. At dam's centre these clayey deposits reach the remarkable thickness of 180-200 m. On the eastern shore the embankment is founded on silty-clayey deposits, and it is very likely that the rocky base can be found, locally, at a depth not exceeding 35-45 m.

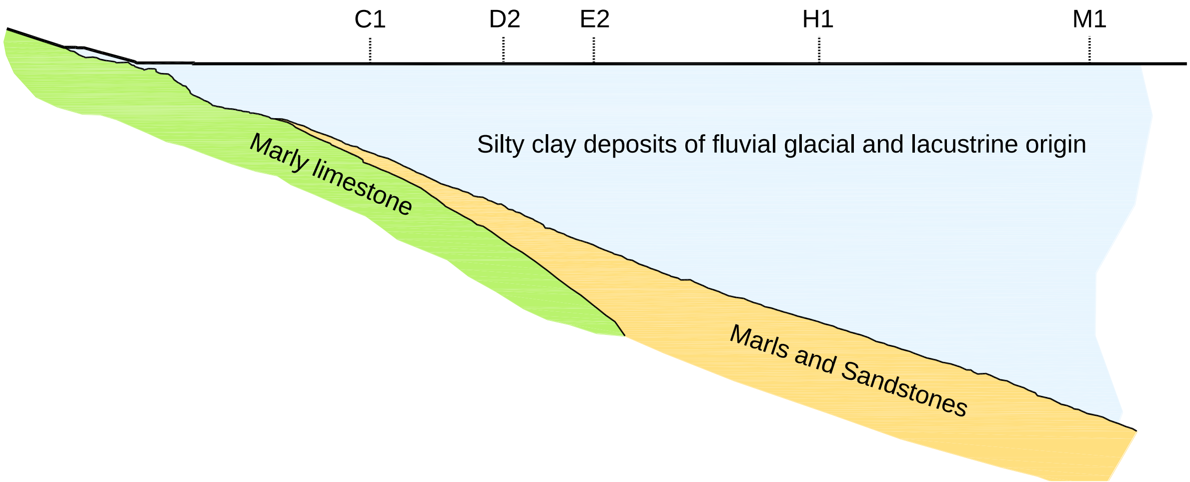

Figure 1. Geological section along dam longitudinal axis with evidence of relevant sections (C1, D2, E2, H1 e M1), after Milli (1986).

2.2. Construction and upgrades

The embankment was built in 1927, while normal operation of the dam began in 1930. Because of the consolidation of the soil foundation, the dam highlighted settlements of the downstream face and a longitudinal depression along the crest, both subject to periodic fills and reloading.

In years 1967-69 the dam has been raised from a crest elevation of 388.70 m a.s.l. to an elevation of 389.20 m a.s.l, to comply with the regulations on the admitted dam freeboard.

Between March 2006 and December 2009, the freeboard was reset (Figure 2), by increasing the elevation of the embankment up to 389.5 m a.s.l.. Work on the dam was carried out using loamy soil (named type A) of the same grain size of the existing dam body, compacted to 90% of the modified Proctor test density, and coarser soil (named type C) for the lower berm built on the downstream face. The fine soils have been taken from loan quarries located within the lake, near the eastern part of the dam, and the coarser soils in the bed of a tributary stream. Today dam's height is variable up to a maximum of 11.2m.

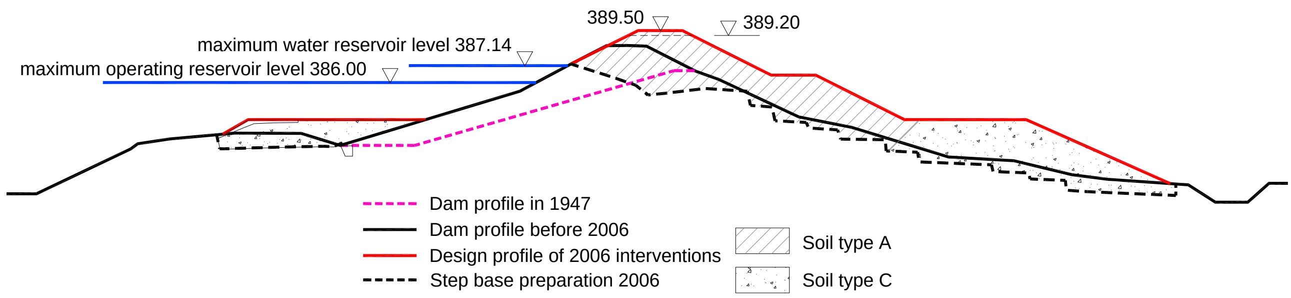

Figure 2. Dam section E2: today section of the dam with evidence of additions.

2.3. Dam behavior from monitoring data

Based on data gathered by the structural and hydraulic monitoring system, it has been possible to identify an overall regular behavior of the dam along its long service life. A peculiar aspect is dam settlement, with a rate of a few cm/year, still in progress since construction, which has been recognized as the response to loads of the low stiffness of the foundation clayey soils, which have been recognized as normally consolidated (Adami et al., 2010). Maximum values and higher rates are measured at the topographic survey points CLSD2, CSLC1, CSLE2, places nearby the dam crest on its upstream face (Figure 1, Figure 3).

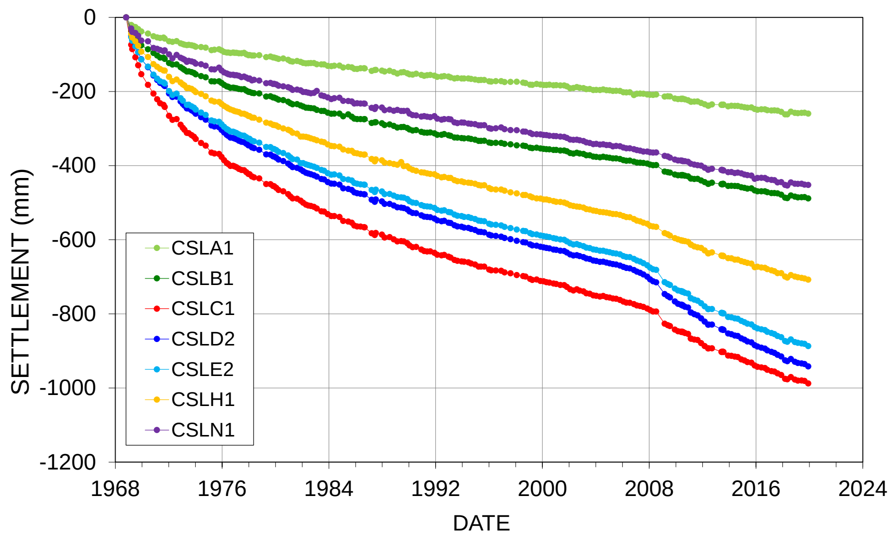

Figure 3. Settlement of the dam measured from 1968 to 2019 at significant topographical survey points. CSLE2 refers to section E2.

2.4. Consolidation analysis

A PLAXIS2D® finite element model (FEM) of dam's Section E2 has been set up. The FEM model represents the entire soil foundation down to the stiff marly base lying about 104 m below the ground level.

The physical and mechanical properties of the dam and its foundation soils have been first determined by processing all the available experimental data and observations gained at different times from construction, typically in preparation of the restoration works, Table 1. Compliance related model parameters (C_r, C_s, e_o) and permeability (k_h, k_v) values of foundation soils have been calibrated seeking the most reasonable agreement with measured settlement values and evolution over time at the dam section E2, pertaining to the most settlement prone portion of the dam. By a process of back analysis, the better agreement has been obtained by varying values of some geotechnical properties as follows, in compliance with the geotechnical available evidence:

- by allowing for a slight overconsolidation of foundation soil down to 10m below the ground level assigning an overconsolidation ratio OCR decreasing from 4 to 1 in 10m.
- by increasing permeability of the deeper soils and assigning flow anisotropy (k_h/k_v) in the upper overconsolidated 10m of foundation soils.

- by increasing the compressibility in the normal consolidated range of deeper foundation soils (parameter C_c).

Table 1. Final geotechnical parameters of the dam and its foundation soils.

SOILS	γ_{wet} [kN/m³]	$\gamma_{saturated}$ [kN/m³]	c' [kPa]	ϕ' [°]	E' [MPa]	υ'	OCR	e_0	C_c	C_r	k_v [m/s]	k_h/k_v
Dam												
Type A	20.0	20.8	4	33	9	0.3	-	0.46	-	-	1e-7	1
Type C	23.0	23.3	-	40	50	0.3	-	0.19	-	-	1e-7	1
Overlay '67-'69	20.0	21.3	-	35	25	0.3	-	0.40	-	-	1e-7	1
Before '67	18.1	18.2	4	33	9	0.3	-	0.92	-	-	1e-7	1
Foundation												
From ground level 379m to el. 368 m	18.1	18.2	-	30	-	-	4÷1	0.92	0.3	0.035	1e-9	4
el. < 368 m	18.1	18.2	-	27	-	-	1	0.92	0.6	0.035	6e-10	1

where: γ: unit weight c' effective cohesion; ϕ' friction angle E': Young's modulus; υ' Poisson's ratio

A hydro-mechanical coupled approach, in large deformations and providing for mesh update has been set. The mesh update allows to represent the effect of buoyancy on the portion of dam foundation settling below the water table, thereby reducing the effective load on foundation soils. A non-associated Mohr Coulomb constitutive model for the embankment soils and a Cam-Clay hardening plasticity model (Soft Soil Model of PLAXIS library) for the clayey foundation soils have been selected.

Settlements calculated at a Node of the model near to the dam crest (Node A) and at a Node nearer to the topographic survey point CSLE2 (Node B), which is placed nearby the crest on the upstream face of the dam, show a satisfying increase rate in time respect to measures. It is observed an almost constant gap of about 0.40 m between measures and model results in the early 30thies, when measures began. It is believed that early settlements measured at dam's operation start may be attributed to imbibition of the embankment material at the first filling of the reservoir, a known effect which cannot be predicted by this model. By increasing the calculated settlements by the 0.40m gap a satisfactory correspondence with measures is achieved starting from year 1940.

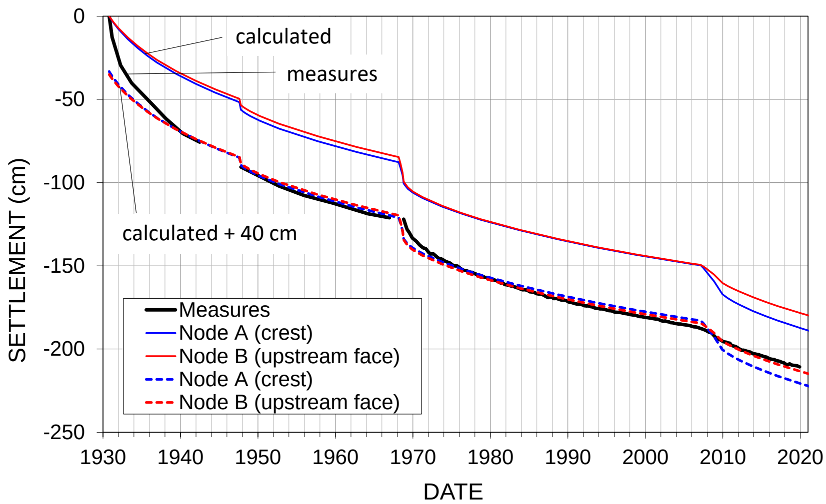

Figure 4 Calculated and measured crest settlement vs time. Dashed curves are obtained by adding to model calculated values (continuous line) a 0.40m settlement.

The current average degree of consolidation of the foundation soils, evaluated as the ratio between the developed and final settlement, is 74%; the final target value of 95% is achieved in about 160 years. According

to the model, the maximum final settlement predicted from construction at the foundation level is 3.42m including settlements developed before the start of the measures (year 1930).
The map of the residual overpressures predicted today, Figure 5, shows maximum values of approximately 59 kPa at a depth of 33 m from the ground level (elevation 346 m a.s.l.). The overpressures are zero in dam embankment. The bulb of residual overpressure is confined within the deeper clayey foundation soils.

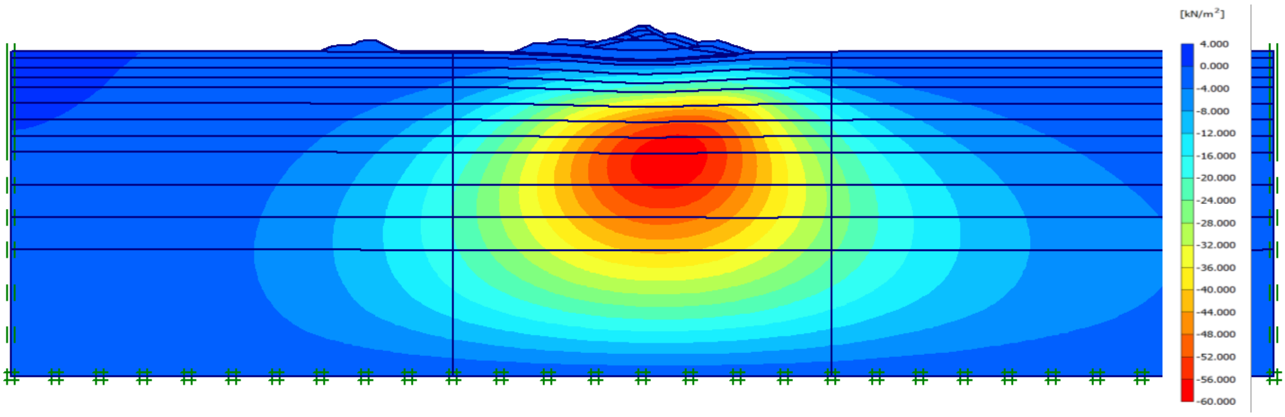

Figure 5. Contour map of predicted current residual water overpressures in Section E2 of the dam (PLAXIS). Maximum over pressure 58kPa. Pressure interval is 4kPa.

In 2001, Section E2 was equipped with a Casagrande piezometer in the foundation soil, placed at about 20 m below the dam foundation. In the period 2002-2006, prior to the last elevation of the dam, the height of the measured piezometric head was on average equal to 382.5 m a.s.l. very close to that predicted by this model. In 2010-2020 period, following the last elevation work, the average height of the measured piezometric surface (382.7 m a.s.l.) unexpectedly differs little from the value measured in the previous period, while the numerical model predicts a piezometric head rise 3m higher (385.6 m a.s.l.). Given the general consistency of the modelling approach with the experimental evidence provided by the dam and its foundation, it was concluded that the overpressure scenario calculated in the fine-grained soils of the dam foundation is a representative, and safe initial condition for the seismic analyses. The dynamic analyses have been however repeated under the scenario that overpressures in soil foundation have been completely dissipated before the seismic shaking to evaluate the sensitivity of results and effects on the seismic safety evaluation.

3. Seismic analyses

3.1. Earthquakes

Given the PSHA spectrum defined for a probability of occurrence of 5% which, according to NTD14 for strategic existing dams, corresponds to a return period Tr=1950 years, seven recorded earthquakes have been selected compliant to conditions such as the lithological quality of site (rock), the ruling momentum magnitude (Mw) - distance (D) couple defined by the site seismic hazard study, and the significant duration. The disaggregation analysis indicated the couple Mw =5.6÷6.3 D=0÷20 km for the assigned collapse ultimate limit state.

It was necessary to scale up the seven horizontal and vertical natural acceleration time histories to meet spectral compatibility to the target response spectrum. The range of vibrational periods identified to target the scaling was determined for both motion components as the estimated range between the fundamental period of the dam and its foundation, T_{o}=0.22s÷1.7s.

Two of the seven acceleration time histories thus determined, which originate from the AQG and AQK recordings, appeared to herald significant seismic effects, due to their intensity and duration. Once applied to the E2 dam section, they are characterized also by a remarkable spectral amplification at periods corresponding to the fundamental period of the foundation soil, Figure 6.

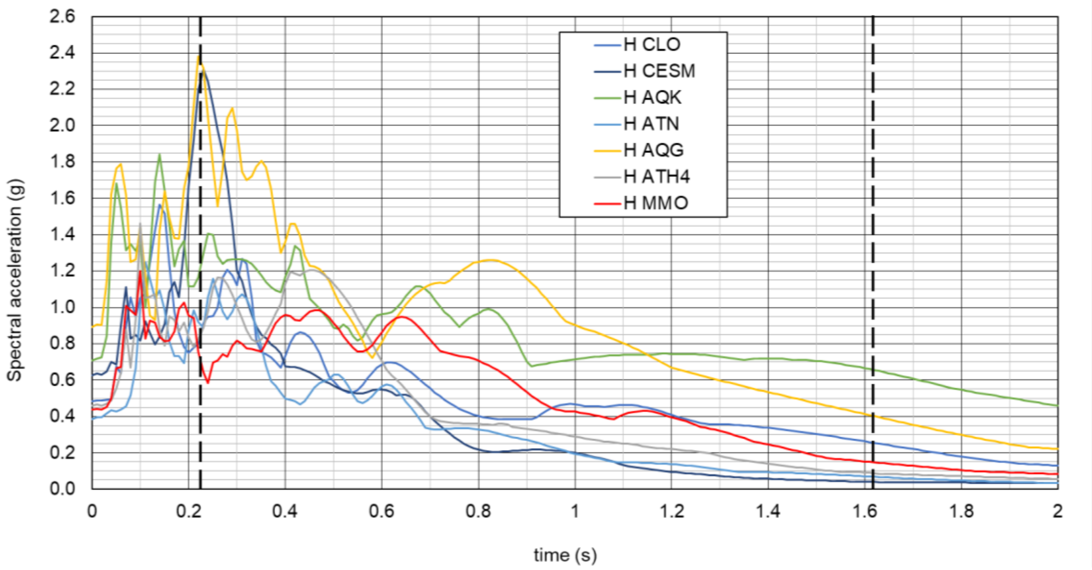

Figure 6. Horizontal response spectra of the seven earthquakes applied. The two dashed lines identify the fundamental period of the foundation (T=1.61s; f=0.62Hz) and that of the dam (T=0.22s; f=4.5Hz).

3.2. Seismic nonlinear coupled model

A plane strain finite difference model of the E2 section and the foundation down to the marly bedrock with the FLAC2D finite difference code.

The Masing nonlinear hysteretic damping model to reproduce cyclic elastic strain energy dissipation and the non-associated Mohr Coulomb constitutive model to describe plastic strain has been adopted. Small strain stiffness properties were determined by cross-hole geophysical tests performed on site, and from correlations with penetrometer CPT tests for the deeper soils, Table 2.

Dam and foundation cohesive soils have been assigned a decay law of dynamic shear stiffness properties (G_0) with the magnitude of induced shear deformation, based on the results of resonant column tests. Reference was made to curves published by Rollins et al., 1998 for granular materials (Type C, 'from ground level 379m to el. 368 m' in Table 2). The assigned evolution of shear stiffness and damping with shear deformation for the different materials has been described by the FLAC *sigmoidal 4* parameter correlation function.

Table 2. Small strain properties assigned to the dynamic model.

SOILS	$\gamma_{saturated}$ [kN/m^3]	V_s [m/s]	G_0 [MPa]	[υ] [-]
Dam				
Type A	20.8	126	29	0.46
Type C	23.3	240	137	0.46
Overlay '67- '69	21.3	200	29	0.46
Before '67	18.2	159	47	0.47
Foundation				
From ground level 379m to el. 368 m	18.2	170	54	0.48
274m< el.< 368 m	18.2	158 to 273 f(depth)	27	0.49
267m< el.< 274m	21.0	550	648	0.49
el.< 267m	21	800	1435	0.46

where Vs=Shear velocity G_0=shear stiffness at small strains

To model the damping that occurs even for very small deformations, a 0.1% Rayleigh viscous damping has been applied, calibrated on the fundamental frequency of the dam, 4.5Hz.

Seven dynamic analyses have been carried out, applying at the compliant base of the model the vertical and horizontal time histories off each earthquake. The dam is subjected to the maximum operational reservoir level (386m a.s.l.) and the effective stress and pore pressure distribution resulting from the static consolidation model. The response to the seismic motion is the average value of results obtained by each earthquake applied.

3.3. Liquefaction potential

The model adopted assumes that the soils of the foundation and the dam body are stable towards liquefaction.

The grain size distribution of the site's soils, including fine-grained ones, requires carrying out checks of susceptibility to seismic liquefaction according to NTC18. The analysis was carried out by methods (Boulanger and Idriss, 2014; Robertson and Wride, 1998) based on correlations with the SPT and CPT penetrometer resistance values. It was possible to conclude that the granular soils of the dam body and its foundation are not prone to liquefaction. As fine-grained soils are regarded, based on the data available, they can be identified as 'clay-like' behaving soils according to Boulanger and Idriss (2006) and, therefore, not prone to liquefaction.

3.4. Results

The stress-strain response of the embankment is characterized by a deformation style consistent with that identified in the literature for earth dams (Swaisgood, 2003; Swaisgood, 2014), identifies as lateral spreading, but with a noticeably higher downstream displacement compared to that observed upstream. Deformation involves the dam body and the entire upper 10m silty foundation soil, Figure 8.

Horizontal displacements, Figure 7, involve dam's downstream portion and are of the same order of magnitude, and generally higher than the corresponding vertical ones: the maximum average value at the downstream face is 0.52 m while the maximum average crest settlement amounts to 0.37m. The observed trend is determined by shear strain bands; the most important originates on dam's upstream face and reaches its downstream toe and affects the upper silty foundation soils. For the most intense earthquakes a band develops in the dam body at the contact between the original 1930 dam and the new portion added in 2006-2009; it is again directed downstream and emerges below the upper berm. A local instability affects also the upstream face involving a thin portion of material where actually a concrete slab, not modelled, is in place protecting and confining most of the upstream face of the dam. Settlements are concentrated in the intermediate portion of the dam (Figure 9), while at the dam's toe some uplift is observed.

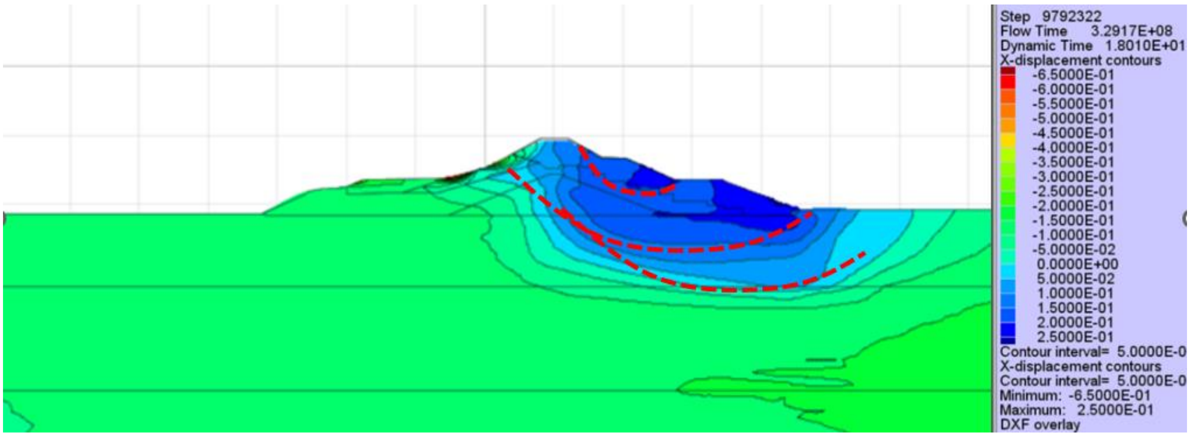

Figure 7. Contour of maximum horizontal displacements [m] and shear bands identifiers (in dashed red lines). CLO accelerogram, representative of the average effects.

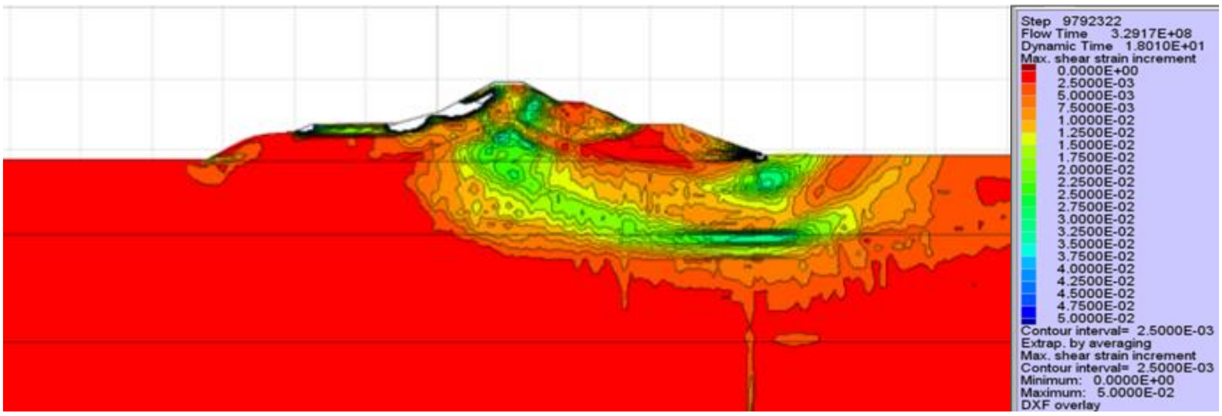

Figure 8 Contour of maximum shear strain increment [-]. CLO accelerogram, representative of the average effects.

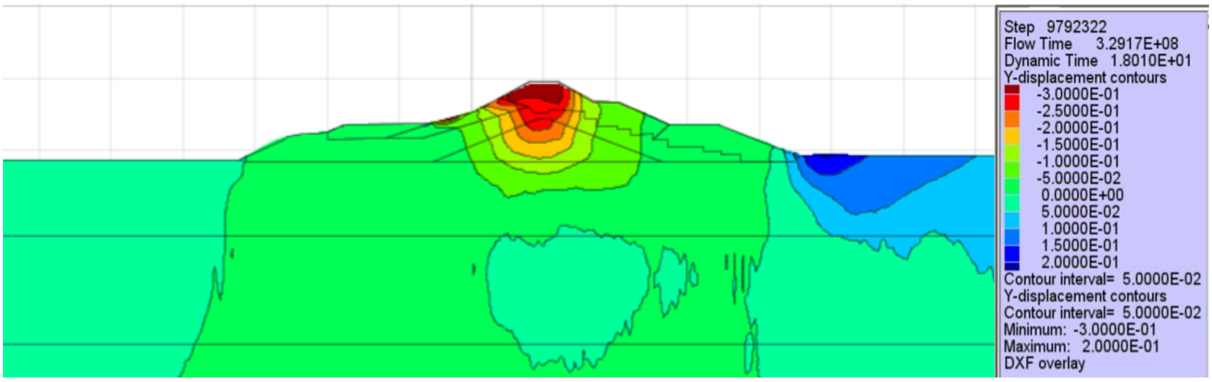

Figure 9. Contour of maximum vertical displacements [m]. CLO accelerogram, representative of the average effects.

The development of nonlinear shear strains and the effective damping of the thick foundation soils depress inertial effects in the dam body. The acceleration at crest (average 5.49m/s^2) is nearly de-amplified respect to that of outcrop (average $a_{max\ b}$=5.59m/s^2) and the amplification factor is lower than unity (F_{AH}=0.93). The result is also consistent with findings by Lanzo (2018) on amplification ratios vs. base acceleration of as much as 30 existing embankment dams, Figure 10, Figure 11.The amplification ratio F_{AH} respect to the dam/foundation interface (average $a_{max\ b}$=3.57m/s^2) is 1.5 compliant to acknowledged ratios (Aliberti, et al., 2019), Figure 10.

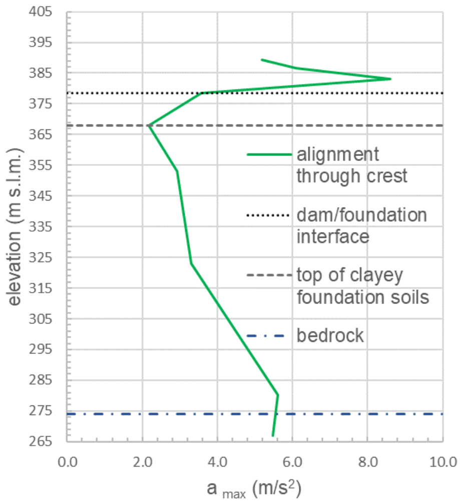

Figure 10. Envelope of maximum acceleration (average values) along the vertical alignment through dam's crest

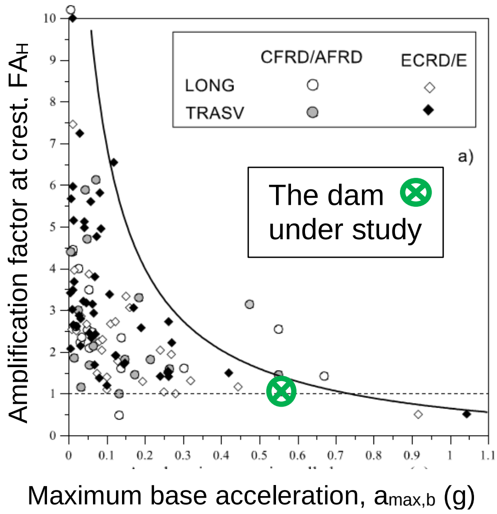

Figure 11. Amplification factor of acceleration at crest respect to base for dams. CFRD (Concrete Face Rockfill Dams); ECRD (Earthcore Rockfill Dam); E (Earthfill dam). After Lanzo (2018).

The shear stress vs. strain behavior sampled at single points of the model indicates that for this collapse ultimate state, hysteretic cyclic strains in the foundation coexist and are overcome by plastic irreversible strains in a large part of the upper foundation soil. Along the seismic induced shear bands, the plastic component prevails compared to the cyclic one. The shear stress-shear strain resulting in a point of the lower foundation soil (C3) and at the top foundation (C6) below the dam are given in

Figure *12* for two earthquakes: CESM, belonging to the group with minor seismic effects and AQG, one of the worse.

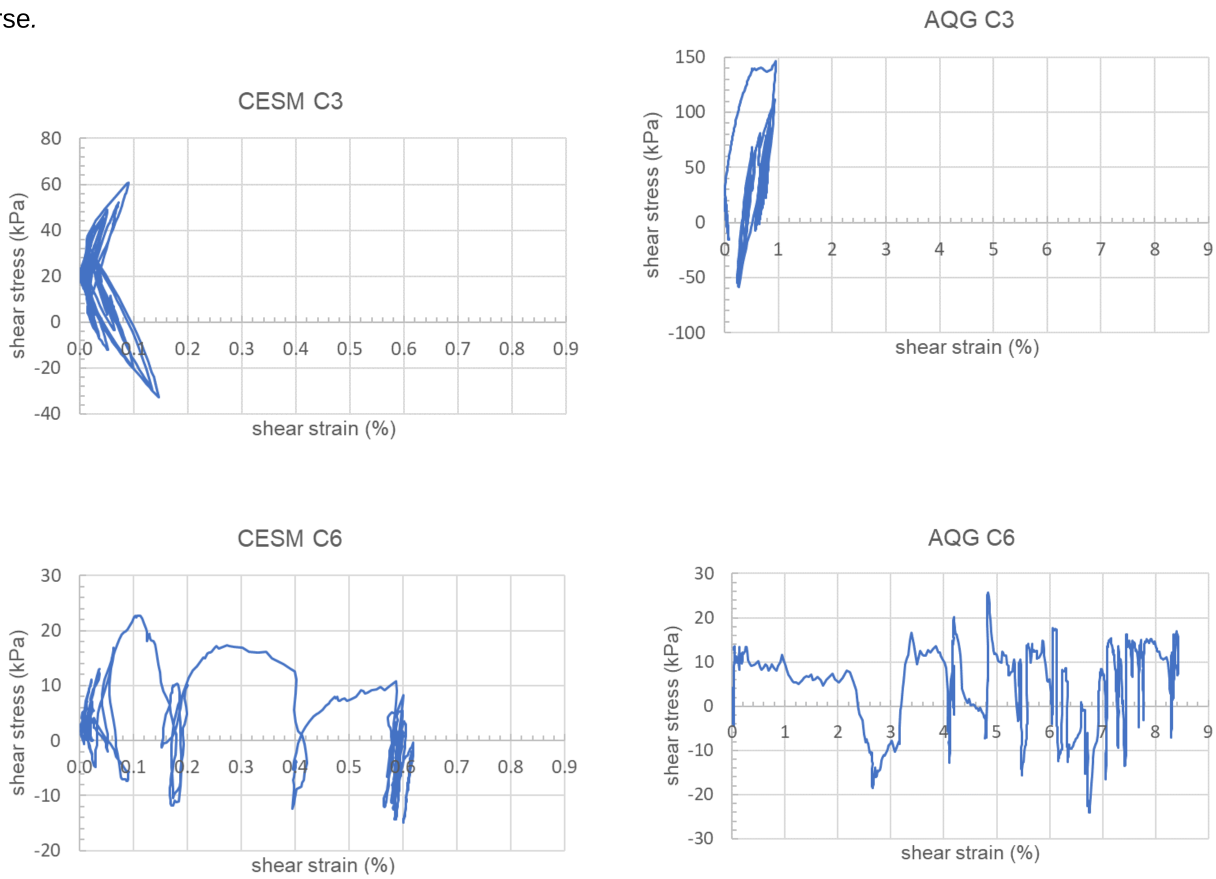

Figure 12. Earthquake AQG (left) and CESM (right) Shear stress vs shear strain during seismic shaking along crest's axis within the lower clay foundation (point C3) and at dam foundation interface (point C6).

The maximum shear strain values reached within the dam and its foundation lay in the range 0.1% to 15% for the seven earthquakes applied. These values are still compatible with the operational strength values assigned to the materials observed in laboratory tests; the final condition of the dam predicted by the model can therefore be still represented by the assigned strength. The settlements at dam crest are correlated with some well-known ground motion severity parameters associated to the input earthquakes, such as the Arias and the Housner intensities, Figure 13.

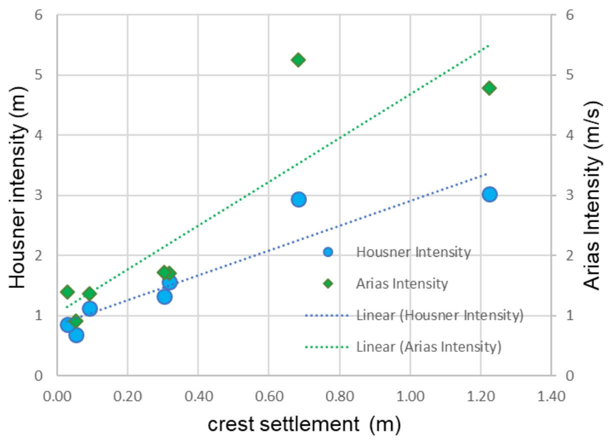

Figure 13. Crest settlement induced by the seven collapse limit state earthquakes as a function of Arias Intensity and Housner Intensity.

Crest settlement is an index of the potential damage suffered by the dam. The average settlement is in this case 0.37 m and is by 46% ascribed to the dam itself, while 42% to the upper foundation soils. It has been compared with thresholds defined by a collection of case studies (Swaisgood, 2003,2014; Pells and Fell, 2003; Sendir et al., 2007) where crest settlement could be associated with the damage occurred to earth dams, and thereby with conditions of post-seismic operativity. It could be concluded that the 0.37m is associated with severe damage which still does not induce the loss of the dam retention capacity of the reservoir water.

3.5. Analyses with no overpressures in dam foundation

The effects induced by the same earthquakes with an initial condition of full dissipation of overpressures within dam foundation leads to the same style of deformation, with effects of a lower magnitude. The dam and its foundation are subject to minor irreversible plastic deformations, Figure 14.

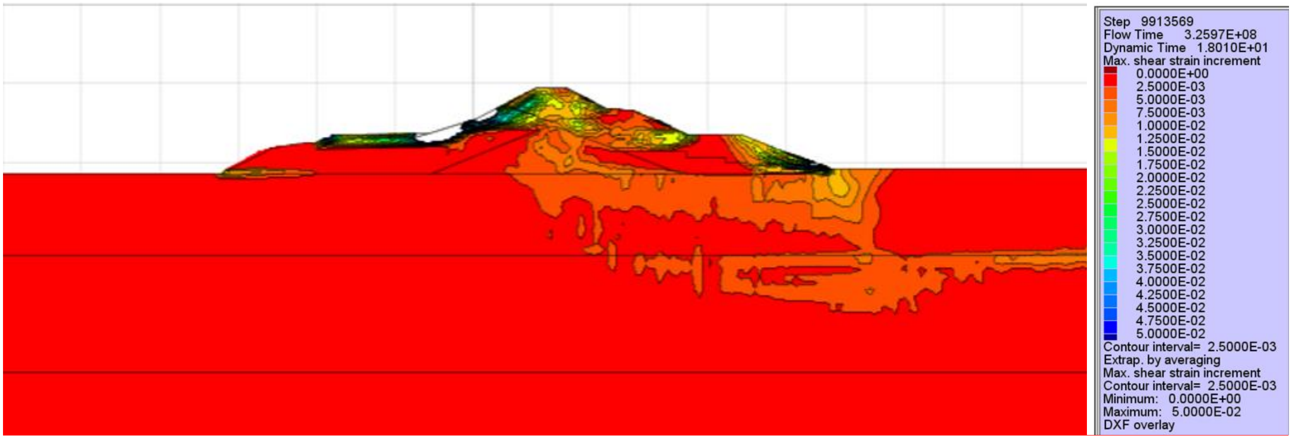

Figure 14. CLO accelerogram: maximum shear strain increment without initial overpressures.

In Figure 15 it may be appreciated how the initial condition with overpressures (left) for a CLO earthquake increases the portion of the model undergoing plasticity respect to the case without initial overpressures (right). In Figure 15 yellow dots indicate that plasticity has occurred and developed; pink areas have behaved according to the nonlinear elastic model applied.

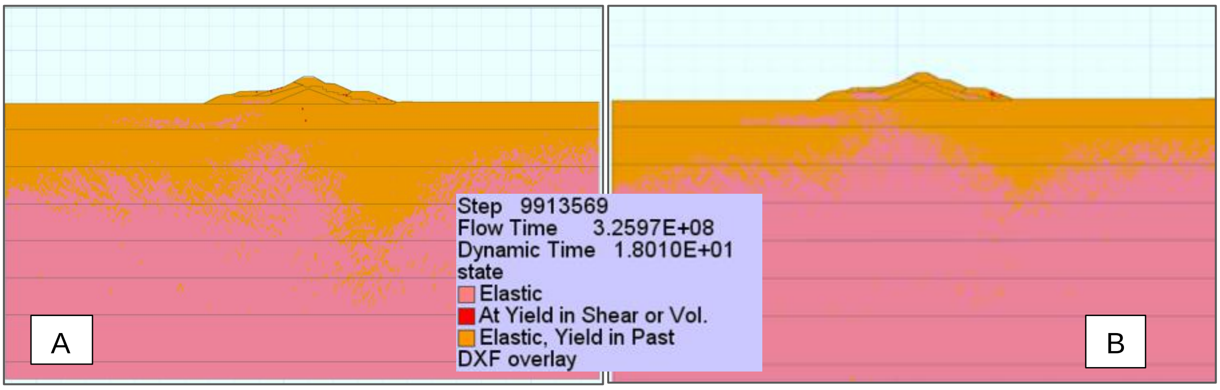

Figure 15. Distribution of the state variable identifying occurrence of plasticity in the model during the time history of CLO accelerogram. Case with initial water overpressures (A) and with zero overpressures (B).

The average crest settlement stands in this case at 0.16m, while maximum horizontal displacement at 0.28m, almost half of the corresponding values obtained considering initial overpressures in dam foundation. The result is due to the lower confinement induced lower strength of the foundation soil when it is still affected by consolidation related overpressures. It could be concluded that the 0.16m is associated with important damage which does not induce the loss of the dam retention capacity of the reservoir water.

4. Conclusions

Nonlinear seismic numerical analyses have been carried out on a plane strain model of an old but still operating earth dam, which still shows crest settlement after about 90 years. The latter is due to the nature of the underlying foundation soils, a thick layer of normal consolidated loamy clay. A nonlinear coupled model in large deformations has used to identify the distribution and values of overpressures in the clay foundation, by which it was possible to conclude that consolidation is still in place, 74% being already developed. This analysis allowed to calibrate the stiffness of the foundation and its permeability.

Seismic analyses have been run applying at the seismic bedrock, placed 104m below ground level, seven ultimate state earthquakes, as horizontal and vertical time histories, with average PGA=5.59m/s^2. Results have been averaged as foreseen by NTC18 and NTD14. The dam and foundation constitutive model is hysteretic nonlinear, associated to a plasticity Mohr Coulomb plastic criterion.

Two different initial stress conditions of the foundation soils have been considered: the presence of interstitial water overpressures representative of the present day 74% consolidation of the deep foundation soils and their complete dissipation.

About two times higher vertical and horizontal residual displacements resulted when water overpressures are considered, due to the lower strength of foundation soils induced by lower effective stresses. The style of final displacement is maintained. The higher shear strains are of irreversible plastic nature; they are organized in broad shear bands that affect the dam and the soil foundation. The resulting strain values are consistent with the assigned strength of soils.

Damage is important, but in both cases the retention capability of the dam is maintained.

5. Acknowledgement

The support of Ing. Fabrizio Pelli and Marino Gotti is gratefully acknowledged.

6. References

Adami S., Chemello P., Gigli P., Salvati S., Vanin V. (2010). Raising of an Earth Dam to Restore the Freeboard reduced by the Consolidation Settlements of the Foundation, *Proceedings of the ICOLD European Club 8th Symposium*, Topic C, Paper C-1:165-170, ATCOLD Ed., TU Graz Publ., Innsbruck, Austria, 22-23 September.

Aliberti D., Biondi G., Cascone E., Rampello S. (2019). Performance indexes for seismic analyses of earth dams, *Earthquake Geotechnical Engineering for Protection and development of Environment and Constructions*, 1066-1073, Silvestri e Moraci Eds., Associazione Geotecnica Italiana Publ.

Boulanger R.W., Idriss I.M. (2014). CPT and SPT based liquefaction triggering procedures, *Report No. UCD/CGM-14/01 Center for Geotechnical Modeling Department of Civil and Environmental Engineering, University of California*, Davis, California.

Charles J.A. (1986). The significance of problems and remedial works at British earth dams, *Proceedings of the BNCOLD-IWES Conference on Reservoirs*, Edinburgh, Paper 9:123-141.

Hunter G., Fell R. (2003). The deformation behaviour of embankment dams, *UNICIV Report No. R-416,* The University of New South Wales, Sydney.

Idriss I.M., Boulanger R.W. (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes, *Proceedings of the 11th International Conference on Soil Dynamics & Earthquake Engineering and 3rd International Conference on Earthquake Geotechnical Engineering*, pp. 32-56, Berkeley, California, USA, January 7-9.

ITASCA. FLAC-fast lagrangian analysis of continua, Minneapolis, USA Itasca Consulting Group, Version 8.0, User's Guide.

Lanzo G. (2018). Alcune considerazioni sulla rivalutazione della sicurezza sismica delle dighe di materiali sciolti, *Atti Conferenze di Geotecnica di Torino-XXV Ciclo*, Paper 11:1-43 Politecnico di Torino Publ. (in Italian), Torino, Italia. 8-9 Novembre.

Milli E. (1966). Considerazioni geologiche e tecniche sui movimenti di assestamento e sulla stabilità dell'argine in terra di Santa Croce, *Technical report,* ENEL Compartimento di Venezia (in Italian).

NTC18. (2018). Aggiornamento delle Norme Tecniche per le Costruzioni. Decreto del Ministero delle Infrastrutture e dei Trasporti*,* Gennaio 17, G.U. n.2018; 42 (in Italian).

NTD14. (2014). Norme Tecniche per la progettazione e la costruzione degli sbarramenti di ritenuta (dighe e traverse)*,* Giugno 26.G.U., vol.156 (in Italian).

Pells S., Fell R. (2003). Damage and cracking of embankment dams by earthquakes and the implications for internal erosion and piping, *Proceedings of 21st Congress on Large Dams, ICOLD, Q83-R17,* ICOLD Paris Ed., Montreal, Canada*.*

PLAXIS2D – Version 9.0 User manual (2008) R.B.J.Brinkgreve, Broere W., Waterman D. Eds.

Robertson P.K., Wride C.E. (1998). Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal*, 35(3): 442-459.

Rollins K.M., Evans M.D., Diehl N.B., Daily W.D. (1998). Shear modulus and damping relationships for gravels*, Journal of Geotechnical and Geoenvironmental Engineering*, 124(5): 396-405.

Sendir S., Sato J.E., Towhata I. (2007). Shaking table tests on residual displacements of earth dams for their performance-based design, *Proceedings of International Workshop on Earthquake Hazard and Mitigation*, Guwahati, India, 7-8 December.

Swaisgood J.R. (2003). Embankment dam deformations caused by earthquakes, *Proceedings of the Pacific Conference on Earthquake Engineering*, Paper No. 14: 1-8, Auckland, New Zealand.

Swaisgood J.R. (2014). Behavior of embankment dams during earthquakes, *Journal of Dam Safety,* 12(2): 35-44.

Tedd P., Charles J.A., Holton I.R., Robertshaw A.C. (1997). The effect of reservoir drawdown and long term consolidation on the deformation of old embankment dams, *Géotechnique*, 47(1): 33-48.