Rehabilitation design of Acciano rockfill dam after the September 1997 earthquake

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SYNOPSIS. The Acciano rockfill dam was originally designed without taking seismic action into account. The area where it is located is now classified in the 2nd seismic zone, according to the current Italian regulations. On September 26th 1997, an earthquake of magnitude M_W=5.5, one of the largest seismic events of the last 20 years in Italy, occurred in that area and caused some visible damage to the dam. Subsequent investigation programmes and structural assessments were carried out to evaluate the residual safety margins of the dam in order to identify possible rehabilitation provisions to comply with the Italian standards for seismic design.
This paper describes the evaluation of the post – earthquake condition of the dam and outlines the assessments to validate the rationale of the rehabilitation project.

THE DAM

The Acciano rockfill dam is located in the centre of Italy (Perugia Province); it was built between 1976 and 1980 to impound water for agricultural use during period of deficient supply. The reservoir capacity is 1.7 million m^3.

The Acciano dam has a zoned embankment with a curvilinear axis; an internal central impervious core and external rockfill shoulders. The embankment is characterised by three berms, at different elevations: two are located at the downstream side and one at the upstream. The structure reaches a maximum height of 28.5 m, and it is 182 m long along the crest, at elevation 531.5 m a.s.l. The faces have a slope (Fig. 2) equal to 1:1.4 from the crest to the first berm (el.513 m), and 1:2.5 in the bottom part. The shoulders were built by dry compaction of two different materials: the zone above el. 513 m with rockfill and the part below with a gravelly sand. The core is of silty-clay.

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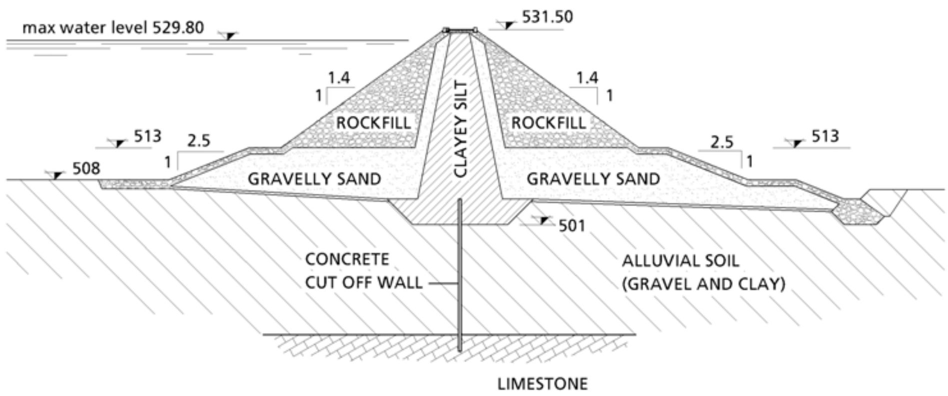


Figure 1. The main cross section

The embankment is founded over an alluvial soil, which is below the main section, about 20 m thick. Between the main section and the shoulders, the thickness of this alluvial layer decreases to zero and the dam is directly founded on a marly limestone rock. To reduce seepage, a concrete diaphragm wall 0.6 m thick was built below the core to bedrock. Cement grouting was carried out to enhance the hydraulic performance of the rock foundation, in particular at the abutments.

At the right abutment the rock is fractured to locally highly fractured: a grout shield 60 m long from the crest elevation and 30-40 m deep into the abutment rock mass was therefore added.

The dam has two outlet works, both located within the rock on the left abutment: a bottom outlet (el. 506.9 m a.s.l) and an overflow spillway (el. 528.50 m a.s.l.) which can release, at the maximum water level, a discharge outflow of 38.8 m^3/s and 86.2 m^3/s respectively, which overall corresponds to the 1000 year flood. The bottom outlet is a tunnel of precast reinforced concrete that, in its central part, lies within the dam body. In this part the discharge gallery is closed by two sliding gates, which are operated from the control tower located in the reservoir near to the left abutment.

The monitoring system comprises: a topographic collimation to observe planimetric displacement evolution and settlement of the crest and at the downstream berms; eight Casagrande piezometers placed downstream respect to the dam body and into the left and right foundation rock. The monitoring records did not show any anomalous response before the earthquake took place. The dam had been operational for 11 years and the reservoir reached 522 m a.s.l. water level before the earthquake struck.

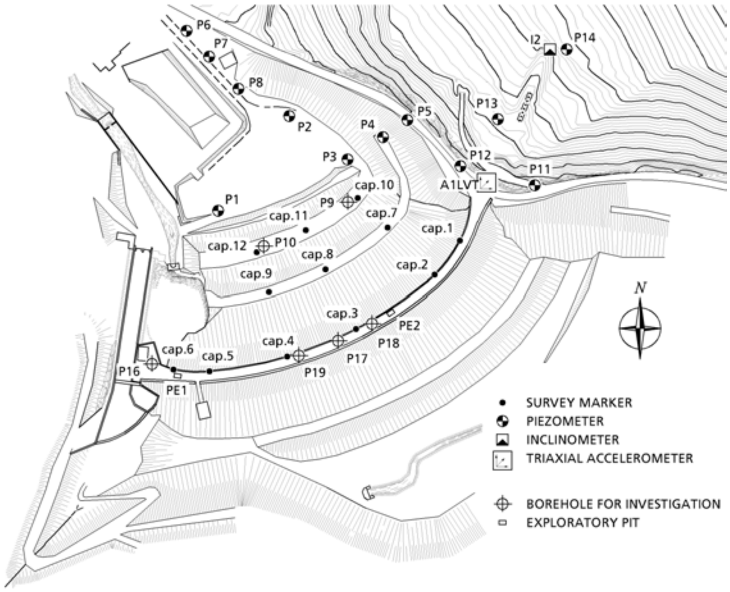


Figure 2. The instrumentation network (partial) and boreholes for investigation

OBSERVED EFFECTS OF THE EARTHQUAKE TO THE DAM

On September 26th 1997, an earthquake of magnitude Mw=5.5 occurred in the area of Nocera Umbra, one of the largest seismic events of the last 20 years in Italy, causing some visible damage to the crest structure.

Shortly after the event some wide cracks appeared in the asphalt paving of the crest: two longitudinal cracks, close to the crest edges, and two transverse ones, near the extreme ends of the dam, were observed (Fig.4). The upstream crack appeared the most significant and spanned the entire crest. Settlements up to 15 cm of the rigid reinforced concrete edges of the crest road were measured in the main cross section, where also a lateral spreading of about 10 cm was also found. The downstream lower berm settled about 2 cm. In the following month, settlements of the dam increased by only a small percentage, mainly due to the dissipation of the excess pore pressure in the foundation soil (a 0.05 MPa increase was measured in the alluvium shortly after the event). The following Figure 3 shows the chronological measurements of displacements on the crest and on the two berms.

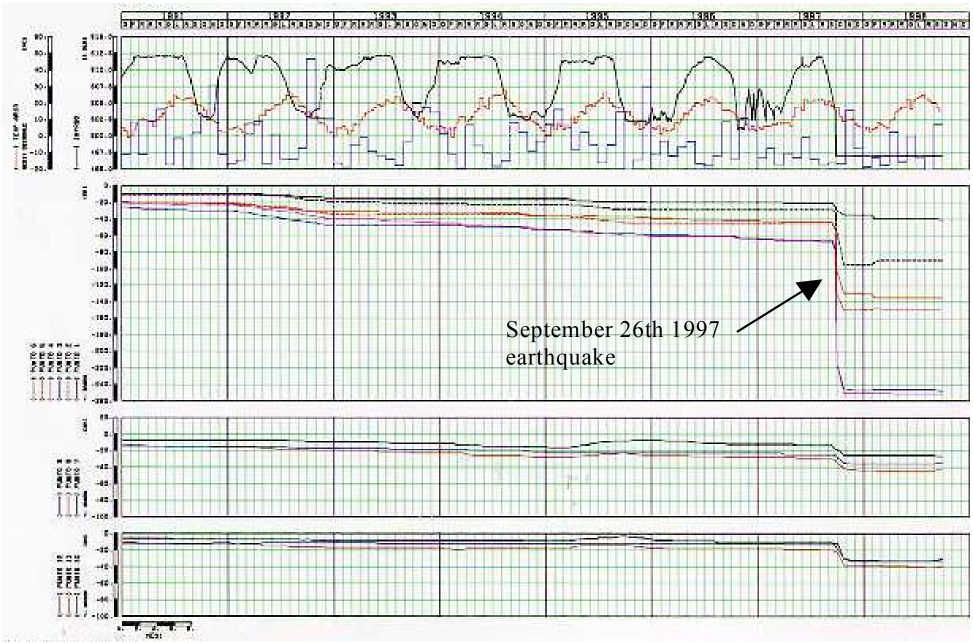


Figure 3. The chronological diagrams of displacements on the crest and berms (Midas® - Ismes Software)

In the following table, settlements of the crest and the two berms before and after the earthquake are reported.

Table 1. The measured vertical displacement values [mm]

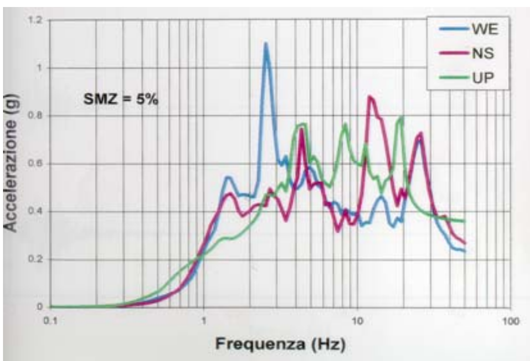
Marker		from jan 1986 to sept,26 1997	After Earthquake	%
Crest	P1	**-25**	**-16**	64
	P2	**-51**	**-84**	165
	P3	**-77**	**-171**	222
	P4	**-74**	**-177**	239
	P5	**-52**	**-97**	187
	P6	**-32**	**-58**	181
Berm 1	P7	**-18**	**-9**	50
	P8	**-32**	**-10**	31
	P9	**-26**	**-9**	35
Berm 2	P10	**-13**	**-18**	138
	P11	**-22**	**-17**	77
	P12	**-15**	**-18**	120

Two square exploration pits 3m wide (PE in Figure 2), dug one year later, did not show any evidence of deepening of the two transverse cracks. No extension of the observed crack path and of complementary damage has been observed.

Figure 4. The damage on the crest

CALCULATED EFFECTS OF THE EARTHQUAKE GROUND MOTION

A large number of stations of the national accelerometer network were operating in the Umbra-Marche region. The nearest to the Acciano dam was that of Nocera Umbra, 11 kmaway. From that record, a site spectrum has been derived as input at the dam foundation for use in structural assessments. Processing of the Nocera Umbra records was made adopting the Sabetta-Pugliese attenuation laws. Site amplification data, based on local measures of micro tremors, gave evidence that no local amplification need to be incorporated. The resulting response spectra (horizontal and vertical component) are depicted in Fig. 5 together with the corresponding accelerograms.



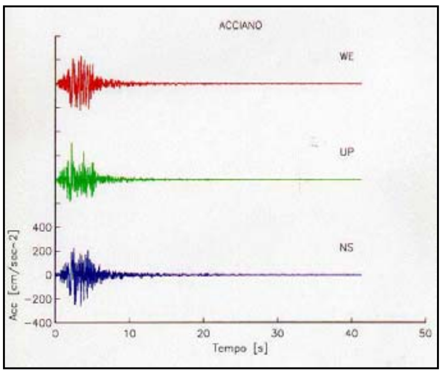


Figure 5. Calculated spectrum and time history below the dam

It may be appreciated that the highest accelerations concentrate within the interval 1.5-10 Hz, where the natural frequencies of the dam also fall. The dam crest could suffer accelerations up to 1.1 g.

A Finite Element (FE) model (Fig. 6), representing the dam body and a portion of the foundation rock, has been set up to determine: the actual distribution of accelerations within the dam body; the principal dynamic properties which are vibration modes and the associated frequencies. The material model is elastic with assigned decay law of material properties (damping and shear modulus). The parameters incorporate the stiffening contribution of the interstitial water and of the short duration of loading.

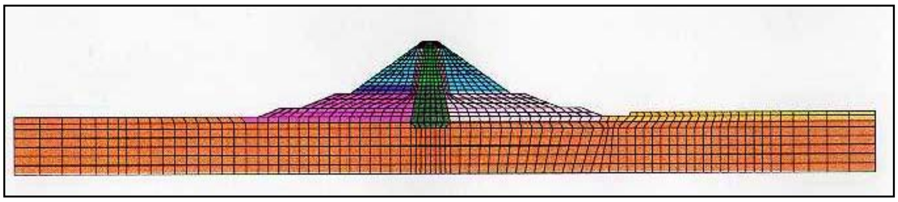


Figure 6. The F.E. mesh for dynamic analyses

The calculated accelerograms were applied at the base of the model. The earthquake (40 s duration) can reduce significantly the stiffness of dam materials: straining reaches the order of magnitude of about 0,01%, a threshold for a significant reduction in stiffness for many materials. This threshold was confirmed by resonant column tests run on specimens taken from the core material.

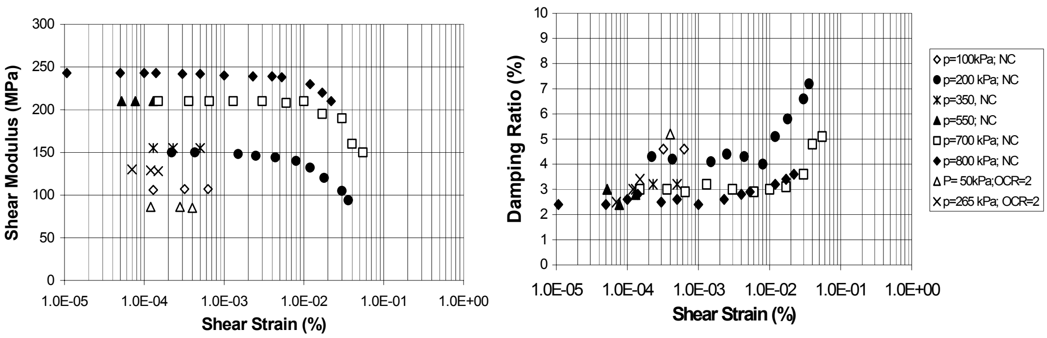


Figure 7. Shear modulus and critical damping by resonant column tests

As a further evidence of the decay, the modal analysis calculated 3.67 Hz for the first natural frequency before the seismic input is applied and 2.60 Hz at the end.

SEISMIC STABILITY EVALUATION

The evaluation of the safety margins after the shock were made by applying the Newmark method, which determines, within a limit equilibrium approach, the residual sliding displacement of a given portion of the dam body suffering a given acceleration record. In this case the accelerograms obtained by processing the actual records have been applied.

The most critical surfaces were determined again by the limit equilibrium method by Bishop (simplified with use of circular potential sliding surfaces), where the Italian regulatory seismic input was applied as a pseudo-static inertial contribution, based on a constant acceleration of 0.07 g. The approach is consistent to that used by the designer, who took into consideration static loads only.

In both assessments the same physical and mechanical material properties were used for the materials in the dam body and in the foundation. They are given in Table 2 and result from the design phase as well as from tests.

Table 2. Design physical and mechanical parameters

Material	γ_d **[kN/m^3]**	**c' [kPa]**	**φ' [°]**
Clayey silt	16.7	30	25
Rockfill shoulders	19.6	0	40
Gravelly sand	21.6	0	35
Alluvial soil (gravel and clay)	17.7	0	30

The water level was taken at the maximum operating (529.8 m a.s.l.) for conventional checks and to the much lower one (514.0 m a.s.l.) present when the earthquake took place.

The assessments confirm that the dam for most critical surfaces complies with the Italian regulation (1.4 and 1.2 for static and seismic conditions respectively). Critical surfaces located in the upper portion of the embankment have a reduced safety margin for static loads (1.17 compared to 1,4) and near to 1 for the seismic condition.

A thorough visual inspection of the embankment slopes did not show even local which could be attributed to sudden unstable conditions. It may be deduced that a higher shear resistance is available at the surface, where confinement due to overburden is a minimum (see Fig 8).

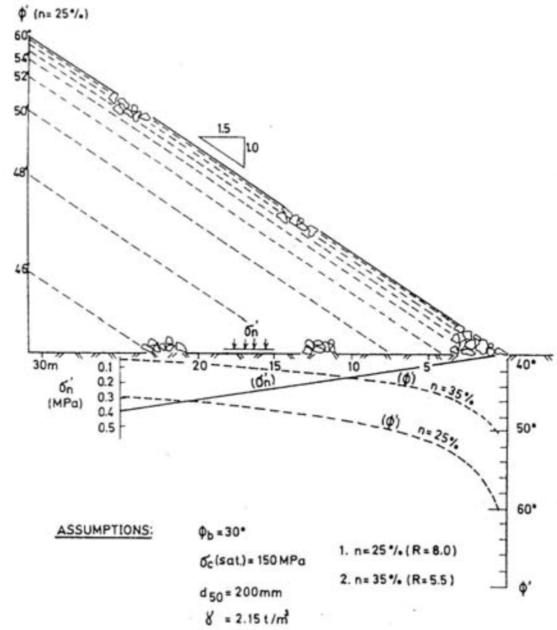


Figure 8. Available shear strength, as friction angle in a rockfill embankment approaching to the surface. σ'_n is the confining normal stress. Source NIT report

The effects of the actual earthquake have been evaluated by the modified Newton method (by Makadisi-Seed) The Newmark method allows the evaluation of the permanent displacement of a slope subjected to an earthquake, assuming that the motion occurs along arcs or planes as in the usual static analysis of stability. Direct integration has been used to compute the magnitude of the dynamic motions produced by the earthquake. The fundamental parameter in the analysis is the critical acceleration Kc, i.e. the pseudostatic acceleration corresponding to a unit safety factor against sliding in the limit equilibrium analysis.

The following steps were taken:

- For each critical surface the critical acceleration was determined at the centre of mass of the given portion of the dam body, defined as that bringing to unit the safety coefficient against sliding. This is the conventional value for unstable response to occur under the form of progressive displacements.
- By checking the critical value with the actual horizontal acceleration record of the given earthquake instant where the critical acceleration is exceeded are determined and the corresponding displacements cumulated.

The maximum critical acceleration value is calculated for critical surfaces originated nearby the crest and ending at the upstream portion of the embankment, and among them for those having the toe below the water table. In any case all the surfaces examined could withstand pseudostatic horizontal accelerations in excess of 015g.

The residual displacement reaches 15 cm, for the most vulnerable surface (Fig. 9) while, for most cases, such displacement is lower than 6 cm.

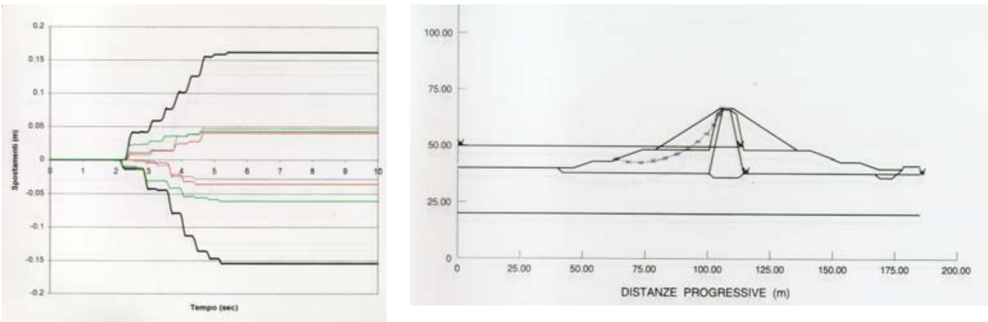


Figure 9. Residual displacements and a potential failure surface

The values obtained are modest if compared with threshold suggested in the literature (Lambe and Whitman for earth dams, NIT report), which are metric.

It may be concluded that the cumulative damage is moderate, and concentrated on the crest area. Considering that the actual earthquake can be associated to a SSE (rare), with a 475 year return period, the overall performance of the dam confirms the high safety margins incorporated in the Italian regulation even for static loads.By processing safety condition in terms of allowable acceleration, Paoliani concluded that millimetric to centimetric displacements can be associated with a OBE to SSE earthquake, which is characterised by PGA's well in excess (0.20-0.28g respectively) of that required by the Italian standards for dams (0.07g at the Acciano dam site). It was concluded that there were grounds for rehabilitating the dam.

EXPERIMENTAL ASSESSMENTS

Evidence of the state of critical materials in the dam body and foundations were acquired by an extensive site and laboratory investigation in support of the rehabilitation design. Emphasis was given to the core material, in order to ascertain its strength and stiffness properties. Self-healing and self-sealing capabilities, which mainly relate to the amount and the mineralogical composition of the clay fraction in the core material were of specific interest. They can counteract the possible formation of damage under the form of shear bands/microcracks, affecting therefore the fundamental barrier function of the core. Tests were also performed on the foundation materials, the alluvial soils and the rock itself.

Some *in situ* standard penetrometer and permeability tests (Lefranc and Lugeon) provided a framework for data obtained in laboratory on specimens. Test boreholes have been drilled from the crest to the foundation through the core, and some from the downstream berm (see Fig.2).

The laboratory test programme for specimens from the core consisted of: triaxial consolidated drained and undrained tests to derive frictional properties and undrained cohesion; direct permeability tests on undisturbed specimen to evaluate a possible anisotropy; load controlled oedometer tests to derive stiffness and permeability, to assess overconsolidation, and, finally, resonant column tests, for further assessment of dynamic properties to determine effects on the dam induced by the earthquake.

A few unconsolidated undrained tests were run on specimens of the alluvial foundation layer resting under the dam body. Uniaxial load tests were run on the foundation limestone rock to derive its mean strength properties.

RESULTS

The core is of a silty-sandy clay with gravel. The clay fraction increases, reaching 50% at increasing depths into the core. The upper material is much more dominated by the sandy and gravel fraction (Fig. 10).

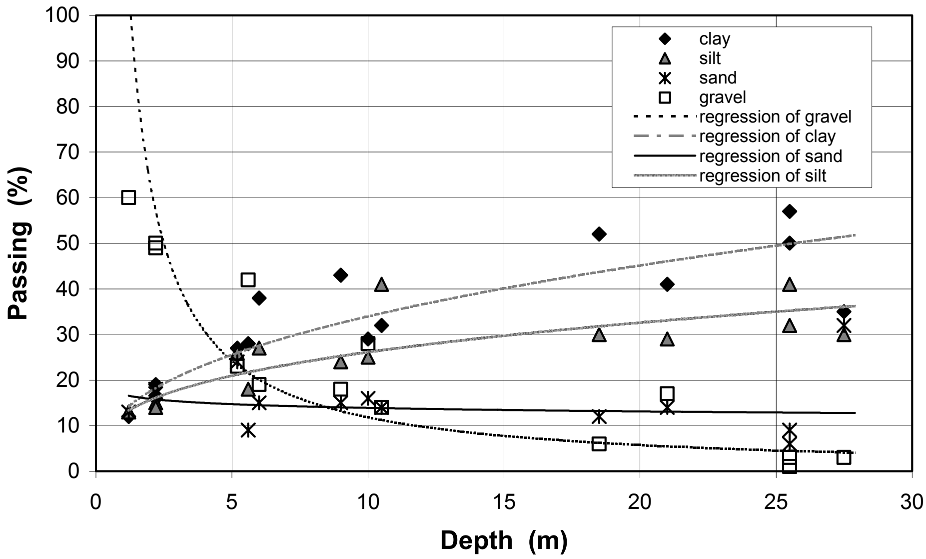


Figure 10. Material composition of the core by grain-size determination.

The clay is inorganic, of plasticity ranging from medium to high, CL (top portion of the core), CH (deeper core) according to Casagrande classification. The consistency index, 0.96, reveals a solid-plastic clay.

According to pocket penetrometers and S.P.T. tests, the core material is of good consistency, stiff in the upper part to very stiff in the deeper one, corresponding to a uniaxial compressive strength (UCS) ranging from 200 to 300 kPa.

The core material was found to be nearly saturated. The average water content is 24% and the wet density of the deeper core material is 19.3 kN/m^3, reaching 20-21 kN/m^3 in the upper, more gravelly portion. The core shows, from oedometer tests, some light overconsolidation. The estimated preconsolidation stress ranges from 400 to 500 kPa. Strength properties from drained and undrained consolidated triaxial tests run at several confining stresses in normally consolidated conditions can be described, according to the Mohr Coulomb criterion, as c' = 45 kPa and φ' = 23°. The same tests run on core upper material suggest a more marked frictional response, c' = 7 kPa and φ' = 34°, in Figure 12.

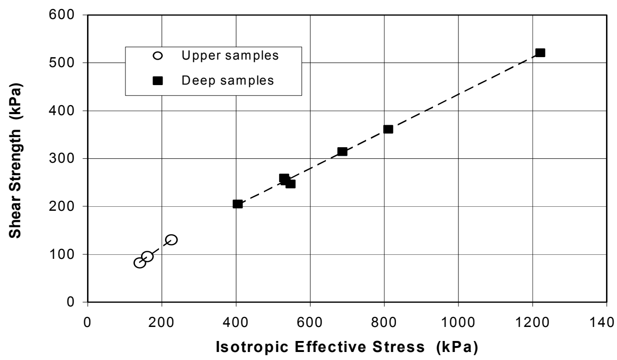


Figure 11. Shear strength vs. isotropic effective stress

Tests run on slightly overconsolidated specimens (OCR=2 corresponding to in situ conditions at the given elevation) show some, peak resistance. All considered, the overall strength envelope for the core can be defined by c'=15-45 kPa and φ'=27-23°, values adopted in the design stage are still represented.

Overconsolidation is weak and the general stress-strain response is quasi-ductile. Specimens collapse in triaxial tests at shear strains from 3.5% to 5% for overconsolidated specimens, and in excess for normally consolidated.

The undrained cohesion (c_u) has been found remarkably high, about 250 kPa. Some overestimation respect to values obtainable by other tests is due to the test bias. Upper specimens, which had to be reconstructed and consolidated to separate the gravelly fraction from the cohesive one gave lower, but still significant values (80-150 kPa).

The oedometric modulus is about M=30-15 MPa, the latter in the overconsolidated range within the prevailing stress state (up to 600 kPa). The consolidation coefficient is estimated as between 1×10^{-8} and 5×10^{-8} m^2/s. The shear modulus varies from 80 MPa a 250 MPa with increasing confinement stress (Fig. 12).

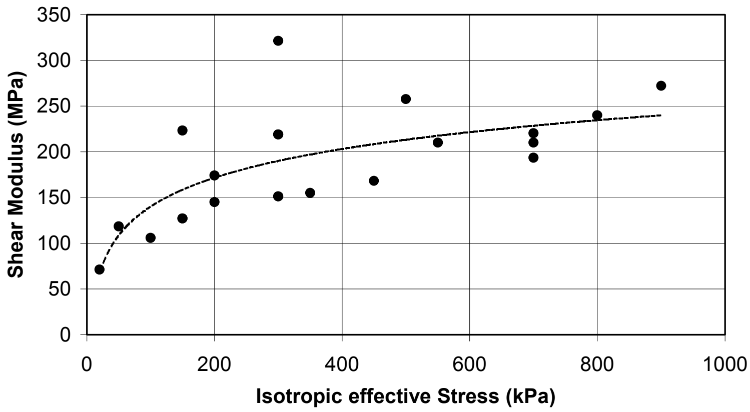


Figure 12. Shear modulus vs. isotropic effective stress by different tests.

Pin Hole test results indicate that the core material is not dispersive. The test has been run with distilled water; determination of chemical species in water taken at the dam site does not reveal any potential adverse effect with the clay minerals stability. Hydraulic conductivity from triaxial test run at several confining stresses and, indirectly, from oedometers, varies, within the stress range of interest (up to 600 kPa) from 1×10^{-10} (extrapolated) to 17×10^{-11} m/s (Fig. 13). No significant anisotropy of hydraulic conductivity has been observed. Lefranc tests indicate values from 5.0×10^{-9} m/s to 5.0×10^{-8} m/s for the core material.

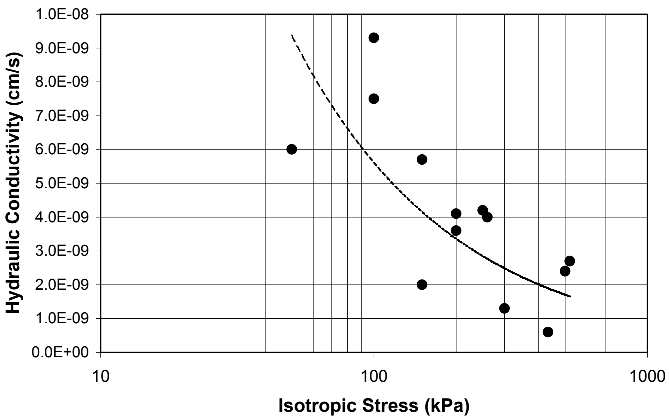


Figure 13. Hydraulic conductivity vs. the isotropic stress

Two principal soils have been identified in the foundation, one with a granular character, the other more cohesive. The latter has displayed undrained cohesion values of 70-80 kPa, and hydraulic conductivity (by Lefranc tests), similar to the average one of the core, 1.0×10^{-9} m/s.

DESIGN CONCEPT FOR REHABILITATION

The assessments of the impact of the Marche-Umbria 1997 earthquake and the properties of the core and cohesive foundation materials investigated by *ad hoc* laboratory tests and by *in situ* determinations revealed that the core retains satisfactory properties after the earthquake shock, which are very near to those adopted in the design phase. It is therefore justified to proceed with a seismic rehabilitation essentially based on providing additional confinement to the core, and higher margins of local safety against sliding to the slopes, by reshaping the embankment slopes. The above objectives can only be achieved by the addition of rockfill material.

The freeboard has been increased to 0.75 m, and the slopes shaped to reach l:2 upstream and 1:1.8 and 1:2.2 downstream. It is proposed to rebuild a small portion of the top of the dam body (the first two metres) . Some overburden is provided, at the downstream toe, to increase the factor of safety against piping.
Grouting is proposed to enhance the performance of the foundation materials, soils and rock, against seepage.

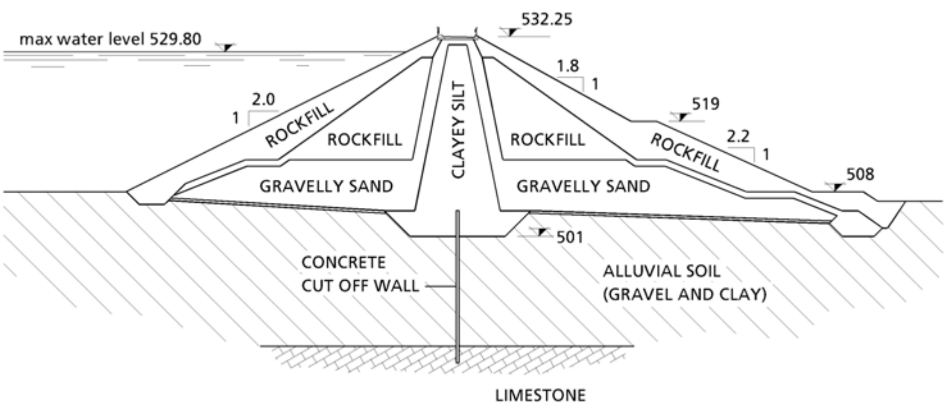


Figure 14. Rehabilitation design: main cross section

Data obtained from the tests allowed all the necessary analyses in support of the remedial works. These were basically:

- Stability checks of the dam body and foundation to comply with Italian standards.
- Seepage evaluation and checks for piping.

- Stability of structures within the dam body, such as the outlets, etc., and seismic checks on the structure of the gate tower.
- Check of punching of the concrete diaphragm into the clay core, during the earthquake motion and possibly reactivated by the consolidation effect of the alluvial soils due to the weight of the new rockfill material (more than 55000 m^3).

The design has proved compliant with regulations with regards to the above effects.

The assessments confirm that the rehabilitation project significantly improves the static and seismic safety margin with respect to the original configuration, varying from 40% for the surfaces located in the upper portion of the shoulders to 25% for deeper surfaces.

The evaluation of settlement of the dam, in the short and long-term conditions respectively, showed a maximum value of 6 cm and 11 cm. The maximum shear strains induced in the material core is 0.3% low compared with deformability of core material as observed in triaxial tests. The core material is able to withstand the overburden without displaying global or local damage effects. The check for local punching, before and after the seismic event, indicates that maximum shear stress (75-80 kPa) keep well below the undrained cohesion of the core.

CONCLUSION

The Acciano dam has provided evidence that factors of safety for seismic design incorporated in the Italian dam design code can effectively provide a significant seismic resistance capacity. The damage is confined to the crest of the dam and the condition of the core has remained suitable to allow remedial works to be implemented.
Such conclusions could only be made following a comprehensive testing and modelling programme outlining the critical role that such methods can play in assessing the current safety condition of existing dams.

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