Design and seismic stability of fill works in a large powerplant

Critères de la stabilité sismique pour ouvrages en remblais

T. SILVESTRI, ENEL, Naples, Italy
F. RICCIARDI, ENEL, Naples, Italy
M. ROSSETTI, ENEL, Naples, Italy

SYNOPSIS The dimensioning criteria and the stability analysis for some of the most important fill structures in a large pumping storage powerplant are described. The plant is located in a site classified, according to the italian by-laws, as of medium seismicity.

INTRODUCTION

The ENEL (National Board for the Electrical Energy Supply - Italy) has the Presenzano pumping storage powerplant, which has a maximum output capacity of 1000 MW, under advanced construction (Figs. 1,2). The daily generating-pumping cycle will operate between an upper and a lower reservoir, both having an active storage capacity of 6.0 millions cubic meters. The maximum gross head between the reservoirs is 495.50 meters. The plant is located at a key point in relation to the big cities of Rome and Naples.

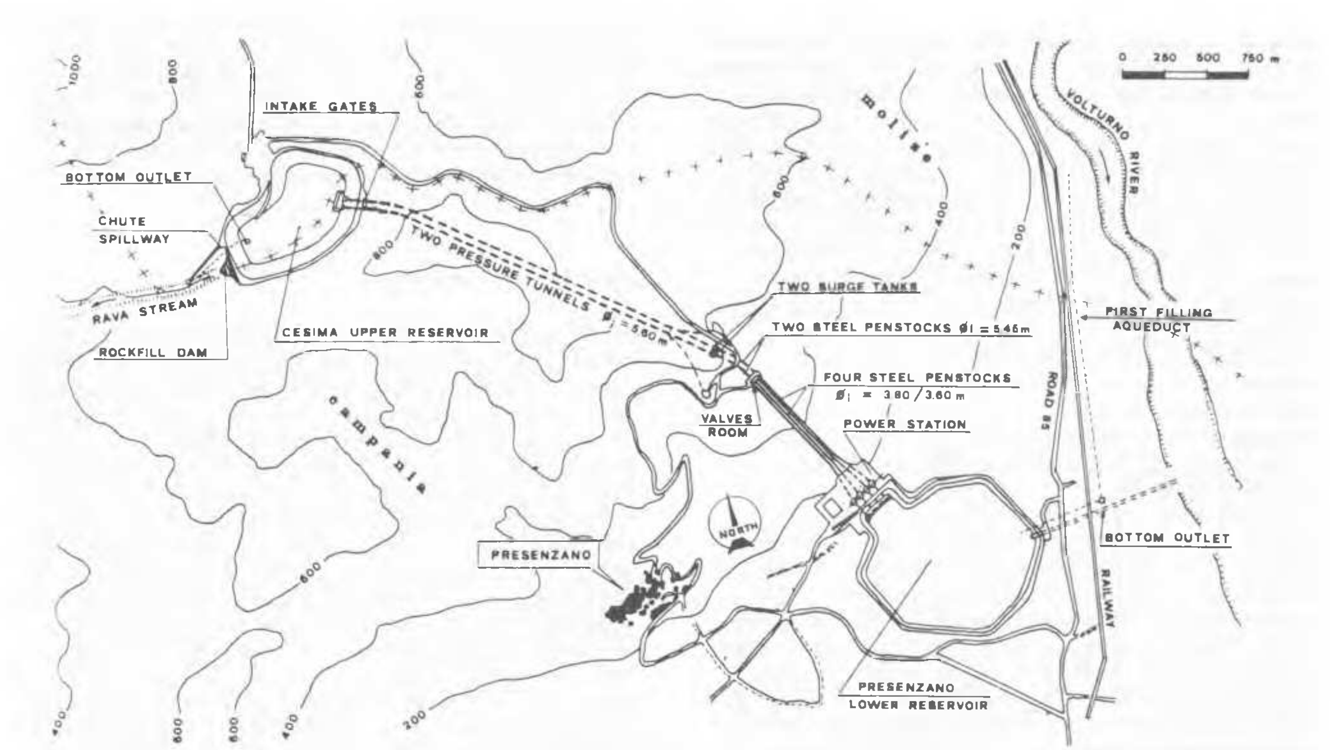

Fig.1 - Layout of the Presenzano Powerplant.

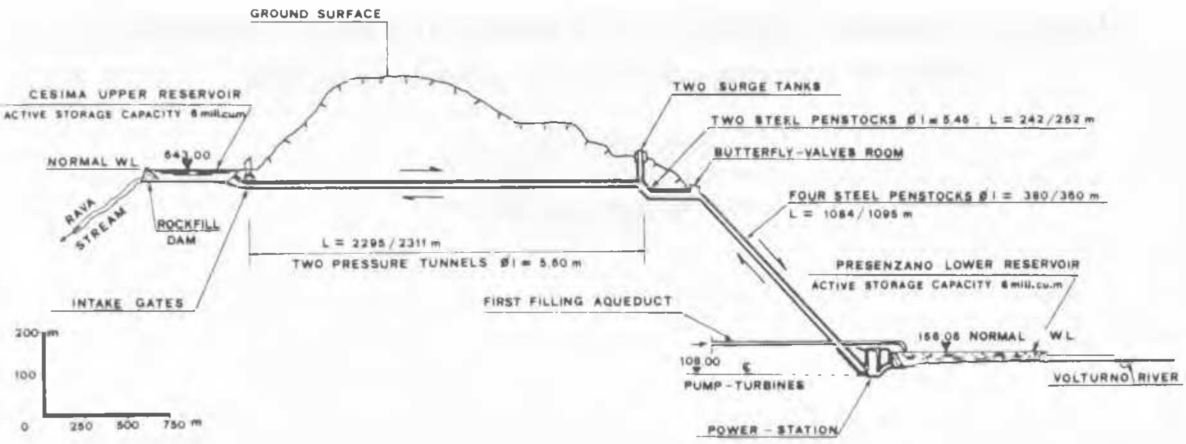

Fig.2 - Presenzano Powerplant - Longitudinal Profile

RESERVOIRS FEATURES

The upper Césima reservoir will be realized intercepting the outlet of a karstic depression with a 45 meters high rockfill dam. The defective natural capacity will be increased by an excavation of 2.2 millions cubic meters along the bottom and the slopes of the valley. The bedrock of Césima depression consists of highly fractured limestones and dolomites. The bottom of the depression is filled with weathered pyroclastic soils, unable to secure its natural imperviousness. The tightness of the reservoir will be obtained fully lining the bottom and the walls of the pool with a bituminous concrete facing. The lining surface amounts to 350,000 square meters and it is made of two impervious layers, each one of 0.05 meters thickness, placed on a binder course with an underlaying drainage layer. Checking and measuring of incidental leakages through the lining is allowed by a drainage system connected by an inspection gallery.
The lower reservoir (located in the plain at the east of Presenzano town) is fully man-made by means of large excavations (5.0 millions cubic meters) and it is bounded by rockfill embankments for a large extent. The soils directly involved in the construction of the reservoir consist essentially of materials having a pyroclastic origin from a nearby extinct volcano, deposited in successive stages and later, at different intervals, weathered, remoulded and resedimented on the bottom of the basin which at present is Presenzano plain. Especially near the surface, these materials are deeply argillified and weathered. The process which led to their deposition justifies the high inhomogeneity of the formation. Nevertheless it is possible to identify within the formation a gradual average increase with depth of the grain-size of the soil, which consists mainly of clays and silts in the upper part of the formation, sandy silts and silty sands and also medium-to-coarse sands in depth. Because of the large excavations due to the reservoir building, the less pervious soils have to be removed. The lower basin so will be completely lined with a bituminous concrete facing extending over 750,000 square meters. The features of the facing and of the drainage system of the lower reservoir are similar to those of the upper one.

EMBANKMENTS FEATURES

As before said, the main retaining structures, consisting in the 45 meters high dam for the upper reservoir, and in the boundary embankments (max. height 20 meters) for the lower one, will be realized in fill materials (Figs.3,4).
The typology of the retaining structures has been greatly conditioned by geotechnical characteristics of the foundation soils and by the location of the reservoirs near sites of well-known seismicity. Both reservoirs sites have been just recently classified as medium seismicity areas, according to the italian by-laws, by Ministerial Decrees of 03/07/81. But the new classification occurred after the official approval of final reservoirs project, obtained on 04/17/80. Nevertheless, for safety sake, the stability analyses were performed as for a high seismicity site, although previously to the afore classification. Actually, as regard the upper reservoir, the

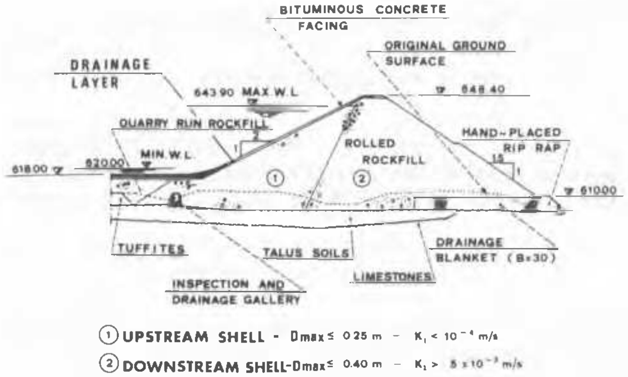

Fig.3 - Cèsima Dam

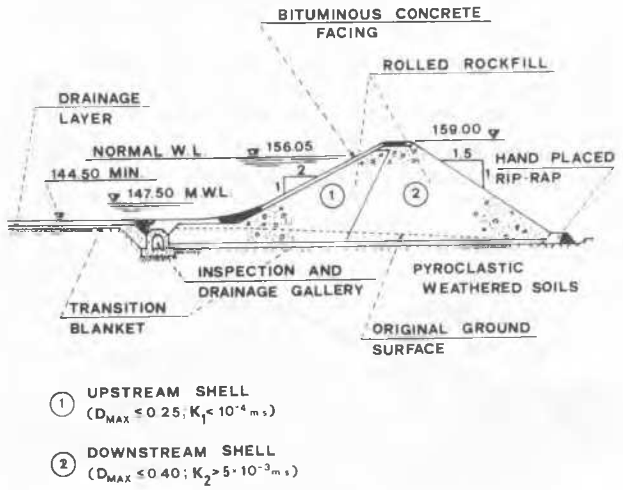

Fig.4 - Presenzano Reservoir - Retaining Embankment

presence of some active faults was noticed since the early geological investigations.
On the other hand the embankment height for the lower reservoir was limited because of the necessity of restraining the foreseeable foundation settlements in the allowable limits. About this matter, during the final design of the embankments, a large scale loading test was perfomed (Brown, 1983).
For safety against seismic actions a worthy care was devoted to the embankments body, that was differentiated in two zones, each one made of a different material. The features of the two zones were defined according to the following considerations.
The upstream shell (Zone 1) has the function of limiting, as far as possible, the amount of incidental leakages through the impervious facing along worthy cracks which could be caused by a seismic event of exceptional intensity. In such a catastrophic event, the drainage layer below the facing should be completely saturated and leakages would penetrate through the embankment. So, the material of the upstream shell has to satisfy two foundamental requirements: it must be absolutely cohesionless, so as to be self-sealing and its permeability must not be too high.
The downstream shell has a mainly drainage purpose and so its material must have such a permeability to hold a fairly low seepage line, which should be as low as possible to let saturated only the lower zone of the embankment.
Furthermore, a hand placed rip-rap has been arranged into the downstream shell of Césima dam, which underlies a head about twice as the lower reservoir one, to improve the drainage capacity in the stream bed.

QUALITY AND CONSTRUCTION CONTROLS

The properties of the materials forming both the zones of the embankments body were specified according to the different functions of the two zones and taking the fact that both materials had to be obtained from the same natural formation. The following specifications were defined as regards the upstream shell (zone 1):
- D max. equal to 0.25 m;
- percentage by weight of the fraction finer than 0.1 mm at least equal to 10%;
- coefficient of permeability less than 10^{-4} m/sec.

The following ones were defined for the downstream shell (zone 2):
the permeability at least 50 times greater than the one of the zone 1;
- exclusion of the boulders larger than 0.400 m.

Furthermore the grain size of the two zones (see Fig.5) was specified so as to respect

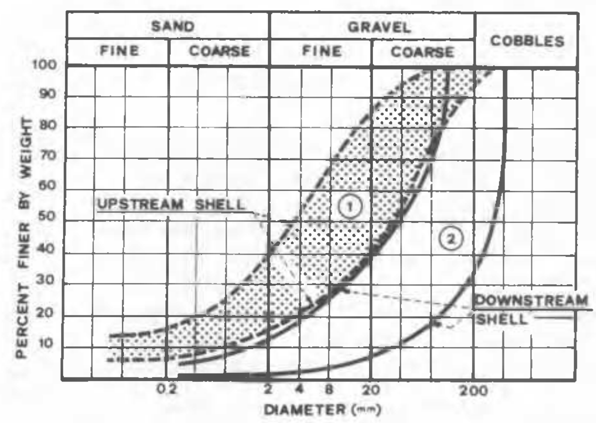

Fig.5 - Grain Size Curves

the well-known filters ratios, considering the zone 1 as the base material to be protected and the zone 2 as the respective filter.

LEAKAGE EVALUATION

What could really happen in the case of a seismic event, causing the cracking of the facing, was checked on the basis of the above mentioned design specifications. Fig. 6 shows, for Cèsima dam, the well known graphical construction of the basic parabola for the seepage line in the embankment, according to Casagrande method. The possible seepage flow was estimated to 0.0034 cubic meters per sec per each meter of embankment, according to the scheme in Fig.6,

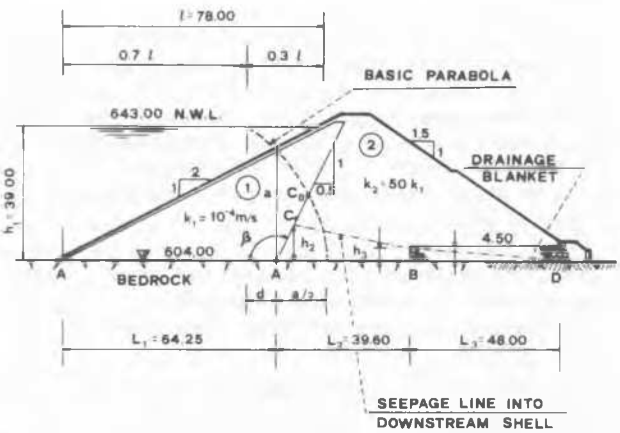

Fig.6 - Cèsima Dam - Seepage Flow (Creager, 1958)

Furthermore, still in this case, the seepage line lays enough far from the downstream face.

STABILITY ANALYSES

Seismic actions were considered, in the stability analysis, as horizontal and vertical static forces, and they were valued according to the Italian by-laws. Specifically, inertial actions of the structural masses were considered proportional to their respective weights w and applied in their self centres of gravity, according to the expressions:

$$F_h = +/- C\ W \quad (1)$$

as regards the horizontal forces, and

$$F_v = +/- 0.5\ C\ W \quad (2)$$

as regards the vertical ones, where C is the coefficient of seismic intensity, connected to the seismicity degree S of the site (*), by:

$$C = (S-2) / 100 \quad (3);$$

instead, the inertial actions of water in the reservoir were considered as a continuous distribution of normal pressures on the upstream face, and of an intensity equal to:

$$P(y) = C \cdot \gamma_w \cdot y_0 \cdot \frac{D}{2}\left[\frac{y}{y_0}\cdot\left(2-\frac{y}{y_0}\right)+\sqrt{\frac{y}{y_0}\cdot\left(1-\frac{y}{y_0}\right)}\right] \quad (4)$$

where, besides the symbols of well-known meaning, D is a coefficient depending on the face slope, and Y_0 is the head, measured from the maximum water level of the reservoir to the lowest point of the bottom.

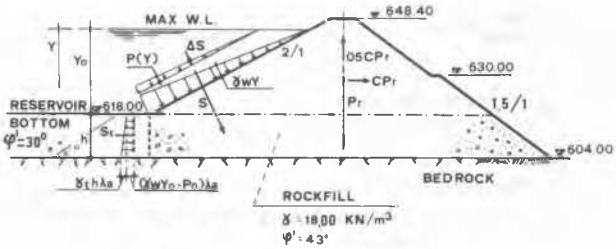

Fig.7 - Cèsima Dam - Load Pattern for seismic Stability Analysis

Cèsima Reservoir

The load pattern, considered for the stability analysis of the dam, is shown in Fig. 7, and is referred to the section of maximum height and to the seismic event taken under consideration. The stability analysis was made at the elevation of the bottom of the reservoir (618 m a.s.l.) and on the foundations (elevation 604).
In the last analysis S_t is the horizontal

(*) The seismic classification of sites is determined by Ministerial Decree. The seismicity degree S is 12 for seismic sites classified in the first class, 9 for the ones in the second class, and 6 for the ones in the third class.

pressure mainly due to the water load on the bottom of the reservoir, and was valued, according to Rankine, with $K_A = 0.333$.

Lower reservoir

The stability analysis was made referring to the two following different cases:

(i) section of maximum height of the embankment, with the reservoir at the normal (and maximum) water level. Analysis was restricted to the slip surfaces, emerging in five different specified points of the upstream facing;

(ii) section of the excavated slopes corresponding to the deepest level of the bottom of the basin, in the condition of the reservoir being empty.

(i) Embankment

The analysis for the embankement was all performed supposing circular slip surfaces, according to the Modified Bishop Method, and using an Olivetti P 652 computer.
As usual in such kind of analysis, the safety factors related to the circles passing through the specified points of the upstream facing and having their centres placed on a reticle with 2.0 meters square meshes, were each time calculated. The reticle was made enough large to identify the slip surface giving rise to the minimum value for the safety factor F_S.

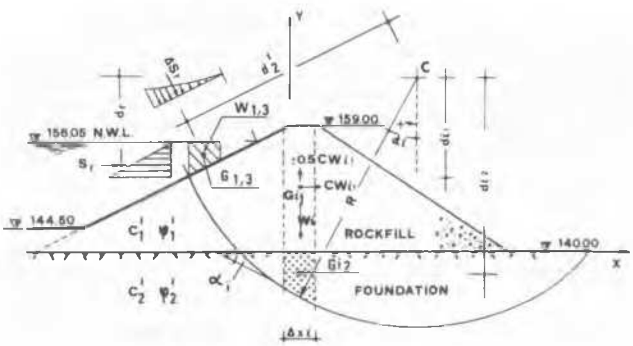

Fig.8 - Presenzano Reservoir - Embankment Stability Analysis by Bishop Modified Method.

The expression used for the seismic analysis is:

$$F_s = \frac{R \sum_{i=1}^{N} \frac{c_i \Delta x_i + \left(W_i \pm 0.5C \sum_{j=1}^{2} W_{ij}\right) \cdot tg\, \varphi'_i}{M_i(F_s)}}{\left[R \sum_{i=1}^{N} \left(W_i \pm 0.5C \sum_{j=1}^{2} W_{ij}\right) \sin\alpha_i + C \sum_{j=1}^{2} W_{ij} d_{ij}\right] + S_r d_r + \Delta S_r d'_2} \qquad (5)$$

where, besides the known symbols and the ones reported in Fig. 8:

$$M_i(F_s) = \left(1 + \frac{tg\,\alpha_i \cdot tg\,\varphi'_i}{F_s}\right) \cos\alpha_i \qquad (6)$$

ΔS_r is the resultant of the dynamic overload due to the stored water, calculated according to the afore mentioned formula (4).

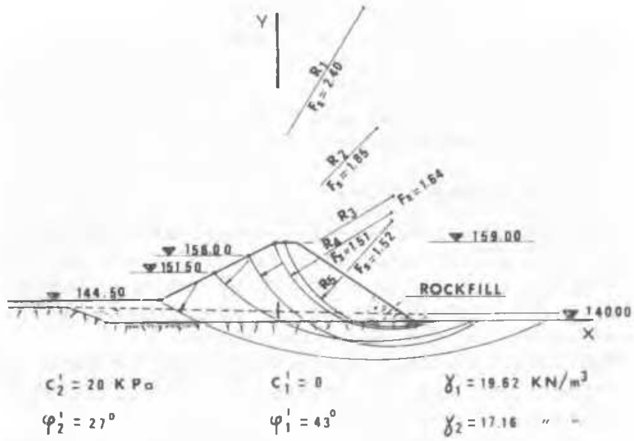

Fig.9 - Presenzano R. - Results of Embankment seismic Analysis

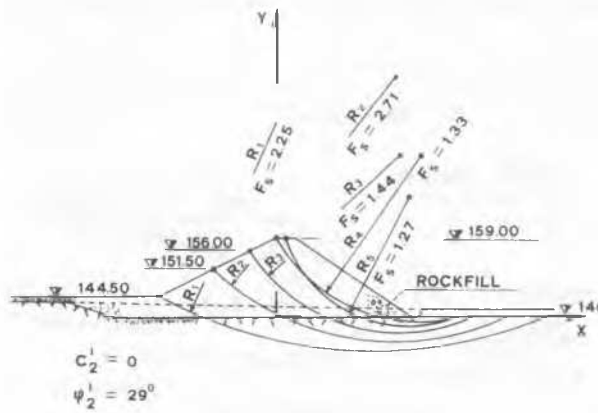

Fig.IO - Presenzano R. - Results of Embankment seismic Analysis

The data relative to the slip circles having the lowest values of F_s always greater than the minimum value (1.20) allowed by the Italian by-laws, and referring to two different hypothesys assumed for the geotechnical foundation soils c'_2 and φ'_2 ($c'_2 = 0$ and $\varphi'_2 = 29°$; $c'_2 = 20$ kPa and $\varphi'_2 = 27°$) are shown in Figs.9 and IO.
The non seismic analysis of the embankment foundation was performed besides under the afore mentioned hypotheses of the parameters

c_2' and φ_2' of the foundations soils, also with two state condition of them different from the natural moisture, and precisely:

- foundation soils in saturation condition, assuming γ_2 =17.65 kN/m^3 and, consequently, γ_2' (weight of the submerged soil):

$$\gamma_2' = \gamma_2 - \gamma_w = 7.84\,kN/m^3 \qquad (7)$$

-foundation soils in non complete consolidation condition, assuming that, when the embankment will be completed, the consolidation degree U is 80%. As regards the last two kinds of analyses, c_2' = 0 and φ_2' = 29° was always assumed, for safety sake.

(ii) Excavated slopes

A preventive cutting of the natural soil with a slope of 3/1 and, subsequently, a definitive shaping with a slope of 2/1 by means of borrow fill,similarly zoned as the embankments, were foreseen for the excavated slopes.
The stability analysis of the slope in its final shape was performed with two different methods, and precisely:

- with the seismic actions, by the Wedge Method, calculating the inertial forces as afore described;

-without seismic actions, by Modified Bishop Method, also applied for the embankment.

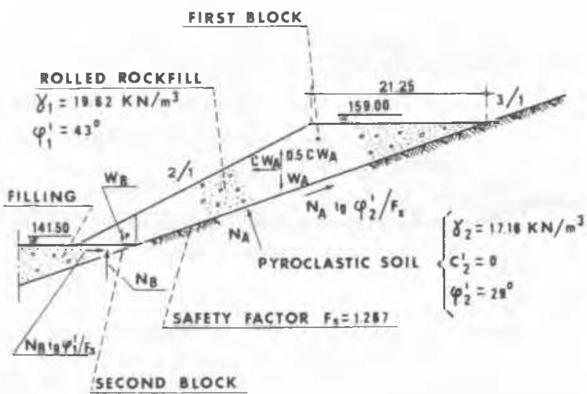

Fig.II - Presenzano R. - Excavated Slopes. Seismic Stability Analysis by Wedge Method.

The data related to the seismic analysis of the slope along the fill subgrade surface and referring to the block subdivision of the rockfill, to which the minimum safety factor is up, are shown in Fig. 11.

FINAL NOTE

The mentioned works, in advanced construction state, were really subjected to the seismic events of the 7th and of the 11th May 1984, having an intensity valued as the seventh degree of the Mercalli Modified Scale,and their epicentres not further than 20 km, without suffering any damage.

REFERENCES

Manfredini, G., and Silvestri,T. (1983). A large-scale in situ permeability test at Presenzano, Italy. Proc.Int.Symp.on Soil and Rock Investigations by in Situ Testing. Vol. I,477-481, Paris.

Brown, P.T., Calabresi, G., Manfredini, G.,Silvestri, T. (1983). A large test embankment at Presenzano, Italy. Proc.Int.Symp.on Soil and Rock Investigations by in Sity Testing. Vol. II, 29-32, Paris.

Creager, W.P., Justin, J.D., and Hinds, J. (1958). Engineering for Dams.Vol.III 6th Pr. pp. 655-698. New York: John Wiley & Sons, Inc. London: Chapman & Hall,Ltd.