DESIGN AND CONSTRUCTION OF A 18M HIGH EARTHWORK ON A SLOPING AREA

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Abstract: The newly built Simeri-Crichi (Southern Italy) electricity production plant (800 MW) required a new station to be constructed in order to ensure connection with the national electricity network. Due to the particular geomorphological features of the site, the transforming station had to be built on a hilly area, affected by several active and quiescent landslides. Furthermore the area was earthquake prone. Quite remarkable earthworks were necessary to provide a flat area of the required size (about 70m x 70m): in fact, while on the uphill side a 15m excavation allowed removal of the weakest soil layers, on the downhill side a 18m high retaining structure had to be constructed on the unstable slope.

The geological and geotechnical conditions of the site, together with the orographic problems previously described suggested construction of a reinforced embankment consisting of three steep reinforced slopes, reinforced with 30 layers of HDPE geogrids, with lengths ranging from 5 to 11m. The conspicuous load that had to be applied on such an unstable slope required a foundation deep enough to provide satisfactory stability conditions to the whole embankment. Therefore the downhill side of the embankment had to be laid on a double row of large diameter piles, connected with reinforced concrete beams. Limit Equilibrium as well as Finite Element analyses were performed in order to study the soil-structure interaction and to assess stability and safety conditions under static and pseudostatic loads. The logistic aspects connected to the location of the project and the most remarkable construction issues are reported and commented as well.

Keywords: case study, earthfill, landslide, pseudo-static method, reinforced earth structure, stability analysis

INTRODUCTION

The building of a new 800 MW Combined Cycle Gas Turbine (CCGT) plant in Simeri-Crichi near the city of Catanzaro, in Southern Italy, required a new power line to be constructed in order to ensure connection to the Italian national power network. The connection would have required a flat square surface, whose minimum dimensions had to be at least 70 x 70m, placed at elevation 410,00 m a.s.l. in order to allow the construction of a transforming station. Other works to be done together with the creation of the platform were the widening of the existing local road and the creation of a new access road to the site. Choosing the site where the transforming station should have been built, appeared to be quite a difficult decision, due to the many constraints that had to be considered such as costs, technical reliability, site accessibility etc, not to mention the fact that the whole area was affected by quiescent and active soil movements (Figure 1).

GEOLOGICAL SETTING

In depth, the Sila Massif if formed by granitic rocks (Palaeozoic Basement), deeply weathered and locally outcropping only at the foot of the hills, along the main creek. In the studied area, the hills are essentially constituted by sedimentary deposits (middle to superior Miocene), outcropping along the creeks as a complex sequence of fine to coarse grained sandstones, inter-layered with sands, silts and silty clays; the bedding, varying from parallel to undulated, is prevalently dipping downslope, with an average inclination of 28 to 30°. The hillslopes are mostly covered by colluvial soils and landslide debris, extending in depth down to about 5 to 8 m, according to the results of geophysical surveying (refraction method) and boreholes investigations (Figure 2).

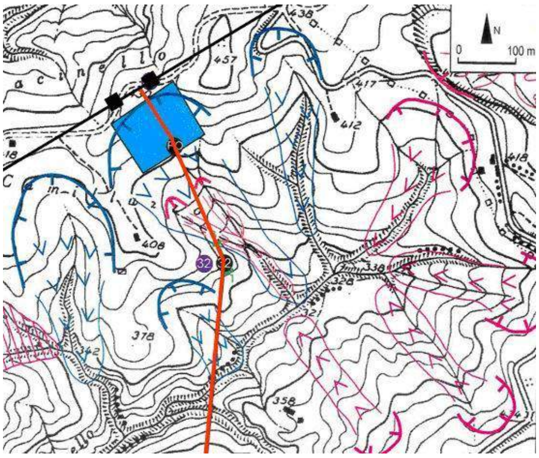

Figure 1. The site was affected by several stability problems

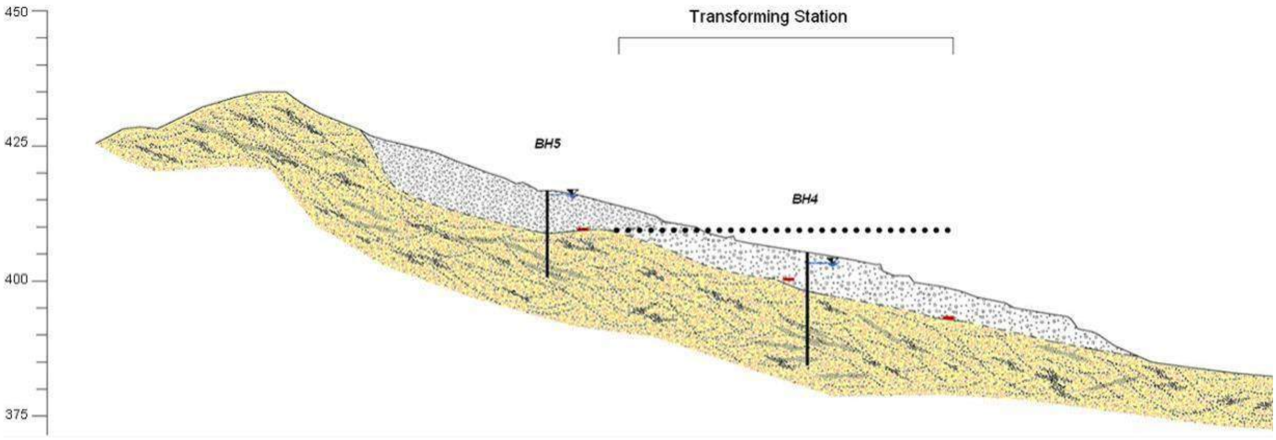

Figure 2. Geological profile

The geomorphological surveying of the site allowed identification of the presence of many slope movements, some of them active (Figure 1). In particular, two main landslides, affecting the area down to a depth of about 7 to 8 m, have been recognized; it was possible to identify a direct relationship between the subsoil setting, the geometry of the failure surface and the direction of the main landslide movements.

The main factor of the slope instability was thought to be the progressive deepening by erosion of the river bed. Moreover, piezometric measurements in the boreholes gave evidence of a high groundwater table (less than 2m), confirmed by the presence of several springs in the lower part of the slope. Furthermore, the area was earthquake prone, being classified, according to Italian Hazard Seismic Atlas, as a seismic zone 2.

DESIGN CHOICE

Quite remarkable earthworks were necessary to provide a flat area of the required size (at least 70m x 70m): in fact, while on the uphill side a 15m excavation allowed removal of the weakest soil layers, on the downhill side a 18m high retaining structure had to be constructed on the unstable slope. Different possible alternatives were evaluated, from concrete walls to unreinforced embankments, but both the solutions were judged to be not acceptable.

The first option would have required very tall concrete structures, which would have resulted too stiff, especially towards slope settlements and seismic actions; furthermore concrete structures would have caused very high environmental impact and, most of all, excessive construction time and cost. The second option would have required huge amount of fill material and excessively large areas to expropriate at the base. If considering that the site is fully covered with valuable olive trees, the cost of such a structure would have become too high even from a social point of view, as olive oil production represents one of the main economies of the area.

For these reasons, a reinforced embankment seemed to represent the best solution: first, it allowed a significant reduction of fill material, and at the same time of the embankment weight, second it ensured enough flexibility to withstand slope deformations.

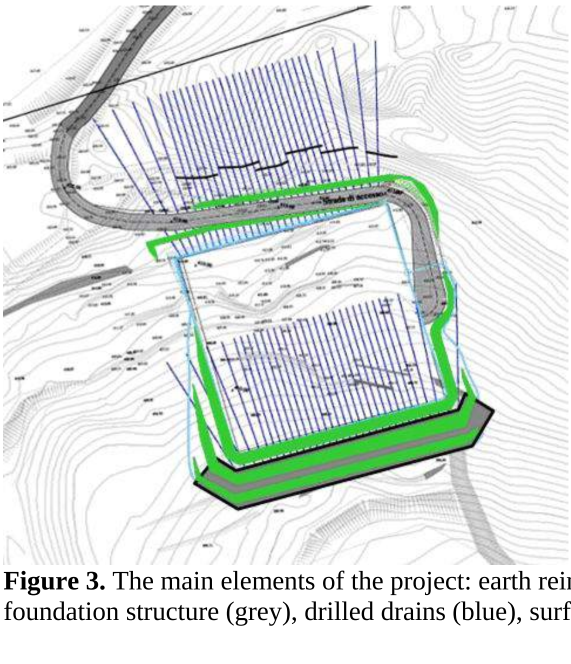

Figure 3. The main elements of the project: earth reinforced slope (green), which are supported downslope by a deep foundation structure (grey), drilled drains (blue), surface drains (light blue) and access road (light and dark grey).

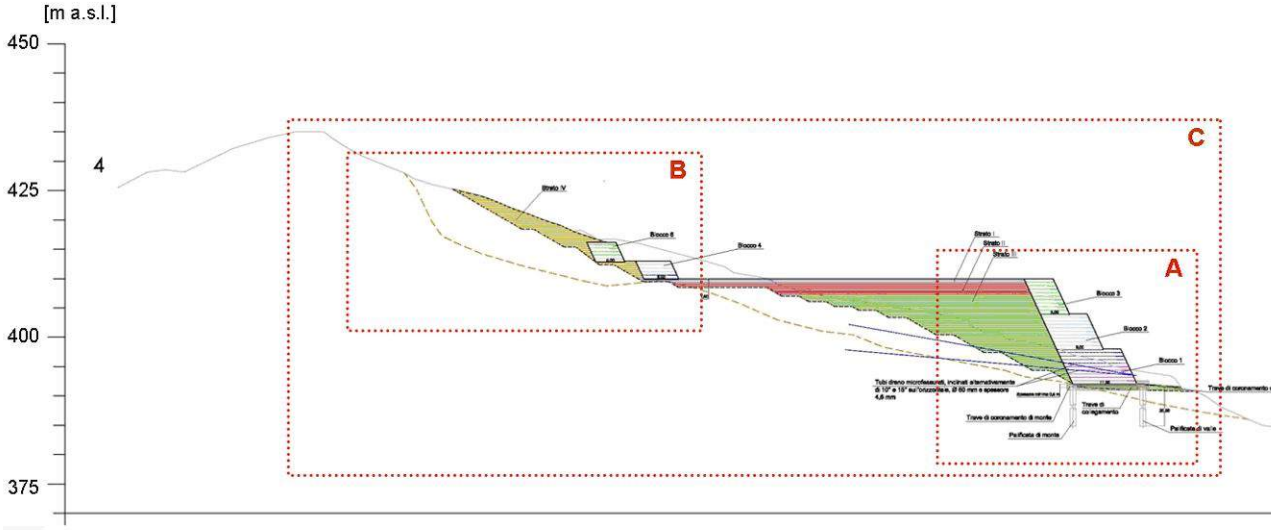

Figure 4. Cross section of the slope, with all of the project main elements. The different colours of the earthfill, green red and blue, refer to different level of compaction. The letters A, B and C refer to different FEM stability analyses (as explained forward in the text).

Therefore the choice of steep earth reinforced slopes was due to several reasons:

- lower global cost, thanks to the possibility to reduce the quantity of the material needed to the possibility use locally available and then cheaper fill material;
- fast and easy installation, with minimum volumes of goods to be delivered on site;
- flexibility of the structure, able to withstand deformations and to resist to dynamic effects induced by seismic forces;
- low environmental impact, thanks to the vegetated face and to the lower amount of fill material to be delivered on site respect to unreinforced embankments.

Nevertheless, the presence of weak foundation soils, which in addition were expected to be loaded by a 18m high embankment, imposed to design a deep foundation structure, so that general failure surfaces under the base and/or excessive deformations could be prevented.

THE PROJECT

The main elements of the project can be briefly described as follows, (see Figures 3 and 4):

- a square shaped embankment at 410m a.s.l., whose dimensions at the crest were 70m x 70m, made with earthfill compacted up to three different levels, so that γ_D should be larger than, namely: 97%, 96% and 95% Proctor mod., the fill material had to comply with the grain size distribution reported in Table 1;
- earth slopes, reinforced with HDPE extruded geogrids, in order to retain excavated soil in the uphill area (two to three blocks for up to 12m), and three blocks to retain earthfill in the downhill area (for up to 18m);
- deep foundation structure (Figure 5), made with two parallel rows of large diameter drilled piles (D1000, L=20m, spaced 2m on centre), the beams on the head of the two rows are connected by secondary reinforced concrete beams (d=1m, spaced 6m on centre), in order to provide a safe foundation for the above reinforced block; since FEM analyses showed that a significant reduction of the moments could be achieved by allowing rotations, special hinge-connections were designed (see text below);

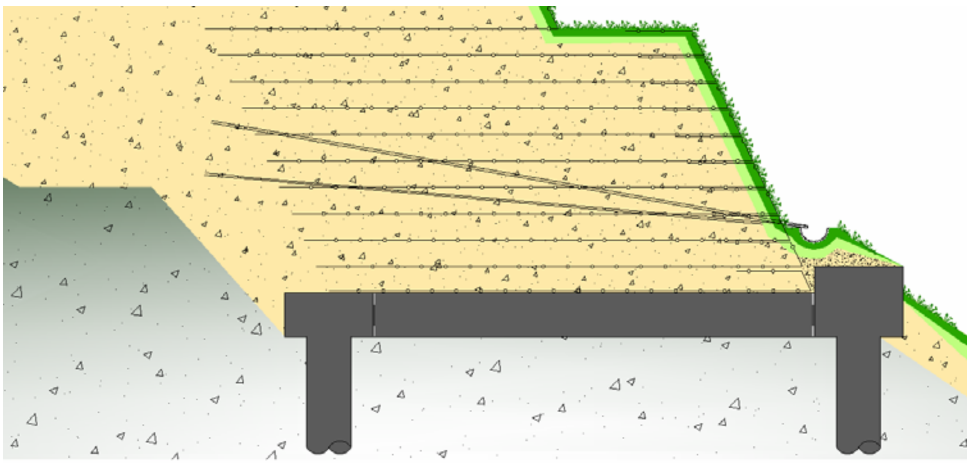

Figure 5. Cross section of the lower part of downhill reinforced slope.

Table 1. Grain size distribution required for earthfill

Grain size [mm]	U.S. Sieve [n°]	Passing [%]
101,6	4”	100
38,1	1” 1/2	100 ÷ 65
9,51	3”/8	70 ÷ 30
4,75	4	55 ÷ 25
0,42	40	25 ÷ 5
0,074	200	12 ÷ 0

- deep groundwater drainage, made with drilled drains (D2”, inclined 10 to 15°), for a length varying up to the subgrade’s depth;
- surface and shallow drainage system, made with gutters, half pipes and French drains, on the top and around the base of the embankment; and
- access paved road, part realised by enlarging an existing road (dark grey in Figure 3), part by building a new one (light grey), the road width was 5m for the whole track.

STABILITY ANALYSIS

As in any case when a steep slope has to be designed, the stability analysis has been first conducted in three steps:

- internal stability
- external stability
- global stability

The internal stability has been studied by determining the additional forces (in terms of number and spacing of the reinforcement layers) necessary to provide equilibrium to the slope with an adequate margin of safety, in both static and seismic conditions.

For this application, in particular, the computation has been made using the software Reslope, which researches log spiral failure surfaces to determine the location of the critical shear surface and, subsequently, the necessary reactive force. For every log spiral failure surface, the program calculates the ratio between the driving moment (given by the own weight of the slope, the surcharge loadings, the seismic forces, the pore water pressure) and the resisting moment (given by the soil shear strength and by the reinforcements reaction). This check is repeated for other potential log spiral failure surfaces until the least stable is found, i.e. until the critical slip surface and the associated maximum required restoring reinforcement force are found.

External stability assessment for reinforced structures needs three different potential failure mechanisms to be considered:

- Sliding
- Limiting the location of the resultant of all forces (overturning).
- Bearing capacity

In this case, as described before the presence of a reinforced concrete platform on piles at the base with a key to prevent direct sliding made the first and third mechanism non critical. The second one is generally the less critical, if the shape of the reinforced block and the position of its center of gravity are considered.

Rotational stability analyses have been done using code PCSTABL, developed by Purdue University, in USA in order to determine both the minimum and the optimal reinforcements layout (Figure 6).

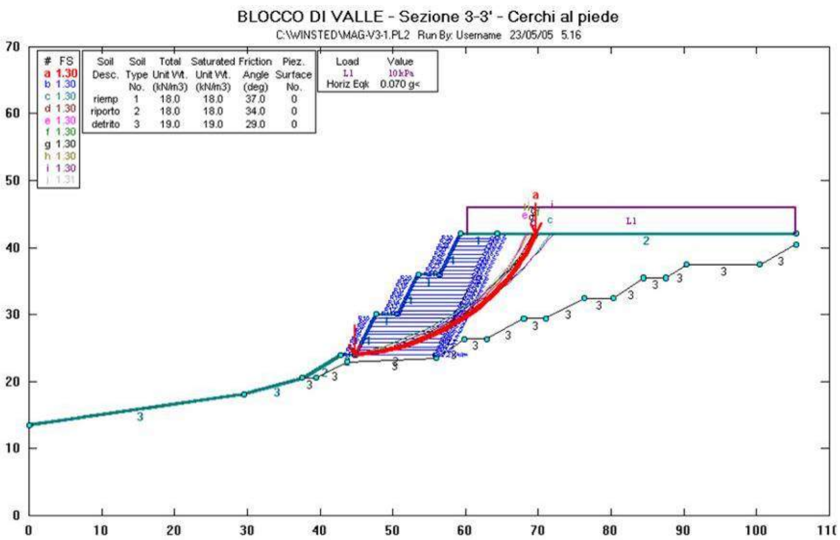

Figure 6. Global stability analysis

The code allows identification of, according to Bishops theory, the circular surfaces with the lower factor of safety, given the geometry of the project, surcharges and geometric intervals within which the circular surfaces must enter into the subsoil and exit form the soil. The job has been designed and constructed using geogrids produced by extrusion and mono-directional stretching of HDPE. Different classes of geogrids have been used, with a peak tensile strength ranging from 45 kN/m to 120 kN/m. The design strength for the geogrids used in the analysis are summarised in Table 2

Table 2. Design strength

Peak tensile strength (kN/m)	Design strength (kN/m)
45	**18.45**
60	**24.60**
90	**36.90**
120	**49.20**

FINITE ELEMENT ANALYSES

Several Finite Element analyses were conducted in order to assess general stability conditions of the slope (eventually validating Limit Equilibrium analyses), but also to predict settlements on the top of the earthfill and, last but not least, to study the interaction between soil and the foundation structure. Such a wide range of goals could only be achieved by studying the problem either on a general or on a local point of view.

For this reason, a first set of analyses was run only on a limited area of the whole slope (downhill blocks: A box in Figure 4), so that it was possible to model all the reinforcements in both static and seismic conditions, the latter by means of a pseudo-static analysis. The same was made for the uphill area (B box in Figure 4), but in this case for multiple cross sections, due to the varying height of the intermediate berm along the area front edge.

Therefore the following general analyses could be calibrated, eventually confirming Limit Equilibrium results with reference to geogrids stress rate and to the reinforced slope collapse mechanism. This third set of analyses, which modelled the whole slope, was performed under static and seismic conditions, but with the inevitable simplification of omitting the reinforcements by defining a homogeneous equivalent soil (C box in Figure 4).

The analyses were performed in plane strain conditions and, according to the library available from the software (PLAXIS) used in the analysis, every soil element has been described with quadratic 6-node triangular elements. In order to represent the soil constitutive law the so-called Hardening Soil model was used (Brinkgreve and Vermeer, 1998): this is an elastoplastic type of hyperbolic model (Duncan and Chang, 1970), formulated on the hypotheses of hardening plasticity.

All the reinforced concrete elements that constituted the structure of foundation (piles and beams) were modelled with perfectly elastic 3 nodes – 4 integration points – beam elements, for which the total deflection is calculated considering also shearing as well as bending. Being a rough, but conservative hypothesis, the entire above soil load was considered transferred to the foundation structure.

If from on the one hand it should be noted that the ratio between the distance on centre and the beam's width would have been 6m / 1m = 6, on the other hand it shouldn't be neglected the geogrids effect in stiffening the base layer over the beams, in literature also referred to "tensioned membrane" effect (Giroud and Noiray, 1981).

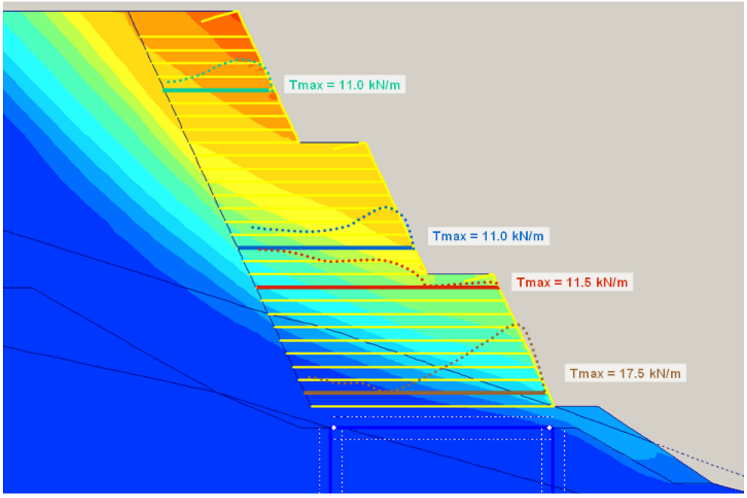

Figure 7. Incremental displacements shaded contours plot under working static conditions

Downhill blocks analyses

Limiting the analysis's domain to the downhill reinforced blocks allowed modelling of the reinforcing geogrids by means of 3-node tension elements. Normally it's also necessary to model the interaction between soil and geogrids by inserting special joint (or interface) elements to allow for reduced values of friction angle.

Since the number of reinforcements was considerably high for the software and hardware resources, modelling interfaces too also have resulted in excessive calculation time. Furthermore, it has been demonstrated from experimental tests on the same kind of geogrids (Cancelli et al., 1992), that due to the interlocking effect between geogrid and soil grain, direct sliding factor is always larger than or equal to one.

The execution of Finite Element Analyses allowed verification of geogrids design strength, as they were calculated by means of General Limit Equilibrium methods, with the additional possibility of calculating strain and stress ratios for reinforcements having different stiffness's (Figure 7).

Uphill block analyses

The same considerations already expressed about the downhill blocks analyses could be repeated for the uphill blocks: analyses allowed to state the effectiveness of the designed reinforcements for static and seismic conditions (Figure 8). In this particular case, the execution of a Finite Element Analysis allowed also a better understanding of the complex failure mechanism. In Figure 9 incremental displacements shaded contours at failure are represented: a connection between two different sliding, on both sides of the road, is quite evident.

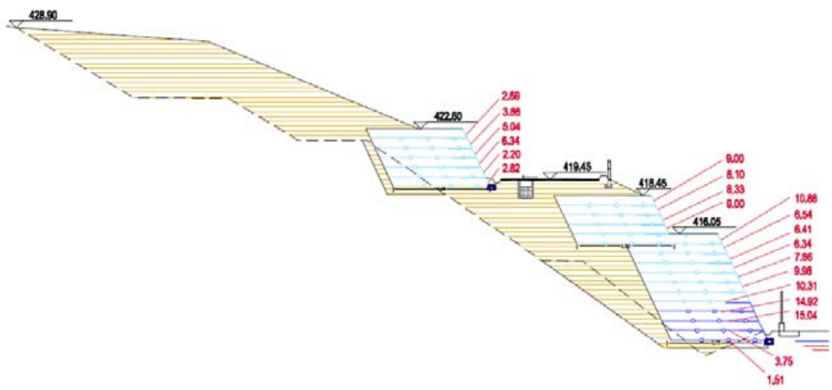

Figure 8. Calculated tensile stress, in kN/m, in uphill blocks geogrids, under seismic conditions.

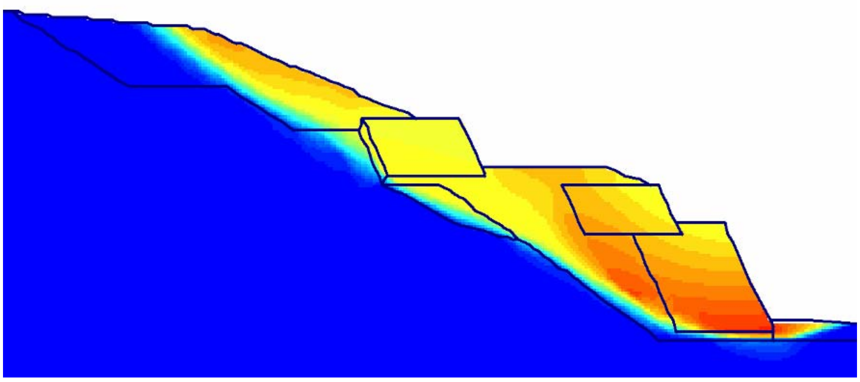

Figure 9. Incremental displacements shaded contours under seismic conditions. The figure relates to failure for (c'-tanφ) reduction , with γ=1,54.

Whole slope analyses

As it has already been said, in order to limit the excessive numerical difficulties that would have arisen if modelling every reinforcement on the whole slope, a homogenization method had to be used. Therefore, a fictitious soil was defined, according to the Mohr-Coulomb law, so to have two terms of strength, $\tan\varphi$ and c', being the first due to its own fill (frictional) contribution and the latter due to reinforcements tensile strength.

When the geogrids spacing is small compared to their length, then the reinforcing effect could be roughly taken into account by "smearing" every geogrid tensile strength along their respective soil layer. It should also be noted that the inclination between geogrids and failure surface, θ, should be also considered, so that the "smeared" strength should be reduced by a factor depending on the reinforcements capability of re-orienting along the failure line.

According to Bonaparte & Christopher (1987), who studied rotational stability of reinforced embankments, the reinforcements capability of bending due to local displacements of the surrounding soil as the onset of failure depends not only on the reinforcements flexibility, but also on the surrounding soil compressibility and/or strain sensitivity. The same authors suggest, when in any doubt, not to consider the possibility of geosynthetic reorientation: for this reason the fictitious cohesion was assumed to be:

$$c' = (T/s) \cos\theta$$

Where T is the geogrid design tensile strength, s is the geogrids spacing, and θ is the angle between geogrids and failure surface, and it was determined from the previous analyses (i.e. for the lower downhill block θ was taken to be 20°, see Figure 7).

ANALYSES OUTCOMES

Finite Element Analyses results allowed verifying all the designed reinforcements, in terms of strength, deformability and anchor length. But most of all, thanks to the execution of multiple numerical analyses it has been possible to confirm the effective interaction between the earth reinforced slope and the deep foundation structure, even under seismic conditions. In fact, a first set of analyses was run according to a structural model which allowed transferring moments and rotations from one row of pile to the other, through the connecting beams: this because the beams were rigidly connected (clamped) at both sides.

Analyses results proved that transferred moments were excessive and that they could be significantly reduced if rotations could be allowed by inserting hinges at every beam’s end (Figure 10). For this reason special hinge-connections were designed by crossing reinforcing bars, so to ensure only horizontal and vertical forces could be transferred, while the arm of the couple is obviously zero (Figure 11).

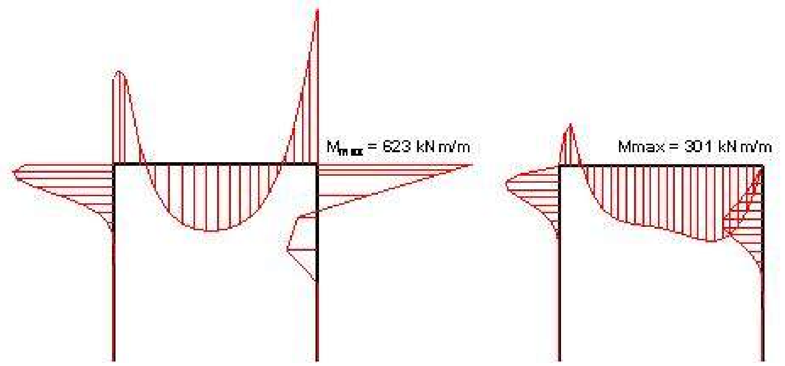

Figure 10. Deep Foundation structure: the bending moment in the frame (1st row of piles – connecting beam – 2nd row of piles) are represented for both connections: clamped (left) and hinged (right). The figure was cut to the higher part of piles, which were actually longer.

Figure 11. The beam connections were designed to transfer only horizontal and vertical forces.

CONSTRUCTION

Construction consisted of the following steps:

- Realization of the access road (unpaved)
- Excavation up to the design depth
- Execution of the foundation structure, piles and beams (Figure 12)
- Building of the embankment and of the earth reinforced slope (Figure 13)
- Final works: electrical works and asphalt paving of the access road (Figure 14)

Figure 12. The construction of the foundation structure (lower left).

Figure 13. The 18m high earth reinforced slope is under construction.

Figure 14. The transforming station after completing the works.

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