Design And Construction Of Tall Reinforced Embankments In Static And Seismic Conditions

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ABSTRACT
The present paper aims to present the design problems associated with very tall reinforced embankments in static and seismic conditions, through the presentation of some of the most challenging projects built in Italy in the last years. The reinforced embankments here presented were built in different Italian regions, with widely differentiated topographic, geotechnical and seismic conditions; for both road and railway applications. The paper introduces the stability analyses in static and seismic conditions, the design layouts and the construction techniques, showing technical drawings, construction details and photos taken during and after construction, thus affording a complete picture of the design and building activities associated with very tall reinforced embankments.

1. INTRODUCTION

Since the early '80 of last century hundreds of reinforced soil structures have been built in Italy in the most diverse environmental conditions, thus allowing the development of design and construction techniques, and the growth of specialized engineering companies and contractors. Green faced structures make the bulk of the projects completed so far. The present paper aims to present the design problems associated with very tall reinforced embankments in static and seismic conditions, through the presentation of some of the most challenging projects built in Italy in the last years. The reinforced embankments here presented, up to 46 m high, were built in different Italian regions, with widely differentiated topographic, geotechnical and seismic conditions; for both road and railway applications. The use of marginal soils, like calcium stabilized silt and clay or tunnel debris, is reported as well.

2. STATE ROAD NR 28 “DEL COLLE DI NAVA”

The Italian National Road Agency, ANAS SpA, Genova Department, is building the new stretch of the State Road Nr 28 “del Colle di Nava”, with the aim of bypassing the town of Pieve di Teco, in the Province of Imperia. The project includes an approx. 2,0 km long tunnel, a viaduct crossing the river valley and three roundabouts placed on the slopes along the course of river Arroscia. The three roundabouts are supported by reinforced soil slopes and walls, built using the wrap-around technique with sacrificial steel formworks, all with vegetated faces. Totally the reinforced soil structures make 9,600 sq.m of vertical face. For a better environmental blending the reinforced soil structures have been designed with tiered pattern, in order to match the old dry walls made up of local stones, which cover all the slopes of the valley. Each reinforced soil structure is comprised of 80° sloped tiers and variable width horizontal berms. The maximum height of the reinforced soil structures is above 30,0 m. The most impressive reinforced soil structure, supporting the Northern round-about, has the following characteristics: maximum height = 30,40 m; length = 260 m; face slope = 80° (each tier); 3 abutments of the viaduct resting on top of the slope. Besides the reinforced soil structure, two concrete channels have been designed, carrying down the water of two creeks, the Rio Teco and the Rio Minore, whose natural courses perpendicularly cross the reinforced soil walls: the channels follow the same tiered pattern of the wall, after passing below the new road stretch inside corrugated steel culverts. Fig. 1 shows the rendering of this very complex structure. The “Acquetico” round-about has been built by filling the valley with debris from tunnel excavation. The fill is supported by a tiered reinforced soil structure, with maximum height of 23.4 m. The Fosso S. Rocco creek, which was flowing through the small and steep valley, has been canalised: after crossing the round-about inside a 3.20 m diameter corrugated steel pipe, it comes to light at the top of the reinforced soil structure, then it flows in a channel made up of gabions, whose bed gradually enlarges to 6.0 m; the channel follows the reinforced soil slope with several jumps; the final jump, almost 10 m high, is made up of huge stones, which also make the base of the reinforced soil structure. Fig. 2 and 3 show the plan view and the cross-section of the Acquetico round-about. The gabion channel is waterproofed with 3 mm thick PVC geomembranes, protected by 500 g/sq.m. nonwoven geotextiles, which run al around gabions, as shown in Fig. 4.

Fig. 1 - Rendering of the Northern round-about, showing the vegetated reinforced soil wall, the Rio Teco channel and the viaduct, whose 3 abutments rest on top of the reinforced soil structure

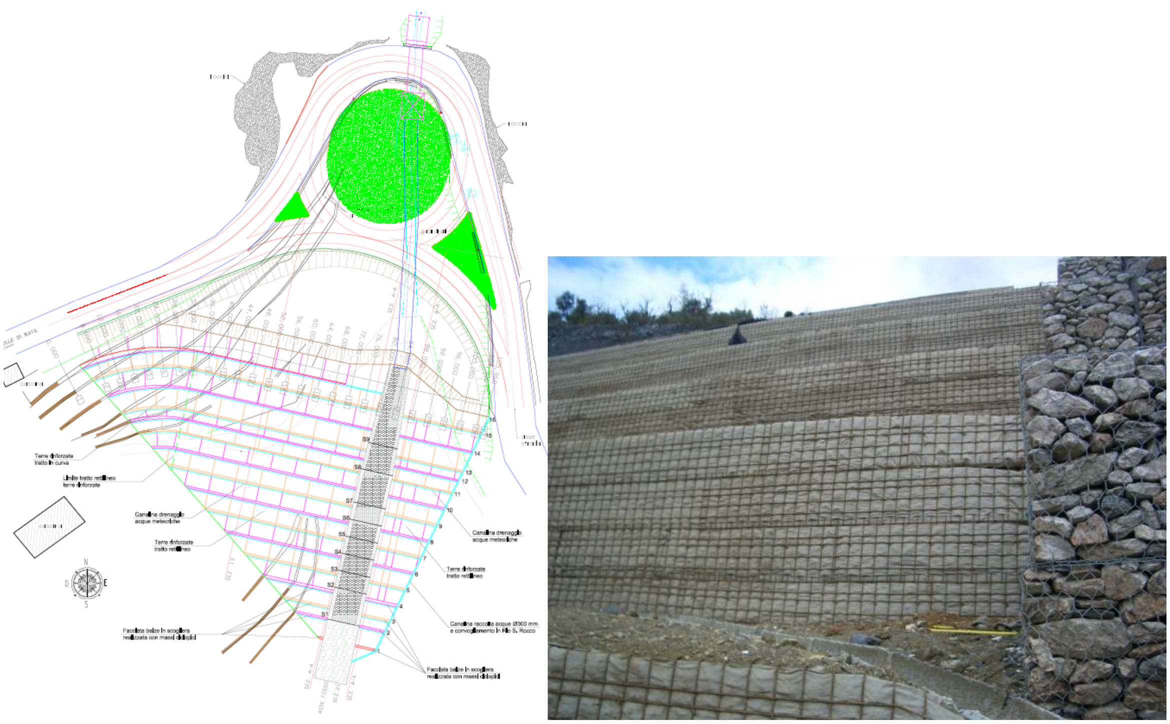

Fig. 2 - Plan view of the Acquetico round-about and picture of the tiered reinforced slope structure aside gabions

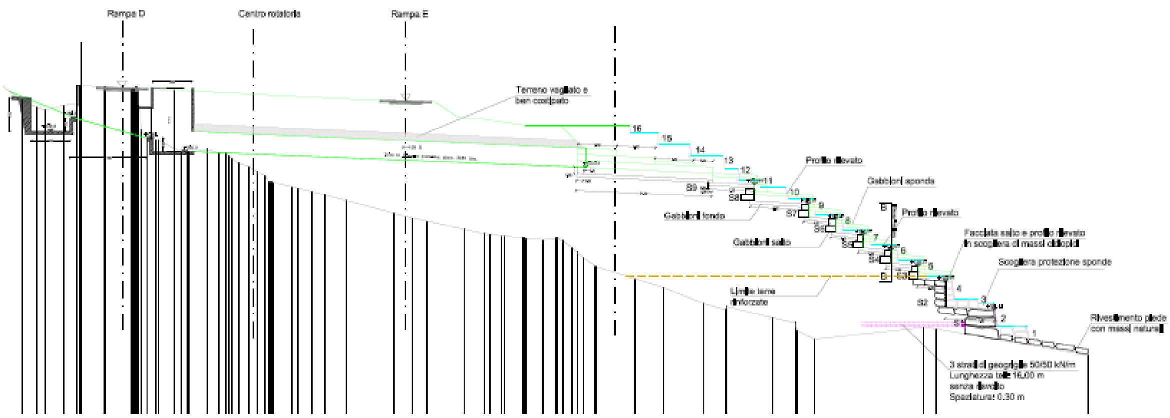

Fig. 3 – Cross-section of the Acquetico round-about with the gabion channel for the Fosso S. Rocco creek

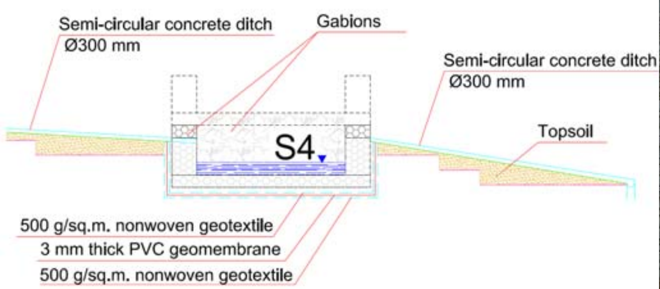

Fig. 4 – Cross section and picture of the gabion channel

2.1 Geotechnical characteristics of soils

The reinforced soil structures have been built using the crushed rocks coming from the excavation of the tunnel between the Southern and the Northern roundabouts, made up mainly by soft limestone. Such a soil can be assimilated to sandy coarse gravel with silt and boulders, with the following geotechnical characteristics: unit weight γ = 18 kN/m^3; friction angle ø = 35°; cohesion c = 0. Looking at the drainage characteristics of the soil for the reinforced structures, we must consider that the permeability of the crushed rocks is medium to low, since there is always a certain percentage of silt, originated by the disgregation of the limestone. Hence the design has taken into account the potential pore pressures by introducing the pore pressure coefficient Ru = 0,10. In any case the design includes drainage pipes, wrapped with nonwoven geotextiles, spaced 3.50 m horizontally and 3.00 m vertically, in staggered pattern, in order to allow the water coming from the back of the reinforced block to flow out at the face without raising up the pore pressure.

2.2 Design tips

In order to simplify construction, a unique value of the vertical spacing of geogrid layers has been set, equal to 0,60 m. For a better environmental blending the reinforced soil structures have been designed with tiered pattern, in order to match the old dry walls made up of local stones, which cover all the slopes of the valley. Each reinforced soil structure is comprised of 80° slope tiers, with a height H = 4,20 m, and variable width horizontal berms. The calculation considered a uniform surcharge of 20 kPa, as required for first class roads. The calculations showed that it is possible to use Polyester woven geogrids with 80 kN/m tensile strength. Since the crushed rock has a wide granulometry, ranging from lime particles to 150 mm boulders, the specification called for geogrids with 60 mm x 60 mm apertures. The actual geogrids were Arter GTS 50-50-60, GTS 60-30-60, GTS 80-30-60, GTS 100-30-60, specifically developed by Alpe Adria Textil. The following construction specs have been set: use Polyester woven geogrids with 60 mm x 60 mm apertures; insert a jute mesh and a 200 – 300 mm thick topsoil layer at the face; use sacrificial steel mesh formworks, with 10 mm diameter bars and 150 mm mesh; formworks shall have 600 mm vertical height, equal to the thickness of two compaction layers; formworks shall be folded at the factory, with one leg at 80° inclination and a 500 mm long horizontal leg; the inclined and the horizontal legs shall be connected by steel hooks made up with 8 mm diameter bars; the face shall be hydroseeded in the most appropriate vegetational period.

2.3 Design Strength of Geogrids

The design strength of geogrids is evaluated according to GRI GG4 specification: the available design strength T_{amm} is calculated by applying partial Factors of Safety (or Reduction Factors) to the peak tensile strength T_{ult} :

$$T_{amm} = \frac{T_{ult}}{\left(FS_{\text{creep}} \cdot FS_{\text{chemical- biological}} \cdot FS_{\text{construction}}\right)} \qquad [1]$$

Moreover for complying with the Italian geotechnical norm, a further Global Factor of Safety FS_G must be applied for getting the Design Strength T_D, to be used in internal stability calculations: $T_D = T_{amm} / FS_G$
According to the importance and the design life of the structure the value of FSg has been set equal to 1.30. Instead for global stability analyses the available strength T_{amm} is used, checking that the overall Factors of Safety are in excess of FS_G. Finally the values listed in Table 1 have been set: for geogrids with T_{ult} = 80 kN/m it results that T_{amm} = 37.5 kN/m.

Table 1: Factors of Safety for Polyester woven geogrids

Characteristics	Values for woven Polyester geogrids
Soil	Silty sand
FS_{creep}	1,66
$FS_{construction}$	1,30
$FS_{chemical – biological}$	1,10
FS_G	1,30
Direct shear factor f_{ds}	1,00
Pull-out factor f_{po}	1,00

2.4 Global Stability and seismic analyses

Once the structure has been designed for satisfying the internal stability conditions, it was necessary to perform the global stability analyses as well, in order to verify that no failure mechanism may occur, involving the reinforced soil mass, the foundation soil and the retained soil at the back. The analyses that were carried out are the following: 1) rotational analyses (circular failure surfaces); 2) translational analyses (horizontal sliding surfaces).
According to the Italian geotechnical norm, the following Factors of Safety in static conditions are set:
- rotational stability: FS_{rot} = 1,30
- traslational stability: FS_{trasl} = 1,30 (generally accepted value for reinforced soil structures)
- overturning: FS_{rib} = 2,00
- bearing capacity: FS_{bc} = 3,00

With the norm "OPCM n. 3274 / 2003" the new seismic classification of Italian territory has been approved: the area of Pieve di Teco has been assigned to the 3rd seismic category. The design horizontal seismic acceleration is:

$$k_h = S\,(a_g / g) / r \qquad [2]$$

a_g = peak bedrock acceleration for the 3rd seismic category = 0,15 g
S = factor accounting for the type of subgrade between the structure and the bedrock = 1,25
r = factor accounting for ductility and elasticity of the structure = 2
Hence for this reinforced soil structure it results: $k_h = S\,(a_g / g) / r = 0{,}093$
Since reinforced soil structures are not gravity structures, it can be assumed: $k_v = 0$

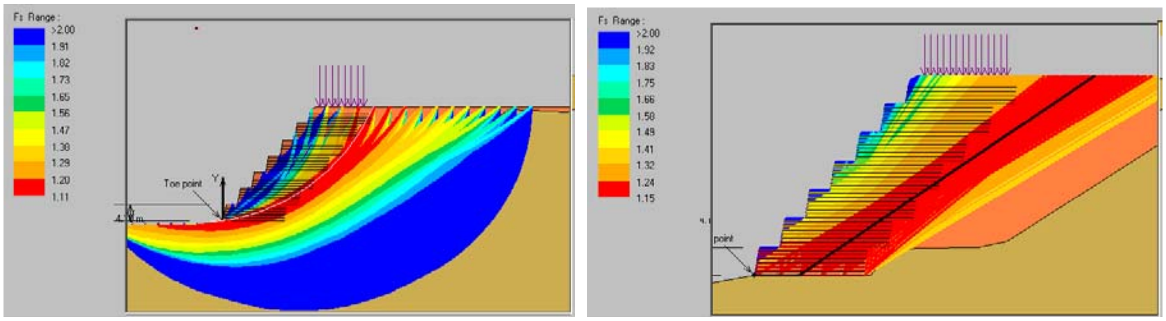

Fig. 5 - Safety maps of rotational and translational analyses in seismic condition for Northern round-about

The norm "OPCM n. 3274 / 2003" requires that the minimum Factors of Safety in seismic conditions shall benot lower than 1,10 for both rotational and translational analysises.
For both rotational analysis and translational analysis a pseudo-static seismic horizontal force F_{PS} is added to each wedge, applied to the wedge center of gravity (where W_i is the weight of the i-th wedge):

$$F_{PS} = a_g \times W_i \qquad [3]$$

The safety maps (Baker and Leshchinsky, 2001) of the seismic analyses for the most critical cross-section of the Northern round-about are shown in Fig. 5. In the design layout geogrids length has been set at 18,0 m, hence L/H = 0.60. Fig. 6 shows some pictures taken during construction of the reinforced soil structure: the accuracy of slope geometry and the connection between the reinforced soil and the concrete channel deserve particular attention.

2.5 Bearing capacity and settlements

Bearing capacity is evaluated, taking into account that the foundation of the reinforced soil structures is made up of the in-situ soil, considering an equivalent rectangular foundation whose width B is the length of geogrids at base and whose length is equal to the face length of the structure at base; the thickness of such foundation is assumed as nil, and the depth of the foundation, in respect of the surrounding in-situ soil, is set to zero as well.
For the calculation of the limit bearing capacity (q_{lim}) the general solution of Brinch-Hansen (1970) is used:

$$q_{lim} = 1/2\ g'\ B\ N_\gamma\ s_\gamma\ i_\gamma\ b_\gamma\ g_\gamma + c'\ N_c\ s_c\ d_c\ i_c\ b_c\ g_c + q'\ N_q\ s_q\ d_q\ i_q\ b_q\ g_q \qquad [4]$$

The meaning of symbols in the above equation is well known and is not reported here.
The allowable pressure (q_{amm}), that is the bearing capacity, is given as: $q_{amm} = q_{lim} / FS_{bc}$.
It was assumed that the soil, at the base of the reinforced soil structure, is made up of Alluvial sediments. Taking into account that this type of soil is essentially granular, and that draining pipes shall be installed inside the reinforced soil mass, it can be reasonably assumed that no pore pressure will arise: hence the bearing capacity calculation can be carried out considering the load application in drained conditions. In such hypothesis, for the tallest embankment (H = 30.40 m), with FS_{bc} = 3.0 it results:

Limit pressure (q_{lim}) =	3,404 kPa
Allowable pressure (q_{amm}) =	1,134 kPa
Applied pressure = $\gamma \cdot h + q = 18 \cdot 30.4 + 50$ =	597.2 kPa

Hence the bearing capacity is enough for supporting the weight of the tallest structure; but, given the proximity of the viaduct, whose column is just 1,0 m distant from the toe (see Fig. 6), a concrete foundation plate was designed all along the toe of the reinforced soil structure, in order to eliminate any horizontal displacement at base.
The settlements of the reinforced soil structure have been evaluated as well: taking into account that the soil is essentially granular, without appreciable cohesion, the calculation of settlements has been carried out using the Burland & Burbidge method, which is based on the results of penetrometric tests; the formula is the following:

$$S = f_S \cdot f_H \cdot f_t \cdot [\sigma'_{v0} \cdot B^{0.7} \cdot I_C / 3 + (q' - \sigma'_{v0}) \cdot B^{0.7} \cdot I_C] \qquad [5]$$

where:

- S = settlement in mm
- f_S, f_H, f_t = correction factors for the foundation shape, the thickness of the compressible soil layer beneath foundation, and the viscous component of settlements;
- σ'_{v0} = effective vertical stress at foundation level, in kPa;
- q' = total vertical pressure applied at foundation level, in kPa;
- B = foundation width in meter;
- I_C = compressibility index, equal to $1{,}706/NAV^{1.4}$,
- NAV = mean value of Nscpt along the effective depth (zi) beneath the foundation.

The following mean value has been assumed: Nscpt = 30

The maximum settlement at the base of the tallest reinforced soil structure resulted as follows: 67 mm immediate settlement and 90 mm settlement after 5 years. The immediate settlement occurs just during construction, hence it has no effect on the long term behaviour of the structure. Since reinforced soil structures are inherently flexible, a settlement of 90 mm for a 30 m high structure, with 18 m long geogrid layers, appears to be fully acceptable; moreover such a settlement at the base of the structure will be completely absorbed by reinforced soil, hence at the top of the structure the settlement will result to be negligible for the road structure and even for the abutments of the viaduct. Therefore calculations showed that both the bearing capacity and the settlements are fully acceptable.

Fig. 6 – Photos taken during construction of the reinforced soil structures

3. S.R. 232 "PANORAMICA ZEGNA"

The Piedmont Regional Road Agency, ARES Piemonte, is building the new stretch of Regional Road SR 232 “Panoramica Zegna”, with the aim of bypassing the town centres of Cossato, Vallemosso and Trivero. The new road stretch crosses many small valleys in the area, hence the design layout includes several

tunnels, viaducts, and reinforced soil embankments, showing the following features: maximum embankments height = 18 m; total vegetated vertical face = 12.700 sq.m; overall length of reinforced soil embankments = 2,000 m (on 5,200 m total project length). The tallest embankments (Fig. 7) have been designed with the cross section composed of three tiers with two horizontal berms; the bottom and intermediate tiers are set at 80° slope, in order to minimize land expropriation, while the top tier is set at 60° slope for better landscaping effects. The designed cross-sections span between 2.40 m and 18.0 m height. All geogrid layers have been designed at 60 cm vertical centres. Design included a 20 kPa variable surcharge, as required for first class roads, plus 20 kPa permanent surcharge, equal to the load provided by the road structure. At each viaduct abutment a reinforced soil structure connects the abutment and the reinforced embankment: these structures have been designed with vertical face and with alignment perpendicular to the embankment; hence the geogrids are placed at mid vertical centres of the embankments geogrids. Draining pipes have been placed inside the reinforced embankments, in order to eliminate dangerous pore pressure. At some positions along the embankments it was needed to place large diameter (up to 3,03 m) corrugated steel pipes, for allowing the flow of water from uphill to downhill area of embankments and for conveying the rain water flowing on the road on top. All reinforced soil embankments have been built using the local soil, mainly made up of silty sand. The soil characteristics for the design of reinforced soil structures have finally been set as: unit weight γ = 18 kN/m^3; friction angle ø = 30°; cohesion c = 0. The permeability of this type of soil is very low, due to the high percentage of silt. Hence for design the pore pressure was taken into account, through the pore pressure parameter Ru, which has been set equal to Ru = 0,125 for all calculations. In any case the design includes drainage pipes, wrapped with nonwoven geotextiles, spaced 3.50 m horizontally and 3.00 m vertically, in staggered pattern, in order to allow the water coming from the back of the reinforced block to flow out at the face without raising up the pore pressure. Woven Polyester geogrids have been specified for soil reinforcement: geogrids shall have a main mesh of 20 – 30 mm, with a second mesh of 2 – 4 mm, made up of thin filaments, inside the main mesh; this gossamer type mesh allows a better interlocking of the silty sand and affords to retain the soil at the face and to provide an excellent medium for supporting growing vegetation.
These special geogrids, called GTM, were specifically produced according to design specs, with tensile strength from 60 kN/m to 150 kN/m according to the height of embankments. Internal and global stability analyses has been performed as above explained. The design seismic acceleration for reinforced soil structures has been set as: $k_h = S\,(ag / g) / r = 0,03$. Results are shown in Fig. 8. Fig. 9 shows scheme and

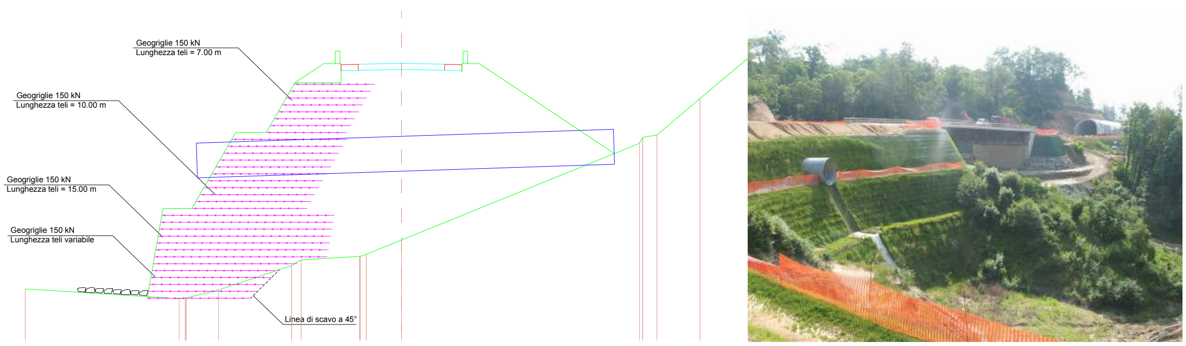

Fig. 7 – Cross-section and picture of the tallest embankment of SR232 project

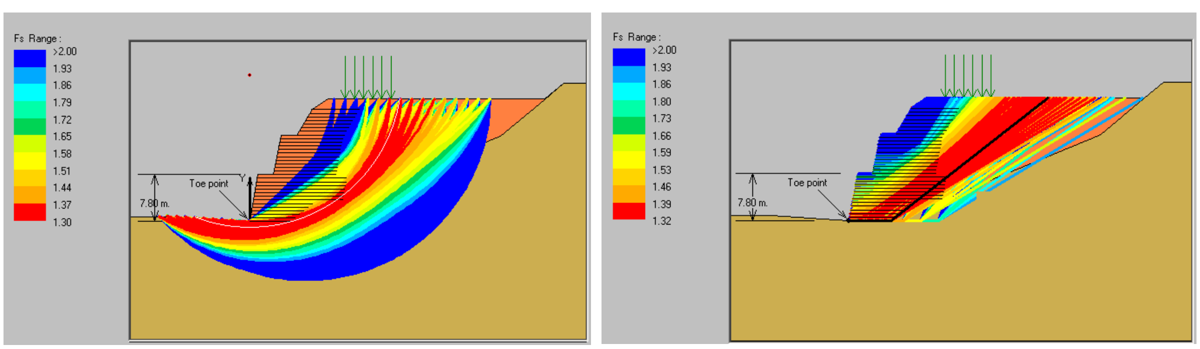

Fig. 8– Rotational and translational stability analyses results for the tallest embankment of SR232 project

details of the road platform drainage system and of the system for carrying water to embankment toe, through a PVC geomembrane and a Biomat. Fig. 10 shows the size and complexity of the project.

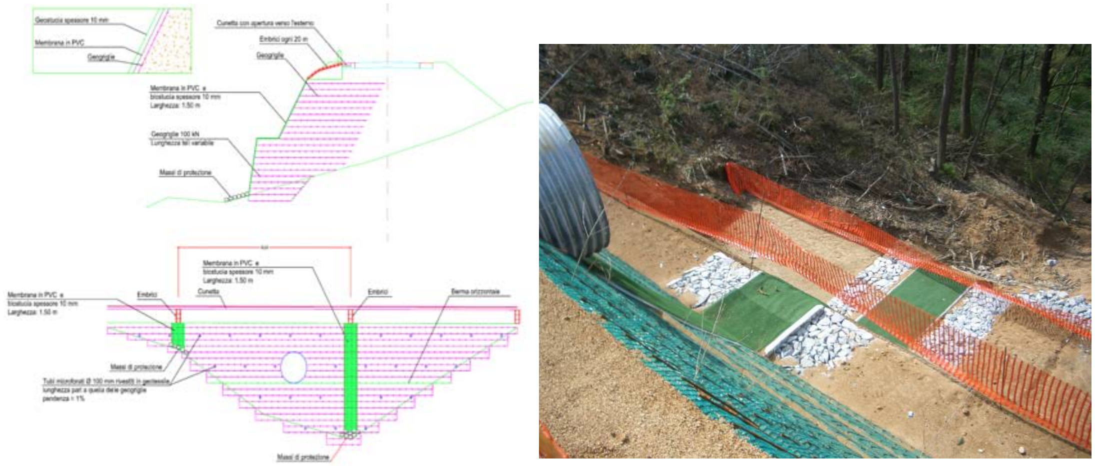

Fig. 9 - Scheme and details of the road platform drainage system and of the system for carrying water to embankment toe, through a PVC geomembrane and a geomat

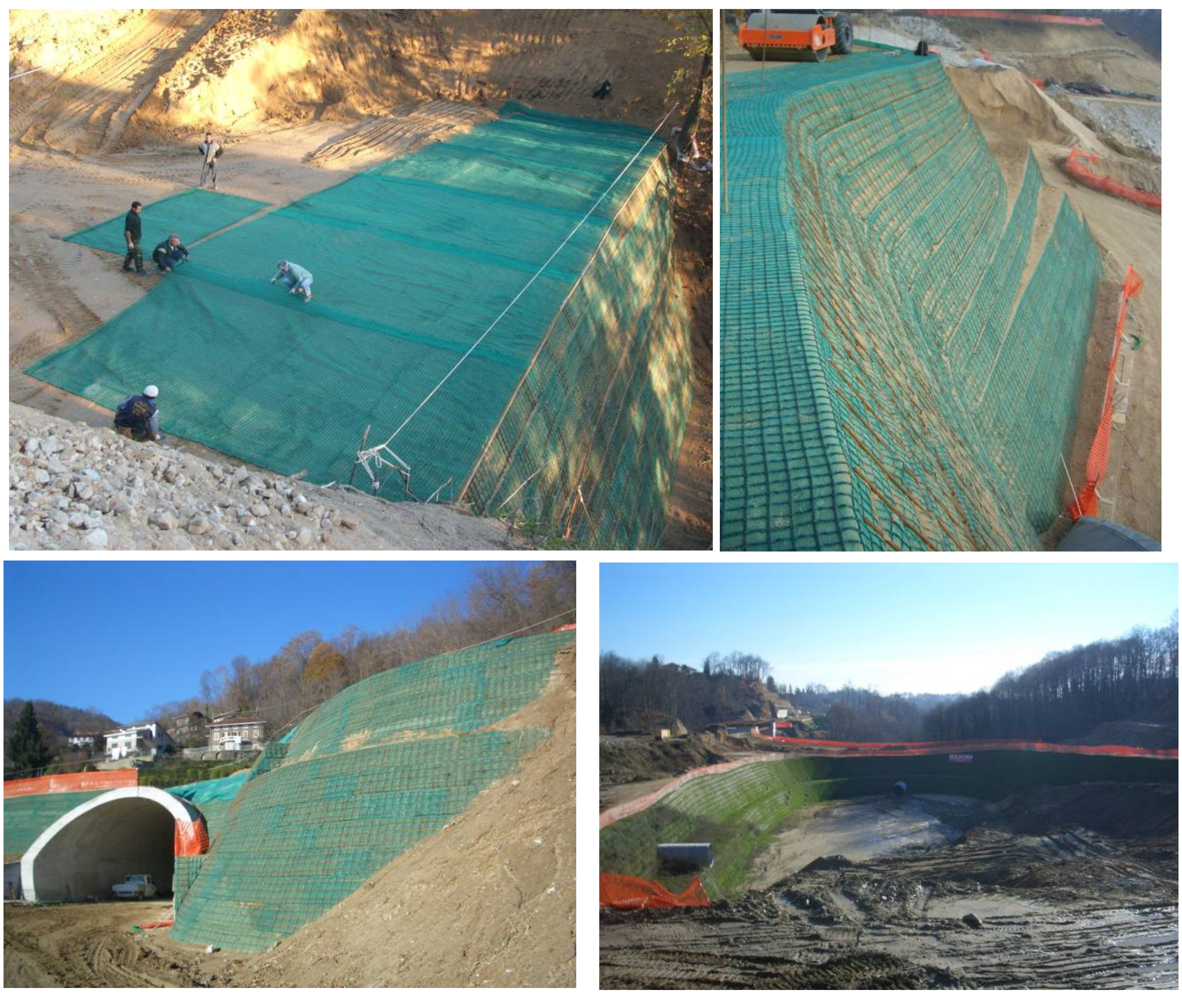

Figure 10. Construction of the reinforced soil road embankments of SR232 project

4. VERONA – BRENNERO RAILWAY LINE

Along the Verona – Brennero railway line, connecting Italy and Austria, in a stretch in front of a curve of Adige river in the town of Peri, a widening of the railway embankment was required for getting a 2.50 m wide service road up to 4.55 m height, 300 m long. Since the embankment toe couldn't be moved forward, a geogrid reinforced slope at 68° inclination had to be built, using the in-situ silty sand soil as fill (unit weight γ = 18 kN/m^3; friction angle $ø$ = 33°; cohesion c = 5 kPa). Design has been carried out by global stability analyses, where the critical condition was the rapid draw-down after the maximum recorded level of Adige river during flooding. Fig. 11 shows the results of such analyses for the most critical cross-section and pictures taken during construction of the reinforced soil structure.

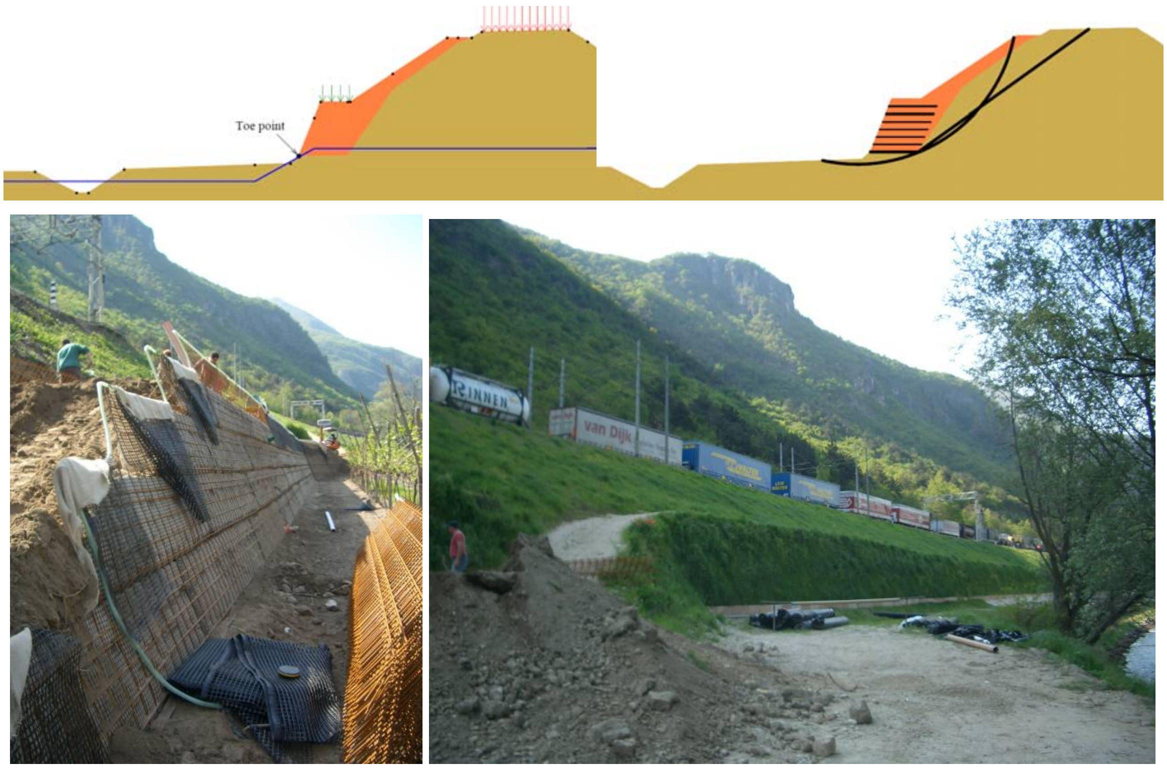

Fig. 11 - Stability analyses in condition of rapid draw-down and construction of the reinforced soil structure

5. CALCIUM STABILIZED AND REINFORCED SOIL STRUCTURES

In many areas of Italy, and particularly in high seismic areas, there is no or very little availability of granular soils: hence, for avoiding the cost and environmental impact of sourcing sand and gravel from very long distance, embankments and retaining structures are often built using the locally available fine soil. For improving the geotechnical characteristics of such soils and/or for building steep faced structures, it is possible to use the technique of calcium stabilized and reinforced soil. Rimoldi and Intra (2008) provide a detailed analysis of such technique. The project of the Provincial Road Ex SS 277 “Trasversale Alta Basentana – Bradanica”, close to the town of Grassano (Matera Province) in Southern Italy, included tall geogrid reinforced embankments, with total length of 840 m, height between 2.10 m and 9.30 m, for a total of almost 8.000 m^2 face in vertical projection. The cross- section includes 65° geogrid reinforced slopes, on both sides of embankments, a 2.0 m wide horizontal berm at crest, and on top a 5.0 m high unreinforced embankment with 2V:3H (34°) side slopes, which carries the road structure, providing 20 kPa uniform surcharge. All embankments had to be built with the locally available soil, that is silt and clay with variable sand content. The project is located in a highly seismic area: the design acceleration was a_g = 0,156 g. As shown in Fig. 12, finally the embankments were designed with a calcium stabilized and geogrid reinforced lower body, while the top unreinforced embankment is made up of compacted silty sand. Figure 12 also shows a picture of the Bradanica highway embankments during construction.

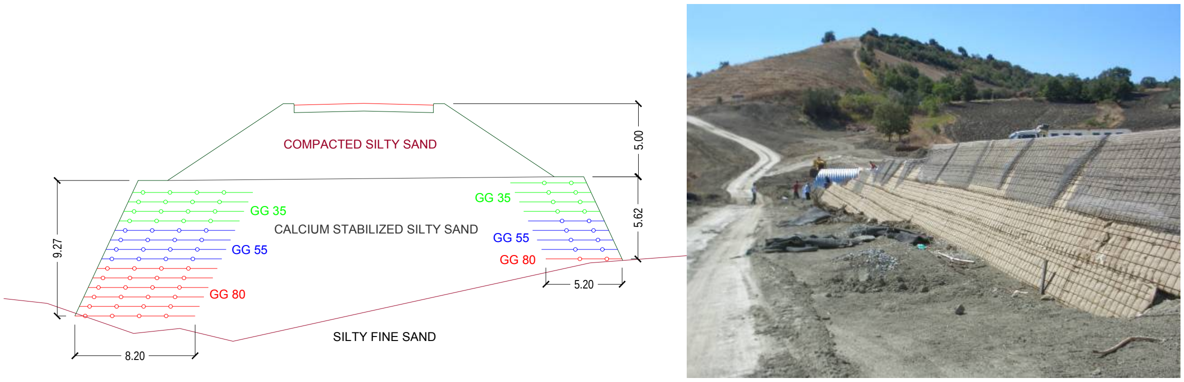

Fig. 12 – Design cross-section and picture of the Bradanica highway embankments during construction

The Bologna – Firenze highway, in Central Italy, is under reconstruction and many tunnels have to be excavated in the Apennines mountains, producing huge amount of debris. In the “Fienile” location outside the town of Barberino del Mugello, a 55.000 m^2 depression in hilly area was selected for dumping part of the debris, thus forming a 46 m high hill. The in situ soil (silt and clay) is sloping downward and the debris show very low friction angle (φ = 20°). Hence a dike, made up of the same debris, had to be designed to stabilize the toe. Barberino is in seismic area with design acceleration a_g = 0,156 g. Toto SpA Contractor, in charge of the construction works, considered both calcium stabilization and geogrid reinforcement for the dike: in this case environmental consideration of the effects of calcium powder forced to select only geogrid reinforcement. Fig. 13 shows the cross section, the global stability analyses in seismic conditions and pictures taken during construction of the dike.

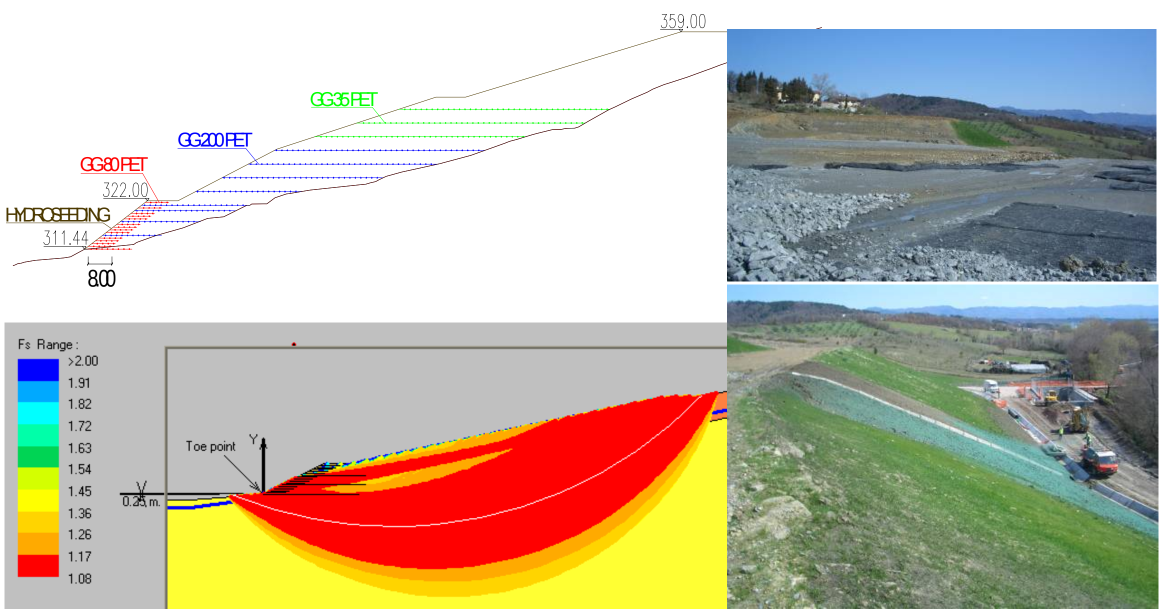

Fig. 13 – Design and construction of the dike for debris stabilization in Barberino del Mugello

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