Impressive Reinforced Soil Structures in Italy

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ABSTRACT
The paper intends to present few of the most impressive green faced reinforced soil structures built in Italy: a reinforced soil wall, 30 m high and 80° face inclination, supporting a bridge deck and a large turnaround of a major state road along the Arroscia river valley, in the Province of Imperia; over 20 m high road embankments for a 5 km stretch of the “Panoramica Zegna” regional road in Piedmont, crossing deep valleys and difficult soil conditions; a huge rockfall protection embankment, 20 m high, along the Simplon road (close to Swiss border). The paper introduces the static and seismic analyses, the design layout and the construction techniques, showing drawings, details and picture during and post construction.

RESUMEN
Se presentan algunas de las mas impresionantes estructuras en tierra reforzada con faza verde construidas en Italia en los ultimos años: un muro de 30 m de alto que suporta la caretera estatal Nr 28 en Pieve di Teco (Region Liguria); 2 km de rellenos reforzados, con 18 m de alto, por la caretera regional SR 232 en Cossato (Region Piemonte); un relleno de 20 m de alto por la protección de la caretera del Simplon, entre Italia y Suisa, contra la caduta de rocas.

1. INTRODUCTION
The application of reinforced soil structures began in Italy in the early '80 of last century. Since then hundreds of structures have been built in the most various environments and conditions, allowing the development of design and construction techniques and the growth of specialized engineering companies and contractors. Green faced walls and slopes make up the largest number of structures presently built.

The paper intends to present few of the most impressive green faced reinforced soil structures built in Italy: a reinforced soil wall, 30 m high and 80° face inclination, supporting a bridge deck and a large turnaround of a major state road along the Arroscia river valley, close to Genoa; over 20 m high road embankments for a 5 km stretch of the “Panoramica Zegna” regional road in Piedmont, crossing deep valleys and difficult soil conditions; a huge rockfall protection embankment, 20 m high, along the Simplon road (close to Swiss border).

Static and seismic analyses, the design layout and the construction techniques are introduced, showing drawings, details and picture during and post construction.

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

2.1 Project description
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 Contractor Lauro SpA of Borgosesia (Biella) carries out all the construction works.
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.

Here we will describe the most impressive reinforced soil structure, supporting the Northern round-about and having 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.
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

2.2 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.

The geotechnical model of the soil includes 7 layers, with the characteristics listed in Table 1.

Table 1 – Characteristics of the soils included in the geotechnical model

<table>
<thead>
<tr>
<th>SOIL</th>
<th>$\phi$ (deg)</th>
<th>$c$ (kPa)</th>
<th>$\gamma$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SURFICIAL DEBRIS</td>
<td>24</td>
<td>0</td>
<td>15,00</td>
</tr>
<tr>
<td>ALLUVIAL SEDIMENTS</td>
<td>34</td>
<td>20</td>
<td>21,00</td>
</tr>
<tr>
<td>UNALTERED BASE ROCK</td>
<td>55</td>
<td>0</td>
<td>24,00</td>
</tr>
<tr>
<td>ALTERED BASE ROCK</td>
<td>40</td>
<td>0</td>
<td>24,00</td>
</tr>
<tr>
<td>ROAD EMBANKMENT</td>
<td>35</td>
<td>0</td>
<td>24,00</td>
</tr>
<tr>
<td>REINFORCED SOIL</td>
<td>35</td>
<td>0</td>
<td>18,00</td>
</tr>
<tr>
<td>SUBGRADE</td>
<td>35</td>
<td>20</td>
<td>18,00</td>
</tr>
</tbody>
</table>

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 disaggregation of the limestone. Hence the design has taken into account the potential pore pressures by introducing the pore pressure coefficient $R_u = 0.10$. 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.

2.3 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. As said, 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 and variable width horizontal berms. Each tier, with a height $H = 4.20 \, \text{m}$, has been designed through internal stability analyses.

Since the reinforced soil structure is made up of many tiers and some cross-sections present the base inclined downward, the global stability analysis has been performed as well: the length and strength of the geogrids have then been increased when required, compared to the results of the internal stability analyses.

The calculation considered, besides the weight of the road structure, 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 design is based on the wrapped-around technique, where the geogrids function as soil reinforcement and soil retention at the face as well. The length of the wraps has been calculated with the internal stability analysis: since the required length is very short, less than 0.50 m, a constant wrapping length of 1.50 m has been specified, in order to simplify installation.

The following construction specs have been set: use Polyester woven geogrids with 60 mm x 60 mm apertures, in order to allow interlocking of the crushed rock soil; 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.

2.4 Internal stability

The design of a reinforced soil structure includes first the definition of the tensile strength, the spacing and the length of the reinforcing layers, in order to guarantee the required factors of safety for internal stability.

Design charts (Jewell, 1991) are available in literature for determining the equivalent soil pressure coefficient $K$ and the reinforcement length, as a function of the face slope, the friction angle of the construction soil and the pore water parameter $R_u$. Through the $K$ value got from the design charts, the total horizontal thrust $T$ that the reinforcing layer must withstand, the minimum reinforcement length at the base (to prevent direct sliding mechanisms) and at the top of the slope (to prevent pull-out of reinforcement) can be evaluated.

It must be noted that the face of a vegetated reinforced soil structure is not considered as a structural element: hence it is not taken into account in the stability analyses.

The design layout of each 4.20 m high tier required 7 layer of 80 kN/m strength woven Polyester geogrids, 0.60 m vertically spaced and 2.90 m long.

2.5 Design Strength of Geogrids

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

$$T_{amm} = \frac{T_{ult}}{FS_{creep} \times FS_{chemical- biological} \times FS_{construction}}$$  \[1\]

Moreover for complying with the Italian geotechnical norm D.L. 11.03.88, 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:

$$T_D = \frac{T_{amm}}{FS_G}$$  \[2\]

According to the importance and the design life of the structure the value of $FS_g$ 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 $FS_G$.

The calculation results in Table 2 have been set: for geogrids with $T_{ult} = 80 \, \text{kN/m}$ it results that $T_{amm} = 37.5 \, \text{kN/m}$.

2.6 Global Stability analyses

Once the structure has been designed for satisfying the internal stability conditions, it is 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 to be carried out are the following:
Table 2: Factors of Safety for Polyester woven geogrids

<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Values for woven Polyester geogrids</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil</td>
<td>Silty sand</td>
</tr>
<tr>
<td>$F_{S_{\text{creep}}}$</td>
<td>1.66</td>
</tr>
<tr>
<td>$F_{S_{\text{construction}}}$</td>
<td>1.30</td>
</tr>
<tr>
<td>$F_{S_{\text{chemical – biological}}}$</td>
<td>1.10</td>
</tr>
<tr>
<td>$F_{S_{\text{total}}}$</td>
<td>2.37</td>
</tr>
<tr>
<td>$F_{S_G}$</td>
<td>1.30</td>
</tr>
<tr>
<td>Direct shear factor $f_{ds}$</td>
<td>1.00</td>
</tr>
<tr>
<td>Pull-out factor $f_{po}$</td>
<td>1.30</td>
</tr>
</tbody>
</table>

- rotational analyses (circular failure surfaces): the soil mass is divided into many wedges limited at bottom by a circular surface (see fig. 2), for each one it is possible to calculate the active moments produced by the thrust of the soil and the resisting moments produced by shear and cohesive forces and the tensile strength of the reinforcing layers; the Factor of Safety is computed as the ratio between the sum of all resisting moments and the sum of all active moments; a very large number of circular surfaces is investigated and for each one the Factor of Safety is computed; the plot of safety maps helps to identify the zones where the reinforcement is required;

- translational analyses (horizontal sliding surfaces): the reinforced soil mass and a portion of the retained soil may slide as a block along the base or along one of the reinforcing layers; hence many bilinear surfaces are investigated (see fig. 2), each one having a horizontal segment which is moved along each reinforcing layer, and finally an inclined segment, whose inclination angle is varied for each length of the horizontal segment; in this way all the potential bilinear surfaces are investigated; the Factor of Safety is computed as the ratio of resisting forces over the active forces.

Fig. 2 – Scheme of rotational and translational stability analyses

According to the Italian geotechnical norm D.L. 11.03.88, the following Factors of Safety in static conditions are set:

- rotational stability: $F_{S_{\text{rot}}}$ = 1.30
- translational stability: $F_{S_{\text{trasl}}}$ = 1.30 (generally accepted value for reinforced soil structures, thanks to the large base length)
- overturning: $F_{S_{\text{rib}}}$ = 2.00
- bearing capacity: $F_{S_{\text{bc}}}$ = 3.00

2.7 Seismic analyses

With the recent 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.

According to the above norm, the design horizontal seismic acceleration is computed as:

$$k_h = S \left( \frac{a_b}{g} \right) / r$$  \[3\]

where:

- $a_b$ = 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 our reinforced soil structure it results:

\[ k_h = S \left( \frac{a_g}{g} \right) / \tau = 0.093 \]  

[4]

Since reinforced soil structures are not gravity structures, it can be assumed:

\[ k_v = 0 \]  

[5]

All reinforced soil structures have been checked in seismic conditions with \( a_g = 0.093 \) g, through the following analyses:

- rotational analysis: the same scheme above explained is used, but a pseudo-static seismic horizontal force \( F_{PS} \) is added to each wedge, applied to the wedge center of gravity:

\[ F_{PS} = a_g \times W_i \]  

[6]

where \( W_i \) is the weight of the \( i \)-th wedge.

- translational analysis: the same scheme above explained is used, but a pseudo-static seismic horizontal force \( F_{PS} \) (given by equation 6) is added to each wedge, applied to the wedge center of gravity.

The safety maps (Baker and Leshchinsky, 2001) of the seismic analyses for cross-section P12 are shown in Fig. 3.

2.8 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:

\[ q_{lim} = \frac{1}{2} \gamma' B N_s \gamma_i b_v g_v + c' N_q \gamma_i d_i b_c g_c + q' N_i \gamma_i d_i b_q g_q + \text{FSbc} \]  

[7]

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: \( q_{amm} = q_{lim} / \text{FSbc} \).

It was assumed that the soil, at the base of the reinforced soil structure, is made up of the following type (refer to Table 1): 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 section \( (H = 30.40 \text{ m}) \) it results:

Limit pressure \( (q_{lim}) = 3,404 \) kPa

FSbc = 3.0

Allowable pressure \( (q_{amm}) = 1,134 \) 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, a concrete foundation plate, with a 1.0 m deep tooth, 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:

\[ S = f_S \cdot f_H \cdot f_i \cdot \left[ \sigma'_{vo} B^{0.7} \cdot I_c / 3 + (q' - \sigma'_{vo}) \cdot B^{0.7} \cdot I_c \right] \]  

[8]
where:
$S$ = settlement in mm
$f_s, f_h, f_l$ = correction factors for the foundation shape, the thickness of the compressible soil layer beneath foundation, and the viscous component of settlements;
$\sigma'_{vo}$ = effective vertical stress at foundation level, in kPa;
$q'$ = total vertical pressure applied at foundation level, in kPa;
$B$ = foundation width in meters;
$IC$ = compressibility index, equal to $1,706/NAV^{1.4}$, where $NAV$ = mean value of $N_{scpt}$ along the effective depth ($z_i$) beneath the foundation.
The following mean value has been assumed: $N_{scpt} = 30$

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.

2.9 Design layout

The two most critical design cross-sections, P8 with the abutment on top and P9 with the concrete channel at mid height, are shown in Fig. 4.

![Fig. 4 – Designed cross-section P8 (H = 30.4 m, with abutment on top) and P9 (with the Rio Teco channel)](image)

2.10 Compaction tests

During construction several in-situ soil density tests and plate loading tests have been carried out, in order to check that design specs for soil compaction (soil density equal or greater than 95 % of maximum density measured in Modified Proctor test) were matched. As shown in Table 3, the soil results to be compacted respecting the design specs.

<table>
<thead>
<tr>
<th>Tab. 3 – Comparison of design specs and results of in-situ tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modified Proctor density</td>
</tr>
<tr>
<td>95 % Modified Proctor density</td>
</tr>
<tr>
<td>Minimum in-situ soil density test result</td>
</tr>
<tr>
<td>% of Modified Proctor density</td>
</tr>
<tr>
<td>300 mm plate loading test result</td>
</tr>
</tbody>
</table>

2.11 Construction

Fig. 5 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.
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 centers of Cossato, Vallemosso and Trivero. The Contractor Lauro SpA of Borgosesia (Biella) carries out all the construction works. 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: approx. 12,700 sq.m;
- overall length of reinforced soil embankments: 2,000 m (on 5,200 m total project length).

The tallest embankments 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 centers. 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 perpendicular to the embankment alignment; hence the geogrids are placed at mid vertical centers of the embankments' geogrids. Draining pipes shall be 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.
3.1 Geotechnical characteristics of soils

All reinforced soil embankments have been built using the local soil, mainly made up of silty sand. Two soil samples have been excavated at 200 m distant locations and sent to laboratory for geotechnical testing: granulometric analysis, slow consolidated direct shear tests and oedometric tests (which allowed indirect measurement of permeability as well) were performed. The soil characteristics for the design of reinforced soil structures have finally been set as reported in Table 4.

<table>
<thead>
<tr>
<th>SOIL</th>
<th>SOIL TYPE</th>
<th>ø (deg)</th>
<th>c (kPa)</th>
<th>γ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>REINFORCED SOIL</td>
<td>REMOLDED SILTY SAND</td>
<td>30</td>
<td>0</td>
<td>18,00</td>
</tr>
<tr>
<td>SUBGRADE</td>
<td>SLIGHTLY COHESIVE SILTY SAND</td>
<td>30</td>
<td>10</td>
<td>18,00</td>
</tr>
</tbody>
</table>

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 second thin 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. Actual geogrids have been Arter GTM 60-30-30, GTM 100-30-30, GTM 150-30-30, specifically produced by Alpe Adria Textil. Table 5 shows the design strength and ultimate strength of the geogrids for this project.

<table>
<thead>
<tr>
<th>T_{ann} (kN/m)</th>
<th>T_{ult} for woven Polyester geogrids (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>29.8</td>
<td>60.0</td>
</tr>
<tr>
<td>49.6</td>
<td>100.0</td>
</tr>
<tr>
<td>74.4</td>
<td>150.0</td>
</tr>
</tbody>
</table>

3.2 Stability analyses and design

Internal and global stability analyses has been performed as above explained. The design seismic acceleration for reinforced soil structures has been set as:

\[ k_s = S \left( \frac{a_{g}}{g} \right) / r = 0.03 \]

Given the small value of the seismic acceleration, the seismic analyses didn't change the design carried out in static conditions. Fig. 6 shows some of the reinforced soil structures built for this project.

4. ROCK FALL PROTECTION EMBANKMENT IN VARZO ALONG STATE ROAD N° 33 "DEL SEMPIONE"

The historical Simplon road in Ossola valley, State Road n° 33 "del Sempione", connecting the Piedmont region in Italy with Switzerland, runs through deep valleys surrounded by steep mountains. A big problem of this road has always been the fall of rocks and large boulders from the steepest slopes. The area of Varzo have been affected many times by these very dangerous phenomena. Hence, for the construction of the new road stretch over passing the town of Varzo, the Italian National Road Agency, ANAS SpA, Torino Department, has designed a huge reinforced soil embankment to protect the road against rock fall. The analysis of the falling patterns of rocks and boulders indicated that a 20 m structure, with impact resistance equal to 15,000 kJ impact energy, was required to protect the road. Given the available space and the steepness of the mountainous area, the only solution was to design a reinforced soil embankment structure, with both the downhill and the uphill faces at 70° slope. The only construction soil locally available was crushed stone, with 35° friction angle and 17 kN/m unit weight. Such soil is extremely aggressive for reinforcement. Moreover the reinforcement shall be extremely robust for withstanding the impact of large falling rocks without excessive damage. The design engineers selected extra heavy duty woven polypropylene Geogrids, with a very fine mesh of 3 mm and 340 kN/m tensile strength, as soil reinforcement. For increasing the resistance to high energy impacts, the reinforcement was specified both in the transversal and longitudinal directions of the embankment. All reinforcement layers had to be placed at 500 mm vertical centers. Laur o SpA Contractor completed the 600 m long, 20 m high protection embankment in 6 months. The geogrid finally approved was Arter GT 340-50-B, green color, purposely produced by Alpe Adria Textil. Fig. 7 shows the impressive rock fall protection embankment after completion and vegetation growth. The design of the rock fall protection embankment provided very hard challenges for engineers, due to the steepness of the mountain and the very little space available. Hence a concrete wall, stabilized by 90 kN/m steel tendons, 24 m long with 12 m long anchorage bulb, and 300 mm diameter micropiles, 12 m deep, was designed to support the embankment; given the proximity of the new viaduct, other stabilizing measures had to be implemented, as shown in the typical cross-section in Fig. 8: sub- horizontal drains made up of 50 mm diameter perforated PVC pipes, inserted into rotary excavated perforations; Tecco steel meshes fixed on the steepest slopes; geomats for erosion protection and vegetation support.
Fig. 6 – Reinforced soil structures built for the SR 232 project: slope protection at the outlet of a corrugated steel pipe; embankment at the exit of a tunnel; embankment connected to a viaduct; circular embankment at the “Berchelle” exit.

Fig. 7 – The reinforced soil rock fall protection embankment along the Simplon road: view of the vegetated embankment showing the cross-section; aerial view showing the embankment protecting the new road stretch and viaduct

ACKNOWLEDGEMENTS

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Fig. 8 – Typical cross-section of the geogrid reinforced soil rock fall protection embankment in Varzo

Fig. 9 – The rock fall protection embankment in Varzo along the Simplon road, during construction

REFERENCES