Base and slope reinforcement of very tall reinforced embankments for the A1 highway in Italy

Rimoldi, P.
World Tech Engineering Srl, Milano, Italy

Talone, F.
Toto SpA, Chieti, Italy

Keywords: Slope Reinforcement, Base Reinforcement, Geogrids, Geocomposites, Gabions

ABSTRACT: for the new stretch of the A1 highway, connecting Milan to Naples in Italy, in Barberino del Mugello, in Tuscany close to Florence, geogrid reinforced embankments have been built, with height up to 46 m, in seismic area. Geogrids were used both for base and slope reinforcement. Gabion channels, waterproofed with geomembranes, have been built as well for discharging the runoff flow from surrounding hills and highway platform. The paper illustrates this impressive project showing the design criteria and methods, construction details, drawings and pictures, thus enhancing the possibility of solving highly demanding engineering projects through the use of geosynthetics.

1 INTRODUCTION

The present paper deals with the A1 highway, connecting Milan to Naples, which is the most important road in Italy. Between Bologna and Florence the highway crosses the Appennine mountains. The original route, built in the '50s and '60s of last century, was tortuous and dangerous, with only 2 lanes per direction and very sharp curves. Finally a new route, the so called "Variante di Valico" has been designed, with 3 lanes per directions and gradual curves, allowing fast and safe driving. The new route includes several tunnels; the valleys between consecutive tunnels are crossed either by viaducts or embankments. In Barberino del Mugello, in Tuscany close to Florence, geogrid reinforced embankments have been built, with height up to 46 m, in seismic area. Geogrids were used both for base and slope reinforcement. Gabion channels, waterproofed with geomembranes, have been built as well for discharging the runoff flow from surrounding hills and highway platform.

2 REINFORCED EMBANKMENT SLOPES

The project herein described deals with the "Lora" interconnection, in Barberino del Mugello between the "Puliana" tunnel at chainage km 17+210 and the diversion channel at chainage km 17+425.

Here the highway pattern has to cross a small valley, deep and steep, whose toe is approx. 40 m below the design road level.

The soil of the valley includes the "Acquerino" formation of sandstone and siltstone, locally covered by chaotic deposits of clay – silt – sand, at limit equilibrium conditions or even being dormant landslides. Hence the geotechnical characteristics are medium to low.

Given the large availability of debris from the Puliana tunnel excavation, Toto Spa Contractor, in charge of building this highway stretch, decided to propose to cross the valley by filling it with a tiered embankment, reinforced with geogrids and, since the debris shows low permeability, with internal drainage through strips of geocomposites. All the zone around Barberino is in level 2 seismic area, according to Italian construction code, hence a reinforced embankment affords the best guarantee of seismic resistance, as shown by the behaviour of reinforced soil structures which recently withstood earthquakes with Richter magnitude higher than 7 (like in Kobe, Japan; Taichung, Taiwan; and S. Francisco, California). Moreover the relatively low slope and the presence of 5 berms along the embankment height afford a proper face vegetation and a pleasant environmental insertion.

The Highway Authority approved the proposal and World Tech Engineering Srl, based in Milano, was charged by Toto Contractor of designing all the relevant structures.

The soils properties were investigated through boreholes, SPT penetrometer tests, and laboratory tests. From this large amount of data the geotechnical model for designing the reinforced soil embankment has been set, as follows:

Base soil:

- unit weight γ = 20,00 kN/m^3
- friction angle ø = 27°
- cohesion c = 15,00 kPa

Surface alteration of the Acquerino formation:

- unit weight γ = 20,00 kN/m^3
- friction angle φ' = 26° - 30°
- cohesion c = 0,00 kPa

Acquerino rock formation:

- unit weight γ = 24,00 kN/m^3
- friction angle φ' = 35°
- cohesion c = 1.000,00 kPa

For the tunnel debris, being a mixture of surface alteration of the Acquerino formation and Acquerino rock formation, the friction angle has been set as the upper limit of the friction angle of the surface alteration soil, that is a φ'_k = 30°; taking into account the remolding of the debris and the consequent loss of the original soil properties, the friction angle has been factorized as follows:

$$\varphi'_d = \arctan(\tan \varphi'_k / \gamma_s)$$

Since γ_s = 1.25, then: φ'_d = 25°.

Finally for the reinforced embankment fill the following parameters have been set:

- unit weight γ = 20.00 kN/m^3
- friction angle φ' = 25°
- cohesion c = 0 kPa

Geocomposite strips have been designed for the internal drainage of the reinforced fill mass; hence for stability analyses all the fill has been considered as self draining, that is with nil pore pressure.

On the top of the embankment, along the edge, a soil dune has been designed as traffic safeguard; such dune, shown in Fig. 1, applies a permanent strip surcharge of 36 kPa along the embankment top edge.

Considering the weight of the road structure and the traffic load, the uniform load on the embankment top surface has been set equal to 40 kPa.

Considering the excavation of the soil at toe, the reinforced embankment has a maximum height of 46.75 m. Hence the reinforced structure has been designed as a tiered embankment; each tier is 8,0 m high, with 2,5 m wide horizontal berms and with face slope at 22°. All geogrids has been designed at vertical centres of 2,00 m, for ease of construction.

Each tier has been designed with internal stability analysis, using the Jewell method (Jewell, 1991), considering the surcharge provided by the tiers on top of it. From such analysis a minimum geogrid length of 25 m has been set for the 2 top layers.

q1	40.00	(kPa)	sovraccarico stradale
q2	36.00	(kPa)	sovraccarico duna
Sovraccarichi			
γ	20.00	(kN/mc)	peso specifico duna
a	2.60	(m)	
b	1.00	(m)	
h	2.60	(m)	
P	93.60	(kN/ml)	peso
q2	36.00	(kN/mq)	sovraccarico

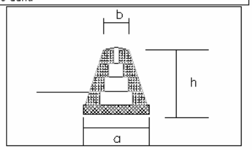

Fig. 1 – The soil dune at embankment top edge

Then global stability analyses, both in static and seismic conditions, has been carried out for a total of 13 different cross-sections.

All the stability analyses showed that the whole embankment could be reinforced with a single type of geogrids, that is high tenacity polyester woven geogrids with ultimate tensile strength of 80 kN/m.

Hence, considering also the type of fill, the following reduction factors have been used: RF_{creep} = 1.67; $RF_{chemical}$ = 1.10; $RF_{construction}$ = 1.10. Therefore the design strength was: T_D = 39.6 kN/m.

The length of each geogrid layer has been set as the maximum value from internal, static, and seismic analyses. The top geogrid layer has a higher length, equal to the overall highway width, in order to have uniform reinforcement conditions below the road structure.

Given the mild slope of each tier, geogrids are not wrapped around the face, but laid horizontally. For vegetating the embankment face, hydroseeding has been specified.

For the stability analyses in static conditions, according to the Italian geotechnical norm, the following Factors of Safety have been set:

- rotational stability: FSrot = 1.30
- translational stability: FStransl = 1.30

Seismic analyses has been performed with the pseudo-static method, according to the Italian norm "OPCM n. 3274 / 2003", which established the seismic classification of Italian territory where the 2^{n}d seismic category has been assigned to the area of Barberino del Mugello,. According to this norm, the design horizontal seismic acceleration parameter is computed as:

$$k_h = S\,(a_g / g) / r$$

where:

a_g = peak bedrock acceleration = 0.19 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 it results: k_h = 0.119.

According to the above mentioned norm the minimum Factor of Safety in seismic conditions shall be equal to 1.00 for both rotational and translational analyses.

All the global stability analyses has been carried out using the ReSSa software developed by Prof. Leshchinsky. The safety maps (Baker and Leshchinsky, 2001) of the seismic analyses for the tallest cross-section are shown in Fig. 2.

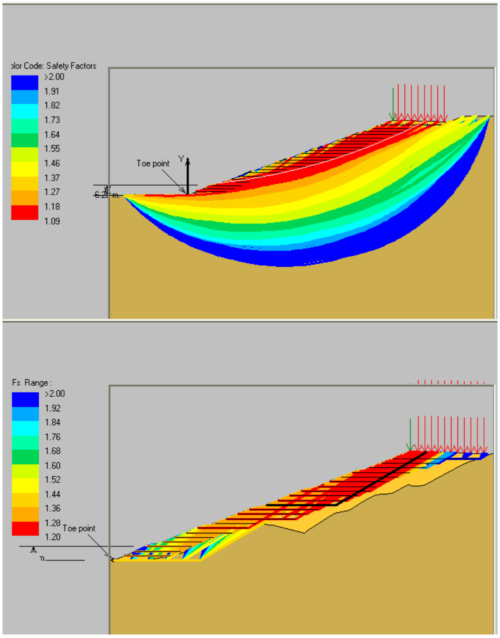

Fig. 2 - Safety maps of the seismic analyses

3 GEOCOMPOSITE DRAINING STRIPS

As said, design of the reinforced soil embankment has been carried out in the hypothesis of self draining fill. This is possible when the fill has a higher transmissivity than the infiltration flow rate.

Let's suppose than an intense rainfall has intensità j = 100 mm/h; considering that the fill has low permeability, we can reasonably suppose that infiltration rate is equal to α = 5 %; hence the unit infiltration flow rate will be equal to:

$$q = 2.78 \cdot 10^{-7} \cdot j \cdot \alpha = 1.39 \text{ x } 10^{-6} \ \ m^3/s/m^2$$

Since the average draining pattern, from the back of the reinforced soil body to the face, has a length L = 40 m, the total flow rate to be drained is:

$$Q = q \cdot L = 5.5 \text{ x } 10^{-5} \ \ m^2/s$$

Horizontal geocomposites draining strips will provide the drainage of infiltration water: draining strips will collect the infiltration water, which moves vertically, and will carry it horizontally to the face, thus keeping pore pressures practically equal to zero.

Let's select GMG 612 geocomposites, placed in 0.50 m wide strips. For sub-horizontal installation (hydraulic gradient i = 0.10) and 20 kPa applied pressure (at embankment top, which is the critical condition), these drains will afford a flow rate Q_d = 4.4 x 10^{-4} m^2/s.

Hence the influence area of each strip is:

$$A = Q_d / Q = 8.0 \ \ m^2$$

Since the draining strips shall have the same vertical spacing as the geogrid layers, equal to S_v = 2.0 m, then the horizontal spacing shall be:

$$S_h = A / S_v = 4.0 \text{ m}$$

Hence the selected draining strips shall be placed in staggered pattern at 4.0 m H : 2.0 m V centres.

4 GABION CHANNELS

The waters coming from the upstream portion of the valley and from the road surface are collected by a draining system and conveyed at the right and left sides of the reinforced embankment. Then these flows shall be conveyed to the embankment toe and released in a natural creek.

Hence two gabion channels has been designed, for carrying the two separate flow down to the toe.

The gabion channel at chainage Km 17+225.00 has been designed for a discharge of 1.35 m^3/s, while the gabion channel at chainage Km 17+425.00 has been designed for a discharge of 0.84 m^3/s, both corresponding to 100 years return time.

Both channels has been designed with water falls every 1.50 m vertically. At the toe both channels discharge into a dissipation tank, made up of gabions as well, from where water is finally released to the creek. The upper edges of the channels at each fall are always lower than the corresponding berms of the reinforced soil embankment, in order to be able to catch the surface runoff on the berms. Considering the standard gabion dimensions, channels have been designed with 2.00 m wide bottom bed, made up with 0.50 m thick gabions, and lateral walls made up with gabions having 1.00 m x 1.00 m and 0.50 m x 1.00 m cross-section, as shown in Fig. 3.

Environmental protection requirements dictated that no water had to infiltrate into subgrade from the gabion channels: hence design included waterproofing of the entire external channel perimeter with PVC geomembranes, protected on both faces with 300 g/m^2 nonwoven geotextiles, as shown in Fig. 3.

Hydraulic channel design has been carried out by considering a mono-dimensional permanent flow and neglecting the solid transport, that is with constant cross-section over time. Validation of such hypothesis required checking that flow velocity and shear stresses are always lower than the critical values at which gabions could be damaged.

Permanent motion profiles has been computed using the well known HEC-RAS V. 3.0 (Hydrologic Engineering Center – River Analysis System) code.

This code affords to model each single hydraulic cross-section, taking into account a different roughness for each stretch of the watercourse and even for different parts of the cross-sections. For our gabion channels roughness has been uniformly modelled using the Gauckler – Strickler parameter Ks = 45.

The hydraulic profile for Hec-Ras modelling of the gabion channel at chainage km 17+225 is shown in Fig. 4. Design flow rate always flows with less than 50 % channel filling.

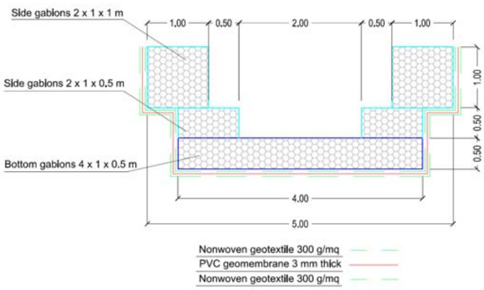

Fig. 3 – Gabion channels cross-section layout

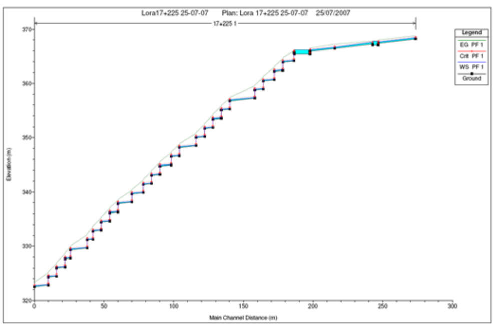

Fig. 4 – Hydraulic profile for Hec-Ras modelling of the gabion channel at chainage km 17+225

5 EMBANKMENT BASE REINFORCEMENT

The highway pattern design in Barberino included the "North Diversion", that is a stretch where the road embankment had to be 9.87 m high (maximum), built on soft silty – sandy clay. Such soil didn't afford adequate bearing capacity, therefore the embankment had to be designed with geogrid reinforcement in order to avoid excessive settlement and/or dangerous tension cracks.

Subgrade had the following geotechnical properties:

- unit weight γ = 19.00 kN/m^3
- friction angle $\o_u$ = 0°
- cohesion c_u = 25.5 + 6.1 x Z kPa

The embankment had to be built using tunnel debris as fill, with the following properties:

- unit weight γ = 19.00 kN/m^3
- friction angle $\o_{cv}$ = 27°
- cohesion c = 0 kPa

Design of the reinforced embankment has been carried out using a specifically developed software based on norm BS 8006:1995 - Code of practice for strengthened/reinforced soils and other fills.

Calculations showed that, for the tallest embankment height of 9.87 m, 7 layers of 200 kN/m tensile strength high tenacity polyester woven geogrids were required, placed at 1.00 m vertical centres. Fig. 5 shows the design layout. Fig. 6 shows the results of F.E.M. analysis with Plaxis 8.2 code: it is interesting to note how geogrids can make embankment settlements uniform.

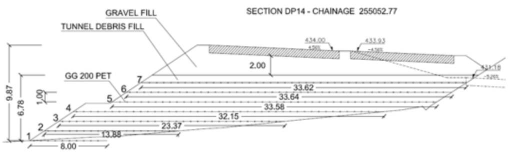

Fig. 5 – Design layout for the 9.87 m high geogrid reinforced embankment

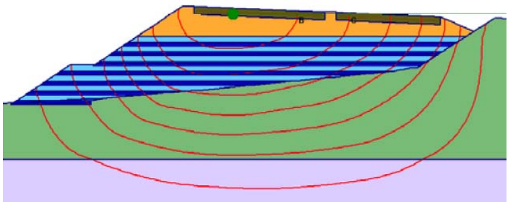

Fig. 6- F.E.M. analysis results

6 CONSTRUCTION

The new highway stretch in Barberino runs next to the old route. Construction works for the above described structures had to be carried out with the old highway always in operation. Safety measures and accessibility for trucks carrying over 400.000 m^3 of tunnel debris caused relatively low construction rates. Anyway construction works proceeded smoothly and at present they are almost completed.

Geosintex Srl (Sandrigo, Vicenza, Italy) supplied all geogrids and geocomposite draining strips; Officine Maccaferri (Bologna, Italy) supplied the gabions. Figures 7 – 10 show the construction of the 46 m tall reinforced embankment; Fig. 11 – 13 show the gabion channel; Fig. 14 – 17 show the base reinforced embankment. The use of geosynthetics afforded to get excellent results both in terms of technical quality, ease of construction and environmental insertion, with full satisfaction both of the Contractor and the Highway Authority.

References

Baker, R, and Leshchinsky, D. (2001) “Spatial Distribution of Safety Factors”, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 2, February 2001, pp. 135-145

Jewell, R.A. (1991). Application of Revised Design charts for Steep Reinforced slopes, *Geotextiles and Geomembranes*, 10.

Fig. 9 – Fill compaction over geogrid reinforcement

Fig. 7 – The valley before embankment construction

Fig. 10 – Filling gabions of the channels

Fig. 8 – Laying the first geogrid layer

Fig. 11 – The gabion channels seen from the toe

Fig. 12 – The reinforced embankment and the gabion channel during construction: the bottom part is already vegetated, the upper part has just been hydroseeded

Fig. 13 – The reinforced embankment and the gabion channel almost completed

Fig. 14 – Construction of the base reinforced embankment

Fig. 15 – Construction of the base reinforced embankment

Fig. 16 – Geogrid installation for the base reinforced embankment

Fig. 17 – The base reinforced embankment near completion