Proceedings of the XVII ECSMGE2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0133

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

A geosynthetic reinforced steep slope 60.0 m high for the stabilisation of the Valpola landslide in Northern Italy

Une structure de 60,0 m de haut renforcée avec géosynthétiques pour la stabilisation du glissement de terrain de Valpola dans le Nord d'Italie

Daniele Cazzuffi
CESI SpA, Via Rubattino , 54, I-20134 Milano (Italy)

Piergiorgio Recalcati
TENAX SpA, Via dell'Industria, 3, I 23897 Viganò (LC) (Italy)

ABSTRACT: A rock avalanche of 3545 million cubic meters collapsed in 1987 on the right side of the Adda river in Northern Italy; the felt-down material created a natural dam over 80 m high and about 2.0 km long, that completely closed the valley. The left side of the valley, that was not interested directly by the landslide but has been struck by the landslide mass, was covered by the coarse debris; the removal of the debris was making necessary the reinforcement of the left side. The slope was reconstructed with 6 geogrid reinforced structure, each 10 m high (thus having a total height of 60 m). This is one of the tallest reinforced structures ever designed and constructed in Europe. One year after the end of construction, the reinforced structure was attacked by fire; a prompt inspection of the site was confirming the job was not suffering any serious damage.

RÉSUMÉ: Une avalanche de roches de 3 545 millions de mètres cubes s'est effondrée en 1987 sur le côté droit de la rivière Adda, dans le Nord de l'Italie; le matériau créé un barrage naturel de plus de 80 m de haut et environ 2,0 km de long, qui a complètement fermé la vallée. Le côté gauche de la vallée, qui n'était pas directement intéressé par le glissement de terrain mais qui avait été touché par la masse du glissement de terrain, était recouvert par les débris grossiers; l'enlèvement des débris nécessitait le renforcement du côté gauche. La côté a été reconstruite avec 6 structures renforcées de géogrilles, chacune d'une hauteur de 10 m (d'une hauteur totale de 60 m). C'est l'une des plus hautes structures renforcées jamais conçues et réalisés en Europe. Un an après la fin de la construction, la structure renforcée a été attaquée par un incendie. une inspection rapide du site confirmait que le travail ne subissait aucun dommage grave.

Keywords: landslide; steep slope; reinforced structure; geogrid.

1 INTRODUCTION

During July 1987 the western Alps of Italy was hit by unusual rainfall: since the end of June there were repeated thunderstorms over the Alpine chain. On July 17, violent storms raged across the Alpine region of Lombardia. On the 18th and 19th of July the heaviest rainfall occurred: in some areas of the Sondrio province peaks of precipitation of 305 mm in 24 hours were reached. The heavy rainfall caused severe flooding throughout Valtellina and in the neighboring provinces; landslides, debris flows with serious damage to infrastructure occurred.

Between July 18th and 20th the first intense flooding along the Valpola, a small valley on the right side of Valtellina near S. Antonio Morignone, occurred. The flooding formed a large fan base which partially obstructed the natural flow of the Adda river and caused the formation of a small lake (250,000 m² and a maximum depth up to 10 m) that reached the village of S. Antonio Morignone. In the immediate aftermath, the river opened a breach in the natural dam and the waters subsided, reducing the lake surface up to about 15 ha. In the meantime, the process of violent erosion channeled along the Pola continued and rockfall and small landslides were observed. On July 28, 1987 at 7.23 a rock avalanche of 3545 million cubic meters collapsed from mount Zandila, falling down into the river 1200 m below and running up about 300 m on the opposite side of the valley (Figure 1), causing 28 casualties.

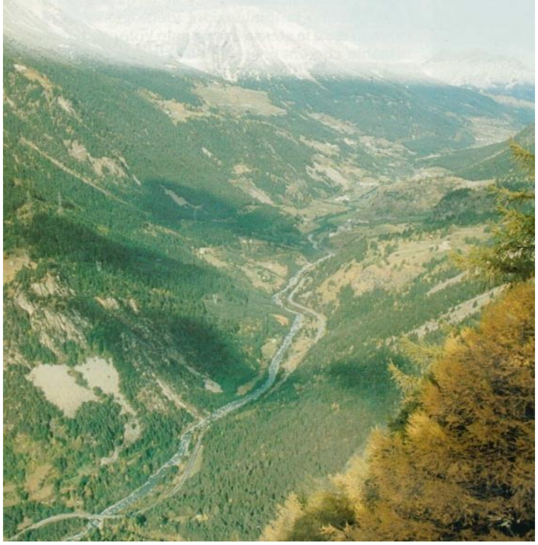

Figure 1 Aerial view of the area before and after the landslide

Figure 2 The lake and the position of the villages destroyed by the landslide

The main rockslide body, detached between the crown located at 2360 m a.s.l. and the toe of the rupture surface at 1750 m a.s.l., fell down in just 30 seconds destroying the villages of Morignone, S. Antonio, Poz, Tirindrè, S. Martino and Aquilone; most of the villages (S. Antonio, Poz, Tirindrè and Aquilone) have not been directly invested by the landslide mass but by a catastrophic wave of water and debris caused by the impact of the rock mass into the lake formed in the previous days (Figure 2).

The collapsed material was accumulated out along the valley of the Adda and climbed the other side of the valley for about 300 meters creating a natural dam 80 to 100 m high and about 2.0 km long.

The landslide produced a new and more massive barrier to the course of the Adda river which caused the rapid formation of a lake much larger than the previous one; in the next few weeks since the end of August, the lake reached an altitude of 1097.62 m with a water head of about 20 m, only 10 m below the elevation of the top of the natural dam, and this created a new and severe danger for the valley below due to the possible collapse of the natural barrier.

The lake was partially eliminated with pumps, and then, during the following year, the river was diverted creating artificial tunnels bypassing the landslide body.

The landslide involved mainly gabbroic rocks and, to a lesser extent, the paragneiss that can be found at higher altitudes. The movement had a strong structural control having developed according to the following and welldefined structures:

- the Valpola fault;
- two major structural planes along which occurred the initial;
- the intense fracturing of rock mass that develops following a second set of discontinuities similar to those of the two main structures above.

In addition to the geological structural features, it is possible to identify some special conditions that have been, as a whole, the triggering factor. First, the intense erosion process that deepened the watershed of the Valpola, reducing the contrast at the foot of the main plane of movement. Secondly, the unusual weather conditions that led to heavy water penetration in the rock mass and to the consequent sharp increase in hydraulic pressure.

2 GENERAL DESIGN PRINCIPLES

The whole plan was based on the concept that it was not only a hydraulic design, but with the aim to give a final and safe configuration to a wide area affected by natural catastrophes over the past 25 years.

The first design choice was to restore the valley to its original appearance before the landslide.

This made it essential to situate the river along the valley and retrieve the flat area between Aquilone and San Bartolomeo, in compliance with the natural restoration of the whole area. To this end, various assessments were carried out relating to issues potentially affecting the riverbed, and mainly due to the risk of runoff, both of liquid and solid flow, and to slopes' stability.

The design was also shaped by several other constraints, concerning the morphology of current location, the flow of the river, the functionality of existing galleries, safety during works and easy maintenance afterwards.

The most binding hydraulic parameter was the river path gradient. While the size of the new path did not represent a problem, because of the vastness of the interventions planned, the tilt constrained the design at multiple points, both because elevations were hard to modify and in terms of the effect of existing works.

Not less important was the presence of the landslide accumulation that elevated the ground by approx. 100 meters, and which was not removable except for a limited area; the presence of Aquilone houses, which require ground levels to protect from the risk of flooding during intense events; the so called “Arginone” on the right side and the landslide body on the left side, which restricted the possibility of deepening the riverbed on left side; and, last but not least, the presence of the bridles made, with significant financial expenditure, at the time of the emergency.

The solution finally designed is different from any other and unique in its genre, as it upgrades the downstream portion using techniques with the least possible impact, and providing the reuse of in loco material, in particular along the left slope through the geogrid reinforced slope method.

Concerning the entire area, the design has involved the following activities:

- ground movement for 3.000.000 m³;
- construction of reinforced soils for 25.000 m² along the left slope;
- recovery and reuse of existing materials on site (prisms of concrete, stone, vegetation, soil, etc..) without any external input;
- implementation of a new river bed for the Adda River;
- reorder of tributaries, both from left and right side, and of water drainage;
- adjustment of the provincial road;
- general environmental restoration (Paoletti and Griffini, 2007).

Bringing back Adda’s path to its site necessitated to design relevant excavations of the existing path, lowering the fund up to 20 meters.

Obviously, the new path has required the design of thresholds and bottom side protections. The most significant design of this area, which is widely described in the following chapters, was the one concerning the reprofiling of the left bank and the building of a safe reinforced soil slope with geogrids.

To allow excavation two series of sheet piles were installed, each 1518 m high and with ties 2830 m long. Along this slope, the provincial road was restored, and a general environmental restoration has been implemented.

3 DESIGN SPECIFIC ASPECTS

In addition to the geological and geotechnical investigations carried out to study the triggering factors of the large landslide and to evaluate residual hazard, a new investigations campaign has been carried out: 40 boreholes (1250 m continuous core drilling and 650 m non-coring drilling), 1350 m seismic refraction survey, 680 m seismic refraction tomographic elaboration, 40.000 m² electrical resistivity tomography (ERT) were executed.

Based on the results of the investigations a geotechnical model has been developed. The geotechnical characteristics have also been defined on the basis of direct surveys in trial pits that allowed to estimate the grain size of the deposits in the cortical portion (Paoletti et al., 2013).

Calculations of the steep slopes have been performed according to Italian “Norme Tecniche delle Costruzioni”, dated 14/01/2008 (NTC08). The Design geosynthetic resistance is obtained by applying partial factors to the long term properties of the reinforcement.

Based on the results of the investigations carried out the following stratigraphic sequence can be recognized (Figure 3):

- Unit 1: surface level (gravel and blocks, including boulders of various meter size in gravel and sandy-gravel matrix), with voids up to 300 mm, moderately dense; velocity of compression waves $V_p \leq 2000$ m/s
- Unit 2: deep level (blocks and boulders with gravel and sandy-gravel matrix), with voids up to 300 mm dense to very dense; velocity of compression waves 2000 m/s $\leq V_p \leq$ 3000 m/s
- Unit 3: bedrock (gabbro and diorite), velocity of compression waves $V_p \geq 3500$ m/s

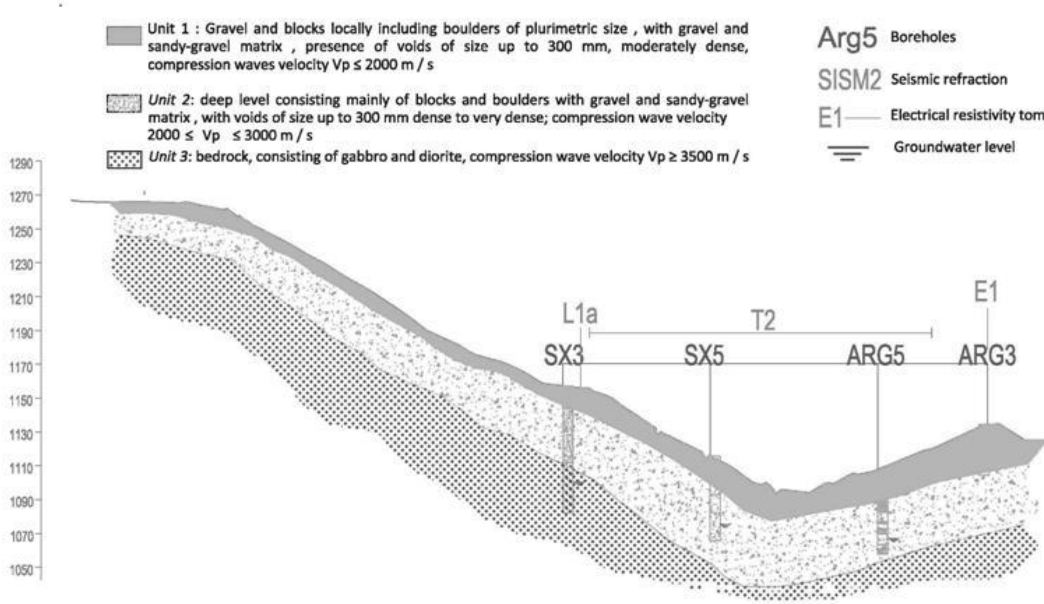

Figure 3 Typical geotechnical cross section

The whole slope was reconstructed with 6 geogrid reinforced slopes, each 10 m high, separated by berms (thus having a total height of 60 m); a road was foreseen over the first slope. Slope inclination was 60°, and the spacing was kept constant and equal to 700 mm.

The geogrids used in the project are manufactured by extruding and mono-directional drawing of high-density polyethylene (HDPE). The choice of this product has been done because of its resistance to installation damage even with coarse aggregate (such as the one used for this job), the good tensile properties and the possibility to have on site rolls with exactly the required length coming from calculation, thus avoiding the need to cut the rolls on site.

The allowable Resistance of a geogrid (see eq. 1) was determined as a fraction of the Long Term Design Strength (LTDS) derived by creep tests by means of a Partial Safety Factor. Those factors in the Italian national The LTDS is a function of the creep phenomena of the geogrids, temperature and time.

$$T_{all} = \frac{LTDS}{\gamma_m} \qquad (1)$$

where the reduction factor γ_m total could be obtained by multiplying several Partials Factors of Safety to account for several possible aging factors (Table 1):

$$\gamma_m = \left(\gamma_{m\,construction} \cdot \gamma_{m\,\mathrm{chemical}} \cdot \gamma_{m\,\mathrm{biological}} \cdot \gamma_{m\,\mathrm{junction}}\right)$$

Table 1. $\gamma_{m\ construction}$ for different types of soil

Soil type	**∅ max. of the particles**	γ_m **construction**
Silt and Clay	< 0.06 mm	1.00
Fine and medium sand	0.06 - 0.6 mm	1.00
Coarse sand and fine gravel	0.6 - 6 mm	1.00
Gravel	6 - 40 mm	1.00
Ballast, sharp stones	< 75 mm	1.03
	< 125 mm	1.07

The biological and chemical Safety Factors for HDPE geogrids are equal to 1.00 for all typical condi-tions found in natural soil.

For the type of geogrids used, the ratio between the Junction strength Rj and the design strength LTDS is always greater than an adequate Safety Factor equal to 1.5.

Furthermore, the tensile creep tests are performed by applying the load through the junctions; the junction Safety Fac-tor fsjunction shall be assumed equal to 1.0. When soil, especially crushed gravel, is spread on geogrids and is compacted, geogrids can suffer damages due to local punctures, indentations, abrasions, cuttings and splitting inferred by the aggregate.

The degree of damage can be assessed by tensile tests performed on both damaged and control (undam-aged) products.

The residual tensile strength after compaction for the geogrids used and different soil types are summarized in the following Table 2.

Considering the type of fill material, the partial factor for damage used was the 1.07.

Table 2. Geogrid properties

Tensile strength (kN/m)	**Creep resistance T_{CS} (kN/m)**	**Elong. at peak (%)**	**Junction strength (kN/m)**	**Allowable strength (kN/m)**
45	18.5	11.5	36	17.24
60	24.6	13.0	50	22.99
90	36.9	13.0	80	34.49

Internal stability analysis was performed using the software Reslope; different cases were studied, corresponding to 4 different typical conditions: base slope (road 12 or 22 m wide on top), intermediate slopes or top slope.

Analyses were performed in both static and seismic conditions.

The same sections were studied also for external stability analysis (sliding, overturning and bearing capacity), assuming the reinforced block to act as a rigid body.

As far as global stability, the complexity of the geometry was suggesting the use of a different software, GStabl 7 allowing to find out, according to Bishops theory, the circular surfaces with the lower factor of safety, given the geometry, surcharges and geometric intervals within which the circular surfaces must enter into the subsoil and exit from the soil.

The road was a line load of 20 kPa over the first berm. The maximum phreatic surface level measured was inserted in the analysis, while the presence of the sheet piles was neglected.

An example of a section studied is shown in Figure 4.

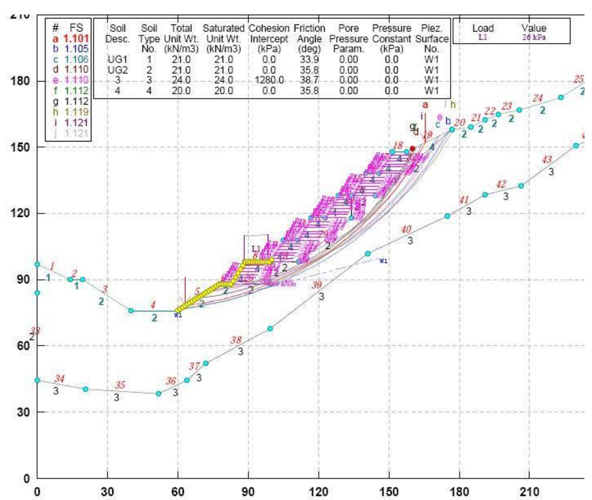

Figure 4 Global stability analysis on the most critical section

4 CONSTRUCTION ASPECTS

The whole slope (over 20.000 m² face) was constructed in less than two years. The use of sacrificial steel mesh formwork was foreseen because of the speed and easiness of installation. Geogrids were delivered on site in rolls clearly identifiable and having a length corresponding to the one that was deriving from calculation, taking into account also face and wrapping length. Spacing was kept constant and equal to 700 mm. The installation procedure was the following

Placement and lining up of the formworks.

Placement of the geogrids in layers perpendicular to the face and along the internal face of the formwork, leaving a portion of grid outside the formworks.

Placement of biomats on the internal side of the geogrids. Taking into account the altitude of the job, the fact that construction was taking place in two years time and that during winter due to the snow the work could be stopped for some weeks or months, a heavy straw mat was used (Figure 5).

Figure 5 Installation: from the left, steel mesh formwork, geogrid, straw mat, and fill material.

Placement of the fill soil in approximately 0.35 m lift and compaction to 95% of Standard Proctor.

Figure 6 The finished job.

Wrapping of the portion of the grid previously kept outside the formworks.

Repeat the phase 1-5 until completion of the work (Figure 6).

The presence of heavy roller and the need to speed up as much as possible the installation in the first month to create the road over the first berm that was used as an access road for the site was leading to an over compaction. The actual thickness of some of the layers was then smaller than the expected 700 mm (some layers were measured to be as low as 600 mm).

This was leading to the need to add some geogrid layers, in order to reach the expected level. Having a narrow spacing was on the safe side in terms of stability; anyway the new layout was checked to provide an analysis "as built" to the owner of the job. After the first tier, the problem was not observed anymore, as taller steel mesh formwork were supplied (and then the control of the thickness was easier) and the compaction close to the face was operated with light weight machines.

The presence of a coarse gravel was sometimes causing rather big boulder (up to 200 mm) to be close to the face. Whenever it has been possible, coarse gravel or boulders were removed, leaving only topsoil in the first 300 mm form the slope face to allow an easier vegetation.

5 FIRE EVENT ON THE REINFORCED STRUCTURE

One year after the end of constructon, the reinforced structure was attacked by fire (Fig. 7); a prompt inspection of the site was confirming the job was not suffering any serious damage (Cazzuffi et al., 2014).

It is important to remember that the structural reinforcing elements (geogrids laid horizontally within the soil) are never affected by the flames, as oxygen is not present. A thorough inspection to assess the integrity of the works has been done, considering the banks from the top to the bottom.

Figure 7 The reinforced structure after the fire event

Samples of geogrid excavated from an area that was interested by the fire were subjected to mechanical tests, and the results obtained were demonstrating that the fire itself was not directly affecting the resistance of the geogrid. In conclusion, such a structure, once vegetated, has then demonstrated to be resistant even to the fire.

The sample of geogrid exhumed was delivered to an accredited laboratory for geosynthetics, where it was subjected to a wide width tensile test according to the EN ISO 10319, as described by Cazzuffi (1996). The test results have provided values of residual strength equal to 85% of the nominal value of the geogrid. These values were due to the damage suffered during installation but especially during the exhumation, and are in no way attributable to the fire, since the geogrid in the portion removed did not show any trace of fusion, burn or change to its geometry due to heat.

6 CONCLUSIONS

The current work is a very interesting application of the use of soil reinforcement in extreme conditions, being one of the tallest structures ever designed and constructed.

The definition of the logistic aspects related to the construction of such a big job, that was lasting over two years, were quite challenging, as it was necessary to provide all the necessary measurements to avoid that during the winter stop (when the whole job was covered by the snow) the part of structure that was not finished was not suffering major damages and that with the spring season the work could start without problems.

Six months after the end of the works the structure was attacked by fire; a prompt inspection of the site confirmed that the structure had not suffered any serious damage. Samples of geogrid excavated form an area which had been affected by the fire were subjected to a tensile strength test; the results demonstrated that the fire had not directly affected the resistance of the geogrid. Such a reinforced structure has demonstrated to be resistant even to fire.

7 REFERENCES

Cazzuffi, D. 1996 Evolution of European standardization on geotextiles, with special reference to mechanical tests. *Proceedings "Index 96" Nonwoven Congress - Construction session*, Geneva, pp. 1-15.

Cazzuffi, D., Griffini, L., Griffini, S., Paoletti, A., Peduzzi, G.B., Recalcati, P. 2014 Reconstruction of the left hand side of the Adda river after the Valpola landslide with a geogrid reinforced steep slope 60.0 high. *Proceedings of XXV Italian Geotechnical Conference*, Baveno, Vol. 2, pp. 431-439 (in Italian).

NTC08 Italian Ministry of Infrastructures. 2008. Norme Tecniche per le Costruzioni (Techncical Guidelines for Construction), Decreto Ministeriale 14.01.2008 (in Italian).

Paoletti, A., Peduzzi, G., Griffini, L., Recalcati, P., Cantoni, A., 2013 A 60.0 m high geogrid reinforced slope in the Italian Alps. *Proc. Int. Conference on Design and Practice of Geosynthetic-Reinforced Soil Structures*, Bologna, pp. 548-560.

Paoletti, A., Griffini, L. 2007. L'Adda di nuovo a cielo aperto: la sistemazione idraulica e il recupero ambientale dell'area della frana di Valpola. *Atti Convegno "Valtellina 2007"* – Morbegno. (in Italian)