Slope Stability Analysis Using 2 & 3-Dimensional Methods of Basal Reinforced Slopes

Carlos Rodriguez[1](✉), July Jaramillo[2], Lizeth Ardila[3], Sina Javankhoshdel[4], and Stephen Hughes[5]

[1] Huesker Incorporated, Charlotte, NC 28120, USA
crodriguez@huesker.com
[2] Huesker Fabrikstraße 13-15, 69121 Gecher, Germany
[3] Huesker Ltda, Sao José Dos Campos, SP 12238, Brazil
[4] Rocscience, Toronto, ON, Canada
[5] Construction Manager, Blacksburg, VA, USA

Abstract. Multiple slopes varying from 25° to 30° were designed and constructed in different areas of a highway project in Northern Virginia. High modulus and high-strength geosynthetics were required to ensure the sliding stability of 11 m slopes up to. For some slopes, polyester geosynthetics with up to 1000 kN/m ultimate strength at less than 10% were used as basal reinforcement instead of ground improvement up to 7 m depth located downstream. The slope stability analyses using a basal reinforcement were carried out in 2D and 3D using Slide2 and Slide3, respectively. The problem formulation, considerations, and final results obtained are presented.

Keywords: Basal reinforcement · Mechanically Earth Slope · High Strength Geosynthetics · Soft Soils

1 Introduction

The extension of a motorway to reduce congestion and accommodate travel demand more efficiently passes through a mountainous countryside in Northern Virginia. The design of a new separate carriage required the construction of very high and steep embankments along 16 km. The project was divided into different areas (4 in total), with varying slope geometries and configurations. Different sections can be identified according to the available and required space for the new lane of the road, as indicated in Fig. 1 and described below.

This work is focused on the stability analysis of a 11 m (40 ft) high and 60 m (200 ft) wide embankment located in an area with several critical boundary conditions and high instability potential. The standard slopes with 2H:1V were designed to provide a minimum factor of safety of 1.4 against failure.

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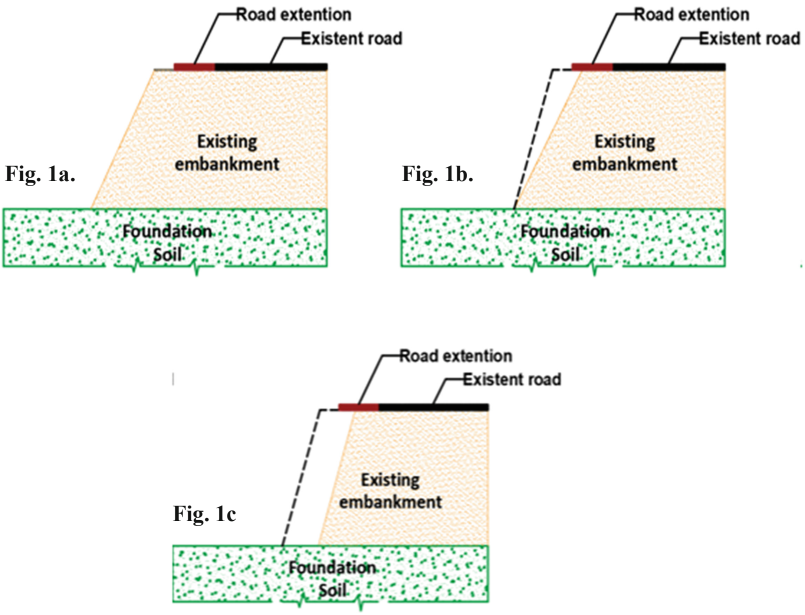


Fig. 1a. **Fig. 1b.** **Fig. 1c**

Fig. 1. Existing slope geometries and configurations in the site.

As it was not possible to achieve an adequate safety factor in some critical areas by using the proposed slope inclination, measures to increase the safety against the several failure modes were proposed (see Fig. 2).

The geotechnical conditions, the calculation of the proposed solutions by means of 2D and 3D models, and its results will be summarized in this paper.

The analysis performed on the reinforced slope suggested a rotational deep-seated general shear failure due to the lack of bearing capacity and the load imposed by the new embankment. The results from the 2D model suggest that the slope can be stabilized using only one layer of high-strength, high-modulus geotextile installed over the existing subsoil on the base of the embankment. Besides confirming the results from the 2D model, the results from the 3D model show the most optimum geotextile response.

2 Basal Reinforcement Geosynthetics General Considerations

2.1 Importance of Row Material, Displacement, and Strains

Row Material: a high-strength woven geotextile (HSWG) is recommended as reinforcement due to the high coefficient of interaction to the existing soil layer to be reinforced and prevent from sliding. Different than geogrids, HSWG provides separation of the soft soil from the embankment fill material.

Reinforced Strength and Length: limit equilibrium analyses (LEA) consider the driving and retaining forces along the entire slope, resulting in one single scalar equation. Weight and seepage as driving forces, the retaining one by friction and adhesion (Plankel

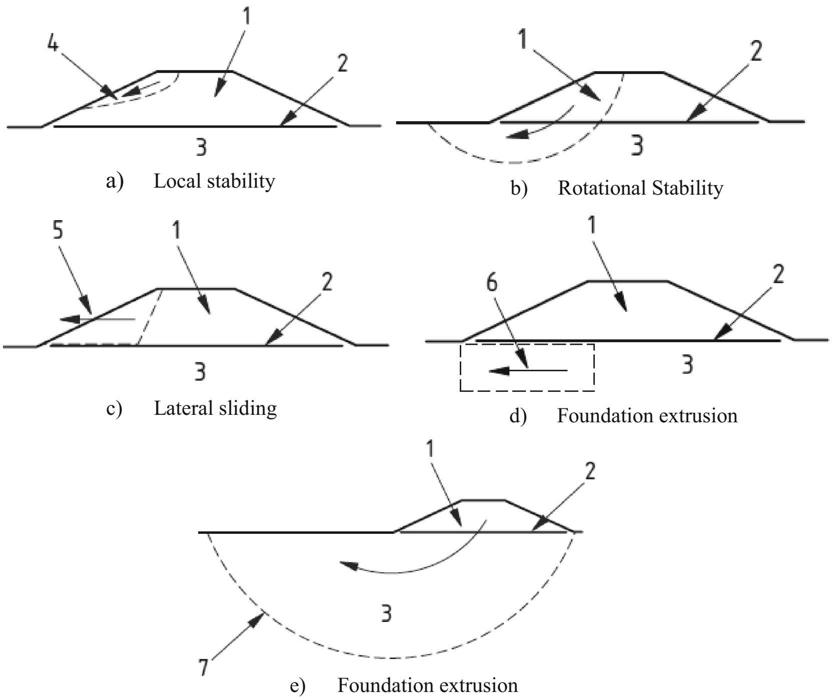


Fig. 2. Possible failure modes of an embankment (British Standard 2010).

and Alexiew, 1998). A diagram of sliding and resisting forces involved in the stability of a geosynthetic reinforced embankment is shown in Fig. 3.

The geotextile allowable design strength T_{min} must be larger than the maximum strength demand calculated based on the limit equilibrium analysis, and it must mobilize

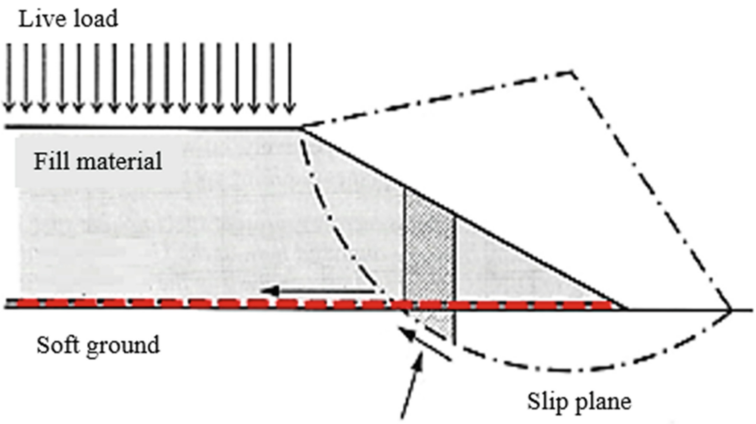


Fig. 3. Global stability analysis with basal reinforcement. Modified from (DGGT 2011)

the slip surface up to a safe position (FHWA 2015). To determine the geotextile embedded length L_{min}, earth pressures are assumed to be linearly distributed using Rankine active earth pressure and at-rest conditions for the soil backfill and the surcharge, respectively (Koerner 2016). The minimum geotextile embedded length must mobilize the critical slip surface up to a safe position.

Separation: Embankment basal reinforcement also requires geosynthetics to prevent separating the overlying granular fill from the underlying fine-grained existing soil. Woven geotextiles significatively reduce drainage and prevent the intermixing of aggregates into a finer underlying soil, as well as restrain fines pumping to the ballast, reducing the drainage and shear strength at the base of the ballast (Whittle and Ling 2002).

3 Case Study

3.1 Description of the Geotechnical Conditions

Generally, the study area is immediately underlain by a combination of artificial fill materials associated with past development activities and natural soils. In most cases, a stabilization method should be proposed for areas where unsuitable subsoils are present. Excavations and fillings are required for foundation construction, retaining walls, utility trenches, stormwater management ponds, and embankment cuts and fill.

The site geology consists of an unconsolidated river and marine sediments. Complex interbedded fine- and coarse-grained sediments are present due to irregular deltaic and alluvial deposition, and strata unconformities are common due to periods of erosion and regional faulting. As a result, strata composition and thicknesses can vary significantly over short horizontal or vertical distances.

A subsurface exploration program consisting of test borings was conducted in 2017; a total of thirty-eight test borings were performed in the studying area which covers approximately 4 km in length. The geotechnical design recommendations were based on 175 test borings, whereof 38 borings were performed in 2017 and 137 right before the project started. The deepest boring was 21 m (70 ft), and the depth average is about 9.0 m (30 ft). Laboratory classification testing was performed on selected soil samples recovered from the subsurface exploration. Additionally, select samples were tested for consolidation and strength properties using CU Triaxial, direct shear, and unconfined compression test methods. In addition, analytical testing consisting of pH, resistivity, and sulfates was also performed. The existing soil profile can be easily simplified as two layers of soft soil.

3.2 Description of the Proposed Solutions for the Critical Zones

As shown in Fig. 1b and 1c, different types of earthworks were required to widen the crest of the existing road embankment. Three different alternative solutions or a combination of them were proposed to achieve the appropriate factor of safety in the critical zones, where a factor of safety of 1.4 against the evaluated failure mode could not be achieved. These solutions are shown in Fig. 4 and described below.

Solution a) Soil replacement at the toe of the embankment:

This solution consisted of the replacement of a saturated normally consolidated clayey soil (c' = 0 kPa and ϕ' = 24°) with a granular material (c' = 0 kPa and ϕ' = 35°) to form a granular foundation body. The applicability of this solution was limited to the cases, where an extension of the embankment footprint was required (see Fig. 4a).

The so-created granular body acts as a friction toe and can increase the safety factor against several failure modes such as lateral sliding, foundation extrusion and some rotational slip surfaces (see Fig. 4a).

In general, can be said, that this type of operation requires excavation in poor soils and at great depths, which may require further measures to stabilize the excavation, as well as compaction works. This will achieve the density and therefore quality of the replacement soil. In cases where the replacement material is not available in the area, significant transport costs may also be incurred.

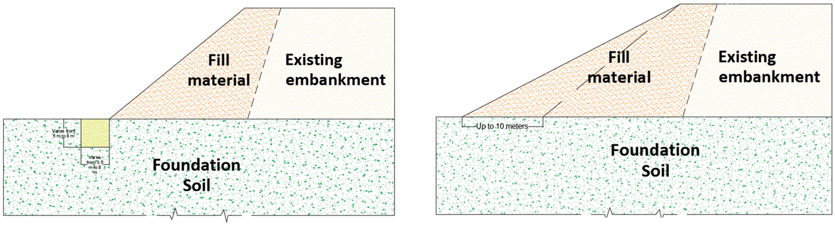
Solution b) Reduction of the Slope Angle (flatten):

This alternative solution consists of flattening the standard slope (2H:1V or 26.6°) to less than 20°. As with the previous solution, this measure could be used in cases where an extension of the embankment footprint is required (see Fig. 1c).

Reducing the slope can increase the safety factor against all the failure modes shown in Fig. 2, but requires more filling and compaction work, so it is only appropriate in cases where embankment material of suitable quality is locally available. Another limitation is that it can only be used where the site boundaries of the infrastructure project allow the embankment to be extended.

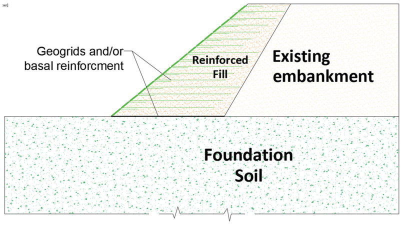
Solution c) Use of Geosynthetic Reinforcement:

The geosynthetic reinforcement could be used both in cases where the extension should be made by maintaining the former footprint of the existing embankment (Fig. 4a) and in cases where the footprint should be extended (Fig. 4b).



a. Slope stability measurement with soil replacement

b. Slope stability measurement flatten-ing of the slope



c. Slope stability measurement with geosynthetic reinforcement

Fig. 4. Schematic proposed solutions for the Critical Zones.

In the first case, where the existing slope of the embankment was steepened and the footprint was maintained, the geosynthetic reinforcements chosen were geogrids placed in layers with a maximum vertical spacing of 60 cm. It is recommended that geogrids be used as reinforcement when placed within the fill material, as they provide optimum interaction with the surrounding soil. The main function of the reinforced slope constructed in this way is to increase the safety factor against failure due to local stability (see Fig. 2a) and rotational slip surfaces (see Fig. 2b).

In the second case, where the footprint of the existing embankment was widened, basal reinforcement could also be applied. Basal reinforcement can increase the safety factor against failure modes that involve the plane of the reinforcement, such as rotational slip surfaces and lateral sliding (see Figs. 2b and 2c). The chosen geosynthetic reinforcement in this type of application is a woven geotextile, which is able to perform the reinforcement function and separate the fill from the underlying saturated soft soil.

3.3 2D Model

The design methods of Bishop Simplified (Abramson et al. 2001) and Spencer (Spencer, 1967) were combined with global search methods to find critical slip surfaces in the slopes. Traditional limit equilibrium (LE) slope stability analysis methods seek to locate the slip surface with the lowest factor of safety (F.S) or the most critical failure surface. In the models below the extent of the geometry, horizontal limits are chosen so that they do not affect the failure surfaces.

The first slope stability analysis was performed on a 2H:1V slope. As can be observed in Fig. 5 this geometry was restricted due to an existing canal structure on the slope toe. Global external overall stability was analyzed to the 2H:1V slope, resulting in an insufficient factor of safety, F.S = 1.1. The figure shows a set of shallow failure surfaces that covers a sliding mass of approximately 326 tons. The critical failure starts 1 m (3.28 ft) from the crest of the slope edge [A], and it corresponds to a compound failure (Ausilio et al., 2001) that curves at both ends and has a level or flat central point. The failures finish at the canal bottom 1.5 m underneath and 3 m to the left of the slope toe [B].

The second analysis compromises an original mitigation system defined by the project owner. In this case, the slope stability analysis was performed on a flattened slope, from 2H:1V to 2.8H:1V, forcing to remove and replace the existing canal structure. The slope without mitigation shown in Fig. 5 has a face inclination of 27 degrees. The solution in Fig. 6 has to be flattened to 18 degrees and includes a fully softened CM/MH as a partial fill at the core of the embankment. Figure 6 shows the result of Spencer LE method F.S = 1.46, a compound failure as described in Fig. 5, occurs in this new configuration. In both cases, the flat level in the compound failure arises at 5.0 m depth, on top of a CH/MH soil layer. In this new scenario, the failure surface starts at the same point as in Fig. 5, however, the exit point was mobilized 14.5 m to the left [C].

As was mentioned earlier, the calculation procedures to determine the reinforcement required strength and the minimum embedded length are based on limit equilibrium analysis. Assuming that it is possible to assess a basal reinforcement partially buried in the fully softened CM/MH soil zone, a third analysis was performed on a 2H:1V slope. Figure 7 shows the results encountered for undrained and drained conditions, respectively. The slope in this section is 198 m long. Since the basal reinforcement

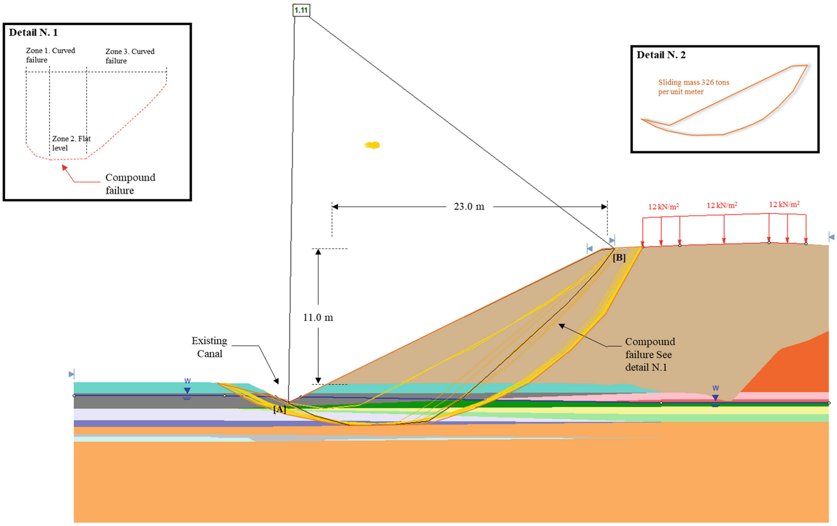


Fig. 5. Analysis of global external stability to no mitigated slope – Spencer method – Drained condition

solution does not consider flattening the slope, the red shadowed area in Fig. 7, 12000 cubic meters of selected fill material are saved, as well as the slope weight is not increased by 47520 tons.

The analysis shows that a woven geotextile comprised of high-tenacity polyester yarns provides the required strength to prevent failure. The geotextile was installed on top of the existing soil and extended from the slope toe up to 14 m. The ultimate tensile strength was determined to be 400 kN/m in the main machine direction, with a maximum elongation at rupture of 9%. The ultimate strength in the perpendicular direction was 50 kN/m and overlapped 0.50 m. The woven geotextile material separates the overlying granular fill from the underlying used as well as fine-grained foundation soil.

Figures 6 and 7 show a base failure, the slip or failure plane goes all the way along through a soft soil layer resting on a stiff layer of soil. The use of a high-strength geotextile yields a different result, as the slip plane does not penetrate the foundation.

The new configuration shows a F.S of 1.40 and a local failure whereby the failure surface passes through the slope.

3.4 3D Model

In this part of the study, the 2D model with the basal reinforcement was imported into Slide3, which is Rocscience's 3D LEM software to carry out the 3D LEM analysis for the exact arrangement of the material and basal geosynthetic reinforcement and extruded with the depth of 27.4 m (90 ft). Figure 8 shows the 3D model. For the 3D analysis, the Spline search method with the Particle Swarm search mechanism and Surface altering optimization technique is used in the Slide3 software (Rocscience 2023).

Similar sensitivity analyses as 2D are carried out to find the minimum length of the basal reinforcement to change the failure mechanism to a slope failure. The sensitivity

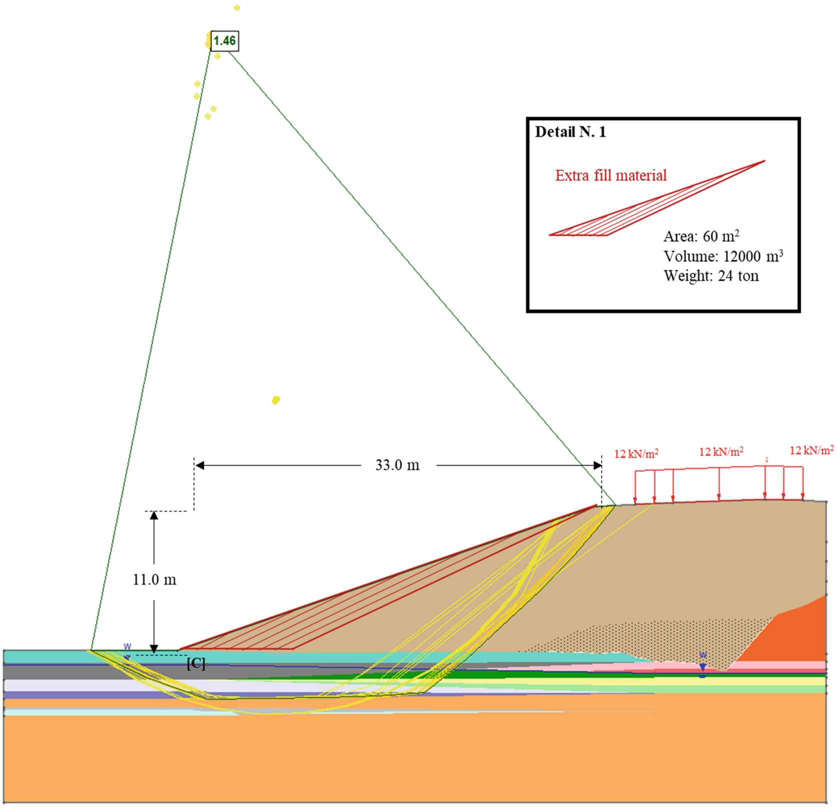


Fig. 6. Analysis of global external stability to mitigated slope – Spencer method - Drained condition.

analyses showed that for the 3D model the required basal reinforcement length is 12 m (39 ft). This value is shorter than the one found in the 2D analysis. The reason for that is, as it is known, 3D analysis gives a higher factor of safety compared to the 2D analysis and are less conservative. That is why it is expected to require shorter reinforcement lengths (less conservative) compared to the 2D analysis to provide the slope failure. However, 3D analysis is more accurate since it is closer to the real-size model and will help to design in a more economical way (Fig. 9).

4 Conclusions

Basal reinforcement can increase the safety factor against failure modes that involve the plane of the reinforcement. The configuration composed of one layer of high strength and high modulus geotextile *Basal Reinforcement* was a suitable solution to achieve the required F.S in this project. The solution is cost-effective if it is compared with other alternatives. The model analyzed in this paper also shows a failure mode transition from a compound failure to a local failure.

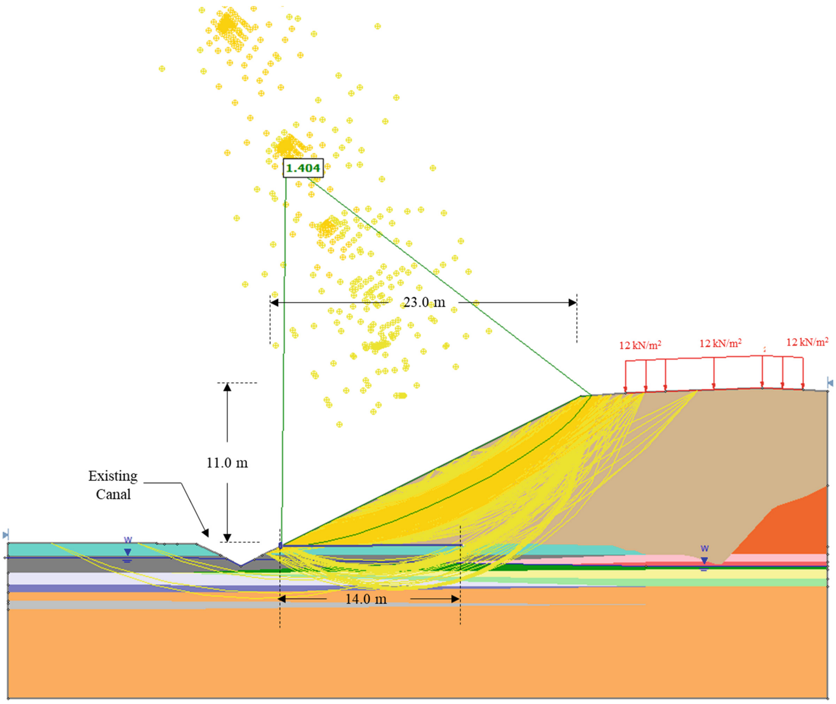


Fig. 7. Analysis of global external stability to geosynthetic mitigated slope – Spenser method - Drained condition.

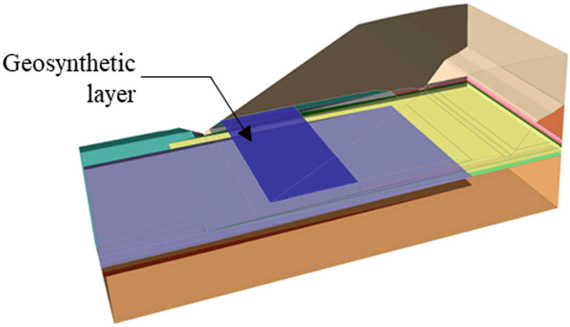


Fig. 8. 3D model used in this study (the 27 m extruded version of the 2D model).

The 3D LEM model is capable of taking into account different cross-sections of the entire slope. Among others, 3D analysis allows for the evaluate of the critical cross-section and its F.S. The results obtained in this paper show congruency with (McQuillan et al.) regarding the factor of safety. Using the same geosynthetic embedded length and type of reinforcement, a higher F.S was found in to evaluate the slope with a 3D mode.

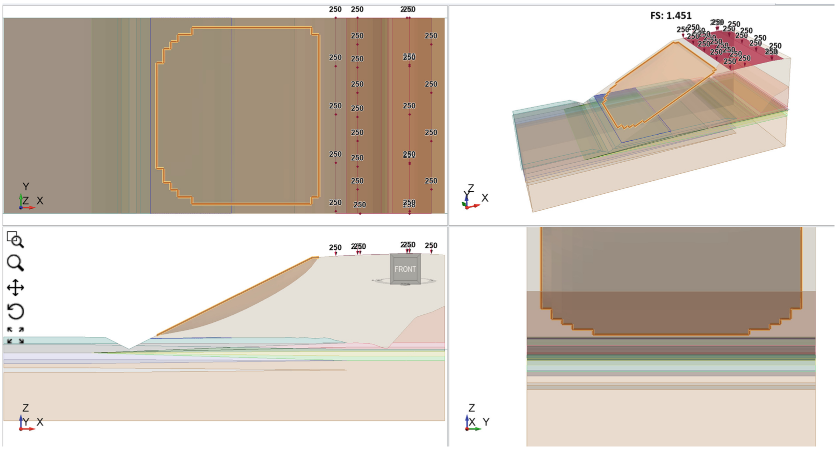


Fig. 9. Results of the 3D LEM analysis in different views.

References

Abramson, L.W., Lee, T.S., Sharma, S., & Boyce, G.M. (2001). Slope Stability and Stabilization Methods, 2nd edition, John Wiley & Sons.

Ausilio, E., Conte, E., & Dente, G. (2001). Stability analysis of slopes reinforced with piles. Computers and Geotechnics, 591–611.

British Standard. (2010). Code of practice for strengthened/reinforced soils and other fills. ISBN.

DGGT. (2011). Recommendations for Design and Analysis of Earth Structures using Geosynthetics Reinforcements - EBGEO. Ernst & Sohn GmbH & Co. KG.

FHWA. (2015). Synthesis of Geosynthetic Reinforced Soil (GRS) Design Topics. Federal Highway Administration Research and Technology, Washington.

Koerner, R. (2016). Designing with Geosynthetics, 6th edition. Xlibris.

Plankel, A., & Alexiew, D. (1998). Hig-Stremgth Aramid Geogrids to Prevent Sliding on Steep Landfill Basal Slopes. Sixth International Conference on Geosynthetics, 475–480.

Spencer, E. (1967). A method of analysis of the stability of embankments assuming parallel inter-slice forces. Geotechnique, 17(1), 11–26.

Whittle, A., & Ling, H. (2002). Geosynthetics in Construction. ScienceDirect - Encyclopedia of Materials: Science and Technology.