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# Mitigation of Savadkouh Landslide Using Non-woven Geotextiles

A.J. Choobbasti, A. Barari, F. Farrokhzal M. Safaei

Babol University of Technology, Department of civil Engineering, P.O. Box 484, Babol, Iran.

**Abstract:** Hazardous landslides are ordinary phenomena in north of Iran. In this paper, mitigation of a landslide in Savadkouh by using non-woven geosynthetic was investigated. Reinforced slope was designed using limit equilibrium analysis and numerical method such as those based on the FHWA guidelines and finite element method and with in-situ residual soil. The main goal of this paper in designing a reinforced slope instead of failed slope is finding number of reinforcement layers and vertical distance of reinforcement layers. Using the non-woven fabric to repair failed slope is a good option to prevent respective failures. When the non-woven fabric is used, it not only generates the tension forces to enhance the overall stability of slopes, but also prevents the development of pore water trapped in clayey soils by providing horizontal drainage through the fabric. Results of analysis showed that positive effects of reinforcement are clear and lead to decrease of horizontal displacements.

**Key words:** landslide; mitigation; geosynthetic; FHWA; finite element method

## INTRODUCTION

Landslides are frequently responsible for considerable losses of both money and lives, and the severity of the landslide problem worsens with increased urban development and change in land use. Given this understanding it is not surprising that landslides are rapidly becoming the focus of major scientific research, engineering study and practices, and land-use policy throughout the world.

In every slope there are forces which tend to promote down slope movement and opposing forces which tend to resist movement. A general definition of the factor of safety, *F*, of a slope results from comparing the down slope shear stress with the shear strength of the soil, along an assumed or known rupture surface. Starting from this general definition, (Terzaghi, K. 1950), divided landslide causes into external causes which result in an increase of the shearing stress (e.g. geometrical changes, unloading the slope toe, loading the slope crest, shocks and vibrations, drawdown, changes in water regime) and internal causes which result in a decrease of the shearing resistance (e.g. progressive failure, weathering, seepage erosion). (However, varnes, D.J. 1978), pointed out there are a number of external or internal causes which may be operating either to reduce the shearing resistance or to increase the shearing stress. There are also causes affecting simultaneously both terms of the factor of safety ratio. Fig. (1) Shows an example of factor of safety variation as a function of time, for a given slope and Table 1 and 2 shows brief lists of landslide casual factors and remedial measures.

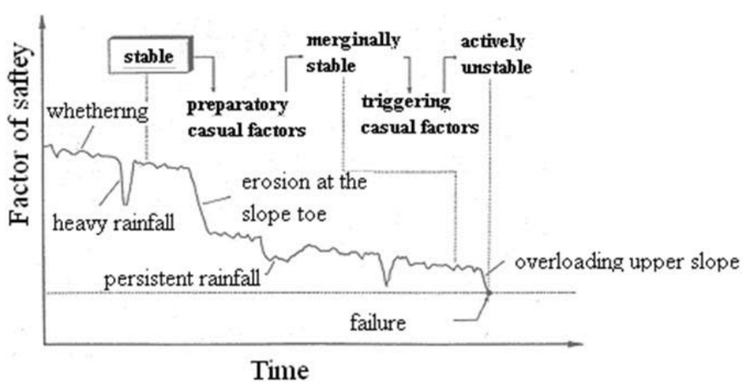


**Fig. 1:** An example of changes in the factor of safety with time.

**Corresponding Author:** A.J.Choobbasti, A. Barari, M.Safaei, Babol University of Technology, Department of civil Engineering, P.O. Box 484, Babol, Iran.

**Table 1:** A brief list of landslide casual factors.

| |
|---|
| 1. Ground Conditions |
| (1) Plastic weak material |
| (2) sensitive material |
| (3) Collapsible material |
| (4) weathered material |
| (5) Sheared material |
| (6) Jointed of fissured material |
| (7) Contrast in permeability and its effects on ground water contrast in stiffness (stiff, dense material over plastic material) |
| 2. Geomorphological Processes |
| (1) Tectonic uplift |
| (2) Volcanic uplift |
| (3) Glacial rebound |
| (4) Fluvial erosion of the slope toe |
| (5) Wave erosion of the slope toe |
| (6) Glacial erosion of the slope toe |
| (7) Erosion of the lateral margins |
| (8) Subterranean erosion (solution, piping) |
| (9) Deposition loading of the slope or its crest |
| (10) Vegetation removal (by erosion, forest fire, drought) |
| 3. Physical Processes |
| (1) Intense, short period rainfall |
| (2) Rapid melt of deep snow |
| (3) Prolonged high precipitation |
| (4) Rapid drawdown following floods, high tides or breaching of natural dams |
| (5) Earthquake |
| (6) Volcanic eruption |
| (7) Breaching of crater lakes |
| (8) Freeze and thaw weathering |
| (9) Shrink and swell weathering of expansive soils |
| 4. Man-Made Processes |
| (1)Excavation of the slope or its toe |
| (2)Loading of the slope or its crest |
| (3)Defective maintenance of drainage systems |
| (4)Irrigation |
| (5)Water leakage from services (water supplies, sewers, storm water drains) |
| (6)Vegetation removal |
| (7)Mining and quarrying (open pits or underground galleries) |
| (8)Creation of dumps of very loose waste |
| (9)Artificial vibration (including traffic, pile driving, heavy machinery) |

The methods used to design reinforced slopes are mainly based on the limit equilibrium concept. Methods such as (Reugger, 1986. Schmertmann, *et al.,* 1987. Leshchinsky, 1989. Jewell, 1980. 1991 and Michalowski, 1997), all utilize limit equilibrium analysis or limit analysis in the design of reinforced slopes. Table 3 lists general information related to the mentioned methods.

Extensive experimental studies have been devoted to the evaluation of effect of reinforcement on stability of soil slopes. One of the most important studies in this field were conducted by (Zornberg *et al.,* 1998. and 2003), who observed the behavior of reinforced slopes in the centrifuge. The effect of reinforcement on stability of soil slopes, and the associated failure mechanisms were assessed in their study. Zornberg *et al.,* showed that if a prototype of actual dimensions is modeled with a reduced scale of $1/N$ and subjected to an acceleration field N times that of gravitational acceleration, a stress field similar to the prototype structure would be reproduced within the reinforced slope model in the centrifuge. Other parameters such as density and internal angle of friction are unchanged while tensile strength of geotextile layers in the model is reduced by a factor of N. Some important findings of (Zornberg *et al.,* 1998. and 2003), from centrifuge tests performed on reinforced soil slopes may be summarized as follows:

1. Failure in the reinforced slope model was observed to pass through the slope toe, which is in good agreement with the assumption of limit equilibrium methods.
2. Failure initiated from mid height of the reinforced model which contradicts the assumption made in limit equilibrium method that failure develops through the toe of the slope.
3. Location of maximum reinforcement load and the associated maximum strain along the potential failure surface depend on slope angle and overburden pressure.
4. Stability of the reinforced slope is governed by peak strength of the soil.

**Table 2:** A brief list of landslide remedial measures.

| |
|---|
| 1.Modification of slope geometry |
| (1) Removing material from the area driving the landslide |
| (2) Adding material to the area maintaining stability |
| (3) Reducing general slope angle |
| 2. Drainage |
| (1) Surface drains to divert water from flowing onto the slide area (collecting ditches and pipes) |
| (2) Shallow or deep trench drains filled with free-draining geomaterials (coarse granular fills and geosynthetics) |
| (3) Buttress counter forts of coarse-grained materials |
| (4) Vertical(small diameter) boreholes with pumping or self draining |
| (5) Vertical (large diameter) wells with gravity draining |
| (6) Drainage tunnels, galleries |
| (7) Vacuum dewatering |
| (8) Drainage by siphoning |
| (9) Electro- osmotic dewatering |
| (10) Vegetation planting (hydrological effects) |
| 3. Retaining structures |
| (1) Gravity retaining walls |
| (2) Gabion walls |
| (3)Passive piles, piers and caissons |
| (4) Cast-in situ reinforced concrete walls |
| (5) Reinforced earth retaining structures with, strip, polymer, metallic reinforcement |
| (6) Rock fall attenuation or stopping systems |
| 4. Internal slope reinforcement |
| (1) Rock bolts |
| (2) Micro piles |
| (3) Geosynthetics |
| (4) Anchors |
| (5) Stone or lime/cement columns |
| (6) Heat treatment |
| (7) Grouting |
| (8) Freezing |
| (9) Electroosmosic anchors |
| (10) Vegetation planting |

**Table 3:** Specifications of limit equilibrium methods for design of reinforced slopes

| Michaloski 1997 | Jewell 1991 | Reugger 1989 | Leshchinsky 1989 | Schmertman, 1987 | Method |
|---|---|---|---|---|---|
| Kinematics limit analysis | Two-part wedge | slices | Internal stability: Variational Limit equilibrium | Two-part wedge | Model |
| (c = 0, φ ) | (c = 0,φ ) | (c = 0,φ ) | (c = 0,φ ) | (c = 0,φ ) | Soil |
| 0,0.25,0.5 | 0,0.25,0.5 | 0 | 0 | 0 | ($r_0$) |
| 30°- 90° | 30°- 90° | 40°- 90° | 45°- 90° | 30°- 80° | Slope Angle |
| Parallel to slope face to slope face | Not parallel to slope face | Parallel to slope face | General case | Not parallel to slope face | Reinforcement Arrangement |
| 20°-50° | 20°-50° | 40°-90° | 15°-40° | 15°-35° | φ |
| 0.5-0.8 | 0.8 | 0.6-1 | 0.6-1 | 0.9 | μ |
| Triangular or Rectangular | Triangular | Rectangular | Triangular | Triangular | Distribution of reinforcemen force with height |

### *Site characterization for Flourd landslide:*

### *Site geography:*

The site is located at Savadkouh Azad University grounds in the rural surroundings of Savadkouh, 5 Km from Pol-Sefid city in the north of Iran. The elevation of the site is 350 *m* above sea level and like the surrounding lands, a forest vegetation cover and a mountainous morphology is dominant. Fig. (2) Shows a scarp of the landslide along with the tilted trees at indicating the extent of the slide.

The area investigated consists of an old slide mass which had several slides at the quaternary age, associated with saturated conditions and dynamic loading caused by severe earthquakes. Peat, lignite remains and coal remains from alder trees which were revealed from boreholes at the site indicate that the site had several previous slides and that generally similar geographical conditions existed in the past.

**Fig. 2:** Scarp of Flourd landslide with tilted trees

***Geo-hydrology and underground water conditions:***

Underground site investigations indicate that no stable underground water table exists. However, underground seepage flow seems to exist within the direction of silt and gravel lenses which exist in the stratification. Bore pits dug in the area show that seepage water exists at the depth of three to five meters below ground level. Water level was observed in several observation wells at depths ranging from 10 to 15 meters, at the interface between the slide mass and the underlying bedrock.

A part of the run-off flow outside the active slide area leaks into the slide area which is in addition to the rainfall induced seepage from surface water absorption. The slide mass is therefore considered to be saturated at the time of flow.

***Geological characterization:***

One of the important factors affecting the instability potential of a landslide hazard area is the engineering geological parameters and characteristics of that site. In this context, the parameters considered usually consist of site geometry, failure mechanisms observed, effect of slide on existing structures, assessment of the causes and the risks for future occurrence of slides, and classification of the landslide and the existing soil characteristics. All these have been studied in Flourd landslide and have revealed the high landslide hazard of the site.

***Site geometry:***

As previously mentioned, the slide mass at the university site is part of a sliding slope facing the south eastern direction. The sliding mass extends to the calcareous sediments on the south west side, and to the river at the foot. Steady state seepage passages through the medium and above the level of the rock foundation. A volume of 15 to 20 liters per second were predicted for these seepage flows. The slide mass does not have any visible surface flow paths and the existing paths are rather scattered in the whole region (Fig. (3)).

**Fig. 3:** Access road to the site and the high seepage flow of 10-20 liters per second observed.

From the lateral scarps, silty clay along with boulders is visible. The slide initiated at the point where little vegetative growth existed. Surface water from rainfalls directly penetrated the underlying soils at theses surfaces and was lead through the shear zone of the sliding mass. Springs visible at the foot of the sliding mass is an indication of this process.

The upper surface of the slide is has a concave form which gathers rainfall water into the sliding mass. Therefore, each period of heavy rainfall caused a reactivation of the slides. Tilting of the existing trees indicates that a creep type of active slide is dominant in the area (Fig. (4)).

**Fig. 4:** A view of the active slide at the site – tilting trees and visible roots indicate an active fault in the area.

The effect of landslide on existing geotechnical structures in the area was generally in the form of slides in the slopes, creep, tilt and structural cracks in the geotechnical structures. These defects were mainly attributed to poor engineering characteristics of the structures, both in design and construction.

***Modeling Flourd Landslide:***

In order to better understand and assess Flourd landslide and the potential applicability of using geotextile reinforced in-situ cohesive soil for mitigation of this landslide, initially, limit equilibrium method was utilized in this study. The method suggested by FHWA was followed in the analyses. Limit equilibrium analysis code Reinforced Soil Slopes (RSS) was effectively utilized to run limit equilibrium analyses in order to evaluate the effect of reinforcement on the factor of safety against landslide. RSS calculates the factor of safety for an existing non-reinforced slope and is able to calculate the required amount of reinforcement in order to reach a desired factor of safety associated with equilibrium or stable conditions. RSS is able to design a reinforced slope by one of the three following procedures:

1. Calculating reinforcement spacing for a given slope configuration in order to reach a desired factor of safety.
2. Determination of the required reinforcement strength in order to reach a desired factor of safety.
3. Calculation of the factor of safety of a reinforced slope with a given configuration of reinforcements.

It is noteworthy that RSS follows all the design procedures suggested by FHWA.

In order to mitigate Flourd landslide using in-situ cohesive soils reinforced with horizontal geotextile layers, an initial reinforcement configuration was first designed, and through limit equilibrium analysis using RSS, the factor of safety of the reinforced slope was determined. Next, the resulting factor of safety was checked by design suggestions from the literature in order to verify the adequacy of the design (Cornforth, D.H.2004). After verifying the given initial reinforcement configuration, the design procedure suggested by FHWA was followed in order to obtain the necessary reinforcement spacing and strength in RSS code.

The final design was obtained by following the above mentioned design procedure for three different slopes of one, 1.5 and two horizontal to one vertical. The reinforced slope area is actually the slope which is responsible for mitigation of the sliding backfill. Section L6 (Fig. (5)) was selected from the topography of the landslide area for analysis. This section is shown in the topography in Fig. (5). Each analysis involves careful determination of the initial unstable slope, and the evaluation of the necessary reinforcement configuration for obtaining stable conditions. The initial design configuration involves a reinforcement vertical spacing of 0.5 meters and a reinforcement length of 15 meters. Reinforcement length is automatically checked for necessary factor of safety against pullout failure, by the program. The results of the initial analysis showed that the slope geometry with a face slope of two horizontal to one vertical was stable under natural conditions and no reinforcement design was required for this design. Therefore, only the slope angles of one and 1.5 horizontal to one vertical were considered in the consecutive analyses. A reinforced soil slope factor of safety of 1.4 was chosen for design following the suggestions of (Cornforth, D.H.2004),. for a medium size landslide with limited site characterization information. The results of the analyses are described in the following section.

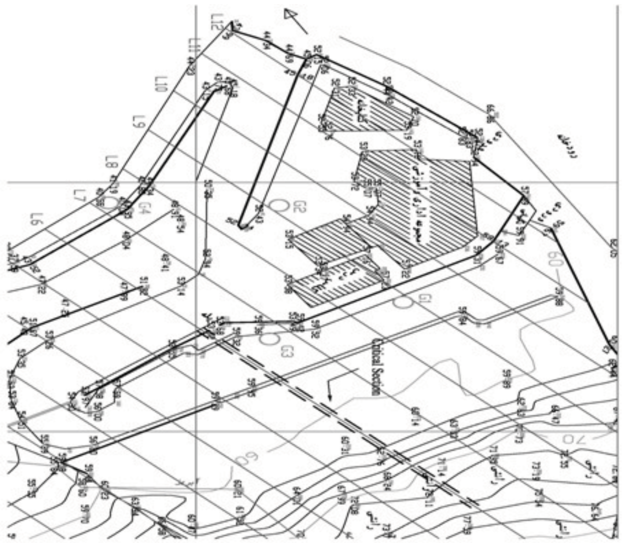

**Fig. 5:** Topography of the landslide area

## RESULTS AND DISCUSSION

Prior to numerical analysis of a landslide, it is necessary to select a critical section of the landslide site in order to perform plane strain calculations on. In this context, Flourd landslide was observed by the topography maps available, and a critical section as depicted in Fig. 5 by line L6 was chosen for analysis. This section is believed to be representative of the behavior of the landslide since a relatively wide area is covered by the slide and therefore the assumption of plane strain conditions is not irrelevant for this section. Moreover, the majority of site investigation was conducted in the vicinity of this section and therefore, more thorough information is available in this vicinity.

Limit equilibrium analysis was first performed on the section chosen, in order to observe the possible failure mechanisms of the slide, as well as to suggest a design for the reinforced soil slope intended to mitigate the landslide. An initial analysis was performed in each case in order to evaluate the factor of safety of the slide section. Then, the design section of RSS was used to give a suitable design for a reinforced soil slope in order to mitigate the landslide. A factor of safety of 1.4 was considered adequate for conditions of equilibrium. The reinforced slope was extended into the bedrock in order to prevent failure surface from developing through the interface between the reinforced mass and bedrock. Figs. (6–9) show several RSS model analysis results, which show design outputs for the reinforcement based on FHWA guidelines. It is clear from the results that a reinforced slope is adequately capable of mitigating the landslide. Tables (4) and (5) list the parameters used for the RSS calculations and finite element model. Also, the results of the limit equilibrium analyses are tabulated in table (6).

Limit equilibrium analysis gives a good initial design for the reinforced slope area in the landslide. However, no information is provided by the method regarding the deformations. In order to verify the suitability of the given design for mitigation of Flourd landslide, FE analysis was performed on one of the models in order to assess the deformation characteristics of the model. PLAXIS finite element code was used for the analysis. 15 node elements were used to create a mesh for the problem. The resulting finite element mesh is depicted in Fig. (10). A reinforcement vertical spacing of 0.5 *m* was chosen for the model, since only the deformation characteristics were intended from this analysis and no design output was to be arrived at in this stage. Lateral boundaries were fixed in the horizontal direction while the bottom horizontal boundary was fixed in both directions. Geotextile elements available in PLAXIS code was used to model geotextile layers. An interaction index of 0.85 was chosen for the interface elements.

Maximum displacements in the reinforced slope were obtained as 11.4 cm, which is equivalent to % 0.9 of the slope height. Total displacement vectors are shown in Fig. (11) which shows a maximum displacement of 0.3 *cm*.

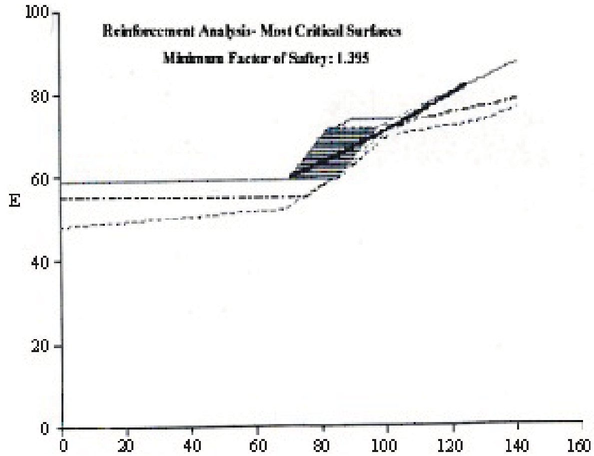


**Fig. 6:** Initial design of reinforced slope for 1.5:1 slope

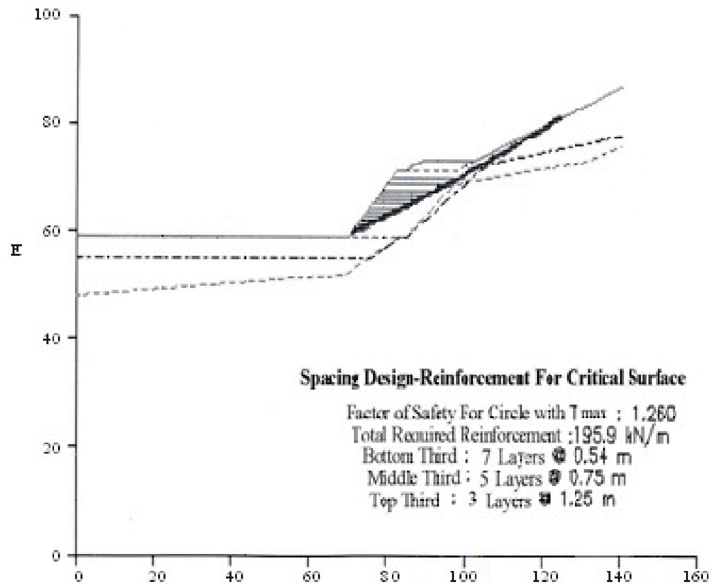


**Fig. 7:** RSS design output for 1.5: 1 slope

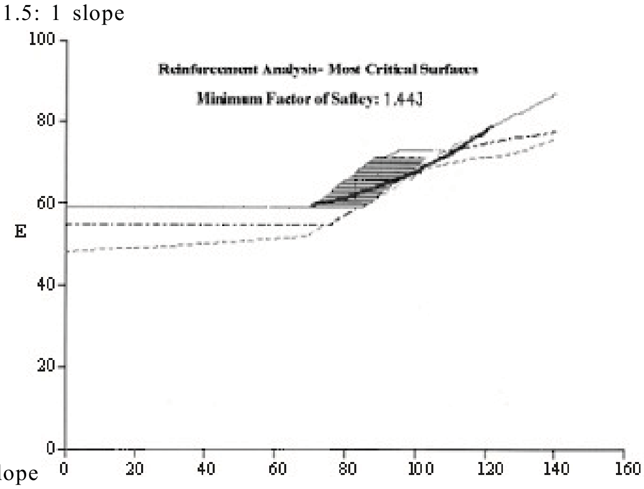


**Fig. 8:** Initial design for 1:1 slope

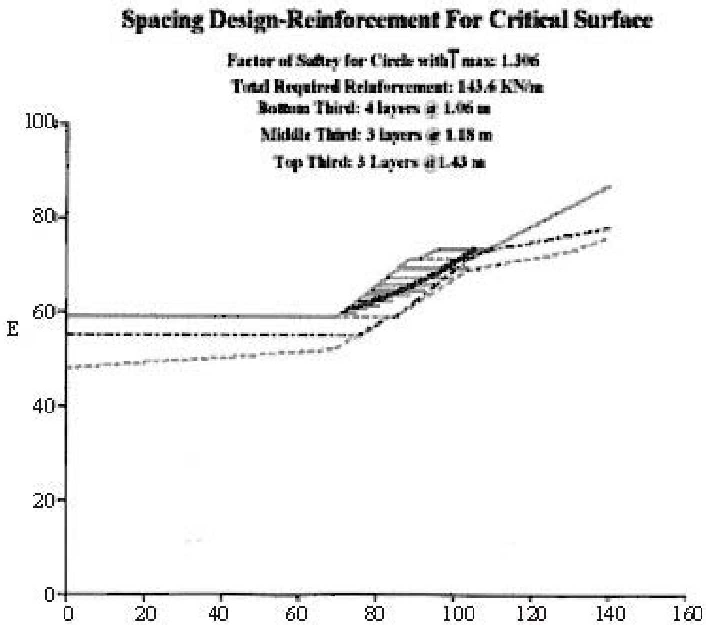


**Fig. 9:** RSS design for 1:1 slope

**Table 4:** Properties of soils used in limit equilibrium and finite element analysis

| Kind of Soil | Moist Unit Weight(kN/m$^3$) | SaturatedUnit Weight(kN/m$^3$) | Internal Friction Angle | Cohesion (kPa) | Modulus of Elasticity (MPa) | Poisson's Ratio |
|---|---|---|---|---|---|---|
| Bed Rock | 20 | 20.5 | 45 | 100 | 100 | 0.25 |
| Unreinforced Soil | 17.4 | 18.5 | 28 | 5 | 7 | 0.35 |
| Reinforced Soil | 18 | 19.2 | 15 | 10 | 15 | 0.3 |

**Table 5:** Properties of geotextile used in limit equilibrium and finite element analysis

| Geotextile Parameters | |
|---|---|
| Ultimate Strength (KN/m) | Geotextile Stiffness (EA) |
| 15 | 45 |

**Table 6:** RSS results for different slope angles

| Reinforced Slope | | | | Unreinforced slope | Slope Angle |
|---|---|---|---|---|---|
| RSS Design | | Initial Design | | Factor of Safety | ---------- |
| Reinforcement layout | Required Reinforcement | Simplified Bishop Factor of Safety | Sliding Block Factor of Safety | | |
| 7@ 0.54m<br>5@0.75m<br>3@1.25m | 180.62 m/m | 1.395 | 1.423 | 1.260 | 1:1 |
| 4@1.06m<br>3@ 1.1 8m<br>3@ 1.43m | 153.1 m/m | 1.443 | 1.812 | 1.306 | 1.5:1 |
| --- | --- | 1.549 | 2.329 | 1.485 | 2:1 |

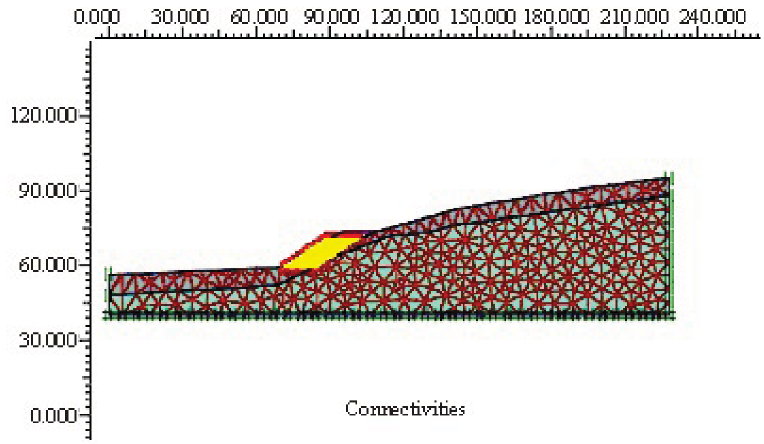


**Fig. 10:** FE mesh for the reinforced slope.

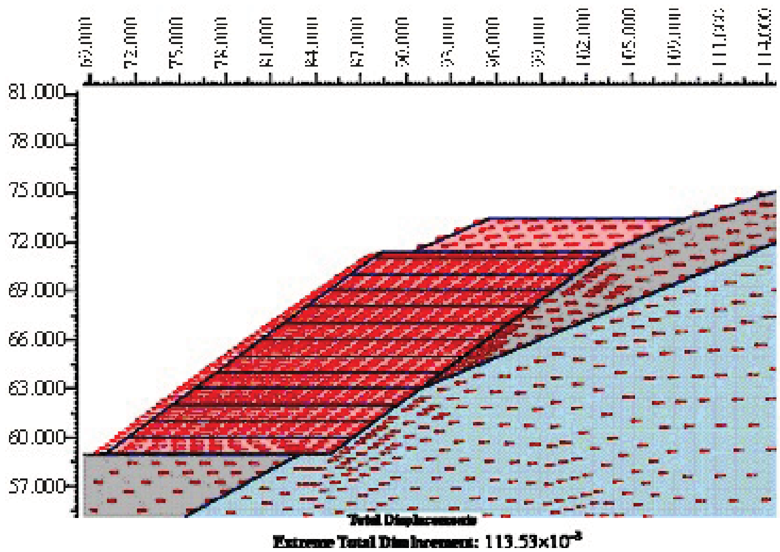


**Fig. 11:** Extreme total displacement vectors in the reinforced slope mass

Incremental shear strains as depicted in Fig. (12) reveals that strain localization occurs mainly in the interface area between soil-geotextile layers. It can be seen that maximum shear strain increment is measured as % 0.93. These low values prove that geotextile layers adequately mitigate the strains within the slide mass. Fig. (13) Shows the plastic points of the slope, once again showing that maximum displacements occur in the soil-geotextile interface, and that no clear failure mechanism is forming in the slope. Moreover, comparison between plastic points and incremental strains show that plastic points are attributed not to the base soil of the embankment, but to the interface area of soil-geotextile layers. This is due to low interface properties of non-woven-geotextile layers. Although woven geotextiles have higher interface properties, however, drainage characteristics of non-woven geotextiles result in an increase of shear strength in the interface area, thus increasing interface properties compared to woven geotextiles. Therefore, the observed behavior in the reinforced soil is inevitable and shear strength properties may only be increased through other methods such as the sandwich technique.

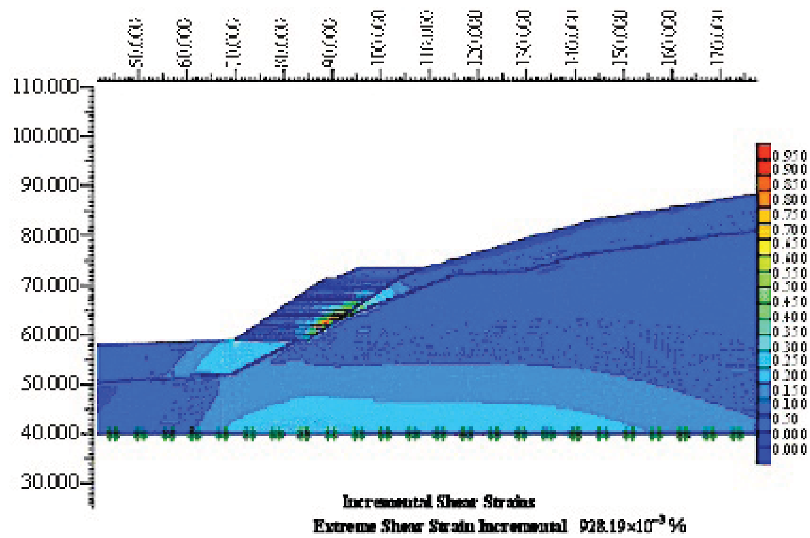


**Fig. 12:** Incremental shear strains within the reinforced mass

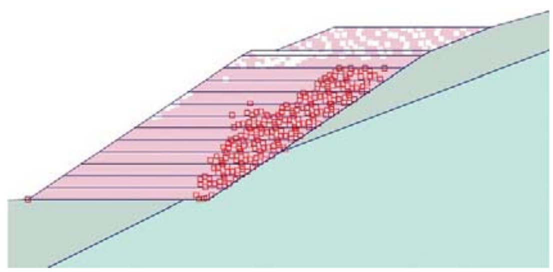

**Fig. 13:** Plastic points from the FE analysis

Stress-strain analysis was also performed by PLAXIS on non-reinforced model of the natural state of the slide area in order to observe actual displacement characteristics of the landslide prior to mitigation. Natural state prior to the slide was predicted from the slide geometry. Displacement vectors revealed that maximum displacement of 52.7 cm occurs in the slope which is equivalent to % 4.25 of the slope height. These high displacement values reveal the circular failure mechanism that is forming within the slope. Figs. (14) and (15) show plastic points and shear strains within the model, better revealing the circular failure mechanism.

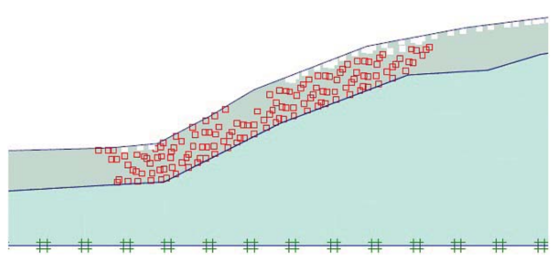

**Fig. 14:** Plastic points within the unreinforced embankment

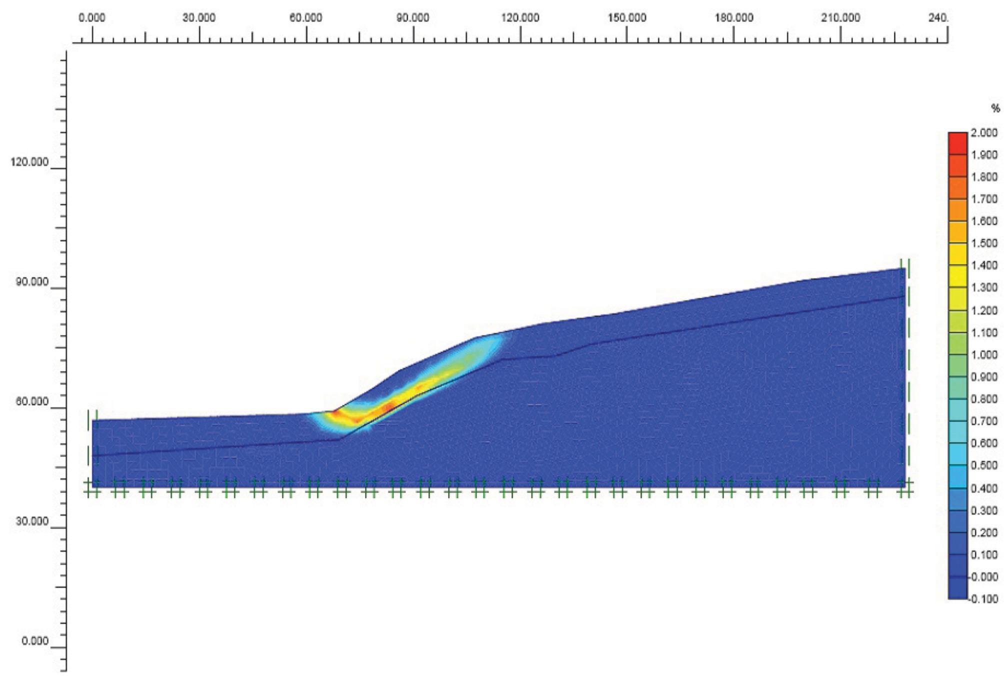


**Fig. 15:** Shear strains within the unreinforced embankment revealing circular failure surface.

***Conclusions:***

Limit equilibrium analysis and stress-strain analysis performed both on unreinforced and reinforced models of Flourd landslide were performed in this study. General conclusions were drawn as follows:

1. Finite element analysis on the slide mass revealed that circular failure surface developed in the landslide, therefore approving the circular failure mechanism assumed in the LE analysis.
2. The analyses showed that using in-situ cohesive soil reinforced with geotextile layers is adequately able to mitigate shallow to medium landslides.
3. In order to make full use of beneficial effects of reinforcement, the reinforced area should extend into the harder medium.
4. Reinforcing the slope reduces both horizontal and vertical displacements substantially.
5. Non-woven geotextile layers are more suitable for reinforcing cohesive soils than woven geotextiles, due to proper drainage properties.

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