Case study on the performance of reinforced soil slopes

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Abstract:

The term paper deals with the performance of reinforced soil slopes in various locations and using various techniques for reinforcement. The four case studies showcase different reinforcement methods like use of soil nailing, geogrids and geo-synthetic sheets. Also the locations of the four slopes are very different from one another as one slope fails due to water pressure, other is an earthen dam. One case study deals with the effect of seismic motions on reinforced soil slopes.

Introduction:

Reinforcement in soil slopes is provided to enhance slope stability. This can be applied in a variety of practical applications such as retaining walls, bridge abutments, dams, seawalls, and dikes. The reinforcing elements used can vary but include steel and geo-synthetics.

Reinforcement placed in horizontal layers throughout the height of the wall provides the tensile strength to hold the soil together. The reinforcement materials of RSS can vary. Originally, long steel strips 50 to 120 mm (2 to 5 in) wide were used as reinforcement. These strips are sometimes ribbed, although not always, to provide added friction. Sometimes steel grids or meshes are also used as reinforcement. Several types of geo-synthetics can be used including geogrids and geotextiles. The reinforcing geo-synthetics can be made of high density polyethylene, polyester, and polypropylene. These materials may be ribbed and are available in various sizes and strengths.

The main advantages of reinforced soil slopes compared to conventional reinforced concrete walls are their ease of installation and quick construction. They do not require formwork or curing and each layer is structurally sound as it is laid, reducing the need for support, scaffolding or cranes. They also do not require additional work on the facing.

Some of the soil reinforcement are described below:

Soil Nailing:

Soil nailing is a technique in which soil slopes, excavations or retaining walls are reinforced by the insertion of relatively slender reinforcing elements into the slope – often general purpose reinforcing bars.

Such structural element which provides load transfer to the ground in excavation reinforcement application is called nail (Fig. 1.1). Soil nails are usually installed at an inclination of 10 to 20 degrees with horizontal and are primarily subjected to tensile stress. Tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation process. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to the other retaining wall systems. As construction proceeds from the top to bottom, shotcrete or concrete is also applied on the excavation face to provide continuity.

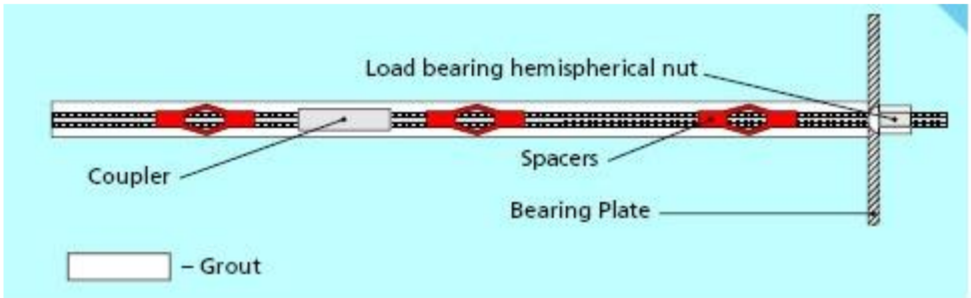


Figure 1: A typical Soil Nail

Elements of Soil Nailing:

Various components of a grouted soil nail are discussed in this section. The cross-section of a nailed wall is presented in Fig. 2.

- **Steel reinforcing bars** – The solid or hollow steel reinforcing bars are the main component of the soil nailing system. These elements are placed in pre-drilled drill holes and grouted in place.
- **Centralizers**- PVC material, which is fixed to the soil nail to ensure that the soil nail is centered in the drill hole.
- **Grout** – Grout is injected in the pre-drilled borehole after the nail is placed to fill up the annular space between the nail bar and the surrounding ground. Generally, neat cement grout is used to avoid caving in drill-hole; however, sand-cement grout is also applied for open-hole drilling. Grout transfers stress from the ground to the nail and also acts as corrosion protection to the soil nail. Grout pipe is used to inject the grout.
- **Nail head** – The nail head is the threaded end of the soil nail that protrudes from the wall facing. It is a square shape concrete structure which includes the steel plate, steel nuts, and soil nail head reinforcement. This part of structure provides the soil nail bearing strength, and transfers bearing loads from the soil mass to soil nail.
- **Hex nut, washer, and bearing plate** – These are attached to the nail head and are used for connecting the soil nail to the facing. Bearing plate distributes the force at nail end to temporary shotcrete facing.
- **Temporary and permanent facing** – Nails are connected to the excavation or slope surface by facing elements. Temporary facing is placed on the unsupported excavation prior to advancement of the excavation grades. It provides support to the exposed soil, helps in corrosion protection and acts as bearing surface for the bearing plate. Permanent facing is placed over the temporary facing after the soil nails are installed.
- **Drainage system** – Vertical geo-composite strip drains are used as drainage system media. These are placed prior to application of the temporary facing for collection and transmission of seepage water which may migrate to the temporary facing.
- **Corrosion protection** - Protective layers of corrugated synthetic material- HDPE (High Density Polyethylene) or PVC tube surrounding the nail bar is usually used to provide additional corrosion protection.

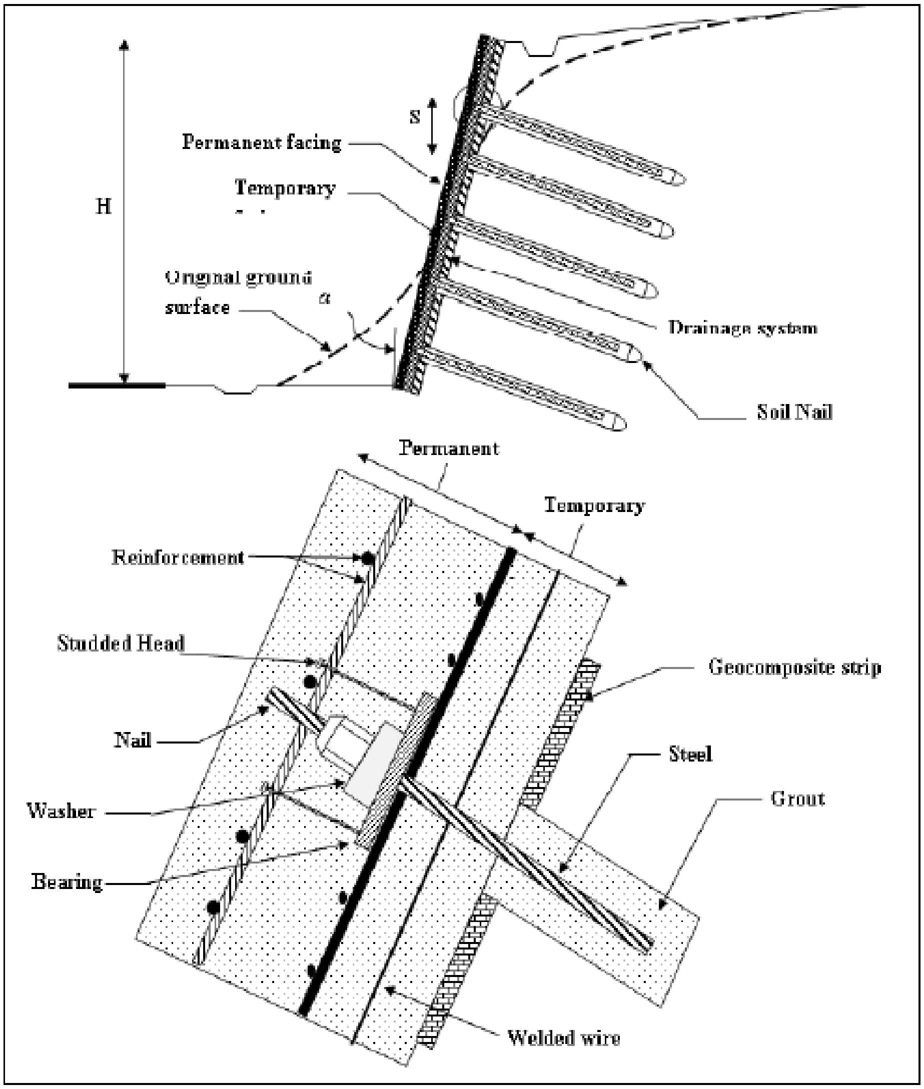


Figure 2: Typical cross-section of a soil nailed wall

Geogrids:

A geogrid is geo-synthetic material used to reinforce soils and similar materials. Soils pull apart under tension but geogrids are strong in tension. This fact allows them to transfer forces to a larger area of soil than otherwise.

Geogrids are commonly made of polymer materials, such as polyester, polyethylene or polypropylene.

Figure 3: Geogrids made from different materials

Geo-synthetics:

Geosynthetics are the generally polymeric products used to solve civil engineering problems. The polymeric nature of the products makes them suitable for use in the ground where high levels of durability are required.

Reinforcement is the synergistic improvement of a total system's strength created by the introduction of a geotextile, geogrid or geo-cell into a soil or other disjointed and separated material. Applications of this function are in mechanically stabilized and retained earth walls and steep soil slopes; they can be combined with masonry facings to create vertical retaining walls. Also involved is the application of basal reinforcement over soft soils and over deep foundations for embankments and heavy surface loadings. Stiff polymer geogrids and geo-cells do not have to be held in tension to provide soil reinforcement, unlike geotextiles.

Case 1: Failure of Five Berms Soil Nailed Slope + Seven Berms Cut Slope

This study presents the investigation results of a soil nailed slope failure in Malaysia and discussion on the lessons learnt. The failure site is underlain by completely weathered Shale facies, with the existence of mudstone and siltstone.

The original slope consisted of the upper cut slope and lower soil nailed slope with the following configuration as shown in Figure 1:

- Seven upper berms of 1V:1H cut slope with a 2m width berm provided at every 6m height interval. The total slope height was 42m.
- Five lower berms of 4V:1H soil nailed slope with a 2m width berm provided at every 6m height interval. The slope was reinforced with 12m length soil nails and the total slope height was 30m.

These cut slope and soil nailed slope were constructed to facilitate the formation of a new road.

The site is located on high ground with reduced level ranging from RL210m to RL330m and is underlain by shale facies consisting of mudstone and siltstone. Post-failure exploration and geological mapping showed that the cut slope face varied from a relatively smooth surface to irregular rough surface. In general, the site was dry and no water seepage was observed. The geological mapping revealed that the joint sets mapped at the slope surface were daylighting (meeting the slope surface) towards the main road. Joints with in-filling material like iron oxide and silt were also observed. Most of the exposed materials on the slope surface were Grade III (moderately weathered) to Grade V (completely weathered).

Methodology

A subsurface investigation consisting of two boreholes and relevant laboratory tests was planned and implemented to establish the subsoil profile and obtain necessary soil strength parameters.

Tests Performed

The following tests were performed from the samples recovered after subsurface exploration:

- Atterberg limits
- Particle size distribution
- Multiple reversal shear box test
- Consolidated Isotropically Undrained (CIU) triaxial test with pore pressure measurements
- Petrographic analysis

Based on the British Soil Classification System, most of the tested samples are silt of intermediate to high plasticity. Three sets of CIU tests were also carried out on samples of Grade III and IV material. In addition, two multiple reversal direct shear box test were also performed on the reconstituted samples from the CIU specimens.

Weathering Grade	Effective cohesion, c’		Effective friction angle, ϕ’	
Grade IV	Peak	30 kPa	Peak	33°
	Residual	0 kPa	Residual	33°
Grade III	Peak	30 kPa	Peak	39°
	Residual	0 kPa	Residual	33°

Checking Rainfall Record

Sometimes, excessive rainfall can cause heavy erosion leading to failure. So, rainfall record was obtained for the failure area and it was found that there was no record of high rainfall before the slope failure event.

Analysis of the slope

Slope stability analysis was carried out using limit equilibrium and finite element methods to investigate the causes of the failure.

- **Limit Equilibrium Method**: Slope stability analysis using Bishop’s Method was performed using the established subsoil profile and shear strength parameters from the subsurface investigation. The results indicated that the global Factor of Safety (FOS) was marginally higher than 1.0 even when all the soil nails were completely installed at the lower 5 berms. Also, the FOS for local stability for the 1V:1H upper cut slope was also very close to 1.0. Following figures show the analyses results for both global and local stability.

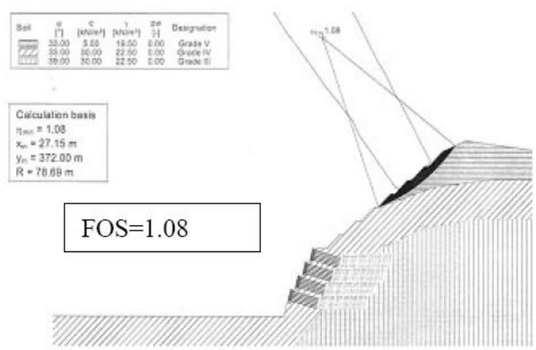


Figure 4: Limit Equilibrium Analysis Results for Local Slope Stability

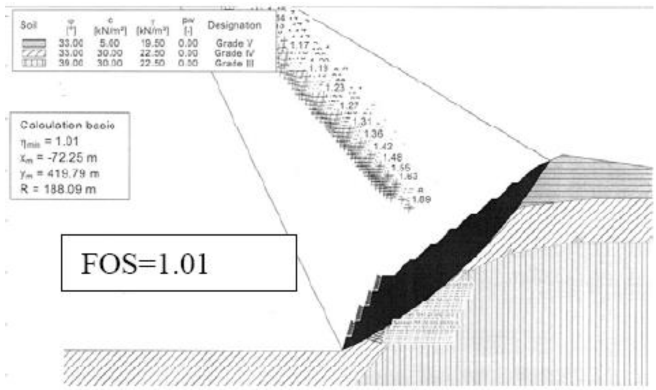


Figure 5: Limit Equilibrium Analysis Results for Global Stability

- **Finite Element Method:** Finite element analysis using the Mohr-Coulomb strength criteria and elasto-plastic model were also carried out to simulate the slope cutting at various stages (Figures 8 and 9) and to reveal the likely failure mechanism. From the modelling of each excavation stage, it was apparent that the development of plastic points within the soil body indicates mobilization of peak strength in these soil elements. When the cutting is in progress, the plastic points gradually develop and propagate to the lower areas. Eventually, a well-defined shear surface is formed when the excavation reached the lower berm.

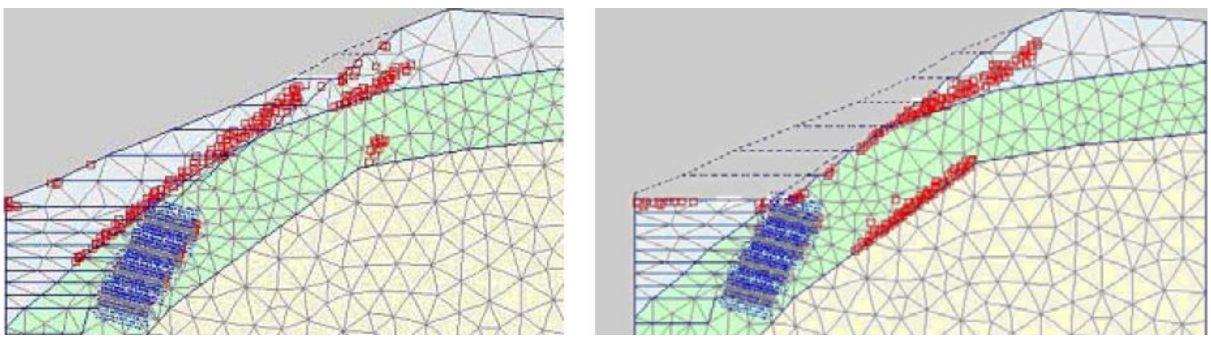


Figure 6: Determination of plastic points during initial stages of cutting

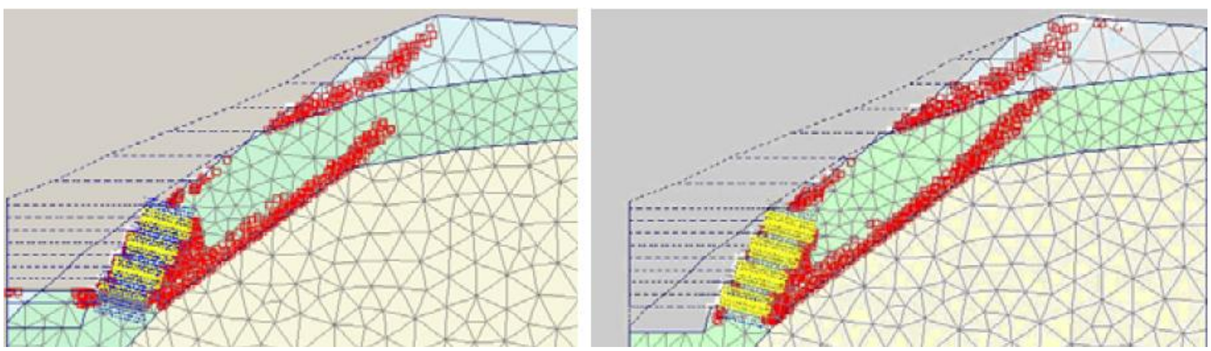


Figure 7: Determination of plastic points during final stages of cutting

Conclusions

- Based on both the analyses results, it was evident that the cause of the failure was due to inadequate Factor of Safety. Limit equilibrium analyses revealed that the FOS against global and local stability are just marginally higher than 1.0.

- The unfavorable day lighting geological structures were likely contributing to the slope failure.
- The finite element analysis shows that the shear failure surface gradually developed as the excavation progresses to the lower berms of the slope, indicating the mechanism of progressive failure.

Possible Solutions

- Increment of reinforcement in the slope
- Reduction in angle of slope to increase FOS if the conditions of the site permit so
- Construction of rock buttresses if the space is limited and slope angle can't be reduced

Case 2: Performance of Geotextile Reinforced Slopes of Earthen Dam

Introduction

This study examines the application of soil reinforcement technique for slope stability of earthen dam, through the case study of the Wasani dam section designed by Water Resources Department of Maharashtra.

The effect on factor of safety is studied by considering the slopes of the dam to be reinforced with horizontal layers of geotextile. Various factors of reinforcement such as spacing, length and offset from the face have been varied and their effect on stability is evaluated through Oasys Slope Software, to obtain an optimum configuration.

Methodology

Table 1 gives details of the designed slopes and berms of Wasani dam.

Top/ Lower Berm Widths (m)			***Slope H:V***	
R.L.	*u/s*	*d/s*	*U/S*	*D/S*
339.90	6.5	6.5	2.5:1	3.0:1
333.90	6.0	7.0	2.5:1	3.5:1
327.90	5.0	14.0	3.0:1	3.5:1

Fig 4 shows the model of the Wasani dam developed in Slope Module of Oasys software. The properties assigned to the dam material are presented in Table 2.

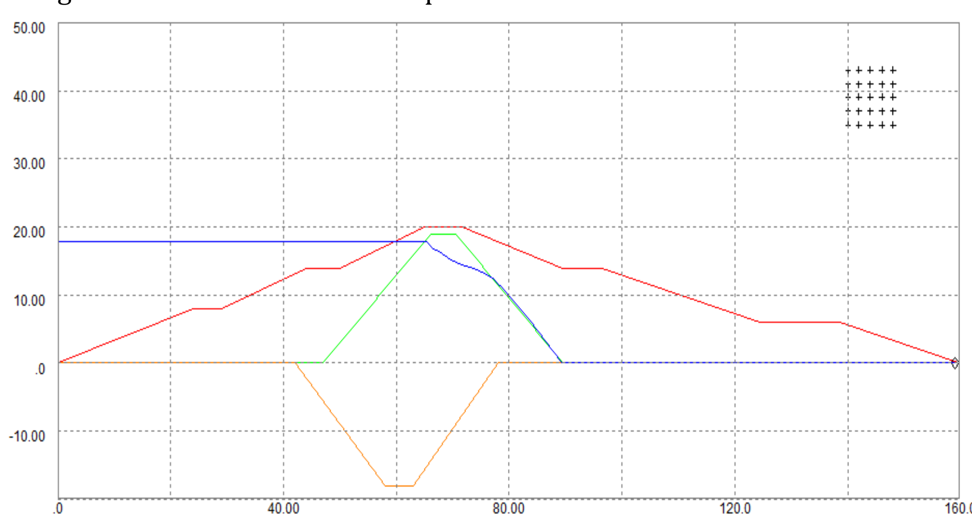


Figure 8: Model of Wasani Dam

Sr. No.	*Particulars*	*Unit Weight*	*Angle of Friction*	*Cohesion (kN/m^2)*
01	Casing	15.8	25.1	7
02	Hearting	15.1	14.5	18.5
03	Foundation	14.9	16.6	18

The tensile strength of the geotextile reinforcement to be provided for reinforcing the slopes of dam is considered as 200 kN/m. Dam section is analyzed using Bishop's method of analysis. The minimum factor of safety for both, reinforced and unreinforced section, is determined by selecting center of critical slip surface. This is accomplished automatically by the software. . The stepwise breakdown of the methodology adopted for analysis is given as below:

- Selection of general parameters like slip surface type (circular or non-circular), type of analysis, direction of slip (downhill, increasing x or decreasing x), value of horizontal acceleration (%g)
- Selecting method of analysis(Bishop Method)
- Assigning material properties such as unit weight, cohesion and angle of friction
- Setting ground water level
- Defining the slip surface
- Analysis and data checking
- Viewing the results
- Graphical output

Results

Stability Analysis of Unreinforced Slopes of Wasani Dam

The results obtained in the present study are presented in Table 3.

Sr. No.	*Conditions*	*Factor of Safety (FOS)*	
		Obtained by Irrigation	*Obtained by Slopes*
0	Steady seepage without	1.50	1.777
0	Sudden Drawdown without	1.31	1.460

Stability Analysis of Reinforced d/s Slope for Steady Seepage Condition

The downstream slope of the Wasani dam was considered to be reinforced with horizontal layers of geotextile in anticipation of increased factor of safety. As a result, steeper slopes may be provided to the downstream direction of dam. The d/s slope of the dam was reduced in increments by 0.25 H: 1V. The spacing between layers (z) was varied from 0.9 to 3 m for each case. Analysis was then carried out for each case separately. During analysis of each case the length of reinforcing layer (L) and their offset (x) from the d/s face of dam was varied in such a way that the reinforcing layers remains activated i.e. they intersected the potential failure surface and governing criteria of failure for the reinforcing layers remained as tensile. This ensured the maximum utilization of reinforcing layers and resulted in solutions that were economical. Typical graphical output obtained from software is presented in figure.

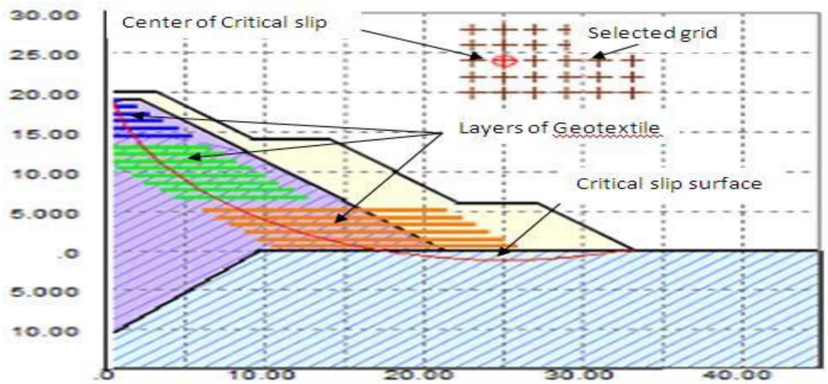


Figure 9: Typical graphical output of d/s slope

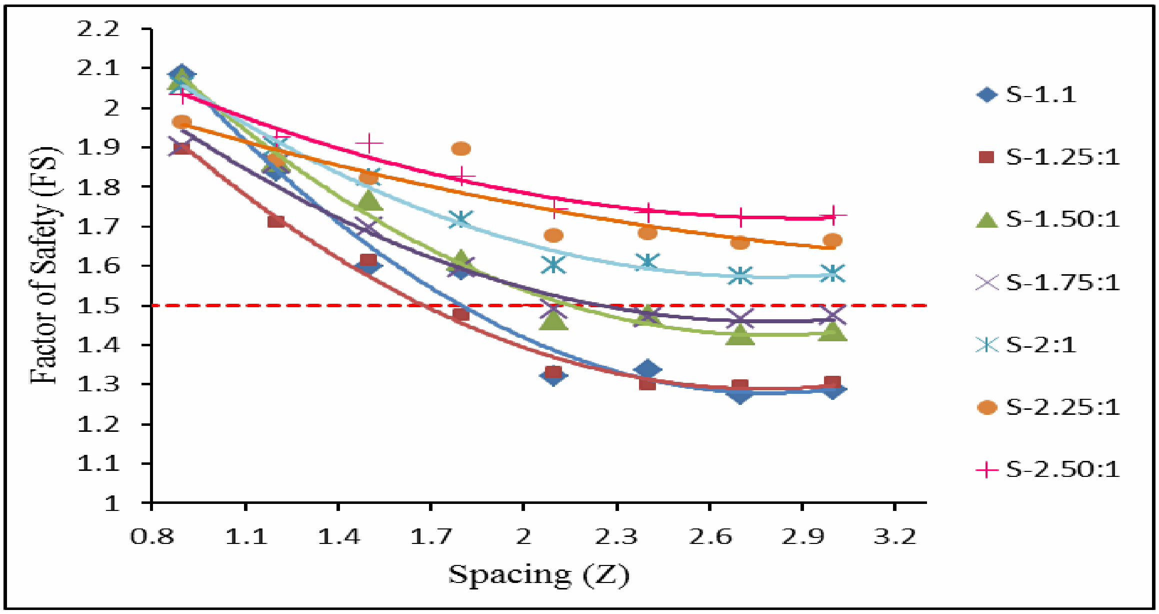


Figure 10: Variation of factor of safety with respect to spacing between geotextile layers for steady

From the figure, the effect of reducing spacing between reinforcement layers on factor of safety can be seen. It has to be ensured that the factor of safety does not fall below 1.5. For different slopes, the relation between factor of safety and spacing between reinforcement layers has been depicted. Hence it is inferred that increasing the spacing up to 3m allowed the downstream slopes of the dam to be steeper up to 2:1. For steeper slopes, the vertical spacing between geotextile layers is required to be reduced up to 1.5 m. The length of the geotextile layers required for reinforcing the d/s slopes were found to be in the range of 4 m to 16 m and offset from d/s slope was found to be in the range of 3 m to 45 m depending upon the location of geotextile layers.

Stability Analysis of Reinforced u/s Slope for Sudden Drawdown Condition

Similar to the downstream case, the upstream slope of the dam was considered to be reinforced with layers of geotextile and similar analysis was carried out. The slope was increased in increments of 0.25H:1V and the spacing was varied from 0.9m to 3m.

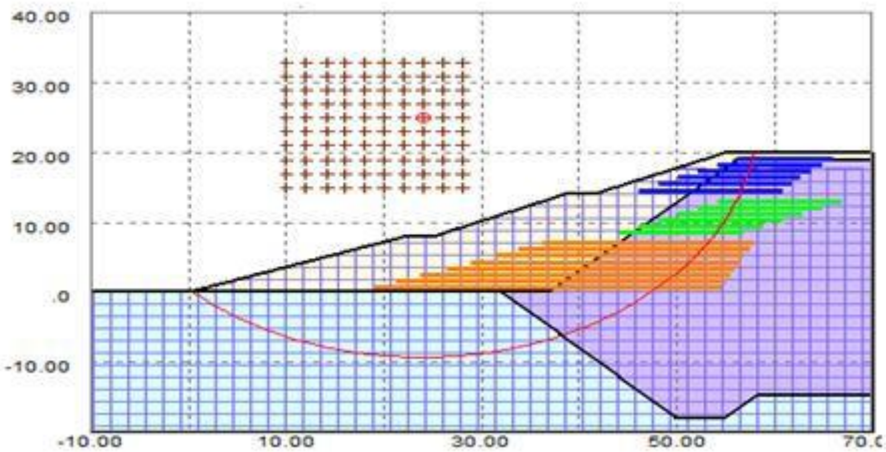


Figure 11: Output of Upstream Slope

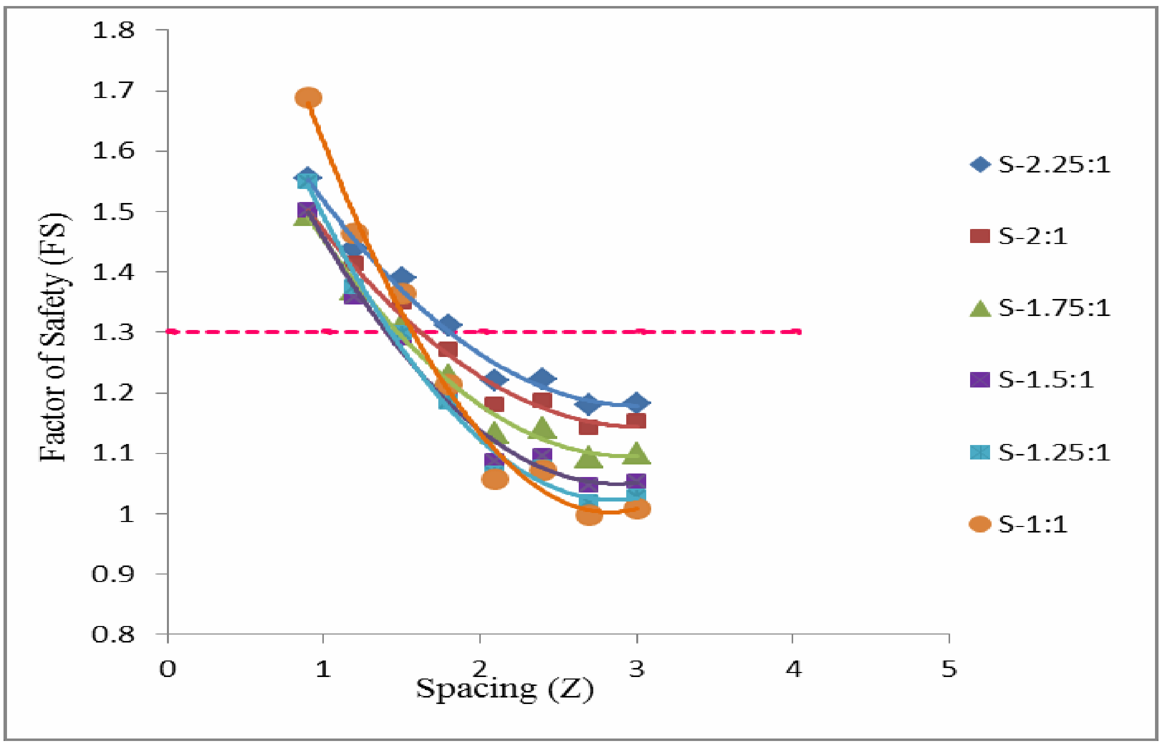


Figure 12: Variation of factor of safety with respect to spacing between geotextile layers for sudden drawdown condition

Similar to the previous case, factory of safety decreased as the spacing between the reinforcement layers was increased. It was also observed that by providing geotextile layers at spacing of 1.5 m, the u/s slope can be made as steep as 1:1. The length of the geotextile layers required for reinforcing the u/s slopes were found to be in the range of 4 m to 27 m and offset from u/s slope were found to be in the range of 3 m to 18 m depending upon the location of geotextile layers.

Economic Analysis

After comparison with unreinforced section, from the analysis, it was observed that the reinforced dam section is 36% more economical.

Case 3: Reinforced Soil Slope (RSS) Failure and Reconstruction US 70 Design-Build Project, New Mexico

Introduction

The paper deals with failure of a reinforced soil slope near Rio Hondo, adjacent to US 70 Design-Build Project in New Mexico.

The above stated project was a design-build highway project. US 70 alignment winds along foothills of mountains to the north and the rivers Rio Hondo and Rio Ruidoso to the south. The highway design required extensive cut slope excavation along the mountains and retaining walls & reinforced soil slope construction to the south along Rio Hondo. This was necessary to reduce the environmental impact of the highway on the immediate surroundings.

Methodology

The geotechnical investigation of the area using two soil borings indicated that the geology in the area was terrace deposits and alluvium overlying mudstone. No signs of slope instability were noticed at the time.

The design of critical slope also included a toe key of excavated and re-compacted soil below the reinforced soil slope. The slope was designed at an 1/2:1 inclination and was evaluated to have a safety factor of 1.3 (it met the slope and project design requirements). The reinforced slope included the use of geogrid as reinforcement.

The contractor which built the reinforced slope changed the design as the toe of RSS was very near the edge of river. So they built a riprap-filled trench to protect RSS from scour. The final slope ranged from 1/4:1 to1/2:1.

Problems

Heavy rains after summer of 2004 resulted in severe cracking of pavement and rotational movement of reinforced soil slope. These vertical cracks, when investigated, were about 5cm deep and were located directly behind the top layer of geogrid.

Borings, piezometers and inclinometers were installed to investigate the subsurface, amount of groundwater and to compute the change in slope inclination and RSS movement respectively.

I. Inclinometer:

Inclinometer systems are used to monitor deformation. The system includes an inclinometer casing, an inclinometer probe and a control cable, and an inclinometer readout unit.

They are used for the measurement of lateral earth movements which can occur in the following:

- Landslides
- Unstable slopes
- Dams
- Embankments
- Landfills.

They are also used to measure deflections in the walls of excavations, shafts, tunnels and in caissons, piles and sheet piling.

Ground movement causes the casing to move away from its initial position. The rate, depth, and magnitude of this movement is calculated by comparing data from the initial readings to data from subsequent readings.

Figure 13: Inclinometer System

The inclinometer readings coupled with that of rain gauge showed that the major slope movements occurred after significant rainfall in the area. Surprisingly, the piezometer readings were significantly below the river water level even after significant storms.

Due to rotational motion of the reinforced soil slope, top 5ft of the soil behind reinforced slope was removed which resulted in considerable slowing of slope movement.

Complete back analysis of the slope using computer simulations of slope stability indicated:

- Water pressures behind the RSS and/or the uplift water pressure beneath the slope were prompting the slope motion and corresponded to periods of heavy precipitation.
- Simulations suggested that the translational sliding of the slope was more likely to occur than deep-seated circular failure.

The analysis concluded that the slope distress was a result of peak flows and saturation during heavy and prolonged rainfall that developed excess pressure in the rear of the slope.

II. Sources of Water Infiltration:

- Rise in river water level due to heavy rainfall
- Surface runoff infiltration
- Subsurface infiltration from north of highway from fractures in bedrock etc.
- Past subsurface structures that were hidden due to construction

Solutions & Reconstruction

Based on the reasons analyzed above for the movement of the slope the following recommendations were made for reconstruction:

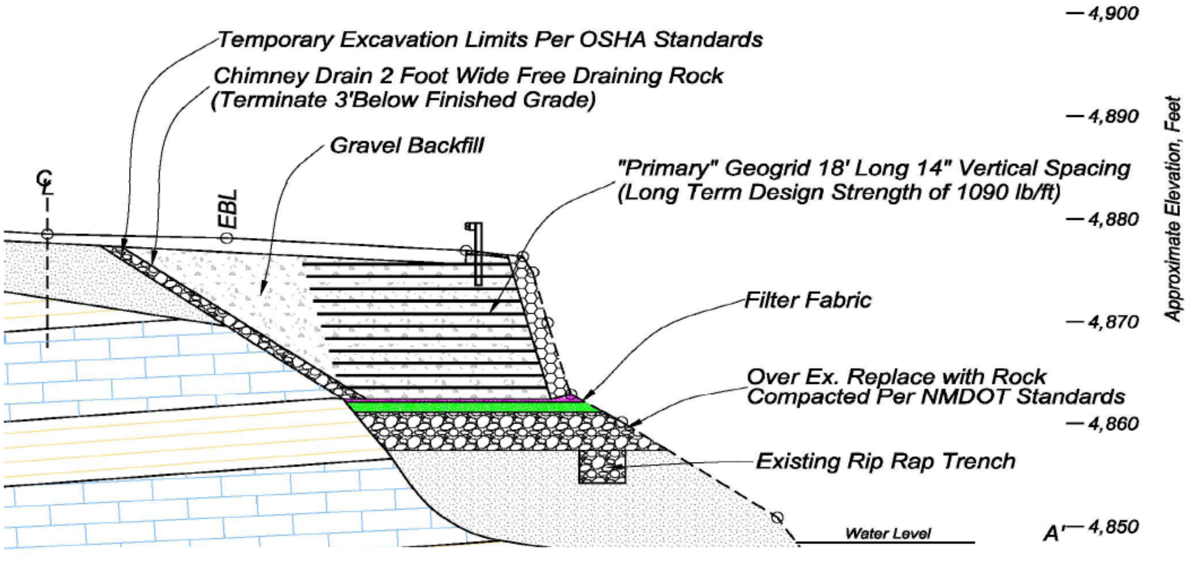


Figure 14: Recommendations for Reconstruction

About 6m long segments to bedrock were excavated to create a keyway and to remove unsuitable material below the embankment. The back and sides of the keyway excavation were lined with filter fabric to reduce the potential for piping of fines from the native, finer-grained materials into the rock fill.

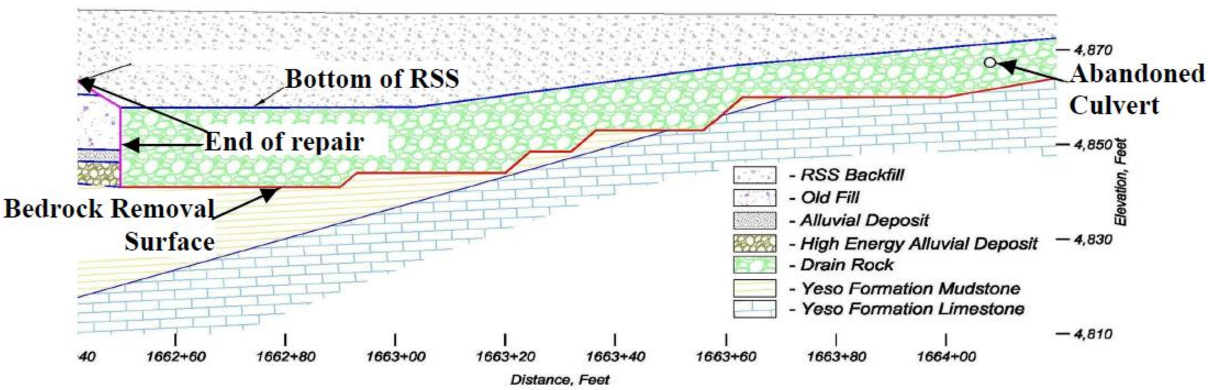


Figure 15: As-Build Longitudinal Profile

Each keyway segment was backfilled with angular rock fill [typically greater than 0.6m (2 ft.) size] to protect against undermining and erosion during peak river flows. Another 0.6 m (2 ft.) of smaller rock fill 15 cm to 0.6 m (6in. to 2 ft.) size was placed over the large rock fill, followed by approximately a 0.3 m (1 ft.) thick layer of 1.9 cm (¾ in.) aggregate base (AB) compacted into the rock mass. The AB layer was smoothed to form a foundation for the new RSS, and filter fabric was used to separate the AB from the RSS fill.

At the back of excavation, a geo-composite drain fabric was extended vertically from the existing pavement down to the smaller rock fill layer. RSS was constructed at a 1/2:1 stepped slope.

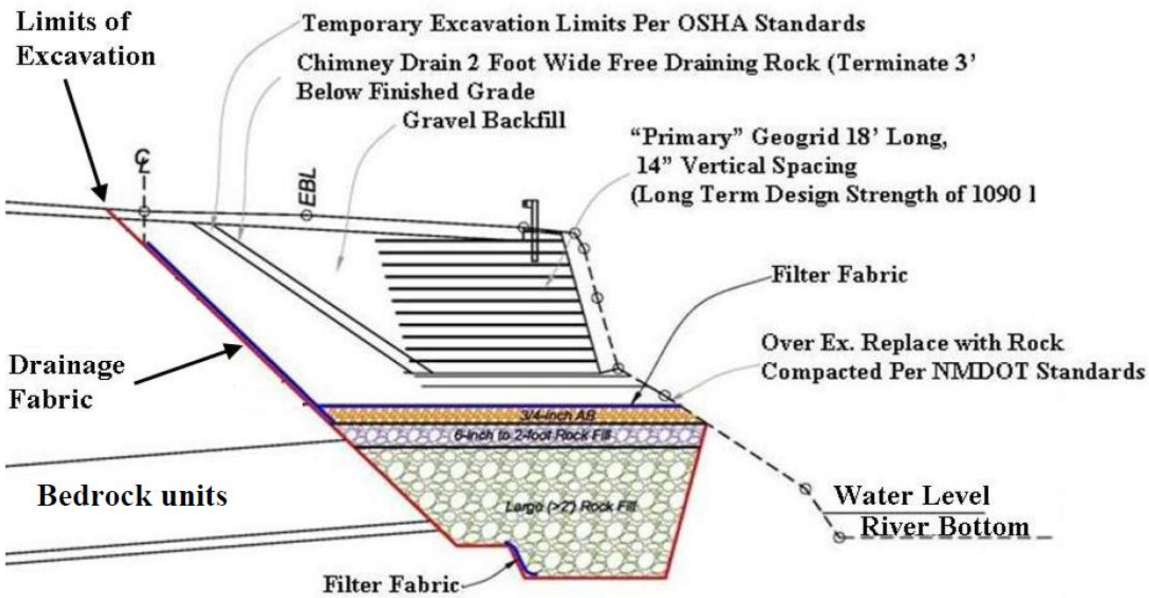


Figure 16: As-Build Cross Section

During reconstruction of the RSS, an existing culvert was exposed. Seepage had been noted in the Rio Hondo riverbank at about this location when the slope distress was noted. The culvert was abandoned with grout during reconstruction of the RSS.

III. Some other possible solutions:

- Installation of drainage pipes through reinforced soil slope to allow the flow of excess water from the north side of the highway and from the rear of slope.
- Construction of reinforced retaining wall to counter the movement of the slope. But this could have side effects like disruption in overflowing of river and removal of toe of slope which reduces its stability more.

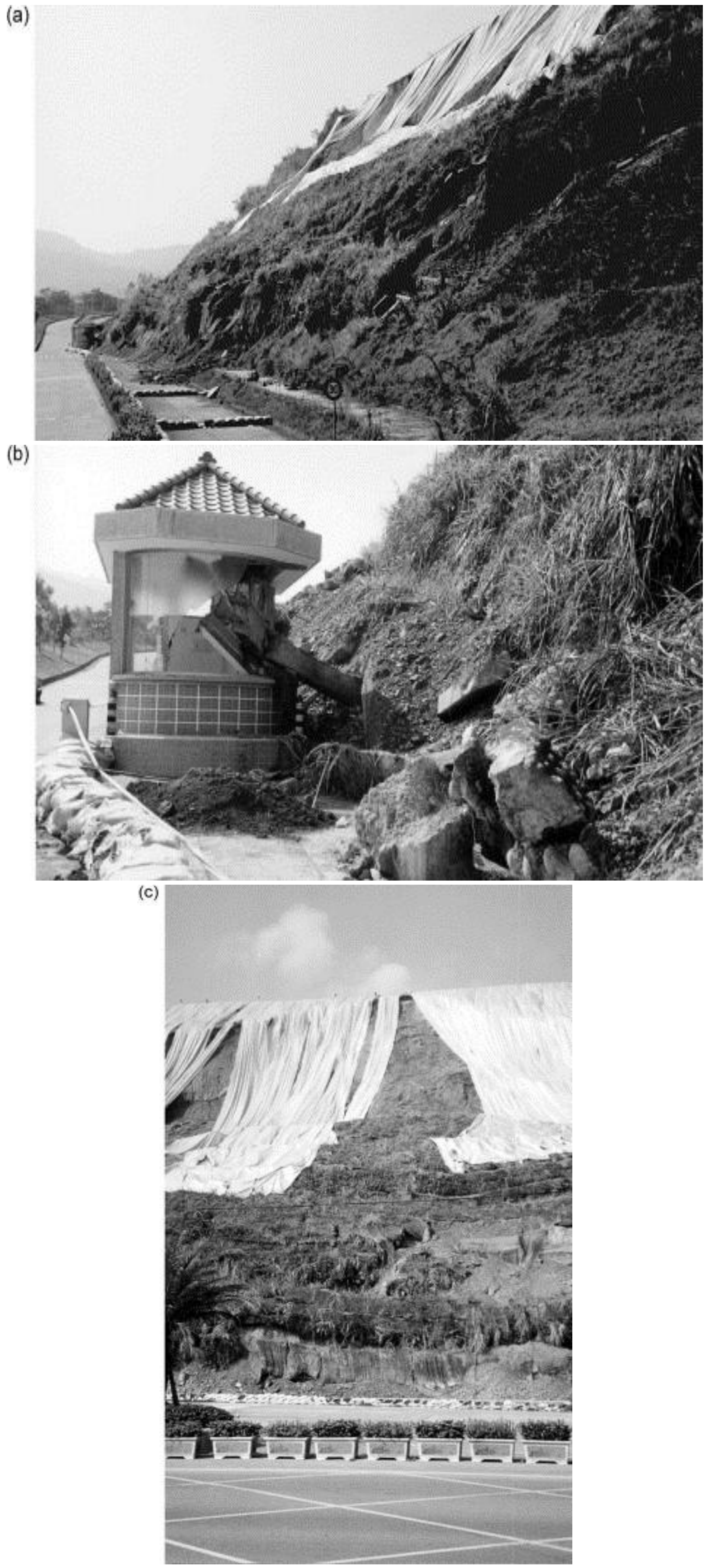
Case 4: Post-earthquake investigation on several geo-synthetic reinforced soil retaining walls and slopes during the Ji-Ji earthquake of Taiwan

Ji-Ji earthquake of Taiwan (Case of Chi-Nan University, Pu Li)

Pu Li is the town that was most severely damaged by the earthquake. It was located at about 25 km from the epicenter. The reinforced slope, 40 m tall, was located at the front gate of National Chi-Nan University, facing east.

Methodology

The geogrids were used as reinforcement and the slope was backfilled by on-site soil, which was silty-clay. The slope had a wrap-around facing. The reinforced structure was constructed by stacking a series of reinforced slopes, with a reinforcement spacing of 1 m. The images of failure slope are shown thus:


(a)
(b)
(c)

(d)

(e)

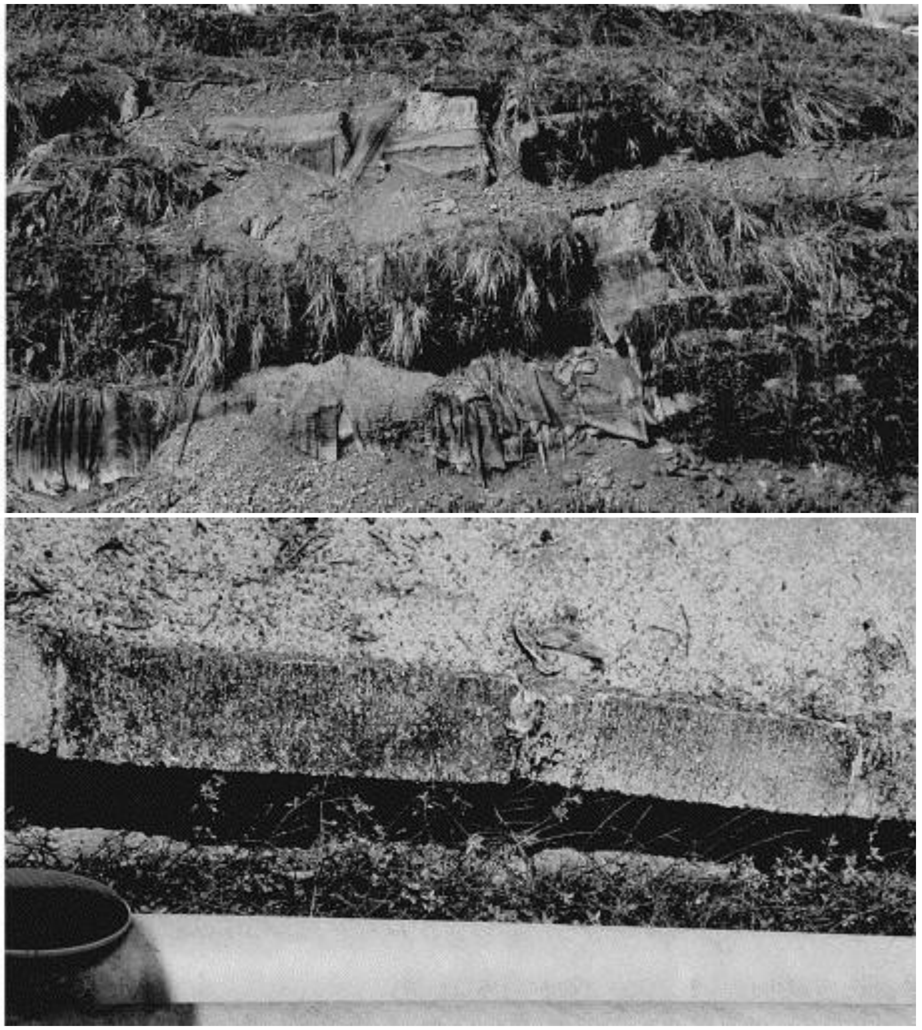


Figure 17: Chi-Nan University geo-synthetic-reinforced slope: (a) side view of failure, (b) damaged security office, (c) front view of failure, (d) close view of failure showing the reinforcement and backfill soil, (e) settlement of concrete pavement along the foot of the slope

Problem

The backfill soils and concrete structures from the slope moved for more than 10 m and buried the road. The security office was damaged. A close view of the slope shows that the reinforcements are seen to pull out of the slope. Note that the concrete pavement around the site, at the foot and crest of the slope, deformed excessively.

It is, however, not certain if the failure of this reinforced structure was attributed to the seismic excitation alone. Excessive deformation of this reinforced slope was reported previously following an excavation at the foot of the slope in 1994. The original configuration of this reinforced slope and the configuration after failure in 1994.

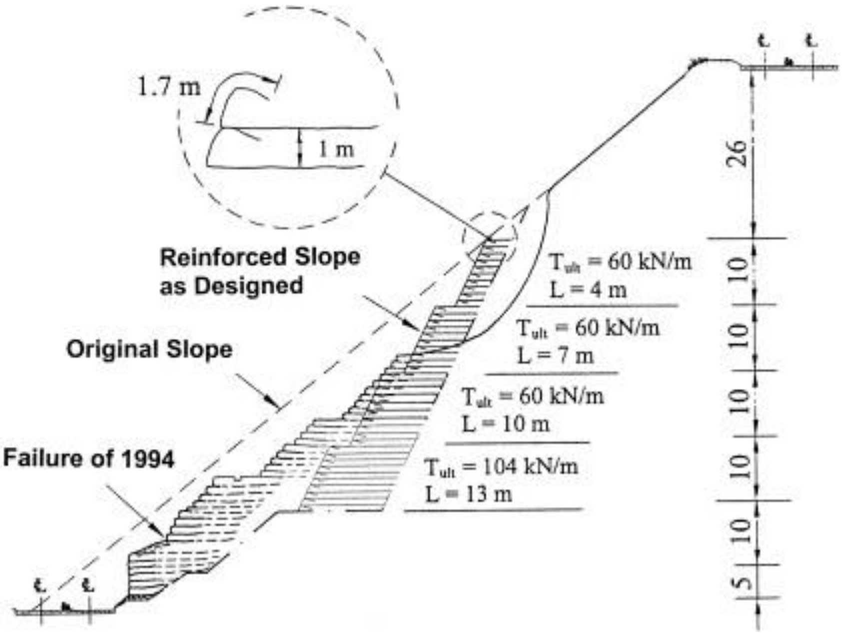


Figure 18: Configuration of Chi-Nan University slope before and after failure of 1994

Solution

According, to paper solution of this problem is another case study of Nai Lu housing development site, Chung Hsin New Village. A 35 m high reinforced structure, located near Chung Hsin New Village, remains stable after the earthquake. The structure was composed of six multiple reinforced slopes, facing south–west. The slope has a wrap-around facing and was fully vegetated. It was the tallest reinforced soil structures at the time of completion of construction. Note that the road pavement along the slope suffered significant damage.

(a)

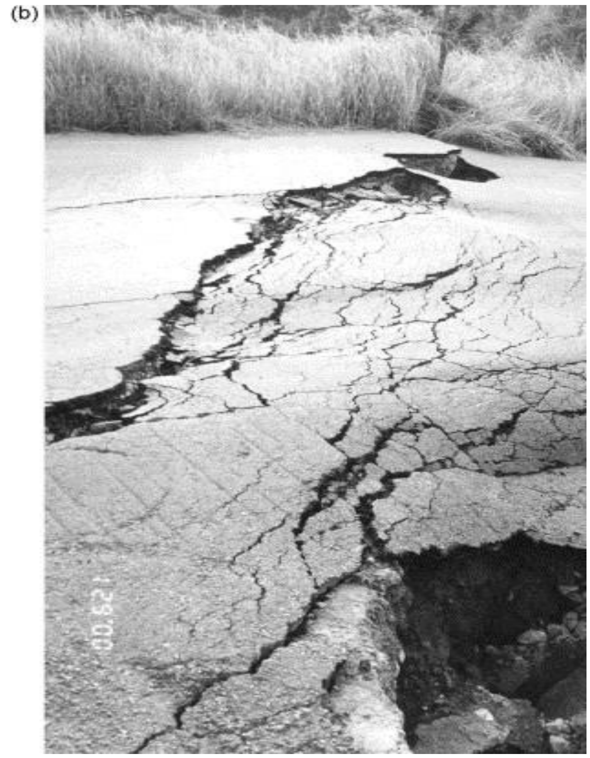


Figure 19: Nai Lu housing development site: (a) stable geo-synthetic-reinforced slope with vegetated facing, (b) severely cracked pavement along the road to the slope

Fig. 20 shows the configuration of this structure. The slope was constructed on a V-shaped valley having an inclination of 2(V):1(H) backfilled with on-site soils. The slope was designed for seismic stability with a seismic coefficient of 0.15. A HDPE geogrid was used. The spacing of reinforced slope was 50 cm and the reinforcement was 18.5 m long with an overlapping length of 2.5 m.

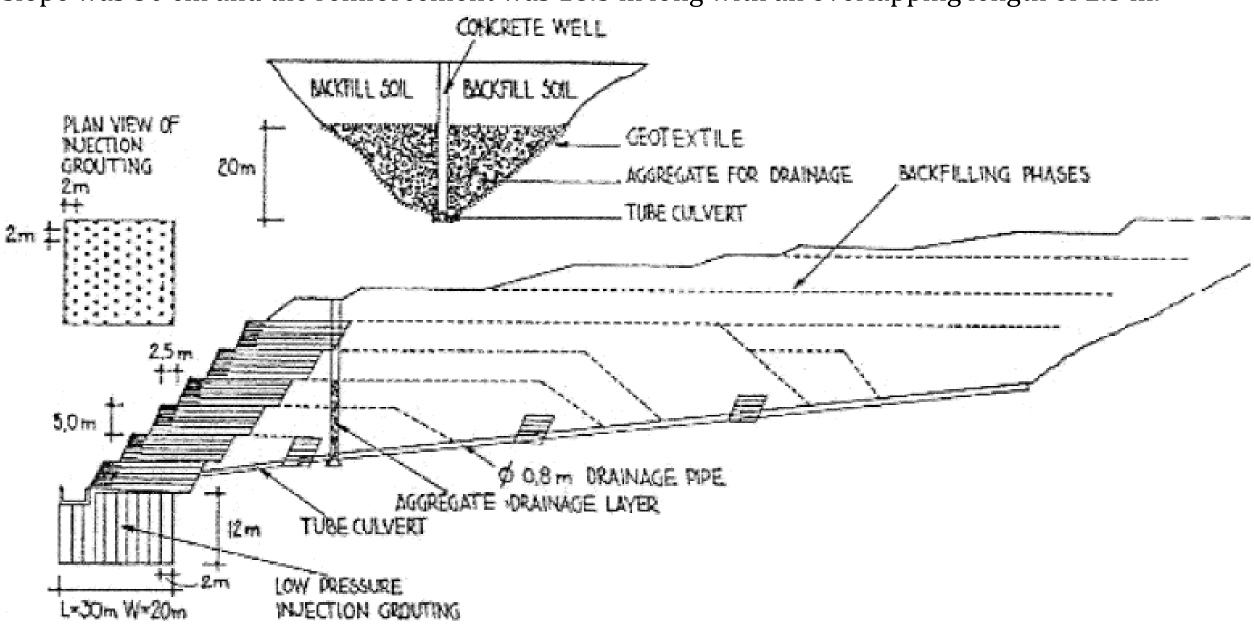


Figure 20: Cross-section of Nai Lu housing development site (Chou et al., 1995).

Note that the width of this slope was less than that of Chi-Nan University and the orientation was different as well. This reinforced slope behaved as an arch-like structure. The end effects could have improved the stability.

Conclusion

The following recommendations and conclusions can be made from the investigation of the above case studies:

- Thorough subsurface exploration and geological examination of exposed materials needs to be done during various stages of construction. If the subsoil profile, geological structures or groundwater table are found to be different from the design model, then the design has to be reviewed with the updated information. This design feedbacks and verifications are crucial in order to ensure safety of a soil nailed slope. Sometimes, further design optimization is possible if the ground condition is more favorable.
- Representative shear strength parameters should be used for the design of a soil nailed slope. Strength tests such as CIU and direct shear box tests should be carried out to obtain the strength parameters for design. For rock mass, the current available approach still relies on empirical method with respect to the database of failure case and visual classification based on rock mass characteristic.
- Proper drainage system should be provided to prevent water from seeping into the soil layers.
- The slopes should not be left exposed to erosion
- For earthen dams, it was found that much steeper slopes may be provided by providing the geotextile reinforcement. A spacing of 1.5 m between geotextile layers is found to be suitable for safe section of dam in all cases. In order to utilize the full strength of geotextile layers, geotextile layers of sufficient length and at appropriate offsets need to be provided. The economic analysis of the reinforced dam section indicates saving of about 36% in the cost.
- The case of reinforced slope failure on US 70 highway in Hondo Valley, gave insight on how excess water pressure can lead to failure of a properly designed structure. The missing information regarding the presence of culvert behind the slope also caused the failure as the culvert was a pathway for infiltration of water. The final solutions by project and geotechnical designers have now given a stable slope keeping in mind its behavior during periods of heavy and prolonged rainfall. The case study also teaches that only proper reinforcement with complete analysis of surroundings is needed for the proper construction and durability of a structure.
- The seismic design of reinforced soil structures has gained attention worldwide only in recent years. However, most of the seismic design procedures do not incorporate compound failure analysis. The failure of reinforced soil retaining walls could be attributed to a lack of professional design as seen by arbitrary spacing used in several of the reinforced soil retaining walls, and with a mixture of unreinforced and reinforced retaining walls within a common structure. The information obtained from post-earthquake investigation is invaluable for the verification and improvement of seismic design procedure.

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