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STABILITY ANALYSIS OF SOIL SLOPE IN SWABI, PAKISTAN

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ABSTRACT

In this study, slope stability analysis for a particular type of soil in Swabi area was carried out using the method of slices. Gravity load and seismic load were considered in the stability analysis. SLOPE/W and SIGMA/W software of Geo studio package were used for analysis under both dry and completely saturated conditions. Pseudo-static approach was used for the seismic stability analysis. The peak ground acceleration value for stability analysis was obtained from the seismic hazard analysis. Two soil slope conditions were considered, and one was found in critical condition. Retaining wall, Soil nails and ground anchors were assumed in the analysis for stability of slope with critical condition. Peak ground acceleration value of 0.235 g is determined with site specific deterministic seismic hazard assessment approach. The maximum displacement in the soil slopes with retaining wall, soil nails, and ground anchors are 0.0220, 0.01665 and 0.01529 feet respectively. Results showed that using ground anchors have a factor of safety 1.165 and both the horizontal and vertical deformation values are within the limit. Based on results, ground anchors technique was found to be a suitable method for slope stability in Swabi.

KEY WORDS: *slope stability, soil nails, ground anchors, land slide, slope failure*

1. INTRODUCTION

Slope failure or landslide is one of the seismic hazards that are commonly observed after ground shaking experienced by the affected area. Landslides or slope failures triggered by earthquakes result in disruption of communication lines, loss of human lives and of course monetary losses. In a developing country like Pakistan which is prone to seismic activities, the chances of losses are even higher. The most recent example of slope failures in the country was during the Kashmir earthquake which triggered 2,424 landslides[1].

Natural slopes that have been stable for many years may suddenly fail because of changes in the topography, seismicity, ground water condition, loss of strength of the slope materials, stress changes, and weathering. Generally, these failures are not understood well because almost no study is made until the failure of the area makes it necessary. In many instances, significant uncertainty exists relating the stability of a natural slope. This has been emphasized by Peck,[2] who said: "Our chances for prediction of the stability of a natural slope are perhaps best if the area under study is an old slide zone which has been studied previously and may be reactivated by some human operations such as excavating into the toe of the slope. On the other hand, our chances are perhaps worst if the mechanism triggering the landslide is (1) at a random not previously studied location and (2) a matter of probability such as the occurrence of an earthquake."

The objective of this study was to carry out the slope stability analysis and propose remedial measures for unstable slope at a site located in Swabi, Khyber Pakhtunkhwa, Pakistan. Captain Karnal Sher Khan Shaheed (NH) Cadet College, is located in Swabi district, 120 Km NE of Peshawar in Pakistan. The site is a part of Peshawar Valey which lies between latitudes 33°402 and 34°352 N and longitudes 71°152 and 72°452 E. Figure 1 shows the city of Peshawar and Swabi. The slope stability assessment includes both gravity (static) and seismic analysis (pseudo-static) analyses. The site is considered in zone-2b (0.16g to 0.24g) per Building Code of Pakistan (Seismic Provisions-2007). This acceleration corresponds to rock of shear wave velocity 760 m/sec. The slope material predominately consists of very dense gravel where standard penetration test would give a refusal. Hence horizontal seismic coefficient is taken to be 0.24g for pseudo-static analysis of slope. This value of seismic coefficient is assumed for analysis, considering (i) amplification of peak ground acceleration due

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to soil conditions being different than rock (ii) topographic amplification (iii) overestimation of seismic demand by considering peak ground acceleration for analysis which is a single momentary peak in the acceleration time history. The upper limit of peak ground acceleration is warranted here because the slope is considered critical as the global failure of the slope can result in collapse of water tank, cadet mess, and hostel-5. This can take around hundred human lives and a monetary loss of around 35 million rupees.

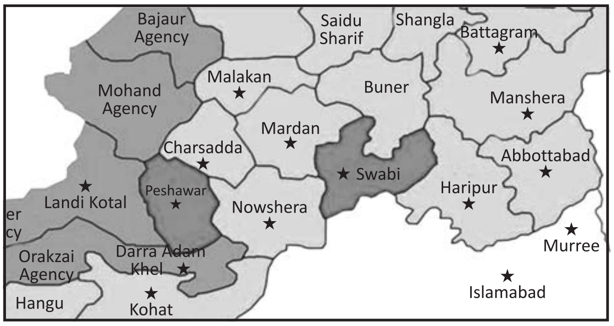

Figure 1. Map showing cities of Swabi and Peshawar

The selected slope is located in an important site and there are located two buildings and 100,000 gallons over-head water tank. The failure of the slope can take hundreds of lives and huge monetary losses. The cross section of this slope and topographic location are shown in Figure 2. This slope was divided into two sub-slopes, herein after would be called slope No. 1 and slope No. 2. The slopes were analyzed for local and global stability. Limit equilibrium methods of slices were used for stability evaluation. Seismic induced forces were modeled using pseudo- static approach. The required input peak ground acceleration (PGA) value was computed from deterministic seismic hazard analysis. Part of the slope was found to require strengthening and was then stabilized with slope stabilizing techniques including retaining wall, soil nails, and ground anchorages. The most feasible technique was proposed for the site.

2. Tectonic setting and regional seismicity

Pakistan is very prone to earthquakes. It lies at the plate boundaries of Arabian, Indian and Eurasian tec-

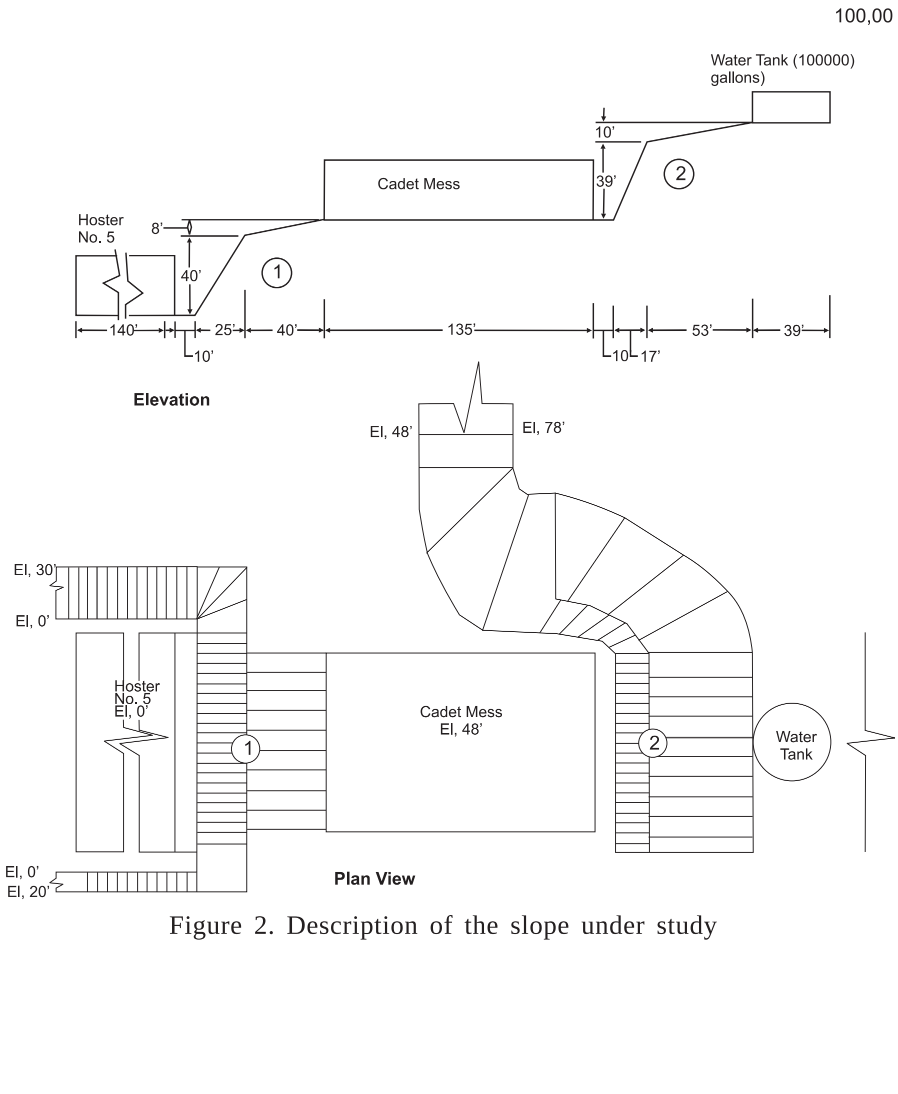

Figure 2. Description of the slope under study

tonic plates. The plate boundary of Indian and Eurasian plates is located in the north of the country while Arabian and Eurasian plate boundary is in the south coast called as Markran subduction zone. The collision of these tectonic plates is a major factor of seismicity in the region. Many devastating earthquakes have occurred in past due to this tectonic activity, such as 7.6 magnitude earthquake of Quetta, 1935, 8.4 magnitude earthquake of Makran Coast, 1945, 7.6 magnitude earthquake of Kashmir, 2005, 7.2 magnitude earthquake of Dalbandine, 2011.

The site under investigation is surrounded by the following faults within one hundred kilometers radius and is shown in Figure 3 and Figure 4.

a. Main Boundary Thrust (MBT)
b. Main Mantle Thrust (MMT)
c. Main Karakorum Thrust (MKT)
d. Raisi Thrust (RT)
e. Himalayan frontal Thrust (HFT)
f. Kalabagh Fault (KF)
g. Jhelum Fault (JF)
h. Punjal Thrust (PT)

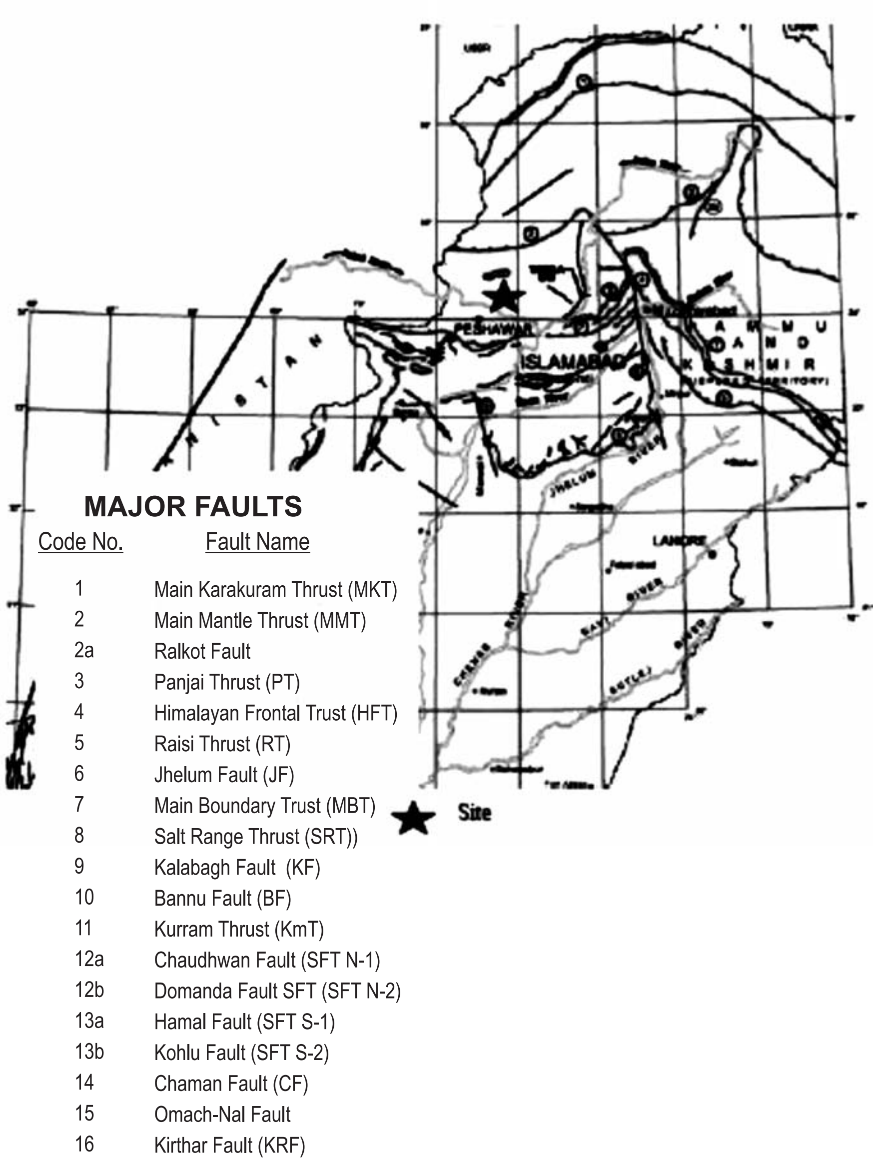

Figure 3: Faulting and seismic sources and recent earthquake localted in the region (Source: Geological Survey of Pakistan)

The area of study is near Tarbela Dam, where a large number of seismographs and strong motion accelerometers are installed. The data collected from this instrumentation shows that about 6000 earthquakes are recorded every year. About 1000 earthquakes have originated within a radius of 160 km of Tarbela Dam. 0.27g PGA is the maximum level of shaking ever observed at the Dam site, which was caused by an Earthquake on May 20, 1996 of magnitude 5.2 and depth 5 Km originated in Topi. The 2005 Kashmir earthquake caused PGA values ranging from 0.1g to 0.16g at different locations of the dam.

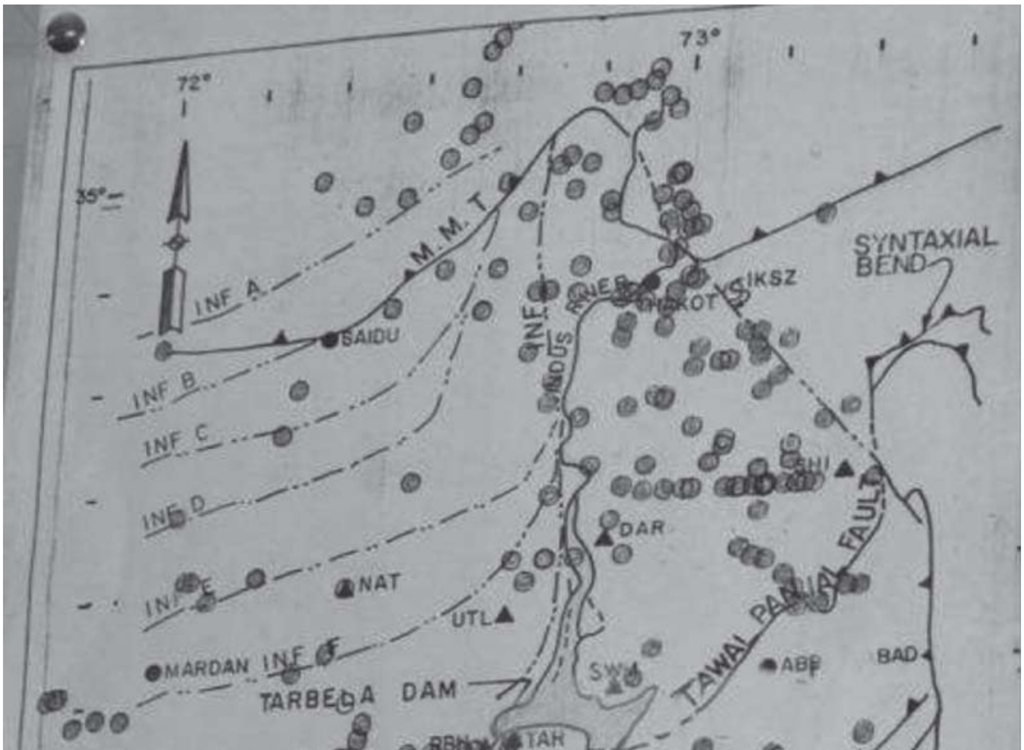

Figure 4. Faulting and seismic sources close to the site (Source: Geological Section of Tarbela Dam Project)

3. SEISMIC HAZARD ANALYSIS

The soil slope was analyzed for expected ground motions due to possible earthquakes, and PGA was selected as the parameter representing the expected ground motions at the site. In order to consider scenario earthquakes at the site, deterministic seismic hazard analysis was performed, and PGA value was estimated. Analysis was performed using Ground Motion Prediction Equation (GMPE) and real time histories were proposed for the site.

4 GROUND MOTION PREDICTION EQUATIONS IN PAKISTAN

Pakistan is a developing country, and amongst other issues in such researches, one essential problem is the choice about the use of appropriate attenuation relations. The researchers, working on seismic hazards, have not developed any predictive relationship for Pakistan due to the lack of strong motion data. They use attenuation relations developed for other regions. However, to the best of the authors' knowledge, none of these studies use recently developed Next Generation Attenuation (NGA) relationships, except for one study which used a single NGA equation.

Researchers have used various equations for seismic hazard analysis[3,4,5]. Seismic hazard map have been developed for Pakistan based on Boore et al. predictive equation[6, 7]. Pakistan Meteorological Department and NOARSAR, Norway jointly worked on seismic hazard analysis and zonation for Azad Jammu and Kashmir, Pakistan[1]. This study is based on Ambraseys et al. attenuation equation[5]. Ahmed et al.[8] used Boore & Atkinson NGA model for developing seismic risk model for Mansehra city, Pakistan.

In this study, five Next Generation Attenuation (NGA) equations and the Ambraseys et al.[5] predictive equation is used.

5. PEAK GROUND ACCELERATION (PGA)

Different distance measures are used in the attenuation relationships. Some attenuation equations consider the shortest distance of the surface projection of the fault to the site called Joyner and Boore distance (Rjb) while others use shortest distance to rupture of the fault (Rrup). These distance measures are given in Table 1. Out of all the faults, MBT produced the highest PGA values for all the predictive equation. Giving equal weights (i.e. 1/6) to all the PGA values calculated from all the attenuation equations used in the study yielded a mean PGA of 0.235 g for the site.

Table 1. Maximum moment magnitude expected

S. No	Name	(M max)	Rjb
1	Main Karakorum Thrust (MKT)	7.9	85.37
2	Main Boundary Thrust (MBT)	8.1	28.80
3	Main Mantle Thrust (MMT)	8.1	36.00
4	Punjal Thrust(PJ)	7.4	29.12
5	Himalayan Frontal Thrust (HFT)	7.8	62.89
6	Raisi Thrust (RT)	7.8	95.70
7	Jhelum Fault	7.2	65.86
8	Kalabagh Fault	7.1	84.83

Using data from Table 1, PGA values are calculated by all the six ground motion prediction equation (GMPE) (attenuation equations) and results are summarized in Table 2.Table 2: Peak Ground Acceleration values

Table 2: Peak Ground Acceleration values

NAME OF GMPE	M.K.T	NMT	PT	H.FT	RS	JF	MBT	KF
Boore & Atkinson NGA	0.019	0.203	0.176	0.118	0.074	0.086	0.230	0.061
Abrahamson & silva NGA	0.093	0.175	0.138	0.105	0.080	0.064	2.210	0.048
Campbell & Bozorgina NGA	0.069	0.160	0.123	0.083	0.060	0.062	0.160	0.048
Chiou & Youngs NGA	0.090	0.214	0.172	0.111	0.074	0.061	0.252	0.042
Idriss NGA	0.072	0.172	0.46	0.090	0.060	0.053	0.214	0.037
Ambraseys et al (2005)	0.144	0.304	0.203	0.161	0.119	0.074	0.346	0.053

6. ANALYSIS PERFORMED

Two types of analysis, slope stability analysis and load deformation analysis are performed.

A) Slope Stability Analysis

Two different slopes, slope 1 and slope 2 were selected for this study. Their loading and drainage conditions are shown in Table 3 and 4. Limit equilibrium method of slices was adopted for slope stability analysis and Morgenstern-Price method of the slice is used in this study. The analysis is performed for both dry and fully saturated drainage conditions. The analysis is performed for gravity loading as well as gravity and seismic loading. To perform slope stability analysis, the slopes are modeled in Slope/W software. In this model, all the surface loads (Building loads) were applied to slope as uniform pressure. Moreover, failure surfaces were assumed as non-circular using inbuilt optimization technique of the software. This software needs the shear strength parameters of the soil. From the observation of sieve analysis results and visual observation of slope material standing in vertical cuts, following soil shear strength parameters are assumed for analysis. Cohesion c=1000 psf, and $\varnothing=35°$. These assumed parameters are verified by considering a vertical cut of 40 feet critical height which gives a factor of safety (FOS) slightly less than 1.0.

The results of analysis sets for slope 1 and slope 2 are given in Tables 3 and 4 respectively.

The values of factor of safety for slope 2 only under gravity load and for fully dry and saturated conditions are well above the value of 1.0 showing that slope 2 would remain stable under gravity loads. However, factor of safety values for slope 1 are above 1.0 for the gravity load and are less than 1 under the combined action of gravity and seismic loads. Factor of safety values less than 1.0 indicate that slope 1 is unstable and needs strengthening.

Table 3: Results for Slope 1 for Different loading and drainage conditions

Loading Combination	Fully Dry Condition	Fully Saturated Condition
Gravity Loading	1.196	1.190
Gravity + Seismic Loading	0.900	0.899

Table 4: Results for Slope 2 for Different loading and drainage conditions

Loading Combination	Fully Dry Condition	Fully Saturated Condition
Gravity Loading	1.466	1.190
Gravity + Seismic Loading	0.091	1.018

Strengthening Techniques

The stability analysis was performed for the slopes without any strengthening measures and with slope strengthening measures. Retaining wall, soil nailing and tiebacks were used as slope strengthening measures. These techniques are as follows:

i. Slope Strengthening with Retaining Wall

The retaining wall was provided at the toe of slopes, which increases the stability of slopes by increasing the slip surface. They are made of materials having different internal angles of friction, unit weight and cohesion values. Trial and error method was used to find most feasible slip surface by changing the stem and base values. Figure 5 shows the failure slip surface with retaining wall. As software packages, that uses limit equilibrium method, provide factor of safety against sliding and the soil material are homogeneous and therefore, Factor of safety for bearing failure is not checked as there is no weak surface at the toe of slope. Checks for other safety parameters are performed. It is observed, by the inclusion of retaining wall that the path followed by slip surface has changed, and factor of safety has increased against global stability of slope. The factor of safety values for various drainage and loading conditions are given in Table 5.

Table 5: Results for Slope 1 for Different loading and drainage conditions

Loading Combination	Fully Dry Condition	Fully Saturated Condition
Gravity Loading	1.287	1.285
Gravity + Seismic Loading	0.171	1.089

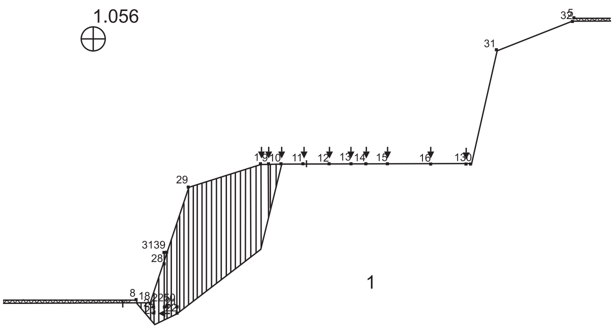

Figure 5. Failure slip surface with retaining wall

ii. Slope Strengthening with Soil Nailing

Soil nailing is a process of reinforcing slope by installing closely spaced steel bars known as nails. The process is effective for broken rock, shale and cohesive soil. The purpose of sail nailing is to improve the factor of safety against failure.

The slope failure slip surface with soil nails is shown Figure 6. The specifications for soil nails that were provided in slope are given below.

Spacing (Horizontal) = 3 ft

Spacing (Vertical) = 6 ft

Iinclination (Nail with horizontal) = 15^o

Grade (Reinforcing steel) = 60 Ksi.

The factor of safety values for various draining and loading conditions are as shown in Table 6.

Table 6: Results for Slope 1 using strengthening with soil nailing

Loading Combination	Fully Dry Condition	Fully Saturated Condition
Gravity Loading	1.329	1.285
Gravity + Seismic Loading	1.172	1.153

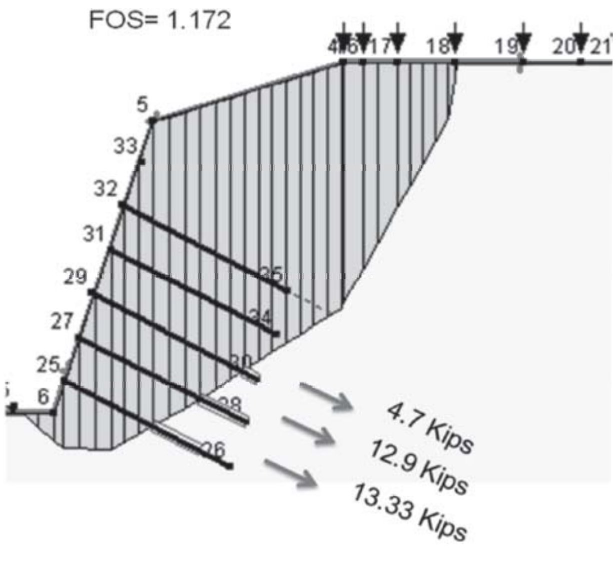

Figure 6. Slip surface with soil nails

iii. Slope Strengthening with Tiebacks (Ground Anchors)

They are made up of pre-stressed reinforcement members which are used to provide stability to slope. They are tied behind the expected location of failure surface which provide stabilization force. In this research, the software package used for analysis of slope is Geo Studio's Slope/W. Safety factor for pull out, steel reinforcement, bound and unbound lengths were specified as these are required by the software. For analysis, the length of anchors was taken as 45 ft of which 25 ft was taken as bound while remaining 20 ft was taken as unbound length.

Table 7 shows factor of safety values for slope strengthening with ground anchors for various conditions. Failure surface with ground anchors is shown in Figure 7.

Table 7: Results for Slope 1 used in slope strengthening with ground anchors

Loading Combination	Fully Dry Condition	Fully Saturated Condition
Gravity Loading	1.583	1.404
Gravity + Seismic Loading	1.166	1.165

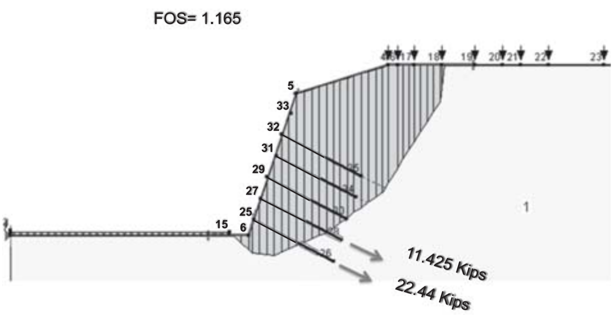

Figure 7. Failure surface with ground anchors

B) Load Deformation Analysis

Load deformation analyses are performed in SIGMA/W software. Since the purpose of the study is to make a comparative study, linear elastic materials are considered in this study.

i. Slope Strengthening with Retaining wall

The components of retaining wall used in the strengthening of soil slope are modeled as beam elements. The point loads are the gravity loads from cadet mess while the normal edge boundary conditions are contact stresses coming from the hostel-5 and over head water tank. Self weight of soil is taken as unit load in the gravity direction whereas pseudo component (horizontal only) is also specified as unit load in the horizontal direction acting away from the slope. All the loading and drainage conditions are modeled, and it is found that gravity and seismic loading combination for saturated condition gives the maximum displacement in the slope (0.264 inches in the horizontal and 0.186 inches in vertical directions). The base and the stem of the retaining wall are taken as beam elements. For analysis of beam elements, E (modulus of elasticity of material) used is 449570000 psf, I (moment of inertia) is 0.1667 ft ^{4} and A (cross-sectional area) used is 2.0 ft^2.

The graphs shown in Figure 8 and Figure 9 represent displacements (in feet) occurring in the slope in X and Y directions respectively.

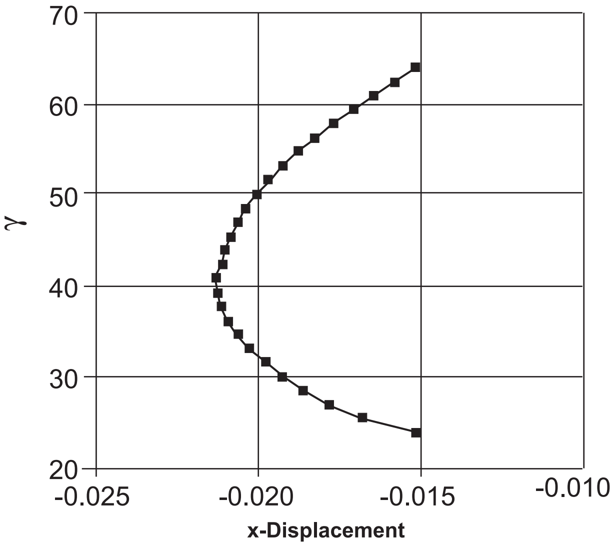

Figure 8. Displacement in horizontal direction in slope with Retaining wall

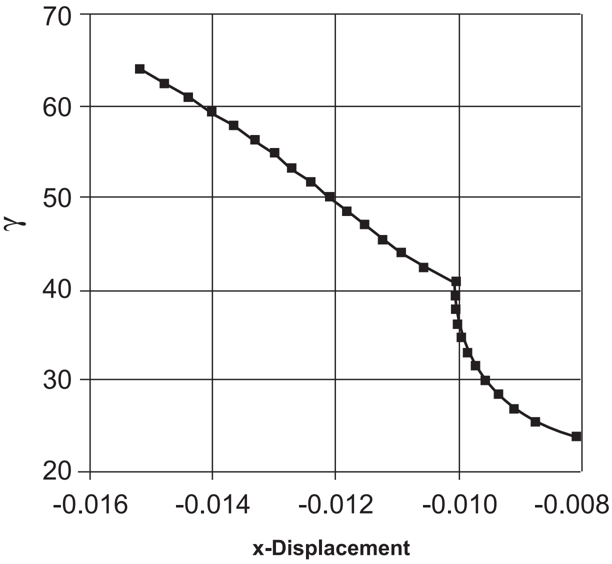

Figure 9. Displacement in vertical direction in slope with Retaining wall

Figures 8 and 9 shows that the maximum X-displacement is 0.264 in and maximum Y-displacement is 0.186 in.

ii. Slope Strengthening with Soil Nailing

Soil nails are modeled as beam elements. For analysis of beam elements, E (modulus of elasticity of material) used is 4176000000 psf, I (moment of inertia) is 0.0061328 ft ^{4} and A (cross-sectional area) used is 0.0054514 ft ^{2}.

The graphs in Figures 10 and 11 show displacement values (in feet) in horizontal and vertical directions respectively in the soil slope.

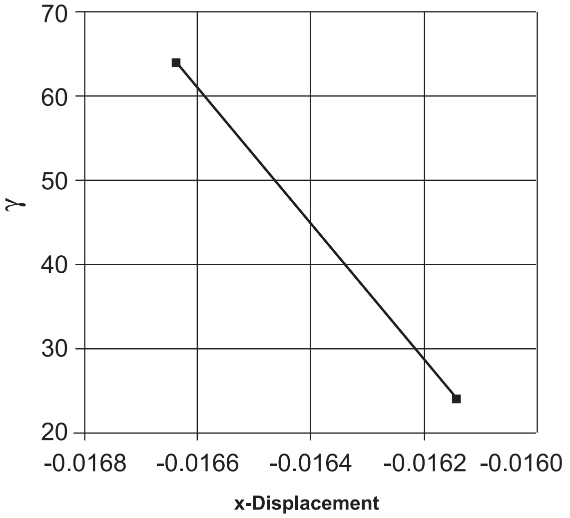

Figure 10. Displacement in horizontal direction in slope with Soil nails

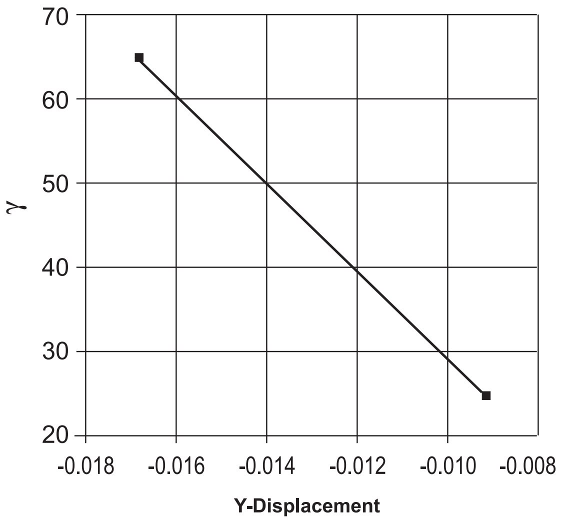

Figure 11. Displacement in vertical direction in slope with soil nails

iii Slope Strengthening with Ground Anchors

Ground anchorages are modeled as beams and steel bars. The bounded portion is modeled as beam whereas unbounded length is modeled as bar. For analysis, E (modulus of elasticity of material) used is 4176000000 psf, I (moment of inertia) is 0.0061328 ft ^{4} and A (cross-sectional area) used is 0.0054514 ft ^{2}.

The graphs in Figure 12 and 13 show displacement values in horizontal and vertical directions respectively in the soil slope.

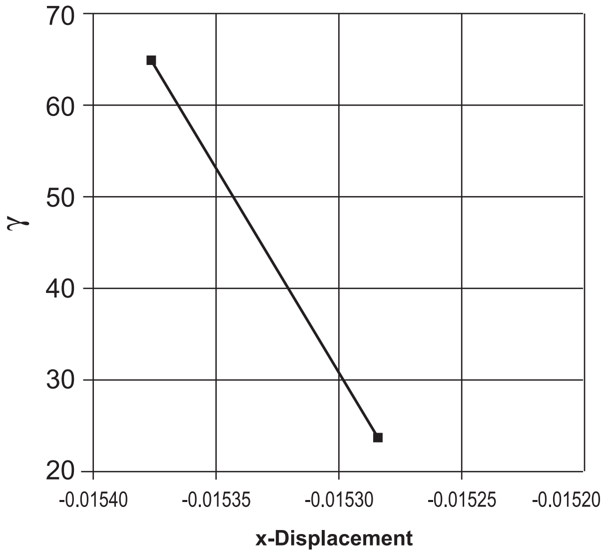

Figure 12. X-Displacement in the slope with ground anchors

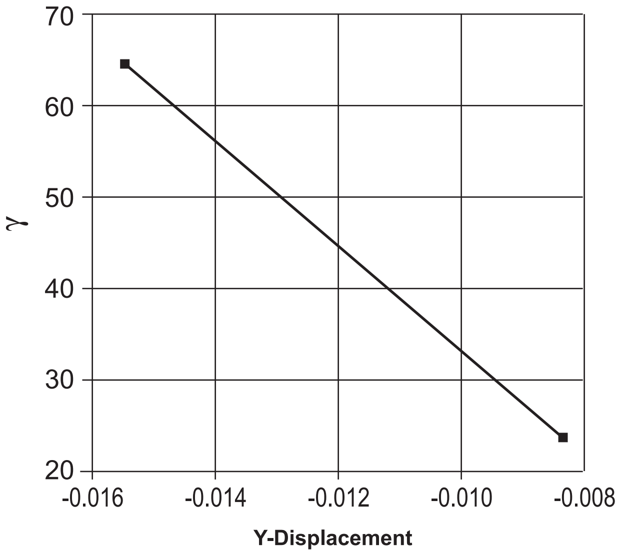

Figure 13. Y-Displacement in the slope with ground anchors

7. RESULTS AND DISCUSSION

The results of the analysis for slope 1 and 2 without any slope strengthening measures are shown in Figure 14 and 15 respectively. The figures show the profiles of the slope and critical failure surfaces. The green color represents the critical area of slip.

For slope 1, the analysis displayed a factor of safety greater than unity for gravity loads while for combined action of gravity and seismic loading, the factor of safety was found as less than unity which indicates that the slope is unstable and needs some strengthening measures.

For slope 2, the analysis displayed a factor of safety greater than unity for all loading and drainage conditions and hence it was considered as stable that does not need any strengthening measures.

The results of stability analysis for slope 1 with retaining wall are shown in Table 5 and Figure 5. The

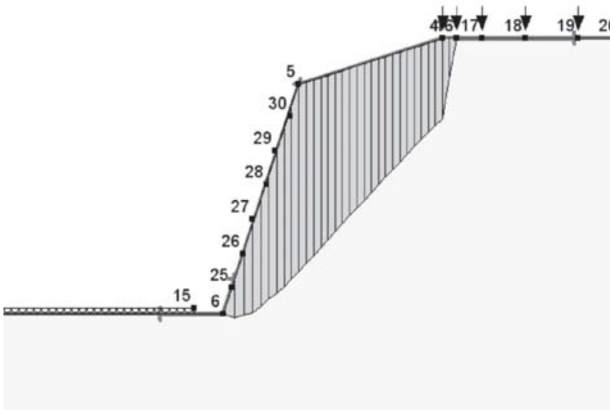

Figure 14. Failure surface of Slope 1

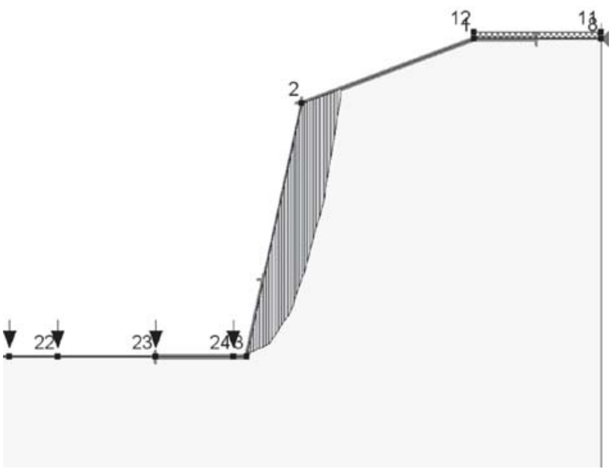

Figure 15. Failure surface of Slope 2

Table 5 gives the values of the factor of safety for different loading and drainage conditions. For all cases, the factor of safety values is quite greater than unity and hence the slope becomes stable.

The factor of safety values and critical failure surface, found by stability analysis for the slope with soil nailing as slope strengthening measures are shown in Table 6 and Figure 6. The factor of safety values for all loading cases is greater than unity and hence the slope is considered as stable.

The factor of safety values and critical failure surface found by stability analysis for the slope with tiebacks as slope strengthening measures are shown in Table 7 and Figure 7 respectively. The factor of safety values are greater than unity for all loading cases and slip surface shows a base failure.

8. CONCLUSIONS

The slopes in general are stable under gravity load case for all drainage conditions. However, under the combination of seismic and gravity loads for fully saturated field conditions, lower slope is unstable having factor of safety value less than 1.0. Peak ground acceleration value of 0.235 g is determined with site specific deterministic seismic hazard assessment using NGA attenuation relationships in conjunction with Ambrassey et al. 2005. This peak ground acceleration value is used as input to the seismic analyses of the slope. Slope is strengthened by considering Retaining wall, Soil nails and Ground anchors. These strengthening techniques showed improvements, but deformation study proved to be the most effective method. Retaining wall showed almost no control over the deformations whereas, soil nails and ground anchors exhibited better control over deformations. The maximum displacement in the soil slopes with retaining wall, soil nails, and ground anchors are 0.0220, 0.01665 and 0.01529 feet respectively.

Practically Soil nails require some displacement for the nails to develop resistance which may cause the building on top of the slope to settle. Whereas, ground anchors impose pre-stress on the slope and do not require any displacement for mobilization of resistance. Therefore, ground anchors are more effective in the present case.

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