Slope Failure Risk Assessment using FEM

Saiful Islam[#1], Zarghaam Rizvi[*2]
[#]Lecturer ,Department of Civil Engineering, King Khalid University,Abha,KSA
[1]saiful.islam.iitr@gmail.com
[*]Research Scholar, University of Kiel,Germany

***Abstract*— Finite element stability analysis of slope stability is carried out. Site investigation (SI) for Geological profile and laboratory testing are performed defining the soil conditions. The slope and the soil beneath are consist of firm to stiff embankment debris, salty clays, limestone's and sandstones. The water table (WT) is observed not to vary too much and lies at the surface of the limestone/sandstone layer. The concept of sliding planes are defined and implemented in the FE analysis. For the considered WT conditions, residual friction angles $\varphi_R = 11°$ and $\varphi_R = 14°$ are respectively evaluated for minimum equilibrium condition. The measured values vary from $\varphi_R = (9 – 13)°$. On these conditions four different engineering measures such as a small embankment at toe, rows of cast-in-place piles, combination of anchors and pile are proposed are analyzed as a measure to stabilize the movement in the slope.**

***Keywords*— *Finite slope stability, Anchor stabilization, FE Analysis, Stabilizing piles, Toe stabilization*Introduction**

I. INTRODUCTION

Slope failure is defined as a downward and outward movement of slope forming materials under the influence of gravity. The phenomena of slope failure described sometimes as landslides but this is neither limited to the "land" nor to "sliding", and usage of the word has implied a much more extensive meaning than its component parts suggest. A lot of scientific investigation has been carried out throughout the years towards slope stability problems. Slope failure risk assessment have drawn worldwide attentions (Sajinkumar et al., 2014) and it is well recognized that the risk of slope failure involves the failure probability and the corresponding consequence. Instead of a single failure mode, past researches have demonstrated that the slope failure is triggered by a finite number of failure modes (i.e., critical slip surfaces or RSS, e.g. Ji et al., 2012; Ji and Low, 2012; Li et al., 2013b, 2014b; Li et al., 2014a; Jiang et al., 2015; Li and Chu, 2015a, b; Chu et al., 2015). Zhang et al. (2011, 2013) have proposed approaches for identifying Representative Slip Surfaces (RSS) which should be incorporated in the determination of slope failure probability The first sign of an imminent finite slope failure is the appearance of surface tension cracks in the upper part of the slope, perpendicular to the direction of the movement. These cracks happen to fill with water, which weakens the soil further and increases the horizontal force that triggers the slide. A finite or infinite slope failure is primarily the result of a shear failure along the boundary of the moving soil or rock mass. Slope Failure calculations indicate that the maximum shear stress occurs at or close to the toe of a slope, the shear strength of the soil is first exceeded at this point and the failure then spreads up the slope.If a slope fails, it is much more probable that the failure has been caused by a gradual decrease in the shear strength of the soil, than by extreme conditions at the time of failure. By adding water to the slope the load increases by substantial amount that percolates in the void of the soil mass.

The shear strength is reduced owing to the increase of the pore water pressures. Water in fact has been implicated as the main controlling factor in most slides. Slope failures can be classified according to their state of activity into: active, dormant, and stabilized. In this paper, stability analysis of a sliding zone has been investigated. Geotechnical investigation and laboratory testing were carried out in the area. The derived soil parameters are applied to the stability calculations. Considering the geometry of the slip zone, the heterogeneity of the soil layers in the slope and extends of the sliding planes; FE modelling is chosen as the most appropriate method for the current landslide stability problem. The analysis is carried outwith PLAXIS FE software. Some engineering measures are proposed such as mechanical stabilization of slope using piles and anchors, which have been implemented to solve the problem. The advantages of FE modelling are that no assumption needs to be made in advance about the shape or location of failure, slice slide forces and their directions. The method can be applied with complex configurations and soil deposits in two or three dimensions to model virtually all types of mechanism. The equilibrium stresses, strain and the associated shear strength in soil mass and in each layer can becalculated accurately. The critical mechanism at failure developed can be extremely general and need not to be simple circular or logarithmic spiral arcs as in case of limit equilibrium analysis.

II. GEOLOGICAL MODEL OF SUB GROUND

A. Geological Condition

The region is having four different kinds of soil layers. Most of these soil layers are either clayey material or limestone. As shown in the Fig. 1 the first layer of dam debris (qhz) is made up of clay/silt, silt and sandy gavel. It also has some traces of gravel and pebbles. This soil layer is soft stiff. The average thickness of this soil layer is 1-4 meters. The layer beneath is not horizontal and is made up of soft material like marl limestone, over the period of time this lime stone is softened and is not too stiff. The average thickness of this layer is more than 15 meters in most of the places. The layers sandwiched in between is the limestone layer, it is made up of weathered limestone, which is the potential region of failure. The thickness of this layer is between 50 cm to 1.5 m.

To test the soil properties, geology and hydrological condition borehole test were performed at locations showed with black dots in the Fig. 2. These bore holes test also helped in finding the subground deformation and rate of movement

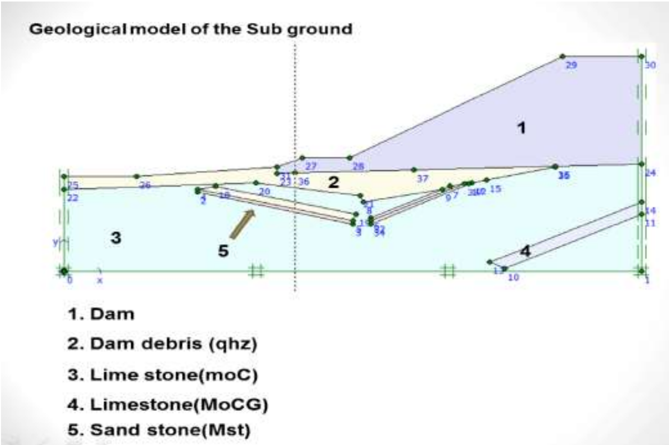

Fig. 1 Geological cross section of the embankment and the subground .

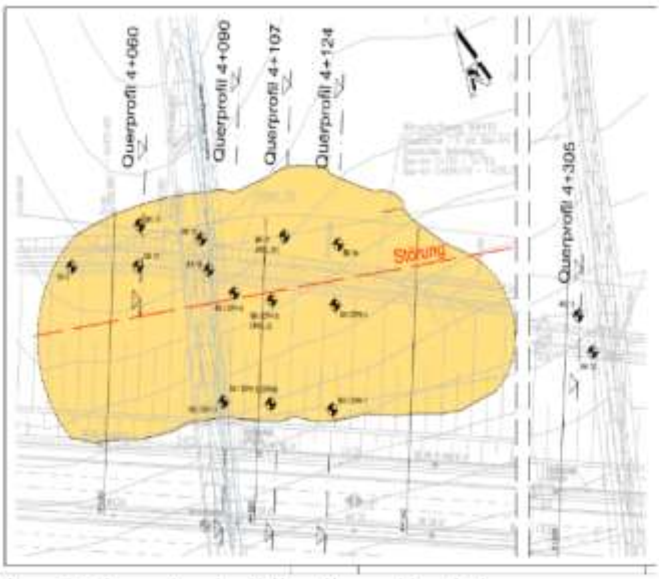

Fig. 2 Figure showing failure line and boreholes.

B. Hydro-Geological Condition

The various effects of running or percolating water are generally recognized as very important in stability problems, but often these effects have not been properly identified. It is a fact that the movement of free water molecules within a soil mass causes seepage forces, which have severe effect than is commonly realized for the stability of slopes.

When there is a drop in the ground water table or of running surface water adjacent to the slope, such as a sudden drawdown of the water surface in a reservoir there is a loss in the buoyancy of the soil mass, which is in effect an increase in the weight. This additional weight causes increase in the shearing stresses, which may or may not be in part counteracted by the increase in internal resistance (shearing strength) developed in the soil mass, depending upon whether or not the soil mass is able to undergo compression, which the additional weight increase tends to cause. If a soil mass is large and saturated with low permeability, practically there is no space to be compacted due to soil weight and therefore no volume changes will occur except gradually, and in spite of the increase of load the soil strength increase may be measurable

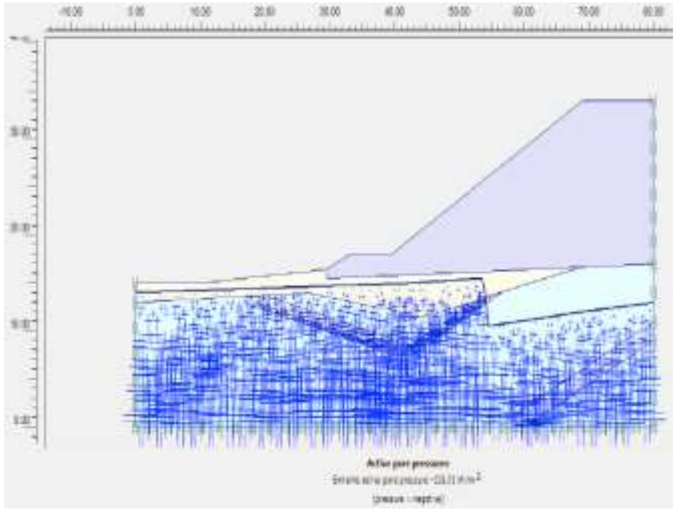

Fig 3 Figure depicting the ground water condition and pore water pressure.

Shear strength in a confined soil mass may be accompanied by a drop in the inter-granular pressure and an elevation in the neutral pressure. A slope failure may result by such a condition in which the entire soil mass undergoes into a state of liquefaction and flows like a liquid. A failure situation of this type may develop if the soil mass is subject to time dependent loading, for example, due to earthquake load machine vibration. At the site of slope failure it is found from the borehole tests that the average fluctuation of the ground water table is 1.55 m during dry and wet months, which is not sizeable, and the finite element.

C. Classification and Characteristics of Soil Layers

As previously explained in the geological conditions of the region most of the material in the area is limestone, clayey silt and some gravels. The soil tests are performed to obtain the soil properties. For modeling purpose the material is modeled as Mohr Coulomb. The following table gives the values obtained from standard laboratory test and standard penetration tests done near the vicinity of the failure zone as shown in Fig. 2.

III. FINITE ELEMENT MODELLING OF SLOPE

The Finite element analysis is carried out with PLAXIS. It is important for the slope stability to model the section, which is the most critical. Twodimensional (2D), plane strain FE modeling is carried out as shown in Fig. 4 where the dimensions are in analysis is carried out based on high water table, which reflects high risk of failure. The Fig.3 shows the water table at various locations. The pore water pressure is generated using the PLAXIS program applying confined boundary conditions at meters giving the real geometry of the considered most critical section.

Fixed boundary conditions are applied at the bottom of the FE model considering no deformations to occur at that depth of Layer. On the left side the model is terminated at the start of the Layer and on the right side at the lowest quote assuming zero horizontal deformations to occur. This might be rather optimistic considering the erosion effect may be due to runoff cuts from the embankment. The traffic load is modeled as uniformly distributed vertical load. The dynamic effects from the traffic or seismic activity are not taken into account, in the current finite element analysis. The soil layers and the soil parameters are employed as referring to the data given in Table 1. The deformation parameters, such as deformation modules are determined on the basis of general experience. The Mohr Coulomb elasto-plastic constitutive soil model is applied for the gravel/sand in undrained conditions. The 15-node triangular plane strain FE elements are applied. The model is composed of 703 elements, 5435 nodes and the stresses are calculated at 7908 stress points. Average FE size is 1.79 m.

Table 1: Table for soil properties of the Embankment and subground.

Mohr-Coulomb		1 Dam	2 qhz	3 moC	4 moCG	5 Mst
Type		Undrained	Undrained	Undrained	Undrained	Undrained
g_{unsat}	[kN/m³]	18.00	17.00	19.00	20.00	19.00
g_{sat}	[kN/m³]	18.00	21.00	23.00	24.00	22.00
e_{init}	[-]	0.500	0.500	0.500	0.500	0.500
c_k	[-]	1E15	1E15	1E15	1E15	1E15
E_{ref}	[kN/m²]	2000.000	2000.000	700.000	10000.000	20000.000
n	[-]	0.350	0.330	0.035	0.032	0.031
G_{ref}	[kN/m²]	740.741	751.880	338.164	4844.961	9699.321
E_{oed}	[kN/m²]	3209.877	2963.291	701.782	10021.202	20039.749
c_{ref}	[kN/m²]	5.00	5.00	20.00	200.00	0.50
j	[°]	22.50	22.50	22.50	40.00	17.00
$R_{inter.}$	[-]	1.00	0.7	0.7	0.7	1.00

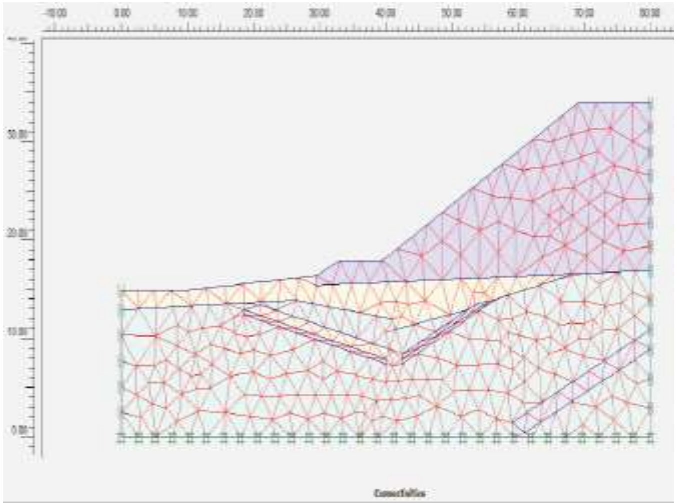

Fig. 4

A. Slip planes Modelling using interface Element

The intermediate sliding planes are modeled by interface elements. At first, the calculations were done to derive the interface strength parameters as a function of the parameters of the soil layers. Based on the parameter calculated a coefficient R was applied to the friction angle and the cohesion, or undrained shear strength of the surrounding layers. One interface element was applied at the contact between the two layers, which took over the soil strength parameters of the corresponding layer reduced by the coefficient R= 0.75. The value of R was derived from the ratio of residual friction angle (at the sliding plane) to the layer friction angle. This method was giving rather favorable results, as except the residual friction angle, cohesion was applied at the interface as well.

Alternatively and more realistically, the interface elements are modeled as independent materials with residual friction angle as given in Table 1 and zero cohesion. The "virtual thickness" of the interface, which is an arbitrary dimension, is calculated as a function of the average FE dimensions multiplied by the virtual thickness factor, which is taken equal to 0.09. This means that the virtual thickness or the width of the sliding planes or fractures is about 2.0 m*Q. = 0.18 m. This thickness is smaller where the FE mesh is denser or otherwise. Interface elements are connected to the soil elements. As the applied soil elements are 15-noded. 5 pairs of nodes define the interface elements. The coordinates of each node pairs are identical.

IV. RESULT

The stability of the slope is analyzed and it is found that the slope has a great chance of failure. The appendix A advocates for some engineering measures to be taken to prevent future failures. The total incremental displacement is found to be 356 mm.

A. Slope stabilization with one anchor and three piles.

The first attempt to stabilize the slope is taken up by driving three piles and one anchor. The piles are place with one pile in the embankment and two piles near the toe. The anchor is place in the slope to prevent the failure. It is found that this scheme reduce the incremental displacement to 192 mm. The diameter of the piles used is 1.09 m. For group of piles, they are separated by a distance of 5D. The thrust resistance of the anchor in either tension or compression is $1x10^{15}$ kN. The grout body of anchor is having the normal stiffness of $1x10^{15}$ kN/m.

B. Slope stabilization with two piles in dam And two piles at toe

The other scheme adopted to stabilize the embankment is to drive two piles in the embankment at the weak sliding zone and two piles at the other weak layer near the toe. The piles in the group are having a diameter of 1.10 m and are separated by 5D apart. A traffic load of 160 kN/m is applied at the top of the embankment to observe the effects to additional dead load on embankment. After analysis it is found that load is not causing any serious damage to the stability of the slope. Therefore in some analysis it is not applied. Total incremental displacement in this case is found to be 72 mm.

C. Slope stabilization with three piles in embankment and one pile at toe.

Three piles are driven in the embankment slope at a distance of 5D. The fourth pile is driven in the second weak zone near the toe. The length of the longest pile is 21 m and the shortest one in the slope is 19 m with all the piles having same diameter of 1.10 m. The incremental displacement is found to be 127 mm in this arrangement of piles. As visible in the incremental strain diagram the piles are failing to give stability as the shear strain is too high for the piles to stabilize.

D. Slope stabilization with one additional embankment at toe.

One more economical solution is proposed to build a small embankment at the toe of the embankment. It is assumed that the extra weight of the small embankment will help to stabilize the slope. The material for this analysis is assumed to be the same as for the embankment debris. A consolidation period of 200 days is given before the c/phi reduction analysis.

It is found that the total incremental displacement is limited to a value of 131 mm, which is comparable that some case assumed in the analysis. It will be better if geogrids are inserted in this secondary embankment and the process of benching shall be implied for constructing this secondary embankment.

V. DISCUSSIONS

As shown above in the different analysis, for each possible case that has been chosen for the stabilizing the slope, the best possible way is to drive two piles in the embankment slope at the weak layer zone and two piles in the toe weak layer of weathered limestone. The analysis contradicts with the general practice of stabilization of slope, which suggests driving three piles in the weak sliding zone and one at the toe. The discrepancy in the result arises due to the fact that manual or limit equilibrium analysis is greatly affected by the choice of failure surface. Also, the material model can be a source of discrepancy due to the fact that in the vicinity of the pile the soil doesn't behave as predicted by Mohr-Coulomb model.

As evident from the analysis tabulated in Table 2 the best solution for this embankment problem is this case with two piles in embankment slope and two piles near the toe in weak limestone layer.

Table 2: Table showing incremental displacement for different scenarios.

Possile stabilizing measures	*one anchor in embankment and three piles*	*two piles in embankment and two piles at toe*	*three piles in embankment and one piles*	*one additional embankment at toe.*
Incremental displacement (mm)	192	72	127	131

VI. CONCLUSION

To stabilize the sliding zone additional engineering measures are necessary such as, lowering the water table by means of Drainage system and building drain chutes at the sliding toe to protect the slope from the erosion. FE analysis has shown that the stability of the sliding zone is not affected greatly by fluctuation in the ground water table. Four rows of concrete cast-in-place piles of D=1.10 m and 5D spacing, long enough to reach bottom layer and incaster minimum 10 m at this layer, are applied .In combination with the drainage system, this maintains the water table at the measured level. Plate elements are employed to model the row of piles in the out-of-plane direction taking into account the pile dimensions, material and pile spacing. The piles behave elastically. The 3D effect can be investigated in a further investigation considering 3D FE modeling. Other methods, (geometrical, hydrological, chemical, and mechanical) can be used to correct the slope failure. Mechanical method, in which the shear strength of the sliding mass is increased, is currently chosen. Stabilizing cast-in-place concrete piles, which have a number of advantages over driven piles, are particularly investigated. For these pile types problems associated with vibration and the remolding of the soil are greatly reduced. The piles applied in such cases are of 1.10 m in diameter and the spacing varies from 5D to 9D.The solution that came out from this finite element analysis is to cast two piles in the sliding weak zone in the slope and two piles in the weak zone near the toe to mechanically stabilize the embankment slope.

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