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Slope Stability Evaluation of a Failed Slope in Electric Sub-Station Site at Reckong Pio, Himachal Pradesh, India

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Abstract: *The Himachal Pradesh Power Transmission Corporation Limited (HPPTCL) has constructed two buildings for housing 660kV and 220kV sub-stations close to Reckong Pio town in Himachal Pradesh, India. These buildings were constructed on two terraces constituted of glacial moraine deposits. A major landslide occurred during the rains of 2013 with the scarp face of landslide being located close to the building site. After geotechnical studies, the slope was treated adequately with grouted anchors and other required measures. The paper discusses the stability of the slopes in natural condition as well as after taking into consideration the effects of control measures.*

I. Introduction

Site selection for various civil structures in hilly areas is an important issue. Civil construction involves cutting of hills and preparation of terraces. In natural condition, the slope may be stable and in equilibrium but if it is cut during construction with steep slope, it tends to become unstable and has to attain a new equilibrium for which suitable measures need to be implemented. In order to construct civil structures, cut slopes are made, but the stability aspects are often not considered, which may cause instability of the slopes leading to landslides. If such a landslide occurs close to the civil structures constructed on the terrace, the overall stability of the civil structure becomes a question, as the landslide may progress further up and affect the foundation of the structure. Hence, a proper investigation in the initial stages of the project may help to understand the status of stability, which may help to adopt suitable control measures.

In this context, the terraces developed for housing a 66kV and 220kV sub-stations near Reckong Pio had caused a major landslide in the area close to the sub-station building. The present paper attempts to evaluate the stability problems of the landslide and to evolve suitable control measures. The efficacy of the control measures has also been analyzed so as to understand the final status of stability after implementation.

II. Area of Study

Reckong Peo, which is the headquarter of Kinnaur district, is located at El ±2670m and the Sutlej river flows below it at El ±1,960m. The present study area (El ±2500m) is located about 7km to the north of Reckong Pio town and is located over glacial moraine deposits. A narrow single lane road gives access to the site from the township. In view of its location in Higher Himalayan terrain, the area receives excessive snow fall during winters and sometimes receives concentrated rain fall during rainy season. The excessive rains in 2013 had resulted in a massive landslide of moraine deposits extending for more than 40m. The crown of the landslide is located about 20m from the foundation of the building of 220kV sub-station. In view of close proximity of landslide, it is necessary to stabilize the slope below the building so that further progress of the landslide can be stopped.

III. Geological Setting

This glacial debris seen at the construction site mainly consists of non-cohesive gravels and sand materials mixed with silt fractions. Angular big boulders are seen occasionally at places. These materials have been cut into two terraces, which houses the buildings. These materials have good amount of porosity and vertical permeability. The Kunzam La Formation of Haimanta Group of Cambrian age, made up of dolomite, sandstone, quartzite, shale/slate, greenish grey siltstone and local pebble beds are exposed in the vicinity of Reckong Pio (Chakrabarty, 2012). However, gneissic rocks of Higher Himalayan terrain are exposed just above the project site forming a steep slope. Thick glacial moraines are exclusively present within the project area.

IV. Site Observations

Two terraces have been developed below the PWD road in order to house two substation buildings – 66kV substation on upper and 220kV substation on lower terraces. During construction, the adjoining slope failed with rotational failure (Anbalagan 2007) due to sustained rainfall in 2013 leaving a gap of just 20m away from the lower sub-station building. If this landslide further progresses during heavy rains, these may lead to substantial damage to the foundation of the sub-station building. The landslide extends for a height of 40m below the crown with an average width of about 25m. Near the crown area, the slope of the landslide is steep (about 65º) and gradually decreases to about 45º in lower levels. The landslide extends up to the PWD road, which is present at a height of about 40m below. If the landslide is not treated, any boulder detached from the head of landslide may roll down and cause damages at road level.

Since a stream course is present in the centre of the landslide area, there may be more erosion and instability during rainfall in addition to creating difficulties to implement control measures. In order to understand the nature of instability and to evolve appropriate control measures, detailed geological mapping and stability analysis were carried out. A geological map (Fig 1) of the area showing various civil structures, extent of landslide and the type of material exposed area have been prepared on a detailed topographical map of scale 1:1000. A section (Fig 2) across the landslide has also been prepared and this has been used for stability analysis.

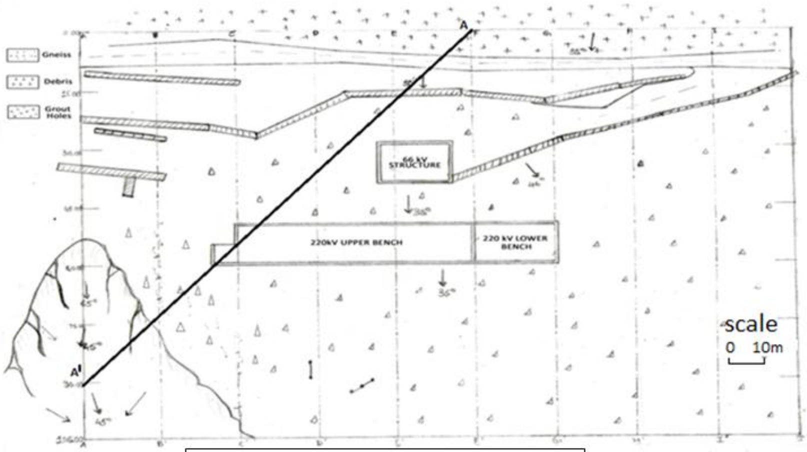

Fig. 1 Plan of area of study

V. Geotechnical Studies

5.1 Mechanical Sieve Analysis

The soil sample of debris materials were collected after removing the boulders. The mechanical sieve analysis carried out indicates that coefficient of uniformity (C_u) and coefficient of curvature (C_c) are 112.5 and 0.39 respectively. The slope material has been classified as poorly graded gravel (GP) based on uniformity coefficient as determined from semi-log graph. On mixing with water, no thread could be formed, which implies non-plastic nature of the sample.

5.2 Light Compaction Test

This laboratory test was performed for a specified compaction effort, to find out the relation between dry density and moisture content of the soil. Both the parameters were plotted (Fig. 3) in order to find the optimum moisture content. From the curve, the values of maximum dry density and optimum moisture content are obtained as 1.893 g/cm3 and 13.26% respectively.

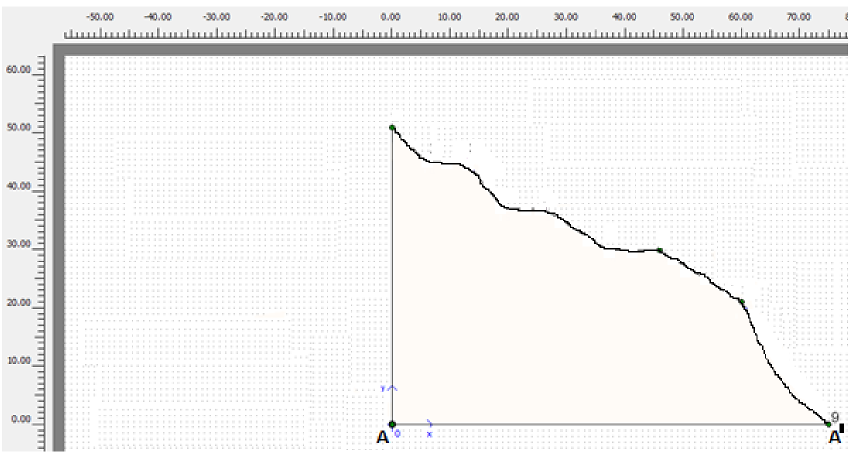

Fig. 2 Geometry of the cross-section drawn on PLAXIS

5.3 Direct Shear Test
In order to carry out direct shear test, samples devoid of boulders were collected and the tests were carried out in dry condition. From the test the following values have been obtained.
- Cohesion - 73.9 kN/m^2 and Internal angle of friction - 22.11°.

5.4 Triaxial Test
The triaxial test on the soil sample under consolidated drained condition was carried out. Based on the test, the Mohr's circles at failure were plotted and the results indicate the following values
- cohesion - 100 kN/m^2 and internal angle of friction - 22°.

From the stress-strain curve, the values of Young's Modulus has also been derived, which is 2700 kN/m^2.

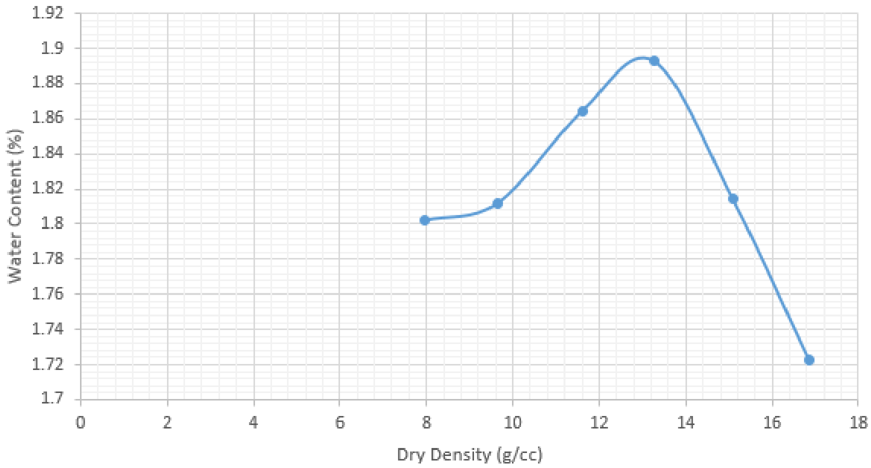

Fig. 3 Curve between Dry density and Water Content in percent

VI. Stability Analysis
The software PLAXIS was used for simulation of both dry and saturated conditions of stability. The software PLAXIS is a finite element program for geotechnical applications, in which soil models are used to simulate the soil behavior. The properties used for this slope are elasto-plastic in nature and hence, the material properties are based on Mohr-coulomb yield/strength criterion. Mohr-Coulomb model is used as a first approximation of soil behaviour in general (Brinkgreve. RBJ, 2001). The model involves the following parameters, namely the dilatancy angle (ψ), the friction angle (ϕ), the cohesion (c), Poisson's ratio (ν) and Young's modulus (*E*).

6.1 Slope stability analysis in dry and saturated conditions - without reinforcement
The slope stability analysis has been carried out using PLAXIS for dry as well as saturated conditions using the material properties indicated in Table 1.

Table 1. Material Properties used for analysis

Parameter	Name	Moraines (dry)
Material Model	Model	Mohr-Coulomb
Type of material behaviour	Type	Drained
Soil unit weight	ϒ	17
Young's modulus	E	7.2×10^5
Poisson's ratio	V	0.3
Cohesion	C	100
Friction angle	ϕ	36°
Dilatancy angle	Ψ	2°

Initially, simulations have been done to find out the displacement behavior in both horizontal and vertical directions.

6.1.1) Horizontal displacement
From the Fig 4, the contours indicate the varying values of horizontal displacement in different locations of the cross-section. The values at the critical points are mentioned in the Table 2 below.

Table 2. Horizontal displacement observed from contours corresponding to these points

Points	4	5	6	7	8
Displacement (m)	14	15	16	17	19

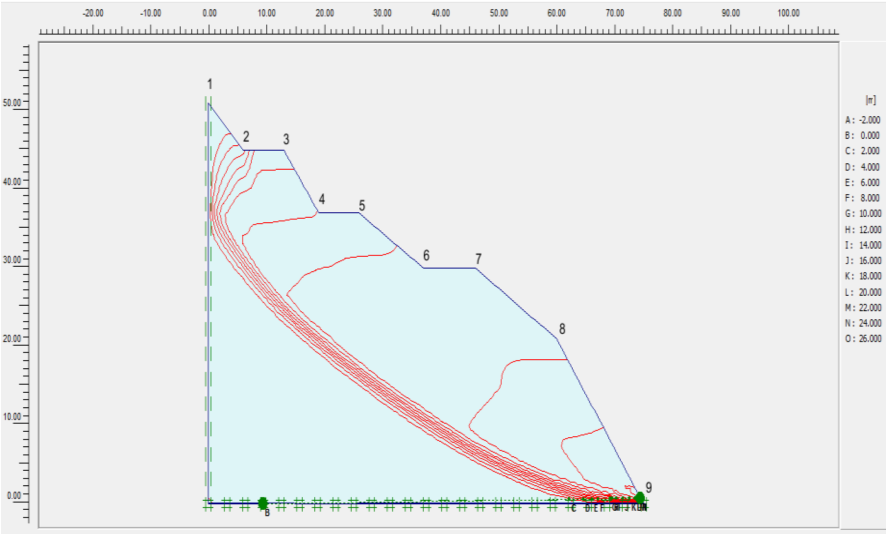

Fig 4. Horizontal displacement contours for dry slope mass without reinforcement

6.1.2) Vertical displacement

From the Fig 5, the contours indicate the varying values of vertical displacement in different locations of the cross-section. The values at the critical points are mentioned in the Table 3 below.

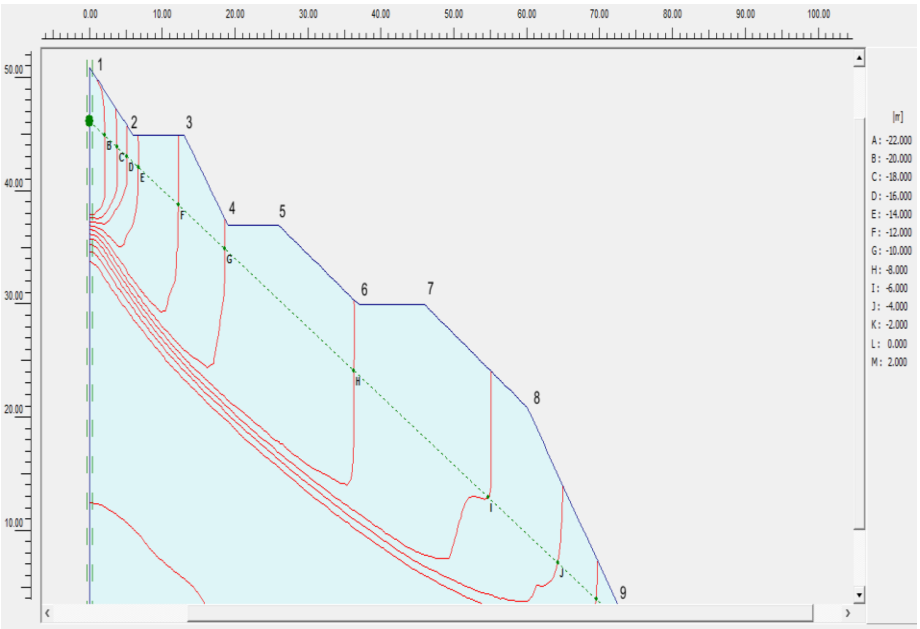

Fig 5 Vertical displacement contours for dry slope mass without reinforcement

Table 3. Vertical displacement observed from contours corresponding to these points

Points	4	5	6	7	8
Displacement (m)	-10	-9	-8	-7	-5

6.1.3 Calculation of FOS

The factor of safety under dry condition using the section given in Fig 2 has been calculated. The material properties indicated in Table 1 have been used for the analysis. The analysis has been carried out for dry as well as saturated conditions. The analysis indicates that the factor of safety for dry condition without reinforcement is 1.57 and hence the slope is stable. However, the factor of safety in saturated condition without reinforcement is 0.92. This indicates that the slope becomes unstable under saturated condition.

VII. Control Measures

Based on observations and investigations at the site, the following control measures are implementing for stabilization of the slope above the crown of the existing landslide.

7.1 Stabilization of slope above crown of the existing landslide

The near vertical landslide scar seen on the south-western side of lower terrace is visibly unstable. The scarp faces is very steep (>65º) close to the crown of the landslide and then gradually flattens further down. This area forms almost the crown portion of a stream, which flows further downstream towards southeast. The unstable area lies about 20m from the edge of 220 kV building. It is essential that this unstable zone below should not progress further up so that the stability of 220 kV terrace is not adversely affected. For that purpose, the following measures had been implemented.

1. A series of NX size (55 mm) drill holes running up to a depth of 20m have been drilled into the slope mass at right angles to the slope surface. The holes were spaced at 2m c/c in either direction and staggered. The bottom most row starts at about 5m away from the landslide scarp face. Similarly, five rows of holes were drilled above the crown area of the landslide between 220 kV building and the unstable zone.
2. The perforated GI pipes of 30mm diameter were inserted into the hole and grouted using cement-sand slurry. These pipes project out of the slope surface by about 100mm-150mm. After grouting, the projecting ends of the pipes were tied to each other by angle sections through welding.
3. The drill holes were drilled using pneumatic drilling machine, so that no water was used for drilling purpose.

7.2 Stabilization of active landslide

The near vertical landslide scarp face present in the crown area of the stream needs to be treated so that its shear strength improves and the erosion is minimised. This area had been treated using the following measures.

1. A series of 40mm dia holes were drilled nearly perpendicular to the steep slope face for a depth of 5m at 1.0m-1.5m spacing in either direction in a staggered pattern. These holes were inserted with 25mm dia steel bars. These steel anchors were hammered into the soil for a depth of about a meter. Later these holes filled with cement water slurry.
2. The anchor heads were provided with 20cm x 20cm bearing plates by welding. These measures were extended for a depth about 25m from the top of the landslide.
3. In order to prevent erosion and increase stabilization of the entire scarp face, a layer of polymer mesh was spread on the entire slope from the crown to a depth of about 50m below. This wire mesh was tied to the projecting anchor heads on the top 25m. Further down, the wire mesh was stitched to the ground with the help of 2m deep dry anchors.
4. Presently, bio-stabilisation measures by means of suitable grass/ plants on the entire surface including the barren stream slopes are under way by spraying seeds on the slope.

VIII. Slope Stability Analysis in Dry And Saturated Conditions –With Rein forcement

In order to improve the stability of the slope, the major treatment carried out at the site includes a series of 20m deep grouted anchors, which form a subsurface barrier preventing the progress of the landslide towards upslope. In order to estimate the efficacy of the treatment, it is essential to carry out stability analysis with the reinforcements in place under dry and saturated conditions. For that purpose, the PLAXIS software has been used with the same material properties. The properties of the grout used in the analysis are given in the Table 4. In addition, the properties of the anchor rod and the subsurface RC wall are given in the Tables 5 & 6. Since a subsurface reinforce wall been created just above the crown of the landslide, the analysis has been done considering this as a RC wall.

Table 4. Grout properties used for analysis in dry and saturated conditions

Parameter	Name	Grouted material (Dry condition)	Grouted material (wet condition)	Unit
Material Model	Model	Mohr-Coulomb	Mohr-Coulomb	-
Type of material behaviour	Type	Drained	Drained	-
Soil unit weight	Υ	25	26	kN/m3
Young's modulus	E	2.1×10^7	2.1×10^7	kN/m2
Poisson's ratio	V	0.2	0.2	
Cohesion	C	12000	10800	kN/m2
Friction angle	ϕ	45°	40°	°
Dilatancy angle	Ψ	2°	2°	°

Table 5. Anchor rod properties

Parameter	Name	Value	Unit
Type of behaviour	Material Type	Elasto-plastic	-
Normal stiffness	EA	36926.4	Kn
Spacing out of plane	Ls	2.5	M
Maximum force	F	1.10^15	kN

Table 6. RC wall properties

Parameter	Name	Value	Unit
Normal stiffness	EA	1.05×10^7	kN
Flexural rigidity	EI	2.1875×10^6	kNm2/m
Equivalent thickness	D	2.5, 0.5	M
Weight	W	62.5, 12.5	kN/m/m
Poisson's ratio	V	0.2	-

The software indicates the extent of horizontal and vertical displacements after incorporating the effects reinforcements within the slope for dry condition. The points 4-8 seen in the slope geometry (Fig. 6) has been used for further displacement analysis. These are critical points in the analysis and the values of displacement are critical in determining the overall safety. The horizontal displacements deciphered from the figure are given in the Table 7. The vertical displacements have also been understood from Fig. 7 and are included in Table 7.

Horizontal displacement

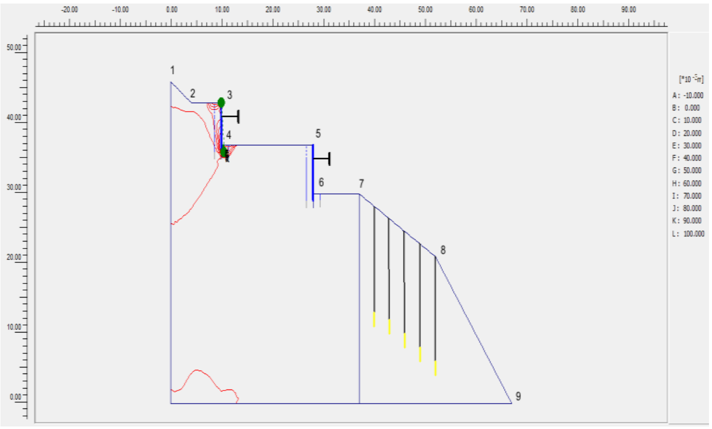

Fig 6 Horizontal displacement contours for dry slope mass with reinforcement

Table 7. Horizontal and Vertical displacements observed from contours corresponding to these points in dry condition with reinforcement

Horizontal	Point	4	5	6	7	8
	Displacement (m)	0.07	0	0	0	0
Vertical	Point	4	5	6	7	8
	Displacement (m)	-0.2	-0.04	-0.04	-0.04	-0.04

Vertical displacement

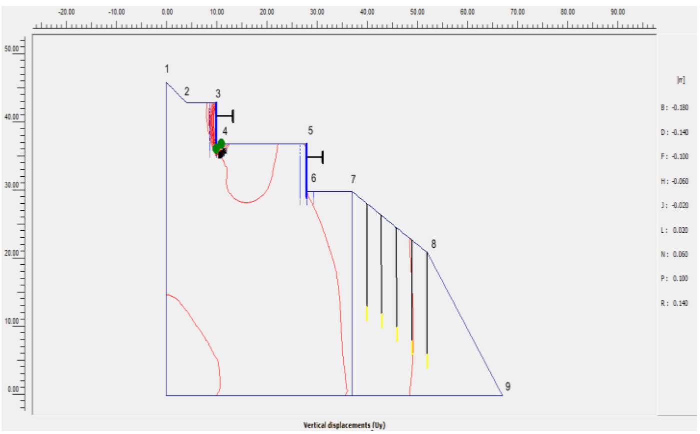

Fig 7 Vertical displacement contours for dry slope mass with reinforcement

Similarly, simulation has been done for saturated condition of the slope with reinforcement to find out the displacement behavior. Further, FOS values have been calculated for both conditions namely dry and saturated with reinforcements.

Factor of safety in dry condition with reinforcement is 2.7

Factor of safety for saturated slope mass with reinforcement is 2.31

IX. Summary and Conclusions

The Himachal Pradesh Power Transmission Corporation Limited (HPPTCL) has constructed two buildings for housing 660kV and 220kV sub-stations close to Reckong Pio town in Himachal Pradesh. These buildings were constructed on two terraces constituted of glacial moraine deposits. The moraine deposits are basically poorly graded gravely sand (GP) which has high vertical permeability.During the heavy rains of 2013, a part of the hill slope failed close to the foundation of lower most building that is 220kV sub-station. The landslide had the tendency to progress upslope particularly during rains. Since a distance of only 20m had been left between the crown of the landslide and foundation of the building, it was essential to stabilize the unstable slope in order to save the building.

The stability analysis of the unstable slopes indicates that the slope is stable under dry condition (FOS - 1.57) but it is unstable under saturated condition (FOS - 0.92).Taking into consideration, the topography and the materials exposed in the area as well as the nature of the landslide materials, a series of grouted anchors extending to a depth of 20m were provided in five rows above the crown of the landslide. This subsurface wall effectively cuts off the progressive tendency of the landslide. Moreover, the loose debris materials seen on the landslide surface were protected with the help of polymer wire mesh layers which were stitched to the ground by means of shallow grouted anchors.The stability analysis was again carried out incorporating the effects of control measures. The value of FOS for the slope under dry condition with reinforcements in place is 2.7 and the value of FOS under saturated condition is 2.31. The analysis clearly indicates that the slopes are stable under dry and saturated conditions with reinforcements in place.

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