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A study of landslide hazards in North Eastern India

Une étude des hasards de glissement de terrain en Nord-Est Inde

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ABSTRACT : The paper discusses the landslide occurred during the period of heavy rainfall and after an earthquake shock on 7th September, 1987 on the north side of a hillock endangering the oil wells. To analyse the problem geotechnical investigations were carried out at field and laboratory. Stability analysis (back analysis) for slopes was carried out using the Bishop's equation. It was found from the stability analysis that the upper part of the hill-slope sections considered was unstable. Therefore, remedial measures were necessary to protect the wells. Remedial measures were suggested as surface pacca drains on the top and the side of the hillock to remove the run off quickly. The dimensions and spacing of the drains were calculated based on rainfall data. Crated masonry and RCC retaining walls on timber/steel pile groups were also suggested at suitable places. The measures so provided were proved to be effective.

1. INTRODUCTION :

The North Eastern Region of India consisting of the states of Assam, Arunachal Pradesh, Nagaland, Meghalaya, Manipur, Mizoram & Tripura bounded by latitude 22° N to 29° N and longitude 90°E to 97°E is mostly hilly terrain, except the Brahmaputra and the Barak Valleys and lies at 150 to 500 meters above mean sea level (msl) at Karachi. The region experiences several landslide hazards specially during moonsoon which are triggered by frequent occurrence of earthquake as the region is highly seismic.

Some of the landslides worth mentioning are one each at Guwahati and Silchar, Assam; three in Nagaland (Rangapahar Military Cantonment Area threatening a Cinema Hall ; Nagaland Pulp and Paper Co. Area, Tuli and Oil and Natural Gas Company Ltd. Oil well area, Borhola) and many more in Arunachal Pradesh. The author was involved in investigating all the three landslides in Nagaland.

The details of the landslide occurred in Borhola is presented in this paper.

2. CHARACTERISTICS OF LANDSLIDE AND SELECTION OF CRITICAL SLOPE :

The landslide occurred in a hillock having three oil bearing wells of Oil and Natural Gas Commission (ONGC), North Eastern Zone, India during a heavy rainfall and simultaneous occurrence of an earthquake, epicentred at Kaziranga, a National Park about 120 km from this hillock towards North West direction (Fig1). The magnitude of earthquake shock was medium (about 5.0 in Richter scale). At first the sliding occurred at foundation soil of P.C.C water tank measuring 6.1 m x 9.15 m used for drilling purposes. The tank was observed sinking down about 3m vertically with distortion of tank bottom. Simultaneously the next slide occurred near the edge of the tank including the slope and moved in an inclined plan. The surface of the

soil was not washed thoroughly but displaced about 2-2.5 m vertically and horizontally. Last slide occurred at the lower part of the second slide. The movements were slow towards an inclined plane dipping in the same direction and displaced about 3.0 m both ways. The last two slides damaged the approach road. The lower part of the last slide was washed away down to the hill toe. Some of the creep materials produced heap on the crated masonry provided for the protection of the road as shown in Fig.1. Wide tension cracks developed all round the slided zone.

For investigation purposes, critical slope was selected along the slipped surface as shown in site plan Fig.1. (Kalita, et al, 1988)

3. FIELD INVESTIGATION :

After selection of the critical section for analyses, the field work was carried out by making six bore holes as shown in Fig.1. In each borehole, standard penetration tests (SPT) were carried out and the value of SPT recorded continuously upto 6 m depth.

Disturbed soil samples were collected to carry out classification test in the laboratory for identification of soil. Undisturbed soil samples were also collected to determine the shear parameter.

4. LABORATORY INVESTIGATION :

Sieve analysis and Atterberg limit tests were carried out with the disturbed soil and the soils were classified up to the depth of boring. The soil profiles are shown in Fig. 2 & 3. From the observed SPT values, the depth of loose soil was found out. Based on field and laboratory testes, the soils parameters were evaluated (Table 1) .

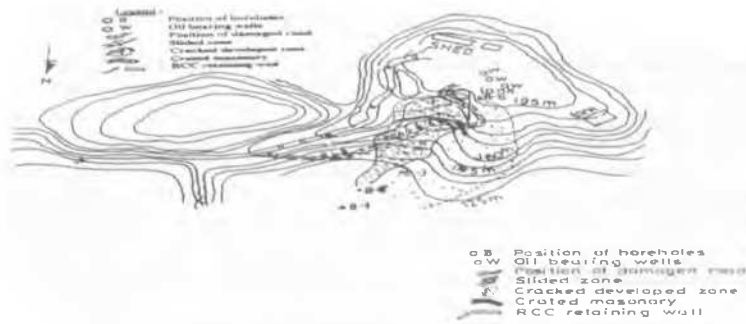


Fig.1. Site plan showing the positions of boreholes

Table 1. Shear parameters

Sample No.	Wet density gm/cc	Dry density gm/cc	Moisture content %	Direct shear test (Undrained)		Triaxial compression tests (Consolidated quick test)		Unconfined compression tests	
				Cohesion c kg/cm ²	Friction ϕ degree	Cohesion c kg/cm ²	Friction angle, ϕ degree	Strength q_u kg/cm ²	Sensitivity
B-2	1.82	1.41	27	0.16	24 ⁰	0.10	14.0 ⁰	0.31	1.10
B-3	1.94	1.59	24	0.14	30 ⁰	0.26	6.3 ⁰	0.29	0.70
B-4	1.94	1.54	25	0.09	27 ⁰	0.19	11.3 ⁰	0.35	0.81
B-6	1.92	1.57	22	0.26	18 ⁰	0.25	8.0 ⁰	0.48	1.20

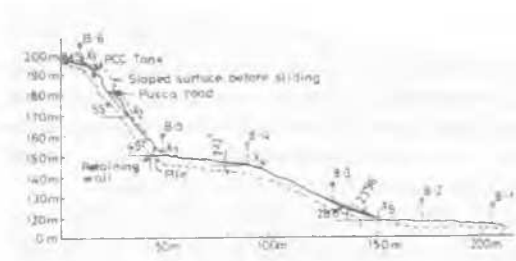


Fig.2. Critical slope section showing positions of boreholes

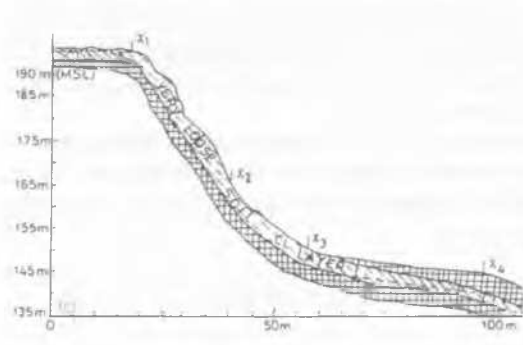


Fig. 3. Longitudinal profile through X₁-X₄

and Juan 1994) As the slided mass included only the loose materials, wedge analysis for the material lying between the slided surface and the original ground surface was considered. Back analysis was carried out for 4.5 m average depth of loose soil along the slope, with the help of computer. In order to consider the effect of pore water pressure, the ground water table was considered to be coinciding with ground level during rainy season. For a factor of safety of unity and angle of internal friction ϕ ranging from 15⁰ to 30⁰, the values of cohesion were obtained for the comparison of the laboratory test data and the results of the back analysis suggest an average values of $c = 0.16 \text{ kg/cm}^2$ and $\phi = 24^0$ for the general slope. These values of shear parameters are thus adopted for the slope stability analysis.

Since the failure was a block type, wedge analysis was used. For better assessment the whole slope was divided into four individual slopes such as $x_1 - x_2$, $x_2 - x_3$, $x_3 - x_4$ and $x_4 - x_5$ as shown in Fig. 2. Wedge analysis was carried out for these slopes and for the whole slope $x_1 - x_5$ to find out the factor of safety by using modified Bishop's equation as given below : (Bishop, 1967).

$$F = \frac{c \cdot \sec^2 i / \gamma_2 + \tan \phi [1 \pm \alpha_v - \alpha_h \tan i - (1 - z_w/z) \gamma_w / \gamma]}{(1 \pm \sigma_v) \tan i + \alpha_h}$$

Where, F = Factor of safety
 c, ϕ = Shear parameters, $c = 0.16 \text{ kg/cm}^2$ and $\phi = 24^0$

5. STABILITY ANALYSIS OF THE COMPLETE SLOPE :

The Back analysis is an indirect method for assessment of strength parameters and particularly reliable when the slope is in failing condition. (Koseki et al, 1999, Santiago

Table 2: Factor of safety from wedge analysis

c- 1.60t/m ²		$\phi = 24.00^\circ$		$\gamma = 1.82t/m^3$		$\gamma_w = 1.00t/m^3$	
Zone	Slope Angle,i deg.	Average depth of backfill/ loose soil in metres	Depth of GWT below ground surface, Zw in metres	Factors of safety, F			
				Static case $\alpha_h = 0.0, \alpha_v = 0.0$		Dynamic case	
						$\alpha_h = 0.12, \alpha_v = +0.06$	$\alpha_h = 0.12, \alpha_v = -0.06$
X ₁ -X ₂	55.00	4.50	0.00	0.5563	0.4559	0.4729	
X ₁ -X ₂	55.00	4.50	1.00	0.5944	0.4892	0.5100	
X ₁ -X ₂	55.00	4.50	2.00	0.6324	0.5225	0.5472	
X ₁ -X ₂	55.00	4.50	3.00	0.6705	0.5557	0.5843	
X ₁ -X ₂	55.00	4.50	4.00	0.7085	0.5890	0.6215	
X ₁ -X ₂	55.00	4.50	4.50	0.7276	0.6056	0.6401	
X ₂ -X ₃	45.00	4.50	0.00	0.5913	0.4785	0.4822	
X ₂ -X ₃	45.00	4.50	1.00	0.6456	0.5245	0.5335	
X ₂ -X ₃	45.00	4.50	2.00	0.7000	0.5706	0.5848	
X ₂ -X ₃	45.00	4.50	3.00	0.7543	0.6166	0.6361	
X ₂ -X ₃	45.00	4.50	4.00	0.8087	0.6627	0.6873	
X ₂ -X ₃	45.00	4.50	4.50	0.8359	0.6857	0.7130	
X ₃ -X ₄	7.12	4.50	0.00	3.1937	1.6603	1.5400	
X ₃ -X ₄	7.12	4.50	1.00	3.6289	1.8757	1.7690	
X ₃ -X ₄	7.12	4.50	2.00	4.0642	2.0911	1.9981	
X ₃ -X ₄	7.12	4.50	3.00	4.4994	2.3066	2.2271	
X ₃ -X ₄	7.12	4.50	4.00	4.9346	2.5220	2.4562	
X ₃ -X ₄	7.12	4.50	4.50	5.1522	2.6297	2.5707	
X ₄ -X ₅	23.96	4.50	0.00	0.9776	0.71401	0.7142	
X ₄ -X ₅	23.96	4.50	1.00	1.0000	0.8321	0.8153	
X ₄ -X ₅	23.96	4.50	2.00	1.2223	0.9241	0.9164	
X ₄ -X ₅	23.96	4.50	3.00	1.3446	1.0161	1.0175	
X ₄ -X ₅	23.96	4.50	4.00	1.4670	1.1081	1.1186	
X ₄ -X ₅	23.96	4.50	4.50	1.15281	1.1541	1.1692	

Table 3: Factor of safety from wedge analysis

c- 1.60t/m ²		$\phi = 24.00^\circ$		$\gamma = 1.82t/m^3$		$\gamma_w = 1.00t/m^3$	
Zone	Slope Angle,i deg.	Average depth of backfill/ loose soil in metres	Depth of GWT below ground surface, Zw in meters	Factors of safety, F			
				Static case $\alpha_h = 0.0, \alpha_v = 0.0$		Dynamic case	
						$\alpha_h = 0.12, \alpha_v = +0.06$	$\alpha_h = 0.12, \alpha_v = -0.06$
X ₁ -X ₅	28.80	4.50	0.00	0.8275	0.6436	0.6264	
X ₁ -X ₅	28.80	4.50	1.00	0.9264	0.7210	0.7118	
X ₁ -X ₅	28.80	4.50	2.00	1.0253	0.7983	0.7971	
X ₁ -X ₅	28.80	4.50	3.00	1.1241	0.8757	0.8825	
X ₁ -X ₅	28.80	4.50	4.00	1.2230	0.9531	0.9679	
X ₁ -X ₅	28.80	4.50	4.50	1.2725	0.9917	1.0106	

i = Respective average slope angle of cross sections as 55⁰, 45⁰, 7.12⁰, 23.96⁰ and 28.8⁰ for sections x₁ - x₂, x₂ - x₃, x₃ - x₄, x₄ - x₅ & x₁-x₅.

γ = Unit weight of loose soil = 1.82 gm/cc.

z = Average depth of loose soil = 4.5 m.

α_h = Horizontal seismic co-efficient = 0.1

α_v = Vertical seismic co-efficient = 0.06.

z_w = Depth of ground water table below existing surface at 0, 1, 2, 3, 4 and 4.5m

It is seen from the result that at an average slope angle of 28.8⁰, the whole slope is stable for static condition when water table lies below 1.0 metre. But for dynamic case, the slope is unstable. For the individual section with respective slope angle, it is seen for the sections x₁ - x₂ and x₂ - x₃ the slopes are unstable for static and dynamic cases, even with water table at 4.5 m depth. The section x₃ - x₄ is found to be stable for both the cases. For section x₄ - x₅, the slope is found to be stable in static and dynamic cases when water table lies below 1m and 3m respectively.

6 MECHANISM OF FAILURE :

A careful observation and investigation to the landslide indicates that it is a landslide of second categories of

The values of factor of safety were computed for both static and dynamic cases for different slope, at various depth of ground water table. The analysis was done by computer and the results are given in Table 2 and 3.

movements involving detritus classification. It was a local, more or less sudden downward movement of a mass of moist clay. The detachment and downward movement of 1st slide followed after an interval by the 2nd and 3rd slide. The soil underneath the tank might become water soaked by the leakage from the bottom of the water tank and later, during a period of heavy rainfall the soil tended to slide down. This tendency to slide down was triggered by the earthquake shock and, therefore, slide occurred locally.

For the general slope at slope angle 28.8° , the soil parameters were found to be $c = 0.37 \text{ kg/cm}^2$ and $\phi = 24^\circ$ for critical condition. However, the value of c found from laboratory test was much low. Therefore, the slope is unstable under above mentioned conditions. Protective measures were needed for stability of the slope.

7. PREDICTION OF LANDSLIDE :

Simple forms of landslides could perhaps be predicted by recourse to conventional procedures using deterministic and probabilistic analysis. In complex situations, landslide occurrences have generally eluded the prediction attempts. Attempts have been made to connect the landslide activity either with rainfall records or with the earthquake occurrence or with slope surface movement measurements. One method of landslide prediction rely on measurement of displacement pattern in the stage of terminal creep (Bhandari & Jeyatharan 1994, Bhandari 1999).

8. PROTECTIVE MEASURES:

- The excavated soil should not be dumped on the side of the cutting putting more surcharged load on the slope. These soils are taken away from the site.
- Trees should be retained as far as possible.
- Sufficient vegetation along with the deep rooted trees should be developed along the slope.
- Jute geogrid can be placed on the slope for growth of vegetations (Sarkar and Venkataraman, 1989).
- Crated masonry wall should be provided in positions as shown in Figure 1 and 2. At least one R.C.C. cantilever retaining wall should be provided in a position as shown in Figure 1 and 2 with sufficient weepholes.
- The sloped area covered by the slided mass needs dressing and protection from percolation of water.
- All open cracks in the slided zone and cracked zone should be sealed with cement mortar.
- The side drain effluents of the new approach road should be discharged away from the threatened area.

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