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STABILITY ANALYSIS OF SOIL SLOPE, KALIYASAUR VILLAGE

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Abstract— Landslides are one of the important natural hazards of Himalaya. Due to landslides, the Himalayan region often faces geo environmental problems posing threats to, both, life and properties. The study was carried out in Kaliasaur village on left bank of Alaknanda river along Rishikesh- Badrinath (NH-58) road in Pauri district of Uttarakhand state, frequent traffic blockages and consequent environmental and economic losses are caused in this area. Considering the importance of Kaliasaur landslide slope stability analysis was conducted. Under this project geotechnical survey of the area was conducted. Factor of safety was calculated using the test results in GEO-STUDIO software, and a value of 0.373 was obtained which is less than 1. This suggests that the slope is not stable. Corrective measures for slope stability are suggested in this study by conducting a parametric analysis and thus reducing the existing slope.

1 INTRODUCTION

Anywhere where the surface of the ground is not level there are gravitational forces acting which desire to cause the movement of soil from the high points to the low points. Often also acting, but not so well recognized, are the seepage forces due to moving water and to a less extent forces due to earthquakes, vibrations and other causes. These forces set up shearing stresses in the soil which, if they are of sufficient magnitude, can cause a mass soil movement. Whether movement occurs depends not only on the many factors controlling the shearing strength of the soil but also on such items as the strength of roots, ice and other material which must be severed if there is to be a mass soil movement. Near the ground surface the soil's shearing strength at a given point may differ greatly during the various seasons of the year. Soils generally swell on wetting, during the rainy season, and shrink on drying, during the dry season. Increased moisture decreases the shearing strength during the wet season and decreased moisture increases the shearing strength. Indeed clays may become quite strong on drying so that these changes in shear strength may be quite large. In cold climates frozen soil may have high strengths which become extremely small during the thaw process. An exact quantification of these strength changes is exceedingly difficult and in many cases a quantitative analysis of stability is relatively reliable only when the failure zone extends well into the ground. Deep in the ground the seasonal variation effect of the soils shearing strength is generally sufficiently small that the shearing strength depth profile may be determined with reasonable accuracy and thus reasonable estimates of stability may be made.

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The strengths which should be used in any analysis would, of course, be those associated with the most unfavourable season of the year. Normally, this is when the water table is highest and the effective stresses lowest. The engineer must deal with many practical problems in which earth slopes are involved. Common examples include the slopes of both cut and fill along railroads and highways, banks of canals and streams, slopes of earth dams, temporary slopes created during construction and natural slopes to mention the more obvious. Such slopes can be analyzed for stability with reasonable accuracy if the shearing strength profiles and geological cross sections are known. Unfortunately many factors introduce complications into the analysis such as heterogeneous soil types, the unknown boundary conditions of flow nets and the like. It is generally necessary, therefore, to adopt simplified, average soil characteristics and average cross sections which are as representative of actual as can be obtained. Those steps which bear on the choice of the simplified section and the simplified soil characteristics are always important in stability analysis but are independent of the actual performance of the analysis. Once the simplified section and soil characteristics are chosen the methods of analysis are relatively consistent and well known. These methods will be explained in the following considerations but it should be realized that the time required for actual analysis, in which many small details and alternative possibilities enter may be many times that indicated by the simple numerical methods.

2 TESTING OF SOIL SAMPLES

2.1 METHODS OF SAMPLING

Samples are of two types:

a) Disturbed Samples - These are taken by methods which modify or destroy the natural structure of the material though, with suitable precautions, the natural moisture content can be preserved.

b) Undisturbed Samples - These are taken by methods which preserve the structure and properties of the material. Such samples are easily obtained from most rocks, but undisturbed

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samples of soil can only be obtained by special methods.

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2.2 DETERMINATION OF WATER CONTENT

This method covers the determination of water content of soils expressed as a percentage of the oven-dry weight. The soil specimen taken shall be representative of the soil mass. The size of the specimen selected depends on the quantity required for good representation, which is influenced by the gradation and the maximum size of particles, and on the accuracy of weighing. The percent of water content shall be calculated as follows:

w=(W2-W3)/(W3-W1)X100

Determination no.	1	2	3
Wt. Of cruisible (W_3)	33.78	34.46	34.67
Wt. Of cruisible+ wet soil (W_1)	40.77	64.06	64.78
Wt. Of cruisible+ dry soil (W_2)	39.97	61.25	61.36
Water content w=(W2-W3)/(W3-W1)X100(%)	11.79	10.39	10.52
Average value	10.9%		

2.3 DETERMINATION OF DRY DENSITY OF SOIL IN-PLACE (CORE-CUTTER METHOD)

a) This standard covers the method for the determination of the in-place density of Fine-grained natural or compacted soils free from aggregates using a core-cutter.

b) For the purpose of the tests described in this standard, a soil shall be termed as fine-grained soil if not less than 90 per-cent. Of it passes a 4.75mm IS Sieve.

	Determination no.	1	2	3
1	Wt. Of core cutter + wet soil (W_S) (gm)	2823	2918	2745
2	Wt. Of core cutter (W_C)(gm)	898	898	898
3	Volume of core cutter (V_C) cm^3	1000	1000	1000
4	Bulk density $\gamma_{b=}$ $(W_S-W_C)/V_C$ g/cm^3	1.925	2.020	1.847
5	Water content (w)	0.12	0.11	0.11
6	Dry density g/cm^3	1.719	1.820	1.663
	DENSITY OF SOIL- 19.30g/cm^3			

2.4 TEST FOR THE DETERMINATION OF LIQUID LIMIT (MECHANICAL METHOD)

A flow curve shall be plotted on semi-logarithmic graph representing water content on the arithmetical scale and the number of drops on the logarithmic scale. The flow curve is a straight line drawn as nearly as possible through the four or more plotted points. The moisture content corresponding to 25 drops as read from the curve shall be rounded off to the near-est whole number and reported as the liquid limit of the soil.

S.NO.	Water content (%)	No.Of blows
1	5	35
2	10	30
3	15	27
4	20	20
5	25	13

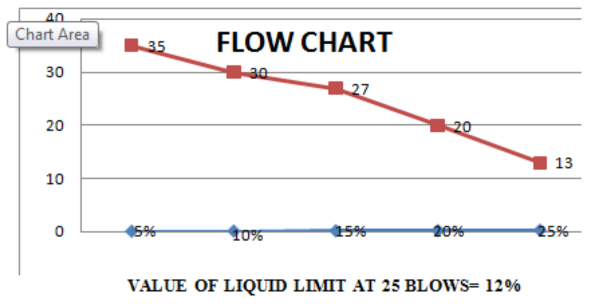

Fig. 1. flow chart

2.5 TEST FOR THE DETERMINATION OF PLASTIC LIMIT

Plastic limit of soil sample is found zero. Because soil begain crumbling at very low water contain without forming in 3 mm diameter thread, and at higher water contain it form a very loose paste as well as slurry.

2.6 GRAIN SIZE DISTRIBUTION OF SOIL

Coarse grained analysis- total mass of soil- 7.002 Kg

Fine grained analysis- total mass of soil- 2.0 Kg

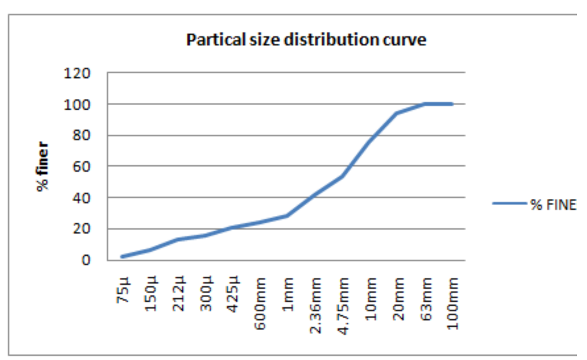

Fig. 2. **Particle size distribution curve**

2.7 CLASSIFICATION OF SOIL

Classification of soil as per unified soil system followed by IS: 1498-1970 and plasticity chart are shown in figure in annexure B. From the particle size distribution curve-

D_{10}= 0.185, D_{30}= 1.16, D_{60}= 6,34

Coefficient of uniformity $C_u = D_{60}/D_{10}$= 6.34/0.185 = 34.27

Coefficient of curvature $C_c = D_{30}{}^2/(D_{60} \times D_{10})$ = 1.162/(0.185x 6.34) = 1.14

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Liquid limit- 12 %
Plastic limit- 0
Plasticity index- 12%

3. RESULTS AND CONCLUSION

After putting all the test results in GEO-STUDIO software and procedure as given in article 3.3 factor of safety come 0.373 which is less then 1. Which shows slope is not stable. Analysis results and failure plane is shown in next graph.

After analyzing the slope by GEOSTUDIO software we find that the factor of safety is coming 0.373, which is than 1. It shows that that soil slope is very critical and one of the reason of failure of that slope and resulting continuous landslide. So it is required to make that slope stable. For this purpose we make a parametric analysis of slope by reducing the existing slope and find out following results (see annexure C) -

Existing slope	Height (H)	F.O.S
57.9946	32	0.373
57.9946	31	0.411
55.4077	29	0.490
43.531	19	0.805
40.3645	17	0.933
41.6335	16	0.988
37.87	14	1.124
39.8055	15	1.053

As we can see after excavating the soil below to 18m the factor of safety is coming 1.053 which shows soil slope is stable in this label. So this can be one of the control measures that we can apply at that particular site. In spite of all this after studying the literatures available about the soil slope or based on previous experiences some more control measures can be recommended for that particular slope.

References

[1] IS code – sp 36 (1 and 2), IS : 2720 (Part I)-1983, IS : 2720 (Part 2)-1973, IS: 2720 (Part 5) – 1985 , to IS: 9259-1979,

[2] “Soil mechanics and foundation engineering” by B.C.Punmia. (Book style with paper title and editor)

[3] “Geotechnical engineering-1997” by G.P. Raymond.

[4] Himalayan geology vol.31 (2), 2010, pp,

[5] BS 1377: 1974 Methods of testing soils for civil engineering purposes.

[6] IS 4434: 1978 which gives a measure of the shear strength of the soil.

[7] Skempton and Hutchinson (1969) have suggested a means of classification of slope failures in fine grained soils which is briefly outlined below and will aid in describing different phenomena which may be observed.

[8] Alireza Ahangar-Asr, Asaad Faramarzi, Akbar A. Javadi, "A new approach for prediction of the stability of soil and rock slopes", Engineering Computations, Vol. 27 Iss: 7, pp.878 - 89

[9] Bishop, 1955, *give modification of the Swedish circle analysis.*

[10] *THE FRICTION CIRCLE (φ - CIRCLE) METHOD* by Professors G. Gilboy and A. Casagrande and presented in detail by Taylor (1937).

[11] Gibson, R.E. and Morgenstern, N.R. (1962) A note on the stability of cuttings in normally consolidated clays, Geotechnique, 12 (3), 212–16.

[12] Bishop, A.W. (1955) The use of the slip circle in the stability analysis of slopes, Geotechnique,5 (1), 7–17.

[13] Murthy, V. N. S. (2003). “Civil & Geotechnical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering”, Marcel Dekker, Inc. Pages: 1029.

[14] Skempton, A.W. (1964) Long-term stability of clay slopes, Geotechnique, 14, 75–102.

[15] . Whitman, R.V. and Bailey, W.A. (1967) Use of computers for slope stability analysis, Journal ASCE, 93 (SM4), 475–98.

[16] Abrams, T. G. and Wright, S. G. (1972). "A Survey of Earth Slope Failures and Remedial Measures in Texas." Research Report 161-1, Center for Highway Research, University of Texas, August 1972.

[17] Corps of Engineers. (1970). “Laboratory Soils Testing.” Engineer Manual EM 1110-2-1906, Department of the Army, U. S. Army Corps of Engineers, Washington, D. C., November 1970 with Change 1 dated May 1980.

[18] Ladd, C. C. and Foott, R. (1974). "New Design Procedure for Stability of Soft Clay, Journal of the Geotechnical Engineering Division.” ASCE, Vol. 100, No. GT7, pp. 763–786.

[19] Kanji, M. A. (1974). "The relationship between drained friction angles and Atterberg limits of natural soils." Technical Note, Geotechnique, Vol. 24, No. 4, December, 1974.

[20] Kayyal, M. K. and Wright S.G. (1991). "Investigation of Long-Term Strength Properties of Paris and Beaumont Clays in Earth Embankments." Research Report 1195-2F, Center for Transportation Research, The University of Texas at Austin, November, 1991.