08 May 1984, 10:15 am - 5:00 pm

Stability of Hill Slopes and Foundation Condition at Radio Astronomy Centre Ootacamand

R. K. Katti
Indian Institute of Technology, Powai, Bombay, India

V. S. Chandrasekaran
Indian Institute of Technology, Powai, Bombay, India

A. Parthasarathy
Indian Institute of Technology, Powai, Bombay, India

K. K. Moza
Indian Institute of Technology, Powai, Bombay, India

Stability of Hill Slopes and Foundation Condition at Radio Astronomy Centre Ootacamand

R. K. Katti, V. S. Chandrasekaran

Professors of Civil Engineering, Indian Institute of Technology, Powai, Bombay - 76, India

A. Parthasarathy

Professor and Head, Earth Sciences Department, Indian Institute of Technology, Powai, Bombay - 76, India

K. K. Moza

Research Officer, C.B.I.P. Project, Civil Engineering Department, Indian Institute of Technology, Powai, Bombay - 76, India

SYNOPSIS Stability aspects of hill slopes and foundation considerations of Radio Astronomy Centre at Ootacamand are described. The analysis of slopes indicated that if joints are not covered, the material in joints may loose strength and the slopes may enter a state of instability. Footings with inclined legs were found to resist the horizontal forces, pull and overturning movements. Lime piles adopted for strengthing soft material at one of the tower locations were found to be effective.

INTRODUCTION

Tata Institute of Fundamental Research Bombay in the year 1967 decided to have a Radio Asto--rnomy Centre for astronomical and astro--physical observations at Fairlawn Reserved Forest Site Ootacamand, India. The area lies between longtitude 76° 44' E and latitude 11°24' N. This location has been selected on the basis of scientific and technical require--ments.

The telescope covers a length of approximately 550 m and consists of parabolic frames. The telescope is operated by a single shaft which is supported by 24 towers spaced 25 m apart. The parabolic frames can be rotated through 62° clockwise and 82° anticlockwise facing north. Section of a typical tower is shown in Fig.1. Under extreme conditions, the tip of the telescope rises to a height of about 40 m.

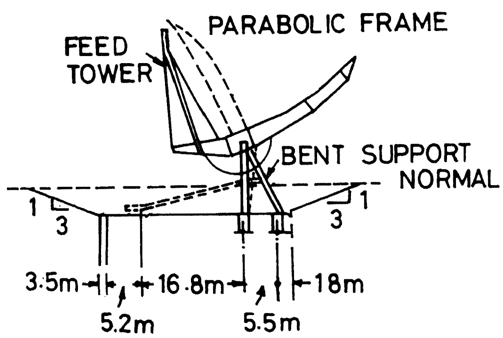

Fig. 1. Section of Tower

In this region even very flat slopes exhibited slides during rainy season. This is experie--nced by Railways, Hindustant Photo Film Factory, etc. The slides generally extended in N or NE to S or SW direction and sliding material took a course approximately parallel to the dip.

The hill slopes have thick mantle of residual soils derived from inplace weathering of charnockites. Some of the major joints were clearly visible in the excavated portion of the site and the parent rock showed effects of weathering. Weathered material could be seen as filling in the joints. The material fill--ing the joints unlike the general soil mass is blocky in nature and it was easy to remove the material from exposed joints. The inten--sity and degree of weathering in the joint material and general soil mass seem to be varying. This indicated that the joints may form surfaces of weakness along which the strength may be low. The foundations are also subjected to moments.

This paper describes various aspects related to stability of hill slopes, design of founda--tion of towers, preventive measures for stabilizing the slopes and the performance till todate.

LOCATION OF SITE

The Centre, is located in the Nilgiri mounta--ins. The mountains in this region rise to heights of about 2440 m characterised by steep spurs. The mountain massif is made up of charnockites of Archaean age. Rocks generally strike in ENE-WSW direction.

From functional considerations, the slope of the site has been adjusted to approximately 11° in southerly direction. A nullah runs in NW-SE direction. The mean annual rainfall at this site is 140 cm. The ground water table as observed in March 1967 was at surface near the nullah and about 9.76 m deep inside the slope at other places.

Major part of the site consists of weathered material and residual clays of kaolinitic nature occuring to considerable depths. Two sets of joints are observed, one set more or less parallel to the strike and the other nearly perpendicular. Fig.2 shows prominant joints observed at the site.

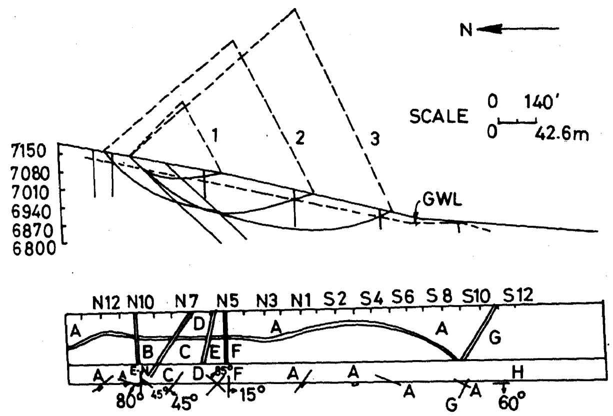

Fig.2. Prominant Joints and Critical Circles for Stability Analysis

APPROACH TO STABILITY ANALSIS

Taking into consideration the nature of sli--des and the influence that joints may have on the stability, the joint pattern on the exposed slopes was carefully mapped. The locations where the observation regarding joints have been made are given in Table I.

TABLE I. Data on Joints

Joint	Location	Strike	Dip (Magnitude) and direction)
A(N-11 to S-9)	N-11	NNE-SSW	Vertical
B	N-10	E-V	45^o (S)
C	N-8	WNW-ESE	45^o (Southerly)
D	N-6	NE-SW	85^o (SE)
E	N-6	NW-SE	Vertical
F	N-5	E-W	15^o (N)
G	S-9	NW-SE	Vertical
H	S-12	N-S	60^o (W)

The predominant joint in the area passes through centre of N-11 to S-9. The joint runs in NNE-SSW direction which confirms to the regional jointing pattern. The joint is nearly vertical.

To conduct the stability analysis, it was necessary to evaluate the properties of soil at the site. Typical soil profile given in Fig.3 along N-S Direction show clays or clay loam occuring upto considerable depths, however in N-1, N-2 and S-6 regions boulders are encountered at shallow depths.

The colour of soil is brown to a depth of around 1.2 m and pink to pinkish white at

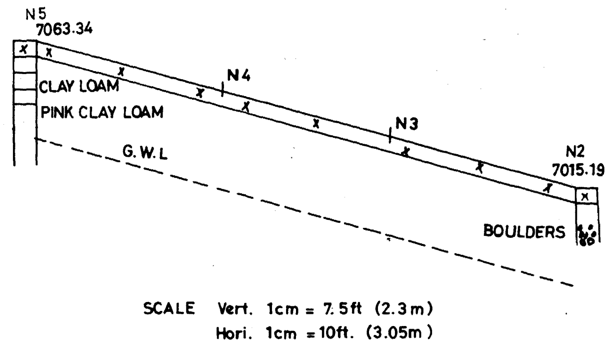

Fig. 3. Typical Soil Profile

lower depths. The colour of material filling the joints is darker in shade as compared to the surrounding soil mass.

The property data from various boreholes showed no significant variation. The representative soil properties are given in Table II.

TABLE II. Properties of Soil

Physical Properties	
Liquid Limit, %	50 to 60
Plastic Limit, %	33 to 33
Shrinkage Limit, %	30 to 41
Specific Gravity at 25^oC	2.68
Textural Composition	
Gravel (> 2.00 mm)	Nil
Sand (2.00-0.06 mm)	20 to 40
Silt (0.06 mm-0.002 mm)	30 to 15
Clay	25 to 48
Engineering Properties	
Standard Proctor Density, g/cc	1.52
Optimum Moisture Content, %	29
Modified Proctor Density, g/cc	1.65
Optimum Moisture Content, %	25.5

According to textural classification the soil is clay or clay loam and as per engineering classification it is MH.

The void ratio pressure relationship obtained from consolidation tests on undisturbed samp--les indicates low compressibility having compression index between 0.12 to 0.16 for pressures beyond 2.18 kg/cm^2. It may be noted that the classification of soil as silt of high compressibility does not reflect properly the insitu compressibility characteristics of this residual clay.

The shear parameters c and ϕ obtained from consolidated slow direct shear test are 0.49 kg/cm^2 and 18^o for clay and 0.24 kg/cm^2 and

A pit of 0.3 m x 0.3 m x 0.3 m dug at N-10 location filled with water indicated the susceptibility of compact material to softe--ning in presence of water when the sides collapsed and sloughing occured around the pit. This indicated that the joints if not covered may loose strength and form surfaces of weakness. This was borne out by frequent occurence of slides in the region.

Keeping in view the topography, the joint pattern, soil properties and the ground water condition it was necessary to examine the stability of N-S slope, slope to the western side of the centre line and the slope to the eastern side of the centre line. The analysis was carried out using Swedish circular are method. The slopes were examined for :

a) fair weather condition in which the water table is located 12.2 m below ground level and and steady seepage occurs. The joints are assumed to be intact.

b) The joints are considered intact but ground water level dropping instantaneously from the surface to a depth of 12.2 m.

c) the water level undergoes sudden drawdown in condition 'b' and the slip surface may run along joints B and F which may soften the material to varying degree under all weather condition. Conditions 'b' and 'c' correspond to severe ones to which the slope may be subjected.

For stability analysis the c and ø values used are 0.49 kg/cm^2 and 18^o for clay, 0.24 kg/cm^2 and 31^o for clay loam and 0.0 kg/cm^2 and 18^o for joint material respectively.

N-S Slope

Typical circles are shown in Fig.2 and the results of the analysis are given in Table III. The slopes are found to be stable for (a) and (b) conditions however condition (c) seems to be severe as the factor of safety indicates that it is necessary to take adequate precautions to ensure that water does not enter the joints leading to loss in shear strength. The preventive measures are discussed separately in section on remedial measures.

TABLE III. Factors of Safety for Critical Slip Surfaces

Cases	Slip circle 1		Slip circle 2		Slip Circle 3	
c, kg/cm^2	0.49	0.24	0.49	0.24	0.49	0.24
ø	18.0	31.0	18.0	31.0	18.0	31.0
(a)	1.8	1.9	1.62	2.16	1.5	2.16
(b)	1.75	1.85	1.3	1.68	1.1	1.3
(c)	1.0	0.85	-	-	-	-

The slopes on Western side is steep (1:25). A major joint A which is nearly vertical passes along the eastern part of the slope. The slope was examined assuming saturated unit weight for actuating forces and partial submergence for resisting forces. Analysis showed that under these conditions the factor of safety is just unity and can be considered adequate if the joints do not open up. A road embankment planned to be built close to top of slope would automatically provide a cover for the joints.

Slope on Eastern Side of Centre Line

Slope on the eastern side of centre line is flat approximately 1 in 6 and the height is comparatively small. The slope of this side is found to be satisfactory.

FOUNDATION CONSIDERATIONS

The load coming on foundation is due to dead load and wind action. As the towers are very high, one can expect high influence of wind loads which may cause reversal of stresses. In view of this it was necessary to design foundations taking into consideration the worst combination of forces. The loading conditions on N and S group towers are given in Table IV and V for typical tower footings shown in Fig.4.

TABLE IV. Loading Conditions on the N-Group Towers

No.	Direction of Wind	Wind Load-Horizontal, Tons	Wind Load Vertical, Tons
Footing No.1	**Dead Load = 6.7 Tons**		
1	East West	0.845	18.15↓
2	East West	0.853	17.70↑
3	North South	6.35	2.37↑
Footing No.2	**Dead load = 0.00 Tons**		
1	East West	6.77	16.7↓
2	East West	7.00	17.2↑

TABLE V. Loading Conditions on S-Group Towe--rs

No.	Direction of Wind	Wind Load-Horizontal, Tons	Wind Load Vertical, Tons
Footing No.1	**Dead load = 7.6 Tons**		
1	East West	3.09	18.9↓
2	East West	2.96	23.5↑
3	North South	3.18	4.24↑
Footing No.2	**Dead Load = 0.00 Tons**		
1	East West	3.09	20.6↑
2	East West	2.96	20.0↓

Plate load test data from a nearby site with similar soil conditions indicated that the soil exhibits general shear failure characteristics. The bearing capacity obtained was 2.5 kg/cm^2. In the light of this for a footing size of 1.8 m x 1.8 m at a depth of 1.8 m the net ultimate bearing capacity works out to be 10 and 15 kg/cm^2 respectively for shear parameters mentioned earlier indicating sufficient factor of safety.

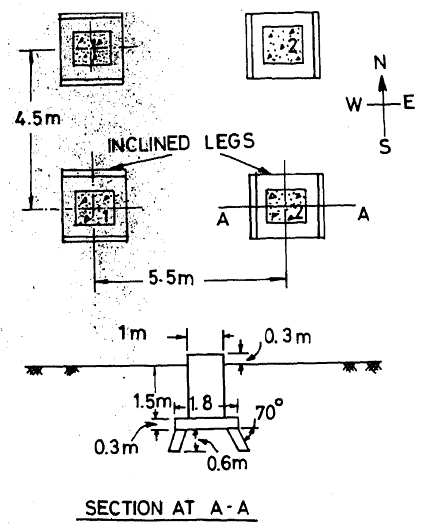

Fig.4. Typical Tower Footings

Design of N-Group Towers

The loading conditions are given in Table IV. when loading corresponding to case (i) was considered the net soil pressures at base were 0.87 and 0.60 kg/cm^2 respectively which were within permissible values. A resistance of 8.44 Tons could be mobilized to resist horizontal wind loads.

For loading corresponding to case (ii) a net upward pull of 3.84 Tons was found to be acting. Besides this net upward pull, no base friction could be mobilized to resist horizontal forces. The foundation required to be modified for adequate resistance against pull and sliding. In the light of this, legs 0.6 m deep and 0.3 m thick inclined at 70° to horizontal in east-west direction on two sides of footing were considered. Fig.4 shows typical tower footings with inclined legs. With the provision of inclined legs a passive resistance of the order of 27.0 Tons could be mobilized. The safe pull that could be resisted worked out to be 30.5 Tons. Foundation as per this design were adopted.

S-Group Towers

For S-group towers the design corresponding to that of N group towers was found to be safe for all conditions of loading.

A study made to see whether local slides would take place under individual towers to various combinations of loads coming at the footing level indicated that the actuating forces mobilized by the soil were far greater than the forces due to loads carried by the structure.

Settlement Considerations

The settlement under N-group towers were estimated, the maximum settlement was found to be 1.6 cm occuring under N-3 tower.

With the recommended type of footings the S-group towers were expected to undergo settlements of the order of 1.45 to 1.81 cm without placement of fill. With the placement of proposed fill, the settlements were 6.8, 13.3, 18.5, 23.8 and 28.2 cm for fill heights of 1.5, 3.0, 4.6, 6.1 and 7.6 m respectively.

As mentioned earlier at S-8 location soft soil was encountered between 6.7 to 11.6 m depths. For proposed fill height of 6 m the settlement worked out to be in the range of 30 cm which is very high. Also an analysis of stress transfer showed that it is necessary to improve the strength characteristics of S-8 location.

Based on these studies, fills serving no functional purpose were eliminated however at some locations being necessary the fills were compacted properly and allowed to settle for a period of 1 year to attain 90 % consolidation

REMEDIAL MEASURES

Following remedial measures were adopted .

For Slopes

For ensuring the stability of slopes, following precautionary measures were adopted.

i) The entire area was covered with 0.3 m thick compacted fill so that the water does not enter the joints and soften the material.
ii) Net work of drains was provided to intercept and drain out surface water. The drains were lined properly with polythylene and asphalt to safe guard against the percolation.

For S-8 Tower Location

The strength characteristics of S-8 location was improved by providing lime pile grid penetrating 1 to 2 m below soft material. Some limited studies had indicated that within around 3 months to a year 10.2 cm hole filled with calcium oxide would produce a lime pile of around 0.6 m in diameter having strength around 1.0 to 2.0 kg/cm^2. The following procedure for construction of lime piles was adopted.

i) Boreholes were drilled to a depth of 12.2 m using 10.2 cm auger.
ii) The layout of boreholes adopted is shown in Fig.5.
iii) Calcium oxide shel type having 98 percent purity mixed with 10 percent sand was poured into the borehole and the borehole was closed.

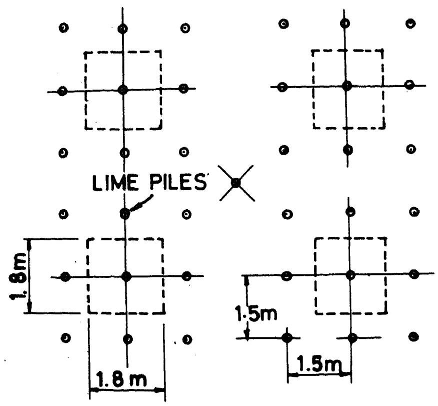

Fig.5. Layout of Lime Piles

The slopes and foundation of towers are per--forming satisfactorily for the last 15 years.

CONCLUSIONS

1. The slides taking place in this region may be attributed to the weakness at the joints. This can be prevented by covering the joints with recompacted material and water proofing the surface.
2. The footings with inclined legs are perfor--ming satisfactorily withstanding high moments.
3. The lime piles are found to be effective in strengthening the soft material.

ACKNOWLEDGEMENTS

The authors take this opportunity to thank Prof. A.K. De, Director, IIT Bombay for his keen interest in this work.

Thanks are also due to Shri C.P. Chavan and other staff members of Geotechnical Engineeri--ng Section of IIT Bombay.

REFERENCES

Pascoe. (1928), Land slides in Nilgiri Railways, Records of Geological Survey of India, Vol.60,

Handy, R.L. (1967), Chemical stabilization of active land slides, Report of Civil Enginee--ring Department, Iowa State, U.S.A.

Katti, R.K. et al (1967), Report on the stabi--lity of hill slopes and foundation conditions at Radio Astronomy Centre Site Ootacamand, Submitted to the Tata Institute of Fundamental Research and Tata EBASCO consulting Enginee--ring Services, India, pp. 1-61.

Kakroo, A.K. (1968), Use of lime piles for stabilization of black cotton soils, Unpubli--shed M.Tech. Dissertation, submitted to Civil Engineering Department, IIT, Bombay.