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Stability of Slopes - A Case History

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SYNOPSIS A Lecture cum-cinema hall was constructed on a terrace developed at the top of a hill--ock at a site located in the north-eastern part of India. At the time of development of the site, excavated soil was dumped on the slopes and retaining walls were constructed to retain the earth. After the first monsoon, the retaining walls gave way. Further distress in the vicinity of the building was noticed in the subsequent three years. The paper describes the details of the above failure, the investigations carried out and the remedial measures suggested.

INTRODUCTION

The site, located in the north-eastern part of India, consisted of cluster of hillocks. The hillocks are covered with forest vegetation. The development of the area necessitated construction of roads and buildings on a large scale. As suitable sites were scarce,most of the construction were made on the top of hill-ocks. The sites were developed by flattening the top of large hillocks forming the required level area. At the time of the selection of sites, no attention was given to the possible stability of the slopes. Further, during the development of sites, the excavated material was possibly dumped on the side slo-pes. These resulted widespread slope failures after heavy rains in the area. The paper des-cribes one such failure which took place at the site of a lecture-cum-cinema hall threatening the safety of the building. The investigations carried out and the remedial measures sugges-ted are also discussed.

DESCRIPTION OF SITE AND FAILURE

The site consisted of a number of hillocks, large and small. A lecture-cum-cinema hall of about 40 x 30 m in plan was constructed at the top of one such hillock. The site is located in earthquake prone zone (IS:1893-1975). The contour map of the site along with the location of the cinema hall is given in Fig.1.

The building was constructed in 1973 at the top of hillock as shown in Fig.1 after developing the site to obtain the requisite flat area for construction. The development of the site involved excavation of soil upto a maximum of about 4.0 m and the excavated material was possibly dumped on the northern and the southern slopes. Retaining walls were constructed on the northern and southern slopes of the building to retain the soil. The flat surface adjoining the building was covered with plain cement concrete hard surfacing.

During the monsoon of 1974, immediately after a heavy rain fall the retaining walls gave way. The northern and southern slopes adjoining the building failed and the retaining walls were carried almost to the toe of the slope. Subse-quently, protective works in the form of crated masonry as shown in Fig.2 were construc-ted to protect the building.

The protective works and slopes experienced further distress during the period 1975-77. The crated masonry got bulged and was slightly displaced from its original position in 1975 (Fig.3). The plain cement concrete surface around the building sank at various spots resulting in cracks. Subsequently, the entire plain concrete surface was covered by a bitu-minus overlay. Surface drains were also cons-tructed covering the plain concrete hard surface area. However, the discharge points of these drains were located on the slopes. Again in 1977, slips occurred on the sothern side of the building after heavy rains. The slips resulted in subsidence of as much as 2.5m of the ground very near to the building as shown in Fig.4. On the nothern side, the crated masonry bulged and got shifted by about a maxi-mum of 2.0 m. Cracks appeared on the bituminus surface. The line of slip on the sothern side and the line of cracks on the northern side are shown in Fig.3. At this stage, the problem was refered for detailed investigation to identi-fy the causes of failure and suggest remedial measures.

SOIL INVESTIGATIONS AND RESULTS

The following field and laboratory investiga-tions were carried out.

i) 10 bore holes,3 each on northern,southern & western sides and one on the eastern side of the buil-ding. The location of the bore holes are shown in Fig.1. The bore holes were made upto 5.0 m below ground level.

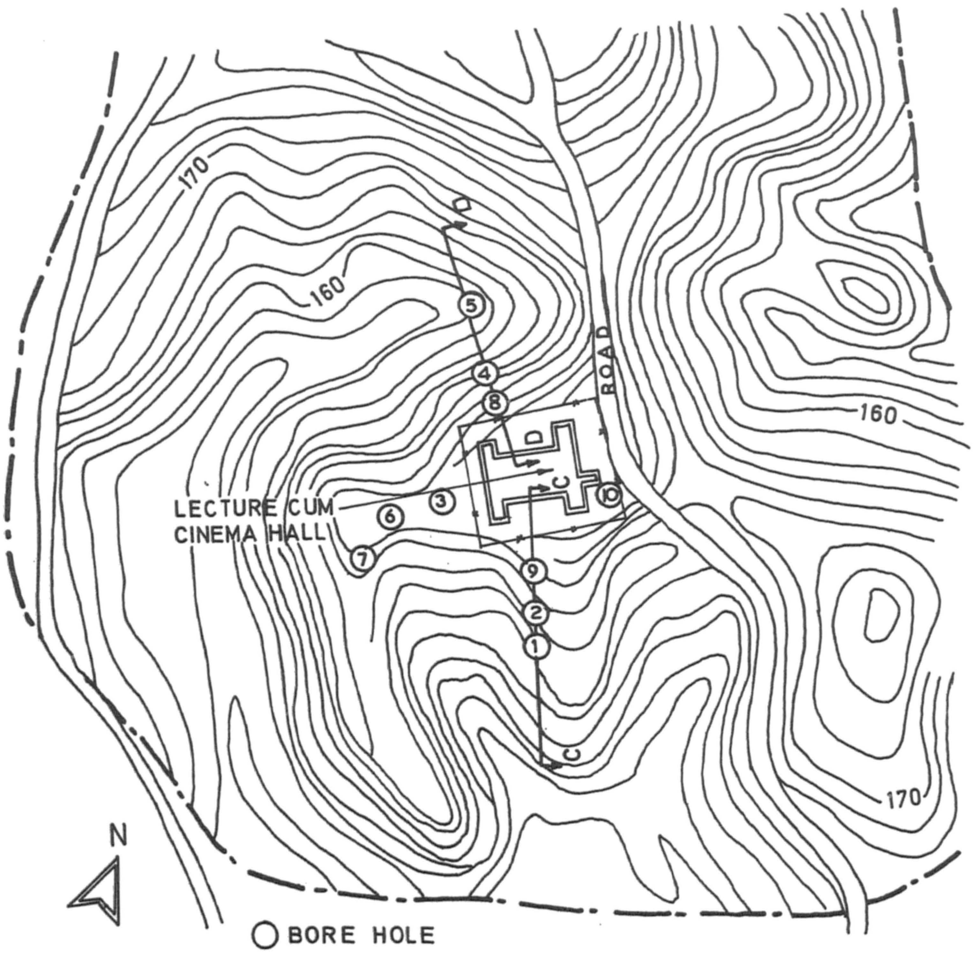

Fig.1 The Contour Map of the Site

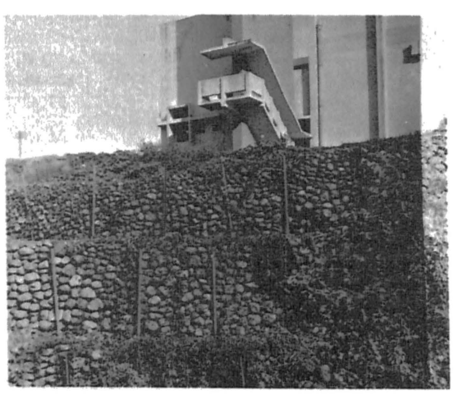

Fig.2 Crated Masonry Walls

ii) Standard penetration test(SPT) at 45 cm intervals in all the bore holes.

iii) Grain size analysis and Atterberg limit tests on soil samples collected from the bore holes.

iv) Bulk density and water content determination, and

v) Direct shear tests on undisturbed block samples collected at a depth of 1.5 m below ground level.

Based on the results of the grain size analysis and the Atterberg limit tests, the soils were classified as per Indian standards specifications, IS:1498-1970. The soils at site consisted of clay of low to medium compressibility. The liquid limit values varied from 30-45 and the plasticity index values varied from 10-25. The average dry density was found to be 1.62 g/cc.

The observed SPT values ranged from 35-60 in bore holes 3,6,7 and 10 (Fig.1) which are located on the western and eastern sides of the building where no distress was experienced. On

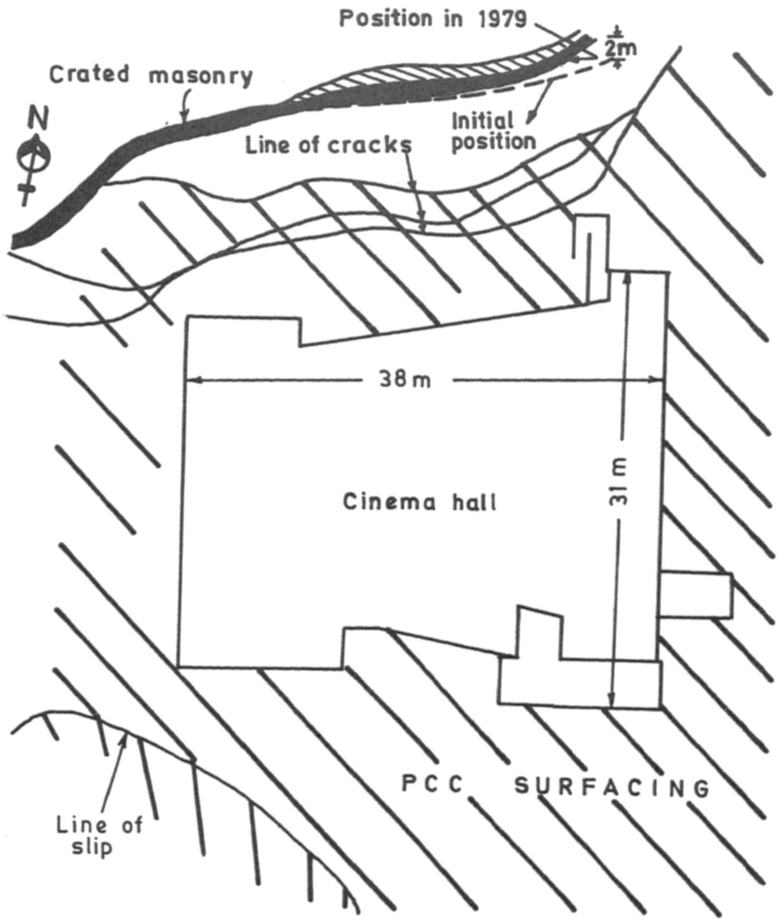

Fig.3 Some Details of Distress

Fig.4 A View of the Slide

the northern and southern sides, low values of SPT ranging from 3-15 upto 3.0 m depth were observed. In general, the values of SPT observed at depths more than 3.0 m were greater than the average values obtained upto 3.0 m.

Direct shear tests (undrained) were conducted on samples at natural moisture content. The degree of saturation of these samples worked out to be about 80-90%. The results of these tests gave values of cohesion, c varying from 0.2-0.6 t/m^2 with an average of $c = 0.35\ t/m^2$ and the values of angle of friction varying from 22-30 degrees with an average of $\phi=25^{o}$. The combination of c and ϕ obtained are given in Table 1.

TABLE 1. The Results of Direct Shear Test

Sl No	Value of c t/m^2	Value of ϕ degrees
1	0.6	22
2	0.3	23
3	0.3	26
4	0.2	30

STABILITY ANALYSIS

The values of c and ϕ were obtained by conducting shear tests on samples collected at a depth of 1.5 m. It was realised, particularly in view of larger values of SPT observed at depths deeper than 3.0 m, that the values of c and ϕ obtained may not be representative values for the complete slope. Therefore, it was decided to back analyse the slope with a factor of safety of 1.0 and obtain the values of c and ϕ required to be mobilized. The stability was evaluated judiciously based on the results of the back analysis, the test values of c and ϕ, the variation of SPT and the location of the critical slip circle.

The back analysis (Singh and Ramasamy, 1979) was carried out using Bishop's simplified method. The site is located in an earthquake prone zone and therefore the Bishop's equation for factor of safety was modified to take into account the earthquake forces. It is assumed that the effect of earthquake is to induce an additional vertical force of α_v times the weight of the slice (α_v - coefficient of vertical acceleration and a horizontal force of α_h times the weight of the slice(α_h - coefficient of horizontal acceleration. The modified equation is given by,

$$F = \frac{\Sigma\,(cb + W\tan\phi)m}{\Sigma\,(W\sin\alpha + \alpha_h W'\cos\alpha)} \quad ..(1)$$

where b = width of slice, F = factor of safety, W' = weight of a slice $W = (1 \pm \alpha_v)W'$, α = slope angle of base of a slice and

$$m = \frac{\sec\alpha}{1 + \tan\phi\,\tan\alpha/F}$$

Equation 1 is applicable when pore water pressure is absent within the sliding mass. Rearranging the terms, Eq.1 may be written as,

$$c = \frac{\Sigma\, W'F\alpha_h\cos\alpha + F\sin\alpha(1+\alpha_v) - \tan\phi\,(1+\alpha_v)m}{\Sigma\, b\, m} \quad ..(2)$$

A slip circle is assumed and the sliding mass is divided into a number of slices.The stabilising forces (the numerator of Eq.1) and disturbing forces (the denominator of Eq.1) are computed for each slice. Using Eq.2, for an assumed value of F, values of c are computed for a set of values of ϕ. Similar computations are done for a number of assumed slip circles and combinations of ϕ and c are obtained. The potential slip circle is the one in which the cohesion mobilised is the maximum for any value of ϕ. (If in a slip surface, the mobilised cohesion is greater than that in an another slip surface, it means the ratio of disturbing force to stabilising force is higher in the former slip surface. Thus, the slip surface in which the cohesion mobilised is maximum, is the potential surface)

The Indian Standards 1893-1975 recommends a value of $\alpha_h = 0.08$ for use in seismic analysis. However, considering the importance of the structure, $\alpha_h = 0.12$ and $\alpha_v = 0.06$ are used in stability computations. The cross-sections of the slopes 'CC' and 'DD'(Fig.1) are shown in Fig.5. This gives the ground slopes as was existing in 1979, i.e. the time of investigation. Unfortunately, no detail was available regarding the ground slope that existed immediately after construction and the actual location and the dimensions of the retaining walls. Therefore, the analysis was carried out to examine the stability of the slope that was existing in 1979.

The upper parts of the northern and southern slopes had experienced slips. Therefore, the analysis was carried out separately considering only the upper part of the slope where the slips have been taken place(A to B in Fig.5) and the complete slope(A to C in Fig.5). The slopes were analysed assuming that pore pressures are absent in the sliding mass. The results of the back analysis are given in Fig.6.

The results of the back analysis show that for the test values of c and ϕ obtained, the factor of safety should be more than 1.0 both for the upper slope and for the complete slope. Further, it was found that the critical slip circle is located much deeper than 1.5 m below the slope surface where it is likely that the values of c and ϕ are larger than test values in view of the higher SPT values observed at greater depths. Therefore, it was concluded that the existing slope is stable and the failure is confined only in the deposit of filled up soil.

REASONS FOR FAILURE AND REMEDIAL MEASURES

It is suspected that the soil excavated at the time of development of the site was dumped along the slopes of the hillock. Low values of SPT observed upto about 3.0 m particularly in bore holes located in the upper portion of the northern and southern slopes give evidence to above view. The failure that took place in 1974 could be due to the rain water seeping into the loose excavated material causing additional lateral pressure on the retaining walls. Subsequent distress observed could also be due to the inadequate drainage measures.

A well laid out surface drainage system was suggested to drain out the rain water to the natural rain water channel at the toe of the hillock. Further, as there was a possibility of rain water seeping into the ground through the loose top soil cover and cracks which have developed during the earlier slips, counterfort walls made of loose stones along the slope were suggested to drain subsurface water(Fig.7). These walls are 1 m thick and buried by about 2.0 m into the ground. The slopes remained stable without any further distress after the above measures have been implemented.

CONCLUSIONS

The analysis indicates that the slopes on the northern and southern boundaries of the cinema hall shall remain safe when provided with proper rain water drainage arrangements. The

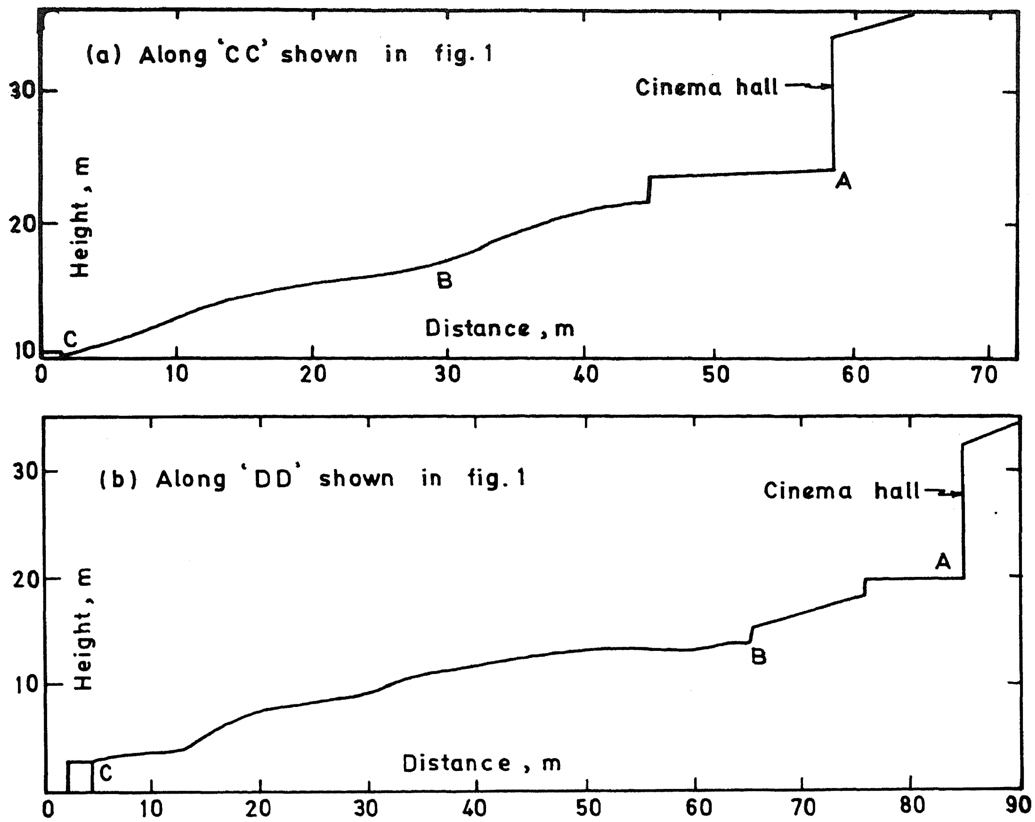

Fig.5 Cross-Section of the Slopes Analysed

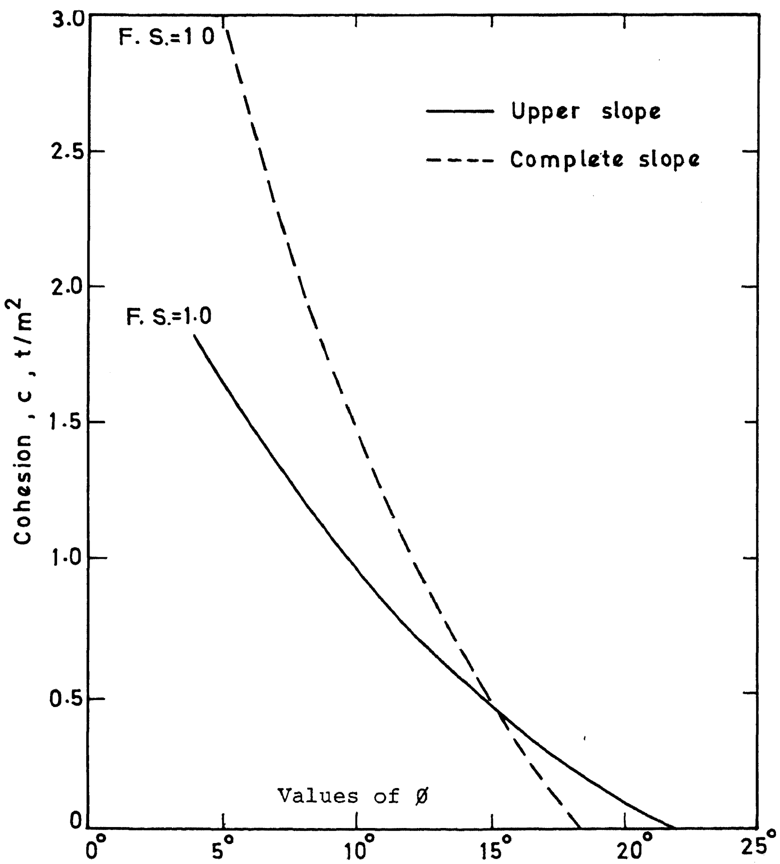

Fig.6 The Results of Back-Analysis

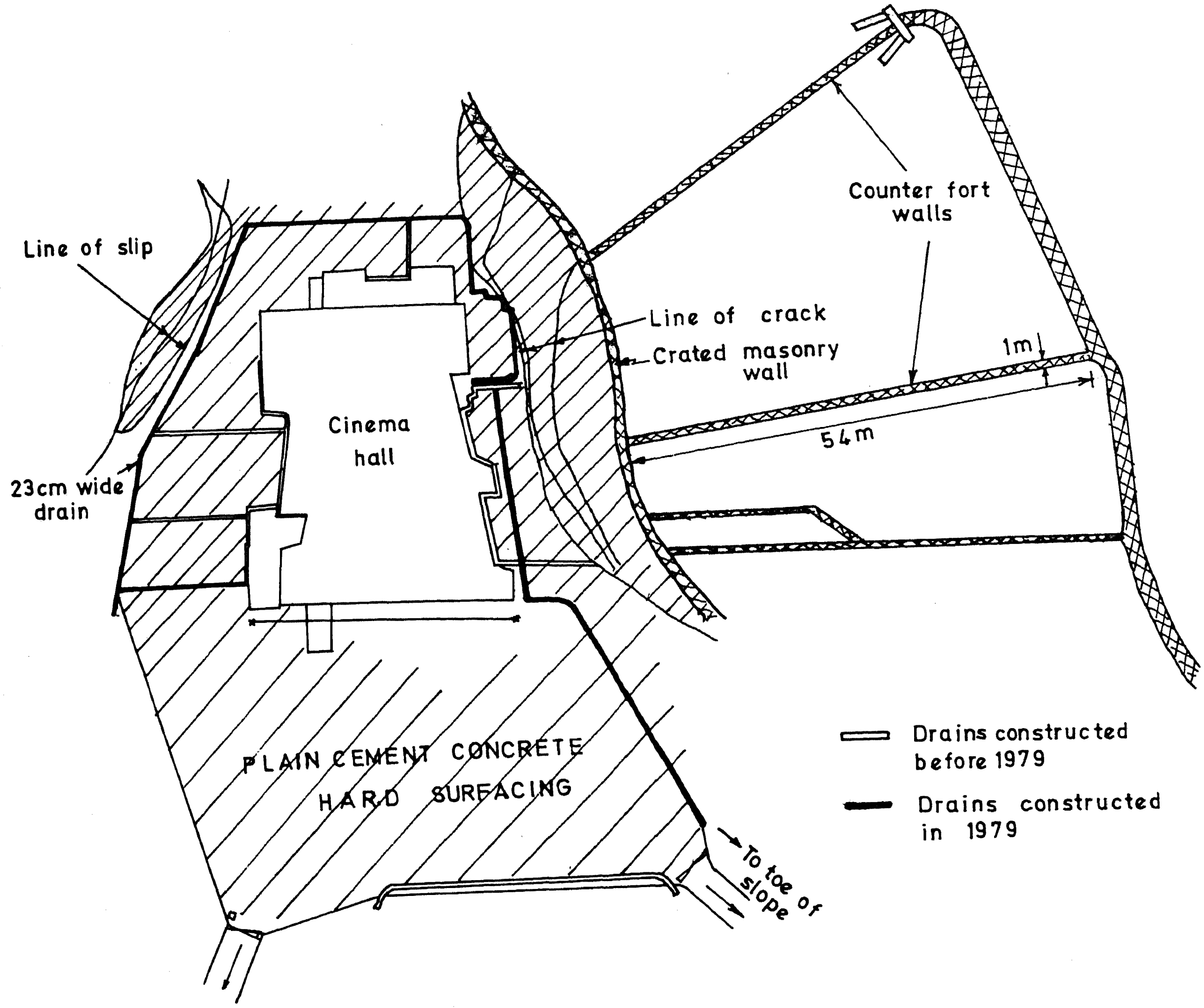

Fig.7 Suggested Remedial Measures

reported slips in the past could be due to the rain water seeping into the loose soil dumped on the slopes at the time of site development.

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