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Damage and Remedial Measures for Buildings on Hill Slopes

Swami Saran
University of Roorkee, India

Gopal Ranjan
University of Roorkee, India

Bhawani Singh
University of Roorkee, India

A. S. R. Rao
University of Roorkee, India

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Swami Saran, Gopal Ranjan, Bhawani Singh

Professors of Civil Engineering, University of Roorkee, Roorkee, India

A. S. R. Rao

Reader in Civil Engineering, University of Roorkee, Roorkee, India

SYNOPSIS The paper deals with details of geotechnical investigations carried out for evaluating the reasons of damage to the buildings resting on hill slopes in northern region of India and also to evaluate the soil parameters for designing retaining walls and other remedial measures for preventing further damage to structures. Three causes of failure were identified namely (i) unstability of slopes, (ii) improper design of retaining walls and (iii) differential settlement of structures. Shear strength parameters of the soil mass required for the analysis of slope stability and stability of retaining wall sections have been chosen based on three methods of investigations viz. large scale direct in-situ shear tests, plate load tests and the back analysis method. The stability analysis of various sections of slopes have been carried out and measures for strengthening of slopes have been suggested.

INTRODUCTION

The site is located in the northern part of India in the foothills of Himalaya and it falls in seismic zone IV. The main construction consists of single and double storeyed residential buildings. The building walls were made of random rubble masonry with sloping roofs of G.I. sheeting. Retaining walls (banded or with dry stone packing) were also to retain slopes. An year after construction some of the buildings developed cracks and a few of the retaining walls failed (Fig.1) damage suffered by the structures built at the site.

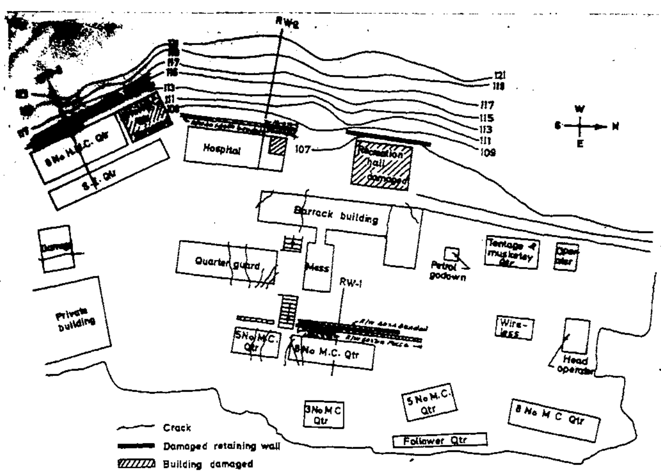

Fig.1 Site Plan

First, the instability of the slopes in the region has to be considered. If the slopes are not stable under the worst combination of field conditions, slides initiate at critical locations and the volume of the moving debris may cause damage to structures located adjacent to the slide. Thus it is necessary to have a knowledge of the shear strength parameters of the soil mass and the drainage conditions. Secondly, failure of breast walls behind the Married Head Constables' Quarters, below the Mess Hall, behind the hospital and the building initiated in the breast wall in front of the Quarter Guard building(Fig.1) indicates the necessity of designing retaining structures in a more rational manner taking into account the shear strength parameters of the existing natural backfill and the surcharge angle of the backfill. Thirdly, the cracks noticed in the walls of the Quarters Guard building and other structures suggest the possibility of differential settlement due to variation in compressibility behaviour of soils under different parts of the structures.

GENERAL DESCRIPTION OF THE SITE AREA

The site is located on the easterly sloping face of the north-south running ridge. With the slope varying from about 30-35° to almost vertical at places. The steeper slopes are towards the northern side.

Initially the area was a cultivated land with terrace shaped topography. But with the development of the site and subsequent slides, the hill slopes have become steeper. The construction of buildings have added additional load on these slopes which have ultimately become very unstable.

The clayey and sandy soil which forms a shallow cover over the phyllitic rocks, have a tendency to slide down these steeper slopes. This slow but active sliding of the soil mass is indicated by the tilting of trees on the slopes (Fig.2).

Fig.2 Tilting of Trees on Slope

The phyllitic rocks have most unfavourable dip of foliation plane towards the slope direction at angles less than the general ground slopes. Added to this are the joints in the rocks which also dip towards the building complex.

The area has poor drainage system. The surface water during rains has a tendency to flood the site area. In the cold winters the rain water which percolates through the joints and foliation planes of the rocks gets frozen, particularly in the upper surfacial parts. This freezing of water might have caused the joints to open up and the development of new cracks.

All these slide providing factors have acted together to make the area most vulnerable to slides.

TEST PROGRAMME

Tests were carried out to determine the following properties:

Field Density

The in-situ unit weight of the deposit is important for the design of foundations. Since the size of particles in the deposit increased with depth the in-situ density was determined by excavating pits and finding out the weight of excavated soil and the corresponding volume of pit. An average in-situ unit weight of 1.80 g/cm^3 was noted.

Shear Parameters

In view of the fact that the soils at site consist of boulders and gravels mixed with fine-matrix, in-situ shear tests are necessary (Prakash and Ranjan 1975).

In-situ direct shear tests at two different locations (Fig.1) on square samples of area 5000 sq.cm and 900 sq.cm in plan (Ranjan, Prakash and Srivastava, 1977) were carried out at normal stresses ranging from 0.45 kg/cm^2 to 0.90 kg/cm^2. The Mohr's envelope from the in-situ shear tests is plotted in Fig.3. The results indicate an average value of angle of internal friction $\phi = 22.4^{\circ}$ with a cohesion $c = 0.075\ kg/cm^2$.

The allowable bearing pressure is determined through vertical plate load tests carried out as per Indian Standard IS:1888-1971. Four vertical plate load tests were carried ou using 30 cm x 30 cm size plate in pits 1.5 m x 1.5 m in plan. These tests were carried out at a depth of 1.25 m at four different locations (Fig.1). Figure 4 shows a typical load intensity vs settlement curve from vertical plate load test at location P-5. Similar curves for locations P-6, P-7 and P-8 were obtained.

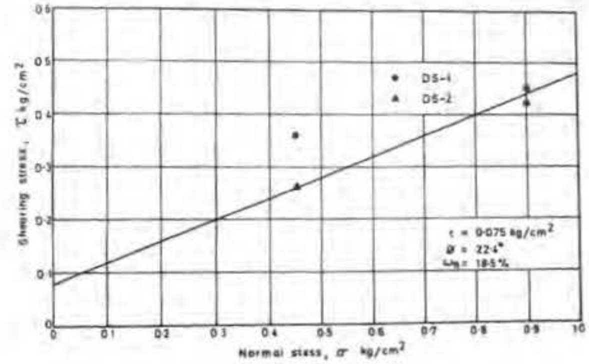

Fig.3 Normal Stress vs Shear Stress Plots

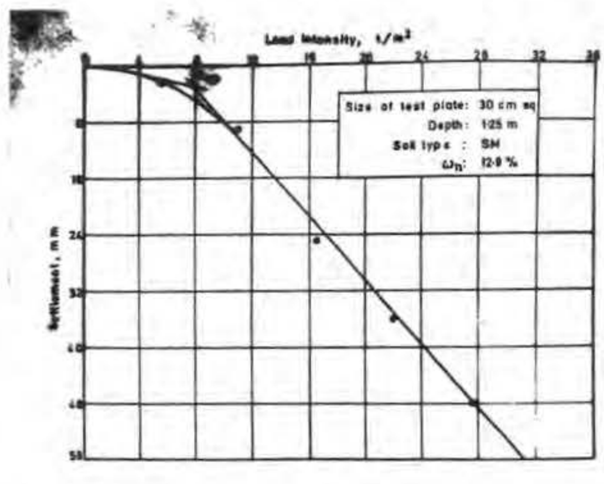

Fig.4 Load Intensity vs Settlement Plot(VPL-5)

From load-settlement curves the failure load of the plate is obtained by intersection tangent method (Leonard, 1962). The values of ultimate bearing capacity of plate, q_{up}, so obtained are listed in column 4 of Table 1.

From a perusal of table 1 and the test plots, the following observations can be made:

a) All the soils tests were non-plastic in nature.

TABLE 1. Results of Vertical Plate Load (VPL) Tests*

Test pit	Depth (m)	Type of soil	Ultimate bearing capacity of test plate q_{up} (t/m^2)
P-5	1.25	SM(ML)	7.8
P-8	1.25	SM	13.75
P-9	1.25	Weathered phyllite rock	Failure not observed upto 80 t/m^2
P-10	1.25	SM	7.4

*Size of test plate used:30 cm x 30 cm

b) Excepting in P-9, settlement under the test plate was considerable in all cases indicating that the soils are likely to be fill material.

c) Results of the test P-9 was quite different as the test plate happened to rest on weathered phyllite at this location.

By taking the average of ultimate bearing capacity and assuming a unit cohesion c of 0.07 kg/cm^2, the angle of internal friction, ø for the soil was worked out to be 28^o.

STABILITY OF SLOPES

Back analysis of slopes ia convenient method of indirect assessment of the strength parameters (Prakash et. al., 1980). It is particularly reliable when slope is in failing condition when the factor of safety may be assumed to be equal to unity.

Following two cases have been analysed:

Slope Along Nala

Figure 5 shows a slope along nala in incipient failure condition. Several tension cracks had developed and slope may collapse in near future.

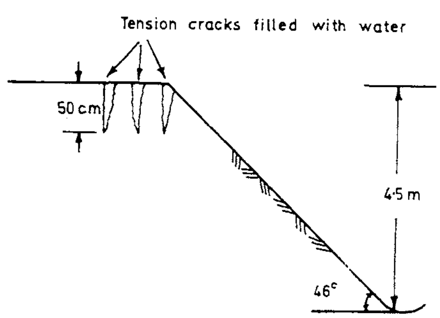

Fig.5 Slope along nala

Hoek (1970) developed charts for factor of safety for slope considering tension cracks. The following assumptions have been made in developing these charts:

(a) Slip surface is circular
(b) Soil is homogeneous and isotropic
(c) Soil obeys Coulomb's law
(d) There is no progressive failure
(e) Tension crack is full of water

Hoek defined two functions X and Y (Eqs.1 and 2) for estimating the factor of safety from these charts (Fig.6).

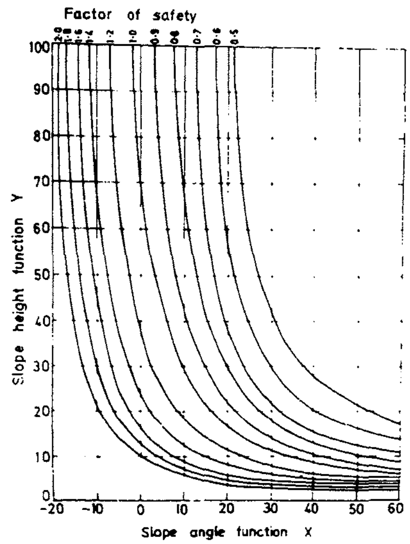

Fig.6 Slope Design Charts

$$X = 1 - 1.2\,\phi \qquad ..(1)$$

$$Y = \left[1 + \left(\frac{i - 10}{100}\right)\frac{z_o}{H}\right]\frac{\gamma H}{c} \qquad ..(2)$$

Where i = angle of the slope, z_o = depth of tension crack assumed filled with water, γ = unit weight of soil, H = height of the slope, c = cohesion of the soil, ø = angle of internal friction of the soil.

Equation 2 takes into account filling of crack with water and Eq.1 is applicable where there is no water table. In the present case taking i = 46^o, z_o = 50 cm, γ= 1.80 g/cc and H = 450 cm, computations were made (Table 2). Column 1 shows the probable values of cohesion for which the corresponding angle of internal friction is to be evaluated. Substituting the value of c and other parameters in Eq.2, the value of function Y is computed. The same is listed in Col.2 (Table 2). For a factor of safety of unity and the computed value of Y, the corresponding value of the function X is read from the chart (Fig.6).The angle of internal friction is then determined from Eq.1 using the value of function X(Tab.2)

Slope Behind Hospital Building

Figure 7 shows the cross-section of the slope (after failure) behind the Hospital building. The mode of failure appears to be 'block type'

TABLE 2. Back Analysis of Slope of Nala

c (kg/cm^2)	Y	X	ϕ (degrees)
1	2	3	4
0.07	12.1	27.0	16^o
0.06	21.0	15.0	26^o
0.025	34.0	8.0	31^o

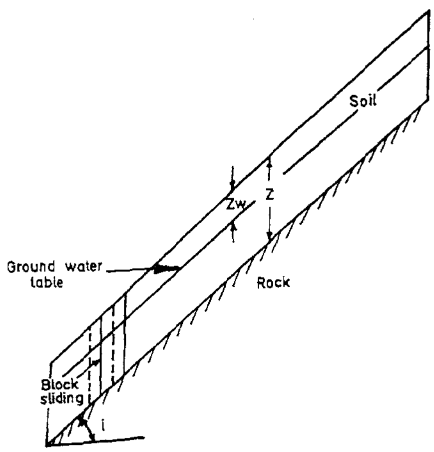

Fig.7 Slope behind Hospital Building

rather than sliding along a curved surface. Analysing the slope, Coates (1969) developed Eq.3 to be satisfied for the failure of slope to occur.

$$\frac{c}{Z} = \cos^2 i \left[\tan i - \left\{ \left(1 - \frac{Z_W}{Z}\right) \frac{\gamma_W}{\gamma} \right\} \tan \phi \right] \quad ..(3)$$

Where Z = average vertical depth of overburden of soil, Zw = vertical depth of water table from ground surface, γ = unit weight of soil, i = angle of slope or rock profile, γ_W = unit weight of water, c = cohesion of soil, ϕ = angle of internal friction of soil.

In the present case, Z varies from 200 cm to 300 cm, Z_W is taken to be zero since ground water table was not seen to exist, γ = 1.80 g/cc, i = 30^o and γ_W = 1.0 g/cc. Using these values computations are made for two different values of cohesion, c and for two different values of overburden. The corresponding values of angle of internal friction, ϕ are computed. The same are summarised in Table 3.

TABLE 3. Back Analysis of Slope behind Hospital Building

Cohesion, c (kg/cm^2)	Angle of internal friction ϕ (degree)	
	Z = 200 cm	Z = 300 cm
1	2	3
0.07	15^o	22^o
0.04	23^o	26^o

Equation 3 proposed by Coates(1969) can be modified to take into account the earthquake forces. The modified equation is as given below

$$F = \frac{\frac{c \sec_i^2}{\gamma Z} + \tan\phi \left[1 \pm \alpha_v - \alpha_h \tan i - \left(1 - \frac{Z_W}{Z}\right) \frac{\gamma_W}{\gamma} \right]}{(1 \pm \alpha_v) \tan i + \alpha_h} \quad ..(4)$$

Where F = factor of safety, α_h = horizontal component of earthquake acceleration, α_v = vertical component of earthquake acceleration and other notations are as indicated earlier.

On the basis of shear test results, plate load test data and back analysis of slopes (Tables 2 and 3), cohesion, c of 0.04 kg/cm^2 and angle of internal friction, ϕ of 26^o have been adopted for the slope. Substituting various parameters in Eq.4, the overall factors of safety for the slopes have been calculated (Table 4). The table indicates that the slopes are unstable particularly under earthquake conditions. It is, therefore necessary that protective measures in the form of flattening the slopes or construction of retaining walls of adequate cross-section are adopted. A typical retaining wall designed for this purpose is shown in Fig.8.

TABLE 4. Overall Factors of Safety of Slope

Location	Factor of safety under static conditions (α_h=0, α_v=0)	Factor of safety with earthquake forces (α_h= 0.08, α_v=0.04)
1	2	3
Slope behind married constables quarters building (Z=420cm) (Fig. 5)	0.80	0.68
Slope behind hospital building, (Z=425 cm) (Fig. 7)	0.92	0.81

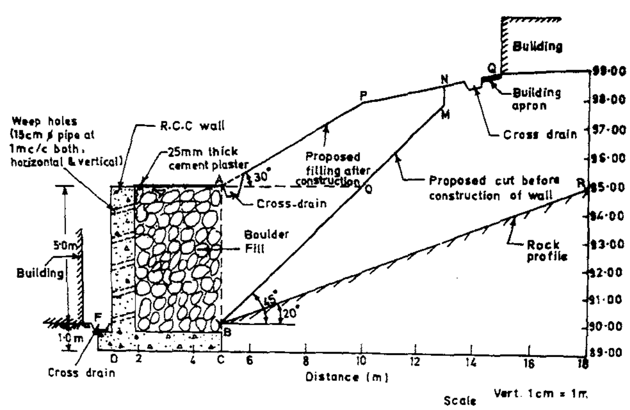

Fig.8 Typical Section of Retaining Wall

DRAINAGE SYSTEM

The site slopes appreciably from the west towards the east. Pauri town, being a hill town, receives quite a large amount of rainfall is expected during monsoon months. If this precipitation is not properly led away, its effect on the constructed buildings and the appurtenant structures like the retaining walls and breast walls, can be manifold. Large ingress of water inside the backfill behind retaining walls means appreciable increase in the thrust on the back of the retaining walls, especially if the weep holes provided are clogged up and not functioning properly. Constant soaking of the soil around and below the buildings may also cause the buildings to settle appreciably if the soil is an unconsolidated one. There are reasons to believe that these two factors may well be operating at the site resulting in distress to structures. It is, therefore, becomes necessary to lay out a series of well designed drains to carry the anticipated maximum discharge and lead it away from the vicinity of the site.

The proposed system of drainage is indicated (Fig.9) and envisages the following:

(a) Provision of a main drain of sufficient capacity along the existing drain on the southern side running from west to east. The drain will be laid at the natural slope of the ground.

(b) Provision of a drain near the crest of the upper retaining wall proposed behind 8 no and 4 no H.M.C. (Fig.9) quarters. This drain will slope in directions indicated by the arrow heads on the plan because of reasons of topography. The drain will run all along the length of the retaining wall.

(c) Provision of drains at the crest of the proposed lower retaining walls viz.i) i) immediately behind the 8 no and 4 no H.M.C. quarters and ii) behind the hospital and the recreation hall buildings.

(d) Provision of drains near the toe of retaining walls indicated at (c) above

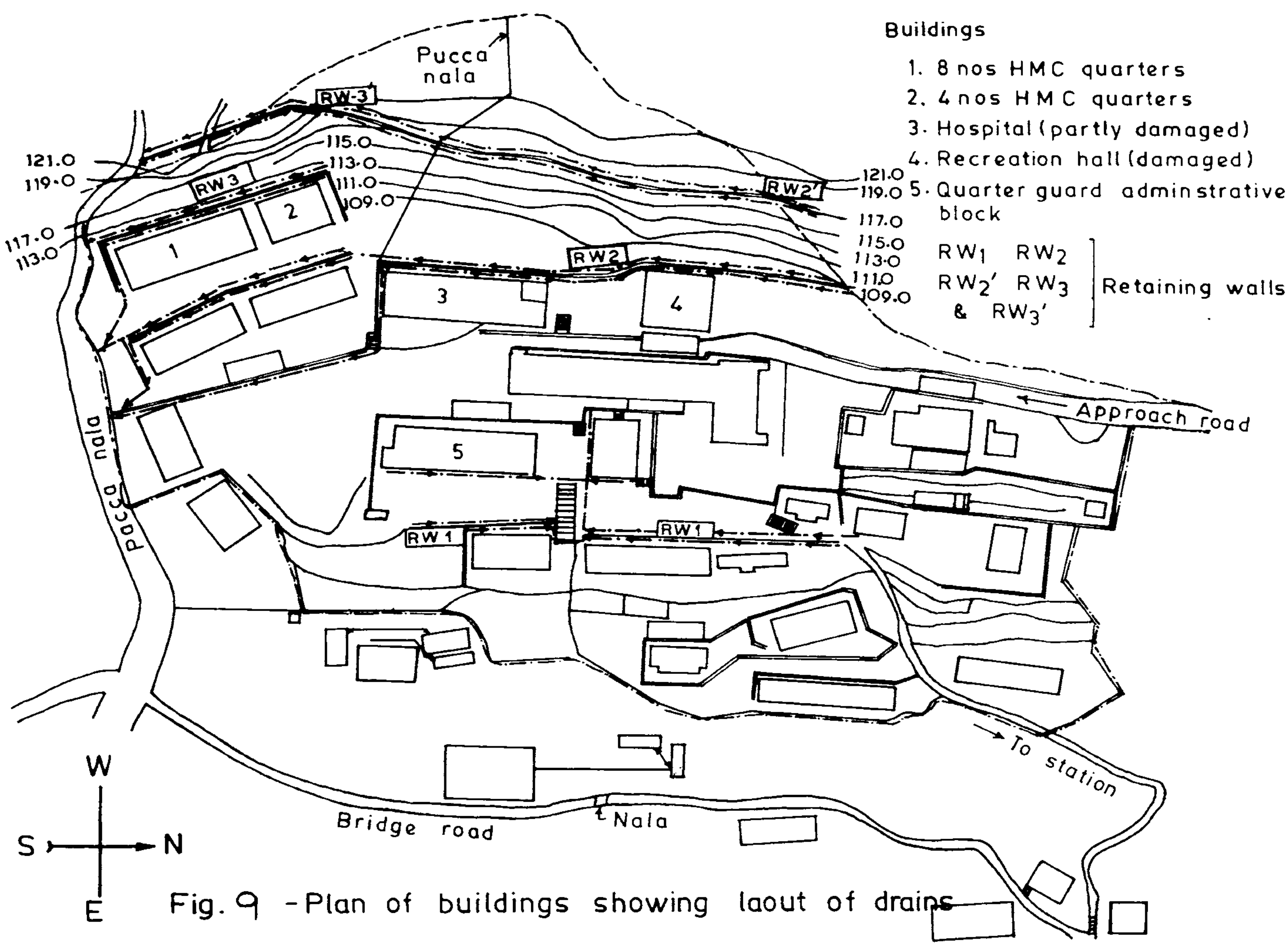

Fig. 9 - Plan of buildings showing laout of drains

in order to collect water emerging from the weep holes in retaining walls. The inter-connection between the drains proposed above is indicated on the plan. All the discharge from drains proposed, at (b), (c) and (d) is collected in drain proposed at (a).

(e) Provision of drains around the administrative block and the mess connected together and then discharging into a main drain of sufficient capacity running along the natural slope of the ground in the west to east direction.

(f) Provision of drains near the crest of the proposed retaining walls in front of the administrative block and the mess.

(g) Provision of drains near the toe of the retaining walls proposed at (f) above.

From the above it can be seen that one set of the proposed drains run either along the N-S direction or along the S-N direction. These drains discharge into 2 main drains, one at the southern side of the site running from west to east and the other in front of the Mess Building from west to east. Both the main drains can be laid along the natural ground slope; the N-S and S-N drains may be laid at a slope of 1 in 300. All drains shall be pucca construction and finished with cement mortar.

CONCLUSIONS

The technique of back analysis is a promising tool to estimate shear parameters of slopes. Water percolation in the backfill was found the main cause of damage of retaining walls. Remedial measures in form of terrace development, retaining walls and adequate drainage system were found adequate.

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