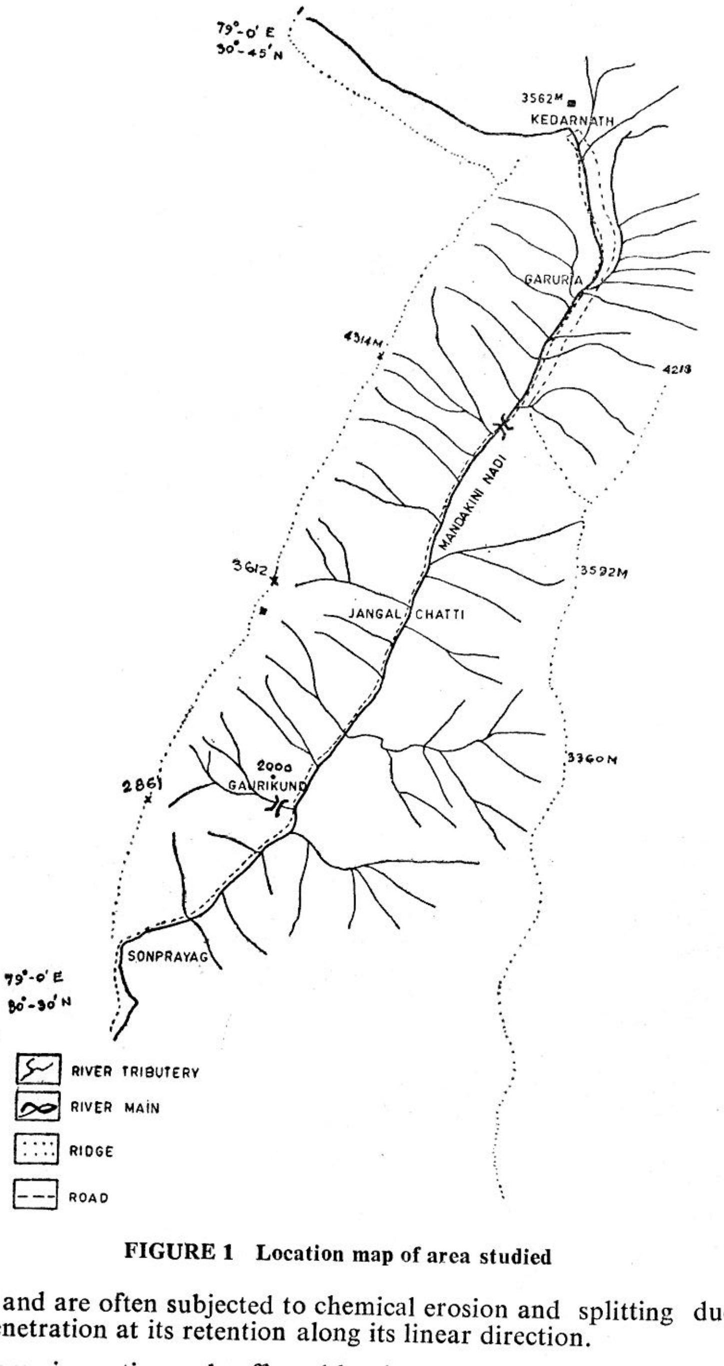

FIGURE 1 Location map of area studied

strength and are often subjected to chemical erosion and splitting due to water penetration at its retention along its linear direction.

The area is continuously affected by the tectonic disturbances due to presence of thrusts (Figure 4). As the Himalayan mountain system is possibly rising against the down cutting three river terrace levels in the Mandakini river vally leads to the neotectism. These tectonic and Neo-

FIGURE 2 (a) Banded Stone masonry wall

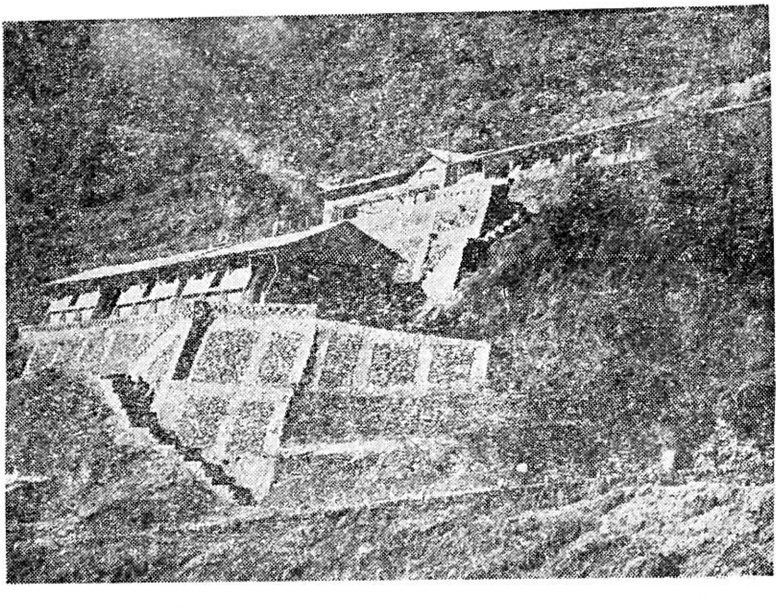

FIGURE 2 (b) Collapsed offset retaining wall

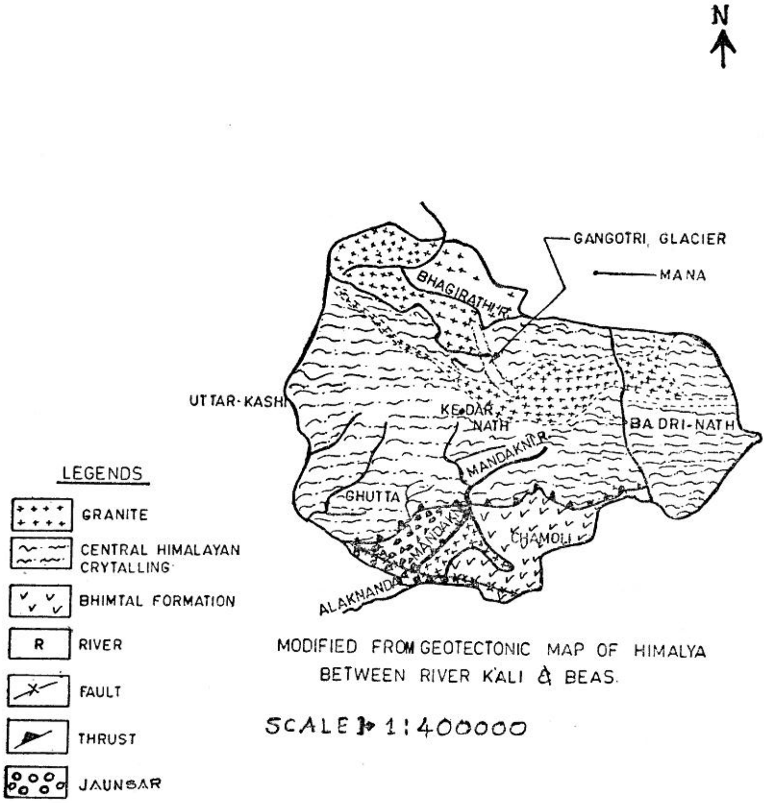

FIGURE 3 Geotectonic map of area

tectonic disturbances had resulted into many folds and faults (Figure 5) making most of the hill slopes of the area unstable and succeptible to frequent land slides.

Design Concept

The design of the curved stone masonry retaining walls have been arrived based on the following assumptions :

1. $c'-\phi'$ soil strength parameters are known or determined in actual slopes.
2. Actual lateral earth presure developed are not more than that evaluated by using classical Rankine's earth pressure theory.
3. Perfect surface drrinage and seepage conditions exist and pore water pressure and seepage pressure are expressed in terms of non dimensional factor excess seepage pressure is assumed to be dissipated through dry pitched stone windows and through weep holes.
4. There exist no rigid base and thus massive gravity retaining wall is undesirable to prevent down slope failure.

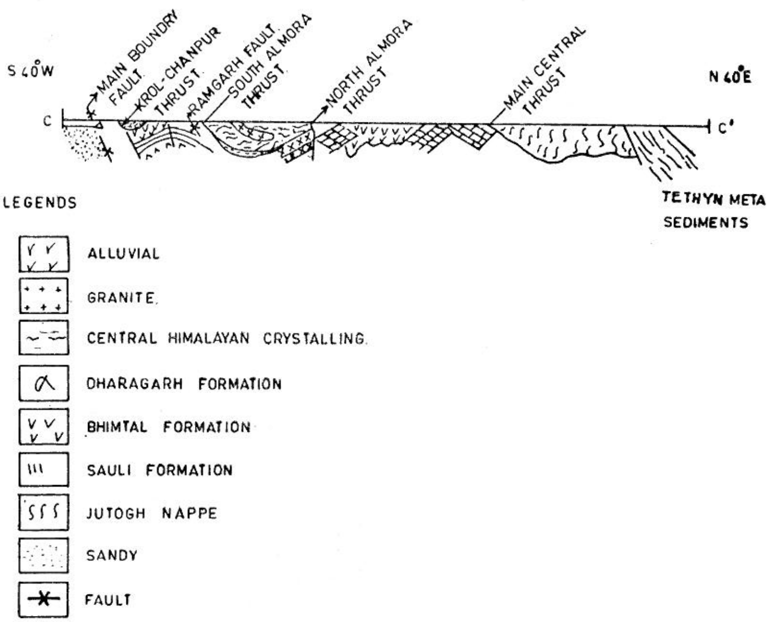

FIGURE 4 Geological section of Kumaon Himalaya

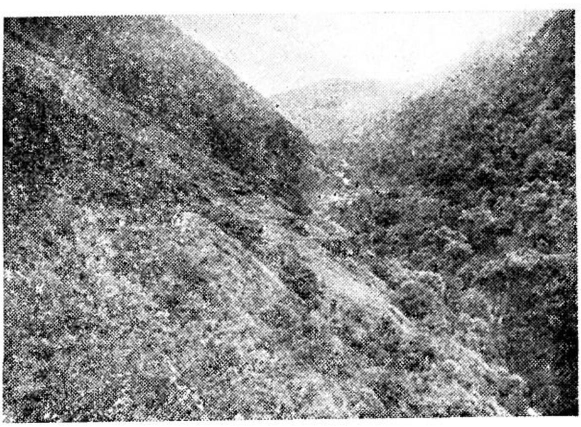

FIGURE 5 Hill slopes of the area

5. Backfill and curued stone masonry wall act together as two insepa rable parts due to curving of the inside face of the wall.

Knowing c'—ϕ' soil strength parameters active (P_A) and/or passive (P_P) earth pressures that wall might be required to retain at various levels are calculated by using following equation or by any rational approach.

$$P_A = \frac{\gamma_s H^2}{2} \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' H \tan\left(45 - \frac{\phi'}{2}\right) \quad \text{...(1)}$$

$$P_p = \frac{\gamma_s H^2}{2} \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' H \tan\left(45 + \frac{\phi'}{2}\right) \quad \text{...(2)}$$

where symbols are given in notations.

Dimensions of the uniformly curved wall or curved wall made up of various straight slops towards backfill at different levels (Figure 7 a) based on similar geological conditions and experience are assumed next and corresponding gravity forces and their line of action determined.

The slope of the resultant at various level from the vertical is computed or determined graphically assuming lateral earth thrust P_A or P_p acting parallel to the slope of the backfill at two third depth from the top of the portion of the wall under consideration as shown in Figure 7 (*a*). The profile of the curved wall is drawn such that vertical mid-section line at various selected levels passes nearly through the points where resultants have crossed the base lines of the selected levels. Wall profile obtained as above, if it is not feasible for existing slope, then procedure is repeated for various assumed dimensions of the curved wall and finally the most suitable profile is selected as shown in Figure 7 (*a*) and (*b*).

Slopes are generally made of loosely packed rocks in soil mass and are likely to result in a series of plans in lieu of a smooth continuous curved failure and are conveniently analysed by sliding block method, In this method, it is usual to divide the sliding mass into two or three large sections or wedges (Figure 7 *b*). The upper and lower wedges are respectively called the active and the passive wedges, In a three wedge system, the middle wedge is generally referred to as the sliding block. Considering the equilibrium of the sliding block (Figure 7 *b*) the factor of safely against sliding is obtained as follows

$$F_s = \frac{W(1-r_u)\tan\phi' + c'L}{P_A - P_p} \qquad \ldots(3)$$

where $r_u = \frac{U}{W}$ = non-dimensional factor

U = unit pore water pressure $a_s \times L$

W = Total weight of the sliding block

Steady state seepage pressure in the same direction as that of the gravitational force of the soil increased the stability by increasing the intergranular pressure and is of little concern to the engineer. However adequate number of dry stone packed windows and weep holes are provided to dissipate excess seepage pressure building up during rainy seasons. Pore-water pressures are succeptible to seasonal variations and are accounted in terms of non-dimensional factor 'r_u' in the analysis. For all practical purposes and based on the topography of the area the 'r_u' factor is linearly related to factor of safety and its range of variation may be taken from 0.0 to 0.4 as worked out in the table.

To safeguard against land slips during excavation foundation depth is normally exacavated not more than 1 M below the ground level to prevent base for frost action and scouring of the toe of the wall from rain water penetration. Base of the wall is usually broadened and made inclined deep towards backfill to keep the stresses on soil evenly distributed within its bearing capacity and to achieve desired factor of safety against sliding.

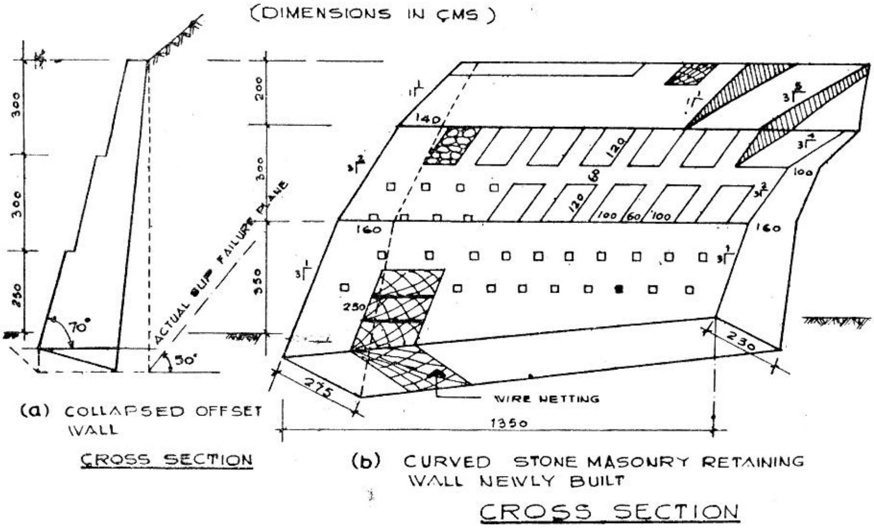

FIGURE 6 Details of offset wall and curved stone masonry retaining wall.

$P_A - P_P$	W	C'	φ'	L	r_u	F_s	REMARKS
					0.0	1.6	
					0.1	1.44	
26.10	69.13	0.08	31°	6.1M	0.2	1.28	$F_s = \frac{W(1-r_u)\tan\phi' + c'L}{P_A - P_P}$
TONNES	TONNES	TONNES/m²			0.3	1.12	
					0.4	0.97	

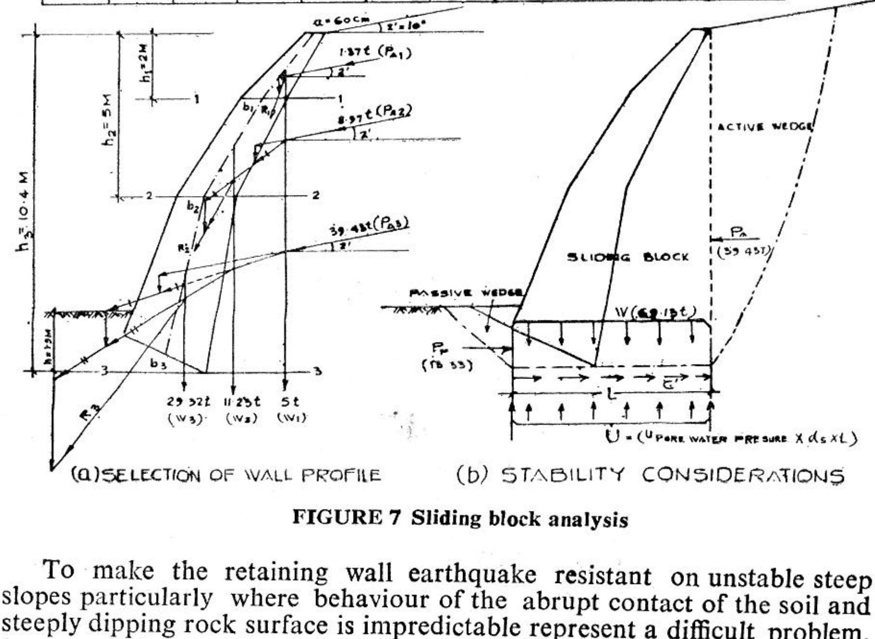

FIGURE 7 Sliding block analysis

To make the retaining wall earthquake resistant on unstable steep slopes particularly where behaviour of the abrupt contact of the soil and steeply dipping rock surface is impredictable represent a difficult problem, however, reinforced bands both horizontally and vertically, installation of the earth and stone anchors at suitable spacing or wire netting the wall are considered adequate measures.

Construction

The wall construction on steep unstable slopes particularly where space limitations exist or caused due to adjacent piece of land is not be encroached pose a formidable task. Probably wall construction from top downward. The Brazion method (Roy et al, 1978) by excavating benches may prove good solution but to install earth or stone anchors where equipments are scarce and are not accessible, do not appear feasible.

It is found practical to excavate the slopes in 1.5 M to 2 M strips and construct the wall either from one end or from both ends upto safe height before exacavating the adjacent portion. Where it is impractical to construct the wall in desired slope, wall is to be strengthened by wire netting at the outer face of the wall and anchoring it to the backfill by inserting spikes at designed spacing (Figure 6*b*). After laying levelling base course often 1 : 4 : 8 Cement Concrete mix of overage 20 cm to 30 cm thick, wall is constructed with good quality of stones in 1 : 6 Cement mortar leaving weep holes at suitable spacings at ground level alongwith about 15 cm loosely packed stones towards backfill to allow seepage of the infiltrated water.

The wall is raised upto 3 M height from the base than about 15 cm thick 1 : 3 : 6 cement concrete band is layed. If the area is earthquake prone RCC band in place of Cement concrete band is recommended. 60 cm wide vertical stone masonry bands in 1 : 6 cement mortar are constructed at design spacing — usually 1 M to 1.2 M centres to centre of height 1 to 2 M and filled with dry packed stones in between them. Construction of band and drystone packing can also be carried out simultaneously. 60 cm thick stone masonry in 1 : 6 cement mortar horizontal bands are constructed over this and the sequence of construction repeated upto top.

For ease in construction the wall is constructed in required number of 2.5 M to 3 M high panels each with the designed safe slope rather than curving it nniformly as shown in Figure 6 *b*. Top of the wall and backfill is made pucca in proper slope to allow quick surface drainage through constructed drains. Surrounding area is also improved to prevent accumulation of water at any place in the vicinity of the top of the wall.

Conclusions

Curved stone masonry unreinforced walls are economical and advantageous over conventional cantilever gravity stone masonry walls and design can be conveniently worked out without the aid of computers.

The curring of the wall towards backfill provides high retention of the backfill thrust at a relatively half or less thickness.

The wall can be constructed in strips of 1.5 M to 2 M wide avoiding the conventional necessity of excavating the entire unsupported slope.

Existing structures near the top of the exacavation can be retained safe without carrying out underpinning operation for their safety.

Foundation excavation in small strips do not require shearing and bracing and excavation can be carried out free of obstruction.

Dismentling and strengthening of existing distressed and buldged retaining walls can be carried out by constructing additional or replaced curved retaining walls without endangering the stability of the retained slope.

Notations

c' = Cohesion intercept.

φ' = Angle of shearing resistance.

P_A = Active pressure on a plane.

P_p = Passive pressure on a plane.

γ_s = density of the soil mass.

H = depth of the base of the levels or foundation from the top.

F_s = Fator of safety.

W = Weight of sliding block based on average density of the masonry and soil mass.

r_u = Non-dimensional factor.

U = Total pore water force acting at the base of sliding block.

L = Length of the base of the sliding block.

a_s = Correction factor for pore water pressure due to steady state seepage condition.

u = Unit pore water pressure.

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