A Large Shear Box for Tests on River Bed Material

by

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Introduction

Two zoned earth and boulder fill dams, 126 meter high on the river Ramganga and 72 meter high on a tributary to close the saddle are being constructed at Kalagarh, Uttar Pradesh. The upstream pervious shell of these dams consists of river bed material. River bed material is fairly a well graded sand, gravel and cobble mixture containing boulder sizes upto 900 mm. It does not contain more than 3 percent of minus 8 No. I.S.S. material. Large quantities of processing plant rejects, containing cobbles of sizes 38 mm—340 mm, (available at processing plant as a residue after screening the river bed material for production of the transition filter, coarse drainage filter and concrete aggregates have been placed in the dam fill in atleast 50 percent width of the upstream pervious shell. The river bed material and the processing plant rejects are placed in 600 and 900 mm thick layers and compacted by two to three passes of 10 Ton Vibratory roller. The effect of passes of roller is substantial in increasing the density of the river bed material as the fines move and pack in the voids between large particles, while in case of processing plant rejects, it is usually small as cobbles in its structure in absence of fines, are supported at the contacts, and compaction only improves arrangement and pattern of packing of cobbles.

A test apparatus using a shear box of size 1200 mm × 1200 mm was devised at the Project Laboratory to test the shear strength of the compacted layers of the river bed material and the processing plant rejects.

Details of Direct Shear Box (1200 mm × 1200 mm)

The details of the large scale shear box apparatus have been shown in Figure. 1. The internal size of the two halves (consisting of 450 mm thick R.C.C. walls) of the box was 1.2 m by 1.2 m square by 0.6 m in height. The upper half of the box was seated on 160 mm dia G. 1 pipe rollers (the annular space being filled in by concrete) resting on the lower half of the box. Two rolled steel joists were centrally placed below the lower half of the box and were connected to top girder assembly by means of two vertical mild steel

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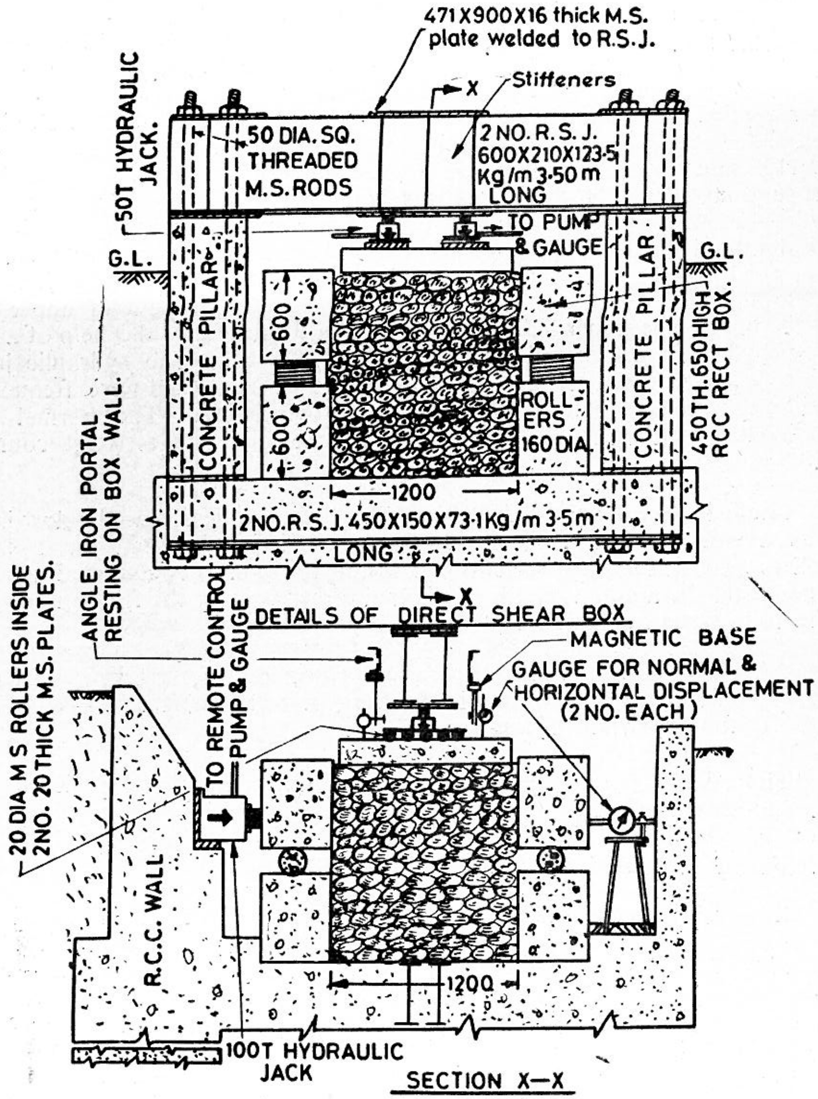

Note – All dimensions are in mm unless otherwise stated.

FIGURE 1 : Detail of Shear Box

bars (of 50 mm diameters) at each end, to provide reaction loading upto 100 Tons on the specimen. The vertical mild steel bars were provided with square threads and were encased in concrete columns. Top girder assembly consisted of two rolled steel joists (size 600 mm x 210 mm x 123. 5 kg/m); the joists were weld jointed to each other by 16 mm thick mild steel plates at midspan and at the ends. Plate stiffeners at midspan and at the ends were also provided. To provide horizontal thrust upto 100 Tons, 3.5 m long R.C.C. wall was constructed parallel to girder assembly.

The test material was filled and compacted by heavy rammers in layers of 500 kg each. To close the gap between two halves of the box at the time of placement and compaction of the test material, a wooden box (of internal size equal to the internal size of the box) was used. The wooden box was bolted at the corners, so that its sides could be unfastened and taken out after application of the normal load on the material. 190 mm thick R. C. C. bed plate (heavily reinforced both by top and bottom bars) of size 1150 mm by 1150 mm square was placed on the test material to uniformly distribute the normal load. Normal load was applied by the help of two 50 Ton hydraulic jacks (remote control type) which rested against the girders. 20 mm diameter mild steel rollers (properly greased) in between the two 20 mm thick mild steel plates were placed below the jacks, so as to keep the hydraulic jacks in true vertical position during displacement of upper half of the box. The upper half of the box was displaced by the help of a 100 Ton hydraulic jack placed against the R. C. C. wall. The hydraulic jacks were attached with dial gauges (least count—1 Ton) and were frequently calibrated by the help of a 100 Ton proving ring. The normal and horizontal settlements were measured by the dial gauges (least count—0.01 mm).

Coefficient of friction of 20 mm dia greased rollers placed below jacks was assumed as 0.02. The coefficient of friction between the 160 dia rollers placed between the two halves of the box was assumed as 0.1. Therefore, the amount of the horizontal force taken by the friction between the rollers was calculated as below :—

$H_f = 0.02\ N + 0.1\ W$, Where H_f=Horizontal force taken by friction of rollers, N=Normal load over the test material, W=Weight of the upper half of the box in Tons.

The value of H_f was deducted from the horizontal force applied by the jack to obtain the net horizontal shear force.

Maximum Particle Size Permitted in the Tests

Research indicates that the maximum particle size of gravel in a triaxial compression test should not exceed 15 per cent to 20 per cent of the specimen diameter and the particle size in direct shear test can not exceed 5 per cent of the minimum dimension of the shear surface without distorting the results (Lewis, 1953 and Zeller, and Wullimann, 1957). However the precise relationship between the maximum grain size suitable for testing and instrument diameter or size of the box is not yet settled. According to COESFELD (1958), the maximum grain size for gravels should be 1/40 of the shear box dimension. SHULTZE (1955) gives 1/5 as limit for single grains. SOWERS and GORE (1961) tested broken sand stone, and mixture of weathered sand stones and sandy clay consisting particle sizes upto 300 mm in a shear box of size 1.8 m x 1.8 m (thus, keeping the ratio as 1/6). The tests on this box were conducted on materials containing particles of maximum sizes upto 200 mm.

Spacing Between the Two Halves of the Box

The spacing between two halves of the box is dependent on the maximum size of the soil particles and the denseness of the soil which is being tested. The parts of the box should be further apart than the diameter of the largest particle to prevent the top half from riding upon a grain which gets between the edges (Lambe, 1951). Because the tests were conducted on the material permitting maximum particle sizes ranging between 75 mm to 200 mm, the spacing between the two halves of the box was kept as 160 mm.

Presentation and Discussion of Test Results

Three tests at different normal stresses were conducted on each grade of material placed at the same density. Normal load at shearing plane was calculated by summing up the loads applied by the hydraulic jack, self load of the jack, R. C. C. bed plate and of the test material over the shearing plane. After stabilisation of normal settlement under the normal force, the horizontal forces were applied in small increments, until loads very near to failure were reached. The normal force was kept constant during the test, by adjusting the jack suitably. The horizontal force and then the normal force was released after completion of the tests and readings of the horizontal and normal settlements were taken to observe the rebound in the material.

TABLE I : Shear test results at normal stresses higher than 1.5 kg/cm²

S. No.	Size of particles in the material					—4.76 mm	Density obtained in direct shear box in gm per c.c.	Angle of shearing resistance	Cohesion in kg/cm
	305 mm to 152 mm	152 mm to 76 mm	76 mm to 38 mm	38 mm to 9.52 mm	9.52 mm to 4.76 mm				
1.	80%	20%	—	—	—	—	1.77	43.5	Nil
	Size 200 to 152 mm								
2.	40%	40%	20%	—	—	—	1.97	42.8	Nil
3.	30%	30%	20%	20%	—	—	2.02	43.0	Nil
4.	—	75%	25%	—	—	—	1.78	45°	Nil
5.	—	50%	20%	20%	10%	—	1.97	44.5°	Nil
6.	—	—	55%	35%	10%	—	1.94	43.5°	Nil
7.	16.5%	28%	11%	7.9%	3.3%	33.3%	2.16	42°	0.178
8.	20%	15%	15%	23%	7%	20%	2.15	38°	0.07

Shear Parameters

The tests were conducted at normal stresses higher than 1.50 kg/cm² on different grades of the processing plant rejects and the river bed material. The results are shown in Table—I. It may be seen from this table that the values of angle of shearing resistance of the processing plant rejects have ranged from 42.8 degrees to 45 degrees; the value of cohesion being zero. The materials shown by serial 1 in Table—I consisted of only two cobble sizes and therefore attained low density, while the material shown by serial 3 and 5 consisted cobbles and gravels and therefore attained a higher density; however the value of angle of shearing resistance has remained in almost the same range in all these cases.

Packing Arrangement of Cobbles

Uniformly sized spheres when arranged immediately over one another vertically in a rectangular arrangement (their centres falling in a vertical line) are in loosest state, their porosity then being $n=0.477$. When uniform sized spheres are arranged one particle above the other rhombically (*i.e.* each sphere resting on two spheres) denser arrangement similar to as obtained after compaction, is achieved, their porosity then being $n=0.412$. The densest state of packing with uniform spheres is attained when one sphere rests upon three or four spheres, their porosity then being $n=0.260$ (Jumkis, 1967). The porosity of cobbles in the tests shown at serial 1 to 4 in Table—I was 0.31, 0.24, 0.21, 0.305. The arrangement of cobbles which were not spherical but of haphazard round shapes in these tests therefore can reliably said to be in the denser state.

Compressive Strength of River Bed Cobbles

The gravels, cobbles and boulders found at Ramganga river bed were of haphazard well rounded shapes and were hard and tough. They were made up of quartzite. The compressive strength tests in 200 Ton Universal Testing machine were performed on five specimens of size 7 cm cube; cut and dressed from cobbles by the help of stone saw machine. The compressive strength of cobbles so determined was found to vary from 837.0 kg/cm^2 to 1337 kg/cm^2 (average value being 1160 kg/cm^2). The cobbles failed by fracture through the whole grain.

Stress Strain Relationship

Figures 2 and 3 show direct shear plots for river bed cobbles. The shear stress versus shear displacement curves are approximately in straight line in their initial portion-indicating elastic behaviour of the material. These curves show several failure peaks. It was noticed that shear stress rises till a peak value, instantaneously followed by a loud sound of fracture of cobbles and sharp drop in the shear stress. The phenomenon is repeated on further displacement of the upper half of the box. The presence of fractured cobbles were noticed during the removal of the material after completion of the tests indicating occurence of crushing phenomenon during shear. Weight of crushed cobbles in the test material after tests was found to be in the range of 3 kg to 10 kg.

Crushing Phenomenon

Studies of SMITH and LIN (1953) in relation to the distribution of stresses due to normal and tangential forces on the contact of rolling bodies revealed that a rolling contact leaves behind it a tensile stress while introducing a compressive stress ahead of it (Figure-4). The tensile stress developed behind the contact is of the order of 1/3rd of the yield stress. When the tensile stress of the material on which rolling is taking place is

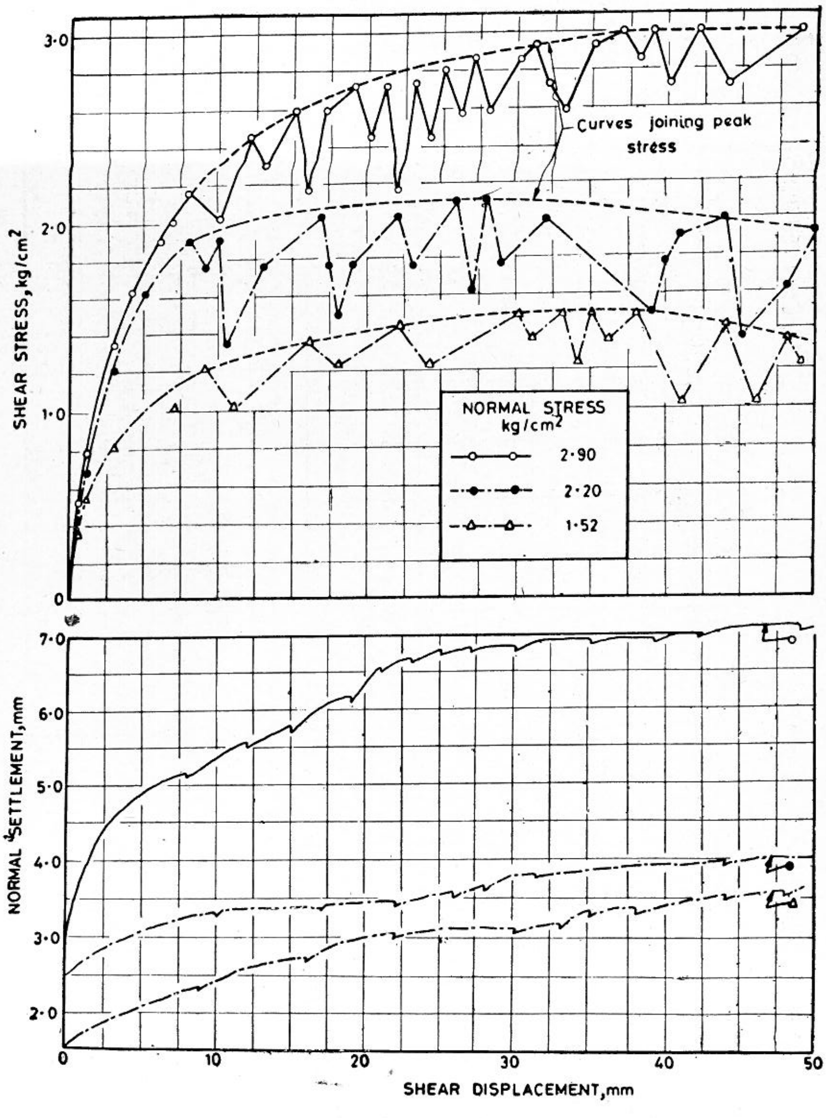

FIGURE 2 : Direct shear plots for cobbles (38 mm to 152 mm)

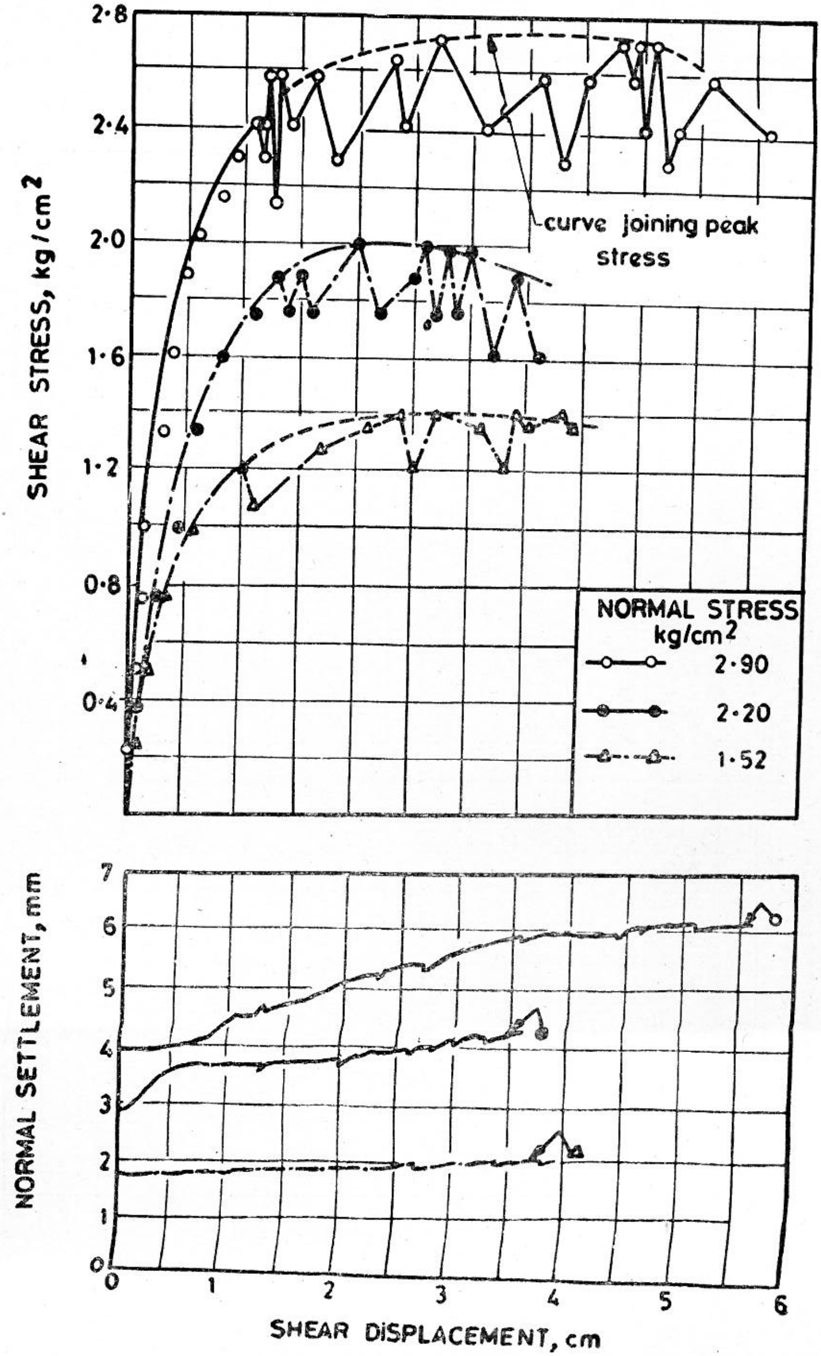

FIGURE 3 : Direct shear plots for cobbles (size 10 mm to 200 mm)

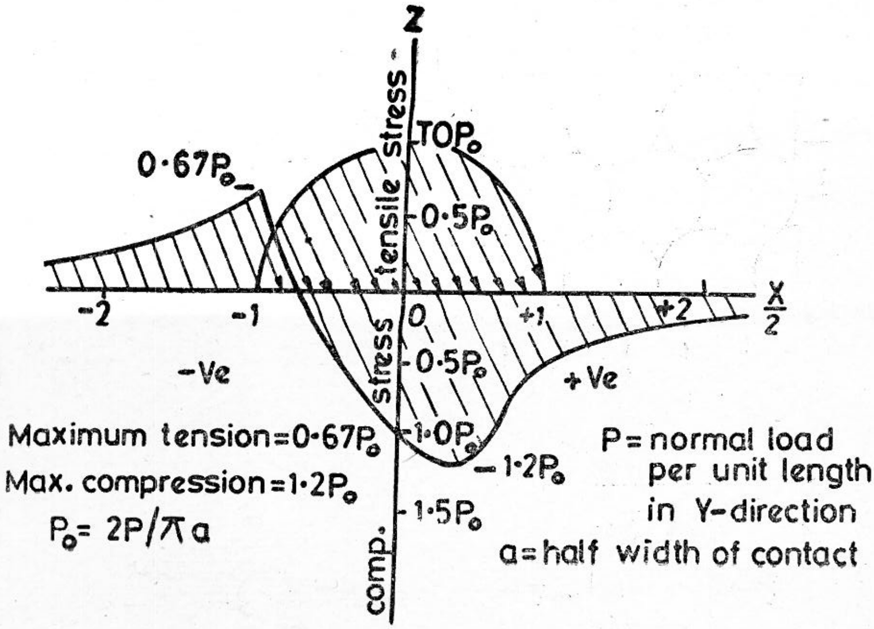

FIGURE 4 : Variation of stress as a load moves past a point (Smith and Lin 1953)

less than 1/3rd of its yield stress, break down of small asperities and angularities may take place. The crushing of granular soils and coarse grained soils under high confining pressures and its influence on strength deformation characteristics have been reported by BISHOP (1965), BRAUNS (1967, 1968), HALL and GORDON (1963), HIRCHFELD and POULOS (1963), RAMAMURTHY (1966, 1969), MARSAL (1967), VESIC and BARKSDALE (1963), VESIC and CLOUGH (1968).

Shearing Process under Normal Stresses Higher than 1.5kg/cm²

The cobbles in its matrix are in close contact with each other as shown in Figure 5(*a*). Figure 5(*b*) shows the possible planes of sliding of cobbles during shear. Figure 5(*c*) shows the normal and tangential stresses being developed at the contact surface of cobbles, when acted upon by a normal stress alongwith shearing stress. During shear, when a soil particle slides over the surface, the shear resistance is given by the mineral to mineral friction angle. However in case of cobbles or other actual soils, the planes through the contact points are inclined to horizontal and in order to have a shear failure between the particles, it is therefore, not only necessary to overcome the mineral to mineral frictional resistance, it is in addition necessary to make particles move up and over one another. Hence the shear failure shall be made up of two components (*a*) one whose magnitude is controlled by mineral to mineral friction angle (*b*) a second component whose magnitude is related to the degree of interlocking. The greater the degree of interlocking, the greater will be the overall shear resistance.

Cobbles in their matrix are firmly supported at contacts of each other under applied normal stress and therefore shear failure is not possible

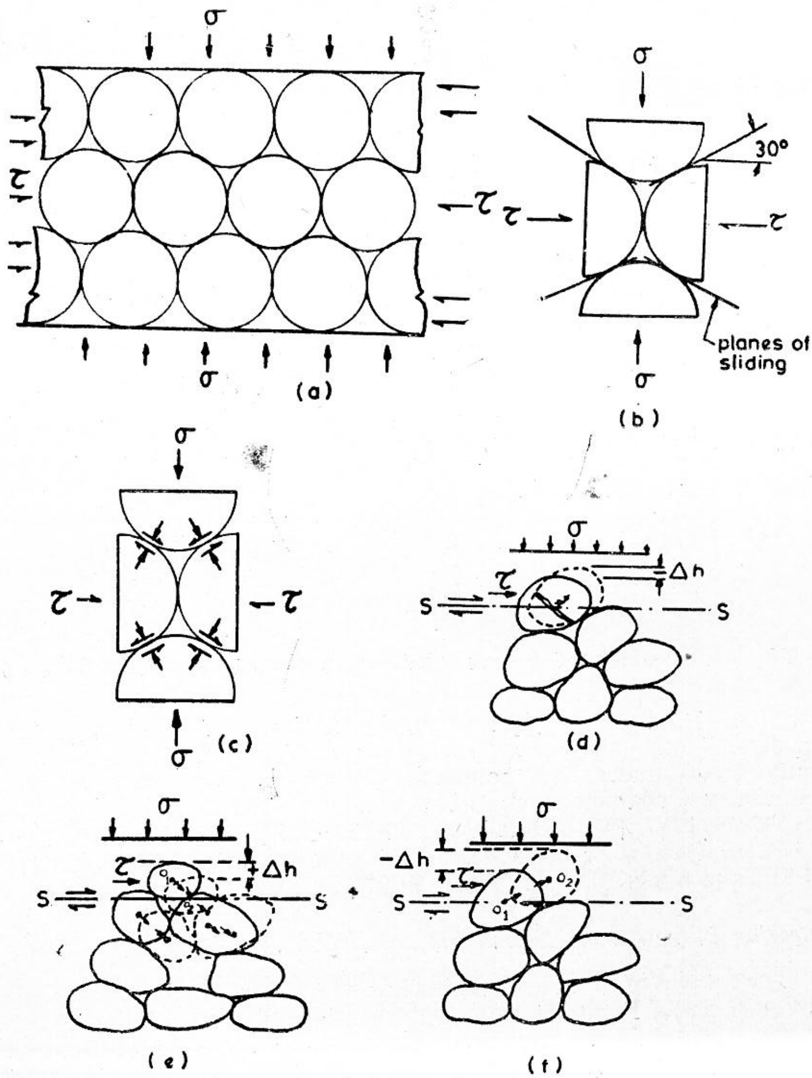

FIGURE 5 : Diagram illustrating (a) a dense single grained spherical cobble structure
(b) possible planes of sliding
(c) normal and tangential stresses being developed at contact surface
(d) crushing of cobble taking place somewhere midway in its effort to move up and over another under high normal loads
(e) compressive settlements taking place in cobbles when moving downward adjacent to large voids
(f) Volumetric expansion taking place in cobbles while moving up and over another, under low normal loads.

unless accompanied by loosening up of the packings or followed by crushing of grains. In the present case, the shear deformations were initially accompanied by compressive settlements due to downward movement of cobbles adjacent to large voids (Figure 5 *c*) or due to movement of cobbles to denser arrangement (Figure 6), but after such initial adjustment further rolling activity of cobbles (involving steep planes of sliding) to permit shear deformations required large volume expansion in the material and before this could become permissible under the applied normal stress, the tensile stresses developed behind the contacts became greater than tensile strength of cobbles; and therefore the shear failure took place at the peak value of shear stress due to crushing or fracture of cobbles (Figure 5 *d*). This resulted in sudden horizontal displacement and some normal settlement. On further horizontal displacement, the cobbles were again subjected to rearrangement of their packing pattern resulting in gain of shear stress with repetition of the same phenomenon. The general increase initially in the values of peak shear stress at the same normal stress (Figure 2 & 3) appears to be due to improvement in packing pattern of cobbles during process of shear.

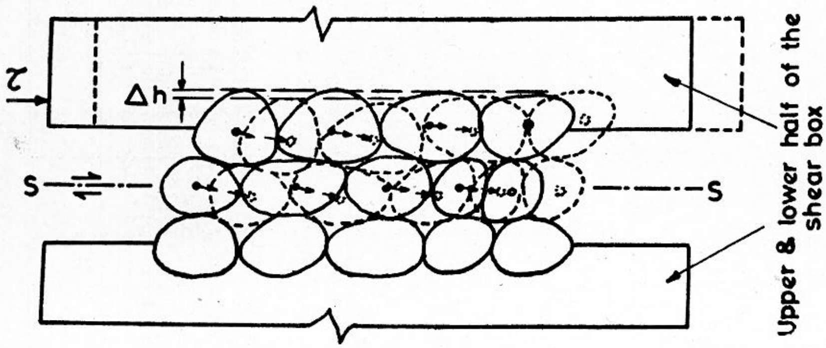

FIGURE 6 : Diagram illustrating a situation occuring in some portion of the structure of cobble in the shearing plane, when cobbles rest on each other

The shear strength of cobbles which undergo crushing phenomenon during shear would therefore be largely related to the strength, and structural resistance of the cobbles and their surface hardness.

Process of Shear in Cobbles under Normal Stresses Lower than 1.2 kg/cm²

The shear tests on cobbles (38 mm to 150 mm in size) at normal stresses lower than 1.2 kg/cm² were also performed (Figure-7). Each time before shear failure took place, cobbles experienced volume expansion (normal settlement gauge showed expansion in general in the order of 0.05 mm to 0.15 mm), indicating loosening of their packings and rolling of cobbles upwards over one another accompanied with rise in shear stress till a peak value. When the cobbles had moved up over another's apex (see Figure 5 *f*) peak shear stress dropped and volume compression alongwith shear displacement took place. This phenomenon was repeated on further horizontal displacement. No sound of fracture of cobbles was heard and the fractured cobbles were not found during removal of the material; this therefore indicates that crushing phenomenon does not take place at normal stresses lower than 1.2 kg/cm².

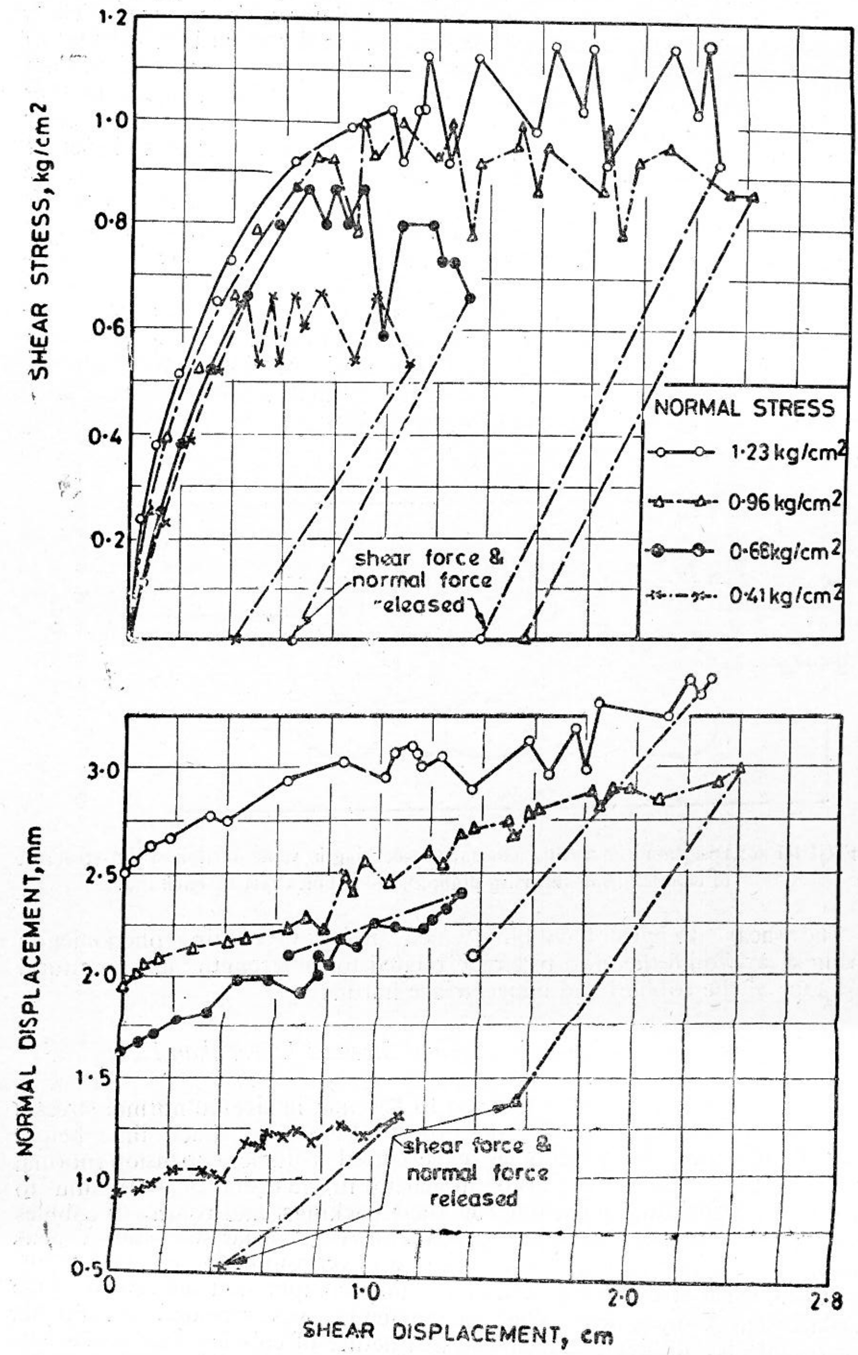

FIGURE 7 : Direct shear plots for cobbles (Size 38 mm or 150 mm) under low normal stresses

Volume expansion (ranging from .02 mm to 0.5 mm) under normal stresses higher than 1.5 kg/cm² was quite low in comparison with that at low normal stresses; this indicates that requisite stress concentration for developement of crushing phenomenon took place before the cobbles could roll up over one another's apex (figure *5d*). This marked difference was probably responsible for the ratio of peak shear stress and normal stress being more than 1 at low normal stresses and being 1 or slightly less than 1 at high normal stresses. In case of low normal stresses, the shear resistance at failure was controlled by (*i*) cobble to cobble friction and (*ii*) in addition, effort required for making cobbles move up and over another. However, this second factor was only partially utilised in building up shear resistance under high normal stresses.

This marked difference in shearing process under low and high normal stresses change the pattern of the Coulomb's line of rupture, representing normal stress V/S peak shear stress (Figure-8). The line of rupture if drawn and extended on the basis of results at high normal stresses passes through the origin of coordinates, however when drawn on the basis of results at low normal pressures, it does not pass through origin of coordinates, but intersects the ordinate at $C=0.44$ kg per cm². This apparent cohesion for this non-cohesive material, which was ascertained in tests, must be related to a certain degree of interlocking of cobbles. Apparent cohesion in non-cohesive soils have also been reported by Zellur and

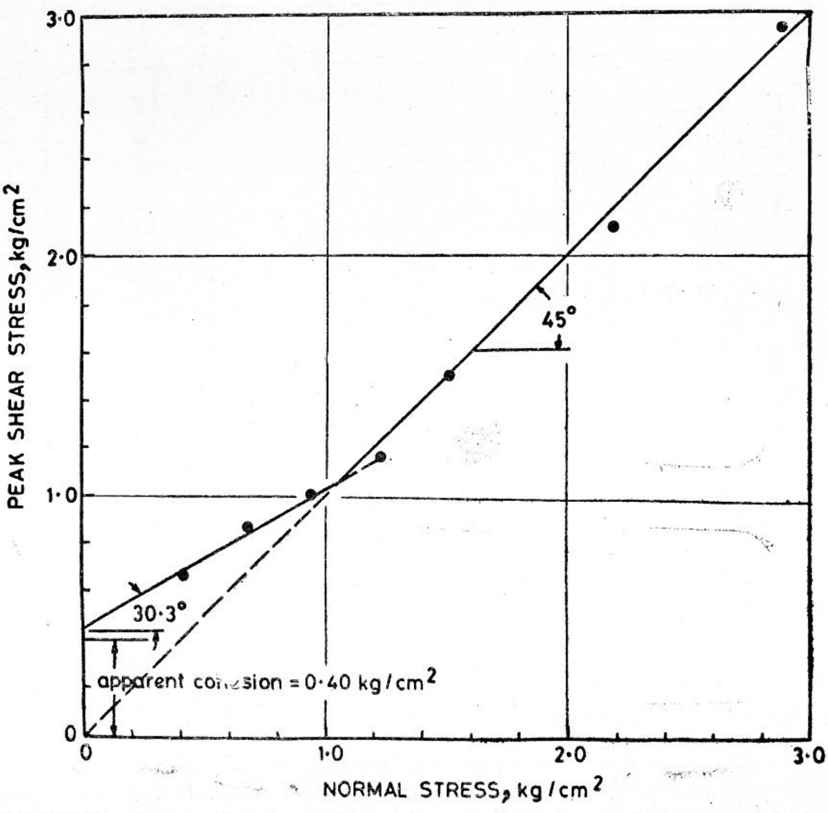

FIGURE 8 : Relation between normal pressure and shearing resistance for cobbles.

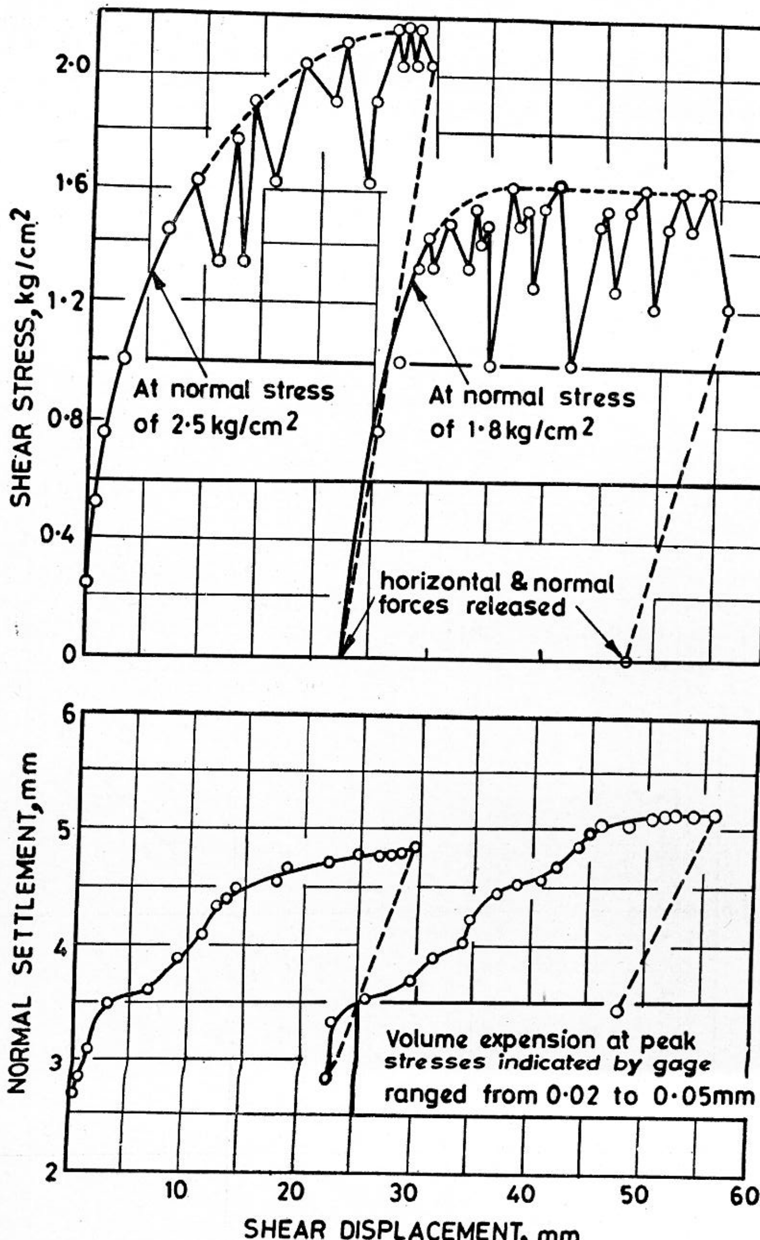

FIGURE 9 : **Direct shear plots for cobbles (Size 50 mm—100 mm).**

Wulliman (1957), who determined its value to be on average upto 0.5 Kg per cm² for a material consisting cobbles, gravels and sand, when tested on large scale triaxial apparatus.

The energy stored in cobbles during application of normal and horizontal forces caused the rebound of part of the normal and horizontal settlement, when the forces were released (Figures 7 and 9).

Keeping the ratio between maximum cobble size and the size of box as 1 : 12, shear tests on cobbles (size 50 mm-100 mm) were also conducted (Figure 9); the results show the same stress strain relationship as was found when this ratio was kept only as 1 : 6. This indicates that there shall be no distortion of results when the ratio between maximum cobble size and size of box is kept even as 1 : 6.

River Bed Material

When the tests were conducted on river bed material (which is a mixture of cobbles, gravels and sand) at the same normal stress at which cobbles were tested no distinct crushing phenomenon took place. After shear failure the shear stress gradually decreased accompanied with steady increase in horizontal displacement (see Figure-10). Since the gradation of the river bed material after test was not determined, the comparison with gradation before test could not be made for detection of crushing phenomenon in it, however the absence of any loud sound of fracture and sudden drop in peak shear stress does speak to some extent about the probability of crushing phenomenon having not taken place.

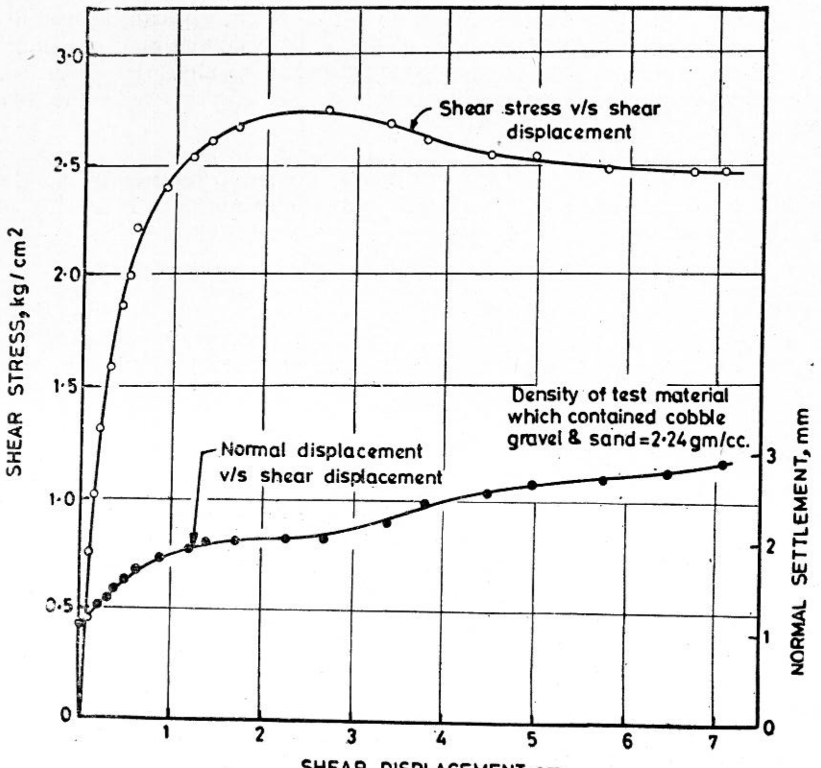

FIGURE 10 : Direct shear plot for sizes smaller than 200 mm River Bed material at normal stress of 2.92 kg/cm².

The results of tests made on cobbles show that it attains a low density and high void ratio, still it has a high angle of shearing resistance (about

45°). Whereas the river bed material consisting cobbles, gravel and sand attains a higher density, but its angle of shearing resistance ranges from 33° to 45°, depending upon its relative density and gradation (Sherard et. al. 1963). This shows that the material containing only cobbles shall have higher shear strength than that of the river bed material. During earthquake, the actuating forces are proportional to the mass, of the material (i.e. density of the material) and acceleration. Therefore, in case of the fill containing only cobbles, the actuating forces shall be smaller owing to its low density, while its shear strength which shall resist these forces shall be higher. This consideration, therefore, may make the fill of cobbles (like processing plant rejects laid at the Ramganga Main Dam) more suitable than the river bed material.

Conclusions

1. The value of the angle of shearing resistance of the material containing only cobbles (like the processing plant reject) shall fall within a close range of 42° to 45°.

2. Large scale shear box of size 120 cm by 120 cm sq. can be usefully employed to test the shear strength of the materials containing maximum particle sizes upto 20 cm without distortion of results. However the additional precaution of keeping a particular spacing between the two halves of the box is considered essential.

3. The cobbles shall undergo crushing phenomenon repeatedly at the peak shear stress during the process of shear under normal stresses higher than 1.50 kg per sq. cm. When normal stresses are lower than 1.0 kg per sq. cm. crushing phenomenon shall not take place and the shear resistance shall be related to mineral to mineral friction angle and in addition the effort required for making cobbles move up and over another (this being related to the degree of interlocking). The material consisting of a mixture of cobbles, gravels and sand shall not experience crushing phenomenon under the same range of normal stresses.

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References

BISHOP, A.W. (1965) : Discussion, *6th Int. Conf. on Soil Mech. and Foundation Engg.*, Montreal, Vol. III.

BISHOP, A.W. (1950) : "Discussion on Measurements of Shear strength of soils" *Proc. Conference in the Measurement of Shear strength of soils. Geotechnique* Vol. 2. p. 113, London.

BISHOP, A.W. (1948) : "A Large Shear box for testing of sands and Gravels", *Proc. of 2nd International Conference on Soil Mechanics & Foundation Engineering*, Vol. I, p-p 207-11, Rotterdam.

BRAUNS, J. (1967): Zu den Grenz-Spannung Szustaenden Von regelmaessigen Packungen aus Spreeden Kugeln im Dreiaxialverstucx," *Publication of the Institute for Soil Mech. and Rock Mech., Technical University, Karlaruhe, W. Germany,* Helt 31.

BRAUNS, J (1968) : Ueber den Einfluss des Einzelkorn Bruches auf dia Belastbarkeit von Haufwerken, besonders von regalmaessigen Kugelpackungen, im Dreiaxialversuch". *Publications of the Institute for Soil Mechanics & Rock Mec., Technical University, Karlsruhe, W. Germany, Heft. 31.*

DVROAK, A. and PETER, P. (1961) : "Field Tests on Soils & Rocks", *Proc of V Inter. Conf. on Soil Mech. & Found. Engg.* Vol. II pp. 453-460.

HALL, E.B. and GORDON, B.B. (1963) : Triaxial Testing with Large Scale High Pressure Equipment" *Sym. Laboratory Shear Testing of Soils, ASTM,* S.T.P. 361, 315-328.

HIRCHFELD, R.C. and POULOS, S.J. (1963) : "High Pressure Triaxial Tests on a compacted sand undisturbed Silt" *Symp. Laboratory Shear Testing of Soils, ASTM,* S.T.P. 361, 329-341.

JUMIKIS, A.R. (1967) "*Soil Mechanics*" *Second East & West Reprint,* 1967, Affiliated East West Press Private Limited, New Delhi.

LAMBE, T.W. (1951) : "*Soil Testing for Engineers*" John Wiley & Sons Inc.

LEWIS, J.G. (1953) "The sloping core principle for Earth & Rock Fill Dam"., *Doctor's Thesis. University of London.*

LAMBE, T.W. and WHITMAN, R.V. (1969) : "*Soil Merchanics*" *John Wiley & Sons. Inc.*

MARSAL, R.J. (1967) "Large Scale Testing of Rockfill Materials" *Proc. Am. Soc. Civ. Engrs.,* 93: SM 2: 27-43.

RAMAMURTHY, T. (1966) : "Order-disorder Developments after different strain Paths" *Ph.D. Thesis, Birmingham University.*

RAMAMURTHY, T. (1969) : "Crushing Phenomenon in Granular Soils" Jnl. of *Indian Nat. Soc. Soil Mech. of Foundation Engg.,* 8: 1: 67-86.

RAJU, V.S. (1969) : Discussion, *Journal of Indian Nat. Soc. of Soil Mechanics & Foundation Engg.,* 8:4: 398-401.

SHULTZE, E., (1957) : "Large scale shear Tests" *Proc. of IV Inter Conf. on Soil Mech. & Found.Engg.,* Vol I. pp. 193-199.

SOWERS, G.F. and CLAYTON, E.G. (1961) : "Large Scale Preconstruction Tests of Embankment materials for an Earth-Rock Fill Dam" *Proc. of Fifth International Conferences on Soil Mechanics & Foundation Engineering,* Vol. II, Division 28-7 pp. 717.

SHERARD, J.L., WOODWARD, R.J., GIRIENSKI, S.F. and CLEVENGER, W.A. (1963) : "*Earth & Earth-Rock Dams*", John Wiley & Sons, Inc.

UNITED STATES DEPARTMENT OF THE INTERIOR, BUREAU OF RECLAMATION, "*Earth Manual*".

TIMOSHENKO, S and GOODIER, J.N. (1951) : "*Theory of Elasticity*" McGraw-Hill Book Co.,

ZELUER, J and WULLIMANN, R., (1957) "The Shear strength of the shell materials for the GOSCHENENALP DAM, Switzerland" *Fourth International Conference on Soil Mechanics & Foundation Engineering* (London), Vol. II, p. 399