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DETERMINATION OF DYNAMIC STRENGTH OF BOULDER DEPOSITS

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INTRODUCTION

River bed and terrace material in many parts of North India consists of large boulders mixed with finer material like gravel, sand, silt and clay. The presence of these boulders changes the strength characteristics of the deposits considerably, mainly depending upon the ratio (by volume) of the boulders and fine material. The special nature of the deposits makes the conventional methods of soil exploration difficult to be adopted. Therefore, special tests such as field shear tests and passive pressure tests have been used by various investigators to determine the load carrying capacity of boulder deposits (Narahari, Rao and Jain, 1968, Prakash and Ranjan, 1975).

The Northern part of India being seismically active, the dynamic strength of the foundation soils needs to be determined if structures are to be designed to withstand earthquake forces. No tests has so far been reported in literature to account for the dynamic forces in the mobilisation of strength of boulder deposits.

A field shear test with dynamic (or reversible) shear stresses poses several problems and hence is very difficult to be performed. A passive pressure test under dynamic loading is comparatively simpler and can be conducted without the need for any heavy apparatus and equipment. This paper describes the details of dynamic passive pressure tests conducted to ascertain the effect of vibrations on the strength of boulder deposits at Rajban. Paonta.

DYNAMIC PASSIVE PRESSURE TESTS

The principle involved in this test is to push, a test wall against the vertical face of a cut under the combined action of static and dynamic loads till failure is induced in the soil. After the application of a static load, dynamic loads of varying intensities at different frequencies were superimposed and the wall deflections measured. Next increment in static load and superimposition of dynamic loads were then applied in gradually increasing order till failure is induced in the back fill. Figure 1 indicates schematically the time-wise loading sequence.

Test Set-up

Figure 2 shows a schematic diagram of the test set-up.

Two reinforced concrete walls 1·5m deep 1·0m wide and 0·30m thick were cast opposite to each other and at a distance of 1·25m (clear) in a pit 1·5m deep. One of these walls had foundation bolts embedded to facilitate fixing of the mechanical oscillator and a loading frame. Also on either side of the test wall two side walls of brick in

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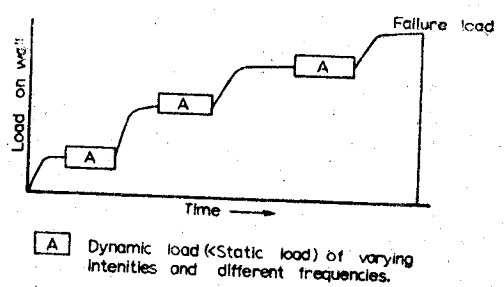

Fig. 1. Schematic Diagram of Loads on Wall in Dynamic Passive Pressure Test.

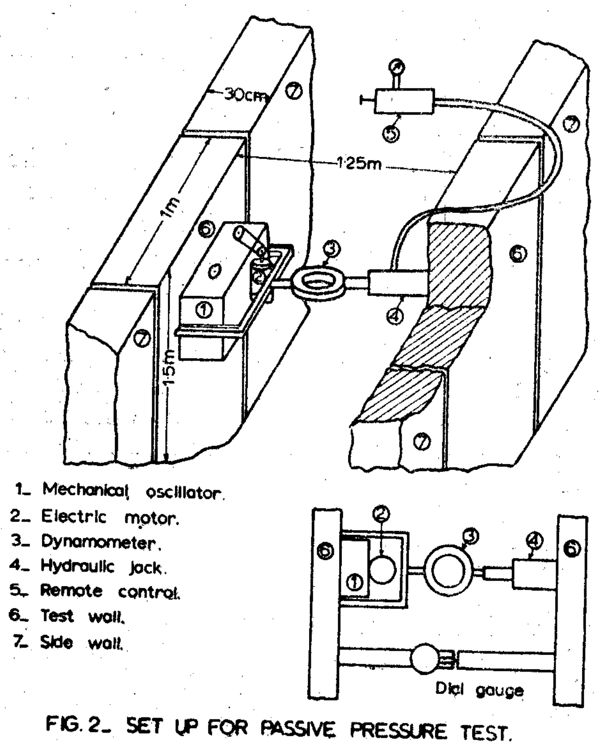

Fig. 2. Set up for Passive Pressure Test.

cement mortar were built for a length of 0·5m. Thus the test wall was isolated from the possible side effects. The mechanical oscillator and the driving unit were fixed on the wall in such a way that vibratory force could be imparted in a horizontal direction at a depth of 1·0m (2/3 the height of the wall) from the top. Then the loading frame was fixed in the same horizontal plane as the oscillator assembly. The portal shaped loading frame facilitated applicaton of static loads without transferring the load to the oscillator assembly. Loads were applied on the test wall by means of a calibrated hydraulic jack which derived a reaction from the wall on the opposite side of the pit. The precautions taken during mounting and testing included support for the jack from below to allow for the same line of loading and also to avoid overturning of the wall during mounting of the frame. These supports did not vitiate the applied loads in any way.

Four dial gauges mounted on magnetic bases which in turn were held on to three angle iron lengths driven into the floor of the pit completed the test set-up. Three of the gauges abutted against the test wall to check any rotation or tilting and the fourth against the reaction wall. Fig. 3 shows the test wall, oscillator assembly, load frame and the dial gauges.

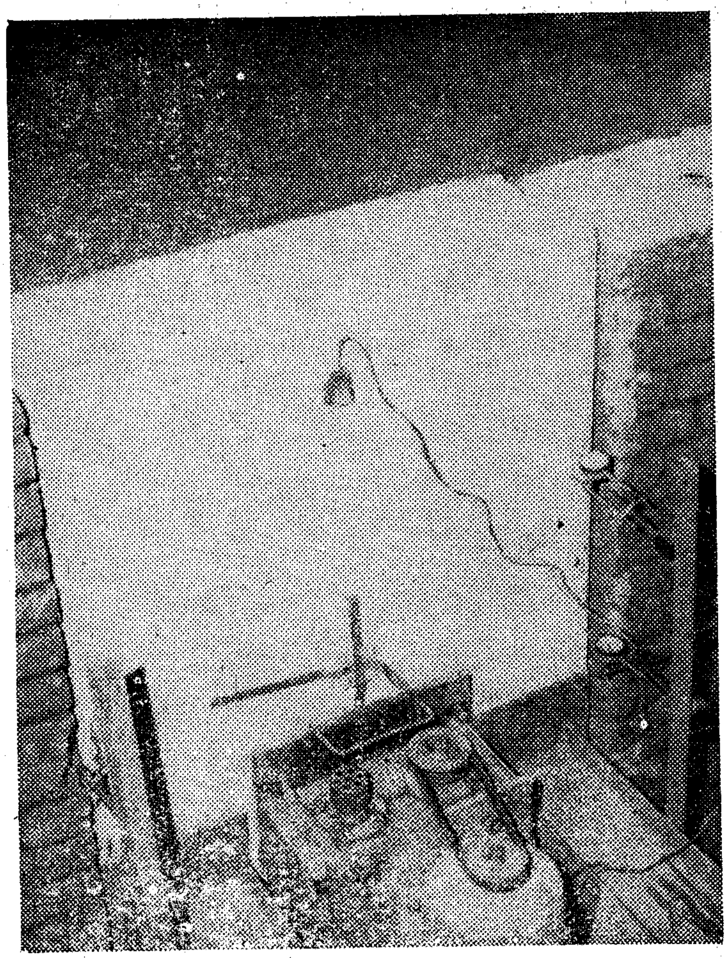

Fig. 3. Passive pressure test arrangement.

Test Procedure

A static load was applied first and maintained till the deflection of the test wall had completely taken place under the load. The travelling rods of the dial gauge were pulled back with threads and the motor of the oscillator-motor assembly was started. To start with, a small eccentricity for the oscillator and a low speed for the motor were selected. After 30 seconds of operation of the motor, it was switched off. The dial gauges were then released and read and arrested again for the next vibratory loading. A higher frequency of the motor was now used and the above procedure repeated. This was continued till the highest possible frequency was achieved.

The eccentricity of the oscillator was then changed and the above tests repeated till different dynamic load levels were applied for a particular static load level. The dynamic load was always kept smaller than the static load level. The hydraulic jack was operated as necessary during the vibrations to maintain the static loads.

After completing the dynamic tests for any particular static load level, the hydraulic jack was operated again to increase the static load to the next value and all the dynamic tests as described earlier were repeated. The same procedure was followed till the soil behind the wall failed, thus developing the full passive resistance.

The details of the tests performed are given in Table 1.

TABLE I

PASSIVE PRESSURE TESTS

Test	Static load level, T	Eccentricities θ	Frequencies cps	Remarks
1	2	3	4	5
PP_1	2 4 7	4°, 16°, 36°, 72°, 140° —do— —do—	4 to 45 6 to 42 6 to 44	Largest eccentricities and frequencies were not used for static load level of $2T$.
	10	—do—	6.5 to 3.7	Maximum Dynamic force=2600 kg
	15	—do—	4.5 to 3.7	Minimum Dynamic force=0.5 kg
	20	—do—	2.5 to 36	Force is given by $F=2.05f^3 \sin \theta/2$ kg
	25	Failure and hence no dynamic load		
PP_2	5	4°, 16°, 36°, 72°, 140°	3 to 31	Maximum dynamic force=3850 kg
	10	—do—	1 to 50	Minimum dynamic force=0.7 kg
	15	—do—	4.5 to 33	
	20	Failure		

RESULTS AND DISCUSSIONS

For obtaining the static load versus wall deflection plot, the wall deflection at any static load level was taken as the sum of the deflection at the previous static load and the additional deflection caused when the static load was increased from the previous value to the present value. The full lines in Figures 4 and 5 show the static load versus deflection plots for the two sites tested.

The effect of dynamic load application with various static loads already applied is also shown in Figs. 4 and 5 as points with numbers assigned to them. These points

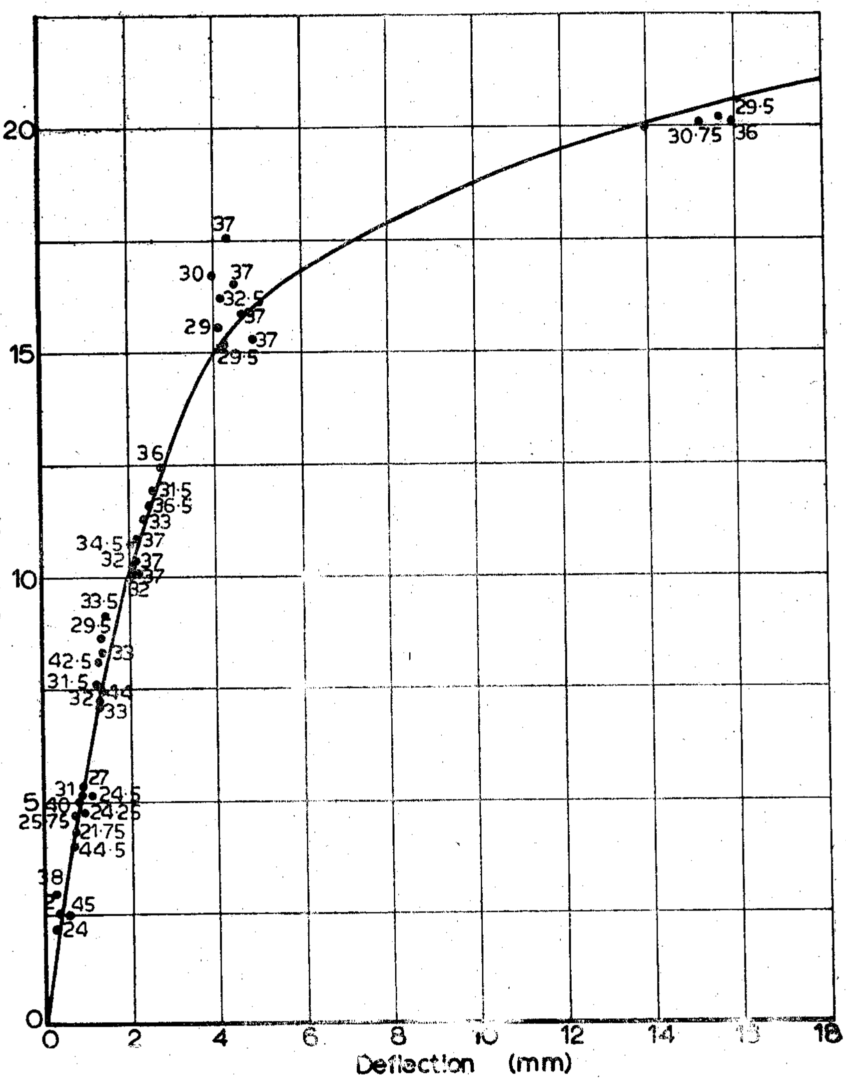

Fig. 4. Load Displacement Plot for Passive Pressure Test, Site PP_1

correspond to a load equal to the sum of the static load and the dynamic load caused by the oscillator and a deflection equal to the sum of deflection at the static load and the additional deflection caused by the particular dynamic load. The number indicates the frequency at which the load was applied.

From the above figures it will be seen that the static as well as dynamic load-deflection points are lying in a narrow band. Thus it is seen that the static and dynamic strengths of the soil are practically the same. This is in conformity with the observation by different investigators from tests on small samples of cohesionless soil in the laboratory.

The static loads on the wall were applied at a depth of two thirds the height of the wall from the top. Test wall in one case moved parallel to itself while in the second case the wall rotated about an axis above the wall top. The examination of the second case showed a lower point of load application than desired due to faulty casting of foundation bolts and also presence of two large boulders in the back fill near the top of the wall. Thus it seems that the distribution of passive pressures may be taken as

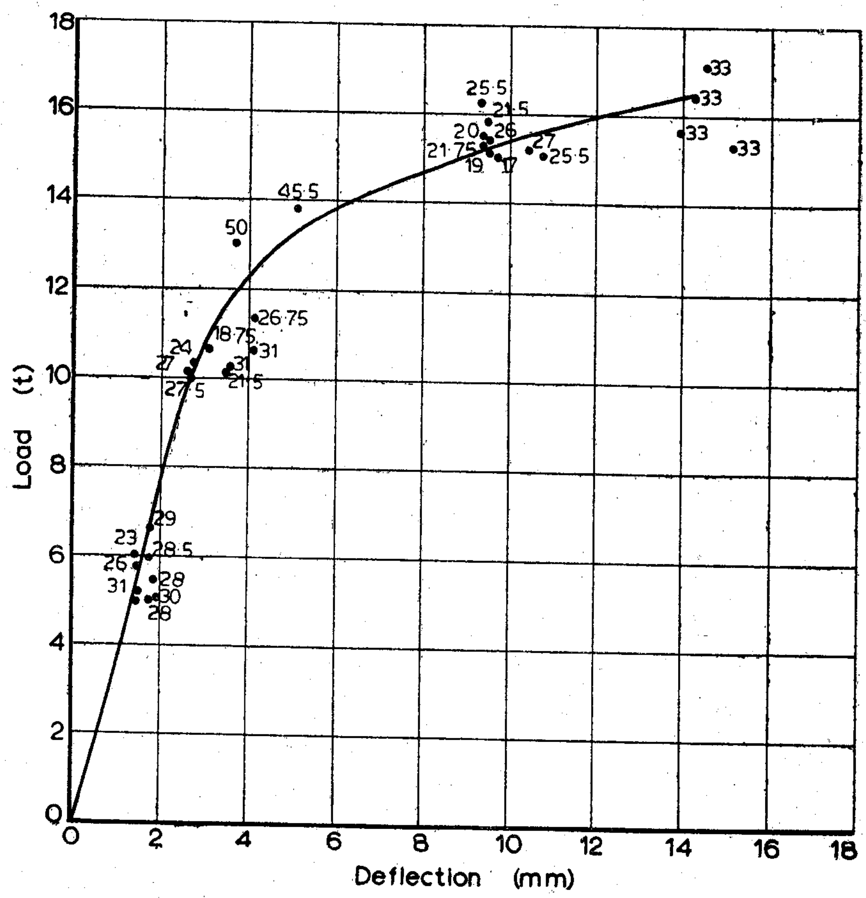

Fig. 5. Load Displacement Plot for Passive Pressure Test, Site PP_2

hydro static. Even at large displacements of the test walls, no clear rupture surface emerged on the ground surface. Cracks radiating from the wall were observed, Fig. 6. Also, general heaving of the ground behind the wall was in evidence.

Because of the above, the failure loads under static conditions were determined by the intersection of tangents of the full lines in Figs 4 and 5 for the tests PP_1 and PP_2 respectively. The influence of dynamic loads on the passive pressures was determined by similarly determining the failure loads by the intersection of tangents to the envelopes of all the points plotted in the above figures. It was observed that the influence was negligible. Thus it was concluded that for the soil tested, the static and dynamic strengths were the same. The failure loads for the two cases are tabulated below:

Test No.	Failure load T	Ultimate load T
PP_1	16.2	25
PP_2	13.6	20

It was also desired to verify whether passive pressure tests would give the same strength parameters as the large direct shear tests conducted at the site (Praksah et al,

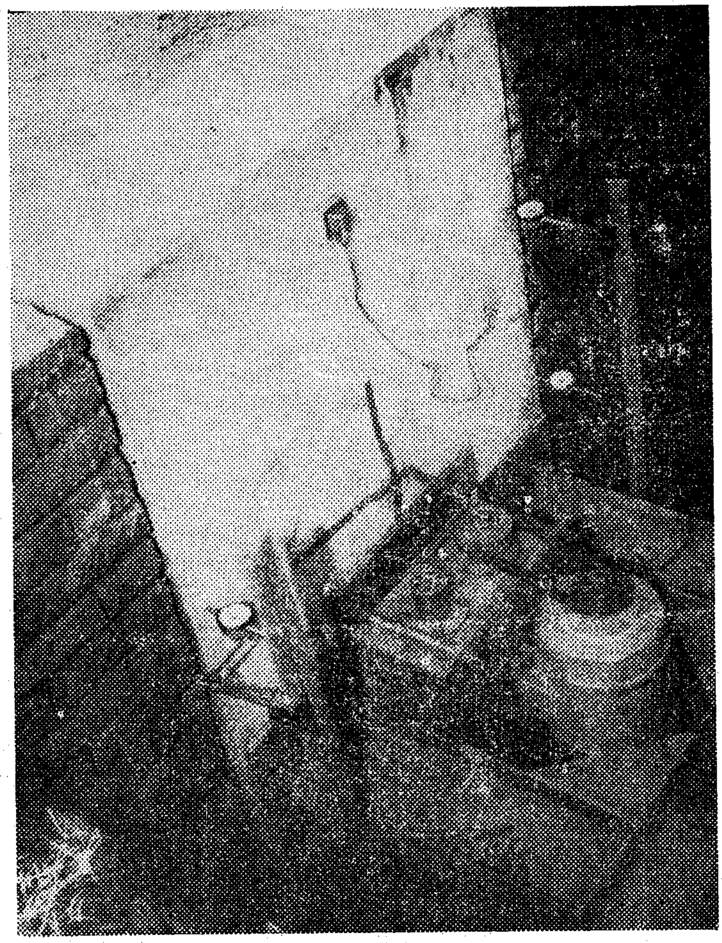

Fig. 6. Test wall during the passive pressure test at site pp_1.

1973). Coulomb theory was used to interpret the test results with the following assumptions.

Case (a)

Angle of wall friction $\delta = \frac{2}{3}$ angle of internal friction ϕ; mobilisation factor wall friction = 1

Case (b)

$$\delta = \frac{2}{3}\phi$$

mobilisation factor = 0.6

In the above two cases, the values of the angle of internal friction were worked out as given below:

Test No.	Value of ϕ Case a	Case b
PP_1	34.15°	38.6°
PP_2	32°	36.4°

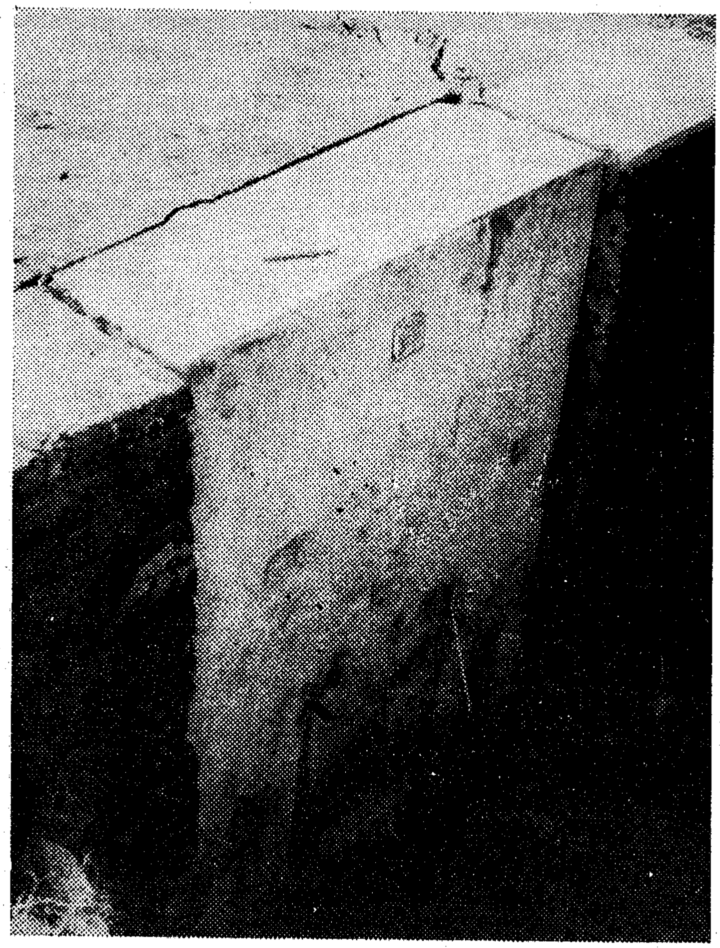

Fig. 7. Crack pattern after passive pressure test.

Since the results of the tests were likely to be affected by the comparatively small width of the test walls (and hence plain strain conditions not likely to be valid), it was felt desirable to use an analysis based on a theory of 3-dimensional passive pressures. Kapur (1972) has given the following relation.

$$(K_p)_{3D} = (K_p)_{2D}\left(1 + C\frac{D}{B}\right) \tag{1}$$

where $(K_p)_{3D} = \dfrac{\text{Ultimate load on the wall} \times 2}{\gamma D^2 B}$

$(K_p)_{2D}$ = value of passive pressure coeff. for the above case of ϕ from 2-dimensional theories

C = Constant depending upon the density of back fill, ν

D = Depth of wall

B = Width of wall

ϕ, δ = angle of internal friction of soil and angle of wall friction.

Kapur (1972) has also given the values ϕ of the constant C for different densities of the soil. Based on the relationship (Eq. 1), the value of 'ϕ' for the two cases already described were calculated as shown below:

Test No.	Angle of internal friction ϕ Case a	Case b
PP_1	31.5°	35.5°
PP_2	30°	33.15°

Since the case of the tests performed at Rajban is best represented by the 3-dimensional theory and a mobilisation factor of 0.6, the angle of internal friction for the soil at site was concluded to have a value between 33° and 35°. The value of ϕ from large scale direct shear tests was 32.5°. The results are therefore thought to be in close agreement.

CONCLUSIONS

A method of evaluating the static and dynamic strength of soils not amenable to sampling has been developed and successfully used in field.

The strength of cohesionless soil, as at the site where the tests were performed is hardly affected by the dynamic loads.

The results obtained by the tests are in close agreement with other tests as far as the static strength parameters are concerned.

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