EXPERIMENTAL DETERMINATION OF DYNAMIC PASSIVE PRESSURE OF SAND

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ABSTRACT

An experimental study was undertaken to understand the dynamic passive pressures. A steel plate was pushed parallel to itself against sand kept in a bin mounted on a horizontal shake table and the ultimate resistance of the soil was measured as a function of the table acceleration. The dynamic passive pressures were found to be marginally smaller than the static value but greater than those predicted by modified Coulomb theory. The observed rupture wedges were however smaller than those from the theory.

INTRODUCTION

The resistance of soil to lateral compression, otherwise known as the passive pressures, has been studied by different investigators during the last two centuries. Amongst those who attempted to study the problems analytically, the name of Coulomb, Rankine, Muller Breslau, Terzaghi, Brinch Hansen, Janbu and Satyanarayana are worth mentioning. All the above except Rankine who considered the state of plastic equilibrium in the soil, have studied the equilibrium of a rupture wedge behind the rigid structure compressing the soil. Very valuable information on the behaviour of the structure - soil system has been made available through carefully conducted experimental studies, mainly by Terzaghi, Brinch Hansen, Rowe and Peaker (1965), Nandakumaran (1967) and James and Bransby (1970). However, the resistance offered by the soil during earthquake loading conditions is little understood. Though most part of India and many other areas in the developing nations lie in active seismic zones and requires the consideration of earthquake forces, this aspect has not received the attention it deserves from the researchers. The available methods belong to two categories one in which a pseudo decrease in the value of the angle of internal friction of the soil is considered (Sano, 1916) and the second in which an inertia force corresponding to a seismic coefficient assigned on the basis of the seismicity of the site is also taken into account when the equilibrium of the rupture wedge is analysed. Both the above approaches lead to the estimation of smaller values of "Dynamic" passive pressure compared to the "Static" values. Since the angle of internal friction of the soil "ϕ" has been shown to be unaffected by the earthquake type of loading (or even loading at very high rates of strains) and since the lateral earth resistance should be a function of ϕ, this presents a very curious picture. Only carefully planned and executed model tests can help in understanding the physical behaviour of the system. No such studies has so far been reported in literature.

This investigation consisted of measuring the lateral resistance offered by a vibrating soil medium contained in a bin mounted on a horizontal shake table when a steel wall was pushed into the back fill of sand. Only

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the case of translation was included. The effect of acceleration on the dynamic passive pressures was noted. The sizes of rupture wedges at different acceleration levels were also investigated.

TEST SET-UP

Tests were done on an apparatus mounted on a shake table capable of being excited at different frequencies and acceleration. The details of the soil used and the apparatus are given below -

Soil Used: The sand used in the test was air-dried uniform Ranipur sand. The relevant properties of the sand are (i) Soil Type - SP (Poorly graded sand with little fines). (ii) Soil grading - Illustrated in Figure 1. (iii) Uniformity Coefficient - 1.83. (iv) Effective Size = 0.12 mm (v) Specific gravity = 2.66 (vi) Average density = 1.595 gm/cc used in test (vii) Void ratio at the test conditions = 0.59 (viii) Angle of internal friction=40°.

Description of Apparatus: The tests were carried out in a vibrating bin 200 cm long, 100 cm wide and 75 cm high Fig. 2. The sides of the bin are of 0.6 cm steel plates rigidly fixed to angle iron frame by means of bolts. The bottom of the bin consists of 12 mm thick steel plate. The bin is mounted on steel wheels which rests on steel plates provided with grooves. The steel plate on which the wheels rest is also provided with guides at the ends to prevent the wheels from moving out of the plates. The construction of the bin is sturdy in view of the vibrations to which it is subjected.

The bin contains a vertical test wall made of steel. It is 100 cm wide, 30 cm high and 1.5 cm thick. The wall is hung on the side walls by means of a frame (C in Fig. 2) of horizontal and vertical rods. The wall can slide on rollers on the top of the side wall. The wall could be given a translatory motion by means of a horizontal driving screw (A in Fig. 2) rigidly attached to the wall at its centre. A dynamometer of 5 tons capacity (B in Fig. 2) is fixed between the driving screw and the wall for measurements of the load applied. A displacement transducer consisting of 2.25 cm wide spring steel strip provided with strain gauges and rigidly fixed at the top of the wall and to the body of the bin was used for measurement of displacements of the wall. A mechanical oscillator was rigidly connected to the base of the bin. The oscillator can be adjusted at different eccentricities and is run by a 2 h.p. D.C. motor. The speed of the motor is controlled by means of a speed control unit. The frequencies and accelerations of bin vibrations are measured by means of an acceleration pick-up attached to the bin and connected to an amplifier and pen recorder. The driving screw was operated manually.

THE PROCEDURE

The bin was filled with sand in 3 layers. After placing each layer of sand, the bin was vibrated for 7 minutes. Oscillator eccentricity, frequency and duration were maintained in each test to achieve uniform density in all the tests. Sand containers of 200 cc capacity were placed with its centre of gravity at 22.5 cms from the top of the wall to obtain average density of the soil. The average density varied between 1.585 gm/cc to 1.605 gm/cc. The top of the fill was levelled with the top of the wall to obtain a horizontal surface of the backfill. The bottom of the wall remained 15 cm above the bottom of the bin so that the wall moved on the sand surface instead of the bottom plate. Before filling the sand, the wall was always brought to its original position by applying a reverse movement to the driving screw. It was checked that the wall remained vertical. After four or five tests, the roller on which the test wall slided was cleaned to remove dust and was oiled. The accumulation of sand particles under the wheels of the bin, which could offer resistance to the bin vibrations, was also periodically removed by means of an air blower. After the bin was filled with sand and levelled up, the displacement gauges and the load cell were connected to Brush amplifiers which were connected to a two channel pen recorder and the bridge was balanced. Then the bin was vibrated at a fixed eccentricity (say 36 rev.) and rpm (say 1500 rpm). The pen recorder was started. Load was then applied to the test wall by rotating the driving screw manually. Effort was made to apply the load at a constant rate. The load was applied till a slip out-crop emerged on the surface. Meanwhile the strains in the displacement gauge and the load cell were recorded simultaneously on the paper of the two channel pen-recorder. The load and displacement of the wall were computed from these recordings. After the slip outcrop appeared on the surface, the motor was stopped and the profile of the slip outcrop was noted. The sand containers were taken out and densities of the backfill were computed to check that the densities remained uniform in all the tests. The tests were repeated for various eccentricities of the oscillator and speeds of the motor. Two static tests were also conducted. In all, 18 tests were conducted and each test was repeated. Pilot tests were conducted at the start of the tests for static as well as dynamic conditions to have a feel of the probable loads and rupture wedges to be expected during the tests.

RESULTS AND DISCUSSION

The translation of the wall and the load applied on the wall were recorded on a two channel pen recorder such that for any wall movement the corresponding load on the wall was automatically recorded. For a particular frequency, of 5 Hz, the load versus displacements curves have been plotted for different accelerations (Fig. 3). It is noted, the load increases gradually with the wall movement. The failure load decreases with the increase in acceleration. The failure takes place at a wall movement of 4.3% of the height of wall in the case of static load. When the acceleration is 0.28g, then also the failure takes place at a wall movement of 4.3%. At acceleration 0.034g the failure takes place at 5.3% of the wall movement. For frequency 20 Hz (Fig. 4), it is observed that the load on the wall increases gradually with the wall movement for each acceleration. For accelerations, 0.182 g, 0.22 g and 0.23 g the failure load decreases with acceleration. In other two cases i.e. for accelerations o.26 g 0.114 g the results do not follow the same trend. The failure load occurs at wall movements of 5% to 8.66%.

The following conclusions are drawn on the basis of results obtained at different frequencies of motion.

1. The load increases steadily with the wall movement till failure.

2. The maximum value of load increases with the decrease in acceleration at the same frequency, at low values of frequencies. At higher frequencies, however, no definite trend was visible.

3. Whether frequency is also a factor in affecting the maximum load is not clearly discernible from these tests. Wider range of frequencies may be applied for evaluation of effect of frequency on maximum load.

4. The failure occurred at wall movements between 2% to 8.66% of the height of wall. Rowe and Peaker have reported that in case of dense sand and static passive pressures, the failure occurred at wall movements of 4 to 5% of the wall height. This indicated that in dynamic condition, the tendency is not different.

Maximum load at failure: The failure load was taken as the load at which there was no further increase in load for corresponding increase in wall movements. The coefficients of passive earth pressure was defined as $K_p = 2P_p/\gamma h^2$ where, where, P_p is Passive Earth pressure γ is Density of the backfill and h is height of wall.

The experimental K_p values are given in Table 1. A plot was made of k_p values versus acceleration (Fig. 5.). On this figure, K_p values (for ϕ = 40° and $\delta = 2/3\ \phi$) as per Mononobe-Okabe for the cases of the inertia forces acting both towards and away from the wall have also been plotted. It is observed that the K_p values obtained experimentally do not follow any pattern with respect to acceleration. The average value of K_p in dynamic condition is 20.47 and the value of K_p under static condition is 22. The reduction in the average dynamic K_p value from static K_p value is 7%, In sand, strength of the soil is not affected much under the dynamic condition including the transient loading condition under which strength variation is found to be maximum. Secondly, since the vibration acts towards and away from backfill, the passive pressure would increase and decrease from the static values. The net resistance available is, therefore, likely to be not much different from the static value. This is in accordance with the observed value.

Rupture wedge: In these studies, only the emergence of the rupture line on the surface have been noted. The shape of the slip surfaces has not been studied. The slip surface is assumed to be a plane passing through the toe of the wall and emerging at the surface. The theoretical rupture wedges for ϕ = 40°, δ = 2/3 and α_h = 0.0, 0.10, 0.20, and 0.30 have been computed using M.O theory, plotted in Figure 6. This figure also contains the observed rupture surface for an eccentriently setting of 80 revolution. Comparing the theoretical wedges and those actually obtained, the following conclusions are made, i) The actual wedges are smaller than the theoretical wedges, ii) The rupture wedges gets smaller with decrease in acceleration. The same trend was observed regarding the extent of rupture wedges in the theoretical computations also, iii) The role of frequency in affecting the wedge is not clearly established from these tests. Further tests with wider range of frequencies are desirable to study the effect of frequency.

The equilibrium of some of the rupture wedges were also analysed to compare with the actual load applied. This was done for both the cases of inertia forces acting towards and away from the wall. It was observed that the actual force applied was in most cases greater than those obtained from the polygon of forces for keeping the rupture wedge in equilibrium. The maximum variation was 19.4% and the minimum 1.7%. The average increase in the applied load over the calculated one was 3.9%. The extra pressure might have been needed to overcome the frictional resistance of the side walls. This aspect can be verified only after more investigation.

CONCLUSIONS

1. The dynamic K_p values are observed not to follow any pattern with respect to acceleration. However, the average K_p value under dynamic condition falls below the static K_p value by 7%. For design purposes, the static K_p value may be reduced by 10% while applying them to problems of dynamic passive pressure.

2. The failure occurred at wall movements between 3% to 8.66%. This indicated that in dynamic condition the sand tended to behave almost as in the static condition.

3. The experimental rupture wedges are found to be smaller than the theoretical wedges of Mononobe-Okabe.

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TABLE - 1

TABLE SHOWING THE EXPERIMENTAL K_p VALUES AT DIFFERENT ACCELERATIONS

Sl. No.	Fre- quency	Accele- ration (g)	Maximum load = W (kg)	$P_p = \frac{W}{\cos 1} = \frac{W \times 1000}{.895 \times 100} = 11.2\,W$	$K_p = \frac{2 \times P_p}{\gamma \times h^2} = \frac{P_p}{720}$
1	5	0.034	1175	13120	18.2
2	5	0.028	1275	14300	20.2
3	5	0.028	1525	17050	24.0
4	10	0.085	1075	12000	16.9
5	10	0.078	1300	14500	20.4
6	15	0.142	1000	11200	15.8
7	15	0.132	1275	14300	20.2
8	15	0.122	1525	17050	24.0
9	20	0.26	1525	17050	24.0
10	20	0.23	1100	12300	17.3
11	20	0.22	1350	15100	21.2
12	20	0.182	1500	16800	23.6
13	20	0.114	1450	16200	22.8
14	24	0.382	1200	13400	18.8
15	24	0.35	1400	15700	22.0
16	24	0.24	1150	12900	18.1
17	Static load		1400	15700	22.0

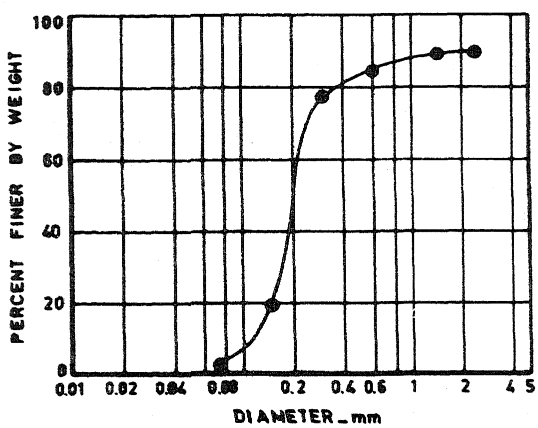

FIG.1 Grain size distribution curve for soil used.

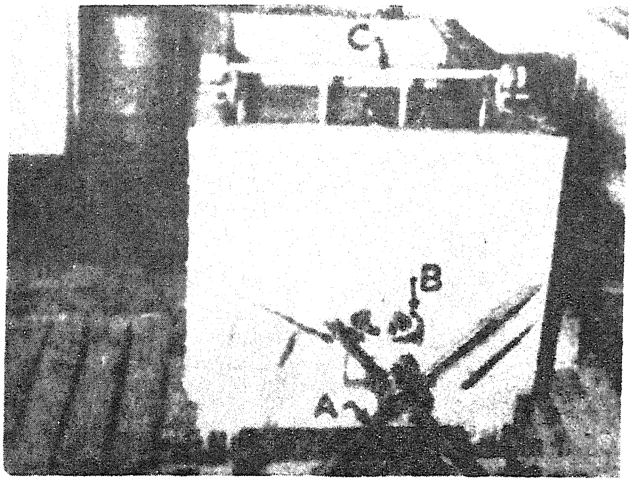

FIG.2 - Photograph of Test Set-up showing bin, driving screw wall etc.

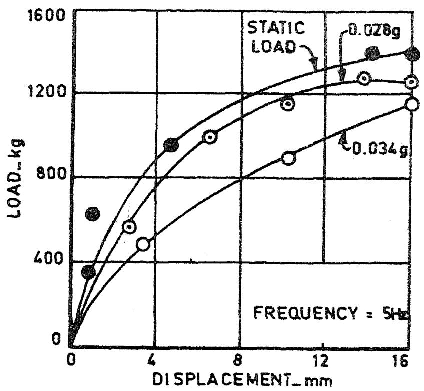

FIG.3 Displacement versus pressure at frequency of 5Hz

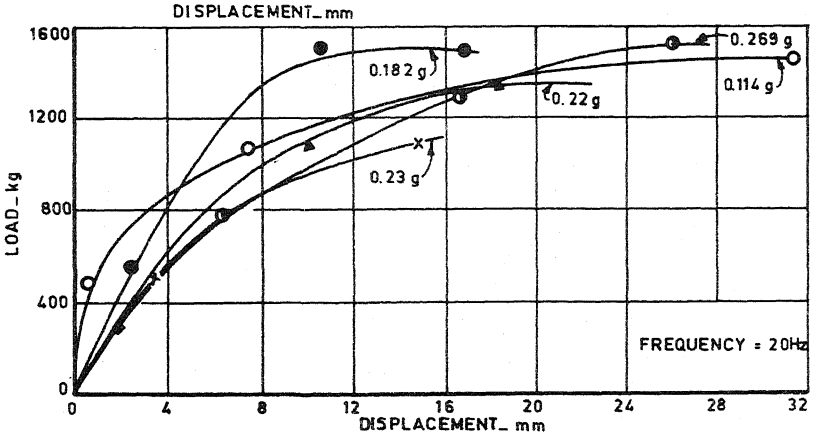

FIG.4-Displacement versus Passive Pressure at requency of 20 Hz

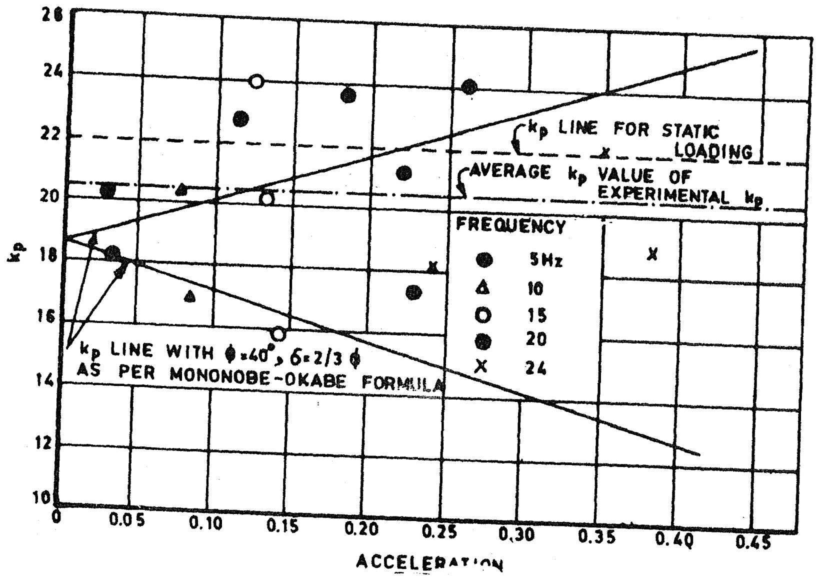

Fig. 5 - Passive Pressure Coefficient K_p versus Acceleration

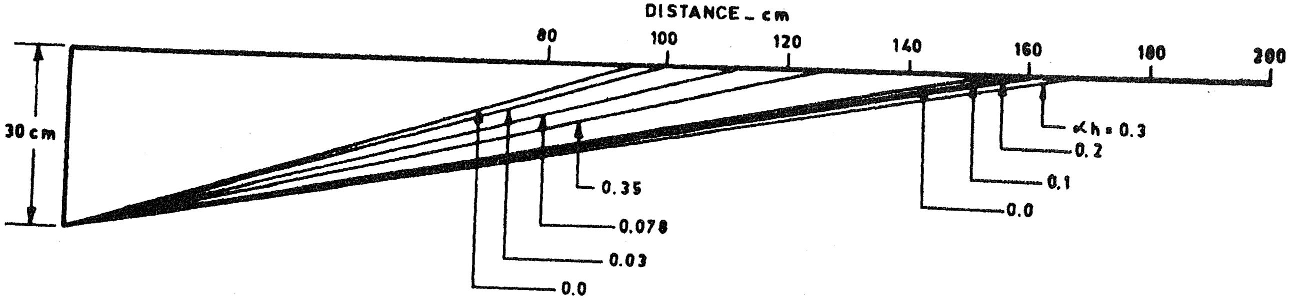

Fig. 6 - Rupture Surfaces