Experimental Investigations on Soil—Structure Interaction of Circular Footings on Sand

by

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Introduction

In the conventional design of footings, the footing is assumed to be stiff and the contact pressure distribution is assumed to be uniform or linearly varying, and the effect of stiffness is not taken into account. It is well known that the actual behaviour of a footing depends upon the stiffness of the footing-soil system. The design of footing considering interaction of the footing-soil system would be more realistic than the conventional design.

Very limited experimental studies have been conducted in the past to determine the actual contact pressure distribution under footings. Kogler and Scheidig (1927) conducted a large number of plate load tests on sand bed to study the effect of the size of the plate on the contact pressure distribution. It was found that the contact pressure depended upon the size of the plate and it became more uniform as the size of the plate was increased. Vesic (1963) carried out a large number of plate load tests to study the behaviour of plates of different sizes on sand. It was shown that the settlement of the footing was a function of the size of the footing and the relative density of sand. Chae et. al. (1965) measured the contact pressure beneath a stiff circular footing on a granular medium. The contact pressure distribution was found to be some what intermediate between a uniform distribution and a parabolic distribution. Drnevich and Hall (1966) observed high contact pressures near the edges of a stiff plate resulting on sand. Smith (1970) investigated the settlement behaviour of the centrally loaded stiff plates resting on overconsolidated sand deposits. The measured settlement profiles compared well with the theoretical profiles obtained using the finite element analysis.

It is observed that experimental studies reported so far had been made on stiff footings of relatively small size. This paper presents a systematic experimental investigation on the interaction behaviour of fairly large size footings with five different stiffness. The effect of the stiffness of the footing-soil system on the load-settlement curve, the contact pressure distribution and the stresses in soil mass is discussed.

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Footing-Soil Systems

Five circular footings of 50 cm diameter and resting on sand were selected for experimental investigation to have a wide range of the stiffness of the footing-soil system. The stiffness of the footing was varied by changing its thickness and Young's modulus. The thickness was varied from 1 to 7.5 cm. The Young's modulus was varied by selecting different materials, viz., aluminium, mildsteel, and the reinforced cement concrete.

It is not possible to define the relative stiffness, K, of the footing-soil system in a non-linearly elastic medium such as sand. However, to get some relative values of the stiffnesses of different footings, the formula for the relative stiffness of footings on elastic medium can be used with some modifications. Borowicka (1936) defined the relative stiffness as

$$K = \frac{1}{2}\frac{(1-\nu_s^2)}{(1-\nu_p^2)} \cdot \frac{E_p}{E_s} \cdot \left(\frac{t}{a}\right)^3 \qquad \ldots(1)$$

where

a = the radius of the footing,

t = thickness of the footing,

E_p = Young's modulus of elasticity of the footing,

ν_p = Poisson's ratio of the footing,

E_s = Young's modulus of elasticity of soil, and

ν_s = Poisson's ratio of soil.

Since E_s for sand changes continuously with increase in the depth and with the change in load level, a representative value of E_s is required. It has been found that, for circular footings on sand bed, the stress in sand at a depth of 0.6 times the diameter can be taken as the average of the stresses in the entire medium, (Arora, 1980). The value of E_s corresponding to the in-situ stress conditions at this depth was taken as a representative value for computation of K. The value of ν_s for sand was taken as 0.30. The values of E_p and ν_p for the footing materials were taken from the stress-strain curves obtained from tests. Table 1 gives the details of the footings tested.

TABLE 1

Details of the Footings

Footing	Material	t (cm)	E_p (kg/cm^2)	ν_p	K
F_1	Aluminium	1.0	6.67×10^5	0.3	0.35
F_2	R.C.C	2.5	1.5×10^5	0.15	1.30
F_3	Mild Steel	1.2	20×10^5	0.3	1.84
F_4	R.C.C.	5.0	1.5×10^5	0.15	10.37
F_5	R.C.C.	7.5	1.5×10^5	0.15	35.00

Foundation Tank

A masonry foundation tank $2.5 \times 2.5 \times 1.5$ m was designed and constructed. This size of the tank was selected in order to keep the pressure bulb within the boundaries. The brick walls of the tank were cement plastered. The top of the tank was kept 15 cm above the ground surface. The tank is shown in Fig. 1.

Pressure Cells

Integrated diaphragm type stainless steel pressure cells for the measurement of stresses in soil mass and the contact pressure were designed and fabricated. Pressure cells were designed as free-earth pressure cells because the design criteria for such cells are more rigid as compared with that for boundary cells. Criteria suggested by various investigators (Brown, 1973) were taken into account while designing the cells. The thickness of the diaphragm was calculated using the following formula (Hanna, 1973).

$$t = D \sqrt[3]{\frac{2000\, p}{96\, E}} \qquad \ldots(2)$$

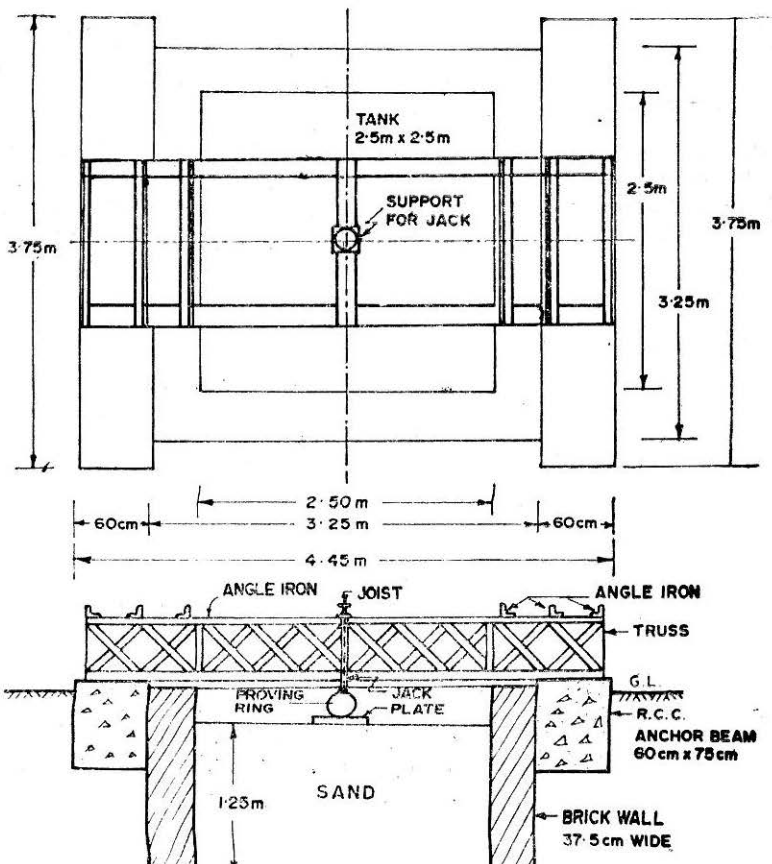

FIGURE 1 Details of Experimental Set-up

where,

D = diameter of the diaphragm,

p = design pressure,

E = Young's modulus of the material.

The design pressure was taken as 2.5 kg/cm² corresponding to the maximum expected pressure on the footing under a load of 3 tonnes. Details of the pressure cells used are given below :

Overall diameter	= 35 mm
Diaphragm diameter	= 25 mm
Overall height	= 7 mm
Height of the main body	= 5 mm
Thickness of brass cover	= 2 mm
Thickness of diaphragm	= 0.75 mm

Two electrical resistance strain gauges of size 3 mm (Rohits, Roorkee) were pasted parallel to each other on the inner surface of the diaphragm near its centre. The leads of the strain gauges were taken out of the brass cover through a small hole (Figure 2).

Pressure cells were calibrated using a standard triaxial cell having suitable modifications. The base of the Cell was fitted with a detachable pedestal containing a central hole for taking out the leads of the pressure cells mounted on the pedestal with its diaphragm at the top (Figure 3).

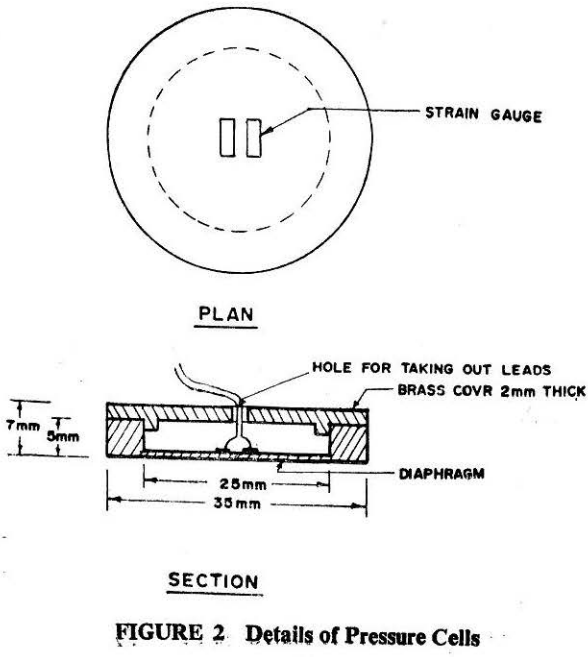

FIGURE 2 Details of Pressure Cells

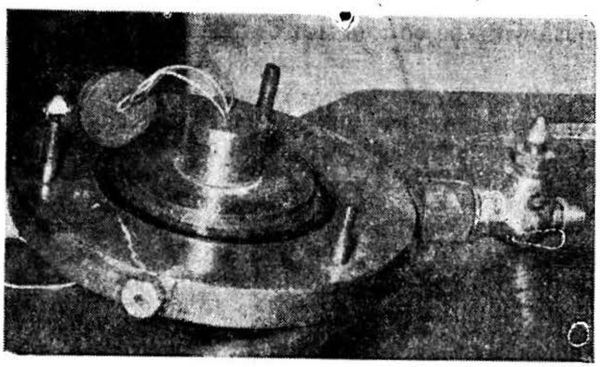

FIGURE 3 Apparatus for Calibration of Pressure Cells

Rubber membranes and O-rings were used for sealing the cell to the pedestal. The triaxial cell was then assembled and filled with water, and connected to the self-compensating mercury pot system. Strain gauges of the pressure cell were connected to a strain recorder on opposite arms. Two strain gauges pasted on an identical dummy cell were connected to other arms of the recorder. The pressure cell was subjected to several cycles of loading and unloading by increasing and decreasing the cell pressure before actual calibration. The strain recorder was balanced at zero cell pressure and the strain indicated was noted. The cell pressure was increased in several increments upto the design pressure of 2.5 kg/cm^2. A calibration curve between the pressure and the strain was prepared for each cell. Figure 4 shows a typical calibration diagram.

Construction of Footing

Footings F_1 and F_3 were cut from metal sheets. For fixing the boundary pressure cells, twelve grooves of 35 mm diameter and 7 mm depth were cut along two diametrical planes at right angles to each other. On each radial line, grooves were made near the centre, the quarter point and the

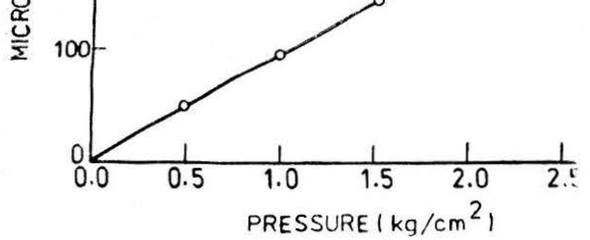

FIGURE 4 Calibration Curve for a Typical Pressure Cell

periphery. Boundary pressure cells were fixed in these grooves with their diaphragms flush with the bottom surface (Figure 5).

Footings F_2, F_4 and F_5 were constructed of cement concrete (1:2:4). Footings F_4 and F_5 were designed to carry a concentrated load of 4 tonnes at the centre. The reinforcement consisted of a 3 mm diameter mild steel weld-mesh with 18 mm spacing in the two directions. The hoop reinforcement was in the form of a 47 cm diameter circular ring made of 10 mm diameter mild steel. Two additional mild steel rods 45 cm long and 10 mm diameter at right-angles to each other were tied to the circular ring. These rods were also required for proper positioning of the pressure cells which were fixed in position before casting of the footing. Thin walled copper tubes about 15 cm long and 2 mm in diameter were soldered to the brass cover of each boundary cell in order to keep the leads of the pressure cells free from the concrete. Boundary pressure cells could not be fixed to the footing F_2 owing to practical difficulties.

Concrete was carefully placed and vibrated with the surface vibrators. Wet curing was done for 28 days. Control cubes (15 cm size) were also cast, vibrated and cured. The stress-strain characteristics of concrete were obtained by testing these cubes after 28 days of casting.

Preparation of Sand Bed

The sand used in the investigation was obtained from the river Yamuna, near Delhi. The sand had 90 per cent of its particles in the range of 0.1 to 0.45 mm size with a uniformity coefficient of 2.1 (Figure 6). The particles were subrounded to subangular in shape. The specific gravity of the particles was 2.67. The angle of shearing resistance as obtained from a *CD* test on a standard triaxial apparatus over the range of 0.5 to 2 kg/cm^2 confining pressure was found to be 41° at a relative density of 67 per cent.

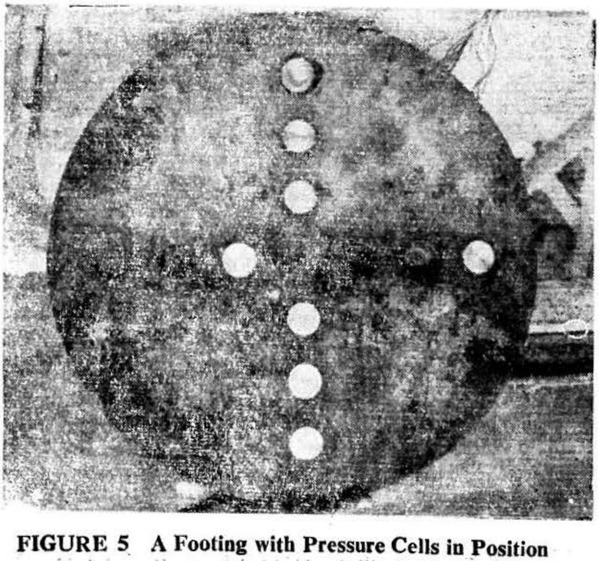

FIGURE 5 A Footing with Pressure Cells in Position

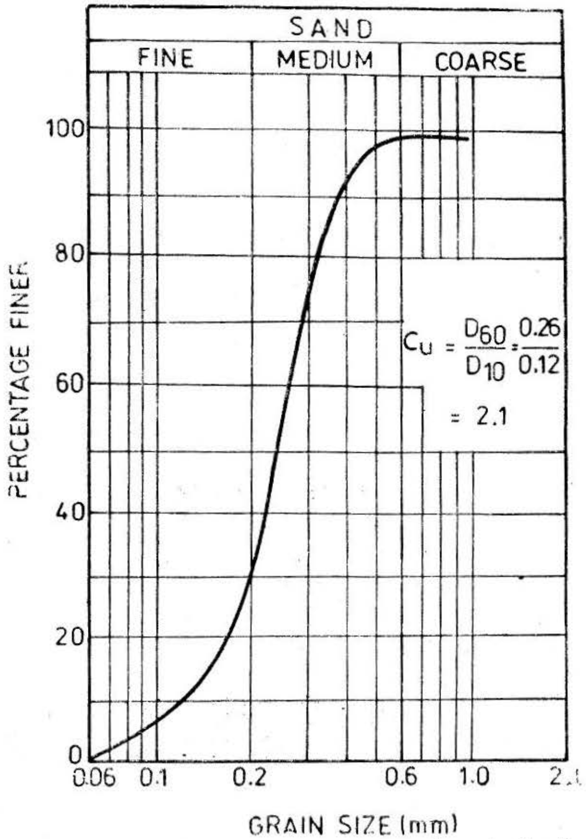

FIGURE 6 Particle Size Distribution Curve for Sand

Vibration techniques can not be used for obtaining a uniform density of sand when earth pressure cell are to be embedded in it. Raining techniques are quite suitable in such conditions (Walker & Whitkar, 1967). The density of sand in the raining technique depends upon the intensity of raining and the height of fall. A special type of raining equipment was designed and fabricated which consisted of container *2.75 × 0.3 × 0.15m made of mild steel plates* having a perforated plate at its base. Perforations of 3 mm dia, were made in the base plate at a spacing of 25 mm in the longitudinal direction and 18 mm in the transverse direction. The container was mounted on a trolley which could move on a track laid at a suitable height above the tank.

The trolley was brought near one end of the tank and the container was filled with air-dry clean sand. The trolley was then moved slowly manually. As the trolley moved, the sand was deposited in the tank by raining. The intensity of raining was constant since the size and spacing of perforations in the plate were fixed. Different densities were obtained by varying the height of fall. A large number of preliminary tests were conducted to obtain a relationship between the height of fall and the dry density of sand. The plot revealed that a dry density of 1.6 g/cc (Relative density 67 per cent) was achieved when the height of fall was equal to or more than 1.375 m. (Figure 7). The sand bed for the experimental

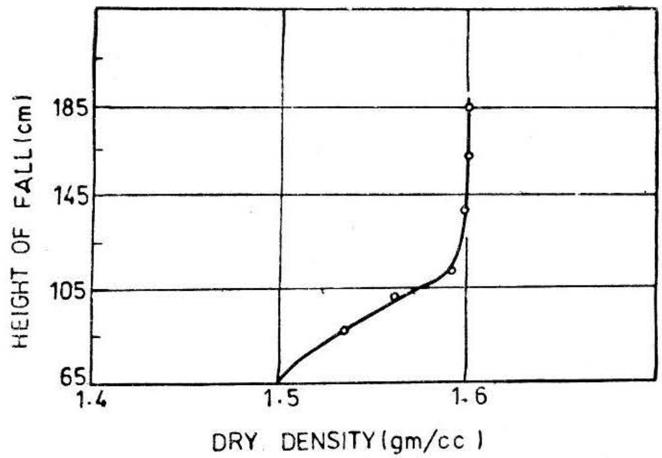

FIGURE 7 Relationship Between the Height of Fall and the Density of Sand

investigation was formed by adjusting the level of the trolley track such that the height of fall was 1.375 m or more.

Embedment of Earth Pressure Cells

For determination of stresses in the sand mass at various depths below the centre line of the footing, three free-earth pressure cells were embedded at depths of 12.5, 25 and 50 cm below the surface of the footing. These cells were placed on sand with their diaphragms at bottom as soon as the required level of sand was attained during the process of deposition of sand. The leads of the strain gauges of the free-earth pressure cells were taken out horizontally towards the side walls of the tank. The process of deposition of sand was continued after the pressure cells had been placed. The strain gauges of these pressure cells were connected to a separate strain recorder.

Loading Arrangement

Trusses were bolted to suitably designed anchor beams constructed on either sides of the foundation tank as shown in Figure 1. A steel joist was bolted across the trusses to support the hydraulic jack and provide reaction to it during loading. The jack was adjusted just above the centre of the footing. A proving ring of 5 t capacity was used to measure the load applied.

Dial Gauge Frame

A dial gauge frame was fabricated from the mild steel box sections. The frame was placed directly on the top of the walls of the tank (Figure 8). Dial gauges were fitted to the frame through slotted angles near the centre, at quarter points and near the periphery of the footing along the radial lines at right angles to each other.

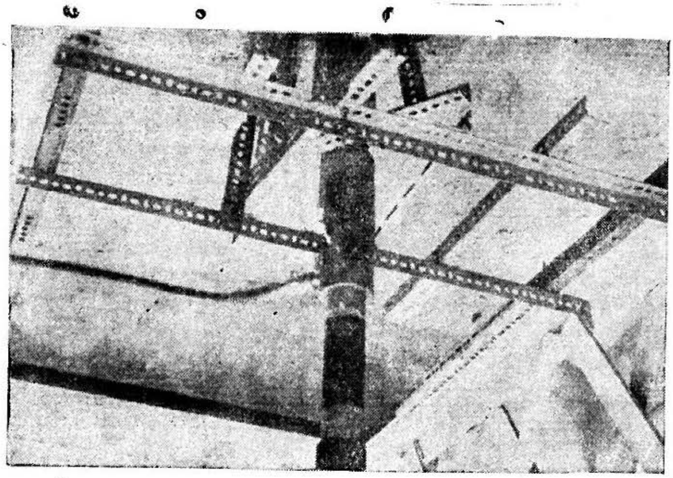

Figure 8 The Dial Gauge Frame

Procedure

After the sand bed had been properly formed, the footing was carefully placed over it. Dial gauges were fixed at the appropriate position over the footing. The boundary and free-earth pressure cells were connected to their respective strain recorders. The hydraulic jack was then gradually lowered over the footing. For reinforced cement concrete footings, a steel loading plate of the size $10 \times 10 \times 1.0$ cm was placed above the centre of the footing to safeguard against the punching shear. Initial readings of all dial gauges and pressure cells were recorded. The load was applied in increments of 0.25t. Observations of the dial gauges and pressure cells were taken after 10 to 15 minutes of the application of load when the readings became almost stationary. The loading was continued upto a load of 4 tonnes except in the case of footings, F-1 and F-2 in which excessive bending was noticed at a load of 2 tonnes and further loading had to be stopped at that level.

After the maximum load had been applied, the jack pressure was released gradually. The footing was then removed. After every test, the tank was emptied. The sand removed was dried, sieved and redeposited.

Results and Discussions

Load-Settlement Curves

Figure 9 shows the load-settlement curves drawn between the central load and the settlement for the five footings. It may be noted that as the stiffness of the system decreases, the settlement increases. The settlement of the most flexible footing *F-1* is about 67 per cent greater than that of the stiffest footing F-5 at a load of 2 tonnes. There is not much difference in the settlements of the three stiff footings F-3, *F-4* and *F-5*, the maximum variation in the settlements is only 5 per cent at a load of 4 tonnes.

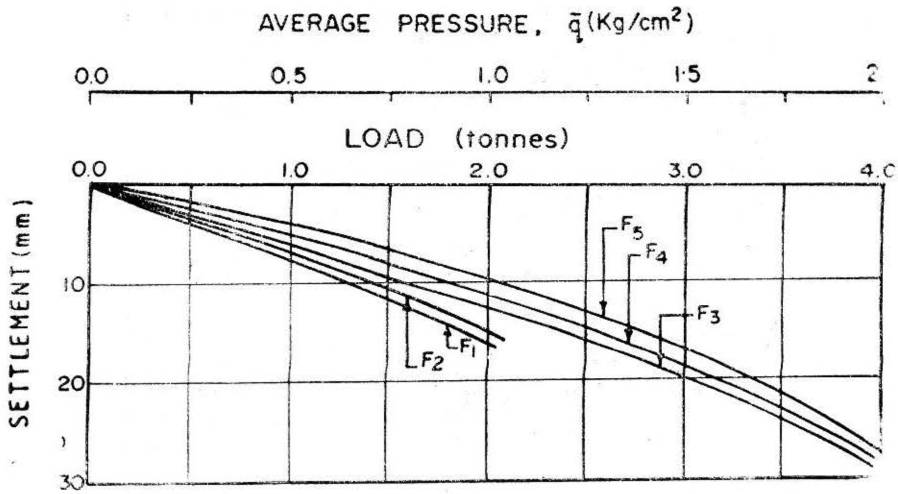

Figure 9 Load-Settlement Curves

The load-settlement curves for all footings are almost linear at low load levels and become slightly curved at higher loads. The load-settlement curves for the three stiff footings *F-3*, *F-4* and *F-5* are almost linear upto a load of 2.5t, which corresponds to about one-third of the ultimate bearing capacity as determined by Terzaghi's theory. It is observed that as the stiffness of the footing decreases, the range over which the load-settlement curve is linear is also reduced. For the most flexible footing *F-1*, the load settlement curve becomes non-linear even at a load of 1.5t. In order to keep the loading of the footings in the linear range, a factor of safety of 5 or so will have to be adopted for flexible footings, whereas the usual factor of safety of 3 is sufficient for the stiff footings.

For a settlement of 2.5 cm, the load at the centre of the stiff footings is relatively small, the settlement criterion gives large load. The settlement of 2.5 cm cannot be permitted in the flexible footings because they bend excessively before a settlement of 2.5 cm is attained.

Contact Pressure Distribution

Figures 10 and 11 show the normalised contact pressure distribution for the four footings *F-1*, *F-3* *F-4* and *F-5* at loads of 0.5 t, 1.0t, 1.5 t and 2.0t. Figure 12 shows the normalised contact pressure distribution for the three stiff footings *F-3*, *F-4* and *F-5* at loads of 2.5t and 3.0t. The contact pressure diagrams are parabolic. The measured contact pressures satisfy the equilibrium conditions of the footings.

It is observed that as the stiffness of the footing increases, the contact pressure distribution becomes more uniform. However, the parabolic shape is retained at all stiffnesses. In flexible footings, since the contact pressure is very high near the centre, it would be more economical to design the footings based on actual pressure distribution instead of the usual uniform distribution. Even at a load of 1 t, about 15 per cent saving can be effected by the use of the actual contact pressure distribution. For the relatively stiffer footings, the saving is not significant.

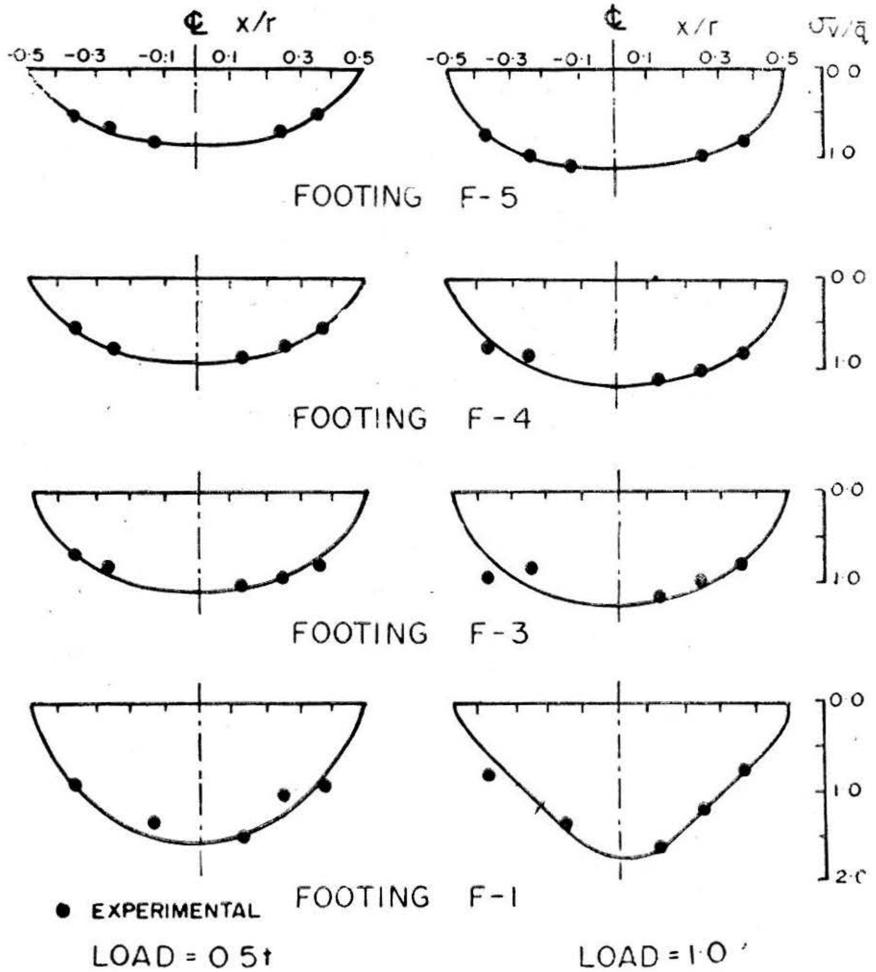

Figure 10 Contact Pressure Distribution at Loads of 0.5t and 1.0 t

It may be noted that for a particular footing, as the load increases, the normalised contact pressure increases at the centre and decreases near the edges. The phenomenon can be explained in terms of the relative stiffness of the footing—soil system. With an increase in load, the confinement pressure of the sand beneath the footing is increased. This causes an increase in the modulus of elasticity, E_s, for sand, and a corresponding decrease in the relative stiffness of the system. Thus at higher loads, the system becomes more flexible, and it would be more economical to design the footings based on actual parabolic distribution.

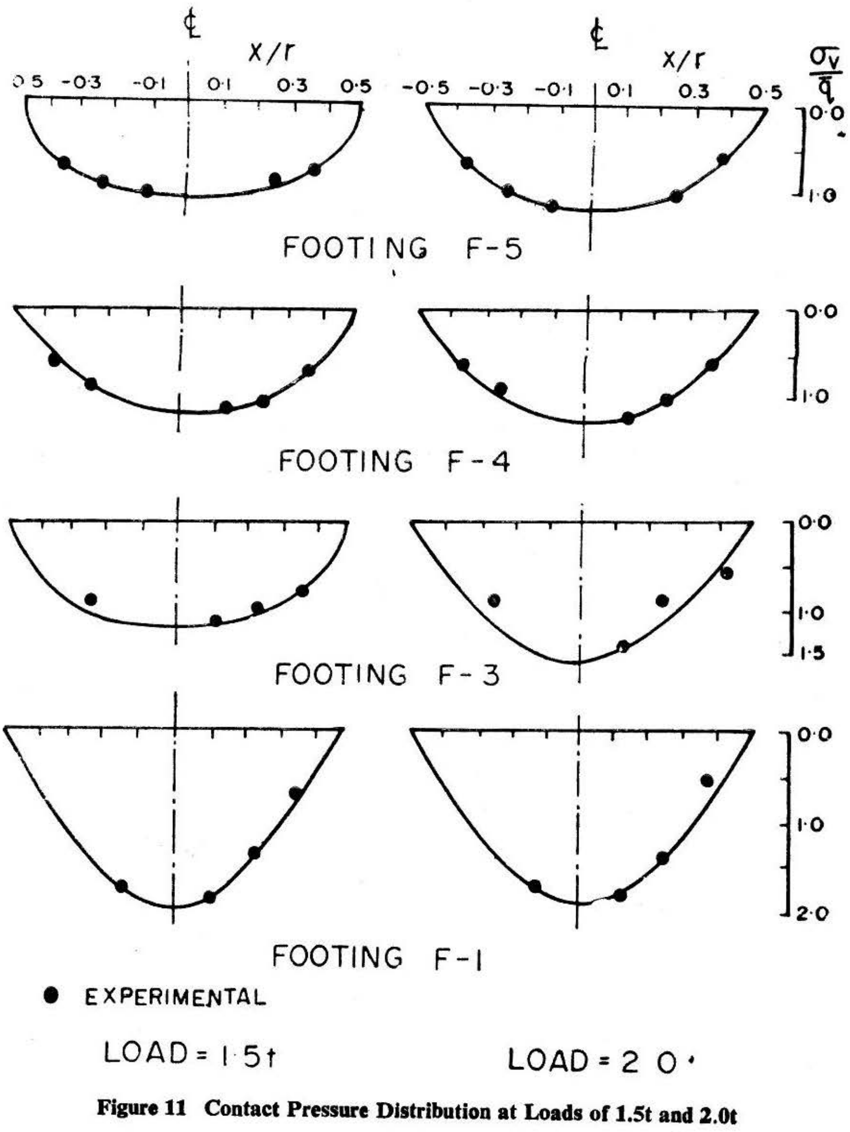

Figure 11 Contact Pressure Distribution at Loads of 1.5t and 2.0t

Vertical Stresses in Soil Mass

The normalised vertical stresses in sand below the centre of the footing *F-3* at loads of 1 t and 2 t are shown in Figure 13. The normalised vertical stresses due to Boussinesq's solution for absolutely flexible footings are also shown. It is noted that the measured vertical stresses are greater

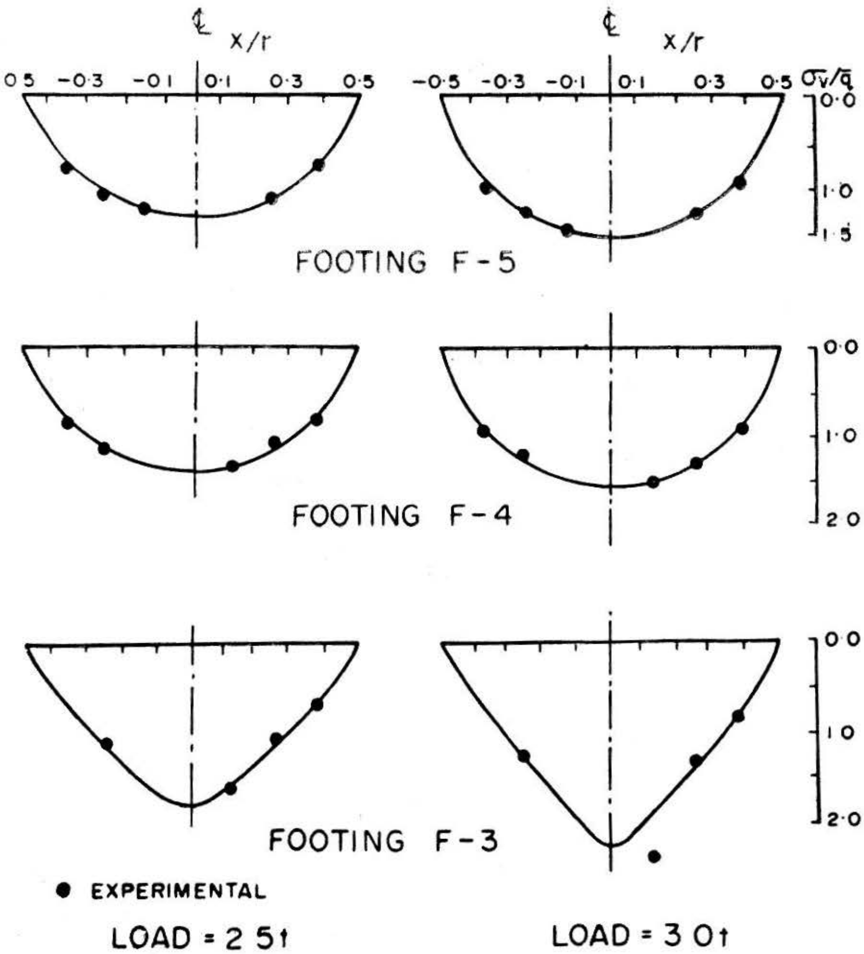

Figure 12 Contact Pressure Distribution at Loads of 2.5t and 3.0 t

than those given by the Boussinesq solution. As the load is increased from 1 to 2 t, the normalised vertical stresses increase at shallow depths upto Z/a equal to 0.75 but at greater depths, the stresses decrease. However, the measured vertical stresses are greater than those given by the Boussinesq solution at all loads and at all depths.

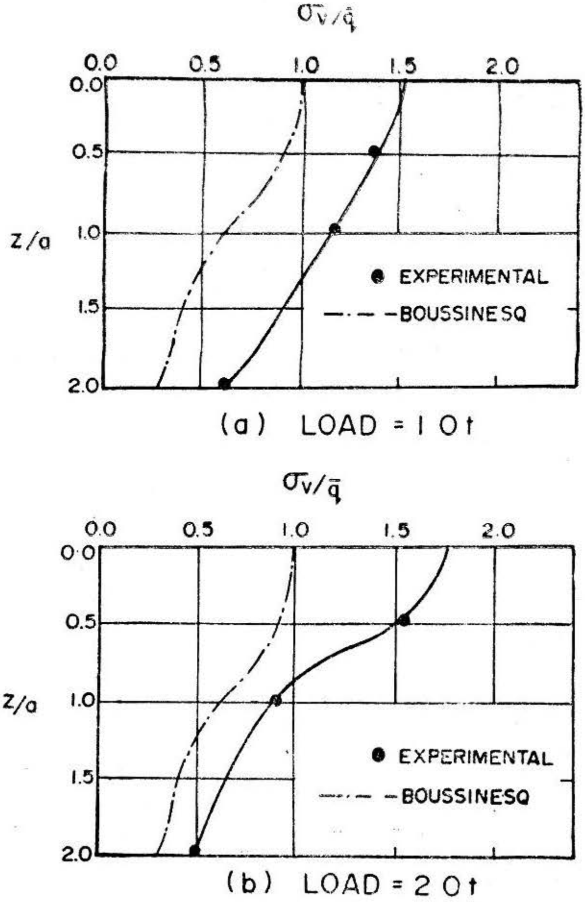

Figure 13 Variation of Normalised Vertical Stress with Depth

Conclusions

The relative stiffness of the footing-soil system has an important effect on the behaviour of the footings placed on sand. As the relative stiffness of the system decreases, the settlement at the centre increases. For a given footing, the load-settlement curve becomes non-linear as the load is increased. The range over which the curve is linear decreases with the decrease in stiffness of the system. In order to keep the working loading on the footing in the linear range, the factor of safety against bearing failure should be kept large for flexible footings. The usual 2.5 cm settlement criterion is not applicable for the flexible footings.

The contact pressure distribution becomes more uniform as the stiffness of the system is increased. For a given footing, with an increase in load, the relative stiffness is decreased. This results in an increase in contact pressure at the centre. Considerable saving can be effected if actual contact pressure distribution is used in design of flexible footings, especially at higher loads.

The normalised vertical stresses for stiff footings are greater than those given by the Boussinesq solution. The use of Boussinesq solution is not desirable in such footings.

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