3/45

An Experimental Study of Dynamic Bearing Capacity of Footings on Sand

Etude sur la variation de la capacité portante des fondations sur le sable sous une charge dynamique

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SUMMARY

The effects of variation of the loading rate on the bearing capacity of footings on sand are studied in this paper. Circular, rigid plates, placed on the surface of dry or submerged sand, were loaded to failure by dynamic, single loads. Loading velocities varied from about 10^{-5} in./sec to over 10 in./sec. Continuous records of load and settlement were secured.

Observations show a limited variation of the bearing capacity of dry sand with the loading rate. There is first some decrease, as the velocity increases to still moderate values of about 0.002 in./sec, followed by an increase for faster tests. This agrees with some observations of triaxial shear-strength variation. A similar trend exists if sand is submerged, and the loading rate is not excessive. For very rapid loadings, a pronounced increase due to negative pore water stresses occurs, along with changes in the failure mode.

SOMMAIRE

On présente une étude de la variation de la capacité portante des fondations sur le sable en fonction de la vitesse de chargement. Des essais dynamiques jusqu'à la rupture ont été faits sur des semelles circulaires et rigides, placées sur la surface d'un sable sec ou submergé. On a fait varier les vitesses de chargement de 0.2 μ/sec jusqu'à plus de 30 cm/sec.

On trouve une influence limitée de la variation de la capacité portante du sable sec avec la vitesse de chargement. Il y a d'abord une reduction, si l'on charge plus vite, jusqu'à près de 50 μ/sec. Au delà de cette vitesse, la capacité portante croit. Ceci est en accord avec quelques observations de la résistance au cisaillement au triaxial. Dans le sable submergé, on retrouve une tendance similaire pour les vitesses modérées. Si les charges sont très rapides, on note des accroissements de la capacité portante, dus aux pressions interstitielles négatives, ainsi que des changements du type de rupture.

DURING THE PAST DECADE, in connection with design of missile-launching and blast-resistant structures, an increased interest was shown in the problem of the bearing capacity of footings on sand subjected to single, transient loads, that is loads applied in single pulses of very short duration.

First light was shed indirectly on this problem through basic investigations of transient shear-strength and deformation characteristics of sand (Casagrande and Shannon, 1949; Seed and Lundgren, 1954; Whitman, 1957; Whitman and Healy, 1962) which indicated some variation of shear strength of sands with the rate of loading. The results of these investigations became especially meaningful after recent advances in understanding of shear and deformation phenomena under statically loaded shallow and deep foundations in sand (De Beer and Vesić, 1958; Vesić, 1963), as well as after the discovery of a rational correspondence between modes of failure of footings in static and dynamic loading conditions (Heller, 1964).

Useful data related to the problem can be gathered from results of small-scale model tests with impact-loaded footings (Shenkman and McKee, 1961; Cunny and Sloan, 1961; Fisher, 1962). Still, little information is available on the variation of bearing capacity of footings on sand as the rate of loading is varied through the entire range between static and impact loading conditions.

Following the extensive static loading tests with model footings performed at the National Geotechnical Institute of Belgium at Ghent in 1956–58 (De Beer and Vesić, 1958), a series of tests was performed in which the loading rate was increased up to ten times the normal static rate. The results of these tests, shown in Fig. 1a, indicate a slight, but definite decrease in bearing capacity with increased loading rate. It is of interest to note that this decrease was accompanied with reduction of size of the wedge of soil involved in shear (Fig. 1b).

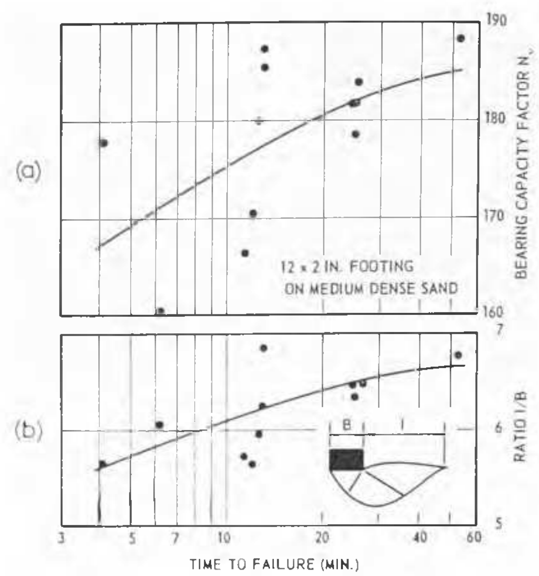

FIG. 1. Results of Ghent bearing capacity tests.

The trend of results of these early tests did not appear to be consistent with the findings of the investigations with impact-loaded footings, which all reported higher bearing capacities under impact loading conditions as compared to static loading conditions. It should be remembered, however, that triaxial tests on samples of sand described by Whitman and Healy (1962) showed a decrease of shear strength in the same range of loading rates (cf. their Figs. 17 through 19).

The purpose of the present investigation is to study the variation in bearing capacity of surface footings on sand as a function of the rate of loading. The basis for this paper is small-scale model investigations which have been performed in the Soil Mechanics Laboratory of the Georgia Institute of Technology since 1961.

DESCRIPTION OF TESTS

All tests have been performed with a circular, rigid, rough plate, having a diameter of 4 in. (10.2 cm) and resting on the surface of homogeneous models of dense sand. Models were prepared in a watertight steel box 50 in. (127 cm) square and 70 in. (178 cm) deep (Fig. 2). The box was equipped with an outside piezometer for control of water level.

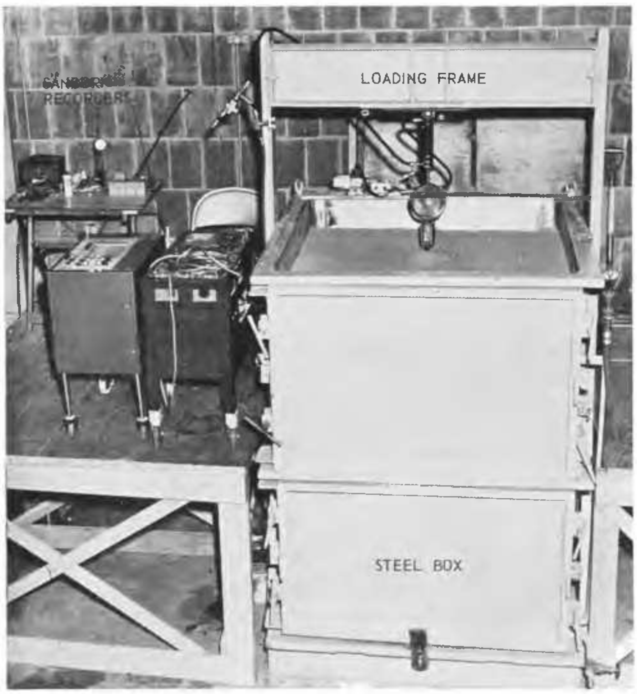

FIG. 2. Test area and principal testing equipment.

The characteristics of Chatahoochee River sand used in the investigations have been described in detail in a former paper (Vesić, 1963) and are summarized in Table I. Triaxial

TABLE I. CHARACTERISTICS OF CHATAHOOCHEE RIVER SAND

Mean particle size	0.37 mm
Coefficient of uniformity	2.5
Specific gravity	2.66
Maximum void ratio	1.10
Minimum void ratio	0.61

tests on cylindrical, air-dry samples of that medium, uniform, slightly micaceous sand indicate that its angle of internal friction ϕ in the low pressure range ($\sigma_3 < 10$ lb/sq.in.) varies with void ratio e as $\phi = \arctan (0.68/e)$.

Models of air-dry sand were prepared by surface vibration of 4-in.-thick sand layers obtained by pouring the material 30 in. from a perforated container. Electric vibrators attached to a rigid steel plate were used, with all possible care to obtain uniform densities throughout models. For models of submerged sand the material was brought in and first compacted by rolling in moist condition, followed by surface vibration in 4-in.-thick layers in submerged condition. The density and homogeneity of sand models was checked by static cone penetrometer soundings. For this purpose the previously described micropenetrometer (Vesić, 1963), with a point diameter of ½ in. and a shaft diameter of ⅜ in. was used. A total of 50 loading tests was performed, in three series. Two different loading systems were used.

The tests of series I, numbered 1 to 21, were performed on air-dry sand. To vary the loading rate, a mechanical loading arrangement was used (Banks, 1963). A steel beam was pivoted on one end and counterweighed, so that the model footing, attached to its other end, as well as the entire system, was in indifferent equilibrium. Different load rates were obtained by attaching the beam in a lever arrangement, by means of a connecting rod, to a 450,000-lb testing machine with a variable speed loading head. This and another similar arrangement permitted loading of the footing at rates varying from 10^{-5} in./sec to 1 in./sec.

Tests of series II and III, numbered 101 to 112 and 113 to 121, were performed on models of air-dry and submerged sand, respectively. A hydraulic loading system, described in detail by Woodard (1964), was used in these series. An electronically controlled hydraulic pumping assembly, designed to maintain an internal oil pressure of 1,400 lb/sq.in., was connected to a loading cylinder with a double acting piston (Fig. 3) through a system of valves (partly visible in Fig. 2). Owing to the high overpower factor in the system, practically constant pressure and piston velocity rate were

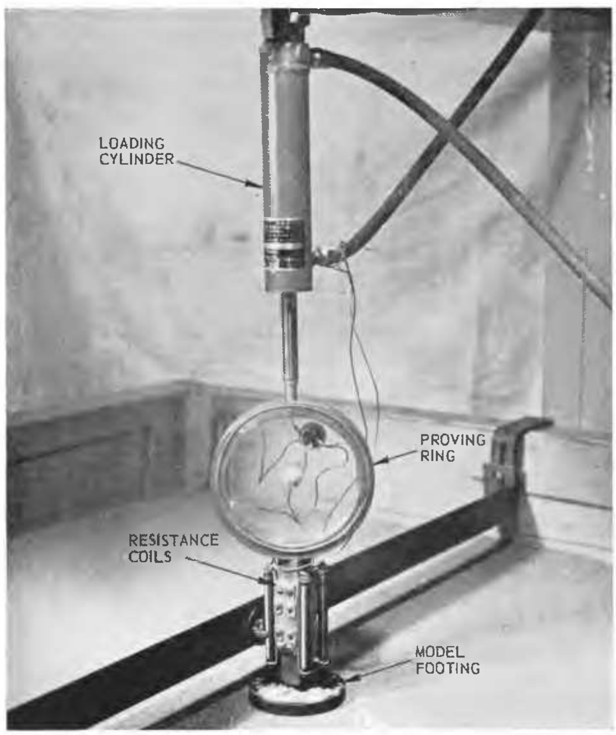

FIG. 3. Load and displacement measuring apparatus.

TABLE II. SIGNIFICANT RESULTS OF TESTS ON SUBMERGED SAND

Test no.	Dry unit weight γ_d (lb/cu. ft.)	Loading velocity (in./sec)	Ultimate settlement (in.)	Time to failure (sec)	Ultimate pressure q_{ult} (lb/sq. in.)	$\frac{q_{ult}}{1/2\ \gamma' B}$
113	96.8	0.000096	0.117	2,232	31.9	457
114	96.5	0.000288	0.156	820	30.3	435
115	96.5	0.000355	0.209	636	27.4	393
116	95.9	0.0020	0.200	106	15.5	222
117	96.4	0.0020	0.210	54	27.1	388
118	96.7	0.0021	0.279	135	23.9	342
119	96.3	0.0139	0.209	11.6	20.4	292
120	96.4	0.040	0.260	6.3	33.5	478
121	95.9	0.041	0.250	3.2	21.9	316
122	96.8	0.042	0.320	5.6	30.7	437
123	96.8	0.317	0.250	0.6	40.7	580
124	97.0	0.356	0.513	1.3	36.9	512
125	96.5	0.466	0.340	0.7	21.9	313
			(1.320)	(2.7)	(63.0)	(900)
126	96.3	0.576	0.348	0.7	(31.9)	455
			(0.925)	(2.0)	(63.8)	(910)
127	96.7	0.683	0.352	0.6	18.5	264
			(1.550)	(2.5)	(63.4)	(905)
128	96.4	0.744	0.352	0.6	30.9	442
			(1.125)	(2.0)	(66.1)	(946)
129	94.6	0.790	0.356	0.6	16.9	247
			(1.100)	(2.0)	(24.0)	(350)

NOTE: $\gamma' = \gamma_d (G_S - 1) / G_S$ = submerged unit weight. Numbers in parenthesis correspond to the second ultimate load.

TABLE III. SIGNIFICANT RESULTS OF TESTS ON DRY SAND

Test no.	Dry unit weight γ_d (lb/cu. ft.)	Loading velocity (in./sec)	Ultimate settlement (in.)	Time to failure (sec)	Ultimate pressure q_{ult} (lb/sq. on.)	$\frac{q_{ult}}{1/2\ \gamma_d B}$
1	95.9	0.931	0.324	0.58	26.3	237
2	95.7	0.878	—	0.66	28.6	258
3	95.2	0.842	0.349	0.71	25.8	234
4	96.3	0.641	0.313	1.0	23.3	209
5	95.8	0.500	0.228	1.3	28.4	256
6	95.6	0.265	0.274	2.3	27.4	248
7	95.8	0.120	0.274	5.1	21.8	197
8	95.5	0.102	0.278	6.4	24.7	223
9	95.3	0.0762	0.210	9.2	29.4	267
10	94.2	0.0283	0.544	31	21.3	196
11	95.7	0.0163	0.291	40	20.9	189
12	96.2	0.00621	0.212	100	25.9	232
13	96.1	0.00302	0.242	117	30.7	276
14	96.8	0.00090	0.358	861	21.0	188
15	96.1	0.00085	0.179	336	19.6	176
16	95.8	0.00076	0.306	894	21.9	198
17	95.8	0.00053	0.294	460	32.6	294
18	95.8	0.00052	0.281	536	29.3	265
19	96.4	0.00022	0.437	1,604	27.0	242
20	95.8	0.00011	0.466	2,664	24.7	223
21	96.0	0.00005		5,364	29.4	265
101	97.1	0.00002	0.163	7,066	43.3	385
102	96.7	0.00016	0.263	1,690	39.4	352
103	97.4	0.00030	0.301	1,560	35.7	316
104	97.4	0.00037	0.279	924	36.9	328
105	97.1	0.00176	0.333	400	30.3	269
106	97.1	0.00203	0.384	195	32.0	285
107	97.0	0.00220	0.286	142	31.2	278
108	96.8	0.080	0.329	4	33.3	297
109	96.6	0.110	0.311	3	36.9	330
110	96.7	2.54	0.356	0.3	38.3	341
111	97.2	9.26	0.407	0.04	39.5	351
112	97.0	12.26	0.419	0.05	40.1	357

assured for even the fastest tests. This system permitted loading rates to vary from 10^{-5} in./sec to over 10 in./sec.

Details of the measuring apparatus are shown in Fig. 3. The piston transmitted the load to the model footing through a proving ring, which enabled electronic recording of load intensity, as well as through a pair of resistance coils wired in parallel for electronic recording of displacements. A Sanborn strain-gauge amplifier, Model 64-500 B, was used to record the strains in the proving ring. Through the sensitivity chosen, the load was read directly to approximately 10 lb and estimated to 1 lb. To amplify the voltage change in the resistance coils a Sanborn D.C. amplifier, Model 64-300 B was used, enabling direct readings to 0.07 in. and estimated readings to 0.007 in. of displacement. Both load and displacement measuring devices were connected to Sanborn recorders, Model 60, which provided a continuous record of the two channels in function of time.

DISCUSSION OF TEST RESULTS

Typical load-settlement curves recorded in tests on dry sand are shown in Fig. 4 by full lines. Each curve had an initial linear section which, after reaching a maximum and a minimum, turned to another linear section, with a considerably reduced slope. Similar curves were recorded in all but the five most rapid tests on submerged sand (Nos. 125 through 129). In these tests the initial linear section gradually turned to the final linear section (Fig. 4, dotted

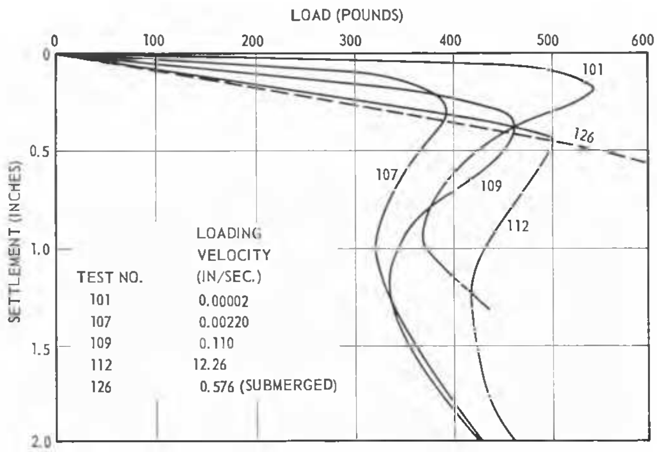

FIG. 4. Typical load-settlement curves.

line), with steadily increasing load. It is significant that general shear failure was observed in tests with the first type of load-settlement curve, while punching shear failure occurred with the second type of load-settlement curve.

Significant results of all the loading tests performed are assembled in Tables II and III. Ultimate loads given in these tables are defined in the same way as in an earlier bearing capacity investigation (Vesić, 1963). In the case of general shear failure, these are the first peak loads, corresponding to the appearance of failure surface at the sand surface and to an abrupt change of the rate of settlement from positive to negative. In the case of punching shear failure, two ultimate loads are defined. First ultimate load, distinguishable only in stress-controlled tests, can be noted when settlements reach magnitudes at which general shear failure would occur. In addition, a second ultimate load can be defined as the load at which the settlement rate first reaches its maximum.

The settlements corresponding to ultimate loads, expressed as percentage of the foundation diameter, are presented in Fig. 5. It is seen that, as in former observations (Vesić, 1963) ultimate loads on dry sand are reached at settlements of about 8 per cent of the foundation width. There appears to be some increase in ultimate settlements at very rapid loading rates, however. This is particularly evident from tests with footings on saturated sand. These footings appeared to fail at lower settlements than those recorded in corresponding tests on dry sand. The dotted line in Fig. 5 is the best fitting straight line that can be traced from available data for saturated sand.

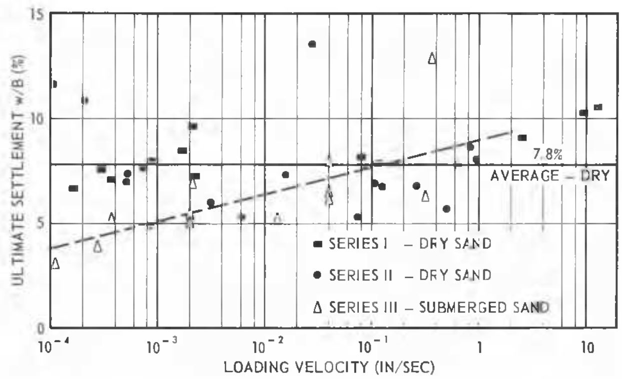

FIG. 5. Ultimate settlement *versus* loading velocity.

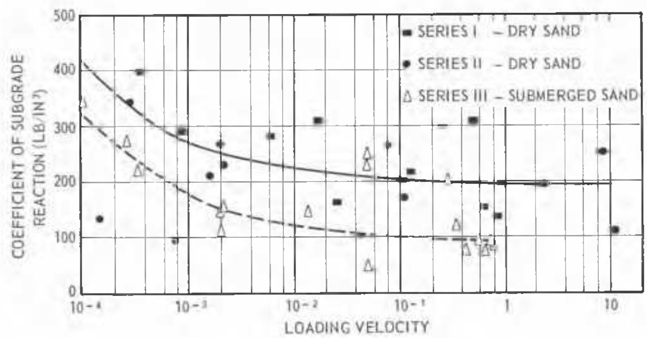

FIG. 6. Coefficient of subgrade reaction *versus* loading velocity.

Fig. 6 shows the slope of the initial part of the load-settlement curve, or the coefficient of subgrade reaction of the footing, expressed in lb/cu.in. as a function of the loading velocity. Two conclusions can be drawn from this figure. First, it re-confirms that the compressibility of submerged sand is higher than that of dry sand under analogous conditions. The coefficients of subgrade reaction appear to be proportional to the effective unit weights of the material. Second, and more important, it appears that the compressibilities of both dry and submerged sand increase as the loading velocities increase from slow to moderately rapid values. This is in agreement with observations made in the Ghent bearing capacity tests. However, further increase of loading velocities to very rapid rates does not seem to affect

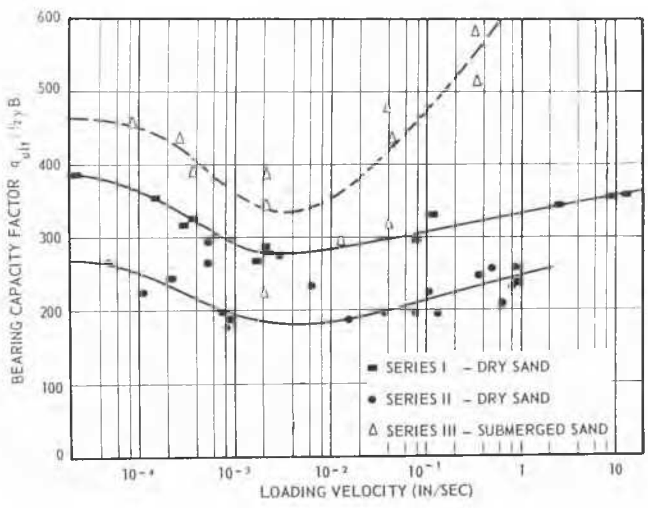

FIG. 7. Bearing capacity factor *versus* loading velocity.

the compressibility any more. It is of interest that in all investigations conducted with triaxial apparatus decreases of compressibility with the increase of loading rate were reported.

The variation of bearing capacity with the loading rate is presented in Fig. 7. Excellent results, with a relatively small scatter, were obtained in tests with dry sand. Considerably more scatter occurred in tests with submerged sand. However, it still can be seen that tests on dry and on submerged sand both indicate the same general trend. The bearing capacity first decreases as the loading velocity increases to still moderate values of about 0.002 in./sec. For more rapid tests there is a steady increase of bearing capacity. In tests with dry sand this increase is not very pronounced, so that the bearing capacities in the fastest tests, performed at velocities of over 10 in./sec, barely exceed the static bearing capacities. In tests with saturated sand, however, the increase is very pronounced, and the final bearing capacities are several times higher than static bearing capacities.

The observed static bearing capacities were generally in agreement with those observed earlier on the same sand (Vesić, 1963). As before, they were several times higher than the theoretical bearing capacities computed by using the Prandtl-Caquot or Terzaghi bearing capacity factors and an angle of internal friction determined by conventional triaxial tests. (The cause of this well-known discrepancy has been widely discussed and falls outside the scope of this paper.)

The observed minimum dynamic bearing capacities were about 30 per cent lower than the static bearing capacities. This difference corresponds to a decrease in the angle of internal friction of about 2°. A similar difference can be deduced from triaxial shear tests by Whitman and Healy (1962); however, this difference was believed to have been caused by some experimental errors. Since the present results come from a completely different type of experiment, which allows more precision and excludes the possibility of the same errors, it may be stated with certainty that there is a slight shear-strength decrease in dry sand as the loading rates increase from slow to moderately rapid values.

An increase in bearing capacity is observed for more rapid tests (rates exceeding about 0.002 in./sec). In tests on dry sand, the increase is not substantial, and it appears that the bearing capacities at the highest achieved loading velocities approximately equal the static bearing capacities. In tests on saturated sand the increases are considerably higher. However the settlements corresponding to increased ultimate loads are also very high and may reach 40 per cent of the footing diameter.

EXPLANATION OF OBSERVED PHENOMENA

To explain the observed variation of bearing capacity with the loading rate the nature of the rheological response of sand as a material must be understood. In a static, stress-controlled loading test on sand the particles need time to adjust their position to any new load increment. If this time is not allowed, there will be first an apparent drop of shearing strength or bearing capacity and an increase of deformation or compressibility. However, with further increase of loading rate another time effect is felt more and more: particles also need time to be displaced along considerable distances when shear failure occurs. If the loading is too rapid, they will not always be able to follow the paths of least resistance. The consequence will be higher shear strength and higher bearing capacity. This explanation is supported by observations of failure surfaces developed.

The more pronounced increase of bearing capacity of saturated sand at very rapid loadings is obviously due to negative pore water pressures. Such negative pressures have been measured in rapid triaxial shear tests on similar materials. In the present investigation their existence is best illustrated by the fact that the slopes of the depression caused by rapid forcing of the footing into saturated sand stood vertically for an instant, before collapsing onto the footing. This indicates clearly that an apparent cohesion was developed by negative pore water pressures.

Finally, the difference in failure mode observed (punching shear for very rapidly loaded footings *versus* general shear for statically loaded footings) can be explained by the inertial restraint of the soil potentially involved in shear. It can be shown (Heller, 1964) that this restraint has the same effect on the failure mode as an overburden pressure or a deep burial of the footing.

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