Engineering Properties of Dune and Beach Sands and the Influence of Stress History

Caractéristiques des sables de dune et de plage et l'influence de l'histoire des contraintes

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SUMMARY

Much of the coastal plain of Israel is covered by deposits of varying thickness of subangular uniformly graded beach and dune sands. This paper reports on extensive soil engineering studies on these sands with particular reference to the influence of stress history on the *in-situ* characteristics. Results are reported on limiting densities and the angle of internal friction. The angle of internal friction correlates well with the void ratio. Data showing the influence of overburden pressure on the blow count in the standard penetration test are presented. The influence of stress history is demonstrated for both standard penetration tests and plate-loading tests. The effect of the position of the water table on settlements in plate-loading tests is discussed. Data showing the influence of foundation width on settlements are presented for both virgin loading and reloading. For reloading, settlement was found directly proportional to foundation width.

SOMMAIRE

Une grande partie de la côte d'Israël est couverte de sables sous-anguleux de dune et de plage en couches variables et d'une granulométrie uniforme. Cette communication a pour objet l'étude détaillée de ces sables et, en particulier, de l'influence de l'histoire des contraintes sur leurs caractéristiques *in situ*. L'étude rapporte des résultats sur les densités limites et le frottement interne. On y voit une bonne corrélation entre l'angle de frottement et l'indice des vides. Des données sont présentées montrant l'influence de la surcharge sur le nombre des coups dans l'essai de pénetration standard. On y montre en outre l'effet de l'histoire des contraintes dans l'essai de pénétration aussi bien que dans l'essai de chargement de dalle. L'influence de la position de la nappe d'eau sur le tassement dans l'essai de dalle y est également traitée. On présente enfin des données sur l'influence de la largeur des fondations sur les tassements pour le chargement initial ainsi que pour les répétitions de chargement. Pour le rechargement les tassements sont directement proportionnels à la largeur de la fondation.

THE COAST OF ISRAEL consists of a broad belt of beach and dune sands in the extreme south, generally narrowing progressively northwards. The substratum of the coastal plain consists of Pleistocene deposits of sands, sandstones, limestones, and clays. The typical soil of the coastal plain is a slightly clayey sand. Much of the area is covered by deposits of varying thickness of clean uniformly graded beach and dune sands. There is at present most intensive building activity (housing, industry, harbour works, shipyards) in this sandy area. This paper will discuss and report on extensive soil engineering studies on these clean sands conducted by the laboratories of the Israel Standards Institution and the Israel Institute of Technology over the last several years.

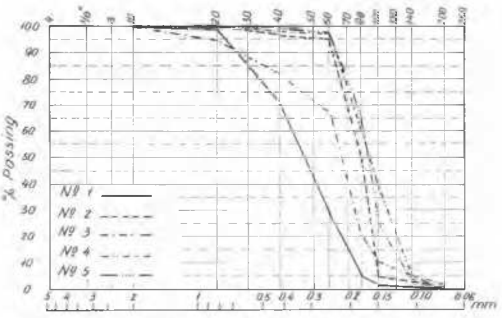

FIG. 1. Grain size curve.

SOIL PROPERTIES

Grain Size and Shape

The sands studied are uniformly graded, fine subangular sands. The grains are primarily quartz and it is generally believed that the beach sands of Israel are derived mainly from the Nile River with some contribution from the streams and sea cliffs of Sinai. Typical grain size curves are shown in Fig. 1, the median size ranging from 0.16 mm to 0.32 mm and the uniformity coefficient from 1.4 to 2.1 (Table I). Examination of the beach sand grains shows a fairly high degree of polish and a visually estimated roundness of 0.3 using the comparison charts prepared by Powers (1953).

Limiting Densities

In estimating the engineering behaviour of natural sand deposits and in specifying the desired degree of compaction of structural fills of sand, the relative density (D_r) is commonly used as the significant parameter. The relative density of a sand describes the *in-situ* density in relation to the maximum and minimum possible densities for the same sand. A study made of the various methods of obtaining maximum densities in the laboratory had shown that laboratory compaction using a 10-pound hammer (ASTM D 1557-58T) achieves densities as high as those achieved by any other method tried to date. (Frequently, maximum density is obtained at zero compaction moisture.) A more common method of describing field density is to determine its percentage of the maximum density. However, the percentage of maximum density can be a misleading measure of the field density. For example, if the ratio of maximum to minimum density of a sand is 1.25 a density of 80 per cent of maximum density corresponds to a relative density of zero or in other words, to the loosest possible packing. The ratio of maximum to minimum density has been found to vary over a comparatively narrow range for most sands tested, that is, from 1.17 to 1.35. Shockley and Garber (1953), in their study of natural alluvial sands of the lower Mississippi River valley, found that the maximum and minimum densities both increase with increasing peak grain diameter. Analyses of their data show that the ratio of maximum to minimum density varied from 1.18 to 1.30. For uniform spheres it can be shown that the ratio of maximum to minimum density is 1.4. The relationships between relative density, percent of maximum density, and ratio of maximum to minimum density are shown in Fig. 2. In Table I the limiting densities for the sands tested are shown.

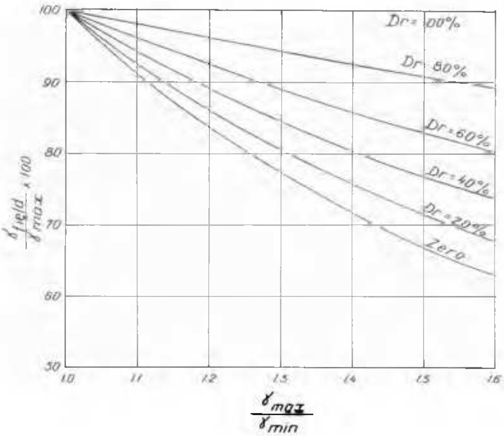

FIG. 2. Relative density, maximum and minimum density relationships.

TABLE I. GRAIN SIZE AND LIMITING DENSITIES

Soil no.	Location	Grain size (mm) D50	D10	$\frac{D60}{D10}$	Limiting dry densities Ton/m³ γ_{min}	γ_{max}	$\frac{\gamma_{max}}{\gamma_{min}}$
1	Ashdod 1	0.32	0.18	2.1	1.49	1.86	1.25
2	Holon 1	0.18	0.15	1.3	1.43	1.74	1.22
3	Kishon 6	0.22	0.15	1.6	1.27	1.72	1.35
4	Haifa Bay 2	0.16	0.11	1.6	1.41	1.65	1.17
5	Kishon 5	0.17	0.13	1.4	—	—	—

Angle of Internal Friction

Over the years numerous triaxial compression tests have been performed on sand samples compacted in the laboratory to various densities. Measured angles of internal friction varied from 30° to 32° at relative densities close to zero but increased to a range of from 36° to 45° at relative densities close to 100 per cent. The angle of internal friction

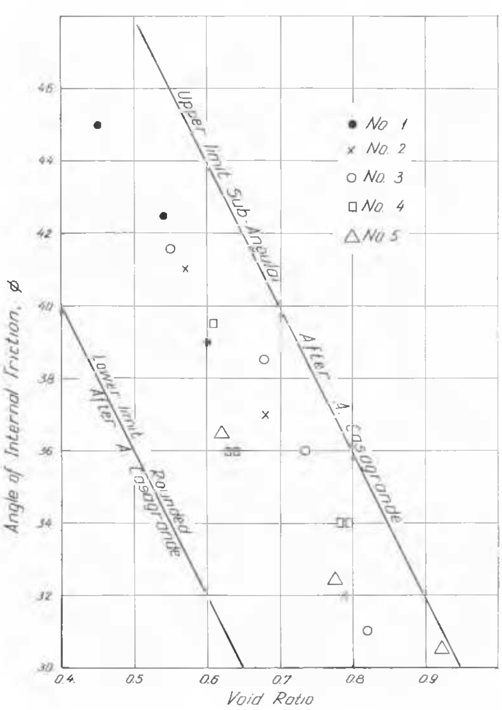

FIG. 3. Angle of internal friction *versus* void ratio.

versus void ratio was found to have less scatter. Some of the test results are shown in Fig. 3 for a range of void ratios between 0.45 to 0.85. A void ratio of 0.5 corresponds to an angle of internal friction of about 44°. For every increase of 0.1 in void ratio there is a corresponding decrease in angle of internal friction of about 4°.

EFFECT OF OVERBURDEN PRESSURE AND STRESS HISTORY ON STANDARD PENETRATION TEST RESULTS

One of the most commonly used methods of estimating the engineering behaviour of foundations on sand deposits is by means of empirical correlations with the standard penetration test. The test is carried out in Israel in accordance with the procedures of ASTM (D1586-63T). In interpreting the results of the standard penetration test, indiscriminate use of the Terzaghi-Peck correlations leads to overconservative design in most cases. In the last five years there has been a growing awareness by the engineering profession of the need for correcting the measured penetration values for the influence of overburden pressure (Gibbs and Holtz, 1957; Sutherland, 1963; Thorburn, 1963). The authors have for several years been using the correction for influence of overburden pressure suggested by Gibbs and Holtz. In general the results of field experience of the behaviour of structures and the results of many loading tests on dune and beach sands in Israel have indicated that the use of the overburden corrections for obtaining the relative density gives sufficiently conservative designs.

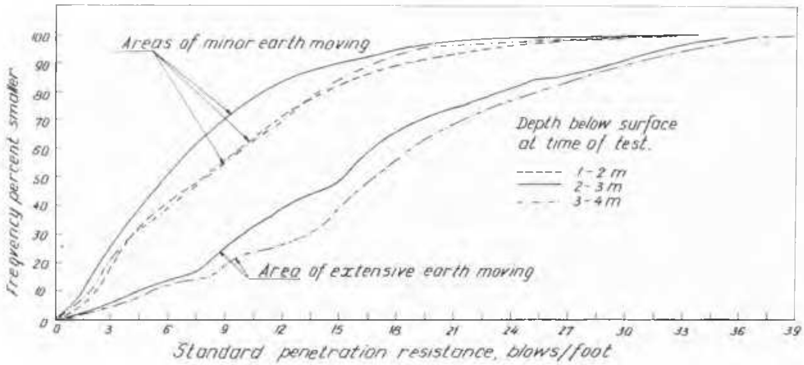

FIG. 4. Frequency distribution of standard penetration resistance for a dune sand.

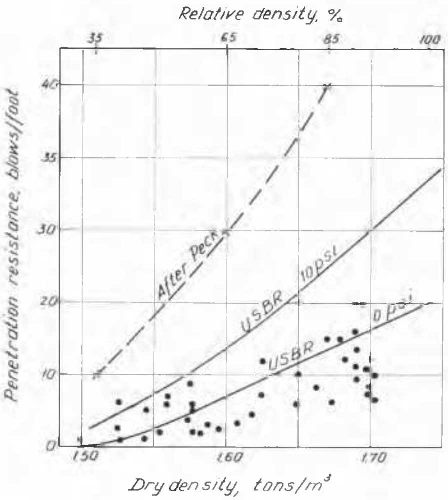

FIG. 5. Measured penetration resistance *versus* dry density for depths from zero to two meters for a dune sand.

In an area south of Tel Aviv (soil no. 2), over a thousand standard penetration tests have been performed in the upper three meters of a uniformly graded sand. The frequency distribution curves are shown in Fig. 4. The results of tests in the upper 2 m, where field densities were also determined, are shown in Fig. 5. The sands ranged in relative density from 35 to 90 per cent and the blows per foot from 1 to 20. Shown in the same figure is the relationship between the relative density and the blow count as suggested by Peck, Hanson, and Thornburn (1953) as well as those published in the *Earth Manual* of the United States Bureau of Reclamation (1960) for 0 psi and 10 psi overburden pressure. It may be observed that the measured correlation between relative density and blow count checks the 0 psi curve of the USBR rather well and for these sands for shallow testing in the upper two meters the curve suggested by Peck, *et al.* underestimates seriously the relative density of the sand.

In connection with the foundation exploration for a 35-storey reinforced concrete structure in Tel Aviv, standard penetration tests in cased holes were carried out from the original ground surface at elevation + 17.5 meters above sea level to a depth of 30 meters below original ground surface. After general excavation of the area to elevation +2.0 meters above sea level, further exploratory borings with standard penetration tests were initiated from elevation +2.0 to a depth of an additional 20 m. Clean uniformly graded sands were encountered over all the site with occasional strata of clayey sands and slightly cemented sands. Groundwater level was at about sea level. The removal of some 30 tons/sq.m. of overburden and the overlapping standard penetration tests allowed a unique opportunity for checking the influence of overburden pressure on the results of the standard penetration test.

Plotted in Fig. 6 are the results of standard penetration tests over a 9-meter depth interval from sea level and below. Each point shows the average results of three tests at each level both before excavation when the overburden pressure ranged from 33 to 43 tons/sq.m. and after general excavation for the same level when the overburden pressure ranged from 2 to 12 tons/sq.m. The results for each elevation have been given the same symbol in the figure and the results before and after general excavation for each layer have been joined by a straight line. Shown on the same plot is the relationship between blow count and overburden pressure for relative densities of 50, 65, and 85 per cent based on curves in the *Earth Manual* by the USBR (1960).

The plot shows the influence of overburden pressure at the time of test as well as the influence of stress history on the measured penetration resistance. The blow count which was originally between 20 to 30 blows per foot at an overburden pressure of about 40 tons/sq.m. corresponds to a relative density of 50 to 60 per cent. Upon removal of some 30 tons/sq.m. of overburden pressure the blow count dropped to 10 to 20 blows/foot indicating the definite influence of overburden pressure at time of test on the measured penetration resistance. It is interesting to note, however, that the reduction in penetration resistance upon reduction of overburden pressure is not as large as would be expected considering that the relative density has remained essentially constant. The explanation for the higher than expected blow count for the unloaded sand very possibly lies in the increased strength built into the sand due to pre-stressing and

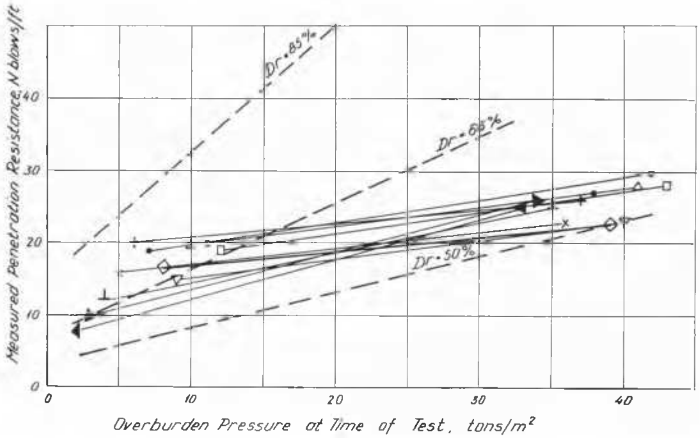

FIG. 6. Measured penetration resistance both before and after excavation.

the residual horizontal stresses remaining after removing the surcharge. In laboratory consolidation tests on similar sands in which lateral pressures were measured when the vertical pressure was reduced from 40 tons/sq.m. to 10 tons/sq.m. the corresponding drop in lateral pressure was only from 16 tons/sq.m. to 8 tons/sq.m. indicating a residual K_0 value of 0.8. Should such high K_0 values exist in the field after excavation this would explain the higher than expected penetration resistance for the corresponding vertical overburden pressure for the unloaded condition. From an inspection of Fig. 6 the pre-stress effects, though high for depths of from 5 to 10 meters, would appear to vanish almost entirely at shallow depths.

Settlement observations on the completed raft foundation

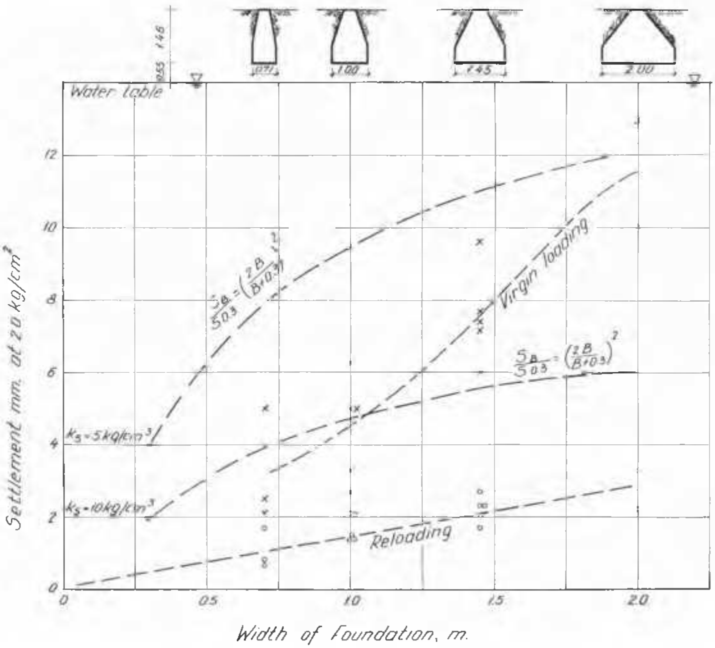

FIG. 7. Measured settlement *vesus* width of footing for a beach sand.

show relatively high values for the coefficient of subgrade reaction, thus confirming the increased rigidity of the sand due to the previous surcharge.

EFFECTS OF STRESS HISTORY, POSITION OF THE WATER TABLE, AND SIZE OF FOUNDATION ON THE RESULTS OF PLATE-LOADING TESTS

In connection with a foundation investigation for a passenger terminal in Haifa port, a series of loading tests on footings of various sizes were carried out on a dense beach sand. The tests were performed on square concrete footings having contact areas of 0.5, 1.0, 2.0, and 4.0 sq.m. All footings had surcharges of 1.45 m of sand and their bases were all 0.55 m above groundwater level. From two to four tests were performed for each footing size. Load increments were generally 0.5 kg/sq.cm. and the load was reduced to zero before proceeding to next load increment. The settlement at 2.0 kg/sq.cm. contact stress is shown for the various sizes of footings in Fig. 7. Shown are both the gross settlement for first loading and the settlement measured when reloading from 0 to 2.0 kg/sq.cm. after having been first loaded to 2.0 kg/sq.cm. Shown on the same plot is the expected settlement as a function of foundation width for coefficients of subgrade reaction of 5 and 10 kg/cu.cm. for 30-cm square plates based on the relationship given by Terzaghi and Peck (1948) for surface loading.

The shape of the curve obtained for both virgin loading and reloading in Fig. 7 would appear to be different from that given by Terzaghi and Peck. The settlement observations for virgin loading show very strongly the influence of the relative position of the water table on the settlement. For the smallest foundation tested, the water table was 78 per cent of the foundation width below the base whereas for the largest foundation plate the water table was only 28 per cent of the foundation width below the base. The data reported here, as well as numerous other tests on 30-cm square plates at various depths above the water table, support the contention that the settlement of a foundation with its base at the water table is twice the settlement to be expected for the same size plate on sand of the same relative density when the water table is a foundation width below the base of the foundation.

The influence of pre-stressing is very marked. In addition to reducing subsequent settlement, the sand behaves as an "elastic" medium, the settlement being directly proportional to the width of foundation for the range considered.

CONCLUSIONS

The following conclusions may be drawn for the subangular uniformly graded fine dune and beach sands studied.

1. Ninety-five per cent of maximum density corresponds to a relative density of from 70 to 80 per cent.
2. The angle of internal friction correlates well with the void ratio.
3. The blow count in the standard penetration test is markedly influenced by the overburden pressure existing at the time of test. The correlations published by the USBR of blow count with pressure and relative density would appear to be on the safe side for shallow depths for dune sand.
4. Consideration of past maximum overburden pressure is important in interpreting the results of standard penetration tests. Residual horizontal pressure are presumed responsible for high penetration resistances.
5. An increase in the ratio of depth of water table to width of foundation, of from zero to about one, reduces the settlement for the same contact pressure by a factor of about one-half.
6. After pre-stressing, foundation settlement tends to be approximately directly proportional to foundation width for foundation widths of up to 2.0 m.

ACKNOWLEDGMENTS

The authors wish to thank the Technion Soil Mechanics Laboratory and the Standards Institution for making the test results available. The authors also wish to thank Professor J. G. Zeitlen for his critical comments.

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