A Study of Stress Distribution in Geogrid-Reinforced Sand

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Abstract

Work conducted in this research aims at measuring the stress distribution with depth during a plate load testing program on geogrid-reinforced sand. A total of five load tests are performed; four on geogrid-reinforced sand and one on unreinforced sand. The tests are performed in a 1.52m x 1.52m x 1.37m(length x width x depth) steel box with a plate dimension of 0.33 m x 0.33 m. Results indicated that the magnitude of measured stresses for the reinforced sand was reasonably predicted using the Westergaard method for applied surface pressure of 28.7 kPa and 229.8 kPa, respectively. At the high stress of 430.5 kPa, methods based on both of Boussinesq and Westergaard distributions overestimated the data measured from the reinforced tests. Reducing the data in accordance with the approximate method (simplified load spreading), higher values of the angle of the stress distribution (α) were estimated for the reinforced sand as compared to the unreinforced samples which maybe indicative of a better attenuation of the stresses due to the inclusion of the reinforcement.

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Introduction

Shallow foundations are commonly used to support commercial and domestic buildings, storage tanks, bridge abutments, and military installations. Construction and performance of shallow foundations in cases where in situ soil conditions encompass loose or soft soils represent a continuous challenge to the profession.

Construction in loose and soft subsurface soil conditions may lead to unacceptable levels of deformation and excessive stresses on the superstructure. Consequently, a means of soil improvement may be needed. An emerging method of soil improvement for shallow foundation construction is to excavate a certain depth of the loose/soft soil and use mechanically stabilized backfill in which polymeric materials, including geogrids and geotextiles, are used for reinforcement. Using proper construction and installation, the merit of using polymeric reinforcement is to reduce the amount of deformation under the applied load by dissipating the stress to a level that can be sustained by the underlying soft soil. Therefore, improved load-deformation characteristics can be realized in terms of reduced total and differential settlement.

The evaluation of the stress distribution with depth due to surface loads for shallow foundation is a major requirement for the computation of the foundation' settlement. Presently, methods based on elastic solutions are used in practice. These methods which provide the distribution of the stresses in the vertical and horizontal directions, are relatively easy to use, and with some manipulation, they can be applied to complex foundation layout. However, there is a dearth of information regarding the adequacy of the elastic methods for predicting the stress distribution in a reinforced soil mass.

Work conducted in this research aims at measuring the stress distribution with depth during a plate load testing program on geogrid-reinforced sand. A total of five load tests are performed; four on geogrid-reinforced sand and one on unreinforced sand. The tests are performed in a 1.52m x 1.52m x 1.37m(length x width x depth) steel box with plate dimensions of 0.33 m x 0.33 m. Three layers of geogrid reinforcement are consistently used per reinforced sample with various spacing between the top layer and the test plate. The pressure distribution with depth is measured using six pressure cells; three are located at 0.33 m and three are located and 0.67 m below the test plate. The three pressure cells per layer consist of two 229 mm diameter cells placed on the outside of the plate boundaries and one 50 mm diameter cell that is placed directly under the center of the plate. The testing procedure regarding load increments and duration of loading is in conformance with ASTM 1194-95. A comparison between the stress distribution in the reinforced and unreinforced sand is presented along with a demonstration of the applicability of the elastic methods to the measured data.

Background

Evaluation of the stress distribution with depth due to surface loads is a major requirement for the design of shallow foundation and for the computation of the foundation' settlement. Traditionally, and as presented in many soil mechanics text books, for example Holtz and Kovacs (1981) and Das (1993), the stress distribution in unreinforced soils is evaluated using one of the following methods:

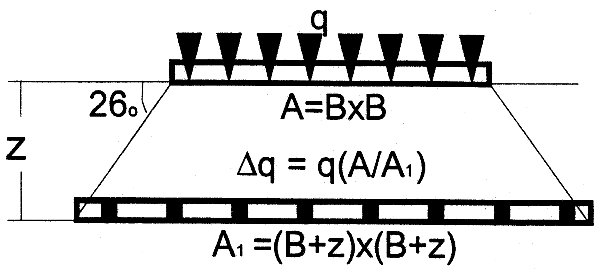

Figure 1. Simplified Load Distribution Scheme

i. Approximate Methods (simplified load spreading, Figure 1): in this case, the angle of propagation of the stresses from the foundation bearing level through the subsurface is assumed based on the soil type. A value commonly used in practice is 26° (which corresponds to 2:1 distribution.)

ii. Elastic Methods: a collection of these methods is presented in the text book by Poulos and Davis (1974). These methods are based on assuming that the soil behaves as a linear elastic medium. Original developments were carried out by Boussinesq and Westergaard with the latter presenting solutions for elastic soil embedded with alternating stiff layers. Elastic solutions for layered soil profiles and for soil profiles with rigid boundaries were later developed by Burmister (1956, 1962) and others.

iii. Numerical Methods: these methods are reserved for complicated foundation configurations or when it becomes necessary to incorporate soil non-linearity in the solution. In this case, the stress distribution and the settlement can be evaluated using the finite element method, the boundary element method, and/or the finite difference method.

There are advantages and disadvantages for each of the above categories of stress-evaluation methods. A main advantage of the approximate method is its simplicity and ease of use. An obvious disadvantage of this method is the lack of accuracy and the nature of the approximate results. Only average stresses can be evaluated at a given depth with no distinction given to the location at which these stresses are being evaluated under the loaded foundation area. In addition, the stress magnitude outside the zone of influence, defined by the distribution angle from the bearing level, is assumed to be zero.

The elastic methods provide the distribution of the stresses in the vertical and horizontal directions and are relatively easy to use. With some manipulation they can be used for complex foundation layout. A main disadvantage of these methods is the inability to account for the soil non-linearity and model soil irregularities. However, this category of methods is the one mostly used in today's state of practice.

The numerical methods are the most versatile and address the shortcomings of the other methods. With the advent of the computer technology, the use of the numerical methods has become more accessible to the engineering community. However, in addition to being relatively expensive and time consuming to apply, these methods are prohibitive in cases where foundation configurations necessitate 3-D modeling. In this case, few programs in the geotechnical arena are deemed suitable and even then the inability to estimate the proper input soil data renders the use of these programs in the state-of-practice questionable.

Past studies reported in literature, as presented for example by Guido et al (1985,1987), Das (1994a,b), Yetimoglu et al (1994), Ismail and Raymond (1995 a,b) and Adam and Collin (1996), reported an increase in the bearing capacity of the foundation when geogrid-reinforcement was included. In several instances, results from past studies indicated the presence of a critical u/B ratio (u=spacing between foundation and first layer of reinforcement and B=foundation width) for which the maximum increase in the BCR (defined as bearing capacity of reinforced foundation/bearing capacity of unreinforced foundation) was obtained. This u/B ratio was estimated to be between 0.25 and 0.75 depending on the number of reinforcement layers, spacing, and reinforcement stiffness.

Experimental Program

A total of five plate load tests were performed on sand. A steel test box was constructed for the performance of the plate load tests under controlled conditions. The dimensions of the box were selected to be 1.52m x 1.52m x 1.37m based on preliminary stress distribution analysis to ensure a minimal interference from the box boundaries on the test results. Three wire strain gages were mounted along the side of the box to monitor the occurrence of any induced strain during the application of the load increments. These gages did not register any increasing strain magnitude during loading. The test plate was 0.33m x 0.33m square plate and the size of the soil samples was 1.52m x 1.52m x 1.22m. The box setup is presented in Figure 2.

The load increments were applied to the test plate using a hydraulic jack with 71 kN capacity. The load was applied in increments with each increment maintained until settlement rate was less than 0.05 mm per hour. The magnitude of a given pressure increment at the surface was monitored using an electronic load cell. The testing procedure regarding load increments and duration of loading was in conformance with ASTM 1194 (1995) "Bearing Capacity of Soil for Static Load and Spread Footings".

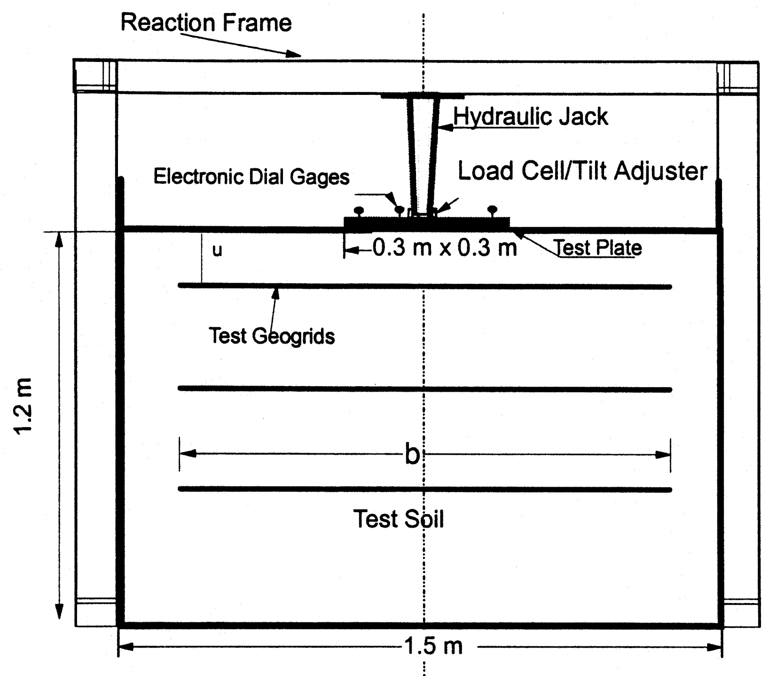

Figure 2. Test Box and Configuration

The pressure distribution with depth was measured using six Model 4800 Geokon earth pressure cells; three were located at 0.3 m and three were located and 0.6 m below the test plate. The three pressure cells per layer consisted of two 228 mm diameter cells placed on the outside of the plate boundaries and one 50 mm diameter cell that was placed directly under the center of the plate. The pressure cells consisted of circular stainless steel plates that were welded together around their periphery and spaced apart by a narrow cavity filled with antifreeze solution. High-pressure stainless steel tubing connects the cavity to a pressure transducer. External pressure acting on the cells is balanced by an equal pressure induced in the internal fluid. This pressure is converted by the pressure transducer into an electronic signal that is transmitted by a four-conductor shielded cable (direct burial type) to the Vibrating-Wire readout box (Geokon Incorporated, 1996).

Test Soil

The test sand known as "Ohio River sand" is typically used in mortar and concrete mixes. Grain size analysis (ASTM D 422-95), specific gravity tests (ASTM D 854-95), and relative density tests (ASTM 4253 and 4254) were performed on sand specimens in accordance with ASTM Standards (1995). Results from these tests indicated that the sand was uniformly graded with an average specific gravity of 2.7, a maximum dry unit weight of 18.8 kN/m^3 and a minimum dry unit weight of 15.7kN/m^3. The grain size distribution of the test sand is shown in Figure 3. The sand contained less than 3% finer than #200 sieve and has a coefficient of uniformity (c_u) equal to 8 and a coefficient of curvature (c_c) equal to 1.0, indicating a well graded

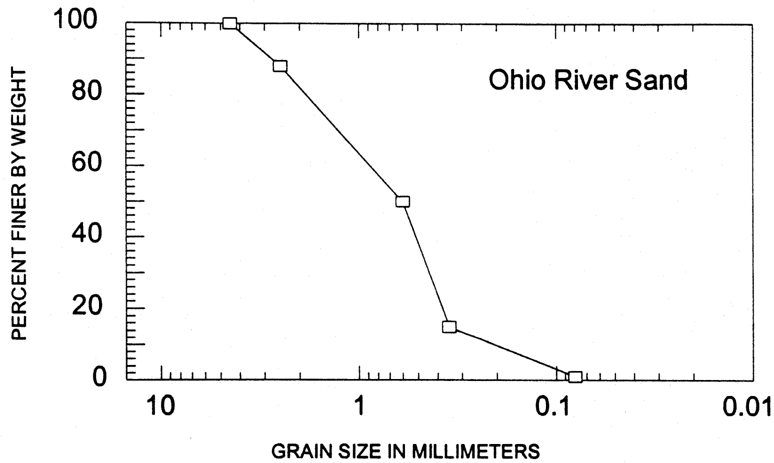

Figure 3. Grain Size Distribution of Test Soil

sand. This sand is classified as SW according to the Unified soil Classification system (USCS).

Test Geogrid

Two grades of biaxial geogrids were utilized in the testing program. The biaxial geogrids were made from polypropylene with apertures of 25 mm and 33 mm in the machine and cross machine directions, respectively, and have a minimum resin content of 99%. The carbon black content was 1%. The geogrids used in testing were trimmed to the size of 1.49m x 1.49m. The tensile strength in the machine and cross machine directions, respectively, was 204 and 292 kN/m for GR1 and 270 and 438 kN/m for GR2.

Sample Preparation Technique

Figure 4 (a,b) shows the lift thickness configuration and the locations of the pressure cells for a top geogrid spacing of 150 mm and 300 mm, respectively, with a uniform spacing of 300 mm. The locations of the pressure cells are also marked on the figure. The number of geogrids for all of the reinforced tests was 3.

The soil was compacted in the test box in lifts using a jackhammer fitted with a 203 mm x 203 mm tamping plate. The jackhammer delivered approximately 40 mm-kN blows at the rate of 1100 blow/minute. The compaction commenced in one corner and proceeded to the other corner while staying on each plate footprint for

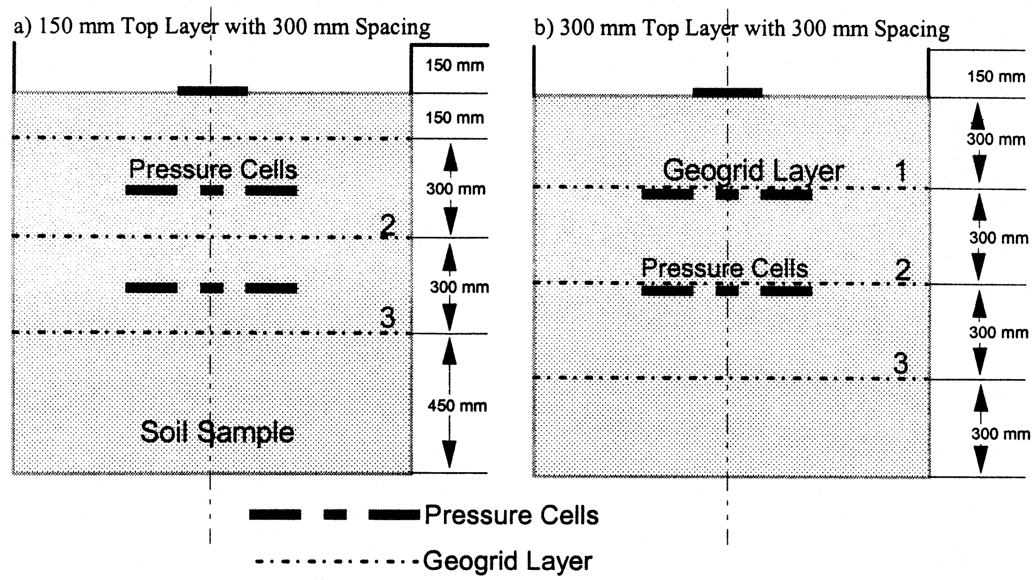

Figure 4. Configuration and Test Spacing

approximately five seconds. This process was repeated until the entire lift surface was uniformly compacted. After the completion of compaction, a layer of geogrid was installed and the next lift was consequently prepared.

The nuclear density/moisture gage was used to measure the moisture content and density distribution per lift according to ASTM D2922-95 for density and D3017-95 for moisture content. The nuclear gage was oriented in the long direction with its sides parallel to the box's sides and the nuclear moisture/density tests were performed for a duration of one minute in the direct transmission mode. Five tests per lift were performed; at the four corners and at the center of the soil sample. The unit weight achieved for the tests ranged from 17.3 kN/m^3 to 17.9 kN/m^3 (relative density on the order of 60%) with water content that ranged from 1.00% to 2.50%. Results of triaxial tests on the sand yielded an effective friction angle equal to 38.6°.

Testing Program

Table 1 shows the number of tests and test configuration used in the experimental program. A total of five tests were conducted on the sand samples. Two tests with the GR1 grids were performed to estimate the effect of the spacing of the first layer (152 mm and 305 mm were used in this study) on attenuating the stresses. One test was performed using GR2 geogrids with u=229 mm to discern the effect of the geogrid stiffness.

Table 1. Testing Program: Number of Tests and Geogrids Spacing

	Reinforcement Spacing				No Reinforcement
	GR1			GR2	
Top Layer Spacing	152 mm[1]	305 mm[1]	229 mm[2]	229 mm[2]	
u/B ratio	0.5	1	0.75	0.75	
Sand	1	1	1	1	1

[1] Bottom layers uniform spacing =305 mm
[2] Bottom layers uniform spacing =229 mm

Pressure Distribution

The measured pressure distributions for the five tests are shown in Figure5 (a,b,c). The results are presented in terms of stress influence factor (I) which is defined as the measured stress at a given depth divided by the surface pressure. The stresses measured by the pressure cells are the total stresses induced under the center of the test plate.

As apparent in Figure 5 (a,b,c), the stress influence factor (I) is a function of the load level and increases as the loading level is increased. This behavior may be expected as the stress-strain characteristics of the test soil are non-linear and can be attributed to the variation of the soil modulus with the increased loading level. Under the applied surface stress of 229.6 kPa, an I value of 0.55 was obtained at 0.3 m depth for the unreinforced case. In comparison, an I value of 0.25 was obtained for the case of GR2 with 228.6 mm uniform spacing versus 0.35 for the GR1 geogrids with the same spacing. At the same surface stress level and for GR1, reducing the top spacing from 305 mm to 152 mm resulted in reducing the I value from 0.3 to 0.25.

Boussinesq and Westergaard Stress Distributions

The stress distribution under the corner of a rectangular loaded area over semi infinite isotropic elastic mass was given by Newmark (1935) and was obtained by integrating Boussinesq's equation for a point load to obtain the following:

$$\Delta q = \frac{q}{4\pi}\left[\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2+m^2n^2+1}\,\frac{m^2+n^2+2}{(m^2+n^2+1)} + \tan^{-1}\frac{2mn(m^2+n^2+1)^{1/2}}{m^2+n^2-m^2n^2+1}\right] \quad (1)$$

where q=applied surface stress, m =B/z and n=L/z, BxL= area dimensions, and z=depth.

In comparison, the stress distribution beneath a rectangular area over a semi-infinite mass reinforced by horizontal strata that prevent the deformation in the horizontal

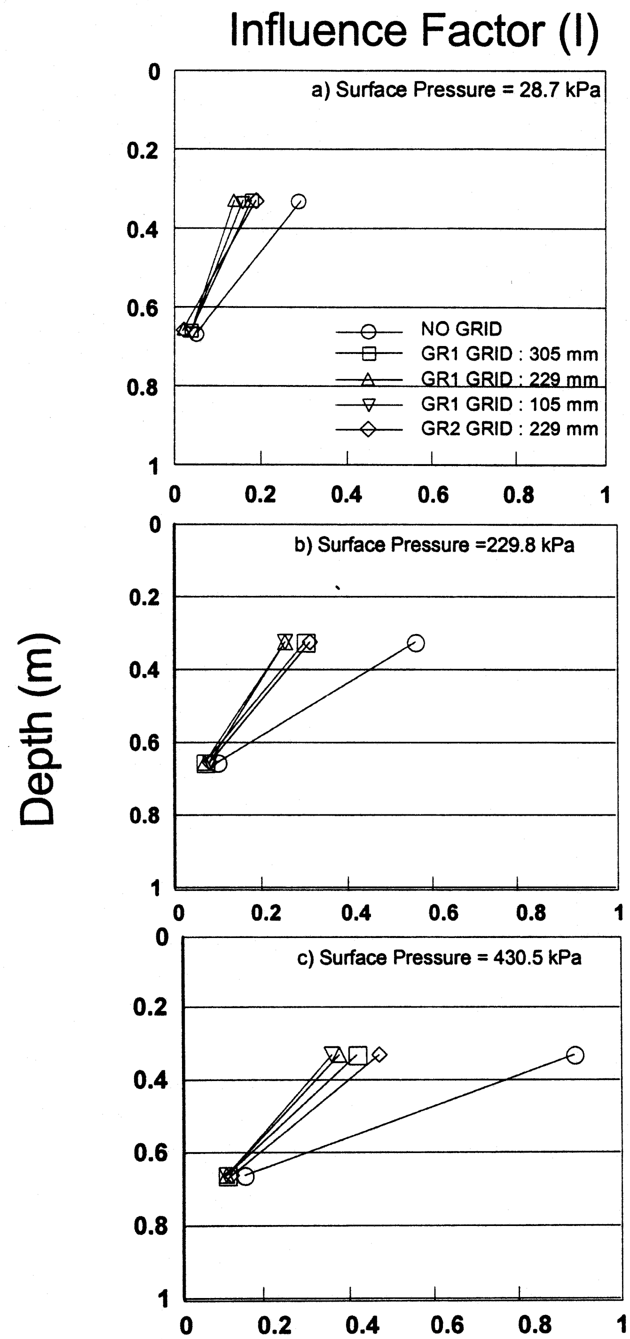

Figure 5: Stress Influence Factors from Measurements
as a Function of Applied Stress

direction was given by Fadum (1948) in the form of influence factors ($\Delta q = q.I$). The solution was developed by integrating the following equation by Westergaard (1938) over a rectangular area:

$$\Delta q = \frac{Q}{z^2}\frac{C}{2\pi}\left[\frac{1}{C^2+(r/z)^2}\right]^{3/2} \text{ and}$$
$$C = \sqrt{\frac{1-2\mu}{2(1-\mu)}} \tag{2}$$

where Q=point load, z=depth of the point of interest at which the stress is evaluated, and r=offset distance of the point of interest from the location of load application. Harrison, and Gerrard (1972) indicated that the Westergaard solution corresponds to the limiting case of horizontal soil modulus=∞.

For the sake of clarity, the distributions of the stress with depth at a relatively low surface pressure (28.7 kPa), medium (229.8 kPa), and relatively high pressure (430.5 kPa) are presented in Figure 6(a,b,c) along with stress estimation from methods based on Boussinesq and Westergaard stress distributions. These stress data indicated a zone of influence that was approximately 1 m (3B).

In comparing measured and computed stresses, the Westergaard stress distribution matched the reinforced data while the Boussinesq distribution was rather conservative under surface stress of 28.7 kPa. On the other hand, and at the relatively high surface stress level of 430.5 kPa, both methods overpredicted the measured data with a closer match with the reinforced data obtained using the Boussinesq distribution. In this case, the measured data were underpredicted by approximately 50% when the Westergaard stress distribution was used. Both methods yielded rather unconservative predictions for the unreinforced case under surface stresses of 430.5 kPa. This may be due to the fact the both of these methods were developed with the assumption of a material having linear stress-strain characteristics which is not the case under relatively high stresses.

Approximate Method

Applying the concept of the approximate stress distribution to these data, the angle of stress propagation (α) can be estimated for the tests on the reinforced sand. Assuming a "2:1" distribution for the unreinforced sand, the angle of the stress distribution for the reinforced sand (α) was estimated for the different tests and presented in Figure 7. The following assumptions were advanced for computing α using the approximate method:

1) The angle α is estimated using the stresses measured under the center of the plate

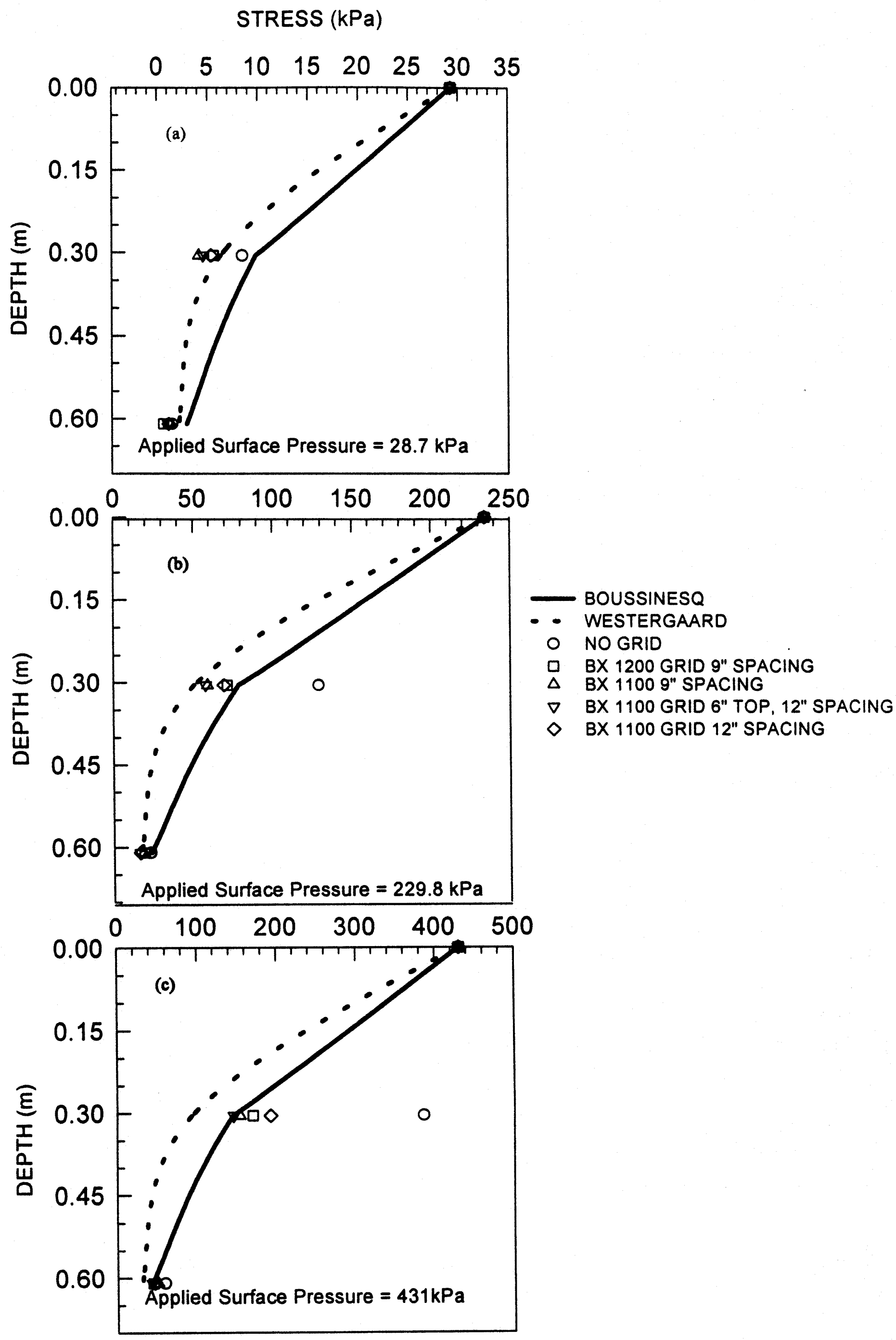

Figure 6: Measured and Predicted Stress Distribution with Depth for Three Surface Stresses

2) The stress magnitude outside the zone of influence delineated by the α values is zero.

As shown in Figure 7, the angle of the stress distribution (α) for the unreinforced case ranged from 23° at surface pressure of 28.7 kPa to 1.6° at surface pressure of 430.5 kPa. In comparison, the angle of the stress distribution (α) increased as the geogrid reinforcement was introduced. Compared to the case of no reinforcement, the largest increase in α was observed in the tests where the GR1 geogrid was used. At the relatively low surface pressure of 28.7 kPa, the α estimated from the GR1 tests was increased from 34° for the 305 mm spacing to an average of 40° when 152 mm top spacing was used. A significant dependancy on the stress level was observed by which the α values decreased with increasing load level.

A smaller α was estimated for the GR1 test with 229 mm uniform spacing. In this case, α decreased from approximately 39.5° for surface stress of 28.7 kPa to approximately 18.5° for surface stress of 430.5 kPa. The test utilizing the GR2 and 229 mm uniform spacing indicated an α value of 32.5° for surface stress of 28.7 kPa and 16.5° for surface stress of 430.5 kPa, respectively.

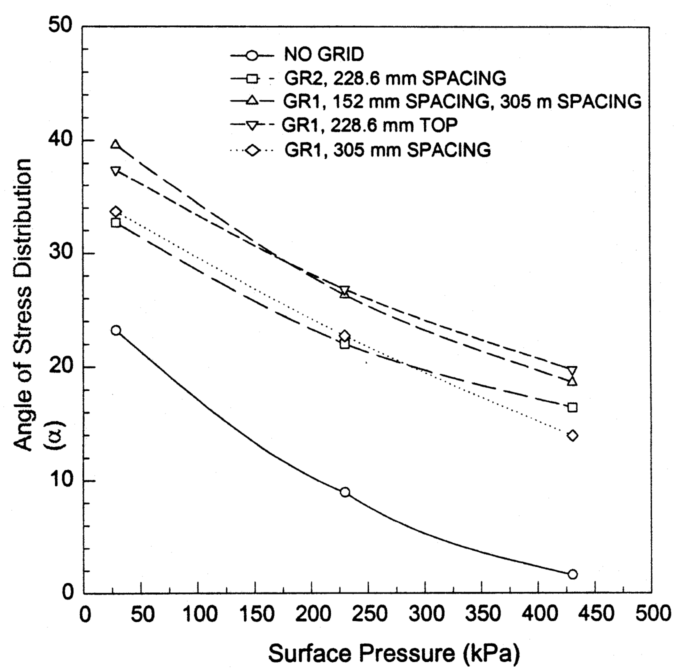

Figure 7: Variation of Angle of Stress Distribution α as a Function of Applied Surface Pressure

Summary and Conclusions

An experimental study was conducted to evaluate the stress distribution in sand reinforced with stiff polymeric geogrids. Two types of geogrids were used; GR1 and GR2 and a total of five plate load tests were performed. Six pressure cells were placed throughout the testing box in two layers to measure the stress distribution with depth. A non-linear stress increase was measured as the applied surface pressure was increased with the consistent trend of lower measured stresses with the inclusion of the geogrids. Based on the results obtained from this study, the following conclusions can be advanced:

1. The magnitude of measured stresses for the reinforced sand was reasonably predicted using the Westergaard method for applied surface pressure of 28.7 kPa and 229.8 kPa. At the high stress of 430.5 kPa, computed data from both Westergaard and Boussinesq distributions overpredicted the measured stresses.

2. The measured data for the unreinforced test were consistently underpredicted using both methods (Boussinesq and Westergaard) when the applied surface stresses exceeded 28.7 kPa.

3. Reducing the data in accordance with the approximate method, higher values of the angle of the stress distribution (α) were estimated for the reinforced sand as compared to the unreinforced samples.

4. In the case of the unreinforced sand, the angle of the stress distribution (α) ranged from approximately 23^0 at surface pressure of 28.7 kPa to 1.5^0 at surface pressure of 430.5 kPa. In comparison, the (α) for the GR1-152mm top spacing case ranged from 40^0 at surface pressure of 28.7 kPa to 19^0 at surface pressure of 430.5 kPa.

5. Lower (α) values were observed as the surface pressure was increased for both the reinforced and the unreinforced tests. However, the rate of α-reduction was larger for the case of the unreinforced sample.

6. The applicability of the results, and conclusions, presented in this study is limited to conditions similar to those used during the testing program.

The results presented in this study indicated an improvement in the plate load carrying capacity and reduction in the stresses with depth as the geogrids were included. This may be explained by the confining and interface friction effects of the test geogrids. In effect, the inclusion of the geogrid layers transform the compacted soil to a highly anisotropic material with relatively larger modulus, as compared to the unreinforced soil, due to the soil-geogrid interface shear resistance as well as the influence of confinement on the lateral deformation and stress propagation. However, in order to fully understand the load-deformation behavior of reinforced soils, instrumentation of the geogrids should be performed to estimate the stress taken by the geogrids and correlate it to the phenomenon of stress reduction.

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