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Lateral Pressure Developed During Compaction

ZVI OFER

The relation between the horizontal and vertical components of stress at a point within a soil mass depends on the physical and mechanical properties of the soil and its stress and strain histories. A series of compression tests and repeated loading tests were performed on sand and clay by using the lateral soil pressure ring MkII. This newly developed apparatus allows the laboratory determination of the lateral soil pressure response to vertical loading at rest condition or, alternatively, when limited controlled lateral strain develops. Correlations between the horizontal to vertical stress ratio (K) and the vertical load are presented. After placement of a cohesionless sample, when the material is loose, K is high and it decreases to an ultimate value. Unsaturated clay specimens exhibit somewhat different behavior—K decreases to a minimum value and then it increases with an increase in the load. During the unloading process K increases with an increase in the overconsolidation ratio, and a general relation between K and the overconsolidation ratio is suggested. However, repeated loading results in a decrease in K with an increase in the vertical load. It was also found that the grain size and distribution of grain size affect K. The present testing system does not have facilities for pore water pressure and dynamic loading measurements, which would enable more comprehensive testing and determination of soil parameters in terms of effective stress.

The relation between the horizontal and vertical components of stress at a point within a soil mass depends to a great extent on the stress and strain histories of the soil mass and the degree to which it is remembered. Other basic soil properties such as grain size and shape, grain size distribution, moisture conditions, and Atterberg's limits also affect the stress ratio. The elastic parameters of the soil [e.g., Poisson's ratio (μ)] and the modulus of elasticity (E) depend on the stress-strain characteristic of the soil as well. These parameters are major engineering considerations in many facets of road design and are used extensively in the design of layer thickness, earth-retaining structures, culverts, and slope-stability analysis.

The horizontal-to-vertical stress ratio at a point within a soil mass is defined as the coefficient of lateral earth pressure. This coefficient varies between a lower limit, when a soil element is allowed to expand laterally subsequent to vertical loading, and an upper limit, when a soil element mobilizes as a result of lateral thrust. These two extremes are defined as the coefficient of active lateral pressure (K_a) and the coefficient of passive lateral pressure (K_p), respectively. When lateral yielding is prevented, the ratio of the horizontal to vertical stress is known as the coefficient of lateral pressure at rest (K_o). These coefficients are correlated to the angle of internal friction of the soil (ϕ). However, this correlation is invalid in the case of highly overconsolidated materials. During a process of compaction or while clay soil swells as a result of an increase in its moisture content, the coefficient of lateral pressure at rest (K_o) may exceed unity value that results from combined effect of soil dilation and locked-in stress and, therefore, it cannot be correlated to ϕ.

The objective of this study was to examine the development of lateral stress in a remoulded soil mass during the compression process and to correlate it to several basic elastic and mechanical soil parameters that are of interest to the road engineer. This paper describes an instrument that allows laboratory determination of the lateral soil pressure response to vertical loading without lateral yielding and with limited controlled lateral yielding. A series of repetitive loading tests performed on river sand and remoulded norite clay is described and test results are presented and discussed.

LABORATORY K_o COMPRESSION TESTING

A laboratory K_o compression testing method must satisfy two requirements:

1. The soil must deform freely in the vertical direction and
2. The lateral yielding must be either zero or of negligible magnitude.

If either of these requirements is not satisfied (e.g., either the vertical yielding is restrained or lateral yielding occurs) the tested specimen is mobilized toward an active or passive failure condition. At rest condition is simulated in the laboratory by using either a modified triaxial testing or a modified oedometer compression testing technique.

A simple sensing device that consists of a metal band clamping a triaxial soil specimen was developed by Murdock (1). The metal band has an adjustable screw and a lamp connected to a power source. Before a vertical load increment is applied, the electrical circuit is closed by adjusting the screw until the lamp lights up. The radial deformation that results from an increase in the vertical load cuts the electrical circuit. A subsequent increase in the cell pressure recovers the radial deformation until the electrical circuit is closed and the lamp lights up again. At this stage no lateral strain occurred and the horizontal-to-vertical stress ratio at rest (K_o) is determined.

A visual lateral strain indicator was introduced by Bishop and Henkel (2). It consists of two curved metal strips joined together by a low friction hinge on one end and a mercury reservoir with a diaphragm and a capillary tube outlet on the other end. The indicator clamps a soil sample placed in a triaxial test apparatus. The lateral deformation of the specimen is indicated by the level of the mercury in the outlet tube and it is possible to maintain the soil sample at rest condition by changing the vertical load and the cell pressure simultaneously. A similar concept of monitoring the radial yielding of a triaxial soil specimen has been used by many researchers and various visual and electrical indicators were developed and used (3-8). The radial deflection of a diameter is measured and at rest condition is reinstated by means of adjusting the triaxial cell pressure until the lateral yielding is zero.

The horizontal-to-vertical stress ratio during a loading test can also be determined by means of a modified oedometer test. The concept behind this testing method is that the lateral stress developed in a soil specimen placed in the modified oedometer ring during a loading test results in a small but measurable yielding of the oedometer ring wall. The resultant strain is recorded by means of strain gauges cemented to the oedometer ring wall and connected to a strain recorder, or alternatively, by means of a pressure transducer attached to the oedometer or test chamber wall (9-11). A section of the oedometer ring wall is trimmed to a thickness of less than 1 mm and the sensor is attached to this section. Note that some strain develops during the test that, in return, may affect the recorded lateral stress (12-14). True K_o conditions can be achieved only when the oedometer thin wall section is a part of an annular chamber filled with fluid. The strain of the thin section is registered electrically and it is maintained constantly at zero by continuous adjustment of the annular chamber pressure to compensate for the change that results from the variation in the vertical load (15,16). This concept is implemented in the instrument used for this study. The instrument is a null-type apparatus that allows the laboratory determination of the lateral soil pressure response to vertical loading during either at rest conditions or when lateral strain develops.

LATERAL SOIL PRESSURE RING MkII

A section through the lateral soil pressure ring MkII (LSP MkII) is shown in Figure 1. The apparatus is a modified oedometer ring that consists of a main ring (A) and a tightly fitted casing (b). The main ring has an inside diameter of 70 mm, outside diameter of 103.4 mm, and height of 42 mm. The middle section of the ring is trimmed to a wall thickness of 0.8 mm and a long strain gauge (C) is cemented to the thin section along its center line. Electric wires (D) connect the strain gauge to a digital strain indicator, and air pressure is supplied through a vent (E) drilled in the main ring. When the casing is placed on the main ring, an air-tight annular chamber is formed. A photograph of the components of the calibration and testing system is shown in Figure 2. The LSP MkII (A) and a temperature compensator (B) are connected to a digital strain indicator (C). The ring and a piston (D), which simulates a soil specimen, are connected independently to an air pressure supply system that consists of a main air pressure supply line (E), pressure regulators (F), and pressure gauges (G).

For the calibration of the ring the piston is placed inside the ring. Two calibration tests are performed. First, air pressure is introduced into the piston and the resultant hoop strain of the ring wall that corresponds to each pressure increment is recorded. The relation between the air pressure in the piston and the corresponding hoop strain gives a calibration chart that represents the relation between the lateral soil pressure developed during an actual test and the ring wall hoop strain that results from the soil pressure when a small lateral strain is allowed to develop. Another calibration test is performed by using a null method--air pressure is introduced into the piston in small increments. Simultaneously, the pressure in the annular chamber is increased so that the ring wall hoop strain maintains a null condition. This test correlates the lateral pressure developed in a soil specimen with the annular pressure for an actual test.

Both calibration tests render consistent results and linear pressure-strain and pressure-annular pressure relations. The coefficient of linear correlation for any of the test recordings of 500 random readings within a simulated soil sample lateral pressure range of 0-600 kPa was better than 0.999.

MATERIALS

Two materials were used in this study:

1. Honeydew sand and
2. Rustenburg black clay.

Honeydew sand is river sand of weathered granite origin obtained from a sand quarry located about 25 km northwest of Johannesburg. The dominant minerals are quartz and felspar with some mica. The specific gravity of the particles is G_s = 2.65 and the particles' shape is angular and subangular. Nine different gradings were tested. Their physical properties are listed in Table 1.

Rustenburg black clay is a highly expansive clay obtained from the Rustenburg platinum mine about 100 km west of Pretoria. X-ray diffraction analysis of the clay shows that the dominant mineral is smectite with some traces of kaolinite. The clay is residual of norite origin, and it is listed among the most highly expansive soils in South Africa (17). The particle size distribution of the clay is shown in Figure 3 and its physical properties are listed in the table below.

Item	Amount
Liquid limit	80 percent
Plastic limit	28 percent
Plasticity index	52 percent
Linear shrinkage	18 percent
Clay fraction, D <0.002 mm	52 percent
Activity	1.00
Potential expansiveness (18)	Very high
Unified soil classification	CH
Specific gravity of particles (G_s)	2.71
Maximum standard AASHO dry unit weight (γ_d)	14.22 kN/m³
Optimum standard AASHO moisture content	26 percent

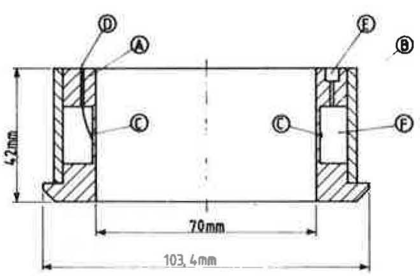
TEST PROCEDURES

All sand specimens were molded and tested in an air-dried condition (moisture content, w = ±2 percent). The sand was placed in the LSP MkII ring in a very loose condition (D_r < 15 percent) and the desired density was achieved by vibrating the specimen lightly under a vertical load of 12.7 kPa. The specimen was then statically loaded up to a vertical pressure of 634.5 kPa and unloaded back to

12.7 kPa. During the loading and unloading cycles, the vertical deflection and the lateral pressure that correspond to each vertical load increment were recorded. Each specimen was subjected to four loading and unloading cycles. Some specimens were tested when lateral deflection was allowed, and some comparative triaxial compression tests were carried out by using Bishop and Henkel's K_o belt (2).

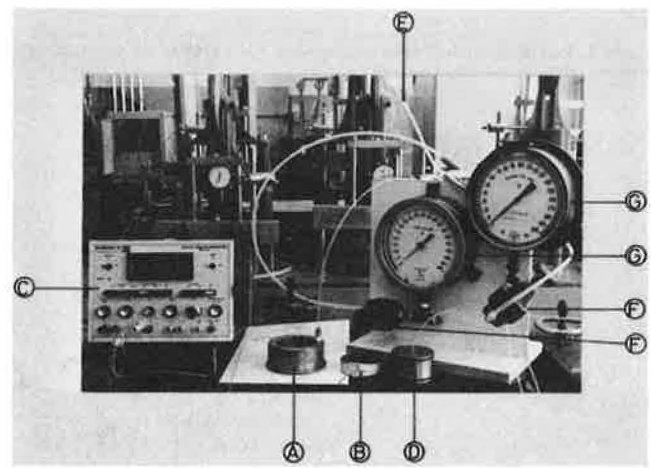
The clay was air dried before the test to a moisture content of 6 percent. Samples at four different moisture contents (20, 24, 27, and 30 percent) were prepared and stored at a constant temperature of 26°C for two weeks. Each specimen was placed in the LSP MkII ring in three layers, 10.8-mm thick (0.425 in), and the desired initial density was achieved by means of static vertical load. Samples at initial dry unit weight of 10.8 kN/m³, 12.3 kN/m³, and 13.7 kN/m³ were prepared. At the end of the initial compression process the vertical load was removed and the ring was placed in a consolidation loading frame under a vertical load of 12.7 kPa. The clay specimen was allowed to relax for 15 min and then the vertical deflection and lateral pressure were recorded. The specimen was then loaded in increments of 126.9 kPa up to a load of 761.4 kPa and then unloaded in double increments back to a vertical load of 12.7 kPa. The load increments were applied at a frequency of one increment per minute and at the end of each period the vertical deflection and the lateral pressure were recorded. Four loading and unloading cycles were performed and the clay specimen was allowed to relax for 15 min between the loading cycles. Some samples

Figure 1. Section through LSP MkII.



A = Main ring
B = Casing
C = Strain gauge
D = Electric wires
E = Air pressure vent
F = Air-tight chamber

Figure 2. LSP MkII calibration and testing system.

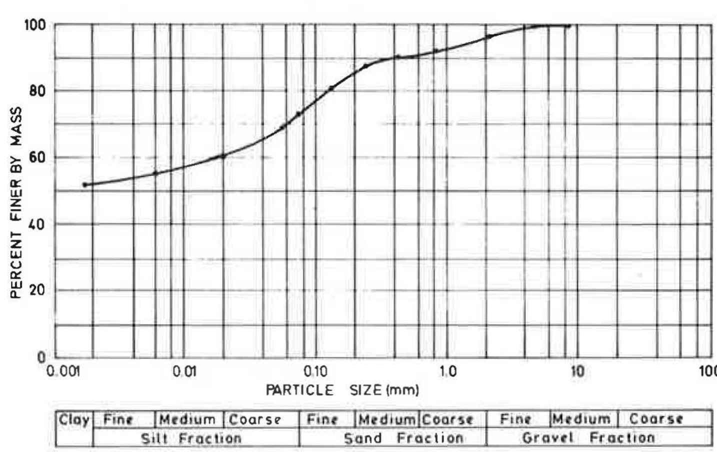


A = Lateral soil pressure ring
B = Temperature compensator
C = Digital strain indicator
D = Calibration piston
E = Air pressure pipe
F = Air pressure regulator
G = Pressure gauge

Table 1. Physical properties of Rustenburg black clay.

	Uniform Sand				Poorly Graded Sand				
Item	1	2	3	4	5	6	7	8	9
Maximum grain diameter (mm)	2.360	1.200	0.600	0.300	2.360	2.360	2.360	2.360	2.360
Minimum grain diameter (mm)	1.200	0.600	0.300	0.075	0.075	0.075	0.075	0.075	0.075
Effective size (D_{10})	1.420	0.750	0.360	0.108	0.260	0.200	0.160	0.128	0.095
30 percent fractile diameter (D_{30})	1.570	0.085	0.390	0.147	0.720	0.540	0.350	0.275	0.172
60 percent fractile diameter (D_{60})	1.720	0.885	0.440	0.182	1.450	1.050	0.790	0.525	0.392
Coefficient of uniformity (C_u)	1.21	1.18	1.22	1.69	5.58	5.25	4.94	4.10	4.13
Coefficient of curvature (C_c)	1.01	0.98	0.96	1.10	1.38	1.39	0.97	1.13	0.79
Unified soil classification	SP	SP	SP	SP	SP	SP	SP	SP	SP

Figure 3. Particle size distribution of Rustenburg black clay.



were tested when the period between the loading cycles was longer, and two specimens were subjected to normal consolidation tests to study the effect of time on the development of lateral pressure.

TEST RESULTS AND DISCUSSION

Tests on Sand

Some results of sand compression tests are presented in Figures 4-7. Typical relations between the lateral-to-vertical stress ratio and the vertical pressure are shown in Figure 4. At a very loose condition and under a low vertical pressure the lateral-to-vertical stress ratio is high and it decreases to an ultimate value with an increase in the vertical pressure and the subsequent densification of the sand. The stress ratio is also affected by grain size--grading 1 is a uniform coarse sand and grading 4 is a uniform fine sand. The stress ratio of the uniform coarse sand is considerably lower than that of the fine sand. Grading 9 is poorly graded sand, 50 percent of its mass is fine sand (grading 4) and 12.5 percent of its mass is coarse sand (grading 1). The stress ratios during a compression test of grading 9 are somewhat lower than those of grading 4.

Figure 4. Lateral-to-vertical stress ratio-vertical pressure relations for tests on sand.

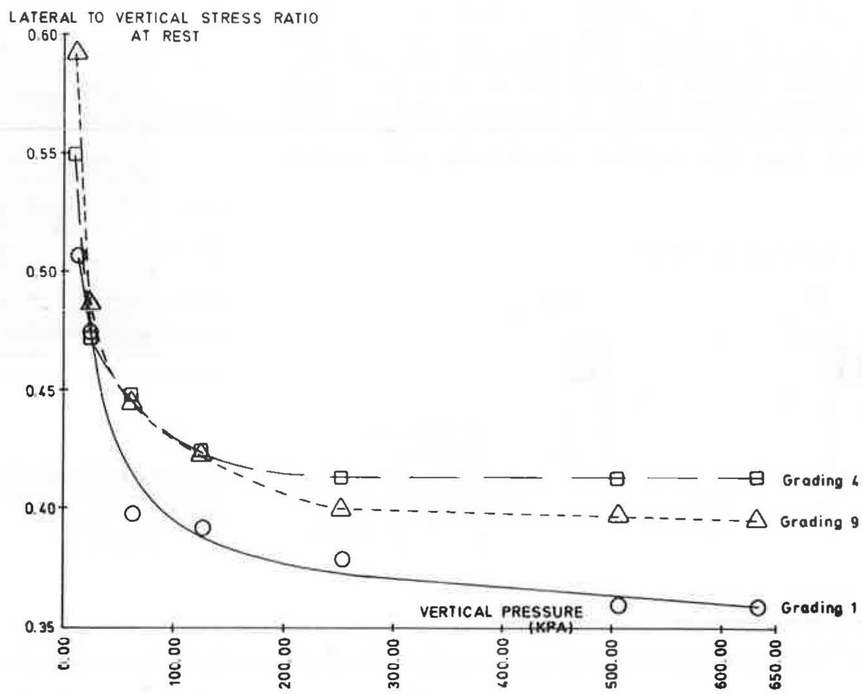
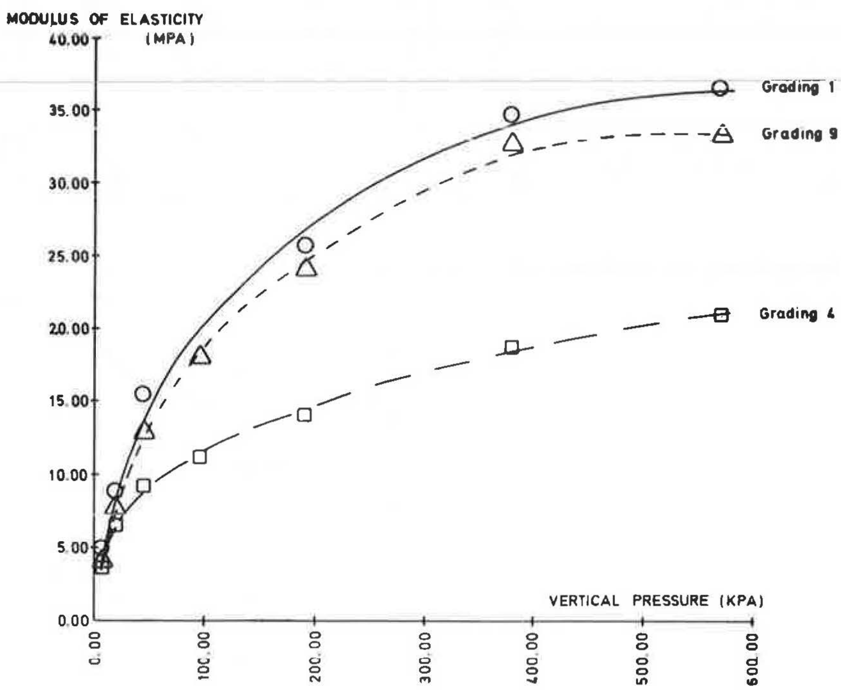


Figure 5. Modulus of elasticity-vertical pressure relations for tests on sand.



The elastic properties of the sand were determined by using the stress-strain characteristic of the sand, and the following relations. For isotropic elastic materials, the value of Poisson's ratio (μ) may be expressed as

$$\mu = K_o/(1 + K_o) \quad (1)$$

The constrained tangent modulus of deformation (D), which is the ratio of vertical stress difference ($d\sigma_v$) with respect to vertical strain difference ($d\epsilon_v$) under zero radial strain may be expressed by definition as

$$D = d\sigma_v/d\epsilon_v \quad (2)$$

The modulus of elasticity of an elastic material (E) may be expressed in terms of Poisson's ratio and the constrained modulus as

$$E = D(1 + \mu)(1 - 2\mu)/(1 - \mu) \quad (3)$$

or, alternatively, in terms of the constrained modulus and the lateral-to-vertical stress ratio at rest, as follows,

$$E = D(1 + 2K_o)(1 - K_o)/(1 + K_o) \quad (4)$$

Equations 3 and 4 do not apply for overconsolidated materials where K_o is equal to or greater than 1.

The relation between the modulus of elasticity (E) and the vertical pressure (σ_v) during compression tests of gradings 1, 4, and 9 are shown in Figure 5. E increases with an increase in the ver-

Figure 6. At test stress ratio-overconsolidation ratio relation for tests on sand.

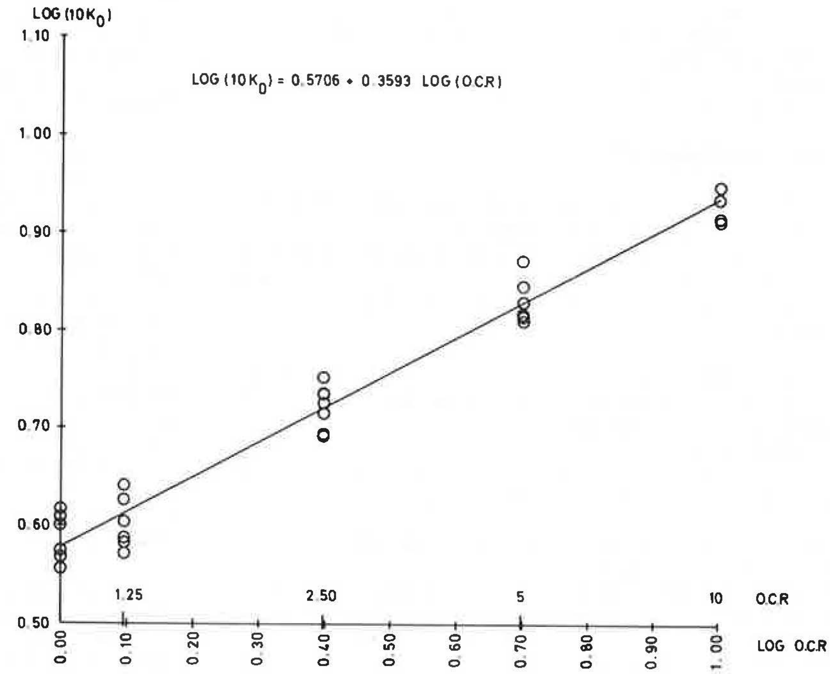
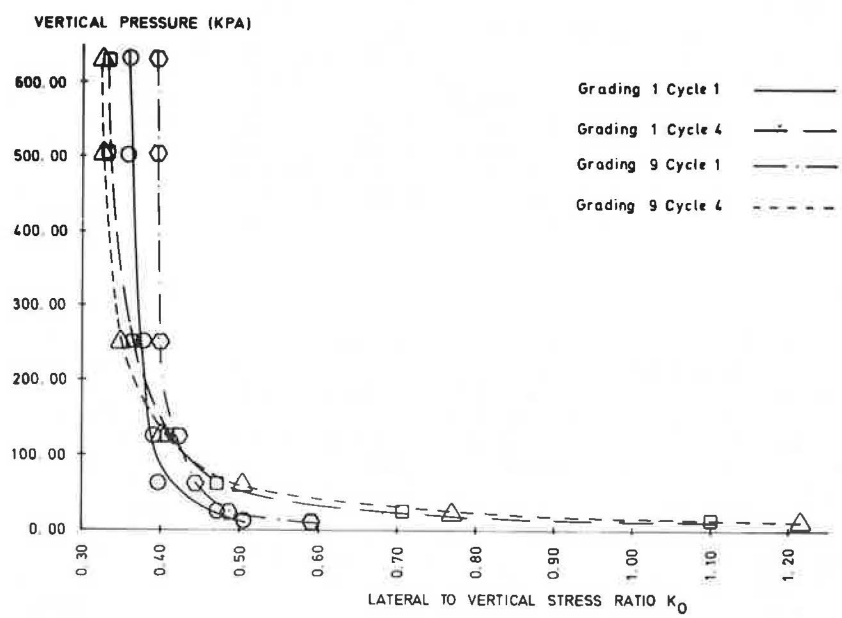


Figure 7. Vertical pressure-at rest stress ratio relations for repeated loading tests on sand.



tical pressure and the subsequent densification of the sand. However, note that the mass of coarse sand developed a much higher modulus of elasticity than the mass of fine sand, although the initial test conditions were similar. The characteristic of the modulus of elasticity of the poorly graded sand (grading 9) was similar to that of the uniform coarse sand, although only 12.5 percent of its mass was coarse sand (2.360 mm > particle size > 1.200 mm).

The correlation between the lateral-to-vertical stress ratio at rest and the overconsolidation ratio during the unloading stage of a compression test is shown in Figure 6. The correlation is linear if presented in double logarithmic scale. During the unloading stage the Honeydew sand may be characterized by the expression

$$\mathrm{Log}(10K_o) = 0.5786 + 0.3593 \log(\mathrm{OCR}) \tag{5}$$

where K_o is the lateral-to-vertical stress ratio at rest and OCR is the overconsolidation ratio.

Different linear double log correlations were obtained for other materials, and it may be expressed generally as

$$\mathrm{Log}(10K_o) = R_1 + R_2 \log(\mathrm{OCR}) \tag{6}$$

where R_1 and R_2 are the load release parameters that characterize the soil behavior.

The effect of repeated loading is shown in Figure 7. Only the first and fourth loading cycles are shown for gradings 1 and 9. Two distinct features are noticed:

1. The stress ratio decreases with an increase in the number of loading cycles and the subsequent compaction of the soil and
2. When the overconsolidation ratio is greater than five, the stress ratio decreases sharply with an increase in the vertical pressure.

These phenomena were common to the uniform and poorly graded sand specimens.

Some comparative triaxial K_o loading tests were carried out by using Bishop and Henkel's K_o belt (2) and the results were similar. However, this technique yielded somewhat higher lateral-to-vertical K_o values. Another series of comparative tests was carried out to evaluate the effect of radial strain on the stress ratio. We found that, for uniform sand, irrespective of grain size, the stress ratio decreased by about 5 percent when the radial strain was 0.000 06. In poorly graded sand radial strain of 0.000 06 resulted in an 11 percent decrease in the stress ratio, and a radial strain of 0.000 13 resulted in a 17 percent decrease in the stress ratio. This decrease is attributed to the dissipation of vertical stress in the sand specimen due to the development of shear stress within the soil mass.

Tests on Clay

Results of compression tests on clay samples are shown in Figures 8-11. During the compaction operation, which was simulated in the laboratory by rapid loading, pore fluid pressure developed during the compression process only dissipates partly. Since the LSP MkII is not equipped with facilities for measuring pore pressure, only an analysis in total stress terms could be carried out. However, total stress analysis represents the actual condition in a clay mass during, and shortly after, a compaction operation, because the dissipation of excess pore pressure is a time-consuming process.

A typical relation between the lateral-to-vertical pressure ratio and the vertical pressure during a loading and unloading cycle is shown in Figures 8 and 9. After the initial placement operation is completed and under a small vertical load, the clay specimen is in an overconsolidated condition and the stress ratio is high. The stress ratio decreases with an increase in the vertical pressure and the subsequent decrease in the overconsolidation ratio. A further increase in the vertical load results in an increase in the stress ratio. During the unloading operation the stress ratio increases with a decrease in the vertical load and the subsequent increase in the overconsolidation ratio. On reinstatement of the vertical load to its initial magnitude, the stress ratio is higher than the stress ratio recorded at the beginning of the loading cycle. However, under a small vertical load in a partly saturated condition, the lateral pressure developed in the clay specimen decreases with time. If evaporation takes place, the clay sample shrinks and full dissipation of the lateral stress may occur.

Results of statistical regression analysis of all the loading tests performed on clay are shown in Figure 10. Test results were analysed irrespective of the placement density, and it is apparent that the moisture content affects the stress path during the loading test. During placement of a dry clay specimen, a high vertical pressure is required to achieve the desired initial dry density. However, the initial vertical pressure required in order to achieve identical placement dry density decreases if the moisture content of the clay is increased in the dry of optimum range. The lateral-to-vertical pressure ratio at the beginning of a loading cycle is high for a dry clay specimen, and it decreases with an increase in the moisture content (Figure 9). An increase in the vertical pressure results in a decrease in the stress ratio, and this characteristic is similar to that found in sand.

Bear in mind that all the observations were made on clay samples that have an initial dry density lower than maximum standard American Association of State Highway Officials (AASHO) dry density and moisture contents in the range of 0.20-0.30. The samples are partly saturated and consequently the pore pressure response to loading is only partial. This condition is shown in Figure 10, which demonstrates the lateral pressure response to an increase in the vertical pressure. When a clay sample is fully saturated, an increase in the vertical pressure is reflected by an identical increase in the lateral pressure. This condition has never been encountered in this series of tests (the maximum degree of saturation achieved in any of the tests was 0.92).

The relation between the lateral-to-vertical pressure ratio and overconsolidation ratio when the clay specimens were unloaded is shown in Figure 11. The general relation suggested in Equation 6 is valid for the clay; e.g.,

$$\mathrm{Log}(10K) = R_1 + R_2 \log(\mathrm{OCR}) \tag{7}$$

However, note that in terms of total stress R_1 decreases and R_2 increases simultaneously with a decrease in the moisture content, and this relation is valid for the range of clay properties tested.

Two more important observations were made during this series of tests. First, repeated loading up to five cycles did not affect the lateral pressure response to loading, which was almost identical during any loading and unloading cycle. The other observation relates to the effect of time on the lateral pressure developed during compression. The time interval between the application of load increments was 1 min, which is insufficient for full dissipation of pore pressure in the clay tested. In another series of slow consolidation tests performed on the same clay, during loading the immediate lateral pressure developed in the clay specimens was high and it decreased if a period of 24 h was allowed between loading increments. This trend is reversed during the unloading process. When a load was removed, the lateral pressure decreased instantly and subsequently increased with time. If the clay sample is not kept under constant controlled moisture condition, evaporation takes place, which results in shrinkage and a subsequent decrease in the lateral pressure.

A comparison between the test results for sand and clay demonstrates the similarities and differences in the characteristics of the lateral pressure developed in the soil samples during compaction. The general stress-strain and vertical pressure-lateral pressure relations are similar in nature. However, in sand these relations are independent of time, whereas in clay time and moisture content affect the behavior of the tested specimen and its physical parameters. Quantitatively, the values of vertical deflection and lateral pressure response to

Figure 8. Typical total stress ratio-vertical pressure relation for test on clay specimen.

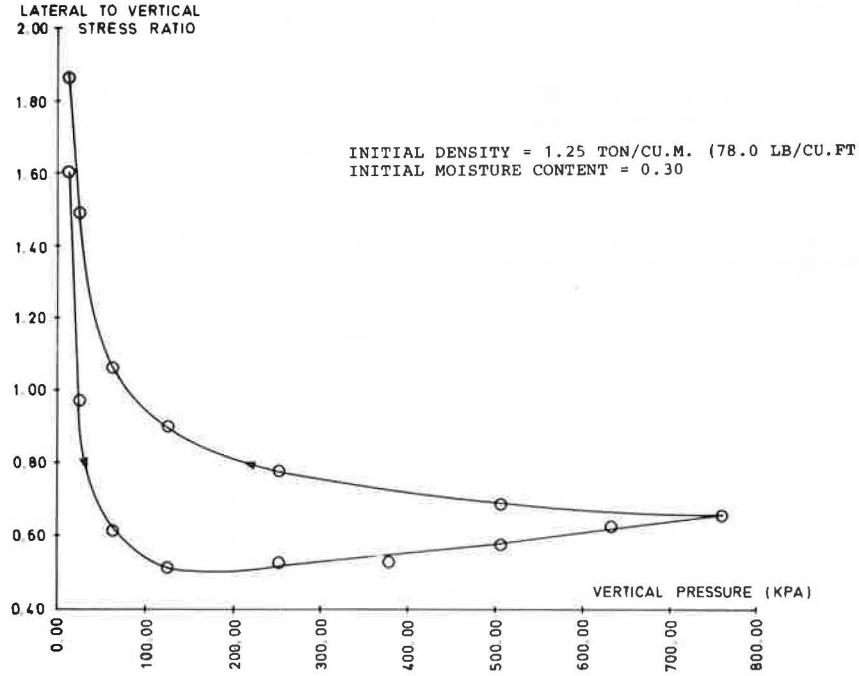
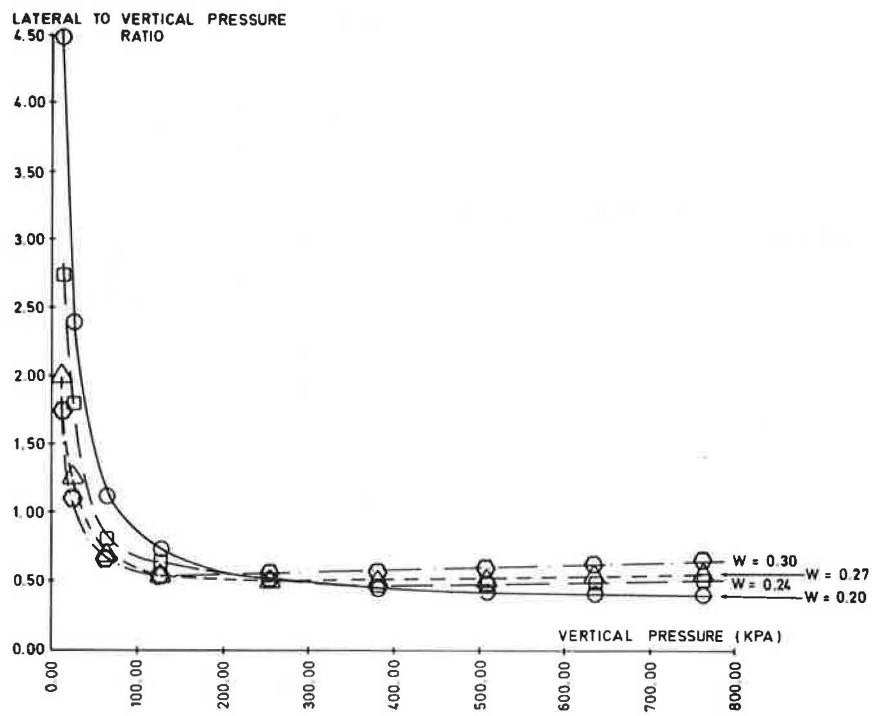


Figure 9. Effect of moisture content on the pressure ratio-vertical pressure relation for tests on clay samples.



axial loading increase with a decrease in the effective grain size. Therefore, we expected that the magnitude of the parameters determined from test results of clay specimens would follow this trend. Determination of the effect of pore pressure and swelling characteristics on the measured parameters was beyond the scope of this study.

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

A series of static compaction tests on sand and clay was performed by using LSP MkII. The apparatus is a modified oedometer ring that measures the lateral pressure developed in a soil specimen during compression in a null method (at rest condition), or alternatively, when some lateral strain is allowed to develop. The following characteristics were found.

1. After placement and under a low vertical pressure, the lateral-to-vertical pressure ratio (K_o) of the soil specimen is high, and it decreases with an increase in the vertical pressure. In sand the pressure ratio decreases to an ultimate value. In clay, the pressure ratio decreases to a minimum value and then it may increase again to a value less than unity for unsaturated clay.

2. When a sand or clay specimen is unloaded, the lateral-to-vertical pressure ratio increases with an increase in the overconsolidation ratio, and a general relation between the pressure ratio (K) and the overconsolidation ratio (OCR) is suggested

$$\mathrm{Log}_{10}(10K) = R_1 + R_2\log_{10}(\mathrm{OCR}) \quad (8)$$

This relation is insensitive to moisture, particle size, particle size distribution, moisture content, and time for the sand tested. However, it is sensitive to moisture content and time for clay. Hence, the stress ratio is a function of the stress history of the soil.

3. Small lateral strain results in a decrease in the lateral-to-axial pressure ratio in sand, but it does not affect the total stress ratio during quick

Figure 10. Pressure increment ratio-vertical pressure relations and loading of clay samples for initial dry unit weight = 13.7 kN/m^3.

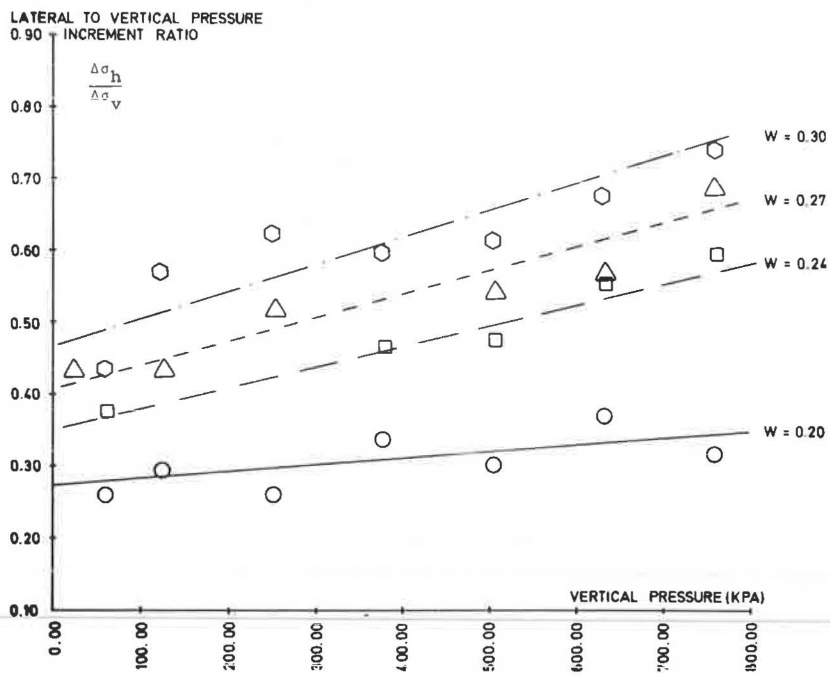
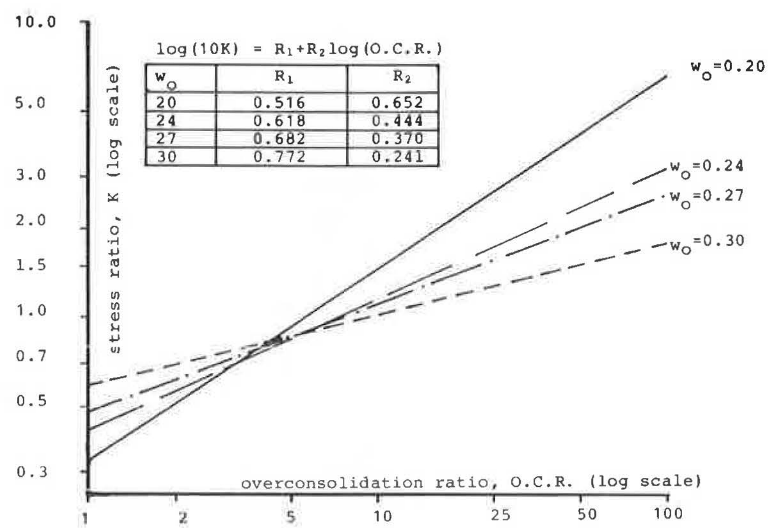


Figure 11. Stress ratio (K)-OCR relations for unloading of clay samples.



loading and unloading of clay samples.

4. Repeated loading results in a decrease of the pressure ratio in sand. The pressure ratio followed a similar path during repeated loading of the clay specimen.

5. Tests that use the LSP MkII provide reliable and consistent results. It was convenient and easy to use for compression tests of at rest condition. Bishop and Henkel's K_o belt (2) tended to give slightly higher K values in sand; however, its use for clay testing is inconvenient.

The results of this research should be developed further by means of incorporating pore pressure measuring facilities to the LSP MkII. The recording facilities of the LSP MkII should be improved for quicker and dynamic loading tests. Finally, a study should be conducted that will investigate the time and pore pressure parameters on several nonfree draining soils.

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