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Effect of Confining Pressure on Lateral Strain of Cohesionless Soil

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Abstract: Most of the available design methods for reinforced soils deals with the limit equilibrium of the system. They also involve certain assumptions regarding the properties of the materials used and the behaviour of the system at failure. In many of these the same assumptions are made as in classical earth pressure and slope stability analysis and adopted to develop simple analytical models. In addition, the internal stability of these soil inclusion systems is generally analysed by considering: (a) The rupture of reinforcing element, (b) Failure by lack of adherence between the soil and the reinforcement. However, for soil incorporating polymers, which are strain controlled systems, a limiting serviceability condition related to structures deformation must be considered and the operational properties of both the soil and the reinforcement established. Moreover, for such systems there must be strain compatibility between the soil and the reinforcement at all times. Hence the strenght parameters for reinforcement must be measured over the same range of lateral strain as can occur in the soil. This study presents the effect of confining pressure on the lateral strain of two different granular materials in a 100 mm x 100 mm diameter triaxial with lubricated ends and checks the validity of some of the assumptions made regarding these strains. The results show that all the lateral strains developed from the end of consolidation stage are tensile strains. The lateral tensile strain corresponding to the peak stress ratio is constant irrespective of the cell presures used. The lateral strains developed at constant volume are different from one soil to another.

Key words: Sand, strain, strength, soil reiforcement, soil properties

INTRODUCTION

A recent innovation for soil improvement utilises structural materials, such as steel strips or polymeric sheets in the soil. It involves strengthening soil by placing tension-resistant strips or sheets in the soil to modify its mechanical properties. The main advantage of the composite system are reduction in construction costs, ease of construction and its flexibity to withstand large deformations without significant damage. A large number of materials can be used for the reinforcing elements such as galvanised mild steel, stainless steel, aluminium, glass fiber and woven and non-woven geotextiles and geogrids. However, as the galvanised steel and stainless steel possess the disadvantage of being susceptible to corrosion when used in highly alkaline or acidic soils and their cost is high, it appears that there is an increasing interst in the use of polymeric elements as reinforcements in soil.

Most of available methods for the design of these reinforced soil structures cover the limit equilibrium of the soil-inclusion system. These methods involve certain assumptions regarding the properties of the materials used and the behaviour of the system at failure. In many of these procedures the same assumptions made in the classical earth pressure theories, are adopted to develop simple analytical models. The internal stability of reinforced mass is generally analysed by considering: (i) rupture of the reinforcing elements under induced tensile forces and (ii) failure by lack of adhrence between the reinforcement and the soil. However, unlike the metallic reinforcements, polymers are relatively extensible materials[1-3]. Therefore a limiting serviceability condition related to structures deformations must be introduced. This should relate both to constructional and post construction deformations[4]. To do this an appropriate means of predicting the behaviour of the structure must be identified and the operational properties of both the

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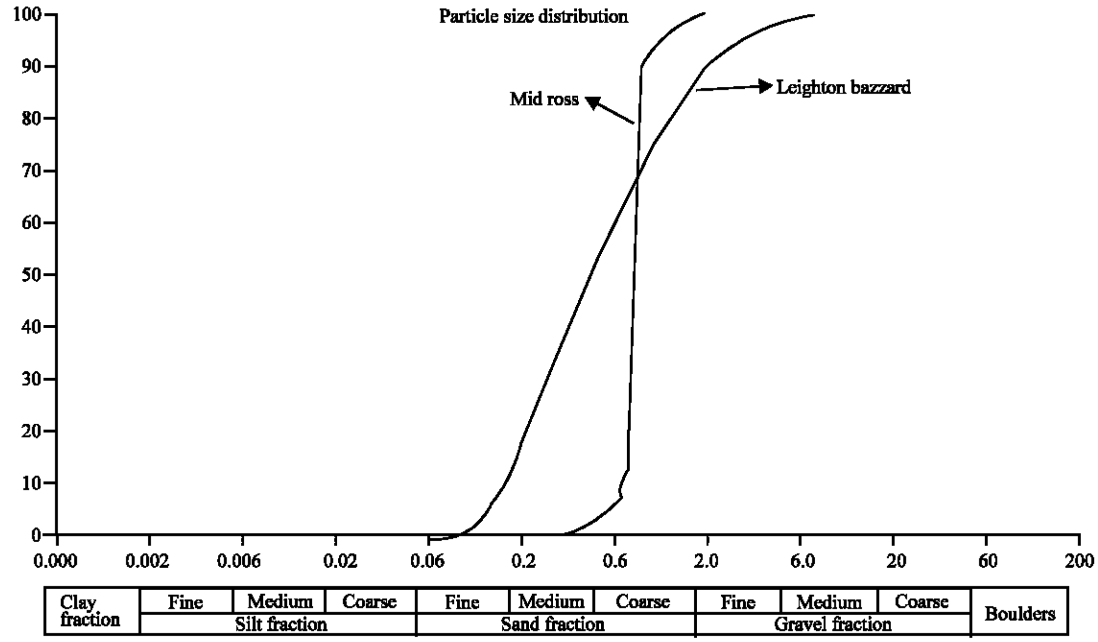

Fig. 1: Particule size distribution for both materials used

Table 1: Properties of materials used

Material used	Mineral composition	Particule size	C_u	G_s	Porosity loose	Porosity dense	Particule shape
Mid-Ross	Mixed composition with large fraction consisting of: highLand shists, Vien quartz and dolorite	0.06-6.0	5.76	2.69	40.0	30.0	Subangular
Leighton-Buzzard	Manly quartz	0.045-8.0	1.19	2.65	45.0	34.0	Subrounded

soil and the reinforcement established. Moreover, soil structure incorporating polymers are strain controlled systems[4,5,3]. Hence, there must be strain compatibility between the soil and the reinforcement at all times[3,5-7]. Thus the strength parameters for the reinforcement must be measured over the same range of lateral strain, as can occur in the soil. In classical soil mechanics, engineers measure shear or compressive strains using different types of apparatuses. Very few have studied the lateral strain in the soil. This is maily due to the difficulties of taking direct mesurements. However, it is sometimes possible to calculate the average tensile strains from the interchangeable relationship $\varepsilon_v=\varepsilon_1+2\varepsilon_3$ in a triaxial test apparatus.

With the above in mind, it was decided that in this work a study must be made of the tensile strain behaviour of two different granular materials in order to check the validity of some of the assumptions made in earth-pressure theories and the analytical techniques employed for reinforced and unreinforced structures.

Test materials: The physical properties of Mid-Ross and Leighton-Buzzard sands is best given in terms of: (a) Particule mineralogy: Chemical analysis of the two particulate materials showed that Mid-Ross sand has a mixed composition including layer fraction consisting of highland shists, Vien quartz and dolomite, while Leighton-Buzzard sand is maily composed of quartz. (b) Particule size and distibution: Sieve analyses were performed on the sands using BS1377(1975, test B). (c)Specific gravity: the specific gravity for each sand was obtained using The BS1377(1975, test 6A). The avrage of three tests on each soil was within 0.03, which was taken as representative of the sample. (c) Limiting porosities: Maximum porosities for the sands used were determined using the method suggested in[8]. Minimum porosities were obtained following the suggestion of[9,10] 100 mm high x 100 mm diameter saturated triaxial specimen were normally vibrated in several layers. The minimum porosity obtained for each sand was taken as the minimum limiting porosity. (d) Particule shape: Photographs illustrating grain shapes of Mid-Ross and Leighton-Buzzard were taken and the classification was made according to[11]. Table 1 summerize the properties of materials used and Fig. 1 shows the grading curves for boths soils.

Mid ross sand		100 x 100 d, am Triaxial			
Cell pressure (kN0Sq.m)		50	100	150	200
Sheering	Relative den (%)	90▲	92□	87○	91▼

Leighton bzn sand		100 x 100 d, am Triaxial			
Cell pressure (kN0Sq.m)		50	100	150	200
Sheering	Relative den (%)	95.5▲	95.5□	92.0○	92.0▼

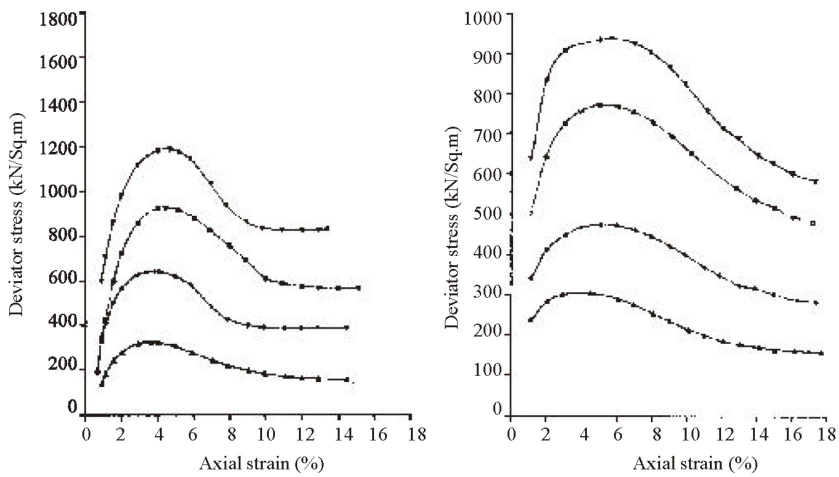

Fig. 2: Deviator stress versus axial strain

Mid ross sand		100 x 100 d, am Triaxial			
Cell pressure (kN0Sq.m)		50	100	150	200
Sheering	Relative den (%)	90▲	92□	87○	91▼

Leighton bzn sand		100 x 100 d, am Triaxial			
Cell pressure (kN0Sq.m)		50	100	150	200
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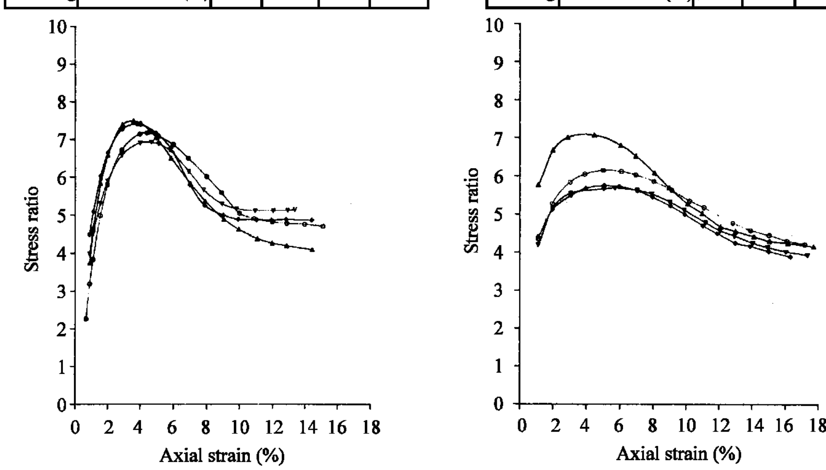


Fig. 3: Stress ration versus axial strain

Equipment and testing techniques: In this investigation and following the suggestion of[12], all triaxial test spcimens were prepared with a height/diameter ratio of approximately 1.0. The actual size of the samples was 100 mm x 102 mm. The method used to prepare the specimen was similar to that proposed by[10]. The usual corrections were applied to the results as suggested by[13]. The triaxial apparatus used to carry out the tests programme was composed of: (a) former, made out of alluminium, enable the application of a small vaccum to exend the 100mm diameter latex menbrane. It was manufactured slightly bigger than the top platen to avoid any formation of a neck. The disturbance of the specimen when removing the former was eliminated by adopting a split form. (b) a 100mm cell consisting of three principal components, which incorporates the various inlet and outlet connections. (c) Platens, These were cut from a solid piece of aluminium and the bottom one was permanently sealed to the base. Each platen was fitted with a 19mm porous plastic drainage plate. (d) Loading system, the controlled rate of strain method of applying the axial load to the sample was used. (e) pressurising ,

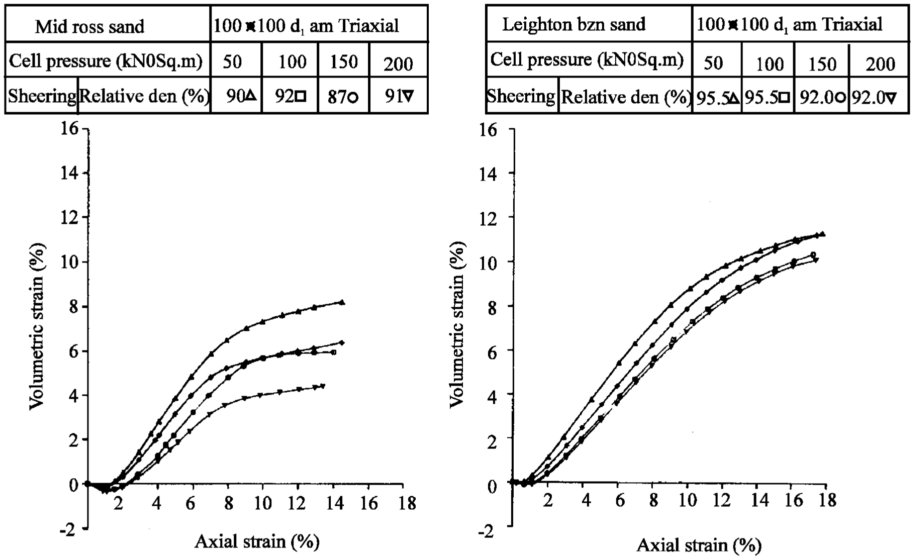


Fig. 4: Volumetric strain versus axial strain

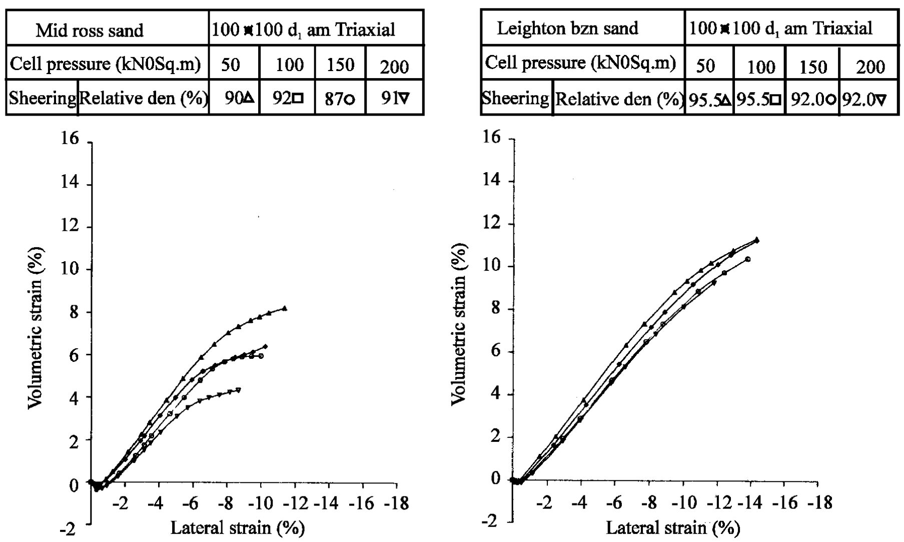


Fig. 5: Volumetric strain versus lateral strain

system, the principle of self composating air-water system was used to provide cell pressure. (f) Mesurising devices the instruments used to mesure the axial load were 500, 1000 and 2500 kg capacity prouving rings. The axial deformation and cell pressure were respectively measured using a 0.01 mm Mercer dial gauge and 250 mm diameter standard Budenberg gauge. The reading of volume change was given by a 100 mL burette. All these measuring devices were calibrated before runing the tests.

Laboratory results: To establish the influence of confining pressure on the stress-strain relationship of both Mid-Ross and Leighton-Buzzard sands, the specimens were tested under four different confining pressures. These were 50, 100, 150 and 200 kN/m^2, respectively. In order to compare the results, the state of compaction was expressed in terms of similar relative densities. This required that a large number of tests be carried out in order to obtain data from tests having practically the same relative densities.

Deviator stress and axial (compression) strain: Deviaiors are plotted in Fig. 2 as a function of axial strains for tests on Mid-Ross and Leighton-Buzzard sands. These plots illustrate several important effects of confining pressure on the stress-deformation of the two soils. The axial strain corresponding to the maximum deviator (or stress ratio) increases slightly with the increase of the confining

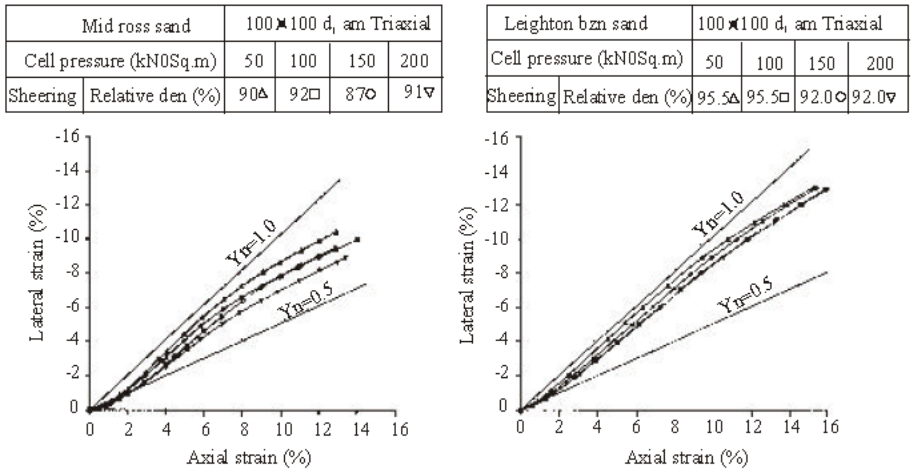


Fig. 6: Lateral strain versus axial strain

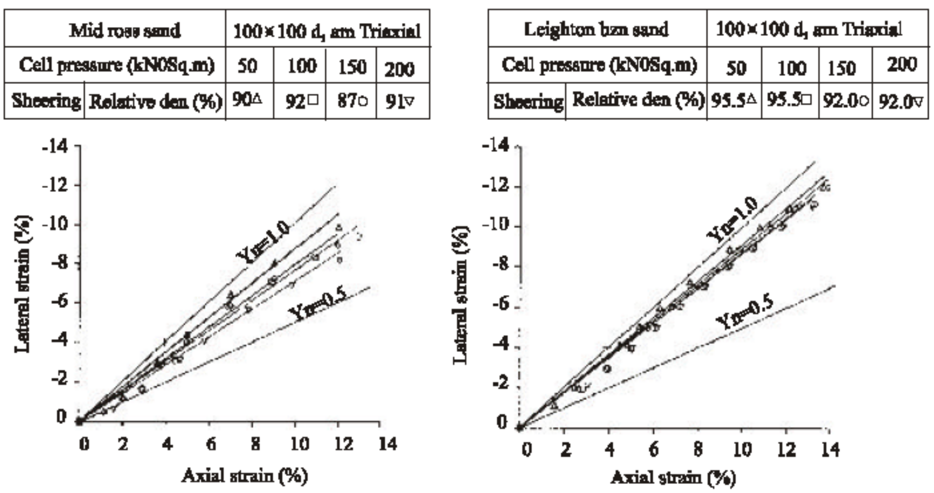


Fig. 7: Approximation of lateral strain versus axial strain

pressure. At strains larger than the axial strain to the peak, the deviator stress (or stress ratio) decreases to attain a constant ultimate strength. Unlike the relative density the confining pressure has a large influence on both the peak and ultimate strength when represented by the deviator stress. However, for the stress ratio-axial strain relationship given in Fig. 3, the ultimate strength seems to be constant irrespective of the cell pressure. This observation has also been made by[14].

Volumetric strain: Figure 4 illustrate the effects of increasing confining pressure on the volume change characteristics of samples of the same sand prepared at the same, or very similar, relative densities. The data show that the shapes of the curves are the same for specimens with relatively l-ow and high density, except that at low pressures the tendency for dilatation is not so strong, while at high pressures the tendency for contraction is greater. The same finding have been obtained by[14].

Lateral strain: The effect of cell pressure on the development of lateral tensile strain is studied through the volumetric strain, the axial strain, mobilized angle, stress ratio and strain ratio $\varepsilon_3/\varepsilon_1$. The relationship between the volumetric strain and the lateral strain is presented in Fig. 5. these curves show that increasing the confining pressure will lead to an increase in the initial contraction of the specimen and a decrease in its tendency to dilate. The shapes of the curves are similar to those of (ε_v versus ε_1) relationship for both materials. At large strains, the rate of dilatation reduces, leading to a constant volume. This stage was stopped at 14% lateral strain.

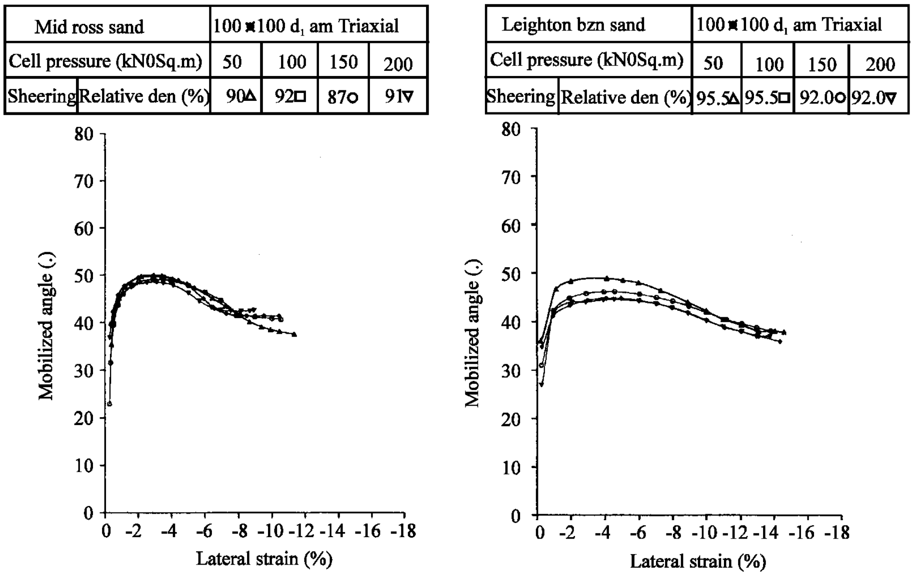


Fig. 8: Moilized angle of friction versus lateral strain

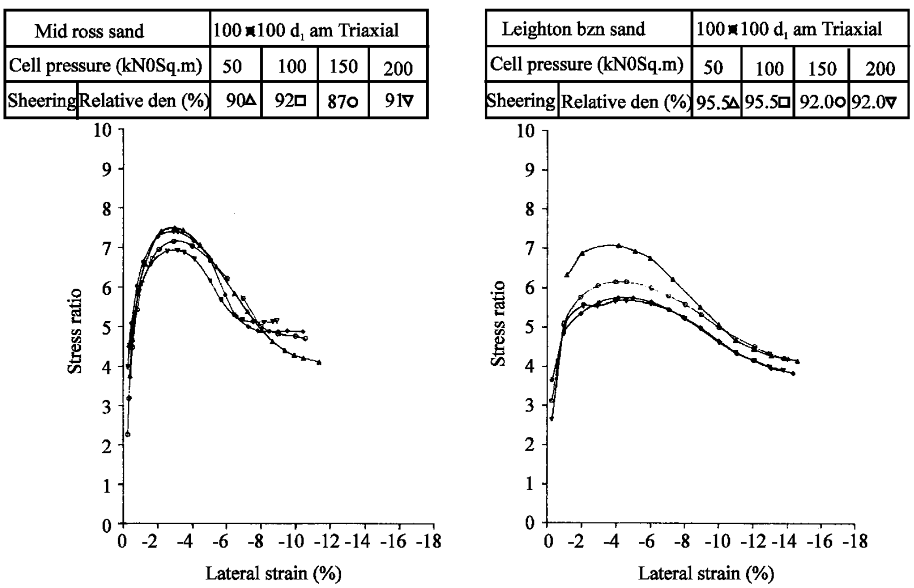


Fig. 9: Stress ratio versus lateral strain

The realtionship between the lateral tensile strain and the axial compressive strain given in Fig. 6 show that both sands undergo a small contraction before they strated to dilate and reach a constant value as in the (ε_v versus ε_l) relationships. Unlike the Leighton-Buzzard sand where the effect of cell pressure is insignificant, the Mid-Ross sand seems to be slightly affected by the change of confining pressure. Indeed the increase of cell pressure gave an increase of lateral strain at the same axial strain.

An approximation of these curves to straigt lines, is made in Fig. 7. These show the slopes of the lines corresponding to the confining pressures used in the tests on Leighton-Buzzard, are unaffected by changes of the cell pressure. Also that for Mid-Ross sand these slopes decrease with increasing confining pressure. The changes in slopes are not very significant and an average value of 0.6 for Mid-Ross and Lieghton-Buzzard may be considered as being reasonable.

Following the suggestion of[14] the mobilized angle of friction (ϕ_m) has been plotted versus the lateral strain. This variation of mobilized angle with the development of lateral tensile strain in soil is shown in Fig. 8. Theses plots show that both Mid-Ross and Leighton-Buzzard increase in strength to reach a peak value of ϕ_{max} of 49.8° and 45°,

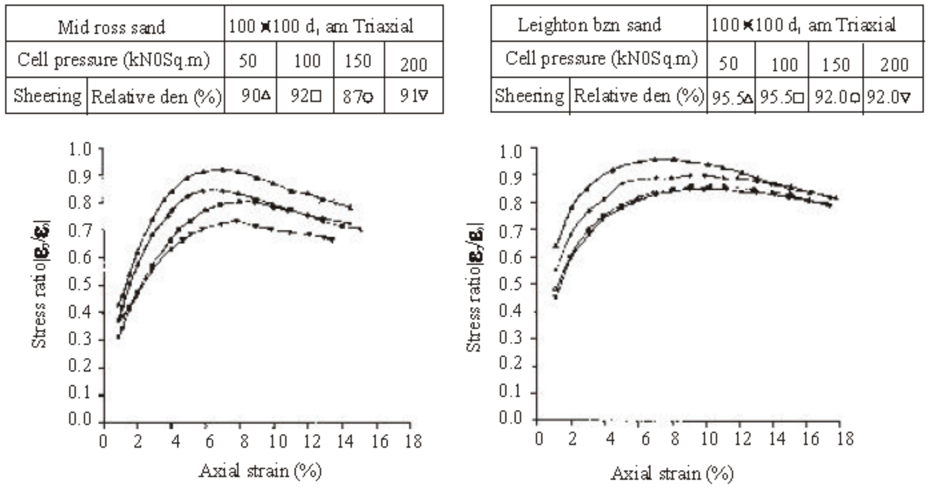


Fig. 10: Strain ratio versus axial strain

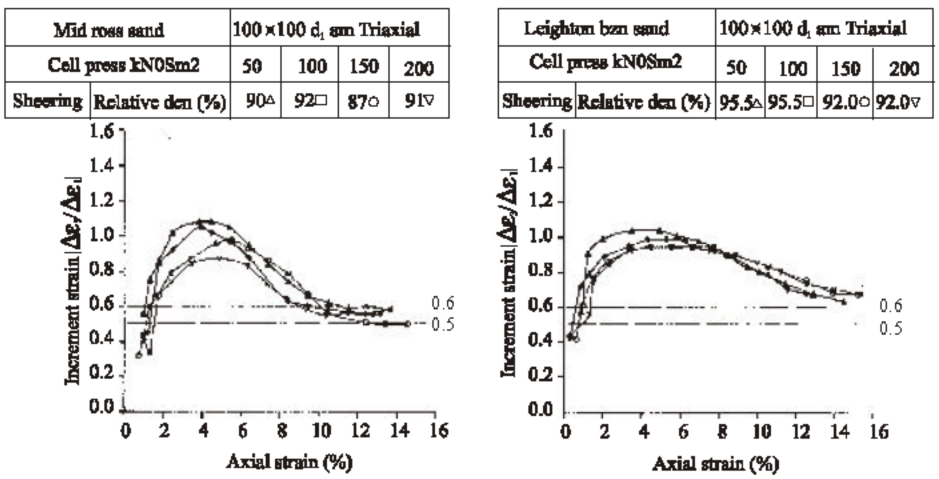


Fig. 11: Increment strain ratio versus axial strain

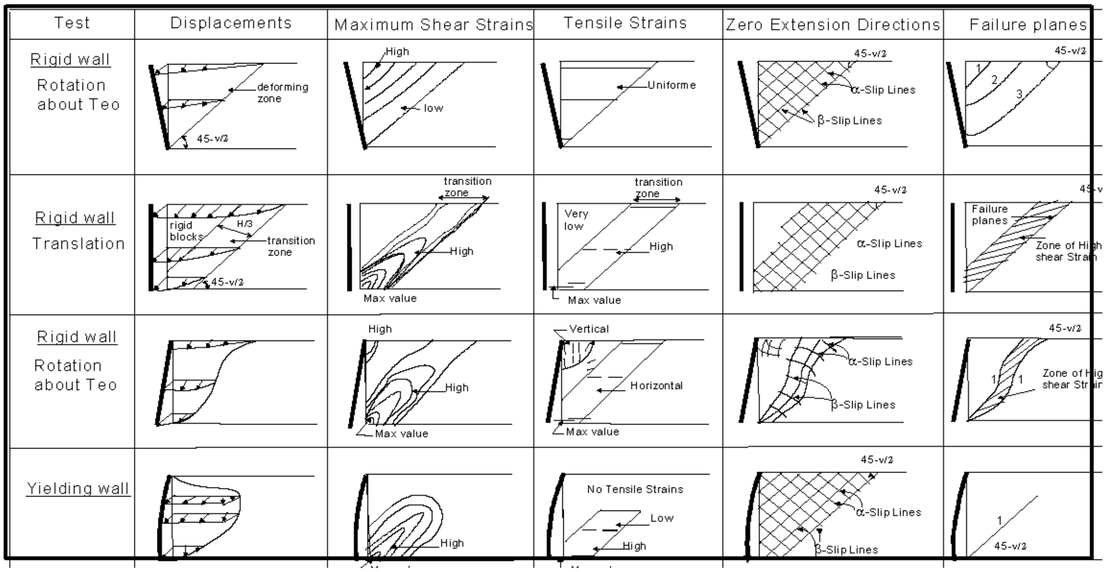
respectively. With continuing shear their strength decreases slowly to attain an ultimate constant mobilized angle of 35° and 36° at about 10 and 13% lateral strains, respectively. Moreover, these relationships show that the effect of cell pressure on mobilized angle-lateral strain relationship is not significante. This is probably the case since the range of lateral stresses used is low and no crushing of soil grains occurred. A set of triaxial published data from[14,16] were replotted to confirm the trend of the (ϕ versus ε_3) relationship curves. [16]Obtained similar results concerning the effect of lateral stress since his sand was the same material used in the present work and the pressure used was in the range 69-276 kN/m^2 and 103-552 kN/m^2.

The above observations have also been made when studying the relationship between the stress ratio and the lateral strain in Fig. 9. The peak and ultimate stress ratios were about 7 and 5 for Mid-Ross and 5.8 and 4.3 for Leighton-Buzzard sand.

The illustration. of the strain ratio $\varepsilon_3/\varepsilon_1$ versus the axial strain is given in Fig. 10. As can be seen, with the progress of shearing, the strain ratio $\varepsilon_3/\varepsilon_1$ increases to a peak value before decreasing at large axial strain. Moreover, the influence of cell pressure is significant and an increasing of confining pressure has two effects on the strain ratio-axial strain relationship; it reduces the brittleness characteristics of the curve and increases the axial strain at peak strain ratios $(\varepsilon_3/\varepsilon_1)_{max}$.

The effect of the confining pressure σ_3 on the incremental strain ratio $(\Delta\varepsilon_3/\Delta\varepsilon_1)$ is examined by plotting $(\Delta\varepsilon_3/\Delta\varepsilon_1)$ versus axial strain as shown in Fig. 11. The first figure shows that for Mid-Ross sand the incremental strain ratio-axial strain relationship is affected by the change of cell pressure. As can be seen, an increase in the cell pressure results in a decrease in the incremental strain ratio at peak from 1.05 to about 0.83. moreover, this increase of confining pressure leads to a decrease in the brittle characteristics of the curve. However, for Lieghton-

Table 2: Displacements and strains behind a wall[17]



Buzzard sand the effect of cell pressure seems to be less significant. This difference in behaviour between the two sands is thought to be due to their composition. At early stage of the test the incremental strain ratio ($\Delta\varepsilon_3/\Delta\varepsilon_1$) is less than 0.5 resulting in contraction of the specimens. With increasing in the axial strain this ratio increases reaching a peak value and then decreases to a constant value of approximately 0.6 at large axial strains.

In practice these results seem to be important in design. Knowing the relationships from the triaxial apparatus in conjunction with the pattern of lateral tensile strains behind a retaining wall, it will be possible to estimate the mobilized shear strength throughout the deforming zone at any stage of the deformation. The test results of[17] presented in Table 2 showing the tensile strain patterns behind retaining walls, together with the (ϕ_m-ε_3) relationship obtained in this investigation, show that the mobilized angle is constant in the deforming zone and along the repture line in the case of wall rotating about the toe. However, this is not the cas for other modes of wall deformation. Hence, The Rankine and Coulomb assumption of constant ϕ_m seems to be valid only when the wall rotates about the toe.

For soil reinforced with extensible material, the constant ϕ_m implies a constant ε_3 at failure Therefore, when considering the distribution of tensile forces in the reinforcement, the forces at each layer should be equal since the system is strain controled. This is only true for the cas of rotation about the toe and the optimum spacing of reiforcement (i.e non uniform spacing).

Furthermore, since reinforced soil structures are generally constructed with compacted granular materials and lateral tensile strain to achieve Rankine state are 2-3% then metallic or alluminium reinforcement must allow a larger amount of slippage if ϕ_m is to reach ϕ_{peak}. For reinforced earth systems using extensible reinforcement and granular fill, the mobilized angle at constant volme ϕ_{cv} and the corresponding lateral strain may be suggested for limiting equilibrum analysis, ϕ_{cv} =35° at ε_3=10% (ϕ_{cv} =36° at ε_3=13%) would appear to be more appropriate for design.

CONCLUSION

For both Mid-Ross and Lieghton-Buzzard sands the following may be concluded:

- The (ε_v versus ε_1) and (ε_v versus ε_3) relationships were very similar. The curves had an S shape form showing contraction of the samples followed by dilatation.
- The lateral strains (ε_3) developed from the end of the consolidation stage were all tensiles strains.
- The (ε_3 versus ε_1) relationships showed that an increment of axial strain does not necessarily cause an equal increment of lateral strain. On applying the deviator stress the tensile strain ε_3 increased at lower rate than ε_1., then increased and assumed a greater rate thanε_1.
- The confining pressure was found to affect the (ε_3 versus ε_1) relationships. At fixed axial strain the value of incremental strain ratio $\Delta\varepsilon_3/\Delta\varepsilon_1$ increased with decreasing confining pressure. However, for Lieghton-Buzzard this relationship was fairly insensitive to the change of cell pressures used in testing programme.

- The approximation of (ε_3 versus ε_1) relationships to straight lines showed that the slope of the lines lies within the range 0.5 to 1.0. With the variation in confining pressure Leighton-Bzzard exibited little variation.
- The shape of mobilized angle of friction (ϕ versus ε_3) or (σ_1/σ_3 versus ε_3) relationships were found to develop a peak value before decreasing to a constant value of angle of friction (or stress ratio) at large lateral tensile strains.
- The confining pressure appeared to have only a slight effect on both (ϕ versus ε_3) and (σ_1/σ_3 versus ε_3) relationships for the range of cell pressures used.
- Mid-Ross and Lieghton-Buzzard achieved their peak mobilized angle of friction at about ε_3=2.1% and ε_3=2.7%, respectively. (i) The ($|\varepsilon 3/\varepsilon 1|$ versus ε_1) relationships for variation in confining pressures indicated that increasing the axial strain, the strain ratio increased to attain a peak value and with the progress of shear it decreases. An increase in the confining pressure had two effects on the ($|\varepsilon 3/\varepsilon 1|$ versus ε_1) relationships. They reduced the strain ratio at peak and increased its corresponding axial strain.
- The relationships between the incremental strain ratio $|\Delta\varepsilon_3/\Delta\varepsilon_1|$ and the axial strain ε_1 varied confining pressure of soil. Specimens with low confining pressure had higher peak values at lower axial strains than specimens with high confining pressures.
- At large axial compressive strains (ε_1) the incremental strain ratio $|\Delta\varepsilon_3/\Delta\varepsilon_1|$ tends to becom constant irrespective of confining pressure.

The comparision of the results of Mid-Ross and Lieghton-Buzzard sands illustrated the following points:

- The strenght of Mid-Ross sand was always greater than that of Lieghton-Buzzard sand. This varies from 1° at low high cell pressure to about 3° at high cell pressure, the difference is thought to be due to the difference of gradation, grain shape an angularity and also their surface roughness.
- Mid-Ross sand exhibited more initial contraction and less dilatation than Lieghton-Buzzard sand. This behaviour was attributed to the necessity for the uniforme sized particules of Lieghton-Buzzard sand to slip up over each other more than the graded Mid-Ross particules.
- The lateral tensile strain ε_3 and the strain ratio ($\varepsilon_3/\varepsilon_1$) at the peak stress ratio of Mid-Ross sand were less than those of Lieghton-Buzzard sand for a given cell pressure.
- Mid-Ross sand achieved a mobilized angle at constant volume of 35° at ε_3=10%, a similar value 36° was reached by Lieghton-Buzzard sand at approximatively ε_3=13%. The differnce in strain is believed to be due to the composition of the two soils as discussed previously.
- Mid-Ross sand attained a value of incremental strain ratio $\Delta\varepsilon_3/\Delta\varepsilon_1$ =0.6 at approximatively 11% axial strain and the same value was reached at 16% axial strain for Lieghton-Buzzard sand. This implies that different sands do not necessarely achieve the same value of incremental strain $\Delta\varepsilon_3/\Delta\varepsilon_1$ at the same axial strain.

In practice, for unreiforced and reinforced soils the Rankine and Coulomb assumption of constant ϕ_m made in the design of retaining walls seems to be valid only when the wall rotates about the toe. this is not the cas for other modes of wall deformation.

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