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Equal Strain Consolidation of Clays under Radial Drainage

Robinson, R. G.* and Shilpa, D**

Introduction

Large areas of Earth's surface are covered with soft clay deposits with very high water content. Typically India has a long coastline of over 6500 km covered with soft marine clay deposits. These soil deposits exhibit low bearing capacity and high compressibility characteristics. Due to rapid increase in population and urbanisation, very often suitable soil conditions are not available and the structure has to be founded on such soft soils. Therefore, appropriate ground improvement technique is to be adopted for using these grounds for foundation, as the traditional pile foundations are expensive.

Preloading is one of the successful ground improvement techniques, in which the soft soil strata is forced to consolidate under a surcharge pressure equal to or more than the anticipated foundation pressure. The consolidation is usually accelerated by providing vertical drains, which are in the form of sand drains, sand wicks, band drains or pre-fabricated vertical drains (Mitchell and Jardine, 2002). As the permeability in the horizontal direction is higher than vertical permeability, the horizontal drainage usually predominate the consolidation.

Figure 1(a) shows a typical unit cell within a network of vertical sand drains of triangular pattern, in a clay stratum of thickness h, and Figure1(b) shows the cross section of the unit cell of diameter d_e. At the centre of the unit cell is a drainage well (sand drain) of diameter d_w. For conditions of radial drainage, the top, bottom and outer boundary of the clay in the unit cell are impermeable where as the sand drain is freely draining. Drainage of water within a cylinder of clay takes place in the radial direction towards the sand drain during the consolidation process.

Rendulic (1935) was one of the first to give an approximate solution based on Terzaghi's one-dimensional consolidation solution for this problem and is commonly known as Terzaghi-Rendulic solution. However, Barron's theory (Barron, 1948) is more popular and is very widely used to arrive at the design parameters such as drain distance and consolidation period for the design of vertical drains because of its simplicity and ease of use (Leo, 2004).

* Assistant Professor, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India E-mail: robinson@iitm.ac.in

** Formerly Postgraduate student, Department of Civil Engineering, Indian Institute of Technology Madras, Chennai-600 036, India

Barron (1948) developed solutions for two types of boundary conditions such as:

"free strain" resulting from a uniform distribution of surface load and

"equal strain" which results from imposing same settlement at all points on the surface.

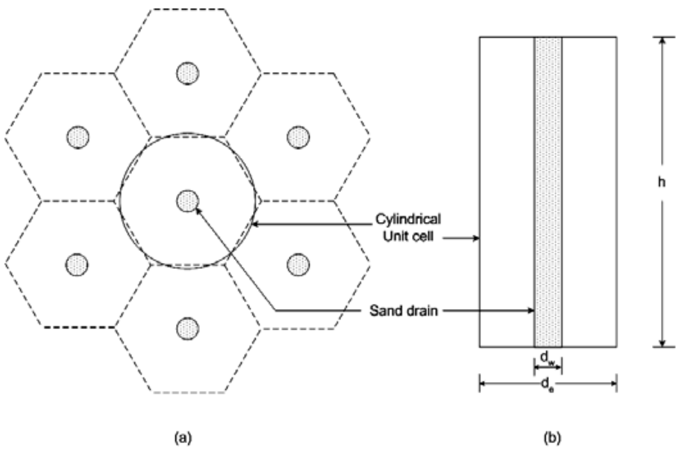

Fig. 1 (a) Typical Unit Cell within a Network of Vertical Sand Drains and (b) Typical Unit Cell

However, it was realized that the difference of average settlement between the free and equal strain solutions is not of engineering significance, and therefore, due to simplicity equal strain case is widely adopted for the design. Later, many researchers studied the problem by including drain resistance, smear zones, varying loading conditions, varying void ratio and permeability, *etc.* (Hansbo *et al*. 1981, Indraratna and Redana, 2000; Nash and Ryde, 2001; Indraratna *et al*. 2005, to name a few). Biot (1941) provided a general solution for the three-dimensional problem. Cryer (1963) observed a distinct difference between the Biot solution and the Terzaghi-Rendulic solution that for special boundary conditions an initial increase in pore pressure more than the applied pressure increment occurs during the consolidation process. This was also reported earlier by Mandel (1953) for plane strain conditions. This phenomenon is commonly referred to as Mandel-Cryer effect.

Several laboratory studies were also reported in the literature based on reduced scale unit cell experiments under conditions of radial drainage. In many of the experiments, only the surface settlement was monitored. Suits *et al*. (1986) carried out a testing program to determine the effectiveness of several types of prefabricated vertical drains, using a 254 mm diameter consolidation apparatus specifically designed for this purpose. Sridharan *et al*. (1996) carried out laboratory consolidation tests under radial consolidation in a modified oedometer to determine the radial coefficient of consolidation from the time-settlement data. Similar experimental technique was adopted by Robinson (1997) and Robinson and Allam (1998). Seah *et al*. (2004) developed procedure for constant rate of strain consolidation test to determine the consolidation characteristics under radial drainage. The above studies focused on the surface settlements only.

Experimental investigations on the pore pressure response of clays with vertical sand drains under radial drainage are limited in the literature. Indraratna and Redana (1998) and Sharma and Xiao (2000) used large-scale model tests in the laboratory and pore pressure measurements were made to study the smear zone developed due to the installation of vertical drain. Fang and Yin (2006) investigated the pore pressure response under sustained loading on Hong Kong marine clay with prefabricated vertical drains. Drainage was allowed to take place both in the radial and vertical directions.

One of the very important parameters required in the design of vertical drains is the permeability in the horizontal direction. While measurements of permeability in the vertical direction is relatively simple to perform, permeability measurements in the horizontal direction requires special equipment or special techniques. Al-Tabba and Wood (1987) devised an apparatus to measure the horizontal permeability. Rowe cell (Rowe and Barden, 1966) is widely used to determine the horizontal permeability (Head, 1986). Indraratna and Redana (1998) reported the values of horizontal permeability from samples recovered in the horizontal direction of a vertically consolidated sample. The procedure adopted in the above studies are either complicated or require expensive apparatus. Therefore, simplified methods are needed to determine the permeability in the horizontal direction.

In this study, a reduced scale model test was conducted using a 100 mm diameter consolidometer that was specially fabricated. In the case of relatively long vertical drains, the radial flow predominates and vertical flow can be ignored (Indraratna *et al*. 2005). Therefore in the present test, vertical drainage was not permitted. In addition to the surface settlement, the pore pressure developed at the boundary of the unit cell during consolidation was measured, using a miniature pore pressure transducer. The end condition of the sample after the test was also carefully studied. The anisotropy in permeability was determined by analysing the time-settlement data obtained from radial and vertical consolidation tests through a matching method. The problem was analysed using finite element analysis and the insights gained from the analysis in terms of stress transfer and soil movement during the consolidation process and the stress state at the end of consolidation were studied.

Test apparatus and procedure

Consolidation test with radial drainage was conducted on a reconstituted soil sample of IIT clay taken from the lake of Indian Institute of Technology Madras, Chennai, India. The soil was processed and the fraction passing through 1 mm sieve was used for carrying out the consolidation tests. The physical properties of the soil are given in Table 1.

A consolidation cell of 100 mm diameter and 40 mm thick was devised in the present study. The cell is made up of stainless steel and the inner surface of the ring is highly polished. The cell was suitably designed to simulate the boundary conditions of the unit cell that was shown in Figure 1(b). A schematic of the apparatus is shown in Figure 2. The soil was mixed thoroughly into slurry to water content of 1.5 times the liquid limit and subjected to a negative pressure of 95 kPa for 8 hours to remove the entrapped air. A collar of 15 mm thickness was positioned over the consolidation ring. The collar and the ring were lubricated with Silicone grease to reduce the side friction during consolidation. The clay slurry was then carefully poured in to the set-up to a thickness of 50 mm. Care was taken to avoid entrapment of air bubbles. While placing the clay slurry, a miniature pore pressure transducer (DRUCK Model PDCR81, Druck Limited 2007) of 6.4 mm diameter and 11.4 mm long was introduced at the boundary as shown in the figure. The miniature pore pressure transducer has a fast response time (Mair, 1979; Lee 1990) and capable of measuring pore pressures with an accuracy of 0.1 kPa. The slurry was then gradually loaded and subjected to one dimensional consolidation under double drainage up to a pressure of 65 kPa. Once the consolidation was over, the set-up was dismantled and the soil was trimmed to the size of the consolidation ring, of 100 mm diameter and 40 mm thick. The water content of the soil under a consolidation pressure of 65 kPa was 37%. A rubber sheet of 0.5 mm thickness with a central hole of 20 mm was placed above the bottom porous stone, on which the consolidation ring with the trimmed soil was placed.

Table 1 Physical Properties of the Soils Used

Property	***Value***
Liquid limit (%)	50
Plastic limit (%)	18
Plasticity index (%)	32
% Sand	40
% Silt	23
% clay	37

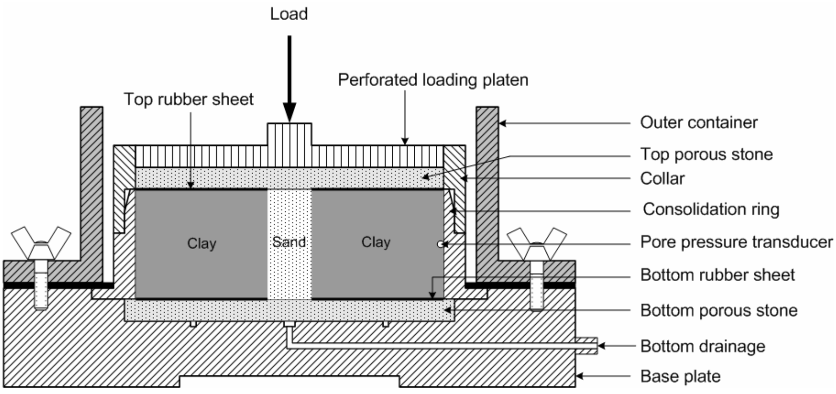

Fig. 2 Schematic of the Apparatus Used

The set-up was assembled again and a Perspex guide of 99.5 mm diameter and 10 mm thickness was then placed above the soil sample. The Perspex guide has a central hole of 20 mm through which a sampling tube of outer diameter of 20 mm and inner diameter of 19 mm was pushed through the soil. The outer surface of the sampling tube was highly polished so that the disturbance to the soil is minimized. The Perspex guide was used to ensure proper positioning and alignment of the sampling tube to make the sand drain. The tube filled with the soil was then carefully taken out leaving a central hole in the soil sample. The hole was then filled with clean sand in loose condition as shown in Figure 3. Since the soil in the central portion was removed using a thin wall sampling tube and sand was placed in a loose state the smearing effect is reduced and Barron's theory for the case of radial consolidation without smear can be applied (Berry and Wilkinson, 1969; Sridharan *et al.*, 1996). As the diameter of the sand drain, d_w, is 20 mm and the diameter of the consolidation cell, d_e, is 100 mm the drain spacing ratio n (= d_e/d_w) is equal to 5.

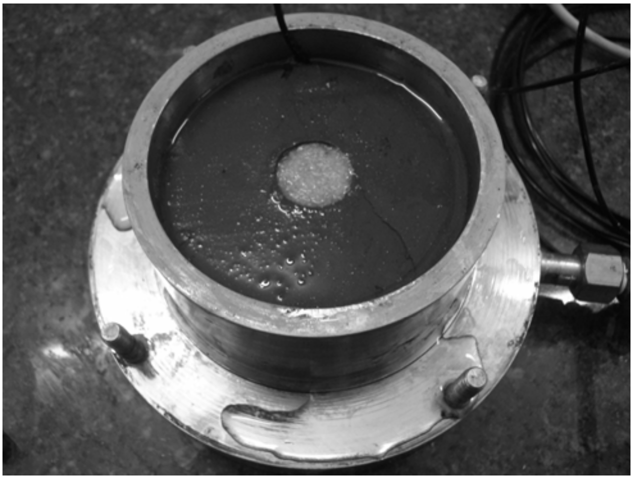

Fig. 3 Photograph of the Apparatus Showing the Sand Drain

In the experiments, drainage was allowed only in the radial direction. The top cap was placed above the soil sample after introducing the rubber sheet, as schematically shown in Figure 2. The top rubber sheet snugly fits with the cell and prevents the drainage through the top. During consolidation, the water drains towards the central well and drains out through the bottom drainage valve.

A consolidation pressure of 65 kPa, which was the preconsolidation pressure of the soil before installation of the sand drain, was initially applied on the soil sample and allowed to reconsolidate. The required consolidation pressure increment of 72 kPa (total applied pressure = 137 kPa) was then applied through a rigid perforated loading platen so that conditions of equal strain hold good. During consolidation, pore pressure and settlement data were collected. Once the consolidation was over, the soil sample was very carefully ejected and water content variation from the drain boundary to the unit cell boundary was measured.

Conventional one-dimensional consolidation test was also conducted on the IIT clay as per standard specifications (IS: 2720, Part: XV, 1965) so as to find out the consolidation properties, using the same consolidation cell without the sand drain. Falling head permeability test was also conducted to obtain the permeability in the vertical direction at the end of consolidation under every pressure increment.

Experimental results

Typical time-settlement curve under radial drainage is shown in Figure 4a. The shape of the time-settlement curve in the logarithmic plot is very similar to the curves usually obtained under one-dimensional consolidation. The pore pressure variation with time is shown in Figure 4b.

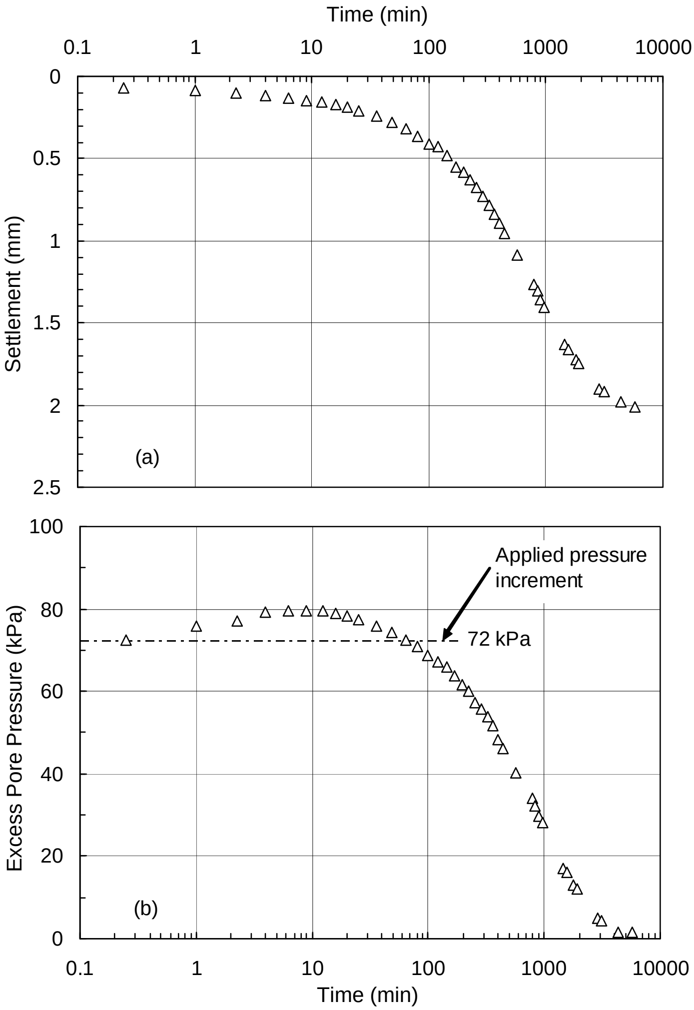

Fig. 4 (a) Surface Settlement and (b) Pore Pressure Variation with Time at Point C during Consolidation

It is interesting to note that the pore pressure increases initially, before dissipation, to a maximum value of 79.6 kPa, which is 10.5% higher than the applied pressure increment of 72 kPa. This is attributed to the Mandal-Cryer effect (Mandel 1953; Cryer 1963) that was reported to occur in two and three-dimensional consolidation problems where stress redistribution takes place so as to maintain strain compatibility in the soil, which has undergone different degree of consolidation at a given time interval.

The variation of water content in the clay from the drain boundary to the outer boundary is shown in Figure 5a. The water content is not uniform but varies along the diameter of the sample. The water content at the outer boundary of the sample is maximum and minimum near the drain. Atkinson *et al.* (1985) also observed similar results in triaxial samples under radial drainage. The triaxial samples were consolidated by providing radial drainage strips, a common technique adopted to accelerate the consolidation phase before shearing. At the end of consolidation, it was observed that the water content is maximum at the centre and minimum near the drainage boundary.

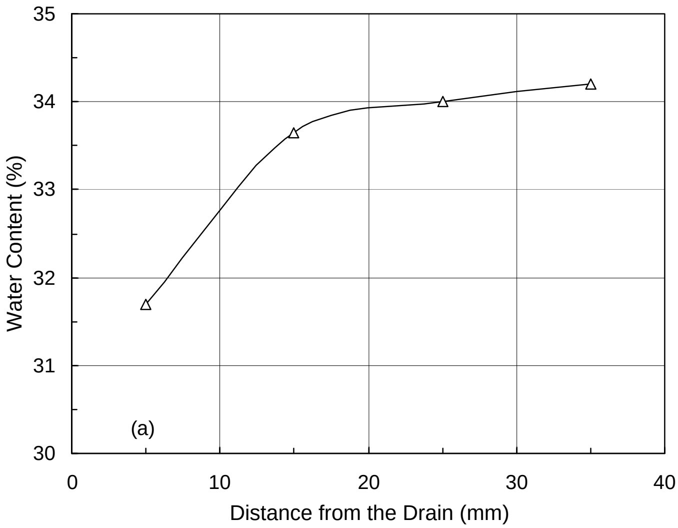

Fig. 5 (a) Variation of Water Content

The difference in water content between the boundary and near the drain is only about 3%. However, the shear strength of the clay along the diameter of the soil may be significant. In order to evaluate this indeed the case, the undrained shear strength of the soil (c_u) was predicted based on the critical state concept (Schofied and Wroth 1968) using the expression:

$$c_u = 0.5M \exp\left(\frac{\Gamma - 1 - wG}{\lambda}\right) \tag{1}$$

where M = slope of the critical state line in the p'-q stress space; p' = mean effective stress; q = deviatoric stress; Γ = specific volume of soil at critical state when p'=1.0 kPa. G, w and λ are the specific gravity, water content and the slope of the specific volume-ln p' plot, respectively. The variation of the undrained shear strength along the diameter of the soil is shown in Figure 5b. The difference in strength is as much as 25 kPa. The result shows that the end state of the soil sample after consolidation under radial drainage is highly non-uniform. The reason behind this non-uniformity is evaluated through finite element analysis.

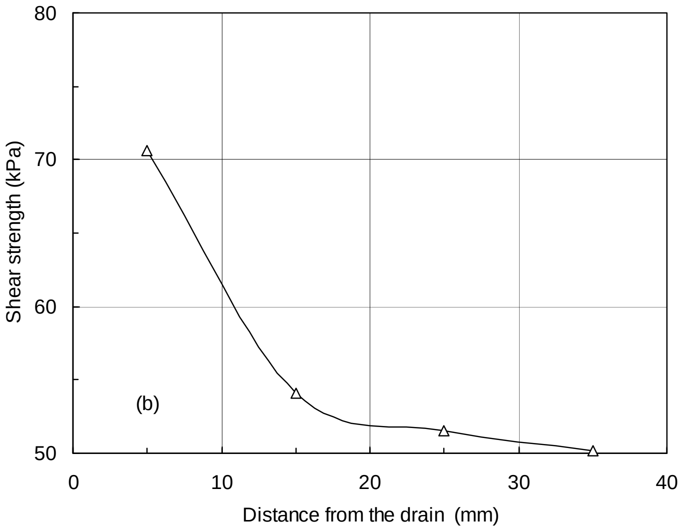

Fig. 5 (b) Strength along the Radius of the Sample

Finite Element Analysis

One of the important input parameters required for the finite element analysis is the permeability in the horizontal direction. In the present study the vertical permeability was determined by performing a falling head permeability test. From the measured permeability values under 65 kPa and 137 kPa, the average vertical permeability in the pressure range of 65-137 kPa was determined to be 7.89 x 10^{-11} m/s. The horizontal permeability value was indirectly calculated from the radial consolidation test results, as described below.

Anisotropy in permeability, which is defined as the ratio of horizontal permeability (k_h) to the vertical permeability (k_v) may be estimated from consolidation test results. As the coefficient of volume change (m_v) is the same under one-dimensional consolidation and under radial consolidation,

$$\frac{k_h}{k_v} = \frac{c_r}{c_v} \tag{2}$$

where, c_r is the coefficient of consolidation under radial drainage and c_v is the coefficient of consolidation under vertical drainage. Values of c_r and c_v can be determined by curve fitting procedures using Barron's theory (Barron 1948) and Terzaghi's theory (Terzaghi 1943), respectively. In the present study, a matching method suggested by Robinson and Allam (1998) was adopted to evaluate both c_v and c_r. Based on Terzaghi's solution for the one-dimensional consolidation, the settlement (S), at any time (t) may be expressed as,

$$S = (S_{100} - S_0)\left[1 - \frac{8}{\pi^2}\sum_{n=0}^{\infty}\frac{1}{(2n+1)^2}\exp\left(-\frac{\pi^2}{4}(2n+1)^2\frac{c_v t}{d^2}\right)\right] + S_0 \tag{3}$$

where,

So = Initial compression under one-dimensional consolidation,
S100 = Settlement corresponding to 100% primary consolidation under one-dimensional consolidation, and
d =length of drainage path

By performing a non-linear regression analysis on the set of time-settlement data obtained from the laboratory consolidation test, the constants S_o, S_{100} and c_v are determined as 0.04 mm, 1.87 mm and 1.12 x 10^{-4} cm^2/s, respectively. Eqn. (3) is also shown in Figure 6, which clearly shows that the matching of the theoretical solution with the experimental data.

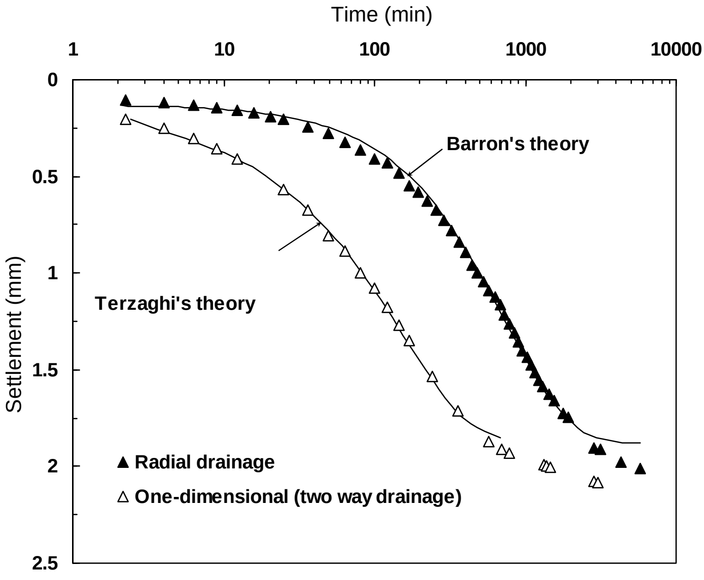

Fig. 6 Matching Method to Determine the Anisotropy in Permeability

The settlement (S_r) that occur at any time (t) under radial drainage, in the primary consolidation phase, for a given value of degree of consolidation (U_r) can be expressed as (Robinson and Allam, 1998),

$$S_r = S_{or} + (S_{100r} - S_{or}) \times U_r \tag{4}$$

From Eqn. (4) and using Barron's solution for U_r (Barron 1948),

$$S_r = S_{0r} + (S_{100r} - S_{0r})\left[1 - \exp\left(\frac{-8}{F(n)} \times \frac{c_r t}{d_e^{\,2}}\right)\right] \quad (5)$$

Where, S_{0r}, S_{100r} are initial compression and 100% primary consolidation, respectively, under radial drainage and and

$$F(n) = \frac{n^2}{(n^2 - 1)}\ln(n) - \frac{(3n^2 - 1)}{4n^2} \quad (6)$$

Very similar to the case of one-dimensional consolidation, non-linear regression analysis was performed and the values of S_{0r}, S_{100r} and c_r are found to be 0.13 mm, 1.88 mm and 2.71 x 10^{-4} cm^2/s, respectively. Equation (5) is also plotted in Figure 6. The theoretical curves match well with the respective response curves generated experimentally. Using Eqn. (2), the value of anisotropy in permeability is determined to be 2.4. Therefore, the permeability in the horizontal direction in the pressure range of 65-137 kPa is 1.89 x 10^{-10} m/s. This value of permeability is adopted in the analysis.

The finite element simulations were performed using the SAGE-CRISP finite element package (SAGE, 2001). In the analysis deformation has been coupled with Biot's (Biot, 1941) pore pressure dissipation equations and the advantage of axi-symmetry was utiliesed. The layout of the finite element mesh used in the analysis is shown in Figure 7. Quadrilateral consolidation elements were used to represent the clay. The vertical boundaries were modeled as frictionless. The draining boundary was modeled by setting the excess nodal pore water pressure to zero. The Cam-clay soil model was used and the relevant soil parameters, determined for this clay, are listed in Table 2. It was assumed in the analysis that the stiffness of the sand drain is equal to the stiffness of the clay and the sand drain act as a perfect drain.

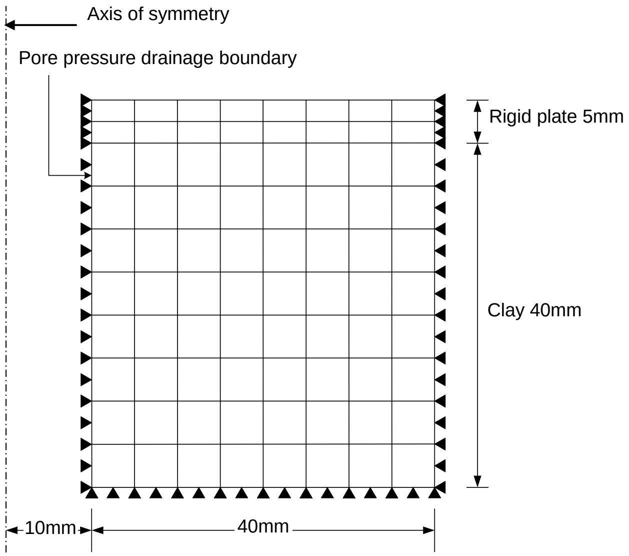

Fig. 7 Finite Element Mesh Used for the Analysis

Table 2 Properties Used for the Finite Element Analysis

Parameter	***Value***
Slope of the normal consolidation line (λ)	0.19
Slope of the recompression line (κ)	0.03
Slope of the critical state line (M)	0.77
Void ratio on critical state line at unit stress	1.81
Permeability in the vertical direction	7.89×10^{-11} m/s
Permeability in the horizontal direction	1.89×10^{-10} m/s

The settlement at the surface and the pore pressure at location C with coordinates (48.91, 20) are shown in Figure 8a and 8b, respectively.

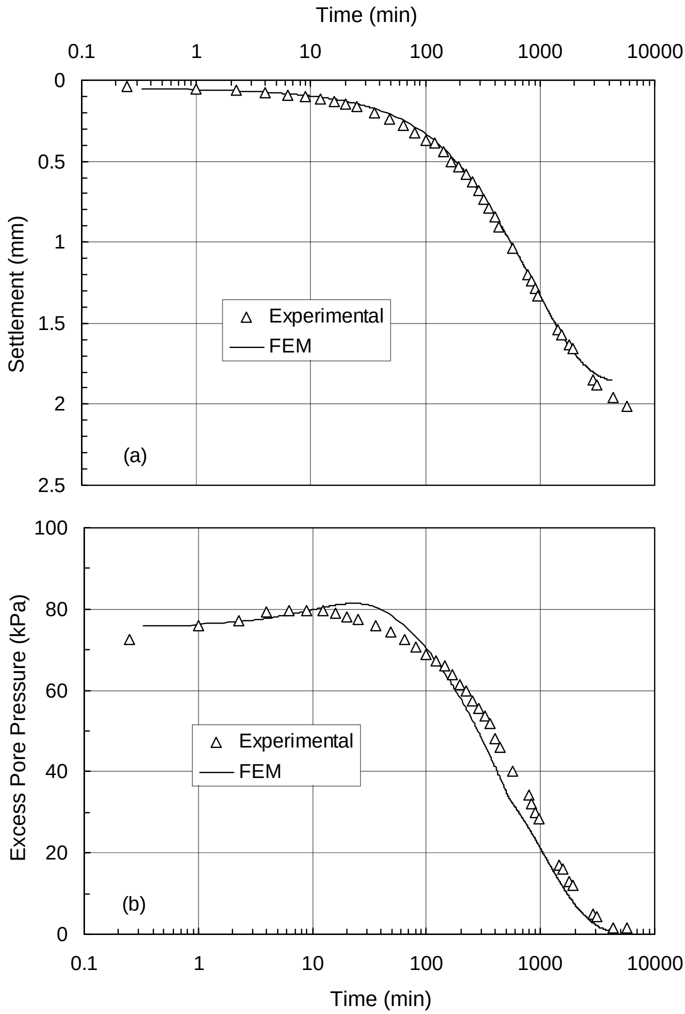

Fig. 8 (a) & (b) Comparison of Experimental Results with Finite Element Analysis

The experimental data are also shown in the figure and good agreement is observed. The finite element analysis also predicts the Mandel-Cryer effect in the early stages of consolidation. Guided by this, the stress condition at the end of consolidation is further evaluated.

The total stress, pore pressure and effective stress variation at the three locations, A(14.1, 20), B(20.69, 20) and C(48.91, 20), are shown in Figures 9a, b and c, respectively.

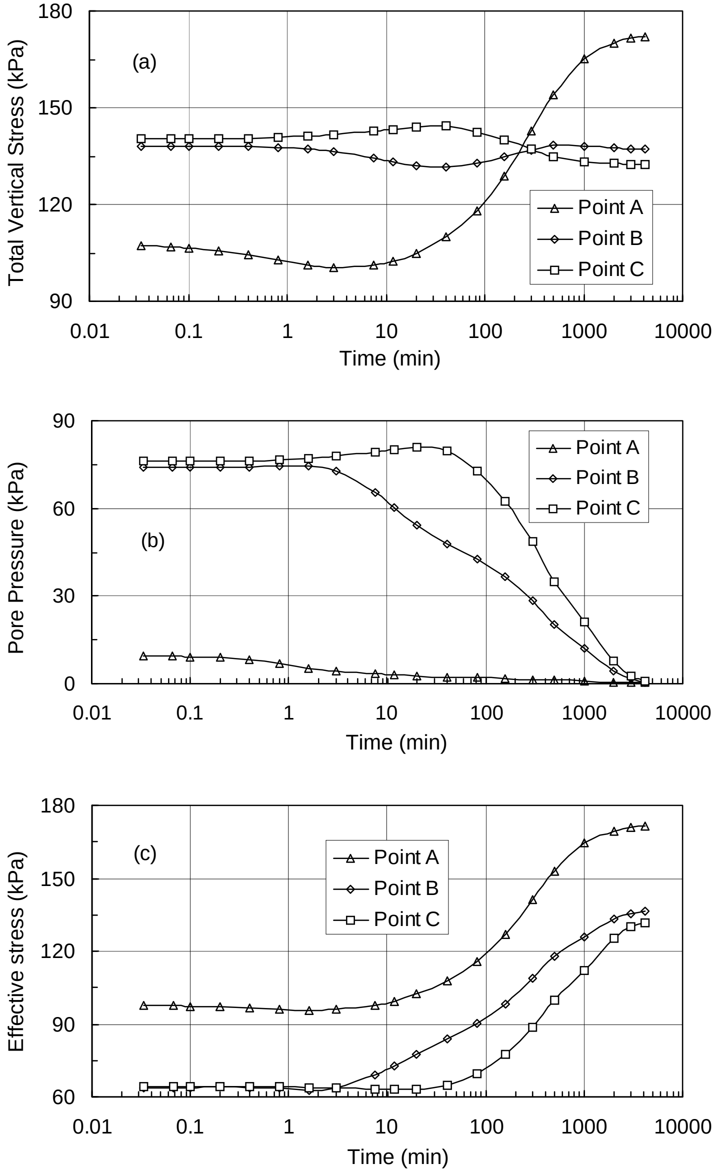

Fig. 9 (a), (b), (c) Variation of Total Stress, Pore Pressure and Effective Stress

The plot clearly shows that stress redistribution takes place during the consolidation process. As expected, the pore pressure near the drain dissipates faster and the effective stress is higher near the draining boundary than at other locations in the early stages of consolidation. This leads to stiffer clay around the drain at the centre, which attracts more loads towards it to maintain equal strain. Similar observations were reported by Al-Tabba and Wood (1991). This redistribution of stresses continues until the end of consolidation. The variation of effective stress from the drain to the edge of the unit cell is shown in Figure 10a. Clearly, large percentage of stress is transferred to the clay near the drain. Knowing the effective stress variation, the water content is calculated from the following void ratio (e)-effective stress (σ_v') relation obtained from one-dimensional consolidation tests:

$$e = 1.83 - 0.19 \log_e \sigma_v' \tag{7}$$

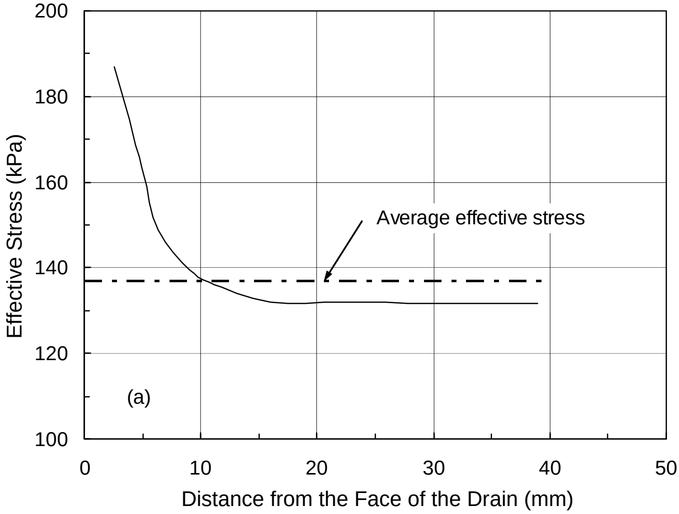

Fig. 10 (a) Variation of Effective Stress

The predicted water content variation is shown in Figure 10b along with the experimental values. The expected water content under for effective stress of 137 kPa under one-dimensional conditions is also marked in the figure. Both experimental results and the calculated values based on Eq. (7) for the effective stresses obtained from the finite element analysis show that the end condition of the soil after consolidation, under radial drainage, is not uniform.

The predicted horizontal and vertical displacements at three locations in the clay during the consolidation are shown in Figure 11. The plot depicts that the consolidation process during radial drainage is not one-dimensional. Similar results were earlier reported by Pyrah *et al*. (1999). The soil initially moves towards the drain and therefore, the soil adjacent to the drain is subjected to a large lateral compressive strain together with vertical strain. This leads to a denser soil near the drain with lesser water contents and higher stiffness. The stiffer material attracts more loads leading to higher effective stress at the end of consolidation at which the excess pore water pressure is fully dissipated. At later stages of consolidation, the soil moves back towards its original position. The soil at the outer boundary of the cell moved back to its original position. However, the soil near the drain did not move back to its original position.

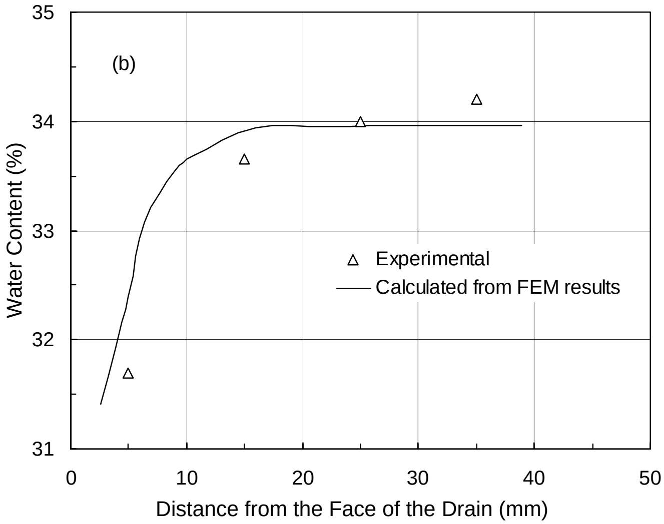

Fig. 10 (b) Predicted and Measured Water Content along the Radius of the Sample

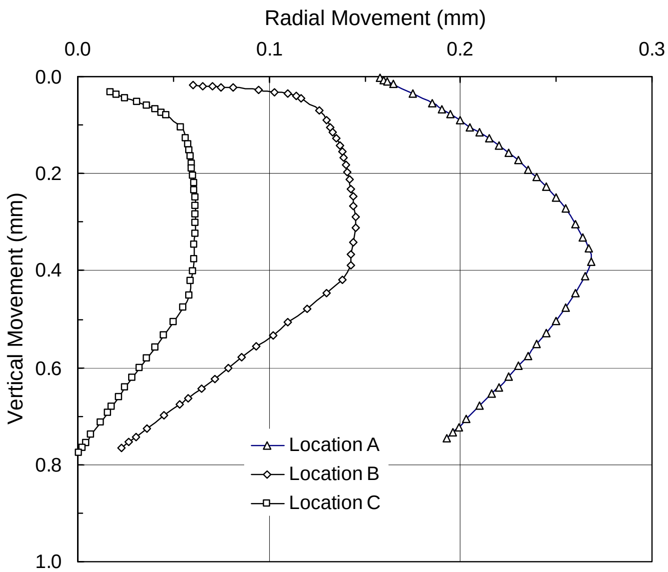

Fig. 11 Soil Movement During the Consolidation at the three Locations

The above results based on laboratory study suggest that the end state of a consolidated ground in the field after preloading under radial consolidation is likely to be not uniform. The consequence of this non-uniformity on subsequent construction of structure needs further investigation.

Conclusions

Unit cell experiment with pore pressure measurements on a clay under radial drainage indicates that the pore water pressure initially rise before dissipation showing the presence of Mandel-Cryer effect. The end condition of the soil after consolidation is non-uniform, with higher water content at the outer undrained boundary and less water content near the drain. Finite element analysis shows that stress redistribution takes place during consolidation and the effective stress near the drain is considerably larger than that at the outer boundary. This leads to non-uniform water content. The analysis also suggest that in the early stages of consolidation soil moves towards the drain in addition to moving vertically, leading to a complex vertical and lateral displacement pattern. The non-uniformity of the ground needs evaluation and the consequence of this non-uniformity needs further study.

Acknowledgements

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References

Al-Tabba, A. and Wood, D. M. (1987): ‘Some measurements of the permeability of kaolin’, *Geotechnique*, 37(4), pp. 499-503.

Al-Tabba, A. and Wood, D. M. (1991): ‘Horizontal drainage during consolidation: insights gained from analyses of a simple problem’, *Geotechnique*, 41(4), pp. 571-585.

Atkinson, J. H., Evans, J. S., and Ho, E. W. L. (1985): ‘Non-uniformity of triaxial samples due to consolidation with radial drainage’, *Geotechnique*, 35(3), pp 353-355.

Barron, R. A. (1948): ‘Consolidation of Fine Grained Soils by Drain wells’, *Transactions of ASCE*, 113, pp. 718-724.

Berry, P. L. and Wilkinson, W. B. (1969): ‘The Radial Consolidation of Clay Soils’, *Geotechnique*, 19(2), pp. 253-284.

Biot, M. A. (1941): ‘General theory of three dimensional consolidation’, *Journal of Applied physics*, 12(2), pp. 155-164.

Cryer, C. W. (1963): ‘A comparison of the three dimensional consolidation theories of Biot and Terzaghi’, *Quarterly Journal of Mechanics and Appied Mathematics*., 16(4), pp. 401-412.

Druck Limited, (2007): DRUCK Model PDCR81, Miniature Series-06/99, High performance pressure transducers [on-line]. Available from http://www.druck.com, Druck Ltd., UK.

Fang, Z. and Yin, J. H. (2006): 'Physical modeling of consolidation of Hong Kong marine clay with prefabricated vertical drains', *Canadian Geotechnical Journal*, 43, pp. 638-652.

Hansbo, S., Jamiolkowski, M. and Kok, L. (1981): Consolidation by vertical drains, *Geotechnique*, 31(1) pp. 45-66.

Head, K. H. (1986): *Manual of soil laboratory testing*. Vol. 3, Pentech Press, London.

Indraratna, B. and Redana, I. W. (1998): 'Laboratory determination of smear zone due to vertical drain installation', *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 124(2), pp. 180-184.

Indraratna, B. and Redana, I. W. (2000): 'Numerical modeling of vertical drains with smear and well resistance installed in soft clay', *Canadian Geotechnical Journal*, 37, pp. 132-145.

Indraratna, B., Rujikiatkamjorn, C., and Sathananthan, I. (2005): 'Radial consolidation of clay using compressibility indices and varying horizontal permeability', *Canadian Geotechnical Journal*, 42, 1330-1341.

IS: 2720 (1965): Methods of Test for Soils - Part XV: Determination of Consolidation Properties, *Bureau of Indian Standards*, New Delhi.

Lee, F. H. (1990): 'Frequency response of miniature pore pressure transducers in centrifuge model tests', *Geotechnical Testing Journal*, ASTM, 13(3), pp. 201-207.

Leo, C. J. (2004): 'Equal strain consolidation by vertical drains', *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130(3), pp. 316-327.

Mair, R. J. (1979): *Centrifugal modeling on tunnel construction in soft clay*. PhD thesis, University of Cambridge, UK.

Mandel, J. (1953): 'Consolidation des sols (etude mathematique)', *Geotechnique*, 3(7), pp. 287-299 (in French).

Mitchell, J. M. and Jardine, R. (2002): *A guide to ground treatment*, CIRIA, London.

Nash, D. F. T. and Ryde, S. J. (2001): 'Modelling consolidation accelerated by vertical drains in soils subject to creep', *Geotechnique*, 51(3), pp. 257-273.

Pyrah, I. C., Smith, I. G. N., Hull, D. and Tanaka, Y. (1999): 'Non-uniform consolidation around vertical drains installed in soft ground', Geotechnical Engineering for Transportation Infrastructure, Barden et al. (Eds), Balkema, pp. 1-7.

Rendulic, L. (1935): 'Der hydrodynamische spannungsausgleich in zentral entwässerten Tonzylindern.' *Wasserwirtschaft und Technik*, 2(23–26), pp. 250–253.

Robinson, R. G. (1997): 'Determination of radial coefficient of consolidation by the inflection point method', *Geotechnique*, 47(5), pp. 1079-1081.

Robinson, R. G. and Allam, M. M. (1998): 'Analysis of consolidation data by a non-graphical matching method,' *Geotechnical Testing Journal*, ASTM, 21(2), pp. 140-143.

Rowe, P. W. and Barden, L. (1966): 'A new consolidation cell', *Geotechnique*, 16(2), pp. 162-170.

SAGE (2001). SAGE-CRISP, Crisp Consortium UK. User Manual. *Ver 4.3a*.

Schofield, A. N. and Wroth, C. P. (1968): *Critical State Soil Mechanics*, McGraw-Hill, London.

Seah, T. H., Tangthansup, B. and Wongsatian, P. (2004): 'Horizontal coefficient of consolidation of soft Bangkok clay', *Geotechnical Testing Journal*, ASTM, 27(5), pp. 1-11.

Sharma, J. S., and Xiao, D. (2000): 'Characterisation of a smear zone around vertical drains by large scale laboratory rests', *Canadian Geotechnical Journal*, 37(6), pp. 1265-1271.

Sridharan, A., Prakash, K., and Asha, S. R. (1996): 'Consolidation Behaviour of Clayey Soils Under Radial Drainage', *Geotechnical Testing Journal*, 19(4), pp. 421-431.

Suits, L. D., Gemme, R. L. and Masi, J. J. (1986): 'Effectiveness of prefabricated drains on Laboratory Consolidation of Remoulded soils', *Consolidation of Soils: Testing and Evaluation, ASTM, STP 892*, R. N. Yong and F. C. Townsend Eds., American Society for Testing and Materials, Philadelphia, pp. 663-683.

Terzaghi, K. (1943): *Theoretical Soil Mechanics*, Wiley, New York