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CONSOLIDATION BEHAVIOR OF SOIL-CEMENT COLUMNS IMPROVED GROUND

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CONSOLIDATION BEHAVIOR OF SOIL-CEMENT COLUMN IMPROVED GROUND

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Abstract: Columnar inclusion is one of the effective and widely used methods for improving engineering properties of soft clay ground. This article investigates the consolidation behavior of composite soft clay ground using physical model tests under an axial-symmetry condition and finite element simulations by PLAXIS 2D program. It is found out that the final settlement and the rate of consolidation of the composite ground depend on the stress state (of what ?). For an applied stress much lower than the failure stress, the final settlement of the ground is insignificant and the consolidation is fast. When the soil-cement column fails, the stress on column suddenly decreases (due to strain-softening) meanwhile the stress on soil increases to maintain the force equilibrium. Consequently, the excess pore pressure in the surrounding clay increases immediately. The cracked soil-cement column acts as a drain, accelerating the dissipation of the excess pore pressure. The consolidation of the composite ground is mainly in vertical direction and controlled by the area ratio, the ratio of diameter of soil-cement column to the diameter of composite ground, a. The stress on column is low for the composite ground with high value of a, resulting in less settlement and fast consolidation. For a long soil-cement column, the excess pore pressures in the surrounding clay and the column are practically the same at the same consolidation time for the whole improvement

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depth. It is proposed thatthe soil-cement column and surrounding clay be assumed to form a compressible ground and the consolidation is in the vertical direction, the composite coefficient of consolidation ($c_{v(com)}$) obtained from the physical model test on the composite ground can be used to approximate the rate of consolidation. This approximation is validated via the finite element simulation. The proposed method is highly useful for geotechnical engineers because of its simplicity and reliable prediction.

Keywords: composite ground, deep mixing, finite element analysis, physical model

1. INTRODUCTION

The method of mixing cement slurry or powder into soft ground (deep mixing) to create soil-cement columns is widely used to improve the engineering properties (shear strength and compressibility) of the thick deposit of soft ground (Broms and Boman, 1979; and Bergado et al., 1994). Studies on the prediction of engineering performance of soil-cement column improved ground have become attractive nowadays to both practitioners and researchers in Asia. Most of the available researches related to the deep mixing method were confined to the strength and the overall stiffness of the soil-cement columns (Broms, 1979; Kawasaki et al., 1981; Kamon and Bergado, 1992; Walker, 1994; Kamaluddin and Balasubramaniam, 1995; Schaefer et al., 1997; Lin and Wong, 1999; Fang et al., 2001; Porbaha et al., 2001; Miura et al. , 2001; Yin, 2001; Porbaha, 2002; Tan et al., 2002; Horpibulsuk et al., 2004a, b; 2005; 2011a and b). The field mixing effect such as installation rate, water/cement ratio and rate of rotation on the strength development of soil-cement columns was investigated by Nishida et al. (1996) and Horpibulsuk et al. (2004c and 2011b). Based on the available compression and shear test results, many constitutive models were developed to describe the engineering behavior of cemented clay (Gens and Nova, 1993; Kasama et al., 2002; Horpibulsuk et al., 2010; Suebsuk et al., 2010 and 2011).

Even though the understanding of consolidation behavior of composite ground is significant for determining the final settlement and rate of settlement, the study on the topic is very limited. Terashi and Tanaka (1981) studied the bearing capacity and consolidation of the composite ground. Yin and Fang (2006) and Chai and Pongsivasathit (2010) investigated the consolidation behavior of the laboratory model composite ground under axial-symmetrical condition. Yin and Fang (2006) hypothesized that the rate of consolidation of the composite ground was fast because the soil-cement column had high permeability and worked as a drain. Chai et al. (2006) made a discussion on this issue and explained that the soil-cement column accelerated the consolidation process due to its high stiffness, which resulted in a higher coefficient of consolidation rather than higher coefficient of permeability. Basically, the soil-cement column and the surrounding clay were consolidated under the same overburden pressure after deep mixing execution; therefore, they possess practically the same void ratio and coefficient of permeability (Horpibulsuk et al., 2011b).

This article aims to illustrate the consolidation mechanism of the composite ground and propose a practical method of estimating the consolidation settlement with time. Both laboratory tests on model composite grounds and numerical simulations were performed for this objective. The laboratory tests were carried out in different conditions of the applied vertical stress, cement content and area ratio, which is the ratio of the soil-cement column diameter to the composite ground diameter. Numerical simulations were performed by using PLAXIS program and compared with the laboratory test results to analyze the consolidation mechanism and to understand effects of area ratio and cement content on the consolidation response under different vertical stresses. Based on the laboratory and numerical studies, a practical method of estimating consolidation settlement is proposed. The proposed method is highly useful for geotechnical engineers because of its simplicity and reliable prediction.

2. LABORATORY INVESTIGATION

2.1 Soil Sample

The soil sample was soft Bangkok clay collected from Chidlom district, Bangkok at a depth of 3 meters. The clay was composed of 11% sand, 17% silt and 72% clay. The natural water content was 80% and the specific gravity was 2.68. The liquid and plastic limits were

81% and 34%, respectively. Based on the Unified Soil Classification System (USCS), the clay was classified as high plasticity (CH). Groundwater was about 1.0 m from surface.

2.2 Physical Model

A cylindrical stainless steel mold 300 mm in diameter and 450 mm high was used for the present experimental research as shown in Figure 1. The soil-cement column was installed in the middle of mold. This physical model test is to simulate a composite foundation where soil-cement columns are installed vertically in a triangular or square pattern at the same spacing into a horizontal clay layer and are subjected to uniform vertical fill loading over an extensive area (Yin and Fang, 2006). The consolidation around a soil-cement column was approximately axisymmetrical with an equivalent diameter, at the boundary of which lateral displacement were not permitted. The vertical load on the model ground was applied by dead weights on a level hanger.

2.3 Preparation of Model Ground

The soft Bangkok clay was thoroughly mixed with water to attain a water content of about twice liquid limit. The remolded clay slurry was transferred to the cylindrical stainless mold having 30 mm sand at the base and a plastic PVC tube (either 50 mm or 100 mm in diameter) in the middle. The tube had many small holes and covered with thin geotextile to accelerate the consolidation. Because the tube was not directly contact to the surrounding clay, it was easily removed after consolidation. During transferring clay slurry, the mold was vibrated to remove air bubble. A 8 mm thick acrylic plate was placed on the top of the model composite ground to apply a vertical consolidation stress of 20 kPa. The plate had a hole (either diameter of 52 or 102 mm) in the middle. During the consolidation process, the settlement was measured to ensure the end of consolidation. The final height of the clay after consolidation was about 200 mm with a water content of about 60%. The unconfined compressive strength of this reconstituted clay was 25 kPa and the compression and recompression indexes obtained from oedometer test were 0.498 and 0.186, respectively. The effective strength parameters in compression were $c' = 1$ kPa and $\phi' = 21°$.

The model soil-cement column was made up by mixing Portland cement with cement slurry at cement contents, *C*, of 20 and 40%. The clay-cement paste was mixed thoroughly in a soil mixer for 10 min. The paste was poured into a cylindrical plastic PVC tube with internal diameters of 50 and 100 mm and height of 200 mm. After 24 hours, the model columns were dismantled and wrapped in vinyl bags. They were cured in a humidity room of constant temperature (20±2°C) for 28 days. The water contents of the model columns after curing were 80 and 70% for *C* = 20 and 40%, respectively. The unconfined compressive strengths were 500 and 1200 kPa for *C* = 20 and 40%, respectively. The effective strength parameters were $c' = 200$ kPa and $\phi' = 25°$ for *C* = 20% and $c' = 500$ kPa and $\phi' = 27°$ for *C* = 40%.

After finishing the consolidation of the reconstituted clay, the PVC tube and geotextile were removed and the soil-cement column was inserted into the hole in the middle of the reconstituted model ground to form the composite model. Six miniature pore water pressure transducers (PPTs) were installed at certain positions and used to measure the pore water pressure in the surrounding clay (*vide* Figure 1). All PPTs were penetrated into the surrounding clay through pre-drilled holes of the cylindrical mold. PPT 1 and PTT 2 were fixed on the bottom, PPT 3 and PTT 4 were on the middle and PPT 5 and PTT 6 were on the top of the surrounding clay ground. One small earth pressure (EPC) (10 mm thick and 50 mm in diameter) was placed on the surface of the surrounding clay and the other was on the top of the soil-cement column. Both EPC were covered with 10 mm sand. In addition, two linear variable differential transformers (LVDTs) were used to measure the settlement of the model composite ground. The drainage was only allowed at the top of the composite ground (single drainage). Table 1 summarizes the test conditions for the model composite ground.

4. TEST RESULTS

4.1 Consolidation behavior of the model composite ground

Figure 2 shows a relationship between vertical stress loading and time for a composite ground with a = 1/6 and q_u = 1200 kPa (diameter of the column, D_{col} = 50 mm and C = 40%). The load applied is the step loading that the next loading was applied after finishing consolidation by the previous loading. Figure 3 shows the relationship between settlement and time for each load increment. The final settlement increases with the applied load until failure. The total final settlement was 15 mm and the final vertical strains were 0.49, 1.09, 1.42 and 4.98% for vertical stresses of 20, 40, 60 and 90 kPa, respectively. The relationship between average degree of saturation and time is depicted in Figure 4. With the increase in applied vertical stress, the average degree of saturation increases. The maximum average degree of saturation occurs for the applied vertical stress of 80 kPa (failure stress of the composite ground). This consolidation characteristic is similar to that of the natural clay. When the applied vertical stress is far lower than the yield (failure) stress, the final vertical strain is small and consolidation is fast. On the other hand, when the applied vertical stress approaches the yield stress, the final vertical strain is large and consolidation is slow.

Figure 5 shows the relationship between the stresses on the soil-cement column and the surrounding clay for a = 1/6 and q_u = 1200 kPa. Figure 6 shows the stress concentration, n, which is the ratio of the stress on the column to the stress on the surrounding clay. Immediately after applying the applied vertical stress, both the stresses on the soil-cement column, σ_{col} and on the surrounding clay, σ_{col} increase sharply. But the σ_{col} is large compared with the σ_{soil} because of the high stiffness of the soil-cement column. With time, the σ_{col} increases as the σ_{soil} decreases, therefore the stress concentration increases. At 80 kPa (failure) vertical stress, the σ_{col} increases with time for the stress states in pre-failure of soil-cement column. At the failure of the column, the σ_{col} decreases suddenly while σ_{soil} increases, indicating that the stresses sustained by the column transfer to the surrounding clay. The strain softening of the soil-cement column is caused by the crushing of soil-cementation structure (Miura et al., 2001; Horpibulsuk et al., 2004b; Horpibulsuk et al., 2005; Horpibulsuk et al., 2010; and Suebsuk et al. 2010 and 2011). The failure vertical stress on the column measured from the EPC was 1400 kPa, which is slightly higher than the unconfined compressive strength.

Figure 7 shows the relationship between excess pore water pressure in the surrounding clay and radial distance at the applied vertical stresses of 40 and 80 kPa. Before failure (at 40 kPa vertical stress), excess pore pressures increase sharply after applying vertical stress and then decreases with time (consolidation). The excess pore pressures decrease with radial distance toward the soil-cement column but the variation is small. This test result is consistent with that reported by Yin and Fang (2006). Under 80 kPa vertical stress (failure load), at early time ($t < 240$ min) the excess pore pressures decrease due to consolidation associated with the decrease in σ_{soil}. At $t = 240$ min (failure starts), the excess pore pressures increase suddenly due to the increase in σ_{soil} and then begin to dissipate. The excess pore pressures near the column decrease quicker than those points away from the column because the cracked column acts as the drain (Yu et al., 1999) and increases its permeability.

4.2 Numerical Simulation

The performance of composite ground (prior to column failure) was simulated by using the PLAXIS 2D program and the simulations were compared with the test results to understand the consolidation mechanism of composite ground and to understand the role of a and C on the consolidation behavior. The finite element mesh was comprised of 15-nodes triangular elements (*vide* Figure 8). Table 2 shows the model parameters for the surrounding clay and the soil-cement column. Overall, the numerical simulations are comparable with the test results.

Figure 9 shows the comparison between simulated and measured settlements versus time for different area ratios, a. Both settlement and rate of consolidation are governed by a. The composite ground with high a value exhibits low settlement and high rate of consolidation. The cement content does not play any significant role on the final settlement and consolidation time because the elastic modulus, E' of the column is almost the same for both cement contents tested. The simulated σ_{col} and σ_{soil} for different area ratios and cement contents are compared with measured ones in Figure 10. The σ_{soil} increases rapidly after applying vertical stress and then decreases with time due to the stress transfer to the soil-cement column. As the a decreases, the σ_{col} increases.

Figures 11 and 12 depict the change in excess pore pressure with radial distance at different consolidation times. The excess pore pressures in the soil-cement column dissipate quicker than those in the surrounding clay only at the top of the composite ground while the excess pore pressures in both the soil-cement column and the surrounding clay at the same consolidation time are practically the same for deeper improvement depth. The rate of consolidation is governed by the area ratio (*vide* Figure 11) and insignificantly by the cement content (*vide* Figure 12). As the area ratio increases, the rate of consolidation increases. Figure 13 shows the typical relationship between the excess pore pressure and depth at different consolidation times. The consolidation behavior of the surrounding clay for the points near and far away from the soil-cement column (at 5 and 10 cm from the column) is similar to the one-dimensional consolidation (Terzaghi, 1925) showing the maximum excess pore pressure at the bottom.

5. ANALYSIS AND DISCUSSION

From this study, the coefficients of permeability of soil-cement column and surrounding clay are practically the same; therefore the soil-cement column does not act as the drain. Because the soil-cement column enhances the yield stress and the stiffness to the composite ground, the composite ground is in over-consolidated state under the applied vertical applied stresses. At this state, the rate of consolidation is fast due to high coefficient of consolidation. The consolidation is mainly in vertical direction as indicated because the excess pore pressure dissipation in the surrounding clay is similar to the one-dimensional consolidation of the soft clay ground (*vide* Figure 13). The soil-cement column acts as the drain only when the soil-cement reaches the failure state. After failure, the stress sustained by the soil-cement column transfers to the surrounding clay, resulting in the decrease in stress concentration, n. Consequently, the average degree of consolidation of the composite ground increases (*vide* Figure 4). This situation is impossible in actual projects that the soil-cement columns are generally designed under working state. Because the cement content is mainly controlled unconfined compressive strength and effective strength parameters (Horpibulsuk et al., 2004a; Horpibulsuk et al. 2010; and Suebsuk et al., 2010), the final settlement is insignificantly dependent upon the cement content. The composite ground with the larger area ratio sustains lower stresses on the column and on the surrounding clay (*vide* Figure 10). As such, the rate of consolidation increases as the area ratio increases.

The effect of the drainage condition (single and double drainages) on the excess pore pressure development is also investigated using the numerical simulation. A typical composite ground, generally performed in practice, is simulated and shown in Figure 14. The diameters of the soil-cement column and composite ground were 1.0 and 6.0 m (a = 1/6) and the depth of soft clay was 15 m. The values of the model parameters presented in Table 2 and the elastic model of the soil-cement column of 120,000 kPa were used for this simulation. Figure 15 shows the change in the excess pore pressure with radial distance for both single and double drainage conditions. For both drainage conditions, the excess pore pressures in the soil-cement column around the drainage dissipate slightly quicker than those in the surrounding clay. The dissipation rate in the other parts of the soil-cement column is practically the same as that of the surrounding clay at the same improvement depth. From Figures 15 and 12, it is noted that as improvement increases, the variation in excess pore pressure in both soil-cement column and surrounding clay around the drainage decreases. For the long soil-cement column, which is a typical application, the variation in excess pore pressure around the drainage is insignificant and the excess pore pressures in both soil-cement column and surrounding clay at the same consolidation time are practically the same for the whole depth.

The change in excess pore pressure with consolidation time in the surrounding clay for both single and double drainage conditions is presented in Figure 16. For both conditions, the change in excess pore pressure with depth is similar to that of one-dimensional consolidation (Terzaghi, 1925) for both positions close to and far away from the soil-cement column. The final settlement of the composite ground is the same for both single and double drainage conditions (*vide* Figure 17). In other words, the drainage path controls the rate of consolidation but not the final settlement. The soil-cement column inclusion not only reduces the final settlement and but also enhances the rate of consolidation of the soft clay (increase the coefficient of consolidation). The time at 90% degree of consolidation, t_{90} of the composite ground is 687 min for single drainage while the t_{90} of the soft clay (without soil-cement column) is 8650 min. It is concluded from this study that the composite ground is in the over-consolidated state under the applied load and hence high coefficient of consolidation. The consolidation of the composite ground is mainly in vertical direction as indicated by the change of excess pore pressure in the surrounding clay with the improvement depth.

6. A METHOD FOR ESTIMATIING THE FINAL SETTLEMENT

From thisinvestigation, it is found that consolidation is mainly in the vertical direction. The excess pore pressures in both soil-cement column and surrounding clay are practically the same for the whole improvement depth. The fast consolidation rate in the soil-cement column occurs only at a particular limited portion (close to the drainage) and can be ignored for the long soil-cement column. Assuming that the consolidation of the composite ground is one-dimensional and the excess pore pressures in the soil-cement column and the surrounding clay are the same at the same consolidation time for the whole depth, the composite coefficient of consolidation, ($c_{v(com)}$) obtained from the laboratory model test can be used for estimating the change in settlement with time. This assumption considers that both soil-cement column and surrounding clay form a new compressible ground. The comparison of the settlement versus time relationship predicted by the finite element method and one-dimensional consolidation using $c_{v(com)}$ is being presented. The $c_{v(com)}$ values obtained from the laboratory model test for a = 1/6 were 0.096, 0.035 and 0.026 m^2/day for the applied stresses of 20, 40 and 60 kPa, respectively. Figures 18 and 19 show the comparisons of the settlement versus consolidation time relationship between finite element simulation and one-dimensional theory for single and drainage conditions, respectively. It is noted that relationships predicted by both finite element simulation and one-dimensional theory are in very good agreement. In practice, the consolidation settlement of the field composite ground is thus simply predicted using the laboratory consolidation test results of the model composite ground for a designed area ratio.

A stepwise procedure for estimating the consolidation settlement of the composite ground is summarized as follows:

1. From a designed (dead and live) load on the soft clay ground, design the diameter, strength and spacing of the soil-cement columns.
2. Prepare the model composite ground with the designed a and strength of the soil-cement column.

3. Perform a consolidation test on the model composite ground under the designed field vertical stress and determine the final strain, ε and $c_{v(com)}$.
4. From the final strain, determine final settlement of the composite ground from $\varepsilon \times H$ where H is the length of the soil-cement column.
5. Determine the consolidation settlement versus time.

5. CONCLUSIONS

This paper presents both the laboratory study and numerical simulation of the consolidation behavior of composite ground. The consolidation mechanism of the composite ground is revealed and the effect of the area ratio and cement content on the consolidation characteristics is presented. The following conclusions can be advanced from this study.

1. The soil-cement column inclusion enhances the yield stress and stiffness to the composite ground. The consolidation behavior is dependent upon the stress state. The settlement is less and consolidation is fast when the applied vertical stress is far below the yield (failure) stress. When the applied vertical stress is close to the yield stress, the settlement and consolidation time increase.
2. After applying the vertical stress on the composite ground, both the stresses on the soil-cement column and the surrounding clay increase suddenly. After that the stress on the surrounding clay decreases (stress transfer to the column) and hence the stress on the column increases with consolidation time. The decrease in stress on the surrounding clay is associated with the decrease in excess pore pressure.
3. At failure state of the composite ground, the stress on the soil-cement column decreases immediately whereas the stress in surrounding clay increases to maintain the force equilibrium. This results in the sharp increase in excess pore pressure. This is attributed to strain softening caused by the crushing of soil-

cementation structure. It has been observed that unlike reconstituted soils soft clay with strong cementation exhibits strong softening behavior in undrained situation in both normally consolidated and over consolidated states (Horpibulsuk et al, 2004b; Suebsuk et al, 2011). With time, the excess pore pressure decreases toward the cracked soil-cement column, which acts as the drain.

4. The area ratio, *a*, significantly affects the consolidation behavior of the composite ground whereas the cement content is usually insignificant. The composite ground with high *a* value, has a high load capacity and carries low stress. As such, the settlement is low and consolidation is fast.
5. Both the physical model test results and numerical simulations show that for a long soil-cement column, which is a typical field application, the variation in excess pore pressure around the drainage is insignificant and the excess pore pressures in both the soil-cement column and the surrounding clay are practically the same at the same consolidation time for the whole depth.
6. During working condition, the soil-cement column does not act as a drain and the consolidation is mainly in the vertical direction. Assuming that the soil-cement column and surrounding clay form a new compressible ground with high coefficient of consolidation, the consolidation settlement with time can thus be predicted simply from the composite coefficient of consolidation obtained from the physical model tests. It was seen that the predicted consolidation settlements by using the composite coefficient of consolidation and that by using the finite element method are comparable within an acceptable error. A stepwise procedure for predicting consolidation settlement is finally presented and the proposed method is useful for geotechnical engineers.

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Figure Captions

Figure 1: Location of instruments in the physical model: (a) plan view of positions of various transducers; (b) side view.

Figure 2: Relationship between applied vertical stress and time.

Figure 3: Relationship between measured settlement and time.

Figure 4: Relationship between average degree of saturation and time for each applied vertical stress.

Figure 5: Relationship between stresses on column and surrounding clay versus time.

Figure 6: Relationship between stress concentration and time.

Figure 7: Radial distribution of excess pore pressure at different times under vertical stresses of 40 kPa and 80 kPa.

Figure 8: Finite element model for the model composite ground.

Figure 9: Comparison between simulated and measured settlement with time for different area ratios and cement contents.

Figure 10: Comparison between simulated and measured stresses on column and surrounding clay with time for different area ratios and cement content.

Figure 11: Relationship between simulated excess pore pressure and radial distance at different times and area ratios.

Figure 12: Relationship between simulated excess pore pressure and radial distance at different times and cement contents.

Figure 13: Relationship between excess pore pressure and depth at different consolidation times for a = 1/6 and q_u = 1200 kPa.

Figure 14: Finite element model for studying the effect of drainage condition.

Figure 15: Change in excess pore pressure with radial distance for single and double drainage conditions.

Figure 16: Change in excess pore pressure with time for single and double drainage conditions.

Figure 17: Relationship between settlement and consolidation time for both single and double drainage conditions.

Figure 18: Settlement versus time relationship for single drainage condition.

Figure 19: Settlement versus time relationship for double drainage condition.

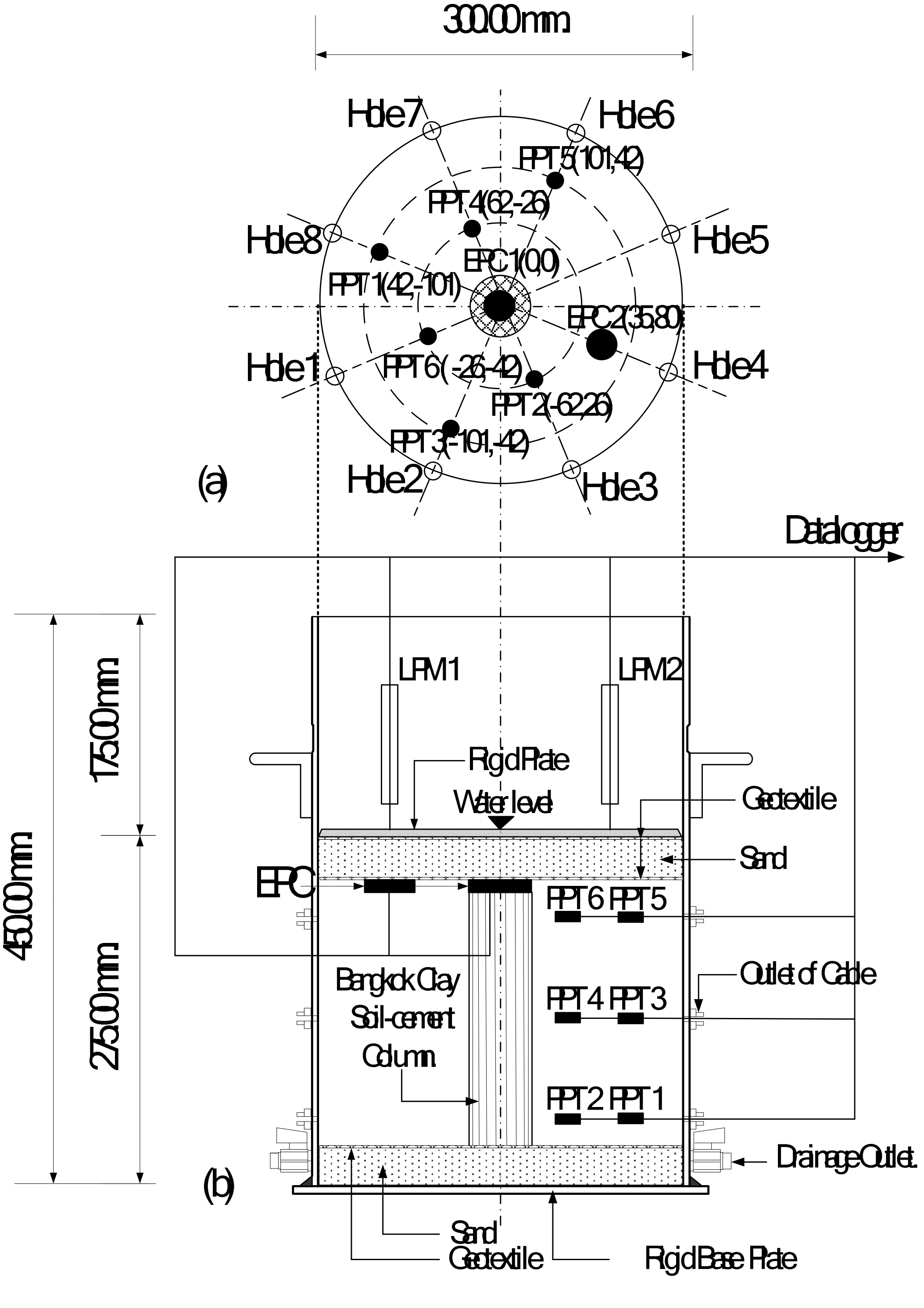


Figure 1: Location of instruments in the physical model: (a) plan view of positions of various transducers; (b) side view.

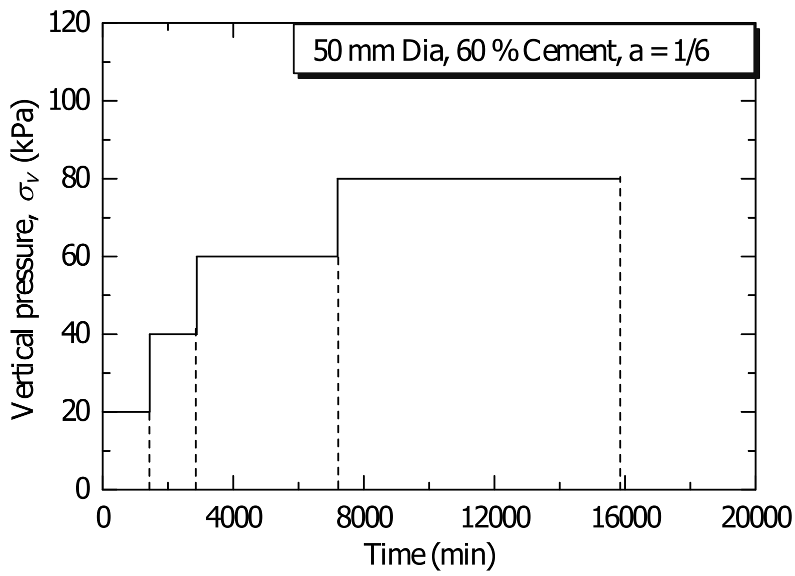


Figure 2: Relationship between applied vertical stress and time.

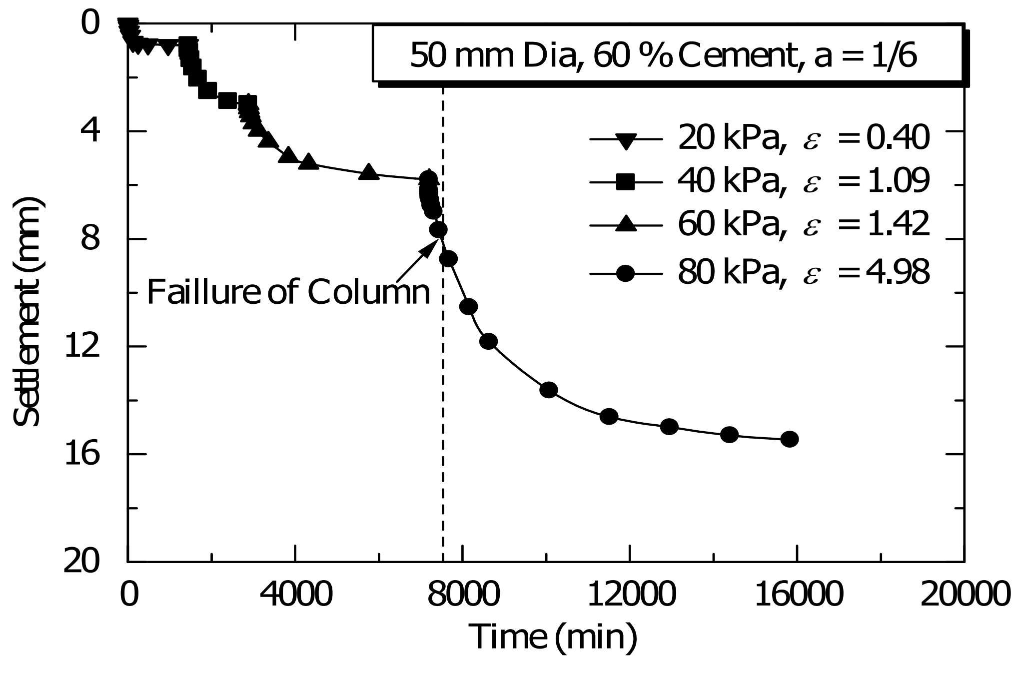


Figure 3: Relationship between measured settlement and time.

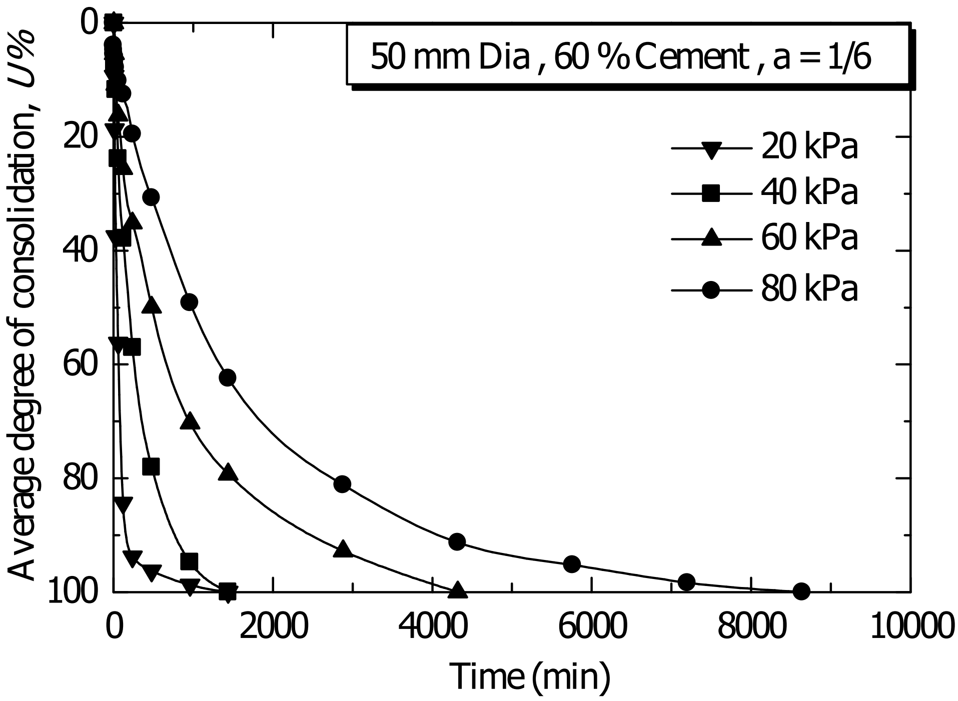


Figure 4: Relationship between average degree of saturation and time for each applied vertical stress.

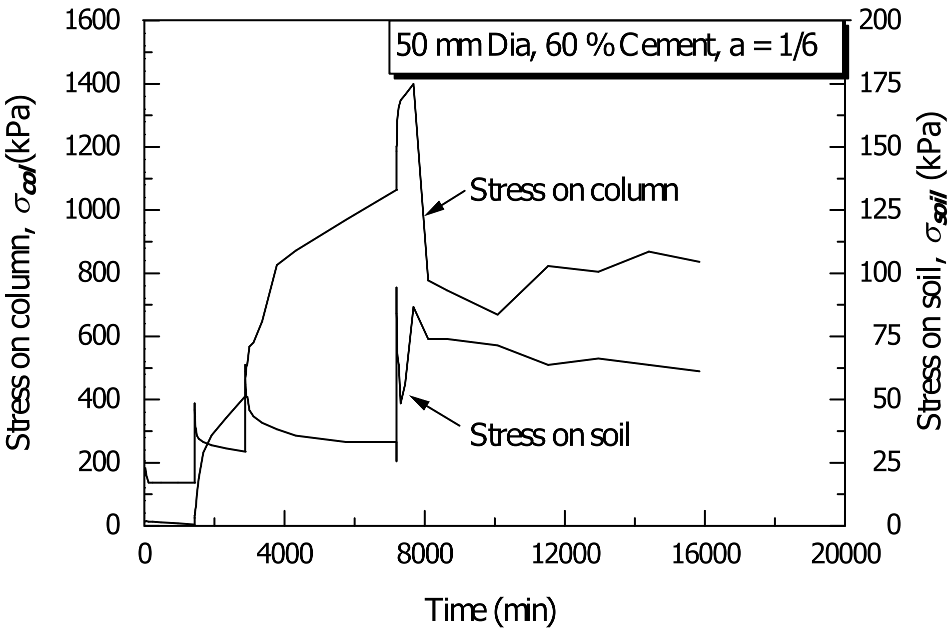


Figure 5: Relationship between stresses on column and surrounding clay versus time.

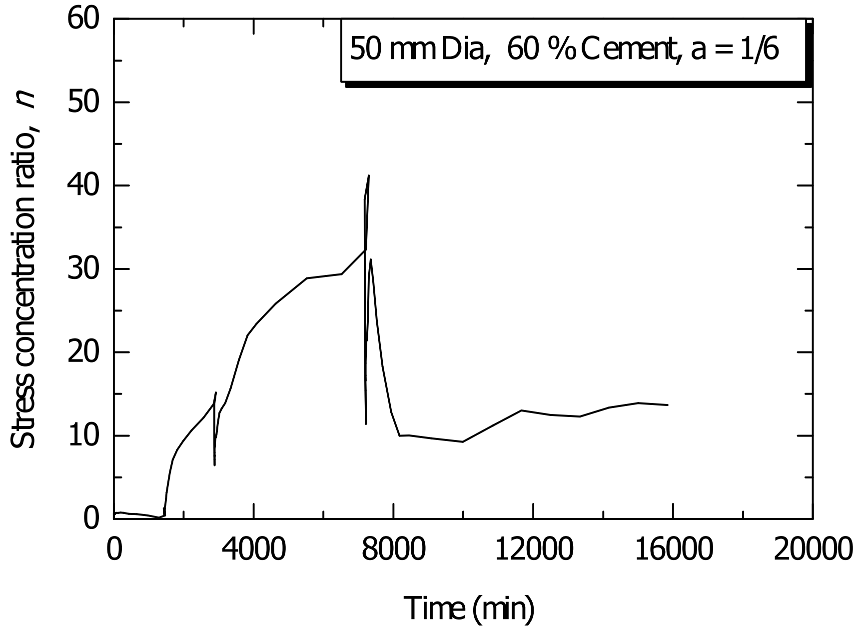
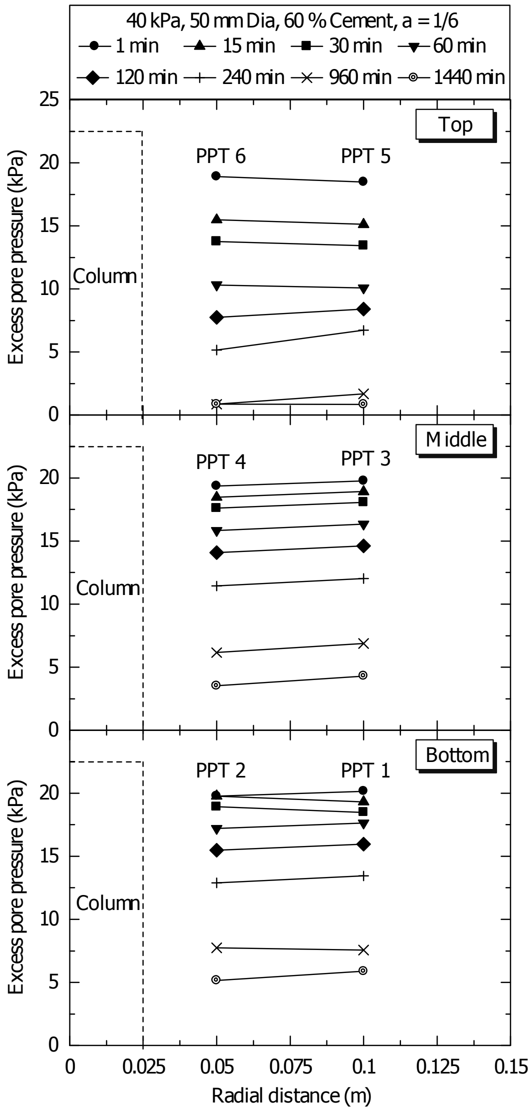
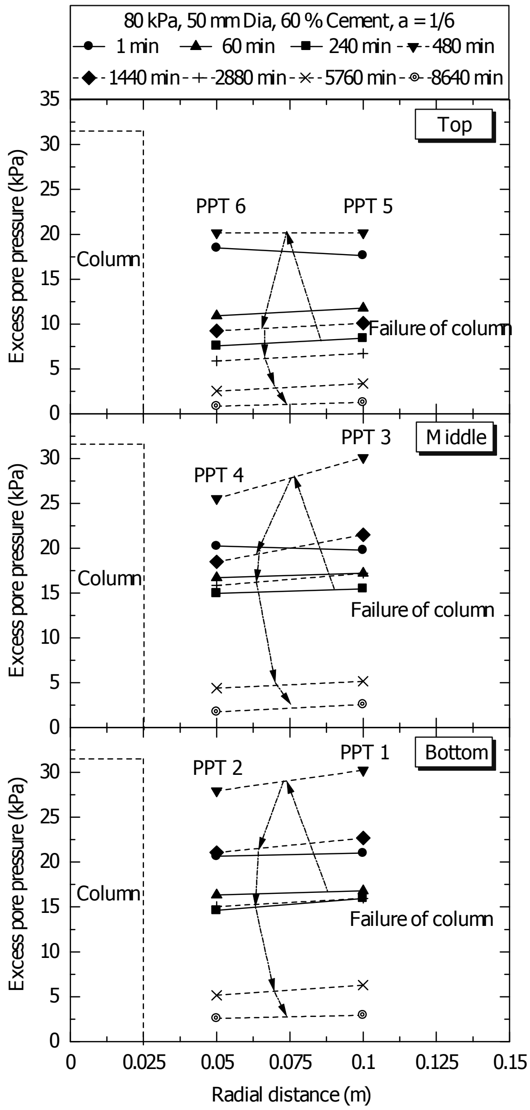


Figure 6: Relationship between stress concentration and time.


40 kPa, 50 mm Dia, 60 % Cement, a = 1/6
1 min
15 min
30 min
60 min
120 min
240 min
960 min
1440 min
Top
Middle
Bottom
PPT 6
PPT 5
PPT 4
PPT 3
PPT 2
PPT 1
Column
Excess pore pressure (kPa)
Radial distance (m)
0
0.025
0.05
0.075
0.1
0.125
0.15
0
5
10
15
20
25



(a) 40 kPa (b) 80 kPa

Figure 7: Radial distribution of excess pore pressure at different times under vertical stresses of 40 kPa and 80 kPa.

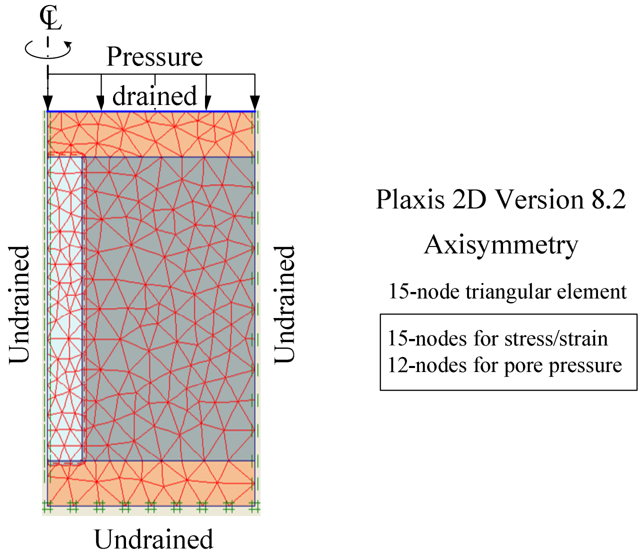


Figure 8: Finite element model for the model composite ground.

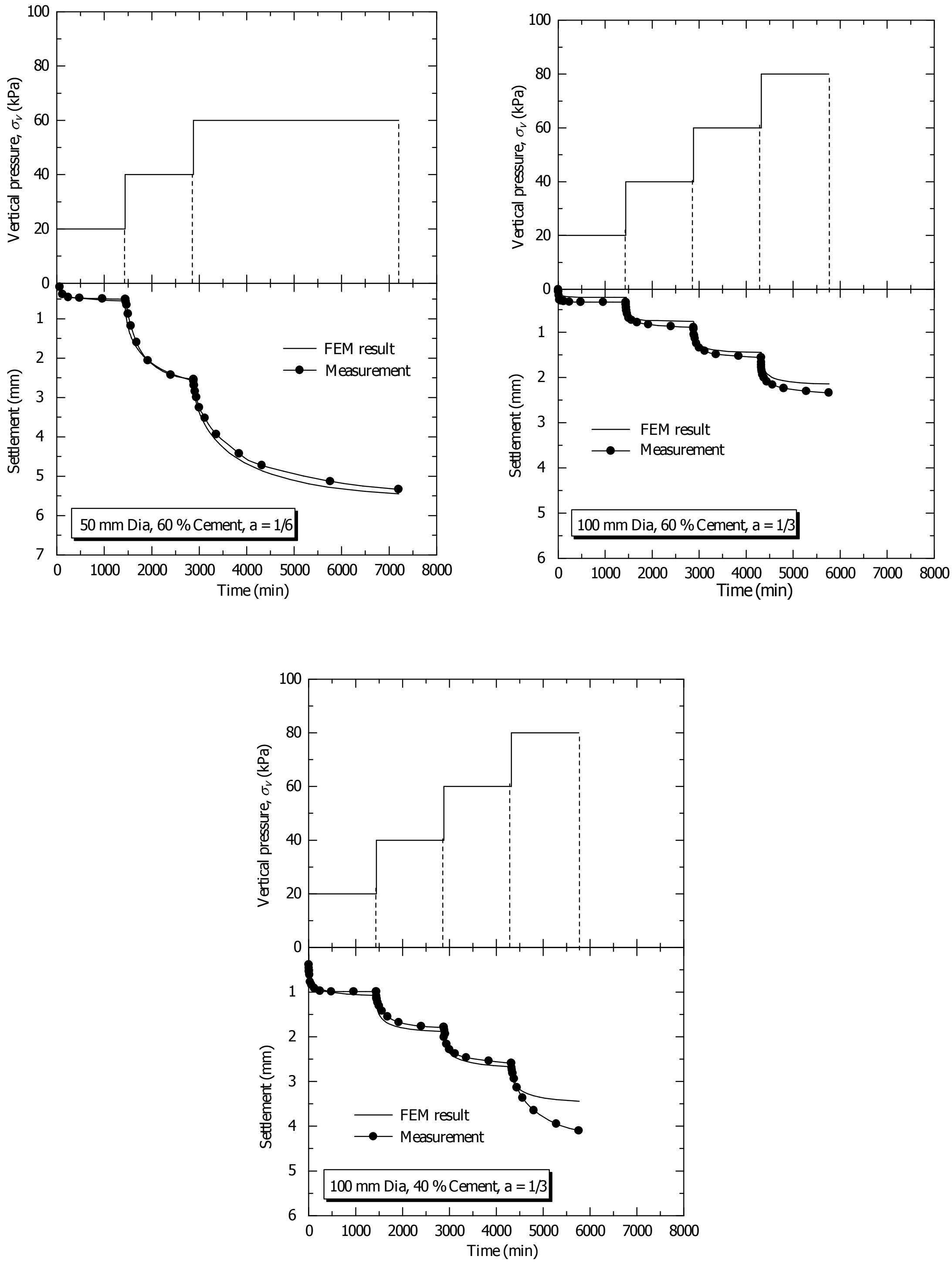


Figure 9: Comparison between simulated and measured settlement with time for different area ratios and cement contents.

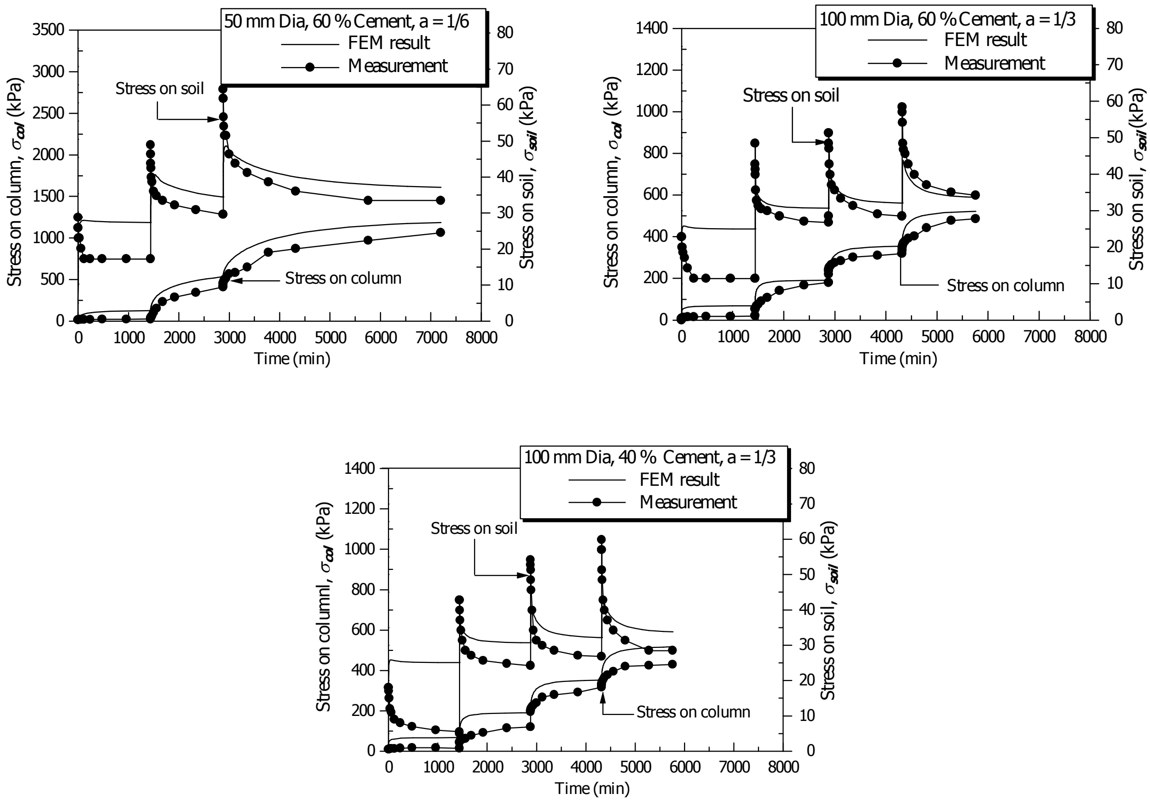


Figure 10: Comparison between simulated and measured stresses on column and surrounding clay with time for different area ratios and cement content.

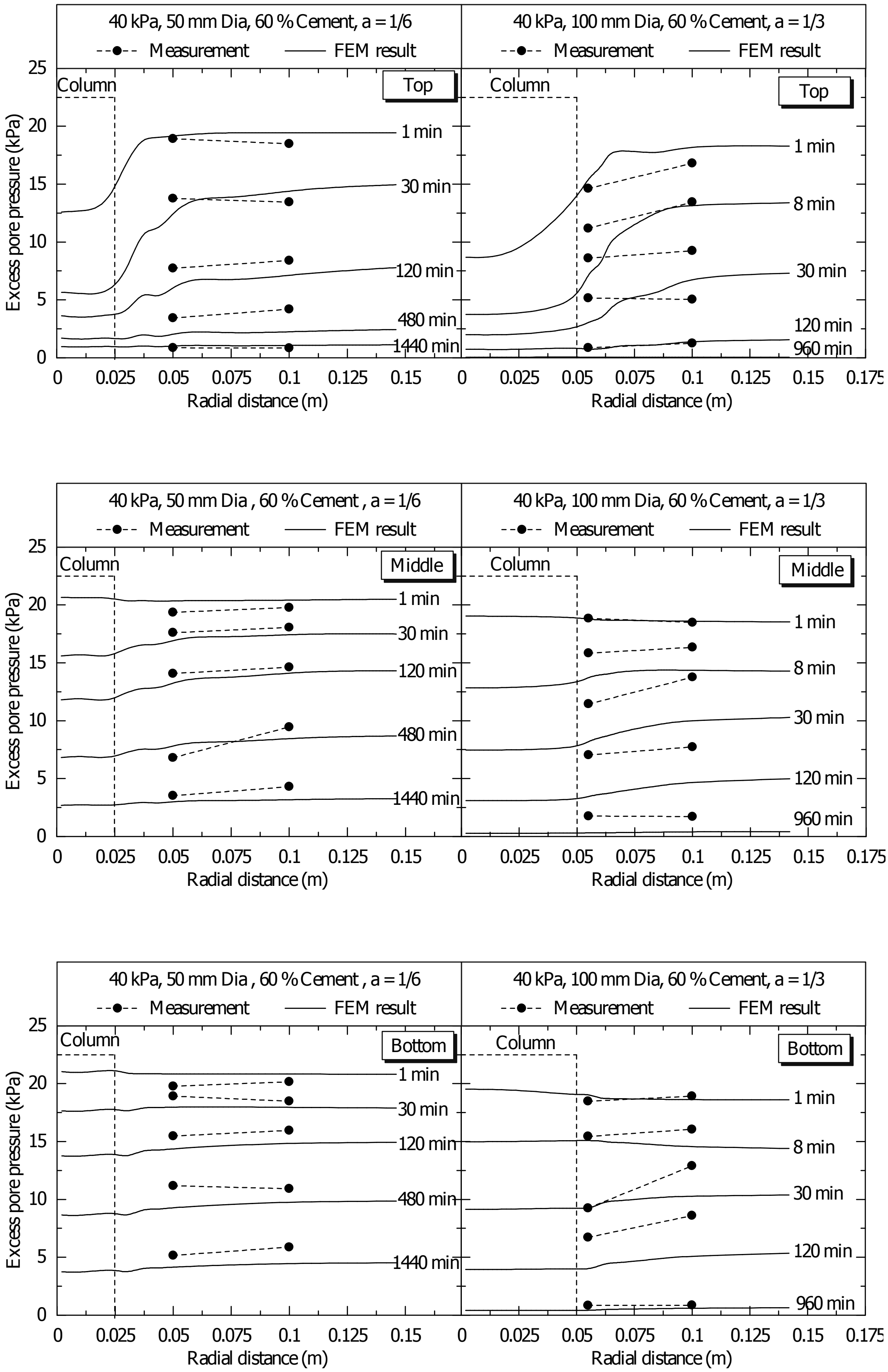


Figure 11: Relationship between simulated excess pore pressure and radial distance at different times and area ratios.

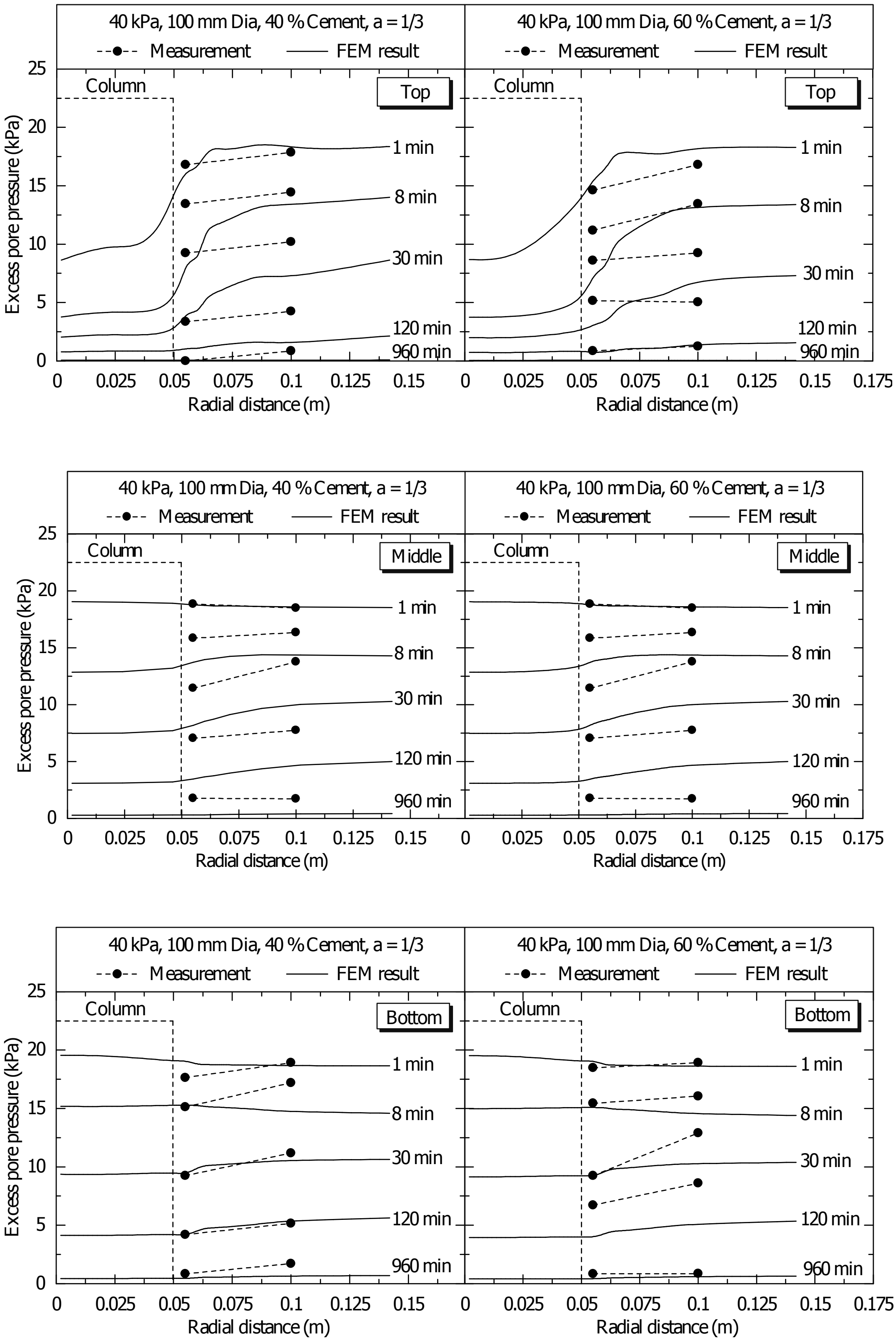


Figure 12: Relationship between simulated excess pore pressure and radial distance at different times and cement contents.

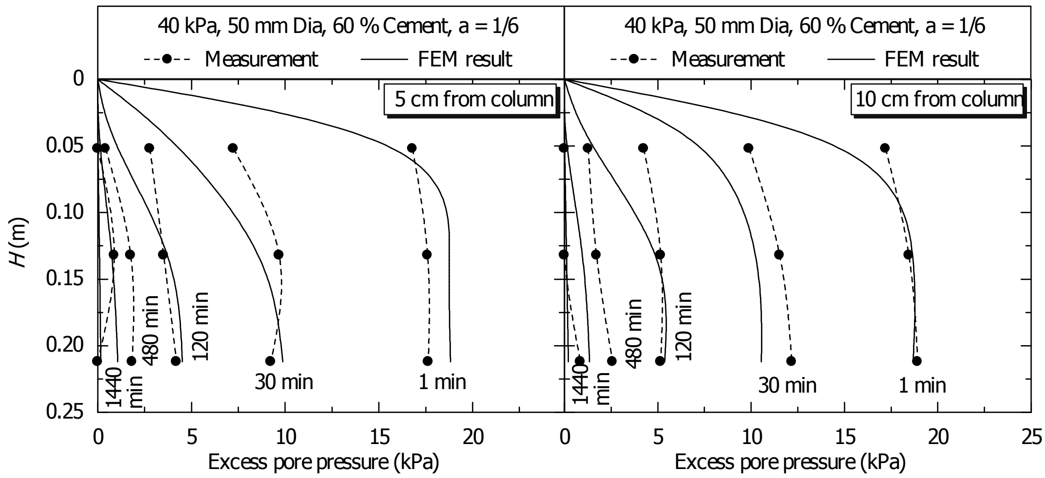


Figure 13: Relationship between excess pore pressure and depth at different consolidation times for a = 1/6 and q_u = 1200 kPa.

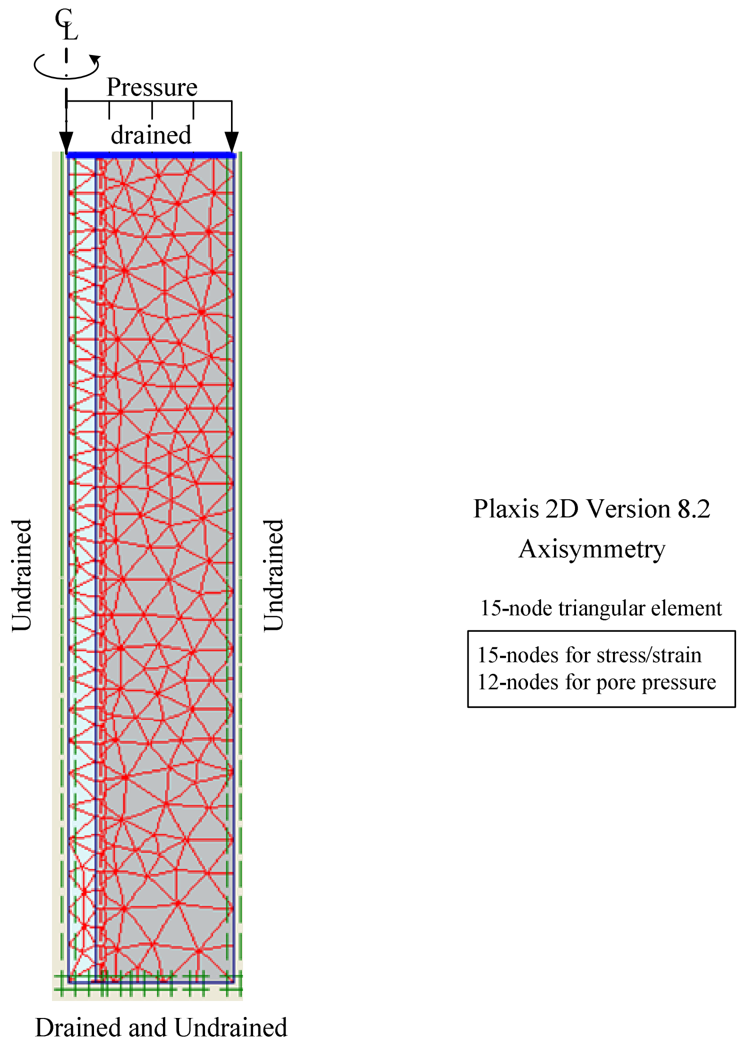
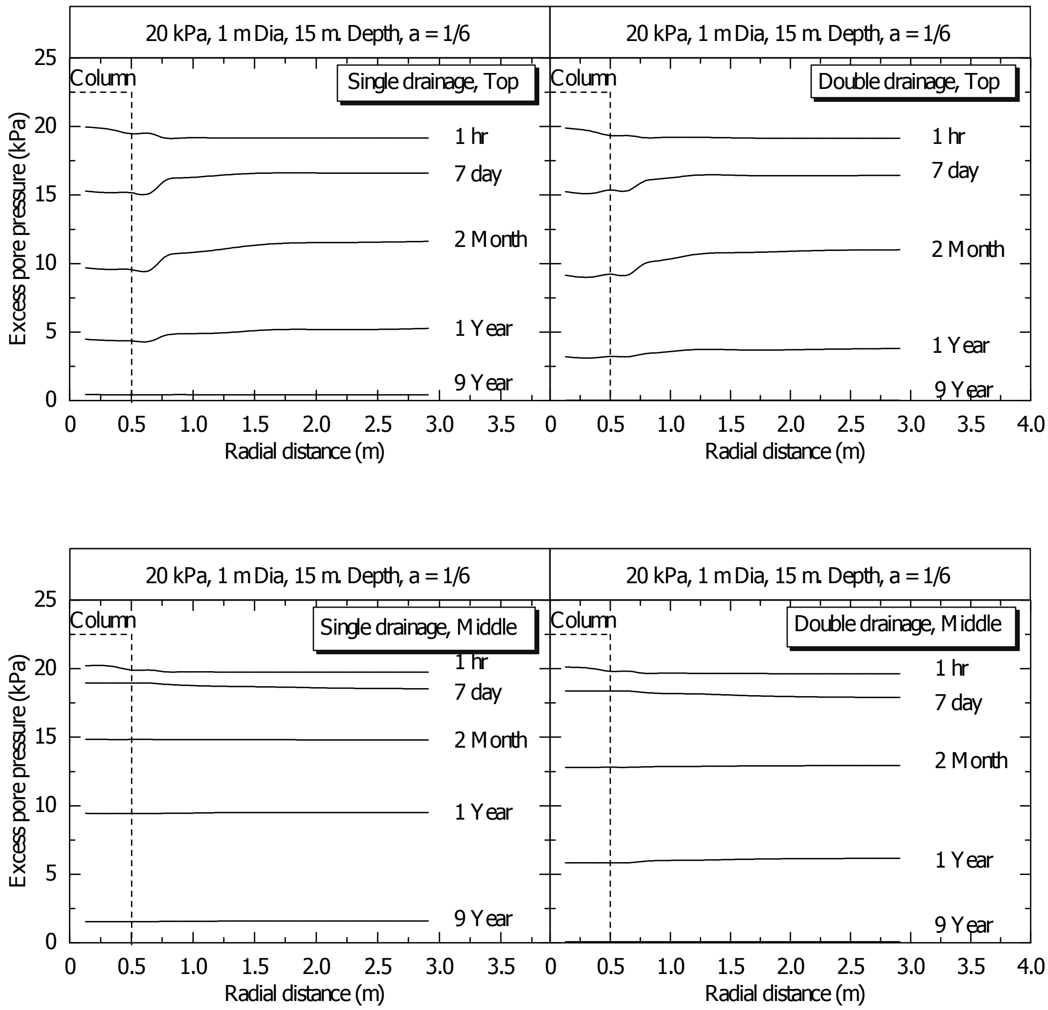


Figure 14: Finite element model for studying the effect of drainage condition.


20 kPa, 1 m Dia, 15 m. Depth, a = 1/6
20 kPa, 1 m Dia, 15 m. Depth, a = 1/6
Column
Single drainage, Top
Column
Double drainage, Top
1 hr
7 day
2 Month
1 Year
9 Year
1 hr
7 day
2 Month
1 Year
9 Year
Excess pore pressure (kPa)
Radial distance (m)
Radial distance (m)
20 kPa, 1 m Dia, 15 m. Depth, a = 1/6
20 kPa, 1 m Dia, 15 m. Depth, a = 1/6
Column
Single drainage, Middle
Column
Double drainage, Middle
1 hr
7 day
2 Month
1 Year
9 Year
1 hr
7 day
2 Month
1 Year
9 Year
Excess pore pressure (kPa)
Radial distance (m)
Radial distance (m)

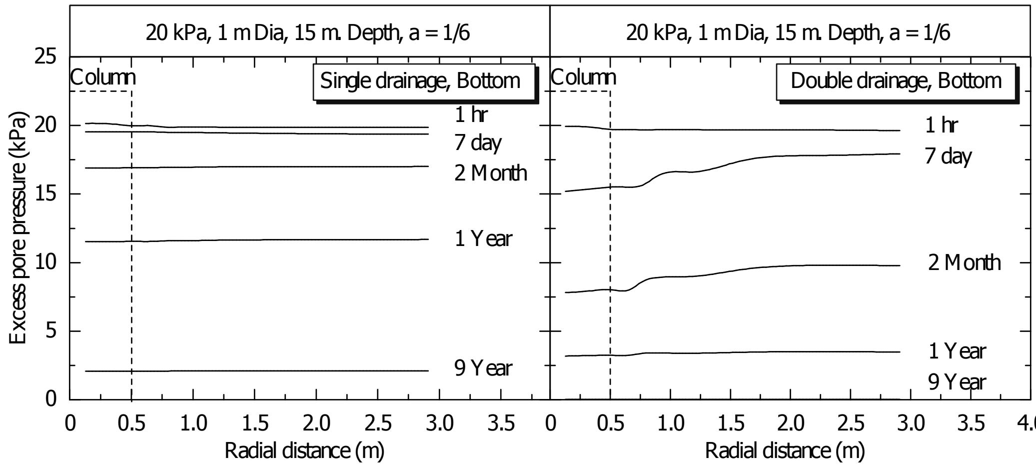
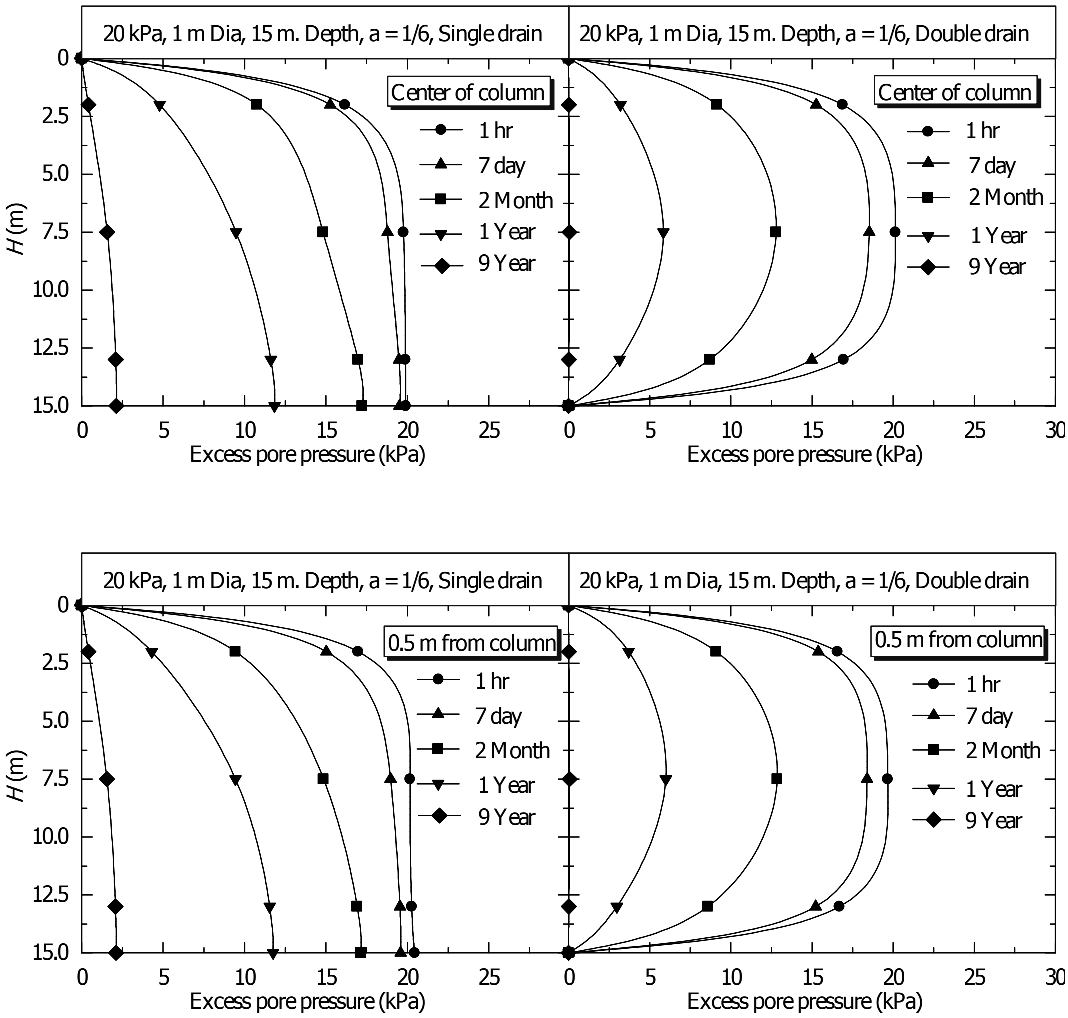


Figure 15: Change in excess pore pressure with radial distance for single and double drainage conditions.



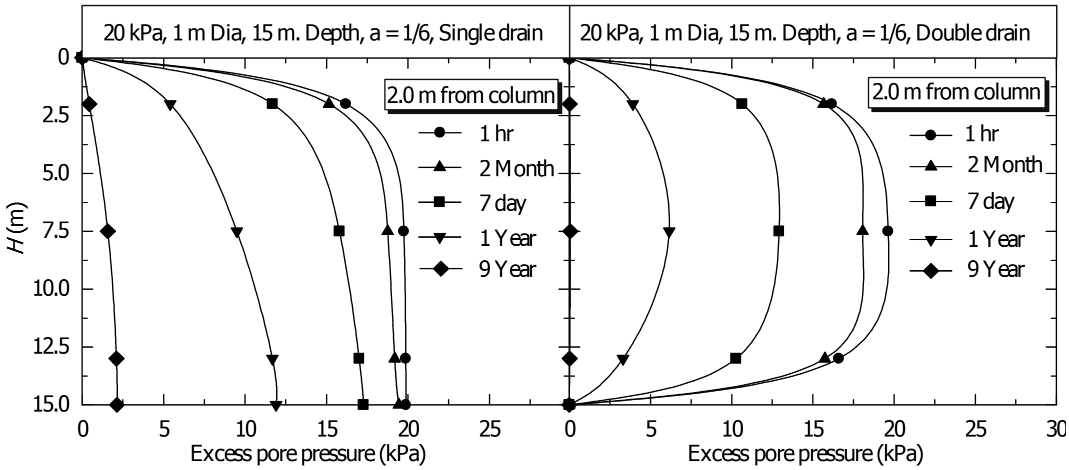


Figure 16: Change in excess pore pressure with time
for single and double drainage conditions.

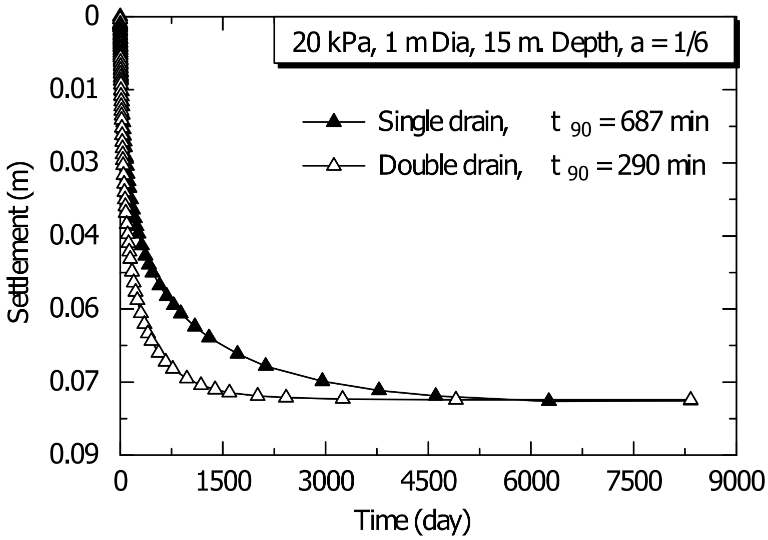


Figure 17: Relationship between settlement and consolidation time
for both single and double drainage conditions.

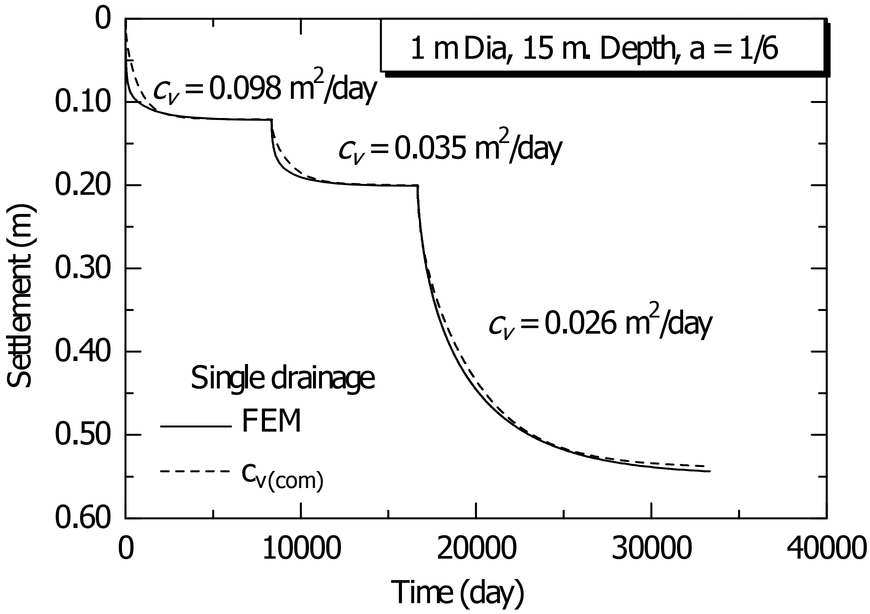


Figure 18: Settlement versus time relationship for single drainage condition.

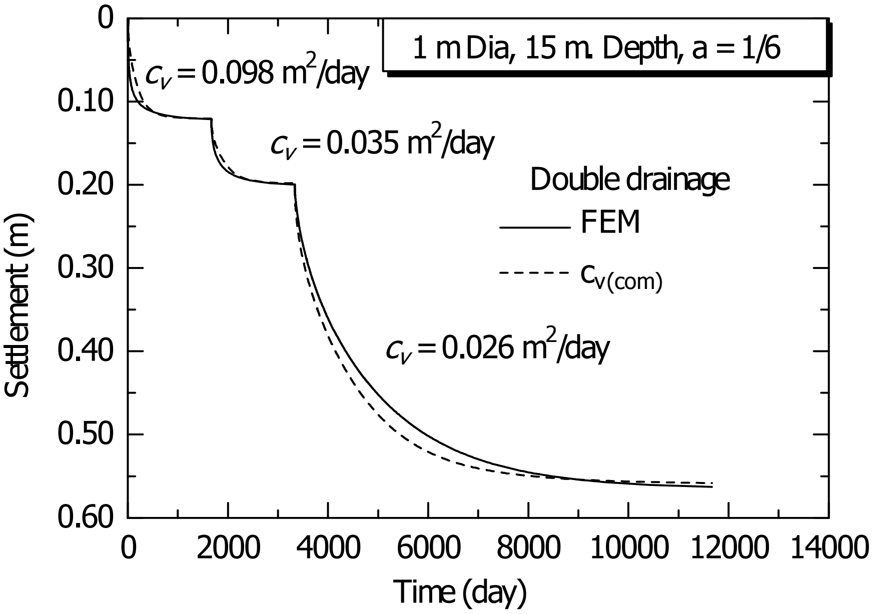


Figure 19: Settlement versus time relationship for double drainage condition.