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INVESTIGATING OF THE BEHAVIOR OF PILE GROUPS UNDER THE EFFECT OF SUCTION POTENTIAL OF UNSATURATED EXPANSIVE SOIL

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Abstract: This research presents an experimental study and numerical modelling to investigate the load-carrying capacity of a group of piles embedded in unsaturated expansive soil. The experimental work consists of a model of group of piles tested under unsaturated condition. The matric suction of the soil is measured by the Tensiometers to estimate the soil water characteristic curve (SWCC) by applying a fitting method. The experimental model is performed to validate the numerical modeling of the investigated problem using finite element method. A three-dimensional numerical modelling of a large-scale model is adopted to perform a parametric study on a group of piles in unsaturated expansive soil to investigate the effect of different parameters, such as depth of phreatic surface and initial degree of saturation on the load- carrying capacity. The results showed that a good agreement for the validation process. In addition, the load-carrying capacity increases when the depth of phreatic surface from the ground surface increases, also the decreasing in the initial degree of saturation of the soil lead to an increase in load-carrying capacity.

Keywords: *Finite element, Group of piles, Unsaturated soil, Expansive soil.*

التحري عن تصرًف مجموعة الركائز تحت تاثير الطاقة الماصَة للتربة الأنتفاخية غير المشبعة

الخلاصة: يقدم هذا البحث دراسة عملية ونمذجة عددية للتحري عن قابلية تحمل مجموعة الركائز المضمنة في التربة الأنتفاخية غير المشبعة. يتضمن الجزء العملي أعداد نموذج من مجموعة ركائز تم فحصها تحت حالة التربة غير المشبعة. أن طاقة الأمتصاص التي تحدث في التربة غير المشبعة تم قياسها بأستخدام جهاز قياس طاقة الأمتصاص (التينسوميتر) وذلك لتخمين منحني خصائص التربة-الماء. أن النموذج العملي أستخدم لتصديق النموذج العددي بأستعمال طريقة العناصر المحددة. تم تبني النموذج الثلاثي الأبعاد ذات المقياس الكبير لمحاكاة مجموعة ركائز في تربة أنتفاخية غير مشبعة لدراسة تأثير عدد من الخواص مثل عمق مستوى المياه الجوفية، ودرجة التشبع الأبتدائية للتربة. أظهرت النتائج حصول توافق جيد بين نتائج النموذج العملي والنمذجة العددية. بالأضافة الى ذلك، بينت الدراسة أن قابلية تحمل مجموعة الركائز تزداد عندما يزداد عمق المياه الجوفية من مستوى سطح التربة. وأن الأنخفاض في درجة التشبع الأبتدائية للتربة أدت الى حصول زيادة في قابلية تحمل مجموعة الركائز.

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1. Introduction

Expansive soil is that kind of problematic soil which shows a significant amount of volume changes upon wetting and drying. The amount of swell generally increases with the increase in soil's plasticity index [1].

The change of volume in expansive soil results from the minerals of clay when the soil undergo dehydration due to evaporation and hydration due to rainfall. The absorption of water in soil depends on two factors: first factor, is the soil structure (mircopores within soil peds and macrospores between the soil peds), the second factor, is the soil state (void ratio and degree of saturation) [2].

Soil is one of the porous materials that have the ability of holding water, this phenomenon is known as suction, where the free energy of water in the soil is defined as suction. The suction is composed of two compounds osmotic suction and matric suction [3]. The matric suction is defined as the variation of pore air and pore water pressure (u_a-u_w), and is occurred as a result of rising water in the pores of the soil (capillary properties), while osmatic suction occurs as a result of the effect of dissolve salts in soil water [4].

2. Experimental Work

2.1. *Soil Used*

The soil used in this study is an expansive soil composed of clay soil and bentonite, where the percentage of bentonite is 30% of the total weight of the soil. The experimental tests were performed to determine the physical properties of the soil used shown in Table 1.

Table 1: Physical properties of the expansive soil used.

Property	Value
Specific gravity (Gs)	2.86
Liquid limit (L.L), %	65.4
Plastic limit (P.L), %	27.28
Permeability (k), cm/sec	1×10^{-7}

The grain size distribution of the soil (sieve analysis and hydrometer test) is performed according to the ASTM D-422-00 Specification. The particle size distribution curve of the expansive soil used is shown in Figure 1.

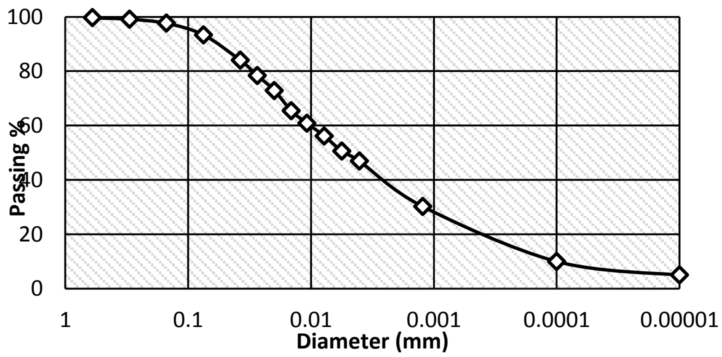


Figure 1: Grain size distribution of the expansive soil used.

2.2. *The Measurement of Matric Suction of the Soil*

In this study, the direct method is used to measure the matric suction of the expansive soil using the Tensometer shown in Figure 2.

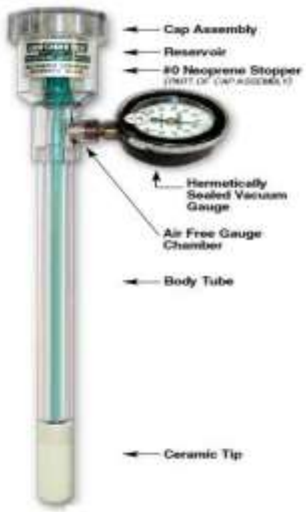


Figure 2: Tensometer

In this method, a small ceramic cup of the Tensometer is attached to a narrow very stiff plastic tube where the distilled water or the Tensiometer solution in the tube is connected to a pressure measuring device (vacuum gauge). The ceramic tip of the Tensometer is saturated by filling the tube with distilled water and then allowing it to drain through the Tensiometer tip for 24 hours to ensure a full saturated condition in the ceramic tip is occurred. Then, applying the vacuum to reduce the water pressure by drying up the cup and removing any air bubbles that may be trapped in the high entry ceramic tip. Any entrapped air within the cavitations of the filter element will be forced to dissolve in the water under the application of the high pressure. Thus increasing the accuracy of the measurable suction by the system.

After that the Tensiometer is transported to the soil with covering the tip with wet towel to prevent drying after filling the Tensometer with its solution. The Tensiometer is inserted using the coring unit to provide the required diameter. Intimate contact between the soil and the Tensiometer should be provided to prevent accumulation of water at the base through cavities between the soil and the sides of the Tensiometer. The hand vacuum pump supplied with the Tensiometer is used to apply a vacuum pressure between (80-85 kPa) to release the air entrapped in the ceramic disc and the tip is submerged in the soil.

The measurement of matric suction of the expansive soil is conducted by using two Tensometers as shown in Plate 1. The first Tensiometer was placed at 100 mm below the soil surface, while the second Tensiometer was installed at 300 mm below the soil surface. The water table raised in the container from the bottom to a distance of 60 mm in the soil, where the height of water (60 mm above the bottom of soil) is monitored by piezometers. Then, the model is left for a period of 23 hours to let the capillary suction occurs in the soil (i.e., the soil in unsaturated condition). After that the matric suction is measured.

Plate 1: Measuring the matric suction of the soil by using Tensiometers.

The suction profile set is shown in Figure 3. The results obtained from suction profile set (i.e., variation of matric suction above the water table) are used to determine the soil-water characteristic curve of the soil to achieve a pre-decided matric suction profile of the soil below the model of foundation prior to conducting bearing capacity tests. The variation of matric suction with depth is shown in Table 2

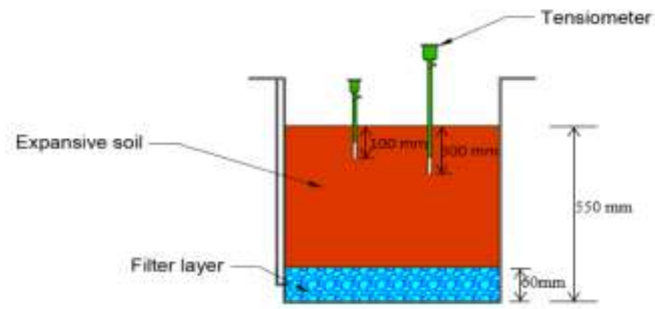


Figure 3: Suction profile set

Table 2: The variation of matric suction.

Depth of Tensiometer (mm)	Water content%	Matric suction (kPa)
100	6	80
200	8	74*
300	10	68

*This value of matric suction is determined by interpolation..

2.3. *Swelling Pressure of the Expansive Soil*

The constant volume method is used to measure the swelling pressure of the expansive soil. In this method, the specimen volume is maintained constant throughout the test by varying the load on the specimen as required. The final equilibrium pressure (the pressure applied to maintain the heave of soil equal to zero) after 48 hours from the start of testing is the swelling pressure. Figure (4) shows the results of constant volume method for the expansive soil.

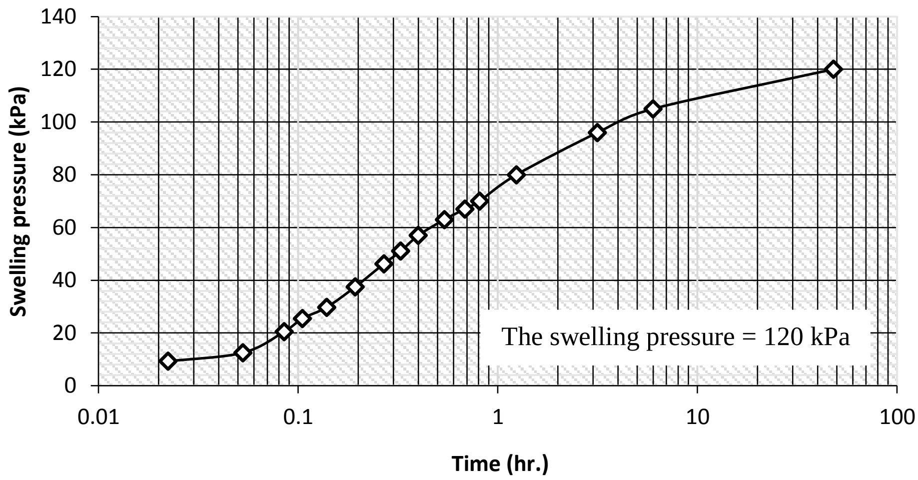


Figure 4: The swelling pressure test of the expansive soil.

2.4 *The Model of Pile Groups*

The model of pile groups (2×2) used in this study is made of reinforced concrete. The diameter of the pile (D) is 20 mm and the length of the pile (L) is 400 mm. The embedment ratio L/D is 20. However, the spacing (S) between piles is kept constant at 60 mm c/c (S= 3D) as shown in Figure 5.

The pile cap model was also made from reinforced concrete with a thickness of 25 mm, and the reinforcement of the cap is placed in two directions. The diameter of each bar is 2 mm, and the yield strength of the reinforcement (fy) is 175 MPa. The spacing between the reinforcement c/c is 24 mm. In addition, the end of each bar is twisted in the vertical direction with length of 15 mm as shown in Plate 2.

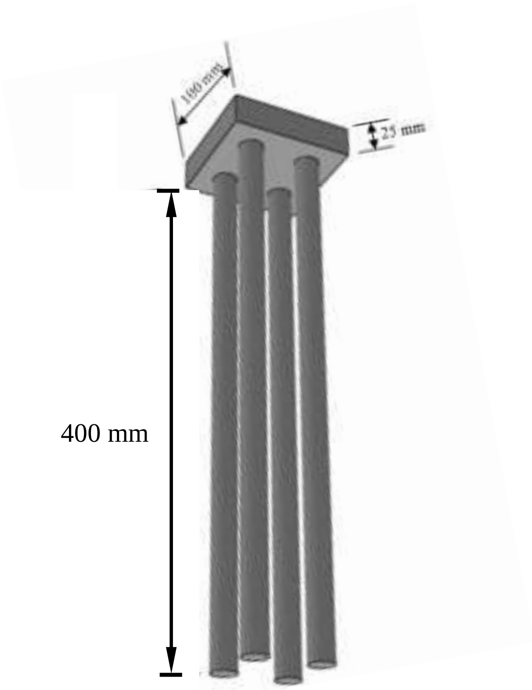


Figure 5: The model of pile groups.

Plate 2: Reinforcement of the pile groups.

The concrete material consists of gravel passing through sieve No.6 (3.35 mm). The water-cement ratio (w/c) is 0.35 by weight. An admixture known as Structuro 520 (Superplasticisers) is used in the mixture of concrete with amount of 5 liter/m^3. This allows to produce a high-performance concrete and concrete with high workability. Table 3 shows the details of concrete mixture used for casting the model of pile groups by many attempts.

Table 3: The concrete mixture.

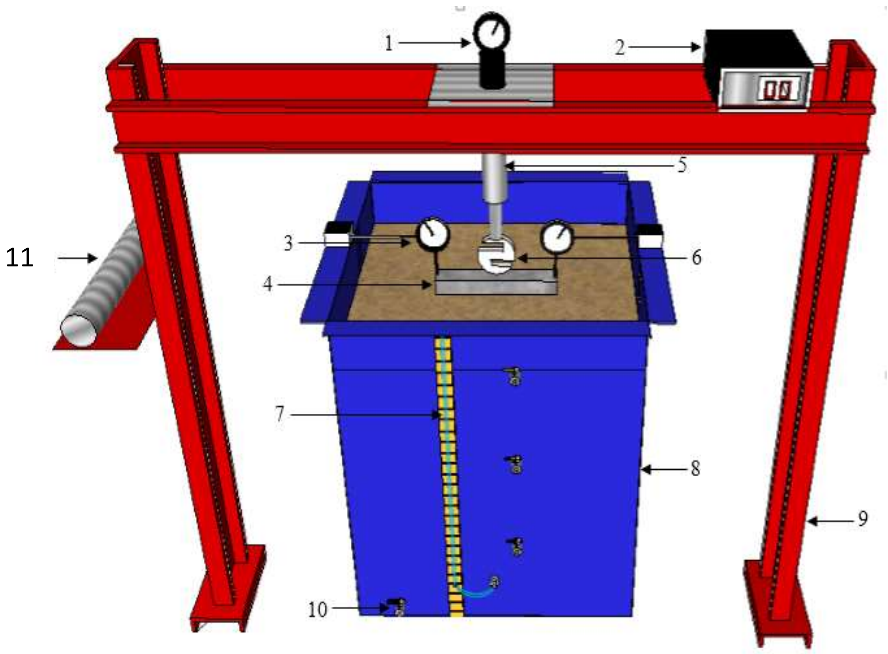
Cement Kg/m^3	Aggregate		Water Kg/m^3	Admixture L/m^3	Slump 150±5mm	W/C by weight
	Sand Kg/m^3	Gravel Kg/m^3				
500	725	875	175	5	155	0.35

3. Test Apparatus

The model test was conducted using the setup shown in Figure 6, which is consist of steel frame, steel tank, model of group of piles. The vertical load is applied on the model of the foundation by means of hydraulic compression manual jack with capacity of 10 ton. The loading rate is kept constant during the test. The applied load is measured by using a load cell with a capacity of 0.5 ton. A digital weighing indicator is used to read the value of the applied load. Two dial gauges with 0.01 mm sensitivity are used for measuring the displacement of the group of piles model.

3.1. *Soil Container*

In this study, a soil container is used with dimensions of 600, 600 and 700 mm of length, width and height, respectively. It contains a one valve at the base of the container to allow the flow of water into the model. Also, a piezometer is placed at each side of the container to monitor the height of the water table in the soil.



1- Pressure gauge
2- Digital Weighing Indicator.
3- Dial gauge.
4- Pile groups
5- Axial loading system.
6- Load cell
7- Piezometer.
8- Soil container.
9- Steel frame.
10- Drainage valve.
11- Hydraulic Jack.

Figure 6: Set up of the experimental model.

3.2. *Soil Layer*

The height of soil in the container is 550 mm to satisfy that the bulb of pressure does not reach the boundary of the container. The soil height is divided into 5 layers, each layer is 110 mm height except the first layer at the base of the model is divided into 50 mm soil and 60 mm filter material to prevent the erosion of soil particles. A layer of mesh made of geosynthic material is placed above the filter layer as shown in Plate 3.

3.3 *The Process of Pile Installation*

After carrying out the preparation of the soil bed, the installation of piles in the soil is done by making four borehole with diameter of 20 mm and length of 400 mm for each one.

The borehole are drilled using a small hand auger manufactured for this purpose. In addition, to ensure the verticality of the hole, a special cover with a steel neck is welded vertically at the top center of the cover as shown in Plate 4 to make the auger penetrates in the bed of soil vertically. Finally, the pile groups are carefully inserted into the holes to prevent any soil disturbance.

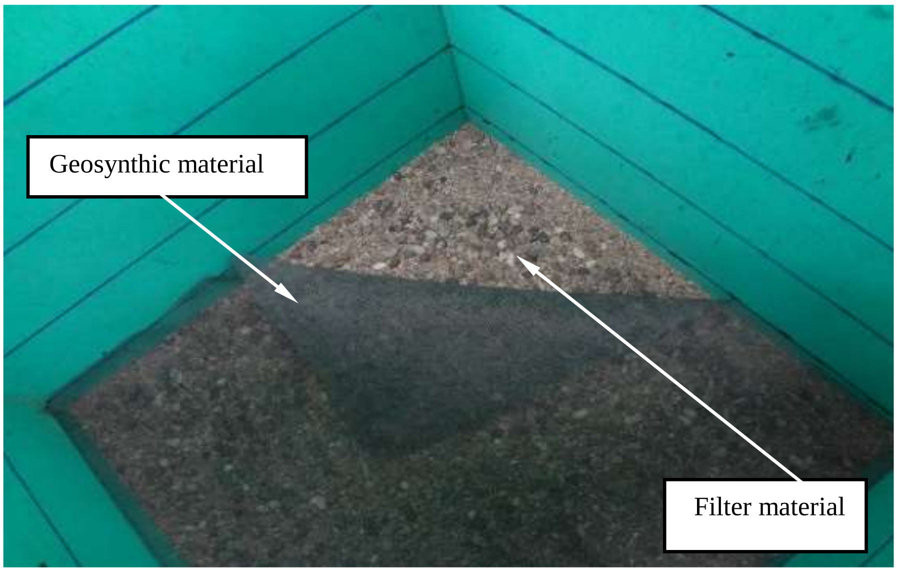


Plate 3: Filter material for the soil model.

Plate 4: The neck cover.

4. Experimental Model Test

The experimental model test was conducted using the setup shown in Plate 5. The test was conducted on group of piles embedded in soil compacted at dry density and initial moisture content of 18 kN/m^3 and 12.5%, respectively. The flow of water table is allowed by the upward flow of water through the filter layer using a water tank. The elapsed time for the flow of water through the model of soil was three hours, and then the model of the foundation is tested after 23 hours.

In this test, two dial gages are used one on each side of the deep foundation. The load cell is placed above the system of the model of foundation directly and join it by digital weight indicator to record the magnitude of the applied load. The load is applied incrementally until the failure is occurred.

For each interval of loading, the load is kept constant for a period of time 8 minutes according to ASTM D-1143 which is specified that the interval for the load increment between 4-15 minutes. The failure occurs when the value of load is kept constant and the dial gauge is still reading the settlement of the foundation.

Plate 5: The experimental model test.

5. Soil Water Characteristic Curve (SWCC) of the Expansive Soil

The soil water characteristic curve (the relationship between the gravitation water content and the matric suction) is predicted through applying fitting methods [5]. In this study, the method proposed by Fredlund and Xing which is implemented in SoilVision software is used [6]. The soil water characteristic curve of the expansive soil is shown in Figure 7.

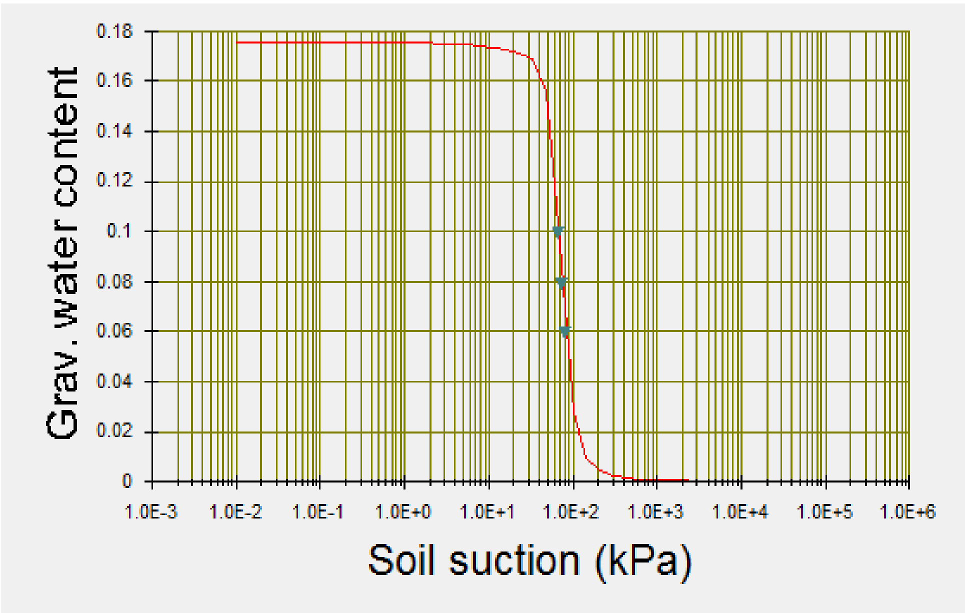


Figure 7: Soil water characteristic curve of the expansive soil (fitting method-SoilVision software).

6. Numerical Modelling of Pile Groups Embedded in Unsaturation Expansive soil

A numerical modelling based on the finite element method through Abaqus program is utilized to analyze the present model test. A three-dimensional numerical modelling of the experimental model is performed for this purpose.

6.1. *Geometric Modeling*

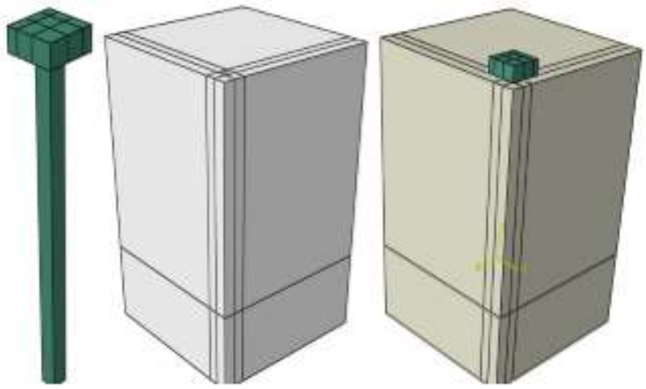
The circular pile is most commonly used in structural construction. On the other hand, the numerical simulation can be best performed by considering the octagonal or square cross-sectional shape of the pile.

The simplification of circular pile to square one leads to considerable savings in the number of finite elements used to generate the mesh and thus, in the calculation time period.

The hexahedrons are the best choice for the three-dimensional analysis, so to accommodate these elements for a fast, less expensive and more accurate simulation, the consideration of square pile is decisive. The same peripheral surface area must be taken into account, because for this type of foundation, the frictional resistance is the main concern rather than the tip resistance.

The pile group in this study has a square arrangement; therefore, the finite element mesh of the pile group and the surrounding soil can take advantage of this symmetric condition. Only one-fourth of the geometry is considered as shown in Figure 8 for simulation purpose.

The soil is considered as an isotropic homogenous single phase medium. To limit the computation cost of the numerical simulation, it is essential to set the suitable boundary extent and the boundary condition of the selected continuum.



(a) Pile part (b) soil part (c) soil-pile assembly

Figure 8: Three-dimensional axisymmetric Parts of the numerical model.

Figure 9 depict the boundary conditions to idealize the numerical simulation. In all cases no x, y and z translations at the bottom nodes are allowed. Because of the symmetry, one fourth of the model is simulated, setting the boundary condition along the mid axis as XSIMM (symmetry about a plane x is constant) and YSIMM (symmetry about a plane y is constant).

No x-translation is allowed in y-z plane and no y-translation is allowed in x-z plane. The rotational degree of freedom Rx and Rz in x-z plane are not allowed and the same condition is considered for Ry and Rz in y-z plane.

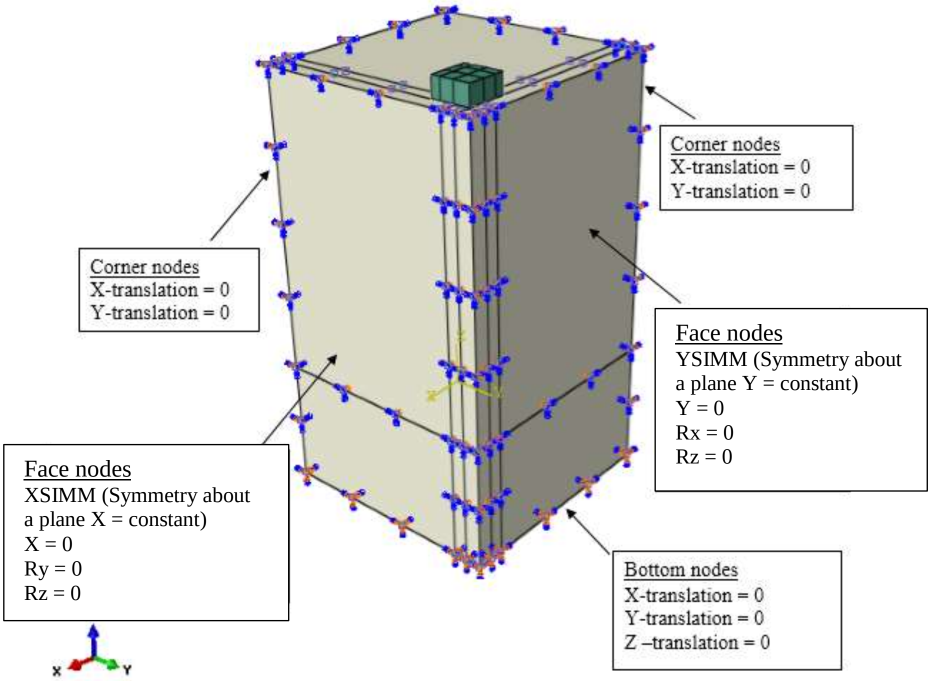


Figure 9: The boundary conditions of the problem.

6.2 *Material Properties*

The values obtained from the soil water characteristic curve which are the degree of saturation from the gravitational water content and the values encountered of the matric suction of the soil is implemented in the numerical analysis as part of the properties of the unsaturated soil. The properties of materials used in numerical analysis are shown in Table 3.

Table 3: The properties of the material used

Material	γ_d kN/m^3	Modulus of elasticity kPa	Poisson's ratio, υ (assumed)	e_o	Cohesion, C kPa	Angle of internal friction ϕ °	Coefficient of permeability cm/sec
Expansive soil	18	10000	0.3	0.5	133	14	1×10^{-7}
Concrete	24	25743000	0.25	-	-	-	-

6.3 *Interaction Model*

Interaction models are used to manage mechanical interactions between regions of the model and the surrounding material. In this study, the contact zones of the pile groups are simulated using surface-to-surface contact which is used to describe the contact between two deformable surfaces or between a deformable surface and a solid surface. Two surfaces are specified namely master and slave surface.

6.4 Type of the Element Used

The element used to simulate the soil is 3D quadrilateral element named C3D20RP. Which means 20-node brick with pore pressure, quadratic displacement, linear pore pressure and reduced integration. While the pile foundation system is modelled by the C3D20R element which is also a 20-node quadratic brick, reduced integration. The discretization of the numerical model is shown in Figure 10.

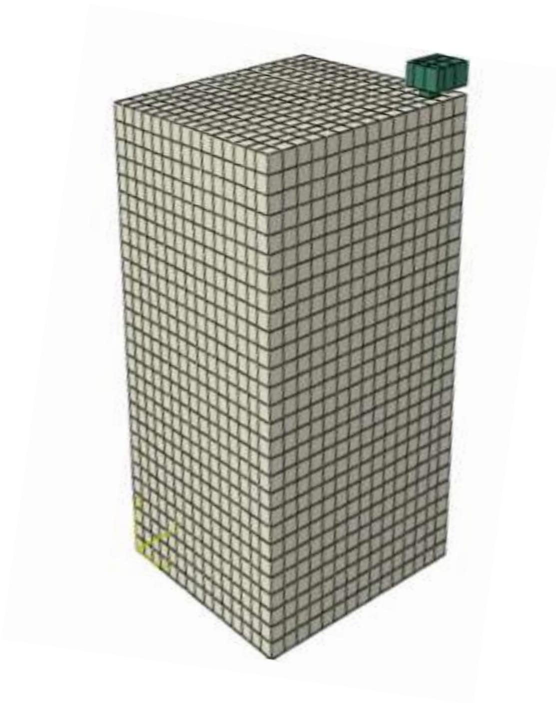


Figure 10: Discretization of the numerical model

7. Validation of the Numerical Modelling

In the numerical analysis, the load is applied incrementally with a pattern similar to that applied in the experimental model.

The pile group capacity (= pile load × 4 because of symmetry) versus settlement curve obtained from the finite element analysis for a node located at the top surface of the pile foundation which is the same position of the of the dial gage in the experimental model and load-settlement curve for the group of piles that obtained from the experimental model test are shown in Figure 11. From this figure, it can be seen that, the ultimate load carrying-capacity determined from the experimental model test of the pile groups embedded in unsaturated expansive soil is 6 kN while the numerical analysis using finite element method gives a value of 6.4 kN.

Figure 12 illustrates the distribution of negative pore water pressure along the soil depth at the end of analysis. The negative pore water pressure is gradually decrease with depth of soil, starting with maximum negative value at the top surface to a minimum value at the bottom of soil.

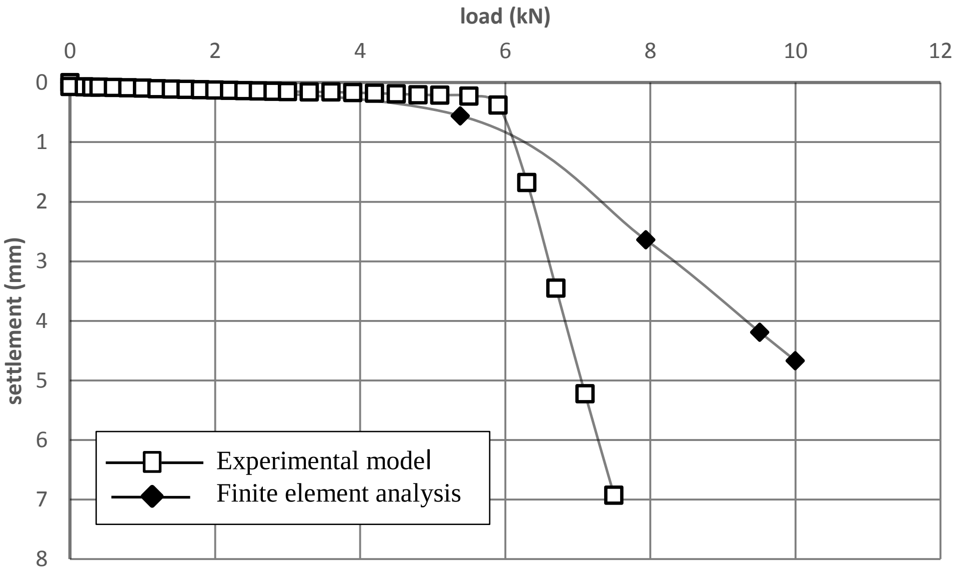


Figure 11: Load-settlement curves of group of piles from experimental model and finite element analysis

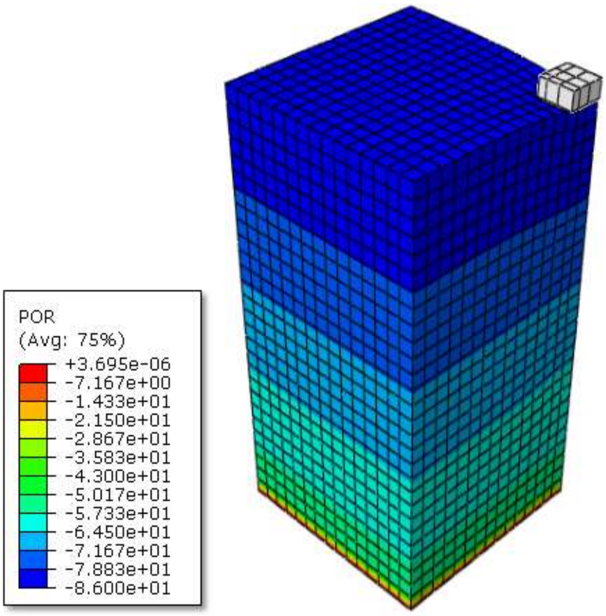


Figure 12: The distribution of negative pore water pressure at the end of Analysis.

8. Parametric Study on a Large Scale Model of Group of Piles Embedded in Unsaturated Expansive Soil.

In this section, a parametric study on a large scale model of group of piles embedded in unsaturated expansive soil is performed to study the effects of some parameters on the load carrying capacity. The geometry of the problem is adopted from Helwany [8]. The pile group has a square arrangement; therefore, the finite element mesh of the pile group and the surrounding soil can be taken the advantage of the symmetric condition, where only one-fourth of the geometry was considered. The simplifying of the three-dimensional of the soil profile with group of piles is shown in Figure 13, where the advantage of symmetry about two orthogonal planes is considered.

The simplifying three-dimensional finite element modelling comprises two parts: the concrete pile and the soil. The interface elements between the two parts are used. The dimensions of the simplifying model is 15 m long (in the *x*-direction), 15 m wide (in the *y*-direction), and 15.15 m high (in the *z*-direction). The concrete pile behavior is modeled as linear elastic. A Young's modulus of 25743000 kPa and a Poisson's ratio of 0.25 are used in the analysis. However, the properties of expansive soil in unsaturated condition are given in Table 3. The three-dimensional finite element mesh is shown in Figure 14.

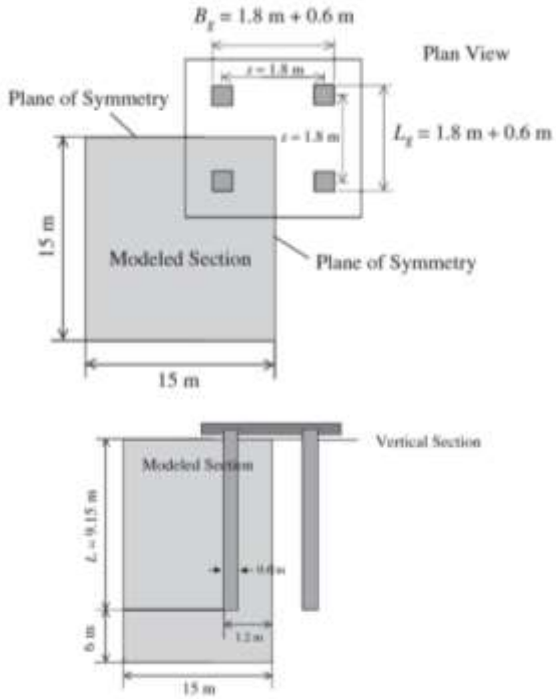


Figure 13: Simplifying of the of the three-dimensional pile groups.

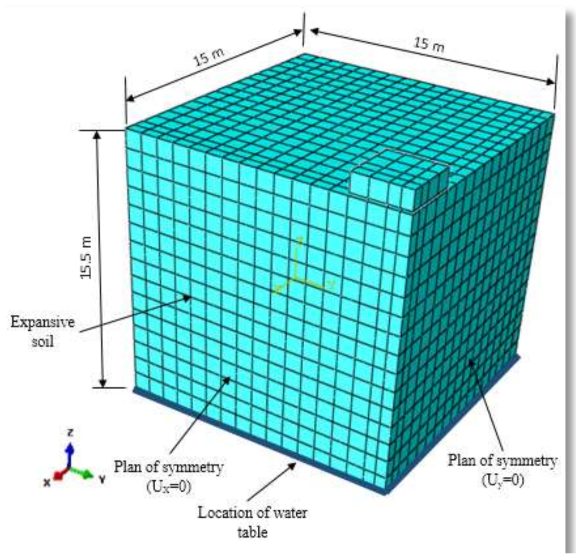
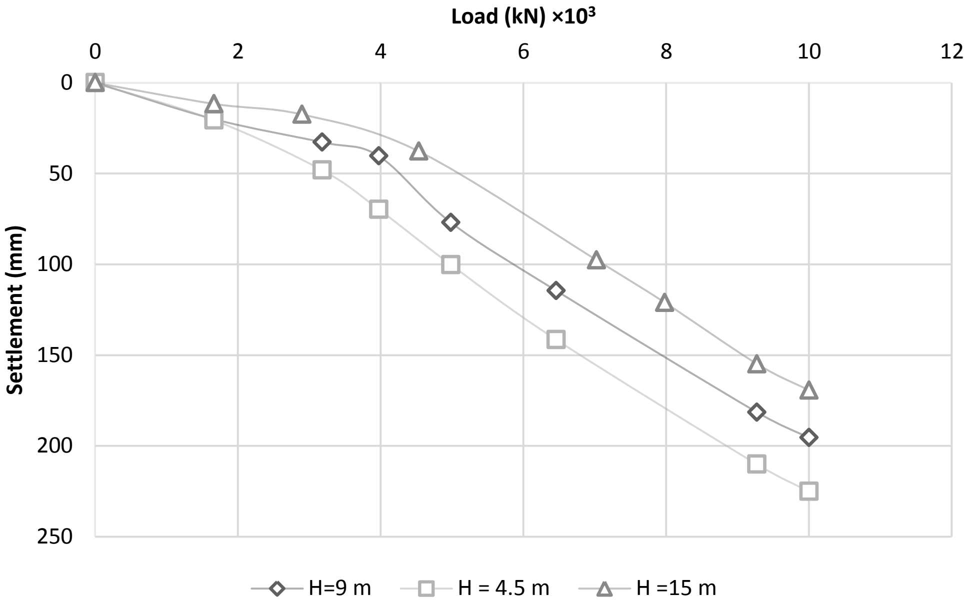


Figure 14: The finite element discretization of the simplified large-scale model.

8.1. *Effect of Location of Phreatic Surface on Load-Carrying Capacity of Pile Groups Embedded in Expansive Soil*.

Figure 15 shows the load-settlement curve for pile groups embedded in an expansive soil with different location of phreatic surface from the soil surface (H), the results are summarized in Table 4.



Figures 15: The load-settlement curve for pile groups embedded in an expansive soil with different location of phreatic surface for one quarter of the foundation.

Table 4: Results of load-carrying capacity of pile groups with different locations of the phreatic surface.

Depth of phreatic surface (m)	Load-carrying capacity (kN)×10^3
15	4.4
9	3.6
4.5	3

From the results shown in Table 4 revealed that the load carrying capacity of the group of piles in unsaturated expansive soil increased with increasing the depth of phreatic surface from the soil surface.

8.2. Effect of Initial Degree of Saturation on Load-carrying Capacity of Pile Groups Embedded in Expansive Soil.

In this section the analysis is performed on four models of pile groups embedded in expansive soil with different values of the initial degree of saturation.

The load-carrying capacity of the group of piles with different values of initial degree of saturation are shown in Figure 16 and Table 5.

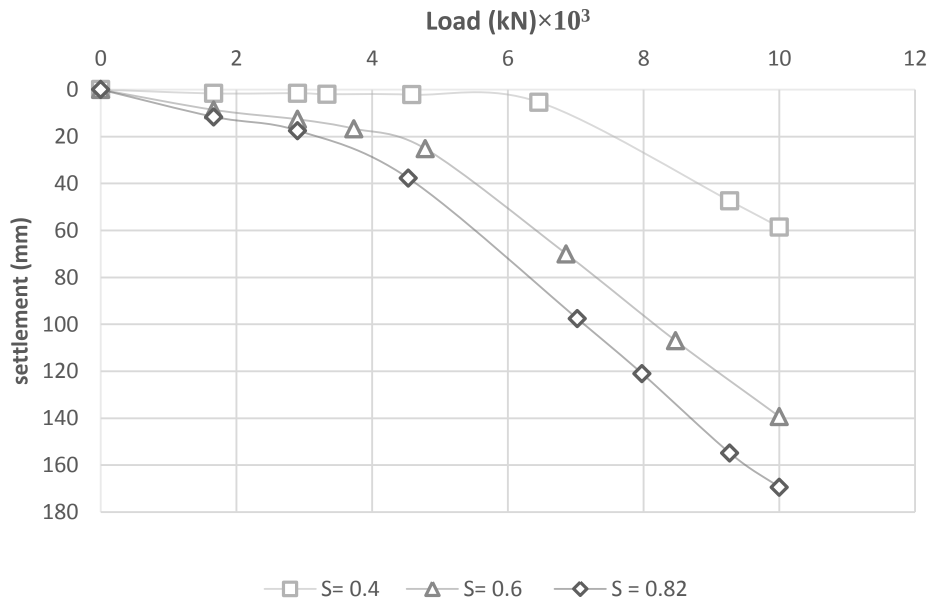


Figure 16: load-settlement curve of pile groups in unsaturated expansive soil with different values of initial degree of saturation for one quarter of the foundation.

Table 5: Results of load-carrying capacity of pile groups under different initial degree of saturation.

Initial degree of saturation (%)	Load-carrying capacity (kN)×10^3
82	4.4
60	4.8
40	6.4

The relationship between load-carrying capacity of the group of piles and the initial degree of saturation is shown in Figure 17.

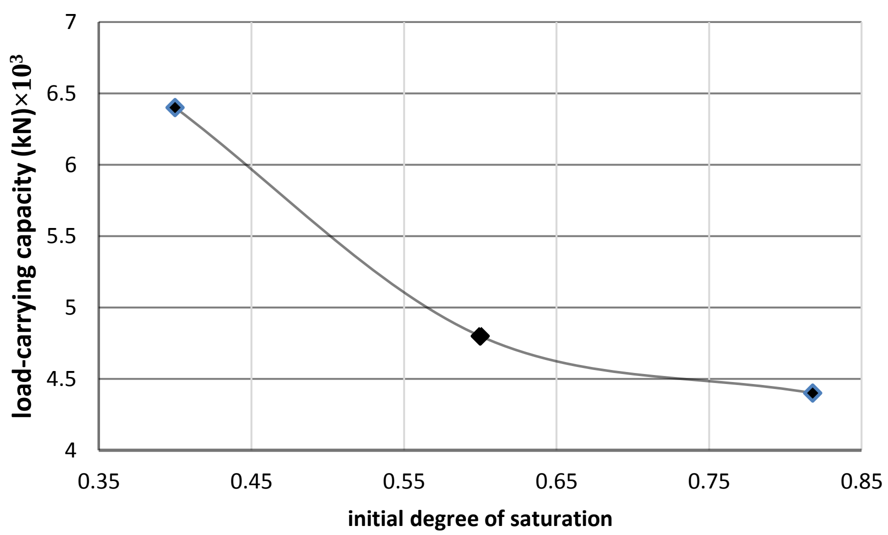


Figure 17: The relationship between load-carrying capacity and initial degree of saturation for the group of piles.

The results shown in Figure 17 show that an increase in the load-carrying capacity with decreasing the initial degree of saturation of the expansive soil. The decrease in the initial degree of saturation leads to an increase in negative pore water pressure (matric suction). These results are due to the contribution of matric suction, which leads to an increase in the shear strength of the soil.

9. Conclusions

(1) The load carrying capacity of the group of piles in unsaturated expansive soil increased with increasing the depth of phreatic surface from the ground surface.
(2) The load-carrying capacity increased with decreasing the initial degree of saturation of the expansive soil.
(3) The decrease in the initial degree of saturation leads to an increase in negative pore water pressure (matric suction). This behavior can be attributed to the contribution of matric suction, which leads to an increase in the shear strength of the soil.

10. References

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