

ANALYSIS OF REPLACEMENT LAYER PROPERTIES AND ITS EFFECTS ON A TYPICAL BUILDING ON SWELLING CLAY

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ABSTRACT

Extensive damage to the structures founded on expansive soils can be reduced using different techniques. Among these techniques, replacement of the expansive soil with a non-expansive soil is considered as one of the most popular. The main objective of the present work is to investigate the effectiveness of the replacement method under typical buildings. The finite element software ABAQUS was implemented in the present study in order to analyze the surface heave and the differential heave between the footings of a typical building. The present investigation considers the variation in replacement layer thickness, stiffness, and permeability. The soil-structure interaction was also considered. The present numerical results show a significant effect of the replacement soil thickness and permeability on the heave and differential heave. Reducing the replacement soil permeability results in significant reduction of the heave and the differential heave under different replacement soil thicknesses.

Keywords: Expansive soil; Fluid flow equation; Finite element method; Replacement; Stiffness Suction.

1 INTRODUCTION

Expansive soils are found all over the world. These soils are considered a worldwide problem because they absorb large quantities of water and expand during the wet periods and become sticky and heavy. They shrink during the dry periods and become stiff and dry. The change in soil volume causes extensive structural damage to structures built on it [1]. There are different methods to reduce the swelling of expansive soil at the ground surface in order to prevent the damage caused to the structures. Expansive soil replacement technique introduces a feasible solution to this problem. In Egypt, the use of this technique is considered as one of the most common and practical techniques to control the swelling behavior of expansive soils. This technique was also recommended by geotechnical consultant engineers [2, 3]. Although soil replacement is a very common technique, there are no definitive guidelines for this technique and it depends on the geotechnical engineer expertise [2]. Expansive soil is found in an unsaturated state. So it follows the concepts of unsaturated soil mechanics. Accordingly, the volume change behavior of an unsaturated soil requires the solution of the equilibrium equation and the water continuity equation. These equations are numerically solved using the finite element method [4].

Arafat and Ebid [3] developed a simplified formula that depends on a constant volume oedometer test to estimate the heave of swelling soil while taking the effect of the replaced layer into consideration. They utilized the developed formula to estimate the required replacement depth to avoid damage due to soil heave. Morsi [2] performed a parametric study using the CRISP, SEEP/W programs to study the effect of replacement layer on the magnitude of soil settlement and soil heave under a single footing. The parametric study included the effect of depth, lateral

exten

sion and the relative density of replacement layer. Morsi [2] also performed a parametric study for the impact of several water sources on the footing heave. The investigated sources of water were climate, lawn, pipe leakage, and infiltration effect. Bharadwaj [5] performed a parametric study using SV Flux and SV Solid packages to evaluate the effectiveness of the replacement method for reducing the surface deformations in expansive soils. The behavior of a slab placed on an unsaturated swelling soil was analyzed using various non-expansive replacement layers and different surface flux boundary conditions. The comparison between the various replacement layers was based on the extent, the degree of wetting and the ground surface movements. The slab was modeled as an impermeable surface which does not resist heave [5].

In the present work, a parametric study for the effect of the replacement layer characteristics on the soil heave and the differential heave between the footings of a typical building was performed. The soil- structure interaction was taken into account. The finite element software ABAQUS was implemented in the present study due to its capability of solving the coupled stress deformation-water flow equations. The objective of the present work is to define the replacement soil characteristics that reduce the heave and differential heave to acceptable levels. The investigated replacement soil characteristics include the thickness, stiffness, and permeability.

2. NUMERICAL MODEL

Figure 1 shows the present Finite Element Model (FEM). It consists of a typical four-story building resting on a Regina clay expansive soil. Each story is 3 m in height, 16m × 16m inplan, and divided into 4 bays. The utilized 3D soil domain for the present simulation is 28m×28m×8m (Length × width × height) [6]. The FEM analysis was carried out using the commercial software ABAQUS. The analysis was performed with the transient soils consolidation procedure using automatic time incrementation[7]. Three-dimensional reduced-integration pore pressure elements (type C3D8RP) were used in this research work to perform the analysis. This is because the C3D8RP element is suitable for modeling the fluid flow in a deforming porous medium where the pore pressure can be varied through this element. An initial negative pore water pressure of 415 kPa was assigned at the bottom surface of the model. This pore water pressure was linearly increased up to 495 kPa at the surface of the model. The ABAQUS software calculates the initial condition for the degree of saturation according to the suction value at each depth based on a linear suction distribution and a soil water characteristics curve (SWCC). The initial value of the soil void ratio is 0.955. The initial conditions for the effective stress were calculated from the densities of the dry material and the fluid, the initial saturation, the void ratio, and the initial pore pressures using the equilibrium considerations and the effective stress principle [7].The initial geostatic status of the soil in mechanical stress was calculated with a coefficient of lateral earth pressure at rest equal to 0.67 which is based on the assumption that its value is equal to $\nu/(1-\nu)$ where ν is the Poisson's ratio [8].

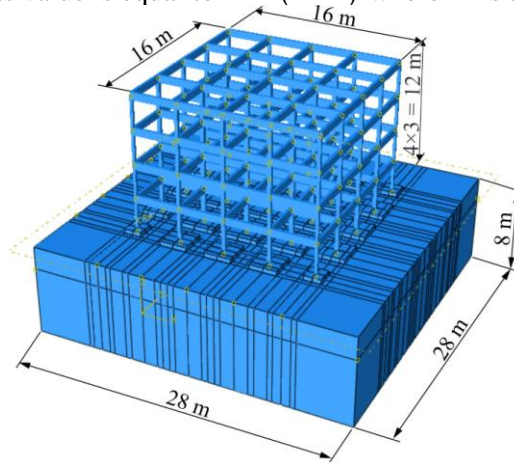


Figure 13D finite element model geometry used in this analysis

Different mechanical boundary conditions were utilized for the simulated domain. The bottom surface of the model was restrained from moving in the vertical direction. The four side surfaces of the model were restrained from moving in the perpendicular direction and were free to move vertically. The top surface of the model was free to move vertically. Considering the hydraulic boundary conditions, the horizontal flow on the four sides of the simulated domain is zero. The model was run in several modes and three steps. The initial step was performed to specify the initial conditions using the predefined values. This step is usually the first step of a geotechnical analysis[7].The second step was a geostatic step. This step was performed to establish an initial equilibrium state. Therefore, no deformation takes place through this step. The weight of the soil and building are applied by gravity loading. The last step was performed in “soil consolidation” mode. In this step, the model was loaded by applying a surface pore pressure equal to zero on the model around the building to represent rainfall infiltration. The duration of this step was 175 days. The weight of the building was transferred to the soil surface by contact elements [9].These contact elements were used in order to simulate the soil-concrete footing interaction. The upper side of the contact element is the bottom surface of the concrete footing and the lower side is the soil surface[9]. In the present model, the master-slave algorithm was used to define the contact between the soil and the footings. The friction coefficient was taken as 0.3.

3.SOIL PROPERTIES

Regina clay was selected for use in the present numerical analysis. This is because its properties were investigated by several researchers (e.g., Shuai [10]). Shuai [10] measured the test data of the Regina clay along the wetting path under the confined condition. Table 1 presents the index properties and mineralogical composition for the investigated Regina clay soil.

Table 1 Index properties of the Regina clay soil [10]

Soil:	Regina clay
Location:	Regina, Sask., Canada
Atterberg limits:	LL=69.9%, PL=31.9%, PI=38.0%
Grain size distribution (based on ASTM D422,1988):	Sand: 2.2%, Silt: 32.9%, clay: 64.9%
Unified Soil Classification System:	CH, Inorganic clay of high plasticity
Standard compaction:	Maximum dry density:14.01 kN/m ³ Optimum water content: 28.5%
Mineralogical composition:	Montmorillonite: 20%, Illite: 42%, Kaolinite: 14%

The soil mechanical behavior was modeled with a non-linear elastic model to describe the elastic portion of the constitutive model and Drucker- Prager plasticity model to describe the soil behavior in the plastic region. The porous bulk modulus κ was calculated using equation (1) [11].The Drucker-Prager parameters were calculated from the Mohr-Coulomb parameters using equations (2) and (3) [12]. The yield function of the hardening model was calculated from equation (4) [13]. Table 2 introduces the mechanical properties of the Regina expansive clay while Figure 2 shows the yield function of the hardening model.

$$\kappa = \frac{C_s}{2.303} \dots\dots\dots (1)$$

Where C_s is the swell index from the conventional 1-D consolidation test.

$$\tan \beta = \frac{6 \sin \phi'}{3 - \sin \phi'} \dots\dots\dots (2)$$

$$d = \frac{18 c' \cos \phi'}{3 - \sin \phi'} \dots\dots\dots (3)$$

$$\varepsilon_{vol}^{pl} = \frac{\lambda - \kappa}{1 + e_0} \ln \frac{p'}{p_0} = \frac{C_c - C_s}{2.3(1 + e_0)} \ln \frac{p'}{p_0} \dots\dots\dots (4)$$

Table 2 Mechanical properties of Regina expansive clay [10,14,15]

Soil Properties	Values
Specific gravity, G_s	2.83
Swelling index, c_s	0.0645
Soil unit weight, γ_b	17.27 KN/m ³
Effective frictional angle, ϕ°	20 degree
Effective cohesion, c	0 kPa
Porous bulk modulus, κ	0.028
Poisson's ratio, ν	0.4
Flow stress ratio, K	1
Compression index, C_c	0.276
Log hardening constant, λ	0.12
Modulus of elasticity, E	10 MPa
Swelling pressure, p_s	350 kPa

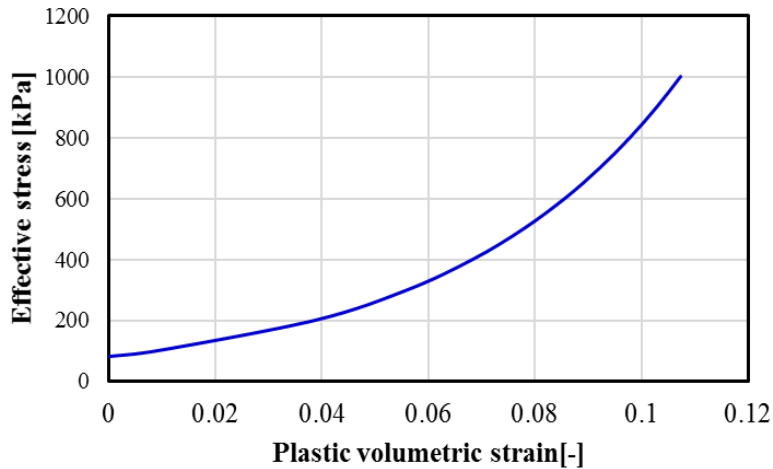


Figure 2 The yield function of the Regina clay

In the present work, the void ratio-matric suction relationship for Regina clay soil was fitted using the Vu Hung equation [4] with four fitting parameters A, B, C, and D:

$$e = A + B \log\left(\frac{1 + C \cdot U}{1 + D \cdot U}\right) \dots\dots\dots (5)$$

Where e is the void ratio, and U is the matric suction. The values of the fitting parameters for Regina expansive clay are shown in Table 3. Variation of the void ratio with the matric suction for the Regina clay is shown in Figure 3.

Table 3 Fitting parameters of the void ratio–matric suction for Regina clay [4]

A	B	C	D
1.1078	-0.0624	0.2936	4.80E-04

The volumetric water content-matric suction relationship for Regina clay soil was fitted using the Vu Hung equation [4] with four fitting parameters E, F, G, and H:

$$\theta = E + F \log\left(\frac{1 + G \cdot U}{1 + H \cdot U}\right) \dots\dots\dots (6)$$

Where θ is the volumetric water content and U is the matric suction. The values of the fitting parameters are shown in Table 4. Variation of the volumetric water content with the matric suction for the Regina clay is shown in Figure 4.

Table 4 Fitting parameters of the volumetric water content–matric suction for Regina clay [4]

E	F	G	H
0.5015	-0.047	1.7146	1.78E-03

The degree of saturation (S) for Regina clay was calculated from the volumetric water content θ_w and the void ratio using equation (7). Figure 5 introduces the degree of saturation-matric suction relationship for the Regina clay soil.

$$S = \frac{\theta_w(1+e)}{e} \dots\dots\dots (7)$$

The permeability function of Regina clay was represented using Gardner (1958) equation. Equation (8) was implemented with k_o = 0.4x10⁻⁸ m/s, a=0.01, b= 18.5, and n=1.1[4] to evaluate the relationship between the coefficient of permeability and matric suction for the compacted Regina clay. Afterward, relative permeability was calculated using equation (9).The relationship between void ratio and saturated coefficient of permeability for Regina clay was calculated using equation (10). Figure 6 presents the relationship between the void ratio and the saturated coefficient of permeability for Regina clay while Figure 7 introduces the relative permeability as a function of the matric suction.

$$k_w = \frac{k_o e^b}{1 + a\left(\frac{u_a - u_w}{\rho_w g}\right)^n} \dots\dots\dots (8)$$

where:

a = an empirical constant

n = an empirical index

ρ_w = water density (kg/m³)

g = gravitational acceleration (m/s²)

k_0 = saturated coefficient of permeability at void ratio of 1 (m/s)

e = void ratio

b = empirical index.

$$k_r = \frac{k_w}{k_s} \dots\dots\dots (9)$$

k_r = relative permeability

k_w = unsaturated coefficient of permeability

k_s = saturated coefficient of permeability

$$k_s = k_0 e^b \dots\dots\dots (10)$$

Where:

$$k_0 = 0.4 \times 10^{-8} \text{ m/s and } b = 18.5. \quad [4]$$

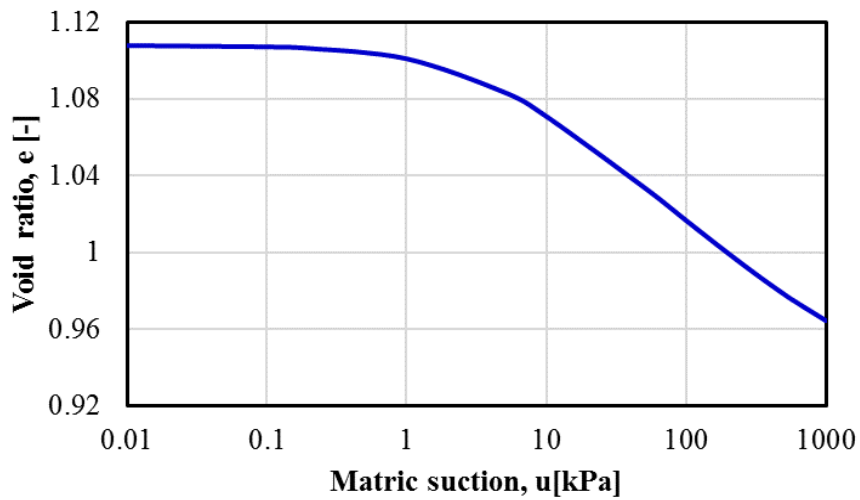


Figure 3 Best-fit void ratio versus matric suction curve for Regina clay

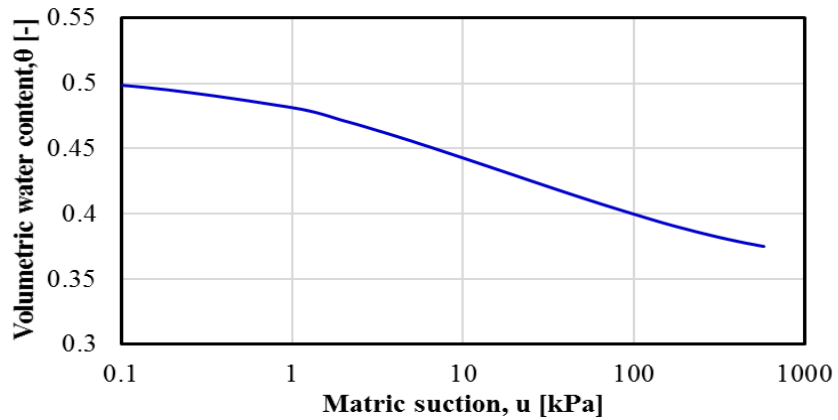


Figure 4 Best-fit volumetric water content versus matric suction curve for Regina clay

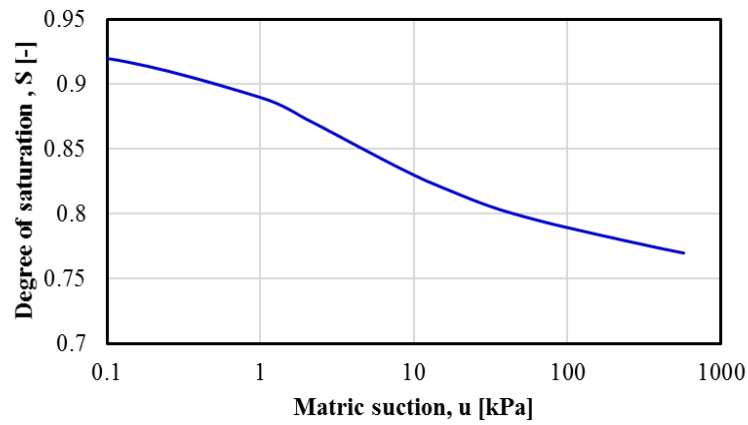


Figure 5 Degree of saturation-matric suction relationship for Regina clay

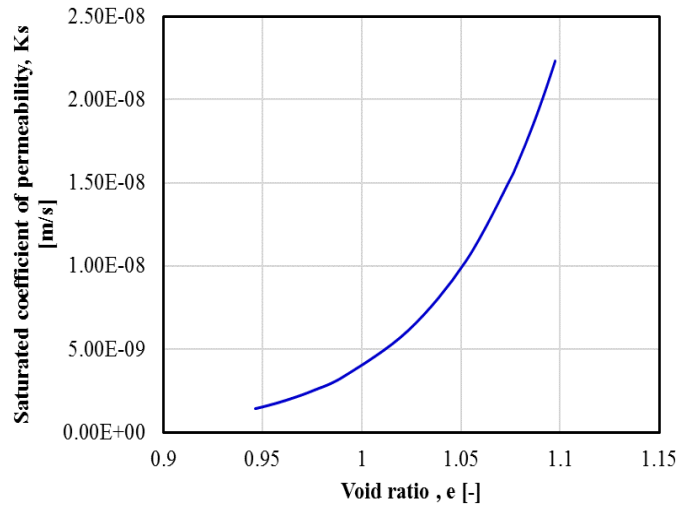


Figure 6 Saturated coefficient of permeability as a function of the void ratio for Regina clay

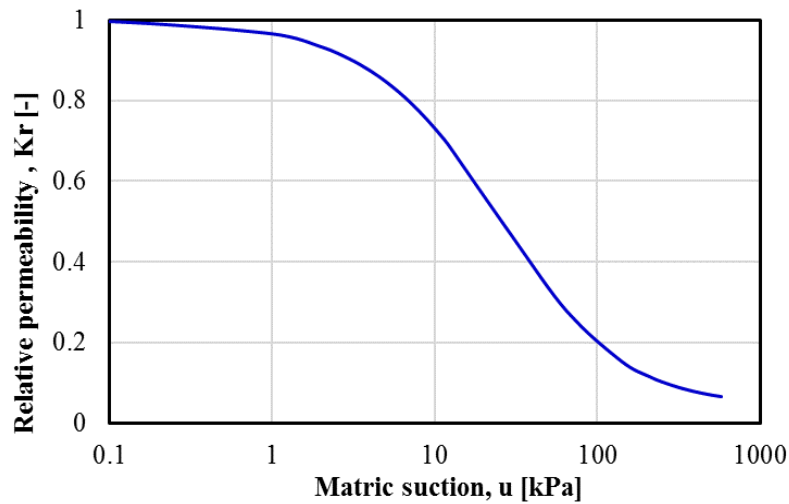


Figure 7 Relative permeability of Regina clay as a function of the matric suction

The volumetric strain (ε_v) was calculated from the change in void ratio (de) as follows

$$\varepsilon_v = \frac{de}{1+e_o} \quad (11)$$

where e_o is the initial void ratio of the soil. Figure 8 represents the relationship between the volumetric strain and the matric suction for Regina clay.

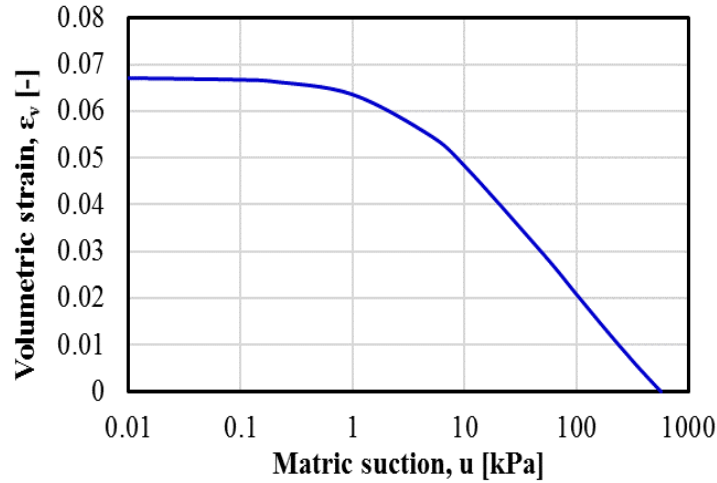


Figure 8 Volumetric strain-Matric suction relationship for Regina clay

The footings, beams, and columns of the building are made of concrete. The concrete mass density was considered as 24 kN/m^3 . The Poisson's Ratio of the concrete was assumed as $\nu = 0.25$ while its Young's Modulus was assumed as $E = 20 \text{ GPa}$.

The properties of the replacement soil were collected and obtained from [2, 5]. The replacement soil is a non-expansive soil (i.e., insensitive to moisture change). This means that it does not change in volume when the soil suction is changed. The dry density of the replacement soil is 18 kN/m^3 .

4 Results of Analysis

The results of the present numerical model are presented in the following subsections. Many paths were established through the model in order to obtain the numerical results along these paths as shown in Figure 9. The present numerical results are presented along path 1 and 3. Footings along each path are numbered as shown in Figure 9 where the first footing along each path takes number 1. The water source for all cases is uniformly distributed around the building perimeter.

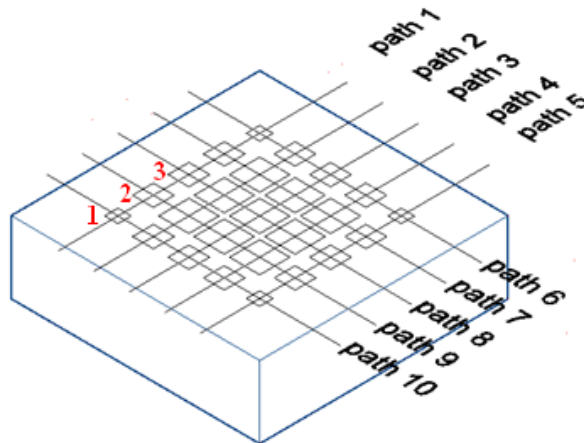


Figure 9 Different paths through the model

4.1. Effect of replacement layer thickness

The replacement layer thickness was gradually varied from 1 to 2m and the model performance was simulated under different replacement thicknesses. Figure 10 shows the variation of the final matric suction at the soil surface (under the footings) along path 1 and 3. The highest matric suction takes place at the center of the building due to the uniform distribution of the water source around the building. The matric suction along path 3 is higher than that along path 1. This means that the soil is more saturated along path 1 compared to path 3 due to the fact that the water source is more close to Path 1. Accordingly, a higher heave takes place along path 1 compared to that along path 3 as shown in Figure 11.

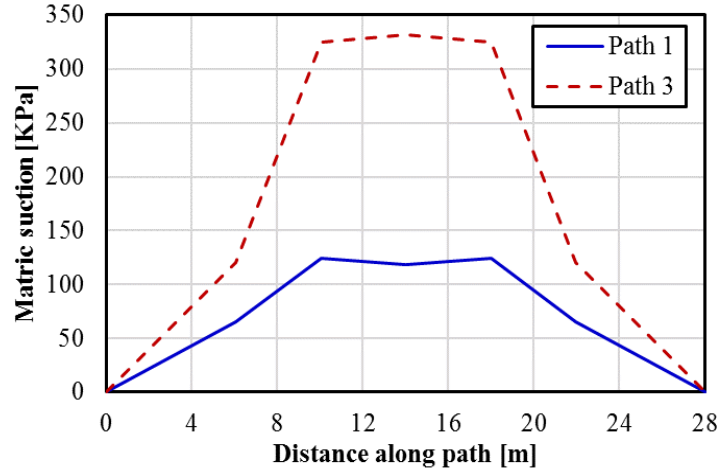


Figure 10 Variation of final matric suction at soil surface along paths 1 and 3

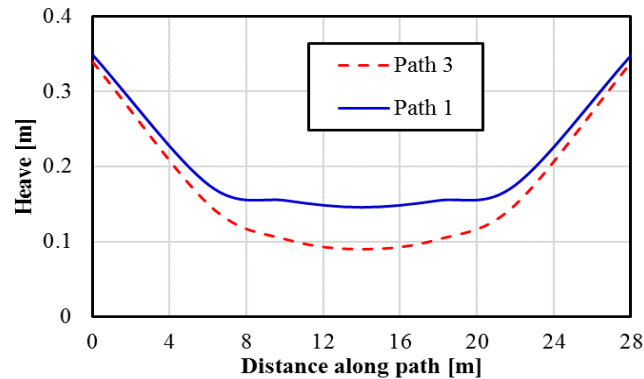


Figure 11 Variation of heave at soil surface along different paths

Figure 12 introduces the heave of the entire numerical domain (28m×28m) along Path 3 while Figure 13 and Figure 14 presents the footings heave along Paths 1 and 3 at different replacement thicknesses. The replacement soil depth has a significant effect on the soil heave and the differential heave between footings. The footings' heave and differential heave decreases with increasing the replacement thickness due to the decrease in the depth of the expansive soil zone. Along path 3, the footings' heave decreases by 20% when using 1m depth replacement layer, by 31% when using 1.5m depth replacement layer, and by 40% when using 2m depth replacement layer. Along path 1 the heave decreases by 16% when using 1m depth replacement layer, by 26% when using 1.5m depth replacement layer, and by 34.5% when using 2m depth replacement layer.

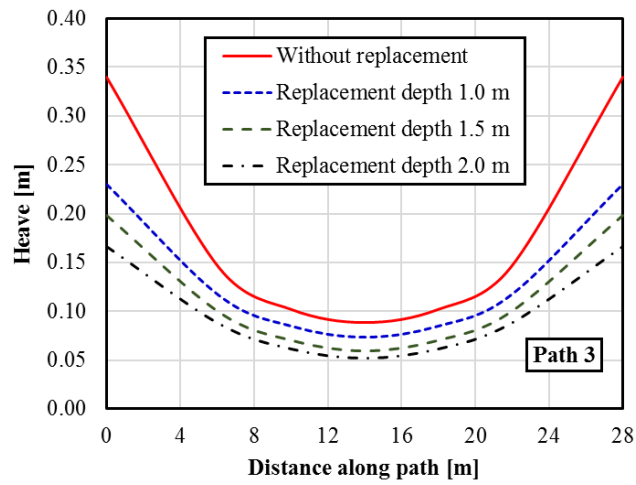


Figure 12 Effect of replacement soil depth on the heave

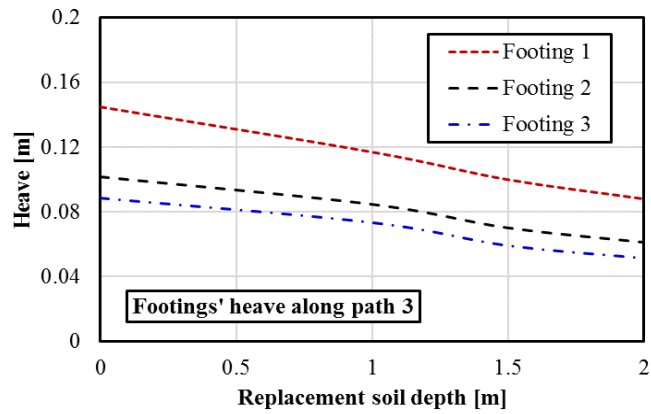


Figure 13 Variation of heave at soil surface along path 3

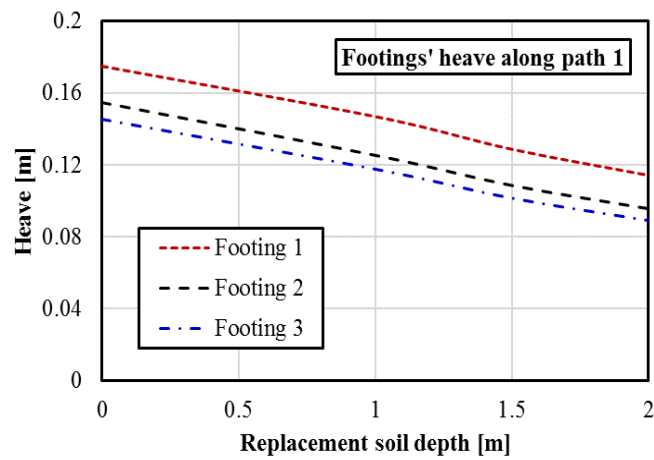


Figure 14 Variation of heave at soil surface along path 1

Figure 15 shows the effect of the replacement soil depth on the normalized differential heave between footings. The normalized differential heave is defined as the ratio between the differential heave between footings at any replacement thickness and the differential heave between footings in the case without replacement

$$\text{Normalized differential heave} = \frac{\text{Differential heave at any replacement soil depth}}{\text{differential heave in the case without replacement}} \quad (12)$$

The differential heave between Footings 1 and 2 is higher than that between Footings 2 and 3 due to the higher variation of the matric suction between these footings as shown in Figure 10. The normalized differential heave between the footings along path 3 decreases by 25% when using 1m depth replacement layer, by 31% when using 1.5m depth replacement layer, and by 38% when using 2m depth replacement layer. The red line represents the critical differential heave. It was calculated from the allowable differential heave (slope 1/150) in the design code. Using 2m replacement soil thickness was found to reduce the differential heave between footings to the acceptable level.

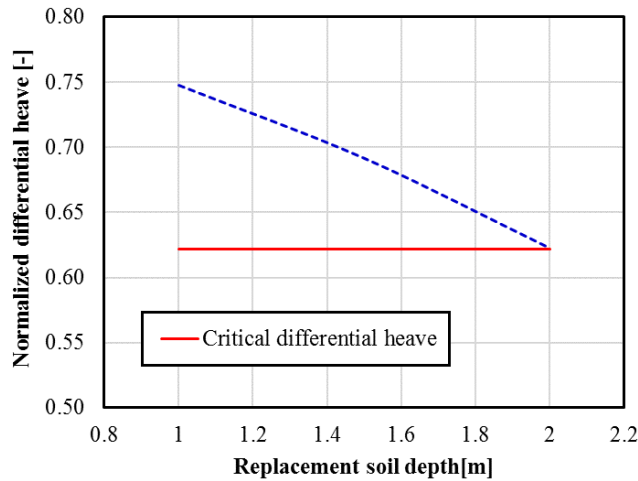


Figure 15 Effect of replacement soil depth on differential heave along path 3

4.2 Effect of the replacement layer stiffness on the heave

The replacement layer stiffness was gradually varied from 12.5 Mpa to 75Mpa and the model performance was simulated under these different values. Figure 16 shows the effect of the different replacement layer stiffness on the heave between footings along Path 1. The normalized replacement soil stiffness is defined as the ratio between the stiffness of the replacement soil and the stiffness of the expansive soil.

$$\text{Normalized replacement layer stiffness} = \frac{\text{replacement layer stiffness}}{\text{expansive soil stiffness (10 MPa)}} \quad (13)$$

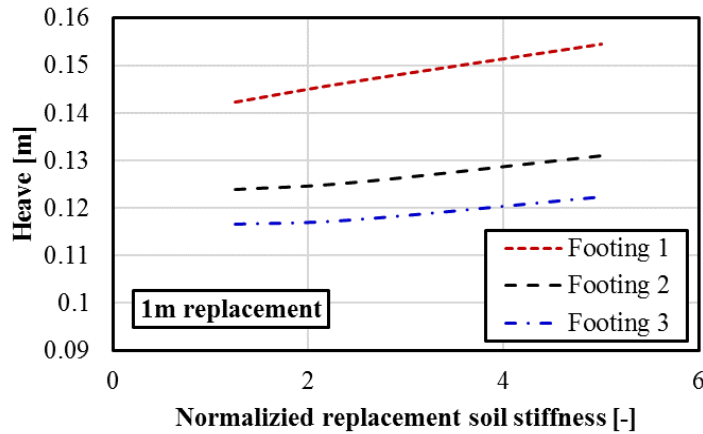


Figure 16 Effect of the stiffness of replacement layer on heave along path 1

Figure 6 shows that the heave between footings along Path 1 increases with increasing the replacement soil stiffness due to the increase in the rigidity of the replacement soil. For 1m replacement soil thickness, the heave of Footing 1 increases by 3 % when the replacement soil stiffness increases from 12.5MPa to 25MPa and by 5% when the replacement soil stiffness increases from 25MPa to 50MPa. For Footing 2, the heave increases by 1.2 % when the replacement soil stiffness increases from 12.5MPa to 25MPa and by 4.4% when the replacement soil stiffness increases from 25MPa to 50MPa. For Footing 3, the heave increases by 0.85 % when the replacement soil stiffness increases from 12.5MPa to 25MPa and by 3.99 % when the replacement soil stiffness increases from 25MPa to 50MPa.

Figure 17 shows the effect of the different replacement layer stiffness on the differential heave between footings along Path 3 under different replacement thicknesses. It shows that the differential heave increases with increasing the replacement soil stiffness. Increasing the replacement soil stiffness from 25MPa to 75MPa increases the differential heave by 5 % for a 1m replacement soil thickness, 10.5% for a 1.5m replacement soil thickness, and 6.75% for a 2m replacement soil thickness. The 2m replacement soil thickness with the lowest replacement soil stiffness (25MPa) reduces the differential heave between footings to the acceptable level.

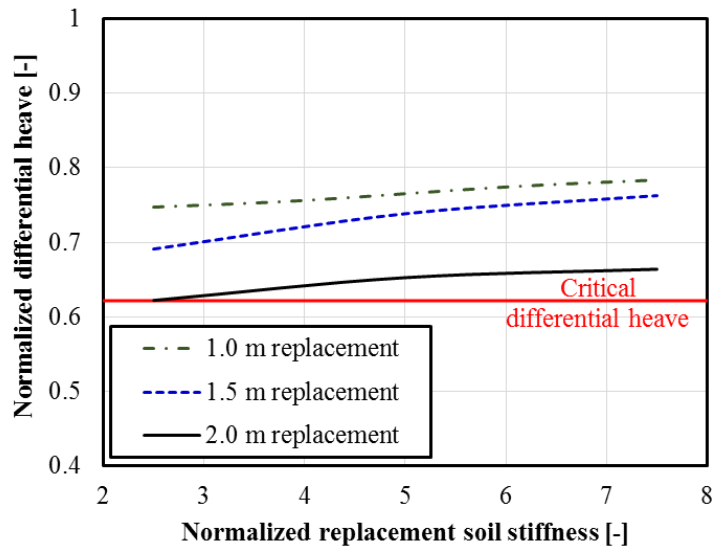


Figure 17 Effect of the stiffness of replacement layer on differential heave along path 3

4.3 Effect of replacement layer permeability on the heave

The replacement layer permeability was gradually varied from 1.296×10^{-3} to 8.64×10^{-4} m/day and the model performance was simulated under these different values of the replacement layer permeability and replacement thickness. Figure 18, Figure 19, and Figure 20 show the results of the analysis along Path 3. The heave decreases with decreasing the permeability of the replacement layer.

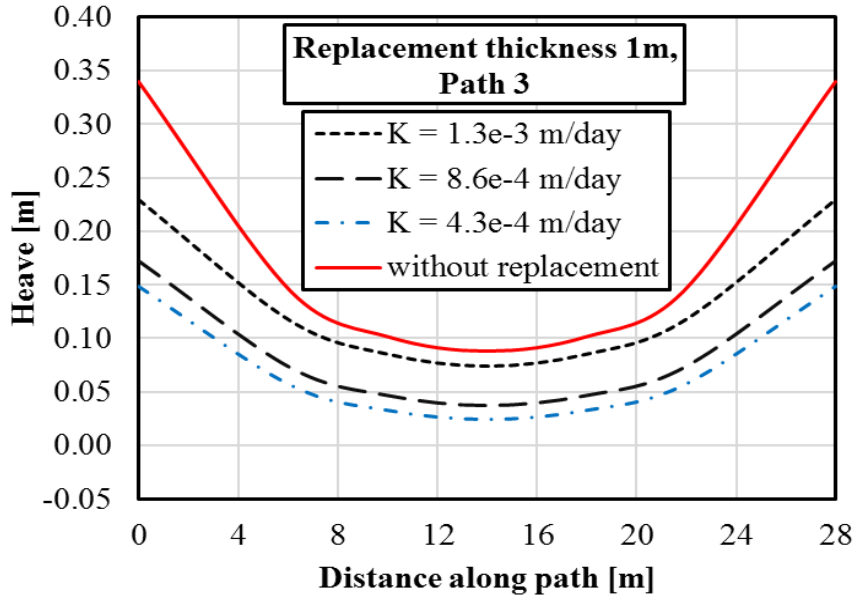


Figure 18 Effect of replacement soil permeability on heave for 1m replacement thickness

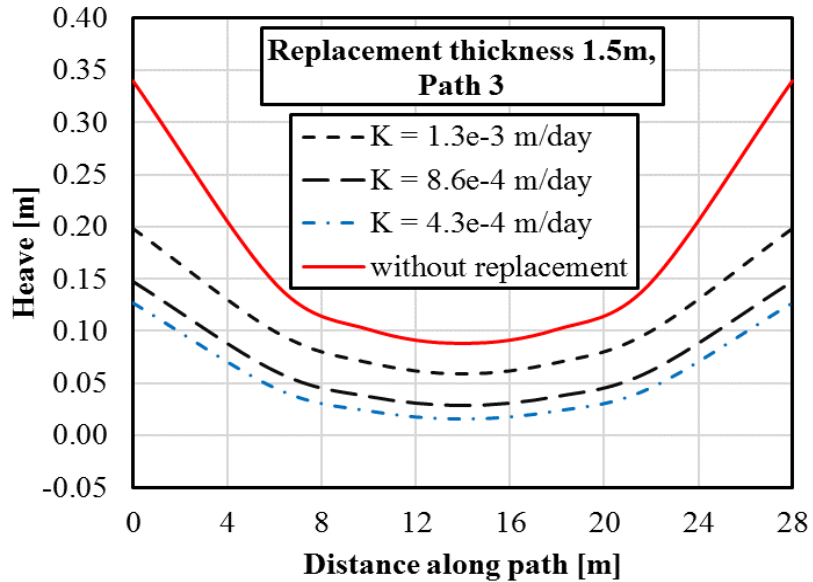


Figure 19 Effect of replacement soil permeability on heave for 1.5m replacement thickness

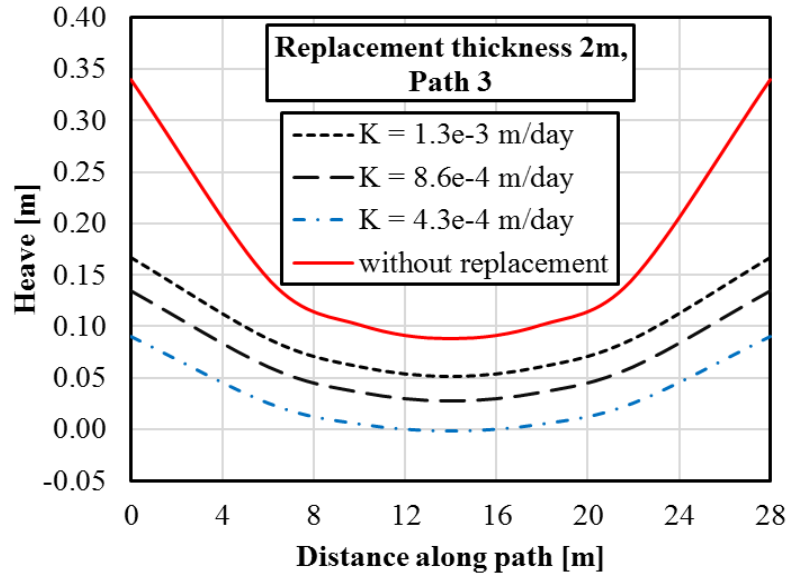


Figure 20 Effect of replacement soil permeability on heave for 2m replacement thickness

For a 1m replacement soil thickness, the heave decreases by 30 % when the replacement soil permeability decreases by 33 % and by 41 % when the replacement soil permeability decreases by 67 % as shown in Figure 18. Meanwhile, the heave decreases by 26 % when the replacement soil permeability decreases by 33 % and by 37.6 % when the replacement soil permeability decreases by 67 % as shown in Figure 19 for a 1.5 m replacement soil thickness. For 2 m replacement soil thickness, the heave decreases by 43 % when the replacement soil permeability decreases by 67 % as shown in Figure 20. Accordingly, sand is not suitable as a replacement layer due to its high permeability.

Figure 21 shows the effect of the different replacement layer permeability on the differential heave between footings along Path 3. The differential heave between footings decreases with decreasing the permeability of the replacement layer.

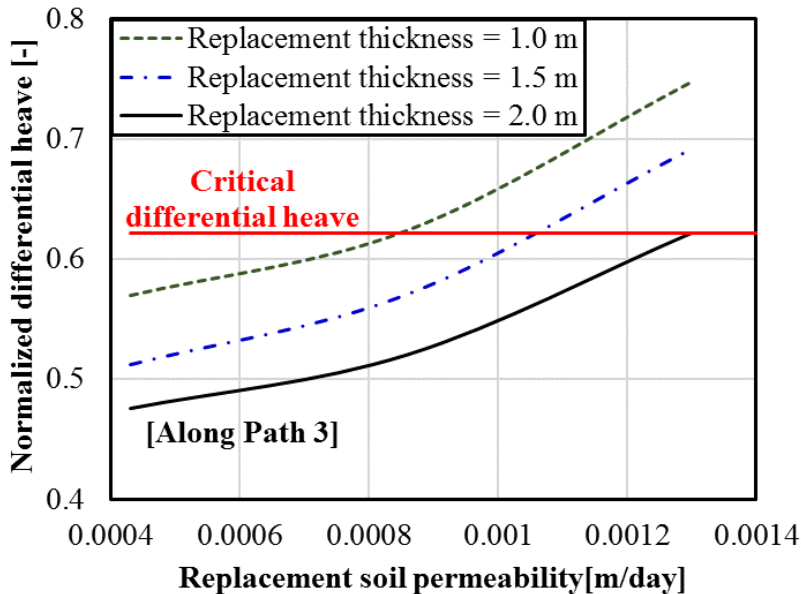


Figure 21 Effect of replacement soil permeability on differential heave between footings for different replacement layer depths

Figure 21 shows that the differential heave decreases by 16.5% when the replacement soil permeability decreases by 33.85 % and by 23.8% when the replacement soil permeability decreases by 67% for a 1m replacement soil. For a 1.5 m replacement soil, the differential heave decreases by 17.5% when the replacement soil permeability decreases by 33.85 % and by 26 % when the replacement soil permeability decreases by 67 %. For a 2 m replacement soil, the differential heave decreases by 16.5 % when the replacement soil permeability decreases by 33.85% and by 23.5% when the replacement soil permeability decreases by 67 %.

6. CONCLUSIONS

The present work concerns with the numerical simulation of the replacement method for the expansive soil in order to evaluate its effectiveness in reducing the surface heave and the differential heave between footings under buildings to an acceptable level. This includes the investigation of the replacement layer characteristics such as its thickness, stiffness, and permeability. A finite element analysis was implemented using the commercial software ABAQUS. The replacement of expansive soil was thoroughly investigated. The following conclusions can be drawn:

- a. The replacement layer characteristics such as thickness, stiffness, and permeability significantly affect the total heave and differential heave under a typical building.
- b. The heave along the building's centerline decreases by 20% when using 1m depth replacement layer, by 31% when using 1.5m depth replacement layer, and by 40% when using 2m depth replacement layer.
- c. The differential heave between the footings along the building's centerline decreases by 25% when using 1m depth replacement layer, by 31% when using 1.5m depth replacement layer, and by 38 % when using 2m depth replacement layer.
- d. The differential heave between footings increases with increasing the replacement soil stiffness due to the rigidity of the replacement soil. A replacement soil stiffness of 25MPa is recommended at a replacement layer thickness of 2m.
- e. Decreasing the replacement soil permeability by 67% reduces the heave by 43% and the differential heave by 23.5% with a replacement layer thickness of 2m.
- f. Untreated Sand would still cause problems as a replacement soil because of its high permeability.

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