Effect of Tunneling in Cohesive Soils on Existing Structures

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Received on 29/4/2012 & Accepted on 13/5/2014

ABSTRACT:

The present work is focused on the influence of shallow tunneling on the settlement of existed two storey building supported on different soil properties ranging from medium to stiff clayey soil that having young modules of "50,75 and 100 MPa". Eight locations of the tunnel center "diameter = 4m" were fixed below the building strip footing "width = 2m" at different depths and locations to determine the critical location of the tunnel at each depth.

A total of 24 Finite element CAD "ANSYS" solutions were performed on the eight locations for each of the three types of soil.

The results of the FEM analysis show that the effect of tunneling was to increase the surface settlement and creating differential settlement at the different locations and the critical location of the tunnel was when the tunnel center is located below the center line of the footing at both depths.

Keywords: tunneling, cohesive soils, ANSYS, Non-linear analysis, Differential settlement.

تأثير الأنفاق في الترب المتماسكة على الأبنية الموجودة

الخلاصة:

تم في هذا البحث النظري التركيز على تأثير عمل الأنفاق على أعماق قريبة من سطح التربة على مقدار الهبوط المتوقع لبناية موجودة مسبقاً مكونة من طابقين ومستندة على تربة طينية متماسكة مختلفة الخصائص تتراوح كثافتها بين المتوسطة الى القوية وتم أختيار معامل المرونة لهذه الترب (50،75،100) ميكاباسكال . تم اختيار ثماني مواقع للنفق المزمع تنفيذه و بقطر "4 متر " أسفل احد الأسس الشريطية " ذو عرض 2متر" للبناية الموجودة وعلى أعماق مختلفة لأجل تحديد الموقع الحرج لكل عمق محدد.

الدراسة أجريت بطريقة العناصر المحددة بإستعمال برنامج حاسوبي معروف(ANSYS) لغرض الدراسة وتم تنفيذ (24) حل لهذه المسألة على المواقع المفترضة للنفق وعلى الثلاث أنواع من التربة التي تم تحديدها.

بينت النتائج من البرنامج الحاسوبي بأن تأثير عمل الأنفاق كان بزيادة الهبوط السطحي وكذلك زيادة الهبوط التفاضلي للأسس في جميع المواقع التي تم تحديدها للنفق وأقصى تأثير للنفق يكون عندما يقع مركز النفق على أمتداد مركز الأساس ولجميع الأعماق المحددة.

https://doi.org/10.30684/etj.32.10A.12
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INTRODUCTION:

During the development and design of a project involving tunneling it is necessary to assess the damage that could be caused to existing or planned structures. Early initial assessment of possible effects may enable a scheme to be modified and the risk to structures minimized or avoided. For larger or complex scheme in an urban area with many structures in the area of influence a simple means is required to assess which structures may be affected and to what degree. Appropriate action can then be determined.

The three-dimensional frame in superstructure, its foundation and the soil, on which it rests, together constitute a complete system. With the differential settlement among various parts of the structure, both the axial forces and the moments in the structural members may change. The amount of redistribution of loads depends upon the rigidity of the structure and the load settlement characteristics of soil (4).

Prediction of ground movements within the soil mass surrounding excavations is a major design issue, particularly in densely populated urban areas. Numerical modeling has been used for evaluation of the behavior of excavation projects. However, the accuracy of the numerical modeling effort depends to a large extent on the adequacy of the stress-strain-strength relationships used to represent the behavior of the soils surrounding the excavation. Specifically, the constitutive model should be able to capture the soil behavior under stress paths typical in excavation projects.

The finite element simulation of a tunnel excavation through soils should ideally be performed using a 3D analysis. However, due to high computational costs, nonlinear 3D analyses are typically not performed for most projects. Thus, a methodology that approximately accounts for 3D effects using a 2D analysis used for the numerical evaluation by Azevedo et al. (2002) showed the ability of a 2D elastoplastic finite element analysis to evaluate the deformation induced in residual soils by tunnel excavations.

Prediction of ground movement due to tunneling:

The settlements caused by tunneling are often characterized by the term "ground loss" parameter, which is defined as a percentage of the ratio of the surface settlement through volume and the tunnel volume per unit length (Loganathan and Poulos, 1998). In reality, the ground loss values may vary, depending on the tunneling methods, tunnel configuration, soil types, etc. The ground loss occurs in two stages :(1) loss in the undrained state, immediately after the passing of the tunnel head; and(2) loss due to time-depending consolidation and creep of the ground.

In practice, as pointed out by Rowe and Kak (1983), the radial ground movement is not uniform since the equivalent 2Dgap (tail void) around the tunnel is noncircular (e.g., typically oval-shaped). When the portion of the soil above the tunnel crown touches the tunnel lining, the soil at the side of the tunnel displaces towards the bottom of the tunnel. Therefore, the upward movement of the soil below the tunnel is limited. When the tunnel lining settles on the bottom of the annulus gap (due to self-weight) the distance between the crown of the tunnel lining and the crown of the excavated surface becomes twice the thickness of the annulus gap. Before the formation of the gap, all the initial stresses in the soil are in equilibrium. The stresses around the tunnel are released in a non-uniform manner due to the soil movement into the oval-shaped gap that basically determines the ground deformation pattern around the tunnel (Loganathan and Poulos, 1998).

Good design should incorporate some basic principles, such as soil-structure interaction and 'arching' in the ground. The act of excavating the tunnel modifies the stress distribution in the ground. Fig.(1) shows one illustration of this, using an analytical solution for a hole in an elastoplastic plate under stress. Introducing a hole into the plate converts a distribution of principal stress in the vertical and horizontal directions into one with high tangential stresses arching around the hole and a radial stress of zero at the edge of the hole. At points far from the hole the stress pattern is unaffected by it. By means of arching, a certain amount of the initial stresses are redistributed around the tunnel, leaving the remainder to be borne by the lining (internal pressure, Pi). Hence deformation of the ground is inevitable and it must be controlled to permit a new state of equilibrium to be reached safely. The arching occurs in three dimensions and so adjacent excavations may interact. It is important for designers to be able to visualize – even if only in their own mind's eye – their tunnels in three-dimensional form.

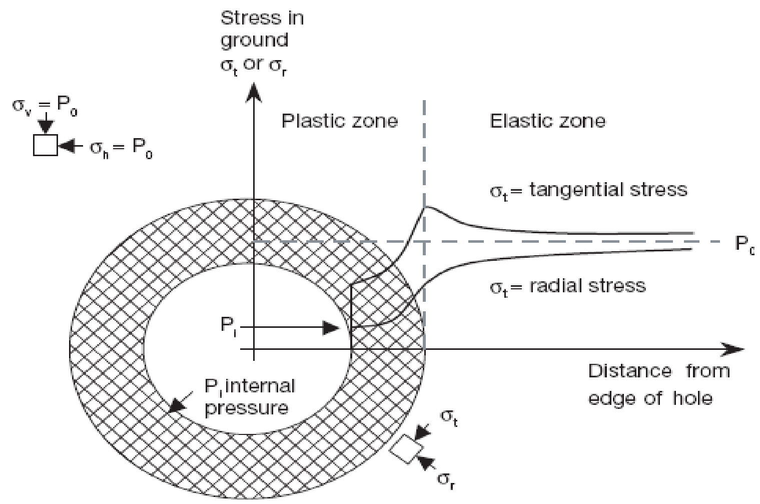
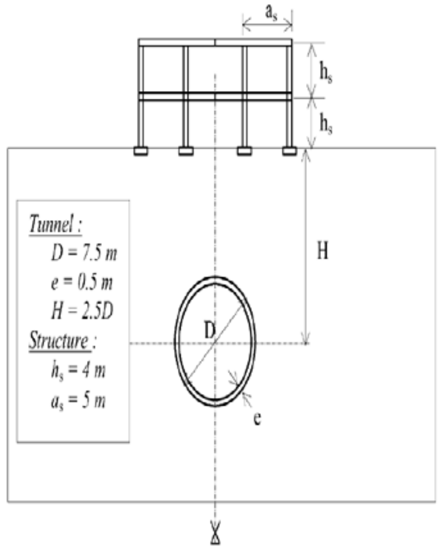


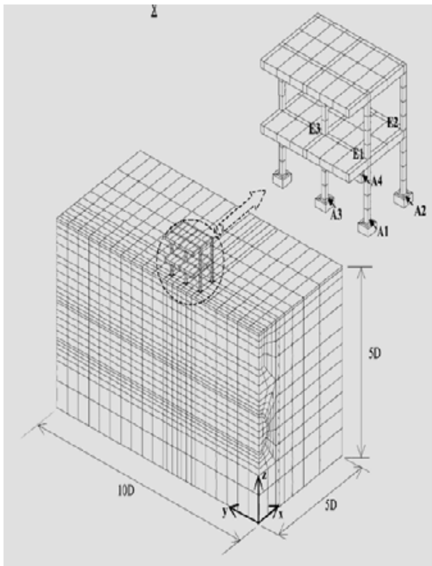
Figure.(1):"Arching" of stresses around a hole in stressed plate (Thomas,2009).

Interaction between tunnels and adjacent structures:

Mroueh and Shahrour(2003) studied the interaction between tunneling in soft soils and adjacent structures. Analysis is performed using a full three dimensional finite element model, which takes into consideration the presence of the structure during the construction of the tunnel. The soil behavior is assumed to be governed by an elastic perfectly-plastic constitutive relation based on the Mohr–Coulomb criterion. This research is composed of three parts. The first part describes the numerical model as seen in Figure (2-41a), the second part is concerned with a full three-dimensional analysis of the construction of a shallow tunnel close to a two level building as shown in Figure(2-41b). The last part includes a comparison between the full 3D analysis and a simplified approach, which neglects the influence of the presence of the structure in the determination of the tunneling-induced ground movement.



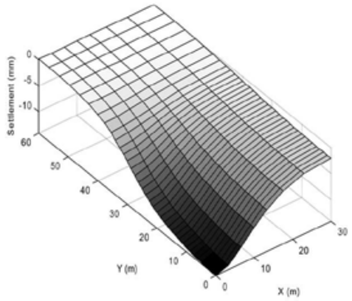
(a) Geometry under consideration



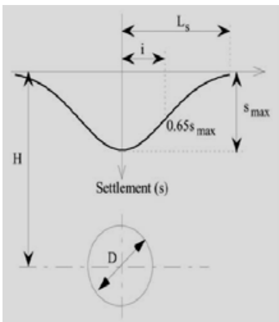
(b) finite element mesh (3912 hexahedral elements HEX20; 19017 nodes; 52533 DOF)

Figure (2): Full 3-D coupled analysis: presentation of the example (After Mroueh and Shahrour, 2003)

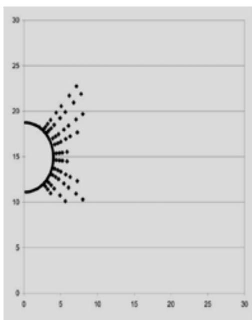
Figure (3-a) presents the 3D soil element profile computed in free field condition. It shows a normal Gaussian distribution as proposed by Peck in transverse section as shown in Figure(3-b). The maximum ground settlement (Smax) is equal to (13.5 mm), which is about (0.18%) of the tunnel diameter (D). The horizontal distance from the tunnel centre line to the point of inflection (i) on the settlement trough is (1.25D). This value agrees well with values proposed by Attewell (1977) and O'Reilly and New (1982). Figure (3-c) illustrates the distribution of the tunneling-induced soil plasticity in the transverse section of the tunnel located at a distance 4D behind the tunnel face.



(a).Settlement profile at the end of excavation



(b).Theoretical profile after Peck(1969)



(c). Extension of plasticity in transverse section

Figure (3):Free field analysis soil movement and plasticity(after Mroueh,2003)

Peng et.al. (2007) investigated cavity effects on the bearing capacity of footing foundations and the calculation method. These effects on the bearing capacity of footing foundation were investigated analytically using two dimensional elasto-plastic Finite Element Method (FEM). Several factors, such as cavity location, shapes of footing and cavity, cavity size and soil type, affect the bearing capacity of footing foundation. In his study, the effect of a single cavity on bearing capacity of footing foundations was analyzed for various conditions. The failure mechanism of ground was also examined in the study. In the FEM analyses, the geomaterial was assumed as an elastic- perfectly plastic material. The analytical results indicate that there exists a critical region for a cavity under the footing foundation, and the bearing capacity is significantly affected by the cavity only when the cavity is within the critical region. In addition, based on the above results of FEM analyses, a simple and practical calculation method is proposed for the cavity effect on the bearing capacity of footing foundation.

Ma Keshuan & Ding Lieyun (2008) investigated the interaction between the tunneling in soft soils and adjacent structures. Full three-dimensional finite element models, which take into account the presence of the building during the excavation of the tunnel, is well analyzed. The soil behavior is assumed to be governed by an elastic perfectly plastic constitutive relation based on Mohr-coulomb criterion with a non-associative flow rule. This work consisted of three parts. The first part presented the 3-D finite element numerical model, the second part provided a full analysis of the construction of a shallow tunnel close to a five level building. Comparision between the full couple model analysis and the full 3-D free-field analysis is given in the final part. The corresponding comparison results provide a fundamental guidance for the shield tunnel design and construction. Figure (4) shows the settlement during twin tunnel excavation.

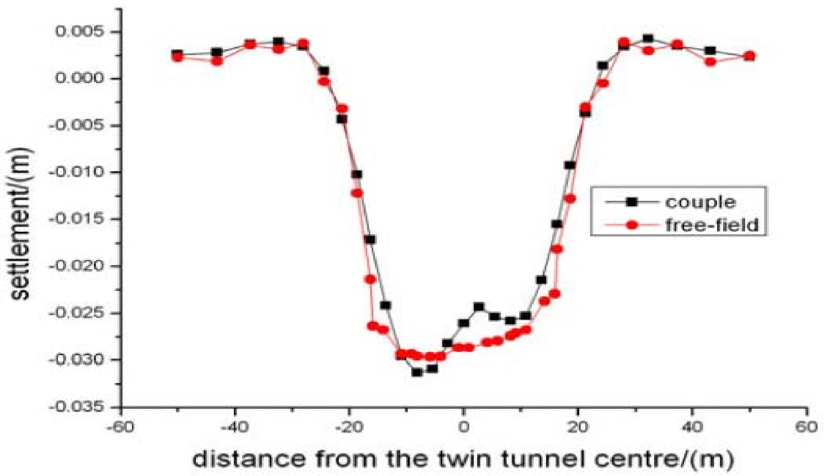


Figure (4): Settlement during twin tunnel excavation (after Ma Keshuan et.al., 2008).

Modeling the soil-foundation-structure interaction system:

Several approaches are readily used for prediction of the ground deformations associated with tunneling namely empirical, analytical and numerical methods. The selection of the appropriate method depends on the complexity of the problem.

However, the accuracy of the numerical modeling effort depends to a large extent on the adequacy of the stress-strain-strength relationships used to represent the behavior of the soils surrounding the excavation. Specifically, the constitutive model should be abled to capture the soil behavior under stress paths typical in excavation projects.
Modeling the system through discretization into a number of elements and assembling the same using the concept of finite element method has proved to be very useful method, which should be employed for studying the effect of soil-structure interaction with rigor. In fact, the technique becomes useful to incorporate the effect of material nonlinearity, non-homogeneity and anisotropy of the supporting soil-medium if needed to be accounted due to the case specific nature of any particular problem.

In the soil-structure interaction analysis, nonlinear behavior of soil mass is often modeled in the form of elasto-plastic element. Up to a certain stress level deformation occurs linearly and proportional to the applied stress. This behavior may represented by ideal reversible spring. A Hookean spring element is the best suitable representation for the same. The perfectly plastic deformation of the soil mass can be well represented with the help of a Coulomb unit (Zeevart, 1972). But when an elastic element (Hookean spring) is connected in series with a plastic element, a new schematic system known as St. Venant’s unit is formed. Use of such a single element generally shows an abrupt transition of soil from elastic to plastic state. Instead, the use of a large number of St. Venant ,s units in parallel represents the elasto-plastic behavior of the soil more accurately. Use of number of springs helps to facilitate the simulation of the gradual transition of soil strain from elastic to plastic zone (Dutta, and Roy, 2002).

Description of the Problem:

Many studies have been reported to verify the effect of tunneling but none of them examine the effect of the location of the tunnel on the behavior of the super and under-structure. This study will examine the effect of shallow tunneling on existed structure and the critical location of the tunnel under the structure foundations will be examined to determine the most critical case that could be prevented or evaluated on both the super and under-structure.
The case study consist of a two storey building of width of "8m"and length is continuing for long distance , the building is supported on strip footings "of width 2.0m" located on a depth of "1.0m" beneath the surface of a compressible clayey layer. The building skeleton is made of concrete for the beams and columns. Three different modulus of elasticity and cohesions were chosen for the clay layer to investigate the influence of the soil type on the results.
Figure (5) shows the case study and the element formation , a zero horizontal displacement is assumed for the vertical sides "roller support" while a zero vertical and horizontal displacement is assumed for the bottom side " hinge support". Table(1) shows the properties of the soil in the three cases.

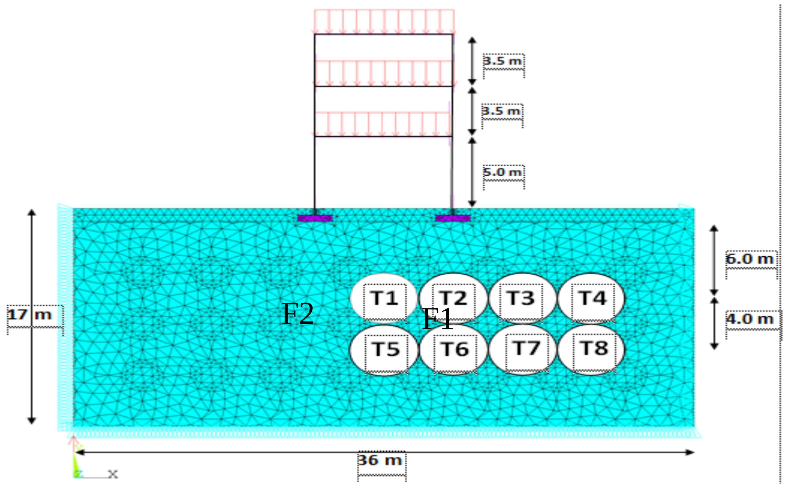


Figure (5) Element formation and study case with tunnel excavated at proposed eight locations below the super-structure

Table (1) cohesive soil properties

Item	E kN/m^2	γ kN/m^3	μ	c kN/m^2	Ø degree
Case-1	50000	19.5	0.499	50	0
Case-2	75000	19.5	0.499	75	0
Case-3	100000	19.5	0.499	100	0

The case study consist of two stages of loading , the first stage of loading "load step-1" , for each soil type , consist of loasing the building to it's designed loads in addition to the geo-static pressure , which was applied in the first sub-step ofd the first load step , until stress reaching equilibrium under these app;ied stresses. The second stage of loading "load step-2" consist of removing part of the soil underneath the existed structure at different locations and depths , to simulate the construction of the tunnel before the stiffening of the tunnels circumferential by reinforced concrete , the soil condition is then under undrained condition.

Computer Program:

The problem was investigated and solved by using finite element CAD program "ANSYS" the case is analyzed under plain strain condition to simulate the two dimensional problem. Beam3 element is used to define beams and columns. The under structure clay layer is simulated by using "plane82" element with plain strain option.

BEAM3: Is a uniaxial element with tension, compression, and bending capabilities. The element has three degrees of freedom at each node: translations in the nodal x and y directions and rotations about the nodal z axis.

PLANE82: Is a higher order version of the 2-D, four-node element (PLANE42). It provides more accurate results for mixed (quadrilateral-triangular) automatic meshes and can tolerate irregular shapes without as much loss of accuracy. The 8-node elements have compatible displacement shapes and are well suited to model curved boundaries. The 8-node element is defined by eight nodes having two degrees of freedom at each node: translations in the nodal x and y directions. The element may be used as a plane element or as an axisymmetric element. The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities.

Drucker-Prager model is used as yield criterion to characterize the behavior of the soil and the concrete structure. The Drucker-Prager failure surface in the principal stress space is a right circular cone with its central axis as the line of hydrostatic stress. The failure surface of this model can be looked upon as a smooth Mohr-Coulomb surface or as an extension of Von-Mises surface for hydrostatic pressure-dependent materials such as soil (Chen and Baladi, 1985).

The analysis was performed under non-linear analysis for the first loading step ; the load "75 kN/m2" was applied in ten sub-steps " to simulate the construction of the building" , while the geo-static pressure was included at the first sub-step. The second loading step which simulate the excavation of tunnel at fixed locations was performed in one load step to simulate the undrained condition. It is worth mentioning that the excavation process of tunnel is performed by deactivation of the pre-meshed elements "killing element process" that formed the tunnel, the subsequent loading steps is performed under the same meshing elements number and shapes to simulate the formation of tunnel thus the accuracy of the results and the ability of making comparison for a specified node or element of the two loading cases are more realistic than using different meshing element.

Results and Discussion:

Figures (6,7and8) shows the contours of vertical deformation of the whole structure (super & under structure) for all types of soils for the case when the tunnel center are located below the center line of footing (F1) at depth of (6m). the contour of vertical deformation are similar in behavior for the three soil types, the formation of tunnel affect the behavior of the soil leading to increase the settlement beneath the footing (F1) and also the movement of the soil towards the tunnel especially when the soil exhibit more softer properties this movement will increase the settlement of footing (F1) relatively to footing (F2) and thus producing differential settlement which might affect the behavior of the structure skeleton and thus shall be investigated to insure that no harmful effect on the structure and its finishing and partitioning materials.

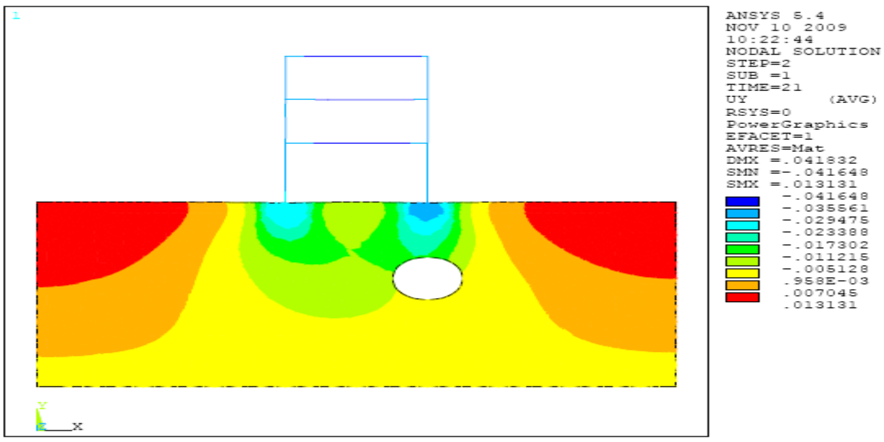


Figure (6) vertical deformation contours for tunnel depth = 6m soil properties : cu=50 kN/m^2 : E = 50000 kN/m^2

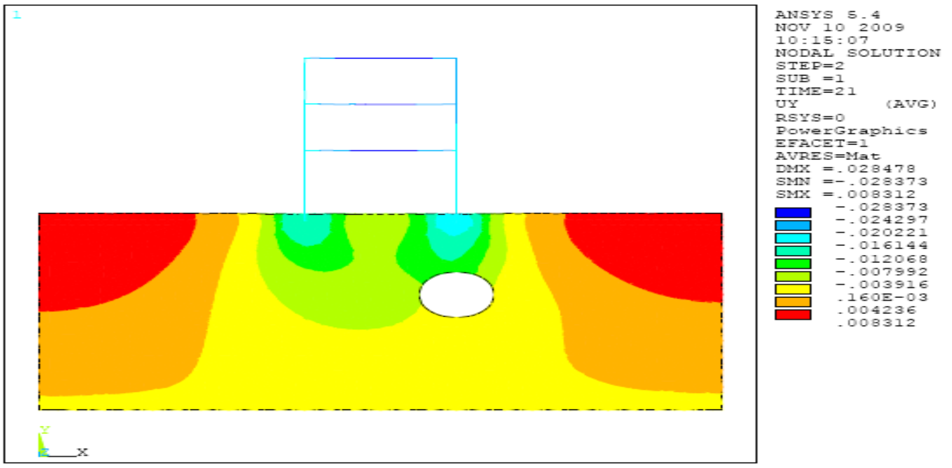


Figure (7) vertical deformation contours for Tunnel depth = 6m soil properties : cu=75 kN/m2 : E = 75000 kN/m2

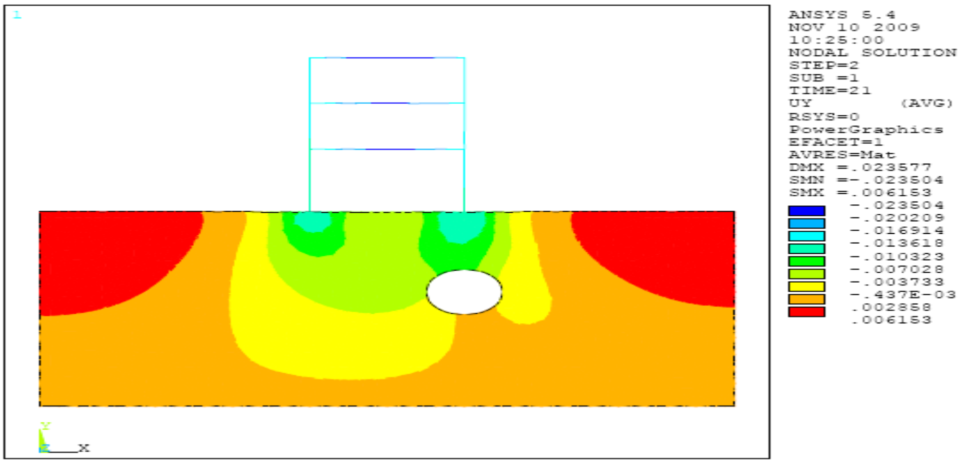


Figure (8) vertical deformation contours for Tunnel depth = 6m soil properties : cu=100 kN/m2 : E = 100000 kN/m2

Since it is not preferred , from the engineering point of view , to make tunnels below footing within the region contains the bulb of pressure of "10% of the external applied pressure and larger stresses", which is approximately corresponding to two times the footing width (2B); hence the effect of the tunneling was investigated below this depth. The first depth was investigated when the tunnel center is located "6m" below the footing i.e.(3B) hence the top boundary of the tunnel will touch the (2B) limit. Four horizontal locations were chosen (T1- T4) in addition to the original case where no-tunnel was formatted (T0) as shown in Fig,(5) to verify the most critical location of the tunnel within the building territory and near of this area.

The results of the vertical surface deformation verses the horizontal locations for different loading cases when the depth of the tunnel was "6m" and the modules of elasticity of "50 MPa" and undrained shear strength of "50 kN/m2" are presented in Fig.(9-a). The results show that for the different tunnels locations the effect of tunneling on the deformation shape was similar to that behavior when no-tunnel existed but with increasing the vertical surface deformation in both sides of the building , tunnel side and opposite side, in different values. This value is at maximum especially below the footing center and decreases when moving outwards the footings center and maximum values are always below the footing in the tunnel side. The most critical case (T2) when the tunnel center location was below the center line of the footing (F1) with maximum vertical surface settlement of (0.0317m) in comparison to the case of no-tunnel (T0) was (0.0258m). the formation of the tunnel leads to increase the vertical surface deformation below the footing in the opposite side from (0.0258m) to (0.0291m). The effect is clearly obvious that the present or formation of tunnels will increase the vertical surface deformation "settlement" regardless of the location of tunnels below or near the structure.

The results of the vertical surface deformation verses the horizontal locations for different loading cases when the depth of the tunnel was "10m" below the footing i.e.(5B) and the modules of elasticity of "50 MPa" and undrained shear strength of "50 kN/m2" are presented in Fig.(9-b). The results show that a similar behavior for the previous depth but with smaller values. The largest deformation when the tunnels center (T6) is below the center line of the footing (F1) with maximum vertical surface settlement of (0.0304m) in comparison to the case of no-tunnel (T0) was (0.0317m). while the opposite side "no-tunnel side" deformation was (0.0298m) which is slightly larger than the previous depth of (0.0291m) this means that the differential settlement is decreases when the depth of the tunnel is increased.

Fig.(9-c) show the total effect of the different tunnels locations (T1-T8) on the vertical surface settlement compared to the original case (T0). The total surface deformation is increased below the two footings in the tunnel side and opposite side with the presence of tunnel ehile the critical case when the tunnel center lies below the footing center at different depths but with all chosen locations the effects were similar on the surface settlement, any increasing in the center line of the tunnel both vertically or horizontally from the center line of footng will lead to decrease the maximum value of surface settlement and also decrease the differential settlement that results from the process of tunnels formation.

Figs,(10) and (11) show the results of vertical deformation caused by the formation of tunnel in two other types of soil having modules of elasticity of (75MPa) and (100MPa) respectively. The vertical surface settlement "deformation" are plotted verses the horizontal location. The results show the same previous behavior are obtained when the soil are stiffer in strength but with smaller deformation values , a maximum deformation of (0.018m) for the soil having (75 MPa) modules of elasticity was observed while the deformation in the absence of tunnel was (0.0152m) , on the other hand a maximum deformation of (0.0129m) for the soil having (100 MPa) modules of elasticity was observed while the deformation in the absence of tunnel was (0.0112m).

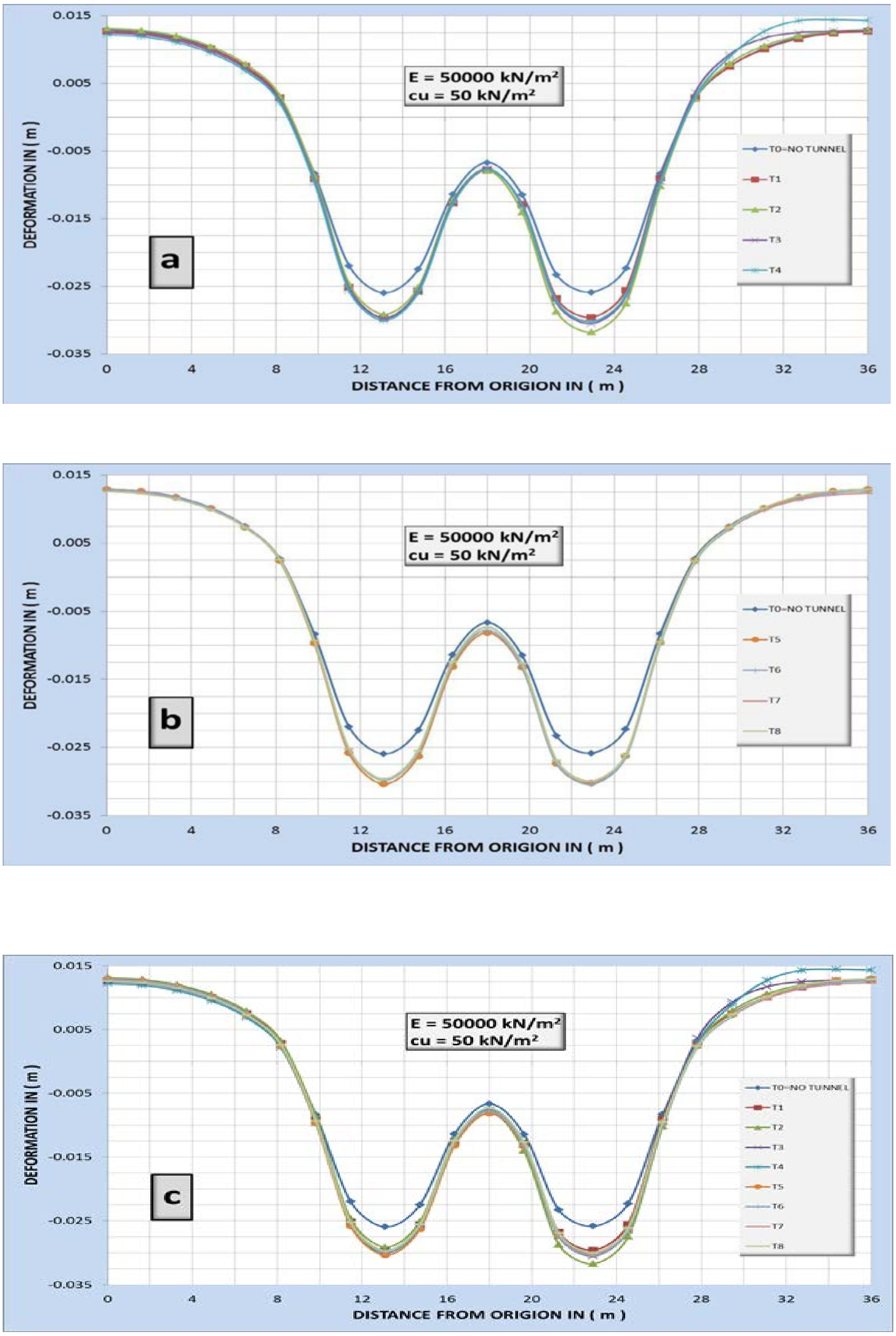


Figure (9) uy variation below footing
soil properties : cu=50 kn/m2 : e=50000 kn/m2
a) tunnel depth=-6m : b) tunnel depth=10m : c) total

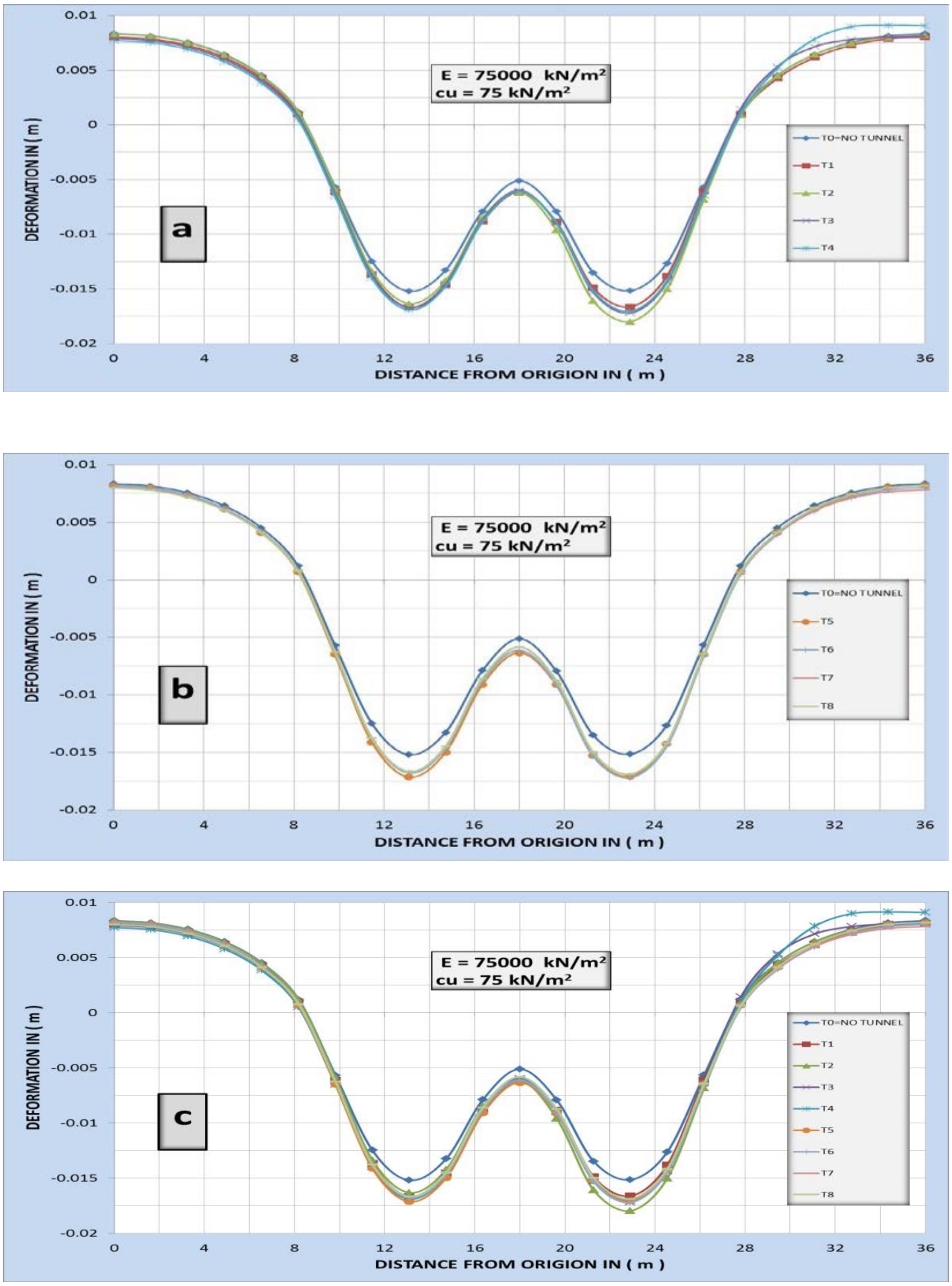


Figure (10) uy variation below footing
soil properties : cu=75 kn/m2 : e=75000 kn/m2
a) tunnel depth=-6m : b) tunnel depth=10m : c) total

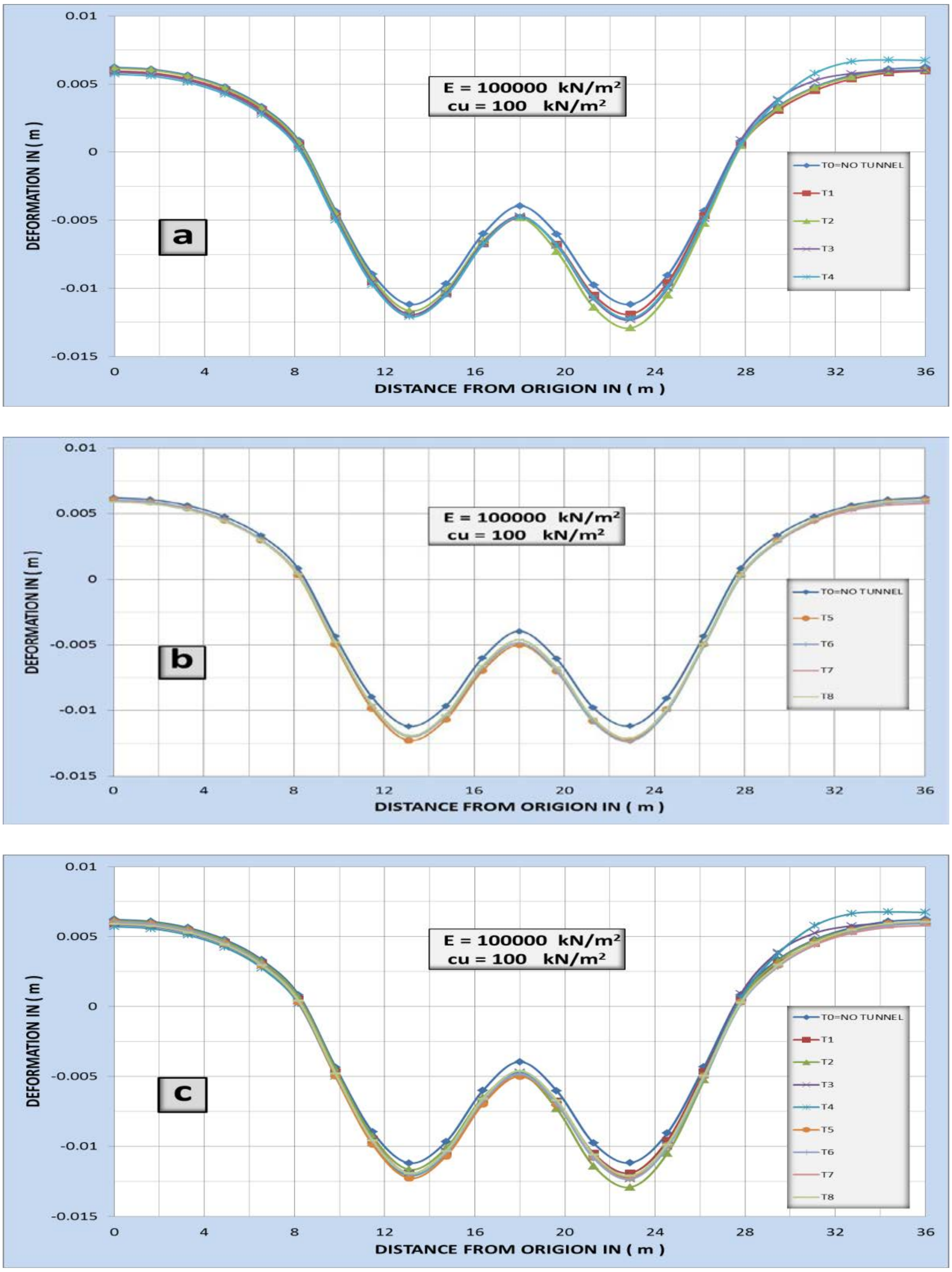


Figure (11) uy variation below footing
soil properties : cu=100 kn/m2 : e=100000 kn/m2
a) tunnel depth =-6m : b) tunnel depth=10m : c) total

Fig.(12) show the variation of the vertical deformation for the critical location of the tunnel for different soil properties when the depth of the tunnel was "6m", the plot indicate that the effect of the tunnel formation is clearly obvious for all types of soil but the effect is more critical with decreasing the soil stiffness which leads to more settlement values and more differential settlement. The same results was observed in the case of lowering the tunnel location to "10m" but the deformation will be less in magnitude , this is because in the second case the intensity of stress on the top boundary of the tunnel will be smaller than the first case which produce more pressure on the top of the tunnel circumferential boundary.

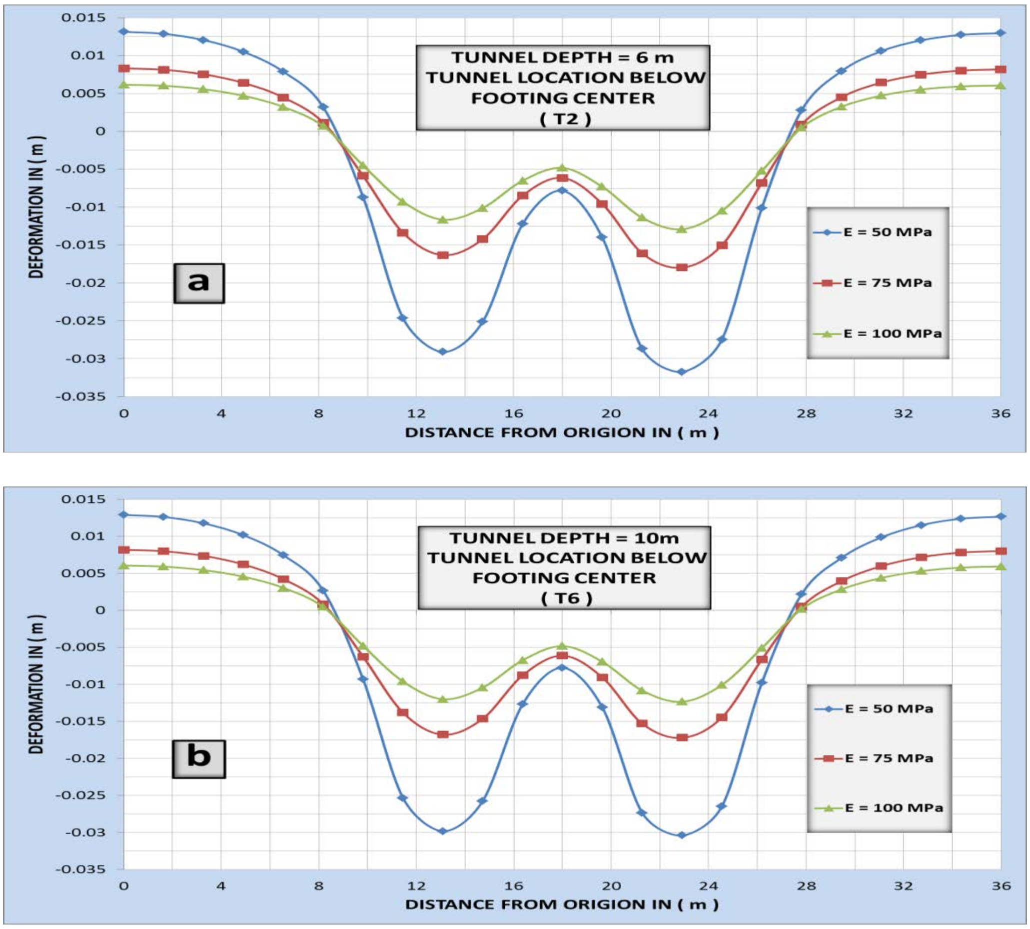
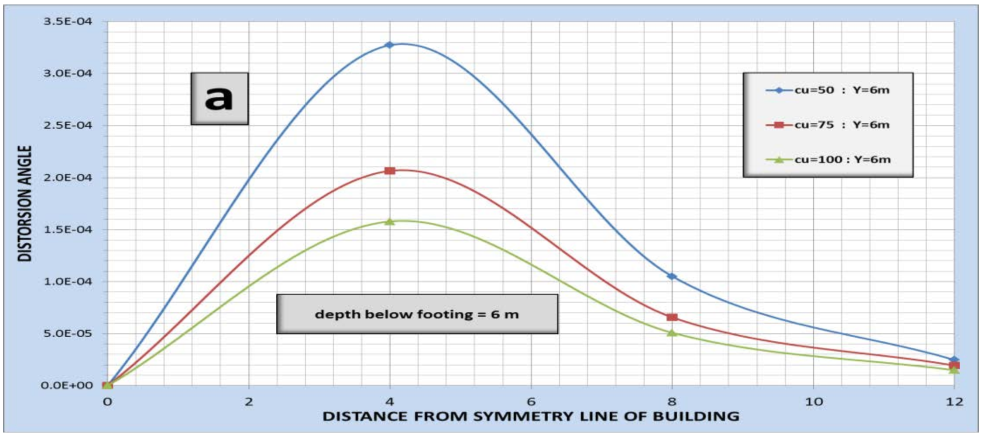
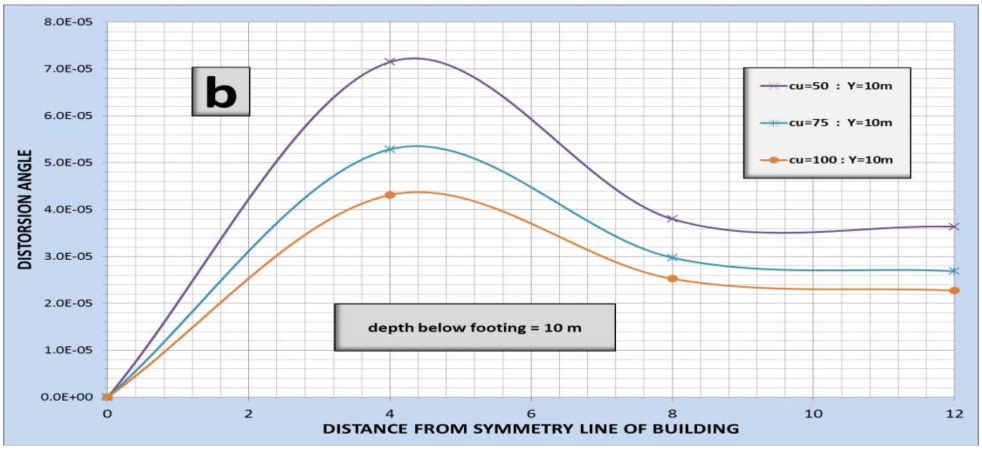


Figure (12) uy variation for critical location of tunnel
a) tunnel depth= 6m : b) tunnel depth=10m

Fig.(13) show the variation of distortion angle ((ΔF1-ΔF2)/L) with the different location of tunnel center at different depth , the charts indicate that with moving the location of the tunnel parallel to soil surface below and outward the super structure symmetry line there is an increase in the distortion angle up to a peak point below the center line of footing for all types of soil properties , beyond which the value will be decreased until reaching a semi-constant value regardless of the soil stiffness when the depth is closer to the footing.





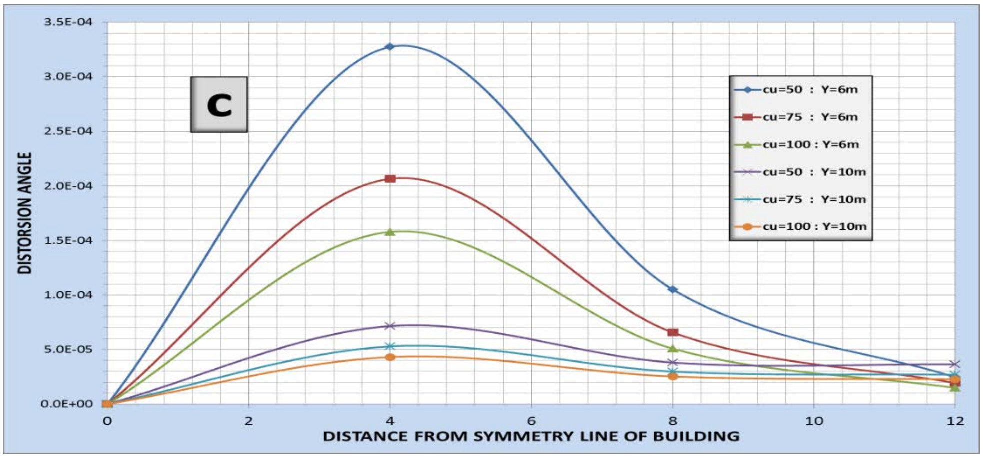


Figure (13) Variation OF Distorsion Angle for different soil types & depth and location of tunnel
a) Tunnel depth =-6m : b) Tunnel depth=10m : c) Total

CONCLUSIONS:

Based on the results obtained from thr finite element program "ANSYS" the following points are obtained:-

1.The effect of shallow tunneling is to increase the vertical settlement of the soil supporting the super structure regardless of the position of the tunnel.

2.The most critical position for the tunnel is below the footing center line , this location gives the largest expected settlement.

3.The presence of tunnel will produce differential settlement except when the tunnel lies below the center line of the structure.

4.The effect of tunneling is more obvious when the soil is weaker in properties.

5.The values of distortion angle for the super structure are within the acceptable limits permitted by the "ACI CODE" for the tunnels deeper than two times the strip footing width.

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