Stress Analysis Around Tunnels During Construction Stages Using Finite Elements Method

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ABSTRACT

In this study, the stresses around a tunnel during construction stages are discussed. For this purpose, the finite element method (FEM) was adopted as an effective approach to analyze the problem using (SIGMA/W) program.

The research includes the study of the behavior of soil due to excavation of tunnel by calculating the displacements and stresses in three positions of tunnel (crown, wall, and invert) during the various stages of construction.

The finite element analyses were carried out using (Elastic- plastic) and (linear elastic) models for the soil and the concrete liner respectively. From the results, it can be noticed that increasing the number of excavation stages (using six stages) decreases the displacement comparing with excavation using one stage.

Keywords: Tunnels, Construction stages, Stresses, Finite elements.

تحليل ألاجهاد حول ألانفاق أثناء مراحل إلإنشاء بإستخدام طريقة ألعناصر ألمحددة

الخلاصة

في هذة الدراسة تم تحليل الاجهادات خلال مراحل الانشاء المختلفة لنفق. تم إعتماد طريقة العناصر المحددة (FEM) كمنهج فعال لتحليل تصرف النفق باستخدام البرنامج (SIGMA/W).

تضمن البحث دراسة سلوك التربة نتيجة حفر النفق عن طريق حساب الازاحات و الاجهادات في مواقع ثلاثة (Crown, Wall, Invert) من النفق خلال المراحل المختلفة من إلإنشاء.

أجريت التحليلات بإستخدام العناصر المحددة باعتماد نموذجين (اللدن مرن) و (المرونة الخطية) لتمثيل التربة و مادة بطانة النفق على التوالي .النتائج اظهرت ان زيادة مراحل الحفر (استخدام ستة مراحل) يقلل الازاحة مقارنة بالحفر في مرحلة واحدة.

INTRODUCTION

A tunnel is an underground structure that passes beneath the ground. The first tunnel was built by the Babylonians about 2180 B.C and passed under the Euphrates River (Susa, 1986). Tunnel may be constructed for many purposes, such as carrying traffic as railway tunnels and highway tunnels. It may be constructed

for conveying utilities such as water supply tunnels and sewer tunnels (Hummadi, 2001). The necessity of these constructions in urban areas has brought about the need for an efficient and safe method for the deep excavations without severely affecting the adjacent structures. In general, excavation refers to the removal of a material within certain specified limits, for construction purposes (Obaid, 2001).

The stresses and the displacements in surrounding soil and tunnel lining depend not only on the soil mass properties and the in situ stresses field, but depend also on the type and stiffness of the lining (Brown, 1983).

PREVIOUS STUDIES ON TUNNEL AND UNDERGROUND OPENING

Karim, (1994) evaluated the stresses and deformations associated with semicircular tunnel underlain by rigid base and surrounded by sandy soil with static loading conditions using finite element method. The study clarified that the soil stiffness increases with increasing angle of internal friction and causes a reduction in both stresses and deformations of the tunnel. An increase in the modulus of elasticity of the tunnel material was found to reduce the deformations of tunnel and increases in stresses.

Dasari et al., (1996) used a finite element program to model the New Austrian Tunneling Method (NATM). The non linear behavior of London clay was modeled by a strain dependent and modified Cam-clay model and the tunnel lining was modeled by constant time-dependent elastic model. The construction process was modeled in two and three dimensions and by removing the elements in sequence. The results obtained from the plane strain and three dimensional analyses are compared to assess the importance of arching of soil ahead of tunnel face. Two dimensional analyses were carried out using plane strain model, type of construction technique was modeled:-
The lining was constructed sequentially as:-
  i.   Excavation of top.
  ii.   Installation of lining of top half.
  iii. Excavate bottom half of the tunnel.
  iv.   Installation of lining of the bottom half.

The conclusion is that used; represent the real construction of the tunnel. There was a significant reduction in ground movement due to an initial increase in stiffness after which there seem to be little effect; this means that there is an upper limit of the tunnel stiffness (or lining thickness). For a given soil, the construction sequence has a significant effect on the predicted ground movements, which is more important than the eventual stiffness.

Kasper and Meschke, (2004) presented the simulation of a tunnel advance in soft cohesive soil below the ground water table using a three-dimensional finite element simulation model for shield-driven tunnel excavation. A Cam-Clay plasticity model is used to describe the material behavior of cohesive soils.

Moller and Vermeer, (2008) suggested the excavation of the tunnel causes relaxation of in situ stress, which is only partially restricted by the insertion of the tunnel support. In fact it is not possible to create a void instantaneously and provide an infinitely stiff lining to fill it exactly. Hence, a certain amount of the deformation of the ground will take place at the tunnel depth; these will a chain of movements, resulting in settlements at the ground surface, which are more significant at shallow tunnel depth.

Salim and Gell, (2011) studied the capabilities of a finite element method for analyzing the influence of a twin- tube highway –tunnel on the stability of a nearby water –transmission tunnel by using the computer program (SIGMA/W). To design tunnel meets existing underground structures in a smaller distance, they found that the maximum increase of stresses occur where the horizontal distance of the tunnel axis reach (14m). Great vertical distance (14m) between the underground openings is shown in Figure (1).
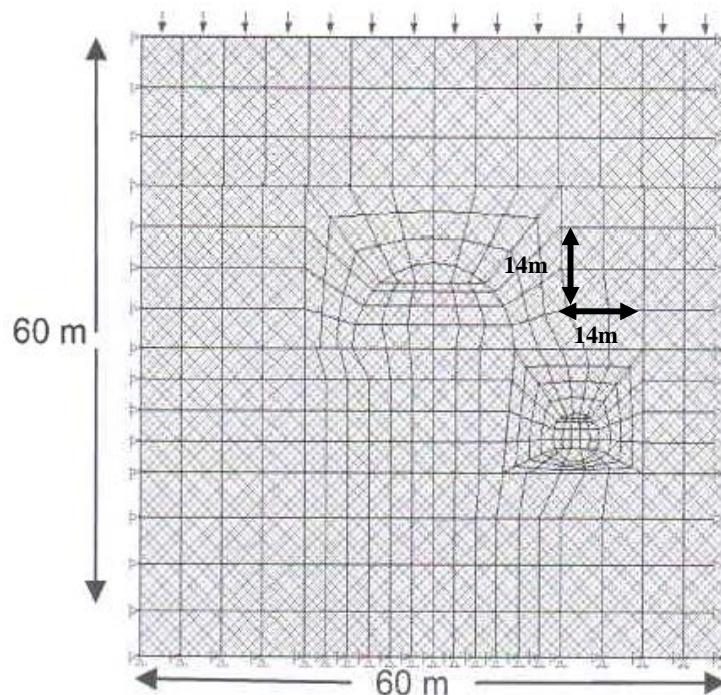

Figure (1) Finite element method, boundary conditions, contours of underground openings (Salim and Gell, 2011).

DESCRIPTION OF THE PROBLEM

The basic problem being analyzed is shown in Figure (2). It consists of construction of 10m diameter circular tunnel and 0.25m thickness of concrete lining. The depth of the tunnel is 15m below natural ground level. The tunnel is to be constructed in a C-□ soil stratum. Excavation of tunnel is carried out through 6 stages. These stages include excavation of each stage and lining it as shown in Figure (3).
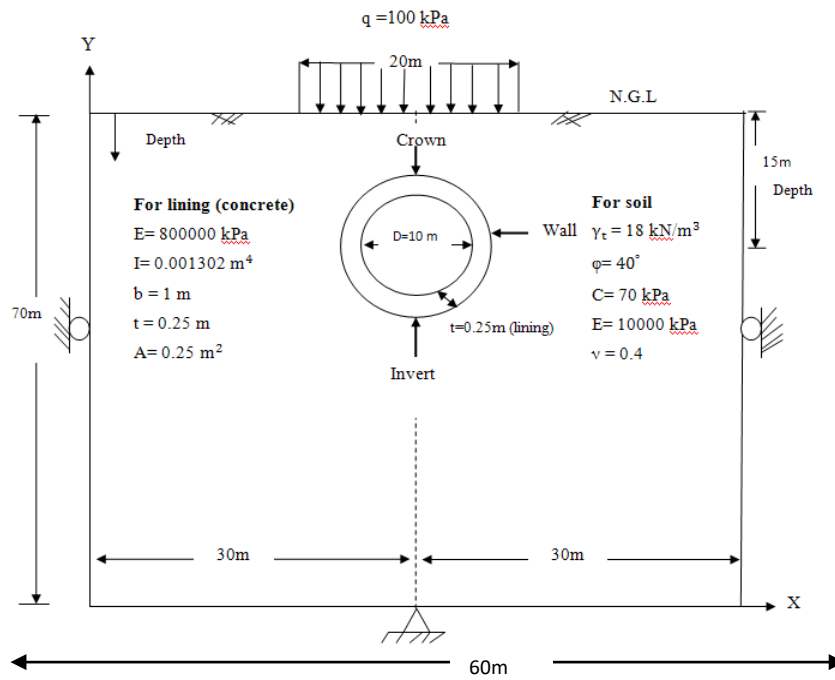
q =100 kPa

20m

N.G.L

Y

Depth

Crown

15m

Depth

For lining (concrete)

E= 800000 kPa

I= 0.001302 m⁴

b = 1 m

t = 0.25 m

A= 0.25 m²

D=10 m

Wall

t=0.25m (lining)

For soil

γ_t = 18 kN/m³

φ= 40°

C= 70 kPa

E= 10000 kPa

ν = 0.4

70m

Invert

30m   30m

X

60m

Figure (2) Soil profile, and soil and concrete lining properties of the basic problem.

q =100 kPa

20m

N.G.L

Crown

Depth

15m

Depth

Stage 2   Stage 1

Wall

Stage 4   Stage3

70m

Stage 6   Stage5

Invert

30m   30m

X

60m
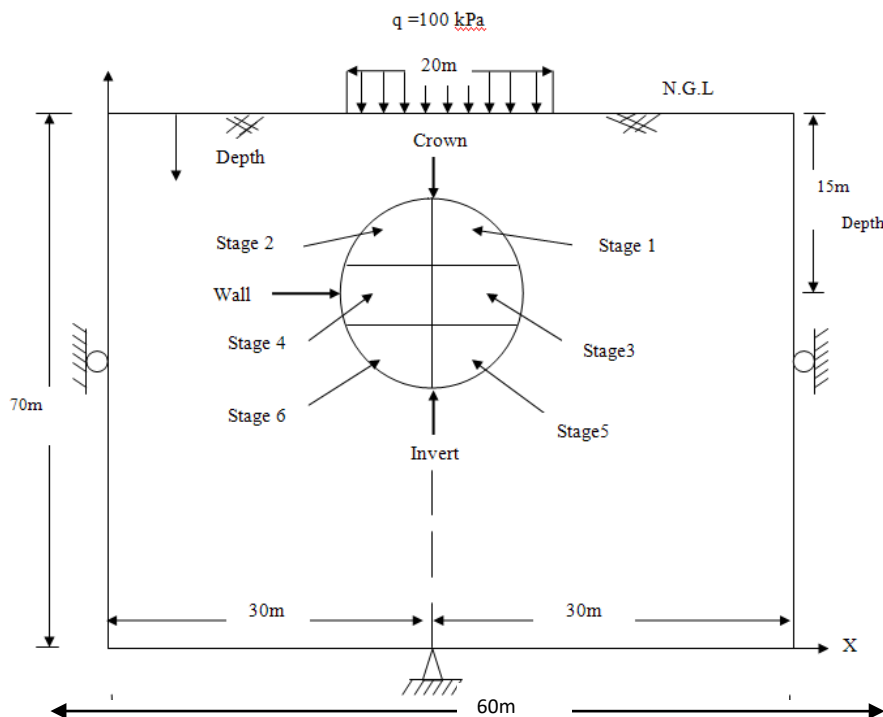
Figure (3) Excavation stages used in the analysis of basic problem.

The finite element mesh used in the analysis of the basic problem is shown in Figure (4). The lining is presented as a (Beam element).   The excavation process is assumed to occur under plane strain conditions (i.e. the strain normal to plane of the mesh is zero). Figure (5) represent the location of points at along which the displacements and the stresses are selected and plotted.
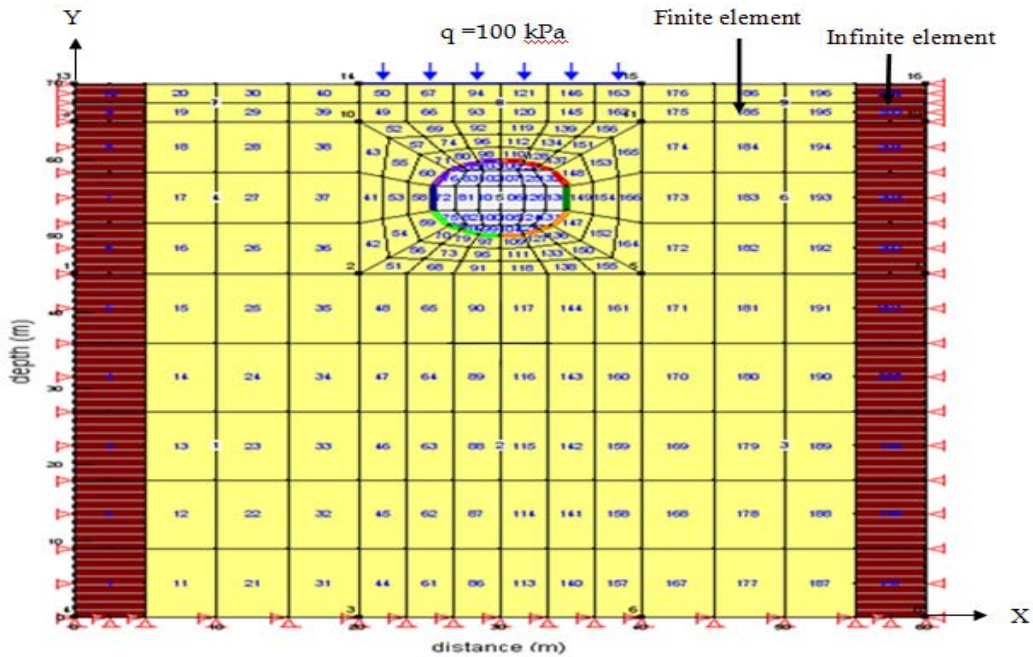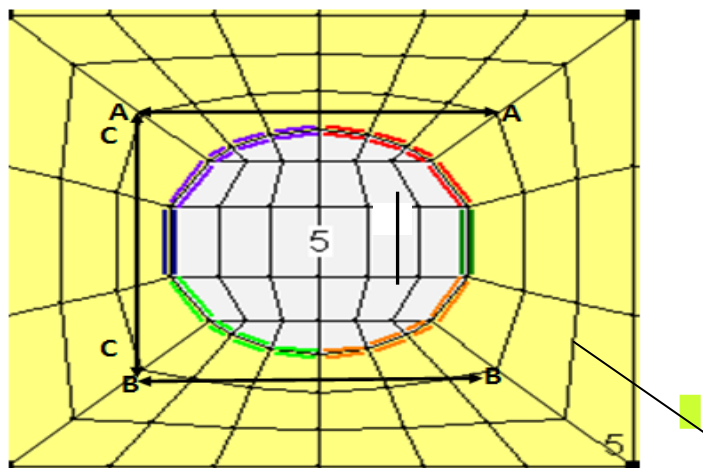

Figure (4) Finite element mesh for the basic problem.

Section A-A represents the points on the crown.

Section B-B represents the points on the invert.

Section C-C represents the points on the wall.

Figure (5) Location of points of interest in the tunnel.

MATERIAL CHARACTERIZTION

The tunnel is surrounding by a C-☐ soil with a coefficient of earth pressure at rest, K◦ equal to 1. Two constitutive models are used, the soil material is assumed to follow elastic-plastic behavior and Mohr-Coulomb failure criterion, while the concrete lining is assumed to behave linear elastic with constant modulus of elasticity and Poisson's ratio. The parameters used in the analysis of the basic problem are presented in Table (1). The lining process is represented as a (Beam element) which is a function of the cross-sectional area, the moment of inertia, the activation stage, and the modulus of elasticity the value of which is obtained using the following equation:-

$$E = 4700\sqrt{f_{c'}} \cong 800000 \text{ kPa (ACI-318, 2005)} \qquad ...(1)$$

Where:-

E: is the young's modulus of elasticity.

$f_{c'}$: is the compressive strength of concrete = 30000 kPa.

$$I = \frac{bh^3}{12} \qquad ...... (2)$$

Where:-

I: is the moment of inertia of the lining beam.

b: is the width of the lining.

h: is the thickness of the lining.

Table (1) Engineering properties of soil and lining used in the present study
(After Metro Baghdad at Bab Al-Muaddam substation)

Parameter	Value	Unit
Tunnel depth(H)	15	m
Tunnel diameter(D)	10	m
Coefficient of lateral earth pressure (K)	1*	-
Modulus of elasticity of soil(E)	10000	kPa
Poisson's Ratio of soil(v)	0.4	-
Cohesion of soil (C)	70	kPa
Angle of internal friction (☐)	40	-
Unit weight of soil (γ_t)	18	kN/m^3
surcharge	100	kPa
Lining thickness(t)	0.25	m
Modulus of elastic of lining(concrete)	800000	kPa
Moment of inertia of lining	0.001302	m^4
Cross-section area of lining	0.25*1	m^2
Number of excavation stages of tunnel	6	-
Type of boundary element	Infinite element	

* assumed.

RESULTS

Settlement due to tunneling

The magnitude of soil movements generally depends on the soil type, construction method, tunnel dimensions, and location. Ground movements around tunnels lead to surface subsidence. The magnitude and distribution of settlement were studied extensively by various researchers (Schmidt, 1969, Clough and Schmidt, 1981, O'Reilly et al., 1982, Rankin, 1988, and Mair and Taylor, 1993).The deflected shapes of excavated face and the vectors during the six stages of construction of the tunnel of the present study are shown in Figures (6) to (8). These figures show the accumulated deformation pattern and displacement vectors for different construction stages. It can be seen that the soil tends to move toward the excavation, and that the maximum displacement occurs within the zone of excavation. The displacements decrease gradually when moving away from the excavation zone. Figure (6) shows that the maximum recorded displacement is (163.7mm) due to the excavation of stage one.   It is found that due to excavation of stage four that the maximum displacement is (290mm) as shown in Figure (7), while Figure (8) shows the maximum displacement reach to (285mm) due to excavation of stage six. Table (2) summarizes the maximum displacement at different construction stages.

Table (2) Maximum displacement at different construction stages

Stages	Maximum displacement (mm)
1	163.7
4	290
6	285

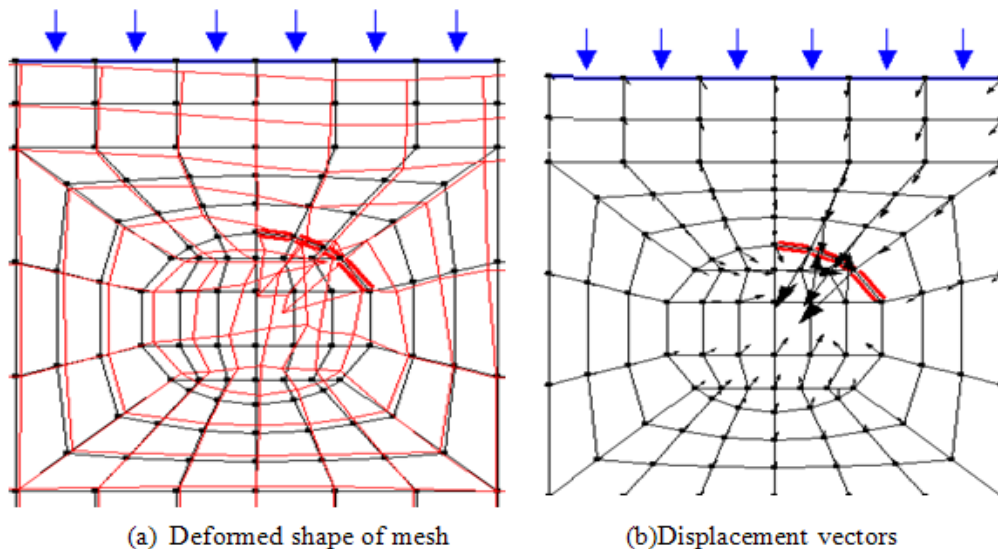(a) Deformed shape of mesh   (b)Displacement vectors

Figure (6) Deformed shape of mesh and displacement vector due to excavation of stage one.

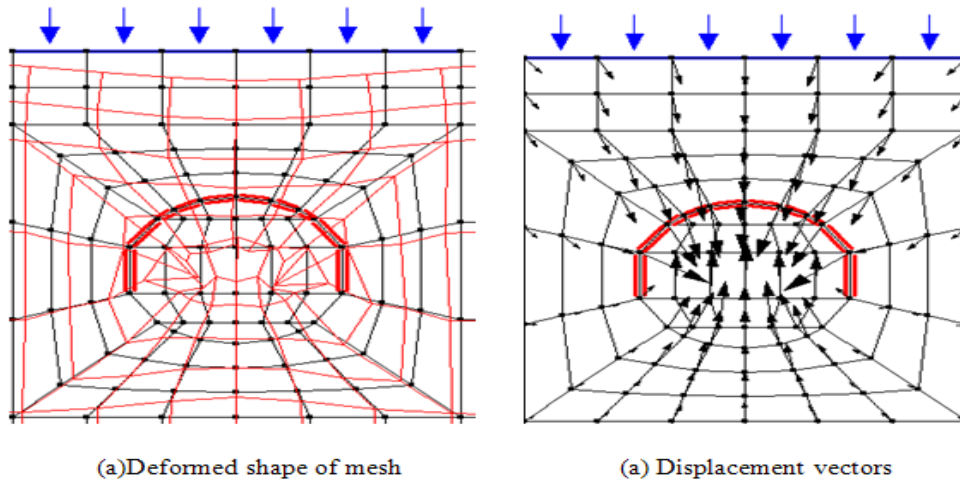(a)Deformed shape of mesh   (a) Displacement vectors

Figure (7) Deformed shape of mesh and displacement vectors due to excavation of stage four.

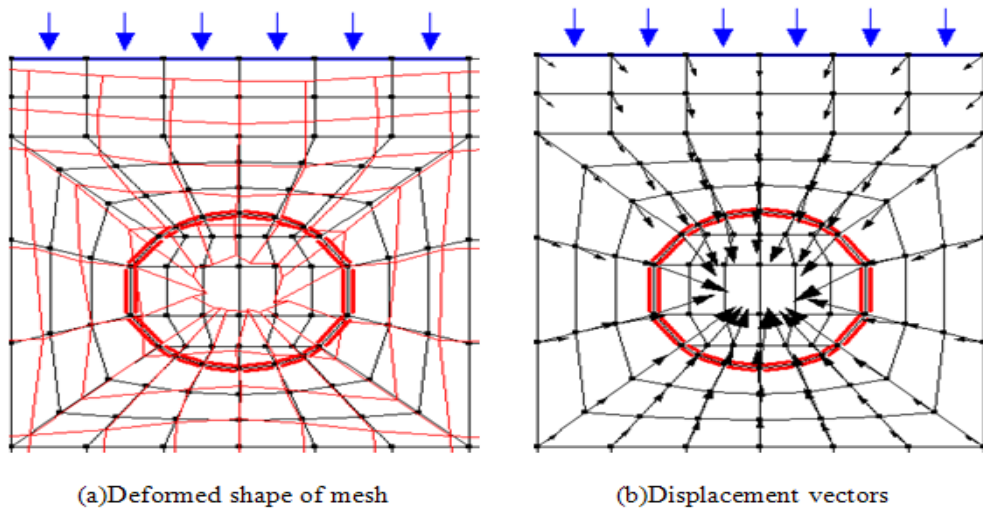(a)Deformed shape of mesh   (b)Displacement vectors

Figure (8) Deformed shape of mesh and displacement vectors due to excavation of stage six.

The vertical displacement contour lines of the basic problem at the end of the sixth (final) stage of excavation, is shown in Figure (9). The contour values of vertical displacement concentrated with higher values above the crown and decrease as going far from the excavation zone.
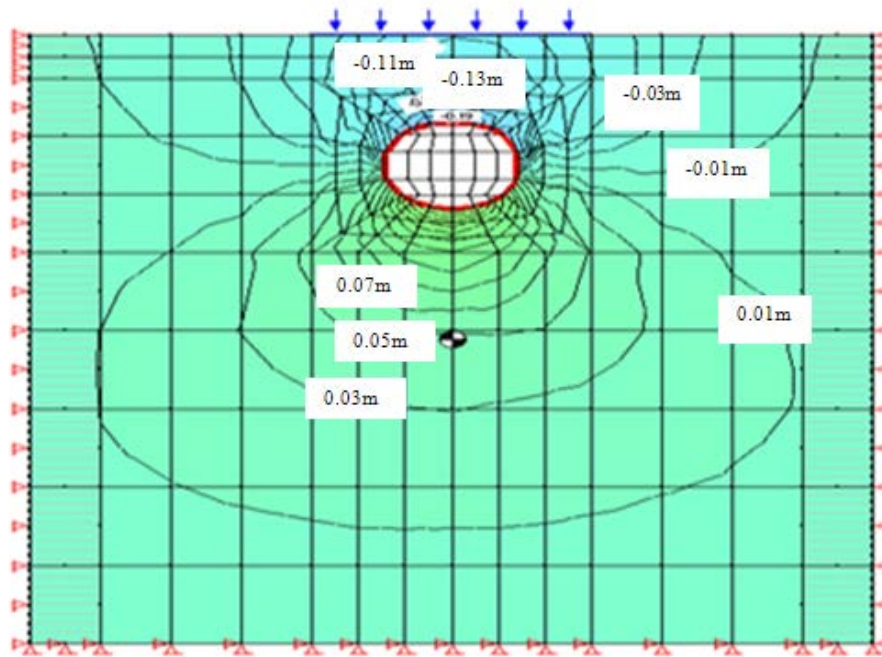
: This symbol related to the program when using infinite element.

Figure (9) Contour lines of vertical displacement after full excavation (stage 6) of the tunnel.

Stress changes around the tunnel

The variations of the stresses for the specific points are shown in Figures (10) to (12). The points were selected at the tunnel crown, invert, and wall. At the crown and the invert excavation induces a reduction in the radial stress (σ_v) and small increase in the tangential stress (σ_h) as shown in Figures (10 a, b and 12 a, b) which causes the radial stress to become the minor principal stress (σ_3) and the tangential stress to become the major principal stress (σ_1). At the wall, excavation induces a reduction in the radial stress (σ_h) to become (σ_3), and an increase in the tangential stress to become (σ_1). This means that the principal stresses do not change at the wall as shown in Figure (11 a, b). These results agree with the finding of (Lee and Rowe, 1989). From this, the conclusion drawn is that the stress above and below the tunnel corresponds approximately to triaxial extension and that the strength and deformation profile of these regions can be estimated from triaxial extension tests, while along the wall, the stress is intermediate between the triaxial compression and extension. For this reason, the modulus above the crown and below the invert is more critical.
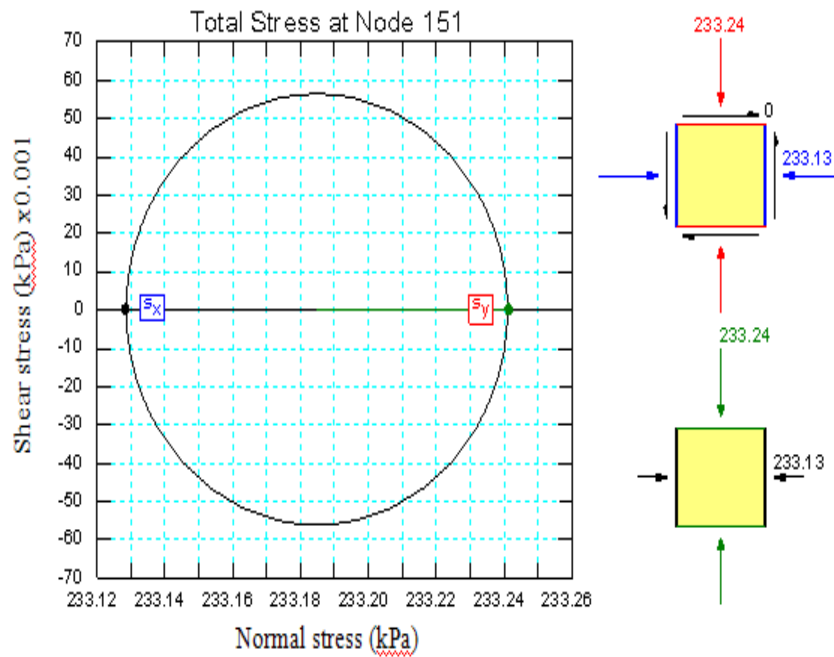
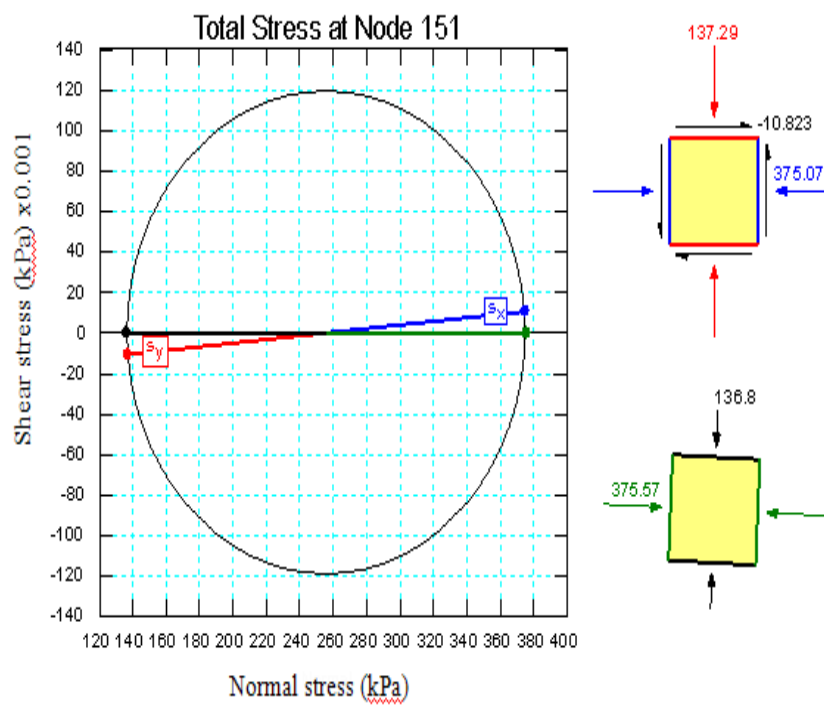**Figure (10a) Mohr circle on the node151 at the crown before tunnel excavation.**

Figure (10b) Mohr circle on the node 151 at the crown after tunnel excavation.
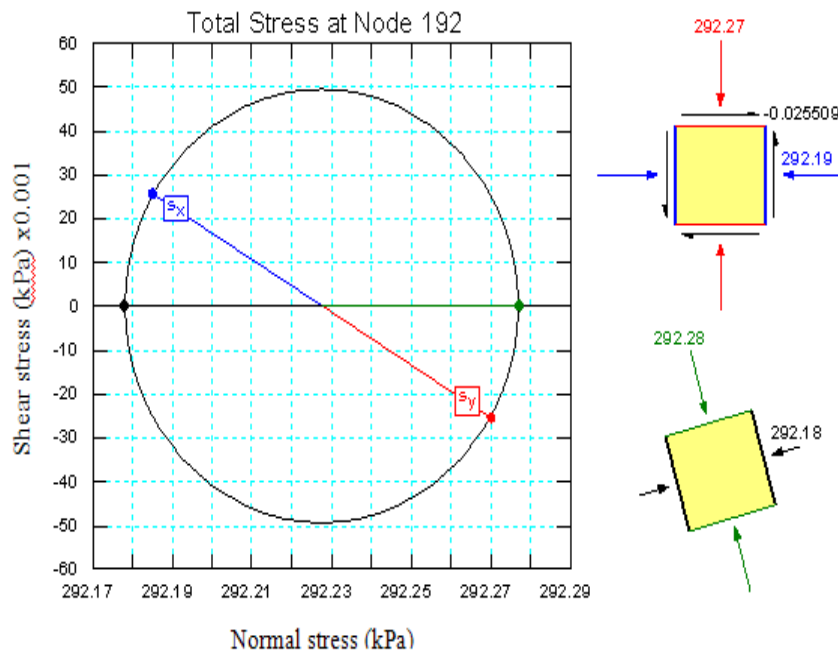
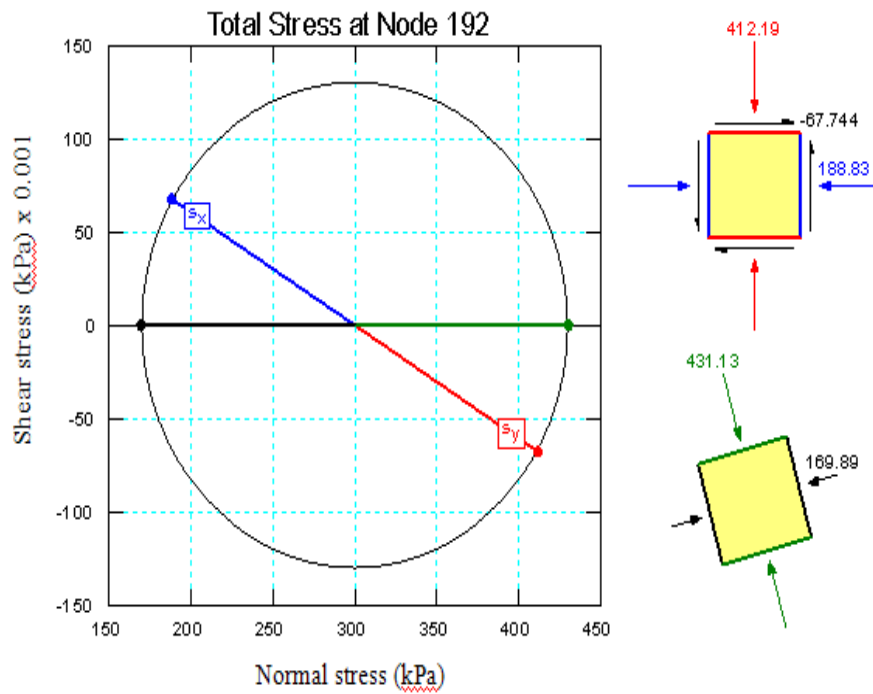**Figure (11a) Mohr circle on the node 192 at the wall before tunnel excavation.**

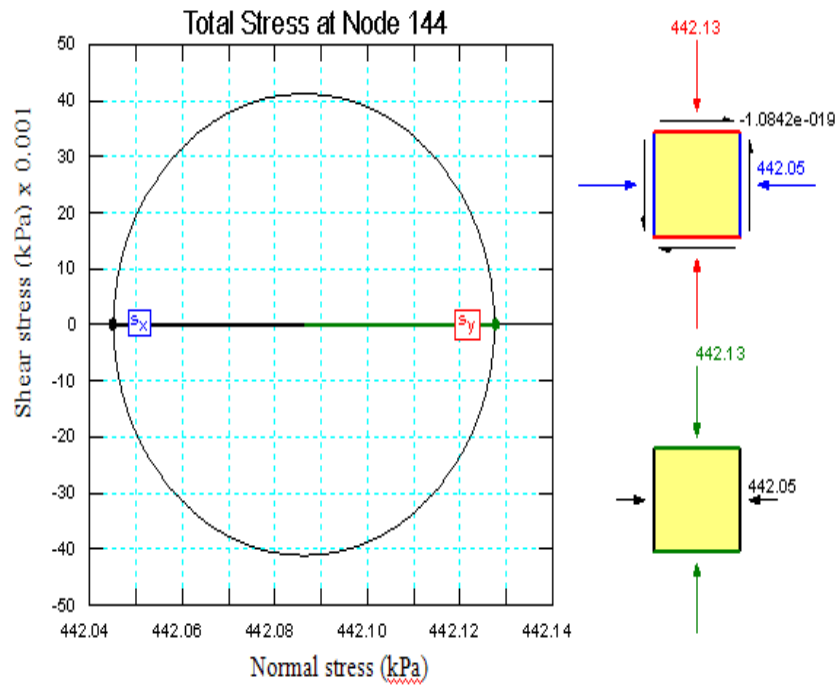**Figure (11b) Mohr circle on the node 192 at the wall after tunnel excavation.**

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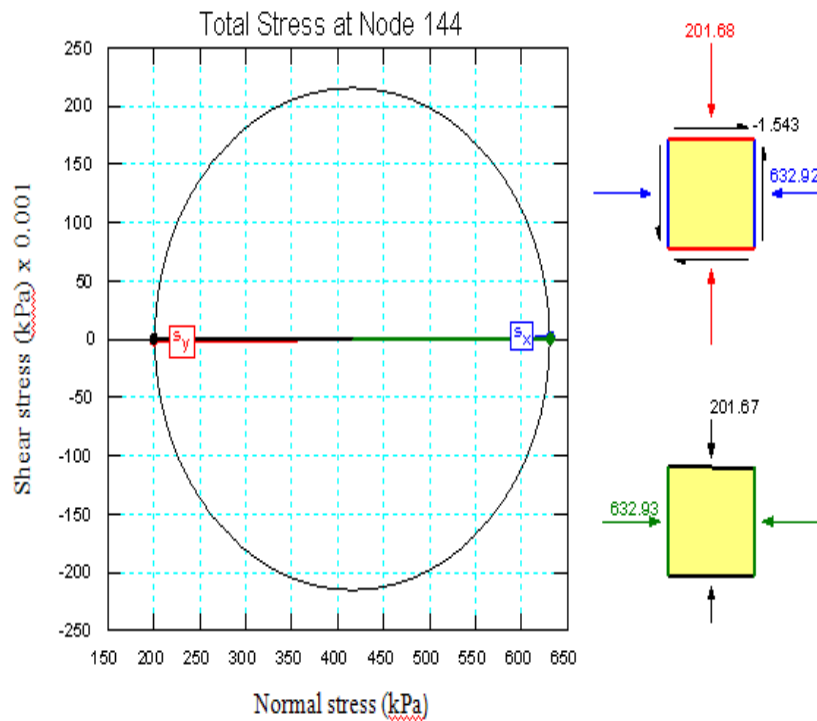**Figure (12a) Mohr circle on the node 144 of at the invert before tunnel excavation.**

Figure (12b) Mohr circle on the node 144 at the invert after tunnel excavation.

Figure (13) shows the vertical stress contour lines after completion excavation of the tunnel. It is clearly shown that the contour values of vertical stress increases with depth, but the higher values concentrate near the tunnel wall. These contour values represent the effect of surcharge plus overburden pressure.
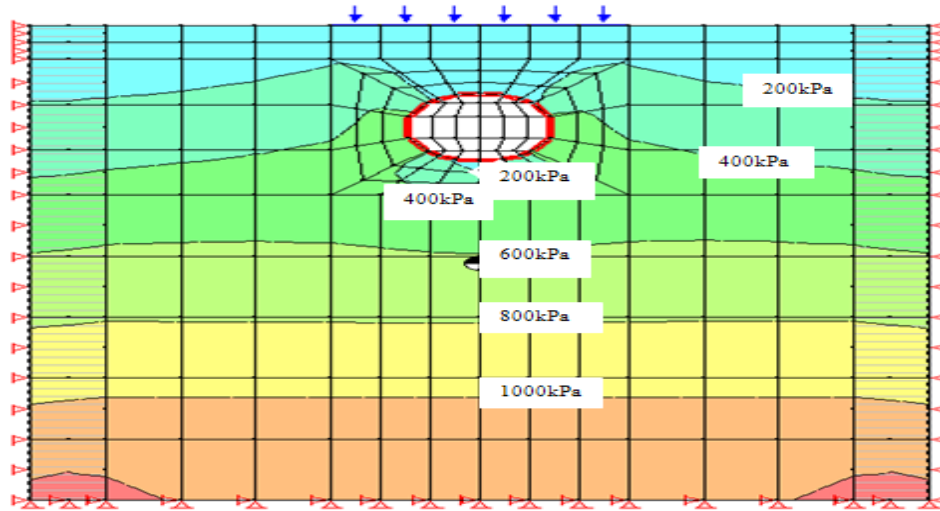

Figure (13) Contour lines of vertical stress after excavation of the tunnel.

From Figure (14) it can be noticed that during excavation stage one and two, the effect of the lining seems to be negative, in other word, the weight of the lining leads to increases the amount of displacement within these stages, while in the third stage, lining the tunnel provide strengthen and stability to the inner surface of hole. This technique reduces the displacement and makes the stress distribution uniform.

A negative movement means an inward movement towards the tunnel's center as shown in Figure (14), while a positive value of vertical movement means upward movement towards the tunnel center as shown in Figure (16).
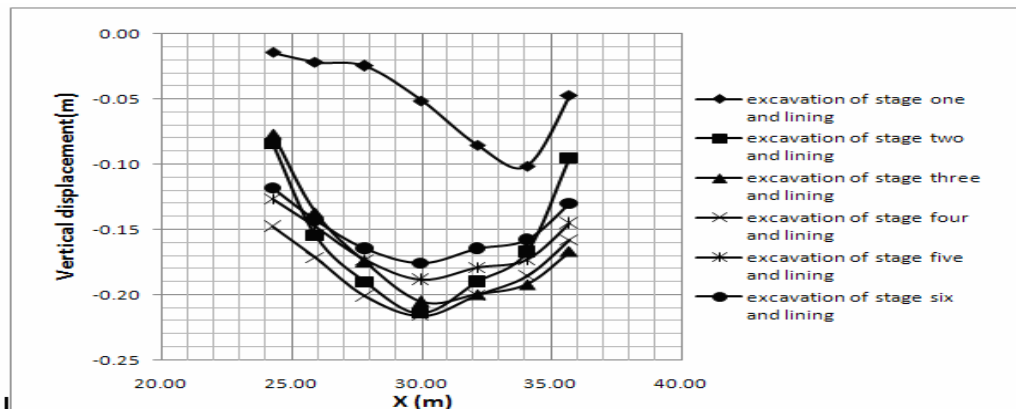

**Figure (14) Variation of vertical displacement throughout the tunnel excavation
at the crown.**

Within the crown zone the stresses above the tunnel shown in Figure (15) increase when progressed excavation stages. The soil above the tunnel cause the stresses concentrate at contact surface with tunnel; the lining works to provide more stability and to prevent collapse of the tunnel. Also, positive stresses are assumed (program SIGMA/W assumption) compressive stresses shown in Figure (15) and (17).
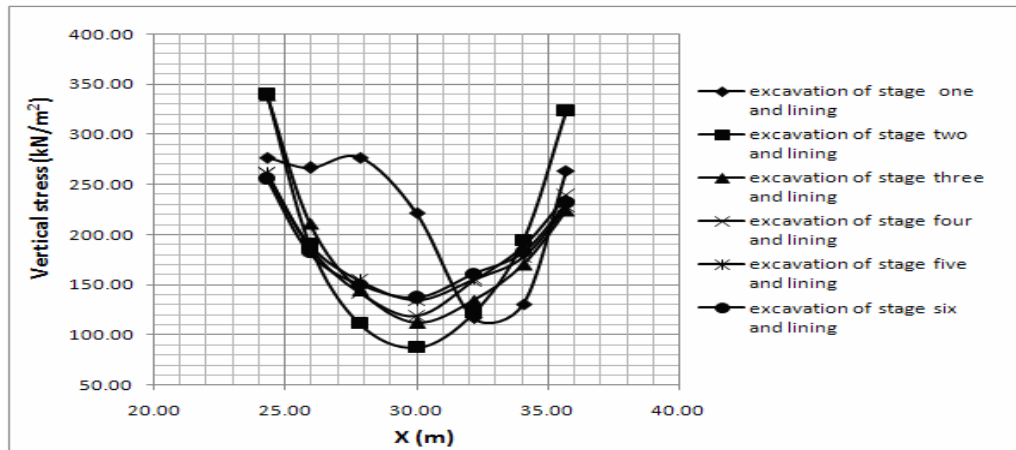

Figure (15) Variation of vertical stress throughout the tunnel excavation at the crown.

It can be seen from Figure (16) that there is an upward movement at the invert which increases with the excavation stages. Comparing Figure (14) and (16) it can be noticed that there is an inward movement towards the center of the tunnel for all points above the crown, while an upward movements are noticed at the invert, for all points below the centerline of the tunnel, this is due to relief in the stresses as a result of excavation of tunnel.
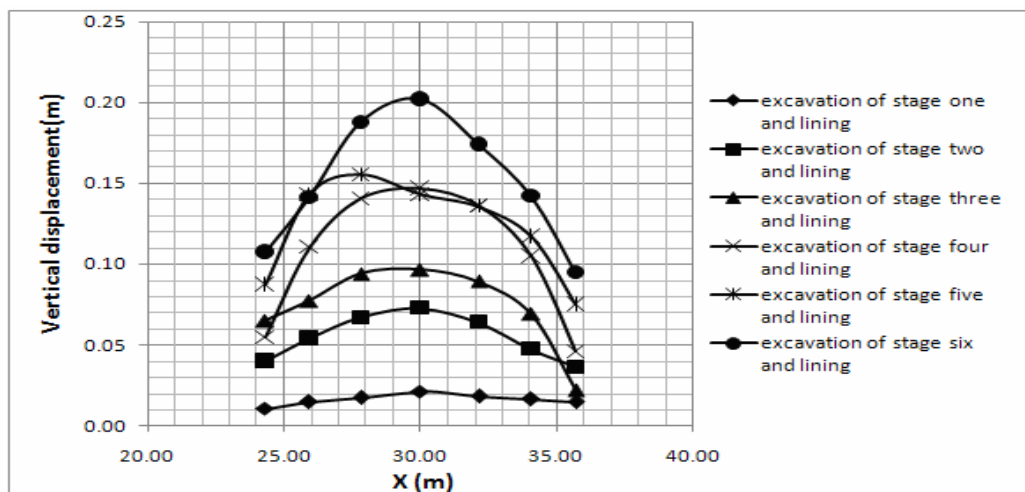

Figure (16) Variation of vertical displacement throughout the tunnel excavation at the invert.

From the comparison between the stresses along the bottom of the tunnel as illustrated in Figure (17), it is obvious that the vertical stresses are reduced during excavation of the tunnel leading to decrease the stresses in conjunction point between soil and tunnel.

The variation of horizontal displacement along the vertical line is shown in Figure (18).The horizontal movement increases with increase in excavation stages and its reaches to a maximum value at stage six which approximately (-0.15m) and this occurs at center of tunnel. This increases in horizontal movement due to increase in confining pressure.
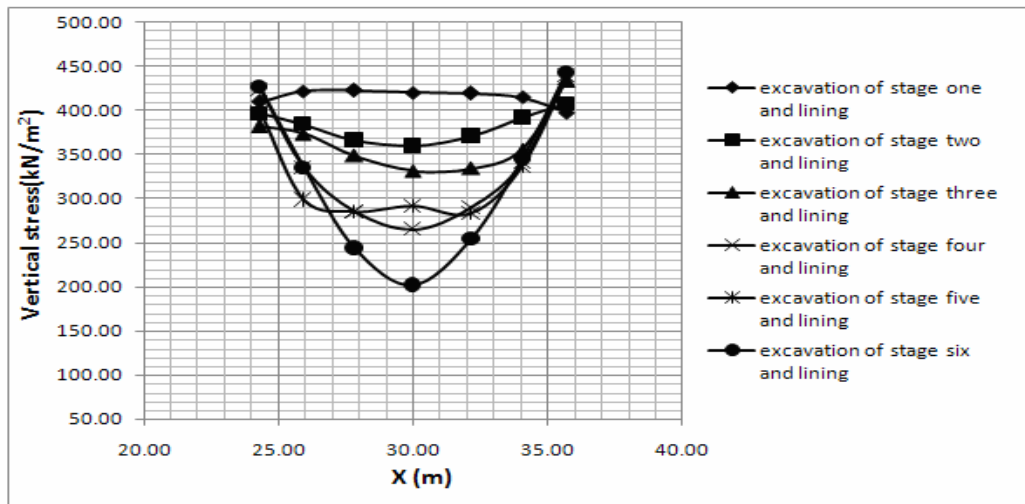

Figure (17) Variation of vertical stress throughout the tunnel excavation at the invert.
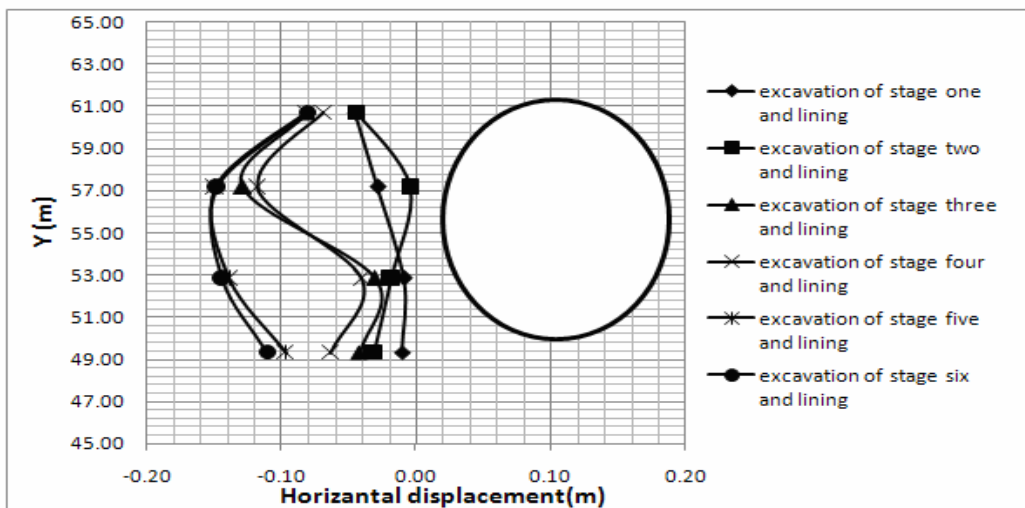

Figure (18) Variation of horizontal displacement throughout the tunnel excavation at the wall.

From Figure (19), it is obvious that the value of the horizontal stress increases with the increase in the depth, this is due to increase in the confining pressure.
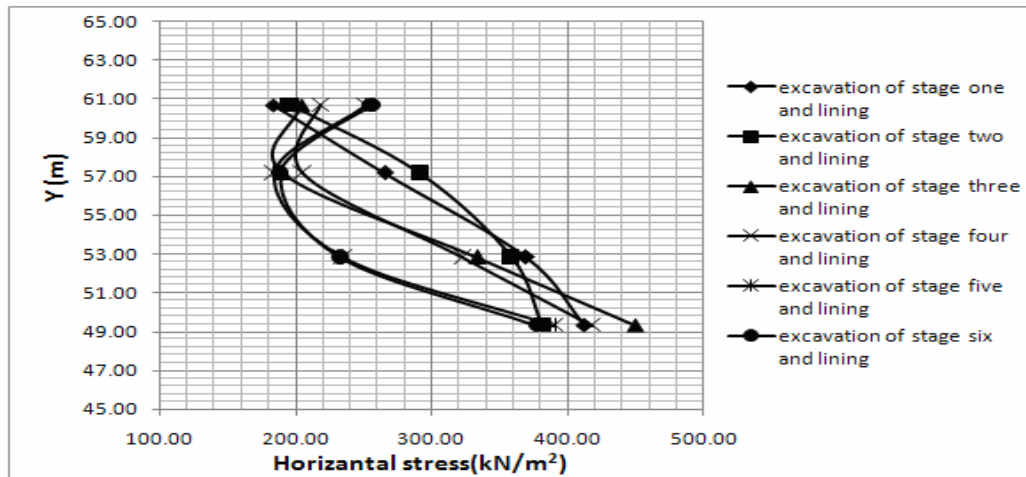

Figure (19) Variation of horizontal stress throughout the tunnel excavation at the wall.

CONCLUSION

From this study the following conclusions can be made:-
1. Increasing the number of excavation stages decreases the vertical displacement above the crown and increases the vertical stress, while the vertical displacement at the invert increases toward the center of the tunnel and the vertical stress decreases.
2. It can be concluded that, the use of an infinite element has no effect on the boundary conditions, and its presence just to extend the range of the calculated displacements.
3. In general, tunneling process causes moving all the points surrounding the tunnel towards the center of the tunnel. This leading, the stresses below and above the tunnel to be changed. This may be referred to the reduction in vertical stresses caused by tunneling.
4. The horizontal displacement along the vertical line increases with depth reaching to the maximum value (-0.15m) at the center of the tunnel.

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