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Numerical Analysis of Tunnel Design Effect of Excavation Damaged Zone on Principal Stresses and Total Displacement

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Abstract:
The development of Excavation damaged zone (EDZ) in an underground opening can significantly affect the overall performance of the opening. Generally, it is believed that the presence of such zone in any kind of underground opening can pose big problem to seepage flow, stability and subsequently impair the general performance and functionality of the excavation. In order to assess the influence of EDZ on excavation, the behavior of the principal stresses and the total maximum displacement was observed in the present study. Therefore, this study is aimed at numerical design of underground tunnel and determining the effect of EDZ on the principal stresses (sigma 1 and sigma 3) and total maximum displacement surrounding the excavation. The principal stresses (sigma 1 and sigma 3) were observed to decrease with increasing EDZ thickness and corresponding increase in elastic modulus, while the shear stress was observed to increase. The total maximum displacement was also observed to decrease with increasing EDZ thickness and corresponding increase in permeability. The general conclusion drawn from the analysis is that mechanical properties principal stresses and (total maximum displacement) decrease with the development of EDZ around the tunnel.

Keywords: *Excavation damaged zone, principal stresses, total displacement*

1. Introduction

1.1. An Overview of Tunnel Design

Due to unavailability and prohibitive expense of ground road networks or rail links in the world's cities, the assignment of transport in tunnels is increasingly becoming popular. Surface disturbance particularly settlement is eminent when underground tunnels are being constructed. Such disturbances can cause significant damage in areas where structures are present. The availability of reliable predictions of potential harmful effects would usually determine the acceptance of tunnel construction in an urban area (Augarde et al., 1997).

The construction of tunnels can virtually be carried out in any type of ground. In rocks, tunneling usually leads to disturbance of surface and possible damage which is normally due to collapse rather than from a continuous deformation if it were to be in soft ground. (Augarde et al., 1997).

The excavation of tunnels within rock masses is quite an arduous and complex process. This is a subject of concern that has not been resolved satisfactorily up to date as a result of the difficulty that the geotechnical account of the soil portrays and the fact that no mathematical model has been found that will be able to simulate all of its complexity (Serrano et al., 2011).

The recognition of the significance of ensuring stability while tunnel driving and controlling deformations on neighboring structures is very important in tunneling. Conventional methods of analyses like stress fields, closed form solutions, plastic analysis, limit equilibrium cannot be used to readily assess ground movement due to the complications in geometry, sequence of construction, soil behavior and stratification which are frequently encountered in the field. In such cases, numerical analysis such as finite element method is frequently employed (Suzuki et al., 2008)

The use of numerical analysis in tunnel design has been on the increase. Much attention should be given to the program selection and simulation process when using numerical techniques in tunnel design so as to capture the fundamental stress regime and the probable ground failure mechanisms. Tunnel behavior and stress redistribution are usually controlled by the existence of geological structures and the 3-D excavation geometry for near-surface tunneling having low in situ field stress (Ghee et al., 2010). In the present study, the finite element program (phase 2) was used to model and analyze the underground tunnel and to investigate the effect of reducing the thickness and permeability of the damage rock zone (DRZ) that may arise as a result of the excavation of the tunnel. Excavation damage zone (EDZ) is a region in which geochemical and hydro mechanical alterations prompt substantial fluctuations in flow and transport properties (Tsang et al. 2005). This zone is sometimes called the disturbed rock zone (DRZ). Small cracks, a redistribution of stresses and repositioning of rock structures will take place in this zone, subsequent extreme variations of permeability to flow, chiefly through the fractures and cracks developed as a result of the excavation (Zhu et al., 2007).

The creation of an EDZ is anticipated around all artificial openings in civil engineering (like transportation tunnels), in underground mining (like stopes) and in petroleum engineering (like borehole). It is considered that there are three essential processes that may aide the development of EDZ around an underground opening; excavation procedure damage, stress redistribution, and weathering or interaction between rock and groundwater. Wide experimental investigations have been designed to deal with the problem of comprehending and predicting the extent of EDZ in previous work. Centered on the in-situ measurements, it is largely accepted that their distribution of excavation-induced stress has greater influence in controlling the extent of excavation disturbance than the excavation method (Zhu et al., 2007).

The instrumentation records comprise unique deformation signatures that provide acknowledge into the mechanical response of rock mass town loading and the development of an EDZ; however, due to limit of tested in-situ records, it is typically challenging to make clear the associated mechanism that is accountable for the development of the EDZ. Therefore, it is of excess worth to develop effective theoretical or numerical models that will be able to capture the damage advancement under the coupling of hydromel chanical conditions, so as to completely label the spatial and temporal development of the EDZ (Zhu et al., 2007).

1.2. Aim of Research

This study is aimed at investigating the effect of excavation damage zone on seepage flow. In order to achieve the aforesaid aim, certain objectives must be attained. These include

- Choice of geometry
- Stage excavation
- Specification of support system: rock bolts and liners
- Stage application and support capacity check
- Seepage analysis

1.3. Scope of Work

The numerical modeling and the analysis were carried out using the finite element program Phase 2. This study is restricted to tunnel design in constant stress field. It is limited to changes imposed on the hydraulic properties on the excavation boundary by the excavation damaged zone. The excavation of the tunnel is a phased rather than the full-face excavation. The whole finite element analysis is a multistage analysis. However, the tunnel is excavated in three parts in the first three stages of the analysis. The application of support will go on concurrently in stage 1as the tunnel is being excavated and finally the tunnel will be fully supported in the last stage of the analysis. The stress and groundwater seepage analyses will be carried out and by varying the thickness and permeability of the EDZ, the variability in pore pressure distribution, hydraulic gradient distribution and discharge velocity distribution around the tunnel will be observed.

1.4. Problem Statement

As the need for underground constructions keeps increasing, so also does the demand for safe construction design as well as performance and functionality increases.

The excavation of underground tunnels often results in the formation of EDZ due excavation damage. Weathering and stress distribution may also contribute to the formation of EDZ. Therefore, since EDZ is usually expected from underground opening, and it is not very clear how it affects the stability and performance of the openings, the damaged zone can be better justified with better understanding of its behavioral effects on the hydraulic properties.

1.5. Hypothesis

The EDZ has significant effect on seepage flow. When the support system is added to the tunnel, it is anticipated that the distribution of yielding depth zone around the tunnel will change (reduce to be specific) thereby leading to a subsequent decrease in permeability of the EDZ since smaller yield zones will subsequently have smaller cracks and thus lesser permeability. As this happen, it is expected that pore pressure around the tunnel will be minimized and the discharge velocity around the tunnel will subsequently reduce.

2. Literature Review

2.1. Assumptions

The following assumptions are made and applied in the study where necessary:

- Yield zone, plastic zone, damaged rock zone and zone of weakness are all assumed to mean the excavation damaged zone in this study.
- The surrounding rock mass is assumed competent with little surface stain and stress problem.
- Due to the stress problem, mild to major rock burst is anticipated.
- Some adjustments will be made to the specification of support system to make room for the anticipated rock burst.
- Any property; material property or strength property that has not been calculated is therefore deemed to be assumed from previous studies.

2.2. Rock Mass Characterization

There are many classification systems that are recognized worldwide today the Q index system developed by Barton et al., (1973); the rock mass rating (RMR) system developed by Bieniawski (1973); and the RMi developed by

Palmstrom (1995). These classification systems have a measurable approximation of the rock mass quality related to an empirical design rule to evaluate sufficient support measures for the rock (Stille and Palmstrom 2003).The Q system is applied in this study.

Barton et al (1974) suggested a quality index (Q) of tunneling for the characterization of rock mass and the determination of tunnel support requirements based on the assessment of huge number of case histories of underground excavations. The value of Q is defined mathematically by the following equation:

$Q = (\frac{RQD}{Jn})*(\frac{Jr}{Ja})*(\frac{Jw}{SRF})$ …………………….. (2.1)

RQD represents the rock quality designation
SRF represents the stress reduction factor
Jn, Jr, JaandJw represent the joint set number, the joint roughness number, the joint alteration number and the joint water reduction factor respectively
The rock tunneling quality Q is deemed to be a function of three main factors which are the basic measures of:

1. $\frac{(RQD)}{Jn}$ i.e. the Block size
2. $\frac{Jr}{Ja}$ i.e. the Inter-block shear strength
3. $\frac{Jw}{SRF}$ i.e. the Active stress

The tables below show the specifications of the parameters of these above-mentioned factors.

	Rock Quality Designation	RQD	Notes
A	Very Poor	0 - 25	1. Where RQD Is Measured As < 10, Q Is Evaluated From A Nominal Value Of 10. 2. RQD Intervals Of 5, That Is 100, 95, 90 Etc. Are Accurate Enough
B	Poor	25 - 50	
C	Fair	50 - 75	
D	Good	75 - 90	
E	Excellent	90 - 100	

Table 1: Rock Quality Designation Values for Various Categories of Rocks

	Joint Set Number	Jn	Notes
A	Massive, no or few joints	0.5 - 1.0	1. increase by a factor of 3 Jn for intersections i.e. 3*Jn 2. increase Jn by a factor of 2 for portals
B	One joint set	2	
C	One joint set plus random	3	
D	Two joint sets	4	
E	Two joint sets plus random	6	
F	Three joint sets	9	
G	Three joint sets plus random	12	
H	Four or more joint sets, random,etc.	15	
J	Crushed rock, earthlike	20	

Table 2: Joint Set Numbers for Various Categories of Rocks

	Joint Roughness Number	Jr	Notes
A	Discontinuous joints	4	1. If the mean spacing of the relevant joint set is > 3m, add 1. 2. Use Jr to be 0.5 for planar, slick sided joints withlineations, as long as the lineations are oriented for minimum strength.
B	Rough and irregular, undulating	3	
C	Smooth undulating	2	
D	Slickensided undulating	1.5	
E	Rough or irregular, planar	1.5	
F	Smooth, planar	1	
G	Slickensided, planar	0.5	
H	Zones containing clay minerals thick enough to prevent rock wall contact	1	
J	Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1	

Table 3: Joint Roughness Numbers for Various Categories of Rocks

	Joint Alteration Number	JA	φr degrees (approx.)	Notes
A	Tightly healed, hard, non-softening impermeable filling	0.75		In the presence of alteration products, the values of φr and the residual friction angle are used as an estimated guide to their mineralogical properties
B	Unaltered joint walls, surface staining only	1	25 - 35	
C	Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2	25 - 30	
D	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3	20 - 25	
E	Softening or low-friction clay mineral coatings, which is kaolinite, mica. Furthermore chlorite, talc, gypsum and graphite etc., and little quantities of swelling clays. Discontinuous coatings,≤ 1 – 2	4	8 - 16	
F	Sandy particles, clay-free, disintegrating rock etc.	4	25 - 30	
G	Strongly over-consolidated, non-softening clay mineral fillings (continuous less than 5 mm thick)	6	16 - 24	
H	Medium or low over-consolidation, softening clay mineral fillings (continuous less than 5 mm thick)	8	12 - 16	
J	Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of Ja depend on percent of swelling clay-size particles, and access to water.	8 12	6 - 12	
k	Zones or bands of disintegrated or crushed	6		
L	rock and clay	8		
M	(for clay conditions) see G, H and J	8 12	6 - 24	
N	Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5		
O	Thick continuous zones or bands of clay	10 13		
P, R	As in M	6 24		

Table 4: Joint Alteration Numbers for Various Categories of Rocks

	Joint Water Reduction	Jw	Approx. Water Pressure kgf/cm²	Notes
A	Dry excavation or slight inflow that is less than 5 l/m locally	1	< 1	1. C to F are rough estimates; increase Jw if drainage installed. 2. Special problems caused by ice formation are not considered
B	Medium pressure, infrequent joint fillings outwash	0.66	1 - 2.5	
C	high pressure in competent rock with unfilled joints	0.5	2.5-10	
D	Large inflow	0.33	2.5-10	
E	Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	
F	Exceptionally high pressure	0.1 - 0.05	> 10	

Table 5: Joint Water Reduction Values for Various Categories of Rocks

Stress Reduction Factor	SRF	Notes
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)	10	Reduce these values of SRF by 25 - 50% but only if the relevant shear zones influence does not intersect the excavation 2. For strongly anisotropic virgin stress field (if measured): when $5<\sigma 1/\sigma 3<10$, reduce σc to $0.8\sigma c$ and σt to $0.8\sigma t$. When $\sigma 1/\sigma 3 > 10$, reduce σc and σt to $0.6\sigma c$ and $0.6\sigma t$.
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50 m)	5	
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50 m)	2.5	
Multiple shear zones in competent rock (clay free), loose surrounding rock at any depth	7.5	
Single shear zone in competent rock (clay free). depth of excavation less than 50m	5	
Single shear zone in competent rock (clay free). depth of excavation greater than 50m	2.5	
Loose open joints, heavily jointed or sugar cube, (any depth)	5	
Low stress, near surface	2.5	
Medium stress	1	
High stress, very tight structure (usually favorable to stability, may be unfavorable to wall stability)	0.5 - 2	
Mild rock burst (massive rock)	5 - 10	
Heavy rock burst (massive rock)	10 - 20	
Mild squeezing rock pressure	5 - 10	
Heavy squeezing rock pressure	10 - 20	
Mild swelling rock pressure	5 - 10	
Heavy swelling rock pressure	10 - 15	

Table 6: Stress Reduction Factors for Various Categories of Rocks

Barton et al (1974) further proposed an additional factor in determining the relation between the values of the Qand the stability and support requirements of underground excavations. This factor is called the equivalent dimension De. This factor is mathematically defined by the following relation:

$$De = \frac{\text{excavation span(height or diameter)}}{\text{excavation support ratio(ESR)}} \ldots\ldots\ldots\ldots\ldots (2)$$

The value of ESRis related to the purpose of excavation and to the degree of safety which is required from the installed support system to preserve the excavation stability. Therefore, the following were proposed by Barton et al (1974):

Excavation Category		ESR
A	Temporary mine openings	3 to 5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defense chambers, portal intersections.	1
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Table 7: Values of Excavation Support Ratio for Different Excavation Categories (Barton Et Al., 1974)

By plotting a graph of De versus Q, number of support groups can be defined in a chart by developed Barton et al (1974). However, was updated recently by Grimstad and Barton (1993) due to continuous rise in use of steel fibre reinforced shotcrete for underground excavation support.

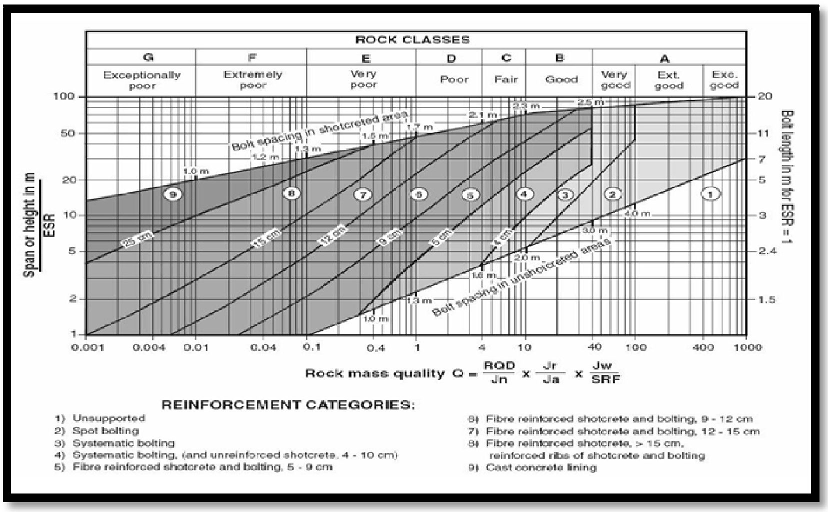

Figure 1: Rock Support Classification Chart for Different Classes of Rock Grimstad and Barton (1993)

Barton et al (1980) provide extra information on rock bolt length, maximum unsupported spans and roof support pressures were provided by Barton et al (1980) to enhance the support recommendations in earlier publications.
The length L of rock boltslength depends on tow parameter: excavation width B and excavation support ratio ESR. The maximum unsupported span (MUS) depends on ESR and Q. These relations are shown below:

$L = 2+\frac{0.15B}{ESR}$... (3)

$MUS = 2ESRQ^{0.4}$.................................... (4)

Based upon analyses of case records, Grimstad and Barton (1993) suggest that the relationship between the value of Q and the permanent roof support pressure Proofis estimated from:

$P_{roof} = \frac{2\sqrt{Jn}}{3Jr}Q^{1/3}$... (5)

2.3. Geology

The rock mas surrounding location of the tunnel excavation isnorite, it is homogeneous and isotropic. In other words, the rock mass has same properties in all directions. It is characterized with a joint set which is rough, undulating and unweathered with little stain on the surface. Based on the laboratory tests of intact rock samples, an average uniaxial compressive strength σ_c of 170MPa was estimated. The directions of the principal stresses are vertical and horizontal.
To evaluate the behavior of the rock mass by numerical models, two important parameters are required: strength and deformation modulus. The generalized Hoek-Brown failure criterion is used in this study. The geological strength index (GSI) was utilized in determining the strength and modulus parameters.
The strength factors: m_b, s and a can be determined using the GSI calculator in Phase 2. However, the Hoek-Brown strength parameters are given by the following equations.

$m_b = mie^{\frac{GSI\ 100}{28-14D}}$.. (6)

$s = e^{\frac{GSI\ 100}{9-3D}}$.. (7)

$a = 0.5 + \frac{1}{6}(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}})$.................................(8)

where mi is a Hoek-Brown constant and D is the degree of disturbance.

Deformation modulus: Hoek et al. (2002) suggested that the mean modulus parameter E for σ_c less than 100MPa is given by the following relation

$E = (1 - \frac{D}{2})\sqrt{\frac{\sigma}{100}}.e^{\frac{GSI-10}{40}}.10^3$ MPa....................(9)

Where $\frac{Jw}{SRF}$ is assumed to be 1, Q can be estimated as

$Q' = (\frac{RQD}{Jn})*(\frac{Jr}{Ja})$.. (10)

Therefore, Q' can be used to estimate the value of GSI using:

GSI =9LogeQ'+44...................................... (11)

2.4. Field Stress

Another factor that yet controls the stability of tunnels is the field stress. The excavation is assumed to be very deep rather than near or surface excavation, therefore, the field stress type is considered constant. The in-situ stress conditions are determined by the field stress before excavation. The in situ stress conditions are given below

Stress	Value
σ_1	85MPa
σ_3	57MPa
σ_z	70MPa
Angle	0^0

Table 8: In Situ Stress Conditions

2.5. Groundwater Conditions

The rock mass is locally damp. The tunnel excavation results in changes in the groundwater conditions in the slope. These changes have a substantial influence on the effective stresses in the rock mass surrounding the tunnel. As a result, a complete analysis of these groundwater conditions is an initiating point for the analysis of the tunnel stability (Hoek et al., 2008). The permeability k of the rock mass is assumed to be 5e-08 m/s

2.6. Geometry

The choice of geometry is another factor which is of equal if not greater importance than the aforementioned factors in determining the stability of tunnels. The surrounding rock mass is a coarse-grained igneous rock. A tunnel of15m span is to be excavated at a depth of about 1100m below the ground surface.

The specification of excavation method by the designer is deemed very important. There are basically two major excavation approaches: full face approach and the top heading and bench approach. However, the choice of which approach to use depends on the size of the tunnel. Tunnels with small diameters are better excavated using the full-face approach as stabilization of the face will relatively be simple. On the other hand, larger tunnels are often excavated in stages from the top down to the bottom **(**Hoek et al., 2008). In the present study, the tunnel is excavated using the top heading and bench approach. The arch is excavated first followed by bench 1 excavation and finally the bottom bench excavation all to be carried out one stage 1 after the other. The geometry of the tunnel is shown in fig. 2.2.

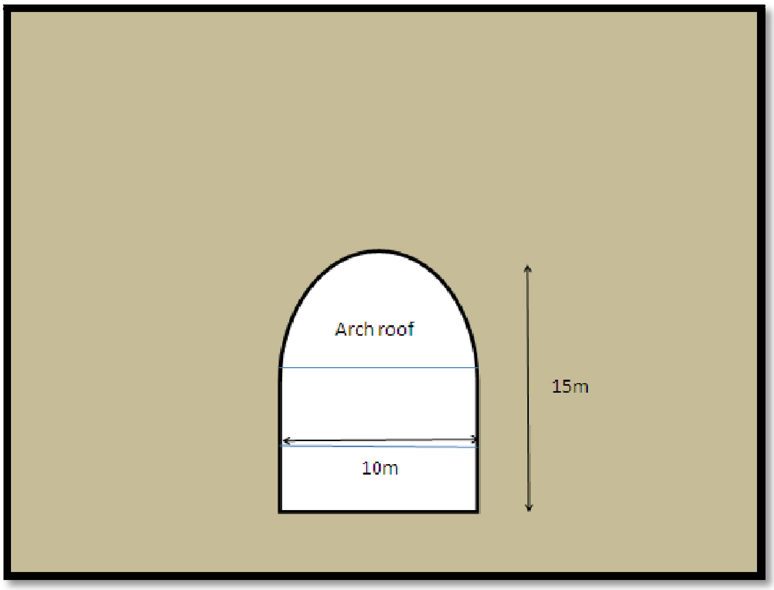

Figure 2: Tunnel Geometry and the Surrounding Rock Mass

2.7. Support System - Rock Bolts, Shotcrete and Concrete Liners

After the excavation of a tunnel, a support system is usually set up to control the closure of thesegment of the tunnel in a pressing rock and to guarantee the security of the opening. If the rock is not proficient,in the absence of support, most often failure may result as a final consequence of undue deformation (wall convergence).Sometimes, even with the presence of the support system, structural failure may resultdue totoo muchpressure exerted on the support by the rock (Cristescu et al., 1987).

Usually before designing a support system, there are certain things that must be attained:

- Certain amount of wall deformation is usually expected as excavation is taking place before the installation of support. This deformation can be determined using various methods but, in this study, the use of empirical calculation suggested by Vlachopoulos and Diederichs.
- At the point of support installation, internal pressure reduction or modulus reduction can be used to determine the pressure or modulus that is responsible for the amount of deformation determined earlier. The internal pressure reduction is used in this study.
- This pressure will be the relaxation pressure and a model can be built and a support added to determine whether the tunnel is stable or not and the liner meet certain factor of safety needed or not.

2.7.1. Support Capacity Diagrams

When the liner used or one of the liners (in the case of composite liners) is reinforced concrete liner, then support capacity diagrams are needed to determine their required safety factor. Capacity envelopes are graphs of shear force space vs. axial force and axial force vs. moment space. The shear force, moment and axial force values are plotted in the capacity envelopes. If any of the liner value falls within an envelope, it means it has a safety factor greater than the value of the envelope. Therefore, if all the calculated values of the liner fall within the design safety factor envelope, the liner safety factor exceeds the design factor of safety and so it is safe.

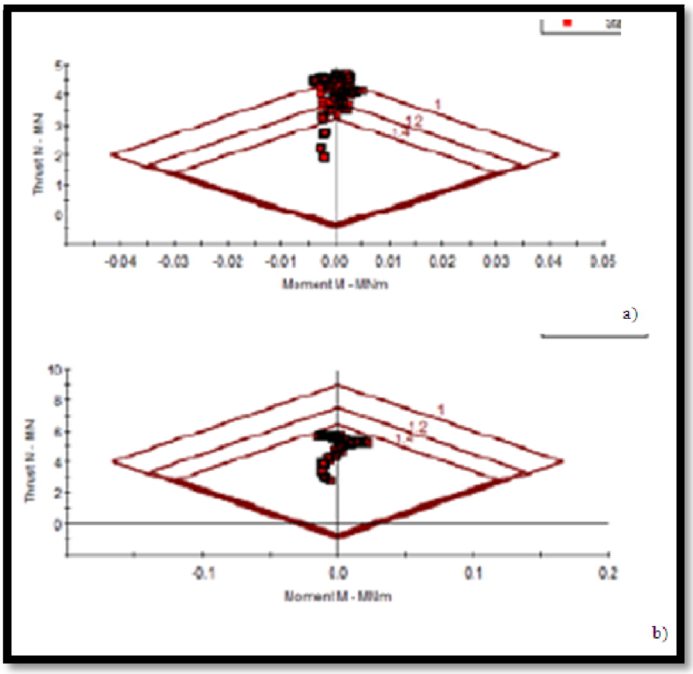

Figure 3: A) Showing Liner Points Falling Outside Capacity Envelope Which Means That They Have FOS Less Than the Required and the Liner Will Experience Cracking If Used B) Showing Liner Points All Falling within Envelope Meaning They Have FOS Higher Than the Required

In the present study, rock bolts together with composite liners are used as support systems.

2.7.2. Rock Bolts

Bolts are anchor bolts long enough to stabilize excavations through rocks, which can be applied either in tunnels or rock cuts. Rock bolt take away the load from the unstable outer part, to the confined and much robust inner part of the rock mass.

Generally, bolting in rocks is very effective in a diverse geo technical and geological conditions. Its key purpose of bolting is the binding together layered or broken rocks such as rocks comprising of bedding planes (sedimentary rocks), rocks with natural fractures and joints, or rocks with unnatural fractures and cracks due to the use of explosives (Peng, 1984).

As aforesaid in the previous paragraph, bolts strengthen or reinforce the rock mass by binding the stratified rock blocks together. The effect of this binding is attained through the friction forces generated as a result of the physical meshing along the anchor and rock boundary. Bolts can be classified based on anchor types as point-anchored bolts and full-length-grouted bolts

However, it is generally believed that effects of bolt binding are achieved by one or a combination of the following basic mechanisms: keying, suspension and beam building (Luo, 1999).

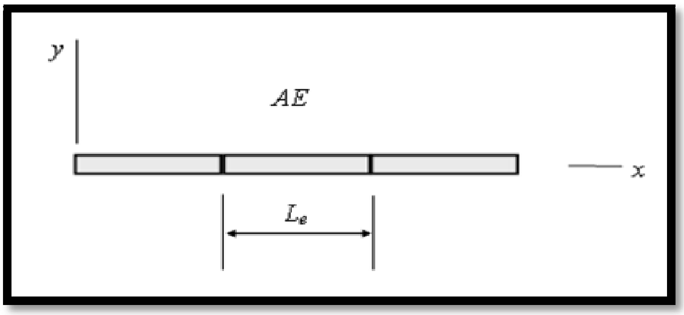

Figure 4: A Fully Bonded Bolt Model from Phase 2

2.7.3. Liners

As aforementioned; composite liners are used in the present study to make part of the support system. A composite liner may be described as a liner consisting of a multiple material layers. It is characterized with different layers of different material properties. The two layers that make up the composite liner in this study are a shotcrete layer and a concrete layer. All these are based on the hypothesis that the rock is poor and will need extensive support system for stability. The verification of this will be seen in the discussion part of the study after the rock mass characterization has been carried out.

2.7.3.1. Shotcrete

A concrete mortar that is projected pneumatically at great velocities onto a surface is called a shotcrete. The application of shotcrete linings is done by the pneumatic shooting of mortar comprising of a full mixture of sand, cement and water into place. Lining thickness fluctuates from about 25 to 75mm. 50mm or less thickness of shotcrete is frequently used on small openings for economic reasons. Moreover, the characteristic difficulty associated with the control of liner thickness application may lead to lining with areas where the thickness is less than the required, thereby creating feeble areas. Shotcrete may be used nearly anywhere a typical concrete mortar would be used. It can be strengthened with steel or may be used alone without steel, and can as well be functional in any thickness(Stevenson, 1999).

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2.7.3.2. Concrete Liners

Benefits justifying their high cost explain the reason the use of concrete linings is widely recognized. Concrete linings are widely used, with benefits justifying their relatively high cost. They are characterized as durable, tough, hydraulically efficient and moderately impermeable. They are suitable for both small and large underground openings and both high and low flow velocities. In fact, concrete liners satisfy every purpose of lining. If concrete liners are properly designed and maintained, they would have at least 40 years average life of service (USBR, 1975; Kraatz, 1977; Stevenson, 1999). In the prevention of cracks from developing and the absence of weakening action of salts, concrete liners can last indefinitely. They are frequently subjected to certain cracking instigated by freeze-thaw action, expansive clays, collapsible soils, and frost heave; nonetheless cracks that allow considerable leakage can be closed with asphaltic compounds. If the installation of concrete liners was performed accurately, no costly maintenance is necessary (Stevenson, 1999).

2.8. Groundwater Seepage

Groundwater inflow is one of the most common and challenging problems faced in tunneling constructions. It may cause delay by slowing down the excavation speed in tunnel construction, while on the other hand it may generate a potential threat to the tunnel stability in the long run. Groundwater inflow and leakage is related to one of the practical difficulties in the construction and maintenance of tunnels. In fact, some of the greatest disasters in construction of tunnels are linked to groundwater inflow of huge volumes in water saturated rocks that are extremely fractured (Li et al., 2008).

In the construction of tunnels below the water table, pore water pressure and leakage are the main hydraulic factors to be considered. Particularly, the pore water pressure, which signifies the penetrating pressure, may possiblyhasthe deterioration of structural components and subsequently increase leakage. Therefore, evading high water pressure is one of the chief concerns when it comes to underwater tunnel design (Shin et al., 2009).

There are many factors that influence groundwater inflow: the rock mass permeability, the water table, rock fracture aperture and excavation size. While it is very challenging to accurately predict the water seepage during tunnel construction, quite a big number of researches have been carried out to face the problem, specifically through numerical models in targeting sane and visible solutions. Goodman (1965) proposed an expression for the flow rate per unit tunnel length, q, in a homogeneous vast water table aquifer as

$$q = \frac{2\pi KH}{\ln\left(\frac{2H}{r}\right)} \ldots\ldots\ldots\ldots\ldots\ (12)$$

Where H represents the depth of the tunnel below the water table, Krepresents the hydraulic conductivity and r is the radius of the tunnel

The groundwater seepage analysis will be carried out to observe the distribution of pore pressures, hydraulic gradients and the distribution of discharge velocities in and around the excavation region. The distribution of such parameters is solely controlled by the permeability and area of the surrounding rock mass.

2.9. Excavation Damaged Zone

In tunnel design, when the magnitudes of stress reach the strength of the rock mass, there will be subsequent yielding of the rock mass (Cai, 2011). This is a major concern in assessing the stability of underground excavations.

A plastic zone may result from the redistribution of stress around the space usually termed the excavation damage zone (EDZ) (Golshani et al., 2005). The in-situstress, the tunnel shape of the tunnel and orientation relative to the maximum stress, method of excavation, pore pressure variation, and the creation of close excavations all affect the advancement of EDZ (Martino et al., 2004).

Actually, damage of a rock massis followed by a subsequent creation of crack networks, which could establish large flow routes, depending on the interaction of crack. As a result, the hydraulic conductivity of the surrounding rock mass will increase and can turn into heterogeneous and anisotropic condition. Crack permeability is usually connected to the aperture of the crack and the main issue is therefore the proposition of a model providing the connection between permeability of the rock mass and the aperture of crack (opening), as well as the development of the crack aperture in the course of the excavation (Severine et al., 2010).

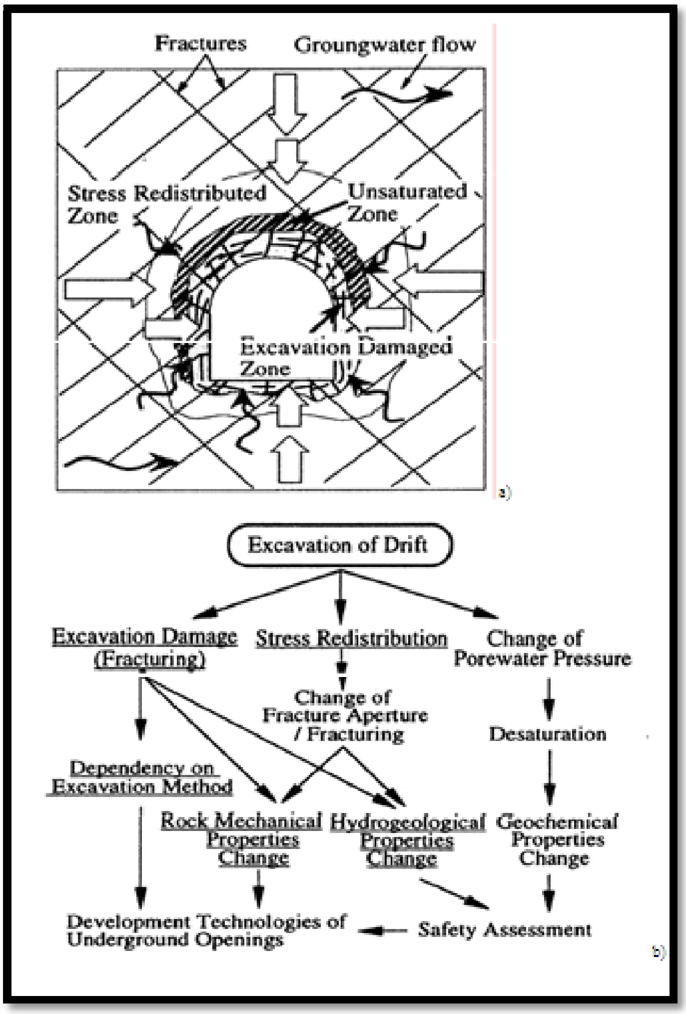

Figure 5: a) An Excavation Damaged Zone Model b) Processes of Change in Rock Properties Related to EDZ (Sato Et Al., 2000)

2.9.1. Characteristics of EDZ

The characteristic features of EDZ around an underground construction can only be assessed appropriately when the appropriate methods are applied with consideration of the geometry of the tunnel, method of excavation, rock mass conditions, etc. In order to get reliable results, application of several methods in EDZ evaluation is recommended rather than the use of one or few methods (Kwon et al., 2008). Most of the EDZ characterizations were carried out with respect to two key properties: mechanical properties and hydraulic properties.

Reduction in mechanical properties is generally observed within the EDZ while an increment is generally observed in the hydraulic properties.

The size of EDZ is reported by Pusch and Stanfors (1992) to depend on the charge density when blasting technique of excavation is used. Where S is the size of EDZ in meters and ρ_b is the charge density in kilogram per meter, then the relation is given as

$S = 2 \times \rho_b^{1.2}$…………….. (13)

From the study of the behavior of EDZ by Saiang (2008), the thickness of the EDZ is ranging from 0.5 to 1m. From the study of EDZ investigation by Kwon et al. (2008), the range of EDZ thickness is from 0.3 to 2.3m. Based on these studies, thickness range of 0.5m to 2.5m will be adopted in the present study.

2.10. Case for Relevant Study

2.10.1. Three-Dimensional Analysis of Tunneling Effects on Structures to Develop Design Methods

A study on the three-dimensional analysis of tunneling effects on structures to develop design methods was carried out by Alan Graham Blood worth in 2002 for the fulfillment of the degree of Doctor of Philosophy at the University of Oxford. The study aimed at verifying a three-dimensional numerical modeling approach in order to predict of settlement damage to masonry buildings due to soft ground tunneling.

It was concluded that the modeling processes are appropriate for application to the detailed valuation of buildings response to tunneling. Specific features of the processes are that the building is modeled together with the ground and a representation of the excavation of the tunnel, and in 3D. It has been established that all these characteristics are needed to model the response of the building, which could include a combination of arching, shear deformation and bending behavior.

2.10.2. Behavior of Blast Induced Damaged Zone around Underground Excavations in Hard Rock Mass

David Saiang in 2008 studied the Behavior of Blast Induced Damaged Zone around Underground Excavations in Hard Rock Mass for the fulfillment of thesis requirement for the degree of Doctor of Philosophy in Rock Mechanics and Rock Engineering. To generally improve knowledge of the damaged zone, significant efforts have been made over the last few decades in a broader area; the excavation disturbed zone. He deemed that to be able to evaluate the importance of the blast induced damaged zone and how it affects the performance of an excavation, the mechanical behavior of the zone must be understood. It was concluded that

There is variation in the thickness of blast induced damaged zone thickness most practical cases between 0.1 and 1.0m with an average ranging from 0.3 - 0.5m subject to the blasting technique used.

The numerical studies revealed that tension is the key failure mechanism within the damaged zone at shallow excavations and shear in deep excavations.

The study on shotcrete-rock interface revealed that the strength of the bond of the interface is essential for the shear strength.

3. Methodology

3.1. Prologue

In every research, work, study, project, etc., it is of vital importance that an organized plan of work is scheduled. Organization and plan of work is important because it will aide in; producing better results in terms of experimental investigations; saving time and money in terms of big construction projects, etc. In the present study, 3-step approach is adopted for the whole process of analysis.

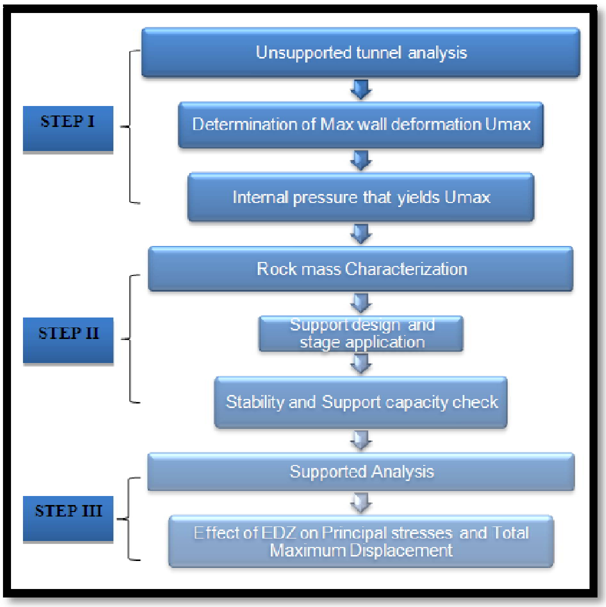

Figure 6: Flow of Analysis

3.2. Step I Unsupported Analysis of Tunnel and Seepage

The rock mass will be characterized to determine the strength parameters (mb, s, a), quality index (Q) and the dimension equivalent (De) to help in specifying the right support system. However, specification of the right support system is carried out in part II of the analysis since part deals with analysis without support.The tunnel is to be excavated first from the arch in stage 1, then the middle bench in stage 2 and finally the bottom bench in stage 3.

In this part of the analysis, the tunnel is to be designed and analyzed without any support system. Usually after excavation before support installation, certain deformation is expected. The aim here is to determine the maximum wall deformation prior to support installation and then to find the internal pressure that yields the aforesaid deformation. To achieve this in phase 2

- The analysis is multistage and the number of stages will be set to 4
- The tunnel is excavated starting with the arch in stage 1,followed by bench 1 in stage 2, and then bottom bench all in stage 3
- An internal pressure is added with stage factors of 1, 0.6,0.2 and 0 for stage 1 to 4 in that order.
- After the analysis, umaxfar from the tunnel face will be determined at stage 4 with 0 internal pressure.
- Radius of yield zone will also be determined as r_y, distance of support installation from tunnel face x and radius of tunnel labeled as r_c.

There are various methods of determining wall deformation prior to support installation. However, the empirical method suggested by Vlacopoulos and Diederichs was used in this study.

With the ratio of $R_y$to $R_c$and X to R_c, the plot by Vlacopoulos and Diederichs as shown in fig. below was used to find the closure. The approximate closure multiplied by Umax will give the maximum displacement by the roof before installation of support.

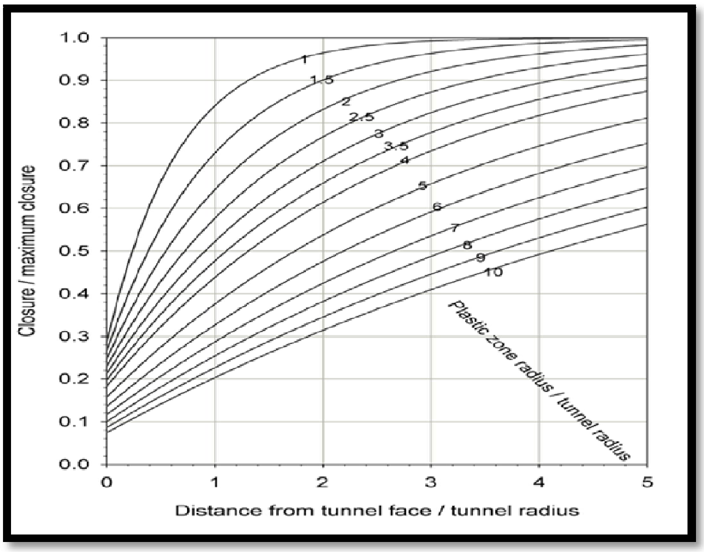

Figure 7: Plot of Ratio of Distance from Tunnel Face to Tunnel Radius and Ratio of Plastic Zone Radius to Tunnel Radius for Closure by Vlacopoulos and Diederichs

Now to find the internal pressure (stage) that yields maximum roof displacement, a plot of total displacement versus stage should be generated and the corresponding stage (internal pressure) that yields the max roof displacement can be found.

The seepage analysis in this part is carried out to determine the distribution pore pressures and total discharge velocities around the tunnel prior to installation of support. As it is assumed there will be various form of damage around the tunnel excavation before support installation, it is then anticipated that the thickness of the yield zone will be high and would result in increase in permeability around the tunnel.

3.3. Step II Support Design and Stage Application

In this part, the right support systems (bolts and liners) will be specified according to the rock mass characterization carried out in part 1 of the analysis. The stages should now be renamed since the relaxation stage was determined in part 1 of the analysis. At stage one, after the excavation of the arch, immediate application of support will follow. As earlier stated, the support system in this study will comprise of rock bolts and composite liners since we are dealing with a hard rock with possible stress problem. However, this will be verified after the rock mass characterization has been carried out in the discussion part of the study.

The rock bolts will be applied all over the excavation boundary in stage one. However, the application of liners will be stage wise. Theshotcrete is added to the arch of the tunnel immediately after its excavation in stage one, excavation of middle bench in stage two will be followed by an immediate application of shotcrete layer and also the application of the shotcrete layer follows the excavation of the bottom bench in stage three. The addition of concrete on top of the shotcrete layer is followed one stage later to make a composite liner. This addition is finished in the fourth stage.

3.3.1. Stability Check

After the analysis, the axial force and axial stress at rock bolt will be known. The shear force and the bending moment at shotcrete will also be known. The stress ratios at rock bolt and shotcrete should each be greater than 1 i.e.$\frac{\sigma s}{\sigma y} > 1$

3.3.2. Support Capacity Check

For the reinforced concrete layer, capacity envelopes are plotted with required factor of safety (FOS). Liner values should all fall inside the envelopes for both moment and shear to show that they have a FOS greater than the required.

3.4. Step III – Supported Analysis

This is the final part of the whole process of the analysis. The stress analysis of the supported excavation will be carried out.

After the final stress analysis, subsequent determination of the distribution of the required properties around the tunnel should follow i.e. principal stresses (sigma 1 and sigma 2) and the total maximum displacement. This can be achieved in phase 2 by querying the excavation boundary and graphing the query data for each property needed.

The plastic zone of a supported excavation is usually anticipated to develop cracks which would in turn increase the pores holding water. Larger cracks would probably mean larger pore water pressures and higher discharge velocities. The above explanation will be clearer after carrying out the analysis.

4. Analysis Result and Discussion

4.1. Classification of Rock mass According to Q-index

The rock mass surrounding the excavation region is to be classified according to the Q-index classification. In order to use the Q-index classification, two important parameters are required and they are the quality index Q and the dimension equivalent De. With the values of these two parameters, then the chart of rock classification after Grimstad and Barton (1993) can be used. to determine Q, the following parameters are determined:
An average value of RQD = 50 will be used in this study.
From Table 2), Jn = 4
From table 3), Jr = 3
From table 4), Ja = 1.0
From table 5), Jw = 1.0
Stress ratio $\frac{\sigma c}{\sigma 1} = \frac{170}{85}$ = 2 (i.e. a rock with rock stress problem)
For such stress ratio, mild to heavy rock burst should be anticipated.
From table 6), SRF lies between 10 and 20
Therefore, SRF = 20 will be used.
From equation (1),
$Q = (\frac{50}{4})*(\frac{3}{1})*(\frac{1}{20}) = 1.88$
From equation (2),
$De = \frac{\text{excavation span(height or diameter)}}{\text{excavation support ratio(ESR)}}$
From Table7, ESR = 1.6
$\therefore De = \frac{15}{1.6} = 9.38$
From Figure 1, for Q = 1.88 and De = 9.38, the excavation falls inside category 6.
This category requires rock bolts at about 1.7m spacing and at least 100mm layer of steel reinforced shotcrete.
For the length of rock bolt equation (3) will be utilized
$L = 2+\frac{0.15B}{ESR}$
B = 10m and ESR =1.6
$\therefore L = 2+\frac{0.15*15}{1.6} = 3.4m$

However, due to the anticipation of rock burst from the stress ratio, 5m long rock bolts at 0.5m out of plane spacing will be used all over the excavation.
For the rock strength properties, the GSI calculator in phase 2 was utilized. The GSI value is determined using equation (10)

GSI =9LogeQ'+44, from equation (11) Q' will be determined
I.e. $Q' = (\frac{50}{4})*(\frac{3}{1}) = 37.5$

Therefore, GSI = $9Log_e37.5+44 = 58$

Initial Element Loading	Field Stress Only
Elastic type	isotropic
Young's modulus	32277.6 MPa
Poisson's ratio	0.3
Failure Criterion	Generalized Hoek-Brown
Material type	Plastic
Dilation Parameter	0
Compressive strength	170 MPa
mb parameter	2.2313
s parameter	0.009404
a parameter	0.503276
Residual mb parameter	2.2313
Residual s parameter	0.009404
Residual a parameter	0.503276
Hydraulic model	Simple
Ks	1e-010 m/s
K2/K1	1
K Angle	0 degrees

Table 9: Summary of Material Properties

4.2. Numerical Modeling

The geometry of the tunnel was designed in phase 2, the roof the tunnel is to take the arch shape and a box like body. The finite element mesh is graded with three node triangles. Figure 8 below shows the model with the finite element mesh (FEM).

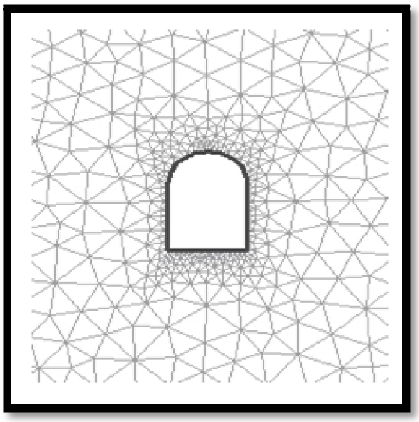

Figure 8: Tunnel Opening with the Finite Element Mesh Generated in Phase 2

4.2.1. Excavation

As discussed in the previous chapter, the top heading method of exaction will be utilized in this study. That is to say the tunnel will be excavated in stages starting from the arch in stage one, followed by the middle bench in stage two and lastly the bottom bench in stage three.

4.2.2. Total Displacement

As observed, the displacement in stage one is almost negligible due to its internal pressure factor of 1.0. As we move to stage two with a reduced internal pressure, the displacement tends to increase. Stage four which is the center of interest tends to yield a total maximum displacement of 0.0436m and a total minimum displacement of about 0.028m and this can be seen in Figure 11 below. From the figure of deformation vectors, it can be seen that the plastic displacement shows an inward displacement of the excavation walls.

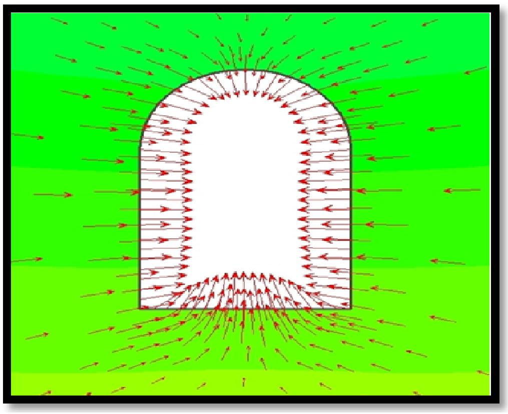

Figure 9: Deformation Vectors around the Excavation at Stage 4

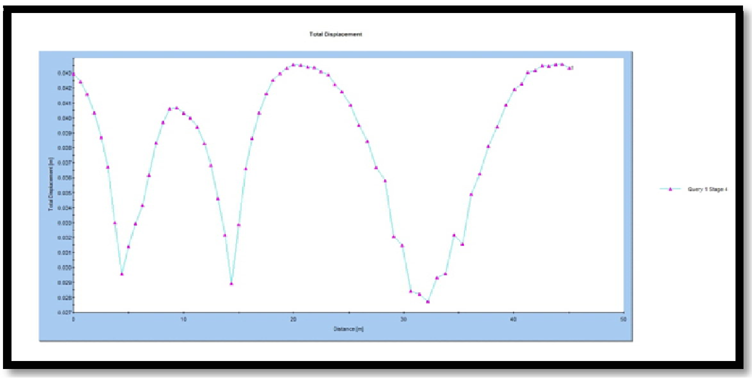

Figure 10: Total Displacement around the Excavation in Stage 4

4.2.3. Plastic Zone

In stage one after the excavation of the arch, quite anextensive zone of weakness was generated at the bottom of the excavation. However, on the roof and sides of the arch, no damage zone was observed. The middle bench excavation led to similar zone of weakness right at the bottom of the excavation. As in stage one, no weakened or damaged zones have been observed on the roof and the side walls. In stage three however, the story is quite different. The bottom excavation led to the generation of damaged zones both at the roof and bottom of the tunnel. The damaged zone is more extensive at the bottom than at the roof. The extensiveness of the zone of weakness has developed all over the tunnel including the side wall in stage 4. However, this is not surprising as no support has been installed. The red sections indicate the zone of yielding in the excavations.

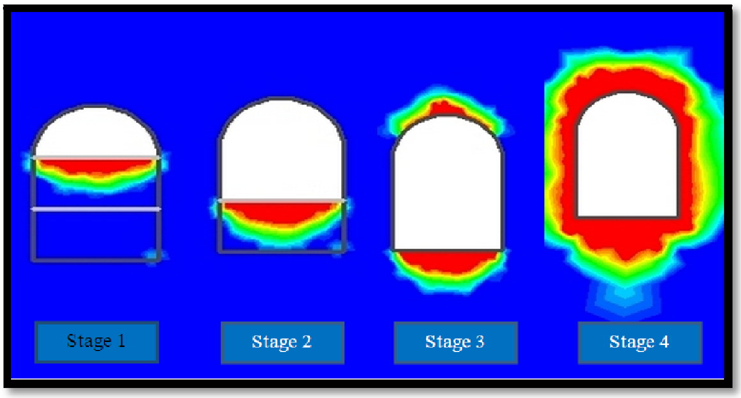

Figure 11: The Generation of Plastic Zones in the Different Stages of the Excavation

The number of yielded elements approximates the size of the plastic zone in each stage. There are 73 yielded elements in stage one, 71 in stage two, 134 in stage three and a total of 355 yielded elements in stage 4. X symbols represent shear failure while the circles represent tensile failure around the excavation.

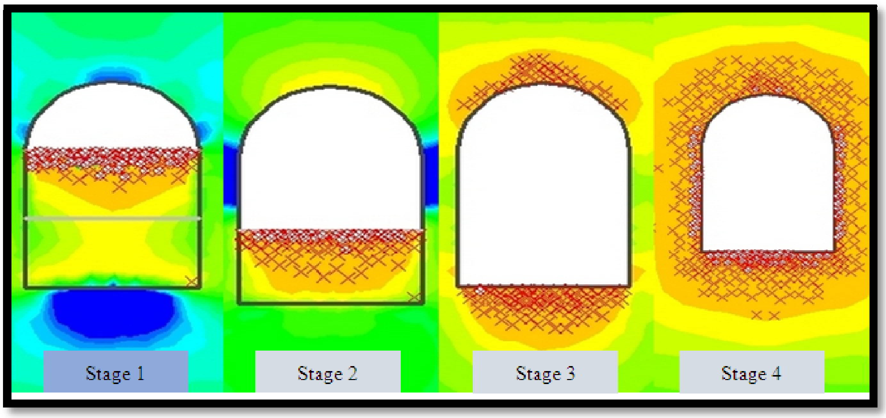

Figure 12: Yielded Elements Showing Tensile and Shear Failures

4.2.4. Tunnel Deformation Prior to the Installation of Support

Usually as a tunnel is being excavated, there is a certain amount of wall deformation that occurs before support system is added. Various methods can be applied to determine the amount of such deformation such as axisymmetric finite element method, field stress observation, Vlachopoulos and Diederichs empirical method, etc. as stated in chapter 3, the Vlachopoulos and Diederichs approach is utilized in this study.

The use of this method requires one to know the maximum displacement that occurs far from the tunnel face which is determined in stage 4 with zero internal pressure; the radius of plastic zone in stage 4 measured from the center of excavation; distance from the tunnel face and the radius of tunnel.

Max wall displacement far from the tunnel face U_{max} = 0.0436m
Radius of plastic zone R_y = 11.42m
Distance from tunnel face X = 2m
Radius of tunnel R_c= 7.5m

$\frac{Ry}{Rc} = \frac{11.42}{7.5} = 1.52$

$\frac{X}{Rc} = \frac{2}{7.5} = 0.27$

From fig. 3.2, closure/max closure is approximately 0.43
Therefore, closure = 0.43 * 0.0436 = 0.019m

This implies the tunnel roof displaces 0.019m prior to support installation. The stage (internal pressure) that is responsible for the above roof displacement is determined from a single point plot of total displacement versus stage below.

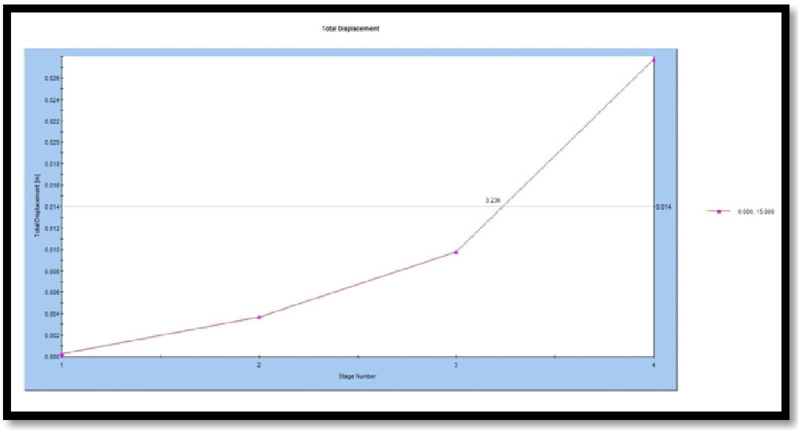

Figure 13: Plot of Total Displacement versus Stage Number

From the above plot, it is seen that 0.019m displacement occurs between stage three and the final stage i.e. stage four with zero internal pressure. Therefore, an internal pressure with factor between 0.2 and 0 yields the above tunnel wall displacement. This is because the tunnel relaxation can occur only after the tunnel has been fully excavated.

4.3. Supports and Stage Application

In the first part of the analysis, the Q-index classification placed the rock mass in a category requiring both bolt and reinforced steel shotcrete support. However, minor adjustments have been made to both the bolts and the liners toenhance the support system required in order to accommodate the minor to major rock burst anticipated as a result of the stress problem the rock is observed to be having.

The bolts are fully bonded that act independently i.e. they don't depend on each other. Adjoining bolt elements do not influence each other directly, but only indirectly through their effect on the rock mass. The bolts are 5m in length and the orientation of the bolt pattern is normal to the boundary. They are installed in an out of plane line spacing of 0.5m. The composite liner is made of a shotcrete layer and a reinforced concrete layer. The bolt and the composite liner properties are given in the tables below.

Bolt name	Bolt 1
Bolt Type	Fully bonded bolt
Diameter	25 mm
Young's modulus	200000 MPa
Tensile capacity	0.2 MN
Residual Tensile capacity	0.2 MN
Pre-tensioning	0 MN
Pre-tensioning force	Constant in install stage
Out-of-plane spacing	0.5 m
Allow Joints to Shear Bolt	Yes

Table 10: Rock Bolt Properties

Name	Shotcrete	concrete
Type	Standard beam	Reinforced concrete
Young modulus (MPa)	30000	50000
Poison ratio	0.2	0.2
Thickness (m)	0.3	0.8
Material type	Elastic	Elastic
Common type	n/a	I beam W460 x 464
Spacing (m)	n/a	1
Compressive strength (MPa)	n/a	300
Tensile strength (MPa)	n/a	190

Table 11: Composite Layers Properties

4.3.1. Stage Application of Support

The bolts are applied all over the excavation boundary in stage one. The first layer of the composite liner is applied to the arch immediately after its excavation in stage one. In stage two, the application of the shotcrete follows the excavation of the middle bench and the concrete layer is also applied to the arch on top of the earlier applied shotcrete. In the third stage, bottom bench is supported with the shotcrete immediately after its excavation and the concrete layer is also applied to the middle bench. The reinforced concrete layer is later applied all over the excavation i.e. on top of the shotcrete layer in fourth stage. The application process can be seen in fig. 4.10, the green indicating the shotcrete layer and the purple indicating the concrete layer.

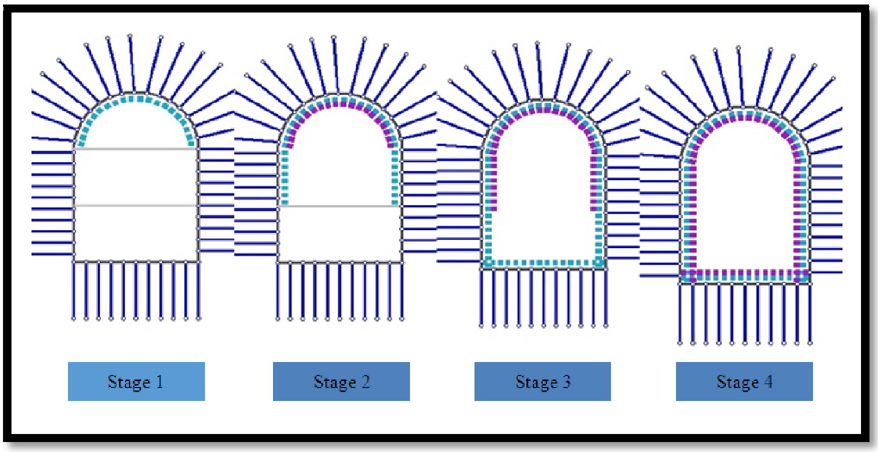

Figure 14: Rock Bolts and Stage Application of Composite Liners

4.3.2. Total Displacement of the Supported Tunnel

After the installation of support, it was observed that at stage one the total displacement is 0.015m at stage one, 0.012m at stage two, 0.024m at stage three and at the final stage there was a total maximum displacement of 0.035m and a total minimum displacement of 0.005m. Compared to the unsupported tunnel, the tunnel displacement has dropped indicating that the support system is quite effective. From the deformation vectors, it was observed that the plastic displacement shows an outward displacement of the excavation walls in stage one. While in stages 2 to 4, there was an inward displacement of the excavation walls with significant floor heave.

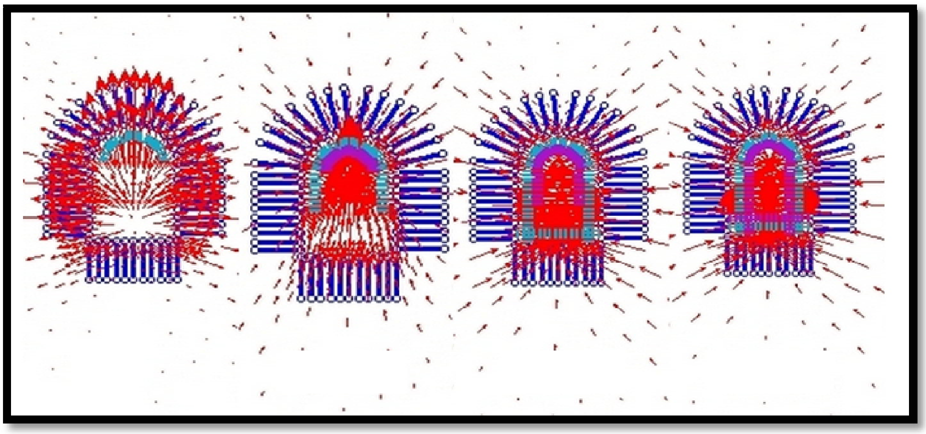

Figure 15: Deformation Vectors at Different Stages of the Supported Excavation

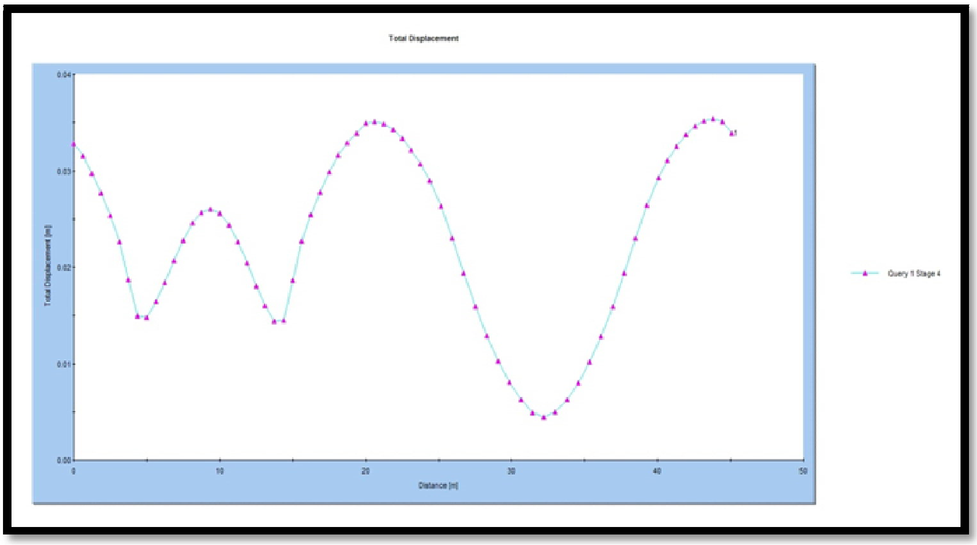

Figure 16: Distribution of Total Displacement around the Supported Excavation in Stage 4

4.3.3. Equivalent Plastic Zone

The size of a plastic zone is directly related to the properties of the rock mass and therefore, any improvement in these properties (strength in particular), as a result of support installation soon after excavation, should be able to reduce the extent of the zone of weakness. Generally, the plastic of a supported excavation is smaller than that of an unsupported excavation. This zone is considered the plastic zone of a material with virtually improved properties and it is referred to as the equivalent plastic zone.

The extent of the yield zones at the floors in stages one and two did not change even with the installation of support. This is not surprising as the supports are installed on the external excavation boundaries only, consequently they will have no effect on the internal boundaries. These zones of weakness are not threatening since they will eventually be excavated in the succeeding stages as the excavation continues. In stages three and four, the effectiveness of the support system is a seen at the roofs where there is extensive reduction in the size of the plastic zone. However, at the floors of these two stages and the side walls of stage four, little or no reduction was observed in the plastic zone. This means that better support system is required to successfully accommodate the extent of failure generated due to the excavation of the tunnel. A better geometry and method of excavation could also mean less failure zone.

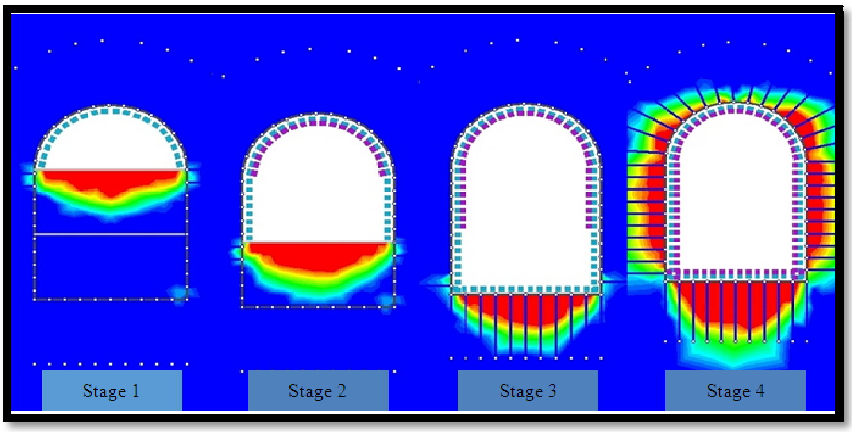

Figure 17: Plastic Zones at Different Stages of the Supported Excavation

The amount of yielded elements has reduced extensively with the installation of support in stages three and four. The number of yielded elements has dropped to 96 in stage three compared to the 134 in stage three when no support was added. In stage four, the number of the yielded elements has dropped to about 40% of the total 355 yielded elements when no support was installed.

4.4. Introducing EDZ to the Model

With reference to certain previous researches on EDZ like EDZ investigation at the KAERI research tunnel by Kwon et al. 2008 and behavior of Blast Induced Damaged Zone around Underground Excavations in Hard Rock Mass by Saiang 2008, it was found that the thickness of the EDZ ranges from 0.5 to 2.5m. However, for the purpose of this study a thickness range of 0.5 to 2.0m has been adopted. For this particular range, four EDZ thicknesses with different material properties (permeability K and elastic modulus E) have been decided and added to the model differently. Each thickness with its material properties can be seen in the table below.

Thickness (m)	Permeability K (m/s)	Elastic modulus E (MPa)
0.5	1.0×10^{-18}	6455.5
1.0	1.0×10^{-16}	12911
1.5	1.0×10^{-14}	19366
2.0	1.0×10^{-12}	25822

Table 12: EDZ Varying Thicknesses with Varying Permeability and Elastic Modulus

The permeability together with the elastic modulus was reduced as the thickness decreases.

4.4.1. Influence of EDZ on Principal Stresses

The decrease in thickness with corresponding reduction in elastic modulus E of the EDZ was to formulate a trend on the possible variation of the principal stresses (sigma 1 and sigma 3) that is anticipated. σ_1 is the vertical stress while σ_3 is the horizontal stress. The development of EDZ in deep excavations affects the structural stability by altering the rock strengths and its deformational behavior (Kwon et al., 2008).

4.4.1.1. Influence on Sigma 1

For a particular point along the excavation boundary, the value of σ_1 was taken for each EDZ thickness and the trend generated can be seen in the figure below.

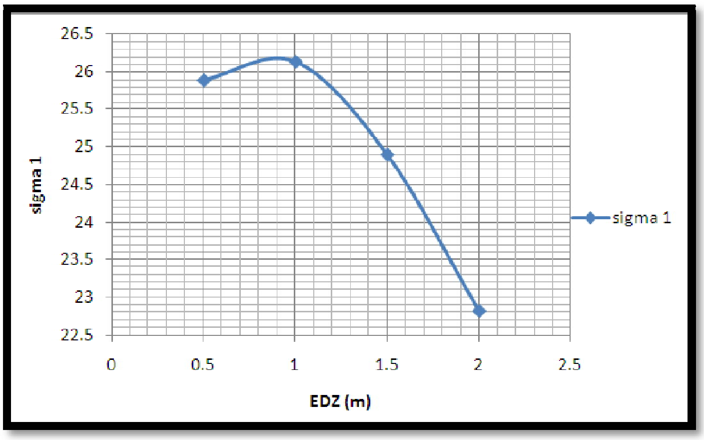

Figure 18: Graph of Sigma 1 against EDZ Thickness

From the graph generated, it was observed that there was a slight increase in σ_1 from thickness of 0.5m to 1.0m, and then drastic fall was observed from 1.0m to 2.0m. Based on the drastic fall that was observed from EDZ thickness of 1.0m with an elastic modulus of 12911MPa to EDZ thickness of 2.0m with elastic modulus of 25822MPa, it can be deduced that increase in EDZ thickness with corresponding decrease in elastic modulus leads to reduction in the vertical principal stress (σ_1).

4.4.1.2. Influence on Sigma 3

At a similar point to that of the σ_1, the value of σ_3 was taken for each thickness and elastic modulus and a trend was generated which can be seen in the figure below.

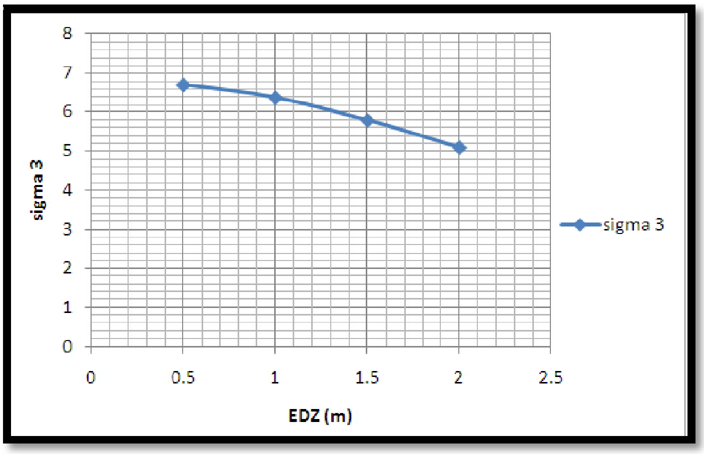

Figure 19: Graph of Sigma 3 versus EDZ Thickness

It is apparent from the figure above that the trend is very similar to that which was generated in the case of σ_1, only that instead of an increase in the stress from 0.5m to 1.0m, there was a continuous reduction and therefore a similar conclusion can be drawn in this case; that decrease in elastic modulus leads to reduction in the horizontal principal stress (σ_3).

4.4.1.3. Influence on Shear Stress

However, it can also be deduced that σ_1 tends to decrease with increasing EDZ thickness due to the development of shear stress τ as a result of the excavation. The shear stress is determined from the failure equation given below

$\tau = \sigma_1 \tan\Phi + C$

Using the equation above the shear stress values were computed and a graph of shear stress versus EDZ was generated.

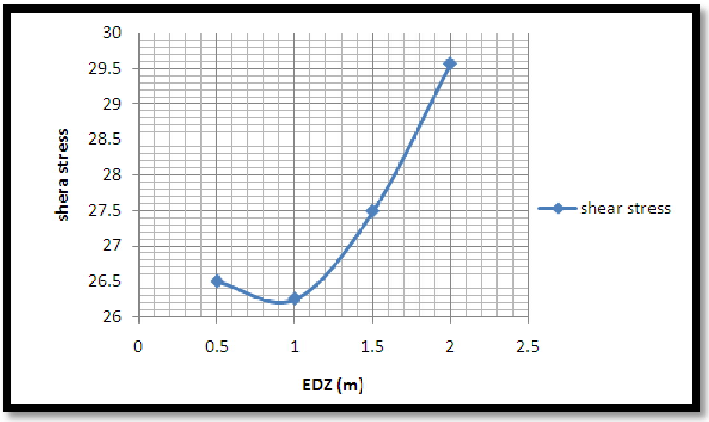

Figure 20: Graph of Shear Stress versus EDZ Thickness

From the above figure, it can be seen that the development of EDZ leads to increase in shear stress. Comparing the above plot with that of sigma 1 versus EDZ, it is apparent that as the sigma 1 goes up, the shear stress goes down. In other words, τ decreases with increasing σ_1 and vice versa. This is however shown more clearly in the figure below

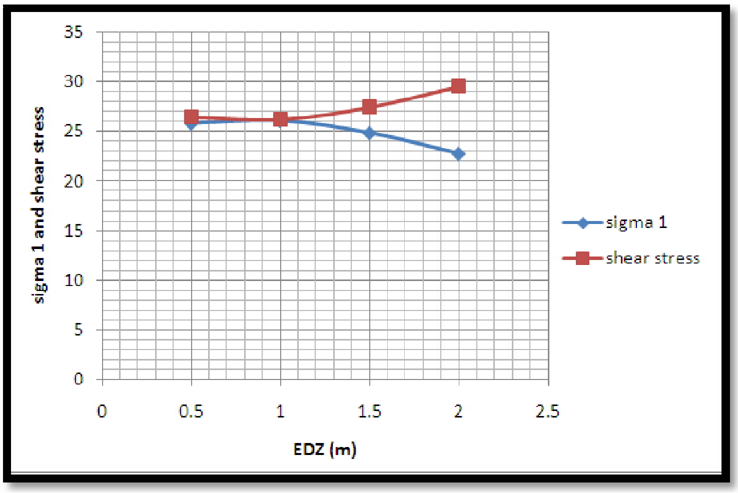

Figure 21: Combination of Sigma 1 and Shear Stress versus EDZ

4.4.2. Influence of EDZ on Total Maximum Displacement

For a point on the roof of the excavation, values of the total maximum displacement were taken and the average was determined. From the average values, it was found that there was a slight descend on the total maximum displacement as the EDZ thickness increases. This trend is shown in the figure below.

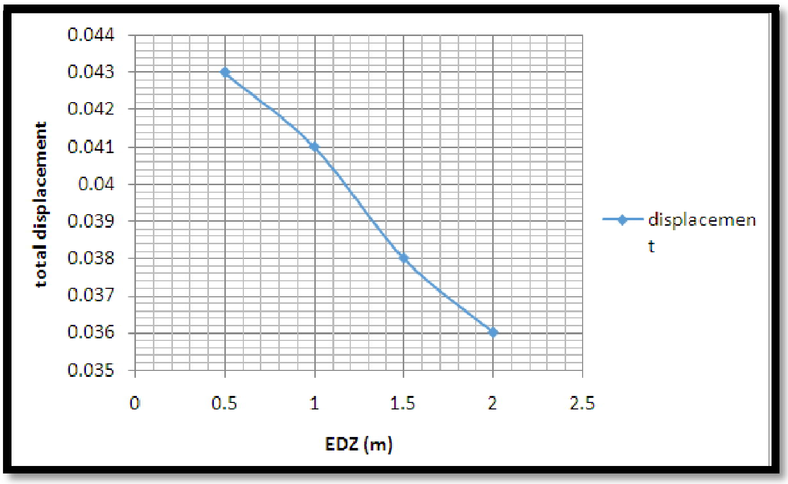

Figure 22: EDZ Thickness versus Total Maximum Wall Displacement

5. Conclusion And Recommendations

5.1. Conclusion

The excavation of the tunnel has resulted in the development of excavation damaged zone round the tunnel. The designed support system: a combination of rock bolts and composite liners has sufficiently reduced the damaged zone on the roof of the tunnel. However, in order to effectively reduce the number of yielded elements both in the walls and the bottom, a better support system (say shotcrete liners with bigger thickness) should be used.

Excavation damaged zone has tremendous effect on mechanical properties. It was found that increase in EDZ thickness with increasing elastic modulus leads to general reduction in principal stresses; sigma 1 and sigma 3. However, the shear stress was increased as elastic modulus decreases and EDZ thickness increases. A slight descend was observed on the total maximum displacement as the EDZ thickness increases with corresponding decrease in elastic modulus.

In general, it can be concluded that the mechanical properties (stresses) and total maximum displacement around the tunnel decrease with the development of EDZ around the tunnel

6. Recommendations

The development of EDZ around the tunnel could be influenced by the geometry of the opening. Therefore, it is recommended that the designer should take tremendous precaution when making a choice of geometry depending on the type of rock mass surrounding the excavation.

The choice of support system should be strictly based on the rock class. Therefore, when classifying the surrounding rock mass, at least two or more classification systems should be used in order to get the best of judgment in the classification and for better support design. This would however, help in providing right support for the excavation in a first-time approach which would in turn save resources like time and money.

7. References

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