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Numerical Simulations for Analysis of Face Stability of Tunnelling

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ABSTRACT

The aim of this paper is to evaluate numerical procedures that are used for analysis of face stability of a tunnelling. Two-dimensional (2D) and three-dimensional (3D) Finite Element (FE) modelling of the Second Heinenoord Tunnel in Netherlands were done by using PLAXIS programs. The models were simulated the tunnel boring machine (TBM) with shell element and the slurry pressure with applied face pressure that measured in the site from the literature. Two methods were used to determine the situation of face failure. The first method was done by reducing the applied face pressure until the failure of the face was occurred, so the minimum required face pressure was determined. The second one was done by reducing the shear strength of the soil until the face failure, so the safety factor was determined. Both methods were applied for 2D and 3D FE-modelling. The results were presented and discussed. Also, the results of the minimum required support pressure were compared to the result of the centrifugal test model for the Second Heinenoord Tunnel found in literature. It was found that, the reducing applied face pressure method is much better than the reducing shear strength method. Also, the result of 3D FE-modelling gave better prediction when comparing with the experimental result .

المستخلص

تهدف هذه الورقة العلمية إلى تقييم الطرق العددية المستخدمة لتحليل ثباتية وجه النفق. حيث تم فيها استخدام نماذج ثلاثية وثنائية الأبعاد لطريقة العنصر المحدد لنفق هينورد الثاني الواقع في هولندا باستخدام برنامج بلاكسس. استخدمت في النماذج العددية عناصر قشرية لنمذجة ماكينة حفر النفق وقوى ضغط مطبقة على وجه النفق لنمذجة ضغط مائع الحفر المستخدم في ماكينة

الحفر. استخدمت طريقتان لتحديد حد الإنهيار لوجه النفق. الطريقة الأولى عن طريق إنقاص قوى الضغط المطبقة على وجه النفق حتى حد الإنهيار, ثم أمكننا بعد ذلك تحديد أقل قوى ضغط يمكن تطبيقها لضمان ثباتية وجه النفق. الطريقة الثانية عن طريق إنقاص مقاومة التربة للقص حتى حد الإنهيار, ثم أمكننا بعد ذلك من تحديد معامل الأمان. كلتا الطريقتين طبقتا على النموذجين الثنائي و الثلاثي الأبعاد. تم بعد ذلك عرض النتائج و مناقشتها. وكذلك أيضا, تم مقارنة نتائج أقل ضغط وجه مطلوب بنتيجة تجربة اختبار الطرد المركزي لنموذج نفق هيننورد الثاني الموجودة في الدراسات المسبقة. تم التحصل على أن طريقة إنقاص قوى الضغط المطبقة أدق بكثير من طريقة إنقاص مقاومة التربة للقص. كذلك نتيجة التحليل ثلاثي الأبعاد أعطت نتيجة أفضل مقارنة مع نتيجة التجربة.

Keywords: Tunnels, numerical modelling, heading stability, safety factor, face pressure

1. INTRODUCTION

At any tunnel project, determining the type of the ground and its strength is the most important factor that decides the construction method. In soft ground when the face failure is a problem or even reducing the amount of deformation in tunnelling process is the first criteria, using Tunnel Boring Machine (TBM) is the best choice in this situation. There are different types of TBM that used for the construction of tunnels. Those types are differing in the method of applying face pressure. Those methods of application are; Mechanical support, Compressed air, Earth pressure balance and Slurry support.

To encountering face failure, the applied face pressure must be within the limits. These upper and lower limits determine the critical conditions that beyond them failure may occurs. So, determining the applied face pressure is an important thing to avoid face failure or excessive deformations. The magnitude of minimum support pressure relates to overburden pressure of the soil and water pressure if found, but there is also the effect of soil arching that could leads to reduce the overburden pressure due to shear strength of the soil. Calculate this minimum pressure by using analytical equations, the affect of the soil arching need to be included separately. While in numerical modelling this affect is added automatically in calculation.

Different analytical and numerical analyses have been established to define the limits of face failure [3], [14] and [17-19]. In spite of the analytical methods are easy to use and give quick interpretation to the problem, the superiority of numerical methods cannot be avoided especially when dealing with complex three dimensional model on different layers of soil in heterogeneity site. An analytical solution that deals with 3D-Model and also the heterogeneity of the ground has been established [3]. But this analytical solution is coming with assumed ideal shape of wedge failure, while in reality idealization is rare to be found, and here numerical procedures can cope with this non-idealization more closely. It was found that numerical methods are more accurate computation techniques than analytical methods [1]. There are different simulation procedures in numerical modelling for accounting of how failure of the face could occur [13, 14]. As those simulations have differences in failure mode, they might give different safety factor with regard to face failure.

For the evaluation of numerical methods, they have to be compared with the reality. Despite the fact that measuring of face failure in tunnelling is hard to be obtained in reality, experimental centrifugal test data has been brought from literature [5].

This paper focuses on the determination of the lower limit for face stability of tunnelling by applying two different numerical procedures. Those procedures were applied in 2D and 3D FE-modelling to determine the minimum required support pressure and the safety factor using PLAXIS programs. The consideration is only for the calculation of the full failure of the tunnel face without taking in account the displacement-stage relationship in the simulation methods. It was found that it is very important to consider this relationship for shear strength reduction method, because the mode of failure is obtained with large amount of deformation. Also, Drained condition was not taken into account.

2. CASE STUDY: THE SECOND HEINENOORD TUNNEL

The cross-section consists of two tubes with an external diameter of 8.30*m*. The total length of the tunnel is 1350*m* with a bored part using slurry type TBM of 950*m* for each tunnel. The top ground layer is a fill underlain by two layer of sand and subsequent sand-clay layer [4].

Table 1: Heinenoord ground parameters [4]

Layer	γ_{dry} [KN/m^3]	$\gamma_{saturated}$ [KN/m^3]	ν [-]	E_{oed} [MPa]	c [kPa]	Φ [º]	K_0 [-]
1	16.5	17.2	0.34	8	3	27	0.58
2	20	20	0.30	40	0.01	35	0.47
3	20	20	0.30	120	0.01	35	0.47
4	20	20	0.32	48	7	31	0.55

Table 1 gives the properties of the soil layers, the depth of the layers from the ground surface was 4*m*, 19.75*m*, 23.25*m* and 27.5*m* for the layers 1, 2, 3 and 4 respectively. The depth of ground water table was 1.5*m* below ground surface. The depth of the tunnel axis was 16.65*m* below the ground surface [4].

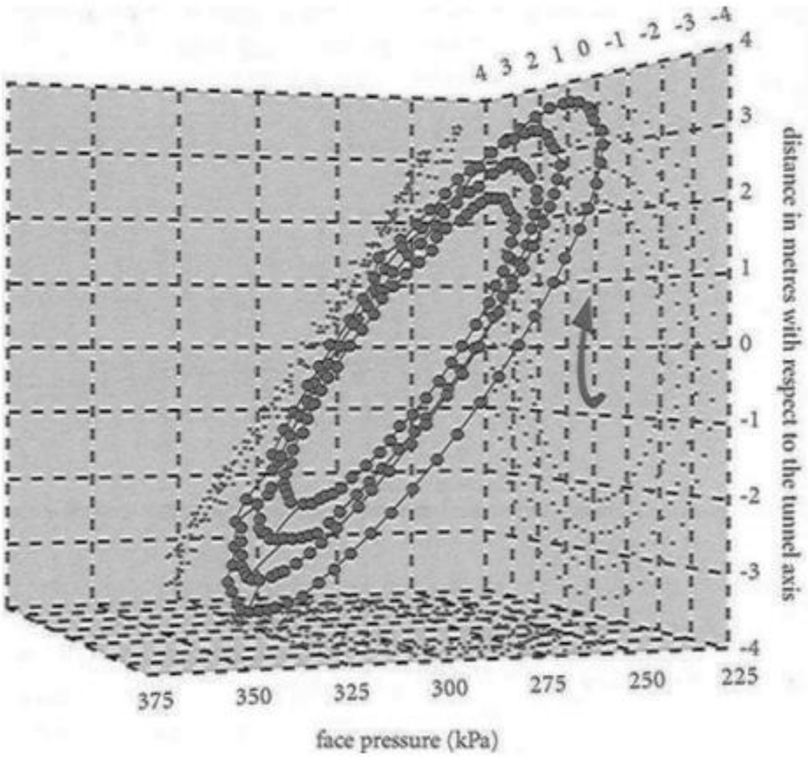

Fig. 1: Example of the measured pressures in the mixing chamber [5]

The Second Heinenoord Tunnel was constructed by using a slurry pressure TBM. The measured applied face pressure was used as applied force in the FE-modelling. The average pressure level and the pressure distribution in the mixing chamber strongly influenced face stability. As shown in Fig. 1, the measured pressure distribution was almost linear, from which it follows that the density in the mixing chamber was almost uniform [5].

3. NUMERICAL SIMULATIONS

In this paper, a finite element programs PLAXIS were used for the analysis. The minimum required support pressure at tunnel face was calculated by applying a support pressure at face which is equal to the total overburden pressure, and then it had been reduced until the face was collapse.

The other method for stability analysis is the Shear Strength Reduction (SSR-FEM). This method has been applied to slope stability analysis in two-dimensional as well as three-dimensional situations. It was used by reducing the shear strength parameters in the weakest surface, and then a value of the reduction for this parameter was obtained, which is defined as a safety factor. In SSR-FEM, the factor of safety is defined as

$$F = \frac{\tan \phi_{real}}{\tan \phi_{min}} = \frac{c_{real}}{c_{min}} \qquad (1)$$

Where: c is the cohesion of the soil, and ϕ is the angle of the internal friction of the soil, c_{min} and tan ϕ_{min} are minimum values as needed for equilibrium. These values are obtained by reducing the real shear strength parameters stepwise down to failure in an elastoplastic FE-analysis. The method originates from [7] and has been used recently for anchored slopes [11].

3.1 Three-Dimensional Finite Element Method

Fig. 2 shows a three-dimensional mesh block discretized using 15-nodes wedge element. This block has a length of 30*m*, a width of 30*m* and a height of 27.5*m*, and is used for the analysis of face stability for the considered case study. This model size is sufficient for analysing face stability without the concern of boundary condition. The depth of the model is ended into a very stiff layer. The soil parameters of the model drained ground behaviour with the Mohr-Coulomb (MC) Model are listed in Table 1. The MC-Model was used because the influence is only come from the strength parameters of the soil without stiffness parameters that concern with ground deformation. The TBM is modelled over 5 slices, each slice 1.5*m* in length and composed of shell (plate) elements, with a flexural rigidity $EI = 50 \cdot 10^3 kNm^2/m$, a normal stiffness $EA = 10 \cdot 10^6 kN/m$ and a weight $w = 38.15 kN/m^2$. The outer radius of TBM is 4.25*m*. A face pressure was applied at the tunnel face to support the soil. The face pressure is 230kN/m^2at the top of the TBM and it increases hydrostatically with depth according to a unit weight of 15kN/m^2 of the slurry.

3.2 Two-Dimensional Finite Element Method

2D block mesh using 6-noded triangular element was modelled. The block dimensions are of width of 40*m* and a depth of 27.5*m*. The soil parameters to model drained ground behaviour with the MC-Model are listed in Table1. The same face pressure and parameters as in the 3D FE-Modelling were applied.

4. DISCUSSION OF THE RESULTS

Two types of calculation were used. The first one was done by reducing the applied face pressure until the face is collapsed, and it is called plastic analysis in this paper. Another calculation method was used is phi-c reduction method, and it was based on the safety factor of the shear strength of the soil.

4.1 3D FE-Modelling of face stability

In 3D FE-modelling, the factor of minimum support pressure to the applied pressure was found to be about 0.5491 from plastic analysis. This leads to average value of minimum support pressure of 161.3kN/m^2, and this result is closely agreed with the experimental results of the centrifugal test, which it is equalled to 163.0kN/m^2 [5]. The difference was found to be only about 1%.

Fig. 2 shows the total displacement of 3D mesh at failure. It is obvious that the deformed zone of the soil did not reach the ground surface; this is because of the soil arching affect. This has been confirmed by Chambon and Corté (1994) [11] according to their centrifuge experiments. They concluded that for *C/D*> 1 (where *C* is the cover of the ground and *D* is the diameter of the tunnel) a soil silo will fully form but not reach ground surface. It is also obvious in the figure that the arch effect was developed at the top of the silo. This is shown by the top curvature shape of the silo. The figure also shows wedge shape failure zone was formed in the front of the tunnel head.

From phi-c reduction method, the safety factor for the failure of the model was found to be 23.31, which is a very high value. Fig. 3 shows the total displacement of failure face with fully formation of the soil silo. In front of the tunnel head, wedge failure mode is appeared.

The safety factor from plastic analysis is about 1.82 while in phi-c reduction analysis is about 23.31. The huge difference between these values is due to the reason that in phi-c reduction method a fully soil silo was reach the surface. Also, the maximum displacement was about 0.95*m* while the maximum one in plastic analysis was about 0.064*m*. This means that phi-c reduction method is not applicable for analysing the face stability of tunnelling. It is better to be used for slope stability problem rather than tunnel heading stability one.

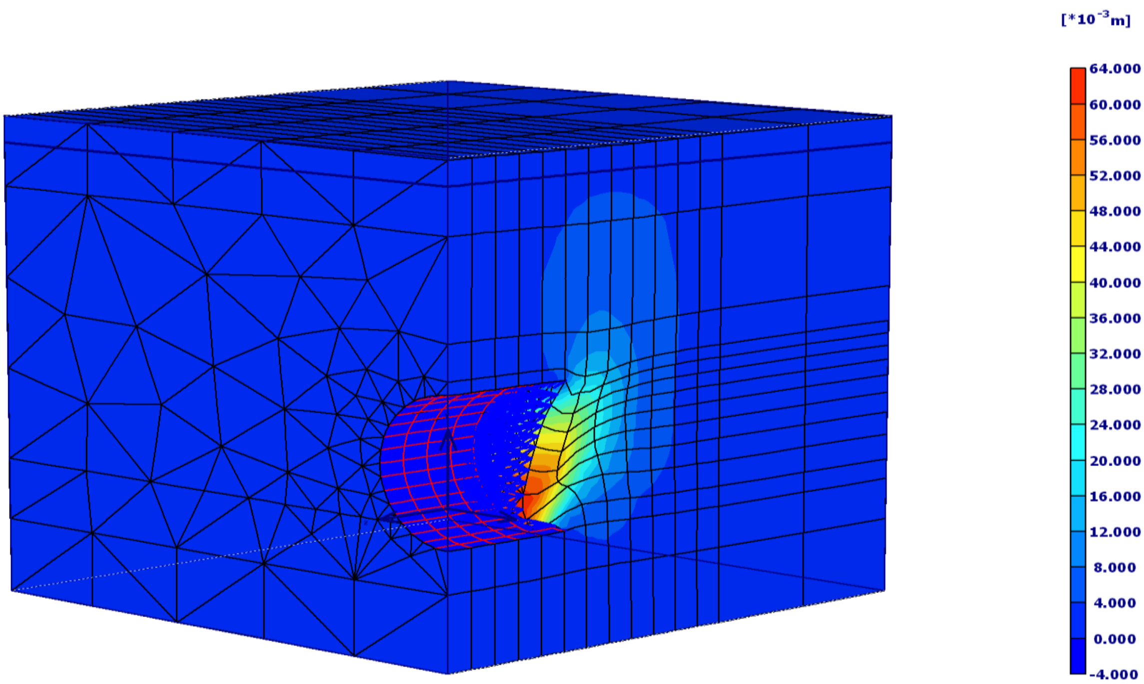

Fig. 2: deformed mesh of 3D plastic analysis when face collapses

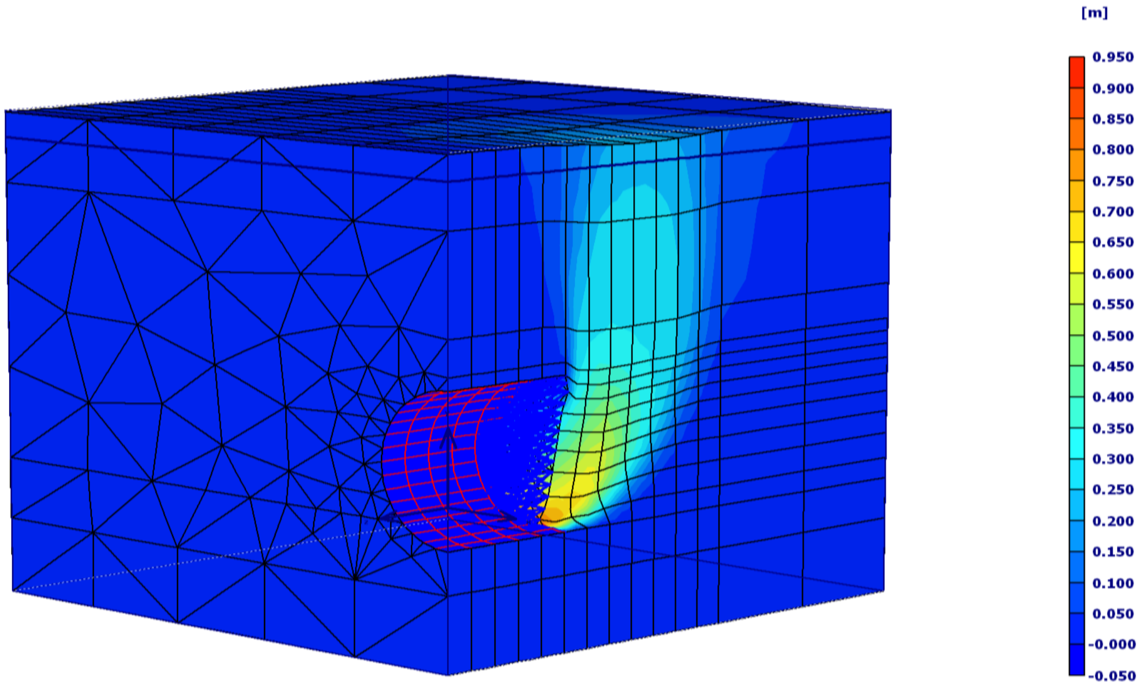

Fig. 3: Total displacement of deformed mesh when face collapses from 3D safety analysis

4.2 2D FE-Modelling of face stability

The calculation of the minimum support pressure using plastic analysis produced a factor of minimum support pressure at failure to applied pressure of about 0.5779. The average minimum support pressure is equal to 169.76kN/m^2. This value is slightly more than the experimental result. The difference was found to be about 4%. Fig. 4 shows the total displacement of deformed mesh when the face failure occurred. The figure shows more wide spread deformation when comparing with 3D analysis. This is because the missing third dimension in 2D analysis for the distribution of the deformation around it, so disturbed zone became wider. Same as in 3D analysis, the arch effect is shown by curved shape at top of the silo. Also, the wedge shape failure in the front of the face was appeared. The size of the wedge in 2D plastic analysis is bigger than for 3D plastic analysis, and also the rate of displacement is higher. This is due to the effect of the missing third dimension.

By using the 2D Phi-c reduction analysis method the factor of safety from the analysis was found to be about 13.579. Fig. 5 shows the total displacement of the deformed mesh when the tunnel face collapsed. Again wedge shape failure is appeared in the figure, and the width of the deformed zone is less than that appeared in 2D plastic analysis. The disturbed zone is full transferred to the surface.

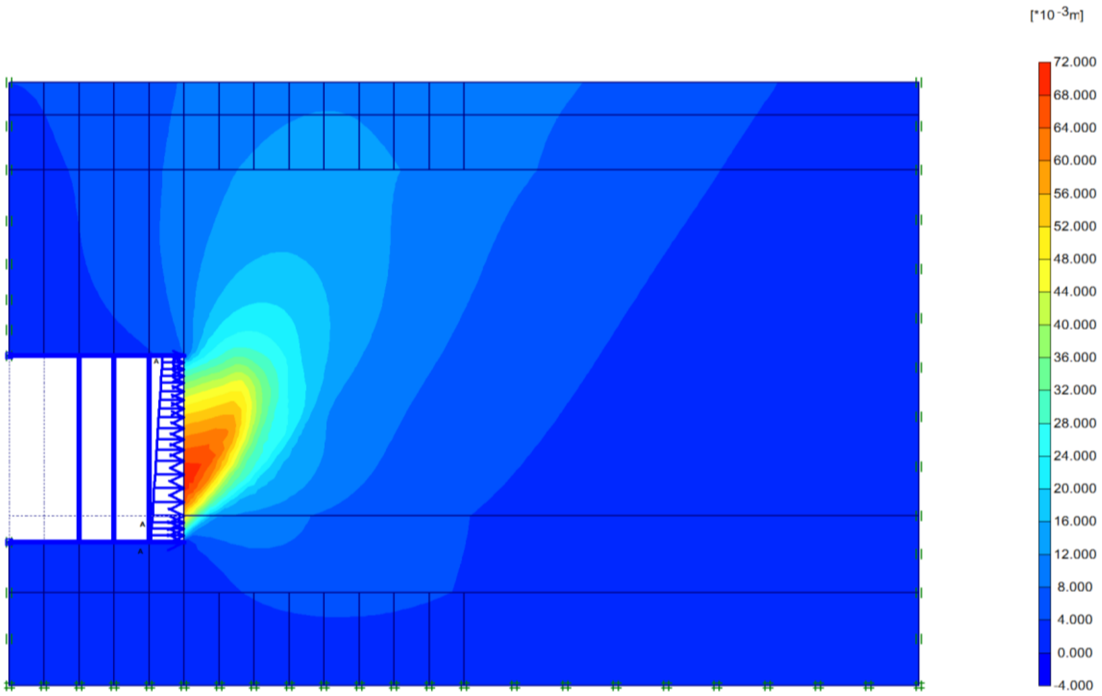

Fig.4: Total displacement when head collapses (2D plastic analysis)

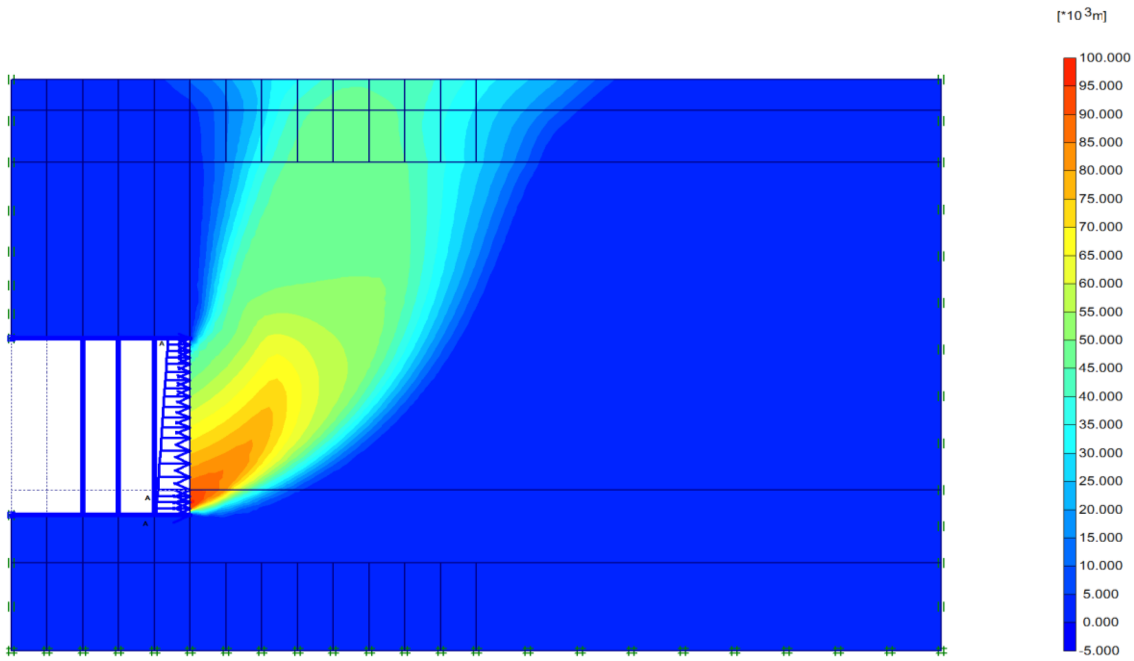

Fig. 5: Total displacement at face failure (2D phi-c reduction method)

The factor of safety from 2D phi-c reduction method is less than that is obtained from 3D one. It is about half the factor of safety from 3D analysis. A wider disturbed zone is not appeared in 2D safety analysis as the once appeared in 3D plastic analysis. 2D plastic analysis gives value closer to that obtained from 3D plastic analysis so it have more wider disturbed zone due to the effect of lost third dimension. In phi-c reduction method, the difference in the factor of safety between 2D and 3D analysis was bigger, so more symmetry in the shapes of failure was found. The effect of lost third dimension in 2D safety analysis was not revealed.

The difference between the time of calculation for 3D and 2D FEM for face stability was found to be not outside the practical time as in the ground deformation analysis [2] and [19]. This is because face stability analysis do not contain a large amount of phases for 3D model as in the ground deformation. And also the size of the model did not need to be large to encounter the effect of the boundary. This is because after the excavation phase, the occurred displacements are set to zero to eliminate the effect of boundary. Also as shown in figures 2 and 3 the disturbed zone did not lie outside the dimension of the model. As using 3D FE-Model in stability did not lead to time consuming, it is recommended to be used for analysis rather than 2D-Model. But also the difference is not huge, and a higher could lead to a more safety in tunnelling process, which is a very dangerous situation and any failure could lead to catastrophic situation.

5. CONCLUSIONS

The result from 3D plastic calculation matched well with the experimental result, and the difference was found to be about 1%. While in the result from 2D plastic analysis, the difference from the experimental result was found to be about 4%. In plastic analysis, 3D FEM gave more accurate result than 2D FEM, and this is because 3D arching effect in the 3D model gives better prediction than the 2D model.

The SSR-FE analysis gave a higher value of the safety factor compared to one deduced from the plastic analysis. This is because in the SSR-FEM analysis, the safety factor is obtained from the failure mode when the soil collapse reached the surface. This showed that SSR-FEM not applicable for tunnel heading stability problems and it may be more useful in slope stability problems.

Small difference was found between the results of the 3D and 2D plastic analysis, with a higher prediction from 2D FE-Model. Also, it was found using 3D FE-Model in face stability analysis is not computational demanding, and could be used in practical without significant effect in consuming time.

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