Analysis Of The Stability And Adequacy Level Temporary Support Using Natm Methods On The Excavation Of Railway Tunnel

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Abstract. NATM (New Austrian Tunneling Method) tunnel construction is able to proceed under instable states of working face, so the stability working face has been regardes as a key scientific issue. This paper based on finite element methods analysis theory established a model for the tunnel face stability. Furthermore, based on the model the influence of rock mass parameter with Generalized Hoek-Brown (2006) criteria and GSI for rock mass quality. NATM tunnel excavation has began top heading first excavate and then bench for fullface excavation. Excavation area are in breccia, sandstone, and claystone with rock mass strength classification is weak rock to intermediate rocks.In this research area, therefore before used for analyzing the opening stability obtained the monitoring data will be used for determining the stress condition of surrounding the openings by trial and error back analysis method. The results of the analysis at monitoring points, the cumulative displacement is average 200mm (<75mm), and displacement rate is average 24mm/days (>1mm/days), therefore it can be classified as unstable condition according to the Cording criteria (1974). Eventhough along the monitoring activy, the observed tunnel condition is in stable condition. Its means that the applied rock supporting system is strong enough. Strata control around the tunnels have significant settlement at the ground surface, and then the structure espescially ground settlement. The determining of ground support will be based for structure and the lithology of the tunnels for the evaluating stability of the existing openings.
Keywords : *displacement, convergence confinement, ground support, finite element.*

1. Introduction

Main problems in infrastructure development are the geological and topographical conditions. The development and progress of a region in fact can not be separated from the ability of the region to interact with other areas on a larger scale. The intended interactions are economic, social and resource activities. To support this, the means of transportation in this case land transportation becomes very vital. The means of land transportation is the highway (expressway) and railway tunnel. Railway Tunnel with a total length of about 473 m have been constructed in Notog, District Patikraja, Purwokerto, Region Central Java. When a large span tunnel or underground space is constructed in soft ground, it inevitably disturbs the in situ stress field, and the causes large ground displacements. Generally, softground is a multi phase porous medium and develops complex stress and strain variation while executing a tunnelling excavation.

During the construction of Notog tunnel in Purwokerto, Banyumas, Central Java is using the New Austrian Tunnelling Method (NATM) has been widely adopted due to its simple, adaptive, cost effective and technical feasibility. While tunnelling in soft ground in order to promote the tunnel face stability and to reduce ground displacement, the tunnel face is partitioned either horizontally or vertically to several temporary support, which is called sequential excavation method (SEM). The ground settlement induced by tunnelling varies according to the geometric parameters of partitioned face such as the size and dsitribution of the tunnels. Two step excavation in the Heading and Bench is the feature of NATM. The Displacement characteristics of the NATM are illustrated, and the numerical simulation results are presented to study of the ground displacement characteristics of various construction approaches. This paper provides an illustration of the influence of the tunnel advancing sequence on tunnelling performance.

2. Methods

2.1 Overview Tunnel

The Geological and hydrogeological conditions in this tunnel are very complex. The design them was revised frequently by the exposed tunnel face condition. As a result of diverse geological, sampling, and the physical and mechanical indices of engineering soft ground showed strong variability in the vertical direction. According the construction characteristics and especially to the construction sequence, Notog tunnel has 5 type of support such as, AN Type, BN Type, CN Type, DN Type and SPN type.

During tunnel excavations, the deformation of the rock mass caused by the unloading of itself will vary with time. The radial shape of the displacement around the tunnel has been estimated. Numerical methods were used to analyze the displacement around the face tunnel, thereby obtaining valuable information on the stability of the tunnel. Monitoring measurement has been extensively used to assess the response of the rock mass based on the field measurement of the rock mass deformation using special instrument (extensometer and total station). The measured results can reflect the behavior of the rock mass and supporting structures as well as evaluate the stability of the tunnel structures. However, the measured displacement of the rock mass is only a part of the complete deformation.

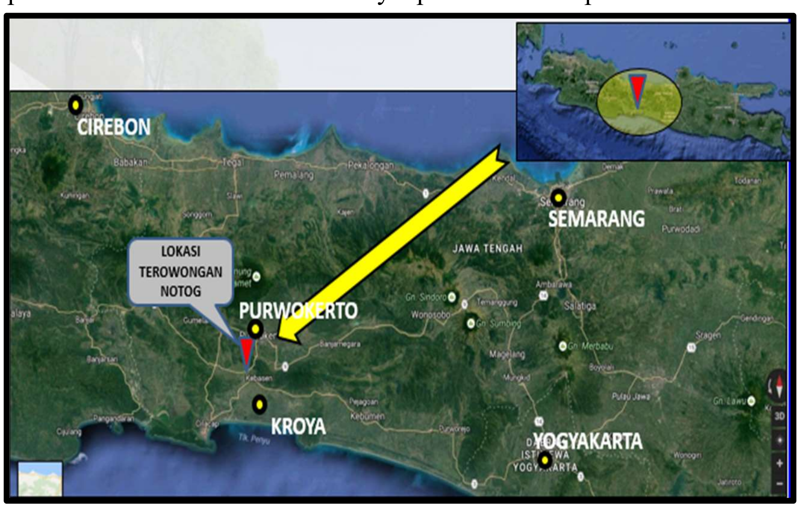


Figure 1. Location of the Notog Tunnel

Complete deformation includes those before, during and after the tunnel excavation. Hoek (1998) used a three-dimensional finite element method (FEM) to analyze the rock mass deformation that surrounds a circular tunnel advancing through a weak rock mass, thereby showing a deformation pattern in the rock mass that surround an advancing tunnel.

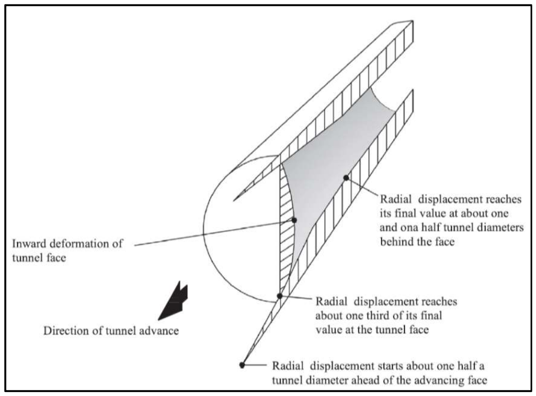


Figure 2. The Deformation pattern in the rock mass that surrounds an advancing tunnel

This study based on a soft ground or sediment medium rock in the Notog tunnel in Banyumas, Central Java Province to measure the tunnel displacement in site, including the crown settlement (vertical displacement and horizontal displacement. The numerical modelling of this tunnel was analyzed by *Phase*^{2} (RS^2) (a simulation software developed by Rocscience). In this paper, the study is improved given that the crown settlement and horizontal displacement were used together to obtain the parameters in a sediment medium rock/soft ground rock mass tunnel. Thereafter, the deformation was studied in a soft ground rock mass tunnel with different construction schemes to guide the construction and verivy the benefit of the optimize construction scheme.

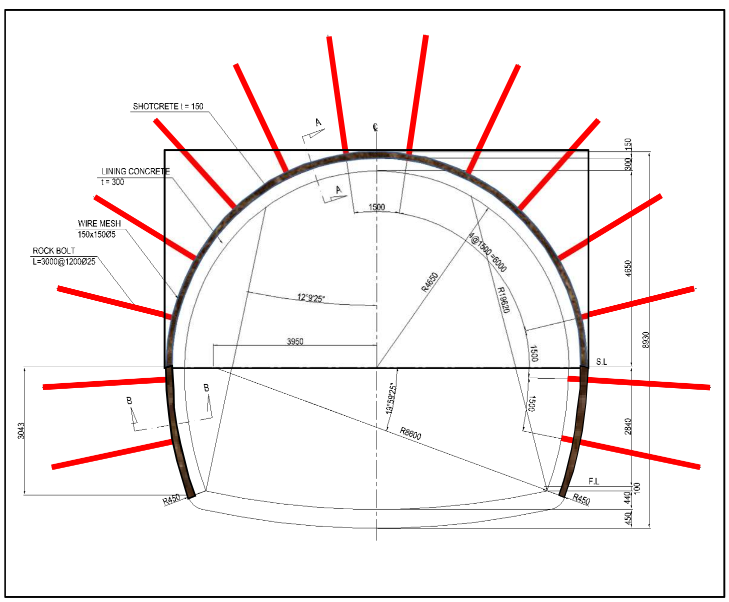


Figure 3. Support structure of tunnel (all unit written in mm)

2.2 Deformation monitoring data

The monitoring results of each section for horizontal displacement and crown settlement are shown in table 1. This results illustrated that the controlling the tunnel deformation using the optimized scheme is better than using the original scheme. We can easily get from Table 1. That the crown settlement of left side and right side are different due to the different rock mass at left and right side. However, the simulated model is always a symmetrical structure, the displacement left and right side is same. So, in order to balance the difference between simulation, we should choose a section of which the left and right side settlement are the same.

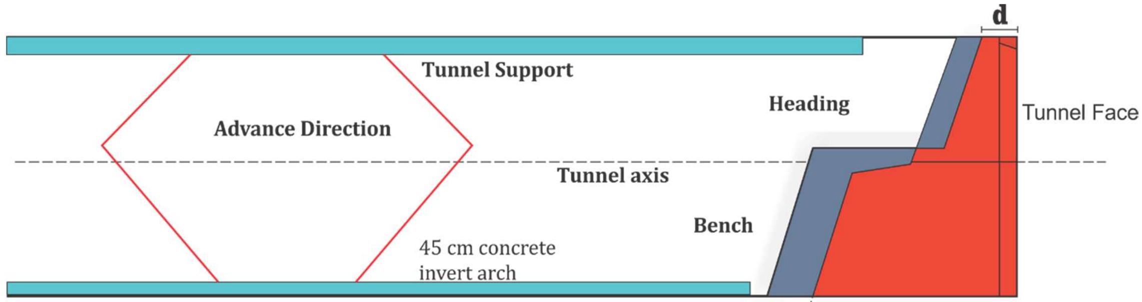


Figure 4. Longitudinal section for Notog Tunnel excavation

Table 1. Monitoring scheme of Notog Tunnel

Measurement content	Instrument	Measurement frequency	
		Time after construction	Frequency
Horizontal Displacement	Total station	1-20 days	once a day
		20 days - 3 month	once a day
		a years	once or a twice a day
Crown settlement	Total station	1-20 days	once a day
		20 days - 3 month	once a day
		a years	once or a twice a day

2.3 Finite element analysis

The tunnel excavation was simulated using the two dimensional finite element analysis with the program RS^2 developed by Rocscience Incorporated. A amulti stage two dimensional plane strain model was developed to simulate three dimensional advance of the tunnel face. This was accomplished using the core replacement technique, which simulates progressive tunnel excavation by incrementally replacing the tunnel core with a new material possessing identical or reduced stiffness without any initial element loading. Since the elements have no initial loading, the internal pressure is reduced to zero each time the material is replaced, causing a disruption to radial equilibrium. To obtain equilibrium by compressing the new tunnel core, the displace- ment at the tunnel boundary increase, simulating the reduction in internal resistance and the increase in convergence as the excavation face advances. The replacement material is then removed altogether to simulate a long open tunnel with plane- strain conditions.

This results in matrix equations relating the input at specified points in the elements (the nodes) to the output at these same points. In order to solve equations over large regions, the matrix equations for the smaller sub-regions can be summed node by node, resulting in global matrix equations, or "element-by-element" techniques can be employed to avoid creating (large) global matrices. The method is already described in many texts, for example, Zienkiewicz and Taylor (1989), Strang and Fix (1973), Cook *et al*. (1989), and Rao (1989), but the principles will briefly be described in this chapter in order to establish a notation and to set the scene for the later descriptions of programming techniques.

If the wall can be considered to be of unit thickness and in a state of plane stress, (Timoshenko and Goodier, 1982) the equations to be solved are the following :

1. Equilibrium

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} + F_x = 0$$

$$\frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_x}{\partial y} + F_y = 0 \quad \text{..........} (1)$$

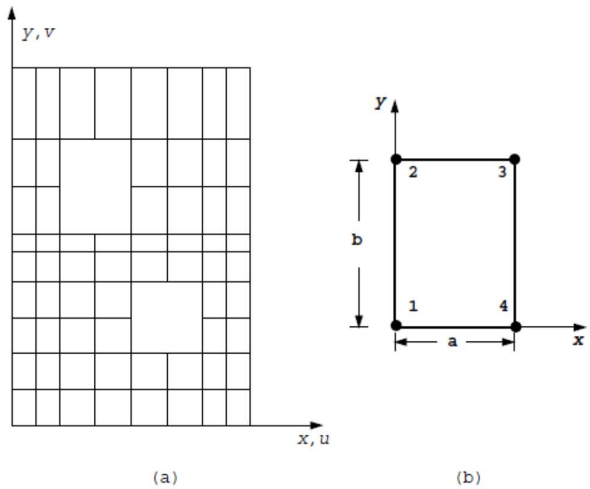


Figure 5. (a) Shear wall with openings. (b) Typical rectangular 4-node element

Where of the component σ_x, σ_y, dan τ_{xy} are the only non-zero component stress *non zero* and F_x, F_y are *body forces* (unit of force/length[3]).

2. Constitutive (plane stress)

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix} = \frac{E}{1-v^2} \begin{bmatrix} 1 & v & 0 \\ v & 1 & 0 \\ 0 & 0 & \frac{1-v}{2} \end{bmatrix} \begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{Bmatrix} \quad (2)$$

where E is *Young's Modulus, v Poisson's ratio* and ϵ_x, ϵ_y and γ_{xy} are the independent small strain components.

3. Strain-displacement

$$\begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{Bmatrix} = \begin{bmatrix} \frac{\partial}{\partial x} & 0 \\ 0 & \frac{\partial}{\partial y} \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{bmatrix} \begin{Bmatrix} u \\ v \end{Bmatrix} \quad (3)$$

where *u* and *v* are the components of displacement in the *x* dan *y directions.* Equations 2 and 3 can be written in form :

$$[\boldsymbol{A}]^{\boldsymbol{T}}\{\boldsymbol{\sigma}\} = -\{\boldsymbol{f}\}$$

$$\{\boldsymbol{\sigma}\} = [\boldsymbol{D}]\{\boldsymbol{\epsilon}\}$$

$$\{\boldsymbol{\epsilon}\} = [\boldsymbol{A}]\{\boldsymbol{e}\} \quad (4)$$

Where ;

$$\{\boldsymbol{\sigma}\} = \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{Bmatrix}, \ \{\boldsymbol{\epsilon}\} = \begin{Bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma_{xy} \end{Bmatrix}, \ \{\boldsymbol{e}\} = \begin{Bmatrix} u \\ v \end{Bmatrix}, \ \{\boldsymbol{f}\} = \begin{Bmatrix} F_x \\ F_y \end{Bmatrix} \quad (5)$$

$$[\boldsymbol{A}] = \begin{bmatrix} \frac{\partial}{\partial x} & 0 \\ 0 & \frac{\partial}{\partial y} \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} \end{bmatrix}, \ [\boldsymbol{D}] = \frac{E}{1-v^2} \begin{bmatrix} 1 & v & 0 \\ v & 1 & 0 \\ 0 & 0 & \frac{1-v}{2} \end{bmatrix} \quad \text{(6)}$$

We shall only be concerned in this book with "displacement" formulations in which $\{\boldsymbol{\sigma}\}$ and $\{\boldsymbol{\epsilon}\}$ are eliminated from 3.15 as follows :

$$[\boldsymbol{A}]^{\boldsymbol{T}}\{\boldsymbol{\sigma}\} = -\{\boldsymbol{f}\}$$

$$[\boldsymbol{A}]^{\boldsymbol{T}}[\boldsymbol{D}]\{\in\} = -\{\boldsymbol{f}\}$$

$$[\boldsymbol{A}]^{\boldsymbol{T}}[\boldsymbol{D}][\boldsymbol{A}]\{\boldsymbol{e}\} = -\{\boldsymbol{f}\} \quad \text{(7)}$$

Writing out (3.18) in full we have :

$$\frac{E}{1-v^2}\begin{Bmatrix} \frac{\partial^2 u}{\partial x^2} + \frac{1-v}{2}\frac{\partial^2 u}{\partial y^2} + v\frac{\partial^2 v}{\partial x \partial y} + \frac{1-v}{2}\frac{\partial^2 v}{\partial y \partial x} \\ v\frac{\partial^2 u}{\partial y \partial x} + \frac{1-v}{2}\frac{\partial^2 u}{\partial x \partial y} + \frac{1-v}{2}\frac{\partial^2 v}{\partial x^2} + \frac{\partial^2 v}{\partial y^2} \end{Bmatrix} = \begin{Bmatrix} -F_x \\ -F_y \end{Bmatrix} \quad \text{(8)}$$

3. Result

Which is a pair of simultaneous partial differential equations in the continuous space variables *u* and *v*. As usual these can be solved by discretizing over element using shape functions (here we assume the same functions in the *x*- and *y*- directions).

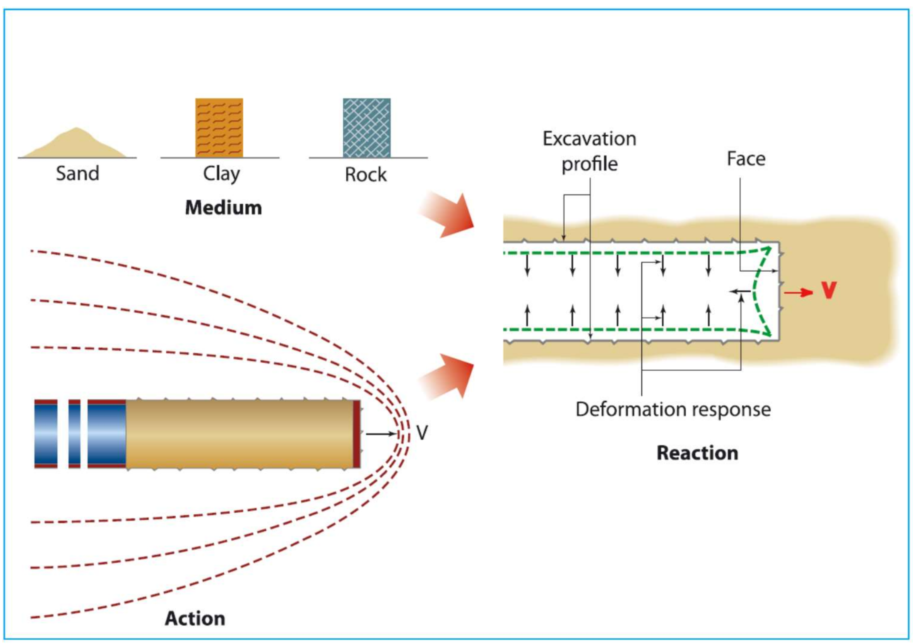


Figure 6. Excavation and stabilization operations appropriately as a function of different stress-strain conditions

The reaction is the deformation response of the medium to the action of excavation. It is generated ahead of the face within the area that is disturbed, following the generations of greater stress in the medium around the cavity. If on passing from triaxial to a plane stress state during tunnel advance, the progressive decrease in the confinement pressure at the face ($\sigma_3 = 0$) produces stress in the elastic range ahead of the face, then the wall that is freed by excavation (the face) remains stable with limited and absolutely negligible deformations. The reaction is also important and the wall that freed by excavation and give rise to a condition of short term stability.

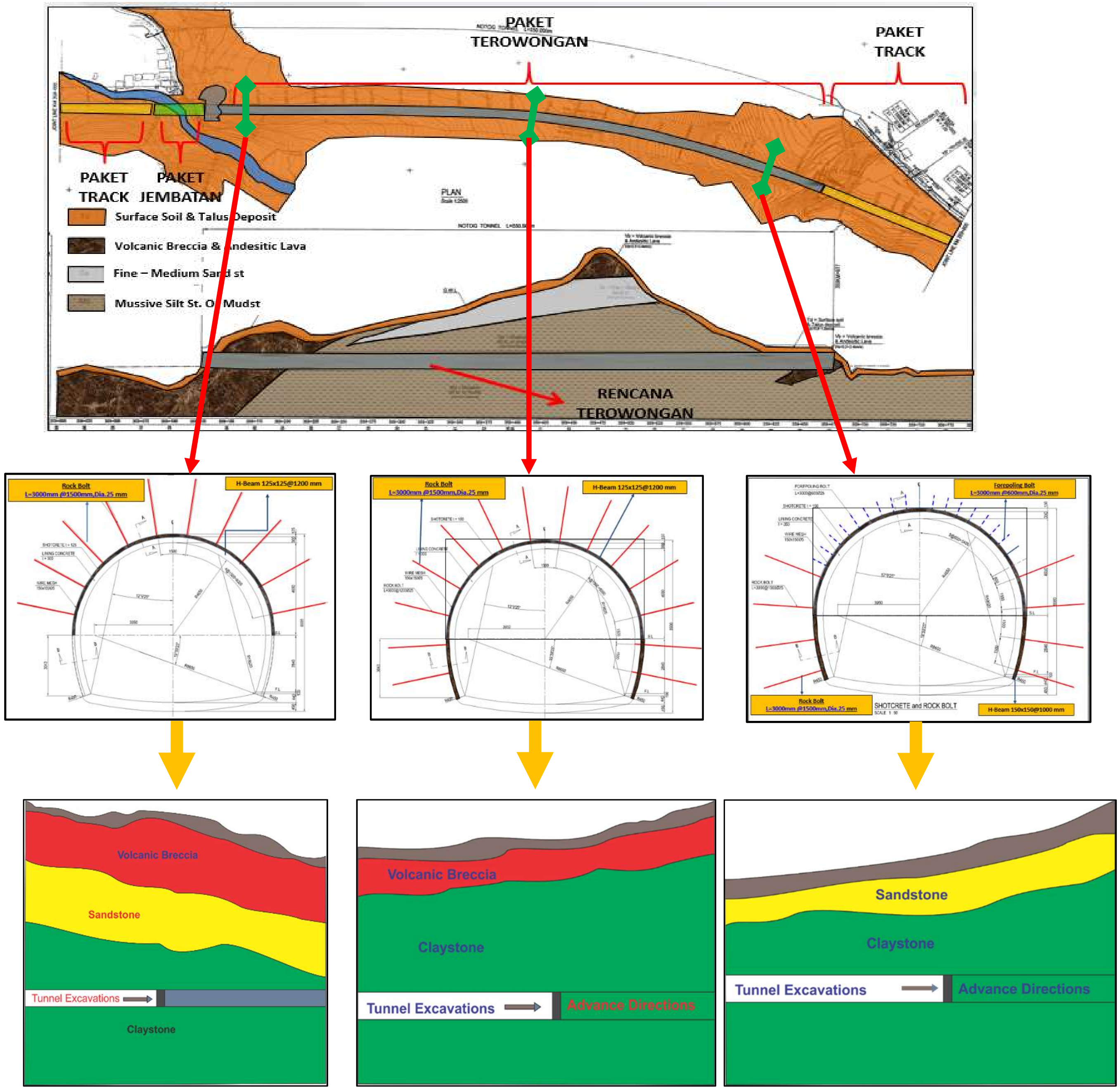
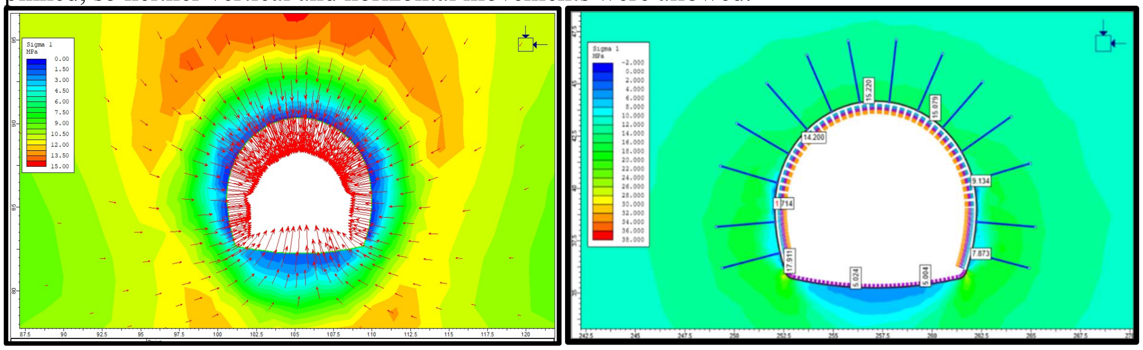


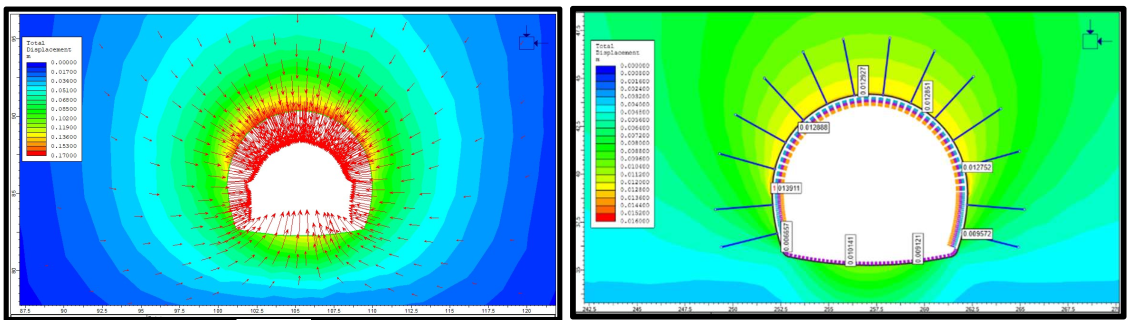
Figure 7. Sketch of whole project, excavations sequenes and geological condition for Notog Tunnel

As mentioned above, the ground surface and subsurface settlement induced by tunnelling varies according to different construction approaches even under the same cross section and geological conditions. Therefore, it makes sense to select a suitable construction approach for large span tunnel projects in loess ground. Numerical simulation has apparent advantages in decision making compared with inference simply by the field data, because different geological conditions, construction qualities and overburden depths are associated with different tunnelling testing sections. In this section, numerical analyses were conducted by a finite element computer program *Rocscience Phase*^{2} are reviewed to evaluate the above mentioned construction approaches based on the same geometrical conditions. The numerical simulations for the three construction approaches followed the actual excavation steps. In this analysis, tunnels were excavated and supported in a total of 9 stages to accomplish the whole cross-section, respectively. The model was fixed in the horizontal direction at each side, which means that vertical movement was allowed, and the bottom part of the boundary was pinned, so neither vertical and horizontal movements were allowed.



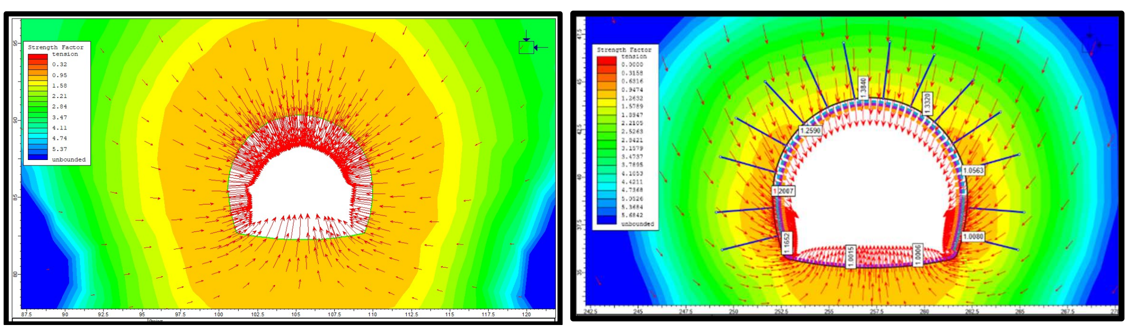
(a) (b)

Figure 8. (a) Simulations final Sigma 1 excavation without support (b). Simulated final Sigma 1 with Supporting.



(a) (b)

Figure 9. (a) Simulated with Total Displacement before Supporting (b) Simulated with Total Displacement after Supporting.



(a) (b)

Figure 10. (a) Simulated Model Strength Factor before supporting (b). Simulated Model with supporting tunnel excavation.

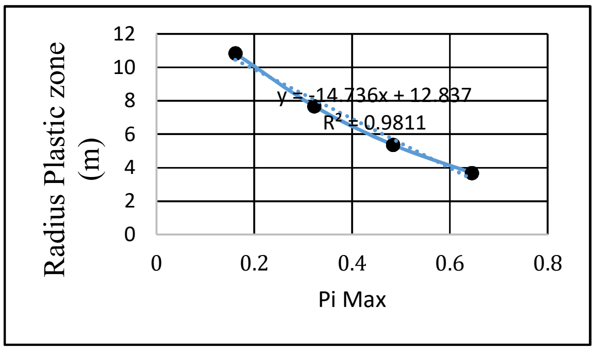
Value of accounted for about 20-30 % the final displacement occurred before the arriving the tunnel face and which was able to be measured. Based on the result of numerical simulation, we get the value of decrease of displacement when tunnel before support and after support. it is indicated in figure 8 that there is a change in the contour where before buffered around the tunnel the sigma value 1 becomes larger, and when the tunnel is propped up there is a decrease in the value of the major stress.

Therefore, strength factor condition there is a change of stress distribution that causes difference before and after support. this can be seen in figures 9 and 10, the displacement and the strength factor values change after supporting.

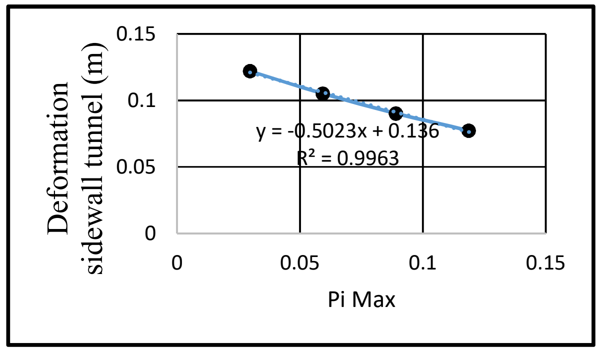
4. Discussion and Conclusion

The Notog tunnelling works have shown that different mechanical behaviors resulted in response to the adoption of different methods for advancing the tunnel face. In this study, a series of insitu stress test and numerical simulations were conducted for revealing the displacement characteristics span tunnels. According the result taken from these analysis, the following conclusions can be drawn :

a. Strata control around the tunnels have significant settlement at the ground surface, and then the structure espescially ground settlement. The determining of ground support will be based for structure and the lithology of the tunnels for the evaluating stability of the existing openings.
b. Based on monitoring data, used Cording (1974), Zhenxiang (1984) and Ghose and Ghoose (1995) the value of displacement the value of displacement can be categorized in stable conditions.



(a)



(b)

Figure 11. The value of Radius of Plastic zone vs *Pi Max* (b). The Value of Deformation sidewall vs *Pi Max*

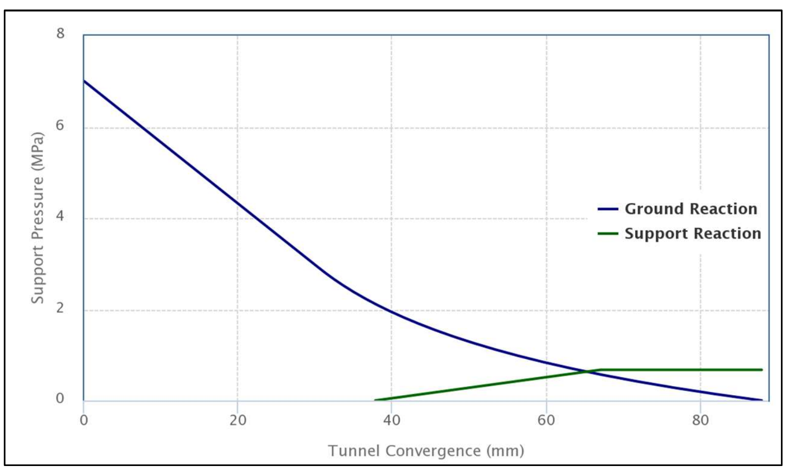


Figure 12. Convergence curve with the support reaction

c. Based on figure 12, can be obtained result that the maximum convergence the tunnel is 38 mm with 7 MPa support pressure, that's the indicates the support adequate for supporting the ground reaction.
d. Value of plastic zone can be obtained result along 10 m until 12 m tend to be affected plastic zone so that adequate supporting is required.

5. Acknowledgment

The author gratefully acknowledge the support by Mr. Eko Septiyanto, Project Manager of PT PP (Persero), Tbk. Notog Tunnel BH 1440 which has allowed to this research, and special thank for Dr. Ir. Singgih Saptono, MT. vice Rector III, UPN "Veteran" Yogyakarta which has guided and patient the completion in this research.

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