Analysis of the double-tunnels interaction by using finite element method: influence of tunnels position and excavation sequence

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Abstract. The advancing of development requires the use of underground area for the construction of transportation infrastructure facilities. Construction of double-tunnel at the same time, with the position of tunnel which is adjacent to the others tunnel may be done horizontally manner. The excavation method which is used in soft soil tunnel construction is three-bench seven-step excavation method that can stabilize the tunnel face but it does not require any additional support, it has been widely used in tunnels with large cross-sections for highway. Since, both the tunnels spacing and the excavation sequence affect the displacement and stresses in the lining, it is major interest to study the influence of these factors on the tunnel design. Numerical simulations are conducted to reveal displacement characteristics and obtain optimal construction approach for tunnels in Indonesia. The method which is used for calculation is very suitable for estimating the stability level of tunnels with reliable result. This paper presents analysis of this issue with a particular interest for the optimization of both tunnels spacing of the double-tunnels and the excavation sequence. The analysis conduct in two dimensions of analysis.

Keywords: soft soil, double tunnels, NATM, excavation sequence, finite element

1. Introduction

The advancing of development requires the use of underground areas for the construction of transportation infrastructure facilities, one of which is the construction of tunnels. Highway tunnel have been constructed in Cisumdawu, Pamulihan District, Sumedang, Region West Java with a total length of about 472 m. The tunnel penetrates the hill with weak rock or soft soil material conditions (Verruijt, 2001). Cisumdawu Tunnel is a double and shallow tunnels. Therefore, the proper excavation method is needed, so the excavation activities will not interfere another activities above the surface.

Tunnel excavation has an effect on the strength of the surrounding rock, due to rock stress distribution changes. Tunnel face will be deformed and allow a new stability of the tunnel. This behavior is indicated by the displacement of the tunnel walls. Tunnel excavation disturbs the in situ stress field, and causes ground displacements. The double tunnels interaction will affect the stress and displacement conditions

around the tunnel, ground surface displacement and load support. Therefore, the study of double tunnels interaction is needed and the interaction effect on the stresses and displacements around the tunnel and the load support must be known. It is achieved by performing a numerical analyzes using the finite element method for multiple tunnel models in the range of parameters. The numerical simulation results are presented to study the ground displacement characteristics of various construction approaches. The results are presented in two parts, in the first part the main parameter examined is influence of the distance between two tunnels. While in the second part, the main parameter examined is the effects of tunnel excavation sequence. In the analysis, influence of tunnel depth and support conditions are considered.

2.   Theoritical Basis

2.1. Soil Mass Classification
Soil classification is a system of regulating different types of soil but having similar properties into groups and subgroups based on usage (Das, 1995). Classification system provides an easy language to explain briefly the general characteristics of soils that varies greatly without detailed explanation. There are several types of soils classification systems that are generally used as a result of the development of an existing classification system. Some of these systems take into account grain size distribution and Atterberg boundaries. These systems are AASHTO classification system and Unified classification system. The AASHTO classification system is generally used by highway departments in all states in the United States. Whereas the Unified classification system is generally preferred by geotechnical experts for other technical needs.

2.2. In-Situ Stresses
Vertical in situ stress on soil or rock is a function of depth.

$$\sigma_v = \gamma h \tag{1}$$

where,
σ_v is vertical stresses
γ is unit weight of the overlying rock
h is depth below surface.

The horizontal stresses acting on rock elements at a depth z below the surface are much more difficult to estimate than the vertical stresses. Generally, the ratio of the average horizontal stress to the vertical stress is denoted by the letter k such that:

$$\sigma_h = k\sigma_v = k\gamma z \tag{2}$$

where,
σ_h is horizontal stresses
k is independent of depth, $k = v/(1-v)$
v is Poisson's ratio.

Horizontal stresses measurements at civil and mining sites around the world show that the ratio of k tends to be high at shallow depths and that it decreases at depth (Brown and Hoek, 1978, Herget, 1988).

2.3. Induced Stresses
Before excavation is carried out, the rock mass is on equilibrium condition. And after excavation, the equilibrium becomes disturbed and the stresses in the vicinity of the new opening are redistributed. As a result of the excavation, the stresses from excavated mass will be transferred to side of tunnel. As a result of the stresses transfer, there is an accumulation of stresses on the surface of tunnel excavation. To find out the distribution of stresses around the tunnel, Kirsch (1898) equation can be used. Illustration of induced stresses due to tunnel excavation can be seen on Figure 1.
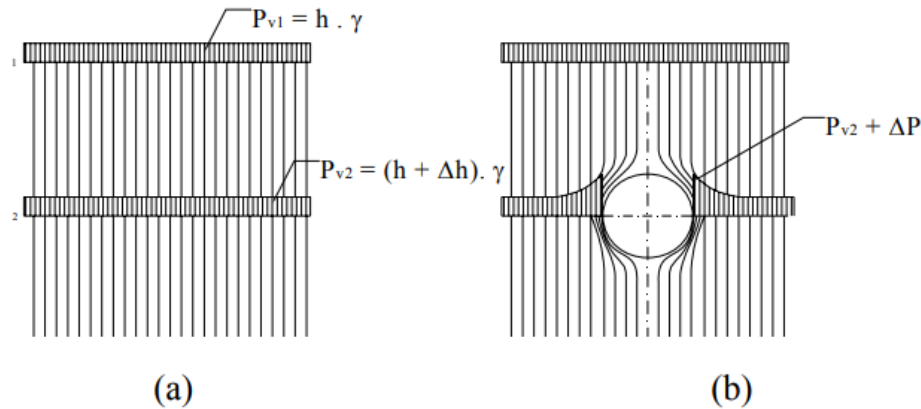
Figure 1. (a) Stress conditions at initial conditions (b) Conditions after tunnel excavation (Szechy, 1973)

Kirsch (1898) formed the radial stress formula (σ_r), tangential stress (σ_θ) and shear stress ($\tau_{r\theta}$) around the tunnel in the following equation.

$$\sigma_r = \left[\left(\frac{\sigma_v + \sigma_H}{2}\right)\left(1 - \frac{R^2}{r^2}\right)\right] + \left[\left(\frac{\sigma_v - \sigma_H}{2}\right)\left[1 - \frac{4R^2}{r^2} + \frac{3R^4}{r^4}\right]\cos 2\theta\right] \tag{3}$$

$$\sigma_\theta = \left[\left(\frac{\sigma_v + \sigma_H}{2}\right)\left(1 - \frac{R^2}{r^2}\right)\right] + \left[\left(\frac{\sigma_v - \sigma_H}{2}\right)\left[1 + \frac{3R^4}{r^4}\right]\cos 2\theta\right] \tag{4}$$

$$\tau_{r\theta} = \left[-\left(\frac{\sigma_v - \sigma_H}{2}\right)\left(1 + \frac{2R^2}{r^2} - \frac{3R^4}{r^4}\right)\sin 2\theta\right] \tag{5}$$

where,
R is tunnel radius
θ is the angle formed clockwise to the point of observation.

2.4. Tunnel Deformation

According to Bray (1967), excavations that produce large stresses (tangential stresses is greater than half of unconfined compressive strength), will cause weakening to certain locations, that is a plastic zone. It is assumed that the circular tunnel with radius r_0 is subjected to hydrostatic pressure p_0 and internal support pressure p_i as illustrated in Figure 2.
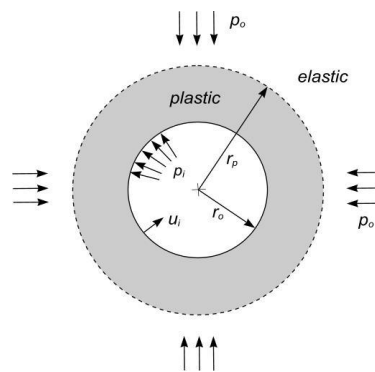

Figure 2. Plastic zone and elastic according to Bray (Goodman, 1989)

Rock mass collapse around the tunnel occurs when the internal pressure provided by support is less than p_{cr} critical support pressure, which is defined by:

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$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1 + k} \tag{6}$$

if internal support pressure p_i is greater than critical support pressure p_{cr}, there is no collapse, the behavior of rock mass around the tunnel is elastic and the radial elastic displacement inward from tunnel wall is shown by the equation:

$$u_{ie} = \frac{r_o(1 + v)}{E_m}(P_o - P_i) \tag{7}$$

where,
E_m is Young's modulus
v is Poisson's ratio

when internal support pressure p_i is less than the critical support pressure p_{cr}, collapse occurs and the radius of plastic zone r_p around the tunnel is shown by the equation:

$$r_p = r_0 \left[\frac{2(p_0(k - 1) + \sigma_{cm})}{(1 + k)\big((k - 1)p_i + \sigma_{cm}\big)} \right]^{\frac{1}{(k-1)}} \tag{8}$$

for plastic failure, the total radial displacement inward from the tunnel wall is shown by the equation:

$$u_{ip} = \frac{r_0(1 + v)}{E} \left[2(1 - v)(p_0 - p_{cr}) \left(\frac{r_p}{r_0}\right)^2 - (1 - 2v)(p_0 - p_i) \right] \tag{9}$$

The curves depicted in Figure 3 and Figure 4 are defined on the equation below:

$$\frac{d_p}{d_0} = \left(1{,}25 - 0{,}625 \frac{p_i}{p_0}\right) \frac{\sigma_{cm}}{p_0}^{\left(\frac{p_i}{p_0} - 0{,}57\right)} \tag{10}$$

$$\frac{\delta_i}{d_0} = \left(0{,}002 - 0{,}0025 \frac{p_i}{p_0}\right) \frac{\sigma_{cm}}{p_0}^{\left(2{,}4\frac{p_i}{p_0} - 2\right)} \tag{11}$$

where,
d_p is plastic zone radius
δ_i is tunnel wall deformation
d_0 is tunnel radius (m)
σ_{cm} is rock mass strength $(2c \cos \emptyset /(1 - \sin \emptyset))$
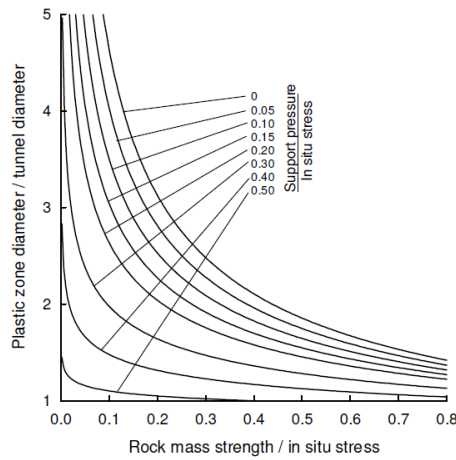

Figure 3. Size of plastic zone compared to support pressure (E. Hoek, dkk., 1993)
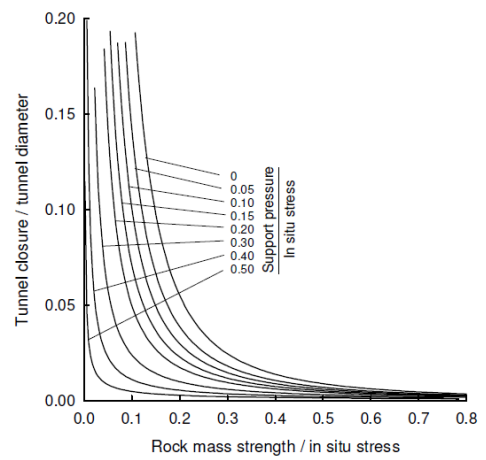
Figure 4. Tunnel deformation is compared to support pressure (E. Hoek, dkk., 1993)

2.5. Mohr-Coulomb Failure Criterion

Mohr-Coulomb failure criterion, it is necessary to determine the friction angle $\emptyset$ and cohesion c for each rock mass and stresses range. Here are the equations for friction angle $\emptyset$ and cohesion c:

$$\emptyset = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma_{3n})^{a-1}}\right] \tag{12}$$

$$c = \frac{\sigma_{ci}\lfloor(1+2a)s + (1-a)m_b\sigma_{3n}\rfloor(s + m_b\sigma_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma_{3n})^{a-1})/((1+a)(2+a))}} \tag{13}$$

where, $\sigma_{3n} = \sigma_{3max}/\sigma_{ci}$

Note that the value of σ_{3max}, the upper limit of the boundary stresses where the relationship between Hoek-Brown and Mohr-Coulomb criteria is considered, must be determined for each individual case. Mohr-Coulomb's shear strength τ, for the given normal stress σ, it is found by substitution of c and $\emptyset$ to the equation:

$$\tau = c + \sigma\tan\emptyset \tag{14}$$

equal plot, in terms of major and minor stresses, defined by:

$$\sigma_1 = \sigma_{cm} + k\sigma_3 = \frac{2c\cos\emptyset}{1 - \sin\emptyset} + \frac{1+\sin\emptyset}{1-\sin\emptyset}\sigma_3 \tag{15}$$
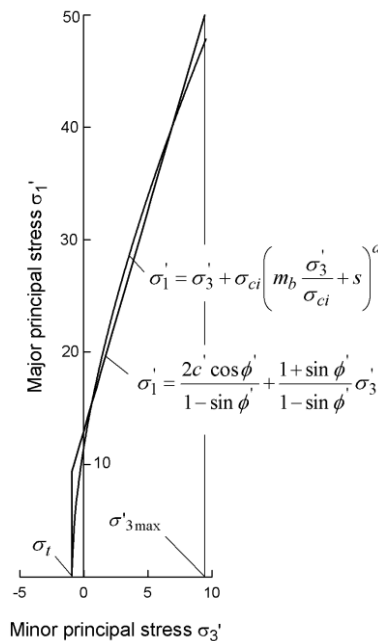

Figure 5. Mohr-Coulomb failure criterion

3.   Research Result

3.1.   Soil Classification

The soil layer in the Cisumdawu tunnel excavation areas are varies greatly based on the size of the soil particles, so it can be referred to as gravel, sand, silt, or clay, depending on the particle size that is dominant in the soil. There is a classification of soil in the Cisumdawu tunnel excavation area based on

grain size analysis. The soils type on Cisumdawu tunnel excavation area are sandy clayey silt, clayey sandy silt, and sandy silty clay.

3.2. *Physical and mechanical properties*

3.2.1. Physical properties. Physical properties test is a test that does not damage rock samples (undistructive test). This test obtains parameters of the physical properties of the soil including water content, unit weight, dry density, specific gravity, saturability, void ratio, and porosity. Table 1 is the results of physical properties test of rock samples.

Table 1. Physical properties data of soil material

Sample no.	Depth (m)	Water content (%)	Unit weight (gr/cm^3)	Dry Density (gr/cm^3)	Specific Grafity (gr/cm^3)	Saturability (%)	Void Ratio -	Porosity (%)
UDS 1	5.50	49.95	1.60	1.07	2.65	89.11	1.48	59.75
UDS 2	10.00	55.75	1.60	1.03	2.67	92.86	1.61	61.62
UDS 3	15.50	64.01	1.58	0.96	2.68	96.34	1.78	64.01
UDS 4	20.00	64.09	1.57	0.96	2.63	96.24	1.75	63.64
UDS 5	25.50	62.09	1.59	0.98	2.63	97.15	1.68	62.73
DS 1	30.00	64.54	1.62	0.99	2.67	100.65	1.71	63.16
DS 2	35.50	60.57	1.60	0.99	2.68	95.93	1.69	62.81
DS 3	40.00	61.86	1.53	0.95	2.67	90.63	1.82	64.53
DS 4	45.50	59.27	1.63	1.02	2.68	98.30	1.61	61.73
DS 5	50.00	63.51	1.63	1.00	2.67	101.48	1.67	62.52
DS 6	55.50	74.41	1.48	0.85	2.68	92.28	2.16	68.39
DS 7	60.00	37.72	1.62	1.17	2.67	78.84	1.28	56.10

3.2.2. Mechanical properties. Mechanical properties test is a test that damages the soil samples (distructive test). Soil mechanical properties shows the behavior of the soil when obtaining a force. Mechanical properties test aims to obtain values of friction angle and cohesion that used for analysis. Mechanical properties test was carried out in 4 types of tests namely Unconsolidated Undrained Test, Triaxial Consolidated Undrained Test, Direct Shear Test, and Unconfined Compressive Test. Table 2 is the results of mechanical properties test of rock samples.

Table 2. Mechanical properties data of soil material

Sample no.	Triakxial CU φ (deg)	C (kg/cm^2)	φ' (deg)	C' (kg/cm^2)	Triaxial UU φ (deg)	C (kg/cm^2)	Direct Shear UU φ (deg)	C (kg/cm^2)	Unconfined q_u (kg/cm^2)	c_u (kg/cm^2)
UDS 1	-	-	-	-	3.731	1.179	-	-	0.792	0.396
UDS 2	-	-	-	-	7.988	0.9	-	-	0.667	0.333
UDS 3	-	-	-	-	9.227	0.493	-	-	0.625	0.312
UDS 4	24.980	0.420	28.212	0.340	-	-	-	-	1.037	0.519
UDS 5	19.859	0.630	22.380	0.600	-	-	-	-	0.287	0.143
DS 1	-	-	-	-	-	-	27.580	0.022	-	-
DS 2	-	-	-	-	-	-	-	-	0.185	0.093
DS 3	-	-	-	-	-	-	35.390	0.078	-	-
DS 4	-	-	-	-	-	-	-	-	0.170	0.085
DS 5	-	-	-	-	-	-	36.520	0.055	0.256	0.128
DS 6	-	-	-	-	-	-	17.390	0.037	-	-
DS 7	-	-	-	-	-	-	-	-	0.340	0.170

3.3.   Tunnel Excavation

3.3.1.   Excavation Sequences. The method of Cisumdawu tunnel excavation is New Austrian Tunneling Method and a supports are used to hold the load and strengthen the rock to prevent collapse. Cisumdawu Tunnel is excavated on soft soil, so that proper excavation methods are needed to avoid collapse during the excavation process. New Austrian Tunneling Method is used with a 3 bench and 7 steps heading system. The excavation pattern scheme that can be applied to the Cisumdawu tunnel excavation can be seen in the Figure 6.

Figure 6. The patterns of tunnel excavation sequences (a) 1, (b) 2, (c) 3

3.3.2.   Supports. The temporary supports used in the construction of the Cisumdawu tunnel consists of several supports, including wire mesh, steel fiber reinforcement shortcrete, and steel rib. The following are the mechanical properties of each support used in the Cisumdawu tunnel.

4.   Discussion
To obtain a two-dimensional model, a perpendicular transverse incision is made to direction of the tunnel progress. Physical properties and mechanical properties of rocks, and support used are listed in previous chapter.

4.1.   The influence of double tunnels excavation on tunnels stability with horizontal distance variations
This section will discuss the influence of double tunnels excavation on tunnels stability that are horizontally separated with a certain distance. Firstly, left tunnel (L) excavation is carried out, after that right tunnel (R) is excavated. Stability parameter used in this discussion is strength factor, which is a safety indication around the tunnels. On first tunnel excavation that is left tunnel (L) excavation, the value of the strength factor shows the smallest value on tunnel wall and greater when away from tunnel wall. The greatest value of strength factor that founded in querry is 14.74 with position point is farthest from tunnel wall. But when right tunnel (R) is excavated with 1 D distance from left tunnel (L) wall, it will be creating a zone of influence from each tunnel excavation. The greatest value of strength factor between two tunnels is 7.65. When right tunnel (R) is excavated with 2 D distance from left tunnel (L) wall, the greatest value of strength factor between two tunnels is 14.20. When right tunnel (R) is excavated with 3 D distance from left tunnel (L) wall, the greatest value of strength factor between two tunnels is 19.11. It shows the effect of horizontal distance applied between two tunnels to the stability, so it needs attention to determine a horizontal distance for double tunnels excavation. If horizontal distance is applied farther, tunnel safety will be improving. Tunnel stability with horizontal distance variations can be seen on Figure 7.
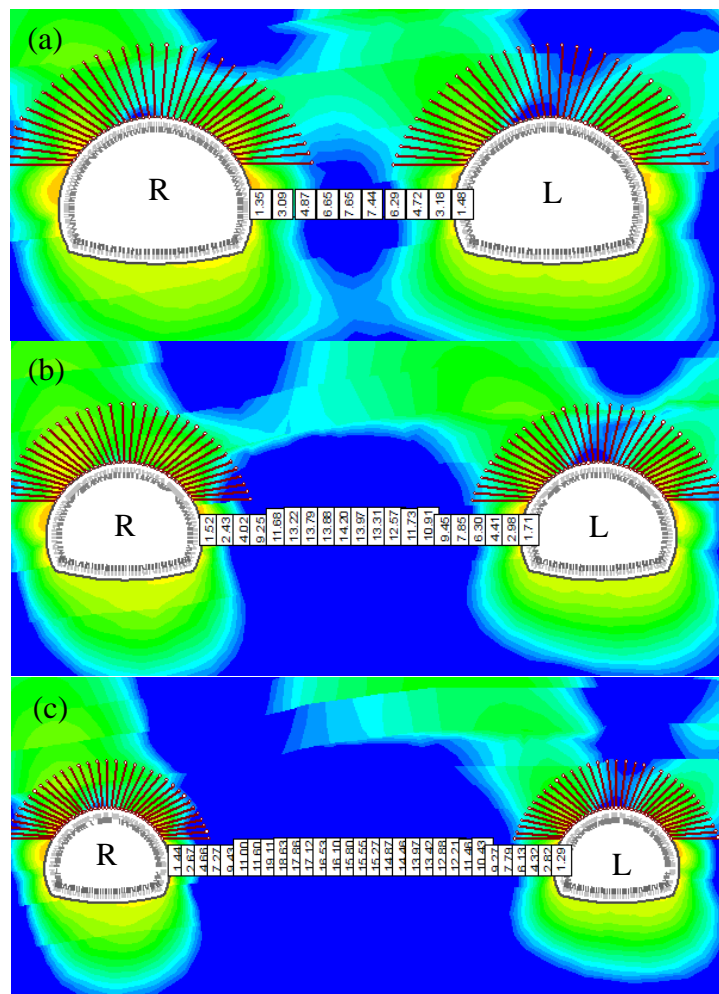
Figure 7. Tunnel stability with (a) 1 D, (b) 2 D, (c) 3 D horizontal distance

4.2. Relationship of Tunnel Stability to Excavation Sequences

In this analysis will be discussed about the influence of tunnel excavation patterns on soft soil using the New Austrian Tunneling Method with 3 excavation patterns. The excavation patterns are applied to same material. In this section, we will observe the influence of the excavation stages applied to the strength factor at each stage. The pattern of excavation sequence that can be applied to tunnel excavation can be seen in the Figure 8, Figure 9 and Figure 10.
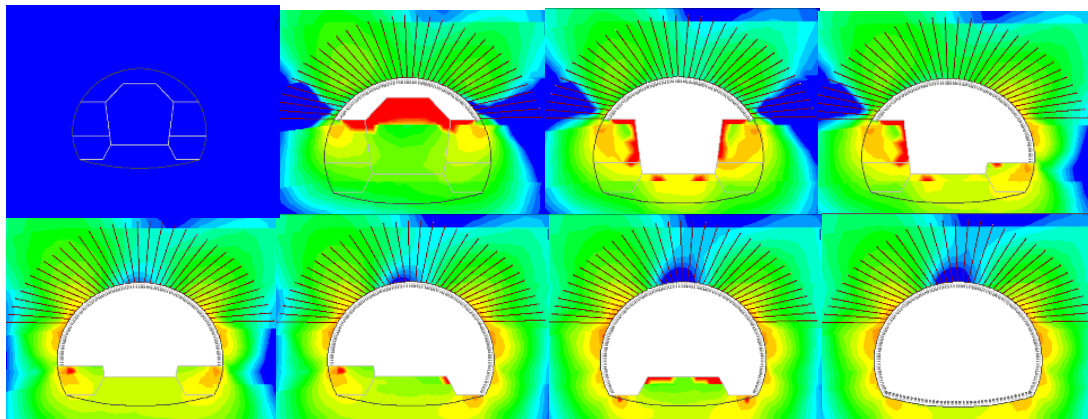

Figure 8. Excavation Pattern 1
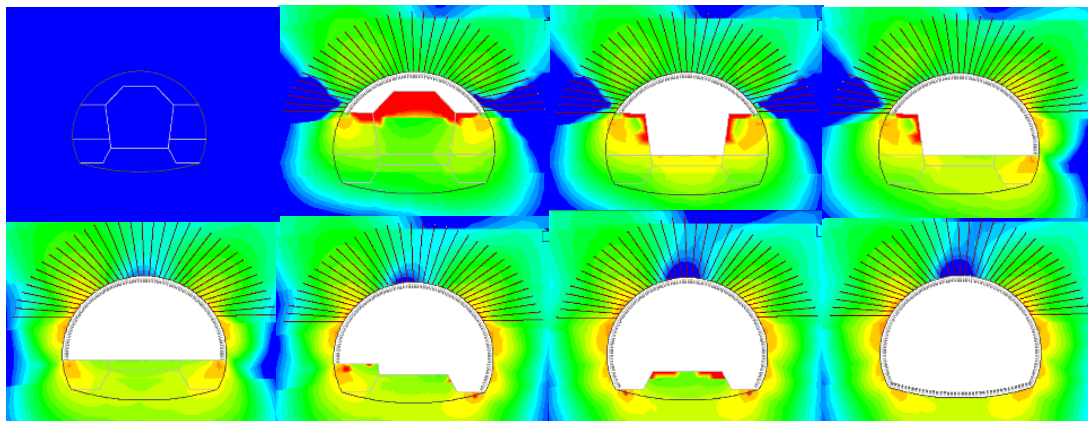
Figure 9. Excavation Pattern 2

Figure 10. Excavation Pattern 3

Based on the results of the strength factor values calculation in each excavation pattern, we can see that the excavation pattern shows the value of the strength factor with various conditions. The excavation pattern 3 shows a safer condition than the excavation pattern 1 and 2, because disturbed conditions that occur are less than the others.

4.3. Comparison of Tunnel Stability Based on Variations in Tunnels Distance

The total displacement parameter shows a difference of total displacement value that occurs based on double tunnel distance. The smallest value of total displacement that occurs is on condition with longest distance applied between two tunnel, which is 3 D. Otherwise, on the shortest distance applied between two tunnel, which is 1 D has the largest total displacement value. It can be concluded that the closer distance between two tunnel excavations more affecting the stability of the tunnels. This statement occurs on the left tunnel and right tunnel analysis. Two tunnels distance applied on the Cisumdawu tunnel excavation is 1D has the largest total displacement value than 2D and 3D tunnels distance. Total displacement is one of parameters to determine level of safety on the excavation activities, so periodic monitoring are needed on the tunnels construction. The comparison of total displacements with tunnel distance variations can be seen on Figure 11.
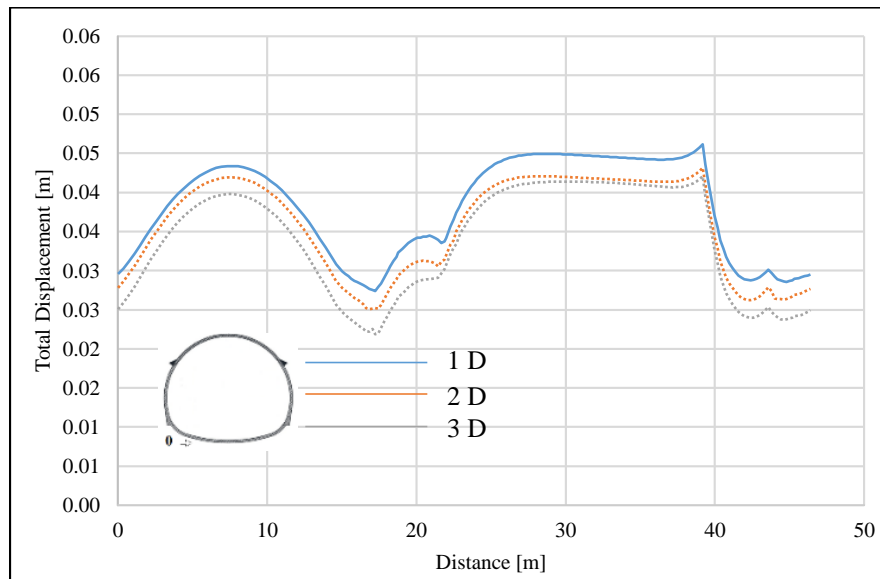
Figure 11. Comparison of total displacements with tunnel distance variations

4.4. *Relationship of Tunnel Horizontal Distance to Total Displacements Value*

Based on the tunnel stability analysis of two tunnels distance variations, it can be seen that the horizontal distance variations affect to the value of total displacement that occurs. The first excavation was carried out on the left tunnel (L) and continued with right tunnel (R) excavation. Tunnel observed in this discussion is the left tunnel. This discussion will analyze the influence of right tunnel excavation to the left tunnel which has been constructed with horizontal distance variations of double tunnels. In this case we will discuss the relationship of the horizontal distance of double tunnels to the value of total displacement that occurs. The horizontal distance applied are 1 D, 2 D, and 3 D (D = tunnel diameter). There are 4 observation points of total displacement, that are on the roof, right wall, left wall, and invert of the tunnel. So the relationship can be seen in Figure 12.
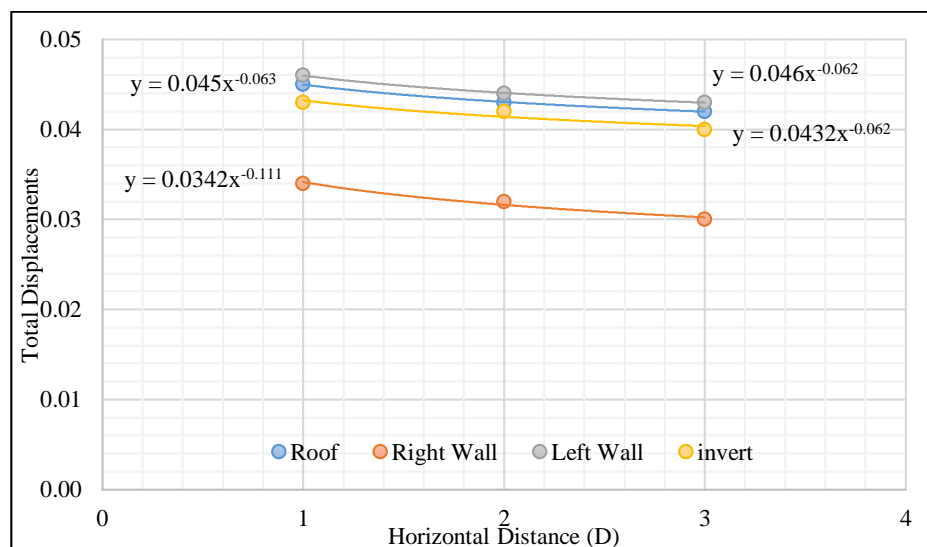

Figure 12. Relationship of Tunnel Horizontal Distance to Total Displacements Value

At each observation point, the total displacement value decreases when the horizontal distance is farther away. With the value of displacement is 1-3 mm / D horizontal distance. Based on the relationship obtained in Figure 17, there are several relationships that occur at 4 observation points. The relationship

that occurs on the roof tunnel displacement to the horizontal distance of double tunnel is $y = 0,045x^{-0,063}$. The relationship that occurs on the right wall tunnel displacement to the horizontal distance of double tunnel is $y = 0,0342x^{-0,111}$. The relationship that occurs on the left wall tunnel displacement to the horizontal distance of double tunnel is $y = 0,046x^{-0,062}$. The relationship that occurs on the invert tunnel displacement to the horizontal distance of double tunnel is $y = 0,0432x^{-0,062}$.

4.5. Monitoring

Tunnels monitoring system is carried out periodically. Monitoring includes monitoring the outside and inside conditions of the tunnel, there are subsidence monitoring and displacement around the tunnel wall monitoring. Monitoring success determined based on the accuracy of the equipment, equipment specifications, and analytical methods. Monitoring is carried out on the roof of tunnel, left wall, and right wall which is carried out periodically. Based on the results of monitoring for 46 days, the value of total displacement on the roof tunnel is 0.050 m. Whereas the total displacement value based on the results of numerical calculations on the roof tunnel is 0.051 m. Difference of value is due to the numerical calculation used the assumption of rock mass characteristics there are homogeneous, isotropic, and continuous. Whereas the results of monitoring used the actual condition of rock mass characteristics there are heterogeneous, anisotropic, and discontinuous. Monitoring results are used as a reference on tunnel construction process because monitoring process used the actual condition of rock mass characteristics. The monitoring results of total displacement can be seen on Figure 13.
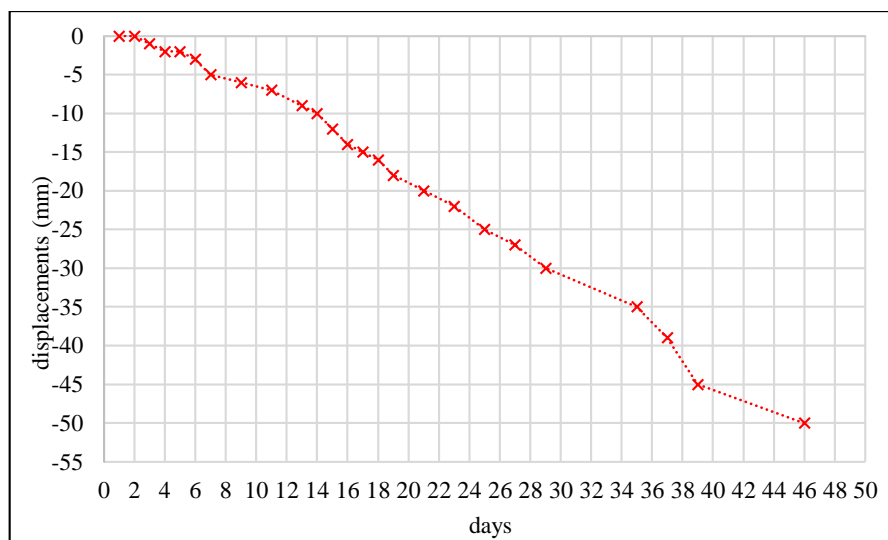

Figure 13. Monitoring results

Conclusion

Based on the description discussed, it can be concluded:

a. Soil mass characteristics on Cisumdawu tunnel is soft soil that that has a poor bearing capacity. So it is required to use a support to strengthen the tunnel and prevent tunnel collapse. The support used consists of steel rib, wire mesh, steel fiber reinforced shotcrete, and forepoling as a temporary support.

b. Based on the results of strength factor analysis of the excavation stages that applied. The excavation pattern that shows the safest condition is excavation pattern 3.

c. Determination of horizontal distance between double tunnels affects the displacement value that occurs which affects the stability of the tunnels. Displacement value decreases when horizontal distance applied is farther away with the difference of displacement value is 1-3 mm / D horizontal distance.

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