Geotechnical Design of Diversion Tunnel Using 2 Dimension Models

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Abstract. Diversion Tunnel plays the important roles to divert flowing water around a construction site. It is required design and analysis of stability and deformation that meets the criteria so that the tunnel can be constructed and operate safely. Stability and deformation analysis was carried out based rock mass strength using Mohr Coulomb and Hoek Brown criteria model. The analysis was conducted using analytical calculations and finite element method software in 2 dimensions (2D) to obtain the required geometry and supports. The results of 2D analysis were compared to analyze the effect geometry modeling. Analysis was also based on several cross sections in the tunnel to investigate design variations for each geotechnical condition according to site-specific environment. The study shows that the design method analytically and numerically fulfills the safety criteria, and the installation of support system indicate the significant reduction of tunnel displacement.

Keywords: Diversion tunnel, Rock mass strength, Deformation, Stability, Numerical modelling

1 Introduction

The Diversion Tunnel is an underground passageway used to divert flowing water around a construction site. In the implementation of dam construction, it is necessary to have an escape channel, to accommodate the flow of the river which is diverted from the original channel, as well as diversion dam to protect the works carried out on the foundation and the work of filling the dam body against disturbances to the relevant river flow. Diversion tunnel other functions as a flood channel and is often used as permanent dam complementary structures such as tapping structures, mud flushing facilities, flood spill buildings and others [1].

Before carrying out tunnel construction, it is necessary to plan beforehand. Planning is carried out to avoid construction failures in the tunnel. There are various things that can trigger the failure of the tunnel implementation either due to internal factors or external factors. Geotechnical planning carried out on the tunnel to ensure the safety. The safety criteria in tunnel geotechnical planning have several assessment parameters where adjusted and depend on the type of standard used. Several other factors need to be considered such as determining the function

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or use of the tunnel to be built, conducting a site review of the implementation and then geotechnical investigations including testing of materials to determine the materials or the type of native soil around the tunnel so that the method can be identified. The consideration of the tunnel design including load that will be imposed on the tunnel such as water pressure, soil pressure, dead load, soil reaction, internal load, earthquake load.

In this study, The New Austrian Tunneling Method (NATM) will be used. This method is based on the philosophy of “Built as you work” which is supported by the statement “not too rigid but not too flexible, not too fast but not too slow” [2] . The New Austrian Tunneling Method (NATM) according to [2] is a method of producing underground space using all available means to strengthen the capacity of the rock or soil itself so as to provide stability to the tunnel. The New Austrian Tunneling Method (NATM) can be used for soft soil excavated by machine or manually, where jointing and overbreak are not dominant.

Here are the basic principles of NATM:
1. Mobilization of rock mass strength,
2. Shotcrete protection to maintain rock mass load bearing capacity,
3. Monitoring the deformation of the excavated rock mass,
4. Provide flexible yet active support and
5. Invert closure to form a load-bearing support ring to control the deformation of the rock mass

1.1 Rock Mass Classification

According to [3] Geomechanics classification system or Rock Mass Rating (RMR) was developed by Bieniawski in 1973. This technical classification assesses rock quality on a scale from 0 to 100. RMR (Rock Mass Rating) classification is a geomechanical classification with the empirical method in determining the weighting of the rock mass. The main reason for using RMR is its ease and flexibility for various practical purposes in engineering [4] Its main use is to predict the required ground support and is based on six universal parameters, namely: a) Uniaxial compressive strength of intact rock materia, b) Rock quality designation (RQD), c) Spacing of discontinuities, d) Condition of discontinuities, e) Ground water condition, f) Orientation of discontinuities.

1.2 Rock Failure Criteria

a. Hoek brown

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{1}$$

notes:
$\sigma_1$and σ_3 are major and minor principal stress, mb, s, a are material parameters.

b. Mohr coulomb

$$\sigma_1 = \frac{2\,c' \cos\emptyset}{1-\sin\emptyset} + \frac{1+\sin\emptyset}{1-\sin\emptyset} \times \sigma_3 \tag{2}$$

notes:

σ_1 = Major principal stress
σ_3 = Minor principal stress
ϕ= frcition angle (°)

1.3 Tunnel In Situ Stresses

The stress in the tunnel is assessed using Kirsch's solution:

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

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

$$\tau_{r\theta} = \frac{P}{2}(1 + \lambda)\left(1 + \frac{2a^2}{r^2} - \frac{3a^4}{r^4}\right)\sin 2\theta \tag{5}$$

notes:
σ_r = radial stress
σ_θ = tangential stress
$\tau_{r\theta}$ = shear stress
P = vertical stress
λP = horizontal stress
R = Distance between tunnel centre lune to observation point
a = inner radius

1.4 Support System

The rock supports used as a reinforcement for rock stability in tunneling. According to there are several types of support systems used for rocks:

a. Shotcrete
Shotcrete capacity can be obtained by the equation below:

$$Psc = \frac{2 \,.\, qsc \,.\, tsc}{Fsc \,.\, B} \tag{6}$$

Notes:
qsc =Uniaxial compression strength of shortcrete (300 t/m^2 atau 0.20 x UCS shortcrete)
tsc = shortcrete thickness (m)
B = tunnel width (m)
Fsc = 0.6 ± 0.05
Psc = shortcrete capacity (t/m^2)
According to [5] the shotcrete support can be calculated by the equation below:

$$K = \frac{Ec\,[r_i^2 - (r_i - t_c)^2]}{(1 + Vc)\,[(1 - 2Vc)\,r_i^2 + (r_i - t_c)^2]} \quad (7)$$

Notes:
K = shotcrete stifness
Ec = Modulus deformation of concrete
tc = shotcrete thickness
r_i= tunnel inner radius

b. Rockbolt

Rockbolt is a rod material made of steel, with a round cross section used to support rock masses. The strength of the rockbolt is measured by carrying out a pull test in the field as shown in

$$P_{sbmax} = \frac{T_{bf}}{S_{l\,.\,s_c}} \quad (8)$$

$$K_{sb} = \frac{E_{sb}\,.\,\pi\,.\,d_b^{\,2}}{4l\,.\,S_{l\,.\,s_c}} \quad (9)$$

Notes:
T_{bf}= ultimate pull-out capacity
Psbmax = maximum capacity of rockbolt
Sl = longitudinal bolt distance
Sc= circular bolt distance
Ksb = maximum stiffness of rockbolt
d_b = diameter of rockbolt
l = bolt length

c. Steel Rib

Steel rib is one of tunnel support which made of steel

- Steel rib capacity

$$P_{sbmax} = \frac{A_s\,.\,O_{ys}}{S_{l\,.\,r_o}} \quad (10)$$

$$K_{sb} = \frac{E_s\,.\,A_s}{S_l\;r_o^{\,2}} \quad (11)$$

Notes:
P_{sbmax} = maximum steel capacity
K_{sb} = maximum stifness of steel
r_o = tunnel inner radius (m)
O_{ys} = Uniaxial compressive strength of the steel (Mpa)
A_s = rib cross section (m^2)
S_l = distance between each steel rib (m)
E_s = Steel rib modulus of deformation

- Steel rib span

$$S_{rib} = \frac{P_{Rib}}{P_{roof} \cdot B} \quad (12)$$

Notes:
P_{Rib} = steel rib capacity (ton)
S_{rib} = distance between each steel rib (meter)
B = tunnel width (meter)
P_{roof} = maximum support pressure on the roof (ton/m^2)

2. Method

Field and laboratory tests were carried out in the early stages of planning with the aim of obtaining several design parameters in the form of rock profiles, geological data, and rock parameters based on five drillhole (DH). Based on the results of field and laboratory testing, several data and parameters were found for tunnel design. The following are data from field and laboratory tests shown in Table 1.

Table 1 Boring Test Result

Location	Drill Point	Unit Weight (KN/m³)	UCS (MPa)	Natural Density (KN/m²)	Natural Water Content (%)	Saturated Density (KN/m²)	Dry density (kN/m²)	Void Ratio	Poisson ratio
0+153	DH-1	20.11	12.07	21.66	8.00	22.48	20.03	0.33	0.36
0+251	DH-2	16.44	6.28	16.46	15.90	18.73	14.20	0.86	0.33
0+369	DH-4	16.39	9.00	16.40	10.59	18.83	14.83	0.68	0.25
0+419	DH-5	17.54	6.67	17.08	9.98	19.61	15.98	0.59	0.34
0+560	DH-5'	17.54	6.67	17.08	9.98	19.61	15.98	0.59	0.34

The rock mass classification system RMR (Rock Mass Rating) was chosen to determine the quality of rock mass according to the type of rock testing in the field. At the tunnel inlet to tunnel outlet, rock types were found in the form of Tufaan Breccia and Tufaan Sandstone, starting from STA 0+0 to STA 0+700 meters as shown in Table 2.

Table 2 Rock Mass Rating

LOCATION	RMR	LOCATION	RMR
STA		STA	
0+0 - 0+20	52	0+345 - 0+370	58
0+20 - 0+45	52	0+370 - 0+395	63
0+45 - 0+70	63	0+395 - 0+420	61
0+70 - 0+95	60	0+420 - 0+445	51
0+95 - 0+120	57	0+445 - 0+470	51
0+120 - 0+145	54	0+470 - 0+495	51

0+145 - 0+170	61	0+495 - 0+520	51
0+170 - 0+195	51	0+520 - 0+545	51
0+195 - 0+220	54	0+545 - 0+570	51
0+220 - 0+245	54	0+570 - 0+600	52
0+245 - 0+270	54	0+600 - 0+625	52
0+270 - 0+295	63	0+625 - 0+650	52
0+295 - 0+320	74	0+650 - 0+680	52
0+320 - 0+345	60	0+680 - 0+700	52

Topographic data will be processed and then the proportional location of the diversion tunnel can be determined. For geological and soil mechanics data is processed and then it can be determined the type of rock that is in the location of the diversion tunnel plan. The calculation of the design flood discharge is processed and then the shape and dimensions of the tunnel are determined, and the earthquake seismic coefficient is obtained from earthquake hazard map. During the design process, calculations performed manually with the selected empirical method will produce various types of values. Stages of analysis are carried out based on the results of calculations and designs. The contents of the analysis phase include analysis of tunnel stability and deformation. The analysis was carried out in two types of conditions, namely with support and without support using Phase 2D and empirical methods. Soil parameters obtained from soil and rock test data and then processed using empirical methods before being processed into data for modeling in software. The expected output of this software is values or safety numbers, deformations and internal forces.

3. Result and discussion

3.1 Tunnel Cross section

The diversion tunnel is designed circular cross section specification. The dimensions of the tunnel are calculated based on the flood discharge during the 25 years return period. From the calculation results, an alternative dimension analysis of the tunnel was carried out. From the calculation results, the tunnel diameters were selected, 8.85 m and 8.65 m, so that the final diameter of the tunnel used was 9 meters. Tunnel cross section is shown in **Figure 1.**

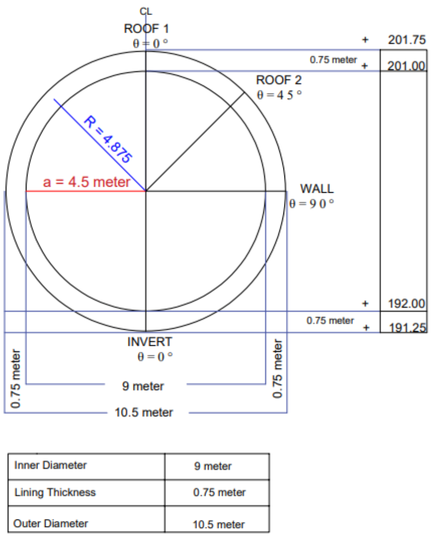

Fig. 1 Tunnel Crossection

3.2 Rock Mass Strength

The rock mass strength using the Hoek-Brown and Mohr Coulomb rock failure methods obtained, the results of σ_1 and σ_3 from each stationing as shown in Table 3.

Table 3 Rock Mass Strength Parameters

LOCATION	H (m)	k	σci (Mpa)	HOEK BROWN		MOHR COLOUMB	
STA				σ3 (Mpa)	σ1 (Mpa)	σ3 (Mpa)	σ1 (Mpa)
0+0 - 0+20	0.81	0.10	12.07	0.13	0.62	0.05	0.35
0+20 - 0+45	16.67	0.26	12.07	0.13	0.62	0.05	0.35
0+45 - 0+70	32.32	0.21	12.07	0.19	1.18	0.11	0.53
0+70 - 0+95	47.81	0.26	12.07	0.35	1.65	0.27	0.96
0+95 - 0+120	72.00	0.32	12.07	0.53	2.05	0.45	1.44
0+120 - 0+145	83.90	0.36	12.07	0.62	2.11	0.54	1.68

LOCATION STA	H (m)	k	σci (Mpa)	HOEK BROWN σ3 (Mpa)	HOEK BROWN σ1 (Mpa)	MOHR COLOUMB σ3 (Mpa)	MOHR COLOUMB σ1 (Mpa)
0+145 - 0+170	67.70	0.28	12.07	0.50	2.21	0.42	1.36
0+170 - 0+195	71.75	0.38	12.07	0.43	1.47	0.35	1.18
0+195 - 0+220	78.74	0.34	12.07	0.47	1.72	0.39	1.29
0+220 - 0+245	89.93	0.35	12.07	0.54	1.91	0.46	1.48
0+245 - 0+270	90.06	0.41	9.00	0.54	1.91	0.46	1.48
0+270 - 0+295	85.58	0.31	9.00	0.63	2.80	0.55	1.72
0+295 - 0+320	79.64	0.21	9.00	0.48	3.30	0.40	1.31
0+320 - 0+345	74.25	0.33	9.00	0.45	1.98	0.37	1.22
0+345 - 0+370	73.46	0.35	9.00	0.44	1.85	0.36	1.21
0+370 - 0+395	71.34	0.27	9.00	0.43	2.11	0.35	1.17
0+395 - 0+420	69.50	0.29	9.00	0.42	1.95	0.34	1.14
0+420 - 0+445	69.19	0.41	9.00	0.41	1.43	0.34	1.14
0+445 - 0+470	67.79	0.41	9.00	0.41	1.41	0.33	1.11
0+470 - 0+495	63.01	0.41	9.00	0.38	1.33	0.30	1.03
0+495 - 0+520	59.62	0.40	9.00	0.36	1.28	0.28	0.98
0+520 - 0+545	58.49	0.40	9.00	0.35	1.26	0.27	0.96
0+545 - 0+570	57.92	0.40	9.00	0.35	1.25	0.27	0.95
0+570 - 0+600	51.33	0.38	9.00	0.31	1.17	0.23	0.84
0+600 - 0+625	35.96	0.35	9.00	0.21	0.90	0.14	0.59
0+625 - 0+650	21.27	0.31	9.00	0.13	0.62	0.05	0.35
0+650 - 0+680	15.17	0.29	9.00	0.13	0.62	0.05	0.35
0+680 - 0+700	9.96	0.26	9.00	0.13	0.62	0.05	0.35

3.3 Loading Conditions

The loading conditions are divided into several combinations in the design. The loading is designed with water flowing conditions during normal times and water flowing conditions during earthquakes. Loading condition was shown in Table 4.

Table 4 Loading Conditions

Drill Hole	DH-1	DH-2	DH-3	DH-4	DH-5
NORMAL CONDITIONS	0+153	0+251	0+369	0+419	0+560
1. Tunnel Self Weight	0.018	0.018	0.018	0.018	0.018
2. Vertical Earth Pressure	0.431	1.486	1.212	1.147	0.956

3. Horizontal Earth Pressure	0.236	0.610	0.509	0.485	0.415
4. Vertical Hydrostatic Pressure	0.019	0.047	0.039	0.050	0.013
5. Uplift	0.011	0.011	0.011	0.011	0.011
6. Internal Water Pressure	0.088	0.088	0.088	0.088	0.088
7. Horizontal Hydrostatic Pressure	0.029	0.058	0.049	0.060	0.023
TOTAL (Mpa)	0.803	2.260	1.877	1.798	1.500
MAX			2.260		
	DH-1	DH-2	DH-3	DH-4	DH-5
Earthquake Conditions	0+153	0+251	0+369	0+419	0+560
1. Tunnel Self Weight	0.018	0.018	0.018	0.018	0.018
2. Vertical Earth Pressure	0.431	1.486	1.212	1.147	0.956
3. Horizontal Earth Pressure	0.282	0.727	0.607	0.579	0.495
4. Vertical Hydrostatic Pressure	0.019	0.047	0.039	0.050	0.013
5. Uplift	0.011	0.011	0.011	0.011	0.011
6. Internal Water Pressure	0.088	0.088	0.088	0.088	0.088
7. Horizontal Hydrostatic Pressure	0.029	0.058	0.049	0.060	0.023
TOTAL (Mpa)	0.877	2.434	2.024	1.952	1.603
MAX			2.434		

3.4 Determination of Stresses in the Tunnel Segment

The stress in the tunnel is calculated at four points in the tunnel, the roof – 1, roof – 2, walls and floor as shown in **Figure 1**. From the calculation of the stress on each stationing based on four points in the tunnel (roof -1, roof -2, walls and floors), the minimum stress is 0.06 Mpa located at roof-1 STA (0+0 - 0+45) and STA (0+625 - 0+700) and the maximum stress located at STA (0+120 - 0+145) which is the floor section of 4.314 MPa. The results of stress calculations are shown in Table 5.

Table 5 In Situ Stress Result

STA	ROOF - 1		WALL		ROOF - 2			INVERT		MAX
	σr (MPa)	σθ (MPa)	σr (MPa)	σθ (MPa)	σr (MPa)	σθ (MPa)	τrθ (MPa)	σr (MPa)	σθ (MPa)	(MPA)
0+0 - 0+20	0.06	0.80	0.01	0.12	0.04	0.48	0.07	0.09	1.16	1.16
0+20 - 0+45	0.06	0.80	0.01	0.12	0.04	0.48	0.07	0.09	1.16	1.16
0+45 - 0+70	0.10	1.20	0.02	0.17	0.06	0.70	0.10	0.13	1.56	1.56
0+70 - 0+95	0.18	2.16	0.03	0.29	0.11	1.25	0.17	0.21	2.60	2.60
0+95 - 0+120	0.26	3.26	0.05	0.42	0.16	1.86	0.26	0.30	3.70	3.70

STA	ROOF - 1		WALL		ROOF - 2			INVERT		MAX (MPA)
	σr (MPa)	σθ (MPa)	σr (MPa)	σθ (MPa)	σr (MPa)	σθ (MPa)	Ʈrθ (MPa)	σr (MPa)	σθ (MPa)	
0+120 - 0+145	0.31	3.80	0.06	0.48	0.18	2.17	0.30	0.34	4.24	4.24
0+145 - 0+170	0.25	3.06	0.05	0.40	0.15	1.75	0.24	0.28	3.50	3.50
0+170 - 0+195	0.26	3.25	0.05	0.42	0.16	1.86	0.26	0.30	3.69	3.69
0+195 - 0+220	0.24	0.00	0.00	0.37	0.00	0.00	0.23	0.27	0.00	0.37
0+220 - 0+245	0.27	3.34	0.05	0.42	0.16	1.90	0.26	0.30	3.70	3.70
0+245 - 0+270	0.27	3.35	0.05	0.42	0.16	1.91	0.26	0.30	3.71	3.71
0+270 - 0+295	0.31	3.87	0.06	0.49	0.19	2.21	0.31	0.35	4.31	4.31
0+295 - 0+320	0.21	2.57	0.04	0.33	0.13	1.47	0.20	0.24	2.93	2.93
0+320 - 0+345	0.22	2.76	0.04	0.35	0.13	1.58	0.22	0.25	3.12	3.12
0+345 - 0+370	0.22	2.73	0.04	0.35	0.13	1.56	0.22	0.25	3.09	3.09
0+370 - 0+395	0.33	1.18	1.25	0.35	0.76	0.77	0.44	0.37	1.34	1.34
0+395 - 0+420	0.21	2.58	0.04	0.33	0.13	1.48	0.20	0.24	2.94	2.94
0+420 - 0+445	0.21	2.57	0.04	0.33	0.13	1.47	0.20	0.24	2.93	2.93
0+445 - 0+470	0.20	2.52	0.04	0.33	0.12	1.44	0.20	0.23	2.88	2.88
0+470 - 0+495	0.19	2.34	0.04	0.30	0.11	1.34	0.19	0.22	2.70	2.70
0+495 - 0+520	0.18	2.21	0.03	0.29	0.11	1.27	0.18	0.21	2.58	2.58
0+520 - 0+545	0.18	2.17	0.03	0.28	0.11	1.25	0.17	0.21	2.53	2.53
0+545 - 0+570	0.17	2.15	0.03	0.28	0.11	1.24	0.17	0.20	2.51	2.51
0+570 - 0+600	0.15	1.91	0.03	0.25	0.09	1.10	0.15	0.18	2.27	2.27
0+600 - 0+625	0.11	1.34	0.02	0.18	0.07	0.78	0.11	0.14	1.70	1.70
0+625 - 0+650	0.06	0.80	0.01	0.12	0.04	0.48	0.07	0.09	1.16	1.16
0+650 - 0+680	0.06	0.80	0.01	0.12	0.04	0.48	0.07	0.09	1.16	1.16
0+680 - 0+700	0.06	0.80	1.00	0.12	0.04	3.00	0.07	0.09	1.16	3.00

3.5 Support System Based on Rock Mass Classification

a. Based on RMR

Based on the rock mass rating value, the smallest and largest value are 51 and 74. If plotted into a stand-up time graph and unsupported span, the number plotted are 51,55,60 and 70 representing adjacent RMR values. The stand-up time and unsupported span for RMR is 51, which is 200 hours with a span of 5 m. For 55 obtained 400 hours with a span of 5 m. For 60 obtained 3000 hours with a span of 5 m. For 70 obtained 20,000 hours with a span of 5 m.
Based on the RMR value, there are several recommendations for the excavation method and the type of support. The following are some recommendations for excavation methods and types of supports that can be used according to the RMR value. Based on the rock mass rating, the value of rock mass rating at the sites were obtained from 51 to 74, which classified in class II and III, with the following recommendations:

RMR class II

a. Excavation: full face, 1-1.5 m advance. Complete support 20 m from face
b. Rock bolts (20 mm diameter, fully grouted): Locally bolts in crown 3 m, spacing 2.5 m with occasional wiremesh
c. Shotcrete: 50 mm in crown where required

RMR class III

a. Excavation: Top heading and bench 1.5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.
b. Rock bolts (20 mm diameter, fully grouted): systematic bolts 4 m long spaced 1.5 - 2 m in crown and walls with wiremesh in crown.
c. Shotcrete: 100 - 150 mm in crown and 100 mm inside.

3.6 Support System Analysis

The support system analysis was determined based on the total load on the tunnel. The section of the tunnel is divided into five parts: the roof, the left upper wall, and the right upper wall, left side wall, right side wall and invert or floor.

a. Rockbolt

Based on the analysis result, rockbolts with diameters between 30 - 40 mm and with anchorage lengths between 4 meters - 7 meters used with the calculation of the capacity of each rockbolt as follows:

Table 6 Rockbolt Capacity, D30

TYPE	Tbf (MN)	Sl (m)	Sc (m)	YOUNG MODULUS (Mpa)	db (m)	L (m)	STIFFNESS (Mpa)	CAPACITY (Mpa)
Type 1	0.297	3	2.6	200000	0.03	4	61.139	0.057
Type 2	0.297	3	2.6	200000	0.03	5	48.910	0.057
Type 2	0.297	3	2.6	200000	0.03	6	40.759	0.057
Type 4	0.297	3	2.6	200000	0.03	7	34.930	0.057

Table 7 Rockbolt Capacity, D40

TYPE	Tbf (MN)	Sl (m)	Sc (m)	YOUNG MODU-LUS (Mpa)	db (m)	L (m)	STIFF-NESS (Mpa)	CAPAC-ITY (Mpa)
Type 1	0.527	3	2.6	200000	0.04	4	108.690	0.101
Type 2	0.527	3	2.6	200000	0.04	5	86.953	0.101
Type 2	0.527	3	2.6	200000	0.04	6	72.461	0.101
Type 4	0.527	3	2.6	200000	0.04	7	62.109	0.101

Table 8 Rockbolt Capacity

Locations	Rockbolt			
	Length (meter)	Amount	Diameter (m)	Capacity (MPa)
DH - 1	5	11	0.03	0.63
DH - 2	5	11	0.03	0.63
				0.85
DH - 4	5	15	0.03	
				1.11
DH - 5	5	11	0.04	
DH - 5'	5	11	0.03	0.63

b. Steel Rib

Table 9 Steel Rib Capacity

Jenis Rib	A (m^2)	Oys (Mpa)	Spacing (m)	Capac-ity (Mpa)	Young Modulus (Mpa)	Stiffness (Mpa)
H (250 x 250 x 14 x 14)	0.010	420	1	12.97	200000	101.728

c. Shortcrete

Table 10 Shotcrete Capacity

Specification	UCS (Mpa)	v	Thickness (m)	Thickness Used (m)	P (Mpa)	Ecs (MPa)	K (Mpa)
K - 300	24.900	0.2	0.092	0.25	0.461	25170	1518.67
K- 275	22.83	0.2	0.10	0.25	0.42	23998	1298.44
K - 250	20.75	0.2	0.11	0.25	0.38	22819	1298.44

d. Concrete Lining

Table 11 Lining Concrete Capacity

Specification	UCS (Mpa)	v	Thickness (m)	Thickness Used (m)	P (Mpa)	Ecs (MPa)	K (Mpa)
K - 300	24.900	4.5	0.75	215200	0.35	3.804	913
K- 275	22.830	4.5	0.75	21520	0.35	3.487	913
K - 250	20.750	4.5	0.75	21520	0.35	3.170	913

e. Support Installation

Table 12 Support for Tunnel

Support Type	Details
Shotcrete	T = 0.25 m K- 300
	D = 0.03 – 0.04 m
	s = 2 – 2.6 m
Rockbolt	L = 5 m
	n = 11 - 15
Steel Rib	H Beam (250 x 250 x 14 x 14)
Lining Concrete	T = 0.75 K- 300

3.7 Numerical Method with 2-Dimensional - Phase 2D Modeling

For 2D modeling of earthquake conditions with no support with Hoek-Brown analysis on DH - 1, the largest displacement is located at the roof with total displacement 0.0195 meters, or 1.95

centimeters and the smallest displacement is on the tunnel floor of 0.009 meters or 0.9 centimeters. Regarding Hoek-Brown and Mohr-Coloumb minor and major stress analysis, the lowest and highest safety factor values are 1.28 and 3.60, on the roof (top) and tunnel walls. Earthquake conditions - support (after the support was installed), a combination of rockbolt, steel rib, shotcrete, and lining concrete, it was found that there was a change in the displacement of the roof with a total displacement 0.018 meters or 1.8 centimeters (declining) and the smallest displacement was on the tunnel floor (invert) 0.009 meters or 0.9 centimeters (no change). All the displacement results are fulfill the maximum allowable tunnel deformation based on [6] which is 10 centimeters. The example of Phase 2D modelling was shown in **Fig. 2.**

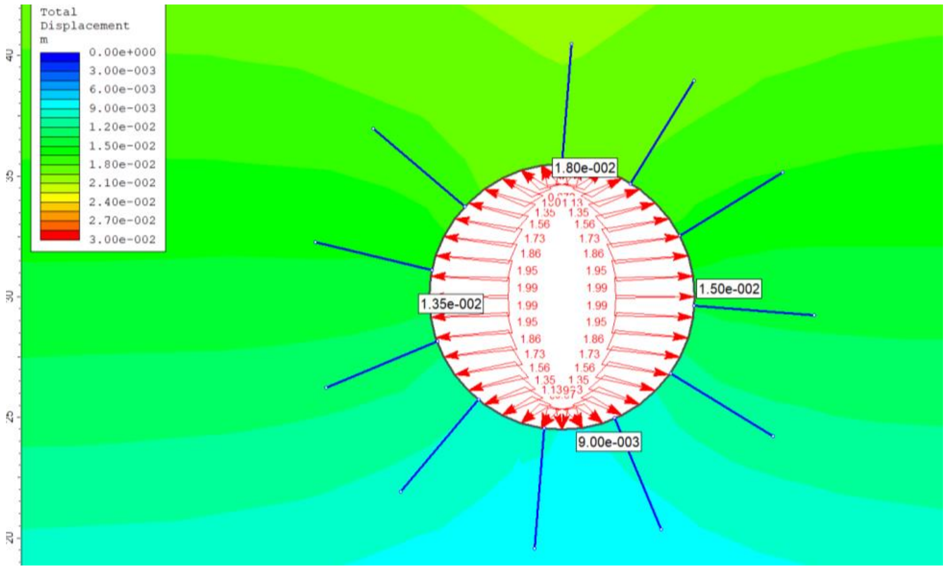

Fig. 2 Total Displacement Phase 2D

3.8 Design Comparison Result

Analysis of the tunnel design is carried out by two types of processing, including analytical methods and 2-dimensional finite element method. Based on the analysis results, the output parameters to be compared are safety factors and total displacement. Both methods are compared based on the conditions: 1) Hoek Brown's and Mohr Coulomb failure criteria, 2) The earthquake acceleration was applied.

a. Support and Excavation System

Table 13 Method Comparison

Parameter	ANALYTICS	PHASE 2D
	Excavation	
Excavation type	Top - Bench	Top - Bench
	Shotcrete	

Thickness	0.015 meter	0.025 meter
Rockbolt		
Length	5 meters	5 meters
Diameter	0.03 - 0.04 meter	0.03 meter
Longitudinal Distance	2.6 meter	2 meters
Circular Distance	2 meters	2 meters
Steel Rib		
Specification	H (250 x 250 x 14 x 14)	

b. Safety Factor

Based on the analysis, obtained safety factor based on analytical methods and 2D numerical methods. The safety factor is analyzed based on the results of tunnel excavation that has been given a complete support. In PHASE 2D the safety factor is obtained based on the analysis of the results of σ_1 and σ_3 from six points in the tunnel (roof, left wall 1-2, right wall 1-2 and floor) which is then processed with the Mohr-Coulomb safety factor equation.

Table 14 Safety Factor Comparison

SAFETY FACTOR								
LOCATION	PHASE 2D				ANALYTIC			
	MOHR COULOMB		HOEK BROWN		MOHR COULOMB		HOEK BROWN	
	No Support	Support	No Support	Support	No Support	Support	No Support	Support
DH - 1	1.61	2.74	1.28	2.67	0.11	1.18	0.20	1.25
DH - 2	1.06	2.50	1.01	1.69	0.19	1.12	0.24	1.17
DH - 4	2.64	2.73	1.04	3.29	0.22	1.13	0.29	1.22
DH - 5	1.15	1.13	0.96	3.81	0.23	1.42	0.36	1.56
DH - 5'	1.43	7.38	1.73	2.23	0.08	1.32	0.12	1.36

c. Displacement Result

Table 15 Displacement Comparison

DISPLACEMENT				
LOCATION	PHASE 2D			
	MOHR COULOMB		HOEK BROWN	
	No Support	Support	No Support	Support
DH – 1	0.010	0.009	0.020	0.018
DH – 2	1.211	1.236	1.011	1.689

DH – 4	0.048	0.023	0.015	0.015
DH – 5	0.070	0.070	0.050	0.050
DH - 5'	0.018	0.017	0.018	0.018

4. Conclusion

The diversion tunnel model and analysis has been done. Two-dimensional tunnel design in this study reach the safety criteria required for geotechnical design which > 1. The New Austrian Tunneling Method (NATM) used as the chosen method as follows:

1. The geometry of the tunnel is design to be circular with the following dimensions of tunnel diameter = 10.5 meters, inside diameter = 9 meters, tunnel length = 700 meter
2. The support system used for the tunnel consist of rockbolt, shotcrete, steel rib and lining concrete with types of classes for the tunnel desin are II and III with maximum excavation length 1.5 – 3 meters.
3. Based on tunnel analysis in the presence of earthquakes and supports, the results show that the support system has a major contribution in reducing the total displacement in the diversion tunnel in terms of changes in total displacement before and after the installation of supports in the tunnel based on 2D modelling.

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