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ANALYSIS OF TUNNEL STABILITY IN VERY WEAK ROCK

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ABSTRACT

In this research, the classification and characterization of rock masses did not use Rock Mass Rating (RMR), because some parameters of the RMR such asjoint spaces, joint conditions namely, apertures, persistence, joint fillings could not be determined during the study, so rock mass classified could be determined by compressive strength using the pocket penetrometer. Tunnel stability in very weak rock was analyzed using numerical modeling, the numerical modeling method used was the finite element method with RS 2019 software (Rocscience). Tunnel stability was assessed bytunnel displacement, strength factors, and probability of failure based on support capacity.

Keywords: *Tunnel, Very weak rock, numerical modelling, deformation, strength factor*.

I. INTRODUCTION

Very weak rocks are important geomaterials in tunnel construction because they present undesirable behaviors such as: low strength, disaggregation, high plasticity, rapid weathering, permeability and others. This material also has medium strength between soil and hard rock. In some cases, the rock is very weak too soft to be tested with rock mechanical equipment and too difficult with soil mechanical equipment. This shows that some adjustments in testing must be developed to be able to characterize weak rock properties well (Kanji, 2014). Tunnels in weak rocks present special challenges for geotechnical engineers, due to prediction errors in tunnel deformation assessments and support system designs can cause fatal failures. In addition, the increasing demand for building large sized tunnels in relatively poor rock mass conditions determines the need for new approaches in the early stages of design. These approaches must combine the influence of depth and size of underground excavation, with qualitative estimates of the classification of rock masses (Mihalis et al, 2001). Based on this, optimization of support systems in very soft rocks has always been an important problem in the engineering field.

II. METHOD & MATERIAL

2.1 Method of Classifying Rock Strength

Hoek & Brown, 1997 provides a field method for estimating uniaxial compressive strength (UCS) of rocks, the UCS value of rocks based on the characteristics in the field by Hoek & Brown, 1997 can be seen in Table.1, Rocks are considered weak if they have UCS <25 MPa, or have 15-90 blows based on the standard penetration test (SPT). For rock mass strength below 500 KPa, the estimated strength of soil compressive strength in table 2 can be used as a reference, where compressive strength is assessed using a pocket penetrometer.

2.2 Tunnel Analysis Method

Numerical modeling is used in analyzing tunnel stability. The numerical modeling method used was the finite element method with RS 2019 software (Rocscience). RS2 is a two-dimensional finite element program for calculating stresses and deformations around underground openings, and can be used to solve various problems because it can incorporate various conditions such as elastic or plastic material, constant field stress or gravity, jointed rock, groundwater, support system analysis and many other things. This software is widely used in the mining or civil project.

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Table 1. Field estimates of uniaxial compressive strength (Hoek&Brown, 1997)

Term	UCS(MPa)	Field Estimate of Strength
Extremely strong	>250	Specimen can only be chipped with a geological hammer
Very Strong	100-250	Specimen requires many blows of a geological hammer to fracture it
Strong	50-100	Specimen requires more than one blow of a geological hammer to fracture it
Medium Strong	25-50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer
Weak	2-25	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer
Very weak	1-5	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife
Extremely weak	0,25-1	By a pocket knife indented by thumbnail

Table 2. Classification of soil compressive strength(Federal Highway Administration ,1997)

Term	Number of blow	Pocket penetrometer (tsf)	Field Test
very soft	0-1	0.25 or less	Squeezes between fingers when fist is closed, penetrated several inchs by fist.
soft	2-4	0.25-0.50	Easily molded by fingers, easily penetrated several inches by thumb.
medium stiff	5-8	0.50-1.00	Molded by strong pressure of fingers, can be penetrated several inches by thumb with moderate effort.
stiff	9-15	1.00-2.00	Dented by strong pressure of fingers, readily indented by thumb but can be penetrated only with great effort.
vey stiff	16-30	2.00-4.00	Readily indented by thumb nail.
Hard	31-60	0ver 4	Indented with difficulty by thumb nail.
Very Hard	>61		

III. CASE STUDIES

In this paper, the Cisumdawu Tunnel located in Sumedang Regency, West Java is used as a case study of tunnels in very weak rocks, the Cisumdawu Tunnel can be seen in Figure 1. The Cisumdawu Tunnel is a tunnel designated as a highway transportation tunnel. The excavation method used is the New Austrian Tunneling Method (NATM) method. Cisumdawu Tunnel is a double tunnel with each tunnel length is 472 m, tunnel width is 14 m, tunnel height is 11 m, maximum height of burden is 52.8 m, and minimum height is 14 m. The types of rocks around the tunnel are silty clay, sandy sit, and sandy clay, which are classified as very weak rock materials with compressive strength values less than 1 MPa.

In general, the type of material around the tunnel can be seen in Figure 2. it can be seen that there are no geological structures such as jointl, faults, and folds in the Cisumdawu Tunnel. It's just that, this tunnel is a shallow tunnel, so it is in very weak rock. The Cisumdawu Tunnel is in a valley between two hills, which is naturally a water gathering area, and it is added that the tunnel is close to the ground water level.

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Figure 1. Cisumdawu Tunnel

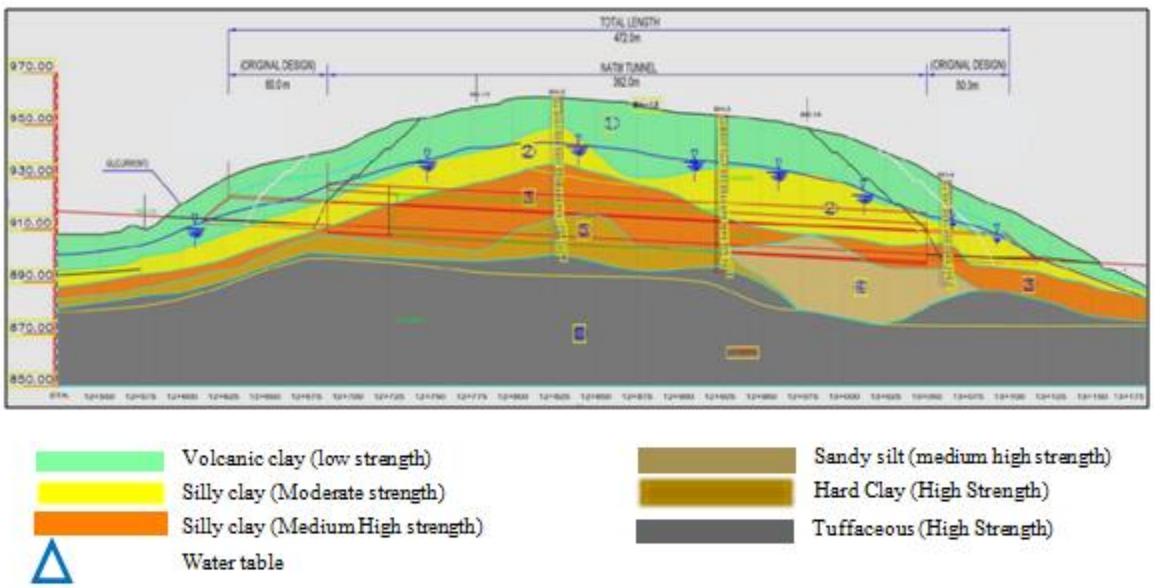

Figure 2. Geological Conditions Around the Cisumdawu Tunnel

The support system used for the Cisumdawu Tunnel can be seen in Figure 3. The condition of the material with very soft rock types, causes the use of pre-support before excavation activities are carried out. The type of pre-support used is steel pipe-grouting which is applied on top of the tunnel crown with an umbrella-like arrangement called the pipe umbrella. The temporary support system used immediately after excavation is steel rib, wiremesh and shotcrete 25 cm.

IV. RESULT & DISCUSSION

4.1 Results of Classification of Rock Strength

In this paper, rock strength classification uses the measurement results of compressive strength with a pocket penetrometer. The number of data taken is 180 data, with an area of 110 m2, with details of 10 m tunnel logitudinal and 11 m is the width of the tunnel. Table 2 is used to classify the strength of rock masses around the tunnel. Theresults of compressive strength with pocket meterometer can be seen in Figure 4. Figure 4 shows that 82% of 180 location points are categorized as "Hard" or hard soil type, 17% very stiff and 1% stiff medium.

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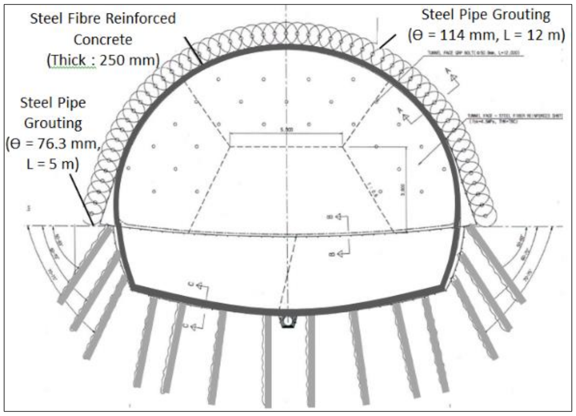

Figure 3. Temporary Support System of Cisumdawu Tunnel

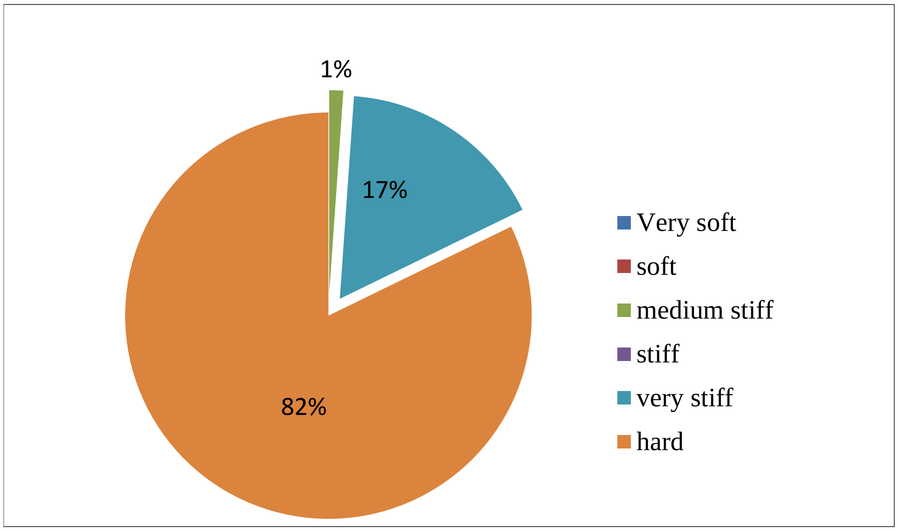

Figure 4. The circle diagram of compressive strength result by pocket penetrometer

4. 2 Numerical Modelling

The material layer around the tunnel is divided into 4 layers according to the results of the core drill test, the layer is classified to be low strength at a depth of 0.0-22.5 m with the type of Silty clay matertial. Medium strength category at depths of 22.5-42.5 m with Clayey silt material. Strength category at depths of 42.5-60 m with the type of Sandy clay material with tuffaceous mixture. While the material layer which is under the layer of strength category is considered as a bottom or bedrock, the tunnel position is at a depth of about 27-37 m. Figure 5 is a figure of numerical modeling for material layers around the tunnel.

The failure criteria used in numerical modeling are Mohr-Coulomb with Plastic material types (Peak = residue). Very weak rock property input used can be seen in Table 3, and the support input data in Table 4.

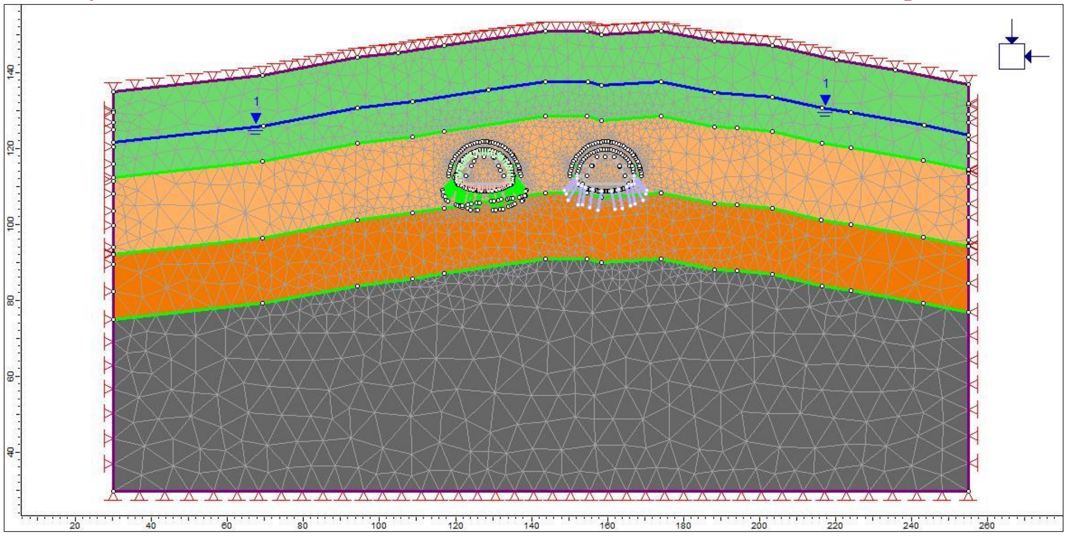

Figure 5. Cisumdawu Double Tunnel Numerical Modeling with RS2 2019

Table 3. Rock Properties Input

Propertis	Satuan	Lapisan			
		Silly Clay	Sandy silt	Sandy clay	Bedrock
γ	gr/cm^3	1,65	1,64	1,65	1,70
c	MPa	0,062	0,041	0,068	0,10
ϕ	^{o}	8,45	13	21,30	25
E	MPa	53,13	45,42	83,75	100

*γ = unit weight, *c=cohesion , *ϕ =friction angle, *E=modulus young

Table 4. Tunnel Support Properties Input

Propertis	Jenis Penyangga						
	Beam (W150X18)	wiremesh	Shotcrete	Mortar	Concrete	Rebar	Steel pipe
E (Mpa)	200000	135000	26315	4444,44	35000	200000	180000
C (Mpa)	400	250	25	37,97	49,07	420	345
v	0,25	0,25	0,20	0,28	0,20	0,30	0,29
T (Mpa)	400	50	2	2	3	560	345

*E = modulus young, *C= compressive strength, **v*=poisson ratio, *T=tensile strength

in determining the pre-support property, namely forepoling or pipe umbrella, property input is calculated using the Evert hook approach, which is a 2D approach by creating a special layer to interpret pre-support (Figure 6) using the formula:

$$\text{Improved layer forepoling} = (\text{weakrock x } 0.8) + (\text{steel pipe x } 0.01) + (\text{mortar x } 0.19)$$

The results of the calculation of the improved layer can be seen in Table 5.

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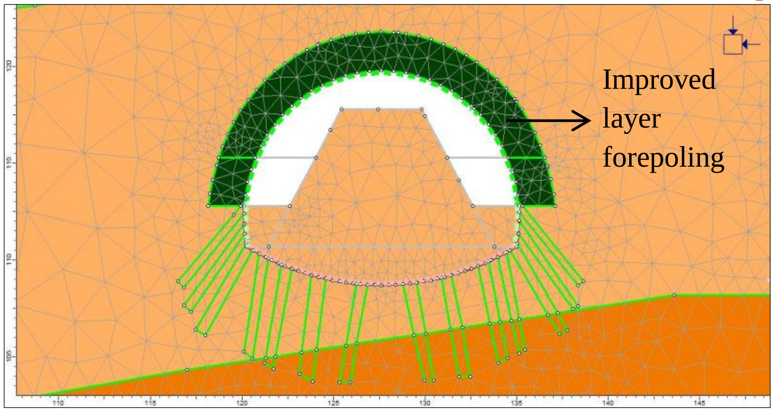

Figure6, Improved layerfor forepoling

Table 5. The input value for the improved layer for forepoling

Parameter	Symbol	Material (weakrock)	Steel Pipe	Mortar	Improved Layer
Unit weight	γ (MN/m^3)	0,0165	0,081	0,017	0,0172
Young's Modulus	E(MPa)	55,6	180000	4444,44	2688,9
Poisson ratio	*v*	0,30	0,25	0,28	0,29
Tensile Strength	T(MPa)	0	345	2	3,80
Friction Angle	ϕ(°)	17,56	High	35	38
Cohesion	c (MPa)	0.036	High	5	1,47

4.3 Numerical Modeling Results

4.3.1 Displacement

The value of displacement that occurs on the roof, walls and floor of the tunnel can be seen in Figure 7. The curve shown in Figure 7 shows the displacement values that occur on the entire surface of the tunnel. The distance of 0-15 m indicates the displacement that occurs on the floor, 15 m -20 m is the distance for the right wall, 20 m - 38 m for the roof and 38-43 m for the left wall. Based on the results, the biggest displacement occurs on the Floor with a max displacement of 79 mm, on the left wall max 30 mm, the right wall max 28 mm and the roof max 23 mm.

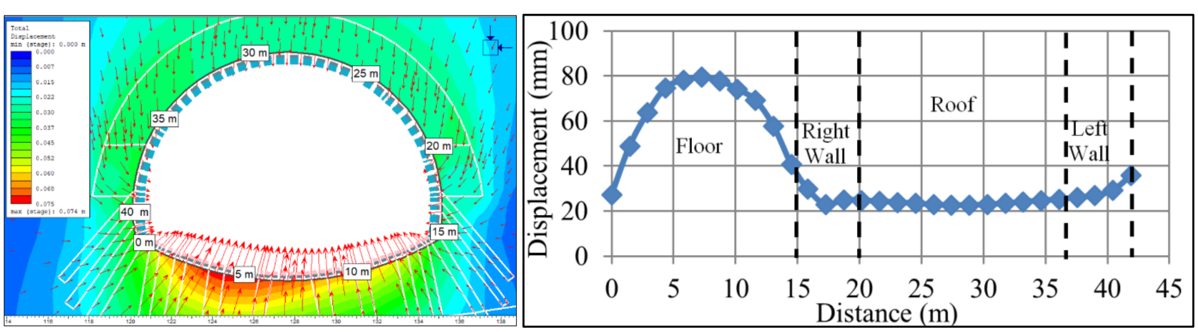

Figure 7. Tunnel Displacement

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An approach to assess tunnel stability must be simple enough to practice at a construction site. Determination of hazard level (d) by Sakurai, 1997 is used to assess tunnel stability based on deformation values. For this purpose, it is assumed that a tunnel is circular, homogeneous and hydrostatic.

$d = \varepsilon_{cr}\, r$, ε_{cr} is critical strain, and r is tunnel radius

Based on the results of the Unconfined Compression Test (UCT), obtained the average critical strain value of very weak rocks is 2.5%. based on the critical strain value, the hazard level of deformation is obtained;

0,025 x 5.5 m = 0,1375 m
= 13,75 cm

So that it can be concluded that the deformations that occur on the surface of the Cisumdawu Tunnel, both on the floor, roof and walls of the tunnel are still relatively safe until permanent support is installed.

4.3.2 Strength Factor

Strength factor (SF) is the ratio of rock strength (based on failure criteria) to induced stress. The SF value on the entire tunnel surface can be seen through the curve in Figure 8. The SF value on the roof tends to be stable with a value of 5 - 7. While on the floor and walls, the SF value has a large difference. The difference in SF values on the walls and floors of the tunnels is caused by the irregular installation of steel pipe grouting for all parts of the walls and floors, so that the stee pipe grouting is provided with a larger FK than the parts without steel pipe grouting. The smallest SF value occurs on the floor with a value of 1.3.

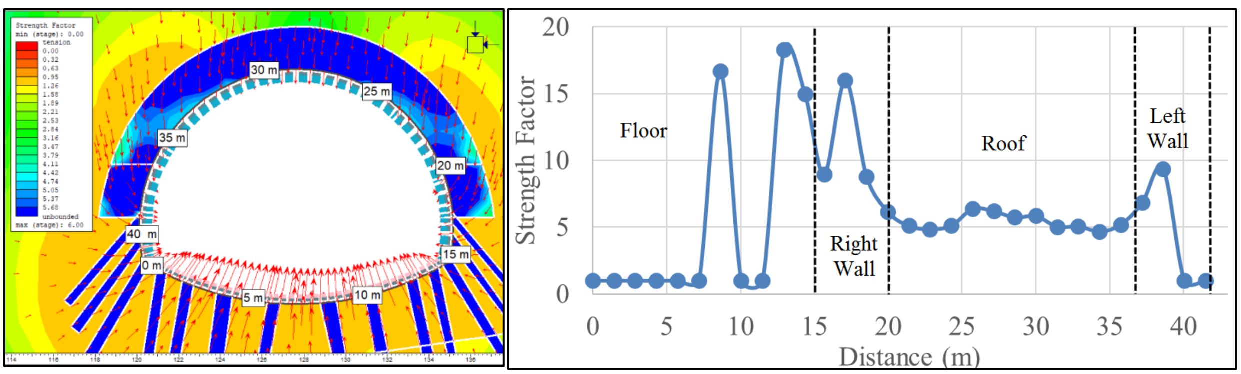

Figure 8. Tunnel Strength Factor

4.3.3 Probability Failure of Tunnel Support

The results of the supportr failure probability are obtained from the support capacity plot. This analysis aims to determine the response of the support system to the stages of excavation and to the deformation that occurs in the tunnel. The RS2 program provides output support capacity plots, which display the stress on the support and the envelope of strength based on factors of safety(FS). Support capacity is displayed in the form of a thrust vs moment plot curve and thrust vs shear. In this study, the support capacity plot curve is displayed for e types of support, namely beam, wiremesh, and shotcrete. The probability of failure analyzed in each type of support is assessed based on the value of the factor safety. In this study, the support type is considered failure if FS <1, or the point is outside the envelope of FS.

1. Beam

The plot curve of beam support capacity can be seen in Figure 9. Based on the figure obtained the beam has a FK value> 1.4. So the probability of a beam failure is 0%. This is indicated that all points are in the envelope of FK 1.4.

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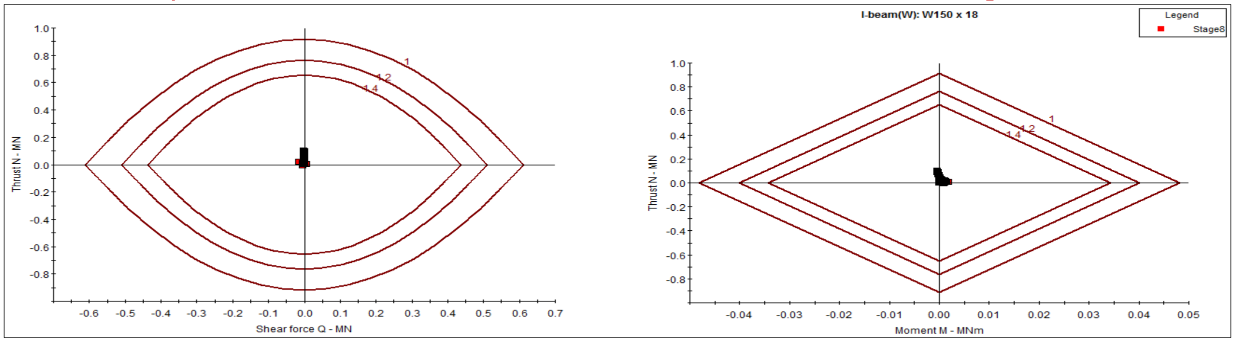

Figure 9. Support Capacity of Beam

2. Wiremesh

The plot curve of Wiremesh support capacity can be seen in Figure 10, based on the figure obtained by Wiremesh has a FK value> 1.4. So the probability of failure is equal to 0%.

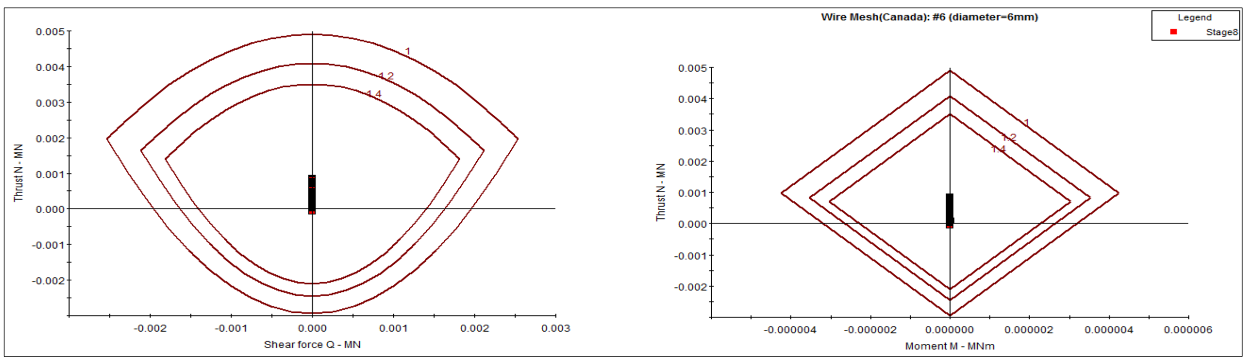

Figure 10. Support Capacity of Wiremesh

3. Shotcrete

The shotcrete support capacity and shotcrete failure location are presented in Figure 11, based on the Figure it can be seen that there are several points outside the strength envelope or FK> 1. FK values on the entire tunnel surface are 50 FK values, 5 of which have FK values> 1, so obtained a failure probability of 5/50 = 0.1 or 10%. The location of possible shotcrete failures can be seen in Figure 12.

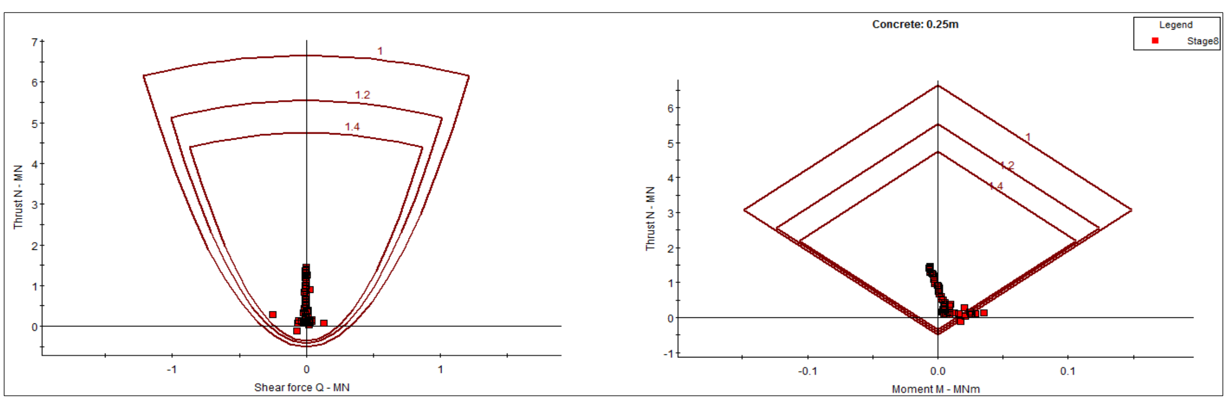

Figure 11. Support Capacity of Shotcrete

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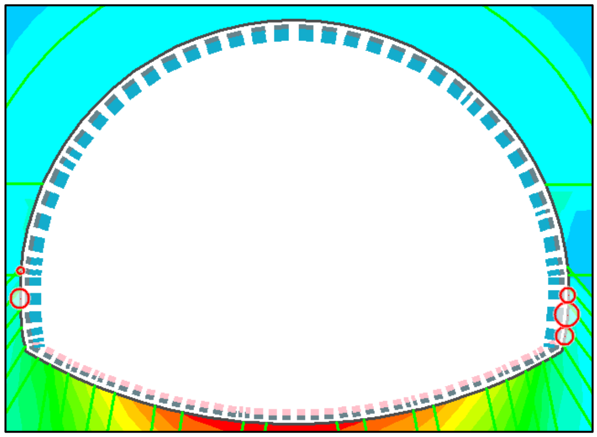

Figure 12. Location of Shotcrete Failure

The red circle sign shown in Figure 12 is the possibility of a shotcrete failure, the smaller the red circle, the smaller the FK value. When viewed on the curve in Figure 11, shotcrete failure is caused due to bending moment.

V. CONCLUSION

Many methods can be used in analyzing tunnel stability. Numerical modeling is the most widely used method, but numerical modeling is only a calculation tool, so a limit state is needed to determine whether the tunnel is stable or not, some simple approaches we can use are critical strain or hazar warning level, and probability of failure based on the support capacity plot.

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