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SENSITIVITY ANALYSIS OF ROCK MASS PARAMETERS ESTIMATE INFLUENCE ON DECLINE SUPPORT DESIGN USING NATM

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Abstract:

Capital mine development is often faced with limited geotechnical databases and designers are faced with more or less accurate estimates of missing parameters. GSI classification if often used with numerical modelling and its rounding unit is ±5 as suggested by its creators. In situ stresses are usually estimated in such manner that vertical component is equal to the weight of the rocks above, while horizontal components may vary in wide range, starting with ratio to vertical component of 0.3 and even be several times higher than vertical component.

Influence of estimate error of GSI and horizontal stress is analyzed for the Cukaru Peki location near Bor in Serbia. Zone in the rock mass valued with GSI of 40 at depth 160m is analyzed for the change of GSI value of ±5 and horizontal stress ratio between 0.5-1.5. Change of the unsupported length of decline and shotcrete layer thickness is tracked for different values of input parameters. Finally, best case and worst case scenarios are analyzed with results showing that shotcrete layer thickness could vary in range between 4-33cm, and unsupported lengths between 0.6-2m.

Keywords: Rock mass; Tunneling; NATM; GSI; Stress; Cukaru Peki; Bor;

1 INTRODUCTION

Design of the underground openings requires that crucial data about rock mass conditions should be determined, such as its strength, deformability and stress conditions that are present. Strength and deformability of the rock masses are commonly determined by classification systems such as Q (Barton et al., 1974), RMR (Bienawski, 1993) or GSI (Hoek, 1994). GSI is commonly used due to its applicability in numerical codes and possibility to determine it by limited database. However, this implies possible errors in determination and consequences that may occur.

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Stress filed is also one of crucial parameters that influence the stability and design of support for underground openings. Vertical stress component is generally equal to the weight of the above lying rock masses, while horizontal stress components are usually expressed as fraction of the vertical component. Terzaghi and Richart (1952) suggested method to determine horizontal stress ratio by relation $\frac{\nu}{1-\nu}$ which estimates it to around 50% of the vertical stress intensity. Numerous works (Zhang et al., 2012; Torbica and Lapčević, 2016) point out that horizontal stresses may be several times higher than those estimate using only Terzaghi's approach and that previous method is insuficient.

Herein, sensitivity analysis for the Cukaru Peki near Bor, Serbia site is analyzed by variation of the possible GSI and stress ratio values. Intention is to emphasize the influence that misjudgment of these parameters can have on advancement of excavation and support design.

2 METHODOLOGY

Decline is meant to be excavated down to the depth of almost 600m and along its trajectory passes through several weak zones. One of such weak zones is expected to be passed at the depth of the 160m. This zone is estimated to be around 120m long and passes through the jointed and weak sandstone layer. Rock mass properties of this zone are shown in Table 1.

Table 1 Initially estimated rock mass properties in the analyzed zone

Parameter	Estimated value
Compressive strength	50 MPa
Young's modulus	28 GPa
Poisson's ratio	0.3
GSI	37

Decline cross section is given in **Figure 1** and total cross sectional area is around 25m^2.

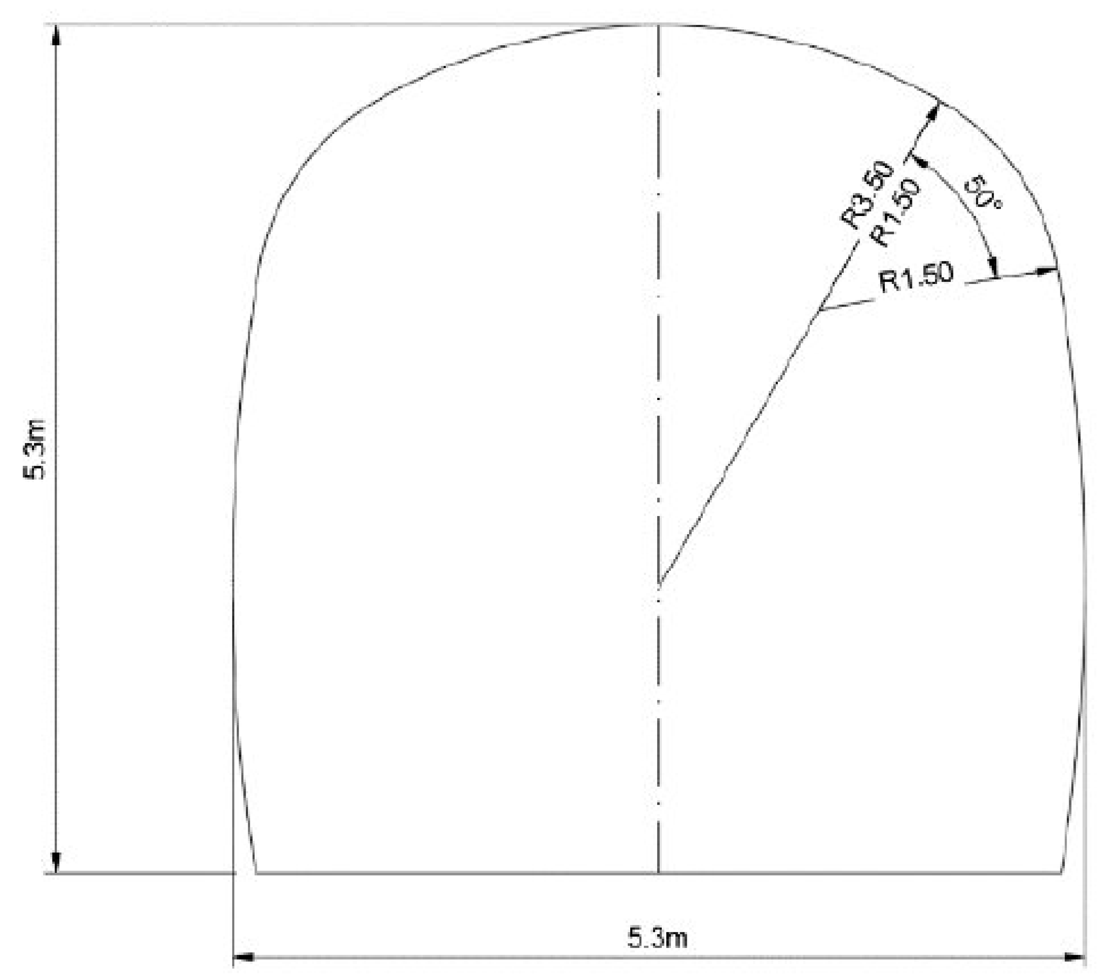

Figure 1 Decline cross section

Estimated GSI value of 37 (as given by geologist who logged the exploratory cores) is always just estimate and engineers should be aware of possible error of this estimate. Analysis herein is performed for 3 different GSI values of 35, 40 and 45 since these are estimated to be possible margins of error that is easy to made.

Initial stress condition are not know and therefore are estimated in such way that gravitational component is equal to the weight of above laying rock mass. Horizontal components are changed between 0.5-1.5*gravitational component in order to assess the model sensitivity to the actual stress state in to emphasize its importance. Orientation of the horizontal stresses is such that one component is perpendicular to the decline trajectory meaning that second component is aligned with decline trajectory.

As summary, GSI and stress value estimate error is tested for the impact on support and advancement step of decline development.

Analysis is based on the parameters given in Table 2 and Table 3.

Table 2 Rock mass parameters used in analyses

Parameters	GSI		
	35	40	45
σ_{ci} (MPa)	50	50	50
E_i (MPa)	28 000	28 000	28 000
E_m (MPa)	3175	4470	6260
Poisson's ratio	0.3	0.3	0.3
m_i	19	19	19
m_b	1.865	2.229	2.665
s	0.00073	0.001	0.002

Table 3 Stress data used in analysis

Stress components	k		
	0.5	1	1.5
σ_1 (MPa)	4.32*	4.32	6.48
σ_2 (MPa)	2.16	4.32	6.48
σ_3 (MPa)	2.16	4.32	4.32*

**vertical*

2.1 FEM model

Methodology used herein is based on the research proposed by Vlachopoulos and Diederichs (2009) that made it possible to estimate 3D progression of the underground opening with utilization of 2D FEM models. This method was originally developed for the circular shaper of the tunnels/drifts, however it has been shown to be applicable for other shapes of the underground openings. Main advantage of this method is that is eliminates complex 3D numerical modeling and reducing time necessary for analysis while providing results reliable enough.

FEM model is created using Phase2 software by Rocscience (2019) and has 20 stages. At the internal boundary of excavation distributed load is applied (Figure 2), where in the first stage this load is equal to the field stress and at every stage it is being decreased for 5% of initial load. In final stage complete distributed load is removed and plain stress strain state is simulated. In this manner progressive excavation is simulated.

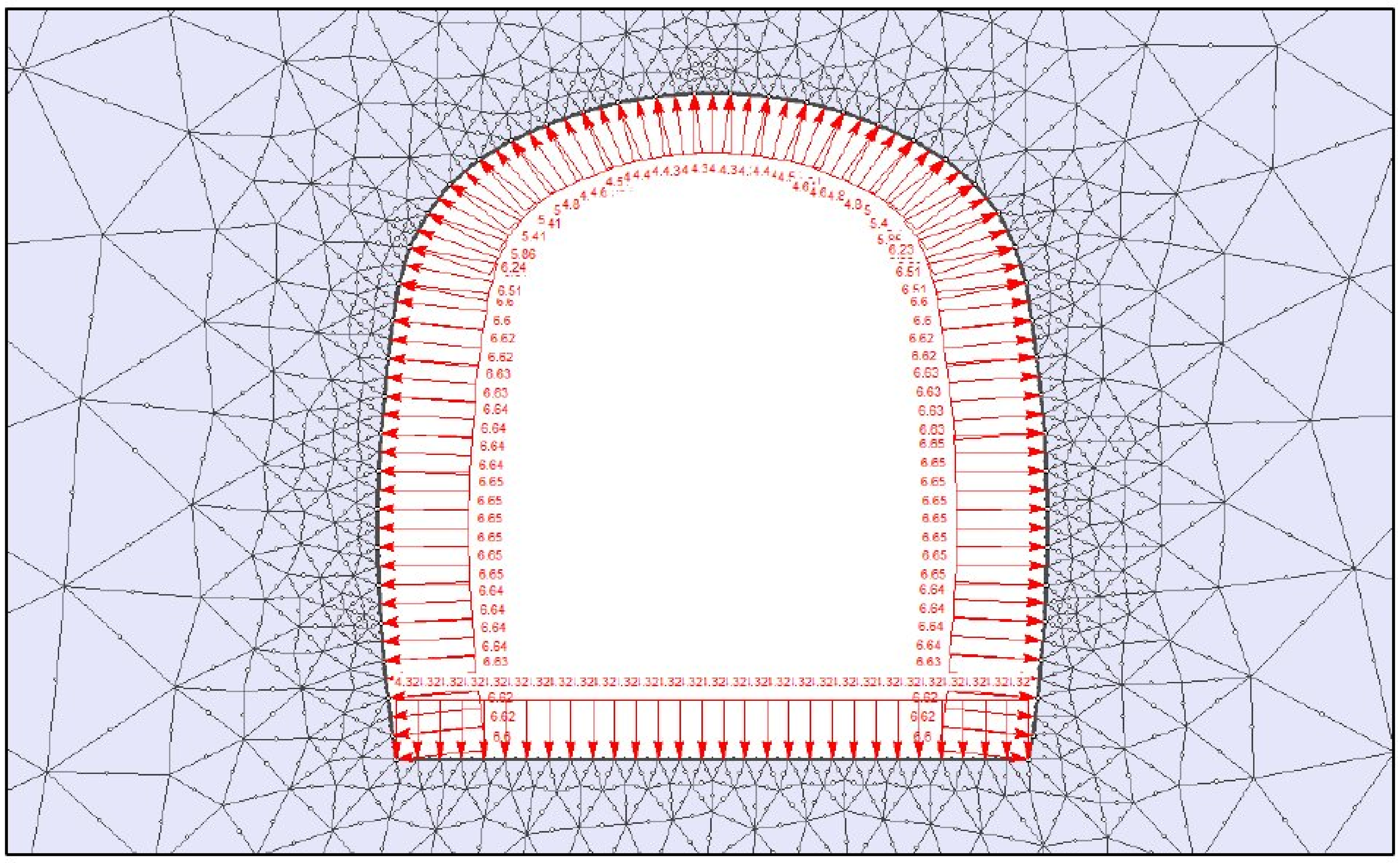

Figure 2 Distributed load at excavation boundary

After model convergence it is necessary to determine radius of plasticization around the underground opening (Figure 3) as well as deformation diagram for point with largest total displacement (Figure 4).

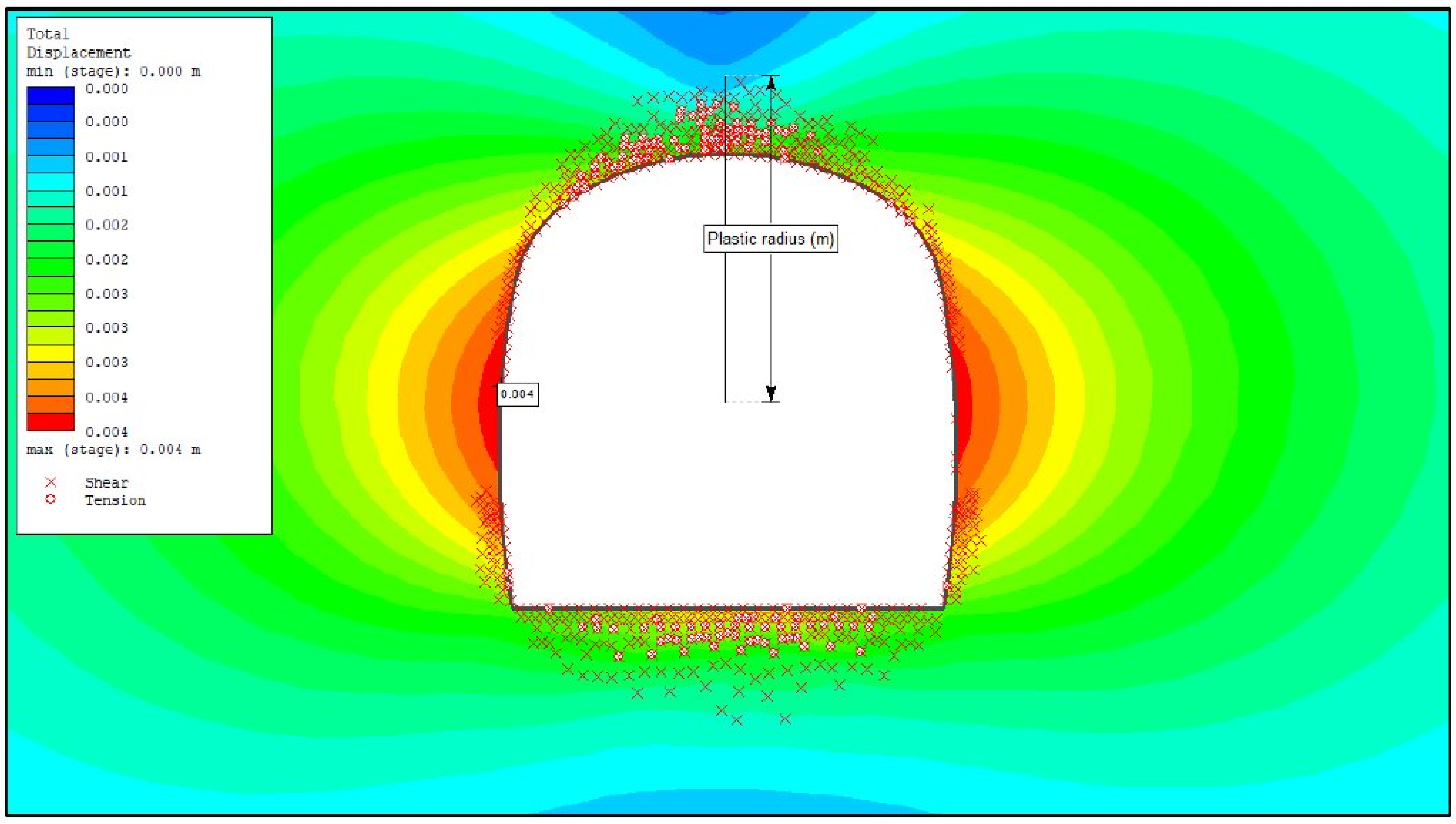

Figure 3 Plastic radius and total displacement in final stage of the model

Main principle is to determine longest unsupported length of the decline, or, put in other words, distance between supported part of decline and its face. Therefore, in FEM model we are determining the stage before failure of the rock mass occurs as it is illustrated in Figure 4. It is important to mark down the total displacement value in this stage since it is necessary for the next step of the procedure.

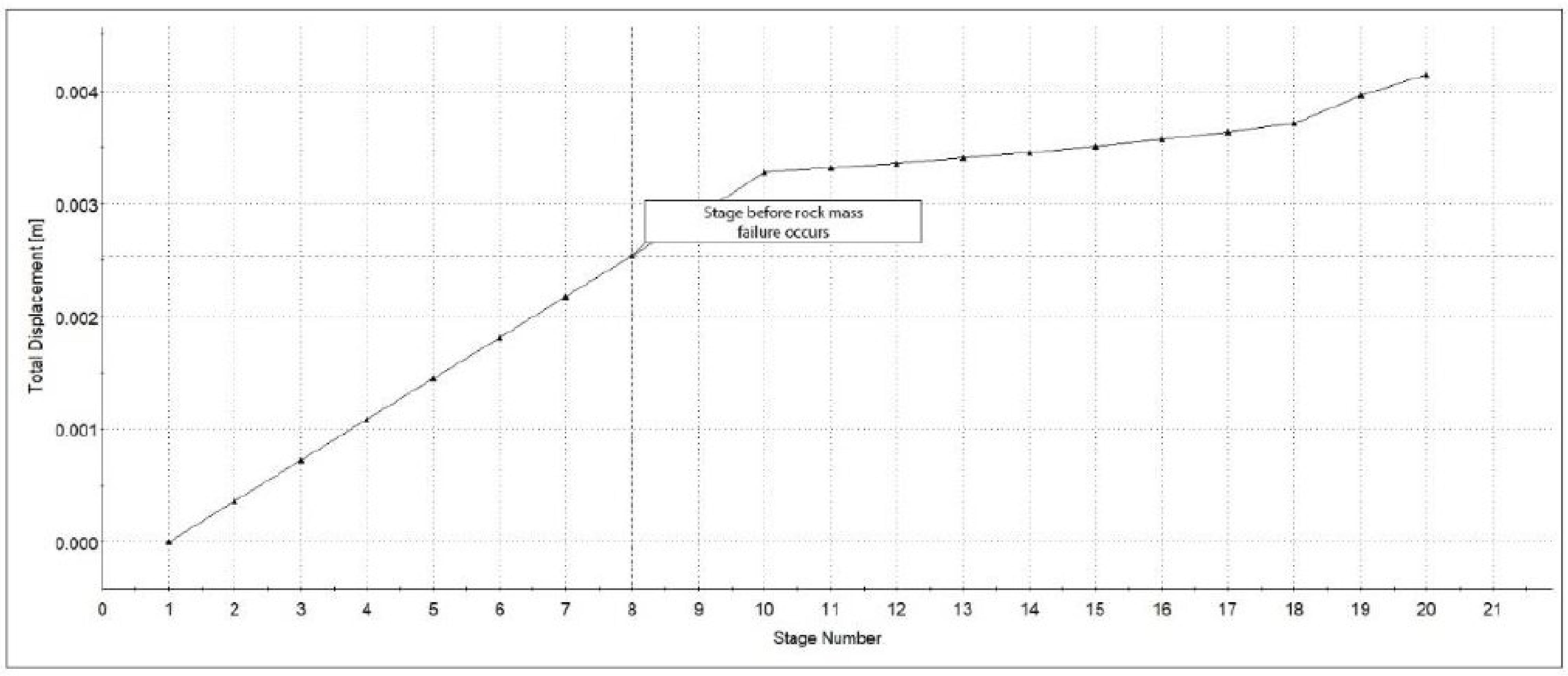

Figure 4 Deformation diagram for the point with largest total displacement

By knowing the ratio between the total displacement in the stage before the failure of the rock mass occurs and the maximum displacement (Closure to maximum closure as illustrated in Figure 5) we are using the diagram created by Vlachopoulos and Diederichs (2009) to determine the unsupported length of decline.

Diagram curves (Figure 5) represent the ratio between plastic zone radius and decline radius. For example if we determine following values:

Closure / maximum closure = 0.8

Plastic zone radius / decline radius = 1.5

Then we read from the diagram:

Distance from decline face / decline radius = 1.3

Finally, unsupported length of the decline is:

Distance from decline face = 1.3* decline radius

This procedure is repeated for each case.

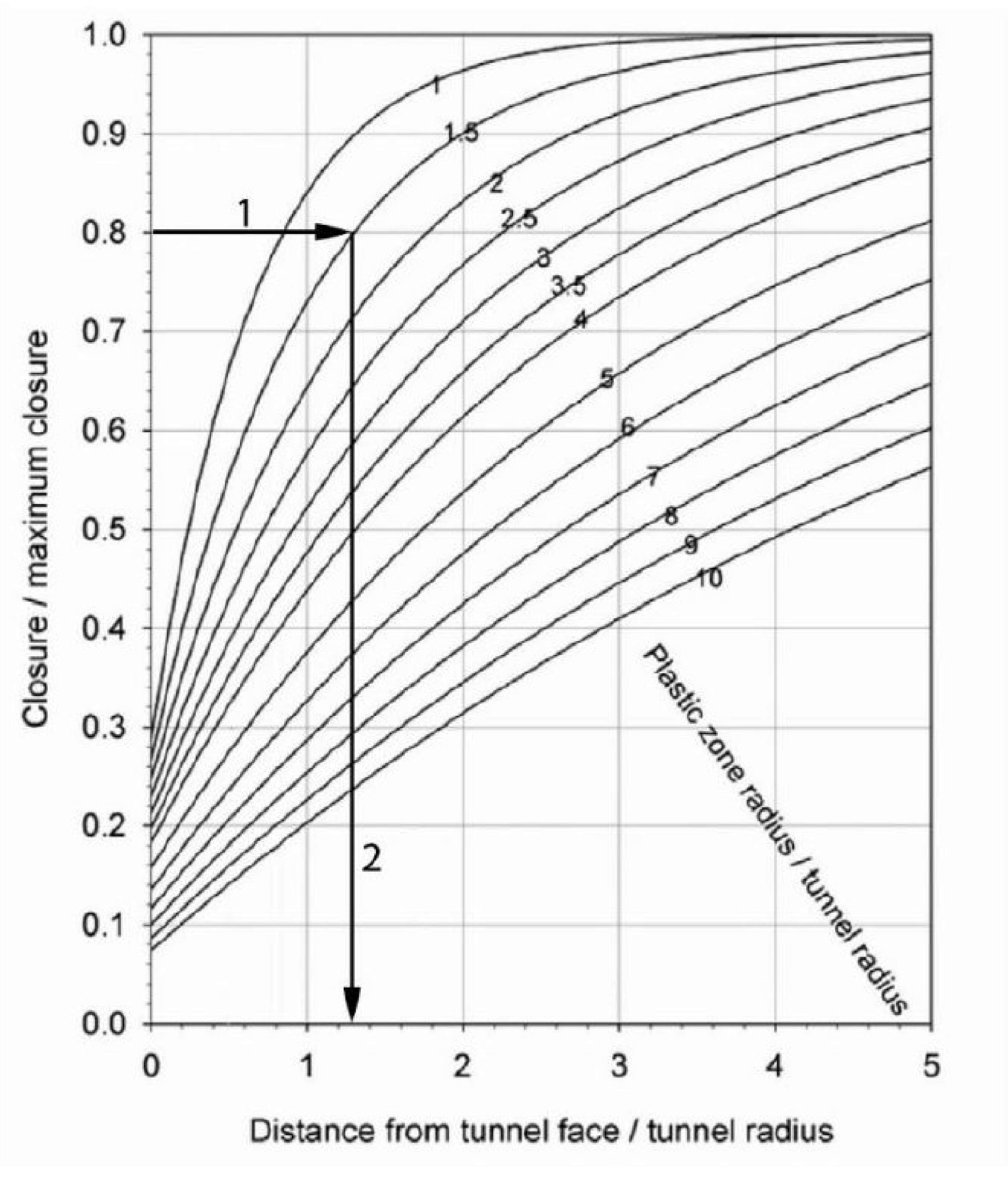

Figure 5 Diagram by Vlachopoulos and Diederichs (2009)

Determining the unsupported length of decline is first part of the analysis, second one is to determine necessary support for the decline. For simplicity, only thickness of the shotcrete layer is determined for the required FOS=1.4.

Support design procedure assumes that shotcrete layer is installed in the stage of FEM model before the failure of the rock mass, stage that has been determine in previous analysis. In each case same properties of shotcrete layer are used as shown in Table 4.

Table 4 Shotcrete properties used for the analysis

Parameter	
Young's modulus	30000 MPa
Poisson ratio	0.15
Compressive strength	30 MPa
Tensile strength	3 MPa

3 RESULTS

3.1 GSI error influence

As it was originally suggested by the Hoek (2007) GSI value should be rounded by increments of 5, meaning that value of GSI=37 that was initially estimated should be 35 or 40. Herein, it is assumed that initial GSI value is 40 and with consideration of the possible estimate error it can spread in span between 35-45. Therefore, there GSI=35, 40 and 45 values are analyzed.

Change of the GSI influences change of the rock mass strength properties for the Hoek-Brown's failure criteria and deformation modulus (estimated by Hoek and Diedrich (2006)). Change of these parameters influences support loading as well as size of the plastic zone around underground opening.

All the stress components are equal ($\sigma_v = \sigma_H = \sigma_h = 4.32MPa$). Intensity corresponds to the intensity of the gravitational component for the depth of 160m assuming that average unit weight is $0.027MN/m^3$.

Analysis results are presented in Figure 6, Figure 7 and Table 5. It is seen that decreasing GSI value determines lower unsupported lengths ranging from 1.4m to 0.6m. Roughly talking unsupported length of the decline is around 1m and it determines the advancements and excavation organization. If we compare extreme values, it implies that advance of 1.4m is more than doubled compared with 0.6m, meaning that twice as much time and costs are necessary to develop decline in worse conditions.

Table 5 Analysis results

Parameters	GSI		
	45	40	35
Tunnel radius, Rt (m)	2.8	2.8	2.8
Plastic zone radius, Rp (m)	3.25	3.8	3.8
Rp / Rt	1.16	1.36	1.36
Maximum closure, u_{max} (m)	0.0030	0.0064	0.0110
Closure in support installation stage, u (m)	0.0018	0.0034	0.0047
u / u_{max}	0.60	0.53	0.43
Distance from face / tunnel radius	0.5	0.4	0.2
Distance from face	1.4	1.1	0.6

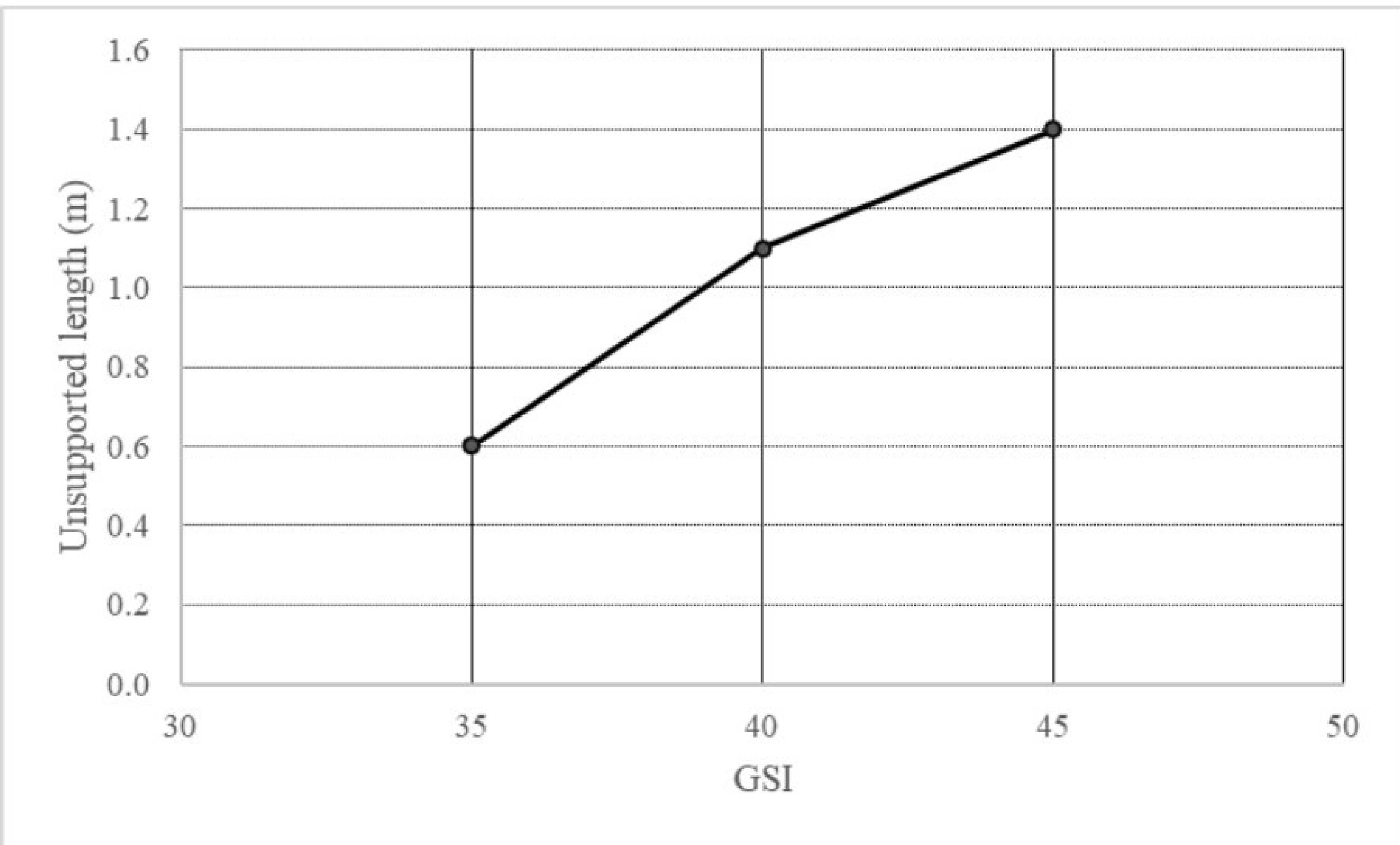

Figure 6 Unsupported length vs GSI change

Figure 7 illustrates change of the necessary shotcrete thickness for different GSI values. It is seen that for the lowest GSI value of 35 shotcrete layer with the FOS=1.4 has thickness of 9cm, while in the case of GSI=45 shotcrete layer with same FOS has thickness of 4cm. It is clear that amount of support that is necessary in these cases is enormous. Considering differences in advancements that could be achieved it is obvious that costs may be significantly different.

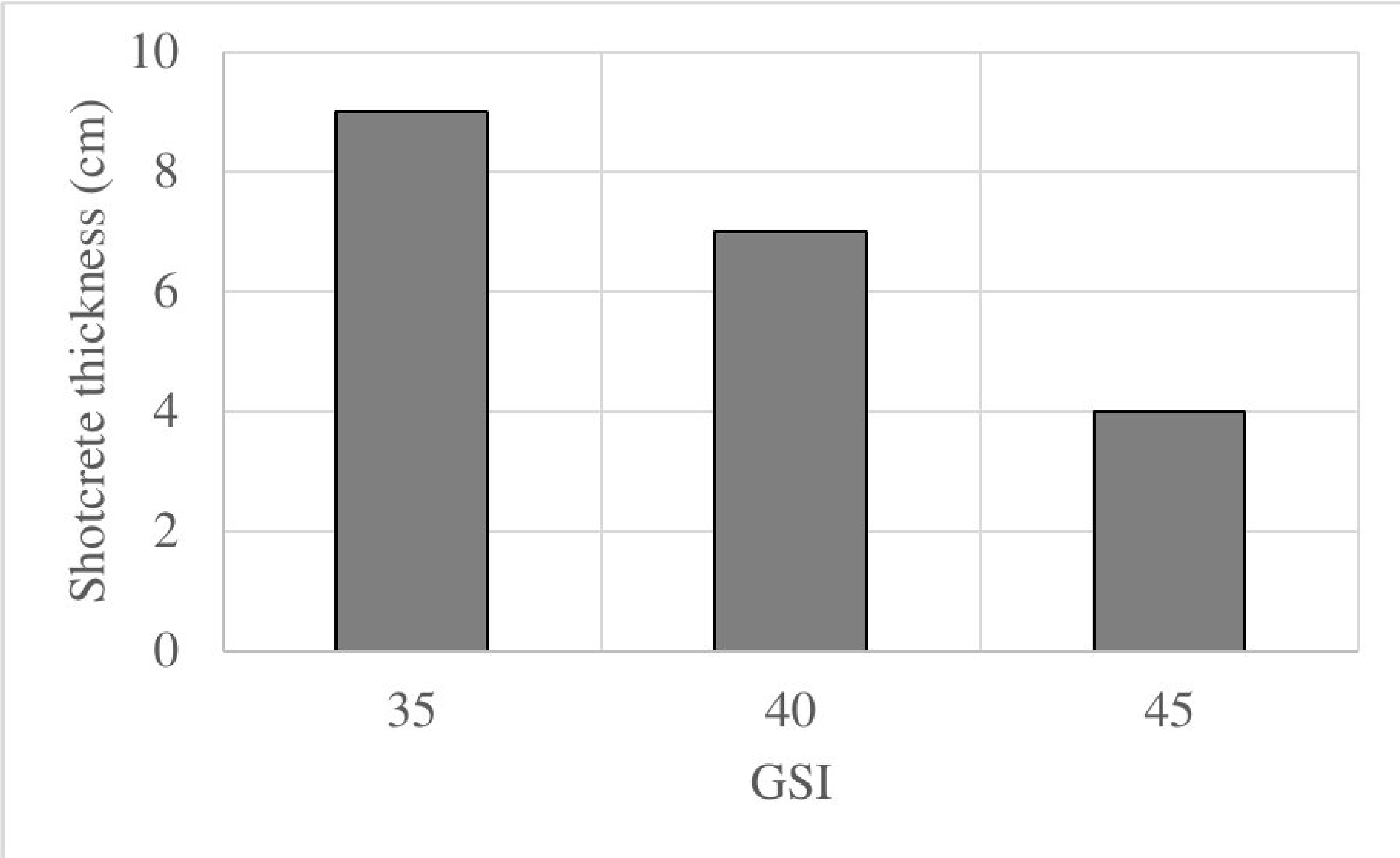

Figure 7 Shotcrete thickness vs GSI change

3.2 Horizontal stress influence

Common approach in geomechanical analyses is that horizontal stress is expressed as portion of the gravitational stress component. Horizontal component often estimated by the expression $\frac{\nu}{1-\nu}$ which estimates horizontal stress between 30-50% of the vertical component depending on actual Poisson's ratio value. However, horizontal stress ratio is know to be higher than this (1-1.5) especially at shallow depths. Zhang et al. (2012) provided useful estimate for the different rock types.

Analysis herein is done for 3 different stress ratios (0.5, 1 and 1.5) with equal minor and major horizontal components. Rock mass parameters are taken for the average GSI value of 40. Results are given in Table 6, Figure 8 and Figure 9.

Table 6 Analysis results

Parameters	k		
	0.5	1	1.5
Tunnel radius, Rt (m)	2.8	2.8	2.8
Plastic zone radius, Rp (m)	3.9	3.8	4.13
Rp / Rt	1.39	1.36	1.47
Maximum closure, u_{max} (m)	0.0053	0.0064	0.010
Closure in support installation stage, u (m)	0.0034	0.0034	0.0046
u / u_{max}	0.64	0.53	0.46
Distance from face / tunnel radius	0.7	0.4	0.3
Distance from face	2	1.1	0.8

As it is seen, unsupported length of decline decreases with the increasing horizontal stresses as it could have been expected. In case when horizontal stress is 50% of the vertical component is possible to achieve advance of 2m before support installation and roof collapse. Unsupported length drops significantly for ratios 1 and 1.5 and is around 1m.

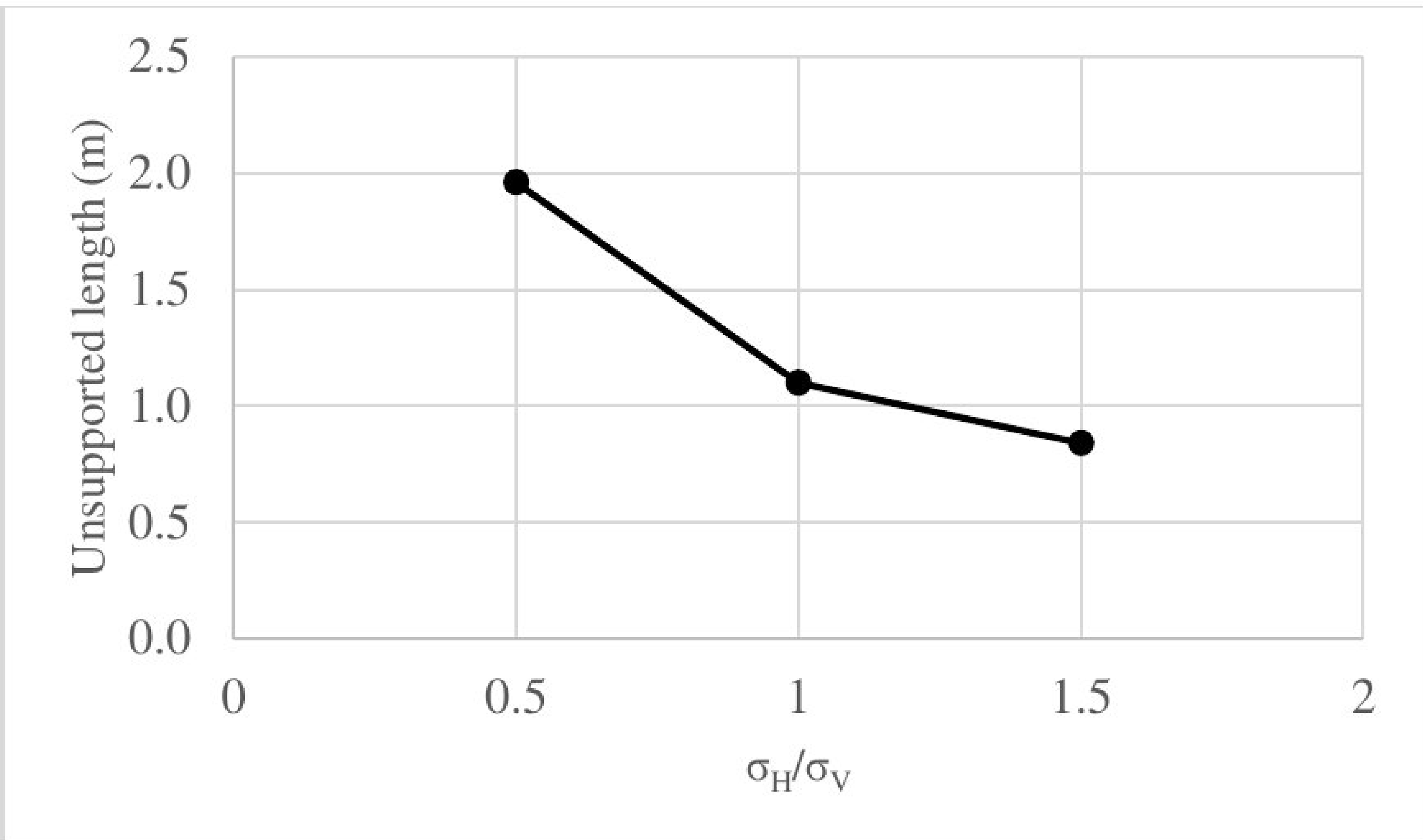

Figure 8 Unsupported length vs stress ratio

For the previously determined unsupported lengths shotcrete thickness is determined using explained methodology. In the case of highest thickness of the shotcrete layer with the FOS=1.4 is 15 cm while for the case where stress ratio is 1 shotcrete layer is 7cm thick. This illustrates the stress influence on the needed support.

For the case where stress ratio=0.5 shotcrete layer is 10cm thick. This comes from the fact that in this case unsupported length of the underground opening is almost 2 times higher than in previously discussed cases.

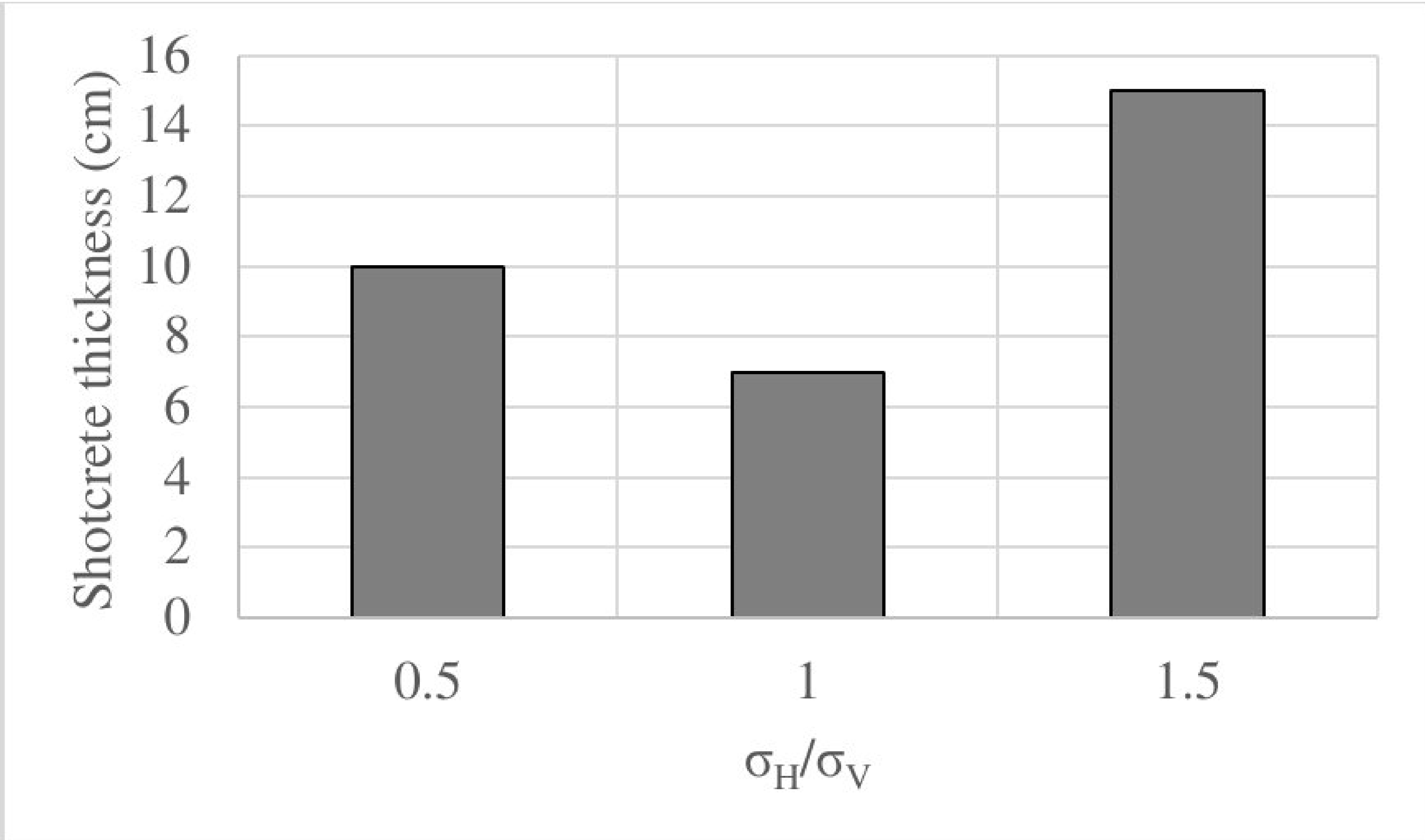

Figure 9 Shotcrete thickness vs stress ratio

3.2 Best- and worst-case scenarios

As it was shown in previous sections, possible estimate errors in stress and GSI values may have high impact on excavation advancing and required support. Intention is to illustrate range of possible outcomes and to point out what dangers are lying under deterministic selection of analysis parameters without testing the sensitivity of their influence.

In best-case scenario it is like to assume that rock mass is stronger than estimated and that stresses are low. Therefore, in our case this would mean that rock mass has GSI of 45 and that horizontal stress ratio is 0.5.

For the worst-case scenario we would like to assume that rock mass is weaker than initially estimated and that stress ratio is higher. In our case this would mean that GSI value is 35 while horizontal stress ratio is 1.5.

Results presented in Table 7, Figure 10 and Figure 11 show that two cases may differ significantly. If most favorable conditions are experienced unsupported length of the underground opening can be around 2m and shotcrete layer with FOS=1.4 has thickness of 4cm.

On the other side, if lower strength and higher stress are to be present unsupported length of the underground opening is significantly reduced to 0.6m. Shotcrete layer thickness is dramatically increased to 33cm in order to maintain desired FOS.

Table 7 Analysis results

Parameters	Worst	Best
Tunnel radius, Rt (m)	2.8	2.8
Plastic zone radius, Rp (m)	4.7	3.75
Rp / Rt	1.68	1.34
Maximum closure, u_{max} (m)	0.015	0.0027
Closure in support installation stage, u (m)	0.0062	0.0018
u / u_{max}	0.41	0.67
Distance from face / tunnel radius	0.2	0.7
Distance from face	0.6	2

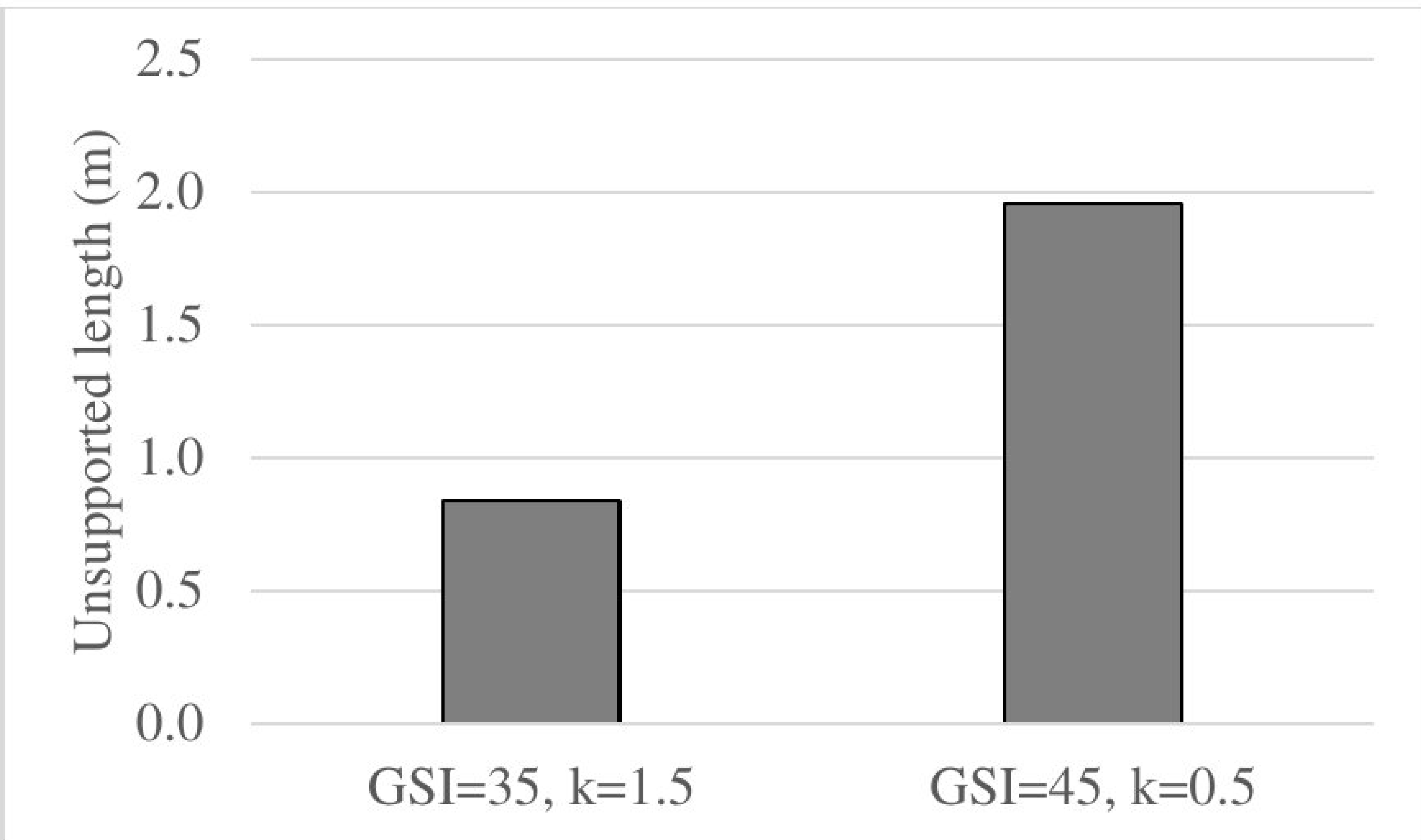

Figure 10 Unsupported length vs stress ratio

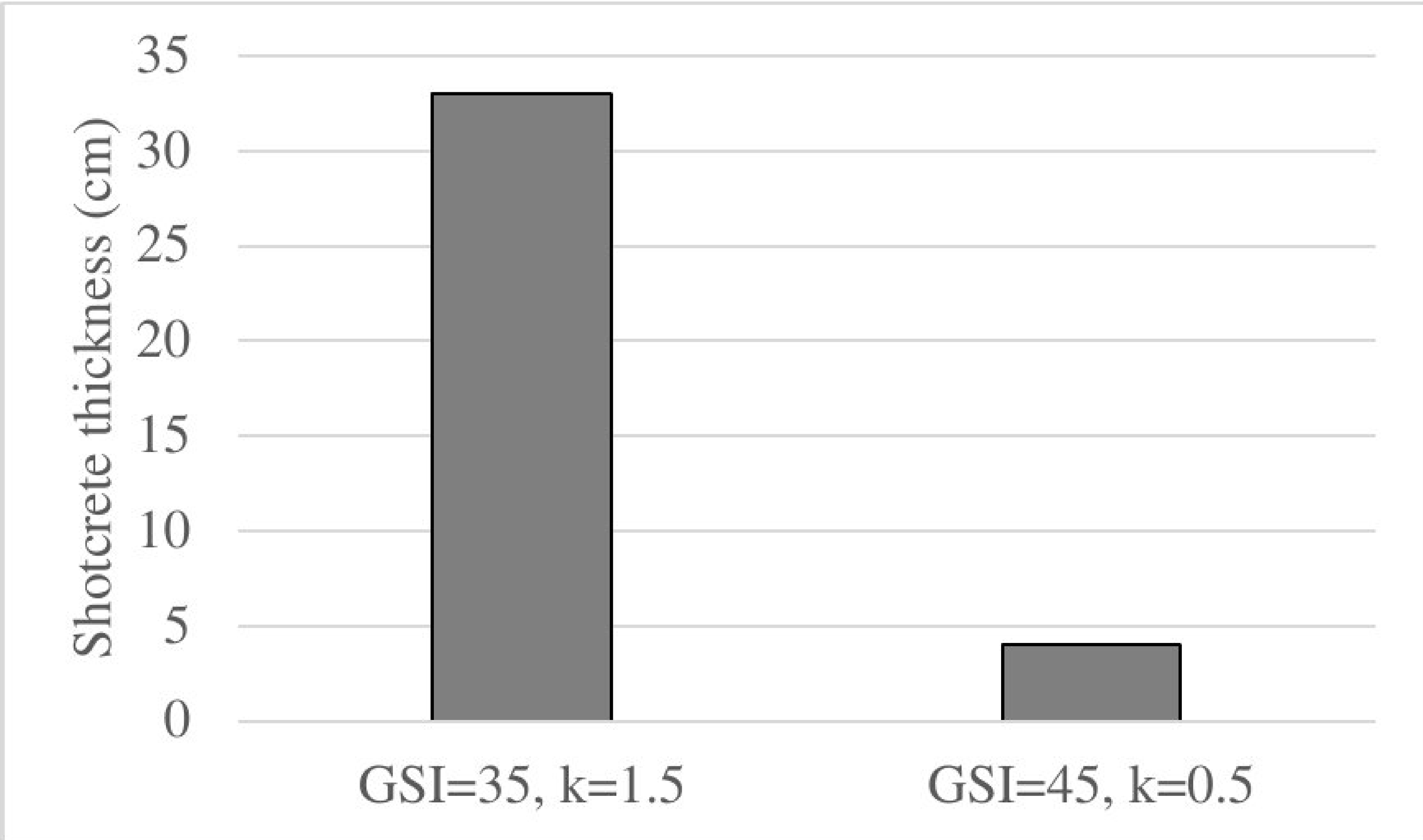

Figure 11 Shotcrete thickness fro best- and worst-case

4 CONCLUSION

Analysis herein was undertaken in order to emphasize the importance of sensitivity analysis of crucial parameters for design of the underground openings. Geological strength index is commonly estimated in increments of 5 and therefore its estimation is highly subjective. Range of GSI values ± 5 of the estimated value is possible error that could be made. It is not unusual that horizontal stress components are not treated with care they require, especially in mining applications.

Results show that GSI change influences both unsupported length of decline as well as thickness of the shotcrete layer. With increasing GSI value from 35 to 45 unsupported length increases from 0.6m to 1.4m, while required shotcrete layer thickness ranges from 4cm to 9cm (FOS=1.4) for corresponding lengths. Major reason for these changes comes from the estimate of the deformation modulus of the rock mass that is directly related to GSI.

Stress change was analyzed by changing the ratio between horizontal and vertical components from 0.5 to 1.5, while rock mass strength and deformability parameters were kept constant (GSI=40). Unsupported length goes from 2m in case of the ratio of 0.5 to 0.8m in case when stress ratio is 1.5. For case with ratio of 0.5 shotcrete layer thickness is 10cm (FOS=1.4) while in the case of ratio of 1.5 thickness is 15cm. In case where stress ratio is 1 shotcrete layer is 7cm thick, but this comes from the fact that unsupported length is 1.1m and should not be misinterpreted if compared with the case with lowest stress ratio.

In best case where strength of the rock mass is the highest and the stresses are lowest it is shown that unsupported length is around 2m and necessary shotcrete thickness is 4cm. on the other side, in worst possible case unsupported length is 0.6m with shotcrete layer of 33cm.

By comparing best and worst possible cases it is clear that possible estimated errors could lead in serious misjudgments about necessary support and advancing of excavation. Other implications may include delays and work safety issues, as well as planning of required materials.

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