

Estimating extent and properties of blast-damaged zone around underground excavations

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Estimating extent and properties of blast-damaged zone around underground excavations

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Abstract

A common technique in hard rock tunneling, and underground excavation in general, is drilling and blasting. This method of excavation assumes that damage will be done to the surrounding rock mass depending on its quality. Herein is a proposal for how to estimate blast-damaged zone extent and shape, and how to quantify rock mass properties in this zone. Comparison was made, using FEM, between models with and without blast-damaged zone consideration through analyzing its impact on the support load. Results showed greater support loads in the case of blast-damaged zone modeling. Also, comparison was made between the proposed method for blast-damaged zone quantification and quantification using blast damage factor D. Results showed that in both cases, the support loads are in same magnitude order. In other words, compared to the blast damage factor D method, the proposed is more objective and makes it possible to describe the blasting process before it is actually done.

Keywords: Tunneling in hard rock, rock blasting, blast-damaged zone, rock breakage by explosives.

1. Introduction

Tunneling in hard rock is hard to imagine without using explosives for rock excavation. In the case of this excavation method, the surrounding rock mass that represents the tunnel walls is damaged by blasting. Depending on how well the blasting pattern is designed, determines how damaged the surrounding rock mass will be. Many researches have been conducted to describe and quantify blast-induced damage of surrounding rock mass and many techniques have been invented to prevent this damage. Many tests were conducted by Swedish Rock Engineering Research Organization (SveBeFo) and these test offered new explanations on how rock breakage by explosives occurs Langefors and Kihlstrom (1973) and Brinkmann (1987, 1990). Järnvägs (1996) introduced a contour blasting technique that reduced blasting damage to surrounding rock mass. Ouchterlony (1997) gave an equation for estimating radial crack length, and later this equation was expanded by correction factors, Ouchterlony *et al.* (2002). Many papers that consider damaged zone extent were

published by Hustrulid. Hustrulid (1999) analyzed the energy and work done by an explosive charge in a borehole. In 2002. Hustrulid and Lu presented their approach to determine peak particle velocity (PPV), which was later revised by Hustrulid and Johnson (2008). Hustrulid (2010) used Ash's (1963) a classic approach to develop the extent of damage based on explosive energy. Based on Holmberg's (1982) paper, Hustrulid (2010) presented the Rock Constant Approach. Senek (1979) presented his method for crack zone radius estimation. This method was based on the relationship between borehole pressure and tensile strength. Kanchibotla *et al.* (1999) offered an equation to estimate the crushing zone radius based on the relationship between borehole radius, borehole pressure and compressive strength of rock. Most of models cited here use parameters such as PPV, borehole radius and pressure inside borehole, and strength parameters usually consider only compressive strength of rock, although it is very well known that failure in rock occurs by tension or

shear. Tensile strength of rock was incorporated in the theoretical work published by Drukovanyi *et al.* (1976), although this work does not explain mechanisms by which blast damage in rock occurs. Also, practice showed that this approach was applicable only for rocks with a compressive strength over 100 MPa. These models are only considered for estimating the extent of the blast damaged zone and do not provide any suggestion as to how to estimate the mechanical properties of rock mass in this zone.

Kwon *et al.* (2009) conducted a series of tests to determine the extent and properties of the blast damaged zone at the KAERI underground research tunnel. It is reported that the blast-damaged zone (BDZ) had an extent of around 2 m; the RQD in this zone decreased around 17% in comparison with the RQD in undisturbed rock mass; and the elastic module decreased around 56%. A decreased RQD value means that the rock mass in the damaged area is becoming more jointed, which is an important fact for this paper.

A description of rock damage by

blasting was incorporated in Hoek-Brown's failure criterion by introducing the blast damage factor D , Hoek et al (2002). Factor D at first was used for estimating parameters of Hoek-Brown's failure criterion m_b , s and a . Later on, it was incorporated in the estimation of a rock mass modulus Hoek and Diederichs (2006). Values for the blast damage

factor and extent of damaged zone are suggested by Hoek (2002, 2012). This suggestion is made on empirical investigations and there is no solution for the exact estimation of these parameters. A common mistake made by engineers is that factor D is applied on whole rock mass instead of just the close perimeter around excavation. This mistake

is caused by lack of exact methods to estimate the damage zone extent and to estimate the mechanical properties of this zone. With this in mind, the authors herein has now mainly focused on this problem, and presents a method for estimating the extent and rock mass properties of the blast-damaged zone around the excavation.

2. Rock blasting theory

According to a new rock blasting theory, Torbica and Lapcevic (2014), it is possible to calculate the radii of zones having a different density of radial cracks around the

blasthole. This theory explains the fracturing mechanism of rock under explosive load. Here, the main part of that theory is presented, since it is the basis of this paper. Detonation of

an explosive charge in rock results in dynamic loading of the walls of the borehole and generation of a pressure wave that transmits energy through the surrounding medium.

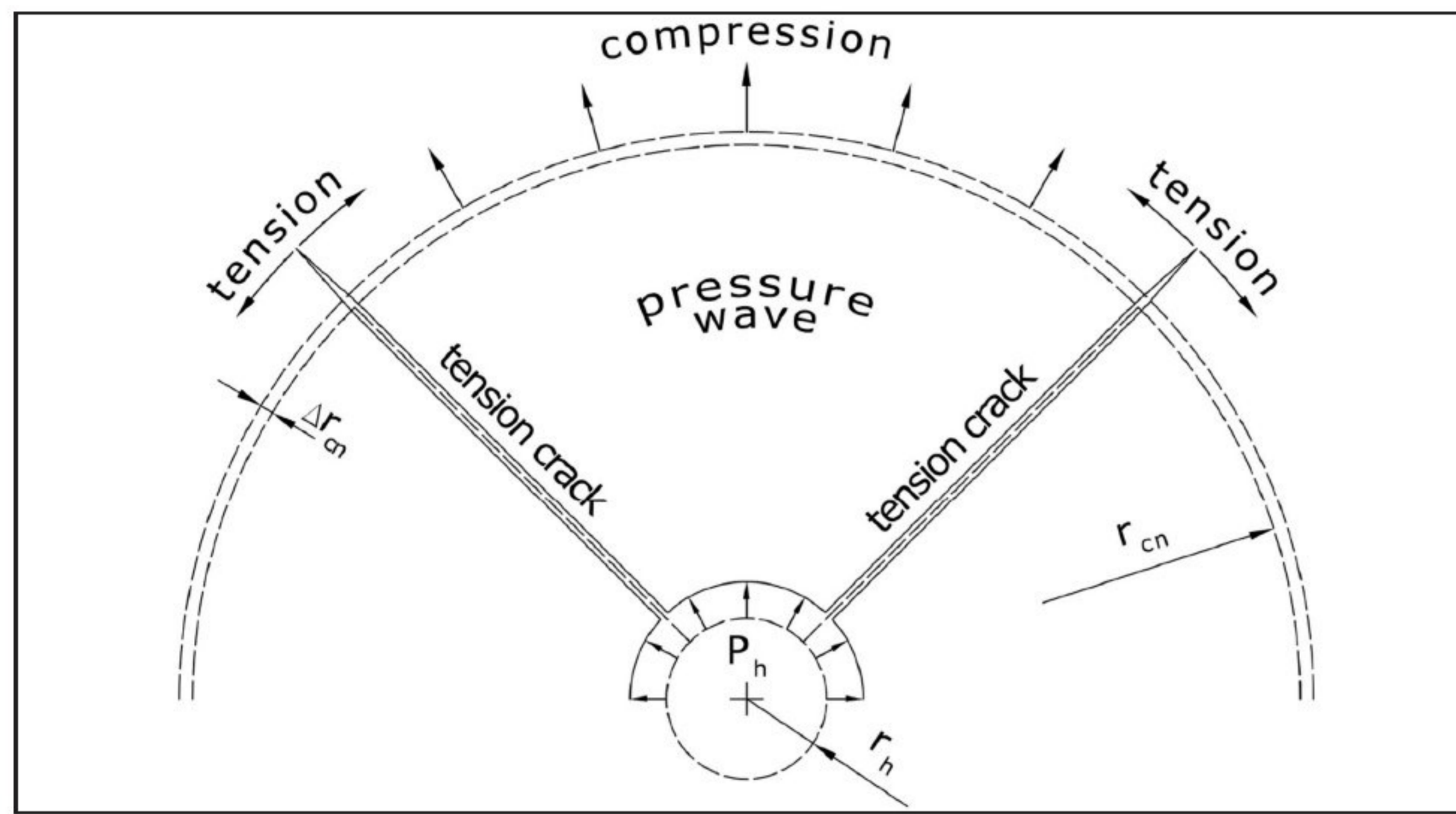


Figure 1
Radial crack formation mechanism.

The pressure wave extends from borehole walls circularly around the

borehole (Figure 1). At the distance r_{cn} from the borehole compressive stress of

the rock in the radial direction is:

$$\sigma_{rc} = P_h \frac{r_h}{r_{cn}} \tag{1}$$

Where:
 σ_{rc} - radial compressive stress

P_h - borehole pressure
 r_h - borehole radius

r_{cn} - crack zone radius

On the other hand:

$$\sigma_{rc} = M \cdot e_r \tag{2}$$

Where:

$$M = E \cdot \frac{(1-\nu)}{(1+\nu)(1-2\nu)} \tag{3}$$

$$k = \frac{(1-\nu)}{(1+\nu)(1-2\nu)} \tag{4}$$

$$\sigma_{rc} = E \cdot k \cdot e_r \tag{5}$$

M - pressure wave modulus
 e_r - radial strain

E - Young's modulus of rock
 ν - Poisson's ratio

Or:

$$(6) \quad e_r = \frac{\sigma_{rc}}{E \cdot k}$$

Therefore:

$$(7) \quad e_r = \frac{P_h \cdot r_h}{E \cdot k \cdot r_{cn}}$$

At the distance r_{cn} , before the pressure wave gets to it, the perimeter of the closed circular ring zone of rock mass is:

$$(8) \quad O_r = 2\pi r_{cn}$$

When the pressure wave reaches the closed circular ring zone of rock mass, it is moved to a new position with a radius $(r_{cn} + \Delta r_{cn})$, and with the perimeter:

$$(9) \quad O_{(r_{cn} + \Delta r_{cn})} = 2\pi(r_{cn} + \Delta r_{cn})$$

Therefore:

$$(10) \quad O_{(r_{cn} + \Delta r_{cn})} = 2\pi(r_{cn} + e_r \cdot r_{cn})$$

Once the closed circular ring zone of rock mass is subjected to tension with a lateral strain:

$$(11) \quad e_l = \frac{O_{(r_{cn} + \Delta r_{cn})} - O_{r_{cn}}}{O_{r_{cn}}} = e_r$$

For the formation of the radial tension cracks, tensile strain is required:

$$(12) \quad e_t = \frac{\sigma_t}{E}$$

Where:
 e_t - tensile strain
 σ_t - tensile strength
 E - Young's modulus of rock

In addition, the number (n) of radial tensile cracks formed at a distance r_{cn} will be:

$$(13) \quad n = \frac{e_l}{e_t}$$

Therefore, it is:

$$(14) \quad n = \frac{P_h r_h}{k \sigma_t r_{cn}}$$

Therefore:

$$(15) \quad r_{cn} = \frac{P_h r_h}{k \sigma_t n}$$

Therefore, for the borehole radius $r_h = 0.051\text{m}$ and the borehole pressure $P_h = 1.6 \text{ GPa}$ in limestone with tensile strength $\sigma_t = 7\text{MPa}$, Poisson's ratio $\nu = 0.3$ (Table 1) will be:

n	2	4	8	16	32
r_{cn} (m)	4.32	2.16	1.08	0.54	0.27

Table 1
Zone radius with crack density

In Figure 2, the schematic illustration of tension crack length and density

around blasthole is shown. Practical application of this theory was demonstrated

by Torbica and Lapcevic (2014) for the estimation of blasted rock fragmentation.

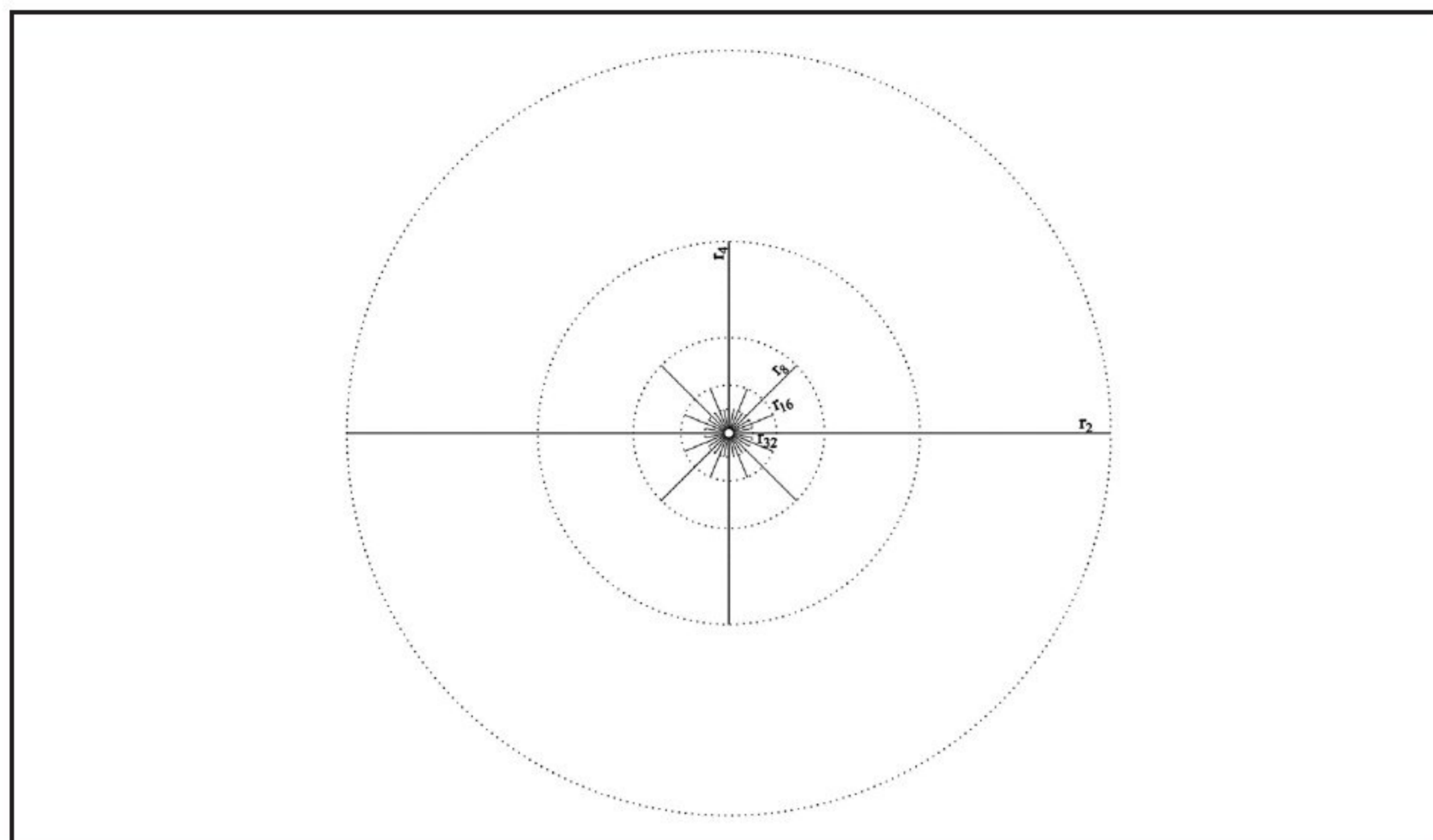


Figure 2
Schematic illustration of tension crack length and density around blasthole.

3. Estimating the extent and properties of the blast-damaged zone around the underground excavation

3.1 Damaged zone extent and shape

The extent and shape of the damaged zone around the excavation depends on the blasting pattern, cross section of the excavation and the structure of rock mass (primary block sizes). One should imagine rock mass as a set of interlocked monolith blocks that are separated by preexisting rock joints. In this manner rock blocks may be considered to be an elastic part of rock

mass and their plasticization is done through the blasting process when blast-induced radial cracks are formed. It is well known that discontinuities as joints and fractures in rock mass limit pressure wave propagation and therefore may limit propagation of radial cracks induced by explosive charge, Figure 3. This also means that preexisting joints and fractures define the extent and

shape of the blast damaged zone around the excavation, as explosive charge will break only the rock block in which it is placed. In other words, preexisting joints define domain for pressure wave propagation. Numerical investigations, using distinct element modeling, on radial crack and pressure wave propagation in jointed rock mass are presented by Aliabadian *et al.* (2014).

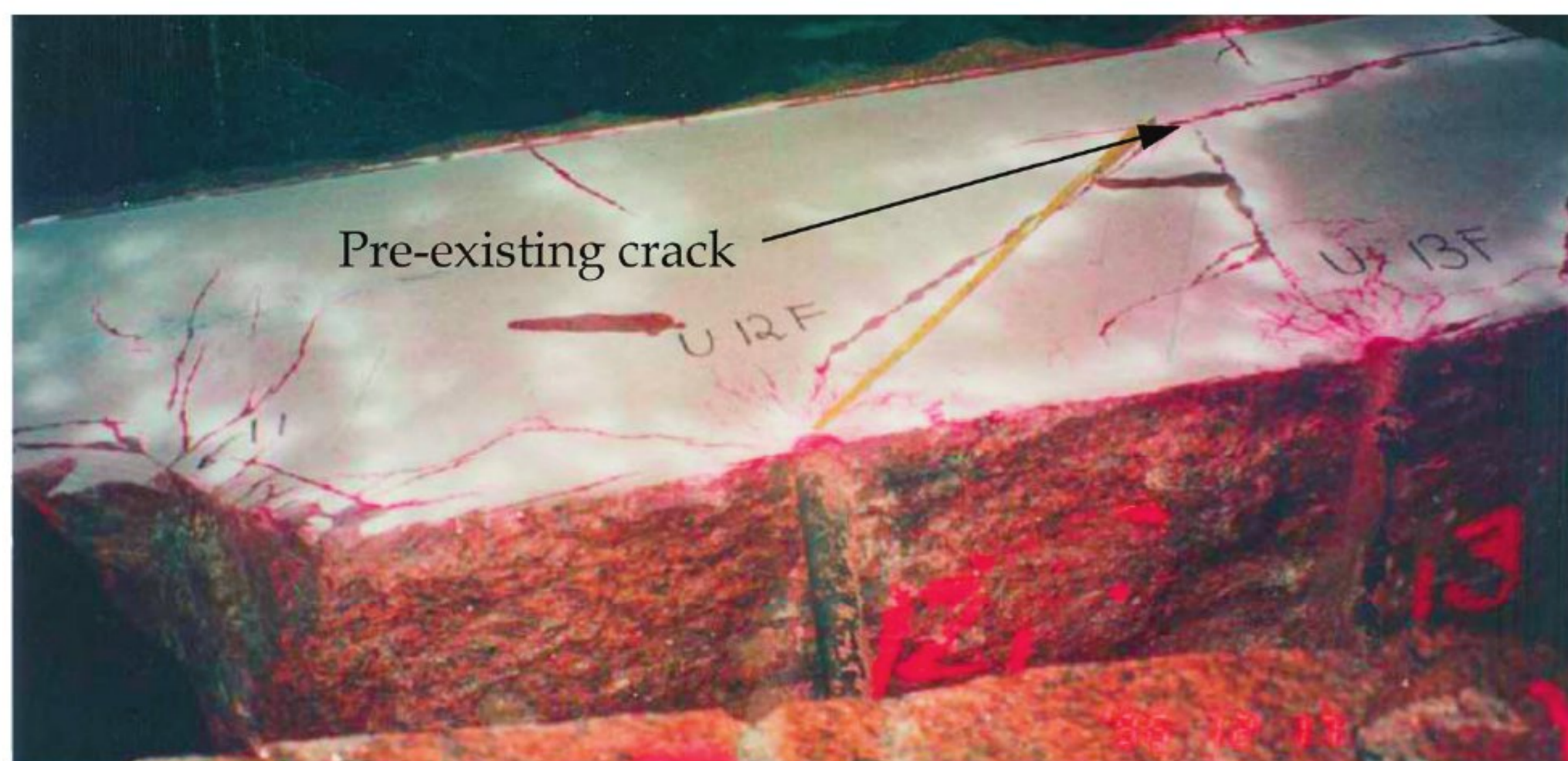


Figure 3
Impact of preexisting joints in rock mass on blast induced radial crack length, published in Saiang (2008) with permission from Mats Olsson.

As it is possible to determine the radii of cracking zones around the blasthole, we can use this principle to determine the extent of the damaged zone in the rock mass surrounding the excavation. As first, the damaged zone extent depends on the explosive pressure in the contour blastholes. It is well known that contour blastholes are placed closer and charged with less explosive than other blastholes in the blasting pattern.

Therefore, if blastholes are charged less, the length of the tension cracks around the excavation decreases and vice versa. The shape of the damaged zone depends on the excavation cross section and its shape is the same as the shape of excavation cross section, but it is offset for the radius of the damaged zone.

Depending on how much rock mass is jointed we can differ two possible situations. The first situation is

when primary block size is larger than the maximum length of the blast induced radial cracks in zone r_4 . In this case pressure wave that induces radial cracks is not limited by preexisting joints in rock mass and radial cracks may reach their maximum length, Figure 4. Therefore, for this situation, the extent of the blast damaged zone is equal to the maximum length of the radial cracks in cracking zone r_4 .

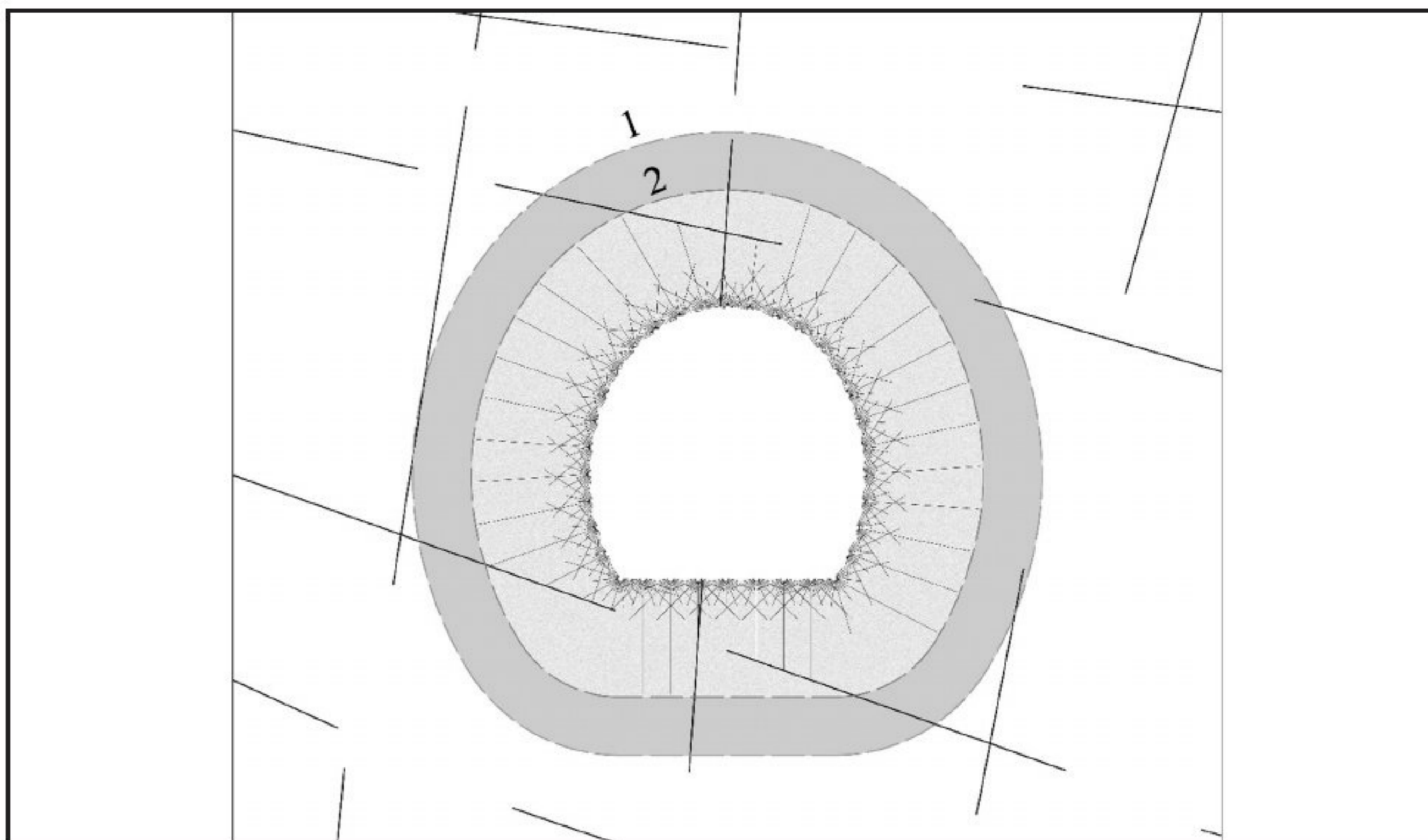


Figure 4
Blasting in rock mass where primary block sizes are larger than the maximum length of the blast induced radial cracks. Line 1 marks maximum length from contour blastholes to preexisting rock joints. Line 2 marks maximum length of blast induced radial cracks in monolith block.

The second situation is when the primary block size is smaller than the maximum length of blast induced radial cracks. In this case, pressure wave propagation is being limited by pre-

existing joints in rock mass, and therefore blast-induced radial cracks are limited in their length by preexisting joints, Figure 5. It is obvious that more jointed rock masses are less subjected to blast

induced damage and vice versa. The size of the blast damaged zone, in this case, depends primarily on maximum distance between preexisting joints in rock mass, or primary block size.



Figure 5
Blasting in rock mass where primary block sizes are smaller than the maximum length of the blast-induced radial cracks. Line 1 marks maximum length from contour blastholes to preexisting rock joints. Line 2 marks maximum length of blast induced radial cracks in monolith block. As can be seen, preexisting joints in rock mass are limiting radial crack propagation and therefore the blast damaged zone around the excavation has a smaller extent.

3.2 Quantification of the rock mass properties in the blast-damaged zones

Rock mass is presented as a set of monolith blocks intersected by joints and the first parameter that described the blocky structure of rock mass was the Geological Strength Index (GSI) (Hoek 1994). GSI describes rock mass based on the number of joint sets and the quality of the joint walls and is incorporated in the Hoek-Brown failure criterion (Hoek and Brown 1988; Hoek *et al.* 2002). Explosive detonation creates tension cracks in monolith blocks in the rock mass. The rock breakage by explosives theory (Torbica and Lapčević 2014) made it possible to predict blast-induced radial crack length and density around the blasthole. Explosive detonation in

contour blastholes creates the shape of excavation, but inevitably creates radial tension cracks in surrounding rock mass and damages it. Therefore, it is clear that rock mass around the excavation becomes more jointed. Said in other words, the zone around the excavation has a decreased GSI value when compared to the undisturbed rock mass. Using this principle, it is possible to determine strength parameters of rock mass in the blast damaged zone by choosing the proper GSI value for its quantification. Since rock mass in a blast damaged zone becomes more jointed, the GSI value for this part of rock mass should be chosen from the adjacent row below

the GSI value for undisturbed rock mass. This corresponds to the fact that rock mass in the blast damaged zone has one joint set more (blast-induced cracks) than that of the undisturbed rock mass. In this manner, the GSI value of undisturbed rock mass is being decreased for 10 points. Quality of surface walls for blast-induced radial cracks will not be decreased in comparison with preexisting joints and therefore the GSI value should not be decreased regarding this criterion. Figure 6 illustrates the selection of the GSI value for blast damaged zone and equation (16) shows how this value can be calculated.

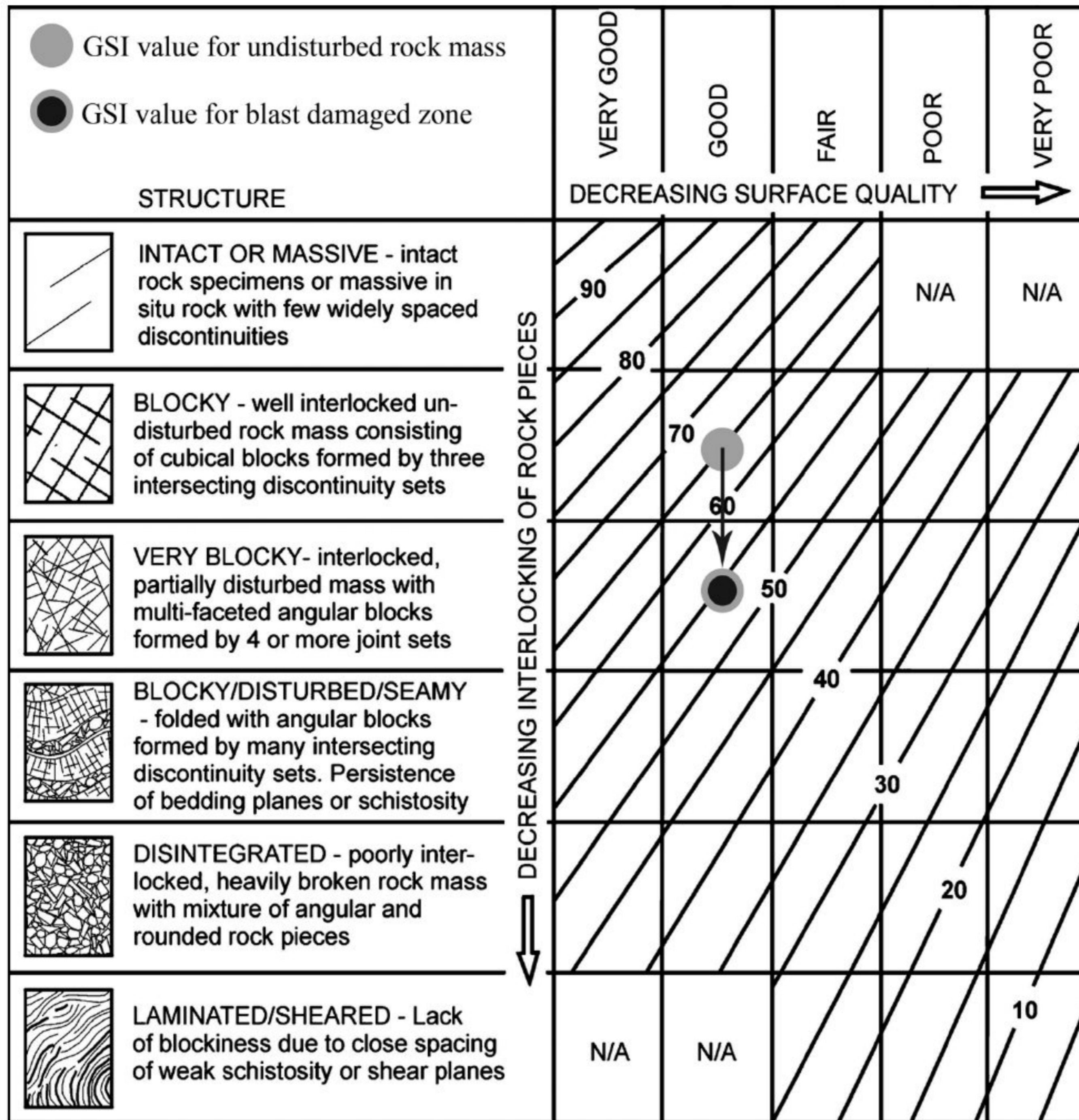


Figure 6
Choosing GSI value
for blast damaged zone.

$$GSI_{bdz} = GSI_{urm} - 10 \tag{16}$$

where: GSI_{bdz} – GSI value in blast damaged zone

GSI_{urm} – GSI value of undisturbed rock mass

4. Comparison between the modeled rock mass, with and without the damaged zones

To describe and show the practical usage of the proposed method of blast-induced damage quantification, a typical tunneling situation was modeled using FEM and Phase2 software (Rocscience Inc.) for modeling. Table 2 presents the monolith and rock mass parameters (Hoek-Brown failure criterion) of the

undisturbed rock mass and blast-damaged zones. The field stress components used in these examples are equal to 10 MPa ($\sigma_1 = \sigma_3 = \sigma_z$). Figure 7 shows two models, one that is used for analysis of the tunnel excavation without the blast-damaged zone and another for analysis with the blast-damaged zone ($r_4 = 2.16m$

= radius of damaged zone). Each model has the same support system installed and analyzed. This system consisted of 5 cm thick concrete ($\sigma_c = 40MPa$, $\sigma_t = 3MPa$, $E = 30GPa$, Poisson ratio $\nu = 0.15$) and fully bonded bolts with 1.6 m length ($D = 19mm$, $E = 200GPa$, and Tensile capacity of 0.072 MN).

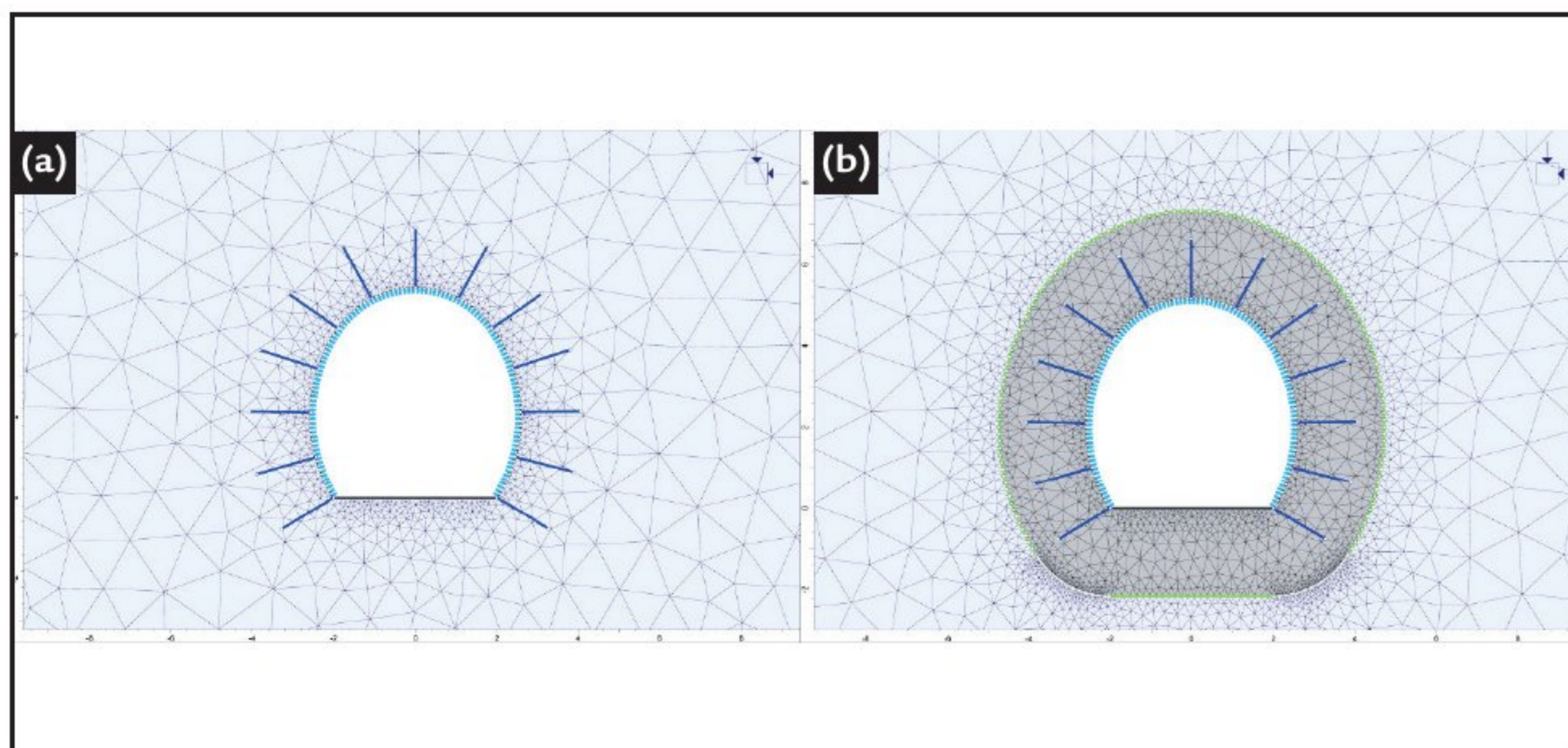


Figure 7
FEM models used for analysis
a) without
b) with blast-damaged zone

Table 2
Rock mass and monolith properties used for FEM models

Zone description	Rock type	Failure criterion	σ_c (MPa)	σ_t (MPa)	Poisson ratio	GSI	m_i	m_b	s	Em (MPa)
Undisturbed rock	Limestone	Hoek-Brown	70	7	0.3	65	10	2.865	0.0205	19840
Damaged zone r_4	Limestone	Hoek-Brown	70	7	0.3	55	10	2.0046	0.00673795	11160

Figure 8 presents the support capacity plots for the two models. Figure 9 presents a comparison between the maximum axial forces in the bolts with

the marked tensile capacity for the two models. From these, it is obvious that in the case where blast-damaged zones are included in the analysis, the

support system is subjected to higher loading which leads to safety factor decrease. Table 3 presents the maximum support loads.

Table 3
Maximum support loading

Model	Concrete			Bolts
	Bending moment (MNm)	Thrust N (MN)	Shear force (MN)	Axial force (MN)
Model without blast-damaged zone	0.0008	1.11	0.007	0.039
Model with blast-damaged zones	0.0007	1.52	0.006	0.057

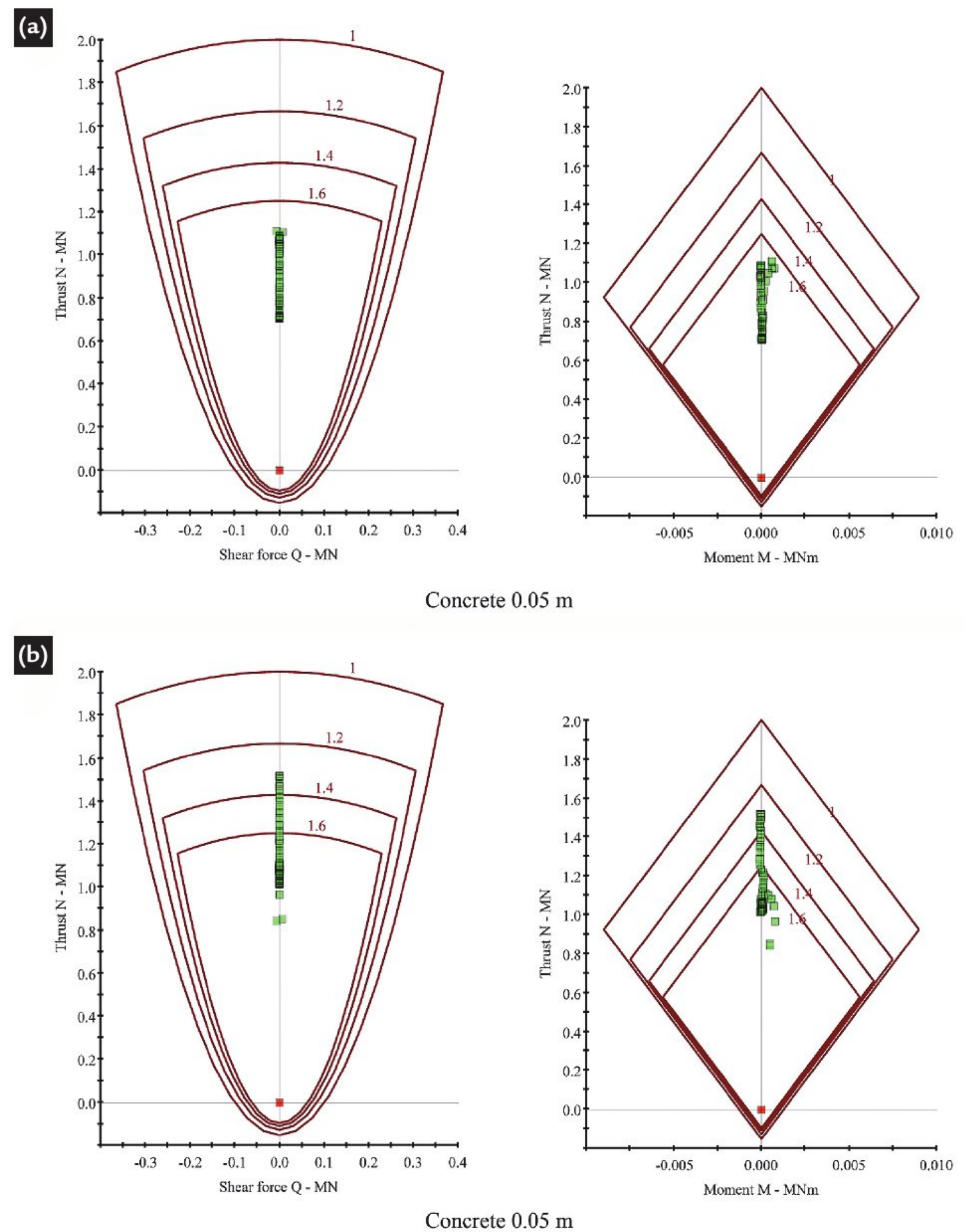


Figure 8
Support capacity plots –
a) model without blast-damaged zones
b) model with blast-damaged zone

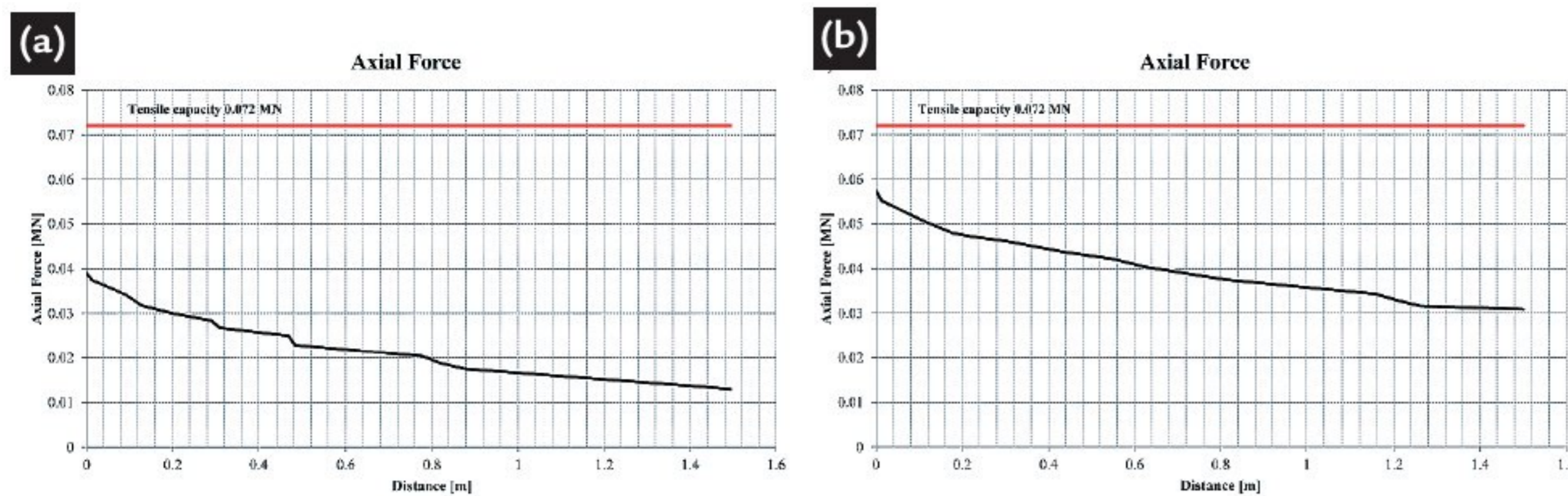


Figure 9
Maximum axial forces induced along bolts
a) model without blast-damaged zone
b) model with blast-damaged zone

5. Comparison between the proposed method for blast-damaged zone quantification and quantification using the blast damage factor D

As mentioned by Hoek (2012), in the case of poor blasting, it can be expected that the blast-induced damage in the surrounding rock mass extends 2-3m. Kwon *et al.* (2009) reports values in the same range. To quantify damage in the excavation walls, Hoek introduced the blast damage factor D and incorporated it into Hoek-Brown's failure criterion. If blasting is poorly done, it is suggested that $D = 0.8$. It has to be mentioned that this suggestion does not explain what is poor blasting and how to determine whether the used blasting pattern will give poor or good results.

So, it is possible to describe the blasting process as poor only if it is already done. But, how to quantify and describe blasting quality before it is done?

We define poor blasting as blasting where contour blastholes are drilled with a smaller span between them and charged with the same amount of explosive as all other blastholes. If the contour blastholes, in the blasting pattern used to create tunnel shape shown in Figure 10, are charged the same as all other blastholes, we have a typical example of poor blasting. To compare these two methods for quantification of blast-induced dam-

age in the rock mass, two FEM models were set up under the same loading and geometry conditions. The first model describes quantification using the blast damage factor D ($D = 0.8$). The damage zone extent is considered to be equal to the extent of zone r_4 (2.16 m). This makes it possible to compare the two models. The second model's analyzed damaged zone was as described in section 4. In Table 4, the rock mass parameters used in these models are shown. In Figure 10, the FEM models used for analyses are shown. The support system is the same as used in the examples of section 4.

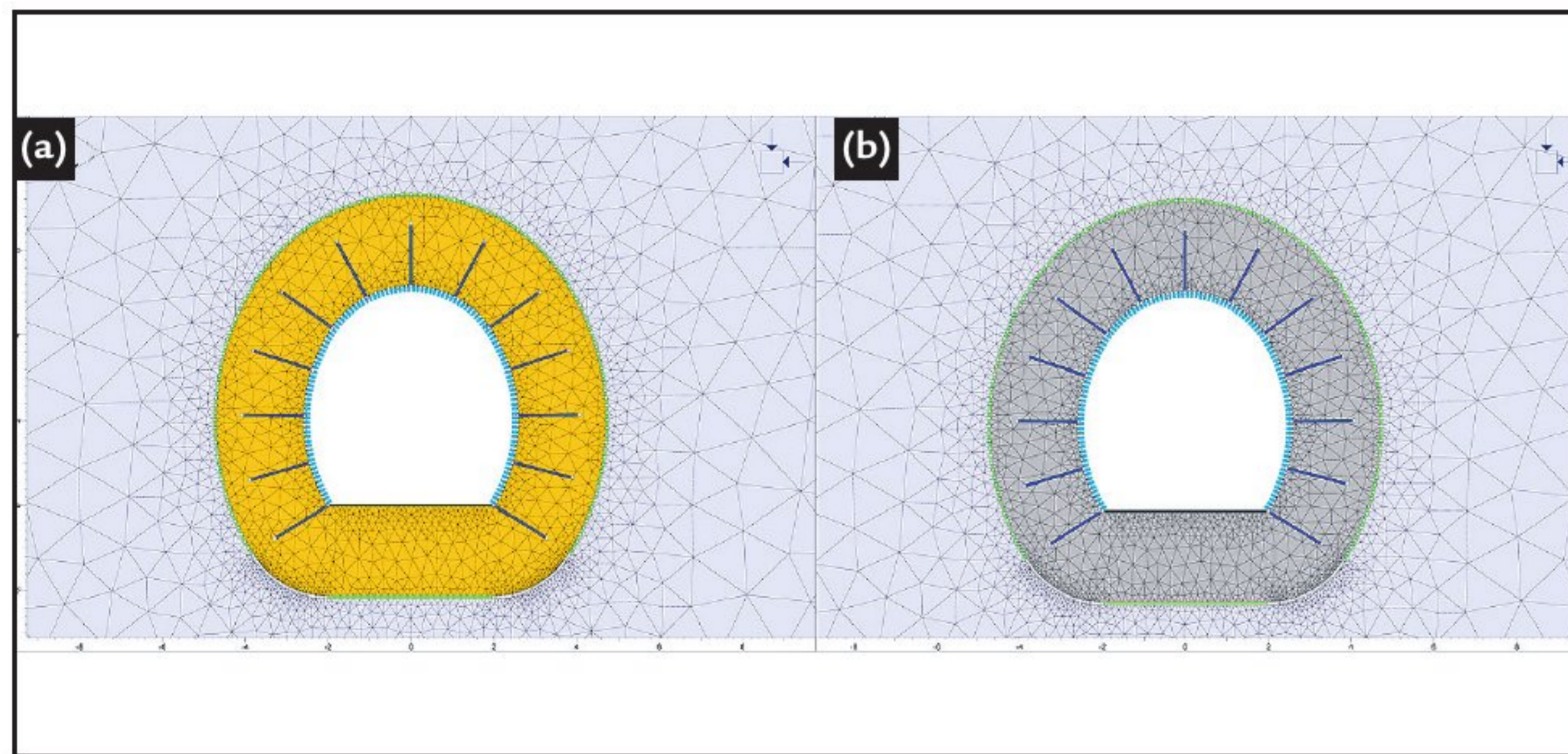


Figure 10
FEM models used for analysis
a) damaged zone quantified using $D=0.8$
b) damaged zone quantified as proposed in this paper

Zone description	Rock type	Failure criterion	σ_c (MPa)	σ_t (MPa)	Poisson ratio	GSI	m_i	m_b	s	E_m (MPa)
Undisturbed rock	Limestone	Hoek-Brown	70	7	0.3	65	10	2.865	0.0205	19840
Damaged zone r_4	Limestone	Hoek-Brown	70	7	0.3	55	10	2.0046	0.00673795	11160
Damaged zone $D=0.8$	Limestone	Hoek-Brown	70	7	0.3	65	10	1.24514	0.00497649	11904

Table 4
Rock mass and monolith properties used for FEM models

Figure 11 presents the support capacity plots for concrete. Figure 12 presents the maximum axial forces along the bolts

for the two models. From these plots, it can be seen that the support loading intensity is similar in both cases. Table

5 presents the maximum loading of the support systems in these cases.

Model	Concrete			Bolts
	Bending moment (MNm)	Thrust N (MN)	Shear force (MN)	Axial force (MN)
Blast-damaged zone quantified using $D=0.8$	0.0008	1.51	0.006	0.066
Blast-damaged zone quantified as proposed in this paper	0.0006	1.46	0.005	0.056

Table 5
Maximum support loading

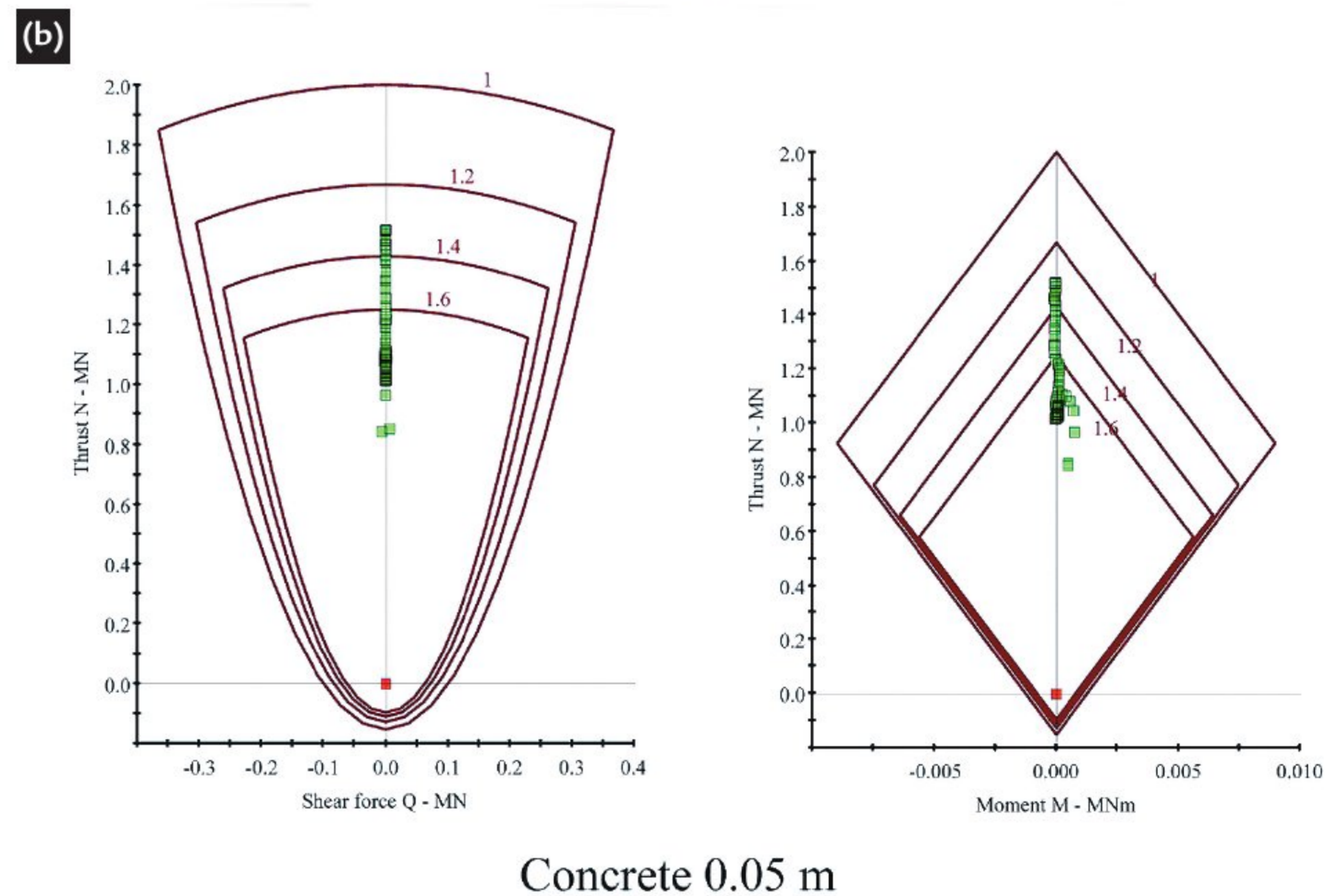
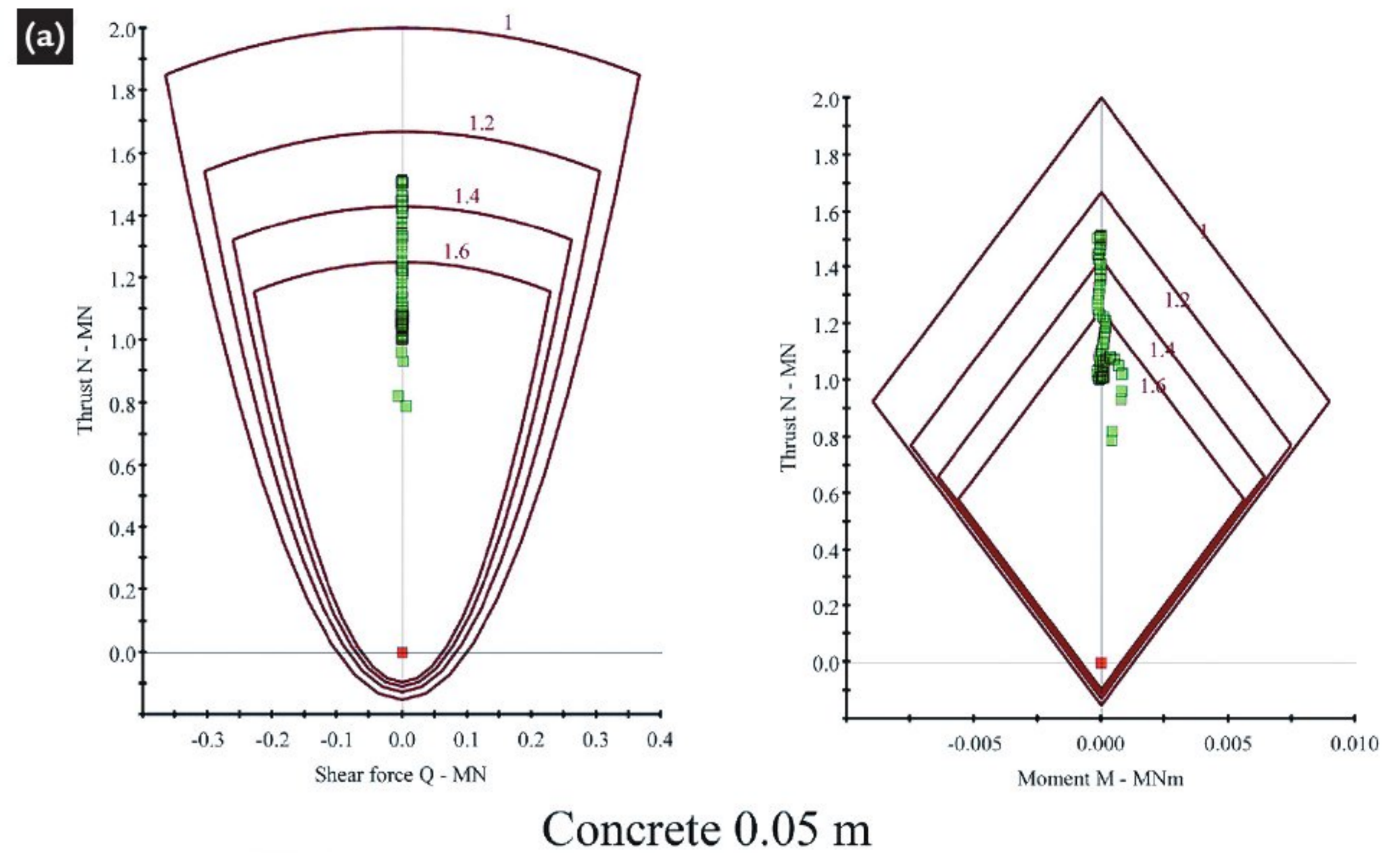


Figure 11
Support capacity plots –
a) model where blast-damaged zone is quantified using $D=0.8$
b) where blast-damaged zone is quantified using method proposed in this paper

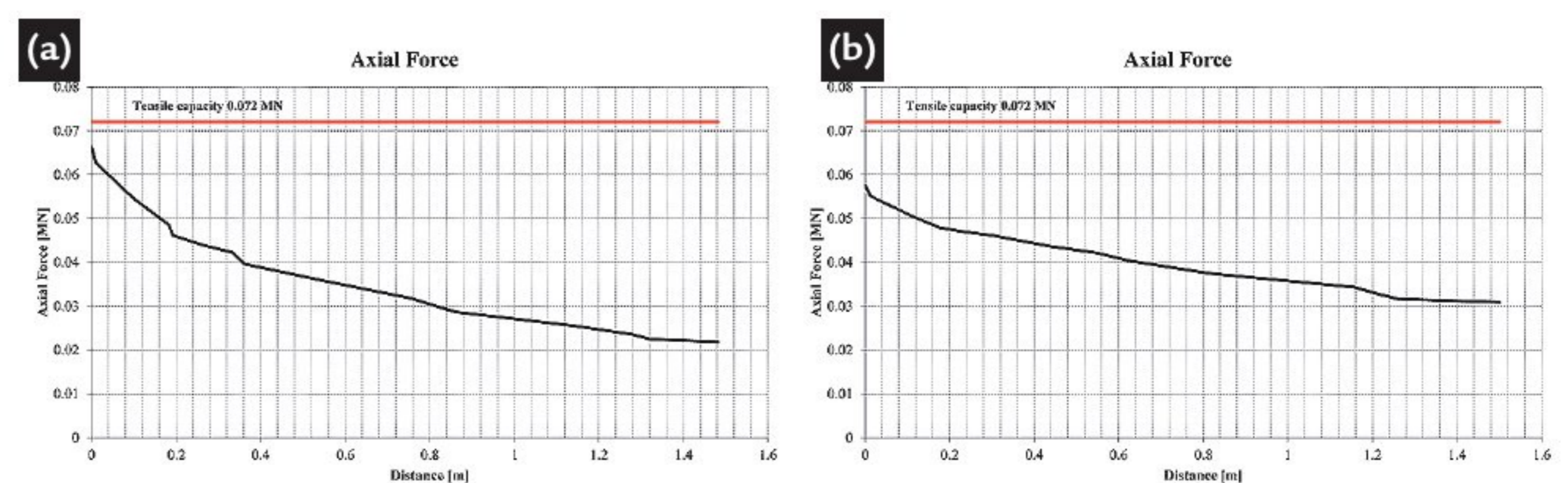


Figure 12
Maximum axial forces induced along bolts a) model where blast-damaged zone is quantified using $D=0.8$
b) where blast-damaged zone is quantified using method proposed in this paper

6. Controlling blast-induced damage

It has been shown that the extent of blast damaged zone (BDZ) depends on several factors, such as pressure on blasthole walls, size of primary blocks in rock mass and strength of monolith

rock. One of these factors that can be controlled is blasthole pressure. Pressure on the blasthole walls can be controlled by changing amount of explosive charge inside the blastholes

or changing radii of explosive charge and blastholes.

According to Torbica and Lapcevic (2014) burden for a blasthole can be calculated by the equation:

$$B = r_4 \cdot \cos 45^\circ \tag{17}$$

Where: B - burden; r_4 – radius of cracking zone according to eqn. (15)

After insertion of equation (15) into (17) for $n=4$:

$$B = \frac{0.17 \cdot P_h \cdot r_h}{k \cdot \sigma_t} \tag{18}$$

Form equation (18) pressure on the blasthole walls, which is the only control-

lable factor, can be expressed as:

$$P_h = \frac{B \cdot k \cdot \sigma_t}{0.17 \cdot r_h} \tag{19}$$

Using equation (19), it is possible to calculate required pressure in order to break burden B that is left for contour blastholes. Therefore, for the borehole radius $r_h = 0.051\text{m}$ in limestone with tensile strength $\sigma_t = 7\text{MPa}$, Poisson's ratio $\nu = 0.3$, the required pressure to break 0.5 m thick burden equals to 543 MPa (~0.5 GPa). At this point it is possible to determine the maximum

extent of blast damaged zone created by contour blastholes by calculating radius of cracking zone r_4 for pressure of 0.5 GPa. For pressure 0.5 GPa using equation (15) radius of cracking zone r_4 equals 0.68 m (~0.7 m) instead of 2.16 m ($P_h = 1.6$ GPa) as shown in Table 1. Figure 13 illustrates how thickness of blastholes burden impacts damaged zone extent. It has to be mentioned

that one should take care about the explosive charge in blastholes that are initiated before the contour blastholes. In fact, the explosive charge in these blastholes may produce much longer radial cracks than those from contour blastholes, if pressure is unreasonably high. Therefore, care should be taken when choosing explosive amount for each blasthole the in blasting pattern.

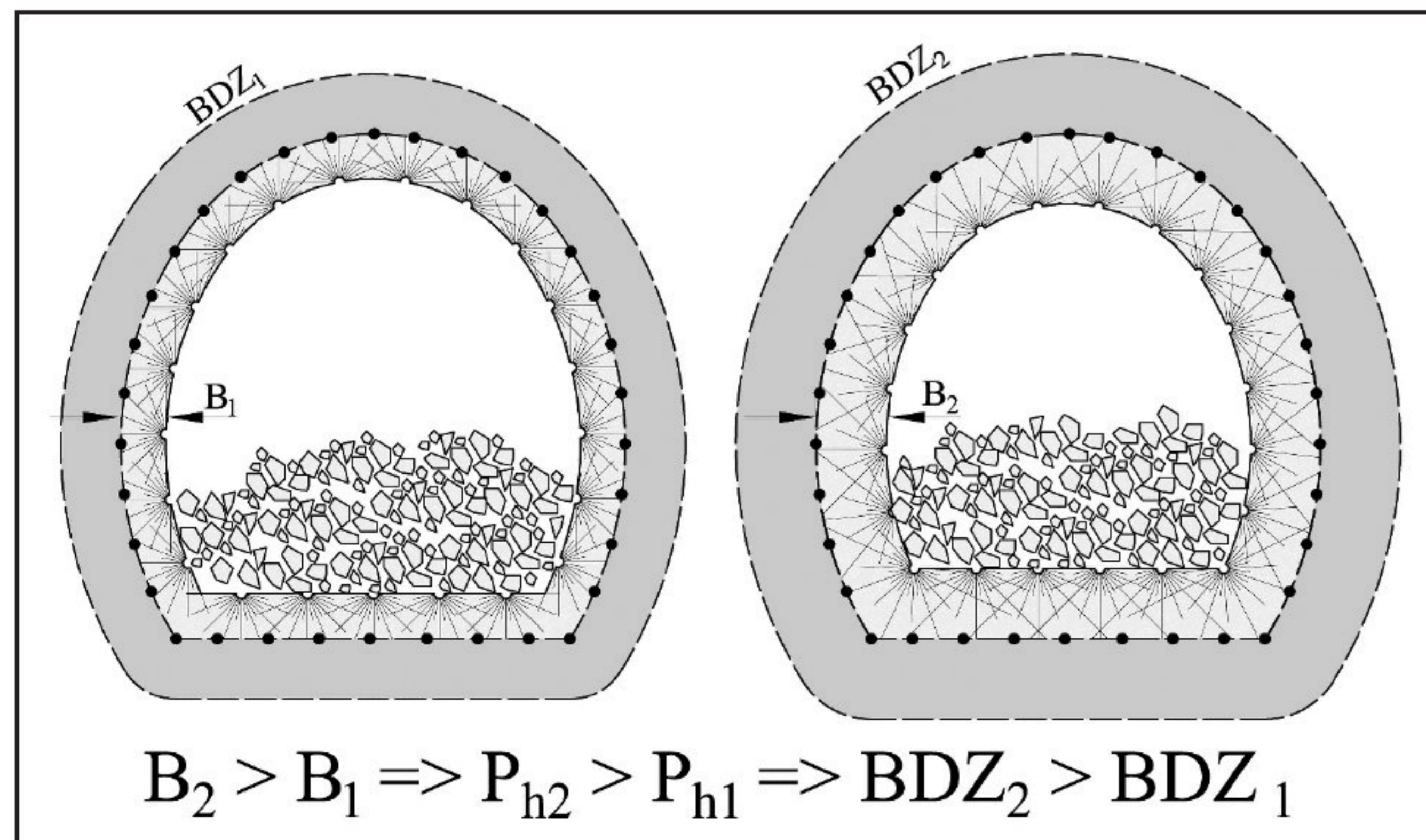


Figure 13
Illustration on how blast damaged zone extent can be controlled. Situation before contour blastholes initiation is shown. B_1 and B_2 are burdens for contour blastholes in two different situations. B_2 is larger than B_1 and therefore higher pressure in contour blastholes is required in order to break thicker burden B_2 . By careful design of blasting pattern, the burden of contour blastholes can be optimized in such manner that pressure in these blastholes is minimized.

According Chapman-Jouguet detonation theory, Chapman (1899) and

Jouguet (1905), pressure on the blasthole walls for explosives with density

above 1 g/cm³ can be calculated as:

$$P_d = \frac{\rho_e \cdot D^2}{8} \tag{20}$$

Where: ρ_e – density of explosive (g/cm³); D – detonation velocity of explosive (km/s).

For explosive with density below 1 g/cm³ pressure on the blasthole walls

is calculated by:

$$P_d = \frac{\rho_e \cdot D^2}{4.5} \quad (21)$$

Equations (20) and (21) assume that blasthole is fully filled with explosive, if

blasthole radius and radius of explosive charge differ or blasthole is not fully filled

with explosive, pressure on the blasthole walls is calculated by:

$$P_h = P_d \cdot \left(\frac{d_e}{d_h} \right)^3 \quad (22)$$

Where: P_h – Pressure on the blasthole walls (GPa), P_d – Detonation pressure

(GPa), d_e – Diameter of explosive charge, d_h – Blasthole diameter

Equation (22) provides a very useful tool for estimating the amount of explo-

sive needed to acquire specific pressure value. From equation (22) we are able to

express diameter of explosive charge as follows:

$$d_e = d_h \cdot \sqrt[3]{\frac{P_h}{P_d}} \quad (23)$$

To achieve pressure of 0.5 GPa on blasthole walls using boreholes with 0.102 m in diameter and explosive

with detonation pressure of 1.6 GPa, diameter of explosive charge should be ~0.069 m which corresponds to

a standardized diameter of 70 mm explosive patron.

7. Discussion

It has to be stated that there is no model for the exact assessment of the extent and quantification of blast-damaged zones. Hoek-Brown's failure criterion has incorporated the blast damage factor D as a parameter that describes the blasting damage in the rock mass. This parameter only describes the blasting process that has been done, but one cannot describe the blasting process beforehand, in the project phase. On the other hand, there is a common misuse of blast damage factor in the way it is applied to the whole rock mass. In this way, one underestimates the whole rock mass properties instead of the limited zone around the excavation. Herein, a method for the precise assessment of blast-damaged zone's shape and extent is proposed, and also a method for the quantification of the rock mass parameters in these zones. With this suggestion, it is possible to define the blasting process as "good" or "bad" before it is done. One can easily assess the damaged zone extent based on the rock and explosive properties and quantify it using the

Hoek-Brown's failure criterion.

Results presented in this paper show that there is an obvious need to differentiate between the undisturbed rock mass and the blast-damaged zones. It has been shown that in the case of modeling the blast-damaged zones, higher support loading occurred as was expected. When comparing the proposed method and quantification using the blast damage factor D , results showed that in both cases similar support loads occurred. The main difference between these methods is in fact that when using the blast damage factor, one has to rely on the supposed extent of the blast-damaged zone being assessed, while the proposed method herein is able to offer a precise assessment of the blast-damaged zone extent. On the other hand, quantification using the blast damage factor is highly subjective, since it is not possible to say whether the blasting pattern that will be used for excavation will provide severe rock mass damage or not. In contrast, the proposed method offers an objective quantification of the

damaged zone in the surrounding rock mass. In addition, the nature of the blast damage factor is empirical and its values are estimated based on reverse analysis. This makes it highly case dependent. The method that is proposed here is based on constitutive relationships that describe rock breakage by explosives.

At the end of this paper, it is explained how blast-induced damage around excavations can be controlled. Using principles described here and relationships that are used to calculate pressure on the blasthole walls, it is possible to determine exact amounts of explosive inside contour blastholes that are necessary to break their burden. It has to be emphasized that it is not the size of the blast damage zone that is crucial for analysis, but the relative size of the blast damage zone size and the size of primary rock blocks. In fact, small primary blocks may even be crushed by small amounts of explosive. Said in other words, with decreasing primary block size impact of blast-induced damage increases.

8. Conclusion

As it is already known, there are plenty of methods for assessment of blast-damaged zones. Also, there is a lack of precise methods for assessing the extent and quantification of these zones.

Many of the existing methods are empirical and highly case dependent, while on the other hand, theoretical methods have limited applicability. An important part of the new rock breakage theory is

presented, making it possible to estimate the length and density of the tension (radial) cracks caused by explosive charge initiation. Based on this theory, the method for the blast-damaged zone

extent and quantification of its rock mass properties has been proposed. It explains how to assess the zone's shape and extent based on the blasting pattern that is used, the main properties of rock mass, and the explosive used. To show the importance for analysis of blast-damaged zones, a typical tunneling situation was modeled using FEM. Analysis showed that in case when blast-damaged zones were included in the model, much

higher support loads occurred, as was expected. Furthermore, a comparison between the quantification of the blast-damaged zones using blast damage factor D and the herein proposed method was done. Results showed that in both cases, the support loads are in the same magnitude order, which also confirms that the proposed blast-damaged zone quantification method is correct. Finally, a discussion between these two methods

for blast-damaged zone quantification has been presented, whereupon it can be concluded that the proposed method is more objective than quantification using the blast damage factor D. Controlling the blast-induced damage around underground excavations is of great importance and here it is explained how pressure on the blasthole walls can be controlled in order to reduce the extent of blast-damaged zone.

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