Study of Rock Truss Bolt Mechanism and its Application in Severe Ground Conditions

A thesis submitted in fulfilment of the requirements for the degree of Master of Engineering

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Declaration

I certify that except where due acknowledgement has been made, the work is that of the author alone; the work has not been submitted previously, in whole or in part, to qualify for any other academic award; the content of the thesis is the result of work which has been carried out since the official commencement date of the approved research program; any editorial work, paid or unpaid, carried out by a third party is acknowledged; and, ethics procedures and guidelines have been followed.

Behrooz Ghabraie
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Abstract

Instability of underground excavations is an ever-present potential threat to safety of personnel and equipment. Further to safety concerns, in the event of failure, profitability may reduce significantly because of loss of time and dilution of the ore, raising the importance of support and reinforcement design in underground excavations both in civil and mining engineering.

The truss bolt reinforcement system has been used in controlling the stability of underground excavations in severe ground conditions and preventing cutter roof failure in layered rocks especially in coal mines. In spite of good application reports, working mechanism of this system is largely unknown and truss bolts are predominantly designed based on past experience and engineering judgement.

In this study, the reinforcing effect of the truss bolt system on an underground excavation in layered rock is studied using non-linear finite element analysis and software package ABAQUS. The behaviour of the rock after installing reinforcement needs to be measured via defining some performance indicators. These indicators would be able to evaluate the effects of a reinforcing system on deformations, loosened area above the roof, failure prevention, horizontal movement of the immediate layer, shear crack propagation, and cutter roof failure of underground excavations. To understand the mecha-

nism of truss bolt system, a comparative study is conducted between three different truss bolt designs. Effects of several design parameters on the performance of the truss bolt are studied. Also, a comparison between the effects of truss bolt and systematic rock bolt on different stability indicators is made to highlight the different mechanism of these two systems.

In practice, site conditions play a vital role in achieving an optimum design for the reinforcement system. To study the effects of position of the bedding planes and thickness of the rock layers, several model configurations have been simulated. By changing the design parameters of truss bolt, effects of thickness of the roof layers are investigated and a number of optimum truss bolt designs for each model configuration are presented.

CHAPTER 1

Introduction

1.1 Human's need and Underground Mining

Since ancient times, man understood his need for raw material to produce shelter, weaponry and other devices to survive the wild nature and achieve a better life condition. Our ancestors could satisfy their early needs like making a shelter by exploring the earth's surface to find pieces of stones and make a home. But the increase in world's population expanded the need for raw materials during the years. The demand for a better life entails more and more raw materials. More and more materials are required for building new structures, scientific developments and even exploring other planets to find new material sources. Yet there is no practical way to import raw materials from outside our planet, the only way to provide enough material is extracting from the earth itself.

Generally, orebodies are not at the surface of the earth but deep inside the crust, especially energy sources like coal, gas and oil. The simplest method to reach the orebody is to remove the overburden material and create an open pit mine (surface mining). But, removing the overburden is not always the most efficient way. An alternative method is to dig into the ground, extract the ore and carry it to surface (underground mining).

Tunnels and shafts are the pathways to reach the orebody in an underground mine. Workers, equipments and fresh air need to transport to stopes. A common concern in any kind of underground excavation is to make it stable for a certain period of time. Providing safety of personnel and equipments is the most important issue after excavating an underground excavation. In addition to safety issues, when failure happens, dilution of ore and rock can affect the profitability of the mining operations (Hoek et al. 1998). Stability of an underground opening can be achieved by installing external support or improving the load-carrying capability of rock near the boundaries of excavation or a combination of both.

1.2 Stability of Underground Excavations and Rock Bolts

Excavating an underground excavation is like removing the reaction forces on the boundary of the opening. This changes the stress distribution around an underground excavation. Depending on the in-situ stress distribution, material properties of the site and presence of geological features, such as bedding planes and faults, instability of a tunnel can happen as rock fall-out, rock slip, roof deflection, wall convergence, floor heave, etc. The simplest solution to overcome these problems is to design a support system, which can be installed on the inner boundary of the tunnel and has a load bearing capacity equal to the imposed load on the tunnel's boundary. For a long time, support systems such as timber and steel sets, have been designed to carry the dead weight of the overburden rock above the tunnel.

The concept of reinforcement has been brought to mining engineering in 1913 by the request of a technical patent to German authorities (Kovári 2003). Reinforcement is to improve the strength and increase the load carrying capability of rock mass from within the rock by installing rock bolts, cable bolts, ground anchors, etc (Brady and Brown 2005). During 1970s rock reinforcement techniques, especially rock bolts, experienced a very fast growth in use and nowadays rock bolts are widely used to reinforce underground excavations (Bobet and Einstein 2011). The wide practice of rock bolts is because of simple and fast installation, being appropriate for various types of rocks and structures, and usage as immediate support after excavations.

1.3 Truss Bolt System

In highly stressed areas and severe ground conditions, especially in response to cutter roof failure in laminated strata and coal mines, conventional rock bolt patterns could be inadequate and risky to use. In these circumstances, Peng and Tang (1984) suggest using a special configuration of rock bolts called Truss Bolt systems. Truss bolt system, in its simplest form, consists of two inclined members at two top corners and a horizontal tension element called tie-rod joining the two bolts on the roof of the opening. A common truss bolt system, known as Birmingham truss, consists of two long cable bolts which are connected at the middle of the roof. Horizontal tension is applied by means of a turnbuckle at the connection point of the cables at the roof and transferring a compression to the rock (Gambrell and Crane 1986). A schematic view of Birmingham truss is shown in Fig. 1.1.

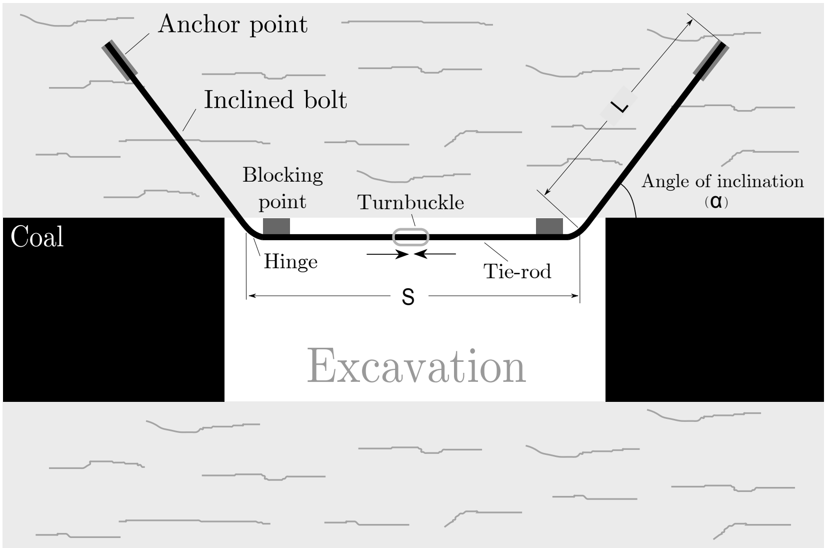


Figure 1.1 Birmingham truss bolt system.

Since the invention of the truss bolt in 1960s, it has demonstrated to be an effective application in practice and has been frequently used by the industry. It has been used in a vast variety of ground conditions from severe to moderate such as poor roof conditions in room-and-pillar mining, long wall road-ways, intersections, and cross-cut entries as permanent support (Cox 2003). These successful applications of truss bolt have led researchers to develop different truss bolt systems which resulted in several patents (White 1969; Khair 1984; Sigmiller and Reeves 1990). Alongside with these developments, several researchers initiated some studies to understand the mechanism of the truss bolt system and publishing a number of practical design schemes. A number of these works has been done by means of photoelastic study during 1970s and 1980s (Gambrell and Haynes 1970; Neall et al. 1977, 1978; Gambrell and Crane 1986). In design schemes for truss bolt systems, just a few number of rational, analytical and empirical design methods are available in the literature (Sheorey et al. 1973; Cox and Cox 1978; Neall et al. 1978; Zhu and Young 1999; Liu et al. 2005). Further to these studies, some field investigation and a small number of numerical analyses can be found in this content (Cox 2003; Seegmiller and Reeves 1990; O'Grady and Fuller 1992; Stankus et al. 1996; Li et al. 1999; Liu et al. 2001; Ghabraie et al. 2012).

Despite these efforts in understanding the truss bolt mechanism, the complicated effects of truss bolts on load distribution around an underground excavation is still largely unknown (Liu et al. 2005). The lack of knowledge forces engineers to consider large safety factors while using these schemes. Understanding the effects of the truss bolt system on reinforcing the rock around an underground excavation is the most important and the first step in obtaining a practical, liable and easy to use design scheme. This project is aimed at understanding the mechanism of truss bolt systems on stability of underground excavations and preventing cutter roof failure. To achieve this, several stability indicators are introduced. Using these indicators, they can evaluate the effects of different parameters of truss bolt pattern and some geological features. The author believes that this study provides the necessary understanding of the mechanism of truss bolt which is a preliminary step to achieve a comprehensive guideline to design a truss bolt pattern.

1.4 Structure of the Thesis

The next chapter introduces the concept of reinforcement and theories behind the design of systematic rock bolt systems. Different elements in truss bolt pattern and a preliminary understanding of the mechanism of truss bolt systems are explained. This chapter addresses the previous research on mechanism of truss bolt, current design techniques and briefly explains the advantages and disadvantages of each design scheme.

To understand the mechanism of truss bolt systems on controlling stability of underground excavations, numerical modelling techniques are used. Numerical models can capture the complicated behaviour of truss bolt system. Once a comprehensive numerical model is established, one can repeat numerous tests for various input parameters at relatively no little cost. The third chapter starts with a brief overview of major numerical modelling techniques. Details of modelling an underground excavation and truss bolt system in a layered rock strata (a typical coal mine) are explained using Finite Element Modelling technique and ABAQUS software package (ABAQUS 2010). In the end, verification process, sensitivity analysis on the dimension of the model and a reference model for further investigations are presented.

The fourth chapter discusses the mechanism of truss bolt system on controlling stability of an underground excavation and cutter roof failure. Different stability indicators are defined to evaluate the reinforcing effects of the truss bolt system. Using these indicators, one can evaluate the mechanism of a reinforcing system on deformations, loosened area, failure prevention, horizontal movement of the immediate layer, shear crack propagation and cutter roof failure of underground excavations. To illustrate the application of these indicators, a comparative study is conducted between three different truss bolt designs. Effects of each parameter on the mechanism of truss bolt system are discussed. Finally, a preliminary comparison between the effects of truss bolt system and systematic rock bolt on different stability indicators is carried out to capture the differences and similarities in mechanism of these two systems.

Chapter five discusses the effects of changing the thickness of the roof layers on the optimum design of truss bolt system. Several different model configurations are modelled and, using three of the stability indicators, a group of optimum truss bolt designs are presented for each model configuration. In chapter six, conclusions and some recommendations for future investigations are presented.

CHAPTER 2

Literature Review

2.1 Reinforcement and support

The main problem after excavating an underground excavation is to maintain the stability of the excavation for a certain period of time. Failure in meeting this demand is a threat to safety of men and equipment. In addition to safety issues, stability of an underground excavation can be achieved by either installing support and/or reinforcement systems. Support and reinforcement are different instruments with different mechanisms. Brady and Brown (2005) in their book clearly distinguished these two instruments.

> ***Support*** *is the application of a reactive force to the surface of an excavation and includes techniques and devices such as timber, fill, shotcrete, mesh and steel or concrete sets or liners.* ***Reinforcement****, on the other hand, is a means of conserving or improving the overall rock mass properties from within the rock mass by techniques such as rock bolts, cable bolts, ground anchors.*

These definitions highlight the difference in practice and mechanism of the reinforcement and support in underground excavation. For instance, the effect of support in continuous and discontinuous rock material is about the same where by applying load at the surface of the excavation, prevents displacement of the rock fragments. While reinforcement system has different mechanism in discontinuous and continuous rock. In continuous rock material reinforcement increases the strength characteristics of rock by acting in a similar way to reinforced concrete whilst in discontinuous rock reinforcement makes the rock to act as a continuous medium by inhibiting displacements at discontinuities (Hudson and Harrison 1997).

Nowadays reinforcement systems are being used widely in underground excavations and rock bolts are one of the most regular reinforcing devices in this content (Palmström and Stille 2010). Rock bolts can be installed in a short time straight after excavation as both primary and secondary reinforcement. This common practice is because of simple and fast installation, being appropriate for various types of rocks and structures and usage as immediate support after excavations. Several usages of the rock bolts provoke different mechanisms of acting and transferring load to the rock material. Consequently, doing any kind of research in this subject entails a good understanding of the mechanism of rock bolting systems.

2.2 Theories of Rock Bolting

Understanding the mechanism of rock bolts on the surrounding rock was of the concern of the researchers for many years. These efforts resulted in several theories about the mechanism of rock bolts which can be classified into three main categories (Huang et al. 2002): *a*) suspension effect[1]; *b*) improving rock material property, and *c*) beam building effect[2]. Here we briefly explain these theories.

Suspension

One of the most common usages of rock bolts is to stabilize an unstable block. This can be achieved by individual bolts or a number of bolts which are anchored behind the unstable block (Hoek and Brown 1980). This effect is shown in Figure 2.1a.

Improving material property

Similar to concrete, tensile strength of rock, by nature, is low. The solution to increase the tensile strength of concrete is to put reinforcement bars which have high tensile strength in the concrete material. Rock bolts in rock can be considered as steel rods in reinforced concrete which act as tensile elements and increase the tensile strength of rock. Further to this, when a rock bolt passes through a discontinuity, because of the applied compression, it makes the rock to behave similar to continuous rock. This effect is because the compression force applied by rock bolt which tightens up the rock fragments together with increased resistance against sliding on the discontinuity surface. Further to these effects, in case of fully grouted rock bolts, grout increases the cohesion and angle of friction on the plane of weakness which make it more stable (Fig. 2.1b).

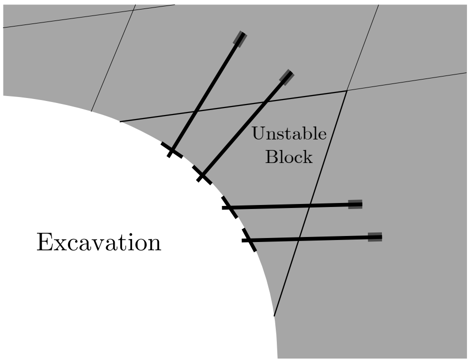
[1]Also known as key bolting

[2]Also known as arch forming effect in curved roof openings

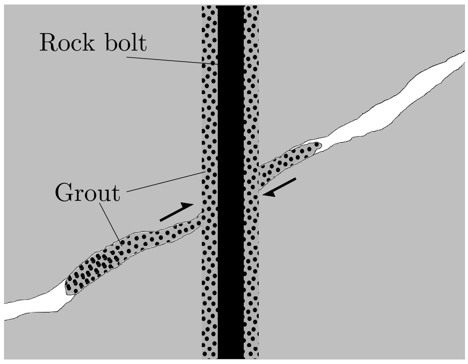
Beam building

> Lang (1961), on the basis of his experience in Australia's Snowy Mountains project, showed a special practice of installing rock bolts in a systematic manner on *an uncoherent crushed rock mass* by a simple experiment. He filled up a rectangular box with fractured rock and compacted to fill the free spaces. After installing rock bolts in position and tightening them up, the material was successfully supported. He did this test on an ordinary household bucket and not only the material was supported, it was able to carry more loads as well. By carrying out several photoelastic analysis on the systematic rock bolt pattern using the material which represented fractured rock material, he reckoned this effect of systematic rock bolt is because of producing a *uniformly compressed area* between the bolts which acts like a beam and can carry the load (Fig. 2.1c). This concept has been further theoretically and experimentally analysed by Lang and Bischoff (1982), Lang and Bischoff (1984) and Bischoff et al. (1992).

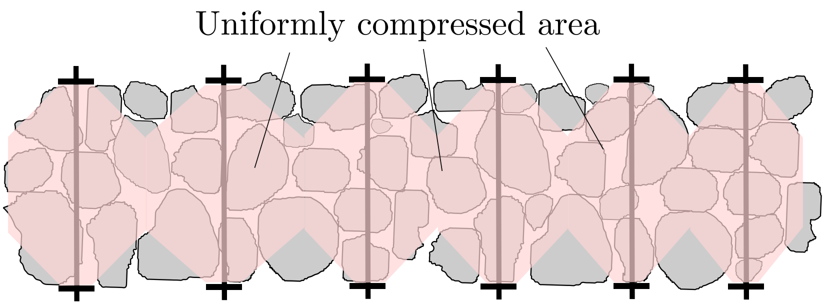
Among these theories the beam building theory is the most proper one as most of present rock bolt patterns are based on the beam building effect of rock bolts (Bischoff et al. 1992; Li 2006). It should be noted that systematic rock bolt pattern improves the rock material properties and suspends individual blocks (prevents from falling) as well as building a reinforced beam so it can be considered as a combination of all of the theories which makes it more complex.



(a) Suspension



(b) Improve material property



(c) Beam building

Figure 2.1 Schematic view of theories of rock bolting

2.3 Truss Bolt System

In highly stressed areas in underground mining, or in poor ground conditions and when fallout is frequent between installed bolts, common bolting patterns are not adequate and usually unsafe. These areas need a more effective and safe support system. Many researchers reported good application of another reinforcement system, named *truss bolt system*, for these areas (Seegmiller and Reeves 1990; Stankus et al. 1996; Cox 2003; Liu et al. 2005). Also, it has been reported that truss bolt systems are more reliable, cost-effective and easy to use in underground excavations (Sheorey et al. 1973; Liu et al. 2005).

Truss bolt system, at the simplest form, consists of two inclined bolts, usually at an angle of 45 degree, and a horizontal element (tie-rod) connecting the heads of the two inclined bolts. These inclined bolts will be anchored above the walls and tensioning force applied horizontally at the middle of the tie rod. As a result, compressive force will be applied on the rock in the area near inclined bolts. To have more space to apply tension and also to prevent penetration of bolts at the hole collar because of applying horizontal tension (especially when cable bolts are being used), normally, two blocks will be used near the connection of tie-rod and inclined bolts (blocking points in Figure 2.2). This system was first introduced by White (1969) as a patent. This design has been improved during the years and the installation procedure become easier (Wahab Khair 1984). As truss bolt systems showed very good application in controlling severe ground conditions, several truss bolt system configurations have been introduced (Seegmiller and Reeves 1990). This development even resulted in production of a truss system suitable for curved roof excavations (Seegmiller 1990).

Generally, truss bolt systems can be categorized into two groups: 1) Birmingham Truss[3]; this truss consists of two cables which will be connected at the middle of the roof, i.e. tie-rod and inclined bolts are not separate, and tension will be applied at the connecting point of the cables (turnbuckle in Fig. 2.2). 2) In-cycle Truss; this truss is a combination of two inclined bolts and a separate tie-rod. The main difference between these two types of truss bolt system is about the time and the way that horizontal tension applies. In

[3]Also known as Classic Truss

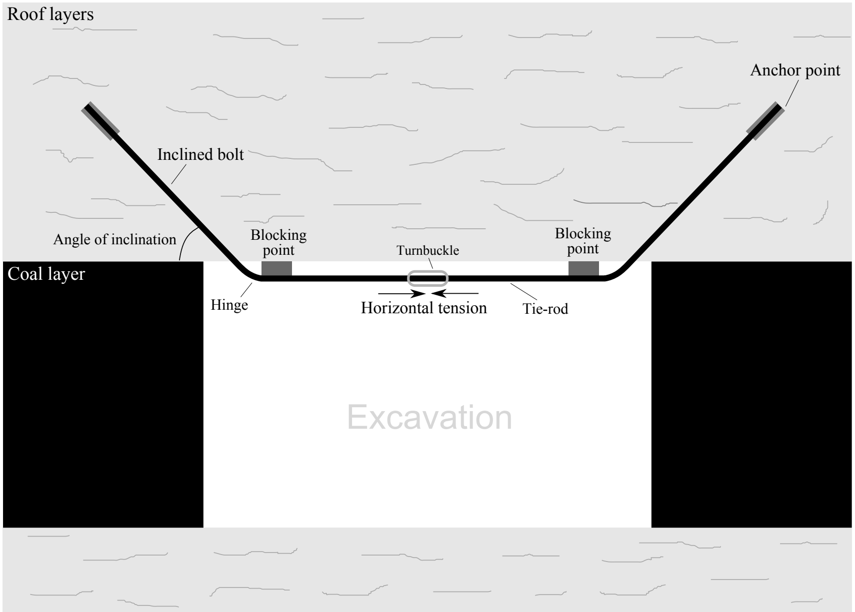


Figure 2.2 Schematic view of Birmingham truss bolt system.

the Birmingham truss the horizontal tension is applied once after installing while in the in-cycle truss, first inclined bolts will be tensioned and after this, tension will be applied on tie-rod (Gambrell and Crane 1986, 1990).

Truss bolt system can be used as either active or passive reinforcement. If inclined bolts are fully grouted and the tie-rod is just attached to them, the system is passive where by increasing deformation in rock, tension increases in truss bolt system. On the other hand, if the inclined bolts are point anchored and pretension applies to tie-rod, the system is active (Wahab Khair 1984). Opinions of researchers in this area are quite contradictory. Cox (2003) believed that after installing end-anchored inclined bolts and tie-rod, a tension should be applied to tie-rod which means the passive installation while O'Grady and Fuller (1992) pointed out that truss system should be installed with end-anchored inclined bolts and in some cases just a small amount of tension should be applied to the tie-rod which means the active installation. These differences in researchers' experience are probably because of changes in the geological features, in-situ stress distribution and site specification of different projects.

2.4 Truss Bolt Mechanism

Figure 2.3 shows a schematic view of the applied load by truss bolt on the surrounding rock. The thing that makes truss bolt totally different from angled bolts is the horizontal tension which is applied at tie-rod. This tension places the roof rock at compression, which is favourable, and reduces the tensile stress at the middle of the entry. By increasing the tension, more roof layers will be placed in compression and the tunnel will be stable (Wahab Khair 1984; Soraya 1984).

In order to understand the effects of truss bolt on the surrounding rock, researchers carried out several photoelastic analysis. Gambrell and Haynes (1970) by comparing angled roof bolts and classic roof truss system concluded that classic truss bolt creates a compression force, with the major axis parallel to the roof of the opening, between the heads of the inclined bolts above the roof of the excavation which is because of horizontal tensioning of the tie-rod. This compressive field, immediately above the roof of the excavation, reduces the excess of the tensile stress which is the main cause of the failure at the mid-span area in lots of cases. Also, as Gambrell and Haynes (1970) reported, diameter and physical characteristics of tie-rod do

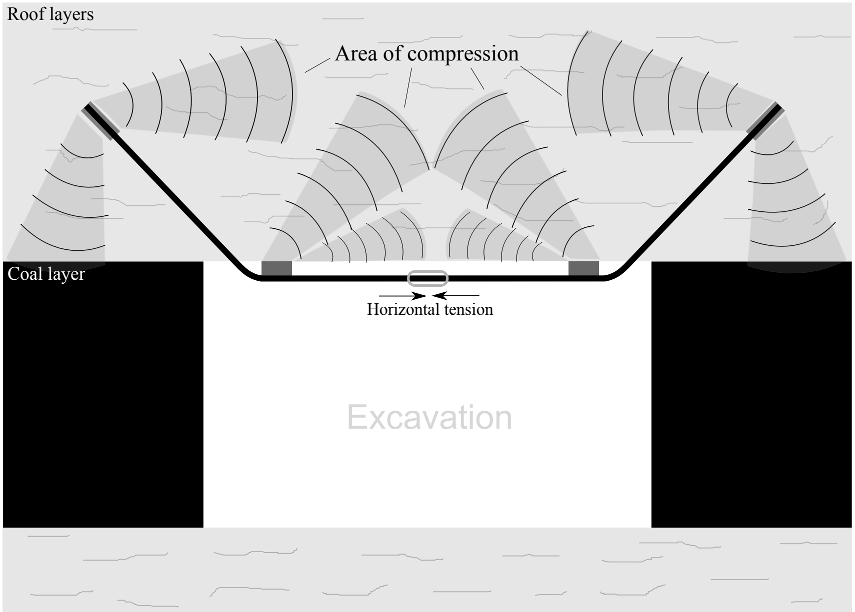


Figure 2.3 Load distribution around truss bolt system (after Wahab Khair (1984)).

not have significant influence on the capacity of truss bolt system. The supporting effect of a small diameter steel rod is about the same as a wide-flange steel beam. This shows that tie-rod element is just to provide the horizontal tension and not a load bearing element.

Neall et al. (1977) by doing photoelastic analysis on the effects of truss bolt in laminated strata model concluded that truss bolt successfully closes the separation of the layers. In addition, Neall et al. (1978) using the same photoelastic model, conducted a research on the load distribution around several truss bolt patterns. Results showed that truss bolt creates a compression field in layers above the roof and reduces the shear stress at the mid-span together with an area above the rib. Their work is more focused on delivering a design procedure and an optimum design which is discussed in Section 2.6.2.

Gambrell and Crane (1986) compared the effects of in-cycle and classic trusses. They concluded that both systems create a compressive area between the heads of the inclined members, however classic truss bolt shows better application in this case. This difference is because of the initial tension of the inclined bolts in in-cycle truss which creates a tensile field at the middle of the roof and as a result less compression after tensioning the horizontal tie-rod. Their models showed that compressive area above the roof in classic truss bolt is similar to a beam in pure bending. After applying simulated in-situ stress on the model, the compressive area reduced and the tension in horizontal tie-rod increased. Also, Gambrell and Crane (1986) concluded that both of the systems create tensile stress at the corners of the roof. This tensile stress is also greater for in-cycle truss bolt system.

It should be noted that rock mass behaviour is different from materials which have been used in photoelastic analysis. This evokes an uncertainty in the results and special care should be considered while using these results (Gambrell and Crane 1986).

Results of the physical modelling of truss bolt system carried out by Wahab Khair (1984) showed that truss bolt controls the roof sag by controlling the tensile stress development in the upper layers and increasing the shearing resistance at the roof of the excavation. In addition, he found that the thickness of the immediate roof changed the effects of truss bolt on the surrounding rock. Thinner immediate roof results in less effect of truss bolt system on the immediate adjacent rock.

In addition to physical and photoelastic analysis, some researchers according to their experience made their comments about the mechanism of truss bolt system. Cox (2003); Cox and Cox (1978) pointed out that truss bolt systems would reinforce the ground by a combination of suspension and reinforced arch building effect. Stankus et al. (1996) examined truss bolt systems in high horizontal stress fields where cutter roof failure was the problem. They reported that high capacity systematic rock bolt would just be able to control high vertical in-situ stress fields but truss bolt systems, because of inclined bolts, successfully control both vertical and horizontal stress fields and abutment pressure together with preventing the shear failure around the rib area. This effectiveness of truss bolt in controlling horizontal displacement of roof is also reported by Seegmiller and Reeves (1990).

2.5 Design of Reinforcement Systems

Design methods of reinforcement systems can be split into several categories based on the rock bolting theory. In this case, designing individual rock bolts to support an unstable block or suspend the roof layers is simpler than designing a systematic rock bolt pattern. In suspension, capacity of rock bolts should be large enough to overcome the weight of the unstable block minus the friction effect on the sliding surface. Figure 2.4 shows an unstable block which would slide towards the opening by its weight. Total required bolt load will be (Brady and Brown 2005)

$$T = \frac{W(F.\sin\psi - \cos\psi\tan\phi) - cA}{\cos\theta\tan\phi + F.\sin\theta}$$

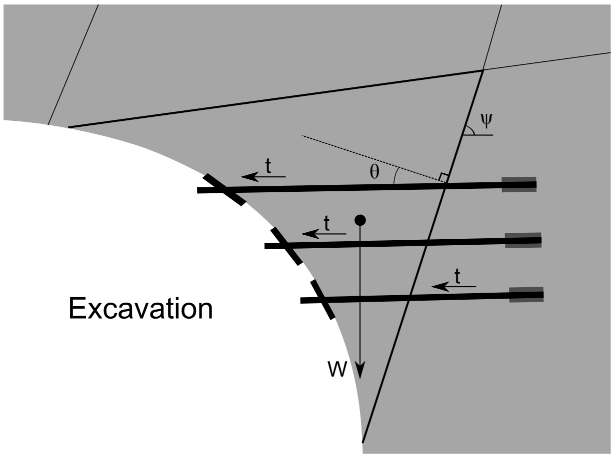


Figure 2.4 Reinforcing a potentially unstable block, $T = \Sigma t$ (adapted from Hoek and Brown (1980)).

where W is the weight of the wedge, T is the load in the bolts, A is the area of sliding surface, ψ is the dip of the sliding surface, θ is the angle between the rock bolt and normal to the sliding surface, c and ϕ are respectively the cohesion and angle of friction of the sliding surface and F is factor of safety. Depending on the damage that sliding would result and grouting condition, a desired factor of safety (usually 1.5 to 2) should be used (Hoek and Brown 1980).

The required load can be applied by number of bolts with respect to capacity of each bolt. This solution can be used to have a first determination of the required number and capacity of bolts. To have a more comprehensive design, other factors should be taken into account, e.g. the wedging action between two planes (Hoek and Brown 1980).

A more comprehensive design of reinforcement systems is to design the systematic rock bolt pattern. The systematic rock bolt design should be based on several parameters such as length and spacing of rock bolts, capacity of rock bolts, amount of tension (in pretension rock bolts) and type of anchors. Lang (1961), on the basis of his experience, proposed number of recommendations to design and check the systematic rock bolt pattern. These recommendations were based on the minimum requirements of length and spacing of the rock bolts. Minimum length of rock bolts should be the greatest of the following (Hoek and Brown 1980):

(a) Twice the bolt spacing.

(b) Three times the width of critical and potentially unstable rock blocks defined by average joint spacing in the rock mass.

(c) For spans of less than 6 meters, bolt length of one half of the span, for spans of 18 to 30 meters, bolt length of one quarter of span in roof and for excavations higher than 18 meters, sidewall bolts one fifth of wall height.

And, minimum spacing of rock bolts should be the least of:

(a) One half the bolt length.

(b) One and one half times the width of critical and potentially unstable rock blocks defined by the average joint spacing in the rock mass.

(c) When weldmesh or chain-link mesh is to be used, bolt spacing of more than 2 meters makes attachment of the mesh difficult (but not impossible).

Further to these recommendations, Barton et al. (1974) proposed a design scheme for reinforcement systems based on the tunnelling quality index,

Q. Excavation support ratio (ESR) and span of the opening are the other parameters in this scheme (Fig. 2.5). These empirical design procedures are based on a number of experience and investigations in different ground conditions. However, properties of adjacent rock and design conditions for any underground excavation, which is a unique characteristic of any project, would differ from case studies that were used for developing the recommendations (Brady and Brown 2005). This is why Hoek and Brown (1980) mentioned that these design schemes should be used with special consideration. This can be achieved by using numerical and comprehensive analysis of rock bolt design[4].

2.6 Truss Bolt Design

After the invention of truss bolt systems and observing the good practice of these systems in controlling severe ground conditions, many attempts have been made to publish proper design guidelines for variety of ground conditions. These attempts are based on industrial experience, field observation, static, rational and numerical analysis. Here some of these design guidelines are briefly discussed.

[4]As we are not discussing the comprehensive design of rock bolt systems we will not expand this concept here.

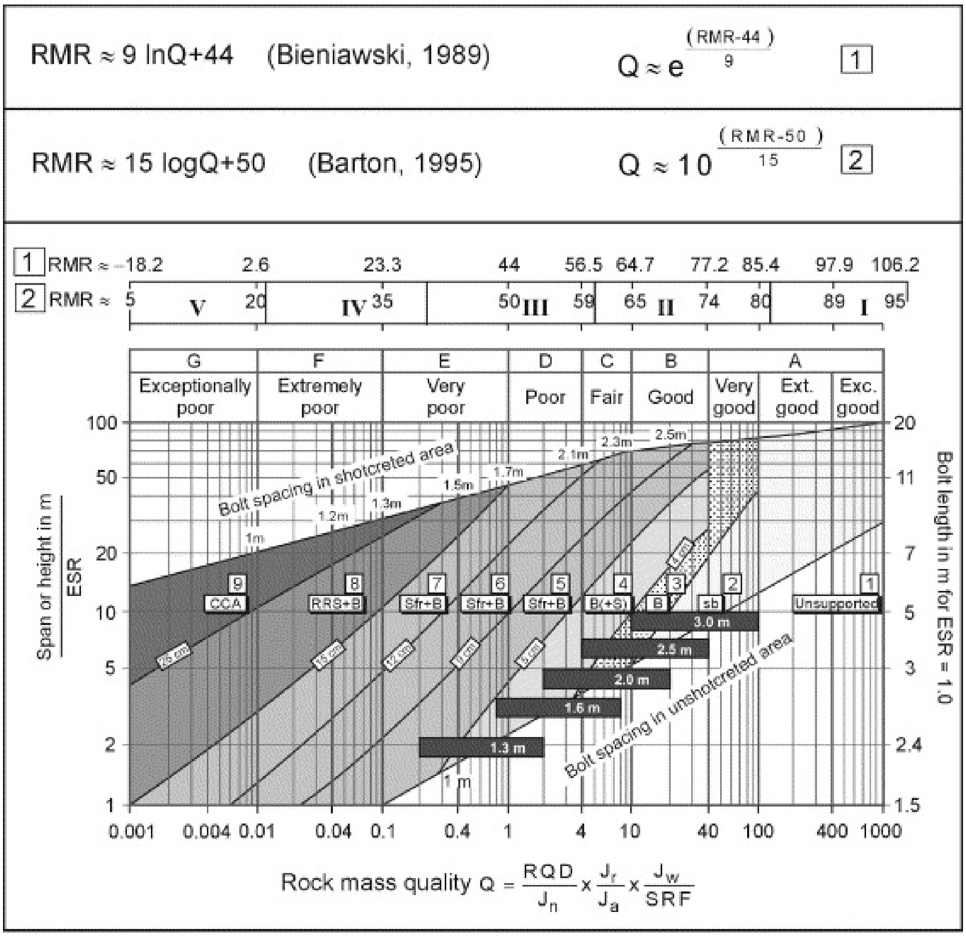


Figure 2.5 Design recommendations for permanent support and reinforcement (after Barton (2002)).

2.6.1 Design Recommendations

Researchers according to their experience and observations in different field conditions proposed several installing procedure and design recommendations. These criteria are based on several parameters of truss bolt system (length and angle of inclined bolts and length of tie-rod). O'Grady and Fuller (1992) and Cox (2003) emphasized the importance of anchoring inclined rock bolts in the safe area above the rib, out of the plastic area. Also, the length of anchorage in safe area should be long enough to make the system capable of carrying dead weight of the loosened area (O'Grady and Fuller 1992).

Wahab Khair (1984) based on his physical model, recommended 45° inclined bolts in comparison with 60°. The reason is that in the results of his investigation, there was not much difference in influence of 45° versus 60° inclined bolts. Angle of inclination equal to 45° would be more cost effective as it can cover a bigger tunnel span and be anchored in a safer area with the same length of inclined bolts. 45° inclined bolts are also recommended by Cox (2003).

Another design factor which proposed by O'Grady and Fuller (1992) is *stiffness* which basically can be defined by the free (unbonded) length of the inclined bolts. This parameter specifies the amount of roof deformation which develops adequate load in truss bolt system to prevent further deformation. Pullout capacity of inclined bolts together with the position of the collars (collars' position specifies the amount of deformation at the head of the inclined bolts) are other factors which control the stiffness of the system.

The importance of installation procedure of truss bolt systems is also emphasised by Cox (2003). He believed that small number of the observed truss failures were due to the failure in anchoring the inclined bolts out of the rib line or improper installation of the system. Consequently, he proposed an installation and design guideline to properly install the system. In this scheme, he mentioned that the length of tie-rod should be one fifth of the entry span, angle of the inclined bolts should be 45° and the length of the inclined bolts should be at least 1.4 times the distance from the walls plus length of the anchorage (0.6 to 1m). This length is to place the whole length of anchorage out of the rib line.

2.6.2 Rational and Analytical Design Schemes

Sheorey et al. (1973) statically analysed the load distribution around the collar head and blocking point of the truss bolt. They considered the reaction forces of rock at borehole head (R_1 and R_2) alongside with friction effect on the blocking point ($\bar{R}_2$) to understand the effective parameters which control the load distribution (Fig. 2.6). These controlling parameters are angle of inclination (α), thickness of the blocking point (b) and the distance between blocking point and borehole (l). These variables can be calculated as (Sheorey et al. 1973)

$$
\begin{aligned}
P &= \frac{T}{\mu b + a + l}\big((a+1)\cos\alpha + b\cos\alpha\big) \\
R_1 &= \frac{T}{\mu b + a + l}\big((a+1)\cos\alpha - b\cos\alpha\big) \\
R_2 &= \frac{T.b}{\mu b + a + l} \\
\bar{R}_2 &= \frac{T.b}{\mu b + a + l}(\sqrt{1-\mu^2})
\end{aligned}
$$

By parametric analysis of the variables in these equations, they proposed a couple of recommendations for choosing the design properties of truss bolt which would result in maximum reaction force of the system. These recommendations are *a*) angle of inclination of 60° would be the optimum angle and it should not be less than 45° and *b*) the optimum thickness of blocking points and distance of the blocking points from the borehole (with respect to

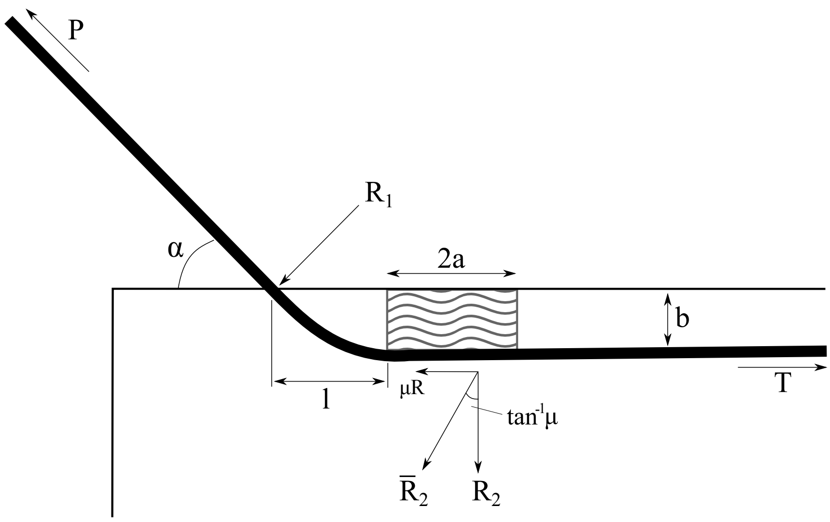


Figure 2.6 Load distribution around blocking point (after Sheorey et al. (1973)).

block width of $2a = 20$ cm) are shown in Table 2.1. These variables depend on the length of the tie-rod or hole to hole span.

Truss bolt systems, like rock bolts, can be designed with respect to the theories of rock bolting. Cox and Cox (1978) used suspension and reinforced arch theories to calculate the design parameters of truss bolt. Equation 2.2 shows the required tension (T) to suspend the weight of the loosened area

Table 2.1 Optimum tie-rod length values corresponding to block width of $2a = 20$ cm (Sheorey et al. 1973).

Tie-rod length (m)	b (cm)	l (cm)
2.6	8	20-22
3.0	8	20-22
3.6	10	25-30

(W in Eq. 2.1) above a tunnel with span of L.

$$W = \gamma h L b \tag{2.1}$$

$$T = \frac{W}{2\sin\alpha} \tag{2.2}$$

where, h is the height of the rock fall, b is spacing of the truss systems, γ is the unit weight of the rock and α is the angle of inclination of inclined bolts. In Equation 2.2 the required tension should be equal to the weight of the loosened area to successfully support the roof. This amount of tension is usually much higher than the required tension to stabilize an underground excavation. This value can be used as the upper limit for the design purpose. On the other hand, Cox and Cox (1978) proposed another design scheme which is based on the reinforced arch theory of rock bolts. In this design it has been assumed that truss bolt system creates a reinforced arch like systematic rock bolt systems which can carry the load. In this scheme, the horizontal and vertical reactions of the rock load (weight of the loosened rock) in the roof truss reinforced arch (H_t) and the abutment (V_t) can be calculated as

$$H_t = \frac{\gamma h L^2}{8Z} - \frac{T}{bZ}\big(\frac{L}{5}\sin\alpha + (\frac{t-Z}{2})(1-\cos\alpha)\big) - \frac{T}{b}\sin\alpha \tag{2.3}$$

$$V_t = \frac{\gamma h L}{2} - \frac{T}{b}\sin\alpha \tag{2.4}$$

where γ, h, L, T, b and α are the same as in Equations 2.1 and 2.2 and Z is the rise of the rock arch axis (typically $Z = \frac{3}{4}t$ where t is the thickness of the arch). The performance of the rock arch depends on several parameters such as unity of the arch, compressive strength of the rock material, shear strength of the rock at abutments and deformation parameters of rock. They also compared the resultant reaction values (Eq. 2.3 and 2.4) of two typical truss systems with normal roof bolting patterns and inclined bolts. This led them to the conclusion that truss bolt systems are much more successful in controlling roof loads which cause failure.

Neall et al. (1978) used the beam theory to theoretically analyse the effect of truss bolt on a beam roof layer which is under the tabular overburden load. They used the superposition technique to add the different load components of the truss bolt which act on the roof layer. They added four different load components of truss bolt and rock load which are *a*) tabular loading that is the weight of the overburden layers, *b*) equal symmetric loads which apply vertically at the blocking points (vertical components of the applied load at blocking points), *c*) axial loads which are the result of the horizontal load component at the blocking point and *d*) moment which is due to the applied horizontal load at blocking points that act at a distance from the neutral axis. Then they calculated the resultant stresses of truss loads (items *b*, *c* and *d*) where should be equal to the overburden load (item *a*), so (Neall et al. 1978)

$$w = \frac{24T}{SL^2}\left(\frac{2t(1-\cos\theta)}{3} + \frac{a^2\sin\theta}{L}\right) \tag{2.5}$$

where, w is the tabular load per unit, T is truss tension, S is truss spacing, L is beam length, t is thickness of the roof layer, θ is the angle of inclination and a is the distance of blocking point to the wall of the excavation.

By differentiating the Equation 2.5 with respect to θ and solving for zero, the extremum points, if any exists, can be found. For a given condition, a maximum point represents the optimum angle of inclination (Eq. 2.6).

$$
\begin{aligned}
0 &= \frac{2t\sin\theta}{3} + \frac{a^2\cos\theta}{L} \\
\tan\theta &= -\frac{3a^2}{2tL}
\end{aligned}
\tag{2.6}
$$

They solved this equation for the given parameters in their photoelastic model and came to the conclusion that the optimum angle of inclination would be 90° from roof of the excavation. They modified Equation 2.5 to calculate required tension (T) to eliminate the tensile stress at the bottom line of the roof layer, as a measure for stability, and checked the results with results of photoelastic analysis. They reckon the theoretical results were 15 times greater than the observed values in photoelastic analysis. In this case, they proposed the use of correction factors which depended on the field variables such as thickness of the roof layers.

Neall et al. (1978) also proposed an empirical approach to determine the truss spacing (S) as

$$
S = \frac{C}{W}
$$

where C is truss capacity which is a function of truss tension, immediate layer tickness, angle of inclination, blocking point configuration, truss span, depth of anchor and W is the roof load which is a function of thickness of roof layers, moduli of roof layers, shear strength of roof layers, depth below surface, time factor, residual or tectonic forces, opening width, mine geometry, joint density, joint orientation, fluid or gas pressure, density of roof layers and so on. They also mentioned that if it is possible many of these variables should be eliminated or blended to other variables in order to make it more simple. It should be noted that Neall et al. (1978) emphasised the uncertainty of their theoretical calculation as a result of simplified assumptions at the beginning of the calculation.

Another closed-form design procedure of truss bolt systems was proposed by Zhu and Young (1999), using arching theory (beam building theory) . They believed that their proposed design can be used to calculate and/or check the preliminary values of length of tie-rod and minimum horizontal tension of the system. This design considers the angle of inclination, α, span of the tunnel, L, span of the truss system (length of tie-rod), S, and thickness of the immediate roof layer, h. Assuming that truss bolt reduces the bending stresses at middle and corner of the roof to zero and calculating coefficients of A, B and C as

$$
\begin{aligned}
A &= 4.5\cos\alpha \\
B &= -6(L\cos\alpha + h\sin\alpha) \\
C &= 1.5L^2\cos\alpha + 4Lh\sin\alpha
\end{aligned}
$$

the length of tie-rod in the truss bolt system, S can be derived as Equation 2.7.

$$AS^2 + BS + C = 0 \tag{2.7}$$

As Zhu and Young (1999) suggested, this equation can be used to calculate the length of tie-rod when other design parameters such as angle and length of inclined bolts are usually predefined (angle normally between 38° and 60° and inclined bolts should be long enough to be anchored over the rib). In addition, the minimum tension in truss system should be great enough to create shearing resistance against vertical reaction of the abutment. This tension for a tunnel with weight of roof beam and overburden equal to W can be obtained as

$$T = \frac{WL^3}{\left[12(L-S)^2\cos\alpha + 16(L-S)h\sin\alpha\right] \times \sin\alpha}$$

The factor of safety for an unsupported roof to resist shear failure at abutment can be calculated as (Wright 1973)

$$F_0 = L^2 \tan\phi\big(3.16hL - 1.76h^2\big)$$

Now by comparing the maximum shear resistance and the shear force at abutment, the factor of safety against shearing for a supported tunnel with truss bolt can be derived as

$$F_s = \left\{\left[3\cos\alpha(L^2 - S^2) + 6hS\sin\alpha\right] \times \frac{T\sin\alpha}{LW} - L^2\right\} \times \frac{\tan\phi}{Bh} + F_0$$

where B is the longitudinal truss spacing. Further to this, Zhu and Young (1999) expanded their closed-form solution for a single truss bolt to multiple truss bolt systems which are two separate truss bolt systems that can be installed within each other (two systems with different tie-rod lengths) or one overlapping the other (two systems with different positions and same length of tie-rod). This solution is much the same as single truss bolt design and can be found in Zhu and Young (1999).

The latest available analytical approach to design the truss bolt pattern in the literature has been proposed by Liu et al. (2005). This design is on the basis of three controlling parameters (design principals) as

(a) *Minimum pre-tightening force* which should be adequate to create an arch shape reinforced structure.

(b) *Maximum pre-tightening force* which is to prevent the failure of inclined bolts and failure of rock at abutment and tail of the bolts. The effective parameters in this case are strength parameters of rock, stiffness of the truss bolt system and the contact condition between truss bolt system and the adjacent rock.

(c) *Minimum anchorage force* which can be defined by the length of the anchorage. This anchorage length should be beyond the plastic zone around the tunnel and be greater than pre-tension force and weight of the rock above the roof, below the axes of rock arch.

To analytically calculate these parameters they proposed that inclined bolts apply different load distributions at plastic and elastic rock material around the tunnel (Also, prior to Liu et al. (2005), Li et al. (1999) used this theory and analytically calculated the imposed forces by inclined bolts in a truss system and verified their work with field investigation[5]). The applied

[5]These two works are much the same in this content.

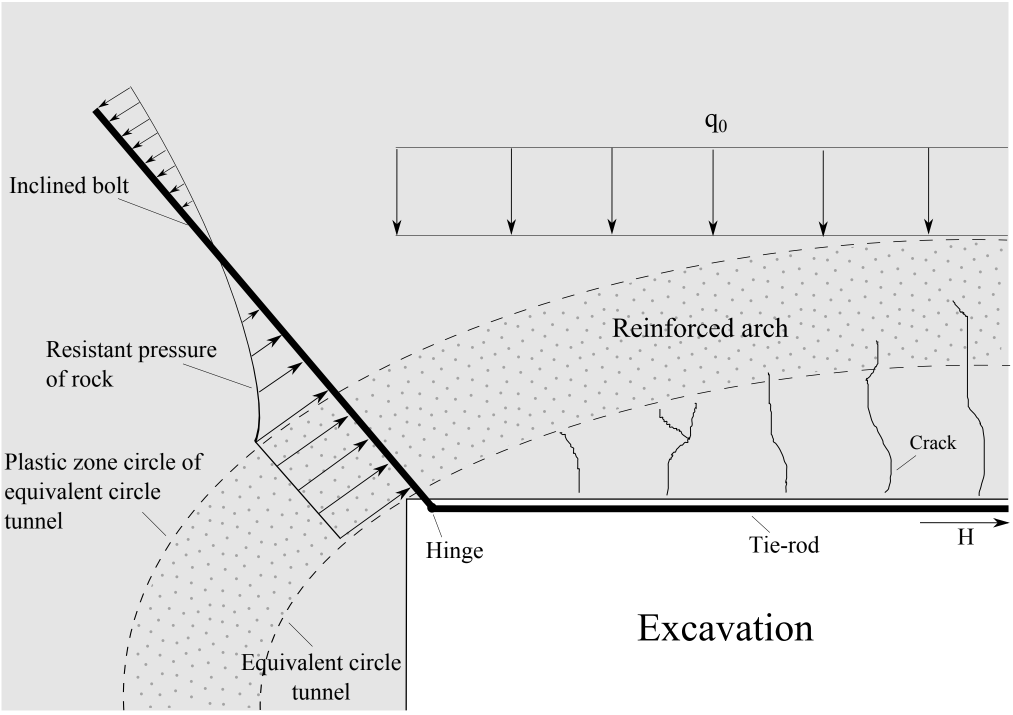


Figure 2.7 Lateral behaviour of inclined bolts and reinforced arch (after Liu et al. (2005)).

load at the plastic zone creates a reinforced arch above the tunnel which reduces the bending moments and tensile stresses in the rock. In this model, the thickness of the reinforced arch is equal to the thickness of the plastic zone around the tunnel (Fig. 2.7). And thickness of this plastic zone has been assumed to be equal to the plastic zone around an equivalent circular tunnel with diameter equal to the diagonal of the rectangular tunnel (Liu et al. 2005).

After deriving the analytical equations, they used the finite element analysis to parametrically analyse the effects of some of the variables on design parameters on the basis of their proposed equations. This part of their analysis has been carried out on effect of depth, angle of friction of rock, shear strength of rock, cohesion, span of the tunnel, truss system spacing, angle of inclination and factor of safety on thickness of the plastic zone around the tunnel and lower and upper pre-tightening force limits. One of their conclusion to this part of their study is that the optimum angle of friction would be between 45° and 75°.

Liu et al. (2005) also published a flowchart that shows the procedure of designing a truss bolt system on the basis of their analytical results. Here, just an overview of this design scheme is explained. The first step of their design procedure is to determine basic parameters of rock, in-situ stress condition and dimension of the opening together with setting up the initial design parameters for truss such as truss system spacing b, inclined bolt length, bolts diameter B, tie-rod diameter and angle of inclination α. Next step is to determine the three design principals and use the proposed upper and lower limits to check the design dimensions and structural parameters. At this stage, the bolt and tie-rod strengths should be checked to prevent their failure. Finally, using trial and error technique, design parameters should be changed and checked to achieve an optimum design and required factor of safety.

As Liu et al. (2005) mentioned, this design scheme estimates the lower bounds of pre-tightening force and axial anchorage forces. In practice, these forces should be between the lower and upper bounds to satisfy the safety concerns. To provide a safe design, they first calculate the minimum required length of anchorage that provides the lower bound of pre-tightening force and lower bound of anchorage. After this, with respect to the upper limit of pre-tightening force, they measured the desired length of anchorage to provide the upper bound of the axial anchorage. Finally, they choose the maximum of these lengths as the anchorage length. The length of inclined bolts should be equal to this length plus the thickness of the plastic zone (to ensure the anchorage in a safe area) plus the accessional length (normally 0.1 to 0.2 m). Furthermore, they mentioned that this length of inclined bolts can be checked by an empirical equation proposed by Lang and Bischoff (1982) as $L = s^{2/3}$ where L is the length of inclined bolts and s is the span of the tunnel.

There are several simplifying assumptions that Liu et al. (2005) have made in their analysis which is worthwhile to be mentioned here. They assumed that truss bolt creates a span-wide reinforced arch shape structure above the tunnel and the arch's thickness is equal to the thickness of the plastic zone. This thickness of the plastic zone around a rectangular tunnel was assumed to be equal to the plastic zone around a hypothetical circular tunnel with radius of half of the diagonal of the actual rectangular tunnel. The length of tie-rod was assumed to be approximately equal to the width of the opening which caused the reinforced arch to cover the whole span. They did not consider the effect of blocking and anchor points, and the arching action of truss system was considered to be just the result of the lateral behaviour of rock bolts at the plastic area. Considering these factors would significantly change the response of truss bolt system. Finally, this design mainly determined the capacity of the truss bolt and length of inclined bolts while parameters like angle of inclination and truss bolt spacing should be chosen by trial and error and the position of inclined bolts (tie-rod length) should be predefined and was not considered at the analysis.

2.6.3 Numerical Analysis

There is a small number of numerical analyses on the behaviour of truss bolt system available in the literature and none of them comprehensively discussed and considered the effects of various parameters of truss bolt on the adjacent rock. Liu et al. (2001) used the finite difference method and FLAC software to model a tunnel and truss bolt system. They investigated the effects of length of tie-rod, angle of inclination, tension and anchorage force on the stability of the excavation. In their model, the material properties of roof, floor rock and coal seam were different while no bedding plane was modelled. Using maximum displacement at the middle of the roof together with the area of the plastic zone, they investigated the effects of truss bolt on stability of the tunnel. They showed that truss bolt system successfully controlled the plastic behaviour of rock around the corners of the roof and reduced the deformation at the middle of the roof. Also, by changing the design parameters, they proposed some recommendations to obtain the optimum values of design parameters for their model as angle of inclination equal to 60°, tie-rod should have a distance of 0.3 meters from the sidewalls and they mentioned that the large amount of tension was not necessary as increasing the tensioning force of truss bolt did not have great influence on the practice of the system. It should be noted that they studied the effects of each parameter by changing them in the model while other parameters were constant in each model.

Ghabraie et al. (2012)[6] used finite element modelling technique and ABAQUS program to model truss bolt system acting on a continuum material model.

[6] This paper is actually a part of this thesis which has been published in the ANZ-2012 conference, Melbourne VIC, Australia and will be more explained in section 4.3.1

They proposed that truss bolt would reduce the area of the loosened rock above the roof by changing the position of the natural roof arch. This is different from the effect of systematic rock bolt which creates a reinforced beam in the loosened area. Using the area of the loosened rock as a measure for stability and practice of the truss bolt system, Ghabraie et al. (2012) changed the design parameters of truss bolt (angle and length of inclined bolts and tie-rod length) and by solving 125 models, proposed a group of optimum patterns. On the basis of these patterns they pointed out several recommendations to choose the truss bolt design parameters as *a*) angle of inclination should be between $45°$ and $75°$, *b*) length of inclined bolts should be more than 80% of the width of the excavation and *c*) tie-rod length should be between half and 70% of the span of the opening. In addition, they highlighted the importance of the anchoring the inclined bolts in the safe area above the rib.

CHAPTER 3
Numerical Analysis

As mentioned in section 2.6, current analytical and rational design procedures for truss bolt systems are based on several simplifying assumptions while empirical designs are based on a number of input parameters, which requires experience in specific project sites. These assumptions and a large number of variables make it necessary for engineers to use these design schemes with large safety factors for several types of problem domains (Neall et al. 1978). Additionally, regular closed form and analytical methods of stress analysis are largely weak in facing discontinuous, inhomogeneous, anisotropy and not-elastic nature of the rock, known as DIANE (Jing 2003). There is no analytical solution for these types of rock. Only very simple conditions of these problems can be solved analytically. Furthermore, when it comes to the interaction of the rock and reinforcement system, this problem becomes even more complex as several different effects of reinforcement systems on the total behaviour of the adjacent rock should be considered.

The influence of a large number of variables on the stability of an underground excavation together with the complex and changing nature of coherent rock material make it hard to understand the mechanism of reinforcement and reach an optimum design with regular analytical design schemes. A more comprehensive and practical solution to solve these complicated problems can be obtained by using numerical analysis (Brady and Brown 2005). Numerical methods would be able to solve these complex interactions under less simplifying assumptions and time and can give us reasonable results. These methods are able to monitor the effective parameters of the rock during excavation and loading procedure which makes engineers able to study the detailed effects of different parameters on the stability of an underground excavation.

In this chapter using a common numerical modelling technique, namely Finite Element Method, we explain the basics of the modelling an underground excavation with several geological features together with the verification and sensitivity analysis of these basic models.

3.1 Current Numerical Techniques

There are a number of classifications for numerical modelling methods on the basis of the nature of these methods. Brady and Brown (2005) categorized the computational and numerical modelling techniques to five main groups as

- Boundary Element Method (BEM),
- Finite Element Method (FEM),
- Finite Difference Method (FDM)
- Distinct Element Method (DEM) and
- Hybrid Methods which are combinations of two different methods (e.g.

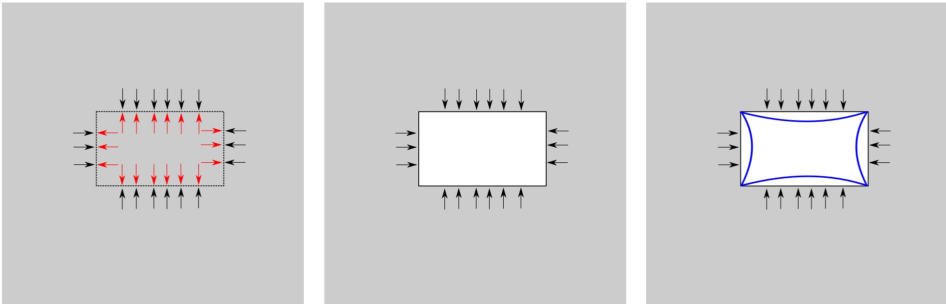
FEM/BEM)

Among these methods, FEM is the most popular and commonly used technique for modelling rock mechanics problems (Jing 2003). FEM was the first numerical method with adequate flexibility to cope with special nature of the rock mass such as discontinuities and anisotropy, inhomogeneous and not-elastic material (DIANE features). Also, the ability to model complex boundary conditions together with moderate efficiency to model discontinuities make it widely applied across rock mechanics problems (Jing and Hudson 2002; Jing 2003). Consequently, FEM has been chosen as the most appropriate method for the scope of this study and ABAQUS (ABAQUS 2010) as a powerful package in dealing with complex soil and rock problems has been chosen as the software package to make a use of FEM.

3.2 Modelling Underground Excavations

3.2.1 Modelling In-situ Stress

Excavating an underground excavation changes the initial stress distribution around the tunnel. In fact, deriving a tunnel is like removing the reaction forces on the boundary of a 'to be driven' tunnel. Before excavating the underground excavation, forces and reaction forces are at equilibrium on the hypothetical boundary of the tunnel (Fig. 3.1a). By removing material, i.e excavating the tunnel, the reaction forces become zero and the equilibrium is no longer valid (Fig. 3.1b). At this stage, a new state of in-situ stress



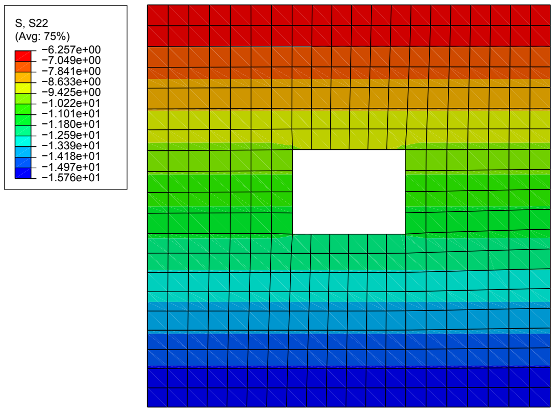
(a) (b) (c)

Figure 3.1 (a) stresses and reactions before excavation, (b) stresses after excavation and (c) deformation after excavation.

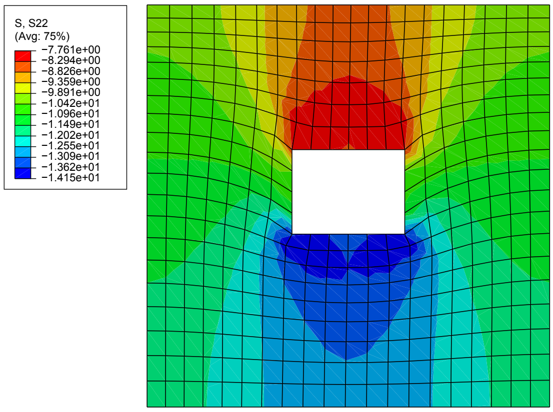
distribution will be applied to the rock around the tunnel. This new stress distribution results in the deformation of surrounding rock and the boundary of the opening (Fig. 3.1c). Shape and amount of this deformation depends on many factors such as magnitude and direction of the new in-situ stress distribution, dimension and shape of the tunnel and physical properties of the rock material.

In numerical analysis, this process can be precisely modelled in three steps (Fig. 3.2). First, in-situ stress applies as initial condition (Fig. 3.2a). At this stage, stresses and reaction forces are at equilibrium at every element and no deformation happens. After this, the tunnel will be excavated by removing some elements from the model while the boundaries of the tunnel are restrained with no deformation on X-Y plane[1] (Fig. 3.2b). At this stage, according to the dimensions of the excavation, a new state of stress distribution, i.e. in-situ stress, will be applied to the tunnel. Finally, by removing

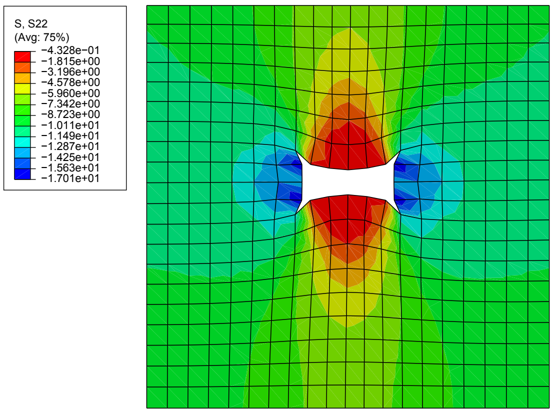
[1]Note that the tunnel is supposed to be very long which can be modelled under plain strain condition.



(a) Step 1



(b) Step 2



(c) Step 3

Figure 3.2 Numerical modelling of an underground excavation (S22 is σ_y).

the boundary conditions from the surface of the opening, deformation happens at the tunnel boundaries and the model reaches to its final condition (Fig. 3.2c). This process can be modelled in at least two steps as steps one and two can be modelled at the same time.

3.2.2 Modelling Rock Material

There are several different models available in the literature for modelling the rock behaviour and several researchers used different models for their own objectives. Among these models, two of them, namely Mohr-Coulomb and Hoek-Brown failure criterion, are considered as classic models which are the most adopted models by researchers (Hoek and Brown 1997; Jing 2003). During the years, these models have been implemented into the FEA and developed to represent several features of rock behaviour (Yingren et al. 1986; Adhikary and Dyskin 1998; Jing 2003). Also, these models would be appropriate to model jointed rock mass. According to Carranza-Torres (2009) and Hoek and Brown (1980), in heavily jointed rock mass, where joints do not have a major orientation and are in several directions, rock mass can be modelled as continuum material with modified or reduced strength parameters that represent the presence of the joints.

Mohr-Coulomb failure criterion is provided by ABAQUS and regularly used to model rock or soil material (Jing 2003; ABAQUS 2010). In this model, by increasing the applied stress, the material undergoes linear elastic deformation to reach the failure point. The expression of the failure in this criterion, on the basis of principal stresses, is

$$\sigma_1 = \sigma_3 \frac{(1+\sin\phi)}{(1-\sin\phi)} + \frac{2c\cos\phi}{1-\sin\phi} \tag{3.1}$$

where σ_1 and σ_3 are respectively the major and minor in-situ stresses, ϕ is the angle of friction and c is the cohesion of the material. By considering the Uniaxial Compressive Strength of the rock mass, σ_c, and k as

$$\sigma_c = \frac{2c\cos\phi}{1-\sin\phi} \tag{3.2}$$

$$k = \frac{(1+\sin\phi)}{(1-\sin\phi)} \tag{3.3}$$

Equation 3.1 changes to the following form (Hoek et al. 1998)

$$\sigma_1 = \sigma_c + k\sigma_3 \tag{3.4}$$

This equation shows that for a given k and σ_c, the amount of major and minor in-situ stresses equal to σ_1 and σ_3 result in failure of the rock material and the linear elastic model is no longer valid. This model does not consider the after failure behaviour of the material. ABAQUS, by considering the non-associated flow rule, models the perfectly plastic behaviour of the material after failure (Fig. 3.3). In this model, after yielding, by increasing deformation, load carrying ability of the rock mass does not change and remains constant.

In fact, Mohr-Coulomb failure criterion can be shown as a straight line.

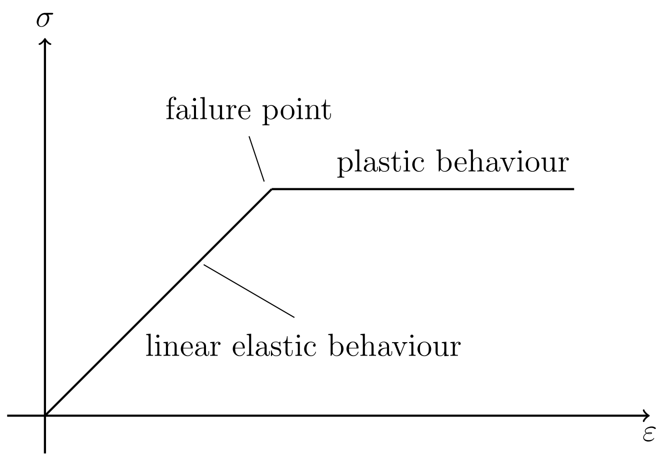


Figure 3.3 Elastic-perfectly plastic behaviour on the stress-strain (σ-ε) plane.

The Mohr-Coulomb failure envelope can be drawn as the closest straight tangential line to the Mohr's circles at failure under different in-situ stress conditions (various σ_1 and σ_3). Figure 3.4 shows two different Mohr's circles at failure corresponding to different σ_1 and σ_3 on the τ-σ plane where the Equation 3.1 changes to the following form

$$\tau = c + \sigma \tan \phi$$

where, τ and σ are shear and normal stress respectively.

3.3 Modelling Rock Bolts

A regular way to model bolted rock material, in macroscopic scale, is to use the equivalent reinforced rock material with modified properties (Maghous et al. 2012). On the other hand, to precisely model an installed rock bolt in rock, all the effects of rock bolt on the rock material (Section 2.2) should be taken into account. In addition to these effects, the influence of deformation of the tunnel should be considered. Because of the applied in-situ stress, tunnel's boundaries converge inward. This results in changing the amount of tension in rock bolts. This change in amount of tension is of high importance especially in passive rock bolts, where rock bolts are installed without or with a small amount of tension. However, this increase of tension in passive rock bolts can sometimes lead to the failure of pretension rock bolts material which should be prevented. Other effects of rock bolt which can be seen in fully grouted rock bolts and can be considered in the numerical model are increasing the strength parameters of joints (cohesion and friction angle), because of grout material, and applying a compressive force against separation of joint. Some examples of these models can be found in Chen et al. (2009); Deb and Das (2011).

In ABAQUS there are several ways to model a rock bolt acting on the

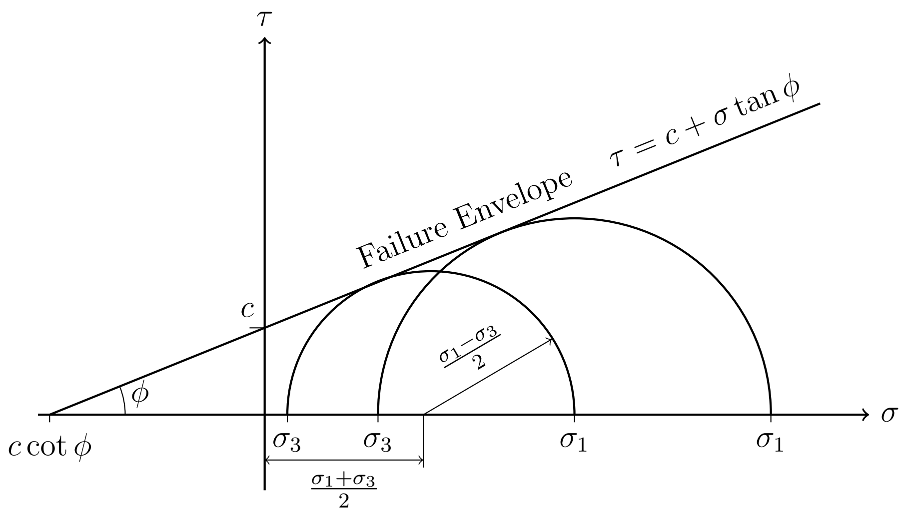


Figure 3.4 Mohr-Coulumb failure envelope on τ-σ plane.

rock material. The simplest way is to simulate the effects of bolt with two concentrated forces at two ends of the rock bolt acting on opposite directions. This model only can be used when there is no discontinuity passing the axis of the rock bolt, i.e. continuum material (Bobet and Einstein 2011). A more comprehensive model of rock bolt can be achieved by considering the changes in tension of the rock bolt element during the loading period. This can be modelled by using *truss elements* in ABAQUS. Truss elements are one dimensional which can only sustain tension in the direction of their axis. In these elements, by increasing the deformation at two ends of the element, the induced stress at the element increases until failure is reached. ABAQUS calculates this failure point with respect to the predefined strength parameters (Mises yield model) and cross sectional area of the element. Note that this model does not consider the effect of rock bolt on increasing the shear resistance of discontinuities. However, by applying a stress component in direction of the normal to the discontinuity's surface, it has significant effect on decreasing the slip and separation of the discontinuity. In this model, for pretension rock bolts, as Bobet and Einstein (2011) mentioned, it can be assumed that axial resistance of rock bolts are much higher than their shear resistance, i.e. one dimensional elements. Another possible model for rock bolts is to use *beam elements* in ABAQUS which have resistance to axial stretch as well as bending. These elements are appropriate only in fully grouted rock bolts where rock bolts significantly increase the shear resistance of rock discontinuities.

Apart from modelling the rock bolt element itself, one of the most difficult parts of numerical modelling is to model the interaction between different materials such as bolt-grout and grout-rock interfaces. A simplified assumption can be made on modelling end-anchored rock bolts that rock bolt and rock are strongly tightened together and no separation is allowed between them. In this case, end-anchored rock bolts can be modelled by tightening the two ends of the rock bolt elements to the rock. Tightening several nodes of rock bolt elements to rock material can represent fully grouted rock bolt.

In this study, *truss elements* are used to model end-anchored rock bolts and tie-rod in Birmingham truss bolt systems. It is assumed that there is no waste of energy in connection of inclined and horizontal members of truss bolt system, resulting in the same amount of tension in inclined bolts and tie-rod. Anchorage and head of rock bolts are modelled by constraining the two ends of the truss elements to the rock material where no separation is allowed. By applying in-situ stress on the excavation, the rock material undergoes different amounts of deformation in various distances from the boundary of the excavation. Obviously, the maximum deformation will be at the boundary of the opening. This results in different amounts of deformation at two ends of an installed rock bolt (truss element), which increases the stress in the rock bolt element and as a result, more load applies to the rock material.

3.4 Modelling Bedding Planes

ABAQUS presents an option, named *contact pair* to model bedding planes. With this option, two surfaces will be identified for two sides of a bedding plane. These surfaces have zero spacing at the beginning of the analysis, presenting a closed plane of weakness. Two sides of this discontinuity transmit shear stress as well as pressure and follow the simple Coulomb friction model. In this model shear stress (τ) is a function of a fraction of the pressure at contact (p).

$$\tau = \mu p \tag{3.5}$$

where μ is the coefficient of friction. During solving the model, ABAQUS calculates a critical shear stress ($\tau_{critical}$) at the surface of each element and judges sliding or sticking (zero sliding) behaviour for contact. Sliding occurs when the shear stress becomes equal or greater than the critical shear stress, where $\tau_{critical} = \mu p$ and μ is $\tan \phi$. On the other hand, if shear stress does not exceed the critical value, stick region, no sliding occurs (Fig. 3.5).

One difficulty in modelling contact pairs in ABAQUS is choosing the correct formulation for pressure-overclusure behaviour of contact. To prevent penetration of the surfaces of the discontinuity, ABAQUS provides so-called *hard* formulation. Graphical illustration of this *hard* pressure-overclosure formulation is shown in Figure 3.6. In this model, when clearance becomes zero, contact pressure increases without undergoing any overclosure (penetration).

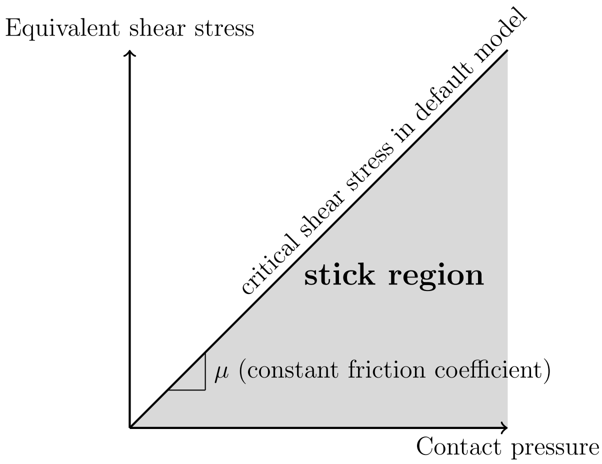


Figure 3.5 Sliding and sticking regions, adapted from ABAQUS (2010) manual.

3.5 Verification

3.5.1 In-situ Stress and Rock Material

As mentioned before, Mohr-Coulomb failure criterion can be used to model rock mass behaviour. For verification purpose, an analytical solution proposed by Hoek et al. (1998) can be used. This solution gives the amount of displacement at the boundary of a circular tunnel with radius of r_0 under hydrostatic in-situ stress equal to p_0 and a uniform internal support pressure acting outward on the boundary of the opening equal to p_i (Fig. 3.7). In this solution, critical support pressure (p_{cr}) can be defined as

$$p_{cr} = \frac{2p_0 - \sigma_c}{1 + k}$$

where σ_c and k are the same as Equations 3.2 and 3.3. If the internal pressure is greater than critical pressure no failure occurs and rock remains elastic

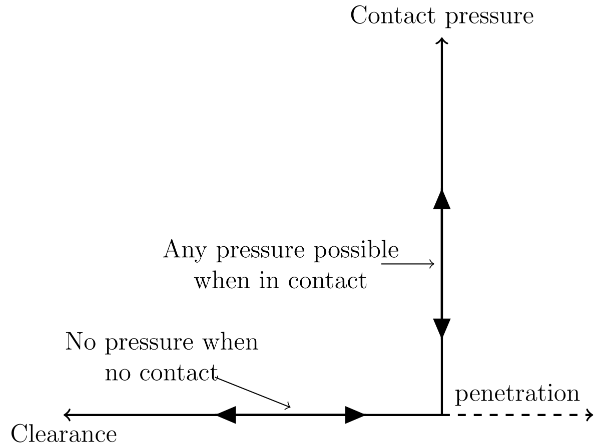


Figure 3.6 Pressure-overcloasure behaviour, adapted from ABAQUS (2010) manual.

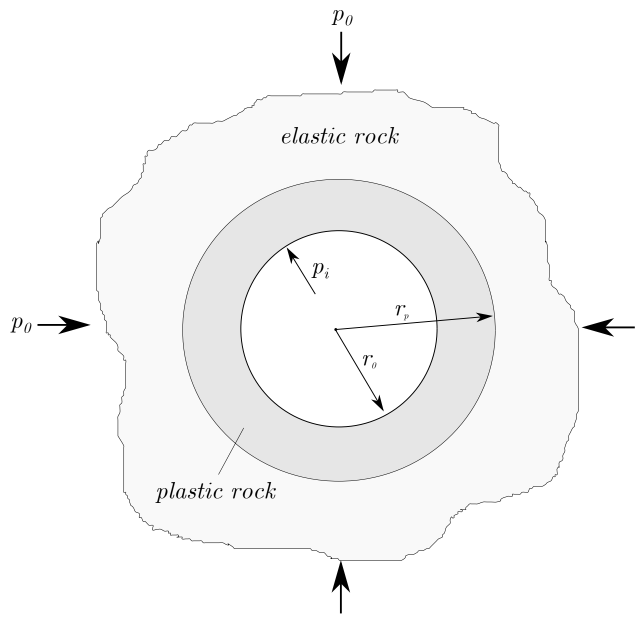


Figure 3.7 Plastic and elastic rock around a tunnel (after Hoek et al. (1998)).

where the inward radial elastic displacement of the tunnel can be calculated as

$$u_{ie} = \frac{r_0(1+\nu)}{E}(p_0 + p_i) \tag{3.6}$$

where E is the modulus of elasticity and ν is the Poissons ratio. On the other hand, if the internal pressure of support is less than the critical pressure, failure of rock occurs. In this condition, the thickness of the failed rock and the total inward displacement of the boundary of the tunnel, u_{ip} are given by (Hoek et al. 1998)

$$r_p = r_0 \left[\frac{2\big(p_0(k-1) + \sigma_c\big)}{(1+k)\big((k-1)p_i + \sigma_c\big)} \right]^{\frac{1}{k-1}} \tag{3.7}$$

$$u_{ip} = \frac{r_0(1+\nu)}{E} \left[2(1-\nu)(p_0 - p_{cr}) \left(\frac{r_p}{r_0}\right)^2 - (1-2\nu)(p_0 - p_i) \right] \tag{3.8}$$

where r_p is the radius of plastic zone from the centre of the tunnel (Fig. 3.7). A graphical illustration of the variables and behaviour of the tunnel is shown in Figure 3.8. This plot is a result of Equations 3.6 and 3.8. It shows that the inward displacement is zero when the internal pressure and in-situ stress are equal. By decreasing the internal pressure to the critical support pressure, elastic inward displacement increases and internal pressure less than critical support pressure results in plastic inward displacement of boundary of the tunnel.

To verify the numerical model, the amount of inward displacement on the crown of the tunnel (Eq.3.8) is calculated. After obtaining the numerical and analytical solutions for the same problem, the resultant relative error of the numerical analysis to analytical solution has been calculated to be at 0.31% which is not remarkable.

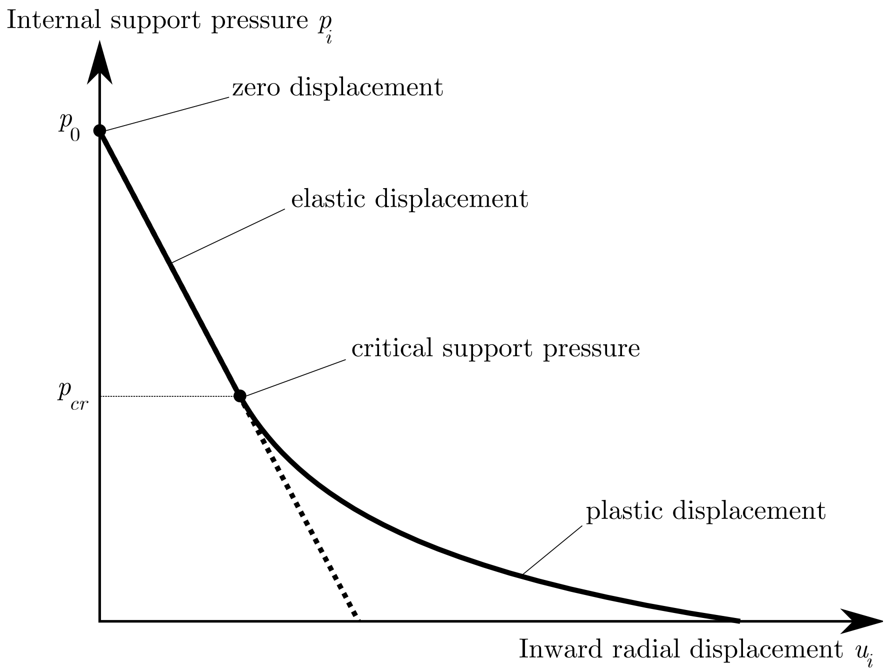


Figure 3.8 Changes in inward radial displacement with respect to support pressure (after Hoek et al. (1998)).

3.5.2 Bedding Planes

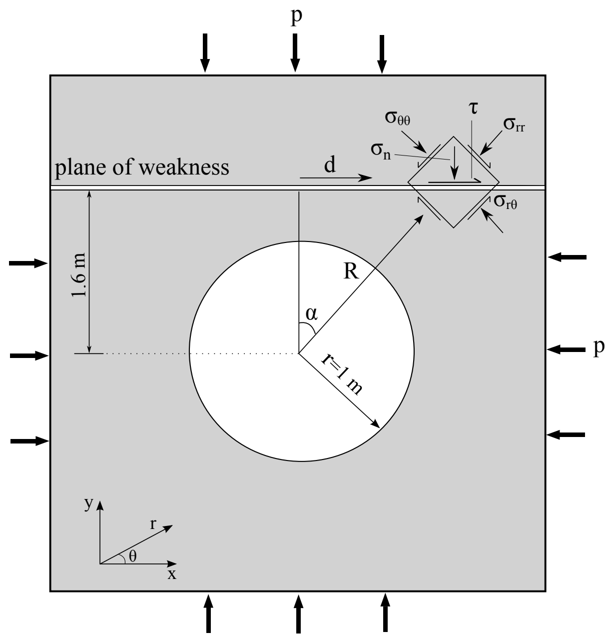
To verify the behaviour of bedding planes in the numerical model, analytical solution suggested by Brady and Brown (2005) has been used. This solution calculates the normal and shear stresses on the plane of weakness at the distance of R from centre of the tunnel with radius of r (Fig. 3.9a). It is assumed that discontinuity has zero tensile strength, zero cohesion and is non-dilatant in shear. Considering Kirsch (1898) equations (Section 3.6) for a circular tunnel under hydrostatic stress field in elastic material, normal and shear stress on the plane of weakness without sliding at the distance of R

from the centre of a tunnel can be derived as

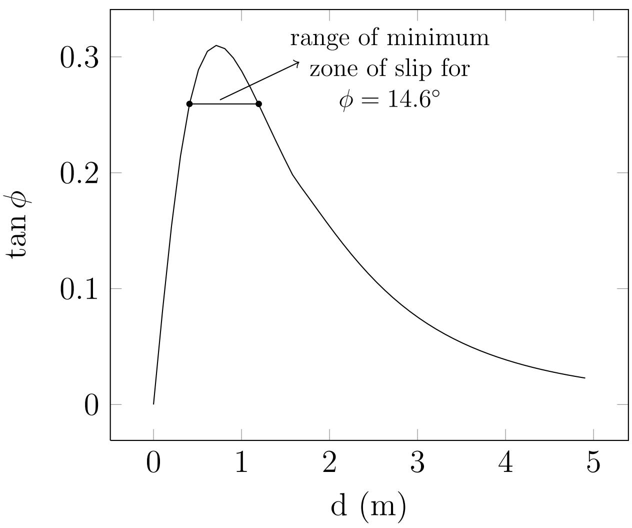
$$\begin{aligned}
\sigma_n &= \frac{1}{2}(\sigma_{rr} + \sigma_{\theta\theta}) + \frac{1}{2}(\sigma_{rr} - \sigma_{\theta\theta})\cos(2\alpha) \\
&= p(1 - \frac{r^2}{R^2}\cos(2\alpha)) \qquad (3.9) \\
\tau &= \sigma_{r\theta}\cos(2\alpha) - \frac{1}{2}(\sigma_{rr} - \sigma_{\theta\theta})\sin(2\alpha) \\
&= p\frac{r^2}{R^2}\sin(2\alpha) \qquad (3.10)
\end{aligned}$$

Plotting the value of $\tau/\sigma_n = tan\phi$, determined by the analytical solution, versus horizontal distance from the centre of a tunnel with the given dimensions at Figure 3.9a results in a curve which is shown in Figure 3.9b. It can be seen that the value of shear stress increases from zero exactly above the centre of the tunnel and reaches the maximum value at the horizontal distance less than a radius of the tunnel. For the given dimension of the tunnel, angle of friction equal to 17.2° ($\tan\phi = 0.31$) depict the maximum shear stress (critical shear stress value). Angle of friction bigger than this value results in no sliding behaviour of discontinuity and the elastic solution can be maintained. On the other hand, angle of friction slightly less than this value results in sliding. The predicted area of sliding for $\phi = 14.6°$, obtained from elastic solution, is shown in Figure 3.9b.

If sliding happens, the problem is no longer elastic and Equations 3.9 and 3.10 can not be used. However, the elastic solution can give us a valuable preliminary insight to the problem (Brady and Brown 2005). In this case, numerical analysis should be used to give a more accurate solution of stress distribution around the tunnel.



(a)



(b)

Figure 3.9 (a) shear and normal stresses on the plane of weakness (after Brady and Brown (2005)) and (b) minimum range of slip for $\phi = 14.6°$.

Numerical Solution

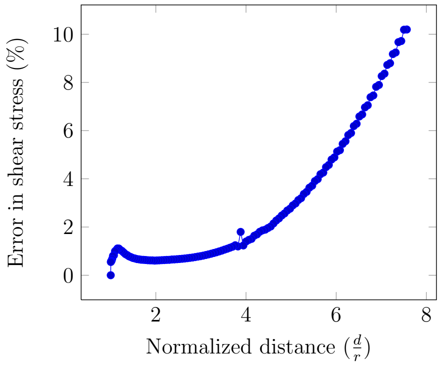
For a tunnel with dimensions same as Figure 3.9a, results of the numerical model showed a negligible error compared to analytical results. The most important parameter that should be verified is sliding which will be one of the parameters to be investigated in next chapters. In this case, analytical solution showed that for the given dimensions of tunnel, the angle of friction greater than $\phi = 17.2°$ (peak point in Fig. 3.9b) resulted in no sliding and vice versa. Results of the numerical analysis showed the exact agreement with the analytical results (Table 3.1).

Table 3.1 Results of numerical modelling compared to analytical solution.

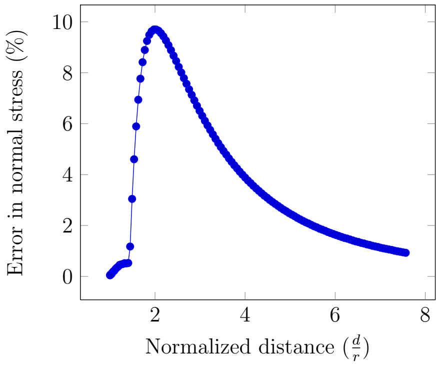
	$\phi = 10$	$\phi = 16$	$\phi = 18$	$\phi = 20$
Analytical results	sliding	sliding	no sliding	no sliding
Numerical results	sliding	sliding	no sliding	no sliding

In case of shear stress, magnitude of error exceeds 2.4% and increases gradually for the distance of more than 5 times radius from the centre of the tunnel (Fig. 3.10a). It has been shown by Brady and Brown (Brady and Brown 2005) that the stress distribution does not change significantly after the distance of 5 times of radius of the tunnel. So, this amount of error does not have considerable influence on the results of analysis.

The peak error for normal stress on the contact surface is less than 10 percent, which can be seen at the distance of 1.7 to 3 times radius from the centre of tunnel (Fig. 3.10b). This amount of error is relatively high and is probably because of the little amount of penetration on the contact surfaces, which is inevitable. The value of contact pressure does not play any



(a)



(b)

Figure 3.10 Percentage of error in shear (a) and normal (b) stress obtained from numerical results

significant role in future analysis in this thesis.

3.6 Sensitivity Analysis

Rock material around an underground excavation can be considered as infinite material with respect to the dimension of the tunnel. In numerical simulation, dimension of the model should be large enough to represent an infinite material. In other words, effects of boundary conditions on the stress distribution around a tunnel should be minimum. In order to study the effect of boundary conditions on the results of excavating a tunnel, sensitivity analysis on the dimension of the model has been carried out. A closed-form solution, namely Kirsch equations, has been used to check the results of the numerical analysis. This solution calculates the stress condition and displacement after excavating a circular tunnel in an elastic isotropic homogenous material subject to biaxial in-situ stress as (Brady and Brown 2005)

$$
\begin{aligned}
\sigma_{rr} &= \frac{p}{2}\left[(1+K)\left(1-\frac{a^2}{r^2}\right)-(1-K)\left(1-\frac{4a^2}{r^2}+\frac{3a^4}{r^4}\right)\cos 2\theta\right]\\
\sigma_{\theta\theta} &= \frac{p}{2}\left[(1+K)\left(1+\frac{a^2}{r^2}\right)+(1-K)\left(1+\frac{3a^4}{r^4}\right)\cos 2\theta\right]\\
\sigma_{r\theta} &= \frac{p}{2}\left[(1-K)\left(1+\frac{2a^2}{r2}-\frac{3a^4}{r^4}\right)\sin 2\theta\right]\\
u_r &= -\frac{pa^2}{4Gr}\left[(1+K)-(1-K)\left(4(1-\nu)-\frac{a^2}{r^2}\right)\cos 2\theta\right]\\
u_\theta &= -\frac{pa^2}{4Gr}\left[(1-K)\left(2(1-2\nu)+\frac{a^2}{r^2}\right)\sin 2\theta\right]
\end{aligned}
\tag{3.11}
$$

where σ_{rr}, σ_θ and $\sigma_{r\theta}$ are the total stresses after excavation, u_r and u_θ are radial and tangential displacements induced by the tunnel, p is in-situ stress value, K, is the fraction of horizontal to vertical stress, a is tunnel radius, r is radial distance from the centre of the tunnel, θ is the angle of the polar coordinates (shown in Figure 3.11) and G is modulus of rigidity of the material where $G = \frac{E}{2(1+\nu)}$.

To study the effect of dimensions of the model, several models with different dimensions have been created. The properties of the model are shown in Table 3.2. Sensitivity analysis has been carried out using the radial displacement at the crown of a circular tunnel (Eq. 3.11) and results of numerical models have been compared to closed-form solution. These results are shown in Table 3.3 where it can be seen that by increasing the dimensions of the model, amount of radial displacement at the crown of the tunnel becomes closer to the analytical solution. This also can be seen in Figure 3.12 that for a large dimension, the amount of displacement converges to the result of

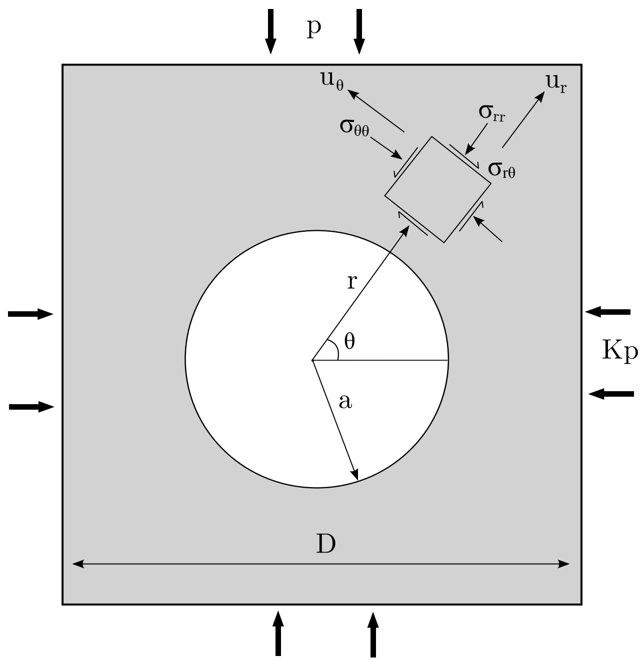


Figure 3.11 Circular tunnel under biaxial in-situ stress.

the closed-form solution. In infinity, these results will be exactly the same. In this Figure, horizontal axis shows the dimensionless fraction of dimension of the model (D) to radius of tunnel (a) in semi logarithmic scale.

Large models require high amount of computational cost so smaller models with negligible amount of error can be used for further analysis. Looking at Table 3.3, it can be concluded that models with dimension greater than 20 times of the tunnel radius result in under 0.83% error which is acceptable.

Table 3.2 Model parameters for a circular tunnel in a typical rock material.

Model characteristics	
Module of elasticity	$E = 5.74$ GPa
Poissons ratio	$\nu = 0.35$
In-situ stress	$\sigma_v = \sigma_h = 10$ MPa
Tunnel radius	$a = 1$ m
Model dimension	D

Table 3.3 Results of numerical and closed-form solutions.

D (m)	u_r (cm)	Error (%)
5×5	1.43	9.21
10×10	2.06	2.87
15×15	2.21	1.41
20×20	**2.27**	**0.83**
30×30	2.31	0.39
40×40	2.33	0.21
50×50	2.33	0.17
100×100	2.34	0.08
Analyitical	2.35	0

3.7 Comprehensive Model of an Underground Excavation

A rectangular underground excavation in laminated adjacent rock has been modelled using the introduced features in this chapter. The tunnel is assumed to be long enough to validate plain strain assumption. As long as the problem is symmetrical, half of the tunnel can be modelled by putting suitable boundary conditions on the symmetry line, i.e. restraining displacement in Y direction. Dimension of the mesh in an area near the tunnel has

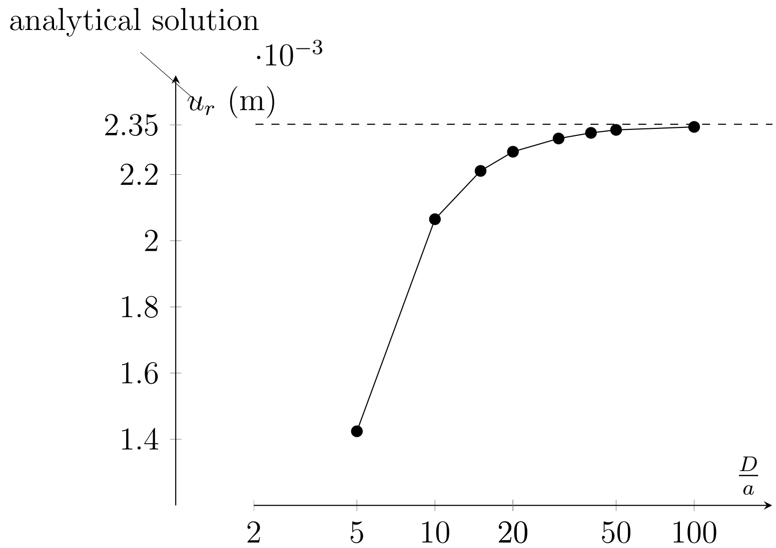


Figure 3.12 Numerical and analytical results for different tunnel dimensions

been chosen as one tenth of the shortest length of rock bolt (inclined bolt) in this study (1 m). This results in square mesh with dimension of 0.1 m. Because of high calculation cost, this high density mesh can not be applied to the whole model and has been used for only a certain area around the tunnel. This area has been chosen with respect to Brady and Brown (2005) which pointed out that the influence of an underground excavation is limited to an area of 5 times of the tunnel's dimension around the tunnel. This also has been observed in early models that the effect of excavating a tunnel on adjacent rock reduces and becomes negligible beyond the distance of 5 tunnel diameter from the centre of the tunnel.

The model contains 4 bedding planes, two above at distance of 90 and 150 cm from the roof and two beneath at distance of 1 and 3 m from the floor of the tunnel. Rock has been modelled as elastic-perfectly plastic material which yields under Mohr-Coulomb failure criterion. Different rock properties have been chosen to represent several rock layers and suitable angle of friction assigned to each bedding surface. Finally, truss bolt elements (inclined bolt and tie-rod) have been modelled using *truss elements* with pretension being applied as initial condition.

To apply the in-situ stress condition, this model is solved in three steps (Section 3.2.1). Similarly, pretension in truss elements can be applied using initial condition and changing the boundary conditions during several steps. For this purpose, during steps one and two, two ends of all of the truss bolt elements are restrained against X-Y displacements. At step three, with excavating the tunnel, boundary conditions of these nodes will be removed and the pretension will be applied to truss bolt elements. This entails an assumption that reinforcement is installed right after excavating the tunnel. Figures 3.13 and 3.14 show two views of the model mesh and the structure of the model[2]. These figures show FEM mesh for a rectangular tunnel with height and span of 2 and 3 m respectively with a typical truss bolt system.

[2]Model properties, e.g. rock, truss properties, etc. are presented in next chapters.

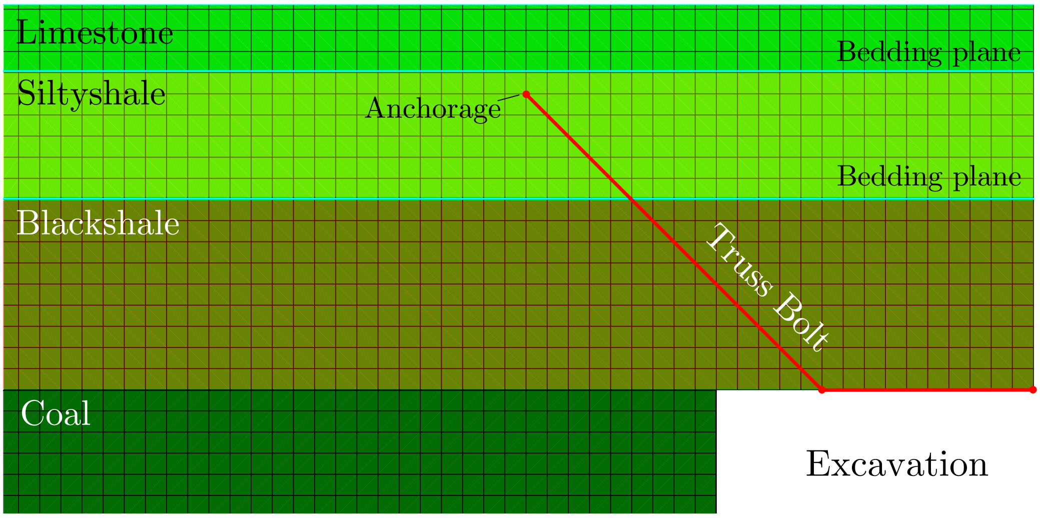


Figure 3.13 Close view of ABAQUS FEM meshing around a tunnel with truss bolt and different rock layers.

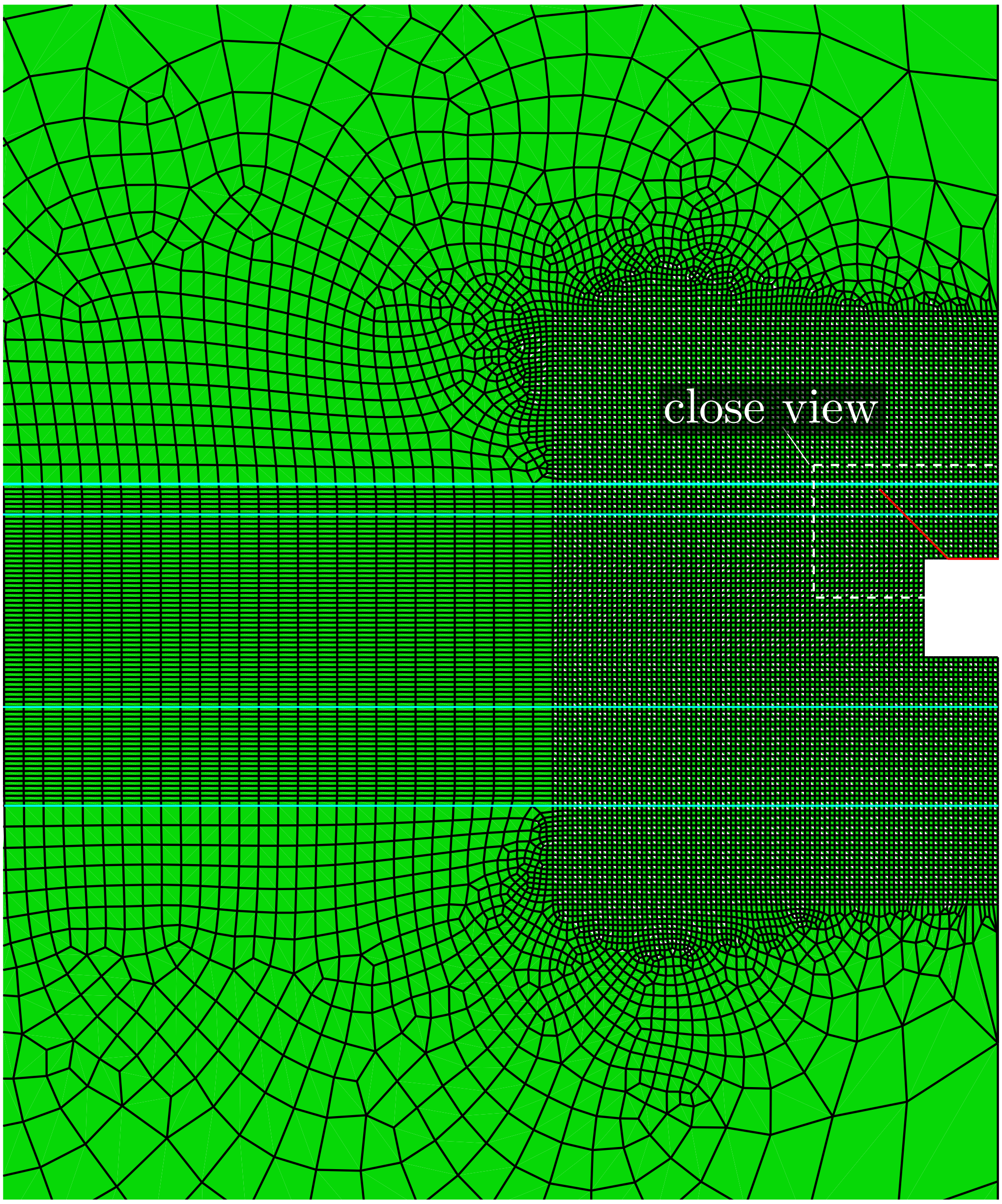


Figure 3.14 Wide view of ABAQUS FEM meshing around a tunnel with truss bolt and 4 bedding planes.

CHAPTER 4

Truss Bolt Mechanism

As discussed in previous chapters, truss bolt demonstrated very good application in providing the stability of underground excavations according to several researchers' experience. Despite the benefits of truss bolt systems in support of underground excavations, the working mechanism of this system under deformation of roof is largely unknown (Wahab Khair 1984; Liu et al. 2001), and most of the design parameters are chosen predominantly based on engineers' judgement and experience. In unfamiliar ground conditions, such experimental design may cause several stability problems. These problems regularly happens in underground mines where the condition of surrounding rock may frequently change during a mine's life. Ground parameters such as bedding thickness, properties of rock material, geological features such as faults and many other factors could possibly affect the optimum design of reinforcement system. Without knowing the effects of these parameters on working mechanism of truss bolt, achieving an optimum design is practically impossible.

To investigate the mechanism of truss bolt system, numerical modelling as a powerful method can give us the ability to monitor the detailed effects of reinforcement systems. This allows engineers to observe, gather the required data and analyse the output to obtain the desired information.

In this chapter, mechanism of truss bolt is investigated using numerical modelling techniques. Parametric study on different design parameters of truss bolt is carried out by modelling some regular design patterns with different design parameters. These patterns are mainly based on existing practice and recommendations in the literature. The behaviour of the rock after installing reinforcement needs to be measured via defining some performance indicators. For the scope of this study, these indicators should be able to evaluate the reinforcing effect of the truss bolt system, roof deflection and effects of truss bolt on preventing cutter roof failure. Additionally, to have a preliminary insight of the difference between mechanism of systematic rock bolt and truss bolt system, a comparison between these reinforcement systems is made using several stability indicators.

4.1 Regular Truss Bolt Patterns

Parametric study is perhaps, one of the best ways to understand the different effects of several design parameters of truss bolt system on stability of an underground opening. To monitor the effects of each design parameter, three regular truss bolt patterns with different design parameters have been chosen to be modelled. All of these designs have been chosen from literature and adapted to the tunnel dimensions in this study. These truss bolt patterns with different parameters are adopted to highlight the different effects of each parameter on the stress distribution around the tunnel:

Pattern 1-Liu et al. (2005): The first pattern has been chosen with respect to a pattern that Liu et al. (2005) used to verify their analytical method. Here we adjust the design parameters to the dimensions of our model to obtain the desired pattern. The characteristics of this truss bolt pattern, after adjusting to the dimensions of a tunnel (Fig. 4.1), are $L = 2$m, $\alpha = 60°$ and $S = 2.8$m (these parameters are shown in Fig. 4.1). It is necessary to mention that Liu et al. (2005) used this pattern in combination with ground anchors to support side walls.

Pattern 2-Cox and Cox (1978): As mentioned in Section 2.6 Cox and Cox (1978) pointed out several recommendations for design and a scheme for installing truss bolt system. On the basis of their recommendations, the second truss bolt pattern will be $L = 2$m, $\alpha = 45°$ and $S = 2$m.

Pattern 3-Ghabraie et al. (2012): They have pointed out a group of optimum designs on based on the effect of truss bolt system on reducing the area of loosened rock beneath the natural reinforced arch. The third pattern has been chosen with respect to one of the optimum designs of their study. The parameters are $L = 3$m, $\alpha = 60°$ and $S = 1.6$m.

4.2 General Properties of the Model

A reference model has been created to examine the effects of several reinforcement patterns on the stability of the excavation. This model consists of four bedding planes, two above the tunnel and two beneath the tunnel.

A schematic view of the tunnel showing dimensions and rock formation is shown in Figure 4.1 and Table 4.1. Meshing and model dimensions are the same as the model discussed in Section 3.7. The thickness of layers and rock properties have been chosen with respect to Xuegui et al. (2011), Zhang (2005) and Yassien (2003). Other properties of the model are as follow.

Table 4.1 Physical properties of different rock layers (Yassien 2003; Zhang 2005).

Type of the rock	Angle of friction (ϕ)	Cohesion (MPa)	Elastic Modulus (GPa)	Poisson's Ration (ν)
Limestone	29	3.01	17	0.3
Blackshale	20	1.0	2	0.27
Siltyshale	27	1.37	4.5	0.27
Coal	22	1.09	2.4	0.34

Thickness of the first roof layer is an important factor that changes the practice of truss bolt system and can affect the optimum design (Wahab Khair 1984). In this part of the study we model only one layer configuration as shown in Figure 4.1. Effects of this factor will be investigated in the next chapter.

Properties of the bedding planes are chosen with respect to Zhang (2005) and shown in Table 4.2.

Physical characteristics of rock bolts have been chosen with respect to the design catalogues of Minova (Orica) Company for cable strata reinforcement. These properties are shown in Table 4.3.

Tension force increases by the deformation of the host rock. To prevent failure, as Hoek et al. (1998) mentioned, pre-tension stress at bolts should not exceed 70% of the yield stress of rock bolts. In all of the models, tensile

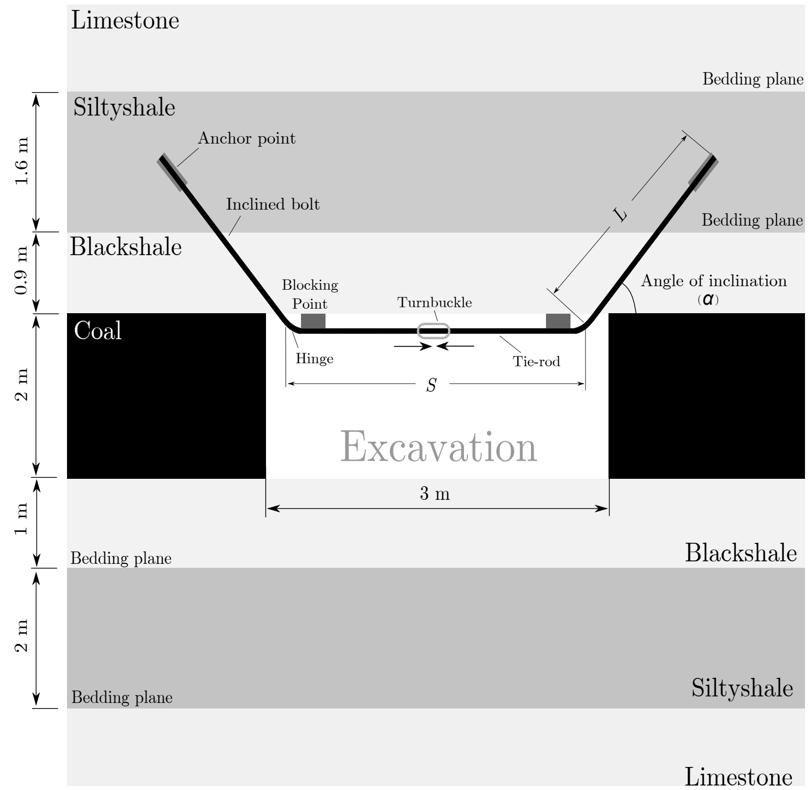


Figure 4.1 Reference model.

stress in truss bolt system has been chosen as 0.314 MN which is equal to 60 percent of yield stress (1670 MPa).

In-situ stress distribution has been chosen as hydrostatic type of stress equal to 1.9 MPa ($\sigma_v = \sigma_h = 1.9$ MPa). In Section 4.4 for investigating the effect of truss bolt on cutter roof failure other in-situ stress distributions will be used.

Table 4.2 Coefficient of friction on bedding surfaces (Zhang 2005).

Bedding planes	Coefficient of friction (μ)
First bedding plane (above)	0.364
Second bedding plane (above)	0.46
First bedding plane (beneath)	0.364
Second bedding plane (beneath)	0.46

Table 4.3 Typical mechanical properties of the cable bolts used for modelling truss bolt and systematic rock bolt.

Cross-sectional area	313 mm^2
Module of elasticity	200 GPa
Ultimate tensile strength	1670 MPa
Elongation on 600mm length	6-7%
Mass per meter-Cable	2.482 (kg/m)

4.3 Stability Indicators

Finite Element Method is not able to model separations and rock falls. This limitation makes it difficult to judge the stability of an underground excavation with a simple "yes or no" function. The behaviour of the rock after installing reinforcement can be measured via defining some performance indicators. These indicators are derived from Mohr-Coulomb material model, failure criterion and elastic-plastic deformation in rock and can be adopted to monitor reinforcing effect of truss bolt, elastic-plastic behaviour of rock, horizontal and vertical deformations in rock, roof deflection and shear crack propagation in cutter roof failure. Although these stability indicators are not able to indicate the failure of the tunnel, but can evaluate the effects of each design parameter. In this Section, these indicators will be introduced and the results of installing different truss bolt patterns on the surrounding rock will be discussed.

4.3.1 Reinforced Roof Arch and Area of Loosened Rock

After excavating a tunnel, redistribution of the in-situ stress forms a pressurized arch above the tunnel. This arch is stable and can carry the load to the sides of the tunnel. The rock material beneath this arch is considered as loosened material (Figure 4.2). This phenomenon can be observed in almost all types of coherent rock formations (Li 2006) and is proved by experience as well as numerical analysis (Bergman and Bjurstrom 1984; Huang et al. 2002). Position of this arch changes drastically by changing the in-situ stress distribution. High horizontal stress is favourable in forming a closer natural arch to the roof, i.e. smaller loosened area. It should be noted, however, that extensive horizontal stress has negative effects on cutter roof failure and also causes stability problems in pillars.

Usually, the natural arch is positioned far above the tunnel and the loosened area beneath it should be stabilized (Li 2006). This can be achieved by either removing or reinforcing the loosened rock. In coal mines, however, where the shape of the tunnel is normally governed by the shape of the coal layer, removing the loosened rock is not an option, thus a suitable reinforcement system should be used.

Choosing parameters of the reinforcement systems to carry the load of the loosened area, without considering reinforcing effects of the system, normally leads to overdesign. The load of the loosened area can be used as only to achieve an upper limit (ultimate capacity) for the parameters of the reinforcement system (Cox and Cox 1978). To have a safe and economic design, the reinforcing effect of truss bolt on the loosened rock area should be taken into account. By applying a new load distribution around the tunnel, truss bolt system reinforces the loosened area and repositions the natural roof arch which results in smaller loosened area.

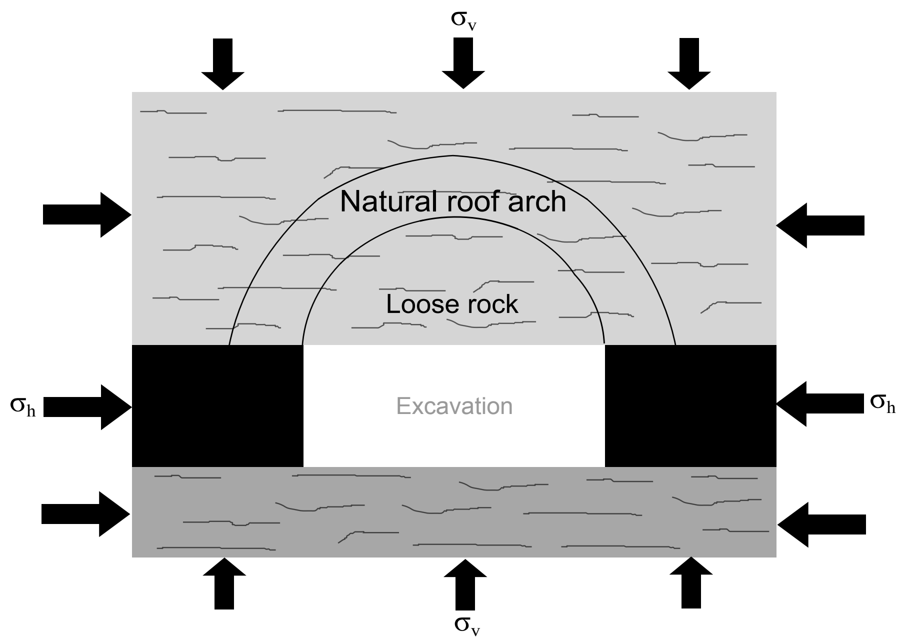


Figure 4.2 Natural roof arch and loosened area around a tunnel in laminated rock, adapted from Cox and Cox (1978).

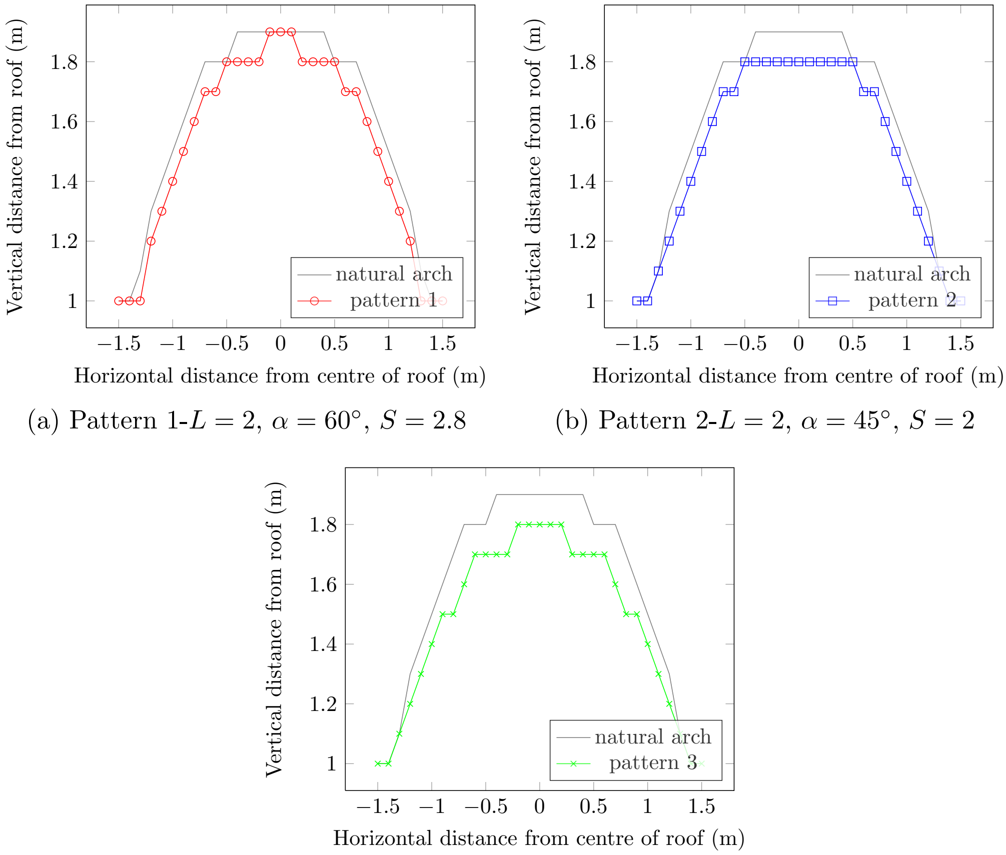
To specify the position of the reinforced arch, Huang et al. (2002) used the concept of invert stress cone to find the natural arch position around an underground excavation. In their model the thickness of the arch has been governed by the direction of principal stresses. According to Huang et al. (2002), reinforced arch is the area in which principal stresses are not in vertical or horizontal direction except on the apex of the arch. Another approach to find the reinforced arch is to use the vertical deformation of the rock above the roof. In this approach, the reinforced arch is defined by the points with the closest amount of vertical deformation to a certain fraction of the maximum vertical displacement of the tunnel roof. This fraction is the amount of displacement which predicts the stable/unstable rock. This condition can be expressed as (Ghabraie et al. 2012)

$$|d_i - (n \times d_{max})| = \text{Minimum} \tag{4.1}$$

where d_i is the vertical displacement at points above the roof in FE mesh, d_{max} is the maximum vertical displacement on roof and n is a fraction between 0 and 1.

In this approach, $n \times d_{max}$ is a threshold (a certain amount of displacement) which predicts the area of the loosened rock. Areas with less deformation than this threshold are considered to be stable and vice versa. The fraction (n) can be chosen with respect to the sensitivity of the tunnel to displacement and can be different from case to case. In this study, $n = 50\%$ has been chosen which implies that areas with less than 50% of the maximum displacement on the roof are loosened area. The output of this method is a line which connects all the points resulting from Equation 4.1. It should be noted that this approach does not necessarily predict the actual area of loosened rock and is only used to define a basis for comparing different designs.

Using $n = 50\%$, the position of the reinforced arch and area of the loosened rock for different truss bolt patterns have been derived. These results are shown in Figure 4.3. It can be seen that truss bolt system repositions the reinforced arch and reduces the area of loosened rock around a tunnel under hydrostatic in-situ stress. These results highlight the importance of the position and the angle of the inclined bolts. The truss pattern with short span and wide angled inclined bolts (pattern 3, Figure 4.3c) shows the best result. On the other hand, pattern 1, which has a bigger span, has a small



(a) Pattern 1-$L = 2$, $\alpha = 60°$, $S = 2.8$

(b) Pattern 2-$L = 2$, $\alpha = 45°$, $S = 2$

(c) Pattern 3-$L = 3$, $\alpha = 60°$, $S = 1.6$

Figure 4.3 Reinforced roof arch before and after installing several truss bolt patterns.

effect on the area above the middle of the roof but shows a good response on the areas near the corners (Figure 4.3a). This is because in this pattern the inclined bolts are closer to the corners of the roof.

Table 4.4 shows the amount of reduction in the loosened area as a result of installing different truss bolt patterns. Pattern 3 shows the best response on reducing the area of the loosened rock. Pattern 1 and 2 reduce the area of the loosened rock almost the same amount but result in different shape of the reinforced arch (see Figures 4.3a and 4.3b). One reason is that the major area of the loosened rock is above the middle of the roof and pattern 3 has better coverage on this area compared to the other truss bolt patterns. Also a wide angle of inclination in pattern 3 produces a greater vertical component which can control the vertical displacement on the roof.

Table 4.4 Reduction in the loosened area after installing three truss bolt patterns

Different truss bolt patterns	Reduction in the loosened area (cm^2)
Pattern 1	2200
Pattern 2	2300
Pattern 3	3300

4.3.2 Roof Deflection

One definition for the stability of an underground excavation can be expressed by the amount of deformation in the surrounding rock. Failure means an excessive deformation of the rock material. This deformation can happen as roof deflection, rock falls, rock slip, floor heave or convergence of side walls. The amount of deformation which denotes failure varies case to case and depends on the application and purpose of the tunnel. On the basis of this concept, the area of the deflection at roof after installing each truss bolt pattern can be evaluate as a stability indicator. It seems necessary to mention that truss bolt systems do not have significant effect on the reduction of the roof sag (O'Grady and Fuller 1992) but control the stability of the tunnel by improving the load carrying ability of rock. However, this measure shows the response of the truss bolt pattern on controlling the vertical deformation on the roof of an underground excavation and can be used to understand the mechanism of the reinforcement system.

Table 4.5 shows different amounts of reduction in the area of roof deflection after installing different truss bolt patterns. It can be seen that the most effective truss bolt pattern in reducing roof sag is pattern 3 which has long and high angled inclined bolts and are installed close to the center of the roof. In contrast, patterns 2 and 3 show approximately the same effect on this measure which is significantly less than the effect of pattern 3. This is probably because of the position and angle of inclined bolts. When truss pattern is more similar to systematic rock bolt pattern, i.e. high angled inclined bolts, where direction of inclined bolts is close to the direction of the major displacement component on roof (vertical displacement), the reinforcement system has better effect on controlling the deformation on the roof. Further to this parameter, installing inclined bolts near the major area of roof sag (centre of the roof), i.e. short tie-rod length, results in higher amount of reduction in deformation.

Table 4.5 Reduction in the area of roof deflection for three truss bolt patterns

Different truss bolt patterns	Reduction in roof deflection (cm^2)	Reduction in roof deflection (%)
(1) $L = 2$m, $\alpha = 60°$, $S = 2.8$m	1.33	7.5
(2) $L = 2$m, $\alpha = 45°$, $S = 2$m	1.51	8
(3) $L = 3$m, $\alpha = 60°$, $S = 1.6$m	2.11	12

4.3.3 Stress Safety Margin (SSM)

As mentioned in Section 3.2.2 Mohr-Coulomb failure criterion is well known and has been widely used to analyse the elastic-plastic behaviour of rock material (Jing 2003). The yield function in Mohr-Coulomb failure criterion on the basis of principal stresses ($\sigma_1 \geq \sigma_2 \geq \sigma_3$) is

$$f = (\sigma_1 - \sigma_3) - (\sigma_1 + \sigma_3)\sin(\phi) - 2c\cos(\phi) \tag{4.2}$$

In this model compressive stresses are considered as positive. The negative values of f mean the elastic behaviour of the rock and f equals to zero means the yield point. This model is not able to show the post failure behaviour of rock.

In this criterion, if the Mohr's circle corresponding to the stress condition at a point in rock material touches the Mohr-Coulomb failure envelope, rock will yield and the elastic solution is no longer valid. By increasing stress on the surrounding rock around an excavation, more points will undergo failure and the tunnel would collapse. The area beneath the failure envelope represent elastic behaviour of rock with no failure and can be considered as safe area. The failure in Mohr-Coulomb failure criterion is a function of two key parameters: a) radius of Mohrs circle ($\frac{\sigma_1-\sigma_3}{2}$) and b) position of centre of the circle ($\frac{\sigma_1+\sigma_3}{2}$). Failure happens by increasing radius of the circle or/and decreasing the amount of $\sigma_1+\sigma_3$. Figure 4.4 shows two possible Mohr's circles for these two paths of failure. It can be seen that the possibility of failure by decreasing radius of the circle is always more than failure by decreasing the amount of $\sigma_1 + \sigma_3$, in fact, $x_c > x_r/\sin\phi$. Hence, the shortest distance to failure is x_r, where x_r equal to zero represents failure. Now the stress safety margin can be defined based on this parameter. The mathematical expression for x_r can be derived as (Ghabraie et al. 2008)

$$x_r = c\cos(\phi) + (\frac{\sigma_1 + \sigma_3}{2})\sin(\phi) - (\frac{\sigma_1 - \sigma_3}{2}) \tag{4.3}$$

Using a dimensionless expression of this factor makes it easier to compare the results of several models. This can be achieved by the following equation

$$\mathrm{SSM} = \frac{r + x_r}{r} = \frac{2c\cos(\phi) + (\sigma_1 + \sigma_3)\sin(\phi)}{\sigma_1 - \sigma_3} \tag{4.4}$$

where r is the radius of the Mohr's circle

$$r = \frac{\sigma_1 - \sigma_3}{2}$$

In Equation 4.4, SSM equal to one represents failure and plastic behaviour of rock while SSM greater than one means elastic behaviour of rock and safe Mohr's circle.

Figures 4.5 to 4.7 show contours of ΔSSM, which is the difference of SSM before and after installing the three truss bolt patterns around a tunnel under hydrostatic stress distribution ($\Delta\mathrm{SSM} = \mathrm{SSM}_{before} - \mathrm{SSM}_{after}$). By this definition, negative values represent areas in which truss bolt has favourable effect. The green line in these graphs shows the line in which truss bolt does not have any significant effect on the value of SSM around the tunnel. This line demonstrates the border of favourable and unfavourable effects of truss

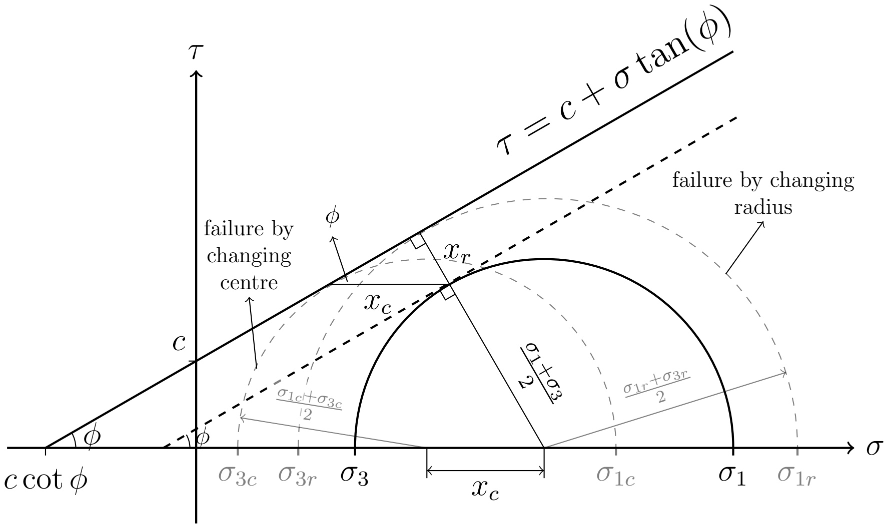


Figure 4.4 Shortest distance from Mohr's circle to failure envelope.

bolt. It can be seen that truss bolt effectively increases the value of SSM around the roof and abutments of tunnel.

Comparing the three truss bolt patterns reveals that short tie-rod, wide angle of inclination and long inclined bolts (pattern 3) results in better effect on the area above the roof but less favourable effect on the rib area (Figure 4.7). On the other hand, in patterns 1 and 2, the most effective areas around truss bolt are near inclined bolts (Figures 4.5 and 4.6). This makes truss bolt patterns 1 and 2 capable of reinforcing the area above the walls of the excavation (rib area). Length of inclined bolts, in current design schemes, is a function of the required load carrying capacity of the reinforcement systems. Inclined bolts should be long enough to ensure sufficient length of anchorage in the safe area (behind the rib line) to provide enough capacity to the truss bolt system (Liu et al. 2005; Cox 2003). Figures 4.5 to 4.7 show that the

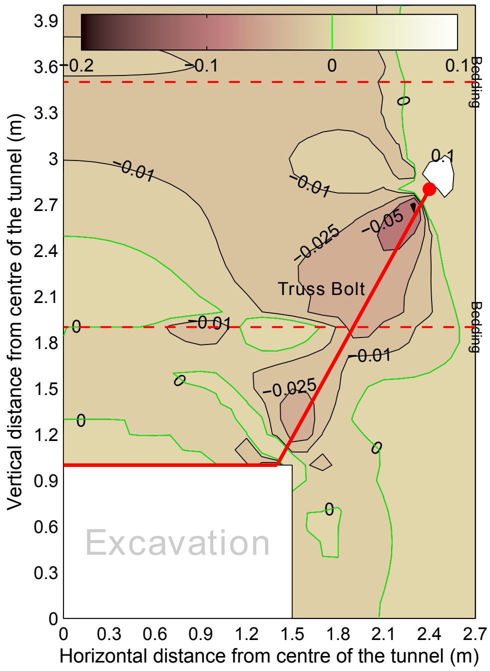


Figure 4.5 ΔSSM by truss bolt (pattern1-$L = 2$, $\alpha = 60°$, $S = 2.8$).

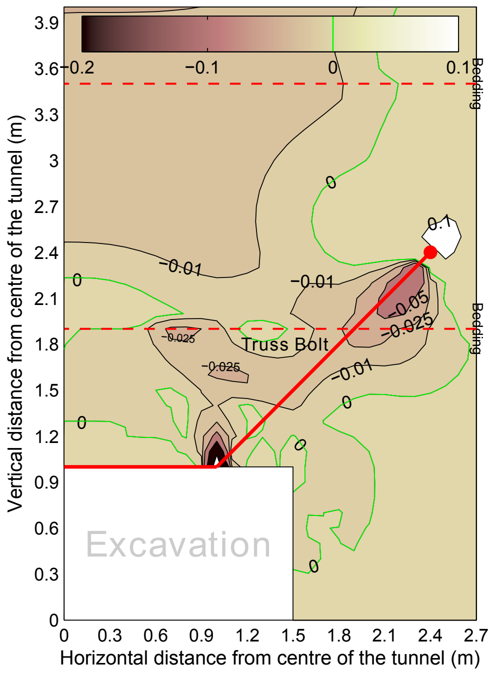


Figure 4.6 ΔSSM by truss bolt (pattern2-$L = 2$, $\alpha = 45°$, $S = 2$).

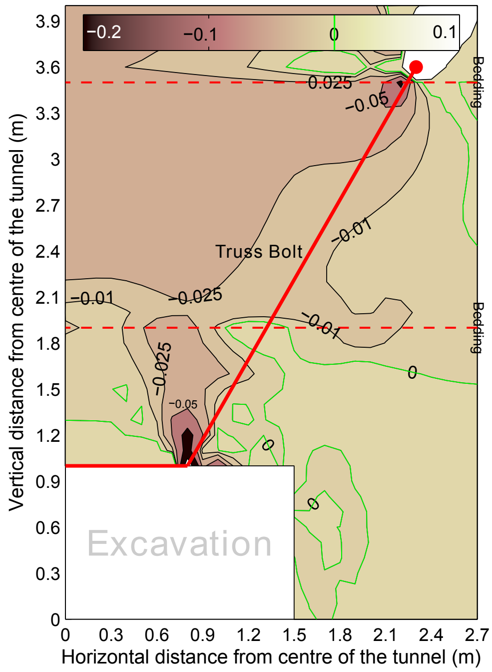


Figure 4.7 ΔSSM by truss bolt (Pattern3-$L = 3$, $\alpha = 60°$, $S = 1.6$).

length of inclined bolts even changes the load distribution around the truss bolt where long inclined bolts (Figure 4.7), in comparison with short inclined bolts (Figures 4.5 and 4.6), are not able to produce a highly reinforced area around inclined members. On the other hand, failure in providing enough length of anchorage results in failure of the truss bolt system. Consequently, the required length of anchorage to carry the applied load on truss bolt system can be always used to find the the lower limit for the length of inclined bolts while this length can be adjusted with respect to the required amount of reinforcing effect near corners of the roof.

Figure 4.8 shows a different illustration of effects of pattern 3 on SSM around the tunnel. Contour lines in this figure have been chosen to represent three

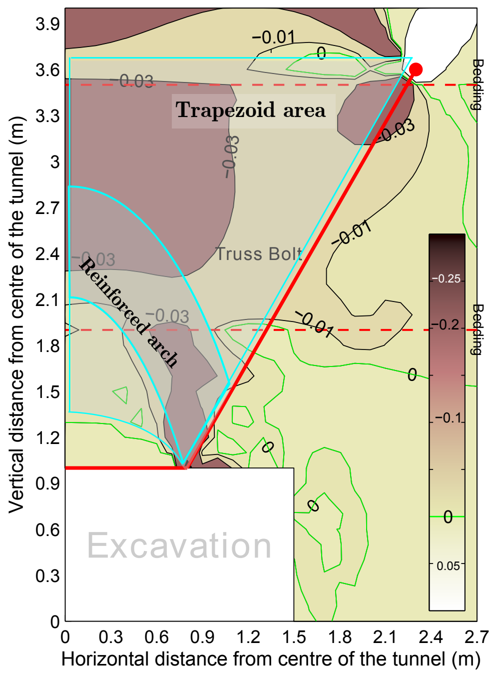


Figure 4.8 ΔSSM and different reinforced areas around a truss bolt pattern ($L = 3$, $\alpha = 60°$, $S = 1.6$).

different areas, namely *a*) major area of reinforcing effect (areas with less than −0.03), *b*) minor area of reinforcing effect (between −0.03 and 0) and *c*) unfavourable effect of truss bolt (bigger than 0). It can be seen that the major reinforced area approximately fits in an arch shape above the roof while the minor reinforced area is more like a trapezoid area which is located above the roof and between the inclined bolts. In other patterns the major reinforced area can be seen around the inclined members (Figures 4.5 and 4.6). However, load distribution around these patterns also shows arch shape borders. The applied horizontal tension at tie-rod can be well transferred to the rock at blocking points and by lateral behaviour of inclined bolts. This load produces an arch shape compressive area above the roof. The reinforced areas in Figures 4.5 to 4.8 match the compressive areas of

Figure 2.3 in Section 2.3.

On the other hand, horizontal tension in tie-rod places the area behind inclined bolts in tension. This unfavourable area is mostly located on sides of the tunnel and can cause stability problems, especially when the side rock is relatively weak. Patterns 1 and 2 which have inclined bolts near the corners of the roof show less unfavourable effect on this area in comparison with pattern 3. In this case, installing truss bolt can shear the side rock which causes rock sliding in this area. Individual rock bolts or rock anchors can be used to stabilise this area (Liu et al. 2005).

4.3.4 Plastic Points Distribution

Following excavating an underground excavation, stress concentration on the adjacent rock around the excavation causes failure in rock material. As discussed in Section 3.3, after installing reinforcement, rock undergoes elastic-plastic deformation. This displacement induces an amount of pressure on the reinforcement system which increases the tension force in the system. Hence, more load is transferred to rock by truss bolt. This increase in load continues to reach an equilibrium in which the stress in rock will be equal to the applied pressure by reinforcement. This effect of reinforcement system prevents some areas of rock from failure and plastic deformation. Figures 4.9 to 4.11 show the effects of the three truss bolt patterns on the plastic behaviour of rock before and after installing reinforcement system.

It can be seen that truss bolt prevents the plastic points to propagate around

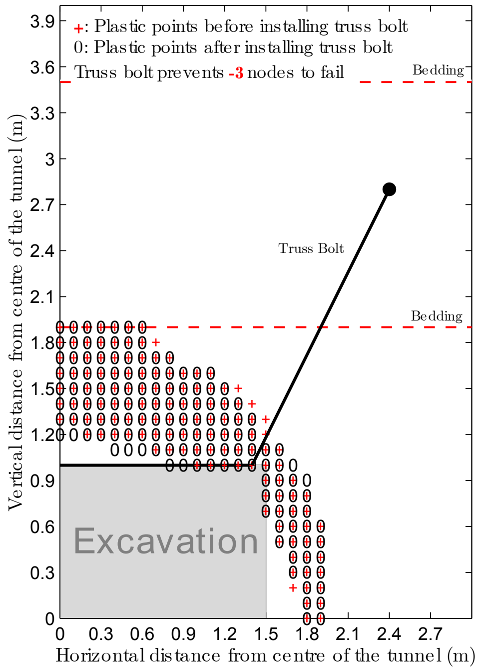


Figure 4.9 Plastic points before and after installing Pattern 1.

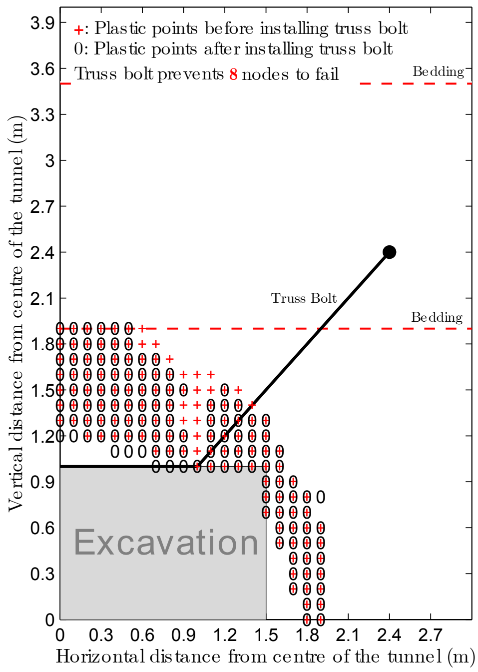


Figure 4.10 Plastic points before and after installing truss bolt pattern 2.

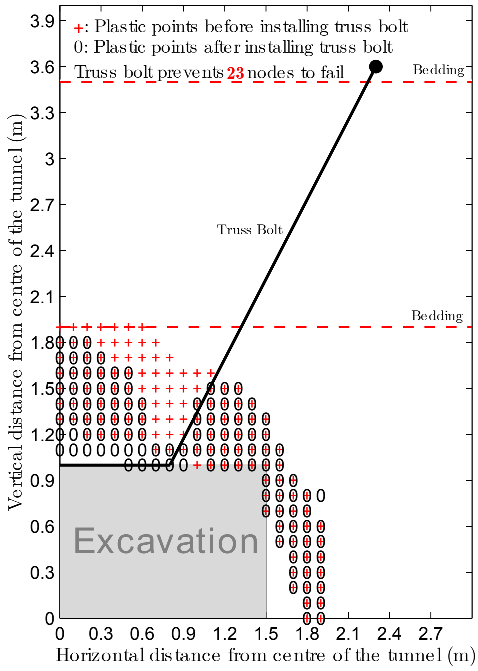


Figure 4.11 Plastic points before and after installing truss bolt pattern 3.

an underground excavation. Effects of truss bolt on the plastic behaviour of the surrounding rock is deeply related to the parameters of the truss bolt pattern. This difference can be well monitored by comparing the effects of two truss bolt patterns in Figures 4.9 and 4.11. When inclined bolts are positioned well in the failed area, truss bolt shows a good practice in reducing the number of plastic points (Figure 4.11). On the other hand, truss bolt pattern in Figure 4.9 prevents some points to fail around the corner of the excavation but more points fail above the roof of the tunnel. Another conclusion to these results is that, pattern 3 prevents an arch shape area to fail above the roof (Figure 4.11). In fact, this area is completely similar to the results of SSM where the area of the major reinforcing effect of truss bolt in Figure 4.8 for pattern 3 can be fit in an arch shape area.

4.4 Effects of Truss Bolt System on Cutter Roof Failure

Cutter roof is a kind of failure which normally happens in laminated roof rock, especially when the immediate roof layer is relatively weak (Su and Peng 1987; Gadde and Peng 2005). Cutter roof failure applies a huge amount of pressure on support or reinforcement system which makes a massive block of rock to fail. In some cases, re-opening and stabilizing a site after cutter roof failure has no efficient solution and the site would be abandoned (Su and Peng 1987).

Mechanism of cutter roof failure is well discussed in the literature (Su and Peng 1987; Gadde and Peng 2005; Coggan et al. 2012; Altounyan and Taljaard 2001). After excavating an underground excavation in laminated rock, shear cracks start to appear near the rib area. During the time after excavation, cracks propagate with an angle depending on the in-situ stress distribution. High horizontal stress causes low angle shear crack propagation from the roof meanwhile high vertical stress causes shear cracks to propagate under an angle close to perpendicular to the roof (Su and Peng 1987). Other factors mentioned by researchers are entry width, relative stiffness between coal and the immediate layer, ground surface topography, geological anomalies, separation of bedding and gas pressure(Su and Peng 1987; Gadde and Peng 2005). When fractures reach a bedding plane or an area above the rock bolt anchorage, because of overburden pressure and weight of the rock, a massive block separates from the roof (Su and Peng 1987). Figure 4.12 illustrates a schematic procedure of crack propagation and cutter roof failure.

As mentioned by several researchers, conventional systematic rock bolt patterns are not able prevent this type of failure and the whole system together with a huge block of rock fail into the excavated area (Su and Peng 1987; Gadde and Peng 2005). In contrast, truss bolt system has shown very good practice in controlling the cutter roof failure (Stankus et al. 1996). In this section, effects of different truss bolt patterns on preventing cutter roof failure is investigated. Mechanism of truss bolt on preventing cutter roof failure can be studied by monitoring horizontal movement of the immediate roof layer and shear crack propagation in models under high horizontal or vertical in-situ stresses.

4.4.1 Shear Crack Propagation

One of the main limitations of FEM method is in modelling fracture growth (Jing 2003). Capturing crack propagation is only possible by employing relatively new methods such as enriched FEM and generalized FEM (Deb and Das 2011; Duarte et al. 2000). Using these techniques in a compressive model of underground excavation with complex geometry is hard and needs extensive calculation costs. This problem becomes more complicated when the model contains pretensioned elements (rock bolts) and geological features such as bedding planes.

Based on Mohr-Coulomb failure criterion, shear failure can happen under compressive stresses when the maximum shear stress reaches the critical value

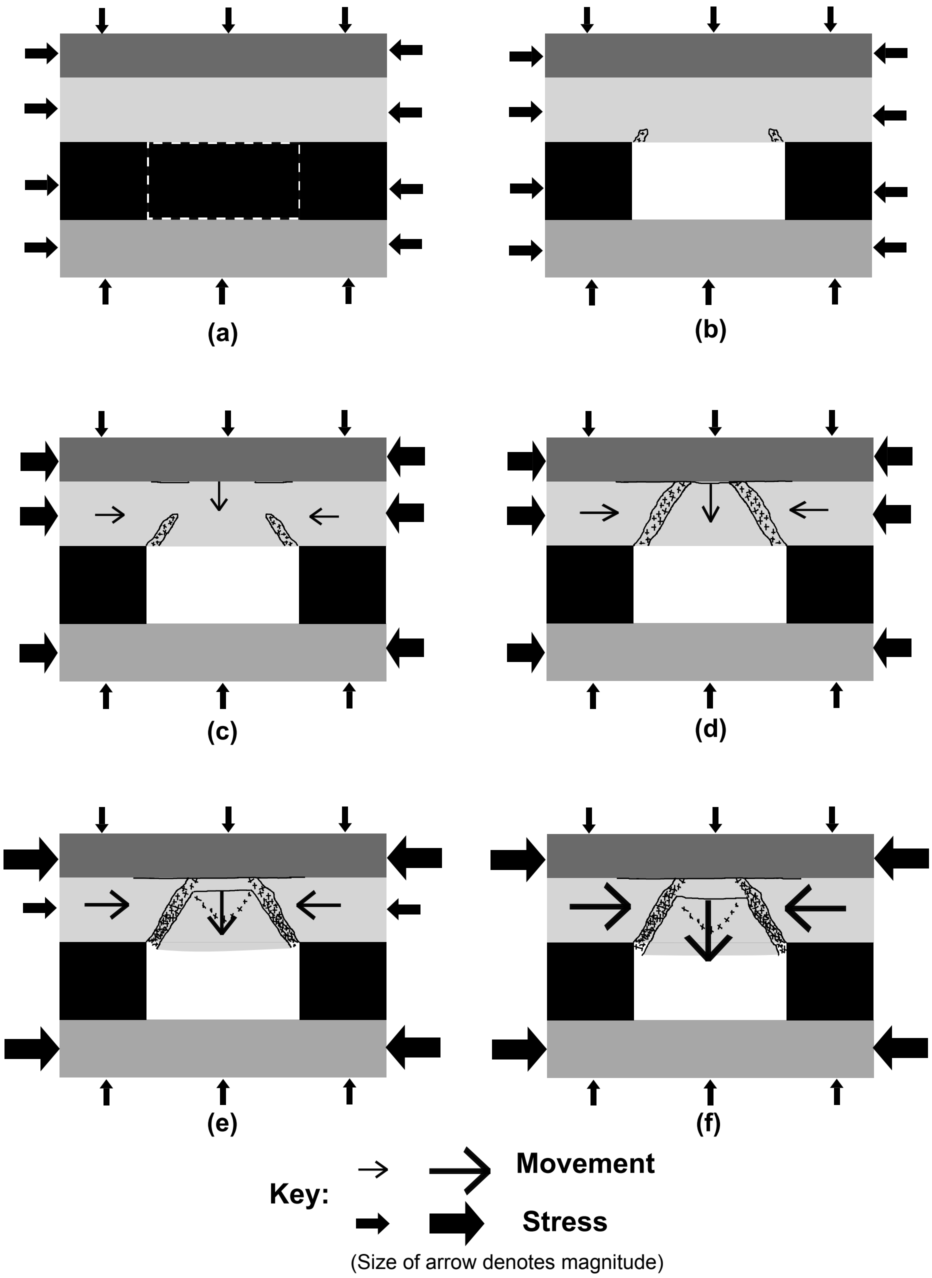


Figure 4.12 Schematic progressive shear and cutter roof failure, after Altounyan and Taljaard (2001) .

defined by the Mohr-Coulomb yield function. After shear failure the rock behaviour could be assumed to be plastic. This failure could thus be captured using an elastic-plastic material model in FEA. Hence the yielded areas resulted from elastic-plastic FEA, provided that the stresses are compressive, could be assumed to represent the shear crack propagation. However, if the failure occurs in tension, due to the separation in material, the post failure behaviour could not be captured appropriately using an elastic-plastic FEA.

To monitor the effects of truss bolt on cutter roof, progressive failure (shear crack propagation) around the tunnel is modelled using a simplified interactive approach. For this purpose, the model is solved with elastic-plastic material model once, and then the *most likely area to yield* is found with respect to the Mohr-Coulomb yield function and SSM factor (Equation 4.4).

As discussed in Section 4.3.3 changes in radius of Mohr's circle is always smaller than the required change in the amount of pressure to satisfy the failure criterion ($x_r < x_c$). From Equation 4.4, SSM equal to one ($x_r = 0$) denotes failure (Figure 4.13).). Increasing load in rock material results in changing the radius of Mohr's circle and causes an increase in the number of failure points in rock. Modelling this progressive failure in rock is possible by gradually increasing values of x_r and finding the yielded points for the new stress condition corresponding to the new x_r. This approach is essentially a linear extrapolation which helps us estimate shear crack propagation. The increase in the amount of x_r can be defined through several increments (I_n) where

$$\text{SSM} - 1 = I_n \tag{4.5}$$

In this equation SSM = 1 represents yielding. By replacing the definition of

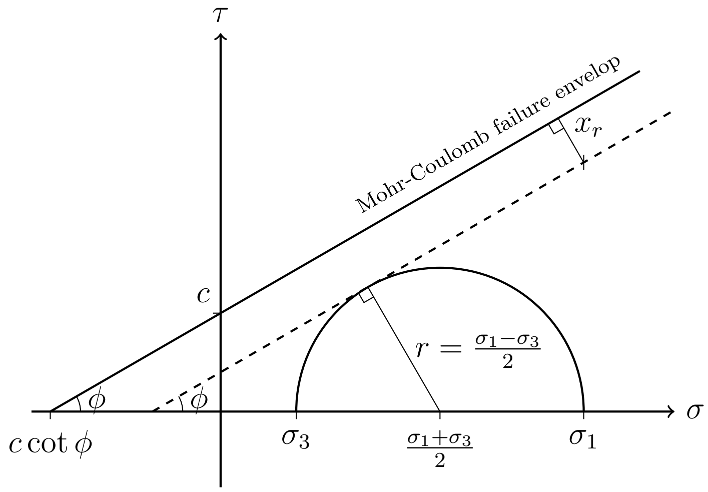


Figure 4.13 Shortest distance from Mohr's circle to failure envelope.

SSM (Equation 4.4) in Equation 4.5, different increments can be derived as

$$I_n = x_r/r \tag{4.6}$$

This equation identifies the locations where rock will undergo shear failure at increment I_n. I_n equal to zero interprets $x_r = 0$ which shows the area of the failure under current loading condition. Increasing the amount of I_n shows propagation of yielded as loads increase. Four possible conditions of rock for different increments from Equation 4.5 are shown in Figure 4.14. It can be seen that after excavating an underground excavation, rock undergoes the elastic deformation (condition: elastic and safe). By increasing the load, the Mohr's circle becomes bigger and as it touches the failure envelope, rock yields (condition: yield in increment 0). Now, by considering various increments (I_n in Equation 4.5), smaller Mohr's circles will also undergo failure and plastic deformation. Points corresponding to these smaller Mohr's cir-

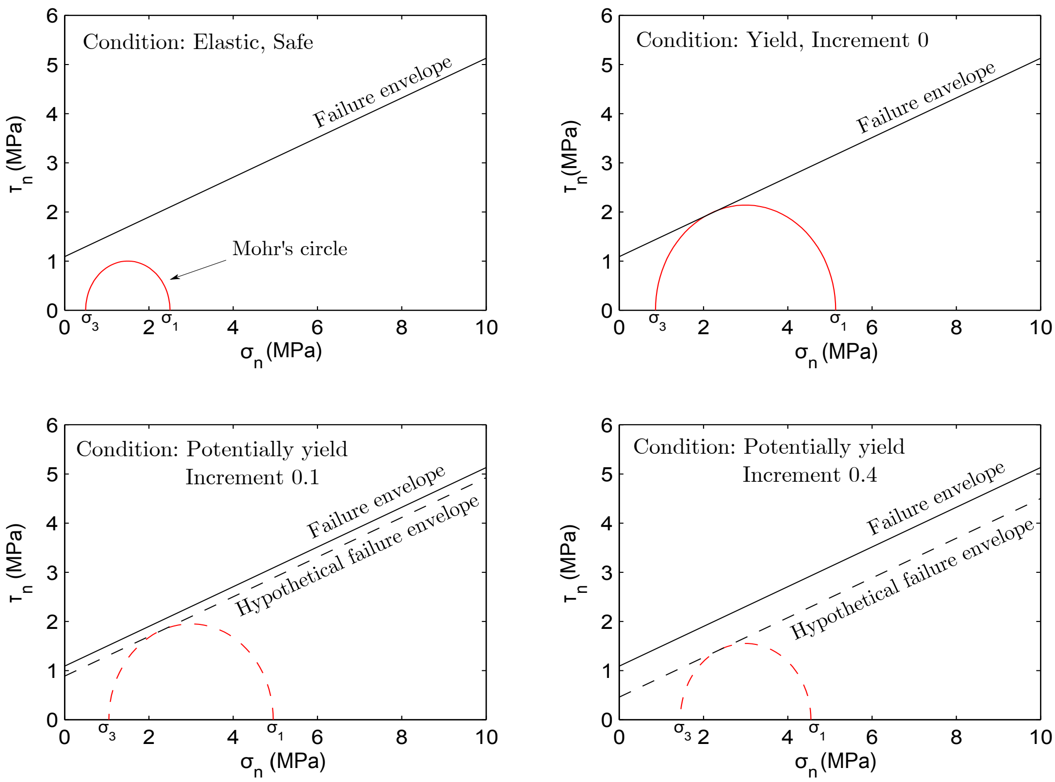


Figure 4.14 Different stages of rock behaviour during the analysis.

cles can be considered as the potentially yield area for the given increment (condition: potentially yield, increment 0.1). By increasing the increment, more points undergo the plastic deformation and a bigger area is considered as the potentially yield area (condition: potentially yield, increment 0.4).

It should be noted that the resulting yielded areas for different increments do not necessarily mean that these areas are yielded but shows the pattern of potentially yielded area (shear cracked area) in different time spans after excavation.

With respect to the definition of cutter roof by Su and Peng (1987), when shear cracks reach the plane of weakness, cutter roof happens. Four different increments have been chosen to represent the shear cracks just after excavation ($I_n = 0$) to cutter roof failure (when shear cracks reach the plane of weakness). Two different in-situ stress distributions have been modelled. Results showed that when the horizontal stress is high ($\sigma_v = \frac{1}{2}\sigma_h$) shear cracks tend to propagate with a sharp angle to the roof of the opening. Stars in Figure 4.15 show yielded points for different increments. Different increments are shown by different colours. The hypothetical lines in this figure show the areas of yielded rock for different increments. As it can be seen, at the final increment ($I_n = 0.015$) shear cracks reach the plane of weakness and the cutter roof happens. Similarly, using the same method for a tunnel under high vertical in-situ stress ($\sigma_v = 2\sigma_h$) the pattern of shear crack propagation can be obtained as shown in Figure 4.16. Comparing these two figures illustrates that the angle of shear crack propagation and shape of the unstable block is deeply related to the condition of the in-situ stress. In high vertical stress, shear cracks propagate at an approximately right angle to the roof while in high horizontal stress this angle is less than 90°. Su and Peng (1987) on the basis of numerical analysis, using FEA and safety factor, together with field observations reported the same pattern of cutter roof in high vertical and horizontal stress conditions.

Figures 4.17 to 4.22 show results of installing three different truss bolt patterns on two identical tunnels under high horizontal and vertical in-situ stresses. Comparing these results with Figures 4.15 and 4.16 (pattern of shear cracks before installing truss bolt), it can be concluded that truss bolt system reduces the possibility of cutter roof by controlling shear crack prop-

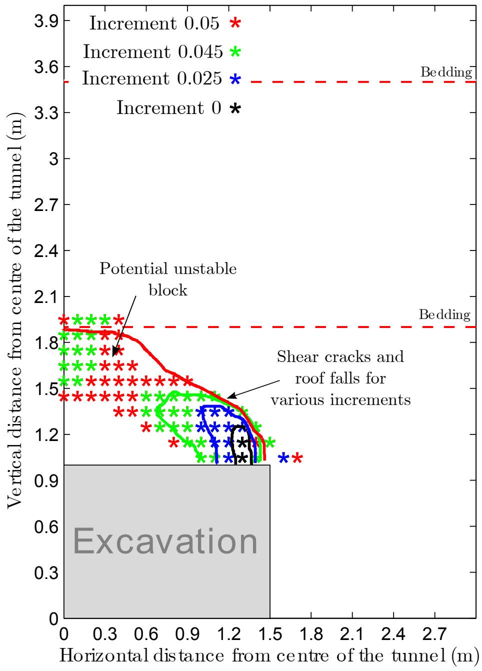


Figure 4.15 Pattern of shear crack propagation ($\sigma_v = \frac{1}{2}\sigma_h$).

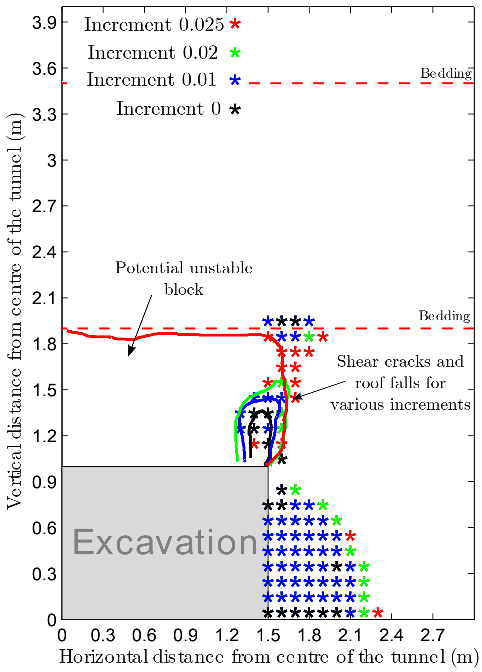


Figure 4.16 Pattern of shear crack propagation ($\sigma_v = 2\sigma_h$).

agation. It appears that truss bolt system by having inclined bolts near the area of initial shear cracks (around the corners of the roof) prevents continuous cracking and reduces the possibility of cutter roof. It has been shown in Section 4.3.3 that, because of the pretension force and induced compressive stress around the inclined bolts, a reinforced area will be created near the corners of the roof. In high vertical stress, where inclined bolts are well located at the area of shear crack propagation, the applied compressive stress by inclined bolts prevents continues shear crack propagation. In addition to this, investigating the results of SSM factor around truss bolt system shows another major reinforced area which is similar to an arch shape between inclined bolts above the roof (Figure 4.8). Comparing patterns of shear cracks before (Figure 4.15) and after installing truss bolt (Figures 4.17 to 4.19) in high horizontal stress shows that truss bolt prevents propagation of cracks at areas near blocking points and above the roof. In fact, this area is identical to the produced reinforced arch area by truss bolt.

Results of installing different truss bolt patterns on preventing cutter roof illustrate that, depending on design parameters of truss bolt and in-situ stress distribution, effectiveness of the system on preventing shear crack propagation varies. It can be seen that in high vertical stress (Figures 4.20 to 4.22), pattern 2 shows the best application. Inclined bolts in this pattern exactly pass through the initial area of cracking and, by reinforcing this area, this pattern prevents further crack propagation (Figure 4.21). Figure 4.22 shows that pattern 3 is also able to reduce the possibility of cutter roof in this in-situ stress condition. On the other hand, inclined bolts in pattern 1 are

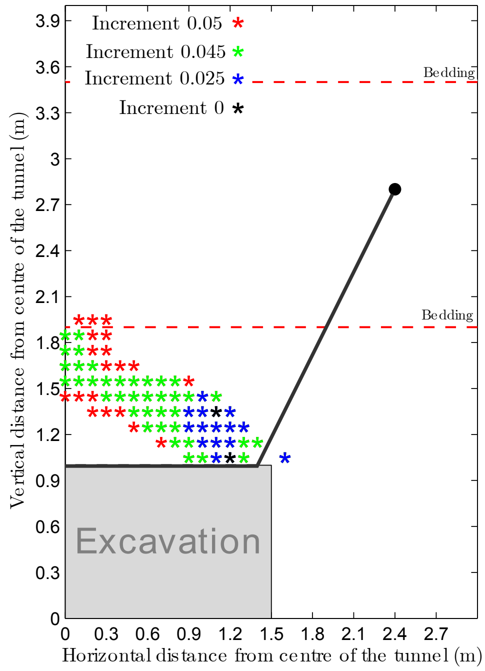


Figure 4.17 Truss bolt pattern 1 in high horizontal in-situ stress.

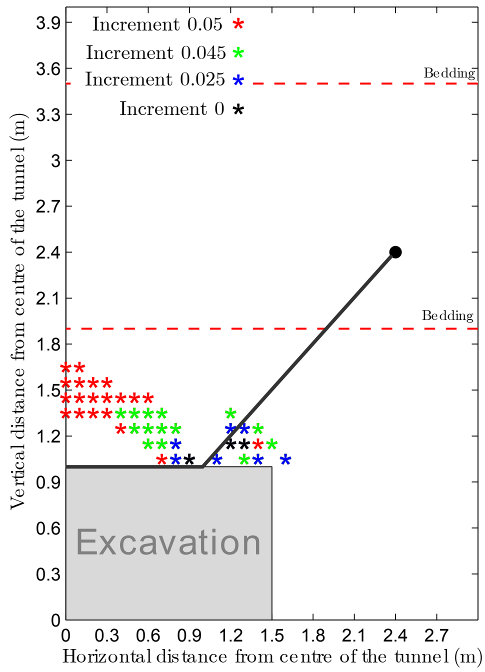


Figure 4.18 Truss bolt pattern 2 in high horizontal in-situ stress.

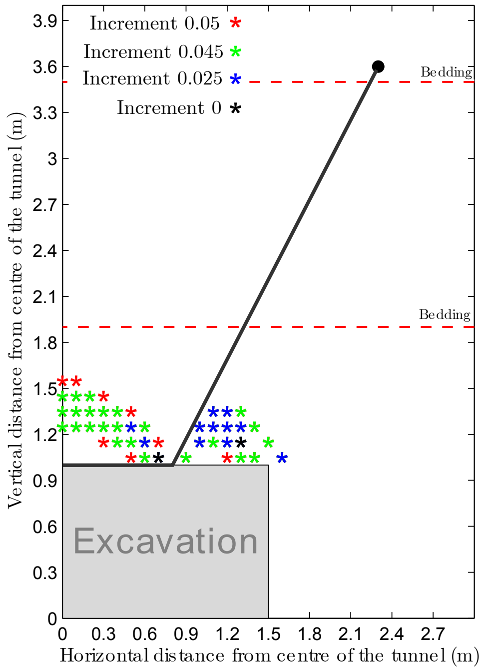


Figure 4.19 Truss bolt pattern 3 in high horizontal in-situ stress.

located behind the area of initial cracking and even push the crack propagation pattern slightly towards the middle of the roof instead of controlling it (Figure 4.20).

Comparing results of installing different truss bolts on a tunnel under high horizontal stress shows that patterns 2 and 3 prevent shear crack propagation to reach the plane of weakness (Figures 4.18 and 4.19). Whilst pattern 1 does not have any significant effect on preventing cutter roof and shear cracks reach the plane of weakness around the middle of the roof (Figure 4.17). This is probably because of the position of inclined bolts in pattern 1 which, similar to Figure 4.20 in high vertical stress, are located behind the area of initial crack propagation. As discussed in Section 4.3.3, pattern 3 by having

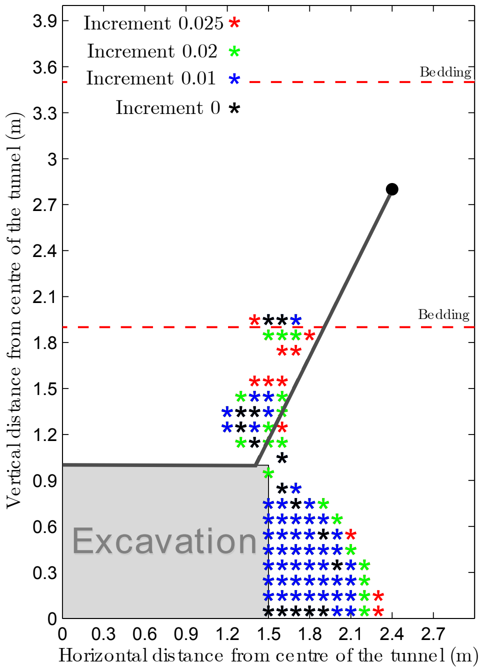


Figure 4.20 Truss bolt pattern 1 in high vertical in-situ stress.

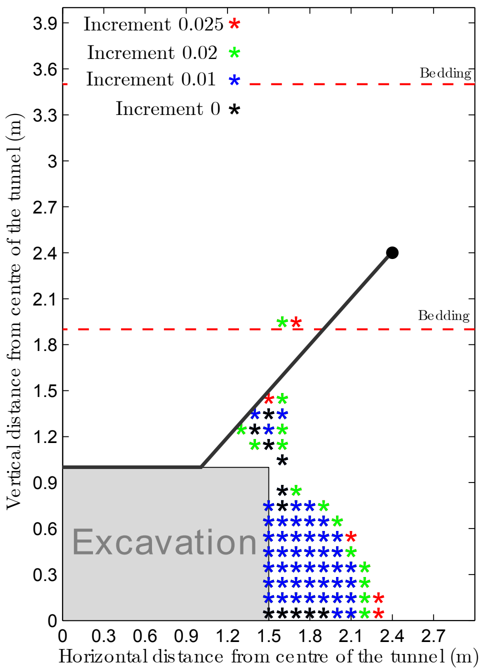


Figure 4.21 Truss bolt pattern 2 in high vertical in-situ stress.

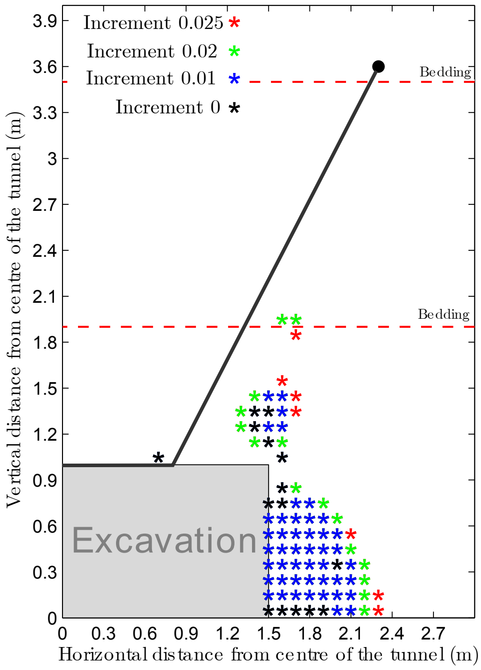


Figure 4.22 Truss bolt pattern 3 in high vertical in-situ stress.

long inclined bolts and short tie-rod length produces a stronger reinforced arch compared to other patterns. This enables it to effectively control the shear crack propagation above the roof and show the best response.

4.4.2 Slip On the First Bedding Plane

In numerical modelling, slip on the first bedding plane can be precisely studied by monitoring the relative displacement of bedding surfaces. This parameter can be interpreted as the relative horizontal movement of the immediate rock layer. Figures 4.23 and 4.24 show the relative horizontal displacement between surfaces of the first bedding plane before and after installing truss bolt patterns on two different in-situ stress distributions (high vertical

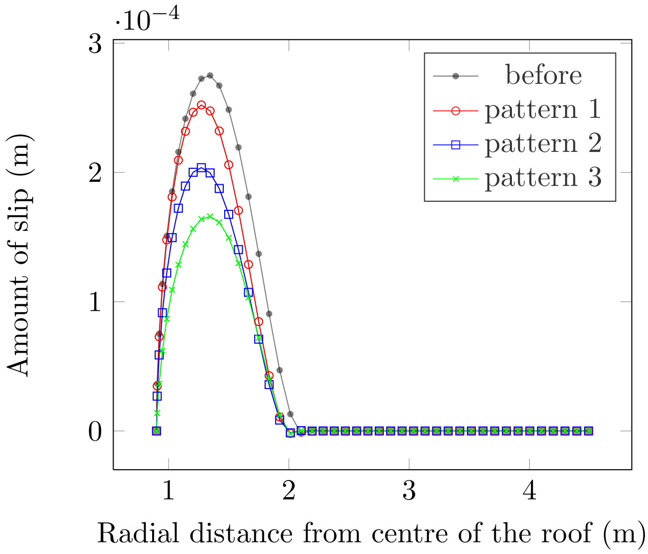


Figure 4.23 Amount of slip on the first bedding plane for different truss bolt patterns ($\sigma_v = 2\sigma_h$).

$\sigma_v = 2\sigma_h$ and high horizontal $\sigma_v = \frac{1}{2}\sigma_h$ stresses). These figures show that truss bolt reduces the amount of horizontal movement in the immediate rock layer in both models.

A closer inspection at Figure 4.23 reveals that, in high vertical stress the major area of slip before installing truss bolt is approximately above the roof. This slippage approaches zero near the rib area (radial distance of 2 m). After installing different truss bolt patterns, pattern 3 shows the best response which is due to the location of the inclined bolts that pass through the major area of the slip. By increasing the length of tie-rod, effectiveness of truss bolt reduces dramatically and pattern 1 shows relatively little effect on this factor.

In contrast, when horizontal stress is high, the slippage on the first bedding plane reaches a peak above the roof and extends to almost 1.5 times of

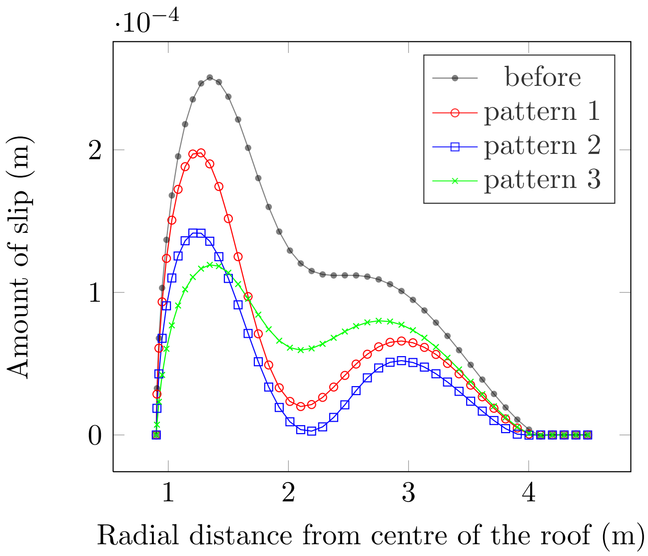


Figure 4.24 Amount of slip on the first bedding plane for different truss bolt patterns ($\sigma_v = \frac{1}{2}\sigma_h$).

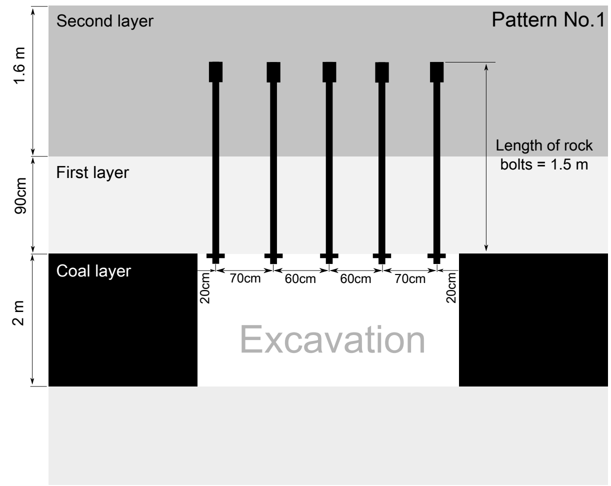
the span of the opening (radial distance of 4 m) and smoothly approaches zero after this distance (Figure 4.24). To prevent the cutter roof failure, horizontal displacement, especially above and behind the rib area, need to be controlled. Figure 4.24 shows that for the area above the tunnel short span truss bolt has the best effect (similar to results of high vertical stress, Figure 4.24). However, for the area around corners of the roof (radial distance of 2 m) pattern 2 shows the best results. In this area pattern 1 and 2 are more effective than pattern 3 due to having inclined bolts passing through this area. Also, angle of inclined bolts in pattern 2 is another reason for effective application of this pattern where 45° inclined bolts produce a larger horizontal component than 60° degree for the same amount of pretension. This component is in the opposite direction to the horizontal stress and reduces the effect of this stress.

4.5 Comparison Between Truss Bolt and Systematic Rock Bolt

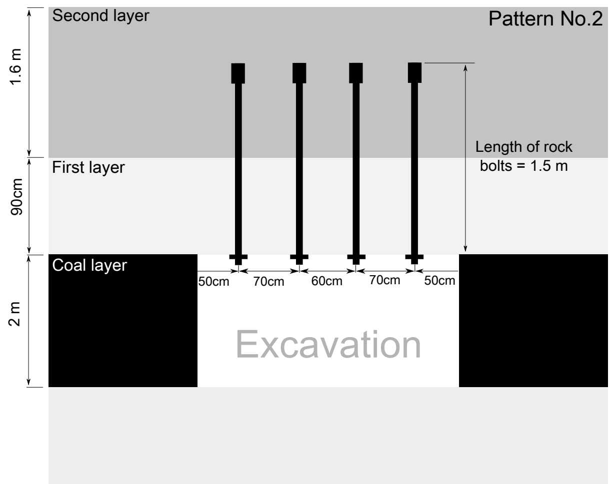
4.5.1 Systematic Rock Bolt Pattern

Comparing two different reinforcement systems needs considering several factors at the same time such as installing procedure, length of drill-holes, total length of rock bolts, time of installation, required number of workers, total price and so on. Considering all of these factors is out of the contents of this study. To have a simple and fair comparison between truss bolt and systematic rock bolt systems, several conditions can be made to design a systematic rock bolt pattern which is relatively comparable to the truss bolt system. These conditions result in couple of design controlling factors. Here, we use two factors as *a*) total length of the drill-wholes and *b*) sum of the tension in all rock bolts to design the systematic rock bolt pattern. Using truss bolt pattern 3 as the reference truss bolt pattern[1], total length of rock bolts and total tension in rock bolts for the systematic rock bolt pattern can be chosen. The amount of pretension for each rock bolt in the systematic rock bolt will be the total amount of tension divided by the number of rock bolts in the pattern. Number of rock bolts is judged by spacing of rock bolts which have been chosen to meet the conditions in Lang's empirical design criteria (Lang 1961). Length of rock bolts is chosen as equal to total length of drill-wholes divided by number of rock bolts. In addition to this pattern,

[1]Pattern 3 has the greatest amount of material and longest drill-hole length amongst three truss bolt patterns which make it the least economic pattern.



(a) Systematic rock bolt pattern derived from Lang (1961) recommendations



(b) Systematic rock bolt pattern using design control criteria

Figure 4.25 Two systematic rock bolt patterns.

another systematic rock bolt pattern has been modelled just on the basis of Lang's empirical design criteria (Section 2.5) and using the same amount of total tension for the system. These patterns are shown in Figure 4.25.

The differences in mechanism of truss bolt and systematic rock bolt systems can be understood by comparing the effects of these two systems on stability of an underground excavation by investigating the stability indicators. It should be noted here that the purpose of this comparison is to examine the difference in *mechanism* of truss bolt systems and systematic rock bolts not to compare the applicability of these systems on controlling stability of an underground excavation.

4.5.2 Stress Safety Margin (SSM)

Effect of systematic rock bolt on the SSM is shown in Figures 4.26 and 4.27 for two different patterns. It can be seen that the induced reinforced areas by both systematic rock bolt patterns are mainly above the roof and between the head and anchorage area. These figures show a little difference in response of these two systematic rock bolt on the area above the roof. This shows that greater number of rock bolts with the same amount of tension does not necessarily produce a better reinforced area above the roof. On the other hand, systematic rock bolt pattern 1 (Figure 4.26) by having rock bolts near the corners of the roof has a slightly better respond on reinforcing the sides of the tunnel.

Figure 4.28 shows the major reinforced areas and areas of unfavourable effects of the systematic rock bolt pattern 2. Contour lines in this figure is the same as Figure 4.8 in Section 4.3.3. Figure 4.28 illustrates that systematic rock bolt produces a beam shape reinforced area above the roof. In fact, this reinforced beam confirms the beam building theory of rock bolting which has been discussed in Section 2.2.

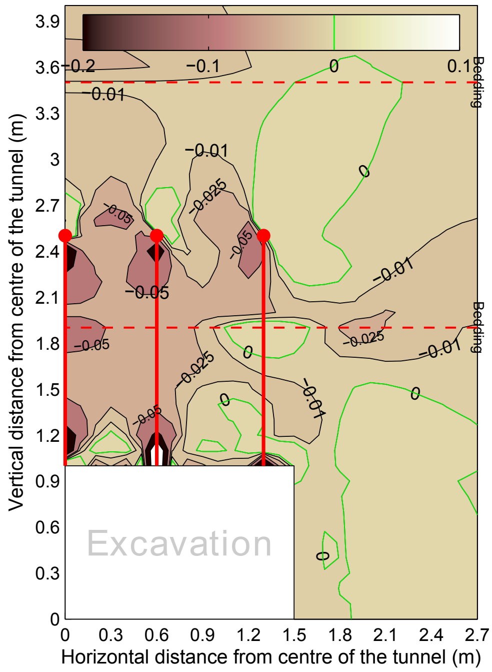


Figure 4.26 ΔSSM by systematic rock bolt pattern 1.

Comparing results of installing truss bolt and systematic rock bolt systems on SSM around a tunnel shows a significant difference in the mechanism of these systems. Truss bolt system is able to reinforce a trapezoid area between the inclined bolts and above the roof. The major reinforced area in this trapezoid is an arch shape structure which is located between the blocking points of the truss bolt system (Figure 4.8). In contrast, the produced minor and major reinforced area around the systematic rock bolt are about the same shape. This area is like a beam shape structure between the anchorage area and heads of the rock bolts and covers the area above the roof (Figure 4.28). Also, comparing results of installing truss bolt patterns 1 and 2 (Figures 4.5 and 4.6) with systematic patterns 1 and 2 (Figures 4.26 and 4.27) shows that

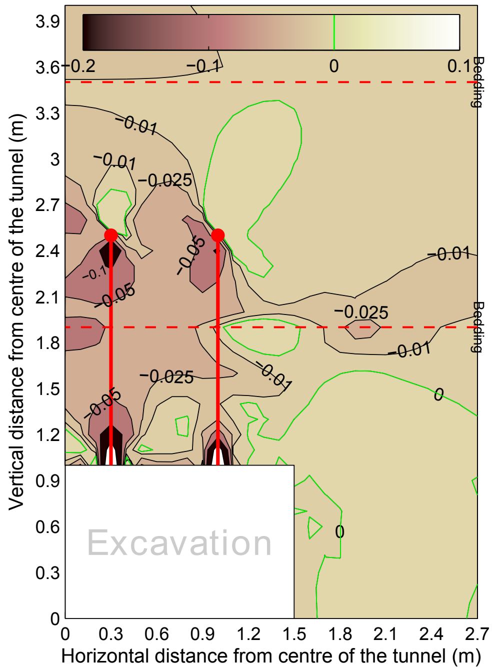


Figure 4.27 ΔSSM by systematic rock bolt pattern 2.

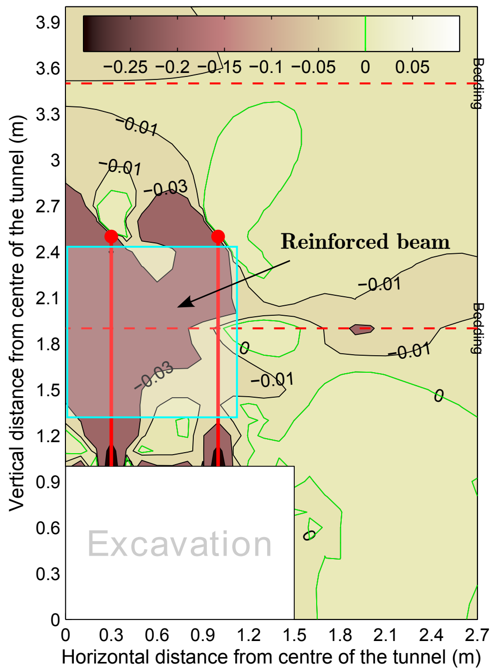


Figure 4.28 ΔSSM and reinforced areas with systematic rock bolt pattern 1.

truss bolt systems with short length of inclined bolts are able to produce a better reinforced area around the inclined bolts and above the abutments of the tunnel.

Both truss bolt and systematic rock bolt systems show an unfavourable effect on the sides of the opening. The shape of this area changes with respect to the pattern of the reinforcement system. In general, a truss bolt pattern or systematic rock bolt pattern with rock bolts or inclined bolts near the corners of the roof (Figures 4.5 and 4.26) shows less unfavourable effect on the side rock in comparison with patterns which have rock bolts or inclined bolts around the middle of the roof (Figures 4.7 and 4.27).

4.5.3 Plastic Point Distribution

Figures 4.29 and 4.30 show the plastic points before and after installing reinforcement systems. It can be seen that systematic rock bolt has very good application in controlling the plastic behaviour of the rock above the roof. This is probably because of having rock bolts at the major area of the plastic behaviour above the roof. The area which systematic rock bolt prevents the failure in rock is quite similar to the major reinforced area shown in Figure 4.28 which is like a beam shape structure.

Comparing the total amount of reduction in the number of plastic points for two different systematic rock bolt patterns reveals that systematic rock bolt pattern 2 (61 points) is more successful than pattern 1 (54 points). Similar to the result of SSM in Section 4.5.2, these figures show that more rock bolts does not necessarily produce a better reinforcing effect on the roof of the

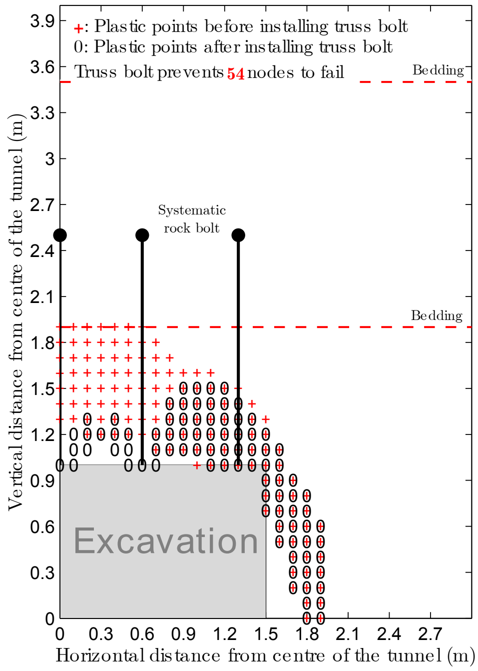


Figure 4.29 Plastic points before and after systematic rock bolt pattern 1.

excavation. This is because of the different locations and the amount of tension of rock bolts in these patterns. These results highlights the importance of the location and amount of tension in rock bolts rather than the number of bolts in design of systematic rock bolt pattern to achieve an optimum design.

Comparing results of installing truss bolt and systematic rock bolt systems on the failure of the rock material around the tunnel shows the difference in shape of the reinforced areas around two different systems. Figure 4.11 shows an arch shape area above the roof while systematic rock bolt in Figure 4.30 reinforces a beam shape area above the roof. Figure 4.30 also shows that systematic rock bolt is more successful in controlling plastic behaviour of the area above the middle of the roof while a truss bolt pattern with long tie-rod

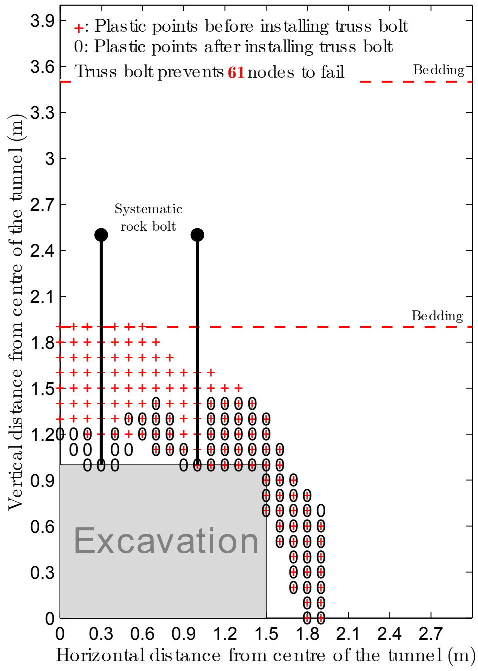


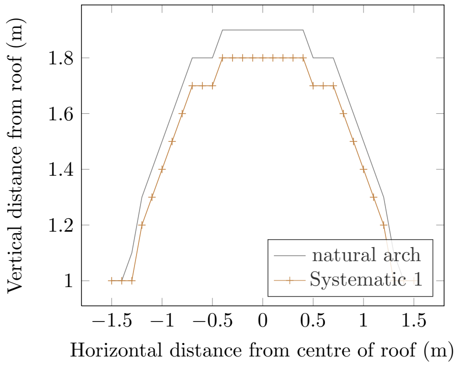
Figure 4.30 Plastic points before and after systematic rock bolt pattern 2.

(Figure 4.9) does not have any significant effect on this area.

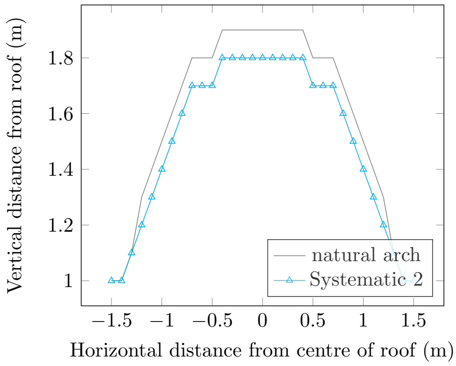
4.5.4 Area of Loosened Rock And Roof Deflection

Similar to truss bolt systems, systematic rock bolt systems are able to reduce the area of the loosened rock beneath the natural roof arch. Figure 4.31 shows reinforced arches before and after installing two different systematic rock bolt patterns. It can be seen that both systematic rock bolt patterns show the same response on the area above the roof, but pattern 1 has a slightly better application on the area near the corners of the roof.

Table 4.6 shows the amount of reduction at the area of the loosened rock after installing two systematic rock bolt patterns. The amount of reduction in the



(a) Systematic rock bolt pattern 1



(b) Systematic rock bolt pattern 1

Figure 4.31 Reinforced roof arch before and after installing systematic rock bolt

area of the loosened rock for both patterns are about the same. These results, similar to the results of plastic points and SSM (Sections 4.5.2 and 4.5.3), indicate that although the number of rock bolts in pattern 1 is greater than pattern 2 but the response of these systematic rock bolt patterns are about the same.

Table 4.7 shows the reduction in the area of roof deflection after installing two systematic rock bolt patterns. It can be seen that pattern 1 has better response in controlling the vertical deformation of the roof in comparison with pattern 2. This good practice of pattern 1 is probably because of having one vertical bolt right at the middle of the roof which can cover the major area of the vertical deformation in hydrostatic in-situ stress distribution.

Investigating the results of installing truss bolt and systematic rock bolt on the position of the reinforced arch and area of deflection reveals that truss bolt pattern 3 (which has the best result for these stability measures among

Table 4.6 Reduction in the loosened area after installing two systematic rock bolt patterns

Different systematic rock bolt patterns	Reduction in the loosened area (cm^2)
Pattern 1	2700
Pattern 2	2500

Table 4.7 Reduction in the roof deflection for two systematic rock bolt patterns

Systematic rock bolt patterns	Reduction in roof deflection (cm^2)
Pattern No.1	2.62
Pattern No.2	2.42

truss bolt patterns) is more capable of reducing the area of loosened rock than systematic rock bolt patterns. On the other hand, both systematic rock bolt patterns show better response in reducing the area of deflection on the roof than truss bolt pattern 3. In fact, systematic rock bolt decreases the vertical displacement at the roof more than truss bolt system by forming an artificial reinforced beam at the loosened area above the roof that can carry the load. However, truss bolt system makes an arch shape reinforced rock by moving the reinforced roof arch towards the roof. A reason for good application of truss bolt system on reducing the area of loosened rock is that, because of the angle and length of inclined bolts, the anchor points in truss bolt are far away from the loosened area, i.e. inclined bolts are anchored in a safe area. But in systematic rock bolt, anchorage area is just above the roof and close to the loosened area. This makes systematic rock bolt just able to reinforce the loosened area (by applying compressive stress and making a reinforced beam) rather than changing the position of the reinforced arch.

4.5.5 Cutter Roof Failure

Shear Crack Propagation: Figures 4.32 to 4.35 show the pattern of shear crack propagation for two in-situ stress distributions and two patterns of systematic rock bolt. The method and various increments for each in-situ stress distribution are identical to Section 4.4.1. It can be seen that in high horizontal in-situ stress both systematic rock bolt patterns successfully control the crack propagation and cracks does not reach the first bedding plane (Figures 4.32 and 4.33). This is because the reinforcing effect of vertical rock bolts on the area above the roof (beam shape structure), which is located at the major area of cracking, and prevents the shear crack propagation.

In high vertical in-situ stress (Figures 4.34 and 4.35), both systematic rock bolt patterns prevent shear cracks to reach the bedding plane. It can be seen that, despite having a vertical bolt at the corner of the roof in pattern 1, pattern 2 shows slightly better response on controlling the shear crack propagation in this case.

Slip on the First Bedding Plane: Figures 4.36 and 4.37 show the effect of installing systematic rock bolt patterns on the horizontal movement of the immediate roof layer. It can be seen that systematic rock bolt pattern 1 shows a smoothly better response on controlling slip on the first bedding plane in high horizontal in-situ stress compare to pattern 2 (Figure 4.36). The difference between results of these two patterns becomes greater around the radial distance of 2 m (above the rib line). This is because of a vertical bolt in pattern 1 which is exactly located at this area. Vertical rock bolts

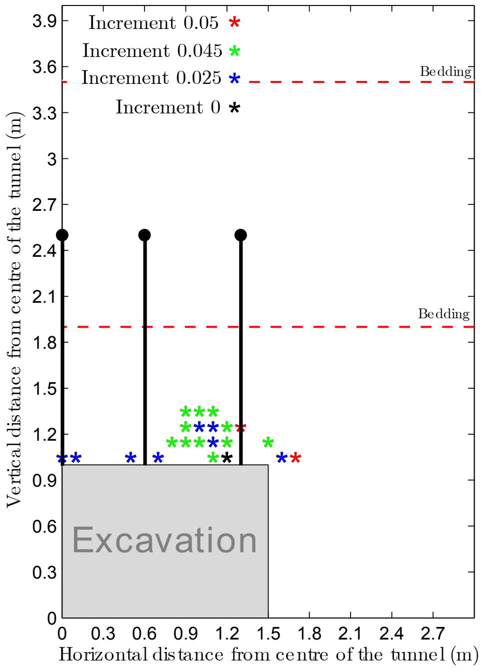


Figure 4.32 Crack propagation for pattern 1 ($\sigma_v = \frac{1}{2}\sigma_h$).

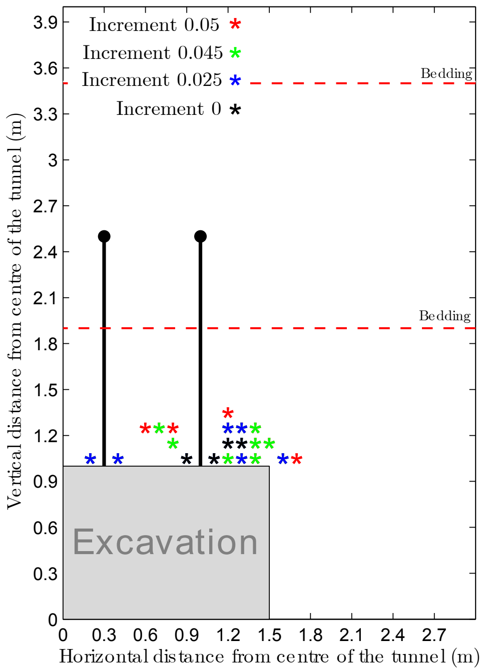


Figure 4.33 Crack propagation for pattern 2 ($\sigma_v = \frac{1}{2}\sigma_h$).

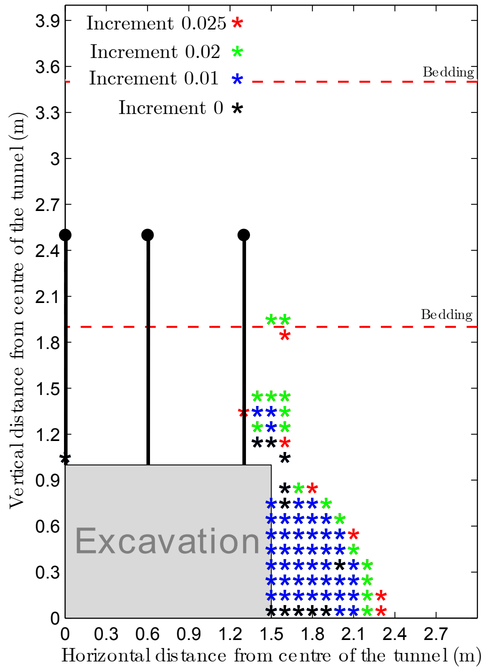


Figure 4.34 Crack propagation for pattern 1 ($\sigma_v = 2\sigma_h$).

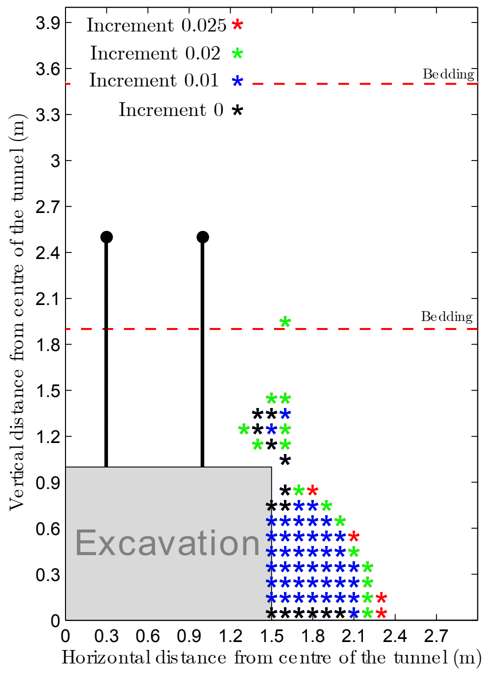


Figure 4.35 Crack propagation for pattern 2 ($\sigma_v = 2\sigma_h$).

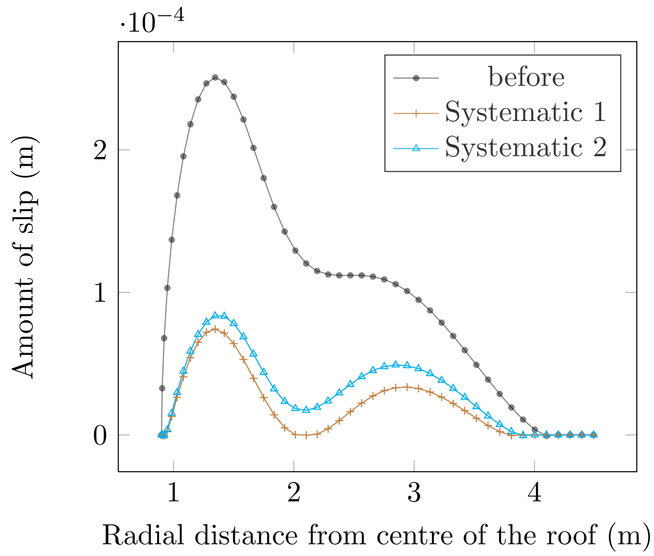


Figure 4.36 Effect of systematic rock bolt patterns on slip on the first bedding plane ($\sigma_v = \frac{1}{2}\sigma_h$)

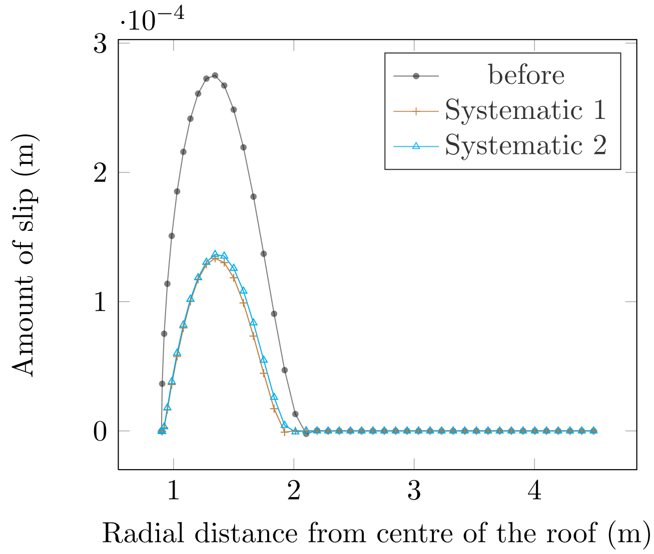


Figure 4.37 Effect of systematic rock bolt patterns on slip on the first bedding plane ($\sigma_v = 2\sigma_h$)

by applying compressive stress to the bedding surfaces increase the amount of normal stress and prevent sliding (Section 3.4). On the other hand, when the vertical in-situ stress is high, the two systematic rock bolt patterns have the same response on this measure.

Comparing results of installing truss bolt and systematic rock bolt systems on shear crack propagation shows different mechanism of these two systems on controlling this factor. In high horizontal in-situ stress systematic rock bolts (both patterns) are able to prevent shear crack propagation to reach the bedding plane by applying compressive stress and reinforcing the area between the anchorage and roof of the tunnel (beam shape area, Figures 4.32 and 4.33). But truss bolt system (especially pattern 3, Figure 4.19) reinforces the area around inclined bolts and top of the immediate roof layer (arch shape area). When vertical in-situ stress is high, both systematic rock bolt patterns show approximately the same response as truss bolt pattern 2 (Figure 4.21).

Figures 4.38 and 4.39 show the results of installing truss bolt and systematic rock bolt systems on slip on the first bedding plane at the same time. In Figure 4.38, when horizontal in-situ stress is high, systematic rock bolts is more capable of controlling slip on the area above the roof. When it comes to the area above the rib line (radial distance of 2 m), truss bolt pattern 2 shows a better effect than systematic rock bolt pattern 2. As discussed before, this is because of the inclined bolt in truss bolt system which passes through this area. Systematic rock bolt pattern 1 shows about the same response as truss bolt pattern 2 on this area by having a vertical rock bolt around the corner of the roof.

In high vertical in-situ stress (Figure 4.39) systematic rock bolt is more capable of controlling horizontal movement of the immediate layer by having

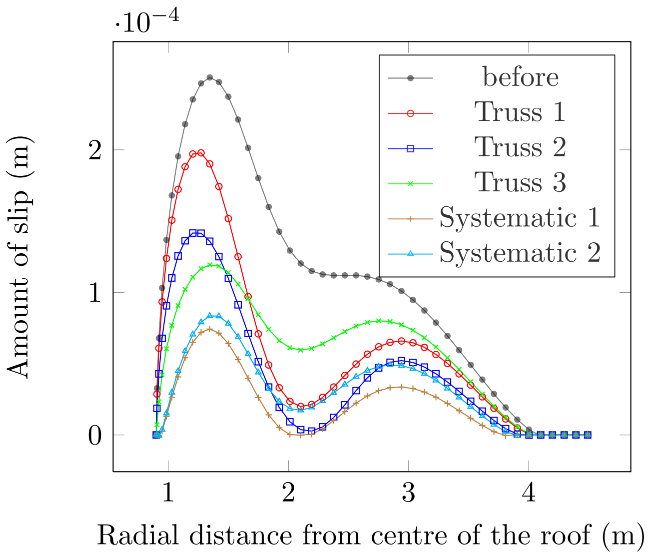


Figure 4.38 Comparing effects of truss bolt and systematic rock bolt on slip on the first bedding plane ($\sigma_v = \frac{1}{2}\sigma_h$).

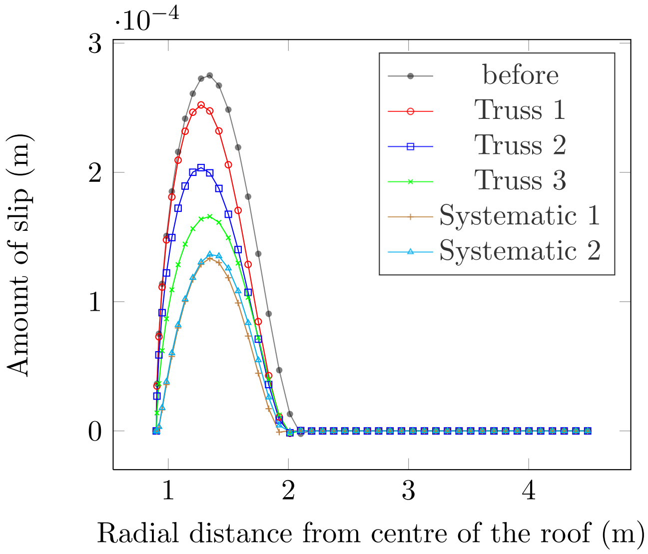


Figure 4.39 Comparing effects of truss bolt and systematic rock bolt on slip on the first bedding plane ($\sigma_v = 2\sigma_h$).

vertical bolt right at the major area of the slip. In this case, a truss bolt pattern which is more similar to systematic rock bolt (truss bolt pattern 3) shows the best result.

4.6 Discussion

The importance of a comprehensive consideration of all the design parameters and site variables can be concluded here. It has been shown that the shorter length of inclined bolts produce better reinforced area around the inclined bolts compared to longer bolts. If truss bolt system with short inclined bolts is located in the right place to prevent crack propagation in high vertical in-situ stress (by choosing suitable tie-rod length), it can effectively prevent the cutter roof failure. On the other hand, longer inclined bolts have the advantage of adequate length of anchorage in passive zone behind the rib line. The length of anchorage is a key parameter to determine the capacity of the system. If the applied load on truss bolt system exceeds the capacity of truss bolt, the whole block with truss bolt will fail.

The length, position and angle of inclined bolts are also important in controlling horizontal movement and the area of the loosened rock. If inclined bolts pass through the major area of slip (depending on in-situ stress distribution), the response of truss bolt on preventing horizontal movement increases significantly. The area of slip changes by changing in-situ stress condition. Results showed that medium length tie-rod locates the inclined bolts at the best possible location to prevent slip on the first bedding plane in high horizontal stress. Further to the importance of tie-rod length in truss bolt, choosing an angle closer to horizon would result in producing higher resisting force against high horizontal stresses. It should be mentioned that bolt angles less than 45 degree will result in significant reduction in the capability of truss bolt to control the area above the roof. Reinforcing this area above the roof is vital to prevent cutter roof failure when horizontal in-situ stress is high. In contrast, the area of slip in high vertical stress is mainly above the roof where short length tie-rod shows the best response. Same as the latter case, capability of this truss bolt pattern in controlling crack propagation should be taken into account. Truss bolt with medium length of tie-rod and 45 degree inclined bolts shows the best response in controlling shear crack propagation in high vertical in-situ stress.

Studying the effects of installing truss bolt on the position of natural roof arch also shows that changing the design parameters of truss bolt would result in reinforcing different areas above the roof and corners of the tunnel. These results match perfectly with results of SSM factor where short span truss bolt with wide angle inclined bolts are able to reinforce the area above the roof. By increasing the length of tie-rod and decreasing the length of inclined bolts, the main area of reinforcing effect of truss bolt shifts from an area above the middle of the roof to the area around inclined bolts.

It has been shown that, impact of truss bolt system changes with respect to the condition of the in-situ stress distribution. There are many other geological features that might have significant influence on the practice of truss bolt systems, such as thickness of the rock layers, strength parameters of rock, condition of discontinuities, time factor, etc. (Neall et al. 1978). Consequently, it can be concluded that obtaining an optimum design for truss bolt systems entails consideration of effects of each individual design parameter alongside with comprehensive study of all of the external geological and ground controlling parameters.

Effects of systematic rock bolt on the stability indicators showed that greater number of vertical rock bolts not necessarily result in a better response of the reinforcement system. In spite of having less number of rock bolts, pattern 1 showed a better response in controlling plastic behaviour of rock and preventing shear crack propagation. Comparing the results of truss bolt and systematic rock bolt systems on the stability indicators showed the different mechanism of these systems. Truss bolt produces a trapezoid reinforced area above the tunnel in which the major reinforced area fits in an arch while systematic rock bolt reinforces the area above the roof in a beam shape area. Effects of these systems on preventing plastic behaviour of the rock also showed the same areas of reinforcing effects.

CHAPTER 5

Truss Bolt Optimum Design

Variable rock mass qualities around underground excavations is always a problem in finding the optimum design pattern for reinforcement devices. Despite good efforts in designing the truss bolt pattern, none of the design procedures considers the changes in geology and properties of the coherent rock material (Section 2.6). An optimum truss bolt design can vary with respect to changes in the geological features such as strength parameters of rock layers, joint directions, thickness of the rock layers, induced stresses by advancing stopes and changing the overburden load. To have a better understanding of the effects of each factor and finding the optimum design patterns, these variables should be changed alongside with the design parameters of the truss bolt. This only can be achieved by using numerical methods which are able to consider lots of variables at the same time. In this chapter, using the finite element modelling techniques we use three design parameters (length and angle of inclined bolts and length of tie-rod) and several thicknesses of the rock layers and change them to find the optimum design pattern of truss bolt for each bedding configuration.

5.1 Numerical Modelling

Tunnel dimensions, material properties of different rock types and rock bolt elements, horizontal tension force and the mesh are chosen as the same as Section 4.2. In-situ stress distribution is hydrostatic and equal to 1.9 MPa. Seven bedding configurations are modelled which are shown in Table 5.1. Here we call each model by two numbers whereas the first number is the distance of the first bedding plane from the roof and the second number is the distance of the second bedding plane from the roof. For example, 30150 is a model with thickness of the first layer equal to 30 cm and the second layer equal to $150 - 30 = 120$ cm (Table 5.1).

Table 5.1 Different bedding configurations.

Name of the Models	Thickness of the first layer (cm)	Top level of the second layer (cm)
3090	30	90
30150	30	150
30250	30	250
90150	90	150
90250	90	250
120250	120	250
150250	150	250

As mentioned before, three design parameters of truss bolt patterns are chosen to be changed in models with different bedding configuration. These variables and their values are shown in Table 5.2. As a result total number of $5 \times 5 \times 4 = 100$ models are generated for each bedding configuration. Considering seven types of bedding configuration results in $7 \times 100 = 700$ models.

Table 5.2 Different truss bolt design parameters.

Design parameters					
α	15	30	45	60	75
L (m)	1	1.5	2	2.5	3
S (m)	1.6	2	2.4	2.8	

according to several reports, inclined bolts should be anchored far enough from the loosened area, above the rib of the tunnel to provide a safe anchorage (Cox and Cox 1978; O'Grady and Fuller 1992; Liu et al. 2005). This factor should be controlled during the model generation, while the design parameters of the truss bolt are being changed. A rejection criterion is developed to reject the models those have less than 0.6 m length of inclined bolts behind the walls of the tunnel, i.e. not anchored in the safe area (Cox and Cox 1978). This criterion is simply based on the length and angle of inclined bolts and the position of the drill-hole which is defined by the length of the tie-rod.

A number of Matlab codes are developed to generate the models, run them, get the results, tabular the outputs and draw the required graphs. Three different stability indicators, which have been introduced in Chapter 4, are chosen to compare the results. These indicators are 1) reduction in the number of plastic points (we call it *plastic points*), 2) reduction in the area of the loosened rock beneath the reinforced arch (we call it *loosened area*) and 3) reduction in the horizontal movement of the first roof layer, or slip on the first bedding plane (we call it *slip*). Note that to compare the effect of truss bolt on the slip on the first bedding plane, the reduction in the area beneath the graph of slip versus radial distance from centre of the roof, is calculated. These stability indicators are chosen to represent the effects of truss bolt on strength parameters of rock, vertical deformation of the roof rock and the horizontal movement of the immediate roof layer. Obviously, each of these indicators represent different aspects of the behaviour of the surrounding rock mass.

To have a fair comparison, the selected indicators should be weighted with respect to their significance on the stability of the tunnel. This needs an intense statistical study on the way that each of these indicators control the stability of an underground excavation. Here we simply normalize these indicators to have dimensionless values and add them together with the same weight. This means that the significance and effect of each indicator, on the stability of the underground excavation, are considered as the same. The normalized indicator can be expressed as

$$a_{in} = a_i \times \frac{100}{max(a_i)}$$

where a_{in} is the normalized value and a_i is the initial value resulting from each indicator. In this calculation the maximum value for each indicator will be 100 and the most optimum design is a pattern which its total result is a value closer to 300.

5.2 Results and Discussion

After running the models and performing calculations to measure the normalized stability indicators, the upper 15% of the results are considered as optimum design patterns, i.e. 15 models out of 100 models for each bedding configuration. Tables 5.3 to 5.9 show these optimum design patterns together with the values of every stability indicators. These 15 patterns are split in four groups which are defined by different colours: upper 5% as red, between 5% to 10% as yellow, between 10% and 15% as green and the rejected models as gray[1].

Table 5.3 shows the optimum designs for model 3090, which represents a highly laminated rock formation. It shows that the optimum angle of inclination changes between 30° and 60° while the optimum tie-rod length changes between 1.6 and 2 m. Considering the change in the length of inclined bolts for a specific angle of inclination and tie-rod length (e.g. 45° and 1.6m tie-rod), it can be concluded that not necessarily the longer inclined bolts are favourable as by increasing the length of inclined bolts the overall points of the pattern decrease (changing colour from red to yellow or yellow to green in Table 5.3).

The major number of optimum designs for 30150 are placed under the 45° angled inclined bolts (Table 5.4). 4 out of 5 most optimum patterns (red cells) lie under 2 m tie-rod. Long length, 30° inclined bolts and a number of models with 45° inclined bolts are also ranked as green and yellow while the

[1] The rejection criterion is based on the truss parameters not the bedding configuration, consequently they are the same in every group.

optimum tie-rod length varies from 1.6 to 2.4 m.

Comparing the results of increasing the thickness of the second layer from 60 cm to 220 cm, while the thickness of the first layer is constant (models 3090 to 30250, see Tables 5.3 to 5.5), reveals that patterns with 30° inclined bolts are no longer the optimum designs for models with thick second layer. Instead, truss bolt systems with 45° inclined bolts, 1.6 and 2 m tie-rods and various length of inclined bolts show the best response. Also, in all of these three model configurations (Tables 5.3 to 5.5) 60° with 2 to 3 m inclined bolts and 1.6 and 2 m tie-rods show fairly good response, by having a number of yellow and green ranked designs.

Tables 5.6 and 5.7 are mostly the same, showing the negligible effect of the changing the thickness of the second bedding plane while the first bedding plane is relatively thick (comparing with thickness of 30 cm for the first layer). Most of the optimum designs in these two model configurations are patterns with long inclined bolts, angle of inclination of 45° and 60° and short tie-rod length (1.6 m). Also, from Table 5.7, most of the patterns with 75° inclined bolts and short length tie-rod are rejected. However, using longer inclined bolts, if possible, would result in anchoring the inclined bolts out of the rib area and good response of truss bolt system as two of these patterns are in upper 5% of the optimum designs in 90250 model. The same result can be seen in Tables 5.8 and 5.9 for 120250 and 150250 models.

By increasing the thickness of the first layer, while the second layer remains constant (comparing Tables 5.7, 5.8 and 5.9), the optimum angle of inclined bolts increases from 45° to 60° and 75° and the longer inclined bolts show better response. This change is probably because of the changing in the nature of the models, where a model with thick rock layers tends to behave similar to a continuum material model. Furthermore, changing the optimum angle of inclination from 45° to 75° shows that higher angle of inclination is more favourable in models with thick layers (or continuum material). High angled inclined bolts (ultimately 90°) represents a pattern similar to systematic rock bolt. It can be concluded that, in continuum material or thick layers, systematic rock bolt would have better application in comparison with truss bolt pattern. It should be noted that considering the effect of horizontal tension to create a reinforced arch area is vital and this effect on 90° inclined bolts should be compared with vertically tensioned systematic rock bolt to have a better understanding in this content.

Table 5.3 Optimum truss bolt designs for a model with 30 cm and 90 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	30			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	45			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	60			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	75			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
3090-L=3A=60S=2	90.91	87.18	30.63	208.72
3090-L=2.5A=60S=2	90.91	87.18	32.12	210.21
3090-L=3A=45S=2	81.82	71.79	57.40	211.01
3090-L=2A=45S=2	81.82	66.67	63.69	212.18
3090-L=1A=30S=1.6	63.64	51.28	100.00	214.92
3090-L=1.5A=45S=2	81.82	66.67	68.31	216.79
3090-L=3A=45S=1.6	81.82	74.36	61.69	217.86
3090-L=2A=30S=2	100.00	51.28	66.82	218.10
3090-L=1.5A=45S=1.6	72.73	74.36	71.22	218.30
3090-L=2.5A=45S=1.6	81.82	74.36	63.86	220.04
3090-L=1.5A=30S=2	100.00	46.15	74.56	220.71
3090-L=2A=45S=1.6	81.82	74.36	67.69	223.87
3090-L=1A=30S=2	90.91	46.15	87.54	224.61
3090-L=2.5A=60S=1.6	90.91	94.87	38.87	224.65
3090-L=3A=60S=1.6	90.91	100.00	37.85	228.76

Table 5.4 Optimum truss bolt designs for a model with 30 cm and 150 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	30			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	45			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	60			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	75			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
30150-L=3A=60S=1.6	76.92	87.23	48.71	212.87
30150-L=1.5A=45S=2.4	76.92	74.47	62.66	214.06
30150-L=3A=30S=2	92.31	61.70	60.79	214.80
30150-L=3A=45S=1.6	76.92	74.47	66.47	217.86
30150-L=2A=60S=1.6	76.92	91.49	50.22	218.63
30150-L=1A=45S=2	61.54	70.21	87.00	218.75
30150-L=2.5A=45S=1.6	76.92	74.47	68.47	219.86
30150-L=3A=60S=2	84.62	95.74	40.15	220.51
30150-L=2.5A=45S=2.4	92.31	78.72	51.62	222.65
30150-L=2A=45S=2.4	92.31	78.72	55.40	226.43
30150-L=3A=45S=2.4	100.00	78.72	49.74	228.47
30150-L=2A=45S=2	84.62	78.72	67.12	230.46
30150-L=2.5A=45S=2	92.31	78.72	62.02	233.05
30150-L=1.5A=45S=2	84.62	74.47	74.88	233.97
30150-L=3A=45S=2	100.00	82.98	60.05	243.03

Table 5.5 Optimum truss bolt designs for a model with 30 cm and 250 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	30			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	45			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	60			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	75			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
30250-L=2A=60S=2	83.33	89.29	45.84	218.46
30250-L=3A=60S=1.6	83.33	89.29	50.38	223.00
30250-L=2A=60S=1.6	91.67	78.57	53.44	223.68
30250-L=1.5A=45S=1.6	75.00	67.86	82.36	225.22
30250-L=1A=45S=2.4	75.00	73.21	79.01	227.23
30250-L=3A=60S=2	91.67	96.43	41.16	229.26
30250-L=1A=45S=2	75.00	67.86	87.72	230.58
30250-L=2.5A=60S=1.6	91.67	89.29	52.66	233.61
30250-L=2.5A=45S=1.6	91.67	71.43	70.70	233.79
30250-L=2.5A=45S=2	100.00	75.00	63.45	238.45
30250-L=2A=45S=1.6	91.67	71.43	77.03	240.13
30250-L=3A=45S=1.6	91.67	82.14	67.28	241.09
30250-L=3A=45S=2	100.00	82.14	60.18	242.32
30250-L=2A=45S=2	100.00	75.00	70.20	245.20
30250-L=1.5A=45S=2	100.00	75.00	77.32	252.32

Table 5.6 Optimum truss bolt designs for a model with 90 cm and 150 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	30			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	45			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	60			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	75			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
90150-L=2A=60S=2	57.58	100.00	67.42	225.00
90150-L=2.5A=75S=1.6	75.76	78.38	77.52	231.66
90150-L=1.5A=60S=2	57.58	94.59	80.07	232.24
90150-L=3A=45S=1.6	100.00	67.57	64.95	232.52
90150-L=2A=45S=1.6	96.97	67.57	68.86	233.40
90150-L=2.5A=45S=1.6	100.00	67.57	66.30	233.87
90150-L=1A=75S=1.6	81.82	56.76	97.31	235.88
90150-L=3A=75S=1.6	75.76	83.78	76.54	236.08
90150-L=2A=75S=1.6	84.85	78.38	79.39	242.62
90150-L=1A=60S=1.6	93.94	51.35	100.00	245.29
90150-L=1.5A=75S=1.6	84.85	78.38	82.60	245.83
90150-L=2A=60S=1.6	90.91	83.78	81.00	255.70
90150-L=3A=60S=1.6	96.97	83.78	75.59	256.35
90150-L=1.5A=60S=1.6	90.91	72.97	95.04	258.92
90150-L=2.5A=60S=1.6	96.97	83.78	78.28	259.03

Table 5.7 Optimum truss bolt designs for a model with 90 cm and 250 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	30			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	45			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	60			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	75			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
90250-L=2.5A=75S=2	43.48	94.87	61.24	199.59
90250-L=2A=45S=1.6	78.26	58.97	67.27	204.50
90250-L=1.5A=45S=1.6	82.61	53.85	72.10	208.55
90250-L=2.5A=45S=1.6	91.30	58.97	62.56	212.83
90250-L=1A=75S=1.6	69.57	53.85	92.01	215.42
90250-L=3A=45S=1.6	95.65	69.23	60.17	225.05
90250-L=2A=60S=1.6	82.61	64.10	80.52	227.23
90250-L=1.5A=75S=1.6	91.30	64.10	81.91	237.32
90250-L=1.5A=60S=1.6	82.61	64.10	93.72	240.43
90250-L=2A=75S=1.6	86.96	79.49	76.51	242.95
90250-L=2.5A=60S=1.6	100.00	74.36	74.92	249.28
90250-L=1A=60S=1.6	100.00	53.85	100.00	253.85
90250-L=3A=60S=1.6	100.00	84.62	69.82	254.43
90250-L=3A=75S=1.6	95.65	94.87	72.60	263.13
90250-L=2.5A=75S=1.6	95.65	94.87	73.43	263.95

Table 5.8 Optimum truss bolt designs for a model with 120 cm and 250 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	30			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	45			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	60			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

α	75			
S	1.6	2	2.4	2.8
$L=1$				
$L=1.5$				
$L=2$				
$L=2.5$				
$L=3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
120250-L=2A=45S=1.6	92.86	59.18	61.60	213.64
120250-L=2.5A=75S=2	59.52	87.76	70.90	218.18
120250-L=2.5A=45S=1.6	92.86	63.27	62.64	218.76
120250-L=2A=75S=2	57.14	83.67	78.99	219.81
120250-L=3A=45S=1.6	100.00	63.27	61.94	225.20
120250-L=3A=75S=2	61.90	93.88	69.48	225.26
120250-L=1.5A=60S=1.6	85.71	69.39	87.88	242.98
120250-L=1.5A=75S=2	54.76	100.00	91.35	246.11
120250-L=2.5A=60S=1.6	95.24	77.55	80.10	252.89
120250-L=2A=60S=1.6	90.48	77.55	85.42	253.45
120250-L=3A=60S=1.6	100.00	85.71	75.07	260.79
120250-L=2A=75S=1.6	85.71	85.71	89.50	260.93
120250-L=3A=75S=1.6	90.48	89.80	81.66	261.94
120250-L=2.5A=75S=1.6	90.48	89.80	82.90	263.17
120250-L=1.5A=75S=1.6	88.10	77.55	100.00	265.65

Table 5.9 Optimum truss bolt designs for a model with 150 cm and 250 cm bedding planes.

α	15			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	30			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	45			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	60			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

α	75			
S	1.6	2	2.4	2.8
$L = 1$				
$L = 1.5$				
$L = 2$				
$L = 2.5$				
$L = 3$				

Upper 5%	
Upper 5 to 10%	
Upper 10 to 15%	
Rejected	

Truss bolt pattern	plastic points	loosened area	slip	total
150250-L=1.5A=75S=2	55.81	68.63	95.17	219.61
150250-L=2.5A=60S=2	58.14	68.63	95.11	221.87
150250-L=3A=60S=2	55.81	76.47	94.42	226.70
150250-L=2.5A=45S=1.6	79.07	60.78	90.75	230.60
150250-L=1.5A=75S=1.6	79.07	60.78	93.26	233.11
150250-L=3A=45S=1.6	88.37	60.78	91.74	240.90
150250-L=2A=60S=1.6	90.70	60.78	100.00	251.48
150250-L=2A=75S=1.6	90.70	64.71	99.99	255.39
150250-L=2.5A=60S=1.6	90.70	64.71	100.00	255.40
150250-L=2A=75S=2	65.12	96.08	100.00	261.19
150250-L=3A=75S=2	62.79	100.00	99.63	262.42
150250-L=2.5A=75S=2	65.12	100.00	99.92	265.04
150250-L=3A=60S=1.6	100.00	68.63	99.96	268.59
150250-L=3A=75S=1.6	90.70	80.39	100.00	271.09
150250-L=2.5A=75S=1.6	93.02	80.39	100.00	273.41

CHAPTER 6

Conclusions

Truss bolt systems have proved competency in controlling the stability of underground excavations in severe ground conditions particularly in coal mines and layered strata. Despite this, knowing the mechanism of truss bolt systems on reinforcing an underground excavation is vital. This study has tried to understand the mechanism of truss bolt by means of numerical modelling. This study involves a review of theories of rock bolting and previous efforts on understanding the mechanism of truss bolt system. Available design schemes of truss bolt system have been reviewed and a reviewing summary is presented. Also, basics and different components of modelling an underground excavation in FE, verification process and sensitivity analysis on the dimension of the model have been discussed in detail. The main contributions of this study can be concluded to the following points:

- Several stability indicators have been introduced to evaluate the effects of truss bolt on stability of a tunnel. None of the individual indicators is able to determine the stability of an underground excavation, but in combination, they help us to understand the effects and mechanism of truss bolt system.

- Truss bolt stabilize underground excavations in several ways such as repositioning the natural reinforced arch and reducing the area of loosened rock above the roof, creating a trapezoid reinforced area in which an arch shape structure is the major reinforced area, reducing horizontal movement of rock layers, preventing shear crack propagation, and decreasing the chance of cutter roof failure.

- Changing the angle and length of inclined bolts and the span of the system change the effectiveness of the system in facing different stability problems.

- To reinforce the loosened area and preventing roof deflection, a short span truss bolt with wide angled inclined bolts is more appropriate.

- Results of employing Stress Safety Margin (SSM) show that short inclined bolts are able to reinforce the area near inclined bolts better than longer bolts. And a short span truss bolt responds better on the area above the roof while a wide span truss bolt results in better reinforcing effect on the area above pillars.

- To prevent horizontal movement of the immediate layer in high horizontal in-situ stress, a wider span and sharper angle of inclination (from horizon) respond better.

- To prevent shear crack propagation in high vertical stress, a pattern with medium length of tie-rod and inclined bolts and 45 degree inclined bolts results in the best performance.

- Systematic rock bolt and truss bolt show different reinforcing mechanism. Systematic rock bolt is able to produce a highly reinforced beam shape area above the roof between the anchorage and head of the vertical bolts. On the other hand, truss bolt reinforces an arch shape area above the roof which is a part of a minor reinforced trapezoid shape area between the inclined bolts.

- Changing the thickness of the roof layers to find the optimum design parameters of truss bolt system showed that

  - By increasing the thickness of the immediate roof layer while second roof layer is constant, the optimum angle of inclined bolts changes from $45°$ to $75°$ (from horizon) and longer inclined bolts response better.

  - And by increasing the thickness of the second layer while the thickness of the immediate layer is constant, optimum angle of inclined bolts changes from $30°$ to $60°$ (from horizon).

  - When the rock layers are thick, the surrounding rock tends to behave similar to continuum material and highly inclined bolts, which make a truss bolt pattern similar to systematic rock bolt, represents the best response.

6.1 Recommendations for Further Analysis

The numerical models in this thesis are mostly a simplified condition of the real problem. In practice, the rock-reinforcement interaction is more complicated. a number of variables are included in the behaviour of rock and stability of underground excavations. The author believes that possible improvements can be applied in these analyses by modelling interactions between rock-grout and grout-bolt, considering more sophisticated elastic-plastic material model for both rock and rock bolt, presenting discontinuities and considering fracture growth around the excavation. Also, carrying out field investigations and experimental studies to validate the numerical results would give a very good credit to the results of the numerical analysis. A comparative study and sensitivity analysis on effects of truss bolt parameters and different site variables such as in-situ stress distribution, thickness of the layers, joints direction and dimension of the tunnel would result in a comprehensive design guideline which would be a useful tool in designing truss bolt systems.

In the end, the author wishes that this study provides engineers with a basic understanding of the mechanism of truss bolt systems and effects of several design parameters and site variables which would be useful in achieving an optimum design of truss bolt systems.

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