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Rock bolting and pull out test on rebar bolts

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Abstract

Pull out tests were carried out in the laboratory of rock mechanics at Norwegian University of Science and Technology (NTNU) for the purpose of determining the critical embedment length of fully grouted rebar bolts. The 20-mm rebar bolts and the grouting material, "the Rescon Zinc bolt cement" used in the testing are widely used in underground projects in Norway. Different embedment lengths, ranging from 10 cm to 40 cm, were employed in the tests under different water-cement ratios for the grouting mortar. The critical embedment length for a given water-cement ratio is determined on the diagram of the pull - out load versus the embedment length. A chart of the critical embedment bolt length versus the water-cement ratio as well as the uniaxial compressive strength of the cement mortar is established based on the testing results. In the theoretical part of the thesis, the main focus is on rock bolting. Bolting principals are introduced along with different types of rock bolts, design of bolting systems and stability problems caused by rock stresses. In the final part of the theoretical part the procedure of pull out testing is described and the load bearing capacity of rock bolts are categorized into the groups in accordance with load deformation performance. Finally, previous pull out test research is presented.

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1 Chapter – Introduction

1.1 In general

For a long period, rock bolts have been used for the support of underground excavations in civil and mining engineering to increase a structure's stability. Rock bolts transfer pressure from the unstable face of the rock structure to a more stable core. To measure relative performance of different anchor systems in rock mass, pull out test are performed on rock bolts. Different anchor systems, such as mechanical anchors or different bond materials, and lengths for grouted anchors are tested and the data are used to choose anchor types and select the correct bolt length, spacing and size of bolts.

In the theoretical part of the thesis, the main focus will be on rock bolts in general. After an introduction of rock bolts and their history, the bolting principals are presented. It is believed that bolting's binding effect is accomplished by a basic mechanism that depends on the geology and the stress regime. Different types of rock bolts are then categorized into three main groups according to their anchoring mechanisms. The groups are mechanically anchored rock bolts, friction anchored rock bolts and fully grouted rock bolts. The main focus will be on fully grouted bolts with cement and the water-to-cement ratio used in the grouting. Things like the setting time of the mixture are an important issue in the application of cement-grouted bolts, and this will be introduced in more detail. Stability problems caused by rock stresses are a factor in underground excavations. The need for rock support can be estimated from the rock mass properties and the possibilities of optimizing the excavation geometry. In the chapter about stability problems caused by stresses, it is explained how the *in situ* stresses change with greater depth, and what effect stress has in weak and hard rock. Rock bolts are suitable choices in rock mass with stress problems.

Geotechnical discontinuities in the intact rock, material properties, distribution and magnitude of rock stresses are all factors that affect the design of a rock bolting system. To create a complete and appropriate rock bolting system design, it is important to properly investigate parameters such as length, bolt type, spacing and pattern. In the chapter about design of bolting system, these parameters are described in more detail. The final chapter in the theoretical part is about pull out testing. The testing procedure is described and the load bearing capacity of rock bolts are categorized into the groups in accordance with load deformation performance. Finally, previous pull out test research is presented.

In the case study section, results from pull out tests are introduced. The pull out tests were carried out in a laboratory at Norwegian University of Science and Technology (NTNU). In collaboration with Statens Vegvesen and Professor Charlie Li, supervisor of this project, it was decided to perform testing on 20 mm rebar bolts grouted with Rescon Zinc bolt cement mortar. Both rebar bolts and the cement mortar are usually used in underground projects in Norway. This specific test was performed to evaluate the critical length of fully grouted rebar bolts. From a plot of load versus displacement, the ultimate and working capacities of the bolt can be calculated. Ultimate capacity is the maximum load sustained by the anchor system and working capacity is the load on the anchor system at which significantly increasing displacement begins. In this pull out test different embedment length and variation in cement-water mixing ratios of grout was used. A sufficient number of tests were taken to determine the average bolt capacities. Finally, a uniaxial compressive strength test was performed on the cement mortar with variation in cement to water mixing ratios.

1.2 Purpose of the thesis

The main purpose of the thesis is to obtain understanding of the load bearing capacity of fully grouted rebar bolts with different water-to-cement ratios and variations in embedment length. In spite of much research, in which pull out tests have been conducted on fully grouted bolts, there is still a lack of understanding as to how different embedment lengths, grouted with different ratios, affect the load bearing capacity. In this thesis, the relationship between critical bolt length and w/c numbers is estimated, along with the relationship between the critical length and the uniaxial compressive strength of the grout. Also, the aim of the thesis is that the reader will gain greater knowledge of rock bolts in general and their function.

2 Chapter – Rock bolts

2.1 Introduction

In underground excavations, stability is improved by adding rock support. The most dominant support method in the Scandinavian tunneling industry has historically been rock bolts (Nilsen and Palmström, 2000).

The rock bolting system improves the competence of disturbed rock masses by preventing joint movements, forcing the rock to support itself. The rock bolt support system binds together a discontinued, fractured, laminated and jointed rock mass. In addition to strengthening and stabilizing jointed rock masses, rock bolting has a marked effect on the stiffness of the rock. Through a friction effect, a suspension effect or a combination of both, rock bolts reinforce rock mass. Because of this, the technique of rock bolts is approved for mines and tunnels in all types of rock (Kılıc, Yasar and Celik, 2002). Rock bolts are used both as an initial support and as the final rock support. They are used at the tunnel face as an initial support to ensure safe working condition for workers (Nilsen and Palmström, 2000). Rock bolts can be installed individually to fix individual loose blocks at the excavation face (called spot bolting) and afterwards with systematic bolting. Systematic bolting is a pre-planned pattern of bolts that is based on geological conditions.

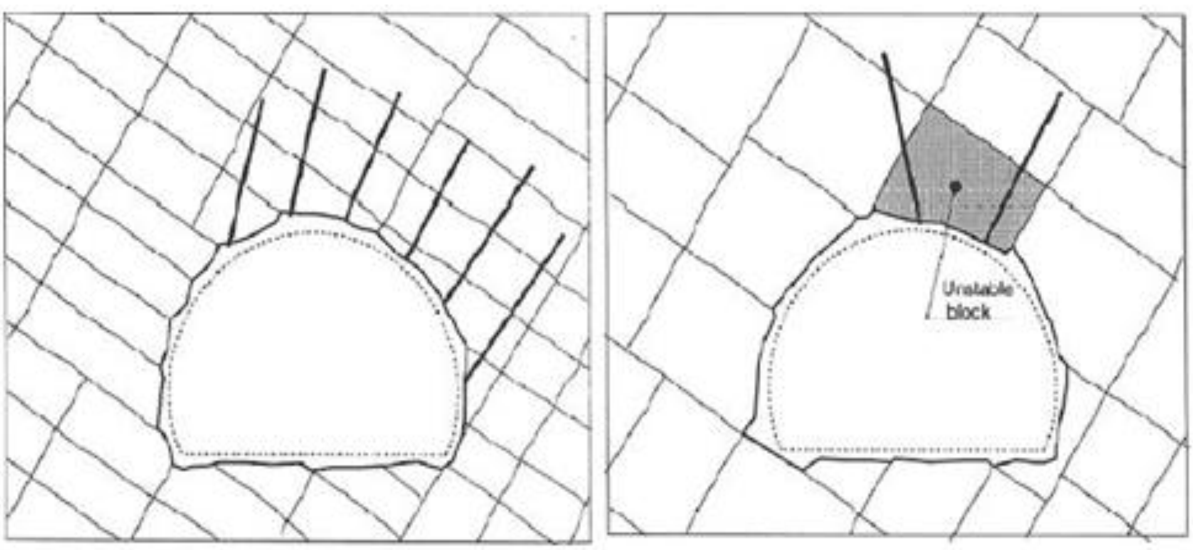

Figure 1- Rock bolts can be installed both individually and with systematic bolting. Systematic bolting is a pre-planned pattern of bolts based on geological conditions, but individual bolts are also installed to fix single loose blocks (Nilsen and Palmström, 2000).

There are various methods used for the design of the required pattern to secure the rock mass. Geological mapping and Q value are mainly used. Systematic bolting is used both during the excavation of the tunnel and at the end of the excavation. It is intended to give a more general support in failure zones (Nilsen and Palmström, 2000).

Table 1 – Main areas of application for systematic and spot bolting (Bjørn Nilsen, Professor at Department of Geology and Mineral Resources Engineering, personal communication, 10. October 2013).

Main areas of application for systematic bolting or spot bolting		
	Spot bolting	Systematic bolting
Fractured rock	Support against rock slide/rock falls	X
Heavily fissured rock	X	Used in combination with other methods
Rock burst/high stress	X	In combination with fiber-reinforced sprayed concrete
Weakness zones	X	As one element in a larger concept

Generally, a rock bolt consists of a plane steel rod with an anchor at one end and a face plate and a nut at the other. The anchor may be mechanical or chemical. The length of the drilling hole, which the bolt is injected into, should be at least 100 mm longer than the rock bolt. This prevents the bail from dislodging when it is forced against the end of the hole. After installation the anchor will be seated with a sharp pull. To increase the anchor force, the bolt is tightened to force the cone into the wedge. If rock bolts are meant to provide a temporary support, they are usually left ungrouted. If they are, on the other hand, used to provide a permanent support, or in rock with corrosive groundwater, the gap between the rock wall and the bolts may be filled with cement or resin grout. These materials lock the anchor in place and shield it from disturbing blasting vibrations. One of the primary causes of rock bolt failure is rusting or corrosion. The grout, filled in the gap, can prevent the bolts from rusting (Hoek, 2007a).

2.2 History of bolts

For a long period, rock bolting has been used widely for rock reinforcement in civil and mining engineering to increase structures' stability. They play a crucial role for construction companies that work with natural material, especially rocks and stones, while providing protection for underground caverns, tunnels, and rock cuttings *(*n. fjellskæring) and slopes (Li, 2009).

In the years of 1855-1861, rock bolts were used for the first time in construction when the Telemark Canal, which connects the coast of Telemark with the interior from Skien to Dalen in Norway, was built (Statens Vegvesen, 1999). Telemark Canal was completed in 1893 and it was called "the eighth wonder of the world" (Telemarkskanalen, 2009). The rock bolts in the canal were short, with a diameter of 40 mm, and their main use was to stabilise blocks in the mountainside. Bolstad and Hill (1983) reported the use of mechanical bolts in a metal mine in the United States 1927. Later, bolts became the most dominant support method in underground construction around the world. Mining companies were pioneers in using rock bolts, primarily companies from the United States, South Africa and Norway. The U.S Bureau of Mines (USBM[1]) was the first company to use roof bolting technology in 1947; the

[1] Primary United States government agency conducts scientific research and disseminate information on the extraction, processing, use, and conservation of mineral resources.

technology was employed to reduce the number of fatal accidents caused by roof falls. Five years later annual roof bolt consumption had reached 25 million.

Rock bolts quickly became an inexpensive and practical support method and today they are used as a support in tunnels, caverns, rock cuttings and construction pits. They are also used as a base reinforcement when building bridges, docks and dams in rock. The usage of bolts as a rock support has increased significantly since 1970. Mechanization is probably the main reason, but as it grows the production rate of drilling and other construction methods also increased. To cope with this, the rate of reinforcement had to increase and new types of rock support evolved to resolve problems related to rock. Most stability problems today are possible to solve with rock bolts and shotcrete. Used with modern equipment, it provides an effective and quick support method. Specific equipment like bolt rigs and robots are also used to a greater extent than before (Statens Vegvesen, 1999).

The first type of rock bolts used underground were split and wedge bolts. These first types are not in use anymore because they are considered unsafe. Since then, rock bolts have evolved significantly; the main focus today is that bolts have to be strong enough to sustain the dead weight of the unstable blocks. Safety is an important factor in modern construction. All regulations have become stricter to provide safer work conditions for workers and also to ensure that the finished work is safe for use. It also depends a lot on the nature of the work. For example, more emphasis is placed on safety in large road tunnels or storage caverns than in water tunnels. In the case of road tunnels, damage from a collapse would be much more than if a water tunnel collapsed. Advanced methods based on empirical data are used more today than before. The main reason is because of the builders' requirements in connection with difficult and complex projects (Statens Vegvesen, 1999).

Nowadays, in US coal mines about 100 million roof bolts are installed every year in excavated entries (Yassein et al., 2004). Similar numbers of rock bolts are installed in Australia. In 1997 the usage of rock bolts worldwide was in excess of 500,000,000 annually (Winsdor, 1997). In Norway approximately 500,000 rock bolts are installed every year. They are used as a temporary support system in underground excavations or as a permanent support system. Figure 2 shows how the usage of rock bolts has grown in the coal mining industry from 1920 until the beginning of 2000 (Junlu, 1999).

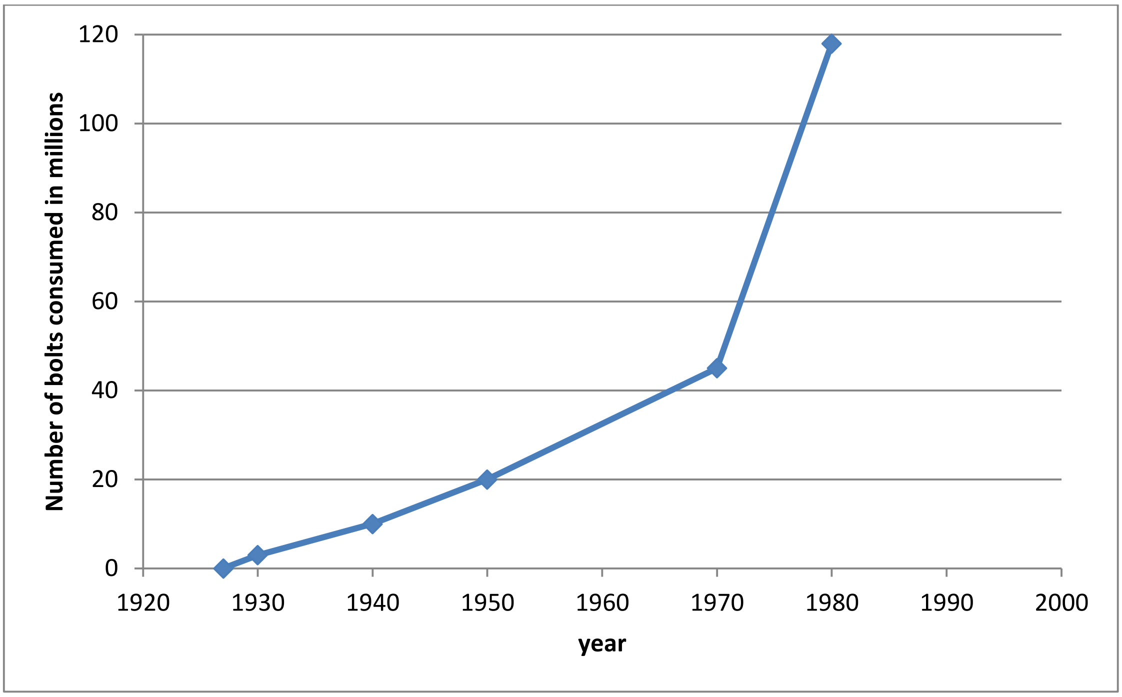

Figure 2 – The growth in usage of bolts in the coal mine industry increased greatly from 1920 until the beginning of 2000. Mechanization is probably the main reason for the increase (JunLu, 1999).

2.3 Bolting principles

Rock bolting is a very successful method in a variety of geological and geotechnical conditions. As mentioned earlier, the main function of bolting is to bind together broken rocks such as sedimentary rocks that contain bedding planes, jointed and fractured rocks, or rocks with artificial cracks caused by explosions or excavations (Peng and Tang, 1984). It is believed that bolting's binding effect is accomplished by the following basic mechanism depending on the geology and the stress regime:

- Skin control
- Suspension
- Stitching (beam building)
- Supplemental support

The bolt binding is achieved by one or a combination of these mechanisms (JunLu, 1999). A brief description of these four mechanisms will be given below.

2.3.1 Skin Control

At the skin of the opening of underground excavations, where the strong and massive roof is essentially self-supporting, cracks, cross beds, joints and slickensides may create occasional hazardous loose rock. In that kind of situation, it is important that the function of the bolts is to prevent local rock falls. It is not necessary to prevent a major collapse in such an environment. A sufficient support system should be a pattern of relatively light and short roof bolts. In weaker ground, skin control may also be an important secondary function of roof bolts. In figure 3, a skin control mechanism is described graphically (Mark, Molinda and Dolinar, n.d.)

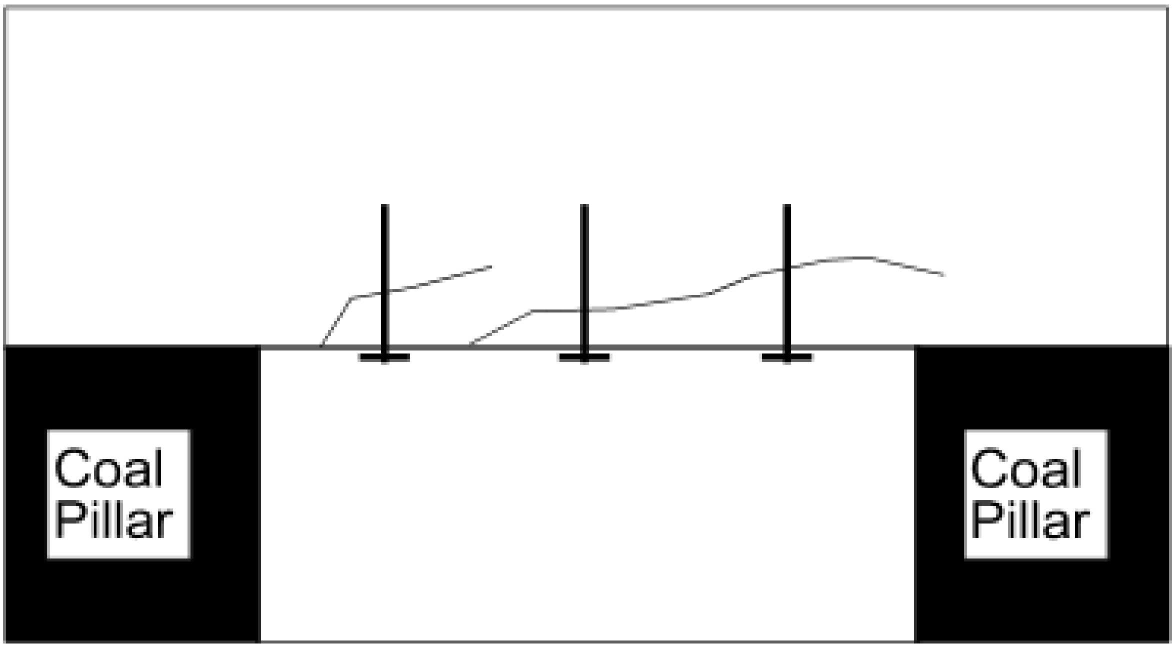

Figure 3 – Skin control. It is important that the function of the bolts is to prevent local rock falls. A sufficient support system should be a pattern of relatively light and short roof bolts (Mark, Molinda and Dolinar, n.d.).

2.3.2 Suspension – Hang up to the above stable layer

In many underground openings, a stronger unit that is largely self-supporting overlies a weak immediate layer. This weak layer tends to sag (Mark, Molinda and Dolinar, n.d.). If the layer is not properly supported in time, the laminated immediate roof may separate from the main roof and fall out. In this situation, roof bolts act to suspend the weaker layer. The bolts anchor the immediate roof to the self-supporting main roof by the tension applied to them (JunLu, 1999). In the suspension mode, experience has shown that rock bolts are very efficient. However, suspension becomes more difficult if the weak layer gets too thick; more than 3 ft becomes difficult to handle (Mark, Molinda and Dolinar, n.d.).

In suspension, the essential bolt force is calculated by predicting that the whole weight of the lower-strength beddings will be borne by the supported bolts. The length of the bolts used in the openings, which must be anchored in the stable rock, is determined by the thickness of the bedding. The bolt spacing is not critical; it is based on the bolts' strength and the weight of the rock (Li, 2013).

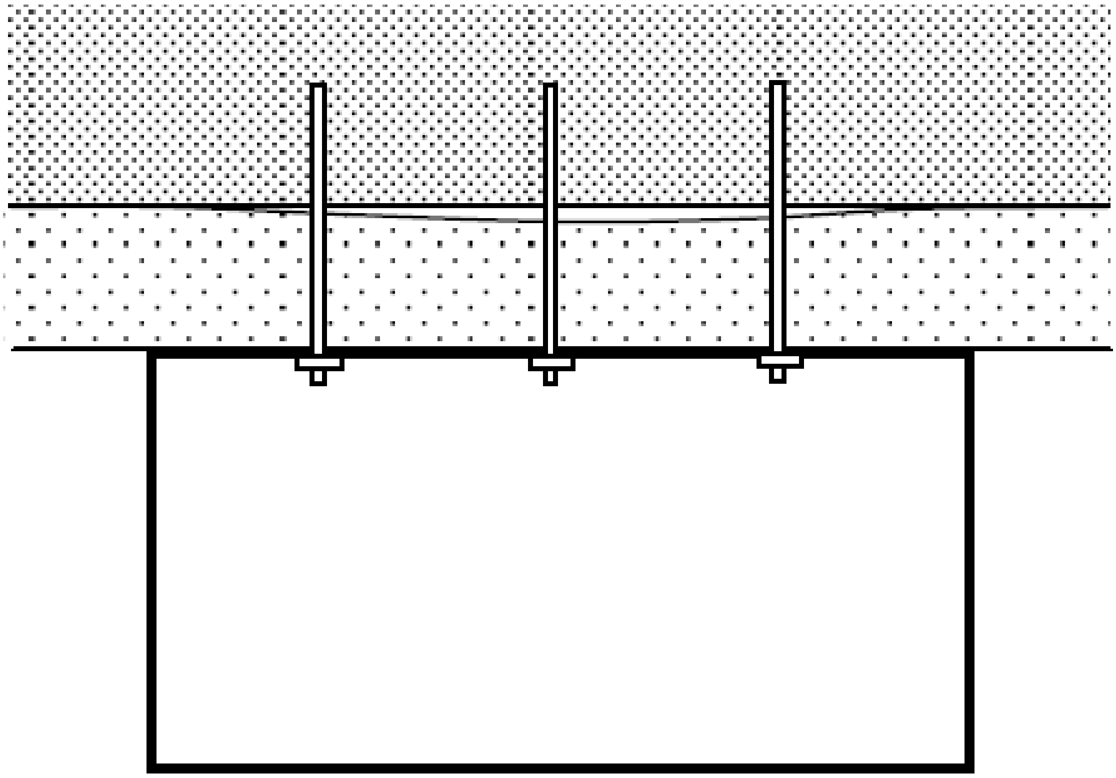

Figure 4 - Suspension bolting. Length of the bolt is important for anchoring in stable rock (Li, 2013).

2.3.3 Stitching (beam building)

It is very common that no self-supporting bed is within reach; in this case, the bolts must tie the roof together to create a so-called "beam." The ordinary roof bolts are not able to reach an anchor for suspension. In this situation, the bolts can still be applied with good success. Both vertical and horizontal movements along the bedding interfaces are caused by sagging and separation of the roof laminate. The horizontal movements are greatly reduced by bolting through these layers; tension is applied to the bolts manually on installation or induced by the vertical displacement of the rock, forcing the layers together. That makes all the layers move with the same magnitude of vertical displacement. Along the bedding interface, frictional forces, which are proportional to the bolt tension, are induced and also make horizontal movement difficult. Beam building is very similar to clamping a number of thin and weak layers into a strong layer, forming a fixed-end composite beam. Theoretically, given that all the thin layers that are clamped into a single strong layer are made of the same material, the maximum bending strain at the clamped ends of the composite beam is:

$$\varepsilon_{mac} = \frac{wL^2}{2ET} \tag{2.1}$$

Where

W = Unit weight of the immediate roof

L = Length of the immediate roof

E = Young's modulus

T = Thickness of the composite beam

The equation above shows that the thicker the beam is, the smaller the maximum strain induced at the clamped ends. Research has implied that if the bolt spacing is decreased, and tension, the number of bolted laminates, and the roof span are all increased, then the beam building effects increase (JunLu, 1999). Generally, higher densities of rock support are needed in beam building than in suspension, as the bolts have to work much harder. Beam building applications along with supplemental support (described later) have been the most troublesome for design (Mark, Molinda and Dolinar, n.d.).

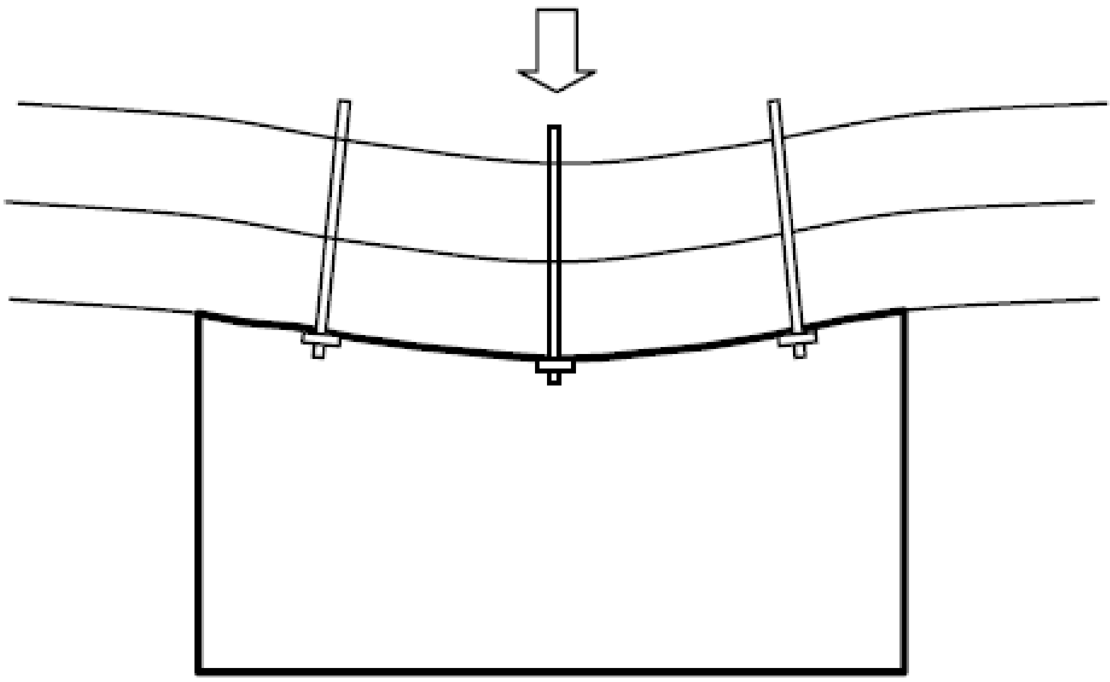

Figure 5 – Beam building/stitching. It is very common that no self-supporting bed is within the reach so the bolts must tie the roof together to create a so-called "beam." If the bolt spacing is decreased, and tension, the number of bolted laminates, and the roof span are all increased, then the beam building effects increase (Li, 2013).

2.3.4 Supplemental support

The roof in underground excavations may sometimes be extremely weak and/or the stress extremely high. In that situation, roof bolts may not be able to prevent the failure of the roof from progressing beyond a reasonable anchorage horizon. Rock support like cable bolts, cable trusses or standing support is necessary in these situations to carry the deadweight load of the broken roof. The roof bolts used in these cases are primarily to prevent unravelling of the immediate roof (Mark, Molinda and Dolinar, n.d.).

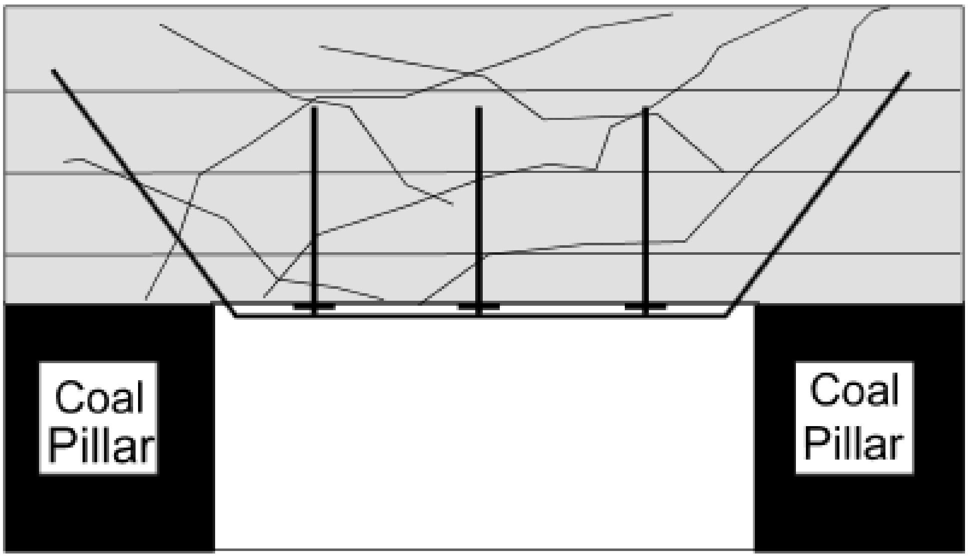

Figure 6 – Supplemental support in failing roof. In some situations where the roof is extremely weak or the stress is extremely high, roof bolts are not able to prevent the failure of the roof from progressing beyond a reasonable anchorage horizon. Cable bolts, cable trusses or standing support are then necessary support. (Mark, Molinda and Dolinar, n.d.).

2.4 Different types of bolts

Different types of rock bolts are categorized into three main groups according to their anchoring mechanisms. The groups are as follows:

- Mechanically anchored bolts
- Friction-anchored bolts
- Fully grouted rock bolts

(Li, 2011).

Below, these groups of rock bolts will be described in more detail. Table 2 describes the advantages and disadvantages of these different groups and Table 3 collects information about some typical technical data for different bolt types.

2.4.1 Mechanically anchored bolts

Mechanically anchored bolts can be divided into two groups: expansion shell anchor and slit and wedge type rock bolts. The anchoring part is either fixed by a wedge-shaped clamping part or by a threaded clamping (Kılıc, Yasar and Celik, 2002). Mechanically anchored bolts are one of the first rock reinforcements used in underground mining and they are still used around the world, including in Canadian mines. If a rock bolt with an expansion shell anchor is well seated in a rock and the rock is hard enough to provide a good grip for the anchor, the expansion shell anchor will allow the rock bolt to be tensioned to its maximum load-bearing capacity. In fact, if the rock bolt is overloaded, failure is most likely in the threads at either the faceplate or anchor end rather than by anchor slip. Mechanically anchored rock bolts provide very effective support in many conditions, such as when rock blocks have been loosened by intersecting joints, bedding planes in the rock or when blocks loosen because of poor quality blasting. Mechanically anchored bolts hold up the dead weight of the loose material. They are often used with the addition of mesh where small rock pieces can fall out between bolt heads. These rock bolts are normally tensioned up to about 70% of their ultimate breaking load in order to tighten the loose block and wedges. This provides as much interlocking between the blocks as possible and helps the rock to support itself.

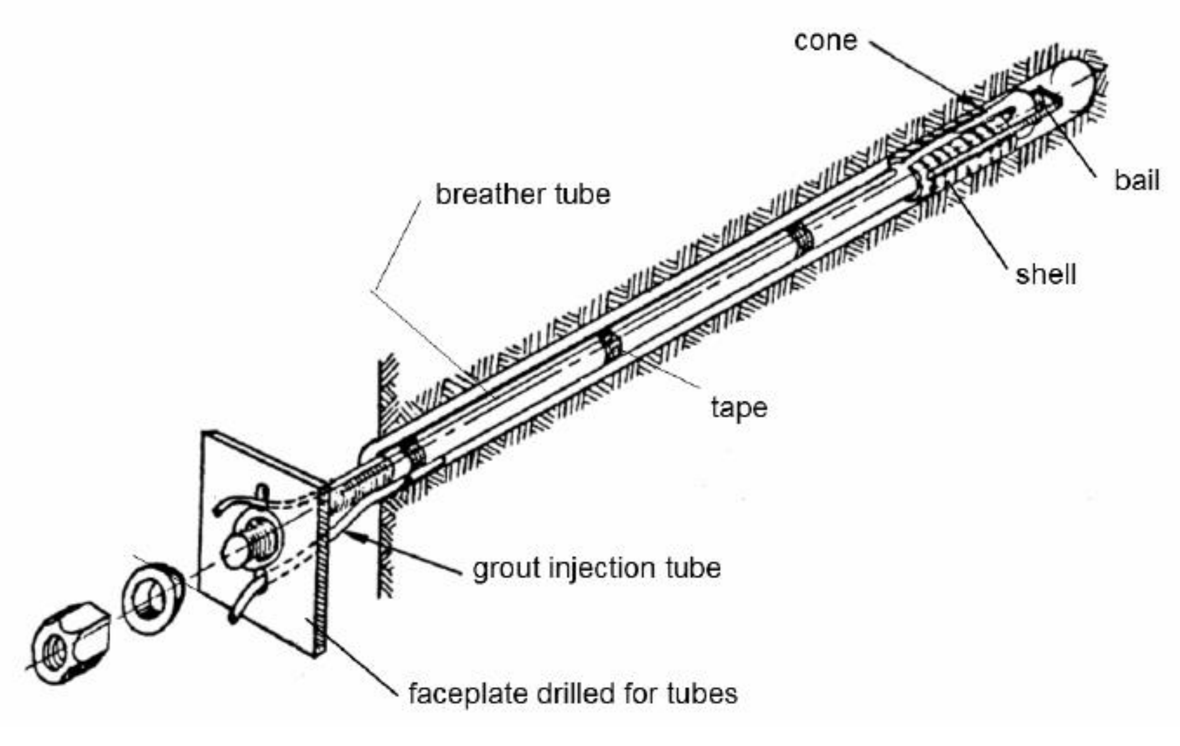

Figure 7 – Mechanically anchored rock bolt. When the bolt is rotated the wedge is pulled into a conical expansion shell. The shell is forced to expand against and into the rock wall of the hole (Hoek, 2012).

There are unfortunately some problems with mechanically anchored rock bolts. Old rock bolts which have lost all their tension are often seen underground; anchors tend to slip progressively with time. This is most likely a result of vibrations induced by blasting nearby. Rusting of bolts in rock masses with aggressive groundwater is another problem. In cases like this, the life of unprotected bolts may be less than a year even though it requires bolts with long-term life. Mechanically anchored rock bolts should be grouted in place in such circumstances (Hoek, and Wood, 1987).
The most common type of mechanically anchored rock bolts are expansion shell anchored bolts. A wedge is attached to the bolt shank; when the bolt is rotated, the wedge is pulled into a conical expansion shell. When that happens, the shell is forced to expand against and into the rock wall of the hole. The anchoring mechanisms of this type of bolt are friction and interlock between the expansion shell and the wall of the borehole. Because of the limited length of the expansion shell, the load-bearing capacity of the bolt is relatively low. Expansion shell bolts have wide application in mining as well as in civil engineering projects. When the bolts are used as a permanent support it can be necessary to have the void between the bolt and the borehole post grouted. It is not recommended to use expansion shell bolts in a very hard rock, as it will prevent the shell from gripping the rock and the anchor will slip under load (Li, 2013).

If the anchor slips or the mechanically anchored rock bolt breaks, the capacity of the bolt drops to zero. That may lead to a rock fall from the supported rock. This is one of the main disadvantages of mechanically anchored bolts. This is not a problem for the other types, such as fully grouted bolts and friction-anchored. If slip occurs or the face plate breaks off from the friction-anchored bolts, the remaining length of the bolt is still anchored and still provides support (Hoek and Wood, 1987).

2.4.2 Friction – anchored bolts

Friction-anchored rock bolts are the second group. Frictional bolts are bolted in a special way, using frictional resistance to sliding generated by a radial force against the borehole wall over the whole bolt length (Kılıc, Yasar and Celik, 2002). Friction bolts stabilise the rock mass by the friction, without needing any important auxiliaries like mechanical locking devices or grouting to transfer load to the reinforcing element (Li, 2013). One of the main advantages of friction-anchored rock bolts is that they accommodate large rock formations. However, they do not tolerate high load-bearing capacity (Li, 2011).

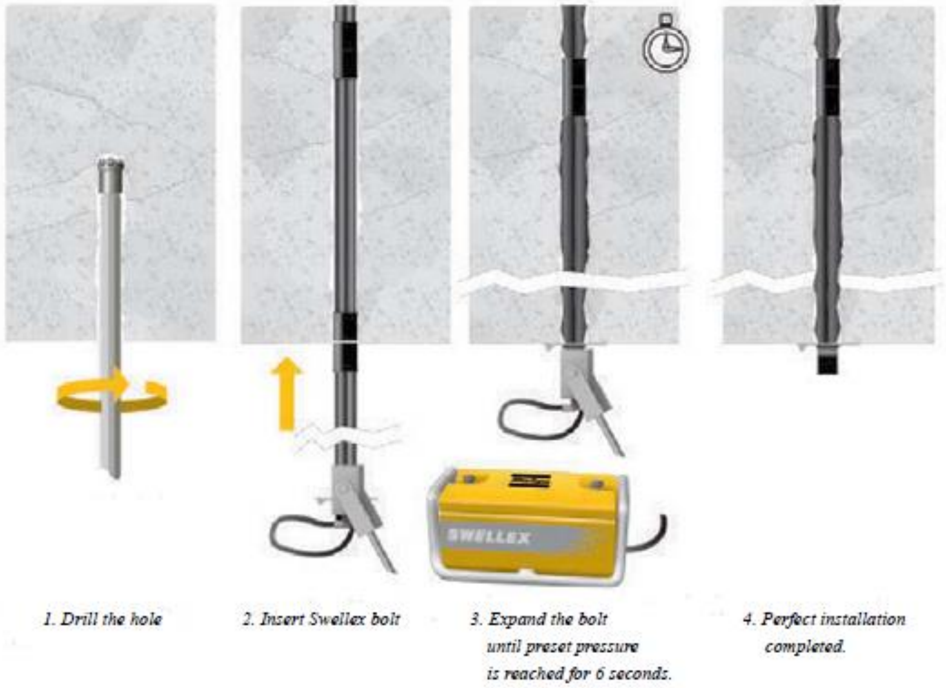

Figure 8 – Friction anchored rock bolt. Injection of Swellex (Atlas Copco, 2012).

Both fully grouted bolts and friction-anchored bolts, like Split Set, cannot be tensioned and therefore they have to be installed before any large movement takes place in the rock. The support action for Split Set is similar to that of an untensioned rebar and therefore it must be installed very close to the face. However, this apparent problem can be turned to advantage. Experience has shown that a combination of careful blasting and installation of frictional bolts can provide an effective support system that supports a much wider range of rock conditions than mechanically anchored bolt are able to handle (Hoek and Wood, 1987). Frictional bolts can be divided into two types of bolts available on market:

- Swellex
- Split Set

2.4.2.1 Swellex

Swellex bolts were developed by Atlas Copco AB. The Swellex rock bolting system has become standard in mines and tunnels all around the world. They offer an environmentally safe solution with high efficiency and immediate support. Today there are three versions of Swellex bolts on market offering different yielding properties to match rock mass conditions. They have been used successfully in many tunnels without any application of external systems.

The cost of support has reduced greatly with the use of the Swellex system, as has the overall cost of the mining and tunneling projects (Atlas Copco, 2012). The anchoring mechanism of the Swellex is both frictional and mechanical interlocking. The initial form of the bolt is a folded tube that has a diameter smaller than the diameter of the borehole. When the folded tube has been installed in the borehole it expands to the size of the borehole using high-pressure water. The Swellex bolt will start to slide when it is subjected to a load that approaches the ultimate tensile strength of the bolt (Li, 2013). Mechanical interlocking of the bolt and the rock prevents the sliding of the bolt. The expanded folded tube leads to some reduction in length, which puts the Swellex into tension (Hoek and Woods, 1987).

2.4.2.2 Split Set

Split Set was developed by Scott in collaboration with the Ingersoll-Rand Company in the United States (Hoek and Wood, 1987). Split Set is a true friction-anchored rock bolt. It is a bolt for temporary stabilization and consists of a high strength steel tube slotted along its length and a plate. The diameter of the bolt is greater than the diameter of the borehole so the bolt is driven into a slightly smaller hole (Hardi Rock Control Europe b.v., 2012). The anchor mechanism of the Split Set is the friction between the bolt and the wall of the borehole (Li, 2012). When the bolt is hammered into the borehole the frictional force is induced by the spring action of the compressed steel tube along the length of the tube and anchors the tube into the wall. If the Split Set is not installed close to the face, or the stresses imposed on the tubes are not very large, then the installation may be quite effective. They are also simple and quick to install (JunLu, 1999).

2.4.3 Fully grouted rock bolts

The third and final group is fully grouted rock bolts. They can be divided into two groups depending on how they are anchored:

- Resin-grouted rock bolts
- Cement-grouted rock bolts

In U.S. coal mines, approximately 10,000 miles of underground entries are developed every year. To support these roadways about 80 million roof bolts are installed and more than 80% of the total are fully grouted rock bolts (Mark et al., n.d).

The main characteristic of fully grouted bolts (dowel) is that they are bolts without any mechanical anchors. Usually they consist of ribbed rebar installed in a bore hole and bonded over its full length to the rock mass. Fully grouted rock bolts are commonly used in mining when stabilizing tunnels, roadways in mines, drifts and shafts for the reinforcing of its peripheries. Compared to other rock bolts, the fully grouted rebar bolts have benefits such as simplicity in installation, relatively lower cost and more versatility.

It is important to install grouted bolts as soon as possible after excavation. The reason is that they are self-tensioning when the rock starts to move and dilate, so they have to be installed before the deformation in the rock occurs and before the bolts lose their interlocking and shear strength. Fully grouted bolts are passive bolts, not activated in the installation phase (Kılıc, Yasar and Celik , 2002).

Fully grouted bolts are often used for systematic bolting, as the grouting gives the bolt protection from corrosion (Statens vegvesen, 1999). Poor anchorage of fully grouted rock bolts can be a problem. Roof movements within the anchorage zone in underground excavations can pull the rock bolt out of the upper portion of the hole at loads that are not as high as the yield strength of the rod. Weak rock and poor installation quality are the main causes for poor anchorage (Mark et al., n.d).

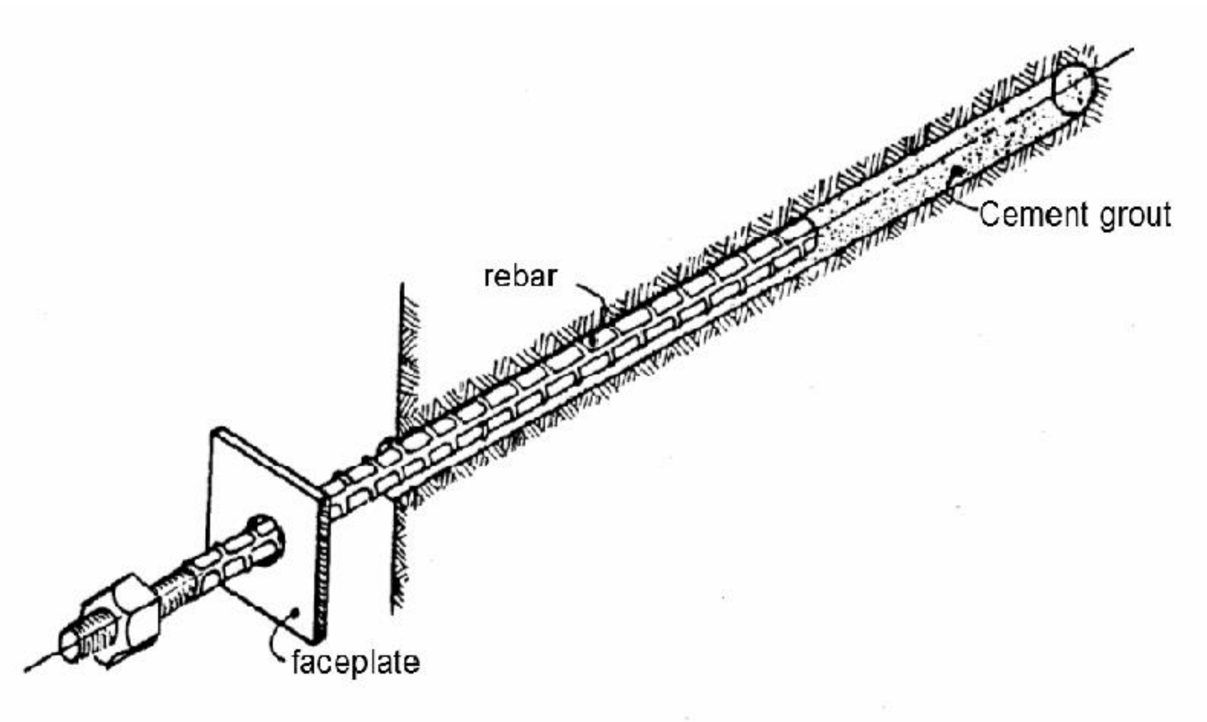

Figure 9 – Fully grouted rock bolt. The bolt consists of three parts; rod, face plate and a bonding (Hoek, 2012).

Fully grouted rock bolts generally consist of three parts: steel bar, bearing plate and bonding material. The first two parts are similar to those used with mechanical bolts, but the last part characterizes the bolt type. The difference with the steel bar from mechanical bolts is that its surface is always deformed or ribbed to both ensure better mixing for catalyst and resin and to increase frictional grip between the steel bar and resin. The bearing plate for fully grouted bolts is only used for coverage.

The bonding material comprises either cement or resin. These two different bonding materials for fully grouted rock bolts will be described more specifically (Peng and Tang, 1983).

2.4.3.1 Resin-grouted rock bolts

Fully resin-grouted bolts are the most sophisticated rock bolt system currently used. It combines most of the advantages of other bolt systems. Resin and a catalyst are packaged in a plastic tube and separated from each other to prevent chemical interaction. These plastic capsules are then placed in the borehole with a loading stick before the bar is inserted. The bar is rotated into the hole, which breaks the plastic tube and mixes the resin and catalyst together (Hoek and Wood, 1987). Resin-anchored bolts can be installed in all types of rock or concrete. They are mostly used where expansion shells are inappropriate rock support (DSI, 2014). They are also increasingly used in critical applications where cost is not as important as speed and reliability (Hoek and Wood, 1987). The anchorage length of the resin bolts can be adjusted to varying rock conditions and it is possible to install them at any angle above or below horizontal. To apply a compressive force across layered rock strata, tension bolts with resin point anchorage are used. The bolt tension is applied before the slow-setting resin cures but after the fast-setting resin has cured. Untensioned resin bolts have to rely on the movement of the rock strata to load the bolts (DSI, 2104).

The development of resin as a bonding material is a further improvement from the cement bonding agent. The major advantages from the cement agent are better anchorage over a wider range of strata types and shorter setting time than cement. However, the resin's high cost is the main disadvantage (Peng and Tang, 1983).

There are some features that are important to achieve successful installation using resin-grouted bolts. First, there is the relation between the hole diameter and the bolt diameter. To ensure proper mixing and higher anchorage capacity, the diameter between the hole and the rebar cannot be too great. According to Wilding and Thomson, the optimum difference is 6, 4 mm (1981).

Mixing and curing time are also important factors to proper installation. If the resin is over-mixed, the anchor strength will weaken or be destroyed. Probably the most important factor to achieve successful installation is the curing time. If the curing is disturbed before it is completed, then the ultimate strength of the resin will never be realized (Peng and Tang, 1983).

2.4.3.2 Fully cement -grouted rock bolts

Grouting with cement is the oldest method of full column anchor bolting. This improved anchorage method works best in weak or fractured strata. The main disadvantage of fully cement-grouted bolts is the uncertainty about cement shrinkage and the longer setting time of the bolts, which limits its use to underground excavations where speed is not required (Peng and Tang, 1983).

There are many types of grout available on the market today, but in situations where the rock has a measure of short-term stability, the most sufficient grout type is simple Portland cement with reinforcing dowels (Kılıc, Yasar and Celik, 2002).

When using Portland cement-grouted rock reinforcement, the grout is pumped into the bore holes and the bars are driven into the filled holes. The grout is introduced through a central hole in the bar as the grout intake or air vent, or a plastic tube alongside the bar. It is important for grouted reinforcement to follow the manufacturer's instructions or engineer's specifications so the grout is properly and fully distributed around the bar when it is driven into the hole. These requirements include information such as hole diameter appropriate for the bar diameter, number and size of packages for the bar diameter, maximum drilling depths of holes, and the length to achieve full grout distribution and encapsulation of the bar and anchor. The bar will be exposed to corrosion if the grout is not fully distributed around it (Kendorski, 2003).

When testing the load-bearing capacity of fully cement-grouted rock bolts there are a few important parameters that need to be taken under consideration, including bolt shape, bolt diameter, length of the bolts, rock strength and grout strength (Kılıc, Yasar and Celik, 2002).

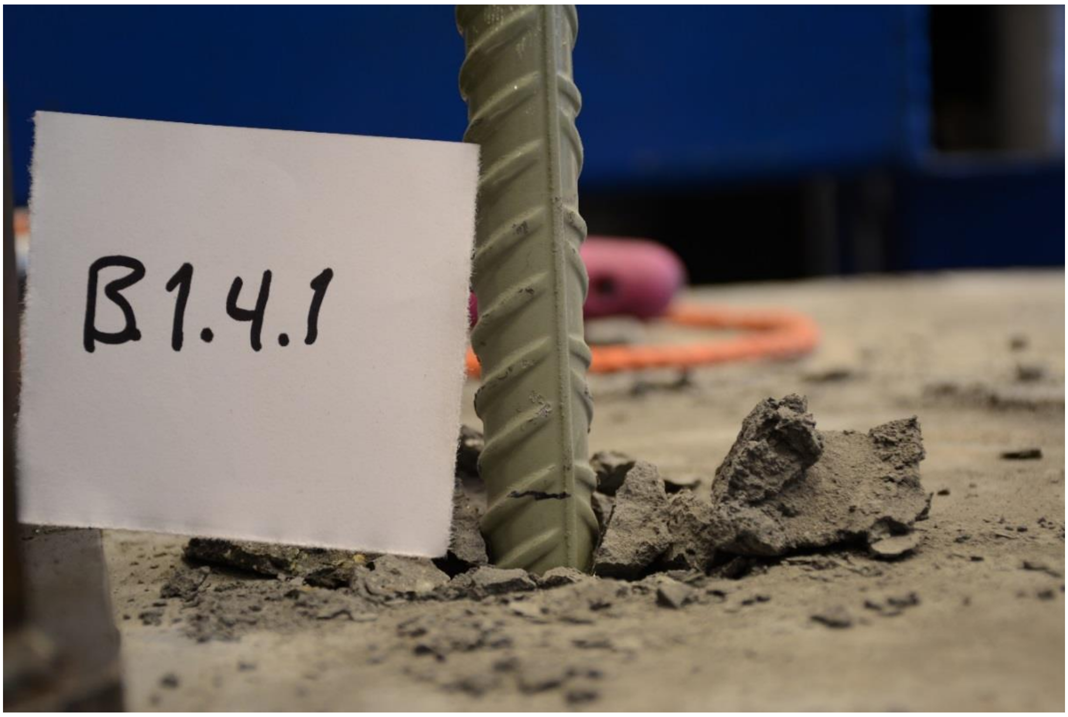

Figure 10 – Pull out test performed on rebar bolt grouted with cement mortar. The figure is taken after the testing has been performed.

Fully grouted rebar bolt

The most common grouted rock bolt is the fully grouted rebar or threaded bar made of steel. The grouting agent for rebar bolts are cement or resin. Rebar bolts are untensioned bolts and are used as both temporary and permanent support in mining and civil engineering. Rebar tolerates high load and is a suitable support system in hard rock conditions. However, they cannot tolerate much deformation and it takes the cement grout anywhere from a few hours up to days to cure, which can be a big disadvantage (Li, 2013).

Cement grouted cablebolt

Grouted cablebolt is a reinforcing element that has been used for reinforcement of structures in rock. It is made of steel wires and is installed untensioned or tensioned with cement grouting. Grouted cablebolts are used in the mining industry and civil engineering but play different roles under these situations. In mines, cablebolts are installed untensioned and fully grouted for temporary support but in civil engineering they are a permanent support.

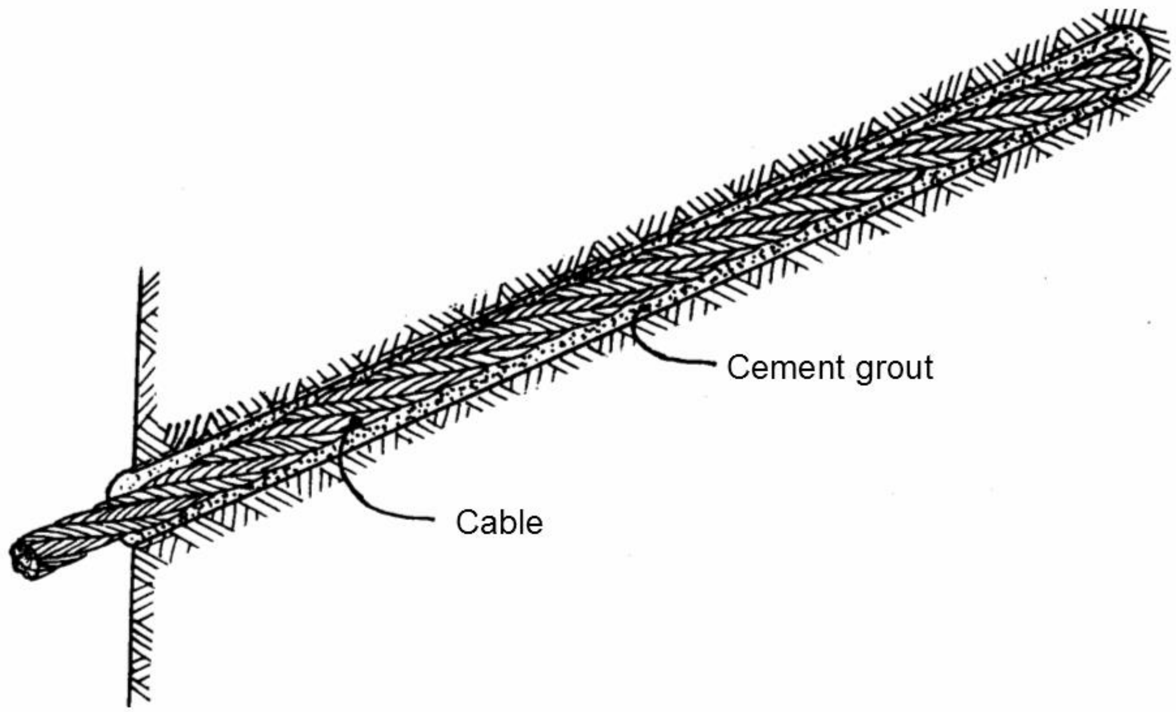

Figure 11 - Grouted cables can be used in place of rebar when more flexible support is required (Hoek, 2012).

Table 2 – Advantages and disadvantages of the bolt types described in the chapter.

Bolt types	Advantages	Disadvantages
Mechanically anchored (expansion shell)	1)Relatively inexpensive 2)Immediate support action after installation((Li, 2013)	1) Limited to use in moderately hard rock. 2) Difficult to install reliably. 3) Must be monitored and checked for proper tensioning. 4) Loses its reinforcement capacity as a result of blast vibrations or chips of rock spalling from underneath the face plate due to high contact forces. 5) Only used for temporary reinforcement unless corrosion protected and post-grouted. (Li, 2013)
Friction anchored bolts Swellex	1) Quick and simple installation. 2) Immediate support action after installation 3) Used in a variety of ground conditions. 4) Little training required to use the equipment 5) Swellex requires no environmental harmful chemical grouts to anchor the bolt in the rock (Atlas Copco, 2012)	1) Corrosion a problem in long term installations 2) Requires a pump for installation 3) May require a sleeve at the collar to prevent spalling under certain rock condition (Hoek and Wood, 1987). 4) Relatively expensive 5) Relative low shear strength (Li, 2013).
Friction anchored bolts Split Set	1) Relative cheap rock bolts 2) Easy to install (Hardi Rock Control Europe b.v., 2012). 3) Useful in moving and bursting ground 4) Immediate support action after installation	1) Cannot be tensioned 2) Sensitive to corrosion and that is why they cannot be used in long term installations 3) The device cannot be grouted 4) Relatively expensive 5) Successful installation of longer bolts can be difficult 6) It has a very resistance to shear displacement
Fully grouted rock bolts (resin)	1) Convenient system and simple to use 2) Very high strength anchors can be formed in rock of poor quality 3) Good anchorage over a wider range of strata types (Hoek and Wood, 1987)	1) Effective resin mixture needs a careful adherence to recommendations provided by manufacturers 2) Resins are expensive material 3) Limited shelf-life, particularly in hot climates (Hoek and Wood, 1987

	4) Permanently effective throughout its full bonding length 5) Prevents both vertical and horizontal strata movements 6) Seal wet holes and exclude air 7) Damage to the bolt head, bearing plate, or rock at the collar of the hole will not cause the bolts to become ineffective 8) Can absorb blast vibrations without bleed-off of the bolt load (Peng and Tang, 1983)	
Fully grouted rock bolts (cement) Rebar	1) Competent and durable reinforcement system with proper installation 2) High load-bearing capacity in hard rock conditions (Li, 2013)	1) Time consuming for cement grout to cure before bolt can take full load 2) Cement grout cures in hours or days. 3) Quality of grouting is difficult to control. 4) Cannot be used in water carrying boreholes 5) Are not able to tolerate much deformation (Li, 2013)
Fully grouted rock bolts (cement) Cable bolts	1) Relatively cheap 2) Competent and durable reinforcement system 3) It provides high loadbearing capacity in hard rock conditions 4) Can be installed to any length (Li, 2013)	1) Standard cement used in the grout- long-time of curing 2) Quality of the grouting is difficult to control 3) It cannot be used in water carrying boreholes. 3) Tensioning of the cables is possible only if installation procedures are followed specifically (Li, 2013)

Table 3 – Typical technical data for different bolt types (Li, 2013).

Typical technical data for different bolt types							
Bolt type	**Steel strength (MPa)**	**Steel diameter (mm)**	**Yield load (kN)**	**Ultimate load (kN)**	**Ultimate axial strain (%)**	**Bolt length (m)**	**Hole diameter (mm)**
Expansion shell	700	16	140	180	14	Any length required	35-38
Swellex	X	X	100/205	110/215	20/15 (steel)	Any length required	35±/48±
Split Set	X	39/46	90/135	110/163	16 (steel)	0.9-3.0/0.9-3.6	35-38/41-45
Fully grouted rebar	570	20	120	180	15	Any length required	35±5
Fully grouted cable bolt	1950	2*15,2	500	500	4,8	Any length required	48-64

2.5 W/C ratio for fully grouted bolts

Water-to-cement (w/c) mass ratio is the most important factor in concrete mix because it controls the mechanical properties and durability of the hardened concrete. Water-to-cement ratio is the weight of water and cement in freshly prepared concrete, and is defined by dividing the water placed in the mixture with the weight of the cement, which is set in the same concrete mixture.

$$ratio = \frac{w}{c} \tag{2.2}$$

By increasing the water in the mixture, the water-to-cement ratio gets higher; as the cement volume increases, the ratio gets lower. A low ratio gives better properties than a high ratio. Therefore, it is recommended to use the exact amount of water to get the necessary workability but no more than that. An overly high water-to-cement ratio reduces the long-term strength of the concrete, increases the drying shrinkage, and decreases the water resistance (Hákon Ólafsson, 1991).

Cement grout is formed by mixing cement powder with water. The ratio is more or less similar to that of concrete. Setting and hardening of cement grout in a hole are important processes which affect the performance of the grout. The presence of excessive voids also affects the strength, stiffness and permeability of grout. Cement mortar is a mixture of cement, water and sand. The typical proportion by weight is 1:0.4:3. While cement grout is constructed by pouring the grout in the holes, cement mortar is placed and packed (Chu, 2010).

Sometimes additives are introduced into the cement mix to provide extra features. Adding bentonit clay in a proportion of up to 2% of the cement weight creates a plastic grout. Other additives can accelerate the setting time of the grout, improve fluidity (allowing injection at lower water-to-cement ratios), and expand the grout and so pressurize the borehole. Extreme caution should be taken when using additives to avoid side effects such as corrosion and weakening of the grouting (Kılıc, Yasar and Celik, 2002).

A comprehensive investigation was undertaken by Hyett, Bawden and Reicher (1992) in order to determine the physical and mechanical properties for Portland cement grouts with a variety of water-to-cement ratios. The range was between 0.25 and 0.7. The grouting was investigated for pull out testing to understand major factors influencing the bond capacity of grouted cable bolts. The samples were mixed and then left to cure for 28 days at a relative humidity of 95%. The results in Figure 12 show that for water-to-cement ratios ranging from 0.7 to 0.35 the uniaxial compressive strength (UCS), tensile strength and Young's modulus all increased.

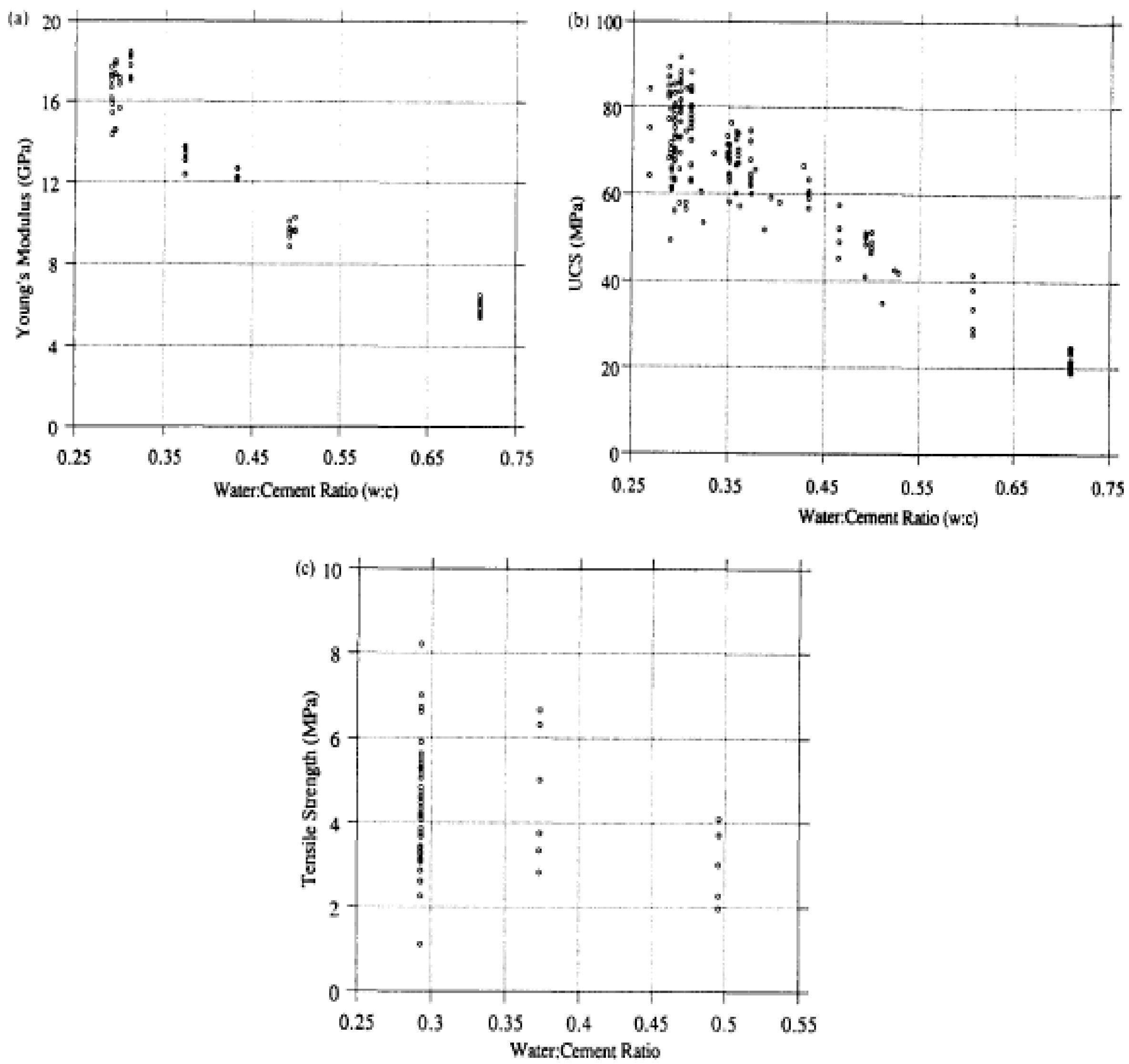

Figure 12 – Mechanical properties of 28-day cure Portland cement (Hyett, Bawden and Reicher, 1992).

However, when the ratio became less than 0.35 only the Young's modulus continued to increase, while any trend in the strength data was overshadowed by an increase in the scatter of both results from the UCS and the tensile strength. Even though the details obtained in the tests above are undoubtedly a result of the particular mixing system used, rather than being true material properties, they may still apply to many grout pumping systems. From these results, it can be presumed that the grout of water-to-cement ratio: 0.30 may be both impractical and undesirable for cable bolts. The reason is because the cement mix is too thick, so it is difficult to pump and because of its variability in strength. According to the results, the most suitable ratios range from 0.35 – 0.40.

The evolution of grout pumps has always been a critical factor in underground construction. It is important that the pumps are capable of pumping grouts with a low enough water-to-cement ratio to achieve adequate strengths. A range of grout pumps are on the market today that are capable of pumping viscous grouts and can operate under typical conditions. Hyet et al (1992) reported a comprehensive testing program on Portland cement grouts. The cement grout had hardened for 28 days before uniaxial compressive strength (UCS) and deformation modulus were measured. The results (Figure 13) showed that the UCS and the deformation modulus decreased with increasing water-to-cement ratio.

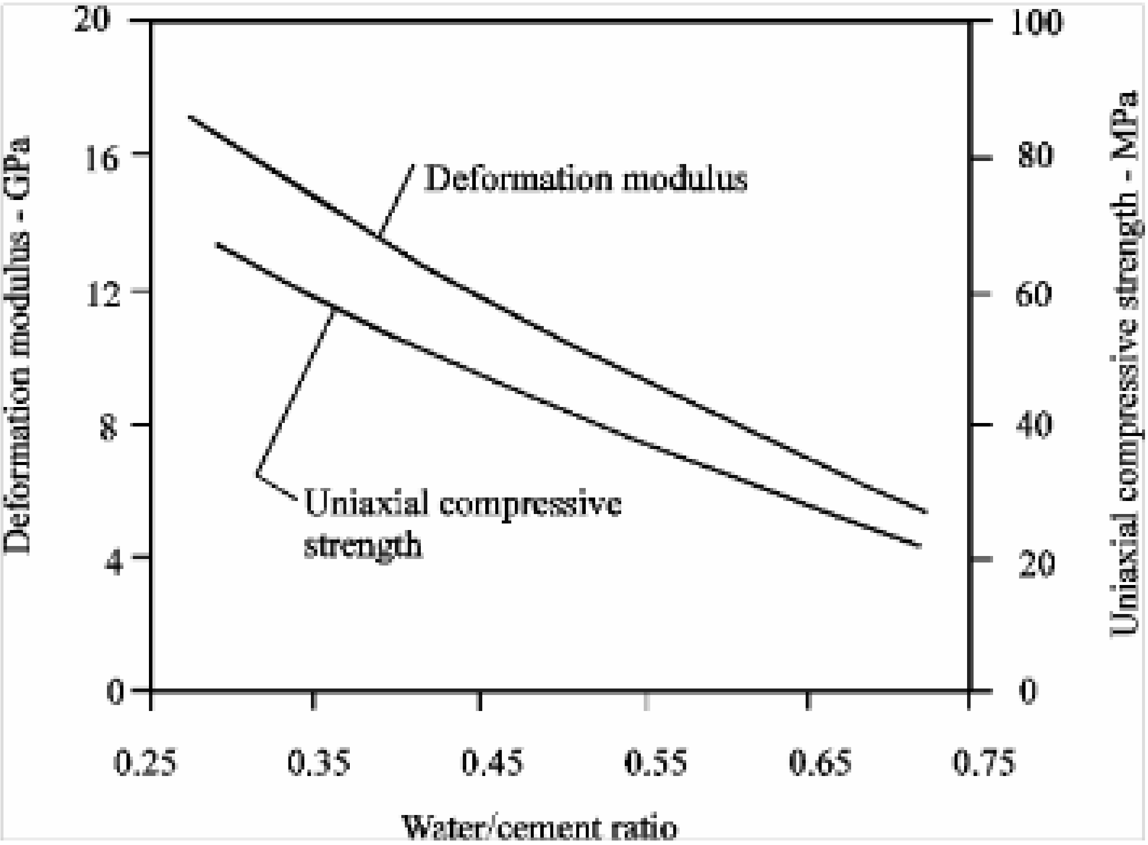

Figure 13 – Relationship between the w/c ratio and the average UCS and deformation modulus. The tests are taken 28 days after grouting (Hoek, 2007).

The results from three ratios were also reported with Mohr failure envelopes. They showed that grouts with ratios of 0.35 to 0.4 are much better than the ratio of 0.5. The scatter in the test results for ratios of less than 0.35 increased markedly, according to Hyett et al. The implication is that the ideal water-to-cement ratio for cable reinforcement lies between 0.35 and 0.4. The table below shows the characteristics of grouts with different ratios:

Table 4-Characteristics of grouts with different water-to-cement ratios (Hoek, 2007).

w/c ratio	Characteristics at end of grout hose	Characteristics when handled
0.30	Dry, stiff sausage structure.	Sausage fractures when bent. Grout too dry to stick to hand. Can be rolled into balls.
0.30	Moist sausage structure. ’Melts’ slightly with time.	Sausage is fully flexible. Grout will stick to hand. Easily rolled into wet, soft balls.
0.35	Wet sausage structure. Structure ‘melts’ away with time.	Grout sticks readily to hand. Hangs from hand when upturned.
0.40	Sausage structure lost immediately. Flows viscously under its own weight to form pancake.	Grout readily sticks to hand but can be shaken free.
0.50	Grout flows readily and splashes on impact with ground.	Grout will drip from hand – no shaking required.

The effects of grout properties on the pull - out load capacity of rebar bolts were tested and results presented by Kilic, Yasar and Celik (2002). This project was conducted to develop a safe, practical and economical support system for engineering works. The results showed that water-to-cement ratios in grouting materials significantly affect the pull out strength of rock bolts (Figure 14).

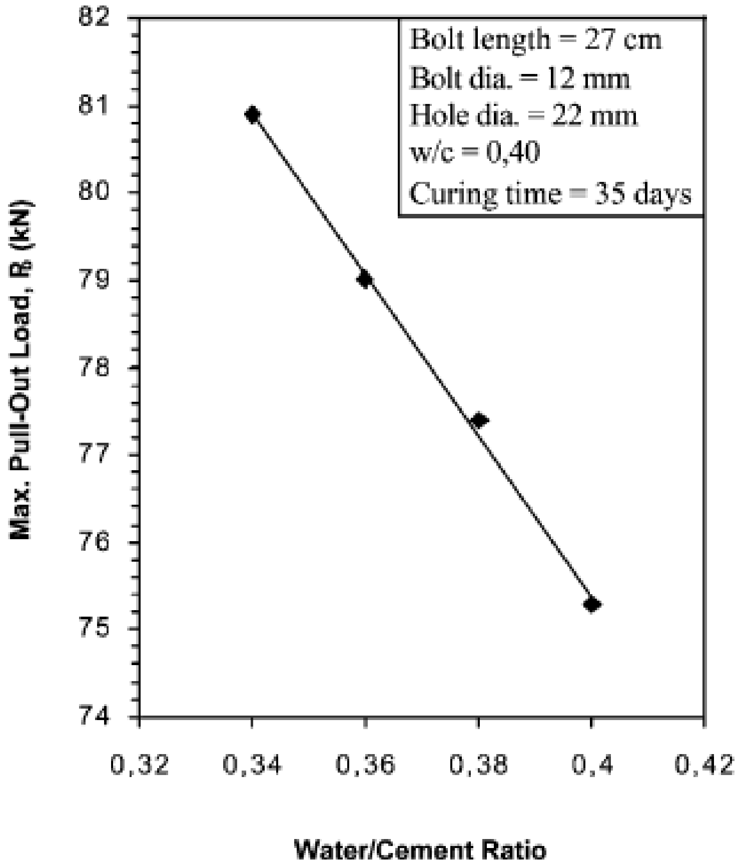

Figure 14 –The figure shows the influence of w/c ratio on the bolt pull - out testing. Maximum pull out load decreases as the w/c ratio gets higher (Kilic, Yasar and Celik, 2002).

When the ratio increases, the values for the UCS and shear strength will become lower (as shown in Table 5).

Table 5 – Variation in water-to-cement ratio influences the uniaxial compressive strength and the shear strength of grouting (Kilic, Yasar and Celik (2002).

w/c	**UCS_g (MPa)**	**T_g (MPa)**	**A_b (cm^2)**	**P_b (kN)**	**τ_b (MPa)**
0.34	42.0	11.9	102	80.9	7.93
0.36	38.9	11.3	102	79.0	7.75
0.38	33.3	10.7	102	77.4	7.59
0.40	32.0	10.3	102	75.3	7.38
Rock: Basalt; diameter of the bolt:12mm					

A ratio between 0.34 and 0.40 presents the best results. The ratio of 0.34 gives the best bond strength (τ_b) (see figure 15). However, other problems occur with that ratio. The pumpability of the grout decreases and a number of difficulties appear in the application. The pumpability increases as the ratio becomes higher, so it will be easier to fill the drill holes. However, that affects the bond strength. The bond strength is primarily frictional for fully cement-grouted bolts and relies on the shear strength at the bolt-grout or grout-rock interface. In other words, every change in the shear strength affects the bond strength and load capacity of the bolts (Kilic, Yasar and Celik, 2002).

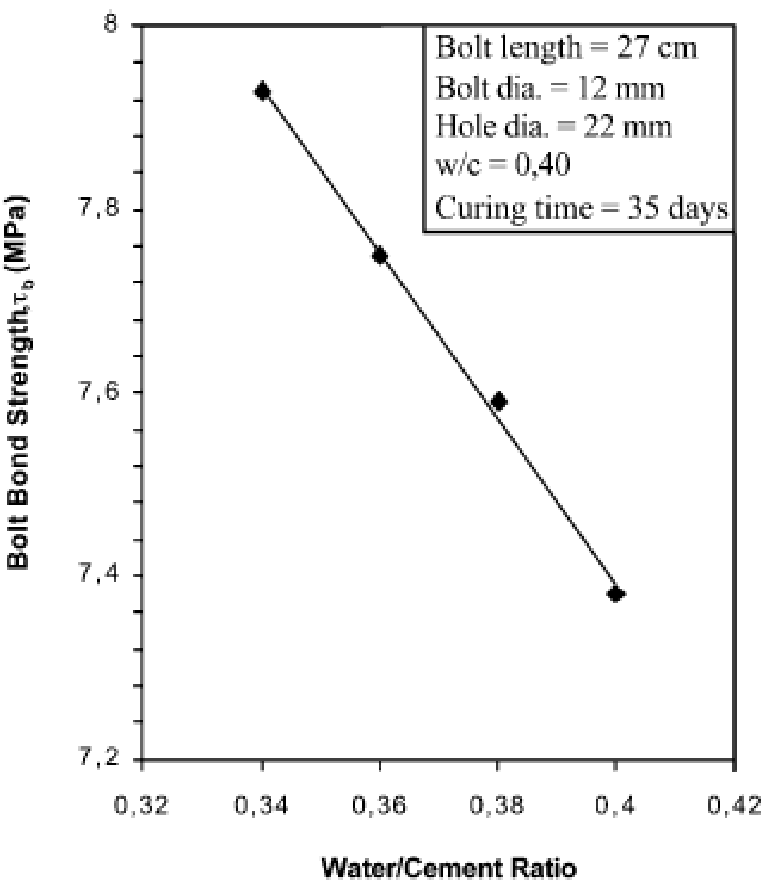

Figure 15 – The ratio of 0.34 gives the best bond strength. However, other problems occur with that ratio, such as decreasing pumpability (Kilic,Yasar and Celik, 2002).

2.5.1 Curing time of grouting

The setting time of cement mortar is an important issue in the application of cement-grouted rock bolts. It affects the stabilizing ability of bolts. It takes time for the cement to set and harden; therefore, grouted bolts cannot be used for immediate support. Eight groups of bolts of the same length and mortar with a w/c ratio of 0.4, were pull-out tested by Kilic et al (2002). They were tested to determine the effects of curing time on the bolts' bond strength. Each group of the rock bolts used a different curing time. Results showed that the first 7 days the bolt bond strength and the maximum pull-out load of bolts increased rapidly. After 7 days, the tests still increased but more slowly.

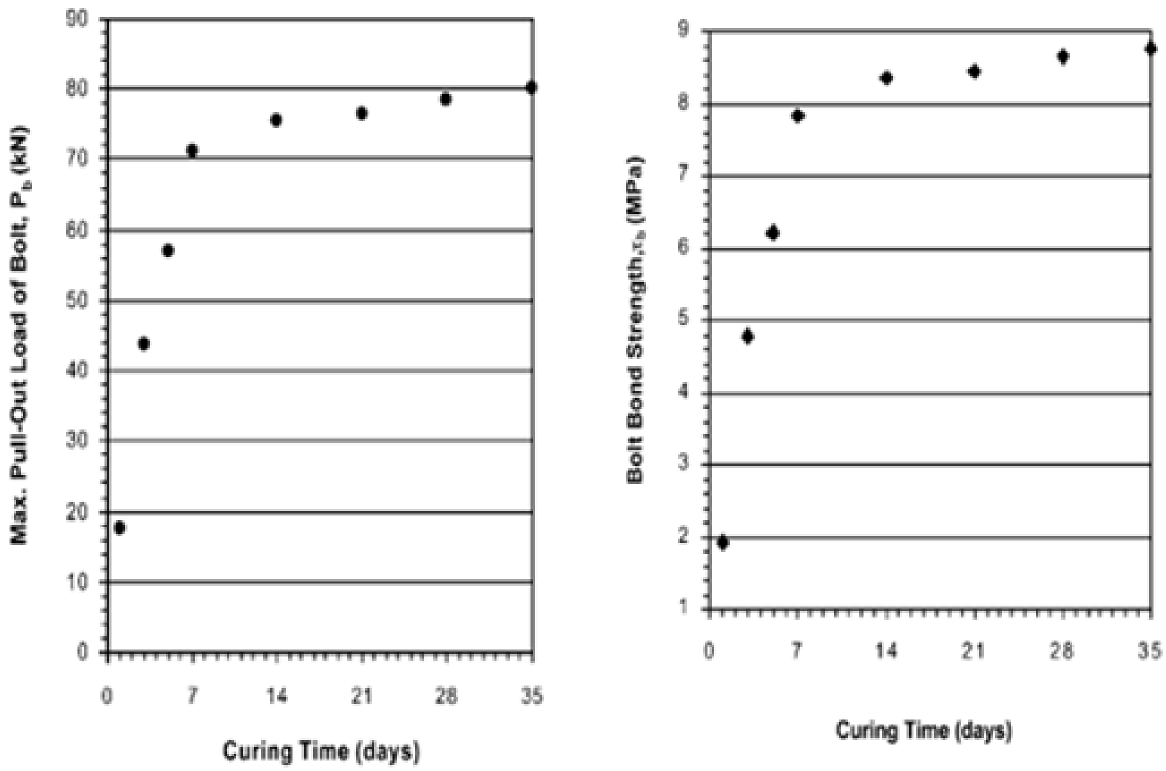

Figure 16 – Different curing times for each group of the rock bolts. In the first 7 days the bolt bond strength and the maximum pull-out load of bolts increases rapidly. After 7 days the tests still increase, but more slowly (Kilic et al, 2002).

Rock bolts may lose their supporting ability through the yielding of rock bolt materials, failure at the bolt-grout or grout-rock interface and unravelling of rock between bolts. However, according to laboratory tests and field observations, the main failure mode is shear at the bolt-grout interface. Therefore, it is important to focus on that in rock bolt testing.

2.6 Use of bolts in Norway

The tectonic history of the Norwegian bedrock is a complicated story of structural patterns and discontinuities, caused by intense folding and metamorphism that are hard to explain. Different rock types and features are detected at the surface and sub-surface in Norway. Even though the Norwegian bedrock is quite stable (a good rock quality), numerous discontinuities in terms of joints and weakness zones are detected. Rock stresses causing stability problems in underground excavations in Norway are partly related to high rock cover and tectonic stresses. Also, most Norwegian underground excavations (like tunnels) are located in the saturated zone of the rock mass. Water inflow and groundwater are therefore important issues to deal with. Despite a great variety of ground conditions, causing problems in tunneling, the overall conditions for underground excavations are considered favorable in Norway. In 1909, when the railway line between Oslo and Bergen was opened, 184 tunnels had been excavated; in the 1970s, when the Norwegian oil and gas era began, experiences grew in underground excavations for storage, transport tunnels and pipeline shores. (NFF,n.d.a) The most commonly used support method in Norwegian excavations is rock bolts and sprayed concrete. Over the years, rock bolts with end anchoring by an expansion shell have been mostly used as a temporary support, and fully grouted bolts with cement as a permanent support. Lately, end anchored resin rock bolts have been used much more often, mainly in the tunneling industry, as a permanent support. About 250,000 rock bolts are used in Norway each year, mostly for tunnel support. The long-term durability of bolts is a crucial parameter and much research has been conducted by the Norwegian Public Road Administration, the main producer of bolts in Norway. The most durable product, according to the Public Road Administration, is fully grouted rebar bolts. One of the newest rock bolts is the CT bolt, which is a steel rebar bolt with an ordinary expansion shell and an arrangement for the injection of cement. In this way, the CT bolt can be used both as an ordinary end anchored bolt for temporary support and then as a permanent grouted bolt (NFF,n.d.b)

Figure 17 – CT bolt. It is a steel rebar bolt with an ordinary expansion shell and an arrangement for the injection of cement (NFF, n.d.b).

3 Chapter – Stability problems caused by rock stresses

3.1 Introduction

Stresses have great impact on stability at the site of an underground excavation. Stability problems occur when the stresses around the excavation exceed the strength of the rock mass and it yields – higher stress causes instability. Low stress may also reduce the stability in jointed rock mass because of normal stresses on joints. This causes rock blocks to slide. It is possible to analyze potential stability problems caused by stresses if the rock mass properties are known. The need for rock support can be estimated from the rock mass properties and the possibilities of optimizing the excavation geometry. Stress information regarding the magnitude and direction of principal stresses are fully obtained by performing rock stress measurements. For each individual case, rock measurement has to be performed because rock stresses vary considerably from place to place. There have been many different methods used to measure *in situ* stresses throughout the years, but the most common method today is triaxial stress measurements by drill hole overcoring and hydraulic fracturing. In Norway, the so-called “door stopper method,” or two-dimensional overcoring, is used in cases of heavy fracturing or for measurements in pillars (Nilsen and Palmström, 2000).

3.2 Definition of stress

The definition of stress is a force over area. Force is, according to Newton’s first law of motion, the product of mass (m) times acceleration (a):

$$F = ma \tag{3.1}$$

Force is defined in Newton (N) as the physical quantity required applying acceleration (a) of one meter per second squared to a mass of one kilogram.

$$F = 1N = kg * \frac{m}{s^2} \tag{3.2}$$

The acceleration due to gravity on earth is $a = g = 9{,}8m/s^2$ and a kg of mass creates a force of

$$F_{(earth)} = 1\ kg * 9{,}8\frac{m}{s^2} = 9{,}8N \tag{3.3}$$

When a force of 1N is acting over a one square meter area the stress is called one Pascal or Pa. In the world of engineering, this is a relatively small stress; most often, it is preferable to work with the mega Pascal (MPa), which is equal to 10^6Pa $(\text{Herget}, 1988)$.

3.3 Stresses in rock mass

Rock stresses exist within the rock mass and are characterized by their directions and magnitudes. Conventionally compressive stress is positive, and tensile stress is negative. Normal stresses on planes with no shear stresses are called principal stresses and are referred to as:

- σ_1 = major principal stress (largest compressive stress)
- σ_2 = intermediate principal stress
- σ_3= minor principal stress (smallest compressive stress)

Stresses found in the rock can be grouped according to origin, into natural stresses and induced stresses; these are the main components influencing magnitude and direction of the stress field. Natural stresses are stresses found in rock before excavation and are comprised of gravitational stresses, topographic stresses, residual stresses, tectonic stresses and thermal stresses (Appendix B). Induced stresses will now be described in more detail.

3.3.1 Induced stresses

When an opening is excavated in a rock mass, the stress field is locally disrupted. This leads to a new set of stresses induced in the rock surrounding the opening, due to manmade excavations.

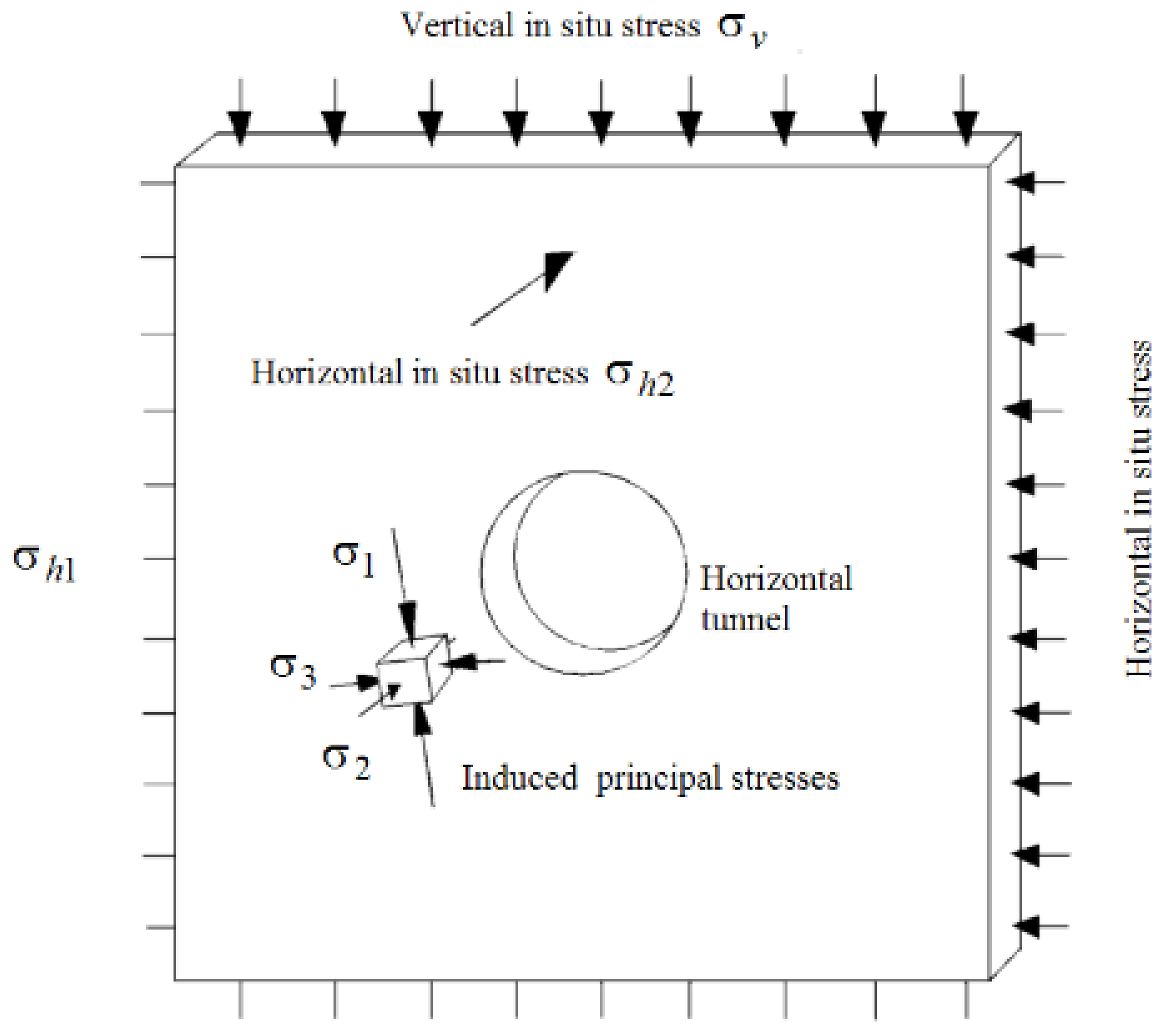

Figure 18 - Illustration of principal stresses induced in an element of rock that is close to an underground excavation. The figure shows the vertical in situ stress, σ_v, the horizontal in situ stress σ_{hi} that is in a plane normal to the tunnel axis, and finally the horizontal in situ stress σ_{h2} that is parallel to the tunnel axis (Hoek, 2007b).

Figure 18 shows an example of the stresses induced in the rock when a horizontal circular tunnel is excavated in a rock mass. After the excavation, the stresses in the immediate vicinity of the tunnel are changed and new stresses are induced. The redistribution of stresses is concentrated in the rock mass close to the tunnel. But farther away from the opening, say a distance of three times the radius from the center of the hole, the disturbance to the *in situ* stress field is negligible. Three principal stresses acting on a typical element of rock are shown in Figure 18.

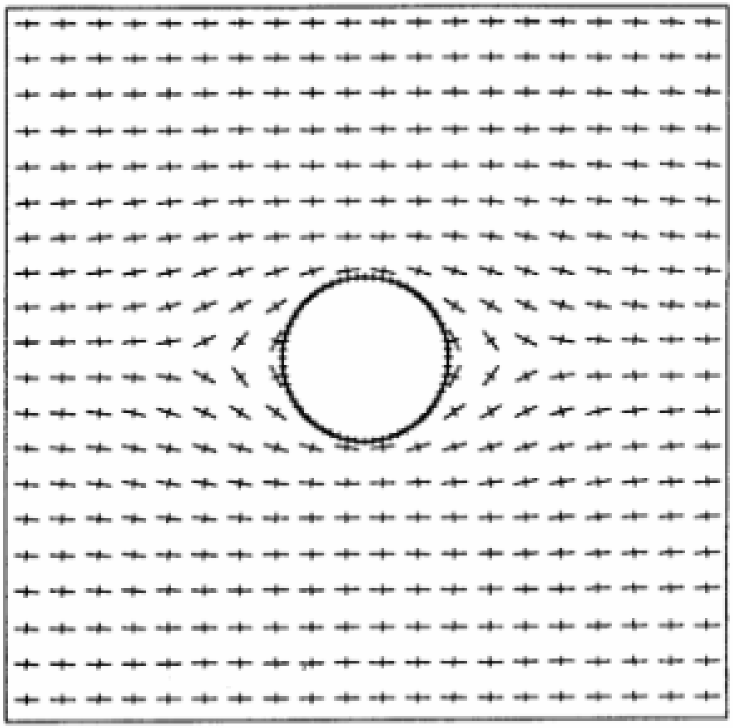

Figure 19 - Principal stress directions in the rock surrounding a horizontal tunnel (Hoek, 2007b).

The principal stresses are perpendicular to a plane, but they may be inclined to the direction of the applied *in situ* stress. Figure 19 shows the direction of the principal stresses surrounding a horizontal tunnel subjected to a horizontal *in situ* stress σ_{h1} equal to 3*σ_v where $\sigma_v$is the vertical *in situ* stress (Hoek, 2007b).

3.4 Stability problems caused by rock masses

The most fundamental concern in a design of underground excavations - such as tunnels, mine slopes or caverns - for civil and mining engineering projects are *in situ* stresses in rock. They play a crucial role in the stability of the rock mass. If the rock stresses are too low, this will lead to de-stressing in the rock after excavation; when rock stresses become too high, rock failure will happen due to stress concentration (Li, 2011). The only way of applying rock strength determination and failure theories to a rational design of rock excavation is to make an assessment of stresses in rock. This increases understanding of rock burst phenomena and predicts the destressing of rock (Herget, 1988).

3.4.1 Stresses in depths

Most excavations in earlier times were conducted in locations closed to the surface. It is a fact that *in situ* stresses become higher with depth. The rock stresses at low depth are most often lower than the strength of the rock, and the rock is characterized by well-developed rock joint sets. At low depths the most concern is falls of loosened rock blocks. In these cases, rock support is used to prevent the falling of blocks. The maximum load exerted on the support system is the dead weight of the falling block; that is, a load-controlled condition. It is important that the support system used in the excavation project is strong enough to bear the dead weight of the loosened rock blocks. For that reason, it is appropriate to use a safety factor[2] for the rock support design in a load controlled condition. This principle states that the load applied to a structure cannot be beyond the strength of the structure. It means that the safety factor should be larger than 1 for a safe design. In underground openings at shallow depths, this principle is valid because the load on the rock support system is mainly the dead weight of loosened rock blocks. The quality of a rock mass becomes better at greater depth, due to reduction in geological discontinuities compared to conditions in more shallow depths. Rock stresses, however, get higher as depth increases. Then, the greatest stability concerns in underground openings are rock failures caused by high stresses. Two consequences of high stresses are large deformation in soft and weak rock, and rockburst in hard rocks (Li, 2011).

3.4.2 In hard rock

When maximum tangential stress is greater than the rock mass strength, fracturing at the excavation boundary may occur in hard or brittle rocks. That is called spalling and causes instability problems during tunneling and other underground excavations. The spalling phenomenon occurs when a compressive stress induces crack growth behind excavated surfaces, causing buckling of thin rock slabs. The severity of spalling can vary from minor spalling to the complete collapse of excavations. Spalling may also increase chance for water loss from water tunnels during operation (Siren, Martinelli and Uotinen, 2011). When the rock stresses are very high (i.e., tangential stresses are much higher than rock mass strength) the rock mass may fail with loud sounds in brittle rocks, leading to rock burst. If one were standing close to the failure zone, one would hear the snapping noises of the rock failure accompanying slice ejections. This noise may continue for a period of hours or days afterwards, then gradually change to discrete muffled sounds when the fracture goes deeper into the rock (Li, 2012). A rock burst event is classified as either strain burst or fault-slip burst. If there is stress concentration in the nearby field of an underground opening the burst event is called strain burst. The stresses become so high that the rock is not capable of sustaining them so the rock

[2] Safety factor is defined as the ratio of the strength of the support system to the weight (i.e. load) of the block.

bursts out. Fault-slip burst occurs when strain waves[3] reach the walls and roof of openings and trigger the rock burst event. The fault-slip rock burst is usually more powerful than the strain rock burst (Charlie and Doucet, 2011). Many deep-level mines around the world have suffered from rock burst problems and there are reports available since 1942 where rock burst has been experienced. Rock burst is a serious problem that may lead to catastrophic situations in underground excavations; in extreme circumstances, loss of human life results (Hoek, 2007c).

3.4.3 In weak rock

Weak and soft rock is characterized by relatively low strength and low deformation modulus. Rock types like mudstone, chloritic and talc-rich rocks contain soft minerals and therefore belong to weak rock category (Li, 2012). When tunneling or excavating through very weak, highly sheared and deformed rocks, the rock mass may squeeze. The squeezing occurs when overburden stress exceeds the rock mass strength, and the pressure gradually builds up. This plastic deformation intensifies with increased time and tunnel advancement. When the strain exceeds 10%, extreme squeezing happens. Extreme squeezing can cause severe stability problems, which are quite challenging to address (Stefanussen, 1999).

Rock squeezing is always correlated with high stresses. The rock deformation increases with time because of the creeping behavior of weak and soft rocks. There will not be any volume increase related to the squeezing, but dilation occurs in the less confined direction, toward the opening space (Charlie and Doucet, 2011). In a mine drift located at 900m depth, observation showed that the wall convergence reached up to 600 mm in a three-month period. To constrain the rock squeezing to an acceptable extent in the area, a yieldable rock support system consisting of rock bolts and meshes was used. The rock bolts functioned as hanging elements to transfer the load on the meshes to the relatively competent rock mass behind (Li, 2012).

[3]Excavations of underground openings may result in reductions in the normal stresses on some pre-existing faults near the undergrond openings. This in turn brings about reductions in the shear resistance of the faults, and slippage induces strain/stress waves that propagate spherically outward from the site of slippage.

Figure 20 – Squeezing in a 10 meter span tunnel. Approximately 1 meter of inward displacement is in the roof and sidewalls (Hoek and Marion. 2000).

3.5 Bolts used as support in rock stresses

The major stability concern in shallow depths, where *in situ* stresses are usually low, are rock falls under gravity. The blocks are stabilized by installing rock bolts and other support devices. In these circumstances the bolts have to be strong enough to sustain the dead weight of the rock. Therefore, when selecting a bolt type for these situations, the most crucial parameter is strength. Fully grouted rebar bolts are the ideal type of bolt for this purpose since they fully utilize the strength of the bolt steel (Li, 2011).

However, the major shortcoming of rebar bolts is that they do not tolerate much deformation and therefore could not survive large rock dilations. A small amount of fracture opening would likely result in premature failure of the bolt material. In high-stress rock conditions at great depth, rock bolts are subjected to both shear and pull loads. The rebar bolts are too stiff to accommodate large displacement in creeping conditions, similar to the case of rock burst. When the bursting occurs the bolts simply fail because of their stiffness (Charlie and Doucet, 2011). Therefore, rebar bolts are neither appropriate for rock support in creeping conditions nor in bursting conditions. Bolts in high-stress conditions should have both high load bearing capacity and be able to accommodate large rock dilations; that is, they should be able to absorb a large amount of energy prior to failure (Li, 2009). Rebar bolts absorb little energy because of their small deformation, and Split Set (frictional bolts) absorb little energy because of their small load bearing capacity (Charlie and Doucet, 2011). Energy absorbing bolts like D bolts or cone bolts should be a suitable choice in high stress at great depth (Charlie and Doucet, 2011). In rock mass where rock burst occurs, minimum tensioning is applied to expansion shell-anchored bolts and resin-anchored bolts. Large steel plates are also used to avoid crushing the surrounding rock. In all situations, except where rock burst occurs, the bolts are only tensioned to 25-50 % of their yield strength (Hoek, E, 2012).

The safety factor for rock support systems in high-stress conditions is calculated on the basis of the released energy and the energy absorption of the support system. The safety factor is expressed as:

$$SF = \frac{Energy\ absorption\ of\ rock\ support}{Released\ energy\ from\ rock} \tag{3.4}$$

The principle above is also valid for weak rock where squeezing occurs, although it is not as easy to determine the released energy from squeezing rock as in the case of a rock burst event (Li, 2012).

4 Chapter – Design of bolting system

4.1 Introduction

Geotechnical discontinuities in the intact rock, material properties, distribution and magnitude of rock stresses, and size and shape of excavation openings are all factors that affect the design of a rock bolting system. To create a complete and appropriate rock bolting system design, it is important to properly investigate the following parameters:

- Selection of bolt type
- Bolt length
- Anchor capacity
- Pattern and spacing

Interactions between these parameters are very complex and our understanding of their mechanics remains imperfect.

4.2 Selection of bolt type

Different types of rock bolts are designed and manufactured to meet all kinds of geological situations and support requirements. For instance, some rock bolts are used as a temporary support while other types are permanent. Then there are bolts that are a more appropriate choice for hard rock conditions, while others are used mainly in weak rock mass. It is essential for a successful bolt type selection to examine carefully the geological conditions and understand the performance of different types of bolts under different settings (Smith, 1993). Guidelines for selecting the right bolt type are presented below:

Mechanically anchored rock bolts are used in:

- Harder rock condition in cases where the rock properties will not affect the gripping force of the anchor (Smith, 1993)
- Rock as spot bolting to secure unstable blocks at the tunnel face (Statens vegvesen, 1999)
- Rock as a temporary reinforcement with the exception of areas of high rock stress (Statens vegvesen, 1999)
- Rock that is not highly fractured (Smith, 1993)
- Water tunneling (Statens vegvesen, 1999)
- Where bolt tension can be checked regularly
- Rock that does not experience high shear forces
- Areas far from blast sites. In this situation, the bolt tension may be lost

(Smith, 1993)

Friction-anchored rock bolts are designed for use in:

- A variety of ground conditions
- Mining industry for medium-term support requirements – Swellex bolts
- Moving and bursting ground – Split Set
- Rock that needs relatively light support duties – Split Set
  (Hoek and Wood, 1987)
- Rock effectively as spiling as well as face, roof and wall support in any material where hole diameters can be maintained – Swellex (Atlas Copco, 2012)
- Daily support for optimum cost efficiency in mining and tunneling areas not requiring rock bolts' ductility (Atlas Copco, 2012)
- Rock not needing long-term installations. Corrosion can be a problem in these situations (Hoek and Wood, 1987)

Grouted bolts are used in:

- Conditions where the usage of mechanical bolts is not recommended
- Permanent reinforcement systems
- Rock where wide fractures and voids are not present. This may lead to a loss of significant amounts of grout
- Boreholes where continuous water run-off will not interfere with the installation
- Rock to prevent both vertical and horizontal strata movements
- Rock blasting. The bolt absorbs blast vibrations without bleed-off of the bolt load (JunLu, 1999)

Untensioned grouted bolts are recommended in:

- Rock that is highly fractured and deformable as long as adequate bolt installation is feasible

Tensioned grouted bolts are recommended:

- Where additional frictional forces may enhance roof stability in combination with a grouted column (JunLu, 1999)

As mentioned earlier, it is a general rule that mechanical bolts cannot be reliably installed in critical areas as permanent support. Furthermore, they are not suitable where roof deformation may likely occur, or in rock like shales and clays that are extremely soft and exhibit highly plastic behavior. However, when selecting temporary reinforcement systems, mechanical bolts should be the first option. To ensure the best performance of mechanical bolts, if they are chosen, the following guidelines should be followed:

1) To prevent strata separation it is important to bolt the roof as soon as possible after the excavation.
2) On-site pull out tests should be made to determine the optimum expansion shell and the optimum anchorage horizon.
3) To reinforce the strata effectively, employ substantial structural components, such as high strength bolts and plates with low deformation characteristics
4) Ensure all bolts are working together. That is done by installing them with equal tension (Peng and Tang1984).

Table 6 - Suggested support type for various rock conditions (Hoek and Wood, 1987).

Rock Conditions	Suggested support type
Sound rock with smooth walls created by good blasting. Low in situ stresses.	No support or alternatively, where required for safety, mesh held in place by grouted dowels or mechanically anchored rock bolts, installed to prevent small pieces from falling.
Sound rock with few intersecting joints or bedding planes resulting in loose wedges or blocks. Low in situ stresses.	Scale well then install tensioned, mechanically anchored bolts to the blocks into surrounding rock. Use straps across bedding planes or joints to prevent small pieces falling out between bolts. In permanent openings, such as shaft stations or crusher chambers, rock bolt should be grouted with cement to prevent corrosion.
Sound rock damaged by blasting with a few intersecting planes. Low in situ stresses.	Chain link or weld mesh held by tensioned mechanically anchored rock bolts, to prevent falls of loose rock. Attention must be paid to scaling and to improving blasting to reduce amount of loose rock.
Closely jointed blocky raveling from surface causing deterioration if unsupported. Low stress conditions.	Shotcrete layer, approximately 50 mm thick. Addition of micro-silica and steel fiber reduces rebound and increases strength of shotcrete in bending. Larger wedges are bolted so that shotcrete is not overloaded. Limit scaling to control raveling. If shotcrete not available, use chain-link or weld mesh and pattern reinforcement such as split sets or Swellex.
Stress-induced failure in jointed rock. First indication of failure due to high stresses are seen in borehole walls and in pillar corners	Pattern support with grouted dowels or Swellex. Split sets are suitable for supporting small amounts of failure. Grouted tensioned or untensioned cables can be used but mechanically anchored rock bolts are less suitable for this application. Typical length of reinforcement should be about ½ the span of openings less than 6 m and between ½ and 1/3 for spans of 6 to 12 m. Spacing should be approximately ½ the dowel length. Support should be installed before significant movement occurs. Shotcrete can add significant strength to rock and should be used in long-term openings (ramp etc.). Mesh and straps may be required in short-term openings (drill-drives etc.)
Draw points developed in good rock but subjected to high stress and wear during blasting and drawing of stopes.	Use grouted rebar for wear resistance and for support of draw point brows. Install this reinforcement during development of the trough drive and draw point, before rock movement takes place as a result of drawing of stopes. Do not use shotcrete or mesh in draw points – place dowels at close spacing in blocky rock.
Fractured rock around openings in stressed rock with a potential for rock bursts.	Pattern support required but in this case flexibility required absorbing shock from the rock burst. Split sets are good since they will slip under shock loading but will retain some load and keep mesh in place. Grouted dowels and Swellex will also slip under high load but some face plates may fail. Mechanically anchored bolts are poor in these conditions. Lacing between heads of reinforcement helps to retain rock near surface under heavy rock bursting.
Very poor rock associated with faults or shear zones. Rock bolts or dowels cannot be anchored in this material.	Fiber-reinforced shotcrete can be used for permanent support under low stress conditions or for temporary support to allow steel sets to be placed. Note that shotcrete layer must be drained to prevent build-up of pressure behind the shotcrete. Steel stets are required for long-term support where it is evident that stresses are high or that rock is continuing to move. Capacity of steel stets are estimated from amount of loose rock to be supported.

4.3 Bolt length

Bolt length is generally based on the total thickness of unstable strata, especially when they are used to control stability. In underground situations such as caverns or tunneling, the maximum length of a rigid bolt is limited by the roof-to-floor height of the opening. There are, however, some types of bolts that can be bent before they are inserted into the boreholes and then straightened afterwards. Fully grouted rebar bolts can be bent and then straightened afterwards, but the bolt strength is reduced. Cable bolts can overcome this limitation, as they are bolts with great flexibility. When rock bolts mainly act in suspension, they have to be long enough to be firmly anchored in a competent rock mass. If suspension is not allowed in the situation, the bolts have to be long enough to create beam-building effects or keying effects (See chapter theories of rock bolting) (JunLU, 1999).

Biron and Arioglu (1982) developed a guideline to determine bolt length in underground openings. There is a linear relationship between bolt length and the roof span. According to the guidelines, bolt length for strong roofs is about one-third of the roof span, and for weak roofs bolt length is about one-half of the roof span. Bolting in a very strong roof, where they are only meant to prevent spalling, has a minimum recommended bolt length of 3 feet 4 inches. Some general empirical rules for bolt lengths have been provided; these rules are as follows:

Estimated minimum bolt length should be the greatest of the following three:

- Twice the bolt spacing;
- Defined by the average discontinuity spacing in the rock mass the minimum bolt length should be three times the width of critical and potentially unstable rock blocks;
- If the span is less than 20 feet, bolt length should be one-half the span. For spans between 20 to 60 feet, linearly interpolate between 10- to 16-foot lengths, respectively. Higher than that, sidewall bolts in excavations should be one-fifth of wall height. According to section 74 of Code of Federal Regulations 30 (from 1977), no length of rock bolts used in a roof should be less than 2.5 feet plus 1 foot if anchored in the stronger strata to suspend the immediate roof.

(JunLU, 1999).

When the length of rock bolts is chosen, it matters whether they are to be used, for example, in tunnels or caverns. The typical length of rock bolts in tunnels is 2-4 meters and the diameter of the bolts is 20-25 mm. In caverns, the bolts' length is generally quite longer. The typical length is 6 meters with a diameter of 25-32 mm. In Norway, when the length of rock bolts is selected for tunnels, the following equation is applied:

$$Lb = 1.4 + 0.184Dt \tag{4.1}$$

Dt stands for the diameter of the tunnels in meters.

Another equation has been suggested for selecting the length. This equation takes into account the rock mass condition when designing the bolts, especially the size of the block.

The equation is:

$$Lb_{roof} = 1.4 + 0.16\mathrm{Dt}\left(1 + \frac{0.1}{Db}\right) \tag{4.2}$$

$$Lb_{roof} = 1.4 + 0.08(Dt + 0.5Wt)\left(1 + \frac{0.1}{Db}\right) \tag{4.3}$$

Wt stands for the height of tunnel wall in meters and Db for the block diameter (Nilsen and Palmström, 2000).

4.3.1 Q system used for selecting bolt length

In 1974 the Norwegian Geotechnical Institute (NGI) developed the Q system for rock mass classification. This classification system is for estimating tunnel support and is based on numerical assessments of rock mass quality:

- Rock quality designation - RQD
- Joint set number - Jn
- Joint roughness - Jr
- Joint alteration - Ja

- Joint water reduction - Jw
- Stress reduction factor - SRF

The parameters above are grouped into three quotients that give the overall rock mass quality. The first two represent the overall structure of the rock mass. Their quotient is a relative measure of the block size. The second quotient is described as an indicator of inter-block shear strength and the third quotient is described as the active stresses. The Q value relates to tunnel support requirements by defining the equivalent dimensions of the underground opening. Equivalent dimension is a function of size and type of the excavation. By dividing the diameter, height or the span of the excavation by the excavation support ratio (ESR) we will get the equivalent dimension. ESR is not the same for different types of underground openings. For power stations, roads and railway tunnels, ESR is 1.0

By using the Q system chart (see Figure 21) we get an indication of the support system that should be used for each case, whether it is bolt support or use of shotcrete. The length of bolts can be decided using the chart and bolt spacing both in un-shotcreted and shotcreted areas. Other necessary information from the chart can be seen in the figure below (Nilsen and Palmström, 2000).

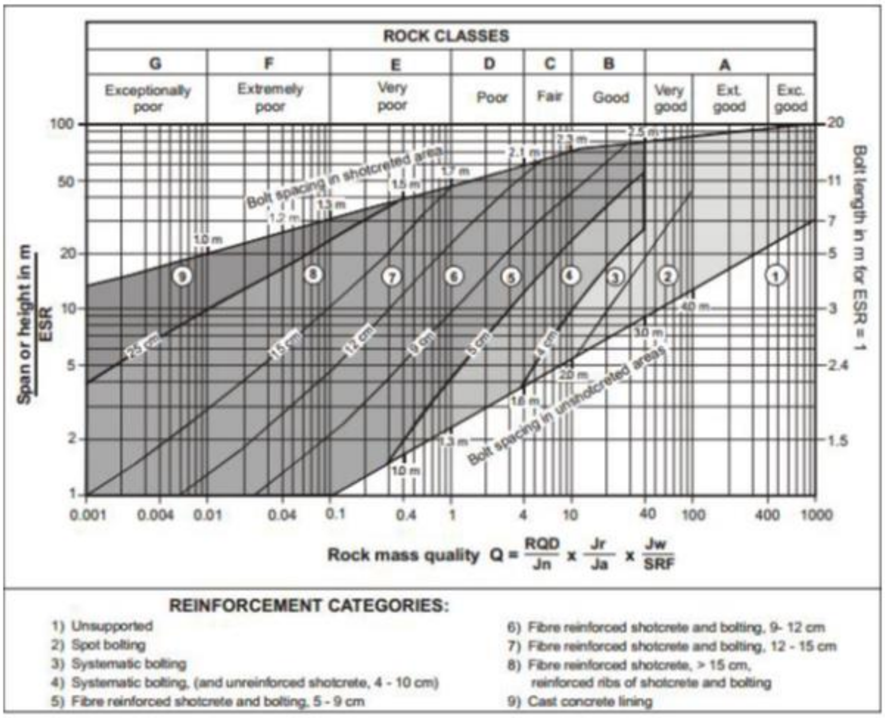

Figure 21 – Q system chart gives an indication of the support system; length of bolts, bolt spacing and shotcrete (Barton et al, 1974).

4.4 Anchor capacity, pattern and spacing

The intensity of support provided by a rock bolting system is determined by three main factors:

• Capacity of the individual bolts

• The density of the bolt pattern

• The length of the bolts

The capacity of rock bolts is determined by the diameter and size of the bolt, its strength and its anchorage capacity (Mark, Molinda and Dolinar, n.d.)

The anchorage capacity depends on different things for various bolt types. Anchorage capacity for mechanical bolts depends on how firmly the expansions shell grips against the borehole wall. The maximum gripping is determined by the rock type and the integrity of the rock surrounding the anchorage area. In most cases, stronger rock or rocks of higher integrity provide better anchorage for mechanical bolts. The best way to determine the maximum carrying capacity, where there is no anchoring slipping or failure, is an underground *in situ* pull test. When defining the difference between good anchoring and poor anchoring, the movement and the yield strength are measured. A good anchoring system has minimum movement and an anchorage capacity exceeding the bolt yield strength. A fair anchorage is one whose capacity is equal to or slightly exceeds the bolt yield strength. Finally, poor anchorage moves excessively with loads below the bolt yield strength (Peng, 1984). Research has shown that fully grouted rock bolts are at least twice as strong as a mechanical bolt of the same size. The clearance between fully grouted bolts and the borehole is important for anchoring capacity and rigidity. By increasing the clearance, both rigidity and capacity decrease (JunLu, 1999).

The density of rock bolt patterns in underground openings depends on the number of bolts per row and the spacing between rows. In the United States, coal mines follow a systematic pattern of bolting, regardless of the bolt length. It has become the near-universal standard to support four bolts per row (Mark, Molinda and Dolinar, n.d.).

The bolts are most often installed vertically, and in some situations at an incline. Systematic bolting does not work under all circumstances. Fairhurst and Singh (1974) pointed out that sometimes a systematic pattern works too well by leaving the supported rock intact. In that case, the entire section of the supported rock may fail as a unit. This failure, according to

Jorstad (1967), is associated with artificial fracturing caused by the longest bolts. Other research has shown that fractures are more likely to form as the bolt spacing is reduced. There are many factors that affect bolt spacing, such as location of weakness planes, strata thickness, roof conditions in underground excavations, bolt tension, and bolt characteristics such as yield strength, length, and diameter. These factors affect bolt spacing because bolt patterns and strata interact (JunLu, 1999).

The bolt spacing can be estimated as:

$$b = \frac{2}{3} l \tag{4.4}$$

Or

$$b = \frac{2}{9} L \tag{4.5}$$

Where:

$b = \text{Bolt spacing}$

$l = \text{Bolt length}$

$L = \text{Roof span}$

To measure the intensity of the rock bolts, a summary variable has been developed, ARBS (*Analysis of Roof Bolt Systems*) that includes all three factors mentioned above. The equation is as follows:

$$ARBS = \frac{(L_b)(N_b)(C)}{(S_b)(W_e)} \tag{4.6}$$

Where:

Lb = Length of the bolt

Nb = Number of bolts per row

C = Bolt capacity

Sb = Spacing between rows of bolts

We = Entry width (span) (Mark, Molinda and Dolinar,n.d).

There are some general rules to check for rock bolt spacing. The maximum bolt spacing should be at least:

- One-half the bolt lengths;
- One and one-half the width of critical and potentially unstable rock blocks;
- 6 feet.

Minimum bolt spacing should not be less than 3 feet (JunLu, 1999).

5 Chapter – Pull-out test

5.1 Introduction

Following is a definition of a pull out test:

> *Rock bolts are installed in the same manner and material as their intended construction use. They are pulled out hydraulically and at the same time the displacement of the bolts head is measured. The rock bolts are pulled until the anchor system or the rock fails. The results from the testing are calculated from the plot of load versus displacement and that gives us the ultimate and working capacities of the rock bolts* (Scribd, 2011).

The objective of pull out testing is to measure the working and ultimate capacities of a rock bolt anchor. Ultimate capacity is the maximum load sustained by the anchor system and working capacity is the load on the anchor system at which significantly increasing displacement begins. Load is the total axial force on the rock bolt. The displacement is an important measure to find the ultimate and working capacities. The displacement is the movement of the rock bolt head. Failure in rock bolt testing happens when the anchor system or rock are not able to sustain increased load without rapidly increasing deformation. Sometimes the peak load itself cannot be sustained.

Pull-out testing does not measure the entire roof support system nor include tests for pre-tensioned bolts or evaluation on mine roof support system. Pull-out tests apply to mechanically, cement- and resin-grouted, or other similar anchor systems.

Information gathered from pull-out tests may give a quantitative measure of the relative performance of different anchor systems in the same rock type. Data from the testing can be used to select a suitable anchor type and determine bolt length, spacing and size.

It is important to conduct anchor pull tests in all rock types in which construction bolts will be installed. Also, a sufficient number of tests should be taken to determine the average bolt capacities (ASTM International, 2014)

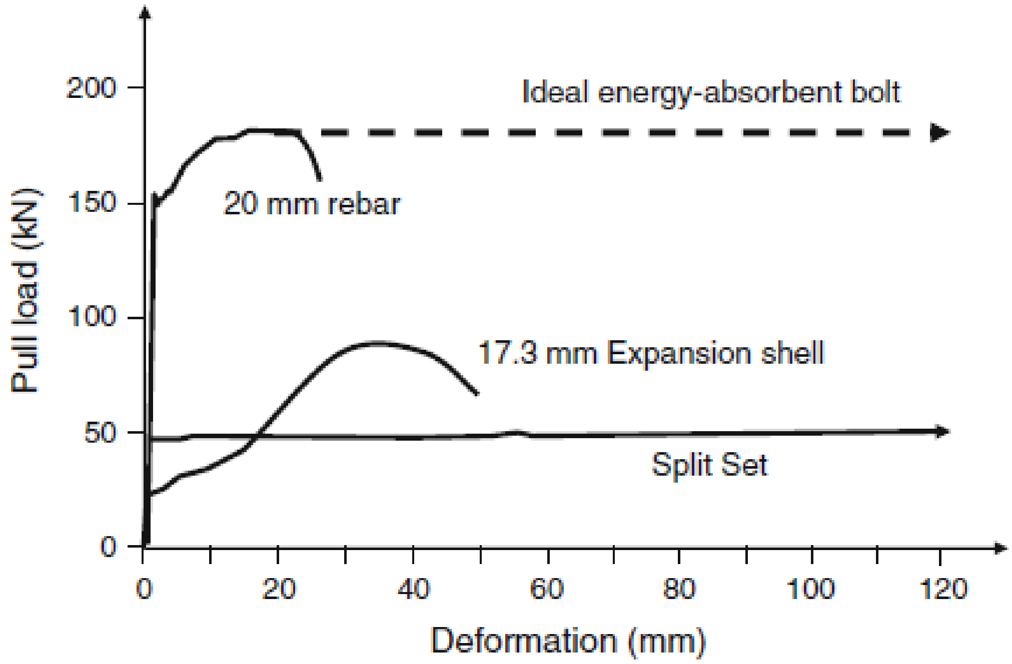

Figure 22 - Typical pull load–displacement curves for fully bonded rebar, expansion shell and frictional bolts are presented in the figure (Li, 2011).

5.2 Testing procedure

When a pull-out test is performed the anchors used should be from the manufacturer's standard production stock. The testing should be adjusted to the specific anchor system to get the most relevant results. If mechanical anchors are tested, it is important to make sure that no anchors are defective. The anchors should be correctly sized for the hole's diameter, and the size of the anchors must be known. If grout or resin are used as anchors, they have to be fresh. The resin cartridge size should be compatible with the borehole diameter, bar diameter and the length of anchorage required. If grout is injected, the mixing and the injection equipment have to be compatible with the recommendations of the manufacturer.

Before the bars are injected, the boreholes are washed or blown to keep all cuttings away from the hole. There is no need for the holes to be as deep as the length of the bolts. However, they have to be deep enough to keep the anchor from the zones of disturbance caused by any excavation during field testing. The borehole has to be straight for a pull-out test. If it is not possible to see more than one-half of the hole's bottom, the hole is not straight enough and therefore cannot be used (ISRM, n.d.)

Pull-out tests are always performed on untensioned rock bolts[4]. If tests are performed on fully grouted bolts it is important to consider the recommended curing times for the resin or grout. The time varies considerably and this may have significant effects on strength (See chapter about water-to-cement ratio). To assess the effect of grouted bond length on anchor strength, more than one anchorage length should be tested, using similar curing times (ASTM, 2014). Finally, when the estimated curing time is completed, the rock bolts are pulled out hydraulically while the displacement of the bolt's head is measured. The equipment used for pulling out the bolts should be capable of applying a load greater than the strength of both the anchor and the bolt to be tested. It is vital that the applied load is axial with the bolt; a wedge is used to ensure this (ISRM, n.d.).

Figure 23 - It is vital that the applied load is axial with the bolt; a wedge is used to ensure this.

The anchor is tested by increasing the load until the bolt yields, breaks, or slides from the grouting without any yielding. Fully grouted rebar bolts start to yield around 170 kN, and from

[4] Tension bolts can provide extra force across the discontinuity surfaces and hence inhibit further block displacement. However, in tensioned bolts the tension may not be sustained over the design life, due to relaxation. The advantage of untensioned bolts is that block displacements induce the necessary tension within them, due to dilation of the shearing discontinuity (Harrison and Hudson, 2000).

that point up to 200kN the rebar yields before it breaks (Charlie Li, Professor at Department of Geology and Mineral Resources Engineering, oral source, 15. March 2014).

5.3 Load-bearing capacity of bolts

Results from pull-out tests give information about axial load bearing capacity of rock bolts. They also give recommendations for selecting appropriate bolt types for specific purposes (Li, 2013). It is important that an installed support system is in harmony with the ground behavior to reach optimal equilibrium in an economical and timely manner without any possibility of jeopardizing safety (Foo et al., 2011).

In accordance with load deformation performance, rock bolts can be categorized into three groups:

- **Ductile bolts**
- **Strength bolts**
- **Energy-absorbing bolts.**

Ductile bolts are capable of accommodating large deformation but do not bear high load. Strength bolts, on the other hand, have high load-bearing capacity but low deformation. The third group, energy-absorbing bolts, are capable of both, bearing high load and also accommodating large deformation. They are a suitable choice for rock reinforcement in highly stressed rock (Li, 2012).

Figure 24 presents results from pull-out tests carried out in a laboratory. It shows examples of non-linear support reaction curves for various types of rock bolt.

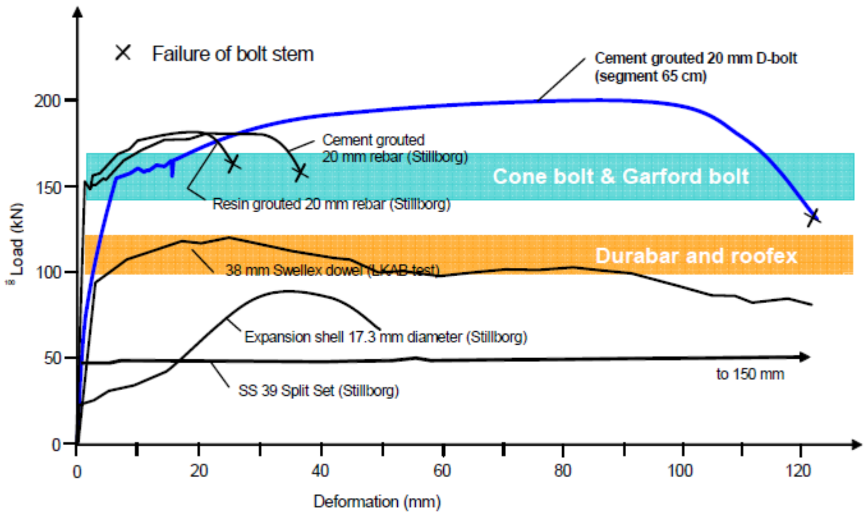

Figure 24 – Results from pull-out tests carried out in a laboratory (Li, 2013).

The figure shows that all grouted bolts have rather low deformation, meaning that they accommodate relatively low displacement. For example, the displacement when 20 mm cement-grouted rebar bolts rupture is around 30-40 mm. They can be described as stiff reinforcing elements. However, all grouted rock bolts have high load-bearing capacity, as they tolerate more than 150 kN (Li, 2013). Fully grouted rebar bolts qualify as an example of strength bolts (Li, 2012). Swellex and Split Set are friction-anchored rock bolts. Friction bolts can accommodate large rock formations, but their load-bearing capacity is rather low (Li, 2011). The figure shows that Swellex bolts have rather high load-bearing capacity (around 120kN) compared to Split Set, which have a load bearing capacity around 50kN. However, both types can accommodate large displacement, up to 150 mm Charlie, 2013). Both Split Set and Swellex are classified as ductile bolts (Li, 2011).

Mechanically anchored rock bolts can bear a relatively high load and are able to deform a bit more than the grouted rock bolts. Expansion shell bolts are mechanically anchored and can bear loads up to75 kN and may accommodate a displacement up to 50 mm. The blue curve in Figure 24 represents the results from pull-out testing on D bolts. The results show that D bolts can tolerate high loads and also large deformation of the rock. None of the other bolt types can both tolerate large deformation and bear high loads as well as D bolts. It can be estimated from this information that D bolts may be important for getting maximum safety benefits. D bolts are categorized as energy-absorbing bolts (NTNU, 2014).

Energy-absorbing bolts are used both in mines and civil tunnels today. When the bolts are put in a borehole, the shank is fully encapsulated. A sleeve that contains a thin-walled steel tube is connected to the bolt head, and when the bolt is pulled in the rock the steel tube is pressed by the head. When the tube starts to move, the sleeve stops it and the tube is forced to buckle to accommodate the rock's deformation (Li, 2011). Conventional rock bolts often fail prematurely in high rock stresses. The ideal rock bolts in that circumstances should be able to accommodate large rock dilations and absorb a large amount of energy prior to failure. Energy-absorbing bolts are the ideal bolts in that case.

D bolts (short for "deformable bolts") are different from other energy-absorbing bolts in that they are multi-point-anchored when all other types are two-point anchored. D bolts are fully bonded in the borehole with resin or cement grout (Li, 2012). The anchors in D bolts are stronger than the shank of the bolt and they are firmly fixed in the grout. The smooth bar section between anchors is weakly bonded to the grout. When the anchors dilate, they will restrain the dilation so that a tensile load is induced in the smooth bar. The bar is able to elongate plastically until its ultimate strain limit is reached. All the smooth bar sections in the rock bolt act independently. If one section fails, it does not affect other sections of the bolt, localizing any effects (Li, 2010).

5.4 Pull-out testing - previous research

Pull-out tests on fully grouted cable bolts

Investigation on major factors influencing the bond capacity of grouted cable bolts was conducted both in the laboratory and the field by Hyett, Bawden and Reichert (1992). The bolts were grouted using Portland cement. The results from their investigation revealed the most critical components for cable bolt capacity: first, the cement properties which are controlled by water/cement ratio; second, the embedment length; finally, the radial confinement that acts on the outer surface of the cement annulus. Grout with rather low water/cement ratio (<0.40) can increase peak cable bolt capacities by 50-75%. However, if the ratio goes under 0.30, it will become a super-thick paste and will both be impractical and undesirable. Different embedment lengths in pull-out testing indicated that capacity of the cables increased with embedment length. When determining the embedment length for cables, the joint spacing along the axis of the bolt should be examined. Results from RQD[5] may give valuable information about the rock mass quality of where the bolts are injected.

Influence of the bolt diameter, length and variant in water to cement ratio on bond strength of fully grouted bolts

It is important to have a good understanding of the mechanism of load transfer in rock bolts as well as the behavior of the bolt-grout interaction. This knowledge improves the performance of fully grouted bolts. As the performance of fully grouted bolts depends on bond strength, pull-out tests were performed by Karanam and Dasyapu (2003) with various bolt diameters, lengths and water-cement mixing ratios of grout. The investigations were conducted on smooth bolts to check the influence of the bolt diameter, bolt length and variant in w/c ratio on the bond strength of the fully grouted bolts. The grout composition consisted

[5] **Rock quality designation or RQD is determined by measuring the core recovery percentage of core chunks greater than 100 mm in length. Core that is not hard or sound should not be counted even if they are 100 mm in length.**

of the following w/c ratios: 1:1, 0.67, 0.5, 0.4 and 0.38. Water/cement ratio under 0.38 was not possible because the grout was too thick. The borehole was fixed at 33 mm diameter, but changes in bolts diameter were made to check the effect of grout thickness on the bolt diameter. The diameters of the bolts were: 9.525, 12.7, 19.05 and 25.4mm. After three weeks of curing, a pull-out test was performed on dry samples. The bond strength was found by dividing the maximum load when the bolts failed with the surface area of the bolts. Table 7 gives all the bond strength values obtained by pull-out test with different values. The results showed that the bond strength became higher with lower water to cement ratio and greater bolt diameter. The bond strength also increased with longer bolts.

Table 7 - Bond strength values obtained by pull-out tests with different values (Karanam and Dasyapu, 2003).

Bolt diameter (mm)	**Bond strength (MPa)**				
	c/w: 1:1	c/w: 0.67	c/w: 0.50	c/w: 0.40	c/w: 0.38
9.525	0.5358	1.0280	1.0500	1.0798	1.1098
12.70	0.9644	1.6072	1.6480	2.0090	2.2050
19.05	1.0046	1.6740	1.7690	2.0580	2.3040
25.4	1.2446	1.7808	1.8032	2.1560	2.4000

Pull-out tests performed on different bolt types

Pull-out tests were conducted in a laboratory by Fumio et al (2001). Bolts were grouted with cement paste into an artificial rock made of concrete. The curing time of the artificial rock was 14 days. Different bolts types were installed into the rock and grouted with Portland cement with a water-to-cement ratio of 0.35. The same embedment length (35 cm) was used in all the test cases. The curing time of the grout was 17 days and its uniaxial compressive strength was measured to be 89.7 MPa. Two types of rock bolts were used for the pull-out testing; a deformed bolt with diameter of 25 mm and a twist bolt with diameter of 24 mm. Two types of cable bolts were also used; a plain strand cable bolt and a bulb strand cable bolt; both bolt types were 15.2 mm diameter. The bolts were all pulled up with a hydraulic cylinder linked with an electric oil pump which contained a pressure cell to measure the pulling force. Force and displacement were recorded.

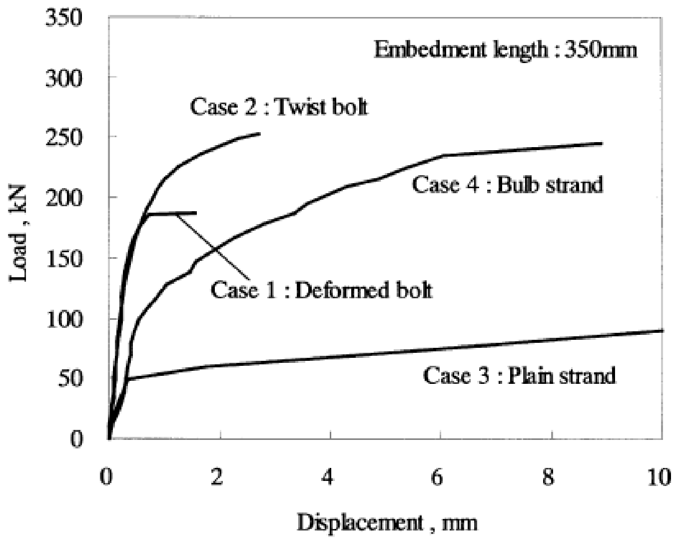

Figure 25 – Load displacement curves for all the cases (Ito et al, 2001).

Figure 25 shows results from the pull-out tests with load-displacement curves for all the bolts. The results show the strong effects of bolt type on the deformational behavior. The point where the bolt begins to slide away from elastic behavior is defined as the yielding point. The curves in the figure show that the yield forces in case 1 and 2 are 170 kN and 215 kN, but the values for cases 3 and 4 are lower. However, the initial stiffness is greater for case 1 and 2 and the displacements at the yielding points are smaller. To summarize this from the design point of view, the yield force of the rock bolts are greater than that of cable bolts, but the displacement of the rock bolts are lower.

Investigation on the effect of confining pressure on bond capacity of bolts

Laboratory test were performed by Mossavi, Jafari and Khosravi (2003). The aim was to investigate the effect of confining pressure on bond capacity of bolts. Two different types of rock bolts were considered for the research: 20 mm Dywidag continuous thread (CT) bar and a 22 and 28 mm ribbed rock bolts. The bolts were pulled axially through a Portland cement grout with 0.40 in water/cement ratio. The bond may be defined as the gripping effect of the cement on an embedded length of steel bar to resist forces tending to slide the bar. Properties of both the cement and the bar play large roles in developing high or low values of bond capacity. Compressive strength of the grout and the smoothness and shape of the bar are examples of important properties. Slip between rebar and grout is mainly a frictional failure. The confining stress (normal stress) plays a big part in this load utilization. Stresses vary significantly in underground excavations. In some situations the stress increases, while in others it decreases. These changes affect the bond capacity of the rock bolts.

It is also important to examine the quality of the Portland cement grout. The strength of the grouting may vary if it is produced by different factories, even though it has been mixed with exactly the same water/cement ratio. Uniaxial compressive strength (UCS) tests of the grout were performed in this research to avoid any inconsistency between the strength of different cement grouts. Often, UCS testing of the grout samples is recommended, rather than only mentioning the water/cement ratio of the grout. The UCS tests on the grout gave results equal to 30 and 42 MPa. During the pull-out testing, the axial load and the displacement of the bolts as well as the radial dilation of the grout were recorded. The results were presented in load and bond capacity. The load-bearing capacity results showed that the axial load increased linearly to a point where divergence began from the elastic behavior. As confining pressure increased, the peak point in most cases shifted more to the right, which means higher axial displacement. At this load level, radial cracks in the sample have most likely fully developed, leading to a decrease in bond strength. Figure 26 shows the results of a pull-out test for a 20 mm Dywidag bar with UCS=42MPa for the grout.

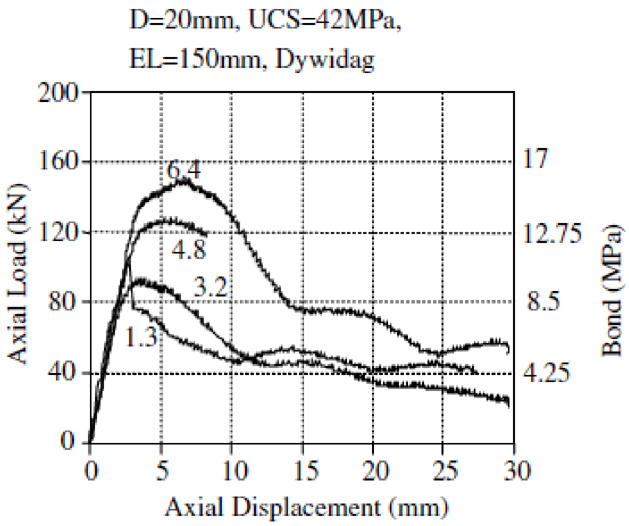

Figure 26 - Results of a pull-out test for 20 mm Dywidag bar with UCS=42 (Mossavi, Jafari and Khosravi, 2003).

Effect of grout properties on the pull-out load capacity of fully grouted rock bolt

Approximately 80 laboratory pull-out tests were performed on rebar bolts grouted into basalt block with cement mortar. The tests were carried out in order to explain and develop the relations between untensioned rebar bolts and the grouting material. This simple pull-out test program evaluated the relations between: bolt diameter and pull-out load of bolt; bolt area and pull-out load of bolt; bolt length and pull-out load of bolt; water-to-cement ratio and bolt bond strength; and mechanical properties of grout material and bolt bond strength, curing time and bolt strength. The grouting mixture was Portland cement with a water/cement ratio of 0.34, 0.36, 0.38 and 0.40. The curing time was 28 days. The bolts had been inserted to the center of the boreholes and when the curing time was over the rebar bolts were axially loaded and the load gradually increased until the rebar failed. The bond strength was calculated by dividing the load by the surface area of the rebar. More pull-out test were performed with various grout types, bolt dimensions and curing times.

The results from the testing showed that the bolt capacity depends mainly on the mechanical properties of the grouting material. Changes can be made in the grouting, as in water-to-cement ratio, mixing time, additives and curing time, which can lead to both increases and decreases of the bolt bearing capacity. Increasing the curing time increased the bolt bond strength. The first day, the bolt bond strength was 19 kg/cm, 2.77 kg/cm2 in 7 days and 86 kg/cm2 in 35 days (see Figure 16). By increasing the bolt diameter and the length, the bearing capacity will become higher. However, this increase is limited to the ultimate tensile strength of the bolt material. In the pull-out test, bond failure occurred between the bolt and the cement grout (Kılıc, Yasar and Celik, 2002).

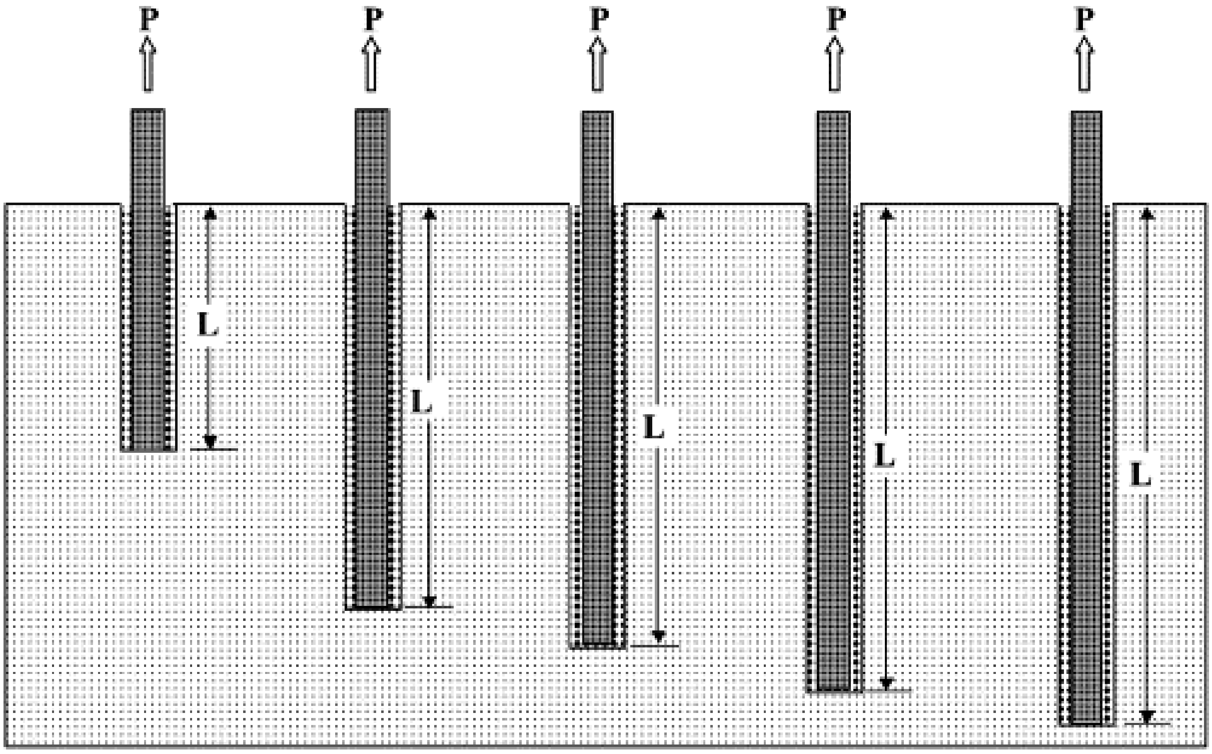

Figure 27 – Rock bolts grouted with different embedment length. By increasing the length, the bearing capacity will become higher (Kılıc, Yasar and Celik, 2002).

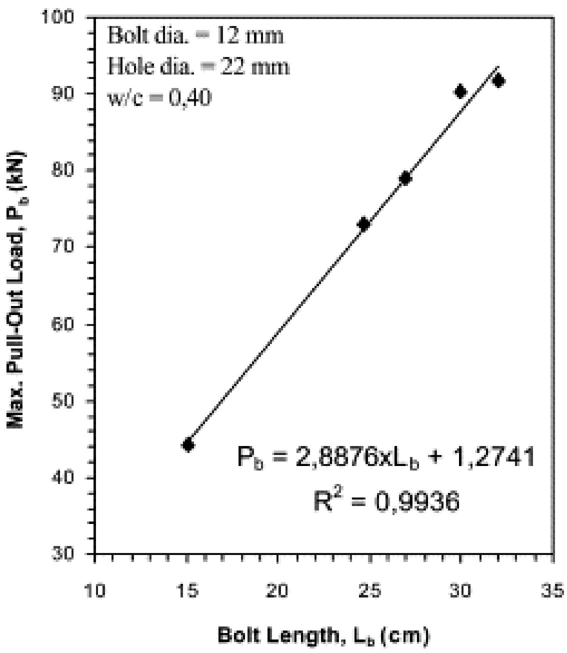

Figure 28 – Maximum pull-out load versus bolt length. The load increases with more bolt length (Kılıc, Yasar and Celik, 2002).

Performance of D-bolts under static loading

Pull out tests on D bolts were performed by Charlie and Doucet (2011) in the rock mechanics laboratory at the Norwegian University of Science and Technology. The bolts were 20 mm in diameter and 0.9 m/0.8-m long (test section/ stretch length). To simulate the rock mass, two concrete blocks of high strength were used. The blocks were placed in a test rig and holes were drilled. The bolts were encapsulated in the boreholes with cement mortar and the water-to-cement ratio of the mixture was 0.35. The curing time was 3 days, and then the tests were performed. Results are shown in Figure 29.

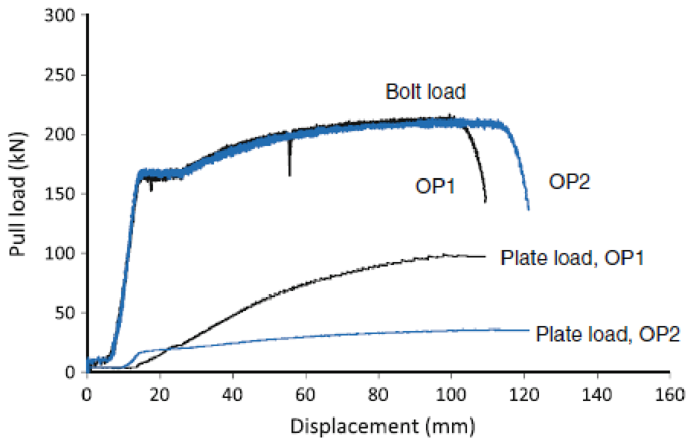

Figure 29 – Results from pull-out test (Charlie and Doucet, 2011).

At a displacement of 110 mm the bolt OP1 failed, while OP2 failed at a displacement of 120 mm. Both bolts started to yield at 170 kN and had a tensile strength up to approximately 210 kN. Figure 30 shows the bolt, OP1, after the pull-out test.

Figure 30 – Bolt OP1 after the pull-out test. The bolt failed at the displacement of 110 mm. It started to yield at 170 kN and had a tensile strength up to approximately 210 kN (Charlie and Doucet, 2011).

Rebar bolts and D bolts have the same tensile strength and in the rock mass they react in quite similar ways until the yielding starts. Until the ultimate strain of the D bolt steel is reached, every section of the D bolts can elongate at yield. The D bolts have significantly larger plastic deformation capacity than rebar bolts.

6 Chapter – Pull-out test performed on rebar bolts

6.1 Introduction and purpose

Pull-out testing was carried out in a laboratory at Norwegian University of Science and Technology (NTNU). In collaboration with Statens Vegvesen and Professor Charlie Li, supervisor of this project, it was decided to perform pull-out tests on 20 mm rebar bolts and grout the bolts with Rescon Zinc bolt cement mortar (Appendix A). The material is the same as its intended construction use. The rock bolts are also installed in the same manner. Both rebar bolts and the cement mortar are commonly used in underground projects in Norway.

This test was performed to evaluate the critical length of fully grouted rebar bolts (see figure 31). This critical length is defined as the greatest grouted length of the bolt wherein the bolt is pulled out hydraulically without the failure of the rod. From a plot of load versus displacement the load bearing capacity of the bolt can be seen. In this pull-out test different embedment lengths and variation in cement-water mixing ratios of grout were used. Uniaxial compressive strength testing was also performed on the cement mortar with variation in cement-to-water mixing ratios. The ultimate load bearing capacity of the 20 mm rebar bolts used in the tests is 200kN. The steel strength of the rebar is 630 MPa, where it is found by multiplying the cross section of the rebar bolts with the ultimate load.

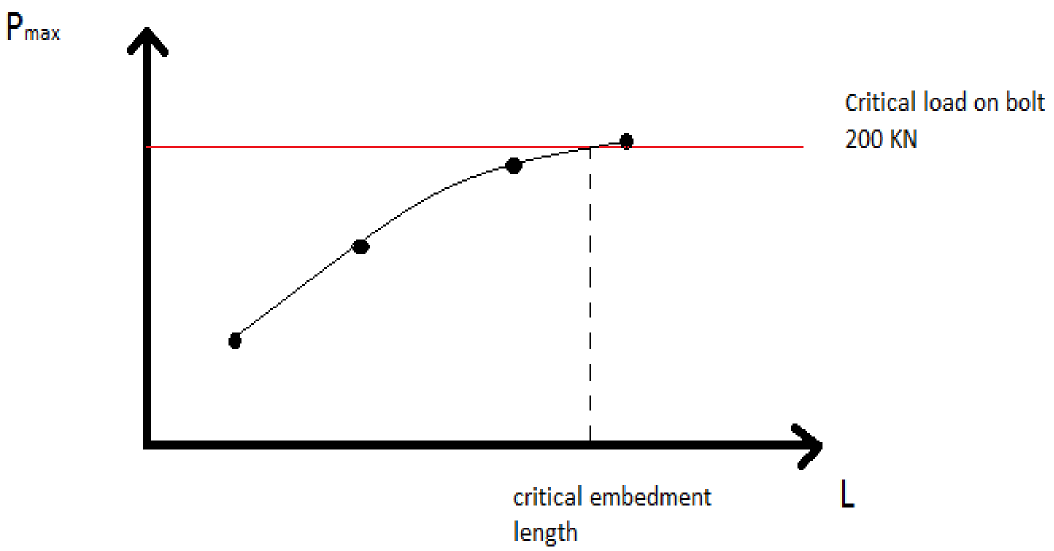

Figure 31 - The critical length is defined as the greatest grouted length of the bolt wherein the bolt is pulled out hydraulically without the failure of the rod. The first three points (bolts) on the graph are pulled out without any failure. The embedment length for point number four is longer than the critical embedment length. It means that before the bolt is pulled out it breaks. It is necessary to test two bolts, one just before the critical point and other right after. The rebar bolt used in this test is capable of bearing 200 KN before it breaks. As embedment length increases, more loads are required to obtain failure.

6.2 Water-cement ratio and embedment length

The capacity of rock bolts depends on the water-to-cement ratio. For example, a lower amount of water used in the mix may increase the bolt capacity by 50-75%. However, if the ratio becomes too low the mix will be undesirable and impractical. The Rescon Zinc bolt cement mortar used in the mixture consists of both cement and silica. The silica has many abilities. It makes it easier to pump the grout and shortens the curing time of the grout (Are Håvard Høien, Statens vegvesen, oral source, 6. May 2014). Therefore, to find the accurate water-to-cement ratio of the mix the following calculations have to be made (Ratios from Table 8 are used in the calculations):

$$\frac{(\frac{water}{cement})}{(\frac{water}{cement + silica})} = \frac{0,49}{0,29} = 1,7 \tag{5.1}$$

For instance, if the aim is to find w/c+s ratio that corresponds to w/c ratio: 0. 49, the calculations above gives:

$$\frac{0,49}{1,7} = 0,29 \tag{5.2}$$

The same method is used to find other w/c+s in this project.

Table 8 – This table was used to find the accurate water to cement ratio of the mixture in the test (Skjølsvold, 2011).

Mix	1	2	3	4	5	6
Water per 25 kg	7,25	6,96	6,36	6,07	5,77	5,48
Water/cement+silica	0,29	0,28	0,25	0,24	0,23	0,22
Water/cement	0,49	0,47	0,43	0,41	0,39	0,37

A lot of time was spent on determine the most suitable water-to-cement ratios for the testing. Finally, three ratios were used to evaluate the influence of the water-to-cement number of the critical length; 0.40, 0.46 and 0.50. For all three ratios different embedment lengths of the bolts were tested to find out how the capacity of the rebar bolts would change with different length. Table 9 shows all the different specimens used in the pull-out test.

Table 9 – Number of bolt samples. The main focus on w/c ratios: 0.4, 0.46 and 0.5. However, two bolts tested with w/c ratio: 0.375.

Embedment length (cm)		Water-cement ratio		
	0.375	**0.40**	**0.46**	**0.50**
10	1	3	3	3
15	X	3	X	X
20	X	3	3	3
25	X	X	3	X
30	X	3	3	3
40	1	1	X	2
Total	**2**	**13**	**12**	**11**

Three cement mortar samples were also added for each of the water-to-cement numbers. The samples were then tested to determine the uniaxial compressive strength (UCS) of the cement mortar (see Table 10).

Table 10 – Number of cement mortar samples. Dimension of the cubes are: 100*100*100 mm.

w/c-ratio	0.40	0.46	0.50
Cubic samples	3	3	3

From the test results the aim was to find:

1. Critical grouted bolt length to the typical 20 mm rebar bolts
2. Relationship between the critical bolt length and w/c-number
3. Relationship between the critical bolt length and the uniaxial compressive strength of the cement mortar

6.3 Materials and equipment

Significant time was spent on the preparations of the pull-out test, which required a great variety of equipment and material. Below is a list of things necessary for the test preparation and the pull-out test itself;

- Concrete block (see figure 32)
- Drilling machine with a 46mm diameter cutter
- Frame to estimate the location of every borehole (see figure 32)
- Water (different amount for each different w/c ratio)
- Rescon Zinc bolt cement mortar(25 kilo bags)

- Hand-held drill mixer for the cement mortar
- Grout mixer with a pump to fill up the boreholes
- Rebar bolts with different embedment lengths. However, the length of the bolts from the top of the concrete block up to the end of the bolts should be the same for every bolt or 75 cm (see figure 34)
- Frame to keep the bolts from dropping into the holes(see figure 33)
- Tape to fasten the bolts to the frame (see figure 33)
- Hydraulic jack with a hand pump and cylinder
- Cylinder base. The measured length of the base and the cylinder was 60 cm. This length is called the stretch length of the bolt.
- Wedge (fastening device) attached to the bolt. Transfer tensile load from the jack to the rebar bolt
- Ropes to keep the cylinder steady while performing the test
- Extensometer used to measure the displacement
- Caliper used to measure the displacement for some bolts.
- Measuring tape and marker pen
- Computer for test results
- Safety equipment such as protective clothing, glasses, gloves, helmets, hearing protection, and protection wall.

Figure 32 - Strong concrete block, USC ca. 100 MPa. The frame is put on the block and holes are drilled into the block. The diameter of the holes are 46mm and the depth varies between 40 and 50 cm.

Figure 33 – Frame is used to keep the bolts from dropping into the holes and keep the vertical while the grout hardens.

Figure 34 - The length of the bolts from the top of the concrete block up to the end of the bolts is the same for every bolt or 75 cm. The stretch length of the bolt (the length of the base and the cylinder) is however, 60 cm.

6.4 Procedure

6.4.1 Drilling and grouting

Rebar bolts with a diameter of 20 mm were grouted vertically, with cement mortar, into a concrete block in a laboratory. First, boreholes were drilled into a strong concrete block, USC ca. 100 MPa. Approximately 50 holes with a diameter of 46 mm were drilled into the block with a depth ranging between 0.40 and 0.50 m. but there was no need for the holes to be as deep as the length of the bolts. A drilling machine with a 46 mm diameter cutter was used for the work along with a frame to estimate the location of every borehole (figure 32). The drilling was performed horizontally and afterwards all the boreholes were washed carefully to keep all the cuttings away. Then the block was turned with the holes facing the ceiling.

With all the holes ready, the next step was preparing the bolts for grouting. The estimated embedment lengths of the rebar bolts are shown in table 9. However, despite the different embedment lengths, the same bolt length was measured from the top of the concrete block up to the end of the bolts for every bolt. The measured length was 75 cm. 60 cm of that was covered with the cylinder and the base. That length is the stretch length of the bolt. The bolts were cut as Figure 35 shows and then prepared for injection.

Figure 35 – The bolts were first cut and then grouted afterwards. The total length of the bolts was 75 centimeter plus the different embedment length for every bolt.

Water-to-cement ratios of the grout used for the testing are shown in Table 10. A hand-held drill mixer was used while mixing the Rescon Zinc bolt cement mortar with the water. When the mixture was fully mixed, it was poured into a grout mixer that contained a pump. Then the prepared grout mortar was pumped into the holes and the bolts inserted to the center of the drilling holes with different embedment lengths. It is important to ensure that the grout mortar is perfectly compacted in the hole to prevent inaccurate results. A frame was used to keep the bolts from dropping into the holes (Figure 33). Estimated curing time for the grout was seven days and afterwards the plan was to pull the bolts up under axial loading.

6.4.2 Pull-out test

After the grouting's curing time, the pull-out test was executed. A hydraulic hand pump with cylinder was used for the test (Appendix C). The cylinder and base were placed on the rebar bolt along with a wedge (fastening device) to ensure that the applied load is axial with the load. The wedge transfers tensile load from the jack to the rebar. Robes were then used to keep the cylinder steady while the test was carried out. Finally, an extensometer was arranged above the rebar to measure the displacement.

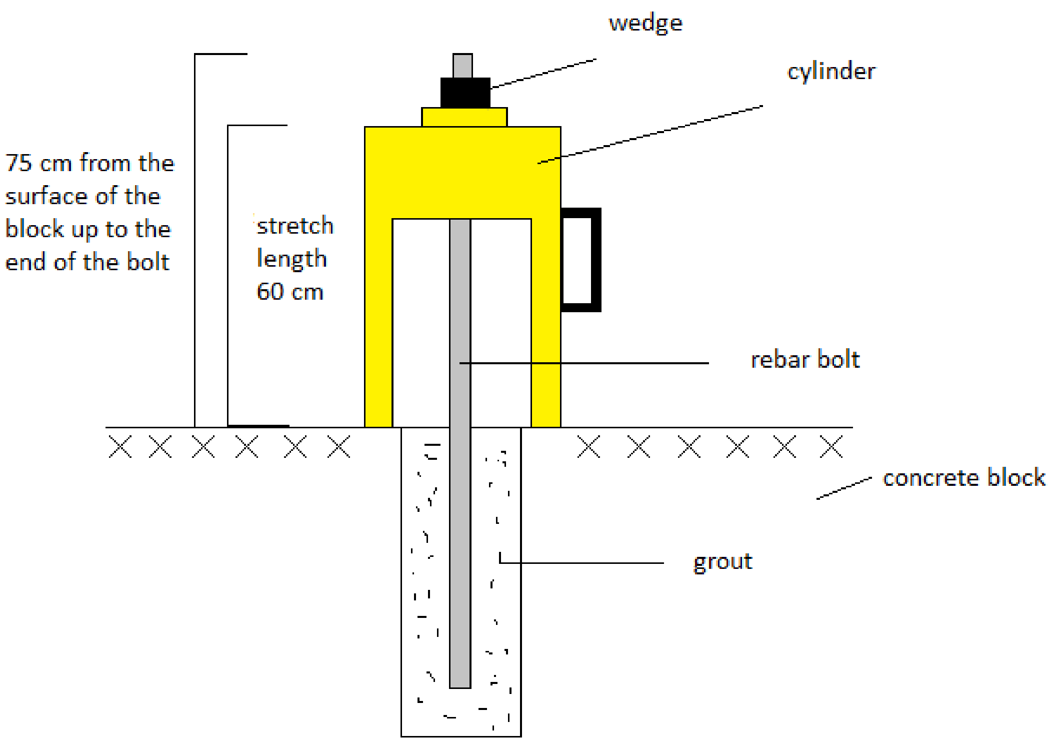

Figure 36 – The pull-out test setup. Rebar bolt grouted in concrete block and hydraulic jack used to pull out the bolt. The tested stretch length is 60 cm.

When the entire setup was completed, the rebar bolts were loaded hydraulically and at the same time the displacement of the bolts head was measured. The load gradually increased until the bolts slid from the grouting or they started to yield or break. The pull-out test was repeated for various grout types and bolts with different embedment length. All load and displacement results from the testing were collected graphically in a computer. Pull-out length was estimated, changes in bolt diameter were measured in bolts that yielded, and the strain of the yielded bolts was calculated. While performing the pull-out tests, all necessary safety equipment was used.

Figure 37 – Wedge used as a fastening device.

6.4.3 UCS test

UCS tests were performed on the cement mortar with different ratios to find the uniaxial compressive strength. 10x10 cm cubic boxes were filled up with cement mortar and tested seven days later. The samples were put in a triaxial compressive machine and loaded axially to failure, with no confinement. In this uniaxial compressive strength test the σ_3=0.

Figure 38 – 10x10 cm cubic boxes filled up with cement mortar and seven days later tested to find the uniaxial compressive strength of the grout.

7 Chapter – Results

In this chapter, results from pull-out testing on rebar bolts with diameter of 20 mm, grouted with different water-to-cement ratios and variations in embedment length, will be presented both graphically and in writing. First the results from the pull-out testing will be introduced followed by the results from the uniaxial compressive strength test (UCS test) performed on cement cubes with different water-to-cement ratios.

The pull-out test was performed vertically, which led to number of problems; the test preparation changed throughout the whole testing process to accommodate these issues. Some displacement measurements were performed manually for safety reasons, in case the bolts break, but using extensometer to measure the displacement of the bolt is much quicker and more accurate way.

7.1 Pull-out tests on rebar

38 rebar bolts were grouted and then pulled out afterwards in a laboratory. The estimated grouting time was seven days, but for some bolts the grouting time went up to eight days; there were two bolts that were pulled out after eleven days (All different grouting time is showed in Appendix D). This may have affected the results. Every rebar bolt was identified with a specific number, subject to the embedment length, water-to-cement ratio and the sample number. In Table 11 the identification system is explained. In Appendix E are figures of all the pull out tested rebar bolts and the measured pull out length is shown.

Table 11 – Identification system for tested rebar. For instance, if rebar bolt is grouted with w/c ratio: 0.40m, the embedment length is 30 cm and this is the second sample, the identification number of the bolt would be B232.

B=Bolt	w/c ratio	w/c number	Embedment length (cm)	Embedment number	ID (three types of every sample
B	0.59	0	10	1	1,2,3
B	0.375	1	20	2	1,2,3
B	0.40	2	30	3	1,2,3
B	0.46	3	40	4	1,2,3
B	0.50	4	14	5	1,2,3
B			25	6	1,2,3

Results from these 38 rebar bolts are presented below.

7.1.1 Water to cement ratio: 0.40

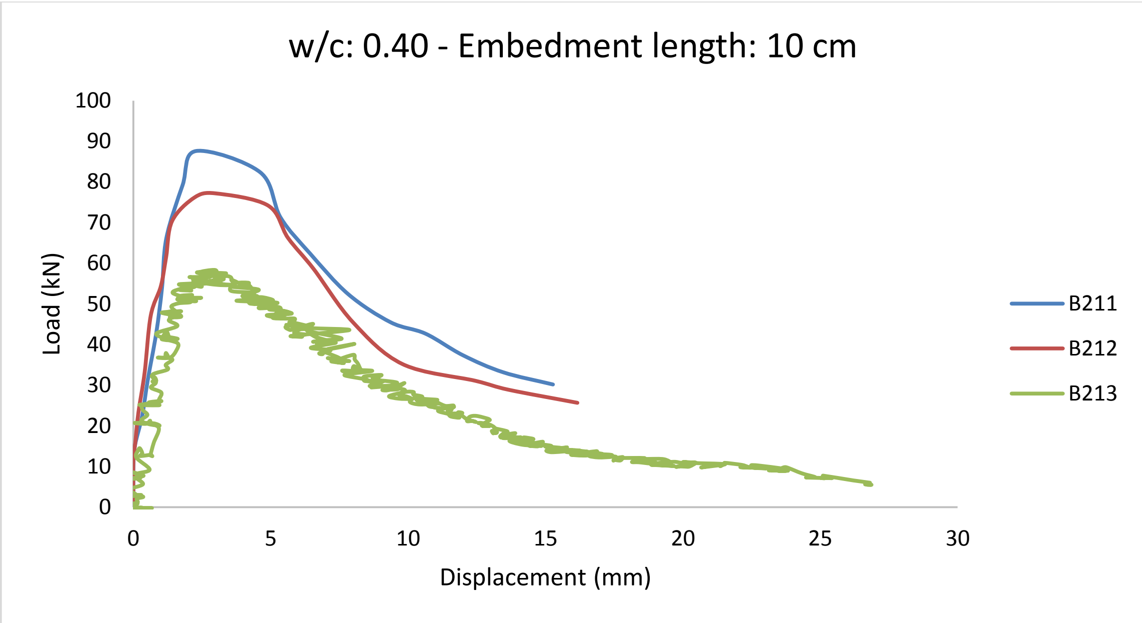

Figure 39 - The maximum load is 87.6 kN for B211 before the bolt starts to slide from the grout. The bolt had been grouted 8 days earlier like the other two. The maximum load for B12 is lower or 77.2 kN. The grout got cracked and the bolt began to slide. B213 was measured using extensometer. The maximum load was around 60 kN before sliding like the other two. The embedment length is too short and therefore the bolts do not tolerate high load.

Embedment length: 10 cm

Three rebar bolts were grouted with w/c ratio: 0.40 and 10 cm in embedment length. Figure 39 shows the results with load versus displacement curves. The displacement of rebar bolt number B213 was measured using an extensometer but the displacement for the other two, B211 and B212, were measured manually. The maximum load for B213 is lower than the maximum load for the other two but all the bolts slid from the grout. The entire bolt number B213 slid from the grout. B211 had sunk little further than the other two so the embedment length was 2-3 cm longer. Hence, the maximum load was higher for B211. Grouting the bolts with 10 cm embedment length is too short; the rebar bolts did not tolerate much load and therefore they began to slide from the cement mortar. The grouting time was eight days.

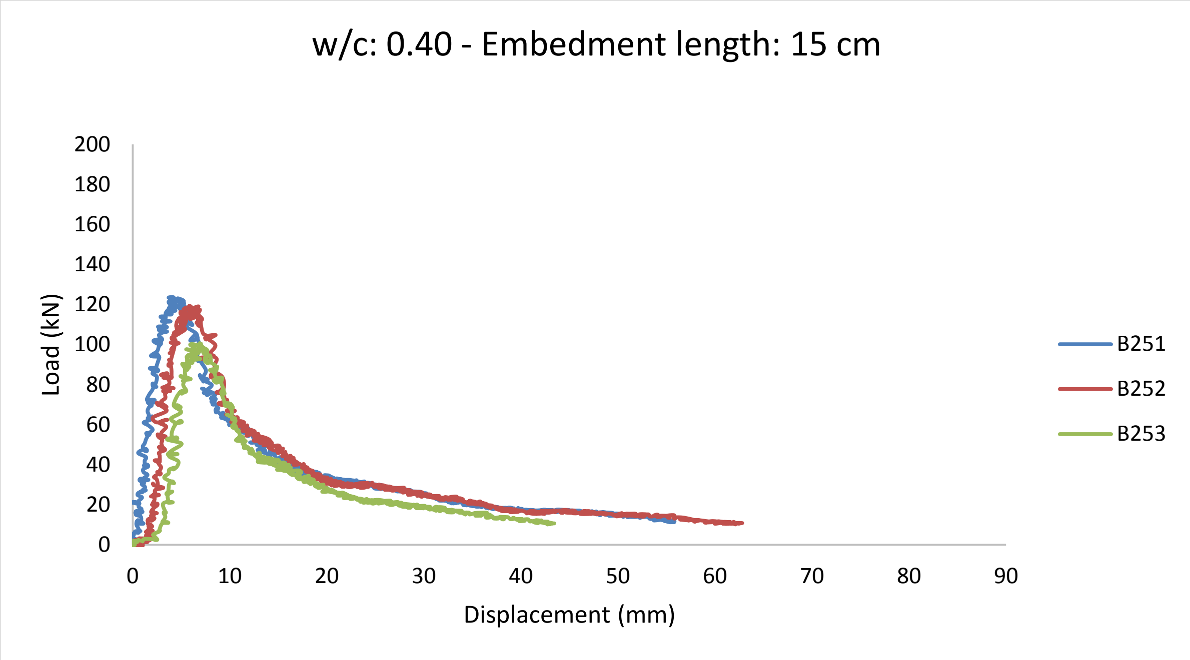

Figure 40 - Maximum load for B251 around 120 kN. Rebar starts to yield around 170 kN so the bolt just slid from the cement mix when the maximum load had been reached. Maximum load for B252 is around 120kN. The bolt starts to slide at the displacement of 10 cm. The final bolt reaches 100 kN. The grout did not fill up the hole completely which resulted with lower load.

Embedment length: 15 cm

Three rebar bolts were grouted with w/c ratio: 0.40 and 15 cm in embedment length. Figure 40 shows the results with load versus displacement curves. The results were similar for all three bolts. The maximum load for B251 and B253 was between 115-120 kN but B253 was little lower or around 100 kN. All bolts slid from the grout. The pull-out length for B251 was measured at 6 cm (figure 41). The grout for B253 did not fill up the hole completely and the cohesion between the grout and the concrete block was not strong. Therefore, the maximum load was lower. The grouting time was seven days.

Figure 41 – The black mark on the bolt is to measure the pull-out length. The length is 6 cm. The bolt slid from the grout with maximum load around 120 kN.

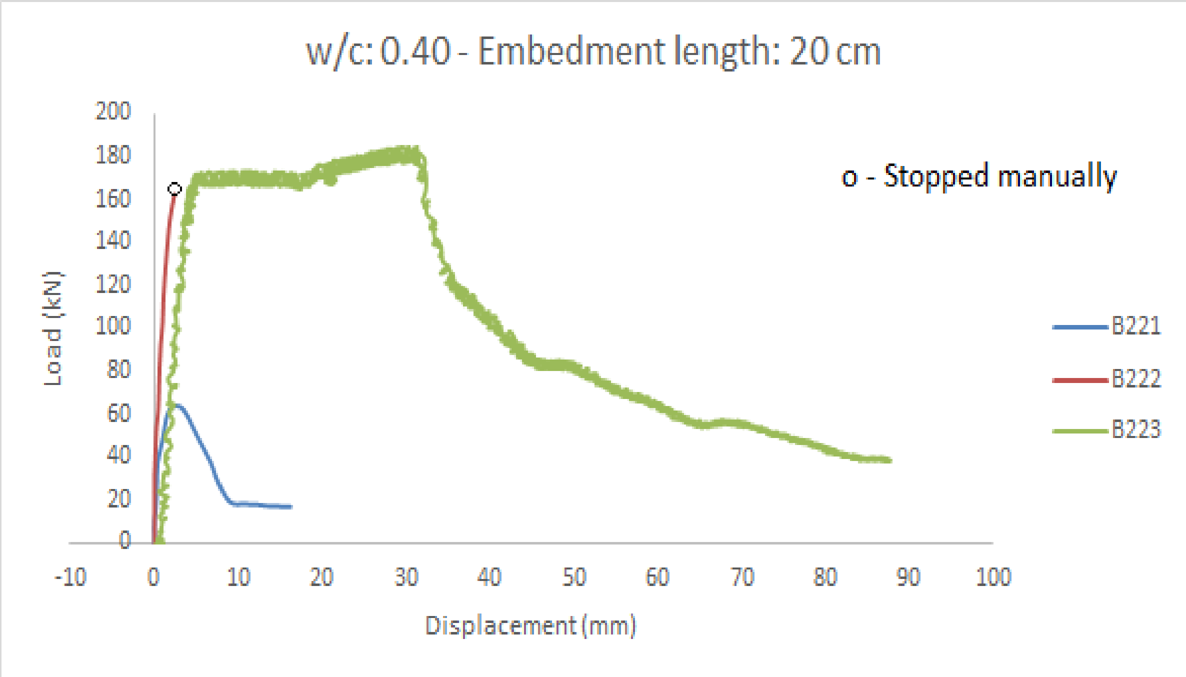

Figure 42 - The maximum load reached only 64 kN for B221. The reason is bad grouting but it did not fill up the hole like it should. Therefore, the maximum load was so low. Maximum load for the other two bolts, went over 180 kN and as the test results shows the bolts reached the yielding point (around 170-175 kN). B223 yielded up to 30 cm in displacement before the bolt slid from the grout and the load started to decrease. All three bolts slid from the grout.

Embedment length: 20 cm

Three rebar bolts were grouted with w/c ratio: 0.40 and 20 cm in embedment length. Figure 42 above shows the results graphically. The results for B221 are an example of bad grouting. The grout did not fill up the hole like it should. The form of the grouting was like squeezing tooth paste; in this case, it is important to stir the grout completely, which can be a rather difficult task. The maximum load was around 60 kN and the displacement, measured manually just like B222, was minor. Hence, B222 and B223 were grouted better than B221, the results were more accurate. The maximum load of both bolts went over 180 kN and as the test results show, the bolts reached the yielding point (around 170-175 kN). B223 yielded up to 30 cm in displacement before the bolt slid from the grout and the load started to decrease. Before and after the testing, the length of B223 was measured. The bolt had stretched 2.5 cm after the test and the pull-out length was 6 cm. The grouting time was 9 days.

Figure 43 – Pull-out length is 6 cm. This bolt yielded before it slid from the grout. The figure shows how the surface of the grout breaks around the bolt.

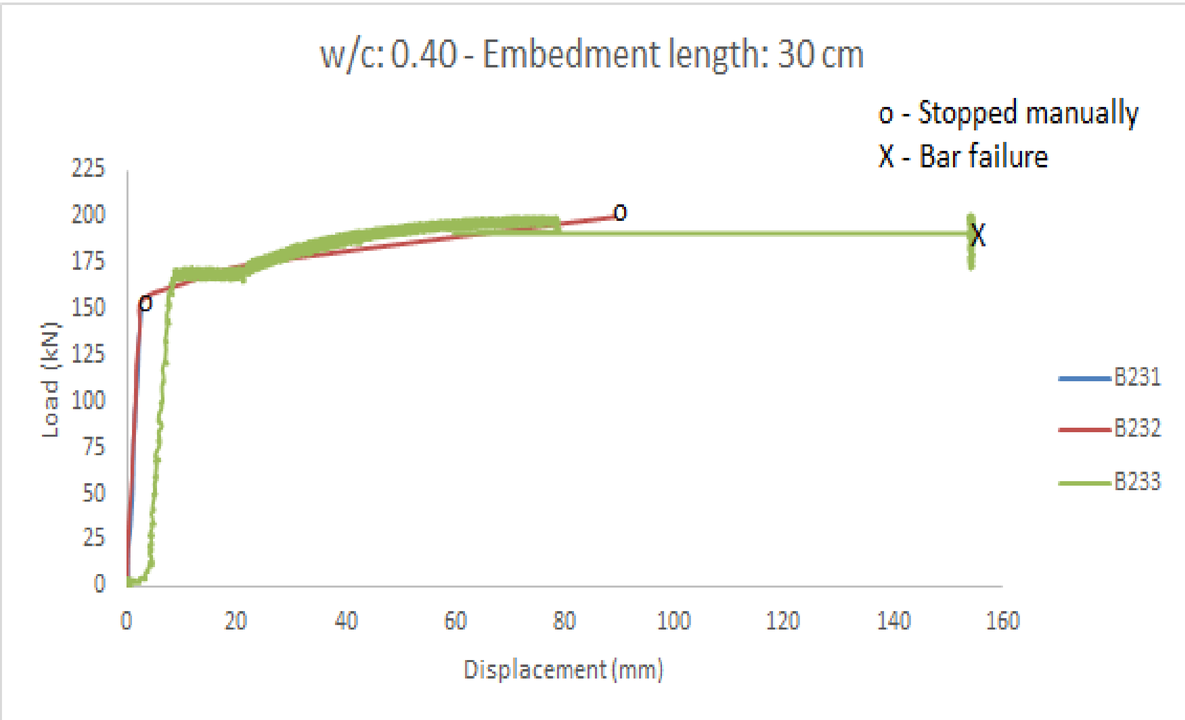

Figure 44 – The load was increased steadily and all three bolts reached the yielding point, 170 kN. The displacement for B231 was low because the test was stopped right away as the bolt had reached the maximum load of 200 kN. The displacement of B232 is longer and the bolt yielded much more than B231. It was going to break when the test was stopped cause of safety reasons. The third rebar, B233, however, broke. The maximum load was 200 kN. The stretch length of the bolt yielded a lot until the bolt broke. The Figure shows how the bolt is necking; the load drops quickly and the bolt breaks.

Embedment length: 30 cm

Three rebar bolts were grouted with w/c ratio: 0.40 and 30 cm in embedment length. Results are presented in figure 44. The load was increased steadily and all three bolts reached the yielding point, 170 kN. Both B231 and B232 were measured manually, because of safety concerns. The displacement for B231 was low because the test was stopped right away as the bolt had reached the maximum load of 200 kN. However, when B323 reached the maximum load of 200 kN, the pumping was kept going to test the deformation of the bolt. The tension in B323 was high and the diameter of the bolt became smaller. The measured diameter after the test was 1 mm smaller than before. The bolt was going to break when it was decided to stop the testing for safety reasons. The third rebar, B233, however, broke. The maximum load was 200 kN. The length of the bolt stretched a lot until the bolt broke. The curve shows how the bolt is necking; the load drops quickly and the bolt breaks. The diameter of the B233 was measured close to the breaking point, when it had reduced by 1 mm. The grouting time was nine days.

Figure 45 – The embedment length was longer then the critical length, so the bolt broke. The reduction in diameter closest to the breakage increased until the bolt broke. The maximum load reached 200 kN.

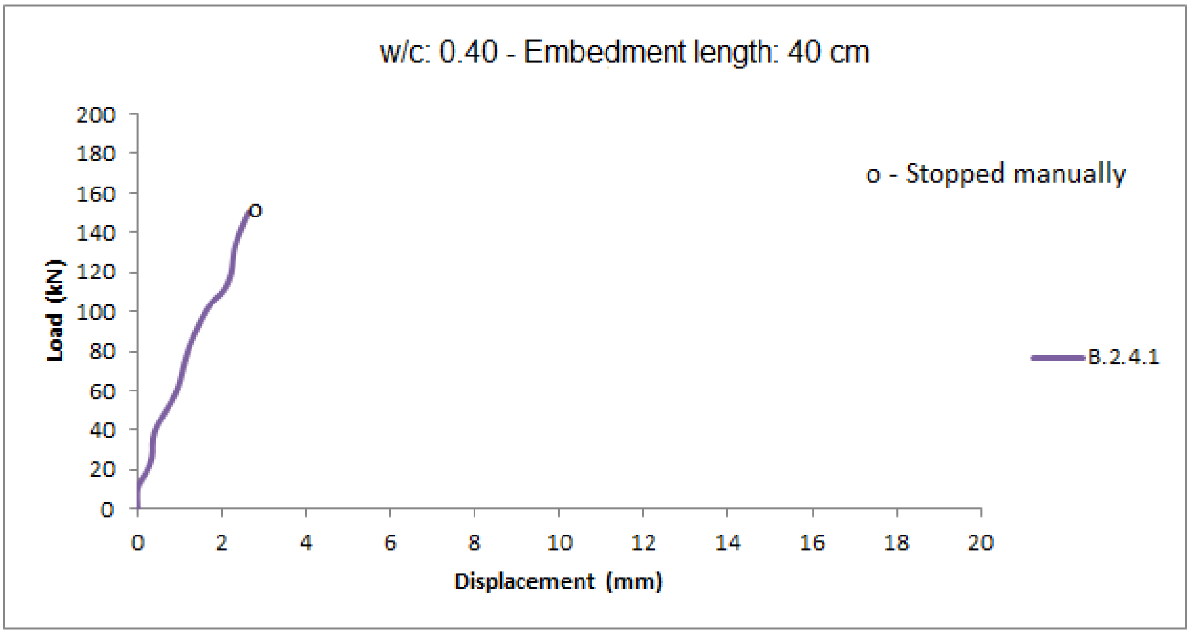

Figure 46 – The displacement of the bolt was measured manually and the load was increased until the load was 183 kN. The bolt had yielded a little bit but before the bolt broke the test was stopped for safety reasons. The stretch length of the bolt had increased much.

Embedment length: 40 cm

Only one rebar bolt, B241, was grouted with w/c ratio: 0.40 and 40 cm in embedment length. The reason was because the bolt with 30 cm embedment length broke, so the embedment length for that bolt was longer than the critical embedment length. Figure 46 shows the result for B241. The displacement of the bolt was measured manually and the load was increased until the load was 183 kN. The bolt had yielded a little bit but before the bolt broke the test was stopped for safety reasons. The stretch length of the bolt had increased significantly. The cohesion between the bolt and the cement was strong and the pull-out length was around 1 cm. Grouting time of the cement mortar was nine days.

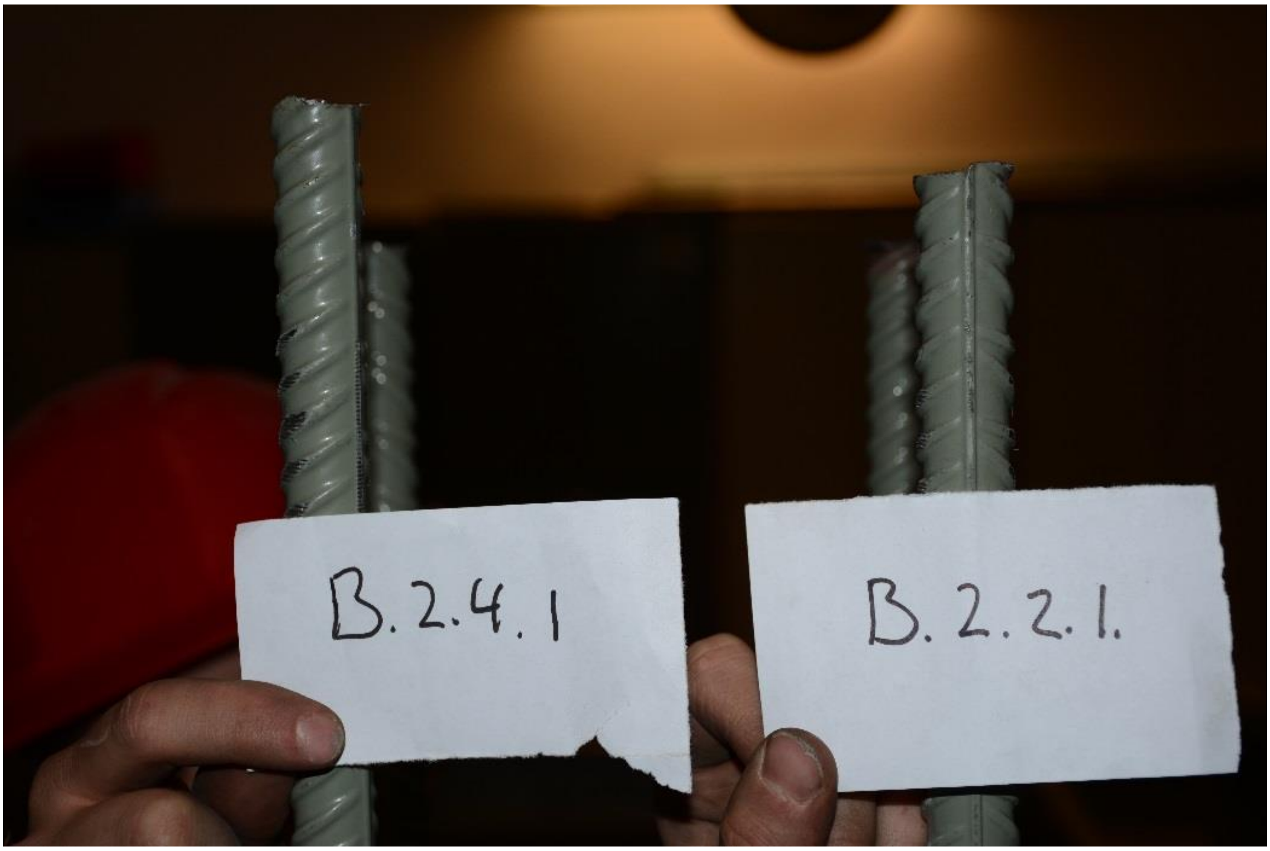

Figure 47 - The figure shows two bolts that were the same length before the test. However, the stretch length for B241 increased significantly while B221 slid from the grout with maximum load around 60 kN.

7.1.2 Maximum load versus different embedment length for w/c ratio: 0.40

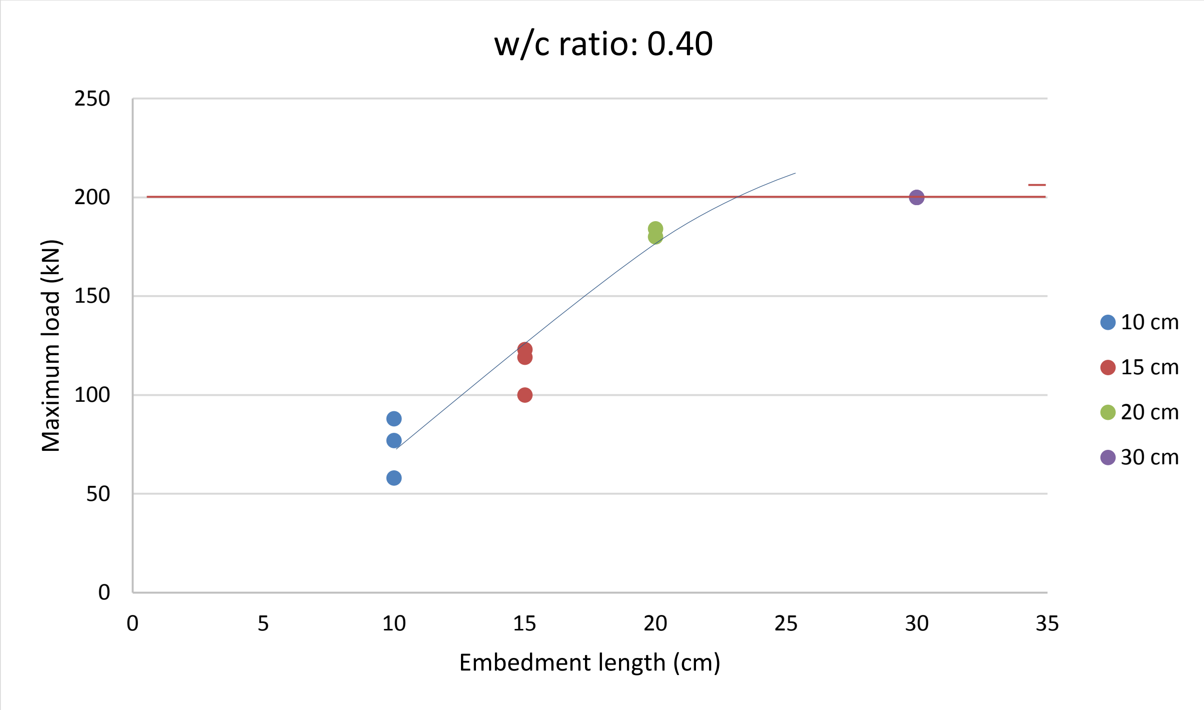

Figure 48 – The maximum load increases with greater embedment length. The bolts with the first three lengths, 10 cm, 15 cm and 20, all slid from the grout. The bolts with 20 cm length were however, close to the critical length. The maximum load reached approximately 180 kN and the bolts began to yield until they slid from the cement. 30 cm embedment length is, however, longer than the critical embedment length. The bolts with 30 cm length reached 200 kN which resulted in breakage of the bolts.

The critical length is defined as the greatest grouted length of the bolt wherein the bolt is pulled out of the grouting without the failure of the rod. Figure 48 shows the results for bolts grouted with different embedment length with w/c ratio: 0.40. The maximum load increases with greater embedment length. The bolts with the first three lengths, 10 cm, 15 cm and 20, all slid from the grout. The bolts with 20 cm length were close to the critical length. The maximum load reached approximately 180 kN and the bolts began to yield which led to increasing of the stretch length. Then the bolts slid from the cement. 30 cm embedment length is, however, longer than the critical embedment length. The bolts with 30 cm length reached 200 kN which resulted in breakage of the bolts – the rebar used in this test is only capable of bearing 200 kN before it breaks.

7.1.3 Water-to-cement ratio: 0.46

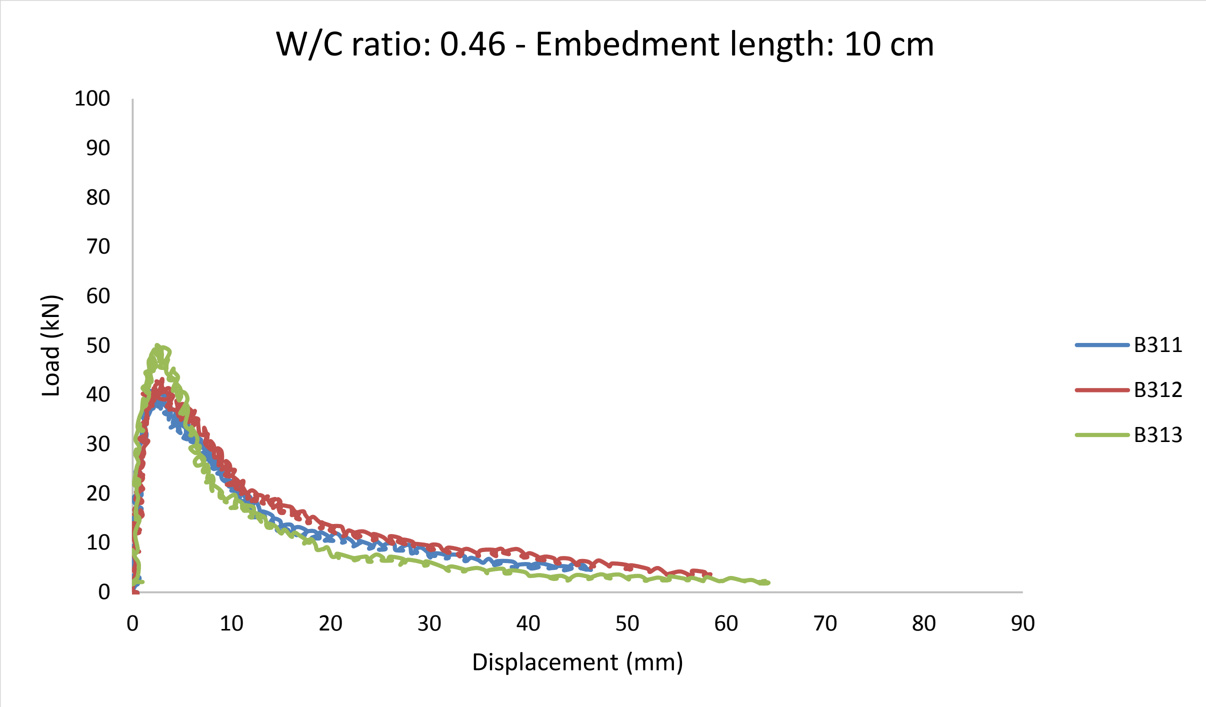

Figure 49 –**Similar results for all three bolts. The maximum load reached approxamately 50 kN before the bolts slid from the grout. The embedment length was too short so the bolts did not bear much load before sliding.**

Embedment length: 10 cm

Three rebar bolts were grouted with w/c ratio: 0.46 and 10 cm in embedment length. Figure 49 shows the results with load versus displacement curves. The grouting time for all bolts, B311, B312 and B313, was eight days. The maximum load was similar for all three rebars, between 40 and 50 kN. The embedment length was too short so the bolts did not bear much load before they all slid from the grout. The measured pull-out length for the bolts was between 4 and 6 cm.

Figure 50 –The cohesion between the grout and the concrete block was weak and when the bolt was pulled out the grout was stuck on the bolt.

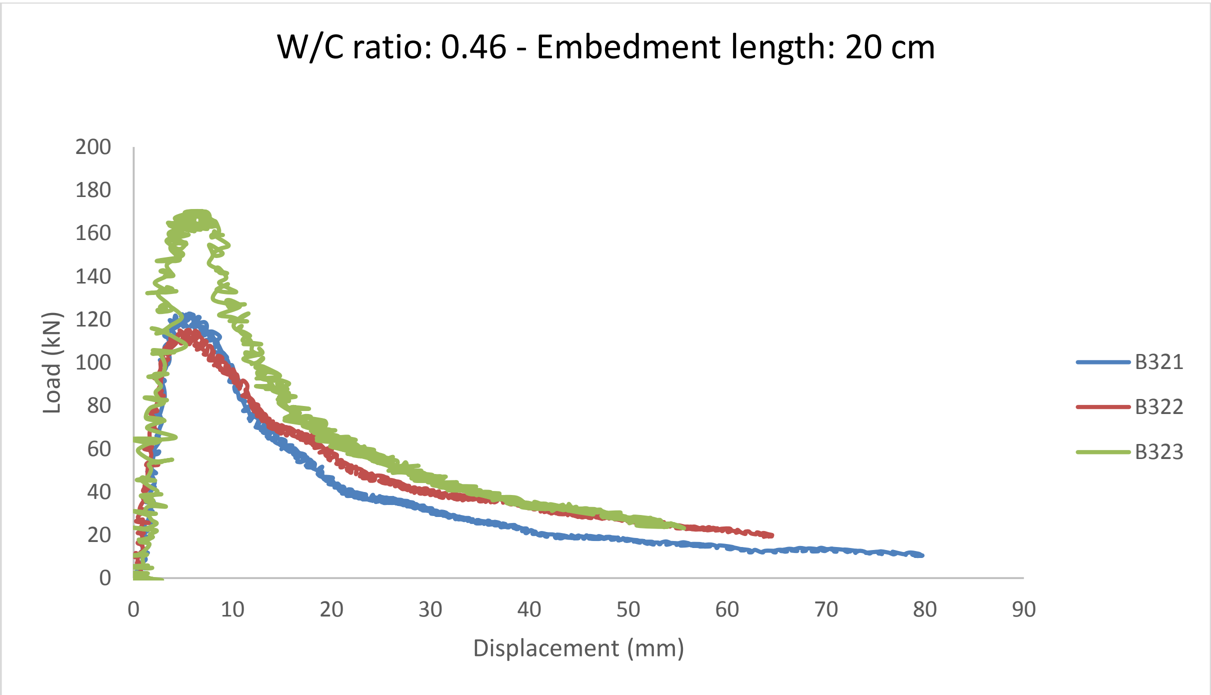

Figure 51 –Similar results for B321 and B322. The maximum load were approximately 120kN for both bolts and they slid from the cement at the displacement of 5-8 cm. The results from B323 were different. The maximum load was much higher, up to 170kN. It is close to the yielding point for the rebars, which is 170 – 175 kN. The bolt had sunk down in the borehole so the embedment length was closer to 25 cm. Therefore, the load was much higher than for the other bolts.

Embedment length: 20 cm

Three rebar bolts were grouted with w/c ratio: 0.46 and 20 cm in embedment length. Figure 51 shows the results with load versus displacement curves. B321 and B322 have similar results. The maximum load was approximately 120kN for both bolts and they started to slide with displacement around 5-8 cm. The grouting time for both bolts was the same, seven days. However, the results from B323 were different. The maximum load was much higher, up to 170kN, which is high for bolts grouted with 20 cm embedment length. It is close to the yielding point for the rebars, which is 170 – 175 kN. The reason for this result was because the bolt had sunk down in the borehole so the embedment length was closer to 25 cm. Therefore, the results for B321 and B322 are more accurate. The grouting time for B323 was one day shorter than for the other two. The pull out length of the bolts was between 6 and 7 cm.

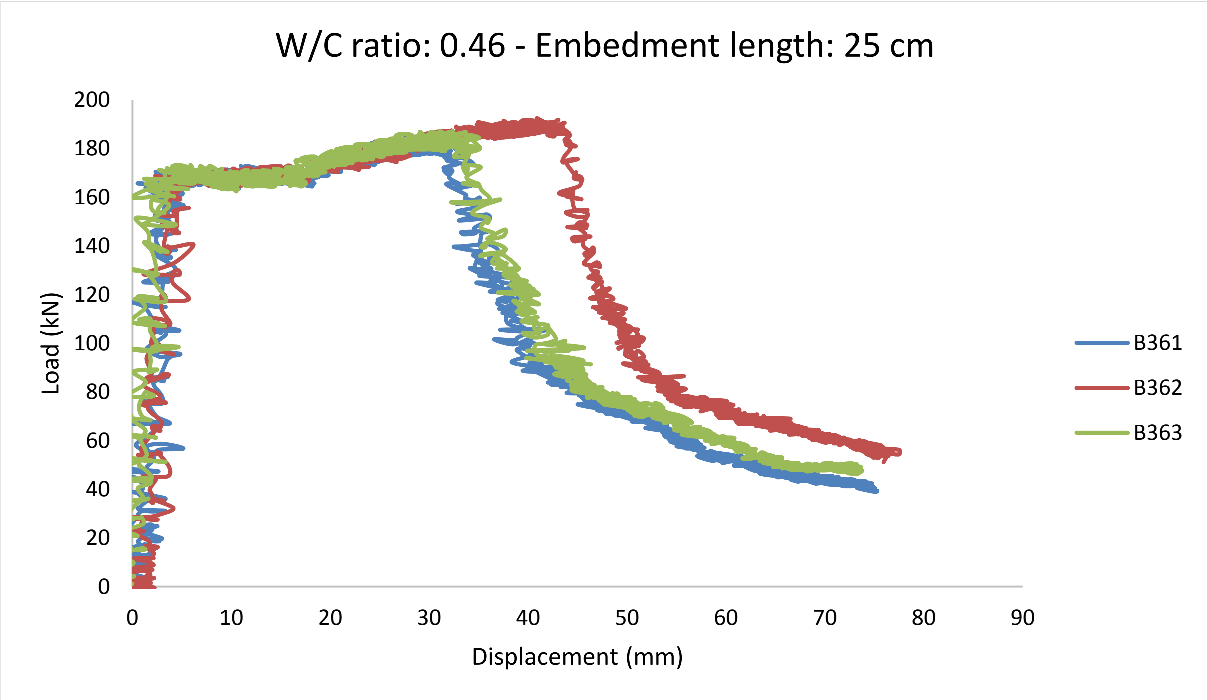

Figure 52 – Similar results for all three bolts. They all reached the yielding point in the testing. The maximum load went up to 192 kN for B362 and like the other two it began to slide afterwards. The displacement when the bolts slid was between 30 and 50 mm. The stretch length stretched when the bolts started to yield.

Embedment length: 25 cm

Three rebar bolts were grouted with w/c ratio: 0.46 and 25 cm in embedment length. Figure 52 shows the results with load versus displacement graph. All three bolts, B361, B362 and B363 yielded in the testing. The maximum load went up to 192 kN for B362 and like the other two, it began to slide afterwards. The displacement when the bolt slid was between 30 and

50 mm. The length stretched when the bolts started to yield; B361 increased by 2 cm and the pull-out length was 5 cm. For B362 the pull out length was 4.7 cm and the stretch length afterwards had increased by 3.3 cm. The pull-out length for the third bolt, B363, was 5.5 cm and the stretch length had elongated by 2.2 cm. Grouting time for all three rebar bolts was seven days.

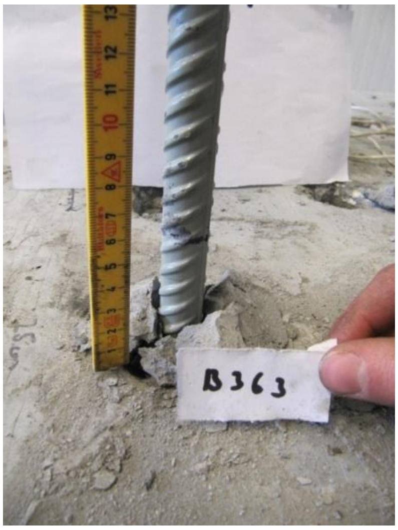

Figure 53 – Pull out length is 5.5 cm and the grout has broken in the surface. This bolt yielded before it slid from the grout.

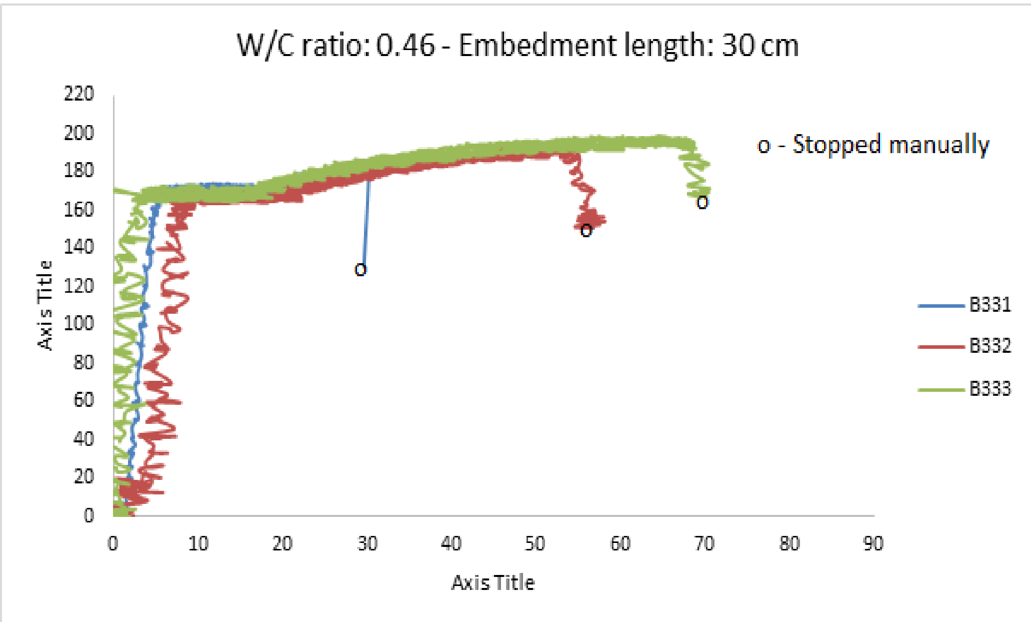

Figure 54 – B331 started to yield when the load reached 170 kN. The bolt had yielded up to 180 kN with displacement of 30 mm when it was decided to stop the test for safety reasons. The maximum load for B332 was higher, almost 200 kN. The bolt started to yield when the load reached 170 kN and from that up to approximately 200 kN the stretch length of the bolt increased. The bolt slid when the displacement reached 70 mm. The same result was obtained from B333. However, the displacement went up to 70 mm before the bolt started to slide.

Embedment length: 30 cm

Three rebar bolts were grouted with w/c ratio: 0.46 and 30 cm in embedment length. Figure 54 shows the results with load versus displacement curves. The grouting time for the bolts was eight days. The first bolt, B331, started to yield when the load reached 170 kN. When the load reached 180 kN with displacement of 30 mm, the pull-out test was stopped for safety reasons. The maximum load for B332 was higher, almost 200 kN with zero pull out length. The bolt started to yield when the load reached 170 kN and from that up to approximately 200 kN the stretch length of the bolt increased. The bolt started to slide when the displacement reached 60 mm. The same result was obtained from the third bolt, B333. However, the displacement went up to 70 mm before the bolt started to slide. The rebar bolts were very close to the critical embedment length which would end with the breakage of the bolts. The diameter of B332 was measured before and after the testing. The measurement showed that the diameter had decreased by 1 mm. Hence, the stretch length had increased by 5 cm.

Figure 55 – The bolt was barely pulled from the grout, but was at the verge of breakage.

7.1.4 Maximum load versus different embedment length for w/c ratio: 0.46

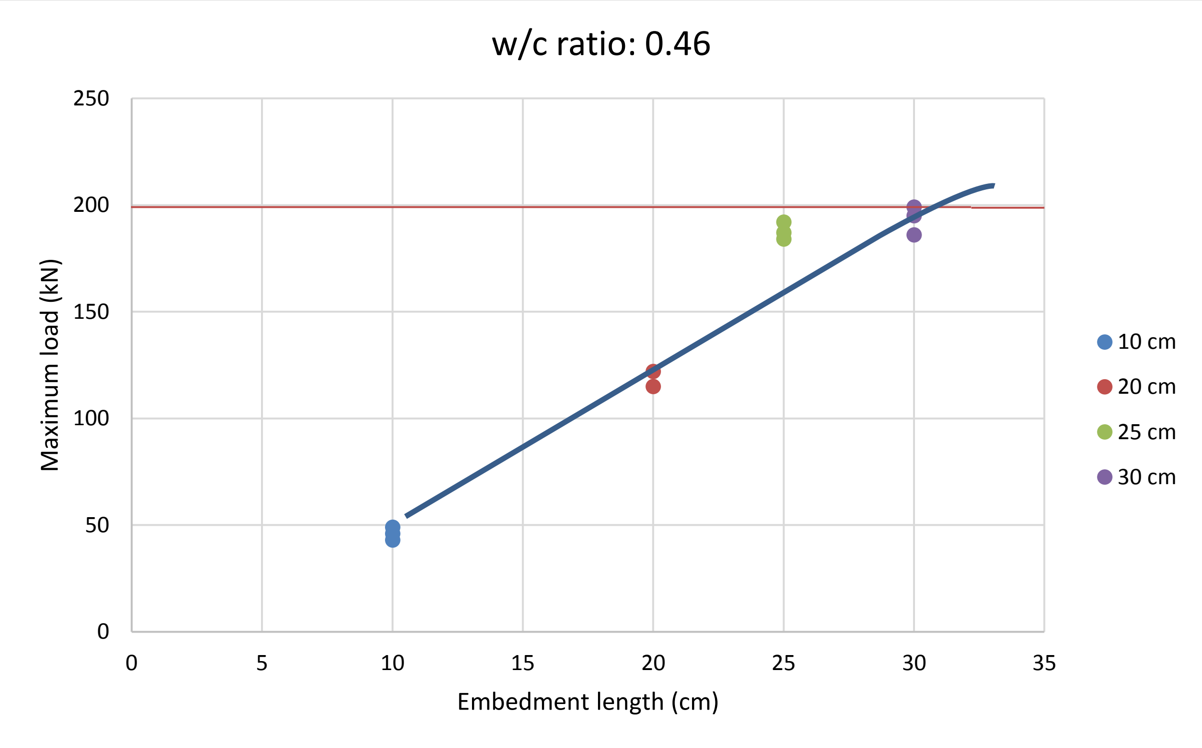

Figure 56 - The first three lengths are under the maximum load at which the rebar bolts will break. The fourth embedment length, 30 cm, was on the verge of being the critical length. The maximum load reached 200 kN but the curve dropped before the bolts broke at the displacement of 50 – 70 cm. The bolts slid from the grout. Therefore, there was no bolt grouted with a water-to-cement ratio of 0.46 that had longer embedment length than the critical length. Embedment length between 30 and 35 cm is a recommended length to see if the bolt will break.

Rebar bolts with four different embedment lengths were grouted with w/c ratio: 0.46. Figure 56 shows the maximum load of the bolts versus different embedment lengths to find the critical length, wherein the bolt with the greatest length is pulled out without failure. The first three lengths are all under the maximum load wherein the rebar bolts will break. The fourth embedment length, 30 cm, was on the verge of being the critical length. The maximum load reached 200 kN but the curve dropped before the bolts broke at the displacement of 50 – 70 cm. and the bolts slid from the grout. Therefore, there was no bolt grouted with a water-to-cement ratio of 0.46 that had a longer embedment length than the critical length. Embedment length between 30 and 35 cm is a recommended length to see if the bolt will break.

7.1.5 Water to cement ratio: 0.50

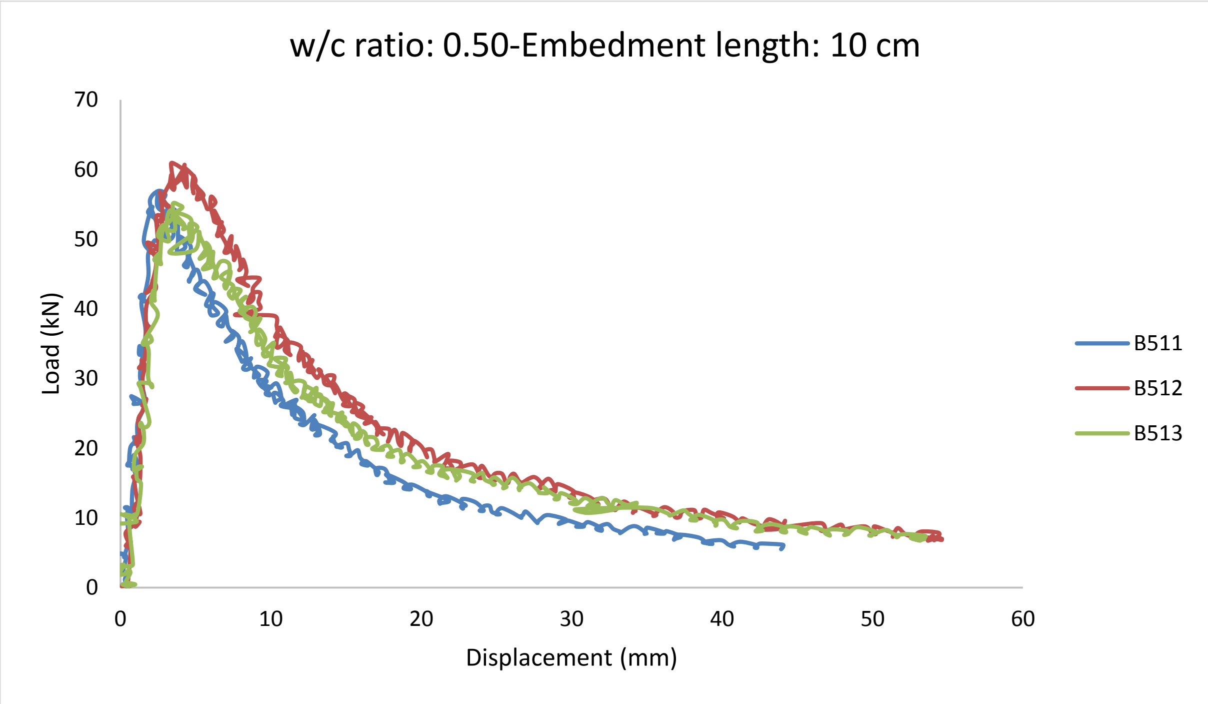

Figure 57 – Similar results for all three rebar bolts. The maximum load is between 50 and 60 kN and the displacement is 5-10 mm when the bolts slid from the grouting. The bolts do not tolerate high load wherain the embedment length is too short and the water to cement ratio is rather high.

Embedment length: 10 cm

Three rebar bolts were grouted with w/c ratio: 0.50 and 10 cm in embedment length. Figure 57 presents the results. There were similar results for all three rebar bolts. The maximum load was between 50kN and 60kN before the bolts slid from the grout. The pull-out length is also similar for all bolts or around 5 cm. The embedment length is too short and therefore the bolts do not tolerate high load. Also the water-to-cement ratio is rather high and the grouting becomes weaker at the higher ratio. The grouting time was seven days.

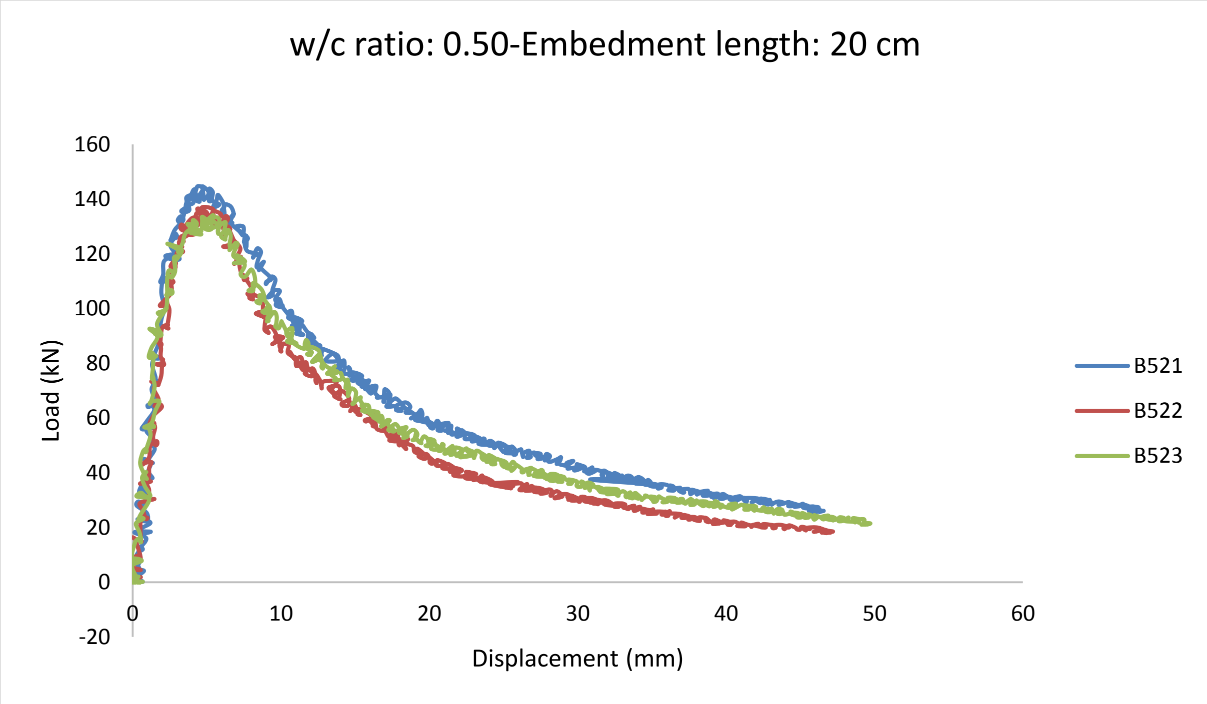

Figure 58 – Similar results for all three bolts. The maximum load was between 130 and 145 kN. All three bolts slid from the grouting with displacement up to approximately 10 mm.

Embedment length: 20 cm

Three rebar bolts were grouted with w/c ratio: 0.50 and 20 cm in embedment length. Figure 58 shows the results with load versus displacement curves. There were similar results for all three bolts. The maximum load was between 130 and 145 kN and little difference between the pull-out length of the bolts. B211 had a pull-out length of 3.8 cm; the other two, B212 and B213, had pull-out lengths of just over 4 cm. All three bolts slid from the grouting. The grouting time was seven days for all three bolts.

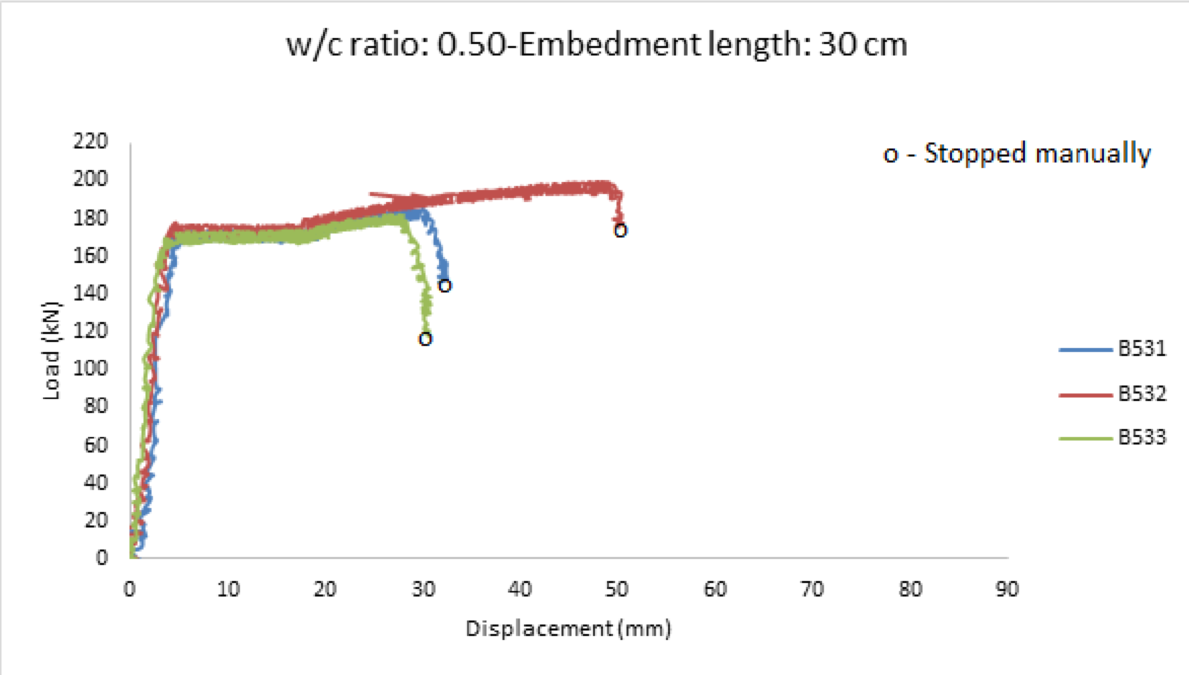

Figure 59 - All bolts yielded when the load reached 170 kN. From that up to the maximum loads, with 30-50 mm in displacement, the stretch length of the bolts increased until they slid. The maximum load of B532 was 199 kN before it slid from the cement mortar. The rebar bolts were close to the critical embedment length. However, the bolts did not break and if the pull out test had been carried on the curves would have shown the load decreasing gradually, not dropping quickly as happens if bolts break.

Embedment length: 30 cm

Three rebar bolts were grouted with w/c ratio: 0.50 and 30 cm in embedment length. Figure 59 shows the results with load versus displacement curves. The grouting time for all three bolts was seven days. All bolts, B531, B532 and B533, began to yield when the load reached 170 kN. From that up to the maximum loads, with 30-50 mm in displacement, the stretch length of the bolts increased until they started to slide. The maximum load of B532 went up to 199 kN before it slid from the cement mortar. However, the pull-out length was zero. When the bolts yielded the diameter decreased for all three bolts. The diameter of B531 was, before testing, 21.21 mm and 20.56 mm afterwards. The stretch length had also gone from 73 cm up to 74.2 cm. The stretch length for B532 had increased by 3.8, with the diameter changing by 0.71mm. The diameter of B533 decreased by 0.76 mm, and the stretch length elongated by 2 cm. The rebar bolts were close to the critical embedment length, which would end with the breakage of the bolts. However, the bolts did not break and if the pull out test had been carried on, instead of stopping as soon as the load began to drop, the curves would have shown the load decreasing gradually, not dropping quickly as happens if bolt breaks.

Figure 60 - the pull-out length was zero. Reduction in diameter of B533 was 0.76 mm and the stretch length elongated by 2 cm.

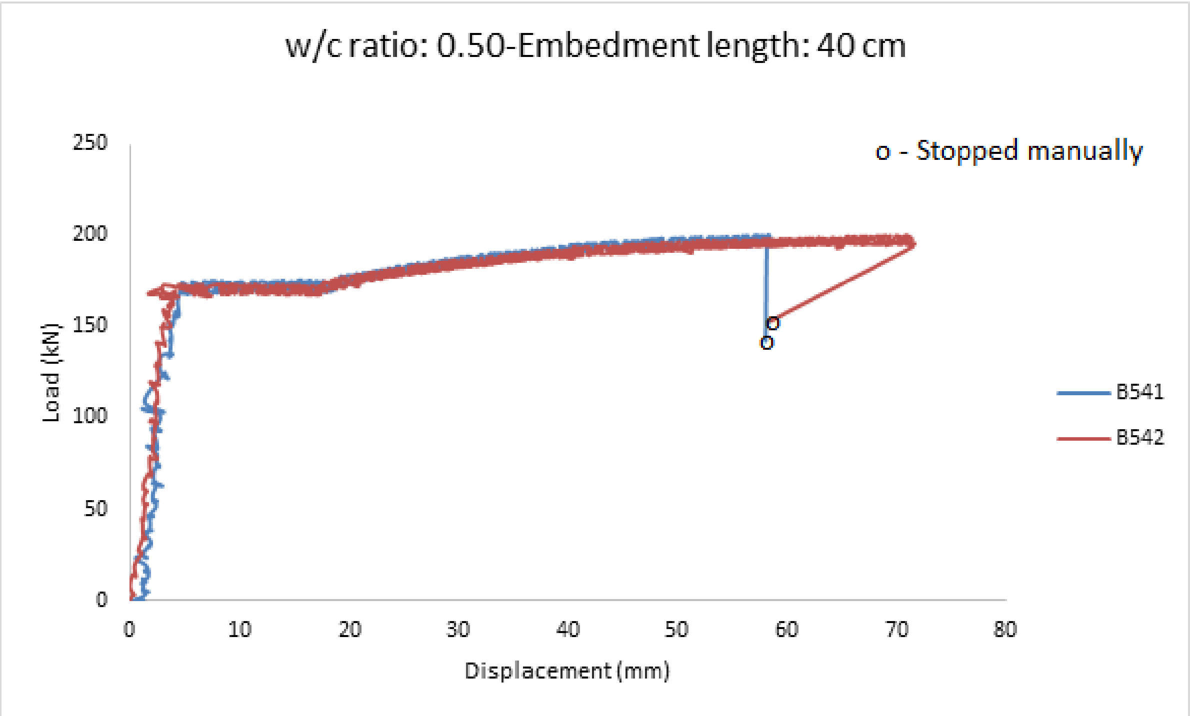

Figure 61 –Both bolts, B541 and B542, started to yield around 170 kN and the maximum load for both bolts went up to 200 kN. B541 was going to break but to keep the extensometer safe the load was decreased and the pull out test stopped. B542 was also going to break but the test was carried out until the displacement had reached 70 mm and then it was stopped.

Embedment length: 40 cm

Two rebar bolts were grouted with w/c ratio: 0.50 and 40 cm in embedment length. Figure 61 shows the results with load versus displacement curves. Both bolts, B541 and B542, started to yield around 170 kN. The maximum load for both bolts went up to 200 kN. For B541 the stretch length increased by 4.2 cm and the pull out length was zero. The diameter of the bolt decreased by 0.26 mm. The pull-out test was stopped when displacement reached up to 60 mm. The bolt was going to break but to keep the extensometer safe the load was decreased and the pull-out test stopped. The diameter of B542 had decreased from 21.20 mm to 19.69 mm after the pull-out test. The stretch length had increased by 6 cm before the test was stopped. The bolt was going to break but the test was carried out until the displacement had reached 70 mm. The grouting time was seven days.

7.1.6 Maximum load versus different embedment length for w/c ratio: 0.50

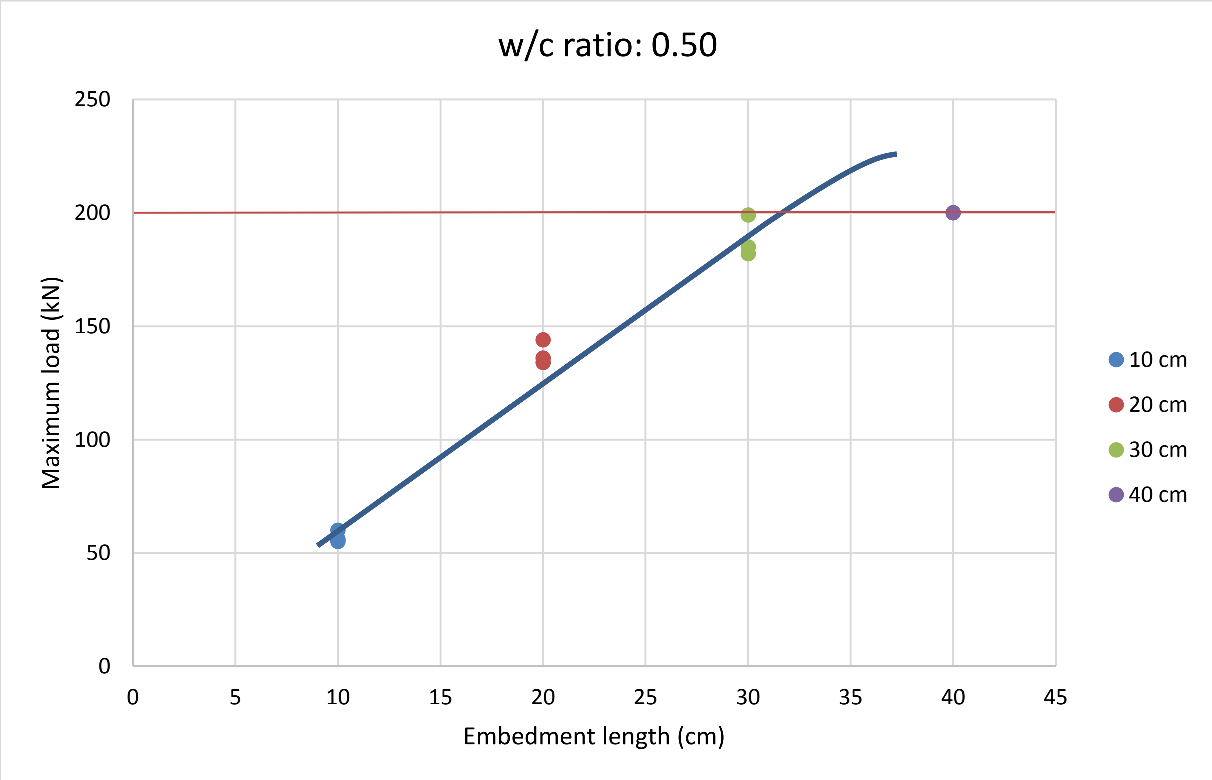

Figure 62 - The maximum load for the three first lengths is below 200 kN but the rebar bolts used only tolerate 200 kN and up to 95 cm in displacement. The bolts with 40 cm embedment length went over the critical embedment length. They were tested and stopped just before they were going to break. The critical embedment length, or the greatest length were the bolts are pulled out without breakage, is therefore between 30 and 40 cm, closer to 30 though.

Rebar bolts with four different embedment lengths were grouted with a water-to-cement ratio: 0.46. The aim was to find the critical length. The Figure 62 shows three lengths below the critical length, wherein the bolts are pulled out without the failure of the rod. The maximum load for the three first lengths is below 200 kN but the rebar bolts used only tolerate 200 kN and up to 95 cm in displacement. The final 40 cm bolts, however, went over the critical embedment length. They were tested and stopped just before they were going to break. The critical embedment length, or the greatest length were the bolts are pulled out without breakage, is therefore between 30 and 40 cm, and closer to 30.

7.1.7 Water to cement ratio: 0.375

In the beginning the plan was to grout 12 bolts with a water-to-cement ratio of 0.375. However, it was decided that the ratio was not a suitable choice because of poor pumpability; the grout was too thick. Therefore, only two bolts were tested with that grouting. Results from these two measurements are presented in figures 63 and 64. The first bolt, B111, was grouted with 10 cm in embedment length but the second one, B141, was grouted with 40 cm in embedment length.

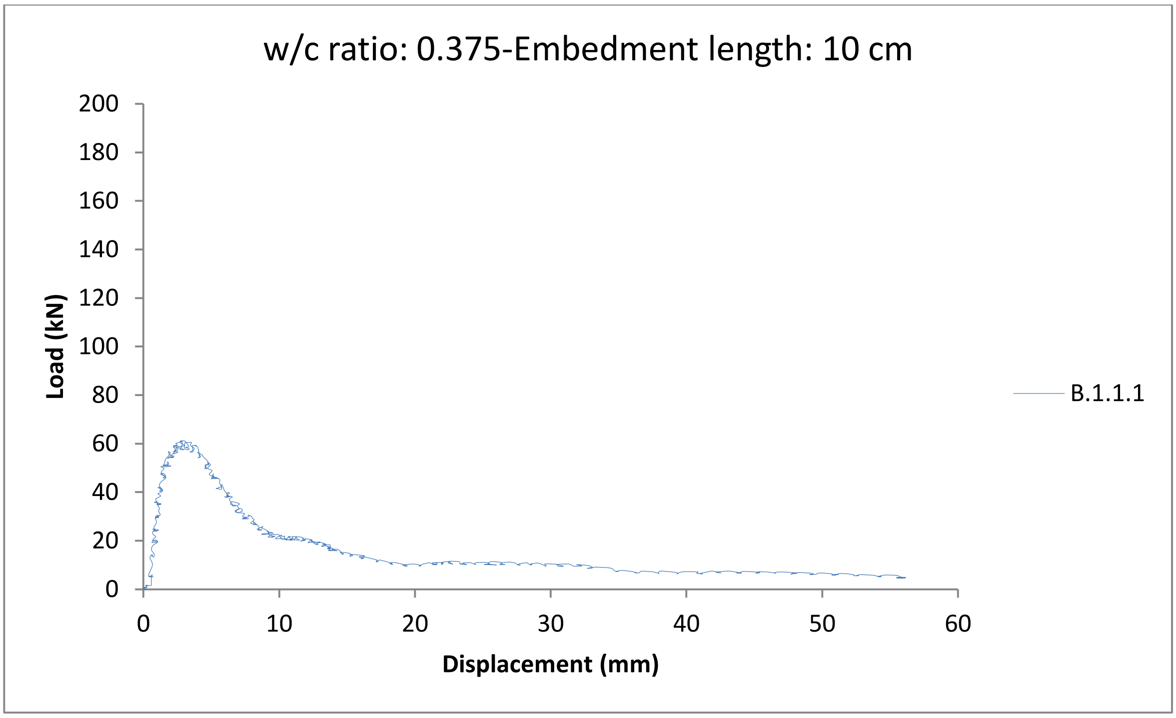

Figure 63 – The maximum load reached 60 kN and the bolt slid from the grout with displacement approximately 5 mm. The embedment length was too short so the bolt did not tolerate much load.

The grouting time was 11 days for both B111 and B141. B111 slid from the grout. The embedment length was too short so therefore, the maximum load only reached 60 kN and then the bolt easily slid from the grout. Different results were obtained with B141. The bolt tolerated a 200 kN load and the displacement went up to approximately 96 mm. Then there was a high pop and the bolt broke. The broken bolt, along with the extensometer, flew meters up in the air and fell down with loud noises. Afterwards, more time was spent on safety rules to keep things like this from happening again.

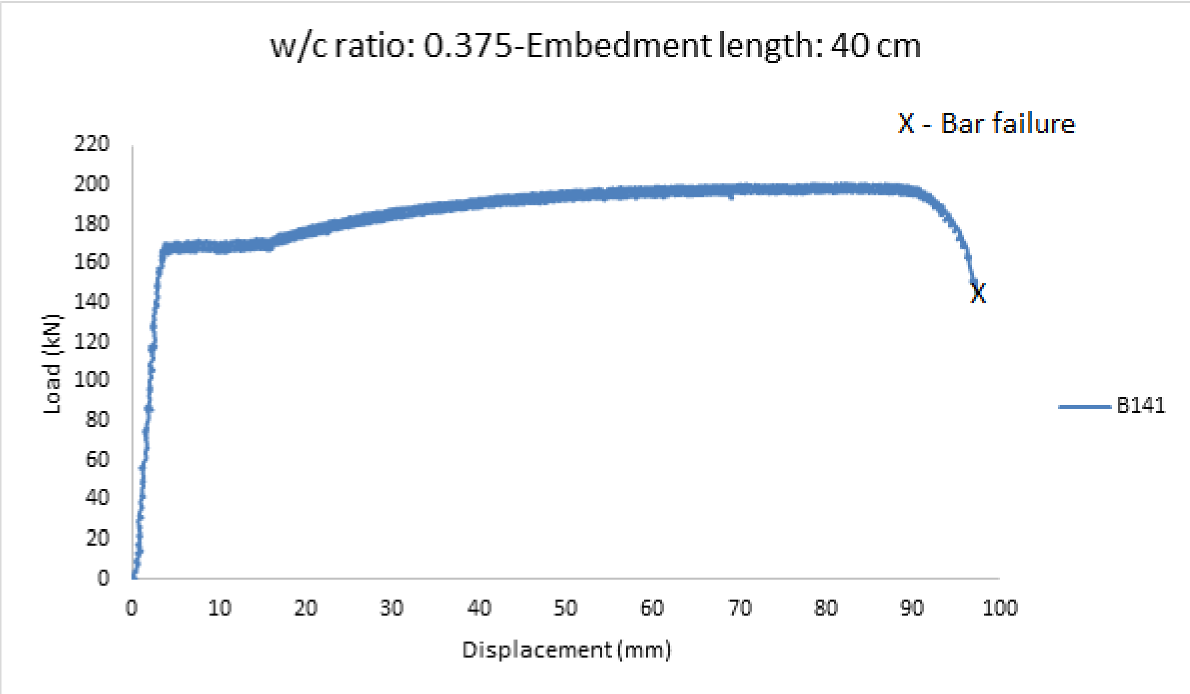

Figure 64 – The bolt tolerated 200 kN load and the displacement went up to approximately 96 mm. The figure shows clearly the necking of the bolt before it breaks. The rebar used in the testing tolerates only 200kN and displacement up to approximately 95 mm.

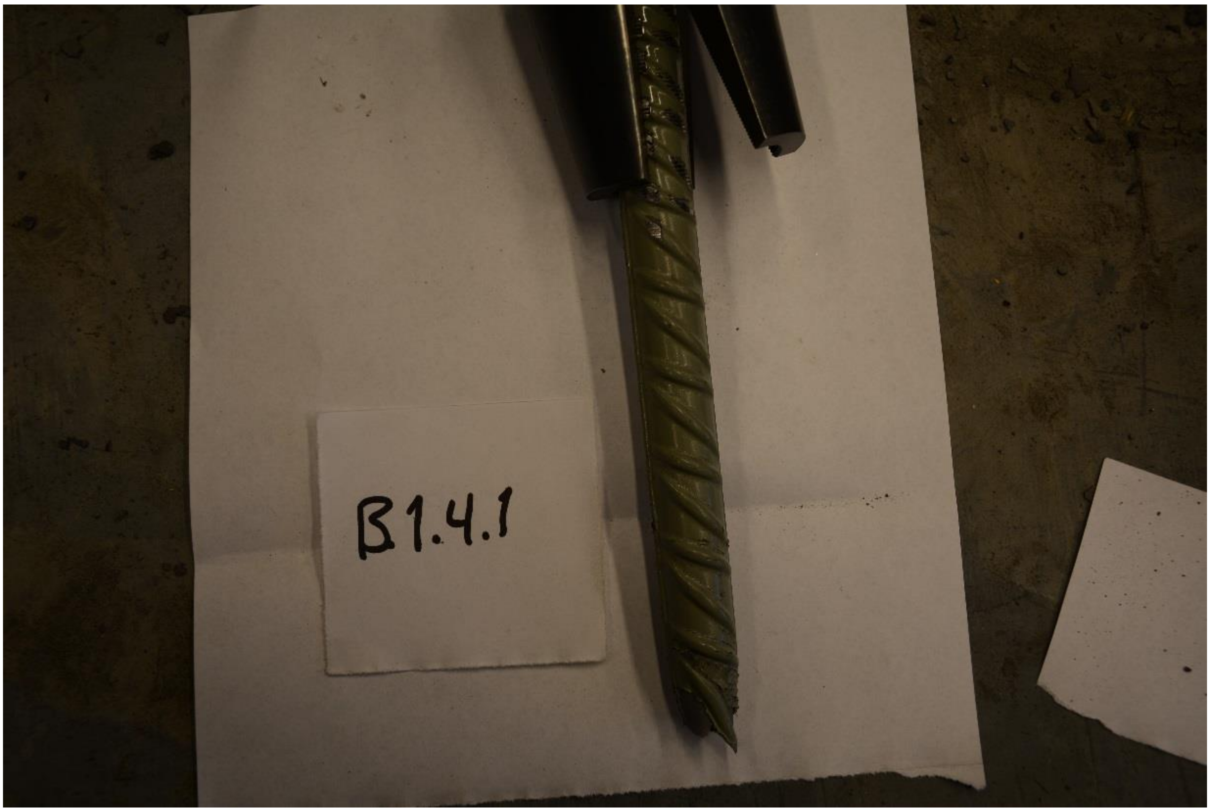

Figure 65 – the figure shows B141 afterwards. The bolt broke and took the wedge with it. The bolt flew 2-3 meters in the air when it broke.

7.1.8 Reduction in diameter and strain measurements

Equation 6.1 below was used to find the strain of the yielded bolts:

$$\varepsilon = dl/l_0 \tag{6.1}$$

Where

dl = change of length (cm)

l_o = initial length (cm)

The stretch length of the bolts was measured before every test and then measured afterwards. Two marks were put on the bolts to identify the stretch length so there would not be any mistake when the stretch length was measured afterwards. The first mark was on the bolt, closest to the surface of the concrete block, and the second one was 60 cm up from the concrete block. To find the most accurate results of the diameter changes, a caliper was used.

The diameter of the rock bolts was measured before testing and then afterwards when the stretch length had increased because of yielding. Results for both strain and reduction in diameter are shown in table 12. The most reduction in diameter was where the greatest stretch length was recorded. The strain is also highest where the greatest stretch length is. The bolts that tolerated most load bearing capacity in the tests had the greatest stretch length and therefore, the most reduction in diameter and highest strain.

Table 12 – Reduction in diameter and strain measurements of yielded bolts taken together in a table. The measured stretch length before testing was 60 cm. The reduction is more as the load is higher. Some tests were stopped before the bolts break and therefore the reduction would be more than the measurement shows.

Bolts number	Reduction in diameter (mm)	Strain (%)
B222	0.15	2%
B223	0.85	4.2%
B231	0.20	2.5%
B232	1.4	10%
B233	Breakage	Over 10%
B361	0.80	3.3%
B362	0.95	5.3%
B363	0.85	3.6%
B331	0.50	3.5%
B332	0.85	7.3%
B333	1	8.3%
B531	0.65	2%
B532	0.69	6.1%
B533	0.76	3.3%
B541	1.26	7%
B542	1.51	10%

7.2 Uniaxial compressive strength test (UCS test)

The uniaxial compressive strength of the grouting materials has an important role in the behavior of rock bolts. Increasing grout compressive strength considerably increases the bond strength of the grouted bolts. The aim was to find the relationship between the critical bolt length and the uniaxial compressive strength of the cement mortar. Therefore, uniaxial compressive strength tests were conducted on cement cubes with three different water-to-cement ratios of the cement mortar. The samples are shown in table 10. First, the plan was to use grout with the water to cement ratio of 0.375. However, the grout was too dry and the pumpability was not good enough (See figure in Appendix F). Therefore, grout with higher water to cement ratio was used for grouting.

The uniaxial compressive strength test was performed in a triaxial compressive strength machine. To get the most accurate results the maximum load (when the specimen broke) was divided with the area of the specimen. All specimen in the test had the same area which was 10*10 cm. The answers are given in MPa which is the unit for stress. Table 13 shows the results for every specimen. Appendix F shows figures from all the specimen before and after the UCS test.

Table 13 - Uniaxial compressive strength test performed on cement cubes with different water-to-cement ratios. The results from the cube with the mark (*) would most likely be higher but before the test was performed there was breakage in the samples.

Uniaxial compressive strength test performed on cement cubes				
specimen for every w/c ratio				
W/C ratio	**1. (MPa)**	**2. (MPa)**	**3. (MPa)**	**4. (MPa)**
0.40	33*	49	43*	43*
0.46	29	31	29	X
0.50	45	43	43	X

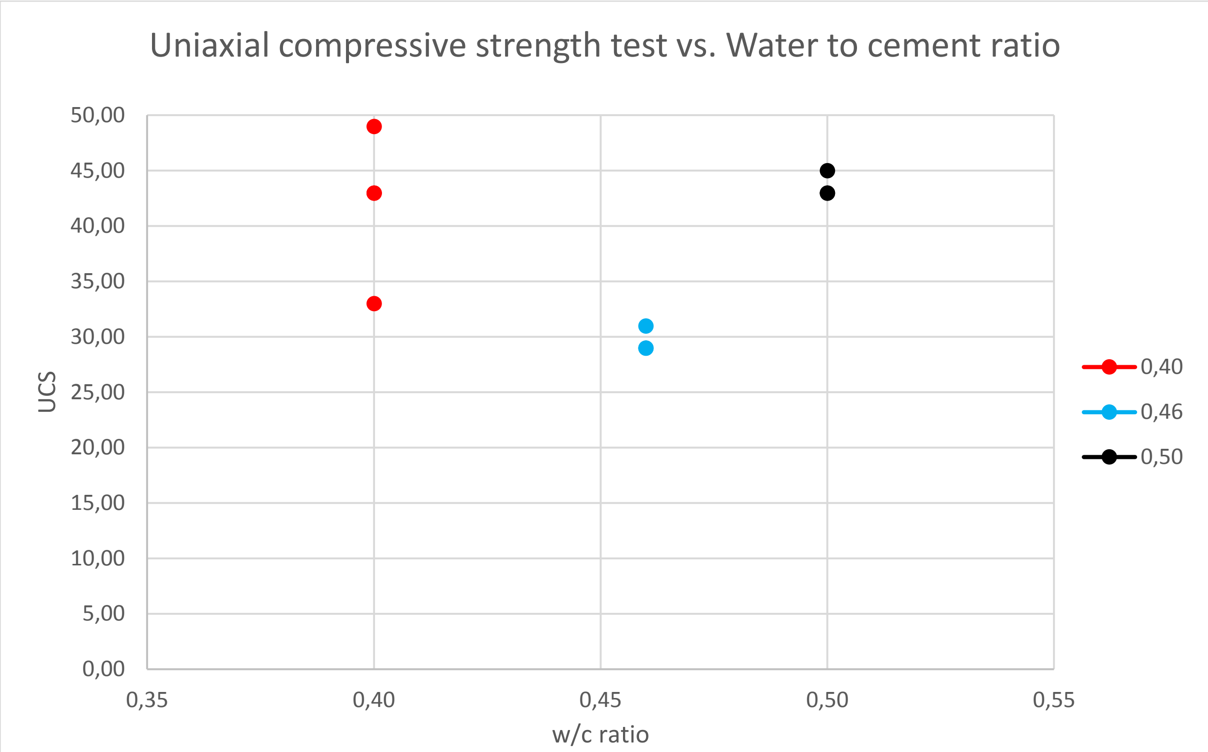

Figure 66 – UCS test versus water/cement ratio. From the results it was not possible to find the relationship between the critical bond length and the uniaxial compressive strength of the cement mortar.

The tests were carried out in two separate days. First, three cement cubes were tested with the water to cement ratio of 0.46 and one cube with the ratio of 0.40. The grouting time for these cubes was seven days. Then the rest of the cubes, three cement cubes with water to cement ratio of 0.40 and three with the ratio of 0.50, were tested few weeks later with the same grouting time. According to Hyet et al (1992) and Kilic, Yasar and Celik (2002) the UCS is supposed to decrease with increasing water to cement ratio. The results given in table 13 and figure 66 do not show similar results.

The results from the previous testing day, were much lower than the results obtained from the UCS tests that was performed few weeks later. The cement cube with the ratio of 0.40 had UCS around 33 MPa while cement cubes with the same ratio, tested few weeks later, had UCS around 50 MPa. The cement cubes with the ratio of 0.50 showed also higher UCS than the cubes with lower ratio or 0.46. It is not possible to estimate what results are the most accurate and which are not because there are too much difference between the values. From the results it can be estimated that some kind of error has occurred. It is not certain what causes this error but most likely it was either mistakes made when the grout was mixed or some technical problems with the triaxial compressive strength machine.

The aim was to find the relationship between the critical bolt length and the uniaxial compressive strength of the cement mortar but with these results it is not possible. If these results are used they would give incorrect relationship between the critical length and the UCS of the cement mortar. Due to lack of time it was not possible to test new specimen and present them in this thesis. However, it is highly recommended that UCS test is performed again on cement cubes with given ratios and then be sure that all mixture of the grout is correct as well as the triaxial compressive strength machine. When that is done it should be easy to find the relationship between the critical length and the UCS strength of the grout.

8 Chapter – Conclusion

Pull-out tests were performed on fully grouted rebar bolts, grouted with the following water-to-cement ratios: 0.40, 0.46 and 0.50.

The diameter of the tested rebar bolts was 20mm and the ultimate load bearing capacity of the bolts was 200 kN with steel strength of 630 MPa. The bolts were grouted with Rescon Zinc bolt cement mortar. The maximum displacement of the rebar, before breakage, was approximately 95 cm.

The main purpose of the testing was to find the critical embedment length for bolts grouted with different water to cement ratios. Different embedment lengths, ranging from 10 cm to 40 cm, were employed in the tests under different ratios for the grouting mortar. The critical length was determined on diagrams of the pull - out load versus the embedment length.

The results showed that for rebar bolts grouted with a water-to-cement ratio of 0.40 the bolts with 20 cm embedment length were close to the critical length. The maximum load reached approximately 180 kN and the bolts began to yield until they slid from the cement. 30 cm embedment length, however, was longer than the critical embedment length. They reached 200 kN and the bolts broke. For rebar bolts with the ratio of 0.46 the 30 cm embedment length was on the verge of being the critical length. The maximum load reached 200 kN but the curve dropped before the bolts broke at the displacement of 50 – 70 cm and the bolts slid from the grout. There were no bolt tested with a longer embedment length and therefore, there was no bolt grouted with a water-to-cement ratio of 0.46 that had longer embedment length than the critical length. However, from the results it can be recommended to test bolts with embedment length between 30 and 35 cm to see if that length goes over the critical length. Finally, the critical length for the ratio of 0.50 was between 30 and 40 cm. The bolts tested with 30 cm embedment length were close to the critical length but bolts with 40 cm broke and therefore, they went over the critical length.

From the test results the relationship between critical embedment length and water-to-cement ratios can be seen. The critical length increases with higher water-to-cement ratios. For example, the rebar bolts grouted with a water-to-cement ratio of 0.40 are capable of tolerating more load when they are grouted with shorter embedment lengths than, for instance, bolts with the ratio of 0.50. In most cases the maximum load was higher for bolts grouted with lower water-to-cement ratio even though the bolts were grouted with same embedment length.

As the tests results show, the water-to-cement ratio is an extremely important factor and it has a great impact on the load bearing capacity of the fully grouted rebar. Before the tests were performed some investigations were made on possible ratios for the grouting. Results showed that grout with a water-to-cement ratio lower than 0.40 were not usable for the testing. Specimens with water-to-cement ratios of 0.35 and 0.375 were too dry and did not have good pumpability. It is important that the grout is pumpable without being too fluid or too dry. Also, it is important to ensure uniform mixing between grout and the water. That was not possible with a ratio lower than 0.40. When bolts are grouted it is important that the space between the bolt and the hole wall is completely filled with grout. The grout did not fill up the hole like it should for bolt number B221. The form of the grouting was like squeezing tooth paste; in that case, it is important to stir the grout better so it is easier to fill it and then the bond strength between the bolt and grout will become higher. The maximum load for that bolt was therefore much lower than for a bolt grouted with same ratio and the same embedment length. The grout ratio for bolt number B221 was 0.40 and it was the driest ratio used in these testing. It was possible to stir the grout but if the ratio had been lower, then the form of the grout would have also been like tooth paste and that would have influenced the pull out tests results. The estimated grouting time was seven days but many tests were performed eight or nine days after the grouting. It seems that it did not have great effect on the results.

The reduction in diameter was measured for all the bolts that yielded after they had been pulled out. The strain was also measured for the yield bolts. The strain for bolt number B233 reached more than 10% before it broke. The reduction in diameter increases as the strain becomes higher. For bolt number B233 the reduction was more than 1.5 mm before it broke.

One of the aims with this thesis was to find the relationship between the critical bolt length and the uniaxial compressive strength of the cement mortar. It was not possible because of some error that occurred when the UCS tests were performed on cement cubes with different water to cement ratio. Therefore, it is recommended to perform UCS tests again to find this relationship.

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Appendix

A)

01.03

Zinkbolt

Mørtel for innstøping av varmforsinkede bolter

PRODUKTBESKRIVELSE

Zinkbolt er en tixotrop, sementbasert, setningsfri tørrmørtel som ekspanderer 1 – 3 % før avbinding.
Zinkbolt er sammensatt av Portland-sement, tilslag med Dmax. 0,5 mm, ekspanderende, plastiserende og stabiliserende stoffer.

BRUKSOMRÅDE

Zinkbolt er utviklet for innstøping av varmforsinkede bolter.
Zinkbolt kan brukes til innstøping av fullt innstøpte, ikke forspente bolter og kombinasjonsbolter.
De ulike boltetypene er nærmere beskrevet i `Håndbok i Fjellbolting`, april 1994.
Mørtelen skal kun tilsettes vann.
Den ekspanderende komponenten gjør at massen ekspanderer 1 – 3 % i fersk tilstand.
Dette sikrer at Zinkbolt fyller godt ut det hulrom mørtelen fylles i, og slutter godt rundt bolten.
Massen må være plassert senest 45 minutter etter blanding. Dette for å få full utnyttelse av ekspansjonen.

BRUKSANVISNING

Zinkbolt blandes i egnet blande- og pumpeutstyr som kan være både mono- og stempelpumper.
Det er viktig at det ikke blir brukt mer vann enn nødvendig for å plassere mørtelen på en sikker måte.
Konsistensen skal være kremaktig.
Husk: Jo mer vann man bruker, jo lavere blir fastheten.
Dessuten vil for mye vann gi fare for sig i massen. Dette kan føre til dårlig innstøping av bolten.
Zinkbolt skal benyttes ved temperaturer over +5°C.

VERNETILTAK

For helse-, miljø- og sikkerhets-informasjon - se eget HMS-datablad. HMS-databladene finnes på www.resconmapei.com.

MERK

De tekniske anbefalinger og detaljer som fremkommer i denne produktbeskrivelse representerer vår nåværende kunnskap og erfaring om produktene.
All ovenstående informasjon må likevel betraktes som retningsgivende og gjenstand for vurdering.

Enhver som benytter produktet må på forhånd forsikre seg om at produktet er egnet for tilsiktet anvendelse. Brukeren står selv ansvarlig dersom produktet blir benyttet til andre formål enn anbefalt eller ved feilaktig utførelse.

Alle leveranser fra Rescon Mapei AS skjer i henhold til de til enhver tid gjeldende salgs- og leveringsbetingelser, som anses akseptert ved bestilling.

N.B! BØR UTFØRES AV FAGFOLK.

TEKNISKE SPESIFIKASJONER		
Trykkfastheter MPa:		
Døgn	*V/+5°C*	*V/+20°C*
1	15,0	24,0
2	22,0	31,0
3	30,0	37,0
7	41,0	45,0
14	46,0	56,0
28	52,0	60,0

Prøvingen er foretatt på 40 x 40 x 160 mm prismer.
Benyttet vannmengde: 6,5 liter pr. sekk á 25 kg.

Zinkbolt er testet m.h.p. permeabilitet i overgangssonen mørtel-varmforsinket bolt i henhold til beskrivelser i NS 3420 og ISO/DIS 7031, og den er testet m.h.p. forankrings-kapasitet for 18,5 mm varmforsinket kamstålsbolter innstøpt i Zinkbolt med verdi på 102,7 kN.
Forsøkene er utført ved Sintef FCB.
Prøvingsresultatene oversendes ved forespørsel.

Materialforbruk:	*1,6 kg tørrmørtel pr. liter ferdig masse ved et vannforbruk på 6,5 ltr. pr. 25 kg mørtel.*
Emballasje:	*Zinkbolt leveres i 25 kg sekker.*
Lagring:	*Må lagres tørt. Er i uåpnede sekker holdbar i minst 12 måneder.*

Produsent:
Rescon Mapei AS
Vallsetvegen 6, 2120 Sagstua, Norway
Tlf: +47 62 97 20 00 Fax: +47 62 97 20 99
post@resconmapei.no
www.resconmapei.com

Rescon Mapei AS
et selskap i

B)

Natural stresses

Natural stresses are stresses found in rock before excavation and comprise of gravitational stresses, topographic stresses, residual stresses, tectonic stresses and thermal stresses

Gravitational stresses

Gravitational stresses are caused by the weight of the rock per unit area above a specific point in the earth's crust. Assuming the surface is horizontal then the vertical gravitational stress at a depth z is:

$$\sigma_z = \rho * g * z$$

ρ=density=mass/volume

g=acceleration=9,8 m/s2

z=depth

Over 1000 meters from gravity the gradient of stress is 0.026 MPa/m but increases to 0.029MPa/m when the density of rocks reaches 3000 kg/m3 (Herget, 1988). The magnitude of the vertical stress may be the same as the magnitude of the gravitational vertical component with some deviations. As illustrated in the figure ___the deviations are particularly at great depths.

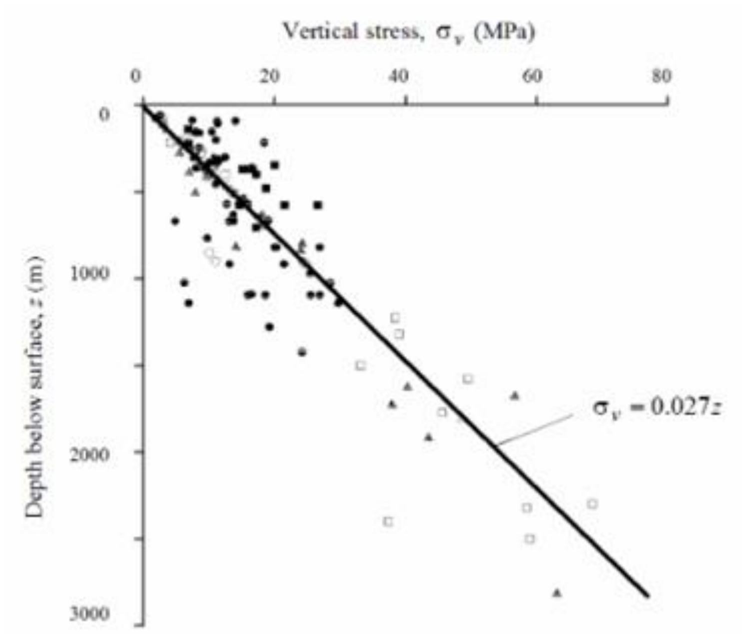


Figure – As the depth below surface increases the vertical stresses increases as well. The magnitude of the vertical stress may be the same as the magnitude of the graviational vertical components with some deviation. As seen from the figure, the deviations are particulary at great dephts (Hoek, 2007b).

It is more complex to define the horizontal component than the vertical component. The reason is different boundary conditions and the effect of rock mass properties. It depends for example whether the material in the crust is strictly elastic or not. If the material is strictly elastic and there are no horizontal displacement the Poisson's ratio (v) is used to describe the relationship between the two components of the gravitation stresses. The equation for horizontal stresses is:

$$\sigma_H = \frac{v}{1-v} * \sigma_z$$

The Poisson´s ratio can varies between 0.15 to 0.35 for most of the rocks but $v = 0{,}25$ is a very common value for rock masses. It means that the horizontal stress induced by gravity is 1/3 of the vertical stress (Herget, 1988).

Topographic stresses

The topography has a great influence on the rock stress situation when the surface is uneven. Topographic stresses are stresses caused by topographic effects. In deep valleys and mountain slopes where underground excavations are often located the stress situations by topographic effects will be dominated. σ_1 Is more or less parallel to the slope of the valley near the surface and the minor principal stress,σ_3 is perpendicular to the slope in same situation (Nilsen and Palmström, 2000).

Tectonic stresses

It can be difficult to predict magnitude and direction of tectonic stresses unless there have been recent movement caused by tectonic or seismic activity (Herget, 1988). Tectonic stresses are mainly caused by plate tectonic or the continental drift. Incidents such as faulting and folding are occur because of tectonic stresses. The total horizontal stresses are normally much higher than the horizontal stress caused by gravitation. That is mainly due to the existence of tectonic stresses. This is mainly in places at shallow or moderate depths (Nilsen and Palmström, 2000).

Residual stresses

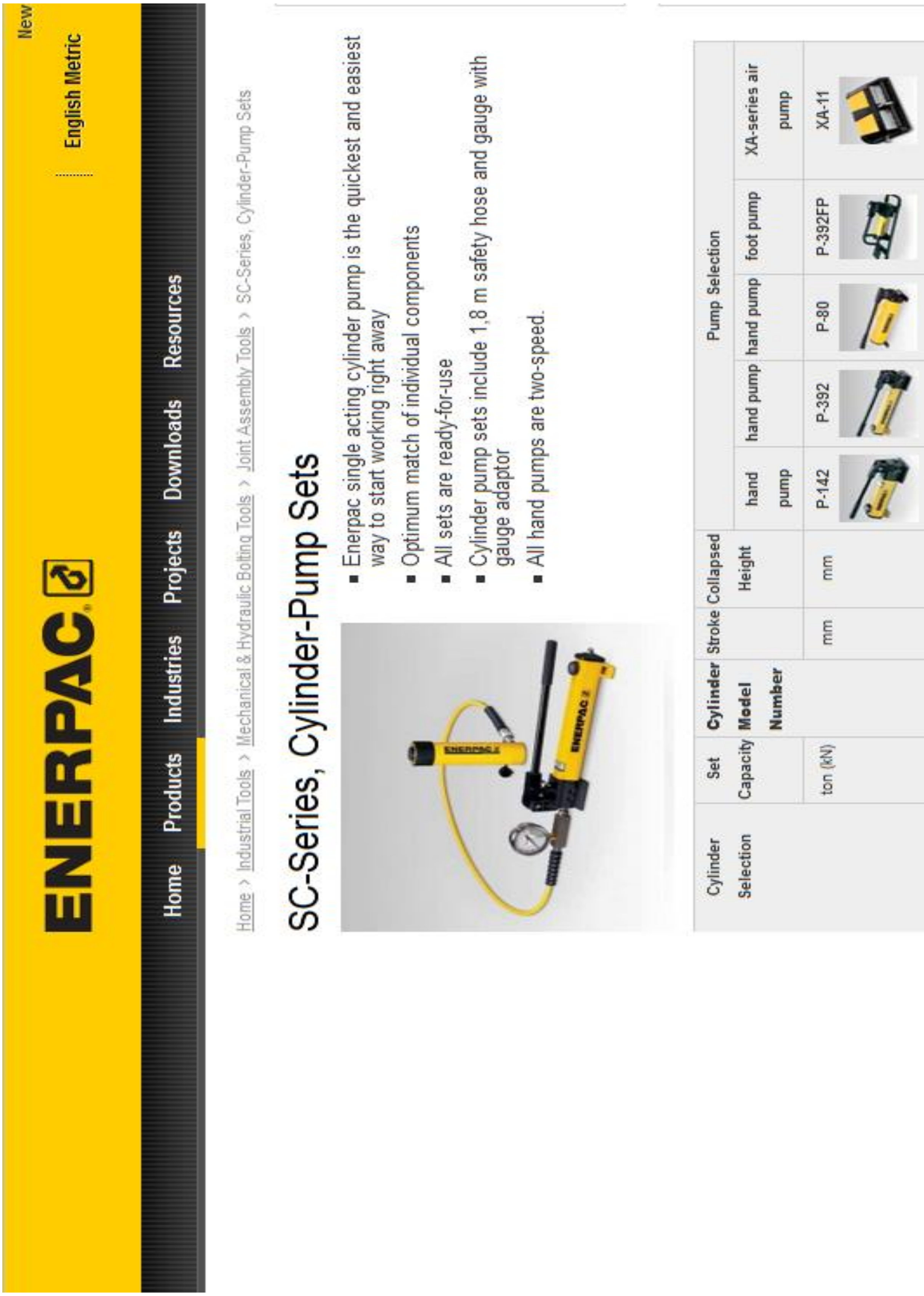
Stresses that are locked into the rock material during earlier stages of it geological history are referred to as residual stresses. Stress caused by contraction during a cooling of rock melt is an example of residual stress. By using strain recovery measurements on rock samples of different size it is possible to identify residual stresses. Abnormally high vertical stresses are often explained as being caused by residual stresses (Nilsen and Palmström, 2000).

Thermal stresses

Thermal stresses are due to cooling or heating of rock. Close to the surface the rock is exposed to the sun that causes thermal stresses. Results of heating occur also from the interior of the earth by radioactivity and other geological processes (Herget, 1988).

C)

Hydraulic jack used for pull out test



D)

Table for different grouting time	
Rock bolts	**Time (days)**
B211	8
B212	8
B213	8
B251	7
B252	7
B253	7
B221	9
B222	9
B223	9
B231	9
B232	9
B233	9
B241	9
B311	8
B312	8
B313	8
B321	8
B322	8
B323	7
B361	7
B362	7
B363	7
B331	8
B332	8
B333	8
B511	8
B512	8
B513	8
B521	7
B522	7
B523	7
B531	7
B532	7
B533	7
B541	7
B542	7
B111	11
B141	11

Grouting time for cement cubes was seven days for all specimen

E)

Figures from pull out testing. Pull out length shown for every bolt tested.

B111

B141

B211

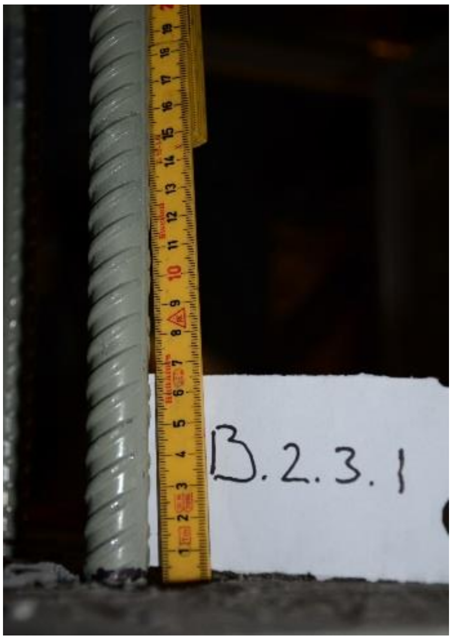
B212

B213

B221

B222

B223



B231

B232

B233

B241


B311


B312


B313


B321


B322

B323

B331

B332

B333

B361

B362

B363

B251


B252

B253

B511

B512

B513

B521

B522

B523


B531


B532

B533

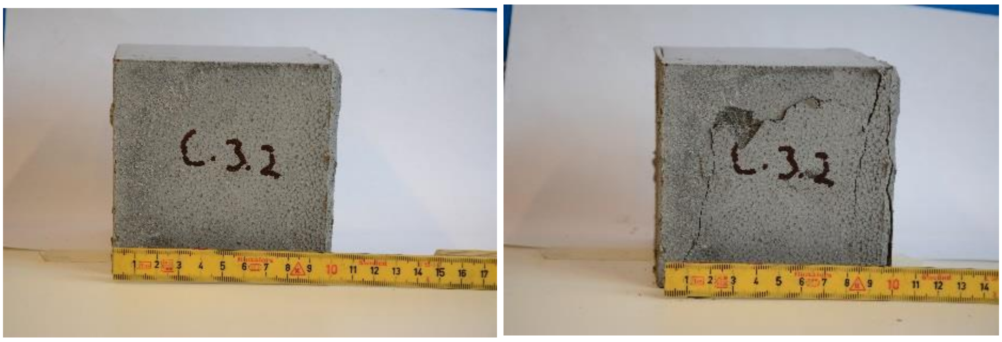
B542

B541

F)

Figures of UCS specimen

w/c ratio: 0.46 before and after



w/c ratio: 0.46 before and after

w/c ratio: 0.46 before and after

w/c ratio: 0.40 before

w/c ratio: 0.40 after

w/c ratio: 0.40 before

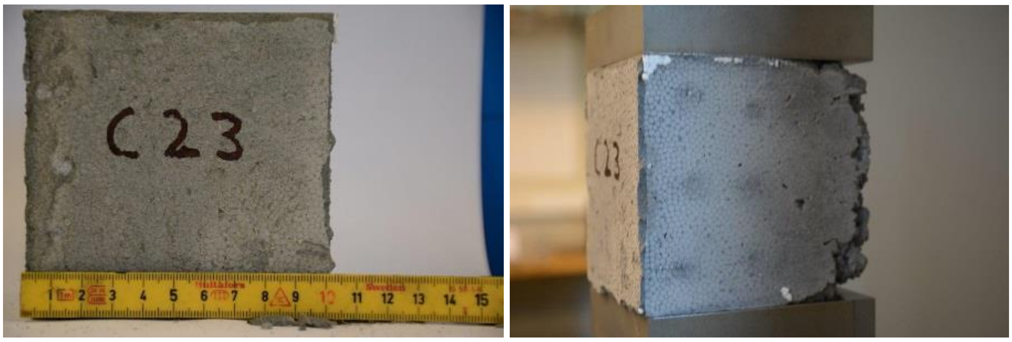
w/c ratio:0.40 after

w/c ratio: 0.50 before and after

w/c ratio: 0.50 before and after

w/c ratio: 0.50 before and after

w/c ratio: 0.40 before and after



w/c ratio: 0.40 before

w/c ratio: 0.40 after

w/c ratio: 0.40 before and after

w/c ratio: 0.375. It was too dry. Not possible to make cement cubes