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Rock bolts - Improved design and possibilities

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FOREWORD AND ACKNOWLEDGEMENT

This master thesis, "ROCK BOLTS : IMPROVED DESIGN AND POSSIBILITIES", is the achievement of the engineer formation of Capucine Thomas-Lepine in the civil engineer school ENTPE, France, in summer 2012, and was realized in partnership with NTNU, Norway.

Works consists of full scale testing and analysis of tensile capacity from 50 passive rock bolts (length 0,4meter) with regards to rock mass quality. It concludes with a first proposal for a renewed design method in rock bolts, and the associate domain of validity and factor of safety. This thesis is a continuation of the thesis from Lars Kristian Neby, NTNU 2011. Critic of the works and recommendations for further research are explicit at the end of the rapport

The writer is grateful to the followings for their contribution :

I would like to thank my supervisor Leif Lia (NTNU vann og miljø, professor for dams design) who proposed me the thesis and remains available to support the research. I am also grateful to Lars Kristian Neby (Norconsult), co-supervisor, who advised with its experience from the 1st thesis on the subject he wrote last year. My thanks extend as well to Eivind Grøv (Sintef), co-supervisor who helped with rock assessment and searching for a quarry to perform tests. Thesis is based on the design from NVE, which is thanked for its responses and the interest manifested when it was contacted for details on its design method.

Energi Norge is thanked for supporting funding that made the research possible.

Finding a place to perform testing was crucial in the thesis and challenging. Several quarries from Franzefoss responded to our demand, few of them had the expected cracked rock mass conditions. Thanks are addressed to Geir Roar Berg (Verdalskalk) for use of the quarry and its excavator.

For assistance with equipment for tests, thanks are extended to the following :
Trond Larsen (Sintef) : core drilling, Ole Mollerud (Kmaskin) : dynamometer, Vik-Ørsta : bolts
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MASTER TEXT

Master of science thesis

Stud.techn: Capucine ThomasLepine

Rock bolts – improved design and possibilities

INTRODUCTION

Rock bolts is used in concrete dams to improve their stability. From the first design criteria developed by hydropower industry in the 80's, NVE have made new regulations several times. Each time the design criteria got more rigorous. Use of rock bolts and how to design them is a continuousdiscussion within the hydropower industry in Norway. A new regulation applies for all existing dams as well as for new ones. This means that many old dams have to be rehabilitated to stand new requirements.

BACKGROUND

During 2010/2011 Lars KristianNeby did a literature survey on un tensioned fully grouted rockbolts design ”*Dimensjoneringav slake fjellbolter i damanlegg*” As an extension of this work a series of full scale capacity tests were done in Neby's master work, presented in "*Fjellbolter i dammer*". The survey reviled that rock bolt design in Norway are based on the method used for pre-stressed anchors. Tests showed that rock bolts have high capacity even when the rock mass is poor. There is need of a new design method for rock bolts since the current one underestimate the capacity.
The main goal with this thesis is to extend number of full scale capacity test of rock bolts and to improved design method for rock bolts.

EXECUTION OF TASK

The master thesis can be divided in several phases:

Initial work

This thesis should be based on the work done by Lars KristianNeby, report from EnergiNorge and other. Results from master thesis "Fjellbolter i dammer, forventa kapasitet" should be evaluated to predict bolt capacity versus rock mass quality. Analyse of crack distance and crack opening with respect to bolt capacity might give interesting results.

Planning of experiments

Based on earliertests (Neby, Sweco/Statkraft), other literature and own assessments a method for testing and a test program should be planned. Assumptions and gear requirement for rock bolt testing should be listed.
There should be planed one test series to find critical depth were rock bolts will yield regardless of rock mass quality. Is important that the rock mass quality is "poor" and homogenous along the hole grouted length of the bolt. Critical depth is expected to be 1 meter. Rock mass for each bolt should be evaluated and later analysed to see if there is any correlation between capacity and rock mass quality.
Another series with as similar conditions as possible due to rock mass parameters and grouting depth should also be planed.

To check the importance of flushing the drill hole a third test series should be planed. This test should be done in an area with high quality rock and short drill holes. To evaluate the result a series of flushed and not flushed holes in similar rock mass and same grouting should be planned.

Full scale testing of rock bolts

For all experiments planned in 3.2 the candidate should organize and perform the tests.

A location were test should be executed should be planed early in the work period. There is also necessary to establish contact with a firm who can do the drilling and loading of bolts to failure.If possible testing should be done in different rock types. Earlier test gives indications of rock quality needed to get failure in rock mass.

Capacity tests of rock mass should be done by a large excavator;this method makes failure in rock mass possible and provides necessary lifting capacity.

In addition capacity of a series of bolts in 1-2 meter deep holes filled with gravel and/or sand in high quality rock mass should be checked.

A test program for testing the differences between core drilled holes and hammer drilled holes should be carried out.

Results of testing

Based on test results, a relationship between measured capacity and key parameters of rock mass should be established. Crack opening and distance between cracks is possibly the most important parameters. But it might be other parameters which are of great interest. Results in 3.3 should be compared with values from today's dimension criteria, (the NVE method). If no relationship or critical depth can be established further test should be proposed.

Conclusion

If possible the conclusion should lead to a design procedure for necessary anchor depth for foully grouted rock bolts. Design procedure might be based on existing method or self-made method based on test results. The conclusion should also pinpoint subjects which need more research and propose how new test can be done.

Report

The final report should be written as a scientific report. All figures, tables, pictures should be of high quality. All used reference should be listed at the end of thesis and continuously in the report. Methods of analyse and performance of test should be described in scientific manner. The final report must be in A4-format in three copies, submitted in pdf-format. Standard cover for master thesis is available online. Students are economically responsible for print costs for three items. If more than three copies are required the department will take the extra cost. Together with the final report a CD with online version of the report and all data files (pictures, excel work sheet and so on) should be delivered

A summary of maximum two pages should be placed first in the report. The summaryshouldinclude a presentation of the assignment, work method, discussion of key results and a conclusion.

CONTACTS

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Sintef/NTNU	EivindGrøv, Professor II (co-supervisor)
Norconsult	Lars Kristian Neby
NVE	Rune Engesæter

Trondheim, 16. January 2012

Leif Lia
Professor

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SUMMARY

Key words : *rock foundation, small concrete dam, rock mass classification, rock joints, shear strength of rock discontinuities, fully grouted passive rock bolts design*

Masters Thesis : “Rock bolts, improved design and possibilities” is a continuation from the Masters Thesis NTNU 2011 “Rock bolts in dams, expected capacity” by Lars Kristian Neby. Internationally, dam engineering focuses mainly on pre-stressed anchors in rehabilitation and improvement of stability of large dams, which is undergoing constant research in North America. Passive rock bolts are used in small concrete dam foundations to ensure sufficient stability against overturning moment from ice loads. This concerns the majority of dams in Norway, over 98% of whose electricity comes from hydropower developed over the last 100 years and still developing. Design is ruled by regulation from NVE (Norwegian water resources and energy directorate) published first in the 1980s, and regularly revised until the retroactive “Retningslinjer for betongdammer” in 2005. This design method for passive rock bolts is conservative with regards to rock capacity, as it is worldwide. The model, developed in the early age of rock bolt development in 1977 by Littlejohn and Bruce, considers the rock resistance as equivalent to the weight of the cone of rock around the bolt. Rock engineering has improved since, often with regards to underground engineering, which is not necessarily transposable to dam engineering. The inherent uncertainty in rock mass characterization slowed development of new design method for passive rock bolts. This is however of great interest in the Norwegian hydropower industry, and for applications to other civil engineering structural foundations.

This thesis is meant to develop knowledge of qualitative and quantitative rock mechanisms in passive rock bolts in order to improve their design.

The work is composed of three parts, as follows :

A study on rock mass capacity and mechanisms in dam foundations, comes first. An empirical and quantitative estimation of rock mass strength with regards to recognized classification Q, RMR or GSI is proposed, based on Wyllie (1992) and results from Lars K. Neby.

Full scale tests are then performed to the assess validity of empirical relationships developed in the first part between rock mass quality and rock bolt capacity (maximal tension load in pull out tests). 50 steel bolts with diameter 25mm and mortar grouted length 0.4m were pulled out with logging of strength and deformation in a rock quarry presenting various degrees of rock quality (RMR 40 to 80), representative of expected conditions for dam foundations. Rock quality was assessed for each bolt by laboratory testing (intact material properties) and rock mass characterization on site supplemented by core drilling or video inside hammer drilled holes. This program of tests was an improvement on the protocol developed and performed by Lars K. Neby on 18 bolts, whose results give the first relevant clues on parameters such as limit range of quality of rock (RMR>40), length of bolt (0.4m), maximal capacity (more than 20 tons, when conservative calculations assessed 0.2 tons). The results of testing confirm the relationships developed in a more statistical approach.

Conclusions from these two parts lead to a proposition for a new design model, with a higher resistance contribution from the rock. The three modes of failure (rock, steel, grouting) are considered for ensuring resistance of a maximal stress of 180MPa (to avoid important deformations in the structure). Factors of safety and the range of validity in the rock mass condition for the proposed design are also considered.

The thesis concludes with propositions for further works in order to :

- Further extend the domain of validity of the proposed design.
- Review methods of control of installed passive rock bolts.
- Document and improve knowledge of load transfer mechanisms in passive rock bolts in dams.

Sammendrag

Stikkord: *bergfundament, små betongdammer, bergmasseklassifisering, skjærstyrke av diskontinuiteter, dimensjonering av slakke fjellbolter*

Masteroppgaven «Rock bolts, improved design and possibilities» er en videreføring av masteroppgaven "Fjellbolter i demninger, forventet kapasitet" av Lars K. Neby ved NTNU i 2011.

Slakke fjellbolter brukes i små demninger for å sikre tilstrekkelig veltestabilitet. Dimensjoneringsmetoden til NVE (Norges vassdrags- og energidirektorat) for slakke fjellbolter er konservativ med hensyn til fjellkapasitet. Modellen, utviklet i 1977 av Littlejohn og Bruce, vurderer fjellmotstand ut fra påhengt fjellprisme. Metoden krever lang forankringslengde for å mobilisere kapasiteten i boltene, noe som er et problem for mange eldre norske dammer hvor boltene er satt ned med kort forankringslengde.

Målsettingen med oppgaven er å utvikle kunnskap om kvalitative og kvantitative fjellmekanismer slik at dimensjoneringsmetoden kan forbedres.

Arbeidet har bestått i tre deler:

Første del er utført som et studie av bergmassens kapasitet og mekanismene i bergfundamentet. En empirisk og kvantitativ metode er foreslått basert på Wyllie (1992) og resultater fra Lars K. Neby for estimering av fjellkapasitet.

Andre del er utført med fullskala feltforsøk for å validere de empiriske sammenhengene mellom kvaliteten i bergmassen og forventet strekkapasitet av fjellboltene. 50 bolter ble trukket med logging av styrke og deformasjon i forskjellig fjellkvalitet i et steinbrudd ved Verdal i Trøndelag. Berget er vurdert fra bergmekaniske tester utført i forbindelse med oppgaven til Lars K. Neby. Resultatene er validert med en lineær sammenheng mellom kvaliteten av bergmassen og forventet kapasitet av en bolt.

Tredje del har bestått i å foreslå en ny dimensjoneringsmetode med større bidrag fra fjellstyrken. De tre måter brudd utviler seg (berg, stål, mørtel) vurderes for å sikre en belastning begrenset av 180 MPa for å unngå store deformasjoner i strukturen. Sikkerhet og gyldighet er vurdert.

Oppgaven foreslår følgende områder for videre forskning:

- Validere foreslått dimensjoneringsmetode
- Metoder for kontroll av slakke fjellbolter
- Dokumentere bruddmekanismen i slakke fjellbolter

RESUME

Mots-clés : *fondations rocheuses, petites barrages en béton, classifications des massifs rocheux, résistance au cisaillement d'un joint rocheux, ancrage passif*

Le mémoire de master « Rock bolts, improved design and possibilities » est une poursuite de celui de Lars Kristian Neby « Rock bolts in dams, expected capacity », NTNU 2011 . A l'échelle internationale, les tirants d'ancrage précontraints sont largement utilisés dans l'ingénierie des grands barrages pour les réhabilitations ou confortement, et font l'objet d'un programme de recherche étendu en Amérique du Nord. Les ancrages passifs sont utilisés pour les petits barrages pour assurer un renfort de stabilité au moment renversant créé par la surcharge de la glace. Ces ouvrages représentent la majorité des barrages en Norvège, alimentée à plus de 98% par l'hydroélectricité dont le développement débuta il y a un siècle et se poursuit encore aujourd'hui. Les règles de conception sont contrôlées depuis les années 1980, et en 2005 entrent en vigueur de nouvelles contraintes de dimensionnement plus exigeantes et rétroactives. Le dimensionnement des ancrages passifs y est conservatif, comme il l'est aujourd'hui mondialement. Le modèle considère la résistance du massif rocheux équivalente au poids du cône de roche autour de l'ancrage. Les développements de la mécanique des roches, majoritairement dans le domaine des ouvrages souterrains, ne sont pas transposables aux conditions rencontrées dans les fondations des barrages. L'incertitude inhérente à la caractérisation du massif rocheux a freiné le développement de nouvelles méthodes de dimensionnement pour les ancrages passifs. Cela n'en présente pas moins un intérêt certain tant pour l'industrie hydroélectrique en Norvège que plus généralement pour les ouvrages d'art.

L'objectif de ce mémoire est de développer les connaissances qualitatives et quantitatives des mécanismes de résistance du massif rocheux dans les ancrages passifs pour proposer une nouvelle méthode de dimensionnement. Le travail s'est organisé en trois étapes.

Une étude des principaux modèles de mécaniques des roches et des particularités présentées par les fondations des barrages est réalisée dans un premier temps. Une estimation empirique de la résistance à la traction du massif rocheux caractérisés par les indices Q, RMR et GSI est proposée à partir du calcul de la résistance au cisaillement d'une discontinuité rocheuses de Wyllie (1992), ou directement par formule empirique déduite des résultats de Lars K. Neby.

Des tests à grande échelle d'arrachements d'ancrage avec mesure de la résistance en tension et des déformations du massif sont ensuite menés pour 50 ancrages en acier 500MPa de diamètre 25mm scellés au mortier sur une longueur de 0,4m dans un massif rocheux présentant différents degré de fracturation (RMR de 40 à 80). Pour chaque ancrage, la qualité du massif est estimée en laboratoire (ISRM, propriétés de la matrice rocheuse) et sur site, ainsi que par des carottes de forages, ou des inspections vidéo à l'intérieur des forages destructifs (RQD). Ce programme de test est une reprise améliorée du protocole développé par Lars K. Neby pour 18 ancrages de 0,1 à 1m. Ses résultats ont mis en évidence que pour un massif rocheux moyen au regard du classement RMR (RMR supérieur à 40), la résistance en tension maximale atteint plus de 200kN pour des ancrages de longueur inférieur à 1m, alors que le calcul conservatif obtient 3kN pour un ancrage de 0,5m. Les tests réalisés dans ce mémoire ont confirmé et précisé dans une approche davantage statistique les résultats de Lars K. Neby, et une relation linéaire est validée entre la résistance à la tension de l'ancrage passif, sa longueur et l'indice RMR du massif.

Les conclusions précédentes permettent de formuler une première proposition de révision du dimensionnement actuel qui prend en compte la résistance du massif rocheux. Les facteurs de sécurité et domaine d'application associés sont également précisés.

Le mémoire conclus sur des propositions de poursuite de recherche pour :

- Augmenter le domaine d'application du dimensionnement proposé
- Préciser les connaissances actuelles sur les scellements et étudier les méthodes de contrôles des ancrages installés
- Préciser le transfert des efforts par les ancrages passifs dans les barrages

INTRODUCTION

Foundation is crucial for dams, and is one of the first causes of failure. Rock mass capacity differs from the intact rock, depending mostly on cracking conditions. Passive rock bolts ensure stability of small concrete dams since the beginning of this technology, about 50 years ago. Small dams, under a height of 7 meters, have lower self weight than large dams. Reinforcement is especially required against ice load in Norway.

Design of passive rock bolts is conservative worldwide, and has been discussed in many years in hydropower industry in Norway. This thesis is a contribution to improve their design.

Lars Kristian Neby initiated the research in NTNU in 2010. First a project work (2010) reviewed design methods for passive rock bolts. Then a master thesis (2011) performed full scale test of 18 bolts with grouted length 0.1 to 1 meter in cracked rock mass to measure the maximal pull out capacity [tons] and determine rock mass parameters related with rock bolt capacity. This work concluded, as follows :

- Bolts have far higher capacity than expected by the conservative worldwide used design method. Capacity is up to 20 tons (400MPa) for bolts about 0.5 meter long, when conservative calculations expects 0.3 tons. The conservative model considers the resistance of the rock as the self weight of a cone around the bolts. Design method considering shear strength in rock discontinuities, such Wyllie based on GSI-rock mass classification, gave relevant capacity values.
- Few bolts, and in extremely different rock mass conditions did not permit to propose a reliable relation between rock mass quality and rock bolt capacity. The relevant parameters are : distance between discontinuities, opening and surface conditions or filling of discontinuities.

This thesis analyses further L.K. Neby results and perform full scale test with 50 bolts in a statistical approach and presents the obtained achievements in order to

1) Develop a relation between rock mass quality and rock mass capacity
2) Propose a new design method for rock bolts
3) Highlight limits and propose further research

These 3 points are presented in this report, after a theoretical presentation on the rock material with regards to concrete dam foundations requirements.

1 ROCK FOUNDATIONS IN CONCRETE DAMS

1.1 Rock mechanism and strength in dam foundation

1.1.1 Rock requirements in concrete dam foundation

“Rock is now considered in the design as a full part of the dam” (ICOLD)

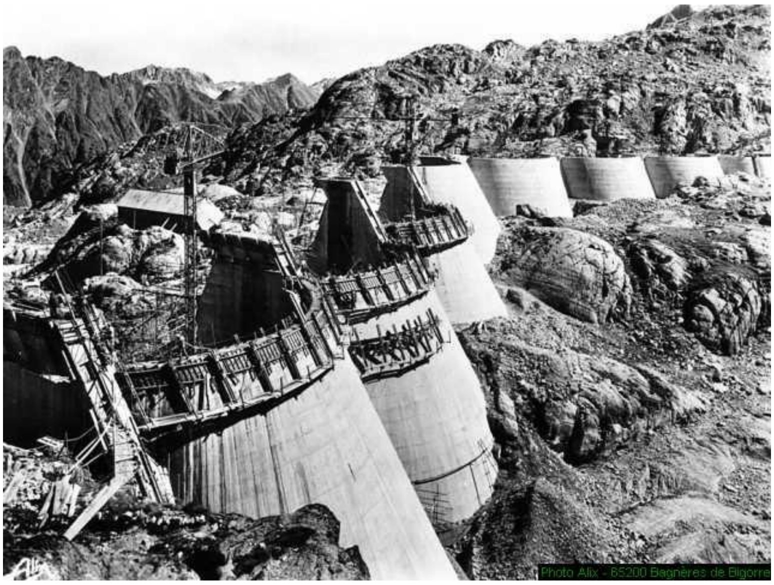


Figure 1 : Migouélou, Pyrénées 1956-1958

Concrete dams foundations must be on fair rock mass. Small dams are less demanding than large ones, as the maximal stress can be first approximate proportional to the height. Yet the good rock quality is required for the following reasons [5]:

- The rigid structure of the dam can’t tolerate differential movements. Rock foundations can take further load from the structures than soil, without deformations.
- Stress diagram is radically different between a full or empty reservoir. This can induce fatigue on a bad rock mass over the filling and emptying of the reservoir.
- Hydraulic gradient are high in foundation. Internal erosion is a risk for poor rock.

A proper investigation of the rock on site is essential. Rock mass capacity is indeed lower than intact rock capacity. Discontinuities in rock mass imply modification of distribution and concentration of loading. Most dam failures are related to rock foundation failure, which happens mainly along existing weakness of the rock (Malpasset, 1959). A treatment of the foundation (for instance grouting) can be performed before building of the dam.

Dam foundation engineering has specificities [6] :

- Dam foundation concerns the rock surface up to a depth of about 10 meters.
- The loading of dams in foundation is first horizontal, from the pressure of water. The vertical contribution of the weight of the concrete structure is less important.
- Foundation has intern loading, with modification from water condition inside the rock (hydraulic gradient).

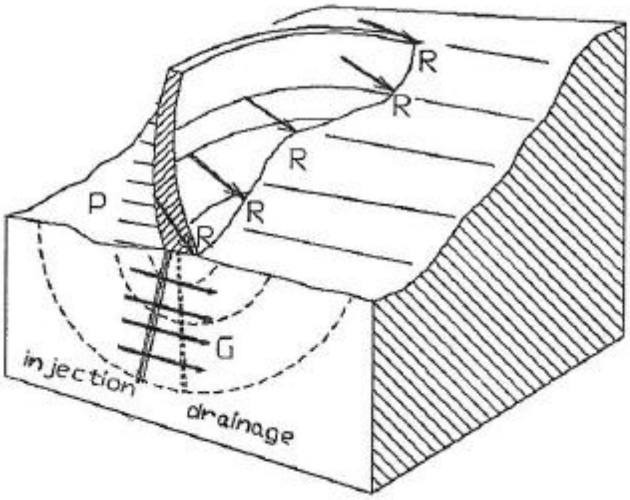


Figure 30-3 – Forces de poussée (R) et de gradient hydraulique (G)

Figure 2 : Forces transferred by the dam in the foundation : Thrust force (R), hydraulic gradient (G), after [6]

1.1.2 Stability assessments in dams foundations

Qualitative mechanism and quantitative analyses of the eventual failure modes of the rock should be performed while evaluating the foundation. A general rock engineering process for dam has the following components :

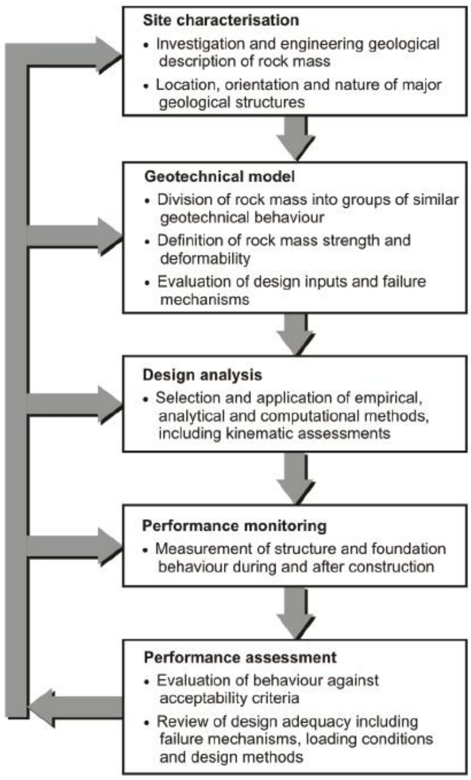


Figure 3 : Components of a general rock engineering process for dams, after [2]

Assessment for rock stability in a dam foundation can be conducted with :

- Stereographic projection

Stereographic projection is a method to plot data in structural geology engineering, and assess eventual risk of failure [appendix]. It can be used to present the rock mass structures at the location of the abutment of the dam, as shown in the figure below.

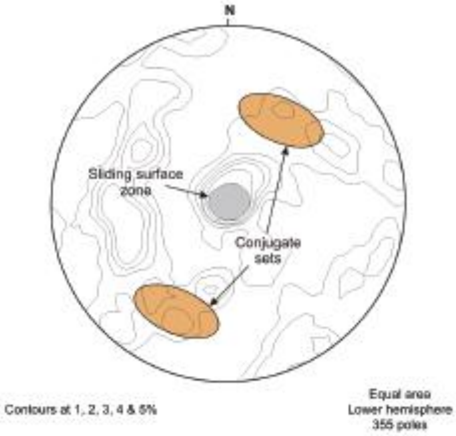


Figure 4 : Stereographic plot of schist defects showing toe area sets, Roxbergh, after [2]

- Method from Londe [6]

The simplified modell from Londe, shown in figure 5, is an elementary bloc of the rock mass foundation, under the dam. It has the geometry of a tetrahedral corner defined by three planes discontinuities P1, P2, P3. It is subject to its weight W, to the dam thrust Q and to the water pore pressure U1, U2 and U3 on the three discontinuity plans. The bloc resists because of friction, cohesion is neglected. Kinematic failure is depending only on geometry. Stability is depending on non exceeding of friction. The advantage of the method is the parametric consideration of uncertain parameters pore pressure and friction. In each single case, abacus can be draw from the geometry to link the unknowns. For a given pore pressure, abacus gives the necessary friction for safety. For a given friction, abacus gives the necessary drainage. For each combination of parameters, a safety factor can be estimated. Once the critical locations are defined, they can be studied with more accuracy in local models, if there is enough data from site. Possible models of rock mechanism are described in the following paragraph 1.1.3..

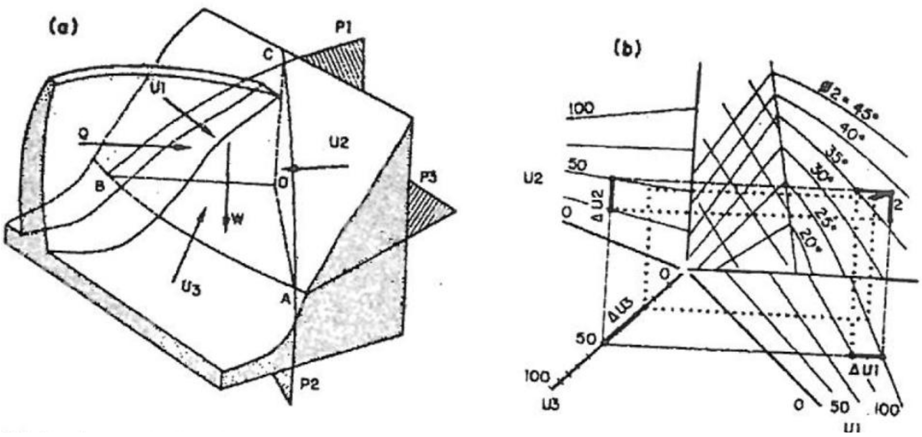


Figure 5 : Method from LONDE : coin rocheux (rock corner) and abacus for results discussion

- Numerical model

Numerical model such as distinct element methods can describe precisely the rock behavior in a discontinued model. In large dam engineering, finite element model are more relevant to obtain stress distribution in the dam structure. Rock is considered as a single homogeneous material, with reduced characteristic values to ensure safety (weakness undetected, fatigue of the rock charged by the structure in long term). Transfer of stress can be estimate at the interface between rock and concrete. The figure below presents the repartition of normal and tangent stress at the interface between concrete and rock foundation. Only compressive strength is plot : tensile stress corresponds to an opening of the heel (calculated of 1 cm for the 87-meter-high Bimont dam), characteristic of the effect of active arch in arch dams.

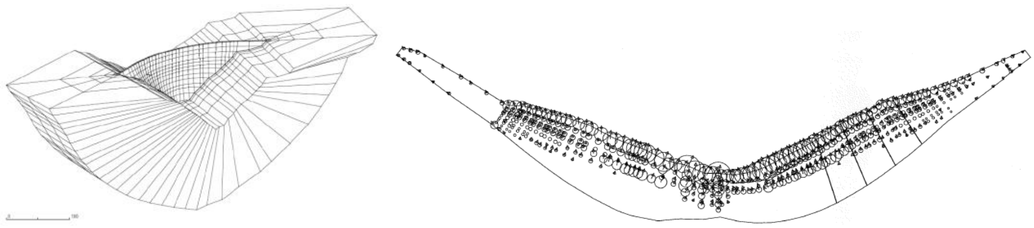


Figure 6 : Numerical finite element model, Bimont dam, France

1.1.3 Scale and model of the rock material

Rock mass, assessed in a scale of 1 to 10 meters presents discontinuities and lower characteristic mechanical values than intact rock material, assessed of sample in a scale of 0.1 meter. Rock mass can be considered either continuum or as combination of blocks separated by discontinuities with kinematic mechanism structurally defined. The choice of a continued or discontinued model is made with regards to the discontinuity pattern in the scale considered.

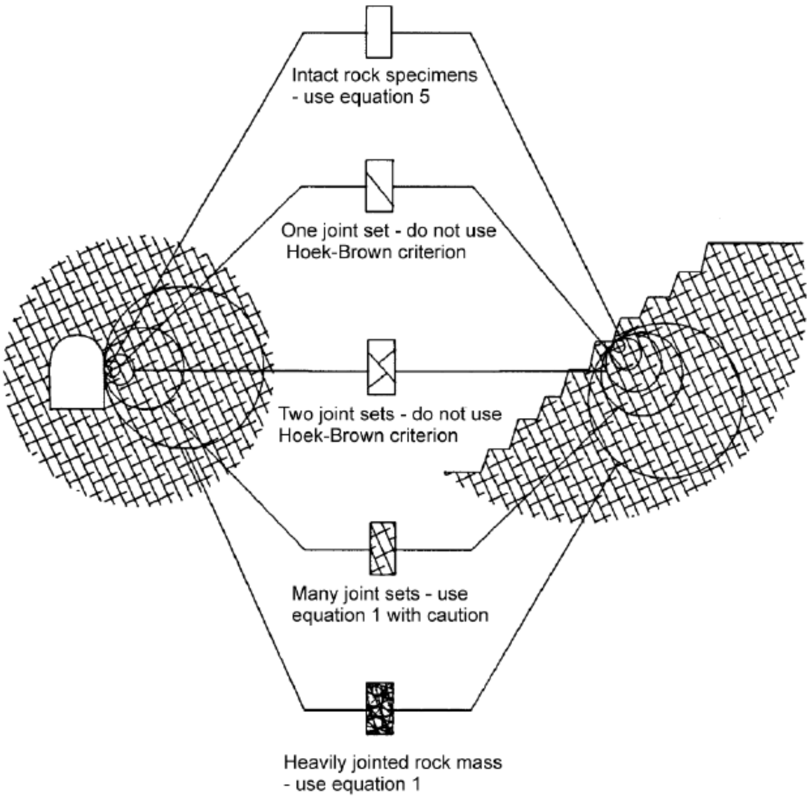


Figure 7 : Transition from intact to heavily jointed rock with regards to scale considered, after [9].

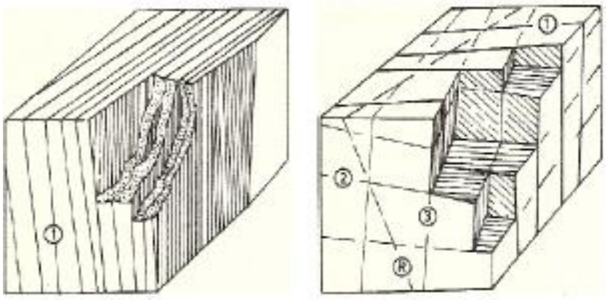


Figure 8 : Pattern of discontinuities and model of rock mass, after [8]

If no direction is privileged by discontinuities (figure 8, drawing on the right), rock mass can be assessed as a continuum material. An anisotropic model would imply rupture mechanism related to blocs kinematics. Analytic model can handle mechanism implying one bloc, such plane sliding or overturning, while numeric method handles more complex geometry.

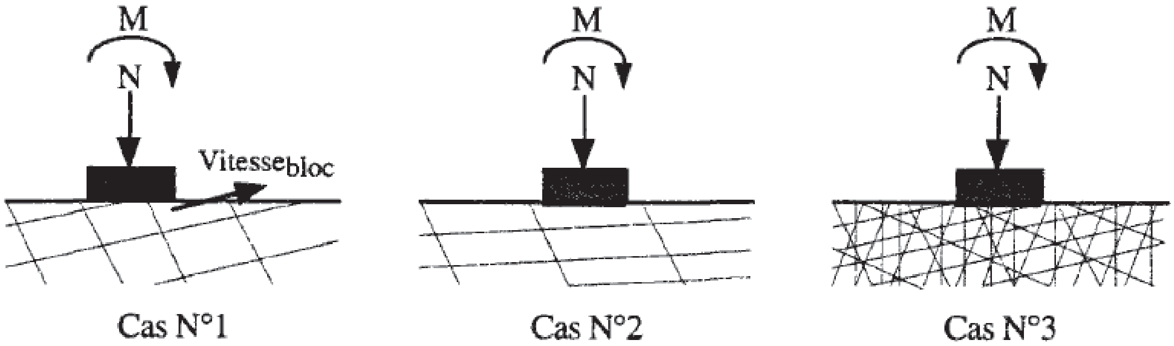


Figure 9: Influence of discontinuities pattern on risk of failure, after [10]

1.1.4 Rock mass strength

Rock mechanics is a young subject in comparison with related engineering sciences such as soil mechanics. Stresses in rock mass knowledge were developed for the design of support in underground excavations. Rock surface is considered unstressed. The geological structures of discontinuities illustrates history of strength in the rock, as illustrated in figure 11. In some particular case, high stress in the rock mass is still present in the rock surface, and could be recognized by the specialists from characteristics signs in the surface.

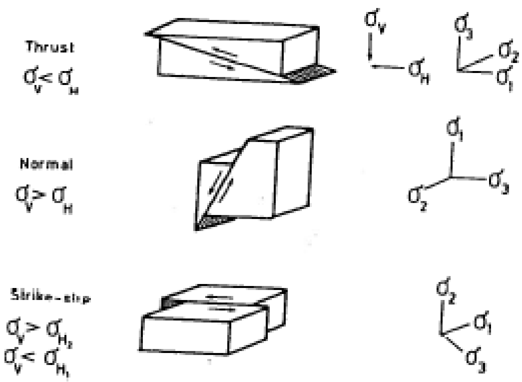


Figure 10 : Faults development in relation to principal stresses, Charlie Li, rock stress and measurements

Mechanism of the unstressed rock mass resistance is related to discontinuity pattern. When it comes to assessment for rock strength for superficial civil foundation, such dams, first step in assessment of the rock mass strength is consideration of the discontinuities pattern, to define if the model is isotropic or anisotropic. This depends on the scale considered (cf. 1.1.3.).

- In isotropic model, global rock mass strength is the major contribution of rock capacity.

Global rock mass strength can be determined from the generalized Hook and Brown failure criterion for jointed rock mass [9] :

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

Formula 1 : Generalized Hook and Brown failure criterion for jointed rock mass

Where input parameters are :
- σ1, σ3 : maximum, minimum effective principal stresses at failure,
- mb : Hoek-Brown constant m for the rock mass,
- s,a : constants which depend upon the rock mass characteristics,
- σ_{ci} : uniaxial compressive strength of the intact rock pieces.

The Mohr Coulomb criteria describes the shear resistance of homogeneous rock, with input parameters cohesion c and friction ø (figure 13). Cohesion is characteristic for rocks, which have shear strength even when there is no normal stress applied, due to their internal structure resistance. These parameters, estimated in laboratory from samples are taken lower to match rock mass conditions. They are also delicate to estimate as their value is function of the normal stress applied. Fair rocks are considered ø=45degree. Cohesion can be far lower for rock mass than samples, and is often taken c=0.

- In anisotropic model, shear strength of discontinuities is predominant.

All rock masses contain discontinuities such as bedding planes, joints, shear zones and faults. At shallow depth, where stresses are low, failure of the intact rock material is minimal and the behavior of the rock mass is controlled by sliding on the discontinuities. Shear strength of discontinuities has been measured by Hencher and Richards (1982) with a shear machine, illustrated below.

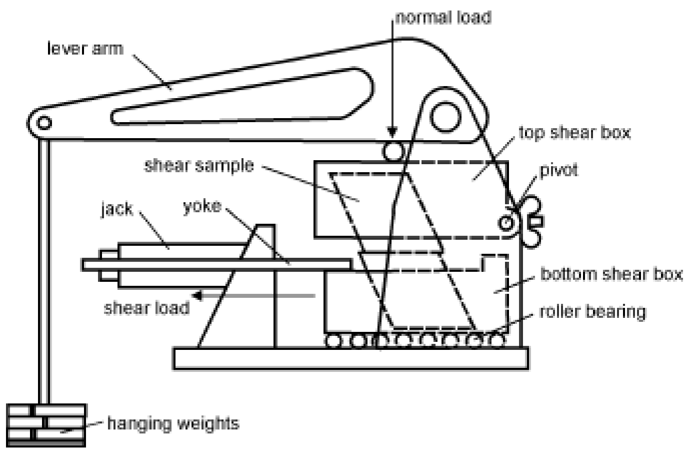


Figure 11 : Test of shear strength of joints with a shear machine, after Hencher and Richards, 1992

The shear resistance of a joint is illustrated by the following diagram. The cohesion c (cohesive strength) has dropped to zero in the case of residual shear strength.

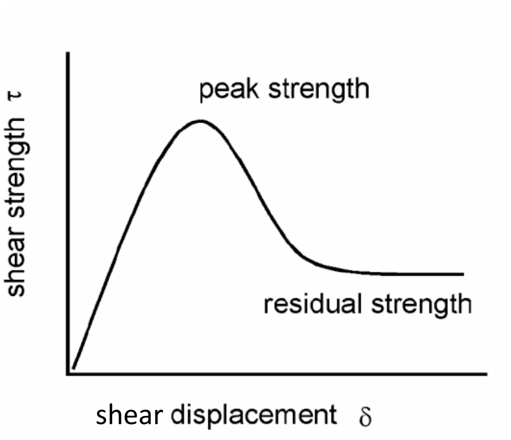


Figure 12 : Shear strength of a discontinuity diagram, after Barton

Peak shear strength can be calculated from normal stress and friction and dilation angle, as illustrated below.

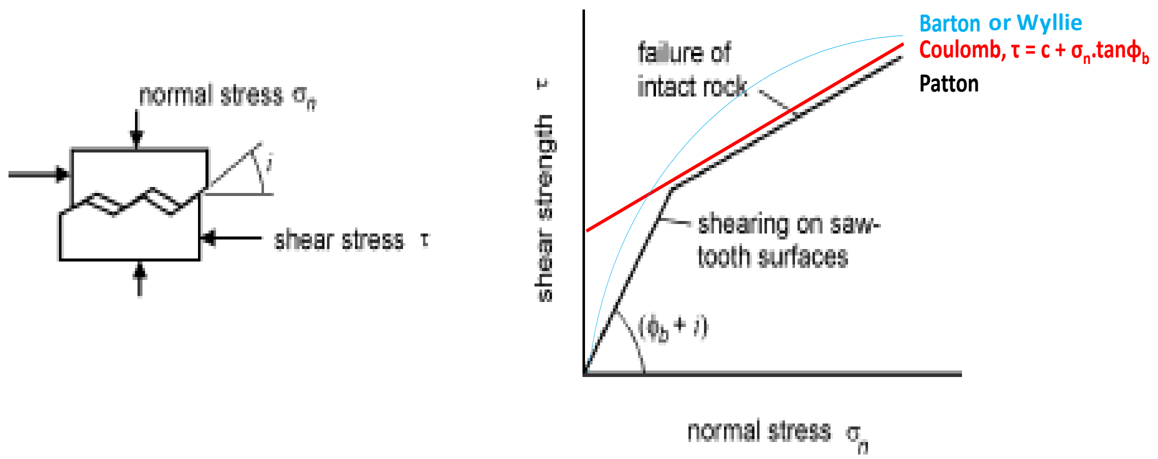


Figure 13 : Dilation and shearing contributions in shear strength of discontinuities

Literature ([13], [15]) proposed shear strength calculations from rock mass parameters with non-linear relation.

Shear strength of discontinuity can be calculated either :

➔ after Barton&Choubey, 1977 [13], in slightly fractured rock mass conditions.

$$\tau = \sigma_n \tan\left(\phi_r + JRC\log_{10}\left(\frac{JCS}{\sigma_n}\right)\right)$$

Formula 2 : Shear strength of discontinuity, Barton and Choubey, 1977

Where input parameters are :

- σn : normal stress applied to the discontinuity
- Dilatance angle : from JRC (joint roughness coefficient, see table in appendix or estimation from Jr in Q-index) and JCS (joint wall compressive strength, see abacus in appendix from hammer Schmidt test)
- φ_r residual friction angle from φ_b (basic friction angle), r (Schmidt rebound number on wet and weathered fracture surfaces) and R (Schmidt rebound number on dry unweathered sawn surfaces)

$$\phi_r = (\phi_b - 20) + 20(r / R)$$

➔ after Wyllie, 1992 [15], in highly fractured isotropic rock mass conditions.

$$\sigma_t = \frac{\sigma_u}{2}[m - (m^2 + 4s)^{0,5}]\frac{1}{F}$$

Formula 3 : Shear strength of discontinuity, Wyllie, 1992

Where input parameters are :

- σ_u : compressive strength, uniaxial test
- m, s : empirical constants on fragility of the rock material and cracking conditions, obtained either from Hoek&Brown table[appendix]or from triaxial test or GSI values [9].

1.2 Rock mass evaluation

1.2.1 Rock mass evaluation issues

Rock mass condition influences highly the intact rock material properties. Evaluation of rock mass quality is necessary in civil engineering but challenging, experience is required. In addition to structural and material consideration, it should be considered as well scale effect and long-term weathering of the rock mass.

1.2.2 Parameters of rock material and discontinuities

- Rock material properties

Testing to obtain rock characteristics can be performed on samples in the range of 0.1 meter in laboratory, or in situ with core drilling (RQD) or geophysical methods. The ISRM suggested methods [appendix] are an international reference.

	Fair rock or hard chalk	Weathered or fractured rock
Pressiometric modulus EM (Mpa)	>100	50 to 100
Limit pressure pi (Mpa)	>5	2,5 to 5
Compressive strength (Mpa)	>10	1 to 10
Velocity of shear wave (m/s)	>800	300 to 800
Velocity of longitudinal wave (m/s)	>2500	400 to 2500

Table 1 : Common values for principal parameters in rock identifications (NFP 06-013), after [26]

Classe de sol	Module de Young E (MPa)	Coefficient de Poisson	Cohésion (MPa)	Angle de frottement φ (°)	Angle de dilatance ψ (°)	Poids volumique γd (kN/m³)
Sables, Graves	3000-20000	0.21-0.3	0	30-40	0-10	18
Limons	2000-15000	0.21-0.3	0.002-0.005	15-25	0	17
Argiles	1000-10000	0.21-0.3	0.005-0.05	5-20	0	16
Pierre calcaire	50000	0.21-0.3	30	25	0	24
Craie	100-12000	0.21-0.3	0.1-2	10-45	0-15	13-22
Grès	3000-80000	0.21-0.3	1-50	20-50	0-20	19-27
Granite	10000-90000	0.21-0.3	10-110	30-60	0-30	26-31
Schiste	5000-80000	0.21-0.3	2-15	50	20	24-29

Table 2 : Physical characteristics for different rock types, After Solem, Switzerland. Tunnel, didacticiel

- Rock mass discontinuities are recognized to be determinant in rock behavior.

Numerous parameters can describe these discontinuities. Parameters considered relevant with the objective of the thesis are :

- Spacing of discontinuities,

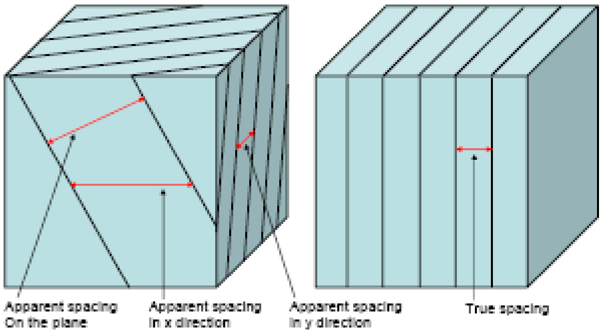


Figure 14 : Apparent and true spacing of discontinuities, after [8]

- Orientation of discontinuities,
- Conditions of discontinuities (roughness, filling, opening).
- Rock quality designation index RQD (Deere et al 1967), defined as the percentage of intact core pieces longer than 100 mm in the total length of core.

1.2.3 Classifications of rock mass : Q, RMR and GSI

Recognized rock mass classifications such Q, RMR and GSI already compute rock mass parameters to obtain a global rating of its quality. Such classifications were first develop with regards to underground engineering. One objective of the present work was to assess the relevance of such classifications in dam foundation engineering. Full scale tests of 50 fully grouted rock bolts were conducted, in order to find a relation between measured maximal tensile capacity of the bolts and rock mass rating.

- Q

Tunneling quality index Q(Barton et al, 1974) compute rock mass characteristics in order to estimate tunnel support requirements. The numerical value of the index Q varies on a logarithmic scale from 0.001 to a maximum of 1.000 and is defined by:

$$Q=\frac{RQD}{J_n}\times\frac{J_r}{J_a}\times\frac{J_w}{SRF}$$

Formula 4 : Q-index

where :

- *RQD* is the Rock Quality Designation
- *Jn* is the joint set number
- *Jr* is the joint roughness number
- *Ja* is the joint alteration number
- *Jw* is the joint water reduction factor
- *SRF* is the stress reduction factor

The value of the classification GSI, presented after, can be estimated from the Q-value :

$$GSI = 9\ln Q'+44,\ Q'=\frac{RQDJr}{JnJa},\ \text{assuming } SRF=Jw=1$$

Formula 5 : Estimation of GSI from Q-index

- RMR

Rock Mass Rating system RMR (Bieniawski, 1976 and revision in 1989) deals with estimating the strength of rock masses. RMR takes values between 0 and 100, with an accuracy of plus or less 5 points. It is the summation of ratings for the six following parameters :

- Uni-axial compressive strength of rock material.
- Rock Quality Designation (*RQD*).
- Spacing of discontinuities.
- Condition of discontinuities.
- Groundwater conditions.
- Orientation of discontinuities.

The value of GSI, presented after can be approximate with RMR_{89} where water conditions are equal to 15 and no corrections are made with regard to discontinuities.

$$GSI=RMR_{89}-5$$

Formula 6 : Estimation of GSI from RMR-rating

The value of the deformation modulus of the rock mass can be estimated from the RMR value (Serafim & Pereira, 1983, from deformations in dams) :

$$20 < RMR < 85 ;E_m = 10^{(RMR-10)/40} \text{ (GPa)}$$

Formula 7 : Estimation of deformation modulus of the rock mass from RMR

- GSI

Geological Strength Index GSI (Hoek&Brown 1998 adjust from Hoek 1994) is a prolongation of the RMR rating adapted for poor rock mass conditions (RMR<20). GSI takes value from 0 to 90. High precision in the value of GSI rating is illusory, GSI can be reasonably defined with a precision of about plus or less 5.

Value can be estimated directly from a table with input parameters :

- Structure (interblocking of rock pieces)
- Surface conditions quality

GSI value is estimated without any measurements. Reservations were also found in the literature on the formulas linking GSI to Q and RMR. GSI is considered in this thesis as an eventual basis to complete (with measured value such as distance between cracks)for propostion of a new rock classification, if no satisfactory relationships were found between tensile capacity of the rock bolt with RMR or Q.

Empirical parameters m and s are linked to the GSI value. They can be determined from triaxial test or estimated from GSI value. Parameters m and s are used in the calculation of rock mass strength (cf 1.1.4.).

Tables to estimate Q, RMR and GSI are presented in appendix.

1.2.4 Scale for rating

The influence of scale in rock mass rating was highlighted in 1.1.3. The protocol for the performed test program was to assess first the rock mass in the direct vicinity of the bolt (characteristic length 1 meter : local break for one anchor), and to correct the value by considering a larger scale (characteristic length 10 meter : global break of the surrounding rock mass).

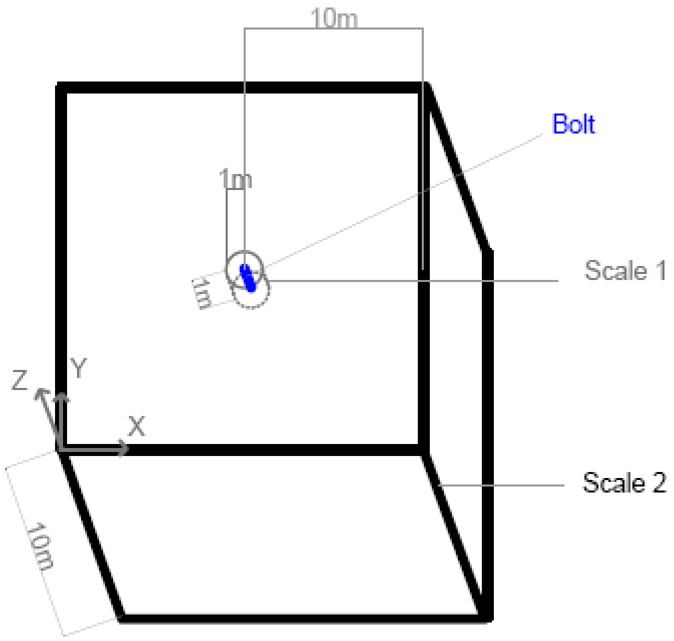


Figure 15 : Scale for rating of rock mass around bolts in full scale test program

1.3 Use of rock bolts in dam reinforcement

1.3.1 Active anchorage and passive rock bolts

A rock bolt is a system that transfers tensile force from the structure into the rock foundations. It appeared first in mining engineering in 1949, and then extended to civil engineering, and first fully grouted anchors were used in 1960. An anchor is a superficial foundation, based on fair rock surface. It is considered permanent if installed for more than 2 years. An anchor can be either pre-stressed anchorage or passive rock bolts.
Unstrained rock bolt is grouted in the rock without tensioning. Load in the support element is mobilized by internal deformation of the rock mass, and such a bolt works with tension and shear. Fully grouted steel rock bolts are protected against corrosion by the grouting, the risks of corrosion is linked to the quality of concrete used. Passive rock bolts can't easily be controlled in operation. Ultrasound can be used exceptionally for detection of corrosion.
Active anchor are support elements installed in the rock mass which imposes a predetermined load to the rock mass at the time of installation. It works only with tensile strength. Pre-stressed rock anchor can take higher loads, transmitted deeper in more solid rock conditions. For active anchorage, load can be easily and steadily controlled with force-gauges system glötzel.

As they are not working in same physical mechanism, it's normally not recommended to use active and passive anchors in the same structures. In special cases, calculations should justify such solutions.

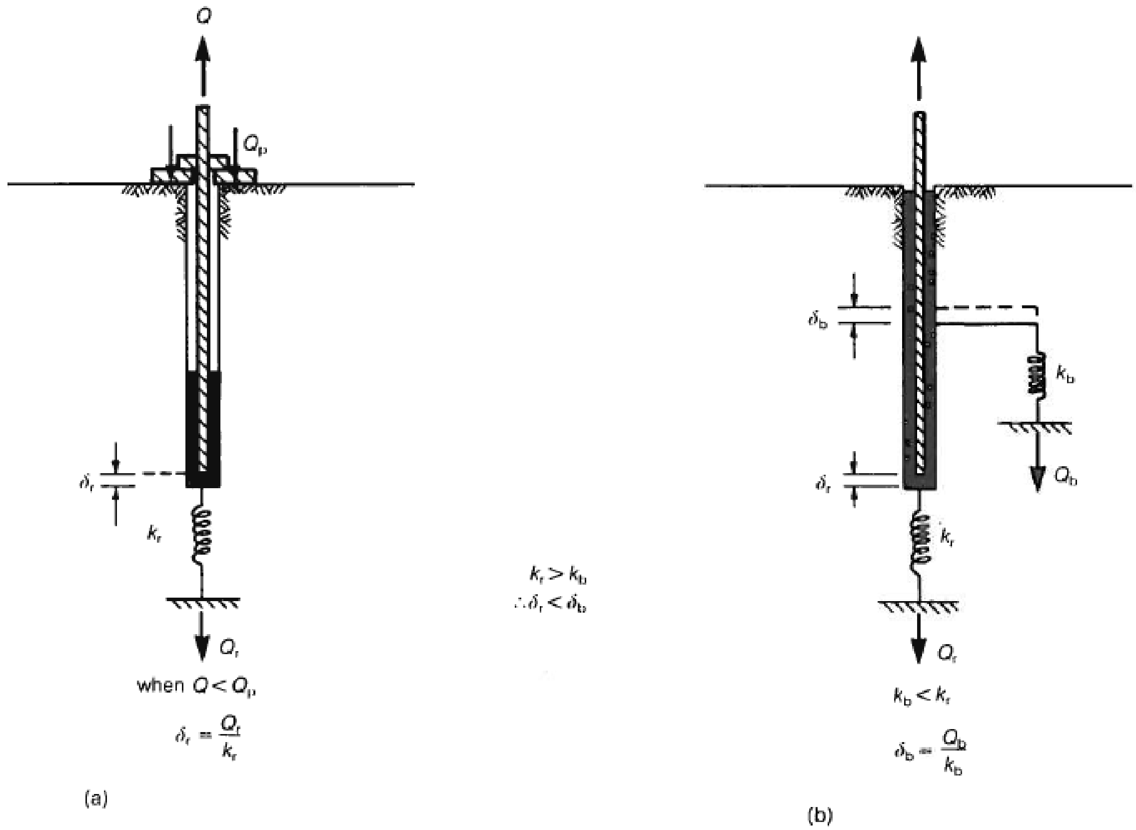


Figure 16 : Simplified model of support mechanism of (a) prestressed and (b) passive anchors, after Wyllie

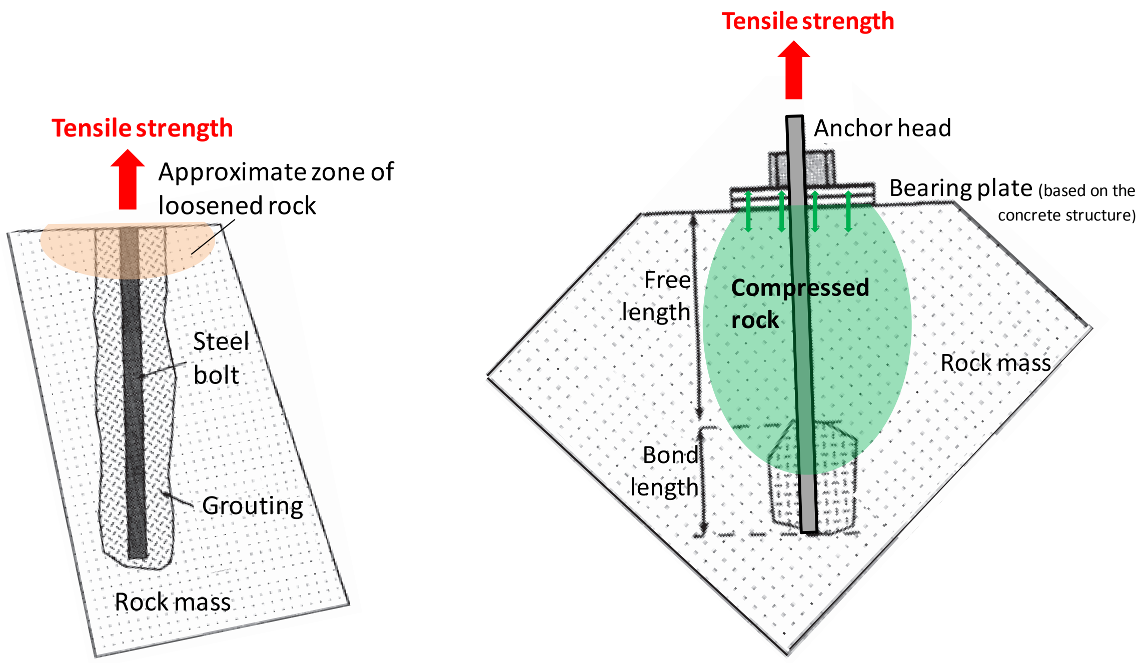


Figure 17 : (a) passive rock bolts, (b) pre-stressed anchor

Passive rock bolts are used in Norway for small concrete dams (H<7meters) to ensure sufficient stability against overturning, especially with regards to ice load in winter, see figure 19. They are designed (NVE) to support a maximal tensile stress of 180 MPa (88 kN), to avoid important deformation. Pre-stressed anchors are used in large dams as reinforcement to improve capacity, it was performed in rehabilitations in the French Alpes by EDF, and up to a capacity of 4500kN per anchor for Eder dam (Germany, 1992). Pre-stressed anchor are preferred for large dams with higher loads to avoid deformations, and eventual tension, unfavorable for the upstream grout curtain, see figure .

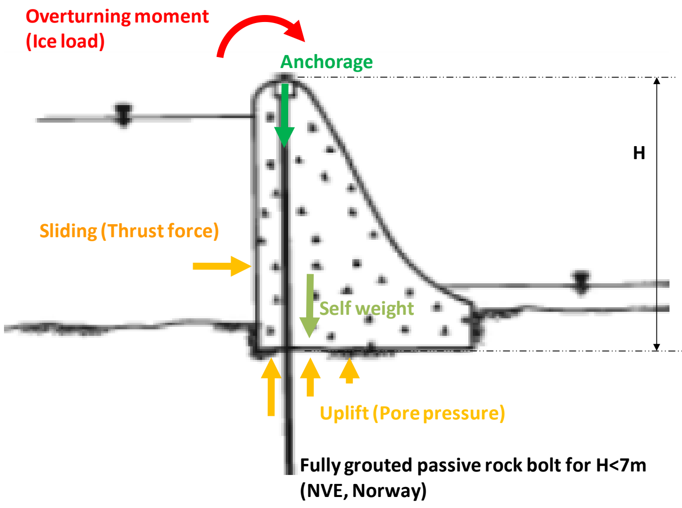


Figure 18 : Rock anchors and loadings in a concrete gravity dam

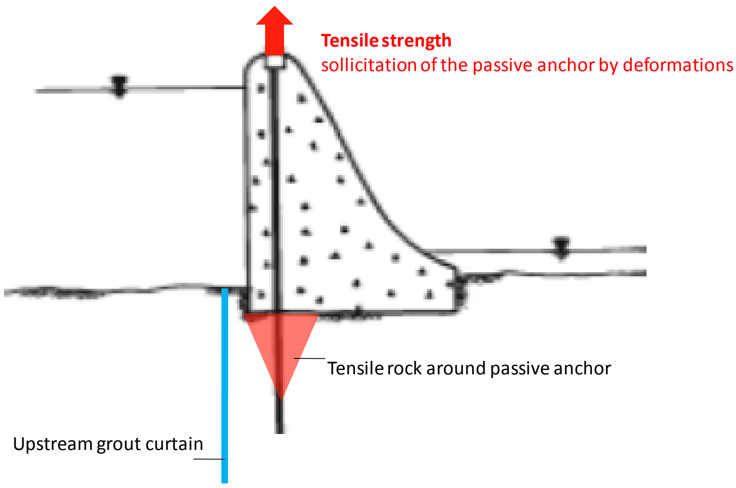


Figure 19 : Operation of a passive anchor in a gravity dam

The design of passive rock bolts in Norway (NVE) is representative of the design worldwide. The necessary grouted length for passive rock bolts is determined with regards to failure from steel, rock or grouting. The steel capacity (400MPa) is far higher than expected loading of the bolts (limited to 180 MPa to avoid important deformation), and therefore not consider critical in the design. The necessary grouted length is calculated from rock-grouting and grouting-steel bond strength to resist a tensile stress equal to the steel yield criterion 400MPa. The required position of the center of the grouted length is then determinate with regards to the weight of the rock surrounding the bolts. The length D is calculated to obtain a cone of rock with sufficient weight to resist a tensile stress of 180 MPa. The position of the center of the grouted length L is then placed at depth D. Parameters of rock-grouting bonding strength and self weight of rock depends of type of rocks and are defined by table from NVE [appendix]. The cone has angle 45° for fair rock conditions. With steel and mortar characteristics used in Norway today, and depending on rock types [table from NVE], calculations for characteristic values give a length of required grouted length L between 1,43m and 5,56m, and a position of the center of length D between 1,44m and.1,52m D is always superior than L/2. Finally, required bolts length in the rock is D+L/2, which varies from 2,16 m to 4,30m. Calculations are detailed further in appendix.

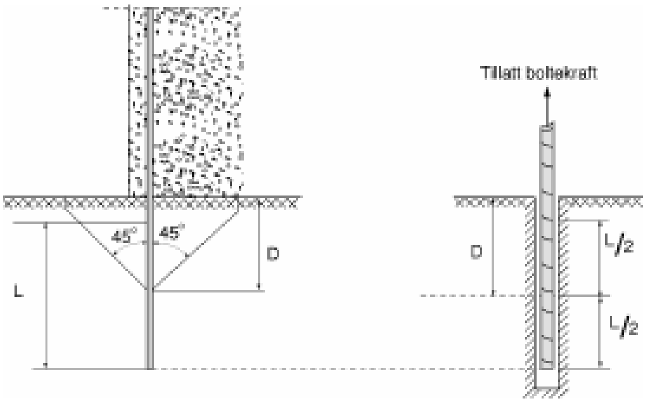


Figure 20 : Rock bolts, rock cone to determine hanging weight, after [27].

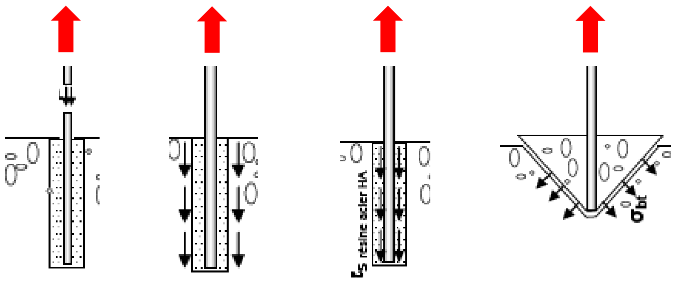
Design of pre-stressed anchor is under steady research in North America (Post Tensioned Institute). "With all tensioned structural-anchor systems, a major consideration is determining how deep to install the anchors. An anchor system that is too shallow may cause tension and cracking to occur along potential failure planes in the foundation, and a system too deep is uneconomical. PTI recommends normal bond length not less than 3.0m (10ft) for bars and 4.5m (15ft) for strand. Bond lengths greater than 10m (35ft) are normally not used. PTI recommends free stressing lengths to be at least 3.0m (10ft) for bar tendons and 4.5m (15ft) for strand tendons. Center to- center spacings between anchors shall be at least 1.5m (5ft) unless unusual circumstances dictate. The fixed end (dead end) anchorages should be staggered." References can also be found in Europe. Rehabilitation of Pontabouland dam (15m, France) used pre-stressed anchor of total length 6 meters for anchor capacity of 1700 kN. Rehabilitation of Eder dam (47m, Germany) used a free length of 20m and a bond length of 10m for extremely high anchor capacity of 4500 kN.

Figure 21 : Installation of anchor, Eder dam, after [1]

1.3.2 Research on passive rock bolts

Failure of rock bolts are either due to failure from steel, grout or rock (figure 22). Actual design is still based on development from Littlejohn and Bruce 1977 [14]. Research in fully grouted passive rock bolt design is still going on today, with interest from the hydropower industry in Norway and transfer of horizontal load from civil engineering structures in France. Steel is today well documented. Steel capacity is considered at the yield criterion in bolt capacity, which ensure additional safety. Grouting has been and is still subject to great improvements. Failure from grouting can be either grout failure or bond failure between grout and steel or between

grout and rock. The technologies vary : mortar (Norway), cement (France and Sweden), resin (more recent). Grout performance is depending in equal proportion to the characteristic of the product and the conditions of its installation. Grouting research is still conducted today. The bond strength value between rock and grout are commonly few documented and underestimate, exception made to Sweden who use far higher values [appendix].
The rock resistance against tension is the weight of the cone of rock around the bolt according to conservative design. Researches were conducted to precise bolt behavior under tensile and shear solicitation where rock was modeled by concrete with different modulus. Stjern realized in particular research on bolt and grouting similar to this thesis in a test rig in a lab, both in tensile and shear solicitations (figure 23). Results will be compared with results from the thesis in 2.1.3. But no comprehensive program of research have been found in rock conditions. Energi Norge, involved in this domain of research, will likely publish soon a report which inventories what is currently going on or about to start on research on passive rock bolts.



(a)steel capacity, (b)rock-mortar bond, (c) mortar-steel bond, (d)rock capacity.

Figure 22 : Drawing of 4 failure modes of a rock bolt

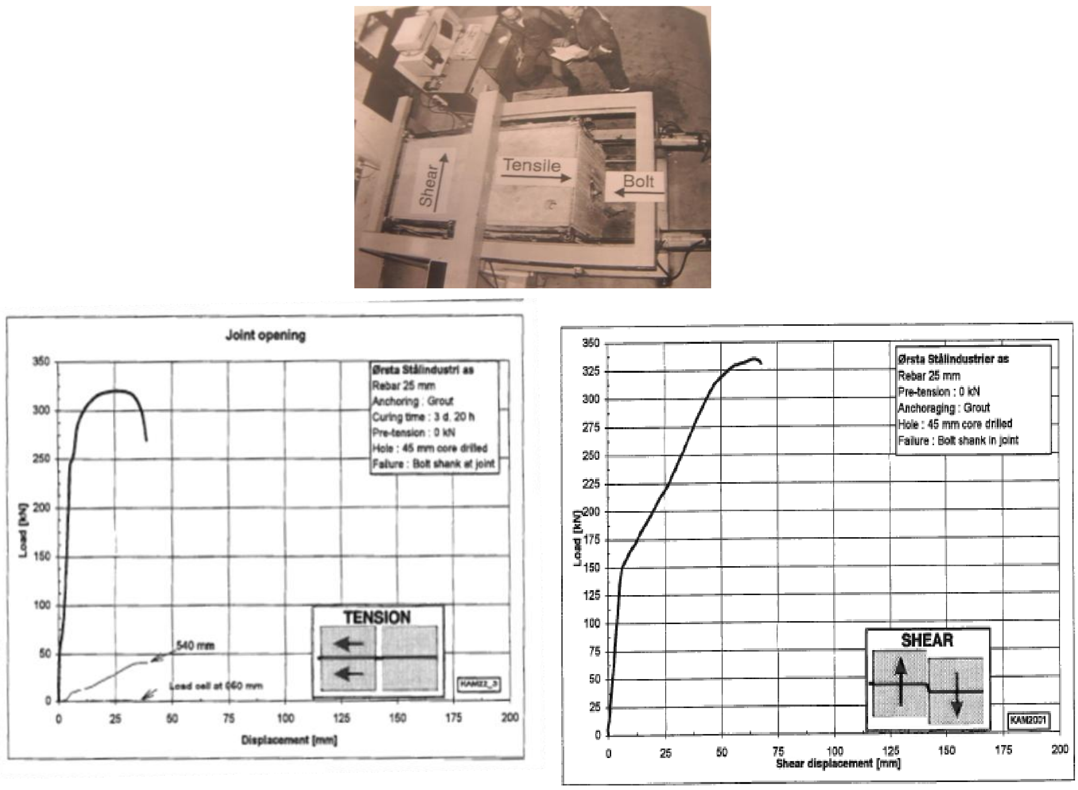


Figure 23 : Tensile and shear test of ø25mm bolts in a test rig, after Stjern

1.3.3 Design of passive rock bolts with regards to rock mass

The present research considers possible improvements of the current design method for passive rock bolts. Results from Lars K. Neby full scale tensile test program measures capacity far higher than the one expected with the design method from today. From these results and literature [15], hypothesis of the work is that shear strength of the rock discontinuities is the major contribution of tensile capacity of fully grouted passive anchors.

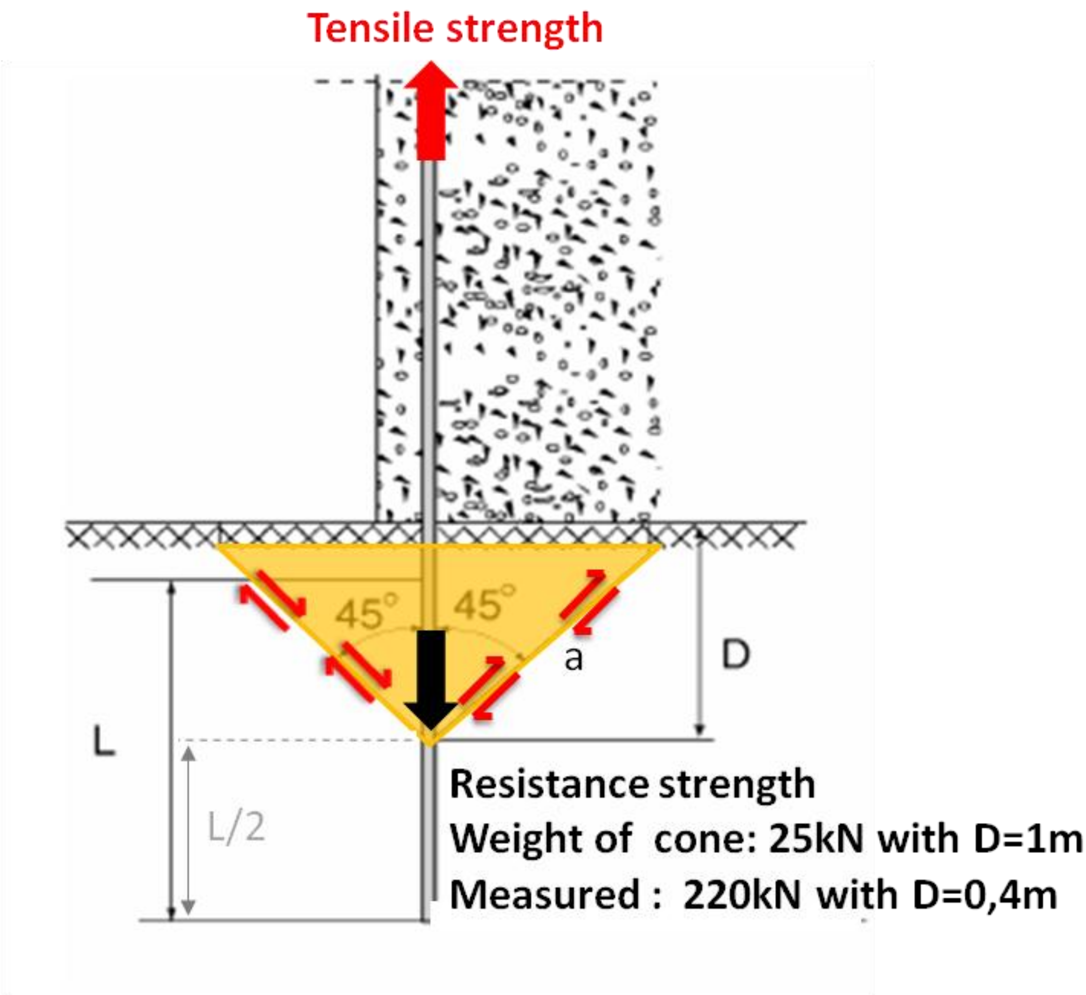


Figure 24 : Predominant contribution of rock shear strength in fully grouted rock bolt tensile capacity

Capacity can be then calculated as the sum of shear on the surface of the cone around the bolt.
$Fr=\sigma t.A$
$Fr=\sigma t.\pi.r.a$

$$Fr=\sigma t.\pi.D^2\sqrt{2}$$

Formula 8 : Shear strength resistance of the rock against tensile loading of the rock bolt

σt calculated from Wyllie or Barton(1.1.4.)

boring	Core drill, diameter 45mm
Steel bolts	Grouted length from 0,1 to 0,99 meter Diameter 25mm Yield limit 500MPa
Pull out test	Results read on a dynamometer Rock mass failure, excepted for 9 bolts see table below Capacity from 10 to 220 kN

Table 3 : characteristics of testing from Lars Kristian Neby

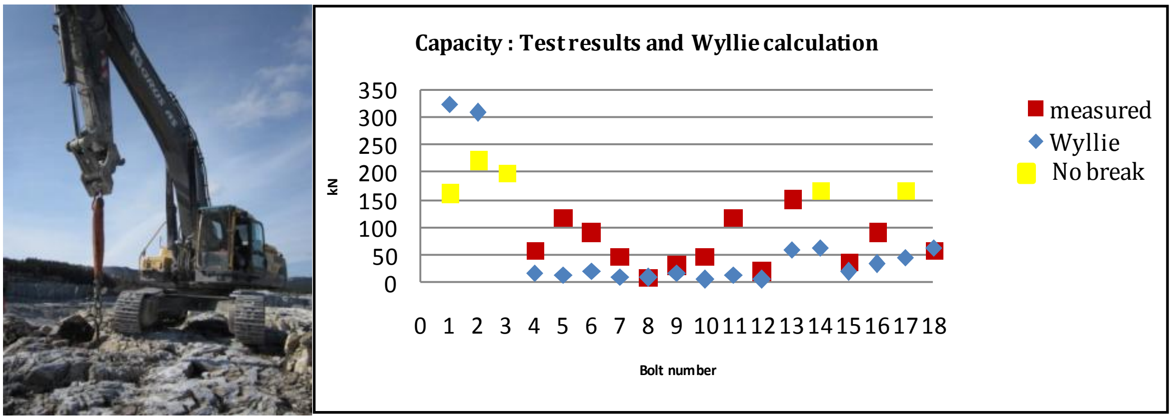


Figure 25 : Pull out testing, measured vs calculate capacity from Wyllie

The shear strength of discontinuities σt was calculated from Wyllie and not Barton because the conditions were considered highly fractured (bad rock according to RMR classes).

Bolt nr	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
No break	x	x	x											x			x	
Loose block								x				x						
Mortar break															x	x		
Relevant bolts				x	x	x	x		x	x	x		x					x
Length (m)	0,4	0,4	0,5	0,6	0,5	0,6	0,5	0,4	0,8	0,3	0,5	0,3	1,0	1,0	0,1	0,1	0,9	1,0
Capacity (kN)	160	220	200	60	120	90	50	10	30	50	120	20	150	170	40	90	170	60
Weight(kN)	2	2	4	5	3	6	2	2	16	1	4	1	24	26	0	0	17	26
Shear strength(kN)	323	307	588	14	10	16	7	7	4	4	11	4	38	40	20	34	30	40

Table 4 : results of 18 bolts tested by Lars Kristian Neby

Calculation of maximal tensile capacity of bolts with shear strength consideration was far more accurate than calculation with only regards to weight of cone (table 4). Limit is : parameters estimation is complex and small difference in evaluation of parameters is highly amplified in capacity calculated. Two empirical formals developed from a further analysis of Lars K. Neby results are proposed to link directly maximal tensile capacity of the bolts to simple assessment from the rock mass. The formals are calibrated to give a capacity inferior or equal to the measured capacity (green curve in the following diagrams).

The first formal is linear with input parameter RMRrqd5, which is equal to RMR1989 rating with RQD5 instead of RQD. RQD5 is defined as the sum of the bits longer than 5cm over the total length of the core. The strength F [kN] for a bolt of length in rock L[m] is proposed as :

$$F = L.\,(12.RMR_{RQD5}\text{-}505)$$

Formula 9 : Tensile resistance from RMR_{RQD5} and length of bolt

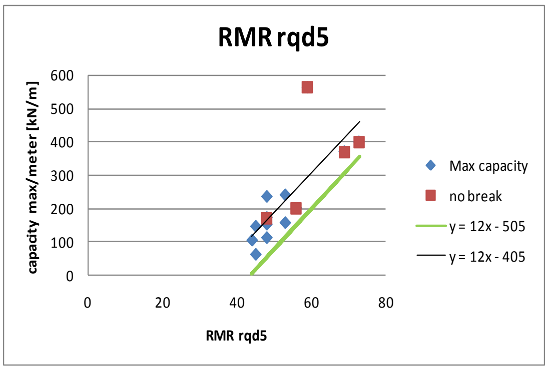


Figure 26 : Relation between maximal measured tensile capacity of the bolt and RMR_{RQD5} rock mass rating, after the 18 bolts tested by Lars K. Neby, 2011

The second formal is polynomial of dgree 2 with input parameter GSI-value derivated from Q-value by a logarithmic expression from the literature. The strength F [kN] for a bolt of length in rock L[m] is :

F = 0,557 GSI2 -46,6 GSI + 1114,

with GSI= 9 ln(Q')+40 where Q' = Q with Jw=SRF=1.

Formula 10 : Tensile resistance from GSI_Q and length of bolt

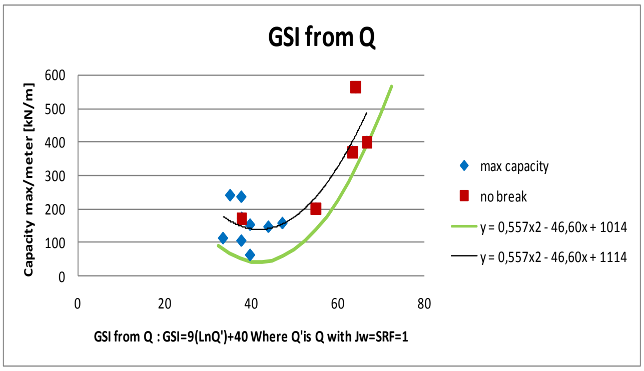


Figure 27 : Relation between maximal measured tensile capacity of the bolt and Qrock mass rating, after the 18 bolts tested by Lars K. Neby, 2011

2 PASSIVE ROCK BOLT DESIGN IN SMALL CONCRETE DAMS

2.1 Full scale test

2.1.1 Presentation

Full scale tensile test of fully grouted passive rock bolts in a rock mass of various conditions is performed in order to improve knowledge on rock mechanical behavior in fully grouted passive rock bolt technology.

The test performed in 2011 by Lars K. Neby assessed bolt tensile capacity was higher than expected from the design today (200kN vs 20kN). First conclusions were obtained on critical length (0,4m) of bolts and rock mass conditions (RMR>40). 18 bolts were tested. Rock break was the failure mode for the majority. Mortar collapse for 2 bolts shorter than 0,15m, and 5 bolts were loaded higher than 160kN without any break (table 4).

In order to validate further in a statistical approach results from Lars K. Neby, 50 bolts were tested with the same protocol. Continual measurement of deformation and last during test to obtain load – deformation diagram was added.

The characteristics of bolts and grout are representative of what is used today in Norway (NVE). Steel bolts (Vik Ørsta) have diameter 25mm and yield limit 500MPa. A lifting consol was also design to fix at the head of the bolt and perform loading. Mortar NONSET 50FF adptated to low temperature conditions, was used. Bolts were grouted with water of 25°C and air temperature 2°C. Curing time was 13 days in negative temperature but few rain or snow. Tests are performed in the limestone rock quarry Verdalskalk (Verdal). Properties tested in laboratory assessed an intact rock of high properties (compressive strength of 90MPa), but rock mass was found in a wide range of quality (RMR 40 to RMR 80).Bolts were spread over two locations. One with high quality rock mass (RMR up to 90) and one with poor quality rock mass (RMR up to 40, where Lars K. Neby did tests in 2011). Drilling was performed both with hammer drill (40 holes, diameter 50mm) and core drill (10 holes, diameter 45mm). Tensile testing is performed with an excavator up to a capacity of 26t (260kN), corresponding to the yield limit of steel (500Mpa).

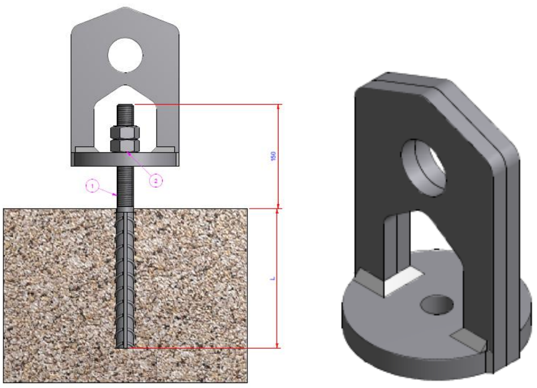


Figure 28 : Design for lifting console for bolts, Vik Ørsta

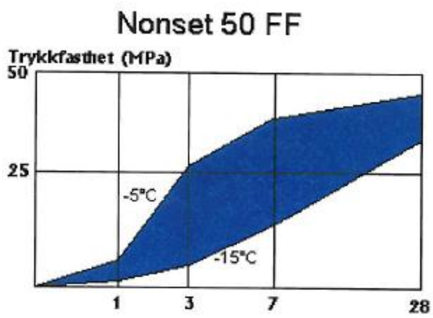


Figure 29 : Mortar Nonset50FF

Figure 30 : Location of Verdalskalk rock quarry

Figure 31 : Different rock mass quality in Verdalskalk rock quarry

Figure 32 : Hammer drilling

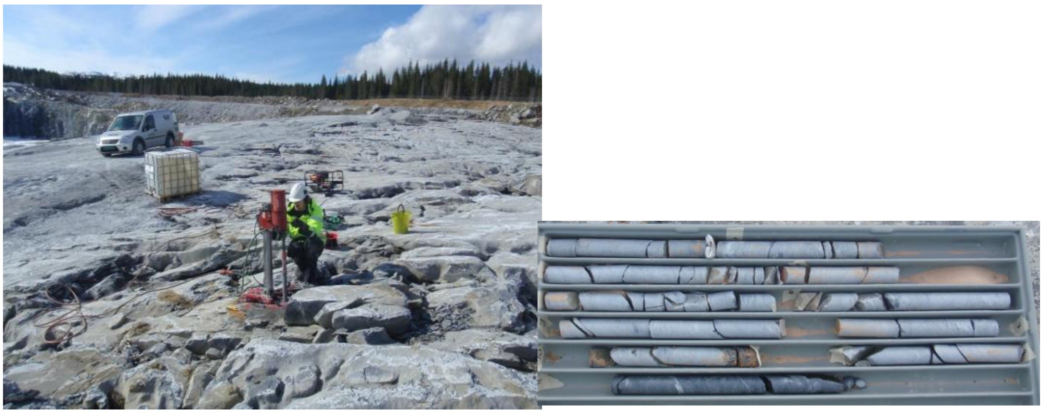


Figure 33 : Core drilling

Figure 34 : Excavator CAT 450, Verdalskalk

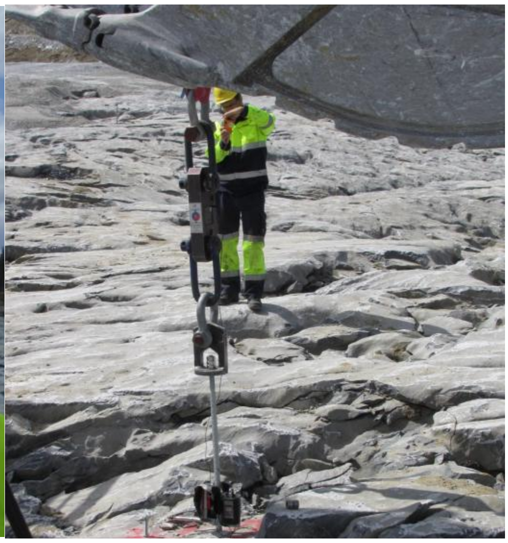


Figure 35 : Bolt ready for test

2.1.2 *Measurements in the test program*

2.1.2.1 *Rock mass rating*

Assessment of the rock mass was made from :

- Visual inspection for Q, RMR and GSI rating
- ISRM lab test from last year in the same rock quarry, see appendix
- Inspection inside the holes by core or video-camera to determine RQD

Figure 36 : drill core : some cracks are due to coring

Figure 37 : clean performing of hammer drilling

Figure 38 : 0,1 m long crushed rock zone under the fair rock surface at the bottom of the bolt nr13

2.1.2.2 Monitoring of tests

- Load and deformation measurement

Figure 39 : System for load and deformation measurement, recorded by a video camera

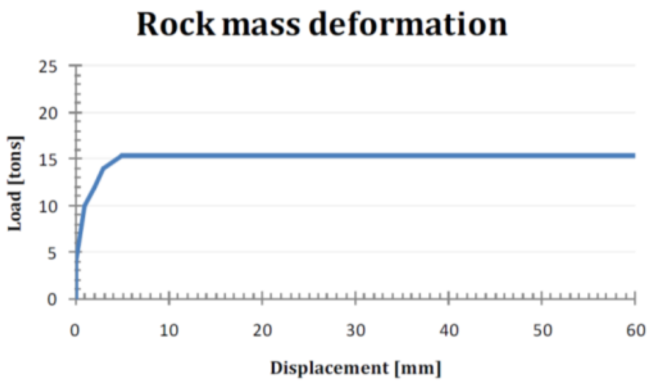


Figure 40 : Load-Deformation diagram obtained for bolt nr 9

- Video and picturesof testing

Figure 41 : 2 video cameras to record large rock behavior around bolt and strength-deformation logging devices

2.1.3 Results of testing

2.1.3.1 Analysis with regard to master test program

Experiments : Full scale tensile test of 68 fully mortar grouted steel bolts 500 MPa ø25mm in ø45mm drills from 0,1 to 1 meter length in a rock quarry of limestone (Verdalskalk) with high intact rock properties (uniaxial compressive strength 90MPa) and different rock mass conditions from very good (RMR=90) to very poor (RMR=20).

Context : Lars K. Neby concluded in 2011 from testing 18 bolts that extremely poor rock mass conditions (RMR=30) and extremely short bolts (L=0,5 meter) have average capacity of 200MPa, 100 times more than conservative design. Strength in rock surface was shown to be the major parameters of maximal tensile capacity of fully grouted bolts. Conservative design do not consider own rock strength, as it's delicate to estimate (local effect and numerous parameters).

Objective : Develop an applicable improved design method related to an easy rock mass rating. Theory developed in shear strength of discontinuities are relatively complex with regards to input parameters and applied in limited conditions.

Initial work
→ Prediction of bolt maximal tensile capacity from rock mass quality :
Three formals were proposed from RMR, Q and Wyllie (with input parameters calculated from RMR) after a further analysis from Lars K. Neby results.

Results from test
All major conclusions from test are listed below. The author emphasize the point number
→Global analysis :
The results of testing confirm relations developed from LKN results in a more statistical approach. Results of testing include 18 bolts without break or with steel break for loading between 19 tons and 26 tons, 21 bolts with break of rock mass from 5 tons to 23 tons and 11 bolts with break of mortar from 5 to 17 tons (mortar was not dry, curing time 13 days was probably not enough with regards to the climatic conditions).

Result	Number of bolts	Comments
No break	18	between 19 and 26 tons
Break of rock mass	21	7 above 19 tons 10 between 8,95 and 18 tons 4 under 4,85 tons (extremely poor rock condition, crushed or loose block)
Break of mortar	11	9 between 4,85 and 11 tons 2 equal to 17 tons

Table 5 : Repartition of bolts with regards to failure mode

1) The maximal tensile capacity measured is comparible than the one obtained from rig test in concrete bloc (Stjern, see 1.3.2.) and in empirical table from swiss directives presented below. Wyllie also write in his book that the maximal tensile capacity was actually from 7 to 54 times higher than the weight of cone of rock considered around the bolt.

Longueur de l'ancrage (m)	Diamètre du forage (mm)	Effort admissible de traction (kN)
1.0	30	47
1.5	45	212
2.0	45	283

Table 6 : Tensile capacity with regards to diameter of drilling and anchor length in few cracked rocks, Directives suisses, after [26].

2) The problematic with mortar conditions is not entirely solved. Control of grouting conditions could be subject to further research.

→ Test serie to find critical depth where rock bolts will yield regardless of rock mass quality :

1) Break is certain for bolts shorter than the critical length 0,4 meter in any rock mass conditions.

Justification

Critical length is higher than 0,15 meter (mortar-rock bond failure)

Critical length is higher than 0,3 meter and lower than 1 meter (At same location and rock mass conditions, bolt 35 of length 0,3 meter breaks from rock break, but bolt 14_{2011} of length 1 meter doesn't)

Critical length is lower than 0,4 meter (5 bolts of length 0,4 meter didn't break in similar fair rock mass conditions)

2) Bolts will not break if they are higher than 1 meter (?) and with conditions from the rock mass such as RMR>50 and lateral support is not defavorable and loose block geometry is not detected.

This affirmation is of major interest for rock bolt design. The author is prudent on the value, and suggested to conduct further research especially in other rock quarry to confirm it. Proposed test is bolts between 1 or 2 meters in different rock mass conditions to confirm minimal length where bolts will not break in minimal rock requirement characteristic for dams foundations : RMR>40 and lateral support is not defavorable and loose block geometry is not detected. Then a second test program should assess the validity of that length 1) in horizontal loading (same protocol, excavator can load with an angle), and 2) in disfavorable water conditions.

→ Other tests :

3) Series with as similar conditions as possible due to rock mass parameters and grouted depth.
   Bolts in similar locations conditions obtained same maximal tensile capacity, repeatability is therefore validated.
4) Flushed and unflushed test serie was not performed as holes were perfectly clean both after hammer drilling and core drilling. It was yet observed for some holes remaining loose rocks pieces at the bottom, or huge faults at the bottom of the hole. Yet such defaults were correctly handled with grouting. The mortar indeed consolidates the rock discontinuities.
5) Friction testing : 2 bolts were "grouted" with gravel in high quality rock mass. It was easily and steadily pulled out with a strength inferior to 1 ton.
6) A test to assess difference between core drilled holes and hammer drilled holes was also performed in same rock conditions. No differences were noticed.

→ Mechanism of break in the rock mass

- Load-deformation diagram. From the 18 break from the rock mass in the 2012 Verdalskalk testing of 50 bolts, load-deformation diagrams illustrate break can be either Instanteneous break (7 bolts) or slow break with deformation (11 bolts). Instantaneous break occurred up to 23 tons (B41, grouted length 0,3 meter, RMR_{RQD5}=59). Slow breaks with deformations occurs up to 20 tons with 5 mm deformation (B40, grouted length 0,3 meter, RMR_{RQD5}=54), and up to 55mm with 18 tons (B11, grouted length 0,47 meter, RMR_{RQD5}=73). Deformations are not necessary linear with load but can occurred with steps, representing the relative interblocking of blocks of rock. All the testing where deformation of rock mass was recorded went to break.

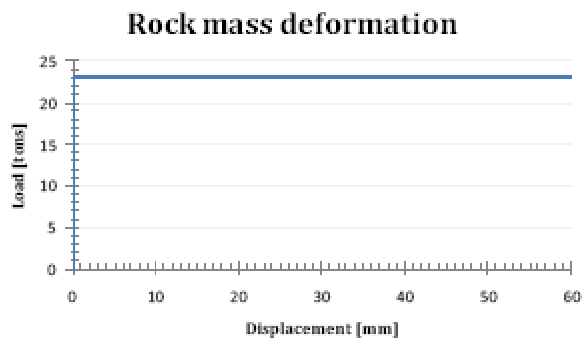


Figure 42 : load-deformation curve, instantaneous break, B41

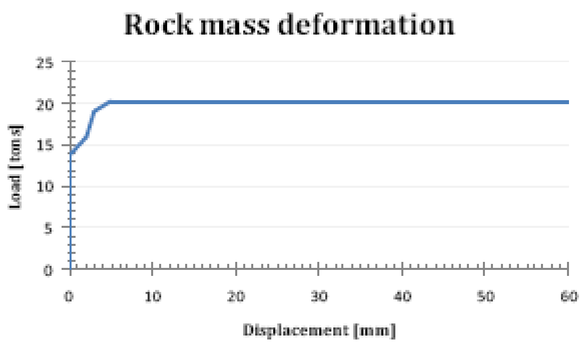


Figure 43 : Load-deformation curve, slow break with deformation, B40

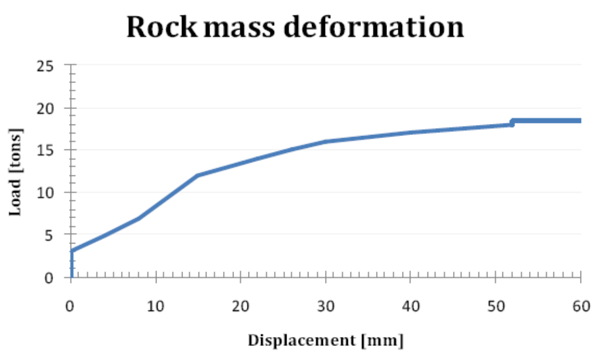


Figure 44 : Load-deformation curve, slow break with deformation, B11

- Geometry of failure in rock mass : (A) liberation of a block or (B) rock mass totally cracked around the bolt libered clean.

(A) liberation of a block

Figure 45 : Influence of structural geology on the shape of cone of rock mobilized by uplift anchors, Verdalskalk 2012, bolt nr 9

Break can occur further around the bolts in the rock mass, and liberate a bloc where the bolt is grouted. Such a bloc is defined by existing discontinuities, or by new break in the rock mass. This illustrate the theory of the cone of influence around the rock, whose have actual shape with regards to orientation of rock mass discontinuities.

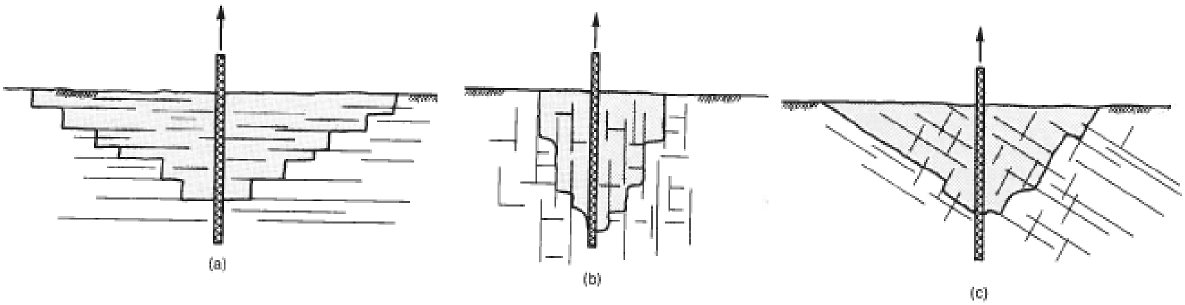


Figure 46 : Influence of structural geology on the shape of cone of rock mobilized by uplift anchors, Wyllie

(a) Wide cone formed in horizontal bedded formation.
(b) Narrow cone formed along vertical fractures
(c) Surface of cone formed along inclined fractures

(B) rock mass totally cracked around the bolt libered clean.

Figure 47 : Break inside the cone of influence, Verdalskalk 2012, bolt nr 3

Rock mass break can occur inside the cone of influence where the rock is grouted. Then bolt is released clean or with small pieces out of the outcracked zone.

→Formal between rock mass quality and maximal tensile capacity of bolt.
The three proposed formals: RMR, Q and Wyllie were further confirm with the tests on 50 bolts. The linear formal from RMR is the most relevant.

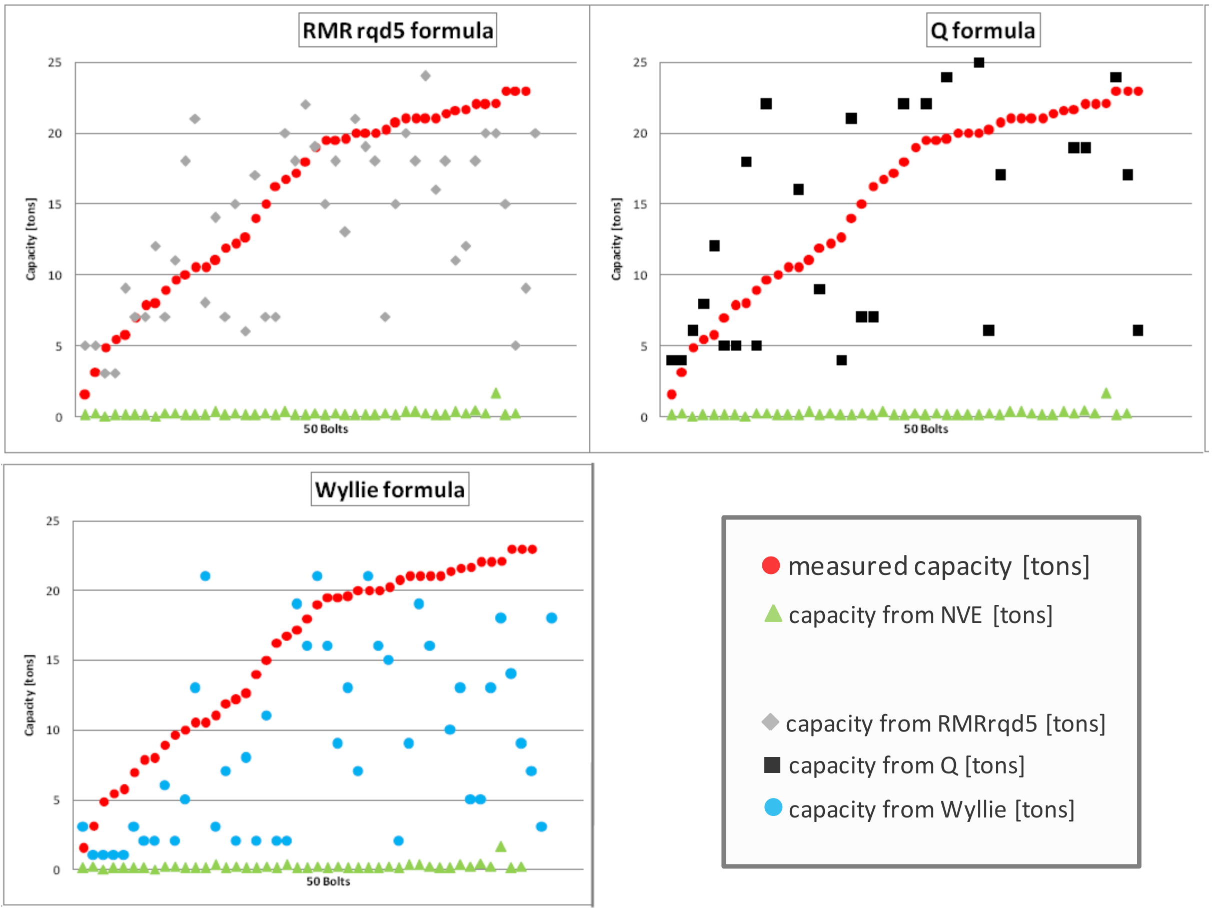


Figure 48 : Diagram of measured versus expected tensile capacity of bolts : RMR, Q, Wyllie and NVE

(1) RMR: Linear relations which give minimal tensile capacity for a passive rock bolt.
F = L. (12.RMR$_{RQD5}$-505), with force in kilonewton and length in meter

Input parameter is the recognized RMR$_{1989}$ rock mass rating [1.2. and table in appendix] with following modifications :

- RQD$_5$ instead of traditional RQD$_{10}$ rating. This modification takes into consideration the scale of the rock mechanism in the vicinity of the bolt.
- Surrounding geometry correction. The rating should be modified up to 20 points (over 100) to traduce a special favorable or unfavorable condition of the rock mass in a scale of about 10 meters. The geometry of the discontinuities, and the lateral support around the bolt should be considered. Such corrections were already proposed in some versions of RMR published since its first definition.

(2) The formals from Q and Wyllie gave relevant results for the present testing, but the author considered they are less reliable. They indeed came from more complex relations (exponential) and therefore are sensitive to small mistakes in rock estimations, but rock mass estimation accuracy is always limited.

After the author, the linear formal to obtain rock bolt tensile capacity from RMR$_{RQD5}$ is satisfying, and an improved design method could be suggested without need of developing a new rock mass rating system. Tests were still only performed in one quarry. Like all geotechnical problematic, the relation developed should be considered as a guide and precise in further research. The author also issues the reservation that only tensile solicitations, and no lateral solicitations (shear) was performed.

2.1.3.2 Detailed results available

Bolt	L[m]	C[tons]	C_{RMR}	C_Q	C_{wyllie}	C_{NVE}	RMR_{RQD5}	Q	GSI_Q	GSI_{RMR}
1mA	1,01	0					80			
1mB	0,48	0,7					80			
42	0,38	0	9	8	3	0,1	54	1,0	52	30
39	0,24	1,5	5	4	1	0	51	0,1	34	27
38	0,22	3,15	5	4	1	0	51	0,1	34	27
31	0,30	4,85	3	6	1	0,1	43	0,1	32	19
52	0,36	4,85					30			
20	0,40	5,45	3	8	1	0,2	39	1,0	52	15
53	0,26	5,7	9	12	3		63	4,2	65	39
47	0,35	6,95	7	5	2	0,1	51	0,4	43	27
48	0,33	7,8	7	5	2	0,1	51	0,4	43	27
32	0,27	7,95	12	18	6	0,1	71	10,0	73	47
46	0,32	8,95	7	5	2	0,1	51	0,4	43	27
5	0,24	9,65	11	22	5	0	71	20,0	79	47
8	0,40	10	18	26	13	0,2	71	9,4	72	47
30	0,42	10,5	21	29	21	0,2	76	10,0	73	52
35	0,33	10,5	16	16	3	0,1	73	5,0	67	49
2	0,36	11	14	33	7	0,1	66	19,5	79	42
36	0,29	11,9	7	9	2	0,1	54	2,0	58	30
3	0,49	12,15	15	36	8	0,3	59	11,8	74	35
45	0,31	12,6	6	4	2	0,1	51	0,4	43	27
9	0,43	14	17	21	11	0,2	66	5,1	67	42
44	0,29	15	7	7	2	0,1	54	1,3	55	30
43	0,29	16,25	7	7	2	0,1	54	1,3	55	30
26	0,40	16,75	20	33	19	0,2	76	15,0	77	52
27	0,36	17,2	18	29	16	0,1	76	15,0	77	52
11	0,47	18	22	22	21	0,3	73	4,7	66	49
18	0,37	19	19	34	16	0,1	76	20,0	79	52
7	0,33	19,5	15	22	9	0,1	71	10,0	73	47
16	0,45	19,5	18	59	13	0,2	68	52,2	88	44
33	0,29	19,55	13	24	7	0,1	71	15,0	77	47
17	0,42	20	21	58	21	0,2	76	60,0	89	52
19	0,37	20	19	33	16	0,1	76	18,0	78	52
28	0,35	20	18	25	15	0,1	76	11,4	74	52
40	0,30	20,2	7	6	2	0,1	54	1,0	52	30
1	0,35	20,7	15	17	9	0,1	69	5,0	67	45
12	0,40	21	20	55	19	0,2	76	60,0	89	52
23	0,36	21	18	28	16	0,1	76	13,5	76	52
25	0,47	21	24	38	27	0,3	76	15,0	77	52
29	0,48	21	16	33	10	0,3	62	10,5	73	38
24	0,44	21,4	18	32	13	0,2	68	12,0	75	44
15	0,38	21,6	11	48	5	0,1	58	45,0	87	34
10	0,31	21,7	12	19	5	0,1	66	8,0	71	42
4	0,47	22	18	19	13	0,3	66	3,6	64	42
22	0,39	22	20	29	18	0,2	76	12,0	75	52
21	0,54	22,1	20	29	14	0,4	64	6,0	68	40
6	0,40	23	15	24	9	0,2	66	7,7	71	42
13*	0,84	23	5	17	7	1,6	39	1,0	52	15
41	0,30	23	9	6	3	0,1	59	1,0	52	35
14*	0,39	26	20	68	18	0,2	76	120,0	95	52

Bolt
Mode of failure
- Mortar not dry
- Loosed rock
- Rock mass break
- No break (or *Steel break)

Capacity
NVE design : 180MPa <-> 8,8 tons
- Inferior to 8,8 tons
- Superior to 8,8 tons
- Twice superior to 8,8 tons

$RMR_{RQD5} \sim RMR + 10$

Table 7 : Synthetic table of test results

Detailed results from the 50 bolts tested are in appendix :
- Detailed Q and RMR_{RQD5} rating
- Table of maximal tensile strength measured and expected from NVE, RMR_{RQD5}, Q and Wyllie. Modes of failure and grouted length are also in that table.
- Documentation from testing : core or view inside drilling, bolt before and after break, load-deformation curve

2.2 Proposition of design method

First considerations have been made in part 1.3. about rock bolt design. They are precised here, in order to improve the design.

2.2.1 Parameters to be considered for design

Relevant parameters for design are related to mechanisms of load transfer from the structure to the rock foundation operated along the bolt. It should be first considered which load will take the anchor. Passive rock bolts are working in reaction to deformation, either tensile or shear. Shear solicitation at the contact rock/concrete could be withstanded up to a displacement of few millimeters, while normal solicitation can be withstanded up to displacement of centimeters [26]. The load transmitted by the structure should be identified. For a vertical anchor in dams, overturning moment (ice load) implies a tensile solicitation. Non exactly vertical loading, and eventual sliding of the structure would imply shear solicitation. Pore pressure who implies uplift is particular as it's acting as an internal loading in the rock. In addition, seismic loading could be considered as horizontal loading and uplift. More widely, civil engineering structures like bridge or viaduct (Viaduc de Millau, studies from 1987 and inauguration 2004) are concerned with uses of anchor against horizontal loading.

- ➔ Rock mass condition and strength

"The most important factors influencing the selection of the bond length are the strength and fracture characteristics of the rock in the bond zone. " (Wyllie)

Finding out some precisions in quantitative and qualitative considerations from the rock mass with regards to maximale tensile capacity for passive rock bolts was the objective of the thesis.

The design is conservative worldwide. The zone of rock mobilized around the bolt is considered as a cone with angle 45 degrees. The cone of rock is a model for calculations. Shape is actually not exactly a cone with regards to rock discontinuities characteristic, as it was explained in the analysis of results. The conservative design estimate the weight of the cone. The author meant that the geometry of a cone around the bolt is a relevant approximation for the zone of influence around the bolt. But in that volume of rock surrounding the bolts, it's not self weight but shear strength of discontinuities which will be the major contribution of the rock mass resistance to tensile solicitations.

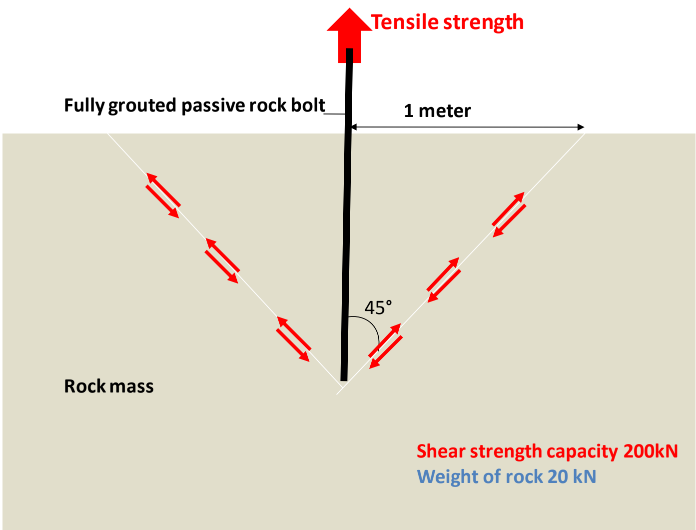


Figure 49 : Mechanism of resistance of the rock mass solicited around the bolt.

2 modes of failure were identified :

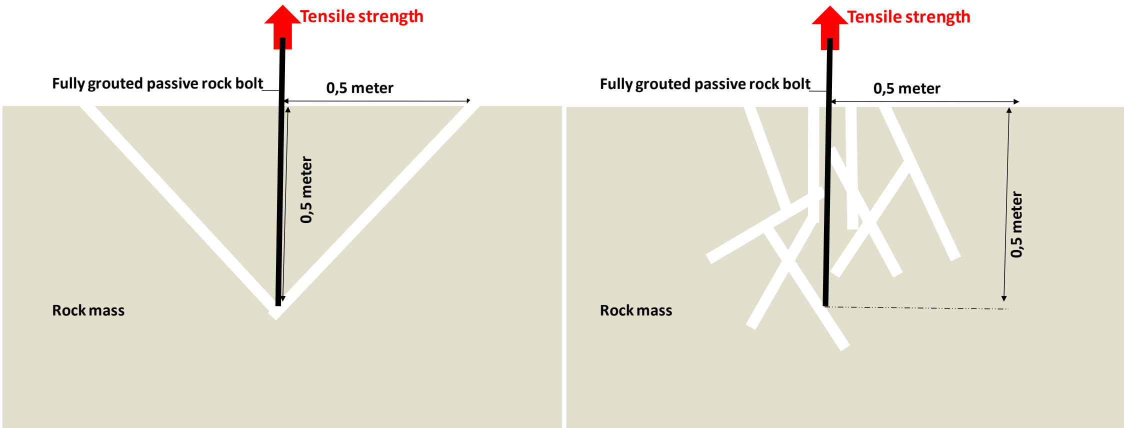


Figure 50 : 2 modes of failure of the rock mass (a) liberation of a bloc (b)generalized cracking

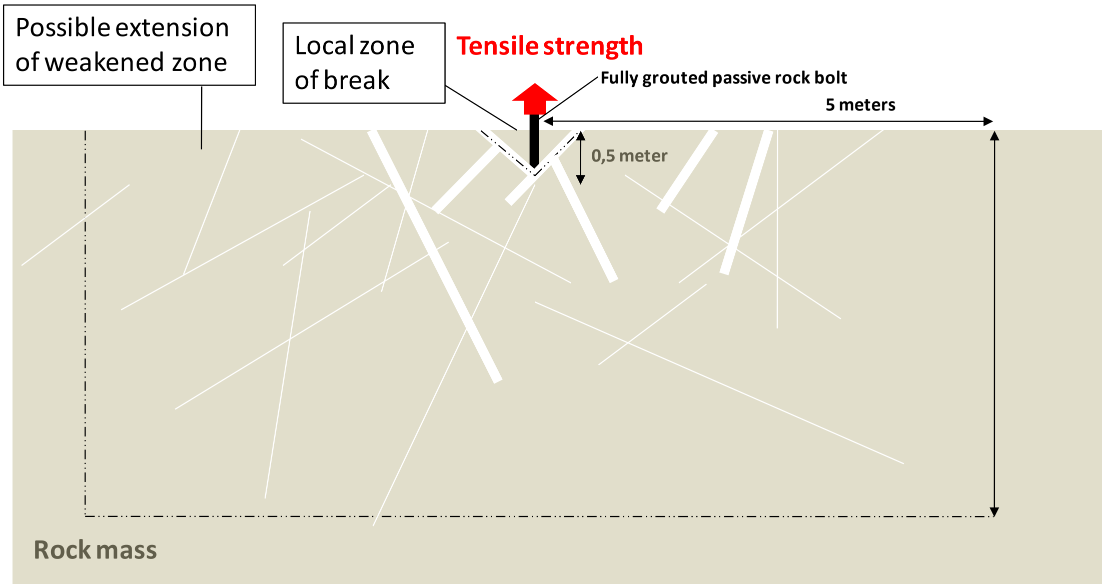


Figure 51 : Extension of weakened zone.

If block liberated was not observed bigger than 1meter around the bolt, deformations of rock mass were sometimes noticed in several meters further around. This problematic could be handled with the consideration of action of sevarl bolts combined, which was not object of this thesis.

➔ Grouting properties

Compressive and shear strength have been compared in the literature between different types of grouting. Mortar used in full scale test program has compressive strength 50MPa. Grout properties are not only depending on material properties of the type of grouting, but also on the conditions of performing. To ensure required correct performance, grouting should be properly performed (ratio water/cement, correct distribution in the hole around the bolt), and not in load before a sufficient curing time is passed (cement 28 days).

Type of grouting	Compressive strength	Shear strength	Borehole diameter	Bolt diameter
Cement (28 days)	65 to 85 MPa	10 to 15 Mpa	35 to 44 mm	20 to 22 mm
Resin	120 to 140 MPa	25 MPa	22 to 25 mm	18 to 20 mm

Figure 52 : Comparison between cement and resin grouting for passive rock bolts, after [16]

➔ Influence of several bolts

2.2.2 Rock bolts, recommendations for design improvement and post-control

Here are proposed possible ideas to improve actual design of passive anchorage technology, which has been presented in 1.3. Propositions made below for design of passive rock bolts have considered references from design in Norway (cf 1.3.1. and Retlingslinjer for betongdammer NVE), France (Guide ancrage passives CETRA) and Switzerland (Directives suisses), as well as literature from Wyllie and Tome 2 Mecanique des Roches. Internationally, princip is :

- (1) L : determination of the required grouting length : with regards to bond grout-steel and grout-rock to support a design maximal capacity (400MPa = steel capacity for NVE) Value for bond strength [appendix] are worldwide conservative after researches, but higher in Sweden. A material factor γ_m=2 is taken in addition after NVE design.
- (2) D : position of the center of the grouted length : at a depth giving cone around bolt with sufficient volume so that weight of rock will support tensile capacity against a maximal stress limited to avoid large deformation (180MPa ←→88kN for NVE)
- (3) 0 : Steel has required elastic limit (400MPa for NVE) and is not consider critical in the design

➔ Domain of application

- A single anchor doesn't contribute considerably to the bearing capacity of the structures, or of part of the structures : a single anchor doesn't take more than 3 to 5 percent of it, after the Swiss directives.
- The displacements of the structure inherent to the system are acceptable.
- Passive rock anchors react directly to external solicitations. The effect of variable loading on the long-term capacity of the anchors should be considered.
- Anchors should not be loaded to early, grouting must consolidate itself first.

For these reasons it's understandable that passive rock anchors are not recommended for large dams.

➔ Control when performed

Control to accept implementation of anchors can be found in different regulations.

➔ Monitoring after implantation

As it was exposed in 1.3.1., passive anchors can not be straightly controlled in service, unlike active anchors. In special cases, direct assessment can be performed with use of ultra sound. Traditionaly, it's recommended to monitor and observe displacement of the structures, cracks apparition, incoming of water, obturation of drainage or local deformations. Such information can be organized with program like KUBA-DB, data bank for civil engineering structures.

➔ Method proposed with associate range of application and factor of safety, presented comparible to actual NVE design presented just above is :

(1) L_{180MPa} : Grouted length required calculated for a maximal tensile capacity 180MPa. This gives L_{180MPa} between 0,6 meter and 2,5 meter (0,7m in characteristics of performed tests). As values for bond strength are likely to be underestimated (Sweden reference and results of present test) and as material factor of 2, this gives a factor of safety of 2.

- (2) d : According to the present thesis, rock resistance doesn't have weight as major contribution, but shear strength of discontinuities for cracked rock mass, or cohesion for fair rock mass. Critical length under which bolt go to break regardless of rock conditions is under 0,4 meter.

  It's therefore suggested, for rock mass such RMR>50, to use the length of grouting L_{180MPa} directly as required length of bolt inside the rock, with an additional extra security length of 0, 5 meter to prevent eventual superficial loosed surface (weathering, ice effect). Designed required length would then be proposed between 1,1 and 3,0 meter. This applies for rock mass conditions with RMR greater than 50. Remark : After Eder dam research, no load is transfer to the rock in a depth greater than 3,0 meter (see 3.2). The 50 bolts tested confirm the proposition.
- (3) no changes with regards to steel : Steel has required elastic limit (400MPa for NVE) and is not consider critical in the design

3 PROPOSITIONS FOR FURTHER WORK

3.1 Precise and extend the relation developed

3.1.1 Critical length without break

The minimal length under which bolts will necessarily break has been assessed at 0,4 meter. It's of interest to assess in addition the length above which bolts will not break in rock mass condition superior to a limit which is proposed RMR=50. Hypothesis from the present is L superior or equal to 1 meter.

3.1.2 Location for testing

The two programs of test, from 2011 and the present were performed in Verdalskalk. That quarry presented very different rock mass conditions, from high fractured rock mass conditions to intact rock and was extremely relevant for the work developed so far. The intact rock properties in Verdalskalk are high : very pure marble with a compressive strength of 90 MPa. It would be of interest to perform tests further in a quarry with still cracked rock mass but lower intact material characteristics. Finding a place for testing is challenging, and should be considered early in the next thesis. Franzefoss quarry or rock around dam under construction or rehabilitation could be investigated.

3.2 Grouting

Grouting is not the subject of the present thesis and parallel research are currently conducted, for instance in France. Over the 50 bolts tested yet, 11 broke because of mortar. The correct implementation of grouting, is indeed necessary both for capacity of the grout to transfer compressive and shear load, but also to protect steel from corrosion. In our testing, the break for mortar was maybe due to a too short curing time (14 days), but that time was acceptable according to the diagram from fabricants and testings from last year, and if the climatic conditions were difficult, the mortar choose was adaptated to negative temperatures, and bolts performed in enough same conditions gave either solid or not dry mortar without clear justifications. It is therefore considered of interest to study further grouting. The difference in grouting type (mortar in Norway, cement in Sweden and France, development of resin) and in grouting conditions (diameter of hole, climatic conditions, other?) can be considered. Methods to control and monitor performance of the grouting could be interesting developments as well, and used as recommendations for practice.

3.3 Loading mechanisms

3.3.1 Shear

A limit of this thesis in the development of renewed design is that only tensile strength tests are performed. It's of interest to perform in addition lateral load for evaluation of shear behavior. The results obtained were likely to the tensile strength obtained by Stjern in a full scale test rig. Results from that proposed "shear testing" could then be probably comparable with the ones from Stjern. This can be performed in the same protocol developed by Lars K. Neby and in this thesis. It's suggested that excavator load with an angle. For eventual stiffness and interface considerations with a structure, a small bloc of concrete can be realized in addition above rock surface.

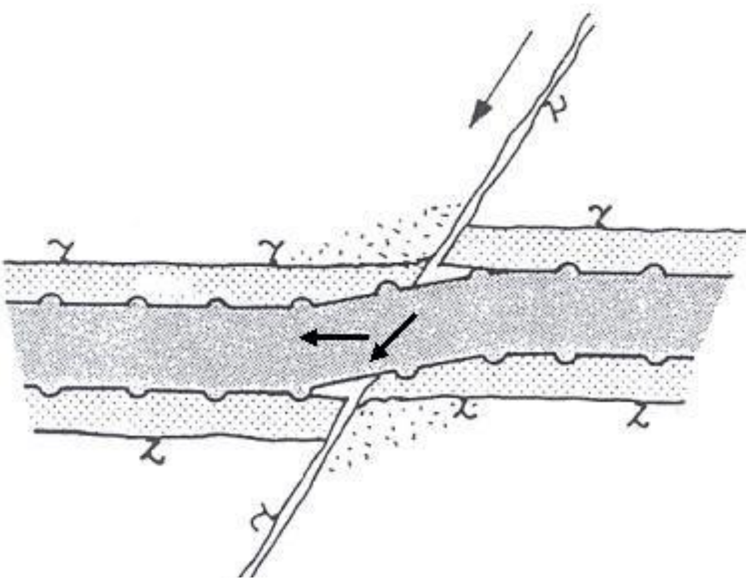


Figure 53 : Grouted rock bolt subject to lateral force, after [20]

The shear resistance of a bolt reinforced rock joint is normally found by shear box tests. These tests are performed on both a plain joint and a bolt reinforced joint (figure).

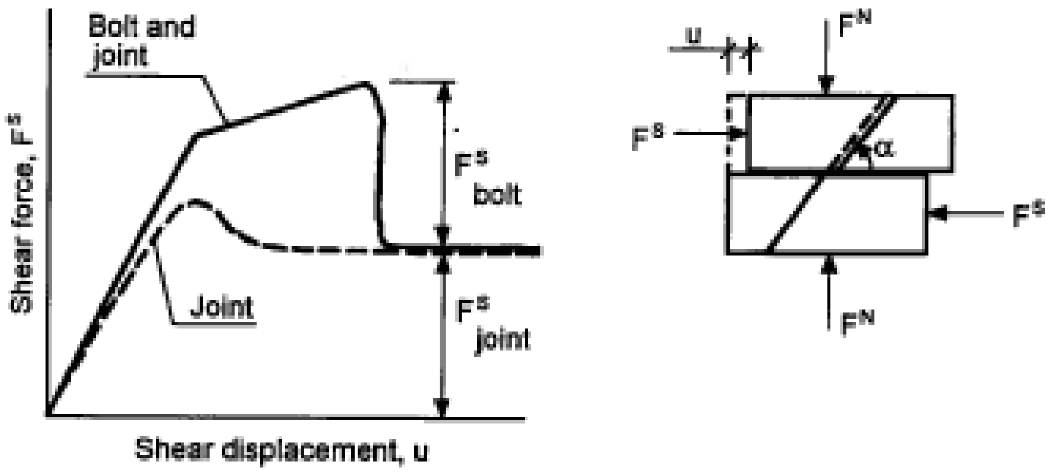


Figure 54 : Shear resistance F_S versus shear displacement u for a bolt reinforced joint under constant normal load F_N, after Ludvig, 1984

3.3.2 Load transfer and distribution

Supplementary monitoring could be proposed to obtain documentation to precise the maximal length where load can be transferred along the bolt. In the Eder dam, optic fiber was implanted inside the anchor to obtain deformation along bolt. That case is a high dam with active anchor, but such technology could maybe be envisaged as well for passive anchors. Description of the method is presented in figure 55. The complete load transfer into the ground ends after 3 m fixed anchor length, as shown in figure 56. After the French "Guide ancrages passifs en montagne" [26], a fair rock mass can take load from the bar over a length of 1 meter. The ratio Length over diameter has characteristic values 40 for fair rock after SNCF (and 50 for concrete after BAEL). For 25mm diameter steel bolts, length would be 1m with ratio 40. Influence of water level affects only the beginning of the grouted length, as shown in the figure 57. NORUT in Narvik is currently developing a program of research in load transfer mechanisms. It was also found in literature model of exponential distribution of strain along the bolt (figure below) This highlights that an higher length will not necessarily bring more security if most of the load is transferred only on a short length on the part of the grouting closest to the surface.

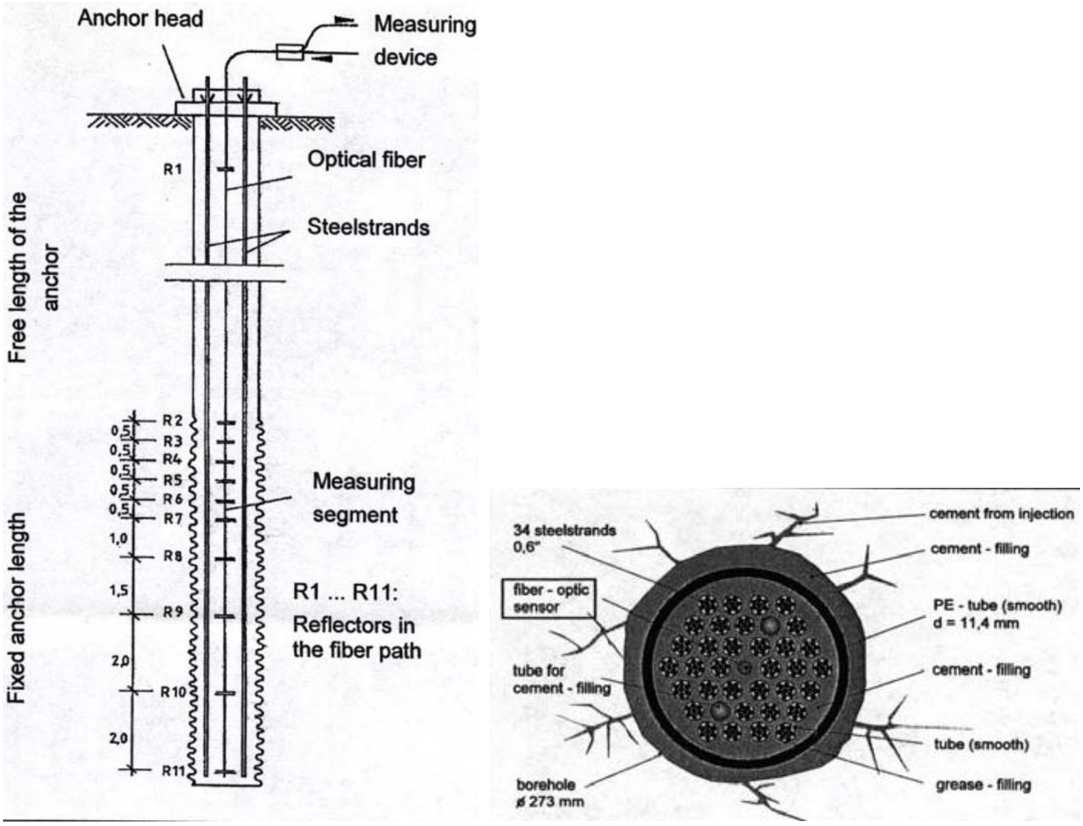


Figure 55 : Sensor segments inside the anchor, Eder dam

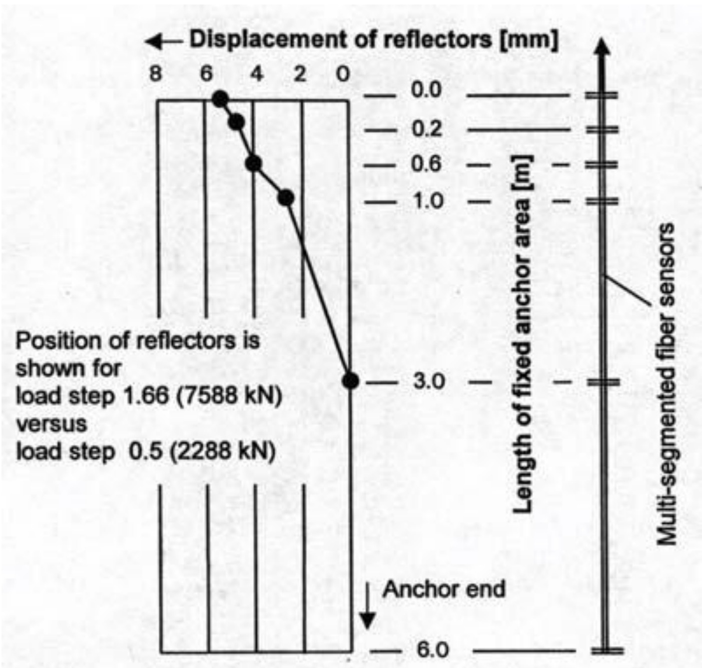


Figure 56 : Suitability test in a stilling basin, Eder dam

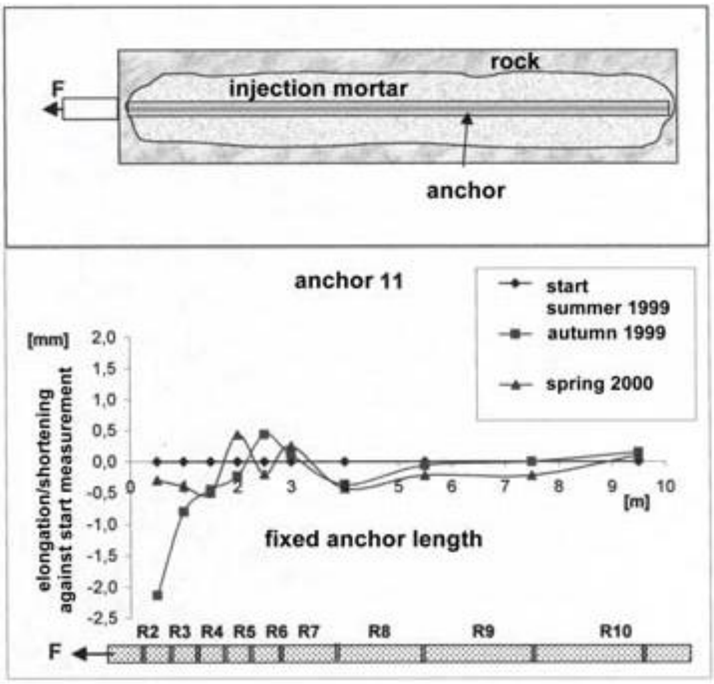


Figure 57 : Measured displacement in the fixed anchor length by OTDR device, Eder dam

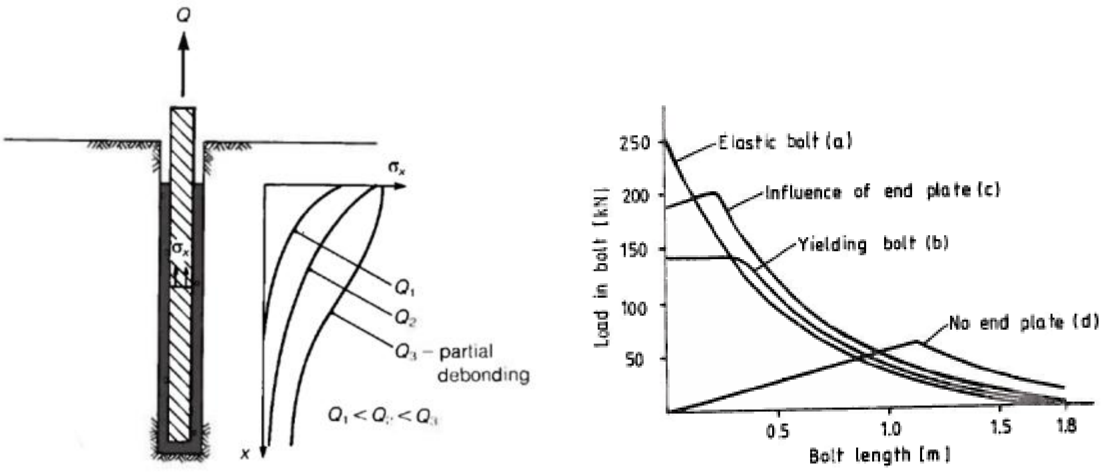


Figure 58 : Distribution of tensile stress along length of anchor

(a) Variation in distribution of tensile stress along length of anchor zone with increasing applied load, after Farmer 1975
(b)Theoretical distribution-load curve along grouted rock bolts, after [18]

3.3.3 Influence of water

Hydraulic gradient is an internal loading characteristic and critical in dam foundations. Ideas could be submit to assess its effect on passive anchors.

Main conclusions

The present thesis performed further work from Lars K. Neby's in 2011 on improved design of fully grouted passive rock bolts, with considerations of resistance from the rock mass to tensile solicitation. This thesis is again meant to be followed by further research, which can precise and eventually correct the ideas developed. It concludes the followings.

From literature study on rock strength, especially from shear strength discontinuities, and tensile full scale test on 68 fully grouted steel bolts in a cracked rock mass, it was conclude :

1. Critical length, under which bolts will always break is 0,4 meter.
2. From rock mass such RMR>50, minimal tensile capacity can be estimated proportional to the length of bolt, with factor of proportionality from a linear relation with input parameter RMR_{RQD5}.

From results named above and consideration to practice of design which is pretty similar worldwide with a necessary grouted length from bond strength mortar-steel and steel-rock, and a supplementary length to obtain a sufficient weight of rock in the cone around the bolt, some suggestions are made to improve the design :

3. Grouted length still estimated from adherence
4. A length of 0,5 meter, corresponding to loosened crack conditions and freezing area should be add in security. No supplementary length added to go deeper in the rock if rock conditions is superior to RMR=50 and there is not unfavorable specificities. This results is conform with Mine et Carrieres 1992, who wrote that for length greater than 0,5 meter for steel rebar grouted with cement, break came from steel and reinforcement is fully mobilized.
5. Development of an organized program to follow regularly the state of rock bolts after their installation, with special regards to swiss directives.

The author highlights the necessity to go further with the research. The work is presented as a first guide with ideas to review the design of passive rock bolts, considered conservative with regard to rock mass resistance. Renewing design is often considered delicate as rock mass surface capacity is linked with numerous parameters in complex relations, and bolts monitoring after installation is not as simple as control of tension on pre-stressed anchor. In particular further work is recommend with :

- Go further with the developed test program both 1) in other rock quarry to confirm minimal length where bolts will not break, and 2) with shear solicitation.
- Precise methods to assess rock bolt capacity and conditions after installation

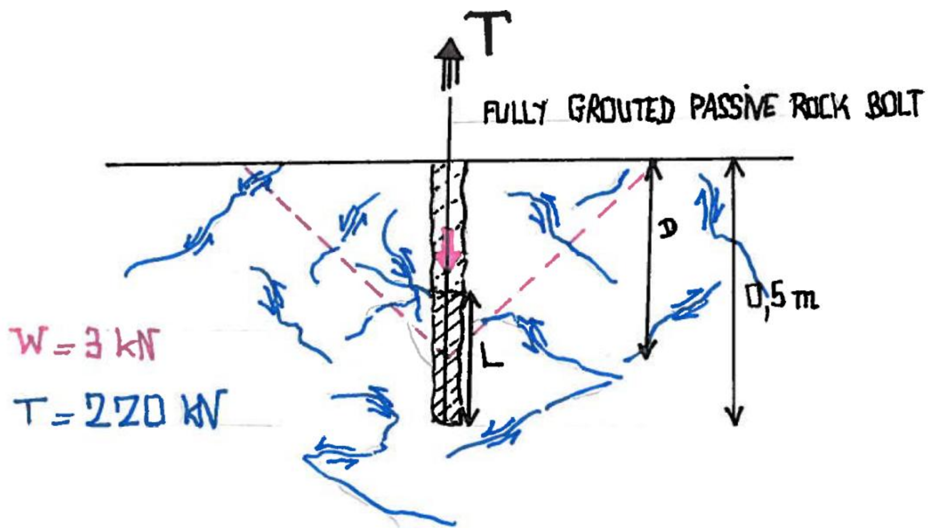


Figure 59 : Resistance from the rock against tensile load of bolt results from shear strength of discontinuities

APPENDIX

A. LITERATURE

A1 Dam safety
A2 Plotting of structural geology data : stereographic projection

B. CALCULATIONS AND TEST RESULTS

B1 Rock mass rating
- Q
- RMR
- GSI and parameters for shear strength of discontinuities after Wyllie
- Parameters for shear strength of discontinuities after Barton

B2 Calculations for tensile capacity of passive rock bolts
- NVE
- Wyllie

B3 ISRM test of rock properties from Lars K. Neby 2011
B4 Bolts documentation
- Pictures of results for the 50 bolts from lowest to highest tensile capacity and with regards to mode of failure : rock, mortar or no break (or steel)

A.1. DAMS SAFETY

A.1.1. Norway

The practice of public control and supervision of dams in Norway started in 1909 with the foundation of the Control Department in the former Norwegian Water Administration, which was succeeded by the Norwegian Water Resources and Energy Administration (NVE) in 1920.

NVE, which is now a directorate under the Ministry of Petroleum and Energy, is still the governmental authority on dam safety in Norway. Today the function of the former Control Department is taken care of by the the Licensing and Supervision Department, Section for Dam Safety in NVE. The main activities of the Section for Dam Safety are related to supervision of dams and appurtenant structures, including approval of plans for construction and rehabilitation, and administration of the legal framework for dam safety, including development of new technical guidelines.

The main goal of the public supervision is to ensure a uniform high level of safety on Norwegian dams and appurtenant structures, and thereby ensure that these structures are not posing a threat to life, property or the environment. NVE give the highest priority to the dams in the highest consequence classes. The number of dams subject to public supervision, and thereby included in the dam register held by NVE, are approximately 2600. More than 290 of these dams are classified as class 3 dams, and 335 are large dams (height > 15 m). The dam register also include some technical data for other structures such as intake structures, penstocks, tunnels and canals.

Classe	Residential units	Infrastructure, community features	Environment and properties
4 (since 2010)	>150		
3	21-150	Damage to the heavily trafficked roads or railways, or other infrastructure, with particular importance to life and health	Major damage to particularly important environmental values or particular damage to foreign property
2	1-20	Damage to medium trafficked roads or railways or other infrastructure that are crucial to life and health	Heavy damage to important environmental values or major damage to foreign property
1	Temporary residence equivalent to <1 permanent housing unit	Damage to the less congested roads or other infrastructure of importance for life and health	Damage to environmental assets or foreign property
0	other		

Table 8 : Definition of consequence classe in dam safety in Norway, NVE 2010

Konsekvensklasse	Antall dammer
Klasse 4	32
Klasse 3	333
Klasse 2	659
Klasse 1	1312
Klasse 0	751

Table 9 : Repartition of Norwegian dams in the consequence classes, NVE 2010

A.1.2. France

Sécurité des ouvrages hydrauliques et de protection

7 octobre 2009 (mis à jour le 17 janvier 2011) - Prévention des risques

Les barrages servent à retenir temporairement une quantité plus ou moins grande d'eau pour différents usages (production d'énergie hydroélectrique, alimentation en eau potable, irrigation, régulation des débits des cours d'eau, activités touristiques….). Les digues de protection contre les inondations ont pour but de guider l'eau en dehors des zones densément habitées ou sensibles afin d'éviter leur submersion, par exemple lors de fortes crues. En retenant l'eau, ces ouvrages accumulent des quantités importantes, voire considérables d'énergie. La libération fortuite de cette énergie est une source de risques importants.

Les responsabilités des différents acteurs

La sécurité des barrages et des digues est de la responsabilité des propriétaires ou concessionnaires des ouvrages. Cette responsabilité inclut le respect d'obligations fixées par l'Etat. La DGPR est chargée au sein du MEEDDM d'organiser le contrôle par l'Etat du respect de ces obligations. Ce contrôle de la sécurité des ouvrages hydrauliques s'appuie sur les DREAL (hors Île-de-France et départements et territoires d'outre-mer).

Le dispositif réglementaire

Le dispositif réglementant la sécurité des barrages et des digues s'appuie principalement sur la loi sur l'eau et les milieux aquatiques du 30 décembre 2006 et le décret 2007-1735 du 11 décembre 2007, lui-même complété par plusieurs arrêtés. Les barrages les plus importants doivent par ailleurs faire l'objet d'un plan particulier d'intervention tel que prévu par le décret n° 92-997 du 15 septembre 1992.

Les classes de barrages et de digues

Les obligations des propriétaires et concessionnaires sont, en application du décret 2007-1745, modulées en fonction de l'importance des risques et des enjeux. Pour cela, les barrages et les digues sont répartis en quatre classes de A (pour les ouvrages les plus importants) à D en fonction de leurs caractéristiques géométriques (leur hauteur, le volume d'eau stocké) et de la présence éventuelle d'enjeux importants à l'aval (le nombre de personnes dans la zone protégée par les systèmes d'endiguement..). Le décret définit pour chacune des classes les études, les vérifications, les diagnostics… et leurs périodicités que doivent mettre en œuvre les responsables des ouvrages.

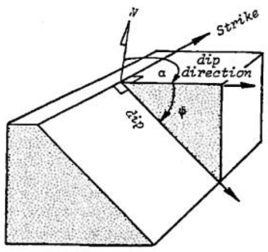
Il y a environ 300 barrages de classe A, 300 de classe B et environ 500 de classe C. Quant aux barrages de classe D, il en existe plusieurs dizaines de milliers répartis sur l'ensemble du territoire. Le recensement des digues est en cours, avec plus de 7700 km de digues dores et déjà identifiées.

Les études de dangers

Les barrages des classes A et B, ainsi que les digues des classes A, B et C devront faire d'une étude de dangers. Le contenu de ces études est précisé par un arrêté du 12 juin 2008 pris en application du décret 2007-1735 du 11 décembre 2007. Cette étude a pour objet de caractériser les risques intrinsèques à l'ouvrage ainsi que ceux susceptibles de se manifester à l'occasion de phénomènes exceptionnels tels que crues ou séismes et d'identifier les parades et moyens de prévention et de protection permettant de maîtriser les risques.

A.2. Plotting of structural geology data : Stereographic projection

Analysis of structural geology data involves first plotting poles representing the orientation of each fracture. Fracture sets are then identify from these plots. It can then been drawn great circles representing the average orientation of each set, majore fractures such as faults, and the dip and dip direction of the cut face.



definition of geometrical terms

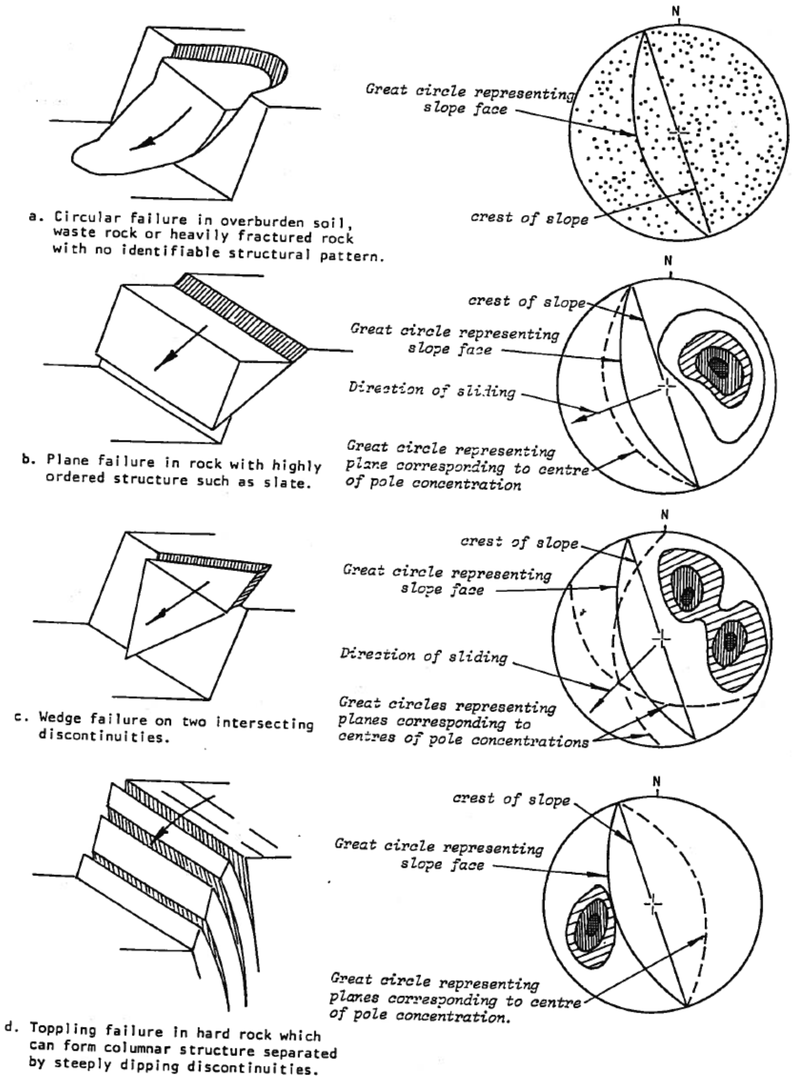


Figure 60 : Main types of slope failure and stereoplots of structural conditions likely to give reise to these failures, after Hoek&Bray 1981

B.1. ROCK MASS RATING

B.1.1. Q

Input parameters to Q system

Rock quality designation (RQD)

Very poor	RQD = 0 - 25%
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100
Notes:	
(i) Where RQD is reported or measured as < 10 (including 0), a nominal value of 10 is used to evaluate Q	
(ii) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate	

Joint set number (Jn)

Massive, no or few joints	Jn = 0.5 - 1
One joint set	2
One joint set plus random	3
Two joint sets	4
Two joint sets plus random	6
Three joint sets	9
Three joint sets plus random	12
Four or more joint sets, heavily jointed, "sugar-cube", etc.	15
Crushed rock, earthlike	20
Notes: *(i) For tunnel intersections, use (3.0 x Jn); (ii) For portals, use (2.0 x Jn)*	

Desciption and ratings for the parameter Jr *(joint roughness number)*

a) Rock-wall contact, b) rock-wall contact before 10 cm shear		**c) No rock-wall contact when sheared**	
Discontinuous joints	Jr = 4	Zone containing clay minerals thick enough to prevent rock-wall contact	Jr = 1.0
Rough or irregular, undulating	3		
Smooth, undulating	2	Sandy, gravelly or crushed zone thick enough to prevent rock-wall contact	1.0
Slickensided, undulating	1.5		
Rough or irregular, planar	1.5	***Notes:***	
Smooth, planar	1.0	*i) Add 1.0 if the mean spacing of the relevant joint set is greater than 3 m*	
Slickensided, planar	0.5	*ii) Jr = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are oreintated for minimum strength*	
Note*: i) Descriptions refer to small scale features, and intermediate scale features, in that order*			

Descriptions and ratings for the parameter Ja *(joint alteration number)*

	JOINT WALL CHARACTER		**Condition**	***Wall contact***
Contact between joint walls	CLEAN JOINTS	Healed or welded joints:	filling of quartz, epidote, etc.	Ja = 0,75
		Fresh joint walls:	no coating or filling, except from staining (rust)	1
		Slightly altered joint walls:	non-softening mineral coatings, clay-free particles, etc.	2
	COATING OR THIN FILLING	Friction materials:	sand, silt calcite, etc. (non-softening)	3
		Cohesive materials:	clay, chlorite, talc, etc. (softening)	4

	FILLING OF:	**Type**	***Partly wall contact*** *Thin filling (< 5 mm)*	***No wall contact*** *Thick filling*
Partly or no wall contact	Friction materials	sand, silt calcite, etc. (non-softening)	Ja = 4	Ja = 8
	Hard cohesive materials	compacted filling of clay, chlorite, talc, etc.	6	5 - 10
	Soft cohesive materials	medium to low overconsolidated clay, chlorite, talc, etc.	8	12
	Swelling clay materials	filling material exhibits swelling properties	8 - 12	13 - 20

Description and ratings for the parameter Jw *(joint water reduction factor)*

Dry excavations or minor inflow, i.e. < 5 l/min locally	p_w < 1 kg/cm^2	Jw = 1
Medium inflow or pressure, occasional outwash of joint fillings	1 - 2.5	0.66
Large inflow or high pressure in competent rock with unfilled joints	2.5 - 10	0.5
Large inflow or high pressure, considerable outwash of joint fillings	2.5 - 10	0.3
Exceptionally high inflow or water pressure at blasting, decaying with time	> 10	0.2 - 0.1
Exceptionally high inflow or water pressure continuing without noticeable decay	> 10	0.1 - 0.05
Note: *(i) The last four factors are crude estimates. Increase Jw if drainage measures are installed* *(ii) Special problems caused by ice formation are not considered*		

Description and ratings for parameter SRF *(stress reduction factor)*

A. Weakness zones intersecting excavation	Multiple weakness zones with clay or chemically disintegrated rock, very loose surrounding rock (any depth)			SRF = 10
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation < 50 m)			5
	Single weakness zones containing clay or chemically disintegrated rock (depth of excavation > 50 m)			2.5
	Multiple shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation < 50 m)			5
	Single shear zones in competent rock (clay-free), loose surrounding rock (depth of excavation > 50 m)			2.5
	Loose, open joints, heavily jointed or "sugar-cube", etc. (any depth)			5
Note: *(i)*	*Reduce these valued of SRF by 25 - 50% if the relevant shear zones only influence, but do not intersect the excavation*	σ_c / σ_1	σ_θ / σ_c	
B. Competent rock, rock stress problems	Low stress, near surface, open joints	> 200	< 0.01	2.5
	Medium stress, favourable stress condition	200 - 10	0.01 - 0.3	1
	High stress, very tight structure. Usually favourable to stability, may be except for walls	10 - 5	0.3 - 0.4	0.5 - 2
	Moderate slabbing after > 1 hour in massive rock	5 - 3	0.5 - 0.65	5 - 50
	Slabbing and rock burst after a few minutes in massive rock	3 - 2	0.65 - 1	50 - 200
	Heavy rock burst (strain burst) and immediate dynamic deformation in massive rock	< 2	> 1	200 - 400
Notes: *(ii)* *(iii)*	*For strongly anisotropic stress field (if measured): when 5 < σ_1/σ_3 <10, reduce σ_c to 0.75 σ_c. When $\sigma 1/\sigma 3$ > 10, reduce σ_c to 0.5 σ_c* *Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for low stress cases*		σ_θ / σ_c	

C. Squeezing rock	*Plastic flow of incompetent rock under the influence of high pressure*	Mild squeezing rock pressure	1 - 5	5 - 10
		Heavy squeezing rock pressure	> 5	10 - 20
D. Swelling rock	*Chemical swelling activity depending on presence of water*	Mild swelling rock pressure		5 - 10
		Heavy swelling rock pressure		10 - 15

Table 10 : Table for Q-index rating, Barton

Bolt	RQD	Jn	Jr	Ja	Jw	SRF	Q	GSI from Q
1	50	6	1,5	1	1	**2,5**	**5,0**	67
2	65	2	1,5	1	1	**2,5**	**19,5**	79
3	59	3	1,5	1	1	**2,5**	**11,8**	74
4	36	6	1,5	1	1	**2,5**	**3,6**	64
5	100	3	1,5	1	1	**2,5**	**20,0**	79
6	77	6	1,5	1	1	**2,5**	**7,7**	71
7	100	6	1,5	1	1	**2,5**	**10,0**	73
8	94	6	1,5	1	1	**2,5**	**9,4**	72
9	76	9	1,5	1	1	**2,5**	**5,1**	67
10	53	4	1,5	1	1	**2,5**	**8,0**	71
11	94	4	1,5	3	1	**2,5**	**4,7**	66
12	100	1	1,5	1	1	**2,5**	**60,0**	89
13	40	3	1,5	8	1	**2,5**	**1,0**	52
14	100	0,5	1,5	1	1	**2,5**	**120,0**	95
15	75	1	1,5	1	1	**2,5**	**45,0**	87
16	87	1	1,5	1	1	**2,5**	**52,2**	88
17	100	1	1,5	1	1	**2,5**	**60,0**	89
18	100	3	1,5	1	1	**2,5**	**20,0**	79
19	90	3	1,5	1	1	**2,5**	**18,0**	78
20	30	6	1,5	3	1	**2,5**	**1,0**	52
21	60	6	1,5	1	1	**2,5**	**6,0**	68
22	100	5	1,5	1	1	**2,5**	**12,0**	75
23	90	4	1,5	1	1	**2,5**	**13,5**	76
24	80	4	1,5	1	1	**2,5**	**12,0**	75
25	100	4	1,5	1	1	**2,5**	**15,0**	77
26	100	4	1,5	1	1	**2,5**	**15,0**	77
27	100	4	1,5	1	1	**2,5**	**15,0**	77
28	95	5	1,5	1	1	**2,5**	**11,4**	74
29	70	4	1,5	1	1	**2,5**	**10,5**	73
30	100	6	1,5	1	1	**2,5**	**10,0**	73
31	10	15	1,5	4	1	**2,5**	**0,1**	32
32	100	6	1,5	1	1	**2,5**	**10,0**	73
33	100	4	1,5	1	1	**2,5**	**15,0**	77
35	100	12	1,5	1	1	**2,5**	**5,0**	67
36	30	9	1,5	1	1	**2,5**	**2,0**	58
38	10	15	1,5	3	1	**2,5**	**0,1**	34
39	10	15	1,5	3	1	**2,5**	**0,1**	34
40	30	6	1,5	3	1	**2,5**	**1,0**	52
41	30	6	1,5	3	1	**2,5**	**1,0**	52
42	30	6	1,5	3	1	**2,5**	**1,0**	52
43	40	9	1,5	2	1	**2,5**	**1,3**	55
44	40	9	1,5	2	1	**2,5**	**1,3**	55
45	15	12	1,5	2	1	**2,5**	**0,4**	43
46	15	12	1,5	2	1	**2,5**	**0,4**	43
47	15	12	1,5	2	1	**2,5**	**0,4**	43
48	15	12	1,5	2	1	**2,5**	**0,4**	43
52	100	9	1,5	1	1	**2,5**	**6,7**	69
53	42	6	1,5	1	1	**2,5**	**4,2**	65
1mA	100	1	1,5	1	1	**2,5**	**60,0**	89
1mB	100	1	1,5	1	1	**2,5**	**60,0**	89

Table 11 : Q rating for the 50 bolts in Verdalskalk test 2012

B.1.2. RMR

INPUT PARAMETERS TO RMR_{1989}

(from Bieniawski, 1989)

	PARAMETER		Range of values // RATINGS				
1	Strength of intact rock material	Point-load strength index	> 10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range uniaxial compr. strength is preferred
		Uniaxial com-pressive strength	> 250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa / 1 - 5 MPa / < 1 MPa
		RATING	**15**	**12**	**7**	**4**	**2** / **1** / **0**
2	Drill core quality RQD		90 - 100%	75 - 90%	50 - 75%	25 - 50%	< 25%
		RATING	**20**	**17**	**13**	**8**	**5**
3	Spacing of discontinuities		> 2 m	0.6 - 2 m	200 - 600 mm	60 - 200 mm	< 60 mm
		RATING	**20**	**15**	**10**	**8**	**5**
4	Condition of discon-tinuities	Length, persistence	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
		Rating	**6**	**4**	**2**	**1**	**0**
		Separation	none	< 0.1 mm	0.1 - 1 mm	1 - 5 mm	> 5 mm
		Rating	**6**	**5**	**4**	**1**	**0**
		Roughness	very rough	rough	slightly rough	smooth	slickensided
		Rating	**6**	**5**	**3**	**1**	**0**
		Infilling (gouge)	none	Hard filling		Soft filling	
			-	< 5 mm	> 5 mm	< 5 mm	> 5 mm
		Rating	**6**	**4**	**2**	**2**	**0**
		Weathering	unweathered	slightly w.	moderately w.	highly w.	decomposed
		Rating	**6**	**5**	**3**	**1**	**0**
5	Ground water	Inflow per 10 m tunnel length	none	< 10 litres/min	10 - 25 litres/min	25 - 125 litres/min	> 125 litres /min
		$p_w / \sigma 1$	0	0 - 0.1	0.1 - 0.2	0.2 - 0.5	> 0.5
		General conditions	completely dry	damp	wet	dripping	flowing
		RATING	**15**	**10**	**7**	**4**	**0**

p_w = joint water pressure; $\sigma 1$ = major principal stress

RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS

		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
RATINGS	Tunnels	**0**	**-2**	**-5**	**-10**	**-12**
	Foundations	**0**	**-2**	**-7**	**-15**	**-25**
	Slopes	**0**	**-5**	**-25**	**-50**	**-60**

ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 - 81	80 - 61	60 - 41	40 - 21	< 20
Class No.	I	II	III	IV	V
Description	**VERY GOOD**	**GOOD**	**FAIR**	**POOR**	**VERY POOR**

MEANING OF ROCK MASS CLASSES

Class No.	I	II	III	IV	V
Average stand-up time	10 years for 15 m span	6 months for 8 m span	1 week for 5 m span	10 hours for 2.5 m span	30 minutes for 1 m span
Cohesion of the rock mass	> 400 kPa	300 - 400 kPa	200 - 300 kPa	100 - 200 kPa	< 100 kPa
Friction angle of the rock mass	< 45°	35 - 45°	25 - 35°	15 - 25°	< 15°

Table 12 : Table for RMR rating, Bienawski, 1989

Bolt	RQD5	Spacing of discontinuities	Conditions of discontinuities	Ground water	Strength intact rock material	RMR_{RQD5}
1	20	8	23	10	8	**69**
2	17	8	23	10	8	**66**
3	13	5	23	10	8	**59**
4	17	8	23	10	8	**66**
5	20	10	23	10	8	**71**
6	17	8	23	10	8	**66**
7	20	10	23	10	8	**71**
8	20	10	23	10	8	**71**
9	17	8	23	10	8	**66**
10	17	8	23	10	8	**66**
11	20	15	20	10	8	**73**
12	20	15	23	10	8	**76**
13	8	5	8	10	8	**39**
14	20	15	23	10	8	**76**
15	17	8	15	10	8	**58**
16	20	10	20	10	8	**68**
17	20	15	23	10	8	**76**
18	20	15	23	10	8	**76**
19	20	15	23	10	8	**76**
20	8	5	8	10	8	**39**
21	13	10	23	10	8	**64**
22	20	15	23	10	8	**76**
23	20	15	23	10	8	**76**
24	17	10	23	10	8	**68**
25	20	15	23	10	8	**76**
26	20	15	23	10	8	**76**
27	20	15	23	10	8	**76**
28	20	15	23	10	8	**76**
29	13	8	23	10	8	**62**
30	20	15	23	10	8	**76**
31	5	5	15	10	8	**43**
32	20	15	18	10	8	**71**
33	20	15	18	10	8	**71**
35	20	15	20	10	8	**73**
36	8	8	20	10	8	**54**
38	5	8	20	10	8	**51**
39	5	8	20	10	8	**51**
40	8	8	20	10	8	**54**
41	8	10	23	10	8	**59**
42	8	8	20	10	8	**54**
43	8	8	20	10	8	**54**
44	8	8	20	10	8	**54**
45	5	8	20	10	8	**51**
46	5	8	20	10	8	**51**
47	5	8	20	10	8	**51**
48	5	8	20	10	8	**51**
52	20	10	23	10	8	**71**
53	17	5	23	10	8	**63**
1mA	20	15	23	10	8	**76**
1mB	20	15	23	10	8	**76**

Table 13 : RMR rating for the 50 bolts in Verdalskalk test 2012

B.1.3. GSI and parameters for shear strength of discontinuities after Wyllie

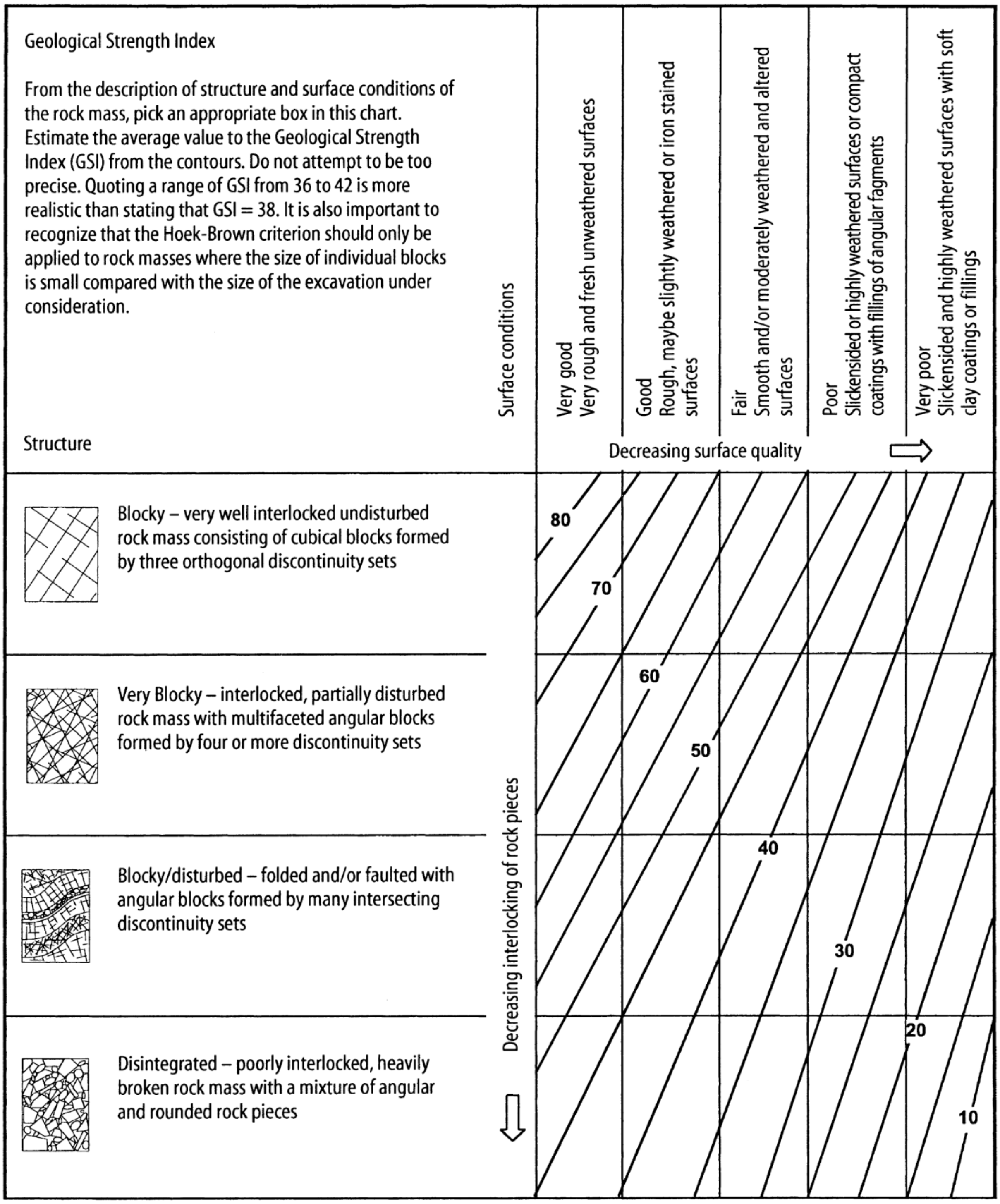


Table 14 : Table for GSI rating, characterization of rock masses on the basis of interlocking and joint alteration (Hoek and Brown 1998 adjusted from Hoek 1994)

B.1.4. Parameters for shear strength of discontinuities after Barton

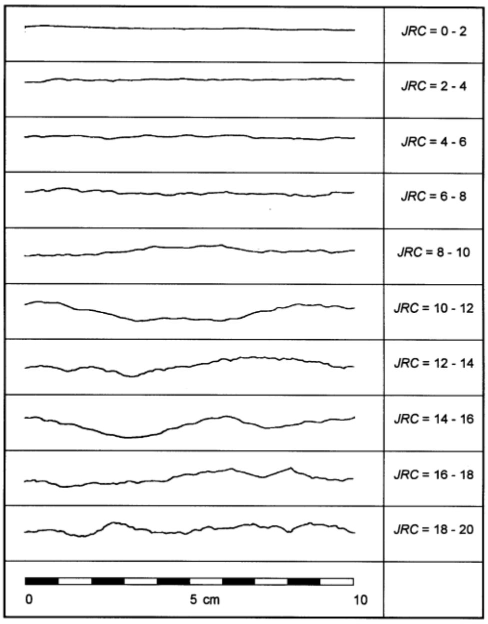


Table 15 : Roughness profiles and corresponding *JRC* values (After Barton and Choubey 1977)

JRC can also be estimated from Jr value in Q index.
Correction of JRC is proposed by Barton with regards to the scale effect of the sample size.

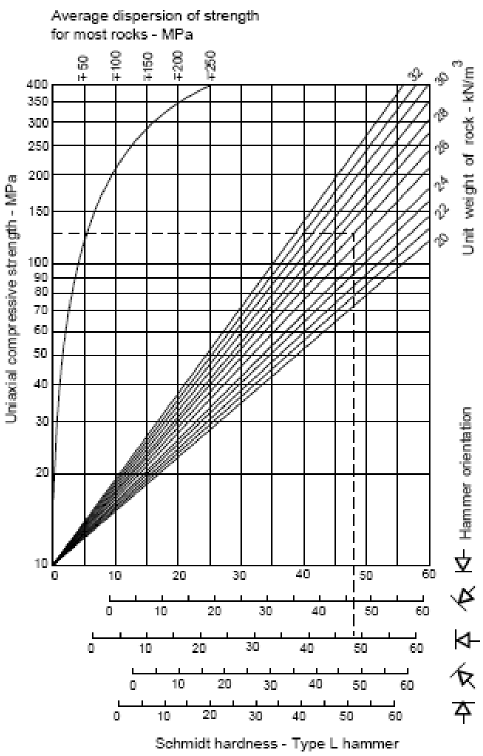


Figure 61 : Estimation of joint wall compressive strength from Schmidt hardness

Correction of JCS is proposed by Barton with regards to the scale effect of the sample size.

B.2. CALCULATION OF CAPACITY OF PASSIVE ROCK BOLTS

B.2.1. NVE

Grouted length L

Grouted length L is the maximum of grouted length with regard to rock-mortar bond strength and mortar-steel bond strength :

$$L = \frac{d_b}{4} \cdot \frac{f_{sk}}{f_{bb}} \qquad\qquad L = \frac{d_b}{4} \cdot \frac{f_{sk}}{f_{bf}} \cdot \frac{d_b}{d_h}$$

d_b	boltediameter	25mm
d_h	borhullsdiameter (min. 5 mm klaring til borhullsvegg)	45mm
f_{sk}	stålets flytegrense	400MPa
f_{bb}	dimensjonerende heftstyrke mellom stål og mørtel	3/γm= 3/2=1,5MPa
f_{bf}	dimensjonerende heftstyrke mellom mørtel og fjell	Fbf/ γm with fbf from NVE table and γm=2

Table 16 : Values for calculations of grouted length L, representative to performed full scale tests and practice in Norway

Bergart	Heftstyrke til mørtel [MPa] (NVE)	Heftstyrke til mørtel [MPa] (Brendeberg)
Granitt	2,0	10,0
Gabbro	2,5	
Gneis	1,5	7,0
Sandstein	1,2	3,0
Kalkstein	2,0	5,0
Leirskifter	0,5	
Kvartsitt	2,5	

Table 17 : Bond strength value from different type of rocks

Position of the center of the grouted length, depth D of the cone of rock

F = γ.V = 1/3. γ.π.D^3

D calculated for F = 180MPa/(π.12,5^2) = 88kN

B.2.2. Shear strength after Wyllie

Input parameter σc has value 90MPa (lab test). The input parameters for the rock mass conditions m and s come either from the table from Hoek and Brown (used for calculations from Lars Kristian Neby) or from the formel from Hoek and Brown (used for the 50 bolts tested in this thesis).

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values — *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION

$$\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$$

σ_1' = major principal effective stress
σ_3' = minor principal effective stress
σ_c = uniaxial compressive strength of intact rock, and
m and s are empirical constants.

		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS – *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES						
Laboratory size specimens free	m	7.00	10.00	15.00	17.00	25.00
from discontinuities	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
NGI rating: Q = 500	*s*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
VERY GOOD QUALITY ROCK MASS						
Tightly interlocking undisturbed rock	m	2.40	3.43	5.14	5.82	8.56
with unweathered joints at 1 to 3m.	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
NGI rating: Q = 100	*s*	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS						
Fresh to slightly weathered rock, slightly	m	0.575	0.821	1.231	1.395	2.052
disturbed with joints at 1 to 3m.	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
NGI rating: Q = 10	*s*	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*
FAIR QUALITY ROCK MASS						
Several sets of moderately weathered	m	0.128	0.183	0.275	0.311	0.458
joints spaced at 0.3 to 1m.	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
NGI rating: Q = 1	*s*	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*
POOR QUALITY ROCK MASS						
Numerous weathered joints at 30-500mm,	m	0.029	0.041	0.061	0.069	0.102
some gouge. Clean compacted waste rock	s	0.000003	0.000003	0.000003	0.000003	0.000003
CSIR rating: RMR = 23	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
NGI rating: Q = 0.1	*s*	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS						
Numerous heavily weathered joints spaced	m	0.007	0.010	0.015	0.017	0.025
<50mm with gouge. Waste rock with fines.	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
CSIR rating: RMR = 3	*m*	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
NGI rating: Q = 0.01	*s*	*0.00002*	*0.00002*	*0.00002*	*0.00002*	*0.00002*

Table 18 : Approximate relationship between rock mass quality and material constant, after Hoek and Brown, 1988

The formel from Hoek were used : $m = m_i . e^{((GSI-100)/28)}$ and $s = e^{((GSI-100)/9)}$ with mi from the table below and GSI = RMR_{RQD5}-24 in the present research.

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerate (22)	Sandstone 19 Greywacke (18)	Siltstone 9	Claystone 4
	Non-clastic	Organic		Chalk 7 Coal (8–21)		
		Carbonate	Breccia (20)	Sparitic limestone (10)	Micritic limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
Metamorphic	Non-foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25–31	Mylonites (6)	
	Foliated[a]		Gneiss 33	Schists 4–8	Phyllites (10)	Slate 9
Igneous	Light		Granite 33 Granodiorite (30) Diorite (28)		Rhyolite (16) Dacite (17) Andesite 19	Obsidian (19)
	Dark		Gabbro 27 Norite 22	Dolerite (19)	Basalt (17)	
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

[a] These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane

Table 19 : Values of the constant m_i for intact rock, by rock group

Note that values in parenthesis are estimates

The results from Wyllie method are considered in this research not very reliable because of inaccuracy in determination of GSI and consequent large influence in the formal to deduce rock mass constants from GSI. Constants m and s can be determined with higher accuracy from triaxial tests, after [17].

B.3. ISRM TEST OF ROCK PROPERTIES FROM VERDALSKALK, LARS K. NEBY 2011

ISRM, Commission on Testing Methods :

"WHAT IS THE PURPOSE OF ISRM SUGGESTED METHODS?
The purpose of ISRM Suggested Methods is to offer guidance for rock characterization procedures, laboratory and field testing and monitoring in rock engineering. These methods provide a definitive procedure for the identification, measurement and evaluation of one or more qualities, characteristics or properties of rocks or system that produces a test result.

WHAT IS ISRM SUGGESTED METHODS?
Rock mechanics is a young subject in comparison with related engineering sciences such as soil mechanics. Engineering knowledge in this area has been largely uncoordinated with each individual group of engineers developing their own methods and experience outside the framework of an established academic and professional discipline. Since the establishment of ISRM, there has been a coalescence of previously disparate undertakings into a single rock mechanics discipline, an improvement in communication between engineers in the field and a very marked expansion in research and publication. A key element in this expansion has been the need to develop a common terminology for rock properties and tests by which they are measured, so that if, for example, an engineer in one country were to describe a rock as "strong", a colleague in another country or field of application would understand and correctly interpret his description (Brown, 2011). This need was recognized by ISRM and ISRM Commission on Testing Methods was established. The Commission aims to generate and publish Suggested Methods (SMs) for testing or measuring properties of rocks and rock masses, as well as for monitoring performance of rock engineering structures. The term 'Suggested Method' has been carefully chosen: these are not standards *per se*; they are explanations of recommended procedures to follow in the various aspects of rock characterization, testing and monitoring. The "ISRM Suggested Methods" is a document that has been developed and established within the consensus principles of the ISRM and that meets the approval requirements of ISRM procedures and regulations. If someone has not been involved with a particular subject before and if this subject is part of a Suggested Method, they will find the guidance to be most helpful. The Suggested Methods can be used as standards on a particular project if required, but they are intended more as guidance."

Intact rock properties from Verdalskalk with ISRM methods are presented below :

SINTEF **TESTRAPPORT**

Densitet og lydhastighet

SINTEF Byggforsk
Geologi og bergteknikk

GRUNNLAG: ***ISRM Suggested Method for Determining Sound Velocity***

Oppdragsgiver: Lars Kristian Neby Dato: 05.05.11
Prosjektnummer: 11030BM
Saksbehandler i lab: Kjartan

Bergart: Kalksten
Prøvemerkning: 0
Kjernediameter: 38,5 mm
Belastningsretning: Uspesifisert
Vanninnhold: Uspesifisert

Kjerne nr.	Romvekt [kg/m^3]	Lydhastighet [m/s]
1	2715	4673
2	2752	5003
3	2726	4190
4	2680	3178
5	2693	5248
Gjennomsnitt:	2713	4458
Standardavvik:	28,3	818,2

SINTEF

TESTRAPPORT

E-modul og Poissons forhold

SINTEF Byggforsk
Geologi og bergteknikk

GRUNNLAG: ***ISRM Suggested Method for Determining Deformability of Rock Materials in Uniaxial Compression***

Oppdragsgiver:	Lars Kristian Neby	Dato:	05.05.2011
Prosjektnummer:	11030BM		
Saksbehandler i lab:	Kjartan		

Bergart:	Kalksten
Prøvemerkning:	0
Kjernediameter:	38,5 mm
Belastningsretning:	Uspesifisert
Vanninnhold:	Uspesifisert
E og ν beregnes med:	Tangentmetoden

Kjerne nr.	E-modul [GPa]	Poisson's forhold
1	64,4	0,250
2	61,7	0,290
3	57,5	0,240
4	68,5	0,260
5	57,2	0,230
Gjennomsnitt:	61,8	0,254
Standardavvik:	4,77	0,023

E-modul og Poisson's forhold er beregnet ut i fra tangent metoden.

SINTEF

TESTRAPPORT

Trykkfasthet

SINTEF Byggforsk
Geologi og bergteknikk

GRUNNLAG: ***ISRM Suggested Method for Determination of the Uniaxial Compressive Strength of Rock Materials***

Oppdragsgiver:	Lars Kristian Neby	Dato:	05.05.11
Prosjektnummer:	11030BM		
Saksbehandler i lab:	Kjartan		

Bergart:	Kalksten
Prøvemerkning:	0
Kjernediameter:	38,5 mm
Belastningsretning:	Uspesifisert
Vanninnhold:	Uspesifisert

Kjerne nr.	Trykkfasthet [MPa]	Bruddvinkel
1	104,2	24
2	81,7	24
3	100,0	29
4	106,0	20
5	80,5	24
Gjennomsnitt:	94,5	24,200
Standardavvik:	12,41	3,194

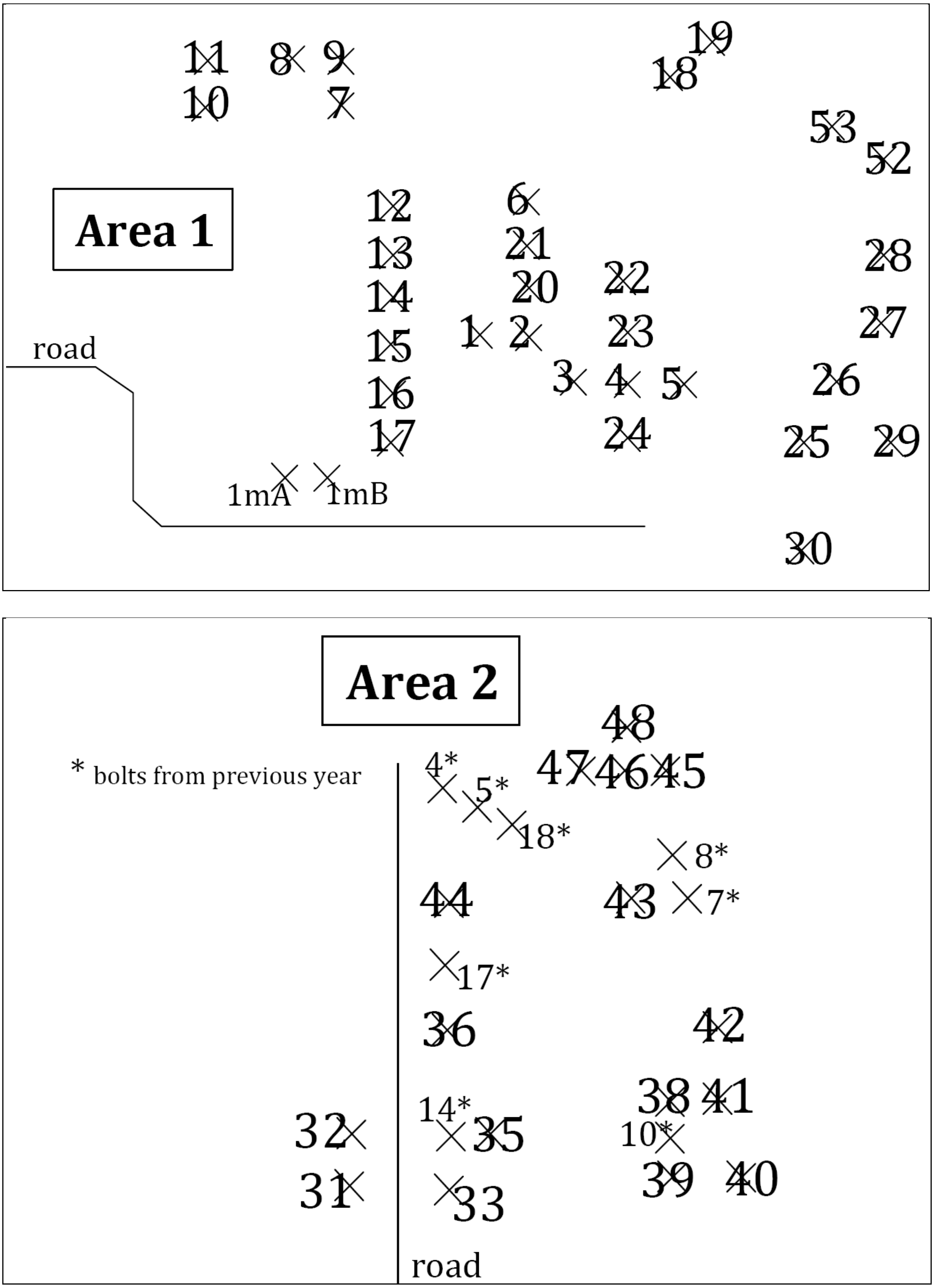
Table 20 : Intact rock properties in Verdalskalk, results from Lars K. Neby, 2011

B.4. BOLTS DOCUMENTATION

Fifty bolts were tested.
They can be divided in three groups with regards to the results of their pull out testing.

no break (18 bolts)	
18 bolts above 19 tons	4, 6, 10, 12, to 19, 14bis, 21 to 24, 28 and 29
break of the rock mass (21 bolts)	
7 bolts above 19 tons 10 bolts between 8,95 and 18 tons 4 bolts under 4,85 tons (crushed or loose block.)	1, 7, 17bis, 25, 33, 40, 41 3, 8, 9, 11, 35, 36, 43, 44, 45, 46 38,39,42,52
Break of mortar (11 bolts)	
(9 bolts) between 4,85 and 11 tons 2 bolts about 17 tons	2, 5, 30, 32, 47, 48, 53, 20, 31 26, 27

The bolts tested in 2012 in Verdalskalk are presented below from the lowest to the highest supported capacity in three parts : rock mass break, mortar break and no break.

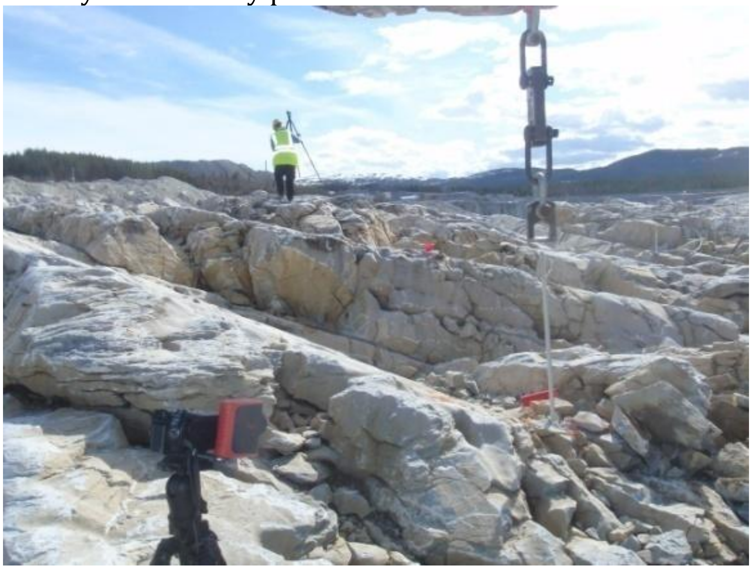


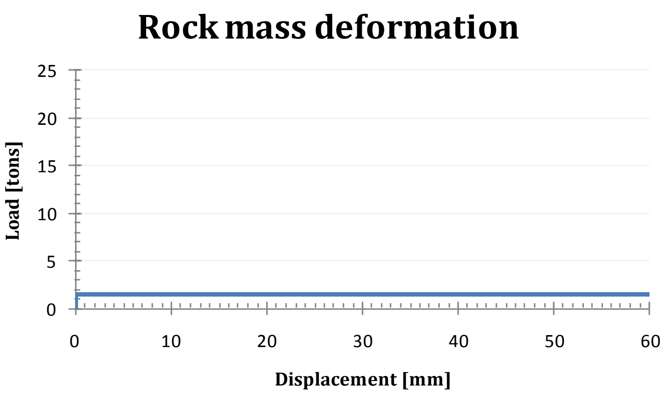
Location of bolts

B.4.1. Rock Mass Break

Bolt 39 : 1 ton, 0,2m

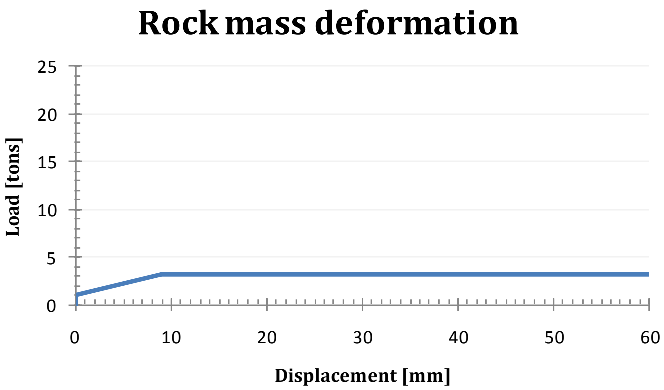
Crushed and shallow conditions. Slowly and steadily pull out.





BOLT 38 : 3 TON, 0,2M

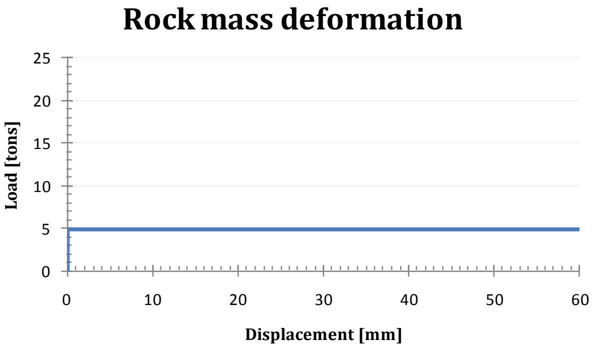
Crushed and shallow conditions. Slowly and steadily pull out.



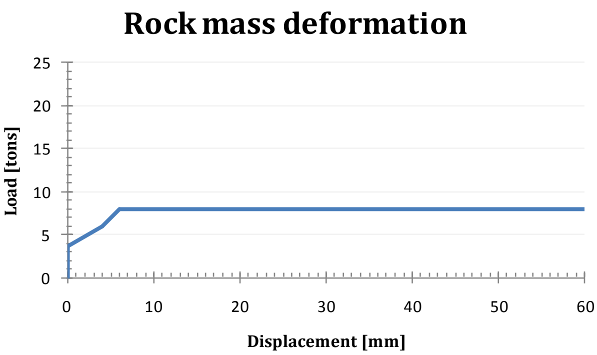
BOLT 42 : 1 TON, 0,4M

BOLT 52 : 5 TONS, 0,4M

loose block, quickly and steadily released.

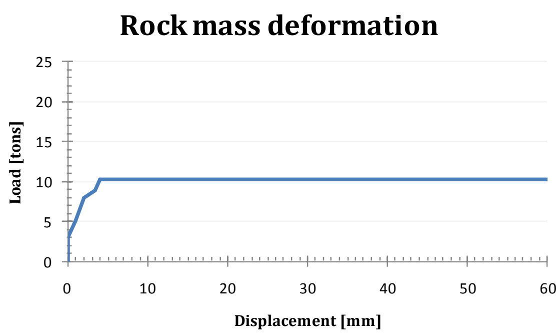


BOLT 46 : 9 TONS, 0,3M



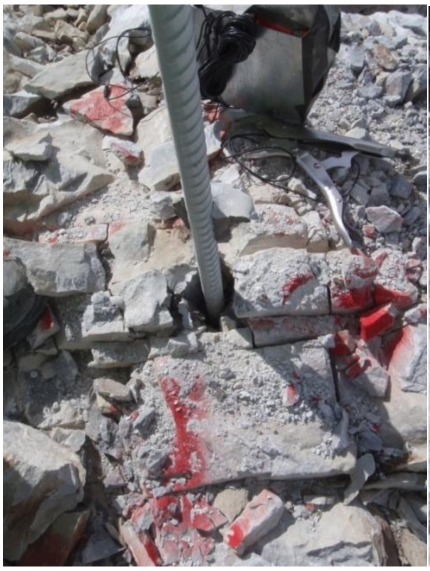
BOLT 8 : 10 TONS, 0,4M

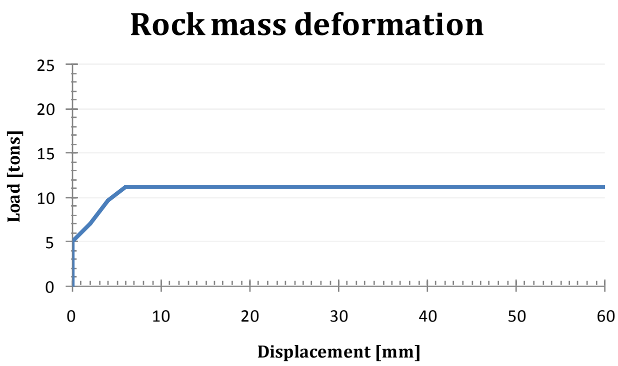




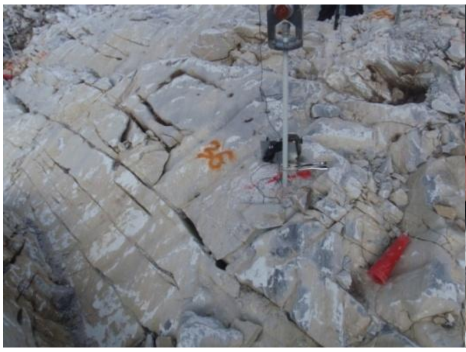
BOLT 35 : 11 TONS, 0,3M

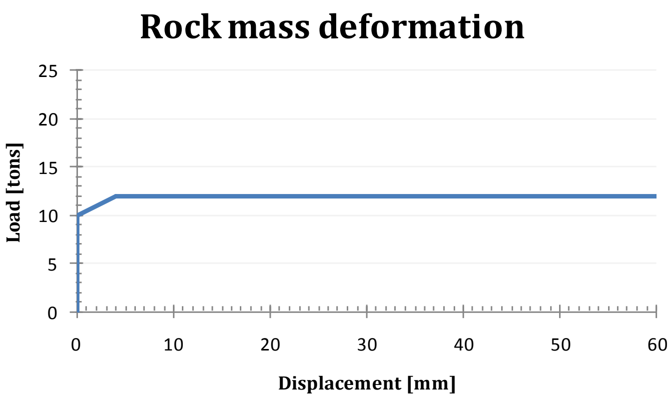
Bolt 14 from previous year was tested again after B35 broke. It was loaded up to without break. The bolts were distant from less than 0,4m

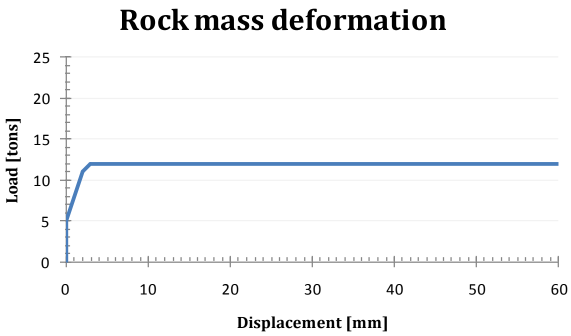


BOLT 36 : 12 TONS, 0,3M

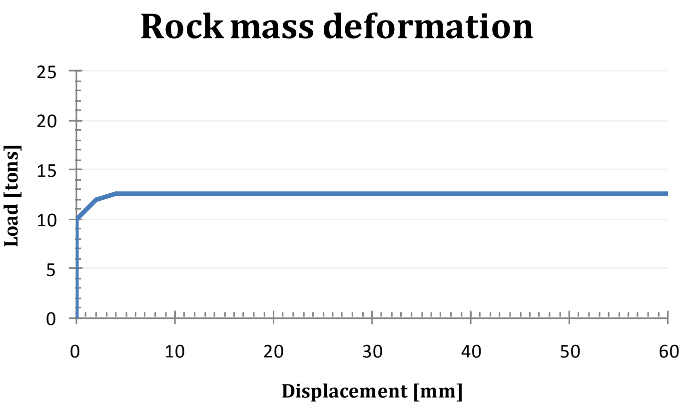




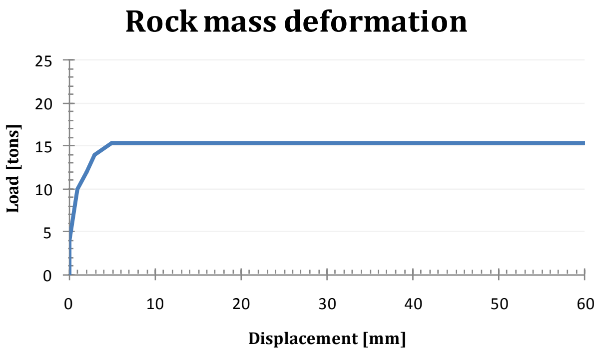
BOLT 3 : 12 TONS, 0,5M



BOLT 45 : 13 TONS, 0,3M

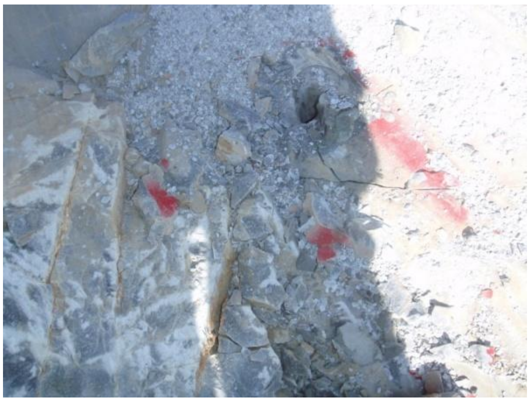


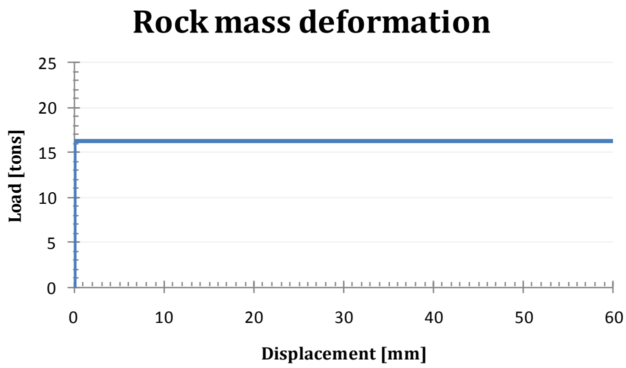
BOLT 9 : 14 TONS, 0,4M



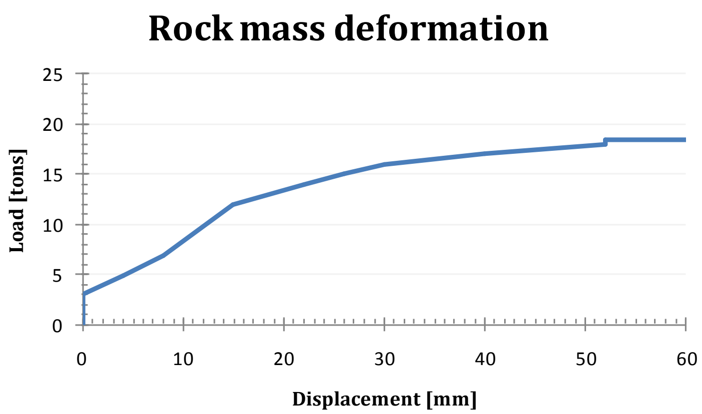
BOLT 44 : 15 TONS, 0,3M

BOLT 43 : 16 TONS, 0,3M

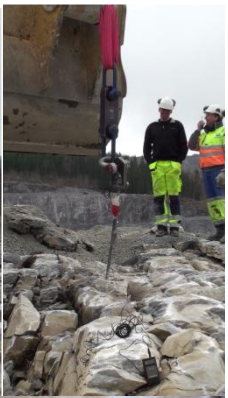


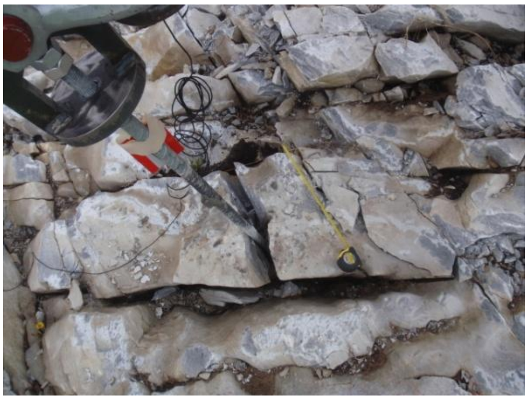


BOLT 11 : 18 TONS, 0,5M

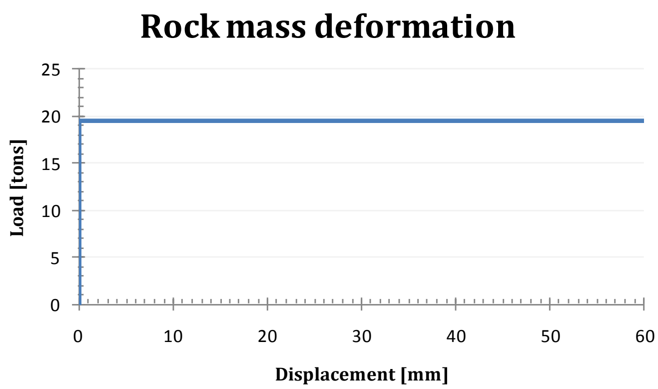


BOLT 17*GROUTED PREVIOUS YEAR : 19 TONS, 0,8M

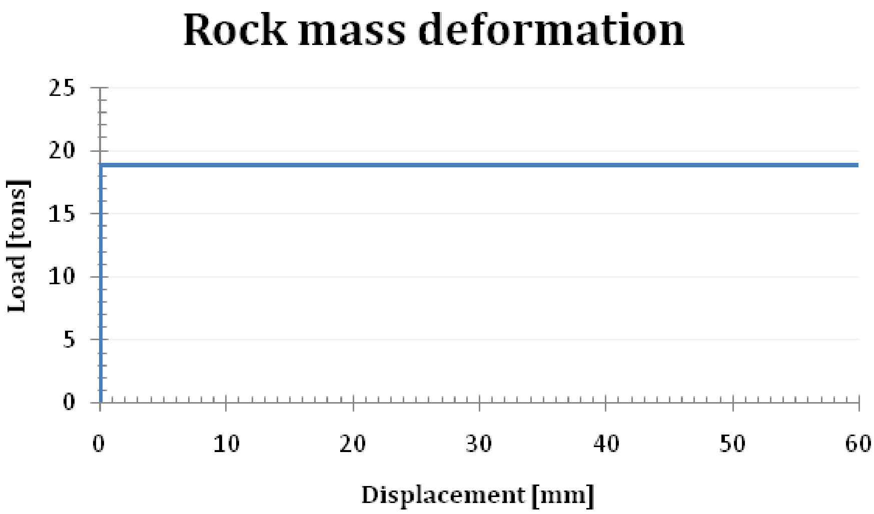




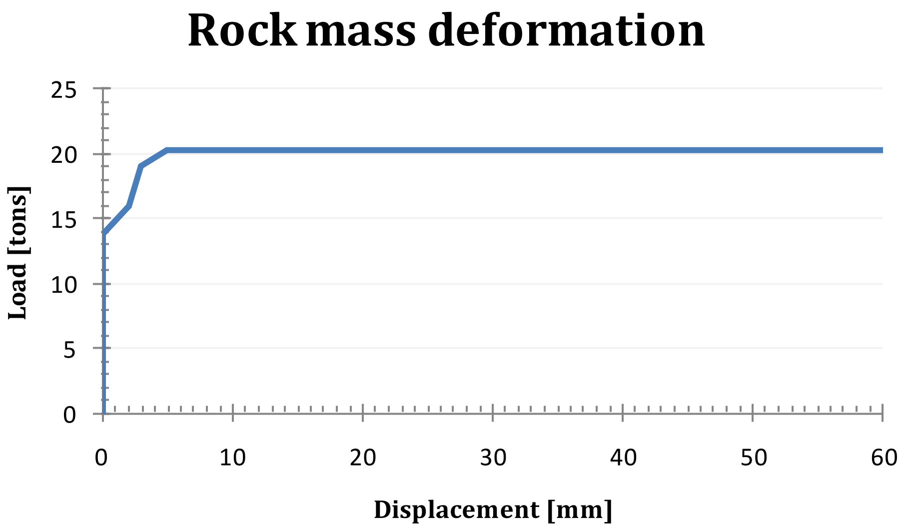
BOLT 7 : 20 TONS, 0,3M



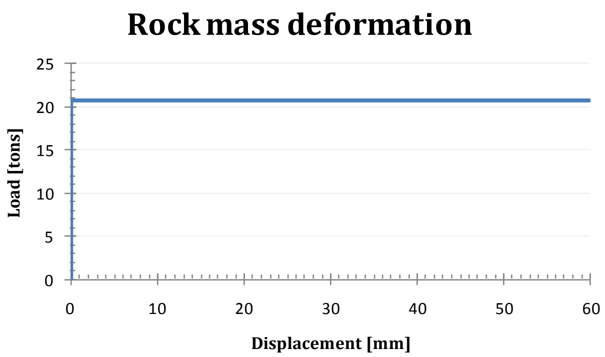
BOLT 33 : 20 TONS, 0,3M



BOLT 40 : 20 TONS, 0,3M

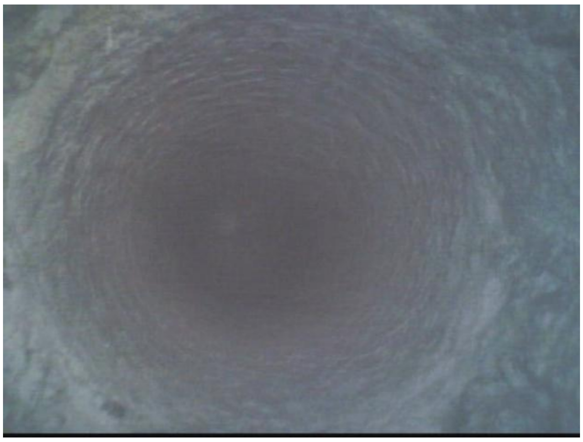


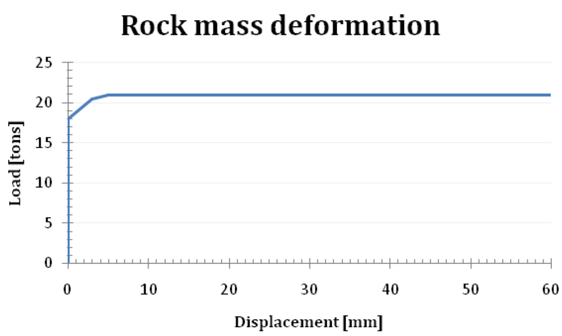
BOLT 1 : 21 TONS, 0,4M



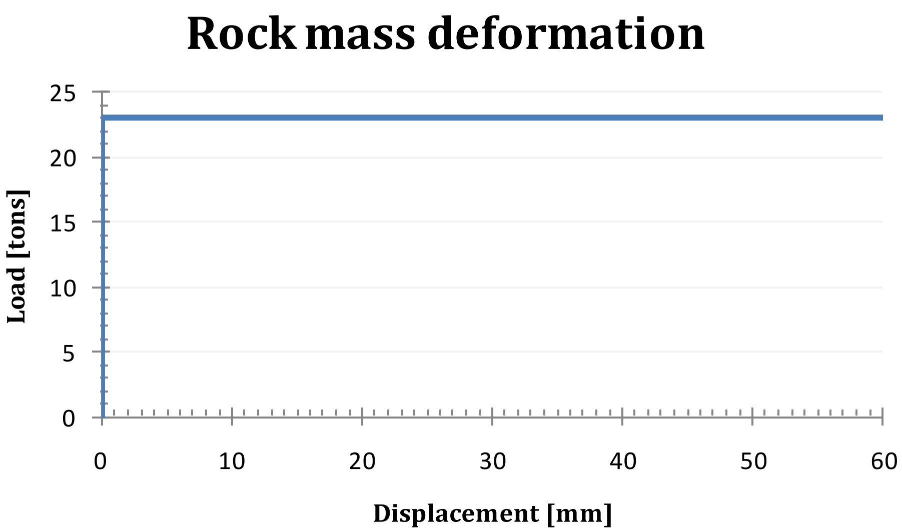
BOLT 25 : 21 TONS, 0,5M

Mortar was not dry, but yet rock broke first at 21 tons





BOLT 41 : 23 TONS, 0,3M



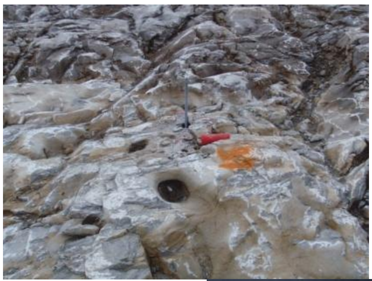
B.4.2. MORTAR BREAK

Rock mass remains intact, and mortar break between 4,85 tons and 17 tons. These bolts give a minimum value of the capacity os the rock mass, which is with a minimum of 5 tons still higher than the conservative design expecting 2 tons from the rock mass resistance. The bolts are presented from the lowest to the highest loading before break.

BOLT 1MA AND 1MB - GRAVEL "GROUTING" :

1TON, 0,5 AND 1M.

BOLT 31 : 5 TON, 0,3M



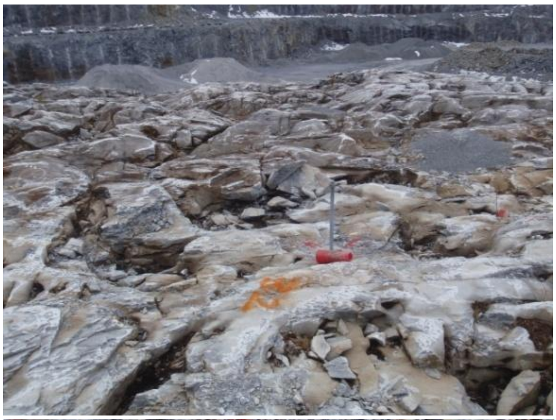
Bolt 20 : 5ton, 0,4m

bolt 53 : 6ton, 0,25m

Bolt 47 : 7tons, 0,35m

Bolt 48 : 8tons, 0,3m

BOLT 32 : 8TONS, 0,3M

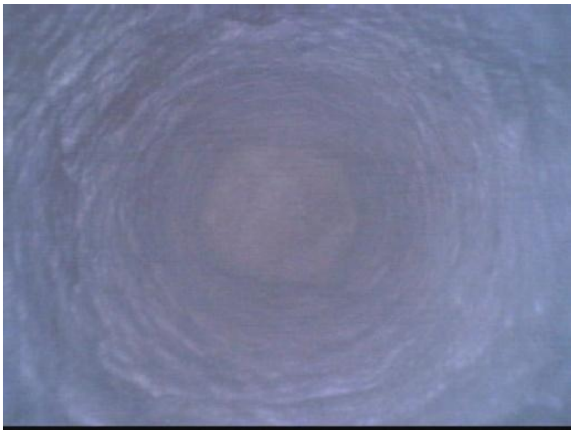



BOLT 5 : 10TONS, 0,25M



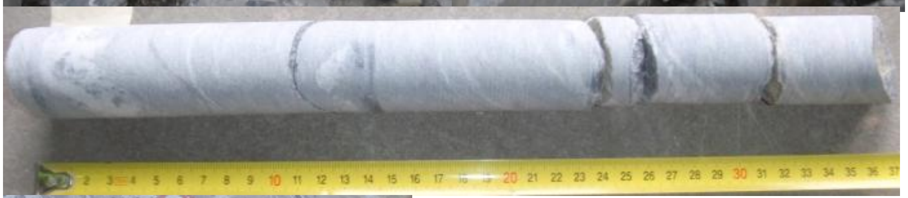

BOLT 30 : 10,5TONS, 0,4M





BOLT 2 : 11TONS, 0,35M





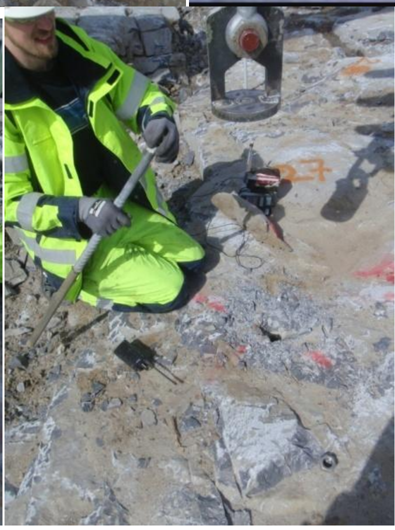

BOLT 26 : 17TONS, 0,4M





Bolt 27 : 17 tons, 0,4m

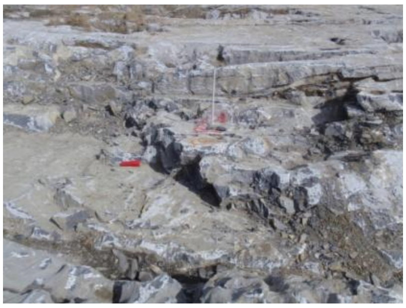


B.4.2. No Break (or steel break)

Bolt 4, 22tons, 0,5m

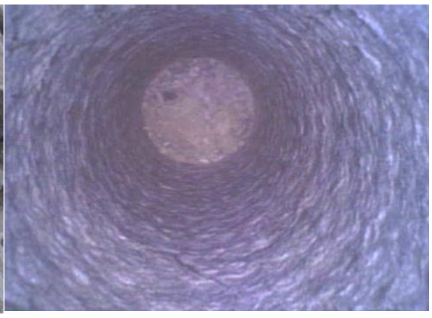
Bolt 6, 23tons, 0,4m



Bolt 10, 22tons, 0,3m

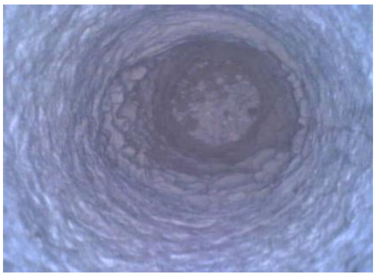
BOLT 12, 13, 14, 15, 16 AND 17 : 19-23TONS, 0,4-0,8M


Bolts 12 to 17 were grouted in the same fair location with length 0,4m. Exception made to bolt 13 : a large fault was meet while drilling from 0,4m depth to 0,8m depth. The grouting of bolts 13 was performed in 2 times, as mortar was sinking into the faults.None of the bolts in that area broke.

Bolt 12 


Bolt 13 


Bolt 14 

Bolt 15  

Bolt 16   

Bolt 17  

<table>
<tr>
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Rock mass conditions : Fair on 0,4m length. One exception with longer bolt 13, who meets the faults for 0,4m long under the fair rock mass.
Testing : No break nor deformation in rock mass for maximal loading from 19,5 tons to 26 tons.

</td>
<td>

Break of the steel at the head of the bolt for loading higher than 23 tons
</td>
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</table>

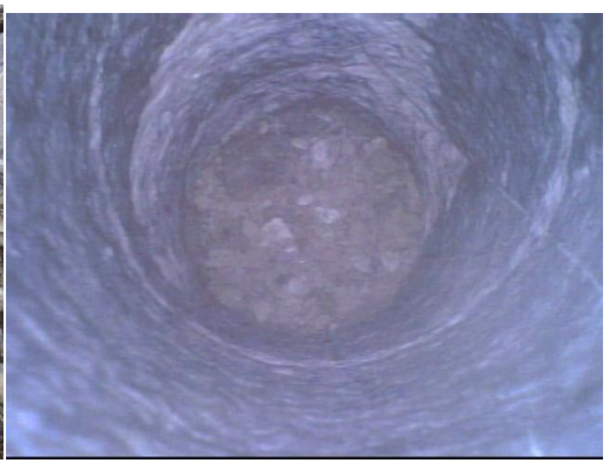
Bolt 18 : 19tons, 0,4m



Bolt 19 : 20 tons, 0,4m



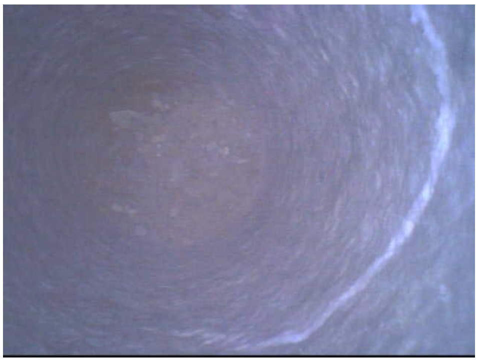
Bolt 21 : 22 tons, 0,5m

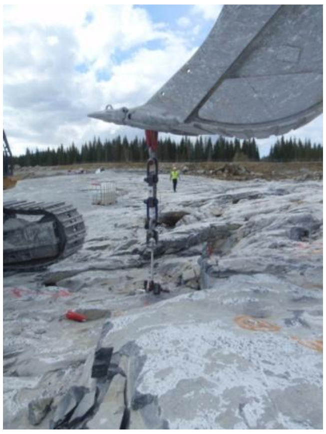



Bolt 22 : 22tons, 0,5m

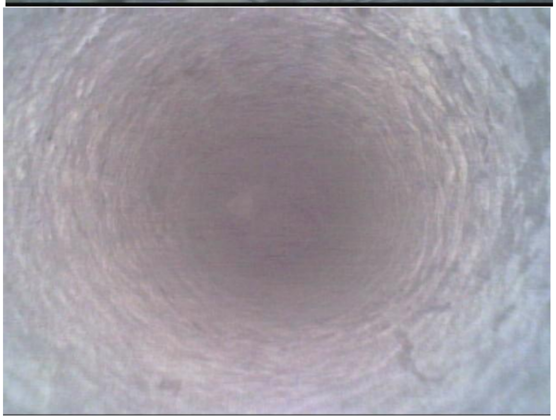
Bolt 23 : 21tons, 0,4m

Bolt 24 : 21tons, 0,4m

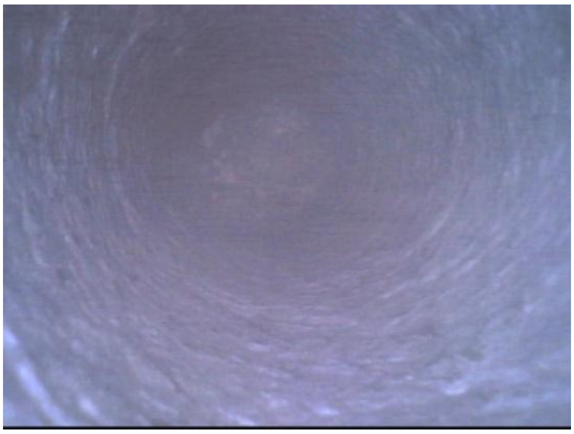




Bolt 28 : 20tons, 0,3m



Bolt 29 : 21tons, 0,5m



Bolt 14 from 2011

See bolt 35 in the section "Rock mass break"