ROCK MASS QUALITY Q USED IN DESIGNING REINFORCED RIBS OF SPRAYED CONCRETE AND ENERGY ABSORPTION

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SUMMARY

A second updating of the Q-system for rock mass classification (1st updating: Grimstad and Barton, 1993) is in progress. In this updating emphasis has been placed on the wide application of fibre reinforced sprayed concrete (Sfr), even in the lowest rock mass qualities, and the recent changes in rock support practice and material properties. In extremely poor rock, where some deformation may be expected, the toughness and the energy absorption of the sprayed concrete has been taken into consideration in the improved Q-support chart.
A substantial amount of data from recent projects has been gathered and analysed. Analytical research has been carried out with respect to the thickness, spacing and reinforcement of reinforced ribs of sprayed concrete (RRS) as a function of the load from the rock and the rock mass quality. The basis for the RRS design is the calculated deformation and bending moment. The analytical calculations are calibrated with numerical analyses and deformation measurements in tunnels. Modern practice indicates that RRS may replace traditional cast concrete lining in extremely poor rock. This has been observed in hundreds of practical cases. An improved Q-support chart that gives details of RRS in very poor to exceptionally poor rock mass is presented.

ENERGY ABSORPTION IN POOR ROCK MASS QUALITY

In a poor rock mass quality there is always some deformation in jointed or weak rock after excavation of an underground opening. When deformations of 2-10cm or more occur in an underground opening with span of 10m, it is of utmost importance that flexible primary layers of sprayed concrete in combination with rock bolts are used to control the deformations of the rock mass without collapse. Hence, it is important to apply shotcrete with high quality fibres, which can deform to a large degree and still contain some residual strength.

Empirical observations have linked the deformations to the variation in rock mass quality, Q [1] and the span or height of the underground opening as seen in Figure 1. For completeness a short description of the rock mass quality, Q is given underneath:
The Q-value is expressed in 6 independent and observable parameters, which are given numerical values defined in a table (See [1] and [2].)

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} \quad \textbf{(1)}$$

RQD = degree of jointing (Rock Quality Designation)
Jn = number for joint sets
Jr = joint roughness number
Ja = joint alteration number
Jw = joint water reduction factor
SRF = Stress Reduction Factor
The figures used for SRF was completely changed during the updating in 1993 [2].

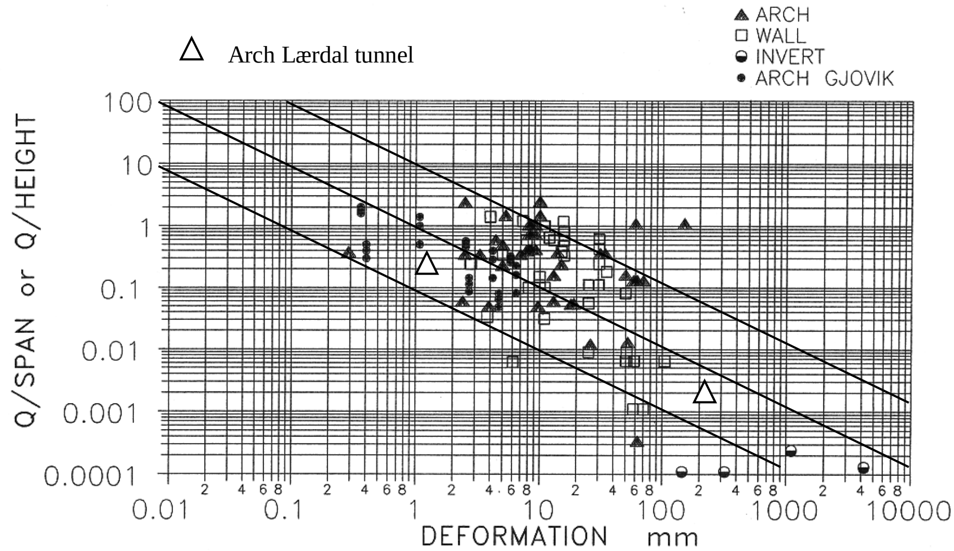


Figure 1. Deformation in rock mass related to rock mass quality, Q and the ratio Q/span or Q/height. [2]

As seen from Figure 1, increased deformations may be expected for low Q-values combined with large spans or heights. Due to this observation the temporary support, and to some extent the permanent support has to be designed to tolerate some deformation. Some researchers, including Bernard [3] have performed plate tests on sprayed concrete for studying the performance of fibres. They have demonstrated that fibre reinforced sprayed concrete (S_{fr}) may be able to take significant load and consume energy during deformation provided that a proper mix of the appropriate fibres and concrete is chosen. A relevant panel test method is described in European Specification for Sprayed Concrete, (EFNARC1996) [4].

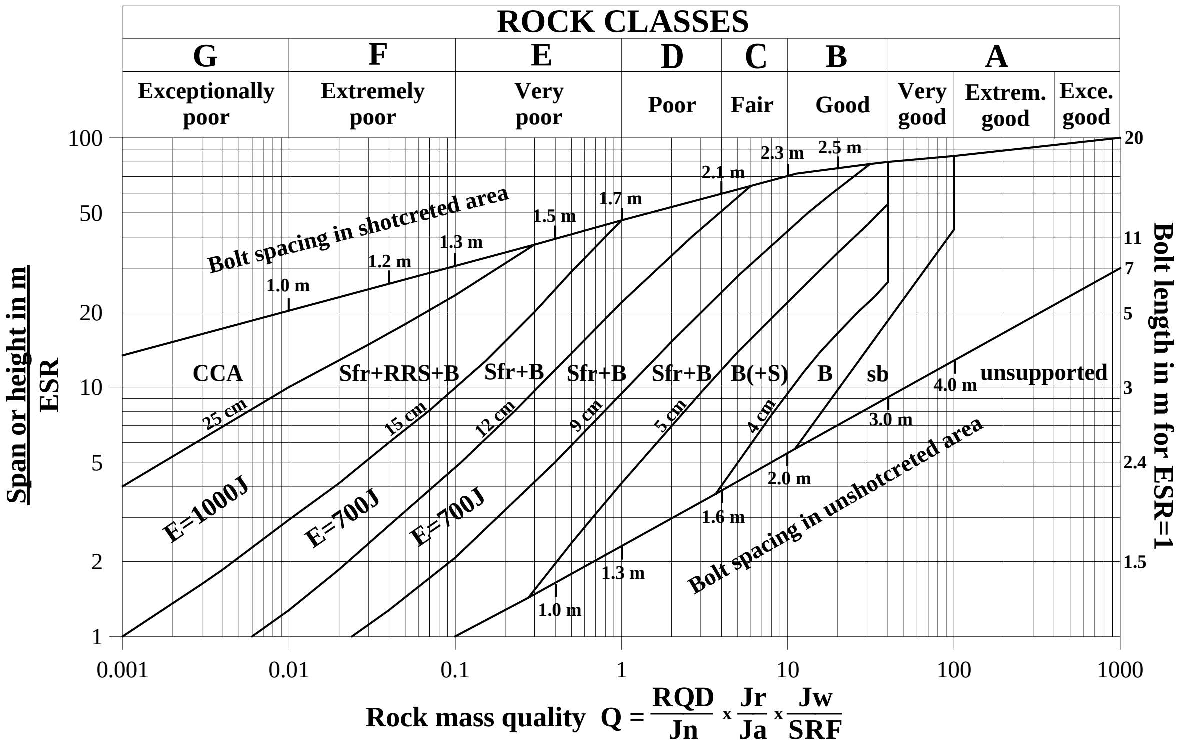
Based on the empirically expected deformations in different rock mass qualities, the energy absorption classes have been included in the support chart of the Q-system [2]. These energy absorption classes in the Q-chart correspond with the toughness classes defined by EFNARC and are given in the guidelines of Norwegian Concrete Association's Publication no. 7 – 1999 [5], see Table 1.

Table 1. Energy absorption classes using the plate test as described in Norwegian Concrete Association Publication no. 7 - 1999. [5]

Energy-absorption class	Documentation		Min. energy-absorption class in Joule for deflection not exceeding 25 millimetres
	In the project	Declaration	
E700		X*	700
E1000	X*		1000

** Explanation given in section 1.2.3.1 in [5]. "In the project" means that toughness/energy absorption of the mix design and the planned method of implementation used in the project must be documented with a certain frequency. "Declaration" implies that the toughness/ absorption energy must be proven through general documentation carried out by an independent test laboratory. The supplier of fibres should document the fibre properties according to appendix A in [5]*

It can be seen from Figure 2 that the variation in Rock mass quality, Q, and the span or height has an almost equivalent influence on the rock support and the energy absorption classes which follow the rock support classes. As an example, a rock mass quality, Q = 1 in a tunnel with 10m span is on the borderline between no specific demand for energy absorption and the energy absorption class E700. This tunnel may have a long-term total deformation in the order of 10 mm with minor support. (Figure 1). A rock mass quality, Q = 0.4 in a tunnel with 10m span is in the reinforcement category 6 in Figure 2. This category is in the energy absorption class E700, and may give approximately 25mm long term deformation with minor support. E700 requires a sprayed concrete, which can give an energy absorption of minimum 700Joule when deflected 25mm in a standard test panel. It should be pointed out that not only the amount of fibres, but to a larger degree the type of fibre is important for the residual flexural strength and the deformation energy. The tests carried out by Bernard [3] show that 35kg/m^3 of superior quality fibre gives almost twice the deformation energy compared to 50kg/m^3 containing a poor type of fibre.



REINFORCEMENT CATEGORIES:
1) Unsupported
2) Spot bolting, **sb**
3) Systematic bolting, **B**
4) Systematic bolting, (and unreinforced shotcrete, 4-10 cm), **B(+S)**
5) Fibre reinforced shotcrete and bolting, 5-9 cm, **Sfr + B**
6) Fibre reinforced shotcrete and bolting, 9-12 cm, **Sfr+B**
7) Fibre reinforced shotcrete and bolting, 12-15 cm, **Sfr+B**
8) Fibre reinforced shotcrete > 15 cm, reinforced ribs of shotcrete and bolting, **Sfr+RRS+B**
9) Cast concrete lining, **CCA**

E) Energy absorption in fibre reinforced shotcrete at 25 mm bending during plate testing

Figure 2. The Q-chart published in 1993 [2] with classes of energy absorption.

When the Q-value is descending down to 0.1 in a 10m wide tunnel the average total long-term deformation with minor support may be in the order of 100mm. In this case the Q-chart recommends a change to the highest energy absorption class E1000.

When the rock mass quality, Q descends to 0.01 in a 10m wide tunnel, the long term deformation without support may be in the order of 1 m, which in reality could be a collapse. In such cases it is important to apply a (E1000) temporary support as flexible as possible, which can control and reduce the deformation without any collapse. The temporary support sometimes has to include some minor reinforced ribs of sprayed concrete, which may deform. Installation of a rigid support, at an early stage, in extremely poor rock mass qualities often gives uncontrolled deformations and collapse. When the controlled deformation speed has decreased to a desired level, the permanent support has to include systematic application of reinforced ribs of sprayed concrete, RRS, which should be designed in accordance with the demand. In some cases CCA may be preferred when the thickness of concrete is large.

REINFORCED RIBS OF SPRAYED CONCRETE (RRS) AS A FUNCTION OF ROCK MASS QUALITY, Q

Need for analysis

During the last 15 years reinforced ribs of sprayed concrete (RRS) have in many cases been installed in Norwegian tunnels instead of the in situ cast concrete lining (CCA). CCA was the common rock support in extremely and exceptionally poor rock (Q= 0.1 – 0.001) earlier. RRS is in shape and support capacity similar to sprayed lattice girders, but are more flexible in application because RRS is not prefabricated.

In most cases RRS have been installed without any analysis of the support pressure or calculations of the capacity of the RRS. Hence, in most cases it can be an over-support with waste of resources. Occasionally, collapses or uncontrolled deformations are experienced because of under support.

Support pressure from the rock mass may be estimated from the Q-value, the roughness number, and the number of joint sets

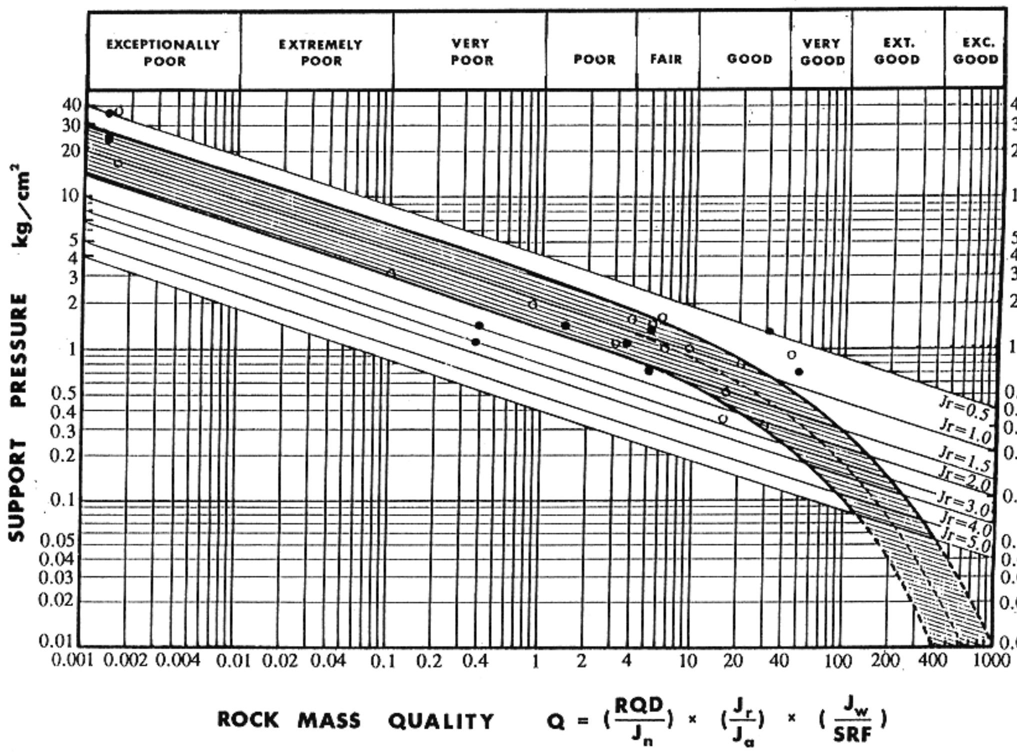


Figure 3. Support pressure related to rock mass quality, Q and joint roughness coefficient. [1]

From the rock mass Quality, Q, the support pressure can be estimated from the empirical diagram in Figure 3 based on case records from 1974, [1]. The support pressure in Figure 3 can be expressed by the following equation [1] :

$$P_{arch} = \left(\frac{2 J_n^{\frac{1}{2}} Q^{-\frac{1}{3}}}{3 J_r} \right) \quad \textbf{(2)}$$

For example the rock mass quality, Q = 0.01 will require a support pressure, P ≈ 1.1 MPa, when the joint roughness number, J_r = 1.0 and the joint set number, J_n = 12. (see equation (2).

In order to calculate the net support, which acts on the RRS, the pressure from the rock mass has to be reduced by the support pressure given from the temporary layers of fibre reinforced sprayed concrete (Sfr) and rock bolts (B). To be on the safe side, the support pressure corresponding to a rock mass quality, Q = 0.3, which gives a support pressure, P ≈ 0.30 MPa (taken from Figure 3), is considered as the "basic" support pressure from the temporary support. This corresponds to a 12cm thick layer of Sfr and rock bolts (B) with spacing 1.5m for a tunnel with 10m span. To be clear: For the design of RRS the "basic" support pressure (P = 0.30 MPa) has to be subtracted from the total support pressures for found in Figure 3, or more accurately, calculated from equation (2). To be less conservative, support pressure related to the real thickness of Sfr sprayed according to observed Q-value may be subtracted from the total support pressure giving a smaller dimension of RRS. But this may cause a problem because no empirical support pressure is available related to the Sfr without RRS or cast concrete arches, CCA, in the lower rock classes, where RRS or CCA traditionally have been used. Another reason to be on the safe side is the inaccuracy of the empirical calculated support pressure.

For 5 m span the Sfr thickness corresponding to a specific Q-value is normally lower than for 10 m span. Similarly for 20 m span the Sfr thickness at a specific Q-value is higher than for 10 m span. This can be seen in the Q-chart (Figure 2). For instance, following the borderline between reinforcement categories 7 and 8, where demand for RRS traditionally starts, initial Sfr thickness is the same for all spans as the Q-value increases with increasing span.

Comparison between actual deformation measurements, numerical analyses and static, analytical calculations of RRS

In the last few years, deformation (convergence) measurements in tunnels in extremely poor rock masses have been compared with numerical analyses (UDEC B-B code). There have been fairly good correlations between the observations and the numerical analyses.

Figure 4 shows convergence measurements in a weakness zone ($Q_{average}$ = 0.04) which was temporarily supported with 25cm thick Sfr + B c/c 1.5 + cast concrete lining in the invert in a 10m wide tunnel. The maximum convergence after 11 months was 23mm, of which 18mm was observed the first 3 months. Since the measurements were started 13 days after blasting, another 10-15mm can be added. In total, the convergence should be approximately 35mm.

The 2D numerical analyses predicted a maximum displacement of 21mm in areas supported with 25cm thick Sfr + B c/c 1.5m. In numerical analysis it is anticipated that 50 % of the total deformation of the rock mass happens immediately after excavation, before application of rock support. Hence the total deformation may be 2x21 = 42 mm.

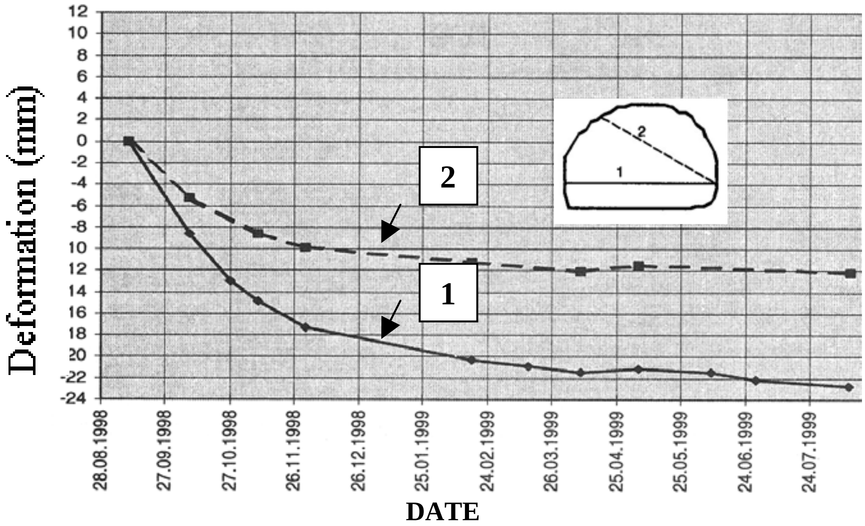


Figure 4. Convergence measurements from a weak zone in the Frøya Subsea Tunnel. [8]

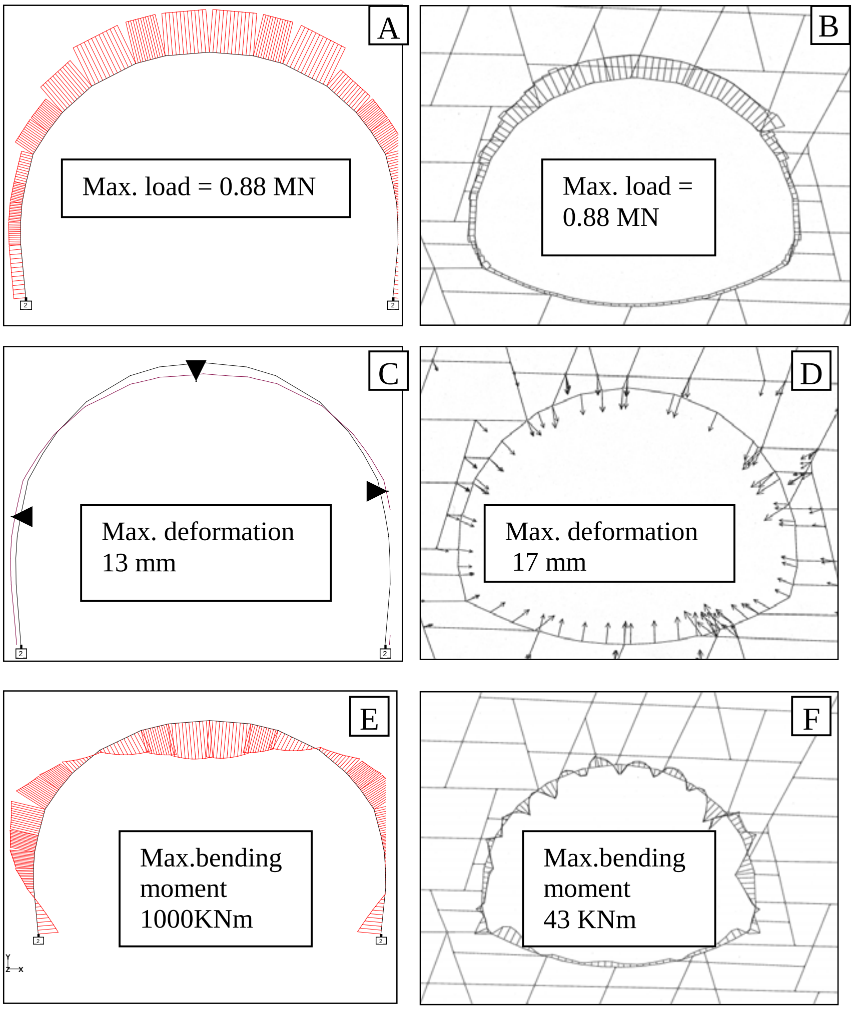


Figure 5. Comparison between numerical analysis[6] and static analysis with STAAD for load distribution (A+B), deformations (C+D) , and bending moments (E+F) in a 10m wide tunnel. (see text). The column ACE shows the analysis with STAAD. The column BDF shows the numerical modelling with blocks and joints.

2D numerical analyses of examples with two layers of sprayed concrete including RRS, with total thickness 25cm, gave a maximum displacement of 17mm. In these cases the maximum loading on the rock bolts was approximately 10ton. This equals approximately 0.05 MPa when the spacing is 1.5m.

Convergence measurements were carried out in 27 weakness zones in the Frøya Subsea Tunnel. Many cracks appeared in the sprayed concrete in the tunnel during the deformations. Hence, it was necessary to install RRS as a permanent support in the weakness zones.
A total of 82 ribs (RRS) were installed. In some places a total of 53m of in situ cast concrete lining was placed as permanent support in addition to the temporary 20-30 cm Sfr + B c/c 1.5m. Cost analysis indicated that cast concrete lining was 50% more expensive than RRS.

The results from convergence measurements and numerical analyses from the Frøya Subsea Tunnel in West Norway have been used in a static analyses of structural frames using the computer program STAAD. STAAD is a program specialised for analysis of concrete constructions.

Figure 5 illustrates the comparison between STAAD –analysis and numerical analysis in a block model (UDEC B-B). A and B show the load distribution, which is similar in static analysis (A) and numerical analysis with UDEC (B-B). C and D show the deformations calculated with STAAD (C) and numerical analysis (D). We can observe that the deformation in STAAD-analysis allows a movement outward in the walls, because of plastic behaviour under the higher load in the crown. We can also see that the defined blocks in the UDEC B-B model rules the deformation around the tunnel. E and F show the bending moment calculated by STAAD (E) and numerical analysis (F) with block movements.

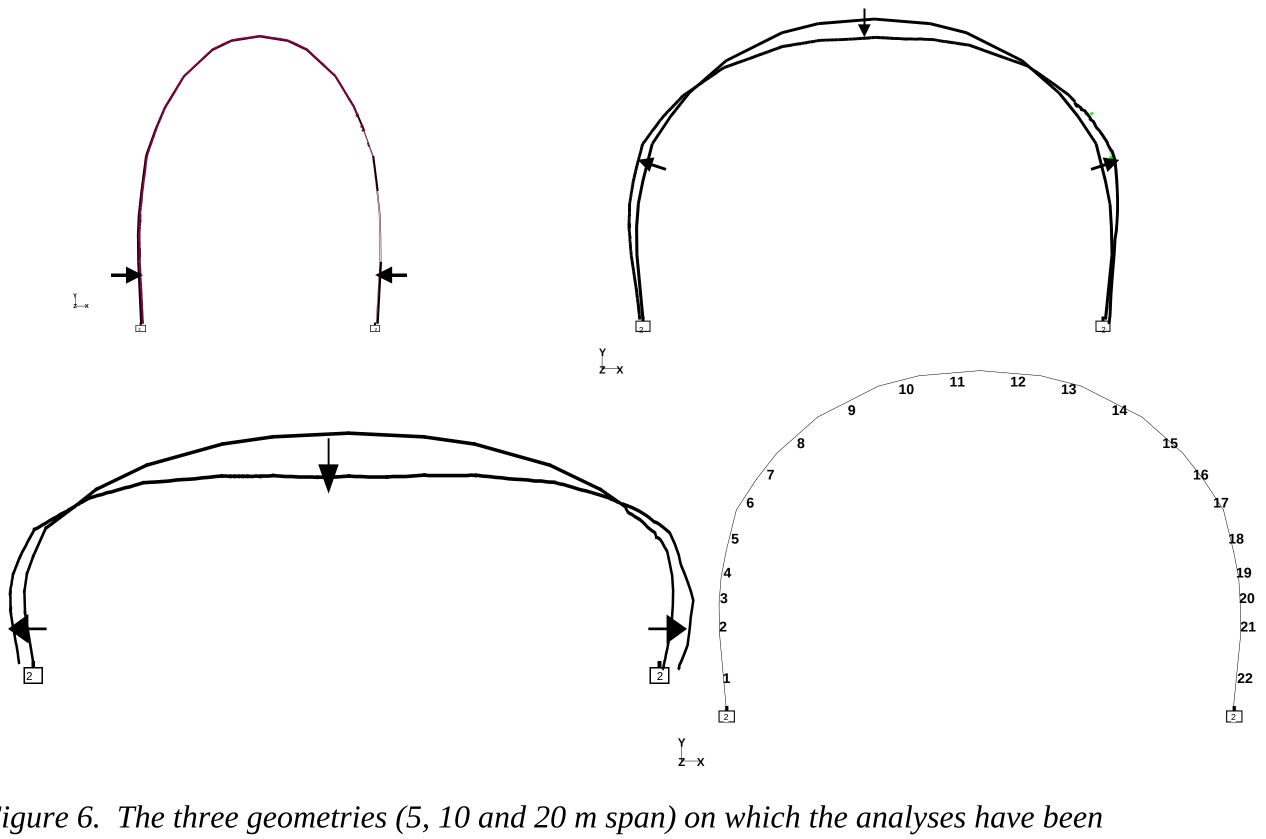


Figure 6. The three geometries (5, 10 and 20 m span) on which the analyses have been carried out in STAAD. The calculated deformation is visualised with arrows and a second line. The lower right illustrates the 22 elements in the STAAD –models.

It should be mentioned that only the ends of the arch (at the invert) in the STAAD –model are fixed (with springs), while the UDEC B-B model has taken into account the adhesion between rock and sprayed concrete. Lack of support from the rock (rock-structure interaction) in the STAAD model allows large outward movements of the walls, and hence overestimates the bending moment in the springline and the roof as shown in Figure 5, E&F. Instalment of stiff springs in the upper part of the walls may reduce this effect.

Analysis of the shear forces are carried out both in the numerical model and in the STAAD – model. In the model with 10 m span and 0.88 MN maximum load the STAAD-model gave a maximum shear force of 1,7 MN, while the numerical model gave 0,35 MN. This big difference probably is due to the effects described above.

In the UDEC B-B model the following material properties have been used for the intact rock in addition to the joint parameters:
Q-value, RMR-value, E (Young's modulus) or M (Deformation modulus) (GPa), ν (Poisson's ratio), ρ (Density) (kN/m^3), K (Bulk modulus) (GPa), G (Shear modulus) (GPa) [6].

In the STAAD model the following material properties are required used for the concrete construction in the RRS:
Thickness of RRS, E (Young's modulus) = 21 GPa, ν (Poisson's ratio) = 0.15, ρ (Density) = 25 (kN/m^3), Diameter of the reinforcement bars = 16 mm. All the models are divided in 22 elements as shown in Figure 6.

Description of the models analysed with STAAD

The deformation and the bending moment is acting differently on different parts of the profiles for 5, 10 and 20 m span as shown in Figures 6 and 7. The shape, load and the size of the underground opening causes this distribution of bending moment. The 20m wide model has a relatively flat crown, and would have less deformation, and would be able to take a higher load in the crown if the arching of the crown had been higher. In contrast, in the relatively high walls in the 5m wide model, there are large deformations in the walls, where the bending moment is high.

The inversion of bending moments from positive to negative in the springline might cause shear failure in the RRS if the load is too high compared to the strength of the RRS. Such effects have been observed in the Lærdaltunnel in West Norway, where high stress with squeezing conditions deformed even 90 cm thick sprayed concrete including RRS combined with rock bolts (c/c<1m) [7]. In most cases however, moderate deformations will give tension cracks in the shotcrete lining.

Figure 8 shows the distribution of bending moments in the crown, springline and walls for different loads in the model with 10m span. It is noteworthy to observe that the bending moment of the walls is decreasing with increasing thickness of the RRS, while it is increasing with increasing thickness in the roof and to a smaller extent in the springline.

Similar curves are shown for a 20m wide span, while a 5m span gives an increasing bending moment when the thickness of the RRS increases in all parts of the profile. The maximum bending moments for 5, 10 and 20 m span are 2700, 6400 and 35000 kNm respectively.

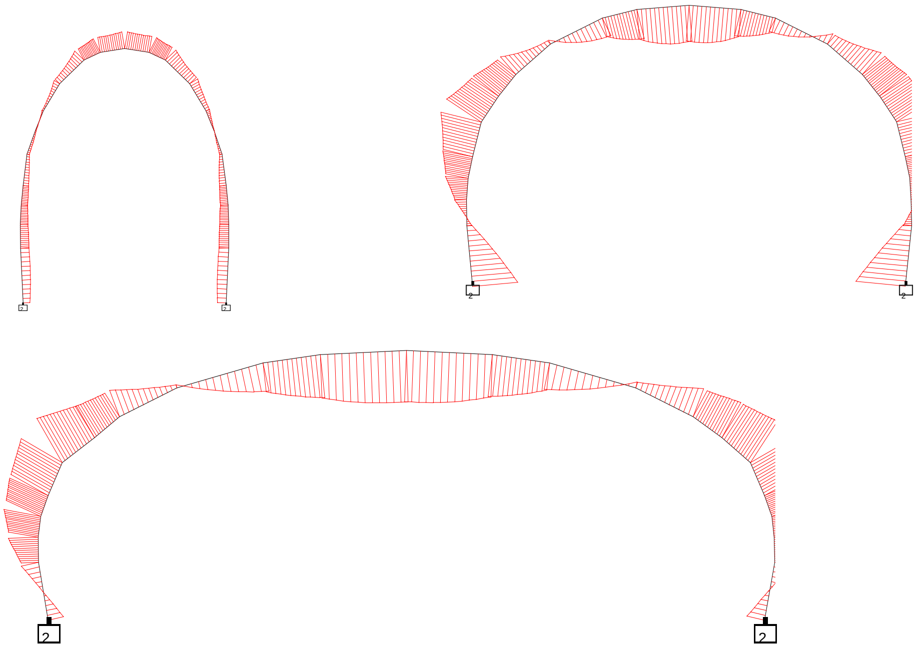


Figure 7. Bending moment of the 5, 10 and 20 m span models

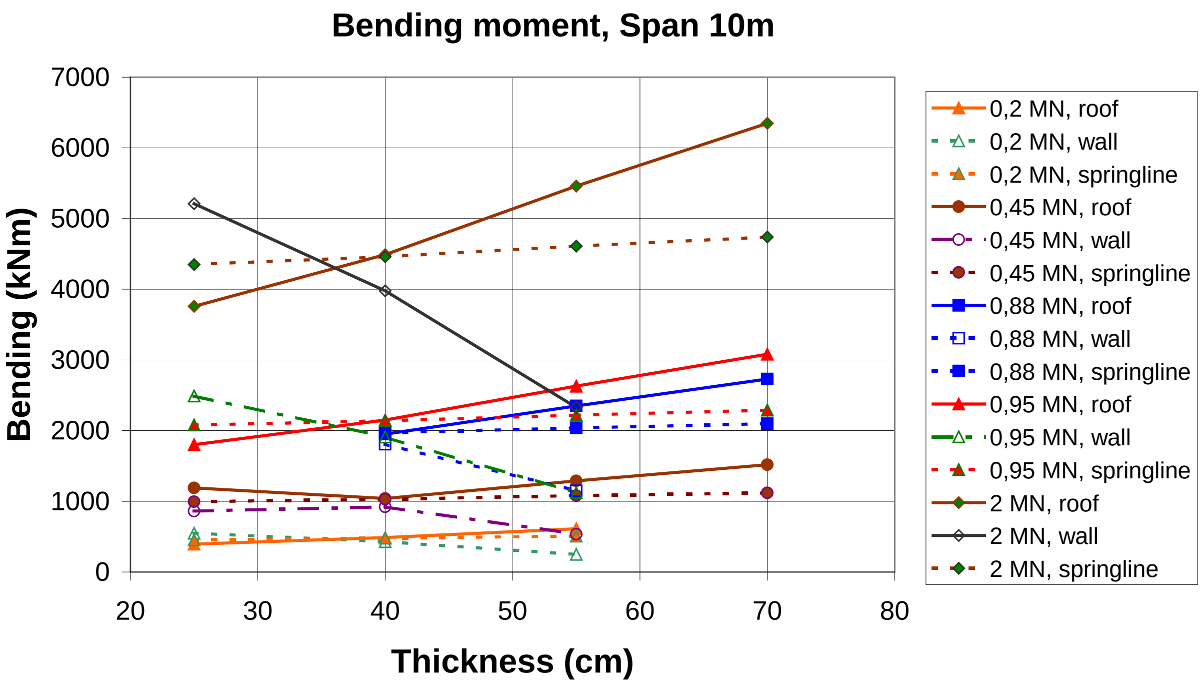


Figure 8. Distribution of bending moment related to thickness of RRS and maximum load in the model with 10m span.

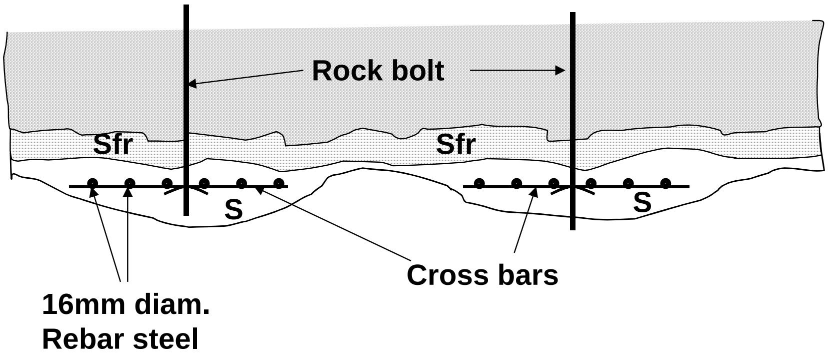


Figure 9. Principal sketch of two reinforced ribs of sprayed concrete with one layer of reinforcement, seen perpendicular to the tunnel axis..

Calculation of the deformation of the ribs related to thickness and load

Figures 10, 11 and 12 show the deformations in relation to the thickness and maximum load for the three models with 5, 10 and 20m span. Lines are drawn for both the roof and the walls for each load ranging from 0.1 to 2.0 MN.

In Figure 10 the maximum deformation in the roof in a 25cm thick rib in 5 m span with high walls is 2 mm when the maximum load is 2.0 MN. It can be seen from Figure 10 that the deformations of the walls, compared to the roof, are far less influenced of the thickness of the ribs. This is most distinct under high loads. In the model the load is much higher in the roof than in the walls. A line is drawn at 1mm deformation is considered as an acceptance limit in 5m span after the permanent support is installed.

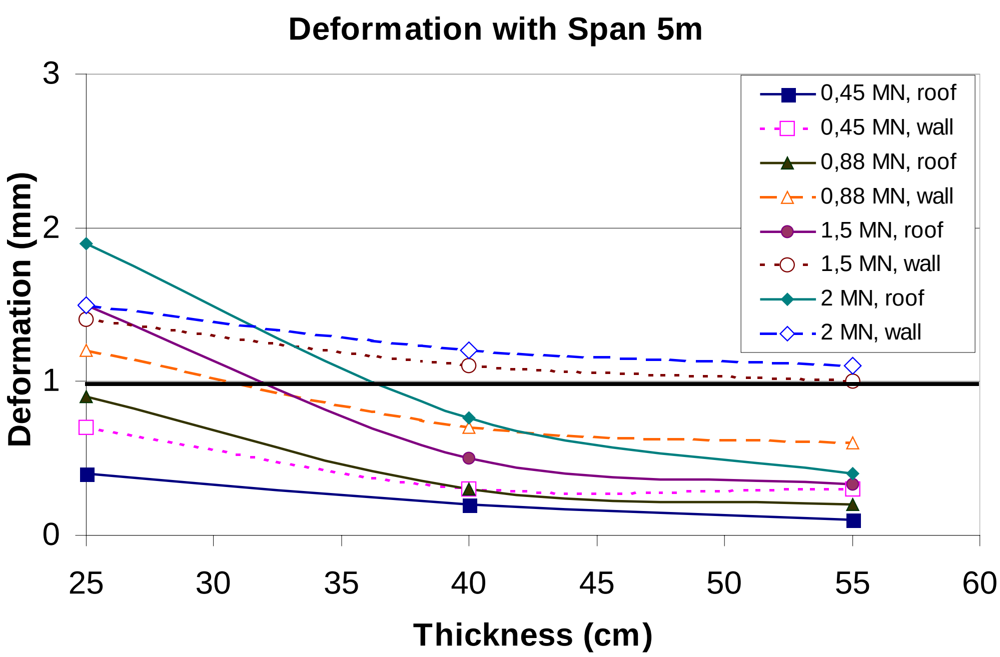


Figure 10. Deformations in the crown and the walls related to thickness of RRS and load in a 5m wide tunnel. Thick horizontal line at 1 mm is the considered limit for acceptable deformation.

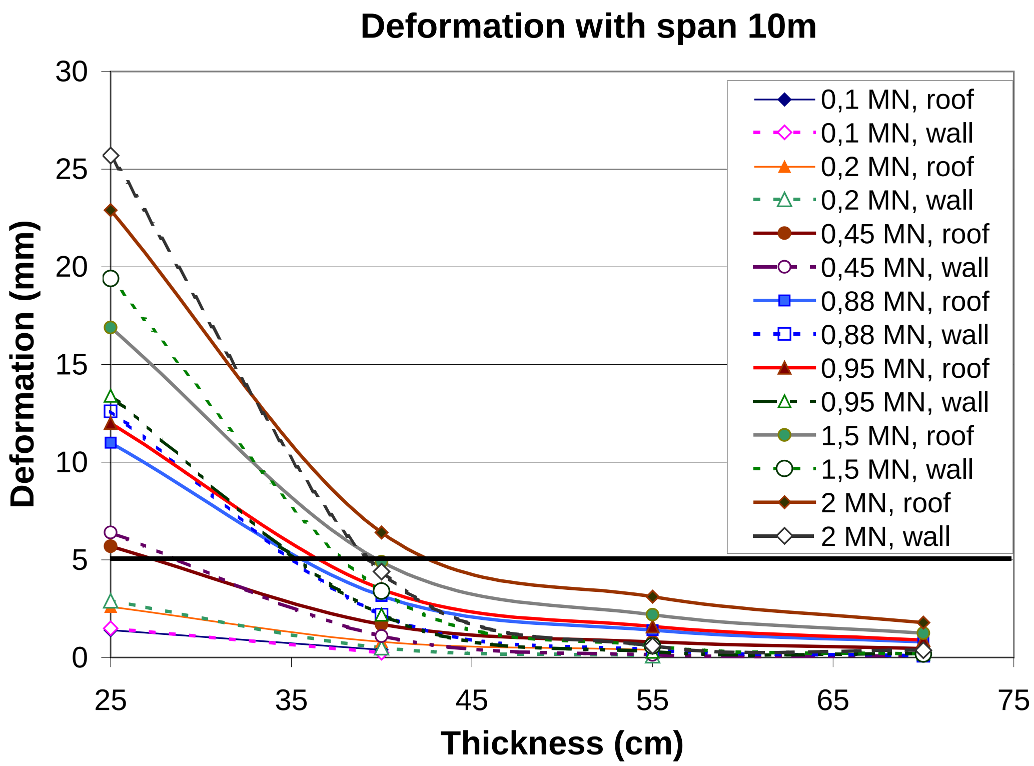


Figure11. Deformations in the roof and the walls related to thickness of RRS and load in a 10m wide tunnel. Thick horizontal line at 5 mm is the expected limit for acceptable deformation.

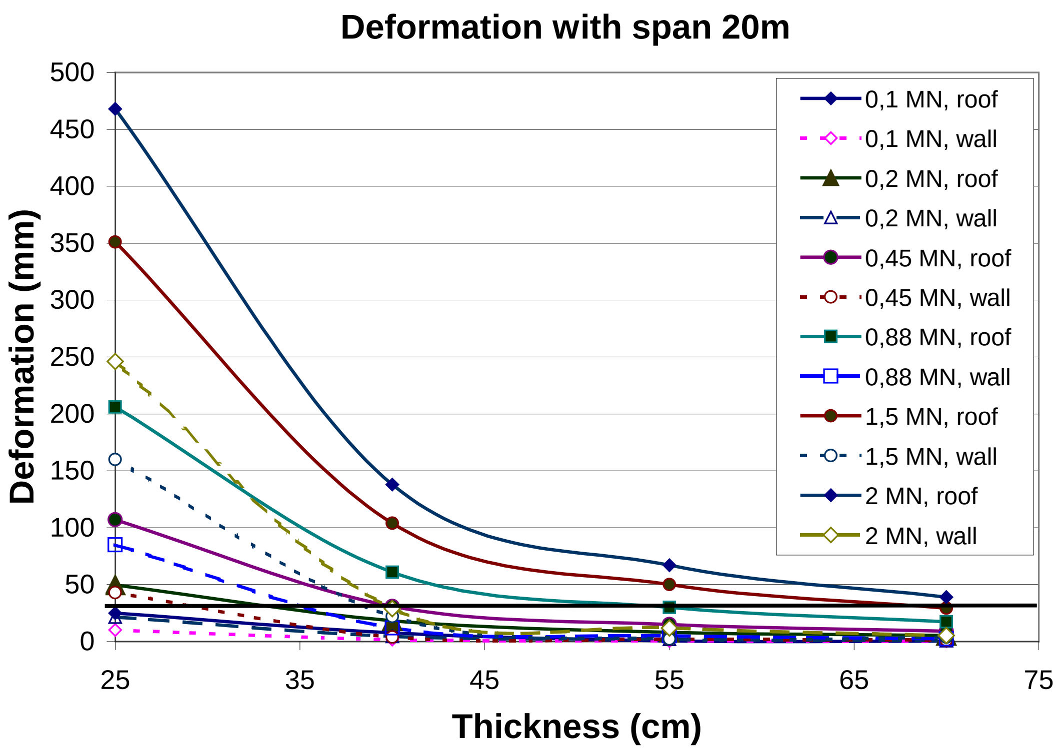


Figure 12. Deformations in the roof and the walls related to thickness of RRS and load in a 20m wide tunnel. Thick horizontal line at 30mm is the expected limit for acceptable deformation.

In Figure 11 the maximum deformation in the roof in a 25cm thick rib, in a 10 m wide tunnel is 23mm when the maximum load is 2.0 MN. The maximum deformation of the walls is 25 mm with maximum load = 2.0 MN. In the model with 10m span the deformation of the walls has an opposite development compared to a 5m span. In 10m span the deformation of the walls is more reduced with increased thickness of the ribs than the roof. This is most pronounced with high loads, i.e. 1.5 and 2.0 MN. A line drawn at 5mm deformation is considered as an acceptance limit in 10m span after the permanent support is installed**.** These acceptance limits are based on practical experience in different spans.

In Figure 12 the maximum deformation in the roof is calculated to be 470mm in a 25cm thick rib in a 20m wide tunnel when the maximum load is 2.0 MN. The maximum deformation in the walls under the same conditions is 250mm. This is to a large degree ruled by the shape of the model with the 20m wide span that has relatively low walls. This difference between roof and walls is most pronounced with high loads from 0.45 to 2.0 MN. Similar to the 10m span the deformation of the walls is more reduced than the roof with increased thickness, particularly with high loads.

In Figure 12 a line drawn at 30mm deformation is concsidered as an acceptance limit in 20m span after the permanent support is installed. For high loads, in the order of 2.0 MN the diagram indicate a thickness of 75 cm in the rib in order to remain under the 30mm line. But in most cases the arch is higher than in the model, and 70cm thickness may be sufficient to avoid deformations greater than 30mm. And the “basic” support is likely to give more than the calculated 0,30 MPa. Another reason why the calculated deformations may be too high for the 20 m span is the application of the same anchored strength of the springs at the invert for all three spans. By adjusting the anchored strength of the springs for 5 and 20m span relative to the strength for the 10 m span, which is calibrated, the deformation will be increased for 5 m span and reduced for 20 m span. The spring strength has to be increased many times its original magnitude (1000 kN) in order to give a significant reduction in deformation in the 20 m span. If the radius of curvature in the crown is reduced, the deformations also will be reduced.

In most cases high loads will cause deformations, which will lead to tension cracks in the crown. After moderate deformation the reinforcement bars will start to act. Because of this it is very important to place most of the reinforcement bars in the lower layer (furthest off from the rock surface). Based on the deformation curves in Figures 10, 11 and 12 it is possible to recommend the optimum thickness of the ribs. This thickness should not allow too much deformation in the permanent structure, but should also avoid over-support.

Analysis of the number of rebars for reinforcement in RRS

The application of the calibrated STAAD-analysis makes it possible to calculate the necessary number of reinforcement bars in the ribs. It also gives a recommendation where to place the reinforcement in the ribs relative to the rock surface.
Figures 13, 14 and 15 show the number of reinforcement bars per rib in a tunnel related to thickness and load for 5, 10 and 20 m span. These are based on the bending moment and deformations calculated in STAAD. Cases with too large deformation are avoided. For 5 m span the expected borderline for unwanted deformations is set to 1 mm in the RRS for permanent support. For 10 and 20m span the limit for accepted deformation is set to 5 and 30 mm respectively.

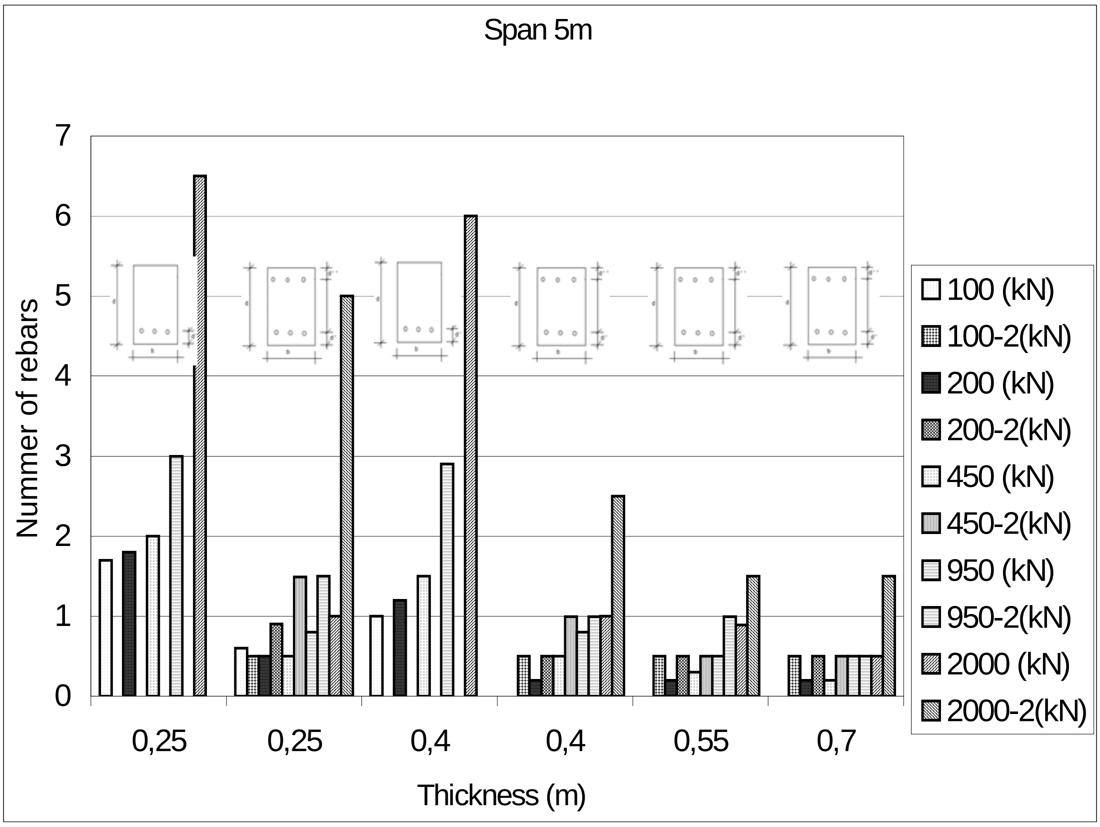


Figure 13. The number of Ø 16mm reinforcement bars per rib related to the thickness of RRS and maximum load on the structure in a 5 m wide tunnel. In cases with two layers of reinforcement, see text for explanation.

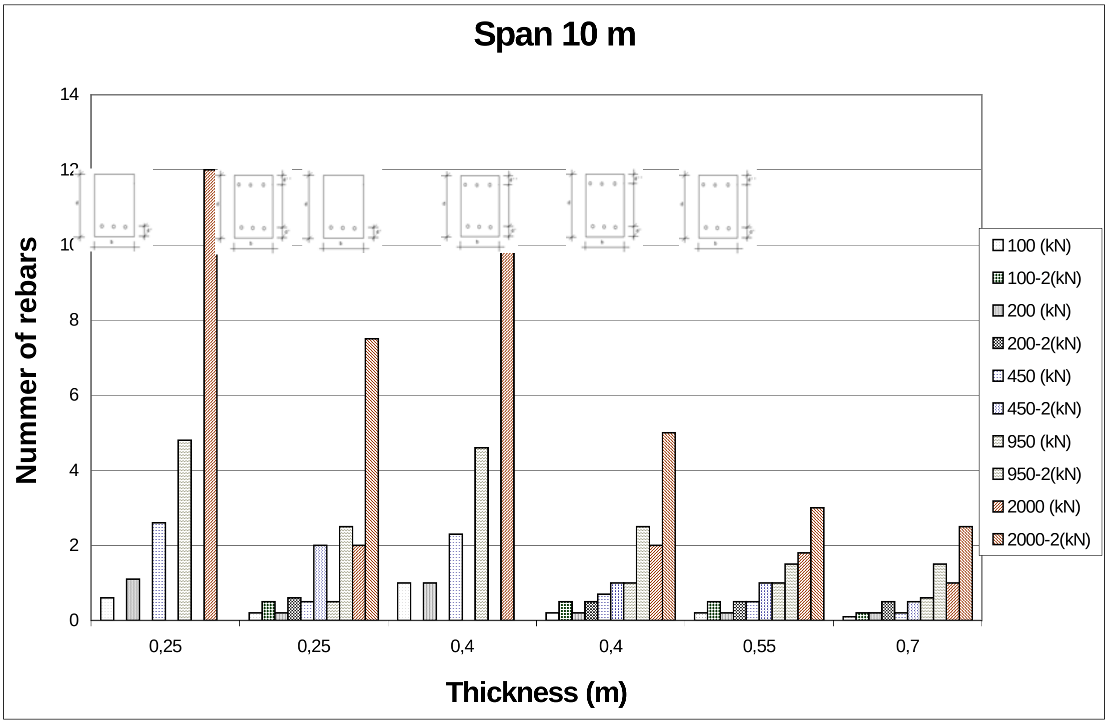


Figure 14. The number of Ø 16mm reinforcement bars per rib related to the thickness of RRS and maximum load on the structure in a 10 m wide tunnel. In cases with two layers of reinforcement, see text for explanation.

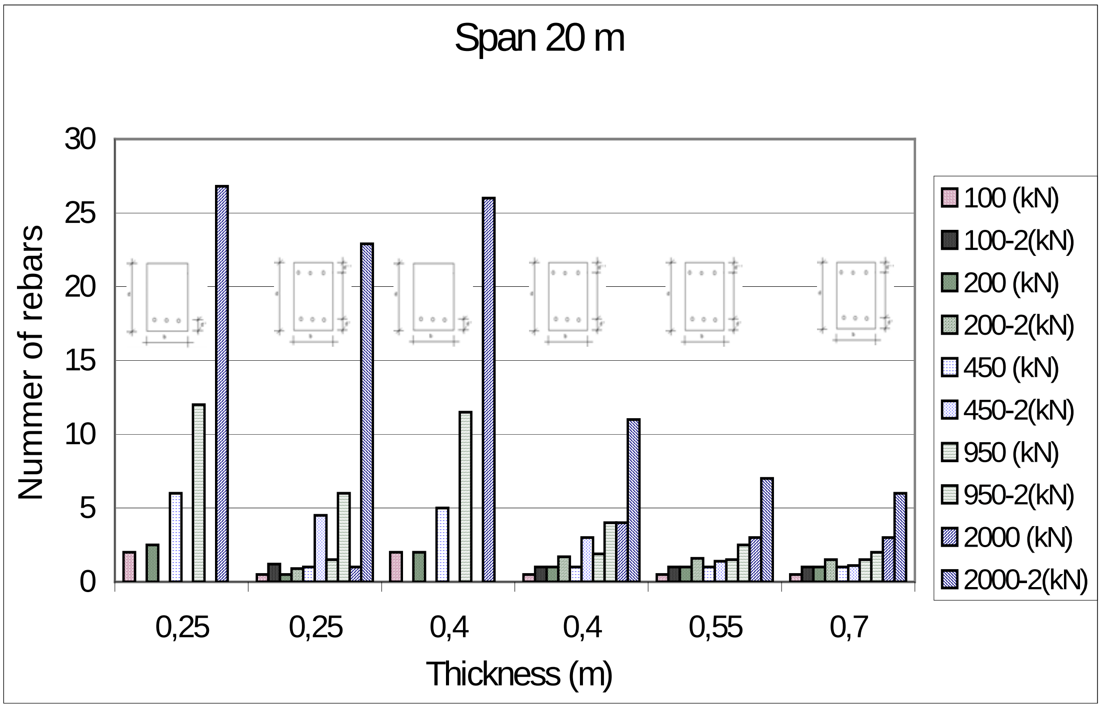


Figure 15. The number of Ø 16 mm reinforcement bars per rib related to the thickness of RRS and maximum load on the structure in a 20 m wide tunnel. In cases with two layers of reinforcement, see text for explanation.

The insets in the diagrams show RRS with one or two layers of rebars for each thickness. Hence, the thickness 25 and 40cm are shown for both single and double layers. For double layers in each load class, for instance 0.95 MN, two columns are shown. The right hand column represent the number of rebars in the lower layer (most important with highest number of rebars, marked 0.95 –2 (MN), while the left hand columns indicate the number of rebars in upper layer, marked 0.95 (MN). The sum of these two columns has to be used in combination with the deformation diagrams in order to design ribs for different load (support pressure) and span.

In general, double layers, which can take higher loads than single layers of the same thickness, should be preferred if the total thickness is sufficient for two layers. It can be seen from the histograms in Figure 14 and 15, that a thickness of 25cm combined with 2.0 MN load in 10 and 20 m span requires a large number of reinforcement bars and combined with rock bolts in order to compensate for the large deformations. In these cases, the rebars will in large extent take over the bending moment after opening of cracks or partial crushing of the concrete. However, large deformations with possibility for shear failure should be avoided. If shear failure is expected, increasing number of rock bolts and cross reinforcement of the ribs is recommended.

Application of the RRS-calculations in the Q-chart for rock support in all rock classes

The calculated thickness for different loads (= support pressure) which is linked to rock mass qualities has been compared to practical cases in Norwegian tunnels. Most of the data originate from the Frøya Subsea Tunnel, the Lærdal Tunnel (25 km long and up to 1500 m overburden) and the E18 twin tube highway tunnels in Vestfold County, South of Oslo.

Based on the calculations described above, the recommended thickness, number of rebars and spacing between the ribs has been placed in the rock support chart linked to rock mass quality, Q. For each Q-value (0.001, 0.004, 0.01, 0.04, 0.1 and 0.4) with 5, 10 and 20 m span, required thickness, number of rebars and spacing is given in the Q-chart. Similar to the thickness of the sprayed concrete in the original Q-chart published in 1993, the same thickness of the ribs can be followed from lower left to upper right. But because of the increasing spacing between the ribs with increasing Q-values the gradient is lower for the ribs than for even layers of sprayed concrete.

The same spacing is recommended between the ribs for similar Q-value irrespective of the span. The net load on each rib of the same strength should approximately be the same, because the reduced support pressure compensates for the increased spacing with increased rock mass quality, Q.

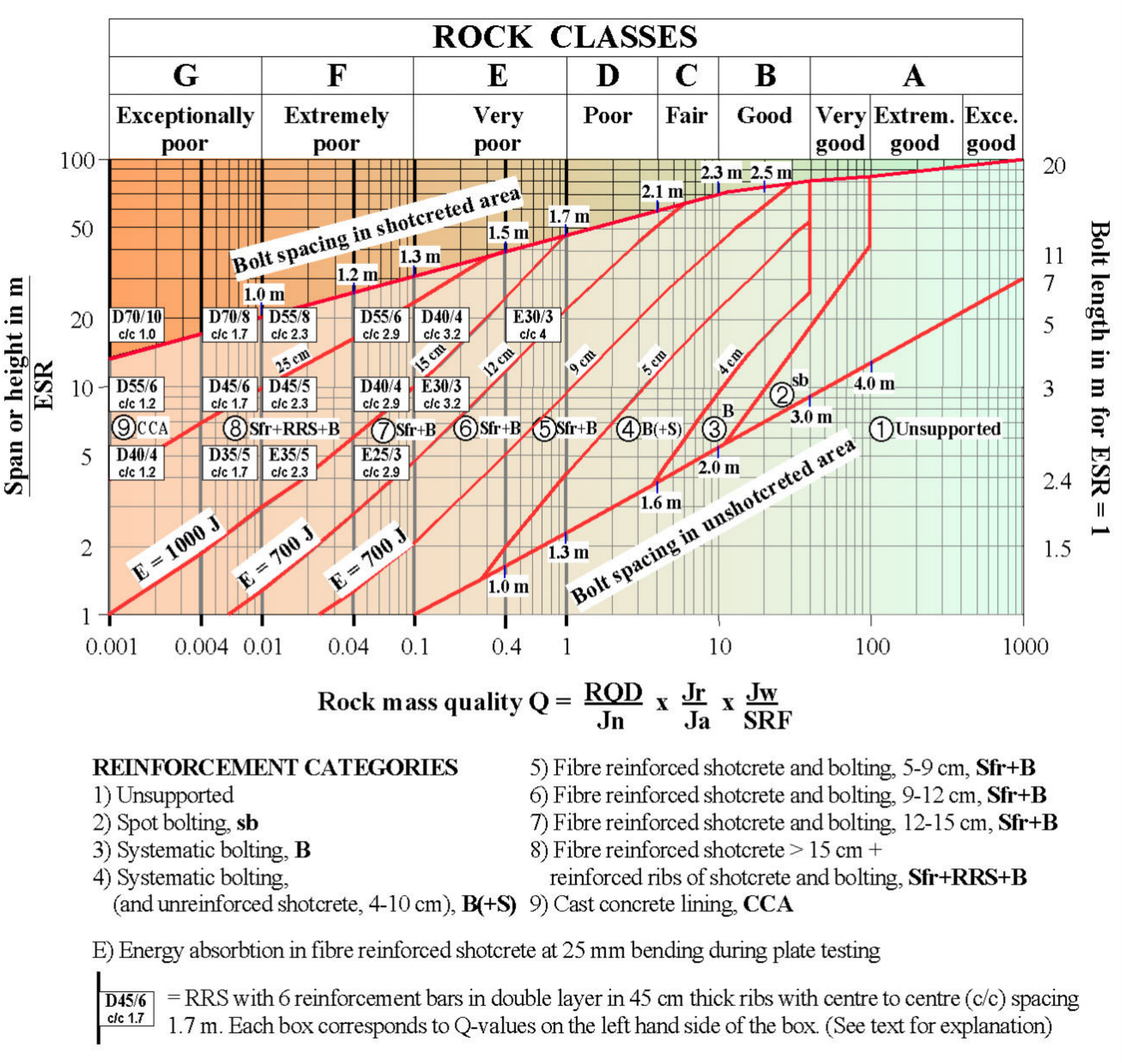


***Figure 16**:Q-chart with recommended thickness, number of rebars in single (E) or double layers (D) and spacing between the ribs (RRS) in different rock mass qualities, Q for 5, 10 and 20m spans.*

For a Q = 0.001 there is no spacing between the ribs for 20 m span. That means a continuous spraying of reinforced ribs with 70cm thickness in addition to the temporary support comprising of 12cm of Sfr + Bc/c1,5m. The 10 reinforcement bars are distributed over the 1m wide rib.

In practical cases the layer of Sfr might be 25cm, and the spacing of the rock bolts might be 1.1m in addition to spiling bolts before the instalment the RRS. In this case, the thickness of the RRS can be reduced with the corresponding increase in thickness of the Sfr ≈ 25 – 12 = 13cm. For other cases, the spacing between the ribs (centre/centre) is given for each case. Between the different boxes which show calculated thickness, number of rebars and spacing between the ribs, interpolation is recommended for improving the support. For Q = 0.001 it is often more economic to apply cast concrete arches (CCA). However, in cases with changing profiles or small weakness zones it is practical and, hence more economic to use RRS.

It should be stressed that the support pressure might vary significant related to corresponding Q-values (See Figure 3). Therefore, in the lower rock mass qualities it is wise to observe the deformations some time after temporary support, before making the design for the RRS.
In the better rock mass qualities stress reduction will often occur during deformation. Hence, the rock mass will take a large part of the load after deformation, and the need for extra support will be reduced. This can be controlled by deformation measurements.

Figure 17. Reinforced ribs of Sprayed concrete, in the 22 m vide National Theater railway station in Oslo. The spacing between the ribs is 2 m, and the thickness is ≈0,85 cm

CONCLUSIONS AND DISCUSSION

For absorbing the deformations in rock masses of low quality, the toughness of sprayed concrete should be high. Based on laboratory tests and observations in tunnels it has been possible to incorporate the deformation energy for different rock mass qualities, Q, in the rock support chart. Design of rock support can be improved by changing to higher quality of fibres or by increasing the amount of fibres. This is in accordance with the guidelines and technical specifications of the Norwegian Concrete Association publication no. 7. When high stresses occur, it is recommended to move up one toughness class because of the high forces and the long term deformations.

In the lower rock mass qualities ($Q<0.1$ for 10m span) the Q-support chart published in 1993 [2] recommends application of RRS or cast concrete arches (CCA) in addition to fibre reinforced sprayed concrete (Sfr) and rock bolts (B). No design model has been available for construction of the RRS. The research work referred in this paper is based on the computer program STAAD, which gives deformation and bending moments in RRS that are dependant on the thickness and number of rebars for different loads (support pressure) and span. The analytical work has been compared to, and calibrated with deformation measurements and numerical analyses of tunnels of approximately 10m span. The calculated deformations agree well with the observed ones. For 5 and 20m span there are few practical examples where RRS is applied. Therefore, some uncertainties concerning the deformations and bending moments for 5 and 20m span exist. Just a few examples of 22-23m span is recorded with deformation measurements. Hence, it is recommended that support with RRS in small and large spans should be done with some care. Regular observations of deformations after installing the temporary support will provide guidance in the design of the permanent support with RRS in extremely poor rock masses.

In general, a higher safety can be expected when crossing the border from the reinforcement category 7 to 8, because the addition of the RRS to the temporary support with Sfr + B will give a sudden substantial increment in support. This sudden increment may be smoothened if some part of the temporary layer of Sfr is taken as a part of the rib close to the borderline. It is important to remember that the ribs shall carry not only the rock mass above, but also the distributed load from the rock mass between the ribs. In the lowest Q-values in reinforcement category 9, the Q-system published in 1993 recommended CCA. However, both practical experience and analyses show that it is possible to replace any CCA with RRS for all rock mass qualities.

Small deformations and the first part of large deformations of RRS are usually elastic dependent only on the concrete, irrespective of the reinforcement, without any cracking of the concrete. The first deformation is not dependent of the reinforcement. The reinforcement will start to act gradually during the deformations. Sufficient reinforcement in the RRS, may prevent further deformation and collapse if the design is correct related to the rock mass quality, Q.

For practical reasons it should be mentioned that all reinforced ribs of sprayed concrete have to be combined with rock bolts evenly distributed throughout the arch. Usually the spacing is 1.5 m. This helps to avoid or reduce shear failure during large deformations. It is important that the deformations have to be controlled before the final instalment of RRS. Sometimes large deformations may occur in the lower part of the walls. If no cast concrete ring is installed in the invert, then it is very important to install extra rock bolts through the lower part of the ribs, near the invert in order to avoid unwanted deformations or collapses.

The most important parameters for designing of RRS are support pressure taken from the Q-value, span, geometry and accepted deformation.

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