



A Hybrid Methodology of Rock Support Design for Poor Ground Conditions in Hard Rock Tunnelling

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Abstract

The design of rock support in hard rock tunnelling is often assisted by the use of empirical engineering rock mass classification systems. This has involved support design in a wide variety of geological- and rock mechanical-conditions, including poor ground conditions like fault zones and weak rocks. For most of the encountered situations, the empirical support designs have performed well. However, the empirical classification systems pose several limitations to describe ground behavior and derive optimal rock support design in poor ground conditions as discussed in several publications during the last decades. In the present study, an analysis of 118 case records, mostly from monitored Norwegian tunnel sites, has been undertaken to investigate the performance of the current empirical design practice with the Q-system when exposed to poor ground conditions. The results have revealed significant improvement possibilities which relate to a wide variety of ground conditions. Among others, weak rocks, anisotropic rock, and rock mass conditions in the transition from self-bearing rock masses to rock masses requiring load-bearing support in the tunnel, and typically with rock mass quality $Q < 1$. As a result, a set of improvements and design recommendations have been developed and an integrated or hybrid design methodology specific for poor ground conditions proposed. The hybrid approach combines the main components and advantages of different design methodologies and provides guidance for design optimization where the empirical approach presents limitations.

Highlights

- The empirical approach of tunnel rock support design has limitations to capture ground behavior, a relevant parameter for support design in poor ground conditions.
- The study of a database with more than one hundred case records has revealed important improvement possibilities in the current design practice using the empirical approach.
- The involvement and integration of different design approaches and tools through a hybrid design methodology can allow for design optimization in poor ground conditions.

Keywords Empirical design · Q-system · Poor ground conditions · Ground behavior · Rock support · Hybrid design

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Abbreviations

CCM	Convergence confinement method
DEM	Discontinuum element modelling
ESR	Excavation support ratio in the Q-system
FEM	Finite element modelling
GBT	Ground behavior type
GRC	Ground reaction curve
GSI	Geological strength index

LDP	Longitudinal displacement profile
LG	Lattice girder
NATM	New Austrian tunnelling method
NFF	Norwegian association of tunnelling
NGI	Norwegian geotechnical institute
NPRA	Norwegian public roads administration
RMi	Rock mass index
RMR	Rock mass rating
RRS	Ribs of reinforced sprayed concrete
SCC	Support characteristic curve
SRF	Strength reduction factor in the Q-system

List of Symbols

A_b	Cross sectional area of rock bolt
E_i	Young's modulus of intact rock
E_m	Rock mass deformation modulus
J_r	Joint roughness number in the Q-system
K_n	Joint normal stiffness
K_s	Joint shear stiffness
M	Bending moment
M_R	Moment of resistance
M_{RC}	Moment of resistance of sprayed concrete
M_{RS}	Moment of resistance of steel
Q_b	Axial load in the bolt
T_b	Tensile capacity of rock bolt
V	Shear force
V_R	Shear resistance
V_{RC}	Shear resistance of sprayed concrete
V_{RS}	Shear resistance of steel
f_b	Yield strength of steel for rock bolt
k_o	Horizontal to vertical stress ratio
m_i	Hoek–Brown material constant of intact rock
p_{eq}	Support pressure at equilibrium
p_i	Internal pressure in tunnel
p_{max}	Maximum allowable pressure of support system
p_o	In-situ rock stress
p_s	Support pressure in tunnel
u_r	Radial displacement of tunnel wall
σ_c	Unconfined rock strength
σ_{fl}	Flexural capacity of sprayed concrete
σ_h	Horizontal stress
σ_v	Vertical stress
σ_θ	Tangential stress in the rock mass
σ_{ci}	Uniaxial compressive strength of the intact rock material
ν	Poisson's ratio of intact rock
σ_{cm}	Rock mass strength
σ_{sh}	Shear strength of sprayed concrete
ϕ_r	Residual friction angle in rock joint
FS	Factor of safety
FS_b	Limit state at axial failure of rock bolt
JCS	Unconfined compressive strength of rock joint wall

JRC	Joint roughness coefficient of rock joint
L	Distance to the face in the CCM
N	Axial thrust
ϵ	Tunnel strain
S	Joint spacing
s	Critical perimeter between rock bolts
t	Thickness of sprayed concrete

1 Introduction

The empirical approach of tunnel rock support design has generally shown successful results when applied to hard rock conditions. In poor ground conditions, however, tunnel engineers have been normally challenged with the evaluation of ground-behavior and -loads. This is something that has occurred, at least, since the first known reference about the use of rock behavior observations as a basis for tunnel support design (Simms 1844). With the development of modern and multiparameter classification schemes such as the RMR (Beniawski 1973), the Q-system (Barton et al. 1974), and the Rock Mass index RMi (Palmstrom 1995), the classification of rock masses and corresponding recommendation of rock support have improved. However, classification systems for tunnel rock support design still present limitations in addressing ground behavior and assessing the optimal rock support in poor ground conditions (Palmstrom and Broch 2006).

In the case of the Q-system, support recommendations are done by assigning rock support classes to rock mass classes based on the evaluation and mapping of the parameters in the Q-system (Barton and Grimstad 2014; NGI 2015). A wide range of support designs are then prescribed, that result from different combinations of sprayed concrete thickness, rock bolt spacing and load-bearing support like the ribs of reinforced sprayed concrete (RRS). However, there have been situations where hard rock tunnels affected by poor ground conditions have required the use of more elaborated design methods such as the analytical convergence confinement method, CCM (Carranza-Torres and Fairhurst 2000), and numerical analyses with input from lab and in-situ testing, and/or deformation monitoring.

The study presented in this article has therefore two main objectives. The first is an assessment of the performance of the Q-system for the design of tunnel rock support in poor ground conditions. This is done by evaluating the ground behavior and the support loading from analytical and numerical back-calculations of case records from tunnels that present rock support designed empirically. The second objective is to propose an improved design procedure based on the performance assessment of the empirical approach.

To fulfill these objectives, a review of the design principles behind the design of rock support in hard rock tunnels

was required. The self-bearing capacity principle of rock masses was emphasized and applied to poor ground conditions. The study concludes with a set of proposed improvements which are implemented in a new hybrid methodology. This hybrid methodology is a design procedure that combines several design tools and approaches, which aims at design optimization of rock support in poor ground conditions.

2 Characterization of Poor Ground Conditions

2.1 Structure and Composition

In hard rocks, the rock material surrounding an underground opening is often intersected by discontinuities which split the rock volume in blocks or pieces of different sizes and shapes. The in-situ and fractured rock medium containing these discontinuities forms the rock mass. The nature and distribution of these discontinuities within the rock mass is referred as the rock mass structure. Definition of isotropic or anisotropic rock mass structure will therefore be linked to the relative spacing and orientation of the discontinuities or structures in relation to the scale of the problem. To describe the rock mass composition, three main parameters are normally addressed: (1) the intact rock properties, i.e., rock strength, Young's modulus, (2) the occurrence of rock mass with discontinuities and (3) zones with low mechanical properties within the rock mass. When a rock mass is subjected to in-situ rock stresses, groundwater, and other external loads, the term ground is used (Palmstrom 1995). And the response of the ground to tunnelling is called ground behavior.

The term poor ground conditions can therefore be used to address adverse combinations of rock mass structure and composition, in-situ stresses, groundwater, and external loads. In the context of this article, poor ground conditions are mainly referred to as distinct weakness zones, weak rock, anisotropic rock mass structure, fractured rock masses and grounds subjected to low stresses condition causing tunnel instability and thus, requiring rock reinforcement in hard rock tunnels (Fig. 1). Mixed ground conditions could be also considered poor ground conditions in the sense that different materials or layers with different rock mechanical properties intersecting the opening can lead to instabilities as well. However, mixed ground conditions have not been particularly studied in this article as there were no case records covering this ground condition.

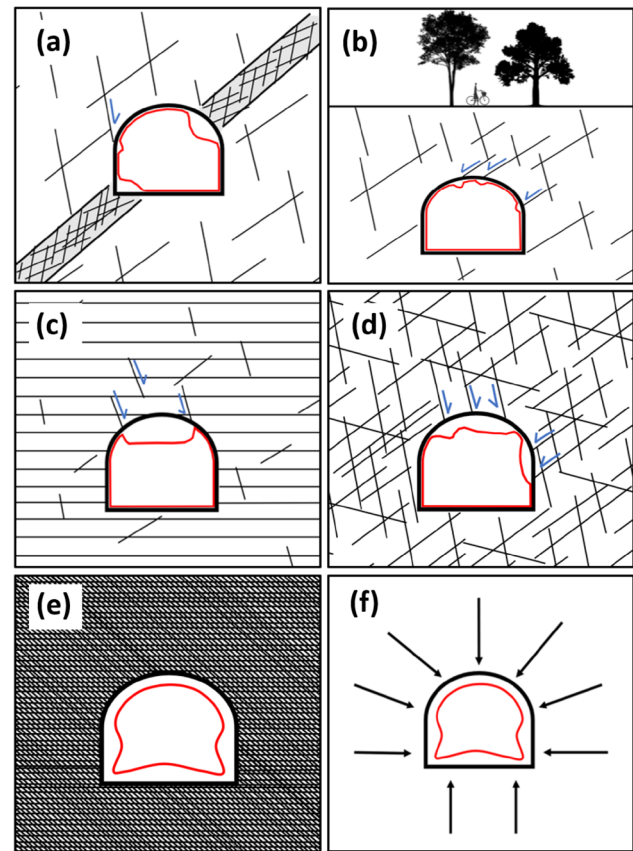


Fig. 1 Proposed classification of GBT in hard rock tunnelling affected by poor ground conditions, in low-moderate and hydrostatic in-situ rock stress conditions. Based on the study of the database in Sect. 5.3. Blue arrows represent the relative direction of rock block movements while red contours show examples of the resulting deformation profile. Description of GBT as labelled a–f in the text. (Color figure online)

2.2 Strength and Deformability

The rock mass strength (σ_{cm}) and the deformation modulus of the rock mass (E_m) are important parameters in any analysis of ground behavior considering deformations in an underground opening (Hoek and Brown 1980a, b; Hoek and Diederichs 2006). Strength and deformability properties are not only relevant to estimate ground behavior of overstressed grounds, but also in any other rock mass characterized by failure mechanisms that are dependent on the in-situ stresses and the rock mass structure, as suggested by Brady and Brown (2006).

The “Generalized Hoek–Brown” failure criterion (Hoek 1994; Hoek et al. 2002) is one of the most widely used means to estimate rock mass strength in isotropic and jointed rock. Geological input in the criterion is provided by the Geological Strength Index (GSI) of Hoek (1994), which is evaluated on the basis of a qualitative assessment of the joint characteristics and the rock mass structure. Since the

Hoek–Brown criterion accounts also for intact rock properties such as the uniaxial compressive strength of rock material (σ_{ci}), a wide spectrum of rock types and ground conditions can be captured, especially in the range of poor and weak rocks. In turn, the Mohr–Coulomb failure criterion (Coulomb 1776; Mohr 1900; Labuz and Zang 2012) is seen more adequate for either rock masses of hard and isotropic rock, or for the estimation of the shear strength of distinct and throughgoing structures within the rock mass.

From the analysis of rock mass strength and the situ rock stresses, the rock mass competence, a parameter that gives insight on the ground behavior and tunnel stability can be evaluated. A *competent* rock mass is defined by a rock having higher strength than the stresses acting on it (Palmstrom and Broch 2006). Therefore, without significant deformations and little or no failure of the rock material. The term *self-bearing* rock mass can be also used to define rock mass competence in jointed hard rock. In this case, the effect of discontinuities in rock stability is involved. The self-bearing capacity of a rock mass is therefore dependent on not only its capability to bear induced stresses, but also on its ability to utilize the stabilizing effect provided by the generation (or conservation) of a pressure arch and the associated increase in normal stresses on joints around the opening.

In turn, a rock mass is considered *weak* or incompetent when its strength is low, making it unable to withstand the induced stresses, causing failure of the rock material followed normally by bulking and plastic deformations. According to Hoek (1999), this occurs when the in-situ uniaxial compressive strength of the rock mass is less than about one third of the in-situ stress acting in the rock.

Estimates of rock mass deformation modulus can be obtained from field tests, but empirical correlations have generally shown to be quite reliable while cheaper and more practical (Hoek and Diederichs 2006). One of the most used empirical correlations for isotropic rock masses is developed by Hoek and Diederichs (2006). For anisotropic rock mass structure, the deformation modulus can be derived by the simplification of the problem to a rock mass with an “equivalent” isotropic deformation modulus. This demands the estimation of the joint stiffness of each of the joint sets in the rock mass and the explicit representation of the joints in the models. Joint stiffness can be then estimated from the deformability of the intact rock and the rock mass together with geometrical data (joint set spacing, S) as suggested by Barton (1972).

2.3 Ground Behavior

Ground behavior is dependent on the rock mass strength, deformability, structure, and the in-situ stresses acting on it. Different failure mechanisms can take place and be physically expressed by a variety of behaviors in the tunnel like

block falls, cave-in, plastic deformations, swelling, and failure of rock support, among others (Stille and Palmstrom 2008). Authors like Schubert et al. (2001) and Palmstrom and Stille (2007) developed methodologies for the assessment of Ground Behavior Types, GBT to identify the governing failure mechanisms in rock masses with focus on tunnel support design. Despite the comprehensive and detailed character of the mentioned methodologies, these are seldomly used systematically in hard rock tunnelling. Based on literature review and the study of the database utilized for this article (Sect. 5.3), a practical classification of GBT for poor ground conditions suited for the context of hard rock tunnelling is presented (Fig. 1). The in-situ rock stresses in the studied case records correspond to tunnel overburdens between approximately 1 tunnel span (in portals) to about 315 m, and absence of high horizontal in-situ stresses. The resulting behavior, the proposed classification (Fig. 1), and the interpretations that follow in this study are therefore limited to these boundary conditions.

- (a) Distinct weakness zone: distinct weakness zones of width up to $1/2 \times$ tunnel span, with weak and fractured infilling and/or gouge material, possibly with alteration minerals and clay seams that can develop loosening, ravelling, swelling and/or slaking. The poor quality and the reduced stiffness of the zone can lead to bulking of the zone and/or relative block movements at each side of the zone, deriving into differential tunnel deformations. Mixed ground conditions may fall within this category as well. That is, provided that different materials of different rock mechanical properties intersect the excavation, leading to some form of instability.
- (b) Low rock stresses: tunnels with low overburden (generally, $< 2 \times$ tunnel span), typically at portals and urban tunnelling. Failure mechanisms are related to the relatively low confining rock stresses, leading to low normal stresses acting on the joint planes, as well as the relative orientation of the joints to the opening. Typical failure is initiated by block sliding, resulting into possible rock fall and point loads on the support.
- (c) Anisotropic rock mass structure: layered rock masses in hard rock can lead to delamination and rock slab falls. Delamination is normally related to the combination of weak joint planes and unfavorable in-situ rock stresses (in terms of orientation and magnitude respect to the tunnel direction and the rock mass structure). Loosening of slabs can lead to differential deformations.
- (d) Hard and fractured rock mass: fractured rock masses which exhibit dense jointing whilst part of the original rock strength is conserved. The stability and behavior of the rock blocks around the opening will depend on the in-situ rock stresses and the ability of the rock to arch and on the utilized means of reinforcement. Block

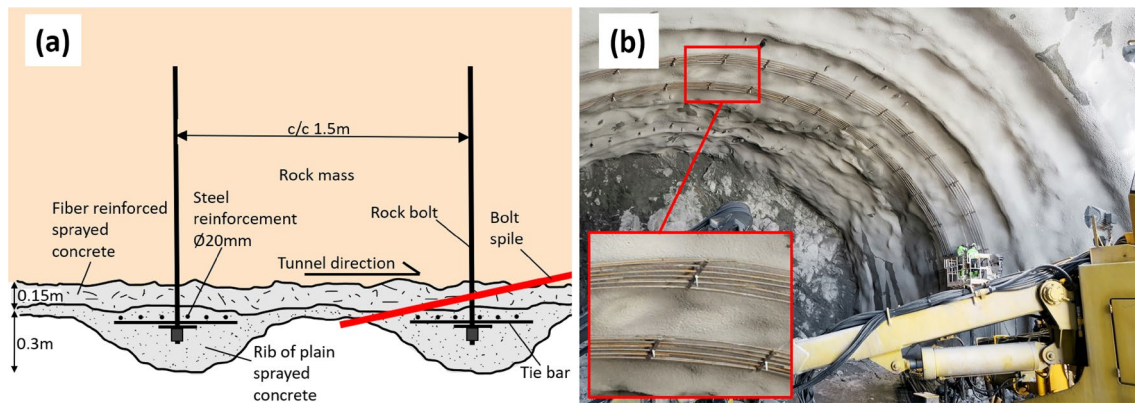


Fig. 2 **a** Typical longitudinal cross section with details of RRS-support configuration Si30/6 @ 1.5 m (Rib of reinforced sprayed concrete -Sfr- 30 cm thickness, reinforced with single steel layer containing

6×20 mm rebars; arches spaced 1.5 m). **b** Sequential construction of (arched) RRS-support behind the face in jointed rock (modified from Høien et al. 2019a, b)

falls, overbreak or even cave-in are typical failures, which can result in differential deformations and asymmetrical loads.

- (e) **Weak rock mass:** poor and weak rock masses composed by intensely fractured or crushed rock, rock material with low mechanical strength and clay seams, subjected to moderate in-situ rock stresses are prone to suffer plastic deformations. The distribution of deformations will vary as a function of the in-situ stresses (magnitude, orientation, and anisotropy), the rock support design, and the ground conditions. Time-dependent deformations may take place as well upon weakening of the rock and infilling material over time causing creep. In the case of overstressing of the rock material having ductile or deformable properties (i.e., weak rock and/or heavy jointed rock), the time-dependent deformations are termed squeezing.
- (f) **Swelling ground:** swelling is the time dependent volume increase of the rock mass caused by physio-chemical reaction of rock and water in combination with stress release, leading to (normally) inward movement of the tunnel perimeter (Palmstrom and Stille 2007). Swelling and squeezing often cause tunnel deformations and are normally interrelated (Einstein 1996).

3 Rock Support in Poor Ground Conditions

3.1 Rock Support Methods

Rock support methodologies based on Scandinavian practice for hard rock tunnelling differ from those used in Central Europe and Alpine countries. In the latter, the permanent support in soft- and squeezing-prone grounds is a double-shell structure composed of an initial, temporary support that

is supplemented by additional layers of mesh, sprayed concrete, lattice girders (LG), or steel sets, upon the observed ground behavior. Cast-in-place concrete lining is frequently added to satisfy functionalities such as load-bearing, reduced maintenance, waterproofing, friction, and aesthetics, among others. On the other hand, permanent rock support under the principles of Scandinavian hard rock tunnelling considers all the rock support materials to be of a permanent function and durable material quality. This means that the initial support is also part of the permanent support. The permanent support is therefore based on the combined application of fully grouted rock bolts, fiber reinforced sprayed concrete, and RRS-support, which together forms a final single-shell tunnel lining (Barton 2017; NFF 2017).

The RRS-support is normally installed on a primary layer of sprayed concrete (typically 15 cm thickness) which provides immediate support until the RRS is installed. The RRS arches are formed by ribs of sprayed concrete with embedded layers of steel reinforcement, disposed circumferentially, and bolted to the rock mass (Fig. 2a, b). Different configurations of RRS-support (designs) can be set by adjusting the longitudinal center spacing, thickness, the number of steel layers (Si-Single, or D-Double) and the number of steel rebars in the RRS-arches (Grimstad et al. 2002; Barton and Grimstad 2014). The RRS-support therefore provides design and construction flexibility since different RRS-designs can be configured to suit the expected load conditions.

A cast concrete invert can be added and connected to the tunnel support, thereby enhancing the overall ring-support effect in deformable ground. The addition of anchors in the invert or the upgrade to a curved invert shape are also methods used if the expected loads are significant. Rock bolt spiling, although not formally recognized as “rock support”, is also a common stabilizing method employed to reinforce poor and unstable rock masses ahead of the face to keep the

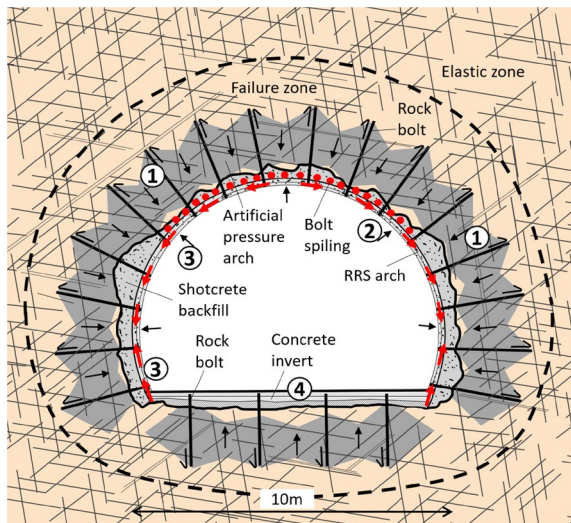


Fig. 3 Schematic illustration showing the principles of the mechanical interaction ground-support for a tunnel excavated in poor ground conditions. Hydrostatic and moderate stress conditions. Rock support consisting in a primary layer of fiber reinforced sprayed concrete, bolted RRS, and spiling. Numbering in figure as described below in the text

roof stable immediately after excavation. Cast concrete liners and LG are solutions used mainly in special cases with large and/or complex loading conditions.

Circular and semicircular tunnel geometries promote uniform load redistribution and transformation of the radial loads into axial stresses, minimizing then moment and shear forces (Oreste 2003). In that sense, the arched geometry of the RRS-support used in the static calculations done by Grimstad et al. (2002) to determine the capacity of different RRS-designs, suggests the use of arched geometries in the practice as well. In Norway, the requirement for *arched* geometry of load-bearing support in rock tunnels was adopted later for road tunnels. In particular, with the guidelines published in Pedersen et al. (2010), that defined requisites for arched geometry of load-bearing support with RRS (Fig. 4c, d). *Unarched* geometry in a load-bearing support is therefore that installed onto an uneven tunnel excavation profile, resulting in a non-circular geometrical shape (Fig. 4b).

3.2 Design Principles

A basic design principle in rock engineering of tunnels is to utilize, conserve and take advantage of the inherent self-bearing capacity of the rock mass (Engesser 1882; Rabcewicz 1969; Hudson and Harrison 1997; Kovari 2003; Brady and Brown 2006; NFF 2017). In practice, this is achieved by allowing the rock mass to deform prior to support installation with the objective of (1) triggering the generation of

a natural rock pressure arch and (2) release part of the rock load prior to support installation. In rock tunnels, the term natural pressure arch is frequently used to define the formation of a self-bearing arch which results from the increase of interblock compressive stresses -and frictional shear in joints- following the redistribution of in situ stresses and generation of induced tangential stresses upon tunnelling. The natural pressure arch can be represented in a simplified manner by a line which defines the innermost location and geometry of the induced tangential stresses surrounding the opening through rock in elastic state (as indicated by dashed line in Figs. 3 and 4). The pressure arch line therefore defines the boundary between self-bearing rock in elastic state -or elastic zone- and failed rock in the failure zone. In poor ground conditions, the natural pressure arch can be located further away from the tunnel contour, leading therefore to larger failure zones. The failure zone must then be stabilized, which is often done by the installment of rock support devices to create an artificial pressure arch (Li 2017). In the case of weak rock and squeezing ground, terminology like plastic zone (Fenner 1938; Hoek 1999) or yield zone (Duncan Fama 1993) may be also used for the failure zone containing plastic failure surrounding the tunnel (Fig. 4d).

For the understanding of the performance and mechanics of support systems in poor ground conditions, the classification of rock support from Kaiser et al. (1996) has been used. It is based on three main support functionalities and displays the mechanical interaction of each element with both the support system and the ground. Reinforcement refers to the measures used to limit rock bulking by conserving or improving the interlocking of rock blocks. Retention, the establishment of surface layers that retain or bridge the broken rock between the bolts. Holding, the action of securing either the rock, the retention layer, or both, by introducing support devices that resist against rock movement. Transfer is a fourth functionality not considered by the mentioned authors, but equally important in poor ground conditions. It refers to the action of distributing, conveying loads through the rock support. A more detailed and adapted description of the ground-support interaction of the typical rock support elements used in hard rock tunnelling is given in Fig. 3.

- (1) Rock bolts: the primary function is rock reinforcement since bulking and movement of blocks are restricted. If systematic rock bolts are installed, an artificial pressure arch of improved rock is formed as soon as the bolts are tensioned by rock movement (Lang 1961; Li 2017). The artificial pressure arch can then take part of the load-bearing function of the ground. Another functionality of bolts is to hold specific rock blocks or hold back (tie) retaining structures like layers of sprayed concrete and RRS-support.

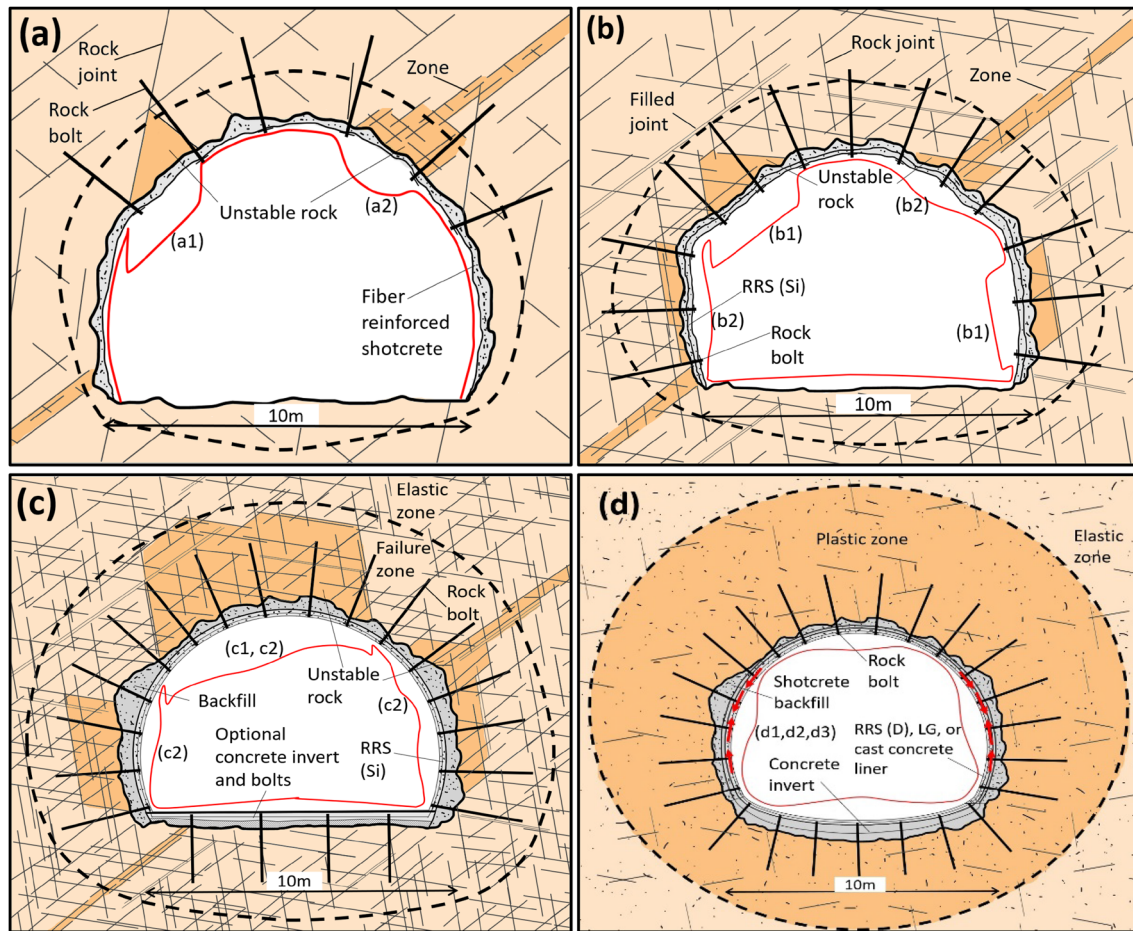


Fig. 4 Classification of tunnel support loading in the context of poor ground conditions in hard rock tunnels (moderate and hydrostatic in-situ stresses). In red, it is illustrated the assumed “deformed tunnel

profile” (exaggerated). **a** Local loading in hard and jointed rock, **b** local loading in poor rocks, **c** generalized loading in strongly jointed rocks, **d** global loading

- (2) Sprayed concrete: the primary function is the retention of loose rock not able to bridge between rock bolts. In poor ground conditions, sprayed concrete can also function as reinforcement since prevention of loosening maintains rock block interlocking. In the case of thick and circular sprayed concrete layers, the liners can then function as holding structures. Similarly, thick and arched liners of sprayed concrete can transfer loads as denoted by the red arrows (Fig. 3) resulting in conveyed tangential stresses in two opposite directions. One, originated from the transformation of radial ground loads to compressional load. The second, a reaction force of opposite direction but exerted by the rock foundation. If sufficient bonding rock-sprayed concrete is available, thrust can also be transferred back to the rock by shear resistance (Malmgren 2005).
- (3) Load-bearing support: RRS-support is extensively used for weakness zones and poor rocks in hard rock tunnels. The steel reinforcement of the RRS is normally bent and adjusted following the tunnel contour and

- so installed directly onto the tunnel surface (Fig. 2). Provided its arched geometry, RRS-support can act as a load-bearing structure. In case of differential loads and/or unarched tunnel geometry, the reinforcement of the RRS-support can also contribute to minimizing failure by distributing moments and increasing shear resistance. LG has, in general, a higher moment capacity than RRS-support. However, the construction of pre-shaped LG-segments in hard rock conditions with overbreak requires a backfilling of the LG’s extrados to achieve the needed mechanical interaction with the ground. This may require a longer time before the LG-support becomes mechanically functional, compared to RRS-support. Concrete liners as a final support structure are designed to work in compression and have the main function of holding or restraining deformations in deformable grounds with tunnel convergences over time.
- (4) Concrete invert: flat concrete inverts like in Fig. 3 have a limited support contribution, apart from restricting

horizontal deformations of the lower walls into the opening. If deformations are expected, a typical solution is the steel reinforcement and the anchoring of the flat invert to the underlying rock with rock bolts. In weak rocks with significant deformations, curved concrete inverts may be a better solution to hold uplift by utilizing the bearing capacity and transferring compressive stresses up to the walls as observed in Fig. 4d.

3.3 Design Methodologies

3.3.1 Empirical Approach

The empirical approach for tunnel rock support design is mainly based on the combined application of experience and rock mass classification systems. The Q-system (Barton et al. 1974) was developed to provide a practical engineering tool that could assist on the evaluation and classification of rock masses and rock support design. It was built mainly from experience in hard rock tunnels, but also from a few cases in weakness zones and weak rocks (Barton 2022). Accordingly, rock mass parameters judged to be more representative on the fundamental cause of tunnel instabilities were selected to form part in the empirical equation of the rock mass quality Q . Following the empirical rating and calculation of the mentioned rock mass parameters, a Q value is calculated. The obtained value is then assigned a support design following the recommendations in a support chart.

In poor ground conditions the Q-design chart contains several RRS-designs for $Q < 0.4$. The selection is normally done upon a mapped Q value, tunnel span, and the Excavation Support Ratio (ESR), an empirical measure of the safety required in the empirical design. The available RRS-designs in the latest versions of the Q-system (Barton and Grimstad 2014; NGI 2015) are the result of the research done by Grimstad et al. (2002). From an empirical evaluation of rock loads and deformations, the authors employed static calculations to back-calculate the necessary support in the form of RRS-arches of different thickness and amount of steel. This allowed the numerical incorporation of the current RRS-designs in the Q-system. In Norway, the Norwegian Public Road Administration (NPRA) has also published national guidelines for the empirical design of tunnel rock support in road tunnels (also applicable to Norwegian rail tunnels). The guidelines partly follow the Q-system, using the Q rock mass classes—but define their own and more conservative support classes (Pedersen et al. 2010).

Deriving rock engineering correlations from classification systems have been also attempted, such as the Q-deformation chart to derive deformation from the empirical relationship of tunnel span to Q value (Barton et al. 1994), and the competence factor of Grimstad and Barton (1993). The latter used to categorize subcategories of the Strength

Reduction Factor (SRF), and defined as the relationship between the maximum tangential stress (σ_θ) to the intact rock strength (σ_c).

However, rock mass classifications and quality indices alone present limitations to address ground behavior and permanent rock support in poor ground conditions (Palmstrom and Broch 2006; Anagnostou and Pimentel 2012). For example, important rock properties to evaluate the behavior of weak and/or mixed grounds like the rock strength can be addressed with the Q-system by normalizing Q values with uniaxial rock strength (Barton 2002), resulting in adjusted values of Q , (Q_c). However, design recommendations regarding the ground conditions where (Q_c) should be utilized instead of the conventional “hard rock” Q , or as to how utilize deformation monitoring in the Q-system to optimize designs in weak ground, are not found in the literature. Similarly, the Q-system considers the case of weakness zones and the multiple occurrences of them intersecting the excavation. The empirical approach does also present limitations in the sense that the relative width and direction of the weakness zones to the excavation are not covered, important parameters to define the stabilizing component provided by the rock arching into the competent, host rock (Løset 1990). More research is therefore needed to better describe and classify poor rock mass conditions. More elaborated design procedures and geomechanical data should be then used to supplement empirical designs of permanent tunnel rock support when facing poor rock mass conditions, as suggested by Hoek and Brown (1980a, b), Bieniawski (1997), Hudson and Harrison (1997), and Hoek (2007).

3.3.2 Analytical Approach

The analytical approach for rock support design in tunnels has been traditionally linked to the ground-support interaction approach. The Convergence Confinement Method (CCM) is an analytical model based on the ground-support interaction approach with an elasto-plastic solution of rock deformation around an excavation. CCM permits the study of the ground-support interaction in weak rock masses experiencing plastic deformation (Fenner 1938; Pacher 1964; AFTES 1978; Detournay and Fairhurst 1987; Carranza-Torres and Fairhurst 2000).

The basic components of the CCM are the Ground Reaction Curve (GRC), Support Characteristic Curve (SCC), and the Longitudinal Displacement Profile (LDP) as shown in Fig. 7a. The GRC represents the relationship between the gradual deformation of the tunnel boundary -in terms of radial displacement (u_r)- as a function of a decreasing internal pressure (p_i) that simulates the advance of the tunnel face. The SCC is defined by the relationship between the increasing support pressure (p_s) and u_r . The intersection of the SCC with the GRC determines the equilibrium pressure

(p_{eq}) where the tunnel stops converging in the CCM. If p_{eq} is then compared with the maximum allowable pressure of the support system (p_{max}), the factor of safety (FS) against support failure can be determined. The LDP can be used to determine the location of the face in the analytical model, a relevant tool to *reconstruct* deformations occurred ahead of the face (Vlachopoulos and Diederichs 2009). Similarly, the CCM permits the study of the best support timing in relation to the examination of loads and deformations in the tunnel.

However, there are several premises and limitations in the CCM. It assumes hydrostatic and lithostatic in-situ rock stresses (p_o), circular tunnels with evenly distributed support, and isotropic rock masses that can be described with the Hoek and Brown failure criteria (Hoek and Brown 1980a, b; Hoek et al. 2002). An important limitation is also that the elastic–plastic solution utilized by Carranza-Torres and Fairhurst (2000) is limited to the instantaneous -elastic and plastic- tunnel deformation that occurs due to tunnel excavation and the inherent loss of the supportive effect provided by the tunnel face. Time-dependent deformations from rock weakening over time are therefore not captured by the CCM.

The analytical approach considers other type of design calculations as well. For example, simple limiting equilibrium analyses of local stability for sprayed concrete shells. These are mainly based on structural engineering analysis techniques for straight beams (Timoshenko 1976; Holmgren 1992) or curved composite members (Oreste 2003; Carranza-Torres and Diederichs 2009).

3.3.3 Numerical Approach

The significant advances in numerical coding during the last decades have provided the designer with powerful numerical tools for rock support analysis and design. However, numerical analyses normally present several limitations. Many of the commercial rock engineering softwares that are typically used in rock support analyses in poor grounds have constitutive models written with elasto-plastic solutions. Such characteristic limits the calculated deformational behavior to that generated by the true tunnel excavation (instantaneous deformation), without estimating time-dependent deformations since the in-built solutions do not contain creep-laws in many of these softwares. For example, RS2 (Rocscience 2021).

Another limitation of numerical analysis is the accounting of anisotropy in the rock mass structure and the effect of distinct joints in the overall ground behavior. This obviously generates challenges on the decision of the proper modelling approach to use in jointed rocks between finite element modelling (FEM) and distinct element modelling (DEM). In general, the selection of one or another approach depends on the size of the excavation relative to the discontinuity

spacing, the imposed stress level and the orientation and strengths of the discontinuities, as suggested by Barton (1998) and Brady and Brown (2006). It should be similarly accounted the possible limitations derived from the use of FEM in hard and dilatant rock masses (upon failure), or the use of DEM in hard and strongly fractured rock masses that may be simplified to “equivalent” isotropic ground.

Because of the above limitations, calibration and verification of the models are normally recommended to minimize part of the potential uncertainty and deviations that the selection of the modelling approach (FEM or DEM) and the modelling itself may print in the results. In general, the use of deformation monitoring of tunnel sections provides a good basis for the needed back-calculations and calibration in the design process (Sakurai 2017). That is, the *a posteriori* ground behavior is likely the best rock engineering indicator reflecting the rock mass properties and ground conditions that are representative of a rock model. On the other hand, there might also be limitations to differentiate between elastic and plastic deformations in back-calculated models, where several combinations of rock mass properties and boundary conditions may give similar results in terms of total deformations.

Another important characteristic of the numerical approach is that it permits the evaluation of the support performance by the study of capacity diagrams (Hoek et al. 2008). They are graphical representations of the induced axial thrust (N) and the resulting bending moment (M) and shear force (V) acting on the reinforced liner, in relation to a plot of failure envelopes representing the maximum allowable state of load for the steel and sprayed concrete.

4 Loading and Failure of Tunnel Rock Support

In poor ground conditions, the evaluation of the permanent rock support design should include, in addition to a description and classification of the rock mass, a study of the associated loading of the support that may be derived from the ground behavior and its interaction with the rock support. Accordingly, a qualitative classification of tunnel support loading has been introduced (Fig. 4) for its practical implementation in design for poor ground conditions similar to those represented in Fig. 1 and Sect. 5.3. The classification presents four loading categories (a” to “d” in Fig. 4), and is based on the combined study of the ground-support interaction and the loading conditions that may be endorsed in rock support as a result of identified failure mechanisms in rock. In addition, identification of the type of failure is included and numbered accordingly for each of the loading categories, from shear failure (1) to flexural failure (2), and compressional failure (3). For simplicity, the interpretation

of loading and rock failure is based on a two-dimensional and in-plane assessment of the failure mechanisms and tunnel loading, and under the condition of hydrostatic and low-moderate in-situ rock stresses.

- (a) Local loading in hard, jointed rock: rock failure is mainly structurally controlled and has the form of either block falls or instabilities provoked by zones of fractured rock intersecting the excavation. Point loads and differential loads can induce shear failure (a1) if related to blocks, and flexural failure (a2) if related to fractured portions of the rock mass (Fig. 4a).
- (b) Local loading in rock masses of poor quality: mainly structurally-controlled rock failures which can promote overbreak and irregular excavation contours, leading to development of unarched zones between the natural pressure arch and the tunnel perimeter. Local addition of load-bearing support (i.e., RRS) may be then needed. Shear failure (b1) in unconfined rock portions, while flexural failure (b2) related to bulking of fractured and unconfined regions of the rock mass (Fig. 4b). Layered rock masses may fall within this category as well.
- (c) Generalized loading in strongly jointed rock masses: failure can be both structurally controlled in one hand and stress driven in the other hand. Although the rock mass conserves part of the original strength, the failure zone is enlarged due to the presence of weaker and more fractured rock. Overbreak, cave-in and/or mild-moderate deformations may be triggered (Fig. 4c), requiring arched load-bearing support (i.e., Si RRS), and anchored concrete invert depending on the loading conditions of the ground. Shear failure (c1) can be related to well defined and unconfined rock regions, while flexural failure (c2) linked to uneven loads provoked by asymmetric deformations. Layered rock masses with significant thinning can also fall within this category.
- (d) Global loading in weak ground: the rock mass is intensively fractured and has low strength. Stress-driven failure within the plastic zone can induce deformations of different magnitude and distribution as a function of the relative rock mass strength to the in-situ stresses, stress anisotropy, and the capacity of the load-bearing support (i.e., D RRS, LG, or cast concrete liners). Shear failure (d1) and flexural failure (d2) may result from a situation where buckling of the support is induced by an excess of compressive load, or upon ground conditions inducing differential deformations (Fig. 4d). If the compressive strength of the sprayed concrete liner is surpassed by the induced thrust forces, compressive failure (d3) may also occur.

5 Analysis Methodology of the Case Records

5.1 Analysis Methodology

The case record study is based on the analysis of the ground behavior and the performance of the rock support of monitored rock tunnels excavated in poor ground conditions. Back-calculations using the analytical and the numerical approach have permitted the study of the performance and stability of the in-situ rock support design (from Q-system) against the calculated tunnel support design which may have been applied if ground behavior had been considered. The results have been used further to derive improvements and recommendations in the current empirical design methodology of tunnel rock support. A summary and description of the methodology is presented in Fig. 5, followed by two worked case examples (Sect. 5.2) to illustrate the mechanics of the calculations in two different ground conditions.

- Step 1. The initial conditions in the analytical and numerical modes are set based on the registrations of rock mechanical data, face mapping, deformation monitoring, in-situ investigations, Q and GSI-values from the studied case records. The numerical analyses for Steps 1 to 3 have been done with the 2D rock mechanical software RS2 (Rocscience 2021).
- Step 2. Calibration of the models using deformation monitoring and back-calculations with CCM and RS2 has considered (1) the effect of time-dependent deformations, (2) the effect of non-registered deformations due to first readings of convergences done behind the face, (3) the effect of using elasto-plastic models in case records with hard rock and, (4) anisotropic rock mass structure.

As denoted from Table 6, time dependent deformations have been measured in convergence sections positioned outside the effect of the tunnel face. Since the measured time-dependent deformations are of little magnitude and the RS2 models are calibrated against the total (and nearly stabilized) registered deformation (instantaneous + time-dependent), the back-calculations are deemed to have minimized possible deviations.

The non-registered deformations have been reconstructed. This is because to allow calibration, one needs to compare the total deformations calculated with RS2 towards total deformations in the tunnel. The CCM approach has been then used to reconstruct the non-registered deformations (Sect. 5.2.1) for both ahead of the face and behind the face. The deformation ahead of the face cannot be measured but determined analytically from the construction of the LDP in the CCM (see Fig. 7a). The non-registered deformation behind the face is derived by utilizing the Vlachopoulos and

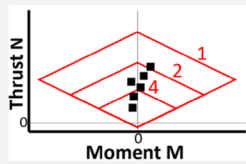
Working flow				
STEP	1	2	3	4
PROCESS	Model of Initial conditions	Calibration of numerical model	Analyses with FEM	Interpretation
OBJECTIVE	Modelling of initial conditions	Calibration of the model to the in-situ conditions	Evaluate tunnel deformations, ground loading and performance of rock support design	Find improvements in the empirical design methodology for tunnel support
INPUT	Rock mass properties and quality, in situ stress, boundary conditions, tunnel geometry, support design, joint conditions	Model in Step 1, input from CCM, and deformation monitoring	Calibrated model in Step 2	Results in Step 3, ground conditions, Ground Behavior Types (Fig. 1), ground-support interaction (Fig. 3), and loading categories (Fig. 4).
OUTPUT	Modelled Initial conditions	-Reconstructed deformation curves in the monitoring records -Calibrated model to reflect the (in situ) ground- and load-conditions	- FS obtained from capacity diagrams (RS2) or calculated analytically 	Overview of: -Rock mechanical conditions controlling ground behavior and tunnel support selection -Tunnel stability, performance of support, and design optimization - Possible improvements in the current empirical method of rock support in poor rock masses

Fig. 5 Methodology used for the analysis of ground behavior and the performance of empirical rock support. Description of the working steps is numbered and presented below the text

Diederichs (2009) approach that permits the study of tunnel deformations with respect to the tunnel face. The total and reconstructed deformation is then compared to the total deformation in RS2 in an iterative process until fit is reached in the RS2 deformations. Then the RS2 model is deemed calibrated.

Poor quality rock masses of $GSI < 40$ (ca. $Q < 1$) could in general be assumed as presenting perfectly plastic post failure behavior with no associated volume change. However, in the context of hard rock conditions like in some of the case records, the post peak strength of rock mass can be reduced in the RS2 models (ca. 10–25%) in compensation to the dilatant behavior that these rocks may have upon failure.

In the case of anisotropic rock mass structure, an explicit joint network was modelled in the RS2 models, involving the Barton (1972) approach for determining the stiffness properties of joints, and the Barton and Bandis (1990) shear strength criterion of discontinuities.

For the modelling of load bearing support like RRS in RS2, the equivalent section approach has been used. It is a well-known procedure to derive the equivalent thickness of sprayed concrete and spacing of steel in composite supports of varying thickness and embedded steel in the tunnel direction. If well implemented, such simulation has been proved effective as suggested by Carranza-Torres and Diederichs (2009), Chrystanthakis (2015), and Høien and Nilsen (2018).

- Step 3. The calibrated models have been used not only to study the ground behavior (tunnel deformation) but also the performance of the support through the examination of the capacity diagrams produced in RS2. A factor of safety is then derived by RS2 for the sprayed concrete and steel in composite supports from the computation of the acting bending moments (M) and shear forces (V) against the moments of resistance (M_R) and the shear resistance (V_R) of the support as defined by Eqs. 1, 2, 3 and 4.

$$FS_{c-bending} = \frac{M_{RC}}{M} \tag{1}$$

$$FS_{c-shear} = \frac{V_{RC}}{V} \tag{2}$$

$$FS_{s-bending} = \frac{M_{RS}}{M} \tag{3}$$

$$FS_{s-shear} = \frac{V_{RS}}{V} \tag{4}$$

where $FS_{c-bending}$ is the factor of safety of the sprayed concrete to fail in bending, M_{RC} is the moment of resistance of the sprayed concrete, $FS_{c-shear}$ the factor of safety of the concrete to fail in shear, V_{RC} the shear resistance of the sprayed concrete, $FS_{s-bending}$ the factor of safety of the steel to fail in bending, M_{RS} the moment of resistance of

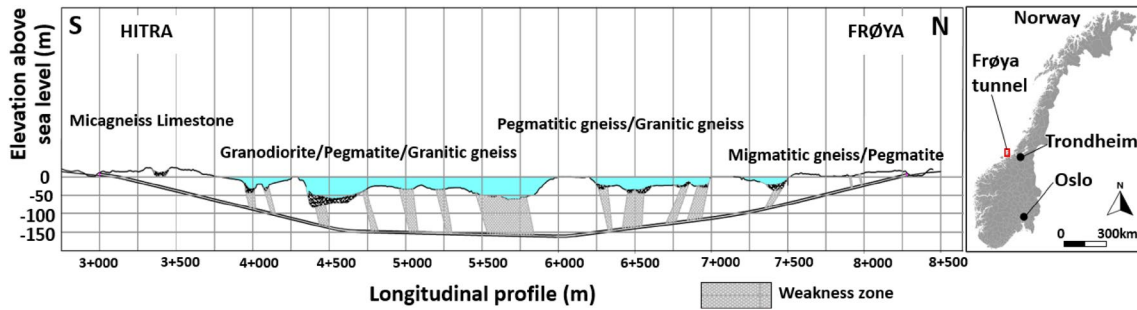


Fig. 6 Longitudinal profile and location map for the subsea Frøya tunnel. Courtesy of Statens vegvesen

Table 1 Rock mass properties and in-situ stresses used for the numerical modelling in the Frøya tunnel 5 + 585

Tunnel	Profile	Q value	GSI	E_i (GPa)	σ_{ci} (MPa)	1m_i	2E_m (GPa)	$^3\sigma_{cm}$ (MPa)	ν	σ_v (MPa)	σ_h (MPa)
Frøya	5 + 585	0.14	23	25	35	20	1.33	2.72	0.3	3.3	3.3

¹Hoek-Brown material constant of intact rock (Hoek et al. 2002)

²Generalized Hoek and Diederichs (2006) approach

³Generalized Hoek–Brown criteria (Hoek et al. 2002)

the steel, $FS_{s-shear}$ the factor of safety of the steel rebars embedded in the sprayed concrete to fail in shear, and V_{RC} the shear resistance of the steel rebars.

In the case records where the sprayed concrete is not steel reinforced, but only formed by fiber reinforced sprayed concrete liners and bolts, RS2 cannot produce capacity diagrams. Then, M_{RC} and V_{RC} have been analytically calculated following the recommendations of Holmgren (1992) and Bernard (2008). See Eqs. 5 and 6.

$$M_{RC} = \sigma_{f1} \frac{t^2 s}{6} \quad (5)$$

$$V_{RC} = \sigma_{sh} t s \quad (6)$$

where (σ_{f1}) is the flexural capacity of the fiber reinforced sprayed concrete, (t) is the thickness of the sprayed concrete layer, (s) the critical perimeter, and (σ_{sh}) the shear strength of the sprayed concrete.

Rock bolts have been modelled as well in the RS2 models. But for simplification purposes, the results have not been explicitly presented in this article. Assuming that failure of the fully bonded bolts occurs in the steel, the limit state at axial failure of the bolt (FS_b) is defined by Eqs. 7 and 8.

$$FS_b = \frac{T_b}{Q_b} \quad (7)$$

$$T_b = A_b f_b \quad (8)$$

where (T_b) is the tensile capacity of the bolt, (Q_b) the axial load acting on the bolt, (A_b) is the cross sectional area of the bolt and (f_b) is the yield strength of the steel.

- Step 4. Interpretation has been possible through the examination of the results. Spreadsheets in Excel (Microsoft 2022) were programmed to study and plot selected combinations of selected data series. As observed through the results, focus was put into the investigation of design optimization possibilities through the study of rock mass properties having an impact on ground behavior and support but not sufficiently accounted in the Q-system. The findings have finally served to build an integrated or hybrid design procedure.

5.2 Examples of Case Records Analysis

5.2.1 Example 1–The Frøya Tunnel (5 + 585)

The Frøya tunnel is a road tunnel in the western part of Central Norway, connecting the islands of Hitra and Frøya by a 5.3 km long subsea tunnel. A 3.6 km long portion of the tunnel is located under the Frøya fjord. Approximately 1.7 km were excavated in fault zones of rock mass ground quality $Q < 1$, (Fig. 6). During construction, systematic deformation monitoring was carried out due to deformations in the weakness zones. A total of 27 tunnel sections were monitored (Holmøy and Aagaard 2002) and used to verify the design of the permanent support, in addition to the recommendations given by the Q-system.

Table 2 Material properties of the rock support modelled for Frøya tunnel 5 + 585 and Skarvberg tunnel 2 + 228 (Sect. 5.2.2)

Parameter	Value
Thickness of sprayed concrete (m)	0.26–0.29 ¹
Compressive strength of sprayed concrete (MPa)	35
Young’s modulus of sprayed concrete (GPa)	30
Tensile strength of sprayed concrete (MPa)	3
RRS rebar diameter (m)	0.02 ¹
RRS Rebar spacing -evenly distributed- (m)	0.25 ¹
RRS rebar tensile strength (MPa)	400 ¹
Bolt modulus (GPa)	200
Bolt tensile capacity (MN)	0.188
Bolt residual tensile capacity (MN)	0

¹Applies for Skarvberg tunnel 2 + 228, with equivalent distribution of thickness in sprayed concrete and steel reinforcement

The tunnel section at profile 5 + 585 represents ground conditions of very poor quality with Q -value 0.14. A wall-to-wall convergence of 13 mm was monitored with the first measurement section installed 4 m behind the face and prior to the installation of the concrete invert. The input parameters for the rock mass together with the characteristics of the rock support design are given in Tables 1 and 2, respectively. Rock properties and GSI-values have been determined from literature (Høien et al. 2019a, b) and calibrated towards monitored deformations. The in-situ stresses have been interpreted from the rock-and-water overburden and a non-published NGI report dated 1998 that was produced to perform numerical analyses of tunnel stability at weakness zones of the Frøya tunnel.

The back-calculation process considering the analytical calculations with CCM and the RS2 calculations are presented in Figs. 7 and 8, respectively. By setting the parameter “distance to the face” (L) equal to zero in the graphical CCM construction, the elastic deformation occurred ahead of the face can be depicted from the intersection of the LDP with the tunnel face ($L=0$ in Fig. 7a). This yields a radial

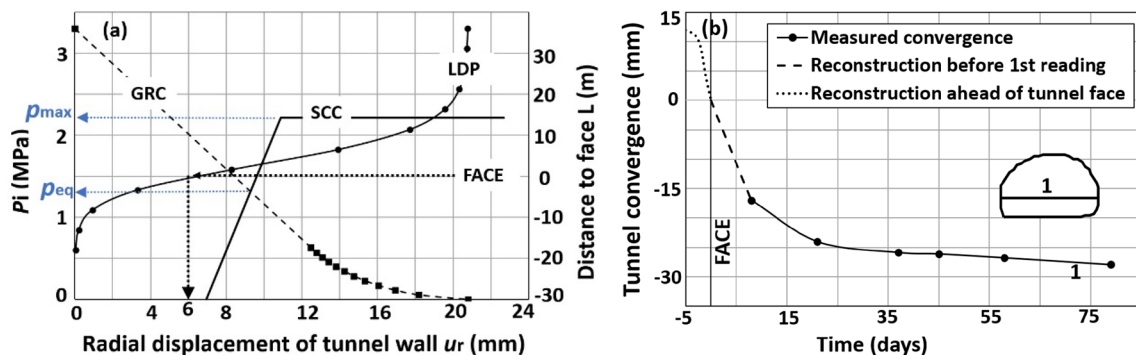


Fig. 7 a CCM diagram for the Frøya tunnel profile 5 + 585, calculated after Carranza-Torres and Fairhurst (2000) and Vlachopoulos and Diederichs (2009). b Reconstruction of tunnel deformation curve

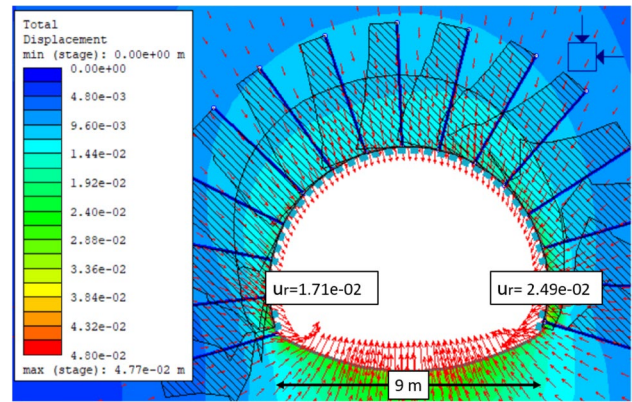


Fig. 8 RS2 model in isotropic and poor ground conditions at profile 5 + 585. Distribution, orientation and the relative magnitude of total displacements and axial loading in bolts and support are plotted. Rock support consisting in a liner of fiber reinforced sprayed concrete $t=26$ cm, bolting c/c 1.2 m (cast concrete invert installation delayed approx. 120 days from tunnel face)

displacement $u_r = 6$ mm (12 mm convergence). The elastic–plastic deformation occurred between the face and the first reading has been estimated using a separate analysis with the “core replacement” approach of Vlachopoulos and Diederichs (2009), yielding $u_r = 7$ mm (14 mm convergence). The measured convergence at 5 + 585 in segment 1 was 13 mm (Fig. 7b). As a result, the reconstructed and actual convergence (12 + 14 + 13 mm) sums 39 mm which, if compared with the numerical calculation in RS2 (convergence 42 mm, Fig. 8) denotes a reasonably good fit which confirms the calibration of the model.

The computed bending moments and shear forces in the calibrated RS2 model are $M=0.03$ MNm and $V=0.05$ MN, respectively. Rock bolts in the side-walls of the tunnel are loaded to max. $Q_b=0.14$ MN. From Eqs. 5 and 6, M_{RC} and V_{RC} of the liner have been analytically calculated. Similarly, the axial load capacity of the 20 mm diameter bolts ($f_b = 500$ MPa) has been used to evaluate the FS of rock bolts following Eqs. 7 and 8. The results are presented in Table 3.

Table 3 Loading condition of tunnel support for the Frøya tunnel profile 5 + 585

M (MNm)	M_R (MNm)	$FS_{c\text{-bending}}$	V (MN)	V_R (MN)	$FS_{c\text{-shear}}$	T_b (MN)	Q_b (MN)	FS_b
0.03	0.045	1.5	0.05	0.624	12.48	0.18	0.14	1.28

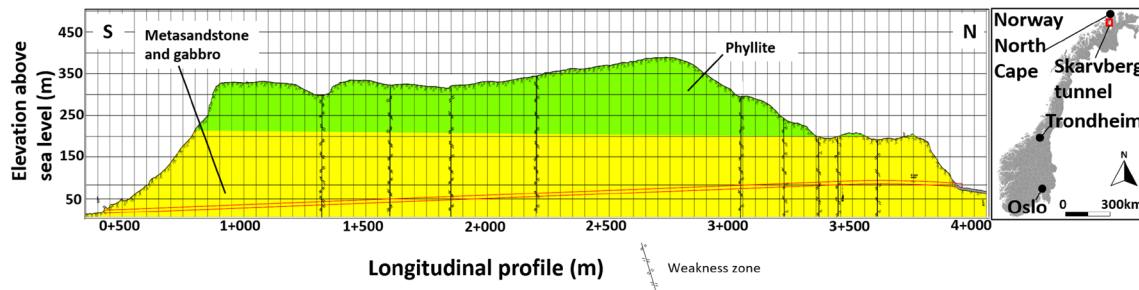


Fig. 9 Longitudinal profile and location map of the new Skarvberg tunnel. Statens vegvesen (2017)

Table 4 Rock mass properties and in-situ stress used for the numerical modelling of Skarvberg tunnel profile 2 + 228

Tunnel	Profile	Q value	GSI	E_i (GPa)	σ_{ci} (MPa)	1m_i	2E_m (GPa)	$^3\sigma_{cm}$ (MPa)	ν	σ_v (MPa)	σ_h (MPa)
Skarvberg	2 + 228	0.3	35	70	170	19	7.94	29	0.32	7.5	5

¹Hoek-Brown material constant of intact rock (Hoek et al. 2002)

²Generalized Hoek and Diederichs (2006) approach

³Generalized Hoek–Brown (Hoek et al. 2002)

Table 5 Geometrical and mechanical properties of non-filled joints of Skarvberg profile 2 + 228 (tunnel azimuth 180°)

Discontinuity	Dip direction/dip (°/°)	J_r ¹	S (m)	ϕ_r ¹ (°)	JRC ¹	JCS ¹ (MPa)	K_n ² (GPa/m)	K_s ² (GPa/m)
Bedding S_0	340/10	1–1.5	0.2–0.6	28	2–4	150	29	10.4
Joint set J_1	310/70	1.5–2	2–6	29	6–8	170	4.5	1.6
Joint set J_2	160/80	1.5–2	6–10	29	6–8	170	1.8	0.6

¹From joint characterization after Barton et al. (1974) and Barton and Bandis (1990)

²Joint normal stiffness (K_n) and joint shear stiffness (K_s) estimated after Barton (1972)

Satisfactory FS is found for the three stability conditions analyzed.

5.2.2 Example 2–The New Skarvberg Road Tunnel (2 + 228)

The busy E69 highway to the North Cape in Northern Norway is being upgraded by NPRA. This includes the construction of the new 3.5 km long Skarvberg tunnel to replace the old Skarvberg tunnel, in operation since 1968. The new Skarvberg tunnel has been excavated through sequences of hard and layered metasandstones with repetitive and inter-layered intrusions of metagabbro. The bedding planes and

the foliation are subhorizontal, penetrative and formed by weak discontinuities that often generated tunnel roof delamination problems during excavation (Fig. 9).

The design of the permanent rock support in the new Skarvberg tunnel was mainly done on the basis of the Q-system and the NPRA-guidelines (Statens vegvesen 2020). RRS and spiling was used extensively in tunnel sections through zones or in areas with dense bedding planes and/or roof delamination. In fact, the poor and unstable ground conditions lead to an increased use of both RRS and rebar bolt spiles corresponding to 13.4 and 6.6 times higher than the design prognosis (Gildestad and Bakk-evold 2021). Despite of the heavy support measures, a

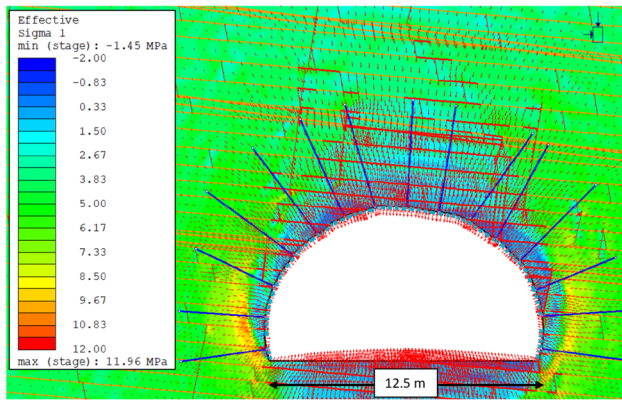


Fig. 10 RS2 model with explicit joint network for Skarvberg tunnel 2+228. Visualization of distribution of principal stress, yielded joints, and displacement vectors

roof collapse took place on March 2020 at profile 3 + 162, involving significant damages in the RRS-support in a 9 m long tunnel section. An investigation with further studies and a monitoring program of the rock mass was initiated for the entire tunnel. Details of these investigations are documented in Bøgeberg and Skretting (2021). The relevant rock and joint properties are presented in Tables 4 and 5, respectively. The closest 2-dimensional stress measurement was done at profile 2 + 886 (SINTEF 2020). Whilst the vertical stress (σ_v) was in good agreement with the overburden (ca. 225 m \approx 6 MPa), the measured in-plane horizontal stress (σ_h) showed variations in the range of 0 to 10 MPa over a distance of a few meters near the excavation reflecting the different stiffnesses and stress concentrations of the layered rocks. The virgin horizontal stress was interpreted to 5–6 MPa.

Finite element numerical modelling with explicit modelling of the joints has been used to simulate the ground mass behavior and support performance in the monitored section at 2 + 228. The rock support in this section consists of fiber reinforced sprayed concrete with thickness 15 cm, 4 and 5 m long rock bolts and RRS Si30/6 c/c 1.5 m. The deformation monitoring from the roof borehole extensometer placed at 2 + 228 was used to calibrate the model. Since the first reading was done when the tunnel face was still very close, it has been assumed that only 1/3 of the total deformation in the hard rock had taken place ahead of the face (Hoek et al. 1997). The calibrated model with plot of the major principal stress and distribution of displacements is shown in Fig. 10.

From the distribution of stresses, yielded joints and the displacement vectors in relation to the rock mass structure, it becomes rather clear that the ground behavior is influenced by the layered rock mass structure and the relatively low horizontal in-situ stresses. As observed in Fig. 10, an unconfined “cone” is generated right above the tunnel roof and between the two tunnel abutments, defined by decompressed

rock (bluish colors) and slipped/detached joints (thick red overprint lines). This motivates the migration of the natural pressure arch further into the rock mass, which derives into a bigger ground load to be taken by the support. In this case, the exerted load on the support in the calibrated model has yielded a FS for the respective bending and shear in concrete of 2.5 and 10, while the FS for bending and shear of the embedded steel (of the RRS-support) is above 10 for both cases.

5.3 Database Description

The database used in this article is formed by 118 case records from 10 different tunnels with a D-shaped excavation section. The studied cases are mostly from Norwegian road projects constructed during the last twenty five years and have been grouped in Table 6 mainly according to tunnel project. The tunnel span for most of the case records is ca. 9 m, but several cases have a span of 12.5 m or even larger. In all the tunnel projects where the case records are collected, the Q-system was used during construction as a basis for permanent rock support design. The database has been mainly limited to case records with rock mass quality $Q < 1$ since this is the approximate boundary where rock masses start indicating issues with the self-bearing capacity (Barton et al. 1980).

The case records are mostly represented by tunnels excavated in hard rock, except for a hydropower tunnel project excavated in weak flysch rock, which represents 19 case records out of the total. Nevertheless, documentation of the ground conditions and design of rock support were made with the Q-system for both geological conditions, hard Norwegian rock, and weak/soft flysch rock. The use of the Q-system in the studied cases includes rock masses exhibiting both isotropic and anisotropic rock mass structures. Anisotropic rock mass structure is only represented by the new Skarvberg tunnel, with 27 case records.

Each of the case records represents datasets containing geomechanical, geological and monitoring data from tunnel sections in poor ground conditions. The geological and geomechanical data are mostly collected from tunnel mapping, deformation monitoring, in-situ- and lab-testing performed by geologists and tunnel personnel or interpreted from literature review when unavailable.

The GSI-values correspond to correlated values that have been calibrated in the back-calculation process, except for the case records of the anonymized tunnel site where GSI was mapped. The two last columns in Table 6 represent back-calculated tunnel displacements in calibrated RS2 models, either for the wall or for the roof. Back-calculated displacements correspond to the total inwards displacements in the support from support installation to equilibrium (i.e.,

Table 6 Summary of the studied 118 case records in poor ground conditions

Tunnel	No. cases	Span (m)	Year of completion	Ground Behavior Type ⁵	Main rock type	Overburden ⁶ (m)	Q	GSI	σ_{cm} (MPa)	E_m (GPa)	Measured displac. tunnel wall (mm)	Distance of face at first reading (m)	Back-calculated wall displac. ⁸ (mm)	Back-calculated roof displac. ⁸ (mm)
Anonym ¹	1	6.3	2017	Weak rock	Ophiolite	245	0.35	28	5.93	3.58	3.5	28	18	7
Anonym ¹	1	6.3	2017	Weak rock	Serpentinite	350	0.05	25	4.97	2.99	9.55	12	30	12
Anonym ¹	19	6.3	2017	Weak rock	Flysch	196–290	0.04–1.13	22–37	2.33–4.68	1.49–4.17	2–39.05	0.5–27	11–40	4–20
Skarvberg ²	26	12.5	2022	Anisotropic	Metasandst	151–284	0.01–1.9	15–45	5.81–26	2.45–15.7	0.1–14 ⁷	3–55	1.3–41	3.2–48
Skarvberg ²	1	20	2022	Anisotropic	Metasandst	285	1.3	38	20.73	9.05	14 ⁷	20	5.5	19
Løren ²	2	12.5	2013	Fractured	Sandstone	32–35	1.04–2.5	48–50	9.8–10.43	10–12.9	2.16–2.39	0	1	2–3
Sørås ²	2	9.5	2016	Fractured	Gneiss	32	0.25	36	10.82	6.07	0.6–1.95	6	4–6	0.1–2
Rå ²	3	9.5	2016	Fractured	Gneiss	27–35	0.15–2.67	27–46	3.53–13.7	2–11.9	0.3–1.15	14–75	1.7–8.2	0.05–0.2
Nordkapp ³	1	9	1999	Fractured	Mylonite	60+130	0.55	36	3.16	3.03	4.22	200	35	20
Nordkapp ³	5	12	1999	Weak rock	Schist	(53–77)+(70–140)	0.008–0.033	15–20	1.69–2.41	0.98–1.23	0.43–5.96	3–148	30–41	14–19
Nordkapp ³	11	9	1999	Weak rock	Schist	(56–81)+(60–120)	0.008–0.017	15–20	1.69–2.41	0.98–1.23	0.1–5.88	9–140	21–45	13–21
Nordkapp ³	1	9	1999	Zone (1.5)	Schist	73+110	0.1	23	2.69	1.44	0.07	100	25	14
Honningsvåg ²	11	9	1999	Fractured	Gabbro	200–320	0.015–0.021	20	8.69	3.42	0.02–8.49	30–60	3–14	5–10
Honningsvåg ²	1	9	1999	Fractured	Hornfjels	95	3.75	50	26.92	24.57	0.06	30–60	0.3	1.5
Honningsvåg ²	2	9	1999	Fractured	Schist	100–210	0.37–0.62	29–37	7.03–9.36	1.9–3.25	0.8–0.87	30–60	2–14	5–16.5
Honningsvåg ²	2	9	1999	Weak rock	Gabbro	335	0.015	20	8.69	3.42	0.41–1.26	30–60	13	10
Frøya ³	6	9	1999	Fractured	Gneiss	(101–120)+(24–55)	0.11–0.22	23–28	2.72–6.84	1.33–2.15	0.12–6.52	4–380	3–11.7	3.5–10.25
Frøya ²	3	9	1999	Low stress	Gneiss	111–19	0.12–0.25	26–29	6.42–7.06	1.9–2.29	2.75–10.45	30–60	23–28	14–27
Frøya ³	12	9	1999	Weak rock	Gneiss	(52–103)+(22–36)	0.003–0.1	10–22	1.04–2.80	0.76–1.26	0.13–1.37	30–250	12–35	8–17
Frøya ²	1	9	1999	Zone (1)	Gneiss	55	0.25	18	1.37	1.03	6.75	202	8	7
Frøya ³	1	9	1999	Zone (1.5)	Gneiss	121+25	0.18	25	6.21	1.79	0.19	8	3	3.5
Frøya ³	1	9	1999	Zone (3)	Gneiss	114+34	0.12	20	5.30	1.37	0.25	270	8	8
Frøya ²	1	9	1999	Zone (5)	Gneiss	31	0.015	19	1.91	1.08	5.55	70	37	36
Frøya ³	1	9	1999	Zone (4)	Gneiss	57+21	0.01	17	1.79	0.99	2.07	6	17	9
Frøya ³	1	9	1999	Zone (0.5)	Gneiss	98+24	0.33	30	7.29	2.44	2.32	120	4.8	4.5

Table 6 (continued)

Tunnel	No. cases	Span (m)	Year of completion	Ground Behavior Type ⁵	Main rock type	Overburden ⁶ (m)	Q	GSI	σ_{cm} (MPa)	E_m (GPa)	Measured displac. tunnel wall (mm)	Distance of face at first reading (m)	Back-calculated displac. ⁸ (mm)	Back-calculated roof displac. ⁸ (mm)
Finnfäst ³	1	8.5	2008	Weak rock	Gneiss	90+50	0.01	20	1.98	1.14	1.1 ⁷	0	23	9.5
Bærum ⁴	1	14	2007	Low stress	Micaschist	4.5	0.43	29	0.95	0.76	7 ⁷	0	0.5	2

¹Hydro

²Road

³Road Subsea

⁴Rail

⁵Zone width (m) in parenthesis

⁶In subsea tunnels, rock cover is the first digit or interval and separated of water cover (second digit or interval) with the + sign

⁷Displacement registered in the roof

⁸Back-calculated displacements with support

the sum of the two deformation segments behind the face in Fig. 7b).

From 2010 on, load-bearing support in the studied Norwegian cases needed to conform with the design guidelines published by Pedersen et al. (2010). The rock support in the studied cases therefore consisted of load bearing support types, either as RRS-support or by thick liners of fiber reinforced sprayed concrete. Bolt spiling of the tunnel roof was also used in part of the case records. However, it was impossible to account for this in the back calculations since the extent during execution and design was unknown. Pre-grouting of the rock mass was also done in some of the case records. The stabilizing effect of the pre-grouting is similarly unaccounted for in the analyses. This is further discussed in Sect. 8.

6 Analysis Results of Ground Behavior and Rock Support Performance

6.1 Prediction of Ground Behavior

The prediction of the ground behavior of the case records has been studied with three different methodologies (Fig. 11). The study has similarly served to evaluate the performance of the three methodologies to capture ground behavior, as a function of rock mass quality Q, ground behavior type, and the calculated tunnel strain (ϵ). Tunnel strain is defined as the ratio of the tunnel closure to tunnel span.

One of the most notable features in Fig. 11 is the debranched distribution of the tunnel strain predicted by the empirical Q-system, if compared with the other two prediction models, the analytical CCM and the numerical analyses. In Fig. 11a, the end of the overlap in the calculated strain for the three prediction models at Q 0.1 suggests that prediction of tunnel behavior (and support design) should be accompanied by one or more methods when $Q < 0.1$. The debranched geometry of the empirical Q-deformation prediction does similarly suggest that relevant properties of rock masses affecting ground behavior when Q is lower than 0.1 are unlikely well addressed. In other words, the properties of a rock mass with quality Q 0.01 can make the tunnel to not necessarily exhibit a tunnel strain of 10% (though close to collapse), but other type of failures where deformation may of course be involved as well.

In Fig. 11a, it is also observed that the empirical prediction of strain (for $Q > 0.1$) is mostly overlying the numerical and the analytical prognoses. It may be argued that the Q-system is conservative since its prediction model is based on successful and *a posteriori* tunnels. However, it may be also argued that the application of CCM and FEM models in hard rock can overestimate tunnel strain. In the context of this discussion, the empirical prediction of tunnel strain for

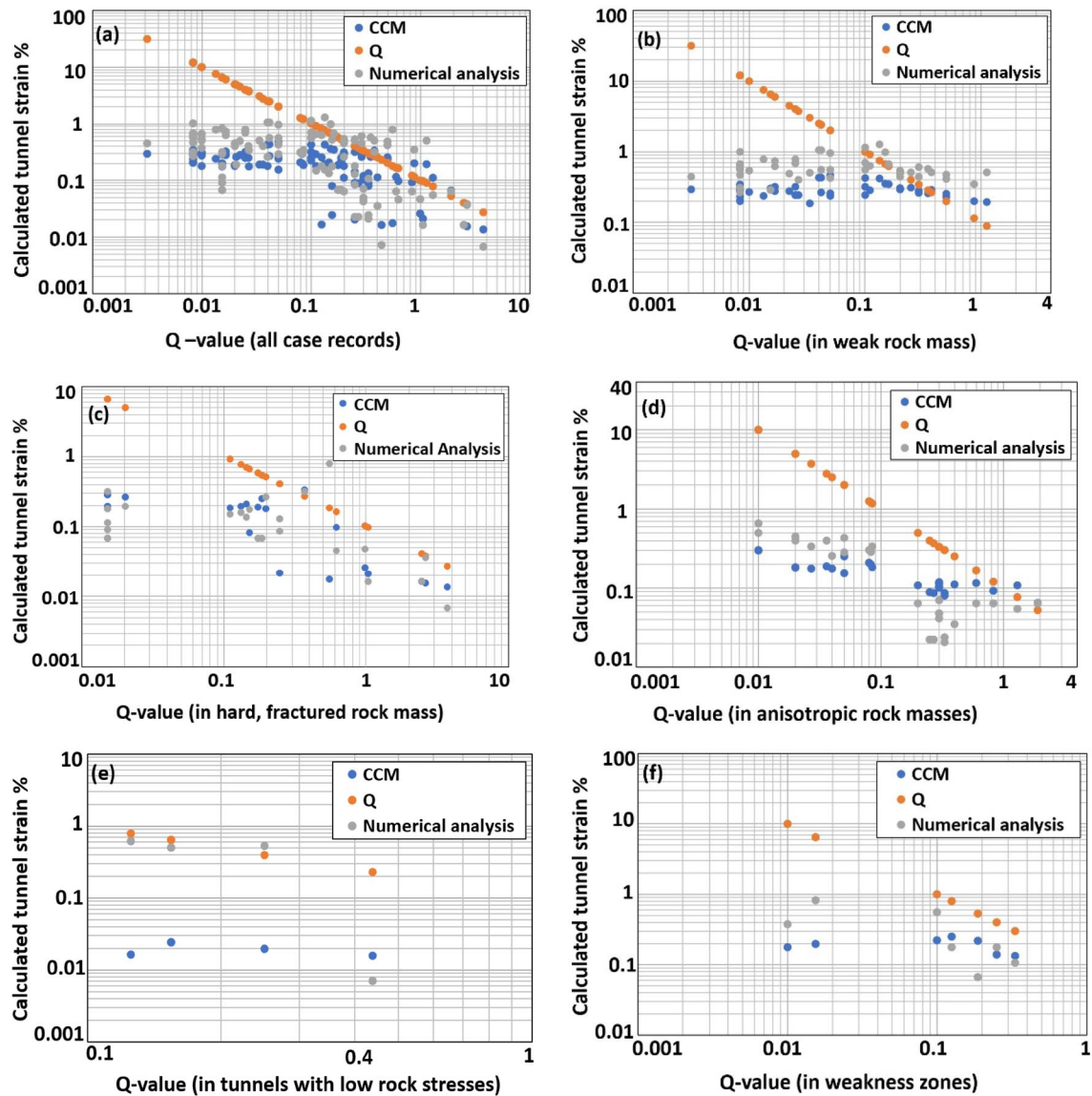


Fig. 11 Prediction of ground behavior on the basis of rock mass quality Q , GBT, and calculated tunnel strain with empirical, analytical, and numerical approaches in supported tunnel case records

$Q > 0.1$ seems to be projected over to the trend for $Q < 0.1$. This gives a prediction of tunnel convergence around 10%, for tunnels excavated and supported in rock masses with quality $Q \approx 0.01$, which is significantly higher than the values obtained in the monitored and calibrated tunnel sections presented in this study. It may be then argued that the tunnel strain calculated with calibrated CCM and FEM (for $Q < 0.1$) can be influenced by specific design requisites to keep deformations below specific levels in the case records. But, provided the magnitude and the increasing deviation of calculated tunnel strain with declining values of Q in Fig. 11a, and the limitations to derive ground behavior when $Q < 0.1$, involvement of more elaborated methods seems a valid recommendation when $Q < 0.1$.

The Figs. 11b–f show the calculated tunnel strain, split in different ground behavior types as described in Sect. 2.3. Weak rock masses as shown in Fig. 11b have been included in this study for those meeting the condition $\sigma_{cm}/p_o < 1$. It is observed that the CCM tends to yield less strain in general than the calibrated numerical calculations. This tells about the mentioned limitations of CCM to capture deformations in hard rock and the theoretically greater support effect of considering circular tunnels if compared to the modelled D-shape tunnels.

For tunnels excavated in hard and isotropically fractured rock masses (Fig. 11c), the empirical prognosis of tunnel deformation seems to correspond reasonably well with that of the numerical method when $Q > 0.4$. Supplementary

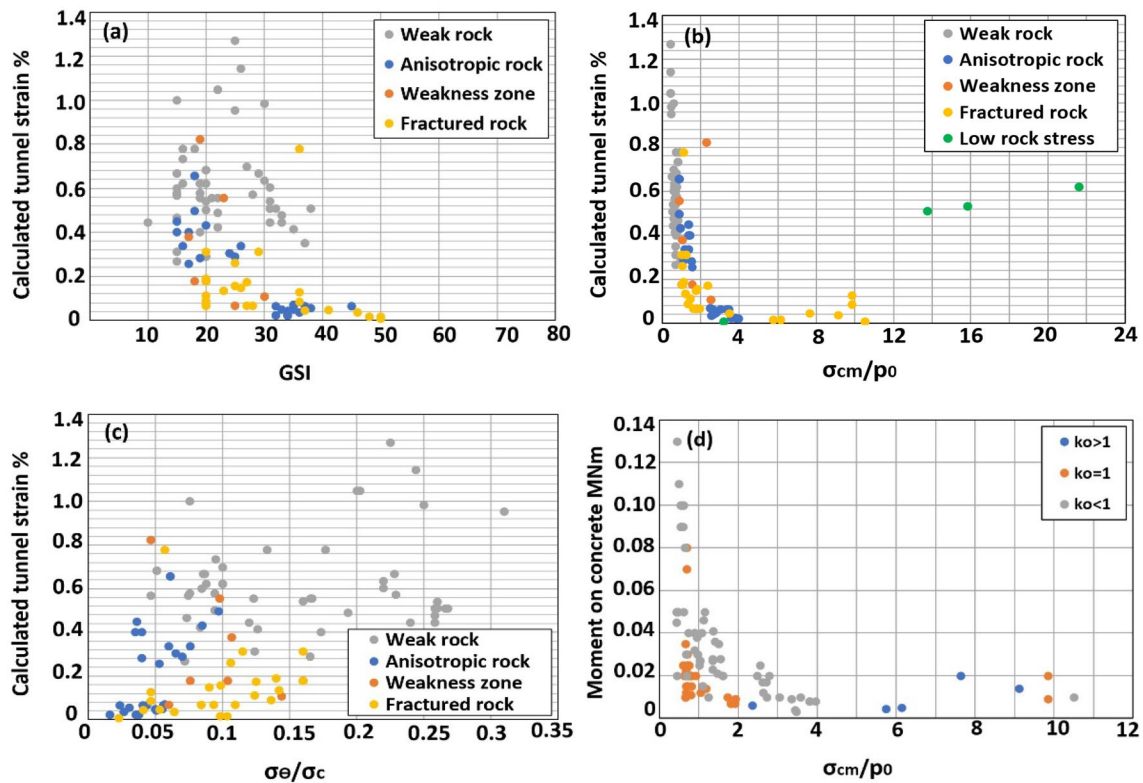


Fig. 12 Calculated tunnel strain and support loading as a function of rock mass properties, GBT, and in-situ stresses in supported tunnel case records

design approaches should be adopted in hard rock conditions with rock mass quality Q below 0.4. In rock masses with an anisotropic structure (Fig. 11d), prediction models like the CCM and the empirical method have limited applicability since they are built upon the premise of rock isotropy. It could also happen that the empirical prediction becomes appropriate in cases where the rock mass anisotropy is reduced by the relative increase of joint spacing in relation to tunnel span, as denoted from Fig. 11d for $Q > 1$ (Skarvberg tunnel with 12.5 m span). Independently, the applicability of the empirical method to predict ground behavior of anisotropic rock masses should normally be based on a study of the relative joint spacing and the in-situ stresses (Perras and Diederichs 2009).

The “low rock stress” category (Fig. 11e) correspond to monitored case records in portals and urban tunnels with an overburden less than 2 tunnel spans while Fig. 11f represents distinct and narrow weakness zones with width between 0.5 and 5 m. The amount of case records in these two GBT is limited. It can, however, be anticipated that in any of the two GBT the CCM is not usable and that the approach to capture best the failure mechanisms involved in tunnels subjected to low stresses and tunnels intersected by narrow zones is likely a combination of the empirical and the numerical approaches.

6.2 Ground Behavior as a Function of Rock Mass Properties and in-situ Stresses

The influence of rock mass properties and in-situ stresses on ground behavior is presented in Fig. 12. Ground behavior is represented in the analysis by tunnel strain (ϵ), and indirectly by the exerted moment loading on support. Both ϵ and loading moments are numerically calculated from the calibrated models with rock support. As a qualitative index to represent geological strength, the GSI has an impact on ground deformation as observed in Fig. 12a. The general trend is a declining deformation with an increasing GSI. However, local variability of intact rock properties or ground conditions, rock support design, or a combination may have produced the observed scatter in the weak rock masses within the region of GSI 10–35 and tunnel strain in the range of 0.4–1.2%. The latter suggests that for any study considering ground behavior, additional rock properties and rock mass parameters need to be accounted for, among them rock mass strength σ_{cm} , in-situ rock stresses, and the rock support design.

Another important parameter in a study of ground behavior is the strength factor condition σ_{cm}/p_o as shown in Fig. 12b. The distribution of the results has served to evaluate the influence of the rock mass strength and in-situ

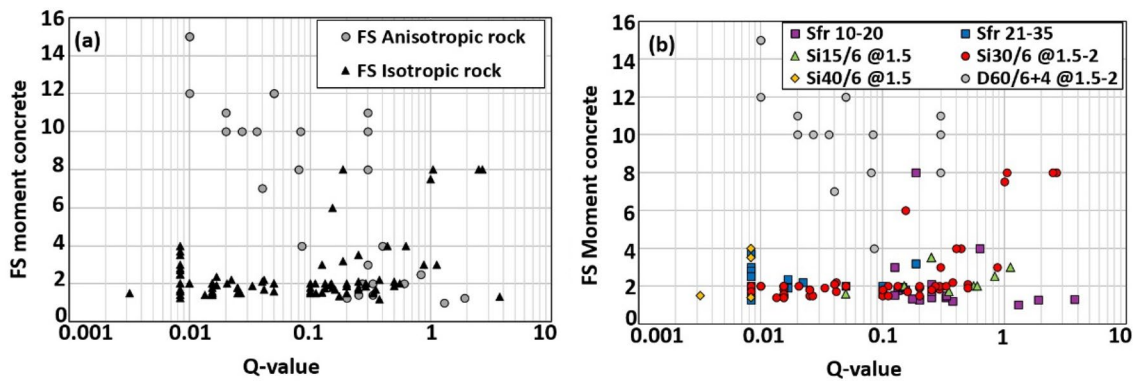


Fig. 13 **a** Rock support performance as a function of mapped Q, FS for moment loading of sprayed concrete, rock mass structure and **b** rock support design

stresses on ground behavior and as a function of the GBT. A well-defined trend can be observed for values of $\sigma_{cm}/p_o < 4$ where deformation increases with declining σ_{cm}/p_o . In turn, the trend from $\sigma_{cm}/p_o > 4$ indicates an increase of deformations despite of the rock mass being harder. This observation is likely related to the deformations that can take place in connection to block movements in hard and distressed rock masses, as denoted by the jump of tunnel strain registered at $\sigma_{cm}/p_o > 12$ for the category “low rock stresses”. The latter points towards the need for involving additional parameters that are relevant for ground behavior of hard and distressed rock masses, joint properties, and in-situ stresses.

The results in Fig. 12c represent tunnel strain as a function of the competence factor σ_θ/σ_c . The distribution and scatter of points over the plot reflect the performance of the SRF parameter to empirically describe rock mass competence in poor ground conditions. The ratio σ_θ/σ_c ranges between 0.01 and 0.35, which approximately corresponds to the SRF-subcategory G in the Q-system – “medium stress, favorable stress condition”, where SRF value of 1 is recommended. Apparently, there is an agreement between the empirical description of the stress condition “medium stress” with the reported “moderate” in-situ stress conditions in most of the studied case records. However, the scatter of the results (especially in weak rocks with tunnel strain $> 0.2\%$, and in anisotropic rocks with $\sigma_\theta/\sigma_c < 0.1$), suggests not only the involvement of additional parameters when designing rock support in these rock conditions, but also refinement in the definition of SRF.

From the analysis of the loading moments exerted in the support (Fig. 12d), it is observed that the magnitude of this is related to the in-situ rock stresses, stress anisotropy (measured as the horizontal to vertical stress ratio k_o), and rock mass strength. It can be observed that loading moments are generally lower when both conditions are met, the rock is weak and upon the presence of hydrostatic stress conditions ($k_o = 1$). Such tendency changes when the supportive

effect of rock arching is reduced due to relatively low lateral confinement ($k_o < 1$), as observed in the rapidly increasing moments in the region ($k_o < 1$) and $\sigma_{cm}/p_o < 1$. The magnitude of moments varies in up to 7 times in that region. Such variation is possibly related to the effect of varying

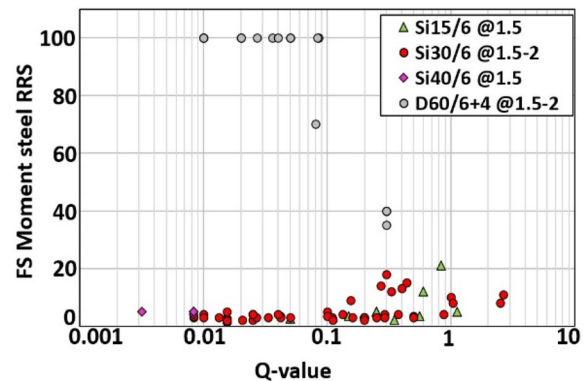


Fig. 14 Calculated loading of the embedded steel reinforcement in RRS-support

magnitudes of the k_o ratio, variations of the rock mass and/or rock support stiffness, or a combination. It is similarly observed how moment loading is reduced by increasing rock mass strength, as denoted by the declining moments when $\sigma_{cm}/p_o > 1$.

6.3 Performance of Empirical Rock Support Design

The performance of the support design has been studied with the evaluation of FS for the bending moments in the sprayed concrete and steel as shown in Figs. 13 and 14. Because of the clear influence of rock mass structure on ground behavior, the analysis has split isotropic rock masses from anisotropic rock masses as shown in the two different trends in Fig. 13a. While rock masses with isotropic

Table 7 Qualitative classification of the performance of design methodologies in poor ground conditions

Performance	Empirical method (Q-system)	Analytical method (CCM)	Numerical method
Appropriateness of the support design can be evaluated	B	A	A
Evaluation of FS	C	A	A
Consideration of other tunnel geometries than circular	A	B	A
Ability to identify and include weak rock	B	A	A
Ability to include in-situ rock stresses other than hydrostatic	B	C	A
Possibility to evaluate optimal timing for support	C	A	A
Ground-support interaction is defined/accounted	B	A	A
Possibility to consider anisotropic rock mass structure	B	C	A
Possibility to predict time-dependent deformations	C	B	B
Possibility to predict the effect of swelling ground	B	B	B
Rock mass description and classification	A	C	C

A Good performance/suitable
 B Limited performance/indirect use
 C Not usable/not suitable

structure present a decreasing FS with decreasing Q -value, layered rock masses (like the Skarvberg tunnel) present an increasing FS with declining Q -values. The trend for the rock masses with anisotropic structure is further accentuated by the systematic implementation of heavy RRS-support (starting from $Q < 0.4$) constructed at the tunnel face for back-supporting the bolt spiles and reinforce rock prone to delamination. The excess of tunnel support for the rock mass quality interval $Q 0.01-0.4$ is translated into high FS in the range of 6–15. Hence, these results are unable to explain the actual ground loads. An FS of 6–15 indicates the limitations of both the Q-system and the NPRA-guidelines to capture the ground behavior and recommend rock support in layered rock masses with $Q < 0.4$.

For isotropic rock masses in Fig. 13a, the trend of FS indicates, in general, reasonable values of FS (1.5–2) in the interval of $Q 0.01-0.1$. A significant increase of FS up to 8, starting from $Q > 0.1$, and a peak of up to FS 4 at $Q 0.008$ can be observed. In general, this suggests a potential for design optimization under these two conditions ($Q < 0.01$, or $Q > 0.1$). The scatter of FS in these Q -value ranges suggest that a more precise characterization of the ground conditions can be favorable.

A more detailed analysis has included rock support design in the study (Fig. 13b). It is observed that the high FS (6–15) for anisotropic rock observed in Fig. 13a corresponds to double RRS (D60/6 + 4). A downgrade of the double RRS-support should therefore be possible without affecting the intended performance of the original design. For isotropic rock masses with $Q > 0.1$, the FS between 4 and 8 shown in Fig. 13b affects mainly the arched RRS Si30/6 @ 1.5–2 m, a standard RRS-support design recommended by NPRA for tunnels with Q -values in the interval 0.1–0.2

and tunnel span of ca. 10 m. A downgrade may be possible in the form of either a leaner RRS type, an unarched RRS geometry, a wider center distance, or a combination of these. A hint of such optimization is actually given in the graph by the leaner rock supports falling within similar tunnel and ground conditions but yielding lower FS. For example, the liner of 10–20 cm thickness in $Q 0.1-0.3$ (FS 1.5–3), the RRS Si15/6 @ 1.5 m for $Q 0.1-1$ (FS 2–4), and the RRS Si30/6 @ 1.5–2 m in $Q 0.008$.

The calculated moment loading in the steel reinforcement of the RRS-support installed in some of the studied case records is presented in Fig. 14. The trend in isotropic rock masses suggests a combined or shared loading of both the sprayed concrete and the steel of the RRS when rock mass quality $Q < 0.1$. For $Q > 0.1$, there is also a shared loading with the sprayed concrete, but with a tendency of a declining steel loading (FS up to 20) if compared to the FS 3–4 when $Q < 0.1$. For anisotropic rock masses represented by the heavy RRS D60/6 + 4 @ 1.5–2 m, the reinforcement function of the steel is nearly neglected by the thick sprayed concrete taking up the loads, as denoted by the increasing FS with declining Q -value (and with shortening of RRS center distance). For $Q < 0.1$, FS is rather high, mostly over 100, and for the readability of the graph, FS is cut off at 100.

7 The Hybrid Design Methodology for Poor Ground Conditions

As shown in the results and evaluation presented in the previous sections, the design of rock support in poor ground conditions should involve a more comprehensive characterization of the ground conditions and the analysis of ground

behavior. This naturally leads to the need of integrating different design tools and approaches, which has resulted in the development of the *hybrid methodology*.

The hybrid methodology is a procedure that combines different design approaches in one procedure and utilizes the advantages of each of the involved design tools to cover limitations that other design tools (including the Q-system) may have in poor ground conditions. A hybrid design therefore benefits from the incorporation of different tools that aim at the identification of failure mechanisms, ground behavior and the interaction ground-support. Such hybrid procedure should therefore integrate the empirical method, the analytical and the numerical approaches along with the involvement of design tools aiming at the identification of ground behavior.

To develop a hybrid procedure, firstly the assessment of the performance for the design tools intended to be included within the hybrid procedure was necessary. A qualitative classification has then been done (Table 7), based on the findings of this study.

The hybrid procedure is, based on the rock mechanical conditions covered by the database, applicable to rock masses with quality $Q < 1$ and/or $GSI < 40$ and subjected to low-moderate in-situ rock stresses. Although testing and further development of the procedure will improve its validity from the addition of case records with actual hybrid support into the database, the presented results show clear relationships regarding the extent and type of improvements for tunnels of up to 12.5 m span. The procedure should therefore be used as a living guideline, and under the premises and limitations provided by the ground- and tunnel -conditions described for the database and the analyses.

The hybrid procedure is divided into six columns in which the different GBT are allocated (Table 8). A logical design sequence is followed from the identification of the GBT (as described in Sect. 2.3) and the evaluation of potential failure mechanisms in the rock support (as in Sect. 4) to a final recommendation of rock support design. Once the GBT is selected, the procedure is open to move through the different design tools within the column. However, lateral movement across the columns should be avoided unless the ground conditions clearly cover two or more GBT.

As observed in the hybrid procedure, Q-values are used as provisions in many of the design recommendations. This reflects the nature of the dataset, mainly based on the application of the Q-system. It also reflects the need for correlating the findings of the study to recommendations in a practical manner in the improved procedure. Hence, the Q-system can be utilized as a reference point or base within the procedure to describe rock masses and estimate basic categories of support. The assessments in

the proposed hybrid procedure will then enable a more correct design of support types.

8 Discussion and Recommendations

The proposed hybrid methodology is a complementary tool to other methodologies (included the empirical Q-system) in the detailed design of tunnel rock support in poor ground conditions. The use of this method requires knowledge of the existing and frequently used methods. Input data from other methods, i.e. the Q-system will be an advantage in many cases. This places the use of this procedure closer to detailed design stages in the project, than to the preliminary planning stages.

The assessed performance of the Q-system has been based on back-calculations of the ground behavior (tunnel deformation) and support loading (FS for support failure) using RS2 and assisted with the CCM. The Generalized Hoek–Brown failure criterion (Hoek et al. 2002) and the elasto-plastic material function were used for that purpose in both calculation modes. The approach similarly assumes that the response of a jointed rock mass can be addressed from the mechanical strength and stiffness of an equivalent continuum rock mass, i.e., utilizing the Hoek and Brown (1980a, b) failure criterion and the Hoek and Diederichs (2006) approach. It similarly assumes that the deformational ground behavior is primarily governed by the combined effect of induced plastic failure of rock and the rock mass stiffness. The potential impact from simulating poor rock masses with continuum models is therefore associated to the potential overrepresentation of plastic zones, which may in principle exaggerate the prediction of deformations at the more frictional rock masses (i.e., $Q > 0.4$, $GSI > 35$) within the studied range of poor rock masses. Hence, the likely outcome is conservative solutions under these conditions.

In the case of time-dependent deformations or creep due to possible rock weakening over time in weak/soft rocks, the simulation with RS2 would need explicit input of rock properties to account for the creep. Such simulation exercise would require either an extensive and long term testing of in-situ rock or the assumption of long-term behavior of rock material. In turn, most of the back-calculations of the ground behavior considering deformation monitoring over approximately one year are deemed to include (simulate) most of the time-dependent behavior. Hence, the calibrated models and the back-calculated ground/support behavior account for the gross effect of time-dependent deformations.

For rock masses with anisotropic structure where the behavior of blocks of hard rock is principally defined by the geometrical/mechanical properties of joints (like in

Table 8 Hybrid design procedure for rock support in tunnels excavated in poor rock ground conditions, rock mass quality $Q < 1$, and moderate to low in-situ rock stresses

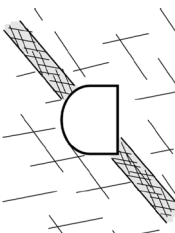
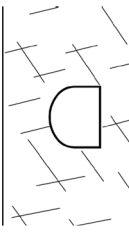
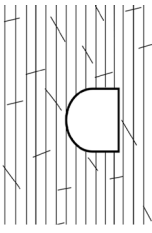
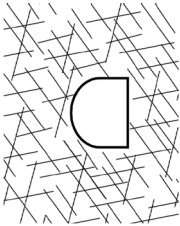
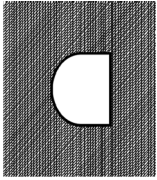
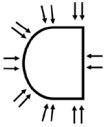
Ground behavior type	Weakness zone	Low rock stresses	Anisotropic rock masses
Illustration of GBT (Sect. 2.3)			
<p>Loading and failure mechanisms (Sect. 4)</p> <p>Site investigation</p>	 <p>a1, a2, b1, b2, c1, c2</p> <p>In weakness zones (thickness up to 1/2 × tunnel diameter) with $Q < 0.4$, GSI < 30 or if swelling is expected: rock properties like rock strength, Young's modulus, swelling and weathering of the infilling material are relevant. Assessment of joint properties is also recommended</p>	 <p>a1, a2, b1, b2, c1, c2</p> <p>If $Q < 1$ and/or unfavorable joint conditions: assessment of joint properties and rock strength/deformability is relevant. Core drilling and geophysics can give detailed information of the rock mass conditions</p>	 <p>b1, b2, c1, c2</p> <p>In-situ rock stresses, joint properties, rock strength and rock mass modulus should be assessed in stratified rock masses when $Q < 1$. Measurement of in-situ rock stresses when $Q < 0.4$</p>
Empirical method	<p>The Q-system is recommended. To describe the effect of orientation and the strength of the infilling, the respective RMR and GSI may be used</p>	<p>The Q-system and the GSI are recommended to determine the respective basic support needs and the rock mass strength</p>	<p>The Q-system have limitations to catch anisotropy when $Q < 1$, especially from $Q < 0.4$. Evaluate SRF upon a detailed study of the relationship σ_θ/σ_c. Q-system to obtain basic means of support. Compare to RMR</p>
Analytical method	<p>Analytical calculations may be used to determine the local capacity of sprayed concrete members at the zone. CCM has very little applicability</p>	<p>Study of block stability with limit equilibrium analyses is recommended regardless rock mass quality</p>	<p>Limit equilibrium analyses to derive the depth of the pressure arch and block stability are relevant when $Q < 1$</p>
Numerical method	<p>Numerical calculations are recommended to address the deformational behavior and for design verification. Selection of FEM or DEM modelling to be evaluated (see Sect. 3.3.3)</p>	<p>Design verification with numerical analyses is recommended in tunnels with low rock stresses provided complex ground conditions and/or $Q < 1$</p>	<p>If $Q < 0.4$: numerical analyses recommended to determine failure mechanisms, rock mass behavior and depth of the pressure arch in relation to the studied support designs</p>
Monitoring	<p>Convergence monitoring is recommended in the presence of unfavorable combinations of width/orientation of the zone and/or evidence of swelling</p>	<p>Monitoring of deformation is recommended in the tunnel and in the nearby infrastructure to control stability and design verification</p>	<p>Deformation monitoring is recommended for the back-calculations of rock mass properties (recommended in tunnel sections with $Q < 0.4$)</p>
Support design	<p>If $Q < 0.4$ and/or GSI < 30, design is recommended with the empirical approach, verified with analytical calculation, and optimized with numerical analysis. Unarched support to be evaluated when zone $Q > 0.1$. Leaner RRS, i.e., reduction of steel reinforcement in RRS, can be evaluated when $Q > 0.1$</p>	<p>Reinforcement of the tunnel ahead of the face is recommended. RRS-support and shortening of round length are seen relevant construction approaches. If $Q < 0.4$, the empirical design to be verified/supplemented with kinematic analysis of blocks, and verified with numerical analysis</p>	<p>If $Q < 0.4$ and/or GSI < 30, the empirical design should be verified with analytical evaluation, optimized with numerical analyses simulating rock mass structure and in-situ stresses. Spiling and round length shortening are relevant when $Q < 0.4$; Leaner RRS-support and/or hybrid-tailored support to be evaluated when $Q > 0.1$</p>

Table 8 (continued)

Ground behavior type	Hard and fractured rock masses	Weak rock mass	Swelling ground
Illustration of GBT (Sect. 2.3)			
Loading and failure mechanisms (Sect. 4)	b1, b2, c1, c2	d1, d2, d3	d1, d2, d3
Site investigation	If $Q < 0.4$ or $GSI < 40$, assessment of rock strength/deformability and joint properties is recommended. Measurement of in-situ stresses if $\sigma_{em}/P_o < 1$	If $Q < 0.1$ or $GSI < 35$: assessment of rock strength and deformability. Measurement of in-situ stresses if practically possible. Testing of rock properties to determine rock mass strength and deformability	Zoning of swelling (Høien et al. 2019a, b; Terron-Almenara 2020; Nilsen 2021) together with rock mass properties to estimate rock mass strength and deformability are recommended
Empirical method	The Q-system is deemed to work well when $Q > 0.4$. Use also GSI when $Q < 0.4$	Q-system can give reliable estimates of basic support for $Q > 0.1$. Evaluate SRF upon a detailed study of the relationship σ_p/σ_c . It is also recommended to register GSI to estimate rock mass strength and deformability	The Q-system should only be used to derive basic needs of support
Analytical method	CCM may be applied when $Q < 0.4$. Limit equilibrium analyses to check rock stability and/or to derive capacity of support members against failure when $Q < 1$	CCM may be used when $Q < 0.1$ for estimates of loading, deformation, and timing of support	Structural analysis of sprayed concrete “plates” should be used to study the load capacity against failure
Numerical method	Numerical analyses of wedge stability are recommended. When $Q < 0.4$, numerical analyses are relevant to verify rock support design. FEM or DEM to be evaluated	For rock masses with $Q < 0.1$, numerical analyses are recommended to study the support performance upon the ground loads. FEM may be used. Stress anisotropy becomes especially important in the calculations when $\sigma_{em}/P_o < 1$	Numerical analyses are recommended. Structural analyses with numerical programs are also relevant to verify failure states of liners and invert
Monitoring	Deformation monitoring when $Q < 0.4$ for design optimization	Deformation measurements are relevant for 1) design verification and optimization and 2) calibrating rock properties. Other means of monitoring may be added	Deformation monitoring is relevant, also in the invert. For the zonation of swelling, systematic convergence sections are recommended
Support design	Support design should be done upon a rational combination of the empirical and analytical approaches. Numerical verification when $Q < 0.4$. Unarched and/or leaner RRS-support to be evaluated when $Q > 0.1$	The empirical approach should be accompanied by other tools when $Q < 0.1$ and $GSI < 35$. Optimization based on the combined application of Q-system, CCM, and numerical analyses for design verification of load-bearing support and optimal support timing. Unarched support is not recommended, but leaner RRS-solutions may be evaluated	The design of the RRS-support and structural invert should be based on the actual ground and swelling pressure in the tunnel. Unarched support to be evaluated depending on the extent and magnitude of the in-situ swelling pressure

Tunnels with span < 12.5 m

Skarvberg tunnel) and the direction of in-situ rock stresses, the addition of an explicit joint network was done in RS2. This included the mapped geometry and the properties of discontinuities (Fig. 10) and the evaluation of unconfined blocks/rock portions, yielded joints and displacement vectors to investigate ground behavior in relation to the given in-situ stresses. Although the discontinuum approach with DEM may have given a detailed analysis (and overview) in terms of shear displacement through discrete joints, it is observed that, in general, the continuum-based RS2 model with explicit geological structure has given reasonable results that correspond with the measured behavior. The extent of a possible under-representation of the ground deformation as a consequence of the limitations of FEM to capture dilatant rock behavior is, however, not derived in FEM. The possible outcome of the latter limitation is the potential overestimation of deformations in harder and jointed rock where failure and movements mainly occur through preferential discontinuities.

It should be also noted that the studied ground behavior in the calculations and in the monitored tunnels belongs to *supported* tunnels. Still, the results have provided reasonable descriptions of the ground behavior that allow for interpretation and comparison. In this context, the effects that the distribution of the selected case records may have on the general results should be similarly noted. For example, case records representing anisotropic rock mass structure in this study belong only to the Skarvberg tunnel where heavy RRS-support was mostly caused by the prominent rock mass delamination issues along the bedding during excavation. In view of this, the design optimization inferred from the high FS (Figs. 13 and 14) should not be extrapolated directly to other cases with layered rock, but mainly as an indication of the expected ground behavior and the needed design tools to address appropriate support design. Other assumptions have been done in the study. For example, the reinforcing effect of bolt spiling or grouting of the rock mass were not modelled on purpose. That was for two main reasons. First, because most of the spiling and grouting design data was unavailable. Second, to ensure more comparability through the case records by avoiding the introduction of further variables (though not belonging to true-rock support).

As observed in the database used for this article (Table 6), most of the registrations regarding rock mass quality were done with the Q-system. Rock mass classification is based on the visual inspection, mapping, and rating of individual rock mass parameters. Such work is normally performed by geologists and tunnel engineers of different backgrounds, experience, and eyes. Inherently, classification of rock masses can therefore carry some level of subjectivity and uncertainty as to how representative and accurate certain Q-values might be for the given ground conditions. This aspect of cognitive biases in rock engineering and its impact

in rock mass classification have been well addressed by Elmo and Stead (2021). One of the conclusions they obtained is that besides the attempts of minimizing uncertainty and subjectivity by increasing knowledge (of the ground conditions), a certain level of uncertainty and subjectivity would still remain due to, among others, the own limitations of the (empirical) method. For this reason, involvement of several design tools and approaches have been utilized and combined in this study.

The involvement of different tools and approaches in this study has permitted the evaluation of possible deviations, as demonstrated in the comparison and testing of different tools shown in Fig. 11, and through the qualitative classification of the performance of individual tools presented in Table 7. In addition, interpretation of the results (in Sect. 6) has been mostly based on the evaluation of trends, distributions and value-intervals of tunnel strain and FS, rather than the mere analysis of single values. This has likely filtered part of the potential deviations produced by the mentioned simplifications. As any a posteriori study, the addition of further case records and the testing of the introduced procedure will naturally turn into improvement. For that reason, the following recommendations are given:

- (1) Further case records representing the different GBT should be studied and included as a basis for potential updates of the hybrid procedure. This may also include sensitivity analyses (i.e., tunnel span, in-situ stresses, σ_{cm} , E_m), and studies with numerical analyses that consider discontinuum modelling.
- (2) A comprehensive testing program (on site) for the hybrid procedure should be done as a basis for further improvement. For example, with instrumented tunnel sections covering different GBT to quantify the appropriateness of both the methodology and the hybrid support design.
- (3) Study of the stabilization effect that spiling and grouting has in poor ground conditions. The outcome used for optimization of the hybrid procedure.
- (4) Involvement and testing of either modified or hybrid support solutions in poor ground conditions requiring load-bearing support.

9 Conclusions

Based on the field registrations and deformation monitoring data from tunnel projects, the ground behavior and loading conditions in more than one hundred case records were studied through analytical and numerical back-calculations.

The study has revealed several limitations in the Q-system that are relevant in any study aiming at optimal rock support design in poor ground conditions. In particular, challenges

to identify and capture the effect of rock mass structure, failure mechanisms in the rock mass, ground behavior, rock mass properties and stress anisotropy are evident. The limitations are reflected in the performance of the Q-system, as shown on the case records with underloaded RRS-supports. The need for design optimization has been therefore found possible in the ground conditions showing underloaded (or conservative) support. That is, for isotropic rock masses of $Q > 0.1$ and in layered rock masses of $Q < 0.4$. A hybrid design approach combining several design tools and methods will therefore enable a better design optimization for these ground conditions. Based on the limitations of the Q-system and the wide spectrum of ground types affected, the use of the hybrid methodology will also enable improved design in other types of poor ground.

For the development of an integrated or hybrid design methodology for poor ground conditions in hard rock tunneling, the main elements and principles of hard rock engineering must be integrated with the principles of weak ground engineering. In this context, the development of the hybrid methodology has involved the ground-support interaction approach, the CCM, a classification of ground behavior types, deformation monitoring, numerical analysis, and tools able to identify failure mechanisms in rock masses and in rock support. The hybrid methodology therefore avoids deriving of rock support classes such as the support recommendations of the Q-system.

As in any a posteriori study using past design experience from selected project sites to build a procedure, the representativity and distribution of the case records can have an effect on the obtained results and interpretations. The hybrid approach needs to be used with this in mind and only in underground conditions comparable to those studied in this article. Hence, further additions of detailed case records and site-testing of this procedure will contribute to updates and expand the areas of use of the hybrid procedure.

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Declarations

Conflict of Interest The authors confirm that there are no known conflicts of interest associated with the publication of this article, and that are no financial or non-financial interests that has affected the outcome of this work.

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