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18.05.2026

Statens vegvesen

Rock support in Norwegian road tunnelling

Current practice and future regulations

NORWEGIAN PUBLIC ROADS ADMINISTRATION REPORTS No. 1132

Are Håvard Høien/NPRA

Statens vegvesens rapporter

NPRA reports
Norwegian Public Roads Administration

Tittel
Bergsikring i norske vegtunneler

Undertittel
Nåværende praksis og fremtidens regelverk

Forfatter
Are Håvard Høien

Avdeling
Konstruksjoner

Seksjon
Konstruksjonsteknikk

Prosjektnummer
L10439

Rapportnummer
1132

Prosjektleder
Are Håvard Høien

Godkjent av

Emneord
Tunneldriving, bergsikring, tunneldriving i hard berg, berggrunnsoppførselstyper

Sammendrag
Sammendrag finnes på side i. Rapporten er publisert på både norsk og engelsk.

Rapporten utgjør det faglige grunnlaget for revisjon av regelverket for bergsikring og tunneldriving for norske vegtunneler, vegnormal N500 Vegtunneler og veiledning N-V521 Geologi og bergsikring i tunnel, i forbindelse med tilpasningen til andre generasjon eurokoder.

Title
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Subtitle
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Project number
L10439

Report number
1132

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Approved by

Key words
Tunnel excavation, rock support, hard rock tunnelling, ground behaviour types

Summary
Abstract can be found on page i. The report is published in both Norweigan and English.

This report constitutes the professional basis for revising the regulations for rock support and tunnel excavation for Norwegian road tunnels, spesification N500 Road tunnels and guideline N-V521 Geology and rock support in tunnels, as a part of the adaptation of the second generation of the Eurocodes.

Antall sider 64

Pages 64

Summary

This report contains a review of current practice for rock support in Norwegian road tunnels and outlines the development of a future regulatory framework for tunnelling in hard rock based on the Observational Method in Eurocode 7.

The report is divided into two main parts. Part 1 addresses historical and engineering geological prerequisites as well as challenges associated with current practice. It presents Norwegian ground conditions, outlines what is meant by “tunnelling in hard rock”, and identifies deficiencies and problems in existing practice. Part 2 provides a theoretical background for different types of failure and deformations that occur in rock, categorisation of ground behaviour, an assessment of methods and tools for rock engineering design, and finally discusses how the Observational Method can be applied to the design and construction of tunnels in hard rock.

The current regulations for rock support in Norwegian road tunnels are largely based on empirical methods developed 30–50 years ago. Although this has resulted in a standardised practice that functions under most conditions, research and experience show that the system has weaknesses, particularly in complex rock conditions and non-standard tunnel geometries. The theoretical basis for the regulations has limited scientific basis, and the current regulations do not fulfil the requirements for verification of stability given in Eurocode 7.

The report presents a new rock support strategy based on the Observational Method, founded on the principle that rock support shall be designed according to the behaviour of the rock mass, by accounting for the expected failure and deformation processes of the rock in the design of stability measures. The new strategy involves categorising the ground along the tunnel according to different ground behaviour types (GBT), which will be governing for the selection of support design. The Norwegian practice of mapping and determining support at the tunnel face is continued; however, the design work, particularly in the detailed design phase, will become more extensive.

GBT is the behaviour of the ground as a result of tunnelling without rock support. The systematisation of stability problems in the report is based on these behaviours and is further linked to loading conditions, rock mass type, and the failure and deformation processes expected to occur. Based on this, overall/principled proposals are given for how rock support can be designed to counteract and take care of the actual failure and deformation processes in the ground.

Central to both design and construction in the new rock support strategy is also the system behaviour (SB), which is the result of the interaction between the rock mass and the rock support. In the Observational Method, as described in Eurocode 7, stability shall be verified through observations where deviations trigger predefined measures, and it is deviations from the expected SB that will trigger these measures.

Furthermore, an important point highlighted in the report is that for many types of stability problems, reliable methods for the design of rock support and assessment of stability in the supported profile are lacking. Awareness of this uncertainty in the methods are crucial, as it allows for compensating measures through increased design effort where needed. This stands in contrast to a practice where empirical methods are applied in ground conditions for which the methods do not have adequate basis.

The plan further is to update the regulations for Norwegian road tunnels harmonised with the second generation of Eurocodes, which are expected to be issued in 2028, based on the content of this report. The regulations for excavation and rock support of road tunnels consist of mandatory requirements given in specification N500 Road Tunnels. The descriptive guideline N-V521 Geology and Rock Support in Tunnels elaborates methods and practice. In a revision, aspects related primarily to design using the Observational Method will be implemented in N500, while guidance on geological, rock mechanical and rock engineering matters will be implemented in N-V521.

At the time of publication of this report, a revision of N500 is ongoing that will not be influenced by the content of this report.

Acknowledgements

Thanks to Jorge Terron-Almenara for review of GBTs for tunnelling in hard rock and for the valuable ideas presented in his PhD thesis. Thanks also to the supervisors of the PhD projects that have led to this report: Bjørn Nilsen, Karl Gunnar Holter, Roger Olsson and Charlie Chunlin Li. Additional thanks to Anette Wold Magnussen, Arild Neby, Arild Palmström, Henki Ødegaard, Mari Lie Arntsen, Mona Lindstrøm and Vilde Møllerhaug for review of the report and valuable comments.

Content

1 General introduction

The purpose of this report is to lay the foundation for a new strategy for rock support of road tunnels in Norway. As indicated by the title, the report will describe current practice, how it has evolved, the engineering geological and rock engineering prerequisites for tunnelling in hard rock, and outline how a new regulatory framework may be formulated.

The following chapters will show that the current regulations – including engineering geological mapping at the tunnel face, registration of Q-values, and use of the rock support table in N500 Road Tunnels – cannot be regarded as “prescriptive measures” in accordance with Eurocode 7 for all ground conditions. The current regulations are therefore not in conformity with the Eurocodes, and a revision is necessary. Furthermore, the current regulations are detailed only for easy ground conditions and tunnel geometries, and lack adequate descriptions of practice in more complex cases.

The new concept is based on the Observational Method described in Eurocode 7 and will essentially involve designing the rock support in response to the actual behaviour of the ground. The Norwegian practice of mapping and determining support at the tunnel face is continued; however, this will be accompanied by more extensive design work, particularly in the detailed design phase.

A main weakness with the current regulations is that their scientific basis is narrow and built on a limited number of older publications, based on practice and design from 30–50 years ago. Since then, the rock mechanical and rock engineering basis has developed, while also tunnel technology and society’s safety requirements have changed significantly. From a rock engineering perspective, there is a major difference between a single tube 40-year-old avalanche protection tunnel in Western Norway and today’s urban twin-tube tunnels. Not to mention todays advanced tunnel systems with roundabouts and caverns for fans several hundreds of metres below sea level. The new regulations will have a broad scientific foundation, including modern research and practice in rock mechanics and rock engineering, and will at the same time be adapted to the European structural standards.

The report is divided into two main parts:

- **Part 1** addresses historical and engineering geological prerequisites as well as challenges related to current practice. Norwegian ground conditions and what is considered as “hard rock tunnelling” are presented, and deficiencies and problems with existing methods are identified.
- **Part 2** provides a theoretical background for different types of failure and deformation, categorisation of ground behaviour, and how the Observational Method can be applied in design and excavation. An important new contribution is the establishment of the relationship between ground behaviour, failure and deformation types, and principles for rock support.

This report forms the technical basis for further revision of N500 Road Tunnels and N-V521 Geology and Rock Support in Tunnels, with the aim of harmonisation with the forthcoming generation of Eurocodes expected to be published in 2028. At the time of publication of this report, a revision of N500 is ongoing that will not be influenced by the content of this report.

In this report, *engineering geology* refers to the branch of geology concerned with mapping geology and characterising rock and rock mass for engineering purposes. *Rock mechanics* refers to the theoretical and applied science of the mechanical behaviour of rocks and rock masses. *Rock engineering* refers to the design and dimensioning of stability measures for structures in/on rock mass.

Part 1 – Historical and engineering geological prerequisites and limitations of current rock engineering practices

2 Introduction

This part establishes the context for the new rock support concept presented in Part 2. It begins with an overview of Norwegian road tunnels, including the number of tunnels, total lengths and years of opening. This is followed by a description of the road tunnel as a construction, before Norwegian ground conditions are reviewed in order to define which geological conditions are covered and to set the framework for tunnelling in hard rock. Also, facts regarding rock mass quality in Norwegian road tunnels are presented.

This is followed by a review of rock support in Norwegian road tunnels, including a historical overview and a description of the current system for rock support. Further details are then given on practice related to excavation and support in weak rock. Subsequently, a brief introduction is provided to the European regulatory framework applicable to rock engineering design, Eurocode 7, as well as an introduction to fundamental principles of rock support.

Finally, systematic limitations, weaknesses in the theoretical basis, and specific problems related to current rock engineering design practice are presented. These issues form the argumentative basis for the need for a new rock support strategy.

2.1 Road tunnels in Norway

In Norway, there are more than 1,570 road tunnels with a total length of approximately 1,500 km, of which about 30 are subsea tunnels. Over the past 15 years, approximately 200 tunnels with a combined length of 450 km have been opened, of which about 50 have twin tubes (see Fig. 1). In most cases, road tunnels in Norway are excavated using conventional drill-and-blast methods; however, there are a few exceptions historically where tunnelling has been carried out using TBM.

Various contract forms are used for the construction of road tunnels in Norway, and the choice depends on the complexity of the project, the risk profile, and how the client wishes to organise the works. The most commonly used form has been design–bid–build contracts, where the client is responsible for the design. In such contracts, the contractor builds according to completed drawings and descriptions, and pricing is based on unit rates for all delivered items. This gives the client a high degree of control, but also greater responsibility for errors in the design.

Another common form is turnkey contracts, where the contractor is responsible for both design and construction. In these cases, the client primarily specifies functional requirements and refers to overarching regulations. The contractor is responsible for the design and has the freedom to develop own solutions that fulfil the client's requirements. This results in less detailed control by the client and greater risk for the contractor.

In some complex tunnel projects, collaborative contracts have been used. This involves close cooperation between the client and the contractor in an early phase to jointly develop the project. The objective is to identify the best solutions together and to allocate risk in an appropriate manner. After the collaboration phase, a design–build contract is normally used for the execution of the project. In some cases, PPP contracts (Public–Private Partnership) are also used, where execution typically is carried out as a design–build contract and the contractor is responsible for maintenance for a certain period after completion.

The pricing of rock support is managed in several different ways. The most common approach is unit pricing of the various support elements, also in design–build contracts, but pricing per support class and other solutions are also used.

For road tunnels in Norway, the regulations specify requirements for the content of engineering geological reports at different stages of planning: early planning phase, municipal sub-plan, zoning plan, and detailed design. However, not all of these reports are prepared for all tunnel projects, as this depends partly on planning pace and contract form. An important fixed point is the zoning plan report, which is always produced and shall, as a general rule, contain all engineering geological information required to construct the tunnel. For some contracts, for example unit price contracts, a separate detailed design report is prepared and attached to the contract documents when the project is tendered. For other contract types, such as design–build contracts, the contractor prepares the documentation for the detailed design.

The clients for road tunnels in Norway are the Norwegian Public Roads Administration and Nye Veier AS (a state-owned development company) for national roads, the county authorities for county roads, and the municipalities for municipal roads.

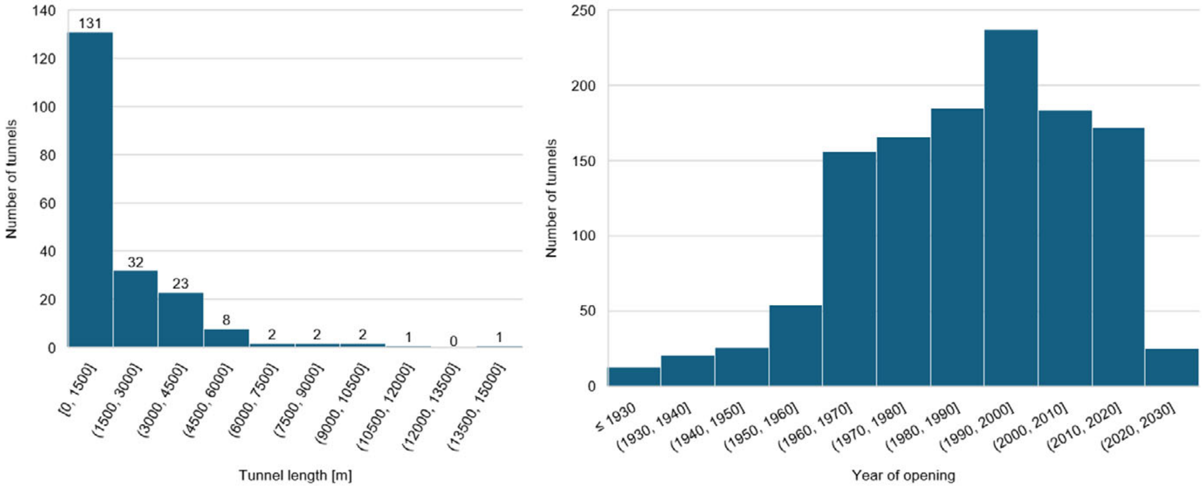

Fig. 1 Left: Length of tunnels opened the last 15 years. Right: Number of opened road tunnels in Norway sorted on decade.

2.2 The road tunnel as a construction

Norwegian road tunnels are designed such that all installed rock support forms a part of the permanent support system, which results in a relatively low consumption of support measures. Due to the need to protect the rock and the rock support against frost, to protect the road from water, to facilitate operation and maintenance, and to improve traffic safety, a tunnel lining is installed. Different solutions are applied based on traffic volume, tunnel length, frost conditions, and project-specific requirements. In general, three main solutions are used (Fig. 2): a) frost-insulated mesh-reinforced shotcrete shell with a concrete kerb at the bottom, b) prefabricated concrete elements in the sidewalls and a frost-insulated mesh-reinforced shotcrete shell in the crown, and c) prefabricated concrete elements from kerb to kerb.

For all three solutions, there is an air gap between the rock support and the tunnel lining. For the first solution, this space is enclosed and, in most cases, inaccessible to personnel. In the case of the other two solutions, the gap between the rock support and the lining usually is sufficiently wide to permit inspection. However, ensuring satisfactory health, safety and working environment (HSE) conditions during such inspections is challenging, and for new tunnels it is intended that inspections are only carried out in exceptional cases. Tunnels are periodically inspected every five years where access is possible. The linings are designed to withstand the fall of smaller rock blocks.

2.3 Norwegian ground conditions

Norwegian ground conditions represent what is internationally referred to as *hard rock tunnelling*. There is no unambiguous and clear definition of what hard rock tunnelling entails; however, it is generally understood to refer to rock mass of sufficient strength such that excavation requires blasting or the use of tunnel boring machines designed for hard rock. These conditions are characterised by high strength of the intact rock, which is consistent with the general rock mass conditions in Norway, where hard rock types dominate. Furthermore, a key characteristic of Norwegian geology is that the rock mass is intersected by weaker zones consisting of altered and/or more heavily fractured rock.

Although strong and very strong rocks dominate, there are also areas with weaker rock types whose intact properties would be classified as moderately strong, and not uncommonly weak according to the

ISRM classification (ISRM 1978). This applies, for example, to certain calcareous schists, marble, phyllite, greenstone and soapstone found in various parts of the country.

The following subchapters describe Norwegian geology, including mechanical properties of intact rock, as well as the distribution of rock mass quality mapped in Norwegian tunnels.

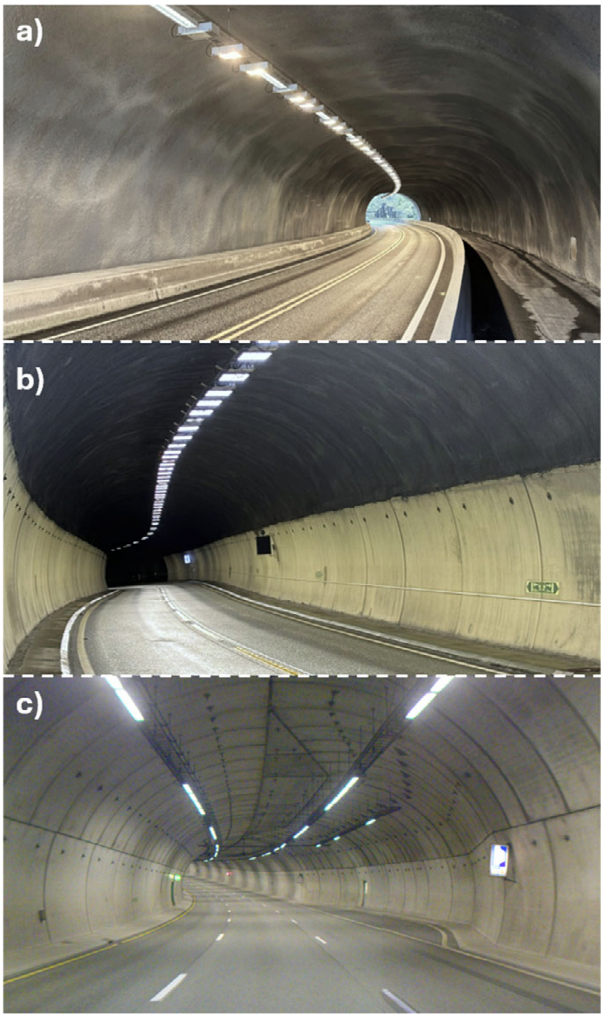

Fig. 2 Different types of tunnel linings. a) PE foam covered with shotcrete and a concrete kerb (walkway on the right). b) Concrete elements in the walls; PE foam covered with shotcrete in the crown. c) Concrete elements with membrane for waterproofing and, where applicable, XPS for frost insulation

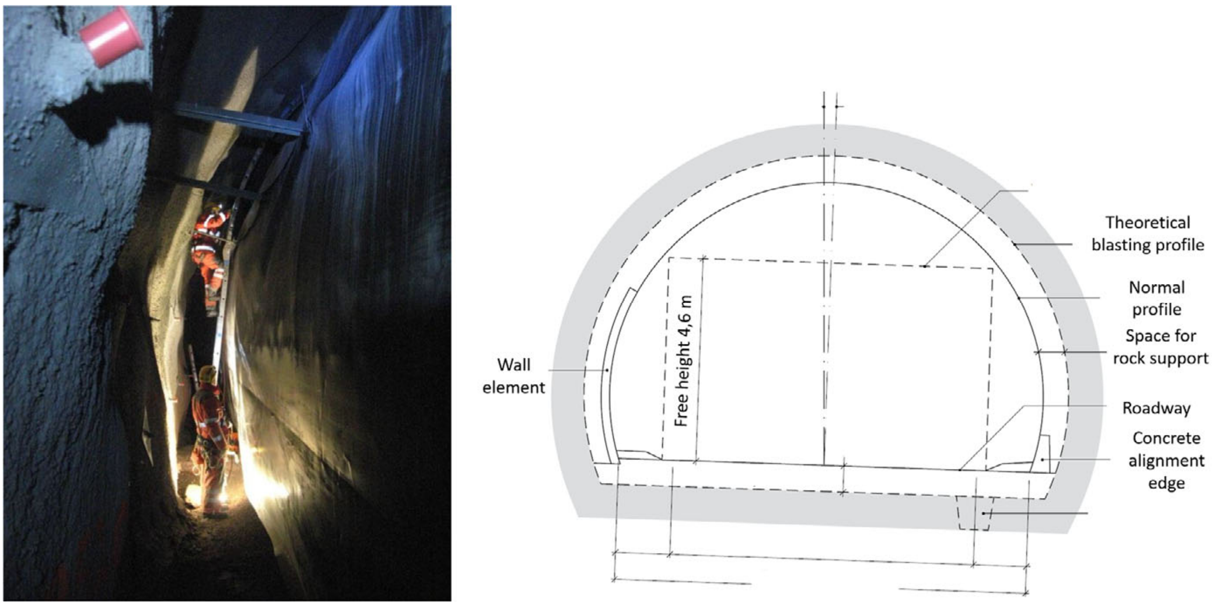

Fig. 3 Left: Inspection of the space between rock support and tunnel lining (Lindstrøm, Magnussen and Langelid 2013). Right: Standard tunnel profile for two traffic lanes

2.3.1 The geology of Norway

The ground in Norway consists mainly of Precambrian and Cambro–Silurian (Caledonian) rock types, see Fig. 4. The Precambrian rocks mainly comprise gneiss and various intrusive rocks, such as granite and gabbro, while the Caledonian rocks are primarily metamorphosed sedimentary and volcanic rocks. These rocks were thrusted over the country during the Caledonian orogeny. In Southeastern Norway, volcanic and magmatic rocks from the Carboniferous to Permian periods associated with the Oslo Rift are also present. In Western Norway, and to some extent in Trøndelag, rocks of Devonian origin are also found. During the Quaternary, glaciers shaped the landscape through erosion. This glacial erosion removed loose and weathered rock, and as a result, the present-day ground mainly consists of unweathered hard rock intersected by weakness zones of varying extent and character. These weakness zones were formed by tectonic activity and, in some cases, Mesozoic deep weathering.

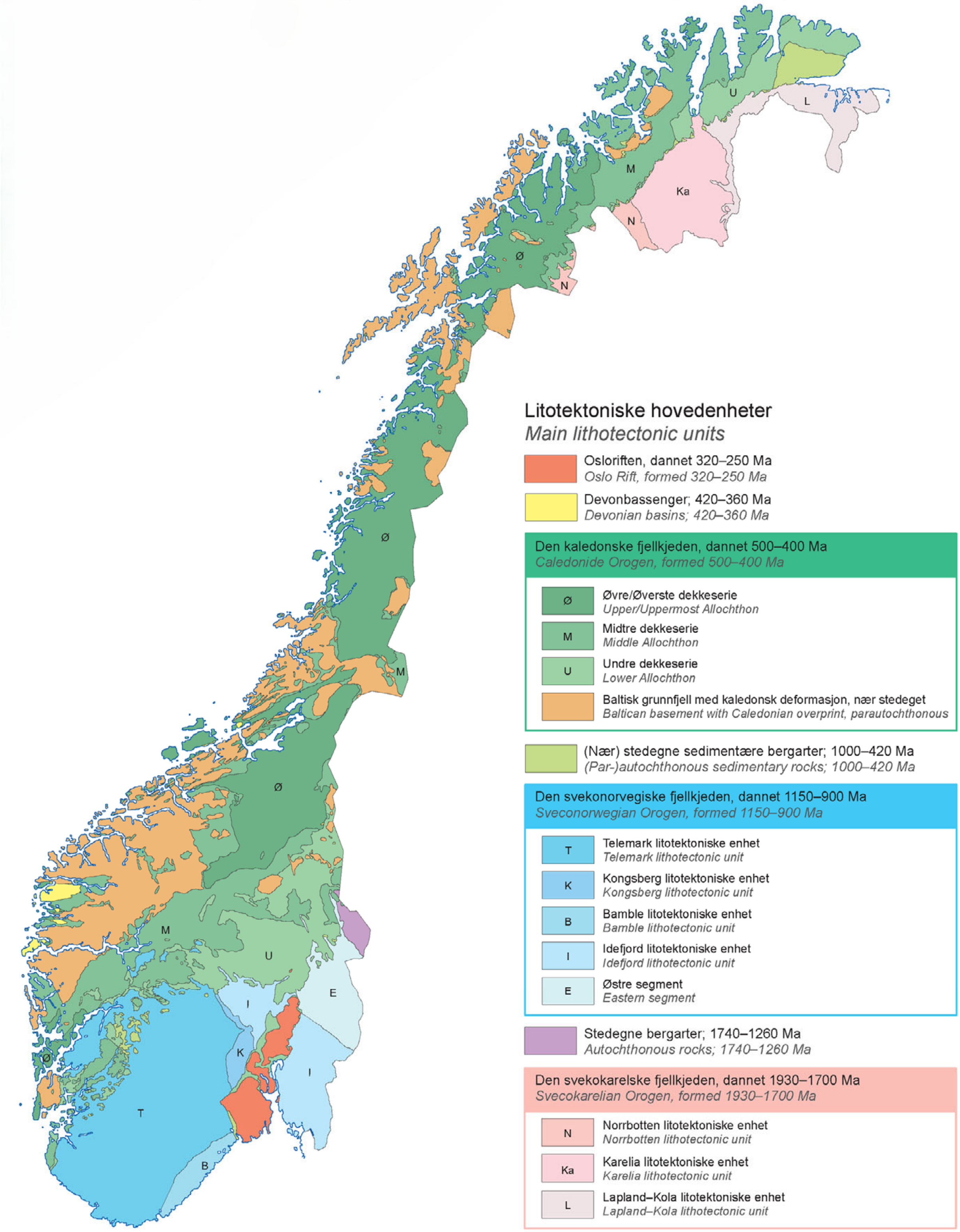

Fig. 4 Main lithotectonic units. Cutout from NGU (2021)

2.3.2 Characteristics of weakness zones

The character of weakness zones may vary within wide ranges. It is not the intention here to provide a complete description of a weakness zone, but rather to give a few examples to illustrate some of the possible variations that may be encountered during tunnelling in hard rock conditions. For a more comprehensive description, reference is made to e.g. Hoek, Kaiser and Bawden (1997) or Nilsen and Palmström (2000), and more specific for the Norwegian region (Braathen and Gabrielsen 2000)

Fig. 5 shows a network of weakness zones as seen from the air. This illustration is from a mountainous area in southeastern Norway, where no Quaternary deposits or vegetation hide the bedrock structures as they usually do in the lower parts of the country.

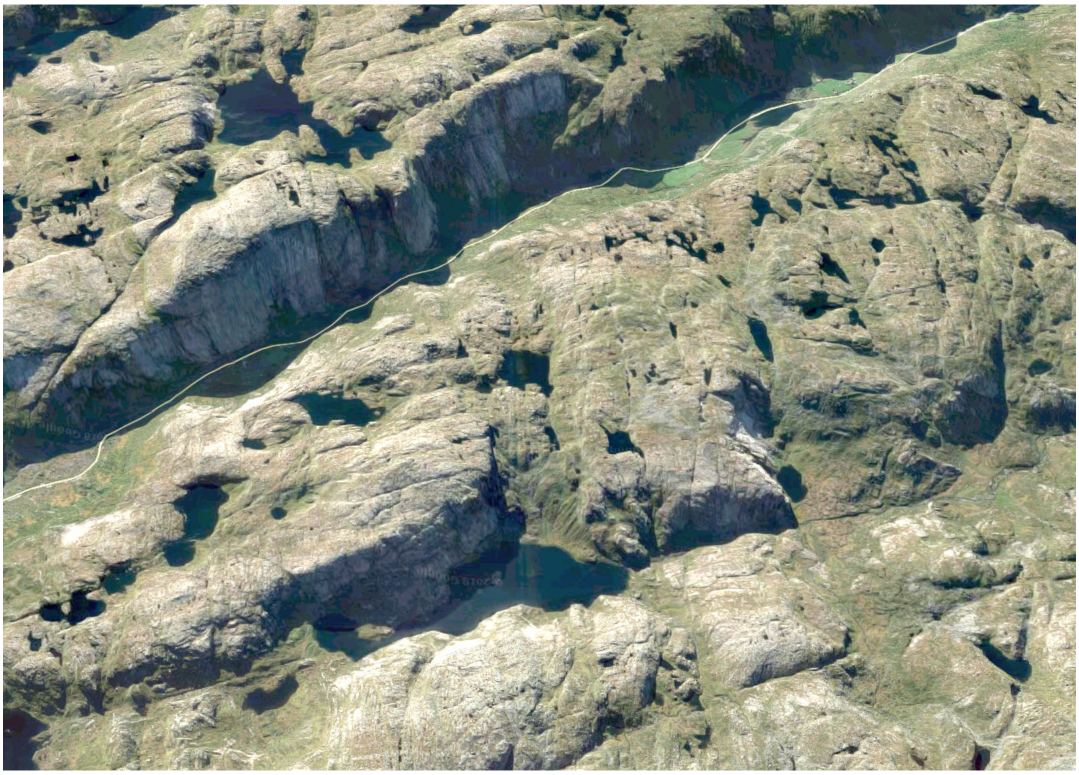

Fig. 5 Network of weakness zones in a mountainous area in southwest Norway. One can see the potential weakness zones as lines in the landscape due to glacial erosion. The depth of depressions is mainly a result of the rock quality, zone width and glacial motion direction. The road crossing the image is 10 km from edge to edge (Google Maps 2018).

In some cases, i.e. as shown in Fig. 6 and Fig. 7, a weakness zone is characterized by intense fracturing/crushing, with limited extent of gouge material or joint filling. In other cases, the weakness zone contains more clay gouge. Fig. 8 and Fig. 9, represent examples from the E39 highway tunnel Svegatjørn–Rådal in Bergen. Fig. 8 shows weak rock mass in the entire face, but highest content of clay at the right side. A hydraulic hammer could with little effort excavate this rock mass. In Fig. 9, a more limited zone with a high content of clay is shown, which was so loose that one could easily dig it out by using the back end of a geological hammer.

In Fig. 10 a zone close to the Martineås tunnel at highway E18 near Larvik is shown. The structure running diagonally in the middle of the photo is a very clay-rich zone with material that can be easily scraped loose by using the back end of a geological hammer. Some places, there are harder rock fragments separated by filled joints. The rock fragments can also with little effort be loosened using the back end of the geological hammer and broken into several pieces with a firm blow.

The zone shown in Fig. 11 is situated close to the Kleivene at highway E18 in Drammen. The left side of the zone is very clay rich and the pointed end of a geological hammer may easily be pushed into the material. On the right hand side, small pieces may be broken loose by using the geological hammer and further broken by hand into small pieces.

The examples shown in this section represent quite narrow weakness zones. Zones forming large valleys and fjords may be of much larger sizes with widths of tens of meter or more, causing fjord-crossing subsea tunnels to be especially vulnerable to stability problems.


Fig. 6 Weakness zone in rhomb porphyry at the E16 Sandvika-Skaret project.


Fig. 7 Detail of the weakness zone shown in Fig. 6.

Fig. 8 Chainage 13251 in tube T52 at the E39 Svegatjørn–Rådal project

Fig. 9 Chainage 13616 in tube T52 at the E39 Svegatjørn–Rådal project.

Fig. 10 Road cut just outside the Martineås tunnel at the E18 Bommestad–Sky project close to Larvik

Fig. 11 Road cut just outside Drammen. This weakness zone crosses the Kleivene tunnel on highway E18

2.3.3 Rock stress

As for many other regions in the world, the *in-situ* rock stresses in Norway vary considerably in magnitude as well as orientation. Myrvang (2001) stated that when measured, the horizontal stresses are usually much higher than the vertical component would indicate. He also stated that the vertical stress in most cases corresponds well to the overburden and is normally the minor principal stress, at least down to 500 m. An overview map of Norwegian geology and horizontal stresses is shown in Fig. 12. According to Myrvang (1993), the overall tendency is that the principal stresses seem to be parallel/normal the Caledonian mountain range.

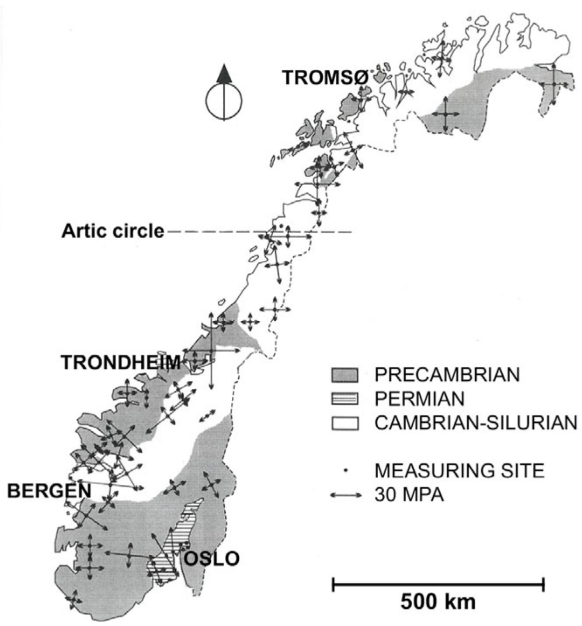

Fig. 12 Directions of horizontal stress in Norway (Myrvang 1993).

2.3.4 Properties of intact Norwegian rock

Box plots of uniaxial compressive strength (UCS) and E-modulus based on the SINTEF rock mechanical properties database (SINTEF 2016) are shown in Fig. 13 and Fig. 14, respectively. The circles and stars are outliers and were not included in the distribution calculation. The dataset is based on testing about 3300 samples and one can see there is considerable variation in both the UCS and the E-modulus. According to ISRM (2007) classification the UCS values are mainly described as strong or very strong, however, there are also test results in the medium strong rock (25- 50 MPa) and weak rock (5 – 25 MPa) categories.

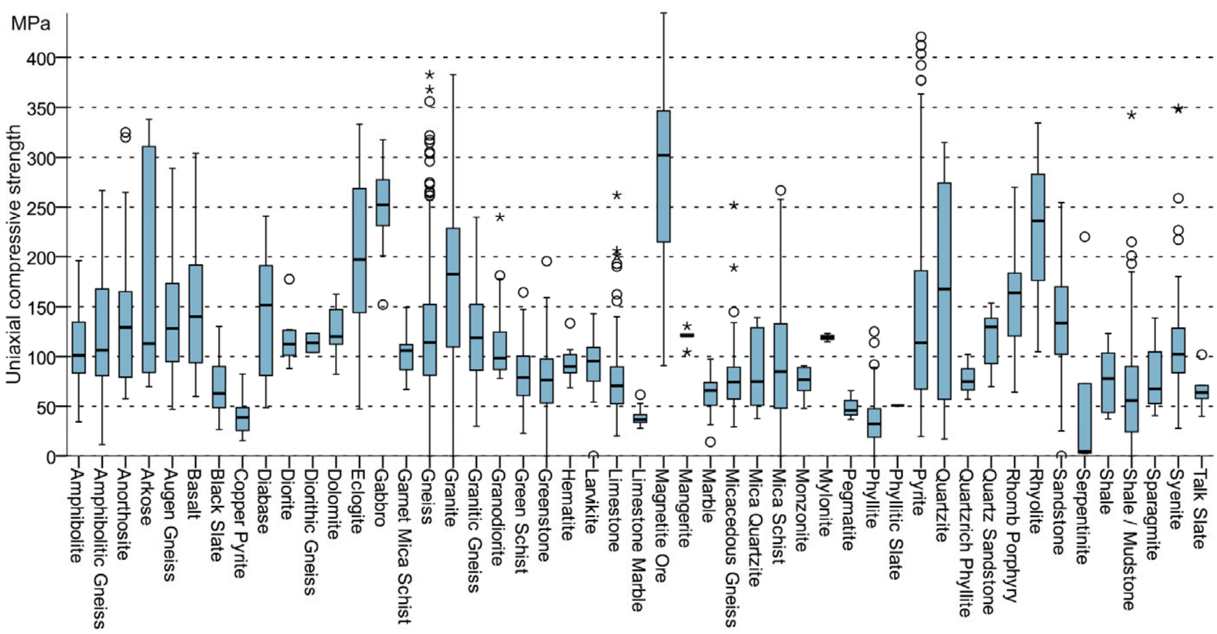

Fig. 13 Uniaxial compressive strength (UCS) in MPa for Norwegian rock types extracted from the SINTEF rock mechanical properties database (SINTEF 2016; Høien, Nilsen and Olsson 2019a).

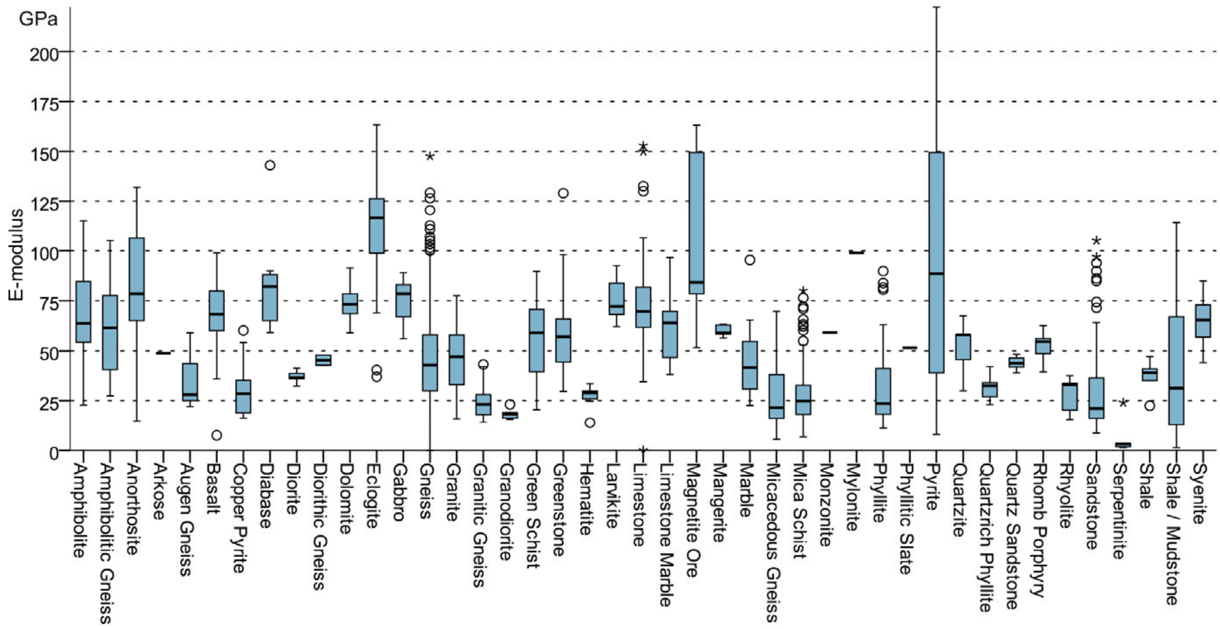

Fig. 14 E-modulus in GPa for different Norwegian rock types extracted from the SINTEF rock mechanical properties database (SINTEF 2016; Høien, Nilsen and Olsson 2019a).

2.3.5 Rock mass quality in Norwegian road tunnels

Registration of the Q-value has been a mandatory part of the mapping during excavation of road tunnels in Norway since 2010. Fig. 15 presents the distribution of rock mass quality from the excavation of 215 km of tunnels carried out over the past 15 years. GSI has been converted from the Q-value using a formula based on RQD and the Q-parameters J_r and J_a (Hoek, Carter and Diederichs 2013). The transition to what is referred to as a weakness zone for tunnelling in hard rock lies in the range GSI 30–40.

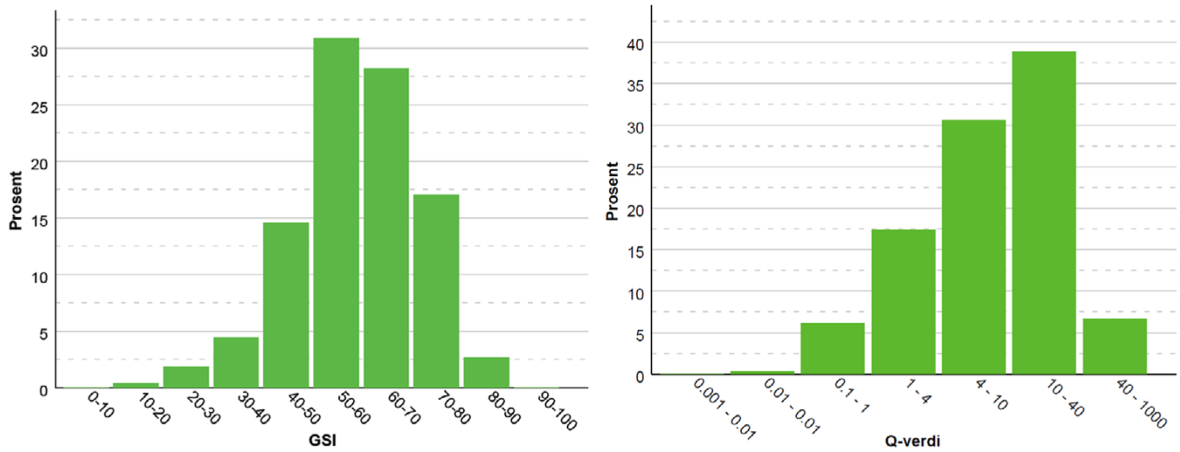

Fig. 15 Distribution of rock mass quality in Norwegian road tunnels built after 2011

3 Rock support in Norwegian road tunnels

3.1 Historical background

The manner in which road tunnels in Norway are excavated and provided with stability support historically originates from the hydropower industry and the tradition of unlined headrace and pressure tunnels. In the decades following the Second World War, there was extensive development of hydropower in Norway, and experience gained from hydropower construction was later transferred to infrastructure development as the share of hydropower projects declined and infrastructure development increased during the 1980s. During this period, technology and experience were developed that are still of benefit today. A road tunnel from this era is shown in Fig. 16. Perhaps one of the most important aspects carried forward from this period is the tradition that decisions regarding support and further excavation are made at the tunnel face. In addition to this, another essential factor in the development of the Norwegian tunnelling industry has been, and continues to be, the presence of highly qualified and solution-oriented crews carrying out the work at the tunnel face.

Fig. 16 The Kleinslått Tunnel on highway E16 in Vaksdal municipality between Bergen and Voss. The tunnel has a length of 55 m and was opened in 1970. A typical example of a road tunnel excavated according to principles adopted from hydropower development. Rock supported with rock bolts and some shotcrete. Top: Viewed from the south. Bottom: Viewed from the north

3.2 The system used today

Rock support in Norwegian road tunnels was, up until 2010, mainly determined subjectively on site for each blast round. From 2010 onwards, mapping using the Q-system (Barton, Lien and Lunde 1974) was introduced to register rock mass quality, along with engineering geological mapping of each blast round. In addition, a dedicated rock support table was introduced to determine the support measures (see Fig. 17).

In competent rock, stability is ensured by rock bolts and shotcrete. In weak rock masses, the use of spiling bolts and rebar reinforced ribs of sprayed concrete became increasingly common during the 1990s (Holmøy and Aagaard 2002). This development was driven by advances in shotcrete technology and the introduction of alkali-free accelerators, which made it possible to apply thicker layers of shotcrete. Rock support based on ribs of sprayed concrete has since replaced cast-in-place concrete linings, which were previously the most common support method in weak rock mass. A main reason for this is that ribs of sprayed concrete combined with spiling bolts allows for much faster advance rates, and that safety during construction is significantly improved; in particular, installation of formwork for cast concrete linings was a considerable challenge. Details of the combined excavation and support process based on the use of ribs of sprayed concrete are described in NFF (2008) and Pedersen, Kompen and Kveen (2010).

Pedersen, Kompen and Kveen (2010) also developed a support classification table, which was introduced as part of the Norwegian Public Roads Administration's regulations in 2010 (Statens vegvesen 2010). The recommendations were based on the Q-system support chart, with certain adjustments derived from their own experience. The table has undergone several minor revisions, including changes to the minimum shotcrete thickness to ensure durability requirements, and the 2024 version is shown in Fig. 17. The support classes are used to determine permanent support based on Q-value registration and an overall assessment, and are supplemented with additional support where necessary. In Fig. 18 the percentage distribution of the different rock support classes based on a dataset comprising mapping from 215 km of road tunnels are shown.

The Norwegian Public Roads Administration's rock support table has a clear threshold at Q = 0.2, where higher values indicate rock reinforcement using bolts and shotcrete, while lower values require a load-bearing structure in the form of symmetrical rib of sprayed concrete. In principle, for rock reinforcement with bolts and shotcrete, the bolts ensure the overall stability of the rock mass surrounding the tunnel, while the shotcrete stabilises the surface by forming wedges that prevent sliding along discontinuities and by bonding the rock mass surface together. The principal idea of symmetrical ribs of sprayed concrete, with their load-bearing design, is to establish equilibrium with the loads from the rock, thereby creating a stable tunnel. Rock support using ribs of sprayed is described in the following chapter and discussed further later in the report.

The Norwegian Public Roads Administration's regulations are fundamentally designed such that compliance with them also implies compliance with Eurocode 7 (Standard Norge 2020). In this context, use of the Q-system is regarded as *design by prescriptive measure* as described in Eurocode 7. In recent years, deformation measurements have become more common in Norway as a means of stability control; however, these are not yet systematised in a manner that would qualify them as application of the *observational method* as defined in Eurocode 7.

Rock mass class	**Rock conditions** **Q-value**	**Support class** **Permanent support**
A/B	Weakly jointed rock mass Average joint spacing > 1 m. Q = 100-10	**Support class I** - Scattered bolting - Sprayed concrete B35 E700, thickness 80 mm
C	Moderate jointed rock mass Average joint spacing 0.3 – 1 m. Q = 10-4	**Support class II** - Systematic bolting, c/c 2 m - Sprayed concrete B35 E700, thickness 80 mm
D	Strongly jointed rock mass or bedded schistose rock. Average joint spacing < 0.3 m. Q = 4-1	**Support class III** - Sprayed concrete B35 E1000, thickness 100 mm - Systematic bolting, c/c 1.75 m
E	Very poor rock mass. Q = 1-0,2	**Support class IVa** - Systematic bolting, c/c 1.5 m - Sprayed concrete B35 E1000, thickness 150 mm
	Q = 0,2-0,1	**Support class IVb** - Systematic bolting, c/c 1.5 m - Sprayed concrete B35 E1000, thickness 150 mm - Reinforced ribs of sprayed concrete: Rib dimension E30/6 Ø20 mm, c/c 2 – 3 m, alt. lattice girders Bolting along arch c/c 1.5 m, length 3 – 4 m - Consider need for invert cast concrete
F	Extremely poor rock mass. Q = 0,1-0,01	**Support class V** - Systematic bolting, c/c 1.0 – 1.5 m - Sprayed concrete B35 E1000, thickness 150 – 250 mm - Reinforced ribs of sprayed concrete: Rib dimension D30/6+4 Ø20 mm, c/c 1.5 – 2 m, alt. lattice girders Bolting along arch c/c 1.0 m, length 3 – 6 m - Consider need for invert cast concrete
G	Exceptionally poor rock mass, mainly soil like Q < 0,01	**Support class VI** - Excavation and support design to be evaluated for each case

Fig. 17 Rock support table used in Norwegian road tunnels from specification N500 Road tunnels (Statens vegvesen 2024)

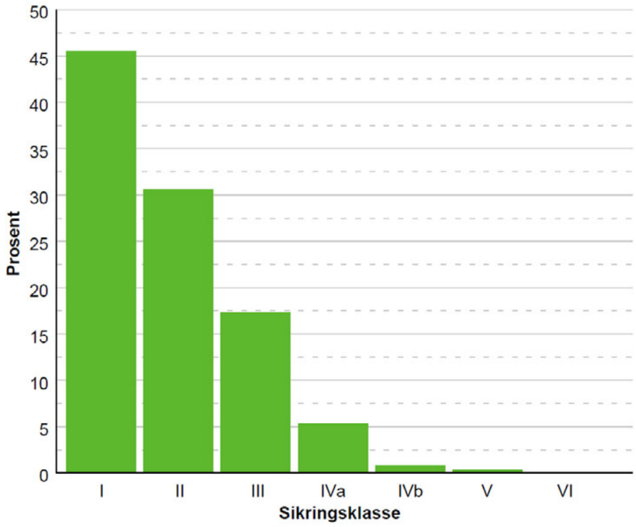

Fig. 18 Distribution of support classes based on 215 km of tunnel excavation and 36 500 individual face mappings

3.3 Rock support of weak rock and ribs of sprayed concrete

Current practice in weak rock is to provide support using ribs of sprayed concrete. One of the first systematic applications of ribs of sprayed concrete was during the construction of the Fjellinjen tunnel beneath Akershus Fortress, now part of the Opera Tunnel, in order to pass a weakness zone with low overburden (Berge and Fromreide 1989; Garshol and Blindheim 1989). The ribs of sprayed concrete were used in combination with spiling bolts, a technique that is still commonly applied today.

In Norway, ribs of sprayed concrete have completely replaced the use of cast-in-place concrete as rock support in weak rock mass and are used as an integrated part of tunnel excavation. Fig. 19 illustrates the process of constructing ribs of sprayed concrete. To stabilise the ground after blasting of the next round, spiling bolts are installed close to the tunnel face in drill holes located just outside the theoretical crown, with a slight outward inclination. Spiling bolts are usually grouted Ø32 mm reinforcement bars, secured at the rear end using steel straps and radial rock bolts to keep them in place when loaded. The inner anchoring is also important and the embedded bolt length should be to meters past the new face. After blasting, an initial layer of shotcrete is applied as temporary support. Radial rock bolts are then installed for permanent support and for mounting the reinforcement bars of the ribs. As shown in Fig. 19, threaded extension pieces are mounted on the rock bolts to provide sufficient projection for fastening the reinforcement brackets. A levelling layer of shotcrete is subsequently applied before installing the reinforcement brackets and the longitudinal reinforcement bars. To ensure a circular shape, a total station is used to mark the position of the brackets. The reinforcement bars are then embedded in shotcrete, and if required, additional radial rock bolts are installed between the ribs. For further details on the construction of ribs of sprayed concrete, see Holmøy and Aagaard (2002) and Statens vegvesen (2023).

In the early application of ribs of sprayed concrete, the levelling layer was applied as thin as possible. Rock bolts were then installed, and Ø16 mm reinforcement bars were fastened to transverse bars on the rock bolts, following the tunnel surface. These asymmetric ribs of sprayed concrete, or perhaps more accurately bar-reinforced shotcrete, may be regarded as rock support that assists the rock itself in carrying the loads by holding it in place. This represents one of the two main support concepts illustrated in Fig. 20 a).

The other main design is symmetric ribs of sprayed concrete (Fig. 20 b), which are shown in Fig. 19 and constitute the most commonly used solution today. These are considered a load-bearing structure. The principle is that when the rib of sprayed concrete is curved, the load acting on the structure is transferred as compressive force tangential to the rib, as other ring-shaped rock support systems.

The transition from asymmetric to symmetric ribs of sprayed concrete was introduced by Grimstad et al. (2002), who analytically and numerically designed symmetric ribs of sprayed concrete and linked them to the support chart of the Q-system. These calculations are commented on in Section 6.3. In Norway, the actual transition took place in 2010, when the Norwegian Public Roads Administration made the use of the Q-system mandatory as a rock mass classification method and introduced its own rock support table.

Grimstad et al. (2008) instrumented symmetric ribs of sprayed concrete in two tunnels and measured deformations, concluding that the ribs had significant excess capacity. Subsequently, Høien and Nilsen (2018) analysed the stabilising effect of symmetric ribs of sprayed concrete in the Løren tunnel in Oslo, based on instrumented ribs, deformation and stress measurements, and rock mass quality. The main conclusion from their analysis was that the ribs did not act as load-bearing elements, but rather as a form of rock reinforcement, since only minimal deformations occurred in the rock mass that could transfer loads to the rock support system. Nevertheless, in these rock mass conditions, more support than fibre-reinforced shotcrete and bolts alone was required for stability. It was further suggested that a solution that is not load-bearing, but more robust than shotcrete and bolts, could be used as an alternative to load-bearing rock support. This research was followed up by Høien, Nilsen and Olsson (2019a), who investigated deformations and rock support in Norwegian road tunnels. They evaluated a large dataset covering 85 km of Norwegian road tunnels supported according to the Norwegian Public Roads Administration's rock support table. For the 14,000 cases in the dataset, where each case represents mapping of a single blast round, rock mass quality parameters, rock support, and cover depth were included. The data were evaluated using parametric numerical analysis, the critical strain concept of Sakurai (1981), and strain–rock support relationships (Hoek 1999; Hoek 2001; Hoek and Marinos 2000; Sakurai 1983). The numerical parametric study forming part of this analysis is presented in Section 8.2. The results showed that only a very small portion of the excavated tunnel length supported with load-bearing ribs of sprayed concrete exhibited rock mass behaviour for which load-bearing support was likely necessary. Deformation measurements from several tunnels, as well as the case study of the Frøya tunnel (discussed in the following paragraph), supported the findings of the parametric study. It was concluded that the use of symmetric ribs of sprayed concrete for all ground conditions with Q-values below 0.2 is probably unnecessary, and that a system better adapted to the actual deformation conditions should rather be applied.

Holmøy and Aagaard (2002) describe the rock support of weakness zones in the subsea Frøya tunnel (overburden = 37–155 m), a case also studied by Høien, Nilsen and Olsson (2019a). In the Frøya tunnel, which included a large proportion of extremely poor rock mass quality (Q = 0.1–0.01), both asymmetric ribs of sprayed concrete (Fig. 20 a) and the use of spiling bolts fixed with rock straps, in combination with bolts and shotcrete, were used as permanent rock support. During excavation, an extensive monitoring programme was implemented to observe deformations and to ensure stable permanent rock support. Final deformations of up to 2–3 cm were recorded.

Another notable development in the use of ribs of sprayed concrete in road tunnels in Norway is the use of deformation measurements for verification of stable design. In both the Fjellinjen tunnel (Garshol and Blindheim 1989) and the above-mentioned subsea Frøya tunnel, which opened in 2000, deformation measurements were used to evaluate stability during construction. Thereafter, it is the author's impression that deformation measurements were seldom used for design verification in conjunction with ribs of sprayed concrete until recent years, when total stations and reflectors have increasingly been applied to confirm structural stability. Most recently, extensive use of deformation measurements combined with ribs of sprayed concrete has been carried out in an area with hard moraine and highly altered rock in the Bergsås tunnel, where numerical modelling was also performed in parallel with the measurements to ensure stability (Jakobsen, Nilsen and Lund 2024).

Fig. 19 a) Spiling bolts to support the next blasting round are pre-mounted with steel straps and radial rock bolts. Radial rock bolts for three ribs of sprayed concrete are mounted with thread protectors (yellow). b) A layer of sprayed concrete is applied to smoothen the surface. The spiling bolt mounting is also covered with sprayed concrete to secure it. Pre-bent rebars are mounted in brackets fixed to the rock bolts. c) The ribs of sprayed concrete is sprayed with concrete to form the ribs (Høien 2018)

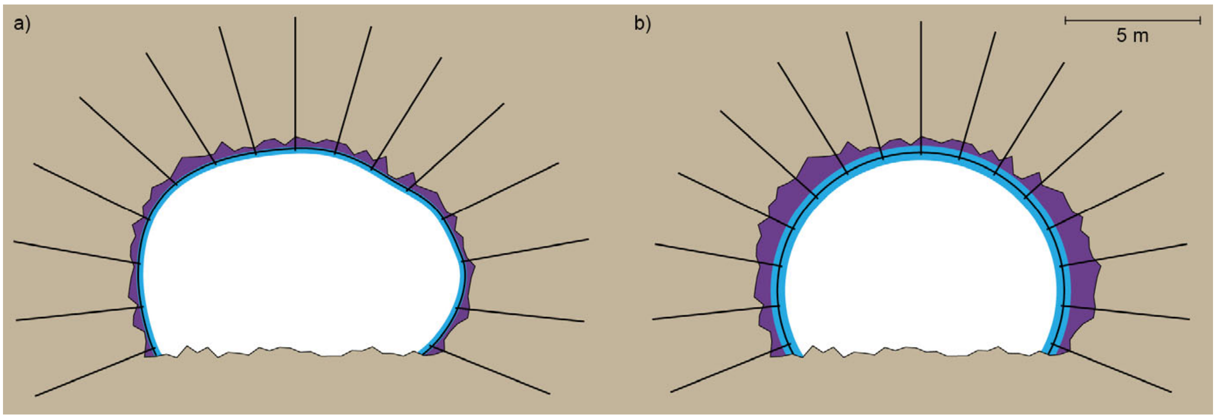

Fig. 20 Principle of a) an asymmetrical rib of sprayed concrete and b) a symmetrical rib of sprayed concrete. In these diagrams, black lines illustrate the rebars and rock bolts, the purple area illustrates the sprayed concrete smoothing layer, and the light blue area represents the final layer covering the rebar. The rough surface in the diagram corresponds to the blasted profile (Høien, Nilsen and Olsson 2019a)

4 Short on rock engineering design in Eurocode 7

The Eurocodes constitute a common European set of standards for the design of buildings and structures. They provide guidelines for dimensioning and safety and are intended to ensure uniform practice across countries. Eurocode 7 (Standard Norge 2020), which addresses geotechnical design, applies to the assessment of stability in both soil and rock, including stability of tunnels. It provides a framework for how safety against failure and deformation shall be documented and describes different methods for design and verification of stability.

Eurocode 7 defines four main methods for geotechnical design and verification of stability. These methods are described in Clauses 2.4 to 2.7 of the standard and are used to ensure that a structure is stable under various conditions. Below, the four types of verification are briefly described, with reference to the Norwegian Group for Rock Mechanics' guideline for Eurocode 7 (NBG 2011) for a more detailed explanation.

Geotechnical design by calculation involves the use of analytical or numerical calculations to document stability. Performing such calculations requires very good knowledge of material parameters, ground conditions, stresses, and related factors.

Geotechnical *design by prescriptive measures* is intended to provide satisfactory stability based on experience and normal practice. In the English original version, this approach is referred to as *prescriptive measures*, which may be translated into the more general terms prescribed measures or standardised measures. These methods or measures may be used when calculation models are not available or not necessary and are based on conventional and conservative design rules. According to the NBG guideline for Eurocode 7, measures based on this approach may be used, for example, for the excavation and support of tunnels in simple rock conditions.

Load tests and tests on experimental models involve verification of stability by performing load tests in situ or by means of physical models, and are considered to be of limited relevance for the stability assessment of tunnels and caverns.

The *observational method* is, in principle, based on verifying assumptions and completed design through measurements and observations during construction. Plans shall be prepared during design that include effective countermeasures if a predefined acceptable level of stability is not achieved during construction. Documentation of conditions by means of deformation measurements may be relevant for verification of overall stability, while for local or detailed stability, normal visual inspections during construction follow-up will usually be considered sufficient.

According to the NBG guideline, the observational method is not an independent design method, but a systematic method for documenting compliance with the design criteria. It is used as a system for documenting design carried out using one of the other three methods. Reference is also made to Clause 2.7 of Eurocode 7 (Standard Norge 2020).

The design shall ensure that the structure satisfies requirements for safety, functionality and durability throughout its service life. In the Eurocodes, this is linked to limit states, where a limit state is defined as a condition beyond which the structure no longer fulfils one or more of the above mentioned requirements. Eurocode 0 divides limit states into two groups:

- **Ultimate limit states (ULS):** when the structure loses load-bearing capacity or stability, potentially leading to collapse or serious damage.
- **Serviceability limit states (SLS):** when the structure remains standing but no longer functions as intended, for example due to excessive deformations.

In design, each governing design situation shall be checked to ensure that the limit states are not exceeded. In a tunnel, different design situations may occur along the alignment, while at the same time it is difficult to predict the exact ground conditions at a given location in advance. This requires continuous design and follow-up to ensure that limit state requirements are maintained. Reference is made to Clause 2.1 of Eurocode 7 (Standard Norge 2020) and Section 3 of Eurocode 0 (Standard Norge 2016).

5 Short on principles of rock reinforcement and support

The type and extent of rock reinforcement and support required to achieve a stable tunnel vary considerably, ranging from minimal reinforcement in competent rock masses with favourable stress conditions to extensive support in weak and deformable rock masses subjected to unfavourable stresses.

In competent rock masses with relatively high strength, low deformability, a limited degree of fracturing and weathering, and an absence of time-dependent deformations, rock reinforcement is often sufficient to achieve stable conditions. This may involve the use of sprayed concrete as surface support, where it binds fragments and smaller blocks together, forms concrete wedges that hold the rock in place, and thereby maintains a stable rock surface (NB 2022). Because deformations in the rock mass are negligible, the surface support is not subjected to loads from the rock stresses, but rather to local forces that mainly arise from gravity acting on locally fractured rock. In addition, rock bolts are installed to reinforce the rock mass and to retain larger unstable volumes. Bolting is commonly applied either as spot bolting, where each bolt is placed to stabilise a specific volume or key block, or as systematic bolting. In systematic bolting, bolts are installed in a regular pattern with equal spacing, on the assumption that they will intersect the volumes and key blocks requiring stabilisation. In more heavily fractured rock masses, and when using pre-tensioned bolts, systematic bolting may create a compressive arch in the rock that is favourable for stability. See also NFF (2020).

In contrast, in very weak rock masses with relatively low strength, high deformability and possible time-dependent deformations, the stresses in the rock mass may lead to large deformations. To establish a stable tunnel, it is essential to control and ultimately stop the deformation caused by the rock stress. This is achieved by installing rock support that is loaded by the deforming rock mass. These load-bearing systems interact with the ground and balance the forces between the two to create a stable condition, an equilibrium. In such cases, both numerical and analytical methods exist to assess the required extent of support. One analytical method is the convergence–confinement method (CCM). According to Hoek (2001) and Carranza-Torres and Fairhurst (2000), the concept describing the interaction between rock mass and rock support can be traced back to the 1930s and was further developed during the 1960s and 1970s. The fundamental idea behind CCM is that the behaviour of the rock mass is described by a ground reaction curve (GRC), which expresses tunnel convergence as a function of excavation advance. Correspondingly, the behaviour of the support system is described by a reaction curve, where the support pressure increases from the time of installation as the rock mass deforms, until equilibrium is reached (see Fig. 21).

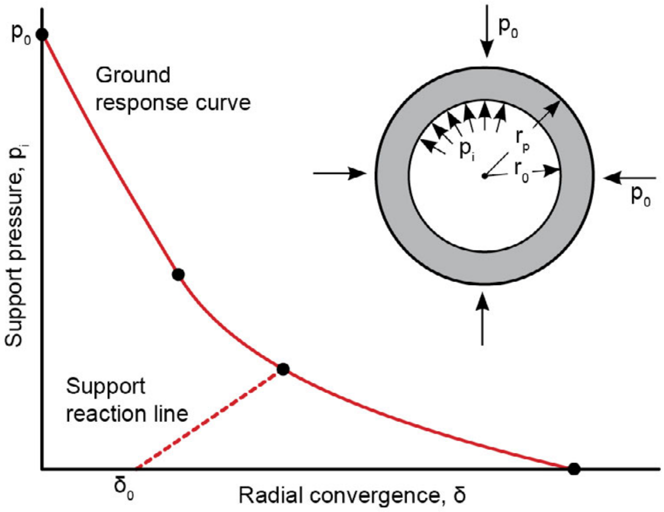

Fig. 21 Principle of CCM with equilibrium between the rock mass and rock support. Based on (Hoek 2001)

Between the two end-members represented by the principles of rock reinforcement (shotcrete and bolts) and rock support based on equilibrium between the ground and the support system, there exists a wide spectrum of stability problems. Under such ground conditions, tunnelling in Norway has traditionally been carried out using spiling, fixed with steel straps and rock bolts, followed by support with shotcrete and bolts, often supplemented by ribs of sprayed concrete that either follow the rock surface or are symmetrical.

In these ground conditions, the load regime acting on the rock support is highly variable, ranging from gravitational loads in the crown due to closely fractured rock, to displacements along discontinuities caused by the overall rock stresses, and squeezing of clay-rich material in weakness zones. Assessing stability under such ground conditions is highly complex. It is recommended to become familiar with the hybrid approach presented by Terron-Almenara, Holter and Høien (2023) and Terron-Almenara (2025).

6 Limitations of the current system for rock support in road tunnels

Mandatory geological mapping at the tunnel face, rock mass classification using the Q-system, and the introduction of a dedicated rock support table represented a major step forward in improving the determination of rock support and increasing structural safety when these measures were introduced in 2010. This approach has functioned relatively well since then; however, over the years a number of both practical and theoretical weaknesses in the methodology have been identified, as well as inconsistencies with the European regulatory framework for ensuring structural safety, the Eurocodes.

Alejano (2024) provides a general assessment of the use of rock mass classification systems and points out that while these systems possess many valuable qualities, they are also often misused. He notes that they were developed in the 1970s in response to the need for establishing guideline recommendations for rock support. As part of this development, various engineering geological properties were assigned numerical values to describe an overall rock mass quality. To enable their use as design tools for rock support, parameters not directly related to the rock mass itself were included, such as the orientation of geological structures relative to the tunnel axis and stress conditions. This broad approach, combined with inherent limitations, has led to some confusion in their application. Alejano (2024) further argues that rock mass classification systems have proven to be useful tools for systematising rock characterisation and providing an initial assessment of rock mass behaviour, for planning and evaluating possible types of rock support and excavation methods, and not least for facilitating communication among professionals. He also states that rock mass classification systems are undoubtedly useful tools for approaching sound rock engineering design solutions and for decision-making, but that they do not function adequately in all cases. To arrive at good solutions, he argues that assessments based on a combination of different design methods are required, together with proper verification of the design through observations, measurements, and monitoring of the excavation and support process.

6.1 Practical weaknesses

Although regulations and systems for determining rock support may appear to be adequate, they depend on the ability to implement them in practice at construction sites, and on whether the specified approaches *can* in fact deliver the intended results. To investigate how tunnel stability has been assessed over time, Høien, Andersen and Glenjen (2024) conducted a review of inspection reports for tunnels supported according to the post-2010 regulatory framework. The objective was to examine how stability may have evolved over time and which measures had been implemented to improve rock support. For each tunnel, the measures were assigned an intervention level consisting of four categories: none, minor, significant, and extensive measures. *Minor measures* typically involve the installation of up to a few tens of bolts, placed where sounding indicates voids or where cracking has developed in the sprayed concrete. *Significant measures* typically include installation of more than 100 bolts, systematic bolting due to insufficient original support, exposed clay-rich rock, and/or the risk of long-term development of stability problems. *Extensive measures* indicate deficiencies in rock support that pose a risk to overall stability and require redesign of the rock support system.

Among the tunnels in the dataset, 18 % required minor measures and 10 % required significant measures, while no cases were classified as extensive at the time of the study. After publication of the report, however, a periodic inspection identified one case that falls within the category of extensive measures, involving problems with overall tunnel stability and requiring substantial remedial support. The report concludes that although the systems function satisfactorily in most cases, these figures are too high and indicate that under the current system, the intended 100-year service life of the rock support, as required by the regulations, is not achieved in too many cases. The discovery of a case involving overall stability problems after the publication of the report suggests that the somewhat moderate conclusions of the report should be reconsidered. This finding shifts the consequence of using the rock support system from being a matter of repair to a stability issue that could lead to a serious accident. This must also be evaluated in light of the fact that large parts of the tunnels are lined, making inspection impossible. As a result, the actual condition of stability and support is unknown, which is particularly problematic given that it is now known that application of the system may lead to severe stability problems.

The way tunnels are supported cannot be considered independently of the excavation approach. Especially in weak rock masses, the use of spiling, other supplementary rock support, adjusted round lengths, and assessment of continued advance constitute an integrated process. Feedback has been given that, in many cases, the Q-value is adjusted so that the perceived conditions and the level of rock support, based on one's own subjective experience and the recommendations in the rock support table in N500 Road Tunnels, correspond to each other. One may argue that such an approach is “incorrect”; however, it also indicates that the Q-values obtained from mapping do not always recommend what professionals consider to be appropriate rock support. Whether this should be characterised as misuse of the Q-system or as shortcomings within the Q-system itself is of secondary importance. The key issue is that when such practices occur in the field—as reported by experienced professionals—it is problematic that the framework for excavation and support does not function as intended.

An important element in verifying that a tunnel is stable is the requirement in N500 Road Tunnels to carry out an inspection of the rock support before installing the tunnel lining. Tunnels are generally fully supported close to the face; however, it is not uncommon to identify weaknesses in the rock support during the excavation stage behind the face, requiring supplementary measures. Such cases may have multiple underlying causes and may in some instances be anticipated, for example when supplementary measures are planned in predefined monitoring zones. Nevertheless, they also illustrate a limitation of the current rock support system, as it struggles to predict certain time-dependent factors and relies on these becoming apparent in the period between blasting and completion of construction. At the case level, instances have been observed where relatively weak rock types, such as clay shale and soapstone, become overstressed even at relatively low stress levels or overburden, with soapstone exhibiting deformations that may develop over many years.

It is also not uncommon for unforeseen events to occur during tunnelling, including collapses at the face where the rock support fails. Such incidents are often the result of ground conditions differing from assumptions and are therefore not necessarily caused by insufficient support, but rather by incorrect assessment of conditions ahead of the face prior to blasting. Most such cases are not documented in publicly available sources. However, one documented case in which stability problems were not captured by the rock support system is described by Bøgeberg and Skretting (2021) and Terron-Almenara et al. (2024). In this case from the Skarvberget tunnel in Finnmark, ribs of sprayed concrete were bent downwards in the crown behind the face in a rock type characterised by pronounced, continuous horizontal fracturing. As noted above, cases involving overall stability problems are rare, and since there is no systematic reporting system, a statistical approach for identifying correlations is not possible

6.2 Weaknesses in the theoretical basis

Rock support is determined through an overall assessment of the engineering geological mapping, the determination of the input parameters in the Q-system, and the resulting Q-value. The corresponding support level is then specified by the rock support table in N500 Road Tunnels, shown in Fig. 17. For the system to function as intended, it is crucial that the calculated Q-value leads to a correct choice of support for ground conditions in question.

Palmstrom and Broch (2006) carried out a thorough review of the input parameters of the Q-system and its overall framework. They concluded that, when used with awareness of its limitations, the system is suitable for stability classification and for estimating rock support in fractured rock during the planning phase. However, it is less suitable as a basis for selecting rock support during the excavation phase across its entire stated range of applicability. They identified an interval in which the Q-system support chart performs best, corresponding to Q-values between 0.1 and 40. Later research has also pointed out problems with support recommendations in the lower range of the Q-system. Høien, Nilsen and Olsson (2019a) concluded that for Norwegian road tunnels, load-bearing rock support such as symetrical ribs of sprayed concrete, as described in the Q-system support chart and the Norwegian Public Roads Administration’s support table, is in most cases not required for Q-values below 0.2. They further stated that it may be possible to apply lighter rock support solutions than those currently used, in combination with systematic deformation measurements to ensure tunnel stability.

The Q-system and the Norwegian Public Roads Administration’s support table prescribe increasing amounts of rock support with decreasing Q-value. From a Q-value of 1000 down to approximately 0.2–0.4, progressively larger quantities of bolts and sprayed concrete are prescribed. At this point, a transition occurs and the support concept shifts to ribs of sprayed concrete with increasing dimensions as the Q-value decreases further. A key strength—and at the same time a weakness—of the Q-system is that all types of stability problems are assigned these same support types. Palmstrom and Broch (2006) noted that support recommendations are generally appropriate within the range where sprayed concrete and bolts are specified. However, for areas with lower Q-values, the stability challenges present a level of complexity that this method cannot fully address. This has also been highlighted by Terron-Almenara (2025) and Terron-Almenara, Holter and Høien (2023), who showed, based on a large number of numerical analyses using rock mass classification and deformation measurements, that ribs of sprayed concrete are in many cases significantly oversized. An important point in this context is that they may also be undersized, as failures have been observed in some cases (Brende, Lia and Nilsen 2018; Terron-Almenara et al. 2024).

Fig. 22 illustrates areas within a tunnel system that deviate from conventional tunnel geometries consisting of single or twin parallel tubes. The empirical relationship between Q-value and rock support is primarily based on the most common tunnel geometries from hydropower and road tunnels (Grimstad and Barton 1993), and becomes weaker as tunnel cross-section increases. In the Norwegian Public Roads Administration’s regulations, the use of the rock support table in N500 Road Tunnels is limited to tunnel widths of up to 12.5 m (Fig. 17). Palmstrom and Broch (2006) defined an upper limit of 30 m in the support chart itself and also stated that the Q-system should not be applied at Q-values below 0.1. The areas shown in Fig. 22 lie outside the range geometrically covered by the empirical basis of the Q-system and are considered not to fulfil the definition of geotechnical *design by prescriptive* measures in Eurocode 7.

Consequently, the requirements for structural safety in the Eurocodes are not met when the Q-system is applied under excavation conditions corresponding to Q-values below approximately 0.1–0.4 and/or in non-standard geometries such as those shown in Fig. 22, including situations with low overburden. In addition, other factors also influence whether the Eurocode requirements are satisfied when using prescriptive measures and the Q-method, such as personnel qualifications and the extent and quality of stability follow-up/control during the excavation phase.

Fig. 22 Areas where the geometry is such that mapping using Q-values and rock support based on specification N500 or the Q-system’s support recommendations is not sufficient. Example from the Ring Road West project in Bergen

6.3 The Q-system and dimensioning of ribs of sprayed concrete

An important element of the Norwegian Public Roads Administration’s support chart (see Fig. 17) is the dimensioning of ribs of sprayed concrete for Q-values below 0.2. The recommendations by Pedersen, Kompen and Kveen (2010) represent a simplification of the initial proposals for dimensioning presented by Grimstad et al. (2002), see Fig. 23 (left). Later, when NGI published the first edition of the handbook *Use of the Q-system* (NGI 2013), certain adjustments to the dimensioning were introduced, partly based on the recommendations and simplifications proposed by Pedersen, Kompen and Kveen (2010). The newest version from NGIs handbook is shown in Fig. 23 (right).

In Høien (2024), the dimensioning recommendations for ribs of sprayed concrete in the Q-system support chart presented by Grimstad et al. (2002) are critically reviewed. Since the Norwegian Public Roads Administration's support recommendations are based on Grimstad et al. (2002), this critique also applies to the current regulations for road tunnels. (Høien 2024) identifies four separate issues, each of which is considered to represent a significant source of error rendering the dimensioning approach unsuitable from a rock engineering perspective:

- the inability of the Q-system to indicate the need for load-bearing rock support,
- dimensioning based on load input derived from the P_{arch} formula,
- modelling of free-standing ribs of sprayed concrete without interaction with the rock mass
- use of an "acceptance limit" for permissible deformation of the ribs of sprayed concrete.

The arguments presented by Høien (2024) for the issues listed above are summarised in the following subsections.

In the conclusions of Høien (2024), it is pointed out that the dimensioning calculations are technically structured such that ribs of sprayed concrete are treated as load-bearing rock support, dimensioned solely on the basis of a mapped Q-value. It is further emphasised that a Q-value does not adequately predict deformations for the purpose of dimensioning this type of rock support. An "acceptance limit" for allowable deformation of the ribs is also introduced in the calculations. This limit is defined based on *engineering judgement* and overrides all other calculations in such a way that the resulting dimensions in the chart fall within a range Grimstad et al. (2002) consider reasonable. This effectively renders the preceding calculations irrelevant, and it will be shown later that this approach also has its own limitations. Furthermore, it is noted that the chain of reasoning—from Terzaghi's theory to the determination of rib dimensions for all types of rock mass behaviour—lacks a logical and consistent structure. A general concern raised in Høien (2024) is that stability problems in the lower range of the Q-system are highly diverse, and that a single rock support solution—such as that currently prescribed—is not well suited to all situations. The stability problems, ranging from heavily fractured rock and gravity-dominated behaviour at low stress levels to squeezing ground and rockburst-type behaviour in competent rock at high stress levels, are too varied for a single solution to be sufficient.

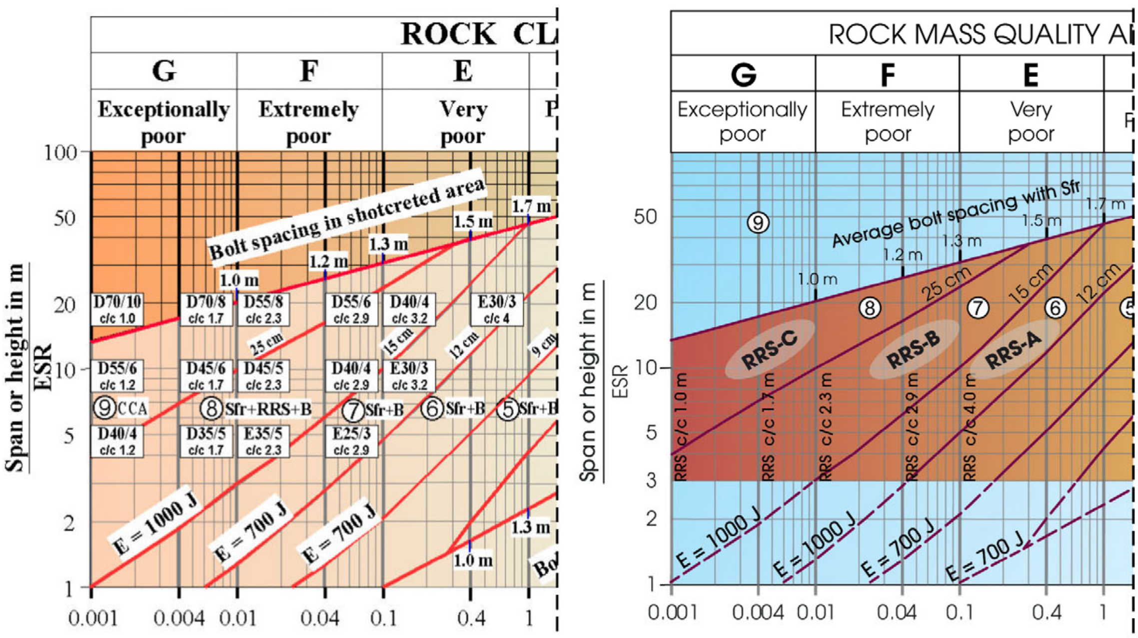

Fig. 23 Clipping of the rock support diagrams by Grimstad et al. (2002) (left) NGI (2025) (right). In the "flags", E/D denotes single/double layers of reinforcement, followed by the rib thickness in cm and then the number of reinforcing bars. c/c indicates the centre-to-centre spacing between the ribs in the tunnel direction. In the most recent diagram, the dimensions are specified in the support categories as RRS-A, B, and C. Details for dimensioning is given in an own table for each category with different dimensions for tunnel widths 5, 10 and 20 m.

6.3.1 The Q-systems ability to indicate the need for load-bearing support

In principle, a Q-value must be able to predict the deformation and the loads from the rock mass resulting from tunnel excavation in order to be used for the dimensioning of load-bearing rock support. The principle is that it must be possible to calculate an equilibrium between the rock mass and the rock support, which is briefly explained in Section 5.

The equation for the Q-value is divided into three main factors: degree of jointing (RQD/J_n), joint friction (J_r/J_a) and active stress (J_w/SRF) (NGI 2025). Two key factors that determine the load-carrying capacity of the rock support are rock mass quality and stress conditions. In the Q-system, these are represented by the first two and the last main factor, respectively. The first two factors describing rock mass quality will not be commented on here, whereas the last – active stress –will be discussed in more detail below.

The active stress factor consists of two parameters: the joint water factor, J_w, and the stress reduction factor, SRF. J_w will not be discussed in detail further, but whether it can actually be regarded as an "active stress" can be questioned, since the descriptions in the table mainly concern practical challenges related to water ingress rather than stresses caused by water pressure. The table used to determine the SRF value is shown in Fig. 24 and is divided into four categories: (a) weak zones intersecting the underground opening, which may cause loosening of rock mass; (b) competent, mainly massive rock, stress problems (c) squeezing rock: plastic deformation in incompetent rock under the influence of high pressure; and (d) swelling rock: chemical swelling activity depending on the presence of water.

The descriptions in category (a) consist mainly of geological conditions rather than stress-related conditions, except for some differentiations below/above 50 m overburden and stress redistribution related to filled joints and weak zones. According to Høien (2024), however, this information is not suitable for differentiating load-bearing support. In many cases, the stability problems in this category are geological and geometrical, and more gravity-controlled than stress-related. Category (b) includes competent rock leading to spalling and rockburst. These are conditions that do not necessarily require load-bearing support, but which nevertheless require extensive support measures.

The squeezing category (c) describes the response of the rock mass to stress and is, in that sense, a stress parameter. However, the rock mass response to stress also depends on rock quality and is therefore not a pure stress parameter, but also geologically controlled. This is supported by the σ_θ/σ_c ratio (tangential stress/intact rock strength) shown in the SRF table (Fig. 24), where note (iv) refers to Singh et al. (1992) for correlations between Q-value and squeezing. An important point is that Singh et al. (1992) themselves point out that their conclusions are preliminary and that it may be "premature to draw final conclusions" from these correlations. Grimstad and Barton (1993) used the work of Singh et al. (1992) to derive the values for the σ_θ/σ_c ratio presented in Fig. 24. Although the method can be discussed, it is not elaborated further here. Palmstrom and Broch (2006) are highly critical of the squeezing parameter and question how a single Q-value can effectively determine rock support for such complex ground conditions. They also point out that the New Austrian Tunnelling Method (NATM) has largely been developed to handle such complex ground conditions by applying an integrated methodology that combines analysis and excavation methods. Furthermore, NGI (2025) emphasises in the latest edition of the Q-handbook that, in squeezing ground, it is important to combine the Q-system with deformation measurements and numerical modelling, and to adapt the timing of support installation to the deformation development, in a manner similar to the principles of CCM, which is commonly used in NATM. The σ_θ/σ_c relationship is for squeezing rock removed from the SRF-table Q-handbook in the 2025 edition.

Category (d), chemical swelling, includes moderate and intense swelling and is, in that sense, a stress parameter. Nevertheless, Høien (2024) argues that the sub-categories are very vague, and that the descriptions in the literature and guidelines are insufficient in light of the large influence this has on the Q-value and the associated support. Palmstrom and Broch (2006) point out that special investigations and measurements must be carried out in such ground conditions, together with thorough analyses, to assess the effects of excavation and rock support. It is also a question whether chemical swelling in clay-rich materials is a relevant parameter for determining rock support (Høien 2022; Høien, Nilsen and Olsson 2019b), and whether it is at all possible to distinguish between creep, mechanical/chemical swelling, consolidation and squeezing (Einstein 1996; Høien et al. 2019).

As mentioned earlier, the ribs-of-sprayed-concrete calculations are based on a load calculated from a Q-value. The Q-value must therefore be able to predict the deformations in the rock mass for the underlying assumption of the calculation to be satisfied. Considering the categories in the SRF table described above, Høien (2024) is of the view that the stress input parameter in the Q-system, in the form of SRF, does not have sufficient quality to predict deformations with the accuracy required to be used as input for the dimensioning of load-bearing rock support.

6	**Stress Reduction Factor**			**SRF**
a)	*Weak zones intersecting the underground opening, which may cause loosening of rock mass*			
A	Multiple occurrences of weak zones within a short section containing clay or chemically disintegrated, very loose surrounding rock (any depth), or long sections with incompetent (weak) rock (any depth). For squeezing, see 6L and 6M			10
B	Multiple shear zones within a short section in competent clay-free rock with loose surrounding rock (any depth)			7.5
C	Single weak zones with or without clay or chemical disintegrated rock (depth ≤ 50m)			5
D	Loose, open joints, heavily jointed or "sugar cube", etc. (any depth)			5
E	Single weak zones with or without clay or chemical disintegrated rock (depth > 50m)			2.5
Note:	i) Reduce these values of SRF by 25-50% if the weak zones only influence but do not intersect the underground opening			
b)	*Competent, mainly massive rock, stress problems*	σ_c/σ_1	σ_θ/σ_c	**SRF**
F	Low stress, near surface, open joints	>200	<0.01	2.5
G	Medium stress, favourable stress condition	200-10	0.01-0.3	1
H	High stress, very tight structure. Usually favourable to stability. May also be unfavourable to stability dependent on the orientation of stresses compared to jointing/weakness planes*	10-5	0.3-0.4	0.5-2 2-5*
J	Moderate spalling and/or slabbing after > 1 hour in massive rock	5-3	0.5-0.65	5-50
K	Spalling or rock burst after a few minutes in massive rock	3-2	0.65-1	50-200
L	Heavy rock burst and immediate dynamic deformation in massive rock	<2	>1	200-400
Note:	ii) For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.75\,\sigma_c$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c to $0.5\,\sigma_c$, where σ_c = unconfined compression strength, σ_1 and σ_3 are the major and minor principal stresses, and σ_θ = maximum tangential stress (estimated from elastic theory) iii) When the depth of the crown below the surface is less than the span; suggest SRF increase from 2.5 to 5 for such cases (see F)			
c)	*Squeezing rock: plastic deformation in incompetent rock under the influence of high pressure*		σ_θ/σ_c	**SRF**
M	Mild squeezing rock pressure		1-5	5-10
N	Heavy squeezing rock pressure		>5	10-20
Note:	iv) Determination of squeezing rock conditions must be made according to relevant literature (i.e. Singh et al., 1992 and Bhasin and Grimstad, 1996)			
d)	*Swelling rock: chemical swelling activity depending on the presence of water*			**SRF**
O	Mild swelling rock pressure			5-10
P	Heavy swelling rock pressure			10-15

Fig. 24 Table for selection of SRF value in the Q-system (NGI 2022)

6.3.2 The support pressure, P_{arch}

The input values for the load in the dimensioning calculations include an equation for support pressure (Eq. 1) and a chart showing support pressure based on rock mass quality from the original Q-system publication by Barton, Lien and Lunde (1974). Both the chart and the equation are based on Terzaghi's support pressure criterion from 1946 (Terzaghi 1946), design support pressures for selected underground openings from 1972, and some measured data.

$$P_{arch} = \frac{2J_n^{\frac{1}{2}}Q^{-\frac{1}{3}}}{3J_r} \qquad (1)$$

In Grimstad et al. (2002), the net support pressure acting on a rib of sprayed concrete for a given Q-value is determined by subtracting a "basic" support pressure from P_{arch}. This net support pressure is the load that a rib of sprayed concrete must be dimensioned to withstand. The basic support pressure is set to 0.3 MPa and is intended to represent the support pressure for a Q-value of 0.3 when inserted into Eq. 1. The basic support pressure of 0.3 MPa is the same for all Q-values and is intended to represent the support pressure from bolts and sprayed concrete used for immediate support.

The support pressure in Eq. 1 is, as mentioned, based on Terzaghi's criterion. According to Høien (2024), this criterion is not suitable for covering the range of rock mass conditions, tunnel spans and overburdens that the Q-system claims to cover, as it is intended for shallow tunnels with active, gravity-controlled collapse. The data basis is also limited, and Høien (2024) argues that the scientific basis for Eq. 1 is not sufficient for prescribing support based on a support pressure.

Another serious error in the use of this equation is also pointed out in Høien (2024). A support pressure, as shown in Section 5, depends on deformation of the rock mass, the stiffness of the support, and the timing of support installation, with a resulting equilibrium between rock mass and support. P_{arch} does not take into account anything other than the ground. In addition, as described in Section 6.3.1, the Q-system has problems describing the stress conditions required to predict rock mass deformation through the SRF parameter. There is also a computational inconsistency, where Eq. 1, in addition to the total Q-value, requires input of the individual Q-parameters J_r and J_n.

6.3.3 Lack of interaction between support and rock mass

The modelling of ribs of sprayed concrete in Grimstad et al. (2002) was carried out using a program specialised for static analysis of concrete structures, called STAAD. The ribs of sprayed concrete in the STAAD model were represented as a free-standing arch fixed with springs at the base. The stiffness of the springs controlled the ability of the arch to move (rotate about the end point). The rock mass was not included in the model. Three spans were modelled: 5, 10 and 20 m, all with the same height. The load varied from 0.1 MN to 2 MN, where 0.88 MN is assumed to correspond to a vertical stress of 1.3 MPa (overburden of 50 m).

Since the STAAD model was a free-standing arch with applied forces, rather than a model in which the rock mass is included, the deformation of the modelled arch will depend only on the properties of concrete and steel. Furthermore, the stiffness of the springs at the ends of the arch, which governs the ability of the arch to rotate at this point, has a major influence on the deformation ability of the ribs of sprayed concrete. This spring-parameter is very difficult to determine, as the "reality" represented in the model differs significantly from actual conditions, and an unknown uncertainty is therefore introduced.

6.3.4 Acceptance limit for deformation

One of the main results presented in Grimstad et al. (2002) is a diagram showing the deformation of the arches for each span with different thicknesses and loads (see example in Fig. 25). The deformation occurs in the plane of the paper, downwards in the crown and outwards in the walls. An "acceptance limit" for arch deformation is introduced for each span. The acceptance limits are set to 1, 5 and 30 mm for spans of 5, 10 and 20 m, respectively, and are extremely small from a practical point of view, and must be regarded as highly theoretical. No references or justification are given for the determination of these limits, apart from the statement: "These acceptance limit lines are based on practical experience from various spans".

The calculation of the spacing between the ribs is not described in the article. The "net support pressure" was probably used to determine the load that a given ribs-of-sprayed-concrete configuration can withstand. Empirical data may also have been used. The spacing varies from 1 m to 4 m. For practical purposes, 1 m means that the ribs of sprayed concrete is continuous, and 4 m means that a 30 cm thick construction with three reinforcing bars is intended to support the rock mass 2 m on each side.

The "acceptance limit" for allowable deformation in the rib of sprayed concrete controls to a very large extent the resulting dimensions, and appears to be used to fine-tune the dimensions to reasonable values. As mentioned above, the limits are defined "based on practical experience", which means that the dimensioning of the arches is in practice based on undocumented engineering judgement.

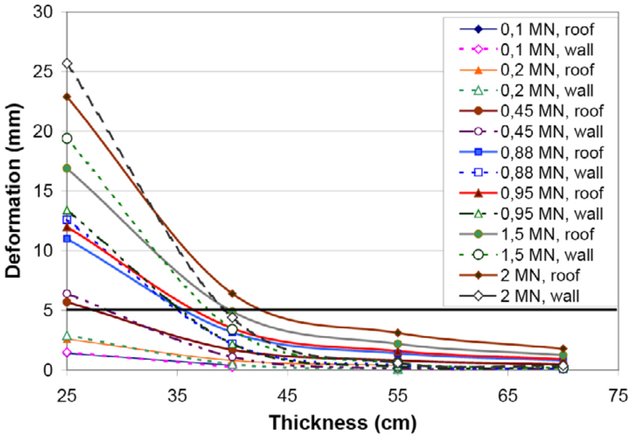

Fig. 25 Example of a deformation/arch thickness graph for a tunnel span of 10 m from Grimstad et al. (2002). The black horizontal line shows the “acceptance limit” for deformation of the rib of sprayed concrete at a span of 10 m.

6.4 Specific principles and practices to be aware of

The following subsections address key principles and practices that require particular attention in the dimensioning and assessment of rock support. Both challenges related to material properties, modelling methods, and empirical approaches are discussed, as well as uncertainties that may arise in different situations. The purpose is to highlight how various factors influence stability and the selection of support, and to provide an overview of relevant issues that should be considered in order to achieve robust and safe solutions.

6.4.1 Mineral swelling in weakness zones and gauge material

In Høien (2022), mineral swelling and its relationship to stability problems are discussed. It is pointed out that no causal relationship has been demonstrated between material exhibiting swelling properties in oedeometer tests on sieved and dried weakness zone material and an increased occurrence of stability problems in zones containing this type of clay material. It has also not been shown that the clay actually swells due to mineralogical swelling in nature.

Furthermore, it is concluded that: “It appears that the mineral swelling has already occurred in nature and that other time-dependent deformations, such as consolidation, mechanical swelling, creep, and squeezing, are likely the dominant deformation processes in weak zone material.”, and further: “This implies one should refrain from swelling pressure testing on dry clay powder, and that focus should instead be placed on other assessments when determining the extent of support. Swelling pressure testing may give a ‘false sense of security’, while more critical stability problems, which may later lead to collapse, are overlooked. In addition, these tests are characterised by significant uncertainty and have limited practical applicability.”

The essence of the citations above is that it is not possible to assess whether a weakness zone containing swelling clay minerals or not leads to more or fewer stability problems, and that it is most likely other properties of the clay and the surrounding conditions (tunnel geometry, width of the weakness zone, and rock stresses, etc.) that are the main causes of stability problems.

There is also no established relationship for how rock masses containing these swelling minerals should be supported compared to rock masses of similar quality that do not contain them. The values obtained are subject to very large uncertainty and depend to a large extent on how the sample is prepared prior to testing. In other words, the tests are unreliable, have no practical application, and most likely indicate an effect that is not observed in nature. Such an effect, even if present, would in addition be impossible to distinguish from more likely deformation and failure mechanisms.

In the discussion of swelling in the international literature, the emphasis is on swelling occurring due to stress relief, negative pore pressure (mechanical swelling), and access to water, rather than on material that swells and exerts pressure on the support on its own (see, for example, Einstein (1996)). The theoretical basis for this issue is discussed further in Part 2, Section 8.1.2.

Clay-filled joints and weakness zone material cause stability problems regardless of whether they contain swelling minerals or not, due to their low compressive strength and low shear strength. The presence of water aggravates these problems, particularly if the material is not sealed with sprayed concrete, as increased water content may further reduce strength and shear strength. This is especially critical in water tunnels where water may enter the rock mass over time and change material properties and stability conditions.

6.4.2 Portals/tunnel intersections and multiplication of the Q-system parameter J_n

In the Q-system (Barton, Lien and Lunde 1974; NGI 2025), it is recommended that the joint set number, J_n, is multiplied by 2 and 3, at portals and tunnel intersections, respectively. This results in a reduction of the Q-value, which leads to an increased support extent. The relationship between Q-value and support recommendations is empirical, but experience data for rock support at portals and tunnel intersections based on this reduction factor does not appear to exist, at least not to a sufficient extent. Consequently, other methods for assessing stability and support requirements must be used. In addition, it is implicitly assumed that the number of joint sets (J_n) is the main cause of the stability problem, which is not necessarily the case.

6.4.3 The Q-system, deformations and support pressure

In Sections 6.3.1, 6.3.2 and Høien (2024), it has been argued that the Q-system cannot predict deformations to a sufficient degree and that the support pressure given by the formula P_{roof} (Barton, Lien and Lunde 1974) is neither logical nor provides reliable values. Consequently, the use of formulas and figures that use or show these relationships should be avoided when dimensioning rock support or assessing stability. This may, for example, involve determining or calculating input values for analytical or numerical methods.

6.4.4 The difference between fibre- and bar/mesh-reinforced sprayed concrete

Different types of reinforcement in sprayed concrete differ significantly in how they influence the capacity of the concrete/support structure. Fig. 26 shows the principles of reinforcement under overloading of fibre-reinforced and bar/mesh-reinforced sprayed concrete, where fibre-reinforced concrete depends on the friction between the fibres and the concrete, whereas bar/mesh-reinforced concrete depends on the tensile strength of the steel.

This difference is important when assessing which type of support to use for different stability problems, so that the behaviour of the reinforced structure is adapted to the stability problem.

Another point concerns numerical modelling, particularly in the presentation of results in capacity plots with safety factors. Here, there are difficulties to implement fibre-reinforced sprayed concrete in a way that satisfies the assumptions in the calculations, even if relevant values are used and the thickness is such that load-bearing behaviour can be expected. Fibre-reinforced sprayed concrete is often used in areas where the main effect is to bind fragments and smaller blocks together and form wedges in joint corners, which numerical models are also unable to represent satisfactorily.

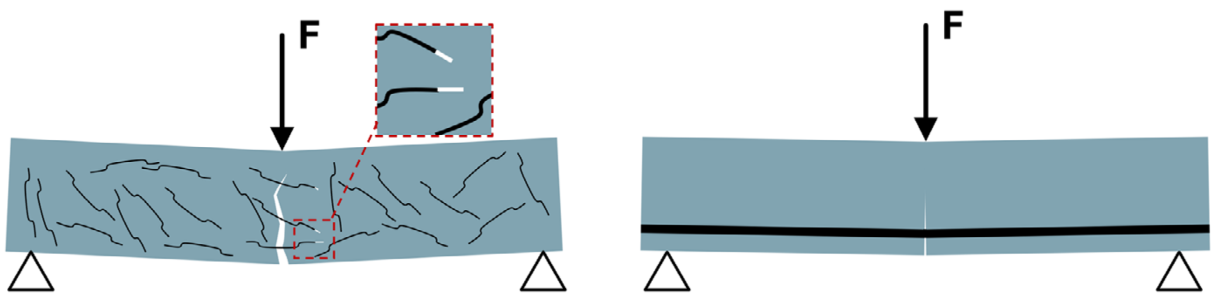

Fig. 26 The difference of fibre reinforcement and bar reinforcement in sprayed concrete. Under overloading of fibre reinforcement (left), the fibres slip within the concrete, and the strength is therefore dependent on the friction between the steel and the concrete. Under overloading of bar reinforcement (right), the strength is dependent on the tensile strength of the steel and, where relevant, the compressive strength of the concrete at crushing

6.4.5 Measurement of rock stresses

Rock stress measurements are used, among other things, as input in numerical modelling for the assessment of stability and rock support. It is essential to have knowledge of the uncertainties and limitations of the measurement methods when implementing them in the models, so that these can be accounted for in the evaluations carried out.

A compilation of results from stress measurements performed using the 3D overcoring method has shown that the borehole in which the measurements are taken appears to influence the results. The data are presented in Table 1 and graphically in Fig. 27. The table shows 19 measurements carried out at 12 different locations/projects using the same type of measuring cell.

In the rightmost column, the direction of the major principal stress has been changed from being relative to north to instead being relative to the borehole direction. In the figure, the orange dashed lines indicate ±15° from the borehole direction, and 13 of the 19 measurements fall within this range, with one located just outside. Two pairs of measurements are indicated by red dashed lines. In these pairs, the measurements were carried out at 90° relative to each other, with a distance of approximately 100 m in one case and 400 m in the other.

The presentation shows a predominance of measurements where the major principal stress is oriented in the direction of the borehole. This suggests that the conversion from measured values to stresses and their orientations does not sufficiently compensate for the influence of the borehole. Four of the remaining five measurements lie perpendicular to the borehole direction, which may also indicate an influence from the borehole, as the principal stress directions are perpendicular to each other and the overall calculated stress field is oriented according to the borehole direction.

Since the measured principal stress directions appear to be influenced by the borehole, one may also assumed that the stress magnitudes are affected and therefore uncertain.

Table 1 Results from 3D overcoring stress measurements

Lokation	**Hole dir. [°]**	**Hole dip [°]**	**σ_1 [MPa]**	**σ_1 dir. [°]**	**σ_1 dip [°]**	**σ_1 dir. to hole [°]**
Location 1	21	5	29	213	37	192
Location 2	0	5	25	166	23	166
Location 3 - 1	0		24	169	3	169
Location 3 - 2	258		25	90	7	192
Location 4	32		17	213	51	181
Location 5	40	5	14	216	2	176
Location 6	69	4	32	154	20	85
Location 7	110	5	15	298	12	188
Location 8 -1	335	5	1	247	50	272
Location 8 -2	58	5	5	11	52	313
Location 8 -3	43	5	5	30	26	347
Location 8 -4	113	5	3	99	2	346
Location 9	185	6	18	263	71	78
Location 10	2	6	20	165	54	163
Location 11 -1	300		18	120	28	180
Location 11 -2	30		17	205	30	175
Location 11 -3	30		22	21	10	351
Location 12 -1	240	5	11	56	15	176
Location 12 -2	238	5	-4	327	65	89

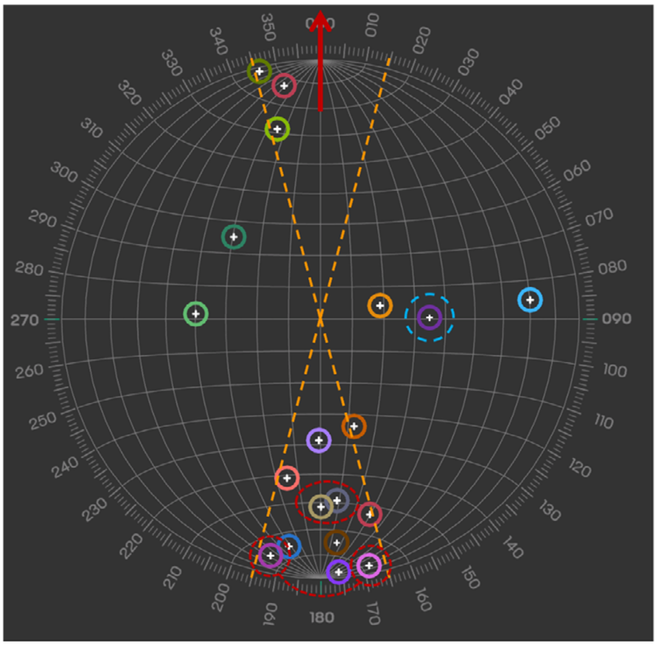

Fig. 27 Dip and dip direction for 3D rock stress measurements, where the measurements have been reoriented to be relative to the borehole direction (red arrow) and the lower hemisphere. The red dashed line mark measurements carried out close and at 90° relative to each other at the same location. The orange dashed line represents ±15° relative to the borehole direction. The blue dashed line indicates a negative major principal stress.

6.4.6 Numerical modelling, safety factor, and the limit state method

When assessing stability and using numerical models, it is important to use software that is capable of representing the relevant stability problem and the function of the rock support in a satisfactory manner.

Input parameters for numerical models have very large uncertainties, and when using a single model, one can never assume that the correct values have been obtained or that the behaviour will be as indicated by the results. However, parameter analyses can be carried out, in which various values are varied and an analysis is made of which conditions may lead to stability problems and how these may potentially be resolved.

However, even in parametric analyses, caution must be exercised when interpreting the results, and it must be ensured that the processes present in practice are also those being analysed in the model. Failure criteria, such as Mohr–Coulomb and Hoek–Brown, are elastic and plastic and therefore have limitations in analysing time-dependent deformations such as creep and squeezing occurring in e.g. weakness zone materials. Furthermore, as noted in Section 6.4.4, there is a difference in the stabilising effect of fibre-reinforced and bar-/mesh-reinforced sprayed concrete, where numerical models have difficulties in implementing the bonding and wedging effects of fibre-reinforced sprayed concrete. Attempting to calibrate possible correction factors into the values used in the model is also a double-edged sword, as each additional factor introduces further uncertainty. In many cases, an overall simple model will be preferable to one that attempts to account for all possible eventualities.

Caution should also be exercised in back-analyses, for example against deformation data. In general, measured deformations during tunnel excavation in hard rock will be small. It then becomes particularly difficult to identify the “correct” parameters that produce a given deformation, as there are countless possible combinations of input data that can make the model match the observed deformation. It is also not certain that the stability problem in question produces deformations, either in practice or in the model, as in the case of detached wedges or parallel structures in hard rock.

A common way of presenting the level of stability in a model is to use capacity plots with safety factors, for example for a reinforced concrete structure (see, for example, Fig. 24 in Høien and Nilsen (2018)). Firstly, what is presented must be representative of the stability problem encountered in practice. Fractured rock mass is one case where rock bolts provide the overall stability and the sprayed concrete acts only as surface support, in which case the stability shown in the model will not be representative. Secondly, the plots illustrate the difficulties in assessing stability in models in relation to the principles for assessing stability using the limit state method required by Eurocode 0 (Standard Norge 2016).

In the limit state method, a distinction is made between *acceptable* and *unacceptable* behaviour, based on a certain probability of occurrence (Harrison 2020). The plots use safety factors and do not apply partial factors as required by the Eurocodes, which would in any case be difficult to implement. More generally, considering all the limitations described above, including the uncertainty in input parameters, how these should be evaluated in combination with the weaknesses of the models, and the ambiguity regarding what constitutes *acceptable* behaviour, it is almost impossible to rely on numerical modelling alone. It is therefore very difficult to determine whether a support design is acceptable based solely on such models, and they must form part of a broader assessment that makes use of several relevant evaluation methods for the problem at hand.

Part 2 – Theoretical background and principles for design using the observational method for tunnelling in hard rock

7 Introduction

The principle of the new rock support strategy is to design the stability measures based on the behaviour of the ground. The core of this behaviour lies in how the rock mass fails and deforms, and a key point is therefore which types of failure and deformation processes govern this behaviour. When designing the stability measures, they must counteract or manage these processes. The interaction between the ground and the rock support, the system behaviour, will constitute a central principle in both design and execution. This implies that the rock support has an expected behaviour that can be observed to verify that it performs as intended, and that it is supplemented with additional stability measures if necessary.

This part therefore begins with a review of relevant failure and deformation types. Subsequently, published literature on various classifications of ground behaviour is summarised, which is used as a basis for determining suitable ground behaviour types for hard rock tunnelling. These categories are grouped according to three types of stress conditions and are related to ground types, failure and deformation processes, as well as proposed corresponding stability measures. The systematics presented are adapted to tunnelling in regions with hard rock; however, given the diversity and complexity of geology, it must still be recognised that weak rock types may also occur in such regions. The delimitation of the system in such cases is not specifically defined, as many factors are involved, but the boundary cases will, as a general rule, fall within the later defined GBT "Squeezing, creep and/or consolidation/mechanical swelling".

A key point that is emphasised in later sections is that there are situations that may be encountered which there are an absence of reliable methods for the design of rock support and for evaluating the actual stability of the supported profile. Awareness of this uncertainty is important, as it enables compensatory measures through increased design effort in such cases. This in contrast to a practice of uncritically applying empirical methods in ground conditions outside the range where they perform well.

In the following chapters some terms are used which is defined as follows:

- **Rock type**
  The intact rock consisting of different minerals.
- **Rock mass**
  Consists of the different rock materials in an area and includes joints, fractures, and weakness zones.
- **Ground**
  The rock mass including in-situ stresses and stresses from water.
- **Ground type (GT)**
  Ground with similar properties.
- **Ground Behaviour Type (GBT)**
  The response of the ground to tunnel excavation without stabilising measures.
- **System behaviour (SB)**
  The behaviour resulting from the interaction between the ground, the excavation (removal of material), and the rock support. SB is divided into three:
  - SB (1) excavation area/face
  - SB (2) supported area
  - SB (3) permanent condition (entire service life)

Furthermore, the term **rock support** is used as to include both **rock reinforcement** and **load-bearing rock support**, see Section 5. The term *rock reinforcement* includes support using bolts and sprayed concrete for both local and global stability problems. It also includes the use of reinforcement mesh and bar-reinforced sprayed concrete, including spiling fixed with rock straps and sprayed concrete, used to reinforce the surface such that the rock remains in place and local deformation is prevented. The term *load-bearing rock support*, is used for load-bearing support systems that interact with the ground and balance the forces between them (equilibrium) to create a stable condition.

8 Theoretical background – failure types and deformations

During tunnel excavation, two main types of deformations may be encountered: immediate and time-dependent (Barla 1999). The immediate deformations, together with the stresses, depend on the elastic and plastic material properties of the rock mass (see Fig. 28). In clay-bearing weakness zone materials saturated with water, immediate and time-dependent deformations are associated with an undrained and a drained phase, respectively. The immediate deformations occur without or prior to movement of pore water caused by pore pressure changes, whereas the drained, or time-dependent, deformations occur with movement of pore water. Purely drained and undrained behaviour rarely occur in practice, but since they represent fundamental types, they are nevertheless valuable in the interpretation and understanding of the actual behaviour (Terzaghi and Peck 1967). For further information on the following topics, reference is made to Høien (2018), Høien et al. (2019), Høien, Nilsen and Olsson (2019b) and Høien (2022).

8.1 Failure and deformation types

8.1.1 Immediate deformation

Immediate deformation arises from stress changes in the remaining rock mass as a result of the removal of rock mass during tunnel excavation. See Fig. 28. The magnitude of the deformation depends on the in-situ rock stresses and the elastic and plastic properties of the rock mass. Elastic properties describe the reversible deformations of the rock mass prior to failure, whereas plastic properties describe the behaviour after the rock mass has yielded by stresses exceed its strength. Post-failure properties, i.e. the plastic properties, of a rock mass will generally have lower values than the elastic properties. For very weak rock masses, however, they may be similar to the elastic properties (Crowder and Bawden 2004).

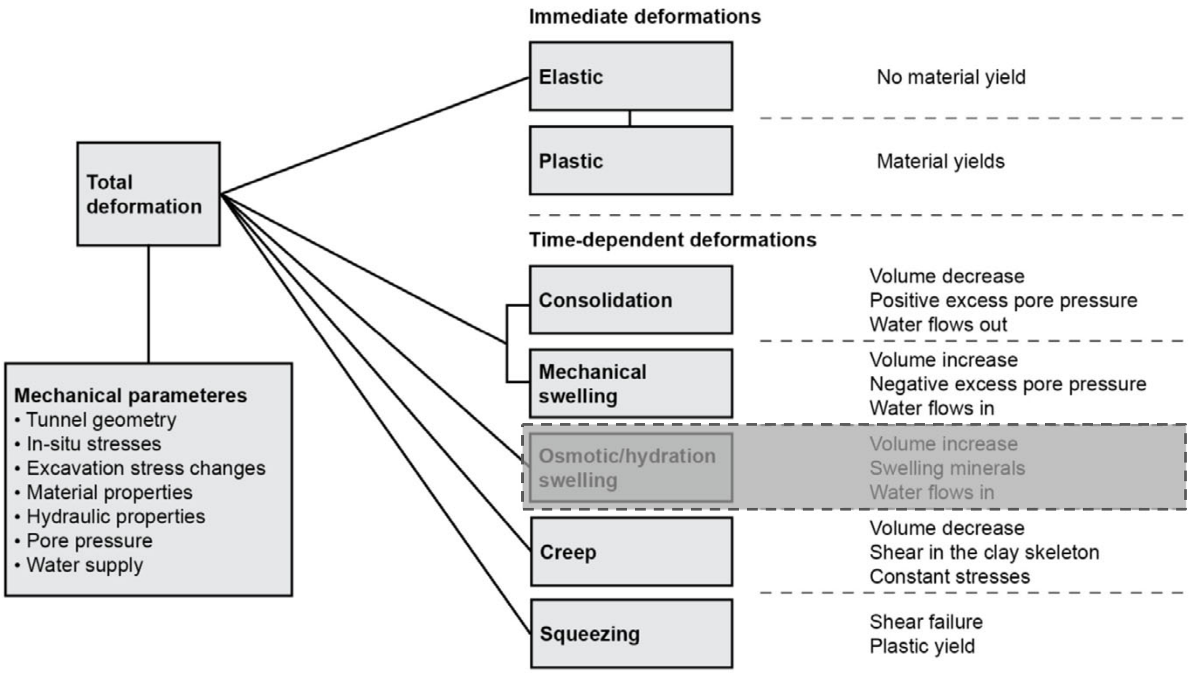

Fig. 28 Failure and deformation types in the rock mass. Due to lack of scientific evidence, mineral swelling (osmotic/hydration of minerals) is shaded in grey in the figure, see Section 6.4.1.Based on Høien (2018)

8.1.2 Time-dependent deformation

Time-dependent deformations are more complex than immediate deformation and may arise from several different processes. As shown in Fig. 28, the main processes are squeezing, creep, consolidation, and mechanical swelling, all of which lead to convergence of the tunnel walls. Which processes occur depends on the properties of the rock mass and the stress conditions, and more than one may occur simultaneously.

With regard to mechanical swelling and squeezing, Einstein (1996) pointed out that these phenomena are often associated with each other and that one may lead to the other. He also provided concise definitions of the two, where swelling is defined as "time-dependent volume increase of the ground, leading to inward movement of the tunnel perimeter", and squeezing as "time-dependent shearing of the ground, leading to inward movement of the tunnel perimeter”. Furthermore, Einstein (1996) states that swelling can only occur after the material has experienced stress relief. Squeezing is a result of a combination of stress changes and loss of confinement due to the removal of rock mass. Certain weak rock types, such as soapstone, may also exhibit squeezing behaviour in that they can undergo time-dependent plastic failure and deformation when overloaded.

As the tunnel advances, stress changes occur which, in weak materials with pore water present, may result in either positive or negative changes in pore pressure. In the case of a positive pore pressure change, water will flow away from the area, and the weak material will consolidate. Conversely, if a negative pore pressure change occurs, water may flow towards the area, which may lead to mechanical swelling. These processes are also referred to as the first consolidation phase (Bellwald 1990) and are illustrated in Fig. 29. Considering the pore pressure changes in Fig. 29 c), the red areas may experience outward flow of water and consolidation. In the blue areas with negative pore pressure, the water content may increase, and Einstein (1996) shows that failure in the material may occur as a result of increased water content by reduction of strength properties.

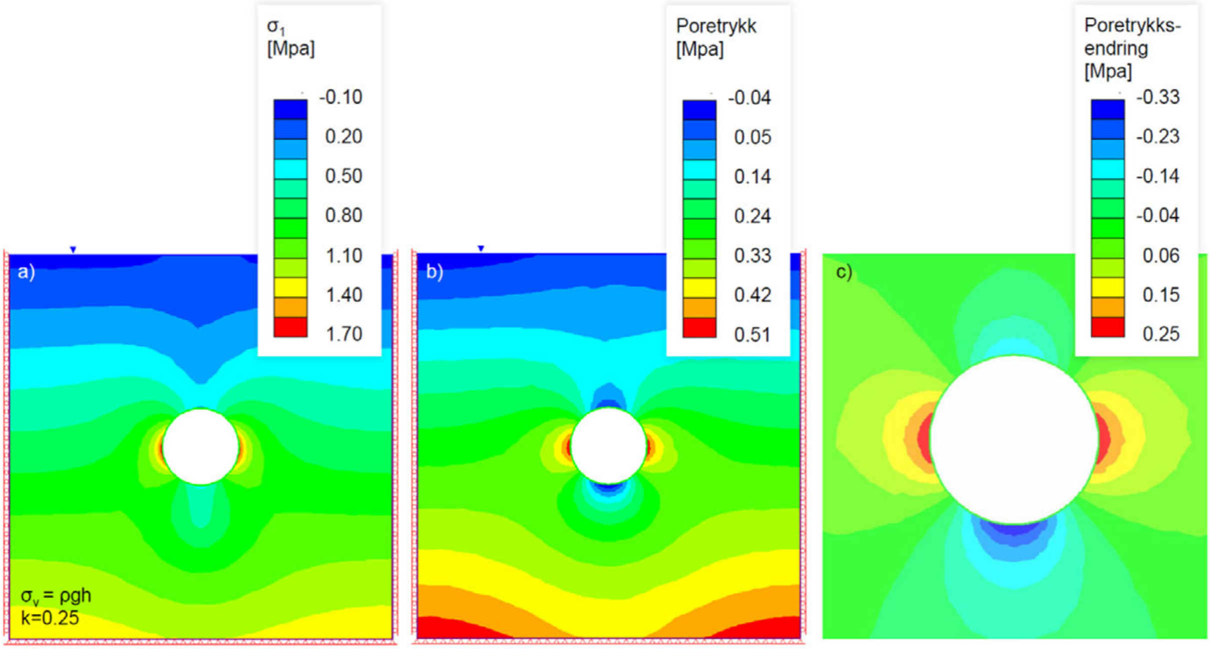

Fig. 29 Stresses (a), pore pressure (b) and pore pressure changes (c) resulting from tunnel excavation for a tunnel with a diameter of 10 m in clay-bearing materials (Høien 2022)

Creep is defined as deformation over time involving shear failure under constant stresses. It may occur simultaneously with mechanical swelling and consolidation, but this is not necessarily the case, and it is also referred to as the second consolidation phase. Creep depends on the time-dependent properties of the grain skeleton and involves a progressive reorganisation of grain contacts. It typically occurs under drained conditions, where pore water is able to move.

During time-dependent deformation, there will be a continuous redistribution of stresses around the tunnel profile. The new stress regime may initiate new deformation processes in areas that were previously stable. As described above, the processes causing these deformations are difficult to determine, and more than one process may occur simultaneously. Different types of deformation may also result in different loading of the rock support. For example, squeezing in a narrow zone crossing the tunnel may generate outward pressure on the support, whereas creep may produce compressive forces along the tunnel axis.

In Norway, deformation associated with the presence of clay minerals in the smectite group in weakness zones has been considered a contributing factor to stability problems, based on the assumption that these minerals produce a swelling pressure due to water uptake (see also Section 6.4.1). A review of the literature on this topic (Høien 2022) show that it is not provided a basis for establishing a causal relationship between the presence of swelling minerals in weakness zone materials and stability problems. Such a relationship was proposed as early as the 1960s but has neither then, nor subsequently, been supported using methods of sufficient scientific quality. What has been shown is that a swelling pressure from clay minerals extracted from weakness zone materials can be induced in laboratory tests by manipulating the material and perform testing in conditions that are unrealistic from a practical perspective. Arguably, these tests do not demonstrate that such swelling actually occurs in practice in tunnels. According to Høien (2022), it is most likely that intracrystalline swelling processes are responsible for the swelling pressures observed in laboratory tests, and that these have already taken place *in-situ* due to naturally occurring moisture. Based on what has been described and recommended as support for weakness zones with swelling clays, reported among others in NFF (2008), it is most likely that what has been interpreted as mineral swelling is in reality a deformation process dominated by squeezing.

For swelling rock types, the deformation processes differ from those described above, even though some of them also contain swelling minerals from the smectite group. Swelling mechanisms in such rocks may also occur due to the transformation of anhydrite to gypsum or the oxidation of pyrite producing sulphates (Einstein 1996). These rock types may also be weaker than the surrounding rock and are therefore more susceptible to other types of failure than swelling.

8.2 Deformasjon og spenninger

To illustrate, at a conceptual level, how overburden and rock mass quality influence deformation in a twin-tube tunnel, Høien, Nilsen and Olsson (2019a) carried out two parametric studies using the FEM software RS2 and RS3. Such general parametric studies are naturally strongly influenced by the limitations and choices made in the development of the applied method. The absolute deformation values must therefore be regarded as an indication of trends and general principles rather than as representative of actual behaviour in a tunnel project.

The first parametric study was two-dimensional (RS2) and examined how deformations vary with different rock mass qualities (infinite extent) and overburden. The second study was three-dimensional and investigated two of the cases from the first study, assessing how deformation develops when the weak rock type is located in a zone of varying width.

Parameters representing a relatively strong rock type were used as a basis, with intact compressive strength σ_{ci} = 125 MPa and intact elastic modulus Ei = 20 GPa. These were adjusted for GSI values between 80 and 10, in intervals of 10, using the Generalized Hoek–Diederichs (2006) approach. Vertical stresses were based on the weight of the rock mass and the overburden. Horizontal stresses were four times higher than the vertical stresses at the surface and became equal at a depth of approximately 300 m. An example of results from RS2 is shown in Fig. 30, with a model section corresponding to 500 m overburden and GSI 20.

The analyses were performed using an ideal elastic–plastic material, meaning that the material undergoes failure but retains the same deformation properties before and after failure. In relation to the different deformation types described in the previous sections, this in principle represents only immediate deformation. By definition, squeezing is a plastic failure process, but it is uncertain in what extent squeezing is represented in an FEM model using an elastic–plastic failure criterion.

Fig. 31 shows the results from the parametric study. The values represent the maximum deformation of the tunnel contour, in mm, in the model. The colours in the figure represent different classes defined by, Hoek (2001) (see also Hoek and Marinos (2000)) where:

- A The green area represents < 1% deformation, no squeezing, with sprayed concrete and rock bolts described as rock support
- B The orange area represents 1–2.5% deformation, minor squeezing, where light steel sets may be required
- C The red area represents 2.5–5% deformation, significant squeezing, where steel ribs are recommended and face support must be initiated
- D The purple area represents 5–10% deformation, very severe squeezing, where forepoling, face support, and steel ribs are typically required

As can be seen from the proposed stability measures above, the distinction between rock reinforcement and load-bearing rock support lies at the boundary between classes A and B. Both the results from the parametric study shown in Fig. 31 and the squeezing classes represent unsupported rock mass.

In Norwegian conditions, what is typically referred to as weakness zone material corresponds to a GSI value of approximately 30–40 and lower. The results from the parametric study show that even very weak rock masses may be self-supporting when bolts and sprayed concrete are used for reinforcement, provided that the stresses are not too high.

Another key observation from this study is that large deformations at the tunnel contour are associated with significant deformations extending far into the surrounding rock mass. For the case with 500 m overburden and GSI 20 (see Fig. 30), deformation at the tunnel contour is 170 mm, while deformation in the rock mass 50 m above the tunnel is 40 mm. This indicates that the weak material must have a considerable extent in order to produce large deformations. This is further illustrated in the parametric study evaluating zone width, presented later in this section, where a significant reduction in deformation is observed when the extent of the weak material is reduced from infinite extent in the plane of the 2D study to limited extent in the 3D study.

Basarir (2008) conducted a similar study based on the Rock Mass Rating (RMR) classification system for a circular tunnel with overburden ranging from 100 to 500 m, investigating how different levels of support influence the deformations. The results from this study are generally comparable to those shown in Fig. 31.

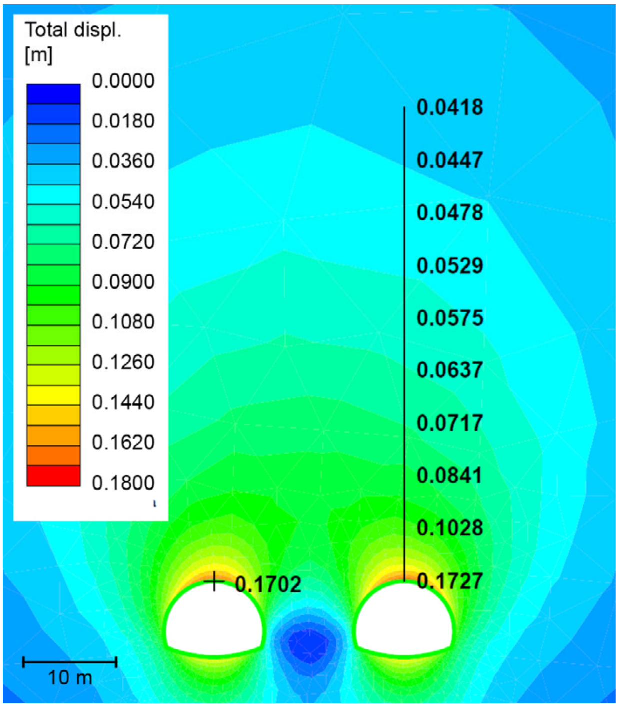

Fig. 30 Result plot from the two-dimensional parametric study. The plot shows the model corresponding to 500 m overburden and GSI 20. Of particular interest is the extent of the deformations, and thus the distance over which the ground influences the deformations in the tunnel (Høien, Nilsen and Olsson 2019a)

		GSI							
		80	70	60	50	40	30	20	10
Overburden [m]	35	2	2	3	6	11	23	43	74
	100	2	2	3	6	11	22	40	69
	200	3	3	5	8	15	31	60	106
	300	4	5	7	12	23	48	92	170
	400	6	7	9	16	32	66	131	249
	500	7	8	12	21	40	85	170	340
	600	8	10	14	25	50	106	227	451
	700	10	12	17	30	60	130	271	585
	800	11	13	19	34	69	148	315	696
	900	13	15	22	39	80	174	370	857
	1000	14	17	25	43	90	200	422	1030

Fig. 31 Findings from the parametric study indicate that even very weak rock masses may be self-supporting using bolts and sprayed concrete for reinforcement, as long as the stresses are not large (Høien, Nilsen and Olsson 2019a)

As noted above, Høien, Nilsen and Olsson (2019a) also examined how the width of a weakness zone affects deformation. The 3D models used were 150 m in tunnel length and 500 m wide, with two different overburdens of 100 m and 500 m. In the centre of the models, a material with weaker rock mass properties than the surrounding rock mass was placed to represent a weakness zone. The zone was oriented perpendicular to the tunnel axis, and six different models were created with zone widths of 50, 25, 10, 5, 2.5, and 1 m. An example of the model is shown in Fig. 32 a). Material parameters were the same as in the 2D study, where the surrounding rock had a GSI of 80 and the weakness zone had a GSI of 20.

To illustrate the distribution of deformations, a profile section through the centre of the zone and a longitudinal section along one tunnel tube are shown for 100 m overburden and 25 m zone width in Fig. 32 b) and c). The results (see Fig. 33 and Fig. 34) demonstrate that the deformations are strongly influenced by the zone width. Measurements were taken high in the left wall (maximum deformation) for the model with 100 m overburden, and at the crown for the model with 500 m overburden. The figures indicate that zone width has a very significant influence on the magnitude of deformation that may be expected, and thus on potential stability problems.

Andresen (2022) performed a parametric analysis using approximately 4000 models in RS2 to investigate how zone width, location within the cross-section, and overburden affect deformation for a longitudinal weakness zone. An important finding from this study was that the largest deformations occurred when the zone was tangential to the tunnel wall, and that it influenced the deformation also when located outside the tunnel. Longitudinal weakness zones may therefore affect tunnel stability before they become visible within the tunnel opening. Such zones may have a substantially greater influence, as they do not benefit from lateral confinement in the same way as transverse zones. Deformations may therefore develop in larger areas in the rock mass.

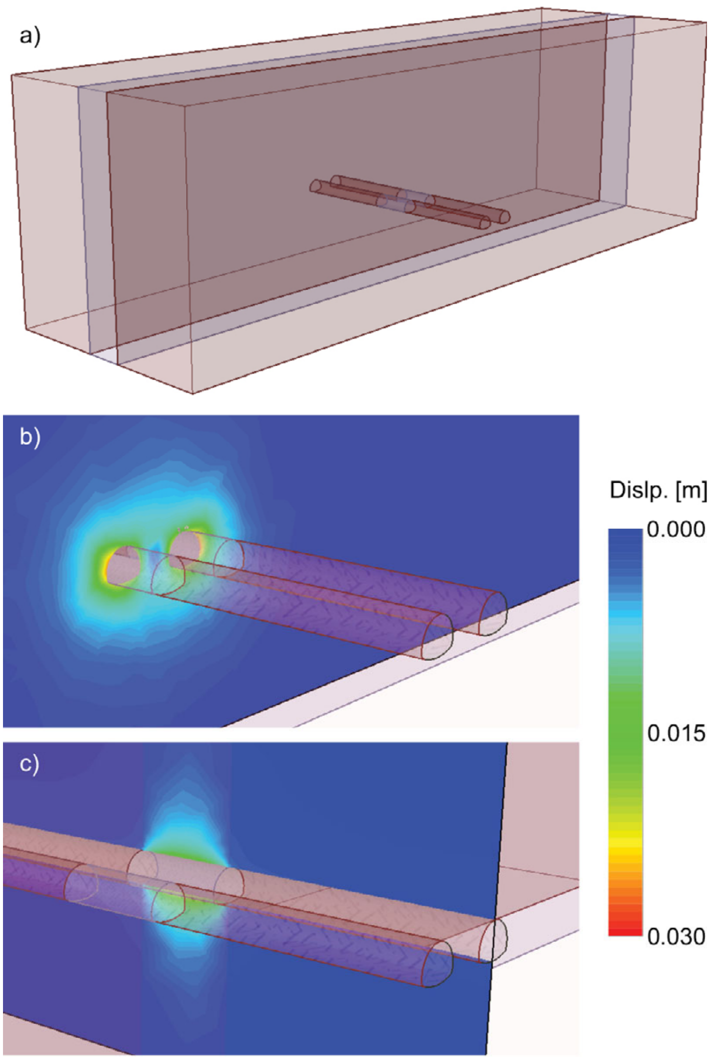

Fig. 32 3D model with an overburden of 100 m and a weakness zone width of 25 m. a) Model overview, b) profile section at the centre of the zone, and c) longitudinal section at the centre of the right tunnel tube (Høien, Nilsen and Olsson 2019a)

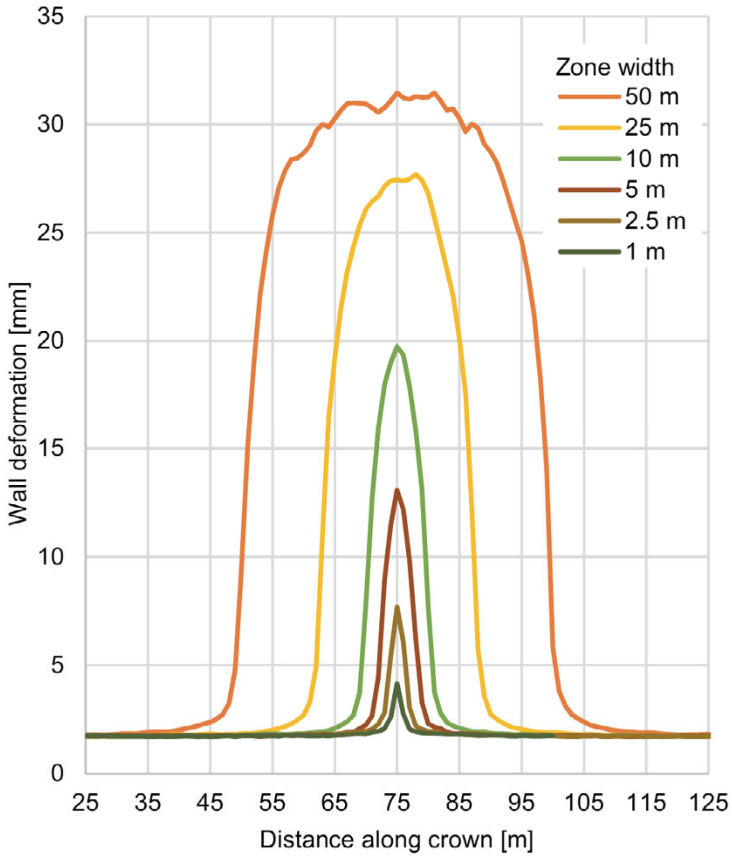

Fig. 33 Maximum deformation along a 150 m long tunnel section with different zone widths, with an overburden of 100 m. The centre of the zone is at 75 m (Høien, Nilsen and Olsson 2019a)

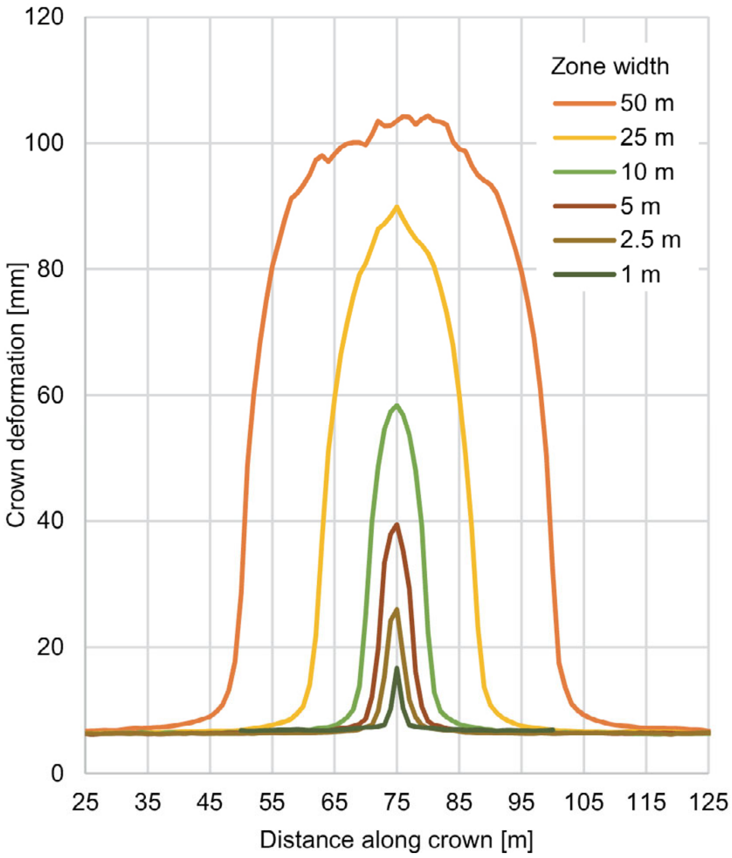

Fig. 34 Maximum deformation along a 150 m long tunnel section with different zone widths, with an overburden of 500 m. The centre of the zone is at 75 m.(Høien, Nilsen and Olsson 2019a)

8.3 Brittle failure and brittle shear failure

Brittle failure is primarily used as a term for the initiation and propagation of new cracks in hard rock. Further can *brittle shear failure* be used to describe movements along existing, only slightly altered joints or, for example, within foliation layers. During tunnel excavation in hard rock, different types of brittle failure and brittle shear failure occur depending on the geometry of the joints, the degree of fracturing, and the stress conditions. In Martin, Kaiser and McCreath (1999) and Kaiser et al. (2000), failure types for excavation in hard rock are categorized into nine classes, as shown in Fig. 35, based on the RMR and the ratio between the major principal stress and the uniaxial compressive strength. As can be seen in the figure, shear failure at low stress is typically associated with block movements, whereas brittle failure in competent rock occurs at higher rock stresses. In general, moderate rock stresses will increase stability along joint planes, particularly due to increased normal stress on the joint plane, but depending on the geometry of the joints and the rock stresses, they can also create unstable zones by increasing the shear stress on the joint plane.

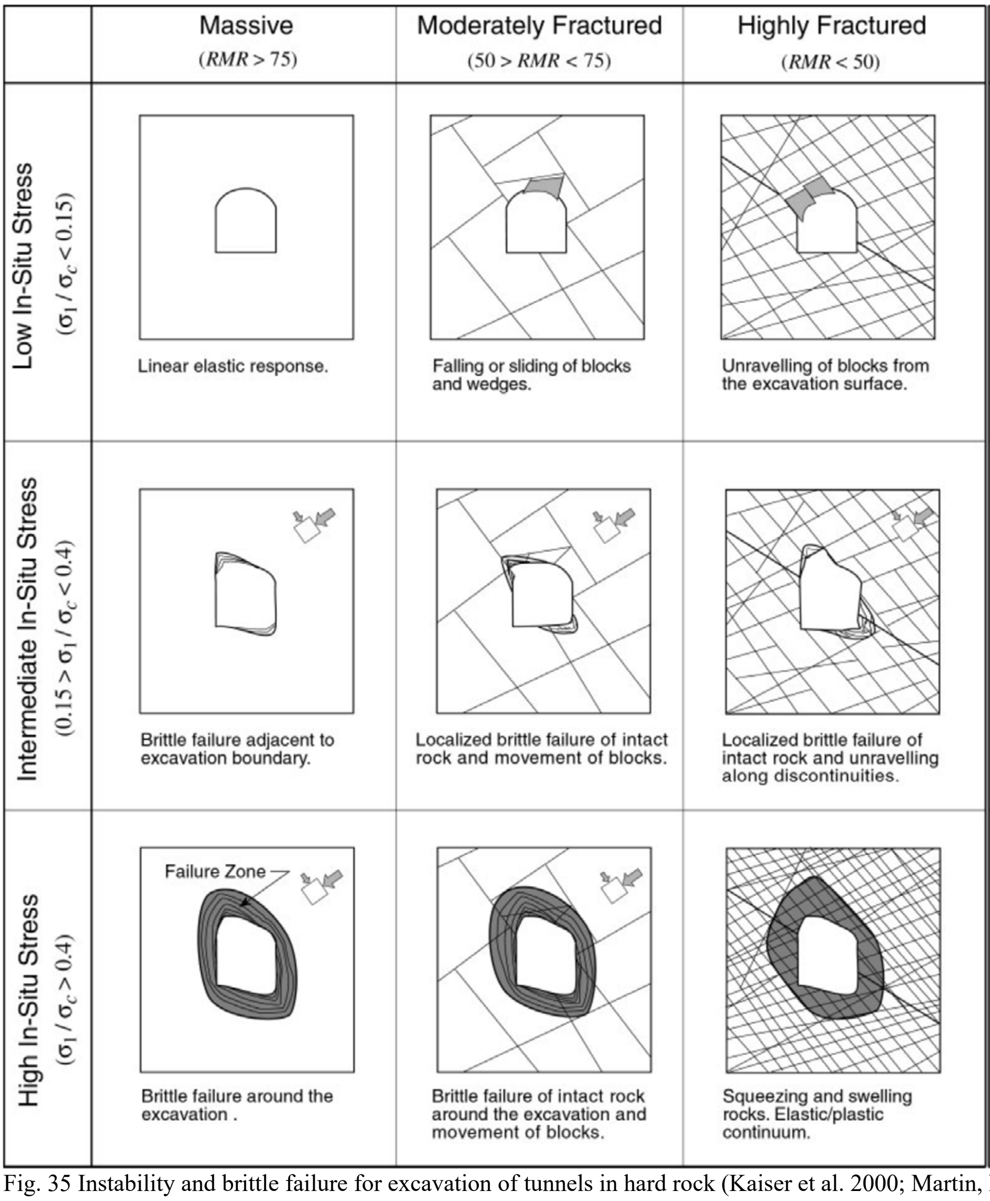

Fig. 35 Instability and brittle failure for excavation of tunnels in hard rock (Kaiser et al. 2000; Martin, Kaiser and McCreath 1999)

If the stresses become large compared to the strength of the rock mass, this may lead to brittle failure in the rock. This is commonly referred to as spalling and rock burst, depending on the severity. Excavation of tunnels under such ground conditions places high demands on both design and execution, as the behaviour of the rock can be unpredictable. Diederichs (2025) reviews the historical development of theory for assessing spalling and rock burst, from early developments in the theory of brittle failure to how the extent can be evaluated in situ and how the phenomenon can be modelled numerically. Two important concepts that can be used to describe the risk and extent of spalling/rock burst are crack initiation (CI) and crack damage (CD). CI is the stress level at which microcracks begin to form in the rock, without the rock losing its load-bearing capacity, and typically occurs at 40–50% of the uniaxial compressive strength. CD is the point at which the cracks coalesce and result in irreversible damage and reduced stiffness in the rock by propagating beyond grain boundaries, interacting with each other, and developing into visible cracks. This occurs at 70–80% of the uniaxial compressive strength (Diederichs 2025).

Diederichs (2025) presents types of stability problems resulting from brittle failure, and Fig. 36 shows an S-shaped curve based on CI and CD under different stress conditions. At high confining pressure, crack growth is restrained and damage at the grain scale accumulates such that shear failure may occur. Close to the tunnel, where the confining stress is low, cracks propagate immediately after initiation at CI. This results in the rock mass strength being reduced to the CI level near the tunnel. In the transition zone between high and low confining stress, the two failure mechanisms are linked through a spalling limit, where crack growth is partially suppressed. Based on the same parameters, empirical relationships have also been developed to assess the depth of spalling in more detail (Diederichs 2025; Diederichs, Kaiser and Eberhardt 2004; Martin, Kaiser and McCreath 1999). See also Dammyr (2016) for a case study of brittle failure in Norwegian rock.

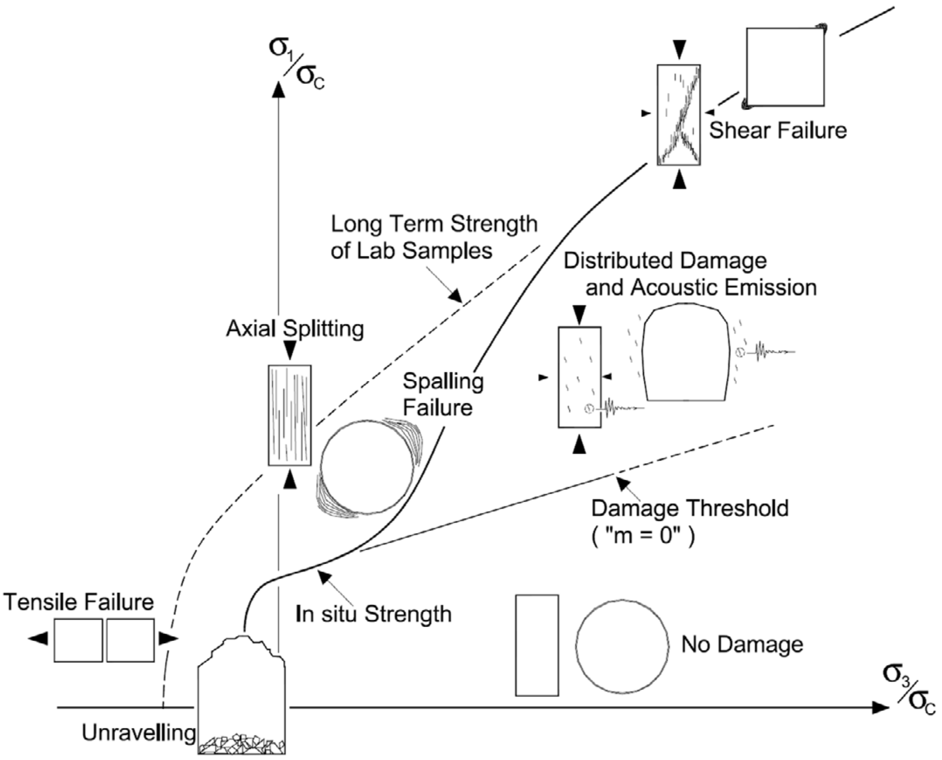

Fig. 36 Schematic failure envelope for brittle failure showing four zones with different mechanisms: no damage, shear failure, spalling and fall-out. σ_c denotes the uniaxial compressive strength measured on laboratory specimens (Diederichs 2025; Diederichs 2003)

9 Existing categorisation of rock mass and ground behaviour

Empirical methods for rock mass classification such as the Q-system and RMR are based on classifying the mapped rock mass into different classes with an associated proposal for rock support. A limitation of this approach is that stability problems caused by very different mechanisms may receive similar rock support recommendations. An example from the Q-system is that both overstressed intact rock, spalling/rockburst, and weakness zones may fall within the same support category (Høien 2024), and another example from RMR is that wedge failure along smooth joints and a progressing failure/fall-out with the presence of groundwater give similar rock support recommendations (Marinos 2014).

A different approach rather than classifying the rock mass is to categorise according to different characteristic types of ground behaviour (Ground Behaviour Types, GBT), and for each of these to have a method for assessing stability and designing stabilization measures. According to Palmström (1995), a rock mass including in-situ stresses and stresses from groundwater can be referred to as *ground*, and furthermore, the ground's response to tunnel excavation is defined as *ground behaviour*. Ground behaviour depends on the strength of the rock mass, its deformation properties, the stresses acting on the rock mass, as well as the interaction between the geometry of the tunnel and the orientation and extent of geological structures.

GBT thus describes which type of stability problem is present if the ground is not stabilized. The advantage, compared with empirical methods and rock mass classification, is therefore that the rock support can to a greater extent be adapted to the specific stability problem at hand.

9.1 Categorisation of GBT based on Central and Southern European practice

Within the New Austrian Tunnelling Method (NATM), a classification into eleven "General categories of Ground Behaviours" with associated descriptions of failure mechanisms has been adopted, as shown in Table 2 (Austrian Society for Geomechanics 2010). A similar characterisation has been carried out by Marinos (2012), from the University of Thessaloniki, Greece, who proposed ten different "Tunnel Behaviour Types" as shown in Table 3 for rock types with an intact strength of approximately 15 MPa. Marinos' different types are further associated with "Rock Mass Structure", which is one of the two basic parameters in the GSI, and also rock stresses.

Table 2 Categories of behaviour types and associated failure mechanisms according to the Austrian Society for Geomechanics (2010), developed from Schubert et al. (2001)

Basic categories of Behaviour Types (BT)		Description of potential failure modes/mechanisms during excavation of the unsupported ground
1	Stable	Stable ground with the potential of small local gravity induced falling or sliding of blocks
2	Potential of discontinuity controlled block fall	Voluminous discontinuity controlled, gravity induced falling and sliding of blocks, occasional local shear failure on discontinuities
3	Shallow failure	Shallow stress induced failure in combination with discontinuity and gravity controlled failure
4	Voluminous stress induced failure	Stress induced failure involving large ground volumes and large deformation
5	Rock burst	Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy
6	Buckling	Buckling of rocks with a narrowly spaced discontinuity set, frequently associated with shear failure
7	Crown failure	Voluminous overbreaks in the crown with progressive shear failure
8	Ravelling ground	Ravelling of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion
9	Flowing ground	Flow of intensely fractured, poorly interlocked rocks or soil with high water content
10	Swelling ground	Time dependent volume increase of the ground caused by physical-chemical reaction of ground and water in combination with stress relief
11	Ground with frequently changing deformation characteristics	Combination of several behaviours with strong local variations of stresses and deformations over longer sections due to heterogeneous ground (i.e. in heterogeneous fault zones; block-in-matrix rock, tectonic melanges)

Table 3 Types of behaviour suggested by Marinos (2012)

	Abbr.	Tunnel Behaviour types
1	St	Stable ground
2	Br	Brittle failure
3	Wg	Wedge failure
4	Ch	Chimney type failure
5	Rv	Ravelling ground
6	Fl	Flowing ground
7	Sh	Shear failure
8	Sq	Squeezing ground
9	Sw	Swelling ground
10	San	Anisotropic strains

9.2 General categorisation of GBT

The methods mentioned in the chapter above are adapted for categorising rock in the "softer" rock types, internationally referred to as soft rock, which are commonly encountered in Central and Southern Europe. In Palmstrom and Stille (2007), the authors proposed a more detailed and comprehensive classification for all types of ground. In Fig. 37, their compilation of previous work on ground behaviour carried out by Hoek, Kaiser and Bawden (1997) and Martin, Kaiser and McCreath (1999) (see Fig. 35) is presented. Based on this, as well as Terzaghi (1946), Schubert et al. (2001), and their own experience, they identified different types of ground behaviour as presented in Appendix 1. Palmstrom and Stille (2007) divided the behaviour types into three groups: gravity-driven, stress-driven, and water-influenced, where the first two groups were proposed by Hoek and Brown (1980) og Hudson (1989), while the last was proposed by themselves.

In type 1, gravity-driven, the dominant mechanism is failure along discontinuities, where existing fragments or blocks in the crown and sidewalls are loosened and may fall or move as the tunnel advances. In type 2, stress-driven, failure is caused by local overstressing, where the stresses in the ground exceed the local strength of the material. These may occur in two main forms: (i) as buckling (see Fig. 37), spalling or rockburst in brittle, massive rock; and (ii) as plastic deformation, creep or squeezing, in continuous but deformable, soft/ductile rock, or in particulate materials such as heavily fractured rock or soil. For type 3, water-influenced, the main mechanisms are failure in the form of flowing ground in particulate materials exposed to large amounts of water, and unstable conditions caused by, for example, swelling and slaking (disintegration). Water may also influence block fall in type 1, as it can reduce the shear strength of joint surfaces, especially slickensides and joints with soft infilling or coatings.

Important factors controlling ground behaviour are whether the ground is continuous or discontinuous, the stresses to which it is subjected, and whether failure/deformation is instantaneous or time-dependent. In Fig. 38, the different GBTs from Appendix 1 are distributed according to these parameters. Such a classification may be useful, for example, in a pre-investigation phase when categorisation is required.

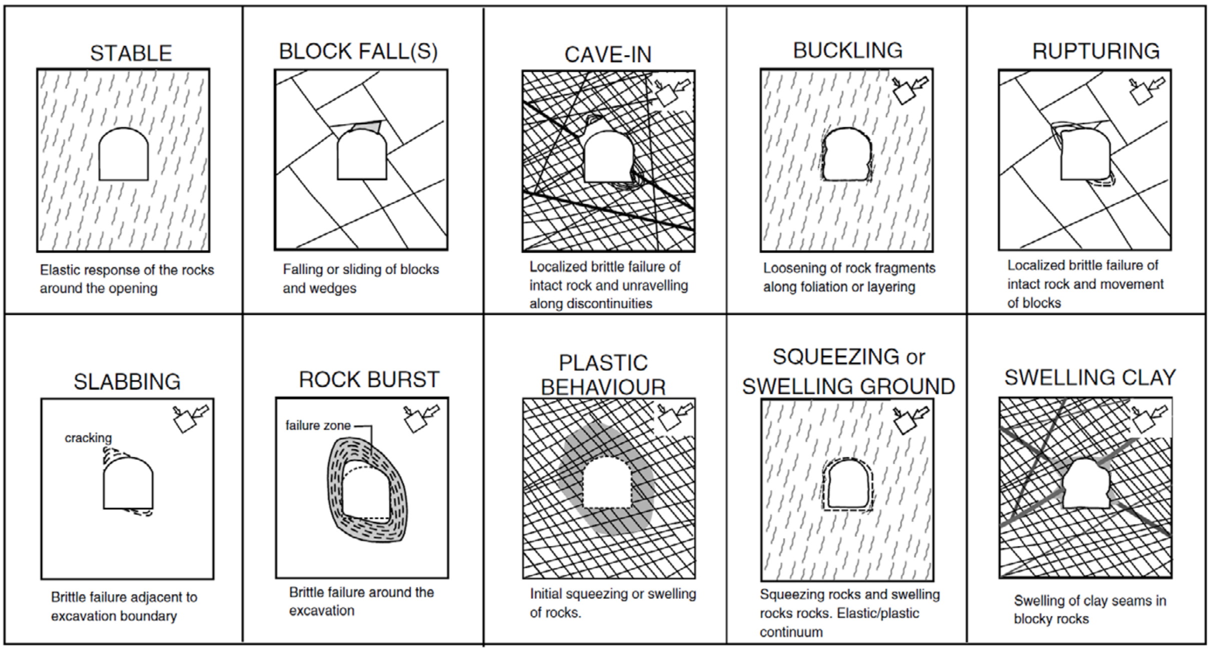

Fig. 37 Different ground behaviours suggested by Palmstrom and Stille (2007) based on Hoek, Kaiser and Bawden (1997) and Martin, Kaiser and McCreath (1999). (Palmstrom and Stille 2007)

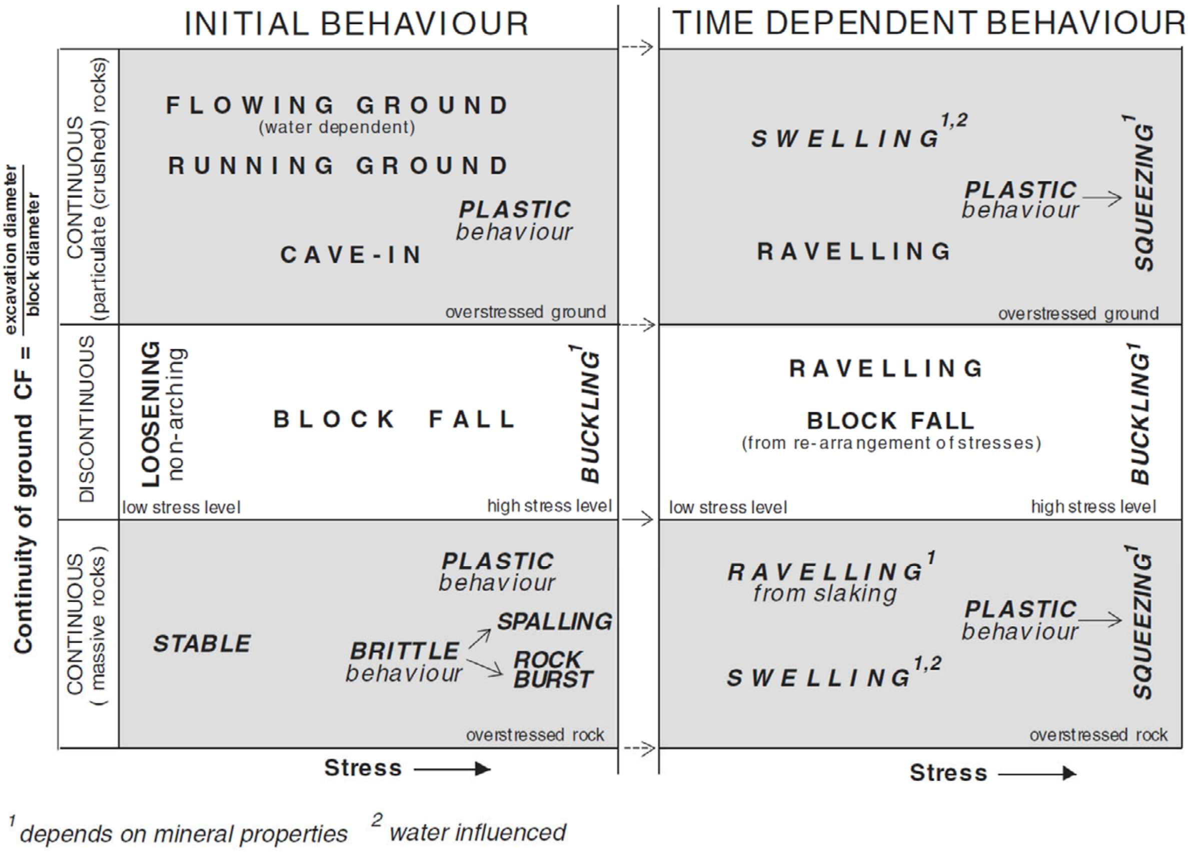

Fig. 38 Ground behaviour as described in Appendix 1 distributed by block size, stresses and time (Stille and Palmström 2008)

9.3 Categorisation of GBT based on Norwegian/Scandinavian conditions

As mentioned previously, it is typical for Norway and the rest of Scandinavia that one mainly encounters hard and massive rock types intersected by weaker zones resulting from weathering or tectonic activity, internationally referred to as *hard rock conditions*. Weak rock mass in this context refers to weakness zones, rock types with low strength, layers of rock with low strength, anisotropic structure, and fractured rock mass.

Terron-Almenara, Holter and Høien (2023) have specifically examined poor ground conditions excavating such rock mass. In this work, rock mass under moderate to low rock stresses was considered, and the classification into different GBTs is presented in Fig. 39, while the description of the different types by Terron-Almenara (2025) is presented in Table 4.

Each type of ground behaviour also has an associated proposed design procedure for rock support, which is shown in Appendix 2. This approach is referred to as the *hybrid method* and involves applying the most suitable rock engineering tools available for each type of ground behaviour. It starts by recommending how the site should be investigated, and further provides recommendations on how to apply empirical, analytical and numerical methods, how stability should be monitored/controlled, and finally overall recommendations for rock support. This approach thus corresponds well with what is described in Eurocode 7 (Standard Norge 2020) as the observational method.

For ground behaviour in fractured hard rock and rock stress-related problems, reference is made to Fig. 35 and the references in Section 8.3.

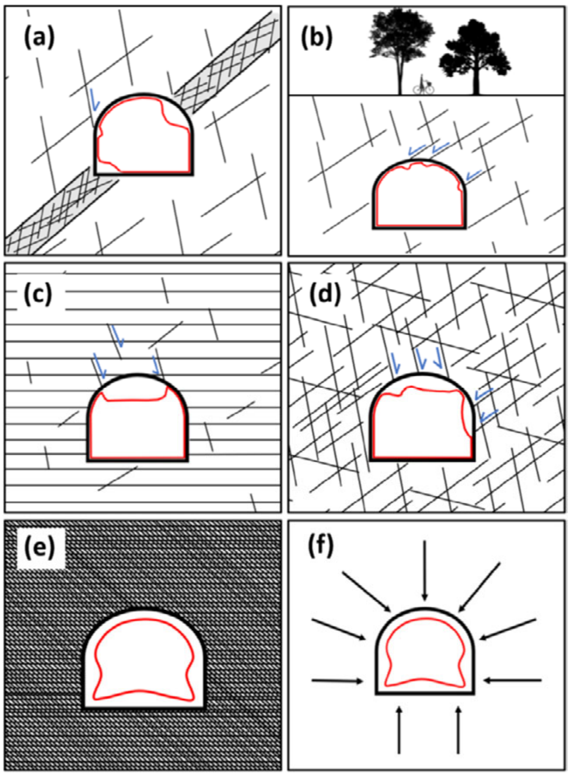

Fig. 39 Similar types of ground behaviour for poor ground in typical Scandinavian rock conditions after Terron-Almenara, Holter and Høien (2023). See explanation in Table 4

Table 4 Description of the types of rock mass behaviour given in Fig. 39. Proposal for different types of ground behaviour for poor ground in typical Scandinavian rock conditions. Quoted from Terron-Almenara (2025)

GBT	Rock mass condit.	Description
(a)	Distinct weakness zone	Distinct zone of width up to 1/2 x tunnel diameter, and intersecting the rock mass and the excavation. The zone can present fractured, weathered rock, possibly with alteration minerals and clay seams that can develop loosening, ravelling, swelling behaviors. The poor quality of the material of the zone with respect to the host, hard rock, can lead to local bulking and deformations, block movements and/or falls, and differential tunnel deformations.
(b)	Low rock stresses	Tunnels in hard rock subjected to very low in-situ stresses, for example at tunnel portals and/or tunnel crossings, or urban rock tunnelling. The ability of the rock mass to arch will control the stability and behavior of the tunnel. Such ability is controlled by the rock mass structure, the geometry and mechanical characteristics of the joints, and the degree of jointing. Typical failures are related to gravitative failure of roof blocks, resulting in point and uneven support loads.
(c)	Anisotropic rock mass structure	Layered and anisotropic hard rock mass. A combination of weak subhorizontal bedding planes and unfavorable in-situ rock stresses can lead to loosening of roof slabs and roof failure, leading to uneven loads of different magnitude and distribution as a function of the rock mass structure, tunnel span and stress confinement.
(d)	Hard and fractured rock mass	The stability of the opening is governed by the in-situ stresses, the degree of jointing, and the ability of the rock mass to arch. As a result, a variety of failure mechanisms can be triggered, from block fall and overbreak, to cave in, leading therefore to differential deformations and uneven loads.
(e)	Weak rock mass	Weak rock mass composed by weak rock material, intensely fractured -or soil-like- crushed and weathered rock, likely clay seams and subject to moderate in-situ rock stresses. This GBT could be represented by large fault zones. The magnitude and distribution of deformations can vary as a function of the in-situ stresses, the characteristics of the support system and the ground conditions. Upon uniform far-field stresses, deformations and loading are more even around the opening, compared to previous GBT described above.
(f)	Swelling ground	Swelling is produced by swelling minerals inside the rock matrix forming the rock mass, and not in joint infillings as gouge material. This GBT could be represented by, for example, volcanic rock masses altered hydrothermally with embedded minerals of the montmorillonite group, or rock masses presenting alum shales or anhydrites. Swelling behavior may require heavy support depending on swelling activity.

10 The observational method for hard rock tunnelling

The observational method in Eurocode 7 (Standard Norge 2020) is an approach used when there is uncertainty related to ground conditions or the behaviour of the structure, and it is difficult to determine all parameters with sufficient reliability prior to construction. The method is based on an assumed model of how the structure and the ground will behave. The main principles are that one:

- defines acceptable limits for the behaviour of the structure
- prepare a monitoring plan with methods and frequency
- have predefined measures that can be implemented rapidly if the limits are exceeded

The purpose is to verify that the actual behaviour corresponds to the assumptions made during design. If observations show deviations from the expected behaviour, the predefined measures shall be implemented to ensure stability and safety. The observational method thus provides flexibility and the possibility to adapt the structure during the construction period.

The overall principle in the Eurocodes for determining the level of structural safety is the concept of limit states described in Eurocode 0 (Standard Norge 2016). As mentioned in Section 4, there are two limit states: ultimate limit state and serviceability limit state. The concept is that the structure is designed such that the limit states are not exceeded. This is achieved by taking into account all relevant design situations that may affect the limit states during the design. For tunnels, design situations may, for example, include local stability, global stability, short-term and long-term stability, as well as failure in both the ground and the support structure.

To systematise the design situations related to the interaction between the ground and the tunnel, one can use the GBT as a starting point. Further, in order also to include the rock support in the design situations, the concept of “system behaviour” (SB) from NATM (Austrian Society for Geomechanics 2010) may be applied. This refers to the behaviour resulting from the interaction between the ground and the rock support as a consequence of the tunnel excavation. SB can be divided into three:

1) the excavation area
2) the supported area
3) the permanent condition

In the design, for each GBT that is relevant for an area of the project, acceptable limiting values, monitoring plans, and additional measures must be defined as described in the list above. SB may in this respect play a central role in fulfilling the fundamental principles of the observational method mentioned above, as the GBTs are thereby related to the excavation process. As an example, SB for the excavation area may indicate which stabilization measures are required ahead of the face; SB in the supported area indicate which limiting values must be defined and the monitoring plan, as well as which measures must be implemented in case of exceedance. SB in the permanent condition may indicate limiting values prior to opening and any follow-up that must be carried out during the operational phase.

10.1 Characterisation of ground and methods for assessing stability

In the following two subchapters, principles for the characterisation of ground and the assessment of stability during tunnel excavation in hard rock are presented. An important point in this respect is the delimitation of the ground conditions covered by the system.

As explained in Section 2.3, regions that can initially be characterised as “hard rock” may also contain zones of weathered and tectonically fractured rock, as well as occurrences of different moderately strong and weak rock types (as defined by ISRM (1978)). A system adapted to tunnel excavation in “hard rock” must therefore also take into account that such weak zones may be encountered and that rock types with low intact properties may be present. However, an essential point here is the extent, or size, of the weak zone or the occurrence of the rock type. In most cases, the extent will be limited, with contributions to stability from the more competent surrounding rock. Depending on the extent, mechanical properties, and stress conditions, one may in exceptional cases approach the boundary of, or fall within, what is commonly referred to as “soft rock tunnelling”, also in tunnel excavation in hard rock regions. Such cases will, from a rock mechanics perspective, largely be covered by the *characterisation* in the following section, however will require more extensive design and follow-up than is generally assumed in the descriptions that follow.

10.1.1 Ground behaviour types for hard rock tunnelling

As described above, dividing the ground into different GBTs is a way to assign different limiting values, monitoring plans, and measures to different parts of the tunnel system together with SB. Based on the literature presented earlier in Part 2, a set of GBTs has been derived, which is assumed to cover the various conditions that may be encountered during tunnel excavation in hard rock. These are presented in Fig. 40. The twelve types are divided into three main groups: (1) gravity-driven, (2) gravity-driven and/or stress-induced, and (3) stress-induced.

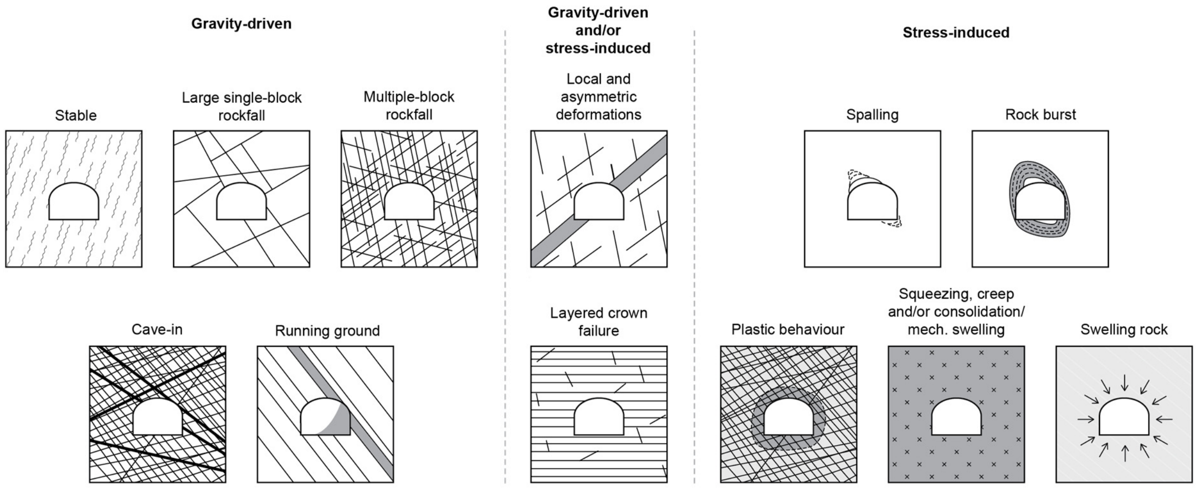

Fig. 40 Ground behaviour types for hard rock tunnelling. *Larger version of figure is found in Appendix 4*

Considering the definition of ground behaviour, it includes the rock mass itself, stresses in the rock and from water, as well as the geometry of the tunnel. In NATM, different types of conditions for ground, groundwater, stresses, and tunnel geometries are considered in order to determine a GBT, and the number of possible combinations of these is very large, which is necessary in order to be able to adapt to all possible ground conditions. If one limits the scope to tunnel excavation in hard rock, the number of alternatives that must be covered is reduced, and one may therefore establish a system with simpler relationships and also include additional relationships with the GBTs beyond those used in NATM.

In Fig. 40, the GBTs are distributed according to three types of rock stress conditions, and in Table 5, additional relationships are included for each GBT. From left to right in the table, it starts with the rock stress conditions encountered. Further to the right are different rock mass descriptions which, together with load/stress, define ground types. The next column is the behaviour of the rock mass. Important to emphasise is that the geometry of the tunnel/underground opening is included, and that this in some cases may determine which behaviour type that applies for a given situation. For example, one may move from *multiple-block fallout* to *cave-in* as the tunnel cross-section increases.

Furthermore, based on what is presented in Section 8, different failure and deformation processes are associated with each GBT. In order to assess the cause of the observations made in the tunnel, it is important to make this assessment on the basis of the correct rock mechanical processes, which in turn is decisive for designing an adequate rock support. An important point is that there are different failure/deformation processes that give rise to different types of stability problems, and that one selects a stabilization measure that is adapted to and most effectively addresses the specific type of problem. Furthermore, the rock support within a GBT in a given area may be differentiated to adapt to variations in the properties of the ground, such as the degree of fracturing.

Based on the GBT and the failure/deformation process, the table further proposes an overall concept for how the rock support may be carried out. For some of the GBTs, there are large variations in the recommended overall rock support, but the key point is that the support measures should be applied such that their mode of action is adapted to the stability problem. Taking the example of the GBT involving *large single-block rockfall* and the failure/deformation process involving plastic shear failure or tensile failure, the bolts will hold the blocks in place and prestressing will increase the strength of the joints. In this case, the bolts ensure global stability, while, if fibre-reinforced sprayed concrete is applied, it will bond and wedge smaller blocks in place and ensure local stability.

Finally, there is a comment column with further elaboration on limitations for each GBT, where to find additional information on stability assessment, as well as presumed excavation procedure (reduced blast length, etc.). The excavation procedure is particularly important when discussing excavation in hard rock, at least for Norwegian road tunnels, because the excavation process and the rock support are integrated, as all rock support forms a part of the permanent support. Here, the key concept of SB becomes central, as it is necessary to determine what constitutes acceptable observable behaviour for the different SB phases, based on the safety level of the tunnel/underground opening.

During tunnel excavation in hard rock, the presence of water will typically constitute an excavation problem, for example by making the installation of rock support more difficult, causing practical challenges, as well as reducing service life and lowering the groundwater table. This is because most of the tunnel is excavated in ground conditions where the presence of water only to a limited extent affects stability (see Fig. 15), and is usually handled by pre-grouting of the rock mass. Nevertheless, the presence of water may be a challenge that reduces stability for many of the GBTs, especially for cave-in and running ground, where the ground type consists of weak, fractured, and altered rock, particularly in subsea tunnels. In the design, it is therefore important that the probability of encountering water and how this affects stability are assessed. This also includes assessing the need for probe drilling for water and how water is to be managed, particularly with regard to pre-grouting.

Table 5 Ground behaviour types for hard rock tunnelling. See also Appendix 4 for additional details regarding suggested rock support

Loading condition	Ground type	Ground behaviour type[1]	Failure/deformation process	Suggested reinforcement and load-bearing support [23]	Comment
Gravity-driven	Massive, durable rocks at low and moderate depths	**Stable**	Elastic deformations	No support	Support may be added for durability purposes.
	Hard rock with few joints and joint sets	**Large single-block rockfall**	Plastic shear failure or tension failure of joints	Spot bolting (tensioned).	For support estimation e.g. the Q-system (NGI 2025) may be used (for approx. Q-value > 0.2). At low overburden see Terron-Almenara et.al (2023) Fig.1 (b) and table 8[5]. Fibre reinforced sprayed concrete in the crown for durability measures
	Jointed hard rock with medium to small blocks, sometimes with clay filled joints	**Multiple-block rockfall**	Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for weathered and clay filled joints.	Systematic bolting with tensioned bolts and fibre reinforced sprayed concrete.	See Terron-Almenara (2023) Fig.1 (d) and table 8[5]. For support estimation e.g. the Q-system may be used (for approx. Q-value > 0.2). Volume < 10 m3. Systematic pattern of tensioned rock bolts are used to create a pressure arch in the rock mass around the tunnel.
	Highly jointed and/or crushed rock. Soil-like. Several smaller weakness zones, clay filled joints. Often with crossing weak layers.	**Cave-in**	Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for weathered, clay filled joints and zones.	Spiling bolts fixed with rock straps and rock bolts. Surface support with reinforcement meshs and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete.	Rapid displacement of large volumes (> 10m3) of rock mass. Usually crushed and altered rock, possibly relieved by crossing weak joints and layers. May occur due to small time-dependent deformations or changes in material properties. Often reduced blasting length and/or sequential excavation.
	Enclosed granular material e.g. intensely fractured rock, sand and/or gravel.	**Running ground**	Shear failure in weak material (low cohesion) resulting in running loose material or tension failure between poorly interlocked rock particles.	Spiling bolts fixed with rock straps and rock bolts. First layer of fibre reinforced sprayed concrete and rock bolts. Surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete. Pre excavation rock mass grouting.	Material with low/no cohesion and tension strength. Moves like soil. Covers running and ravelling ground. If water under pressure is involved, flowing ground. Reduced blasting length and/or sequential excavation. For subsea tunnels a plan to handle running and flowing ground always need to be made.
Gravity-driven and/or stress-induced	Distinct Weakness Zones	**Local and asymmetric deformations**	For gravity driven: Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for clay filled joints. For stress driven: Volume decrease due to consolidation and creep, and deformation in zones due to squeezing.	Rock bolts and sprayed concrete. / Rock bolts and sprayed concrete reinforced with rock straps. / Spiling bolts fixed with rock straps. First layer of fibre reinforced sprayed concrete and rock bolts. Sprayed concrete reinforced with reinforcement mesh together with additional tensioned rock bolts. Possibly asymmetrical ribs of rebar and sprayed concrete. / Spiling bolts together with lattice girders, rock bolts and sprayed concrete. Pos. invert castconcr.	Gravity driven and stress induced failure and deformation processes may occur at the same time. Zones with a width up to 1/2 x tunnel diameter. See Terron-Almenara (2023) Fig.1 (a) and table 8[5]. Reinforcement dependent on the rock/tunnel geometry and stresses. To stop gravity driven failures rock reinforcement (asymmetrical) is applied. For stress induced problems support designs with equilibrium between the rock mass and the support is applied.
	Anisotropic Rock Mass Structure	**Layered crown failure**	Tension failure of joints and intact rock. Plastic failure (brittle crushing) of corners and intact rock.	Systematic bolting with tensioned bolts and fibre reinforced sprayed concrete. Crown surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete.	See Terron-Almenara et al. (2023) Fig.1 (c) and table 8[5]. See also Diederichs and Kaiser (1999).
Stress-induced	Hard brittle rock with few joints	**Spalling**	Plastic failure. May be time-dependent due to redistribution of stress field caused by the failure and/or tunnel excavation.	Dynamic rock bolts and fibre reinforced sprayed concrete. Possibly also reinforcement mesh covered with sprayed concrete.	See Martin CD, Kaiser PK, McCreath DR (1999) and Diederichs MS (2025) Eurock keynote. Often reduced blasting length and/or sequential excavation.
	Hard brittle rock with few joints	**Rock Burst**	Plastic failure. May be time-dependent due to redistribution of stress field caused by the failure and/or tunnel excavation.	Dynamic rock bolts, fibre reinforced sprayed concrete. Reinforcement mesh with sprayed concrete. Closed ring structure.	Martin CD, Kaiser PK, McCreath DR (1999) and Diederichs MS (2025) Eurock keynote. Often reduced blasting length and/or sequential excavation.
	Weak rock material[4], intensely fractured -or soil-like- crushed and weathered rock, likely clay seams. Weakness zones > 1/2 x tunnel diameter (dependent on the stress). Heavily jointed brittle/hard rock.	**Plastic Behaviour**	Plastic failure.	Spiling bolts and lattice girders. Dynamic rock bolts and fibre reinforced sprayed concrete between. Possibly also reinforcement mesh and sprayed concrete. Ring structure.	In heavily jointed brittle/hard/rock there may be failure around the tunnel due to overstressing of the rock mass. For more weak materials this may be the initial deformation followed time-dependent squeezing, creep and/or mechanical swelling - dependent on stresses and water situation. See Terron-Almenara (2023) Fig.1 (e) and table 8[5]. Reduced blasting length and/or sequential excavation.
	Weak rock mass, composed by weak rock material[4], intensely fractured -or soil-like- crushed and weathered rock, likely clay seams. Weakness zones > 1/2 x tunnel diameter (dependent on the stress).	**Squeezing, creep and/or mechanical swelling/consolidation**	Plastic failure, followed by squeezing and/or creep and/or mechanical swelling/consolidation	Spiling bolts and lattice girders. Dynamic rock bolts and fibre reinforced sprayed concrete between. Possibly also reinforcement mesh and sprayed concrete. Ring structure.	See Terron-Almenara (2023) Fig.1 (e) and table 8[5]. See also section 8.1.2 Time-dependent deformation. Reduced blasting length and/or sequential excavation.
	Certain rock types like bentonite, tuff, alumn shale, anhydrite, some types of flysh.	**Swelling rock**	Volume increase due to reactions between the rock mass and water/air.	Spiling bolts fixed with rock straps, dynamic rock bolts and fibre reinforced sprayed concrete as excavation support. Surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete. Possibly spiling bolts and lattice girders with reinforcement mesh and sprayed concrete together with dynamic rock bolts.	See Terron-Almenara (2023) Fig.1 (f) and table 8[5]. Dependent on the properties of the rock type, e.g. decrease of rock mass strength, geometry and stress conditions the swelling may lead to stability problems more associated with other GBTs. Often reduced blasting length and/or sequential excavation.

[1]The presence of water may affect ground behaviour and measures to detect and control water should be considered. [2]Suggestions are based on rock support practices and the acceptable risk levels commonly applied in Norwegian road tunnels. See also Appendix 4. [3]In ground conditions that requires spiling boreholes may collapse and the need for self-drilling bolts and anchors may be necessary. The use of tensioned rock bolts is preferably executed using combination-bolts (mechanical end-anchor and post-grouted). [4]May also include medium strong rock types (ISRM 1978) dependent on the rock stress. [5] See Appendix 2

10.1.2 Methods for assessing rock support and stability

As mentioned in Section 4, the observational method is a form of systematisation of other methods that can be used for design, where "acceptable limits of behaviour shall be established", and "the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits" (Standard Norge 2020).

In order to assess what constitutes an acceptable probability and acceptable limits, one must apply the methods that best suit the different ground conditions. Terron-Almenara (2025) carried out an review of different rock engineering methods for evaluating stability for the GBTs shown in Fig. 39 (see also Table 2 in Terron-Almenara (2025)). The different rock engineering assessment methods, with a somewhat revised descriptions, are presented in Table 6.

In Table 7, the review made by Terron-Almenara (2025) of suitability of rock engineering methods have been extended, and somewhat revised, to apply to all GBTs for tunnel excavation in hard rock. How well a method will perform will, of course, depend on the amount of effort invested in the assessment and on where the threshold for "suitability" is set. The "scoring" should therefore not be regarded as absolute, but rather as a simple ranking. For example, one may encounter cases where few suitable methods are available, but considerable effort is invested in several less suitable methods, and still achieve an adequate overall assessment. Note also the fundamental differences between the evaluation methods described in the notes in Table 7.

The type of monitoring that is appropriate will depend on the types of GBT present. For example, for the GBT involving fall-out of individual large blocks, deformations that can be detected by total station measurements are not expected, and visual inspection and, if relevant, hammer sounding of the sprayed concrete may be alternatives.

As exemplified above, with the rock engineering methods at hand, it is necessary that limiting values are be expressed through qualitative observations and simple field tests, in addition to quantitative measurements. Such criteria are particularly relevant where conventional instrumentation is not expected to provide meaningful information, or where the governing ground behaviour type does not involve measurable displacements. In these cases, limiting values can be defined in terms of observable conditions that shall not be exceeded. Consequently, the specification of limiting values should be adapted to the expected response of the system and may include both quantitative thresholds and qualitative indicators derived from observations, tests and measurements.

Table 6 Rock engineering assessment methods. Based on Terron-Almenara (2025)

Rock engineering assessment	Description
Empirical methods[1]	Use of, for example, the Q-system and RMR. Prediction of brittle failure. Investigations, including documentation, of previously applied solutions for specific cases.
CCM[1]	Use of the Convergence Confinement Method by Carranza-Torres and Fairhurst (2000)
Analytical methods[1]	Includes simple static equilibrium analyses of local stability of wedges and blocks, kinematic analyses of wedges, stability analyses of horizontal layering using the voussoir analogy
Numerical methods[1]	Continuous, discontinuous or combined models depending on the ground conditions. Performed as parametric analyses. For large deformations, parameter-based back-analysis.
Deformation measurements[2]	Systematic monitoring in the form of deformation measurements
Observation and control[2]	Systematic visual monitoring/observation of rock support, hammer sounding of sprayed concrete, etc.
Tunnel mapping[3]	At the face. Rock mass classification (e.g. GSI, Q-value and RMR value). Engineering geological mapping and documentation. Assessment of the rock mass ahead of the face using interpretation of drilling parameters and the engineering geological site investigations. Investigations such as probe drilling, core drilling, testing of rock and weakness zone materials, measurement of rock stresses, etc.

[1]Rock support design [2]Monitoring and/or control [3]Overall (excavation) planning, documentation and execution

Table 7 Suitability of different methods and tools for rock engineering design for GBTs in hard rock. (A) Suitable (B) Limited suitability/indirect use (C) Not applicable/not suitable. The table is an extended version of Table 2 in Terron-Almenara (2025), where the original GBTs are shown in grey rows. The names of some GBTs have been modified from the original table.

Ground behaviour type	Empirical methods	CCM	Analytical methods	Numerical metods[2]	Deformation measurements	Observation and control	Tunnel mapping
Stable	A	C	C	C	B	A	A
Large single-block rockfall	A	C	A	B	B	A	A
Multiple-block rockfall	A	C	B	B	B	A	A
Cave-in	B	C	C	B	A[3]	A	A
Running ground	B	C	C	B	A[3]	A	A
Local and asymmetric deformations	B	C	C[4]	A	A[3]	A	A
Layered crown failure	B	C	A	A	A[3]	A	A
Spalling	A[1]	C	C	A	A	A	A
Rock burst	A[1]	C	C	A	A	A	A
Plastic behaviour	B	A	B	A	A	A	A
Squeezing, creep and/or mechanical swelling/consolidation	B	A	B	A	A	A	A
Swelling rock matrix	B	C	B	C	A	A	A

[1]Empirical prediction of brittle failure (Kaiser, McCreath and Tannant 1996; Diederichs 2025), not, for example, the Q-system. [2]Provided that it is carried out with high quality and with a critical assessment of the results. [3]Deformations are not expected to be measurable. Measurements are carried out to control/verify a stable structure. [4]Suitability changed from Terron-Almenara (2025)

10.2 Project planning using the observational method in hard rock tunnelling

A fundamental principle when applying the observational method is system behaviour. This is included in both the design and construction phases, in that during planning, the rock support is designed and acceptable limits for system behaviour are established based on the expected ground behaviour. During construction, verification that the actual conditions and the behaviour of the system correspond to the assumptions made in the design is performed, and any deviations are managed by means of predefined measures.

In N500 Road tunnels, as mentioned in Section 2.1, there is a requirement that, as part of the zoning plan work, all investigations necessary for constructing the tunnel shall in principle have been carried out, and that these shall be documented in an engineering geological report. The subsequent phase in the planning work is the preparation of documentation for the detailed design. The process for detailed design varies depending on the type of contract. For example, for unit price contracts, one combined detailed design report is prepared for the entire tunnel system, which is included in the tender documents. For design–build contracts, the planning work may be carried out in a more stepwise manner, with reports prepared for parts of the tunnel system both before and during construction.

Fig. 41 shows, at an overall level, how the observational method can be incorporated into the planning and construction process, and into operation and maintenance. For the work up to and including the zoning plan phase, most activities will be carried out as previously, but with some additions. The main change is that it is necessary to assess which GBTs may be expected in different areas of the tunnel system. This implies that overviews must be prepared for the tunnel alignments, cross passages, caverns for pumping stations, etc., and that expected GBTs must be assigned. In addition, the rock mass must be classified using GSI and Q-values. GSI is a purely engineering geological parameter describing the rock mass without considering the tunnel, support, etc., and is useful for describing the rock mass itself. Additionally, GSI may also be useful as input in numerical modelling. For cost estimates of support and excavation, it may still be appropriate to map Q-values, as these can be used to differentiate the required amount of rock bolts and sprayed concrete in competent and fractured rock. The Q-value may also be useful for identifying the transition from support with only sprayed concrete and bolts to excavation with spiling bolts, reduced blasting rounds, and more comprehensive rock support.

In the preparation of the detailed design, the application of the observational method will lead to relatively significant changes compared with previous practice, as this is the phase that the framework for the observational method must be established. The rock support must be designed for the different areas of the tunnel based on GBT and SB. This implies that acceptable limiting values must be defined, a monitoring plan must be described, and measures must be specified in case the limits are exceeded. An important point is that during construction, the actual conditions in an area are mapped and assessed so that stability is ensured based on the correct GBT and the appropriate support level within that GBT. For areas with special geometry, for example entrance/exit ramps, and uncertain geology, it may be necessary to design rock support for two (or more) GBTs, with corresponding SBs. This is because one does not necessarily know the actual ground conditions until approaching or encountering them.

Phases		*The observational method*
Planning	Geological preliminary investigations (early planning phase, municipal sub-plan, zoning plan)	- Engineering geological mapping and preliminary investigations, including establishment of ground model - Determination of GBT along the tunnel alignment, including Q-value (or RMR) and GSI
	Geological report for detailed design	- Possible supplementary investigations and updating of ground model - Divide the tunnel into sections with corresponding GBT - Rock engineering design of rock support and excavation method, establish SB, for the various sections
Construction	At the face	- Mapping of the rock mass at the face - Assessment of GBT and execution of designed rock support, based on SB (2) - Assessment of the ground ahead of the face and further excavation method, based on SB (1)
	Behind the face	Based on SB (2) - Implement monitoring program - Assess against limiting values - Implement countermeasures if needed
Operation and maintenance	Periodic inspections	- Implement planned monitoring, based on SB (3)

Fig. 41 Execution of the observational method through planning, building, and operation and maintenance

As indicated above, SB, i.e. the interaction between the ground and the rock support, is a key concept in both the detailed design phase and the excavation of the tunnel. By dividing SB into three parts, (1) the excavation area, (2) the supported area, and (3) the permanent condition, it can be used to define acceptable limits for behaviour, monitoring plans, and predefined measures in case limits are exceeded. SB and the associated framework for execution of excavation, support, and monitoring must be established for each GBT and its corresponding area. The objective is to ensure a stable tunnel throughout the excavation cycle and to the permanent phase. For SB (1), decisions must be made regarding the excavation method, for example blast round length and whether support ahead of the face using spiling bolts is required. Investigations ahead of the face (interpretation of drilling parameters/core drilling) to detect transition to more unstable conditions may also be relevant to include in this SB. Furthermore, for SB (2), it must be described how stability is evaluated and how stabilization is carried out in detail. A plan must also be established for monitoring stability, including limits for when supplementary support must be applied.

As mentioned previously, all rock support is considered part of the permanent support, and at the SB2 stage, when the limits for acceptable behaviour are considered to be satisfied, the support is regarded as completed. An important point is to include time-dependent deformations and failure processes in the assessment and to evaluate whether these have fully developed during the construction period.

In Austrian Society for Geomechanics (2010), SB (3), the permanent phase, is described as the phase after the tunnel lining has been installed. For tunnels constructed according to NATM, this lining may be designed to carry loads from the rock. Such a solution will not generally be applicable for Norwegian road tunnels, but also modern Norwegian road tunnels have tunnel linings designed to withstand fall-out of smaller blocks. The design of rock support and tunnel lining must be coordinated such that requirements related to structural safety, user safety, and traffic safety are satisfied, and this must be in place for SB (3). An example may be that if sprayed concrete is omitted in the walls for a GBT involving fall-out of individual blocks, this must be compensated for by a lining in the walls that can protect users and ensure traffic safety.

As a general rule, stability must be ensured for the permanent phase, SB (3), but also for this phase it must be specified how stability is to be monitored during the service life of the tunnel. It is therefore important that the rock support system, together with the associated lining, is designed such that monitoring can be carried out.

11 Future work

The *N500 Road Tunnels* is the governing standard for the design of Norwegian road tunnels, while guidance related to site investigations and the design of rock support is provided in *N-V521 Geology and rock support in road tunnels*. The content of this report is intended for use in updating both of these documents. The framework for the application of the observational method will be incorporated into N500, while matters related to rock engineering will be incorporated into N-V521. This further implies that the rock support classes in N500, shown in Fig. 17 in this report, will be removed.

A main limitation of the current standard is that it is detailed for the least complex cases, whereas for situations involving more complex geometry and difficult ground conditions, it simply require "design specifically" without providing further direction. Since the design is subjected to the Eurocodes, "design specifically" (Fig. 17) implies the use of the observational method, but current practice in such cases is to a large extent not sufficiently systematic to satisfy the requirements given in the Eurocodes. By implementing the framework described in this report, one will obtain a practice that fulfils the Eurocode requirements for design for all ground conditions, while also having a broader and more robust scientific basis.

If one considers the basis for assessing the stability of an underground opening, for example as summarised in Table 5 and Table 7, it can be seen that in some cases there will be a lack of good methods that alone provide reliable answers on how to design the rock support and evaluate stability. As mentioned previously, one approach in such cases may be to refer to previously completed cases, as well as to invest more effort in methods that are less suitable. A key point in this regard is to approach the problem with an awareness of the limitations in the assessments, so that one can compensate for these in the best possible way. This goes to the core of the use of empirical methods, and to some extent also other methods such as numerical modelling, when these present themselves as applicable over a broader range than they actually are. They appear to provide solutions to the problems at hand, but contain hidden limitations that one may not necessarily be aware of and therefore cannot compensate for.

Finally, reporting and sharing of experience from the design and execution of tunnel projects will be essential for the further development of systems and theory for stability assessment under different types of ground behaviour and stability requirements. The theory and compilations presented in this report should therefore by no means be regarded as an end point, but rather as a basis for professional discussion, and as a foundation for identifying topics for further research and development.

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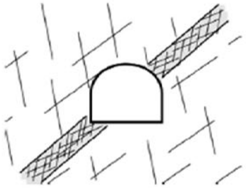
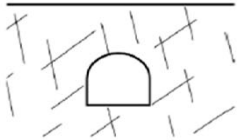
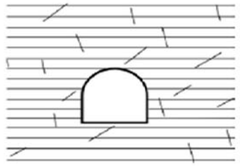
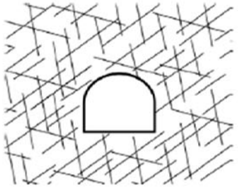
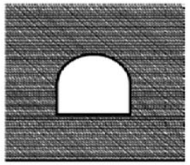
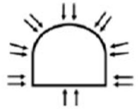
Behaviour type	**Definition**	**Comments**
Type 1 Gravity driven		
a. Stable	The surrounding ground will stand unsupported for several days or longer	Massive, durable rocks at low and moderate depths
b. Block fall(s)		
– of single blocks	Stable with potential fall of individual blocks	Discontinuity controlled failure
– of several blocks	Stable with potential fall of several blocks (slide volume <10 m^3)	Discontinuity controlled failure
c. Cave-in	Inward, quick movement of larger volumes (>10 m^3) of rock fragments or pieces	Encountered in highly jointed or crushed rock
d. Running ground	A particulate material quickly invades the tunnel until a stable slope is formed at the face. Stand-up time is zero or nearly zero	Examples are clean medium to coarse sands and gravels above groundwater level
Type 2 Stress induced		***Brittle behaviour***
e. Buckling	Breaking out of fragments in tunnel surface	Occurs in anisotropic, hard, brittle rock under sufficiently high load due to deflection of the rock structure
f. Rupturing from stresses	Gradually breaking up into pieces, flakes, or fragments in the tunnel surface	The time dependent effect of slabbing or rock burst from redistribution of stresses
g. Slabbing	Sudden, violent detachment of thin rock slabs from sides or roof	Moderate to high overstressing of massive hard, brittle rock. Includes popping or spalling.[a]
h. Rock burst	Much more violent than slabbing and involves considerably larger volumes. (Heavy rock bursting often registers as a seismic event)	Very high overstressing of massive hard, brittle rock
		Plastic behaviour
i. Plastic behaviour (initial)	Initial deformations caused by shear failures in combination with discontinuity and gravity controlled failure of the rock mass	Takes place in plastic (deformable) rock from overstressing. Often the start of squeezing
j. Squeezing	Time dependent deformation, essentially associated with creep caused by overstressing. Deformations may terminate during construction or continue over a long period	Overstressed plastic, massive rocks and materials with a high percentage of micaceous minerals or of clay minerals with a low swelling capacity.
Type 3 Water influenced		*Hydrazination*
k. Ravelling from slaking	Ground breaks gradually up into pieces, flakes, or fragments	Disintegration (slaking) of some moderately coherent and friable materials. Examples: mudstones and stiff, fissured clays
l. Swelling		*Swelling minerals*
– of certain rocks	Advance of surrounding ground into the tunnel due to expansion caused by water adsorption. The process may sometimes be mistaken for squeezing	Occurs in swelling of rocks, in which anhydrite, halite (rock salt) and swelling clay minerals, such as smectite (montmorillonite) constitute a significant portion
– of certain clay seams or fillings	Swelling of clay seams caused by adsorption of water. This leads to loosening of blocks and reduced shear strength of clay	The swelling takes place in seams having fillings of swelling clay minerals (smectite, montmorillonite)
		Flowing water
m. Flowing ground	A mixture of water and solids quickly invades the tunnel from all sides,including the invert	May occur in tunnels below groundwater table in particulate materials with little or no coherence
n. Water ingress	Pressurized water invades the excavation through channels or openings in rocks	May occur in porous and soluble rocks, or along significant openings or channels in fractures or joints

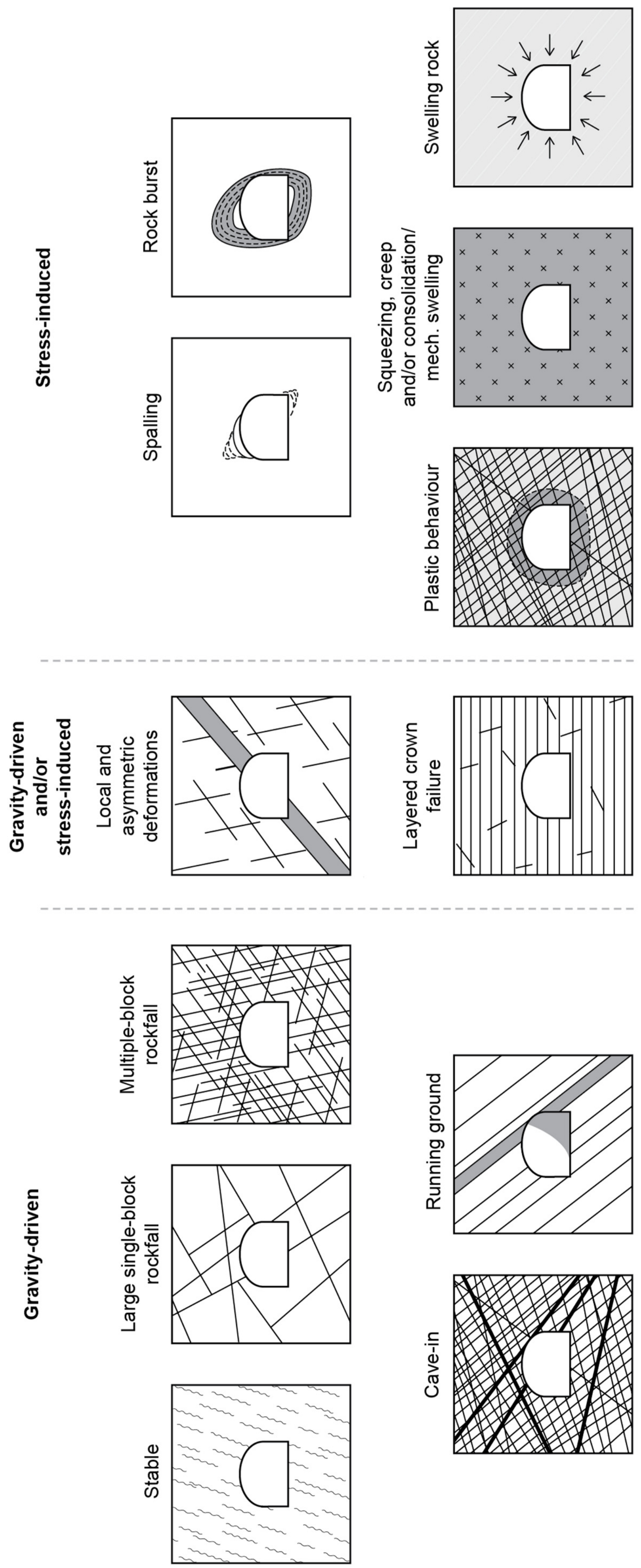
[a] This term was often used by Terzaghi (1946) as synonymous with the falling out of individual blocks, primarily as a result of damage during excavation

Ground behavior type	Weakness zone	Low rock stresses	Anisotropic rock masses
Illustration of GBT (Sect. 2.3)			
Loading and failure mechanisms (Sect. 4)	a1, a2, b1, b2, c1, c2	a1, a2, b1, b2, c1, c2	b1, b2, c1, c2
Site investigation	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	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	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 $Q < 0.4$
Empirical method	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	The Q-system and the GSI are recommended to determine the respective basic support needs and the rock mass strength	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
Analytical method	Analytical calculations may be used to determine the local capacity of sprayed concrete members at the zone. CCM has very little applicability	Study of block stability with limit equilibrium analyses is recommended regardless rock mass quality	Limit equilibrium analyses to derive the depth of the pressure arch and block stability are relevant when $Q < 1$
Numerical method	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)	Design verification with numerical analyses is recommended in tunnels with low rock stresses provided complex ground conditions and/or $Q < 1$	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
Monitoring	Convergence monitoring is recommended in the presence of unfavorable combinations of width/orientation of the zone and/or evidence of swelling	Monitoring of deformation is recommended in the tunnel and in the nearby infrastructure to control stability and design verification	Deformation monitoring is recommended for the back-calculations of rock mass properties (recommended in tunnel sections with $Q < 0.4$)
Support design	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 $Q > 0.1$	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	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 $Q > 0.1$

Hybrid design procedure for rock support in tunnels in poor rock mass conditions, and moderate to low in-situ rock stresses (Terron-Almenara, Holter and Høien 2023) (cont.)

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 $\sigma_{cm}/p_o < 1$	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_θ/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 $\sigma_{cm}/p_o < 1$	Numerical analyses are recommended. Structural analyses with numerical programs are also relevant to verify failure states of liners and inverts
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


Gravity-driven
Stable
Large single-block rockfall
Multiple-block rockfall
Cave-in
Running ground
Gravity-driven and/or stress-induced
Local and asymmetric deformations
Layered crown failure
Stress-induced
Spalling
Rock burst
Plastic behaviour
Squeezing, creep and/or consolidation/ mech. swelling
Swelling rock

The following table elaborates the connection between the Ground behaviour type, its failure/deformation process and the suggested reinforcement and support type.

The suggested reinforcement and support in Table 5 are based on rock support practices and the acceptable risk levels commonly applied in Norwegian road tunnels. These recommendations may not be suitable for builders in other countries, as their traditions in rock support methods may differ and they may have different safety requirements, which influence the acceptable risk level.

Ground behaviour type *Failure/deformation process*	**Suggested reinforcement and load-bearing support type**	**Comment**
Stable Elastic deformations	No support	No need for support if nothing fails
Large single-block rockfall *Plastic shear failure or tension failure of joints*	Spot bolting (tensioned). Fibre reinforced sprayed concrete in the crown for durability measures	Tensioned rock bolts to increase the normal force in joints to increase resistance to shear failure, and to prevent tension failure. Over time rock fragments and smaller blocks may loosen which can be held in place by applying sprayed concrete.
Multiple-block rockfall *Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for weathered and clay filled joints.*	Systematic bolting with tensioned bolts and fibre reinforced sprayed concrete.	Tensioned rock bolts to increase the normal force in joints to increase resistance to shear failure, and to prevent tension failure. Systematic bolting to secure key blocks and possibly create arching in the rock mass. Fibre reinforced sprayed concrete to create wedges that holds blocks in place, and to generally hold the rock surface in place.
Cave-in *Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for weathered, clay filled joints and zones.*	Spiling bolts fixed with rock straps and rock bolts. Surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete.	Spiling bolts to keep the rock in place until sprayed concrete can be applied to create surface support. Spiling bolts are fixated at the collar end of the borehole by rock straps and radial rock bolts, and is sprayed with concrete before blasting (see Fig. 19 a)). Extra surface support may be necessary and can be executed by sprayed concrete reinforced with reinforcement mesh. Also asymmetrical rebar rod reinforced ribs of sprayed concrete may be used, however they do not have reinforcement in the length direction of the tunnel, compared to net reinforced. The grouted, tensioned rock bolts should have the plate placed on the surface support to hold the rock in place and help create an arching effect in the rock mass. As the load is local the support will be subjected to a situation more similar to point load on a beam, rather than a uniform load creating tangential compression forces in a ring structure, hence curved support is not required.
Running ground *Shear failure in weak material (low cohesion) resulting in running loose material or tension failure between poorly interlocked rock particles.*	Spiling bolts fixed with rock straps and rock bolts. First layer of fibre reinforced sprayed concrete and rock bolts. Surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete. Pre excavation rock mass grouting.	Tight pattered spiling bolts to keep the rock in place until sprayed concrete can be applied to create surface support. Spiling bolts are fixated at the collar end of the borehole by rock straps and radial rock bolts, and is sprayed with concrete before blasting (see Fig. 19 a)). Pre excavation rock mass grouting to increase the strength of the rock mass. However the effect of grouting is uncertain. Surface support should be executed by reinforcement mesh, rather than rods, and sprayed concrete for better interaction between good and weak rock. Self drilling spiling/anchors or pumpable pipes may be needed. Grouted, tensioned rock bolts, with the plate placed outside on the surface support to fixate and keep the rock in place.

Local and asymmetric deformations *For gravity driven: Plastic shear failure or tension failure of joints. Brittle for fresh joints, ductile for clay filled joints. For stress driven: Volume decrease due to consolidation and creep, and deformation in zones due to squezing.*	Rock bolts and sprayed concrete. / Rock bolts and sprayed concrete reinforced with rock straps. / Spiling bolts fixed with rock straps. First layer of fibre reinforced sprayed concrete and rock bolts. Sprayed concrete reinforced with reinforcement mesh together with additional tensioned rock bolts. Possibly asymmetrical ribs of rebar and sprayed concrete. / Spiling bolts together with lattice girders, rock bolts and sprayed concrete. Pos. invert castconcr.	Suggestions separated by slashes represents increased support needs. The zone direction to the tunnel axis, material properties, extent and the stress conditions determines the amount of needed reinforcement and support. In gravity driven situations the principal will be to reinforce to keep the rock in place and help it to be self bearing. A relatively small vertical zone 90 degree on the tunnel axis may be supported by rock straps and sprayed concrete together with tensioned rock bolts. The same zone, but sub vertical and with a small acute angle to the tunnel axis may cause large portions of the crown to be "free" and create a need for support that can handle large shear forces crossing the zone. The rock stress may cause clay rich gouge material to consolidate or creep, or with increased stress, squeeze. Dependent on the geometry and stress state the weak material may create pressure on the support by itself and/or make more solid parts of the rock mass move relative to each other. In stress induced problems the support solutions may be similar to that of gravity driven, but with larger loads acting. Hence, the possible need for lattice girders that better can handle local loads than net reinforced sprayed concrete and symmetrical ribs of rod reinforced concrete. The most common way of designing double-reinforced arches, with connections between the layers only at the anchorage bolts, may have difficulties in handling shear movements. Need for invert support may come from movement of the walls, or if the invert deform over time in a way that may cause implications for future constructions.
Layered crown failure *Tension failure of joints and intact rock. Plastic failure (brittle crushing) of corners and intact rock.*	Systematic bolting with tensioned bolts and fibre reinforced sprayed concrete. Crown surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete.	A main task for a tensioned bolt support in such cases is to hold up the crown slabs and create a "thicker package" by increasing the strength in the joints. Dependent on the strength of the rock and joints, and the stress state, one might need more surface support that fibre reinforced sprayed concrete (mesh reinforced). In cases of high horizontal stresses and smoot joints one should also be aware of the possibility to have shear failure along the joints in the springline area. In such stability problems the wedging effect of sprayed concrete together with tensioned rock bolts may help the rock to stay in place.
Spalling *Plastic failure. May be time-dependent due to redistribution of stress field caused by the failure and/or tunnel excavation.*	Dynamic rock bolts and fibre reinforced sprayed concrete. Possibly also reinforcement mesh covered with sprayed concrete.	Rock bolts in these conditions need to be able to withstand sudden loads and large deformations. Tensioning of the bolts would theoretically increase the confinement of the rock mass prone to fail and reduce the spalling. Surface support is important to keep the rock in place, also to reduce spalling effect.
Rock Burst *Plastic failure. May be time-dependent due to redistribution of stress field caused by the failure and/or tunnel excavation.*	Dynamic rock bolts, fibre reinforced sprayed concrete. Reinforcement mesh with sprayed concrete. Ring structure.	Same as above. In severe cases a ring structure may be needed to create pressure against the rock mass to help the bolts handling the load.
Plastic Behaviour *Plastic failure.*	Spiling bolts and lattice girders. Dynamic rock bolts and fibre reinforced concrete between. Possibly also reinforcement mesh and sprayed concrete. Ring structure.	Spiling bolts to keep the rock in place until sprayed concrete can be applied to create surface support. Spiling bolts are fixated at the collar end of the borehole by rock straps and radial rock bolts and is sprayed with concrete before blasting (see Fig. 19 a)). Extra surface support may be necessary and can be executed by sprayed concrete reinforced with reinforcement mesh. Due to rock stress induced failure the invert is subjected to the same loads and deformations as the crown and a ring support may be needed to keep walls from moving inward and invert to rise. Dynamic rock bolts may be needed for bolts anchored in more solid side rock.

Squeezing and/or creep and/or mechanical swelling/ consolidation *Plastic failure, followed by squeezing and/or creep and/or mechanical swelling/consolidation*	Spiling bolts and lattice girders. Dynamic rock bolts and fibre reinforced concrete between. Possibly also reinforcement mesh and sprayed concrete. Ring structure.	Same as above. However, the deformations will continue with time, not only from advancement of the face.
Swelling rock *Volume increase due to reactions between the rock mass and water.*	Spiling bolts fixed with rock straps, dynamic rock bolts and fibre reinforced sprayed concrete as excavation support. Surface support with reinforcement mesh and sprayed concrete (without fibres). Possibly asymmetrical ribs of rebar and sprayed concrete. Possibly spiling bolts and lattice girders with reinforcement mesh and sprayed concrete together with dynamic rock bolts.	Same as above. The swelling may induce new joints in the rock mass altering the behaviour.

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