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Design of reinforced sprayed concrete arches in rock tunnelling

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Abstract

Reinforced sprayed concrete arches are a tunnel support measure commonly used in ground conditions with poor rock mass and low or insufficient rock cover, weaknesses, or fault zones. This is a typical solution for rock excavation support in a single–layer tunnel lining.The initial chapters outline the empirical, analytical, and numerical principles governing the design process. Subsequently, two case histories from Swedish tunnelling practice—the Högdalsdepån and Flemingsbergstunneln projects—are analysed to demonstrate how shotcrete ribs are implemented within single-layer linings to optimise structural stiffness and load-bearing capacity.

Crucial uncertainties remain regarding rock-shotcrete deformation compatibility and interface interaction,the precise load-transfer mechanisms, and design criteria for integrated arch-spile systems.

Keywords Shotcrete arches/ribs, Insufficient cover, Weaknesses, Fault zones, Single–layer tunnel lining, Swedish tunnelling practice.

1 Introduction

Tunnelling mainly involves excavating rock or soil, supporting or lining the tunnel walls, and mucking (removing debris). Depending on the ground conditions, construction methods, and water inflow (pressures), supporting measures can be either permanent or temporary. For rock tunnelling, shotcrete combined with rock bolts has been the dominant support method since the 1980s (Sjölander, 2020).

The traditional approach uses two separate support measures: an initial lining and a final lining. However, the modern trend in rock tunnelling is to design permanent single-shell tunnel linings. In Sweden, this approach has been followed for decades, but these efforts have been strengthened by the European Union (EU) 's ambitious new goal of becoming climate neutral by 2050. The rock support design is often based on a combination of empirical, analytical methods and numerical analysis. In difficult ground conditions, especially where “poor” rock mass prevails, such as thin rock cover and/or poor rock conditions, reinforced sprayed concrete arches are frequently employed.

This paper presents a design approach for reinforced sprayed concrete arches as a permanent single-shell tunnel lining, illustrated by two examples from Sweden. A brief explanation of design principles is provided in the following chapters, followed by case history examples.

2 Design of Reinforced Sprayed Concrete Arches as Permanent Rock Support

The design of reinforced sprayed concrete arches as permanent rock support is conducted as a load-bearing structure with rockinteraction, in accordance with rock-mechanical design principles.

This indicates that empirical, analytical, and numerical approaches are used to link anticipated rock-mass behaviour to appropriate support measures. Additionally, the structural engineering principles are applied to verify the load-bearing capacity of reinforced sprayed concrete arches by calculating their moment and shear capacities in accordance with Eurocode SS-EN 1992-1-1 (Standarder, 2023).

To clarify, the support for the rock tunnel was evaluated through multiple methods: (1) an empirical approach, (2) modelling the rock support as a beam on an elastic foundation, and (3) numerical analysis of the rock and tunnel using the finite element method (FEM). All these design approaches are described in the chapters below.

2.1 Design with an empirical approach

The stability of the rock tunnel and the support measures should primarily rely on a proper understanding of how the rock mass and tunnel structure might behave. To do that, rock mass behaviour andthe design of rock support are initially based on empirical methods using a rock mass classification system.

Recent experience with large tunnelling projects managed by the Swedish Transport Administration generally involves using the Q-method developed by Barton et al. (Barton, Lien , & Lunde, 1974), which has been adopted by the Norwegian Geotechnical Institute (NGI, 2025).

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_W}{SRF} \tag{1}$$

In the Q-method, the rock mass quality Q is calculated from the three sets of ratios that represent the rock mass structure ($\frac{RQD}{J_n}$), roughness, and joint alternation ($\frac{J_r}{J_a}$), and active ($\frac{J_W}{SRF}$) (Barton, Lien , & Lunde, 1974).To estimate therock mass quality index (Q), 6 parameters are required. The *Q*-value and the equivalent dimension (D_e), which is the ratio of the effective length to the excavation support ratio (*ESR*), are empirically linked to certain support measures. Guidelines for using various supporting elements—such as spiles, arches/ribs, and bolts—in relation to the Q-value and equivalent dimension (D_e) are shown in Fig. 1.

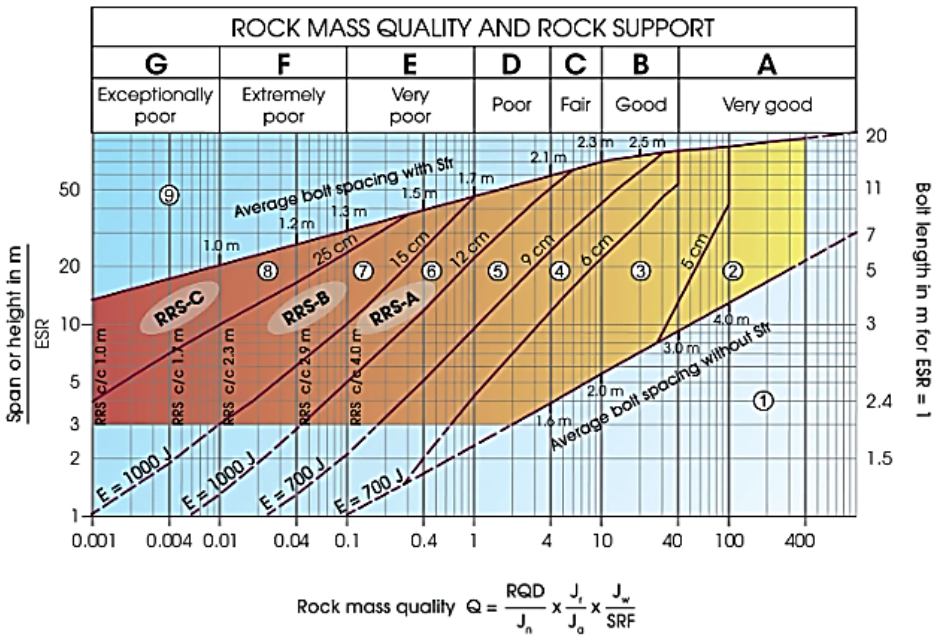


Fig. 1 The latest updated Q-system for design of rock support (NGI, 2025).

This rock mass classification has been significant updated based on recent experience(Grimstad, Kankes, Bhasin, Magnussen, & Kaynia, 2022)in the construction of underground openings in hard rock, using various support methods and excavation techniques.

2.2 Design with a structural design approach

In a structural design approach, moment/thrust interaction is considered, whereby rock support in direct contact with the surrounding rock benefits from rock-structure interactions like natural rock arches and movement restraint. Such a structure can generate moments and thrusts simultaneously, providing a high degree of load-bearing (structural) capacity.

Stable natural rock arching occurs when rock support interacts with the surrounding rock mass. In this phenomenon, the support system and the rock mass act as a composite structure; loads imposed on the support by the rock can be supported by both the lining and the rock, depending on the magnitude of the strain distribution.

Movement restraint refers to rock support placed in direct contact with the rock to prevent the lining from developing large moments.

In the beam-spring method, the rock support is generally modelled as a structural element,a flexible continuous beam. The beam structural elements are modelled as linear elastic, with their stiffness determined by the thickness and elastic modulus of the constituting material. The rock is also assumed to behave as a linear-elastic material, accounting for the stiffness of the surrounding rock. Each segment of the tunnel, from the spring line to the bottom, is supported by springs, whose elasticity represents the ground reaction (K_s); i.e., the rock is modelled as a continuum medium. It is assumed that the ground reaction is generated by the displacement of the rock support (lining) proportional to the rock deformation (Kerr, 1964), (Hetenyi, 1963), (Pasternak, 1954).

$$p(x) = k \cdot \delta(x) \tag{2}$$

$$p(x) = k \cdot \delta(x) - G \cdot \delta''(x) \tag{3}$$

$$p(x) = k \cdot \delta(x) - D \cdot \delta^{IV}(x) \tag{4}$$

where k is the stiffness of the surrounding rock, G is the stiffness of the shear rock layer in the Pasternak model(Pasternak, 1954), and D is the bending stiffness in the Hetényi model (Hetenyi, 1963).

The entire calculation involves defining the geometry with linear elements, modelling rock-structure interaction using elastic springs, determining load size, direction, and nature, analysing load combinations and worst-case scenarios, performing static calculations to find internal forces, moments, and displacements, and finally, moment and shear force capacity verification per Eurocode 2(Standarder, 2023). The final step is to check total displacement, plastic yielding of rock, and stresses at the tunnel boundary and along the bolts and spiles.

2.3 Design with the numerical analysis by using the finite element method

Numerical analysis (FEM) can be employed to identify suitable support measures and to verify if the selected ones are appropriate and sufficient.

The rock support is modelled using a 2D or 3D implicit continuum model that simulates the interaction between the lining structure and the surrounding rock, reproducingactual boundary conditions and all construction phases. 2D models are often used for their computational efficiency, whereas 3D models provide the most realistic representation of the excavation face. Both 2D/3D FEM analysis can predictrock movements, structural forces in the support, and overall stability during construction.

Usually, the rock mass is modelled as an elastic-plastic material, using quadrilateral or triangular continuum elements, while shotcrete or shotcrete arches/ribs are modelled as linear, shell, or solid elastic or elastic-plastic elements. Rock bolts or spiles are modelled as linear or solid elastic or elastic–plastic elements.

FEM results provide spatial insight into the distribution and redistribution of stresses at the tunnel boundary and along bolts, spiles, and other elements; the plastic yielding of rock; internal force distributions; and displacement (settlement) values throughout the entire spatial model.

The stresses in the rock support are calculated from the major and minor principal stresses at the periphery of the tunnels or in other structural elements. These stresses induce the axial thrust, shear

force, and bending moments in the structural elements (shotcrete, bolts, etc.). For shotcrete or shotcrete arches, plots of the thrust-bending moment and thrust-shear force (support capacity diagrams) are constructed to determine whether the rock loads exceed the load-bearing capacity. A similar approach is also adopted for the rock bolt and spile elements. Rock mass movement generates stresses in rock bolts and spiles. These elements are checked to determine whether the loads exceed the bolt or spile capacity or not.

3 Case Study of Reinforced Sprayed Concrete Arches Design

In this chapter, two cases have been reviewed that utilise reinforced sprayed concrete arches as permanent rock support – single-shell tunnel linings.

3.1 Högdalsdepån, service/access tunnel

The Högdalsdepån project began construction in 2020 and is scheduled for completion in 2026. It is a2.5-kilometre-long subway tunnel that will double the capacity of the Högdalen depot. The subway tunnel includes three new tracks: a connecting track and two parking tracks, with the parking tracks located in a separate, large underground storage area. The expanded depot will be able to service trains on both the green and blue metro lines of the Stockholm subway. The total cost is expected to be 800 million Swedish crowns.

As part of the construction of the Högdalsdepån, it is planned to first excavate the access tunnel from the north. The excavation work will be carried out predominantly from this tunnel, without disrupting traffic on other roads. The working tunnel will serve to transport machines and personnel underground and to remove blasted rock. Later, two more working faces will be excavated to the west and east. Once construction is finished, the access tunnel will continue to be used for service and emergency evacuation.

A high-arched tunnel profile with a total cross-sectional area of 38.90 m² has been designed. The tunnel span is 5.4 m, the maximum height along the axis is 6.87 m, and the sidewall height is 5.35 m. The total length of the working tunnel is 342 m.

After construction, the tunnel will serve as a service and emergency evacuation tunnel. Therefore, it includes niches for electrical and technical equipment, service niches, a water supply and sewer station niche, vehicle parking and turning niches, and a fire-protection pressurisation chamber, Fig. 2.

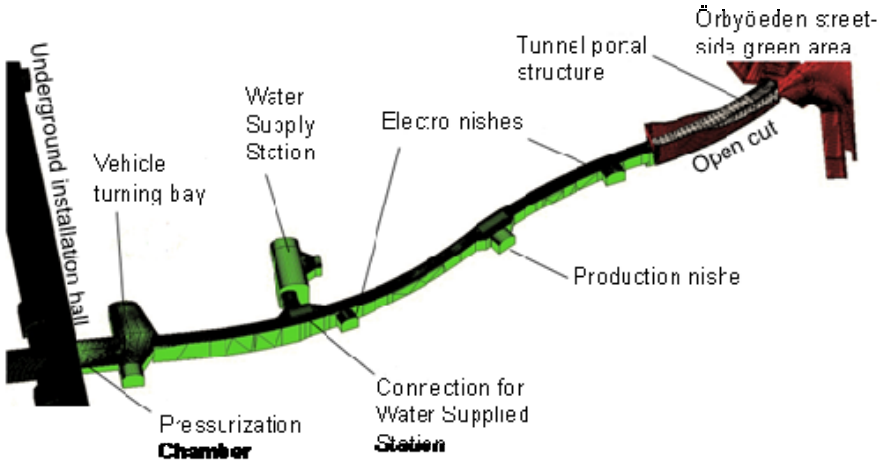


Fig. 2 Overview illustration of the access tunnel with niches for electrical and technical equipment.

The surrounding rock mass consists mainly of well-consolidated fine- to medium-grained orthogneiss, locally intersected by diabase and pegmatite veins, and generally exhibits a blocky structure with occasional weakened or fractured zones.Between km 0+138 and km 0+155, immediately after the first electrical niche, the tunnel passes through a shallow overburden zone with a weakness zone, Fig. 3. The rock cover above the crown ranges from 2.10 to 2.30 m, while the total overburden, including clayey soil, is about 6.60 m. According to the RMR classification, the rock mass in this section is at the boundary between fair and poor rock (RMRbas ≈ 48).

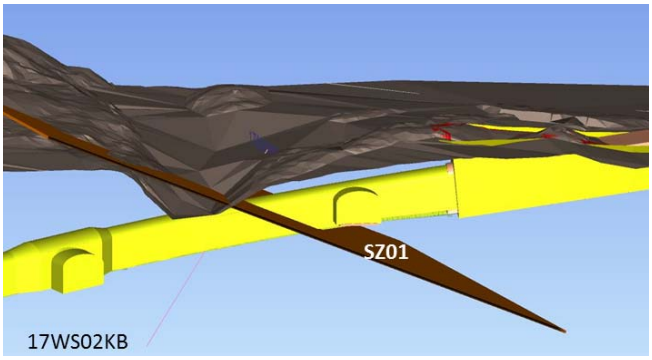


Fig. 3 Location of the low overburden and unfavourably orientedweakness zone relative to the access tunnel.

Due to the low overburden and the unfavourable orientation of the weakness zone relative to the excavation direction, excavation is expected to be difficult.

The initial design of bolts, shotcrete, and reinforced shotcrete arches is performed using the Q-system support chart from Chapter 2.1. The Q-chart indicates the need for reinforced shotcrete arches/ribs (RRS1), with a single layer of 3 rebars, each 16-20 mm in diameter, and covered with 250 mm of shotcrete. The c/c spacing between each arch was determined to be 2.8 m.The Q-chart does not specify requirements for spiling bolts; this decision should be based on experience. Low overburden, the quality of the rock mass, and the unfavourable orientation of the weakness zone restrict the development of a stable natural rock arch above the tunnel roof. This can potentially cause substantial deformation and rapid rock-mass failure during excavation. Spile bolts are designed as a temporary pre-support system to ensure the stability of the tunnel roof perimeter until permanent rock support is installed.

Furthermore, the design with a structural approach: a linear elastic beam on an elastic bed (beam-spring method), assumes that all rock pressures are transmitted from spiles to the rock tunnel support/lining, so the external load on the lining equals the support reaction from the spile system.

The sprayed shotcrete arches are designed to behave like a reinforced concrete T-beam: the 115 mm shotcrete layer acts as the flange, while the remaining 135 mm shotcrete serves as ribs. Both work monolithically due to the high-quality shotcrete. The shotcrete has compressive strength C32/40 and steel fibre reinforcement (MAMB). The influence of steel fibres is taken into account when assessing the shear and bending capacities of shotcrete arches, Fig. 4.

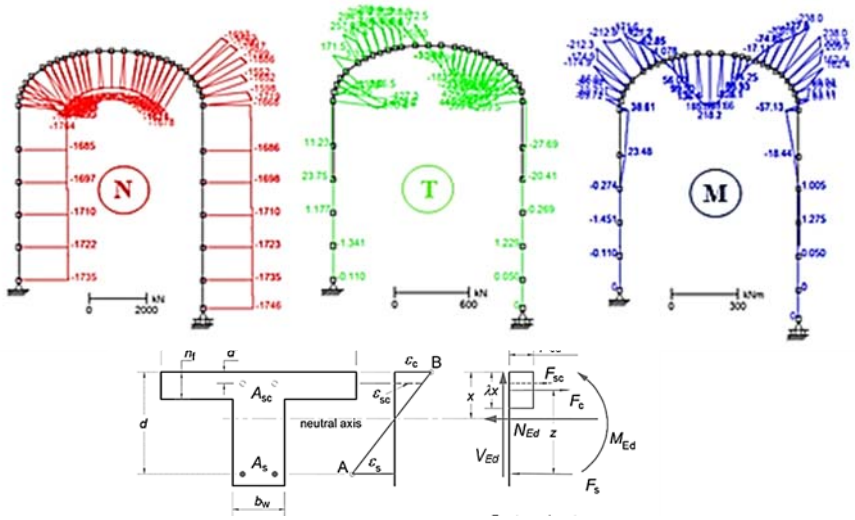


Fig. 4 Distribution of internal forces in a single-shell tunnel lining analysis (beam-spring method) and illustration of sprayed shotcrete T-section in bending and shearing at the ultimate limited state.

A single-layer tunnel lining was analysed using 3D FEM with an implicit continuum model in PLAXIS 3D(Bentley). The model was 61 m long, 46 m wide, and 41 m high, discretised with 10-node tetrahedral elements ranging from 0.02 to 6.40 m. Boundary conditions allowed free movement along the vertical sides, while the bottom boundary was fixed and the top remained free. Shotcrete was

modelled as a linear-elastic material using six-node beam elements with six degrees of freedom per node. The model included both the typical rock support section and a segment reinforced with sprayed-concrete arches (Fig. 5). Bolts and spiles were modelled as linear-elastic materials using three-node beam elements with six degrees of freedom per node.

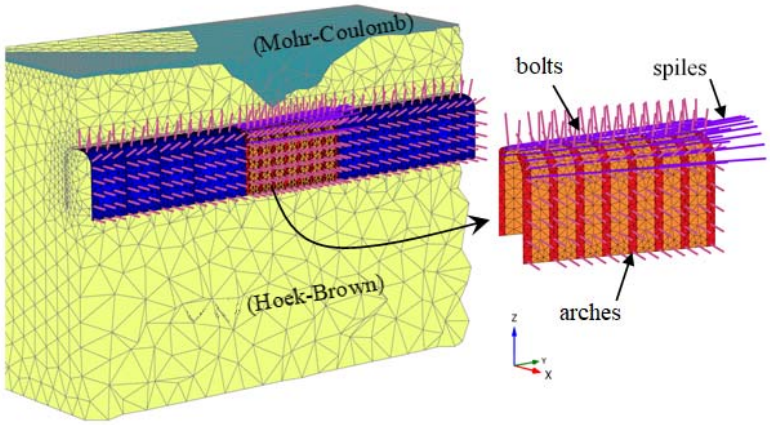


Fig. 5 Location of the low overburden and unfavourably oriented weakness zone relative to the access tunnel.

Excavation and support installation were simulated in 78 construction phases, with advance lengths of 2.0–5.0 m per phase. The rock mass displacements are such that small, limited-height sliding blocks form above the tunnel, allowing the stable rock arch to develop sufficiently (Fig. 6). The largest displacements occur in rock mass depressions, ranging from 0.207 to 0.472 mm.

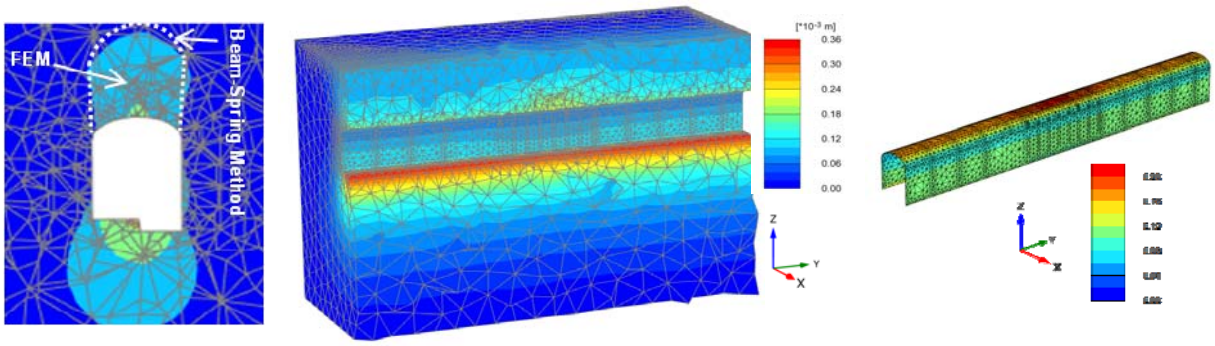


Fig. 6 Contours of the collapse zone are shown by the Finite Element Method (FEM) and Spring-Beam Method on the left. On the right, the displacement of the rock mass and lining after the entire tunnel excavation.

Finally, the self-drilling R51N spiles are designed to be 15 m long, spaced 0.6 m apart.After installation and excavation, sprayed shotcrete arches are constructed to transfer loads from the spiles. First, a 115 mm layer of shotcrete (typical support) is applied, then strengthened with 235 mm shotcrete arches spaced 2.0 m apart, reinforced with 5×RØ25 mm steel bars.In total, seven excavation stages are reinforced with shotcrete arches.

3.2 Flemingsbergstunneln, Tvärförbindelse Södertörn

Flemingsbergstunneln is a 3.1-kilometre tunnel on road 259 Tvärförbindelse Södertörn, between Kungens kurva and the Jordbro interchange in Haninge. The current road 259 is an accident-prone stretch that will now be rebuilt and become a safe road for all road users. The tunnel consists of two tunnel tubes with two lanes in each direction, separated by a 12-meter-wide rock pillar.

The rock mass is mainly made of sedimentary gneiss, with subordinate bands of gneiss-granite, meta-vulcanite, and metabasite. The gneiss has a medium-to-coarse grain size. The rock's structure rangesfrom indistinct to foliated and veined.

The tunnel passes under an existing stormwater tunnel at km 9/693. Road tunnel tubes are trending to the east-south (ES), striking 136° and dipping 2°. The existing stormwater tunnel is trending west-south (WS), i.e., striking 241°. The distance between the designed road tunnel tubes and stormwater tunnel is determined to be 3,00 – 4,30 m. Stormwater tunnel tube has an oval—egg—shaped cross-section, ca 4,50 m wide and ca 4,40-4,70 m high.

The rock class ranges from fair to poor, with $31 \leq RMR_{bas} \leq 45$. Rock mass is with closely spaced discontinuities, which fracture intact rock over small distances.

The designed rock support consists of a typical rock support layer of sprayed concrete (t = 150 mm), located at a specific distance (L = 2.60 m) along the tunnel axis, and additionally strengthened by sprayed concrete arches/ribs (b x h = 600 x 150 mm), Fig. 7. In addition to the arches/ribs, regular rebars are designed to be mounted on brackets and attached to the rock ground using short bolting (L=1.50 m). The short bolts are at a c/c distance of 2,60 m.

Similar to Chapter 2, the rock support is first verified by structural analysis using the beam-spring method and then further investigated by 3D numerical analysis in Plaxis 3D. The full water pressure in the stormwater tunnel is applied in the analysis.

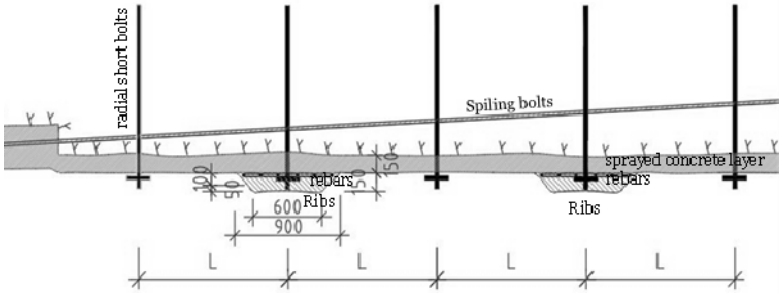


Fig. 7 Sketch of tunnel longitudinal section secured by the spiling bolts, sprayed concrete layer and arches together with short radial bolts.

The Plaxis 3D model measures 86 m × 92 m × 53 m (Fig. 8). The model size is reduced to minimise the likelihood of generating fault-mesh elements and to reduce calculation time.

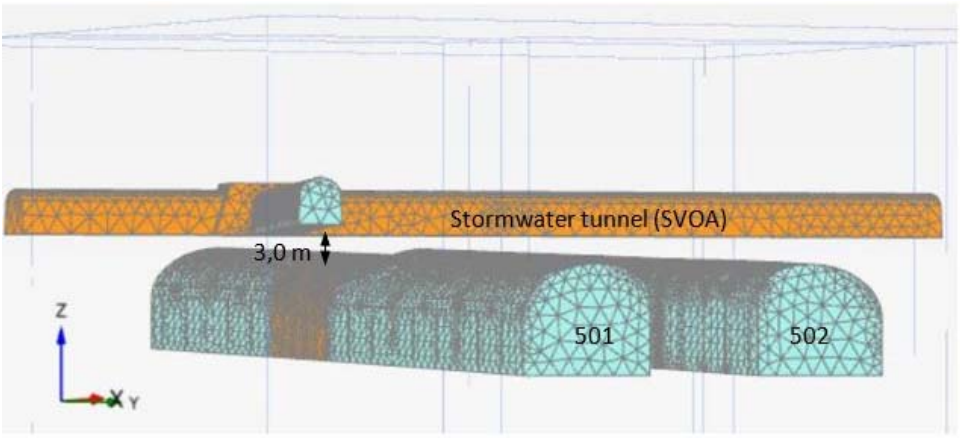


Fig. 8 A 3D finite mesh model for the analysis of the tunnels 501/502 and the existing stormwater tunnel.

The extent of the plastic zone along the periphery of the tunnel walls is limited to the area of 2,0 – 2,42 m. The maximum deformation is observed in tunnel 501, reaching 3,86 mm. Deformations produce critical strains of ε = 0,05 % and are not expected to produce tunnel stability problems.

Bending moments (M), Shear forces (Q), Axial thrusts (N) are maximised at 12,17 kNm, 189,19 kN. and 685,932 kN. A scatter plot of all internal forces on support capacity plots is presented in Fig. 9.

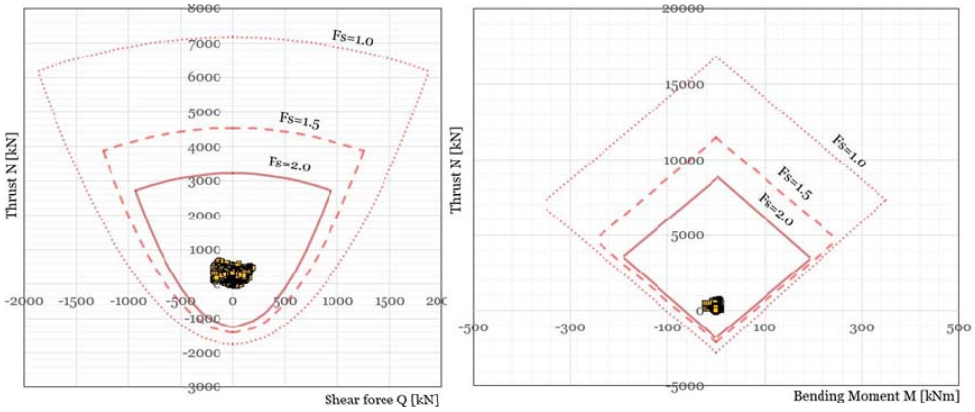


Fig. 9 Support capacity diagrams of shotcrete reinforced ribs.

4 Conclusion

The paper presents case histories from Scandinavian practice on the design of reinforced sprayed concrete arches as a single-layer tunnel lining in complex geotechnical conditions—such as low overburden, poor ground quality, and proximity to objects.

In the above-mentioned practice, it is not clear whether a rock mechanical or a structural engineer should be assigned to the design of shotcrete arches. The author's opinion is that this should be done by personnel with knowledge of both principal structural and rock mechanics. A clear methodology connecting analysis, empirical analysis, and numerical analysis was described, leading to the final design. Reinforced shotcrete arches are designed as load-bearing structures, considering their interaction with the rock mass. There is a critical question about whether shotcrete arches and the rock mass deform in the same way, i.e., whether there is full interaction between the shotcrete arches and the rock mass.

Further work on this topic should examine these questions, as well as the requirements for sprayed concrete arches combined with spiles and the exact mechanism of load transfer.

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