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Engineering geology in bridge design: Tower foundation stability – New Sotra Bridge

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Abstract
Norway has a rich tradition of suspension bridge design and construction, partly owing to the hard rock conditions playing on the strengths of this type of structure. Referring to the latter, the anchorage of the main cables in rock caverns is by many perceived as the main rock engineering task in suspension bridge design. However, the design and construction of a suspension bridge involves numerous areas requiring engineering geological competence. This paper highlights one of these areas based on the experience gained by Norconsult's engineering geologists during the design and construction of the New Sotra Bridge and Drotningsvik Portal. In addition to the structural geological conditions due to the proximity to the Drotningsvik Fault Zone (DFZ), the architectural and structural design of both the suspension bridge and tunnel portal has entailed engineering geological challenges not previously undertaken in Norway.

The stability of the southern tower foundation in Drotningsvik has been a key topic during the entire project. In the detail design phase the tower foundations were shifted onshore compared to the tender design as much as the zoning plan allowed. Several potential geological stability problems were highlighted in the detail design report, with emphasis on the construction phase residual risk. Accordingly, it was recommended to perform supplementary rock core drillings during construction to facilitate further geological modelling and stability verification. This paper elaborates on the process of verifying the tower foundation stability in the execution phase, i.e. planning of additional ground investigations, follow-up and analysis of all findings.

These additional ground investigations comprised engineering geological surface mapping, rock core drilling/logging, optical and acoustical televiewer investigations, crosshole seismic tomography and drone photogrammetry. All investigation results have contributed to the making and interpretation of a 3D-model (Leapfrog) of the structural geology used as a basis for stability analysis and rock reinforcement design.

Keywords
Engineering geology, suspension bridge, ground investigations, foundation stability, Leapfrog

1 Introduction

1.1 Project description

Rv. 555 Sotrasambandet is a road construction project in Vestland, Norway. It involves the construction of a 4-lane, 608 m-span suspension bridge, the New Sotra Bridge, crossing the fjord between Øygarden and Bergen municipalities. Norconsult Norge AS has designed the New Sotra Bridge and Drotningsvik Portal on behalf of Sotra Link Construction JV ANS.

1.2 Background and purpose

The engineering geological detail design of the New Sotra Bridge started off in September 2021. A ground investigation campaign to supplement the previously conducted geological surveys was launched. During previous project phases it had become clear that additional investigation of the southern tower foundation area in Drotningsvik, on the east side of the fjord, was key to verify the overall geological stability of the bridge.

The detail design phase investigations in this area involved high-resolution bathymetric scanning, subsea ROV-video inspection and two oriented, subhorizontal rock core drillings (Figure 5). Based on the bathymetric scanning, it was concluded that the approximately 15-20 m high subvertical subsea rock slope in front of the southern tower foundation, along with the present joint sets, potentially could compromise the foundation stability. Consequently, it was decided to relocate the tower foundations further onshore. Further detail design works were then conducted implementing the oriented rock core drillings, dated March 2022, to verify the foundation stability at the new location.

During the detail design phase several possible structurally controlled stability problems were identified. The worst-case scenario involved observation of a potential failure plane in one of the two oriented boreholes (Figure 1). The observation correlated well with observations in non-oriented boreholes from previous project phases. The following comments were made in the design report:

- The orientation of the potential failure plane is notably rotated compared to the subsea rock slope in front of the foundation (> 20°). Planar failure is considered unlikely, and the presence of other joints to facilitate wedge failure is required to compromise foundation stability.
- The joint is observed only in one of the oriented boreholes. This could be explained both by large-scale undulations, the presence of intact rock bridges, or limited joint persistence.
- The joint surface conditions are favourable i.e. rough and undulating, with sandy coating, direct rock-wall contact, fresh rock in proximity of the joint, and no clay- or intact rock alteration.
- The joint, as modelled based on the conducted investigations, does not intersect the subsea rock slope in front of the foundation. However, there is a residual risk related to this comment.

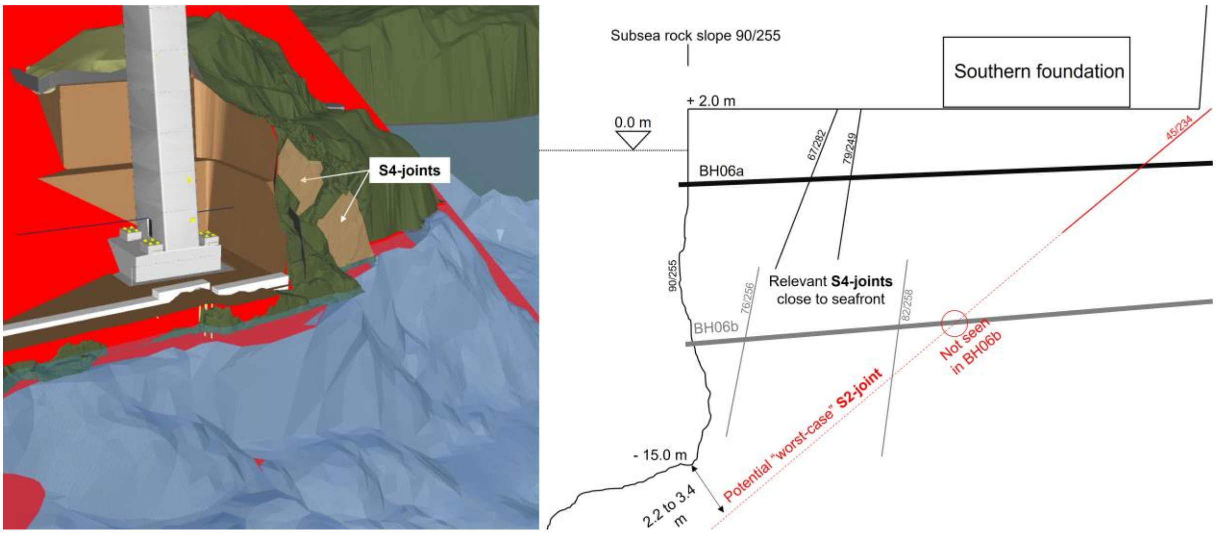

Figure 1: Left: Model excerpt showing the worst-case S2-joint in red, and dominant S4-joint surfaces close to sea. Right: Longtudinal section through BH06a showing the worst-case S2-joint and related S4-joints close to sea. A selection of joint registrations from the rock core logging are drawn to highlight their importance.

The following prerequisites were made in the final version of the detail design report, dated December 2022, to address the residual risk:

- The potential failure plane is to be given particular emphasis while performing geological surface mapping of the rock cuts around the tower foundation during construction.
- Rock core drillings during construction are recommended to verify the plane's properties, and to facilitate further 3D-modelling of the structural geology. The plane's potential intersection with the subsea rock slope in front of the foundation is of particular interest.
- The construction phase geological surface mapping is to focus on the presence of other joints that may facilitate wedge failure, as the most practically relevant kinematic failure mechanism.
- In case the worst-case scenario is identified during construction, further evaluation of lowering the foundation level by means of shaft excavation, along with the detail design of comprehensive passive rock reinforcement, are considered the most suitable measures to ensure the overall geological stability of the bridge.

The purpose of this paper is to elaborate on the engineering geological works performed during construction to verify the foundation conditions for the southern tower foundation in Drotningsvik.

1.3 Stages of construction phase

During the construction phase the verification works were divided into the following stages. The borehole locations are shown in Figure 5.

1) Aug.-Sept. 2023 (el. +33-28): Identification of S2-joints with significant (2-20 cm) sandy, silty, clayey joint filling upon starting excavation, adapting the rock cut design accordingly. One supplementary rock core drilling (BH-16) with optical (OTV) and acoustical televiewer (ATV) was conducted to evaluate rock mass conditions regarding the total stability of the designed rock cut.

2) Oct. 2023 (el. +22): Geological surface mapping of the excavated horizontal rock surface at el. +22. Identification of several S2-joints with significant joint filling. Swelling clay was confirmed by laboratory testing. These were labelled (e.g. "X-PLANE", "Y-PLANE" and "Z-PLANE" with reference to Figure 2) as the primary basis for geological 3D-modelling (Leapfrog). Rock cut design was adapted to include excavation along a dominant S2-joint ("Z-PLANE").

3) Dec. 2023 (el. +12): Three supplementary rock core drillings (BH-14, BH-15 and BH-17) with OTV and ATV were conducted as basis for verifying the foundation stability. The boreholes corroborated the geological surface mapping which identified two S2-joints with the potential of compromising the foundation stability ("Y-PLANE" and "X-PLANE"). The project deemed it appropriate to initiate design works for an alternative back-up design involving lowering of the southern tower foundation level by means of shaft excavation.

4) A) Feb.-Mar. 2024 (el. +2): Geological surface mapping at el. +2 indicated more favourable conditions than at higher elevations. One of the two S2-joints ("Y-PLANE") exposed a too low persistence to compromise foundation stability. The other S2-joint ("X-PLANE") was more persistent in the horizontal direction. However, only a low degree of joint alteration was observed. Partly the joint was noted as "sealed".

   B) Mar. 2024 (el. +2): Eight supplementary boreholes (NGI-BH01-8) were drilled. In all holes OTV and ATV surveys, and cross-hole seismic tomography, were conducted. No clear correlation with any relevant S2-joint was found.

   C) Mar.-Apr. 2024 (el. +2): The seafront was evaluated and found to require rock reinforcement to ensure the permanent foundation stability at el. +2. This was done first and foremost to avoid any propagating failure reaching the foundation area. This way, the required rock mass confinement, assumed for bearing capacity considerations, was ensured. The intactness of the seafront is also a principal assumption for ship collision evaluations.

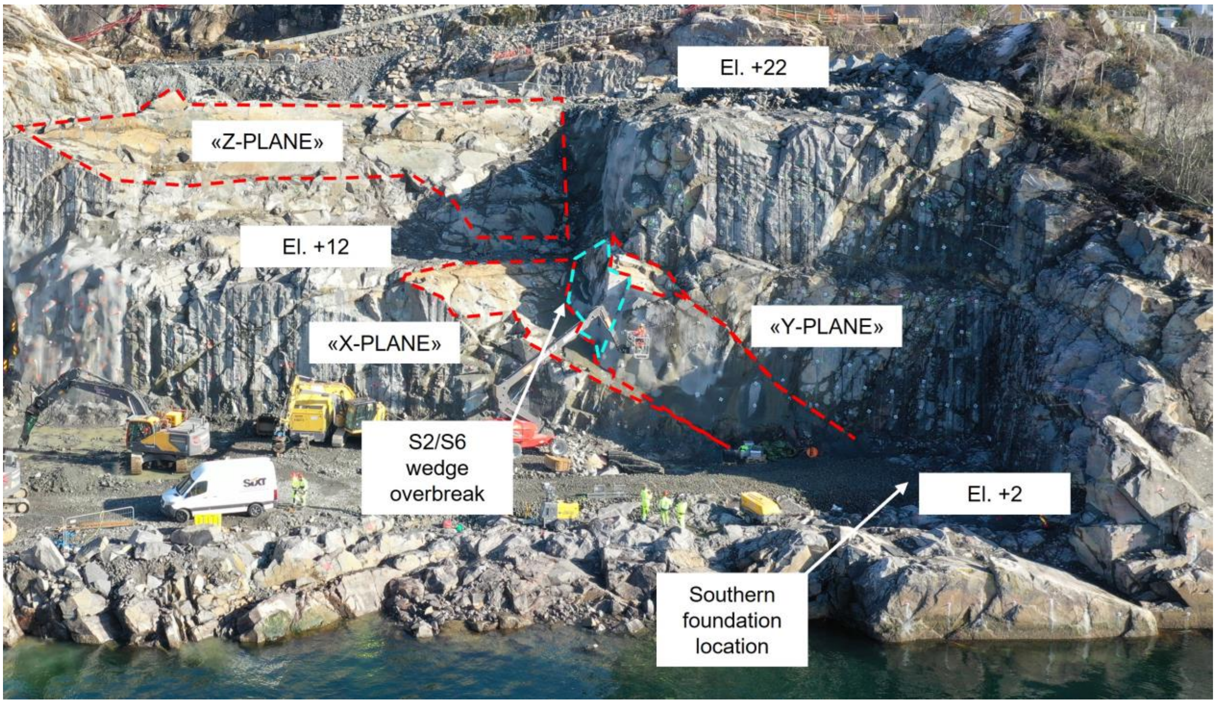

Figure 2: Drone photo of the tower foundation area in Drotningsvik. Upon excavation reaching el. +22 (stage 2)) several dominant S2-joints were labelled (e.g. "X-PLANE", "Y-PLANE" and "Z-PLANE") for reference. These joints were found to be persistent from el. +22 to +2, but with varying persistence below el. +2 based on borehole investigations.

2 Rock mass conditions

2.1 Structural geology and rock mass classification

The joint sets of Drotningsvik (west of the Drotningsvik Fault Zone (DFZ)) are shown in Figure 4. The televiewer investigations conducted during the construction phase verified the joint sets described during the detail design phase.

The rock surface beneath the foundation consists of competent, fresh gneissic rock. The rock mass is moderately to slightly, but complexly, jointed. The joints are rough, and varies from planar to undulating. Most joints appear slightly altered, apart from clay fillings observed along some distinct joints belonging to joint sets S2, S4 and S5. The following observations from el. +2 are highlighted:

- A distinct S2-joint labelled as the "X-PLANE" was visible in the eastern rock cut between el. +12 and +2 (Figure 2), and at the el. +2 rock surface (Figure 3). The position and orientation of this structure, and its influence on the foundation stability, were investigated further based on the 3D-model and the results from ATV/OTV.

- Another distinct S2-joint labelled as the "Y-PLANE" was also visible in the eastern rock cut between el. +12 and +2 (Figure 2). The plane daylights in the foot of the rock cut, but ceases a few metres north of the foundation at the el. +2 rock surface (Figure 3).

- A distinct S5-structure with increased joint density and sporadic clay fillings crossing the foundation from SSE to NNW was also observed (Figure 3). The structure was anticipated based on borehole observations, but was not observed in the northern part of the foundation.

- Numerous S6-joints were registered both in boreholes and during geological mapping of el. +2. These joints are in general less persistent than the other joint sets. Only three S6-joints with persistence in the order of magnitude 5-15 m were mapped (Figure 3).

- Subvertical joints belonging to joint set S4 striking N-S detach the outer part of the el. +2 rock surface from the foundation area (Figure 3). These joints appeared as open joints from approx. 7-8 m from the seafront. It was assumed that the observed joint openings are product of destressing along the Drotningsvik seafront and surface weathering over time.

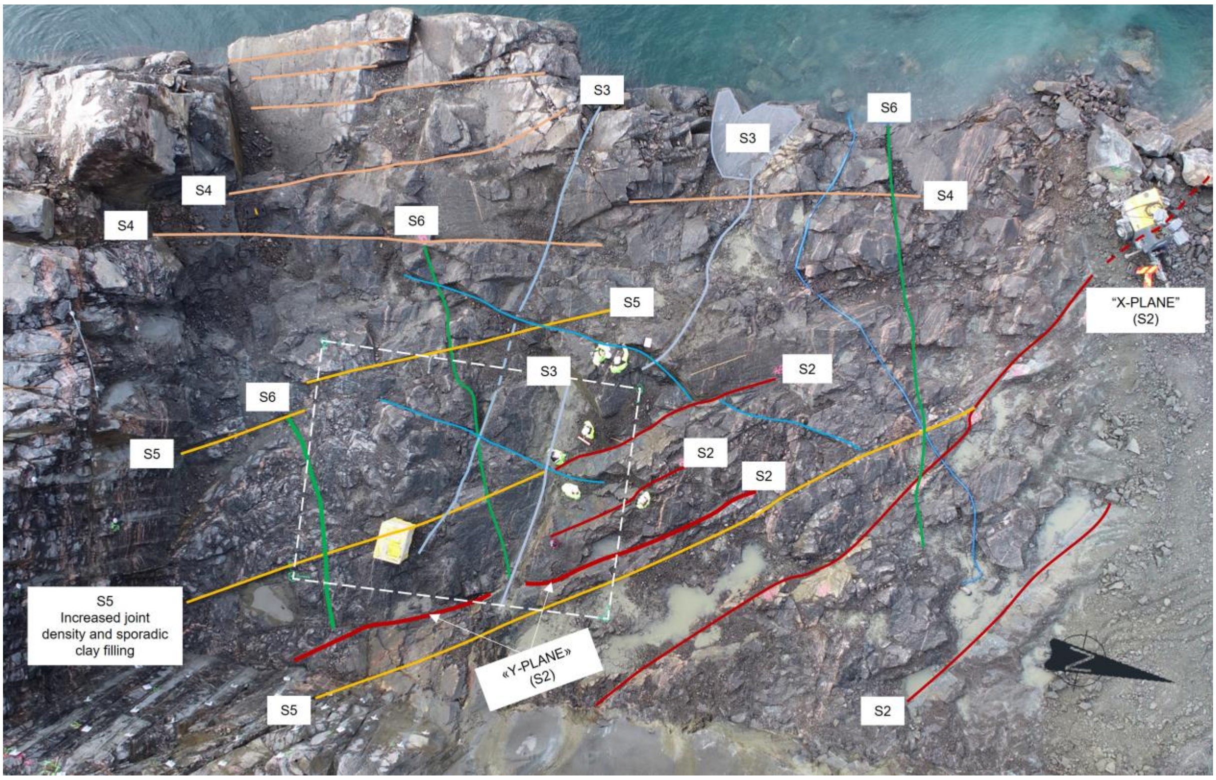

Figure 3: Drone photo showing the foundation area mapped. In colour and labelled the major joints encountered. Note, different coloured lines correspond to the daylighting line of the joints. The white dashed rectangle indicates the southern foundation area. Refer to stereographic projection in Figure 4 for joint set dip and dip direction.

Table 1: Rock mass parameters gathered during field mapping, core logging and laboratory testing. *Value for gneiss from RSData. **Effective cohesion c' calculated under normal stress levels corresponding to a subsea rock slope height of 15 m.

	Worst-case	**Average**	**Best-case**
RQD	60	80	100
Jn	15	13.5	12
J_r: S2 (S6)	1.5 (3)	2.25 (3.5)	3 (4)
J_a: S2 (S6)	8 (3)	6 (2)	4 (1)
GSI	45	55	70
σ_{ci}	145 MPa	250 MPa	375 MPa
m_i*		28	
D		0.7	
σ_{ti}	15 MPa	18 MPa	21 MPa
γ'		17 kN/m^3	
σ_{cm} (σ_c)	21.9 MPa (2.5 MPa)	50.9 MPa (9.3 MPa)	119 MPa (42.4 MPa)
ϕ_{S2} (ϕ_{S6})	11° (45°)	21° (60°)	37° (76°)
c_i	23 MPa	34 MPa	44 MPa
c'**	0.30 MPa	0.80 MPa	3.7 MPa
c'/c_i	1.3 %	2.4 %	8.4 %

2.2 Estimation of rock mass compressive strength (σ_{cm} and σ_c)

The high ground pressure (> 7 MPa), caused by tensioning of the rock anchors integrated in the foundation to prevent tower overturning, required the verification of the rock mass compressive strength σ_{cm} (Hoek et al., 2002). This was calculated using RSData with input as presented in Table 1. Prior to installation of the rock reinforcement as described in ch. 4 the ground pressure evaluations were conducted using the unconfined rock mass compressive strength σ_c (Hoek et al., 2002).

2.3 Evaluation of failure mechanisms

It was verified that planar failure in front of the foundation could be excluded as possible failure mode, due to the relatively high angle between the strike of joint set S2 and the subsea rock slope (> 30°). The most relevant kinematic failure mechanism was confirmed to be wedge failure facilitated by joint sets S2 and S6. During excavation of rock cuts parallel to the subsea rock slope the geologically controlled overbreak was governed by this kinematic behaviour (Figure 2). In the rock cuts, joint sets S4 and S5 were seen to act as tension cracks forming a more complex, but smaller, failure volume.

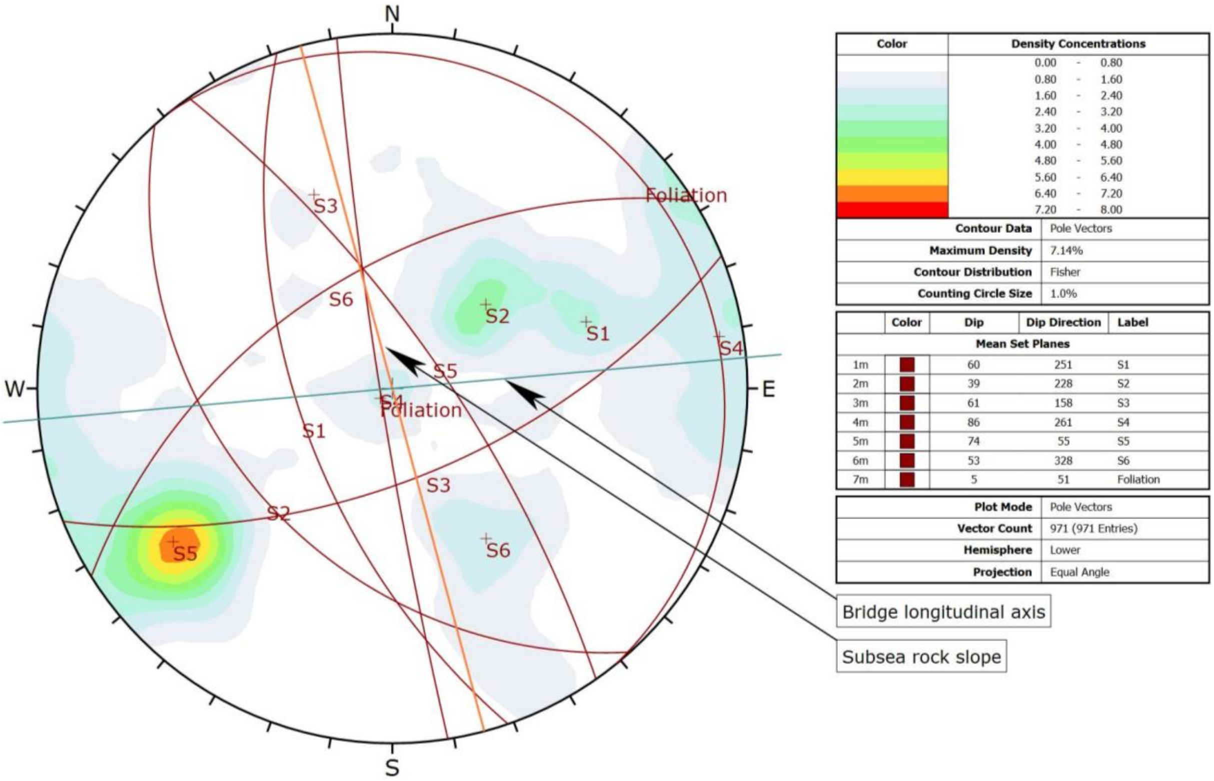

Figure 4: Stereographic projection of joint data from both detail design phase and construction phase collected in Drotningsvik (west of DFZ). Includes data from field mapping, oriented cores, TV-investigations and drone photogrammetry.

2.4 Estimation of joint friction angle ϕ

Based on the relevant kinematic behaviour of the rock mass the friction angle ϕ of two different joint sets (S2 and S6) were estimated using the relationship between joint roughness and joint alteration as suggested by NGI (2022). Table 1 presents the varying J_r- and J_a-values observed during surface mapping, core logging and televiewer investigations, as well as their impact on the friction angle ϕ.

2.5 Estimation of rock mass effective cohesion c'

The estimated rock mass effective cohesion c' accounts for the fact that the S6-joint system is non-continuous. Any large-scale displacement in the direction of S6 would require both displacement along open joints belonging to different joint sets, and failure of intact rock. c' was estimated using a Mohr-Coulomb fit under relevant normal stress levels as described by Hoek et al. (2002). Upon estimating c', this value was compared to the calculated cohesion of intact rock c_i (Barton, 1976), to determine the relative contribution of intact rock cohesion to shear strength (Table 1).

3 Evaluation of structurally controlled total stability

3.1 Geological modelling using Leapfrog

The georeferenced structural measurements from the conducted ground investigations were implemented to create a Leapfrog-model incorporating the most prominent geological structures in proximity of the foundation. The modelling was initiated by stage 2) (ref. subchapter 1.3). Then, the modelled joints were calibrated with observations in the core drillings from the detail design phase. Upon completion of the respective TV-investigations conducted during excavation the plane fits were continuously revised. Lastly, the planes were updated based on field mapping and drone photogrammetry of the foundation surface at el. +2, and the televiewer data from stage 4).

3.2 Borehole interpretation

The crosshole seismic tomography indicates seismic velocities varying from 4500-6500 m/s corresponding to good to very good rock mass quality. The most prominent areas of relatively low velocities correspond to a modelled S2-structure observed during stage 2) labelled as the “Z-PLANE” (Figure 5). Structural readings in the televiewer- and core logs corresponding to the “Z-PLANE” were observed in seven of the fourteen boreholes. Nevertheless, following the model the “Z-PLANE” is located deep below the subsea slope without the potential to compromise foundation stability.

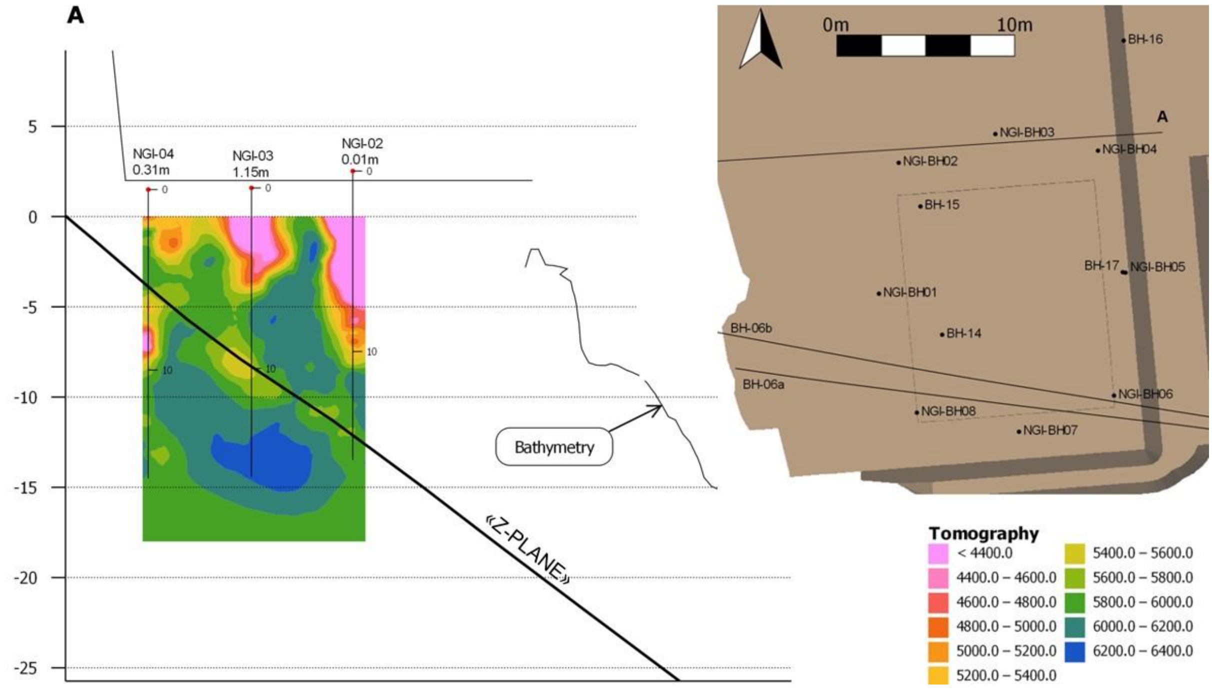

Figure 5: Boreholes conducted in proximity of the foundation. BH-14, -15,-16 and -17 are core drillings with ATV/OTV. BH-06a and BH-06b are oriented, subhorizontal core drillings seen from above. NGI-BH1-8 are ordinary boreholes with ATV/OTV and cross-hole seismic tomography. The dashed rectangle is a recess in the excavation plan from el. +2 to el. +1.675 with 1.5 m offset from the foundation. The “Z-PLANE” interpreted from borehole investigations is shown.

It was not possible to correlate different observations of S2-joints from the respective boreholes, apart from the “Z-PLANE”, into continuous structures. The “X-PLANE” modelled based on geological surface mapping only corresponds with a few structural measurements in the boreholes, and there were very few S2-observations in the proximity above and below the modelled “X-PLANE”. The latter, clearly visible in the rock cut between el. +12 and +2 (Figure 2), is observed to be non-continuous below el. +2 based on extensive borehole investigations.

3.3 Limit equilibrium analysis

A limit equilibrium method (LEM) analysis in Swedge, implementing Eurocode 7 Design Approach 3 (Standard Norge, 2020), was conducted to address the residual risk of the “X-PLANE”-continuity by means of calculation. The orientation of the S2-joint was chosen based on the orientation of the “X-PLANE” as observed in the rock cut between el. +12 and +2, and at the rock surface at el. +2. The orientation of the S6-joint was chosen based on the sample mean orientation.

Two different material models, worst-case and average as presented in Table 1, were used as a basis for sensitivity analysis. Note that c' was only included for the S6-joint. Numerous external loads were considered in the analysis, hereunder seismic loads, water loads (joint water pressure and ponded water load on the slope face), loads from permanent landscaping, and structural loads. The latter included both construction phase loads and 12 nos. operation phase load combinations.

Five different analysis cases were studied. Firstly, a base case (A) with no partial factors, no external loads and no cohesion was analysed for calibration. Thereafter, four additional cases (B-E) were studied as shown in Table 2. The cutting of the wedge by subvertical joints below the foundation was found to be relevant for S5-joints (Figure 3). The worst-case structural load combination was found for the entire wedge, and this load combination was then included in an analysis with a sample mean oriented S5-tension crack at the minimum R_d/F_d-location for analysis cases D and E.

Table 2: Analysis cases and results from limit equilibrium analysis of wedge failure facilitated by joint sets S2, S6 and S5.

Case	Partial factors	Operation phase loads	Construction phase loads	Seismic loads	Landscape loads	c'_{S6}	Min. R_d/F_d Worst-case	Min. R_d/F_d Avg.
A							0.77	1.23
B	X					X	2.46	5.92
C	X		X			X	1.27	2.57
D	X	X			X	X	1.06	2.05
E	X	X		X	X	X	1.03	2.00

4 Rock reinforcement evaluations

Based on the observed conditions at el. +2, as well as observations from ground investigations and ROV-video, rock support to ensure the stability of the seafront was deemed necessary. Particular emphasis was put on the rock mass in front of the open S4-joints separating the competent rock mass beneath the foundation from the subsea slope (Figure 3). It was considered important to ensure that this volume remains intact and stable during the entire lifespan of the bridge, as it provides confinement of the loaded rock mass beneath the southern foundation. As the area is not subjected to any direct loading from the permanent bridge structure, and was evaluated to be independent of the total stability of the foundation area, passive rock reinforcement was considered appropriate. However, the rock bolts should interact to ensure an evenly distributed confining pressure upon potential deformation. The latter was ensured by integrating each bolt in a casted reinforced concrete beam.

A complete 3D-model of the rock reinforcement structure was issued for construction (Figure 6). The rock bolt and -anchors were designed based on prescriptive measures, i.e. a best-practice approach for designing confining rock reinforcement for foundations. The stability of the mentioned area was then confirmed by calculation (LEM) implementing the designed rock reinforcement.

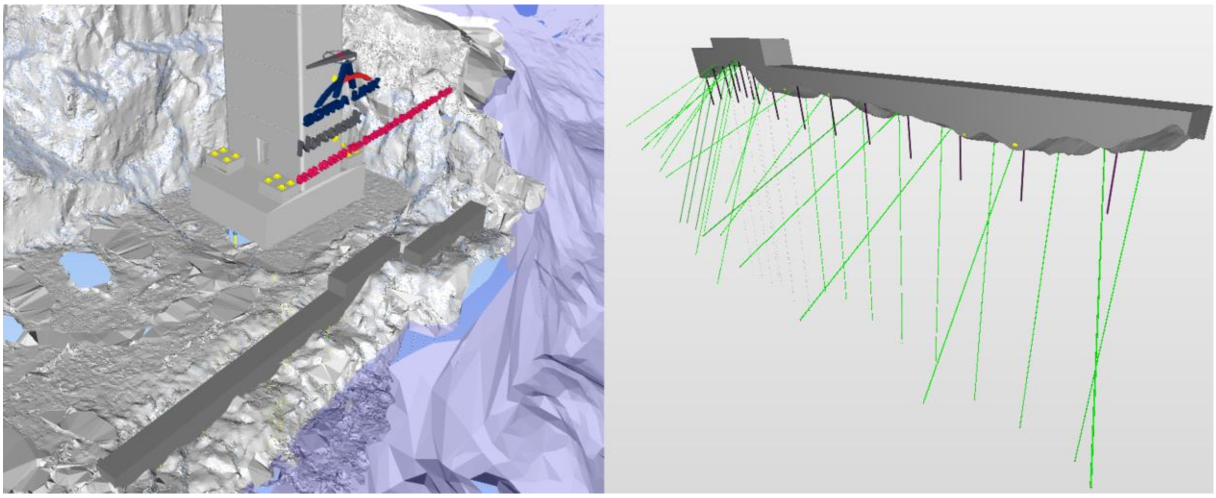

Figure 6: Model excerpts. Left: The southern tower foundation in Drotningsvik along with the reinforced concrete beam. Right: Detail of the rock bolts/-anchors integrated in the reinforced concrete beam.

5 Conclusive remarks

In summary, the stability of the New Sotra Bridge southern tower foundation in Drotningsvik has been successfully verified. The on-site verification works of the foundation level defined during the detail design phase have highlighted the value of construction phase ground investigations combined with geological 3D-modelling to avoid late-hour design changes involving significant residual risk. In this case, the latter refers to lowering of the foundation level by means of shaft excavation.

Moreover, the residual risk related to interpretation of the ground investigations was addressed by means of calculation. The remaining risk is further addressed by the installation of a borehole inclinometer to monitor the rock mass deformation during the continuing bridge construction works.

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