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Site tests on retaining systems and uplift anchors of Nordhavnstunnel in Denmark

Essais d'un système de soutènement et d'ancrages de soulevement pour le Nordhavnstunnel au Danemark

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ABSTRACT: The Nordhavnstunnel, located in the Nordhavn region of Copenhagen, Denmark, extends the existing Nordhavnsvejtunnel over 1,4 km, passing below the Svanemøllehavn and in the Nordhavn area. It is designed as a rectangular reinforced concrete cut and cover tunnel constructed between anchored retaining walls. A successful design and construction in the challenging location and ground conditions require a series of preliminary sheet pile driveability tests and anchor investigation tests. This paper presents a comprehensive summary of the test results, offering valuable insights and recommendations. Furthermore, it delves into the implications of these findings on the updated design of the tunnel's retaining system and uplift anchors. The conclusions drawn from this study provide crucial insights pertinent to the Nordhavnstunnel but also offer valuable lessons for future projects in Copenhagen encountering similar geotechnical conditions

RÉSUMÉ:. Le Nordhavnstunnel, situé dans la région de Nordhavn à Copenhague, au Danemark, s'étend sur plus de 1,4 km depuis le tunnel existant de Nordhavnsvejtunnel jusqu'à la mer, passant sous le Svanemøllehavn et la zone de Nordhavn. Il est conçu comme un tunnel rectangulaire en béton armé construit entre des murs de soutènement. La conception et la construction à cet emplacement et dans ces conditions géotechniques difficiles nécessite une série de tests préliminaires de faisabilité d'enfoncement de palplanches et d'ancrages. Cet article présente un résumé des résultats des essais, offrant des informations précieuses et des recommandations. De plus, il explore les implications de ces résultats sur la conception mise à jour du système de soutènement du tunnel et des ancres de soulèvement. Les conclusions tirées de cette étude fournissent des informations cruciales qui ne sont pas seulement pertinentes pour le Nordhavnstunnel, mais offrent également des leçons précieuses pour les projets futurs à Copenhague rencontrant des conditions géotechniques similaires.

Keywords: Road tunnel; Retaining wall; Uplift anchor; Limestone; Investigation test

1 INTRODUCTION

The new Nordhavnstunnel extends the existing Norhavnvejtunnel to the east in the Nordhavn region of the Copenhagen, Denmark. The project owner is Danish Road Directorate (Vejdirektoratet) and design and build contract is awarded to Besix & MT Højgaard joint venture with design support from Niras (all permanent works), Jacobs UK (M&E design) and Besix Engineering Department (all temporary works).

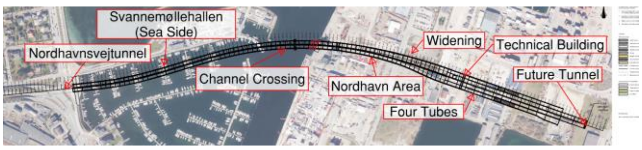

Figure 1. Plan view of the Nordhavnstunnel

The tunnel is 1.4 km long. It is connected at its west end to the existing Nordhavnsvejtunnel. A 800 m long part of the tunnel passes under the existing harbour of Svanemøllen and crosses the access channel to the Nordhavn quays. After construction, the harbour and the access channel will be reinstated. The channel must maintain a width of 40 m all along the construction period. East of the channel crossing, the tunnel continues under the Nordhavn Area, which is intended to be further developed in a nearby future. The Nordhavnstunnel will be extended by a future Eastern tunnel that will connect the Nordhavn area to the Copenhagen airport.

The section in the Svanemøllehavn and channel crossing is a two-tube rectangular section of constant width. East of the harbour and the channel crossing, in the Nordhavn area, the tunnel widens to consist of four tubes: the two outer tubes will act as a ramp for Nordhavn area, and middle two tubes will be kept as blind tunnel for connection to Eastern Ring Road.

The clearance height between the bottom and slab is 6.65 m, allowing for a 4.65 m high traffic clearance profile, 1 m high mechanical and electrical installations profile above traffic and a pavement and ballast concrete layer of about 1m. The width of the tunnel tubes starts from 9 m and increases up to 19 m at the widening section.

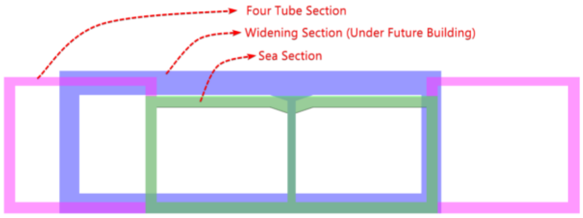

Figure 2. Various Cross-section used in Nordhavnstunnel

The project includes the design and construction of two large pumping stations along the tunnel as well as a main technical building located on top of the tunnel in the Nordhavn area, as well as various road infrastructure and upgrades of existing tunnel structures and buildings.

The geotechnical conditions at the project location consist of following (simplified) layering (from bottom to top):

- Copenhagen Limestone at starting at level -12 to -16 m DVR 90. The limestone is highly indurated in its upper meters, low fractured except at the east end of the project due to the presence of the Svanemolle fault zone.
- Greensand, thickness 2 to 3 m. The Greensand is not found in the Nordhavn area.
- Glacial till, of quite variable thickness 4 to 8 m over the project location. The glacial till can be described as a clayey silt to silty clay.
- Interbedded layers of melt sand in the glacial till at Nordhavn
- The top 2 to 4 m at Nordhavn consist of man-made sandy and very heterogeneous fill

The values of the geotechnical parameters are shown in the Table 1 and Table 2.

Table 1. GIR Parameters (simplified mean value - ½ std dev).

Soil type	γ_b kN/m^3	S_u kPa	φ'_{pl} °	c kPa	E_{50} MPa	E_{ur} MPa	σ'_{pc} kPa
Existing fill -sand	20	-	32	0	25	75	0
Existing fill -clay	20	65	29	5	15	45	0
Clay till	22	400	33	35	30	90	>1000
Glacial Sand	20		37	0	30	85	>1000
Greensand	21	500	32	50	65	200	>1000

Table 2. Limestone properties (Hoek-Brown properties transformed into equivalent Mohr Coulomb properties).

Soil type	γb kN/ m^3	UCS MPa	GSI	mi	D	φ'_{pl} °	c kPa	E_{50} MPa
Limestone (passive)	22	15	65	7	0	45	100 (450)	1200
Limestone (vertical)	22	30	70	7	0	55	900	1200

** Limestone properties for vertical capacity were upgraded based on preliminary vertical bearing capacity estimations through dynamic testing (see discussion in 4.3).*

The limestone has a separate, potentially artesian, hydrological regime compared to the above layers.

The tunnel is secured against uplift by means of post tensioned bar anchors fixed in the limestone. 50 cm toe is used throughout the tunnel to increase the uplift resistance.

The tunnel is constructed by the cut and cover method, from a hydraulic fill forming a temporary working platform in the Svanemøllen harbour, and from the existing ground in the Nordhavn area. The tender documentation provided “for information” by the Road Directorate to the tenderers proposed a secant pile wall retaining system with multiple anchor levels. The depth of the secant pile wall was determined by the geotechnical and hydrological design to reduce the water discharge by dewatering. The proposal of secant pile wall was in-line with the retaining system that had been used for the Nordhavnsvejtunnel. The Contractor decided to adopt a sheet pile retaining wall system in combination with grouting to create a water cut off. This solution was considered to bring in significant productivity and potentially sustainability related benefits to the project. An indicative cross section of the excavation pit and the retaining wall system in the marine area is illustrated in Figure 3.

Risks related to driveability of the sheet piles through the clay till/greensand and in the limestone were identified and from the early design stages different scenarios with various penetration depths in the limestone were assessed. Early driveability testing was placed in the planning, to allow early risk mitigation and design improvement.

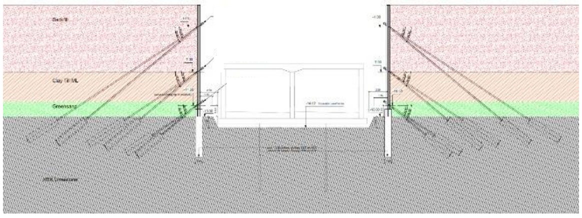

Figure 3. Temporary works cross-section in the marine area, illustrating the excavation pit: sheet pile wall with multiple anchor levels, grout injection, berm support of sheet pile.

This paper presents the results of the investigation tests of uplift anchors and sheet piles performed at the initial phase of the project. To optimize the design, a set of uplift anchor investigation tests, anchors with bond length in the clay till, and sheet pile driveability tests have been carried out. Additional test data and project information are also shared to document the various know-hows obtained during the early phases of the project.

2 UPLIFT ANCHORS

2.1 Design against uplift

Uplift checks of the tunnel have been performed as per the Client's Requirements and DS EN 1997. The requirements are summarized below for completeness.

Load combinations for UPL in Danish annex have been separated into two to consider the additional risks related to frictional resistance. If frictional resistance and tensile elements such as anchors are used, Table A.4-1 should be used: 0.9 x Favourable - 1.1 x K_{fi} x Unfavourable. If only self-weight is used to resist against uplift, the factors are lowered due to reduced uncertainty: 1.0 x Favourable - 1.05 x K_{fi} x Unfavourable (Water). Since a very conservative friction between concrete and backfill is included in all cases, only Table A.4-1 is used in Nordhavnstunnel. Therefore, the overall safety factor against uplift can be calculated as 1.35, considering K_{fi}=1.1 as per the Client's Requirements.

Additional requirements by Client includes future excavations in certain areas, artesian pressures that create additional water pressure from the bottom of tunnel, requirement of post-tensioning the anchors.

The typical failure mode of the anchors (see figure 4) has been checked throughout the design process.

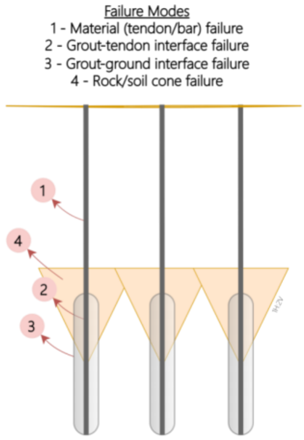

Figure 4. Anchor failure modes

Following criteria are considered for the definition of anchor lengths:

- All bond lengths should be embedded in the limestone with minimum of 1 m from the top of the limestone.
- Free length = Minimum 4 m

Since the anchors are embedded in the limestone, main design criteria become the material failure (failure mode 1). To be able to increase the anchor spacings, S670/800 MPa bar anchors are considered in the design. As per the DS EN 1997-1, material capacity is $R_{t;d} = A_{anchor} \cdot \min\{k_1 \cdot f_{pk}; k_2 \cdot f_{p0.1k}\}$. In this equation, k_1=0.8 and k_2=0.95. The steel bar should be able to carry the proof load. The proof load is determined as the 1.43 x design load. Therefore, the capacity $R_{t;d}$ is reduced by 1.43 to reach the design capacity. Due to conservative requirement on the $R_{t;d}$, the limitation as per the EN 1993-5 does not govern – as the requirement to check for proof load is only for $R_{t;d}$. With these assumptions, Ø50, Ø57.5 and Ø63.5 S670/800 anchors result in $R_{t,d}$ of 874 kN, 1156 kN and 1410 kN, respectively.

2.2 Investigation tests uplift anchors in limestone

The typical bond strength (referring to failure mode 3) assumption for Copenhagen limestone has been increasing over the years from 650 kPa to 900 kPa – the latter value is being used widely in the latest projects.

Considering that the number of anchors in Nordhavnstunnel is more than 700, even a slight increase in the capacity optimizes the project significantly. More importantly, ensuring that the problems during the construction and testing phase is critical, also considering that 10% of the anchors will be suitability tested, and acceptance test load is increased to same level with suitability test load, i.e. 1.43 x design load.

Uplift anchor tests have been carried out in two different locations on the land-side of the project. In total, 8 anchors were tested. The anchor data are summarized below:

Bond length = 3 and 5 m

Top of anchor – top of limestone = 1 m and 2 m

Strand anchor to reach the capacities.

Proof load: 3350 kN – corresponds to 2090 kPa adhesion for 3 m bond length.

Test Method 1 is used in Denmark. The interpretation method for anchor failure in Denmark is the creep rate method. The methodology is described in DS/EN ISO 22477-5:2018. The creep rate is the rate of increase in deformation under constant load with logarithm of elapsed time. As per the DS EN 1997-1, creep rate limits are 2 mm/log(min) for acceptance and suitability tests and 5 mm/log(min) for investigation tests. Based on this methodology, the success vs. failure will be implemented as shown below:

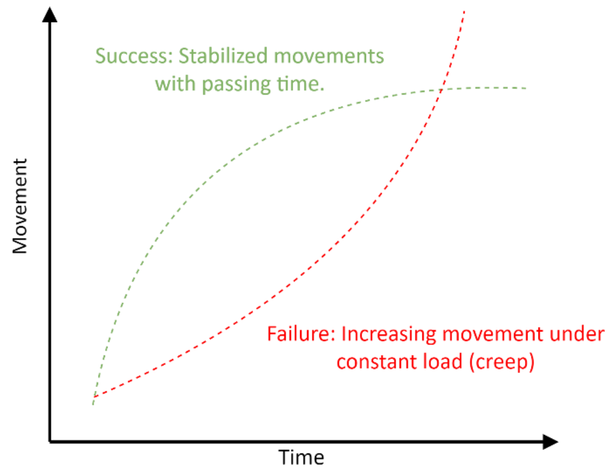

Figure 5. Success vs. failure of anchor test.

The data has processed in a way that all durations starting from t=1 min are checked for maximum creep rate, i.e. b/w 1-3 min, 1-5 min, 1-10 min, 1-15 min, 1-30 min, 1-45 min, 1-60 min. Although ISO recommends using the last stabilized portion of the curve, most critical creep rate has been found from test results conservatively.

The test results have shown that all the creep values were significantly below the 5 mm level for failure. To make sure the results are correct, all individual measurement points are used separately to calculate the creep rate. The results are summarized for adhesion (kPa) – which is the applied load per surface area of the bond.

The highest creep rate observed in the tests were 2.13 mm/log(min) – and not at the highest adhesion. The highest creep is observed for an anchor with 861 kPa loading.

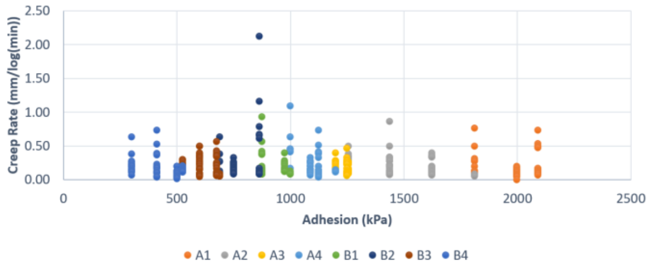

Figure 6. Creep rate vs. adhesion.

The highest anchor loads (in terms of adhesion) are shown in table 1. For a load (friction) of 2091 kPa, the creep rate is less than 1 mm/log(min). Therefore, the failure has not been reached even at the maximum adhesion of 2091 kPa.

Table 3. Creep rates at highest proof loads.

Name	Region	Load (kPa)	Duration (min)	Bond Length (m)	Friction (kPa)	Creep Rate (mm/log(min))
A1-3350-3	A1	3350	3	3	2091	0.73
A1-3350-20	A1	3350	20	3	2091	0.53
A1-3350-15	A1	3350	15	3	2091	0.51
A1-3350-30	A1	3350	30	3	2091	0.49
A1-3350-5	A1	3350	5	3	2091	0.48
A1-3350-10	A1	3350	10	3	2091	0.47
A2-3350-3	A2	3350	3	3	2091	0.17
B2-3350-3	B2	3350	3	3	2091	0.17
B1-3350-30	B1	3350	30	3	2091	0.14
B2-3350-30	B2	3350	30	3	2091	0.14
B1-3350-20	B1	3350	20	3	2091	0.13
B2-3350-20	B2	3350	20	3	2091	0.13
B2-3350-15	B2	3350	15	3	2091	0.13

2.3 Comparison with literature

Several papers in the literature (Kristensen et. al., 2000, Roesen & Trankier, 2021, Lyse, 2022) discuss bond of anchors in the Copenhagen limestone. The maximum tested adhesion was reported by Lyse (2022) as high as 2060 kPa. None of the studies have reported failure in the Copenhagen limestone. Therefore, it can be concluded that the Nordhavnstunnel test results increased the number of tests above 2000 kPa, reaching a new limit.

2.4 Design decisions for uplift anchors in the limestone

To reduce the risk to an acceptable level, an adhesion of 1200 kPa has been selected. Using a common drilling diameter of 150 mm. this resulted in 3.6 m bond length for 63.5 mm diameter anchor to sustain the design uplift load of 1410 kN.

3 ANCHORS IN THE GLACIAL TILL

A series of anchor investigation tests with the anchor grout body in the clay till was set up during the design phase.

16 inclined investigation tests were performed on inclined anchors at two specified test sites adjacent to the tunnel location on land (Nordhavn area). These anchors were all made to investigate the anchors' suitability for the glacial clay till. The test anchors were inclined by 35 degrees to the horizontal plane, drilled in OD Ø170 mm dimension using a Ø152 mm casing, and each was installed with 12 No. of 0,62" Y1860 strands. The test proof load was 1600 kN. 12 anchors with bond length 6,9 and 12m were installed with primary injection during installation and using 6 to 7 bar grout pressure when extracting the casing around the tendon bond length. 4 anchors with 9m bond length were post grouted using two reinjection tubes.

Anchors with no post grouting could take a load between 720 and 960kN, without reaching failure, however a creep rate of 2 mm per time decade was reached even at the lower end of the mentioned loading range.

Post grouted anchors could undertake test loading up to 1440 kN for a 9 m bond length (displacement criterion $\Delta s = s_b - s_a < 0.5$mm was exceeded at the last step of 1600kN or creep rate of 2mm per time decade was reached).

The investigation tests confirmed that anchors in the clay till with post grouting have a resistance of approximately 265kPa (the corresponding adhesion factor α would be equal to 0.65 assuming S_u=400kPa and adhesion=α x c_u). This resistance, however, was not sufficient for the current project demands.

As the sheet pile wall design requires proof load tests ranging up to 1600 to 1800 kN, it was decided not to go forward with the anchors bond in the clay till, but to adopt a grout body in the limestone.

4 TEMPORARY WORKS - RETAINING STRUCTURE

4.1 Design summary

The construction sequence in the Svanemøllehavn foresees a hydraulically installed working platform from which the sheet piles and vertical anchors are installed passing through the fill, clay till, greensand to the limestone. In the Nordhavn area, retaining walls and vertical anchors are to be driven from the existing ground level. Grouting is to be performed to extend the water cut-off role of the sheet piles into the limestone over 2 to 9 m depth. Dewatering (with re-injection) is installed to allow staged dry excavation and installation of the grout anchors at one to three levels depending on the depth of the building pit. The retaining height reaches up to 15 m in the deepest parts. (see Figure 3). In the Svannemolen area, the bottom slab of the tunnel is up to two meters below the limestone top. As the sheet piles do not reach the depth of the bottom slab in these areas, a berm will be left below the toe of the sheet pile to reach full excavation depth.

An extended driveability campaign at 5 locations with different hammers and modifications (e.g. driving shoe at the toe of the sheet pile, pre-trenching/relaxation of the soil) was performed. Heavy duty U-sections with flange thickness around 20 mm, were selected from the beginning and used on all test piling locations. Testing was done, using panel driving of three double piles at each testing location, to simulate the piling process for the production sheet piles, that are properly interlocked. These sheet piles were considered more likely to be driven through the hard underground thanks to their shape and stiffness.

4.2 Sheet pile driveability tests

A variety of hammers were used for the sheet pile driveability tests, including a 9-ton free fall hydraulic hammer along with hydraulic accelerated hammer types IHC S-120 and S-150, both representing equivalent drop weights of 12 ton and 15 ton, respectively. The result of the driveability tests was that the free fall 9 ton hammer proved to be adequate for sheet pile driving to the limestone, as well as the IHC hammers.

Potential benefits from pretrenching/loosening the soil of upper layer were not observed. A driving shoe was found to have benefits (stress reduction during driving) but given the sheet pile section selected (the sheet pile section could also be installed without a driving shoe), a driving shoe was not deemed necessary.

Through the campaign it was proven possible to drive the U-shaped sheet piles smoothly through the glacial till and greensand layers to the top of limestone. However, penetration in the limestone could not be demonstrated. Some sheet piles were extracted at the end of campaign to witness toe damage. Sheet piles in general demonstrated acceptable minor or no damage. However, there were cases where sheet pile toe got damaged to force sheet piles into limestone (Figure 7).

As multiple inclined anchors are necessary for the support of the wall, it is easily understood that the vertical load on the sheet pile is high and vertical bearing capacity is thus critical. The initial design has been adapted to sheet piles stopping with refusal to the top of limestone. The end bearing capacity will have to be confirmed by dynamic testing: PDA-measurements and CAPWAP-analysis and by the introduction of a reliable set criterion when installing the sheet piles. Preliminary dynamic testing allowed for an estimation of an unfactored toe bearing resistance of 4514kN/m (or factored as per DS EN 1997-1 approximately 2257kN).

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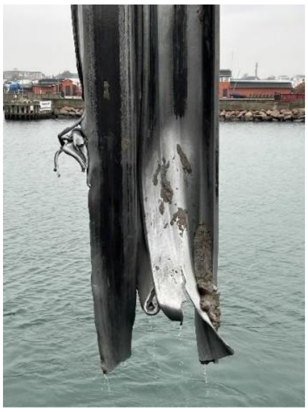

Figure 7 Damaged sheet pile toe as retrieved.

4.3 Limestone berm support at pile toe level

Where excavation extends deeper than the toe of the sheet piles, a berm at the sheet pile toe is provided to ensure sufficient end bearing capacity. This design includes a purpose specific assessment of the effective limestone strength accounting for its anisotropy in the given the loading conditions.

By including anisotropy of the limestone, in combination with the nature and circumstances of the load from the retaining wall onto the limestone berm, two different sets of equivalent strength properties can be attributed to the limestone due to the different failure mechanisms. Thus, one set of strength parameters was used for the vertical capacity and berm stability, and a specific set of values was used for the determination of horizontal passive resistance in the earth pressure calculations.

The design strategy furthermore foresees site assessment of the limestone conditions and properties based on face logging results at the centre of the excavation before excavation towards the sheet piles is continued. Rock bolts are foreseen as mitigation measure in case of unfavourable geological and geotechnical conditions (e.g. unfavourable fissuring).

5 CONCLUSIONS

Uplift anchor investigation tests, inclined anchors investigation tests in the clay till and sheet pile driveability tests have been carried out to validate and optimize the design of the Nordhavnstunnel.

Uplift anchor tests reached confirmed adhesion as high as 2091 kPa and the anchors did even not come close to failure. Therefore, traditional 650-900 kPa values should not be used anymore, and higher values should be adapted in Copenhagen. Nordhavnstunnel design adopts 1200 kPa – while higher values are open to explore. Preliminary anchor testing allowed a refinement of the anchor grout body length, contributing to financial optimisations.

The benefits of post grouting of anchors in the clay till could be demonstrated. An increase of capacity by minimum 60% was reported, with post grouted anchors of 9m grout body in the clay till demonstrating a capacity around 1440kN.

U-shape heavy duty (20mm web thickness) sheet piles were found to be able to be smoothly driven in the clay till and greensand layers of Copenhagen, but no penetration in the limestone could be demonstrated.

Placing the driveability campaign early in the project allowed for an in-time design refinement and risk mitigation and enabled the adoption of a sheet pile solution instead of a traditional secant pile wall. The very good collaboration between design and execution team, the contractor and the client allowed preliminary investigation tests, taking calculated risks/opportunities, and led to an improved productivity and potentially sustainability related performance during design and execution.

6 ACKNOWLEDGEMENT

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