A case study on the design, numerical modelling and preliminary testing of tension piles in weak rock

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Abstract. Over the last twenty years several major tunnel projects have been constructed in the Copenhagen area in the Danian limestone, either in the Copenhagen Limestone or in the Bryozoan limestone; in parallel, developments have been made in the characterization of these rocks. The Danian limestones are weak sedimentary rocks with highly variable properties in terms of their strength, stiffness and in their mass permeability. Their properties are governed by their genesis resulting in a large variation in the carbonate content, their induration and a large variability of the fissuring and distribution of fissuring. The paper describes the experience gained from selected underground construction projects in the Copenhagen area, specifically for the passive uplift anchors/tension piles that are often used as a means to resist uplift forces resulting from the porewater pressure exerted at the underside of the base slab of major underground structures. While tension piles are common in Copenhagen, as they are often used for basement construction too, the authors are not aware of a documented calculation method. The paper presents a summary of the case histories published for ground anchors in Danian limestone and proposes a calculation method for the unit shaft resistance of small diameter tension piles as well as the comparison with numerical modelling and preliminary testing undertaken for the Sydhavn metro in Copenhagen.

1. Introduction
The branch-off to Sydhavn (CRSH) adds five underground stations and 4.5 km of dual single-track tunnel 12 to 30m below ground surface to the existing Cityringen metro in Copenhagen. Due to the ground water being hydrostatic from approximately the ground level the underground stations are subject to a significant hydraulic uplift which is resisted by the self-weight of the permanent structures as well as by tension piles connected to the station base slab. As the tension piles are installed in the Danian limestone formation, a horizontally bedded weak sedimentary rock, a small diameter and short fixed lengths are sufficient to achieve the required axial resistance. For the Sydhavn project, much like for the preceding underground stations in Copenhagen, the tension piles were installed as passive ground anchors with no pre-stress applied between the threaded bar and the slab, i.e. small diameter tension piles.

Perhaps due to the fact that ground anchors are designed on the basis of testing rather than on characteristic geotechnical parameters derived from in situ & lab test, it is uncommon in Copenhagen to justify the initial unit shaft resistance on the basis of a calculation method, but rather to design the investigation tests on the basis of experience. Because the Danian limestone has an intrinsically high
variability of unconfined compressive strength (UCS) based on its diagenesis, it should be expected that relatively short sections of anchors (3 to 6 m) have shaft resistances that are dependent on the average strength of the surrounding ground.

This paper presents the findings of a literature review of ground anchors tests in Danian limestone; it also introduces a calculation method to derive the shaft resistance and documents the results of the preliminary pile testing carried out for the Sydhavn metro project in the Copenhagen and Bryozoan Limestone and the Maastrichtian Chalk. Finally, it presents the findings of a numerical study developed to assess the impact that the grout injected within the annulus of the free length has on the axial stiffness and resistance.

1.1. Sydhavn metro stations
The five underground station and a switch box have been built with a temporary retaining system consisting of secant bored piles embedded in the limestone to below the base slab to provide a soil retaining structure and to form a temporary cut-off to groundwater. The maximum excavation depths range between 17 and 23 m below ground level. The retaining walls are supported by a combination of steel props and ground anchors in the temporary case. The station permanent structure consists of inner lining walls cast against the secant bored pile wall and reinforced concrete slabs.

The uplift forces acting below the base slab are resisted by the self-weight of the permanent structures and by the axial resistance of uplift anchors/tension piles. The tension piles are bored 178 mm diameter with a threaded 75 mm steel bar and a grouted length which varies from station to station between 14 and 28 m.

1.2. Ground conditions
The geology of the Copenhagen area comprises of fill and quaternary deposits overlying the Paleogene deposits, primarily consisting of Danian limestone [1] which is subdivided into the Copenhagen Limestone (CL, Late Danian) and Bryozoan Limestone (BZL, Lower and Intermediate Danian). Below the BZL the Maastrichtian Chalk formation is found. These formations originate from calcareous material deposited in a deep water (marine) environment. Lithification of the sediments to bedrock occurred during syndepositional lithification (including induration) and post-depositional burial.

The Danian limestone is variably indurated and un lithified layers are interbedded with indurated to strongly indurated limestone; its layers are enriched with flint nodules or flint beds. The Maastrichtian Chalk consists principally of a white chalk, horizontally bedded and includes flint. Top of solid geology has uneven topography, representing an erosional surface, which had been subjected to erosion including subaerial erosion, possibly with karst dissolution, and during the Quaternary with glacial and fluvial erosion. For the purpose of this study and considering the depths of the tension piles, the non-glacially disturbed Danian limestone is considered in the following.

In addition to the traditional intact rock classification, strength and compressibility tests, local practice relies on induration for the purpose of Danian limestone characterization: this is defined in [2] and is a semi-quantitative parameter subdivided in a five degrees scale as shown in Table 1 together with the corresponding ISRM rock grade. The bulk density ranges from 18.5 kN/m$^3$ and 26.5 kN/m$^3$ depending on porosity and grain density. The strength increases with increasing degree of induration and density.
Table 1. Degree of induration according to Danish practice

| Induration | Term             | Description                                                                 | ISRM Rock Grade |
|------------|------------------|-----------------------------------------------------------------------------|-----------------|
| H1         | Unlithified      | The material can be easily formed by hand. Grainy material will fall apart when dry. | R0              |
|            |                  | The material can easily be cut with a knife and can be scratched with a fingernail. Individual grains can be picked out with the fingers when the material is grainy. | R1              |
| H3         | Indurated        | The material can be cut with a knife but cannot be scratched with a fingernail. Individual grains can be picked out with a knife when the material is grainy. | R2              |
| H4         | Strongly indurated| The material cannot be scratched with a knife. Fractures will follow grain surfaces. | R3, R4          |
| H5         | Very strongly indurated | The material cannot be scratched with a knife. Cracks and fracture surfaces will go through individual grains in grainy materials. | R5, R6          |

2. Review of case histories

In the following, five case histories of ground anchor testing in Danian Limestone are presented. The case histories are summarised in Table 2 and described in the following sections.

Table 2. Case histories summary

| Case          | Grouted diameter [mm] | Bond length [m] | Test passing criteria                        | Measured shaft resistance [kPa] |
|---------------|-----------------------|-----------------|----------------------------------------------|-------------------------------|
| Christiansbro | 160                   | 3-6 min         | DIN4125, $k_s < 2$ mm                        | 766-1139                      |
| New Maritime Museum | 152                | 5-10            | $k_s < 5$ mm                                | $\leq 691$                     |
| Bryghusprojektet | 152                | 6-7             | $k_s < 0.8$ mm                              | 857-998                       |
| Nordhavn      | 152                   | 4-6.5           | $k_s < 2$ mm                                | 780-904                       |
| Postbyen      | 152                   | 4-6             | $k_s < 5$ mm                                | ~ 1545                        |

2.1. Christiansbro office building, Copenhagen

The project is a 33,000m$^2$ office building in the central Copenhagen harbour area. With a planned installation of almost 1,500 permanent ground anchors to withstand uplift, an extensive series of ground anchor testing was carried out in the Copenhagen Limestone. The objective of the testing was to determine the shaft resistance of ground anchors with shorter bond length (3-6m) than what had previously been common practice (5-10m) for anchoring in Danian Limestone. The case history, including details on geology and testing procedure, is documented in [3]. The testing programme comprised 39 anchors with varying bond length and anchoring level. From the tests that either reached or almost reached failure, a mean mobilised shaft resistance between 766kPa and 1139kPa was derived.

2.2. New Maritime Museum, Elsinore

The new M/S Maritime Museum of Denmark in the port of Elsinore is located in an old dry dock, subject to permanent water pressure. Building of the new museum meant that soil providing ballast to the dry dock would be removed, and ground anchors had to be installed in the Danian Limestone. The case history, including details on geology and testing procedure, is documented in [4]. Six suitability tests and ten investigation tests were carried out. One test resulted in failure between grouting and limestone, from which a shaft resistance of 691kPa can be derived.
2.3. Bryghusprojektet, Copenhagen
Bryghusprojektet (now BLOX) is a multipurpose building located in the harbour area of central Copenhagen. The building is a six-story building with a multi-level basement constructed as a permanent secant pile wall with temporary ground anchors. The ground anchor testing programme (Züblin, 2014) comprised suitability tests on 10% of the anchors and acceptance test on the remaining 90%. For the two suitability test anchors considered in this case study, the mobilised shaft resistance was found to be 857kPa and 998kPa, respectively.

2.4. Nordhavn Metro Station, Copenhagen
Nordhavn Metro Station is part of the Cityringen metro extension in Copenhagen. For the purpose of this case study, results from eight suitability tests (Züblin, 2015) of inclined (40°-45°) ground anchors installed in Copenhagen Limestone were assessed. Based on the eight tests, none of which reached failure, mobilised shaft resistance in the range 780kPa to 904kPa was measured.

2.5. Postbyen urban area, Copenhagen
The project is a new 200,000m² urban area on the former Postal Service Centre parcel in central Copenhagen. Approximately 950 ground anchors were needed to withstand uplift on the two-level parking basement. An extensive suitability testing programme comprising 23 tests of anchors in Copenhagen Limestone was carried out with the goal of minimizing cost and ensuring robustness of the project. The case history, including details on geology and testing procedure, is described in [5]. The authors report that none of the tests reached failure at the applied load, and that a generalised shaft resistance of ~1545kPa can be derived from the tests.

3. Calculation method for unit shaft resistance
The calculated geotechnical resistance for the tension piles was based on the application of an empirical correlation calibrated on the case histories presented in Section 2. The calibration focused on the influence of UCS and degree of induration on the unit shaft resistance.

The shaft resistance \( \tau_{\text{max}} \) of a bored pile in rock was calculated using empirical correlations with unconfined compressive strength of rock \( q_{\text{uc}} \) (or UCS) as generalized in [6]:

\[
\tau_{\text{max}} = \alpha \cdot (q_{\text{uc}})^\beta,
\]

where \( \alpha \) and \( \beta \) are empirical factors. The following reduction factors have been applied: \( \eta_c = 0.9 \) (due to construction method), \( \alpha = 0.2 \) and \( \beta = 0.5 \). Size effects have conservatively been ignored.

The selected UCS values for each induration and limestone type are shown in Table 3 and correspond to the 50th percentile of the analysed data set; the UCS ranges proposed in [7] are also provided. In case of identified core loss, a UCS value of 250kPa has been assumed. For highly indurated materials identified as H5, an UCS value of 100MPa has been adopted.

| Induration | CL UCS after [7] | CL UCS, CRSH | BZL UCS, CRSH |
|------------|------------------|--------------|--------------|
| H1         | 0.25-1           | 0.25         | 0.25         |
| H2         | 1.5              | 5            | 4.5          |
| H3         | 5-25             | 16.3         | 10.9         |
| H4         | 25-100           | 26.6         | 30.4         |
| H5         | 100-500          | 100          | 100          |

The average unit shaft resistances over a given pile length were calculated and plotted as shown in Figure 1. While the figure shows one curve for each project borehole, the investigation locations of a specific station, Ny Ellebjerg in this case, were highlighted to guide the selection of the representative...
calculated unit shaft resistance. Due to the high variability of the UCS values, there is a large scatter in average unit shaft resistance initially which stabilises for pile lengths greater than 2-3m.

![Graph showing calculated average unit shaft resistance over grouted length, non-glacially disturbed limestone, Ny Ellebjerg station (station-specific series highlighted in red)](image)

**Figure 1.** Calculated average unit shaft resistance over grouted length, non-glacially disturbed limestone, Ny Ellebjerg station (station-specific series highlighted in red)

### 4. Preliminary tension piles testing

The preliminary tension pile testing programme was defined to demonstrate that the calculated geotechnical resistance can be provided by a tension pile or an anchor while satisfying specified criteria of creep or load loss. The programme focused on testing anchors with the fixed (tendon) length within glacially disturbed limestone and BZL as no similar case histories have been found with ground anchors in these ground conditions. Tests were also performed on anchors within CL to confirm the design resistance determined based on the calculation method described in Section 3.

The testing programme comprised three anchors in glacially disturbed limestone, six anchors in CL, five anchors in BZL and one anchor in Chalk in order to verify ground-grout bond within specific conditions and at the approximate levels of both retaining wall anchors and tension piles. Calculated design resistances to be confirmed by testing were 400kPa for glacially disturbed limestone, 500kPa for intact CL, 600kPa for intact BZL and for Chalk. Table 4 and Table 5 present the test details.
Table 4. Details of preliminary anchors testing in glacially disturbed limestone and Copenhagen limestone (CL).

| Intended rock type to be tested | ØVK-IT-01 | ØVK-IT-02 | ØVK-IT-03 | EBR-IT-04 | EBR-IT-05 | EBR-IT-06 | EBR-IT-07 | EBR-IT-08 | EBR-IT-09 |
|---------------------------------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|-----------|
| Ground surface level [mDVR90]   | Glacially disturbed limestone | Glacially disturbed limestone | Glacially disturbed limestone | Copenhagen limestone (CL) | Copenhagen limestone (CL) | Copenhagen limestone (CL) | Copenhagen limestone (CL) | Copenhagen limestone (CL) | Copenhagen limestone (CL) |
| Level of bottom of anchor [mDVR90] | -9.6      | -15.0     | -21.0     | -21.0     | -26.0     | -21.0     | -26.0     | -21.0     | -26.0     |
| Total length [m]                | 11.8      | 17.2      | 23.2      | 23.2      | 28.2      | 23.2      | 28.2      | 23.2      | 28.2      |
| Intended free length [m]        | 8.8       | 14.2      | 20.2      | 20.2      | 25.2      | 20.2      | 25.2      | 20.2      | 25.2      |
| Test load $P_p$ [kN]            | 1395      | 2150      | 2150      | 2150      | 2150      | 2150      | 2150      | 2150      | 2150      |
| Measured unit shaft resistance [kPa] | 832      | 1283      | 1283      | 1283      | 1283      | 1283      | 1283      | 1283      | 1283      |
| Strand area [mm$^2$]            | 150       |           |           |           |           |           |           |           |           |
| Number of strands [-]           | 7         | 10        | 10        | 10        | 10        | 10        | 10        | 10        | 10        |
| Young’s modulus [GPa]           | 195       |           |           |           |           |           |           |           |           |
| Intended stiffness EA/L [kN/m]  | 19879     | 18631     | 13479     | 13479     | 10955     | 13479     | 13479     | 10955     | 10955     |
| Measured stiffness EA/L [kN/m]  | 17354     | 16938     | 17049     | 17471     | 17678     | 17830     | 13920     | 13635     | 11670     |

Table 5. Details of preliminary anchors testing in BZL and chalk.

| Intended rock type to be tested | GÅB-IT-01 | GÅB-IT-02 | GÅB-IT-03 | GÅB-IT-04 | GÅB-IT-05 | GÅB-IT-06 |
|---------------------------------|-----------|-----------|-----------|-----------|-----------|-----------|
| Ground surface level [mDVR90]   | Bryozoan limestone (BZL) | Chalk | Bryozoan limestone (BZL) | Chalk | Bryozoan limestone (BZL) | Chalk |
| Level of bottom of anchor [mDVR90] | -6.0      | -12.0     | -23.0     | -26.0     | -33.0     | -26.0     | -33.0     | -26.0     | -33.0     |
| Total length [m]                | 10.0      | 16.0      | 27.0      | 30.0      | 37.0      | 27.0      | 37.0      | 27.0      | 37.0      |
| Intended free length [m]        | 7.0       | 13.0      | 24.0      | 27.0      | 34.0      | 27.0      | 34.0      | 27.0      | 34.0      |
| Test load $P_p$ [kN]            | 2092      |           |           |           |           |           |           |           |           |
| Measured unit shaft resistance [kPa] | 1248    |           |           |           |           |           |           |           |           |
| Strand area [mm$^2$]            | 150       |           |           |           |           |           |           |           |           |
| Number of strands [-]           | 10        |           |           |           |           |           |           |           |           |
| Young’s modulus [GPa]           | 195       |           |           |           |           |           |           |           |           |
| Intended stiffness EA/L [kN/m]  | 34412     | 25435     | 20172     | 11471     | 10263     | 8239      | 10263     | 8239      | 10263     |
| Measured stiffness EA/L [kN/m]  | 30412     | 23422     | 19730     | 14266     | 12418     | 10477     | 12418     | 10477     | 12418     |

All test anchors/piles were drilled from the existing ground surface vertically with 0.178m diameter and had an intended fixed length of 3m. Testing anchors were grouted continuously over the bonded and free lengths resulting in lack of grout discontinuity between the fixed and free length.

The apparent free (tendon) length $L_{app}$ was found to lie below the lower limit defined in [8] for test anchors GÅB-IT-04, GÅB-IT-05, GÅB-IT-06, EBR-IT-09 and 0.1m above the upper limit for ØVK-IT-02. As these tests were not subjected to repeated load cycles to test load $P_p$ to demonstrate reproducibility of load-displacement behaviour, the results should be treated with caution. The apparent free length shorter than the lower limit may be attributed to friction developing along the free length.

The preliminary test anchors did not fail under the applied loads; furthermore, the registered creep rates were well below the 5mm limit, as defined by [8] for investigation tests where a vertical asymptote of creep rate versus proof load diagram is difficult to determine. Therefore, it is reasonable to assume that the tests confirm a higher ground-grout resistance than the calculated. However, a simple calculation of the measured shaft resistance as maximum load applied divided by the fixed length surface area as shown in Table 4 and Table 5 is expected to overestimate the unit shaft resistance of the fixed length due to the contribution to the resistance from the soil and rock layers above the intended fixed length. This is investigated with a numerical model in the next section.
5. Numerical modelling

An axisymmetric finite element model in software Plaxis 2D was developed to simulate a pull-out test for tension piles. Finite elements are used to model the tendon/strands, the grout and the ground while interface elements define the behavior at the boundary between the tendons and the grout and between the grout and the ground. The model geometry, interface detail and analyses results are shown in Figure 2-4. The stratigraphy and length of the modelled tension pile are based on the EBR station test anchors with intended fixed lengths in non-glacially disturbed UCL (i.e. EBR-IT-04,05,06). The modelled cross-sectional bar area corresponds to that of a threaded bar with nominal diameter of 75mm.

The drained Mohr-Coulomb constitutive model with tension cut-off is used to model the ground and the grout with the material parameters illustrated in Table 6.

![Figure 2. Numerical Model.](image)

![Figure 3. Interface detail.](image)

![Figure 4. Unit shaft resistance along depth.](image)

| Material parameters          | Abb. | FY | ML1 | MS | gdUCL | UCL | Grout |
|-----------------------------|------|----|-----|----|-------|-----|-------|
| Bulk Unit Weight [kN/m³]     | γ    | 18.0 | 21.5 | 21.0 | 21.0 | 21.0 | 20.0  |
| Poisson’s Ratio              | ν    | 0.2 | 0.2  | 0.2  | 0.2  | 0.25 | 0.2   |
| Effective Young’s Modulus [MPa] | E’  | 8.0 | 65.0 | 80.0 | -    | -    | 10E3  |
| Angle of Shearing Resistance [˚] | φ   | 30  | 33   | 38   | 40   | 100  | 2     |
| Cohesion intercept [kPa]     | c’   | 0   | 10   | 0    | 70   | 100 - 350 | 10E3 |
| Rock Mass Young’s Modulus [MPa] | Ecm | -   | -    | -    | 600  | 750 - 4000 | -    |
| At Rest Coefficient K₀      | 0.5  | 1.0 | 1.0  | 0.7  | 0.7  | 1.0  |       |

The steel-grout interface adopts the grout material properties by imposing an interface factor of $R_{int}=1$ (i.e. full bonding) along the fixed length and $R_{int}=0$ (i.e. no bonding) along the free length. The grout-ground interface is rigid and adopts the material parameters of the adjacent ground stratum.

Figure 4 shows unit shaft resistance distributions for the tested models. The y-axis has the origin at the top of the fixed length. The x-axis corresponds to the mobilised shear stress $\tau$ at the grout-ground boundary.

In the analysis with no bonding conditions along the free length, approximately 2/3 of the mobilised grout-ground resistance develops within the intended fixed length. The peak of unit shaft resistance
occurs at the top of the fixed length. This analysis confirms the contribution of the layers above the fixed length to the total resistance offered by the tension pile.

When full bonding conditions along the free length are modelled, the mobilised unit shaft resistance develops along multiple strata as the upper soil layers contribute to the overall resistance and follows closely the theoretical shear strength (i.e. $\tau' = c' + \sigma' \tan \phi'$) until the peak in gdUCL is reached. It is seen at strata transitions that a cohesion increase produces an increase of mobilised friction (i.e. gdUCL and UCL). Figure 4 also shows that the shear stress distribution along the pile length varies significantly between the full bonding and the no bonding case.

As expected a tension pile with full bonding along the free length is also stiffer than one with no bonding, in line with the findings reported in [9].

6. Conclusions
The results of preliminary tension pile testing to support the design of 5 underground metro stations in Copenhagen have been presented. The targeted geomaterials were two Danian limestone units and the Chalk formation. The measured unit shaft resistances are generally in line with past experience for the Copenhagen limestone; the authors are not aware of similar case histories having been published for the Bryozoan limestone and the Chalk in the area. A numerical analysis has demonstrated that for fully grouted tension piles, whether bonding is assumed in the free length or not, the layers above the fixed length contribute to the measured resistance and increase the stiffness of the tensile elements.

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