BORED PILES IN COPENHAGEN LIMESTONE

Kirsten M. Iversen[1], Bjørn S. Roesen[2], and Jørgen S. Steenfelt[3]

KEYWORDS

Bored piles, bi-directional testing, shaft & toe resistance, flat-jacks, Copenhagen limestone

ABSTRACT

On the former Postal Service Centre parcel in central Copenhagen a new urban area of 200,000 m² is being constructed, which accommodates service and retail trade as well as apartments for housing. The buildings consist of office buildings in 3-4 stories and 5 high-rise towers up to about 107 m high. Below ground level a parking basement in two levels is established with excavation to 8 m below ground level. Due to exceedingly high concentrated loads the towers and parts of the other buildings with large load concentrations are placed upon bored piles serving as rock sockets in the underlying Copenhagen Limestone. The bored piles are constructed with diameters ranging from 1180 to 1800 mm.

Before installation of the production piles, six test piles were established on the construction site. The purpose of the test piles was primarily to test the shaft resistance of the piles in Copenhagen limestone, as insufficient "Codal comparable experience" was established at the time. However, a second purpose was also to overcome the limitations in the Danish Annex to Eurocode 7 requiring a reduction of the shaft resistance to 30 % of the value for a "comparable driven pile". Further, the purpose was to test bored piles with a flat-jack device at the bottom. The flat-jack engages toe resistance of the piles by a controlled grouting process enabling pre-stressing of the pile and a stiffer pile/structure interaction.

The results of the tests allowed the design to be optimized beyond the limitations of the Code, and the concept of using flat-jacks on the production piles was proven successful.

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[1] COWI A/S, Jens Chr. Skous Vej 9C, 8000 Aarhus C, Denmark, KMIV@cowi.com

[2] COWI A/S, Jens Chr. Skous Vej 9C, 8000 Aarhus C, Denmark

[3] COWI A/S, Parallelvej 2, 2800 Kgs. Lyngby, Denmark

1. INTRODUCTION

The project

Close to the Central Station in Copenhagen a new urban area of 200,000 m² is being constructed on the former Postal Service Centre parcel. The development project consists of a new domicile for a large Danish bank, a building for small and medium-sized companies and five large towers for both business and apartments. Below ground a parking basement in two levels were established with excavation to level -5.5 m, corresponding to approximately 8 m below ground level.

The high-rise buildings are founded on large, bored piles, rock socketed into the Copenhagen Limestone, which forms the sub-base under the entire area.

Limitations within the Danish annex to Eurocode 7 part 1

In the Danish Annex to Eurocode 7 (DS/EN-1997-1 DK NA:2021), Ref. [1], the resistance of bored piles is restricted by Clause (6) to Annex L (informative). "For bored in situ cast piles, the resistance may be considerably less than for corresponding driven piles. The maximum allowable shaft resistance is limited to 30% of the shaft resistance of the corresponding driven pile, and the toe resistance is limited to 1000 kN/m² unless recognized documentation for higher resistance is provided" (COWI translation).

The restriction is not qualified in terms of the soil or rock strata considered. In Denmark, this restriction typically applies to piles in limestone. However, the qualification "30% of the shaft resistance of the corresponding driven pile" implies that the restriction is intended for piles in fine-grained soils where the resistance is mainly derived from shaft resistance.

Unfortunately, this qualification/restriction has been indiscriminately applied to all bored piles, including those in rock. The fact that the qualification/restriction is in an informative Annex seems to be overlooked in its application.

There have been numerous projects in Denmark which proves that the limitations are too strict. Some of the projects are described in the paper “Is it reasonable to reduce the shaft resistance for bored cast-in-situ piles?”, Ref. [2]. However, more examples have seen the world after the paper has been published. E.g. a static load test in Miocene sand and clay in Silkeborg, Ref. [3] and "Loading test experience with bored piles in limestone/weak rock", Ref. [4].

To abstain from the codal limitations on the project it was decided to conduct six load tests on piles on the location to gather information about the shaft and base resistance.

2. GROUND CONDITIONS

The design of the foundation of the new buildings is based on 36 geotechnical boreholes where 24 were carried out as core drillings 10.5 m to 16.7 m into the limestone.

The ground conditions on the location for the construction pit are relatively uniform. The upper layers at the location consist of fill; mull, sand and clay from level -0,1 to -6,3 m DVR90. Some boreholes showed layers of gyttja fill, postglacial sand and gyttja and late glacial to glacial sand.

Fill, postglacial and late glacial deposits are underlain by glacial deposits of meltwater sand, silt, clay and clay till, gravel till and sand till to level –7.2 á -10.2 m DVR90, followed by a 2.5 to 5.0 m thick layer of Selandian Green sand. Due to the Green sand layer, the underlying Copenhagen limestone was not remoulded and glacially disturbed as frequently seen in the Copenhagen area. The limestone surface varied from level -10.2 to -14.2 m DVR90. The induration of the limestone is typically H2-H4 according to the Danish induration scale (corresponding to R1-R4 on the ISRM rock grade scale). However, some 18 % of the limestone showed hardness H5 (corresponding to R5-R6). The strength of the limestone was tested with 32 UCS (Unconfined Compression Strength) test. The tests were generally made on the weaker layers in the limestone. A plot of the measured strength as a function of depth and hardness is seen in Figure 1.

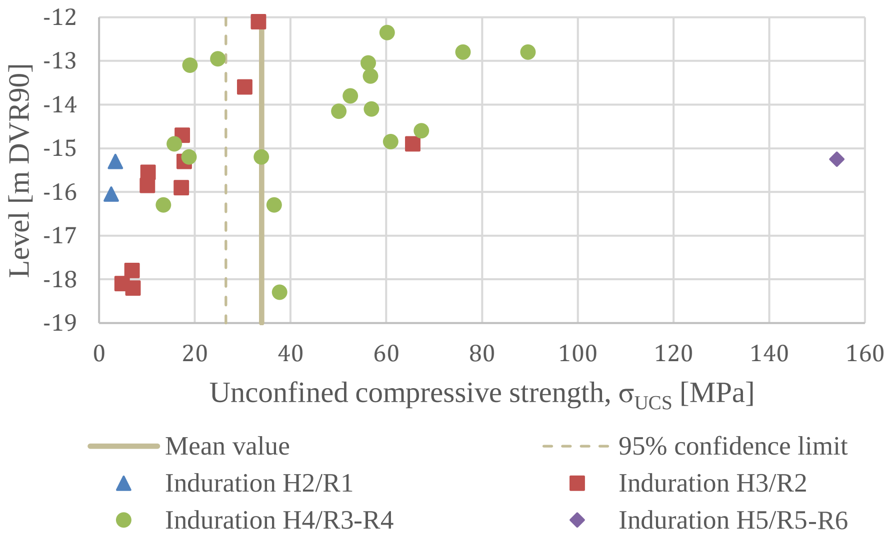

Figure 1. Measured unconfined compression strength as a function of the level of the sample and the induration. The degree of induration was described by the geologist before the conducted UCS tests, which can lead to inconsistency between the judged induration and the resulting strength from the test.

As shown in Figure 1 a lot of scatter is observed in the measured strength of the Copenhagen Limestone. There does not appear to be a relation between the depth of extraction and the measured strength in the limestone. As expected a trend was registered that the strength was a function of the geologically described degree of induration.

3. GEOSTATIC CALCULATION

As described in "Loading test experience with bored piles in limestone/weak rock" the design of bored piles follows the international standard adopted for numerous projects with sockets in weak rock carried out by COWI. The characteristic shaft, τ_{char}, and the toe resistance, $q_{toe,char}$, are based on the measured unconfined compressive strength, σ_c, found by UCS testing. According to Fleming et al., Ref. [5] and Tomlinson, Ref. [6], the characteristic resistance for soft rock may be found as, respectively:

$$\frac{\tau_{char}}{p_a} = 1.3\sqrt{\frac{\sigma_c}{p_a}}; \quad \frac{q_{toe,char}}{p_a} = 3\,\frac{\sigma_c}{p_a}$$

$$\tau_{char} = \alpha\,\beta\,\overline{\sigma}_c$$

In the equations σ_c represents the characteristic value and $\overline{\sigma}_c$ represents the cautious mean value of the strength distribution of individual or total layers of the rock, p_a is the atmospheric pressure and α and β are empirical factors. This methodology has been applied on numerous projects and have been found to be conservative for all the pile load tests interpreted by COWI.

The contribution of the layers above the limestone is assessed to be insignificant. In the evaluation of the bearing capacity the bearing capacity of the surface in these layers has been disregarded.

The skin resistance in the limestone has been evaluated to 1300 kPa according to Flemming et. al., Ref. [5] and Tomlinson, Ref. [6].

4. CONCEPT OF FLAT-JACK

The quality and resistance of bored piles depend highly on the workmanship, the cleaning of the toe and not least the properties of the surrounding soil/rock. Surprisingly, the strict rules pertaining to ground anchors (investigation, suitability, and acceptance tests) are not implemented for bored piles. To gain the same reliability and robustness of bored piles post grouting, particularly at the toe of the pile, suggests itself.

For the piles at Posten, the post grouting was established by means of flat-jacks where the grouting pressure is ensured over a well-defined area almost corresponding to the cross-sectional area of the pile.

This allowed for (i) enhancement of the toe resistance at a displacement commensurable to the displacement required to mobilize the shaft resistance; (ii) verification of pile capacity to a value of minimum twice the toe capacity; (iii) a cheap and efficient quality assurance; and finally (iv) an increased stiffness of the foundation system where the SLS load may be taken at very limited pile top displacement.

5. TEST SETUP

The bored piles were tested by means of an O-cell loading set-up. The O-cell is a bi-directional load cell, that is expanded through hydraulics and can thus apply a load both upwards to test skin resistance and downwards to test the combination of skin resistance below the O-cell and the toe resistance. The piles were not to be considered as production piles and were as far as possible be tested to failure. The load cells (O-cell) were placed 0.7-1.2 m above the pile toe and the piles were instrumented with strain gauges in four levels: halfway between O-cell and top of rock, top of rock and respectively 2 and 4 m above top of rock. The pile was further instrumented with tell-tale extensometers to measure displacement at the toe, the O-cell, top of rock and top of pile. Lastly the expansion of the O-cell is measured with four parallel LVWDTs.

The piles were installed by means of Kelly-boring where the casing is the main drill tool and the soil/limestone inside is brought to the surface with various cutting tools.

The piles were drilled with water pressure as to avoid loosening at the pile toe due to inflow of water and the borehole was carefully cleaned both after reaching the final depth and after raising the casing to a level approximately 0.5 m below top of rock. The reinforcement cage with the O-cell, instrumentation and flat-jack, see Figure 2, was lowered into the borehole and the pile was concreted through a submerged tremie pipe to ensure clean high-quality concrete on the entire length of the pile.

Before the loading sequence was commenced the flat-jack injection was carried out to engage the toe resistance. The first injected pile BP3 was not successful, and the procedure was adjusted, after which the remaining flat-jacks were installed successfully.

Figure 2: Flat-jack and O-cell mounted on reinforcement cage.

The load sequence consisted of two load cycles, with the first load cycle being 5 load steps up to 200 % (40 % each) of the expected serviceability limit state (SLS) load applied and a subsequent unloading in 5 steps. The second cycle was subsequently loading in 15 equal load steps to a total load corresponding to 150 % of the guarantied O-cell capacity or until failure was observed. For each load step in both cycles a 30 min observation time was applied.

6. RESULTS

Data from the test piles are summarized in Table 1.

Table 1. Data from the test piles.

Test pile	Dimension [mm]	Ground level [m]	Level of pile toe [m]	O-cell break-in-pile [m]	Top level of limestone [m]	SLS load [MN]
BP1	1500	+1.55	-16.70	-15.90	-12.00	7.9
BP2	1500	+1.60	-17.10	-16.37	-11.70	7.9
BP3	1180	+1.70	-16.95	-16.26	-12.10	6.3
BP4	1180	+2.35	-17.40	-16.21	-12.65	6.3
BP5	1180	+2.35	-16.85	-16.20	-12.20	6.3
BP6	880	+2.35	-17.40	-16.21	-12.30	3.3

The load tests on the bored piles were conducted between January 15th and January 21st, 2019. The load was taken to failure or the maximum capacity of the load cells.

The first load cycle was conducted to the SLS load. The magnitude of the load is shown in Table 1. The test piles were subsequently loaded in 15 equal load steps to a total load corresponding to 150 % of the guarantied O-cell capacity. Some piles were loaded a bit further to the maximum capacity of the hydraulics.

The load-displacement curves for the pile tests are shown in Figure 3. The horizontal asymptote makes it hard to evaluate failure. Therefore, the rate of creep has been evaluated for each load step for all six test piles as shown in Figure 4. Based on the rate of creep five of the six test piles were loaded until failure. However, test pile BP3 shows a rate of creep of 2 mm/log(t) and has therefore not reached failure.

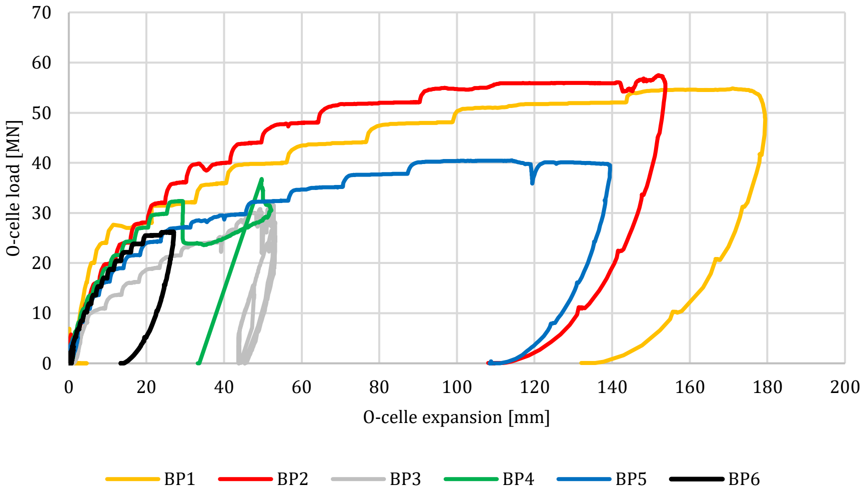

Figure 3. Load-displacement curves for the test piles.

The test piles primarily develop the capacity in the limestone. The test piles were instrumented with strain gauges as described in Section 5. The strain gauges measure the developed strains at the levels of the strain gauges. By using the axial stiffness of the pile (EA) the axial load in the level of the strain gauges can be evaluated. Further this leads to the maximum developed surface resistance between two levels of strain gauges. The result from the evaluation is shown for test pile BP2 in Figure 5.

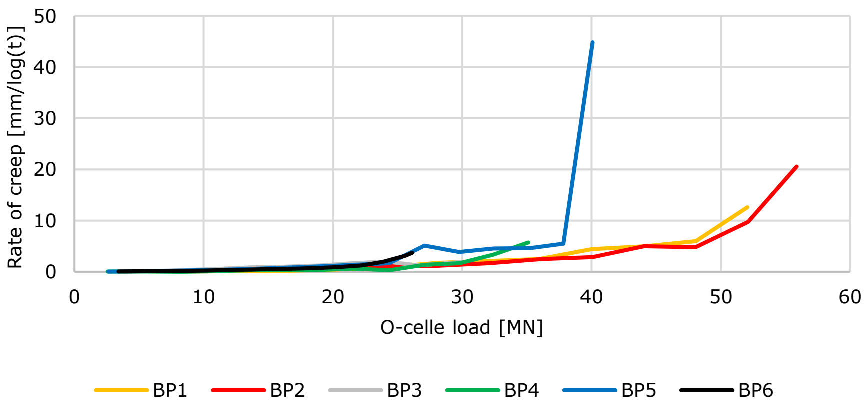

Figure 4. Evaluated rate of creep for all six test piles.

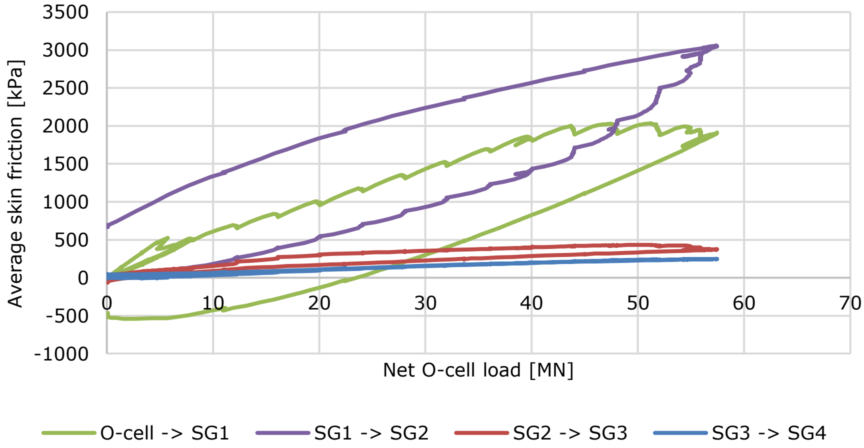

Figure 5. Mobilized shaft resistance for test pile BP2 between O-cell and different levels of strain gauges (SG), respectively.

The maximum and average mobilized skin friction for all the test piles in the limestone were evaluated for all six test piles and summarized in Table 2.

Table 2. Maximum and average shaft resistance mobilized in the limestone.

Test pile	Max. mobilised skin friction [kPa]	Average mob. skin friction [kPa]
BP1	3085	2407
BP2	2952	2402
BP3	1908	1665*

Test pile	Max. mobilised skin friction [kPa]	Average mob. skin friction [kPa]
BP4	2444	2139
BP5	2610	2456
BP6	2697	2083

* Test pile is not loaded till failure.

7. CONCLUSIONS

For a construction in the city centre of Copenhagen, six test piles have been established with sockets in the Copenhagen Limestone. The test piles should form the basis of allowing a larger baring capacity in the production piles compared to the limitation in the Eurocode, which limits the surface resistance to 30 % of the surface resistance of the comparable driven piles.

The piles have been tested with O-cells cast in the pile and with strain gauges to monitor the strains along the pile. The piles are loaded to 150 % of the guarantied capacity of the O-cell and hydraulics and it is found that five of six piles are loaded to failure. The average mobilized skin friction ranges between 2100 – 2450 kPa for the piles loaded to failure. This corresponds to approximately 160 – 188 % of the estimated skin friction.

8. ACKNOWLEDGEMENT

The authors would like to thank the owner of the development project, Project Nord, and the contractor Aarsleff for permission to use the site data and to publish the paper.

9. REFERENCES

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