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# Validation of drilled shafts for a new bridge in Denmark, by means of a holistic approach, CSL testing and pile coring

# Validation des pieux forés pour un pont au Danemark, avec une approche holistique, essais par transparence sonique et réalisation de carottage

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**ABSTRACT:** For the validation of deep pile foundations, a holistic approach was followed throughout design development, pile testing and execution. The design assumptions have been validated by the means of test piles that were loaded by bi-direction hydraulic jacks (O-cell ®). For validation of the executed foundation system, Cross Hole Sonic Logging (CSL) was performed in 100% of the piles. Additionally, some piles were further investigated by means of core drilling and compressive strength tests. It was observed that both measured attenuation and FAT (First Arrival Time) are relevant for the analysis of the CSL test results. The interpretation of these parameters is not yet well defined in guidelines or literature. In addition, a systematic tip grouting injection was performed in all elements to guarantee the aimed design behaviour of tip resistance in ULS.

**RÉSUMÉ:** Pour la validation des pieux forés profonds, une approche holistique a été suivie tout au long de la conception, les essais de pieux et l'exécution. Les hypothèses de calcul ont été validées à l'aide des pieux tests, consistant des vérins hydrauliques bi-directionnels (O-cell®) placées au fond des pieux tests. Pour validation des pieux exécutées, des essais par transparence sonique (CSL) ont été réalisés dans 100% des pieux. En outre, certaines pieux ont fait l'objet de carottages et de tests de résistance à la compression. Il a été observé que l'atténuation de l'énergie et le temps d'arrivée des ondes, mesurée lors des tests CSL, sont 2 paramètres déterminant pour l'analyse. L'interprétation de ces paramètres n'est pas encore bien définie dans les directives ou la littérature. De plus, une injection systématique a été réalisée au niveau de la base de tous les pieux, pour garantir que le comportement des pieux est en ligne avec les considération pris en compte lors de la conception.

**Keywords:** Pile validation ; Deep Foundations ; Rock Socket ; Crosshole Sonic Logging ; Pile tip grouting

## 1 INTRODUCTION

The contract for the design and construction of a 8.2 km expressway in Frederikssund, Denmark, including a 1.4 km bridge crossing the Roskilde fjord (Fig. 1), was signed In September 2016. The project is funded by Danish Government, (Danish Road Directorate) with Arup as employer's representative. The project is designed and build by a Contractors Joint Venture consisting of Besix, Rizzani de Eccher, and Acciona (RBAI JV).

*Figure 1: Illustration of expressway and the High Bridge over Roskilde Fjord ( source DRD)*

The bridge and expressway will be open to the public at the end of 2019.

The High Bridge is supported by deep bored pile foundations, that gain their bearing resistance from a limestone layer that is up to 40m below water level. The foundation consists of 4 piles per pier, with diameters of 1.5m for land piers and 2m diameter for the off shore piers. The rock sockets in the in situ limestone are 6m to 10m long.

The realisation of the pile foundation system is a one of a kind project in Denmark due to its size, the pile type (2m diameter and up to 53m deep bored piles), the rather limited experience with such large bored piles in Denmark, the magnitude load applied (service loads up to 20MN), and due to works being done from water in one of the most sensitive environmental areas in Europe (Natura 2000).

Such a challenging foundation requires a solid and holistic validation all along the different steps of the design development and construction.

The present paper describes the validation methodology applied for the design assumption, the full scale testing by bi-directional cell tests, the execution acceptance criteria and finally an extensive integrity testing campaign, including CSL tests and pile tip grouting.

## 2 DESIGN

### *2.1 Geotechnical Information*

The investigation campaign consists of one or two borings up to 65m deep at each pier and 725m of core samples from the limestone. A set of 850 strength tests (Unconfined Compression Strength, UCS, & Point load Test , PLT) of the in situ limestone has been gathered. The measured compressive strengths of all 850 samples are linked to its geological hardness (or induration) category ranging from H1(very soft) to H5 (flint, very hard). The variation in hardness categories in a core sample is illustrated in figure 2.

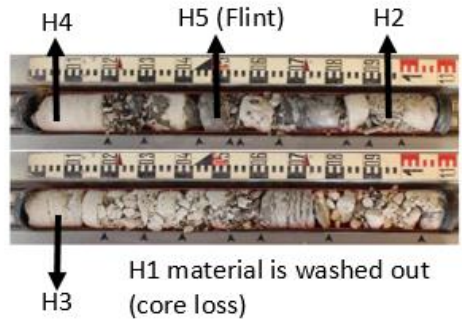


*Figure 2: Illustration of limestone induration*

The variation in rock induration as illustrated above is captured by a variation in strength characteristics as defined in table 1. Table 1 gives the average strength value from the testing campaign as well as the selected characteristic value.

*Table 1. Limestone hardness category strength*

| Induration | Average (MPa) | Character-istic (MPa) | Literature* (MPa) |
|---|---|---|---|
| H1 | 1.4 | 1.0 | 0.25 – 1 |
| H2 | 5.7 | 2.1 | 1-5 |
| H3 | 18.6 | 10.0 | 5-25 |
| H4 | 28.4 | 25.0 | 25-100 |
| H5 | 73.6 | 25.0 | 100-500 |

*Foged and Hansen (2002)

The relation between UCS and hardness is being observed on a quite continuous way on the rock corings, allowing for a continuous mapping of the rock. A rock strength profile as shown as the blue line figure 3 can be created at the location of each pier. The figure also plots the measured UCS or PLT results at the core taken, as well as the variation in GSI.

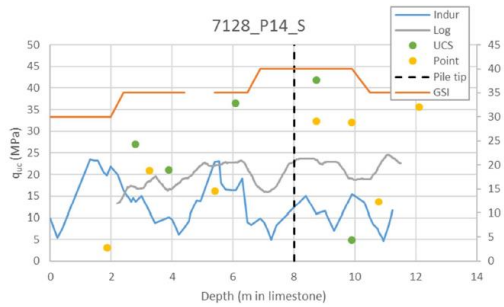


*Figure 3: Strength profile at location of test pile 1*

## 2.2 Design Method

Pile design in rock is challenged by choosing a method to estimate bearing capacity and by spatial variability. These uncertainties need to be considered in the design validation process.

### 2.2.1 Bearing capacity

The compressive resistance is calculated as the sum of shaft and end bearing resistances

$$R_c = R_s + R_b$$

Literature provides several empirical and analytical methods for the assessment of the compressive resistance of the pile socket in the rock.

The shaft resistance has been assessed using(Zhang, 2004):

$$R_{s,ber}/A_{si} = \eta_c \cdot \alpha \cdot q_{uc}{}^{\beta} \cdot \gamma_m$$

Where $q_{uc}$ is the unconfined compression strength, $\eta_c$ is a construction factor (the highest possible construction factor of 0.9 was chosen), α and β are constants (respectively 0.2 and 0.5).

The pile tip resistance in limestone has been estimated based on the method proposed by Serrano and Olalla (1994), based on Hoek-Brown rock model considering a 100% pile tip contact. The input parameters are UCS, m and GSI.

### 2.2.2 Spatial variability: ξ factor

In above formulas one has to account for the spatial variability of the rock compressive strength. The size of the pile compared to the autocorrelation length of the UCS leads to a reduction of the variability of the soil property.

The characteristic UCS used in the bearing assessment is defined as follows:

- Locally low and high rock strengths are averaged over the length of the shaft,
- A quite small volume of rock is involved in the pile tip compression resistance , therefore a local value is considered over a depth corresponding to 1 pile diameter under the pile tip.

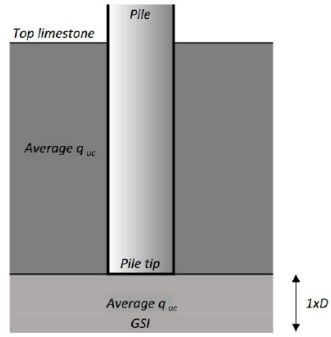


*Figure 4: Pile design parameters*

## 2.3 Pile Test

The design (calculation method, value of ground parameters end effects of pile execution on bearing capacity), is validated by bi-directional static pile tests and correlation of their results with a-priory prediction (type A) of the bearing capacity.

The ground parameters for calculations have been deduced from the local soil investigation (two boreholes and observed log of hardness) ; the local correlation between hardness and $q_{uc}$ appeared to be well in line with the overall correlation obtained from the entire soil investigation

*Table 2. Pile characteristics used for type A prediction*

| Test Pile | Rock Socket (m) | Pile diameter (m) | UCS Shaft (MPa) | UCS tip (MPa) | GIS tip (-) |
|---|---|---|---|---|---|
| 1 | 8.0 | 1.25 | 15 | 12 | 40 |
| 2 | 5.2 | 1.25 | 19 | 7 | 30 |

The test pile has a nominal diameter of theoretically 1,2 m, which is smaller than the final piles. The pile diameter as built has been deduced from actual concrete volume to correct the predicted pile bearing capacity. Further on, the type A prediction was done considering the average rock strength values as given in table 1 and not the characteristic value used for pile design.

The O-cell is placed 1.0m above the tip of the test piles, allowing to measure nearly separately tip bearing and shaft resistance.

The ultimate bearing capacities predicted based on the design methodology as explained above and the soil characteristic as given in table 2 are:

· Test 1: Above O-cell 23.3 MN ; Below O-cell 20.5 MN

· Test 2: Above O-cell 15.8 MN ;Below O-cell 10.9MN

Those values are of be demonstrated by means of the load test. Figure 5 shows the load displacement curve for both piles, including the targeted load mobilization.

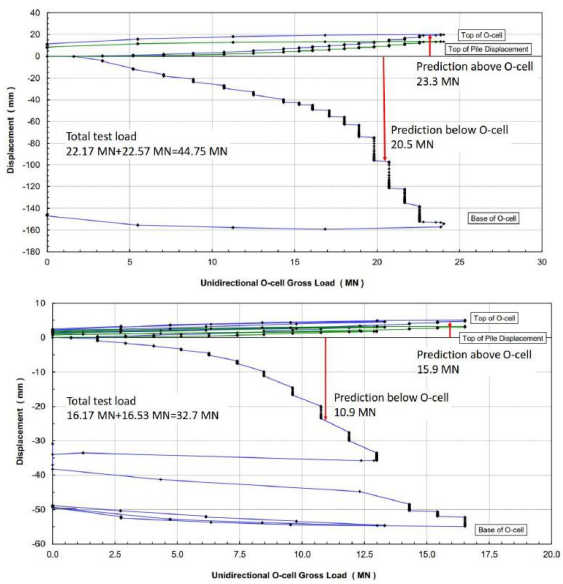


*Figure 5: load displacement curves above and below O-cell and comparison with predicted ultimate resistances*

The load settlement curve of the lower part indicates that a large part of the end bearing has been mobilised during the test, while the load-displacement curve of the upper part shows a lower degree of mobilisation of the ultimate shaft resistance. Both load tests validate the predicted ultimate tip and shaft resistance.

The stiffer behaviour in friction compared to end bearing has been accounted for in design, by the additional requirement that the service load (unfactored) has to be carried by unfactored shaft resistance only.

## 3 EXECUTION CHARACTERISTICS

The bored piles performed onshore were constructed using temporary thick walled steel casing penetrating op to the top of the rock. Piles performed off-shore (in the Fjord) were constructed using a lost thin walled steel casing driven (in sequence with internal excavation) until the top of rock. In both cases, water in excess of 1 m compared to external water level is used as support fluid.

### *3.1 Concrete mix*

Concrete is installed with the tremie pipe, requiring the high slump flow type (>210mm), workability during the entire concreting operation (of up to 5 Hours and 180m³) and controlled setting in order to allow placeability, deformability and to limit wash-out.

*Table 3: concrete mix for bored piles*

| **Property** | **Concrete mix** | |
|---|---|---|
| | kg | VOL% |
| Cem I HS | 372 | 11.6 |
| Fly ash | 75 | 3.3 |
| Microsilica | 22 | 1.0 |
| Water | 170 | 17.0 |
| Sand | 787 | 30.2 |
| 4-8mm | 337 | 13.0 |
| 8-16mm | 482 | 18.5 |
| Plasticizer(VAR) | 8.7 | 2.1 |

### *3.2 Reinforcement and sonic tubes*

The rebar cage and the included sonic tubes have properties as listed in Table 4:

*Table 4. Rebar cage properties*

| Property | Pile D=1.5m | Pile D=2.0m |
|---|---|---|
| Diameter pile/ Diameter socket | 1500mm 1350mm | 2000mm 1830mm |
| Rebar cage pile/ socket | 1200mm/ 1200mm | 1750mm/ 1680mm |
| Rebar spacing | 200mm | 162 |
| Rebar bundle | 2*d25+d20 | 2*d32 |
| Sonic tube | 4/pile D=101.6mm | 5/pile D=101.6mm |

### *3.3 Execution acceptance criteria*

Prior to the start of the pile execution, pile validation criteria were agreed upon between designer, employer and employer‘s consultant and specialised piling sub-contractor. The acceptance criteria were defined clearly in an inspection and test plan (ITP). The following validation checks were agreed upon:

- Validation of top of limestone
- Limitation on inflow of fine
- Validation of casting parameters
- Pile integrity testing

To define acceptance criteria for the pile tip contact, a core was taken at the pile tip of the test pile. however the recovered core had high core loss just below the pile tip and did not allow for assessing the pile tip contact.

Even though more than the estimated pile tip resistance could be mobilised at the pile tip during pile testing, as additional measure, it was decided to perform a grouting at the pile tip. The grouting would take away potential doubts related to execution and also reduce the impact of the spatial variability of the in situ rock.

## 4 PILE INTEGRITY TESTING

Sonic investigation testing was performed on all 52 foundation piles of the High Bridge. Pile anomalies were observed from the sonic integrity tests. Hence further investigations were needed at the locations that anomalies were detected.

### *4.1 Crosshole Sonic Logging (CSL)*

#### *4.1.1 CLS test Report*

The Cross hole Sonic Logging (CSL) method measures the sonic wave speed throughout the concrete of the casted drilled shaft (FHWA Drilled shafts manual, 2010). The variations in measured wave speed give an indication of the concrete integrity.

The factual report of the CSL tests shows:

- First arrival time(FAT) (red in Fig. 6),
- Apparent wave speed (shown in green)
- The recorded relative energy (blue)
- Sequential arrival of the sonic impulses in a waterfall diagram (right side of figure 6).

#### *4.1.2 Acceptance criteria*

Various acceptance criteria have been developed for validation of the pile using the CSL test. The most relevant recommendations are summarised below:

- ASTM D6760-16 states engineering judgement is required for result interpretation
- Federal Highway administration (FHWA) considers a poor quality concrete as a 20% wave speed reduction and an increased attenuation of 9dB
- NF P94-160-1 states an anomaly is observed if FAT>20% AND Attenuation >14 dB

From above it is clear that the recommendations are not uniform, nor that all data obtained from the CSL tests are considered.

### *4.2 CSL test Observations*

#### *4.2.1 Gravel Inclusions*

In figure 6, a CSL test result for 1 profile is shown. The profile contains two anomalies in energy and wave speed.

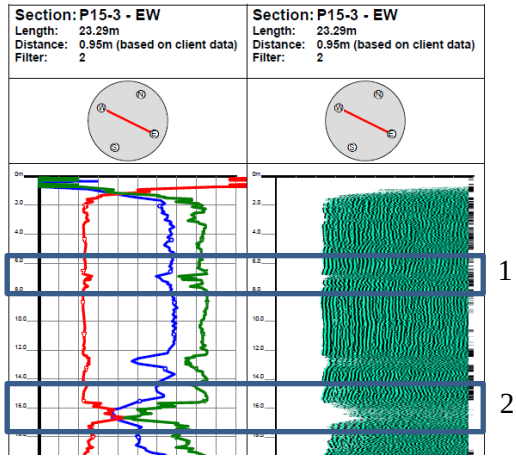


*Figure 6: Integrity test with anomalies*

To investigate the anomalies a core (D100mm) from the pile was retrieved. The location of the core is defined by selecting the most onerous area from the 6 CSL test profiles.

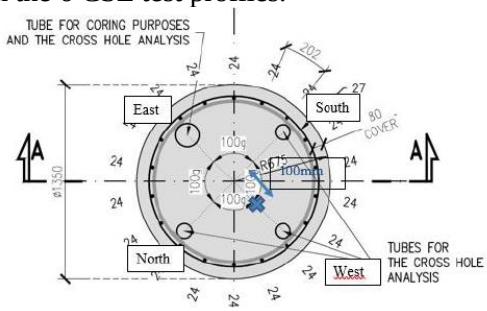


*Figure 7: Selected location for pile coring*

The coring shows important (>50cm) concrete segregation 16m to 18m below top of pile, marked as area 2 in figure 6, as show in figure 8. Hence confirming the CSL effectively identified the deviation of pile integrity.

*Figure 8; Core picture of gravel nests*

### *4.2.2 Mortal Inclusions*

In the same piles where important gravel inclusions were found, inclusions of mortar, with thickness of 40cm, were detected at higher levels in the pile. Tests were done on these samples and the compressive strength ranged from only 5MPa to 15Mpa. Which is considerably less than the required concrete strength (30/37).

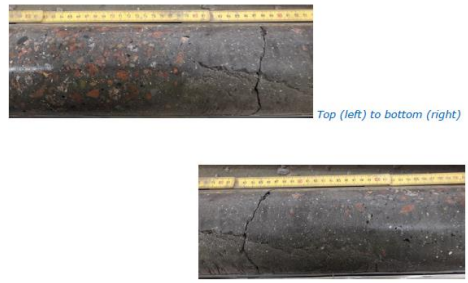


*Figure 9: Core picture of mortar inclusion*

The sonic profile of the concerned pile, as shown in figure 6, seems to primarily show the impurity at the transition between the mortar and the concrete and not the extent of the mortar inclusion. This same observation was made in another pile coring. It is therefore important to consider the entire pile integrity when concrete segregation is observed.

### *4.2.3 Wave speed deviations, but no anomaly*

Further on, in other piles, locations have been found where the apparent wave speed was reduced by more than 20%. 2 Of those events are shown in figure 10.

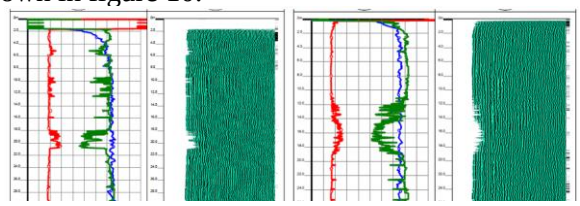

*Figure 10: Variations not confirmed to be an anomaly*

Those locations have been investigated by means of coring and no anomaly was found in the pile. The reduced apparent wave speed could be a result of not recorded first impulses.

### *4.3 CSL acceptance criteria*

At locations in the pile where anomalies were found, a pattern was consistently found , showing both FAT and relative energy deviating in multiple FAT profiles. At these locations, the waterfall diagrams show a continuous disruption

These observations demonstrate that not only is speed of the soundwave is to be considered to evaluate pile integrity but also relative energy and even the sequence of impulses as shown in the waterfall diagram. This is also in line with recommendation from earlier observations made in literature (drilled shafts manual, 2010 ; Beckhaus et al. 2015 ; Amir et al. 2004).

In case a concern on the integrity of the piles is triggered (meaning FAT is increased by minimal 20%), following procedure was applied for the validation assessment of the pile:

1) Are the deviations in wave speed confirmed over multiple signals in the waterfall diagram
2) Is the change in attenuation more than 9 dB, compared to the average attenuation
3) Are at least 2 of the measured paths within the section affected at the same level with the same characteristic pattern

If a variation of the apparent wave speed raised a concern, but no other of the 3 deviations as described above are observed, the testing performed on this project have demonstrated that there is no concern about the integrity of the pile and that the pile can be validated.

## 5 PILE TIP GROUTING

The pile tip grouting, performed to validate pile tip contact is done through the reservations made for the sonic testing (see also figure 7).

At 3 locations in the outer pile perimeter, holes are drilled 300mm below pile tip. Hence, a targeted grouting at the pile tip wasdone. The 3 locations are than put under pressure by means of packers located in the bottom of the piles.

The first grouting is done with a low viscosity grout mix that could penetrate in small openings. If the injected quantities exceed the volume of rock mass directly under the pile tip, a more viscous material is used. The used grout mixes are shown in Table 5.

*Table 5. Grout mix for tip grouting*

| Mix | Water/ cement | plasticizer/ cement | Viscosity (Marsh, s) | Density (g/cm²) |
|---|---|---|---|---|
| M1 | 0.65 | 1.0 | 34 | >1.55 |
| M2 | 0.50 | 1.0 | 38 | >1.70 |

The pile tip grouting is validated if a pressure of 10 bar is reached at each of the 3 injection points per pile. The injected volumes of grout per pile are illustrated in figure 11.

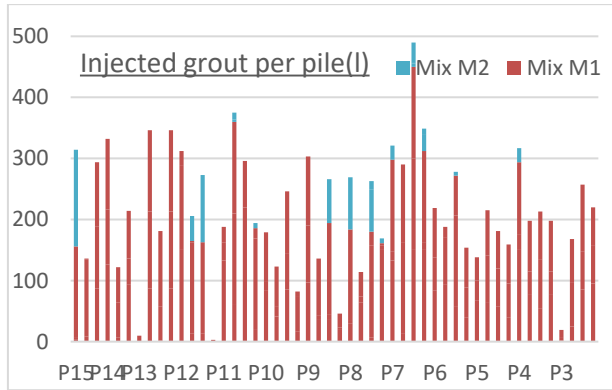


*Figure 11: Injected grout volume per pile*

In average, each of the 3 injection points per pile were injected by 65 liters, resulting in 204l grout/pile. The volume cored prior to the injection and required to be filled by grout amounts to 6.8l/pile.

Only in 19 of the 150 injection points, injection with more viscos M2 mix was needed in order to reach the pressure of 10bars. Only in 1 pile, the threshold of 150l injection was exceeded at all 3 injection points. This indicates that the rock was more porous at that exact location.

At 25 points, none or negligible injection was possible with the pre-defined pressure limit. Overall the injected grout quantities can be seen as relatively small.

## 6 CONCLUSIONS

An integrated approach, managed by a single party throughout all steps of the pile validation process was proven to be beneficial. A persistent philosophy from design through validation testing and validation in execution approach has resulted in both quality and cost benefits.

The applied design method considering a strength profile based on the induration profile and the estimation of shaft resistance based on Zhang and pile tip resistance by Serrano-Olalla, was successfully validated by means of the performed pile tests.

From the observations made when comparing the sonic test results with cores, it is shown that the relative energy is to be considered when assessing the pile integrity. A European wide recognised guideline on validation by means of sonic testing would be beneficial.

Further investigation and uniformisation with regards to validation of pile tip contacts of large bored piles is also in the interest of the industry. The validation of the pile tip is currently being assessed in a similar way as the pile. However, the sonic profiles focus on the outskirts of the pile, the sonic tests are likely to identify anomalies at the pile tip, especially for large diameter piles.

## 7 ACKNOWLEDGEMENTS

The pile validation exercise on the project was an intense cooperation between contractor's site team, the designer Sweco, contractor's specialists from the head offices, the Employer DRD and the Employer supervision from ARUP. The authors wish to thank all those parties for their positive attitude and input.

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