Summary and evaluation of pile test results

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Revision: 1 / 28 March 2014

Time Effects on Pile Capacity
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shall be made without consent from NGL.
The main objective of the project “Time effects on pile capacity” has been to establish how the axial tensile bearing capacity develops over time after a pile has been installed in the ground. The focus is on long term “ageing” effects that take place after possible excess pore pressures set up along the pile during installation are fully dissipated, but procedures for predicting the build-up of shaft friction in clays during the re-consolidation phase are also dealt with.

To achieve the overall objective a comprehensive pile load testing program has been undertaken at six different test sites. The table below presents typical ground conditions and dimensions of the test piles installed.

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### Summary

The main objective of the project “Time effects on pile capacity” has been to establish how the axial tensile bearing capacity develops over time after a pile has been installed in the ground. The focus is on long term “ageing” effects that take place after possible excess pore pressures set up along the pile during installation are fully dissipated, but procedures for predicting the build-up of shaft friction in clays during the re-consolidation phase are also dealt with.

To achieve the overall objective a comprehensive pile load testing program has been undertaken at six different test sites. The table below presents typical ground conditions and dimensions of the test piles installed.
Six test piles were installed at each site (only 5 at the Femern test site). All piles were driven open ended. A main reason for that was to limit the time required for reconsolidation for piles installed in clay.

As originally planned for, all piles were loaded to failure in tension, which reduced any impact of tip resistance and make it easier to deduce ageing effects on the ultimate shaft friction without having to instrument the piles with strain gauges or load cells.

The piles were loaded to failure at 5 different times, typically 1(2), 3(4), 6, 12 and 24 months after pile installation as marked with an “X” in the following table. As an example, Pile No. 3 was loaded to failure first time after 6 months, and then after 12 and 24 months. Test pile No. 6 at each site was first subjected to a sustained load corresponding to about 60 % of the assumed failure load. This sustained load was applied from 6 to 24 months after pile installation, when the pile was finally loaded to failure.

<table>
<thead>
<tr>
<th>Test site</th>
<th>Typical soil conditions</th>
<th>Pile diameter (mm)</th>
<th>Depth to pile tip (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stjørdal</td>
<td>Low plastic normally consolidated silty CLAY</td>
<td>508</td>
<td>23.6</td>
</tr>
<tr>
<td>Onsøy</td>
<td>Medium plastic normally consolidated CLAY</td>
<td>508</td>
<td>19.1</td>
</tr>
<tr>
<td>Cowden</td>
<td>Stiff overconsolidated low plastic glacial till CLAY</td>
<td>457</td>
<td>10.0</td>
</tr>
<tr>
<td>Femern</td>
<td>Stiff highly plastic overconsolidated palaeogene CLAY</td>
<td>508</td>
<td>25.0</td>
</tr>
<tr>
<td>Larvik</td>
<td>Loose to medium dense fine SAND with some clayey silt</td>
<td>508</td>
<td>21.5</td>
</tr>
<tr>
<td>Ryggkollen</td>
<td>Medium fine to coarse, medium dense SAND with some cobbles</td>
<td>406.4</td>
<td>20.0</td>
</tr>
</tbody>
</table>

These systematic pile testing programs confirm that there are significant and positive gains in ultimate shaft friction with time for piles installed in both clay and sand deposits. For piles in clay this is a gain that comes in addition to the...
normal set-up due to dissipation of excess pore pressures generated during pile installation. For piles in sand it is a pure ageing effect, see the following illustration.

The observed ageing effect in clays suggests a linear increase in ultimate shaft friction with logarithm of time after the end of re-consolidation, but the rate of increase depends on the type of clay. The increase is clearly the largest for lean normally consolidated clays with low plasticity index as at the Stjørdal test site (about 55% gain from 3 months to 2 years), and smallest in highly plastic overconsolidated clays like at the Femern test site. The ageing effect also tends to reduce with increasing overconsolidation ratio of the clay. Some older data reviewed and included in the study suggest that the ultimate shaft friction may keep increasing for at least two decades after pile installation.

The gain in capacity with time for piles driven in sand deposits was found to be surprisingly large (increase by factor of about 2), and even larger than for piles installed in clay deposits. Unlike for piles in clay, the gain in ultimate shaft friction with time does not seem to follow a linear increase with logarithm of time, and tends to level off 1 to 2 years after pile installation.

The new pile test results show that there are distinct effects of repeated (staged) loading as well as of sustained loading on the gain in ultimate shaft friction with time. These effects are also very different for piles installed in sand as compared to in clay deposits, and needs to be accounted for when applying the results of this and earlier studies of time effects.

As discussed in the report, there are some uncertainties associated with interpretation of some of the individual load test results. There is also some inherent variability in local soil conditions revealed by the site investigations, especially at the two sand sites. This introduces some uncertainty in the interpreted gain in capacity with time, which should be considered when applying the results of this study in design practice.
The tentative procedures proposed herein for dealing with time or ageing effects should be applicable in design practice, irrespective of pile dimensions, and for piles loaded in compression as well as tension.

For piles subjected to a significant cyclic loading component it is tentatively suggested that the static reference capacity used as basis for addressing cyclic loading effects can be the upgraded “aged” static capacity as defined herein. This means that the cyclic capacity will increase by the same amount as the static capacity as result of ageing effects. Supplementary cyclic tests should be considered to verify this assumption.

Chapter 7 of the report presents a more comprehensive summary of the results, where also some recommendations for further studies are given.
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Review and reference page
1 Introduction

1.1 General objectives

The objective of the joint industry project “Time effects on pile capacity” has been to establish design procedures to account for the increase in axial bearing capacity of driven piles with time primarily resulting from “ageing”. For piles in clay this primarily relates to increase in capacity that comes in addition to the gain in capacity resulting from normal re-consolidation of the excess pore pressures set up in the clay surrounding the pile as a result of the pile installation. Such anticipated gain in pile capacity due to “ageing” is not incorporated in existing Design Codes like the American Petroleum Institute’s (API) guideline or other international codes for offshore or onshore structures.

For the past two decades pile research has been a focal point of geotechnical research internationally, and has played an important role in the development of new and improved design procedures. If the expected increase in bearing capacity with time can be accounted for in design, it will open new possibilities for upgrading of offshore or onshore structures on driven piles that in their lifetime will experience increase in the loads on the foundations.

Accounting for the expected increase in pile capacity with time would also have a positive impact on the “first time” design of pile foundations. For many structures it often takes several months, if not up to a year or two, from the piles are driven to the structure is completed and the full design loads are applied.

There is little doubt that accounting for the potential gain in pile capacity with time has a potential for significant direct and indirect cost savings. As an example, in relation to offshore oil or gas production it often becomes a need to install new and/or heavier production equipment onto existing platforms. If the present piles can be demonstrated to have enhanced their capacity due to ageing it may allow the added loads to be put on to the jacket structure without any measures to strengthen the foundation. In the extreme case that may eliminate the need for installing a new platform at the site. This could imply cost savings in the order of a hundred million USD or more.

As another example, pile foundations for buildings and bridges will often first experience their full design load 1 to 3 years after the piles were installed. At such a time there may already be significant gain in capacity compared to the time for normal set-up. Accounting for such time or ageing effects in the design may significantly reduce the total need for piles or pile materials.

This research project has the following sub-objectives:

- Establish and update present “state of the art” on time effects on pile bearing capacity
- Perform new pile load tests specifically designated to reveal time effects
- Develop new design procedures and ensuring that they will be incorporated into national and international design codes

1.2 Pile load testing program

The research program comprises an extensive series of new pile load tests that have been undertaken at six different test sites, 4 clay sites and 2 sand sites with key soil parameters as summarised in Table 1.2.1. Two of the test sites are located outside Norway, namely the Cowden UK glacial till site and the Femern highly plastic Palaeogene clay site.

Two of the clay sites are in essentially normally consolidated (NC) clays, and two in overconsolidated (OC) clays with a wide range in plasticity indexes. Note that the OCR values given in the Table 1.2.1 are representative for the lower about 2/3 of the test piles, and neglects the upper weathered and higher OCR zone. The sand site at Larvik site is a fresh alluvial sand deposit located at the mouth of the river Numedalslågen. The gravelly sand deposit at Ryggkollen is part of an end moraine. This sand deposit is overconsolidated due to removal of about 15 m of overburden in connection with a sand quarry at the site. It could also to some extent be overconsolidated by actions of the glacier when the end moraine was formed. It has not been possible to take piston samples of this sand. Therefore the in-situ water content is not known.

Six tubular steel test piles were installed at each test site (only 5 at Femern). They have a diameter around 500 mm and were driven into the ground to tip penetrations mostly in the range 20-25 m, but only 10 m at the Cowden UK site. All piles were load tested in tension at various time intervals over a period of about 2 years.

<table>
<thead>
<tr>
<th>Site</th>
<th>Type of soil</th>
<th>Water content (%)</th>
<th>Plasticity index (%)</th>
<th>OCR (clay)</th>
<th>Dr (% sand)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stjørdal</td>
<td>Silty NC clay</td>
<td>28-32</td>
<td>12-16</td>
<td>1.4-1.6</td>
<td>-</td>
</tr>
<tr>
<td>Onsøy</td>
<td>Plastic NC clay</td>
<td>48-70</td>
<td>22-40</td>
<td>1.3-1.6</td>
<td>-</td>
</tr>
<tr>
<td>Cowden, UK</td>
<td>OC glacial till</td>
<td>16-17</td>
<td>17-19</td>
<td>4-10</td>
<td>-</td>
</tr>
<tr>
<td>Femern, Germany</td>
<td>High plastic OC Palogene clay</td>
<td>35-40</td>
<td>70-170</td>
<td>3-8</td>
<td>-</td>
</tr>
<tr>
<td>Larvik</td>
<td>Loose fine silty sand</td>
<td>20-31</td>
<td>-</td>
<td>-</td>
<td>20-40</td>
</tr>
<tr>
<td>Ryggkollen</td>
<td>Dense OC gravelly sand</td>
<td>?</td>
<td>-</td>
<td>-</td>
<td>50-80</td>
</tr>
</tbody>
</table>

Table 1.2.1 – Overview of test sites used for new pile load tests
1.3 Project organisation and financing

The project has a total budget of 46.6 million NOK. The project has been financed by direct or indirect contributions from a consortium of different type of partners as summarised in Table 1.3.1. The major funding of NOK 14.4 million comes from the Research Council of Norway (NRC). Other cash funding amounts to NOK 12.8 million. The remaining 19.4 million come from indirect contributions from the participants. This has been in the form of supplying equipment and materials, assistance with site work at reduced rates, making in-house data on relevant past pile test results available to the project, reviewing and commenting on reports prepared, and participating in meetings.

It should be noted that the pile tests at the Femern test site located in the sea between Denmark and Germany, was fully financed by Femern AS, a state owned company planning a new fixed road and rail link between Denmark and Germany at this location.

Figure 1.3.1 presents a schematic organisation plan for the project. All participants are represented in the Steering Committee (SC) established for the project. Table 1.3.2 presents the names of the key persons from each company that is represented in the SC. Some of the persons have been replaced during the course of the project. The names of earlier participants are shown in parenthesis.

![Schematic organisation plan](image)

Figure 1.3.1 – Schematic organisation plan
**Table 1.3.1 – Overview of organisations participating in the project**

<table>
<thead>
<tr>
<th>Name of organisation</th>
<th>Category</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Research Council of Norway</td>
<td>Main sponsor</td>
<td>Provides funding of NOK 14.4 million</td>
</tr>
<tr>
<td>Multiconsult AS</td>
<td>Main contract partner to RCN</td>
<td>Provides some work, data and funding</td>
</tr>
<tr>
<td>NGI</td>
<td>Responsible technical organisation and contract partner to JIP partners</td>
<td>Provides main part of work, some data and funding</td>
</tr>
<tr>
<td>Aramco</td>
<td>JIP- partner</td>
<td>Provides funding of NOK 2 million</td>
</tr>
<tr>
<td>Total E&amp;P Norge AS</td>
<td>JIP- partner</td>
<td>Provides funding of NOK 2 million</td>
</tr>
<tr>
<td>Statoil</td>
<td>JIP- partner</td>
<td>Provides funding of NOK 2 million</td>
</tr>
<tr>
<td>Femern AS</td>
<td>JIP- partner</td>
<td>Provides funding of NOK 2 million</td>
</tr>
<tr>
<td>Petronas Carigali SDN BHD</td>
<td>JIP- partner</td>
<td>Provides funding of NOK 2 million</td>
</tr>
<tr>
<td>Kværner (Aker Solutions)</td>
<td>Industry- partner</td>
<td>Provides funding of NOK 1 million and some work</td>
</tr>
<tr>
<td>Norwegian Directorate of Public Roads</td>
<td>Government institution</td>
<td>Provides funding and some data</td>
</tr>
<tr>
<td>Rautarruukki Oyj (RUUKKI)</td>
<td>Industry- partner</td>
<td>Provides all pile materials (except Cowden and Femern)</td>
</tr>
<tr>
<td>Entreprenørservice AS</td>
<td>Industry- partner</td>
<td>Provides some equipment and site work</td>
</tr>
<tr>
<td>Kynningsrud Fundamentering AS</td>
<td>Industry- partner</td>
<td>Provides some site work</td>
</tr>
<tr>
<td>SKANSKA Norge AS</td>
<td>Industry- partner</td>
<td>Provides mainly data</td>
</tr>
<tr>
<td>Building Research Establishment, UK</td>
<td>Research partner</td>
<td>Provides work (Cowden) and data</td>
</tr>
<tr>
<td>NTNU</td>
<td>University</td>
<td>Provides student assistance, MSc and PhD candidates</td>
</tr>
</tbody>
</table>
Table 1.3.2 – Names of participant in the Steering Committee

<table>
<thead>
<tr>
<th>Name of organisation</th>
<th>Name of contact person /member of Steering Committee</th>
</tr>
</thead>
<tbody>
<tr>
<td>Research Council of Norway</td>
<td>Jørn Lindstad</td>
</tr>
<tr>
<td>Multiconsult AS</td>
<td>Arne Schram Simonsen</td>
</tr>
<tr>
<td>NGI</td>
<td>Kjell Karlsrud</td>
</tr>
<tr>
<td>Saudi Aramco</td>
<td>Jonathan Grosch (Albert Griffith)</td>
</tr>
<tr>
<td>Total E&amp;P Norge AS</td>
<td>Jean-Louis Colliat-Dangus</td>
</tr>
<tr>
<td>Statoil</td>
<td>Geir Svanø</td>
</tr>
<tr>
<td>Femern AS</td>
<td>Jens Kammer</td>
</tr>
<tr>
<td>Petronas Carigali SDN BHD</td>
<td>Mohammad Joehan B. Rohani</td>
</tr>
<tr>
<td>Kværner (Aker Solutions)</td>
<td>Annette Jahr (Torstein Alm, Gunhild Hennum)</td>
</tr>
<tr>
<td>Norw. Dir. of Public Roads</td>
<td>Steinar Giske</td>
</tr>
<tr>
<td>Rautarruukki Oyj (RUUKKI)</td>
<td>Harald Ihler</td>
</tr>
<tr>
<td>Entreprenørservice AS</td>
<td>John-Petter Holtmon (Trond Øiseth)</td>
</tr>
<tr>
<td>Kynningsrud AS</td>
<td>Erling Omre</td>
</tr>
<tr>
<td>Fundamentering AS</td>
<td></td>
</tr>
<tr>
<td>SKANSKA Norge AS</td>
<td>Lars Bjerkeli (Bjørn Haavardsholm)</td>
</tr>
<tr>
<td>Building Establishment, UK</td>
<td>John Powell</td>
</tr>
<tr>
<td>NTNU</td>
<td>Prof. Steinar Nordal</td>
</tr>
</tbody>
</table>

1.4 Organization and content of the report

Chapter 2 summarizes soil conditions and factual pile test results site by site. Chapter 3 presents a summary of previously published pile load test results that are found of sufficient reliability and relevance to shed light on time effects and can be used as supplement to the new data generated in this project.
Chapter 4 makes an overall assessment of impact of time or “ageing” for piles in clay including the various test results generated in this project as well as previously published pile test data found of most relevance. To understand the contribution of “ageing” in contrast to normal reconsolidation effect on the bearing capacity of piles in clay, it is important to establish what the expected time for reconsolidation is at the different sites. This issue is dealt with in the beginning of Chapter 4. In the assessment of results and for developing new design procedures accounting for time effects it is attempted to separate the effects of re-consolidation, sustained loading and repeated loading on the same pile from “ageing” effects.

Chapter 5 deals in a similar way with piles in sand.

Chapter 6 assesses the pile capacities determined from PDA measurements and CAPWAP analyses made during pile installation and at the end of the test programs.

Chapter 7 presents a summary and recommendations for how to deal with time effects in engineering practice. This chapter also proposes a program for dissemination and publishing of results. This with the goal of ensuring that time effects will be included in standard codes of practice, both offshore and onshore. Recommendations are finally given for further studies that could contribute to widen the data base, and improve the reliability of the proposed design procedures to account for time effects.

2 Summary of testing arrangements, soil conditions and pile test results for each site

This chapter first summarize the general pile testing arrangements, and then the soil conditions and pile load test results at each site one by one. The results are extracted from the factual data reports on soil conditions and on pile test results that have been prepared for each site. Appendix A gives an overview of these data reports as well as progress reports prepared earlier as part of this project.

2.1 General description of testing arrangements- all sites

A brief general description of the general testing arrangements and procedures applied at the different test site is given in the following.

Table 2.1.1 summarize the dimensions of the tubular steel test piles used at the different sites. A casing was in general installed to depths of 1 to 5 m prior to pile driving to avoid influence of fill or special top soils on the pile performance. The only exception was at the Femern test site located in the sea, where no casing was installed. The time of installation of the test piles are given in the last column.
**Table 2.1.1 – Pile dimensions used at the different test sites**

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile diameter (mm)</th>
<th>Wall thickness (mm)</th>
<th>Depth of casing (m)</th>
<th>Depth to pile tip (m)</th>
<th>Embedded pile length (m)</th>
<th>Date installed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stjørdal</td>
<td>508</td>
<td>6.3</td>
<td>1</td>
<td>23.6</td>
<td>22.6</td>
<td>May 2009</td>
</tr>
<tr>
<td>Onsøy</td>
<td>508</td>
<td>6.3</td>
<td>1.4</td>
<td>19.1</td>
<td>17.7</td>
<td>Aug 2009</td>
</tr>
<tr>
<td>Larvik</td>
<td>508</td>
<td>6.3</td>
<td>1.4</td>
<td>21.5</td>
<td>20.1</td>
<td>June 2009</td>
</tr>
<tr>
<td>Ryggkollen</td>
<td>406.4</td>
<td>12.5</td>
<td>5</td>
<td>20.0</td>
<td>15.0</td>
<td>Aug 2009</td>
</tr>
<tr>
<td>Cowden</td>
<td>457</td>
<td>12.5</td>
<td>1</td>
<td>10.0</td>
<td>9.0</td>
<td>Oct. 2009</td>
</tr>
<tr>
<td>Femern</td>
<td>508</td>
<td>22.5</td>
<td>0</td>
<td>25.0</td>
<td>25.0</td>
<td>Oct. 2010</td>
</tr>
</tbody>
</table>

Six test piles were installed at each site, apart from Femern where test pile 6 - for sustained load testing - was not included. The original intention was to load test the test piles at different time intervals after driving as presented in Table 2.1.2. In reality the time for testing varied somewhat from site to site, see subsequent presentations.

**Table 2.1.2 – Planned load testing sequence on the different test piles at each site**

<table>
<thead>
<tr>
<th>Test pile No.</th>
<th>Time of testing after pile installation (months)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1(2)</td>
</tr>
<tr>
<td>1</td>
<td>X</td>
</tr>
<tr>
<td>2</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

The test piles were driven with hydraulic hammers to the desired tip penetration. The level of the soil plug inside the piles was monitored during driving for some of the piles. Table 2.1.3 presents the observed depth to the top of the soil plug inside each pile relative to the bottom of the cored out casings. Note that at the Femern site the plug was removed in parallel with the driving to ensure no plugging effects on the time for set-up. The reason for that was the very low permeability of this clay.

**Table 2.1.3 – Depth to top of soil plug below depth of cored out casing (metres)**

<table>
<thead>
<tr>
<th>Pile no</th>
<th>Onsøy</th>
<th>Stjørdal</th>
<th>Larvik</th>
<th>Ryggkollen</th>
<th>Cowden</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.85</td>
<td>8.9</td>
<td>3.07</td>
<td>2.33</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td>2.69</td>
<td>8.6</td>
<td>3.39</td>
<td>2.87</td>
<td>1.6</td>
</tr>
<tr>
<td>3</td>
<td>2.35</td>
<td>8.3</td>
<td>3.34</td>
<td>3.23</td>
<td>1.1</td>
</tr>
<tr>
<td>4</td>
<td>3.96</td>
<td>9.4</td>
<td>3.31</td>
<td>3.61</td>
<td>1.9</td>
</tr>
<tr>
<td>5</td>
<td>3.57</td>
<td>6.0</td>
<td>3.46</td>
<td>10.27</td>
<td>1.6</td>
</tr>
<tr>
<td>6</td>
<td>2.36</td>
<td>8.2</td>
<td>2.73</td>
<td>5.52</td>
<td>2.05</td>
</tr>
</tbody>
</table>

1) At Femern the plug was drilled out in parallel with pile driving
Calculated times for degree of consolidation at the time of pile testing is presented in Chapter 4.2 for all the clay sites. Apart for the Femern site the calculated times to reach 90 % consolidation is typically from $t_{0.9} = 1$ month and to reach 95 % consolidation $t_{0.95} = 2-3$ months. At Femern it is much longer, respectively $t_{0.9} = 25$ months and $t_{0.95} = 210$ months.

Figure 2.1.1 illustrates an example of the loading frame used at the test sites in Norway. The loading frame was designed to transmit axial tension to the top of the pile through a hydraulic cylinder. A special actuator that can maintain a constant load over a long time period of time, as needed for test pile No. 6, was used to apply the load through the hydraulic jack mounted on top of the reaction frame.

The load was measured with a vibrating wire type load cell mounted above the pile and the hydraulic cylinder.

The loading frame was equipped with wheels supported on railway tracks on top of two reinforced concrete strip foundations, allowing the loading frame to be easily moved from one pile to the next, cf. Figure 2.1.1 and 2.1.2.

![Figure 2.1.1 – Loading frame and loading system setup used at test sites in Norway](image)
To measure the displacement of the pile during pile testing a displacement transducer (Temposonics R-series) was used. The displacement transducer was fitted to the moveable pile collar before every test. The position magnet was connected to the reference beam, which was supported by two concrete footings placed at a distance of approximately 2.5 metres to either side of the pile, Figure 2.1.3. Both load and displacement were logged continuously during the load tests.
The test set-ups used at the Cowden and Femern sites were somewhat different, ref. the reports for these tests listed in Appendix A. At Cowden a main difference was that the sustained load on pile No. 6 was applied by means of dead loads and a cantilever beam rather than by a hydraulic actuator. At Femern the reaction frame and loading system was placed on driven piles at seabed level, see Fig. 2.1.4. The testing at Femern was carried out by Aarsleff AS and presented in a report by Femern AS (2012). No sustained load test was included at Femern.

![Reaction frame and loading arrangement for piles installed at seabed level at Femern (after Femern AS, 2012)](image)

Figure 2.1.4 – Reaction frame and loading arrangement for piles installed at seabed level at Femern (after Femern AS, 2012)

The test piles at all sites, apart from at Femern, had two vibrating wire type earth pressure (e-p) cells mounted diametrically at about mid-depth of the piles. The intention was to see if any significant changes in earth pressures occurred over time. Unfortunately these e-p cells did not work as intended. Some fell out during driving, and others gave partly unstable or unreasonably low or high values. The results are therefore, not included in this report.

At Femern it was decided to install two piezometers on the pile wall instead of e-p cells. These functioned well and the results are included herein.

At all test sites the loads were applied incrementally in steps. Table 2.1.4 presents a typical test procedure for piles installed in clay, with larger load steps during the first part, then gradually smaller. The duration of each load step correspondingly decreased from 5 minutes during the first large load steps, to 3 minutes for the rest. After reaching defined failure, the piles were stepwise unloaded back to zero.

For the piles installed in sand the same stepwise procedure was used but the duration of the load steps was increased to 15 minutes to ensure fully drained conditions during these tests.
Table 2.1.4 – Example of loading procedure for clay sites (Stjørdal case)

<table>
<thead>
<tr>
<th>No. of increments</th>
<th>Load increment size (kN)</th>
<th>Duration of increment (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st loading sequence</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>100</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>50*</td>
<td>3</td>
</tr>
<tr>
<td>Until failure**</td>
<td>25</td>
<td>3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Unloading</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>100</td>
<td>3</td>
</tr>
<tr>
<td>Until zero</td>
<td>50</td>
<td>3</td>
</tr>
</tbody>
</table>

* Load step shall be reduced to 25 kN if the creep rate is indicating approaching failure load.
** Stop loading of pile when peak load is reached, or accumulated displacement exceeds 30mm.

2.2 Clay site Stjørdal

2.2.1 Summary of soil conditions Stjørdal

The Stjørdal test site is located about 35 km north of Trondheim in central Norway, and just on the north side of the Trondheim airport Værnes and the outlet of the river Stjørdalselva.

![Figure 2.2.1 – Location of the Stjørdal test site](image)

The complete site investigation at Stjørdal was documented in report 20061251-00-249-R, Appendix A. The site investigations consisted of 3 standard Norwegian rotary-pressure soundings, more or less continuous 72 mm piston sampling to a depth of 25 m, and 7 CPTU soundings. Testing of the samples has, in addition to routine type classification tests, included 7 constant rate of strain (CRS) oedometer tests, 14 UU triaxial tests, 3 anisotropically consolidated undrained triaxial compression (CAUC) and extension (CAUE) tests, and 3 undrained direct simple shear (DSS) tests.
Below an upper layer of fine sand extending to a depth of 3 m, there is a very uniform silty clay deposit. The clay is of marine post-glacial origin, deposited in the sea about 8000 to 12000 years ago.

The data in Figure 2.2.2 shows that the water content lies in the range w = 28 to 32 %, and the plasticity index range from Ip = 12 to 15 %. Sensitivity values vary between 2 and 8.

The Stjørdal clay has a content of clay size particles (< 2 μ) mostly in the range 23 to 26 %. It contains essentially zero sand size particles. In combination with the plasticity data it can therefore be classified as lean silty Clay. Analyses of the clay minerals show primarily illite (71 %), however some chlorite and quartz are also present. This is quite typical of Norwegian clays.

Figure 2.2.2 – Water content and plasticity index, Stjørdal

A special comment with respect to the plasticity index. It was measured as part of the routine testing carried out on the samples by Multiconsult (MC) at their laboratory in Trondheim, but was also measured on the samples shipped to NGI for triaxial and DSS testing. As can be seen from Figure 2.2.2, MC’s values are consistently lower than NGI’s values. This discrepancy may be due to differences in the detailed procedures followed for determination of the liquid and plastic limits. It seems like MC’s tests give consistently lower liquid limits than NGI’s tests, but higher plastic limits. The data in Figure 2.2.2 suggest however, that there is a correlation between water content and plasticity index, and that some of the differences could be explained by local variations in water content. Considering the uncertainties, it was in this study chosen to give most weight to the plastic limits determined by NGI, mainly because NGI have generated plastic limit data that lie behind a number of different empirical correlations used for characterizing clay behaviour.

Figure 2.2.3 show in-situ stress conditions derived from the soil data. The measured in-situ pore pressure is hydrostatic below the ground water table at depth of 2.0 m.
Only two of the oedometer tests gave results that allowed determination of the pre-consolidation pressure. The sample quality was too poor to allow that for the other five tests. The pre-consolidation pressure, \( p'_c \), and OCR profiles in Figure 2.2.3 were therefore based on back-calculation from the interpreted undrained CAUC triaxial strength (\( s_{u\text{c}} \)) values, as described in the following.

**Figure 2.2.3 – Assumed in-situ stress conditions, Stjørdal**

It is well established that the undrained strength of clays is closely related to the overconsolidation ratio, \( \text{OCR} = \frac{p'_c}{\sigma'_{v0}} \). Ladd and Foott (1974) suggested a relationship, based on what was called the SHANSEP procedure, as follows:

\[
s_{u}/\sigma'_{\text{v0}} = S \cdot \text{OCR}^m \tag{2.2.1}
\]

where,

- \( S = s_{u}/\sigma'_{\text{v0}} \) for OCR=1.0 is the normalised strength for a sample consolidated well beyond the pre-consolidation pressure, \( p'_c \)
- \( M = \text{power} \)

The value of \( S \) for OCR =1.0 would correspond to a young truly normally consolidated clay which has not had the opportunity to develop any apparent pre-consolidation pressures due to secondary (creep) consolidation, e.g. Bjerrum (1973). Based on samples that were artificially consolidated in the laboratory, Ladd and Foott (1974) suggested typical values of \( S=0.28 \) and \( m=0.85 \) for triaxial compression.

Figure 2.2.4 shows normalised strength derived from triaxial tests on very high quality block samples taken from Karlslrud et al (2005a). From Figure 2.2.4 it is apparent that natural clays do not show such a unique relationship between undrained strength and OCR as has been indicated by testing clays that have been artificially pre-consolidated in the laboratory (e.g. Ladd and Foott, 1974). The reason may be that soil structure and possible local chemical bonding or cementation effects play a role in-situ, which is lost when a sample is artificially pre-consolidated in the laboratory.
Figure 2.2.4 – Normalised CAUC strength values, $s_{uc}/\sigma'_{v0'}$, from tests on high quality block samples in relation to OCR (from Karlsrud et al, 2005a)

The selected OCR profile in Figure 2.2.3 was primarily based on values back-calculated with SHANSEP from the CPTU-based $s_{uc}$ values in Figure 2.2.6 assuming $S=0.28$ and $m=0.85$ for this clay. The data suggest that the clay is lightly overconsolidated, probably mostly as result of secondary creep, e.g. Bjerrum (1967).

Figure 2.2.5 summarize values for the Janbu (1963) modulus number, $m_0$, and the vertical permeability at zero volume change, $k_0$, as determined from the oedometer tests. The values in Figure 2.2.5 are quite typical of marine silty clays. Note that the virgin compression index is linked to the modulus number $m_0$ through the expression: $C_v/(1+e_0) = \ln(10)/m_0$.

Figure 2.2.5 – Modulus number and permeability from oedometer tests, Stjørdal
Figure 2.2.6 presents undrained shear strengths from the triaxial and DSS tests carried out on samples from 3 levels, as well as a CAUC strength profile derived from the tip resistance measured in the CPTU tests. The CPTU derived $s_{uc}$ strength profile was established using the correlations proposed by Karlsrud et al (2005a) between corrected tip resistance and $s_{uc}$ strengths, which was based on tests on very high quality undisturbed block samples. The cone factor used is typically $N_{kt}= 9.2$, but varies somewhat with depth. It may be noted that the pore pressure response in the CPTU tests at Stjørdal for some reason was very poor and had to be disregarded as basis for determination of undrained strengths.

The CPTU derived $s_{uc}$ strengths in Figure 2.2.6 agree very closely with the triaxial tests, which appeared to be of good quality. The same applies to the DSS tests which show a strength typically corresponding to $s_{ud} = 0.67s_{uc}$ (range 0.62-0.71). The selected $s_{ud}$ profile tries to reflect the variations in the CPTU derived values, but neglects the tendency for local high values from 22.5 to 26 m depth. (That is of little importance as the pile tip is at 23.6 m).

The curve marked “measured tus” in Figure 2.2.6 represents the average measured shaft friction in test S1-1, as will be presented later.

\[
s_{u} \text{ (kPa)}
\]

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure226.png}
\caption{Assumed undrained shear strength profile, Stjørdal}
\end{figure}

The triaxial tests show that the peak effective friction angle is typically $\phi = 34.2^\circ$. 
2.2.2 Summary of pile test results Stjørdal

The 6 test piles at Stjørdal had diameter of 508 mm and wall thickness of 6.3 mm. They were driven open ended through a 1m deep casing to a tip penetration of 23.6 m.

At the end of driving the depth to the top of the soil plug was measured to be from 6.0 to 8.9 m below bottom of the casing, corresponding to 27 to 39 % of the pile length. That these piles showed such a large tendency for plugging came as a surprise. The impact of partial pile plugging on time for re-consolidation or set-up is analysed and discussed in a later Chapter 4.2. At the Stjørdal site these analyses suggest that 90 % consolidation was reached after 0.9 month and 95 % consolidation after 2.2 months.

All the piles were driven with a hydraulic hammer with hammer weight of 60 kN, and using a constant drop height of 10 cm throughout. As seen from Figure 2.2.7 the blow count increased gradually with depth, and was practically identical for all piles, confirming that the ground conditions are homogeneous across the site.

The plug levels was measured more or less continuously during driving of pile S3, and at selected levels during driving of piles S5 and S6. Figure 2.2.8 presents the measured depth of the soil plug relative to the ground surface. The data suggest that the pile penetrated in a more or less fully plugged mode until the tip had reached 8 m depth, after which the tendency for plugging reduced rapidly. This change in plugging behaviour with penetration can be more clearly seen from Figure 2.2.9 presenting the incremental plugging ratio as function of pile penetration depth. The plugging ratio, p, is defined by Eq (4.2.5) in Chapter 4 as the volume of clay that has entered the pile for an incremental penetration of the tip divided by the gross outer volume of pile entering the ground.
Figure 2.2.7 – Summary of recorded blow counts during installation of the piles, Stjørdal (hammer weight 60 kN and drop height 10 cm throughout)

Figure 2.2.8 – Observed plug levels during pile driving
Figure 2.2.9 – Measured incremental plugging ratio versus pile tip penetration - Pile S3, Stjørdal

Figure 2.2.10 and Figure 2.2.11 show examples of load displacement curves for the repeated load testing on Pile S1 at Stjørdal, whereas Figure 2.2.12 compares the load-displacement curves only for the first time tests on each pile. The increase in capacity both with repeated loading on the same pile and with time is apparent.

Figure 2.2.10 – Accumulated Load-displacement response from the 5 repeated load tests on pile S1-1, Stjørdal site
Figure 2.2.11 – Load-displacement response from the 5 repeated load tests on pile S1-1, Stjørdal site

Figure 2.2.12 – Load-displacement response from 1st time load tests on piles S1 to S6, Stjørdal site
The load-displacement curves in Figure 2.2.10 to Figure 2.2.12 show that the failure developed in a rather non-brittle manner at the Stjørdal site, and a clear peak was not reached in many of the tests. This was especially the case for the 1st and 2nd tests on each pile. This is most likely due to a tendency for dilation and generation of negative pore pressures at large displacements. In the later tests on the same pile, the load levelled more off towards the end of the test. For this reason, the failure criterions for these piles have been defined as the largest of when:

- The pile top displacement exceeds 30 mm if a clear peak is not reached before that
- The rate of creep under sustained loading exceeds 1 mm/min.

The latter is the same failure criterion used by Karlsrud (2012).

Figure 2.2.13 show creep data for the 1st time tests on each pile. The creep rates for some of the tests tend to level off at somewhat less than 1 mm/min, which probably also is due to tendencies for dilation. The displacement criterion of 30 mm was used to define failure for these tests.

Note that for pile S6-1 the sustained load was reduced to zero in advanced of the loading to failure. That is probably the reason for small creep rates during loading up to about 600 kN in this test.

![Graph showing load-displacement curves](image_url)
Table 2.2.1 presents the ultimate failure loads defined from each test as well as the relative increase in failure load using test S1-1 as reference. Figure 2.2.14 presents the failure load versus time for all tests.

Table 2.2.1 and Figure 2.2.14 show that after 25 months the capacity had increased to 1.9 times of the initial capacity in the 1st time tests. The piles also show even larger increase in capacity when the same pile is loaded repetitively to failure, up to 3.27 times the initial ultimate capacity for test S3-3. Pile S6, which was subjected to a sustained tension load of 350kN over 2 years before being loaded to failure, showed capacity that is a factor of 2.42 times that of test S1-1. Thus such sustained loading also gives a further increase in capacity than what comes from only time, compare results of test S5-1 and S6-1.

The data from pile S1 suggest that repeated loading on the same pile reaches a plateau level, but the magnitude of this plateau level also tends to increase with time the pile is left in the ground before the 1st test on the pile is carried out.

Table 2.2.1 – Summary table of failure loads measured at the pile top, $Q_{utop}$, Stjørdal

<table>
<thead>
<tr>
<th>Stjørdal</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 1-1</th>
<th>Time (months)</th>
<th>Change factor relative to test 1-1</th>
<th>Time (months)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile No.</td>
<td>1,6 3,6 6,6 13,3 25,2</td>
<td>1,6 3,6 6,6 13,3 25,2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>480 860 975 1320 1380</td>
<td>1 1,79 2,03 2,75 2,88</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>700 910 1160 1470</td>
<td>1,46 1,90 2,42 3,06</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>700 1050 1570</td>
<td>1,46 2,19 3,27</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>760 1390</td>
<td>1,58 2,90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>910</td>
<td>1,90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>1160</td>
<td>2,42</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 2.2.14 – Ultimate pile top capacity, $Q_{utop}$, versus time – all tests Stjørdal
For each test the ultimate shaft friction has been determined from the failure load measured at the pile top and accounting for the weight of the pile. This gives the following equation for the ultimate load, $Q_{us}$, carried by shaft friction:

$$Q_{us} = Q_{utop} - W_{pile} + Q_{utip} \quad (2.2.2)$$

$Q_{utop}$ = Ultimate failure load measured at the pile top

$W_{pile}$ = Total weight of pile and soil plug inside the pile

$Q_{utip}$ = Ultimate load at the pile tip, considered positive in upward direction

The load carried by the tip has been calculated theoretically assuming full mobilization of tip resistance over the gross area of the pile as follows:

$$Q_{utip} = (\gamma z - 9 \cdot s_{ud}) \pi D^2/4 \quad (2.2.3)$$

$D$ = Pile diameter

For the Stjørdal site $W_{pile} = 105$ kN and $Q_{utip} = -9$ kN. The tip correction only amounts to 0.7 to 1.9 % of the failure load measured at the pile top.

The average ultimate shaft friction has further been normalized with respect to both the average undrained DSS strength, $s_{ud}$, taken from Figure 2.2.6, and the average vertical effective stress, $\sigma'_v$, in Figure 2.2.3. The average ultimate shaft friction, $\tau_{us}$, was found by dividing the deduced shaft load, $Q_{us}$, by the outside surface area of the piles. Figure 2.2.15 shows that the deduced values of $\alpha = \tau_{us}/s_{ud}$ starts off at an extremely low value of $\alpha = 0.32$ for the first test S1-1 and approach $\alpha = 0.68$ for the first time tests when the pile has been left in the ground for 25 months. Repeated load testing makes the $\alpha$-value approach 1.0. Pile S6 subjected to a sustained load of about 350 kN, corresponding to 60 % of the failure load in test S1-1, gave a value of $\alpha = 0.88$.

**Figure 2.2.15 – Mobilised $\alpha$-value versus time- all tests Stjørdal**
Figure 2.2.16 shows correspondingly the deduced values for $\beta = \frac{\tau_{us}}{\sigma'_{v0}}$ for all the tests. The first initial value corresponds to $\beta = 0.073$ and the 1st time tests reached $\beta = 0.16$ after 2 years, with repeated loading giving a maximum value around $\beta = 0.27$. Sustained loading over two years gave $\beta = 0.21$.

Figure 2.2.16 – Mobilised $\beta$-value versus time- all tests Stjørdal

Figure 2.2.17 presents the relative change (increase) in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested. A staged loading effect can be defined by a factor:

$$C_{st} = \frac{Q_{staged}}{Q_{non-staged}} \quad \text{(2.2.4)}$$

Values of $C_{st}$ were arrived at by taking the ratio between the ultimate capacities determined at the same time after pile installation relative to the 1st test on each pile, e.g. the ratio between the capacity for test (S1-2)/(S2-1), (S1-3)/(S3-1) etc.

The trend suggests that the staged loading factor increase in test 2, 3 and 4 but it then tends to stabilize. The maximum value is to about $C_{st} = 1.8$
2.2.3 PDA testing

PDA testing was performed in January 2013. The results were very difficult to interpret, partly due to questionable data quality. For that reason the results have not been included in this report, but can be found in the data report for the Stjørdal site, ref. Appendix A.

2.3 Clay site Onsøy

2.3.1 Summary of soil conditions

Figure 2.3.1 shows the location of the Onsøy test site, located along the National Road no. 110 (Riksveg 110) approximately 3 km NW of the city of Fredrikstad. The site is about 1 km away from the test site used in earlier pile studies by NGI, ref. Karlsrud et al (1990) and Karlsrud (2012). The clay characteristics are also very much the same as at that site.
The complete site investigation at Onsøy was documented in report 20061251-00-248-R, Appendix A. The site investigations consisted of 6 standard Norwegian rotary-pressure soundings, more or less continuous 72 mm piston sampling to a depth of 21 m, and 3 CPTU soundings. Some 54 mm piston samples were also taken at an early stage to check the feasibility of the site. Testing of the samples have, in addition to routine type classification tests, included 10 constant rate of strain (CRS) oedometer tests, 17 UU triaxial tests, 3 anisotropically consolidated undrained triaxial compression (CAUC) and extension (CAUE) tests, and 3 undrained direct simple shear (DSS) tests.

As shown in Figure 2.3.2, this normally consolidated clay deposit has water content in the range \( w = 47-70 \% \) and plasticity index of \( I_p = 24-40 \% \). The sensitivity of the clay is mostly in the range \( S_t = 4 \) to \( 7 \), but increase to \( S_t = 10 \) to \( 30 \) from depths \( 15 \) to \( 20 \) m.

The Onsøy clay has a content of clay size particles \( (<2 \mu) \) mostly in the range \( 44 \) to \( 66 \% \). It contains essentially zero sand size particles. In combination with the plasticity data it can therefore be classified as medium to highly plastic Clay. Analyses of the clay minerals show primarily illite and chlorite (about 90 \%). Some quartz and plagioclase are also present. This is quite typical of the marine clays in south eastern part of Norway.
Figure 2.3.2 – Index data, Onsøy

Figure 2.3.3 shows in-situ stress conditions derived from the soil data. The measured in-situ pore pressures show a slight artesian condition as compared to the hydrostatic, with measured ground water table at a depth of 0.8 m. The pre-consolidation pressures derived from oedometer tests were judged to be on the low side due to some apparent effects of sample disturbance. The selected OCR profile in Figure 2.3.3 was therefore primarily based on values back-calculated with SHANSEP from the CPTU-based $s_{uc}$ values, assuming $S=0.28$ and $m=0.85$ for this clay. The procedure used is the same as presented for the Stjordal site in Section 2.2.1.

Figure 2.3.3 – Assumed in-situ stress conditions, Onsøy

Figure 2.3.4 summarize values for the Janbu (1963) modulus number, $m_0$, and the vertical permeability at zero volume change, $k_0$, as determined from the oedometer
tests. The pronounced drop in modulus number below 15 m depth is clearly related to the increase in sensitivity beyond this depth.

Figure 2.3.4 – Modulus number and permeability from oedometer tests, Onsøy

Figure 2.3.5 shows undrained shear strengths from the CAUC and DSS tests carried out on samples from 3 levels, compared to the $s_{uc}$ strength derived from the pore pressure response in the CPTU tests using the Karlsrud et al (2005a) correlations. These CPTU derived $s_{uc}$ strengths are on the high side of the strengths from the triaxial tests. The triaxial tests were, as the oedometer tests, to some extent influenced by sample disturbance as compared to high quality block samples, which the CPTU correlations are based upon. For that reason the selected $s_{ud}$ profile also lies somewhat higher than the DSS test results. (As a comment, due to the impact of stress rotation during DSS tests, the strength in DSS test is normally less influenced by sample disturbance effects than CAUC tests). The triaxial tests showed that the peak effective friction angle of the clay is $\varphi = 33.8^0$. 
Figure 2.3.5 – Assumed undrained shear strength profile, Onsøy

### 2.3.2 Summary of pile test results Onsøy

The 6 test piles at Onsøy had as at the Stjørdal site a diameter of 508 mm and wall thickness of 6.3 mm. They were driven open ended through a 1.4 m deep casing to a tip penetration of 19.1 m (Table 2.1.1).

All the piles were driven with a hydraulic hammer with hammer weight of 50 kN, and using a constant drop height of 10 cm throughout. As seen from Figure 2.3.6 the blow count increased gradually with depth. The data suggest that the driving resistance was somewhat larger at the location of Piles O1, O4 and O5 than for the 3 other piles. This may reflect some variability in the sensitivity of the clay across the site.
Figure 2.3.6 – Summary of blow counts recorded during pile installation, Onsøy (hammer weight 50 kN, and constant drop height of 10 cm throughout)

As was shown in Table 2.1.3 the depth to the top of the soil plug was at the end of driving measured to be from 2.35 to 3.96 m below the bottom of the casing. For Pile O3 the depth to the top of the plug was also measured during driving. As presented in Figure 2.3.7 some plugging occurred at pile penetration around 10 m and 15 m, see also Figure 2.3.8.

The impact of partial pile plugging on the time for re-consolidation or set-up is analysed and discussed in a later Chapter 4.2. At the Onsøy site these analyses suggest that 90% consolidation was reached after 1.5 month and 95% consolidation after 4.4 months.
Figure 2.3.7 – Measured depth to top of soil plug during driving of Pile O3, Onsøy

Figure 2.3.8 – Measured incremental plugging ratio versus pile tip penetration for Pile O3, Onsøy

Figure 2.3.9 and Figure 2.3.10 show examples of load displacement curves for the repeated load testing on Pile O1 at Onsøy, whereas Figure 2.3.11 compares the load-displacement curves only for the first time tests on each pile.
Figure 2.3.9 – Accumulated Load-displacement response from the 5 repeated load tests on pile O1-1, Onsøy site

Figure 2.3.10 – Load-displacement response from the 5 repeated load tests on pile O1-1, Onsøy site
The load-displacement curves in Figure 2.3.9 to Figure 2.3.11 show that the development of failure was rather brittle and well defined. A clear peak was reached at about 14 mm pile top displacement in most of the tests, followed by a tendency for loss in load with accumulated displacements for the last load-step in each test. The failure criteria for these piles have been defined as for the Stjørdal pile tests as the largest of when the peak load was reached, or the rate of creep under sustained loading exceeded 1 mm/min. Figure 2.3.12 show creep data for the 1st time tests on each pile. The creep rates define in this case failure more clearly than was the case at the Stjørdal site.
Table 2.3.1 presents the ultimate failure loads defined from each test as well as the relative increase in failure load using test O1-1 as reference. Figure 2.3.13 presents the failure load versus time for all tests.

Table 2.3.1 and Figure 2.3.13 show that after 24 months there is an increase in capacity of about 1.17 times the initial capacity just due to time alone, which is much less than at the Stjordal site. Re-loading the same pile tends to give some increase in capacity (beyond only time effect) during the 2nd and 3rd tests, the capacity then levels off or reduces in subsequent tests. Figure 2.3.9 and Figure 2.3.10 suggests that the reduction in capacity with re-loading for pile O1 is due to strain-softening effects, which in this case tends to override the impact of repeated loading.

Pile test O6-1 with sustained loading of 420 kN over 2 years show an increase in capacity of 1.34 times test O1-1. Note that the sustained load was reduced to zero before loading started. Probably for that reason, the response of this pile is much stiffer than the other piles. Thus sustained loading also in this case gives a further increase in capacity than what comes from only time. As can be seen in Figure 2.3.11 it also led to a stiffer pile response and smaller displacement at failure than in the other tests.

The load-displacement curve for pile O6 also confirms the impact of strain softening. At 40 mm pile top displacement, corresponding to about 4 times the displacement at peak, the load at the top is reduced to about 69\% of the peak value. Tests O4-1 and O5-1 loaded to respectively 17 and 23 mm pile top displacement, also show clear evidence of strain softening, ending at loads that are 91 and 85\% of the peak load respectively.

Table 2.3.1 – Summary table of failure loads measured at the pile top, $Q_{utop}$, Onsøy

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time (months)</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
<td>12,3</td>
<td>24,3</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
</tr>
<tr>
<td>Defined capacities (kN)</td>
<td>590</td>
<td>770</td>
<td>700</td>
<td>680</td>
<td>640</td>
<td>1</td>
<td>1,31</td>
<td>1,19</td>
</tr>
<tr>
<td>Time (months)</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
<td>12,3</td>
<td>24,3</td>
<td>1,14</td>
<td>1,14</td>
<td>1,19</td>
</tr>
<tr>
<td>Defined capacities (kN)</td>
<td>690</td>
<td>690</td>
<td>700</td>
<td>640</td>
<td>700</td>
<td>1,17</td>
<td>1,17</td>
<td>1,19</td>
</tr>
<tr>
<td>Time (months)</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
<td>12,3</td>
<td>24,3</td>
<td>1,14</td>
<td>1,14</td>
<td>1,19</td>
</tr>
<tr>
<td>Defined capacities (kN)</td>
<td>680</td>
<td>790</td>
<td></td>
<td></td>
<td></td>
<td>1,08</td>
<td>1,19</td>
<td>1,15</td>
</tr>
<tr>
<td>Time (months)</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
<td>12,3</td>
<td>24,3</td>
<td>1,14</td>
<td>1,14</td>
<td>1,19</td>
</tr>
<tr>
<td>Defined capacities (kN)</td>
<td>790</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1,34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time (months)</td>
<td>2,6</td>
<td>5,4</td>
<td>7,9</td>
<td>12,3</td>
<td>24,3</td>
<td>1,14</td>
<td>1,14</td>
<td>1,19</td>
</tr>
<tr>
<td>Defined capacities (kN)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
For the Onsøy site the contribution of pile weight and tip resistance as defined in Eq. (2.2.2) and Eq. (2.2.3) are $W_{pile} = 60.9 \text{ kN}$ and $Q_{utip} = -4.8 \text{ kN}$. As at Stjørdal the tip correction has little impact, and only amounts to 0.6 to 0.8 % of the failure load measured at the pile top.

Figure 2.3.14 shows that the values of normalised ultimate shaft friction $\alpha = \frac{\tau_{us}}{s_{ud}}$ deduced on this basis starts off at a value of $\alpha = 0.80$ for the first test O1-1 and reach about $\alpha = 0.95$ for the first time tests when the pile has been left in the ground for 24 months. Repeated load testing gave a maximum $\alpha$-value of 1.14. Pile O6 subjected to a sustained load of about 420 kN, corresponding to 71 % of the failure load in test O1-1, gave an value of $\alpha=1.16$.

![Figure 2.3.13 – Ultimate pile top capacity, Qutop, versus time – all tests Onsøy](image1)

![Figure 2.3.14 – Mobilised $\alpha$-value versus time- all tests Onsøy](image2)
Figure 2.3.15 shows correspondingly the deduced values for $\beta = \frac{\tau_{us}}{\sigma'\alpha_{0}}$ for all the tests. The first initial value corresponds to $\beta = 0.25$ and the 1st time tests reached $\beta = 0.32$ after 2 years, with repeated loading giving a maximum value around $\beta = 0.37$. Sustained loading over two years gave $\beta = 0.38$.

Figure 2.3.15 – Mobilised $\beta$-value versus time- all tests Onsøy

Figure 2.3.16 presents the change in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested, determined as explained for the Stjørdal site. The trend suggests that the relative capacity increase by factor of about 1.06 in test 2 but further load repetitions tend to reduce this gain.

Figure 2.3.16 – Impact of repeated (staged) loading- Onsøy
2.3.3 **PDA testing**

During clean-up of the test site at Onsøy two piles were subjected to PDA measurements during re-strike. This took place in October 2012, about 14 months after the last pile load tests were carried out. The detailed data and analyses can be found in the Onsøy data report 20061251-00-264-R (see Appendix A), but are also summarized in Table 2.3.2. It appears that the PDA/CAPWAP analyses tend to predict capacities on the low side of what was measured by the static tests. This could to some extent be explained by a possible reduction in capacity due to repeated loading when 4 or 5 load tests have been carried out on the same pile, ref. Figure 2.3.16.

**Table 2.3.2 – Summary of PDA/CAPWAP results, Onsøy**

<table>
<thead>
<tr>
<th>Pile no.</th>
<th>Measured shaft capacity last load test $Q_{us}$ (kN)</th>
<th>Shaft capacity from PDA/CAPWAP $Q_{us,CAP}$ (kN)</th>
<th>Ratio $Q_{us,CAP} / Q_{us}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>O 3</td>
<td>636</td>
<td>503</td>
<td>0.79</td>
</tr>
<tr>
<td>O 4</td>
<td>636</td>
<td>575</td>
<td>0.90</td>
</tr>
</tbody>
</table>

2.4 **Clay site Cowden**

2.4.1 **Summary of soil conditions**

The Cowden glacial till test site has been used by the Building Research Establishment (BRE) UK, since the 1980’s, which has included testing of steel and concrete piles as well as a range of in-situ and laboratory testing, e.g. Powell and Butcher (2003). Imperial College has also carried out testing with their model pile in the same area (e.g. Lehane, 1992; Lehane and Jardine, 1992).

Figure 2.4.1 shows the location of the Cowden test site on the east coast of England.

The soil characteristics described in the following are taken from a comprehensive presentation of soils data given by Powell and Butcher (2003).

Figure 2.4.2 shows that within the pile testing depth, the Cowden till has a rather uniform water content and plasticity index. The plastic limit lies very close to the water content, as is typical of heavily over-consolidated clays.
Figure 2.4.1  –  Location of the Cowden (UK) test site

Figure 2.4.2  –  Water content and plasticity index, Cowden

This glacial till has clay size particle fraction in the range 25-35%. Silt size constitutes about 35-40% and sand 10-15%. The clay also contains some fragments of chalk and flint within its matrix.

Figure 2.4.3 presents in-situ stress conditions derived from the soils data. The measured in-situ pore pressure is hydrostatic below the ground water table at depth of 0.8 m. The selected OCR profile in Figure 2.4.3 was based on values back-calculated with SHANSEP from the CPTU based $s_{uc}$ values (Figure 2.4.4), assuming $S=0.28$ and $m=0.80$ for this clay (see following). It agrees well with the range of OCR-values given by Powell and Butcher (2003) based on oedometer tests and in-situ tests.
Oedometer tests show that the modulus number typically lies in the range of \( m = 18 \) to 25. Based on a combination of laboratory tests and in-situ tests, the in-situ vertical permeability has been reported to be typically \( 1.0 \times 10^{-10} \), but varies from 0.15 to 2.0 \( \times 10^{-10} \) m/s.

Figure 2.4.4 shows the undrained shear strength profile arrived at from the CPTU test. Values of \( N_{kt} \) were based on Karlsrud et al (2005a), and range from \( N_{kt} = 11.1 \) to 13.7. For the highest OCR clay in the top, it was assumed \( s_{ud} = 0.65s_{uc} \), and below 5m depth it was taken as \( s_{ud} = 0.70s_{uc} \).

2.4.2 Summary of pile test results Cowden

The dimensions of the Cowden tubular steel test piles were diameter of 457 mm and wall thickness of 12.5 mm. They were driven open ended through a 6 cm deep pre-augered hole to a tip penetration of 10.3 m. The pile tests were carried out by BRE.
on contract from NGI as part of the current project. Factual data and results were presented in report 20110444-00-7-R (see App. A), and was based on the results provided by BRE.

The piles were driven with a Dawson HPH 2400 hydraulic hammer with a maximum rated energy of 24 kNm. The applied driving energy was increased stepwise, but was 18 kNm during the last 3 m of driving. The blow count towards the end of driving was in the range 240 to 300 blows pr. m.

The piles partially plugged during driving. At end of driving the distance from the ground surface to the top of the plug was from about 1.1 to 2.05 m below the casing, corresponding to a plugging ratio (see later Eq. 4.2.5) in the range $p=0.21$ to 0.31. The impact of partial pile plugging on time for re-consolidation or set-up is analysed and discussed in a later Chapter 4.12. At the Cowden site these analyses suggest that 90 % consolidation was reached after 1.3 months and 95 % consolidation after 3.4 months.

Figure 2.4.5 and Figure 2.4.6 show an example of load displacement curves for the repeated load testing on Pile C1 at Cowden, whereas Figure 2.4.7 compares the load-displacement curves only for the first time tests on each pile.

![Figure 2.4.5](image-url)
Figure 2.4.6 – Load-displacement response from the 5 repeated load tests on pile C1-1, Cowden site

Figure 2.4.7 – Load-displacement response from 1st time load tests on piles C1 to C6, Cowden site
The load-displacement curves in Figure 2.4.5 to Figure 2.4.7 show that the development of failure was fairly neutral, but with some hardening tendency in the 1st tests and more neutral (no pronounced strain hardening or softening) in subsequent tests on each pile.

Table 2.4.1 presents the ultimate failure loads defined from each test as well as the relative increase in failure load using test C1-1 as reference. Figure 2.4.9 present the failure load versus time for all first time tests. The failure criterions for these piles have also been defined as for the Stjørdal site as largest of when the pile top displacement exceeds 30 mm if a clear peak is not reached before that, or the rate of creep under sustained loading exceeds 1 mm/min. Figure 2.4.8 show creep data for the 1st time tests on each pile. As for the other piles subjected to sustained loading the sustained load was reduced prior to loading to failure.

![Rate of creep displacement towards the end of each load increment-first time tests, Cowden](image)

Table 2.4.1 and Figure 2.4.9 show that after 24 months there is a an increase in capacity of 1.20 times the initial capacity just due to time alone, which is a bit higher than at Onsoy. Re-loading the same pile tends to give some, but not very pronounced, increase in capacity for all tests (in addition to only time effect), but the increase is not very different from what comes from the time effect alone.

Pile C6 with sustained loading of 595 kN over 2 years show an increase in capacity of 1.23 times test C1-1, about the same as the time effect alone. Thus at Cowden,
neither repeated loading nor sustained loading seems to have any significant impact on the pile capacity.

Table 2.4.1 – Summary table of failure loads measured at the pile top, $Q_{\text{utop}}$, Cowden

<table>
<thead>
<tr>
<th>Cowden</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (months)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1,6</td>
</tr>
<tr>
<td>Pile No. 1</td>
<td>1020</td>
<td>1070</td>
</tr>
<tr>
<td>Pile No. 2</td>
<td>1050</td>
<td>1200</td>
</tr>
<tr>
<td>Pile No. 3</td>
<td>1110</td>
<td>1250</td>
</tr>
<tr>
<td>Pile No. 4</td>
<td>1300</td>
<td>1360</td>
</tr>
<tr>
<td>Pile No. 5</td>
<td>1210</td>
<td>1210</td>
</tr>
<tr>
<td>Pile No. 6</td>
<td>1225</td>
<td>1225</td>
</tr>
</tbody>
</table>

Figure 2.4.9 – Ultimate pile top capacity, $Q_{\text{utop}}$, versus time – all tests Cowden

For the Cowden site the contribution of pile weight and tip resistance as defined in Eq. (2.2.2) and Eq. (2.2.3) are $W_{\text{pile}} = 39.4$ kN and $Q_{\text{utip}} = -109.4$ kN. The tip correction only amounts to 0.8 to 1.0 % of the failure load measured at the pile top.

Figure 2.4.10 shows that the values of normalised ultimate shaft friction $\alpha = \tau_{\text{ud}}/s_{\text{ud}}$ deduced on this basis starts off at a value of $\alpha = 0.51$ for the first test C1-1 and reach a maximum of about $\alpha = 0.68$ for the first time tests when the pile has been left in the ground for 12 months. Repeated load testing gave a maximum $\alpha$-value approach of 0.66. Pile C6 subjected to a sustained load of about 600 kN, corresponding to 60 % of the failure load in test C1-1, gave an value of $\alpha=0.64$, practically the same as for time effects alone as observed for pile C5.
Figure 2.4.10 – Mobilised $\alpha$-value versus time - all tests Cowden

Figure 2.4.11 shows correspondingly the deduced values for $\beta = \tau_{us}/\sigma_{v0}$ for all the tests. The first initial value corresponds to $\beta = 0.82$ and the 1st time tests reached $\beta = 1.07$ after 2 years, with repeated loading giving a maximum value around $\beta = 1.16$. Sustained loading over two years gave $\beta = 1.04$.

Figure 2.4.12 presents the change in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested, determined as explained for the Stjørdal site. The trend suggests that the relative capacity tend to increase somewhat in test 2, but further load repetitions tend to reduce this gain and produce a net relative loss in capacity after the 3rd load repetition. This loss may be due to strain softening effects.
**Figure 2.4.12 – Impact of repeated (staged) loading- Cowden**

**2.5 Clay site Femern**

**2.5.1 Summary of soil conditions**

The Femern test site is located in the German sector of the sea between Denmark and Germany, Figure 2.5.1. The water depth at the pile test site is about 12 m. The clay at this site is a rather uncommon deposit of very plastic and old Palaeogene clay (dating back about 50-60 million years).

**Figure 2.5.1 – Location of the Femern (Germany) test site-site indicated by red dot**

The pile testing program is part of ongoing plans for a fixed link across the strait, either bridge or submerged tunnel. Femern A/S wanted to undertake a pile testing program at the site similar to the test program carried out at other sites as part of this
study. Furthermore, the Femern project kindly made all soil data and pile test results available to this project. NGI took part in the planning and execution of the tests and developed part of the monitoring system. Per Aarslef A/S was contracted by Femern A/S to act as the general contractor with overall responsibility for execution of the tests.

An extensive site investigation program was carried out in several stages for the planned fixed link. The soil data presented in the following were mainly extracted from the summary report by Femern A/S (2012), which has been made fully open and can be downloaded from [www.femern.com](http://www.femern.com). The focus herein is on soil data from within reasonable proximity of the pile test area.

At the pile test site there is an upper 2-3 m thick layer of sand followed by some more recently deposited or re-worked clay. Below about 5 m there is folded and fissured Palaeogene clay of the so-called Røsnes formation. Figure 2.5.2 shows that the water content below the top part is typically 36%. The average plasticity index increases from about 50% at 5 m to 120% at 20 m depth. Notice however, the upper and lower \( I_p \) ranges. The real \( I_p \) values actually vary within these ranges throughout the deposit, that is, on a very local level. The plastic limit is very close to, but typically 5% lower than the water content, giving a liquidity index that typically decrease from 0.08 at 10 m to 0.05 at 30 m depth. The clay content is high and varies also locally from about 50 to 85%. Silt size particles constitute most of the remaining part.

![Figure 2.5.2 – Water content and plasticity index, Femern (from Karlsrud, 2012)](image)

A large number of CRS and IL type oedometer have been carried out on Femern clay.

Figure 2.5.3 shows the pre-consolidation pressures reported from the soil borings closest to the pile test site. Definition of the pre-consolidation pressure from oedometer tests on stiff clays can be difficult, considering also the possible impact of sample disturbance. In the current case, samples were taken with the Geobore-S core drilling system. Even if great attention is given to the coring procedure, it is hard to eliminate any effect of sample disturbance. The individual oedometer tests have not been reviewed as part of this study, and the pre-consolidation pressures and OCR...
values are assumed as they were reported by Femern AS (2011), knowing that highly qualified and experienced people undertook that work.

![Figure 2.5.3 – Assumed in-situ stress conditions, Femern (from Karlsrud, 2012)](image)

The permeability of the clay is very low, with measured values ranging from about $2 \times 10^{-12}$ to $2 \times 10^{-11}$ m/s. This is about 2 to 3 orders of magnitude lower than what is commonly observed for normally consolidated marine clays.

The soil testing at Femern also included a series of DSS tests. Two test types were run:

- The samples that were first consolidated up to but not above the assumed pre-consolidation pressure, and then un-loaded back to $\sigma'v_0$ prior to undrained shearing. The purpose was to try to re-store the in-situ horizontal effective stress prior to shearing. This is also standard procedure used at NGI on OC clays, e.g. Lunne et al (2006).

- The samples that were first consolidated to well beyond the pre-consolidation pressure and then un-loaded to various stress levels prior to undrained shearing. This is according to the SHANSEP procedure, Ladd and Foott (1974).

Figure 2.5.4 shows the normalised strengths derived from the SHANSEP consolidated samples. They define average values of $S= 0.23$ and $m=0.85$. Figure 2.5.5 shows the DSS strengths obtained from samples consolidated with the $p'_c - \sigma'v_0$ procedure. Herein is also shown the calculated strength using the SHANSEP procedure with the assumed OCR values from Figure 2.5.3. The two approaches led to quite similar $s_{ud}$ values.
A large number of CAUC, CAUE and UU triaxial tests were also made on the Femern clay. Geo (2011) concluded on that basis that statistically there were no difference between these strengths.

A number of CPTU tests have also been carried out at the Femern site. Figure 2.5.6 summarize the measured corrected tip resistance values, \( q_t \). The tests 09.B001 and 09.B002 in the diagram on the left were carried out in an early phase, and were located somewhat up-slope and down-slope from the pile test site. These tests were of the stepwise downhole type. The CPTU tests in the diagram on the right were carried out in closer proximity of the test piles, and were based on direct push-in from the seabed. As seen in the left hand diagram, the average values from the two types...
of CPTU tests are in good agreement. The selected typically assumed $q_t$ profile is also shown herein.

In the Femern A/S (2012) report, the measured DSS strengths were presented in relation to the cone tip resistance. Figure 2.5.7 shows a correlation between $N_{kt}=(q_t-\sigma_v)/s_{ud}$ and OCR as proposed by Karlsrud (2012) on that basis. Figure 2.5.5 shows the $s_{ud}$ profile established using this correlation and the typically assumed $q_t$ profile in Figure 2.5.6. The CPTU deduced $s_{ud}$ profile falls somewhat on the low side of the other values, especially in the top part. The $s_{ud}$ profile proposed and used herein, and shown with black line in Figure 2.5.5, gives most weight to the $s_{ud}$ strengths derived from the laboratory tests.

\begin{figure}
\includegraphics[width=\textwidth]{summary}
\caption{Summary of corrected cone tip resistance values, Femern (from Karlsrud, 2012)}
\end{figure}

\begin{figure}
\includegraphics[width=\textwidth]{correlation}
\caption{Cone factor $N_{kt}$ relating $s_{ud}$ to OCR, Femern (from Karlsrud, 2012)}
\end{figure}
2.5.2 Summary of pile test results Femern

The 5 tubular steel piles used at Femern have a diameter of 508 mm and wall thickness of 22.5 mm. They were driven open-ended from a jack-up barge through a piling and loading frame pre-installed at the sea bottom. The piles were driven to a tip penetration of 25.0 m. During driving of the piles, the soil entering the piles was more or less continuously removed to try to avoid plugging, which could adversely affect (prolong) the pile set-up. For that reason, the pile installation took several days. The loading frame was installed at the sea bottom and anchored with separate piles in the corners.

Pile F5 was instrumented with two pore pressure sensors on the pile shaft at both depths of 14.5 and 19.5 m. The two sensors at the same depth were placed diametrically opposite each other. The sensors were designed to measure directly the difference between the actual pore pressure against the pile shaft and a hydrostatic pore pressure relative to the sea level. Figure 2.5.8 shows the measured excess pore pressures for all 4 sensors versus time after pile installation (30 October 2010). A few comments are given to the results:

- A by-pass valve was kept open during pile driving. The valves were closed some hours after end of installation, but due to an error the valve for sensor no. 3 was kept open for about 3 months before it was closed.
- After closing the valves the excess pressures built up gradually to a peak value 1 to 2 months later. This may be due to delayed response of the sensors, possibly due to less than 100 % degree of saturation of the filters. It has also for some previous instrumented pile tests in high OCR clays been observed a similar trend of first increasing then decreasing pore pressures. As discussed by Karlsrud (2012), it has been difficult to distinguish if or to what extent such a trend may be due to poor saturation of piezometer filters or if it is real.
- To check the response of the sensors and that absolute pressures were correct, all valves were opened 9.6 months after installation when piles F1 to F3 were tested. The excess pressures then correctly dropped to zero, but it took also in this case a little over a month before the pressures built up to their original values.
- There is some difference in pressures at the two sides of the pile. That could be due to the pile being slightly curved, leading to different pressures against on the two sides of the pile.
Figure 2.5.8 – Measured pore pressure with time, Pile F5, Femern

Figure 2.5.9 presents the measured excess pore pressures (above hydrostatic) at end of installation and onset of each load test. Due to the delayed hook-up of the pore pressure sensors, the measured pressures at end of installation were back-extrapolated from the data as indicated in Figure 2.5.8. The average values from the pressures at the two sides were used in Figure 2.5.9. The measured excess pore pressures are also compared to end of installation pore pressures calculated with the procedure recommended by Karlsrud (2012), which come out considerably larger than measured. The difference in excess pore pressures at the two levels is most likely due to a difference in local soil conditions. This is also reflected in the very different responses in pore pressures when pile F5 was finally loaded to failure 22 months after pile installation, Figure 2.5.8.
Using the measured excess pore pressure at end of installation as basis, Figure 2.5.10 shows the average degree of pore pressure dissipation (or consolidation) as function of time. The data suggest that 50 % consolidation was reached after 5 months, and that the degree of consolidation was still only 85 % when the last tests were carried out 22 months after installation.

Figure 2.5.11 and Figure 2.5.12 show examples of load displacement curves for the repeated load testing on Pile F1 at Femern, whereas Figure 2.5.13 compares the load-displacement curves only for the first time tests on each pile.
Figure 2.5.11 – Accumulated Load-displacement response from the 5 repeated load tests on pile F1, Femern site

Figure 2.5.12 – Load-displacement response from the 5 repeated load tests on pile F1, Femern site
The load-displacement curves in Figure 2.5.11 to Figure 2.5.13 show that failure was in general defined by a clear peak value occurring at a pile top displacement of 10-12 mm, followed by a clear tendency for post-peak softening.

Figure 2.5.14 shows the creep data for the first time tests on each pile. The development in the creep rate towards failure as well as the displacement at failure are quite comparable to what was observed at the NC Onsøy site, and much more brittle than at the Stjørdal and Cowden sites. Table 2.5.1 presents the ultimate failure loads defined from each test as well as the relative increase in failure load using test F1-1 as reference.

Figure 2.5.15 presents the failure load versus time for all tests. Failure was for this case defined as the largest of when the peak was reached or exceeded 20 mm, or the creep rate exceeded 1 mm/min.
Figure 2.5.14 – Rate of creep displacement towards the end of each load increment—first time tests, Femern

Table 2.5.1 – Summary table of failure loads measured at the pile top, Qutop, Femern

<table>
<thead>
<tr>
<th>Femern Pile No.</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (months)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.1</td>
<td>3.3</td>
</tr>
<tr>
<td>1</td>
<td>3170</td>
<td>2500</td>
</tr>
<tr>
<td>2</td>
<td>3290</td>
<td>2500</td>
</tr>
<tr>
<td>3</td>
<td>3350</td>
<td>2400</td>
</tr>
<tr>
<td>4</td>
<td>3350</td>
<td>2400</td>
</tr>
<tr>
<td>5</td>
<td>3400</td>
<td>2400</td>
</tr>
</tbody>
</table>

Figure 2.5.15 shows that after 24 months there is an increase in capacity of only 1.06 times the initial capacity just due to time alone. Re-loading the same pile gives a pronounced decrease in capacity for all tests. This reduction due to repeated loading of the same pile to failure can in this case be seen as a clear cumulative strain softening effect (e.g. Figure 2.5.11).
For the Femern site the contribution of pile weight and tip resistance as defined in Eq. (2.2.2) and Eq. (2.2.3) are $W_{pile} = 59.4$ kN and $Q_{utip} = -61.7$ kN. Note that due to plug removal the tip resistance was for this case only calculated for a tip area corresponding to the wall thickness.

Figure 2.5.16 shows that the values of normalised ultimate shaft friction $\alpha = \tau_{us}/s_{ud}$ deduced on this basis starts off at a value of $\alpha = 0.76$ for the first test C1-1 and reach $\alpha = 0.82$ for the 1st time tests when the pile has been left in the ground for 22 months. Repeated load testing gave a reduction to a minimum $\alpha$-value of about 0.52.

Figure 2.5.16 – Mobilised $\alpha$-value versus time- all tests Femern
Figure 2.5.17 shows correspondingly the deduced values for $\beta = \frac{\tau_{us}}{\sigma'_{v0}}$ for all the tests. The first initial value corresponds to $\beta = 0.92$ and the 1st time tests reached $\beta = 0.99$ after 22 months, with repeated loading reducing the value to a minimum around $\beta = 0.60$.

\[ \text{Figure 2.5.17 – Mobilised } \beta \text{-value versus time- all tests Femern} \]

Figure 2.5.18 presents the change in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested, determined as explained for the Stjørdal site. The trend suggests that the relative capacity at the same time after pile installation here decreases systematically for each load test. It is typically reduced by factor 0.73 due to 2nd time loading, and to a minimum of 0.60 after the 5th loading. Figure 2.5.11 to Figure 2.5.13 suggest however, that the decrease in capacity at Femern is primarily due to “strain softening” i.e. a reduction in shaft friction as a result of accumulated displacements more than as a result of repeated loading or number of load repetitions.

\[ \text{Figure 2.5.18 – Impact of repeated (staged) loading- Femern} \]
2.6  Sand site Larvik

2.6.1  Summary of soil conditions

The test site at Larvik is located on the small peninsula “Revet”, close to the Color Line terminal on the southeast side of the city of Larvik, Figure 2.6.1. The test area is located at the mouth of the river Numedalslågen, which is one of the longest rivers in Norway. The test site is located at elevation +2 metres.

![Figure 2.6.1 — Location of Larvik test site](image)

The complete site investigation at Larvik was documented in report 20061251-00-244-R, see Appendix A. The site investigations consisted of 1 standard Norwegian rotary-pressure sounding, 2 total soundings, 5 CPTU-soundings and Ø72 mm piston sampling to a depth of 22 metres at one location. Testing of the samples, in addition to routine type classification tests, included four constant rate of strain (CRS) oedometer tests, four anisotropically consolidated undrained triaxial compression (CAUC) and two undrained direct simple shear (DSS) tests. Mineralogical analyses and radiocarbon dating of samples were also carried out on selected samples.

Below an upper layer of fill material to a depth of about 2 m, the visual classification of the soil samples in Figure 2.6.2 suggest that the deposit mainly consists of fine to medium and partly silty sand with occasional layers of sandy, clayey silt. There are some pockets of organic materials and some shell fragments. The sand is of fluvial origin and was according to the radiocarbon dating deposited at the seafront around 2600 to 1200 years ago.

The data in Figure 2.6.2 shows that the water content lies in the range w =18 to 32 %. Figure 2.6.3 also summarises results from the grain size analyses. The total amount of fines (Silt and clay with grain size less than 0.06 mm) mostly lies in the range of 1 to 20 %. Below a depth of about 14 m there are however, some layers of
sandy, clayey silt with fines content of 53 to 63 %. Hydrometer analyses showed that these sandy, clayey silt specimens had clay content in the range of about 3 to 9 %.

Analyses of the sand minerals reveal quartz, feldspars and plagioclase with perhaps small traces of pyrite.

![Figure 2.6.2 – Water content and soil description, Larvik](image)
Figure 2.6.3 – Water content, silt and clay content from borehole 2, Larvik

Figure 2.6.4 – In-situ stresses, Larvik

Figure 2.6.5 compares measured values of net tip resistance, q_c and friction ratio, R_f, and Figure 2.6.6 values of measured pore pressure and deduced pore pressure parameter B_q.
The 5 CPTU tests carried out at the Larvik site show that the soil conditions may show some variations across the site. The net tip resistance varies from about 2 to 10 MPa. The zones with lowest tip resistance generally correspond with zones showing the largest excess pore pressure compared to the hydrostatic case and largest $B_q$ values. This is believed to reflect the zones with sandy clayey silt, which respond partly undrained during the cone penetration, in contrast to the fully drained response of the zones with low fines content.

![Figure 2.6.5 – Summary of measured cone resistance and friction ratio- all CPTU’s, Larvik](image-url)

Figure 2.6.5 – Summary of measured cone resistance and friction ratio- all CPTU’s, Larvik
Maximum and minimum density tests were determined on three samples from depths of 7.5, 12.5 and 20.5 metres. As shown in Table 2.6.1, the results gave an average minimum dry unit weight of about 13.7 kN/m$^3$ and an average maximum dry unit weight of 17.7 kN/m$^3$. The relative density, $D_r$, for samples at depths 7.5, 12.5 and 20.5 metres were on that basis calculated to be 0.62, 0.35 and 0.81 respectively. According to the definitions in Lambe & Whitman (1969) this indicates that the sands at the Larvik site can be classified as loose to medium dense SAND.
Table 2.6.1 – Maximum and minimum dry unit weights

<table>
<thead>
<tr>
<th>Depth (mbgl)</th>
<th>$\gamma_{d,min}$ [kN/m$^3$]</th>
<th>$\gamma_{d,max}$ [kN/m$^3$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>13.66</td>
<td>17.61</td>
</tr>
<tr>
<td>12.5</td>
<td>13.46</td>
<td>17.68</td>
</tr>
<tr>
<td>20.5</td>
<td>14.00</td>
<td>17.82</td>
</tr>
</tbody>
</table>

The relative densities of the sand have also been assessed from the CPT cone resistance using the correlation proposed by Baldi et al. (1986), and are shown in Figure 2.6.7. However, this correlation is based on relative density tests performed on predominantly clean fine to medium uniform sands that are not “aged”.

Clayey silt layers are present at some depths across the site and give a “disturbed” picture of the relative density in Figure 2.6.7. This interpretation suggests relative densities in the range 20 to 60% for what is assumed to be sand layers with relatively low content of fines. The layers with high silt content are assumed to be those that come out with relative densities well below 20% in Figure 2.6.7.

Relative densities have also been assessed from the CPTU cone resistance, $q_c$, using the relationship given by Baldi et al (1986) for normally consolidated sand assuming $K_0 = 0.45$, and correcting the cone resistances for fines content using the procedure presented by Stark and Olson (1995). For simplicity the fines content was assumed to be 25% and constant throughout. For the silt layers the corrected cone resistance values should be used when interpreting the CPT data. Figure 2.6.8 shows that the relative densities for such clayey silt layers are around 40 to 50%.

Figure 2.6.9 shows CPTU from borehole no. 2 interpreted using the programme “CPTpro”, developed by GEOsoft. This programme defines the relative density only for sands with low content of fines, and gives for these layers a value of $D_r$ in the range of 40 to 60%, agreeing with what was deduced from Figure 2.6.7. Figure 2.6.9 also shows the soil types defined from CPTpro. The interpretation method used is the Robertson (1986) method. This method utilises sleeve friction and excess pore pressure in addition to the cone resistance to define the soil type. These results, although generally agreeing with the impressions obtained from the sampling in Figure 2.6.2 and Figure 2.6.3, probably give a more representative picture of the layering of soil.
Figure 2.6.7 – Dr relationship for clean normally consolidated sand (after Baldi et al, 1986)

Figure 2.6.8 – Dr relationship corrected for fines content of 25%
Figure 2.6.9 – CPT pro interpretation of CPTU from borehole no. 2, Larvik

Combining all the different data from visual sample classification, grain size analyses and CPTU results, it is concluded that the total thickness of soils that can be classified as sandy, clayey SILT vary from about 9-16 % of the total thickness over the upper 20 m. The other 84-91% consist of about equal amounts of fine to medium SAND layers containing little or no fines, and silty fine SAND layers with fines content in the range 5-20 %.

To get a better statistical picture of local variations Table 2.6.2 compares the following from the different CPTU tests:

- The average tip resistance over the length of the test piles
- The percentage of the total length along each pile where the measured Bq-values exceed 0.05.
It appears from Table 2.6.2 that there is a clear correlation between high average \( q_c \) values and low lengths where \( B_q > 0.05 \). This can be interpreted as areas where the content of fines (silt and clay fractions) is smallest, and fine sands are most dominating. This can tentatively also be assumed to be areas where the shaft friction relatively speaking would be the highest, which is in the vicinity of CPT3, and lowest near CPT 5, with the others somewhere in between. There is a clear trend in decreasing CPT resistance with distance from the river. On that basis Table 2.6.3 ranks where the pile capacity relatively speaking would be expected to be high or low.

**Table 2.6.2 – Comparison between CPTU soundings - Larvik**

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Average tip resistance (kPa)</th>
<th>Percentage of length with ( B_q &gt; 0.05 ) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT 1</td>
<td>2981</td>
<td>12.0</td>
</tr>
<tr>
<td>CPT 2</td>
<td>4426</td>
<td>11.8</td>
</tr>
<tr>
<td>CPT 3</td>
<td>5281</td>
<td>8.7</td>
</tr>
<tr>
<td>CPT 4</td>
<td>4106</td>
<td>12.6</td>
</tr>
<tr>
<td>CPT 5</td>
<td>3185</td>
<td>16.0</td>
</tr>
</tbody>
</table>

**Table 2.6.3 – Estimated relative influence of ground conditions on pile capacity - Larvik**

<table>
<thead>
<tr>
<th>Pile no.</th>
<th>Relative influence of local ground conditions on pile capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
</tr>
</tbody>
</table>

* 1 - Lowest  
  7 - Highest  

The triaxial tests show that the peak effective friction angle is typically in the order of \( \phi = 35 - 37^\circ \). The interpreted CPTU from borehole no. 2, using the program CPTpro, show similar friction angles, \( \phi = 35 - 39^\circ \) to those obtained in the triaxial test.

The vertical permeability at zero volume change, \( k_0 \), was determined in one oedometer test on fine silty sand from Larvik, and gave a permeability of 1.9E-05 m/s. This is in the typical range for sand to silty sand. The sandy, clayey silt layers would be expected to have significantly lower permeability, probably in the range 1E-08 to 1E-09 m/s.
2.6.2 Summary of pile test results

The tubular steel test piles used at Larvik had a diameter of 508 mm and wall thickness of 6.3 mm. They were driven open ended through a 1.5 m deep casing to a tip penetration of 21.5 m. Factual data from the installation of the piles and results from the pile testing are presented in report 20061251-00-265-R, see Appendix A.

The piles at Larvik were driven with a 50 kN hydraulic hammer. Records of blow counts were at this site only taken for piles L1 and L7 (due to misunderstanding with field crew). The drop height was generally only 10 cm throughout the driving, but with an increase 20 cm at the end of driving Pile L1.

At the end of driving the depth to the top of the soil plug was measured to be from 2.53 to 3.35 m below bottom of the casing, corresponding to 13 to 17% of the pile penetration length.

Figure 2.6.10 and Figure 2.6.12 show an example of the load displacement curves for the repeated load testing on pile L1 at Larvik, whereas Figure 2.6.13 compares the load-displacement curves for the first time tests on each pile.

As mentioned in Progress Reports 3, and 4, (see Appendix A) the first load test L1-1 was carried out with load step duration (each step) of 3 minutes as for the clay sites. This pile showed no tendency for failure when loading was stopped, most likely prematurely, at a displacement of 15 mm. Considering the content of fines within the deposit it was suspected that the observed response of pile L1-1 could reflect a
tendency for dilation and negative pore pressure response during loading. To ensure fully drained conditions it was therefore decided to increase the duration of the load steps to 15 minutes for all subsequent tests. This change seemed to give the expected effect of a more well-defined failure load.

For this reason it was also decided to install a new test pile at Larvik to check the capacity after 1 month for a load step duration of 15 minutes as for all the other tests. As seen from Figure 2.6.13 this test pile L7-1 gave a capacity which is significantly lower than test L1-1, and also a very well defined failure load as in all the other tests apart from L1-1. It is therefore concluded that pile test L1-1 should be disregarded when considering time effects at the Larvik test site, and test L7-1 should be used instead as reference for the initial 1-month pile capacity.

![Figure 2.6.11 – Accumulated Load-displacement response from the 5 repeated load tests on pile L1, Larvik site](image)

Figure 2.6.11 – Accumulated Load-displacement response from the 5 repeated load tests on pile L1, Larvik site
Figure 2.6.12 – Load-displacement response from the 5 repeated load tests on pile L1, Larvik site

Figure 2.6.13 – Load-displacement response from 1st time load test on piles L1 to L7, Larvik site
To arrive at a consistent definition of failure loads from the individual tests the development of rate of creep displacement under constant load as function of load was considered. Figure 2.6.14 summarise these data for all the 1st time tests on the piles. Note that these creep rates were taken as the average over the last 5 minutes or so of constant load. As for the clay sites the creep rate shows an accelerating tendency towards failure. The failure criterions for these piles have for this reason been defined as the largest of when:

- The pile top displacement exceeds 30 mm if a clear peak is not reached before that
- The rate of creep under sustained loading exceeded 0.2 mm/min.

The assumed failure loads were interpolated from the creep curves.

Table 2.6.4 summarises the failure loads defined from each test as well as the relative increase in failure load using test L7-1 as reference capacity. Figure 2.6.15 shows the relative increase in capacity with time. The results suggest a significant increase in capacity for 1st time testing over the first 7 months, but it seems to level off at about 80 % increase after that. Based on Table 2.6.3 the arrows on the figure indicate where the capacity relatively speaking could be higher (arrow upwards) or lower (arrow downwards). The absolute impact is very difficult to estimate, but the net effect would be to reduce the time effects somewhat when pile tests L7-1 is used as reference, but keep the increase going for longer time than 7 months.

Figure 2.6.14 – Rate of creep displacement towards the end of each load increment-first time tests, Larvik
Pile L6 was subjected to a sustained load of 420 kN for 2 years, and the pile shows an increase in capacity of 1.52 times relative to test L7-1. As these piles are close to each other, this relative impact is probably quite representative.

Most of the piles show a significant decrease in capacity when repetitively loaded, though pile test L1-5 and L2-4 show some increase in capacity again from the 12 months to the 24 months tests.

Table 2.6.4 – Summary table of failure loads measured at pile top, $Q_{us\text{ top}}$, Larvik

<table>
<thead>
<tr>
<th>Larvik</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 7-1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Time (months)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1,4 1,0 4,4 7,1 12,0 24,0</td>
<td>1,0 4,4 7,1 12,0 24,0</td>
</tr>
<tr>
<td>1</td>
<td>980 1080 910 850 940</td>
<td>1,80 1,52 1,42 1,57</td>
</tr>
<tr>
<td>2</td>
<td>990 790 740 800 960</td>
<td>1,65 1,32 1,23 1,33</td>
</tr>
<tr>
<td>3</td>
<td>1160</td>
<td>1,93 1,69 1,60</td>
</tr>
<tr>
<td>4</td>
<td>1065</td>
<td>1,78 1,68 1,60</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>1,80</td>
</tr>
<tr>
<td>6</td>
<td>900</td>
<td>1,50</td>
</tr>
<tr>
<td>7</td>
<td>600</td>
<td>1</td>
</tr>
</tbody>
</table>

Figure 2.6.15 – Ultimate capacity versus time – all tests Larvik

When calculating the load carried by shaft friction the total loads in Table 2.6.4 were corrected for the total weight of the pile and soil plug (78 kN) and the pore pressure acting across the tip of the pile, corresponding to an upward force of 190 kN.

Figure 2.6.16 shows the deduced values for $\beta = \tau_{us}/\sigma'_{v0}$ for all the tests. The first initial value corresponds to $\beta = 0.17$ and the maximum value reached is around $\beta = 0.34$. 
Figure 2.6.16 – Mobilised $\beta$ – value versus time – all tests Larvik

Figure 2.6.17 presents the change in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested, determined as explained for the Stjørdal site. The trend suggests (apart from pile L1) that the relative capacity decreases by a factor 0.66-0.86 as a minimum for 2$^{nd}$ and 3$^{rd}$ load test. The reduction factors tend to relatively speaking increase again for the 4$^{th}$ and 5$^{th}$ test, but the factor still remains well below 1.0. Thus, for the Larvik site, it is quite clear that repeated loading to failure reduces the ageing effect, but the net effect of time is still positive (e.g. Table 2.6.4 and Figure 2.6.15).

Figure 2.6.17 – Impact of repeated (staged) loading- Larvik
2.6.3 **PDA testing**

PDA testing was carried out during pile driving in 2009 by Multiconsult and by NGI in 2011, about 6 months after the last pile tests had been carried out. The detailed data and results are included in the Larvik report 20061251-00-265-R (see App. A).

In 2009 pile L1, L2, L3 were continuously monitored during pile driving, whilst pile L6 was tested during re-striking 1 day after pile installation.

In 2011 the new pile L7 was continuously monitored during pile driving, whilst piles L1, L4 and L6 were tested by re-striking the piles. The data quality for pile L6 was however found to be too poor to be included herein.

Table 2.6.5 compares the total shaft friction deduced from the PDA and CAPWAP analyses to the ultimate shaft capacities deduced from the load tests. The numbers in parenthesis give the time that had passed since pile installation for the respective tests. The ultimate capacities from static load tests are taken from the tests closest to the date when the PDA testing was carried out.

The PDA/CAPWAP data from the pile installation phase show very large scatter in deduced ultimate shaft capacity, from 134 to 1074 kN. For comparison the load tests on piles L1-1 (1.4 month) and L7-1 (1.0 month) gave ultimate shaft capacities of respectively 935 and 555 kN, somewhere in between the range of PDA/CAPWAP results.

Several re-strike tests were carried out on piles L1 and L4 in late November 2011, about 5 months after the last pile tests were carried out. The interpreted values for the ultimate shaft friction presented in Table 2.6.5 represent the range of values obtained for the 3 to 5 re-strikes that were made on pile L1 and L4. These PDA/CAOPWAP values are on the high side of the last static load test result on each pile. Some possible explanations may be that PDA/CAPWAP results reflect shaft friction in compression type loading as compared to tension in the static tests, and/or more tendencies for dilation and rate effects in silt layers during pile during than in static testing.

Compared to the PDA/CAPWAP during pile installation the last PDA/CAPWAP measurements still confirm a significant gain in ultimate shaft friction with time as observed by the static load tests.
Table 2.6.5 – Ultimate shaft capacity from static load tests and PDA/CAPWAP

<table>
<thead>
<tr>
<th>Pile</th>
<th>Load test Qus [kN]</th>
<th>PDA/ CAPWAP at pile installation QusCAP [kN]</th>
<th>PDA/ CAPWAP after pile testing completed QusCAP [kN]</th>
<th>Ratio QusCAP / Qus</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>864 (24)</td>
<td>613</td>
<td>1290-1602 (30 mo)</td>
<td>1.49-1.85</td>
</tr>
<tr>
<td>L2</td>
<td>914 (4.4)</td>
<td>1074</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L3</td>
<td>1039 (7.1)</td>
<td>421</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>L4</td>
<td>934 (24)</td>
<td>-</td>
<td>964-1399 (30 mo)</td>
<td>1.03-1.50</td>
</tr>
<tr>
<td>L6</td>
<td>834 (24)</td>
<td>461*</td>
<td>Poor data quality</td>
<td>-</td>
</tr>
<tr>
<td>L7</td>
<td>524 (1)</td>
<td>134</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

*Restrike 1 day after pile installation

2.7 Sand site Ryggkollen

2.7.1 Summary of soil conditions

The test site at Ryggkollen is located in a sand quarry between a railway line and County Road 28 (Fylkesvei 28), see Figure 2.7.1. The site is located approximately 65 metres south of the river Drammenselva.

![Figure 2.7.1– Location of Ryggkollen test site](image)

The sand deposit is part of an end moraine formed during the glacial retreat in Norway about 9 000 years ago. The test site is located at an elevation approximately + 10 metres above sea level. The sand is overconsolidated, due to removal of about 15 m of overburden. This gives an overconsolidation ratio decreasing from about 3
to 1.75 from the top to the bottom of the test piles. It is possible that the sand deposit is more overconsolidated due to the action of the glacier during its retreat.

The complete site investigation at Ryggkollen was documented in report 20061251-00-262-R, see Appendix A. The site investigations consisted of five standard Norwegian rotary-pressure soundings, 23 total soundings, one down-hole CPTU-sounding and sampling using shovel sampler, auger, ram piston sampler and sampling of drill cuttings from Odex drilling (percussion drilling).

In 2012 NGI carried out 12 special total soundings for easier comparison between the soundings, to identify possible local variations in the soil profile across the site. A rate of penetration of 0.5 m/min with a speed of rotation of 70 RPM was used. In addition, the soundings were carried out with minimum flushing, and percussion was only used when absolutely necessary (i.e. too much friction on the drill string, or when encountering harder gravel layers).

These latter adjusted total soundings suggest SAND of varying density with variable layers or inclusions of gravel or stones. Judging from the quarried slopes, stones up to a diameter of about 20 cm can be encountered locally within the sand deposit. The drilling resistance and inferred “density” vary both in vertical and horizontal direction. A general trend seems to be that it is “denser” along the north side of the test pile area than on the south side.

Table 2.7.1 shows a relative classification of the inferred average “density” at each test pile location based on the drilling data from NGI borehole nos. 23 to 34. The qualitatively interpreted apparent “density” at each test pile location has been given a score from one to six, where one is loosest and six is densest.

Due to lack of successful sampling of undisturbed samples at Ryggkollen only disturbed bag samples and ram piston samples were examined in NGI’s laboratory. In total 40 bag samples and two ram piston samples were opened and described. As summarised in Figure 2.7.2 the soil can mainly be classified as gravelly medium to coarse sand with occasional inclusions of gravel and stones/ cobbles. There also seems to be some layers or inclusions of finer silty sand layers within the deposit.

Figure 2.7.2 shows the estimated in-situ stress conditions. It was in this connection assumed that the sand above the ground water table has a total unit weight of 19.5 kN/m³, and 20.0 kN/m³ below.
Table 2.7.1 – Classification of the Relative Average Drilling Resistance and relative Pile Driving Resistance, Ryggkollen

<table>
<thead>
<tr>
<th>Pile no.</th>
<th>Relative Average Drilling Resistance *</th>
<th>Relative Pile Driving Resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>6</td>
<td>2</td>
<td>4</td>
</tr>
</tbody>
</table>

* 1 - Loosest  
  6 - Densest

Figure 2.7.2 – In-situ stresses, Ryggkollen

Figure 2.7.4 shows the CPT results from borehole 13 with CPT-pro interpretation of soil layering. The CPT-pro soil classification agrees well with the visual description of collected samples. The tip measured resistance is on average around 30 MPa from depth 5 to 9 m and 20 MPa from depth 12 to 17 m.

Figure 2.7.5 shows that the relative density interpreted from the CPT using the Baldi (1986) lies in the range 80-100 % along the top part (5 to 9m) and and 50-80 % in the lower part (12-17 m). According to Lambe & Whitman (1979) these relative densities classifies the sand as very dense in the top part and medium to dense in the lower part.
Figure 2.7.3 – Soil description from borehole 13, 14 and 1, Ryggkollen

Figure 2.7.4 – CPT pro interpretation of CPTU from borehole no. 13, Ryggkollen
2.7.2 Summary of pile test results

The Ryggkollen tubular steel test piles had diameter of 406.4 mm and wall thickness of 12.5 mm. They were driven open ended through a 5 m deep casing to a tip penetration of 20 mbgl. Factual data and results were presented in report 20061251-00-266-R, appendix A.

At the end of driving the depth to the top of the soil plug was measured to be mostly in the range from 2.33 to 5.52 m with the exception of pile R5 which plugged 10.27 below bottom of the casing. The early plugging of pile R5 may be due to encounters of stones/cobles within the deposit.

The Ryggkollen piles were driven with a hydraulic hammer with weight of 50 kN. The drop height increased with depth, and varied between the different piles from 30 to 50 cm during the last 10 m of driving. Figure 2.7.6 presents the blow count and driving energy used to install the different piles. Based on the PDA measurements on piles 4 and 5 the hammer efficiency was close to 1.0. From the driving records it seems quite clear that Pile 5 showed the largest driving resistance and piles 1 and 2 the smallest, with piles 3, 4 and 6 in between. The relative driving resistance is compared to the drilling resistance characterised in Table 2.7.1. It appears that these two classifications broadly agree. The high resistance during driving of Pile 5 also ties nicely in with the early tendency for plugging of this pile, and the pronounced

Figure 2.7.5 – $D_r$ relationship for overconsolidated sand (after Baldi et al, 1986)
jump in blow count when the tip of this pile reached 8 m depth. As suggested above, this is most likely because it hit a stone at this depth.

![Blow count and driving energy during used to install the piles at Ryggkollen](image)

Figure 2.7.6 – Blow count and driving energy during used to install the piles at Ryggkollen

Figure 2.7.7 and Figure 2.7.8 show an example of the load displacement curves for the repeated load testing on pile R1 at Ryggkollen, whereas Figure 2.7.9 compares the load-displacement curves for the first time tests on each pile. Most piles gave a clear defined failure associated with rapid accumulating displacements at the last load step. Some comments to the individual results and how the ultimate capacity was defined should be given for some of the tests:

- In test R1-1 failure actually occurred as the load was increased from 775 to 1040 kN in the last load increment. This was evidenced by very rapidly accelerating displacements in parallel with the load application (it was actually difficult to adjust the load rapidly enough). Towards the end of the second last load step the rate of creep displacement was about 0.03 mm/min. When the load was increased to the maximum value of 1040 kN, the rate of creep suddenly increased from about 0.23 mm/min during the first 3 minutes to 15 mm/min thereafter. Even after the load was reduced to 850 kN the rate of displacement was very high, about 1 mm/min. The creep stopped completely when the load was reduced further to 725 kN. On this basis it seems clear that failure of this pile must be defined somewhere in between 775 kN and 1040 kN, but closer to the lower value. For this pile the failure load was therefore defined at 850 kN as indicated in Figure 2.7.5.
During the last load step in test R5-1, when the load was at 2134 kN, the rate of creep first seemed to stabilise at 0.096 mm/hr, but 13 min after the load had been applied the rate of displacement increased dramatically. The pile actually failed with such a speed that it was not possible to unload the pile in a controlled manner. Hence test R5-1 does not have an unloading curve.

Figure 2.7.7 – Accumulated Load-displacement response from the 5 repeated load tests on pile R1, Ryggkollen site
Figure 2.7.8 – Load-displacement response from the 5 repeated load tests on pile R1, Ryggkollen site

Figure 2.7.9 – Load – displacement response from 1st time load test on piles R1 to R6, Ryggkollen site

Figure 2.7.10 shows the development of rate of creep displacement under sustained load for the first tests on each pile at Ryggkollen. For these piles it seems like failure
can be most consistently be defined as when the creep rate exceeds about 0.20
mm/min towards the end of each load step, as was also chosen for the Larvik site.
This forms the basis for the failure loads defined for each test.

Figure 2.7.10 – Rate of creep displacement towards the end of each load increment-
first time tests, Ryggkollen

Table 2.7.2 summarises the failure loads defined from each test as well as the relative
increase in failure load using R1-1 as reference. The normalised failure loads are also
shown graphically in Figure 2.7.11. For the 1st time tests there is a significant and
clear tendency for increase in capacity with time at this site, with up to 2.47 times of
the initial capacity reached after 29.5 months. The increase shows some tendency for
levelling off in the log-time plot but not as strong as at the Larvik sand site.

Pile R6 shows an increase in capacity of only 1.33 times relative to test R1-1 after
being subjected to a sustained load of 600 kN for 2 years, which is less increase than
observed at the Larvik site, and far less increase than for the piles without sustained
loading.

In Figure 2.7.11 the vertical black arrows indicate where the ground conditions tend
to be on the loose and dense side, as was defined in Table 2.7.1, and how that
relatively speaking would impact the measured pile capacities. This suggests that the
real time effect may be somewhat less than measured by these pile tests.
All piles show significant decrease in capacity when repetitively loaded, and reach a relative capacity of 0.59 at the lowest for pile R2. Pile test R1 shows however, a slight increase in capacity again from the 12 month to the 25 month test.

Table 2.7.2 – Summary table of the failure loads measured at the pile top, $Q_{us \text{ top}}$, Ryggkollen

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Time (months)</th>
<th>Defined capacities (kN)</th>
<th>Change factor relative to test 1-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2,8 950</td>
<td>730 660 675 710</td>
<td>0,86 0,78 0,79 0,84</td>
</tr>
<tr>
<td>2</td>
<td>5,3 950</td>
<td>515 500 520</td>
<td>1,12 0,61 0,59 0,61</td>
</tr>
<tr>
<td>3</td>
<td>8,1 1600</td>
<td>1135 1030</td>
<td>1,88 1,34 1,21</td>
</tr>
<tr>
<td>4</td>
<td>13,0 1910</td>
<td>1550</td>
<td>2,25 1,82</td>
</tr>
<tr>
<td>5</td>
<td>24,8 -</td>
<td>2100</td>
<td>- 2,47</td>
</tr>
<tr>
<td>6</td>
<td>29,5 -</td>
<td>1140</td>
<td>- 1,34</td>
</tr>
</tbody>
</table>

Figure 2.7.11 – Increase in ultimate capacity with time – all tests Ryggkollen

Figure 2.7.12 shows the deduced values for $\beta = \tau_{us} / \sigma'_{v0}$ for all the tests. The first initial value corresponds to $\beta = 0.44$ and the maximum value reached is around $\beta = 1.15$ for the first test on pile R5.
Figure 2.7.12 – Mobilised $\beta$ – value versus time – all tests Ryggkollen

Figure 2.7.13 presents the change in capacity due to repeated (staged) loading only in relation to the number of times the individual piles were tested, determined as explained for the Stjørdal site. The trend suggests (apart from pile L1) that the relative capacity decrease to a factor 0.25-0.48 as a minimum for the 3rd load test. The reduction factor tend to relatively speaking stabilize at that level for the 4th and 5th test. Thus, for the Ryggkollen site, staged loading has the largest negative relative impact, and even cause a net reduction more than outweighing the positive time effects for pile R1 and R2 (e.g. Table 2.7.2 and Figure 2.7.11).

Figure 2.7.13 – Impact of repeated (staged) loading- Ryggkollen
2.7.3 PDA testing

PDA testing was performed on piles R4 and R5 during driving, and new re-strike tests were performed on piles R1, R2, R3, R4 and R6 in October 2012, a little more than a year after the last load test (7 months for pile R6).

As presented in Appendix D to the factual data report 20061251-00-266-R, the CAPWAP analyses of the Ryggkollen pile driving data gave a total shaft friction of only 167 and 219 kN for piles R4 and R5, or about 10% of the total driving resistance. This shaft capacity during driving is only about 25% of the shaft capacity as measured in the 1st static load test on Pile R1-1.

Appendix E to the factual data report 20061251-00-266-R presents results from the re-strike tests. Unfortunately the data quality for Piles R1, R2 and R3 was poor or the data were difficult to interpret. Pile R5 was not tested due to damaged pile top.

Table 2.7.3 presents interpreted PDA/CAPWAP shaft capacities to what was measured by the last load test. Pile R6 was subjected to several re-strikes. The re-strike value is given for the first blow, but typical average for the subsequent 5 blows is also given in parenthesis. As can be seen from Table 2.7.3 the PDA/CAPWAP capacities are smaller than measured in the last load test on the piles.

Table 2.7.3 – Ultimate shaft capacities from static load tests and re-strike tests, Ryggkollen

<table>
<thead>
<tr>
<th>Pile</th>
<th>Capacity from last load test, Qus (kN)</th>
<th>Capacity from re-strike test, QusCAP (kN)</th>
<th>Ratio QusCAP/ Qus</th>
</tr>
</thead>
<tbody>
<tr>
<td>R4</td>
<td>1500</td>
<td>801</td>
<td>0.53</td>
</tr>
<tr>
<td>R6</td>
<td>1089</td>
<td>1081 (790)</td>
<td>0.99 (73)</td>
</tr>
</tbody>
</table>

3 Other relevant pile test data found in the literature

3.1 Sources and approach

A rather comprehensive literature review was included in a report prepared for Petronas Carigali SDN Bhd by NGI (2011) with assistance from Prof. Anders Hust Augustesen of Aarhus University, Denmark. Petronas kindly agreed to make that report available to this project, and the report has been placed on the project website. Data from this report that are found of most relevance to this project are included in the overall assessment of results in Chapters 4 and 5.

A search has also been made for more recent data than what was included in the NGI (2011) report. Such data are presented in the subsequent sections 3.2 and 3.3.

A number of recent papers have presented time effects deduced from re-strike tests on piles using PDA and CAPWAP analyses, or from Statnamic testing (e.g. Powell
and Brown, 2006). Although such tests may give some qualitative indications, the interpretation of bearing capacity from such tests are considered too uncertain (e.g. Rausche et al, 2004) and are therefore not included in this study.

### 3.2 Piles in clay

For clay sites the only new recent data found are those reported by Karlsrud & Mahan (2010). Briefly described, this case involved testing of two steel pipe piles in tension. The site was located in the Oromieh salt lake in Iran. The very deep clay deposit in the lake is essentially normally consolidated with water content mostly in the range 30-40 % and plasticity index of 20-25 %.

Test pile A was an open-ended pipe pile with diameter of 305 mm driven to a tip penetration of 66.1 m below lakebed. It was driven in 2003 and load tested 5 months after end of driving. The open-ended test pile B had a diameter of 356 mm and driven to 30 m penetration. Pile B had been installed in 1988, but was first load tested in 2003, 15 years later.

Although the piles were of different length and slightly different diameter, Karlsrud & Mahan (2010), made a direct comparison between average ultimate shaft friction values normalised with respect to the estimated in-situ direct simple shear strength, that is values of $\alpha = \tau_{us}/\tau_{ud}$, as well as $\beta = \tau_{us}/\sigma'_{v0}$, Table 3.2.1. For a rather homogeneous and in geologic terms normally consolidated clay deposit, such a comparison was considered relevant for revealing time effects.

<table>
<thead>
<tr>
<th>Pile test</th>
<th>Defined ultimate capacity $Q_{us}$ (kN)</th>
<th>Average ultimate skin friction $\tau_{us}$ (kPa)</th>
<th>Average vertical effective stress $\sigma'_{v0}$ (kPa)</th>
<th>Average ultimate normalized skin friction $t_{us}/\sigma'_{v0}$</th>
<th>Apparent $\alpha = \tau_{us}/\tau_{ud}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Old pile</td>
<td>1200</td>
<td>48.7</td>
<td>106</td>
<td>0.46</td>
<td>1.70</td>
</tr>
<tr>
<td>New pile</td>
<td>3000</td>
<td>47.2</td>
<td>236</td>
<td>0.20</td>
<td>0.74</td>
</tr>
</tbody>
</table>

To make a fair comparison between the two tests Karlsrud & Mahan (2010) also made some corrections for possible effects of pile length or strain softening and that pile B had been subjected to some moderate loads for a short period after it was installed. On that basis they concluded that the increase in capacity from 5 months to 15 years was by a factor of 1.67 to 1.99 (1.83 on average), which is quite substantial.
3.3 **Piles in sand**

For piles in sand, a very interesting case record was published by Jardine et al (2006). Although it is briefly described in the report by NGI (2011), it is the only earlier case where 1st time static tests at different time intervals have been available, and also where repeated (staged) load tests had been carried out earlier at the same site.

The sand deposit at the Dunkirk site consists mostly of dense medium to fine sand with CPT tip resistance mostly in the range 10 to 25 MPa, and with ground water level at 4 m depth.

The first Dunkirk tests involved repeated (staged) loading tests of the same piles, the results of which were assessed by for instance Jardine & Chow (1996). The pipe piles tested had diameter of 324 mm and 2 each of length of 11.1 and 22.0 m respectively. The two short piles were subjected to repeated loading in both tension and compression. There were also some re-striking and plug removal in between, which complicates the picture. Nevertheless there were clear indications of a significant gain in shaft friction with time, as indicated by the lower trend line in Figure 3.3.1 (after Jardine et al, 2006).

The 1st time static tests involved testing of piles that had been used as reaction piles for earlier pile testing two piles with diameter of 457 mm driven to 10 m penetration (C1 and JP1). The 6 reaction piles R1 to R6 were open-ended pipe piles of diameter 457 mm driven to depth ranging from 18.9 to 19.4 m below ground. Three of the reaction piles were successfully loaded to failure in tension from 9 to 238 days after installation.

Table 3.3.1 summarizes the measured capacities. A slight adjustment to a common pile length has been made to make the results comparable (choosing R6 as basis). The results reveal that the capacity increased by factor of 2.05 from the 9 day to the 235 day test. As will be discussed further in Chapter 5, this is quite comparable to the increase observed for the Larvik and Ryggkollen tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Tip pen. (m)</th>
<th>Time (days)</th>
<th>Time (months)</th>
<th>Measured capacity (kN)</th>
<th>Adjusted to R6 length (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>19,31</td>
<td>9</td>
<td>0,30</td>
<td>1450</td>
<td>1521</td>
</tr>
<tr>
<td>R6</td>
<td>18,9</td>
<td>80</td>
<td>2,62</td>
<td>2400</td>
<td>2400</td>
</tr>
<tr>
<td>R2</td>
<td>18,85</td>
<td>235</td>
<td>7,70</td>
<td>3210</td>
<td>3120</td>
</tr>
</tbody>
</table>
Gavin et al (2013) presented results from axial load tests carried out at a sand site near Blessington in Ireland. The test piles used were open pipe piles, 7 m long with diameter 340 mm. The piles were driven into a glacially deposited very dense sand with CPT tip resistance increasing from about 10 MPa near ground level to about 20 MPa at 7 m depth. The ground water table was located well beyond the tip of the piles. Four piles where subjected to first time loading to failure in tension at times of 2, 12, 30 and 220 days after pile driving. Figure 3.3.2 presents load-displacement curves from these tests, and Table 3.3.2 the measured total and shaft capacity (total capacity reduced by the weight of pile and soil plug).
Table 3.3.2 – Summary of measured capacities Blessington piles

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Time (days)</th>
<th>Time (months)</th>
<th>Total cap. (kN)</th>
<th>Shaft cap. (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S2</td>
<td>2</td>
<td>0.07</td>
<td>344</td>
<td>331</td>
</tr>
<tr>
<td>S3</td>
<td>12</td>
<td>0.39</td>
<td>665</td>
<td>652</td>
</tr>
<tr>
<td>S4</td>
<td>30</td>
<td>0.98</td>
<td>385</td>
<td>372</td>
</tr>
<tr>
<td>S5</td>
<td>220</td>
<td>7.21</td>
<td>990</td>
<td>977</td>
</tr>
</tbody>
</table>

As illustrated by Figure 3.3.3 the tests at Blessington confirm a significant gain in capacity with time, but there are some anomalies that requires comments:

- Testing of Pile S3 was first initiated after 10 days, but due to problems with the loading frame the pile could not be loaded to failure that day, and was first re-loaded to complete failure 2 days later. This unloading-reloading has probably had little impact on the failure load.
- Pile S4 tested 30 days after installation shows much lower capacity than pile S3. Gavin et al (2013) contribute that to local variations in soil conditions, as evidenced by about 25% lower blow count, a CPT cone resistance that below a depth of 3.5 m is consistently 15-20% lower than measured near the other piles, and finally, that this pile showed less tendency for plugging than the other piles during the last meter or so of penetration.

The Blessington results therefore confirm the observations at Larvik and Ryggkollen that in sand deposits there may be significant variability in soil characteristics and pile capacity over relatively small distances.
4 Overall comparison and assessments for piles in clay

4.1 Factors impacting effects of time on the ultimate shaft friction in clay

The main factors contributing to impact of time on the ultimate shaft friction in clays may be contributed to the following factors:

1. The classical effects of re-consolidation, i.e. dissipation of excess pore pressures generated by the pile installation and increase in effective stress in the clay surrounding the pile. In this effect is also included the impact of the pile installation and re-consolidation on the undrained stress-strain and strength characteristics of the clay and how they may vary with distance from the pile surface.

2. Effects of thixotropy, related to a gain in remoulded strength of the clay under constant volume and no impact of externally generated geochemical effects. This is normally associated with enhanced bonding forces between clay platelets.

3. Effects of long-term ageing, which may be associated with further long term enhanced bonding between clay platelets or other minerals naturally present within the clay that occur during or after the re-consolidation phase. It could however, also be due to stress changes (relaxation) due to creep in the clay surrounding the pile, and thus changes in effective stresses.

4. Effects of geochemical reactions with the pile, which may be caused by cation exchange between the pile material and the surrounding clay, enhancing cementation or bonding forces.

5. Effects of failure surface moving away from the pile surface. The effects above will in most cases lead to a strength that varies with distance from the pile surface. The critical failure surface will therefore with time often tend to move out from the pile surface, increasing the effective pile radius.

Some further explanation and discussion of these 5 different factors is given in the following.

1) Effects of re-consolidation

Installation of piles in clay generates large changes in total stresses and pore pressures in the surrounding clay. The excess pore pressures will reduce as a result of the subsequent reconsolidation process. The total stresses generated also tend to decrease during this reconsolidation process, but the effective stress will still increase (e.g. Karlsrud, 2012).

Figure 4.1.1 presents measured normalised radial effective stresses, $K_c = \sigma'_r / \sigma'_v$, against a number of different instrumented piles at the end of the re-consolidation phase. The data suggest $K_c$ to be generally on the low side of the in-situ horizontal effective stress ratio, $K_0$, in low OCR clays, and mostly so for low-plastic clays. In high OCR clays $K_c$ seems to be mostly greater than $K_0$ but the data show considerable scatter.
In soft to moderately overconsolidated clays, the shaft friction just after the end of pile driving will correspond to the fully remoulded strength of the clay. The shaft friction will then increase as the excess pore pressures reduce and the effective stresses increase, as illustrated by the procedure proposed by Karlsrud (2012) and shown in Figure 4.1.2. The recommended curve in Figure 4.1.2 was mainly developed on basis of small scale in-situ pile probe tests carried out at various stages of reconsolidation, and mainly test results presented by Bogard & Matlock (1990), Bogard et al (2000) and Bogard (2001).
Figure 4.1.2 – Procedure for determination of build-up of shaft friction during the re-consolidation phase, after Karlsrud (2012)

2) Effects of thixotropy

Thixotropy can also impact how the undrained strength of a clay, and also the ultimate shaft friction, will increase with time even under constant volume. This was first demonstrated by Skempton and Northey (1952) and also addressed by Mitchell (1960). They suggested that this time effect on the remoulded strength is related to re-establishment of binding forces between the clay platelets. Andersen and Jostad (2002) summarised remoulded strengths determined from undrained constant volume direct simple shear tests (DSS) on a number of different soft nearly NC clays. The samples were tested at various times after they had been fully remoulded, but the volume of each test specimen was maintained constant during this time. Figure 4.1.3 presents a summary of these results. The data show that the remoulded strength after 60-100 days increased by a factor of about 1.4 to 2.5, which is quite significant. It is believed that the recommended “set-up” curve in Figure 4.1.2 indirectly accounts for such thixotropy effects, as it is based on field tests at various time intervals, but it will not fully capture the impact of plasticity index on the thixotropy effect.
3) **Effects of long-term ageing**

To address ageing effects that come in addition to the normal reconsolidation or set-up effect, it is important to first consider the time required for the reconsolidation process, as dealt in the following Section 4.2.

The increase in pile capacity after the end of the re-consolidation phase may be due to enhanced chemical bonding between soil particles clay platelets (i.e. a geochemical effect) and/or further increase in total and effective stresses due to creep effects. The latter explanation may be most relevant for low plastic and low OCR clays, where the radial effective stresses have been found to be considerably lower than the in-situ effective stresses (e.g. Figure 4.1.1). Based on the original proposal by Skov and Denver (1988) relating the capacity change to an ageing factor, $\Delta_{10}$, to a logarithmic time ratio $\log(t/t_o)$ as expressed by eq (4.1.1), NGI (2000) proposed
that this time ageing factor, $\Delta_{10}$, could tentatively be taken as given in Eq. (4.1.2) and assuming $t_0 = 100$ days. This procedure was also presented with the NGI-05 method for calculating ultimate shaft friction in clays proposed by Karlsrud et al (2005b).

$$Q(t) = Q(t_0) \cdot \left[1 + \Delta_{10} \cdot \log_{10} \left(\frac{t}{t_0}\right)\right], \quad \text{where}$$

$$T = \text{time in days after pile installation in days}$$

$$t_0 = \text{a reference time}$$

$$Q(t_0) = \text{capacity of the pile after a reference time } t_0$$

$$Q(t) = \text{capacity at a later time}$$

$$\Delta_{10} = 0.1 + 0.4 \cdot \left(1 - \frac{I_p}{50}\right) \cdot \text{OCR}^{-0.8}$$

Figure 4.1.4 shows examples of how the pile capacity will increase relative to end of re-consolidation (assumed 3 months) for piles installed in a clay with OCR=1.3 and 5.0 using Eq. (4.1.1) and (4.1.2). It illustrates that an important implication of this proposed ageing factor is that the capacity will increase much more with time in low-plastic clays than in clays with high plasticity, and reduce with OCR.

![Graphs showing pile capacity gain over time](image)

**Figure 4.1.4 – Example of calculated increase in pile capacity following end of consolidation (here assumed 100 days) based on the procedure in Karlsrud et al (2005)**

It is an important principal question if ageing effects actually first starts near the end of the re-consolidation phase or is linked to absolute time, independent of the pile dimensions and time for re-consolidation. If ageing is mostly a geo-chemical effect it may be mostly related to absolute time. It is, however, considered more likely that even such geochemical effects in absolute terms will also be related to the state of effective stress in the ground. This is in some ways similar to the question of whether or not volumetric creep (or secondary consolidation) runs in parallel with primary consolidation or first starts at the end of primary. NGI is of the clear opinion that it is a parallel process. The impact of different assumptions will be presented and discussed in Section 4.4.
4) **Effects of geochemical reactions with the pile material**

Steel piles used for friction piles should normally not have any coating, as such coating may reduce shaft friction. Some corrosion will therefore normally take place with time. The released Fe$^{++}$ ions and reaction with available oxygen will have a positive impact on the bonding forces between soil particles and clay platelets, and enhance the strength of the clay. The speed and radial extent of the impact of this process has not been studied as part of this project, but could have some impact on the observed ageing effect.

Although concrete piles are not directly included in this study, there can also be some geochemical reactions between Ca$^{+}$ and K$^{+}$ ions in the concrete that have a similar impact as corrosion in enhancing the strength of the clay.

5) **Effects of failure surface moving away from the pile surface**

The shear stress imposed by the pile will decrease radially with the inverse of the normalised distance to the pile surface. The undrained shear strength after full re-consolidation will also in general vary considerably with distance from the pile surface, e.g. Karlsrud & Nadim (1990). For clays with OCR less than about OCR = 4, Karlsrud & Nadim (1990) suggested that the strength can be at its lowest some distance from the pile wall, whereas for high OCR clays it will be at or very close to the pile surface. This has been confirmed by observations from piles retrieved after installation and testing, where layers of clay from about 1 to about 4 cm thick have stuck to piles in low OCR clays upon retrieval. In high OCR clays the piles are normally clean upon extraction.

If the other factors 3) and 4) discussed above lead to enhanced strength of the clay close to the pile wall, the impact on the ultimate capacity can be further enhanced by increased radial distance to the critical failure surface. The effect is however likely to be limited to 10 % or so gain in capacity, and is impossible to distinguish from other effects without pulling piles up just after load testing.

### 4.2 Assessment of time for re-consolidation

Both analytical and numerical procedures have in the past been used to analyse the impact of pile installation and reconsolidation on the total stresses and pore pressure conditions in the surrounding clay. In a recent study by Karlsrud (2012), the various models were assessed and compared against a large data base established from instrumented pile load tests. On that basis Karlsrud (2012) proposed a semi-empirical procedure for calculating the time $t_{cal}$ to reach a certain degree of consolidation. This method has been used to calculate the degree of reconsolidation at the time of load testing the various piles at the different test sites included in this study.

\[
T \cdot r_0^2 / c
\]

\[
T = \text{time factor as function of degree of consolidation}
\]

\[
r_0 = \text{outer pile radius}
\]

\[
c = \text{coefficient of consolidation}
\]
The coefficient of consolidation shall be taken as:

\[ C = \frac{M \cdot k_0}{\gamma_w} \text{, where} \]

\[ \gamma_w = \text{Unit weight of water} \]
\[ k_0 = \text{In-situ vertical permeability} \]
\[ M = 4 \cdot m \cdot p'_c \text{ (typical oedometer modulus for re-loading)} \]
\[ M = \text{Janbu’s (1963) modulus number for virgin loading} \]
\[ p'_c = \text{preconsolidation pressure} \]

The procedure furthermore includes the following steps.

1. Assume an initial excess pore pressure distribution that decreases linearly with the normalized radial distance, \( r/r_0 \).
2. The radial extent of the initial excess pore pressure field is assumed to correspond to the plasticized radius, \( r_p \), which shall be computed according the simple CEM-EP approach using Eq. 4.2.3 and a recommended shear modulus in relation to OCR as shown in Figure 4.2.1.
3. Use a relation between time factor \( T \) and degree of consolidation, \( U \), according to linear radial consolidation theory, and assuming that the coefficient of consolidation is constant in time and as function of radial distance from the pile shaft. Furthermore it shall be assumed that the initial excess pore pressure distribution decreases linearly with normalised radius \( r/r_0 \) and is equal to zero at \( r \) equal to the calculated plasticized radius, \( r_p \), calculated according to Eq. (4.2.3).
4. This gives time factors as shown in Figure 4.2.2 or Figure 4.2.3.
5. To get good match to measured consolidation times the calculated consolidation times shall finally be corrected according to Eq. (4.2.6).

The normalised plasticized radius, \( r_p/r_0 \), is given by the following expression:

\[ \lambda = \frac{r_p}{r_0} = \left( \frac{G_{50}}{s_u} \right)^{1/2}.\left( \frac{r_0^2 - r_{ie}^2}{r_0^2} \right)^{1/2} \]

\[ G_{50} = \text{Undrained secant shear modulus at 50 \% mobilization of the undrained strength to be taken from Figure 4.2.1} \]
\[ r_{ie} = \text{equivalent internal pile radius } = (r_0 - t) \text{ for an open pile that does not plug during driving.} \]
\[ t = \text{wall thickness} \]

For a pile that partly plugs during driving, the average equivalent internal pile radius can be computed from the following expression.

\[ r_{ie} = r_0(1-p)^{1/2} \]

\[ p = \text{plugging ratio } = (\text{displaced volume})/(\text{displaced volume of closed-ended pile}) \], and given by:

\[ p = 1 - \frac{(r_0-t)^2(L-z_p)}{r_0^2 \cdot L} \]
\[ L = \text{embedded length of pile in the ground} \]
\[ z_p = \text{depth from surface to top of soil plug within the pile} \]
\[ r_e = \text{equivalent wall thickness} = r_0 - r_e \]

The time factors calculated in this manner shall finally be corrected according to:

\[ t_{\text{corr}} = C_t \cdot t_{\text{cal}} \quad (4.2.6) \]

- \[ C_t = 0.6 + 4.5 \cdot (\log(OCR))^{1.5} \] for correcting \( t_{50} \)
- \[ C_t = 0.32 + 6.4 \cdot (\log(OCR))^{1.5} \] for correcting \( t_{90} \)

For degrees of consolidation in between \( t_{50} \) and \( t_{90} \), a linear interpolation of \( C_t \) against \( U \) may be used.

**Figure 4.2.1** – Typical range of normalised \( G_{50}/s_{uc} \) values obtained from CAUC triaxial tests on high quality block samples (from Karlsrud, 2012)

**Figure 4.2.2** – Time factors in relation to normalised plasticised radius (from Karlsrud, 2012)
Figure 4.2.3 – Time factors in relation to normalised degree of consolidation (from Karlsrud, 2012)

Table 4.2.1 summarise the calculated times to reach 90, 95 and 98 % consolidation at the different sites. The input parameters used in the calculations, as presented in Table 4.2.1A), were based on the soils data presented in Chapter 2.

The OCR values used were generally selected as a typical value at about 2/3 of the pile depth.

The observed fairly large depth to the soil plug at the Stjørdal site was rather surprising, and may be due to the top sand layer in combination with a fairly low sensitivity (St=2 to 8) of this clay deposit as compared to the Onsøy clay (St=6 to 20), as well as its more dilative behaviour.

As was shown in Figures 2.2.8 and 2.2.9 the plugging ratio was initially very large during the first 5-6 m of pile penetration at Stjørdal, but then decreased with penetration. For this case the plugging ratio p was taken as a typical average of what was measured from the lower 2/3 of the pile (p = 0.35).

The overconsolidated Cowden clay till has very low sensitivity, but showed surprisingly little plugging. At Femern the plug was more or less continuously cored out during driving, and therefore the plug depth taken as zero. There is still some uncertainty when it comes to how large radial displacements that actually occurred during the pile driving at Femern. In Table 4.2.1 it was assumed to correspond to the full wall thickness, but it could be both smaller and larger.

The main reason for the very long consolidation times calculated for the Femern piles still lies in the very low permeability of the clay. As presented in Chapter 2.5 the permeability could however, vary within a rather wide range of 2·10^{-11} to 2·10^{-12} m/s. An average value of 2.9·10^{-11} m/s was used in the calculations.
At Femern the pore pressures at the pile shaft were successfully measured at two levels, but the measurements did not start before 1 month after pile driving. Thus, the degree of pore pressure dissipation cannot be inferred precisely, ref. Chapter 2.5. Figure 2.5.9 and Figure 2.5.10 therefore compared these measured pore pressures to initial installation pore pressures calculated according to the semi-empirical procedure proposed by Karlsrud (2012). The data suggest that the average degree of consolidation reached after 22 months was around 83 %. As the time for 90 % consolidation is a factor of about 1.55 larger than the time to reach 83% consolidation, the measured (extrapolated) time for 90 % consolidation would be 1.55 \cdot 22 = 34.1 months. This is a factor of about 2 times shorter than the value for t_90 predicted for Femern in Table 4.2.1. Considering the fairly large uncertainties when it comes to the permeability of the Femern clay, these measured and predicted consolidation times are still considered to agree well.

| Table 4.2.1 – Summary of calculated times to reach 90, 95 and 98 % consolidation |
|---|---|---|---|---|---|---|---|
| Site | D (mm) | t (mm) | Embedded L (m) | Plug.ratio p | Equiv wall te (mm) | Average OCR | Calc. G50/su rp/r0 | Permeability k0 (m/s) | Modulus M (kPa) |
| Stjørdal | 508 | 6,3 | 22,6 | 0,350 | 49,2 | 1,45 | 450 | 12,5 | 1,2E-09 | 20000 |
| Onsøy | 508 | 6,3 | 17,7 | 0,250 | 34,0 | 1,5 | 180 | 6,7 | 1,00E-09 | 4800 |
| Cowden | 457 | 12,5 | 9 | 0,265 | 32,6 | 7 | 50 | 3,6 | 1E-10 | 64000 |
| Femern | 508 | 22,5 | 25 | 0,169 | 22,5 | 4 | 80 | 7,1 | 2,9E-11 | 40000 |

The impact of the calculated consolidation times on what can be interpreted as gain in capacity due to consolidation effects in contrast to ageing effects will be discussed in Section 4.4.

4.3 Static capacity in first tests compared to existing design methods

A large number of different design methods for determination of the ultimate shaft friction in clays have been proposed since the early 1950’s. The methods selected for use herein are presented Table 4.3.1. A short description of the different methods, apart from the Karlsrud (2012) method which is described below, is given in Appendix B. It was extracted from a report by NGI (2012). This report is a user manual for the computer program Pacer developed for NGI by Carl Jakob Frimann Clausen. The Pacer program was used to calculate the capacities with different methods. Input parameters and calculated capacities are documented in Appendix C.
Table 4.3.1 – Overview of methods used for calculating ultimate capacity after normal set-up (end of re-consolidation phase)

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2</td>
<td>API (1993) and API RPGEO (2011)</td>
</tr>
<tr>
<td>ICP-96 (Imperial College)</td>
<td>Jardine &amp; Chow (1996)</td>
</tr>
<tr>
<td>FDV-96 (Fugro)</td>
<td>Kolk &amp; van der Velde (1996)</td>
</tr>
<tr>
<td>NGI-05</td>
<td>Karlsrud et al (2005)</td>
</tr>
<tr>
<td>Karlsrud-12</td>
<td>Karlsrud (2012)</td>
</tr>
</tbody>
</table>

Karlsrud (2012) proposed a new α-method as well as an alternative β-method for determination of shaft friction in clays. The α-method is based on much of the same principles as the NGI-05 method proposed by Karlsrud et al (2005b). Unlike all other methods, these methods account for the important impact of the plasticity index of the clay on the ultimate shaft friction. Values of α and β are given by Figure 4.3.1 and 4.3.2 respectively.

Figure 4.3.3 illustrates the significant impact of the plasticity index on the α-values deduced for the pile test results from the Karlsrud (2012) data base for clays with a normalized DSS strength $s_{ud}/\sigma'_{vo}<0.4$. The pile test results stemming from the 1st time tests on S1-1 and O1-1 are included herein. It appears from Figure 4.3.3 that the α-values are particularly sensitive to the plasticity index when $I_p<25\%$. Relatively speaking there is also more scatter in the data for pile tests in clays with $I_p<25\%$ than when it is larger.

The reason for why the commonly adopted restoration factor $\alpha = \tau_{us}/s_u$ is smaller than 1.0 is partly because the radial effective stress has been reduced (mostly low-OCR and low plastic clays) and/or the original clay structure has been broken down and now corresponds to that of re-consolidated remoulded clay (high OCR clays).

![Figure 4.3.1 – Chart for determination of α-values proposed by Karlsrud (2012)](chart.png)
Figure 4.3.2 – Chart for determination of β-values proposed by Karlsrud (2012)

Figure 4.3.3 – Measured α-values from pile tests in clays with $s_{ud}/\sigma'_{v0}<0.4$ (after Karlsrud, 2012)

Table 4.3.2 presents the capacities calculated with the different methods listed in Table 4.3.1 as compared to the capacity measured in the first test on Pile 1 at each site (e.g. S1-1, O1-1 etc.), whereas Table 4.3.3 gives the corresponding ratio of calculated to measured capacities.
Table 4.3.2 – Calculated capacities for methods considered (measured capacities are based on first test on pile 1 at each site)

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Stjørdal</th>
<th>Onsøy</th>
<th>Cowden</th>
<th>Femern</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2</td>
<td>965</td>
<td>519</td>
<td>696</td>
<td>1903</td>
</tr>
<tr>
<td>FDV-96 (Fugro)</td>
<td>811</td>
<td>461</td>
<td>831</td>
<td>2021</td>
</tr>
<tr>
<td>ICP-96 (Imperial College)</td>
<td>958</td>
<td>251 (502)</td>
<td>644 (850)</td>
<td>1042</td>
</tr>
<tr>
<td>NGI-05</td>
<td>498</td>
<td>434</td>
<td>671</td>
<td>1966</td>
</tr>
<tr>
<td>Karlsrud-12</td>
<td>392</td>
<td>542</td>
<td>831</td>
<td>3013</td>
</tr>
<tr>
<td>Measured</td>
<td>392</td>
<td>526</td>
<td>869</td>
<td>3049</td>
</tr>
</tbody>
</table>

Table 4.3.3 – Ratio between calculated and measured capacities

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Stjørdal</th>
<th>Onsøy</th>
<th>Cowden</th>
<th>Femern</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2</td>
<td>2.46</td>
<td>0.99</td>
<td>0.80</td>
<td>0.62</td>
</tr>
<tr>
<td>FDV-96 (Fugro)</td>
<td>2.07</td>
<td>0.88</td>
<td>0.96</td>
<td>0.66</td>
</tr>
<tr>
<td>ICP-96 (Imperial College)</td>
<td>2.44</td>
<td>0.48 (0.84)</td>
<td>0.74 (0.98)</td>
<td>0.34</td>
</tr>
<tr>
<td>NGI-05</td>
<td>1.27</td>
<td>0.83</td>
<td>0.77</td>
<td>0.64</td>
</tr>
<tr>
<td>Karlsrud-12</td>
<td>1.00</td>
<td>1.03</td>
<td>0.96</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Some comments should be made to the basis for- and the results- of the calculated values:

- The test results presented herein were included in the data base used by Karlsrud (2012) for developing his method. Thus it may not be surprising that this method also gives the overall best agreement with measured values.
- None of the first 2 methods account for the important impact of plasticity index on the ultimate shaft friction, which is why they grossly over-predict the capacity at Stjørdal.
- The API-2, Fugro-96 and NGI-05 α-methods are linked to the undrained strength from UU tests. UU tests were carried out on the Stjørdal and Onsøy clays as part of this project (ref. Figure 2.2.6 and 2.3.5). At Femern UU tests were also carried out, but as described in Section 2.5.1 the UU strengths were at this site on average equal to the DSS (s_u) strength profile. For Cowden no UU data have been found, so for that case the typical strengths were based on the assumed DSS strength profile.
- The ICP-96 method is partly based on interface effective friction from ring shear tests, not available for other than the Cowden site (from other research programs at this site). The Pacer program therefore uses default values as suggested by Jardine et al (1996) for this friction angle. It appears
that these ICP-96 method significantly under-predicts the capacity at both Onsøy and Femern.

- Imperial College have earlier carried out ring shear tests on clay from Onsøy (Ridgway and Jardin, 2007) and Cowden (Lehane, 1992). This suggested interface friction angle of respectively 29° at Onsøy and 26.5° at Cowden. Using these interface friction angles enhance the predicted capacities to those given in parenthesis in tables 4.3.2 and 4.3.3, which improves the ICP-96 predictions for these two sites.

- API-RPA states that calculated capacities are relevant for when all excess pore pressures caused by pile installation are fully dissipated. At the time of the 1st time tests this was not the case for all piles. Based on figure 4.1.2, the measured capacities in Table 4.3.2 could have been corrected for the estimated degrees of consolidation in Table 4.2.1. The degree of consolidation was around 93% for the 1st time tests at Onsøy, Lierstranda and Cowden, and the correction would therefore be only a few %. The Femern case is more special as the degree of consolidation was only about 15% when the test F1-1 was carried out. There was on the other hand very little change in capacity with time from the first to the last test (factor of 1.07, table 2.5.1), so consolidation effects is not an important issue for this case.

4.4 Effects of ageing and repeated loading on capacity

4.4.1 Ageing effect from 1st time testing only

This section only deals with results of 1st time testing on the different piles. The impacts of sustained and staged loading are discussed in subsequent sections.

As a starting point for comparing the results of the 1st time tests, it is first attempted to fit the results into the ageing framework discussed in Section 4.1, and expressed by Eq. (4.1.1).

Figure 4.4.1 presents the measured absolute ultimate shaft capacities measured in relation to time after pile installation for the four clay sites. In each diagram it is also shown a trend line selected to fit a linear log-time curve using t₀ = 100 days (3.29 months) as a reference time. This trend line also defines the reference shaft friction capacity, Q_{us0}, selected for each site. All sites seem to follow reasonably well such a trend line, but some comments are given to the interpreted “ageing curve” for some of the sites as follows:

- At the Onsøy site there is a clear trend in increasing capacity for the first 3 tests, then a drop for the 4th test and a new increase for the 5th test. It seems most reasonable to assume that the shift in the curve after the 3rd test is caused by slightly different soil characteristics at the location of piles 4 and 5 as compared to the other piles. An alternative explanation could be that the gain in capacity more or less levels off after 6 months or so, but that is considered a less likely explanation, and does not fit the overall trend in other data.
• At Cowden the relatively speaking high capacity for test pile 4 is most likely due to local differences in soil conditions. The other tests follow a fairly steady increase in capacity with log(t).

• As was presented in Chapter 2.5 excess pore pressures still remained at the pile shaft during testing of the Femern piles. Thus, it is not quite clear if the slight gain in capacity with time observed at this site is due to re-consolidation and/or ageing. The relative increase with time is however in any case small at this site.
Figure 4.4.1 – Suggested ageing lines for all sites-1st time tests only

Table 4.4.1 summarizes the selected ageing parameters for each site. To make the different pile tests more directly comparable, the shaft capacities are given as average normalized \( \alpha \)-values at the time of the first test on each pile. Based on the selected average values of \( \Delta_{10} \) the \( \alpha \)-values corresponding to \( t = 100 \) days have also been calculated and given in Table 4.4.1. Results obtained from 1st time testing of piles at Haga (Karlsrud and Haugen, 1985), St. Alban (Konrad and Roy, 1987) and Oromieh bridge (Karlsrud and Mahan, 2010) are also included in Table 4.4.1. The Haga and St. Alban results were also included in NGI (2011), and the Oromieh data were briefly reviewed in Chapter 3.2.

Table 4.4.1 – Summary of ageing factors, \( \Delta_{10} \), and average \( \alpha \)-values for reference time \( t_0 = 100 \) days

<table>
<thead>
<tr>
<th>Site</th>
<th>w (%)</th>
<th>Ip (%)</th>
<th>OCR</th>
<th>Time after inst. (days)</th>
<th>Average</th>
<th>At 1st test</th>
<th>At 100 days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1st ref. test</td>
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<td>( \alpha )-value</td>
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Figure 4.4.2 and Figure 4.4.3 suggest that the ageing factor increases significantly with decreasing plasticity index and overconsolidation ratio of the clay. The labels associated with the different data points in these figures represent the plasticity index and the overconsolidation ratio respectively. The following tentative correlation, also illustrated by the curves in the figures, captures the data in Table 4.4.1 reasonably well:

\[
\Delta_{10} = 0.05 + 1.3 \cdot (1 - \frac{Ip}{50})^2 \cdot OCR^{-0.5}
\]  

\[ (4.4.1) \]
The NGI (2011) report prepared for Petronas included results of a number of repeated (staged) load tests. Most of these data are presented in Table 4.4.2. The name and ID of test sites identifying the different tests are the same as used in tables 1 and 2 of Appendix D in the NGI (2011) report. The factual pile test results from these staged tests have been corrected based on the staged loading effects observed in the present study (see the later Section 4.4.3). Both the possible impact of plasticity index and overconsolidation ratio of the clay, and the number of load repetitions were considered when estimating a high and low value for the correction factors presented in Table 4.4.2. The “assumed” capacity values in the last column correspond to the average of the high and low estimates.

![Figure 4.4.2 – Summary of ageing factors in relation to plasticity index- all 1st time tests (numbers attached to data points represents OCR values)](image-url)
Using the corrected “assumed” capacities as basis, Figure 4.4.4 presents for each pile the capacity versus log(t). One important observation that can be made from these results is:

- All test data confirm that there is quite a linear gain in capacity with log(t) over the entire time range, which at the most extends over a period of 25 years. Thus, the data suggest no clear upper limit to the time over which a gain in capacity due to ageing can be counted upon.

Table 4.4.3 presents the assumed corrected values for the ageing factor, $\Delta_{10}$, for the earlier staged tests. These test results are represented by the small black points in Figure 4.4.2 and Figure 4.4.3. It appears that these earlier corrected staged tests do not show the same clear correlation to $I_p$ and OCR of the clays as the non-staged tests, but they still show a clear presence of ageing effects for all clay types. These earlier corrected staged tests suggest that there are some uncertainties with respect to the general applicability of the tentative correlation proposed by Eq. (4.4.1).
### Table 4.4.2 – Results of staged load tests taken from NGI (2011)

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### Table 4.4.3 – Values of ageing factor, $\Delta_{10}$, deduced for earlier staged load tests

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### Diagrams

- Top diagram: Total capacity (kN) vs. Time (days) for various sites.
- Bottom diagram: Total capacity (kN) vs. Time (days) for various sites.

Note: The diagrams illustrate the progression of total capacity over time for different sites, showing the variations in load testing results.
Figure 4.4.4 – Gain in capacity with time from corrected old staged tests (data taken from Table 4.4.2)

The last two columns of Table 4.4.1 present the average shaft friction in terms of mobilized $\alpha$-values at the time of the 1st test on each pile. The data are plotted in Figure 4.4.5 and suggest that the ageing factor generally increases with decreasing $\alpha$-value, but the Oromieh test result falls outside the general trend line.

Figure 4.4.5 – Ageing factor in relation to $\alpha$-value at time of 1st test on each pile

Figure 4.4.6 presents the deduced $\alpha$-values at the time of 1st time tests plotted in the Karlsrud (2012) diagram for $\alpha$-values. The mobilized $\alpha$-values in Figure 4.4.1 come from the trend-lines in Figure 4.4.6. For simplicity only values for 1, 3 and 24 months are included in the figure, but the 24 month cases also include the sustained load case for each pile (piles No. 6).
Figure 4.4.6 shows that:

- Relatively speaking the impact of ageing has a larger effect on the “end of re-consolidation” $\alpha$-values for clays with the lowest plasticity index. In other words, ageing partially compensates for the very low $\alpha$-values that have been observed at end of the normal re-consolidation or set-up face for the low-plastic clays.
- The Onsøy pile test suggest that for normally consolidated plastic clays the $\alpha$-value can become larger than 1.0 as a result of ageing.

An important issue in relation to application of ageing effects in design practice is if it can be applied irrespective of the design method used. All methods apart from the NGI-05 and the Karlsrud-12 methods could in the outset over-predict the capacity for piles in low plastic clays, e.g. see the API-curve in Figure 4.4.6. In such cases it would be un-conservative to apply ageing factors as suggested by Figure 4.4.2. Thus, for low plastic clays ($I_p<20-25\%$) use of ageing factors should only be used with the NGI-05 and Karlsrud-12 methods.

For clays with higher plasticity index ($I_p>20-25\%$), the ageing factors could be applicable with all design methods.

For practical applications it is also important to consider how or if the ageing effect depends on the time for re-consolidation or dissipation of the excess pore pressures...
set up by the pile installation. Apart for the special case of the Femern site, the times for 95% consolidation have been calculated to be about 2 to 4 months (Table 4.2.1) for the piles tested in this study. It would be reasonable to assume that essentially all set-up due to re-consolidation has taken place by then, and that the subsequent increases are mainly due to true ageing effects. Thus for these piles using $t_0= 100$ days as a typical reference for ageing effects seems quite reasonable.

In case of the Haga pile tests the time for full re-consolidation was measured to 7 to 8 days (Karlsrud and Haugen, 1985). As discussed earlier, and shown in a later Figure 4.4.10, the gain in capacity with time continued from that point in time, and did not “wait” until 100 days had passed. This suggests that ageing effects plays a role at all times after pile installation, and that anchoring it at $t_0= 100$ days for all cases may be questioned.

The key question is if ageing is an effect that runs in parallel to the re-consolidation process, or if it first starts after more or less complete re-consolidation is reached. This is an issue which is very much the same as if secondary consolidation takes place in parallel with primary consolidation or first starts at end of primary. NGI is of the opinion that it is a parallel process. The following procedure is therefore suggested to determine ageing effect in clays.

The ageing effect is defined by multiplying the capacity during re-consolidation by an ageing factor basically anchored at 100 days. However, for a case with very rapid consolidation (less than 100 days) it would not be reasonable to assume that the ageing effect first starts after 100 days. Similarly, for a case of slow consolidation (much more than 100 days to reach full consolidation) it is suggested that ageing and re-consolidation is a parallel process. At very large times it is reasonable to assume that the combined effect of consolidation and ageing leads to the same shaft friction irrespective of the time required for re-consolidation. It is assumed that this large time can be taken as 100,000 days.

For a case where the time to reach 95 % consolidation is less than 100 days, it is proposed that the capacity increases due to consolidation effects alone up until 95 % consolidation is reached, and from there on increase linearly with log-time until it meets the ageing curve anchored at 100 days after $t= 100,000$ days.

For a case where the time to reach 95 % consolidation is larger than 100 days the capacity is it proposed that the aged shaft friction, $\tau_{usa}$, at any time during and after the re-consolidation phase is calculated from Eq. (4.4.2) as follows:

$$\tau_{usa} = \tau_{usc} \cdot (1 + \Delta_{10} \cdot \log(t/100))$$

where,

$$\tau_{usa} = \text{Ultimate shaft friction due to the combined effects of consolidation and ageing}$$

$$\tau_{usc} = \text{Ultimate shaft friction due to consolidation alone}$$

$t$ = no. of days after pile installation

$\Delta_{10}$ = ageing factor anchored at $t= 100$ days
The value of shaft friction due to consolidation alone, $\tau_{usc}$, is computed as described by the Karlsrud (2012) procedure described in Section 4.2. At completion of the re-consolidation phase (100% consolidation) the shaft friction is designated as $\tau_{usc100}$ (e.g. ultimate shaft friction without considering any ageing effect).

To illustrate the impact of this procedure, example calculations have been carried out for a closed-ended and open-ended pile with diameter of 500 mm. The key soil parameters assumed in the analyses of consolidation times are typical of a medium plastic clay with OCR=1.4 and coefficient of consolidation (as applicable to such analyses) of $c= 20$ m$^2$/yr. The normalized plasticized radius was taken as $r_p/r_0 = 3$ for the open-ended pile and $r_p/r_0 = 15$ for the closed-ended pile. This means that the consolidation times to reach 95% consolidation are respectively 43 days for the open pile and 2665 days for the closed-ended pile, i.e. a factor of about 50 larger for the closed-ended pile. The ageing factor was in the calculations taken as $\Delta_{10} = 0.2$ for a reference time $t_0=100$ days.

Figure 4.4.7 presents the calculated normalized ultimate shaft friction values of $\tau_{usa}/\tau_{usc100}$. At $t=10,000$ days the closed and open-ended pile will attain the same ultimate shaft friction.

Equation 4.2.1 and the analytical approach presented in Section 4.2 implies that the consolidation times are proportional to the square of the pile diameter. Thus, an open-ended pile with a diameter of 3.5 m would have a consolidation times about $(3.5/0.5)^2 = 49$ times larger than an open-ended pile with a diameter of 0.5 m. This is about the same difference in consolidation time as between the closed-and open-ended pile of diameter 0.5m. The closed-ended pile in the example above could in other words also present a very large diameter open-ended pile with a diameter of 3.5 m. For piles

\[\text{Figure 4.4.7 – Example calculation of increase in ultimate shaft friction with time assuming ageing effects runs in parallel with consolidation. For assumed } \Delta_{10} = 0.2.\]
used for offshore petroleum structures the pile diameters are often in the range 2 to 3 m, which means times to reach 95% consolidation can be several years. As pointed out by Karlsrud (2012) the build-up of capacity during consolidation can therefore, be an important factor to consider when designing pile foundations for such structures.

Further field testing would be beneficial to support the hypothesis that ageing effects develops in parallel with the re-consolidation process. This may be achieved by comparable field testing on two closed-ended piles with different diameters, requiring different times for re-consolidation.

4.4.2 Impact of sustained loading

Piles No. 6 tested at Stjørdal, Onsøy and Cowden were for 2 years subjected to sustained loading at a load level of about 60% of the capacity determined from the first tests on pile 1, before the piles were loaded to failure. The capacity after such sustained loading was at all sites found to be larger than for the piles No.5 that were without load during the 2 year period. The relative impact of such sustained loading can be compared by simply taking the ratio of the capacities measured for piles No. 6 and No.5 at each site. As can be seen from Table 4.4.4 and Figure 4.4.8 the positive impact of sustained loading is largest for the low plastic low OCR silty clay at Stjørdal, and hardly existing for the high OCR clay at Cowden. As a tentative suggestion, the positive impact of sustained loading should be limited to low-OCR clays as suggested in Figure 4.4.8.

Table 4.4.4 – Impact of sustained loading in addition to pure time effect as observed by comparing shaft capacities from piles No. 5 and 6

<table>
<thead>
<tr>
<th>Site</th>
<th>w (%)</th>
<th>Ip (%)</th>
<th>OCR</th>
<th>Shaft capacities (kN)</th>
<th>Capacity ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Pile 5</td>
<td>Pile 6</td>
</tr>
<tr>
<td>Stjørdal</td>
<td>29</td>
<td>14</td>
<td>1,41</td>
<td>787</td>
<td>1037</td>
</tr>
<tr>
<td>Onsøy</td>
<td>60</td>
<td>33</td>
<td>1,47</td>
<td>616</td>
<td>726</td>
</tr>
<tr>
<td>Cowden</td>
<td>16</td>
<td>18</td>
<td>11,93</td>
<td>1061</td>
<td>1076</td>
</tr>
</tbody>
</table>
The approach for determination of the time effect for piles subjected to sustained loading would be to first calculate the capacity at a given time for piles not carrying any sustained load, and then multiplying with the sustained loading factor, $C_{\text{sust}}$, as suggested by Figure 4.4.8.

4.4.3 Impact of repeated (staged) loading on the same pile

For each site the impact of repeated or staged loading, as compared to pure ageing effects, was assessed in Chapter 2 by dividing the capacity determined for staged tests at a given time by the capacity of the 1st time test carried out at the same time. Table 4.4.5 and Figure 4.4.9 present typical average values of the stage loading factor, $C_{st}$, in relation to test number on the same pile.

<table>
<thead>
<tr>
<th>Site</th>
<th>$w$ (%)</th>
<th>$l_p$ (%)</th>
<th>OCR</th>
<th>$s_{u(d)}$ (kPa)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stjørdal</td>
<td>29</td>
<td>14</td>
<td>1,41</td>
<td>32,7</td>
<td>1</td>
<td>1,4</td>
<td>1,6</td>
<td>1,7</td>
<td>1,6</td>
</tr>
<tr>
<td>Onsøy</td>
<td>60</td>
<td>33</td>
<td>1,47</td>
<td>23,5</td>
<td>1</td>
<td>1,06</td>
<td>1,05</td>
<td>1,03</td>
<td>0,98</td>
</tr>
<tr>
<td>Cowden</td>
<td>16</td>
<td>18</td>
<td>11,93</td>
<td>113</td>
<td>1</td>
<td>1,06</td>
<td>1</td>
<td>0,95</td>
<td>0,93</td>
</tr>
<tr>
<td>Femern</td>
<td>37</td>
<td>110</td>
<td>4,62</td>
<td>85</td>
<td>1</td>
<td>0,73</td>
<td>0,67</td>
<td>0,63</td>
<td>0,6</td>
</tr>
<tr>
<td>Haga</td>
<td>38</td>
<td>20</td>
<td>5,74</td>
<td>41,2</td>
<td>1</td>
<td>1,23</td>
<td>1,44</td>
<td>1,27</td>
<td>NA</td>
</tr>
</tbody>
</table>
Figure 4.4.9 – Average staged loading factors in relation to test number on the same pile - all sites

Table 4.4.5 and Figure 4.4.9 also include results of pile tests at the Haga test site reported by Karlsrud and Haugen (1985), and shown in Figure 4.4.10. The Haga tests were the first to really document the potentially large effect of staged loading on the same pile. It may be noted that the time for 100% pore pressure dissipation was about 7 days for the Haga piles. The Haga data still seems to follow the same trend as the other tests with much longer times for consolidation.

Figure 4.4.10 – Results of staged and non-staged load test- Haga (after Karlsrud and Haugen, 1985)
For all the clay sites the effect of staged loading enhanced the capacity as compared to time effects alone. Figure 4.4.11 summarizes the relative increase in capacity from the 2nd and 3rd test on each pile as compared to the 1st test and seen in relation to the plasticity index. The data suggest a clear trend of diminishing staged loading effect with increasing plasticity index. The trend lines suggested in Figure 4.4.11 were used to correct the earlier pile test data from the NGI (2011) report, ref. Table 4.4.3.

![Figure 4.4.11 – Summary of impact of repeated (staged) loading to failure for all site](image)

5 Overall comparison and assessments for piles in sand

5.1 Factors impacting effects of time on the ultimate shaft friction in sand

Apart from the consolidation and thixotropy aspects, time effects for piles installed in sand may be associated with similar factors as for clay discussed in Section 4.1.

Partly based on an overview by Jardine et al (2006), the following discuss various possible causes.

1) Increase in radial effective stress against the pile surface with time

This is believed caused by a gradual loss of circumferential arching (hoop) stresses around the pile, possibly due to creep within the sand skeleton. Axelsson (2000) confirmed a gradual increase in effective stress with time measured against 235mm square concrete piles. Gavin et al (2013) presented some data from a 340 mm pipe pile driven 7m into a dry dense fine sand at Blessington in Ireland. The pile S5 was instrumented with miniature earth pressure cells at 3 levels. Although there were some problems with shifts in zero readings (change in zero readings before and after extraction of the piles), the data suggested some increase in radial stress at the upper two levels (3.43 m and 5.13 m) over the 220 day observation period, and decrease at
the lowest level (6.49 m). As the earth pressures generally were relatively speaking much lower at the top than at the bottom of the pile, it is uncertain what the net effect of the earth pressure changes would be on the total shaft friction.

2) More dilatant response of the sand with time

More dilative response may be associated with a gradual rearrangement of sand particles leading to a denser soil fabric, possibly also due to creep effects. In this connection it is worth to notice that significant grain crushing of sand particles is likely to take place when driving a pile into quartz sands, which can make the sand more susceptible to changes in fabric over time. Bowman (2002) reported some laboratory studies on behavior sand under very high stresses involving grain crushing, and suggested that creep straining makes such sands more dilatant with time. A more dilative response will lead to increased effective stress against the pile surface during pile loading, and thus, increased shaft friction.

Gavin et al (2013) also presented some earth pressure data for the Blessington pile S5 when it was loaded to failure 220 days after it was installed, Figure 5.1.1. These data suggest a very strong increase (about doubling) in effective stress for the lowest e-p sensor, but little effect at the two higher levels. It is possible that such strong dilative response would not have been observed at an earlier time of testing. Assuming a linear interpolation of the measured e-p values at failure for this Blessington pile S5, and an interface effective friction of \( \tan \delta = 0.7 \), leads to a calculated shaft capacity of about 1100 kN, which compares fairly well to the measured capacity of pile S5 of 990 kN. Without this dilative response, and assuming no change in e-p during pile loading, gives a capacity of about 750 kN. Another pile (S3) tested 12 days after installation showed for comparison a capacity of 665 kN. It is therefore possible that the sand contract or behaves neutrally at early times after installation, and then behaves more and more dilative with time.

![Figure 5.1.1 – Earth pressures measured during loading of pile S5 at Blessington 220 days after it was installed (after Gavin et al, 2013)](image)
3) Geochemical reactions leading to increased cohesion and/or interface friction angle

Like in clays, there is bound to be some geochemical reactions between the pile material and the surrounding sand. Corrosion, as for instance observed along part of the test piles used by Imperial College at the Dunkirk test site (Chow et al, 1998), is likely to have some effect, but not necessarily large. Mitchell and Solymar (1984) postulated that formation of silica acid gel at the sand particle contacts would lead to increased cohesion, and could possible explain the significant increase in CPT cone resistance with time observed in sand deposits subjected to dynamic compaction.

4) Method and/or energy used for pile installation

Lim (2013) presented some new pile tests on displacement piles that were jacked into the ground at the Shenton Park sand test bed often used by the University of Western Australia. The test piles were 65, 100 and 135 mm in diameter, and jacked into the ground to depths ranging from about 3 to 5.5 m. The piles were loaded to failure in tension from 1 to 72 days after installation, but showed practically no gain in capacity over this time period. Lim (2013) found that his results on jacked piles were in strong contrast to tests on various driven piles installed in the same sand deposit, as previously reported by Schneider (2007). The driven piles showed a very large and steady gain in capacity with log-time. The normalized shaft friction was initially much smaller than for the driven piles, but surpassed that when more than 70-80 days had passed from pile installation to testing. Thus, these test programs clearly suggest that the method and/or energy used for pile installation both impact the absolute capacities and the relative ageing effects.

More research is definitely needed to determine which of the four main mechanisms discussed above dominate the time or ageing effect for piles in sand.

5.2 Static capacity in first tests compared to existing design methods

For the two sand sites the methods used for calculating the capacity are presented in Table 5.1.1. A short description of the different methods is given in Appendix B. It was extracted from a report by NGI (2012). This report is a user manual for the computer program Pacer developed for NGI by Carl Jakob Frimann Clausen. The Pacer program was also used to calculate the capacities with different methods. Input parameters and calculated capacities are documented in Appendix C.
Table 5.2.1 – Overview of methods used for calculating ultimate capacity in sand

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2</td>
<td>API (1993)</td>
</tr>
<tr>
<td>FBV-05 (Fugro)</td>
<td>Kolk et al (2005)</td>
</tr>
<tr>
<td>ICP-05 (Imperial College)</td>
<td>Jardine et al (2005)</td>
</tr>
<tr>
<td>NGI-05</td>
<td>Clausen et al (2005)</td>
</tr>
<tr>
<td>UWA-05 (Univ. of Western Australia)</td>
<td>Lehane et al (2005)</td>
</tr>
<tr>
<td>UWA-05 Simplified</td>
<td>Lehane et al (2005)</td>
</tr>
<tr>
<td>ISO-07</td>
<td>ISO (2007)</td>
</tr>
</tbody>
</table>

Table 5.1.2 and 5.1.3 present the calculated capacities and compare them to what was measured in the 1st tests on Pile 1 at each site. The following comments pertain to the calculated values:

- The calculated capacities show very large variation at both sites as compared to what was measured, which is rather disheartening. The ratio between the maximum and minimum predicted values varies by a factor 6.2 at Larvik and 2.2 at Ryggkollen.
- The average of all methods gives an under-prediction of the measured capacity at Larvik by 8% and over-prediction at Ryggkollen by 60%.
- The lack of a complete CPT profile for the Ryggkollen site makes the predictions for this site more uncertain than for the Larvik site, but is considered not likely to explain the fairly large over-prediction for some methods for this site.

Table 5.2.2 – Comparison between calculated and measured capacities for all methods considered (capacities are based on first test on pile 1 at each site)-sand sites

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Larvik</th>
<th>Ryggkollen</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2</td>
<td>1441</td>
<td>1751</td>
</tr>
<tr>
<td>FBV-05</td>
<td>232</td>
<td>790</td>
</tr>
<tr>
<td>ICP-05</td>
<td>403</td>
<td>1345</td>
</tr>
<tr>
<td>ICP-05 Simp.</td>
<td>243</td>
<td>1012</td>
</tr>
<tr>
<td>NGI-05</td>
<td>412</td>
<td>1525</td>
</tr>
<tr>
<td>UWA-05</td>
<td>417</td>
<td>1312</td>
</tr>
<tr>
<td>UWA-05 Simp.</td>
<td>185</td>
<td>790</td>
</tr>
<tr>
<td>ISO-07</td>
<td>535</td>
<td>1702</td>
</tr>
<tr>
<td>Measured</td>
<td>557</td>
<td>811</td>
</tr>
</tbody>
</table>
Table 5.2.3 – Ratio between calculated and measured capacities- sand sites

<table>
<thead>
<tr>
<th>Method designation</th>
<th>Ratio of Calculated/Measured capacities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Larvik</td>
</tr>
<tr>
<td>API-2</td>
<td>2.59</td>
</tr>
<tr>
<td>FBV-05</td>
<td>0.42</td>
</tr>
<tr>
<td>ICP-05</td>
<td>0.72</td>
</tr>
<tr>
<td>ICP-05 Simp.</td>
<td>0.44</td>
</tr>
<tr>
<td>NGI-05</td>
<td>0.74</td>
</tr>
<tr>
<td>UWA-05</td>
<td>0.75</td>
</tr>
<tr>
<td>UWA-05 Simp.</td>
<td>0.33</td>
</tr>
<tr>
<td>ISO-07</td>
<td>0.96</td>
</tr>
</tbody>
</table>

There will always be some room for judgment when setting the parameters to go with different methods. For most of the methods the Pacer program sets “default” values that can be inferred from other parameters when specific relevant data are missing. The interface friction angle to go with the API-2 method is for instance in API (1993) described in relation to sand type (dominating grain size) and assumed relative density groups (loose, medium or dense). In Pacer the density group is selected directly on basis of input cone resistance.

NGI asked Carl Jakob Frimann Clausen to make an independent calculation and assessment of capacities for the Larvik and Ryggkollen sites. His calculations are presented in Appendix D (Larvik) and Appendix E (Ryggkollen). Clausen’s predicted values agree within a few % up or down with the values in Table 5.2.2. In Appendix D Clausen also considers the effect if the pile tip is located within a layer of clayey silt rather than in sand at Larvik. In that calculation the tip resistance was calculated assuming undrained conditions and bearing capacity according to equations (2.2.2) and (2.2.3). The impact of such an assumption is to reduce the interpreted shaft friction by about 120 kN. With such a reduction, the interpreted shaft friction at Larvik would be 404 kN and very close to what is predicted with the ICP-05, NGI-05 and UWA-05 methods.

5.3 Effects of ageing and repeated loading

5.3.1 Ageing effects from 1st time testing only

Figure 5.3.1 compares measured shaft capacities with time for the 1st time tests at the Larvik and Ryggkollen sites, as well as the Dunkirk site dealt with in Chapter 3. Two different attempts have been made to fit the capacity increase to a linear increase with log(t/t0) as suggested by Eq. (4.1.1), and anchored at t0=10 days as proposed by NGI (2011).

1. It was first attempted to make a “best fit” to the data neglecting that there may be some variability of the ground conditions across the Larvik and Ryggkollen sites. Such a log-linear approximation fits most of the data rather poorly, and it is difficult to draw a line that captures all the data. The “best
fit” trend lines suggested in Figure 5.3.1 give unreasonable large difference in \( \Delta_{10} \)-values (respectively \( \Delta_{10}=2.4 \) at Larvik, \( \Delta_{10}=5.0 \) at Ryggkollen and \( \Delta_{10}=0.6 \) at Dunkirk and Blessington) when considering the fact that the total capacity gains after about 1 year are quite similar at all sites. The capacity at \( t_0=10 \) days also becomes unrealistically low with this “best fit” log-linear approximation.

2. A second attempt to give a good log-linear fit to the data was made by selecting a common value of \( \Delta_{10}=0.6 \) for all the sites, and also ensuring that the capacity at \( t_0=10 \) days was in fair agreement with predicted capacities. The possible variability in soil conditions across the sites was in this case also considered. For the Larvik site this means that the capacity for test L7-1 (1 month test) and L2-1 (4.4 month test) were considered to be on the low side, and tests L4-1 and test L5-1 on the high side, ref. discussion in Chapter 2.6 and Appendix D. The result of test L1-1 was ignored completely, mostly due to the high rate of loading. At Ryggkollen, tests R5-1 and R3-1 were judged to be on the high side and tests R1-1 and R2-1 on the low side, ref. Chapter 2.7 and Appendix E.

At the Ryggkollen site Pile R5 hit a stone at about 8 m depth, and effectively plugged the pile during subsequent driving. The correction factor applied to this pile (factor 0.8) was based on the relative driving resistance. Due to the stone and early plugging, the driving resistance may for this pile not be quite relevant when considering shaft friction, and the applied correction could be too large.

From Figure 5.3.1 it seems reasonable to conclude that the Case 2) log(t)-fit with \( \Delta_{10}=0.6 \), gives a better approximation to the test results at all four sites than Case 1), but it is still far from perfect.
Figure 5.3.1 – Gain in shaft capacity with time for 1st time tests - sand sites-log-linear plot

- Ryggkollen
  - $t_0 = 10$ days
  - $\Delta_{50} = 0.6$
  - $Q_0 = 750$ kN
  - $t_0 = 10$ days
  - $\Delta_{50} = 5.0$
  - $Q_0 = 180$ kN

- Dunkirk
  - $t_0 = 10$ days
  - $\Delta_{50} = 0.85$
  - $Q_0 = 1490$ kN

- Blessington
  - $t_0 = 10$ days
  - $\Delta_{50} = 0.6$
  - $Q_0 = 500$ kN
The Larvik and Ryggkollen data suggest a tendency for the gain in capacity to level off after 12 months or so. This has also been suggested by Tan et al (2004). For the sand sites it has therefore, been attempted to fit the data into another framework or curve fitting procedure. To facilitate that, the data have also been plotted in a linear time scale, Figure 5.3.2. Some aspects or parameters the fitting model should reflect are as follows:

- Many pile test data on which existing bearing capacity models are based upon come from pile tests carried out fairly shortly after driving (from a few days to a month or so). It is therefore considered reasonable to start the capacity gain at $t_0 = 10$ days.
- For the Larvik and Ryggkollen tests the measured capacities have been corrected for possible variations in ground conditions. For the Larvik site correction factors were based on the relative variations in cone CPT resistance as assessed by C.J.F. Clausen in Appendix D. At Ryggkollen the corrections were based on the relative driving resistance determined by C.J.F. Clausen in Appendix E.
- From the current tests, this initial capacity, $Q_{us0}$, at $t_0 = 10$ days, was defined by manual extrapolation of the trend in the data at different times back to $t_0 = 10$ days. For this purpose the linear time scale was used as supplement to the log-linear time scale.
- The capacity curves are assumed to level off at about 12-24 months after pile installation.
- The variation in capacities between the values of $Q_{us0}$ and $Q_{usmax}$ were at the three sites assumed to follow a curved tanh type function as described by Eq. (5.2.1).

$$Q_{ust} = Q_{usref} + a \cdot \tanh\left(\frac{b}{t-t_{ref}}\right),$$  \hspace{1cm} (5.2.1)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>fitting parameter which corresponds to $(Q_{usmax} - Q_{usref})$</td>
</tr>
<tr>
<td>$B$</td>
<td>fitting factor, unit 1/time</td>
</tr>
<tr>
<td>$t$</td>
<td>time after pile installation</td>
</tr>
<tr>
<td>$t_{ref}$</td>
<td>fitting time at which the slope of the increase with time is the largest</td>
</tr>
<tr>
<td>$Q_{usref}$</td>
<td>reference shaft capacity at $t = t_{ref}$</td>
</tr>
<tr>
<td>$Q_{us0}$</td>
<td>shaft capacity at $t_0 = 10$ days (calculated with fitting parameters)</td>
</tr>
<tr>
<td>$Q_{usmax}$</td>
<td>maximum shaft capacity reached (calculated with fitting parameters)</td>
</tr>
</tbody>
</table>

Table 5.3.1 summarizes the parameters that were selected to give the best fit to measured capacities. The table also includes some selected staged tests discussed later. The part of the table which is in yellow represents values calculated using the selected fitting parameters. The fitted curves are represented by the curves in red color in Figure 5.3.1 and Figure 5.3.2.
Larvik

1st time tests-uncorrected
1st time tests-corrected
tanh-correlation

log(t)-fit, d10 = 2.4
log(t)-fit, d10 = 0.6

\( t_0 = 10 \text{ days} \)
\( \Delta_{10} = 2.4 \)
\( Q_0 = 240 \text{ kN} \)

Ryggkollen

1st time tests-uncorrected
1st time tests-corrected
tanh-fit

log(t)-fit, d10 = 5.0
log(t)-fit, d10 = 0.6

\( t_0 = 10 \text{ days} \)
\( \Delta_{10} = 0.6 \)
\( Q_0 = 520 \text{ kN} \)

Dunkirk

1st time tests

\( t_0 = 10 \text{ days} \)

\( \Delta_{10} = 0.6 \)
\( Q_0 = 240 \text{ kN} \)
Figure 5.3.2 – Gain in shaft capacity with time for 1st time tests - sand sites-linear plot

Table 5.3.1 – Summary of curve fitting parameters and capacities based on tanh() fitting procedure, sand sites

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Larvik</th>
<th>Ryggkollen</th>
<th>Dunkirk</th>
<th>Blessington</th>
<th>Emshav.</th>
<th>Florida</th>
<th>Dunk R1/T</th>
<th>Dunk CS/T</th>
</tr>
</thead>
<tbody>
<tr>
<td>a (kN)</td>
<td>260</td>
<td>540</td>
<td>1300</td>
<td>500</td>
<td>6400</td>
<td>1000</td>
<td>550</td>
<td>350</td>
</tr>
<tr>
<td>b (1/months)</td>
<td>0.4</td>
<td>0.2</td>
<td>0.3</td>
<td>0.2</td>
<td>0.3</td>
<td>0.6</td>
<td>0.4</td>
<td>0.1</td>
</tr>
<tr>
<td>t_{ref} (months)</td>
<td>2.4</td>
<td>7.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.5</td>
<td>0.6</td>
<td>1.5</td>
<td>16.0</td>
</tr>
<tr>
<td>Q_{us,ref} (kN)</td>
<td>603</td>
<td>810</td>
<td>1490</td>
<td>437.0</td>
<td>9540</td>
<td>1940</td>
<td>1360</td>
<td>380</td>
</tr>
<tr>
<td>Q_{us,max} (kN)</td>
<td>1040</td>
<td>1820</td>
<td>3400</td>
<td>1100</td>
<td>19600</td>
<td>3100</td>
<td>2150</td>
<td>1050</td>
</tr>
<tr>
<td>Q_{us,max}/Q_{us,ref}</td>
<td>1.72</td>
<td>2.25</td>
<td>2.28</td>
<td>2.52</td>
<td>2.05</td>
<td>1.60</td>
<td>1.58</td>
<td>2.76</td>
</tr>
<tr>
<td>a/Q_{us,0}</td>
<td>0.43</td>
<td>0.67</td>
<td>0.87</td>
<td>1.14</td>
<td>0.67</td>
<td>0.52</td>
<td>0.40</td>
<td>0.92</td>
</tr>
<tr>
<td>Q_{ref}/Q_{us,0}</td>
<td>1.29</td>
<td>1.58</td>
<td>1.41</td>
<td>1.37</td>
<td>1.38</td>
<td>1.08</td>
<td>1.18</td>
<td>1.84</td>
</tr>
</tbody>
</table>

Some observations that can be made from Table 5.3.1 regarding the un-staged tests are:

- The maximum ageing factor reached for all the four sand sites as defined by the ratio $Q_{us,max}/Q_{us,0}$ are quite similar (range 1.72 to 2.52), especially considering that the four sites cover a reasonable range of grain size and relative density of sands.
- The reference time is longer for the Ryggkollen site (7 months) than the three other sites (2–2.4 months).
- The steepness of the gain curves, as defined by the parameter $b$, is fairly similar for all three sites.

Table 5.3.2 presents data from some of the earlier staged load tests presented in the NGI (2011) report prepared for Petronas. It can be mentioned that the Emshaven site refers to the EURIPIDES test program reported by Kolk et al (2005). This was a large scale test pile with diameter of 770 mm driven to 50 m tip penetration in a dense to
very dense sand with cone resistance up to 70 MPa. The results given in Table 5.3.2 present the tension capacity of test pile II. During the first test on this pile 6 days after installation the tension test was carried out just after a compression test to failure. The test 533 days after installation also started with compression loading, but loading was stopped prematurely due to limitations of the loading equipment. Judging from the reported load-displacement curve the subsequent tension test also was stopped somewhat prematurely. The ultimate capacity could therefore be somewhat larger than the maximum load applied which is given in Table 5.3.2.

Table 5.3.2 shows both the actual measured capacities and capacities corrected for staged load testing effects. The applied correction factors are based on the Ryggkollen and Larvik staged pile test results presented in a later Figure 5.3.6. Note that the measured capacities are divided by the given stage correction factors. Thus, corrected capacities are larger than measured. Three sets of corrected capacities are given. The “large” and “small” correction factors are those representing respectively the Ryggkollen and Larvik results in Figure 5.3.6. The third represents the average of the two.

Table 5.3.2 – Summary of capacities reported from earlier staged pile tests in sand

<table>
<thead>
<tr>
<th>Site ID</th>
<th>Site name</th>
<th>Pile Name</th>
<th>Test no</th>
<th>Time (days)</th>
<th>Time (months)</th>
<th>Capacity (kN)</th>
<th>Correct factor for staged loading</th>
<th>Corrected capacities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Large</td>
<td>Small</td>
<td>Average</td>
</tr>
<tr>
<td>2.1</td>
<td>Dunkirk</td>
<td>CS/T</td>
<td>0</td>
<td>188</td>
<td>6.18</td>
<td>382</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>272</td>
<td>8.94</td>
<td>422</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>1990</td>
<td>65.44</td>
<td>737</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td>2.2</td>
<td>Dunkirk</td>
<td>CL/T</td>
<td>0</td>
<td>176</td>
<td>5.79</td>
<td>444</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>288</td>
<td>9.47</td>
<td>534</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>2086</td>
<td>68.60</td>
<td>796</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td>5</td>
<td>Dunkirk</td>
<td>R1/T</td>
<td>0</td>
<td>9</td>
<td>0.30</td>
<td>1418</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>57</td>
<td>1.87</td>
<td>1468</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>239</td>
<td>7.86</td>
<td>1614</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td>4.3</td>
<td>Florida</td>
<td>SBZ/T</td>
<td>0</td>
<td>4.03</td>
<td>0.13</td>
<td>1916</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>17.9</td>
<td>0.59</td>
<td>2138</td>
<td>0.6</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>69.9</td>
<td>2.30</td>
<td>2207</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>293</td>
<td>9.61</td>
<td>2365</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>1057</td>
<td>34.76</td>
<td>2362</td>
<td>0.35</td>
<td>0.75</td>
</tr>
<tr>
<td>6</td>
<td>Eemshaven</td>
<td>47TII/T</td>
<td>0</td>
<td>7</td>
<td>0.23</td>
<td>9390</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
<td>533</td>
<td>17.53</td>
<td>17590</td>
<td>0.6</td>
<td>0.9</td>
</tr>
</tbody>
</table>

Figure 5.3.3 presents the capacity-time curves for the un-corrected staged test results (left hand side) as well as for the case of “low” correction (right hand side).

Some observations that can be made from these earlier staged tests are as follows:

- Even without correction (increase) for effects of staged loading, all test data show a fairly significant increase in capacity with time.
- The increase in time seems for the most part to follow a linear trend with log(time), but the Florida tests suggest that the increase tends to level off after 2 years or so.
- The Dunkirk tests CS/T and CL/T in the lower diagram suggest a very different, and much larger and later increase with time, as compared to the R1/T pile at Dunkirk in the middle diagram. This could possibly be related...
to the fact that these piles were loaded to failure in compression in between the tension tests.

- A linear extrapolation of the tests CS/T and CL/T in the log(time) diagram to a reference ageing time at $t_0 = 10$ days as commonly used for piles in sand, leads to a negative capacity at this time ($t=10$ days). Thus, a log-t time extrapolation is highly unreasonable for these cases.

As for the new pile tests the corrected staged test results were fitted to a tanh type correlation defined by Eq. (5.2.1). The fitted curves are shown in the right hand side diagrams in Figure 5.3.3, and the curve fitted parameters are included in Table 5.3.1.
The following presents some comments and observations related to how the tanh type curve-fitting parameters determined for the staged tests tie in with the non-staged test parameters:

- The maximum gain factors as defined by $Q_{\text{umax}}/Q_{\text{uso}}$ are on average similar to the non-staged test results, but more variable. That may well be due to the uncertainties associated with the applied correction factor.
- The reference time, $t_{\text{ref}}$, defining when the increase in capacity is steepest, is generally smaller than for the non-staged test. The exceptions are for the Dunkirk CL/T and CS/T tests which suggest much longer reference times than all other tests. As discussed above those results may not be representative.

The differences in observed patterns in increase in capacity with time for the corrected staged tests confirm the impression from the new non-staged test results (Figure 5.3.1 and Figure 5.3.2) that how the capacity increases with time can be quite variable. It may depend on a number of factors such as:

- Grain size distribution and relative density of the sand.
- Mineral composition of the sand (as suggested by for instance Jamiolkowski and Manassero, 1995).
- Pile characteristics (diameter, length, in situ stress conditions, OCR, open or plugged pile)
- Installation method

For staged tests, the loading sequence, load intensity or displacement reached in previous loading may also have a significant impact, e.g. Jardine et al (2006).

Figure 5.3.4 compares the tanh type curve fitted development of normalised capacities, $Q_\text{um}/Q_\text{uso}$, with time for all tests, including the new non-staged tests as well...
as the earlier corrected staged tests. The curve-fitted parameters used are those given in Table 5.3.1. It may be recalled that the capacity $Q_{us0}$ refers to the capacity corresponding to $t_0 = 10$ days. A logarithmic time scale is used in the upper diagram and a linear time scale in the lower diagram.

There is in the outset fairly large variations in how the capacity increases with time, but most data fall into a fairly narrow range. The Dunkirk CS/T case should, as discussed earlier, be given little weight. In relation to the other corrected staged tests it may also be recalled that a “low” correction was made for staged loading effects. Thus, the gain in capacities for the staged tests could well be significantly larger, ref. Table 5.3.2.

Figure 5.3.4 – Development of normalised capacity with time in 1st time testing based on tanh type curve-fitted parameters, all tests (log scale top, linear scale bottom)
5.3.2 Impact of sustained loading

As was shown in Chapters 2.6 and 2.7, the capacities reached for the piles subjected to a sustained tension load at about 60 % of the capacities in the respective 1\textsuperscript{st} tests, were significantly lower than for the piles left in the ground without any loading. Table 5.3.3 presents the shaft capacities determined for Piles 5 (1\textsuperscript{st} time loading) and Pile 6 (sustained load) to the reference capacity at \( t_0 = 10 \) days, \( Q_{us0} \), based on the tanh-type curve fitting of the test results. Note that the given capacities are the values corrected for local variations across the two sites as discussed in Chapter 5.2.

It appears from Table 5.3.3 that the gain in capacity with time is significantly reduced as a result of the sustained loading, and most so for the Ryggkollen site.

As a comment, the Ryggkollen test R6 may be unduly impacted by numerous problems with the hydraulic equipment used to hold the load constant (caused by oil leaks). Pile R6 therefore had to be unloaded and reloaded from the sustained load level several times. This could explain why the ageing factor under sustained loading at Ryggkollen is lower than at Larvik, and should be given less weight.

One reason for the observed negative impact of sustained loading for the tests at Larvik and Ryggkollen may be related to the tensile type loading the piles are subjected to. This may tend to reduce the vertical and lateral effective stresses acting against the pile surface, and thus, tend to reduce shaft friction. The implication could be that for piles loaded in compression, sustained loading would give no negative effect at all, even possibly giving the opposite effect of further enhancing the ageing effect.

Table 5.3.3 – Relative impact of sustained loading on capacity reached after about 24 months

<table>
<thead>
<tr>
<th>Site</th>
<th>Pile 5</th>
<th>Pile 6</th>
<th>( Q_{us0} )</th>
<th>Pile 6/Pile 5</th>
<th>Pile 5/( Q_{us0} )</th>
<th>Pile 6/( Q_{us0} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Larvik</td>
<td>1045</td>
<td>855</td>
<td>603</td>
<td>0.82</td>
<td>1.73</td>
<td>1.42</td>
</tr>
<tr>
<td>Ryggkollen</td>
<td>1640</td>
<td>940</td>
<td>810</td>
<td>0.57</td>
<td>2.02</td>
<td>1.16</td>
</tr>
</tbody>
</table>

For most practical design situations, a sustained load will always be there, but maybe rarely at as high a level as 60 % of the ultimate failure load as in these tests. It would be reasonable to assume that the negative impact of permanently sustained loading would decrease with the relative static load level.

Figure 5.3.5 presents a tentative recommendation for a reduction factor, \( C_{sust} \), to be applied for sustained loading effects in sand. The correction factor is suggested to vary linearly with load level.

In design practice it is normally applied a total safety factor of at least 1.6 to the ultimate bearing capacity. Furthermore, the actual maximum design load (including
variable loads) will in general be larger than the permanent sustained load, which also will tend to reduce the impact of the sustained loading component.

As an example of how the sustained loading factor may be applied in design assume a safety factor of 1.6 is applied to the ultimate bearing capacity, and that the sustained load is a factor of 0.6 times the maximum design load. Based on Figure 5.3.5 this gives then:

$$\frac{Q_{\text{sust}}}{Q_{\text{us0}}} = \frac{0.6}{1.6} = 0.437$$

which from Figure 5.3.6 gives $$C_{\text{sust}} = 0.75$$

Figure 5.3.5 – Tentative suggestion for sustained loading correction factor in relation to normalized load level

5.3.3 Impact of repeated (staged) loading to failure

The impact of repeated loading on the same pile was to cause a reduction in capacity at both the Larvik and Ryggkollen sites. Figure 5.3.6 compares typical reduction factors versus the number of load repetitions for the two sites. This was based on selecting typical average reduction factors for the two sites was presented in respectively Figure 2.6.17 and Figure 2.7.13.

The reduction due to repeated or staged loading was clearly larger and more dramatic for the Ryggkollen tests than for the Larvik tests, levelling off at values of 0.35 and 0.75 respectively at the two sites.

The conclusion that can be drawn from this is that ageing effects in sand will be significantly underestimated if it is based on repeated load testing to failure on the same pile. The data are considered too limited to propose a generally applicable correction factor to account for staged loading effects, but the Larvik and Ryggkollen data could represent close to an upper and lower limit of this effect.
The potential of using PDA re-strike tests with CAPWAP analyses for establishing ageing effects

PDA/CAPWAP analyses of re-strike tests carried out on two piles at the Onsøy site shows fair agreement but 5 to 25 % on the low side of the measured ultimate shaft friction in the last tests on each pile.

At the Stjørdal clay site the PDA/CAPWAP analyses were too unreliable to be of any value.

For the Larvik sand site the capacity determined from the PDA/CAPWAP analyses from pile driving of 4 of the piles showed large variations, from 134 to 1013 kN (Table 2.6.5), which makes it very difficult to make any meaningful comparison to results of the first load tests.

The re-strike tests on piles L1 and L4 after the end of the test program at Larvik confirm a significant gain in capacity with time. These deduced ultimate shaft capacities also varied considerably, but were significantly larger than measured in the last load tests (factor 1.03 to 1.85 larger). The difference could possibly be explained by compression versus tension type loading.

At the Ryggkollen site the PDA/CAPWAP analyses during pile driving of pile R4 and R5 gave shaft capacities that were only about 25 % of the capacity measured in the 1st static test on pile R1.

The re-strike tests at Ryggkollen were only successful for piles R4 and R6. The capacities deduced from these PDA/CAPWAP analyses were in this case clearly lower than measured in the last static tests on the same piles (factor of 0.5 and 0.9 lower) and not higher as observed at Larvik. This difference in behaviour may partly
be due to the much larger loss in capacity observed in repeated load testing on the same pile at Ryggkollen than at Larvik (see Figure 5.2.6).

Some overall conclusions drawn from the PDA/CAPWAP analyses are as follows:

- The PDA/CAPWAP results show in general surprisingly large scatter at all sites. That may be due to poor quality of the data collected and/or uncertainties related to the method of data analyses and interpretation used by the programs that come with equipment.
- For the Onsøy clay site the deduced capacities in the re-strike tests may still be considered in fair agreement with, but on the low side, of what was measured in the final static tests on the piles. For piles in clay, it should in general be noted that it is only re-strike tests after essentially complete re-consolidation has been reached that can be relevant for design.
- The results from both the Larvik and Ryggkollen sand sites show large scatter both during installation and during the final re-strike tests. This suggests that for piles in sand one should generally be very cautious with using PDA/CAPWAP results on a few piles to deduce ultimate shaft friction.
- The deduced capacity during pile driving at Ryggkollen was very much lower than the ultimate shaft capacity measured in the first static tests. Thus, PDA/CAPWAP analyses collected during pile driving can significantly underestimate the capacity also in sand type materials. Because this effect was not so clear at the Larvik site, the sand type seems to play an important role in this respect.
- For the two sand sites the large differences in impact of repeated load-testing on the same pile, also introduces large uncertainties in capacities deduced from re-strike tests after static load testing. Repeated re-strike testing can therefore, as repeated load testing, give an incorrect picture of time or ageing effects.

7 Conclusions and recommendations

7.1 Some overall observations and recommendations

The systematic pile testing programs carried out as part of this project at 4 clay sites and 2 sand sites confirm that there can be a significant and positive gain in ultimate shaft friction with time for piles installed in both types of materials. For piles in clay this is a gain that comes in addition to the normal set-up due to dissipation of excess pore pressures generated during pile installation.

The observed ageing effect in clays suggests a linear increase in ultimate shaft friction with logarithm of time, but the rate of increase depends on the type of clay. The increase is clearly the largest for lean normally consolidated clays with low plasticity index as at the Stjørdal test site (about 55 % gain from 3 months to 2 years), and smallest in highly plastic overconsolidated clays like at the Femern test site. The ageing effect also tends to reduce with the overconsolidation ratio of the clay. Some
older data reviewed and included in the study suggest that the ultimate shaft friction may keep increasing for at least two decades after pile installation.

The gain in capacity with time for piles installed in sand deposits was found to be surprisingly large (increase by factor of about 2), and even larger than for piles installed in clay deposits. Unlike for piles in clay, the gain in ultimate shaft friction with time does not seem to follow a linear increase with logarithm of time, and tends to level off 1 to 2 years after pile installation.

The new pile test results show that there are distinct effects of repeated (staged) loading as well as of sustained loading on the gain in ultimate shaft friction with time. These effects are also very different for piles installed in sand as compared to in clay deposits, and need to be accounted for when applying the results of this and earlier studies of time effects.

The tentative procedures proposed herein for dealing with time or ageing effects should be applicable in design practice, irrespective of pile dimensions, and for piles loaded in compression as well as tension, but the impact of sustained loading must be accounted for.

For piles subjected to a significant cyclic loading component it is tentatively suggested that the static reference capacity used as basis for addressing cyclic loading effects can be the upgraded “aged” static capacity as defined herein. Otherwise the impact of cyclic loading on the bearing can be dealt with as summarised by Jardine et al (2012). This means that the cyclic capacity will increase by the same amount as the static capacity as a result of ageing effects. This issue may be subject to further verification studies as discussed below.

The following sections give a more detailed summary of test results for piles in clay and sand. This include tentative recommendations for how to deal with time effects in design practice.

It should be noted that the pile test results show some scatter, partly due to local variations in ground conditions. This introduce some uncertainty in the selected "ageing curves" observed at the different test sites as well as in the tentatively proposed methods. Further field-testing programs can reduce this uncertainty.

7.2 Summary and tentative recommendations for dealing with time effects for piles in clay

The present test program, as well as older staged load test that have been reviewed as part of this project, clearly document the presence of significant ageing effects on the ultimate shaft friction for piles in clay.

The ageing effect is found to be proportional to the logarithm of time as given by Eq. (4.4.1). Data from earlier staged pile tests that have been reviewed and included in the study, suggest the capacity can increase for several decades after the end of the normal re-consolidation period.
The ageing factor, $\Delta_{10}$, anchored at a reference time of $t_0=100$ days, can be determined on basis of the trend curves proposed in Figure 4.4.2 and Figure 4.4.3. The proposed trend curve implies that the ageing effect increases significantly with decreasing plasticity index of the clay and decreases with increasing OCR. As examples, for the low-plastic Stjørdal clay test site with $I_p\approx14\%$ the pile tests suggest that the shaft friction increased by a factor of 1.56 from $t_0=100$ days to 2 years. At the medium plastic Onsøy clay test site with $I_p\approx33\%$ the increase in the same time period was by a factor of 1.11.

For piles in clay where the predicted time for 90% consolidation is much longer than 100 days, as could be the case for large diameter offshore piles, it is proposed that the ageing effect is accounted for by assuming that ageing is an effect that runs in parallel with the re-consolidation process. Chapter 4.4.2 and Figure 4.4.7 propose how this can be dealt with.

For piles installed in clay with low plasticity ($I_p<\text{about 25}\%$) caution should be used in combining the proposed ageing effects with methods for calculating the ultimate shaft friction that do not account for the impact of plasticity index on the ultimate shaft friction after normal set-up or re-consolidation. That means, the ageing effect should in such cases only be combined with the NGI-05 or Karlsrud-12 methods. For clays with higher plasticity index ($I_p>\text{about 25}\%$), ageing effects may also be combined with other design methods.

For piles in clay subjected to sustained loading there are strong indications that this can further enhance the ageing effect, especially in clay deposits that are in geologic terms essentially normally consolidated. In such cases, the “aged” capacity could tentatively be increased further by multiplying it by a sustained loading factor as suggested by Figure 4.4.8. This staged loading factor was also found to be larger at the low plastic Stjørdal clay site ($C_{\text{sust}}=1.32$) than at the Onsøy clay site ($C_{\text{sust}}=1.16$). The sustained loading factor can be applied on top of the pure ageing effect for assessing the present capacity of existing pile foundations. It should be applicable for both tension and compression type loading.

The new pile tests clearly document that for low plastic clays repeated or staged load tests on the same pile will result in significant over-estimation of ageing effects. For high plastic overconsolidated clays, the effect can be the opposite, e.g. under-estimation. Results from repeated or staged pile load testing on the same pile should therefore, be used with caution when assessing ageing effects. Figure 4.4.11 presents tentative correction factors to be applied to results of staged load test.

### 7.3 Summary and tentative recommendations for dealing with ageing effects in sand

Both results of the present test program as well as earlier staged load test results reviewed as part of this project show that the ultimate axial capacity of piles installed in sand increases substantially with time.
Unlike in clays, the ageing factor in sands does not seem to increase linearly with logarithm of time elapsed since pile installation. The rate of build-up seems to be slower at early stages (when seen in log-time plot), then accelerates and finally seems to level off more or less completely at a time of 1 to 2 years after pile installation.

The maximum ultimate shaft capacity, $Q_{us\text{max}}$, reached after 1-2 years range from a factor of 1.58 to 2.76 times the reference capacity, $Q_{us0}$, as defined in Table 5.2.1 for a reference time of $t_0=10$ days.

The value of $Q_{us0}$ can be assumed to correspond to the ultimate shaft capacity determined by present design methods, or the ultimate shaft capacity determined from a load test on a pile left in the ground for about 10 days prior to load testing.

As was shown in Chapter 5.1, however, the capacities predicted with current design methods for the Larvik and Ryggkollen sites showed disturbingly large scatter. No single method predicts well the capacity at both sites. If, or to what extent, accounting for the positive impact of ageing on the ultimate shaft friction in sands can be combined with all relevant design methods is an aspect that needs further considerations based on a reliability approach.

It is difficult to tell from the present test data how or if the maximum gain factor, $Q_{us\text{max}}/Q_{us0}$, depends on the specific characteristics of the sand type, pile dimensions etc. It is still tentatively suggested that this ratio on the conservative side may be taken as:

- $Q_{us\text{max}}/Q_{us0} = 1.7$ for piles in fine, loose to medium dense sands (like Larvik)
- $Q_{us\text{max}}/Q_{us0} = 2.1$ for piles in medium to coarse, medium to very dense sands (like Dunkirk)

As was shown in Figure 5.3.4, there are considerable variations in time after pile installation when the capacity shows the most significant gain. In some cases the strongest increase took place during the first month or so after installation, in other cases first a year or so after installation. As a general and conservative recommendation, it is proposed to assume a build-up with time similar to that for the Ryggkollen site in Figure 5.3.4, with parameters $b$ and $t_{ref}$ as given in Table 5.3.1.

There are strong indications from present as well as earlier pile driving studies and load tests that the shaft friction during and shortly after pile driving is significantly lower than the reference ultimate shaft capacity $Q_{us0}$ as defined above. How the capacity builds up during these early phases may be a subject to study further.

Unlike for piles in clay, sustained loading on a pile tends to reduce ageing effects rather than enhance it. For piles subjected to sustained loading in tension it is therefore, proposed to reduce the maximum gain factor by a sustained loading reduction factor, $C_{\text{sust}}$ as presented in Figure 5.3.5. For piles subjected to loading in compression it is tentatively proposed to assume no correction, i.e. $C_{\text{sust}}=1.0$. 
7.4 The potential for using pile driving analyses for assessing ageing effects

PDA measurements with CAPWAP analyses have been carried out during pile installation (Larvik and Ryggkollen only) and as re-strike tests some time after the final static load tests had been carried out (Onsøy, Stjørdal, Larvik, Ryggkollen).

The results show surprisingly large scatter and lack of consistency. The reason for that is not clear.

Although the results tend to confirm a gain in capacity with time, the large scatter and uncertainty associated with the PDA/CAPWAP results suggests that such tests should not on their own be used for assessing ageing effects at a specific site.

7.5 Some tentative recommendations for further studies

Similar test programs as carried out as part of the present study should be considered carried out in a wider range of sand and clay deposits than covered herein.

The present study has had its focus on ultimate shaft capacity. It could be worthwhile to use the measured pile head load-displacement curves for the tested piles to back-calculate t-z curves, and how this is impacted by time or ageing effects.

For piles in clay, a test program to verify that ageing effects run in parallel with the re-consolidation process is highly recommended. This could be achieved by installing new test piles at the Stjørdal site (with the largest ageing effect seen so far) that will have much shorter or longer time for re-consolidation than for the piles used in the present study.

There is a need for new test programs to study how the shaft capacity for piles in sand builds up during the first hours and days after pile installation. In this connection it should also be studied how that ties in with PDA/CAPWAP analyses. It could also be worthwhile to carry out test on piles left in the ground for more than 2 years to verify that the capacity levels off before that, as suggested by the Larvik, Ryggkollen and Florida tests.

For piles installed in sand it may be worthwhile to verify that ageing effects are also applicable to piles loaded in compression, and in that connection also how- or if- end bearing is impacted by ageing effects.

Lim (2014) found that piles jacked into the ground showed little or no gain in capacity over a 72 day period, which was in strong contrast to the significant gain observed for piles installed by driving in the same sand deposit. This suggests that both the method and energy used for pile installation may impact both the absolute capacity and how it develops with time. Further parallel testing is needed to sort out if or to what extent both the installation method and the amount of energy used affects the capacity at any given time after pile installation.
For piles installed in both sand and clay it is recommended to initiate test programs to verify that ageing effects can be accounted for also for piles subjected to significant cyclic loading components.

It could be of great interest for both sand and clay to see if or to what extent ageing effects could be revealed from direct simple shear (DSS) testing. That would imply testing samples left to consolidate for periods up to 2 years or so prior to shearing. One challenge with such a test program is that it will occupy the test apparatus for a very long time, and that only limited numbers of DSS testing equipment are readily available for such testing. An option may be to consolidate the samples in special oedometer cells prior to placing them in the DSS apparatus for shear testing.

For clay materials, DSS tests should in case be carried out on fully remoulded-reconsolidated clay samples consolidated to the same stress level for different time periods prior to undrained shearing. A similar DSS test program may be considered on samples pre-sheared to say 100 % strain rather than completely remoulded. The idea behind such DSS tests is to re-create the impact of pile installation on the properties of the clay closest to the pile wall.

Similar DSS test programs should also be considered carried out on sand materials. How to capture the impact of pile installation on the properties of the sand closest to a pile wall is in this case a more difficult issue. Heavy pre-tamping before re-building the sample in the DSS apparatus may be one option to consider.

It is recognised that such DSS testing will not capture the possible impact of stress changes in the ground surrounding a pile, and thus, it will tend to underestimated ageing effects.

To reduce some uncertainties associated with the test results from the present study some supplementary soils investigations are recommended as follows:

- New sampling to verify Ip- values at the Stjørdal site
- Block sampling and new lab tests to verify the in-situ undrained DSS strength at Stjørdal
- Similar block sampling and DSS testing of samples from Cowden
- Supplementary CPT tests at Ryggkollen

7.6 Plans and suggestions for dissemination of results to ensure implementation in design practice

The project and the budget included plans- and allowances- for publishing the test results in relevant fora.

The original confidentiality period set forth in the contract would not allow publishing of the results until 2 years after project completion. The participants have later agreed to reduce that, and make the results open from 1 March 2014.
The following referee papers are planned for:

- A paper presenting factual results and tentative design procedures for piles in clay
- A paper presenting factual results and tentative design procedures for piles in sand

Geotechnique or the ASCE-journal will primarily be considered for such referee publications.

Papers will also be considered submitted to the following conferences:

- Offshore Technology Conference (OTC) 2014- brief summary of results and proposed procedures for both sand and clay
- Int. Symp. on Frontiers in Offshore Geotechnique (ISFOG) 2015 (see below)- summary of test results and proposed design procedures

Draft papers will be sent to the participants for informal review and comments before they are submitted for publishing.

The API and ISO working groups on piles have been asked to review and comment upon the results and recommendations given in this report. Having such a 3-party review by the API/ISO working groups will hopefully, expedite the process of including time effects in future updating of the relevant design codes, and its implementation in practical design.

It is the intention to include these review comments in a revised version of this report. For this 3-party review, the following tentative time schedule is planned for:

1. NGI send the report to the working groups by the end of June 2013.
2. The working group discuss individual review comments at their first planned upcoming meeting on 10 September 2013.
3. The working group summarize their main agreed comments well in advance of their next meeting on 12 December 2013, and forward these comments to NGI.
4. NGI comes to the 12 December 2013 meeting to present and discuss the results of the study and the review comments.
5. The API/ISO working group then finalize their review comments, and NGI incorporate them in a revised version of this report. These final review comments should be sent to NGI by end of December 2013. A revised version of the report will then be issued by the end of January 2014.

To further enhance dissemination of results it is planned to arrange a special seminar in connection with the upcoming ISFOG (2015) Symposium, which will be arranged at NGI that year. In this connection, special invitation will be sent to other research groups known to be working with ageing effects. Some of these participants will be
asked to prepare papers also addressing the results of the present study, which will be made available in due time before the Symposium.

It will also be considered to present the results at special pile seminars arranged by the ECSMGE or the ISSMGE Technical Committees dealing with pile foundations or other relevant piling conferences.

8 References


CPTpro (GEOsoft), http://www.geosoft.com.pl


Appendix A - List of project reports
1 Progress reports

20061251-00-247-R Time Effects on Pile Capacity Progress. Progress Report 1, 28 October 2009

20061251-00-257-R Time Effects Piles. Progress Report 2, 12 May 2010

20061251-00-259-R Time Effects Piles. Progress Report 3, 21 December 2010

20061251-00-268-R Time Effects Piles. Progress Report 4, 15 September 2011

2 Site investigation reports

20061251-00-244-R Factual Report, Test site Larvik Rev. 01, 10 January 2011

20061251-00-248-R Factual Report, Test site Onsøy, 23 September 2011

20061251-00-262-R Factual Report, Test site Ryggkollen, 8 May 2012

20061251-00-249-R Factual Report, Test site Stjørdal, Rev. 01 5 June 2012

3 Installation and load test results

20061251-00-280-R Time Effects on Pile, Installation Details and Load Test Results, Test site Cowden, 12 April 2012

20061251-00-300-R Time Effects on Pile, Installation Details and Load Test Results, Test site Fermern, 12 April 2013

20061251-00-265-R Time Effects on Pile, Installation Details and Load Test Results, Test site Larvik, 10 April 2013

20061251-00-264-R Time Effects on Pile, Installation Details and Load Test Results, Test site Onsøy, 10 April 2013

20061251-00-266-R Time Effects on Pile, Installation Details and Load Test Results, Test site Ryggkollen, 11 May 2011

20061251-00-263-R Time Effects on Pile, Installation Details and Load Test Results, Test site Stjørdal, 10 April 2013
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5.5 FBV-05, Kolk et al (2005)
5.6 NGI-05, Clausen et al (2005)
5.7 UWA-05, Lehane et al (2005)
5.8 ISO-07, ISO 19902:2007

6 Pile Tip Resistance in Calcareous Sand

7 References
1 Pile Skin Friction in Clay

1.1 API-1, API-RP2A, 1975 to 1986

The below formulae are based upon the API-RP2A, 15th edition, from 1984.

**Highly plastic clays, e.g. Gulf of Mexico**

NC clays : \( \tau_{\text{skin}} = S_u \)  
(A.1.1)

OC clays : \( S_{u,nc} = 0.25 \cdot \sigma_v' \)  
(A.1.2)

\[ \tau_{\text{skin}} = \text{Max} \ (48 \text{ kPa, } S_{u,nc}) \]  
(A.1.3)

**Other clays**

\( \tau_{\text{skin}} = \alpha \cdot S_u \)  
(A.1.4)

\( \alpha = 1.25 - S_u / 96 \text{ kPa} \)  
(A.1.5)

\( 0.5 < \alpha < 1.0 \)  
(A.1.6)

In program Pacer "highly plastic" is taken to mean \( I_p \geq 40 \% \). The undrained shear strength, \( S_u \), shall be "determined in accordance with ASTM Methods of Tests for Unconfined Compression Strength of Cohesive Soil, ASTM Designator D-2166-63T, or as determined by miniature vane tests".

1.2 API-2, API-RP2A, 1987 to Present

The below formulae are taken from the API-RP2A, 20th edition, from 1993.

\( \tau_{\text{skin}} = \alpha \cdot S_u \)  
(A.2.1)

\( \psi = S_u / \sigma_v' \)  
(A.2.2)

For \( \psi < 1.0 \), \( \alpha = 0.5 \cdot \psi^{-0.5} \)  
(A.2.3)

For \( \psi > 1.0 \), \( \alpha = 0.5 \cdot \psi^{-0.25} \)  
(A.2.4)

where \( S_u \) is the undrained shear strength and \( \sigma_v' \) is the vertical effective stress.

In the commentary to the code, Section C.2.6.4, it is stated that "Unconsolidated-undrained triaxial compression tests on high quality samples, preferably taken by pushing a thin-walled sampler with a diameter of 3.0 inches or more, are recommended for establishing strength profile variation because of their consistency and repeatability."
1.3 **NGI-90, Karlsrud & Nadim (1990)**

The below formulae and procedures reflect the present use of this method at NGI.

The method assumes that the skin friction between pile and soil, \( \tau_{\text{skin}} \), is proportional to the horizontal effective stress, \( \sigma'_{hc} \), after the excess water pressures due to pile driving have dissipated:

\[
\tau_{\text{skin}} = \xi \cdot \beta^{\text{RR}} \cdot \sigma'_{hc} \tag{A.3.1}
\]

The factor \( \xi \) corrects for stress differences between a soil element at the pile wall, and a soil element in the DSS apparatus. The factor \( \beta^{\text{RR}} \) is the strength ratio \( \tau_f / \sigma'_{ac} \) for DSS tests on remoulded and reconsolidated clay samples. The factor \( \xi \) is established from the clay over-consolidation ratio OCR as follows:

1. Use the Brooker & Ireland (1965) diagram to establish the stress ratios \( K'_o = \sigma'_{ho} / \sigma'_{vo} \) for the in situ conditions, and \( K'_{o,nc} \) for a normally consolidated clay of the same plasticity.

2. Use Figure 4 of the paper to find the effective horizontal stress against the pile:

\[
\sigma'_{hc} = \eta \cdot \sigma'_{ho} = \eta \cdot K'_o \cdot \sigma'_{vo} \tag{A.3.2}
\]

For the Pacer implementation of the method, it was assumed that the \( \eta \) value in equation (A.3.2) can be expressed as:

\[
\eta = 1.80 - \exp(-0.8 \cdot \ln \text{OCR}) \tag{A.3.3}
\]

The \( \xi \) value needed in equation (A.3.1) can then be calculated as:

\[
\sigma'_{ho} = K'_o \cdot \sigma'_{vo} \tag{A.3.4}
\]

\[
\sigma'_{hc} = \eta \cdot \sigma'_{ho} \tag{A.3.5}
\]

\[
K'_c = \sigma'_{hc} / \sigma'_{vo} \tag{A.3.6}
\]

\[
\xi = \frac{1 + 1/K'_c + K'_{o,nc}}{1 + 2 \cdot K'_{o,nc}} \tag{A.3.7}
\]

The remoulded-reconsolidated strength ratio \( \beta^{\text{RR}} \) quoted by Karlsrud & Nadim (1990) is shown on Figure 3 in their paper. The line labelled "assumed relationship" on that figure is represented as:

\[
\beta^{\text{RR}} = 0.36 - \exp(-2.2 \cdot \ln (\text{OCR}+2.0)) \tag{A.3.8}
\]

The above assumes that failure takes place at the pile/soil surface. One therefore needs to check if a failure at some distance outside the pile surface leads to lower...
capacity, see Karlsrud & Nadim (1990). This check has been omitted in the Pacer version of the method.

A default OCR value is calculated from the below expressions for given $s_{u}^{DSS}$, $\sigma_{vo}'$ and $I_p$:

$$s_{u}^{DSS} = \beta^{DSS} \cdot \sigma_{vo}' \cdot OCR^{0.85}$$  \hspace{1cm} (A.3.9)

$$\beta^{DSS} = 0.05 \cdot (1 + 0.7 \cdot I_p^{0.55})$$  \hspace{1cm} (A.3.10)

$$0.15 < \beta^{DSS} < 0.40$$  \hspace{1cm} (A.3.11)

### 1.4 NGI-92, Nowacki et al (1992)

This method is a proposed modification of the API-RP2A procedure API-2. The method takes into account pile test results in over-consolidated clays that were not available when the API-2 method was established around 1985.

The authors propose to adjust the API-RP2A as follows:

$$\psi = \frac{s_u}{p_o'}$$  \hspace{1cm} (A.4.1)

$$\alpha = 0.5 \cdot \psi^{-0.5} \text{ for } \psi < 0.7$$  \hspace{1cm} (A.4.2)

$$\alpha = 0.56 \cdot \psi^{-0.2} \text{ for } \psi > 0.7$$  \hspace{1cm} (A.4.3)

$$\alpha < 1.0$$  \hspace{1cm} (A.4.4)

The authors used UU tests from Tilbrook, Clarke (1992), for their calibration. They warn that the undrained shear strength may vary considerably from one test type to another.

The above expressions for the $\alpha$-factor shall be used together with a length factor FL, proposed calculated as:

$$L1 = 15 \cdot D$$  \hspace{1cm} (A.4.5)

$$L2 = 0.25 \cdot L_{tot}$$  \hspace{1cm} (A.4.6)

$$L = \text{Min} (L1,L2)$$  \hspace{1cm} (A.4.7)

$$FL = 0.5 \text{ from pile head to } L$$  \hspace{1cm} (A.4.8)

$$FL = 1.0 \text{ from } L \text{ to the pile tip}$$  \hspace{1cm} (A.4.9)

where $D$ is the pile outer diameter and $L_{tot}$ is the embedded pile length. The expression for pile skin friction $\tau_{skin}$ then becomes:

$$\tau_{skin} = FL \cdot \alpha \cdot s_u$$  \hspace{1cm} (A.4.10)
Nowacki et al (1992) also considered the effect of open versus closed pile for OC clays. They note that closed piles, directly comparable to open ones, gave 10-20% higher skin friction. However, for one of the piles (Tilbrook pile segment test D) this is attributed to "whipping" effects during driving of an internal pile. They conclude that:

"Although there is room for different interpretations with respect to effect from open- versus closed piles, it is concluded that as long as length effects are accounted for one can use the proposed design curve in Fig. 12 for both open and closed piles."

1.5 FBV-92, Fugro 1992 method

This is an in-house Fugro BV method that was calibrated against the Pentre and Tilbrook large scale pile tests, Clarke (1992). It is similar to the API-2 method, but with a modified $\alpha$ factor:

$$\tau_{\text{skin}} = \alpha \cdot s_u^{UU} \quad (A.5.1)$$

$$\alpha = 0.5 \cdot \left( \frac{s_u^{UU}}{\sigma_{vo'}} \right)^{-0.16} \quad (A.5.2)$$

1.6 FBV-96, Kolk & van der Velde (1996)

This method also uses an approach similar to API-2. The $\alpha$-factors are based upon the work presented by the authors. They propose that the skin friction $\tau_{\text{skin}}$ in a clay layer is calculated as:

$$\tau_{\text{skin}} = \alpha \cdot s_u \quad (A.6.1)$$

$$\alpha = 0.9 \cdot \left[ \left( \frac{L - z}{D} \right) -0.2 \cdot \left( \frac{s_u}{p'_{o}} \right) -0.3 \right] \quad (A.6.2)$$

$\alpha < 1.0$

Where:

$s_u$ = Undrained shear strength from UU triaxial tests
$L$ = Depth from the surface to the pile tip
$z$ = Depth from the surface to the point considered
$D$ = Pile outside diameter
$p'_{o}$ = Vertical effective stress at depth $z$
1.7 **ICP-96, Jardine & Chow (1996)**

The following describes NGI’s interpretation of the method and its implementation in computer program Pacer. The following calculation procedure is proposed by the authors:

\[
\tau_{\text{skin}} = \sigma'_{rf} \cdot \tan \delta_f
\]

(A.7.1)

\[
\sigma'_{rf} = 0.8 \cdot \sigma'_{rc}
\]

(A.7.2)

\[
\sigma'_{rc} = K_c \cdot \sigma'_{vo}
\]

(A.7.3)

\[
K_c = \{ 2.2 + 0.016 \cdot \text{OCR} - 0.87 \cdot \log_{10} ( S_t ) \} \cdot \text{OCR}^{0.42} \cdot ( H/R )^{-0.2}
\]

(A.7.4)

Where:

- \( \sigma'_{rf} \) = Radial effective stress against the pile wall at failure
- \( \delta_f \) = Pile/soil friction angle, depends upon \( I_p \)
- \( \sigma'_{rc} \) = Radial effective stress against the pile wall after full consolidation
- \( K_c \) = Earth pressure coefficient
- \( \sigma'_{vo} \) = In situ vertical stress
- OCR = Apparent over-consolidation ratio, called YSR by the authors
- \( S_t \) = Clay sensitivity
- \( H \) = Distance from the layer considered to the pile tip
- \( R \) = Outer radius of the pile

In program Pacer equation (A.7.4) was used to determine \( K_c \). However, Jardine & Chow (1996) includes alternative expressions, based upon parameters to be determined from oedometer tests, rather than direct use of the sensitivity \( S_t \).

The above expressions apply for piles driven closed-ended in clay. For open-ended piles, the radius \( R \) shall be replaced by \( R^* \), calculated from:

\[
R^* = ( R^2_{\text{outer}} - R^2_{\text{inner}} )^{0.5}
\]

(A.7.5)

The ratio \( H/R \), or \( H/R^* \), shall be equal to or higher than 8.

The authors recommend that the OCR value is determined from oedometer tests, or in the case of non-brittle soils, from triaxial tests consolidated to different OCR values. The ICP-96 model used by Pacer calculates the OCR value from consolidated triaxial compression strengths, and the vertical effective stress, using Figure 14 in the Jardine & Chow (1996) paper.

The pile/soil friction angle \( \delta_f \) is taken from the curve labelled \( \tan \delta_{\text{peak}} \) in Figure 16 of the paper, based upon ring shear tests carried out by Imperial College on inorganic North Sea clays. The authors state that:

"there is no universal and reliable link between the plasticity index and \( \delta_f \) and site-specific ring shear interface tests are recommended for practical design."
1.8  **NGI-05, Karlsrud et al (2005)**

This method is also based on the API-2 format. However, the $\alpha$-factors are adjusted to reflect (1) the low skin friction observed on piles in soft clays of low plasticity, and (2) the effect of pile tip condition upon the skin friction in stiff clays.

The reference strength with this method is the UU triaxial strength. At a given depth the following values are needed:

- $\sigma_{vo}'$ = Vertical effective stress (kPa)
- $s_{uUU}$ = Undrained shear strength (kPa)
- $I_p$ = Clay plasticity (%)
- $\psi$ = Strength ratio = $s_{uUU} / \sigma_{vo}'$

It is first checked that the strength ratio $\psi$ is higher than 0.25. If that is not the case, the method assumes that the $\sigma_{vo}'$ value is correct, and calculates a corrected $s_{uUU}$ value:

$$s_{uUU}^{NC} = 0.25 \cdot \sigma_{vo}'$$  \hspace{1cm} (A.8.1)

The pile skin friction is calculated as:

$$\tau_{skin} = \alpha \cdot s_{uUU}$$  \hspace{1cm} (A.8.2)

where the $\alpha$-value depends upon the strength ratio, the clay plasticity, and the pile tip condition, as shown on Figure A.1.

*For* $\psi \leq 0.25$:

$$\alpha = \alpha^{NC} = 0.32 \cdot (I_p - 10\%)^{0.3}$$  \hspace{1cm} (A.8.3)

$$0.20 < \alpha^{NC} < 1.0$$  \hspace{1cm} (A.8.4)

*For* $\psi \geq 1.0$:

$$\alpha = 0.5 \cdot \psi^{-0.3} \cdot F_{tip}$$  \hspace{1cm} (A.8.5)

where $F_{tip}$ is taken as 1.0 for a pile driven open-ended. For a closed-ended pile, the value is calculated as:

$$F_{tip \, closed} = 0.8 + 0.2 \cdot \psi^{0.5}$$  \hspace{1cm} (A.8.6)

$$1.0 < F_{tip \, closed} < 1.25$$  \hspace{1cm} (A.8.7)

*For* $0.25 < \psi < 1.0$:

Determine the $\alpha$-value by linear interpolation between $\psi = 0.25$ and $\psi = 1.0$ on Figure A.1, taking the log-scale into account, which leads to:

$$\alpha = 0.5 + (0.83 - 1.66 \cdot \alpha^{NC}) \cdot \log_{10} \psi$$  \hspace{1cm} (A.8.8)
1.9  ISO-07, ISO 19902:2007

This method is identical to the API-2 method presented above.

2  Pile Skin Friction in Silica Sand

2.1  API-1, API-RP2A, 1972 to 1984

The below formulae are based upon API-RP2A, 11th edition, January 1980. The unit skin friction is calculated from:

\[ f = K \cdot \sigma_{vo} \cdot \tan \delta \]  

(A.9.1)

where:
- \( f \) = Unit skin friction between pile and soil
- \( K \) = 0.5 to 1.0 for compressive loading, 0.5 for tensile loading
- \( \sigma_{vo} \) = Effective vertical stress
- \( \delta \) = Soil/pile friction angle, see the table below.

Table A.1: Parameters recommended by API-1 for medium dense to dense frictional soils

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( \phi' ) (degrees)</th>
<th>( \delta ) (degrees)</th>
<th>( N_q )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean sand</td>
<td>35</td>
<td>30</td>
<td>40</td>
</tr>
<tr>
<td>Silty sand</td>
<td>30</td>
<td>25</td>
<td>20</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>25</td>
<td>20</td>
<td>12</td>
</tr>
<tr>
<td>Silt</td>
<td>20</td>
<td>15</td>
<td>8</td>
</tr>
</tbody>
</table>

In Section 2.6.4c of the API-RP2A 1983 edition it is stated that:

“For deep foundations, limiting values of \( f \) and \( q \) may be less than indicated by Eq. 2.6.4-2 (=A.9.1) and 2.6.4-3. These limiting values should be determined for local conditions.”

The designer is thus given considerable freedom in selecting the parameters needed to calculate pile skin friction. In program Pacer the following assumptions are made, partly based upon normal practice by McClelland Engineers Inc. during the mid 1970’s:

- \( K = 0.5 \) for tensile loading
- \( K = 0.7 \) for compressive loading, pile driven open-ended
- \( K = 0.8 \) for compressive loading, pile driven closed-ended
- \( 20^\circ < \phi' < 35^\circ \)
- \( f_{\text{limit}} = 3.35 \cdot \phi' - 19.0 \)  

(A.9.2)
48 kPa < \( f_{\text{limit}} \) < 100 kPa \hspace{1cm} (A.9.3)

### 2.2 API-2, API-RP2A, 1984 to present

The below formulae are based upon the API-RP2A, 20th edition, July 1993.

The skin friction is calculated from equation (A.9.1) with the factor \( K \) taken as 0.8 for open-ended piles loaded in both compression and tension. For piles driven closed-ended, \( K \) is taken as 1.0. Values for \( \delta \) and limiting skin friction values are shown below.

**Table A.2: Soil parameters recommended by API-2, from API (1993)**

<table>
<thead>
<tr>
<th>Density</th>
<th>Soil Description</th>
<th>Soil Friction Angle, ( \delta ) Degrees</th>
<th>Limiting Skin Friction Values ( \text{kPa} ) (kips/ft(^2))</th>
<th>( N_q )</th>
<th>Limiting Unit End Bearing Values ( \text{MPa} ) (kips/ft(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>Sand</td>
<td>15</td>
<td>47.8(1.0)</td>
<td>8</td>
<td>1.9( 40)</td>
</tr>
<tr>
<td>Loose</td>
<td>Sand-Silt**</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>Sand</td>
<td>20</td>
<td>67.0(1.4)</td>
<td>12</td>
<td>2.9( 60)</td>
</tr>
<tr>
<td>Medium</td>
<td>Sand-Silt**</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium</td>
<td>Sand</td>
<td>25</td>
<td>81.3(1.7)</td>
<td>20</td>
<td>4.8(100)</td>
</tr>
<tr>
<td>Dense</td>
<td>Sand-Silt**</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very Dense</td>
<td>Sand</td>
<td>30</td>
<td>95.7(2.0)</td>
<td>40</td>
<td>9.6(300)</td>
</tr>
<tr>
<td>Dense</td>
<td>Sand-Silt**</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Very Dense</td>
<td>Gravel</td>
<td>35</td>
<td>114.8(2.4)</td>
<td>50</td>
<td>12.0(250)</td>
</tr>
<tr>
<td>Very Dense</td>
<td>Sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\*The parameters listed in this table are intended as guidelines only. Where detailed information such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.

\**Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

In order to use the above table in practice one needs to estimate the sand relative density \( D_r \) and also the silt content if the sand contains a "significant fraction of silt". The following formulae are used by program Pacer to calculate \( D_r \) from CPT results if \( D_r \) values are not given as input:

\[
D_r = 0.3413 \cdot \ln \left[ \frac{q_c}{(250 \cdot \sigma_m^{0.51})} \right] \hspace{1cm} (A.10.1a)
\]

or

\[
D_r = 0.4 \cdot \ln \left[ \frac{q_c}{(22 \cdot (\sigma_{vo} \cdot \sigma_{atm})^{0.5})} \right] \text{ for the NGI-05 method} \hspace{1cm} (A.10.1b)
\]

where \( q_c \) is CPT tip resistance, \( \sigma_m \) is the average effective stress, \( \sigma_{vo} \) is vertical effective stress and \( \sigma_{atm} \) is the atmospheric reference pressure, 100 kPa. Pacer calculates the \( \sigma_m \) value as \((\sigma_{vo} + 2K_0' \sigma_{vo}) / 3\) with \( K_0' \) taken as 0.5.
For a given (or calculated) Dr value the corresponding friction angle $\varphi'$ (if not given as input) is calculated from:

$$\varphi' = (23^\circ + 21.4 \cdot Dr) \cdot (1 - 0.082 \cdot \log_{10}(\sigma'_{vo} / \sigma_{atm}))$$  \hspace{1cm} (A.10.2)

$$20^\circ < \varphi' < 40^\circ$$  \hspace{1cm} (A.10.3)

With known $\varphi'$, the soil/pile friction angle $\delta$ is taken as $\varphi' - 5^\circ$. The skin friction is then calculated from equation (A.9.1) and the limiting skin friction from equation (A.9.2). For the API-2 method it is required that:

$$48 \text{kPa} < f_{\text{limit}} < 115 \text{kPa}$$  \hspace{1cm} (A.10.4)

It should be noticed that the API-2 calculation method described above leads to a continuous increase of skin friction as the sand relative density increases, where as the actual code (Table A.2) operates with five soil groups with constant parameter values within each group. In addition, the adapted API-2 calculation method does not include the silt content of a sand as such, only indirectly through the selected $q_c$, Dr or $\varphi'$ values.

### 2.3 ICP-96, Jardine & Chow (1996)

Chow (1996) gives the detailed basis for the method. The proposed calculation procedure uses results from in situ cone penetrometer tests (CPT) as the key soil parameter. This procedure is reproduced on Figure A.2 herein. Step A4 on this figure is supplemented with the check that the term $(A + B\eta - C\eta^2)$ is not allowed to become smaller than 0.02.

The interface friction angle $\delta_f = \delta_{cv}$ is shown on Figure 3 of their paper. In program Pacer this angle is calculated as:

$$\delta_f = 23^\circ - 6.0^\circ \cdot \log_{10}(D_{50})$$  \hspace{1cm} (A.11.1)

$$23^\circ < \delta_f < 35^\circ$$  \hspace{1cm} (A.11.2)

where $D_{50}$ is the mean particle size in mm. If $D_{50}$ is not known, a default value of 0.1 mm is used by Pacer, which results in $\delta_f = 29^\circ$.

### 2.4 ICP-05, Jardine et al (2005)

**Full Method**

The sand skin friction calculation by the original/full version of this method is the same as for ICP-96, see above.
**Simplified Method**

In the proposed commentary to the new API RP2A code, API (2005), a "simplified" ICP-05 method is given, where the dilatancy term ($\Delta \sigma_{rd}$ on Figure A.2) is neglected, and some of the coefficients have been "conservatively rounded up/down" according to API (2005). By this method, the skin friction $f_z$ is calculated as:

$$f_z = u \cdot q_c \cdot \left( \frac{\sigma_{vo}}{\sigma_{atm}} \right)^{0.1} \cdot A_r^{0.2} \cdot F^{-0.4} \cdot \tan \delta$$

(A.11.3)

where:

- $u =$ 0.023 for compressive loading, 0.016 for tensile loading
- $q_c =$ CPT tip resistance at the depth considered
- $\sigma_{vo} =$ Vertical effective stress
- $\sigma_{atm} =$ Atmospheric reference pressure, 100 kPa
- $A_r =$ $1.0 - \left( \frac{D_{inside}}{D} \right)^2$
- $F = \text{Max} \left[ \frac{L-z}{D}, 4 \cdot (4.0)^{0.5} \right]$
- $L =$ Pile embedded length
- $z =$ Depth below the ground surface of the point considered
- $D =$ Pile outside diameter
- $\delta =$ Interface friction angle

### 2.5 FBV-05, Kolk et al (2005)

This method uses the same format and approach as the ICP-96 method, but with adjusted parameters, in order to better match Fugro's data base. It should be noticed that in the original paper equation (10) is not correct. The term given as $(h/R^*)^{-0.90}$ should be replaced by $(4.0)^{-0.90}$.

The skin friction in compression, $\tau_{fc}$, is calculated as:

**For $H/R^* \geq 4.0$:**

$$\tau_{fc} = 0.08 \cdot q_c \cdot \left( \frac{\sigma_{vo}}{\sigma_{atm}} \right)^{0.05} \cdot (H/R^*)^{-0.9}$$

(A.12.1)

**For $H/R^* < 4.0$:**

$$\tau_{fc} = 0.08 \cdot q_c \cdot \left( \frac{\sigma_{vo}}{\sigma_{atm}} \right)^{0.05} \cdot (4.0)^{-0.9} \cdot \frac{H}{(4R^*)}$$

(A.12.2)

The skin friction in tension, $\tau_{ft}$, is calculated as:

$$\tau_{ft} = 0.045 \cdot q_c \cdot \left( \frac{\sigma_{vo}}{\sigma_{atm}} \right)^{0.15} \cdot \left[ \text{Max} (H/R^*, 4.0) \right]^{-0.85}$$

(A.12.3)

where:

- $H =$ Distance from the point considered to the pile tip
- $R^* = \left( \frac{R^2_{outside} - R^2_{inside}}{0.5} \right)$
- $q_c =$ CPT tip resistance
- $\sigma_{vo} =$ Vertical effective stress
- $\sigma_{atm} =$ Atmospheric reference pressure, 100 kPa
2.6 **NGI-05, Clausen et al (2005)**

The following expressions are used to calculate the local skin friction:

\[ \tau_{\text{skin}} = \frac{z}{z_{\text{tip}}} \cdot \sigma_{\text{atm}} \cdot F_{\text{Dr}} \cdot F_{\text{sig}} \cdot F_{\text{tip}} \cdot F_{\text{load}} \cdot F_{\text{mat}} \]  
(A.13.1)

\[ \tau_{\text{skin}} > 0.1 \cdot \sigma_{\text{vo}} \]

where

- \( z \) = Depth below ground surface
- \( z_{\text{tip}} \) = Pile tip depth
- \( \sigma_{\text{atm}} \) = Atmospheric reference pressure, 100 kPa

\[ F_{\text{Dr}} = 2.1 \cdot (D_r - 0.1)^{1.7} \]  
(A.13.2)

\[ F_{\text{sig}} = \left( \frac{\sigma_{\text{vo}}}{\sigma_{\text{atm}}} \right)^{0.25} \]  
(A.13.3)

- \( F_{\text{tip}} \) = 1.0 for a pile driven open-ended, 1.6 for a pile driven closed-ended
- \( F_{\text{load}} \) = 1.0 for tensile loading, 1.3 for compressive loading
- \( F_{\text{mat}} \) = 1.0 for steel, 1.2 for concrete

\( D_r \) = Sand relative density, preferably calculated from CPT results using equation (A.10.1)

\( \sigma_{\text{vo}} \) = Vertical effective stress

2.7 **UWA-05, Lehane et al (2005)**

**Full Method**

In the full version of this method the skin friction \( \tau_f \) is calculated from:

\[ \tau_f = f \cdot (\sigma_{\text{rc}} + \Delta\sigma_{\text{rd}}) \cdot \tan \delta \]  
(A.14.1)

where

- \( f \) = 1.0 for compressive loading, 0.75 for tensile loading
- \( \sigma_{\text{rc}} \) = 0.03 \cdot q_c \cdot A_r^{0.3} \cdot [\text{Max}(H/D, 2.0)]^{-0.5} \]  
(A.14.2)

- \( q_c \) = CPT tip resistance
- \( A_r = 1.0 - IFR \cdot (D_i / D)^2 \)  
(A.14.3)

- \( IFR = \text{Min} [1.0, (D_i / 1.5m)^{0.2}] \)  
(A.14.4)

- \( D_i \) = Pile inner diameter

- \( D \) = Pile outer diameter

- \( H \) = Distance from the point considered to the pile tip

\[ \Delta\sigma_{\text{rd}} = 4.0 \cdot G \cdot \Delta r / D \]  
(A.14.5)

\[ G = q_c \cdot 185 \cdot (q_{\text{c1N}})^{-0.7} \]  
(A.14.6)

\[ q_{\text{c1N}} = (q_c / \sigma_{\text{atm}}) / \left( \sigma_{\text{vo}} / \sigma_{\text{atm}} \right)^{0.5} \]  
(A.14.7)

\( \Delta r = 0.02 \text{ mm} = 2.0 \times 10^{-5} \text{ m} \) for a lightly rusted steel pile

\( \delta \) = Interface friction angle, see Equations (A.11.1 & 2)
Simplified UWA-05 Method

In the simplified version of the UWA-05 method, which the authors propose used for large diameter offshore piles, the following values apply:

\[ IFR = 1.0 \]
\[ \Delta \sigma_{rd} = 0.0 \]

Lehane et al (2005) does not give any recommendation for the interface angle \( \delta \). Program Pacer therefore uses the expressions presented above for the ICP-96 method.

2.8 ISO-07, ISO 19902:2007

This method is based upon the API-2 method described above. However, a continuous relationship between sand relative density and pile skin friction has been included in ISO-07. The unit skin friction \( \tau \) is calculated as:

\[ \tau = \beta \cdot \sigma'_{v0} < \tau_{\text{lim}} \] (A.14.8)

where \( \beta \) and \( \tau_{\text{lim}} \) are given in Table 17.4.1 of ISO 19902:2007, included herein as Figure A.6. It is seen that this method is not applicable for loose sands and silts. Values for \( \beta \) and \( \tau_{\text{lim}} \) are given for two soil types, described as "Sand" and "Sand-silt". Footnote D to the table states that "Sand-silt includes soils with significant fractions of both sand and silt".

Rev.2 of the Pacer program therefore reads the contents of fines for each sand layer as input when the ISO-07 calculation method is specified. The layer is taken to be "Sand" if the content of fines is less than 5%, "Sand-silt" if the content of fines is higher than 15%. For content of fines between 5% and 15%, a linear interpolation between the two is used.

Values to be used for \( \beta \) and \( \tau_{\text{lim}} \) for the two soil types are plotted against relative density on Figure A.7.

3 Pile Skin Friction in Calcareous Sand

The calculation methods for pile skin friction in sand described above assume that the sand is siliceous, i.e. that the grains consists of silica/quartz minerals. For calcareous sands (also called carbonate sands), practical experience has shown that the skin friction could be much lower than normally found for silica sands.

Based upon the results presented in Kolk (2000), and a study of the underlying data, it was decided to implement the following computational models in the Pacer code.
for skin friction in calcareous sands. This option may be used for the calculation methods API-2 and NGI-05.

For each sand layer, Pacer reads 5 miscellaneous values S1 to S5, presently interpreted as:

- S1 = Content of CaCO₃ in %
- S2 = Estimated degree of cementation, expressed as a number 0.0 to 1.0
- S3 = K\cdot\tan(\delta) for API-2 calculations
- S4 = \(\tau_{\text{lim}}\), i.e. limiting skin friction for API-2 CaCO₃ calculations
- S5 = Factor on the calculated sand skin friction, both siliceous and calcareous

If S1 is lower than 20 %, the skin friction is calculated as for a 100 % silica sand. If S1 is higher than 80 %, the skin friction both in compression and in tension is calculated as described below.

### 3.1 API-2 Calcareous Sand Skin Friction Model

\[
\tau_{\text{skin}} = \sigma'_{\text{vo}} \cdot K \cdot \tan(\delta) < \tau_{\text{lim}}
\]  

(A.14.8)

where:

- \(\sigma'_{\text{vo}}\) = Vertical effective stress at the depth considered
- K = Horizontal earth pressure coefficient
- \(\delta\) = Friction angle at pile/sand interface, user input is K\cdot\tan(\delta)
- \(\tau_{\text{lim}}\) = Limiting skin friction

### 3.2 NGI-05 Calcareous Sand Skin Friction Model

\[
\tau_{\text{skin}} = 0.16 \cdot \sigma_{\text{atm}} \cdot \left(\sigma'_{\text{vo tip}} / \sigma_{\text{atm}}\right)^{0.25} \cdot F_{\text{tip}} \cdot z / z_{\text{tip}}
\]  

(A.14.9)

where:

- \(\sigma_{\text{atm}}\) = Atmospheric reference pressure, 100 kPa
- \(\sigma'_{\text{vo tip}}\) = Vertical effective stress at the pile tip level
- \(F_{\text{tip}}\) = 1.0 for a pile driven open-ended, 1.5 for a pile driven closed-ended
- \(z\) = Depth below seafloor of the point considered
- \(z_{\text{tip}}\) = Depth below seafloor of the pile tip

For both methods, the interpolation formula proposed by Kolk (2000) is used to find the skin friction for a CaCO₃ content between 20 % and 80 %:

\[
\tau_{\text{skin}} = \tau_{\text{skin si}} - (\tau_{\text{skin si}} - \tau_{\text{skin ca}}) \cdot \log_{10}(\text{CaCo} / 20 \%) / \log_{10}(4.0)
\]  

(A.14.10)

where:

- \(\tau_{\text{skin si}}\) = Skin friction calculated for a 100 % silica sand
τ_{skin\,ca} = \text{Skin friction calculated for a 100\% calcareous sand}
CaCo = CaCO_3 \text{ content in } %

4  \textbf{Pile Tip Resistance in Clay}

For all methods, except ICP-96 (see below), the tip resistance acting against a closed-ended or plugged pile is calculated as:

\[ Q_{\text{tip}} = 9 \cdot s_u \cdot A_{\text{tip}} \quad \text{(A.15.1)} \]

where \( s_u \) is the undrained shear strength and \( A_{\text{tip}} \) is the gross pile tip area, \( 0.25\pi D^2 \).

The undrained shear strength used is the \( s_u^{UU} \) values, except for the NGI-90 method, for which DSS strengths are used.

For a coring pile with its tip in clay, the unit tip resistance against the wall is taken as \( 9s_u \).

4.1  \textit{ICP-96, Pile Tip Resistance in Clay}

Jardine & Chow (1996) recommend that the method shown on Figure A.3 is used to find the pile tip resistance. Two different loading conditions are considered, undrained and drained. It should be noticed that the check for a plugged or unplugged pile tip only includes pile diameter and \( q_c \) at the tip, and that by this method the tip resistance for a coring pile is independent of the inside friction.

5  \textbf{Pile Tip Resistance in Silica Sand}

5.1  \textit{API-1, API-RP2A, 1972 to 1984}

The plugged pile tip resistance is calculated from:

\[ q_{\text{tip}} = \sigma'_{vo} \cdot N_q \quad \text{(A.16.1)} \]

where \( \sigma'_{vo} \) is the vertical effective stress and \( N_q \) is a bearing capacity factor that depends upon the sand angle of internal friction, see Table A.1. For the API-1 method, program Pacer calculates the \( N_q \) factor from the following expressions:

\[ \phi' < 30^\circ : \quad N_q = 11.6 \cdot \tan^2 (45^\circ + \phi'/2) \cdot \tan(\phi') \quad \text{(A.16.2)} \]
\[ 30^\circ < \phi' < 35^\circ : \quad N_q = 4 \cdot \phi' - 100 \quad \text{(A.16.3)} \]
\[ \phi' > 35^\circ : \quad N_q = 2 \cdot \phi' - 30 \text{ and } N_q < 40 \quad \text{(A.16.4)} \]

The tip stress \( q_{\text{tip}} \) is limited to:
q_{tip\ max} = 0.265 \text{ MPa} \cdot [ \tan \left( 45^\circ + \frac{\phi'}{2} \right) ]^{5.5} < 10 \text{ MPa} \quad (A.16.5)

Control of plugged or coring pile tip is carried out as described for the API-2 method below.

5.2 **API-2, API-RP2A, 1984 to present**

The plugged pile tip resistance is calculated from (A.16.1). The $N_q$ factor and the limiting tip stress are determined by linear interpolation between the values shown in Table A.2 for a given $\delta = \phi' - 5^\circ$.

Control of plugged or coring pile tip is carried out assuming that (1) the inside unit friction is the same as the outside one, and (2) that the stress acting against the pile tip wall is given by (A.16.1).

5.3 **ICP-96, Jardine & Chow (1996)**

The calculation method recommended by the authors is shown on Figure A.4. The pile plugging criterion used, see equation D1, means that in a sand with 100% relative density, piles with an inside diameter of more than 1.4 m cannot plug.

5.4 **ICP-05, Jardine et al (2005)**

The calculation method recommended by the authors is shown on Figure A.5. Program Pacer assumes that both plugging criteria in D1 must be met in order for a pile to plug. It should be noticed that the lower limits on $q_b$ have been increased as compared to the ICP-96 method.

5.5 **FBV-05, Kolk et al (2005)**

The method assumes that the pile is either closed-ended or plugged. The stress against the pile tip is given by:

$$q_{\text{tip}} = 8.5 \cdot \sigma_{\text{atm}} \cdot \left( \frac{q_{\text{c\ avr}}}{\sigma_{\text{atm}}} \right)^{0.5} \cdot \left( \frac{R^*}{R} \right)^{0.5} \quad (A.17.1)$$

where:

- $\sigma_{\text{atm}}$ = Atmospheric reference pressure, 100 kPa
- $q_{\text{c\ avr}}$ = CPT tip resistance, averaged over ± 1.5D from the pile tip
- $R^* = \left( R_{\text{outside}}^2 - R_{\text{inside}}^2 \right)^{0.5}$
- $R$ = Outside radius

For a pile driven open-ended, Pacer checks if the pile is plugged, using the following procedure, recommended in API (2005).

**Check 1:** The pile plugs if the cumulative thickness of sand layers in the
soil plug is more than 8 times the pile diameter.

or

Check 2: The pile plugs if the plugged tip resistance is smaller than

\[ Q_{\text{fins clay}} \cdot e^{(L_s / D)} \]  \hspace{1cm} (A.17.3)

where \( Q_{\text{fins clay}} \) is the inside skin friction force from clay layers, \( L_s \) is the sand plug length, and \( D \) is the pile diameter.

For open-ended piles with a modest tip depth, plugging may not yet have taken place. Pacer therefore calculates the coring tip resistance as:

\[ Q_{\text{coring}} = 0.5 \cdot q_c \cdot A_{\text{tip wall}} + Q_{\text{inside}} \]  \hspace{1cm} (A.17.4)

where the inside friction force \( Q_{\text{inside}} \) is calculated under the assumption that the inside unit skin friction is the same as the outside value.

### 5.6 NGI-05, Clausen et al (2005)

For a pile driven closed-ended, the stress against the pile tip is calculated as:

\[ q_{\text{tip}} = 0.8 \cdot q_{\text{c avg}} / (1 + D_r^2) \]  \hspace{1cm} (A.18.1)

where \( q_{\text{c avg}} \) is the CPT tip resistance averaged over \( \pm 1.5D \) from the pile tip, and \( D_r \) is the relative density.

For a pile driven open-ended, the plugged stress against the pile tip is calculated as:

\[ q_{\text{tip}} = 0.7 \cdot q_{\text{c avg}} / (1 + 3D_r^2) \]  \hspace{1cm} (A.18.2)

The tip resistance for a coring pile is calculated assuming that the inside unit friction in sand layers is equal to the outside value times a factor \( f_{si} \) calculated as:

\[ f_{si} = 1.0 + 0.2 \cdot L_s / D_{\text{ins}} < 3.0 \]  \hspace{1cm} (A.18.3)

where \( L_s \) is the sum of the sand layer thicknesses above the pile tip, and \( D_{\text{ins}} \) is the pile inside diameter. If the pile has an internal driving shoe, a lower \( f_{si} \) factor would probably be appropriate. The stress against the pile tip wall is taken equal to \( q_c \) at the pile tip position. Equation (A.18.3) is an extension of the NGI-05 method, Clausen et al (2005) has \( f_{si} = 3.0 \).
5.7 **UWA-05, Lehane et al (2005)**

*Full Method*

This method assumes that the pile is either closed-ended or plugged. The stress against the pile tip is calculated as:

\[ q_{\text{tip}} = q_{\text{av}} \cdot (0.15 + 0.45 \cdot A_r) \quad (A.19.1) \]

where:

\[ q_{\text{av}} = \text{CPT tip resistance, averaged over} \pm 1.5D \text{ from the pile tip} \]

\[ A_r = 1.0 - \text{FFR} \cdot \left( \frac{D_i}{D} \right)^2 \quad (A.19.2) \]

\[ \text{FFR} = \text{Min} \left[ 1.0, \left( \frac{D_i}{1.5m} \right)^{0.2} \right] \quad (A.19.3) \]

\[ D_i = \text{Pile inner diameter} \]

\[ D = \text{Pile outer diameter} \]

*Simplified UWA-05 Method*

The FFR (final filling ratio) is taken as 1.0, otherwise as above.

Pacer carries out the same coring pile check for the UWA-05 method as described above for the FBV-05 method.

5.8 **ISO-07, ISO 19902:2007**

This method is based upon the API-2 method described above. However, a continuous relationship between sand relative density and pile tip stress has been included in ISO-07.

The unit pile tip stress \( q_{\text{tip}} \) acting against a closed or plugged pile tip is calculated as:

\[ q_{\text{tip}} = N_q \cdot \sigma' v_0 < q_{\text{lim}} \quad (A.19.4) \]

where \( N_q \) and \( q_{\text{lim}} \) are given in Table 17.4.1 of ISO 19902:2007, included herein as Figure A.6. It is seen that this method is **not** applicable for loose sands and silts. The implementation in Pacer is as described above for ISO-07 skin friction in sand.

Values to be used for \( N_q \) and \( q_{\text{lim}} \) for the two soil types are plotted against relative density on Figure A.8.

6 **Pile Tip Resistance in Calcareous Sand**

The pile tip resistance in a calcareous sand could be much lower than for a silica sand, depending upon the degree of sand cementation. For two of the calculation
methods, API-2 and NGI-05. Pacer therefore allows special CaCO₃ calculation methods to be used, if wanted by the user.

These special methods are governed by the Pacer input parameters S1 to S5, see the section on skin friction in calcareous sands above. If the CaCO₃ content of a sand layer (given as S1) is lower than 20 %, the tip resistance is calculated as for a silica sand. If the CaCO₃ content is higher than 80 %, the tip resistance is calculated as described below.

It is first assumed that CPT data is not available.

The plugged or closed-ended pile tip resistance in a 100 % calcareous sand is calculated as:

API-2: \[ q_{\text{tip plgd}} = (1 + C) \cdot 3 \text{ MPa} \]  
NGI-05: \[ q_{\text{tip plgd}} = (15 + 45 \cdot C) \cdot \sigma_{\text{atm}} \cdot (\sigma'_{\text{vo}} / \sigma_{\text{atm}})^{0.25} \]  

where

\[ C = \text{Estimated degree of sand cementation, 0.0 - 1.0} \]
\[ \sigma_{\text{atm}} = \text{Atmospheric reference pressure, 100 kPa} \]
\[ \sigma'_{\text{vo}} = \text{Vertical effective stress at pile tip level} \]

For a coring pile tip, the wall tip resistance in a 100 % calcareous sand is calculated for both methods API-2 and NGI-05 by:

\[ q_{\text{tip wall}} = (150 + 250 \cdot C) \cdot \sigma_{\text{atm}} \cdot (\sigma'_{\text{vo}} / \sigma_{\text{atm}})^{0.25} \]  

The interpolation formula referred to above is also used for a coring pile tip in sands with a CaCO₃ content between 20 % and 80 %.

In case the CPT tip resistance \( q_c \) was given as input, the above results are replaced by:

\[ q_{\text{tip plgd}} = 0.2 \cdot q_c \]  
\[ q_{\text{tip wall}} = 0.5 \cdot q_c \]

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Figure A.1

NGI-05 calculation method for pile skin friction in clay, Karlsrud et al (2005)
TABLE 2. PROCEDURES FOR SHAFT CAPACITY CALCULATIONS IN SAND

<table>
<thead>
<tr>
<th>A</th>
<th>Shaft capacity of closed-ended piles</th>
</tr>
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<tbody>
<tr>
<td>A1</td>
<td>$Q = \pi D \int \tau_i , dz$</td>
</tr>
<tr>
<td></td>
<td>Integral of local shear stresses along the</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>A2</td>
<td>$\tau_i = \sigma_{pa} \tan \delta_i$</td>
</tr>
<tr>
<td></td>
<td>$\sigma_{pa} = \sigma_{pa0} + \Delta \sigma_{pa}$</td>
</tr>
<tr>
<td>A3</td>
<td>$\sigma_{pa0} = 0.029 , q_c , (\sigma_{col}/P_s)^{0.11} , (h/R)^{0.38}$</td>
</tr>
<tr>
<td></td>
<td>Function of CPT resistance, free-field vertical effective stress (normalised by atmospheric pressure, $P_s = 100$ kPa) and $h/R$, $q_c$ is not corrected for YSR. $h/R$ is limited to a minimum of eight.</td>
</tr>
<tr>
<td>A4</td>
<td>$\Delta \sigma_{pa} = 2G , \delta h/R$</td>
</tr>
<tr>
<td></td>
<td>$G = q_c \left[ A + B \eta - C \eta^2 \right]^{1}$</td>
</tr>
<tr>
<td></td>
<td>$\eta = q_c \sqrt{(P_s \sigma_{col})}$</td>
</tr>
<tr>
<td></td>
<td>$A = 0.0203$</td>
</tr>
<tr>
<td></td>
<td>$C = 1.2166e-6$</td>
</tr>
<tr>
<td>A5</td>
<td>$\delta_i = \delta_c$</td>
</tr>
<tr>
<td></td>
<td>May be estimated from Figure 3.</td>
</tr>
<tr>
<td>A6</td>
<td>$\tau_i = (0.8 \sigma_{pa0} + \Delta \sigma_{pa}) \tan \delta_i$</td>
</tr>
<tr>
<td></td>
<td>Equation A6 should be used in place of Equation A2.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>B</th>
<th>Shaft capacity of open-ended piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>B3</td>
<td>$\sigma_{pa0} = 0.029 , q_c , (\sigma_{col}/P_s)^{0.11} , (h/R^*)^{0.38}$</td>
</tr>
<tr>
<td></td>
<td>Substituted into Equation A3 to give B3; $h/R^* &gt; 8$.</td>
</tr>
<tr>
<td></td>
<td>$R^* = (R_{oam}^2 - R_{wam}^2)^{0.5}$</td>
</tr>
<tr>
<td></td>
<td>In tension</td>
</tr>
<tr>
<td></td>
<td>$\tau_i = 0.9 , (0.8 \sigma_{pa0} + \Delta \sigma_{pa}) \tan \delta_i$</td>
</tr>
</tbody>
</table>

Figure A.2

ICP-96 calculation method for pile skin friction in sand, Jardine & Chow (1996)
**TABLE 11. PROCEDURES FOR BASE CAPACITY CALCULATIONS IN CLAY**

<table>
<thead>
<tr>
<th>H</th>
<th>Base capacity of closed-ended piles</th>
</tr>
</thead>
</table>
| H1 | Undrained loading: \( q_h = 0.8 \bar{q}_c \)  
Drained loading: \( q_h = 1.3 \bar{q}_c \) | **Pile base resistance** is controlled by CPT resistance at the founding depth and the drainage conditions during loading. \( \bar{q}_c \) is averaged 1.5 pile diameters above and below the tip. |

<table>
<thead>
<tr>
<th>J</th>
<th>Base capacity of open-ended piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>J1</td>
<td>If ( \frac{D_{num}}{D_{CPT}} + 0.45 \frac{\bar{q}_c}{P_A} ) is less than 36, the pile plugs</td>
</tr>
</tbody>
</table>
| J2 | \( q_b = q_b \pi D^2 / 4 \)  
Undrained loading: \( q_b = 0.4 \bar{q}_c \)  
Drained loading: \( q_b = 0.65 \bar{q}_c \) | **Fully plugged piles** develop half of the end resistance of closed-ended piles given by Equation 11 after a pile head displacement of \( D/10 \). |
| J3 | \( q_b = q_{bu} \pi (R_{min}^2 - R_{num}^2) \)  
Undrained loading: \( q_{bu} = q_c \)  
Drained loading: \( q_{bu} = 1.6 \bar{q}_c \) | **Unplugged piles** sustain end bearing on the annular area of steel only. Base resistance is equal to average CPT end resistance at the founding depth. This may be increased by a factor of 1.6 for drained conditions. Contributions from internal shear stresses should be ignored. |

**Figure A.3**

*ICP-96 calculation method for pile tip resistance in clay, Jardine & Chow (1996)*
### TABLE 3. PROCEDURES FOR BASE CAPACITY CALCULATIONS IN SANDS

<table>
<thead>
<tr>
<th>C</th>
<th>Base capacity of closed-ended piles</th>
<th>Pile base resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>$q_b = q_e \pi D^2 / 4$</td>
<td>Related to the CPT end resistance. $q_e$ is averaged</td>
</tr>
<tr>
<td></td>
<td>$q_e = q_e \left[1 - 0.5 \log(D/D_{CPT})\right]$</td>
<td>1.5 pile diameters above and below the pile toe. Depends on pile diameter. Note $D_{CPT} = 0.036$ m and a lower bound $q_b = 0.13 q_e$ applies for $D &gt; 2$ m.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>D</th>
<th>Base capacity of open-ended piles</th>
<th>Fully plugged piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>A rigid basal plug can only develop if: $D &lt; 0.02 [D_r - 10]$</td>
<td>Develop 50% lower end resistance than comparable closed-ended piles after a pile head displacement of $D/10$. D3 provides a lower bound to D2 at large diameters</td>
</tr>
<tr>
<td></td>
<td>Note: $D$ is in metres and $D_r$ is specified in %.</td>
<td></td>
</tr>
<tr>
<td>D2</td>
<td>$q_b = q_e \pi D^2 / 4$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$q_e = q_e \left[0.5 - 0.25 \log(D/D_{CPT})\right]$</td>
<td></td>
</tr>
</tbody>
</table>

| D3  | $q_{bu} = q_{bu} (R_e^2 - R_i^2)$ | Lower bound for unplugged and large open-ended piles |
|     | $q_{bu} = q_{b u}$                | Allow end bearing on the annular base area of steel only, resistance is equal to average CPT end resistance at the founding depth. Contributions from internal shear stresses should be ignored. |

---

*Figure A.4*

Table 3. Procedures for base capacity calculations in sand

<table>
<thead>
<tr>
<th>C</th>
<th>BASE CAPACITY OF CLOSED-ENDED PILES</th>
</tr>
</thead>
</table>
| C1 | \( Q_b = q_b \pi D^2/4 \)          | Pile base resistance is related to the average CPT end resistance at the founding depth, and the relative pile and CPT diameters. A lower limit of \( q_b = 0.30q_c \) is suggested for piles with \( D > 0.90 \) m. See text regarding \( q_c \) selection.  
\( D_{CPT} = 0.036 \) m. |

<table>
<thead>
<tr>
<th>D</th>
<th>BASE CAPACITY OF OPEN-ENDED PILES</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>( D_{	ext{smw}} &lt; 0.02 ) (( D_b - 30 )) ( D_{	ext{smw}} ) are not the same.</td>
</tr>
</tbody>
</table>

\( D_{	ext{smw}} = \frac{0.083q_c}{P_a} \)

| D2 | \( Q_b = q_b \pi R_{	ext{outer}}^2 \) | Fully plugged piles develop 50% of the end resistance of closed-ended piles of the same diameter (C1 above) after a pile head displacement of \( D/10 \). Two lower limits apply: (i) the fully plugged capacity should be no less than the unplugged capacity (D3 below) and (ii) \( q_b \) should not fall below \( 0.15q_c \) (as predicted for \( D > 0.90 \) m). See text regarding \( q_c \) selection. |

\( q_b = q_c \) \( 0.5 - 0.25 \log (D/D_{CPT}) \)

| D3 | \( Q_b = q_{ba} \pi (R_{	ext{outer}}^2 - R_{	ext{inner}}^2) \) | Unplugged piles are assumed to sustain end bearing on the annular pile base area only with \( q_{ba} = q_c \); see text regarding parameter selection. Contributions from internal shear stresses are not considered explicitly. |

\( q_{ba} = q_c \)

---

**Figure A.5**

Table 17.4-1 — Design parameters for cohesionless siliceous soil

<table>
<thead>
<tr>
<th>Relative density(^{B})</th>
<th>Soil classification(^{C})</th>
<th>Skin friction factor (\beta)</th>
<th>Limiting unit skin friction values (f) kPa (kips/ft(^2))</th>
<th>End bearing factor (N_q)</th>
<th>Limiting unit end bearing values (q) MPa (kips/ft(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>Sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>Sand</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Loose</td>
<td>Sand-silt(^{D})</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium dense</td>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dense</td>
<td>Silt</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium dense</td>
<td>Sand-silt(^{D})</td>
<td>0.29</td>
<td>67 (1.4)</td>
<td>12</td>
<td>3 (60)</td>
</tr>
<tr>
<td>Dense</td>
<td>Sand</td>
<td>0.37</td>
<td>81 (1.7)</td>
<td>20</td>
<td>5 (100)</td>
</tr>
<tr>
<td>Dense</td>
<td>Sand-silt(^{D})</td>
<td>0.46</td>
<td>96 (2.0)</td>
<td>40</td>
<td>10 (200)</td>
</tr>
<tr>
<td>Very dense</td>
<td>Sand-silt(^{D})</td>
<td>0.56</td>
<td>115 (2.4)</td>
<td>50</td>
<td>12 (250)</td>
</tr>
</tbody>
</table>

A The parameters listed in this table are intended as guidelines only. Where detailed information such as in-situ CPT records, strength tests on high quality samples, model tests, or pile driving performance is available, other values are justified. Design values relate to the mid-point in each range of relative density.

B The following soil definitions for relative density descriptions are applicable:

<table>
<thead>
<tr>
<th>Description</th>
<th>Relative density (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>0 - 15</td>
</tr>
<tr>
<td>Loose</td>
<td>16 - 35</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>36 - 65</td>
</tr>
<tr>
<td>Dense</td>
<td>65 - 85</td>
</tr>
<tr>
<td>Very Dense</td>
<td>85 - 100</td>
</tr>
</tbody>
</table>

C Soil classifications are according to ISO 19901-4.

D Sand-Silt includes soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

E Design parameters for these relative density/soil description combinations, given in previous API RP2A documents, can be unconservative. Hence CPT-based methods should be used for these soils (see A.17.4.4).

Figure A.6

ISO-07 method for calculation of pile axial capacity in sand, from ISO 19902:2007
\[ \tau_{\text{skin}} = f \cdot \beta \cdot \sigma'_{v0} < \tau_{\text{lim}} \]

\( f = 1.0 \) for a pile driven open-ended, 1.25 for a closed-ended pile

*Figure A7*

ISO-07 method for calculation of skin friction in sand
\[ q_{\text{tip}} = N_q \cdot \sigma'_{v0} < q_{\text{lim}} \]

*Figure A.8*

ISO-07 method for calculation of the pile tip stress \( q_{\text{tip}} \) that acts against a closed or plugged pile tip in sand
Appendix C - Documentation of calculated pile capacities

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   2.1  Clay sites 2
   2.2  Sand sites 10
3  Pile parameters 15
4  Results 16
   4.1  Clay sites 16
   4.2  Sand sites 17
5  References 17
1 Introduction

This calculation summary presents the input parameters and results of pile calculations using the program PACER with different methods of calculation.

2 Soil parameters

2.1 Clay sites

2.1.1 Testsites Onsøy

The soil parameters chosen for input to the PACER calculations are based on laboratory results presented by NGI (2011a). Figure 1 shows the results from triaxial UU-tests with the chosen design profile. Figure 2 shows total unit weight. Figure 3 shows plasticity index. Figure 4 shows sensitivity. A summary of the parameters as they are given in the PACER input file is shown in table 1. The ground water level is assumed to be at terrain (z = 0 m).

![Figure 1: Undrained shear strength from UU tests, Onsøy](image)

Figure 1: Undrained shear strength from UU tests, Onsøy
Figure 2: Total unit weight, Onsøy

Figure 3: Plasticity index, Onsøy
Table 1: Input soil parameters for PACER, Onsøy

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>z_{bottom} [m]</th>
<th>\gamma_{tot} [kN/m^3]</th>
<th>c_{u, top} [kPa]</th>
<th>c_{u, bottom} [kPa]</th>
<th>I_p [%]</th>
<th>S_t [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>17.0</td>
<td>17.5</td>
<td>17.5</td>
<td>35</td>
<td>9</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>16.0</td>
<td>17.5</td>
<td>17.5</td>
<td>38</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>16.0</td>
<td>15.0</td>
<td>15.0</td>
<td>40</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>11</td>
<td>16.6</td>
<td>15.0</td>
<td>19.2</td>
<td>30</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>15</td>
<td>17.3</td>
<td>19.2</td>
<td>27.5</td>
<td>27</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>16</td>
<td>16.0</td>
<td>27.5</td>
<td>28.5</td>
<td>33</td>
<td>11</td>
</tr>
<tr>
<td>7</td>
<td>20</td>
<td>16.2</td>
<td>28.5</td>
<td>32.5</td>
<td>30</td>
<td>25</td>
</tr>
</tbody>
</table>

2.1.2 Testsite Stjørdal

The soil parameters chosen for input to the PACER calculations are based on laboratory results presented by NGI (2011b). Figure 1 shows the results from triaxial UU-tests with the chosen design profile. Figure 5 shows the results from triaxial UU-tests with the chosen design profile. Figure 6 shows total unit weight.

Figure 7 shows plasticity index. Figure 8 shows sensitivity. A summary of the parameters as they are given in the PACER input file is shown in table 2. The ground water level is assumed to be at terrain (z = 0 m).
Figure 5: Undrained shear strength from UU tests, Stjørdal

Figure 6: Total unit weight, Sjørdal
Figure 7: Plasticity index, Stjørdal

Figure 8: Sensitivity, Stjørdal

Table 2: Input soil parameters for PACER, Stjørdal

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>z_{bottom} [m]</th>
<th>\gamma_{tot} [kN/m^3]</th>
<th>c_{u,top} [kPa]</th>
<th>c_{u,bottom} [kPa]</th>
<th>I_p [%]</th>
<th>S_t [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>19.5</td>
<td>20.8</td>
<td>25.0</td>
<td>12</td>
<td>7</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>19.5</td>
<td>25.0</td>
<td>33.3</td>
<td>13</td>
<td>7</td>
</tr>
<tr>
<td>3</td>
<td>20</td>
<td>19.5</td>
<td>33.3</td>
<td>37.5</td>
<td>13</td>
<td>6</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>19.5</td>
<td>32.0</td>
<td>32.0</td>
<td>14</td>
<td>6</td>
</tr>
</tbody>
</table>
2.1.3 Testsite Cowden

The soil parameters chosen for input to the PACER calculations are based on field and laboratory results presented by Powell and Butcher (2003). Figure 9 shows the calculated $c_u$ from CPT-tests with the chosen design profile. Figure 10 shows total unit weight. Figure 11 shows plasticity index. The sensitivity is low and is assumed approximately 2. A summary of the parameters as they are given in the PACER input file is shown in table 3. The ground water level is assumed to be at $z = 1$ m.

![Figure 9: Undrained shear strength from CPT, Cowden](image)

![Figure 10: Total unit weight, Cowden](image)
2.1.4 Testsite Femern

The soil parameters chosen for input to the PACER calculations are based on laboratory results presented by Femern A/S (2012). Figure 12 shows the measured $c_u^{DSS}$ from DSS-tests with the chosen $c_u^{D}$ design profile. Figure 13 shows total unit weight. Figure 14 shows plasticity index, where the measured values are an average for the deposits in the area. The sensitivity is low and is assumed approximately $1.5$. A summary of the parameters as they are given in the PACER input file is shown in table 4. The ground water level is 7-8 m above the piles, and for calculations the ground water depth is set to $z = 0$ m.
Figure 12: Undrained shear strength, Femern

Figure 13: Total unit weight, Femern
Table 4: Input soil parameters for PACER, Femern

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>z_{bottom} [m]</th>
<th>\gamma_{tot} [kN/m^3]</th>
<th>c_{u,top} [kPa]</th>
<th>c_{u,bottom} [kPa]</th>
<th>I_p [%]</th>
<th>S_t [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>18.0</td>
<td>16.7</td>
<td>50</td>
<td>50</td>
<td>1.5</td>
</tr>
<tr>
<td>2</td>
<td>10</td>
<td>18.5</td>
<td>50</td>
<td>75</td>
<td>90</td>
<td>1.5</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>18.5</td>
<td>75</td>
<td>100</td>
<td>105</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>25</td>
<td>18.7</td>
<td>100</td>
<td>150</td>
<td>110</td>
<td>1.5</td>
</tr>
</tbody>
</table>

2.2 Sand sites

2.2.1 Testsite Ryggkollen

The soil parameters chosen for input to the PACER calculations are based on field and laboratory results presented by NGI (2012). Figure 15 shows CPT cone resistance. Figure 16 shows relative density D_r calculated from cone resistance based on Baldi et al. (1986). In the NGI-05 method, D_r is set to 0 and then calculated by the program after equations given by NGI (2011c).

A summary of the parameters as they are given in the PACER input file is shown in table 5. Parameters which are 0 are calculated by the program after equations given by NGI (2011c). The program will choose a default value for D_{50} when not given. The ground water depth is set to z = 9.7 m.
Figure 15: CPT cone resistance with design profile, Ryggkollen

Figure 16: D_e from CPT test
Table 5: Input soil parameters for PACER, Ryggkollen

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>$z_{\text{bottom}}$ [m]</th>
<th>$\gamma_{\text{tot}}$ [kN/m$^3$]</th>
<th>CPT $q_c$, top [MPa]</th>
<th>CPT $q_c$, bottom [MPa]</th>
<th>$\varphi$ [$^\circ$]</th>
<th>$D_r$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9.7</td>
<td>20.0</td>
<td>30</td>
<td>30</td>
<td>0</td>
<td>0.95</td>
</tr>
<tr>
<td>2</td>
<td>12.0</td>
<td>20.0</td>
<td>25</td>
<td>25</td>
<td>0</td>
<td>0.8</td>
</tr>
<tr>
<td>3</td>
<td>25.0</td>
<td>20.0</td>
<td>20</td>
<td>20</td>
<td>0</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Layer no. | $D_{50}$ [mm] | Fines [%] | API $\tau_{\text{lim}}$ [kPa] | API Tip limit [MPa] | ISO $\tau_{\text{lim}}$ [kPa] | ISO Tip limit [MPa] |
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0</td>
<td>10</td>
<td>114.8</td>
<td>12.0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.0</td>
<td>5</td>
<td>95.7</td>
<td>9.6</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0.0</td>
<td>5</td>
<td>95.7</td>
<td>9.6</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

2.2.2 Testsite Larvik

The soil parameters chosen for input to the PACER calculations are based on field and laboratory results presented by NGI (2009). Figure 17 shows CPT cone resistance from sounding no. 2, which all interpretation and calculation is based on.

Figure 18 shows relative density $D_r$ calculated from cone resistance for normally consolidated sand based on Baldi et al. (1986). Figure 19 shows $D_r$ from cone resistance for sand corrected for fines content of 25 %. The design values are chosen from both these figures, depending on layer grain size distributions. $D_{50}$ is chosen based on sieving tests presented in the factual report (NGI, 2009). For the NGI-05 method, $D_r$ is set to 0 and is then calculated by the program after equations given by NGI (2011c).

A summary of the parameters as they are given in the PACER input file is shown in table 5. Values set to 0 are calculated by the program after equations given by NGI (2011c). The ground water depth is set to $z = 9.7$ m.
Figure 17: CPT cone resistance with design profile, Larvik

Figure 18: Dr from cone resistance in NC sand, Larvik
Figure 19: Dr from cone resistance, sand corrected for 25 % fines content, Larvik

Table 6: Input soil parameters for PACER, Larvik

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>$z_{\text{bottom}}$ [m]</th>
<th>$\gamma_{\text{tot}}$ [kN/m$^3$]</th>
<th>CPT $q_c$, top [MPa]</th>
<th>CPT $q_c$, bottom [MPa]</th>
<th>$\phi$ [$^\circ$]</th>
<th>$D_r$ [-]</th>
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</thead>
<tbody>
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<td>2</td>
<td>2</td>
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</tr>
<tr>
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<td>2</td>
<td>2</td>
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</tr>
<tr>
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<td>4</td>
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<td>0.20*</td>
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<tr>
<td>5</td>
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<td>19.5</td>
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<td>4</td>
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<td>0.20*</td>
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<tr>
<td>6</td>
<td>17.2</td>
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<td>7</td>
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<td>19.0</td>
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<td>2</td>
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<td>9</td>
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<td>8</td>
<td>8</td>
<td>0</td>
<td>0.40</td>
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</table>
Cont. Table 7: Input soil parameters for PACER, Larvik

<table>
<thead>
<tr>
<th>Layer no.</th>
<th>$D_{50}$ [mm]</th>
<th>Fines [%]</th>
<th>API $\tau_{\text{lim}}$ [kPa]</th>
<th>API Tip $\tau_{\text{lim}}$ [kPa]</th>
<th>ISO $\tau_{\text{lim}}$ [kPa]</th>
<th>ISO Tip $\tau_{\text{lim}}$ [MPa]</th>
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</thead>
<tbody>
<tr>
<td>1</td>
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<tr>
<td>2</td>
<td>0.2</td>
<td>10</td>
<td>67</td>
<td>2.9</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
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<td>0.4</td>
<td>10</td>
<td>81.3</td>
<td>4.8</td>
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<td>81.3</td>
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<td>0.3</td>
<td>2</td>
<td>81.3</td>
<td>4.8</td>
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</tr>
</tbody>
</table>

*) ISO-07 method is not applicable to loose sands ($D_r < 35\%$)

3 Pile parameters

Table 8: Input pile parameters for PACER

<table>
<thead>
<tr>
<th>Site</th>
<th>Onsøy</th>
<th>Stjørdal</th>
<th>Cowden</th>
<th>Femern</th>
<th>Ryggkollen</th>
<th>Larvik</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{\text{top}}$ [m]</td>
<td>0.508</td>
<td>0.508</td>
<td>0.457</td>
<td>0.508</td>
<td>0.406</td>
<td>0.508</td>
</tr>
<tr>
<td>$D_{\text{tip}}$ [m]</td>
<td>0.508</td>
<td>0.508</td>
<td>0.457</td>
<td>0.508</td>
<td>0.406</td>
<td>0.508</td>
</tr>
<tr>
<td>$t_{\text{wall}}$ [m]</td>
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<td>0.008</td>
<td>0.0125</td>
<td>0.0225</td>
<td>0.0125</td>
<td>0.0063</td>
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<tr>
<td>$\gamma_{\text{wall}}$ [kN/m$^3$]</td>
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<td>78.0</td>
<td>78.0</td>
<td>78.0</td>
<td>78.0</td>
<td>78.0</td>
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<tr>
<td>Pile wall mat.</td>
<td>1 (Steel)</td>
<td>1 (Steel)</td>
<td>1 (Steel)</td>
<td>1 (Steel)</td>
<td>1 (Steel)</td>
<td>1 (Steel)</td>
</tr>
<tr>
<td>$z_{\text{top}}$ [m]</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
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<tr>
<td>$z_{\text{tip}}$ [m]</td>
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<td>10.0</td>
<td>25.0</td>
<td>20.0</td>
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<tr>
<td>$z_{\text{hole}}$ [m]</td>
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<td>1.0</td>
<td>0.0</td>
<td>5.0</td>
<td>1.5</td>
</tr>
<tr>
<td>$z_{\text{plug}}$ [m]</td>
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<td>8.6</td>
<td>2.2</td>
<td>25.0</td>
<td>8.0</td>
<td>4.6</td>
</tr>
<tr>
<td>$z_{\text{water}}$ [m]</td>
<td>2.3</td>
<td>8.6</td>
<td>2.2</td>
<td>8.0</td>
<td>2.0</td>
<td>2.0</td>
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<tr>
<td>$\gamma_{\text{plug}}$ [kN/m$^3$]</td>
<td>16.5</td>
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<td>20.0</td>
<td>19.5</td>
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<tr>
<td>Pile tip cond.</td>
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<td>1 (Open)</td>
<td>1 (Open)</td>
<td>1 (Open)</td>
<td>1 (Open)</td>
<td>1 (Open)</td>
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<tr>
<td>Soil plug friction ratio</td>
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<td>0.3</td>
<td>1.0</td>
<td>1.0</td>
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<tr>
<td>Tip condition</td>
<td>1 (Undrained)</td>
<td>1 (Undrained)</td>
<td>1 (Undrained)</td>
<td>1 (Undrained)</td>
<td>2 (Drained)</td>
<td>2 (Drained)</td>
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</table>
4 Results

4.1 Clay sites

<table>
<thead>
<tr>
<th>Method</th>
<th>Skin tension capacity [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2, 1984-present</td>
<td>519</td>
</tr>
<tr>
<td>Fugro-96</td>
<td>461</td>
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<tr>
<td>ICP-96</td>
<td>251</td>
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<tr>
<td>NGI-05</td>
<td>434</td>
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</table>

Table 10: Resulting skin tension capacity, Stjørdal

<table>
<thead>
<tr>
<th>Method</th>
<th>Skin tension capacity [kN]</th>
</tr>
</thead>
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<tr>
<td>API-2, 1984-present</td>
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<tr>
<td>Fugro-96</td>
<td>811</td>
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<tr>
<td>ICP-96</td>
<td>958</td>
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<tr>
<td>NGI-05</td>
<td>498</td>
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</table>

Table 11: Resulting skin tension capacity, Cowden

<table>
<thead>
<tr>
<th>Method</th>
<th>Skin tension capacity [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>API-2, 1984-present</td>
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<td>Fugro-96</td>
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<td>ICP-96</td>
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Table 12: Resulting skin tension capacity, Femern

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<td>API-2, 1984-present</td>
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<tr>
<td>Fugro-96</td>
<td>2021</td>
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<td>ICP-96</td>
<td>1042</td>
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<td>NGI-05</td>
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</table>
4.2 Sand sites

Table 13: Resulting skin tension capacity, Ryggkollen

<table>
<thead>
<tr>
<th>Method</th>
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<td>API-2, 1984-present</td>
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<tr>
<td>ICP-05, full</td>
<td>1345</td>
</tr>
<tr>
<td>ICP-05, simplified</td>
<td>1012</td>
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<tr>
<td>FBV-05</td>
<td>790</td>
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<tr>
<td>NGI-05</td>
<td>1525</td>
</tr>
<tr>
<td>UWA-05, full</td>
<td>1312</td>
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<tr>
<td>UWA-05, simplified</td>
<td>790</td>
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<tr>
<td>ISO-07</td>
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</table>

Table 14: Resulting skin tension capacity, Larvik

<table>
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<td>ICP-05, full</td>
<td>403</td>
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<tr>
<td>ICP-05, simplified</td>
<td>243</td>
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<td>FBV-05</td>
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<td>UWA-05, full</td>
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<td>UWA-05, simplified</td>
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<td>ISO-07*</td>
<td>535*</td>
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</table>

*) ISO-07 method is not applicable to loose sands (Dr < 35 %)

5 References


Appendix D - Independent Assessments Pile tests Larvik

Prepared by C.J.F. Clausen

Contents

1 Introduction 2
2 Soil Conditions 2
3 Pile Geometry and Installation 2
4 Measured Capacities 3
5 Calculated Capacities 5
6 Pile Stiffness Analysis 6
7 Ageing Effects 7
8 Conclusions 9
9 References 9
1 Introduction

This appendix gives a summary of the site investigations and pile tests carried out at the Larvik test site. The factual results presented in the following are based upon the information given in NGI (2011a, 2012). The purpose of this review is to obtain an independent interpretation of the Larvik test results that can be compared to the preliminary conclusions presented in NGI (2012).

2 Soil Conditions

The soils at the Larvik test site are described in Section 2.6.1 of NGI (2012) as “... fine to medium and partly silty sand with occasional layers of sandy, clayey silt”. NGI (2011a) presents the detailed results of the soil investigations carried out.

Figure D2.1 shows a site plan with the position of the test piles and the boreholes. Measured CPT tip resistance values $q_c$ are summarised on Figure D2.2.

The upper 2.0 m consists of fill material. The ground water level is located 2.0 m below the surface. Pore water pressures at the site are hydrostatic. The average water content between 3 m and 22 m depth is approximately 25 %, Figure 2.6.2 of NGI (2012). With an assumed specific gravity of 2.65, this leads to a total unit weight of:

$$\gamma_{tot} = 2.65 \cdot 10 \text{ kN/m}^3 \cdot \frac{(1+0.25)}{(1+2.65 \cdot 0.25)} = 19.9 \text{ kN/m}^3 \approx 20 \text{ kN/m}^3$$

Between 15 m and 20 m depth the measured $q_c$ values range from say 2 MPa to 10 MPa. Using the NGI-05 pile calculation method, this corresponds to a range in the relative density of less than 0.1 to 0.5. The sand layers can thus be described as loose to medium dense.

3 Pile Geometry and Installation

The Larvik test piles are open ended tubular steel piles with the following geometry:

- Outside diameter: 0.508 m
- Wall thickness: 6.3 mm
- Cross section area: 0.00993 m$^2$
- Tip depth below the ground surface: 21.5 m
- Casing tip depth: 1.4 m
- Average depth to top soil plug: 4.7 m
Hydraulic hammers were used to drive the piles. Piles L1 to L6 were installed 29-30 June 2009. In 2011 a new test pile L7 was installed for the reasons explained in Section 2.6.2 of NGI (2012).

Observations of driving resistance and PDA measurements were carried out. However, these results are only partly complete, and have therefore not been considered in connection with the assessments presented herein. All piles were tested in tension.

4 Measured Capacities

Only the first time testing of each pile is considered in the following. It should however be noted that the Larvik piles all showed a reduction in measured capacity when the piles were reloaded after a rest period, see Table 2.6.4 of NGI (2012). Measured first time loading capacities are summarised below.

Table D4.1: Measured capacities, Larvik

<table>
<thead>
<tr>
<th>Pile</th>
<th>Age (days)</th>
<th>Measured failure load (kN)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
</tr>
<tr>
<td>L1</td>
<td>43</td>
<td>980</td>
<td>1040</td>
</tr>
<tr>
<td>L2</td>
<td>135</td>
<td>990</td>
<td>960</td>
</tr>
<tr>
<td>L3</td>
<td>218</td>
<td>1160</td>
<td>1110</td>
</tr>
<tr>
<td>L4</td>
<td>365</td>
<td>1065</td>
<td>1030</td>
</tr>
<tr>
<td>L5</td>
<td>730</td>
<td>1080</td>
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<td>L6</td>
<td>730</td>
<td>900</td>
<td>920</td>
</tr>
<tr>
<td>L7</td>
<td>30</td>
<td>600</td>
<td>600</td>
</tr>
</tbody>
</table>

*) A: As given in Table 2.6.4 of NGI (2012)
*) B: At 0.05·D = 25 mm pile head displacement

1) Failure load B is extrapolated from 18 mm measured displacement
2) Pile L6 carried a permanent load of 60 % of the expected failure load

It is observed that the difference between failure loads A and B in Table D4.1 is modest. Failure loads A are therefore used in the following.

The Table D4.1 capacities are the forces measured at the top of the pile. For the evaluation of the ageing effect, the outside skin friction only should be considered. The following corrections should therefore be subtracted from the measured capacities:

Total weight of pile steel
\[(0.5 \text{ m} + 21.5 \text{ m}) \cdot 0.00993 \text{ m}^2 \cdot 78 \text{ kN/m}^3 = 17 \text{ kN}\]
Total weight of soil plug
0.193 m² · (21.5 m – 4.7 m) · 20 kN/m³ = 65 kN

Total weight of water plug
0.193 m² · 2.7 m · 10 kN/m³ = 5 kN

Misc. weights
4 kN

Sum self weights
91 kN

If it is assumed that drained conditions exist at the pile tip, with hydrostatic pore water pressure and an open crack under the pile tip, then the upward acting tip force becomes:

\[ Q_{\text{tip drained}} = 0.203 \text{ m}^2 \cdot 19.5 \text{ m} \cdot 10 \text{ kN/m}^3 = 40 \text{ kN} \]

and the correction on the measured capacities is 91 kN – 40 kN = 51 kN.

If it is assumed that the soil at the pile tip is a clayey silt with an undrained shear strength of \((2 \text{ MPa} – 0.42 \text{ MPa}) / 17 = 92 \text{ kPa}\), then the upward acting tip force becomes:

\[ Q_{\text{tip undrained}} = 0.203 \text{ m}^2 \cdot (21.5 \text{ m} \cdot 20 \text{ kN/m}^3 – 9 \cdot 92 \text{ kPa}) = -81 \text{ kN} \]

and the correction on the measured capacities is 91 kN + 81 kN = 172 kN.

This leads to the following measured skin friction values.

*Table D4.2*

*Measured skin friction values after correction for self weights and pile tip force*

<table>
<thead>
<tr>
<th>Pile</th>
<th>Age (days)</th>
<th>Measured skin friction force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Pile tip in sand</td>
</tr>
<tr>
<td>L1</td>
<td>43</td>
<td>929</td>
</tr>
<tr>
<td>L2</td>
<td>135</td>
<td>939</td>
</tr>
<tr>
<td>L3</td>
<td>218</td>
<td>1109</td>
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<tr>
<td>L4</td>
<td>365</td>
<td>1014</td>
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<td>L5</td>
<td>730</td>
<td>1029</td>
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<td>L6</td>
<td>730</td>
<td>849</td>
</tr>
<tr>
<td>L7</td>
<td>30</td>
<td>549</td>
</tr>
</tbody>
</table>

These skin friction capacities are plotted against pile age on Figure D4.1.
5  Calculated Capacities

The expected capacities of the Larvik piles were calculated for each of the 5 CPTs shown on Figure D2.1. These CPT profiles were modelled with 2 m thick layers using the PACER program, NGI (2011b). An example input file is included on Figure D5.1. Calculated skin friction values in tension are given below.

Table D5.1 : Calculated skin friction in tension (kN), Larvik

<table>
<thead>
<tr>
<th>Location</th>
<th>Calculation method</th>
<th>API-07</th>
<th>ICP-05</th>
<th>NGI-05</th>
<th>UWA-05</th>
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</thead>
<tbody>
<tr>
<td>CPT-1</td>
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<td>432</td>
<td>321</td>
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<tr>
<td>CPT-2</td>
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<td>439</td>
<td>432</td>
<td>449</td>
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<tr>
<td>CPT-3</td>
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<td>516</td>
<td>459</td>
<td>536</td>
</tr>
<tr>
<td>CPT-4</td>
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<td>440</td>
<td>438</td>
<td>446</td>
</tr>
<tr>
<td>CPT-5</td>
<td></td>
<td>1273</td>
<td>350</td>
<td>432</td>
<td>354</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>1366</td>
<td>412</td>
<td>439</td>
<td>421</td>
</tr>
</tbody>
</table>

The large difference between the API-07 and the three CPT-based methods agrees with the results presented by Clausen et al (2005), included herein on Figure D5.2.

A comparison between the results in Tables D4.2 and D5.1 shows that:

- Measured skin friction, pile tip in sand 549-1109 kN
- Measured skin friction, pile tip in clay 428-988 kN
- Calculated skin friction 316-536 kN

The calculated skin friction values are thus considerably lower than the measured ones. This difference is partly due to the very low CPT tip resistance at the Larvik site, and partly caused by the ageing effect as further discussed in Section 7.

The very small variation in capacity calculated by the NGI-05 method in Table D5.1 for the different CPTs reflects that this method has a lower limit on the calculated local skin friction:

\[
\tau_{\text{skin}} = \frac{z}{z_{\text{tip}}} \cdot \sigma_{\text{atm}} \cdot F_{\text{Dr}} \cdot F_{\text{sig}} \cdot F_{\text{tip}} \cdot F_{\text{load}} \cdot F_{\text{mat}} \quad (D5.1)
\]

\[
\tau_{\text{skin}} > 0.1 \cdot \sigma'_{\text{vo}}
\]

where

\[
z = \text{Depth below ground surface}
\]
ztip = Pile tip depth

\( \sigma'_\text{vo} = \) Vertical effective stress

\( \sigma_{\text{atm}} = \) Atmospheric reference pressure, 100 kPa

\[
F_{\text{Dr}} = 2.1 \cdot (D_r - 0.1)^{1.7} \quad \text{(D5.2)}
\]

\[
F_{\text{sig}} = (\sigma'_\text{vo} / \sigma_{\text{atm}})^{0.25} \quad \text{(D5.3)}
\]

\[
F_{\text{tip}} = 1.0 \text{ for a pile driven open-ended, 1.6 for a pile driven closed-ended}
\]

\[
F_{\text{load}} = 1.0 \text{ for tensile loading, 1.3 for compressive loading}
\]

\[
F_{\text{mat}} = 1.0 \text{ for steel, 1.2 for concrete}
\]

\[
D_r = \text{Sand relative density} = 0.4 \cdot \ln \left[ \frac{q_c}{22 \cdot (\sigma'_\text{vo} \cdot \sigma_{\text{atm}})^{0.5}} \right] \quad \text{(D5.4)}
\]

When the calculated \( D_r \) value becomes smaller than 0.1, \( D_r = 0.1 \) is used, which gives zero skin friction, and a skin friction of \( 0.1 \cdot \sigma'_\text{vo} \) is then used. The NGI-05 results above thus reflect that the apparent sand density for the Larvik tests is lower than for the pile tests used to calibrate the method.

Section 7 presents the method used to correct the measured Larvik skin friction capacities in order to account for the lateral and vertical variation in sand relative density over the pile test site. This correction had to be based upon the calculated ICP-05 capacities, because of the short-comings of the NGI-05 method explained above.

6 Pile Stiffness Analysis

The data from the Larvik pile tests also include the measured load-displacement curves for the different piles. The interpretation of the pile tests should therefore also include comparisons between the observed and the calculated load-displacement curves.

Figure D6.1 shows the load-displacement curves for the first test on each of the 7 piles. Pile L2 was selected as a “typical” load-displacement curve and compared to calculated curves generated by the SPLICE program, NGI(2011c).

Two cases were analysed by SPLICE, (1) pile tip in sand, and (2) pile tip in clay with an undrained strength of 90 kPa. The analyses included self weights of pile steel and soil plug. The NGI-05 was used to calculate the skin friction for the CPT-2 profile. It should be noted that the NGI-05 capacities represent 10-day values. Since L2 has an age of 135 days, the calculated skin friction was increased by a factor \( f \) calculated for an assumed \( \Delta_{10} \) value of 0.4:

\[
f = [1.0 + 0.4 \cdot \log_{10} (135 \text{ days} / 10 \text{ days})] = 1.45 \quad \text{(D6.1)}
\]

Calculated load-displacement curves are compared to the measured one on Figure D6.2. It is observed that the calculated curves are slightly stiffer, and
have a more well-defined peak capacity, than the measured curve. It is also seen that if the assumed ageing factor of 1.45 is not applied, the initial slope of the calculated and measured curves would agree closely.

7 Ageing Effects

The main purpose of the present study is to determine the increase in pile skin friction with time, as determined by field tests on directly comparable piles. This ageing effect is expressed as:

\[
\tau(t) = \tau(t_0) \cdot \left[1.0 + \Delta_{10} \cdot \log_{10}(t/t_0)\right] \quad \text{(D7.1)}
\]

where

- \(\tau(t)\) = Local unit skin friction at time \(t\) (kPa)
- \(t\) = Time since pile driving (days)
- \(t_0\) = Reference time, taken as 10 days for piles in sand
- \(\Delta_{10}\) = Ageing factor back figured from pile tests

For identical piles in the same soil, loaded to first failure at different times \(t\), \(\Delta_{10}\) can be directly determined from plots of the type shown on Figure D4.1.

However, as clearly demonstrated by the CPT results (see Table D5.1), the sand relative density, and thus the pile skin friction, varies over the Larvik test site. On the Figure D7.1 site plan, the location of the 7 test piles and the 5 CPTs are included, together with the ICP-05 skin friction values from Table D5.1. It is observed that the calculated skin friction is lowest in north-west and increases towards south-east.

The information on Figure D7.1 can be used to calculate a weighted ICP-05 skin friction at each test pile position. It is assumed that for a given pile position, the weight given to the capacity found at each CPT shall be inversely proportional to the squared distance from the pile to the CPT:

\[
\text{Weight} = \left(\frac{10 \text{ m}}{\text{Distance}}\right)^2 \quad \text{(D7.2)}
\]

This leads to the weight factors and the calculated skin friction values shown below.
Table E7.1: Calculated weight factors

<table>
<thead>
<tr>
<th>Boring</th>
<th>Pile test number</th>
<th>ICP-05 (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L1</td>
<td>L2</td>
</tr>
<tr>
<td>CPT-1</td>
<td>0.21</td>
<td>0.31</td>
</tr>
<tr>
<td>CPT-2</td>
<td>1.56</td>
<td>4.00</td>
</tr>
<tr>
<td>CPT-3</td>
<td>2.04</td>
<td>0.83</td>
</tr>
<tr>
<td>CPT-4</td>
<td>0.19</td>
<td>0.28</td>
</tr>
<tr>
<td>CPT-5</td>
<td>0.28</td>
<td>0.28</td>
</tr>
<tr>
<td>Sum</td>
<td>4.28</td>
<td>5.70</td>
</tr>
</tbody>
</table>

As an example, the calculated ICP-05 skin friction for pile L1 then becomes:

\[ Q_{\text{skin } L1} = \frac{(0.21 \cdot 316 \text{ kN} + 1.56 \cdot 439 \text{ kN} + \ldots + 0.28 \cdot 350 \text{ kN})}{4.28} = 464 \text{ kN} \]

This leads to the skin friction correction factors and the corrected measured capacities given below.

Table E7.2: Skin friction correction factors and corrected measured skin friction

<table>
<thead>
<tr>
<th>Pile</th>
<th>L1</th>
<th>L2</th>
<th>L3</th>
<th>L4</th>
<th>L5</th>
<th>L6</th>
<th>L7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Age (days)</td>
<td>43</td>
<td>135</td>
<td>218</td>
<td>365</td>
<td>730</td>
<td>730</td>
<td>30</td>
</tr>
<tr>
<td>ICP-05 skin friction (kN)</td>
<td>464</td>
<td>439</td>
<td>427</td>
<td>412</td>
<td>411</td>
<td>417</td>
<td>346</td>
</tr>
<tr>
<td>Average value (kN)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>417</td>
</tr>
<tr>
<td>Factor</td>
<td>0.90</td>
<td>0.95</td>
<td>0.98</td>
<td>1.01</td>
<td>1.01</td>
<td>1.00</td>
<td>1.21</td>
</tr>
</tbody>
</table>

Pile tip assumed in sand

| Measured (kN) | 929  | 939  | 1109 | 1014 | 1029 | 849  | 549  |
| Corrected (kN) | 836  | 892  | 1087 | 1024 | 1039 | 849  | 664  |

Pile tip assumed in clay

| Measured (kN) | 808  | 818  | 988  | 893  | 908  | 728  | 428  |
| Corrected (kN) | 727  | 777  | 968  | 902  | 917  | 728  | 518  |

These capacities are plotted against pile age on Figure D7.2. A visual best fit to the test results leads to the following \( \Delta_{10} \) values:

Pile tip in sand \( 0.54 \)
Pile tip in clay \( 0.67 \)
8 Conclusions

Based upon the results presented above, the following conclusions are drawn:

**Soil Conditions at the Larvik test Site**

The soils consist of fine to medium partly silty sand with occasional layers of sandy and clayey silt. The sand layers are loose to medium dense. Sand relative density varies over the site. Calculated pile skin friction values, based upon the CPT results, increase by more than 60% from the lowest value in the north-west to the highest value in the south-east.

**Measured and Calculated Pile Capacities**

The directly measured pile capacities range from 600 kN to 1160 kN. When corrections are included for pile and soil plug weights, pile tip force and lateral variations in sand density, then the range in measured skin friction is 518 kN to 1087 kN. Extrapolation back to the reference time $t_0 = 10$ days leads to a measured range of 450 kN to 560 kN.

Using the CPT-based calculation methods ICP-05, NGI-05 and UWA-05, the average calculated skin friction in tension varies from 412 kN to 439 kN. The measured values are thus 10-30% higher than the calculated ones.

**Ageing Factor $\Delta_10$**

The average ageing factor $\Delta_10$ back-figured from the Larvik tests ranges from 0.54 to 0.67. The lowest value assumes that the pile tip is located in a freely draining sand, while the highest value is found for the pile tip in clay.

9 References

Clausen C.J.F., P.M. Aas & K. Karlsrud (2005)  
"Bearing Capacity of Driven Piles in Sand, the NGI Approach".  
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“Time Effects on Pile Capacity, Factual Report, Test Site Larvik”.  
Document 20061251-00-244-R presented to Joint Industry Project,  
Revision 01, 10 January 2011.

Norwegian Geotechnical Institute (2011b)  
“PACER – A Computer Program for Calculation of Axial Capacity of Driven Piles,
Program Documentation”. Report 20061125-1, revision 2, 1 November 2011.

Norwegian Geotechnical Institute (2011c)

Norwegian Geotechnical Institute (2012)
North

Figure D2.1

Larvik pile test site with position of piles and boreholes, 50 m grid
Figure D2.2 : Summary of Larvik CPT results
Figure D4.1

Measured skin friction values after corrections for self weights and tip force
NGI 12.0626 : Pile test Larvik CPT-1 30 Aug 2012

* Date  Sign  Log of file modifications
* 30 Aug 2012 cjfc  Original version, units used are : kN, MN & m

*********** CONTROL SECTION

** SOIL PROPERTIES SECTION **

<table>
<thead>
<tr>
<th>Layer</th>
<th>Z.bot (m)</th>
<th>Gam.tot (kN/m²)</th>
<th>Gam.pwp (kN/m²)</th>
<th>Ko' (kPa)</th>
<th>SigmaPC (kPa)</th>
<th>Top S1</th>
<th>Bot S2</th>
<th>Soil Type</th>
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<tr>
<td>1</td>
<td>2.0</td>
<td>20.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>Sand</td>
</tr>
<tr>
<td>2</td>
<td>4.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>2.0</td>
<td>2</td>
<td>Sand</td>
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<td>3</td>
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<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>2.0</td>
<td>2</td>
<td>Sand</td>
</tr>
<tr>
<td>4</td>
<td>8.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>1.0</td>
<td>2.5</td>
<td>Sand</td>
</tr>
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<td>5</td>
<td>10.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>1.8</td>
<td>2.3</td>
<td>Sand</td>
</tr>
<tr>
<td>6</td>
<td>12.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
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<td>3.2</td>
<td>Sand</td>
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<td>0.0</td>
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<td>1.8</td>
<td>2.5</td>
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<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>2.0</td>
<td>2.6</td>
<td>Sand</td>
</tr>
<tr>
<td>10</td>
<td>20.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>2.6</td>
<td>3.8</td>
<td>Sand</td>
</tr>
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<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>4.2</td>
<td>4.4</td>
<td>Sand</td>
</tr>
<tr>
<td>12</td>
<td>24.0</td>
<td>20.0</td>
<td>10.0</td>
<td>0.0</td>
<td>0.0</td>
<td>4.4</td>
<td>4.4</td>
<td>Sand</td>
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</table>

** Miscellaneous **

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<tr>
<th>Layer</th>
<th>D50 (mm)</th>
<th>Phi (°)</th>
<th>Dr (0-1)</th>
<th>D60or API - K.fact</th>
<th>Tau.lim (kPa)</th>
<th>Tip.lim (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2</td>
<td>0.60</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>S1 S2 S3 S4</td>
</tr>
<tr>
<td>2</td>
<td>0.4</td>
<td>0.00</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>S1 S2 S3 S4</td>
</tr>
<tr>
<td>3</td>
<td>0.6</td>
<td>0.00</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>S1 S2 S3 S4</td>
</tr>
<tr>
<td>4</td>
<td>0.8</td>
<td>0.00</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
<td>S1 S2 S3 S4</td>
</tr>
</tbody>
</table>

** D50 (mm) may be used by ICP/UWA, Fines (%) is needed by the ISO-2007 method **

\x\fil1/prodata\2006\12\20061251\leveransedokumenter\rapport\evaluation report\final\rev.1\20061251-00-279-r-appendix.d.docx
* Tau.lim and Tip.lim for sand layers only apply to the API-2 method
* Present use of SAND miscellaneous values: S1-S4 used for CaCO3 soils
  * S1 = CaCO3 cont. %
  * S2 = Degree of cementation, 0-1
  * S3 = K*tan(delta)
  * S4 = Tau.lim in API-2 method
  * S5 = Factor on calculated skin friction in all sand layers

*************** PILE PROPERTIES SECTION
**************************************************
<table>
<thead>
<tr>
<th></th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.508</td>
<td>Outer diameter at pile top (m)</td>
</tr>
<tr>
<td>0.508</td>
<td>Outer diameter at pile tip (m)</td>
</tr>
<tr>
<td>.0063</td>
<td>Pile wall thickness (m)</td>
</tr>
<tr>
<td>78</td>
<td>Total unit weight of pile wall material (kN nB)</td>
</tr>
<tr>
<td>1</td>
<td>Pile wall material (1=Steel 2=Concrete 3=Timber)</td>
</tr>
<tr>
<td>0.0</td>
<td>Z-coordinate at pile top (m)</td>
</tr>
<tr>
<td>21.5</td>
<td>Z-coordinate at pile tip (m)</td>
</tr>
<tr>
<td>1.4</td>
<td>Z-coordinate of bottom of pre-drilled hole or casing (m)</td>
</tr>
<tr>
<td>4.7</td>
<td>Z-coordinate of top soil plug (m)</td>
</tr>
<tr>
<td>4.7</td>
<td>Z-coordinate of water surface inside the pile (m)</td>
</tr>
<tr>
<td>20.0</td>
<td>Total unit weight of soil plug (kN nB)</td>
</tr>
<tr>
<td>1</td>
<td>Code for pile tip condition during driving (1=Open 2=Closed)</td>
</tr>
<tr>
<td>1</td>
<td>Soil plug friction ratio, Tau.inside = Tau.outside*PLGRAT</td>
</tr>
<tr>
<td>1</td>
<td>Tip condition (1=Undrained 2=Drained 3=Stress free)</td>
</tr>
</tbody>
</table>

* Miscellaneous pile values (1-10)
  * P1 P2 P3 P4 P5 P6 P7 P8 P9 P10
  * Present use of PILE miscellaneous values: None

End of file c:\cjfc\pacer\data\larvik-CPT1.txt

---

**Figure D5.1**

Example input file to program PACER, CPT-1
Figure D5.2

Capacity of tension piles in sand, from Clausen et al (2005)
Figure D6.1

Larvik, measured load-displacement curves for the first testing of each pile, from NGI (2012)
Figure D6.2

Comparison between measured and calculated load-displacement curves
Figure D7.1

Site plan with position of piles and CPTs
Figure D7.2

Measured skin friction. Includes corrections for self weights, tip force and variation in sand density over the test site.
Appendix E - Independent Assessments Pile tests Ryggkollen

Prepared by C.J.F. Clausen

Contents

1 Introduction 2
2 Soil Conditions 2
3 Pile Geometry and Installation 2
4 Measured Capacities 3
5 Calculated Capacities 4
6 Pile Stiffness Analysis 6
7 Pile SRD Values 6
8 Ageing Effects 7
9 Discussion 8
10 Conclusions 9
11 References 10
1 Introduction

This appendix gives a summary of the site investigations and pile tests carried out at the Ryggkollen test site. The factual results presented in the following are based upon the information given in NGI (2012a,b,c). The purpose of this review is to obtain an independent interpretation of the Ryggkollen test results that can be compared to the preliminary conclusions presented in NGI (2012a).

2 Soil Conditions

The soils at the Ryggkollen test site are described as “medium to coarse gravelly sand with occasional inclusions of gravel and stones/cobbles”. This deposit is part of an end moraine formed around 9000 years ago. NGI (2012c) presents the detailed results of the soil investigations carried out.

Figure E2.1 shows a site plan with the position on the test piles and the boreholes. Only one CPT could be carried out at the site, hole NGI-13, located near pile 1. Results from this CPT are shown on Figure E2.2.

The upper 4.5-5.5 m consists of filled material. The ground water level is located at 9.7 m below the ground surface. Hydrostatic pore water pressures are assumed.

Between 12 m and 17 m depth the measured qe value was approximately 20 MPa. Using the NGI-05 pile calculation method, Clausen et al (2005), this corresponds to a relative density of 0.7. The gravelly sand can thus be described as dense.

3 Pile Geometry and Installation

The Ryggkollen test piles are open ended tubular steel piles with the following geometry:

- Outside diameter: 0.4064 m
- Wall thickness: 12.5 mm
- Cross section area: 0.0155 m²
- Tip depth below the ground surface: 20.0 m
- Casing tip depth: 4.4-5.3 m
- Average depth to top soil plug, piles 1-4: 7.9 m
- Depth to soil plug, pile 5: 15.2 m
- Depth to soil plug, pile 6: 9.9 m

Pile driving was carried out with the Junttan PM20 piling rig and a hydraulic hammer with a ram weight of 50 kN. The piles were installed during the period 10-12 August 2009. Observed driving resistance and hammer ram drop heights are plotted on Figures 3.1 to 3.6. It is seen that pile 5 had considerably higher driving resistance than the other piles. In the pile driving log it is stated that “something strong met at approximately 8 m. 1.4 m plug after driving the first section”.

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It thus seems that pile 5 plugged at an early stage. Results from this pile may therefore not be directly comparable to the other piles.

PDA measurements were carried out for piles 4 and 5. However, these results have not been considered in connection with the assessments presented herein. All piles were tested in tension.

4 Measured Capacities

Only the first time testing of each pile is considered in the following. It should however be noted that the Ryggkollen piles all showed a reduction in measured capacity when the piles were reloaded after a rest period, see Table 2.7.1 of NGI (2012a). Measured first time loading capacities are summarised below.

Table F4.1: Measured capacities, Ryggkollen

<table>
<thead>
<tr>
<th>Pile</th>
<th>Age (days)</th>
<th>Measured failure load (kN)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>80</td>
<td>850</td>
<td></td>
</tr>
<tr>
<td>R2</td>
<td>157</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>R3</td>
<td>246</td>
<td>1600</td>
<td></td>
</tr>
<tr>
<td>R4</td>
<td>394</td>
<td>1910</td>
<td></td>
</tr>
<tr>
<td>R5</td>
<td>897</td>
<td>2100</td>
<td></td>
</tr>
<tr>
<td>R6</td>
<td>897</td>
<td>1140</td>
<td>1)</td>
</tr>
</tbody>
</table>

1) Pile R6 carried a permanent load of 60 % of the expected failure load

The Table E4.1 capacities are the forces measured at the top of the pile. For the evaluation of the ageing effect, the outside skin friction only should be considered. The following corrections shall therefore be subtracted from the measured capacities:

- Total weight of pile steel:
  \[ 21.0 \text{ m} \cdot 0.0155 \text{ m}^2 \cdot 78 \text{ kN/m}^3 = 25 \text{ kN} \]  
  
- Total weight of soil plug:
  \[ 0.114 \text{ m}^2 \cdot (20.0 \text{ m} - 7.9 \text{ m}) \cdot 19 \text{ kN/m}^3 = 26 \text{ kN} \]

- Sum self weights: 51 kN

It is assumed that drained conditions exist at the pile tip, with hydrostatic pore water pressure and an open crack under the pile tip. The upward acting tip force then becomes:
Q_{tip} = 0.130 \text{m}^2 \cdot 10.3 \text{ m} \cdot 10 \text{kN/m}^3 = 13 \text{kN} \quad (F4.3)

and the correction on the measured capacities is 51 \text{kN} – 13 \text{kN} \approx 40 \text{kN}. This leads to the following measured skin friction values.

Table F4.2: Measured skin friction values corrected for self weights and pile buoyancy

<table>
<thead>
<tr>
<th>Pile</th>
<th>Age (days)</th>
<th>Measured skin friction force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R1</td>
<td>80</td>
<td>810</td>
</tr>
<tr>
<td>R2</td>
<td>157</td>
<td>960</td>
</tr>
<tr>
<td>R3</td>
<td>246</td>
<td>1560</td>
</tr>
<tr>
<td>R4</td>
<td>394</td>
<td>1870</td>
</tr>
<tr>
<td>R5</td>
<td>897</td>
<td>2060</td>
</tr>
<tr>
<td>R6</td>
<td>897</td>
<td>1100</td>
</tr>
</tbody>
</table>

These skin friction capacities are plotted against pile age on Figure E4.1. It is observed that if the measured values are extrapolated back to an age of say 30 days, a skin friction of zero is found. This matter is discussed in the following.

5 Calculated Capacities

The expected capacity of the Ryggkollen piles was calculated by different methods. Table E5.1 shows the soil parameters assumed.

Table E5.1: Soil parameters used for the Ryggkollen piles

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Depth to layer bottom (m)</th>
<th>Soil type</th>
<th>(\gamma_{total}) (kN/m(^3))</th>
<th>CPT (q_c) (MPa)</th>
<th>Clay undrained strength (kPa)</th>
<th>Plasticity (%)</th>
<th>Notes</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>4.9</td>
<td>Sand</td>
<td>18</td>
<td>0</td>
<td>30</td>
<td>0</td>
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<tr>
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<td>9.7</td>
<td>Sand</td>
<td>18</td>
<td>30</td>
<td>30</td>
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<tr>
<td>3</td>
<td>12.0</td>
<td>Sand</td>
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<td>25</td>
<td>25</td>
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<td>25</td>
</tr>
<tr>
<td>4</td>
<td>21.0</td>
<td>Sand</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>20</td>
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</tbody>
</table>
Input data to the PACER program used for the calculations, NGI (2011a), is included on Figure E5.1. Calculated values for pile skin friction in tension are summarised below.

**Table E5.2 : Calculated pile skin friction in tension**

<table>
<thead>
<tr>
<th>Method</th>
<th>Tensile skin friction (kN)</th>
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<tr>
<td>API-07</td>
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<tr>
<td>Fugro-05</td>
<td>782</td>
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<tr>
<td>ICP-05</td>
<td>1327</td>
</tr>
<tr>
<td>ICP-05 simpl.</td>
<td>1004</td>
</tr>
<tr>
<td>ISO-07</td>
<td>1643</td>
</tr>
<tr>
<td>NGI-05</td>
<td>1560</td>
</tr>
<tr>
<td>UWA-05</td>
<td>1308</td>
</tr>
<tr>
<td>UWA-05 simpl.</td>
<td>807</td>
</tr>
</tbody>
</table>

There is a factor of 2.3 between the lowest (Fugro-05) and the highest (API-07) values. Taking the measured capacity as the average of piles R1 and R2, leads to a value of 885 kN, valid for an age of 120 days. This corresponds to an average unit skin friction of 46 kPa. On Figure E5.2 that value has been compared to key tensile pile tests which formed the basis for the calibration of the NGI-05 calculation method.

It is observed that the Ryggkollen result plots well below the other data points. It is also noted that the other data points from open-ended steel piles all are from sand deposits with a higher relative density than found at Ryggkollen. Finally, there is the still unresolved matter of the effect of pile diameter upon the unit skin friction in sand, see Figure E5.3.

In conclusion, the poor performance of the CPT-based methods ICP-05, NGI-05 and UWA-05 for the Ryggkollen piles is probably due to one or more of the following causes:

* The gravel content of the sand, as discussed in Section 9.
* These 3 methods all over-estimate the capacity of open-ended steel piles in medium dense and dense sand.
* The effect of pile diameter upon unit skin friction in sand.
* For most of the pile tests used to calibrate the CPT-based methods, the time since pile driving is not known, or not accounted for.

However, even if the predicted axial tensile capacities are much higher than measured, the measured relative increase in pile capacity as a result of ageing would still be valid, as long as the ground conditions are reasonably uniform over the pile test area.
6 Pile Stiffness Analysis

The data from the Ryggkollen pile tests also include the measured load-displacement curves for the different piles. The interpretation of the pile tests should therefore also include comparisons between the observed and the calculated load-displacement curves.

Figure E6.1 shows the load-displacement curves for the first test on each of piles R1, R2 and R3. These 3 measured load-displacement curves are compared to “predictions” calculated by program SPLICE, NGI (2011b), for assumed ultimate skin friction values of 800 kN and 1200 kN. It should be noted that the upper part of the R1-1 curve was measured with a high rate of displacement. The failure load for this pile was selected as 850 kN, see Section 2.7.2 of NGI (2012a) for further details.

It is observed that the initial stiffness of the calculated and observed load-displacement curves are in reasonable agreement. The calculated curves reach their theoretical capacity after a displacement of 6-8 mm (1.5-2 % of the pile diameter), while the observed curves need displacements of around 20 mm before the peak is reached.

7 Pile SRD Values

The measured pile driving resistance, see Figures E3.1-F3.6, shows that the ground conditions are not uniform over the Ryggkollen site. It is then difficult to directly compare the test results from the individual piles. At test site Larvik, this difficulty could be partly overcome by using the CPT tests to establish correction factors on the measured capacities.

At the Ryggkollen site, only one incomplete CPT boring could be carried out, as a result of the gravel and cobble content of the sand. For the present study, it was therefore decided to established a set of correction factors based upon the observed driving resistance. The approach taken was to calculate the pile static resistance at time of driving (SRD) from the pile driving data using a simple pile driving formula.

The hammer/pile/soil system considered is sketched on Figure E7.1. When the ram of weight \( W \) falls a distance \( H \), the energy that enters the pile causes an elastic displacement \( \delta_E \) and a plastic permanent set \( \delta_P \). This simple system is governed by the following equations:

\[
\begin{align*}
k_{\text{pile}} &= \frac{EA}{(L + ZQ - Z_{\text{tip}})} \quad \text{(E7.1)} \\
k_{\text{soil}} &= \frac{Q_{\text{MAX}}}{(f \cdot \text{DIAM})} \quad \text{(E7.2)} \\
k_r &= k_{\text{pile}} \cdot k_{\text{soil}} / (k_{\text{pile}} + k_{\text{soil}}) \quad \text{(E7.3)} \\
\text{Energy fed into pile} &= \eta \cdot W \cdot H \quad \text{(E7.4)}
\end{align*}
\]
Energy absorbed by pile and soil = \( Q_{\text{MAX}} \cdot (0.5 \cdot \delta_E + \delta_P) \)  
\( \delta_E = Q_{\text{MAX}} / k_r \)  
\( \delta_P = 1.0 \text{ m} / (\text{measured blows/m}) \)

For given values of \( ZQ/ZTIP \), \( f \) and \( \delta_P \), \( Q_{\text{MAX}} \) can be calculated from the above equations:

\[
Q_{\text{MAX}} = \{ (k_r \cdot \delta_P)^2 + 2 \cdot \eta \cdot W \cdot H \cdot k_r \}^{0.5} - k_r \cdot \delta_P
\]

Since \( Q_{\text{MAX}} \) enters into the expression for \( k_{\text{soil}} \), an iterative solution must be used.

SRD values (= \( Q_{\text{MAX}} \)) calculated by the above approach are plotted on Figure E7.2. These calculations were carried out with \( ZQ/ZTIP = 0.8 \), \( f = 0.01 \) and a hammer efficiency \( \eta \) of 0.9. NGI (2012b) includes results from PDA analyses carried out for the R4 and R5 piles. This work concluded that “Bearing capacity at End Of Drive (EOD) at the Ryggkollen site was about 1300 kN to 1400 kN”. The simple approach above gave capacities of 1100 kN to 1700 kN for the same two piles.

Correction factors on the measured skin friction values can therefore be calculated from the SRD values plotted on Figure E7.2. For each pile, the SRD value is taken as the average of the 3 deepest points. The correction factor is taken as this SRD value divided by the average SRD value.

Table E7.1 : SRD-based correction factors and corrected pile capacities

<table>
<thead>
<tr>
<th>Pile</th>
<th>SRD (kN)</th>
<th>Correction factor</th>
<th>Corrected capacity (kN)</th>
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<tbody>
<tr>
<td>R1</td>
<td>970</td>
<td>1.18</td>
<td>956</td>
</tr>
<tr>
<td>R2</td>
<td>960</td>
<td>1.19</td>
<td>1142</td>
</tr>
<tr>
<td>R3</td>
<td>1400</td>
<td>0.82</td>
<td>1279</td>
</tr>
<tr>
<td>R4</td>
<td>1070</td>
<td>1.07</td>
<td>2001</td>
</tr>
<tr>
<td>R5</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>R6</td>
<td>1327</td>
<td>0.86</td>
<td>946</td>
</tr>
<tr>
<td>Average</td>
<td>1145</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

8 Ageing Effects

The main purpose of the present study is to determine the increase in pile skin friction with time, as determined by field tests on directly comparable piles. This ageing effect is expressed as:

\[
\tau(t) = \tau(t_0) \cdot [1.0 + \Delta_{10} \cdot \log_{10}(t/t_0)]
\]  
where
\[ \tau(t) = \text{Local unit skin friction at time } t \text{ (kPa)} \]
\[ t = \text{Time since pile driving (days)} \]
\[ t_0 = \text{Reference time, taken as 10 days for piles in sand} \]
\[ \Delta_{10} = \text{Ageing factor back figured from pile tests} \]

For identical piles in the same soil, loaded to first failure at different times \( t \), \( \Delta_{10} \) can be directly determined from plots of the type shown on Figure E4.1. However, local variations in the ground conditions could make the direct comparison between individual pile test results invalid. It is therefore proposed to correct the measured capacities as described in Section 7.

Figure 8.1 shows the measured pile capacities, with the SRD-based corrections included, plotted against pile age. The line through piles R1, R2 and R3 has an ageing factor \( \Delta_{10} = 1.8 \) for \( t_0 = 10 \) days. This \( \Delta_{10} \) value is believed to be far too high, which probably demonstrates that the SRD-based attempt to correct for local variations in the ground does not work at Ryggkollen.

9 Discussion

The capacities measured at the Ryggkollen piles are surprising because:

* The skin friction measured at piles R1 and R2 is much smaller than “predicted” by the widely used CPT-based calculation methods. The API-07 and ISO-07 methods lead to even higher calculated capacities.

* The ageing effect indicated by the pile tests is much higher than the expected value.

Low Tensile Capacity

It is likely that the low measured capacities in tension are linked to the content of gravel and cobbles in the sand. It may be argued that the coarse fraction will lead to a strengthening of the soil deposit and an over-estimate of the relative density of the sand based upon the CPT results. When an open-ended pile is driven through such a deposit, pushing and rotation of the gravel and the cobbles at the tip of the wall would cause dilation and formation of local voids near the pile tip. As the pile is driven deeper, the sand near the outside wall may end up in a looser state than initially.

A literature search for papers on capacity of driven piles in gravel did however not give any useful information.

\textbf{HOLD} : Have the piles been pulled out and inspected ?
Ground Water Effect?

The measured driving resistance, and the SRD values plotted on Figure E7.2, all increase with depth until the pile tip is near the ground water level at 9.7 m below the surface. Below that depth, there is little or no increase in the measured pile resistance. This could be a coincidence, or it could by caused by a mechanism not yet identified.

High Ageing Effect

The $\Delta_{10}$ value of 1.8 shown on Figure 8.1 is approximately 3 times higher than measured for the Larvik piles in loose silty sand. Earlier studies by NGI have recommended a $\Delta_{10}$ value of 0.25 for sands. The Ryggkollen ageing results should therefore be considered as not reliable at this stage. One possible explanation is local variations in the ground conditions that cannot be isolated, neither by the available soil investigations, nor by the pile driving records.

10 Conclusions

Based upon the results presented above, the following conclusions are drawn:

Soil Conditions at the Ryggkollen test Site

The soils consist of medium to coarse gravelly sand with occasional inclusions of gravel and stones/cobbles. As a result of this presence of coarse material, only one incomplete CPT hole could be carried out at the site.

Measured and Calculated Pile Capacities in Tension

The directly measured pile capacities range from 850 kN to 2100 kN for the first time loading tests. Capacities calculated by 8 different methods range from 780 kN to 1790 kN.

However, the average measured skin friction from tests R1 and R2 after 120 days is 890 kN. The CPT-based methods ICP-05, NGI-05 and UWA-05 give a range of 1300 kN to 1600 kN, assumed to represent 10-days values. The measured capacity is thus only 55-70 % of the calculated one. Possible reasons for this important difference are discussed in Sections 5 and 9. The content of gravel and cobbles in the sand is believed to play an important role.

Ageing Factor $\Delta_{10}$

It was attempted to include a correction on the measured capacities based upon the observed driving resistance. The intension of such a correction was to compensate for local variations in the ground conditions.
However, even with such a correction, the back figured $\Delta_{10}$ value is much higher than considered reasonable. It therefore seems that the ground condition at Ryggkollen are such that direct comparisons from one pile to another are not valid.

11 References

Clausen C.J.F., P.M. Aas & K. Karlsrud (2005)

Norwegian Geotechnical Institute (2011a)

Norwegian Geotechnical Institute (2011b)

Norwegian Geotechnical Institute (2012a)

Norwegian Geotechnical Institute (2012b)
“Time Effects on Pile Capacity, Installation Details and Load Test Results, Test Site Ryggkollen”. Document 20061251-00-266-R presented to Joint Industry Project, 31 August 2012.

Norwegian Geotechnical Institute (2012c)
Figure E2.1

Ryggkollen pile test site with position of piles and boreholes
Figure E2.2:

Ryggkollen CPT hole NGI-13
Figure E3.1

Driving of Ryggkollen pile R1
Figure E3.2

Driving of Ryggkollen pile R2
Driving of Ryggkollen pile R3
Figure E3.4

Driving of Ryggkollen pile R4
Figure E3.5

Driving of Ryggkollen pile R5
Figure E3.6

Driving of Ryggkollen pile R6
Figure E4.1

Measured skin friction values after correction for self weight and buoyancy

Date  Sign  Log of file modifications
25 Aug 2012 cjfc  Original version, units used are: kN, MN & m

******************************************************** CONTROL SECTION ********************************************************
6 MCCLAY Method of calculation for clay layers, see below
6 MCSSAND Method of calculation for sand layers, see below
1 IDATYP Analysis type (1=Single depth 2=Multiple depth 3=Tau skin)

----------------------------------------------------
04: IC 2005 09: UWA 2005-Simpl 04: NGI 1990 09: Not used

******************************************************** SQL PROPERTY SECTON ********************************************************
4 NUMLAY Number of soil layers
0.1 DZSUBL Thickness of layer sub-divisions (m)
9.7 ZSURF Z-coordinate of soil surface (m)
10 GAMWAT Unit weight of free water (10 kN/m3)
0.0 SIGSRF Vertical stress acting at the soil surface (kPa)
0 ISUCOR Code for undrained strength corrections (0=No 1=Yes)

* Layer Z.bot Gam.tot Gam.pwp D.pwp Ko' D.sigPC CPTqc (MPa) Nspt Soil type
  1 4.9 18.0 0.0 0.0 0.0 0.0 30 30 0 2 Sand
  2 9.7 18.0 0.0 0.0 0.0 0.0 30 30 0 2 Sand
  3 12.0 20.0 10.0 0.0 0.0 0.0 25 25 0 2 Sand
  4 21.0 20.0 10.0 0.0 0.0 0.0 20 20 0 2 Sand

* Sand Z.bot Phi Dr D50or API-K.fact Tau.lim Tip.lim Miscellaneous
  1 4.9 0 0.00 0 0.8 0.8 0 0 0 0 0 0 1
  2 9.7 0 0.00 0 0.8 0.8 0 0 0 0 0 0 1
  3 12.0 0 0.00 0 0.8 0.8 0 0 0 0 0 0 1
  4 21.0 0 0.00 0 0.8 0.8 0 0 0 0 0 0 1

* D50 (m) may be used by ICP/UWA, Fines (% is needed by the ISO-2007 method
* Tau.lim and Tip.lim for sand layers only apply to the API-2 method
* Present use of SAND miscellaneous values : S1-S4 used for CaCO3 soils
  * S1 = CaCO3 cont. %  S2 = Degree of cementation, 0-1
  * S3 = K*tan(delta)  S4 = Tau.lim min - API-2 method
  * SS = Factor on calculated skin friction in all sand layers

******************************************************** PILE PROPERTES SECTON ********************************************************
0.4064 DIATOP Outer diameter at pile top (m)
0.4064 DIATIP Outer diameter at pile tip (m)
.0125 TWALL Pile wall thickness (m)
78 GAMWAL Total unit weight of pile wall material (kN/m3)
1 IDOPCL Code for pile tip condition during driving (1=Open 2=Closed)
1.0 PLGRAT Soil plug friction ratio, Tau.inside = Tau.outside.cmp * PLGRAT
2 ICTIP Tip condition (1=Undrained 2=Drained 3=Stress free)

*Miscellaneous pile values (1-10)
P1 P2 P3 P4 P5 P6 P7 P8 P9 P10
0 0 0 0 0 0 0 0 0 0

End of file c:\cjfc\pacer\data\ryggkoll.txt

Figure E5.1: Example input file to the PACER program
Figure E5.2

Capacity of tension piles in sand, from Clausen et al (2005)
Local unit skin friction in tension (kPa)

Effect of pile diameter upon calculated local skin friction for a 50 m pile in sand loaded in tension

Dr = 85-90%, $q_c = 6.6(z^{0.5}$ MPa, $\gamma' = 10$ kN/m$^3$

Pile diameters $D = 0.5m - 2.5m$, $wt = D/30$

Method: ICP-05

Example effect of pile diameter upon calculated unit skin friction in sand
Figure E6.1

Comparison between measured and calculated load-displacement curves
Figure E7.1

System considered by the selected pile driving formula
Figure E7.2

Calculated SRD values for the Ryggkollen piles based upon the observed driving resistance
Figure 8.1

Measured skin friction with SRD-based corrections included
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