Driven Pile Capacity Assessment Using Installation Energy and Pile Set-Up

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ABSTRACT
This paper describes the process used to verify ultimate pile capacity for a wharf project in Kitimat, British Columbia, Canada. The wharf is in a fjord system, where the Kitimat River meets the Douglas Channel, with a deep foundation supported by open-ended steel pipe piles. The stratigraphy primarily consists of interbedded sands, silty sands, silts and clays. The assessment process first uses hammer energy and penetration per blow to estimate end-of-initial-drive (EOID) pile capacity, based on high strain dynamic testing and static load testing. Then an appropriate set-up factor is applied to the EOID capacity to estimate the long-term capacity of a pile. Set-up refers to the increase of pile capacity over time and is mostly caused by excess porewater pressure dissipation, which is primarily by geology and pile type.

RÉSUMÉ
Cet article décrit le processus utilisé pour vérifier la capacité ultime des pieux pour un projet de quai à Kitimat, Colombie-Britannique, Canada. Le quai est dans un système de fjord, où le fleuve Kitimat rencontre le chenal Douglas, avec une fondation profonde soutenue par des pieux de tuyaux en acier à bouts ouverts. La stratigraphie se compose principalement de couches interstratifiées de sables, de sables limoneux, de limons et d'argiles. Le processus d'évaluation utilise d'abord l'énergie du marteau et la pénétration par coup pour estimer la capacité du pieu en fin-de-battage-initial (FBI), sur la base des tests dynamiques à haute contrainte et des tests de charge statique. Ensuite, un facteur de set-up approprié est appliqué à la capacité de FBI pour estimer la capacité à long terme d'un pieu. Set-up référence à l'augmentation de la capacité du pieu au fil du temps et est principalement causée par une dissipation de l'excès de pression de l'eau, qui est affectée par la géologie et le type de pieu.

1 INTRODUCTION

LNG Canada (LNGC) has retained Worley Canada Services (Worley) to provide Engineer of Record services for the construction of a material loading and offloading wharf for Rio Tinto's aluminum smelter (the Project) in the Kitimat River delta. Worley previously completed the preliminary and detailed design for the project. While the full Worley scope involved multiple disciplines, this paper will focus on the geotechnical component; specifically, the assessment and evaluation of driven pile construction for wharf's foundation.

2 SITE CONDITIONS

2.1 Location

Kitimat is located at the northern end of the Kitimat Arm of the Douglas Channel on British Columbia's west coast, approximately 650 km northwest of Vancouver. It has serviced several large industries since the 1950s when the Aluminum Company of Canada (Alcan) first established the townsite (Kitimat Tourism 2020, Kitimat Project History).

Prior to commencement of the Project, the wharf footprint consisted of, from north to south, an asphalt roadway work zone, a steep coastline with rip-rap protection, tidal beaches, and finally, ocean water. The steep coastline had non-engineered shoreline protection, including broken-down concrete slabs placed by demolition works over past decades.

Figure 1 – Satellite Imagery of Project Location Relative to Kitimat (Google Maps 2020. Satellite View of Kitimat Area)

2.2 Geological History

The surficial geology of the Kitimat River valley is dominated by sediment from the most recent major glaciation event, know as the Fraser Glaciation, which began 25,000 to 30,000 years ago (Clague 1984). Deposition prior to this has been largely eliminated during the Frasier Glaciation and post-glacial period (Clague 1984). This glaciation period saw most of British Columbia covered in ice sheets that covered most land mass, except for mountains higher than 2000 m (Clague 1984). The Frasier Glaciation consisted of complex frontal retreat with alternating periods of retreat, stability, and minor re-advances (Clague 1984). As the highlands first became exposed from the ice-sheet, individual tongues of ice began extending in to each of the valleys. These valley glaciers were the source of the deposition profile that is currently seen in the Kitimat area.

Due to the massive weight of overlying ice, the local crust experienced signification isostatic depression during the Fraser Glaciation. During the subsequent Fraser Retreat, flooding of the sea ensued up the valleys from the current shorelines. In the Kitimat River valley, the local sea level was up to 200 m higher than current sea level, with the shoreline existing almost at where Terrace is now located, approximately 50 km to the north. The shoreline has since moved southward due to the complex interaction of crustal isostatic rebound, global sea level rise due to the melting of glaciers, and aggrading sediment deposits.

The flooding of the Kitimat River valley up to Terrace created regions of glaciomarine sedimentation, resulting in flood plains rapidly aggrading, since the existing streams could not cope with the amount of sediment. During and after the Fraser Retreat, the land experienced isostatic rebound. Any small water flows entering the valley were establishing deltas, but at successively lower elevations, due to the crustal rebound.

In the post-glaciation period, there was significant debris swept in to the valleys before the slopes were stabilized with vegetation. From this there was significant deposition in fans, deltas, and floodplains. Once the slopes had stabilized and vegetation had been established, the rivers started to form incised pathways and deposition occurred primarily in lakes, the sea, and in alluvial fans. These rivers and depositional environments occurred within the deltaic and alluvial deposits that extend up to the Terrace region. Nowadays, the Kitimat River continues to prograde into the Douglas Channel, and small tributaries continue to create and/or expand alluvial fans.

Other post-glacial deposits include colluvium, marine sediments, and organic material. Due to the prograding nature of the Kitimat River, underlying the floodplain and delta deposits are marine sediments (Clague 1984). Marine sediments that were deposited near to shore consist primarily of sand and/or gravel, but sediments deposited further from the delta front consist primarily of silt and clay, following the principles of Stokes Law, which states that larger grain sizes deposit more quickly than smaller grain sizes, consisting primarily of silts and clays (Clague 1984).

The resultant founding material for the wharf construction consisted of a complex interlayered combination of marine sediments ranging from silt to clay to sands and gravels, deltaic sand deposits, and alluvial deposits of sands and gravels.

2.3 Site-specific Soils

The following units describe the different materials encountered during the investigative drilling programs, as described in WorleyParsons (2015b). Note that only the material types to interact with the piles are detailed below.

Unit 1 – Fill

This unit was placed during construction of the existing industrial facility, to reclaim part of the beach and is limited to the northern onshore section of the wharf. This unit is generally sand and gravel that contains varying amounts of inert construction waste materials such as waste concrete. This unit was less than 5 m thick.

Unit 2 – Alluvial deposits

This unit is primarily loose sand and gravel with silt, cobbles, and boulders. This unit was only encountered on the west side of the wharf footprint and was less than 5 m thick.

Unit 3 – Deltaic Deposits

This unit consists of the prograded deltaic deposits and is primarily interbedded coarse-grained and finer grained sediments deposited in beds that dip generally in a south-easterly direction towards the river delta. For engineering assessment, the unit was divided into two sub-units based on based on fines content of the bed:

Unit 3a is predominantly fine-grained, consisting of loose silt and sandy silt, with interbedded sands ranging from 2 m to 32 m thick, increasing from west (closest to the shore) to east (further into the delta).

Unit 3b is a silty sand material with interbedded silts and is typically overlain and interlayered with Unit 3a.

Unit 4 – Flood Plain Deposits

This unit is interbedded with the deltaic deposits and varies from sand to silty sand to gravelly sand and was found to have thin interbedded layers of sandy silt and clayey silt. This layer varied from 5 m up to 50 m and is generally denser, i.e. having higher CPT cone resistance, than the deltaic deposits.

Unit 5 – Marine and Deltaic Deposits

This unit consists of clayey silt to silt containing varying sand quantities and thin interbedded layers of silty sand. This unit underlies either Unit 4 or Unit 3 and is the lowest elevation unit that the driven piles interacted with.

Figure 2 below shows a cross-section from north to south on the western edge of the wharf, which contains each of these units.

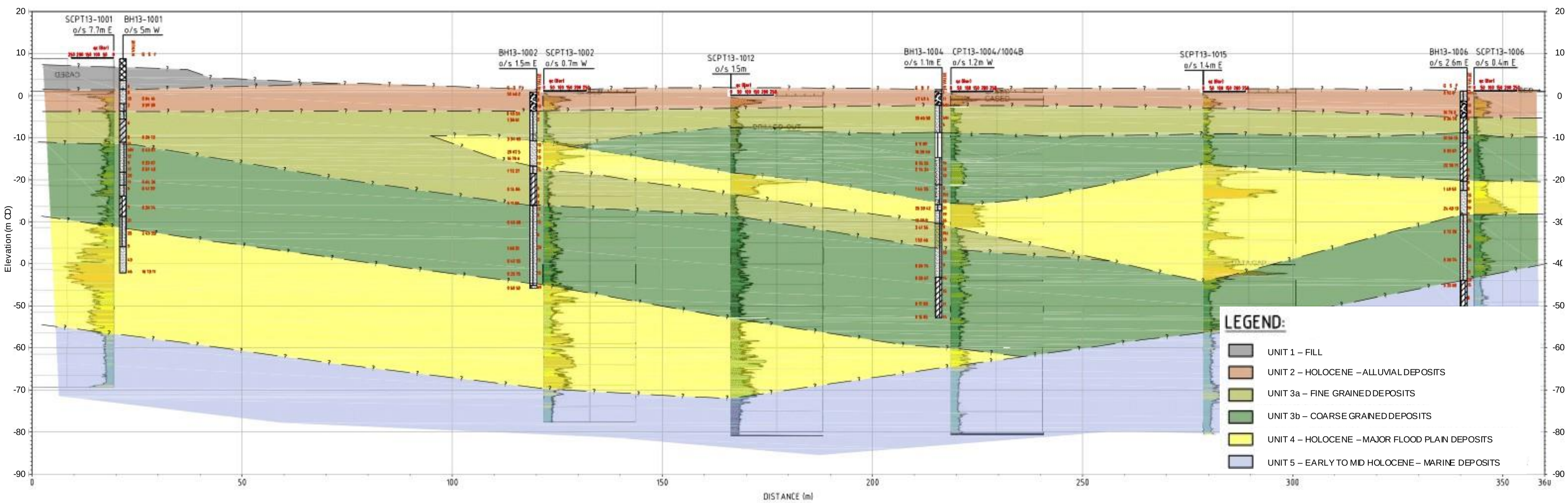

Figure 2 – Geological Cross-Section within Wharf Footprint (WorleyParsons, 2015b)

Note that the datum used on this project and depicted on Figure 2 is Chart Datum (CD). The relationship between CD and geodetic datum (GD) at this site is: 0.0 m CD = −3.2 m GD.

The bedrock elevation is not confirmed within the Project area but based on its location within the western part of the Douglas Channel fjord, it is reasonable to assume it is steeply dipping to the east. Based on available data, it may be in the range of -100 m CD or deeper (WorleyParsons 2015b).

3 PROJECT DESCRIPTION

3.1 Background

LNGC is developing a new shipping terminal for material onloading and offloading for Rio Tinto's aluminium smelter in Kitimat, BC. This wharf is located on the west side of the Kitimat River flood plain adjacent to an existing shipping terminal, where the new construction will tie-in and expand the capacity of the existing RioTinto operations. The footprint of the new wharf is approximately 330 m by 60 m (WorleyParsons 2015b).

3.2 Structure

The structural basis of the wharf project was broken in to five primary areas, which are the abutment, trestle, main wharf, dolphins, and barge berth. The abutment as at the northern end and establishes the beginning of the transition from the existing reclaimed area, behind the existing caisson wharf structure, to the new trestle. The trestle connects the abutment and the new wharf by creating a roadway access over the existing steep rocky shoreline area and tidal beaches. The wharf is the largest area and where most of the loading and offloading activity will be. The mooring dolphins are separate structures located opposite ends of the wharf. The barge berth is located at the southern end of the wharf and includes fender piles and an associated ramp up to the wharf.

All five areas had a similar construction methodology. Construction commenced after dredging to create a slope, under the wharf, down to the berthing pocket in front of the wharf. Firstly, the piles are installed using both a vibratory and a hydraulic impact hammer. The vibratory hammer would be used to pick the pile from horizontal and perform the initial install, and then the hydraulic impact hammer would install the pile the remaining length to achieve the design toe level. Following this, a precast concrete matrix of corbels, beams and panels are constructed on top. These elements are connected using rebar and cast-in-place concrete before the final concrete deck is poured. Supporting elements, such as electrical, water and sewer transportation, were incorporated into this design.

The wharf will allow for 2-way transportation of equipment and vehicles, extensive raw-material storage, material onloading and offloading areas, and a berth for large shipping vessels. The wharf needs to withstand these live loads, in addition to the dead loads associated with the structure itself and seismic loads, with appropriate safety factors.

Figure 3 below shows piles driven in the abutment, trestle, and wharf area during construction, before completion of piling and subsequent placement of the deck structure.

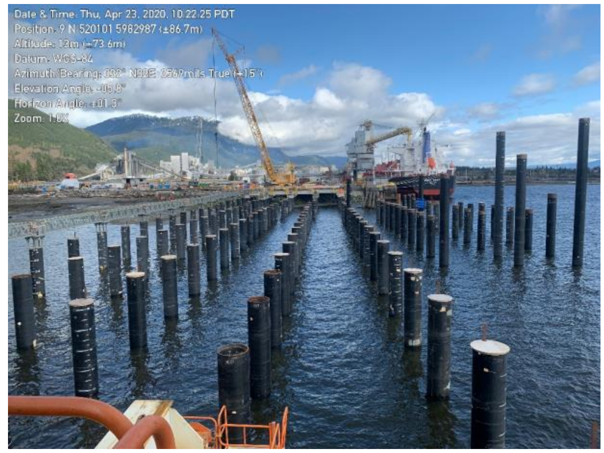

Figure 3 – Piling of abutment, trestle and wharf structures during construction

3.3 Geotechnical Component

As mentioned above, the bedrock depth within the wharf footprint is too deep to be considered in the foundation design. As a result, the foundation design used open-ended steel pipe friction piles. There are approximately 400 piles for the full structure with a diameter of 1,067 mm, wall thickness of 25.4 mm, and embedment lengths that varied from 20 m to 60 m. Because of the large tidal movements and the depth to seabed in the wharf footprint, the total pile lengths varied from 36 m to 78 m.

This paper focuses on the axial component of static pile capacity. Other pertinent geotechnical considerations, such as lateral capacity, settlement estimations, and seismic cases were addressed during design, but they are not discussed further here.

For the abutment and wharf, there is no resultant tension loading on the piles, since the operating and self-weight of the structure exceeded any applied tension load. The trestle and dolphin piles are designed with both compression and tension components considered, however, the ultimate unfactored compression loads governed the pile design length requirements in all situations.

Considering load and resistance factors, the ultimate unfactored compression loads on the piles range from 1,800 kN to 10,000 kN. The ultimate unfactored tension loads are 1,000 kN for the trestle and range from 1,000 kN to 4,900 kN for the dolphins.

As the piles are friction based and did not target a specific end bearing layer, the project design called for the piles to be installed to pre-determined toe levels to achieve pile capacities. Additionally, the construction team were able to procure most piles in their full lengths by taking advantage of sea-based delivery of the pipes in order to reduce the need for splicing on site. Therefore, after the piles were installed to toe level, the capacity of each needed to be verified. The specifics of this process are detailed in the following sections, but the basis of this verification relies on a combination of blow-count measurement on all piles, high strain dynamic testing on some piles, with interpretations initially calibrated by static load testing on two piles. The high strain dynamic testing was analyzed by independent engineers subcontracted to the contractor who used CAPWAP® (Case Pile Wave Analysis Program) software for the interpretation. This software simulates a static load test to estimate the total pile capacity at the time of the test (PDI Pile Dynamics, Inc. 2019a). The test uses high strain dynamic test data collected during impact driving by a Pile Driving Analyzer® (PDA) pile driving monitoring system (PDI Pile Dynamics, Inc. 2019b).

For high strain dynamic testing of piles, strain gauges and accelerometers are installed on the pile for collection of data during a strike test, such as End-of-Initial-Drive (EOID) or Beginning-of-Restrike (BOR). The strain gauge data is interpreted to determine the stress and dynamic force the pile is subjected to and the accelerometers are interpreted to assess velocity and displacement of the pile over time for each strike of the test. This data, along with an understanding of soil profile and parameters, hammer details, and pile parameters are used to interpret static pile capacity at the time of testing. Each interpretation in the CAPWAP® analysis is given a Match Quality index, which measures the confidence in calculated capacity (PDI Pile Dynamics, Inc. 2019.).

A test pile program consisting of driving two test piles, EOID and BOR tests, then static load tests, then further BOR tests was executed prior to construction. This enabled validation of the high strain dynamic testing interpretations against measured static capacity as well as to acquire site-specific data on pile set-up behaviour, discussed in the following section.

4 PILE SET-UP

4.1 Definition

Pile set-up may be defined as the increase of pile capacity over time. The magnitude and duration of set-up is affected by soil type, permeability and sensitivity, pile type and size, and installation technique (Komurka et al. 2003). Set-up may take place over time periods ranging from minutes (Bullock 1999) to years (Skov and Denver 1988) and has accounted for long-term capacities up to 12 times the initial capacity (Titi and Wathugala 1999).

When a pile is driven, soil is displaced and disturbed for a certain radius around the pile. This causes an increase in porewater pressure and an associated decrease in effective stress. Set-up is recognized to be primarily due to porewater pressure dissipation over time that causes a resultant increase in shaft resistance (Komurka et al. 2003). Set-up may also be affected by a process called aging, which continues after porewater pressure dissipation is complete, similarly to secondary compression, and generally increases shear modulus, stiffness, dilatancy and friction angle and decreases compressibility (Komurka et al. 2003; Axelsson 1998; Schmertmann 1981).

The process of pile set-up may be divided into three phases (Komurka et al. 2003). The first and second phases are associated with excess porewater pressure dissipation. The first phase is difficult to model and predict due to the highly disturbed nature of the soil at this time and can last from minutes to days (Komurka et al. 2003). The second phase is simpler to model, as the porewater pressure dissipation is linear with respect to the logarithm of time. The third phase is attributable to the aging process, which refers to the time-dependent change in soil properties at a constant effective stress, such as thixotropy, secondary compression, particle interference and clay dispersion (Komurka et al. 2003). The phase being experienced across a project site may vary in plan and with depth due to soil variability, and these phases may overlap one another. Consideration of these phases may prove useful when estimating a set-up model for a given project site.

4.2 Design Challenges

During the pile design stage of a project, it is common to use static analysis methods, such as the LCPC Method (Bustamante and Gianselli 1982), to estimate required pile lengths. This may be combined with wave equation analysis of the complex hammer, pile and soil system to estimate a required set criterion, i.e. the required penetration per blow of the pile for a given hammer type and energy to verify a specified capacity is achieved.

Design stage pile capacity estimates are typically based on pile capacities that are expected to be achieved in the long-term; however, during construction execution it is important to understand the time based set-up behavior and how it relates to the pile capacity evaluations, testing program and overall construction schedule, which naturally has an urgency to obtain pile approvals so that pile cut-offs and superstructure construction can proceed with minimum delays.

Since set-up is recognized to affect shaft friction more than end bearing (Axelsson 2002; Bullock 1999; Chow et al. 1998) the set-up phenomenon is particularly important for friction piles, which are not typically driven into a hard layer such as glacial till or bedrock, as is the case for the Project.

Several empirical relationships exist for estimating set-up, including Guang-Yu (1988), Huang (1988), Skov and Denver (1988), Svinkin (1996) and Svinkin and Skov (2000). Additionally, historical long-term high strain dynamic tests or static load tests in a similar geological environment may be useful in estimating set-up. Care should be used by the designer to ensure that the empirical relationships or historical data used are applicable to the subject project.

4.3 Construction Challenges

The set-up phenomenon can make both long-term and short-term estimation of pile capacity during construction challenging. It is generally recommended to conduct restrike testing and/or static load tests to ascertain the applicability of the parameters used during the design phase. If a test pile program has not been completed in advance, and set-up is found to be significant, delays may occur in the early phases of construction due to the need to wait for long-term restrikes.

Additionally, if the set-up phenomenon is not recognized and a set criterion has been defined based on long-term capacity requirements, this may lead to unnecessary pile splicing, as the target capacity may have been reached simply by waiting for set-up to occur.

For the Project, piles have generally been driven first using a vibratory hammer, and then driven using an impact hammer to the design toe elevation at some later date. During the period between vibratory and impact driving, significant set-up has been witnessed. However, when impact driving has commenced, the set-up has not immediately been lost, but instead has taken up to 5 to 10 m of driving distance. A simple set criterion approach that did not consider future set-up would have been ineffective in this instance, as the capacity appeared to be dropping with further penetration. This would have been nonsensical, since the piles for the Project are friction piles, with a relatively small end bearing contribution typically in the range of 10% to 15% of the long-term capacity.

While long-term pile capacity is important for the final structure, short-term capacity is also an important consideration for temporary loading during construction. If long set-up times are expected, it may be advisable to recommend longer pile lengths if the costs of this are less than the costs that may be incurred to due construction delays caused by relatively slow set-up.

These examples stress the importance of gathering set-up data during the design phase of the project, preferably in the form of a site-specific test pile program, to optimize design and the testing plan during construction. If this process does not occur, there is significant risk of delays and/or redesign, to account for previous under or over design, during the construction phase.

5 PILE CAPACITY ASSESSMENT

One of the primary duties of an Engineer of Record is to ensure that the design intent is met during construction. For pile driving, this means reviewing relevant information, including pile driving records and available restrike testing and static load testing results, to estimate the long-term capacity of the pile. Should this capacity estimation be found to be less than that required, remedial action, often in the form of a pile splice, may be required, which may involve cost and/or time delays for the subject project. The following sections describe the pile assessment process for the Project.

5.1 Pile Capacity vs. Energy per Set Relationship

The first step in the pile capacity assessment process is to estimate the pile capacity at EOID. This is either measured by PDA on 10 to 15% of piles as described earlier or must be interpreted based on pile driving blow counts for the remainder of piles. For the Project, a reasonable relationship was developed between total capacity (measured in kN) and energy per set (measured in kJ/[mm per blow]) from the EOID and BOR testing completed during the test pile program and during production piling. Energy per set is used to account for the variable hammer energy settings used during driving. A graph of this relationship is found in Figure 4 below.

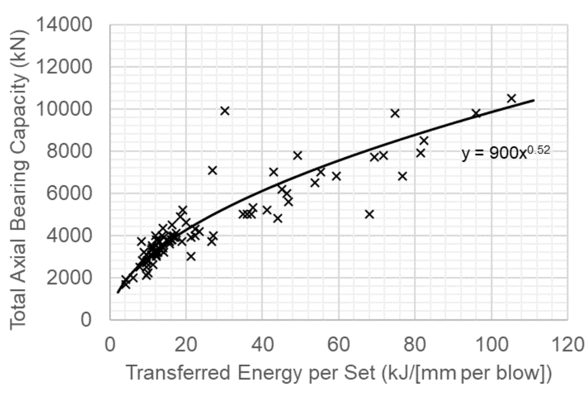

Figure 4 – Graph of Axial Compression Bearing Capacity vs. Energy per Set for the Project

A line-of-best-fit can be drawn through these points to estimate the pile capacity for the specific pile type described in Section 3.3 at the Project site.

While static load test results cannot be directly plotted on this graph, they were used to verify the reliability of the CAPWAP interpretations from BOR dynamic testing on the same piles. For the Project, the two static load tests completed during the test pile program proved capacities that were equal or greater than the BOR dynamic test results, confirming that the restrike test interpretations were reliable and likely conservative.

An important consideration in using the above graph is the efficiency of the hammer in delivering energy to the pile. This can be estimated by comparing the transferred energy, which can be found from the restrike testing results, to the hammer energy setting, which should be available in the pile driving records. The above graph uses transferred energy, so the uncorrected energy from the pile driving records should be adjusted prior to using Figure 4.

5.2 Time-Dependent Set-Up Factors

Once an EOID capacity estimate has been obtained using the above Figure 4, an appropriate total set-up factor (i.e. long-term capacity divided by EOID capacity) needs to be applied to it to estimate the long-term capacity, noting that the set-up factor is highest at EOID and reduces to one in the long-term when the set-up process is complete. Ideally, the total set-up should be based on site-specific long-term restrikes and static load testing.

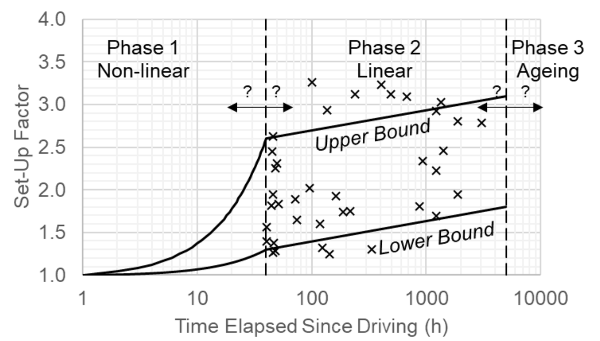

Figure 5 – Graph of Set-Up Factor vs. Time for the Project

Figure 5 above shows the instantaneous set-up factors witnessed by comparing dynamic tests conducted both at EOID and at various BOR times after EIOD, expressed as Current Capacity divided by EOID capacity.

Since the piles at the project site are considered friction piles, the set-up factors were based on shaft plus toe capacity rather than attempting to isolate one from the other. This was needed for simplicity as there is some uncertainty in properly estimating the end bearing component using high strain dynamic testing and it is not possible to distinguish between shaft and end bearing based on blow count records for piles with dynamic testing.

As seen in Figure 5 above, set-up factors after 350 to 3000 hours (i.e. 15 to 125 days) have ranged from 1.8 to 3.3. There is obvious variability in these results, which is likely due to the variable geology at the project site affecting rate of pore pressure dissipation and hence set-up time.

With respect to the phases described in Section 4.1, it appears that some combination of Phases One and Two can be witnessed in Figure 5 above, although the transition point is uncertain, and different Phases may be occurring simultaneously at different locations in depth and plan. The longest-term restrikes available are approximately 120 days after driving, so it is not feasible to ascertain if Phase Three has already occurred by this time, if at all.

5.3 Assessment Process

Given the variability in set-up observed, there are various factors that need to be considered in applying a set-up factor to the EOID or BOR results to interpret long-term capacity. The set-up factor to apply is highest based on EOID dynamic tests or blow counts and needs to be reduced to account for time since EOID when interpreting BOR tests.

The full blow-count record obtained from driving between the end of vibratory driving and the final toe level can help in understanding the geology and set-up behavior of the project site and needs to be considered to assess an appropriate set-up factor to apply to the EOID capacity.

When impact hammering is commenced after a delay since vibratory hammering, there will have been an initial set-up. The initial set on commencing impact driving is an indicator of current capacity. As the pile is driven the initial set-up will be lost as the pile is mobilized and the reduction in set to a minimum as the pile remobilizes can be used as an indicator of EOID at that depth. Comparing the capacities at these two points can enable assessment of a lower bound set-up factor. Typically, further driving results in an increasing blow count as the total length of shaft in contact with the ground and hence pile capacity increases until final toe depth when the EOID set is recorded.

If the pile is not impact driven far enough past the vibratory driving depth, then the pile may not become fully mobilized by impact driving, meaning the set-up factor to apply to EOID needs to be reduced

6 LIMITATIONS

The pile capacity assessment process described above has several limitations, which are listed below. These limitations should be considered, and possibly expanded on, if applying this process to another project.

- Due to the increase in pile capacity, it is possible that piles may not be fully mobilized during restrike testing. This may be evident if the set values are low. In this case, the pile capacity, and therefore the set-up, would be underestimated. This occurred for multiple restrike tests on the Project and were therefore not included on Figure 4.
- Not all piles that had restrike testing had EOID testing completed as well. As such, estimated EOID pile capacities were estimated from Figure 4. While this adds some uncertainty, this still allows for points to be plotted on Figure 5.
- While some allowances have been to adjust for different hammer energies, care should be taken to not vary it too greatly, as a simple linear adjustment likely has limitations. For example, four blows at 50 kJ energy may not be equivalent to one blow at 200 kJ energy. For this project, energy settings typically varied in the range of 150 kJ to 230 kJ.
- Detailed pile driving records are critical for this type of assessment process. Expectations should be clearly defined in the piling specifications. Inconsistent or incomplete recording of pile driving data can lead to unnecessary uncertainty or make this assessment process difficult to apply.
- For some piles, a substantial amount of time passed between vibratory and impact hammering, allowing for significant set-up to take place. Subsequently, it took up to 5 to 10 m of impact driving for set-up to be completely lost. The pile driving record should be carefully examined to see if it is warranted to use the full set-up factor suggested by Figure 5, or some fraction of it, particularly for impact driving lengths less than 5 m.

7 CONCLUSION

Many piling projects use a pile acceptance criterion based on set, i.e. a target amount of penetration per blow for a given hammer and energy setting. This may be effective for some projects but can lead to pile lengths significantly longer than are necessary on other projects. Consideration of the set-up phenomenon can have significant time and cost benefits. If feasible and practical, it is recommended to complete a test piling program in advance of construction to have a better site-specific understanding of pile capacities and set-up behavior.

The design and construction of a wharf in the Kitimat River delta has presented several unique challenges. The deep, soft soils at the Project led to the need for over 400 piles of lengths between 36 and 78 m to support the relevant structures. Consideration of the complex geological history of the site, with emphasis on soils deposited during and since the Fraser Glaciation, was critical for design, in addition to detailed characterization of the site-specific soils.

The pile assessment process described herein assessed capacity by estimating the capacity at the time of driving based on the correlation of capacity of energy per set vs. capacity in Figure 4, and then applying an appropriate set-up factor based on Figure 5. This approach has worked effectively for the Project. The role of the Engineer of Record has been satisfied by adequately assessing pile capacity, and there have been significant time and cost benefits compared to if a set criterion with no set-up consideration was used. The relationships of capacity vs. energy per set (Figure 4) and set-up vs. time (Figure 5) indicate relatively large set-up factors between 2 and 3 in the Kitimat River delta. This information may be useful for other projects in a similar geological environment, and the general process may be particularly useful for projects with significant set-up.

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