Review of capacity of 45 year old piles via restrike PDA testing

Evaluation de la capacité de pieux vieux de 45 ans à l'aide d'essais de chargement dynamique

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ABSTRACT: Dynamic pile testing was undertaken on a nearby structure to assess the capacity of 876 existing piles supporting a wharf being upgraded at an export port. The 45 years old piles were found to be over-utilized by as much as 300% under the design loading based on conventional pile capacity calculations. Restrike dynamic testing was therefore undertaken on several piles from a two-year old nearby jetty due for demolition, for which end of drive and 1-2 day restrike pile driving analyzer (PDA) testing was available. The testing demonstrated shaft setup values of between 2.1 and 4.2 compared to end of drive, and overall capacity setup of between 1.6 and 2.8. When translated to the nearby 45 year old wharf structure, allowing for variances in pile toe elevations and stratigraphy, the total number of piles nominally overstressed reduced from over 300 to 17, based both on changes to geotechnical strength reduction factors and adoption of less conservative geotechnical capacity estimates. When further allowances were made for structural redundancy and other conservatisms in the analysis, only 6 piles were considered unacceptably overstressed. The PDA testing resulted in a significant reduction in potential piling works and considerable savings for the client.

Résumé : Afin d'évaluer la capacité portante de 876 pieux existants supportant le quai d'un port d'exportation en phase de modernisation, on procéda à des essais de chargement dynamique sur un structure voisine. Les techniques de calcul conventionnelles de capacité révélèrent que les nouvelles charges de ces pieux vieux de 45 ans étaient jusqu'à 300% supérieure à la capacité portante. Des tests dynamiques par re-battage furent donc entrepris sur plusieurs pieux appartenant à une jetée vielle de deux ans en passe d'être démolie, et sur lesquels il était possible de réaliser des tests de fin d'enfoncement ainsi que des essais dynamiques pendant 1 à 2 jours. Les tests démontrèrent des valeurs de résistance de fût à l'installation entre 2.1 et 4.2 par rapport aux valeurs de fin d'enfoncement, et une capacité portante totale entre 1.6 et 2.8. Quand ces valeurs furent converties aux pieux vieux de 45 ans appartenant au quai en question en prenant en compte les variations de stratigraphies et de profondeur de pieux, le nombre total de pieux étant nominalement en surcharge réduisit de plus de 300 à 17, en se basant sur des modifications des facteurs de réductions de la résistance géotechnique et par l'adoption de capacités portantes moins conservatrices. De plus, lorsque l'on prit aussi en compte les redondances structurelles et autres aspects conservatifs de l'analyse, il ne restait plus que 6 pieux étant soumis à une surcharge inadmissible. Les tests PDA par re-battage ont permis une importante réduction des travaux de mise en œuvre des pieux ainsi que des économies de budget considérables pour le client.

KEYWORDS: Pile design, Dynamic testing, Pile driving analyzer (PDA), Setup, GRLWEAP

1 INTRODUCTION.

In order to increase the export capacity at a mining port, new shiploaders were proposed to be installed on an existing wharf originally constructed in the late 1960s / early 1970s. The existing structure comprises a reinforced concrete flat slab wharf deck approximately 650 m long by 25 m wide supported on 876 steel tubular driven piles ranging from 628 mm to 1066 mm external diameter. The toe level of the existing piles was inferred from an inspection of the original construction drawings and installation records, and ranged from 2.7 m to 12.5 m below seabed level. In order to accomodate the new shiploader, an assessment of the structural and geotechnical capacity of the wharf was necessary to assess what remediation and strengthing works were required to the structure to accomodate the new loading regime, and extend the design life of the structure. Figure 1 shows the construction of the original wharf in 1970.

Figure 1. Construction of wharf, c1970

Figure 2 shows a typical schematic cross section of the wharf. A recently installed row of piles to support a conveyor (for which pile testing data is also available), is shown in the figure.

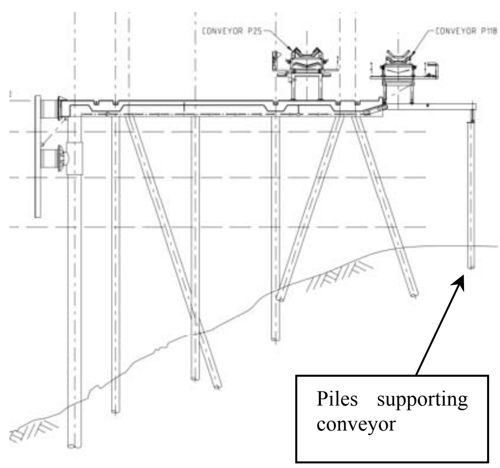

Figure 2. Typical cross section

A 'temporary' jetty, located approximately 20 m north of the main wharf and constructed from 2012 to 2013, was scheduled for demolition during this study (2015), providing a unique opportunity to load test the driven piles for the temporary structure. The temporary jetty was approximately 400 m long, comprising 82 vertically driven 610 mm diameter steel tubuluar piles. The piles were fitted with an internal steel ring to retard the development of internal shaft friciton, allowing full external pile resistance to develop. Figure 3 shows the location of the temporary jetty in relation to the main wharf.

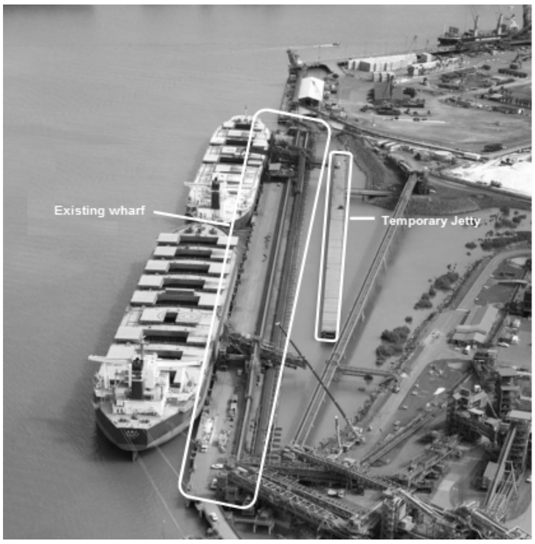

Figure 3. Existing wharf and temporary jetty

2 GROUND CONDITIONS

Ground conditions at the site were inferred from geotechnical investigations ranging from the time of construction of the original wharf, to recent investigations undertaken at the temporary jetty site, adjacent to selected test piles. Ground conditions comprise between 2.8 m and 5 m of loose Holocene marine muds, overlying between 9 m and 14 m of very stiff Pleistocene sandy clay and low strength clayey sandstone, and low to medium strength Pleistocene sandstone. The sandy clay and low strength clayey sandstone are essentially the same material, the difference being the degree of induration and cementation present. Calcium carbonate content of both units is typically less than 20%. The geology of the site is described in more detail in Kristinof 2013, which looked at setup from static load tests at a nearby site in the same geology. The low to medium strength sandstone layer comprises sandstone and clayey sandstone, with an appearance of rock which has been fractured insitu and recemented via groundwater flow and deposition of silcrete and calcrete cements. Median UCS strengths of 0.2MPa, 0.8MPa and 2.0MPa were initially adopted for each unit of the stiff sandy clay, low strength clayey sandstone and low to medium strength sandstone, respectively, based on inferred strengths and laboratory testing from historical boreholes at the site.

3 INITIAL CAPACITY ESTIMATE OF EXISTING PILES

The theoretical capacity of the existing piles was assessed using two different methods, in order to improve the credibility of the estimate. These are discussed in Sections 3.1, and 3.2 below.

3.1 *CAPACITY ASSESSMENT USING SRD CURVES*

Original drawings and pile installation records suggested that three different types of hammers were used to install the driven piles: A Delmag D44, Kobe K42 and an unknown BSP steam hammer. The properties for the Delmag and Kobe hammers were readily idenifiable from manufacturer databases, with rated energies of 122kJ and 107kJ, respectively. The properties of these hammers were input into the software package *GRLWEAP 2010*, along with pile size information and the inferred ground model, in order to prepare a series of 'bearing graphs' for each scenario (refer to Figure 4). The graphs present soil resistance to driving (SRD) versus number of hammer blows, assuming the hammers are in good condition and operating at full stroke. The proportion of total resistance provided by the pile shaft was assumed to be between 50 to 60% based on an inspection of PDA and CAPWAP records from the nearby tempory jetty.

The SRD for each pile was assumed equivalent to the pile capacity at the time of installation. For each pile where final set/blowcount information was available, an end of drive SRD (capacity) could be estimated. A conservative setup value of 1.25 (based on 1-4 day values of 1.2 to 1.6 observed on newly installed piles nearby) was applied to the shaft friction component of the capacity estimate, and further corrections were made for the likely change in seabed level and embedment over time due to erosion and dredging.

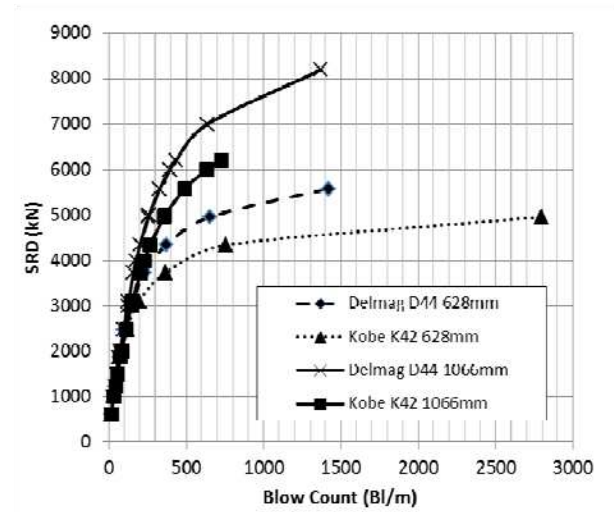

Figure 4. Bearing graphs – SRD versus blowcount

3.2 *STATIC AXIAL CAPACITY USING API METHODS*

The axial capacity of the piles was also assessed using a design methodology based on the guidance provided by API 2007. The Holocene muds were ignored in the analysis, being relatively loose and subject to low effective stresses. Each of the other geotechnical units were modelled as 'clay' owing to their relatively high proportion of fines (typically above 30%). This included the weak sandstone materials, which typically also had a high fines content within their matrix. External shaft friction was assessed as a function of the ratio of undrained shear strength (assumed as 0.5 x UCS) and vertical effective stress via the α-method, with upper limiting values of 300kPa adopted for each layer, based on previous pile design experience in the area. Values were compared to measured CAPWAP shaft resistance values from nearby conveyor piles (refer Figure 2), and shown to compare well with the lower-quartile of that dataset, as shown in Figure 5.

End bearing capacity was taken as 4.5 x UCS (equivalent to 9 x s_u). To allow for potential plugging of the piles, the contribution of internal shaft friction was assumed to be limited to the theoretical bearing capacity of a soil plug. Finally, carbonate content was assumed to be sufficiently low in each material that any influence on the pile capacities could be ignored.

Similar to the SRD models, a setup factor of 1.25 was applied to the calculated shaft capacity. Capacity of each pile was then estimated based on the best estimate of the ground model for each pile (based on the site wide geotechnical model) and the current seabed surface (noting that in most instances it was lower than the originally dredged seabed, due to more recent dredging and scour). Pile toe levels were as per the original design drawings.

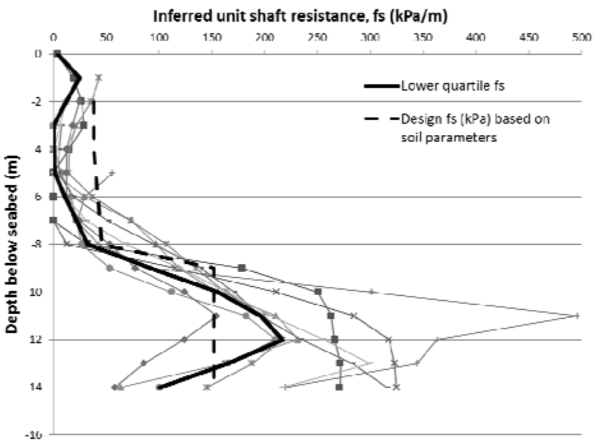

Figure 5. Calibration of static pile capacity against CAPWAP data

3.3 *ASSESSED PILE CAPACITIES*

For piles installed with the Kobe or Delmag hammer, for which piling records and hammer details were available, the ultimate pile capacity was assessed using the SRD method. 349 piles were either installed using the unknown steam hammer or had installation records which were illegible. For those piles, the capacity was taken as the static axial capacity assessed using the API method. A conservative geotechnical strength reduction factor (ϕ_g) of 0.45 was adopted in accordance with the local piling standard (AS 2159-2009), to reflect the uncertainty in the assessments.

As a result of the assessment of pile capacities, a structural assessment of utilisation of the piles in the wharf resulted in 304 piles being regarded as overloaded, when an assessment of design ultimate limit state loads was considered.

4 REASSESSMENT USING TEMPORARY JETTY PILES

As the temporary jetty structure nearby the wharf was due for demolition at the time of this study, the opportunity arose for restrike testing to be undertaken on piles which already had end of drive CAPWAP results from the time of construction. Five of the piles which had been CAPWAP tested during installation were selected for restrike testing. The jetty deck was dismantled to expose the pile heads, and the piles were restruck with a Juntann HHK7 hammer, with nominal rated energy of 107kJ. A borehole was drilled adjacent to each pile to assess the ground conditions and the relative thickness of the geotechnical units. Results from the restrike testing, showing the change in CAPWAP assessed pile capacity over time, are presented in Figure 6.

Setup on pile shaft resistance varied from 2.1 to 4.2, with a mean value of 3.2. The proportion of the resistance which was derived from the shaft also increased to between 80% to 90%, from 50% to 60% originally assumed. Mobilised sets during the restrike testing were only 0.3 mm to 0.4 mm; from the authors' experience values of 3 mm to 4 mm are considered necessary to fully mobilise the pile's capacity, and as a result the assessed CAPWAP restrike capacities were considered to be conservative.

Individual CAPWAP results were inspected to ensure the breakdown of pile resistance with depth matched what was expected from the geotechnical model and original blowcount records of the tested piles. In three of the five restrike tests, there was good agreement with the breakdown of pile shaft resistance versus depth predicted by CAPWAP and the original recorded blowcounts. In two cases however, the CAPWAP results did not agree well with the blowcount pattern (for example, capacity appeared to decrease below 8.0 m, when both blowcounts and the geotechnical model suggest that capacity should increase). The discrepency is likely due to the shortcomings of automated signal matching undertaken during the data processing, as described by Seidel 2015. Where the CAPWAP results were not considered to be geotechnical credible, the results were manually adjusted corrected using the method described by Seidel 2015, ensuring for example, that the infered shaft resistance was consistent with the relative increase in hammer blow count versus depth. An example of the correction performed is shown in Figure 7.

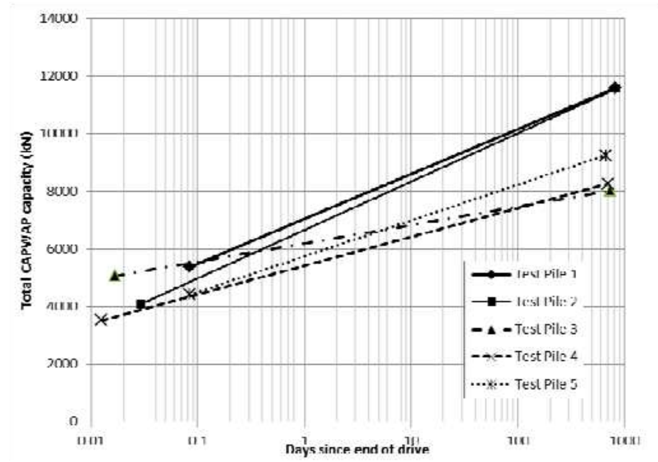

Figure 6. Increase in pile capacity over time

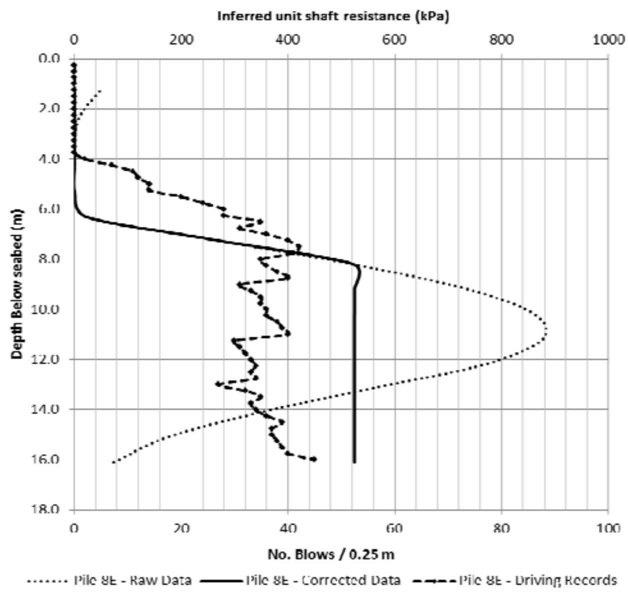

Figure 7. Correction of CAPWAP results

The corrected CAPWAP results and observed setups were then used to reassess the pile capacities. Based on the boreholes adjacent to each test pile, the ultimate shaft resistance for static axial calculations was reassessed for the very stiff sandy clay and low strength clayey sandstone. Values of unit shaft resistance of 78 kPa and 327 kPa were adopted for each unit respectively; being the mean minus one standard deviation value for measured CAPWAP shaft resistance in the two units, overriding the values derived using the API 2007 methodology. The design UCS value of the low to medium strength sandstone was also reduced to 1.1 MPa from 2.0 MPa on the basis of the new ground information that was obtained.

Capacities based on the SRD curve methodology were also reevaluated, by taking into account the increased rates of setup which were observed.

Finally, the additional data which was collected allowed a revision to the geotechnical strength reduction factor (ϕ_g). In parallel with the geotechnical capacity study, structural analyses of the overall wharf confirmed a high degree of structural redundancy, meaning the superstructure itself was able to shed load across multiple load paths in the event that any individual pile became modestly overloaded. The risk based methodology in AS2159-2009 allows for an increase in the value of ϕ_g in this circumstance, and a revised value of 0.64 was ultimately adopted, compared with 0.45 which was considered valid prior to the pile restrike testing program. This revision alone contributed an increase in design pile capacity of over 40%

5. DISCUSSION AND OUTCOMES

As a result of the additional re-strike pile testing and revisions to the geotechnical strength reduction factor, the total number of piles which were nominally overloaded under the revised structural loads decreased from 304 to just 17. Of these, 11 had utilization (ratio of Ultimate Limit State (ULS) structural load to ULS geotechnical capacity) of less than 110%.

The 11 piles which were over-utilized by less than 10% were judged to be nominally acceptable, due to several conservatisms in the analysis:

- The pile test results were unlikely to have fully mobilized the full pile resistance, based on the relatively low observed set values. The derived capacity values used in the analysis are therefore considered to be conservative;
- Little research is available on long term setup of piles. State of the art research (for example, NGI, 2014; Jardine et al 2006; Lim et al 2014) is limited to observations over one to two years. Setup is postulated by the authors to continue beyond the two years over which it was observed in this study (albeit at a decreasing rate with time); the piles in question are in excess of 40 years old and it is probable that the pile capacity would have increased further over time;
- The revised pile capacity assessment did not take into account any potential setup in the underlying low to medium strength sandstone;
- There was considerable variability in the driven embedment depth of the existing piles, based on installation records. It is probable that the piles which have very short embedments have refused at shallow elevations due to encountering hard material, and as a consequence might be expected to have higher base resistance than has been inferred from the overall geotechnical model of the site.

Of the remaining six piles, two had over-utilizations in excess of 250%, which was not considered credible; if these values were accurate then the piles would likely already be overloaded under existing operational loads, and the wharf showing signs of distress in those areas as a result.

Finally, it was noted that in several instances the revised geotechnical capacities, making allowance for setup, actually exceeded the nominal structural capacity of the piles themselves.

5 CONCLUSION

Long term setup of pile capacity is rarely studied, with limitations on the data available an inevitable consequence of not only the cost of pile testing in general, but the practicalities of undertaking such long term tests in a commercial construction setting.

The evaluation of pile capacity by taking into consideration the change in capacity over time of a nearby set of piles has considerably improved the viability of the upgrade of the facility in this case study, and therefore has provided a credible and cost effective solution.

It is these authors' judgement that construction timeframes and methodologies which allow sufficient time and testing in which to take advantage of such setup can significantly improve the pile capacities which can be realised, ultimately cutting material costs by reducing the size or embedment length of piles needed to sustain design loads.

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