Keynote Address

Ground truth, control and design of driven piles: implementing old ways with a new twist

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ABSTRACT

Piling design and verification is a fraught and risky business. The spread of pile capacity estimates submitted to conference predictions exercises is often staggering and sobering. This underlines why design of driven piles does not stop at the design engineer's desk but continues through construction, and relies on the valuable information provided by the installation process. Each installation blow is a test - a test of the ground response to hammer input delivered into the pile. Traditionally, pile capacity has been interpreted from this input-response relationship through various and many pile driving formulae. Five decades ago, measurement systems were first used to measure and interpret the stress waves in piles generated from the hammer inputs and reflected from the ground response to infer capacity in a more sophisticated and reliable way using wave mechanics principles. Today, PDA testing and wave matching are routinely accepted practice. However, each PDA test has direct relevance only to the individual pile which is tested. This paper will argue that our fundamental task as designers and supervisors is to establish ground truth, by synthesizing the results of PDA tests into a locally-evidenced and locally-targeted dynamic formula. Therefore, only dynamic formulae, properly modified and correlated, must be the vehicle for delivering local ground truth and ultimately being the basis for sign-off. On a foundation-wide basis, the role of PDA tests is critical but subservient, and principally to provide the evidence on which a correlated dynamic formula is developed. Consequent implications for the foundation sign-off process, and for a proposed new approach to establishing capacity reduction factors for driven piles will also be discussed.

Keywords: pile driving formulas, pile acceptance, PDA testing, capacity reduction factors, wave equation analysis

1 INTRODUCTION

Piling design is a fraught and risky business. The spread of pile capacity estimates submitted to conference predictions exercises is often staggering and sobering. Fellenius (2013) summarizes the results of such a prediction exercise for a continuous flight auger (CFA) pile installed in clay till with sand and gravel lenses. The predicted load-movement responses from 41 invited foundation engineers are shown in Figure 1 with the actual load test results. The test had to be terminated prematurely for safety reasons and two possible extrapolations are shown in the solid lines. 35 predictions included a full load-settlement curve; 6 provided only a capacity estimate at 50mm movement indicated by the black circles. The average capacity prediction was 1920kN, with the range of predictions between 700kN and in excess of 5000kN. For the purpose of this paper, the issue is not how well the predictions compare with the measured (and extrapolated) test response, but the extreme range of predictions of both pile capacity (7-fold) and pile stiffness (8-fold).

This paper is concerned with driven piles, and in contrast to CFA piles, there is another opportunity to evaluate capacity for individual piles based on monitoring of key installation characteristics. The potential uncertainty of geotechnical design underlines why design of driven piles does not stop at the design engineer's desk but continues through construction, and relies on the valuable information provided by the installation process.

For pile driving, each installation blow is a test. A test of the ground response to hammer input delivered into the pile. Traditionally, pile capacity has been interpreted from this input-response relationship through various and many pile driving formulae (e.g. Hiley, 1930; Chellis, 1961; Olson and Flaate, 1967; Fragaszy et al., 1989; Lawton et al., 1986; Allin et al., 2015) amongst many others. A particularly good summary can be found in Groom, (2019).

The underlying assumption of any pile driving formula is that it should reflect a universal ground truth and by application of this formula, pile capacity can be inferred by measurement or assumption of some key parameters. The many studies which assessed the comparative merits of different formulae by comparison with static load tests all indicated large scatter and poor reliability which resulted in recommended factor of safety of up to 6 (for the Engineering News Record formula).

However, it should be considered that at the time of many of these comparison studies, the effect of pile setup could not be assessed, and the true energy delivered by pile driving hammers could not be measured. These are critical factors which the authors of early studies did not have at their disposal, and would undoubtedly have influenced their finding

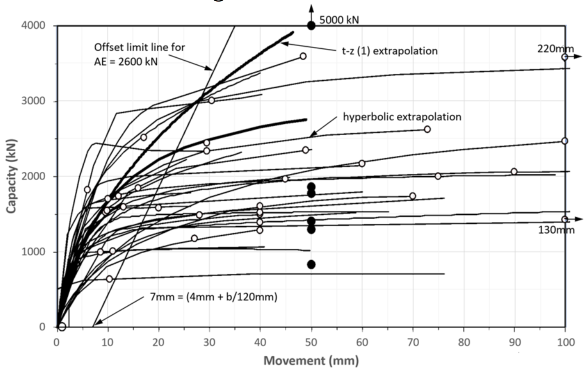

Figure 1. Load-movement predictions from Fellenius (2013)

Fifty years ago, electronic systems were first used to measure and interpret the stress waves in piles generated from the hammer inputs and reflected from the ground response to infer capacity in a more sophisticated and reliable way using wave mechanics principles (Rausche et al., 1972; Goble et al., 1975). Today, PDA testing and wave matching are routinely accepted practice (Hannigan, et al., 2016). But prior to 1985, pile driving acceptance was based only on static load testing and pile driving formulae.

Our fundamental task as designers and supervisors is to ensure that capacity and integrity of each pile installed meets the demands of the structure that it supports. To do this we need to establish a 'ground truth', albeit a locally-evidenced and locally-targeted ground truth capable of application to all piles in a foundation system. Dynamic formulae, properly modified and correlated, must be the vehicle for delivering local ground truth and ultimately being the principal basis for sign-off.

It is important to remember that each PDA test has direct relevance only to the individual pile which is tested. On a foundation-wide basis, the role of PDA tests is critical but subservient, and principally to provide the evidence of ground truth on which a correlated dynamic formula is founded.

Consequent implications for the foundation sign-off process, and for a new approach to establishing capacity reduction factors for driven piles will also be discussed.

2 ANALYSIS AND SYNTHESIS

The engineering design process can be generalized to comprise two phases

1. An analysis phase which includes collecting evidence, organization and processing of data
2. A synthesis phase in which the data collected and analysed is transformed into a connected design

Elsewhere analysis is defined variously as "The process of studying or examining something in an organized way to learn more about it, or a particular study of something" (Cambridge, 2021) or "The process of separating something into its constituent elements" (Oxford, 2021).

Similarly, synthesis is defined variously as "The act of combining different ideas or things to make a whole that is new and different from the items considered separately" (Cambridge, 2021) or "The combination of components or elements to form a connected whole" (Oxford, 2021).

2.1 Geotechnical design

The geotechnical design process is no different, and proceeds through the same analysis and synthesis phases. We explore, observe, sample and test (analysis) and then draw all this together as best we can in the design process, trying to transform the diverse information into simple models which are amenable to solution.

In this section, we will consider the design for a project to be supported on driven piles, and the process of designing a single pile to support a column load.

The purpose of this section is only to provide an analogy to a process that is well understood as a basis for subsequent argumentation regarding the role of dynamic pile formulae.

In the analysis phase of our pile design case, activities include:

- Desktop studies
- Site investigation
- Insitu testing
- Laboratory testing

The synthesis phase activities include:

- Development of simplified site stratigraphy
- Assigning characteristic properties to the layers
- Selecting and applying a pile design method
- Considering the structural loads to establish a design pile size and length

Let's suppose that the pile is a floating pile in a deep clay layer, and that the design method adopted is the α method, in which the pile adhesion, τ at any depth is computed based on the local undrained shear strength, c_u and an adhesion factor, α

$$\tau = \alpha c_u \qquad (1)$$

Figure 2, reproduced after Coduto (1994), is typical of design charts which relate α to c_u. The Coduto curve is similar to the design chart which was included in the Appendix to AS2159 (1978).

Such design curves are often used by designers in the belief that they are based on some universal truth, unaware of the significant scatter which lies behind this design relationship.

Figure 3, includes the 124 test results from which the Coduto relationship was developed. Based on the reference tests alone, it can be seen that if a designer uses this design curve, they could easily overestimate or underestimate adhesion by a factor of 2.

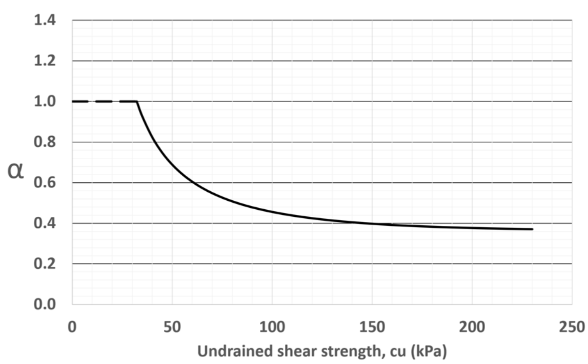

Figure 2. Adhesion factor α after Coduto (1994)

The scatter has an equivalent coefficient of variation (COV) of 29% about the best-fit curve, which is within the bounds of normal expectation for soil investigations. Kulhawy and Phoon (2002) suggest typical COVs of 10 to 30% for laboratory UU triaxial tests; 10-40% for field vane shear tests and 25 to 50% for SPT N tests, amongst others.

The development of this design curve is the embodiment of the synthesis process, in which the engineer necessarily has to reduce the diverse test results into a design algorithm, despite the up to four-fold range of the data on which the algorithm is based.

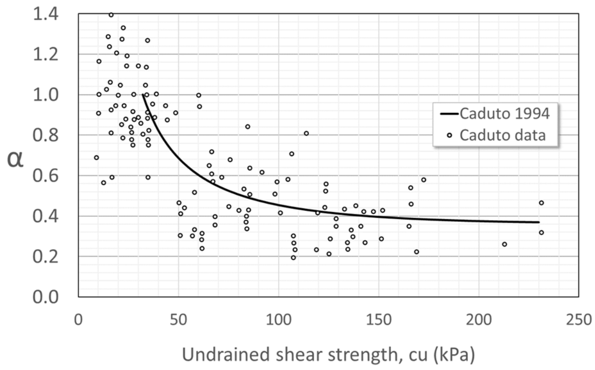

Figure 3. Data sets for adhesion factor α after Coduto (1994)

Coduto is only one of many researchers who have attempted to determine the relationship between α and c_u. Figure 4 compares the Coduto recommendations to other published relationships (Cherubini and Vessia, 2007).

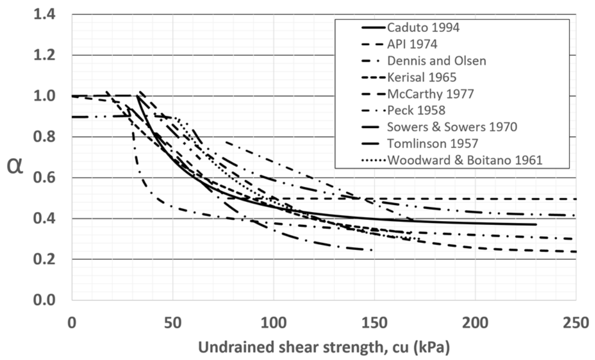

Figure 4. Comparison of published adhesion factor α relationships

It is clear that other researchers, investigating the same relationship but in other geologies and with other pile types have determined and propose distinctly different empirical relationships. This demonstrates that empirical relationships are not universal, but are limited to situations in which the prevailing conditions match the critical parameters on which that relationship are based.

Various authors have identified that α is not only dependent on undrained shear strength, but also a range of other physical properties such as clay mineralogy, Atterberg limits, moisture content, overconsolidation ratio, pile material, geometry, displacement ratio, and installation method.

Improved predictions of α are possible if some of the influencing factors are taken into account. Thus Nowacki et al. (1992) proposed the design chart shown in Figure 5. More sophisticated interpretations of data sets can lead to more reliable assessments with reduced uncertainty.

Putting aside whether the α method is the most suitable design method, it is clear that the variability of results will reduce as the number of variables decreases. By reducing the scope of a relationship to a specific site, or a small region within the same geological setting, the possibility of developing a more reliable relationship increases. Further, if that data is collected for a single pile type, then that will further increase the reliability of prediction.

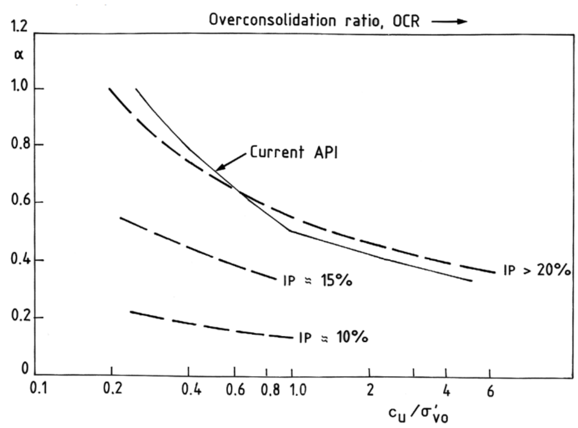

Figure 5. Prediction of adhesion taking into account OCR and plasticity index (IP)

It would be impractical for all but the most significant projects to undertake a program of static load tests to develop site-specific relationships. On the other hand, dynamic load tests conducted adjacent to bore locations provide a practical opportunity to develop the basis for evaluation of site-specific or region-specific α values. Our design-and-construct piling subcontractors have been doing this for decades in order to ensure that they can provide efficient and cost-effective designs. Of course, such an approach must be based on thoughtful, and geotechnically-informed Wave Equation analysis.

Figure 6 is a simulation of the sort of data set which might be generated for a set of tests within a particular geological formation and for a specific pile type. The coefficient of variation of this data set is now less than 5%. The grey zone represents the 95% confidence limit bounds. The lower bound aligns with the definition of characteristic value adopted in Eurocode EC7 (Bond et al., 2013).

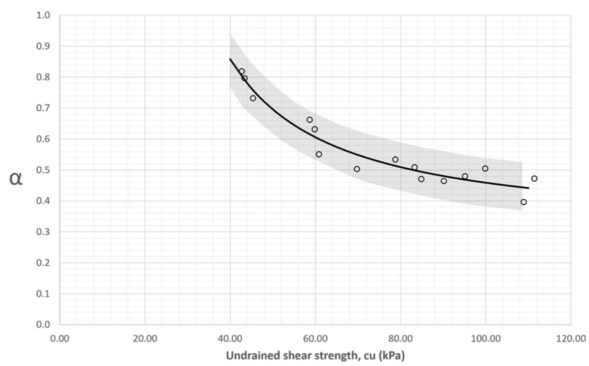

Figure 6. Simulated adhesion relationship for specific geology and pile type

2.2 Key findings

To summarize, the key findings (*kf*) from this review are :

kf1. Geotechnical design relationships are generally based on data sets with wide scatter

kf2. Geotechnical engineers using these relationships must understand and account for the inherent uncertainty in their design (with appropriate reduction factors)

kf3. Empirical relationships are by nature not universal, but are specific to the particular conditions existing at the sites used for data collection

kf4. The reliability of design recommendations can be improved by reassessment of data sets taking into account critical influencing factors

kf5. The reliability can also be improved by developing local relationships for a specific geological formation and particular pile type.

kf6. Statistical analyses can be used to evaluate confidence limits and to determine characteristic values for design. This is fundamental to developing realistic geotechnical reduction factors

kf7. Even in the face of significant variability and uncertainty, engineers necessarily synthesize the individual test results into a unifying relationship (design method) so that the test data can be extrapolated to locations not specifically tested

kf8. The synthesis of test results is fundamental to the design process

2.3 Dynamic Formulae and Pile Acceptance

As noted, the purpose of the previous section was only to provide an analogy to a process that is well understood as a basis for argumentation in this section regarding the role of dynamic pile formulae in regard to pile acceptance. Fragaszy et al., 1989 reviewed the reliability of 10 common pile driving formulae, including the Hiley Formula, in a study of 103 pile tests conducted in Oregon and Washington states by consultants and their departments of transport.

Because of the traditional use of the Hiley Formula in Australia, only the Fragaszy results in relation to this formula will be discussed here.

The Hiley formula is based on a simple theoretical model of pile driving characterized as the inelastic collision with coefficient of restitution, n of 2 masses (the hammer, W_h which drops from a height h with drop efficiency e_h and the pile and helmet, W_p). The pile experiences a permanent set, s , and three components of transient movement, C. Equation (2) predicts the pile capacity, R_u.

$$R_u = \frac{e_h W_h h}{s + (C_1 + C_2 + C_3)} \cdot \frac{W_h + n^2 W_p}{W_h + W_p} \tag{2}$$

The basis of this equation is to estimate the striking hammer energy, determine the energy transferred to the pile, and then to equate that to the work expended in the transient and permanent movements of the pile. With the exception of the Gates Formula (Gates, 1957), pile driving formulae are based on some simplified energy balance equation.

That being said, two key parameters in the Hiley Formula – drop efficiency, e_h and coefficient of restitution, n can only be determined empirically[1].

Subsequent modifications of the traditional Hiley Formula, which substitute PDA-computed energy transfer, EMX, have been proposed by Broms (1989) and Paikowsky and LaBelle (1994). These modifications were well established in general practice prior to these publications, but with the addition of a correction factor, χ as shown in Equation (3)

$$R_s = \frac{EMX}{\chi.(s + C/2)} \tag{3}$$

The empirical factor χ (a correction factor which correlates the formula to PDA or Wave Equation estimates of static resistance, R_s) is traditionally assumed to be constant, however, as will be shown, it can be any empirical function. Although the genesis of Eqn. (3) is theoretical, the use of a correlation factor implies a fundamental reliance on empiricism.

The comparison between Hiley Formula predictions (according to Eqn (2)) and static load tests reported by Fragaszy et al., 1989 are shown in Figure 7. The authors report that of the 103 tests, 38 were rejected because of incomplete data and 2 were rejected because the piles were broken. The remaining 63 usable tests included 6 timber, 20 prestressed concrete, 5 H-section, 4 pipe (open and closed), 7 concrete-filled pipe, 5 hollow concrete, and 16 Raymond step taper piles. Included in these tests were 41 piles driven in cohesionless soil, 11 in cohesive soil, and 11 where the sub-surface conditions consisted of layers of both cohesive and cohesionless soil.

In the case of Figure 7, the 63 individual test comparisons are shown against reference line of 1:1 correspondence – the 'design' line

$$\frac{R_{Hiley}}{R_{static}} = 1 \tag{4}$$

Supplementary lines are also shown for factors of 2 and 3 overestimation and underestimation for reference.

This design line shown in Eq (4) and Figure 7 is equivalent to the α design line in Figure 3, and demonstrates the same key findings *kf*1, *kf*2 and *kf*3 (see Section 2.2).

Although on average the predictions have low bias (the average ratio of Hiley to static capacity is 0.985), the scatter is very wide which is reflected by the coefficient of variation of 72.6%.

Given that the basic premise of monitoring the installation of piles is to reduce the uncertainties inherent in pile design, such a high coefficient of variation undermines the very premise on which pile monitoring is based.

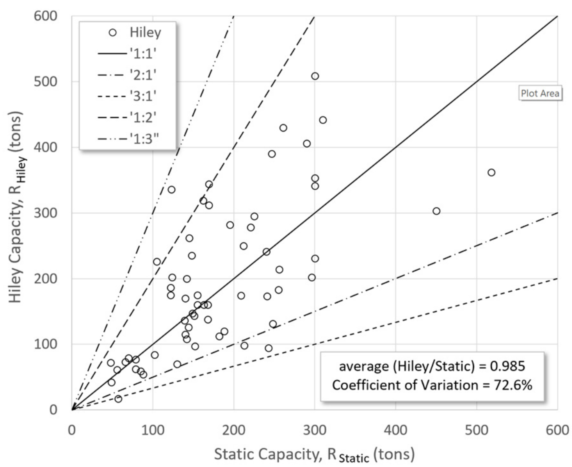

Figure 7. Hiley capacity vs Static capacity (after Fragaszy et al. 1989)

[1] And in practice are generally assumed rather than measured

Fragaszy et al. do not further analyse the data of Figure 7 in order to determine whether the spread is affected by pile type, hammer type or soil type.

The difficulties of comparing end of drive installation data (on which driving formulae are based) with static load tests undertaken some days or weeks later have been discussed elsewhere (Seidel, 2015a). The phenomenon of pile setup is well documented and is particularly associated with post-installation pile capacity increases in cohesive soils (Skov and Denver, 1998; Lee et al., 2010).

Ramey and Johnson (1978) report a data set of 153 piles comprising steel-H, steel pipe, concrete and timber piles. Five dynamic formulae, including Hiley Formula, are compared with static tests. All things being equal, one would expect the Hiley/Static capacity ratio to be relatively highest for the piles installed in cohesionless soils (which are generally not known for setup), and lowest for the piles exclusively in cohesive soils (for which end of drive capacity may be significantly less than the capacity measured later during static load testing).

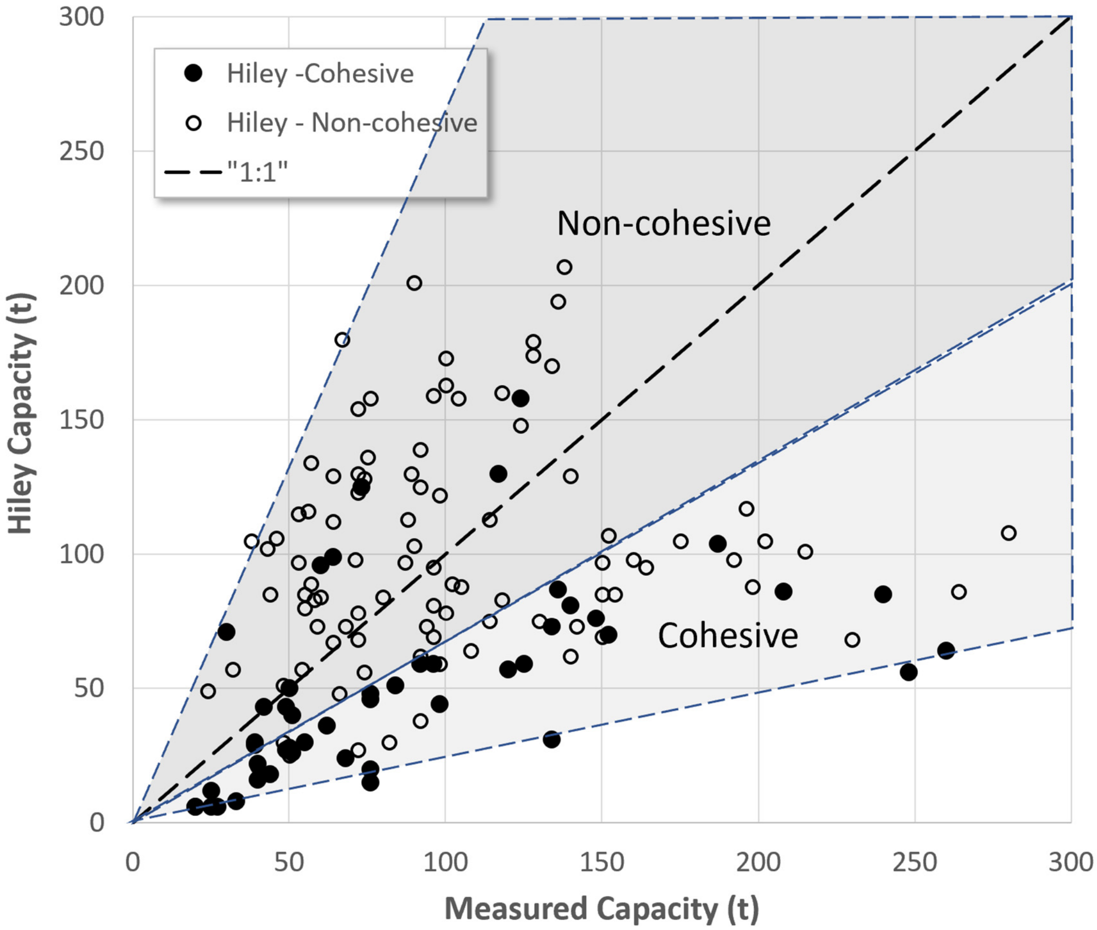

Figure 8. Hiley capacity vs Static capacity after Ramey and Johnson (1978)

Figure 8 shows that the spread of data is similar to the Fragaszy data set. However, the data broadly supports the expected differentiation of test results by soil type, with the spread of cohesive soil data generally being below the non-cohesive data, and showing the influence of pile set-up. Key findings *kf* 4 and *kf* 5 are demonstrated in Figure 8.

As noted in regard to the design parameter, α insightful consideration of the influencing factors (in that case overconsolidation ratio and plasticity index), can reduce uncertainty.

In order to further reduce the uncertainty in the application of dynamic formulae, it is important to understand that dynamic formulae do not predict static pile capacity but rather predict the total resistance of the soil to driving, R_t.

This resistance comprises two parts – a static component, R_s, and a transient dynamic component R_d. thus

$$R_t = R_s + R_d \tag{5}$$

As discussed in Seidel (2018a), the relative proportion of static and dynamic components of total driving resistance vary with the ease of driving. For hard driving, with low set, the total resistance is dominated by the static component. As driving becomes progressively easier (increasing set), the proportion of dynamic resistance increases as indicated schematically in Figure 9.

Referring to Equation (3), the correction function, χ[2] will be relatively small at low sets, and progressively increase as pile sets increase. χ can either be evaluated by wave equation analysis prior to piling, or evaluated empirically from dynamic pile testing, particularly at the commencement of a project.

[2] χ is referred to as Dynamic Reduction Function (DRF) in Seidel (2018)

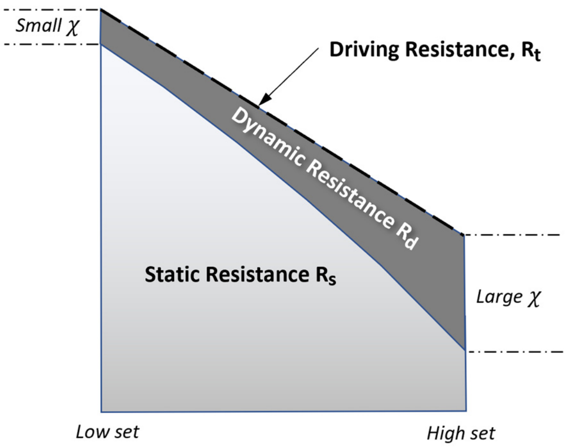

Figure 9. Schematic representation of resistance components during pile driving

The predicted relationship between Hiley Formula and static capacity, was evaluated using the Wave Equation program GRLWEAP (Rausche et al., 2004). for a near-shore piling project with a hydraulic hammer and steel pipe piles driven through bed sediments to refusal on variably weathered rock.

Figure 10 compares the uncorrected Hiley formula capacities inferred from the GRLWEAP predicted values of transferred energy, set and temporary compression with the analysed static capacities. The Hiley formula shown in Equation (3) was used with $\chi = 1$. Hammer strokes of between 0.6m and 1.5m were used in the analyses.

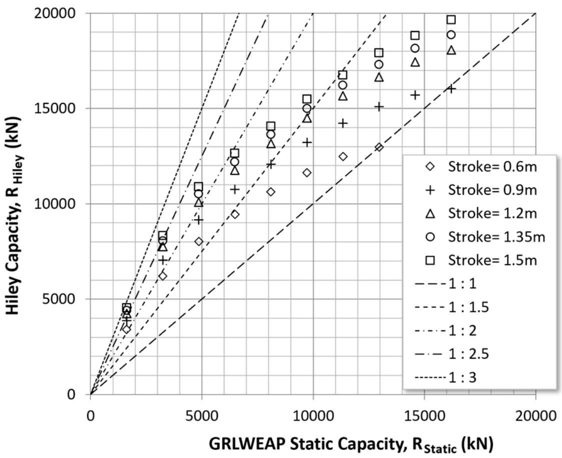

Figure 10. Comparison of Hiley Capacity and Static Capacity predicted by GRLWEAP

The similarity between Figures 7 and Figure 10 is noted. The Hiley formula overpredicts capacity by a factor of up to 3 at low capacities. Unlike the traditional Hiley Formula of Eqn (2), the modified Hiley Formula of Eqn.(3) generally does not underpredict capacity. It is evident that the degree of overprediction progressively decreases with capacity for each stroke, and approaches the equality line (Eqn. 4) at higher capacities.

It is clear that the ratio of Hiley capacity to Static capacity is not constant but varies between 1 and 3 even for a single project with defined soil model, pile type and geometry and hammer. Figure 11, plots the correction factor, χ as a function of pile set. There is a compelling dependency of χ on set across all hammer strokes analyzed, which results in a coefficient of variation of only 1%, and a very tight 95% confidence limit band.

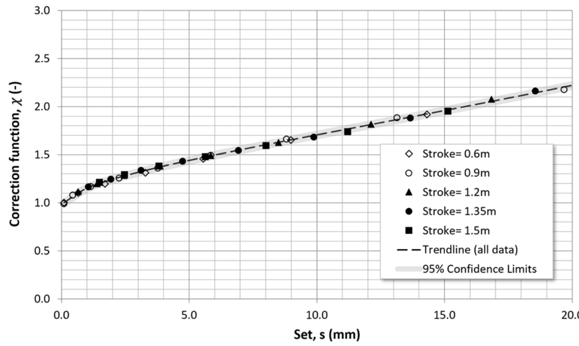

Figure 11. Correction factor function χ computed by GRLWEAP analysis

It is worth reinforcing that by understanding fundamentally the factors underlying the variability of Hiley Formula predictions (see Figure 7) it has been possible to develop a definitive correction factor and to adopt characteristic values which are only marginally below the best-fit relationship for the data (see Figure11). On the other hand, it is noted that the correction function computed here is theoretical. In practice, the data spread will inevitably be greater due to accuracy of set measurements (Denes et al. 2021), quality of PDA testing, uncertainty in wave equation matching (Seidel, 2015a) as well as inherent geotechnical variability. The confidence of any relationship will be reflected in the quality of these inputs. Figure 11 demonstrates the same key findings *kf*4, *kf*5 and *kf*6 (see Section 2.2).

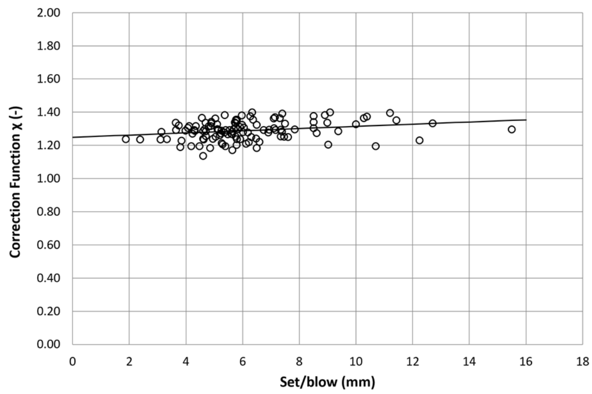

Figure 12. Correction factor function χ computed from project PDA data

Figure 12 presents correction function data for a large project. Each data point represents the wave matching solution for a PDA test. The spread is seen to be larger than for Figure 11, but a distinct trend is still shown. As χ is on the denominator of Equation (3), the upper bound of the confidence limit should be adopted as the characteristic value line.

The data represented in Figure 12 has a very low coefficient of variation (about the trendline) of only 4.2%. This was achieved with remote non-contact set measurements and with high quality PDA testing and mindful (non-automatic) wave equation matching. The consequence is the potential for higher capacity reduction factors (as discussed in the next section).

Figure 12 demonstrates the same key findings *kf*7 and *kf*8 (see Section 2.2). The PDA results are only relevant to the particular piles tested. It is necessary to synthesize these results into a unifying relationship from which a pile acceptance criterion can be developed. Locally correlated driving formulae provide the ground truth, and are fundamental to providing the basis for acceptance of any untested pile.

3 PILE ACCEPTANCE

The current practices in Australia for driven pile acceptance vary considerably. Generally, piles are driven to a capacity rather than a penetration criterion. What might be considered a normal practice is that a percentage of piles are PDA tested and matched with wave equation analysis, and this percentage defines the adopted capacity reduction factor, ϕ_g in accordance with the provisions of Section 4 of AS2159 (2009).

AS2159 (2009) introduced the concepts of a basic geotechnical reduction factor, ϕ_{gb}, an intrinsic test factor, ϕ_{tf} , and a testing benefit factor, K, which allowed the capacity reduction factor, ϕ_g to vary depending on the percentage of piles tested, p.[3]. For a typical project with low perceived risk category and low redundancy $\phi_{gb} = 0.56$, the computed capacity reduction factors are as indicated in Table 1:

Table 1: AS2159 (2009) Effect of increasing percentage of PDA tests

p (%)	3	5	10	25
ϕ_g (−)	0.69	0.72	0.76	0.80

Table 1 suggests that by the very act of testing, a reduced risk is imparted to the pile foundation system. The benefits of testing are manifest, and higher percentages of testing will increase the likelihood of detection of any piles with insufficient capacity. However, there is an implied assumption in AS2159 (2009) that the installation of the 75% to 97%[4] of piles not tested is informed by the results of the tests. This is generally not true, or is not implemented in a rigorous way.

One common practice is to treat the untested piles as unrelated to the tested piles. The tested piles must demonstrate a capacity in excess of the contractual requirement. The untested piles must pass a predefined acceptance criterion in the specification, such as the traditional Hiley Formula. In this approach, the valuable learnings of the PDA tests do not inform acceptance of the untested piles.

The second common practice is to adopt a maximum set criterion which corresponds to the set measured for a tested pile with the target capacity, or inferred if the tested capacity exceeded the target. This implicitly assumes that delivered energy efficiency is constant and that the set-capacity relationship is fixed with no variation. We will see in the next sections that neither of these assumptions is correct.

It will be shown (see Figure 14) that transferred energy cannot be assumed to be constant. Figures 10 to 12 demonstrate that the set-capacity relationship is not constant. Any acceptance criterion must take into account the demonstrated variability of both the energy delivered and of the set-capacity relationship.

[3] Up to a limit of p = 25(%) of piles PDA tested

[4] i.e. (100 – p)%

3.1 Pile acceptance based on energy, set and temporary compression

Despite the certainty provided to the evaluation of χ , there still remain challenges in evaluation of static capacity, R_s, in Equation (3) which is reproduced here for convenience.

$$R_s = \frac{EMX}{\chi.(s + C/2)} \tag{3}$$

There are a range of techniques for measurement of pile set and temporary compression. The possibilities, their benefits and limitations are discussed in Denes et al. (2021).

However, the most significant challenge to implementation is EMX, the energy transferred to the pile. Although some modern hammers measure and report kinetic impact energy, KE, these represent only a small percentage of the existing hammer fleet. Even for these hammers, the challenge still remains to evaluate the loss of energy, $KE - EMX$, through the helmet, particularly for concrete piles, for which the pile cushion can be highly variable in thickness and stiffness, especially when the cushion properties change during driving.

Flynn and McCabe (2016) compare EMX and KE measurements for hydraulic hammers on 5 projects, and demonstrate the uncertainties with inferring EMX from hammer energy measurements, as shown in Figure 13. The Flynn and McCabe data indicates that the percentage of hammer energy delivered to the pile typically varied between 75% and 100%, but in isolated instances was as low as 50%. This variability is just the transfer variability. The variability of the hammer stroke and kinetic energy is not even included in this assessment.

For uninstrumented hammers, transferred energy can only be estimated on the basis of estimated ram drop height, assumed hammer drop efficiency, and an assumed loss of energy in transferring from the hammer to the pile. As just noted, this can be particularly challenging for concrete piles.

Seidel (2018b) presents the energy measurements for a project in which 155 PDA tests were undertaken on prestressed concrete piles over the extent of a large bridge project. The project is distinguished by the requirement for all piling and all PDA tests to be undertaken with a drop height of 0.5m. Despite this control on drop height, delivered energies averaged 69.2kJ (88% efficiency), but with a range of 37.5 to 91.0kJ (48% to 116%[5] efficiency). The coefficient of variation of EMX was 15.4%. The three projects that the paper details demonstrate energy ranges of 36% and 40% for the steel pile projects, and 77% for the concrete pile project.

Li et. al., (2022) report in regard to another case study “The driving hammer energy efficiency recorded variations of up to 40%, casting doubt on the use of achieved set as a means of inferring pile capacities”.

The histogram in Figure 14 presents the sequential EMX values for 51 of the 155 PDA tests.

[5] Efficiency greater than 100% is physically impossible, indicating that the true drop height was significantly in excess of 0.5m

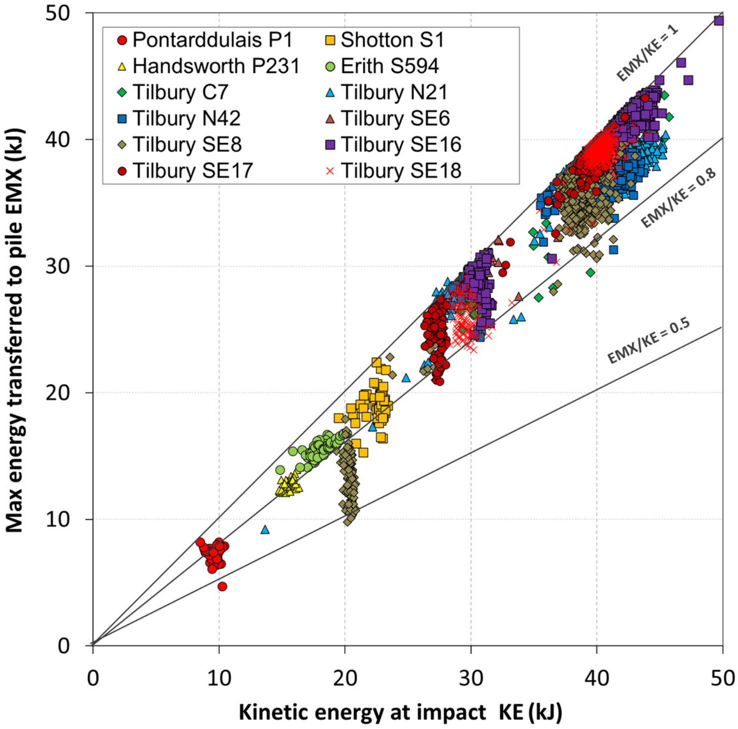

Figure 13. Relationship between kinetic energy at impact and energy transferred to pile

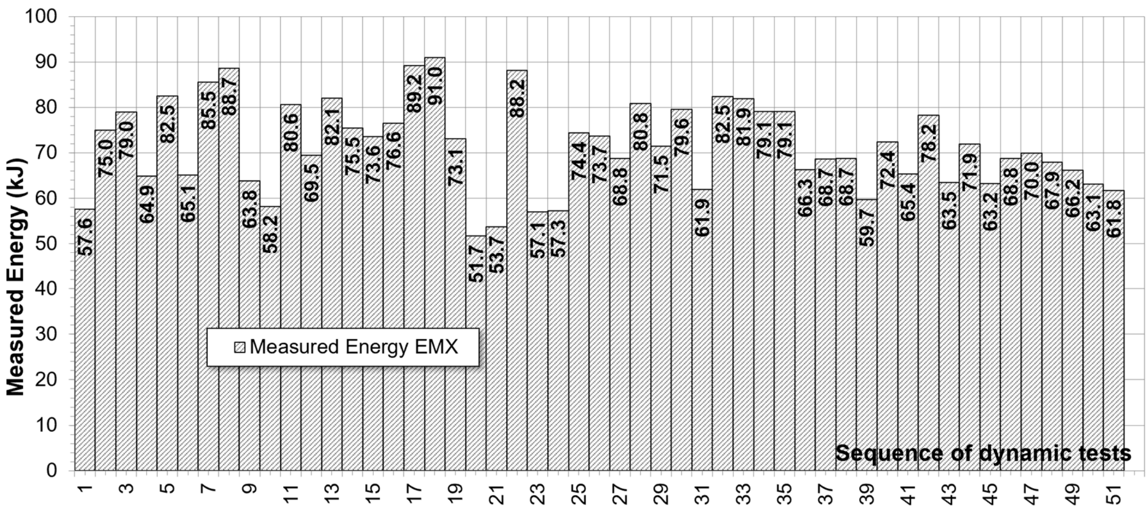

Figure 14. Longitudinal study of variability of delivered energy

The only possibility for individual pile energy measurements[6] is from high frequency remote non-contact displacement measurements. Peak pile velocities can be computed from differentiating the displacement-time signal. It is possible to identify low energy blows from low velocity measurements. Although EMX is related to the square of peak velocity VMX, the relationship is affected by variations in cushion stiffness, so that no unique and reliable energy can be computed in most cases.

Figure 15 is a representation of Figure 12, with upper and lower 95% confidence limits shown. Also shown are the implied geotechnical reduction factors based on a transferred energy uncertainty of 5%[7].

It is noted that the ϕ_g value of 0.70 is derived exclusively from a statistical assessment of the pile testing results and hammer performance and is independent of any assessment of the basic geotechnical reduction factor, ϕ_{gb}. This is a logical approach, because assessment of the installation process should be independent of any of the risk factors considered in developing project average risk rating, ARR and ϕ_{gb} in Table 4.3.2(C) of AS2159 (2009).

[6] Other than PDA testing or attachment of an accelerometer to each pile, both of which have practical issues

[7] The uncertainty in transferred energy evaluation, either from instrumented hammer or pile velocity measurements should be determined from each case.

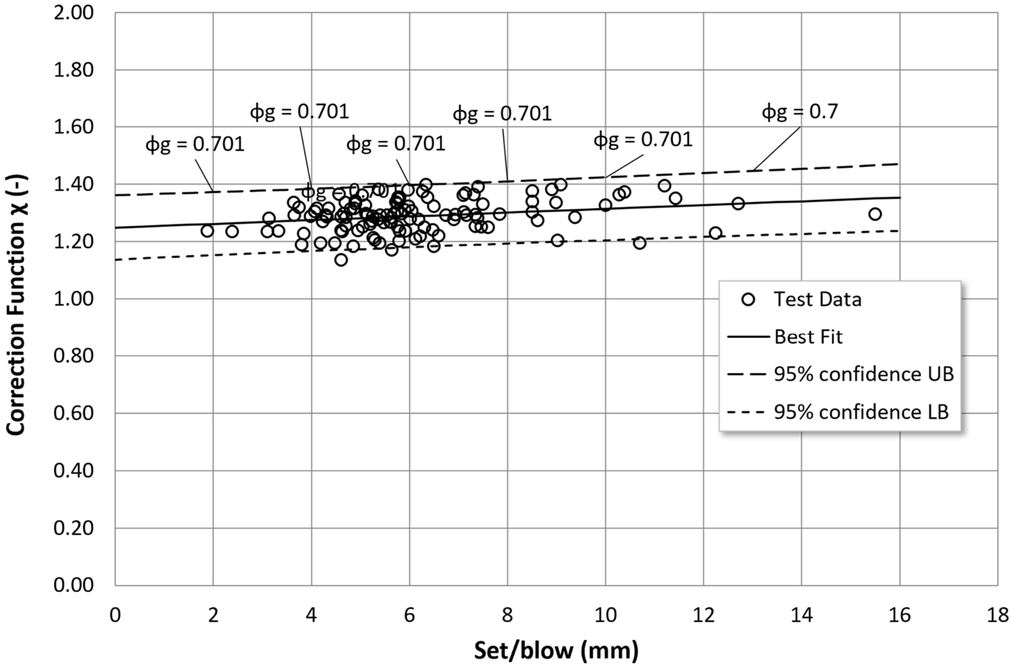

Figure 15. Confidence limits and inferred geotechnical reduction factors for Figure 11 data

3.2 Pile acceptance based on energy and set

Pile acceptance methods based on energy and set alone are fundamentally premised on use of bearing graphs correlated to field test results. Bearing graphs represent the relationship between capacity and pile set, which is of course dependent on hammer energy. Seidel (2015b) and Denes et al. (2021) discuss the importance of Energy-Capacity-Movement (ECM) relationships.

Figure 16 compares bearing graphs for 5 different hammer strokes. The inferred capacities at a blow count of 200 blows/m (5mm set) are shown for each stroke, ranging from 3525kN at 0.4m stroke to 5650kN at 0.8m stroke. Clearly, capacity can only be known if the delivered energy is known.

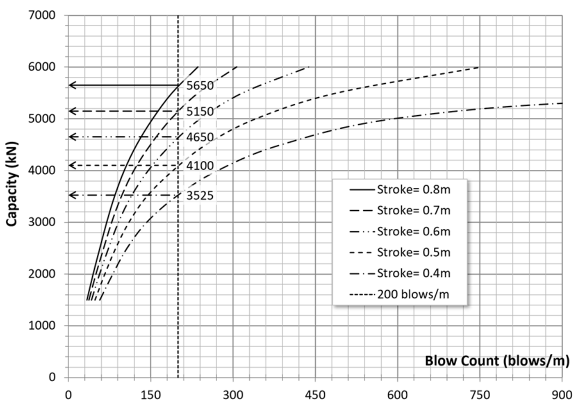

Figure 16. Traditional bearing graphs for a range of hammer strokes / delivered energies

As shown by Seidel (2018b) these bearing graphs can be normalized by the capacity at a specific blow count (in this case 200 blows/m). All the curves collapse to a near-unique relationship as shown in Figure 17, which is an interesting finding.

Three key points are shown on the graph at blow counts of 67, 200 and 870 blows/m (15mm, 5mm and 1.15mm set). What this graph shows is that for any given energy, the capacity at 15mm set will be 50% less than the capacity at 5mm set, and the capacity at 1.15mm set will be 50% more than the capacity at 5mm set. This relationship is unique to this data set, but equivalent relationships will hold for any site.

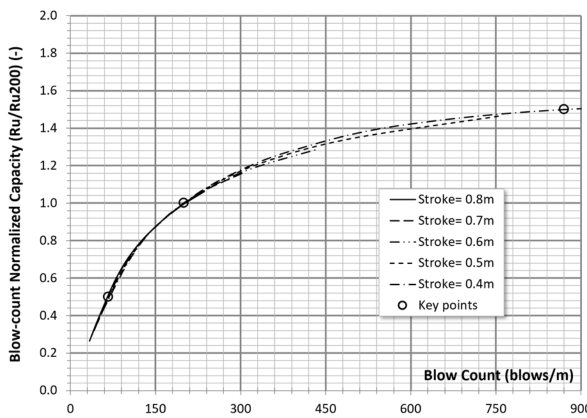

Figure 17. Normalized bearing graphs with normalization to capacity at nominated blow count

The benefit of this approach is that it does not require measurement of temporary compression.

However, all energy-based methods require determination of the energy delivered to the pile. The same comments regarding variability of transferred energy and the reliability of EMX estimates discussed in Section 3.1 apply equally to methods based on energy and set alone.

3.3 Pile acceptance based on force and set

Pile driving formulae are almost universally some expression of a Energy-Capacity-Movement (ECM) relationship, with assumed or measured energy being a fundamental input. As discussed, this is problematic, because of the potential variability of hammer performance, and the uncertainties involved in estimating energy transferred to the piles, even for instrumented hammers (Flynn and McCabe, 2016).

The Gates pile driving formula (Gates, 1957) is an empirical formula which is based not on energy, but the square root of energy, which leads to an equation which is not dimensionally correct. Regardless, the Gates formula, or derivatives of the Gates formula[8] continue to show comparatively the best performance in comparative studies in the United States where it is in common usage (Long, 2009).

To better understand why a non-dimensional equation performs well, consider Equation (6) for kinetic energy KE of the pile driving hammer with mass M_r, which strikes with velocity v_{imp} ,

$$KE = 1/2\, M_r v_{imp}^2 \tag{6}$$

from which it follows that

$$\sqrt{KE} \propto v_{imp} \tag{7}$$

Similarly, the energy transferred to the pile, EMX is computed from pile-top PDA measurements of force, $F(t)$ and velocity, $v(t)$ as

$$EMX = \int F.v.dt \tag{8}$$

One dimensional wave mechanics provides the following relationship which holds between F and v at the commencement of the dynamic event when no reflections from the soil have reached the pile head.

$$F = Z.v \tag{9}$$

where $Z = EA/c$ is the pile impedance, a function of the pile modulus, E, pile cross-section, A and pile wavespeed, c.

From Equations (8) and (9) it follows that there is an (imperfect) relationship between $\sqrt{EMX}$ and pile-top velocity, v, and pile-top force, F.

The Gates formula therefore effectively expresses a relationship between capacity and force or velocity. The FHWA Gates formula is

$$R_u = 1.75\sqrt{eE_r}\log(10N_b) - 100 \tag{10}$$

Without detailing all the components, the influence of blow count, N_b in blows/inch on modifying the estimate of static capacity, R_u , is shown in Figure 18 which has been shows the relationship in equivalent mm set units.

Seidel (2018c) presented results from a parametric study of 194 Wave Equation simulations involving 20 different piling hammers ranging from 7 to 168 tonne ram weight; 25 concrete or steel pile sections varying from 0.2m square concrete to 3m diameter and 80mm wall thickness steel; cushion stiffnesses, short and long piles, and varying resistance quakes, damping factors and resistance distributions. The study therefore ranged across an extreme range of piling scenarios.

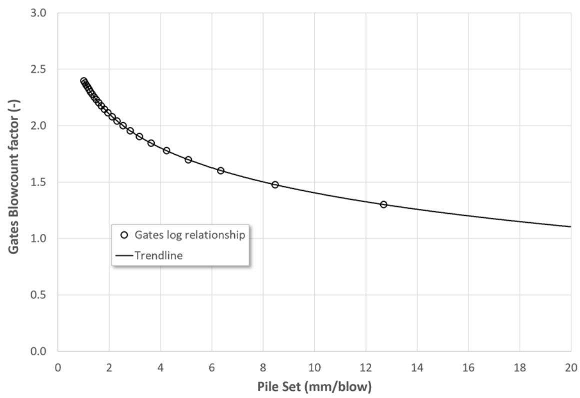

Figure 18. Gates blow-count capacity modifier

The purpose of the parametric study was to investigate the factors affecting normalized capacity with a view to developing reliable predictions of the relationship between normalized capacity[9] and pile set. Only moderate success was achieved, and a reliable prediction method is yet to be developed.

Figure 19 shows the results for 4 of the cases investigated. These cases include concrete and steel piles, and span ratios of (a) ram weights of 10.4, (b) pile impedances of 3.5; (c) damping factors of 3.4 and (d) quakes of 2.1. Despite these large variations in hammer and pile parameters, the indicated normalized capacity-set relationships have a limited range. Furthermore, in practice, the range of this relationship appears to be more limited than indicated by Figure 19, because the parametric study included hammer/pile combinations which could be considered extreme and outside the bounds of practical experience.

Regardless of whether it will be possible to reliably predict these relationships in advance with a simple algorithm, predictions for any project can be made in advance using wave equation analysis.

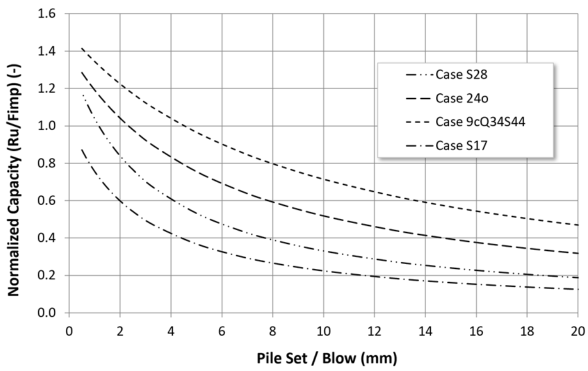

Figure 19. Wave Equation study of Normalized Capacity – Set relationship

Furthermore, any predicted relationship should be calibrated against the results of testing once piling commences. It is recommended that for the initial pile installations, PDA testing and wave matching analyses be undertaken at a range of pile sets – for instance at 10mm, 5mm and 2mm set per blow.

[8] For example FHWA-Gates, modified FHWA-Gates or WSDOT formulas
[9] Capacity normalized by the impact force, R_u/F_{imp}

Figure 20 is an example drawn from a project covering multiple dispersed bridge sites. The data includes both end of drive and restrike tests, but is constrained to a single pile geometry (1200mm diameter steel pipe piles) and hammer system.

Figure 20 also shows the upper and lower 95% confidence limits, and the corresponding geotechnical reduction factors, ϕ_g, which vary depending on pile set and the spread of data. For typical acceptance criteria with sets below 5mm /blow, ϕ_g is in excess of 0.7 and as much as 0.74.

Of course, the relationship is premised on measurement of impact force, F_{imp} , for every (untested) pile. This is not practical, however, peak impact velocity, v_{imp} , can be used as a proxy for F_{imp} using Equation (11).

$$F_{imp} = Z.v_{imp} \tag{11}$$

It is possible to measure v_{imp} with high frequency remote pile measurement devices (see Denes et al., 2021).

The basis for the popularity of the Gates formula can be seen in the similarity of the capacity-set relationships shown in Figures 18, 19 and 20. The empirical success of the Gates formula reinforces the essential validity of a force-based, but locally correlated acceptance criterion.

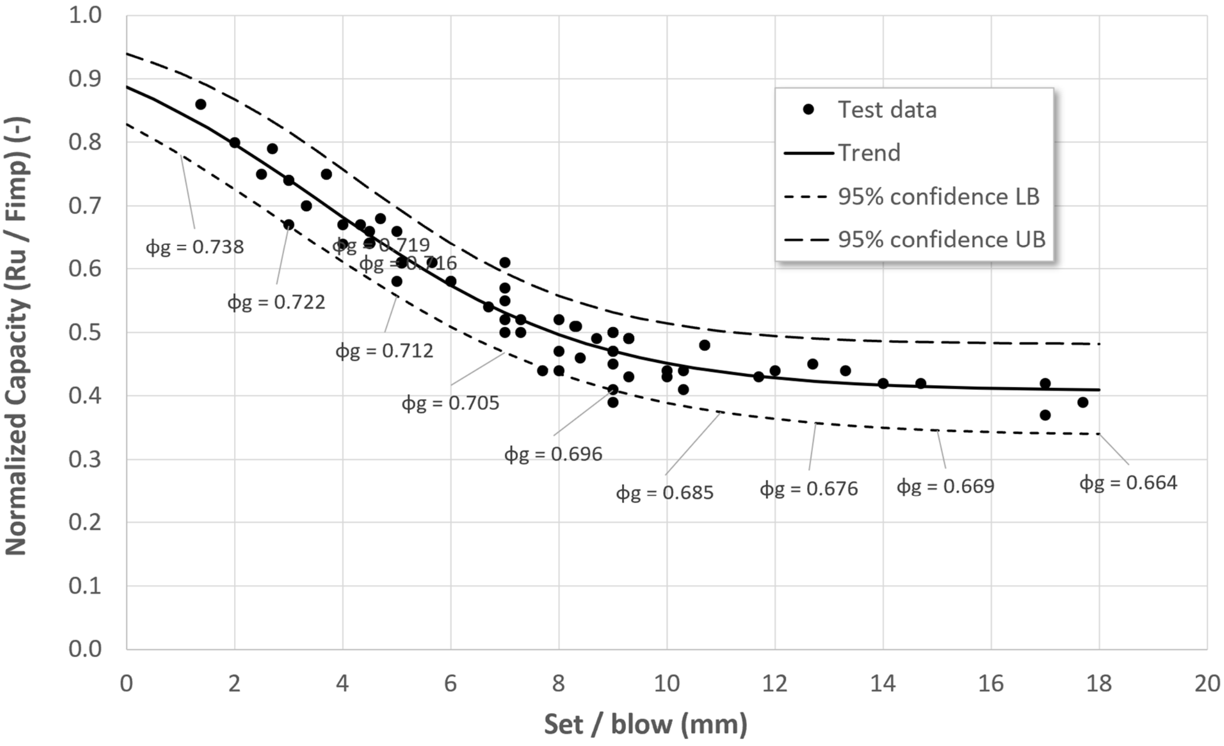

Figure 20. Normalized Capacity – Set relationship for multi-bridge project

4 CONCLUSIONS

Pile driving formulae have long been considered an unreliable approach to evaluation of pile capacity. Although this view may have had historical merit, it misunderstands the central role that pile driving formulae have in modern pile driving control.

Pile driving formulae are not universal truths, but are empirical formulations which have relevance only to the particular set of conditions (piling equipment, pile and soil) on which they are based.

In the modern context, pile driving formulae should have a central role in being the synthesis of the results of PDA testing and wave matching analysis. The pile driving formula, locally correlated against the test results is an expression of the local ground truth, and provides the basis for extension of the individual test results to all piles in the foundation system.

In order to undertake this successfully, the PDA tests must be of high quality and the wave matching must be conducted in a geotechnically relevant and thoughtful approach.

The paper has described three alternative approaches to implementation of pile acceptance criteria – two based on traditional energy-based approaches and a third based on measurement of pile impact velocity (being a proxy for impact force).

It has been shown here and elsewhere that the hammer energy delivered to piles can be extremely variable. The greatest challenge to implementation of the energy-based approaches is the evaluation of delivered energy to the pile for every pile. On the other hand, the alternative based on measurement of pile impact velocity (and pile set) is practically attainable.

Finally, it has been shown that these approaches provide a logical pathway to determining realistic capacity reduction factors, based on the reliability of the adopted empirical pile acceptance relationship, the reliability of the energy, force and movement measurements and the number of tests on which the relationships are based. These approaches do not rely on any evaluation of the basic geotechnical reduction factor defined in AS2159 (2009), which is in itself a logical outcome.

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