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An application of wave equation analysis program to pile dynamic formulae

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Abstract. Wave equation analysis programs (WEAP) such as GRLWEAP and TNOWave were primarily developed for pre-driving analysis. They can also be used for post-driving measurement applications with some refinements. In the case of pre-driving analysis, the programs are used for the purpose of selecting the right equipment for a given ground condition and controlling stresses during pile driving processes. Recently, the program is increasingly used for the post-driving measurement application, where an assessment based on a variety of input parameters such as hammer, driving system and dynamic behaviour of soil is carried out. The process of this type of analysis is quite simple and it is performed by matching accurately known parameters, such as from CAPWAP analysis, to the parameters used in GRLWEAP analysis. The parameters that are refined in the typical analysis are pile stresses, hammer energy, capacity, damping and quakes. Matching of these known quantities by adjusting hammer, cushion and soil parameters in the wave equation program results in blow counts or sets and stresses for other hammer energies and capacities and cushion configuration. The result of this analysis is output on a Bearing Graph that establishes a relationship between ultimate capacity and net set per blow. A further application of this refinement method can be applied to the assessment of dynamic formulae, which are extensively used in pile capacity calculation during pile driving process. In this paper, WEAP analysis is carried out to establish the relationship between the ultimate capacities and sets using the various parameters and using this relationship to recalibrate the dynamic formula. The results of this analysis presented show that some of the shortcoming of the dynamic formula can be overcome and the results can be improved by the introduction of a correction factor.

Keywords: dynamic formula; wave equation analysis; piles dynamic analysis; energy formula

1. Introduction

Pile dynamic formulae are the oldest methods that are still in common use amongst the piling practitioners and consultants according to several research surveys by AbdelSalam *et al.* (2009) and Fleming *et al*. (2008).

The derivation of the dynamic formula is based on the principle of Newtonian impact theory. A full detail of the derivation of the dynamic formula is given by Chellis (1961). These formulae are

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simple and useful in many projects where no or very limited number of piles are tested by higher order method such as dynamic testing.

Essentially these formulae establish the relationship between the driving resistance and set or penetration during pile driving. The formulae are generally classified into empirical, theoretical and combination of both. The theoretical dynamic formula, such as the well-known Hiley formula, takes into account the energy losses in the driving system (hammer, cap and cushion) as well as the losses in the pile due to elastic compression. It also assumes that the soil response is elasto-plastic. The Hiley formula (Hiley 1925) is given as

$$R_u = \frac{e_h W_h h}{s + c/2} \cdot \frac{W_h + e^2 W_p}{W_h + W_p} \tag{1}$$

Where R_u is total resistance or ultimate pile capacity, e_h is efficiency of hammer, W_h is weight of the hammer, h is stroke, e is coefficient of restitution for cushion (COR) and is a material property, W_p is weight of the pile, s is set and c is elastic compression (recoverable movement) of the pile cushion.

A simple and field usable form of the dynamic formula is expressed as

$$R_u = \frac{E_{\max}}{s + c/2} \tag{2}$$

Where $E_{\max}$ is maximum transferred energy, s is set and c is total elastic compression.

Paikowsky *et al.* (1994) and Broms and Lim (1988) proposed the Eq. (2) based on the actual energy evaluation by Pile Driving Analyser (PDA).

The $E_{\max}$ can be evaluated by PDA in the field accurately. The set and the temporary compression parameters can be measured directly manually during driving or more precisely by equipment proposed by Tokhi *et al.* (2011).

Experience and pile test data over the years have shown that the dynamic formula consistently over predict pile capacity compared to the reference static tests. The reason for this over prediction of capacity evaluation by the dynamic formula is that the formula does not take into account the dynamic component of the capacity. Hence a correction factor, f, can be used to adjust for this dynamic component similar to the damping parameter used in the Wave Equation Analysis methods (Tokhi *et al.* 2011). This factor 'f' is assumed to be a function of pile velocity and displacement and the expression for the Hiley formula can be modified as

$$R_u = f \cdot \frac{E_{\max}}{s + c/2} \tag{3}$$

Lowery *et al.* (1969) proposed such a factor to 'bring the formulas into agreement with the wave equation.

If the energy and displacement are measured, then the capacity can be estimated by substituting $s = D_{\max} - c$ into Eq. (2)

$$R_u = \frac{2E_{\max}}{D_{\max} + s} \tag{4}$$

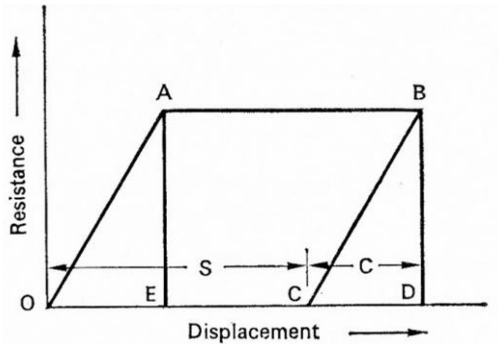

Fig. 1 Energy equilibrium equation relating to resistance and displacement of pile (Whitaker 1970)

Where R_u and D_{max} are ultimate capacity and maximum displacement, respectively. The GRLWEAP program can output calculated theoretical values of R_u, D_{max}, blow count and E_{max} that can be used to assess and compare the theoretical values from the GRLWEAP program and the dynamic formula. Furthermore, the factor f can be used to theoretically bring the dynamic equation into close agreement with the wave equation.

Inherently, pile dynamic formula treats pile as a rigid body and its interaction with soil is modelled as elasto-plastic similar to the wave equation, as shown in Fig. 1. In fact the dynamic formula in its simplest from can be derived by the area under the idealised elasto-plastic line on the plot.

2. Wave equation analysis

The theory of wave propagation provides the proper theory of pile driving. Wave equation was proposed nearly 150 years ago in 1866 by Saint Venant and Boussinesq for longitudinal impact of bars (Timoshenko and Goodier, 1951). Isaac (1931) was the first to point out the application of wave propagation theory to piles and developed a set of graphical charts and formulas to analyse the stresses and displacements in piles. Smith (1960) presented the mathematical method which, with some modifications, could be applied to pile driving problems and solved numerically by computers. Smith modelled the pile, hammer and cushion as a series of springs and the pile-soil interaction as elasto-plastic response shown in Fig. 2.

The stress propagation in a pile during a pile driving is given by the following governing hyperbolical differential equation

$$\frac{\partial^2 w}{\partial t^2} - c^2 \frac{\partial^2 w}{\partial z^2} = 0 \tag{5}$$

Where $c = (E_p / \rho)^{0.5}$, E_p is modulus of elasticity, ρ is density of pile material, $w(z, t)$ is the axial displacement of cross section at distance z and time t.

Eq. (5) is easily solvable by various mathematical methods. The mathematical solutions are often obtained for simple boundary conditions and they give an excellent insight into the behaviour of stress wave induced in solid media as result of hammer impact energy.

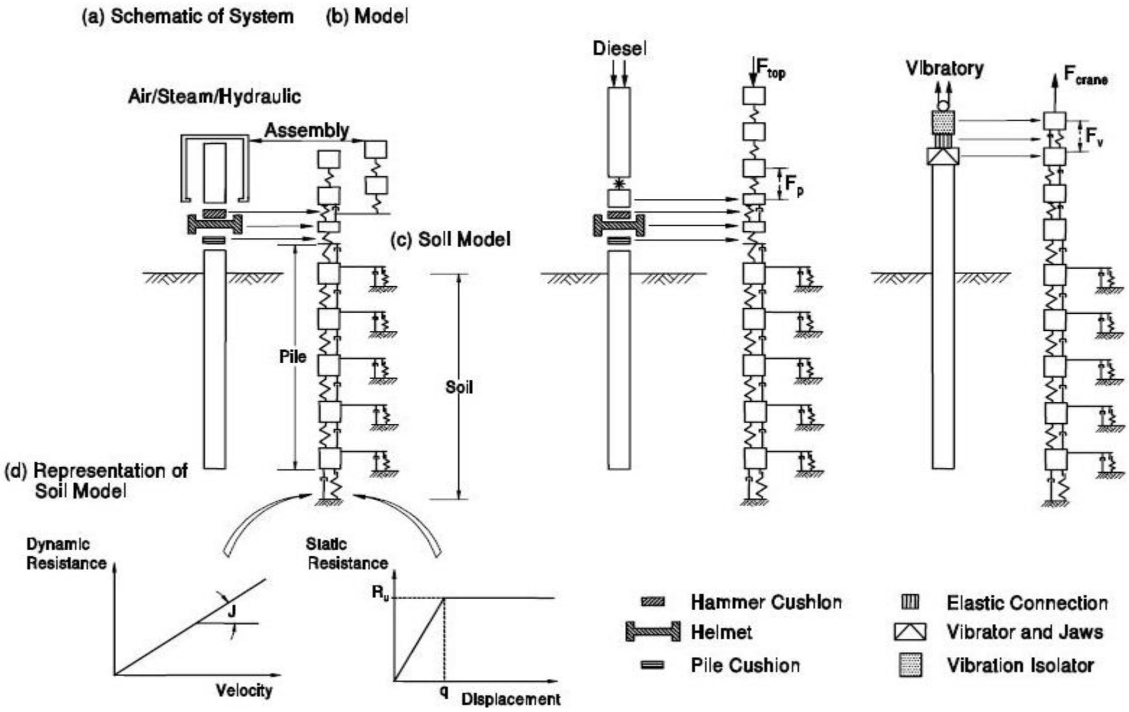

Fig. 2 GRLWEAP pile, soil and driving system (hammer, helmet & cushion) modelling (Pile Dynamic Inc. (PDI) 2005)

When friction resistances are introduced into the partial differential equation, as in Eq. (6), then the solution is neither simple nor practical except for very simple cases where the friction can be expressed as a function.

$$E_p A_p \frac{\partial^2 w}{\partial z^2} - P_p f_s = \rho A_p \frac{\partial^2 w}{\partial t^2} \tag{6}$$

Where, A_p is pile cross section area, P_p is pile perimeter and f_s is frictional force acting on perimeter by soil.

In the real physical world where ground shear resistance is present, the solution for the above differential equation is carried out by numerical finite difference method and in fact the Smith's approximation in itself turns out to be essentially a finite difference technique.

3. GRLWEAP analysis results

GRLWEAP™ software is pre-driving computational analysis tool for simulating pile response based on the solution of one-dimensional wave equation. Smith (1950) developed the numerical solution to the wave equation by discrete element idealisation of the hammer-pile-soil system as a series of mass, springs and dashpots. One of the first programs that were developed by Goble and Rausche in 1976 was named Wave Equation Analysis Program (WEAP) and later it was updated to WEAP87. Amongst the many available programs, currently GRLWEAP is the most widely used program and improvements such as residual analysis, pile-soil modelling and driveability analysis were incorporated in the later versions (Hussein 2004).

The main input data in GRLWEAP™ program are hammer, driving system (cushion) and pile details as well as soil parameters. It outputs driving stresses, hammer performance and the pile bearing capacities versus sets both graphically and in tabular format.

For the hammer, the program provides a large data file which is a compilation of basic properties of many impact hammers, powered by air, steam, hydraulic pressure, or diesel combustion. Once the model is setup, the program then performs an analysis of the model.

The driving system parameters, consisting of striker plate, cushion, helmet and pile cushion (for concrete piles) can be entered via the program's input screen. Again, the standard properties of the driving system are inbuilt into the program and can be modified if one wishes so. The cushions can be specified in the program by the area, elastic modulus and the thickness or by the stiffness alone. Pile cushion is only entered for concrete piles and it can be specified by the thickness and the program assumes the elastic modulus value for a new cushion. However, this assumption may not be correct for the End of Driving (EOD) conditions and the program manual suggests a value of approximately 2.5 times the early driving conditions, for example, 500 MPa instead of 200 MPa.

Pile cushions are only used for concrete piles. In the field during pile driving, the cushion properties change over the short life of the cushion material. For example, plywood may compress to half of its initial thickness and its elastic modulus may increase.

Pile input data consists of total length, cross-section area, toe area, elastic modulus and specific gravity. Many different pile profile options that are available are non-uniform, segmental and open ended piles. Pile mechanical splice and cracks can also be performed by entering 'tension slack', which allows for the extension of the splice with zero tension force. Tension slack was not applicable to this research study.

The coefficient of restitution (COR) for the material of the cushion as a decimal number range from 0.1 to 1.0. The cushion COR is a material property, which indicates the fraction of energy that is temporarily stored in the cushion during compression that is not lost. For man-made materials, if nothing else is known, a 0.8 value is a good estimate. For wood COR may be set to 0.5.

Although GRLWEAP performs calculation of soil static resistance in the analysis, but it is only an estimate and the values calculated is found by the authors to be non-consistent. So for the bearing graph analysis, a separate static analysis program DRIVEN was used to evaluate the static shaft and toe resistances. Although the pile's ultimate capacity is changed, the skin and toe resistance distribution is maintained constant throughout the analysis. When the driving process continues, it is not necessary to recalculate the quake, damping parameters and the skin and toe resistance distribution.

To undertake a GRLWEAP parametric analysis, it was considered necessary to study the effect of different pile-hammer and soil combinations. A summary of the cases that were studied is shown in Table 1.

The numerical simulations were carried out using common hammers such as Delmag, Junttan and MKT together with three ram weights at approximately 30, 50 and 100 kN. Also for each hammer three different drop heights at 0.25 m, 0.5 m and 1m were adopted to produce a wide range of energies and capacities, as shown in Table 2.

Table 1 Soil combinations and cross-section profiles used in the GRLWEAP analysis

Bottom layer →		Clay			Sand		
Top layger ↓		Soft	Stiff	Hard	VL	MD	VD
		17-19 kN/m^3	19-21 kN/m^3	22-23 kN/m^3	11-16 kN/m^3	17-20 kN/m^3	20-23 kN/m^3
		2-4	8-15	> 30	0-5	5-10	> 50
Clay	Soft	∨	∨	∨	∨	∨	∨
	Stiff	∨	∨	∨	∨	∨	∨
	Hard	∨	∨	∨	∨	∨	∨
Sand	VL	∨	∨	∨	∨	∨	∨
	MD	∨	∨	∨	∨	∨	∨
	VD	∨	∨	∨	∨	∨	∨

* VL = Very loose; MD = Medium dense; VD = Very dense

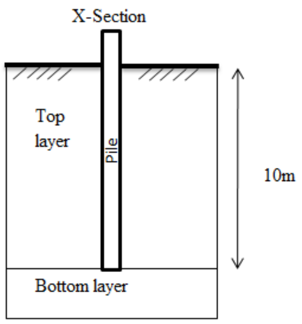

Table 2 Hammer details

Hammer	Model	Ram weight (kN)	Max. stroke (m)	Rated energy (kNm)
Delmag	D15	29.43	1.2	35.32
	D19-42	49.10	1.2	58.86
	D36-13	98.10	1.2	117.72
Junttan	HHK3	14.69	2.5	36.75
	HHK5	17.8	3.3	58.65
	HHK10	35.29	3.22	113.69
MKT	S-8	35.60	1.0	35.27
	MS500	48.95	1.2	59.68
	S-20	89.00	1.0	81.38

In the case of diesel hammers, the percentages of the maximum hammer strokes were selected. As a result, this systematic analysis produced a total of 1944 cases representing all possible combinations of hammer, pile and soil types, as well as different hammer strokes. After the analysis, the data were sifted thoroughly and collated into an excel database. The hammer and pile cushion details that were used in the GRLWEAP analysis are presented in Table 3 below.

Table 3 Hammer and pile cushion details

	Concrete pile							Steel pile			
	Delmag *Single acting diesel*		**Junttan** *Single acting hydraulic*			**MKT** *Single acting air*		**Delmag** *Single acting diesel*	**Junttan** *Single acting hydraulic*		**MKT** *Single acting air*
	Hammer cushion	Pile cushion	Hammer cushion	HHK10	Pile cushion	Hammer cushion	Pile cushion	Hammer cushion	Hammer cushion	HHK10	Hammer cushion
Material	Nylon	Plywood	Alum/ Conbest	Alum/ Conbest	Plywood	Alum/ Conbest	Plywood	Nylon	Alum/ Conbest	Alum/ Conbest	Alum/ Conbest
Area (cm^2)	1465.5	1225	1590.3	4418.1	1225	-	1225	1464.5	1590.3	4418.1	-
Elastic Mod. (MPa)	3654.2	207	2500	2500	207	-	207	3654.2	2500	2500	-
Thickness (mm)	50.8	94	200	210	94	-	94	50.8	200	210	-
COR	0.8	0.5	0.9	0.9	0.5	0.8	0.5	0.8	0.9	0.9	0.8
Stiffness (kN/mm)	10534.6	269.8	1987.9	5259.6	269.8	6329	269.8	10534.6	1987.9	5259.6	6329

Table 4 GRLWEAP resistance parameters for non-cohesive soils

Soil type	SPT N	Friction angle	Unit Weight	β	N_t	R_s	R_t
		Deg.	kN/m^3	-	-	Limit (kPa)	
Very loose	2	25-30	13.5	0.203	12.1	24	2400
Loose	7	27-32	16	0.242	18.1	48	4800
Medium	20	30-35	18.5	0.313	33.2	72	7200
Dense	40	35-40	19.5	0.483	86.0	96	9600
Very Dense	50+	38-43	22	0.627	147.0	192	19000

*B = Bjerrum-Burland beta coefficient, Nt = Toe bearing coefficient, R_s = Shaft resistance, R_t = Toe resistance

The soil types and their consistencies were also based on the parameters given in the GRLWEAP program and range of cohesive and non-cohesive soils with various consistencies were selected. A summary of the consistencies for the cohesionless soil is shown in Table 4. For cohesive soils, the shaft and bearing soil resistance as a function of soil types are given in Table 5.

In the GRLWEAP analysis, shaft and toe quakes and damping are the four Smith soil model parameters that are used to model the soil dynamic behaviour. The program standard quake and damping parameters were selected.

Table 5 GRLWEAP resistance parameters for cohesive soils

Soil type	SPT N	q_u	Unit Weight	R_s	R_t
		kPa	kN/m^3	kPa	
Very soft	1	12	17.5	3.5	54
Soft	3	36	16	10.5	162
Medium	6	72	18.5	19	324
Stiff	12	144	19.5	38.5	648
Very stiff	24	288	22	63.5	1296
Hard	32+	384+		77	1728

* q_u = ultimate soil capacity, R_s = shaft resistance, R_t = toe resistance

In the Smith model, the pile is discretised into lumped masses interconnected by springs. The soil resistance to driving is represented by a series of spring and dashpots. The soil springs are assumed to behave in an elastic- perfectly plastic manner, and the spring stiffness is defined by ratio of the maximum elastic deformation or quake Q. Damping coefficients were introduced to account for the viscous behaviour of the soil.

GRLWEAP is capable of producing results both in graphical and tabular data formats and the Fig. 3 below shows a typical graphical output.

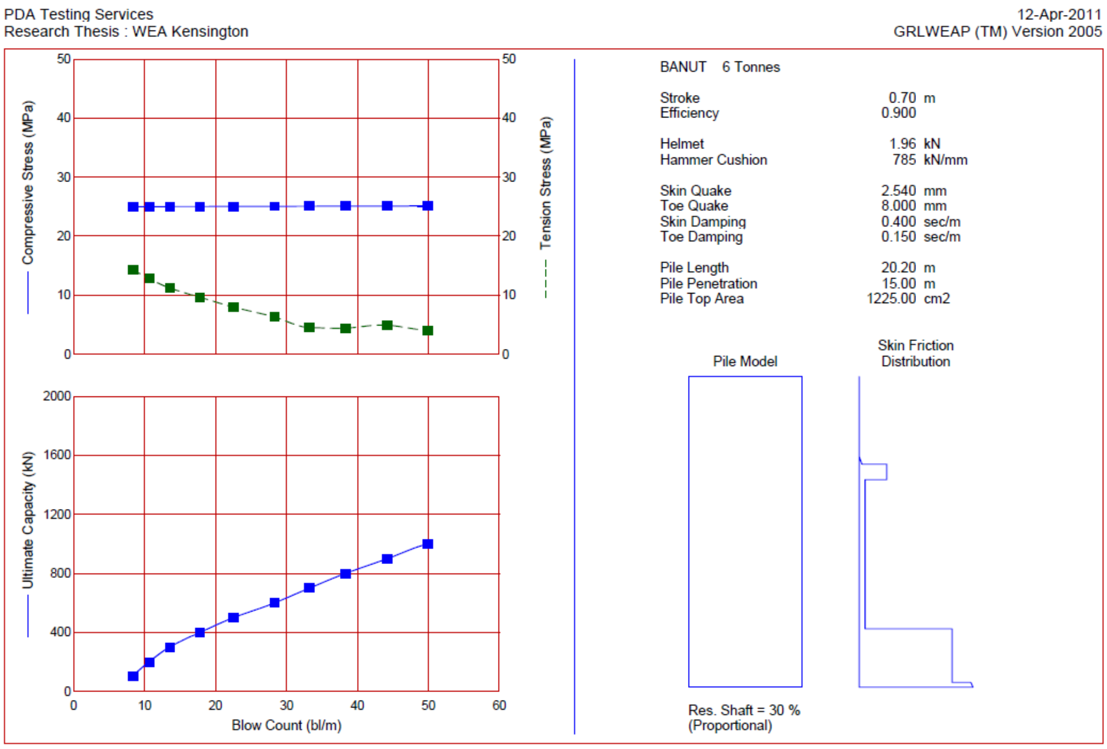

Fig. 3 GRLWEAP graphical output of results

In summary, the results of the GRLWEAP numerical parametric analysis for various hammers and soil combination as well as pile types were collated in a spread sheet and analysed for the following:

(1) Three hammer types: Delmag, Junttan and MKT. Within each type, three different ram weights were analysed.
(2) Two different piles: precast concrete and steel tube pile with similar effective sizes.
(3) For each hammer type and ram weight, three drop heights were analysed.
(4) Three soil resistance models with consistencies of soft, stiff and hard were used for the cohesive soils, and three (very loose, medium dense & very dense) for non-cohesive soils were used.
(5) GRLWEAP analysis results were presented on graphs.

3.1 Delmag hammers

The maximum hammer energies for the three rams with the weights of 3, 5 and 10 tonnes with drop heights at 0.25 m, 0.5 m and 1m were plotted against maximum ram velocity. Delmag diesel hammers seem to perform well in driving both concrete and steel piles, particularly the heavier 10 tonne hammer. All three hammers perform consistantly at low strokes and the trend is fairly predictable. It can be seen from Figs. 4 and 5, that there is reasonable correlation between the maximum energy and the ram impact velocity. Overall, the performance of Delmag seems to be correlating well.

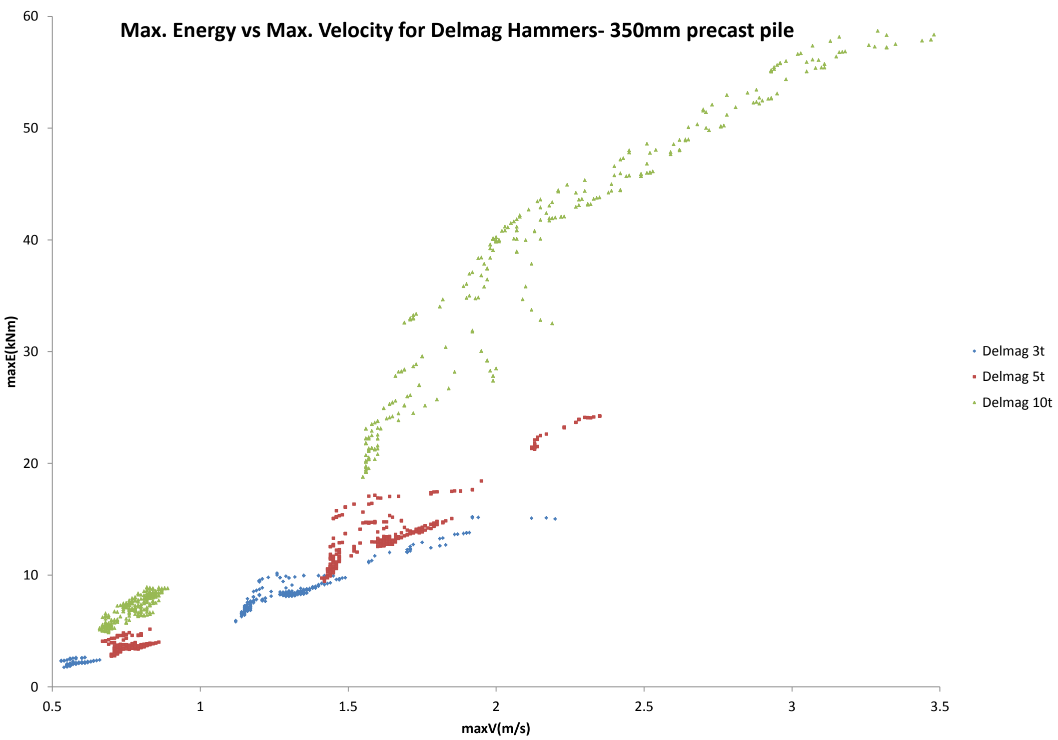

Fig. 4 Maximum energy versus velocity for Delmag hammer - concrete pile

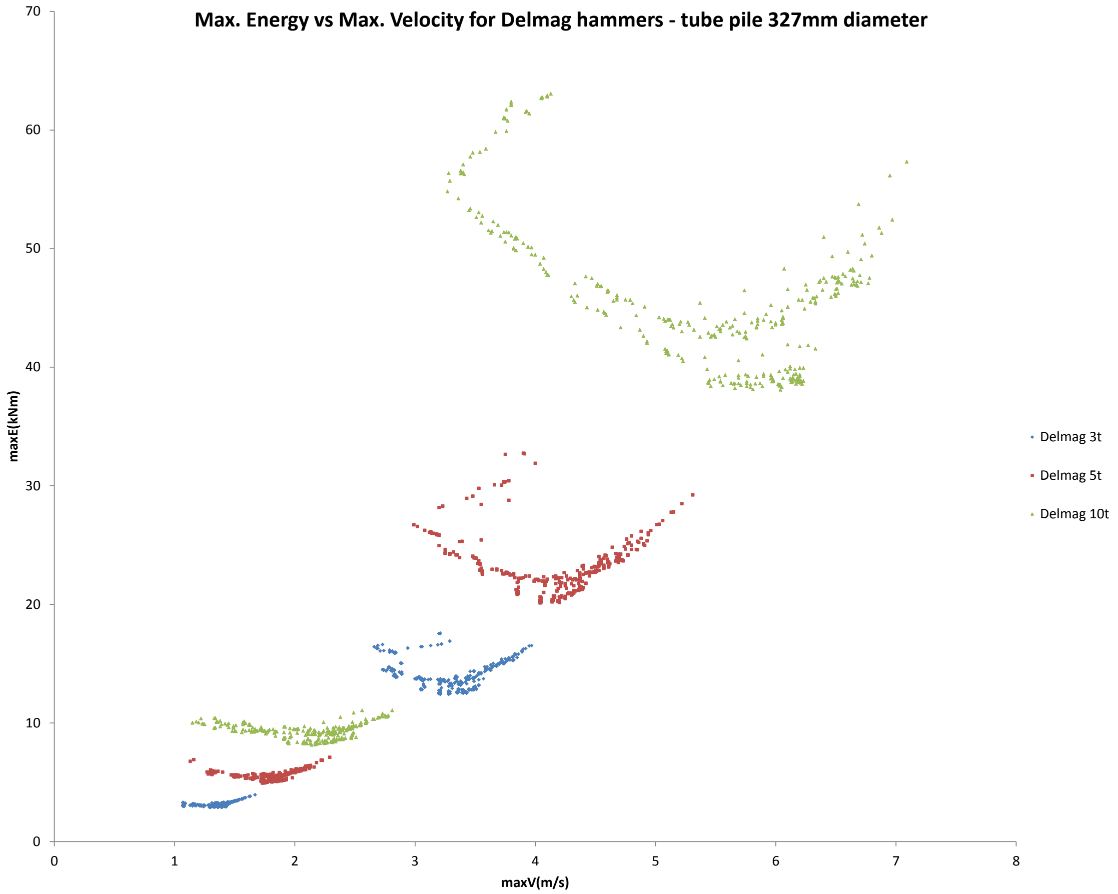

Fig. 5 Maximum energy versus velocity for Delmag hammer - tube pile

On the other hand, the performance of the Delmag for steel pile is unusual for low soil resistance or relatively high set values calculated from GRLWEAP. There seems to be a contradiction in the energy-velocity plot data, where it reflects like an inverted parabola. This gives an indication that there is a minimum energy at a specific set value at which the hammer is most efficient.

It should be born in mind that the pile performance in this case is not efficiencies, but rather the implicit relationship between the ram velocity and delivered energy. This relationship is important in the field where once ram velocity can be established and the ram energy can be inferred by the relationship that would make it possible to use the dynamic formulas with greater accuracy.

3.2 Junttan hammers

Junttan hammers' performances are similar to the Delmag hammers for steel piles, but its performance is relatively poor for the precast piles as the data is widely scattered and no clear trend can be established. However, the velocity-energy graph in Fig. 7 for the steel pile seems to indicate there is a very good correlation and the relationship may be estimated by the given equation on the graph.

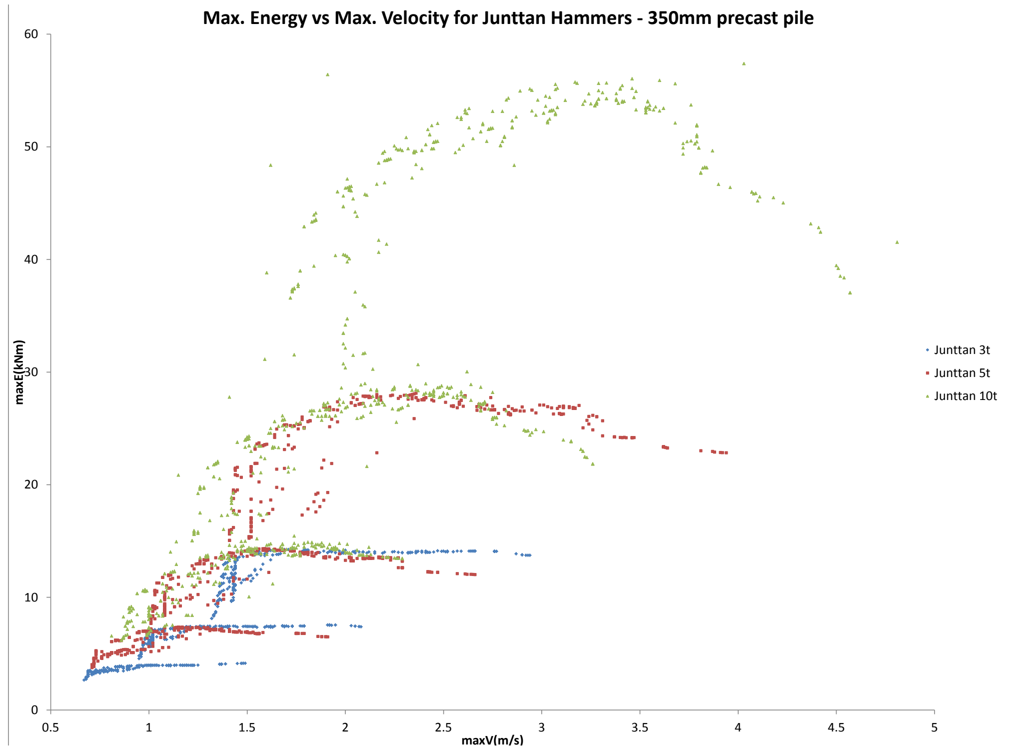

Fig. 6 Maximum energy versus velocity for Junttan hammer - concrete pile

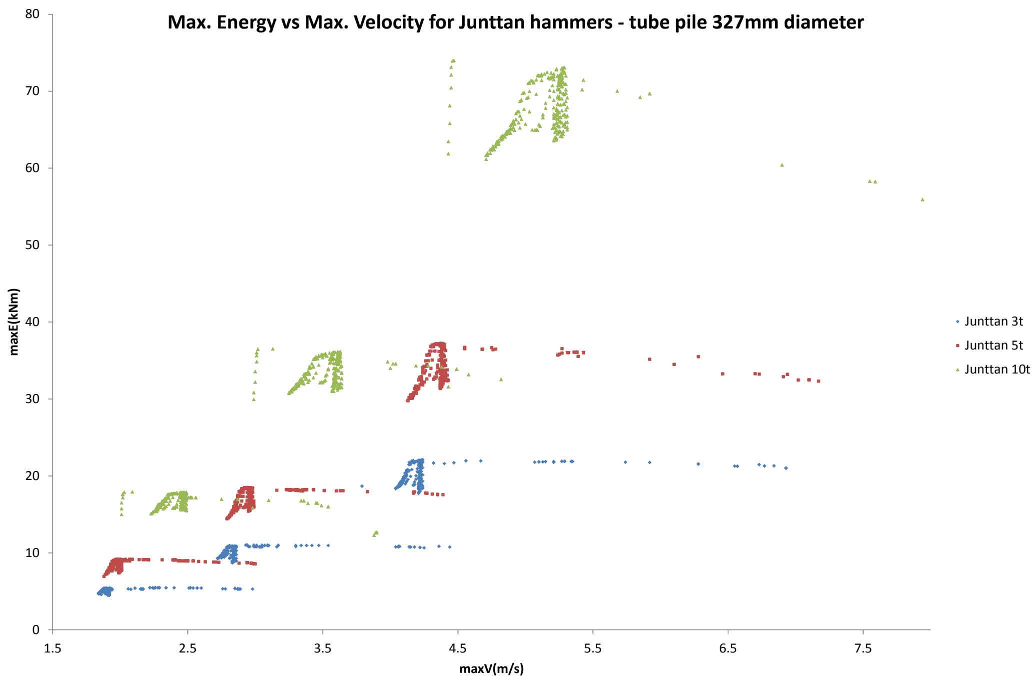

Fig. 7 Maximum energy versus velocity for Junttan hammer - tube pile

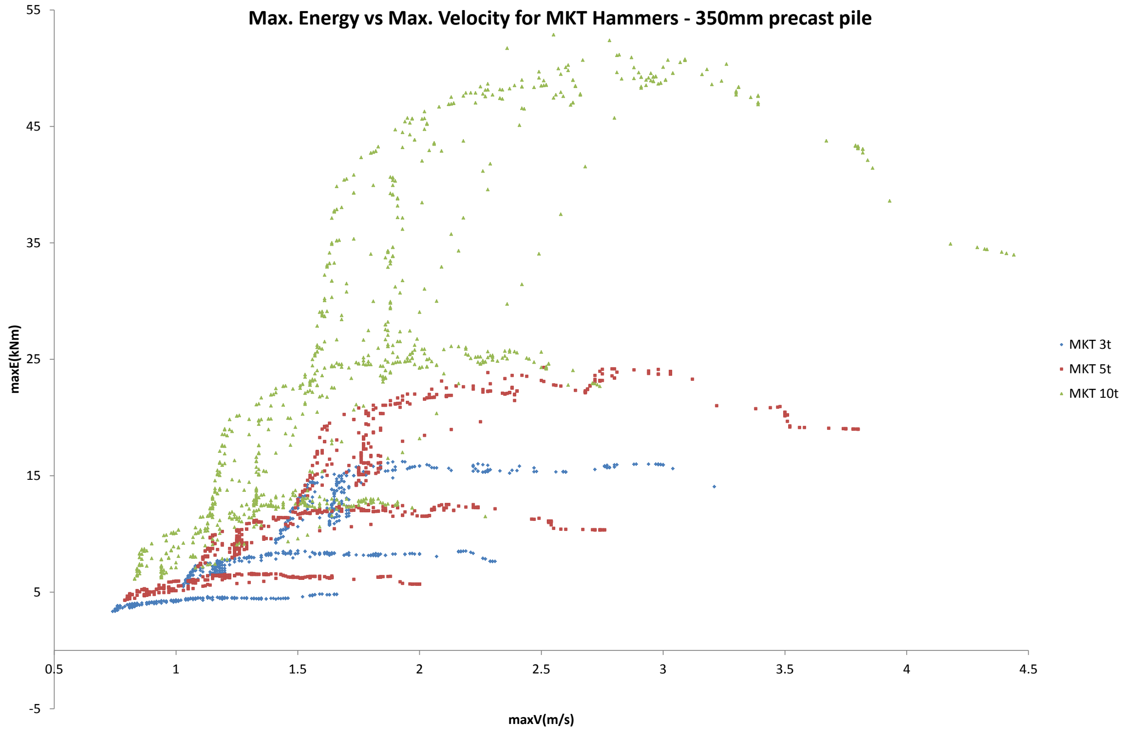

Fig. 8 Maximum energy versus velocity for MKT hammer - concrete pile

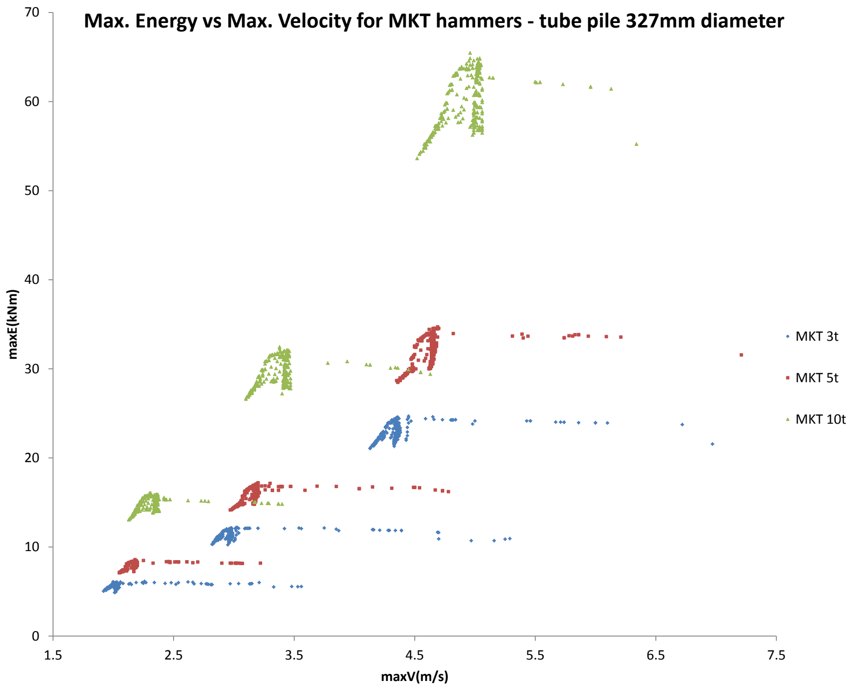

Fig. 9 Maximum energy versus velocity for MKT hammer - tube pile

Table 6 Summary of range of HCF (*f*) for various driving system and soil types

	Precast Concrete Pile					Tube Steel Pile				
		Top	**Bottom**	**HCF***	**Trend**		**Top**	**Bottom**	**HCF**	**Trend**
Delmag	3 tonne	S	S-H	1.51-1.07	Initially steep-linear then horizontal. Nearly constant for low hammer energies.	3 tonne	S	S-H	2.07-1.25	Linear then flat. For low energies initially linear then negative linear.
			VL-VD	1.57-1.00				VL-VD	2.03-1.13	
		H	S-H	1.58-1.06			H	S-H	2.07-1.30	
			VL-VD	1.59-1.02				VL-VD	2.06-1.23	
		VL	S-H	0.95-1.31			VL	S-H	1.07-1.57	
			VL-VD	0.92-1.42				VL-VD	1.08-1.72	
		VD	S-H	0.90-1.19			VD	S-H	1.05-1.38	
			VL-VD	0.90-1.32				VL-VD	1.06-1.60	
Delmag	5 tonne	S	S-H	1.71-1.13		5 tonne	S	S-H	2.29-1.40	
			VL-VD	1.70-1.06				VL-VD	2.26-1.26	
		H	S-H	1.72-1.12			H	S-H	2.25-1.50	
			VL-VD	1.71-1.00				VL-VD	2.28-1.30	
		VL	S-H	0.95-1.37			VL	S-H	1.11-1.68	
			VL-VD	0.95-1.51				VL-VD	1.12-1.88	
		VD	S-H	0.95-1.23			VD	S-H	1.09-1.46	
			VL-VD	0.95-1.39				VL-VD	1.10-1.74	
Delmag	10 tonne	S	S-H	2.09-1.11	Linear then flat. Almost constant for low energies.	10 tonne	S	S-H	2.38-1.28	Power. Negative linear for low energies.
			VL-VD	2.04-1.07				VL-VD	2.26-1.19	
		H	S-H	1.12-2.11			H	S-H	2.40-1.28	
			VL-VD	2.09-1.09				VL-VD	2.35-1.25	
		VL	S-H	1.65-0.95			VL	S-H	1.07-1.70	
			VL-VD	0.97-1.83				VL-VD	1.09-1.74	
		VD	S-H	0.94-1.48			VD	S-H	1.05-1.46	
			VL-VD	0.94-1.68				VL-VD	1.06-1.80	
Junttan	3 tonne	S	S-H	1.96-0.97	Linear then assymptotic	3 tonne	S	S-H	2.59-1.38	Linear then assymptotic.
			VL-VD	1.88-0.99				VL-VD	2.53-1.19	
		H	S-H	1.96-1.18			H	S-H	2.61-1.45	
			VL-VD	1.96-0.93				VL-VD	2.57-1.29	
		VL	S-H	1.60-0.93			VL	S-H	1.10-1.91	
			VL-VD	0.94-1.64				VL-VD	1.07-2.05	
		VD	S-H	0.95-1.26			VD	S-H	1.03-1.62	
			VL-VD	0.92-1.47				VL-VD	1.04-1.82	
Junttan	5 tonne	S	S-H	2.15-1.10		5 tonne	S	S-H	2.54-1.37	
			VL-VD	2.17-0.96				VL-VD	2.50-1.13	
		H	S-H	1.08-2.42			H	S-H	2.60-1.44	
			VL-VD	2.25-1.02				VL-VD	2.50-1.25	
		VL	S-H	1.90-0.93			VL	S-H	1.10-1.93	
			VL-VD	0.90-1.80				VL-VD	1.05-2.14	
		VD	S-H	0.94-1.34			VD	S-H	1.12-1.64	
			VL-VD	0.90-1.60				VL-VD	1.13-1.82	
Junttan	10 tonne	S	S-H	2.30-1.10	Parabolic then assymptotic.	10 tonne	S	S-H	2.41-1.41	Parabolic then linear assymptotic.
			VL-VD	2.30-1.01				VL-VD	2.35-1.15	
		H	S-H	1.05-2.25			H	S-H	2.47-1.45	
			VL-VD	2.13-1.18				VL-VD	2.41-1.27	
		VL	S-H	1.68-0.95			VL	S-H	1.14-1.67	
			VL-VD	1.83-0.94				VL-VD	1.07-1.99	
		VD	S-H	1.43-0.97			VD	S-H	1.13-1.52	
			VL-VD	1.70-0.95				VL-VD	1.14-1.83	
MKT	3 tonne	S	S-H	1.91-1.19	Parabolic then assymptots down.	3 tonne	S	S-H	2.50-1.48	Parabolic. Power for low energies.
			VL-VD	1.93-1.01				VL-VD	2.44-1.15	
		H	S-H	1.91-1.23			H	S-H	2.54-1.39	
			VL-VD	1.85-1.10				VL-VD	2.48-1.35	
		VL	S-H	1.00-1.46			VL	S-H	1.05-1.82	
			VL-VD	1.63-0.96				VL-VD	1.14-2.04	
		VD	S-H	0.98-1.28			VD	S-H	1.01-1.56	
			VL-VD	0.98-1.48				VL-VD	1.05-1.89	
MKT	5 tonne	S	S-H	1.15-2.25		5 tonne	S	S-H	2.42-1.43	
			VL-VD	2.19-1.00				VL-VD	2.39-1.14	
		H	S-H	1.24-2.25			H	S-H	2.47-1.47	
			VL-VD	2.25-1.06				VL-VD	2.40-1.35	
		VL	S-H	1.02-1.55			VL	S-H	1.11-1.81	
			VL-VD	1.03-1.60				VL-VD	1.13-2.03	
		VD	S-H	1.01-1.32			VD	S-H	1.08-1.52	
			VL-VD	1.56-1.02				VL-VD	1.09-1.87	
MKT	10 tonne	S	S-H	2.33-1.18	Parabolic then assymptots.	10 tonne	S	S-H	2.38-1.38	Parabolic. Nearly linear for VD soils.
			VL-VD	2.40-1.03				VL-VD	2.32-1.13	
		H	S-H	2.28-1.24			H	S-H	2.60-1.37	
			VL-VD	2.31-1.05				VL-VD	2.39-1.25	
		VL	S-H	0.97-1.52			VL	S-H	1.09-1.76	
			VL-VD	0.98-1.78				VL-VD	1.04-1.98	
		VD	S-H	1.04-1.37			VD	S-H	1.13-1.51	
			VL-VD	0.97-1.53				VL-VD	1.11-1.82	

*= Hiley Correction Factor   Clay: S= Soft, H= Hard; Sand: VL= Very Loose, VD= Very Dense

3.3 MKT hammers

The energy-velocity relationship for the MKT hammers is scattered and no clear correlation exists. It should be emphasised that although there may be a relationship on a small individual case bases, but overall, for a wide range of ground conditions and different rated energies and strokes, no clear correlation as can be seen for precast piles from the graph in Fig. 8.

MKT hammers seem to perform exceptionally well for the steel piles. Initially there is clear relationship, as can be approximated by regression line, and then it is a horizontal line indicating that the delivered energy cannot increase indefinitely and reaches a maximum regardless of the ram velocity. This is shown in the Fig. 9.

3.4 Hiley corrections factor (f)

Having analysed and established the relative performance of hammers in regards to the velocity-energy relationship, similar analysis were carried out for establishing the Hiley correction factor (HCF, *f*) for each soil types and hammer types. The results of the analyses for each soil and hammer types were produced by plotting the GRLWEAP and Hiley capacity ratio (or HCF) against the sets. Table 6 shows a summary of the range of the HCF for the various hammers, pile types and soil conditions.

Overall, the HCF values are fairly uniform across the different hammers and soil types. The performance of Delmag 5 t, Junttan 3 t and MKT 5 t look well in terms of the statistical analysis data, as shown in Table 7. For the steel tube pile, Delmag 3 t, Junttan 10 t and MKT 5t have performed better statistically. Also from the Table 7, it can be seen that the minimum and maximum range of HCF in the table are between approximately 1.0 and 2.9 with an average about 1.4. An interesting point about the HFC is that generally the lower unit value is derived for the sandy soil and the higher value applicable for the clay soils. This indicates that for clay soils the

Table 7 Statistical analysis of HCF based on the GRLWEAP data for each hammer type

	Precast concrete pile								
	Delmag			Junttan			MKT		
Weight	3 t	5 t	10 t	3 t	5 t	10 t	3 t	5 t	10 t
Min.	0.947	0.897	0.941	0.864	0.831	0.861	0.959	0.926	0.887
Max.	1.719	1.600	2.108	1.965	2.418	2.516	2.909	2.026	2.516
Mean.	1.228	1.164	1.316	1.303	1.384	1.418	1.327	1.282	1.332
STDEV	0.213	0.194	0.298	0.248	0.319	0.314	0.265	0.217	0.283

	Steel tube pile								
	Delmag			Junttan			MKT		
Weight	3 t	5 t	10 t	3 t	5 t	10 t	3 t	5 t	10 t
Min.	1.032	1.016	1.046	0.992	1.013	1.069	0.919	1.052	1.010
Max.	2.099	2.323	2.504	2.676	2.667	2.499	2.802	2.494	2.736
Mean.	1.446	1.555	1.532	1.556	1.514	1.515	1.554	1.527	1.496
STDEV	0.308	0.363	0.391	0.347	0.332	0.299	0.333	0.307	0.294

dynamic effect is greater due to the viscous nature of clay soils and hence the factor must be higher for clay soils. On the other hand, for sandy soils, this dynamic effect is very small and hence the HCF approaches a unity.

4. Conclusions

In this study, a methodology and discussion on the results of numerical parametric analysis using GRLWEAP have been presented. A total of 1944 cases were analysed based on a combination of different hammer type and weight, pile types, soil types and hammer strokes. The data were collated into a spread sheet and further analysis was conducted to calculate and compare the range of HCF based on these variables.

Based on this study, it has been proposed that GRLWEAP analysis could be carried out to establish the:

1. Hammer performance and efficiency;
2. Transmitted energy to piles;
3. Relationship between hammer velocity and peak energy; and
4. Set versus Resistance of piles.

Once the above have been established, the HCF can be used calculated based on these parameters. Subsequently, the HCF can be used to evaluate pile capacity in the field with greater accuracy and reliability.

The study also demonstrates that the pile capacity prediction by the Hiley formula can be improved reliably provided the variation in the energy input can be accurately measured and allowed for in the calculations. Since under normal pile driving conditions, variability in the driving system and the energies delivered to the pile exists, it is important to account for this variability so that the driving formula can be used with greater confidence.

Correction factors to allow for the dynamic effects of soil resistance, similar to the Case damping factor, need to be established and utilised to improve the reliability of the dynamic formula. It has been shown that the correction factors are quite consistent and can be developed for a variety of ground conditions, hammers and pile types. This will allow for the dynamic formula, particularly the Hiley formula to be used with greater accuracy comparable to the wave equation analysis.

More general conclusions in this research can be drawn as follows:

- The wave-equation analysis only describes the energy transfer mechanism from the hammer to the pile toe in a systematic and accurate fashion and if the dynamic formulae are modified to account for the energy losses, then the dynamic formulae should technically fulfil the same function.
- The dynamic formulae, which ignore the dynamic effects, need to be accounted for in the formulae.
- The energy delivered to the pile and its set measurements need to be accurately determined in order to render the dynamic formulae reliable.
- Create a comprehensive database with driving records for various soil conditions, driving systems as well as different piles and establish driving formula correction factors against the

database.

- The correction factors can be established from GRLWEAP and CAPWAP analysis as well as static testing results.

References

AbdelSalam, S., Sritharan, S. and Suleiman, M.T. (2009), “Current design and construction practices of bridge pile foundations”, *Proceedings of International Foundation Congress and Equipment Expo 2009*, Orlando, FL, USA, March, pp. 458-465.

Broms, B. and Lim, P.C. (1988), “A simple pile driving formula based on stress-wave measurements”, *Proceedings of the 3rd International Conference on the Application of Stress-Save Theory to Piles*, Vancouvar, Canada, MONTH.

Chellis, R.D. (1961), *Pile Foundations*, McGraw-Hill.

Fleming, K., Weltman, A., Randolph, M. and Elson, K. (2008), *Piling Engineering*, (3rd Edition), Taylor & Francis.

Hiley, A. (1925), “Rational pile-driving formula and its application in piling practice explained”, *Engineering*, **119**(3100), 657-658.

Hussein, M.H. (2004), “A brief history of the application of stress wave theory to piles: Current practices and future trends in deep foundation”, *Proceedings of Current Practices and Future Trends in Deep Foundations*, Los Angeles, CA, USA, July.

Isaac, D.V. (1931), “Reinforced concrete pile formulae”, *Transactions, Institute of Engineers, Australia*, **12**(370), 305-324.

Lowery, L.L. Jr., Hirsch, T.J., Edwards, T.C., Coyle, H.M. and Samson, C.H. Jr. (1969), “Pile driving analysis - State of the art”, Texas: Texas Transportation Institute.

Paikowsky, S., Regan, J. and McDonnell, J. (1994), “A simplified field method for capacity evaluation of driven piles”, FHWA.

Pile Dynamics Inc. (PDI) (2005), *GRLWEAP Wave Equation Analysis of Pile Driving: Procedures and Models Manual*, Cleveland, OH, USA.

Smith, E.A.L. (1950), “Pile driving impact”, *Proceedings of the Industrial Computation Seminar*, New York, NY, USA, September.

Smith, E.A.L. (1960), “Pile driving analyses by the wave equation”, *J. Soil Mech. Found. Div., ASCE.*, **86**, 35-61.

Timoshenko, S.P. and Goodier, J.N. (1951), *Theory of Elasticity*, (2nd Ed.), McGRaw-Hill Book Company, New York, NY, USA.

Tokhi, H., Ren, G. and Xie, Y.M. (2011), “A new application of radar in improving pile dynamic formula used in the quality control of pile foundation”, *Australian Geomechanic Society (AGS)*, **46**(4), 35-49.

Whitaker, T. (1970), *The Design of Pile Foundation*, (1st Ed.).

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