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BEARING CAPACITY OF PILES FROM DYNAMIC MEASUREMENTS

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SUMMARY New procedures for the control of pile driving installations are compared with traditional Dynamic formula results. The use of the new testing method, including Dynamic measurements with a Pile Driving Analyzer is described extensively for a large piling foundation. Piles were driven to a criteria established by the Dynamic measurements and resulted in shorter pile lengths than for piles driven to the conventional Dynamic formula, thus saving both cost and time in the installation. Static load tests verified that piles driven to the new criteria had adequate factors of safety. Additional analyses of the Dynamic field tests determined the skin friction distribution, effectiveness of a non-negative skin friction coating, and the presence of a large soil strength increase by comparing results from end of initial driving to restrike results of the same piles several days later.

A new technique for evaluating piling installations is being introduced in Hong Kong. Field measurements of force and acceleration of the pile are made during driving or restriking by the hammer. This data then provides information about hammer performance, pile stresses, pile shaft integrity or damage, and pile bearing capacity. These results are provided on site for each hammer blow by a Pile Driving Analyzer using the Case-Goble Method of analysis (2,3).

The Analyzer obtains the total driving resistance from

$$RT = \left\{ F(t_1) + F(t_2) + (v(t_1) - V(t_2) \cdot Mc/L \right\} \; /2 \qquad (1)$$

where F and v are the measured force and velocity (usually near the pile top), M and L are the total mass and length, respectively, of the pile, and c is the wavespeed. The time t_1 is generally chosen as the first major velocity maximum and time t_2 is set to 2L/c later, the time required for the stress wave to travel the length of the pile and return. This equation is a closed form solution to the one-dimensional wave propagation equations. This total resistance RT is the sum of the static and damping soil resistances. The static resistance RS is obtained by

$$RS = RT - J(F(t_1) + v(t_1).Mc/L - RT) \qquad (2)$$

where J is a dimensionless damping factor ralated to the soil grain size.

Energy transferred to the pile is obtained from

$$E = \int F\ v\ dt \qquad (3)$$

To date, the Analyzer has been used on several projects in Hong Kong. One project involved the driving of 943 open end steel pipe piles for Area 24 Phase 1B for the Hong Kong Housing Authority at Tai Po, N.T. (5). Two sizes of pipe piles were driven: 508mm O.D., with 14mm wall thickness and 171 kg/m, and 406mm O.D., with 12mm wall thickness and 117 kg/m.

The site was situated on open reclaimed land in Tolo Harbour. There were four main high rise blocks and a multi-storey car park. The consulting engineer was Maunsell Consultants of Hong Kong.

The block areas were first excavated down to cut-off level, 2.5m below ground level, in order to reduce the gross length of pile. The first two pile sections were pre-welded in the horizontal position using a semi-automatic welding process and transported to the pile rigs. These sections were driven by a McKiernan Terry No. 10B3 air hammer or a Kobe KC-35 diesel hammer. Subsequent third sections were coated with a negative skin friction coating applied at the supplier factory in Japan and welding was performed in-situ using manual welding techniques. The driving of third and other subsequent sections was performed by Kobe KC-45 and KB-60 diesel hammers.

A typical soil boring (BH2) is given in Figure 1 with soil classification and standard penetration blow counts. The soils consisted of a dense sandy overburden above an 8 meter thick layer of marine clay. Undesired negative skin friction therefore had to be eliminated. Below the marine clay were silts of 20 and 30 blow consistency and then decomposed volcanic (mainly silty clays) below 28 meters. Blow counts in decomposed volcanic increased from 90 to over 200 at the bottom of the boring.

For the 232 ton working load and the required safety factor of 2.0, an ultimate load of 464 tons was needed. Verification of the installation was accomplished by ten static load tests.

Initially the piles were driven to satisfy the Hiley criteria. However, they frequently did not meet the criteria at the expected penetrations. Due to the nature of this formula, when pile lengths are increased the set criteria is also increased and it becomes even more difficult to reach an acceptable final result. Piles with total lengths in excess of 50 meters often resulted in an area where the expected lengths were only 38 meters.

The Housing Authority (with the concurrence of the Consultants) approved the use of the Pile Driving Analyzer on this project at the Contractor's risk as an initial step in upgrading pile driving techniques for ultimate future savings that might accrue to the Hong Kong Government. It was agreed that the piles be driven to criteria established by the Analyzer as long as the static load tests results were satisfactory. The intent of the dynamic pile testing was to develop a pile driving criterion that would provide the required bearing capacity with shorter piles as compared to the Hiley formula criterion.

Selected piles from Blocks 1 and 2 were monitored during driving by the Pile Driving Analyzer to develop a driving criterion. These same piles were allowed to set for a period of time to gain additional capacities resulting from the reconsolidation of surrounding soils. They were then monitored with the Analyzer during restrikes. The data from both driving and restriking of these piles was used to establish a criterion for driving the remaining piles. Selected piles driven to this criterion were restruck to confirm their bearing capacity. The driving criterion for Blocks 1 and 2 was used in Block 3. Piles from this block were later restruck to confirm their capacity.

Typically the Analyzer observed less than 450 tons force (200 MPa stress) for the 508mm diameter piles. The largest observed force was 570 tons, a stress level of 250 MPa.

The Kobe 60 hammer is rated (E = Wh) at 14.6 ton-meters at a stroke of 2.45 meters. Observed strokes were at times greater and energies transferred into the pile were as high as 8.6 ton-meters. In general the ratio of measured transferred energy to the potential energy Wh was between 40 to 50 percent, efficiency ratio indicative of a good, normally performing hammer.

Typical force-velocity measurements are shown in Figure 2 for three piles. Piles 32 and 33 were driven in adjacent locations and had similar blow count records. The blow count for pile 32 EOD (End of Driving) was only 33 blows per meter. After a two-day wait, the blow count during a restrike (pile 33R restrike) increased dramatically to 156 blows/meter. Pile 68R was located approximately 18 meters from piles 32 and 33. (These piles were among the first on this site tested by the Pile Driving Analyzer).

Figure 2 shows that for pile 32 EOD a force reduction and velocity increase occurs at time 2L/c, the time necessary for the peak input to travel the length of the pile, reflect and return to the top. Such behavior represents a pile with resistances much less than the peak input force. (The maximum force was 315 tons versus capacity of 219 tons.) The adjacent pile 33R shows at most slight force reductions and velocity increase at 2L/c, a behavior associated with higher capacities (388 tons versus input force 349 tons).

The rate at which F-Iv (where I = EA/c, i.e. the pile impedance) increases gives an indication of the skin friction distribution. The force should be equal to Iv until reflections from pile section changes (or the tip) or from soil resistances. It can be observed that the rate of divergence for pile 33R is greater than for pile 32 EOD, thus the skin friction has increased due to set-up during the two-day wait. A wave equation type program, CAPWAP, has been developed which uses this and other information from the measured force and acceleration/velocity data to determine the skin friction and toe bearing distribution along with damping constants and soil quakes (1,3). Assumed soil resistance models and the measured acceleration are used to compute the forces. This computed force is then compared with the measured force. The soil resistance model is then modified until the computed and measured forces agree.

Figure 3 shows an example of the sequential CAPWAP computed-measured force matching process for pile 33R. For each trial, the total resistance, damping and quake values and the skin friction distribution are presented in Table 1. The final matches are then presented in Figure 4. Minor differences are observed around the time 2L/c (4L/2C) and are considered to be the effect of an incorrect assumed pile length (the pile was marked meter by meter during driving and small errors can accumulated). The error is insignificant and the length is probably not too different.

Referring to Figure 5 and Table 2, both piles show zero resistance encountered in the upper 8 meters; the skin friction reduction coatings were effective. From 8 to 18 meters skin friction was about 45 tons in both cases; it is concluded that set-up in this layer is minimal. From 18 to 28 meters, an increase of 55 to 122 tons was observed; thus, the silts and clays in this layer have a set-up factor of about 2.0. Below this level, in the decomposed volcanic (28 to 35 meters), another set-up of 2.0 (from 92 to 192 tons) was noted. The force in the pile at any depth, shown graphically in Figure 5, represents the total soil capacity below that depth. Steeper slopes then indicate higher unit skin frictions.

Case Method capacity obtained in the field by the Pile Driving Analyzer is presented in Figure 6. Both blow count and capacity showed dramatic increases after the two-day wait period. The CAPWAP results have been included for comparison. The field selected damping constant of J = 0.2 was confirmed for the initial driving. Restrike showed a somewhat lower CAPWAP result compared to Case Method. This indicates that a larger Case Method J is required for restrike and can be computed from

$$J = (RT - RS) / (F + Iv - RT) \tag{4}$$

where RT is the total Case Method capacity and RS is the static capacity, in this case determined by CAPWAP. The Analyzer can be correlated with any static load test failure definition by use of this equation.

Several reasons may be proposed for the apparent increase in soil strengths. During driving, the soil can be remolded, losing strength. This is particularly important in fine grained soils. Pore water pressures and liquefaction can also contribute to reduced capacity during driving. During driving, non-axial pile motions can create an oversized hole, reducing the skin friction. With time the soil pressures will cause the soil to again collapse around the pile, eliminating this hole and thus increasing the friction. These piles also had tip protection (an extra 14mm added to the pile wall thickness on the outside at the bottom only) which probably contributed to skin friction reductions during driving by creating an oversize hole.

To get a true indication of the service load capacity of the piles at Tai Po, it is therefore necessary to test during restrike after a waiting period. Restrikes revealed an increase in bearing capacity. Within 8 hours after driving, pile capacity showed an average increase of 17%. Piles that were restruck one to six days after driving showed an average increase in capacity of 40%.

A conventional wave equation analysis was performed using the WEAP program containing a correct thermodynamic model of diesel hammers (4). Results indicate that for an ultimate capacity of 460 tons, the observed blow count should be 174 blows per meter at a 2.3 meter stroke (43 blows per minute). Stresses calculated were 222 MPa (32.2 ksi). This appears to match the observed field results.

Pile 68 Restrike exhibited behavior similar to Pile 33 Restrike. During driving, the blow counts and capacities were significantly higher than Piles 32 and 33. Blow counts at the end of driving were 125 blows per meter. The CAPWAP results (Table 2 and Figure 5) show behavior which is similar to Pile 33R, but indicate slightly higher resistance.

This pile was marginally adequate for a 230 ton design load and a safety factor of 2.0 after the two-day set-up period. Additional set-up would provide additional safety. Alternatively, driving to lower blow counts (lower capacity) and then allowing the set-up to overcome the initial deficiency would allow even more economic foundations. Additional restrike testing to monitor the blow count and capacity by the Pile Driving Analyzer was recommended to assure quality control.

A total of 46 piles, approximately five percent of the total, were tested dynamically. Ten static tests confirmed the driving criteria. The static load test for Pile 2-154 was the closest to failure and is shown in Figure 7. Yet the pile did not fail using the Davisson failure criteria at a load of 482 tons. The Davisson criteria for quick load tests is obtained from where the line equal to the pile stiffness EA/L beginning at a displacement offset of $\delta = 0.15 + D/120$ - δ and D are in inches and D is the pile diameter - intersects the load test curve. For slow maintained tests, the creep effect should be subtracted out. The Analyzer had predicted 520 tons for the failure load during a restrike.

The concept of load factor design is growing in popularity. Basically, it recognizes the distribution of strengths of individual piles and the variance of design loads. Safety factors are not assigned rigid values but rather are calculated from different factors on live and dead loads and a reduction factor on pile strength related to the method and amount of inspection done to verify bearing capacity. For example, safety factors must be higher if only dynamic formula (like Hiley) are used than if static tests are performed. Additional load tests should be reflected in lower allowable safety factors. Tests by the Pile Driving Analyzer can be performed in still larger quantities in a quick, economical manner and again increase the quality control.

Since this was the first experience of dynamic testing in Hong Kong, a conservative approach was used for the Pile Driving Analyzer. Piles were driven deeper than required by the Analyzer and damping factors were chosen conservatively, resulting in safety factors around 2.2 to 2.4. The Analyzer contributed in reducing pile lengths, but it is difficult to assess the magnitude of the savings.

The piles driven to the Hiley criteria and tested by restriking with the Pile Analyzer often had safety factors in excess of 3.0. These piles were generally longer and thus more costly to install. While large safety factors are comforting, if not dictated by loading uncertainties, they are not economically justified. The Pile Analyzer has proven without any doubt that piles can be driven in Hong Kong by using modern Wave Theory methods which are supported by actual dynamic measurements to effect savings in cost and time.

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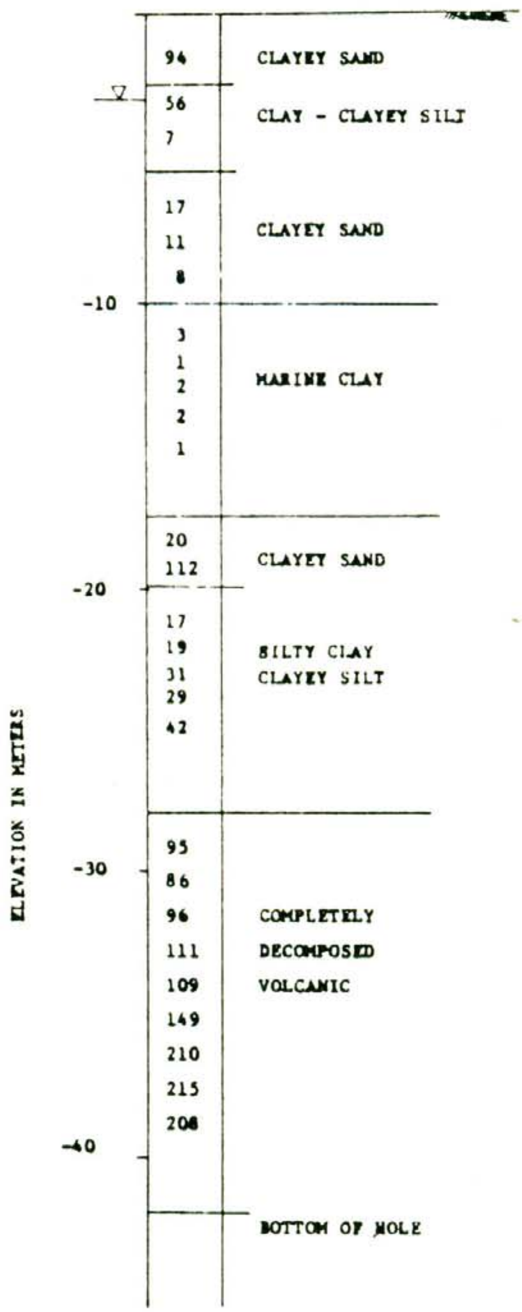
Table 1: Soil Resistance Models for Figure 3 CAPWAP Trials

Trial	Static Resistance - unit skin friction* (tons/meter)							Smith Damping		Quakes	
	Total	Toe	depth in meters					Skin	Toe	Skin	Toe
	tons	tons	6-12	12-18.5	18.5-24.5	24.5-30.8	30.8-35.4	s/m	s/m	mm	mm
5 (Final)	388	33	3.25	5.02	10.93	18.31	26.58	.09	.57	1.5	1.5
4	375	30	3.25	5.31	10.93	16.54	26.58	.09	.61	2.0	2.0
3	341	35	3.25	5.31	8.27	13.59	25.70	.08	.53	2.5	2.5
2	273	30	3.25	5.31	5.31	10.63	19.79	.06	.46	2.5	2.5
1 (Initial)	227	24	3.25	5.31	5.31	7.68	15.36	.07	.59	2.5	2.5

*includes inside and outside friction on pipe. For uplift, inside friction unavailable so divide unit friction by two and ignore toe resistance.

Table 2: CAPWAP Results

Block - Pile	Static Resistance - tons						Smith Damping		Quake	
	Depth in meters				Toe	Total	Skin	Toe	Skin	Toe
	0-8	8-18	18-28	28-35			s/m	s/m	mm	mm
1-32 EOD	0	48	55	92	24	219	.07	.60	1.3	1.3
1-33 R	0	41	122	192	33	388	.09	.57	1.5	1.5
1-68 R	0	48	136	219	40	443	.09	.47	2.0	2.0



SOIL PROFILE - TAI PO

FIGURE 1

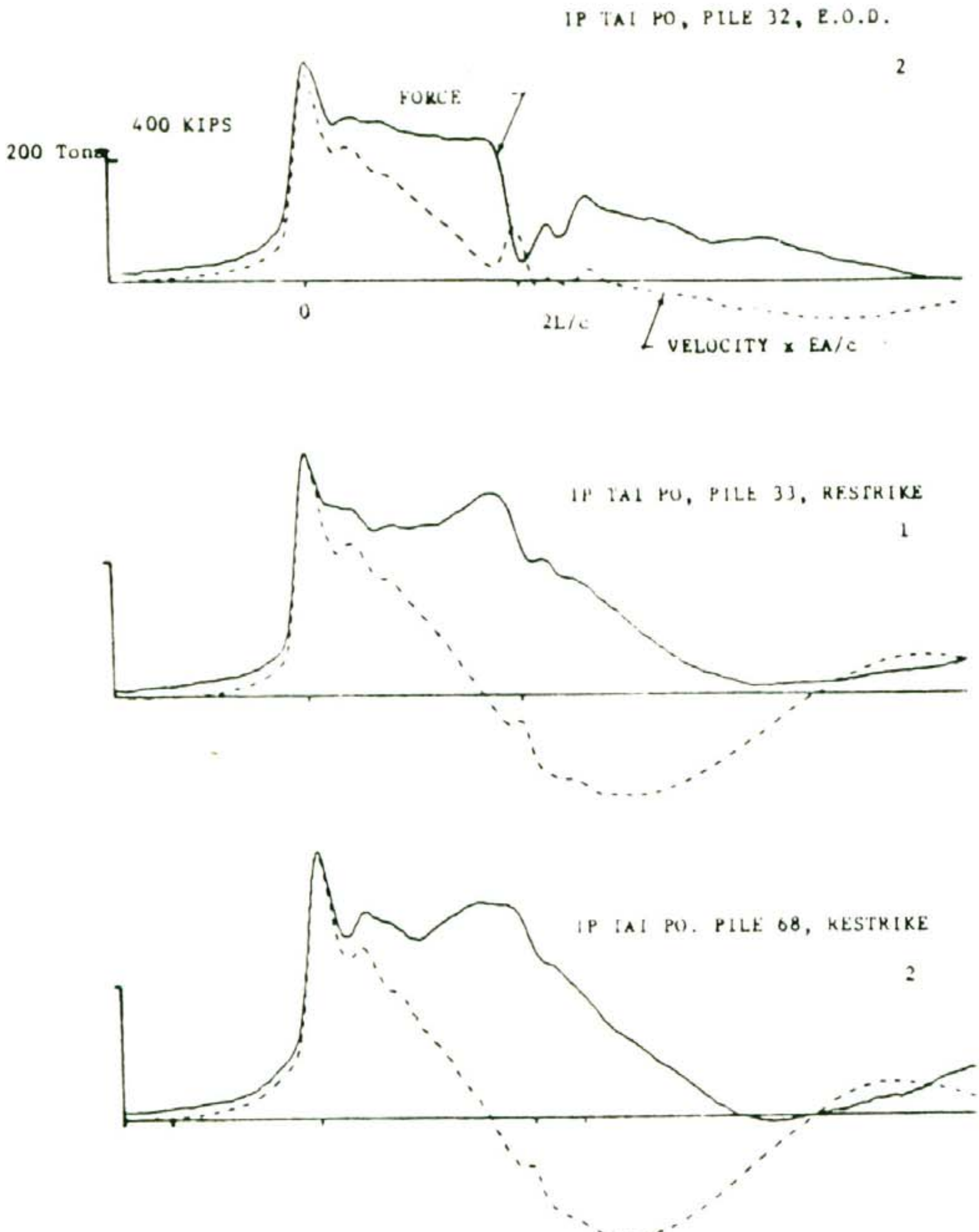


FIGURE 2

TIME SCALES ARE 3.28 MSEC/CM

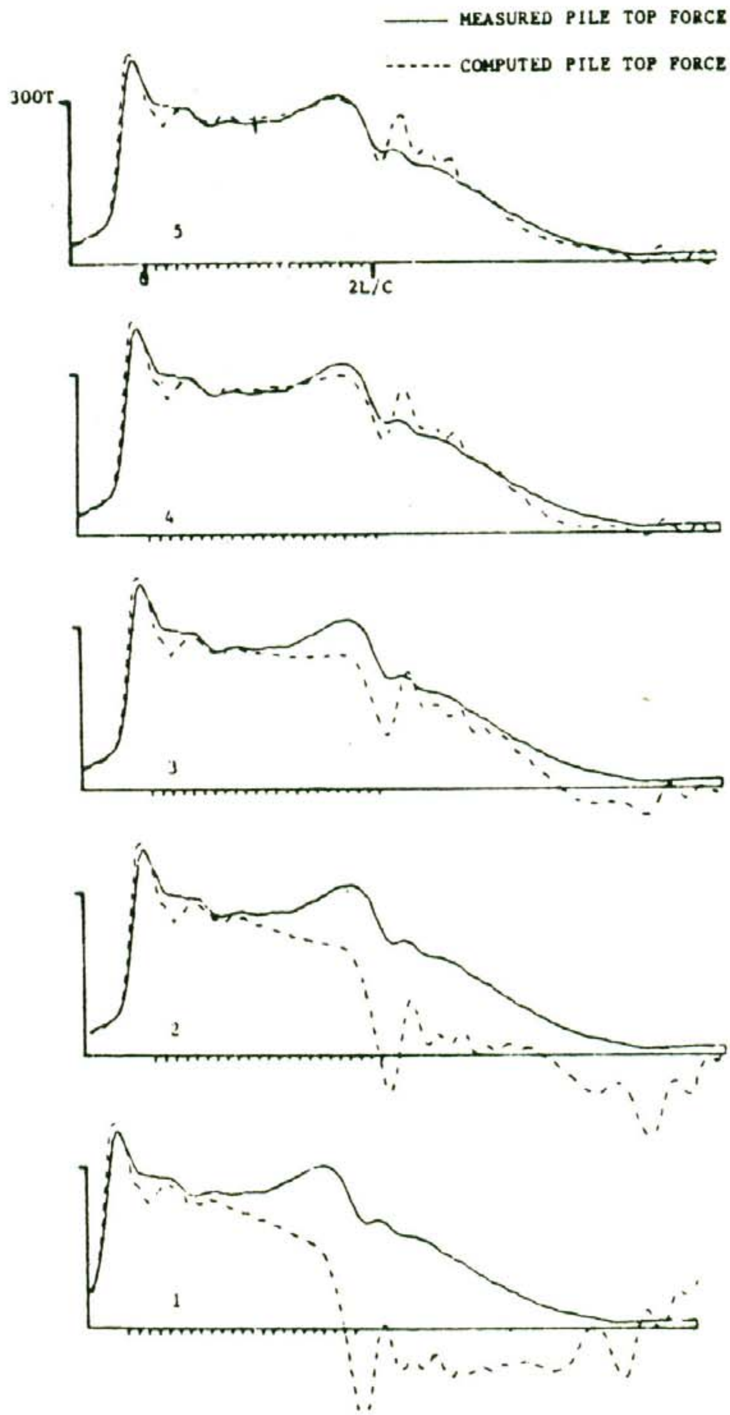


FIG. 3 CAPWAP MATCH FOR PILE 33 RESTRIKE

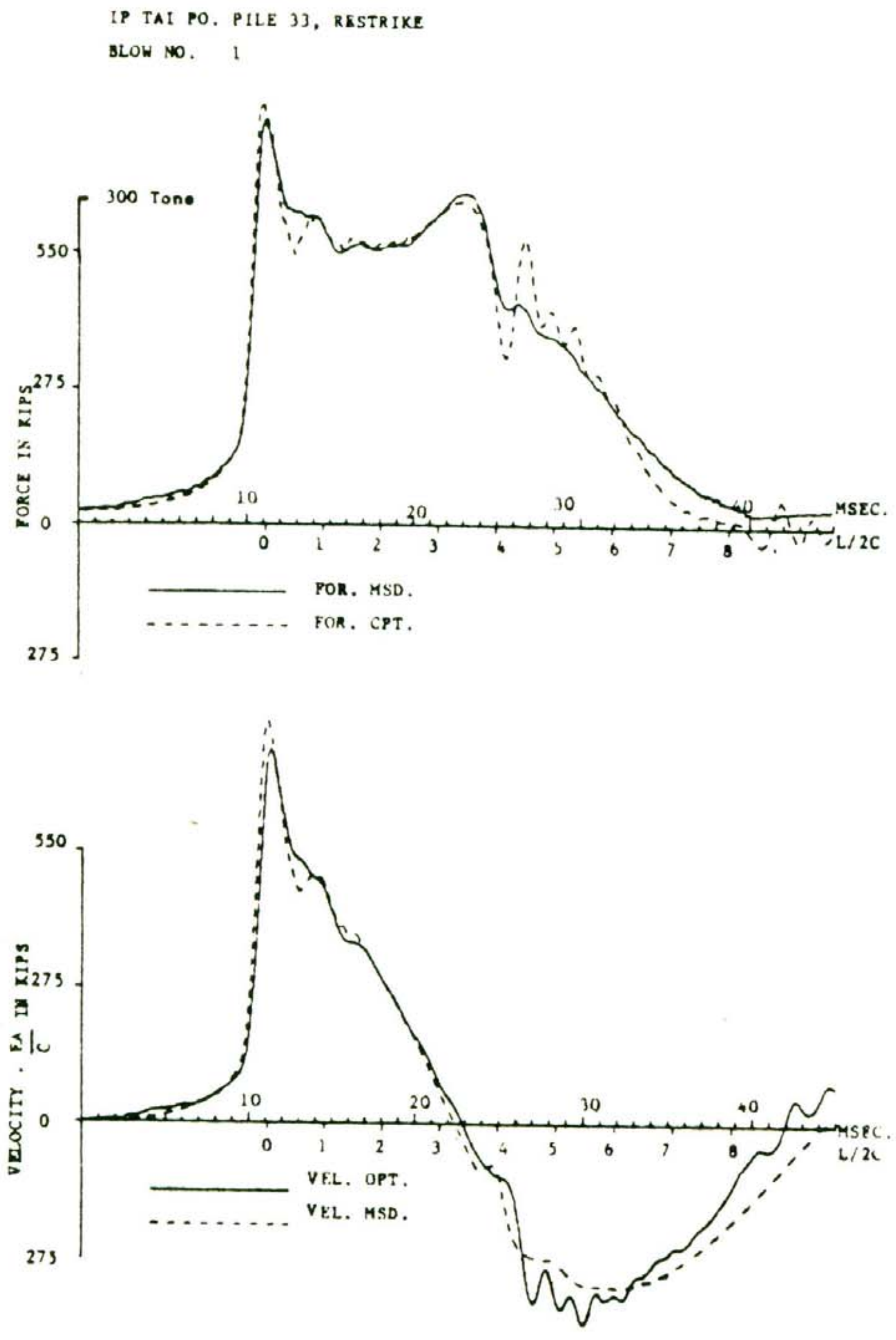


FIGURE 4 - FINAL CAPWAP FORCE/VELOCITY MATCHES FOR PILE 33R

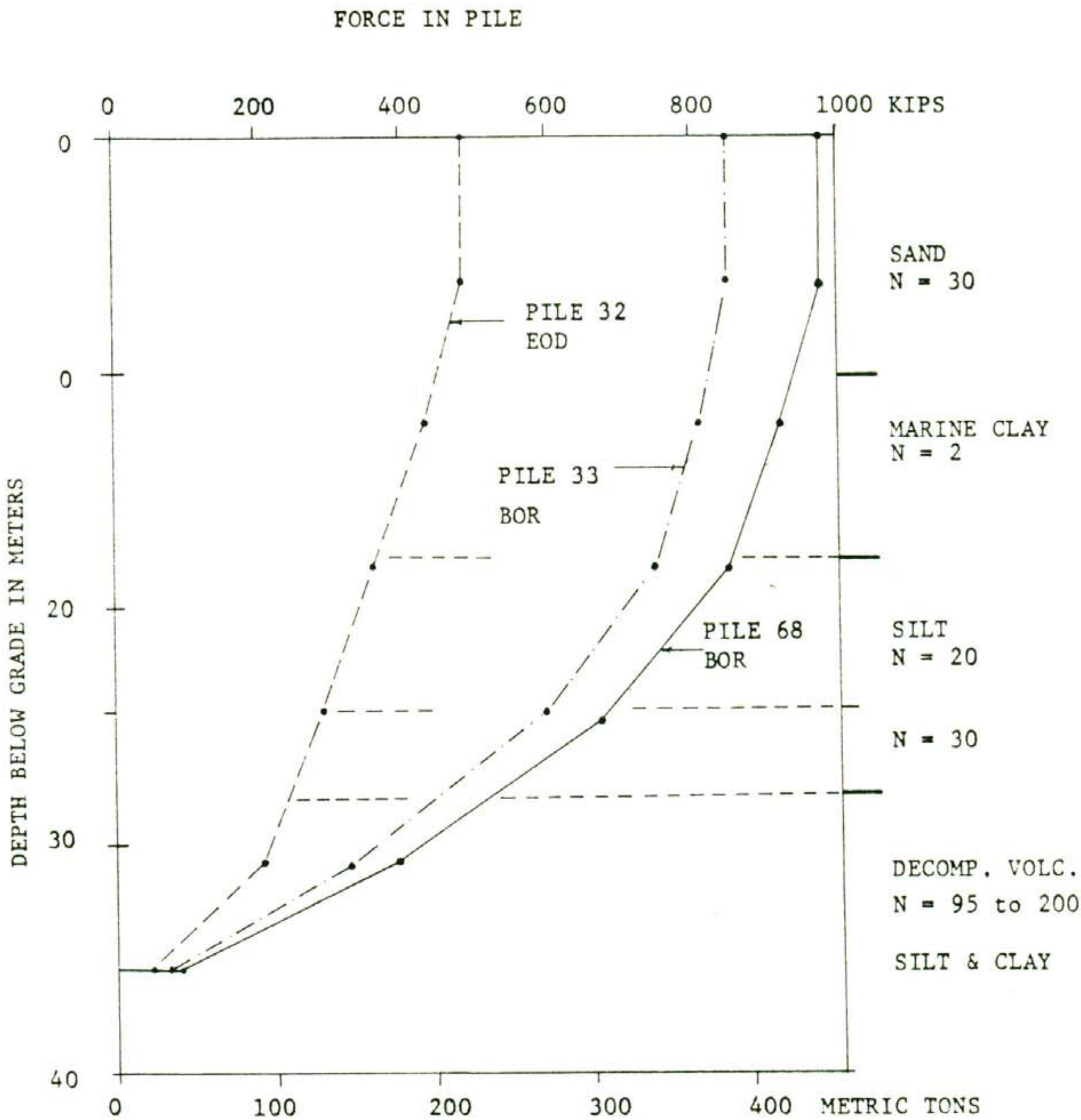


FIGURE 5 RESULTS FROM CAPWAP ANALYSIS

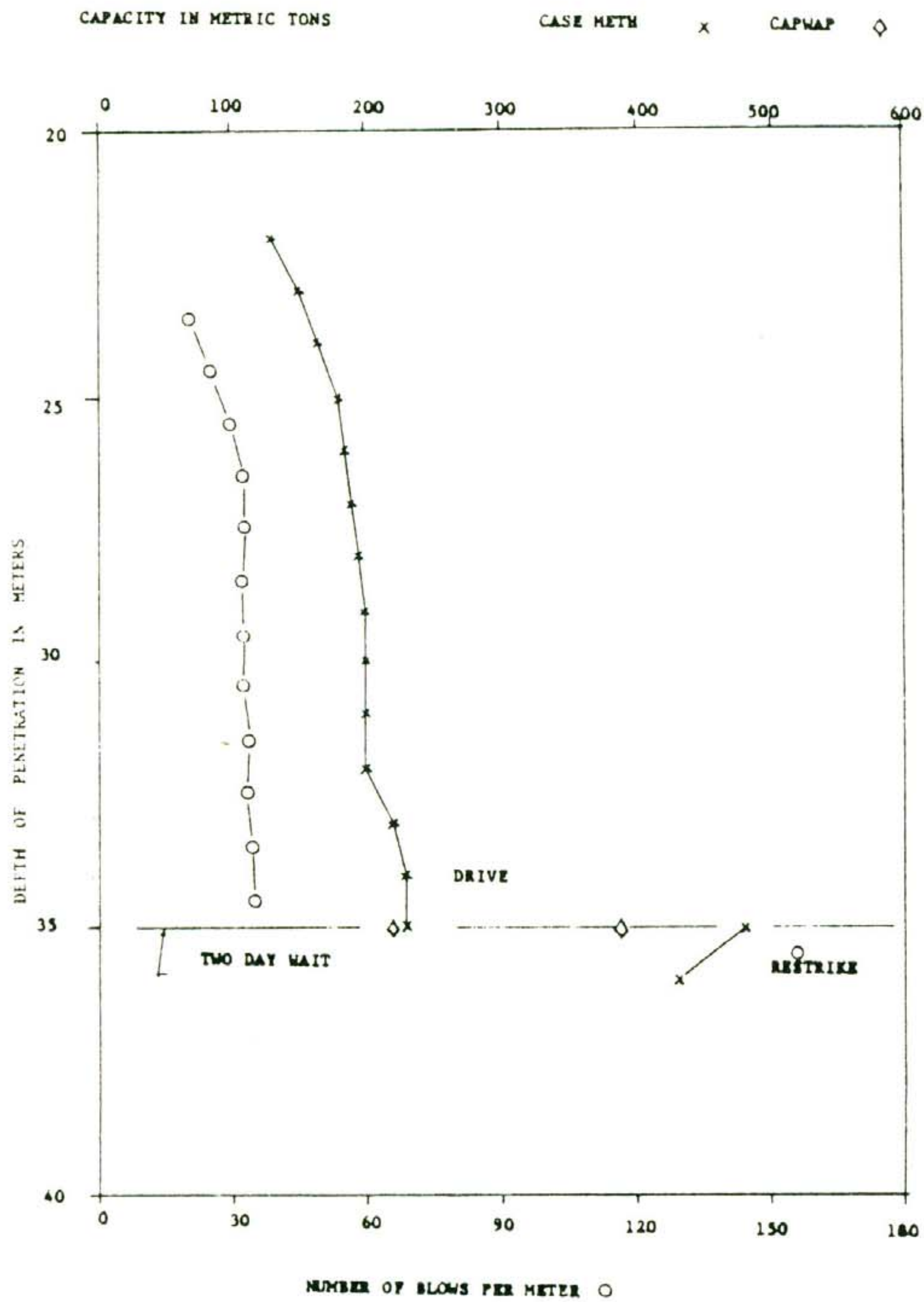


FIGURE 6 FIELD RESULTS FOR PILES 32 AND 33

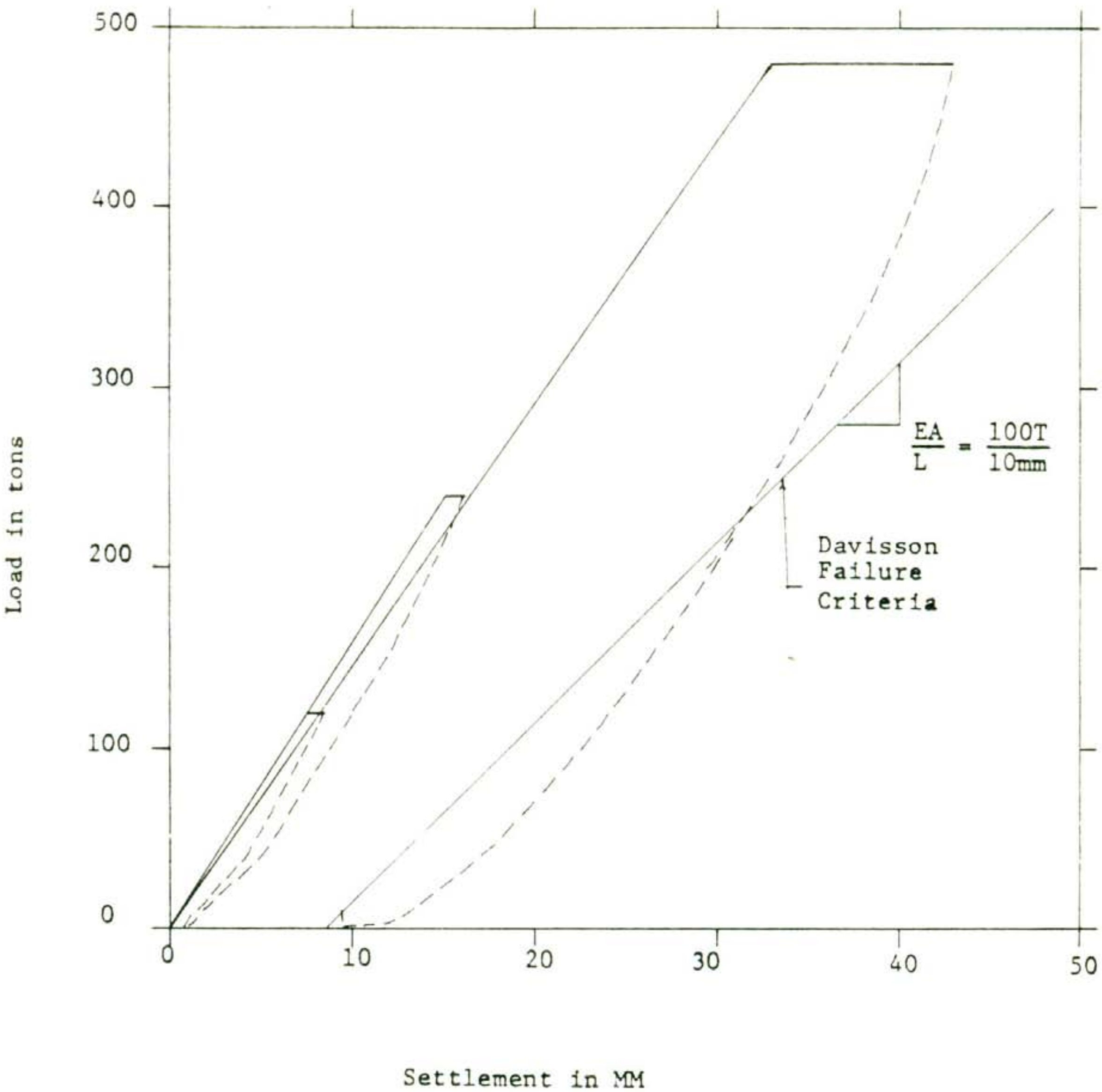


Figure 7 : Static Load Test
For Pile 2-154