Pile Installation Difficulties in Soils with Large Quakes

By Garland E. Likins, Jr.*

INTRODUCTION

With the continued development of dynamic pile testing techniques and analysis procedures, much has been learned about the dynamic resistance properties of soils during pile driving. Three types of quantities completely describe the pile during driving; pile forces, pile motions and the soil boundary conditions. If any two are known, then the third can be derived; the CAPWAP computer analysis program (1) utilizes measured force and acceleration data to determine the actual soil parameters. The measured acceleration and Smith type pile and soil models are used to compute a force curve which is then compared with the measured force. Adjustments are made in the soil parameters until the computed and measured force curves match. Output results are then the ultimate static load and its distribution, skin and toe damping values, and skin and toe quakes, i.e., the displacement at which the initial elastic static soil model achieves its ultimate load and goes plastic.

Prior to this analysis technique, it was concluded from parameter studies (2) of the wave equation with the quake between 0.05 and 0.30 inches that the quake value did not significantly affect any of the basic wave equation results. Based on relatively recent experiences using dynamic pile measurement and CAPWAP, it has become apparent that soil quakes far in excess of previously considered values frequently exist and do in fact significantly alter the wave equation results (3,4,5).

This paper discusses three cases where "high quakes" have been observed in soil conditions ranging from sands to clays. Other cases having "high quakes" (toe quakes between 0.4 and 1.0 inches) have also been observed (5). The only apparent common feature in the soils is that they are saturated. In most every case, displacement type piles have been involved and excess pore water pressure buildup during the cyclic pile driving has been suspected. Dissipation of this excess pore pressure usually is accompanied by, but does not necessarily result in, improved soil friction and lower static quakes.

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*President, Pile Dynamics, Inc., Cleveland, Ohio

EFFECTS OF LARGE QUAKES ON DRIVEABILITY

The occurrence of large toe quakes has complex effects on pile driving which have a great practical importance. First, the ultimate capacity which a hammer attains at refusal driving will be reduced, often requiring the use of a larger hammer. A capacity reduction by a factor of three is easily obtained; as the quake increases, this reduction becomes larger. The reason for this behavior can be observed in Figure 1. For the same maximum toe displacement, a pile with a normal quake will have a much larger permanent set (lower blow count) than will a pile having a large quake. Conversely, to obtain the same blow count, a pile with a large soil quake will require a much larger displacement; thus, more energy is required to mobilize the full resistance for high quake soils. Since the larger energy hammers would not normally be required for the usual soils, refusal blow counts are obtained much earlier, even at low ultimate capacities.

An additional effect of reduced resistance relates to tension reflections. One dimensional wave propagation theory shows that compression impact loads in pile driving cause tension reflections from the pile toe if little soil resistance is present. As soil resistance increases, this tension reflection decreases. If the pile is very short compared to the input pulse length, the continuing input compressive wave superimposed on the upwards traveling reflected tension wave results in little or no net tension. As piles become longer in relation to the input pulse length, net tensions result which can be particularly harmful to concrete piles. In soils with normal quakes, the ultimate load will reach a level approximately 1.2 times the input force at refusal driving and no tension stresses will be present except in easy driving; in the case of piles with medium or long lengths in high quake soils, the ultimate resistance even at refusal driving is much lower than the input force magnitude thus generating these dangerous tension reflections.

These tension reflections are further increased due to the slow response of large soil quakes. With typical pile top cushioning, displacements at the time of arrival of the peak input velocity at every point along the pile are usually comparable in size to normal quakes. Thus, the full resistance effects are mobilized at the time of the first reflection at the pile tip. Under normal conditions, this is enough to prevent damaging tension stresses from occurring. In the large quake case, the displacement at the toe at the arrival of the first input peak (typically 0.1 inch) can be considerably less than the quake. As demonstrated in Figure 1, this of course implies that only a fraction of the toe resistance is initially mobilized (in addition to the ultimate resistance being appreciably reduced) and even higher tension reflections are generated. Only later, after the initial wave peak has been reflected in tension, is the full displacement and resistance achieved. Thus, high tension stresses can be generated even in refusal driving conditions.

The use of dynamic measurements during pile driving has led to real time field evaluation for every blow of capacity, structural integrity, hammer performance and stresses (7). For example, the maximum tension stress T at any location x below the pile top can now be determined from standard Case Method measurements of the force F(t) and velocity v(t) time functions near the pile top from:

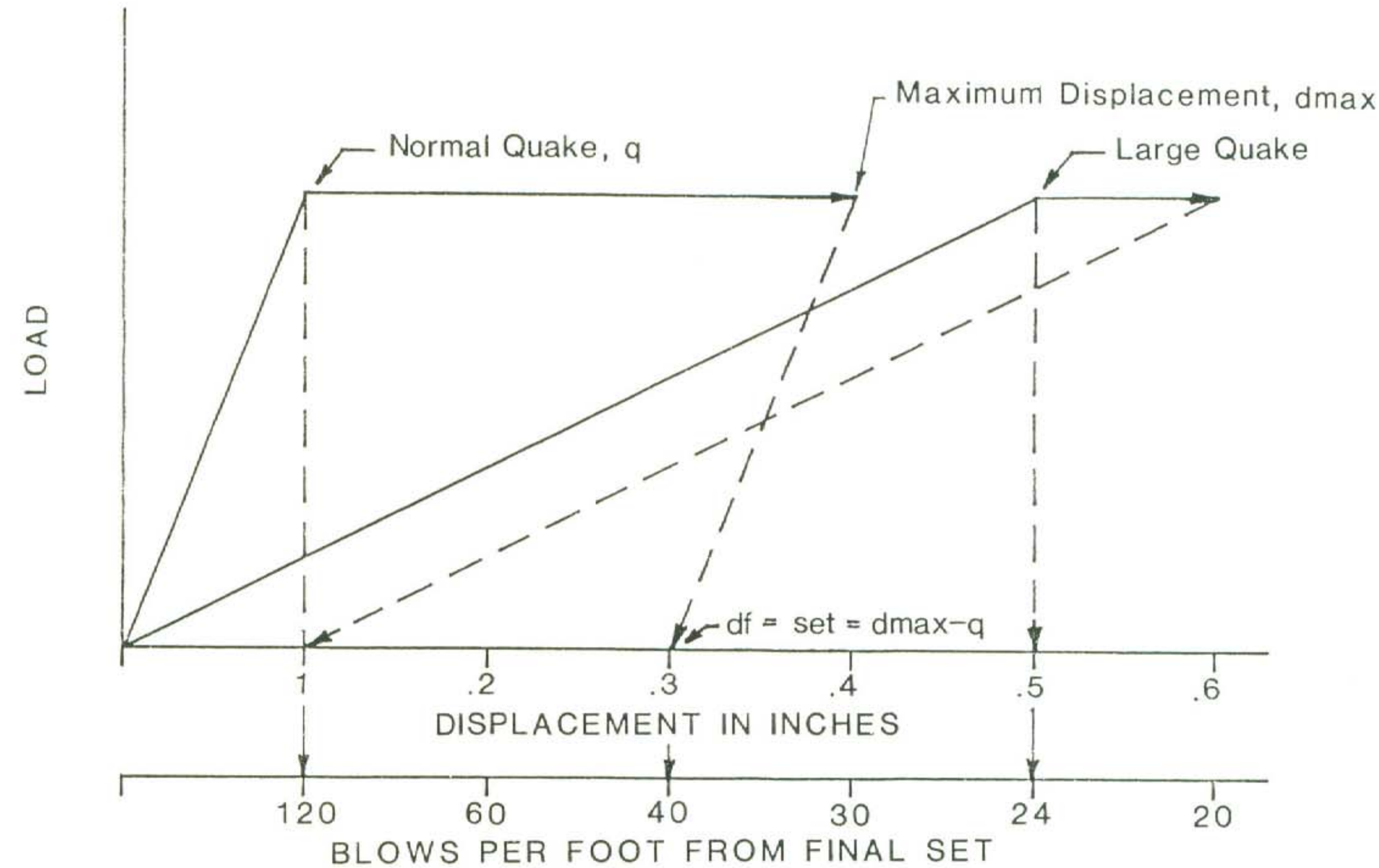

Figure 1: Load Displacement Diagram for Normal and Large Quakes

$$T(x) = \frac{1}{2}\left\{\frac{EA}{C}\, v\left(\frac{2L}{C}\right) - F\left(\frac{2L}{C}\right) - \frac{EA}{C}\, v\left(\frac{2L-2x}{C}\right) - F\left(\frac{2L-2x}{C}\right)\right\} \tag{1}$$

where the times of force and velocity are referenced to the peak input and the E, A, C and L are the pile modulus of elasticity, cross sectional area, speed of stress wave propagation and total length below measuring instrumentation, respectively. The closed form solution for static resistance RS of a one-dimensional travelling wave used in the field analysis equipment is:

$$RS = \left(\frac{1-J}{2}\right)\left\{F(t_1) + \frac{EA}{C}\, v(t_1)\right\} + \left(\frac{1+J}{2}\right)\left\{F\left(t_1+\frac{2L}{C}\right) - \frac{EA}{C}\, v\left(t_1+\frac{2L}{C}\right)\right\} \tag{2}$$

where J is a soil damping constant for soils and time t_1 being the peak input. For large quakes the resistance is low upon the initial arrival of the wave at the pile tip. When time t_1 is delayed from the initial peak, additional toe displacements are introduced into the resistance calculation of Equation 2; these additional displacements will mobilize extra soil resistance and be reflected in the capacity predictions. Investigations of capacity and tension stresses have been made in the field and used to detect the presence of large soil quakes. Although field estimates of quakes can then be made with this information, further laboratory analysis of the data is then usually justified to further define the soil's proper load displacement parameters.

The effects of high input stresses associated with prolonged hard driving, reduced resistance and delayed soil response causing high tension stress from large quakes often then combine to produce unexpected pile damage. Also, the "bounce" or high rebound, often associated with these soil conditions, usually results in a decreased hammer performance. Ram strokes are lower and racking or lift-off of the hammer assembly can become more of a problem due to non-uniform contact stresses and lowering the efficiency of energy transferred into the pile.

CASE STUDIES

Results obtained at three different sites are presented demonstrating the effects of large soil quakes.

Case 1

Several 24-inch (610 mm) octagonal prestressed concrete piles were installed. The piles were hollow, having a cross sectional area of 300 in^2 (1935 cm^2) and were 70 ft (21.3 m) in length. Below 27 ft (8 m), the soil was classified as glacial deposits of hard silty clay. After predrilling the first 12 ft (3.7 m), the pile had been driven to a penetration of 45 ft (13.7 m) with a Kobe K45 open-end diesel hammer which has a rated energy of 91 kip-ft (124 kJ) with a 10-inch (250 mm) plywood cushion and a 3.5-inch (89 mm) Fosterlon capblock. The pile was redriven and tested dynamically after a wait of three days. Blow counts steadily increased to over 21 blows/inch (8 blows/10mm) at 57 ft (17.4 m) penetration. Driving was stopped when the blow count exceeded 50 blows/inch (20 blows/10mm). The cushion was then reduced to only 4 inches (100 mm) of plywood and blow counts decreased to 22 blows/inch (10 blows/10mm) at a ram stroke observed to be 7.7 ft (2.35 m).

Figure 2 shows data taken at the end of driving with the 4-inch (100 mm) cushion. Of special interest is the relative force minimum and velocity maximum at a time 2L/c after the peak input (the time required for the wave to travel the length of the pile, reflect and return to the measuring location which was 60 ft (18.3 m) above the pile toe). Even at refusal blow counts, a net tension of 250 kips (1.1 MN; stress of .83 ksi or 5.75 MPa) is observed at the measuring location 10 ft (3 m) below the pile top. Ordinarily this would indicate a pile with low resistance as compared with the peak force input and structural capacity of the pile. Using techniques previously developed for the calculation of peak tension in the pile from top measurements (6,7), a tension force of 368 kips (1.64 MN) is found 7 ft (2 m) below the transducers. This corresponds to a stress of 1.2 ksi (8.46 MPa). During the actual construction phase of this project, dynamic measurements were again made and indicate that tension stresses as high as 1.5 ksi (10.5 MPa) were present during easy driving.

CAPWAP was used to further investigate the soil response of this pile. Figure 3 shows the final force and velocity matches (Figure 3a uses acceleration as input to compute force, Figure 3b uses force as input to compute velocity) and both are considered good. The total predicted capacity was 500 kips (2.2 MN). The skin friction is distributed rather uniformly with 350 kips (1.6 MN) indicated at the pile tip. However, the indicated toe quake of 0.42 inch (10.7 mm) was equal to the calculated maximum toe displacement, thus accounting for the high blow count. The toe displacement at the arrival time of the first input peak was 0.14 inch (3.6 mm) and therefore mobilized only about half of the total available resistance at the first reflection time. The maximum computed tension force from CAPWAP was 375 kips (1.6 MN). That this tension is high is not surprising considering that the peak force input of 900 kips (4.1 MN, stresses of 3.0 ksi or 21 MPa) is about 1.8 times larger than the bearing capacity during driving.

An equally good CAPWAP match could be obtained with even larger quakes providing the soil stiffness is not changed. It is therefore possible that the toe quake and toe resistance are even larger and the total resistance should be similarly increased. When the hammer in refusal driving is not able to generate sufficient penetration and mobilize the full ultimate soil resistance, dynamic capacity analysis techniques cannot be expected to result in anything greater than the actual mobilized resistance. This, of course, applies equally well to standard wave equation analyses where a driveability limit is obtained or even to static testing when the soil resistance is larger than the jack, reaction capacity or maximum applied proof load; only a lower bound proof load can be determined. A larger hammer would be necessary to achieve additional displacements and mobilize more capacity; however, larger hammers could increase the potential for pile damage.

A second CAPWAP analysis was performed using the same soil constants (resistance distribution and damping factors) except using a standard 0.10 inch (2.5 mm) quake at the pile toe. The toe displacement was equal to the quake at the arrival time of the first input peak at the toe and thus all the available resistance was mobilized. The force and velocity matches shown in Figure 4 are quite poor at 2L/c. The computed

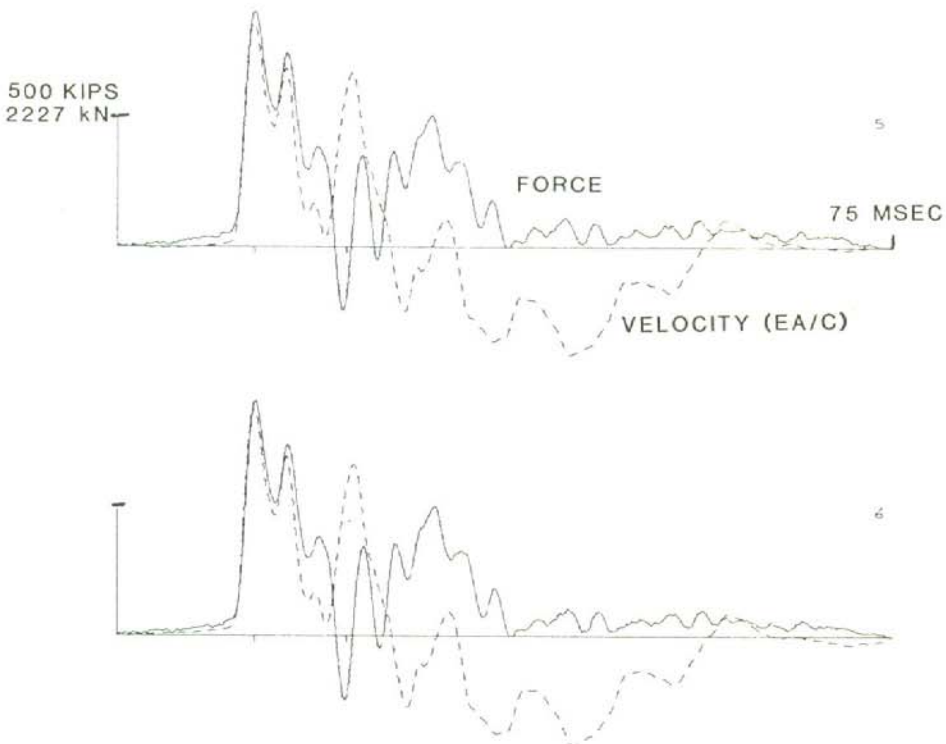

Figure 2: Dynamic Records - Case 1

Case 1

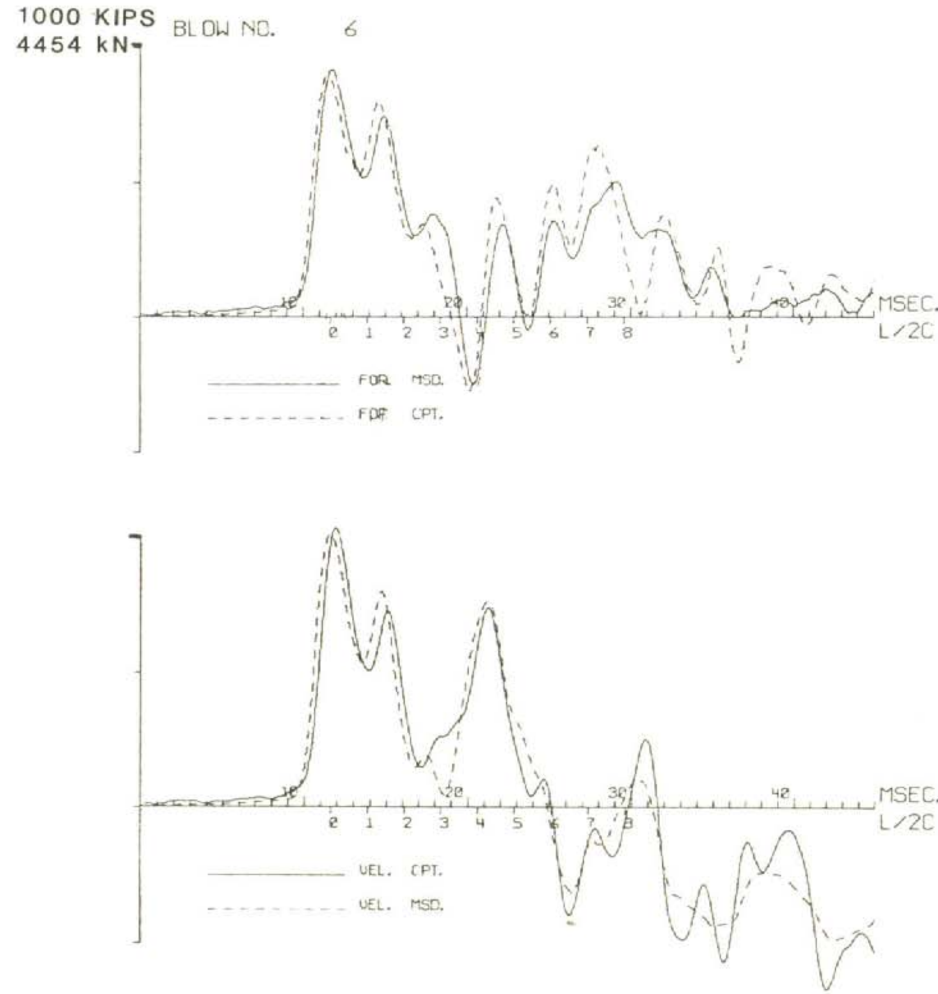

Figure 3: CAPWAP Results with Qt = 0.42 inch

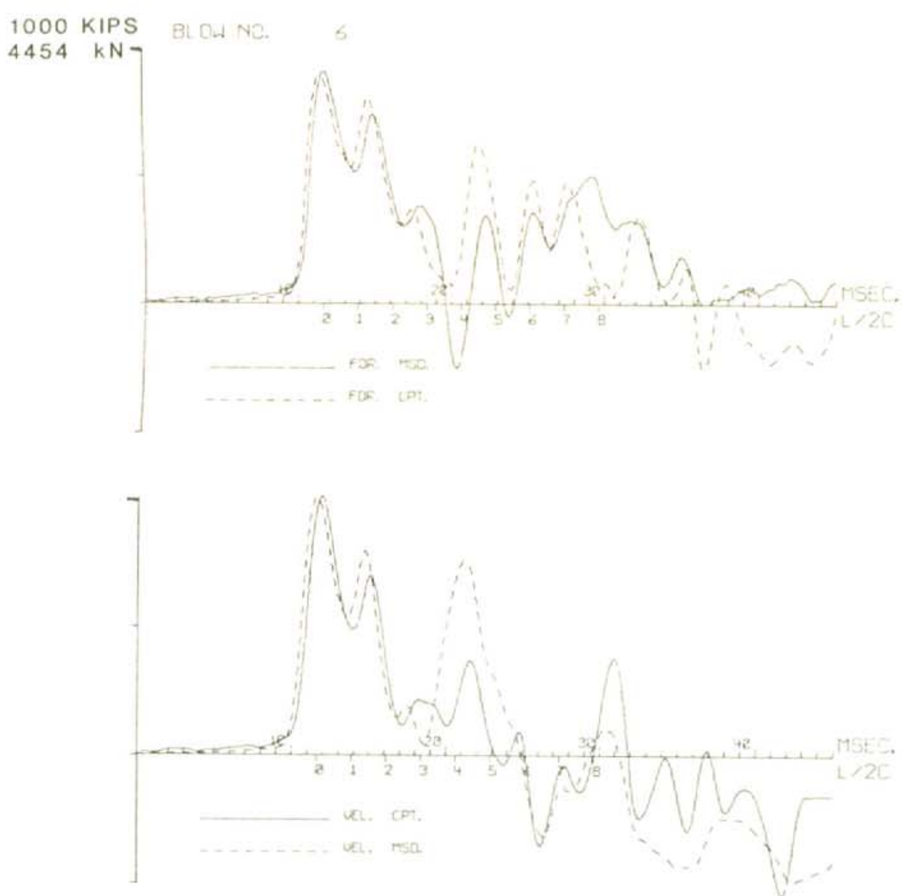

Figure 4: CAPWAP Results with Qt=0.10 inch

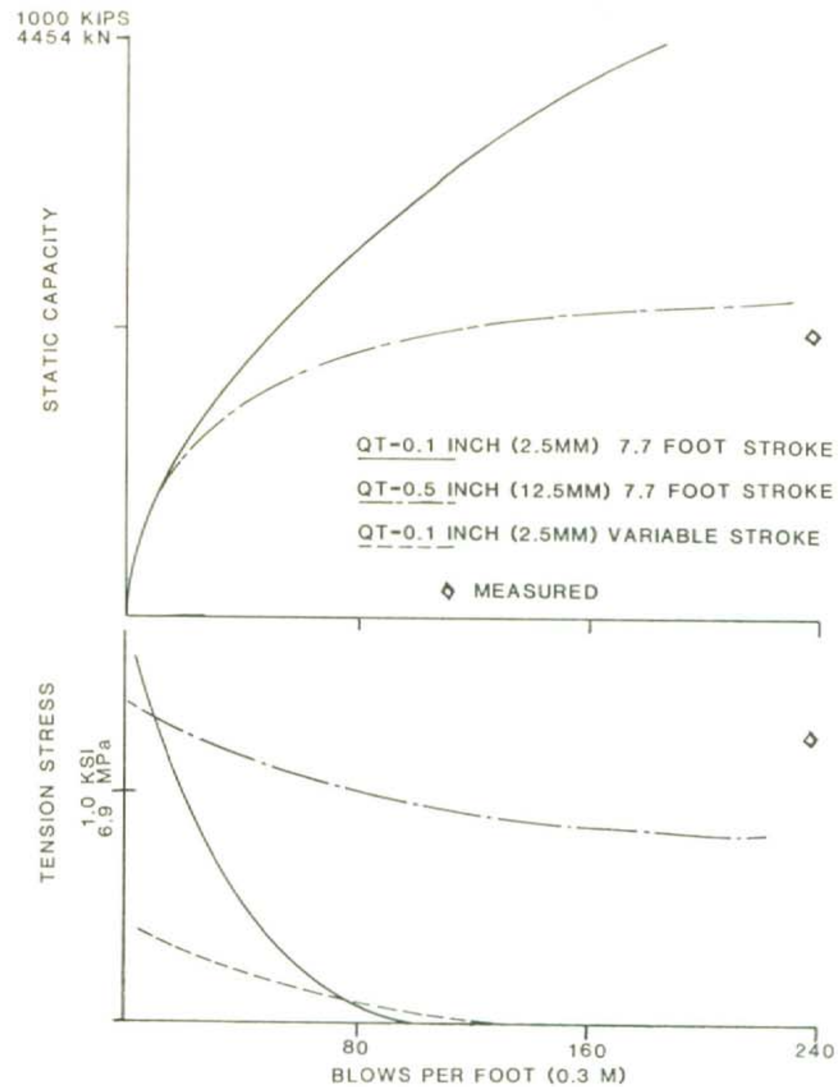

Figure 5: WEAP - Wave Equation Results

force no longer shows a net tension at 2L/c as was actually measured and the computed velocity is significantly reduced. This lack of agreement between measured and computed curves indicate that the soil model with normal quakes is not correct.

The CAPWAP soil model parameters were then input in the WEAP (Wave Equation Analysis Program)(8). Two analysis were made; one with a large quake of 0.50 inch (13 mm) and one with the normal quake of 0.10 inch (2.5 mm). Both analyses used the observed 7.7 ft (2.35 m) stroke. As seen in Figure 5, the capacity using a large quake at 20 blows/inch (8 blows/10mm) is only half the capacity using a small quake. Actually driving beyond 8 blows/inch (3 blows/10mm) yields little increase in capacity. The tension stresses are equally dramatic. Above 6 blows/inch (2 blows/10mm), there is no tension in the pile with normal quakes; with large quakes, the computed tension is never below 0.8 ksi (5.5 MPa) and the measured tension was even higher. The large tension stresses in easy driving may be artificially high as the observed stroke was used throughout. WEAP uses a thermodynamic model for the hammer and if allowed to compute its balanced stroke with a normal quake and 200 kips (890 MN) resistance, a stroke of 5.9 ft (1.8 m) is observed and the maximum tension is then an acceptable 0.3 ksi (2.1 MPa).

This pile was later load tested after several weeks. The Davisson failure load (10) was 1150 kips (5.1 MN). Telltale and strain gage data along the pile length gave excellent correlation with skin friction results from CAPWAP for the first blows at the beginning of this redrive (45 ft or 13.7 m penetration). Equally good results were obtained by comparing restrike capacity information on a 16-inch (405mm) dynamically tested pile driven to approximately 60 ft (18.3 m) penetration and scaling the shaft friction to account for the different diameters, proving the inherent correctness of the dynamic testing techniques (see Figure 6). It is always recommended that at least some piles on each site be tested during restrike to properly assess the soil's static strength. In this manner, setup and relaxation effects are then properly observed.

The large quakes observed dynamically are not in this case reflected in the static load test. It is indeed fortunate that the pore pressure dissipation and soil setup provided additional capacity. Additional testing during production driving, which also included some restrike tests, reconfirmed the indicator pile program results of setup factors of approximately two. Minimum tip elevations resulted in extremely high blow counts for many feet of penetration. A series of blows for one production pile is presented in Figure 7. This prestressed pile had an area of 300 in^2 (1935 cm^2) and a length below transducers of 95 feet (29m). The tension computation of equation 1 shows that tensions of 500 kips (2.3 MN; stresses of 1.7 ksi or 11.5 MPa) are present. Compression forces of 1250 kips (5.7 MN) are much larger than the ultimate capacity of 550 kips (2.5 MN) as determined by CAPWAP (the maximum Case Method estimate from equation 2 with a time search was 540 kips using a damping factor J=0.4 as determined appropriate for this site); the low capacity compared to the peak input is responsible for the high tensions. The CAPWAP analysis showed a toe resistance of 300 kips (1.4 MN). The toe quake was determined to be 0.55 inches (14 mm); this compares with the maximum computed toe displacement of 0.69 inches (17 mm). The driving resistance was in excess of 10 blows/inch (4 blows/10mm). Due to prolonged driving in these high stress conditions, this pile broke suddenly (during blow 6 of Figure 7). The remaining blows of this sequence

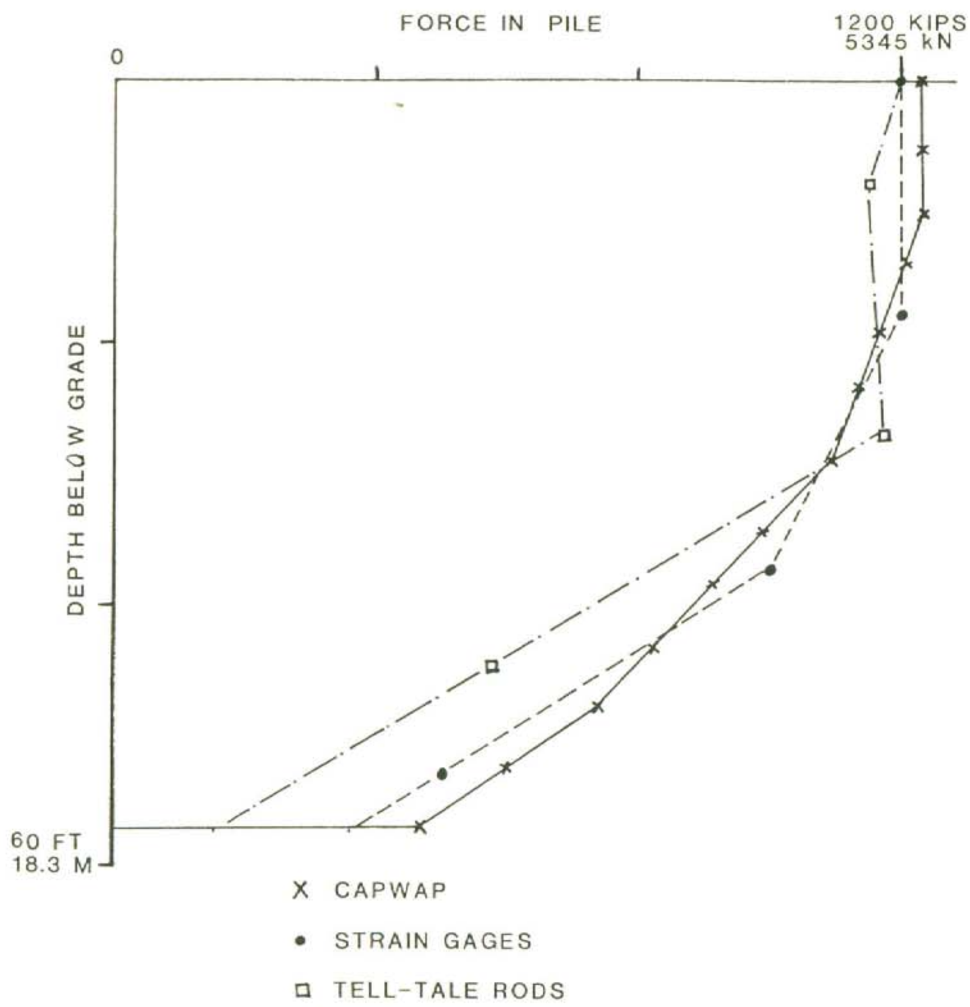

Figure 6: Resistance Distribution

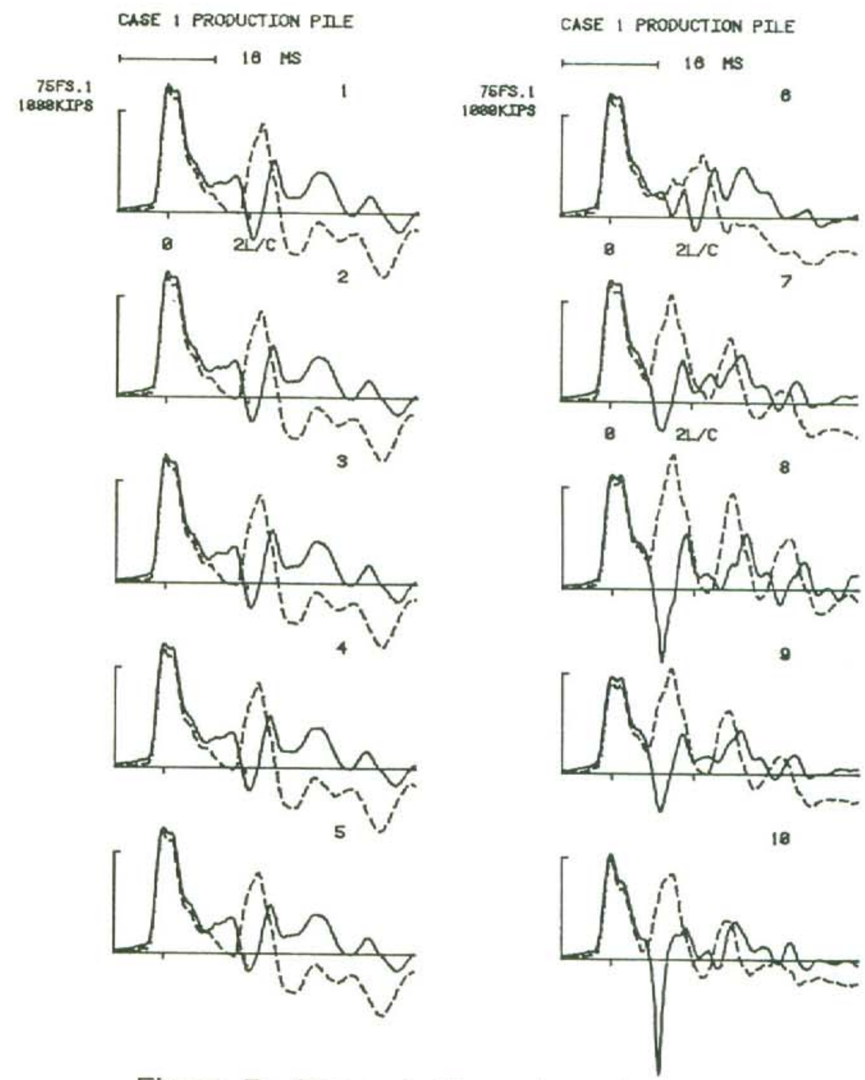

Figure 7: Dynamic Records - Case 1 Production Pile

before the hammer was shut off show a complete break (9) located 56 feet (17m) below the transducers; this is qualitatively observed in the sharp velocity increase and force decrease which is observed before the correct 2L/c time. High tension stresses and some tension cracking were later reduced by preaugering.

Case 2

Seven piles were tested dynamically on this site. All piles were 24-inch (610 mm) square prestressed piles with total lengths below measurements of 122 ft (37.2 m). All were prejetted through gray clayey sand to depths of at least 100 ft (30.5 m) into dense light gray sand. Driving was accomplished by a Raymond 80 hammer with a rated energy of 80 kip-ft (109 kJ).

The dynamic data of two piles is shown in Figure 8. Again, a velocity increase is observed followed by negative (upward or rebound) velocities. In both cases, the blow counts were slightly in excess of 17 blows/inch (7 blows/10mm) and skin friction was minimal. Proportionality between force and velocity for almost the entire first 2L/c indicates no reflections from soil skin resistance or pile cross sectional changes. Observed quakes from CAPWAP were about 0.7 inch (18 mm). Although capacities were around 1000 kips (4454 kN or about 60 percent of the peak force input) in each case, the slow soil response associated with the large quakes produced tension stresses of 0.75 and 1.00 ksi (5.2 and 6.9 MPa) for Piles A and B, respectively. Tension stresses in other piles on this site reached maxima of 1.37 ksi (9.5 MPa) at 4 blows/inch (2 blows/10mm).

No trend in setup or relaxation was observed (verified by restrike testing) on this site as might be expected in a soil described as dense sand.

Case 3

Twelve 18-inch (457 mm) square prestressed piles were driven and tested dynamically using a Delmag D-30 hammer. The pile lengths were 80 ft (24.4 m) and the soil was described as a saturated dense fine sand with some silt or clay content. Again, the piles were prejetted.

Blow counts were erratic at the end of driving ranging from 2 to over 42 blows/inch (1 to 17 blows/10mm). The example case shown in Figure 9 had 17 blows/inch (7 blows/10mm). The maximum computed tension was 0.6 ksi (4.1 MPa) at the end of driving and as large as 1.3 ksi (9.0 MPa) at lower blow counts. Again indicated capacities are low (compared to the peak force input and structural pile strength) as seen by the large velocity increase at 2L/c.

A CAPWAP analysis was not performed on this pile, although analyses of other piles on this site indicated quakes on the order of 0.40 to 0.50 inch (10 to 13 mm).

The analysis of the force and velocity traces revealed that one third of the piles had excessive structural damage (9) below grade, a condition not previously recognized due to the erratic blow counts during driving. It is probable that this damage was caused by the excessive tension due to the large quakes.

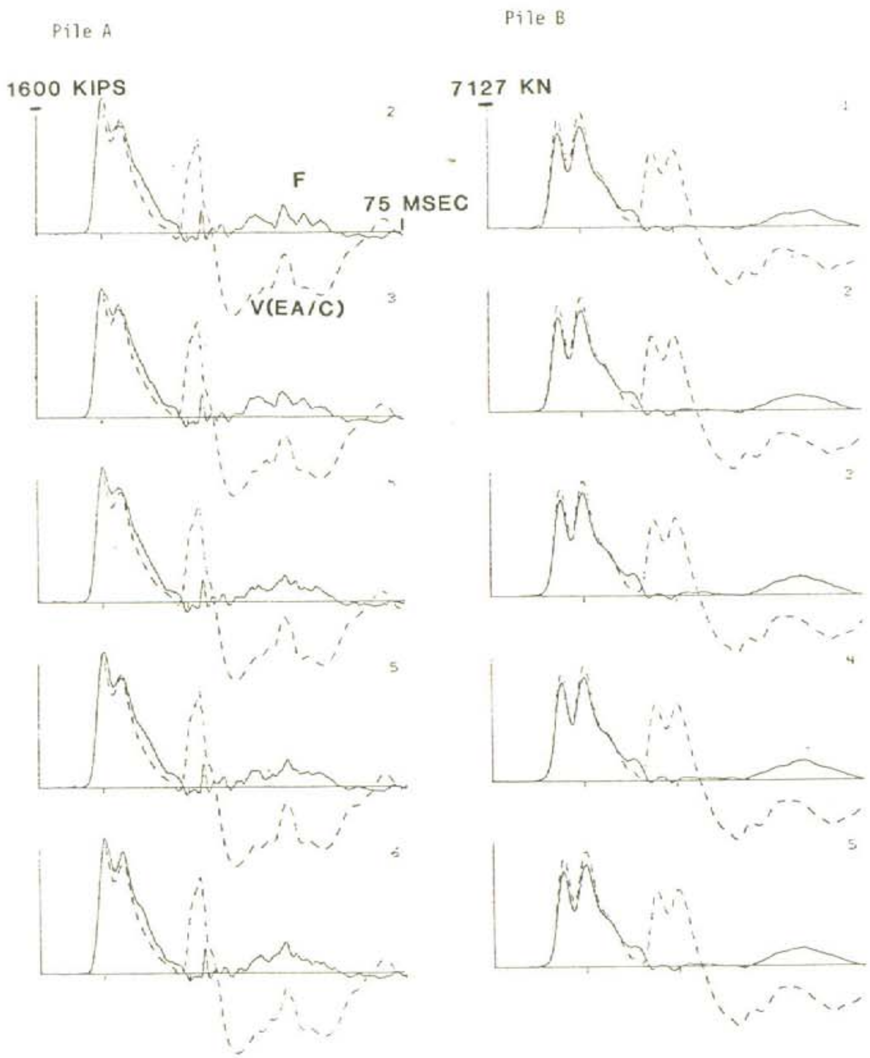

Figure 8: Dynamic Records - Case 2

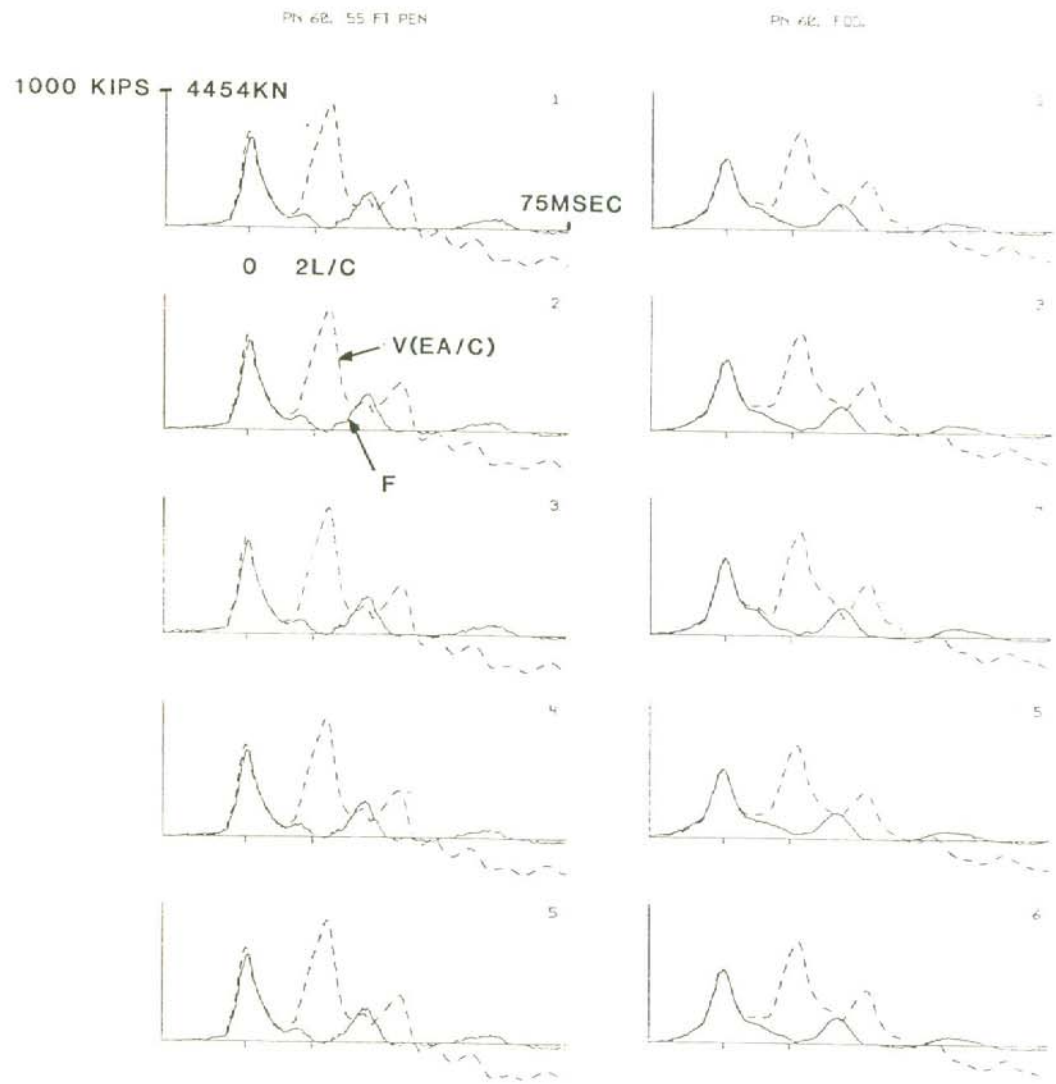

Figure 9: Dynamic Records - Case 3

Large setup factors associated with the fine grained soil and cementation in some layers later provided adequate capacity as determined by restrike testing. However these strength gains were primarily located below the structural damage. These damaged piles would not have been able to support even the design load and detrimental settlements would have resulted.

CONCLUSIONS

The three cases presented here clearly demonstrate the adverse effects of large toe quakes on pile driving. Not only is the driveability and ultimate soil resistance reduced but also increased tension stresses even in refusal driving conditions can and do cause structural damage.

The only common soil condition is saturation. It is felt that excess pore water pressures, caused by displacement piles driven into poorly drained soils is the primary cause of these large quakes.

Reliance only on dynamic formula or wave equation driving criteria could lead to unsafe foundations although in many cases gains in soil strength, as pore pressures decrease, compensate for low initial capacities. The only reliable method of determining the actual soil response during driving is by measurements of force and velocity. Subsequent CAPWAP analysis or low Case Method capacity (when compared with the peak input force) in near refusal conditions can be used to detect this behavior. Restrike testing by Case or CAPWAP Methods should always be performed, especially on sites with saturated or fine grained soils, to confirm service load capacity.

If large quakes are found, several corrective actions may be necessary. Augering with slightly undersize bits through weak layers and even through the problem soils may often be beneficial. Non-displacement pile types could be considered. If concrete piles are long and tension stresses high, the ram weight may be increased and the ram stroke may be reduced, causing lower compressive input and subsequent reflected tension stresses. Pile cushion thickness may be increased resulting in a longer input pulse width and reduced compression and tension wave peaks.

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