HIGH TENSION STRESSES IN CONCRETE PILES DURING HARD DRIVING

By Garland E. Likins, Jr.

It is a well understood principle of one dimensional wave propagation that compression impact loads, such as those occurring in pile driving, cause tension reflections from the pile toe if little or no soil resistance is present. As soil resistances increase, this tension reflection decreases. If the pile is short, the continuing input compressive wave superimposed on the upwards travelling reflected tension wave results in little or no net tension. As piles become longer in relation to the input pulse length, harmful net tensions can result. These tension waves can be particularly harmful to concrete piles.

As concrete piles become longer, additional measures are then necessary to prevent damage. For example, piles are often prestressed, thus superimposing a residual compressive stress to any tensile reflections. By pre-drilling, the pile end is often placed in more competent, higher resistance soils even before driving begins. In more difficult soils, the peak input compressive stress and subsequent tension refelction is limited by reducing the ram stroke. Increasing the cushion thickness or ram weight are ways to lengthen the pulse width and reduce the peak of the compressive input wave to be superimposed on the tension reflection.

Many concrete pile jobs have avoided potentially dangerous tension stresses by successfully using a wave equation analysis prior to construction to investigate and control the driving stresses, both tensile and compressive. In many cases dynamic testing has been used during construction to verify stresses from this theoretical analysis.

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. The CAPWAP computer analysis program (1) processes the measured force and acceleration data to determine the soil parameters. CAPWAP uses the measured acceleration and wave equation type pile and soil models to compute a force curve which is then compared with the measured force. Adjustments are made in the soil model 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 elastic plastic static soil model goes plastic).

Prior to this analysis technique and based on parameter studies of the standard wave equation with the quake between 0.05 and 0.3 inches, it was concluded that the quake value did not change any of the basic wave equation results (2). Based on relatively recent experiences using dynamic pile measurements 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. Authier and Fellenius (3) and Thompson (4) have both demonstrated that these large quakes can cause very hard driving while having low static capacities.

The case histories by Authier and Fellenius were both glacial soils, one being dense sandy silty till and the other dense clayey silty till. Thompson adds that the same observations have also been encountered in more coarse grained materials. The author has observed many such "high quake" cases (toe quakes between 0.5 and 1.0 inches) in a wide variety of soil conditions. The only apparent distinctive 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 does not necessarily result in improved soil friction or lowered static quakes.

The occurrence of large toe quakes has complex effects on pile driving which have great practical importance. First the ultimate capacity which a given hammer attains at refusal driving will be reduced, often requiring the use of a larger hammer. As demonstrated by Fellenius, a reduction in capacity by a factor of three is easily obtained. This reduction becomes larger as the quake increases.

As a secondary effect of the reduced resistance, tension reflections are generated even at refusal driving. If minimum penetrations are required, continued driving at these increased blow counts accompanied by high tension stresses again compound the structural damage potential to long concrete piles.

These tension reflections are further increased due to the slow response of soils with large quakes. With typical pile top cushioning, displacements at the time of arrival of the peak input velocity at every point along the pile are at least equal or comparable to the normal quakes, even in refusal conditions. 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 not only is the ultimate resistance achievable reduced but the displacement at the toe at the arrival of the first input peak can be considerably less than the quake. This of course implies that only a fraction of the reduced toe resistance is initially mobilized and even higher tension reflections are generated. Only after considerable delay is the full displacement and resistance achieved.

The effects of high input stresses associated with prolonged hard driving, reduced resistance and delayed soil response from high quakes combine to often produce unexpected pile damage.

Results obtained at three different sites are presented demonstrating the effects of large soil quakes on tension stresses in concrete piles.

Case 1 - Seattle

Several 24 inch (610mm) octagonal prestressed concrete piles were installed in March 1980. The piles were hollow, having a cross sectional area of 300 square inches (1935cm^2) and were 70 feet (21.3m) in length. Below 27 feet the soil was classified as glacial deposits of hard silty clay. After predrilling the first 12 feet (3.7m), the pile had been driven to a penetration of 45 feet (13.7m) with a Kobe K45 open end diesel hammer (rated at 91 kip feet (1 24kJ) based on a 9.2 foot (3.6m) stroke) with a 10 inch (250mm) plywood cushion and 3.5 inch (89mm) Fosterlon capblock. The pile was redriven and tested dynamically after a wait of three days. Blow counts steadily increased to over 250 blows/foot (100 blows/82mm) at 57 feet (17.4m) penetration. Driving was stopped when the blow count exceeded 50 blows per inch (2 blows/mm). The cushion was then reduced to only four inches (100mm) of plywood and blow counts reduced to 45 blows per two inches (50mm) at a ram stroke observed to be 7.7 feet (2.35m).

Figure 1 shows data taken at the end of driving with the four inch (100mm) 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 feet (18.3m) above the pile toe). Ordinarily this would indicate a pile with low resistance, as compared with the peak input and structural capacity of the pile. Using techniques previously developed for the calculation of peak tension in the pile from top measurements (5,6), a tension force of 368 kips (1640kN) is found. This corresponds to a stress of 1.2 ksi (8.46MPa).

CAPWAP was used to further investigate the soil response of this pile. Figure 2 shows the final force and velocity matches (Fig. 2a uses acceleration as input to compute force, Fig. 2b uses force as input to compute velocity) and both are considered quite good. The total predicted capacity was 500 kips (2227kN). The skin friction is distributed rather uniformly with 350 kips (1560kN) indicated at the pile tip. However the indicated toe quake of 0.42 inches (10.7mm) 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 inches (3.6mm) and therefore mobilized only about half of the available resistance at the first reflection time. The maximum computed tension force from CAPWAP was 375 kips (1670kN).

An equally good CAPWAP match could be obtained with larger quakes provided the soil stiffness is not changed. It is possible that the toe quake and toe resistance are larger and that the total resistance should be similarly increased. When the hammer in refusal driving is not able to mobilize the full ultimate soil resistance, dynamic capacity analysis techniques cannot be expected to result in anything greater than the actual mobilized resistance.

A second CAPWAP analysis was performed using the same soil constants except using a standard 0.1 inch (2.5mm) quake at the pile toe. The force and velocity matches shown in Figure 3 are quite poor at 2L/c. The computed force no longer shows a net tension at 2L/c and the computed velocity is significantly reduced. 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. Tension is greatly reduced.

The CAPWAP soil constants were then used in the conventional wave equation analysis program WEAP (7). Two analysis were made; one with the observed quake of 0.5 inch (13mm) and one with the standard quake of 0.1 inch (2.5mm). Both runs used the observed 7.7 foot (2.35m) stroke. As seen in Figure 4, the capacity using a large quake at 20 blows per inch (25mm) is only half the capacity using a small quake. Actually driving beyond 100 blows per foot (300mm) yields little increase in capacity. The tension stresses are equally dramatic. With normal quakes, above 80 blows per foot (300mm) there is no net tension in the pile. With large quakes, the computed tension never is below 0.8 ksi (5.5MPa) 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 (890kN) resistance, a stroke of 5.9 feet (1.8m) is observed and the maximum tension is then an acceptable 0.3 ksi (2.1MPa).

This pile was later load tested after several weeks to a Davisson failure load of 1150 kips (5122kN). 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 feet or 13.7m penetration). Equally good results were obtained by comparing restrike capacity information on a 16 inch (405mm) dynamically tested pile driven to approximately 60 feet (18.3m) penetration and scaling the shaft friction to account for the different diameters, proving the inherent correctness of the dynamic testing techniques (see Figure 5). It is always recommended that at least some piles on each site be tested during restrike to properly assess the soils static strengths. 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 set-up provided additional capacity. It is perhaps a coincidence that the WEAP prediction with normal quakes matches the observed load test. If this load were used, however, and then a setup factor assumed, the resulting design would not have achieved a proper safety factor.

Case 2 - Mobile

Seven piles were tested dynamically in June 1979. All piles were 24 inch (610mm) square prestressed piles with total lengths below measurements of 122 feet (37.2m). All were prejetted to depths of at least 100 feet (30.5m). Driving was accomplished by a Raymond 80 hammer with a rated energy of 80 kip feet (109kJ).

The dynamic data of two piles is shown in Figure 6. Again a velocity increase is observed, followed by negative (upward or rebound) velocities. In both cases the blow counts were slightly in excess of 200 blows per foot (300mm) and skin friction was minimal. Proportionality between force and velocity for almost the entire first 2L/c indicates no reflections from soil resistance on the skin or pile cross section changes. Observed quakes from CAPWAP were about 0.7 inches (18mm). Although capacities were around 1000 kips (4455kN) in each case, the slow soil response associated with the large quakes produced tension stresses of 0.75 and 1.0 ksi (5.2 and 6.9 MPa) for piles G57A and E56A respectively. Tension stresses in other piles on this site reached maximums of 1.37 ksi (9.5MPa) at 50 blows per foot (300mm).

No trend in setup or relaxation was observed on this site as might be expected in a soil described as dense sand.

Case 3 - Norfolk

Twelve 18 inch (457mm) square prestressed piles were driven and tested dynamically using a Delmag D30 hammer. The pile lengths were 80 feet (24.4m) and the soil was described as a saturated dense fine sand with 30 to 40 percent silt or clay content. Again the piles were prejetted.

Blow counts were erratic at the end of driving, ranging from 20 to over 500 blows per foot (300mm). The example case shown in Figure 7 had 200 blows per foot (300mm). The maximum computed tension was 0.6 ksi (4.1MPa) at the end of driving and as large as 1.3 ksi (9.0MPa) at lower blow counts. Again indicated capacities are low 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 0.4 to 0.5 inches (10 to 13mm).

The analysis of the force and velocity traces revealed that one third of the piles had excessive structural damage (8) 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.

Large setup factors associated with the fine grained soil and cementation in some layers later provided adequate capacity as determined by re-strike 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 can and do cause structural damage, even in refusal driving conditions.

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

Reliance only on wave equation driving criteria could lead to unsafe foundations although in many cases soil strength gains as pore pressures decrease will 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 results in near refusal conditions are the best way to detect this behavior. Restrike testing by Case or CAPWAP methods should always be performed, especially sites with saturated soils, to confirm service load capacity. Although these electronic measurement techniques are preferred, accurate restrike blow count for the first few blows and stroke measurements for correlation with a wave equation analysis is the minimum requirement.

If large quakes are found, several corrective actions may be necessary. If jetting is used, it should be reduced or discontinued if possible. Non-displacement pile types could be considered. If concrete piles are long and tension stresses high, the ram stroke may be reduced causing lower compressive input and subsequent reflected tension stresses. Pile cushion thickness may be increased. In these cases, long spliced prestressed piles should be avoided as the prestress disappears at the splice and the splice will thus see the full tension force. Steel piles could be considered as an alternative. In some cases partial driving, waiting for pore pressures to decrease, and then continuing the driving process may be the only other practical solution.

Acknowledgements

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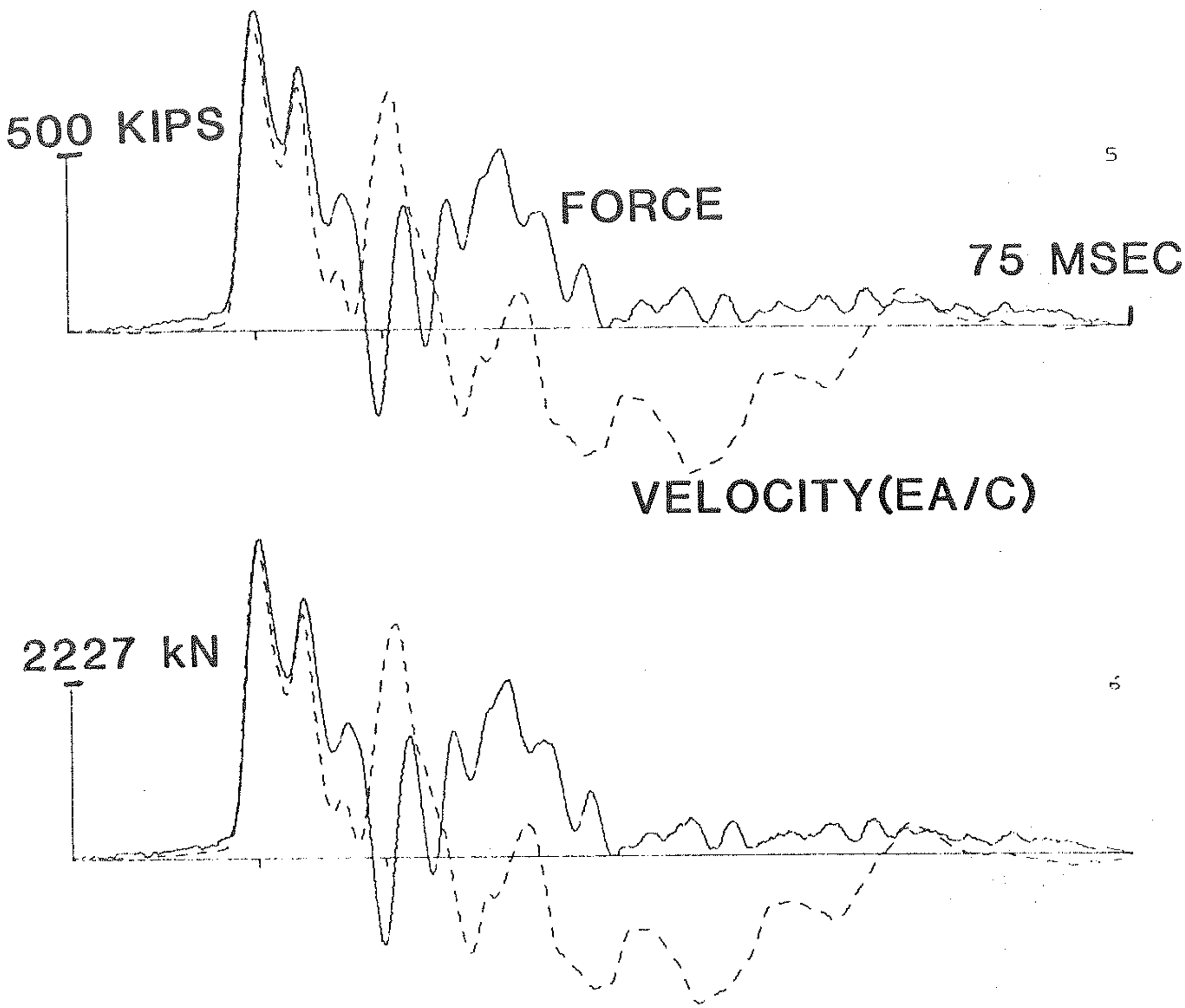

Figure 1 Dynamic Records From Seattle

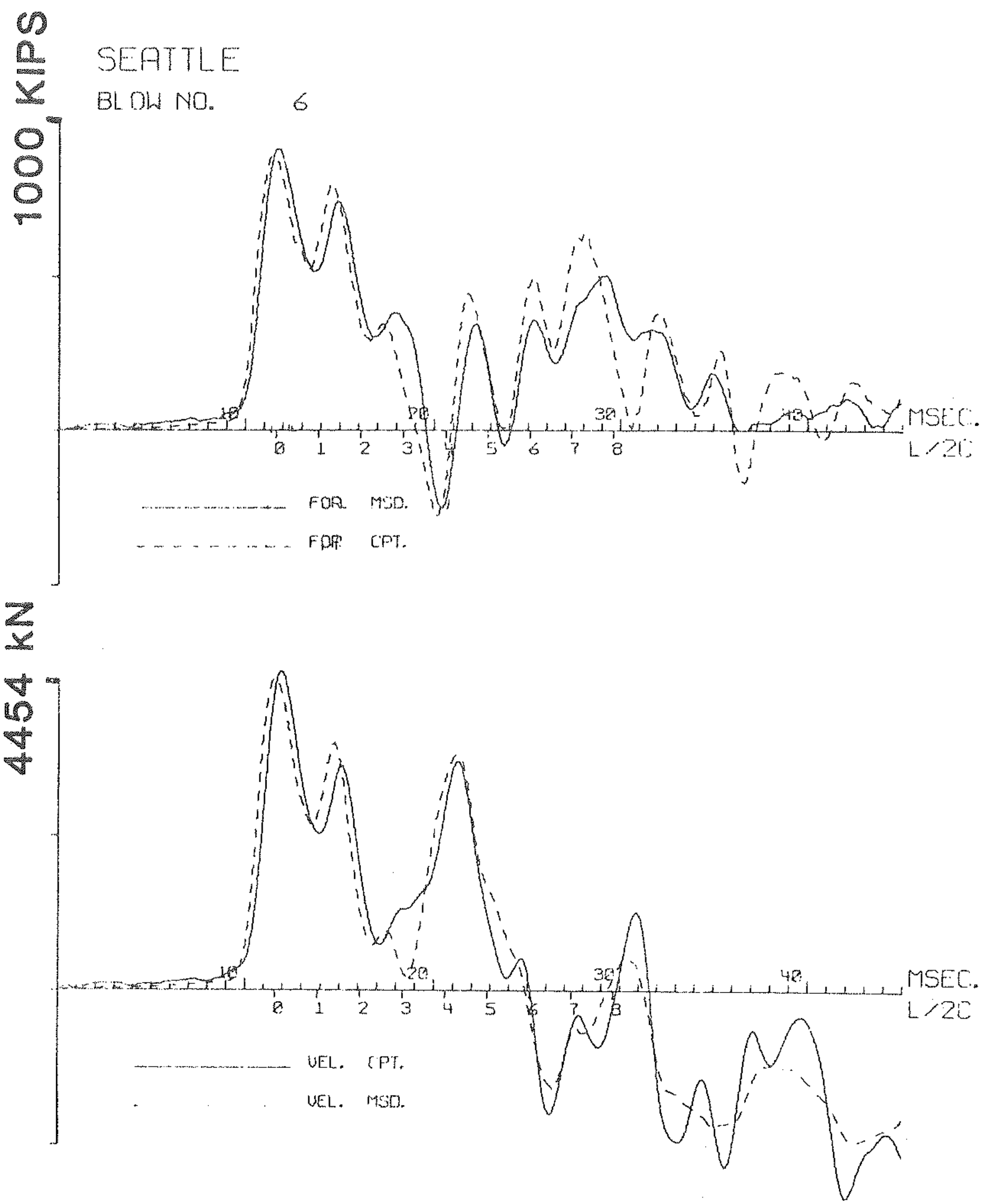

Figure 2 CAPWAP Results With Qt = 0.42 inch

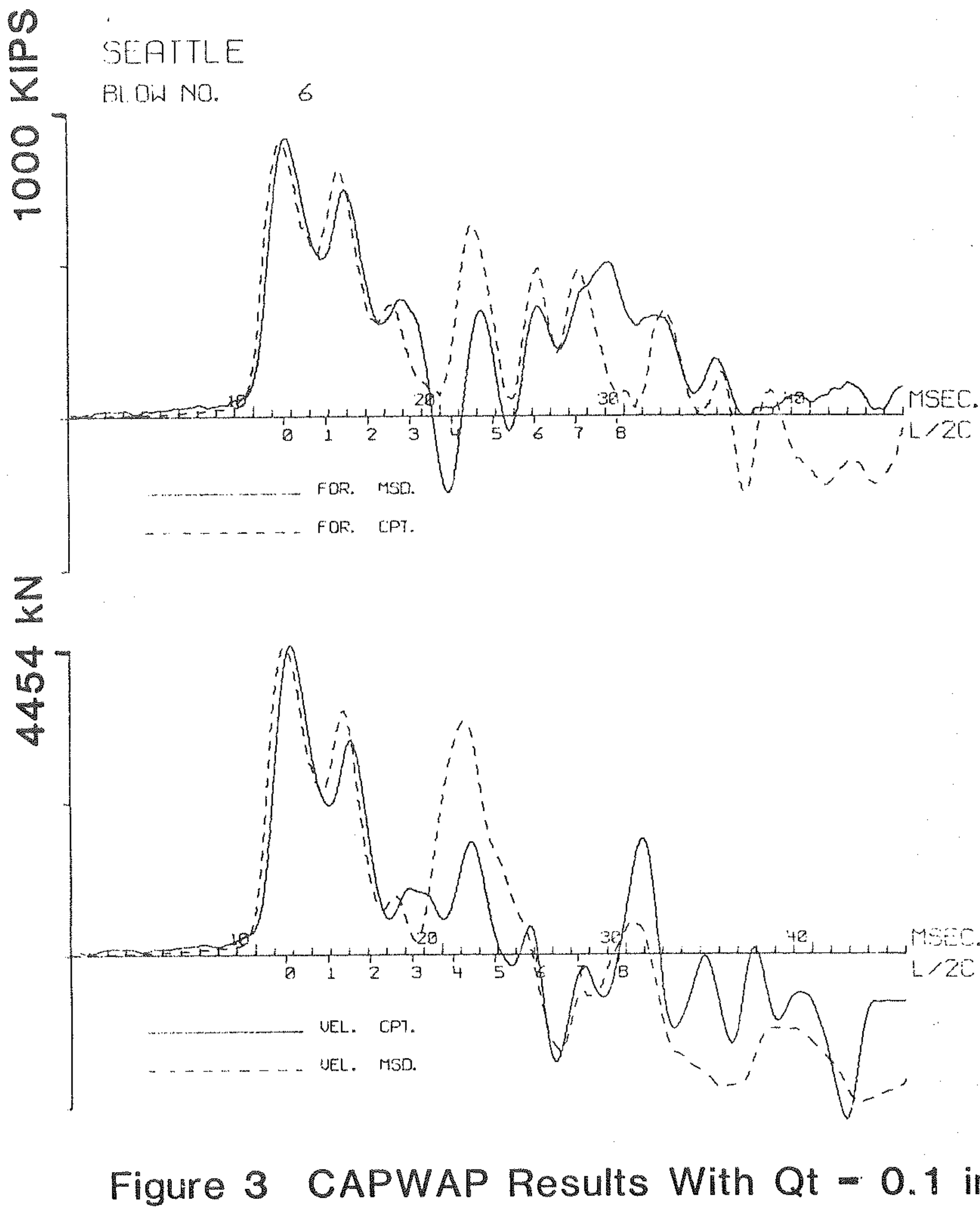

Figure 3 CAPWAP Results With Qt = 0.1 inch

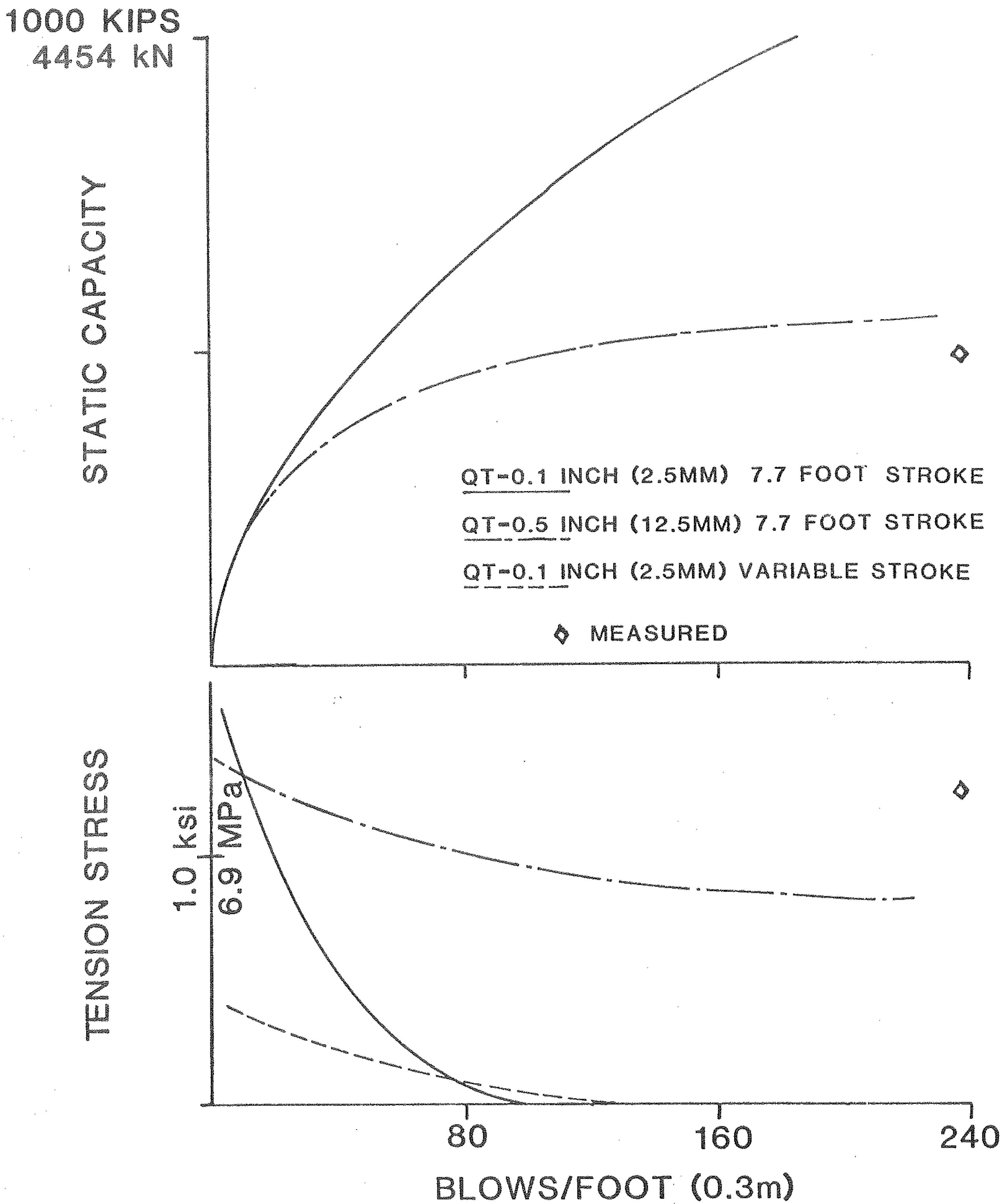

Figure 4 WEAP Wave Equation Results

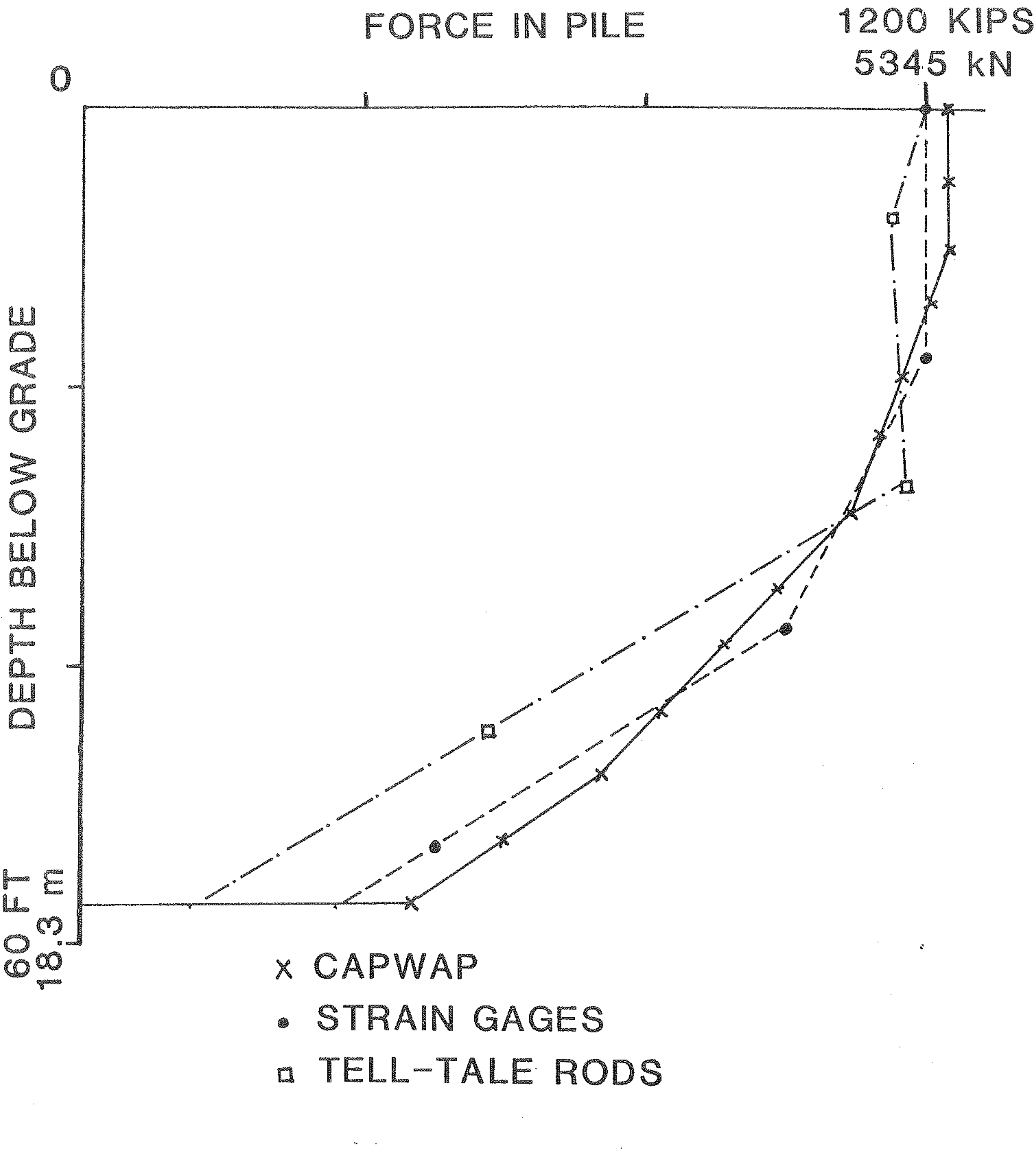

Figure 5 Resistance Distribution

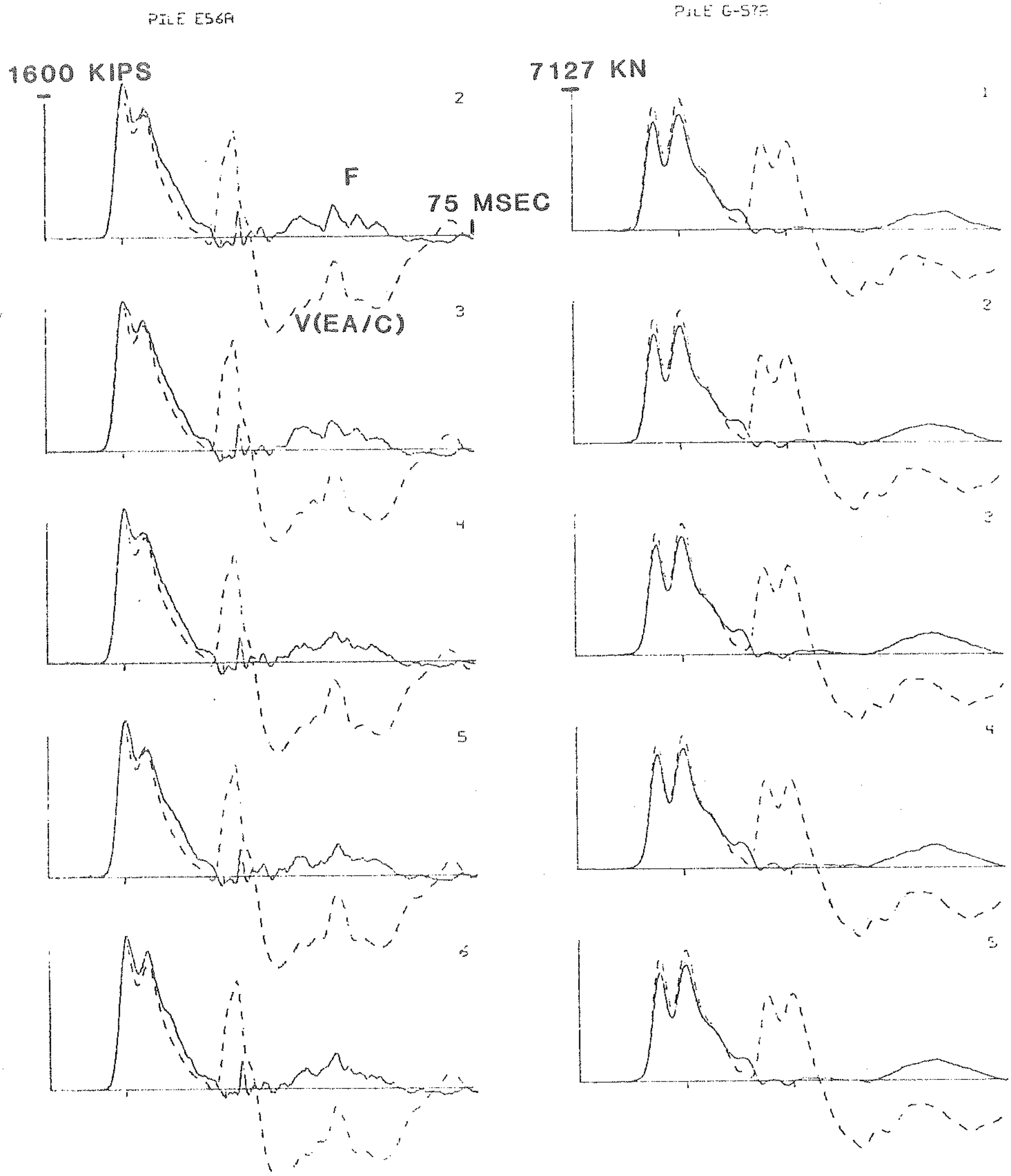

Figure 6 Dynamic Records From Mobile

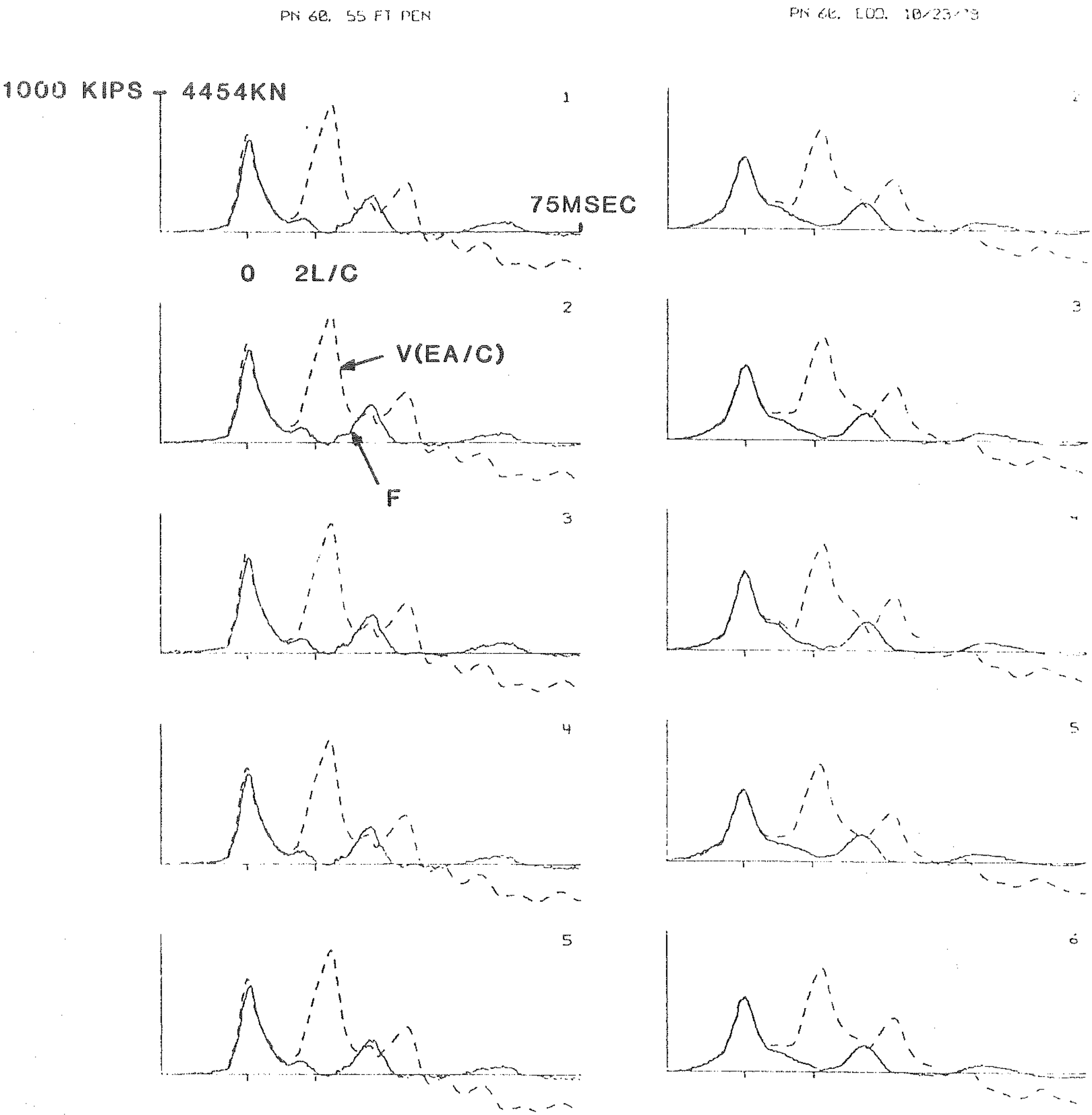

Figure 7   Dynamic Records From Norfolk

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Background of Capacity Interpretation
Using Dynamic Pile Measurements
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High Tension Stresses in Concrete Piles
During Hard Driving
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Load & Resistance Factor
Design of Piles
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Case Method
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