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QUAKE VALUES DETERMINED FROM DYNAMIC MEASUREMENTS

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In wave equation analysis of pile driving, it is ordinarily assumed that quake values are about 2.5 mm (Smith 1962, Goble and Rausche 1976, Hirsh et al. 1976). Although theoretical studies have been made on the influence of the different parameters used in wave equation analysis, the influence of the quake value has received little attention, only. Forehand and Reese (1964) correlated bearing capacity predictions with results from static loading tests using quake values ranging from 1.3 to 7.6 mm (0.05 in to 0.30 in). The same range of values was used by Ramey and Hudgins (1977) in a study of the sensitivity of the wave equation program solution to the soil parameters used in the analysis. From these studies, it was concluded that the original quake od 2.5 mm value proposed by Smith (1962) is sufficiently precise and that variations of this parameter do not greatly influence the program solution.

In the beginning of the use of wave equation analysis, there was no possibility of determining the quake values for the soils other than by the general correlation of the wave equation analysis with results of loading tests. However, some ten years ago, research at Case Western Reserve University, Cleveland, developed a technique of obtaining measurements of force and acceleration at the pile head during the driving by means of the Pile Driving Analyzer (Goble et al., 1970). The continued development work by the same group resulted in the CAPWAP program (Rausche et al., 1972), which processes the measured dynamic data to determine the soil parameters and the amount of static soil resistance acting on the pile.

In the CAPWAP technique, the computer takes the measured acceleration wave and computes by means of wave equation theory a force curve, which is compared (matched) to the measured force trace. The computation uses mainly six variables — side and toe quake, side and toe damping, and static resistance along the pile shaft and at the pile toe. By changing these variables, the operator strives to achieve agreement (a match) between the computed and measured force traces. The report of the results of the analysis include, amongst others, the ultimate static resistance in the pile and its distribution, the soil quakes, and the soil damping values, as they were assumed in the CAPWAP analysis for the final match.

The CAPWAP technique was used to analyze the results obtained at two different sites showing a large soil quake at the pile toe, as presented in the following.

CASE 1

During a research project on the application of the Pile Driving Analyzer techniques to the Canadian practice, sponsored by the Canadian Government (Fellenius et al., 1978), dynamic data were analyzed from a total of 21 sites across Canada. On one site, closed-toe pipe piles, 324 mm O.D. with wall thicknesses of 7.9 mm, 8.4 mm, and 9.5 mm, were driven into a very dense sandy silty glacial till. Four different hammers were used on the site — one drop hammer and three open-end diesel hammers. The nominal (rated) hammer energies were 27 KJ for the drop hammer and 39 KJ, 46 KJ, and 62 KJ, respectively, for the diesel hammers. Sixteen piles were monitored with the Pile Driving Analyzer.

The average impact stresses for the drop hammer and the diesel hammers were 138 MPa and 117 MPa, 172 MPa and 207 MPa, respectively (20 ksi and 17 ksi, 25 ksi, 30 ksi, respectively). The average transferred energy for the drop hammer was 14 KJ (9 ft-kips). For the diesel hammers, the transferred energies were 11 KJ, 20 KJ, and 41 KJ, respectively (6 ft-kips, 13 ft-kips, 26 ft-kips, ft-kips, respectively). The corresponding energy ratios were 50 % and 28 %, 44 %, and 66 %, respectively. Consistently, the lighter the diesel hammer, the smaller the ratio of transferred energy. Static loading tests showed that the lightest diesel hammer was not able to drive the piles to a sufficient bearing capacity at this site, although this hammer had previously been proven to be adequate for the installation of the same size of piles tp the same desired cpaapcity on other sites in the same general area. During the Analyzer monitoring work, it became evident that the characteristics of the pile-soil system were unusual. This was indicated by the force and velocity wave shapes at termination of driving, which showed an apparent lack of toe resistance at time 2L/c followed by a substantial positive reflected force wave. A representative example of the records is given in Fig. 1. The apparent lack, or, rather, the delay of toe resistance to occur after Time 2L/c resulted in values of bearing capacity calculated by the Analyzer using the Case Method Estimate (CMES), which were considered to be on the conservative side.

In Fig. 2, the results are shown of CAPWAP force matches with, first, the ordinary value of toe quake of 2.5 mm and, then, with a value of 20 mm. A good force match was not possible to achieve with the smaller quake value, only with the larger.

The piling work and CAPWAP analyses took place in June 1976. It was the first time that quake values much larger than the generally assumed value of 2.5 mm were indicated.

CASE 2

Recently, another case was encountered where the Analyzer wave traces indicate a large value of soil quake at the pile toe. Twenty-four 305 mm square precast concrete piles were driven through an about 11 m thick clay deposit and into underlying dense clayey silty glacial till. The pile driving was by means of a Berminghammer B-400 open-end diesel hammer having a rated energy of 62 KJ (40 ft-kips). The pile cushion consisted of layers of plywood.

All piles were monitored with the Pile Driving Analyzer and the dynamic data obtained were similar for all piles.

The driving through the clay was very easy and required a few light blows, only. When the pile toe reached the upper surface of the glacial till at about 11 m depth, the penetration resistance was about 5 blows/0.2 m. Within a penetration of about one metre into the glacial till, the resistance increased to about 40 blows/0.2 m. Then, during the last 150 mm of penetration, the resistance increased from an initial value of 10 blows/cm to a final value of 20 blows/cm. Restriking the pile after one hour gave a resistance of 23 blows/cm. Set-rebound measurements at the end of initial driving, and at restriking, indicated a set of 0.5 mm/blow and a rebound of 15 mm giving a maximum displacement of the pile head of 16 mm. The maximum displacement of the pile toe can be estimated to be about 6 mm by subtracting from the pile head displacement value the calculated value of elastic compression, i.e. 10 mm.

Measurements at two depths have been selected for presentation in this paper; at 11.3 m, when the pile end had penetrated only about 0.3 m into the glacial till and the driving was easy, and at 12.5 m, which is the depth at end-of-driving. The observed dynamic data, including the Analyzer measurements, are compiled in Table 1.

The wave traces, which were recorded at the two depths, are shown in Fig. 3. Both sets of wave traces show the same behavior as observed in Case 1, i.e., a velocity increase at time 2L/c and a delay in the toe-force reflection. In easy driving, upper diagram, the velocity increase is very pronounced, almost indicating a total lack of toe resistance, and the reflection delay is almost two L/c units. At the end of driving, lower diagram, the velocity increase and the reflection delay are less pronounced, but still clearly discernible.

To calculate the pile capacity from the Analyzer measurements by means of CMES directly, a damping value, J, of 0.2 should be applied in this soil. However, as shown in Table 1, this results in capacity values, which are smaller than one normally would be willing to accept as representative of the mobilized pile capacity at the driving. Even the capacity applying J = 0 is considered low considering the penetration resistance and previous experience with the hammer-pile system used.

The reason for the low values is, of course, the reflection delay causing the positive toe reflection to be eliminated from the calculation. It has been proposed that a time delay method be applied to offset the effect of the reflection delay. As indicated in Table 1, the maximum time-delay capacity values are about 30 % higher than the undamped conventional Analyzer capacity. It is probable that a damped time-delay capacity would be about equal to a CAPWAP computed capacity (see below), suggesting that the time delay approach could be used to offset the low conventional values. However, until this is further verified in the field, in cases such as the illustrated Cases 1 and 2, the Analyzer data had better be calibrated by means of a loading test and/or CAPWAP analysis.

CAPWAP analyses were subsequently performed on two representative blow records, one from the depth of 11.3 m and one from 12.5 m. The force matches obtained are given in Figs. 4 and 5 for traces from the depth of 11.3 m and 12.5 m, respectively. For reasons of comparison, the best force match which could be obtained, when applying the conventional quake of 2.5 mm, is given in the upper half of each figure. The matches are quite poor, and the results of the calculations, consequently, or illustrative value, only.

In the lower halves of Figs. 4 and 5, the best matches are shown as obtained with quake values of 20 mm and 8 mm, respectively. The matches, in contrast to those shown for the 2.5 mm quake, are quite good, clearly indicating the necessity of adjusting the computations to the quakes.

It is possible that in the termination driving, depth 12.5 metre, the pile did not mobilize the full static resistance of the soil. The computed maximum pile toe displacement is only about 9mm to 10 mm, which is about equal to or smaller than the quake values of 8 mm and 15 mm assumed in the CAPWAP analysis. A good force match can be achieved with any quake value as large or larger than the displacement value, provided the soil stiffness is kept the same. The mobilized static resistance is then obtained by multiplying the value of the displacement with the value of the soil stiffness. Thus, the CAPWAP analysis results in a minimum quake value equal to the maximum pile toe displacement. The actual quake value of the soil could well be considerably larger.

The foregoing discussion is illustrated in Fig. 6, which shows the force match for a quake of 20 mm. The match is about as good as the match obtained with the 8 mm quake. When the purpose of the CAPWAP analysis is to determine the mobilized static capacity of the pile, the actual quake value used is not important. This should not be understood as if the CAPWAP analysis provides a freedom of quake value to choose. Had the Smith model been built in terms of a certain soil stiffness within the zone of elastic static soil resistance, instead of a quake value, this would have been very clear. As seen in Table 1, the two force matches give the same value of mobilized static soil resistance.

Obviously, when the available hammer energy is not sufficient to mobilize the ultimate soil resistance, as in the present case of refusal driving, neither the Analyzer CMES capacity nor the capacity determined in a CAPWAP analysis can result in anything but the mobilized soil resistance. This capacity can, naturally, be regarded as a least capacity and used as such in the technical design or quality control considerations, as the case may be.

If the capacity is established by means of static loading test, the quake can be determined from the value of ultimate static toe-resistance divided by the soil stiffness value computed in the CAPWAP analysis, provided the driving data analyzed are obtained from restriking the pile at the time of the loading test. In the present case, a loading test for proof testing reasons was performed two days after the driving. The pile withstood a maximum load of 2,800 KN without showing signs of failure. An approximate extrapolation of the load-movement curve suggests a Davisson Limit value of about 3,200 KN. However, the loading test was carried out after the pore pressures induced by the pile driving had dissipated. Therefore, a soil set-up (freeze) must have taken place and the capacity, at the time of the loading test, must in all likelihood have been greater than the static capacity available at the refusal driving. As no restriking was carried out after the loading test, neither the pile-toe quake nor the stiffness of the soil at that time is known. It is probable that both these values changed during the reconsolidation of the soil.

The reason for the unusually large quake observed in the two described cases is not known. The Authors believe it to be related to pore pressure build-up in the soil. However, it is not usually observed at other sites, where similar soils are found. It should also be recognized that the pore pressure dissipation does not always have to result in an appreciable soil set-up.

The occurrence of a large quake has practical importance. Where large quakes occur, a given hammer will not be able to drive a given pile to the capacity possible where the ordinary small quake occurs.

Wave equation analysis with the WEAP program (Goble and Rausche, 1976) is performed for the pile at a depth of 12.5 m using data for the actual hammer and applying the damping factors determined in the CAPWAP analysis. The cushion stiffness was determined to 1,300 MN/m by repeated runs matching the computed values of force, energy and velocity to the Analyzer measured values. Several WEAP runs are made using varying values of pile-end quake. The results are shown in the Bearing Graph in Fig. 7.

The curves in Fig. 7 indicate that when the soil quake increases, the soil stiffness decreases, and, consequently, the maximum capacity to which the hammer can drive the pile reduces. At a site, where the ordinary 2.5 mm quake occurs, the particular hammer-pile-soil combination would be able to achieve a capacity of about 3,000 KN at a practical and economical specified termination resistance of 8 blows/10 mm ("refusal"). As the quake increases, and the soil stiffness decreases, not only does the maximum attainable capacity decrease, the limit of the practical and economical termination criterion reduces, also. In the event of a quake of 15 mm, or rather, a stiffness of about 100 MN/m, not much capacity is gained by driving to a greater resistance than about 3 blows/10 mm.

The Authors believe that large quakes occur more often than one at first would think, but that the soil set-up usually improves the final capacity of the piles so that the inadequate capacity to which the pile has been driven goes undetected. There are, however, many case histories told, where contractors have failed to provide piles with the specified minimum capacity, and where they, subsequently, have been accused of not doing the job properly, and held to add piles, or improve the situation by bringing out larger hammers, etc., to a considerable extra cost to themselves, and/or to the owners. It is quite possible that in many of those cases, the contractor and his original equipment were innocent, and that the blame lies in a large quake without a following soil set-up of appreciable magnitude.

Only the analysis of independent measurements of force and velocity can reveal the existence of a large quake. The presented two case histories provide a sound argument for performing such dynamic measurements.

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TABLE 1 DYNAMIC DATA FROM CASE 2

	UNIT	DEPTH 11.3 m	DEPTH 12.5 m	
DRIVING DATA				
Measured Penetration Resistance	BL/0.2 m	5	400	
Measured Net Penetration Per Blow	mm	40	0.5	
Measured Rebound Per Blow	mm	15	15	
Estimated Maximum Pile Toe Displacement	mm	45	6	
ANALYZER DATA				
Peak Impact Force, FIMP	KN	1,600	2,200	
Transferred Energy, EMAX	KJ	20	21	
CMES Capacity (J = 0.2)	KN	200	1,700	
CMES Capacity (J = 0.0)	KN	660	2,200	
Maximum Time-Delay Capacity (J = 0.0)	KN	1,000	2,900	
CAPWAP DATA				
Assumed Total Capacity	KN	800	2,200	3,200
Assumed Toe Capacity	KN	600	1,900	2,900
Assumed Shaft Capacity	KN	200	300	300
Toe Quake	mm	20	8	15
Shaft Quake	mm	4	2.5	2.5
Soil Stiffness at Toe, K_{soil}	MN/m	30	230	200
Soil Coefficient of Restitution at Toe	- -	1	0.8	0.8
Computed Maximum Toe Displacement, DMAX	mm	30	9	10
Mobilized Total Resistance	KN	600	2,000	2000
Mobilized Toe Resistance	KN	800	2,300	2,300
Case Damping Factor, J_{toe}	- -	0.07	0.22	0.24

CASE 1

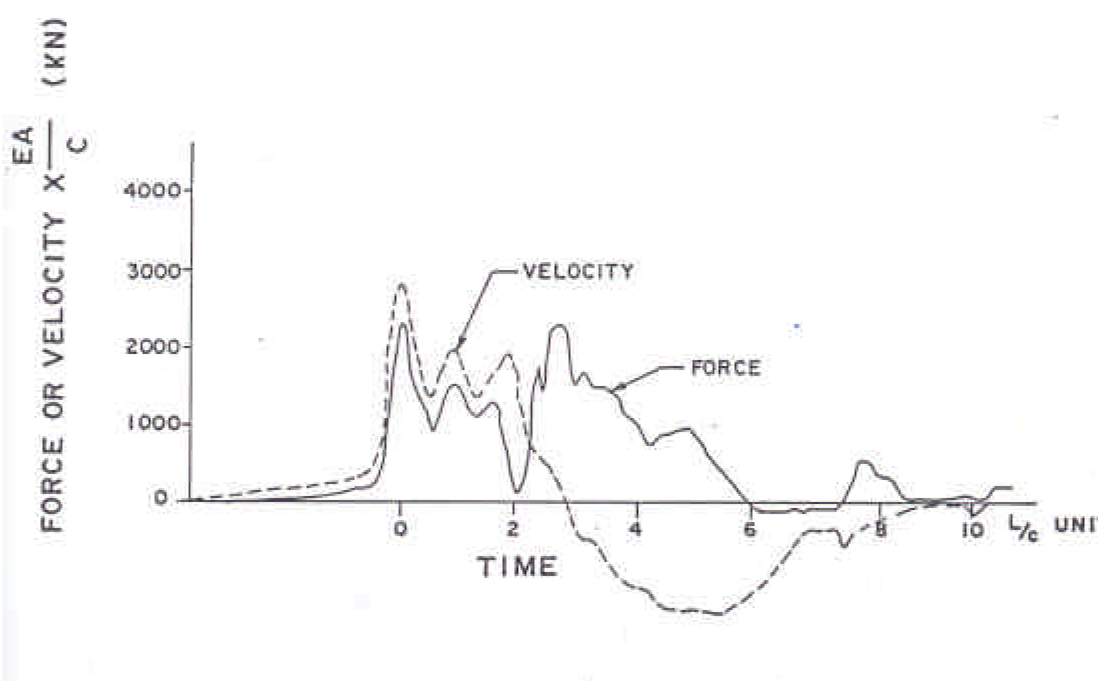

FIG. 1 Force and Velocity Traces, Case 1

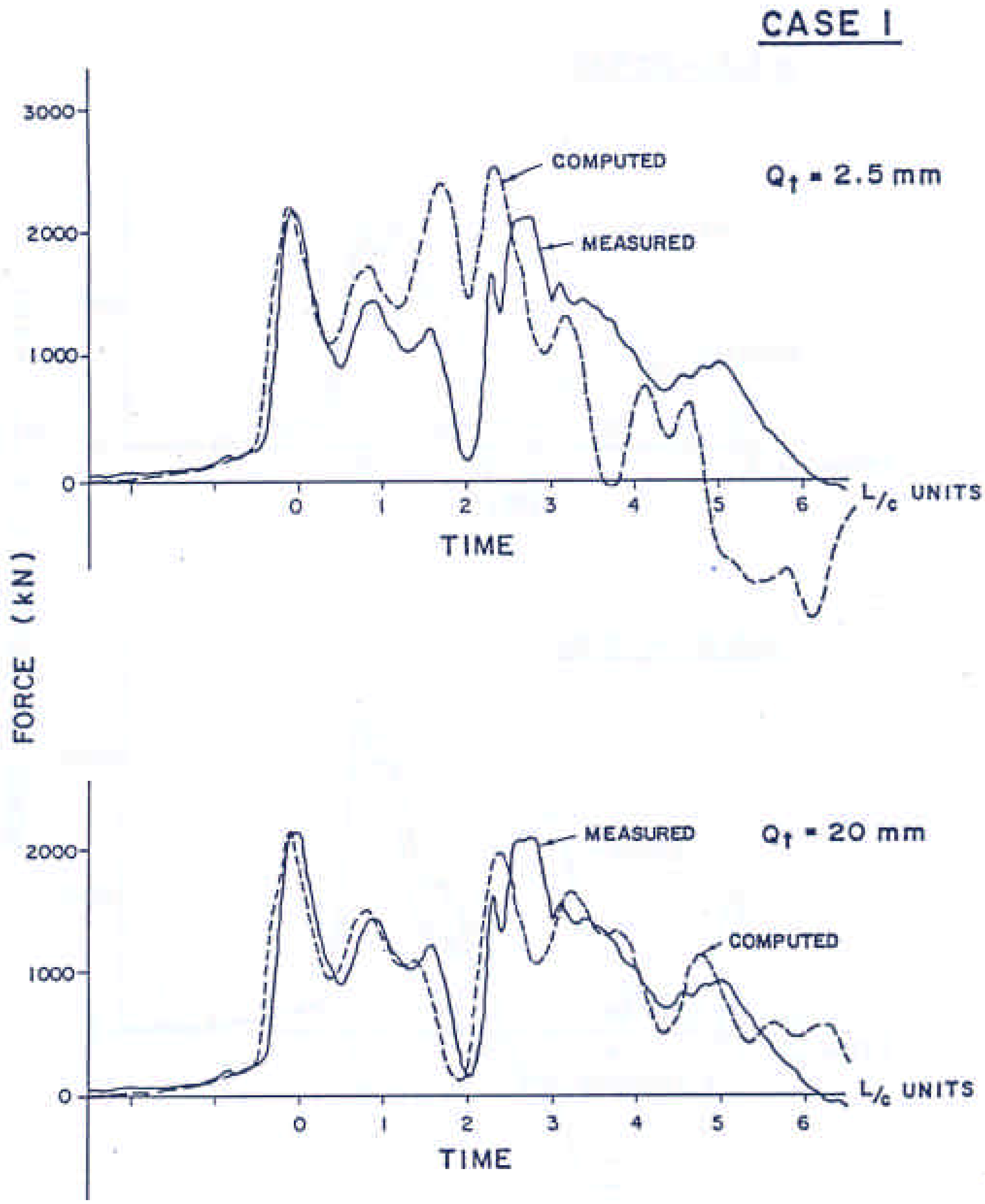

FIG. 2 CAPWAP Force Match for Pile Toe Quake (Q_t) of 2.5 mm and 20 mm, Case 1

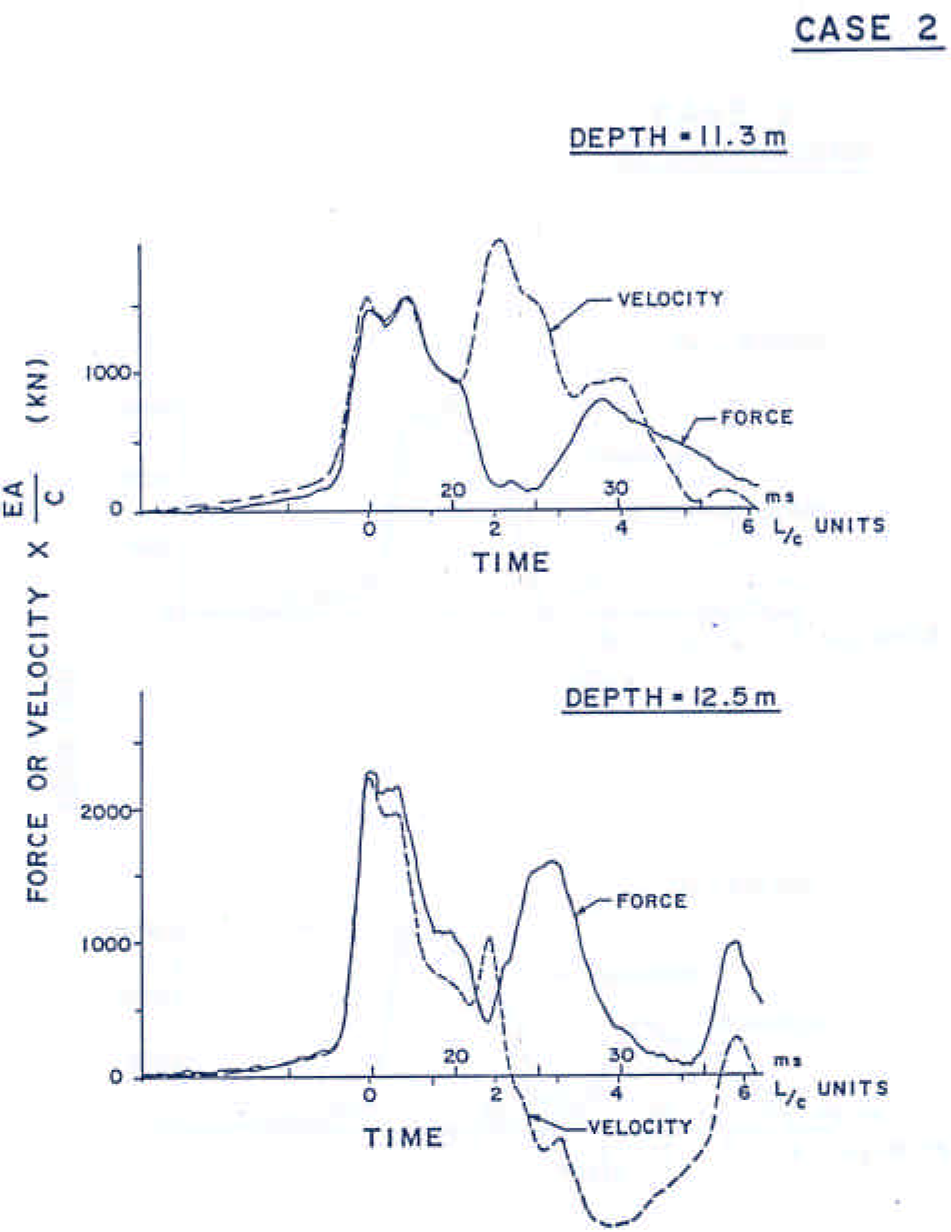

FIG. 3 Force and Velocity Traces at Depths of 11.3 m and 12.5 m, Case 2

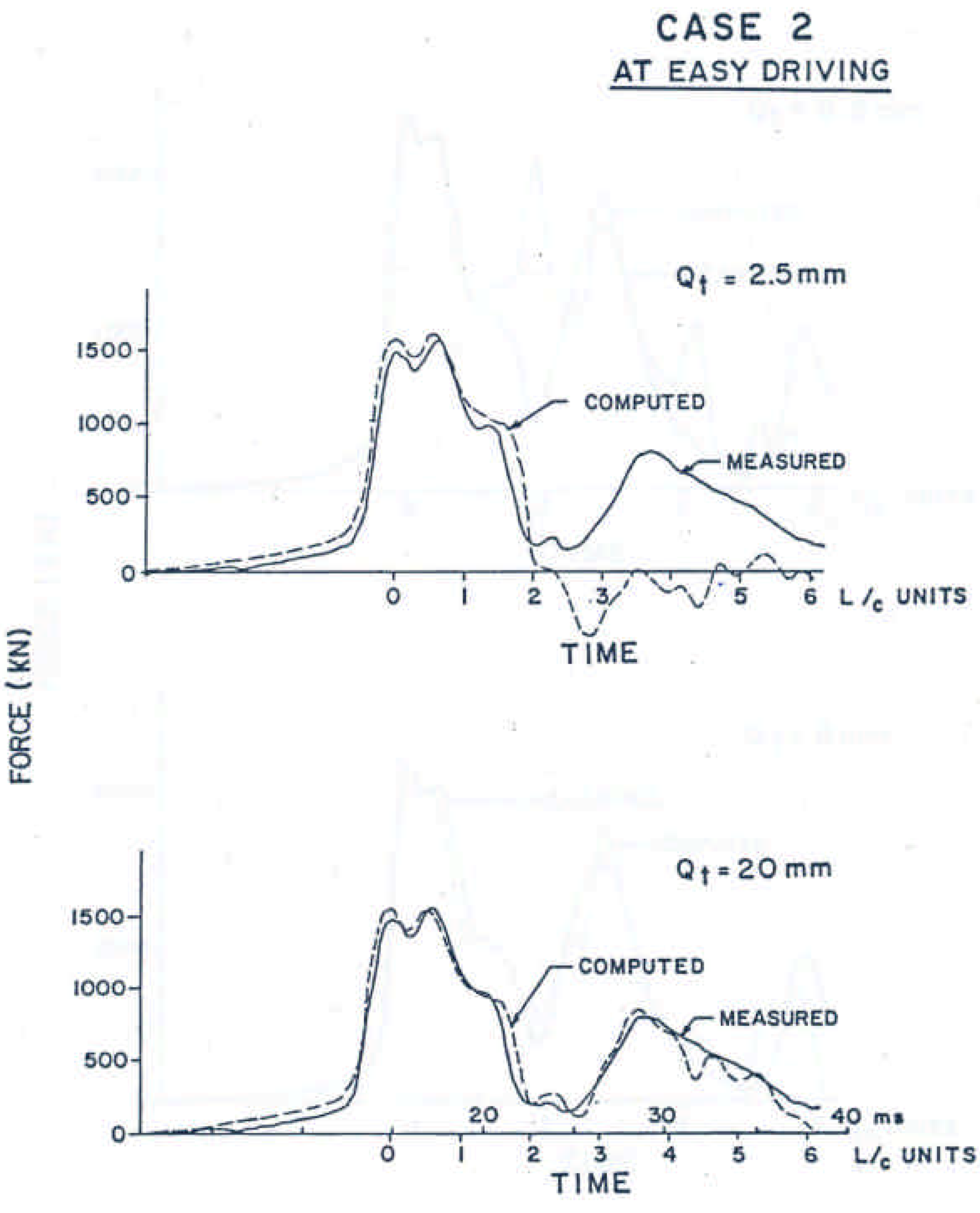

FIG. 4 CAPWAP Force Traces for Pile Toe Quake (Q_t) of 2.5 mm and 20 mm, Depth 11.3 m, Case 2

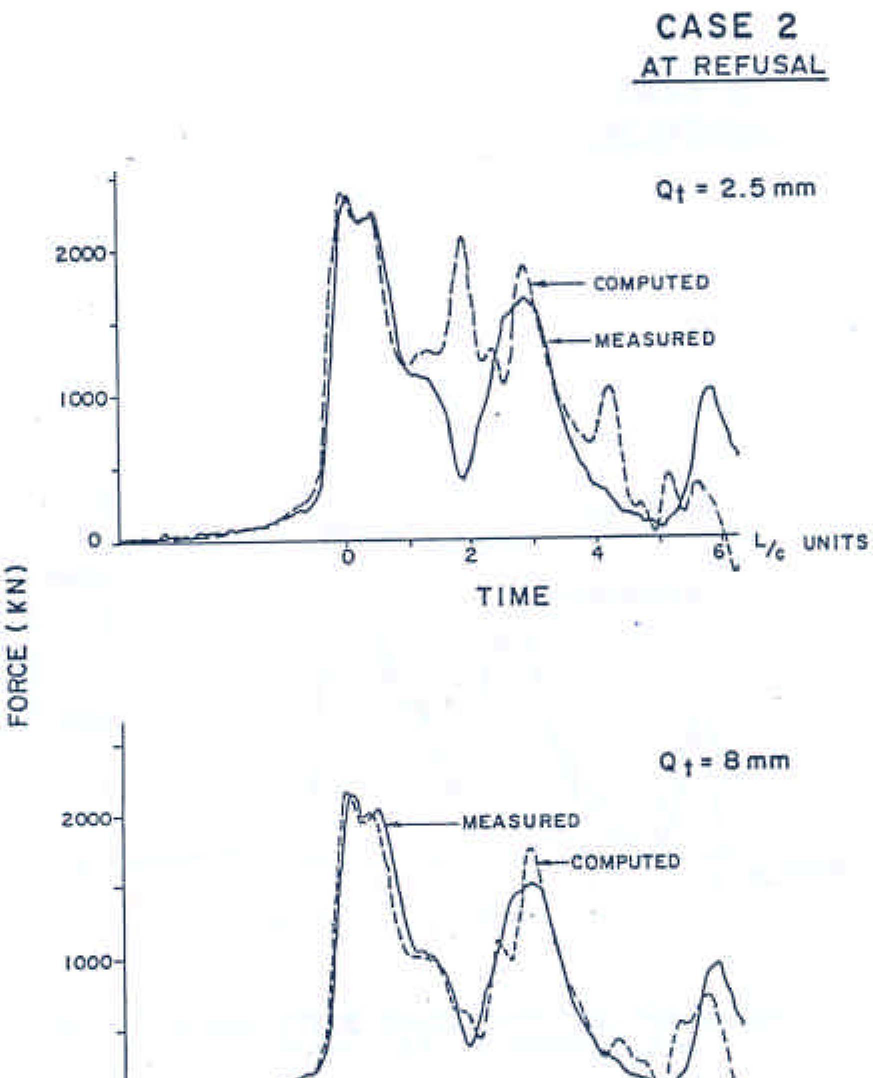

FIG. 5 CAPWAP Force Traces for Pile Toe Quake (Q_t) of 2.5 mm and 20 mm, Depth 12.5 m, Case 2

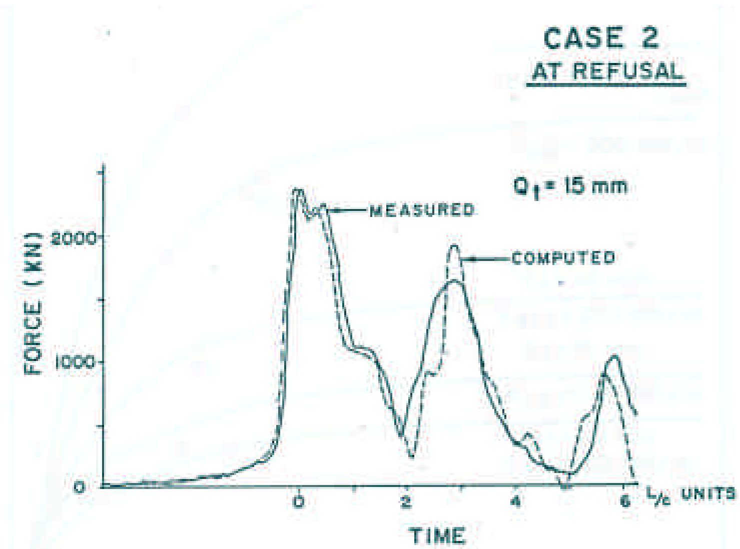

FIG. 6 CAPWAP Force Traces for Pile Toe Quake (Q_t) of 15 mm, Depth 12.5 m, Case 2

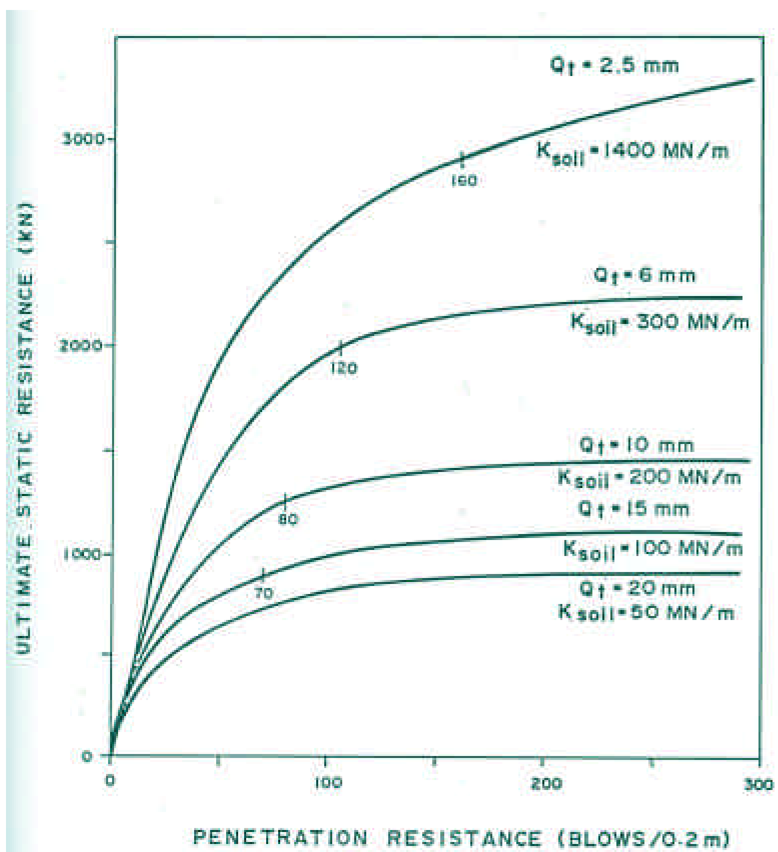

FIG. 7 Bearing Graph from WEAP Analysis with Varying Pile Toe Quake (Q_t) and Soil Stiffness (K_{soil})