INSTALLATION OF HIGH CAPACITY CONCRETE PILES AT BALLANTYNE PIER, VANCOUVER, B.C.

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ABSTRACT

A review of the results of the installation of large diameter concrete piles during the redevelopment of Ballantyne Pier is presented. A total of 154 high capacity 914 mm diameter hollow precast prestressed concrete piles were driven to depths varying from 15 to 40 m through the existing granular embankment fill to end bearing in the dense underlying strata using an HPSI-3505 model hydraulic impact hammer. The mobilized geotechnical capacity of the piles was variable, being influenced by the behavior of foundation soils to repetitive loading, although the stratum supporting the piles has relatively consistent geotechnical characteristics. Dynamic monitoring carried out on 23 piles (15%) was useful in confirming the available geotechnical pile capacity.

INTRODUCTION

Ballantyne Pier is located at the north foot of Heatley Avenue on the south shore of Burrard Inlet in the City of Vancouver, between Centennial Pier to the west and Burlington Northern Pier to the east (see Figure 1a). The pier is approximately rectangular in shape and is about 100 m wide and 360 m long. The central portion of the pier is composed of a granular embankment which varies in thickness from about 10 m at the south end to about 30 m at the north end. The outer aprons are supported on a series of existing 2.1 m diameter concrete caissons installed to firm bearing strata. An aerial view of the pier looking southeast is shown in Figure 1b. A typical cross section, taken at the mid-point of the pier in the east-west direction, is shown in Figure 2.

By the late 1980's, the 70 year old pier was in poor condition, with major deterioration in the tidal zone and apron structures. In addition, the four original cargo sheds had become obsolete due to low ceilings and narrow column spacings. In 1991, to upgrade the aging infrastructure of Ballantyne Pier, the Vancouver Port Corporation (VPC) initiated a redevelopment program for the pier. The intent was to transform the pier into a modern, multi-use marine terminal combining general cargo handling and cruise passenger facilities, while retaining the heritage features. The redevelopment program included strengthening of the existing concrete caissons both for static and seismic loading conditions, replacing the old deck with a new reinforced concrete deck, strengthening the existing reinforced concrete bulkhead wall and installing additional high capacity piles to support the increased loading imposed by the new warehouse apron.

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This paper provides a review of the results of the pile installation carried out at the Ballantyne Pier with emphasis on the factors which affected the driven capacities and variations in driving conditions.

SOIL CONDITIONS

The overall soil stratigraphy at the Ballantyne Pier site can be described by considering the following three main strata having different geotechnical engineering characteristics:

- granular embankment fill
- seabed sediments
- dense underlying strata

The granular embankment, originally constructed by barge dumping of Seymour River delta sands and gravels under water, is in a state of loose to compact in-situ relative density. Typical equivalent SPT N values taken within this fill, which contained some cobbles, varied from about 8 to about 15 blows/0.3 m. Overall, the embankment material is well graded with sand, gravel, cobble and boulder size particles, with the upper 2 to 3 m being generally coarser. Typical gradation variations of the embankment fill material obtained using the Becker open casing (110 mm ID) are shown in Figure 3a.

Seabed sediments of soft to very soft consistency, and up to 3 m in thickness, underlie the embankment fills towards the peripheral and seaward areas of the embankment. In central areas of the embankment, the seabed sediments are inferred to be either mixed with the granular fills or have been displaced non-uniformly during barge dumping.

The dense stratum below the granular fill and seabed sediments consists of a 4 m to 6 m thick layer of silty sand to sand and silt. A majority of the piles were driven into this dense stratum. Typical gradation curves of this material obtained along the pier length are shown in Figure 3b.

A complex sequence of very dense to dense sand and silt (or weak sandstone) inter-layered with very stiff silty clay or weak claystone underlies the upper dense stratum.

Typical penetration resistance measurements observed at several locations at mid-point and along the centerline of the pier are shown in Figures 2 and 4, respectively.

FOUNDATION PILES

One hundred and fifty four (154) hollow precast prestressed concrete piles with lengths varying from 15 to 40 m were installed for foundation support. All the piles were installed vertically. The piles had an outside diameter of 914 mm and a wall thickness of 125 mm. Initially, both closed-toe and open-toe piles were considered.

The ultimate geotechnical compression capacity required was about 7,500 kN/pile, whereas the ultimate geotechnical tension capacity required was close to 1000 kN/pile.

TEST PILE PROGRAM

In order to evaluate the driveability and the likely compression and tension capacities that could be mobilized, a test pile program was undertaken during the summer of 1992. A total of six (6) piles were installed over a variety of fill thicknesses along the pier. The lengths of the installed test piles varied from about 18 to 40 m. The piles were installed using a CONMACO-5300 air/steam hammer with a theoretical maximum rated energy of 203 kJ (150 kip-ft) per blow. The test pile installation was carried out by Fraser River Pile and Dredge Ltd. and was supervised by Sandwell Inc. with periodic monitoring by Golder Associates.

The locations of the six test piles (A1, A2, A3, 93C, 145B, 120D) are shown in Figure 4a. All the test piles were installed through 1.05 m diameter steel casings extending down to a depth of about 3 to 6 m below deck level. The purpose of the steel casings was to guide the concrete piles. The casings were installed using a vibratory hammer and the inside of each casing was cleaned out by augering prior to installing the concrete piles. In most cases, the steel casings were withdrawn during the installation of each concrete pile.

Pile Driveability

Dynamic monitoring of the force and velocity variations with time were carried out on four of the test piles (A1, 145B, 93C and 120D) at the end-of-initial-driving and beginning and end-of restrike to evaluate the pile capacity, hammer performance, driving stresses and pile damage. The dynamic monitoring and interpretation of data for the test pile program was carried out by Anna Geo Dynamics of Ottawa, Ontario.

The results from the reported CAPWAP analyses indicate that the mobilized compression capacity was variable ranging from about 5800 kN to 12,000 kN/pile. It was observed that pile penetration into the dense underlying stratum with closed-toe conditions was difficult resulting in a lower mobilized geotechnical capacity. On the other hand, the piles with open-toes penetrated the dense stratum more readily. The capacity that was mobilized in these open-toe piles was higher than the capacity of closed-toe piles. In most cases, the blow counts at final set were greater than 10 blows/25 mm suggesting that the full capacity of the piles was not mobilized.

Based on the results of the test pile program, it was concluded that the piles should be installed with open-toes to facilitate deeper penetration so that higher capacities could be achieved without the need for excessive driving forces that would likely cause pile damage.

Load Test

Test pile A1 was subjected to a tension load test to confirm the design uplift capacity. The testing was carried out generally in accordance with ASTM Standard D3689. The results indicated that the pile could withstand a geotechnical tension load of 3000 kN (including the self weight of the pile). The tension test was terminated at a load of about 3300 kN due to cracking of pile head. The load-deformation curve for this pile is shown in Figure 5.

For comparison, a CAPWAP analysis carried out by Anna Geo Dynamics on restrike blows recorded for pile A1 indicated an ultimate shaft resistance of about 3400 kN which was slightly higher than the effective tension load applied to the pile.

PRODUCTION PILE DRIVING

Production pile driving was carried out by Peter Kiewit & Sons using an HPSI-3505 hydraulic impact pile driving hammer which had a ram weight of 16,000 kg. The hammer was capable of operating at a wide range of strokes; 0.15 m to a theoretical maximum of 1.5 m with a corresponding range in maximum rated energy from 31 kJ/blow to 235 kJ/blow.

A total of 154 piles were installed over a period of approximately 6 months. Detailed pile driving information was recorded by Sandwell Inc. with periodic review of results by Golder Associates. A typical pile driving record is shown in Figure 6.

The pile configurations consisted of single piles and groups of four (4) and six (6) piles arranged in a rectangular grid pattern. The center to centre spacing between piles varied from about 3 m to 6 m.

Figure 4b shows the measured toe elevations of some selected piles along the length of the pier. A majority of the piles shown in Figure 4b have been subjected to dynamic monitoring including detailed CAPWAP analyses. The inferred extent of upper dumped granular fill and the depth to competent rock strata estimated from the field investigation programs are also shown. As can be seen, the majority of the piles were driven to depths that were several meters into the dense stratum underlying the granular fill and any sediments with the occasional pile tip reaching bedrock.

Driving of each pile was terminated upon achieving the required set and/or confirmation of pile capacity by dynamic measurements. A total of 23 piles (15 percent) out of the 154 piles were subjected to dynamic monitoring. Of these 23 piles, CAPWAP analyses were carried out on 20 selected piles. Dynamic monitoring and data interpretation was carried out by Trow Consulting Engineers Ltd. of Brampton, Ontario.

A total of nine (9) piles (i.e. Piles 5, 6, 11, 11A, 25, 28, 49, 96 and 97), or 5 percent of the total number of piles driven, were heavily damaged during installation and required repair and replacement. Piles 5, 6 and 11 were inferred to have encountered an obstruction prior to reaching the design depth and capacity, in comparison to the surrounding piles. The remaining piles were identified as broken at depth by dynamic monitoring.

Retrofit measures for the broken piles included installation of a 500 mm diameter steel pipe pile through the damaged concrete pile and grouting the annulus to form a composite pile capable of carrying the design loads. In some cases, additional supplementary steel piles were driven outside of the concrete piles.

SHAFT RESISTANCE

In free draining soils such as sand and gravel, it is often assumed that the excess pore water pressures generated during driving-induced soil shearing and compaction would dissipate rapidly after driving. Figure 7 illustrates the shaft resistance variations of Pile 2 as inferred from CAPWAP analyses carried out on dynamic monitoring records obtained at both end-of-initial-driving (EOID) and at beginning-of-restrike (BOR), one day later. Note that the mobilized shaft resistance is considerably smaller at the EOID driving and that the shaft resistance has in fact more than doubled at restrike. These inferred shaft resistance estimates indicate that, contrary to the common belief that "pile-freezing" would not occur in free draining soils, substantial "pile freezing" could occur in granular soils subsequent to installation. Figure 7 also shows the results of the inferred shaft resistance distribution from CAPWAP analyses carried out on closed-end test Pile 93C. Although the two piles are of different lengths and were driven at different locations, Becker Penetration Test results indicated similar penetration resistances of the soils in the vicinity of the two piles. Over the same depth range, however, the CAPWAP computed unit shaft resistance for the closed-toe test Pile 93C is approximately twice that of the open-toe Pile 2, presumably due to the greater soil displacement which occurred during closed-toe driving.

EFFECTS OF PILE QUAKE AND DAMPING ON GEOTECHNICAL CAPACITY

The reported CAPWAP results for all the twenty (20) piles tested excluding the nine damaged piles are summarized in Table 1. The results exhibit a wide range of computed quake and damping values despite the piles being founded within the dense silty sand to sandy silt strata in almost all cases. As illustrated below, variation in these parameters can affect both the predicted pile capacity and driveability of the piles.

Toe and Shaft Quakes

An evaluation of the pile driveability using the wave equation analysis technique requires estimates of toe and shaft "quakes". The "quake" is defined herein as the displacement of the pile required to mobilize the ultimate geotechnical resistance and is schematically illustrated in Figure 8. The representative magnitude of "quake" to be used in the analysis of pile driveability is dependent on the soil type and pile toe conditions (i.e. open-toe or closed-toe). The computed pile capacity, particularly for higher capacities, is known to be sensitive to the "quake" value used as input in the wave equation analysis.

Figure 9a illustrates the variation in the toe capacity of the 914 mm diameter piles as a function of the toe-quake inferred from the CAPWAP analyses carried out on a total of 20 piles. The results indicate a narrow range of toe-quakes generally varying from about 5 mm to about 12 mm. High toe-quakes in the order of 22 mm as well as a low toe-quake in the order of about 3 mm have also been computed in three of the piles that were subjected to dynamic monitoring.

The sensitivity of the magnitude of the toe-quake on the computed (WEAP) compression capacity of a 16 m long 914 mm diameter concrete pile is shown in Figure 9b. The shaft resistance component of the ultimate pile capacity has been assumed to be 20 percent. For analysis purposes, the toe-quake has been varied from the generally assumed 7.5 mm ($d_o/120$) to 15 mm ($d_o/60$). As can be seen, although the relative variations in the pile capacity are small in the lower capacity range (i.e. up to 2000 kN), there is a considerable difference in the final set versus capacity relationship at the high pile capacity range, depending on the actual toe-quake considered in the analysis. For example, at 20 blows/25 mm, the quake assumption gives a pile capacity in a relatively wide range of 6500 to 8500 kN. It follows that the pile driveability is adversely affected by higher quakes in that higher blow counts are required to advance the pile.

The variation in shaft-quakes computed from the CAPWAP analyses (Table 1) ranged from about 1 to 9 mm, with the higher shaft-quakes generally corresponding to closed-toe driving. For the production piles (open-toe), the shaft quakes ranged from about 1 to 5 mm. Where CAPWAP results are not available, shaft-quakes of approximately 2.5 mm are commonly used in wave equation analyses. The relative effect of different shaft-quakes on the computed (WEAP) pile capacity is not as significant as the effect of toe-quake, as shown on Figure 10. Doubling the shaft-quake from 2.5 to 5 mm in this case resulted in only a relatively small decrease in the computed pile capacity.

In published literature, a common feature of sites identified as exhibiting large soil quakes (Thompson, 1980) include high driving resistances, very dense saturated silt deposits, and relatively large displacement piles such as closed-toe pipe and precast concrete piles. All these characteristics were common to pile driving at the Ballantyne Pier. It has been suggested that high toe-quake values result from softening associated with high pore pressures developed from repetitive cyclic loading from repeated hammer blows. Large toe-quakes have also been observed during driving of low displacement H-piles where the development of a soil plug was inferred from the dynamic records (Hannigan, 1984). Thompson (1980) indicates that in most cases, high toe-quakes decrease significantly after pore pressures are allowed to dissipate. Toe-quakes determined on two piles at the Ballantyne Pier site at end-of-initial-driving and at restrike indicate only a small decrease; 9.5 mm to 9.0 mm for Pile 2 and from 12.4 to 11.6 for Pile 93C.

Toe and Shaft Damping

In addition to the effect of toe and shaft-quakes, pile capacity and driveability are also influenced by the velocity-dependent resistance (viscous damping resistance). In wave equation analyses, this dynamic resistance (R_d) is often computed using a Smith damping factor (j) where the dynamic resistance along the shaft or at the toe is assumed to be directly proportional to the pile velocity (v) and static pile resistance (R_s):

[1] $$R_d = j * v * R_s$$

The Smith toe-damping factors computed from CAPWAP analyses carried out on 19 piles were also given in Table 1. As shown on Figure 11a, the computed Smith toe-damping varies widely from about 0.1 to 1.0 sec/m, with most of the values falling between 0.2 and 0.8 sec/m. In WEAP analyses, a Smith toe-damping value of about 0.5 sec/m is commonly considered appropriate when CAPWAP results are not available. As shown on Figure 11b, the effect of 0.2 to 0.8 sec/m toe damping range on the computed pile capacity is significant. For example, over the range of toe-damping analyzed, for a blow count of 20 blows/25 mm the pile capacity varies from 7800 to 9600 kN/pile.

The shaft-damping values determined from CAPWAP analyses (Table 1) ranged from 0.2 to 1.9 sec/m. Shaft-damping values of about 0.2 and 0.7 sec/m are often used in WEAP analyses for non-cohesive and cohesive soils, respectively. Values in excess of 0.9 sec/m are considered very high (Thompson and Goble, 1988). WEAP analyses carried out using damping values of about 0.2 and 1.0 sec/m show that high shaft-damping values reduce the pile capacity at high penetration resistances (Figure 12), but not excessively.

Published literature indicate that at other sites, high damping has been found to be associated with high toe-quakes and high shaft resistance (Thompson and Goble, 1988), and has been attributed to the properties of saturated soil.

CONCLUSIONS

154 large diameter (914 mm) prestressed concrete piles were installed to provide foundation support for the new warehouse and deck of Ballantyne Pier. The piles were successfully driven through a thick granular fill embankment to underlying dense silty sand to sandy silt stratum.

The soil conditions at the test hole locations indicated relatively consistent properties across the site. However, the pile driveability and mobilized compression capacity varied considerably even though the piles were founded within the same dense stratum. The dynamic monitoring of piles to assess driveability, pile damage and mobilized pile capacity was useful. It is inferred herein that the variations in the magnitude of quake and damping contributed to the wide range in computed geotechnical pile capacity at the Ballantyne Pier site. The dynamically generated pore pressure response of the bearing stratum may have been the cause of widely varying quakes and damping inferred to have occurred during pile driving.

The quake and damping values reported from CAPWAP analyses were variable, but generally within the range reported in the literature. Variations in the toe-quake and toe-damping had a considerable impact on the predicted geotechnical capacity of the piles, particularly for ultimate capacities larger than about 2000 kN. Variations in the shaft-quake and shaft-damping, on the other hand, were much less significant in predicting pile capacity.

ACKNOWLEDGMENTS

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TABLE 1
SUMMARY OF CAPWAP RESULTS

	Pile Length (m)	Setup Time	Emax** (kJ)	Blow Count	Quake (mm)		Smith Damping (s/m)		Case Damping		Pile Capacity (kN)		
					Toe	Shaft	Toe	Shaft	Toe	Shaft	Toe	Shaft	Ultimate
TEST PILES													
Open Toe													
145B BOR*	27	1 d	90.7	20/25mm	3.40	1.10	.51	.52	1.11	.90	6695	5295	11990
120 D EOID*	37	-	83.7	8/25 mm	6.40	1.10	.37	.98	.36	1.60	3065	5110	8175
Closed Toe													
A1 EOR2*	19	2 d	93.6	18/25 mm	11.00	8.30	.12	.51	.14	.56	3630	3330	6960
93 C EOID*	19	-	68.5	17/25 mm	12.40	5.40	.15	.38	.16	.23	3480	1920	5400
93 C BOR*	19	1 hr	71.5	24/25 mm	11.60	9.00	.25	.29	.29	.20	3655	2185	5840
PRODUCTION PILES													
Open Toe													
1 BOR	40	1.5 d	95.9	5/0 mm	4.86	2.50	.27	.94	.25	1.47	2894	4919	7813
2 EOID	40	-	118.7	25/25 mm	9.46	2.50	.22	.90	.30	.61	4256	2122	6378
2 BOR	40	1 d	138.5	5/0 mm	9.00	4.58	.26	.61	.22	.86	2794	4689	7483
141 BOR	28	1.5 d	136.5	5/0 mm	9.20	3.56	.09	1.88	.13	1.50	4505	2503	7008
142 BOR	28	2 d	145.5	68/25 mm	8.40	3.56	.34	.67	.41	.83	4401	4499	8900
8 BOR	37	2 d	108.5	31/25 mm	4.85	2.50	.31	.76	.34	1.02	3690	4523	8213
51 BOR	36	2 hr	156.8	16/25 mm	5.34	4.83	.68	1.01	.78	1.36	3901	4553	8454
27 BOR	22	1.5 mth	82.5	12/25 mm	8.10	5.00	.55	1.10	.55	1.23	3304	3704	7008
29 BOR	24	2 d	121.0	7/25 mm	11.30	1.50	.40	.97	.36	1.09	3007	3709	6716
85 EOID	23	-	91.4	31/25 mm	21.50	2.00	.40	.23	.56	.09	4628	1294	5922
86 BOR	23	2 d	113.8	8/25 mm	15.50	8.00	.50	1.05	.46	.70	3000	2200	5200
26 BOR	22	7 d	104.5	150/25 mm	7.20	4.40	.96	1.30	.87	1.58	3011	4014	7025
31 BOR	23	1.7 mth	111.2	125/25 mm	11.30	2.70	.77	1.30	.60	1.39	2450	3350	5800
34 BOR	20	20 d	117.4	150/25 mm	7.90	2.50	.22	.82	.34	.82	5131	3323	8454
77 EOID	16	-	86.8	150/25 mm	7.30	4.60	.45	.50	.90	.25	6934	1741	8675
84 EOID	23	-	95.4	37/25 mm	12.00	4.80	.65	1.10	.71	.85	3500	2500	6000

BOR = Beginning of Restrike
EOID = End of Initial Driving
EOR = End of Restrike
* Conmaco 5300 air/steam hammer. All others HPSI 3505 hydraulic hammer
** Emax is maximum transferred energy
Data from Anna Geodynamics (1992) and Trow Consulting Engineers (1993/1994) reports

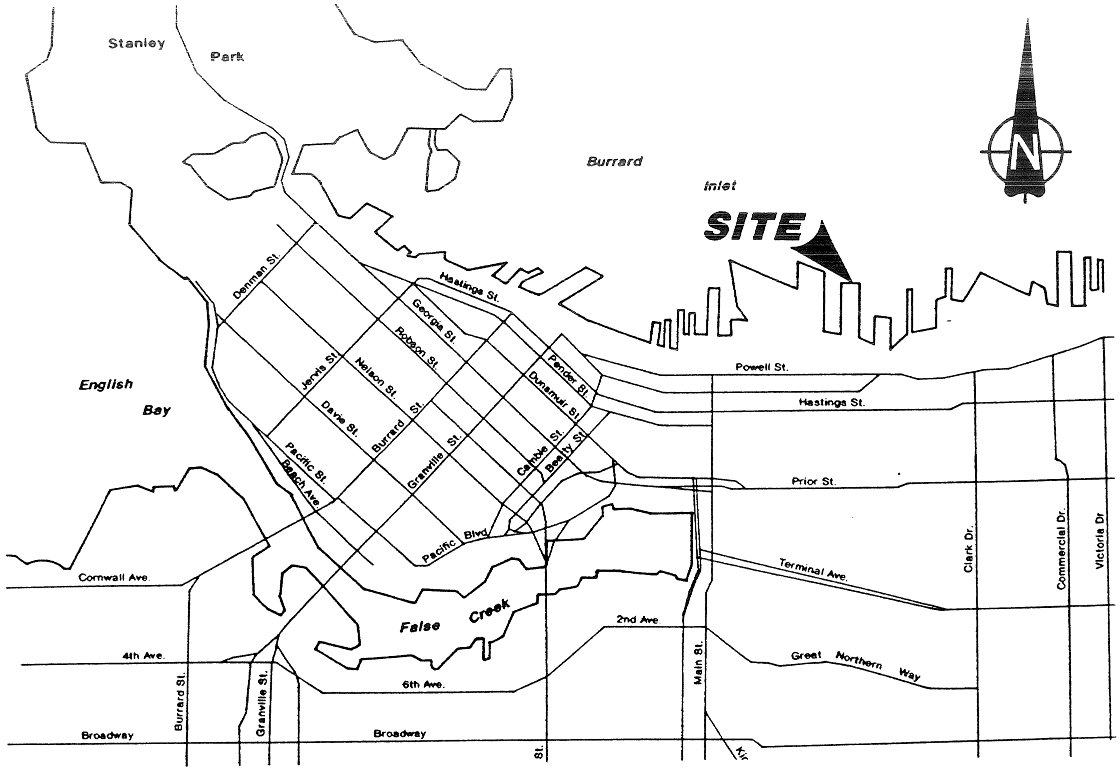

Figure 1a Site Location Plan

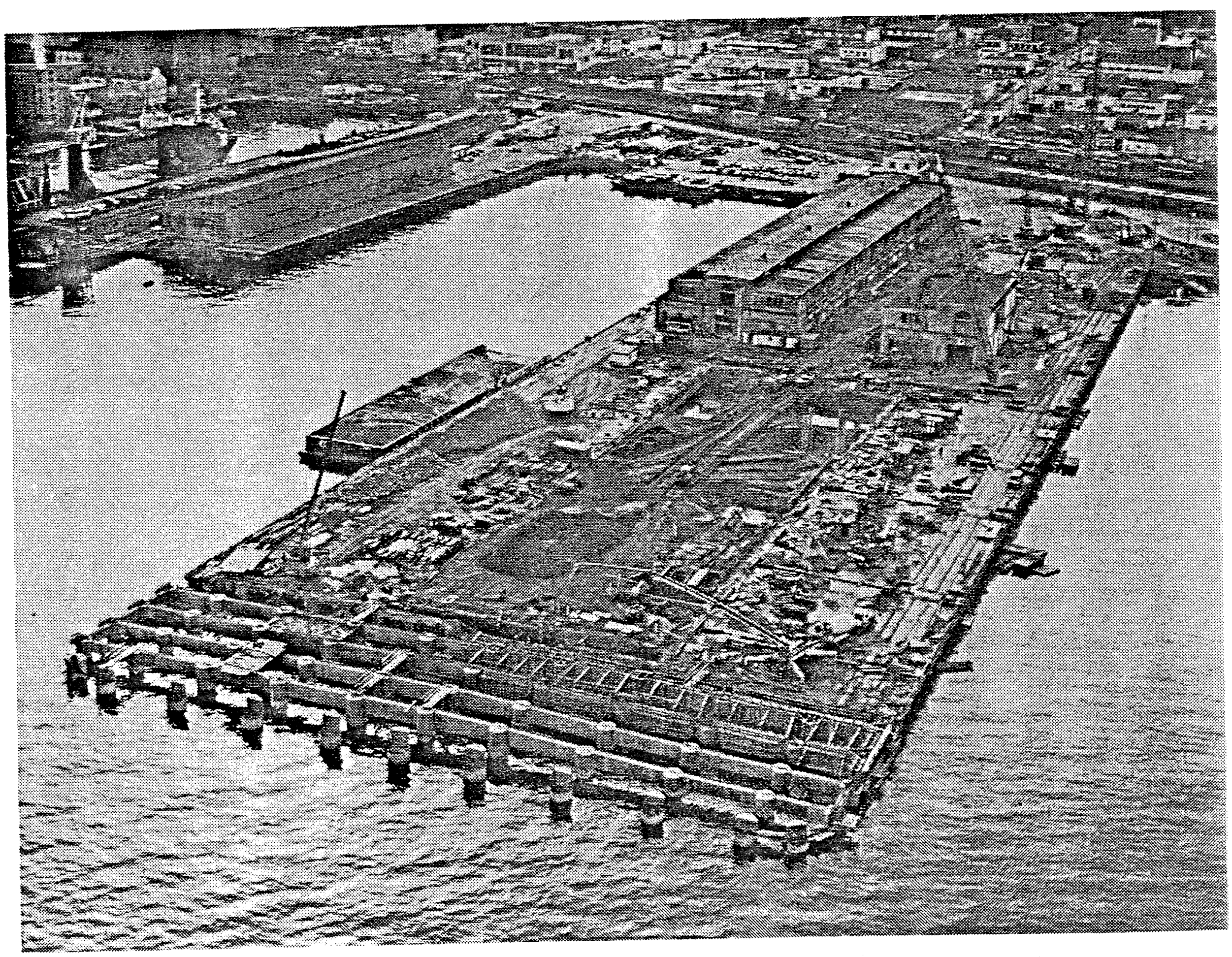

Figure 1b Photograph Showing Ballantyne Pier During Reconstruction (courtesty Peter Kiewit Sons Co. Ltd.)

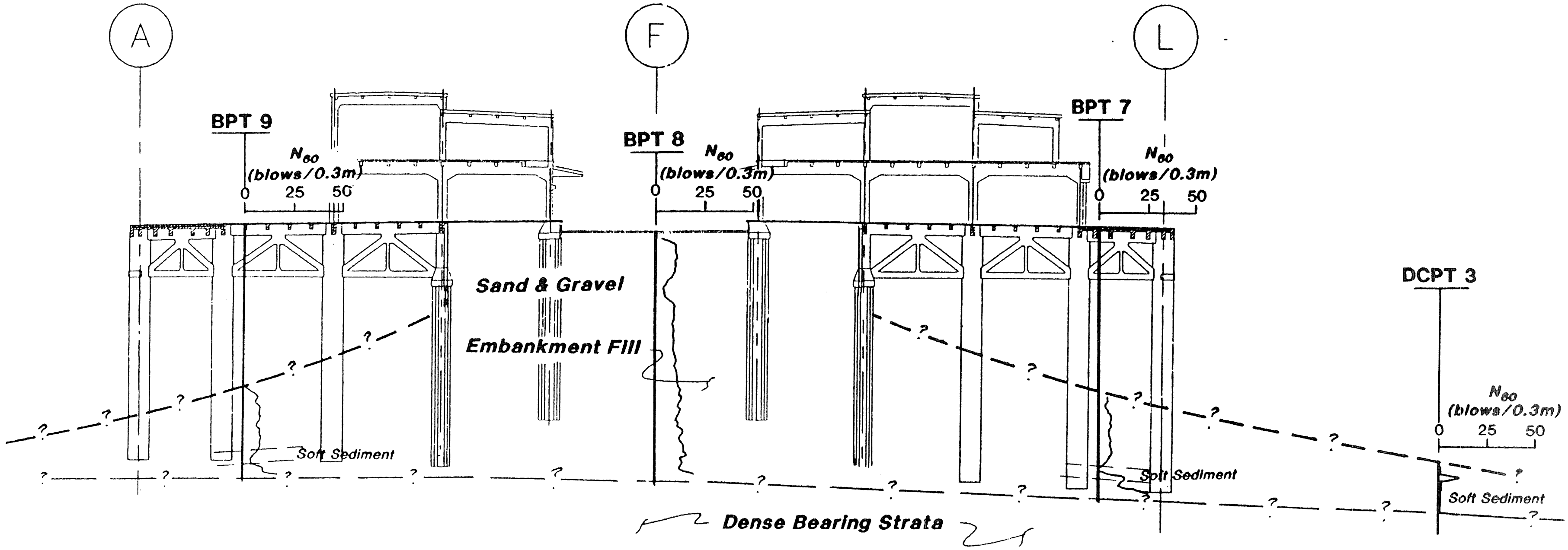

Figure 2 Typical Cross Section Through Pier

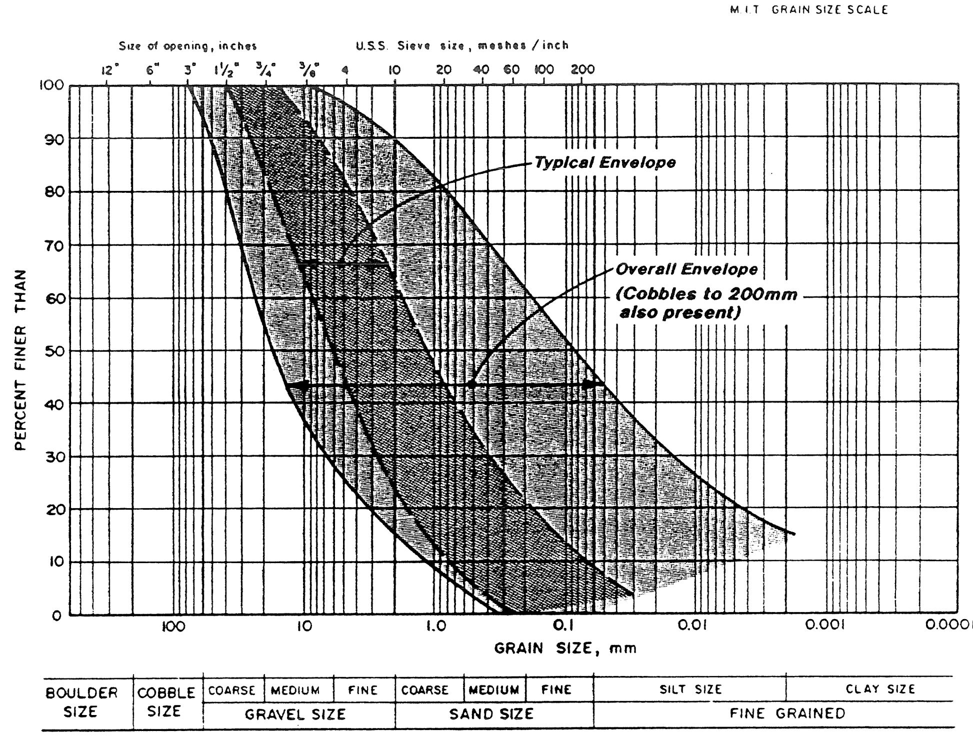

Figure 3a Gradation of Embankment Fill

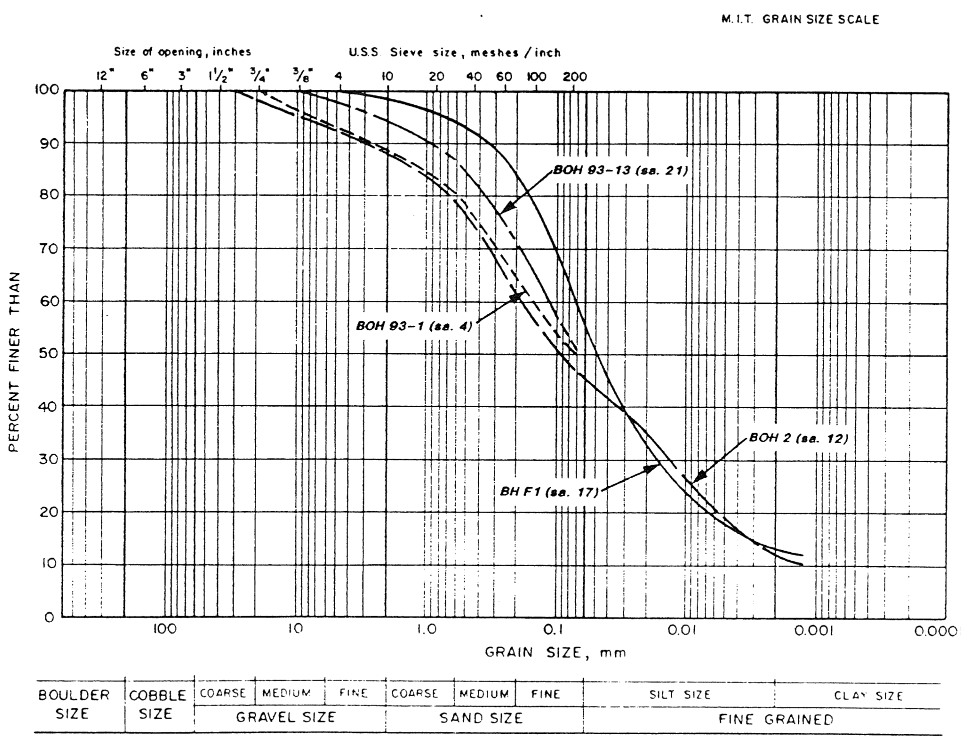

Figure 3b Typical Gradation of Dense Bearing Stratum

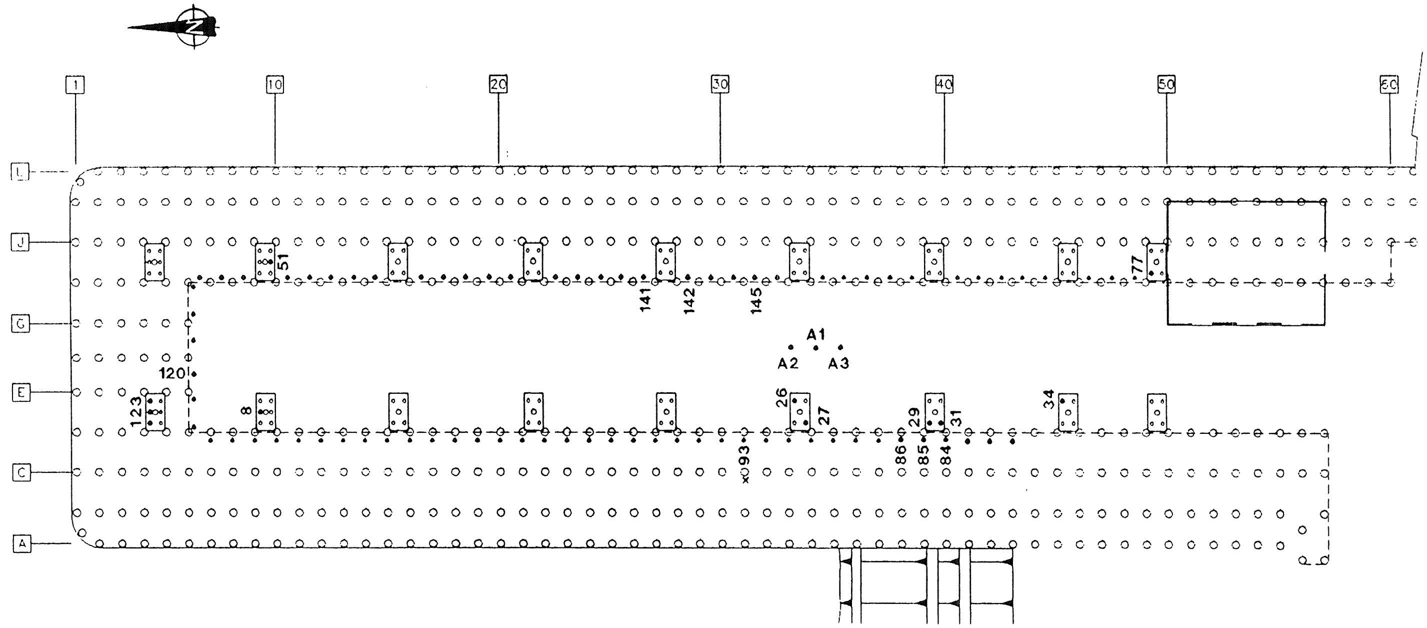

Figure 4a Plan of Pier Showing New Piles and Existing Caissons

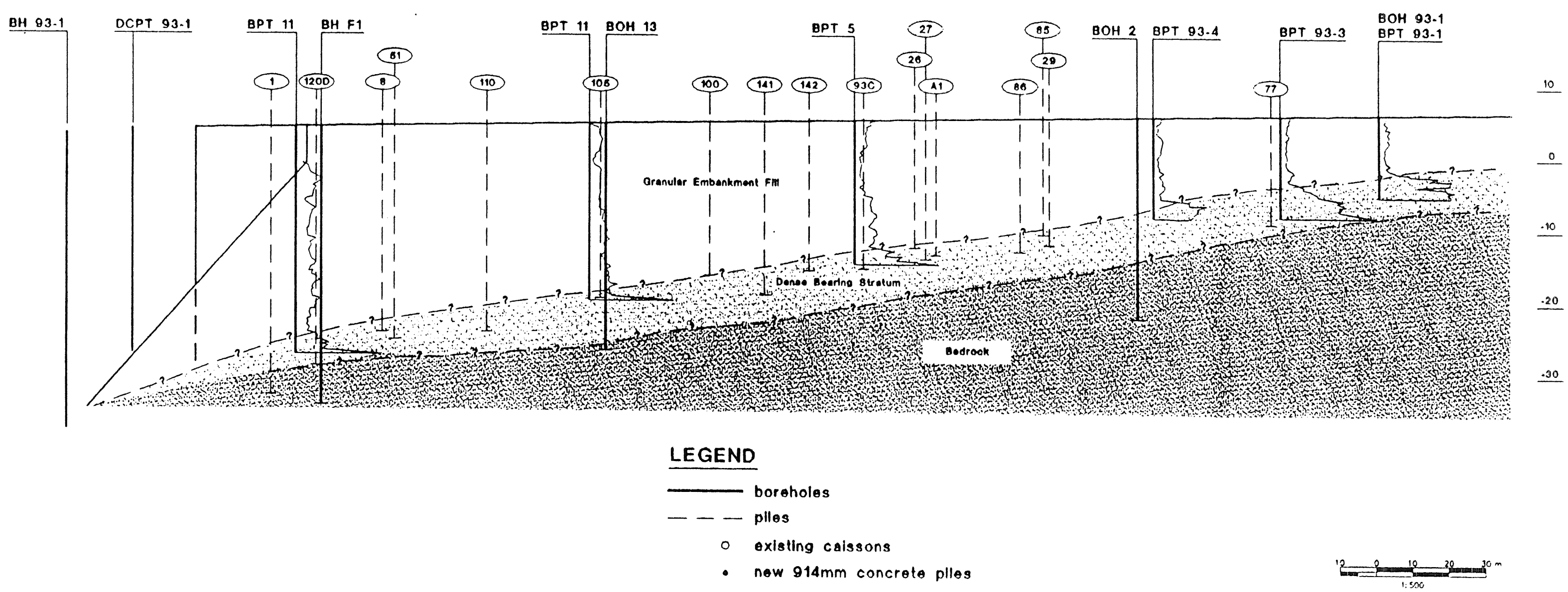

Figure 4b Longitudinal Profile Showing Selected Testhole and Pile Locations

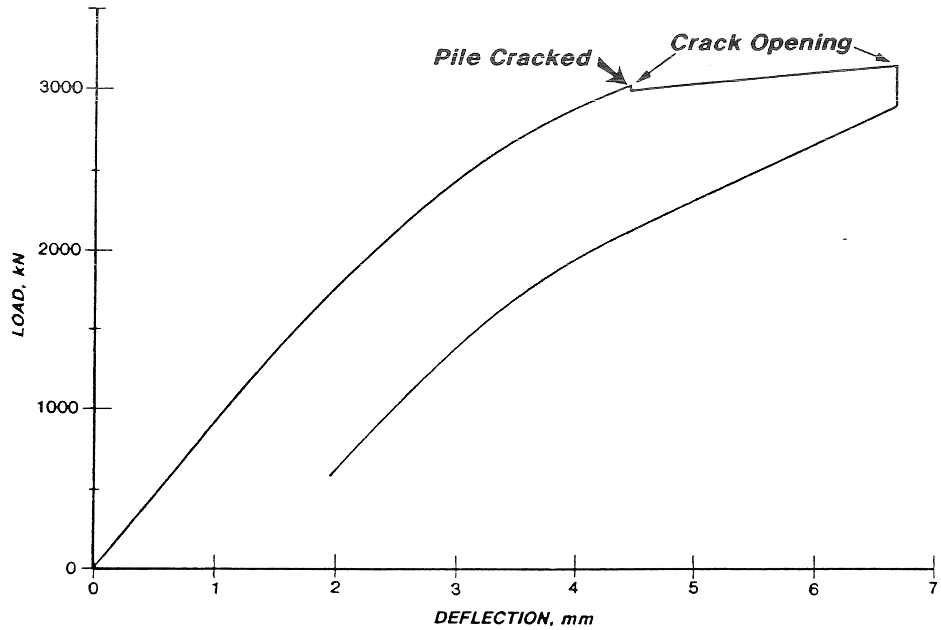

Figure 5 Static Uplift Load Test

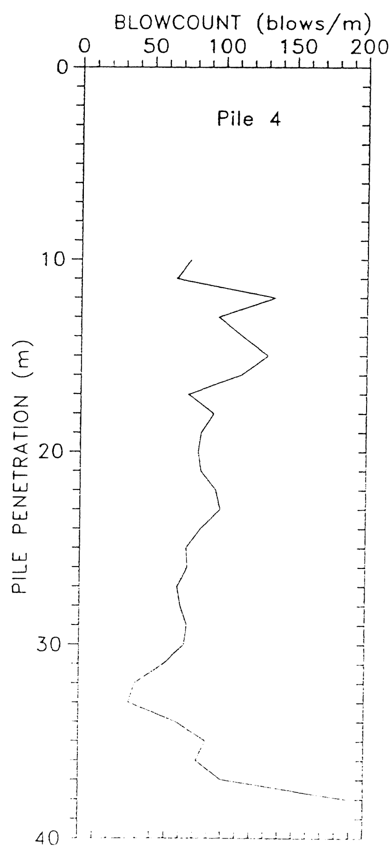

Figure 6 Typical Pile Driving Record

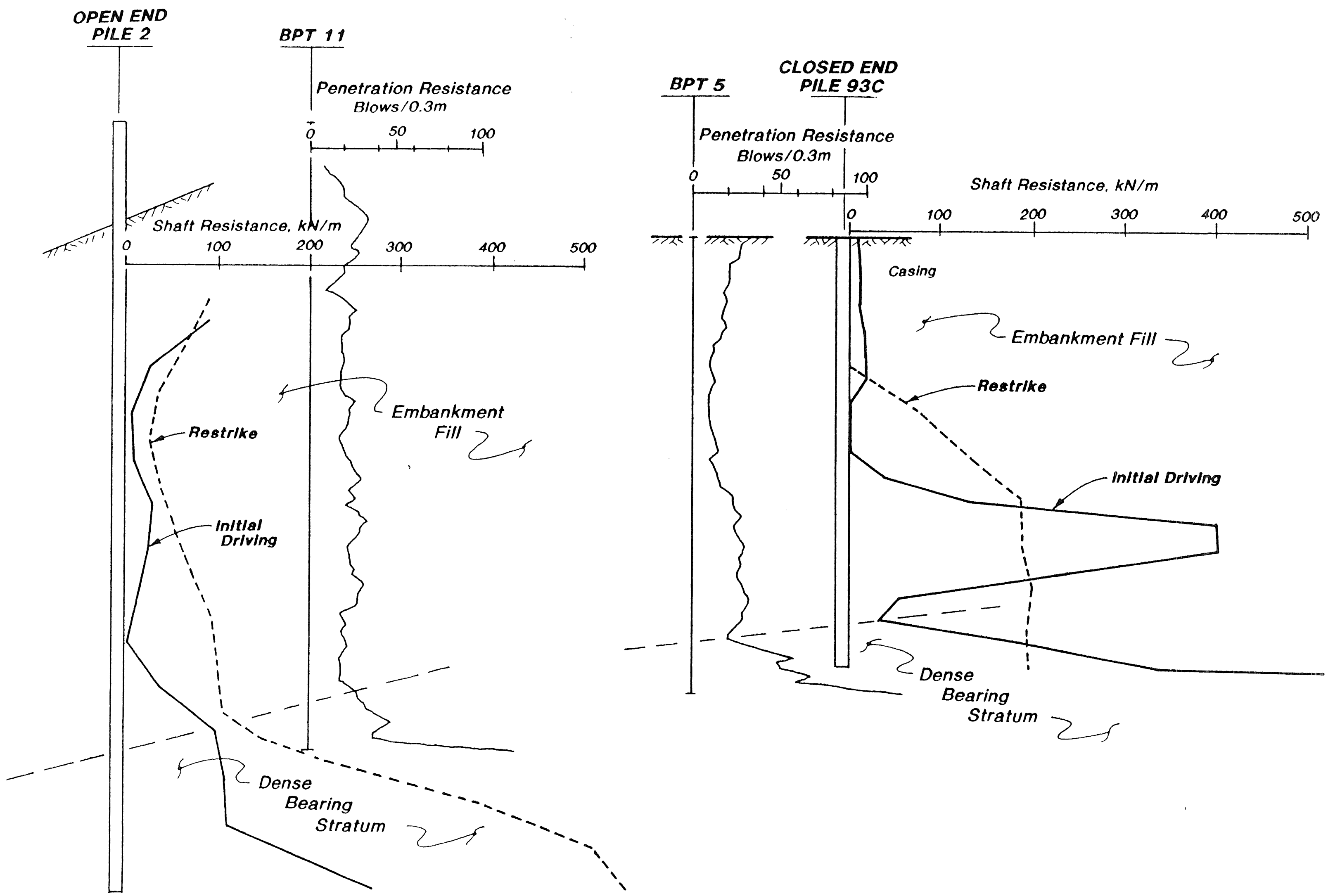

Figure 7 Shaft Friction Set-up For Closed and Open Toe Piles

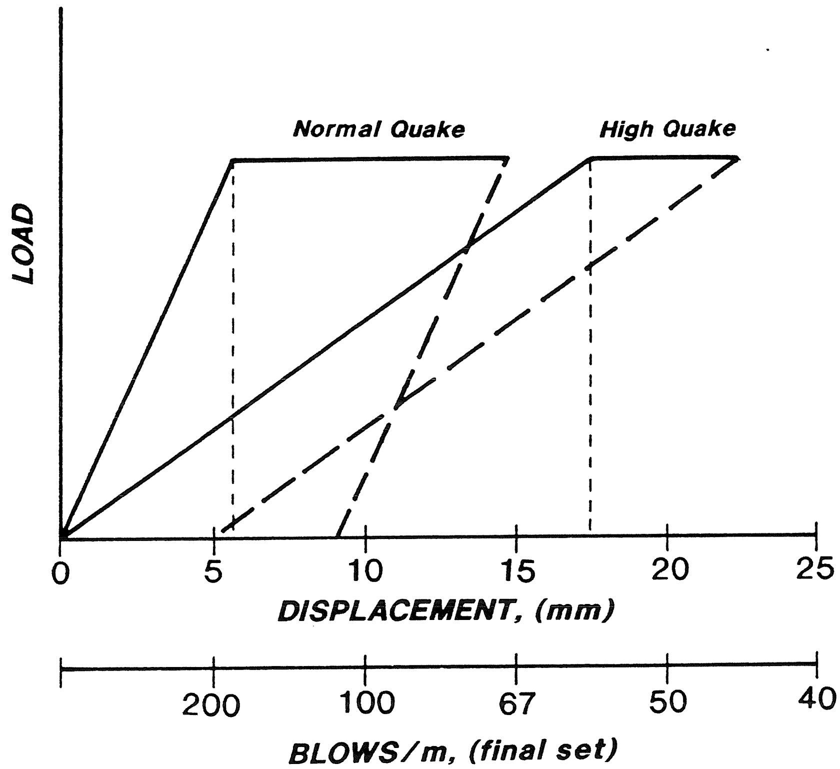

Figure 8 Schematic Illustration of Quake (adapted from Likens, 1983)

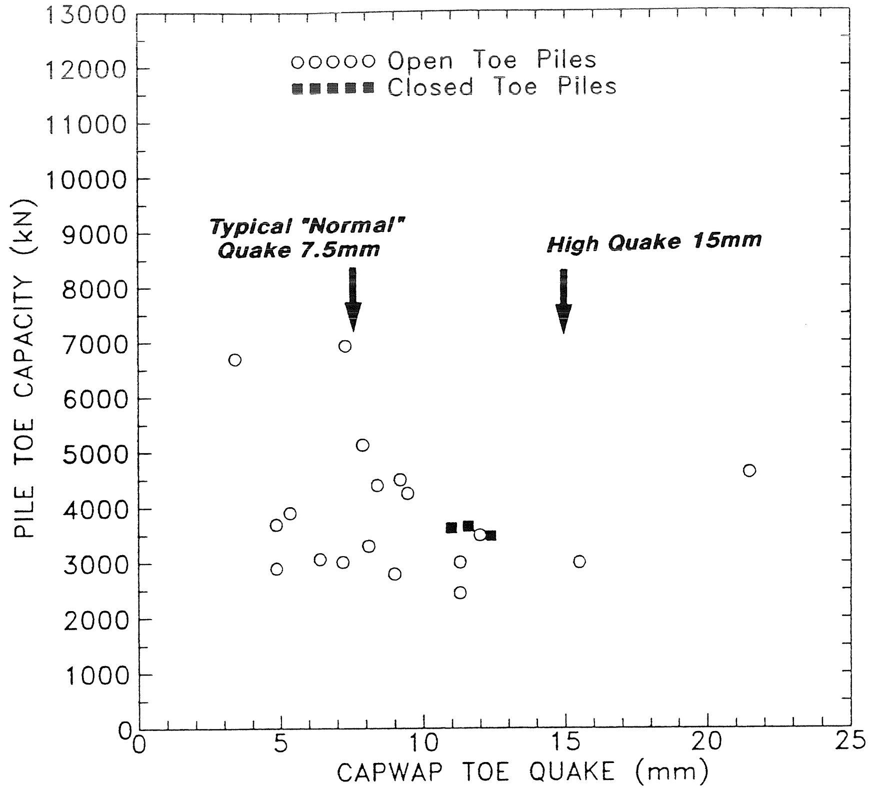

Figure 9a Computed Range in CAPWAP Toe Quakes and Pile Toe Capacity

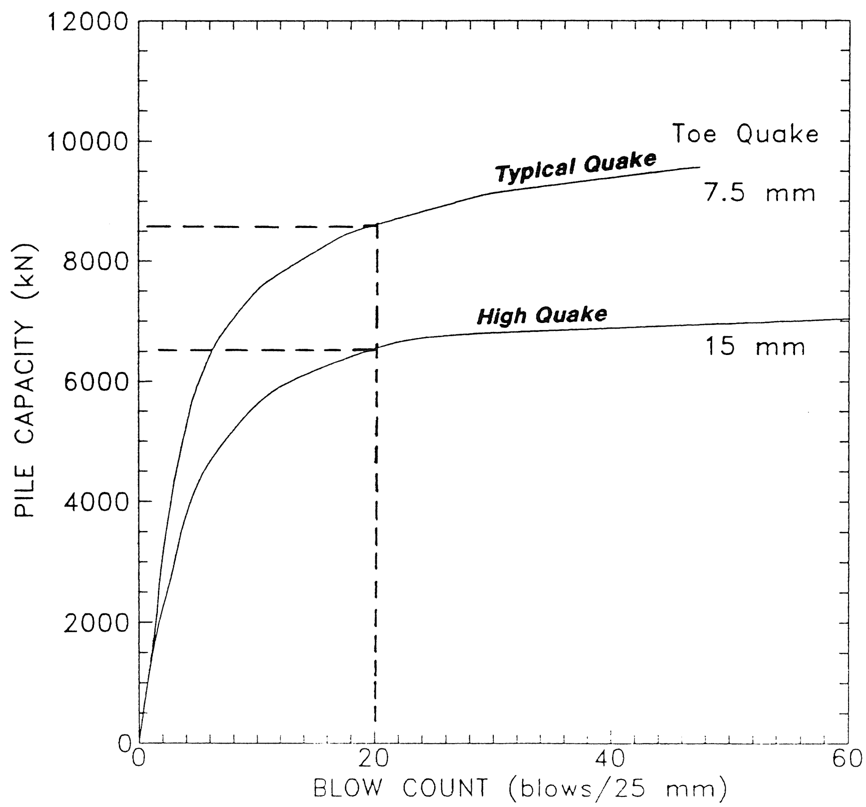

Figure 9b Effect of Typical and High Toe Quakes on Computed Pile Capacity

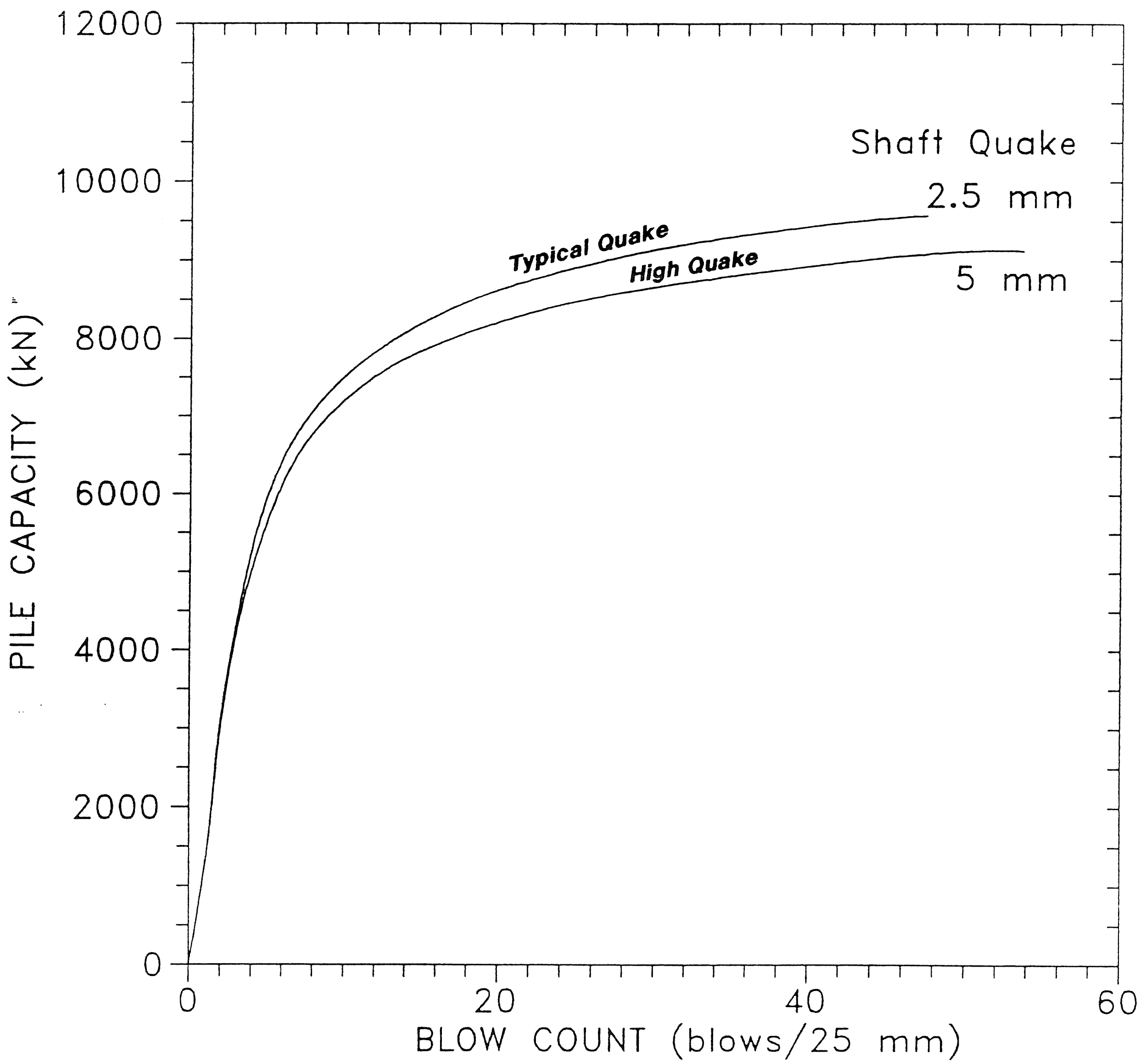

Figure 10 Effect of Normal and High Shaft Quakes on Computed Pile Capacity

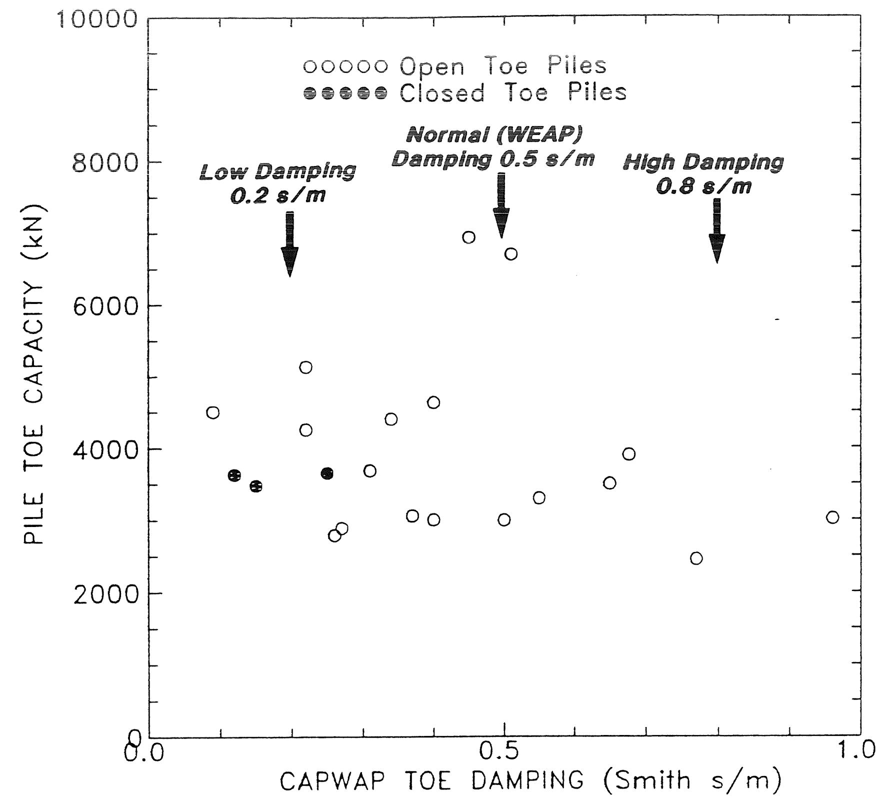

Figure 11a Computed CAPWAP Range in Toe Damping

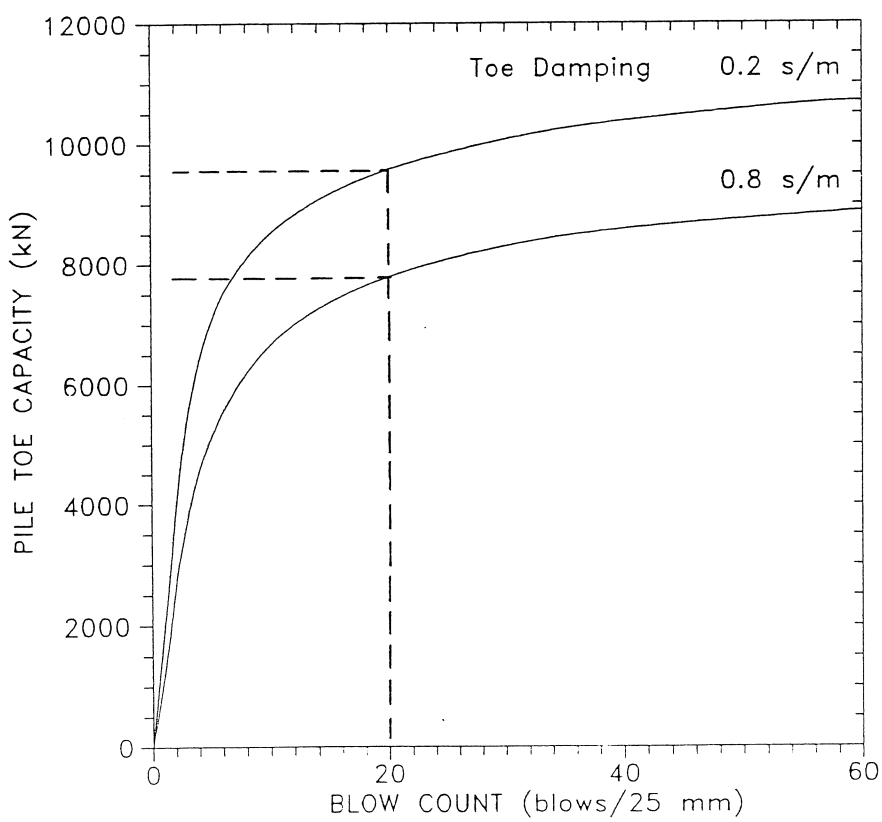

Figure 11b Effect of Toe Damping on Pile Capacity

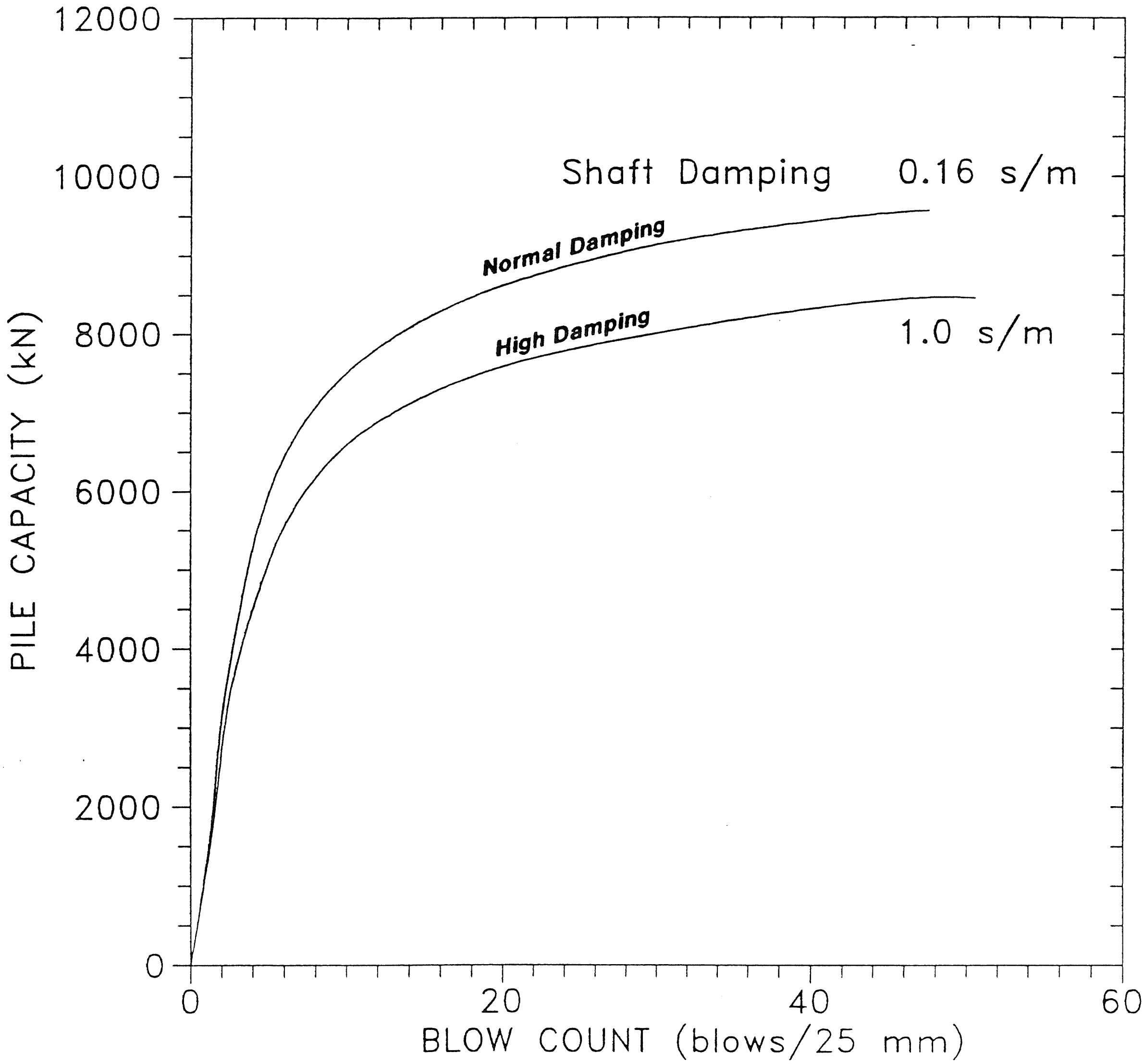

Figure 12 Effect of Normal and High Shaft Damping On Computed Pile Capacity