HIGH CASE DAMPING CONSTANTS IN SAND

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1 INTRODUCTION

The use of Case Method for capacity determination in dynamic monitoring is well established today. Hundreds of jobs have been successfully tested and good agreement is usually achieved with static test results. As experience has been obtained with the use of dynamic monitoring for capacity determination the method has been refined and improved. These improvements usually occurred when problems arose. As explanations were found for the problems it was possible to develop solutions. The purpose of this paper is to present an apparent difficulty that will be called *high damping constant sands*. This problem has appeared infrequently but when it occurred it has caused a great deal of difficulty. Several examples of sites with high damping constant sands will be presented and possible explanations of the problem will be given. A means of protecting against this unexpected condition will be given.

The Case Method is based on the calculation of the total dynamic resistance to penetration using an expression obtained by Rausche, 1970. The total resistance is

$$R_T(t) = 1/2[F(t) + F(t + 2L/c)] + Mc/2L[v(t) - v(t + 2L/c)] \tag{1}$$

where F is the measured force in the pile above the ground surface, v is the particle velocity at the location where the force is measured, L is the pile length, c is the velocity of wave propagation, and M is the mass of the pile. The quantities F and v are given as functions of time.

The total resistance is obtained based on assumptions that are quite general. The pile is assumed to be of uniform cross section, of a material that is linear elastic, and wave propagation is assumed to be one dimensional. The result is given as a function of time.

Case Method uses a simple approach to select the time at which a value of the total resistance to penetration is selected for calculation of the static capacity. This quantity is further reduced by an amount representing the loading rate effect. The dynamic part of the resistance is assumed to be concentrated at the pile toe and its magnitude is assumed to be proportional to the velocity of toe penetration. Using concepts from one dimensional

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wave mechanics the toe velocity can be determined from pile head measurements. If the velocity is multiplied by the pile impedance all quantities are obtained in terms of force, and the damping constant becomes non-dimensional. The assumed rate dependent portion of the resistance to penetration is

$$R_D = j_c(2F_t - R_T) \tag{2}$$

where F_t is the measured pile head force. If this quantity is then subtracted from the total resistance to penetration the predicted static capacity is obtained.

$$R_S = R_T - j_c(2F_t - R_T) \tag{3}$$

In order to determine the damping constant an extensive empirical study was done on the available data where static loading tests had been made on dynamically tested piles. This study was reported by Goble et al, 1975. Thus, the constant, j_c, has a completely empirical basis founded on over 70 sets of test data.

In the derivation of the expression for the dynamic portion of the resistance the assumption was made that all of the dynamic resistance was concentrated at the pile toe. The constant,j_c, was assumed to be a soil property. Thus, two important assumptions have been made, that the dynamic soil resistance is at the pile toe, and that it is purely a soil property (inferring that it is unrelated to the pile type).

A previous problem, much more common than high damping constants in sands, was discovered several years ago. This condition has become known as the *large quake condition*. It was discussed by Likins, 1983 and means of dealing with the problem were presented. It has become possible to predict some, but not all, of the occurrences of large quakes and to include them in wave equation analyses. There is clearly a relationship between quake and toe diameter in sands. Unfortunately, large quakes are not limited to large diameter piles in sands and these other cases are difficult to identify prior to beginning pile driving.

Recently, unusually high Case Damping Constants have been observed at several sites in North America. In what follows these data will be summarized and reviewed. The results will be discussed with the goal of developing the capability to be able, at some future time, to predict the condition prior to going to the field.

2 PROCEDURE

The test results from 25 piles at nine projects, where high Case Damping has been observed, have been examined, and are presented in Tables 1 and 2. The projects were distributed coast to coast across Canada and covered the eastern seaboard of the United States. All of the piles were founded in granular soils. All the piles have been dynamically monitored and have had CAPWAP analyses performed at the beginning of redriving after at least an overnight delay. Five piles at four projects were statically load tested according to ASTM Standard D1143.

Table 1: Site, Soil, Hammer and Pile Data

Project	Location	Soil Conditions	Hammer & Capblock	Pile Description	Pile Number	Embedment (m)	Penetration Resistance (bl/25 mm:)
A	Newcastle, N.B.	Granular Fill to 2.5 - 5 m depth. 10 to 30 bl/300 mm. Organic Clayey silt to 8 - 8.5 m, 4 to 9 bl/300 mm. Silty Sand with some gravel to bottom of boreholes at 31 m depth, 20 to 30 bl/300 mm with up to 40 bl/300 mm. Artesian Conditions reported immediately below Organic Clayey Silt.	Linkbelt 520 Double Acting Diesel Rated Energy (RE) 35.7 KJ Capblock-225 mm of Oak Cushion-6 sheets of 18 mm plywood	400 mm square Prestressed Concrete	1	15.7	10
				400 mm square Prestressed Concrete	2	15.7	7
				400 mm square Prestressed Concrete	3	15.7	9
				400 mm square Prestressed Concrete	4	15.7	6
				400 mm square Prestressed Concrete	5	15.7	30
				400 mm square Prestressed Concrete	6	15.7	10
				400 mm square Prestressed Concrete	7	15.7	10
B	New Westminster, B.C.	Gravel fill from surface to 15 m. Interbedded silt, sand and gravel to -50 m. GWT at 5m.	Delmag D46 open end diesel hammer RE 143 KJ Standard Delmag Aluminum end Conbest Sandwich Capblock	610 mm O.D. by 20 mm thick Steel Pipe	1	41	13
				360 mm O.D. by 174 mm thick Steel H Pipe	2	36	5
				360 mm O.D. by 174 mm thick Steel H Pipe	3	48	25
C	New Westminster, B.C.	Water at 5 to 6 m depth. Clayey silt to between 12.3 and 16 m depth, 20 to 30 bl/300mm. Sand, fine to medium to 28 and 31 m depth, 10 to 46 bl/300mm.	Menck MH96 Hydraulic Hammer for Piles 1 & 2	610 mm Octagonal Prestressed Concrete	1	17.8	21
				610 mm Octagonal Prestressed Concrete	2	23.6	9
			Delmag D36-23 open end diesel hammer for Piles 3 to 5.	610 mm Octagonal Prestressed Concrete	3	23.5	6
		Sand, fine to 48m depth, 17 to 76 bl/300mm.		610 mm Octagonal Prestressed Concrete	4	23.5	48
				610 mm Octagonal Prestressed Concrete	5	23.5	75
D	North York, Ont.	Clayey dilt fill to 10 m depth, 4 - 9 bl/300 mm. Sand, fine to medium to 36 m depth, 11 to 22 bl/ft. GWT at 6 m.	35 KN Drop Hammer falling 1.8 m	298 mm O.D. by 8.5 mm thick Steel Pipe	1	19.2	25
				244 mm O.D. by 8.9 mm thick Pipe Steel	2	24.2	17

Table 1 Cont'd : Site, Soil, Hammer and Pile Data							
				Pile		Embedment	Penetration Resistance
Project	Location	Soil Conditions	Hammer & Capblock	Description	Number	(m)	(bl/25 mm)
E	Toronto, Ont.	Clayey silt fill to 5.5 m depth, 5 - 25 bl/300 mm Organic silt to 10 m depth, 6 bl/300 mm. Sand, fine to 11.5 m depth 25 bl/300 mm. Sand, fine to medium. 36 - 41 bl/300 mm · GWT at 1.2 m.	13.3 kN Drop Hammer falling 1.8 m	273 mm O.D. by 4.8 mm thick Steel Pipe	1	11.1	20
F	Delta, B.C.	Fill, sand to 3 m, 10 - 18 bl/300 mm. Peat, soft to 3 - 4 m, 0 bl/300 mm. Sand, fine to coarse to 37 m, 50 - 87 bl/ 300 mm. GWT at 1.2 m.	Delmag D-36-13 at Setting 2.	610 mm O.D. by 12.5 mm thick Steel Pipe	1	22	6
				610 mm O.D. by 12.5 mm thick Steel Pipe	2	23	5
G	Northeastern U.S.	Fine Sand, > 100 bl/300 mm.	Delmag D36-32	500 x 500 mm PS Concrete	1	45	29
		Fine Sand, 50 bl/300 mm.	Delmag D36-32	500 x 500 mm PS Concrete	2	22.5	3
H	Southeastern U.S.	Fine Sand, 80 bl/300 mm.	Delmag D46-23	500 x 500 mm PS Concrete	1	33.3	11
J	Delta, B.C.	Fill and organic silt to 2.4 m. Clayey silt, firm to very stiff to 8 m. Sand, gravel and cobbles, very dense to 31 m.	Kobe K25 open end diesel, RE 67.8 KJ	335 mm O.D. by 150 mm thick Steel H Pipe	1	17.8	14
			Delmag D30-13 RE at Setting 3 74.4 KJ. Standard Delmag Aluminum and Conbest Sandwich Capblock	335 mm O.D. by 150 mm thick Steel H Pipe	2	17.9	17

Table 2: Case Damping Constant Calculated to match Ultimate Capacity predicted by CAPWAP Analysis

Project	Pile No.	Ultimate Capacity (KN) Static Load Test	Ultimate Capacity (KN) Capwap Analysis	CAPWAP Results: Resistance Skin	Resistance Toe	Case Damping Skin	Case Damping Toe	Smith Damping (1/m/s) Skin	Smith Damping (1/m/s) Toe	Quake (mm) Skin	Quake (mm) Toe	Pile Driving Analyser Results: Impact Force (KN)	Total Dynamic Resistance (KN)	Case Damping Constant
A	1		1350	1100	250	0.60	0.22	0.09	0.14	2.5	1.0	2520	2460	0.43
	2		1000	930	70	0.33	0.10	0.06	0.27	1.3	1.0	2380	2100	0.42
	3		540	360	180	0.61	0.02	0.27	0.02	2.7	5.6	1600	1340	0.43
	4		1040	835	205	0.25	0.19	0.05	0.17	2.2	4.8	2600	2320	0.44
	5	1420	1390	850	540	0.71	0.26	0.13	0.08	3.3	3.6	2860	2680	0.42
	6		1260	1095	165	0.60	0.10	0.09	0.10	2.5	3.0	2230	2450	0.59
	7		470	450	20	0.66	0.04	0.23	0.30	1.3	1.8	1210	1200	0.60
B	1		5470	4070	1400	0.90	0.70	0.31	0.76	3.8	4.3	6300	8550	0.76
	2		3770	3500	270	0.85	0.20	0.21	1.20	3.5	6.0	4920	6350	0.74
	3		4510	4010	500	1.40	0.25	0.31	0.46	4.0	6.0	4760	6810	0.85
C	1		1780	1080	700	0.49	0.26	0.90	0.77	1.6	3.5	2080	2270	0.26
	2	1760	1760	1380	380	0.20	0.22	0.29	1.23	2.7	8.5	3000	2590	0.24
	3		1660	1230	430	0.51	0.12	0.83	0.66	3.0	3.0	6050	4970	0.46
	4		1730	1280	450	0.75	0.26	1.18	1.17	3.5	3.5	3310	3740	0.70
	5		2020	1060	960	0.55	0.25	1.03	0.55	4.0	5.0	3570	3340	0.35
D	1		1830	400	1430	0.37	0.18	0.29	0.04	2.5	9.2	1470	2160	0.42
	2		1460	450	1010	0.55	0.37	0.33	0.10	4.0	6.5	1210	1740	0.42
E	1		550	180	370	0.50	0.57	0.49	0.27	2.5	5.7	470	750	0.58
F	1		2360	2165	195	1.03	0.15	0.46	0.77	2.8	3.0	2900	4150	0.67
	2		2550	2345	205	1.00	0.17	0.41	0.80	2.0	4.0	4420	4670	0.66
G	1	2180	1920	1670	250	1.06	0.30	0.13	0.26	3.3	3.3	2990	3440	0.60
	2	800	930	840	90	0.66	0.09	0.17	0.20	2.0	2.0	3110	2850	0.57
H	1		2750	1090	1660	0.20	0.43	0.14	0.19	6.3	8.9	6560	5970	0.45
J	1	2580	2670	2470	200	1.20	0.15	0.38	0.59	2.0	1.5	3610	4280	0.54
	2		4450	3380	1070	0.48	0.20	0.10	0.14	1.0	2.4	4140	5600	0.43

3 TEST RESULTS

A large variety of test results have been observed at the nine projects which have been examined. The only similarities about the projects are that driven piles were founded in granular soils, and that most of the Case Damping Constants required to duplicate CAPWAP wave equation bearing capacities were high.

A wide variety of pile driving hammers were used for the projects. These ranged from drop hammers, through both single and double acting diesel hammers to hydraulic hammers. The capblock and cushion materials also varied. At some projects, where information is available, hardwood capblocks were used with plywood cushions. At other sites, aluminum and plastic sandwich capblocks were employed.

The tests were all undertaken on concrete or steel piles. The concrete piles ranged from 400 mm square precast to 610 mm octagonal prestressed piles. Pipe piles reanged from 244 mm diameter by 8.9 mm wall thickness to 610 mm diameter by 20.0 mm wall thickness. Steel H-piles ranged from 335 x 150 to 360 x 174 piles.

While the founding soils were all granular, there was a wide range of grain size distribution and relative density. On some sites, the founding soils were silty sand at standard penetration resistance of 20 to 30 blows per 300 mm. At others the soil was interbedded very dense silt, sand and gravel, and yet others, there was fine to coarse sand with standard penetration resistances ranging anywhere between 50 to in excess of 100 blows per 300 mm.

In general terms, the hammers were moving the piles. While there were occasional penetration resistances of more than 20 blows per 25 mm, piles were generally penetrating at 10 blows or less per 25 mm.

The significant similarity between the sites is that high Case Damping Constants were required to duplicate CAPWAP bearing capacity results. However, even these constants were variable ranging from 0.24 to 0.70 in the same soil on the same site and from 0.24 to 0.85 for the complete range of projects. There was, in fact, no clear indication that the type or density of soil was directly related to the required Case Damping Constant. Relatively high Case Damping Constants were observed at some of the sites with denser and coarser soils, such as at Sites B and G. Geologically the soils at the sites were all water borne. Some were beach deposits, some fluvial/deltaic and some were shallow water lacustrine deposits. Mineralogical analysis was available for few of the sites and consequently this aspect could not be examined.

The CAPWAP analyses have shown wide ranges of the input parameters required to achieve matches of the force and velocity traces. These are summarized in Table 3.

An examination of the CAPWAP results indicates little relationship between the required input parameters and the Case Damping Constant needed to match the CAPWAP total resistance. The Case Shaft Damping and Shaft Quake were the only parameters which showed some relationship with the Case Damping Constant and comparisons of these are plotted in Figures 1 and 2. It can be seen that the envelopes encompassing the ratio of Case Shaft Damping to the Case Damping Constant are from 0.8 to 1.8 and for Shaft Quake to Case Damping Constant are 3.1 to 11.3 mm. Ratios for individual projects,

Table 3: Ranges of input parameters for Case Method Damping		
	FROM	TO
Total Resistance (KN)	470	5470
Ratio of Shaft to Toe Resistance (6 of 9 projects indicated frictional rather than bearing support)	0.28	22.5
Case Shaft Damping	0.20	1.20
Case Toe Damping	0.02	0.90
Smith Shaft Damping (1/m/s)	0.05	0.90
Smith Toe Damping (1/m/s)	0.02	1.23
Shaft Quake (mm)	1.0	6.3
Toe Quake (mm)	1.0	9.2

as well as for all 25 piles range widely within the envelope.

4 DISCUSSION

The test results from the nine project sites indicate that high Case Damping Constants for driven piles founded in sands have occured in North America, and especially in Canada. The project sites, which have been examined, are scattered throughout Canada with a large number being along the Fraser Valley near Vancouver, B.C., some in Central Canada and one near the east coast of New Brunswick. The two American examples are on the east coast of the U.S.A. with one being in the north and the other in the south. It is therefore, clear that the high Case Damping phenomena cannot be treated as localized geographic or geological conditions, and that every project involving piles driven into sand should be checked for it.

Higher Case Damping Constants than expected will result in an overestimation of pile bearing capacity unless the dynamic monitoring is checked by means of CAPWAP analyses or load tests. There is usually considerable pressure from others on the project to provide rapid predictions of bearing capacity and values which are too high may be given to clients. These "too high" bearing capacities often "confirm" other methods of calculating bearing capacity from theoretical static analyses and/or conventional pile driving formulae. They are, therefore, readily accepted by the client. The subsequent reduction in capacity, resulting from the CAPWAP analysis, causes concern and skepticism from other parties to the project, especially if the CAPWAP predicted capacity is less than required to provide an adequate factor of safety for the piling. While static load testing, when carried out on four of the nine projects, has confirmed the CAPWAP analyses, the change in prediction can result in a crisis of credibility in relation to the dynamic test procedures. It is, therefore, important that, if there is any concern that higher than normal Case Damping Constants will be encountered, CAPWAP analysis be performed *before* giving test results to those who are unfamiliar with dynamic monitoring as a means of predicting bearing capacity.

At the examined projects the Case Damping Constants varied significantly and with no apparent relationship with any predictable condition. At site C, the Case Damping Constant ranged from 0.24 to 0.70 in the same soil. Such variations result in widely ranging bearing capacities for a given total dynamic resistance. A sufficient number of CAPWAP analyses must, therefore, be carried out to establish the effects of the variations of the Case Damping Constants. It is even necessary on some projects to undertake a CAPWAP analysis for every pile which is dynamically monitored.

5 REASONS FOR HIGH CASE DAMPING CONSTANT IN SANDS

While the High Case Damping Constant phenomenon for driven piles founded in sands appears to be unpredictable, it relates, to some extent, to high Case Shaft Damping and Shaft Quake (Figures 1 and 2) and to a lesser extent to high shaft resistance.

Theoretically this explains the phenomenon, as the Case Method tends to underestimate effects of the skin parameters on bearing capacity of piles in granular soils by assuming piles to be predominately end bearing. The authors have noted on many projects that the toe resistance is substantially less than would be calculated from theoretical static analysis. This can be the case for even very dense granular soils. However, high Case Damping Constants were also noted on three projects (D, E and H) where there was substantial toe resistance. Additionally, some projects exhibited high Case Shaft Damping and Low Shaft Quake or vice-versa (T 4113-G, H, E, D) while others indicated a combination of both. Consequently, although the major reasons for high Case Damping Constants in soils appear to be high Case Shaft Damping, Shaft Quake and Shaft Resistance, these cannot be isolated to represent the only reasons.

It can be stated that there is no relationship between the hammers, capblocks, cushions and piles used on the projects and the Case Damping Constant. This means that the phenomenon is most likely related to the properties of the soils. However, it is difficult to identify specific depositional, geological or minerological characteristics with the presence of high Case Damping Constants. Many of the sites have fluvial/deltaic soils. However, some of the sands are shallow water lacustrine or beach deposits. Some soils have a fairly high micaceous content, but others are thought to be predominately quartzitic. Unfortunately, the minerology of the sand deposits was not available, and consequently this aspect, which may be of significance, could not be examined. Simply by elimination it must be assumed that high Case Damping Constants for driven piles founded in sands are caused by the soil conditions, and it is recommended that the minerology and depositional characteristics be examined in more detail on future projects where high Case Damping Constants are observed.

6 CONCLUSIONS

It is concluded that high Case Damping Constants for driven piles founded in sands are a widely distributed and not uncommon phenomenon. It is recommended that sufficient CAPWAP analyses and static load tests be performed for such projects before presenting bearing capacity predictions. At the sites where the Case Damping Constant is variable, it may be necessary to carry out a CAPWAP analysis on every pile that is dynamically monitored. The major reasons for high Case Damping Constants in sands appear to be high Case Shaft Damping, Shaft Quake and Shaft Resistance or a combination thereof. It would seem likely that the high Constants are caused by the depositional and/or minerological characteristics of the sands, although this aspect needs to be further researched.

7 REFERENCES

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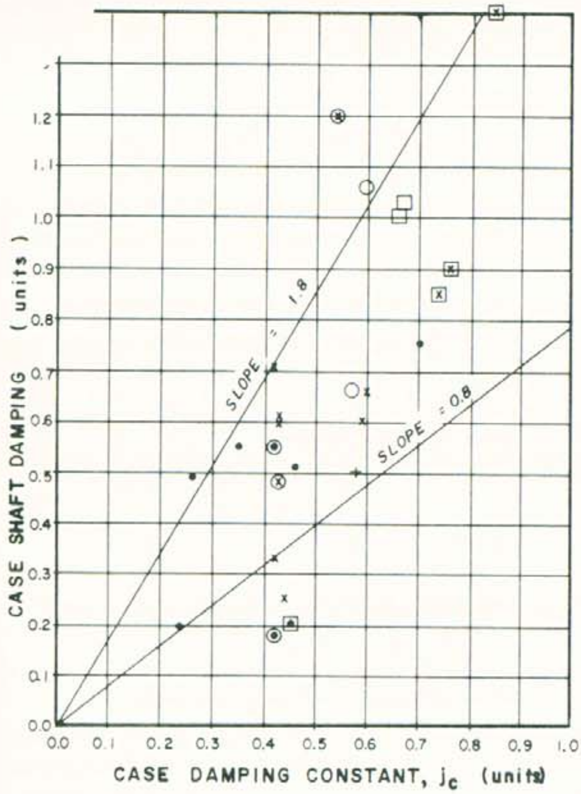

FIGURE 1 - CASE SHAFT DAMPING VERSUS CASE DAMPING CONSTANT

LEGEND

Symbol		
x	PROJECT	A
⊠		B
•		C
⊙		D
+		E
□		F
○		G
▣		H
⊗		J

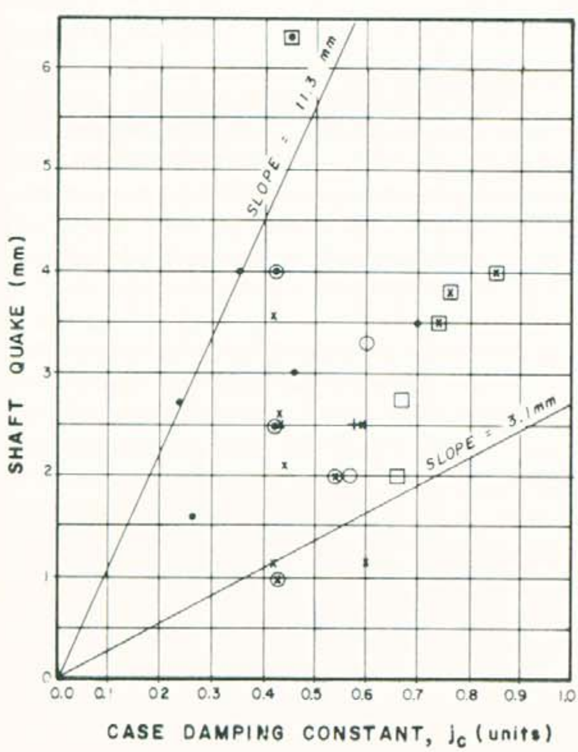

FIGURE 2 - SHAFT QUAKE VERSUS CASE DAMPING CONSTANT