276

DYNAMIC PILE MONITORING REPORT

GEIST ROAD EXTENSION

PEGER ROAD TO COLLEGE ROAD

FAIRBANKS, ALASKA

FHWA
DEMONSTRATION PROJECTS

Federal Highway Administration
Office of Highway Operations
Demonstration Projects Division
Washington, D.C.
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Introduction and Background

Field demonstrations for Demonstration Project No. 66, "Design and Construction of Driven Pile Foundations," include (1) dynamic pile monitoring by pile analyzer (field computer), and (2) static pile load testing using a mobile pile load testing frame. The equipment and technical assistance are made available to requesting State transportation departments and Direct Federal Divisions.

In April 1988, a request for demonstrating the pile analyzer on a construction project was received from the Alaska Department of Transportation. Alaska Department of Transportation (AK-DOT) had selected the pier location for the Geist Road Extension in Fairbanks as the site of the dynamic tests.

The purpose for dynamic testing was (1) to demonstrate the use of newer and more accurate techniques for determining pile load capacity during driving (2) to determine the ultimate pile load capacity for the subject bridge and (3) to use CAPWAP computer analysis to evaluate the dynamic testing data obtained from the uncommonly used pile section (30 inch diameter steel pile with a 2-inch thick steel plate located approximately thirty feet above the pile tip).

The field work was performed on May 19 and 20, 1988, by Mr. H. Clark, Civil Engineering Technician, in the Demonstration Projects Division. On May 20th results of the analysis and recommendations were discussed with Mr. Monte Weaver, Chief of Geotechnical services and several project engineers of AK-DOT. A CAPWAP computer analysis of the dynamic measurement data was conducted in early July. A detailed description of the work performed, test results, data analysis, and conclusions follow in the report.

LOCATION AND PILE DATA

The load test site is located in Fairbanks, Alaska at pier location of the Geist Road Overhead Structure. A general site plan is shown in Figure 1. Both bridge abutments are supported on spread footings founded on mechanically stabilized earth walls. A center pier is supported on a single row of 30-inch diameter pipe piles with a 2-inch thick plate located 30 feet above the pile tip. Four, 3-inch diameter, evenly spaced holes were placed in the plate to allow the escape of air during driving. The pipe pile had a 1/2-inch wall thickness and a design load of 400 kips. The pile plate detail is illustrated in Figure 2.

SUBSURFACE CONDITIONS

The log of boring No. 1 (Figure 3) shows firm to compact, silty, sandy gravel; gravelly sand; and silty sand (SPT "N" values vary 13 to 26) to a depth of 20 feet. This material is underlaid by 70 feet of a loose-to-compact, sandy gravel containing occasional 3-6 inch thick layer/lenses of fine-medium grain sand (SPT "N" values vary 8-30). The design intent was to achieve a minimum length of pile penetration, approximately 30 feet and also attain the minimum design load of 400 kips below that depth.

The AK-DOT pile driving formula was the primary method of construction control for production pile installation. The analyzer data was only used for comparative analysis.

HAMMER DATA

The following information for the hammer system was provided by the contractor:

DELMAG D-46, open end diesel hammer
Rated Energy at 10'-8" stroke = 107,600-foot pounds;
Ram Weight = 10,100 lbs.
Hammer Cushion = 5" aluminum and micarta
Vibratory Hammer = MKT V-20 maximum pull extraction 40 tons.

PILE INSTALLATION AND DYNAMIC MONITORING RESULTS

For Test Pile No. 1, the pile installation sequence was as follows: pre-drill a 36" diameter hole to a depth of 10 feet, vibrate the pile to a depth of 28 feet and then use the impact hammer for remaining depth of penetration. For Test Pile No. 2, the pile installation sequence was as follows: Pre-drill to a depth of 8 feet, vibrate the pile to a depth of 21 feet and then use the impact hammer for the remaining depth of penetration.

Tables 1 and 2 show the summary of the dynamic test results for test piles 1 and 2 respectively. For test pile No. 1, pile monitoring began at a depth of 56 feet and was terminated at a depth of 60.5 feet. A dynamic capacity of 1518 kips (RSP capacity method) was predicted at 60 feet when using a soil damping value (J) equal to 0.15. This damping value is considered to be appropriate for the coarse grained, granular materials at this location. However, subsequent analysis using the CAPWAP computer program indicated the damping value was considerably greater than might be expected. This disagreement will be further discussed under the CAPWAP section of this report. The maximum measured compressive stress was 33.5 ksi or approximately 95 percent of yield stress. The Delmag D-46 performed satisfactorily with a 9.0-foot stroke and a transfer efficiency of 54 percent. The relatively high blow count (approximately 100 blows per foot), indicates that the pile displacement per blow may have been insufficient to completely mobilize the end bearing resistance of the 30-inch diameter pile. If this assumption is correct, then the analyzer capacity predictions would be expected to be conservative as compared to static load test results. Unfortunately, a static load test was not conducted at this location due to the small number of piles. On May 20th, the retap of Pile No. 1 indicated no significant increase in capacity. The AK-DOT dynamic formula indicated Pile No. 1 had an end of driving, allowable load of 818 kips (pile design load equal to 400 kips)

For test pile No. 2, pile monitoring began at a depth of 55.5 feet and was terminated at a depth of 66.2 feet. A dynamic capacity, using the RSP method of prediction of 1270 kips, was recorded using a soil damping value of 0.15 (J). The measured compressive stresses were considerably lower on Pile No. 2, (24.5-27.5 ksi) as compared to Pile No. 1. This might be partially explained by the shorter stroke (8.0' compared to 9.0') and generally lower transfer energies (range of 37-44 percent). Similar to Pile No. 1, the retap on Pile No. 2 showed no significant gain in capacity. Mr. Clark, recorded in his field notes that he had more confidence in the analyzer data quality for Pile No. 2. He attributed this to the possibility of transducer malfunction on Pile No. 1. The AK-DOT dynamic formula indicated Pile No. 2 had an end of driving allowable load of 800 kips.

CAPWAP ANALYSIS

The purpose for performing a "CAPWAP" analysis is to determine the load transfer distribution along the pile and the soil damping parameters. These two items must be assumed when performing a wave equation analysis. When using the pile analyzer, the damping constant, "J", is estimated, based on the soil type. These assumptions introduce some uncertainties in the results obtained from the wave equation and the pile analyzer. The CAPWAP analysis is based on force and acceleration data recorded on magnetic tape during pile driving. A reasonable assumption is made regarding soil parameters and then the motion of the pile is assumed, using the measured pile top acceleration as a boundary value. The output results are pile element motions, soil resistance forces and computed pile top forces. The computed and measured pile top forces are compared and, if they differ, appropriate changes are made to the soil model assumptions. Finally, a computed pile top force will be obtained which cannot be further improved.

A CAPWAP Analysis was performed for Pile No. 2 for the Geist Road Project. The analysis was performed for FHWA by Goble, Rausche, Likins, and Associates, Inc. of Cleveland, Ohio. The hammer blows selected represents both ends of driving and restrike conditions. Results of the analysis are summarized in Table 3. The normal pile structural model was adjusted due to the piles' unusual geometry. It appears the soil column inside the lower pile section acts dynamically, as if the pile were filled with concrete. The pile impedance modification affect the computed force and soil resistances, hence, the actual distribution of resistance is somewhat less reliable than for uniform pile shafts.

A high percentage of the total resistance was observed at the location of the closure plate for all CAPWAP analyses. The predicted resistances were less than the desired 800 kips ultimate capacity (Factor of Safety = 2.0).

A CAPWAP capacity prediction of 769 kips was made using a case damping value of .7 which is highly unusual for coarse, granular materials (typical values of .1 -.15 might be expected) Other occurrences of unusually high case damping values have been reported by others. Several of these are presented in the paper "High Case Damping Constants in Sand," by C. Thompson and G. Goble. A copy of this paper is included as Appendix A.

Two analyses were performed using the pile restrike data, identified as blow No. 3. The first analysis used the data directly obtained using normal calibrations. Because of poor proportionality between force and velocity waves, the calibration of the force data was modified. The quality of the data improved after this adjustment; the total restrike capacity did not change to any substantial degree compared to the end of driving condition. The entire results of the CAPWAP Analysis are included as Appendix B.

CONCLUSIONS

1. The Pile Analyzer performed well in monitoring driving stresses, pile capacities and hammer performance.

2. Poor correlation of ultimate pile capacity was obtained between the pile analyzer and CAPWAP analysis. The unusual values of case damping (as determined by the CAPWAP analysis), cannot be completely explained, but may, in part, be due to the unusual pile geometry.

3. By using the most conservative capacity prediction (769 kips retap via CAPWAP), a resulting safety factor of 1.92 was computed. The author believes the actual safety factor is likely greater because of the inability to completely mobilize pile tip resistance at the observed driving resistance.

4. The AK-DOT dynamic formula agreed very well with the CAPWAP analysis, but disagreed with the pile analyzer prediction. This discrepancy is directly related to the variation of the soil damping value "J" between field measurements and the CAPWAP computer analysis.

   The only method of determining the "true" pile capacity, is by conducting a static load test to failure which, unfortunately, was not performed due to the small quantity of piles. Readers are cautioned not to draw any general conclusions regarding the reliability of any particular prediction method from this case study alone.

GEIST ROAD RAILROAD BRIDGE
FAIRBANKS, ALASKA

TABLE 1 SUMMARY OF DYNAMIC MONITORING RESULTS
SITE NUMBER 1, PILE NUMBER 1.

DATE .... MAY 19, 1988

PILE TYPE ...... 30" OD. 1/2" WALL PIPE PILE

PILE NO. .... # 1

LENGTHS .... 60' + 55' = 115'

PILE AREA ...... 46.3 sq. in.

HAMMER MODEL .... DELMAG, D-46

HAMMER TYPE .... SINGLE ACTING

HAMMER RATED ENERGY .. 105,000FT. LBS

RAM WEIGHT .... 10.1 KIPS

DEPTH FEET *	BLOW COUNT PER FOOT: ANALYZER	BLOW COUNT PER FOOT: DRIVING RECORD	RSP WITH J = .15 KIPS	RSU J = KIPS	SFT KIPS	FMX KIPS	MAX. COMP. STRESS KSI	CTEN KIPS	MAX. TENSILE STRESS, KSI	EMX FT. KIPS	STROKE FEET	HAMMER ENERGY (RAM WT. x STROKE) FT. KIPS.	TRANSFER EFFICIENCY TRANSFER ENERGY / APPLIED HAMMER ENERGY %	REMARKS
56	5	5	1381	--	--	1286	27.8	36	0.78	384	7.5	75.8	50.7	A 2'' THICK STEEL PLATE 2'6" DIA. WELDED 30' FROM TIP.
57	65	65	1493	--	--	1294	27.9	69	1.49	394	8.5	85.7	45.9	PILE VIBRATED TO 46' DEPTH. DYNAMIC EQUIPMENT ATTACHED
58	85	86	1396	--	--	1360	29.4	66	1.43	411	9.0	90.9	45.2	AND MONITORED AT 56' OF PENTRATION.
60	98	95	1518	--	--	1554	33.6	116	2.51	494	9.0	90.9	54.3	
60'6"	85	--	1532	--	--	1544	33.3	107	2.31	475	9.5	96.0	49.5	INITIAL DRIVING COMPLETED AT 66'6".
RETAP MAY 20, 1988														
60'6"	2 / 1"	--	1695	--	--	1668	36.0	36	0.78	453	8.0	80.8	56.1	RETAP AT 60'6". END RETAP AT 60'8".
60'7"			1314	--	--	1360	29.4	125	2.70	280	8.0	80.8	34.7	
60'8"			1322	--	--	1317	28.4	107	2.31	253	8.0	80.8	31.3	

* DISTANCE FROM THE GROUND LINE TO PILE TIP
RSP = ULTIMATE STATIC RESISTANCE USING DAMPING J
FMX = MAXIMUM MEASURED FORCE IN PILE AT THE TRANSDUCER LOCATION
CTEN = MAXIMUM COMPUTED TENSILE FORCE ANYWHERE IN THE PILE
RSU = CAPACITY INCLUDING UNLOADING WITH DAMPING J
BTA = INTEGRITY FACTOR (% DAMAGE)

MAXIMUM ALLOWABLE COMPRESSIVE OR TENSILE DRIVING STRESS = 0.9 x Fy = 0.9 x 36 = 32.4 KSI
J = ASSUMED DAMPING PARAMETER (DEPENDS ON SOIL TYPE)
EMX = MAXIMUM TRANSFERRED ENERGY AT TRANSDUCER LOCATION
SFT = SKIN FRICTION TOTAL (NOT REDUCED FOR DAMPING)
RMX = MAXIMUM CASE - GOBLE CAPACITY (KIPS)
RMN = MINIMUM CASE - GOBLE CAPACITY (KIPS)

GEIST ROAD RAILROAD BRIDGE
FAIRBANKS, ALASKA

TABLE 2 SUMMARY OF DYNAMIC MONITORING RESULTS
SITE NUMBER 1, PILE NUMBER 2.

DATE .... MAY 19, 1988
PILE TYPE ...... 30" OD. 1/2" WALL PIPE PILE
PILE NO. .... # 2
LENGTHS .... 60' + 55' = 115'
PILE AREA ...... 46.3 sq. in.

HAMMER MODEL .... DELMAG, D-46
HAMMER TYPE .... SINGLE ACTING
HAMMER RATED ENERGY .. 105,000FT. LBS
RAM WEIGHT .... 10.1 KIPS

DEPTH FEET *	BLOW COUNT PER FOOT: ANALYZER	BLOW COUNT PER FOOT: DRIVING RECORD	RSP WITH J = .15 KIPS	RSU J = KIPS	SFT KIPS	FMX KIPS	MAX. COMP. STRESS KSI	CTEN KIPS	MAX. TENSILE STRESS, KSI	EMX FT. KIPS	STROKE FEET	HAMMER ENERGY (RAM WT. x STROKE) FT. KIPS.	TRANSFER EFFICIENCY TRANSFER ENERGY / APPLIED HAMMER ENERGY %	REMARKS
55'6"	4	4	1549	--	--	1378	29.8	57	1.23	445	9.0	90.9	49.0	A 2'' THICK STEEL PLATE 2'6" DIA. WELDED 30' FROM TIP.
56	25	--	1413	--	--	1274	27.5	108	2.33	348	8.5	85.9	40.5	PILE VIBRATED TO 44' DEPTH. DYNAMIC EQUIPMENT ATTACHED
57	80	--	1362	--	--	1233	26.6	114	2.46	325	8.0	80.8	40.2	AND MONITORED AT 55'6" OF PENTRATION.
58	78	80	1392	--	--	1275	27.5	144	3.11	347	8.5	85.9	40.4	
59	89	91	1375	--	--	1264	27.3	147	3.17	336	8.0	80.8	41.6	
60	92	91	1367	--	--	1237	26.7	160	3.46	334	8.0	80.8	41.3	
61	114	115	1370	--	--	1274	27.5	165	3.56	345	8.0	80.8	42.7	
62	86	--	1347	--	--	1266	27.3	168	3.63	349	8.0	80.8	43.2	
63	79	--	1339	--	--	1246	26.9	180	3.89	352	8.0	80.8	43.6	
64	95	90	1298	--	--	1205	26.0	179	3.87	316	8.0	80.8	39.1	
65	89	95	1296	--	--	1159	25.0	158	3.41	299	8.0	80.8	37.0	
66	104	--	1302	--	--	1169	25.2	142	3.07	298	8.0	80.8	36.9	
66 2"	30	--	1270	--	--	1141	24.6	156	3.37	309	8.0	80.8	38.2	INITIAL DRIVING ENDED AT 66'2".
RETAP MAY 20, 1988														
66 2"	3 / 1"	--	1317	--	--	1109	24.0	157	3.39	284	8.0	80.8	35.1	RETAP BEGINS AT 66'2". ENDED AT 66'3".
66 3"	16 / 2"	--	1303	--	--	1123	24.3	126	2.72	276	8.0	80.8	34.2	

* DISTANCE FROM THE GROUND LINE TO PILE TIP
RSP = ULTIMATE STATIC RESISTANCE USING DAMPING J
FMX = MAXIMUM MEASURED FORCE IN PILE AT THE TRANSDUCER LOCATION
CTEN = MAXIMUM COMPUTED TENSILE FORCE ANYWHERE IN THE PILE
RSU = CAPACITY INCLUDING UNLOADING WITH DAMPING J
BTA = INTEGRITY FACTOR (% DAMAGE)

MAXIMUM ALLOWABLE COMPRESSIVE OR TENSILE DRIVING STRESS = 0.9 x Fy = 0.9 x 36 = 32.4 KSI
J = ASSUMED DAMPING PARAMETER (DEPENDS ON SOIL TYPE)
EMX = MAXIMUM TRANSFERRED ENERGY AT TRANSDUCER LOCATION
SFT = SKIN FRICTION TOTAL (NOT REDUCED FOR DAMPING)
RMX = MAXIMUM CASE - GOBLE CAPACITY (KIPS)
RMN = MINIMUM CASE - GOBLE CAPACITY (KIPS)

Table 3: CAPWAPC Summary

Data	Ultimate Capacity-kips skin	(at plate)	toe	total	Smith Damping skin	toe	Quakes skin	plate	toe
EOD	605	(518)	43	648	0.178	0.404	0.17	0.26	0.25
RES	667	(554)	99	766	0.148	0.310	0.14	0.24	0.12
RES, ADJUSTED	718	(557)	51	769	0.146	0.564	0.14	0.33	0.12

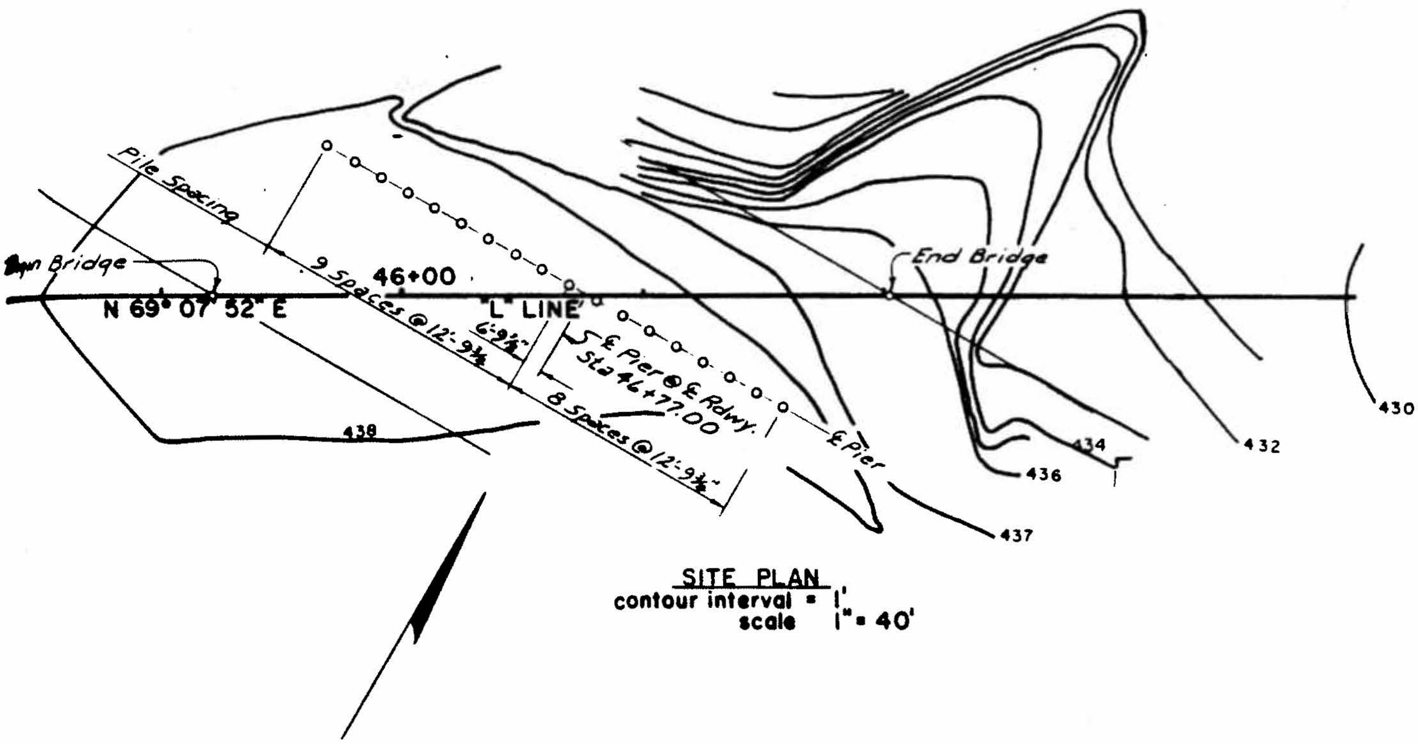
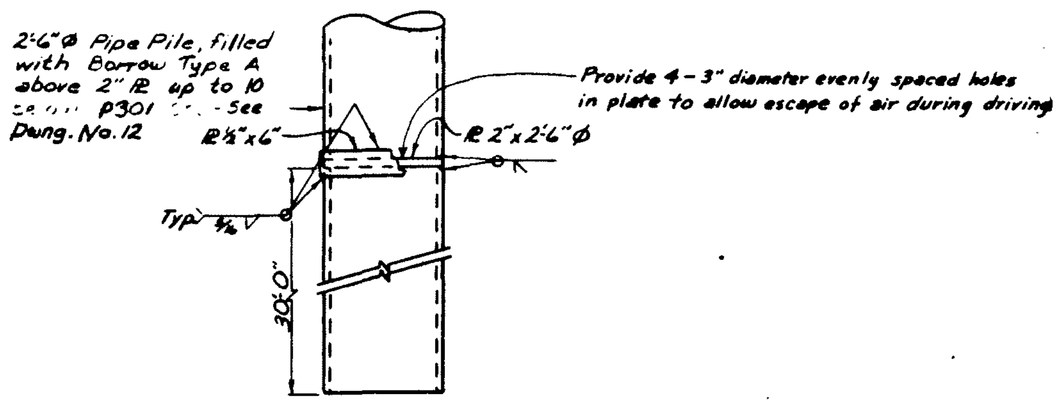

Pile Spacing
Bgn Bridge
N 69° 07' 52" E
46+00
"L" LINE
9 Spaces @ 12'-9¾"
6'-9¾"
℄ Pier @ ℄ Rdwy.
Sta 46+77.00
8 Spaces @ 12'-9¾"
℄ Pier
End Bridge
438
437
436
434
432
430
SITE PLAN
contour interval = 1'
scale 1" = 40'

Figure 1

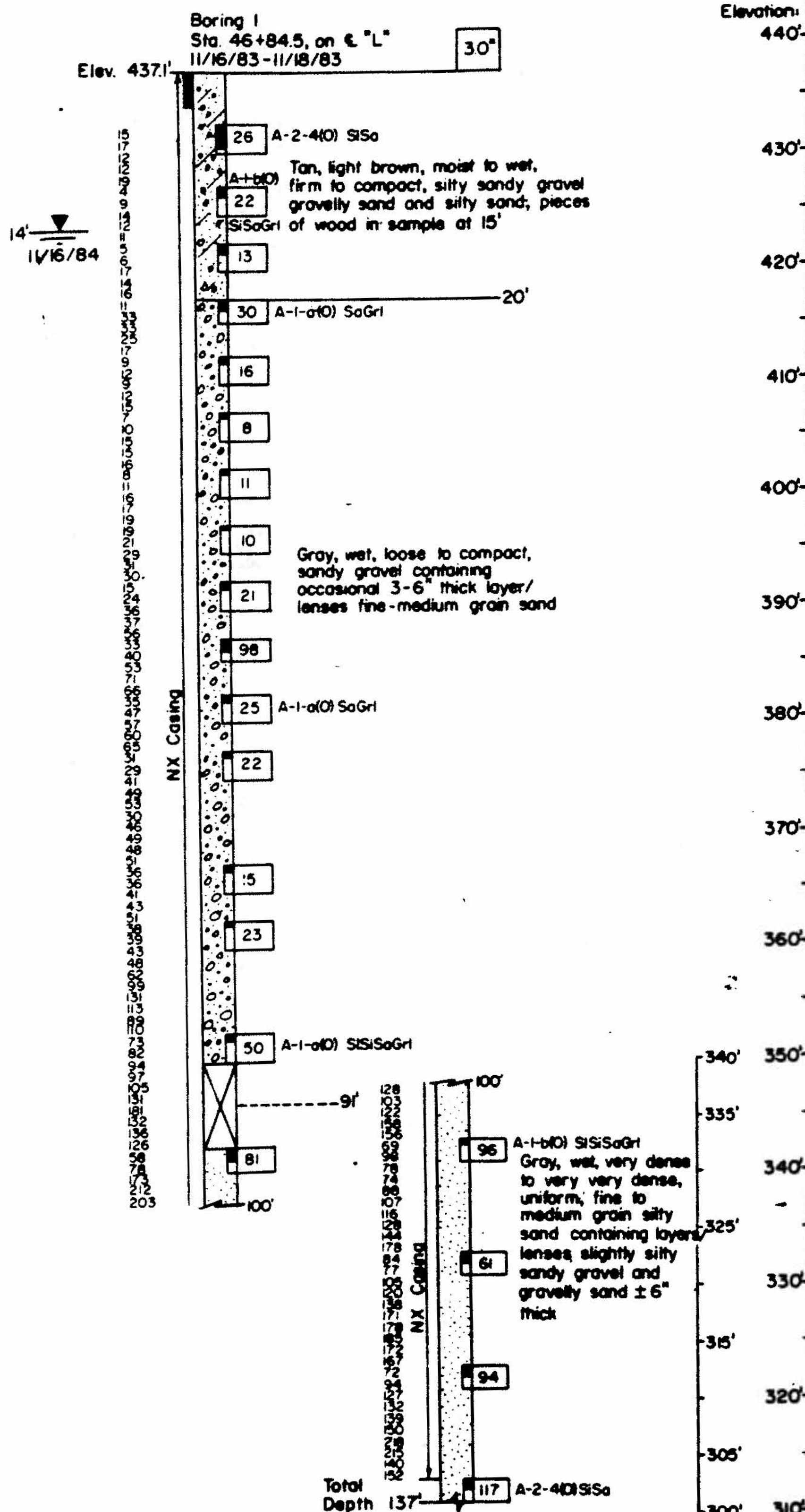


PILE CLOSURE
PL DETAIL

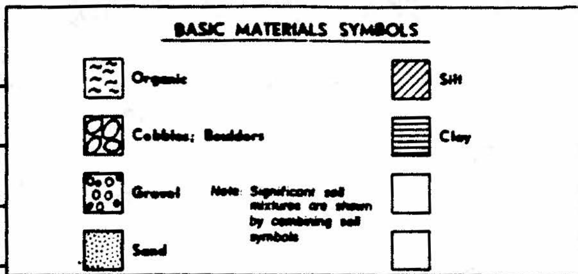
Figure 2

Figure 3

STATE	PROJECT DESIGNATION	YEAR	SHEET NO.	TOTAL SHEETS
ALASKA	RS-RRS-M-000S(52)	1987	95	118



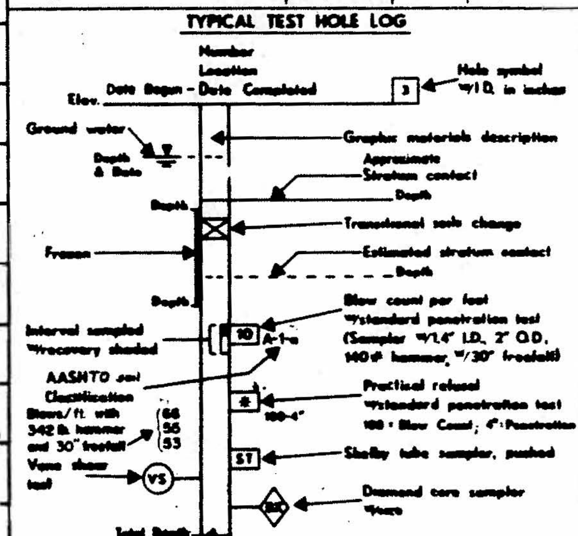
Note: Casing pullout jacking pressure was 19,630 lbs after sampling at 135'

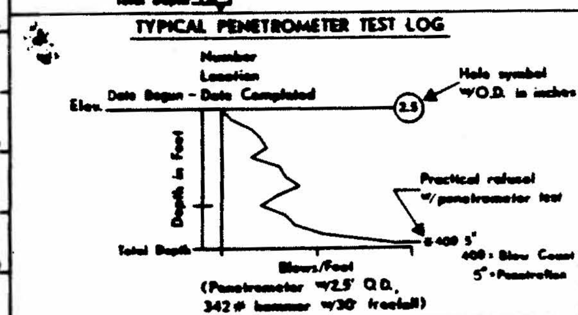


RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

Based on Standard Penetration Test

GRANULAR		COHESIVE	
Blows/ft	Rel. Density	Blows/ft	Consistency
0 - 5	Very Loose	2	Very Soft
6 - 10	Loose	2 - 4	Soft
11 - 20	Firm	5 - 8	Medium
21 - 35	Compact	9 - 15	Stiff
36 - 50	Dense	16 - 30	Very Stiff
51 - 70	Very Dense	31 - 60	Hard
71+	V.Very Dense	61+	Very Hard





LOG OF TEST HOLES

GEIST ROAD OVERHEAD

ROUTE NO. S-665

SOIL GRAIN SIZE DEFINITIONS

Boulder (rounded)............ >12" Diameter
Cobble (rounded)............ 3" Diameter - 12" Diameter
Broken rock (angular)........ >3" Diameter
Gravel (rounded); Stone (angular) #10 Sieve - 3" Diameter
Sand........................ #200 Sieve - #10
Silt/clay................... #200 Sieve

Note: Soil classifications are visual only unless AASHTO soil class is shown on the log

State of Alaska
DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES
Juneau, Alaska

Date April, 1984
Approved [signature]

BRIDGE NO. 1697
DWNG. NO. 19

APPENDICES

12

HIGH CASE DAMPING CONSTANTS IN SAND

Christopher D. Thompson* George G. Goble†

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

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

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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† Professor, Department of Civil and Architectural Engineering, University of Colorado, Boulder, Colorado, U.S.A.

APPENDIX A

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.

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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 - 6 m depth. 10 to 20 bl/300 mm. Organic Clayey silt to 8 - 6.8 m, 4 to 9 bl/300 mm. Silty Sand with some gravel to bottom of borehole 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 19 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	8
				400 mm square Prestressed Concrete	4	15.7	8
				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 40 m. GWT at 5m.	Delmag D46 open end diesel hammer RE 143 KJ Standard Delmag Aluminum and 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	30	8
				360 mm O.D. by 174 mm thick Steel H Pipe	3	40	26
C	New Westminster, B.C.	Water at 5 to 8 m depth. Clayey silt to between 12.2 and 16 m depth, 20 to 30 bl/300mm. Sand, fine to medium to 20 and 21 m depth, 10 to 40 bl/300mm.	Menck MHU Hydraulic Hammer for Piles 1 & 2	410 mm Octagonal Prestressed Concrete	1	17.0	31
				410 mm Octagonal Prestressed Concrete	2	23.6	9
			Delmag D30-23 open end diesel hammer for Piles 3 to 5.	610 mm Octagonal Prestressed Concrete	3	23.8	6
		Sand, fine to 40m depth, 17 to 70 bl/300mm.		610 mm Octagonal Prestressed Concrete	4	23.8	40
				610 mm Octagonal Prestressed Concrete	5	23.8	76
D	North York, Ont.	Clayey silt fill to 10 m depth, 4 - 9 bl/300 mm. Sand, fine to medium to 36 m depth. 11 to 27 bl/ft. GWT at 6 m.	35 KN Drop Hammer falling 1.5 m	290 mm O.D. by 6.5 mm thick Steel Pipe	1	10.2	26
				244 mm O.D. by 6.5 mm thick Pipe Steel	2	24.2	17

APPENDIX A

Table 1 Cont'd : Site, Soil, Hammer and Pile Data							
Project	Location	Soil Conditions	Hammer & Capblock	Pile Description	Pile Number	Embedment (m)	Penetration Resistance (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. 30 - 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-30-13 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 D30-32	500 x 500 mm PS Concrete	1	45	20
		Fine Sand, 50 bl/300 mm.	Delmag D30-32	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	0.5	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		4460	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 avallable 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 thoeretical 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 Toe 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.

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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 recommenced 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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CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 EOD   Blow No 0   03-Jun-88

Final CAPWAPC Capacity: Ru 647.9, Skin 604.7, Toe 43.2 Kips

Soil Sgmnt No.	Depth Below Gages ft	Depth Below Grade ft	Quake in	Soil Damping Case	Viscs Kips/ft/s	Smith s/ft	Ru Kips	Sum of Ru Kips	Unit Skin Frctn Kips/ft2
								647.9	
1	66.5	16.5	.170	.014	1.1	.178	6.5	641.5	.09
2	74.9	24.8	.170	.014	1.1	.178	6.5	635.0	.09
3	82.9	33.1	.170	.014	1.1	.178	6.5	628.5	.09
4	87.0	39.1	.170	.014	1.1	.178	6.5	622.1	.17
5	91.2	43.2	.260	1.114	92.0	.178	518.0	104.1	13.85
6	95.4	47.4	.170	.026	2.2	.178	12.2	91.9	.32
7	99.6	51.6	.170	.026	2.2	.178	12.2	79.7	.32
8	103.9	55.8	.170	.026	2.2	.178	12.2	67.5	.32
9	108.2	60.1	.170	.026	2.2	.178	12.2	55.4	.31
10	112.5	64.4	.170	.026	2.2	.178	12.2	43.2	.31
Sum				1.300	107.4		604.7		
Avrge			.247			.178	60.5		1.59
Toe			.250	.211	17.5	.404	43.2		9.60

Soil Model Extensions		Skin	Toe
Unloading Level	(% of Ru)	0	

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 EOD   Blow No   0   03-Jun-88

EXTREMA TABLE

Pile Sgmnt No.	Depth below Gages ft	max. Force Kips	min. Force Kips	max. Comp. Stress Kips/in2	max. Tension Stress Kips/in2	max. trnsfd. Energy Kips-ft	max. Veloc. ft/s	max. Displ. in
1	4.2	1096.3	-62.2	23.68	-1.34	29.97	13.8	.663
3	12.5	1120.0	-179.9	24.19	-3.89	29.25	13.3	.590
6	25.0	1114.1	-195.3	24.06	-4.22	28.06	13.1	.530
10	41.6	1108.2	-169.6	23.94	-3.66	26.03	12.9	.450
13	54.1	1111.8	-53.3	24.01	-1.15	24.28	12.6	.430
17	70.7	1249.2	-77.7	26.98	-1.68	23.36	11.4	.390
20	82.9	1500.5	-127.5	9.68	-.82	22.66	7.3	.360
23	89.1	1558.6	-200.2	2.21	-.28	21.77	6.8	.340
27	97.5	819.6	-346.5	1.16	-.49	8.16	6.9	.320
30	103.9	756.9	-271.1	1.07	-.38	7.13	7.2	.320
33	110.3	421.3	-68.8	.60	-.10	5.17	10.6	.330
34	112.5	259.8	-43.2	.37	-.06	4.55	11.4	.332
Absolute	79.0			32.75		(T=	25.7 ms)	
	33.3				-4.96	(T=	35.9 ms)	

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 EOD   Blow No 0   03-Jun-88

PILE PROFILE AND PILE MODEL

	Depth ft	Area in2	E-Modulus Kips/in2	Spec. Weight Kips/ft3
1	.00	46.30	30000.0	.492
2	82.50	46.30	30000.0	.492
3	82.50	706.00	30000.0	.492
4	82.66	706.00	30000.0	.492
5	82.66	706.00	2000.0	.135
6	112.50	706.00	2000.0	.120

Segmnt No.	Depth B.G. ft	Impedance Kips/ft/s	Tensn Slack inch	Compr. Slack inch
1	4.16	82.6	.0000	.0000
20	82.92	189.0	.0000	.0000
21	84.97	220.0	.0000	.0000
22	87.04	199.3	.0000	.0000
23	89.11	188.6	.0000	.0000
24	91.19	177.9	.0000	.0000
25	93.29	152.2	.0000	.0000
26	95.39	136.5	.0000	.0000
27	97.49	132.1	.0000	.0000
28	99.61	127.6	.0000	.0000
29	101.74	123.2	.0000	.0000
30	103.87	118.8	.0000	.0000
31	106.02	114.3	.0000	.0000
32	108.17	109.9	.0000	.0000
33	110.33	105.4	.0000	.0000
34	112.50	101.0	.0000	.0000

Pile Damping (%) 2.5, Time Incr (ms) .247, Wave Speed 13373.4

FHWA ALASKA GEIST ROAD PILE 2 EOD

Case Method Capacity Results

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
Rs	1445.	1322.	1199.	1077.	954.	831.	708.	585.	462.	339.
Rx	1445.	1322.	1199.	1077.	954.	831.	708.	585.	549.	549.
Ru	1501.	1384.	1267.	1149.	1032.	915.	797.	680.	563.	445.
Ra Ra2	246.	362.								

STATIC ANALYSIS

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 EOD
Blow 0 Date 03-Jun-88

DYNAMIC D-TOE, E-P R-TOE

I	Top Load Kips	Top Set IN	Bot. Load Kips	Bot. Set IN
37	35.0	.040	2.1	.012
45	68.6	.078	4.1	.024
50	101.9	.116	6.1	.035
52	142.4	.162	8.5	.049
53	187.9	.214	11.3	.065
54	254.4	.289	15.3	.088
55	338.5	.386	20.4	.118
56	428.9	.491	26.1	.151
57	507.5	.586	31.6	.183
58	570.4	.663	36.5	.211
59	622.5	.728	40.5	.235
60	647.9	.767	43.2	.253
76	612.1	.798	41.0	.313
81	578.7	.753	38.9	.295
93	612.7	.792	41.0	.307
98	646.7	.831	43.0	.318
102	609.7	.782	40.8	.299

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES Blow No 3 88 07 20

Final CAPWAPC Capacity: Ru 765.9, Skin 667.4, Toe 98.5 kips

Soil Sgmnt No.	Depth Below Gages ft	Depth Below Grade ft	Quake in	Soil Damping Case	Viscs kips/ft/s	Smith s/ft	Ru kips	Sum of Ru kips	Unit Skin Frctn kips/ft2
								765.9	
1	65.9	15.3	.140	.014	1.2	.148	8.1	757.8	.05
2	74.1	23.5	.140	.014	1.2	.148	8.1	749.7	.05
3	82.4	31.7	.140	.014	1.2	.148	8.1	741.7	.05
4	86.7	38.1	.140	.014	1.2	.148	8.1	733.6	.10
5	90.9	42.3	.240	.993	82.0	.148	554.4	179.2	6.70
6	95.2	46.5	.170	.029	2.4	.148	16.1	163.0	.19
7	99.5	50.8	.160	.029	2.4	.148	16.1	146.9	.19
8	103.8	55.1	.150	.029	2.4	.148	16.1	130.7	.19
9	108.1	59.4	.140	.029	2.4	.148	16.1	114.6	.19
10	112.5	63.8	.130	.029	2.4	.148	16.1	98.5	.19
Sum				1.195	98.8		667.4		
Avrge			.224			.148	66.7		.79
Toe			.120	.370	30.6	.310	98.5		20.06

Soil Model Extensions		Skin	Toe
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kips)		1.10

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES   Blow No 3   88 07 20

EXTREMA TABLE

Pile Sgmnt No.	Depth below Gages ft	max. Force kips	min. Force kips	max. Como. Stress kips/in2	max. Tension Stress kips/in2	max. trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	4.1	1098.6	-37.1	23.73	-.80	31.59	15.1	.747
3	12.4	1188.7	-192.9	25.67	-4.17	31.32	13.9	.590
6	24.7	1189.4	-112.3	25.69	-2.42	30.22	13.8	.540
10	41.2	1185.6	-198.7	25.61	-4.29	28.18	13.7	.460
13	53.6	1187.7	-104.2	25.65	-2.25	24.96	13.5	.380
17	70.0	1263.1	-136.1	27.28	-2.94	24.20	12.4	.330
20	82.4	1622.7	-185.0	35.05	-4.00	23.61	7.5	.300
23	88.8	1599.0	-242.0	2.26	-.34	22.65	7.2	.280
27	97.3	922.7	-301.2	1.31	-.43	11.05	7.0	.270
30	103.8	884.5	-275.3	1.25	-.39	10.11	7.0	.280
33	110.3	828.4	-167.2	1.17	-.24	8.73	7.7	.280
34	112.5	631.5	-134.2	.89	-.19	7.74	9.1	.289
Absolute	82.4			35.05		(T=	25.7 ms)	
	37.1				-4.63	(T=	36.3 ms)	

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES Blow No 3 88 07 20

PILE PROFILE AND PILE MODEL

	Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3
1	.00	46.30	30000.0	.492
2	82.50	46.30	30000.0	.492
3	82.50	706.00	30000.0	.492
4	82.66	706.00	30000.0	.492
5	82.66	706.00	2100.0	.135
6	112.53	706.00	2100.0	.120

Segmnt No.	Depth B.G. ft	Impedance kips/ft/s	Tensn Slack inch	Compr. Slack inch
1	4.12	82.6	.0000	.0000
20	82.39	132.6	.0000	.0000
21	84.61	263.9	.0000	.0000
22	86.70	193.6	.0000	.0000
23	88.80	180.4	.0000	.0000
24	90.91	167.2	.0000	.0000
25	93.03	153.9	.0000	.0000
26	95.16	140.7	.0000	.0000
27	97.30	136.3	.0000	.0000
28	99.45	131.8	.0000	.0000
29	101.61	127.3	.0000	.0000
30	103.77	122.9	.0000	.0000
31	105.95	118.4	.0000	.0000
32	108.13	113.9	.0000	.0000
33	110.33	135.7	.0000	.0000
34	112.53	135.0	.0000	.0000

Pile Damping (%) 3.0, Time Incr (ms) .245, Wave Speed 13506.7

FHWA ALASKA GEIST ROAD PILE 2 RES

Case Method Capacity Results

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
Rs	1601.	1485.	1369.	1253.	1138.	1022.	906.	790.	674.	559.
Rx	1602.	1485.	1369.	1253.	1138.	1022.	906.	790.	675.	616.
Ru	1624.	1510.	1397.	1283.	1170.	1056.	943.	829.	716.	602.
Ra Ra2	37.	252.								

STATIC ANALYSIS

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES
Blow 3 Date 88 07 20

DYNAMIC D-TOE, E-P R-TOE

I	Top Load kips	Top Set IN	Bot. Load kips	Bot. Set IN
51	38.5	.040	6.9	.008
68	78.7	.081	14.1	.017
75	136.5	.141	24.5	.030
76	187.5	.193	33.7	.041
77	265.3	.273	47.6	.058
78	365.1	.377	65.8	.080
79	475.2	.493	86.8	.106
80	570.9	.597	98.5	.132
81	634.2	.672	98.5	.157
82	686.2	.735	98.5	.180
83	730.1	.788	98.5	.199
85	765.9	.842	98.5	.225
101	722.5	.859	90.5	.277
104	672.7	.804	81.2	.263
107	616.4	.742	70.8	.247
111	572.2	.694	62.6	.235
122	530.3	.648	54.9	.223
125	478.5	.591	45.3	.208
127	422.6	.530	34.9	.192
129	353.9	.454	21.8	.172
130	315.4	.411	14.3	.161
131	275.9	.366	6.5	.149
133	208.7	.287	.0	.124
135	157.6	.222	.0	.100
137	116.0	.168	.0	.078
140	64.4	.100	.0	.050

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES   Blow No 3   88 07 21

Final CAPWAPC Capacity: Ru 769.4, Skin 718.1, Toe 51.3 kips

Soil Sgmnt No.	Depth Below Gages ft	Depth Below Grade ft	Quake in	Soil Damping Case	Viscs kips/ft/s	Smith s/ft	Ru kips	Sum of Ru kips	Unit Skin Frctn kips/ft2
								769.4	
1	65.9	15.3	.140	.014	1.2	.146	8.1	761.3	.05
2	74.1	23.5	.140	.014	1.2	.146	8.1	753.1	.05
3	82.4	31.7	.140	.014	1.2	.146	8.1	745.0	.05
4	86.7	38.1	.140	.014	1.2	.146	8.1	736.9	.10
5	90.9	42.3	.330	.986	81.4	.146	557.3	179.6	6.74
6	95.2	46.5	.170	.045	3.8	.146	25.7	153.9	.31
7	99.5	50.8	.160	.045	3.8	.146	25.7	128.3	.30
8	103.8	55.1	.150	.045	3.8	.146	25.7	102.6	.30
9	108.1	59.4	.140	.045	3.8	.146	25.7	77.0	.30
10	112.5	63.8	.130	.045	3.8	.146	25.7	51.3	.30
Sum				1.270	104.9		718.1		
Avrge			.289			.146	71.8		.85
Toe			.120	.350	28.9	.564	51.3		10.45

Soil Model Extensions		Skin	Toe
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kips)		1.10

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES   Blow No 3   88 07 21

EXTREMA TABLE

Pile Sgmnt No.	Depth below Gages ft	max. Force kips	min. Force kips	max. Como. Stress kips/in2	max. Tension Stress kips/in2	max. trnsfd. Energy kips-ft	max. Veloc. ft/s	max. Displ. in
1	4.1	1339.4	-45.2	28.93	-.98	38.51	15.1	.747
3	12.4	1310.8	-197.3	28.31	-4.26	37.86	15.3	.650
6	24.7	1311.3	-110.7	28.32	-2.39	36.50	15.2	.590
10	41.2	1306.1	-198.8	28.21	-4.29	34.02	15.1	.510
13	53.6	1307.7	-105.7	28.24	-2.28	31.00	14.9	.420
17	70.0	1391.2	-135.4	30.05	-2.92	30.12	13.7	.400
20	82.4	1787.0	-186.1	38.60	-4.02	29.40	8.3	.360
23	88.8	1755.1	-239.2	2.49	-.34	28.27	8.0	.340
27	97.3	1027.2	-299.6	1.45	-.42	14.03	7.7	.320
30	103.8	958.6	-287.7	1.36	-.41	12.64	7.8	.320
33	110.3	870.6	-167.0	1.23	-.24	10.23	8.4	.330
34	112.5	705.2	-139.9	1.00	-.20	8.21	10.2	.334
Absolute	82.4			38.60		(T=	25.7 ms)	
	37.1				-4.60	(T=	36.3 ms)	

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES   Blow No 3   88 07 21

PILE PROFILE AND PILE MODEL

	Depth ft	Area in2	E-Modulus kips/in2	Spec. Weight kips/ft3
1	.00	46.30	30000.0	.492
2	82.50	46.30	30000.0	.492
3	82.50	706.00	30000.0	.492
4	82.66	706.00	30000.0	.492
5	82.66	706.00	2100.0	.135
6	112.53	706.00	2100.0	.120

Segmnt No.	Depth B.G. ft	Impedance kips/ft/s	Tensn Slack inch	Compr. Slack inch
1	4.12	82.6	.0000	.0000
20	82.39	132.6	.0000	.0000
21	84.61	263.9	.0000	.0000
22	86.70	193.6	.0000	.0000
23	88.80	180.4	.0000	.0000
24	90.91	167.2	.0000	.0000
25	93.03	153.9	.0000	.0000
26	95.16	140.7	.0000	.0000
27	97.30	136.3	.0000	.0000
28	99.45	131.8	.0000	.0000
29	101.61	127.3	.0000	.0000
30	103.77	122.9	.0000	.0000
31	105.95	118.4	.0000	.0000
32	108.13	113.9	.0000	.0000
33	110.33	109.5	.0000	.0000
34	112.53	105.0	.0000	.0000

Pile Damping (%) 3.0, Time Incr (ms) .245, Wave Speed 13506.7

FHWA ALASKA GEIST ROAD PILE 2 RES

Case Method Capacity Results

	J=0.0	J=0.1	J=0.2	J=0.3	J=0.4	J=0.5	J=0.6	J=0.7	J=0.8	J=0.9
Rs	1714.	1587.	1459.	1332.	1204.	1076.	949.	821.	693.	566.
Rx	1719.	1589.	1459.	1332.	1204.	1076.	949.	821.	693.	668.
Ru	1757.	1634.	1511.	1387.	1264.	1140.	1017.	894.	770.	647.
Ra Ra2	63.	287.								

STATIC ANALYSIS

CAPWAPC - GRL & Associates, Inc.

FHWA ALASKA GEIST ROAD PILE 2 RES
Blow 3 Date 88 07 21

DYNAMIC D-TOE, E-P R-TOE

I	Top Load kips	Top Set IN	Bot. Load kips	Bot. Set IN
53	39.5	.043	4.6	.011
71	80.6	.087	9.4	.022
75	123.7	.134	14.4	.034
76	167.6	.182	19.5	.046
77	235.0	.255	27.3	.064
78	322.8	.351	37.8	.088
79	420.0	.460	50.0	.117
80	496.5	.551	51.3	.147
81	547.0	.620	51.3	.176
82	591.4	.680	51.3	.202
84	659.2	.773	51.3	.242
86	699.6	.828	51.3	.266
89	741.2	.885	51.3	.291
92	769.4	.932	51.3	.315
103	723.9	.901	46.0	.322
106	682.1	.856	41.1	.311
109	640.9	.811	36.3	.300
125	591.2	.758	30.6	.286
127	531.0	.692	23.6	.270
129	455.6	.610	14.6	.249
130	414.7	.565	9.6	.237
131	372.8	.518	4.3	.225
132	332.3	.472	.0	.212
134	271.2	.397	.0	.187
136	229.6	.341	.0	.163
139	179.3	.272	.0	.133
142	136.3	.213	.0	.108
145	91.9	.153	.0	.081
148	51.1	.097	.0	.057