OAKLAND ARMY BASE WHARVES

CONCRETE STRENGTH RESULTS

S7 - Concrete Compressive Strengths

DATE TESTED		3-28-13	3-28-13	3-28-13	4-2-13
ID NUMBER		S7-DC1	S7-DC2	S7-DC3	S7-DW1
DIAMETER,	in.	3.75	3.75	3.75	3.03
CROSS-SECTIONAL AREA,	in.	11.04	11.04	11.04	7.21
LENGTH RECEIVED,	in.	4.63	4.33	4.86	4.19
LENGTH TRIMMED,	in.	4.26	3.92	4.05	3.99
LENGTH TESTED,	in.	4.49	4.15	4.24	4.20
L/D RATIO,		1.20	1.11	1.13	1.39
CORR. FACTOR,		.918	.894	.906	.948
ULTIMATE LOAD,	lbs.	78840	61510	68790	44350
ULTIMATE STR.,	psi	7140	5570	6230	6150
CORR. ULTIMATE STR.,	psi	**6,560**	**4,980**	**5,640**	**5,830**
MAX. AGG. SIZE,	in.	1”	1”	1”	1”
REMARKS					

S8 - Concrete Compressive Strengths

DATE TESTED		3-28-13	3-28-13		
ID NUMBER		S8-DC1	S8-DC2		
DIAMETER,	in.	3.75	3.75		
CROSS-SECTIONAL AREA,	in.	11.04	11.04		
LENGTH RECEIVED,	in.	4.00	5.76		
LENGTH TRIMMED,	in.	3.36	5.05		
LENGTH TESTED,	in.	3.94	5.25		
L/D RATIO,		1.05	1.40		
CORR. FACTOR,		.882	.948		
ULTIMATE LOAD,	lbs.	75250	63620		
ULTIMATE STR.,	psi	6820	5760		
CORR. ULTIMATE STR.,	psi	**6,020**	**5,460**		
MAX. AGG. SIZE,	in.	1”	1”		
REMARKS					

S8 - Concrete Compressive Strengths

DATE TESTED		4-2-13	4-2-13	4-2-13	4-2-13
ID NUMBER		S8-PC1	S8-PC2	S8-PC3	S8-BH1
DIAMETER,	in.	3.03	3.03	3.03	3.03
CROSS-SECTIONAL AREA,	in.	7.21	7.21	7.21	7.21
LENGTH RECEIVED,	in.	5.96	7.36	4.04	4.20
LENGTH TRIMMED,	in.	5.29	5.69	3.28	3.65
LENGTH TESTED,	in.	5.48	5.94	3.53	3.88
L/D RATIO,		1.81	1.96	1.17	1.28
CORR. FACTOR,		-	-	.906	.936
ULTIMATE LOAD,	lbs.	47040	60070	46480	44350
ULTIMATE STR.,	psi	6520	8330	6450	6150
CORR. ULTIMATE STR.,	psi	**6,520**	**8330**	**5,840**	**5,760**
MAX. AGG. SIZE,	in.	1”	1”	1”	1”
REMARKS					

S9 - Concrete Compressive Strengths

DATE TESTED		3-28-13	3-28-13	3-28-13	
ID NUMBER		S9-DC1	S9-DC2	S9-DC3	
DIAMETER,	in.	3.75	3.75	3.75	
CROSS-SECTIONAL AREA,	in.	11.04	11.04	11.04	
LENGTH RECEIVED,	in.	5.68	5.67	6.81	
LENGTH TRIMMED,	in.	4.96	4.78	6.53	
LENGTH TESTED,	in.	5.15	5.00	6.95	
L/D RATIO,		1.37	1.33	1.85	
CORR. FACTOR,		.942	.942	--	
ULTIMATE LOAD,	lbs.	64320	55860	63280	
ULTIMATE STR.,	psi	5830	5060	5730	
CORR. ULTIMATE STR.,	psi	**5,490**	**4,770**	**5730**	
MAX. AGG. SIZE,	in.	2”	2”	2”	
REMARKS					

S10 - Concrete Compressive Strengths

DATE TESTED		3-28-13	3-28-13	3-28-13	3-28-13
ID NUMBER		S10-BH1	S10-DC1	S10-DC2	S10-DC3
DIAMETER,	in.	3.75	3.75	3.75	3.75
CROSS-SECTIONAL AREA,	in.	11.04	11.04	11.04	11.04
LENGTH RECEIVED,	in.	7.82	5.47	3.72	4.81
LENGTH TRIMMED,	in.	7.13	4.41	3.38	4.35
LENGTH TESTED,	in.	7.38	4.64	3.79	4.55
L/D RATIO,		1.97	1.24	1.01	1.21
CORR. FACTOR,		-	.930	.870	.918
ULTIMATE LOAD,	lbs.	75700	75500	75930	51880
ULTIMATE STR.,	psi	6860	6840	6880	4700
CORR. ULTIMATE STR.,	psi	**6,870**	**6,360**	**5,990**	**4,310**
MAX. AGG. SIZE,	in.	1”	2”	2”	2”
REMARKS					

S11 - Concrete Compressive Strengths

DATE TESTED		4-1-13	4-1-13	4-1-13	
ID NUMBER		S11-DC1	S11-DC2	S11-DC3	
DIAMETER,	in.	3.75	3.75	3.75	
CROSS-SECTIONAL AREA,	in.	11.04	11.04	11.04	
LENGTH RECEIVED,	in.	6.80	5.84	7.28	
LENGTH TRIMMED,	in.	5.97	5.47	6.66	
LENGTH TESTED,	in.	6.22	5.68	6.94	
L/D RATIO,		1.66	1.51	1.85	
CORR. FACTOR,		.972	.960	-	
ULTIMATE LOAD,	lbs.	58910	54530	74160	
ULTIMATE STR.,	psi	5240	4940	6720	
CORR. ULTIMATE STR.,	psi	**5,190**	**4,740**	**6,720**	
MAX. AGG. SIZE,	in.	2”	2”	2”	
REMARKS					

S12 - Concrete Compressive Strengths

DATE TESTED		4-1-13	4-1-13		
ID NUMBER		S12-DC1	S12-DC2		
DIAMETER,	in.	3.75	3.75		
CROSS-SECTIONAL AREA,	in.	11.04	11.04		
LENGTH RECEIVED,	in.	4.38	5.76		
LENGTH TRIMMED,	in.	3.85	5.28		
LENGTH TESTED,	in.	4.07	5.49		
L/D RATIO,		1.09	1.46		
CORR. FACTOR,		.894	.954		
ULTIMATE LOAD,	lbs.	69220	71000		
ULTIMATE STR.,	psi	6270	6430		
CORR. ULTIMATE STR.,	psi	**5,610**	**6,130**		
MAX. AGG. SIZE,	in.	2	2”		
REMARKS					

S12 - Concrete Compressive Strengths

DATE TESTED		4-1-13	4-1-13	4-1-13	
ID NUMBER		S12-BH1	S12-PC1	S12-PC2	
DIAMETER,	in.	3.03	3.03	3.03	
CROSS-SECTIONAL AREA,	in.	7.21	7.21	7.21	
LENGTH RECEIVED,	in.	6.75	8.36	4.22	
LENGTH TRIMMED,	in.	5.72	5.64	4.06	
LENGTH TESTED,	in.	5.89	5.88	4.32	
L/D RATIO,		1.94	1.94	1.43	
CORR. FACTOR,		-	-	.954	
ULTIMATE LOAD,	lbs.	37470	36770	38200	
ULTIMATE STR.,	psi	5200	5100	5300	
CORR. ULTIMATE STR.,	psi	**5,200**	**5100**	**5,060**	
MAX. AGG. SIZE,	in.	¾	1 ¼	1 ¼	
REMARKS					

S13 - Concrete Compressive Strengths

DATE TESTED		4-1-13	4-1-13	4-1-13	
ID NUMBER		S13-DC1	S13-DC2	S13-DW1	
DIAMETER,	in.	3.75	3.75	3.75	
CROSS-SECTIONAL AREA,	in.	11.04	11.04	11.04	
LENGTH RECEIVED,	in.	4.57	4.60	6.52	
LENGTH TRIMMED,	in.	4.05	4.15	5.98	
LENGTH TESTED,	in.	4.29	4.40	6.17	
L/D RATIO,		1.14	1.17	1.65	
CORR. FACTOR,		.906	.906	.972	
ULTIMATE LOAD,	lbs.	72390	78240	39840	
ULTIMATE STR.,	psi	6560	7090	3610	
CORR. ULTIMATE STR.,	psi	**5,940**	**6,420**	**3,510**	
MAX. AGG. SIZE,	in.	1	1	2	
REMARKS					

S14 - Concrete Compressive Strengths

DATE TESTED		4-1-13	4-1-13		
ID NUMBER		S14-DC1	S14-DC2		
DIAMETER,	in.	3.75	3.75		
CROSS-SECTIONAL AREA,	in.	11.04	11.04		
LENGTH RECEIVED,	in.	5.68	5.07		
LENGTH TRIMMED,	in.	5.35	4.90		
LENGTH TESTED,	in.	5.52	5.08		
L/D RATIO,		1.47	1.35		
CORR. FACTOR,		.954	.942		
ULTIMATE LOAD,	lbs.	84220	75830		
ULTIMATE STR.,	psi	7630	6870		
CORR. ULTIMATE STR.,	psi	**7,280**	**6,470**		
MAX. AGG. SIZE,	in.	1	1		
REMARKS					

OAKLAND ARMY BASE WHARVES

STEEL TENSILE RESULTS

Legend
S# = correlates to the sheet/map
DC = Deck Core
WC = Wall Core
PC = Pile Cap Core
BH = Bulk Head Wall Core

So S12-DC2 refers to Map S12, Deck Core, specimen #2

TESTING ENGINEERS, INC.

Quality Assurance Services
Materials Consulting
Since 1954

Laboratory No. E384 | TENSILE TESTS | Date: 4/3/13 | Page 1 of 2

Client No. In-House | P.O. No.: E0008

Project : In-House Consulting

Tag: Oakland Army Base

Description or Mark	1/2" Rebar (S7)	1/2" Rebar (S8)	1/2" Rebar (S15)	1/2" Rebar (S14)	Specified Min.
Heat Number	Unidentified	Unidentified	Unidentified	Unidentified	Requirement
Actual Diameter, Ins.	0.249	0.256	0.246	0.252	
Actual Area, Sq. Ins.	0.049	0.051	0.048	0.050	
Yield Point, Lbs.	2241	2835	2500	2569	
Maximum Load, Lbs.	3196	4652	4170	3938	
Yield Strength, Lbs. Per Sq. In.	45700	55500	52000	51500	
Tensile Strength Lbs. Per Sq. In.	65000	91000	87000	79000	
Elongation In 1 Inches	0.331	0.262	0.237	0.280	
Elongation, Percent	33.2	26.2	23.6	28.0	
Post Test Diameter, Ins.	0.150	0.176	0.174	0.165	
Reduction of Area, Percent	64.0	52.0	50.0	57.0	
Fracture	PC	PC	PC	PC	
Test Equipment: Instron 4486 S/N 11313J					SOP: 20.080

Remarks: Yield @ Halt to Force
Machined and tested in accordance with ASTM A370.

Results Reported To: TEI Consulting

TESTING ENGINEERS,INC.

By

Colin Subasa, Materials Testing Technician

The results presented in this report relate only to the items(s) tested.
This report can be reproduced only in its entirety unless written permission from TEI is obtained
Samples pertaining to this report will be discarded 30 days from the date of this report unless other wise advised

\\Teisys04\rmtlarch\In-House\E384_Consulting_E0008 Oak. Army Base_Pg 1 of 2.doc

Corporate Office – 2811 Teagarden Street – San Leandro, California 94577 – (510) 835-3142 – FAX (510) 834-3777
www.Testing-Engineers.com

ACCREDITED

Laboratory No. E384 | TENSILE TESTS | Date: 4/3/13 | Page 2 of 2

Client No. In-House | P.O. No.: E0008

Project : In-House Consulting

Tag: Oakland Army Base

Description or Mark	5/8" Rebar (S9)	5/8" Rebar (S11)			Specified Min.
Heat Number	Unidentified	Unidentified			Requirement
Actual Diameter, Ins.	0.355	0.358			
Actual Area, Sq. Ins.	0.099	0.101			
Yield Point, Lbs.	4601	4595			
Maximum Load, Lbs.	7216	7570			
Yield Strength, Lbs. Per Sq. In.	46500	45500			
Tensile Strength Lbs. Per Sq. In.	73000	75000			
Elongation In 1.4 Inches	0.409	0.401			
Elongation, Percent	29.2	28.6			
Post Test Diameter, Ins.	0.235	0.249			
Reduction of Area, Percent	56	52			
Fracture	PC	PC			
Test Equipment: Instron 4486 S/N 11313J					SOP: 20.080

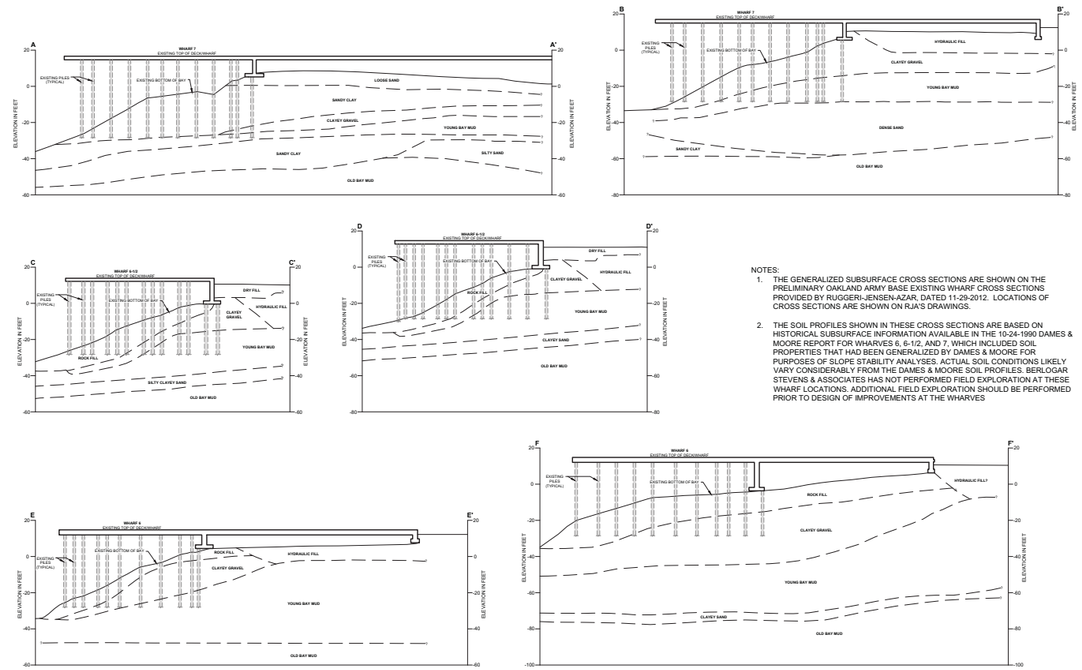
Remarks: Yield @ Halt to Force
Machined and tested in accordance with ASTM A370.

Results Reported To: TEI Consulting

The results presented in this report relate only to the items(s) tested.
This report can be reproduced only in its entirety unless written permission from TEI is obtained
Samples pertaining to this report will be discarded 30 days from the date of this report unless other wise advised

\\Teisys04\rmtlarch\In-House\E384_Consulting_E0008 Oak. Army Base_Pg 2 of 2.doc

Appendix E
Geotechnical Recommendations



NOTES:

1. THE GENERALIZED SUBSURFACE CROSS SECTIONS ARE SHOWN ON THE PRELIMINARY OAKLAND ARMY BASE EXISTING WHARF CROSS SECTIONS PROVIDED BY RUGGERI–JENSEN-AZAR, DATED 11-29-2012. LOCATIONS OF CROSS SECTIONS ARE SHOWN ON RJA'S DRAWINGS.

2. THE SOIL PROFILES SHOWN IN THESE CROSS SECTIONS ARE BASED ON HISTORICAL SUBSURFACE INFORMATION AVAILABLE IN THE 10-24-1990 DAMES & MOORE REPORT FOR WHARVES 6, 6-1/2, AND 7, WHICH INCLUDED SOIL PROPERTIES THAT HAD BEEN GENERALIZED BY DAMES & MOORE FOR PURPOSES OF SLOPE STABILITY ANALYSES. ACTUAL SOIL CONDITIONS LIKELY VARY CONSIDERABLY FROM THE DAMES & MOORE SOIL PROFILES. BERLOGAR STEVENS & ASSOCIATES HAS NOT PERFORMED FIELD EXPLORATION AT THESE WHARF LOCATIONS. ADDITIONAL FIELD EXPLORATION SHOULD BE PERFORMED PRIOR TO DESIGN OF IMPROVEMENTS AT THE WHARVES

OAKLAND GLOBAL
PORT OF OAKLAND · CITY OF OAKLAND
"one vision, one team, one project"

BERLOGAR STEVENS & ASSOCIATES
(925) 484-0220
5307 SUNOL BOULEVARD
PLEASANTON, CA 94566

JOB NO:	3362.217/218
DRAWN BY:	CJC
CHECKED BY:	SI

PROJECT INFO.

WHARFS 6, 6-1/2 AND 7 CROSS SECTIONS A THROUGH F
OAKLAND ARMY BASE
OAKLAND, CA

PRELIMINARY

REV	DATE	COMMENT	CHECKED BY	C.O.D.

DRAWING NO.
PLATE
SHEET 1 OF 1
SCALE: 1" = 20'
DATE: 1-11-13

Jan 11, 2013 – 1:24pm, Hilmi Ercoskun
S:\Projects\362.217\362.218\362 SECTIONS 1-11-13.dwg

OAKLAND ARMY BASE - LPILE PARAMETERS FOR WHARVES 6, 6.5 AND 7 (1/11/2013)

Layers	p-y Curve Soil Model	Effective Unit Weight (pcf)	C (psf)	ϕ (degrees)	k (lb/in^3)	ε_{50}
Dry Fill and Sandy Clay (above water table)	Stiff Clay without Free Water (Reese)	115	1500	0	--	0.007
Sandy Clay (below water table)	Stiff Clay without Free Water (Reese)	53	1500	0	--	0.007
Rock Fill / Clayey Gravel (above water table)	Sand (Reese)	135	0	35	225	--
Rock Fill / Clayey Gravel (below water table)	Sand (Reese)	73	0	35	125	--
Hydraulic Fill / Loose Sand (above water table)	Sand (Reese)	110	0	30	60	--
Hydraulic Fill / Loose Sand (below water table)	Liquefied Sand (Rollins)	48	--	--	--	--
Silty Sand / Clayey Sand (below water table)	Sand (Reese)	53	0	32	60	--
Young Bay Mud (below water table)	Soft Clay (Matlock)	38	400 -12xZ *	0	--	0.01
Dense Sand (below water table)	Sand (Reese)	63	0	35	125	--
Old Bay Mud (below water table)	Stiff Clay without Free Water (Reese)	58	2000	0	--	0.005

PRELIMINARY

* Note: Z = Elevation in feet (e.g. C = 1,000 psf at Elevation = -50 feet)

Eric Lee

From: Troy Swenson
Sent: Tuesday, January 22, 2013 10:24 AM
To: Greg Tolan; Ricardo Rivas
Subject: FW: Oakland Army Base - Wharves 6, 6-1/2 and 7 ARS Curves (BSA Job No. 3362.217)
Attachments: Design Sprectral Acceleration.pdf

Here are the ARS curves.

From: Steve Tsang [mailto:stsang@berlogar.com]
Sent: Tuesday, January 22, 2013 10:08 AM
To: Troy Swenson
Cc: Frank Berlogar; JamesH@archdim.com; tagami@californiagroup.com; Steve Tsang
Subject: FW: Oakland Army Base - Wharves 6, 6-1/2 and 7 ARS Curves (BSA Job No. 3362.217)

Troy, this version is just to correct the table format. The tabulated values are exactly the same.

Thanks,

Steve Tsang

From: Steve Tsang
Sent: Tuesday, January 22, 2013 9:57 AM
To: Troy Swenson
Cc: Frank Berlogar; JamesH@archdim.com; tagami@californiagroup.com
Subject: Oakland Army Base - Wharves 6, 6-1/2 and 7 ARS Curves (BSA Job No. 3362.217)

Troy,

Per your request, attached are the Design Acceleration Response Spectra (ARS) curves for a 10% in 50 year event and a 50% in 50 year event for Wharves 6, 6½ and 7 following Chapter 31 of the 2010 CBC. The Design Spectral Acceleration values are summarized below:

T	Sa (10% in 50 years)
0.0000	0.4505
0.1000	0.8508
0.1688	1.1263
0.2000	1.1263
0.3000	1.1263
0.4000	1.1263
0.5000	1.1263
0.6000	1.1263
0.7000	1.1263
0.8000	1.1263
0.8442	1.1263
0.9000	1.0564
1.0000	0.9507
1.1000	0.8643

T	Sa (50% in 50 years)
0.0000	0.3511
0.1000	0.7657
0.1270	0.8778
0.2000	0.8778
0.3000	0.8778
0.4000	0.8778
0.5000	0.8778
0.6000	0.8778
0.6352	0.8778
0.7000	0.7965
0.8000	0.6970
0.9000	0.6195
1.0000	0.5576
1.1000	0.5069

1.2000	0.7923
1.3000	0.7313
1.4000	0.6791
1.5000	0.6338
1.6000	0.5942
1.7000	0.5593
1.8000	0.5282
1.9000	0.5004
2.0000	0.4754
2.1000	0.4527
2.2000	0.4322
2.3000	0.4134
2.4000	0.3961
2.5000	0.3803
2.6000	0.3657
2.7000	0.3521
2.8000	0.3395
2.9000	0.3278
3.0000	0.3169
3.1000	0.3067
3.2000	0.2971
3.3000	0.2881
3.4000	0.2796
3.5000	0.2716
3.6000	0.2641
3.7000	0.2570
3.8000	0.2502
3.9000	0.2438
4.0000	0.2377

1.2000	0.4646
1.3000	0.4289
1.4000	0.3983
1.5000	0.3717
1.6000	0.3485
1.7000	0.3280
1.8000	0.3098
1.9000	0.2935
2.0000	0.2788
2.1000	0.2655
2.2000	0.2534
2.3000	0.2424
2.4000	0.2323
2.5000	0.2230
2.6000	0.2145
2.7000	0.2065
2.8000	0.1991
2.9000	0.1923
3.0000	0.1859
3.1000	0.1799
3.2000	0.1742
3.3000	0.1690
3.4000	0.1640
3.5000	0.1593
3.6000	0.1549
3.7000	0.1507
3.8000	0.1467
3.9000	0.1430
4.0000	0.1394

Thanks,

Steve Tsang
Berlogar Stevens & Associates
(925) 484-0220

KLEINFELDER
Bright People. Right Solutions.

January 24, 2014
Project No. 134477

Mr. James Heilbronner
Architectural Dimensions
300 Frank H. Ogawa Plaza, Suite 375
Oakland, California 94612

Subject: **Geotechnical Report for Existing Pile Foundation Capacity, Oakland Army Base – Wharves 6, 6½, and 7, (Berths 9, 8, and 7) Oakland, California**

Dear Mr. Heilbronner,

INTRODUCTION

This report presents the results of Kleinfelder's geotechnical investigation, pile load testing program, and engineering evaluation for assessment of existing pile foundation capacity for Wharves 6, 6½, and 7 at the former Oakland Army Base (OAB). The geotechnical services for this project were performed in general accordance with our proposed scope of work letter dated March 18, 2013 to Architectural Dimensions (AD), and is an extension to our on-going geotechnical services for evaluating pile foundations for Wharves 5 through 7 at OAB. These geotechnical services are provided in support of AD, Biggs Cardosa and Associates, Inc. (BCA) structural engineering designers, and the OAB team for planning and engineering design.

The OAB redevelopment program includes some rehabilitation and possible structural retrofitting for on-going use and possible load capacity upgrading of the wharf structures for continuing and future cargo goods transport and shipping operations at the Port of Oakland (Port) outer harbor area. The purpose of our geotechnical services was to evaluate subsurface conditions and deep foundation pile axial capacity for the existing timber and concrete piles supporting the three wharves for use by the structural designer BCA.

Kleinfelder also reviewed BCA's initial structural assessment report for the wharves titled "*Structure Type Selection and Seismic Retrofit Strategy Report for Wharf 6, 6½, and 7 Oakland Army Base*" dated April 15, 2013. The interim results of our geotechnical study program based on an initial desktop 'paper' study using available data were presented in a technical memorandum to the OAB team dated June 14, 2013. Kleinfelder's initial analyses for estimating existing pile foundation axial load capacity were performed based on interpretations of data available at that time for subsurface soil profile conditions, and reference information on pile types and dimensions from the Port documents compiled by AD and BCA.

This report presents superseded information for existing pile foundation capacity, including data from recent field investigations and testing, and recommendations for use in design.

DATA REVIEW, FIELD INVESTIGATION PROGRAM, AND PILE TESTING METHODOLOGY

Based on review of the historic drawing sets and reports for the three subject wharves, no as-built information regarding pile length or geotechnical capacity is available. However the

Copyright 2014 Kleinfelder
1330 Broadway, Suite 1200, Oakland, CA 94612 p | 510.628.9000 f | 510.628.9009

original design allowable pile capacity specified for timber piles (12-inch minimum diameter) was 32 tons, and 40 tons for precast concrete piles (16-inch square). Some pile length information for both timber and concrete piles is shown on the engineering drawings for the adjacent Wharf No. 5 (East Wharf). Design schedules for Wharf No. 5 indicate that pile lengths of 65 to 80 feet for timber piles, and 45 to 65 feet for concrete piles were constructed. Wharf 5 is considered to be of similar construction type, materials and age as Wharves 6, 6½, and 7, so the range of pile lengths between these wharves is also expected to be somewhat similar, but dependent on subsurface conditions including the thickness of Bay Mud deposits and depth to "bearing layers".

The depth to the mudline, or top of the fill or soil surface, varies considerably from the existing ground surface along the northern (land) edges of the wharves to over 30 feet at the southern (water) edges. A key geotechnical assumption used for our preliminary analysis of pile axial compression capacity using the pre-existing data was that the piles were driven in order to be founded (tipped) in significantly denser/stiffer bearing layers underlying the weak and compressible Bay Mud deposits prevalent at the Port. Based on our recent experience providing similar services for the Bay Bridge Toll Plaza structure located about four hundred feet away, and similar studies done for older Caltrans structures in the immediate vicinity near the Port, this was the common practice during the 1930s and 1940s, and in some cases necessitated use of long timber piles up to 100-feet where the Bay Mud layers are very thick.

Based on review of the available subsurface soil profile data from past exploration programs, including Cone Penetration Testing (CPT) and test borings (1950s era up to 2012), several data gap locations were identified. The quantity and quality of available subsurface data from past studies is limited. Descriptions for the subsurface conditions as well as geologic and seismic settings (and seismic hazards) and the available geotechnical data used in our initial study are contained in several previous investigation reports for various sites including the wharves at the OAB and Port areas, and so are not reprinted or attached here in this report in the interest of brevity. A list of reference documents is provided at the end of this report.

Cone Penetration Tests

Additional exploration and in-situ testing for collection of supplemental subsurface data was performed during June 2013 by Kleinfelder using the Cone Penetration Test (CPT) method. Our subcontracted CPT operator was GREGG Drilling and Testing (GREGG) of Martinez, California.

CPT exploration for the current geotechnical investigation program was targeted at nine data gap locations distributed across the three wharves as shown in Kleinfelder's CPT 'workplan' submittal dated May 22, 2013. Temporary coreholes approximately 4-inch diameter were drilled in the existing concrete slabs in order to gain access (for the CPT probe) from wharf deck level and insert metal casing pipe to the mudline. Non-destructive Ground Penetrating Radar (GPR) survey equipment was used at the wharf deck surfaces in order to locate final positions of the concrete coreholes to avoid cutting reinforcing bars embedded in the deck slabs.

Due to the presence of a layer of cobble to small boulder sized rock rip-rap fragments at the mudline surface at the three wharves, it was not possible to advance CPTs at all nine targeted locations. A total of five CPTs were performed, to depths ranging from 100 to 120 feet measured from the top of the wharf deck surfaces. The approximate CPT locations are plotted on the attached Site Plan, Plate 1. The CPT data report prepared by GREGG is attached in Appendix A. The CPT results were used to augment our interpretations of variable soil profiles

Copyright 2014 Kleinfelder

and engineering properties (shear strength, etc.) for the subsurface conditions at the wharves. The narrow CPT holes (approximately 1½ inch diameter) below mudline were grouted upon completion in accordance with our drilling permit from Alameda County, and the wharf deck slab coreholes were patched with rapid-set ready-mix concrete.

High-Strain Dynamic Pile Load Testing

High-strain dynamic pile load testing (PLT) locations were selected in collaboration together with staff from BCA using information from the reference documents as well as results of our reconnaissance visits both above and below the wharf deck slabs. Some wharf areas were inaccessible for PLT type work, or had severely limited access due to various constraints such as fixed or temporary features blocking access at the deck surfaces and/or areas of high activity by current tenants. Other types of constraints which limited the suitability of some piles for PLT testing included structural integrity conditions of existing piles, pile orientation (plumbness), many of the timber piles are covered with thick gunite 'jackets', and related constraints.

PLTs for the geotechnical study program assessment of existing pile capacity were targeted at seven locations (including one 'alternate' location), distributed across the three wharves as shown in Kleinfelder's revised Geotechnical Workplan for Pile Load Testing Activities submittal, dated August 1, 2013. In order to perform the testing, demolition and temporary removal of wharf deck sections, approximately 6 feet by 6 feet square, were required to expose pile caps at each test location. The demolition and reconstruction activities as well as providing the crane to lift and transport the PLT equipment around the wharf sites was performed by the OAB Contractor - Alarcon Bohm hired by California Capital and Investment Group (CCIG). Additional details regarding the demolition and reconstruction were presented in the Structural Workplan by BCA dated September 9, 2013 prepared in support of the PLT program. Kleinfelder's subcontractors included GRL Engineers Inc. (GRL) to provide the PLT equipment, load test instrumentation, and data interpretation of test results.

The PLT program used the dynamic load testing method (reference ASTM D4945 - *Standard Test Method for High-Strain Dynamic Testing of Piles*) at six pile locations. The PLT methods and reference procedures for dynamic load testing using GRL's free-fall 8-ton drop hammer system (a.k.a. APPLE system), including equipment such as ram and frame, load cell transducer etc. were referenced by the document tilted "*Detailed Description of APPLE/PDA Testing*" by GRL Engineers dated 2011 attached to Kleinfelder's PLT Workplan.

The locations of the six load tested piles were distributed across the three wharves, and included three concrete piles and three timber piles. Since all six PLT tests were completed successfully, the team decided to forego testing at the seventh (alternate) PLT location. The approximate locations of the six PLTs are plotted on the attached Site Plan, Plate 1.

Pile Dynamic Analysis (PDA) measurements were performed by GRL on each of the 6 test piles. Dynamic measurements recorded by the PDA consisted of attaching strain transducers and accelerometers near the top of the pile and converting the measurements to force and velocity time histories during each drop hammer blow. A load cell was also used to record the impact force of the drop hammer. This data was used to evaluate drop hammer system performance, measure driving stresses and pile structural integrity, pile length and estimated ultimate axial compression pile capacities.

Case Pile Wave Analysis Program (CAPWAP) analyses were performed by GRL to compute soil resistance forces and approximate distribution along the piles based on force and velocity

Copyright 2014 Kleinfelder

data recorded during field dynamic pile testing. Analyses were performed for the drop hammer blows on each of the 6 test piles. The CAPWAP analyses indicate that the piles derive their total bearing capacity in a combination of shaft skin friction and end bearing. The PLT and CAPWAP results are summarized on Table 1.

TABLE 1 – SUMMARY OF TEST PILE DATA

WHARF	TEST PILE NUMBER	PILE TYPE	APPROXIMATE PILE LENGTH BELOW TOP OF DECK-FT	APPROXIMATE PILE TIP ELEVATION -FT	ULTIMATE AXIAL COMPRESSION CAPACITY-KIPS
7	7A	18-inch-square concrete	77	-62	595
7	7B	13-inch-diameter timber	63	-48	160
6½	8A	16-inch-square concrete	79	-64	410
6½	8B	16-inch-square concrete	73	-58	370
6	9A1	13-inch-diameter timber	52	-37	225
6	9A	13-inch-diameter timber, with 1.5-inch gunite jacket	35	-20	360

For the longer test piles that appear to have been driven deeper to the dense sands or stiff sandy clays below the Bay Mud deposits, the end bearing component of the total capacity is significantly less than the skin friction component. The dynamic pile load test results indicate that the percentage of the total capacity attributed to skin friction generally ranged from about 65 to 83 percent. However, Test Pile 9A at Wharf 6 located at the land side of the wharf is significantly shorter than the other test piles, and may have been driven to hard driving refusal in a very dense sand layer. This pile has a much higher end bearing component, and the percentage of the total capacity attributed to skin friction is only about 25 percent.

It should also be noted, as discussed in the GRL report attached in Appendix B, that the PDA and CAPWAP estimates of total capacity should be considered lower bounds for most of the piles that did not move significantly during the drop hammer blows (i.e. piles that did not move more than on the order of approximately 0.2 inches per blow during testing).

Sonic Pulse Echo Testing (a.k.a. pile integrity testing 'PIT' method) was also done on two exposed test piles for purposes of attempting assessment of pile lengths. However, successful PIT results were not achieved due to various factors including the relatively long pile lengths, and pile head conditions (with concrete pile caps still attached).

CONCLUSIONS AND RECOMMENDATIONS

The results of the CPT investigation and PLT dynamic pile load testing program were used to verify and adjust the assumptions used in our preliminary analyses, and to re-evaluate the pile foundation system geometry and estimation of ultimate axial capacities in more detail. The additional CPT data performed by Kleinfelder was used to supplement the available subsurface data and to provide additional information regarding soil strata thicknesses and shear strength profiles. The PLT data and CAPWAP analyses also provided an estimate of pile length, soil

Copyright 2014 Kleinfelder

resistance forces and approximate pile skin friction resistance distributions along the length of the test piles, as well as the end bearing/pile tip resistance component of the total ultimate capacities. This data was then used to elaborate on our previous ultimate pile capacity calculations and verify the previous soil strength assumptions used in our preliminary analyses.

Ultimate axial compression soil capacity for the modeled subsurface profiles for the three wharves was analyzed for both timber and concrete piles using the software program APILE Plus 5.0 for Windows by Ensoft (2007). This analysis was then compared with PLT data and the estimated skin friction resistance distribution and end bearing components. The results of this evaluation indicate that the measured ultimate pile axial compression capacities measured during the PLT dynamic testing program correlate closely with the calculated ultimate axial compression soil capacities from our static geotechnical analyses.

Based on the BCA Structure Type Selection and Seismic Retrofit Strategy Report, the design dead load (self weight) of the wharf deck is approximately 150 pounds per square foot (psf), and the live loads range from 400 psf for the general wharf area to about 600 psf at the wharf rail track areas (Cooper E60 loading). The pile foundation grid spacing is typically about 10 feet center-to-center, resulting in a tributary wharf deck area of about 100 square feet per pile. The original design allowable pile capacity specified for timber piles (12-inch minimum diameter) was 32 tons, and 40 tons for precast concrete piles (16-inch square).

As discussed above, a key geotechnical assumption used for our preliminary analysis of pile axial compression capacity using the pre-existing data was that the piles were driven in order to be founded (tipped) in significantly denser/stiffer bearing layers underlying the weak and compressible Bay Mud soils prevalent at the Port. The results of our CPT investigation and PLT program generally verify these assumptions.

There is not any definitive as-built information regarding the actual length of the existing pile foundations. However, based on our geotechnical evaluation, we conclude that the existing pile foundation system is capable of supporting the originally designed structural (dead plus live) wharf deck loads.

Consideration can be given to increasing the individual pile axial compression capacities (for static loading case) beyond the original design parameters if required to support additional live loads anticipated for reuse of the three wharf structures. The results of the PLT program indicate that the existing 12 to 13-inch diameter timber piles have an ultimate axial compression capacity ranging from about 160 kips to 360 kips, and the 16-inch square precast concrete piles range from about 370 to 410 kips. Recommended maximum <u>allowable</u> axial compression capacities of existing piles for static load case design are as follows:

- Timber Piles (12 to 13-inch diam.) 90 kips / 45 tons
- Concrete Piles (16-inch square) 180 kips / 90 tons

These recommendations are limited to the geotechnical/soil capacities for the piles, and so pile structural capacities should be assessed by BCA. The OAB team should also understand that the pile geotechnical capacities recommended above are based on evaluation of a very limited number of test/data points, and there is some level of risk associated with applying these capacity values for 9,000+ piles providing foundation support for the three wharves. Capacity of individual piles is dependent upon subsurface profile variability and shear strength of soil layers, pile dimensions (section sizes and embedment lengths), etc. all of which vary considerably

Copyright 2014 Kleinfelder

based on actual conditions. The OAB team should consider whether additional investigation and PLT (dynamic load testing method) on existing piles is desired to reduce the level of uncertainty and risk; however it is expected to be impractical to conduct PLTs at a high frequency on a large percentage of the existing piles.

The pile capacity evaluation methodology also assumes that there will not be any future additional site grading fill placed to raise existing site grades, and there will not be any additional site settlements as a result of consolidation settlement of the Bay Mud deposits and overlying fill soils. Therefore we do not anticipate that the pile foundations need to be designed for drag loads (negative skin friction) or downdrag settlement for the static load cases. Evaluation of seismic hazard related effects, including slope stability, liquefaction, and lateral spreading were not included as part of the geotechnical analysis scope for the three wharf structures.

Several photographs of the field programs for CPT investigation and PLT work are attached in Appendix C.

ADDITIONAL SERVICES

We understand that wharf retrofit strategy and design is not yet complete, and final planning and performance criteria are being refined by the OAB team. As the engineering consultant that performed the geotechnical evaluation for preliminary design of this project, Kleinfelder should continue to be retained for the final design phase to confirm that the recommendations of this report are properly incorporated in the final engineering documents, and properly implemented during construction. This may avoid misinterpretation of the information by other parties and will allow us to review and modify our recommendations if variations in the soil conditions or foundation system geometry are encountered. As a minimum Kleinfelder should be retained to provide the following continuing services for the project for the remaining engineering design phases and during construction for the three wharves:

- For advancement through final design of any replacement piles and new (added) retrofit piles, including driven and/or cast in steel shell (CISS) piles, geotechnical analyses and input for axial and lateral capacity for varying soil profiles/locations;
- Seismic analyses considering updated 2013 CBC code and to confirm Site Class (soil profile types), as well as seismic lateral earth pressure increment for bulkhead or other walls retaining soil;
- Seismic hazard analyses for liquefaction, lateral spreading, and seismic slope stability and impacts;
- Review the final project plans and specifications, including any revisions or modifications;
- Pile foundation construction site visits including geotechnical observation during installation/construction of any indicator and/or test pile programs performed in advance of production pile installation. Kleinfelder geotechnical staff personnel should be on-site full-time during the production pile driving operations for monitoring (for conformance purposes) as required per the specifications;
- Observe and evaluate the site earthwork operations to confirm subgrade soils are suitable for construction of foundations, subgrade preparation, and placement and compaction of engineered fill or backfill per the project specifications;
- Respond to Requests for Information (RFIs) - review and respond to the Contractor's RFIs that are geotechnical-related for foundations and earthwork;
- As-needed meeting attendance and participation, as scheduled and requested by the OAB project team. Meeting attendance could potentially include the timeframe during

Copyright 2014 Kleinfelder

final engineering, pre-construction conference prior to commencement of construction, pre-installation meeting for pile foundations, general status or submittal review meetings, or other meetings as necessary during sitework construction.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's geotechnical engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during foundation and earthwork construction. Furthermore, the contractor should be prepared to handle subsurface contamination conditions that may be encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers.

LIMITATIONS

The recommendations contained in this report are based on our review of data and previous reports and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction which differ from those described herein, we should be notified immediately in order that a review may be made and any supplemental recommendations provided. If the scope of the proposed construction changes from that described in this report, our recommendations should also be reviewed. This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited amount of data. Kleinfelder makes no other representation, guarantee or warranty, expressed or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

This report may be used only by AD and the OAB design team consultants and partners for this project, and only for the purposes stated within a reasonable time from its issuance, but in no event later than two years from the date of the report. Land or facility use, on and off-site conditions, regulations, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder

Copyright 2014 Kleinfelder

from any liability resulting from the use of this report by any unauthorized party and client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

CLOSURE

We appreciate the opportunity to continue providing geotechnical services for the OAB project. We look forward to continued participation with the OAB team in support of the remaining engineering design services and construction at the wharves. If you have any questions regarding this geotechnical deliverable, please contact Brian O'Neill or Robert Fosse of our Oakland office at (510) 628-9000.

Sincerely,

KLEINFELDER

REGISTERED PROFESSIONAL ENGINEER ROBERT FOSSE JR. No. GE 2380 Exp. 3/31/14 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER BRIAN O'NEILL GE 2516 Exp. 6-30-15 GEOTECHNICAL STATE OF CALIFORNIA

Robert Fosse, P.E., G.E.
Principal Geotechnical Engineer

Brian O'Neill, P.E., G.E.
Principal Engineer / Project Manager

Attachments:

List of Reference Documents
Plate 1 – Site Plan
Appendix A – Cone Penetration Test (CPT) Report by GREGG
Appendix B – Dynamic Pile Load Test (PLT) Report by GRL
Appendix C – Photographs from CPT and PLT Field Programs

cc: Biggs-Cardosa Associates – Mr. Troy Swenson, P.E., S.E.

List of Reference Documents:

ASTM D4945 - *Standard Test Method for High-Strain Dynamic Testing of Piles*

Berlogar Stevens and Associates (2013), "65% Geotechnical Report, Oakland Army Base – Horizontal Development, Maritime Street and Highway 880, Oakland, California," June 20, 2013.

Biggs Cardosa Associates Inc. (2013), "Structure Type Selection and Seismic Retrofit Strategy Report for Wharf 6, 6 ½, and 7, Oakland Army Base," April 15, 2013.

Biggs Cardosa Associates Inc. (2013), "Structural Details and Calculations for Oakland Army Base Pile Load Testing Structural Work Plan," September 9, 2013.

Dames and Moore (1990), "Final Report, Geotechnical Engineering Services, Structural Repairs of Wharves 6, 6½, and 7, Oakland Army Base," October 24, 1990.

GRL Engineers. *Detailed Description of APPLE/PDA Testing,* 2011.

Jacobs Engineering Group Inc. (2010), "Oakland Army Base Redevelopment, Preliminary Condition Assessment & Evaluation of Army Wharves 6 ½ and 7," October 5, 2010.

Kleinfelder (2013), "Oakland Army Base – Berth 7 (Wharf 7), Berth 8 (Wharf 6½), and Berth 9 (Wharf 6), Pile Foundation Investigation, Geotechnical Workplan for Pile Load Testing Activities," August 1, 2013.

Kleinfelder (2013), "Interim Geotechnical Recommendations, Pile Capacity Analysis of Existing Timber and Concrete Piles at Wharf Nos. 6, 6½, 7, Oakland Army Base Project, Port of Oakland Area, Oakland, California," June 14, 2013.

Kleinfelder (2013), "Oakland Army Base – Wharves 6, 6½, and 7 Geotechnical Study, Investigation Workplan for CPT Exploration Activities," May 22, 2013.

Kleinfelder (2013), "Oakland Army Base – Wharves 6, 6½, and 7 Geotechnical Study, Workplan for Start-Up Activities," April 30, 2013.

Moffat and Nichol Engineers. Oakland Army Base Wharf 6, 6½, and 7 Condition Study, undated

Copyright 2014 Kleinfelder

KLEINFELDER 1330 Broadway, Suite 1200, Oakland, CA 94612 p | 510.628.9000 f | 510.628.9009

PLATES

CPT-3
9A.1
8B
WHARF 6.5 / BERTH 8
CPT-6
9A
WHARF 6 / BERTH 9
CPT-4
CPT-5
8A
WHARF 7 / BERTH 7
7B
7A
CPT-7
N
LEGEND
CONE PENETRATION TEST (CPT) LOCATIONS
PILE LOAD TESTING (PLT) LOCATIONS
250 125 0 250
APPROXIMATE SCALE (feet)
REFERENCE:
"HIRES AERIAL.PDF" PROVIDED BY ARCHITECTURAL DIMENSIONS, UNDATED.
KLEINFELDER
Bright People. Right Solutions.
www.kleinfelder.com
PROJECT NO. 134477
DRAWN: JAN 2014
DRAWN BY: AG
CHECKED BY: BO
FILE NAME: 134477_SitePlan.dwg
SITE PLAN
WHARVES 6, 6.5 & 7
OAKLAND ARMY BASE
OAKLAND, CALIFORNIA
PLATE
1
OAKLAND, CA

APPENDIX A

CONE PENETRATION TEST (CPT) REPORT BY GREGG

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

June 17, 2013

Kleinfelder
Attn: Brian O'Neill

Subject: CPT Site Investigation
Port of Oakland Wharfs 6-7
Oakland, California
GREGG Project Number: 13-109MA

Dear Mr. O'Neill:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☐
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☒
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

950 Howe Rd • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-2	6/12/13	37	-	-	-
CPT-3	6/12/13	100	-	-	-
CPT-4	6/13/13	110	-	-	-
CPT-5	6/13/13	110	-	-	-
CPT-6	6/13/13	105	-	-	-
CPT-7	6/14/13	120	-	56	-

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., “Cone Penetration Testing in Geotechnical Practice” E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., “Soil Classification using the Cone Penetration Test”, Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., “NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization”, available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, “Geo-Environmental Application of Penetration Testing”, Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemees, “Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies”, Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, “Reliability of Soil Gas Sampling and Characterization Techniques”, International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, “Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT”, 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, “Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers” BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

950 Howe Rd • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT*. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

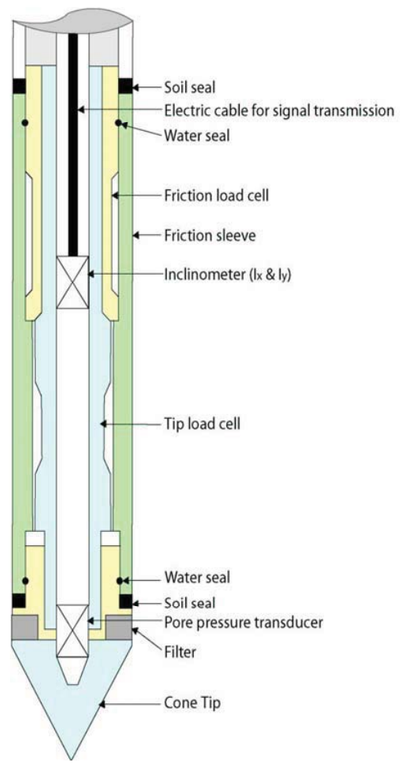


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

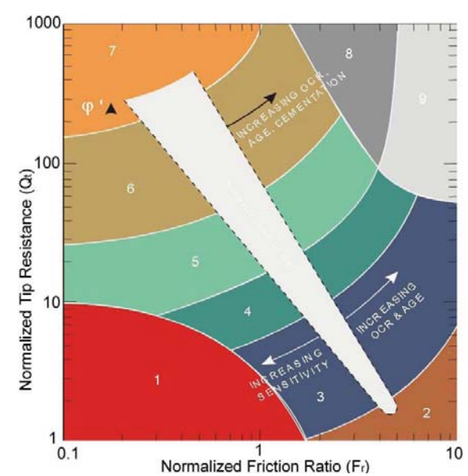
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on the field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



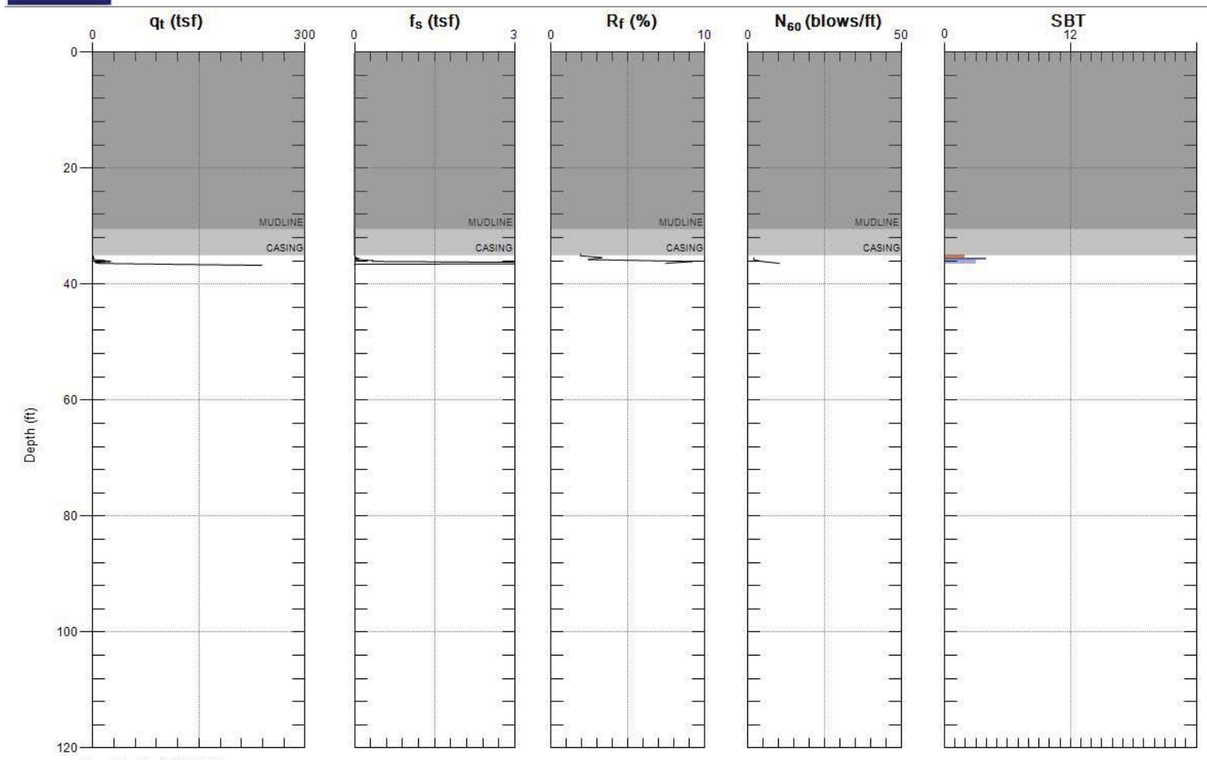
(After Robertson, 1990)

Zone	Normailzed Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

Figure SBTn

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-2
Engineer: B.O'NEILL
Date: 6/12/2013 12:32

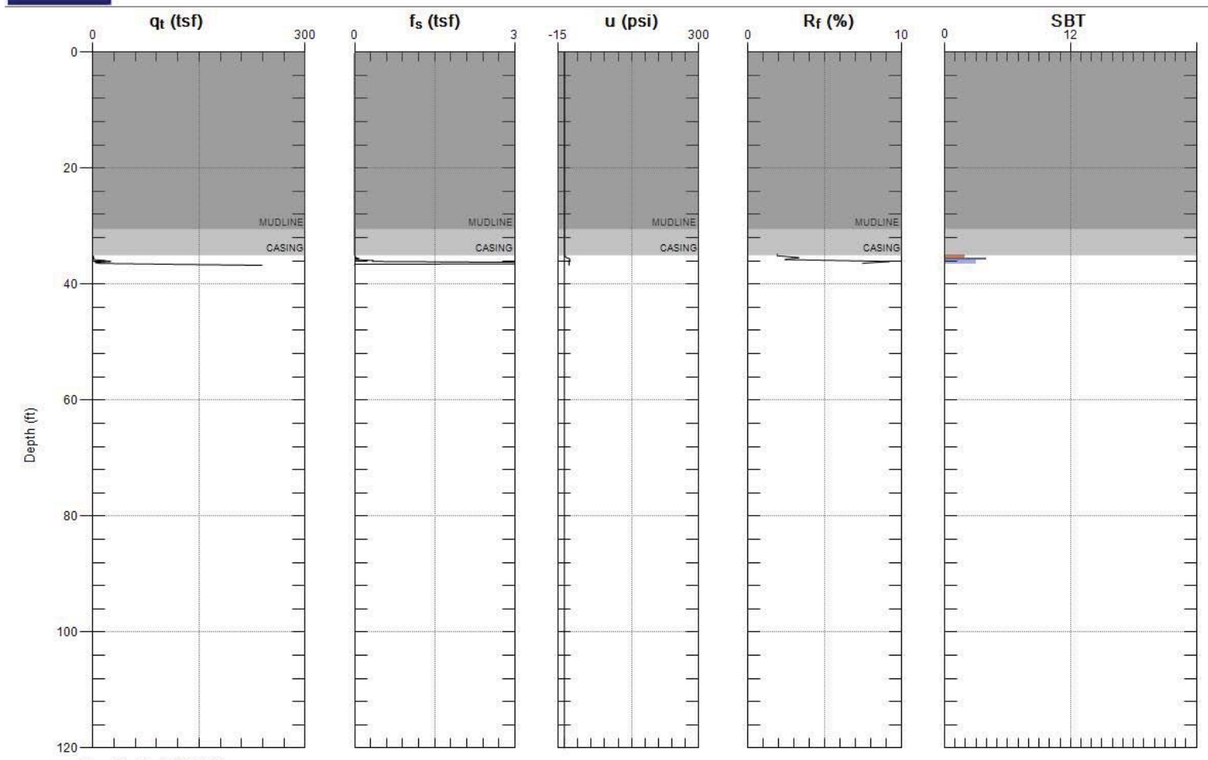


Max. Depth: 36.745 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-2
Engineer: B.O'NEILL
Date: 6/12/2013 12:32



Max. Depth: 36.745 (ft)
Avg. Interval: 0.328 (ft)

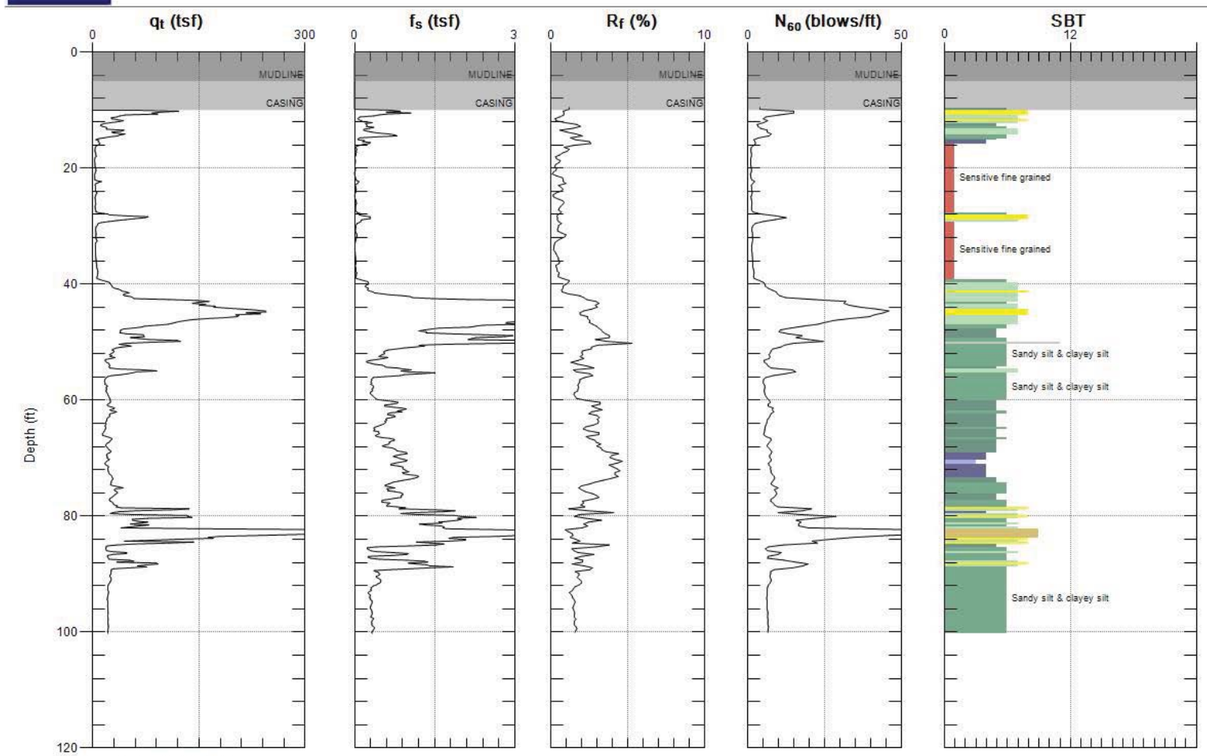
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-3

Engineer: B.O'NEILL
Date: 6/12/2013 02:51



Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

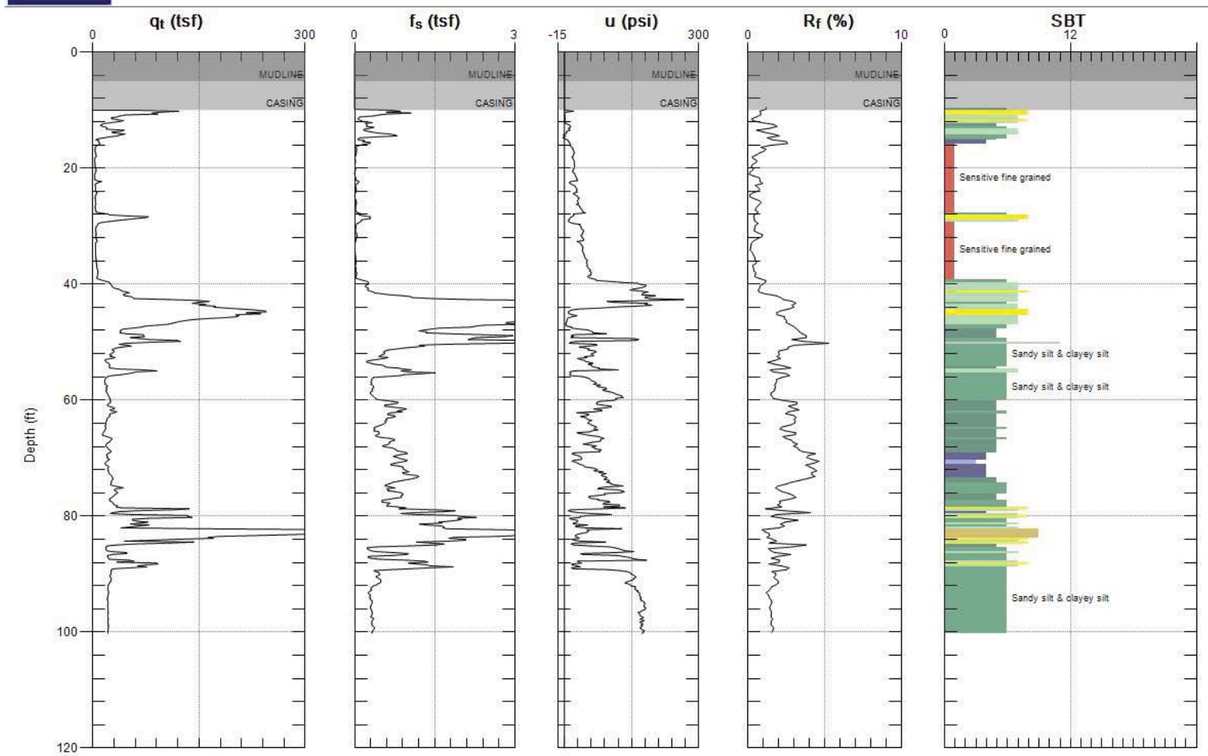
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-3

Engineer: B.O'NEILL
Date: 6/12/2013 02:51



Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

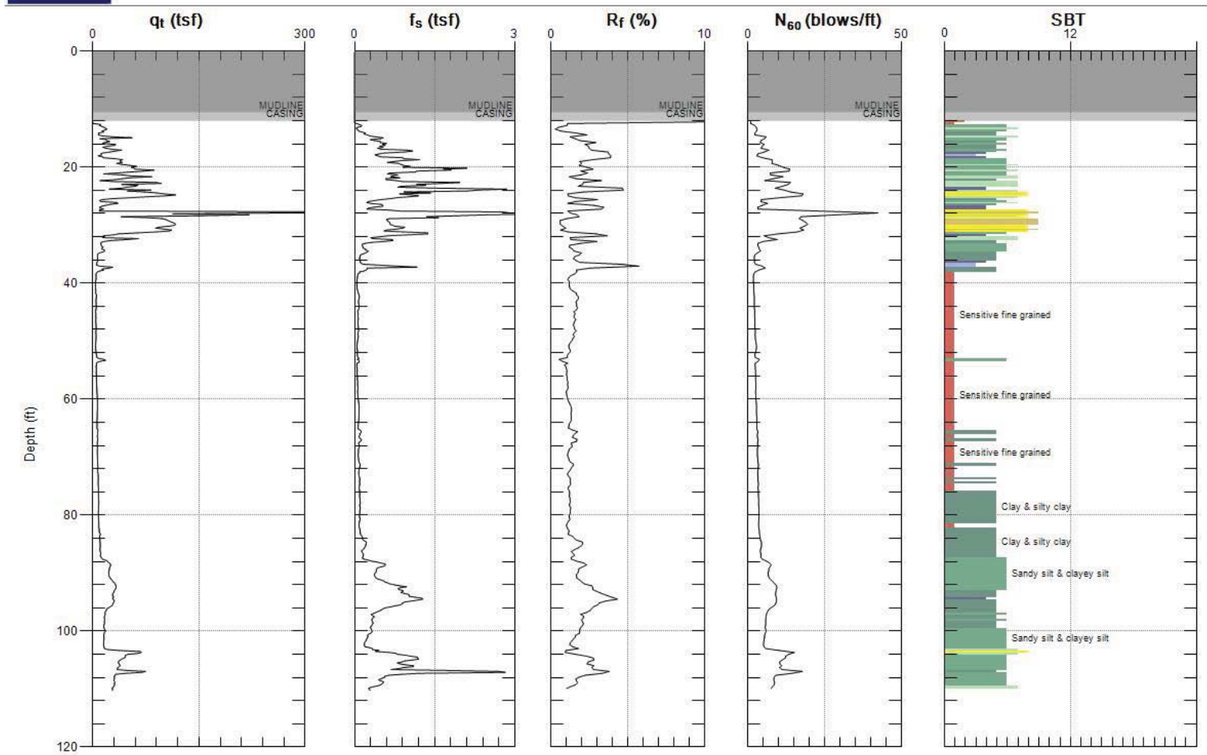
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-4

Engineer: B.O'NEILL
Date: 6/13/2013 08:45



Max. Depth: 110.236 (ft)
Avg. Interval: 0.328 (ft)

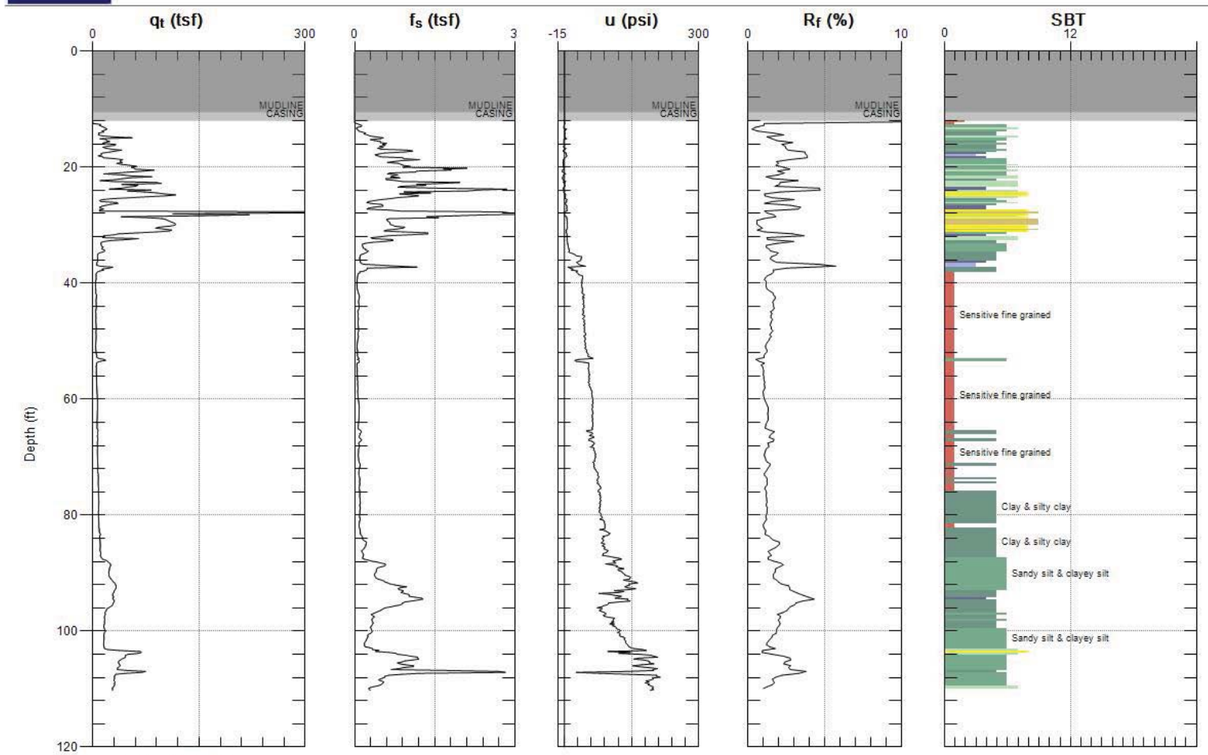
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-4

Engineer: B.O'NEILL
Date: 6/13/2013 08:45



Max. Depth: 110.236 (ft)
Avg. Interval: 0.328 (ft)

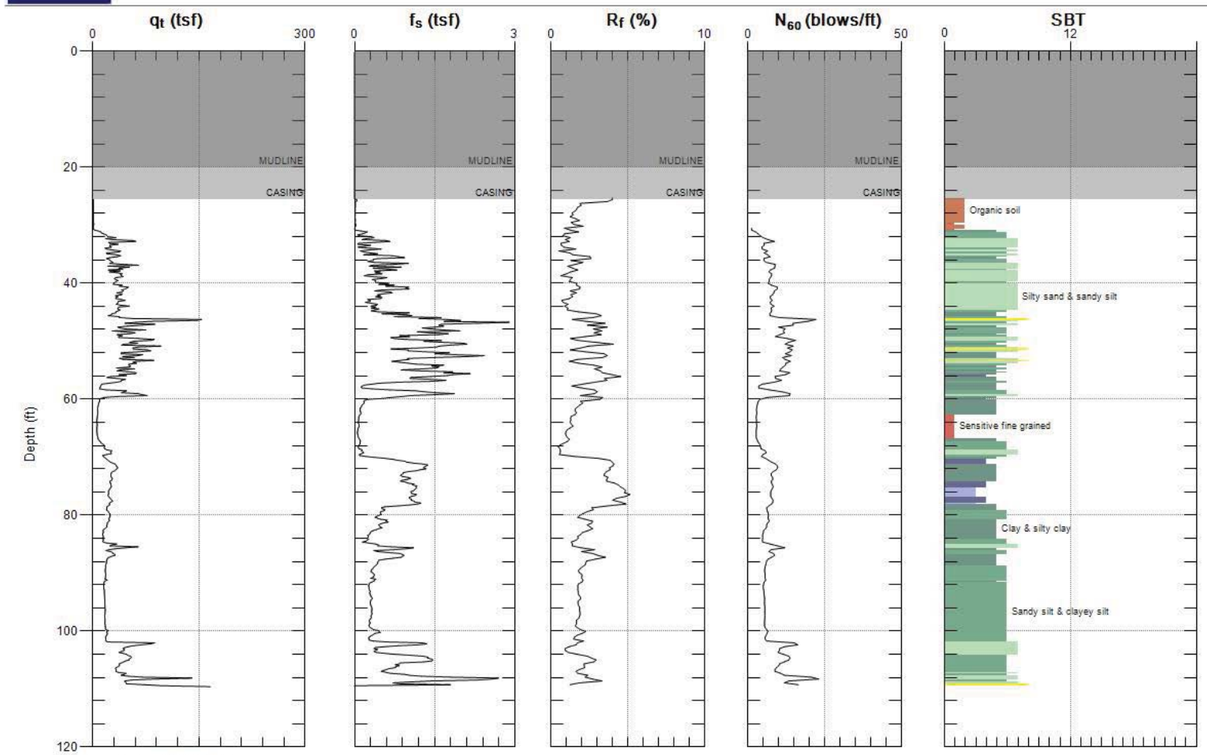
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-5

Engineer: B.O'NEILL
Date: 6/13/2013 12:11



Max. Depth: 109.580 (ft)
Avg. Interval: 0.328 (ft)

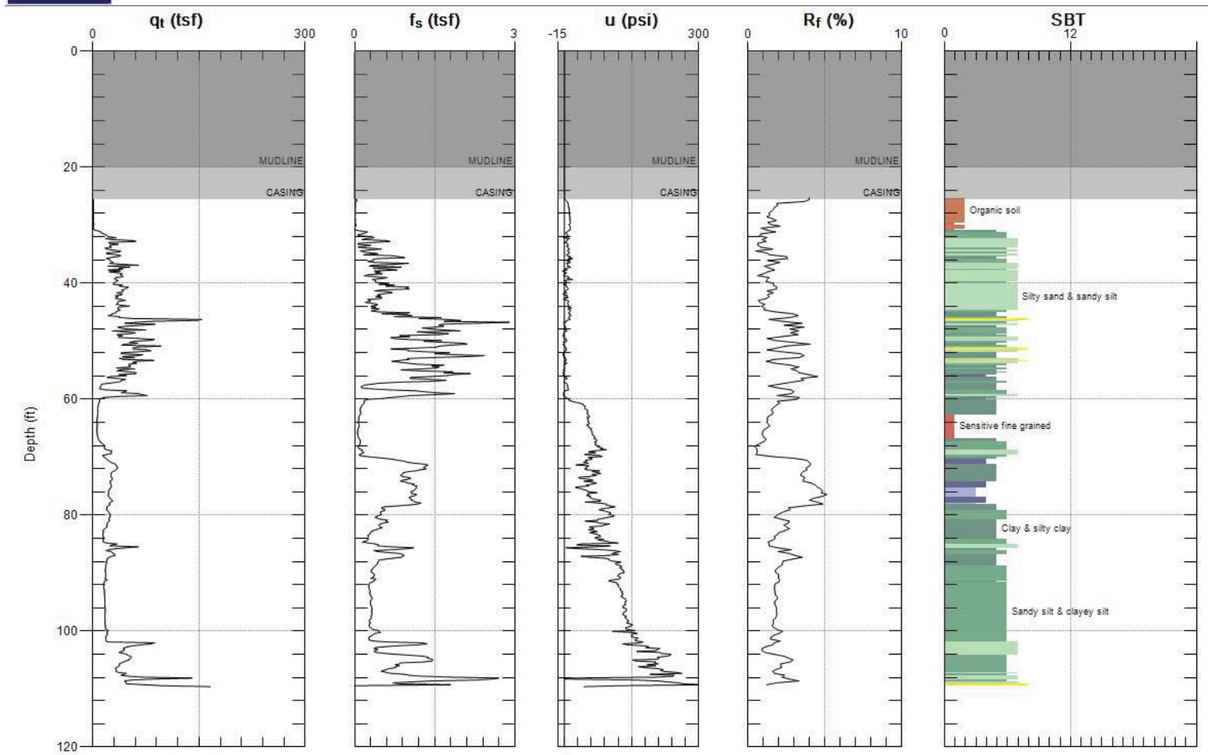
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-5

Engineer: B.O'NEILL
Date: 6/13/2013 12:11



Max. Depth: 109.580 (ft)
Avg. Interval: 0.328 (ft)

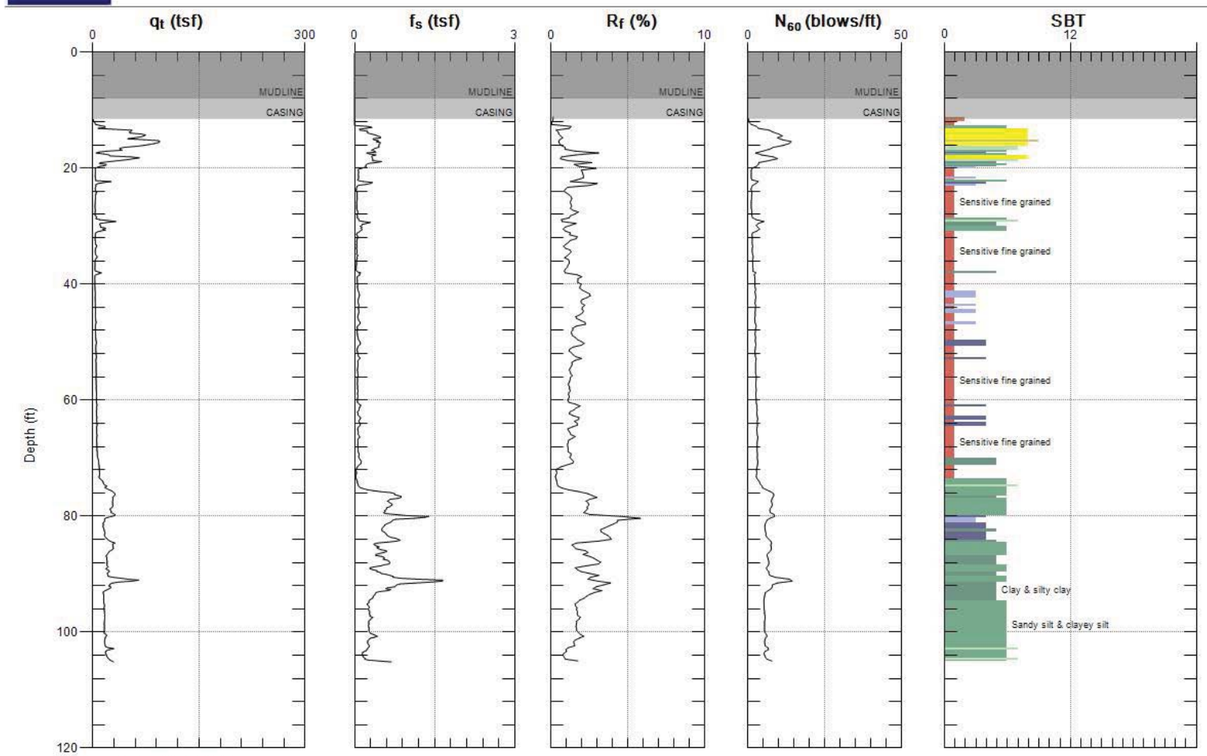
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-6

Engineer: B.O'NEILL
Date: 6/13/2013 04:17



Max. Depth: 105.151 (ft)
Avg. Interval: 0.328 (ft)

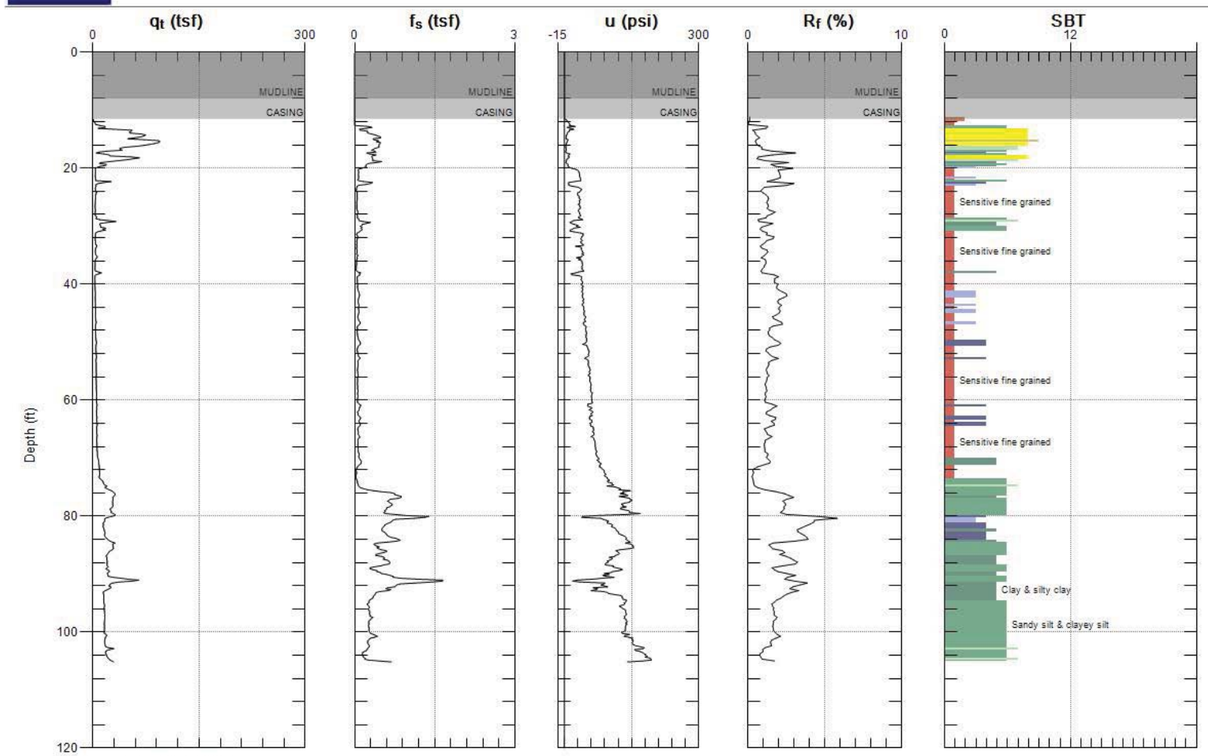
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-6

Engineer: B.O'NEILL
Date: 6/13/2013 04:17



Max. Depth: 105.151 (ft)
Avg. Interval: 0.328 (ft)

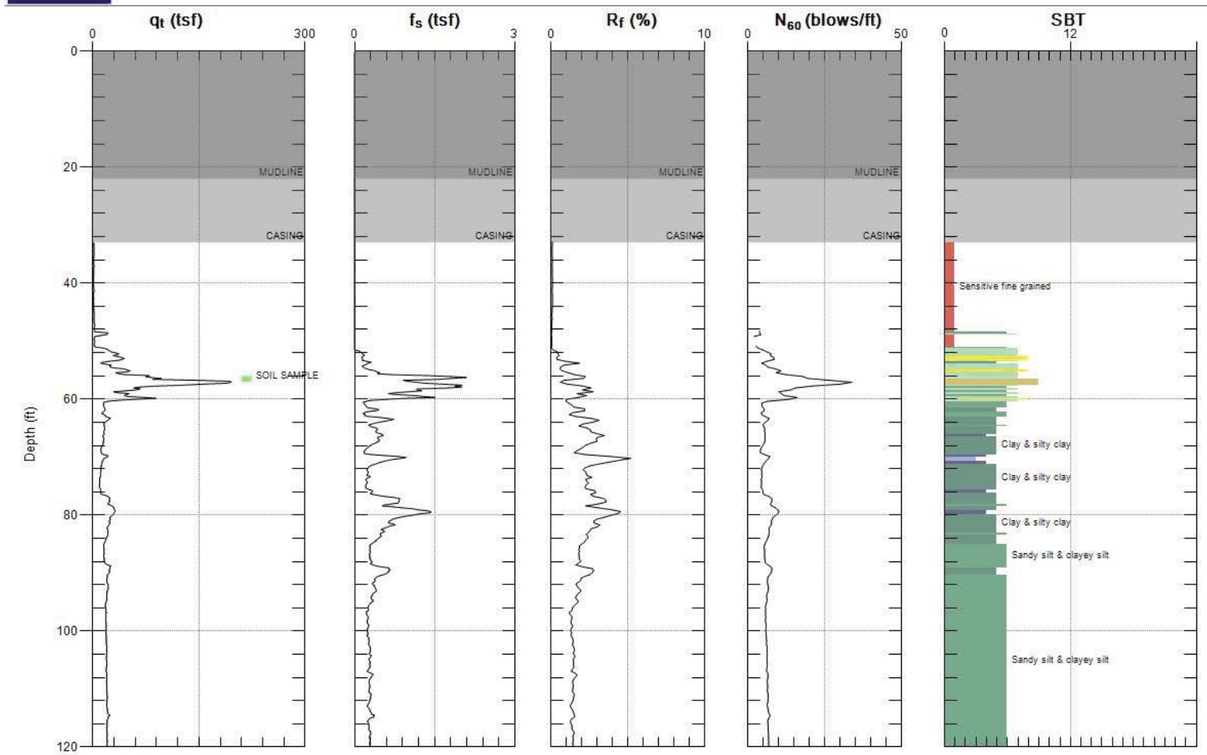
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-7

Engineer: B.O'NEILL
Date: 6/14/2013 10:55



Max. Depth: 120.243 (ft)
Avg. Interval: 0.328 (ft)

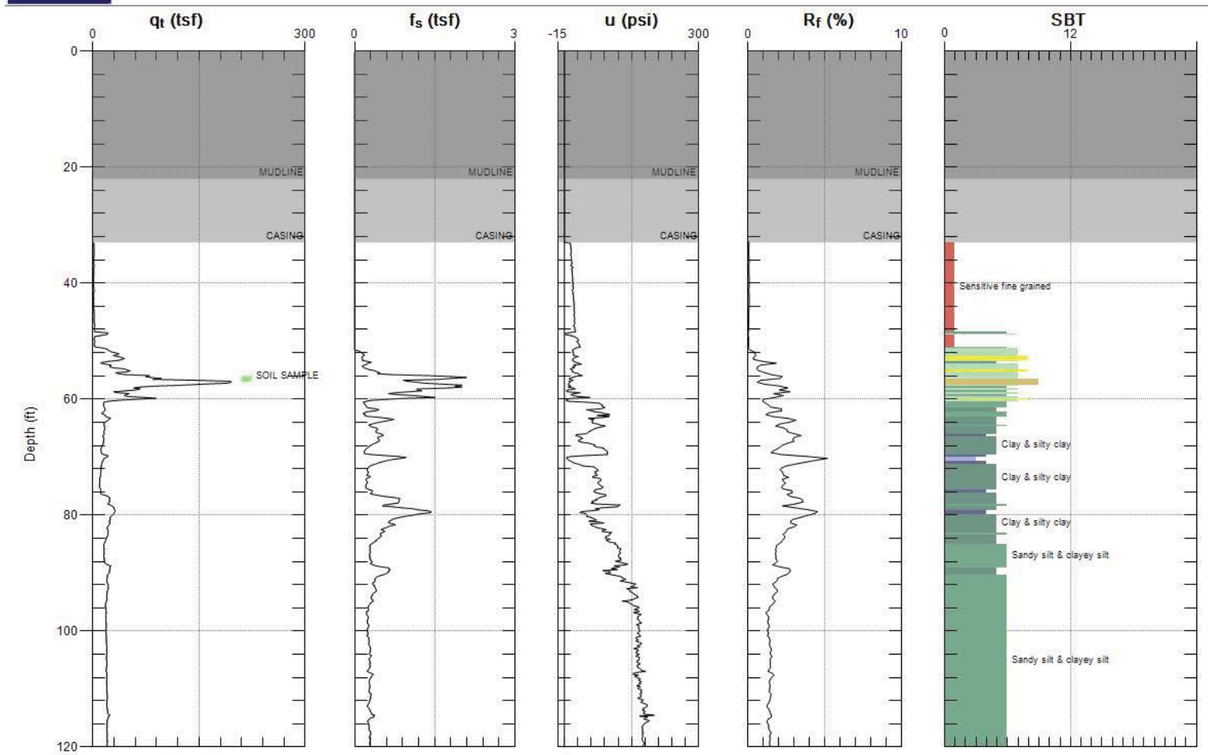
SBT: Soil Behavior Type (Robertson 1990)

GREGG

KLEINFELDER

Site: PORT OF OAKLAND
Sounding: CPT-7

Engineer: B.O'NEILL
Date: 6/14/2013 10:55



Max. Depth: 120.243 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B

DYNAMIC PILE LOAD TEST (PLT) REPORT BY GRL

GRL engineers, inc.

Dynamic Measurements and Analyses

Job No. 138134-1

Report on: Dynamic Pile Measurements
Oakland Army Base – Wharves
Oakland, California

Prepared for Kleinfelder

By Camilo Alvarez, MSCE, P.E. and
Anna M. Klesney, MSCE, E.I.T.

November 12, 2013

www.GRLengineers.com info@GRLengineers.com

November 12, 2013

Mr. Brian O'Neill, PE, GE
Kleinfelder
Principal Geotechnical Engineer
1330 Broadway, Suite 1200
Oakland, CA 94612

Re: Dynamic Pile Measurements - Oakland Army Base - Wharves
Oakland, CA
GRL Job No. 138134-1

Dear Mr. O'Neill:

This report presents results obtained from high-strain dynamic measurements gathered between October 30 and November 1, 2013 on six concrete and/or timber piles at the Oakland Army Base, Wharves Project in Oakland, California. High-strain dynamic pile measurements were collected and processed with a Pile Driving Analyzer® (PDA). The results include evaluations of impact pile stresses, pile integrity, hammer performance, and soil resistance activated at the time of testing. Our high-strain testing and analysis procedures are based on the Case Method and are described in Appendix A of this report. Complete CAPWAP results are included in Appendix B.

Foundation Piles

The tested piles were identified as Piles 7A, 7B, 8A, 8B, 9A and 9A1 with unknown pile length. Pile 7A was reported to be a 18x18 square concrete pile, Piles 8A and 8B were reported to be 16x16 square concrete piles, and Piles 7B, 9A and 9A1 were reported to be approximately 13-inch diameter timber piles. Pile 9A had a surrounding Shotcrete outer of close to 1.5 inches thick.

GRL Dynamic Load Testing System

The impact hammer utilized for the high-strain dynamic testing was the GRL APPLE V drop hammer system. The APPLE V ram mechanism is guided by a boxed lead configuration having approximate dimensions of 8 ft x 8 ft x 20 ft (lwh). Various ram weights can be utilized with the APPLE V. For this project, one 8 ton weight was selected. The ram stroke is fully adjustable up to a maximum drop height of greater than 6 ft. The hammer drop system utilized a hydraulic clamp device to release the ram in a free fall manner. One new sheet of ¾ inch-thick plywood pile cushion was utilized to protect each pile top from high impact compressive stresses. The plywood also helps to widen the hammer impact pulse which helps limit the pile tension stresses.

California Office: 516 Crane Boulevard • Los Angeles, CA 90065 USA • 323.441.0965 • fax 323.441.0975 • GRL-CA@GRLengineers.com
Central Office: 30725 Aurora Road • Cleveland, OH 44139 USA • 216.831.6131 • fax 216.831.0916 • www.GRLengineers.com

Colorado 303.666.6127 | Illinois 847.221.2750 | Florida 407.826.9539 | Louisiana 985.640.7961 | North Carolina 704.593.0992 | Ohio 216.831.6131 | Pennsylvania 610.459.0278

Soils

The reader is referred to the appropriate Geotechnical/foundations report(s) for details regarding the subsurface conditions at this site and the required pile capacities for this project. Information regarding elevations, pile placement, penetrations, and other field details may be found in the project field reports.

Instrumentation

A load cell was used to obtain dynamic strain measurements for all test piles except for pile 7A. Strain and acceleration measurements were also performed 2 to 3 feet below the pile tops for all piles with the exception of Pile 8B. Analog signals from the gages were conditioned, digitized, stored and processed with a Model PAX, Pile Driving Analyzer™ (PDA). Force and velocity records were viewed on the PDA's graphic screen to evaluate data quality, pile integrity and aspects of soil resistance. Stored dynamic data and the PDA field results are the basis of the analyses presented in this report. A schematic of our equipment setup is shown in Appendix A.

FIELD RESULTS

PDA results from the high-strain dynamic tests are presented in Tables 1 and 2. The table quantities were taken from the same hammer impact utilized in the CAPWAP analysis. The analyzed ram drop heights ranged from 12 to 48 inches for the tested piles. When piles (similar to these diameters of 13 to 18 inches) displace more than 0.2 inches for one drop, the soil resistance is considered to be ultimate. Lower sets will most possibly imply that the load is a mobilized load and not an ultimate load.

For Pile 9A1, the load tests for 12 inches generated a displacement close to 0.1 inches and when the drop was increased to 24 inches the permanent set exceeded 0.5 inches. Due to the excessive displacement the previous 12 inch drop was selected for analysis.

Table 1 also presents the results for the maximum CAPWAP computed compression and tension stresses, the hammer transferred energy and the CAPWAP computed ultimate pile capacity.

CAPWAP ANALYSES

Background

CAPWAP (CAse Pile Wave Analysis Program) is a rigorous numerical analysis procedure which uses the measured force and velocity data to solve for soil resistance parameters and distribution. This method combines the wave equation pile and soil models with the Case Method field measurements. The process iteratively determines the best match soil model unknowns by signal matching. The solution includes the estimated static bearing capacity and also the shaft friction, end bearing, damping factors and other dynamic soil properties. Ultimately a simulated static load set curve is developed using the obtained static parameters.

CAPWAP analysis was performed for a selected hammer blow for each test pile. The results are summarized in Table 2 and complete CAPWAP analysis results are presented in Appendix B.

DISCUSSION OF RESULTS

Hammer Energy Transfer

Calculated energy transfer (EMX) to the piles is a function of the ram release mechanism, ram drop height, plywood pile cushion stiffness and pile impedance. The transferred energy to the gage location, below pile top, ranged between 10 and 36 kip-ft for all impacts that were analyzed with CAPWAP.

Pile Stresses

The maximum CAPWAP computed compressive stress values ranged between 1.5 and 3.6 ksi, with values exceeding 3.0 ksi corresponding to the square concrete piles. Maximum computed tension stress levels were at or less than 0.4 ksi.

Pile Integrity- Length

PDA measurements were monitored during the high-strain dynamic impacts for indications of notable pile impedance changes. A clear impedance decrease will be considered to be the end of the pile and based on that response the length will be determined. If the pile has an integrity problem at that elevation instead, we will report that to be the pile length. In general the wave speed of concrete will be close to 12500 ft/s. Depending on the concrete mix, this wave speed may vary plus or minus 10%. Although our estimates on wave speed are also based on obtained proportionality, expect the estimated length in the concrete pile to vary in approximately 5%. For timber wave speed could vary by approximately 15% from an average wave speed value of

10000 ft/s depending on timber type, age and wear. Although as mentioned before, our wave speed estimation is based as well on proportionality, expect the length estimated length to vary approximately 10%.

Depending on which was the best quality data obtained, the top load cell or the instrumentation below the pile top, an estimate on pile length is presented on Table 2. The estimates of pile lengths below the top of deck for Piles 7A, 7B, 8A, 8B, 9A and 9A1 are 77, 63, 79, 73, 35 and 52 feet, respectively.

Bearing Capacity

CAPWAP analysis was performed on one hammer impact record for each tested pile. As is customary in CAPWAP hammer blow selection, the record chosen was first examined for data quality and energy transfer. CAPWAP results are summarized in Table 2 and complete output is presented in Appendix B.

In general, when a permanent set close to 0.2 inches is obtained during dynamic testing for a 13 to 18 inch pile, the pile is considered to reach its ultimate capacity. Lower sets may imply that the pile has more capacity and the measured capacity is considered to be an activated or mobilized capacity.

The capacities calculated by CAPWAP analysis for Piles 7A, 7B, 8A, 8B, 9A and 9A1 were 595, 160, 410, 370, 360 and 225 kips, respectively. Table 2 presents complete results including total shaft friction and end bearing components for each pile. Complete shaft resistance distribution is included in Appendix B.

Limitations

In general, for timber piles, increases in pile size and increases in shaft resistance produce similar compressive stress wave reflections. Therefore, signal matching may either be accomplished by an increased soil resistance or an increased pile size at the location where a strong compressive reflection occurs. This ambiguity leads to some uncertainty about the predicted resistance distribution at a particular point and any calculated local pile impedance values. Fortunately, average resistance values over the whole pile are insensitive to local wave reflections and, therefore, predicted with much greater confidence than individual resistance forces.

It was a pleasure working with you, and we hope that we can assist you on future projects. Please contact us if you have questions regarding the contents of this report.

Respectfully,

GRL Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
Camilo A Alvarez
No. 67938
Exp. 6/30/15
Civil
STATE OF CALIFORNIA

Camilo Alvarez, MSCE, P.E. Anna M. Klesney, MSCE, E.I.T.

CA:AMK:dms

TABLE 1: Summary of Field Results
Port of Oakland

Pile Name	Approximate Drop Height (in)	Approximate Pile Set (in)	Average Transferred Energy (1) (kips-ft)	Average Maximum Compression Stress (2) (ksi)	Average Maximum Tension Stress (3) (ksi)	CAPWAP Capacity (4) (kips)
7A	48	0.2	33	3.4	0.2	595
7B	18	0.2	10	1.5	0.2	160
8A	36	0.3	36	3.5	0.4	410
8B	36	0.2	29	3.6	0.2	370
9A	36	0.2	35	2.5	0.2	360
9A1	12	0.1	10	1.8	0.1	225

Notes:
1. Average transferred energy near pile top.
2. Average Compressive Stress Calculated by CAPWAP
3. Average Maximum Tension Stress Calculated by CAPWAP.
4. CAPWAP Calculated Capacity.

Table 2: Summary of CAPWAP Results
Port of Oakland

Pile	Pile Type	Estimated pile Length below sensors ft	Testing location below top	Approximate pile length below top of deck ft	Pile Set in	Capacity Total kips	Capacity Shaft kips	Capacity Toe kips	Smith Damping Skin s/ft	Smith Damping Toe s/ft	Soil Quake Skin in	Soil Quake Toe in
7A	18"x18" Concrete	75	2	77	0.2	595	390	205	0.40	0.03	0.22	0.34
7B	13" Timber	60	3	63	0.2	160	105	55	0.36	0.02	0.16	0.22
8A	16"x16" Concrete	79	0	79	0.3	410	340	70	0.19	0.02	0.30	0.60
8B	16"x16" Concrete	73	0	73	0.2	370	290	80	0.34	0.16	0.27	0.44
9A	13" Timber + 1.5" Shotcrete	35	0	35	0.2	360	90	270	0.08	0.07	0.35	0.72
9A1	13" Timber	49	3	52	0.1	225	160	65	0.21	0.40	0.17	0.05

Appendix A

An Introduction into Dynamic Pile Testing Methods

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by GRL Engineers, Inc. and may only be copied with its written permission.

1. BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during both preconstruction test programs and production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the "High Strain Test" Method of pile testing. However, for the sake of completeness, two types of "Low Strain Tests" are also mentioned: the Pile Integrity Test (PIT) and Cross Hole Sonic Logging conducted with the Cross Hole Analyzer™ (CHA).

2. RESULTS FROM PDA DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing*.

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both drilled shafts and impact driven piles during restrike.

2.1 DYNAMIC PILE MONITORING

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- *Bearing capacity* at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- *Dynamic pile stresses* axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- *Pile integrity* assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- *Hammer performance* parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

© 1999, 2001, 2002 GRL Engineers, Inc.

2.2 DYNAMIC PILE LOAD TESTING

Bearing capacity testing of either driven piles or drilled shafts employs the basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it is most important that the test is conducted after a <u>sufficient waiting time</u> following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- *Bearing capacity i.e.* the mobilized capacity present at the time of testing

- *Resistance distribution* including shaft resistance and end bearing components

- *Stresses in pile or shaft* calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.

- *Shaft impedance vs. depth*; this is an estimate of the shaft shape if it differs substantially from the planned profile

- *Dynamic soil parameters* for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

3. FIELD MEASUREMENTS

The following is a general summary of dynamic measurements available to solve typical deep foundation problems.

3.1 PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

3.2 HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

3.3 SAXIMETER™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

3.4 PIT

The Pile Integrity Tester™ (PIT) helps in detecting major defects in concrete piles or shafts or assess the length of a variety of deep foundations, except steel piles. PIT performs the "Pulse-Echo Method" which only requires the measurement of motion (e.g., acceleration) at the pile top caused by a light hammer impact. PIT also supports the "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. PIT may also be used to evaluate the unknown length of deep foundations under existing structures.

3.5 CHA

This test requires that at least two tubes (typically steel tubes of 50 mm diameter) are installed vertically in the shaft to be tested. A high frequency signal is generated in one of the water filled tubes and received in the other tube. The received signal strength and its First Time of Arrival (FAT) yield important information about the concrete quality between a given pair(s) of tubes. The transmitting and recording of the signal is repeated typically every 50 mm starting at the shaft bottom and all records together establish a log or profile of the concrete quality between the tubes. The total number of tubes installed depends on the size of the drilled shaft. More tubes allow for the construction of more profiles.

4.ANALYTICAL SOLUTIONS

4.1 *BEARING CAPACITY*

4.1.1 WAVE EQUATION

The GRLWEAP program calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation

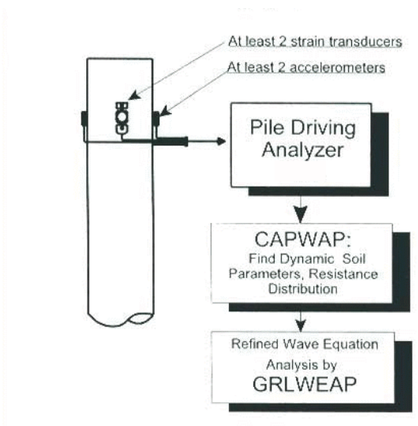


Figure 1. Block Diagram of Refined Wave Equation Analysis

logs, the bearing graph yields a corresponding bearing capacity. This approach requires no field measurements other than blow count. Rather it requires an accurate knowledge of the various parameters describing hammer, driving system, pile and soil. The wave equation is also very useful during the design stage of a project for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (Figure 1.) is often performed by inputting the PDA and CAPWAP calculated parameters. With many of the dynamic parameters verified by the dynamic tests, the RWEA offers a more reliable basis for a safe and sufficient driving criterion.

4.1.2 CASE METHOD

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force, F(t), and pile top velocity, v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time t + 2L/c
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile (ρc^2)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and the pile velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 could be evaluated. Most commonly, T is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed for one record. The capacities for other hammer blows are then quickly calculated for the thus selected Case Method and its associated damping factor.

The static resistance calculated by either Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDI-PLOT program or formerly in the DOS based PDAPLOT program.

4.1.3 CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffness values. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters based on the dynamic measurements.

4.1.4 Capacity of damaged piles

Occasionally piles are damaged during driving and such damage may be indicated in the PDA collected records, if it occurs below the sensor location. Damage on steel piles is often a broken splice, a collapsed pile bottom section, a ripped of flange on an H-pile or a sharp bend (a gradual dog leg is usually not recognized in the records). For concrete piles, among the problems encountered are cracks, perpendicular due to the pile axis, which deteriorate into a major damage, slabbing (loss of concrete cover) or a compressive failure at the bottom which in effect makes the pile shorter.

Damaged piles, with beta values less than 0.8 should never be evaluated for bearing capacity by the Case Method alone, because these are non-uniform piles which therefore violate the basic premise of the Case Method: a uniform, elastic pile.

Using the CAPWAP program, it is sometimes possible to obtain a reasonable match between computed and measured pile top quantities. In such an analysis the damaged section has to be modeled either by impedance reductions or by slacks. For piles with severe damage along their length it may be necessary to analyze a short pile. It should be born in mind, however, that such an analysis also violates the basic principles of the CAPWAP analysis, namely that the pile is elastic. Also, the nature of the damage is never be known with certainty. For example, a broken splice could be a cracked weld either with the neighboring sections lining up well or shifted laterally. In the former case the stresses would be similar to those in the undamaged pile; in the latter situation, high stress concentrations would develop. A sharp bend or toe damage present equally unpredictable situations under sustained loads which may cause further structural deterioration. If a short pile is analyzed then the lower section of the pile below the damage may offer unreliable end bearing and therefore should be discounted.

It is GRL's position that damaged piling should be replaced. Utilizing the CAPWAP calculated

capacities should only be done after a very careful consideration of the effects of a loss of the foundation member while in service. Under no circumstances should the CAPWAP calculated capacity be utilized in the same manner in which the capacity of an undamaged pile be used. Under the best of circumstances the capacity should be used with an increased factor of safety and discounting all questionable capacity components. This evaluation cannot be made by GRL as it involves consideration of the type of structure, its seismic environment, the nature of the loads expected, the corrosiveness of the soil material, considerations of scour on the shortened pile, etc.

4.2 *STRESSES*

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance, R(t), minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress, again averaged over the cross section and therefore not including bending stresses, can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \quad (4)$$

$$W_d = ½[F(t) + Zv(t)] \quad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

4.3 *PILE INTEGRITY BY PDA*

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta = (1 - \alpha)/(1 + \alpha) \quad (6)$$

with

$$\alpha = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the damage reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling reflection wave due to the damage.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.
Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

4.4 *HAMMER PERFORMANCE BY PDA*

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(\tau)v(\tau)\, d\tau \quad (8a)$$

The maximum of the E(t) curve is often called **ENTHRU;** it is the most important information for an overall evaluation of the performance of a hammer and driving system. **ENTHRU** or **EMX** allow for a classification of the hammer's performance when presented as, e_T, the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency.

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the hammer manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\, T_B^2 - h_L \quad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

4.5 *DETERMINATION OF WAVE SPEED*

An important facet of dynamic pile testing is an assessment of pile material properties. Since, n most cases general force is determined from strain by multiplication with elastic modulus, **E**, and cross sectional area, **A**, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \quad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c of the whole pile is lower than the wave speed at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes during the pile installation.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

5. *DATA QUALITY CHECKS*

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

5.1 PROPORTIONALITY

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (12b)$$

or strain

$$\varepsilon = v / c \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

5.2 *NUMBER OF SENSORS*

Measurements are always taken at opposite sides of the pile so that the average force and velocity in the pile can be calculated. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. In that case the averaging of the two strain signals does not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

6. LIMITATIONS, ADDITIONAL CONSIDERATIONS

6.1 MOBILIZATION OF CAPACITY

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

6.2 TIME DEPENDENT SOIL RESISTANCE EFFORTS

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur as a result of soil setup and relaxation. Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

6.2.1 SOIL SETUP

Because excess positive pore pressures often develop during pile driving in fine grained soils (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures

reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze. There are numerous other reasons for soil setup such as realignment of clay particles, arching that reduces effective stresses during pile installation in ver dense sands, soil fatigue in over-consolidated clays etc.

6.2.2 RELAXATION

Relaxation capacity reduction with time has been observed for piles driven into weathered shale, and may take several days to fully develop. Where relaxation occurs, pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically with particular emphasis on the first few blows. Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous overpredictions

6.3 CAPACITY RESULTS FOR OPEN PILE PROFILES

Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

6.4 CAPWAP ANALYSIS RESULTS

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the signal match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

6.5 STRESSES

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90% of yield strength for steel piles

85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements. The above allowable stresses also apply to those calculated by wave equation.

6.6 ADDITIONAL DESIGN CONSIDERATIONS

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,
- loss of shaft resistance due to scour or other effects,
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads general due not cause buckling even though they may exceed the buckling strength of the pile section), corrosion etc.

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

6.7 WAVE EQUATION ANALYSIS RESULTS

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

7. FACTORS OF SAFETY

Run to failure, static or dynamic load tests yield an ultimate pile bearing capacity, R_{ult}. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, it is absolutely necessary that the actually applied load, also called the design load, R_d (or working load or safe load), is less than R_{ult}. In most soils, to limit settlements, it is necessary that R_{ult}, is at least 50% higher than R_d. This means that

$$R_{ult} \geq 1.5\ R_d,$$

or the Factor of Safety has to be at least 1.5.

Unfortunately, neither applied loads nor R_{ult} are exactly known. One static load test may be performed at a site, but that would not guarantee that all other piles have the same capacity and it is to be expected that a certain percentage of the production piles have lower capacities, either due to soil variability or due to pile damage. If, for example, dynamic pile tests are performed on piles in shale only a short time after pile installation, then the test capacity may be higher than the long term capacity of the pile. On the other hand, due to soil setup, piles generally gain capacity after installation and since tests are only done a short time after installation, a lower capacity value is ascertained than the capacity that eventually develops.

Not only bearing capacity values of all piles are unknown, even loads vary considerably and occasional overloads must be expected. We would not want a structure to become unserviceable or useless because of either an occasional overload or a few piles with low capacity. For this reason, and to avoid being overly conservative which would mean excessive cost, modern safety concepts suggest that the overall factor of safety should reflect both the uncertainty in loads and resistance. Thus, if all piles were tested statically and if we carefully

controlled the loads, we probably could live with F.S. = 1.5. However, in general, depending on the building type or load combinations and as a function of quality assurance of pile foundations, a variety of Factors of Safety have been proposed.

For example, for highway related loads and based on AASHTO specifications, the Federal Highway Administration proposes the following:

F.S.= 2.00 for static load test with wave equation.

F.S.=2.25 for dynamic testing with wave equation analysis.

F.S.=2.50 for indicator piles with wave equation analysis.

F.S.=2.75 for wave equation analysis.

F.S.=3.00 for Gates or other dynamic formula.

It should be mentioned that all of these methods should always be combined with soil exploration and static pile analysis. Also, specifications of what are occasionally updated and therefore the latest version should be various consulted for the appropriate factors of safety.

Codes, among them PDCA, ASCE, or specifications issued by State Departments of Transportation specify different factors of safety. However, the range of recommended overall factors of safety in the United States varies between 1.9 and 6.

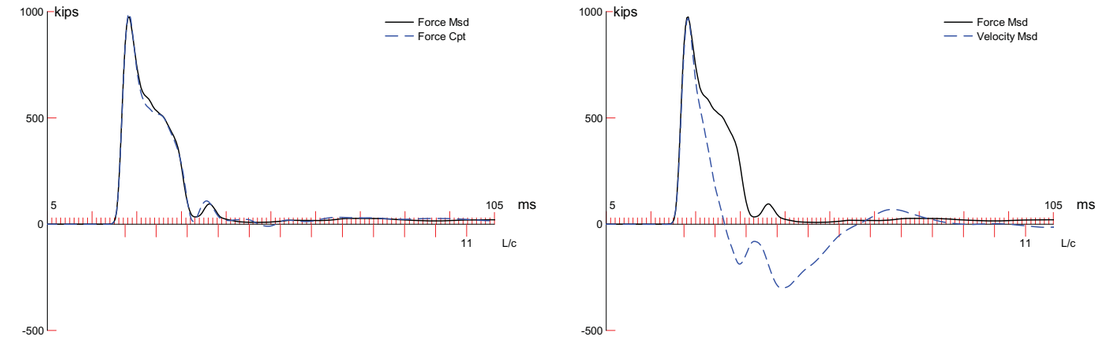
It is the designer's responsibility to identify design loads together with the adopted safety factor concept and associated construction control procedure. The required factors of safety should be included in design drawings or specifications together with the required testing. Only contractors bid for the work and develop the most economical solution. This should include a program of increased testing for lower required pile capacities. This will also help to reduce the confusion that often exists on construction sites as to design loads and require capacities. In any event, it cannot expected that the test engineer is aware of and responsible for the variety of considerations that must be met to find the appropriate factor of safety.

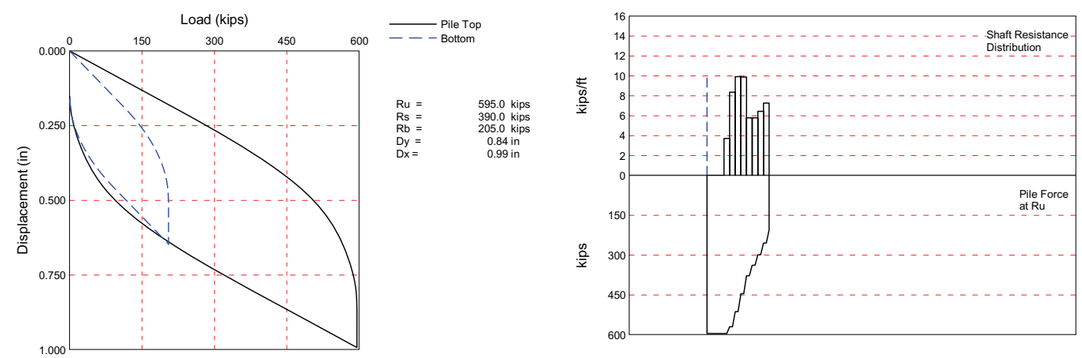
APPENDIX B

CAPWAP Analyses Results

OAKLAND ARMY BASE; Pile: P7A; BH; Blow: 4 (Test: 30-Oct-2013 16:24:)
GRL Engineers, Inc.

30-Oct-2013
CAPWAP(R) 2006-3





CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 595.0; along Shaft 390.0; at Toe 205.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				595.0				
1	6.8	6.8	0.0	595.0	0.0	0.00	0.00	0.000
2	13.6	13.6	0.0	595.0	0.0	0.00	0.00	0.000
3	20.5	20.5	0.0	595.0	0.0	0.00	0.00	0.000
4	27.3	27.3	25.4	569.6	25.4	3.73	0.62	0.400
5	34.1	34.1	57.0	512.6	82.4	8.36	1.39	0.400
6	40.9	40.9	67.7	444.9	150.1	9.93	1.65	0.400
7	47.7	47.7	67.4	377.5	217.5	9.88	1.65	0.400
8	54.5	54.5	39.5	338.0	257.0	5.79	0.97	0.400
9	61.4	61.4	39.5	298.5	296.5	5.79	0.97	0.400
10	68.2	68.2	43.9	254.6	340.4	6.44	1.07	0.400
11	75.0	75.0	49.6	205.0	390.0	7.27	1.21	0.400
Avg. Shaft			35.5			5.20	0.87	0.400
Toe			205.0				91.11	0.025

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.222	0.342
Case Damping Factor		1.378	0.045
Unloading Quake	(% of loading quake)	118	81
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Resistance Gap (included in Toe Quake)	(in)		0.004
Soil Plug Weight	(kips)		0.30
Soil Support Dashpot		1.210	0.000
Soil Support Weight	(kips)	4.27	0.00

CAPWAP match quality = 1.98 (Wave Up Match) ; RSA = 0
Observed: final set = 0.150 in; blow count = 80 b/ft
Computed: final set = 0.124 in; blow count = 97 b/ft

max. Top Comp. Stress = 3.07 ksi (T= 24.0 ms, max= 1.103 x Top)
max. Comp. Stress = 3.38 ksi (Z= 27.3 ft, T= 26.2 ms)
max. Tens. Stress = -0.20 ksi (Z= 27.3 ft, T= 53.0 ms)
max. Energy (EMX) = 32.5 kip-ft; max. Measured Top Displ. (DMX)= 0.53 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.4	993.7	-17.4	3.07	-0.05	32.53	8.4	0.533
2	6.8	993.2	-25.0	3.06	-0.08	32.17	8.4	0.516
3	10.2	994.4	-32.9	3.07	-0.10	31.80	8.4	0.499
4	13.6	998.2	-40.3	3.08	-0.12	31.41	8.4	0.482
5	17.0	1009.1	-47.2	3.11	-0.15	31.08	8.3	0.471
6	20.5	1029.0	-53.4	3.18	-0.16	31.05	8.1	0.467
7	23.9	1059.3	-59.1	3.27	-0.18	31.00	7.8	0.462
8	27.3	1096.0	-64.4	3.38	-0.20	30.93	7.5	0.456
9	30.7	1053.8	-54.7	3.25	-0.17	28.59	7.2	0.449
10	34.1	1093.0	-61.3	3.37	-0.19	28.50	6.8	0.442
11	37.5	973.4	-38.6	3.00	-0.12	24.26	6.5	0.434
12	40.9	1005.9	-44.7	3.10	-0.14	24.17	6.2	0.426
13	44.3	867.4	-18.3	2.68	-0.06	19.70	5.9	0.419
14	47.7	890.0	-23.0	2.75	-0.07	19.62	5.7	0.411
15	51.1	757.1	-0.4	2.34	-0.00	15.41	5.5	0.404
16	54.5	777.8	-5.4	2.40	-0.02	15.34	5.3	0.397
17	58.0	704.0	-0.1	2.17	-0.00	12.71	5.2	0.390
18	61.4	708.1	-4.0	2.18	-0.01	12.63	5.2	0.382
19	64.8	587.8	-0.1	1.81	-0.00	9.97	6.0	0.374
20	68.2	524.1	-3.3	1.62	-0.01	9.90	6.7	0.366
21	71.6	323.0	-0.1	1.00	-0.00	6.87	7.2	0.359
22	75.0	261.6	-3.6	0.81	-0.01	3.76	7.3	0.351
Absolute	27.3			3.38			(T =	26.2 ms)
	27.3				-0.20		(T =	53.0 ms)

OAKLAND ARMY BASE; Pile: P7A — Test: 30-Oct-2013 16:24:
BH; Blow: 4 — CAPWAP(R) 2006-3
GRL Engineers, Inc. — OP: AK

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1048.9	958.3	867.6	776.9	686.2	595.6	504.9	414.2	323.6	232.9
RX	1048.9	958.3	867.6	776.9	686.2	595.6	504.9	414.2	378.0	354.3
RU	1209.5	1134.8	1060.2	985.6	911.0	836.3	761.7	687.1	612.5	537.8

RAU = 270.7 (kips); RA2 = 593.8 (kips)

Current CAPWAP Ru = 595.0 (kips); Corresponding J(RP)= 0.50; J(RX) = 0.50

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
8.60	22.73	974.7	981.0	981.9	0.534	0.156	0.150	32.7	1149.3

PILE PROFILE AND PILE MODEL

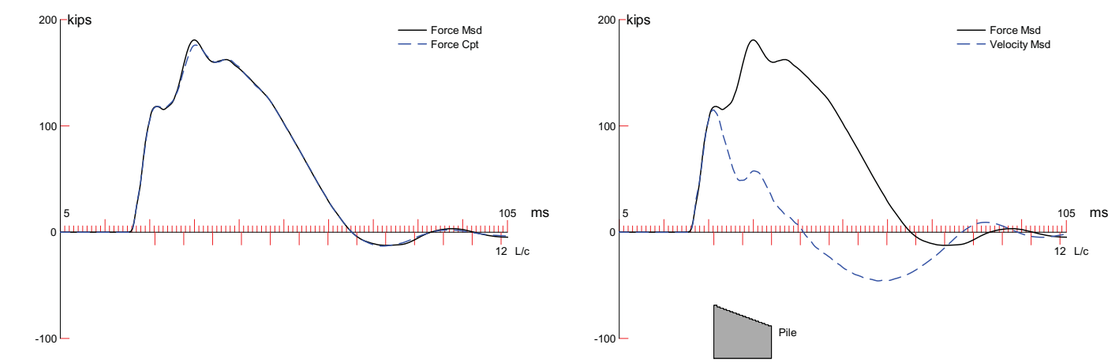
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	324.00	3775.4	150.000	6.000
75.00	324.00	3775.4	150.000	6.000

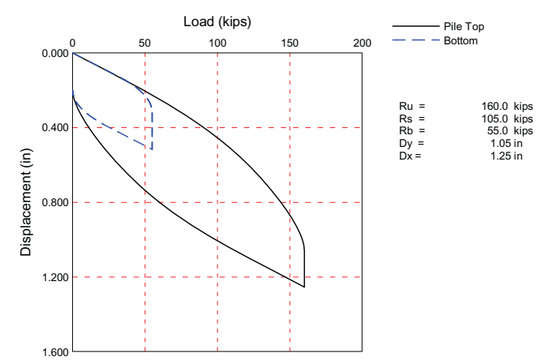
Toe Area 2.250 ft^2

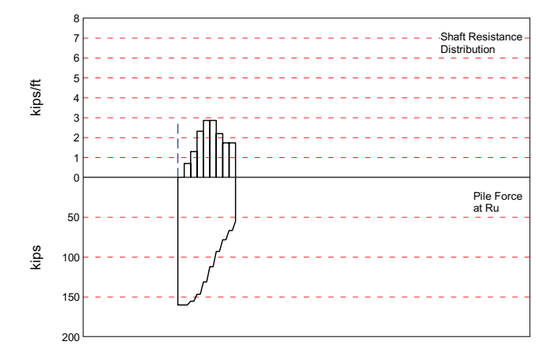
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
1	3.41	113.29	0.00	0.000	0.000	-0.000	0.000	6.000	0.00
2	6.82	113.29	0.00	0.000	0.000	-0.000	0.000	6.000	0.06
3	10.23	113.29	0.00	0.000	0.000	-0.000	0.000	6.000	0.05
22	75.00	113.29	0.00	0.000	0.000	-0.000	0.000	6.000	0.05

Pile Damping 0.5 %, Time Incr 0.316 ms, Wave Speed 10800.0 ft/s, 2L/c 13.9 ms

OAKLAND ARMY BASE; Pile: P7B; BH; Blow: 2 (Test: 30-Oct-2013 19:22:) — 30-Oct-2013
GRL Engineers, Inc. — CAPWAP(R) 2006-3







CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 160.0; along Shaft 105.0; at Toe 55.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				160.0				
1	6.7	6.7	0.0	160.0	0.0	0.00	0.00	0.000
2	13.3	13.3	4.7	155.3	4.7	0.71	0.22	0.363
3	20.0	20.0	8.7	146.6	13.4	1.31	0.41	0.363
4	26.7	26.7	15.5	131.1	28.9	2.33	0.75	0.363
5	33.3	33.3	19.1	112.0	48.0	2.87	0.96	0.363
6	40.0	40.0	19.1	92.9	67.1	2.87	0.98	0.363
7	46.7	46.7	14.7	78.2	81.8	2.21	0.78	0.363
8	53.3	53.3	11.6	66.6	93.4	1.74	0.63	0.363
9	60.0	60.0	11.6	55.0	105.0	1.74	0.65	0.363
Avg. Shaft			11.7			1.75	0.58	0.363
Toe			55.0				60.60	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.161	0.218
Case Damping Factor		2.422	0.085
Unloading Quake	(% of loading quake)	100	55
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	28	
Resistance Gap (included in Toe Quake)	(in)		0.114

CAPWAP match quality = 1.12 (Wave Up Match) ; RSA = 0
Observed: final set = 0.200 in; blow count = 60 b/ft
Computed: final set = 0.225 in; blow count = 53 b/ft

max. Top Comp. Stress = 1.4 ksi (T= 36.2 ms, max= 1.121 x Top)
max. Comp. Stress = 1.5 ksi (Z= 20.0 ft, T= 38.4 ms)
max. Tens. Stress = -0.15 ksi (Z= 13.3 ft, T= 77.8 ms)
max. Energy (EMX) = 10.4 kip-ft; max. Measured Top Displ. (DMX)= 0.97 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	177.1	-14.3	1.4	-0.11	10.38	7.2	0.947
2	6.7	178.2	-15.4	1.4	-0.12	10.11	7.0	0.904
3	10.0	179.4	-16.5	1.5	-0.13	9.83	6.6	0.860
4	13.3	180.8	-17.5	1.5	-0.15	9.54	6.3	0.814
5	16.7	172.5	-15.6	1.5	-0.13	8.57	5.8	0.769
6	20.0	174.3	-16.5	1.5	-0.14	8.28	5.3	0.723
7	23.3	160.7	-12.7	1.4	-0.11	7.00	4.8	0.678
8	26.7	163.3	-13.3	1.5	-0.12	6.75	4.3	0.633
9	30.0	139.5	-7.1	1.3	-0.07	5.18	3.9	0.591
10	33.3	141.1	-7.6	1.4	-0.07	4.97	3.4	0.549
11	36.7	114.6	-1.5	1.1	-0.02	3.56	3.1	0.512
12	40.0	115.4	-1.9	1.2	-0.02	3.40	2.8	0.474
13	43.3	96.9	-0.0	1.0	-0.00	2.33	2.6	0.440
14	46.7	96.9	-0.0	1.1	-0.00	2.21	2.5	0.406
15	50.0	82.4	-0.0	0.9	-0.00	1.52	2.5	0.375
16	53.3	82.3	-0.0	1.0	-0.00	1.42	2.5	0.344
17	56.7	70.3	-0.0	0.8	-0.00	0.97	2.5	0.315
18	60.0	70.3	-0.0	0.9	-0.00	0.58	2.4	0.284
Absolute	20.0			1.5			(T =	38.4 ms)
	13.3				-0.15		(T =	77.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	179.0	173.6	168.2	162.7	157.3	151.9	146.5	141.0	135.6	130.2
RX	194.7	190.3	185.9	181.6	177.3	173.0	168.7	164.4	160.2	156.0
RU	179.0	173.6	168.2	162.7	157.3	151.9	146.5	141.0	135.6	130.2

RAU = 24.1 (kips); RA2 = 172.7 (kips)

Current CAPWAP Ru = 160.0 (kips); Corresponding J(RP)= 0.35; J(RX) = 0.80

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
7.34	26.52	115.5	117.8	180.9	0.971	0.200	0.200	10.5	215.3

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	130.70	1119.8	60.000	3.377
60.00	78.54	1119.8	60.000	2.618

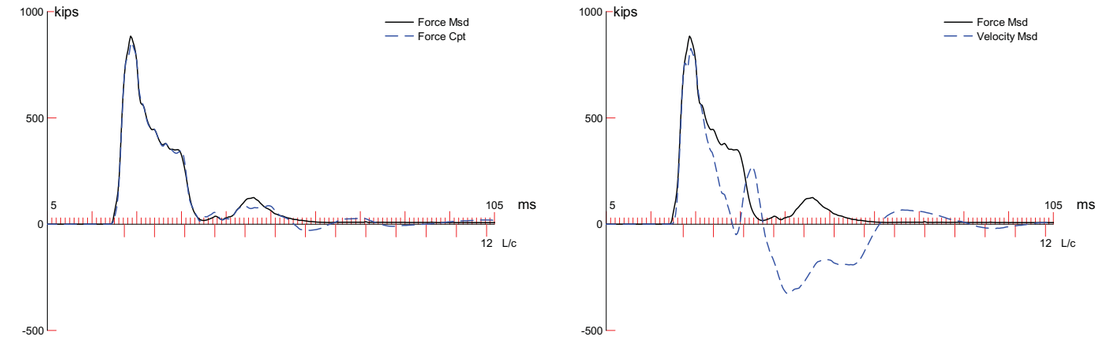
OAKLAND ARMY BASE; Pile: P7B — Test: 30-Oct-2013 19:22:
BH; Blow: 2 — CAPWAP(R) 2006-3
GRL Engineers, Inc. — OP: AK

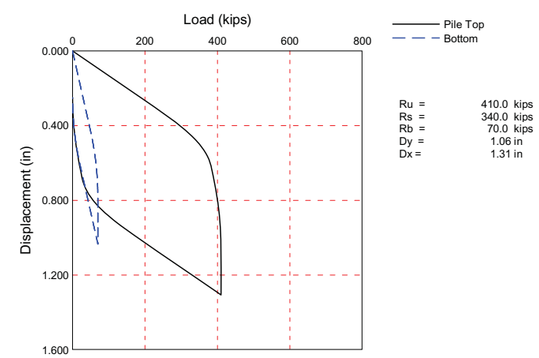
Toe Area 0.908 ft^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
1	3.33	15.74	0.00	0.000	0.000	-0.000	0.000	3.377	0.00
2	6.67	15.22	0.00	0.000	0.000	-0.000	0.000	3.314	0.01
3	10.00	14.87	0.00	0.000	0.000	-0.000	0.000	3.272	0.00
4	13.33	14.52	0.00	0.000	0.000	-0.000	0.000	3.229	0.00
5	16.67	14.17	0.00	0.000	0.000	-0.000	0.000	3.187	0.00
6	20.00	13.82	0.00	0.000	0.000	-0.000	0.000	3.145	0.00
7	23.33	13.47	0.00	0.000	0.000	-0.000	0.000	3.103	0.00
8	26.67	13.12	0.00	0.000	0.000	-0.000	0.000	3.061	0.00
9	30.00	12.77	0.00	0.000	0.000	-0.000	0.000	3.019	0.00
10	33.33	12.43	0.00	0.000	0.000	-0.000	0.000	2.976	0.00
11	36.67	12.08	0.00	0.000	0.000	-0.000	0.000	2.934	0.00
12	40.00	11.73	0.00	0.000	0.000	-0.000	0.000	2.892	0.00
13	43.33	11.38	0.00	0.000	0.000	-0.000	0.000	2.850	0.00
14	46.67	11.03	0.00	0.000	0.000	-0.000	0.000	2.808	0.00
15	50.00	10.68	0.00	0.000	0.000	-0.000	0.000	2.766	0.00
16	53.33	10.33	0.00	0.000	0.000	-0.000	0.000	2.723	0.00
17	56.67	9.98	0.00	0.000	0.000	-0.000	0.000	2.681	0.00
18	60.00	9.63	0.00	0.000	0.000	-0.000	0.000	2.639	0.00

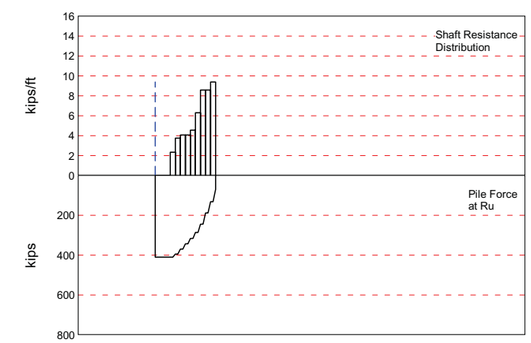
Pile Damping 2.0 %, Time Incr 0.358 ms, Wave Speed 9300.0 ft/s, 2L/c 12.9 ms

OAKLAND ARMY BASE; Pile: 8A; 16 CONCRETE; Blow: 3 (Test: 31-Oct-2013 13:36:)
GRL Engineers, Inc.
31-Oct-2013
CAPWAP(R) 2006-3





Ru = 410.0 kips
Rs = 340.0 kips
Rb = 70.0 kips
Dy = 1.06 in
Dx = 1.31 in



CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 410.0; along Shaft 340.0; at Toe 70.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				410.0				
1	6.6	6.6	0.0	410.0	0.0	0.00	0.00	0.000
2	13.2	13.2	0.0	410.0	0.0	0.00	0.00	0.000
3	19.8	19.8	0.0	410.0	0.0	0.00	0.00	0.000
4	26.3	26.3	15.4	394.6	15.4	2.34	0.44	0.188
5	32.9	32.9	24.7	369.9	40.1	3.75	0.70	0.188
6	39.5	39.5	26.8	343.1	66.9	4.07	0.76	0.188
7	46.1	46.1	26.8	316.3	93.7	4.07	0.76	0.188
8	52.7	52.7	30.0	286.3	123.7	4.56	0.85	0.188
9	59.3	59.3	41.5	244.8	165.2	6.30	1.18	0.188
10	65.8	65.8	56.5	188.3	221.7	8.58	1.61	0.188
11	72.4	72.4	56.5	131.8	278.2	8.58	1.61	0.188
12	79.0	79.0	61.8	70.0	340.0	9.39	1.76	0.188
Avg. Shaft			28.3			4.30	0.81	0.188
Toe			70.0				39.37	0.024

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.295	0.603
Case Damping Factor		0.658	0.018
Damping Type			Smith
Unloading Quake	(% of loading quake)	100	103
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	28	
Resistance Gap (included in Toe Quake)	(in)		0.019
Soil Plug Weight	(kips)		1.22

CAPWAP match quality = 3.37 (Wave Up Match) ; RSA = 0
Observed: final set = 0.250 in; blow count = 48 b/ft
Computed: final set = 0.257 in; blow count = 47 b/ft

max. Top Comp. Stress = 3.34 ksi (T= 24.5 ms, max= 1.057 x Top)
max. Comp. Stress = 3.53 ksi (Z= 26.3 ft, T= 26.4 ms)
max. Tens. Stress = -0.40 ksi (Z= 32.9 ft, T= 63.9 ms)
max. Energy (EMX) = 35.6 kip-ft; max. Measured Top Displ. (DMX)= 0.76 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	854.4	-39.8	3.34	-0.16	35.65	8.7	0.769
2	6.6	854.9	-50.8	3.34	-0.20	35.63	8.7	0.766
4	13.2	861.8	-71.2	3.37	-0.28	35.56	8.7	0.756
6	19.8	878.5	-89.2	3.43	-0.35	35.42	8.5	0.741
7	23.0	889.8	-96.4	3.47	-0.38	35.32	8.4	0.733
8	26.3	902.6	-102.3	3.53	-0.40	35.22	8.2	0.724
9	29.6	880.9	-100.4	3.44	-0.39	33.55	8.1	0.715
10	32.9	894.9	-103.1	3.49	-0.40	33.43	7.9	0.705
11	36.2	854.7	-95.6	3.34	-0.37	30.93	7.8	0.696
12	39.5	869.4	-96.4	3.40	-0.38	30.80	7.6	0.686
13	42.8	828.3	-86.0	3.23	-0.34	28.17	7.5	0.676
14	46.1	845.5	-85.7	3.30	-0.33	28.05	7.3	0.666
15	49.4	810.0	-74.7	3.16	-0.29	25.49	7.1	0.657
16	52.7	831.3	-73.8	3.25	-0.29	25.38	6.9	0.647
17	56.0	795.3	-59.9	3.11	-0.23	22.60	6.7	0.638
18	59.3	810.3	-58.1	3.16	-0.23	22.56	6.5	0.632
19	62.5	739.1	-40.2	2.89	-0.16	18.84	6.9	0.627
20	65.8	719.9	-38.6	2.81	-0.15	18.80	8.1	0.622
21	69.1	605.8	-19.0	2.37	-0.07	13.65	8.9	0.617
22	72.4	569.0	-17.7	2.22	-0.07	13.61	9.3	0.612
23	75.7	418.0	-5.5	1.63	-0.02	8.24	9.7	0.608
24	79.0	297.3	-0.0	1.16	-0.00	1.93	9.8	0.605
Absolute	26.3			3.53			(T =	26.4 ms)
	32.9				-0.40		(T =	63.9 ms)

OAKLAND ARMY BASE; Pile: 8A — Test: 31-Oct-2013 13:36:
16 CONCRETE; Blow: 3 — CAPWAP(R) 2006-3
GRL Engineers, Inc. — OP: AK

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	763.5	673.1	582.7	492.3	401.8	311.4	221.0	130.6	40.2	0.0
RX	770.3	676.6	583.2	492.3	401.8	391.1	390.9	390.7	390.5	390.3
RU	749.3	657.5	565.7	473.8	382.0	290.2	198.4	106.6	14.7	0.0

RAU = 357.5 (kips); RA2 = 438.5 (kips)

Current CAPWAP Ru = 410.0 (kips); Corresponding J(RP)= 0.39; J(RX) = 0.39

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
8.58	22.51	784.5	883.1	889.7	0.760	0.250	0.250	35.7	847.3

PILE PROFILE AND PILE MODEL

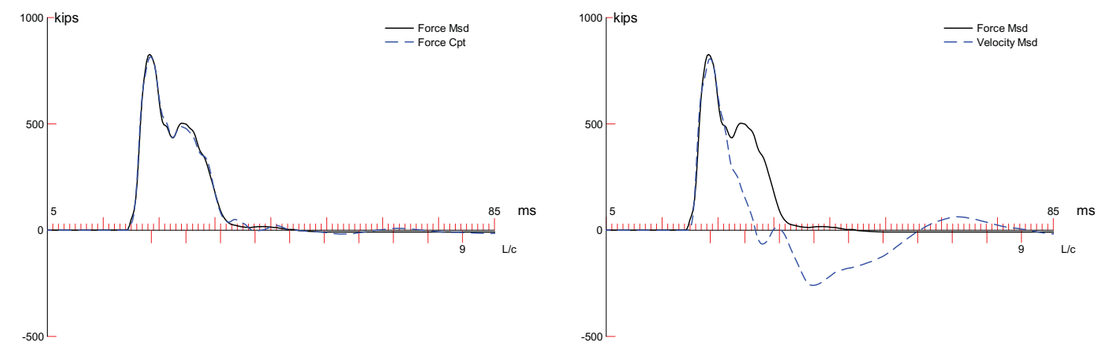
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	256.00	4430.8	150.000	5.333
79.00	256.00	4430.8	150.000	5.333

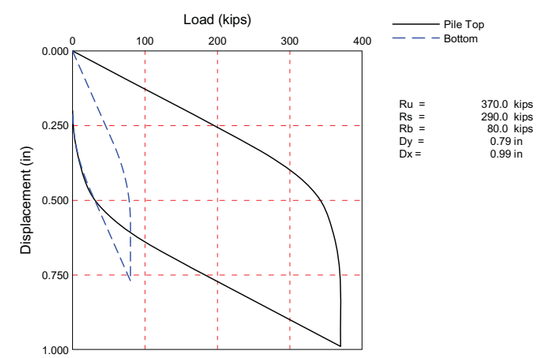
Toe Area 1.778 ft^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
1	3.29	96.97	0.00	0.000	0.000	-0.000	0.000	5.333	0.00
2	6.58	96.97	0.00	0.000	0.000	-0.000	0.000	5.333	0.01
24	79.00	96.97	0.00	0.000	0.000	-0.000	0.000	5.333	0.01

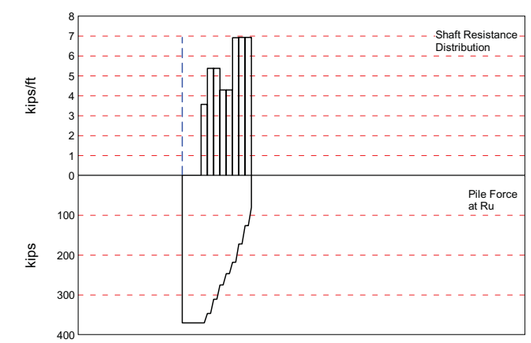
Pile Damping 2.0 %, Time Incr 0.281 ms, Wave Speed 11700.0 ft/s, 2L/c 13.5 ms

OAKLAND ARMY BASE; Pile: 8B; Blow: 3 (Test: 01-Nov-2013 18:42:)
GRL Engineers, Inc.
08-Nov-2013
CAPWAP(R) 2006-3





Ru = 370.0 kips
Rs = 290.0 kips
Rb = 80.0 kips
Dy = 0.79 in
Dx = 0.99 in



CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 370.0; along Shaft 290.0; at Toe 80.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				370.0				
1	6.6	6.6	0.0	370.0	0.0	0.00	0.00	0.000
2	13.3	13.3	0.0	370.0	0.0	0.00	0.00	0.000
3	19.9	19.9	0.0	370.0	0.0	0.00	0.00	0.000
4	26.5	26.5	23.7	346.3	23.7	3.57	0.67	0.336
5	33.2	33.2	35.7	310.6	59.4	5.38	1.01	0.336
6	39.8	39.8	35.7	274.9	95.1	5.38	1.01	0.336
7	46.5	46.5	28.5	246.4	123.6	4.29	0.81	0.336
8	53.1	53.1	28.5	217.9	152.1	4.29	0.81	0.336
9	59.7	59.7	45.9	172.0	198.0	6.92	1.30	0.336
10	66.4	66.4	46.0	126.0	244.0	6.93	1.30	0.336
11	73.0	73.0	46.0	80.0	290.0	6.93	1.30	0.336
Avg. Shaft			26.4			3.97	0.74	0.336
Toe			80.0				45.00	0.160

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.273	0.439
Case Damping Factor		0.995	0.131
Unloading Quake	(% of loading quake)	100	99
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	96	
Resistance Gap (included in Toe Quake)	(in)		0.108
Soil Plug Weight	(kips)		0.88

CAPWAP match quality = 2.26 (Wave Up Match) ; RSA = 0
Observed: final set = 0.200 in; blow count = 60 b/ft
Computed: final set = 0.178 in; blow count = 67 b/ft
Replay Factor: F3:1.411; F4:1.411; V3:1.000; V4:1.000;

max. Top Comp. Stress = 3.18 ksi (T= 23.9 ms, max= 1.120 x Top)
max. Comp. Stress = 3.57 ksi (Z= 26.5 ft, T= 26.4 ms)
max. Tens. Stress = -0.24 ksi (Z= 26.5 ft, T= 57.1 ms)
max. Energy (EMX) = 29.0 kip-ft; max. Measured Top Displ. (DMX)= 0.57 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	815.2	-23.9	3.18	-0.09	29.04	8.3	0.560
2	6.6	816.0	-29.7	3.19	-0.12	28.78	8.3	0.553
3	10.0	820.1	-35.5	3.20	-0.14	28.75	8.3	0.550
4	13.3	831.0	-41.3	3.25	-0.16	28.72	8.2	0.546
5	16.6	846.5	-46.8	3.31	-0.18	28.68	8.1	0.540
6	19.9	866.4	-51.8	3.38	-0.20	28.63	7.9	0.534
7	23.2	889.8	-56.4	3.47	-0.22	28.57	7.6	0.527
8	26.5	913.1	-61.2	3.57	-0.24	28.50	7.4	0.520
9	29.9	863.3	-52.0	3.37	-0.20	26.15	7.2	0.513
10	33.2	883.9	-55.5	3.45	-0.22	26.08	6.9	0.506
11	36.5	801.4	-40.9	3.13	-0.16	22.82	6.7	0.499
12	39.8	820.0	-44.0	3.20	-0.17	22.76	6.5	0.492
13	43.1	742.8	-31.4	2.90	-0.12	19.68	6.4	0.486
14	46.5	764.9	-34.2	2.99	-0.13	19.63	6.2	0.479
15	49.8	715.5	-26.1	2.79	-0.10	17.27	5.9	0.474
16	53.1	734.0	-28.8	2.87	-0.11	17.23	5.7	0.468
17	56.4	680.9	-21.9	2.66	-0.09	14.96	5.5	0.463
18	59.7	679.3	-24.4	2.65	-0.10	14.93	6.0	0.459
19	63.0	552.9	-12.9	2.16	-0.05	11.30	6.7	0.455
20	66.4	502.4	-15.2	1.96	-0.06	11.28	7.1	0.451
21	69.7	354.7	-5.4	1.39	-0.02	7.50	7.4	0.448
22	73.0	263.7	-7.7	1.03	-0.03	3.51	7.6	0.445
Absolute	26.5			3.57			(T =	26.4 ms)
	26.5				-0.24		(T =	57.1 ms)

OAKLAND ARMY BASE; Pile: 8B — Test: 01-Nov-2013 18:42:
Blow: 3 — CAPWAP(R) 2006-3
GRL Engineers, Inc. — OP: AK

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	860.4	783.2	706.0	628.8	551.7	474.5	397.3	320.1	243.0	165.8
RX	860.4	783.2	706.0	628.8	551.7	474.5	397.3	335.9	314.5	293.8
RU	933.6	863.7	793.9	724.0	654.2	584.3	514.4	444.6	374.7	304.9

RAU = 193.0 (kips); RA2 = 500.1 (kips)

Current CAPWAP Ru = 370.0 (kips); Corresponding J(RP)= 0.64; J(RX) = 0.64

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
8.28	23.90	809.4	822.7	828.8	0.569	0.200	0.200	29.1	907.4

PILE PROFILE AND PILE MODEL

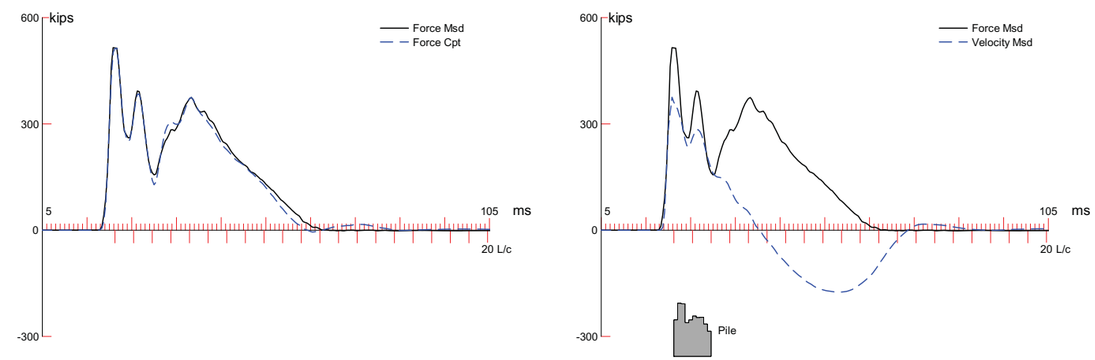
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	256.00	4506.9	150.000	5.333
73.00	256.00	4506.9	150.000	5.333

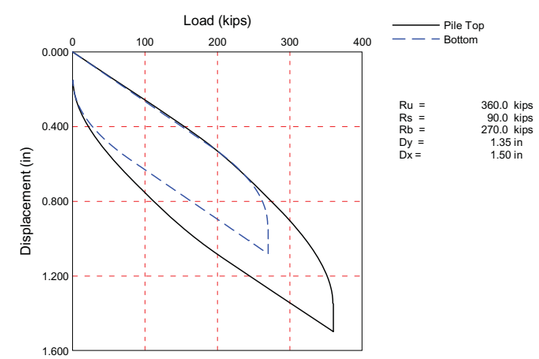
Toe Area 1.778 ft^2

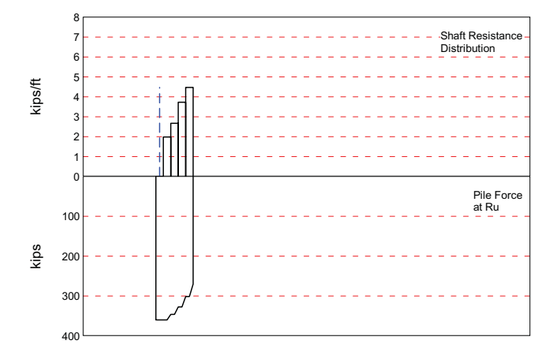
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
1	3.32	97.80	0.00	0.000	0.000	-0.000	0.000	5.333	0.00
2	6.64	97.80	0.00	0.000	0.000	-0.000	0.000	5.333	0.00
22	73.00	97.80	0.00	0.000	0.000	-0.000	0.000	5.333	0.00

Pile Damping 2.0 %, Time Incr 0.281 ms, Wave Speed 11800.0 ft/s, 2L/c 12.4 ms

OAKLAND ARMY BASE; Pile: P9A; TIMBER 51.5; Blow: 3 (Test: 01-Nov-2013 14:03:)
GRL Engineers, Inc.
08-Nov-2013
CAPWAP(R) 2006-3







CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 360.0; along Shaft 90.0; at Toe 270.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				360.0				
1	7.0	2.0	0.0	360.0	0.0	0.00	0.00	0.000
2	14.0	9.0	13.9	346.1	13.9	1.99	0.52	0.076
3	21.0	16.0	18.7	327.4	32.6	2.67	0.77	0.076
4	28.0	23.0	26.1	301.3	58.7	3.73	1.19	0.076
5	35.0	30.0	31.3	270.0	90.0	4.47	1.61	0.076
Avg. Shaft			18.0			3.00	0.90	0.076
Toe			270.0				184.06	0.070

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.350	0.715
Case Damping Factor		0.201	0.551
Unloading Quake	(% of loading quake)	110	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	7	
Soil Plug Weight	(kips)		0.23
Soil Support Dashpot		0.000	10.000
Soil Support Weight	(kips)	0.00	3.14

CAPWAP match quality = 4.60 (Wave Up Match) ; RSA = 0
Observed: final set = 0.150 in; blow count = 80 b/ft
Computed: final set = 0.183 in; blow count = 66 b/ft
Replay Factor: F3:5.500; F4:5.500;
V3:1.000; V4:1.000;

max. Top Comp. Stress = 2.5 ksi (T= 21.7 ms, max= 1.000 x Top)
max. Comp. Stress = 2.5 ksi (Z= 3.5 ft, T= 21.7 ms)
max. Tens. Stress = -0.19 ksi (Z= 35.0 ft, T= 70.0 ms)
max. Energy (EMX) = 35.5 kip-ft; max. Measured Top Displ. (DMX)= 1.34 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.5	525.4	-10.2	2.5	-0.05	35.48	11.5	1.302
2	7.0	477.0	-16.7	2.3	-0.08	35.01	12.1	1.268
3	10.5	464.1	-22.3	2.2	-0.11	34.53	12.1	1.234
4	14.0	476.7	-26.8	2.3	-0.13	33.77	11.4	1.182
5	17.5	459.2	-27.8	2.2	-0.13	31.26	11.3	1.135
6	21.0	452.8	-33.1	2.1	-0.16	30.68	11.6	1.093
7	24.5	411.7	-34.5	1.9	-0.16	27.84	12.2	1.051
8	28.0	403.5	-39.4	1.9	-0.19	27.29	12.2	1.009
9	31.5	372.1	-38.3	1.8	-0.18	23.83	12.9	0.962
10	35.0	369.5	-41.1	1.7	-0.19	20.75	13.5	0.901
Absolute	3.5			2.5			(T =	21.7 ms)
	35.0				-0.19		(T =	70.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	430.6	386.4	342.2	298.1	253.9	209.7	165.6	121.4	77.2	33.0
RX	430.6	394.6	372.5	368.6	364.7	361.9	359.5	357.8	356.3	355.1
RU	430.6	386.4	342.2	298.1	253.9	209.7	165.6	121.4	77.2	33.0

RAU = 323.0 (kips); RA2 = 310.8 (kips)

Current CAPWAP Ru = 360.0 (kips); Corresponding J(RP)= 0.16; J(RX) = 0.58

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
11.38	21.67	358.7	513.6	525.3	1.338	0.149	0.150	36.4	586.4

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	211.24	1618.4	75.000	4.294
35.00	211.24	1618.4	75.000	2.618

Toe Area 1.467 ft^2

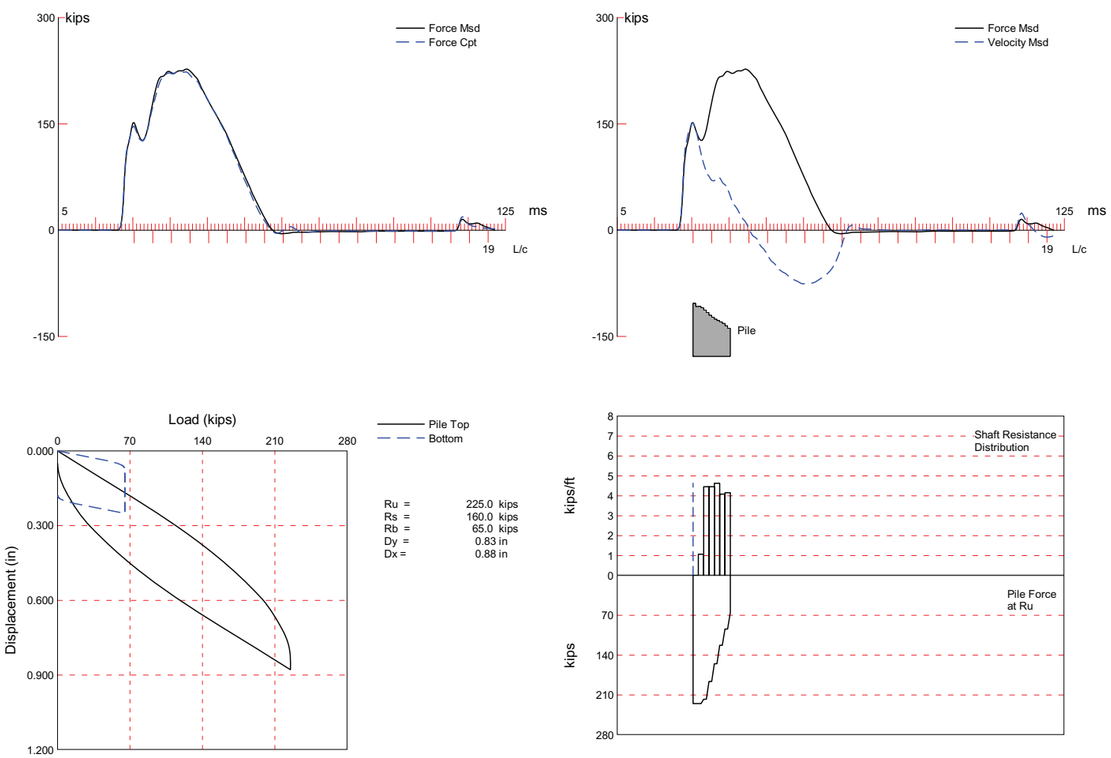
Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
1	3.50	34.20	0.00	0.000	0.000	-0.000	0.000	4.294	0.00
2	7.00	49.72	45.40	0.000	0.000	-0.000	0.000	4.043	0.04

OAKLAND ARMY BASE; Pile: P9A — Test: 01-Nov-2013 14:03:
TIMBER 51.5; Blow: 3 — CAPWAP(R) 2006-3
GRL Engineers, Inc. — OP: AK

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Tension Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft	Soil Plug kips
3	10.50	49.20	43.88	0.000	0.000	-0.000	0.000	3.875	0.04
4	14.00	31.63	-7.50	0.000	0.000	-0.000	0.000	3.707	0.04
5	17.50	34.15	-0.13	0.000	0.000	-0.000	0.000	3.540	0.04
6	21.00	37.76	10.42	0.000	0.000	-0.000	0.000	3.372	0.04
7	24.50	36.34	6.27	0.000	0.000	-0.000	0.000	3.205	0.04
8	28.00	36.52	6.80	0.000	0.000	-0.000	0.000	3.037	0.04
9	31.50	30.30	-11.39	0.000	0.000	-0.000	0.000	2.869	0.04
10	35.00	23.62	-30.93	0.000	0.000	-0.000	0.000	2.702	0.04

Pile Damping 3.4 %, Time Incr 0.417 ms, Wave Speed 8400.0 ft/s, 2L/c 8.3 ms

OAKLAND ARMY BASE; Pile: P9A1; BH; Blow: 2 (Test: 31-Oct-2013 16:23:)
GRL Engineers, Inc.
31-Oct-2013
CAPWAP(R) 2006-3



CAPWAP(R) 2006-3 Licensed to GRL Engineers, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 225.0; along Shaft 160.0; at Toe 65.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				225.0					
1	7.0	7.0	0.0	225.0	0.0	0.00	0.00	0.000	0.167
2	14.0	14.0	7.5	217.5	7.5	1.07	0.31	0.207	0.167
3	21.0	21.0	31.2	186.3	38.7	4.46	1.33	0.207	0.167
4	28.0	28.0	31.2	155.1	69.9	4.46	1.40	0.207	0.167
5	35.0	35.0	32.4	122.7	102.3	4.63	1.53	0.207	0.167
6	42.0	42.0	28.6	94.1	130.9	4.09	1.43	0.207	0.167
7	49.0	49.0	29.1	65.0	160.0	4.16	1.54	0.207	0.155
Avg. Shaft			22.9			3.27	1.03	0.207	0.165
Toe			65.0				58.28	0.400	0.053

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.498	1.178
Unloading Quake	(% of loading quake)	100	97
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	15	
Soil Plug Weight	(kips)		0.40

CAPWAP match quality = 2.45 (Wave Up Match) ; RSA = 0
Observed: final set = 0.050 in; blow count = 240 b/ft
Computed: final set = 0.089 in; blow count = 135 b/ft

max. Top Comp. Stress = 1.4 ksi (T= 37.9 ms, max= 1.283 x Top)
max. Comp. Stress = 1.8 ksi (Z= 21.0 ft, T= 36.8 ms)
max. Tens. Stress = -0.14 ksi (Z= 21.0 ft, T= 66.1 ms)
max. Energy (EMX) = 9.8 kip-ft; max. Measured Top Displ. (DMX)= 0.74 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.5	227.1	-7.7	1.4	-0.05	9.76	7.1	0.724
2	7.0	231.9	-10.6	1.5	-0.07	9.34	7.0	0.678
3	10.5	236.0	-13.5	1.6	-0.09	8.93	6.6	0.632
4	14.0	239.1	-16.3	1.7	-0.12	8.50	6.2	0.585
5	17.5	230.9	-15.6	1.7	-0.12	7.51	5.7	0.537
6	21.0	232.9	-17.8	1.8	-0.14	7.08	5.2	0.487
7	24.5	198.0	-8.8	1.6	-0.07	5.10	4.7	0.441
8	28.0	199.8	-10.9	1.7	-0.09	4.74	4.2	0.393
9	31.5	168.0	-3.7	1.5	-0.03	3.34	3.8	0.349
10	35.0	169.8	-4.7	1.6	-0.04	3.06	3.4	0.305
11	38.5	138.5	-0.0	1.4	-0.00	2.11	3.1	0.267
12	42.0	138.9	-0.0	1.5	-0.00	1.91	2.6	0.228
13	45.5	111.8	-0.0	1.3	-0.00	1.37	1.9	0.195
14	49.0	112.1	-0.0	1.4	-0.00	1.00	1.6	0.159
Absolute	21.0			1.8			(T =	36.8 ms)
	21.0				-0.14		(T =	66.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	231.8	225.4	219.1	212.7	206.4	200.0	193.7	187.3	180.9	174.6
RX	257.8	254.5	251.2	248.0	244.7	241.5	238.2	235.0	231.8	229.3
RU	231.8	225.4	219.1	212.7	206.4	200.0	193.7	187.3	180.9	174.6

RAU = 151.2 (kips); RA2 = 216.2 (kips)

Current CAPWAP Ru = 225.0 (kips); Corresponding J(RP)= 0.11; matches RX9 within 5%

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
6.95	25.71	147.1	148.2	227.5	0.743	0.050	0.050	9.9	298.9

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	160.61	1347.1	65.000	3.744
49.00	78.54	1347.1	65.000	2.618

Toe Area 1.115 ft^2

Segmnt Number	Dist. B.G. ft	Impedance kips/ft/s	Imped. Change %	Slack in	Tension Eff.	Compression Slack in	Compression Eff.	Perim. ft
1	3.50	22.08	0.00	0.000	0.000	-0.000	0.000	3.744
2	7.00	20.73	-0.69	0.000	0.000	-0.000	0.000	3.623
3	10.50	20.79	3.60	0.000	0.000	-0.000	0.000	3.543
4	14.00	20.21	4.92	0.000	0.000	-0.000	0.000	3.462
5	17.50	19.22	4.14	0.000	0.000	-0.000	0.000	3.382
6	21.00	18.25	3.40	0.000	0.000	-0.000	0.000	3.302
7	24.50	17.10	1.52	0.000	0.000	-0.000	0.000	3.221
8	28.00	16.24	1.26	0.000	0.000	-0.000	0.000	3.141
9	31.50	15.46	1.50	0.000	0.000	-0.000	0.000	3.060
10	35.00	14.86	3.01	0.000	0.000	-0.000	0.000	2.980
11	38.50	14.35	5.36	0.000	0.000	-0.000	0.000	2.899
12	42.00	13.74	7.23	0.000	0.000	-0.000	0.000	2.819
13	45.50	12.75	6.18	0.000	0.000	-0.000	0.000	2.739
14	49.00	11.69	4.36	0.000	0.000	-0.000	0.000	2.658

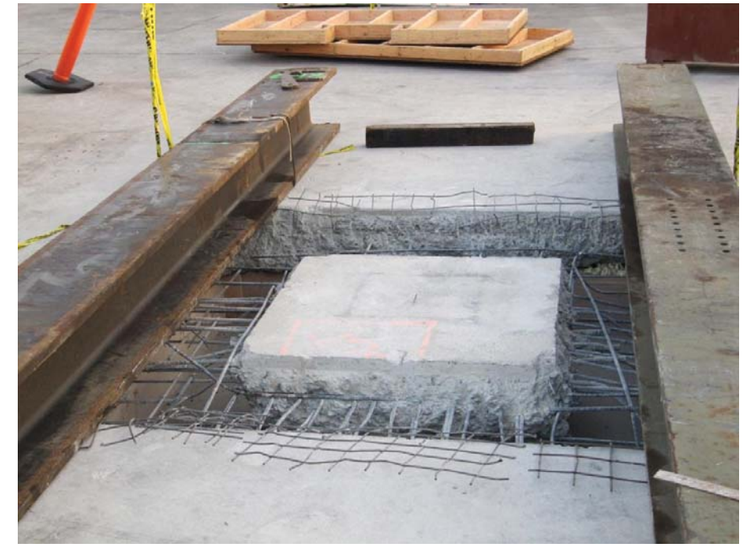
Pile Damping 2.0 %, Time Incr 0.357 ms, Wave Speed 9800.0 ft/s, 2L/c 10.0 ms

APPENDIX C

PHOTOGRAPHS FROM CPT AND PLT FIELD PROGRAMS

CPT Exploration Program

PLT Equipment Positioned for Testing at Pile Location 7A

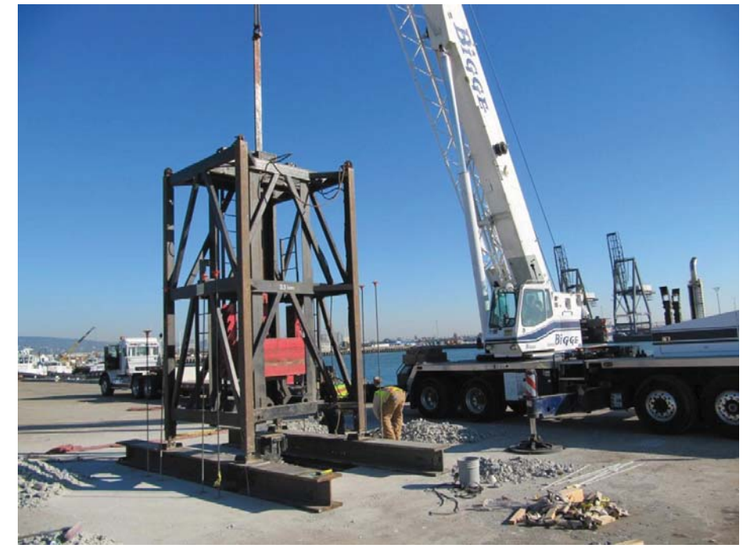


PLT Location 7A - Pile Cap Prepared for Dynamic Load Testing

8-ton Apple Load Test Equipment Set Up at PLT Location 7A

Attachment of PLT Instrumentation at Pile Location 7B

Instrumented PLT Pile 9A - Timber Pile with Gunite Jacket



PLT Equipment Set Up at Pile Location 8B

Pile Driving Analyzer Instrumentation Readout Box