Title

Full-scale Lateral Load Testing of Pier 3 at the Port of Long Beach

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FULL-SCALE LATERAL LOAD TESTING OF PIER 3 AT THE PORT OF LONG BEACH

By

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INTRODUCTION

Before the release of the *Seismic Design Criteria for California Marine Oil Terminals* (Ferrito et. al, 1999), the seismic design of piers and wharves was a non-uniform procedure. Design practices of the past typically underestimated earthquake intensities, a fact that has become clear after seismic events such as the Loma Prieta (1989) and Northridge (1994) earthquakes. Based on the damage to port facilities observed in such events, the Marine Facilities Division (MFD) of the California State Lands Commission, with funding through FEMA and the California Office of Emergency Services, is developing specific regulations for the seismic performance of marine oil terminals in California. The goals of the criteria established by Ferrito et al., (1999) are to (i) ensure safe and pollution-free transfer of petroleum products between ship and land based facilities, (ii) ensure the best achievable protection of public health, safety, and the environment, and (iii) maximize utilization of limited resources. A major component in the effort to realize these goals is the development and implementation of standardized design criteria, hence, the *Seismic Design Criteria for California Marine Oil Terminals.*

Strength design, the philosophy adopted in the new criteria, focuses on "restricting inelastic actions to carefully defined and detailed plastic hinges in piles". The overall performance of a pier directly relates to the displacement capacity of these plastic hinges. In order to ensure acceptable response of piles through proper design of plastic hinges, the criteria specify strain limits within plastic hinges for various pile types (see attached table "Structural Performance Limit State Strains"). Before these strain levels are implemented into the MFD regulations, testing needs to be carried out in order to verify that piles undergoing these strain levels will meet the specific criteria, i.e. "some inelastic behavior, with repairable damage," and that collapse is prevented. The planned demolition of Pier 3 at the POLB, though of relatively old construction, provides and opportunity to verify these strain levels in a full-scale test. Once the performance of the pile foundations at Pier 3 is verified, MFD can proceed with finalization of the regulations.

The second objective of the testing program is to back-calculate horizontal soil springs for silts to use in the analysis of laterally loaded piles. Since many of the foundations soils in ports along the Pacific coast of the US consist of silty soils, verification of these soil springs (e.g. in the form of horizontal modulus of subgrade reaction or *p-y* curves) is important to the efficient rebuilding of our port system. The demolition of Pier 3 provides an ideal opportunity to carry out testing in order to quantify the response of silt to laterally loaded piles.

TESTING PROGRAM

Four specimens were prepared for testing by the Port of Long Beach: a single pile, a 3-pile group, an 8-pile group, and a 24-pile group (see attached pictures "Single Pile & 3-Pile Group" and "8-Pile & 24-Pile Group" and figures "Demolition Detail", "Plan Detail" and "Typical Sections"). Individual piles are 20-inch square precast concrete with eight one-inch diameter smooth steel reinforcing bars and nominal transverse reinforcement. The specimens were then subjected to ambient vibration testing, forced vibration testing, and lateral load testing.

Material Properties

Due to the age of the pier, material properties could not be determined from the drawings. Compression testing of cored samples of concrete indicated a range of strengths much higher than specified in the plans and testing of the steel reinforcement was not possible. Therefore, a range of concrete and steel strengths were considered during analysis.

Ambient Vibration and Forced Vibration Testing

Vibration testing of the specimens was conducted to assist in verifying the accuracy of the analytical models created for post-test analyses. By arranging accelerometers in the x and y-directions at the corners of each specimen, the natural frequencies in the respective directions could be found. Then, the natural frequencies of the actual pile groups could be compared to those obtained using SAP 2000.

Lateral Load Testing

Quasi-static lateral load tests were conducted on the single pile, 3-pile group, and 8-pile group in order to determine the load versus displacement characteristics and concrete strains. A 500-kip capacity, 4-foot stroke, portable actuator was used to apply the lateral load to each specimen (see attached picture "Actuator").

The specimens were instrumented with waterproof concrete strain gauges and string-activated linear potentiometers along the length of the pile, and tiltmeters placed on the deck surface of each specimen. The single pile included additional tiltmeters and steel strain gauges mounted on a 48-foot long piece of steel angle that was inserted into a cored hole in the center of the pile (see attached figures "Instrumentation Setup").

Cyclic testing of the 8-pile group and 3-pile group was conducted at displacement ductilities of $\mu = 1$, $\mu = 2$, and $\mu = 4$. Monotonic testing of the single pile was conducted at displacement ductilites of $\mu = 1$, $\mu = 1.5$, $\mu = 2$, $\mu = 3$, $\mu = 4$, and $\mu = 6$ (see attached pictures "Lateral Load Test Single Pile" and "Lateral Load Test 3-Pile Group").

TEST RESULTS

Ambient Vibration and Forced Vibration Testing

Power spectrum plots from the ambient vibration testing and forced vibration testing were initially compared to each other before comparison with the analytical models (see attached figures "Ambient Vibration Test" and "Forced Vibration Test").

Lateral Load Testing

Load versus displacement plots for the single pile, 8-pile group, and 3-pile group were generated during testing for comparison with those produced by the analytical models (see attached figure "Lateral Load Test: Single Pile"). In addition, displacement profiles for the single pile and 8-pile group were obtained.

ANALYSIS OF TEST RESULTS

In order to verify the performance of the piles (i.e. strain limit states) a series of analyses were performed, including: SEQMC moment-curvature analysis, SAP 2000 dynamic analysis, and LPILE pile response analysis. Also, a composite soil profile was developed based on data collected from Standard Penetration Tests (SPT) and Cone Penetrometer Tests (CPT) tests near the test site. Material properties and performance of the single pile were verified first, then the 3-pile and 8-pile groups.

Moment Curvature Analysis

Given the geometry of the pile section and the concrete material properties, the moment-curvature program SEQMC was used to determine the variation in pile stiffness for increasing applied moment for a single pile. This relationship would then be used in the following LPILE analysis.

Composite Soil Profile

Based on Standard Penetration Test (SPT) data from Diaz Yourman Associates and Cone Penetrometer Test (CPT) data from Gregg In Situ, Inc., a composite soil profile was developed for use in all applicable analyses. The soil profile used in the analyses was simplified into three layers, using average values for unit weight and friction angle from the CPT and SPT tests (see attached figure "Composite Soil Profile").

SAP 2000 Dynamic Analysis

Having determined the initial stiffness of the pile from the SEQMC moment-curvature analysis, a SAP model was developed to verify the natural frequency of the single pile. Winkler springs were used to model the soil. Values for these springs were based on equations developed by Rix and Stokoe (1991), Imai and Tonouchi (1982), Carter (1984), Ling (1988), and values from SPT and CPT testing. Models for the single pile, 8-pile group, and 3-pile group had errors of 10%, 5%, and 14%, respectively. These results were considered good and it was determined that the soil spring values used in the SAP analysis accurately represented the actual soil.

LPILE Analysis

The program LPILE was used to perform pile response analyses to determine load versus displacement, displacement versus depth, and moment versus depth characteristics. The three main variables in all of the analyses are the concrete strength, f'_c, the steel strength, f_y, and the soil properties. In order to determine the influence of each of these parameters on the results of an LPILE analysis, a parametric study was conducted. From the analyses it was determined that varying concrete strength and/or soil stiffness had very little effect on the response of the pile. Only variation of steel strength altered the load versus displacement curve significantly. Based on these findings a steel strength of f_y = 45 ksi and a concrete strength of f'_c = 8.5 ksi were determined to be realistic values to use for the 8-pile and 3-pile analyses. Also, the average values for friction angle obtained from the composite soil profile, $\__1 = 30^{\circ}$, $\__2 = 33^{\circ}$, $\__3 = 40^{\circ}$,were used in all further analyses.

The load versus displacement curves obtained using these material properties provided reasonable matches for the curves generated during testing of the actual specimens (see attached figures (3) "LPILE Analysis: Single Pile/8-Pile/3-Pile").

DISCUSSION

Instrumentation

Proper performance of the instrumentation on the single pile and 8-pile group was vital in order to achieve both of the testing objectives using data acquired directly from the tests. However, some instrumentation failed to operate correctly.

The concrete strain gauges were implemented to define strain profiles to be used for comparison with strain limits set forth by the *Seismic Criteria for California Marine Oil Terminals*. Unfortunately, the adhesive used to bond the gauges to the concrete surface never "set" completely and therefore the concrete strain in the piles was not fully transferred to the gauges and values recorded during testing could not be directly compared with the strain limits states.

The tiltmeters employed during the testing of the single pile were supposed to provide data that would be used to back-calculate soil spring information. However, only one tiltmeter located below the mudline functioned correctly. Thus, it is impossible to determine soil spring information directly from data obtained in the field.

Due to the malfunctioning instrumentation determination of concrete strains, steel strains, and soil properties was impossible. The use of analytical models became the only method for achieving the test objectives.

Strain Limit States

Due to instrumentation failure, the strain limit states set forth in the seismic design guidelines cannot be directly verified. However, use of ductility levels associated with Level 1 and Level 2 earthquakes, defined in *The Seismic Design and Retrofit of Bridges* (Priestly, Seible, Calvi), in conjunction with results from LPILE and SEQMC makes verification of strain limit states possible. The procedure is described below for the strain limit states associated with a Level 1 earthquake, but a similar one can be applied for a Level 2 earthquake. (see attached figures "SEQMC Output Data", "LPILE Moment vs. Depth" and table "Verification of Serviceability Limit States for single pile")

1. SEQMC produces a table of data including concrete strain, steel strain, and moment capacity for every increment. It also specifies the yield of the section based on a tension steel strain of $\_{s} = 0.002$.
2. From the SEQMC output, the moment at yield is determined, M_{yield}.
3. The potential hinge of interest is identified. For pile groups the pile/deck connection is considered the critical hinge. For the single pile, the critical hinge is located below the mudline.
4. LPILE produces moment versus depth plots for each pile-head displacement specified in the boundary conditions. By determining the displacement necessary to create the "yield moment" at the critical hinge of interest, the yield displacement can be determined, $\_{yield}$.
5. Based on the suggested displacement ductility of $\mu\_ = 2$ for the serviceability limit state, the pile-head displacement associated with a Level 1 earthquake (serviceability limit state) is determined (i.e. $\_{Level\ 1} = 2\_{yield}$).
6. $\_{Level\ 1}$ can then be input as one of the pile-head displacement boundary conditions, so that the moment at the critical hinge can be determined, $M_{Level\ 1}$.
7. Using the moment associated with a Level 1 earthquake the concrete compression strain and steel tension strain are found from the SEQMC output.

8. The concrete and steel strains are then compared to the serviceability limit state strains specified in the seismic design criteria.

Using this procedure it was found that the strain limit states for both the Level 1 and Level 2 earthquakes are satisfied for all specimens.

CONCLUSIONS

Based on the moment-curvature and pile response analyses performed on the single pile and pile groups, expected concrete and steel strains due to Level 1 and Level 2 earthquakes were less than the proposed strain limits of the new criteria. Thus, these results support implementation of the seismic design criteria. While this testing provides an idea of how an old pier stands up to the new criteria, future testing of fully instrumented pile specimens, constructed for testing, will enable researchers to verify the strain limit states for concrete and tension steel reinforcement directly.

The failure of the tiltmeters to function properly prevented the back-calculation of soil springs directly. However, it was found that equations developed for sands by Imai and Tonouchi (1982), and Rix and Stokoe (1991) provided soil springs values that resulted in close matches for the natural frequencies of the single pile and pile groups.

This test cannot single-handedly spur the implementation of the new design criteria, or define properties of silt-type soils. However, the successful collaboration between the Ports of Long Beach and Los Angeles and the State Lands Commission was the impetus for additional funding from the National Science Foundation in the amount of $2.4 million for further seismic testing of port structures.

STRUCTURAL PERFORMANCE LIMIT STATE STRAINS

(a) Serviceablility Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 1 earthquake shall not exceed:

Concrete extreme fiber compression strain:	
Pile/deck hinge:	0.004
In-ground hinge:	0.008
Reinforcing steel tension strain:	0.010

(b) Damage Control Limit State: Within potential plastic hinge regions, strains at maximum response to the Level 2 earthquake shall not exceed:

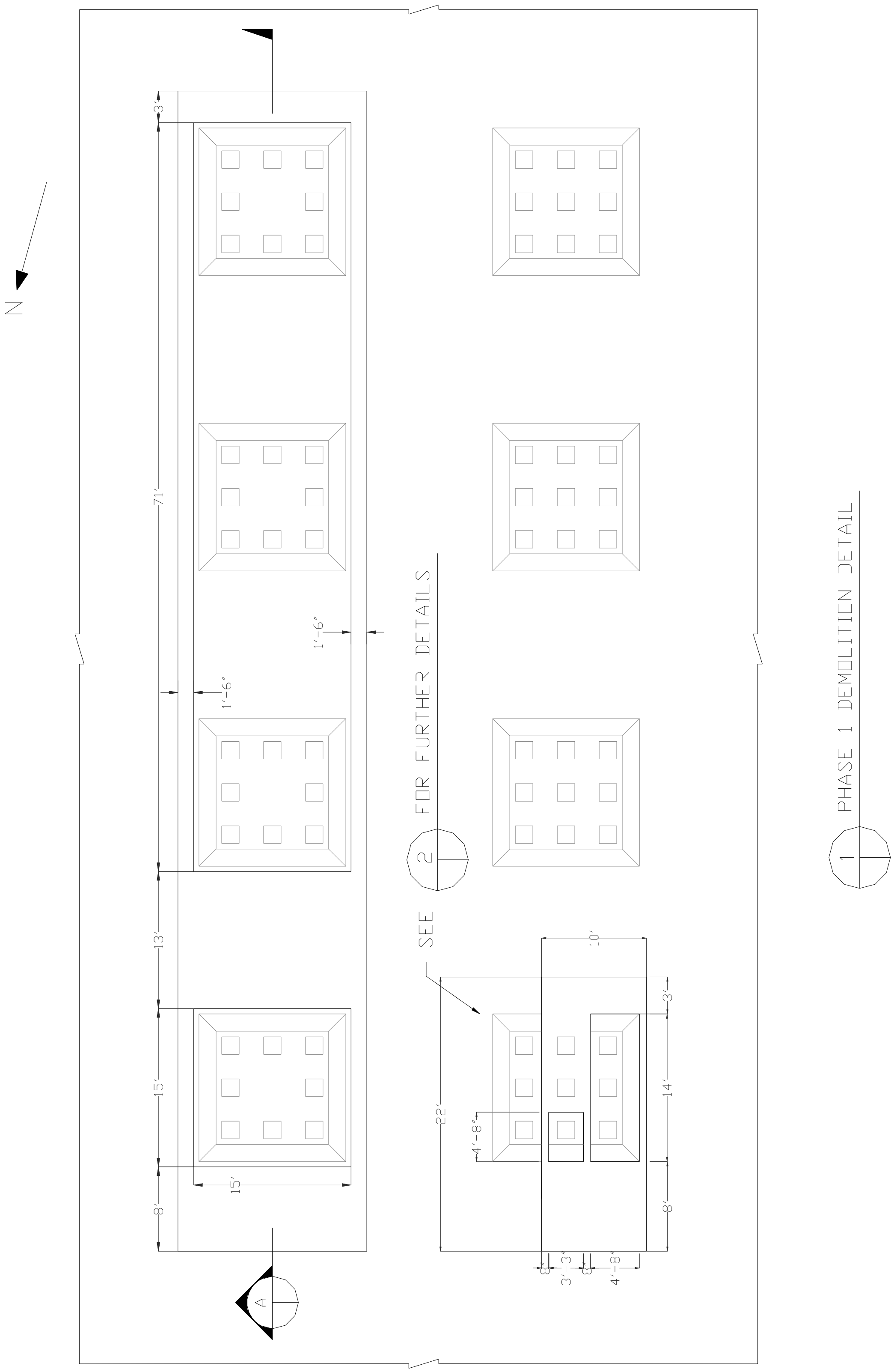
Concrete extreme fiber compression strain:	
Pile/deck hinge:	Value given by equation 7, but < 0.025
In-ground hinge:	Value given by equation 7, but < 0.008
Reinforcing steel tension strain:	
Pile/deck hinge:	0.05
In-ground hinge:	0.01

The design ultimate compression strain of confined concrete may be taken as

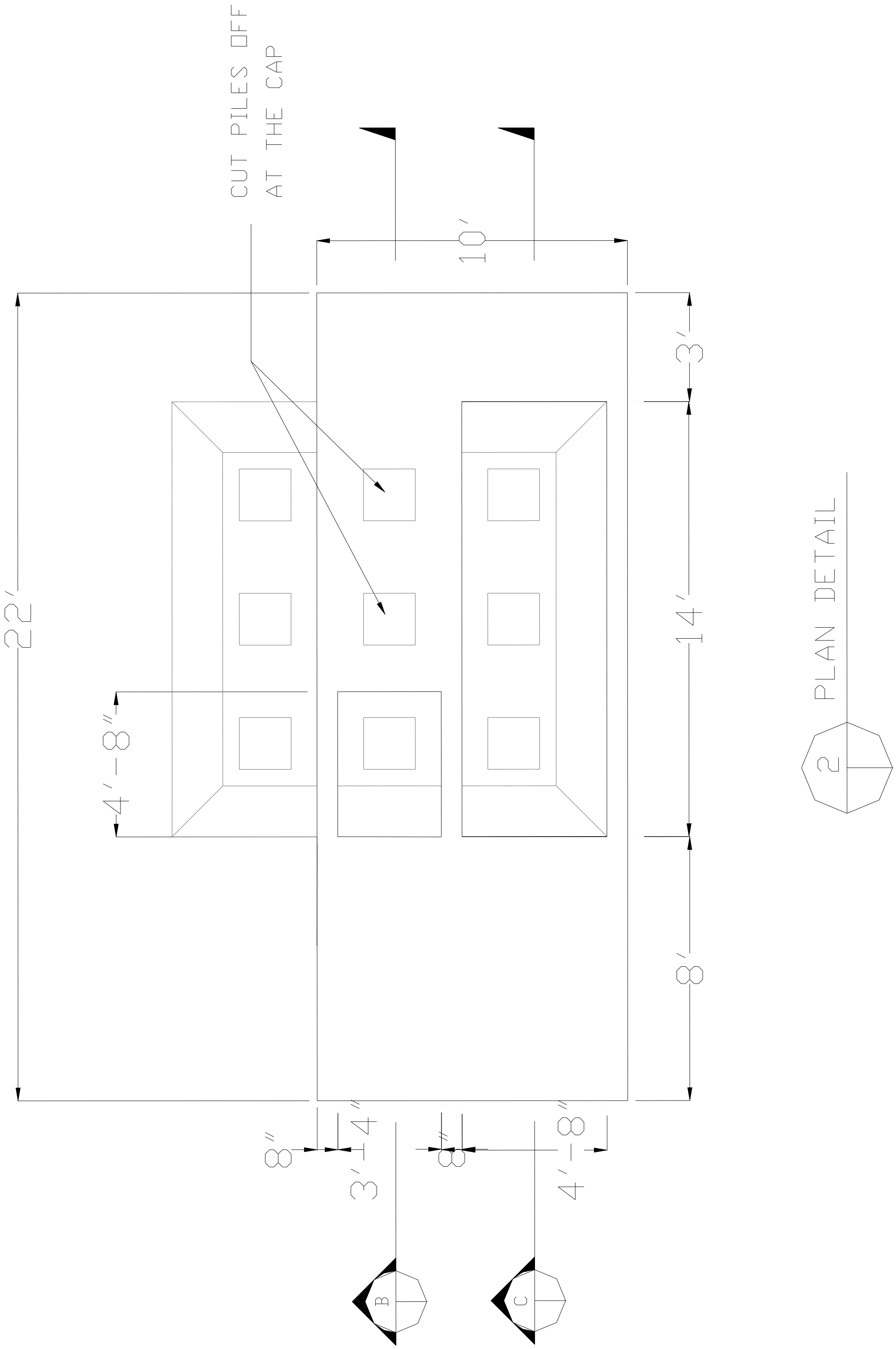
$$\varepsilon_{cu} = 0.004 + (1.4\rho_s f_{yh}\varepsilon_{sm}) / f'_{cc} \geq 0.005 \qquad (7)$$

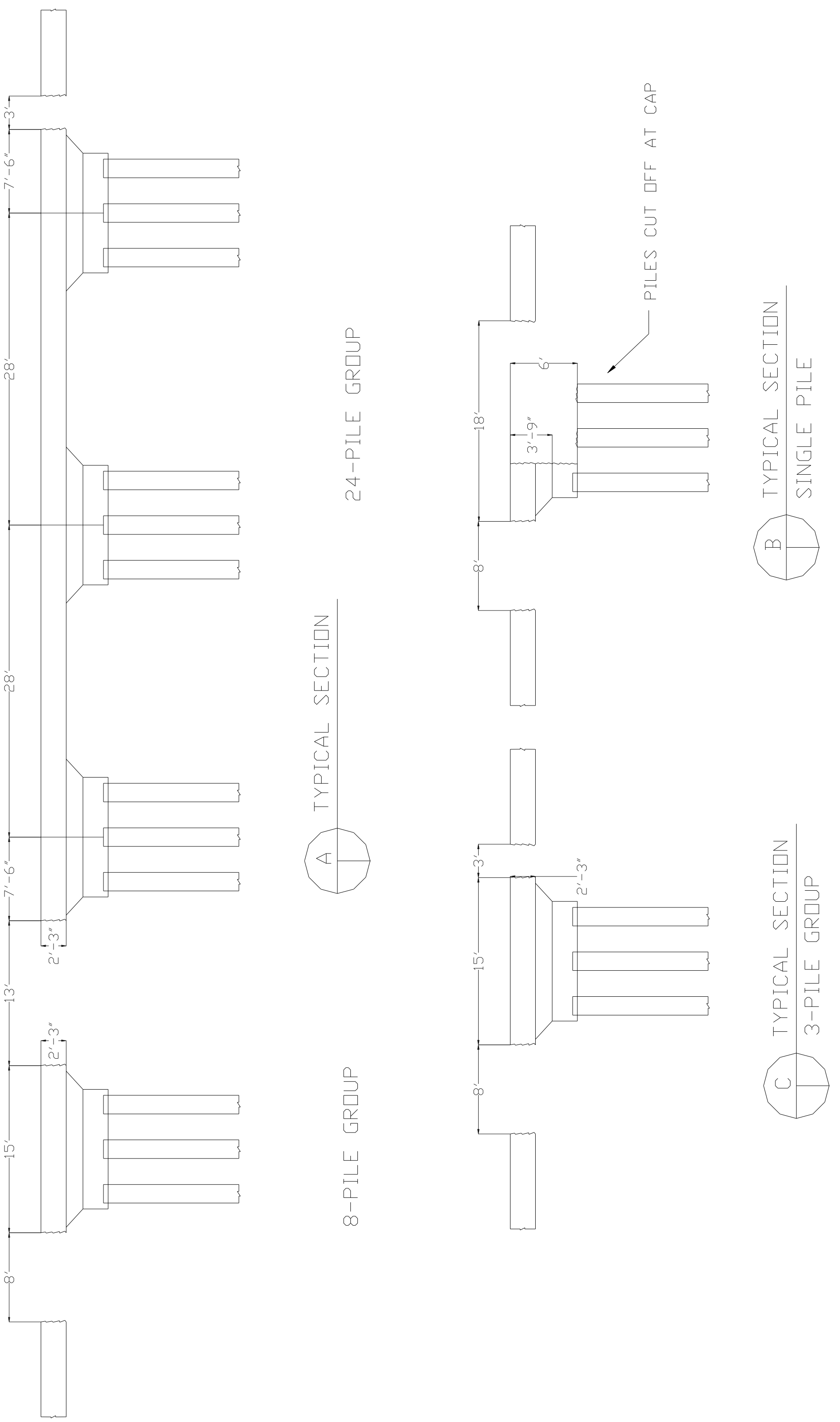
where

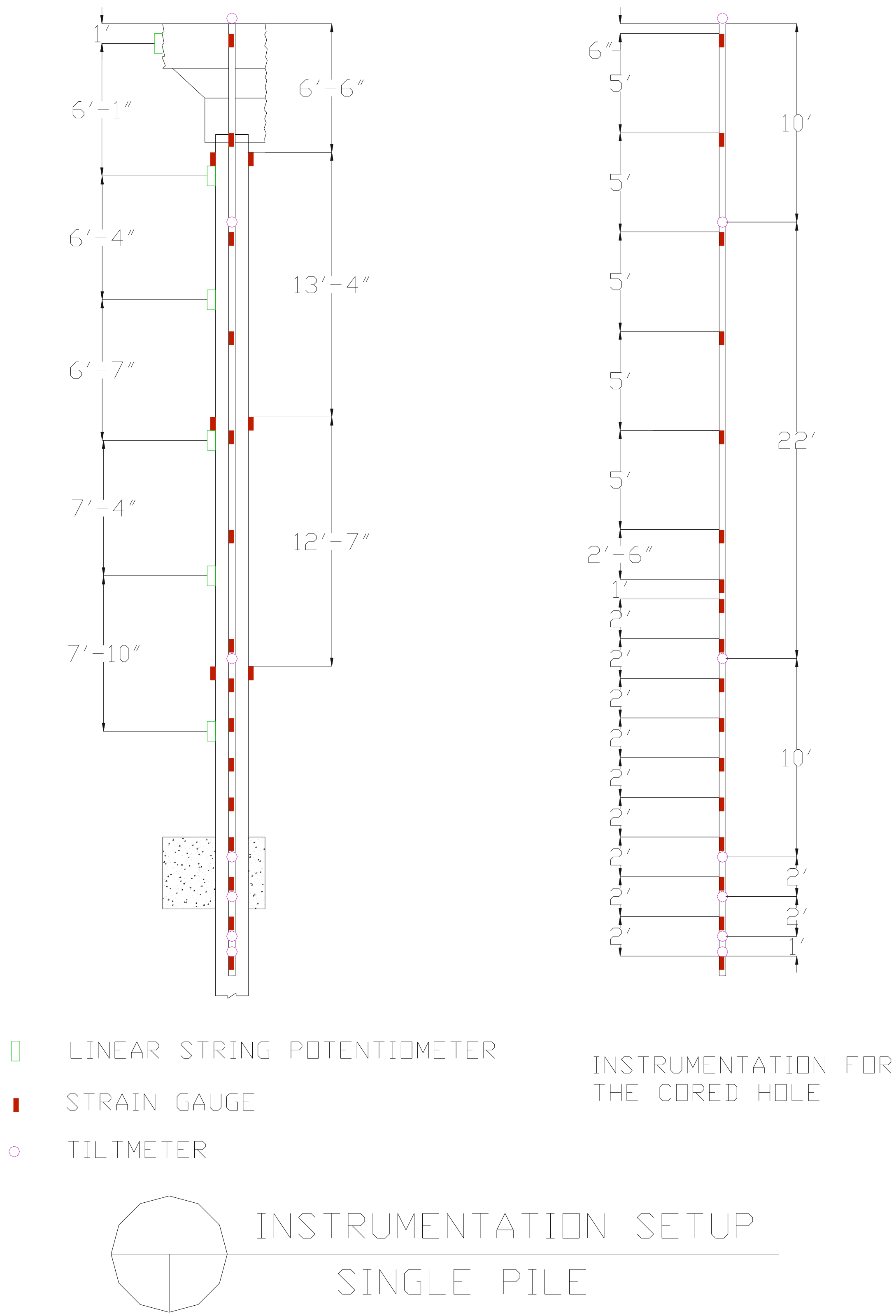
ρ_s	effective volume ratio of confining steel
f_{yh}	yield stress of confining steel
ε_{sm}	strain at peak stress of confining reinforcement, 0.15 for grade 40 and 0.12 for grade 60
f'_{cc}	confined strength of concrete approximated by $1.5f'_c$

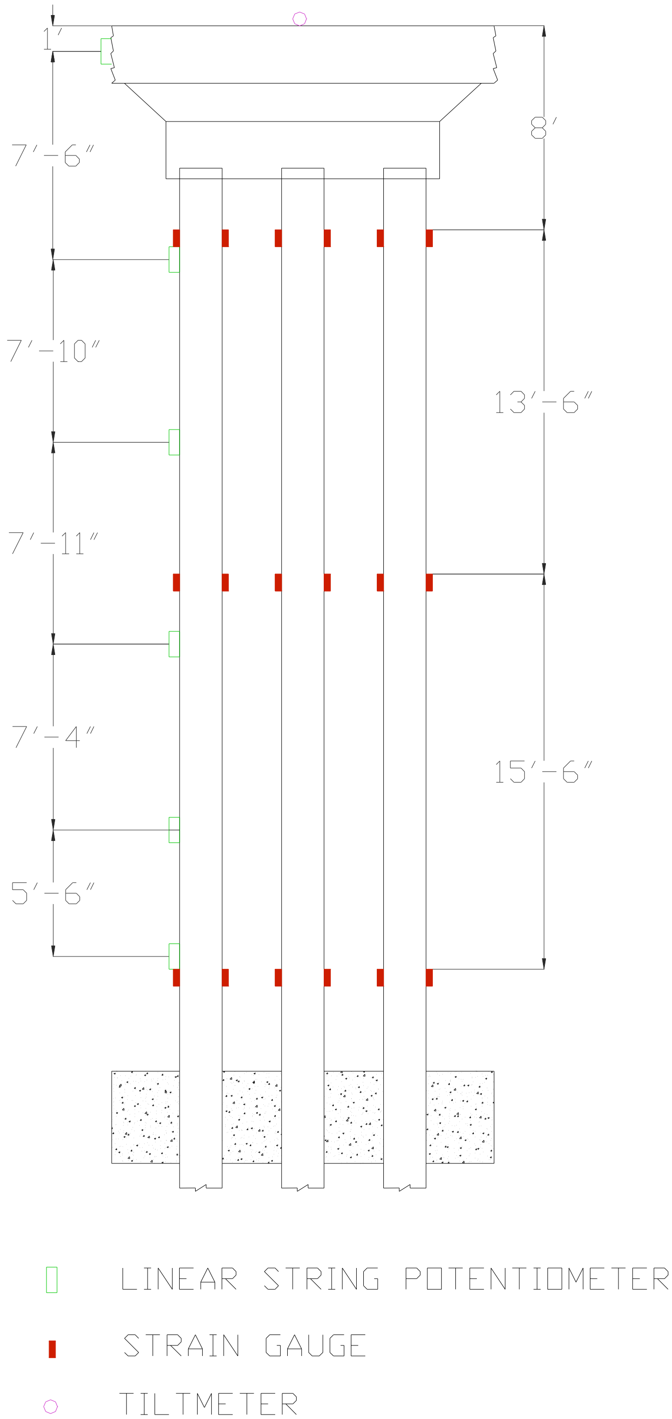


N
3'
71'
13'
15'
8'
15'
1'-6"
1'-6"
A
SEE 2 FOR FURTHER DETAILS
10'
3'
14'
8'
22'
4'-8"
8"
3'-3"
8"
4'-8"
1 PHASE 1 DEMOLITION DETAIL


CUT PILES OFF
AT THE CAP
10'
22'
4'-8"
3'
14'
8'
PLAN DETAIL
2
8"
3'-4"
8"
4'-8"
B
C

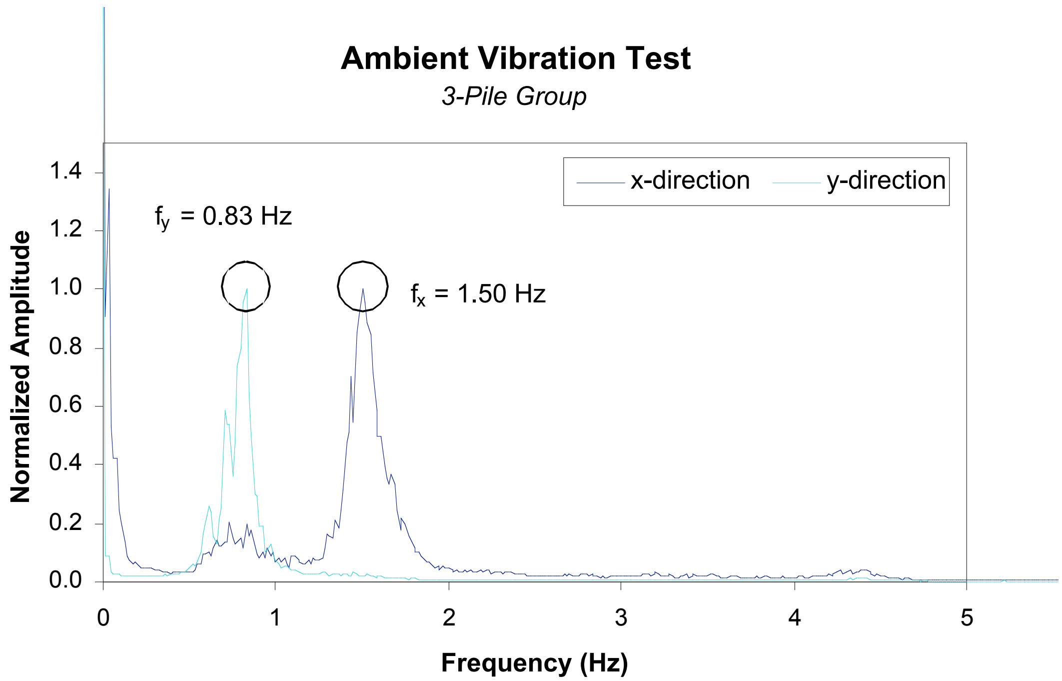

3'
7'-6"
28'
28'
7'-6"
2'-3"
13'
2'-3"
15'
8'
24-PILE GROUP
TYPICAL SECTION
A
8-PILE GROUP
18'
6'
3'-1/2"9"
8'
PILES CUT OFF AT CAP
TYPICAL SECTION
SINGLE PILE
B
3'
2'-3"
15'
8'
TYPICAL SECTION
3-PILE GROUP
C


1′
6′−1″
6′−4″
6′−7″
7′−4″
7′−10″
6′−6″
13′−4″
12′−7″
6″
5′
5′
5′
5′
5′
10′
22′
2′−6″
1′
2′
2′
2′
2′
2′
2′
2′
2′
2′
10′
2′
2′
1′
LINEAR STRING POTENTIOMETER
STRAIN GAUGE
TILTMETER
INSTRUMENTATION FOR THE CORED HOLE
INSTRUMENTATION SETUP
SINGLE PILE

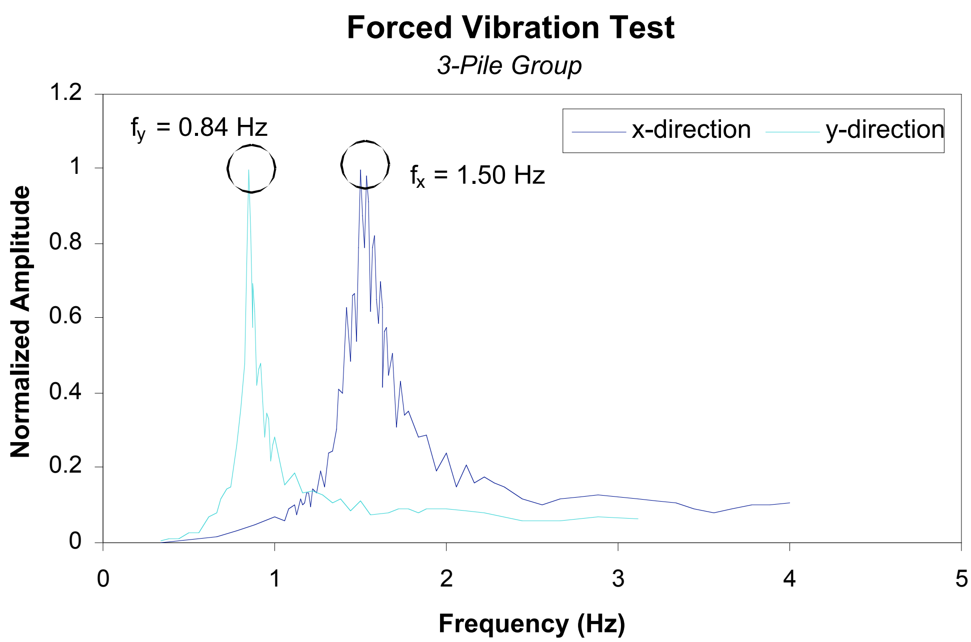


INSTRUMENTATION SETUP

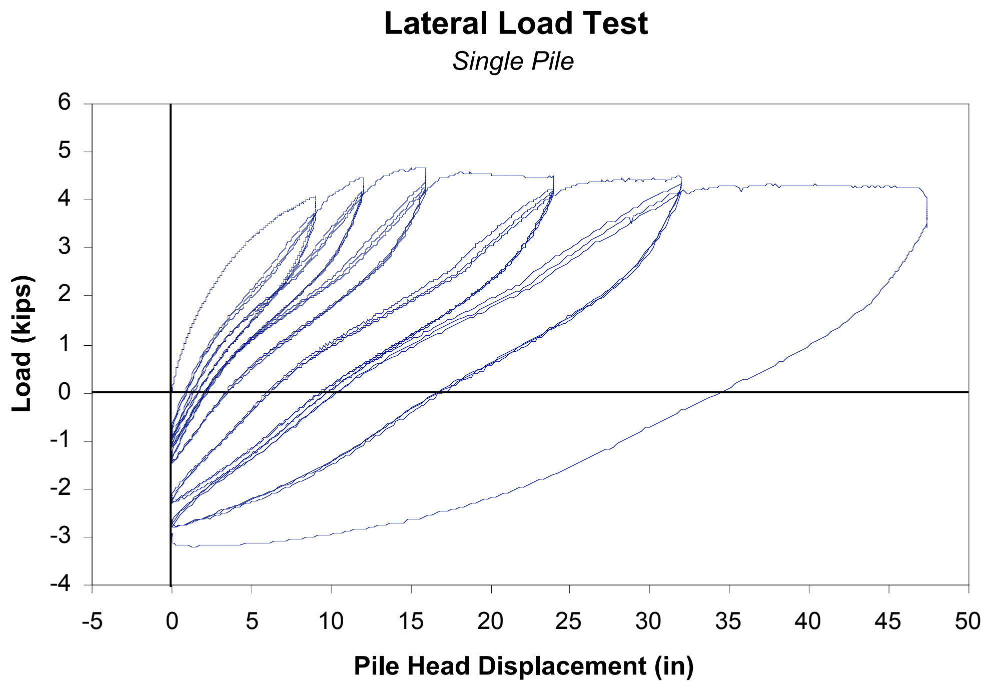
8-PILE GROUP

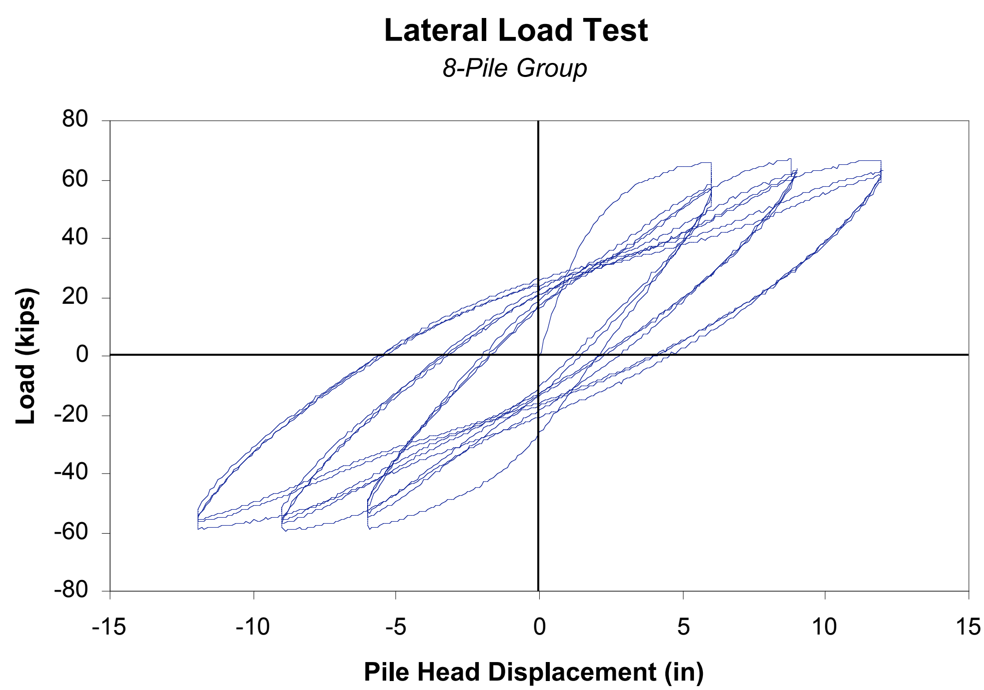


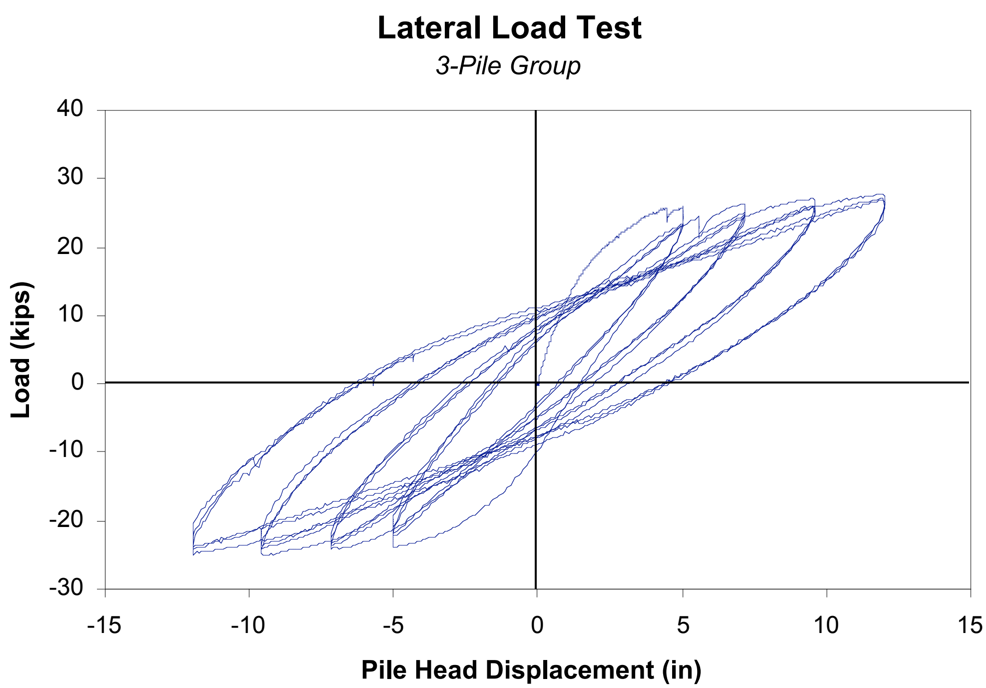
Ambient Vibration Test
3-Pile Group
x-direction
y-direction
f_y = 0.83 Hz
f_x = 1.50 Hz
Normalized Amplitude
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
1
2
3
4
5
Frequency (Hz)



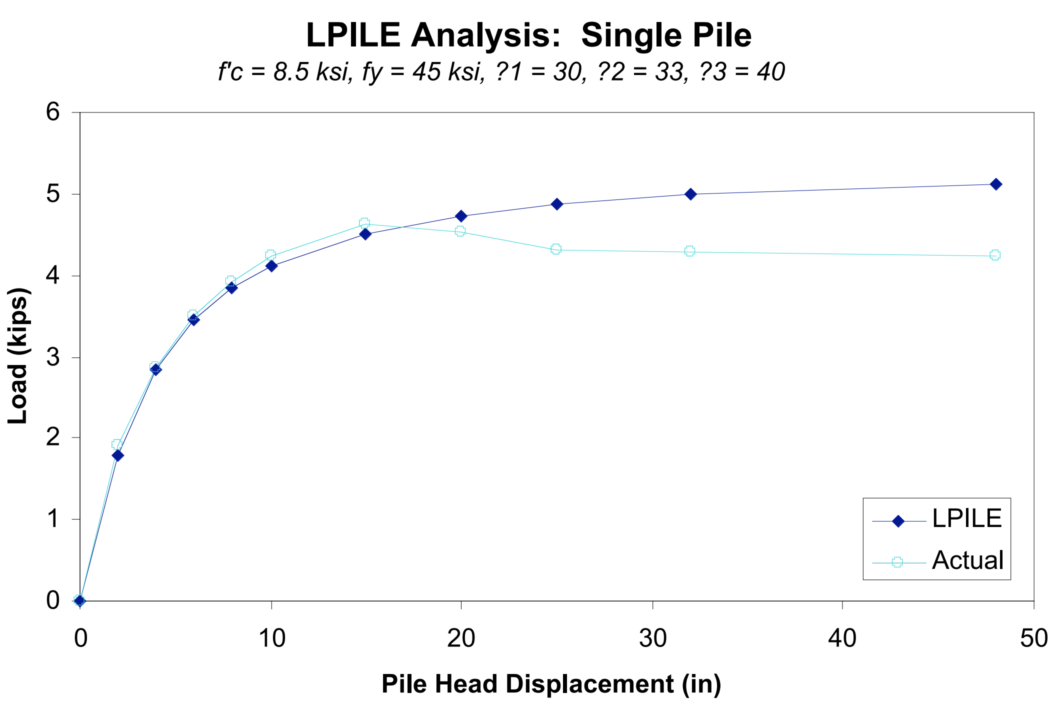
Forced Vibration Test
3-Pile Group
x-direction
y-direction
f_y = 0.84 Hz
f_x = 1.50 Hz
Normalized Amplitude
1.2
1
0.8
0.6
0.4
0.2
0
0
1
2
3
4
5
Frequency (Hz)


Lateral Load Test
Single Pile
6
5
4
3
2
1
0
-1
-2
-3
-4
Load (kips)
-5
0
5
10
15
20
25
30
35
40
45
50
Pile Head Displacement (in)

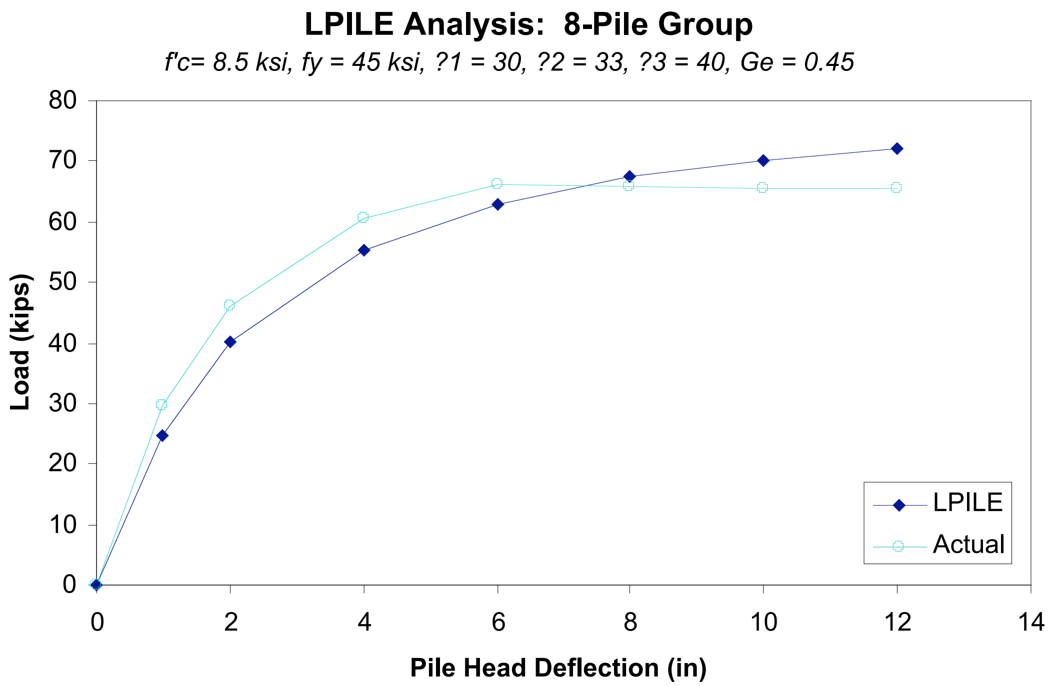

Lateral Load Test
8-Pile Group
80
60
40
20
0
-20
-40
-60
-80
Load (kips)
-15
-10
-5
0
5
10
15
Pile Head Displacement (in)

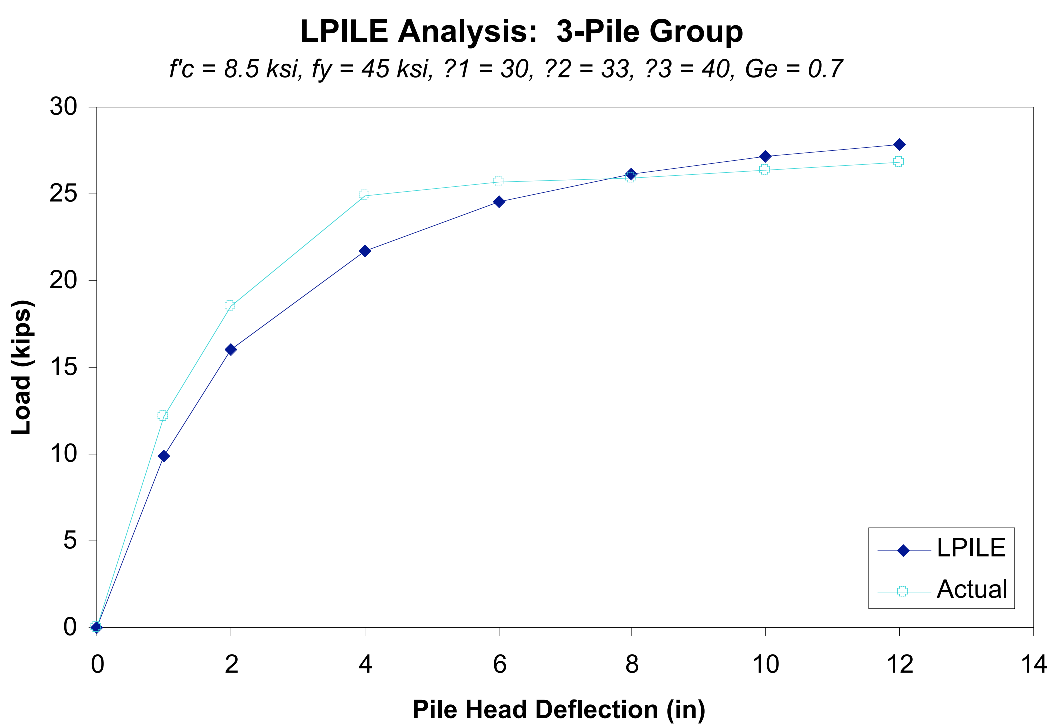


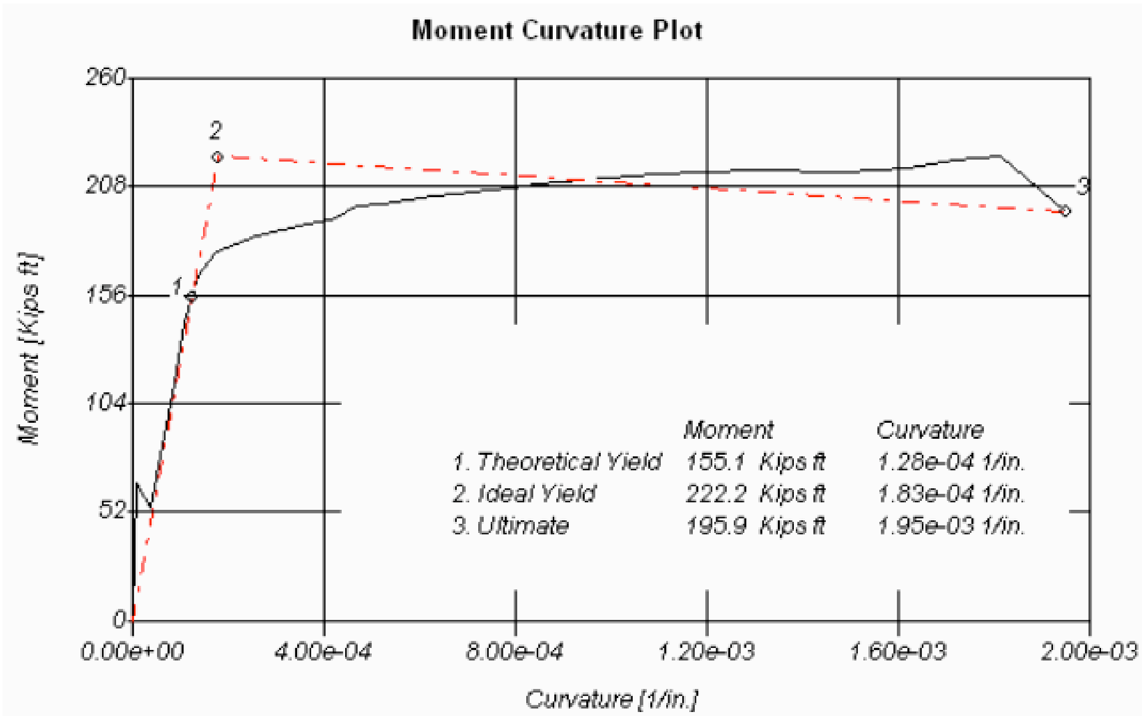
Lateral Load Test
3-Pile Group
Load (kips)
40
30
20
10
0
-10
-20
-30
-15
-10
-5
0
5
10
15
Pile Head Displacement (in)



LPILE Analysis: Single Pile
f'c = 8.5 ksi, fy = 45 ksi, ?1 = 30, ?2 = 33, ?3 = 40
Load (kips)
6
5
4
3
2
1
0
0
10
20
30
40
50
Pile Head Displacement (in)
LPILE
Actual

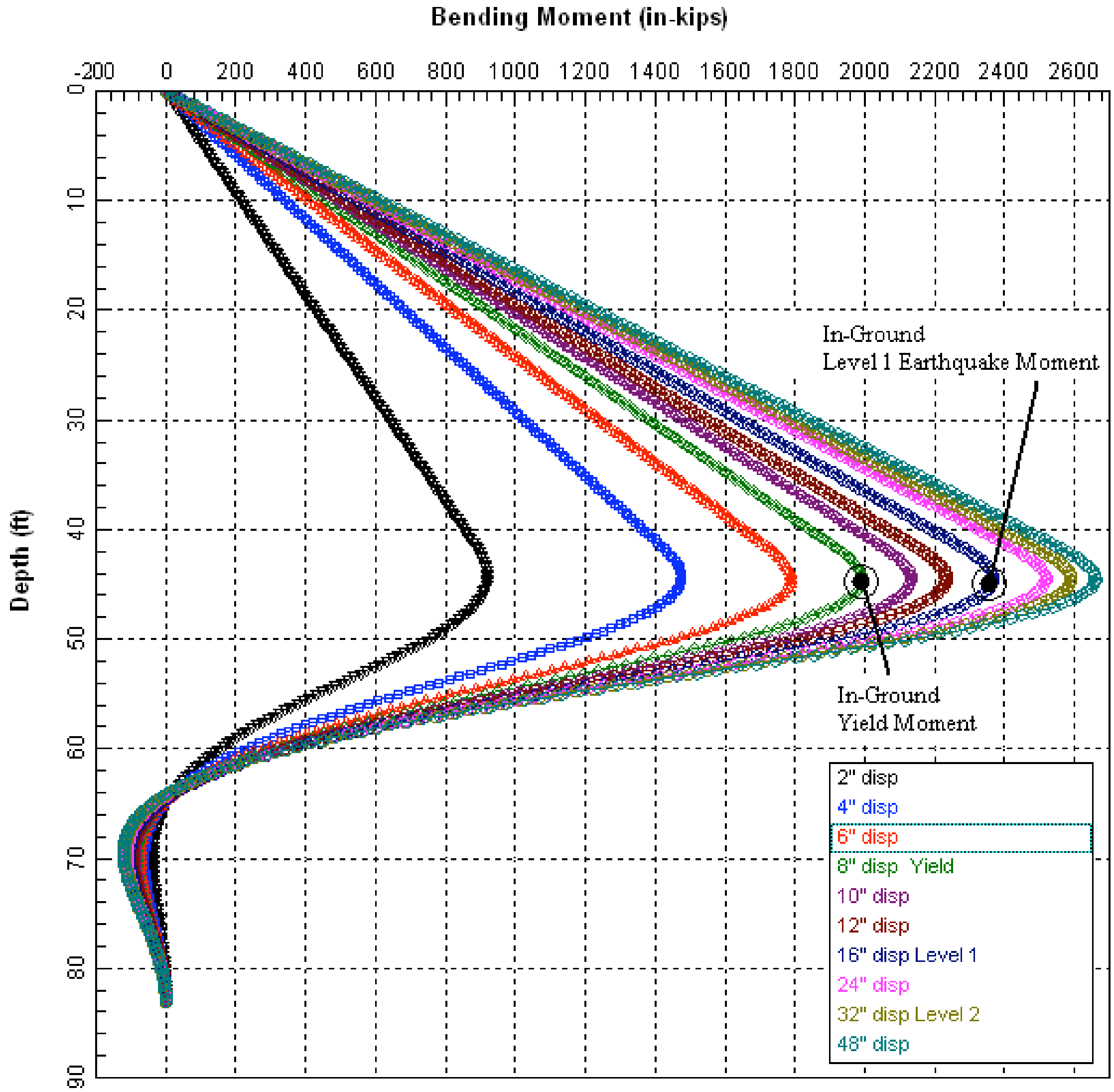

LPILE Analysis: 8-Pile Group
f'c= 8.5 ksi, fy = 45 ksi, ?1 = 30, ?2 = 33, ?3 = 40, Ge = 0.45
Load (kips)
Pile Head Deflection (in)
LPILE
Actual
0
10
20
30
40
50
60
70
80
0
2
4
6
8
10
12
14


LPILE Analysis: 3-Pile Group
f'c = 8.5 ksi, fy = 45 ksi, ?1 = 30, ?2 = 33, ?3 = 40, Ge = 0.7
Load (kips)
Pile Head Deflection (in)
LPILE
Actual
0
5
10
15
20
25
30
0
2
4
6
8
10
12
14


Moment Curvature Plot
260
208
156
104
52
0
Moment [Kips ft]
0.00e+00
4.00e-04
8.00e-04
1.20e-03
1.60e-03
2.00e-03
Curvature [1/in.]
1
2
3
Moment
Curvature
1. Theoretical Yield 155.1 Kips ft 1.28e-04 1/in.
2. Ideal Yield 222.2 Kips ft 1.83e-04 1/in.
3. Ultimate 195.9 Kips ft 1.95e-03 1/in.

Conc. Strain	N.A. Depth	Steel Strain	Moment Cap.	Curvature	
0.0001	10.0 in.	-0.00006	65.8 Kips ft	1.00e-05 1/in.	
0.0002	5.0 in.	-0.00046	54.3 Kips ft	4.00e-05 1/in.	
0.0003	4.6 in.	-0.00078	83.8 Kips ft	6.55e-05 1/in.	
0.0004	4.5 in.	-0.00107	113.1 Kips ft	8.93e-05 1/in.	
0.0005	4.4 in.	-0.00138	143.3 Kips ft	1.14e-04 1/in.	
0.0006	4.3 in.	-0.00170	165.3 Kips ft	1.40e-04 1/in.	__ Yield
0.0007	3.9 in.	-0.00223	176.8 Kips ft	1.77e-04 1/in.	
0.0008	3.6 in.	-0.00284	180.9 Kips ft	2.20e-04 1/in.	
0.0009	3.4 in.	-0.00341	184.6 Kips ft	2.61e-04 1/in.	
0.0010	3.2 in.	-0.00422	187.1 Kips ft	3.16e-04 1/in.	
0.0011	3.0 in.	-0.00494	189.6 Kips ft	3.66e-04 1/in.	
0.0012	2.9 in.	-0.00569	192.0 Kips ft	4.17e-04 1/in.	
0.0013	2.8 in.	-0.00645	198.2 Kips ft	4.70e-04 1/in.	
0.0014	2.7 in.	-0.00723	199.9 Kips ft	5.23e-04 1/in.	
0.0015	2.6 in.	-0.00800	201.5 Kips ft	5.76e-04 1/in.	
0.0016	2.5 in.	-0.00877	203.1 Kips ft	6.28e-04 1/in.	
0.0017	2.5 in.	-0.00953	204.5 Kips ft	6.81e-04 1/in.	
0.0018	2.4 in.	-0.01044	205.9 Kips ft	7.42e-04 1/in.	
0.0019	2.4 in.	-0.01124	207.2 Kips ft	7.96e-04 1/in.	
0.0020	2.4 in.	-0.01202	208.5 Kips ft	8.49e-04 1/in.	
0.0025	2.3 in.	-0.01558	213.5 Kips ft	1.10e-03 1/in.	
0.0030	2.3 in.	-0.01846	215.9 Kips ft	1.30e-03 1/in.	
0.0035	2.3 in.	-0.02110	214.6 Kips ft	1.49e-03 1/in.	
0.0040	2.5 in.	-0.02275	216.9 Kips ft	1.62e-03 1/in.	
0.0045	2.6 in.	-0.02388	220.0 Kips ft	1.72e-03 1/in.	
0.0050	2.8 in.	-0.02497	222.2 Kips ft	1.82e-03 1/in.	
0.0060	3.1 in.	-0.02621	195.9 Kips ft	1.95e-03 1/in.	

SEQMC Output Data (Level 1 Strain Limit States are circled)



LPILE Moment versus Depth Profile for the Single Pile

Moments at the In-Ground Hinge Corresponding to Yield and a Level 1 Earthquake

VERIFICATION OF SERVICEABILITY LIMIT STATE STRAINS FOR THE SINGLE PILE

Strains at the In-ground Hinge

Yield Moment from SEQMC, M_{yield}	M_{yield} = 165 k-ft = 1985 k-in
Displacement from LPILE corresponding to the Yield Moment, $\_{}_{yield}$	$\_{}_{yield}$ = 8 in
Displacement from LPILE corresponding to a displacement ductility, $\mu_{\_}$ = 2 (Serviceability Limit State), $\_{}_{Level\ 1}$	$\_{}_{Level\ 1}$ = 16 in
Moment from LPILE corresponding to a 16" displacement, $M_{Level\ 1}$	$M_{Level\ 1}$ = 197 k-ft = 2367 k-in
Steel Tension Strain from SEQMC corresponding to $M_{Level\ 1}$, $(\_{}_{s})_{Level\ 1}$	**$(\_{}_{s})_{Level\ 1}$ = 0.006 < 0.01**
Concrete Compression Strain from SEQMC corresponding to $M_{Level\ 1}$, $(\_{}_{c})_{Level\ 1}$	**$(\_{}_{c})_{Level\ 1}$ = 0.0013 < 0.008**