ATTACHMENT G

FOUNDATION REPORT

Lower Sacramento Road Bridge
at Bear Creek (Replace)
Bridge No. 29C0443
Stockton, California

Prepared by:

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June 30, 2010

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File No. 879.5
June 30, 2010

Ms. Julie Passalacqua
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Sacramento, CA 95826

Subject: **FOUNDATION REPORT**
Lower Sacramento Road Bridge at Bear Creek (Replace)
Bridge No. 29C0443
Stockton, California

Dear Ms. Passalacqua:

Blackburn Consulting (BCI) is pleased to submit this Foundation Report for the Lower Sacramento Road Bridge at Bear Creek in Stockton, California. BCI prepared this report in accordance with our December 10, 2007 Subconsultant Amendment 1 to our original May 2, 2006 agreement. This report contains our subsurface findings, conclusions and recommendations for bridge design.

Please call if you have questions or require additional information.

Sincerely;

BLACKBURN CONSULTING

Reviewed by:

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Stockton, California

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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Draft Foundation Report for the Lower Sacramento Road Bridge at Bear Creek in Stockton, California. It contains our subsurface findings, conclusions and recommendations for bridge design.

This report is for the project design team and City of Stockton to use during bridge design. It shall not be used or relied upon by others, or for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI performed the following:

1. Discussed the project with Julie Passalacqua and Lance Schrey of Mark Thomas & Company (MTCo).
2. Reviewed the General Plan, Foundation Plan, and foundation loads for the bridge structure prepared and provided by MTCo.
3. Reviewed a “General Plan – Profile”, “Site Plan”, and “Footing & Foundation Plan” for the Bear Creek Project, dated March 26, 1963, prepared by The Reclamation Board, State of California.
4. Reviewed a Log of Test Borings drawing for the Bear Creek Bridge at Sacramento Road, dated December 20, 1961, prepared by Moore and Taber.
5. Reviewed a Scour Analysis for Lower Sacramento Road over Bear Creek, Stockton, CA, Bridge #29C0135, dated December 19, 2008 by Avila and Associates Consulting Engineers, Inc.
6. Reviewed in-house literature pertaining to geologic and seismic conditions in the project vicinity.
7. BCI observed, logged and sampled two borings (B9-08 and B10-08) to depths of about 71 feet at Bear Creek Bridge.
8. Performed laboratory tests on soil samples obtained from the exploratory borings.
9. Performed engineering analysis and calculations to develop our conclusions and recommendations.

2 SITE AND PROJECT DESCRIPTION

2.1 Site Description

The site is located on Lower Sacramento Road at Bear Creek, about 1,400 feet west of the Union Pacific Railroad (UPRR) tracks in Stockton, California. Site coordinates are approximately latitude 38.043ºN and longitude 121.322ºW. We show the site location on Figure 1 in Appendix A.

At this location, Bear Creek flows west within a 75-foot-wide, unlined man-made channel section. The bottom of channel is at/near elev. 7[1], about 17 feet below existing bridge deck grade. The channel slopes are in-place at about 2.5H:1V (horizontal:vertical distance).

The existing bridge is a five-span, concrete flat-slab structure, about 128.5 feet long and 32.5 feet wide, with a super-elevated deck. The substructure consists of concrete wall abutments supported on short piers on isolated spread footings and multi-column piers supported on spread footings. The supports are skewed about 20 degrees to match the channel alignment. The referenced plans show the base of each isolated spread footing at elev. 10.0 feet (1963 project datum) at the abutments and elev. 3.70 feet (1963 project datum) at the piers.

2.2 Project Description

The project will replace the existing structure with a three-span, cast-in-place, post-tensioned concrete voided slab bridge, about 152.7 feet long ("LSR" Sta. 27+68.74 to Sta. 29+21.41) and 112 feet wide. The new deck grade will be on a vertical curve that passes through elev. 27.68 at Abutment-1 (south) and elev. 27.63 at Abutment-4 (north). The bridge substructure will consist of seat-type abutments with cantilever wingwalls and two, multi-column piers.

No channel modifications, other than Rock Slope Protection (RSP) at the abutments, are planned for this project. The bridge approaches will require 5 feet to 8 feet of new embankment fill.

The new bridge will be constructed in two stages. Stage 1 will build the northbound section of the new bridge. The existing bridge will then be demolished and the southbound section constructed as part of Stage 2.

3 SUBSURFACE EXPLORATION

BCI retained V&W Drilling to drill two borings (one at each abutment) on April 4, 2008. The drillers used 6-inch diameter hollow stem auger drilling methods. Each boring was drilled to a maximum depth of 71.5 feet.

A BCI engineer logged the borings consistent with the Unified Soil Classification System (USCS) and retrieved samples for laboratory testing. We obtained 1.4-inch inside diameter (SPT) and 2.4-inch inside diameter (modified-California) drive samples from the borings at various intervals. The samplers were driven into the ground with a 140-pound automatic trip hammer falling 30 inches. At completion of drilling, we backfilled the boreholes with a cement-grout.

[1] Unless otherwise noted, all elevations are referenced to NGVD29 datum.

4 GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the San Joaquin Valley within the southern portion of the Great Valley Geomorphic Province. This province encompasses the San Joaquin Valley in the south and the Sacramento Valley in the north. The province is bound by the Sierra Nevada Mountains to the east, the Coast Ranges to the west, the Mojave Desert and Transverse Ranges to the south, and the Klamath Mountains to the north.

The Great Valley is a broad, elongated, northwest trending, structural trough that has been filled with a thick sequence of sediments. The eastern margin of the valley is formed by the west sloping Sierran bedrock surface that extends westward beneath the alluvium and older sedimentary bedrock within the valley. The western border is underlain by east dipping rock of the Coast Ranges that form a deeply buried trough.

During the late Mesozoic and through most of Tertiary time (approximately 100 million to 20 million years before present), deposition of thousands of feet of marine sediments occurred within the Great Valley. Continental deposits (generally alluvium) of late Tertiary and Quaternary age (approximately 20 million years ago to the present) overlie these marine deposits. Both the continental deposits and the underlying marine sediments form a wedge of sediments that generally thickens from east to west.

4.2 Local Geology

The California Geologic Survey (CGS)[2] maps surface materials at the site as the Modesto Formation, which is predominantly composed of Pleistocene gravelly sand, sand and silt alluvium deposited by streams and rivers.

4.3 Subsurface Conditions

4.3.1 Native Soil

In Borings B9-08 and B10-08, soils consist of predominately of medium dense to dense (locally very dense) clayey/silty sand and sand interbedded with layers of very stiff to hard (locally stiff) sandy silt, clay with sand, and sandy clay to the maximum depth explored (71.5 feet, elev.-49.4). We interpret the low blow count (N=11) recorded for sample number 15 in Boring B10-08 to reflect sluff in the boring.

Refer to the Log of Test Borings drawings in Appendix A for soil descriptions, exploration details and sampling methods.

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[2] "Geologic Map of the Sacramento Quadrangle, California"; Regional Geologic Map Series; Map No. 1A; California Division of Mines and Geology; D.L. Wagner, C.W. Jennings, T. L. Bedrossian, and E. J. Bortugno; 1991

4.3.2 Ground Water

At the time of our field exploration (April 4, 2008), BCI measured ground water at a depth of approximately 50 feet (elev.-26.2 and elev.-27.9) below ground surface in Boring B9-08 and B10-08.

Borings drilled by Moore & Taber in December 1961 indicate that the ground water was encountered 27 feet below ground surface in Boring 1. No ground water level is shown in Boring 2.

BCI reviewed ground water well data at the California Department of Water Resources website for three nearby wells. This data indicates that the groundwater level in project area has been about 30 feet below existing grade during the last 15 years.

Ground water and perched water levels can fluctuate due to changes in precipitation, Bear Creek surface water levels, irrigation, pumping of wells, and other factors.

5 LABORATORY TESTING

To classify the subsurface soil and obtain parameters for analysis, BCI performed laboratory tests on some of the samples obtained from the exploratory borings. Tests included:

- Moisture Content
- Density
- Particle Size Analysis
- Plasticity Index
- pH
- Minimum resistivity
- Sulfate Content
- Chloride Content

BCI performed laboratory tests in substantial conformance with current ASTM and Caltrans test procedures. Test results are presented in Appendix B.

6 CORROSION EVALUATION

Table 1 presents our corrosivity test results.

Table 1: Soil Corrosion Test Summary

Boring/Sample	Depth	Minimum Reistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
B9-08/8	30.0 - 31.5	1,050	6.98	13.3	64.3
B9-08/16	55.0 - 55.8	2,810	7.13	11.4	5.9
B10-08/3	16.0 - 16.5	1,720	7.12	13.8	53.2

Caltrans considers soils corrosive to foundation elements if one or more of the following conditions exist:

- Chloride concentration is 500 parts per million (ppm) or greater,
- Sulfate concentration is 2000 ppm or greater,
- pH is 5.5 or less.

Based on the laboratory test results, the site soils are classified as "non-corrosive" according to the Caltrans Corrosion Guidelines (Version 1.0, Sept 2003).

7 SCOUR EVALUATION

MTCo informed BCI that the proposed Bear Creek Bridge replacement corresponds to Alternative 2 reported in the scour analysis report by Avila and Associates Engineers, Inc. That report indicates approximately 8 feet of total scour, of which 6 feet is pier scour and 2 feet is future degradation. Avila recommends a design scour elevation at -1.0 feet (NGVD-29) for the pier foundations.

The scour analysis report indicates that the abutments should be checked assuming washout to elev. 5 ft.

8 SEISMIC DATA AND EVALUATION

8.1 Caltrans Seismic Design Criteria

Based on the Caltrans "California Seismic Hazard Map 1996", the peak horizontal rock acceleration for the site is approximately 0.14g. The controlling seismic source is the Coast Ranges-Sierran Block Boundary Zone (CSB), located about twenty-two miles west of the site, with an estimated maximum moment Magnitude of 7.0.

Using Table B.1 of Caltrans "Seismic Design Criteria (SDC), Version 1.4 (June 2006), we classify the site soil profile as Type D, with SPT values ranging from 15 to 50.

Based on guidelines and published Caltrans criteria as discussed above, use the following SDC seismic design parameters for design.

- Controlling Fault: Coast Ranges-Sierran Block Boundary Zone (CSB)
- Soil Type D
- Magnitude 7.25 ± 0.25
- Peak Horizontal Rock Acceleration = 0.20g
- Peak Horizontal Ground Acceleration = 0.28g
- Acceleration Response Spectra (ARS) Curve from SDC (Version 1.4) Figure B.8.

We include our recommended ARS Curve as Figure 2 in Appendix A.

8.2 Liquefaction Potential

Liquefaction can occur when loose to medium dense, granular, saturated soils (generally within 50 feet of the surface) are subjected to ground shaking. Based on our preliminary LOTB data and the relatively low peak ground acceleration, we conclude that the potential for liquefaction at the site is very low to nonexistent.

9 BRIDGE FOUNDATION RECOMMENDATIONS

BCI provides the following conclusions and recommendations related to abutment and pier foundations.

9.1 Foundation Data and Loading

The subsurface conditions encountered in our borings indicate that the site is suitable for either driven concrete or cast-in-drilled-hole (CIDH) piles. Undersize pre-drilling will be required for driven piling to penetrate locally dense soil layers and achieve specified tip elevations. CIDH piles may require casing due to potential caving of relatively clean sand layers in the upper 15 to 20 feet, and will need to be at least 24-inch diameter to maintain pile tips above ground water at the abutments and to allow for slurry drilling at the piers. Steel HP piles would require greater penetration than driven concrete piles for an equivalent pile capacity.

We do not recommend spread footing foundations due to the limited soil bearing capacity (likely on order of 3.0 ksf) in the upper 15 feet at the abutments, depth to competent bearing support in the channel and scour potential.

Based on the above information, driven Class 90 (Alt X) precast, prestressed concrete piles were selected for the abutments and 30-inch diameter CIDH piles were selected for the piers.

MTCo provided the following foundation design information in Tables 2 and 3.

Table 2: Foundation Design Data Provided by MTCo

Foundation Design Data								
Support No.	Design Method (WSD or LRFD)	Pile Type	Finish Grade Elev. (ft)	Pile Cut-off Elevation * (ft)	Pile Cap Size (ft)		Permissible Settlement – Service Load (in)	Number of Piles per Support
					B	L		
Abut 1	WSD	Class 90 (Alt X, T=12”)	21.0	16.25	7.5	120.75	1	31
Pier 2	LRFD	30” CIDH	10.0	25.26	NA	NA	1	18
Pier 3	LRFD	30” CIDH	10.0	25.24	NA	NA	1	18
Abut 4	WSD	Class 90 (Alt X, T=12”)	19.0	14.25	7.5	123.17	1	31

Note: * For Piers 2 and 3, pile cut-off elevation is given as average soffit elevation at pier.

Table 3: Foundation Design Loads Provided By MTCo

Foundation Design Loads											
Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Abut 1	2260	85	1880	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	3890	230	2080	6425	375	0	0	2080	130	0	0
Pier 3	3890	230	2080	6425	375	0	0	2080	130	0	0
Abut 4	2430	90	2060	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:

1) For Piers 2 and 3, per support and per pile loads are given at top of column (i.e. average soffit elevation given in Foundation Design Data Table).
2) To obtain Piers 2 and 3 pile loads at ground considering no scour (channel elevation = 7.0 feet), add an additional 14 kips per pile to service and extreme limit state loads, and 17 kips per pile to strength limit state loads.
3) To obtain Piers 2 and 3 pile loads at ground considering total scour (scour elevation = -1.0 feet), add an additional 19 kips per pile to service and extreme limit state loads, and 24 kips per pile to strength limit state loads.

9.2 Foundation Recommendations and Pile Data Table

BCI used the above preliminary foundation design data and loading conditions to evaluate pier foundations using AASHTO LRFD Bridge Design Specifications-4th Edition with Interims Thru 2009 and current Caltrans Amendments (V4). We evaluated abutment foundations using Caltrans November 2003 Bridge Design Specifications for foundations using Working Stress Design methods. We present our foundation recommendations in Tables 4, 5 and 6 on the following pages.

Table 4: Foundation Recommendations for Abutments

Abutment Foundation Design Recommendations										
Support	Pile Type	Cut-off Elev. (ft.)	**LRFD Service-I Limit State Load – Compression (kips)**			Required Nominal Resistance (kips)		Design Tip Elevations (ft.)	Specified Tip Elevation (ft.)	Nominal Driving Resistance (kips)
			Per Support		Per Pile	Comp.	Tens.			
			Total	Permanent						
Abut 1	Class 90 (Alt. X, T=12")	16.25	2260	1880	85	170	0	-20.0 (a) -15.0 (b) -4.0 (c)	-20.0	170
Abut 4	Class 90 (Alt. X, T=12")	14.25	2430	2060	90	180	0	-20.0 (a) -15.0 (b) -4.0 (c)	-20.0	180

Notes:
1) Design tip elevations for **Abutments** are controlled by (a) Compression, (b) Scour, (c) Lateral.
2) The nominal driving resistance required is equal to the required nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the required nominal resistance due to scour.

Table 5: Foundation Recommendations for Piers

Pier Foundation Design Recommendations											
Support	Pile Type	Cut-off Elev. (ft.)	LRFD Service-I Limit State Load Per Support – Compression (kips)	Total Permissible Support Settlement (in.)	Required Factored Nominal Resistance (kips) Per Pile				Design Tip Elevations (ft.)	Specified Tip Elevations (ft.)	Nominal Driving Resistance Required (kips)
					Strength Limit		Extreme Event				
					Comp φ = 0.7	Tens. φ = 0.7	Comp φ = 1.0	Tens φ = 1.0			
Pier 2	30-inch CIDH	25.26	3890	1	399	0	150	0	-43.0 (a) -45.0 (b)	-45.0	N/A
Pier 3	30-inch CIDH	25.24	3890	1	399	0	150	0	-43.0 (a) -45.0 (b)	-45.0	N/A

Notes:
1) Design tip elevations for **Piers** are controlled by (a) Compression (Strength Limit), (b) Scour, respectively.
2) The CIDH specified tip elevation shall not be raised.

Based on our analysis presented in the following sections, BCI presents our recommended Pile Data Table as Table 6:

Table 6: Pile Data Table

Pile Data Table						
Support	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft.)	Specified Tip Elevation (ft.)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 90 (Alt. X, T=12”)	170	0	-20.0 (a) -15.0 (b) -4.0 (c)	-20.0	170
Pier 2	30-inch CIDH	570	0	-43.0 (a) -45.0 (b)	-45.0	N/A
Pier 3	30-inch CIDH	570	0	-43.0 (a) -45.0 (b)	-45.0	N/A
Abut 4	Class 90 (Alt. X, T=12”)	180	0	-20.0 (a) -15.0 (b) -4.0 (c)	-20.0	180

Notes:
1) Design tip elevations for **Abutments** are controlled by (a) Compression, (b) Scour, (c) Lateral, respectively.
2) Design tip elevations for **Piers** are controlled by (a) Compression (Strength Limit), (b) Scour, respectively.
3) The nominal driving resistance required for Abutment piles is equal to the required nominal resistance needed to support the factored load plus driving resistance from the penetrated soil layers, if any, which do not contribute to the required nominal resistance due to scour.

Provide a minimum pile spacing of two pile dimensions, center to center, to achieve the above compressive capacities. BCI presents a discussion of our pile analysis in Sections 9.3 and 9.4.

9.3 Abutments 1 and 4 (Class 90 PPC Piles)

In accordance with current Caltrans specifications, we used Working Stress Design (WSD) for the abutment piles. BCI presents the results of our compressive resistance, settlement and lateral pile analysis below.

9.3.1 Compressive Resistance

The tips of the Class 90 piles will bear in medium dense to dense sand about 32 feet below the existing channel bottom elevation. BCI used both end bearing and skin friction contributions in our compressive resistance analysis. Actual contributions to end bearing and skin friction could vary depending on how the load is transferred to the pile. We neglected the approach fill in our skin friction and end bearing analysis.

We determined the compressive resistance using the Federal Highway Administration's Driven 1.2 (March 20, 2001) computer program developed by Blue-Six Software, Inc. BCI estimated specified tip for a nominal resistance of 170 kips/pile at Abutment 1 and 180 kips/pile at Abutment.

BCI evaluated pile compressive resistance for washout to elev. 5 feet at the abutments. MTCo indicates that under these conditions the foundation piles will be subject only to a maximum dead load of 53 kips/pile (nominal load of 106 kips/pile).

Refer to the Driven 1.2 output files in Appendix C for additional information.

9.3.2 Settlement

We calculated immediate pile settlement of approximately 0.6-inches (for the Service 1 Limit State Load) by the method outlined in Section 16-10 of Foundation Analysis and Design, 5^{th} edition, Joseph E. Bowles, 1996. We do not anticipate significant long-term settlement due to the competent soil conditions above and below the pile tips. We include the pile settlement calculations in Appendix C.

9.3.3 Lateral Load Analysis

We used LPILE Plus Version 5.0 software to evaluate lateral pile capacity for the driven Class 140 (Alt X, T = 12") piles. MTCo requested analysis to determine the allowable lateral pile design loads which would produce approximately ¼-inch top-of-pile deflection and 1-inch top-of-pile deflection assuming a pinned head condition. MTCo requested analysis for the pre-scour condition only. For ultimate scour condition, the lateral pile capacities will be significantly lower than the values shown below.

BCI used a reduced p-multiplier of 0.93 in the longitudinal bridge direction to account for group effects for a pile center-to-center spacing of about 5 pile widths. BCI did not use a p-multiplier in the transverse bridge direction due to the wider pile spacing.

For the longitudinal bridge direction, our analysis yielded a lateral resistance of 12.5 kips for ¼-inch top-of-pile deflection, and 24.3 kips for 1-inch top-of-pile deflection.

For the transverse bridge direction, our analysis yielded a lateral resistance of 13.0 kips for ¼-inch top-of-pile deflection, and 25.3 kips for 1-inch top-of-pile deflection.

BCI calculated a minimum lateral tip elevation of -4.0 ft. (NGVD29) for the piles using a factor of safety of 1.5.

Refer to the LPILE output files in Appendix C for additional information.

9.3.4 Negative Skin Friction

Because the subsurface soil is generally competent with no soft clay or loose sand layers, we do not anticipate negative skin friction at the abutments.

9.4 Piers 2 and 3 (30-inch CIDH Piles)

We used AASHTO LRFD Bridge Design Specifications-4th Edition with Interims Thru 2009 and current Caltrans Amendments (V4) for evaluating the pier pile extensions. BCI presents the results of our compressive resistance, settlement and lateral pile analysis below.

9.4.1 Compressive Resistance

For 30-inch diameter CIDH piles, BCI used skin friction contributions and neglected end bearing in our compressive resistance analysis. We determined the compressive resistance using SHAFT 6.0, the drilled shaft computer program developed by Ensoft, Inc. SHAFT computes the axial capacity and short-term settlement analysis. In general, SHAFT analytical methods are based on methods recommended in the FHWA manual Drilled Shafts: Construction Procedures and Design Methods, by L.C. Reese and M. W. O'Neill, published in November 1999. We used a design scour elevation of -1.0 feet in our analysis for both piers.

BCI determined the required factored nominal resistance by comparing the Factored Strength Limit Load (Geotechnical Resistance Factor = 0.7) with the Extreme Event Load (Resistance Factor = 1.0). We then used the higher value as the required factored nominal resistance under scour conditions. In this case, the Factored Strength Limit Load [(375+24)/0.7 = 570 kips per pile] is controlling over the Extreme Event [(130+19)/1.0 ≈ 150 kips per pile].

Refer to the SHAFT output graphs in Appendix D for additional information.

9.4.2 Settlement

The settlement analysis obtained from SHAFT estimates that the maximum total settlement of CIDH piles established as above will be nominal (less than 0.5-inches for the Service 1 Limit State Load) and occur substantially during construction.

We do not anticipate significant long-term settlement due to the competent soil conditions above and below the pile tips. We include the pile settlement calculations in Appendix D.

9.4.3 Lateral Load Analysis

MTCo requested that BCI provide L-pile parameters for use in their equivalent column length and overturning calculations for the pier foundations. MTCo indicated that BCI not perform lateral load analysis for pier piles.

Table 7 provides our recommended L-pile parameters for equivalent column length calculations at the pier.

Table 7: L-pile Parameters for Equivalent Column Length Analysis

Elevation (NGVD-29)	L-Pile Soil Type (p-y curve model)	Unit Weight (pci)	Friction Angle (degrees)	Cohesion (psf)	ε_{50} (dim.)	Modulus, k (lb/in^3)
7.0 to -1.0	Sand (Reese)	0.0368	33	--	--	60
*-1.0 to -10.0	Sand (Reese)	0.0729	36	--	--	90
-10.0 to -20.0	Stiff Clay w/o Free Water (Reese)	0.0677	--	1,600	0.007	**
-20.0 to -47.0	Sand (Reese)	0.0380 (submerged)	38	--	--	125

*scour elevation; ** L-pile program internally calculates k value for clay.

9.4.4 Negative Skin Friction

Because the subsurface soil is generally competent with no soft clay or significant loose sand layers, we do not anticipate negative skin friction at the piers.

9.5 Abutment Lateral Earth Pressures

We recommend the following equivalent fluid weights (EFWs) be used to design the abutment walls and wing walls.

Table 8: Equivalent Fluid Weight

Condition	Equivalent Fluid Weight	
	Static (lb/ft^3)	Dynamic (lb/ft^3)
Active	38	47
At-Rest	60	74
Passive	2220	203

The values shown above assume level backfill conditions using Caltrans "Structure Backfill" with a soil unit weight of 130 pcf, a minimum angle of internal friction of 33º, and that drainage is placed behind walls in accordance with Caltrans "Standard Plans and Specifications."

We estimated the EFWs for seismic loading conditions using the Mononobe-Okabe equation for active and passive lateral coefficients K_a and K_p. We estimated the at-rest coefficient, K_o, for the seismic condition using an increase ratio similar to the active condition. We used a horizontal acceleration of 0.14g (50% of the peak ground acceleration of 0.28g) in the Mononobe-Okabe equation. We calculated the static EFWs using methods presented in the 1982 Naval Facilities (NAVFAC) Design Manual 7.2.

Apply the resultant of the seismic active and at-rest pressures at a depth of 0.5H from the base of the wall, where H equals the wall height in feet. The passive pressures are applicable for concrete placed directly against undisturbed soil or compacted fill.

For seismic loading into abutments, use a maximum passive pressure of 5.0 ksf for longitudinal abutment response, with the proportionality factor presented in Section 7.8.1 of Caltrans Seismic Design Criteria v.1.4.

For surcharge loads, apply an additional uniform lateral load behind the wall equivalent to 0.3-times the surcharge pressure.

Use a coefficient of friction of 0.45 to resist sliding for concrete placed on native undisturbed soil.

10 APPROACH FILLS

10.1 Fill Material

Embankments will be constructed using imported borrow material, supplemented with material excavated from on-site cuts and existing approach embankment fill. The source(s) of borrow material for construction of approach fills has not been identified. Proposed borrow must be tested and approved for use by the project engineer prior to transporting to the site. Refer to Section 11.5 and 11.6 for Approach Fill and Levee Embankment Fill requirements, respectively.

10.2 Slope Geometry and Stability

The maximum fill height at the bridge abutments will be approximately 8 feet. Approach side slopes and end slopes will have a gradient of 2H:1V, or flatter. Such slopes should be stable provided the new slopes are constructed in accordance with the 2006 Caltrans Standard Specifications. The underlying native soil should provide a stable base on which to construct the fills.

10.3 Settlement

Based on the subsurface conditions, we anticipate about 1 to 3 inches of settlement for 10 foot high embankments, mostly occurring during construction. No waiting period is necessary prior to construction of bridge abutment foundations.

11 CONSTRUCTION CONSIDERATIONS

Where referenced below, "Standard Specifications" refers to Caltrans Standard Specifications (May 2006).

11.1 Abutment Piles

Class 90 (Alt. X) piles shall conform with Section 49-1 of the Standard Specifications. Difficult pile installation is anticipated due to the presence of locally dense soil layers above the specified tip elevations.

At the abutments, perform predrilling through the abutment fill to Elevation 10.0 feet in accordance with Section 49-1.06 of the Standard Specifications. The hole shall have a diameter of not less than the greatest dimension of the pile cross section plus 6 inches. The annulus remaining after driving the piles shall be filled with cement-bentonite grout. Spudding should not be used.

The contractor may perform undersize drilling to assist pile driving through dense native soil to achieve the specified tip elevations. Drilling should be performed prior to pile driving in accordance with Caltrans Standard Specification 49-1.05, except the *drill hole should be no greater than 8 inches in diameter for the 12-inch Class 90 (Alt. X) piles.* Perform undersize drilling to at least Elevation -5.0 feet (NGVD-29) but not deeper than within 10 feet of specified pile tip elevations. The contractor should drill and drive the first pile at the abutment locations, and then adjust the drilling procedure if necessary to achieve the specified tip elevation on remaining piles.

Jetting or vibratory hammers should not be used to obtain the specified pile penetration.

Verify pile capacity during placement using energy equations in accordance with Caltrans Standard Specification 49-1.08. However, in no case shall the required blows (N) be less than that obtained using the Engineering News Formula (P=Er/6(s + 2.54). A pile load test is not necessary.

11.2 Pier Piles

Due to the presence of ground water (above specified tip elevation), construct 30-inch diameter CIDH piles by the “wet” method, slurry drilling and concrete deposited under slurry.

Construct CIDH piles in conformance with Section 49-4 of the 2006 Caltrans Standard Specifications and the Standard Special Provision 49-310 (Cast-In-Drilled-Hole Concrete Piles). Drilling slurry shall conform to Caltrans Standard Special Provision 49-311. The slurry construction method also requires placement of inspection tubes to permit gamma-gamma and crosshole sonic testing of the CIDH pile (Caltrans Memo to Designers 3-1, July 2008).

The CIDH pile excavations will encounter sandy layers based on our boring data. Temporary casing may be required during construction of the CIDH piles to mitigate caving within clean sand layers. The contractor should review the Log of Test Borings and plan accordingly. The contractor is responsible for the design of temporary casing, including actual length(s), to install CIDH piles according to the above specifications without defects.

11.3 Temporary Shoring

The contractor is responsible for design and construction of excavation sloping and shoring in accordance with CalOSHA Standards.

11.4 Dewatering

During the rainy season, infiltrating rain water can pond upon less permeable underlying soil creating a perched water condition. This perched water condition may extend into the late spring or early summer season. If perched ground water or surface water is encountered, sump pumps may be required to facilitate construction. If needed, we expect that surface water in the channel (at low flow) can be diked/diverted if construction takes place during the late spring through early fall months.

11.5 Approach Fill

Construct embankment and place/compact new fill in accordance with Caltrans “Standard Specifications” (including Section 19, “Earthwork”).

Where new fill is to be placed onto existing fill slopes or natural slopes exceeding 5H:1V, fully bond into the existing slope by placing on discrete horizontal benches cut fully into the slope and below any loose/soft or otherwise unsuitable materials (per Section 19 of Caltrans “Standard Specifications”).

Expansive soil (Expansion Index > 50) should not be used as fill within 10 feet behind the abutment backwall.

11.6 Levee Embankment Fill

New levee fill shall meet the following criteria:

- 100% passing the 2-inch sieve
- 90% to 100% passing the No. 4 sieve
- At least 20% passing the No. 200 sieve
- Liquid Limit ≤45
- Plasticity Index ≥ 8 ≤ 40
- Expansive soil (Expansion Index > 50) shall not be placed within 10 feet behind the abutment backwall.
- Shall not contain organics, debris or other deleterious material

Existing levee materials may be reused as engineered fill within the levee, provided that organics, high-plasticity clays (CH), oversize material (i.e., greater than 2-inches), trash, and other deleterious material are removed.

Place fill in maximum 6-inch lifts, moisture condition to within 1% below to 2% over optimum and compact to a minimum of 97% relative compaction per ASTM D 698.

Bench fill into the existing levee a minimum of one foot for every foot of fill placed, or as necessary to remove loose material and provide proper compaction along the zone of transition.

11.7 Construction Monitoring

Pile driving for Stage 1 bridge construction could potentially cause settlement of the native soil below the existing bridge foundations, which could result in excessive settlement of the existing bridge, especially since the structure is founded on shallow spread footings.

BCI recommends that a settlement monitoring program be developed to avoid excessive settlement of the existing bridge during pile driving for the new bridge.

The settlement monitoring program should include performing a pre-construction survey of the existing bridge to develop baseline elevation data and benchmarks. The benchmarks should be continuously surveyed/monitored during all pile driving operations for the new bridge. In the event that settlement at any benchmark exceeds 0.2 inches, discontinue pile driving immediately and contact MTCo and BCI for additional recommendations.

11.8 Potential Pier Pile Conflict

BCI expects the existing bridge will be removed in accordance with Caltrans Standard Specifications Section 15-4. There appears to be potential for conflict between the existing Pier 5 foundation and new Pier 3 CIDH piles.

We understand that MTCo has offset Pier 3 CIDH pile elements to be clear of the existing bridge foundation. In the event that demolition of the existing structure reveals potential conflicts with new Pier 3 construction (or other new support locations), contact MTCo and BCI immediately for additional recommendations.

12 RISK MANAGEMENT

Our experience and that of our profession clearly indicates that the risks of costly design, construction, and maintenance problems can be significantly lowered by retaining the geotechnical engineer of record to provide additional services. For this project, BCI should be retained to:

1. Review and provide written comments on the (civil, structural) plans and specifications prior to construction.
2. Monitor construction to check and document our report assumptions. At a minimum, we should monitor pile installation; approach fill subgrade and fill construction; abutment and wingwall backfill.
3. Update this report if:
   - design changes occur,
   - 2 years or more lapse between this report and construction, or
   - site conditions change.

If BCI is not retained to perform the above applicable services, we are not responsible for any other parties' interpretation of our report, and subsequent addenda, letters, and discussion.

13 LIMITATIONS

This report should only be used for design and construction of the Lower Sacramento Road Bridge at Bear Creek project, as described herein.

BCI performed services in accordance with the generally accepted geotechnical standard of practice currently used in this area. Where referenced, we used ASTM and Caltrans Standards as a general (not strict) *guideline* only. We do not warranty our services.

BCI based this report on the current site and project conditions. We assumed the soil and ground water conditions encountered in our exploratory borings were representative of the subsurface conditions across the site. Actual conditions between borings could be different. Ground water may be higher in other locations than measured in the borings.

The interface between soil types on the logs is approximate. The transition between soil types may be abrupt or gradual. We based our recommendations on the final logs, which represent our interpretation of the field logs and general knowledge of the site and geologic conditions.

Our scope did not include evaluation of flooding or hazardous materials on site.

Modern design and construction is complex, with many regulatory sources, restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and delays.

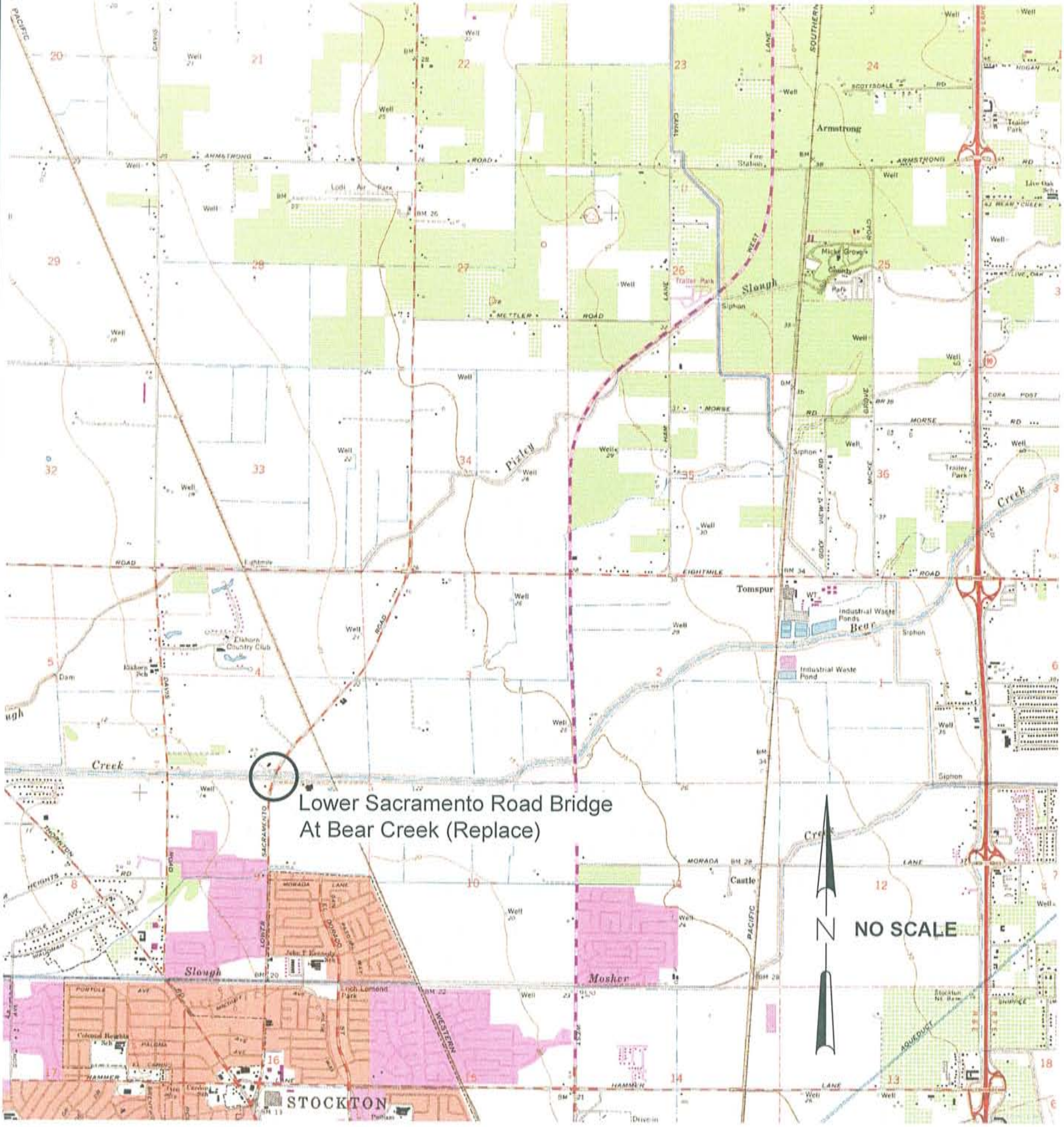
APPENDIX A

Figure 1 – Vicinity Map

Figure 2 – ARS Curve

Log of Test Borings (3 sheets)

ATTACHMENT G



Source: MAPTECH Terrain Navigator Pro, v. 7.01, USGS topographic map, 7.5 minute quadrangle, 1:24000, Lodi South 1968, photorevised 1976.

blackburn consulting

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West Sacramento,CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

VICINITY MAP

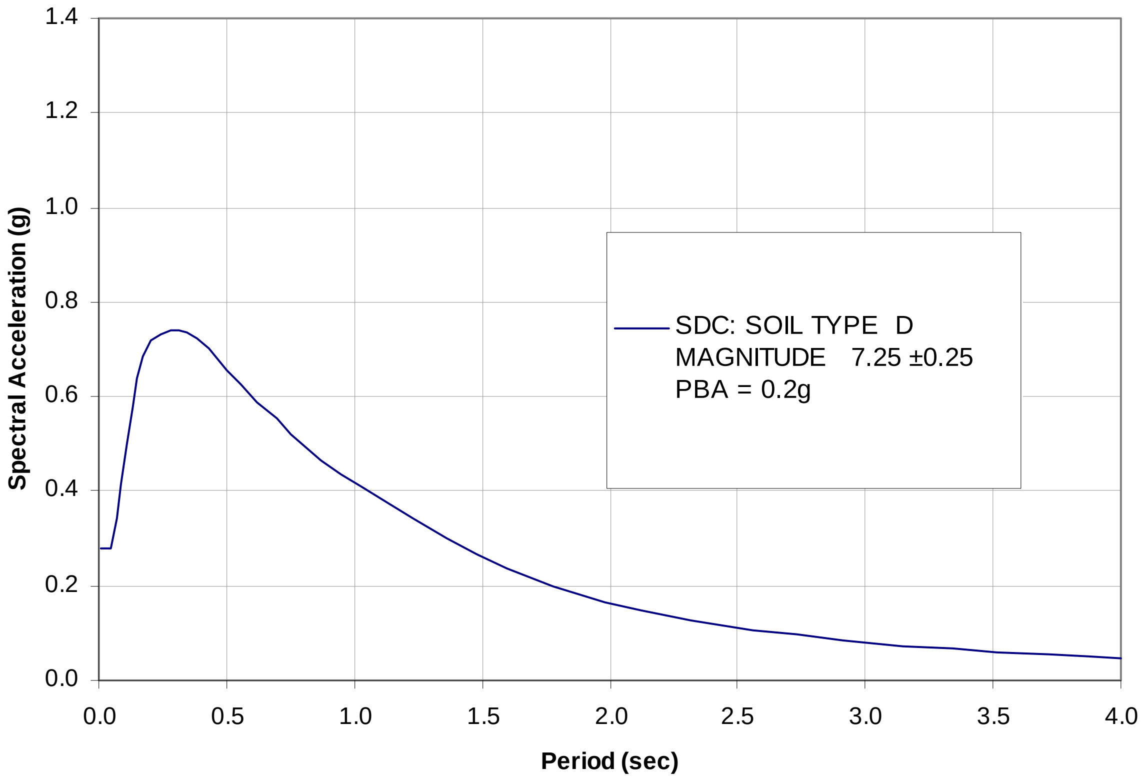
Lower Sacramento Road Bridge
At Bear Creek (Replace)
Stockton, California

File No. 879.5

June 2010

Figure 1

6/29/2010 879.5 Lower Sacramento Road Bear Creek Bridge Replacement Figure 1.dwg



Reference: Figure B.8, Caltrans Seismic Design Criteria, Version 1.4, June 2006.

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ARS CURVE
Lower Sacramento Road Bridge
at Bear Creek (Replace)
Stockton, California

File No. 879.5

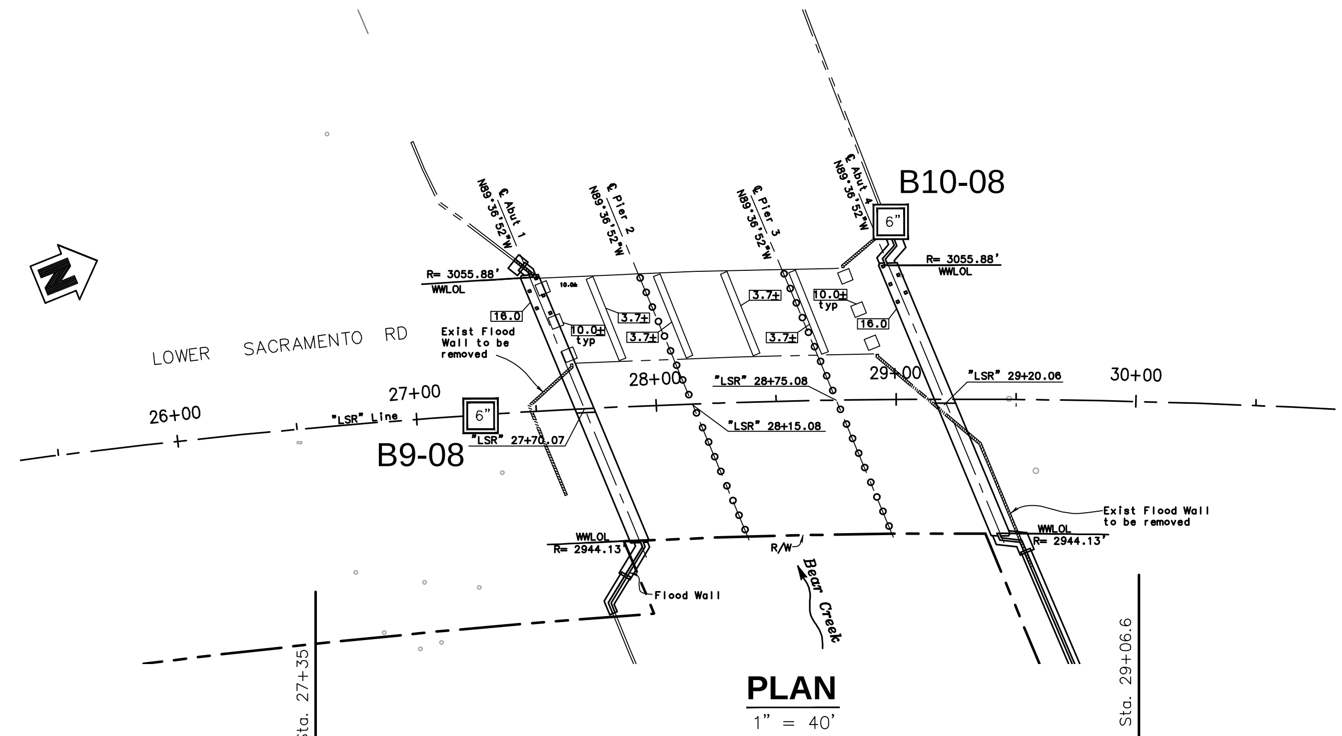
June 2010

Figure 2

ATTACHMENT G

BENCHMARK

City of Stockton BM #4 Monument #IN-10, a Brass Disk in monument well located at the intersection of Davis Road and Eight Mile Road. Elevation 17.53 feet (NGVD 29 Datum)



PLAN

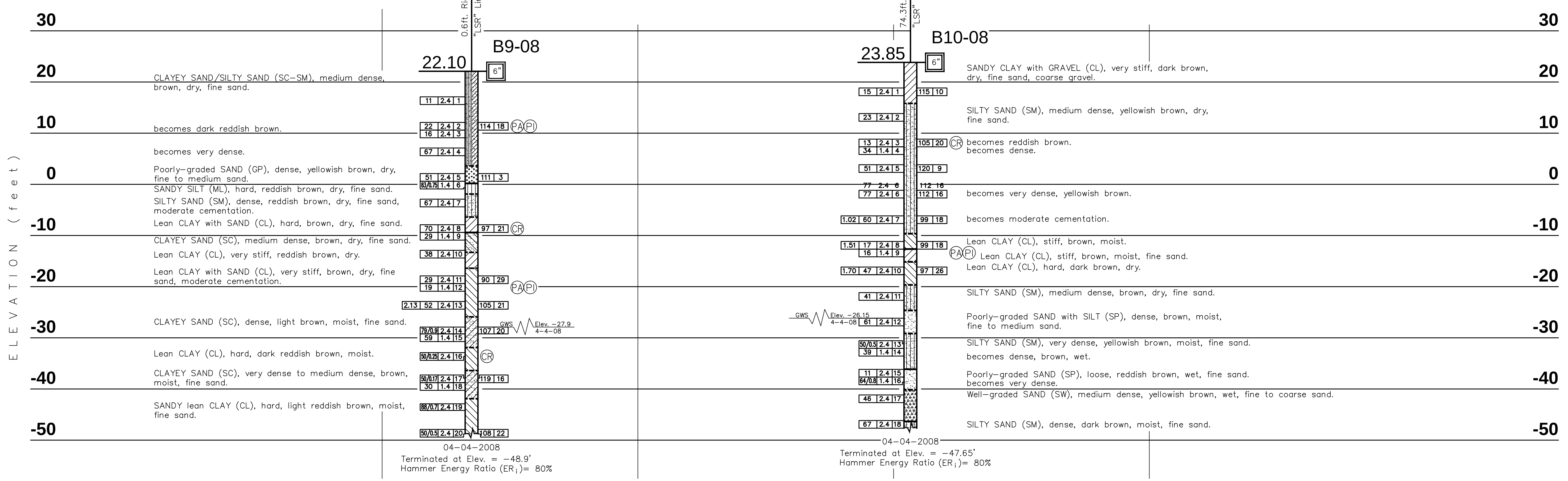
1" = 40'

LEGEND

(rectangle)	Indicates Bottom of Footing Elevation
(small square)	Indicates Precast Prestressed Concrete Pile (All Piles Not Shown)
(circle)	Indicates CIDH Pile
(dashed line)	Indicates Existing Structure

NOTES:

1. Field classification of soils was in accordance with ASTM D 2488-00 "Description and Identification of Soils (Visual-Manual Procedure)".
2. Standard Penetration tests were performed in accordance with ASTM D 1586-99 using a hammer operated with an automated drop system. Drill rods were 1 5/8-inch diameter "A"-rods; sampler was driven without brass liners.
3. "2.5 inch sampler": ID=2.5 inch, OD=2.9 inch. Driven in same manner as SPT ("1.4 inch") sampler.
4. The length of each sampled interval is shown graphically on the boring log. Whole number blow counts ("N") represent the "standard penetration resistance" interval in accordance with ASTM D1586-99. Where less than 1 foot of penetration is achieved, the blow count shown is for that fraction of the "standard penetration resistance" interval actually penetrated.
5. Consistency of soils shown in ( ) where estimated.
6. Ground water surface elevations in the borings indicated on the Log of Test Boring Sheets reflect the fluid level in the borings on the specified date.
7. Ground water surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
8. Electronic media for plan view provided by Mark Thomas & Company, December 2008.
9. The "Log of Test Borings" drawing is included with plans in accordance with Section 2-1.03 of Caltrans "Standard Specifications".



PROFILE

VERT. 1" = 10'
HOR. 1" = 20'

ORIGINAL SCALE IN INCHES FOR REDUCED PLAN

0 1 2 3

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01/13/09 | 08/31/09 | 05/07/10 | 06/30/10 — BCI JOB NUMBER: 879.5

REGISTERED PROFESSIONAL GEOLOGIST — WILLIAM E. NICHOLS No. CEG 2229 Exp. 01/31/12 ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Revision No.	Description	Date	By	Aprvd. By

LOWER SACRAMENTO RD BRIDGE AT BEAR CREEK (REPLACE)

LOG OF TEST BORINGS 1 OF 3

CITY OF STOCKTON
PUBLIC WORKS DEPARTMENT

BRIDGE NO.: 29C0443	APPROVED BY: DATE	SHEET NO.127
DESIGNED BY: WEN		S28 of S30
DRAWN BY: MDR		OF 129 SHEETS
CHECKED BY: WEN	CITY ENGINEER STOCKTON, CALIFORNIA	PROJECT NO. 05-17
RECORD DWG:		

PLOT DATE: Jul 01, 2010-11:01:23am

FILE NAME: 03 NSGS_LSBC LOTB

PATH: Z:\Active Projects\879.X - Stockton Bridges\879.5 - Lower Sacramento Road_UPRR\CAD Drawings\

ATTACHMENT G

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL, (JUNE, 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names	Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly-graded GRAVEL Poorly-graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly-graded GRAVEL with SILT Poorly-graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly-graded GRAVEL with CLAY (or SILTY CLAY) Poorly-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean Clay ORGANIC lean Clay with SAND ORGANIC lean Clay with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly-graded SAND with SILT Poorly-graded SAND with SILT and GRAVEL		
SP-SC	Poorly-graded SAND with CLAY (or SILTY CLAY) Poorly-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTING

- (C) Consolidation (ASTM D 2435-04)
- (CL) Collapse Potential (ASTM D 5333-03)
- (CP) Compaction Curve (CTM 216-06)
- (CR) Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- (CU) Consolidated Undrained Triaxial (ASTM D 4767-04)
- (DS) Direct Shear (ASTM D 3080-04)
- (EI) Expansion Index (ASTM D 4829-03)
- (M) Moisture Content (ASTM D 2216-05)
- (OC) Organic Content-% (ASTM D 2974-07)
- (P) Permeability (CTM 220-05)
- (PA) Particle Size Analysis (ASTM D 422-63) (2002)
- (PI) Plasticity Index (AASHTO T 90-00) Liquid Limit (AASHTO T 89-02)
- (PL) Point Load Index (ASTM D 5731-05)
- (PM) Pressure Meter
- (PP) Pocket Penetrometer
- (R) R-Value (CTM 301-00)
- (SE) Sand Equivalent (CTM 217-99)
- (SG) Specific Gravity (AASHTO T 100-06)
- (SL) Shrinkage Limit (ASTM D 427-04)
- (SW) Swell Potential (ASTM D 4546-03)
- (TV) Pocket Torvane
- (UC) Unconfined Compression-Soil (ASTM D 2166-06) Unconfined Compression-Rock (ASTM D 2938-95) (2002)
- (UU) Unconsolidated Undrained Triaxial (ASTM D 2850-03)
- (UW) Unit Weight (ASTM D 2937-04)
- (VS) Vane Shear (AASHTO T 223-96) (2004)
- (LT) Unconfined Compressive Strength of Lime Treated Soil/Aggregates (CTM 373-00)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} -Value (Blows / 12 in.)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		>12 in.
Cobble		3 to 12 in.
Gravel	Coarse	3/4 to 3 in.
	Fine	No. 4 to 3/4 in.
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

CONSISTENCY OF COHESIVE SOILS

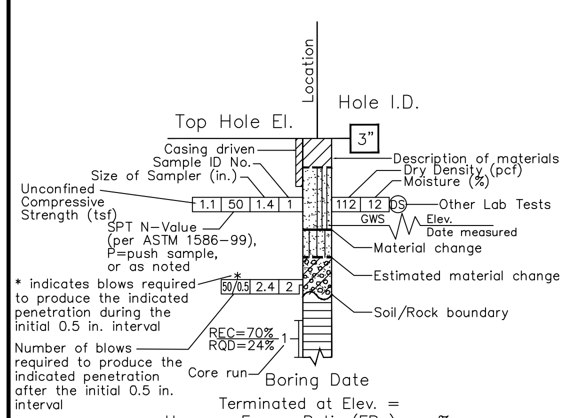
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	<0.25	<0.25	<0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

PLASTICITY OF FINE-GRAINED SOILS

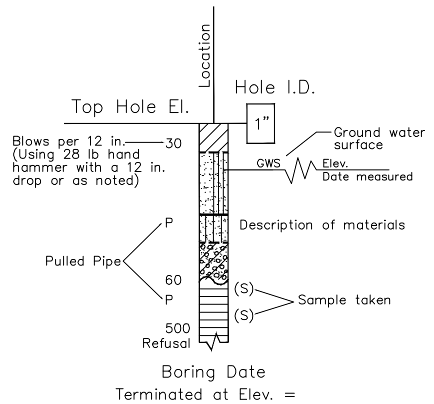
Description	Criteria
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

BOREHOLE IDENTIFICATION

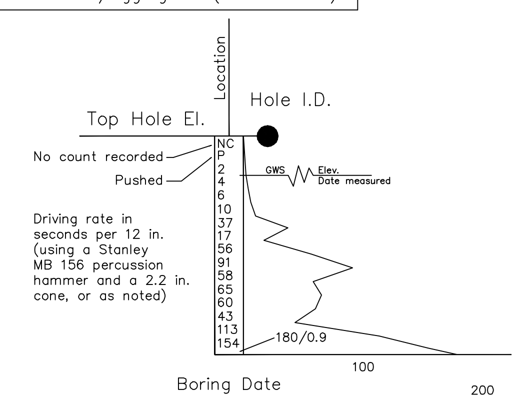
Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
Size	T	Backhoe Test Pit



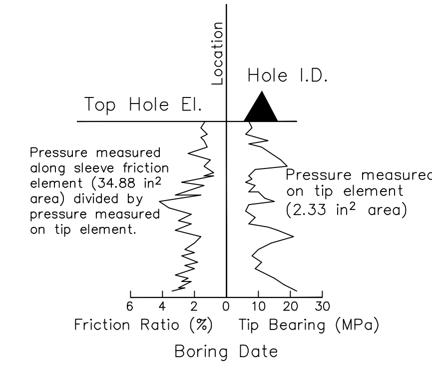
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATON BORING



CONE PENETRATON TEST (CPT) SOUNDING

ORIGINAL SCALE IN INCHES FOR REDUCED PLAN

0 1 2 3

SOIL LEGEND

LOWER SACRAMENTO RD BRIDGE AT BEAR CREEK (REPLACE)

LOG OF TEST BORINGS 2 OF 3

BLACKBURN CONSULTING
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REGISTERED PROFESSIONAL GEOLOGIST
WILLIAM E. NICHOLS
No. CEG 2229
Exp. 01/31/12
ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Revision No.	Description	Date	By	Aprvd. By

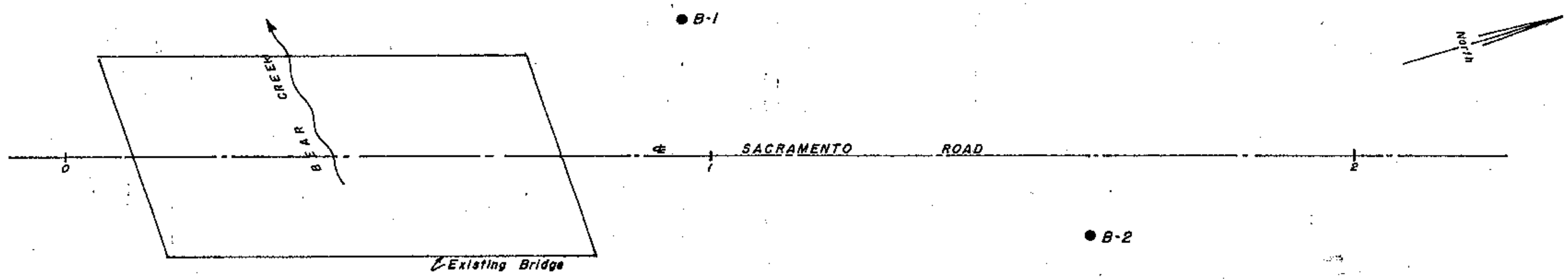
~~01/13/09~~ | ~~08/31/09~~ | ~~05/07/10~~ | 06/30/10 | BCI JOB NUMBER: 879.5

CITY OF STOCKTON
PUBLIC WORKS DEPARTMENT

BRIDGE NO.: 29C0443	APPROVED BY:	SHEET NO. 128
DESIGNED BY: WEN	DATE	S29 of S30
DRAWN BY: MDR		OF 129 SHEETS
CHECKED BY: WEN	CITY ENGINEER STOCKTON, CALIFORNIA	PROJECT NO. 05-17
RECORD DWG:		

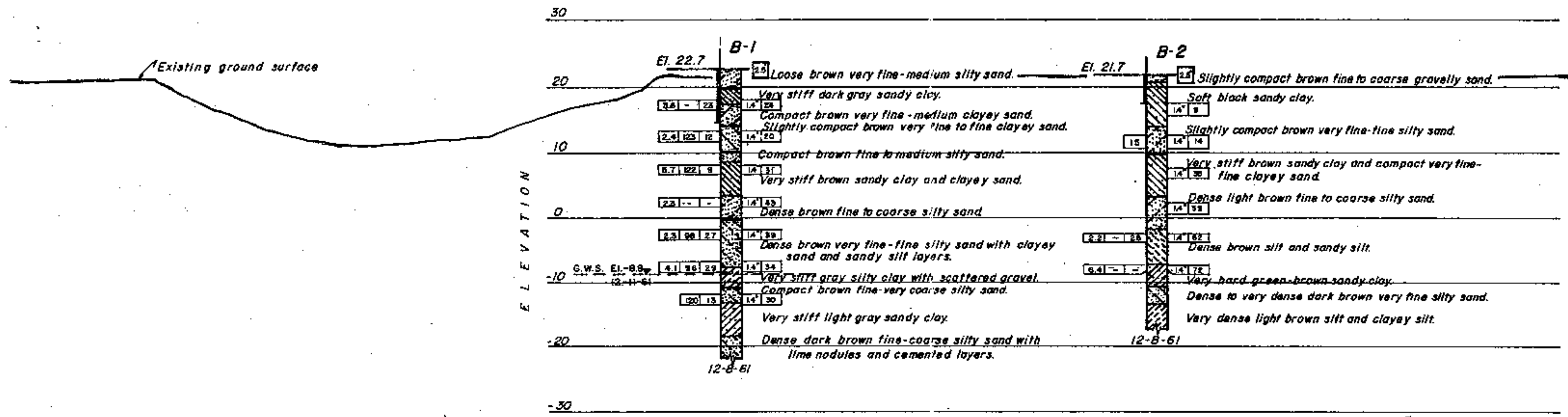
PATH: Z:\Active Projects\879.X - Stockton Bridges\879.5 - Lower Sacramento Road_UPRR\CAD Drawings\ FILE NAME: 03 NSGS_LSBC LOTB PLOT DATE: Jul 01, 2010-11:01:30am

ATTACHMENT G



—— PLAN ——

NOTE —
BM. North end of bridge at ℄ of road.
El. 22.7



—— PROFILE ——

As–Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. It does not attest to the accuracy or validity of the information contained in the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	MILEPOST–PROJECT	SHEET NO.	TOTAL SHEETS
10	SJ	C.R.	101.32		

REGISTERED ENGINEERING GEOLOGIST DATE

LOWER SACRAMENTO RD BRIDGE AT BEAR CREEK (REPLACE)

LOG OF TEST BORINGS 3 OF 3

NOTE: A COPY OF THIS LOG OF TEST BORINGS IS AVAILABLE AT OFFICE OF STRUCTURE MAINTENANCE AND INVESTIGATIONS, SACRAMENTO, CALIFORNIA.

CU:
EA:

BRIDGE NO. 29C0443
SHEET NO. 129
S30 of S30
OF 129 SHEETS
PROJECT NO. 05-17

PROFESSIONAL GEOLOGIST
William E. Nichols
No. 2229
Exp. 1-31-12
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

a.l.s.

LEGEND OF EARTH MATERIALS

SIZE CLASSIFICATION

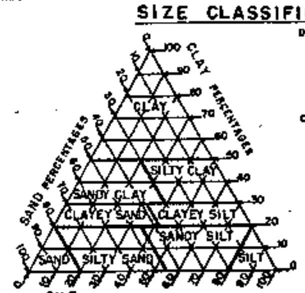


Diagram showing the basis of grain size distribution used in determination of stone names. Size classification is based on the Wentworth grade scale in field classification or the A.S.T.M. grade scale in the laboratory sieve analysis.
Classification of earth material shown on this sheet is based on field inspection and should not be construed to imply mechanical analysis unless so stated.

MATERIAL SYMBOLS

Gravel; Sand; Silt; Clay; Sandy clay or clayey sand; Sandy silt or silty sand; Silty clay or clayey silt; Peat or organic matter; Fill material; Shale; Sandstone; Limestone; Metamorphic rock; Igneous rock

CONSISTENCY CLASSIFICATION

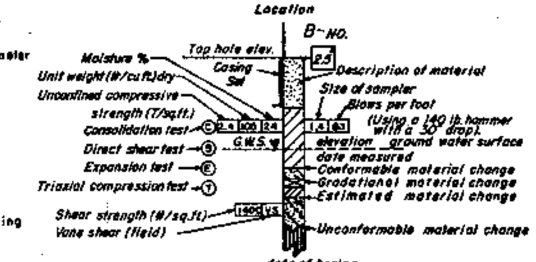
According to the Standard Penetration Test.

No. of blows	Granular	Cohesive
0–5	very loose	very soft
6–10	loose	soft
11–20	slightly compact	stiff
21–35	compact	very stiff
36–70	dense	hard
70+	very dense	very hard

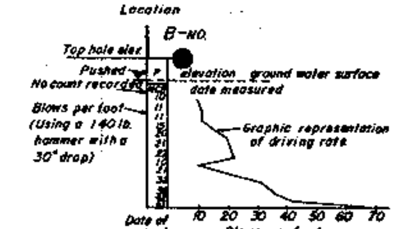
LEGEND OF BORING OPERATIONS

Plan of any boring
Flush penetrometer
2.5" Cone penetrometer
Rotary boring
Auger boring
Sample boring
Jet boring
Diamond core boring
Test pit

ROTARY BORING



PENETRATION TEST



TR. 2-897 | MAP: 41-12

MOORE and TABER
Engineers —— Geologists
Job No. 3164F-3
APPROVED H.R. Taber 12-26-61
LICENSED CIVIL ENGINEER No. 9165

THE RECLAMATION BOARD
STATE OF CALIFORNIA

BEAR CREEK BRIDGE AT SACRAMENTO ROAD

LOG OF TEST BORINGS

Scale 1" = 10' | Date Dec. 20, 1961 | File | Drawing 9 of 24

APPENDIX B

Laboratory Test Results

Project Name: Bear Creek Bridge
BCI File No: 879.5
Date: 5/5/2008
Technician: BWM

blackburn consulting

MOISTURE-DENSITY TESTS

Sample No.	B9-2b	B9-5b	B9-8b	B9-11b	B9-14b	B9-17c	B9-20c
Depth (ft.)	10.5-11.0	20.5-21.0	30.5-31.0	40.5-41.0	50.5-51.0	60.2-60.7	70.5-71.0
Sample Length (in.)	5.50	6.00	5.95	5.86	5.95	5.95	5.76
Diameter (in.)	2.420	2.420	2.400	2.400	2.410	2.410	2.4100
Sample Volume (ft^3)	0.01464	0.01597	0.01558	0.01534	0.01571	0.01571	0.01521
Tare No.	OO	WW	PP	RR	KK	HH	II
Tare (g)	104.9	105.9	105.3	105.5	159.8	156.3	158.6
Wet Soil + Tare (g)	994.7	935.6	935.1	913.5	1076.1	1132.9	1066.5
Dry Soil + Tare (g)	861.4	908.0	790.7	733.4	923.0	1000.5	904.8
Dry Soil Weight (g)	756.5	802.1	685.4	627.9	763.2	844.2	746.2
Water (g)	133.3	27.6	144.4	180.1	153.1	132.4	161.7
Moisture (%)	**17.6**	**3.4**	**21.1**	**28.7**	**20.1**	**15.7**	**21.7**
Wet Density (pcf)	**134.0**	**114.5**	**117.4**	**116.1**	**128.6**	**137.1**	**131.6**
Dry Density (pcf)	**113.9**	**110.7**	**97.0**	**90.2**	**107.1**	**118.5**	**108.2**

Sample: B9-2b Description: Light olive brown / Olive brown sandy clay

Moisture (Appearance): Moist Consistency/Cementation: PP= 4.25

Sample: B9-5b Description: Light olive brown poorly graded sand

Moisture (Appearance): Moist Consistency/Cementation: PP= 0.09375

Sample: B9-8b Description: Light olive brown sandy clay

Moisture (Appearance): Moist Consistency/Cementation: PP= 4.5

Sample: B9-11b Description: Light olive brown clay

Moisture (Appearance): Moist Consistency/Cementation: PP= 2.5

Sample: B9-14b Description: Light olive brown sandy clay

Moisture (Appearance): Moist Consistency/Cementation: PP= +4.5

Sample: B9-17c Description: Light olive brown clayey sand / sandy clay

Moisture (Appearance): Moist Consistency/Cementation: PP= +4.5

Sample: B9-20c Description: Light olive brown sandy clay

Moisture (Appearance): Moist Consistency/Cementation: PP= +4.5

Diameter = 1.44" for 1.5-inch Tubes
Diameter = 1.938" for 2-inch Tubes
Diameter = 2.438" for 2.5-inch Tubes
Diameter= 2.850" for 3.0-inch Shelby Tubes

Project Name: Bear Creek Bridge

BCI File No: 879.5
Date: 5/5/2008
Technician: BWM

blackburn consulting

MOISTURE-DENSITY TESTS

Sample No.	B10-1b	B10-3b	B10-5b	B10-6b	B10-11b	B10-13b	B10-17b
Depth (ft.)	5.5-6.0	15.5-16.0	20.5-21.0	25.5-26.0	45.5-46.0	55.0-55.5	65.5-66.0
Sample Length (in.)	5.70	5.91	5.97	5.74			
Diameter (in.)	2.420	2.420	2.400	2.420			
Sample Volume (ft^3)	0.01517	0.01573	0.01563	0.01528			
Tare No.	DD	MM	FF	EE			
Tare (g)	156.3	168.1	155.8	155.3			
Wet Soil + Tare (g)	1023.3	1066.0	1079.5	1053.9			
Dry Soil + Tare (g)	947.7	914.1	1006.2	931.4			
Dry Soil Weight (g)	791.4	746.0	850.4	776.1			
Water (g)	75.6	151.9	73.3	122.5			
Moisture (%)	**9.6**	**20.4**	**8.6**	**15.8**			
Wet Density (pcf)	**126.0**	**125.8**	**130.3**	**129.7**			
Dry Density (pcf)	**115.0**	**104.5**	**120.0**	**112.0**			

Sample: B10-1b Description: Dark olive brown clayey sand with gravel

Moisture (Appearance): Consistency/Cementation: PP= 4.0

Sample: B10-3b Description: Light olive brown sandy clay

Moisture (Appearance): Consistency/Cementation: PP= 3.5

Sample: B10-5b Description: Olive brown clayey sand

Moisture (Appearance): Consistency/Cementation: PP= +4.5

Sample: B10-6b Description: Light olive brown clayey sand

Moisture (Appearance): Consistency/Cementation: PP= 3.5

Sample: B10-11b Description:

Moisture (Appearance): Consistency/Cementation:

Sample: B10-13b Description:

Moisture (Appearance): Consistency/Cementation:

Sample: B10-17b Description:

Moisture (Appearance): Consistency/Cementation:

Diameter = 1.44" for 1.5-inch Tubes
Diameter = 1.938" for 2-inch Tubes
Diameter = 2.438" for 2.5-inch Tubes
Diameter= 2.850" for 3.0-inch Shelby Tubes

Unconfined Compression Test
ASTM D 2166-00

Project Name	Bear Creek Bridge		
Project Number	879.5		
Sample	B9-13c	Depth	46.0-46.5'
Sample Description	Brown lean clay		
Date	5/7/2008		
Tested By:	JRM		

Original Sample Length	5.80
Original Diameter (in)	2.43
Sample Area (in^2)	4.64

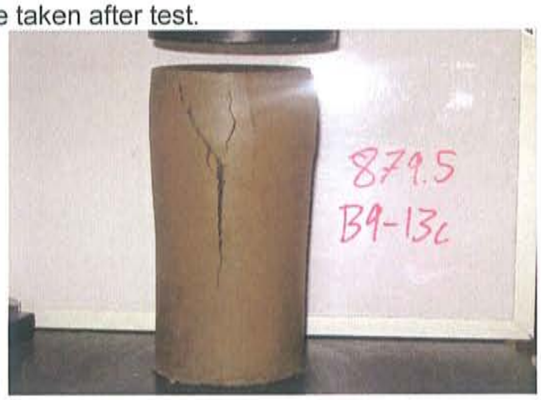
axial strain	**6.8%**
Average cross-sectional area (in^2)	**4.98**
Average cross-sectional area (ft^2)	0.035
Peak Reading	0.395
Maximum Load(lb)	147
Compressive Strength (tsf)	**2.13**

Moisture Density

Wet Sample Weight (g)	1004.7
Tare Number	UU
Tare Weight (g)	105.5
Dry Sample Weight (g)	846.5
Dry Weight (g)	741.0
Water Weight (g)	158.2
Percent Moisture (%)*	21.3
Wet Density (pcf)	127.4
Dry Density (pcf)	104.9

Remarks:

* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

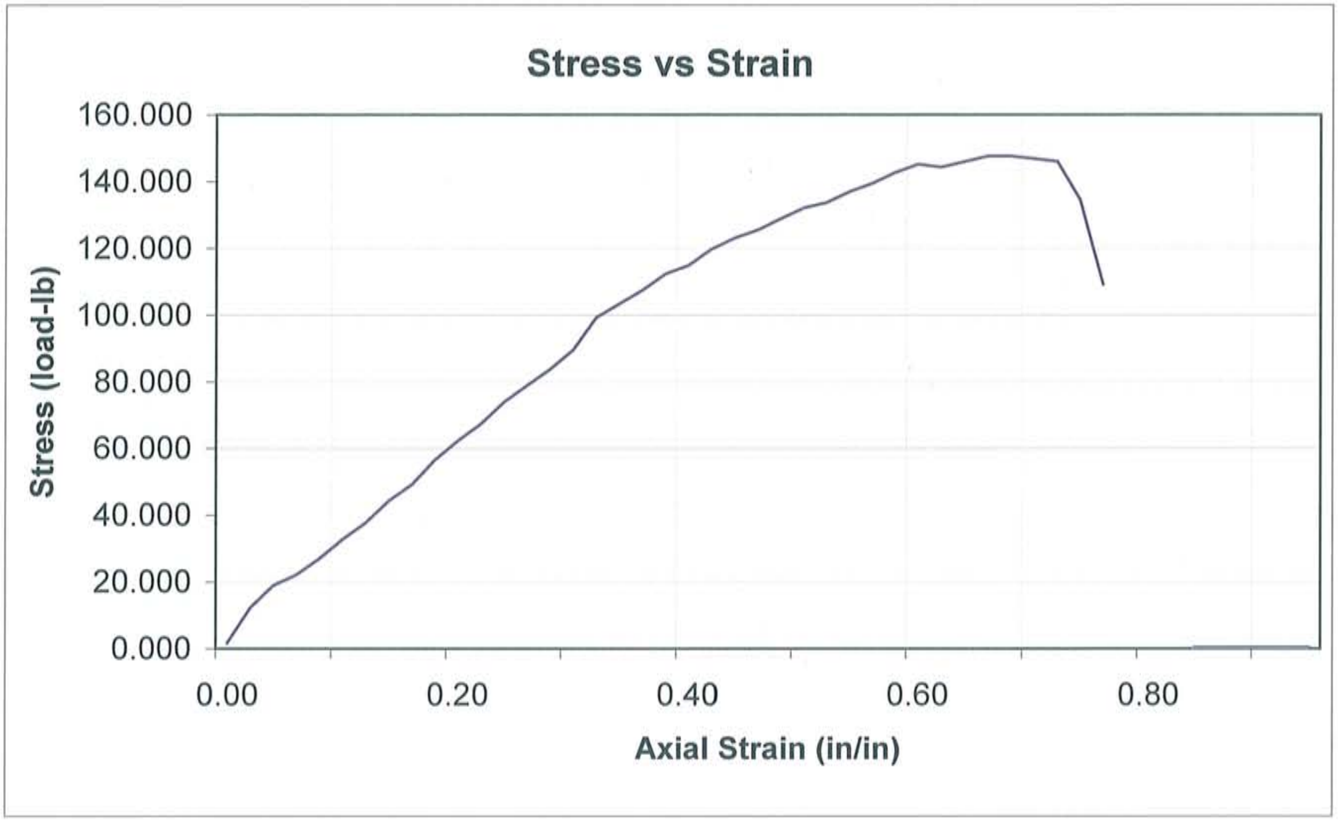
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.005	1.639	0.203	99.148	0.384	145.854		
0.017	12.291	0.214	103.245	0.395	147.493		
0.029	18.846	0.224	107.342	0.406	147.493		
0.040	22.124	0.234	112.258	0.416	146.673		
0.052	27.040	0.246	114.716	0.427	145.854		
0.064	32.776	0.256	119.633	0.437	134.382		
0.077	37.693	0.267	122.910	0.459	108.981		
0.088	44.248	0.279	125.369				
0.100	49.164	0.291	128.646				
0.112	56.539	0.303	131.924				
0.124	62.275	0.315	133.563				
0.136	67.191	0.326	136.840				
0.148	73.746	0.337	139.299				
0.159	78.663	0.349	142.576				
0.171	83.579	0.360	145.034				
0.182	89.315	0.372	144.215				

Project
Bear Creek Bridge
Project Number
879.5
Sample Number
B9-13c
Material Description
Brown lean clay
Tested By
JRM

ASTM D 2166-00



Wet Density (pcf)	127.4
Dry Density (pcf)	104.9
% Moisture	21.3

Unconfined Compressive Strength (tsf) 2.1

Unconfined Compression Test
ASTM D 2166-00

Project Name	Bear Creek Bridge
Project Number	879.5
Sample	B10-7b Depth 30.5-31.0'
Sample Description	Dark yellowish brown sandy clay'
Date	5/6/2008
Tested By:	JRM

Original Sample Length	5.58
Original Diameter (in)	2.42
Sample Area (in^2)	4.60

axial strain	**3.5%**
Average cross-sectional area (in^2)	**4.77**
Average cross-sectional area (ft^2)	0.033
Peak Reading	0.195
Maximum Load(lb)	67
Compressive Strength (tsf)	**1.02**

Moisture Density

Wet Sample Weight (g)	935.2
Tare Number	RR
Tare Weight (g)	150.0
Dry Sample Weight (g)	818.5
Dry Weight (g)	668.5
Water Weight (g)	116.7
Percent Moisture (%)*	17.5
Wet Density (pcf)	116.5
Dry Density (pcf)	99.2

Remarks:
* % moisture taken after test.

Compression Tests

Dial reading @ 0 lb	0.000

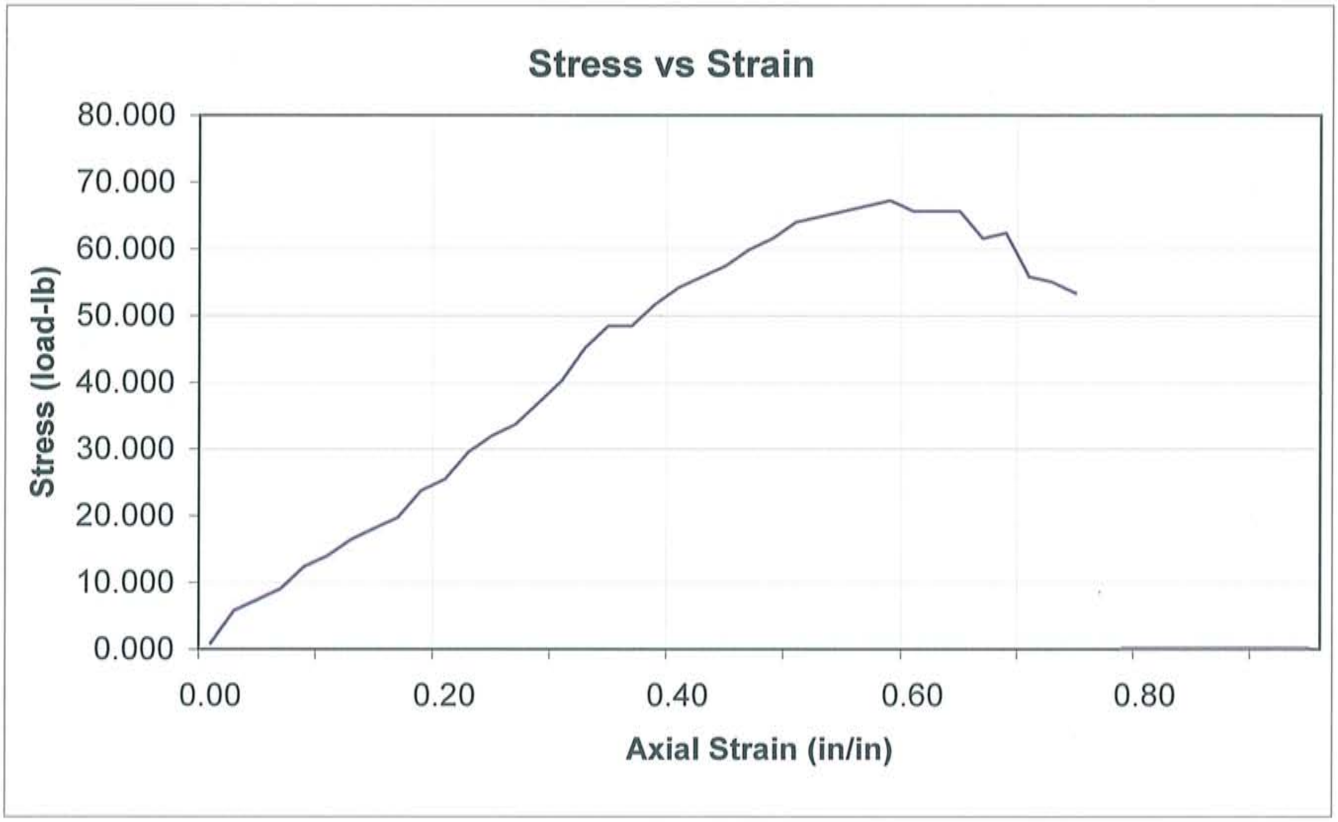
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.001	0.819	0.094	45.067	0.184	65.552		
0.006	5.736	0.099	48.345	0.190	61.455		
0.012	7.375	0.105	48.345	0.195	62.275		
0.018	9.013	0.110	51.622	0.201	55.719		
0.025	12.291	0.116	54.081	0.201	54.900		
0.031	13.930	0.122	55.719	0.201	53.261		
0.037	16.388	0.127	57.358				
0.043	18.027	0.132	59.816				
0.049	19.666	0.138	61.455				
0.054	23.763	0.144	63.913				
0.061	25.402	0.149	64.733				
0.066	29.499	0.155	65.552				
0.072	31.957	0.161	66.372				
0.077	33.596	0.167	67.191				
0.083	36.873	0.172	65.552				
0.088	40.151	0.178	65.552				

Project
Bear Creek Bridge
Project Number
879.5
Sample Number
B10-7b
Material Description
Dark yellowish brown sandy clay'
Tested By
JRM

ASTM D 2166-00



Wet Density (pcf)	116.5
Dry Density (pcf)	99.2
% Moisture	17.5

Unconfined Compressive Strength (tsf) 1.0

Unconfined Compression Test
ASTM D 2166-00

Project Name	Bear Creek Bridge
Project Number	879.5
Sample	B10-8c Depth 36.0-36.5'
Sample Description	Strong brown silt
Date	5/6/2008
Tested By:	JRM

Original Sample Length	5.65
Original Diameter (in)	2.43
Sample Area (in^2)	4.64

axial strain	**9.3%**
Average cross-sectional area (in^2)	**5.11**
Average cross-sectional area (ft^2)	0.036
Peak Reading	0.526
Maximum Load(lb)	107
Compressive Strength (tsf)	**1.51**

Moisture Density

Wet Sample Weight (g)	904.1
Tare Number	WW
Tare Weight (g)	105.7
Dry Sample Weight (g)	783.8
Dry Weight (g)	678.1
Water Weight (g)	120.3
Percent Moisture (%)*	17.7
Wet Density (pcf)	116.1
Dry Density (pcf)	98.6

Remarks:

* % moisture taken after test.

Compression Tests

Dial reading @ 0 lb	0.000

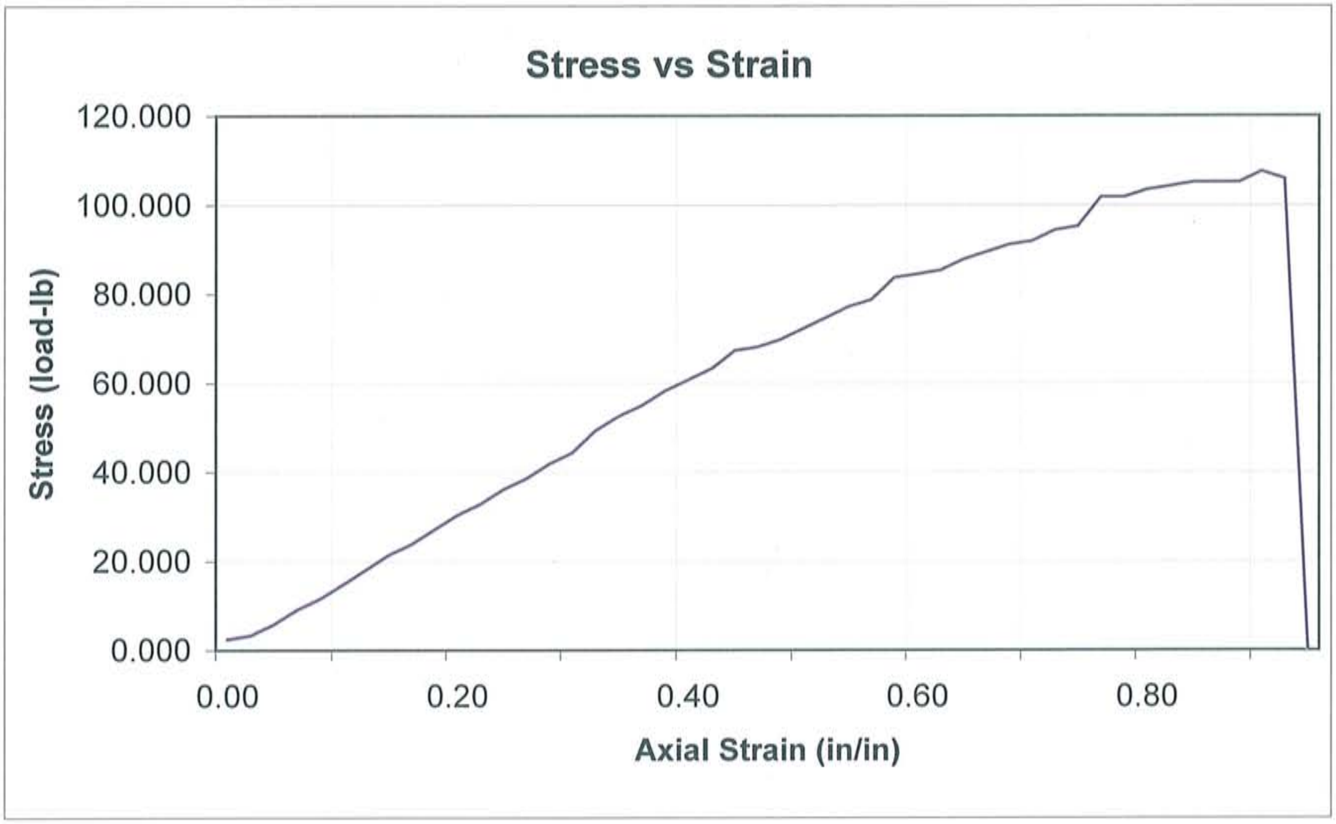
Rate of Strain=0.056in/min

Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
	2.458	0.188	49.164	0.368	87.676		
0.005	3.278	0.200	52.442	0.380	89.315		
0.017	5.736	0.212	54.900	0.392	90.954		
0.029	9.013	0.223	58.178	0.404	91.773		
0.040	11.472	0.235	60.636	0.416	94.231		
0.052	14.749	0.247	63.094	0.427	95.051		
0.063	18.027	0.258	67.191	0.450	101.606		
0.074	21.304	0.269	68.010	0.461	101.606		
0.086	23.763	0.280	69.649	0.472	103.245		
0.097	27.040	0.291	72.108	0.483	104.064		
0.108	30.318	0.302	74.566	0.495	104.884		
0.119	32.776	0.312	77.024	0.504	104.884		
0.131	36.054	0.323	78.663	0.515	104.884		
0.142	38.512	0.334	83.579	0.526	107.342		
0.154	41.790	0.346	84.399	0.536	105.703		
0.165	44.248	0.357	85.218				

Project
Bear Creek Bridge
Project Number
879.5
Sample Number
B10-8c
Material Description
Strong brown silt
Tested By
JRM

ASTM D 2166-00



Wet Density (pcf)	116.1
Dry Density (pcf)	98.6
% Moisture	17.7

Unconfined Compressive Strength (tsf) 1.5

Unconfined Compression Test
ASTM D 2166-00

Project Name	Bear Creek Bridge		
Project Number	879.5		
Sample	B10-10b	Depth	40.5-41.0
Sample Description	Brown clay		
Date	5/7/2008		
Tested By:	JRM		

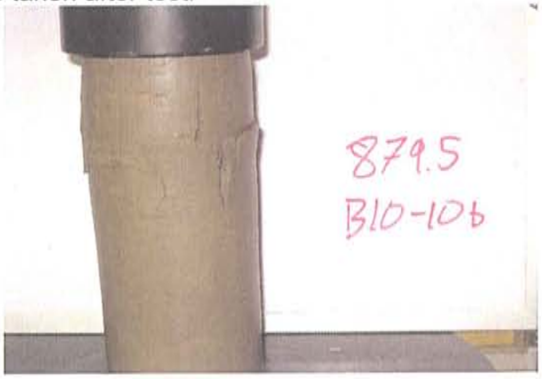
Original Sample Length	6.00
Original Diameter (in)	2.43
Sample Area (in^2)	4.64

axial strain	**5.5%**
Average cross-sectional area (in^2)	**4.91**
Average cross-sectional area (ft^2)	0.034
Peak Reading	0.329
Maximum Load(lb)	116
Compressive Strength (tsf)	**1.70**

Moisture Density

Wet Sample Weight (g)	995.6
Tare Number	WW
Tare Weight (g)	105.6
Dry Sample Weight (g)	811.7
Dry Weight (g)	706.1
Water Weight (g)	183.9
Percent Moisture (%)*	26.0
Wet Density (pcf)	121.8
Dry Density (pcf)	96.7

Remarks:
* % moisture taken after test.



Compression Tests

Dial reading @ 0 lb	0.000

Rate of Strain=0.056in/min

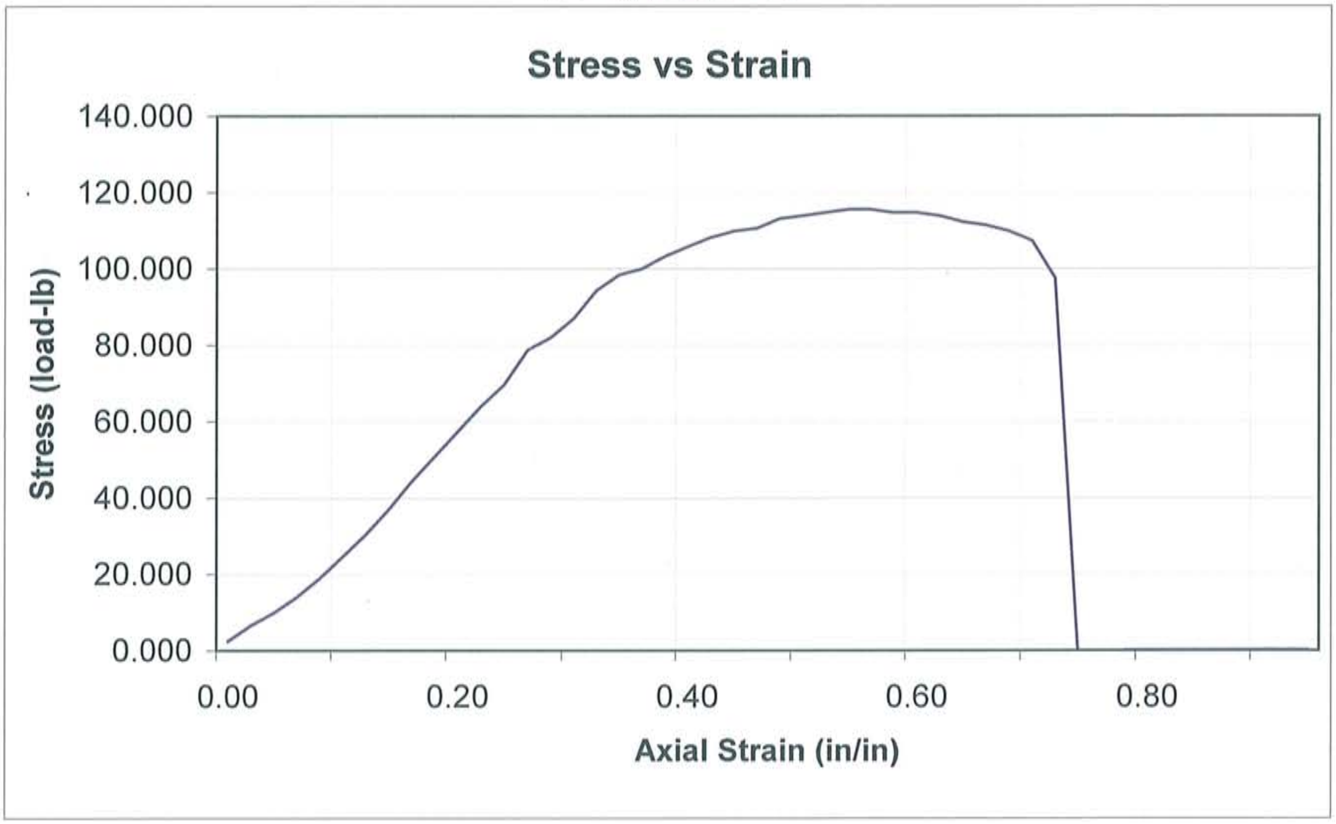
Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.005	2.458	0.204	94.231	0.387	112.258		
0.018	6.555	0.216	98.328	0.399	111.439		
0.030	9.833	0.228	99.967	0.411	109.800		
0.041	13.930	0.241	103.245	0.423	107.342		
0.053	18.846	0.252	105.703	0.428	97.509		
0.064	24.582	0.264	108.161				
0.075	30.318	0.275	109.800				
0.086	36.873	0.286	110.619				
0.098	44.248	0.297	113.078				
0.108	50.803	0.307	113.897				
0.120	57.358	0.318	114.716				
0.131	63.913	0.329	115.536				
0.144	69.649	0.340	115.536				
0.155	78.663	0.352	114.716				
0.166	81.940	0.363	114.716				
0.179	86.857	0.375	113.897				

blackburn consulting

Project
Bear Creek Bridge
Project Number
879.5
Sample Number
B10-10b
Material Description
Brown clay
Tested By
JRM

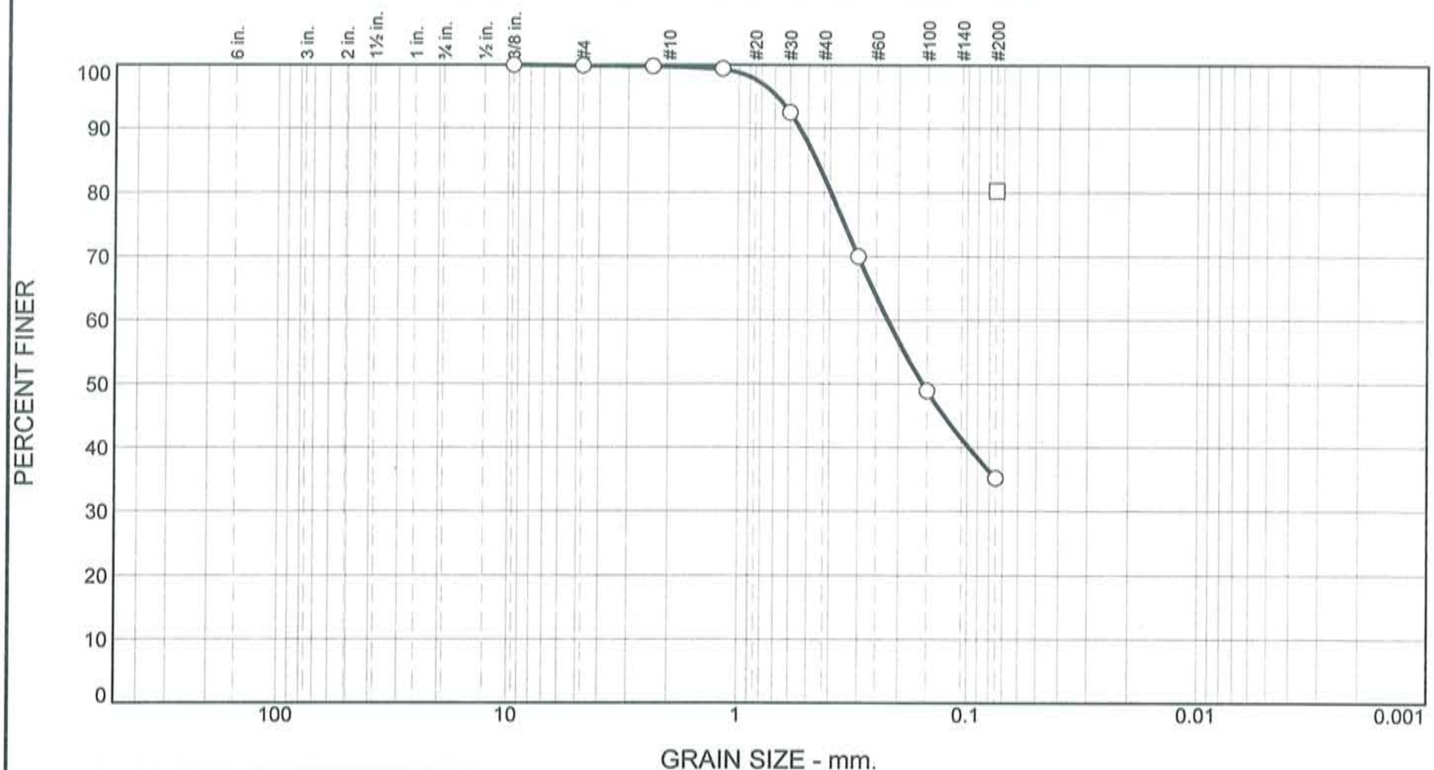
ASTM D 2166-00



Wet Density (pcf)	121.8
Dry Density (pcf)	96.7
% Moisture	26.0

Unconfined Compressive Strength (tsf) 1.7

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.1	0.2	17.3	47.2	35.2	
□							80.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	22	15	0.4588	0.2225	0.1565					
□	49	23								

Material Description	USCS	AASHTO
○ Brown SILTY, CLAYEY SAND	SC-SM	A-2-4(0)
□ Brown Lean CLAY with SAND	CL	

Project No. 879.5 **Client:** Mark Thomas & Company, Inc.
Project: Bear Creek Bridge
○ **Sample Source:** Boring B9 **Depth:** 11.0-11.5' **Sample No.:** B9-08-2c
□ **Sample Source:** Boring B9 **Depth:** 41.5-41.0' **Sample No.:** B9-08-12

Remarks:

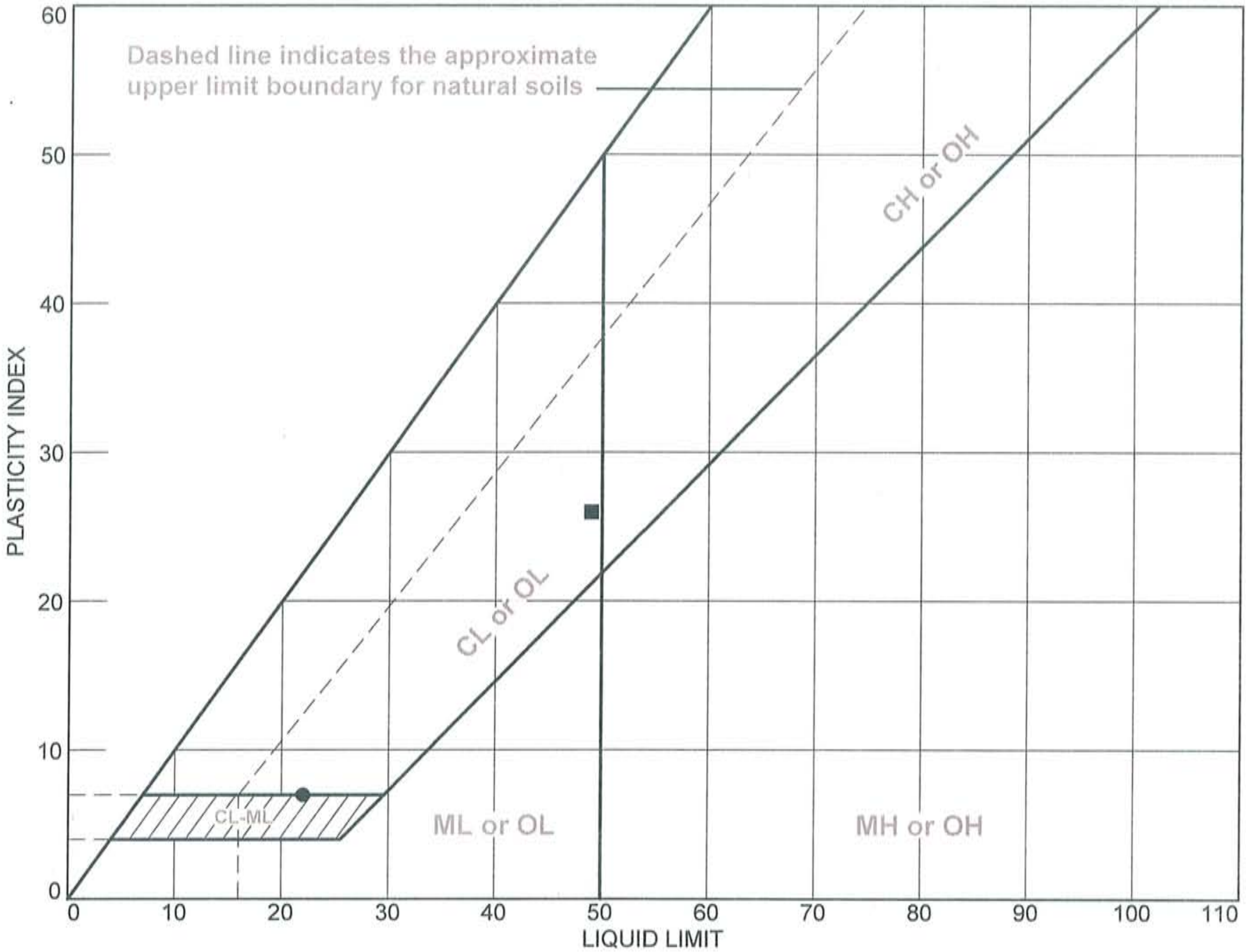
Blackburn Consulting

W. Sacramento, CA

Figure

Tested By: ______________________ **Checked By:** ______________________

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring B9	B9-08-2c	11.0-11.5'		15	22	7	SC-SM
■	Boring B9	B9-08-12	41.5-41.0'		23	49	26	CL

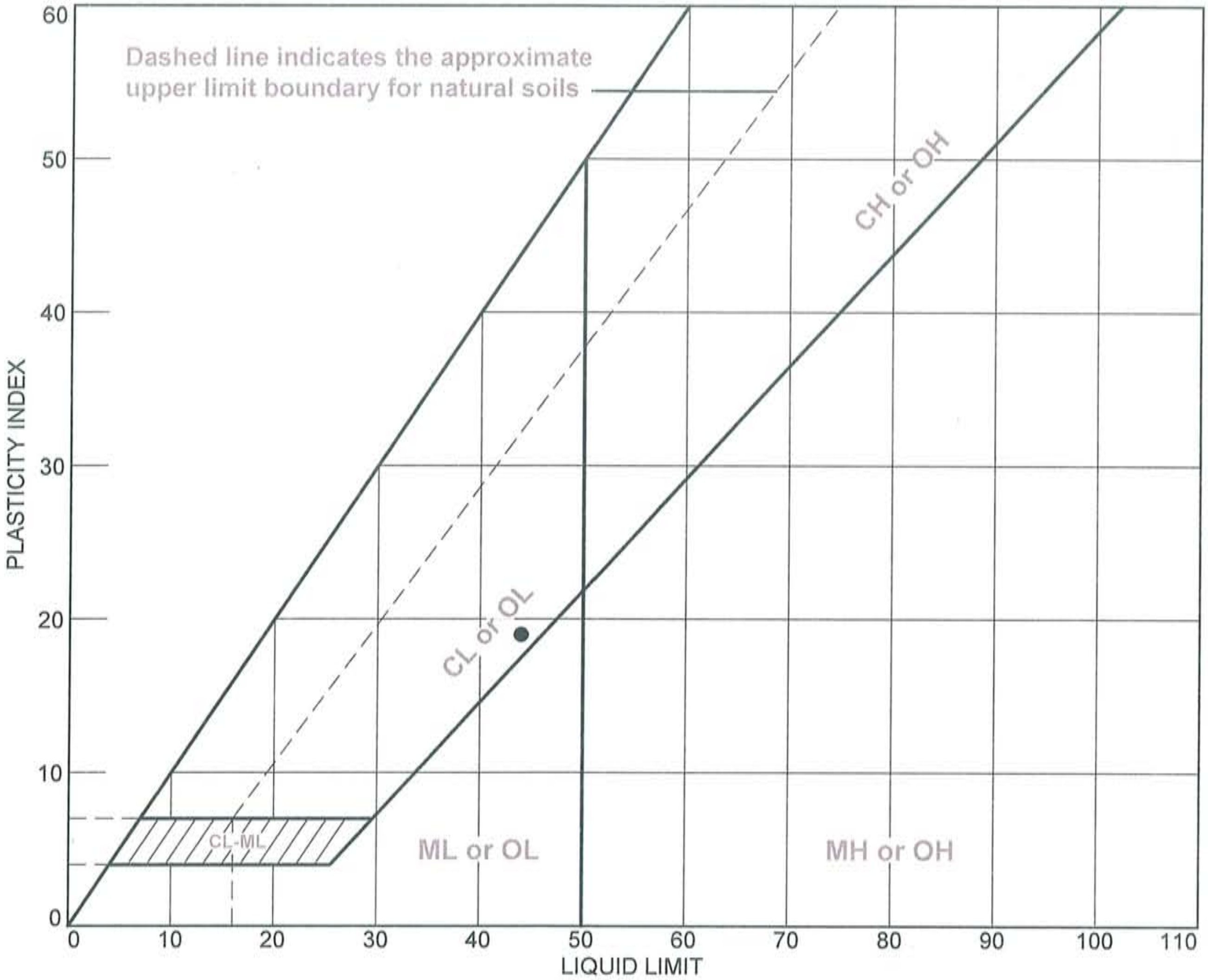
Blackburn Consulting

W. Sacramento, CA

Client: Mark Thomas & Company, Inc.
Project: Bear Creek Bridge
Project No.: 879.5 **Figure**

Tested By: ______________________ **Checked By:** ______________________

LIQUID AND PLASTIC LIMITS TEST REPORT



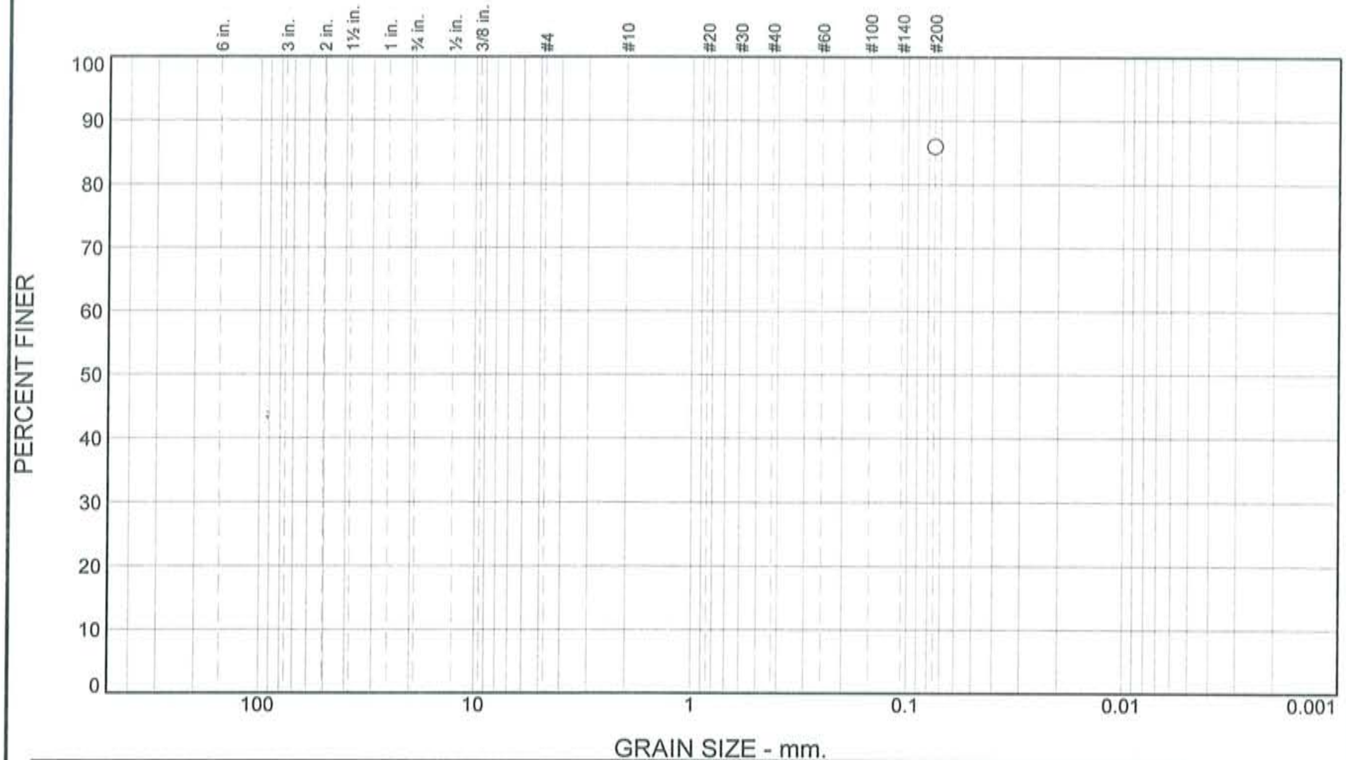
SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	Boring B10	B10-08-9	36.5-38.0		25	44	19	CL

Blackburn Consulting W. Sacramento, CA	**Client:** Mark Thomas & Company, Inc. **Project:** Bear Creek Bridge **Project No.:** 879.5 **Figure**

Tested By: ____________ **Checked By:** ____________

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							86.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	44	25								

Material Description	USCS	AASHTO
○ Brown Lean CLAY	CL	

Project No. 879.5 **Client:** Mark Thomas & Company, Inc.
Project: Bear Creek Bridge
○ **Sample Source:** Boring B10 **Depth:** 36.5-38.0 **Sample No.:** B10-08-9

Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure

Tested By: ____________ **Checked By:** ____________

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/09/2008
Date Submitted 05/06/2008

To: John Massetti
Blackburn Consulting
2437 Front Street
W. Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager RH

The reported analysis was requested for the following location:
Location : BEAR CREEK BRIDGE Site ID : B9-8.
Your purchase order number is 879.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 53179-106450.

EVALUATION FOR SOIL CORROSION

Soil pH	6.98		
Minimum Resistivity	1.05	ohm-cm (x1000)	
Chloride	13.3 ppm	00.00133	%
Sulfate	64.3 ppm	00.00643	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/09/2008
Date Submitted 05/06/2008

To: John Massetti
Blackburn Consulting
2437 Front Street
W. Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager RA

The reported analysis was requested for the following location:
Location : BEAR CREEK BRIDGE Site ID : B9-16C.
Your purchase order number is 879.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 53179-106451.

EVALUATION FOR SOIL CORROSION

Soil pH	7.13		
Minimum Resistivity	2.81 ohm-cm (x1000)		
Chloride	11.4 ppm	00.00114	%
Sulfate	5.9 ppm	00.00059	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422

Sunland Analytical
11353 Pyrites Way, Suite 4
Rancho Cordova, CA 95670
(916) 852-8557

Date Reported 05/09/2008
Date Submitted 05/06/2008

To: John Massetti
Blackburn Consulting
2437 Front Street
W. Sacramento, CA 95691

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager RH

The reported analysis was requested for the following location:
Location : BEAR CREEK BRIDGE Site ID : B10-3.
Your purchase order number is 879.5.
Thank you for your business.

* For future reference to this analysis please use SUN # 53179-106452.

EVALUATION FOR SOIL CORROSION

Soil pH	7.12		
Minimum Resistivity	1.72	ohm-cm (x1000)	
Chloride	13.8 ppm	00.00138	%
Sulfate	53.2 ppm	00.00532	%

METHODS
pH and Min.Resistivity CA DOT Test #643
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APPENDIX C

Abutments 1 & 4: Class 90 Pile Analysis

ATTACHMENT G

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: C:\PROGRA~1\DRIVEN\BEARABUT.DVN
Project Name: Bear Creek Br Abutments — Project Date: 12/05/2008
Project Client: MTCo
Computed By: WEN
Project Manager: WEN

PILE INFORMATION

Pile Type: Concrete Pile
Top of Pile: 0.00 ft
Length of Square Side: 12.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	43.00 ft
	- Driving/Restrike	43.00 ft
	- Ultimate:	43.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	4.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	4.00 ft	0.00%	134.00 pcf	33.0/33.0	Nordlund
2	Cohesionless	10.00 ft	0.00%	126.00 pcf	38.0/38.0	Nordlund
3	Cohesive	20.00 ft	0.00%	117.00 pcf	2500.00 psf	T-79 Concrete
4	Cohesionless	1.56 ft	0.00%	128.00 pcf	38.0/38.0	Nordlund
5	Cohesionless	1.56 ft	0.00%	128.00 pcf	38.0/38.0	Nordlund
6	Cohesionless	3.12 ft	0.00%	128.00 pcf	38.0/38.0	Nordlund
7	Cohesionless	6.25 ft	0.00%	128.00 pcf	38.0/38.0	Nordlund
8	Cohesionless	12.50 ft	0.00%	128.00 pcf	38.0/38.0	Nordlund

ATTACHMENT G

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
3.99 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
3.99 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
4.00 ft	Cohesionless	0.00 psf	0.00	N/A	0.00 Kips
4.01 ft	Cohesionless	536.63 psf	29.15	N/A	0.02 Kips
13.01 ft	Cohesionless	1103.63 psf	29.15	N/A	42.15 Kips
13.99 ft	Cohesionless	1165.37 psf	29.15	N/A	49.35 Kips
14.01 ft	Cohesive	N/A	N/A	1248.54 psf	49.47 Kips
23.01 ft	Cohesive	N/A	N/A	1436.04 psf	101.18 Kips
32.01 ft	Cohesive	N/A	N/A	1623.54 psf	166.38 Kips
33.99 ft	Cohesive	N/A	N/A	1664.79 psf	182.54 Kips
34.01 ft	Cohesionless	4136.64 psf	29.15	N/A	182.80 Kips
35.55 ft	Cohesionless	4235.33 psf	29.15	N/A	210.48 Kips
35.57 ft	Cohesionless	4336.58 psf	29.15	N/A	210.85 Kips
37.12 ft	Cohesionless	4435.33 psf	29.15	N/A	239.86 Kips
37.13 ft	Cohesionless	4536.64 psf	29.15	N/A	240.25 Kips
40.24 ft	Cohesionless	4735.36 psf	29.15	N/A	302.58 Kips
40.26 ft	Cohesionless	4936.64 psf	29.15	N/A	303.00 Kips
42.99 ft	Cohesionless	5111.36 psf	29.15	N/A	362.15 Kips
43.01 ft	Cohesionless	5288.33 psf	29.15	N/A	362.60 Kips
46.49 ft	Cohesionless	5402.47 psf	29.15	N/A	442.30 Kips
46.51 ft	Cohesionless	5517.93 psf	29.15	N/A	442.76 Kips
55.51 ft	Cohesionless	5813.13 psf	29.15	N/A	664.54 Kips
58.99 ft	Cohesionless	5927.27 psf	29.15	N/A	756.33 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.00 psf	0.00	0.00 Kips	0.00 Kips
3.99 ft	Cohesionless	0.00 psf	0.00	0.00 Kips	0.00 Kips
3.99 ft	Cohesionless	0.00 psf	0.00	0.00 Kips	0.00 Kips
4.00 ft	Cohesionless	536.00 psf	47.20	50.00 Kips	16.33 Kips
4.01 ft	Cohesionless	537.26 psf	110.40	268.60 Kips	42.82 Kips
13.01 ft	Cohesionless	1671.26 psf	110.40	268.60 Kips	133.21 Kips
13.99 ft	Cohesionless	1794.74 psf	110.40	268.60 Kips	143.06 Kips
14.01 ft	Cohesive	N/A	N/A	N/A	22.50 Kips
23.01 ft	Cohesive	N/A	N/A	N/A	22.50 Kips
32.01 ft	Cohesive	N/A	N/A	N/A	22.50 Kips
33.99 ft	Cohesive	N/A	N/A	N/A	22.50 Kips
34.01 ft	Cohesionless	4137.28 psf	110.40	268.60 Kips	268.60 Kips
35.55 ft	Cohesionless	4334.66 psf	110.40	268.60 Kips	268.60 Kips
35.57 ft	Cohesionless	4337.22 psf	110.40	268.60 Kips	268.60 Kips
37.12 ft	Cohesionless	4534.72 psf	110.40	268.60 Kips	268.60 Kips
37.13 ft	Cohesionless	4537.28 psf	110.40	268.60 Kips	268.60 Kips
40.24 ft	Cohesionless	4934.72 psf	110.40	268.60 Kips	268.60 Kips
40.26 ft	Cohesionless	4937.28 psf	110.40	268.60 Kips	268.60 Kips
42.99 ft	Cohesionless	5286.72 psf	110.40	268.60 Kips	268.60 Kips
43.01 ft	Cohesionless	5288.66 psf	110.40	268.60 Kips	268.60 Kips
46.49 ft	Cohesionless	5516.94 psf	110.40	268.60 Kips	268.60 Kips
46.51 ft	Cohesionless	5518.26 psf	110.40	268.60 Kips	268.60 Kips
55.51 ft	Cohesionless	6108.66 psf	110.40	268.60 Kips	268.60 Kips
58.99 ft	Cohesionless	6336.94 psf	110.40	268.60 Kips	268.60 Kips

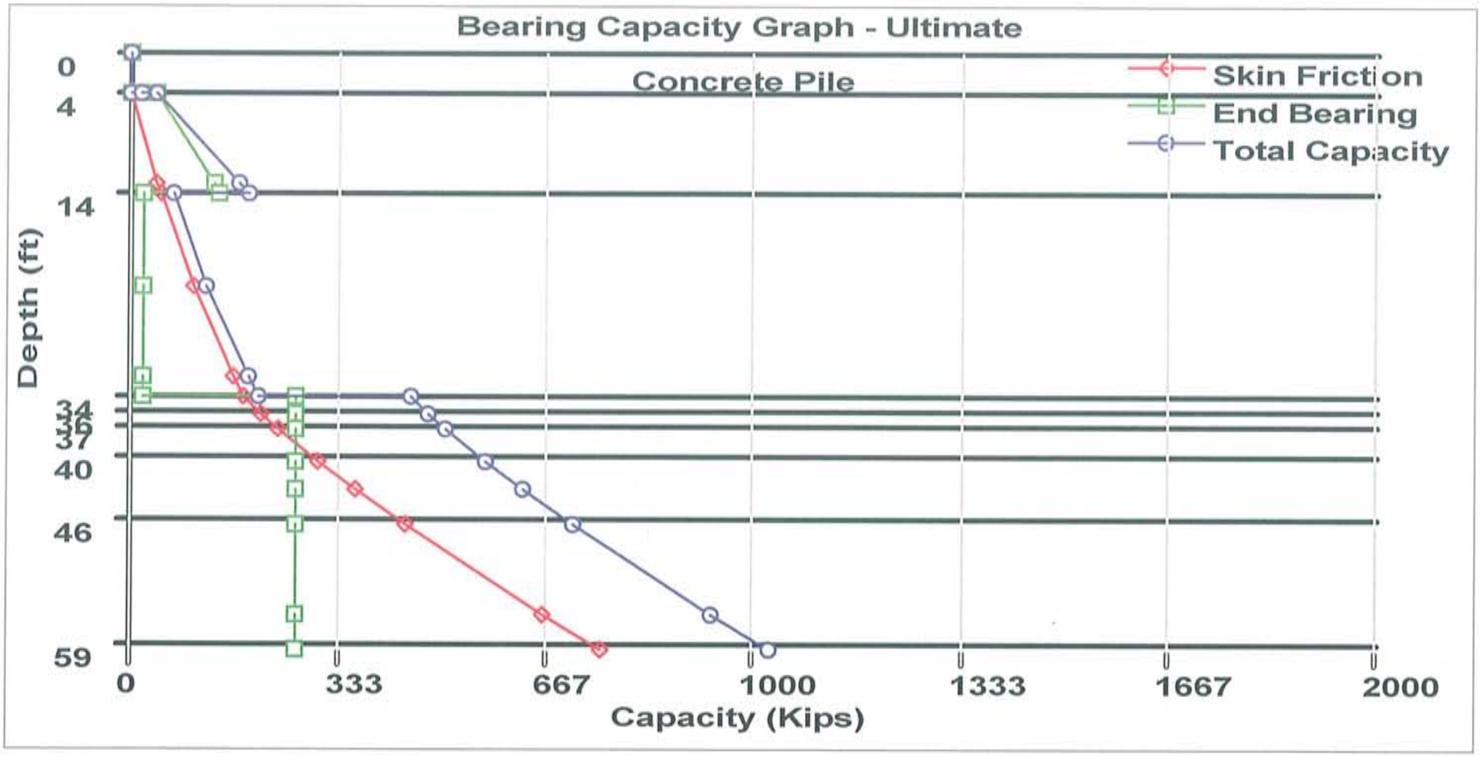
ATTACHMENT G

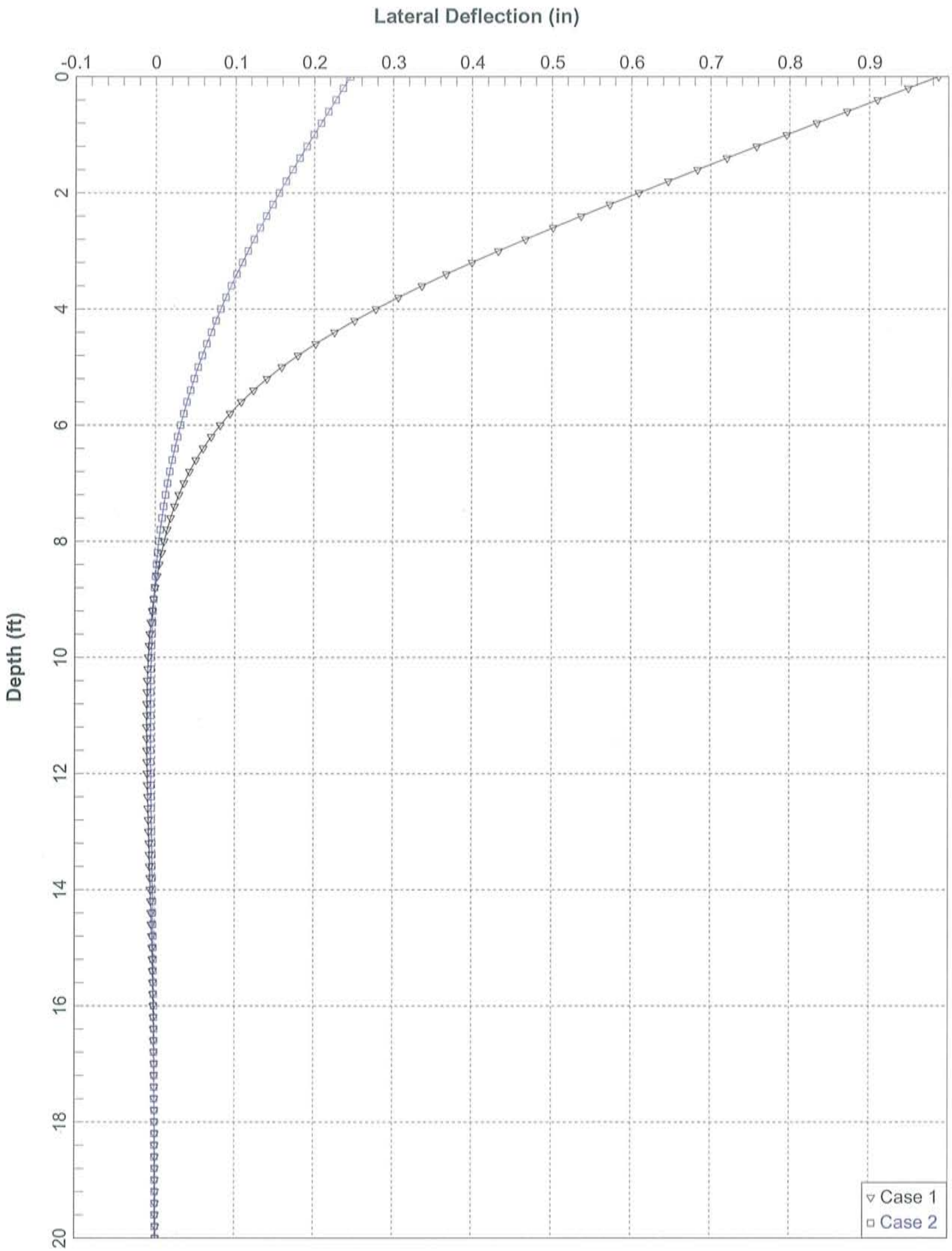
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
3.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
3.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.00 ft	0.00 Kips	16.33 Kips	16.33 Kips
4.01 ft	0.02 Kips	42.82 Kips	42.85 Kips
13.01 ft	42.15 Kips	133.21 Kips	175.36 Kips
13.99 ft	49.35 Kips	143.06 Kips	192.40 Kips
14.01 ft	49.47 Kips	22.50 Kips	71.97 Kips
23.01 ft	101.18 Kips	22.50 Kips	123.68 Kips
32.01 ft	166.38 Kips	22.50 Kips	188.88 Kips
33.99 ft	182.54 Kips	22.50 Kips	205.04 Kips
34.01 ft	182.80 Kips	268.60 Kips	451.40 Kips
35.55 ft	210.48 Kips	268.60 Kips	479.08 Kips
35.57 ft	210.85 Kips	268.60 Kips	479.45 Kips
37.12 ft	239.86 Kips	268.60 Kips	508.46 Kips
37.13 ft	240.25 Kips	268.60 Kips	508.85 Kips
40.24 ft	302.58 Kips	268.60 Kips	571.18 Kips
40.26 ft	303.00 Kips	268.60 Kips	571.60 Kips
42.99 ft	362.15 Kips	268.60 Kips	630.75 Kips
43.01 ft	362.60 Kips	268.60 Kips	631.20 Kips
46.49 ft	442.30 Kips	268.60 Kips	710.90 Kips
46.51 ft	442.76 Kips	268.60 Kips	711.36 Kips
55.51 ft	664.54 Kips	268.60 Kips	933.14 Kips
58.99 ft	756.33 Kips	268.60 Kips	1024.93 Kips

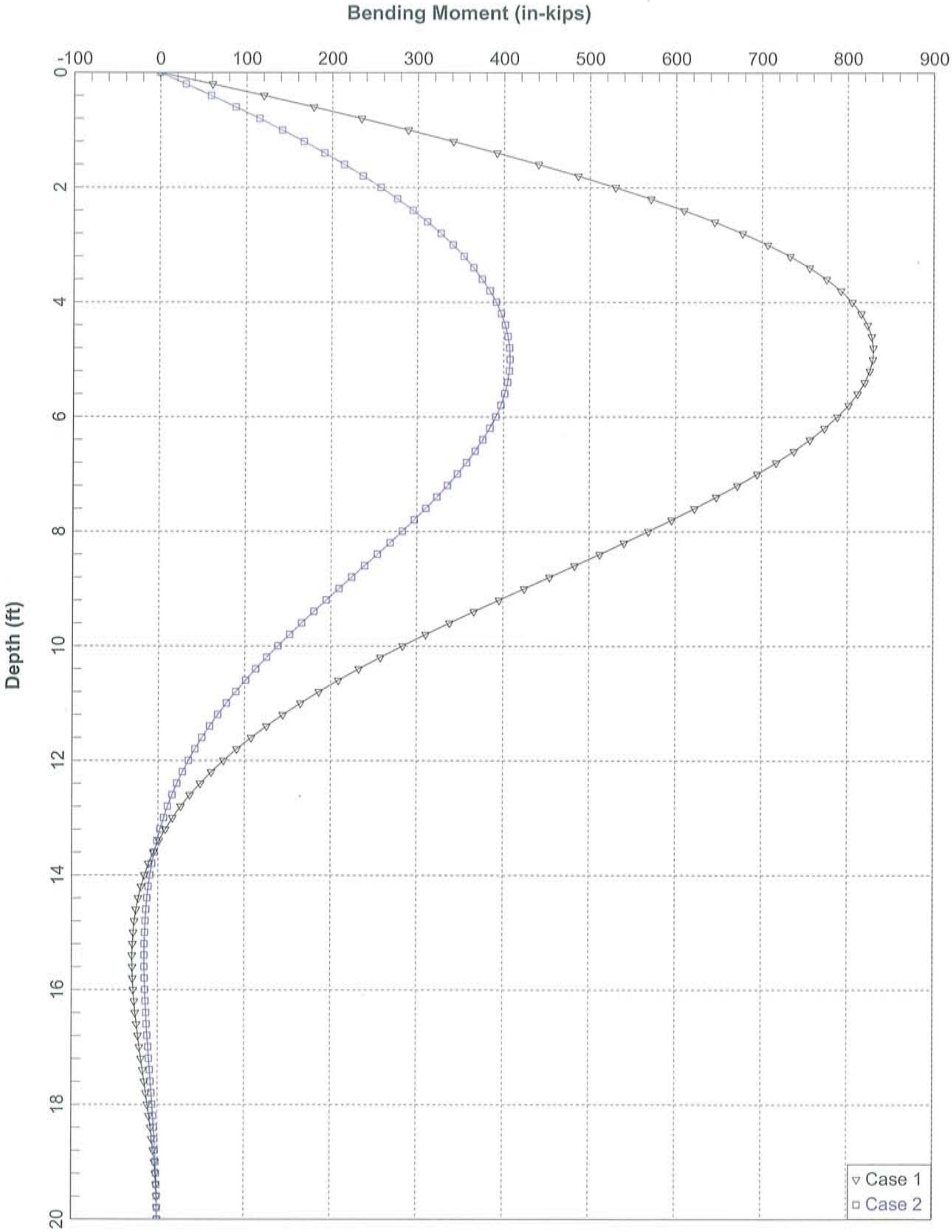
Filename: C:\PROGRA~1\DRIVEN\BEARABUT.DVN

ATTACHMENT G





Abutment Alt X Pile, Longitudinal Direction, Case 1: Nominal, Case 2: Service



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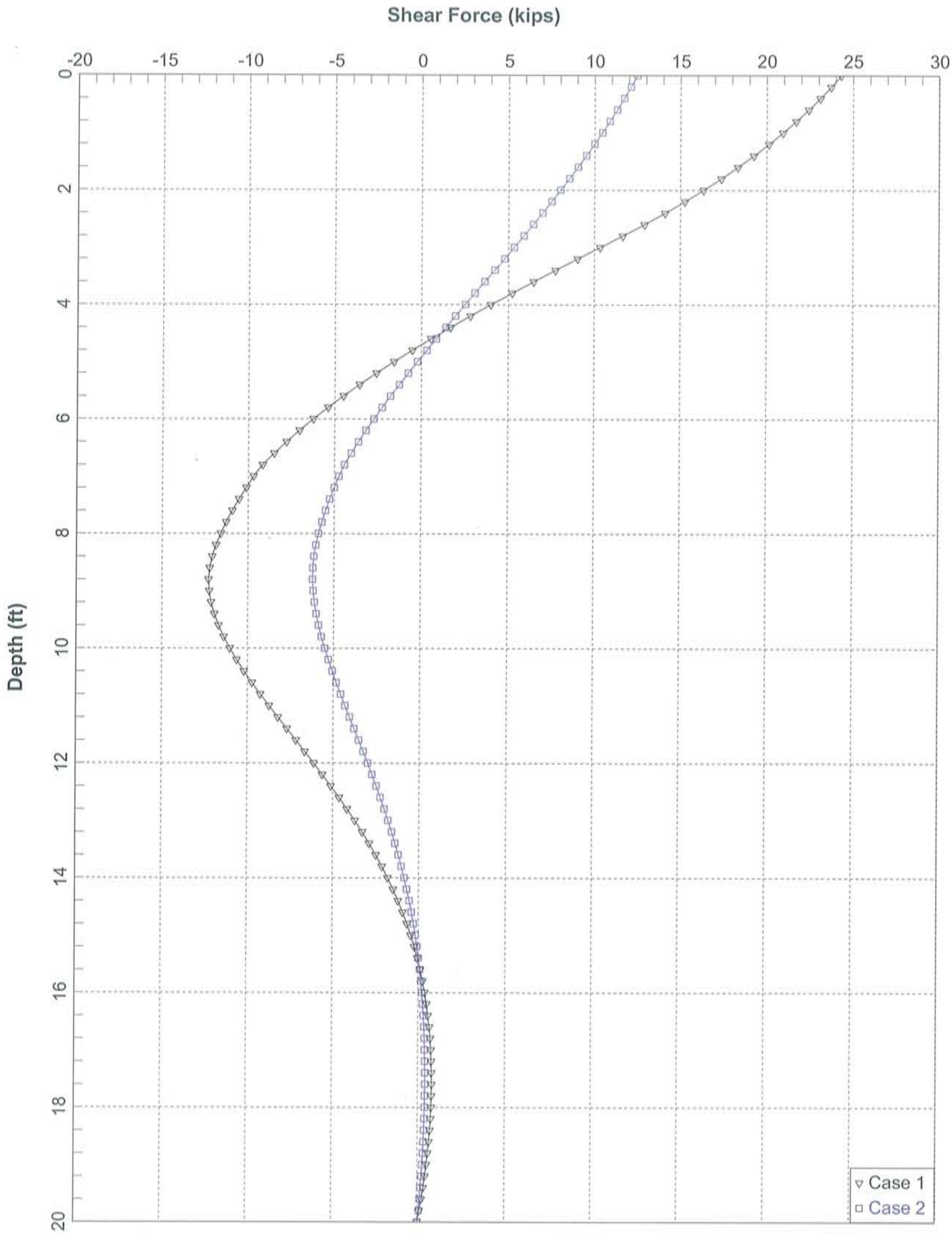

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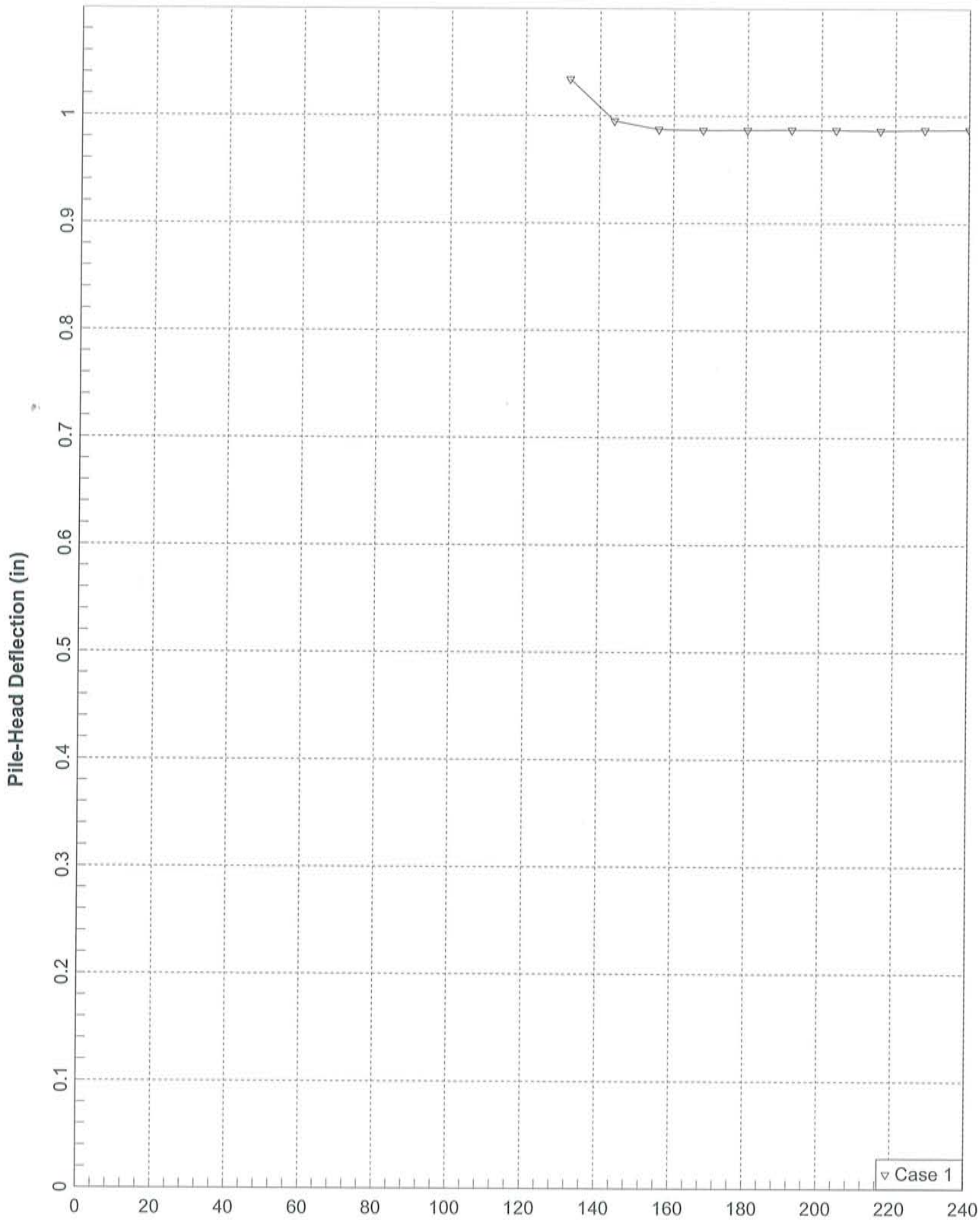
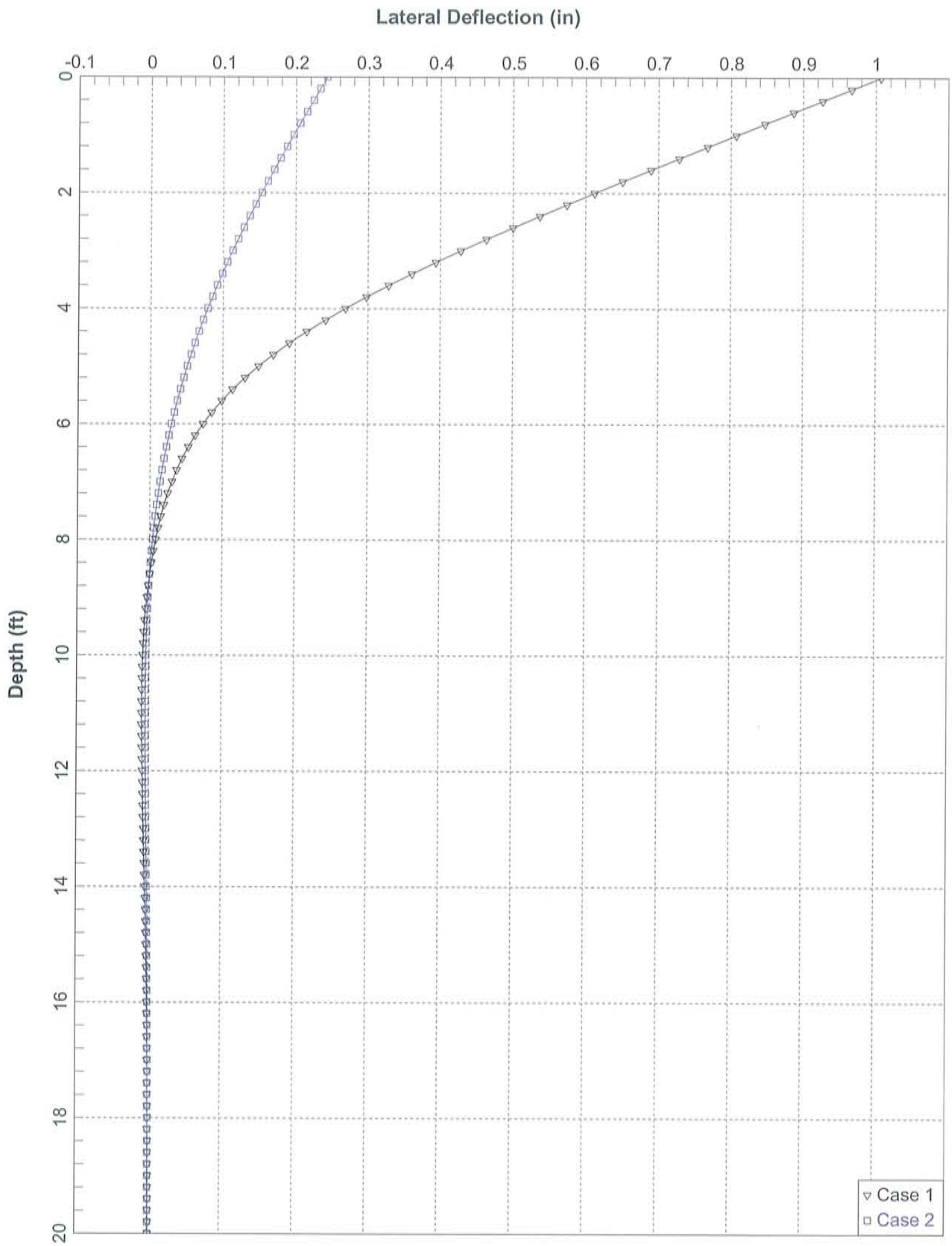


Pile Length (in)

Abutment Alt X Pile, Longitudinal Direction, Nominal Case



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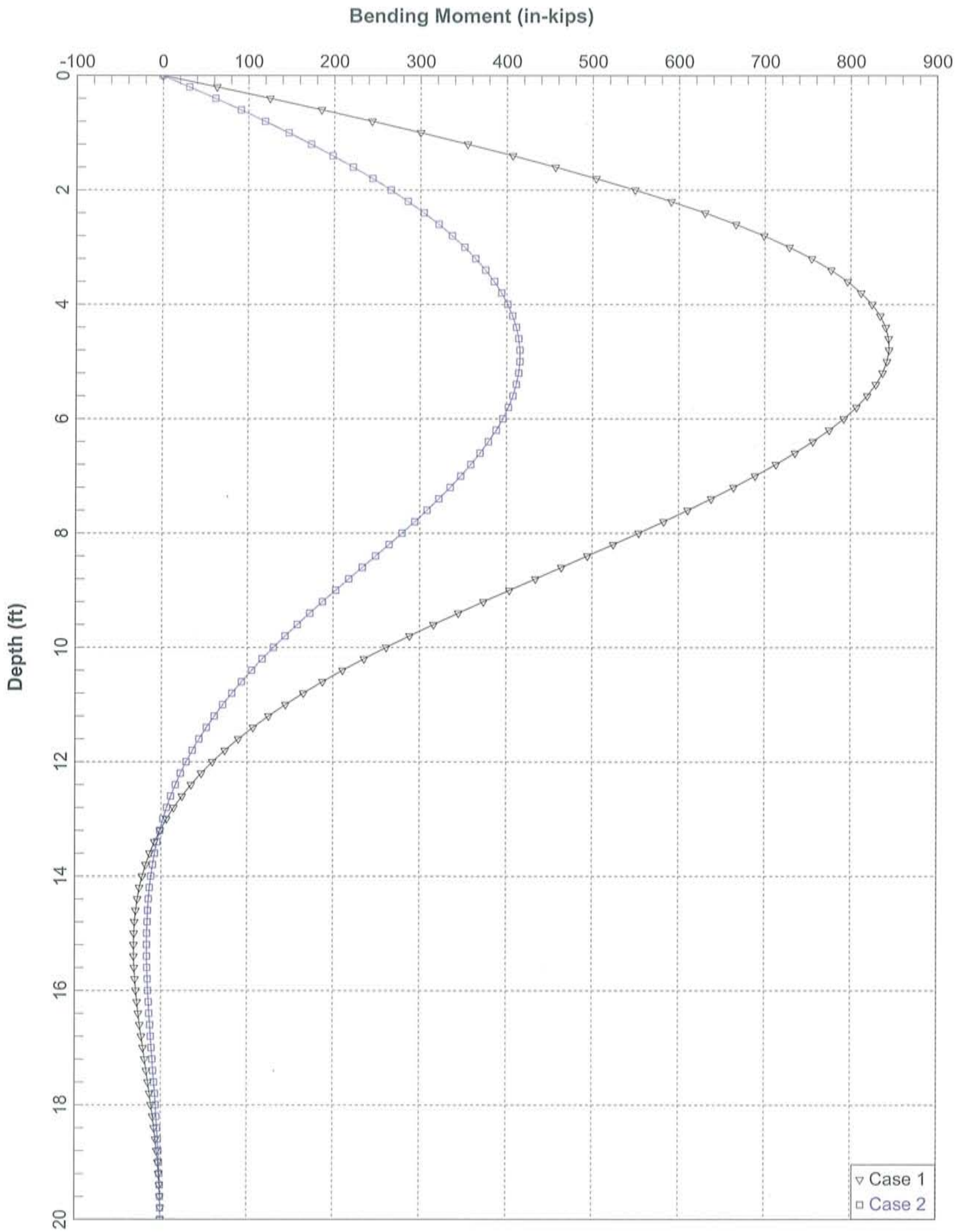

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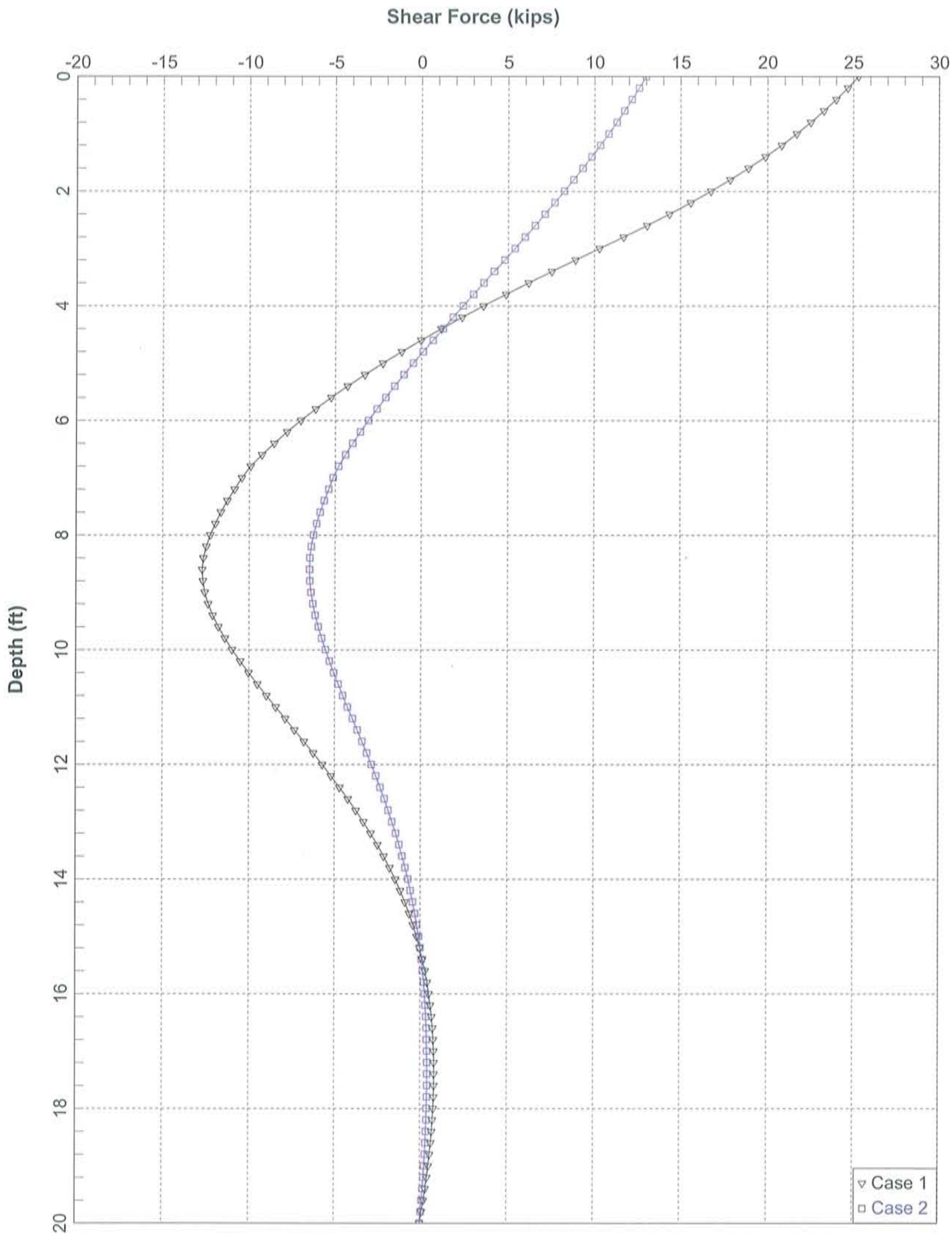

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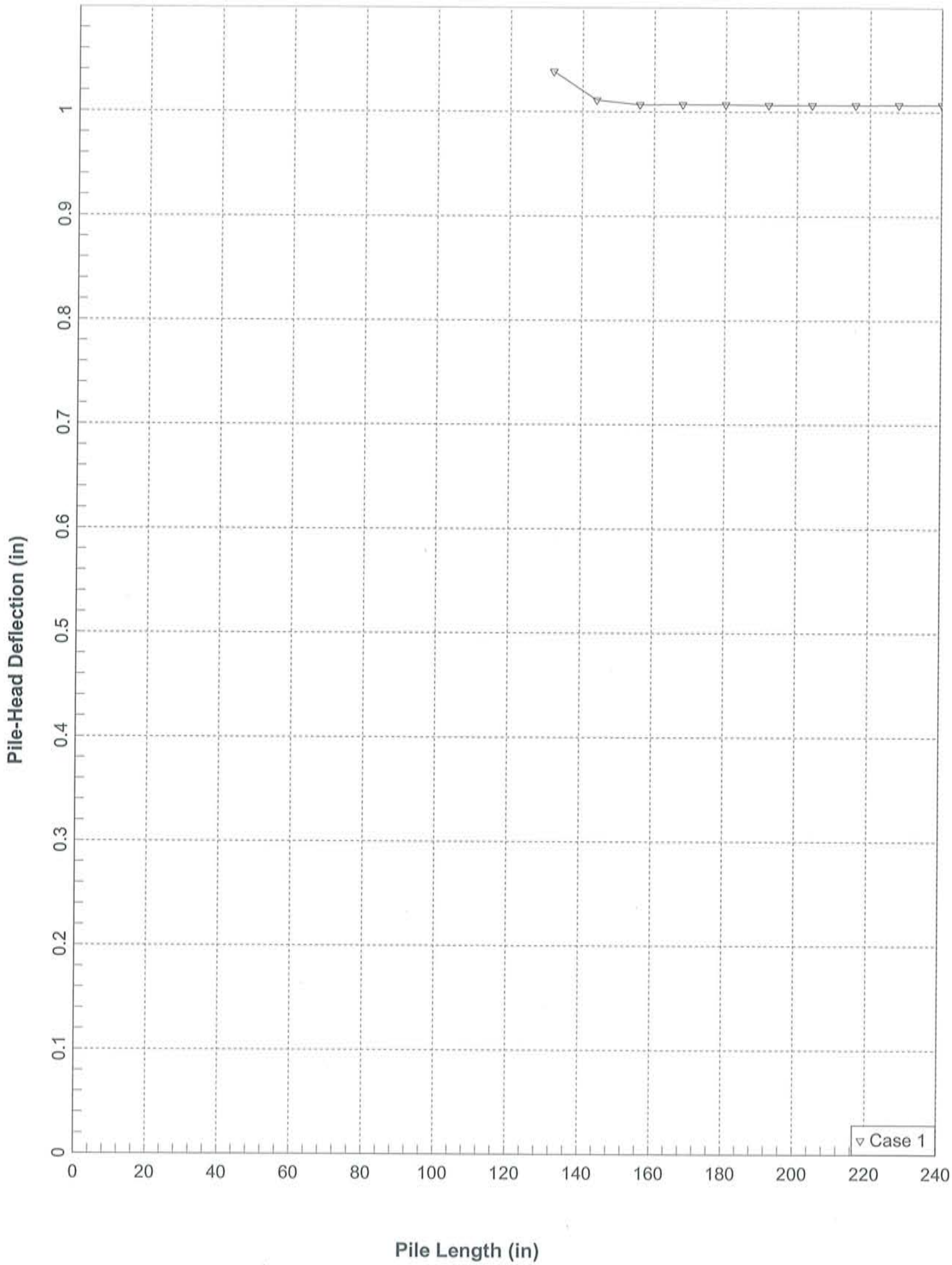


Abutment Alt X Pile, Transverse Direction, Nominal Case

Bear Creek Bridge
BCI No. 879.5
June 28, 2010
By: WEN

Pile Settlement Calculations: Class 90 (Alt. X) Piles
(Foundation Analysis and Design, Bowles, 5th edition, 1996)

Axial Pile Compression

	Allowable Pile Capacity (lbs)	90000
A	*Average Axial Load (lbs)	49500
B	Pile Length (in.)	456
C	Tip Area (sq. in.)	144
D	Concrete Modulus of Elasticity (psi)	4760000
	Axial Compression (in.)	**0.03**

*Allowable Capacity Reduced by 45% Due to Skin Friction

Axial Compression = (A x B)/(C x D)

Point Settlement

A	Point Bearing Pressure (psi)	625.0
B	Pile Diameter (in.)	12
C	Poisson's Ratio	0.35
D	Point Soil Stress-Strain Modulus (psi)	2900
E	Shape Factor	1
F	Fox Embeddment Factor	0.5
G	Reduction Factor for Skin Friction	0.5
	Point Settlement (in.)	**0.57**

A = Allowable Pile Capacity x Tip Area

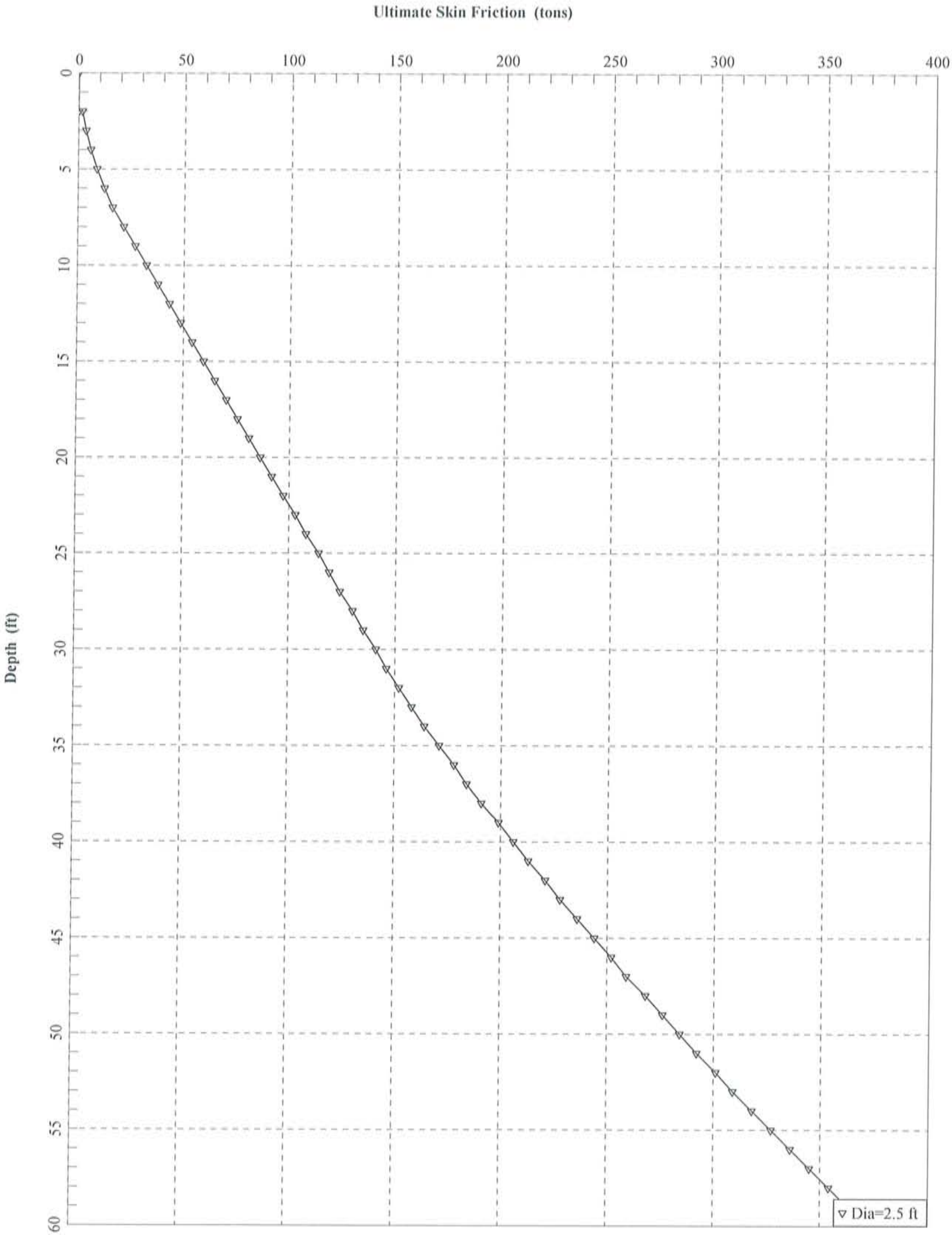
F = 0.55 if L/D </= to 5, 0.5 if greater than 5

Point Settlement = A x {B x (1-C^2)/D} x E x F x G

Total Pile Settlement = **0.60** in.
15.2 mm

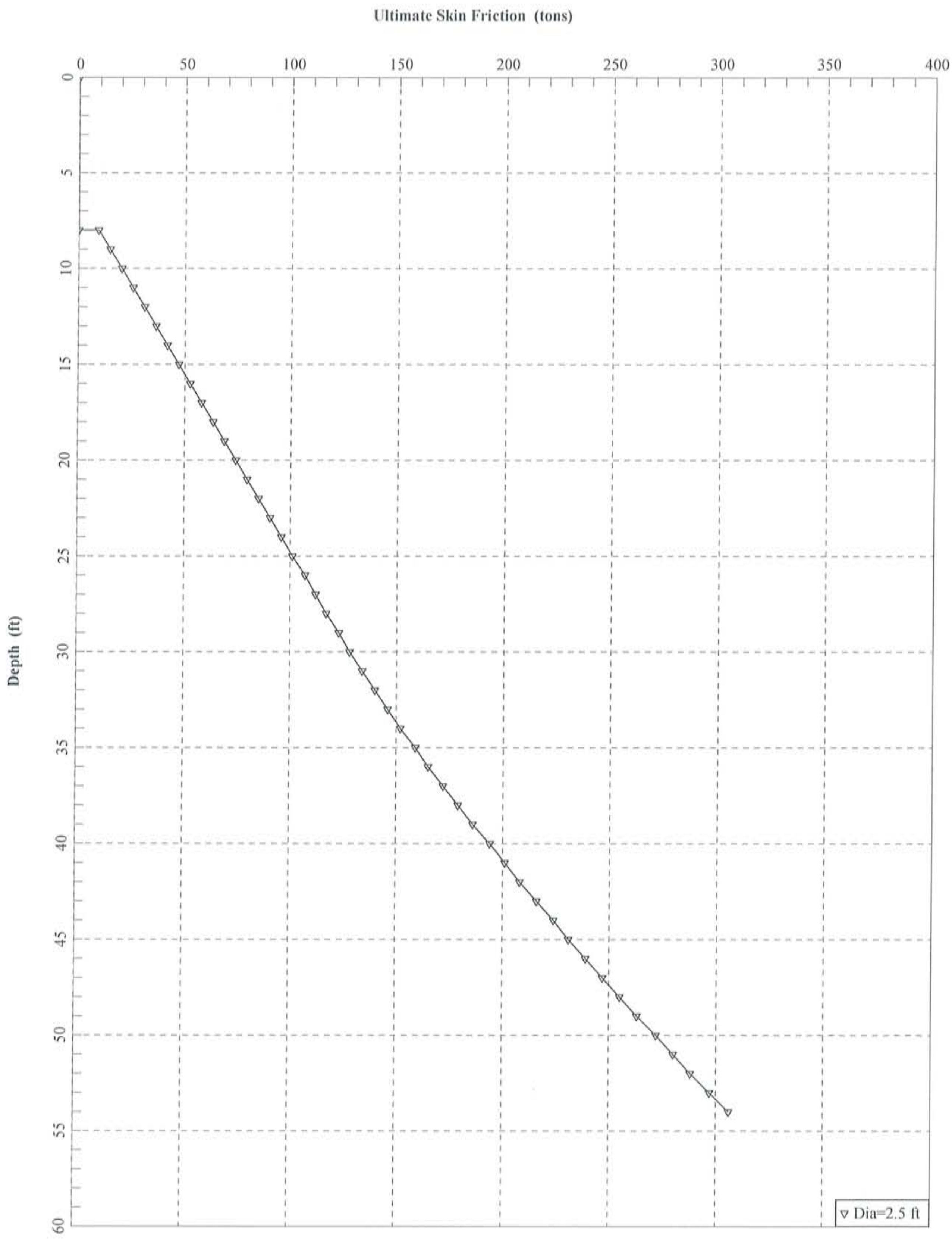
APPENDIX D

Piers 2 & 3: 30-inch CIDH Pile Analysis



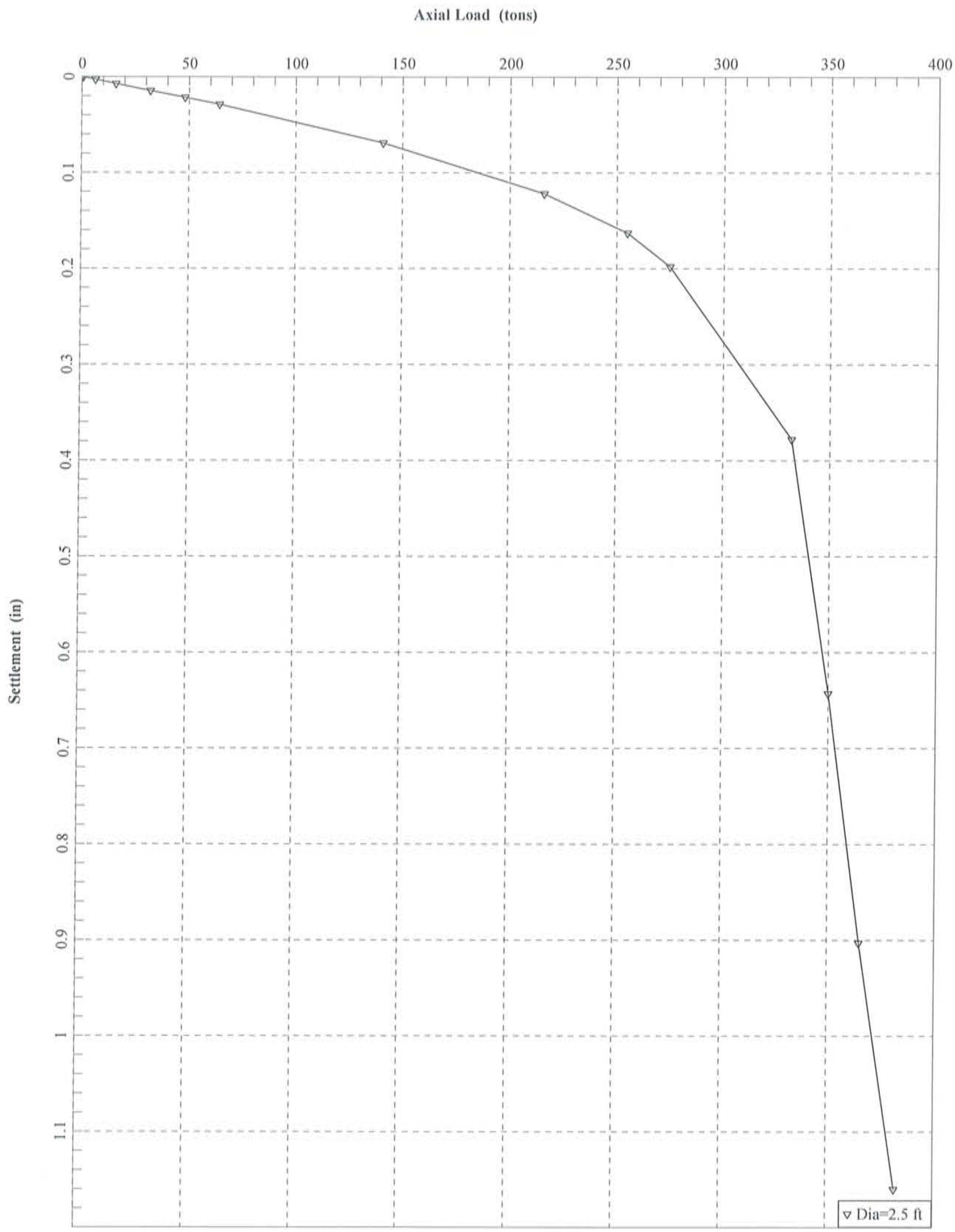
30-inch CIDH Piles, Piers 2 & 3 (No Scour)

ATTACHMENT G



30-inch CIDH Piles, Piers 2 & 3 (Scour at Elev. -1.0)

ATTACHMENT G



30-inch CIDH Piles, Piers 2 & 3 (Scour at Elev. -1.0)