FINAL FOUNDATION REPORT

Thomes Creek Bridge (Replace) at 99W
Bridge No. 08C-0377
Tehama County, CA

Prepared by:

BLACKBURN CONSULTING
2491 Boatman Avenue
West Sacramento, CA 95691
(916) 375-8706

March 2014

Prepared for:

HDR, Inc.
2365 Iron Point Road, Suite 300
Folsom, CA 95630

West Sacramento Office:
2491 Boatman Ave. ▪ West Sacramento, CA 95691
(916) 375-8706 ▪ Fax (916) 375-8709

blackburn consulting

Main Auburn Office: (530) 887-1494
Fresno Office: (559) 438-8411
Modesto Office: (209) 522-6273

Geotechnical ▪ Geo-Environmental ▪ Construction Services ▪ Forensics

File No. 1274.2
March 24, 2014

Mr. John Maniscalco
HDR, Inc.
2365 Iron Point Road, Suite 300
Folsom, CA 95630

Subject: **FINAL FOUNDATION REPORT**
Thomes Creek Bridge (Replace) at 99W
Bridge No. 08C-0377
Tehama County, California

Dear Mr. Maniscalco,

In accordance with our October 22, 2008 Agreement, Blackburn Consulting (BCI) prepared this Final Foundation Report for the proposed Thomes Creek Bridge (Replace) Project at State Route 99W in Tehama County, California. This report contains the findings of our subsurface investigation, conclusions, and recommendations for foundation design and roadway approaches for the new bridge.

Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. GE 2843
GEOTECHNICAL
STATE OF CALIFORNIA

David J. Morrell, C.E., G.E.
Senior Project Manager

Patrick F. Fischer, C.E.G, C.E.
Principal

Copies: 5 to Addressee

FINAL FOUNDATION REPORT
Thomes Creek Bridge (Replace) at 99W
Bridge No. 08C-0377
Tehama County, California

TABLE OF CONTENTS

1 INTRODUCTION 1
1.1 Purpose 1
1.2 Scope of Services 1
2 SITE LOCATION AND PROJECT DESCRIPTION 1
2.1 Site Description 1
2.2 Project Description 2
3 SUBSURFACE INVESTIGATION 2
4 LABORATORY TESTING 2
5 GEOLOGY AND SUBSURFACE CONDITIONS 3
5.1 Geologic Setting 3
5.2 Subsurface Soil Conditions 3
5.3 Groundwater 3
6 SEISMIC RECOMMENDATIONS 4
6.1 Ground Motion Study 4
6.2 Fault Rupture 4
6.3 Liquefaction Evaluation 5
6.4 Seismic Settlement 5
7 CORROSION EVALUATION 5
8 SCOUR ASSESSMENT 6
9 AS-BUILT FOUNDATION DATA 6
10 FOUNDATION RECOMMENDATIONS 7
10.1 Foundation Data and Loading 7
10.2 Pile Data Table 8
10.3 Abutment Pile (HP14x89) Analysis 9
10.3.1 Compressive Resistance 9
10.3.2 Settlement 10
10.3.3 Lateral Load Analysis 10
10.3.4 Negative Skin Friction 11
10.4 Bent Pile (HP14x89) Analysis 11
10.4.1 Compressive and Tension Resistance 11
10.4.2 Settlement 11
10.4.3 Lateral Load Analysis 11
10.4.4 Negative Skin Friction 12
11 LATERAL EARTH PRESSURES 12

FINAL FOUNDATION REPORT
Thomes Creek Bridge (Replace) at 99W
Bridge No. 08C-0377
Tehama County, California

12 APPROACH FILL EARTHWORK 13
12.1 Fill Material 13
12.2 Slope Geometry & Stability 14
12.3 Settlement 14
13 PAVEMENT DESIGN 14
14 CONSTRUCTION CONSIDERATIONS 15
14.1 HP14x89 Piles 15
14.2 Excavation Dewatering 15
14.3 Temporary Excavation and Shoring 15
15 RISK MANAGEMENT 16
16 LIMITATIONS 16

APPENDIX A:
Figure 1 – Vicinity Map
Figure 2 – Geologic Map
Log of Test Borings

APPENDIX B:
Laboratory Test Results

APPENDIX C:
Design Response Spectrum and Tabular Data

APPENDIX D:
Driven 1.2 Input Files and Compressive Resistance Graphs
LPILE Output Graphs

1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (BCI) prepared this Final Foundation Report for the proposed Thomes Creek Bridge (Replace) Project at State Route 99W located about 3.5 miles north of the City of Corning in Tehama County, California. This report contains our findings and recommendations for design of new bridge foundations and roadway approaches.

This report is for HDR and the County of Tehama to use during design and construction of the bridge. Do not use or rely upon this report for different locations or improvements without the written consent of BCI.

1.2 Scope of Services

To prepare this report, BCI:

- Reviewed 60% submittal plans for the proposed structure, provided by HDR.
- Discussed the project with Mr. John Maniscalco and Mr. Titus Keng of HDR.
- Reviewed geologic and seismic maps pertaining to the site.
- Reviewed the June 2013 Hydraulic Design Study for the bridge prepared and provided by HDR.
- Reviewed February 5, 1951 As Built Log of Test Borings for the existing bridge.
- Reviewed June 1, 2007 Log of Test Borings and Pile Data Plans for the Thomes Creek Bridge (Replace) at Interstate 5, located about 1.3 miles west of the site.
- Prepared a February 27, 2009 Preliminary Foundation Report for the project.
- Observed, logged and sampled four (4) exploratory borings to depths of 101.5 to 140.2 feet along the proposed bridge replacement alignment.
- Performed laboratory tests on soil samples obtained during the subsurface investigation.
- Performed engineering and seismic analysis to provide recommendations for new bridge foundations, roadway approaches, and pavement design.

2 SITE LOCATION AND PROJECT DESCRIPTION

2.1 Site Description

The project is located on State Route 99W (99W) at Thomes Creek, about 3.5 miles north of the City of Corning in Tehama County, California. Site coordinates are approximately latitude 39.9796ºW and longitude 122.1767ºN. We show the general site location on Figure 1 in Appendix A.

The site is in a topographically flat area of the northern Sacramento Valley. Natural ground elevation outside the creek channel near the bridge is approximately 265 ft. Thomes Creek flows easterly and is a tributary to the Sacramento River, located about 4 miles east of the site. The present channel is approximately 600 ft wide with bottom at about elev. 245 ft. A railroad

bridge, no longer in service, is located parallel to the project site, about 50 ft. downstream (east) from the existing 99W bridge.

The 99W bridge is a nine-span, 625 ft long and 31 ft. wide, reinforced concrete arch bridge. It was originally constructed in 1920 and widened in 1950. The site has been subject to significant scour, degradation and bedload migration, discussed further in the As-Built Foundation Data section of this report.

2.2 Project Description

The project will replace the existing bridge with a new bridge on a similar alignment. The new bridge will be a five span cast-in-place, prestressed concrete box girder structure, approximately 609 ft long and 43.5 ft wide. Existing approaches will be widened and raised several feet, with side slopes oriented at 2:1 (horizontal to vertical). Rock slope protection (RSP) will be used to mitigate potential scour at the abutments. Scour protection will not be used at the piers.

3 SUBSURFACE INVESTIGATION

BCI retained Diamond Core Drilling to drill and sample 4 exploratory borings along the proposed bridge alignment from October 5-13, 2009 to characterize the subsurface conditions and obtain soil samples for laboratory testing. The drillers used a B-59 mobile truck-mounted drill rig and both 8-inch O.D. hollow stem auger and 3.5-inch O.D. rotary wash drilling methods.

BCI obtained relatively undisturbed samples using both Modified California Samplers (equipped with 2.4-inch I.D. brass liners) and Standard Penetration Test samplers (1.5-inch I.D.). Samplers were driven into the ground with a 140 pound cable-pulley trip hammer falling 30 inches. Samples were sealed in relatively air tight containers and delivered to our laboratory for testing.

BCI logged the borings consistent with the Unified Soil Classification System (USCS). The Log of Test Borings (LOTB) in Appendix A shows the boring locations and logged soil conditions.

4 LABORATORY TESTING

We completed the following laboratory tests on representative soil samples obtained from the exploratory borings:

- Moisture content and dry density (ASTM D2216 / D2937)
- Sieve Analysis (ASTM D422)
- Plasticity Index (ASTM D4318)
- Triaxial Compression – Unconsolidated, Undrained (ASTM D2850)
- Unconfined compression (ASTM D2166)
- Sulfate content (CTM 417), chloride content (CTM 422), pH (CTM 643) and resistivity testing (CTM 643)

Appendix B contains the laboratory test results.

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Geologic Setting

The site is located in the northern Sacramento Valley that is bounded by the Sierra Nevada on the east and the Coast Ranges on the west. The Sacramento Valley is a structural trough that represents the northern third of the Great Valley Geomorphic Province. The relatively flat valley surface is underlain by alluvial, lacustrine, and marine sedimentary deposits that have accumulated as the structural trough formed and the adjacent mountain ranges were elevated. The thickness of the sediments varies from a thin veneer along the valley margins to thousands of feet at the axis of the trough.

Based on published geologic mapping[1], the Thomes Creek channel contains Holocene age alluvium that consists of unconsolidated sand, silt and gravel. The alluvium is underlain by Pleistocene sediments of the Modesto Formation, comprised of interbedded clay, silts, sands and gravels. These sediments are underlain by older Pleistocene and Pliocene-age sediments of the Riverbank, Red Bluff and Tehama formations, including intensely weathered sandstone and siltstone with lenses of cross-bedded pebble and cobble conglomerate. Figure 2 shows the site geology.

5.2 Subsurface Soil Conditions

BCI observed loose to medium dense silty sand and well-graded gravel in the borings from the existing ground surface to approximate elev. 245.0 ft. to 240.0 ft., which we interpret as recent stream channel deposits. Beneath these deposits, we generally observed stiff to hard clay to about elev. 150.0 ft., which was interbedded with occasional dense to very dense layers of clayey sand, silty sand, poorly graded sand with silt, clayey gravel and very stiff to hard silt. Between elev. 150.0 ft. and 106.1 ft. (deepest boring), we generally observed dense to very dense poorly graded sand with gravel, clayey sand with gravel, and silty sand. These deposits were interbedded with occasional very stiff to hard silt and clay layers.

Refer to the Log of Test Borings sheets in Appendix A for more detailed soil descriptions, exploration details and sampling methods. We also include the February 5, 1951 As Built Log of Test Borings in Appendix A.

5.3 Groundwater

We encountered groundwater in all of our borings during drilling in October 2009. Groundwater elevations ranged from 227.5 ft. to 234.3 ft. near the existing abutments, and from Elev. 240.0 ft. to 240.8 ft. within the channel area. We anticipate significantly higher groundwater elevations will be encountered from late Fall through Spring in response to increased precipitation and associated rising creek water levels.

[1] Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley, California Department of Water Resources, Mulder, J., 2007

BCI reviewed groundwater level data made available by the California Department of Water Resources webpage. The closest well data indicates that the regional groundwater table is at a depth of about 20-40 f.t below ground surface, generally between Elev. 230-240 ft.

BCI used a design groundwater elevation of 240.0 ft. in our pile analysis.

6 SEISMIC RECOMMENDATIONS

6.1 Ground Motion Study

BCI used the Caltrans ARS Online Version 2.2.06 (web-based tool) to evaluate the minimum design acceleration response spectrum (Design Response Spectrum) for design of the bridge. Caltrans Seismic Design Criteria Version 1.7 (April 2013) defines the Design Response Spectrum as the maximum design envelope across the period spectrum developed by considering both deterministic and probabilistic spectra.

Table 1 provides a summary of our ground motion study results.

Table 1: Ground Motion Study Results

Peak Ground Acceleration (PGA)	0.27g
*V_{S30} (Small Strain Shear Wave Velocity)	250 meters per second
Near Fault Factor	N/A
Basin Amplification Factor	N/A
Controlling Deterministic Scenario	Minimum Deterministic
Nearest Late Quaternary Fault	Great Valley 01 (Fault ID: 64) • Style: Reverse • Maximum Moment Magnitude (Mmax) = 6.7 Site to fault distance (R_{RUP}) = 22.2 miles
DESIGN RESPONSE SPECTRUM	Recommended Design Response Spectrum is the Probabilistic Spectrum (probability of exceedance equal to 5% in 50 years, a 975-year return period) which controls for this site over all periods.

*Estimated using site boring/laboratory data and equations outlined in Appendix A of Caltrans "Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations", November 2012.

Appendix C contains a graphical display of our recommended Design Response Spectrum along with the tabular data used to create the spectrum.

6.2 Fault Rupture

The site does not lie within or adjacent to an Alquist–Priolo Earthquake Fault Zone for fault rupture hazard (Bryant and Hart, 2007)[2], and no known active faults are mapped within or through the project area based on our ARS Online evaluation which displays an active fault map

[2] Fault Rupture Hazard Zones in California, Special Publication 42, Interim Revision; California Geological Survey

using data from the 2012 Caltrans Fault Database. We consider the potential for fault rupture at the site to be very low to nonexistent.

6.3 Liquefaction Evaluation

Liquefaction can occur when saturated, loose to medium dense, granular soils (generally within 50 feet of the surface), or specifically defined cohesive soils, are subjected to ground shaking. We consider the potential for detrimental liquefaction at the site to be very low to nonexistent based on the soil and groundwater conditions encountered in our borings and the relatively low design peak ground acceleration.

6.4 Seismic Settlement

During a seismic event, ground shaking can cause densification of granular soil above the water table that can result in settlement of the ground surface. We consider the potential for detrimental seismic settlement at the site to be very low to nonexistent based on the soil and ground water conditions encountered in our borings and the relatively low design peak ground acceleration.

7 CORROSION EVALUATION

Table 2 presents our soil corrosivity test results.

Table 2: Soil Corrosion Test Summary

Boring	Sample No.	Depth (ft)	Minimum Resistivity (ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
R-09-B2	7	40-41.5	1,770	7.97	31.2	3.5
R-09-B2	10	55-56.5	2,360	7.97	22.0	3.1
R-09-B2	13	70-71.5	1,630	7.42	26.0	5.2
A-09-B3	12	60-61.5	1,370	7.49	15.1	2.9

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” to structural elements according Caltrans Corrosion Guidelines (Version 2.0, November 2012).

8 SCOUR ASSESSMENT

HDR prepared a June 21, 2013 "Thomes Creek Hydraulic Design Study" for the project which included estimated maximum scour depths at the abutments and piers. Table 3 summarizes HDR's estimated maximum scour depths:

Table 3: Scour Depths

Support	Long-Term Degradation/Contraction Depth (ft.)	Local Scour Depth (ft.)	Total Scour Depth (ft.)
Abutment 1	0	19.22	19.22
Piers 2-5	20.0	14.82	34.82
Abutment 6	0	20.37	20.37

HDR is designing Rock Slope Protection to mitigate potential scour at the abutments so that they can be designed for the no scour case. Scour protection will not be provided for the piers.

9 AS-BUILT FOUNDATION DATA

The 1920 bridge plans show the original abutments and pier walls supported on timber piles with a 5 ft. thick concrete pile cap. The base of the cap is shown at Elev. 245 ft., about 6-9 ft. below the 1920 channel bottom. A 1951 Bridge Report quotes an August 26, 1921 report indicating that the piles are 8-14 ft. long with a few driven to "refusal". The 1921 report describes some scour observed around Pier 4 the first year after construction.

In 1950 the bridge was widened on the upstream (west) side. The widened section, per discussions with Tehama County DPW, is supported on H-piles driven about 11 ft below the pile caps. The February 5, 1951 As Built Log of Test Borings drawing shows a channel bottom between about Elev. 250 and 255 ft. (approximately the same as indicated in 1920). Several inspection reports in the 1950's describe channel shaping, backfilling of potholes around piers and rip-rap to control scour/erosion, although a January 1960 report states that "no scour is evident at any of the piers".

Channel degradation and scour increased after 1960, and a 1983 inspection report states that all piers, except Pier 2, are undermined, and that the piling is exposed at Piers 3 and 4. Other inspection reports from 1960 through 1983 describe drift on the piers.

An inspection report dated May 23, 2002, includes channel cross-sections that show the 2002 channel generally below Elev. 245 ft., and locally as deep as Elev. 234 ft. The base of the abutment footings and pile caps is shown between about Elev. 244 and 246 ft. With the exception of Abutment 1 and Pier 2 (south end), the 2002 channel profile is below the base of all the pile caps and, in the area of Piers 6 and 7, at or below the pile tips. Sacked concrete protection is described in the 2002 inspection report at Abutments 1 and 10, noting channel migration based on a comparison of previous channel cross sections. The 2002 report indicates

that, based on a hydraulic review, the anticipated scour will "go below the Pile Tip Elevation for all the bridge piers". The bridge was therefore categorized by Caltrans as scour critical.

Based on our discussions with Jim Brinkley, former county bridge engineer (now retired), the County Department of Public Works performed a variety of measures since about 1970 to mitigate the scour conditions. These included concrete cutoff walls around the pile caps to protect the piles; grouted rock slope protection around the piers; grouting inside the cutoff walls; and a rip-rap "wall" between the roadway and railroad bridge. Mr. Brinkley estimates that about 15-18 ft. of total channel degradation has occurred at the bridge site.

10 FOUNDATION RECOMMENDATIONS

10.1 Foundation Data and Loading

The subsurface conditions encountered in our borings indicate that the site is conducive for either driven piles, Cast-In-Drilled-Hole (CIDH) piles or Cast-In-Steel-Shell (CISS) piles. Spread footings are not considered feasible for support due to the potential for scour and/or excessive settlement.

CIDH piles would require temporary casing and slurry drilling due to the potential for encountering caving or squeezing soil. We understand that during recent construction of large diameter CIDH piles for the upstream Thomes Creek Bridge Replacement at Interstate 5, pile defects were detected in several piles during gamma-gamma logging, resulting in delays and significant additional foundation installation costs. Therefore, we favor the use of driven piles or CISS piles over CIDH piles.

Driven steel HP piles or open-ended pipe piles are preferred over driven concrete piling in order to penetrate through variably thick layers of dense to very dense sand to reach specified tip elevations. Standard open-ended pipe piles (16 inches or less in outside diameter) could plug during driving and experience driving refusal in dense to very dense sand layers prior to reaching specified tip elevations. In this case, center relief drilling would be required to achieve specified tip elevations.

At HDR's request, BCI performed preliminary analysis for CIDH pile, CISS pile and HP pile alternatives for support of the piers for their cost benefit analysis. Based on the cost benefit analysis, HP14x89 piles were selected at all bridge supports.

At the piers, HDR is designing the bottom of the pile cap elevations at about 20 feet below existing grade (below long-term scour level), and designing the piles for 15 feet of unsupported length to account for local scour occurring after the full long-term scour occurs.

HDR provided the following foundation design information in Tables 4 and 5.

Table 4: Foundation Design Data Provided By HDR

Foundation Design Data								
Support No.	Design Method	Pile Type	Finish Grade Elev. (ft)	Cut-Off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement – Service Load (in)	Number of Piles Per Support
					B	L		
Abut 1	WSD	HP14x89	262.0	254.17	7	45.5	1	12
Pier 2	LRFD	HP14x89	255.0	235.42	19.67	16.33	1	29
Pier 3	LRFD	HP14x89	245.0	225.42	19.67	16.33	1	29
Pier 4	LRFD	HP14x89	242.0	222.42	19.67	16.33	1	29
Pier 5	LRFD	HP14x89	245.5	225.92	19.67	16.33	1	29
Abut 6	WSD	HP14x89	258.0	247.92	10	45.5	1	17

Table 5: Foundation Design Loads Provided By HDR

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	1505	175	1265	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Pier 2	2715	N/A	2540	4285	335	620	70	2805	370	1030	280
Pier 3	3240	N/A	2930	4560	365	670	75	3190	405	1015	280
Pier 4	3280	N/A	2900	4640	365	720	80	3160	394	1015	280
Pier 5	3140	N/A	2775	4710	370	830	91	3100	390	1020	280
Abut 6	2200	185	1950	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

10.2 Pile Data Table

Based on the above information, we provide recommended pile tip elevations in Table 6. We describe our engineering analysis in the following section.

Table 6: Pile Data Table

Location	Pile Type	Nominal Resistance		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression (kips)	Tension (kips)			
Abut 1	HP14x89	350	0	191.0 (1) 224.0 (2)	191.0	350
Pier 2	HP14x89	480	280	145.0 (1) 188.0 (2) 157.0 (3)	145.0	550
Pier 3	HP14x89	530	280	137.0 (1) 178.0 (2) 148.0 (3)	137.0	600
Pier 4	HP14x89	530	280	134.0 (1) 175.0 (2) 146.0 (3)	134.0	600
Pier 5	HP14x89	530	280	136.0 (1) 179.0 (2) 148.0 (3)	136.0	600
Abut 6	HP14x89	370	0	187.0 (1) 217.0 (2)	187.0	370

- Design tip elevations at Abutments are controlled by: (1) Compression, (2) Lateral
- Design tip elevations at Piers are controlled by: (1) Compression (Scour Case), (2) Lateral (Scour Case), (3) Tension (Scour Case)
- Do not raise specified tip elevations without BCI review and approval.
- The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the unsuitable penetrated soil layers (scourable soil), if any, which do not contribute to the required design resistance.

10.3 Abutment Pile (HP14x89) Analysis

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 load analysis below. No tension demand is indicated for abutment piles.

10.3.1 *Compressive Resistance*

Our calculations indicate that the nominal compressive resistance of the piles can be obtained through about 10% end bearing and 90% skin friction. Actual contributions to end bearing and skin friction could vary depending on how the load is transferred to the piles. 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.

Refer to the Driven input files and compressive resistance graphs in Appendix D for the analysis results.

10.3.2 *Settlement*

We expect that settlement will be nominal (less than the permissible 1-inch settlement specified for the Service-I Limit State total load per pile) since pile compressive resistance is primarily derived by skin friction with competent soil along the pile length. We do not anticipate significant long-term settlement at this site.

10.3.3 *Lateral Load Analysis*

We used LPILE Plus Version 5.0 software to evaluate lateral pile capacity. BCI determined the allowable lateral pile design loads which would produce approximately ¼-inch and 1-inch top-of-pile deflection assuming a pinned head condition. For the ¼-inch deflection case, BCI applied the Service-I Limit State maximum per pile axial load in the analysis. For the 1-inch deflection case, BCI applied the pile axial nominal resistance in the analysis.

HDR is designing the pile orientation so that the H-pile strong axis will resist lateral loading in the longitudinal bridge direction, with the weak axis resisting lateral loading in the transverse bridge direction. Table 7 presents the lateral pile capacity results.

Table 7: Abutment Lateral Pile Capacity

Support	Lateral Load Direction	Top-of-Pile Deflection (inches)	Lateral Resistance (kips)
Abutment 1	Longitudinal (Pile Strong Axis)	¼	19.5
		1	50.0
	Transverse (Pile Weak Axis)	¼	14.0
		1	34.0
Abutment 6	Longitudinal (Pile Strong Axis)	¼	16.5
		1	41.5
	Transverse (Pile Weak Axis)	¼	14.0
		1	34.0

To account for group effects at Abutment 1, BCI used reduced p-multipliers of 0.79 and 0.85 for lateral loads in the longitudinal and transverse bridge directions, respectively. For Abutment 6, BCI used reduced p-multipliers of 0.57 and 0.85 for lateral loads in the longitudinal and transverse bridge directions, respectively. These p-multipliers are based on the pile spacing and layout shown on HDR's 60% Submittal Plans for the project. If the pile layout and/or spacing change, BCI will need to perform additional LPILE analysis to evaluate lateral pile capacity.

BCI calculated design lateral tip elevations shown in the Pile Data Table using a factor of safety of 1.5.

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

10.3.4 *Negative Skin Friction*

We do not anticipate negative skin friction at the abutments given the competent soil conditions.

10.4 Bent Pile (HP14x89) Analysis

We used AASHTO LRFD Bridge Design Specifications-4th Edition and current Caltrans Amendments for evaluating driven HP14x89 piles for compressive resistance, tension resistance, settlement and lateral load analysis. HDR is responsible for evaluation of structural pile capacity, including buckling potential of the pier piles under the design 15-feet of unsupported length due to scour.

BCI presents the results of our analysis below.

10.4.1 *Compressive and Tension Resistance*

Our calculations indicate that the nominal compressive resistance of the piles can be obtained through about 10% end bearing and 90% skin friction. Actual contributions to end bearing and skin friction could vary depending on how the load is transferred to the pile.

We determined the required nominal compressive resistance using the Federal Highway Administration's Driven 1.2 (March 20, 2001) computer program developed by Blue-Six Software, Inc.

BCI determined the required factored nominal compressive and tension 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 compressive and tension resistance. In this case, the Factored Strength Limit Loads controlled for compression and the Extreme Event Loads controlled for tension.

Refer to the Driven input files and compressive resistance graphs in Appendix D for the analysis results.

10.4.2 *Settlement*

We expect that settlement will be nominal (less than the permissible 1-inch settlement specified for the Service-I Limit State total load per pile) since pile compressive resistance is primarily derived by skin friction with competent soil along the pile length. We do not anticipate significant long-term settlement at this site.

10.4.3 *Lateral Load Analysis*

We used LPILE Plus Version 5.0 software to evaluate lateral pile capacity for the vertical pier piles under the total scour case (15 feet of unsupported length). HDR indicated that LPILE analysis for the battered pier piles was not needed. BCI determined the allowable lateral pile design loads which would produce approximately ¼-inch and 1-inch top-of-pile deflection assuming a pinned head condition. For the ¼-inch deflection case, BCI applied the highest

Service-I Limit State maximum per pile axial load in the analysis. For the 1-inch deflection case, BCI applied the highest Extreme Limit State per pile axial load in the analysis.

HDR is designing the vertical pile orientation so that the H-pile strong axis will resist lateral loading in the longitudinal bridge direction, with the weak axis resisting lateral loading in the transverse bridge direction. Table 8 presents the lateral pile capacity analysis results.

Table 8: Pier Lateral Pile Capacity (Vertical Piles)

Support	Lateral Load Direction	Top-of-Pile Deflection (inches)	Lateral Resistance (kips)
Piers 2-5	Longitudinal (Pile Strong Axis)	¼	1.1
		1	2.8
	Transverse (Pile Weak Axis)	¼	0.4
		0.9*	0.07

* At higher deflections, L-pile software returned an error message indicating that plastic yielding of the pile may have occurred.

To account for group effects at the piers, BCI used reduced p-multipliers of 0.46 and 0.47 for lateral loads in the longitudinal and transverse bridge directions, respectively. These p-multipliers are based on the pile spacing and layout shown on HDR's 60% Submittal Plans for the project. If the pile layout and/or spacing change, BCI will need to perform additional LPILE analysis to evaluate lateral pile capacity.

BCI calculated design lateral tip elevations shown in the Pile Data Table using a factor of safety of 1.5.

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

10.4.4 *Negative Skin Friction*

We do not anticipate negative skin friction at the bents given the competent soil conditions.

11 LATERAL EARTH PRESSURES

The following equivalent fluid weights (EFWs) may be used to design the abutment walls and wing walls assuming level backfill conditions:

Table 9: Equivalent Fluid Weights

Condition	Static EFW (pcf)	Seismic EFW (pcf)
Active	36	8
At-Rest	56	13
Passive	220	200

For static design, apply the resultant of the static active earth pressure (36 lb/ft^3) at a depth of 0.33H from the base of the wall where H equals the wall height in ft.

For seismic design, calculate the resultant of incremental lateral soil pressure due to seismic loading based on an equivalent fluid pressure of 8 lb/ft^3 and apply the magnitude of the resultant at 0.5H from the base of the wall. Add the resultant of the seismic earth pressure to the resultant of the static active earth pressure.

The values shown above are consistent with Caltrans standards/practice and assume level backfill conditions using Caltrans "Structure Backfill" with a soil unit weight of 125 pcf, a minimum angle of internal friction of 34º, and that drainage behind walls is placed in accordance with Caltrans "Standard Plans and Specifications."

To limit wall deflection to acceptable levels, BCI applied a factor of safety of 2.0 to the ultimate passive pressure to generate the allowable passive pressures provided above.

BCI estimated the EFWs for seismic loading 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. In the Mononobe-Okabe equation, BCI used a horizontal seismic acceleration coefficient (k_h) of 0.14 calculated using the equation in Chapter 11, Section 11.6.5 of the AASHTO LRFD Bridge Design Specifications-4th Edition. This k_h value assumes that the walls displace at least 1-inch during the design seismic event. BCI calculated the above static EFWs using methods presented in the 1982 Naval Facilities (NAVFAC) Design Manual 7.2.

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.1.7 (April 2013). 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 0.48 to resist sliding for concrete placed on native undisturbed soil or compacted fill

12 APPROACH FILL EARTHWORK

12.1 Fill Material

Embankments will be constructed using imported borrow material, supplemented with material excavated from shallow on-site cuts. 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.

Fill placed within the upper 4 feet of finish pavement subgrade shall have a minimum R-value of 15.

Expansive soil (Expansion Index > 50 and Sand Equivalent < 20) shall not be used as fill within 5 ft. of abutment backwalls or wingwalls.

12.2 Slope Geometry & Stability

The existing roadway approaches will be widened but finish roadway grade will be raised only a few feet. New fill slopes will have a gradient of 2:1 (horizontal to vertical) or flatter. HDR's design includes rock slope protection at the abutments to mitigate scour potential.

Proposed 2:1 or flatter slopes will be stable based on the relatively stable condition of the existing slopes, provided the new slopes are constructed in accordance with current Caltrans Standard Specifications.

12.3 Settlement

We anticipate short-term settlement of less than 2 inches will occur beneath new embankment fills. Since the settlement will occur substantially during embankment construction, a settlement waiting period is not required from the end of embankment construction to beginning abutment pile installation.

13 PAVEMENT DESIGN

HDR requested flexible pavement section thickness recommendations for Traffic Indexes of 8, 10 and 12 using a subgrade design R-value of 15 (provided by Tehama County).

Using Caltrans Flexible Pavement Design Methods and a design R-value of 15, we recommend the hot mix asphalt (HMA) pavement sections shown in Table 10.

Table 10: Recommended HMA Pavement Sections (R-value = 15)

Traffic Index	Material Type/Thickness	
	*HMA-Type A	Aggregate Base (Class 2)
8.0	0.40 ft.	1.25 ft.
10.0	0.50 ft.	1.65 ft.
12.0	0.60 ft.	2.05 ft.

*HMA binder grade should be PG 64-10 based on "Inland Valley" site location per Caltrans Pavement Climate Regions Map (2005).

HMA and Class 2 Aggregate Base material quality and construction should conform to the 2010 Caltrans Standard Specifications.

Slope all areas adjacent to pavement at a gradient of 2% or greater to allow for positive surface drainage away from the pavement.

14 CONSTRUCTION CONSIDERATIONS

Where referenced below, Caltrans Standard Specifications (CSS) refer to the 2010 Caltrans Standard Specifications with latest revisions.

14.1 HP14x89 Piles

Difficult pile installation is anticipated due to the presence of dense to very dense sand and gravel layers above the specified tip elevations. The contractor shall provide a Driving System Submittal per Section 49-2.01A(3)(b) of the CSS to verify that the pile driving system is adequate.

Verify pile capacity during driving per CSS 49-2.01A(4)(b). A pile load test is not necessary.

Piles that do not attain the required nominal driving resistance at specified tip elevation during the end of initial driving (EOD) may be allowed to stand for a "setup period" before being restriked. The "setup period" must be at least 16 hours from EOD. At least 25 percent of the piles (rounded up) at each support that do not achieve the required nominal driving resistance at EOD must be restriked after the “setup period” using the same pile driving hammer to confirm that the nominal resistance has been achieved following pile setup. The pile penetration during restrike must be at least 2 inches and the piles with the lowest nominal driving resistance at EOD must be selected for restrike. If all of the restrike piles meet or exceed the required nominal driving resistance, then the remaining piles shall be considered to have achieved the required nominal resistance. If any of the restrike piles do not meet or exceed the required nominal driving resistance, contact BCI for additional recommendations which could include an extended “setup period”.

Do not raise specified tip elevations without the approval of BCI.

14.2 Excavation Dewatering

Abutment and pier excavations extending below the creek water level will require dewatering methods to construct foundations in the “dry.” The contractor is responsible for design and construction of dewatering systems.

14.3 Temporary Excavation and Shoring

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

15 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.
3. Update this report if:
   - design changes occur
   - 2 years or more lapse between this report and construction
   - 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 discussions.

16 LIMITATIONS

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/groundwater 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.

Use this report only for design and construction of the Thomes Creek Bridge (Replace) at 99W project, as described herein.

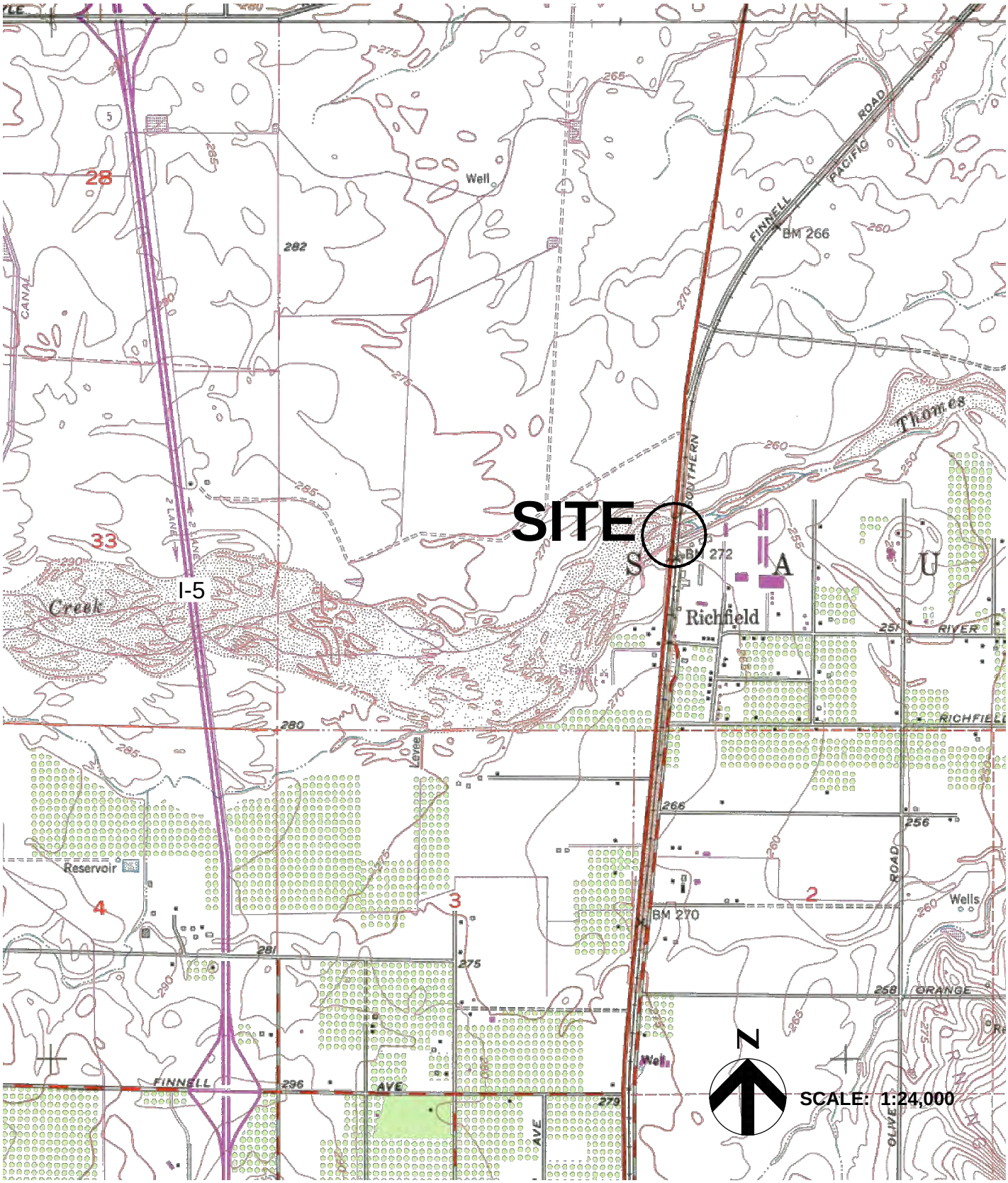
Modern design and construction is complex, with many regulatory sources, restrictions, involved parties, construction alternatives, etc. It is common to experience changes and delays. The owner should set aside a reasonable contingency fund based on complexities and cost estimates to cover changes and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

APPENDIX A

Figure 1: Vicinity Map

Figure 2: Site Plan

Log of Test Borings



Source: USGS 7.5 Minute Quadrangle, topographic, dated 1951, revised 1969.

blackburn consulting
2491 Boatman Avenue
West Sacramento, CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

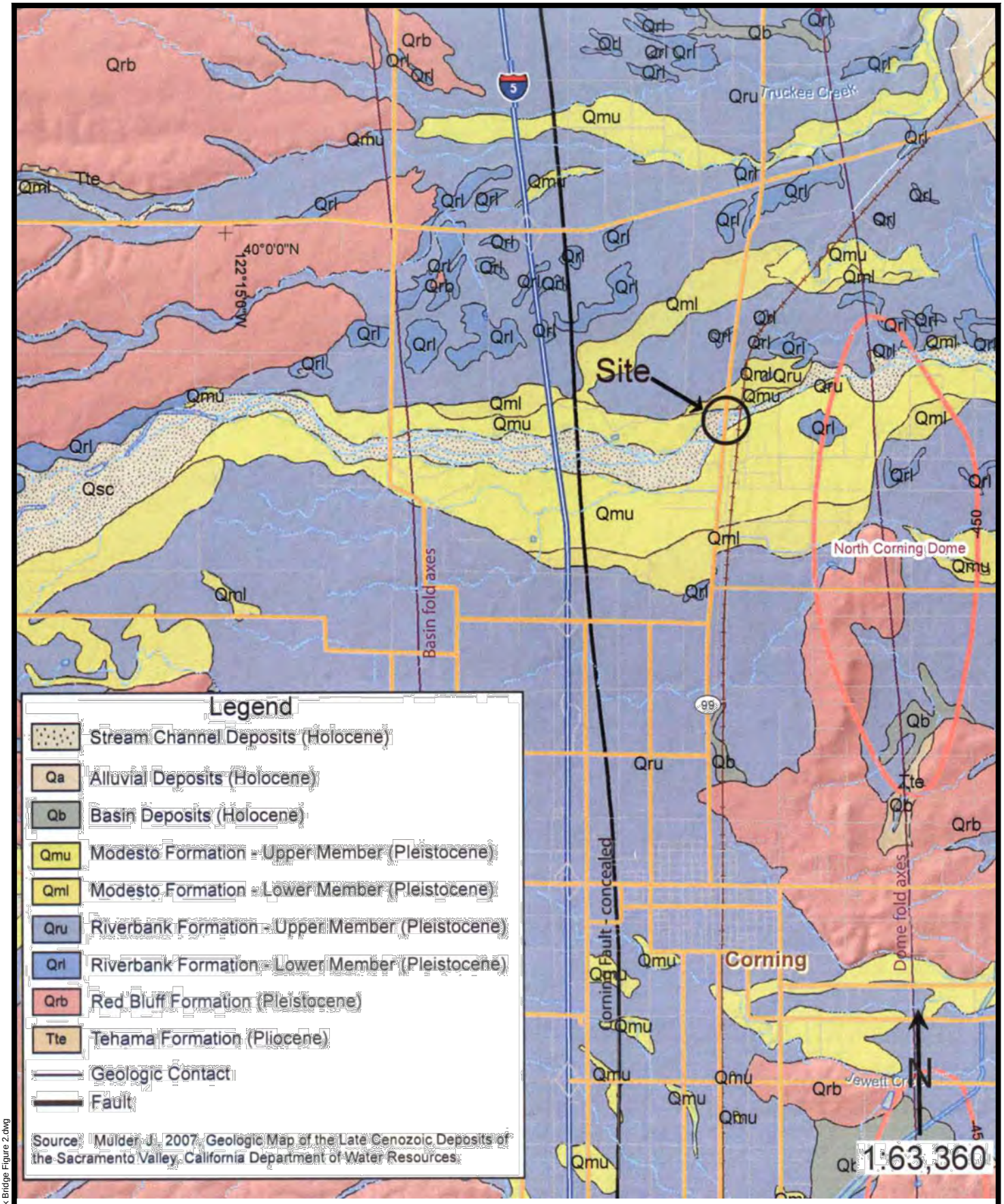
VICINITY MAP
99W at Thomes Creek
Project (Replace)
Tehama County, California

File No. 1274.1

March 2014

Figure 1

3/21/2014 1274.1 Thomes Creek Bridge Figure 1.dwg



blackburn consulting

2491 Boatman Avenue
West Sacramento, CA 95691
Phone: (916) 375-8706
Fax: (916) 375-8709
www.blackburnconsulting.com

REGIONAL GEOLOGIC MAP
99W at Thomes Creek Project (Replace)
Tehama County, California

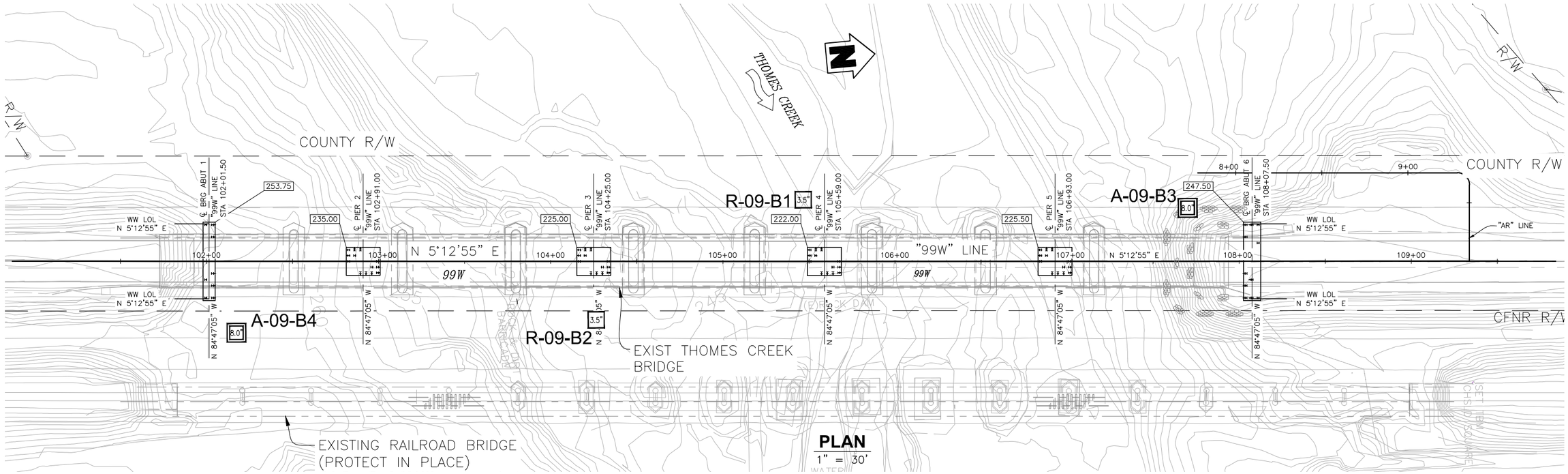
File No. 1274.1

March 2014

Figure 2

3/21/2014 1274.1 Thomes Creek Bridge Figure 2.dwg

BRIDGE NO.
08C-0110



PLAN
1" = 30'

LEGEND:

- INDICATES BOTTOM OF FOOTING ELEVATION.
- INDICATES NEW STRUCTURE.
- INDICATES EXISTING FEATURES.
- INDICATES BATTERED HP PILE.
- INDICATES VERTICAL HP PILE.

NOTES:
1. Field classification of soils was in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (June 2007). See Log of Test Borings 4 and 5 of 6, "Soil Legend".
2. Standard Penetration tests were performed in accordance with ASTM D 1586–99 using a safety hammer operated with an cable–pulley system. Drill rods were 1 5/8–inch diameter "A"–rods; sampler was driven with brass liners.
3. "2.4 inch sampler": ID=2.4 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. Groundwater surface elevations are subject to seasonal fluctuations and may occur at higher or lower elevations depending on the conditions at any particular time.
7. Boring elevations are estimated based on topography provided by Tehama County Department of Public Works.
8. Electronic media for plan view provided by HDR Engineering, Inc., General Plan, March 2014.
9. The "Log of Test Borings" drawing is included with plans in accordance with Section 2–1.03 of Caltrans "Standard Specifications".

95% SUBMITTAL

blackburn consulting
Blackburn Consulting
2491 Boatman Avenue, West Sacramento, CA 95691
File No. 1274.1

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. 60578
Exp. 12-31-14
CIVIL
STATE OF CALIFORNIA

No.	REVISION:	BY:	DATE:

These drawings are instruments of service provided by the Tehama County Public Works Dept. and are not to be used for any type of construction unless signed and sealed by a registered Civil Engineer in the employ of the County of Tehama

TEHAMA COUNTY
PUBLIC WORKS
9380 San Benito Ave.
Gerber, Ca. 96035
(530) 385-1462

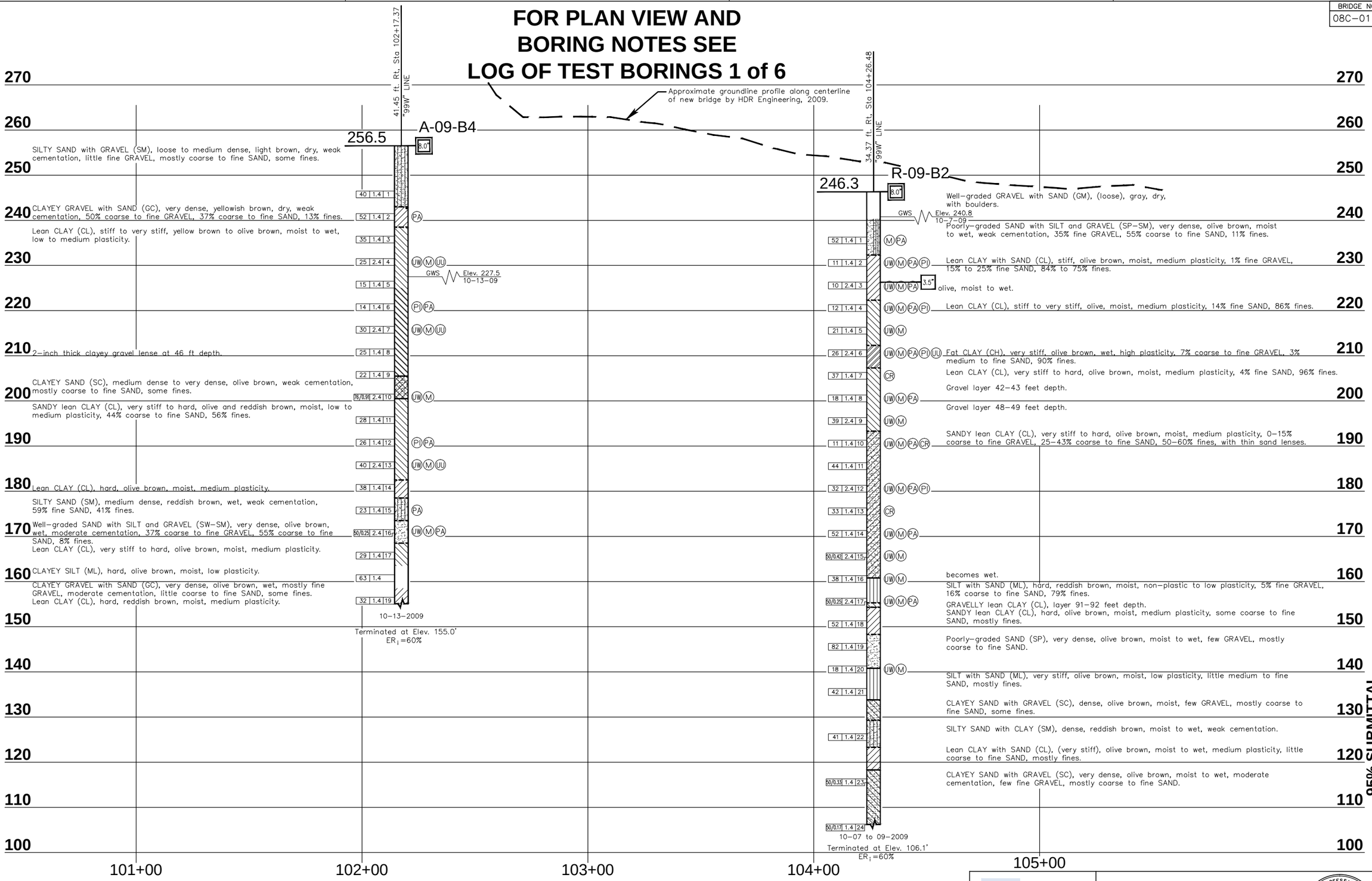
SCALE	HORIZ: AS SHOWN
	VERT: AS SHOWN
DRAWN BY:	M.D.R.
DATE:	11/2009
CHECKED BY:	D.J.M.
DATE:	05/2013

99W at THOMES CREEK BRIDGE
PROJECT (REPLACE)
LOG OF TEST BORINGS 1 OF 6

PROJECT NO.: 370861

SHEET NO.	PAGE NO.
S-23	

FOR PLAN VIEW AND BORING NOTES SEE LOG OF TEST BORINGS 1 of 6



PROFILE

95% SUBMITTAL

blackburn consulting

Blackburn Consulting
2491 Boatman Avenue, West Sacramento, CA 95691
File No. 1274.1

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. 60578
Exp. 12-31-14
CIVIL
STATE OF CALIFORNIA

BRIDGE NO. 08C-0110

No.	REVISION:	BY:	DATE:

These drawings are instruments of service provided by the Tehama County Public Works Dept. and are not to be used for any type of construction unless signed and sealed by a Registered Engineer in the employ of the County of Tehama

TEHAMA COUNTY PUBLIC WORKS
9380 San Benito Ave.
Gerber, Ca. 96035
(530) 385-1462

COUNTY OF TEHAMA

HORIZ:	1" = 20'
VERT:	1" = 10'
DRAWN BY:	M.D.R.
DATE:	11/2009
CHECKED BY:	D.J.M.
DATE:	05/2013

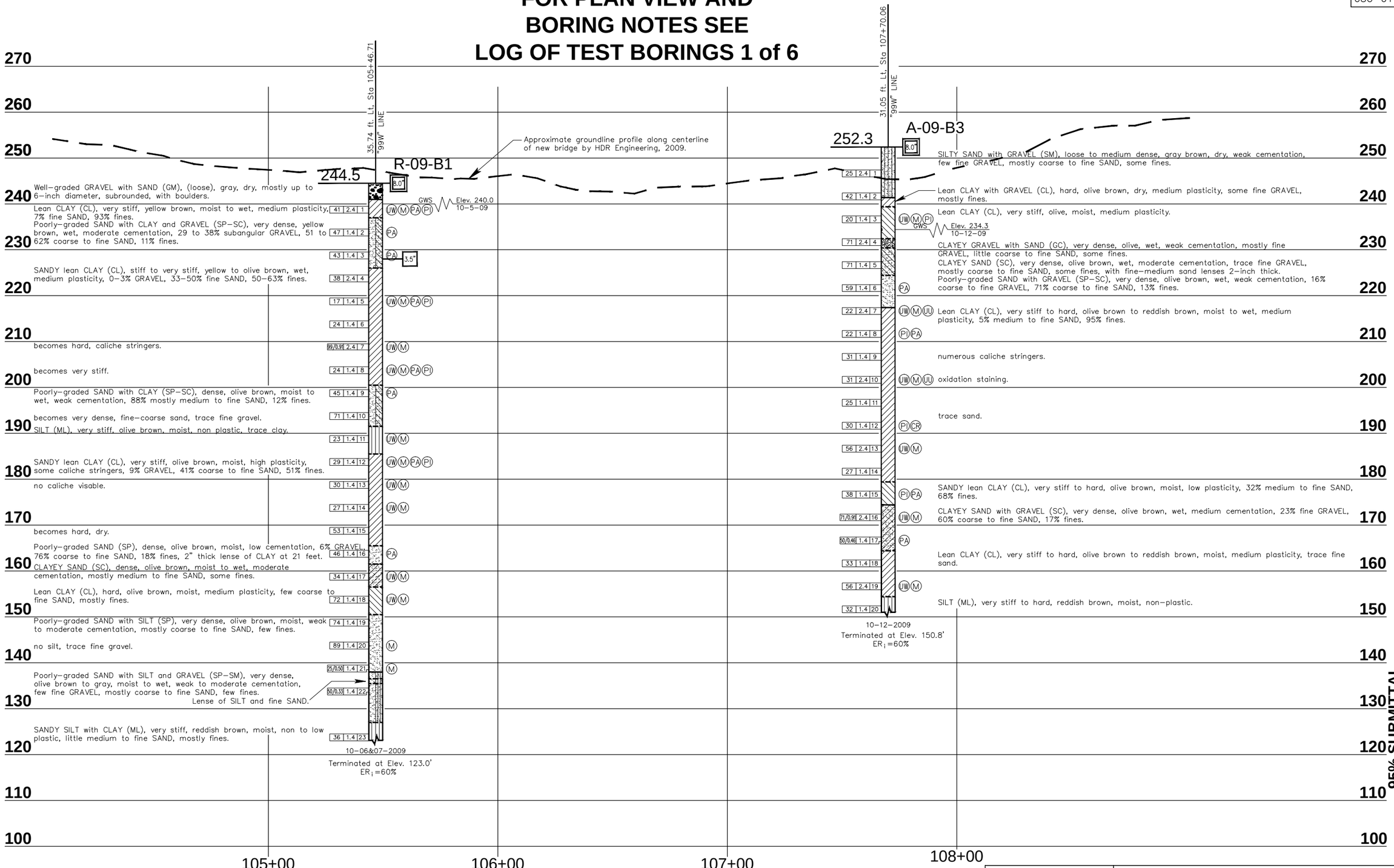
SCALE

99W at THOMES CREEK BRIDGE
PROJECT (REPLACE)
LOG OF TEST BORINGS 2 OF 6

PROJECT NO.: 370861

SHEET NO.	PAGE NO.
S-24	

FOR PLAN VIEW AND BORING NOTES SEE LOG OF TEST BORINGS 1 of 6



PROFILE

95% SUBMITTAL

BRIDGE NO. 08C-0110

No.	REVISION:	BY:	DATE:

These drawings are instruments of service provided by the Tehama County Public Works Dept. and are not to be used for any type of construction unless signed and sealed by a Registered Engineer in the employ of the County of Tehama

TEHAMA COUNTY PUBLIC WORKS
9380 San Benito Ave.
Gerber, Ca. 96035
(530) 385-1462

COUNTY OF TEHAMA

HORIZ:	1" = 20'
VERT:	1" = 10'
DRAWN BY:	M.D.R.
DATE:	11/2009
CHECKED BY:	D.J.M.
DATE:	05/2013

SCALE

99W at THOMES CREEK BRIDGE PROJECT (REPLACE)
LOG OF TEST BORINGS 3 OF 6

PROJECT NO.: 370861

SHEET NO. S-25 | PAGE NO.

blackburn consulting
Blackburn Consulting
2491 Boatman Avenue, West Sacramento, CA 95691
File No. 1274.1

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. 60578
Exp. 12-31-14
CIVIL
STATE OF CALIFORNIA

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

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.

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)
⬚	O	Other

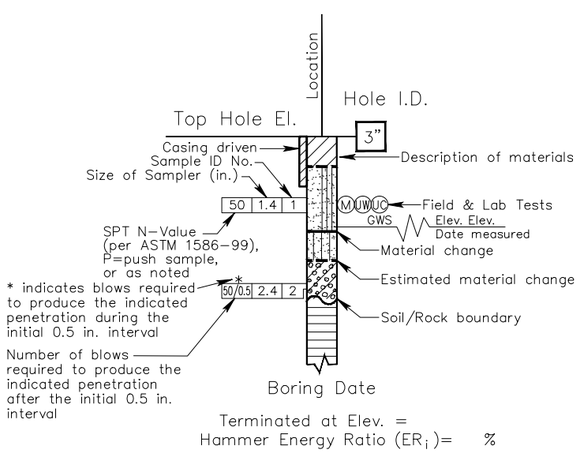
NOTE: Size in inches.

CONSISTENCY OF COHESIVE SOILS

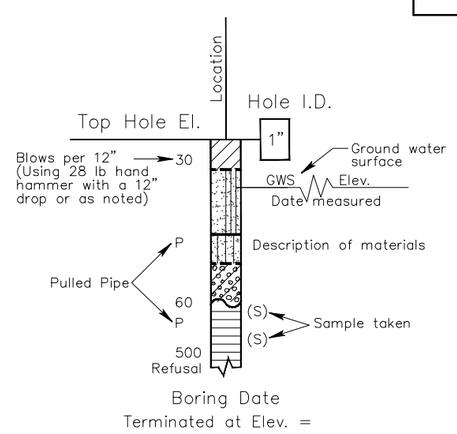
Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	<0.25	<0.25	<0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	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

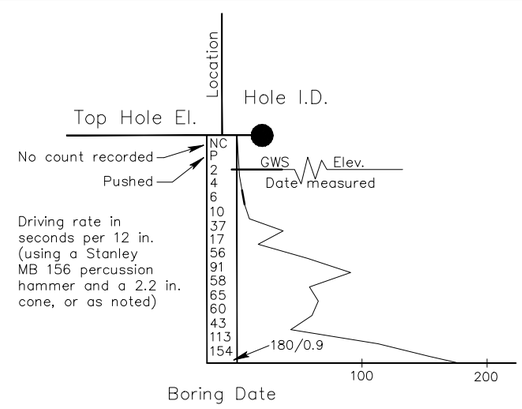
Description	Criteria
Nonplastic	A 1/8–inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be 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.



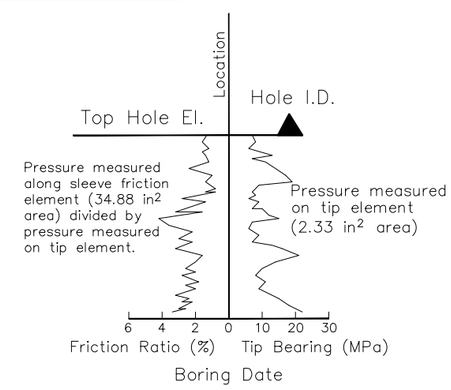
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

95% SUBMITTAL

SOIL LEGEND

blackburn consulting
Blackburn Consulting
2491 Boatman Avenue, West Sacramento, CA 95691
File No. 1274.1

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. 60578
Exp. 12-31-14
CIVIL
STATE OF CALIFORNIA

BRIDGE NO. 08C–0110

No.	REVISION:	BY:	DATE:

These drawings are instruments of service provided by the Tehama County Public Works Dept. and are not to be used for any type of construction unless signed and sealed by a Professional Engineer in the employ of the County of Tehama

TEHAMA COUNTY PUBLIC WORKS
9380 San Benito Ave.
Gerber, Ca. 96035
(530) 385–1462

SCALE
HORIZ: NO SCALE
VERT: NO SCALE
DRAWN BY: M.D.R.
DATE: 11/2009
CHECKED BY: D.J.M.
DATE: 05/2013

99W at THOMES CREEK BRIDGE
PROJECT (REPLACE)
LOG OF TEST BORINGS 4 OF 6

PROJECT NO.: 370861

SHEET NO. S-26
PAGE NO.

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

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL 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
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
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
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly-graded SAND Poorly-graded SAND with GRAVEL
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)
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)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
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
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
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT 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
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
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
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
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
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
OH/OL	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

FIELD AND LABORATORY TESTING

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

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60}-Value (Blows / 12 inches)
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"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

95% SUBMITTAL

SOIL LEGEND

REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER DAVID J. MORRELL No. 60578 Exp. 12-31-14 CIVIL STATE OF CALIFORNIA

blackburn consulting
Blackburn Consulting
2491 Boatman Avenue, West Sacramento, CA 95691
File No. 1274.1

BRIDGE NO. 08C-0110

No.	REVISION:	BY:	DATE:

These drawings are instruments of service provided by the Tehama County Public Works Dept. and are not to be used for any type of construction unless signed and sealed by a Registered Engineer in the employ of the County of Tehama

TEHAMA COUNTY PUBLIC WORKS
9380 San Benito Ave.
Gerber, Ca. 96035
(530) 385-1462

SCALE: HORIZ: NO SCALE; VERT: NO SCALE
DRAWN BY: M.D.R. DATE: 11/2009
CHECKED BY: D.J.M. DATE: 05/2013

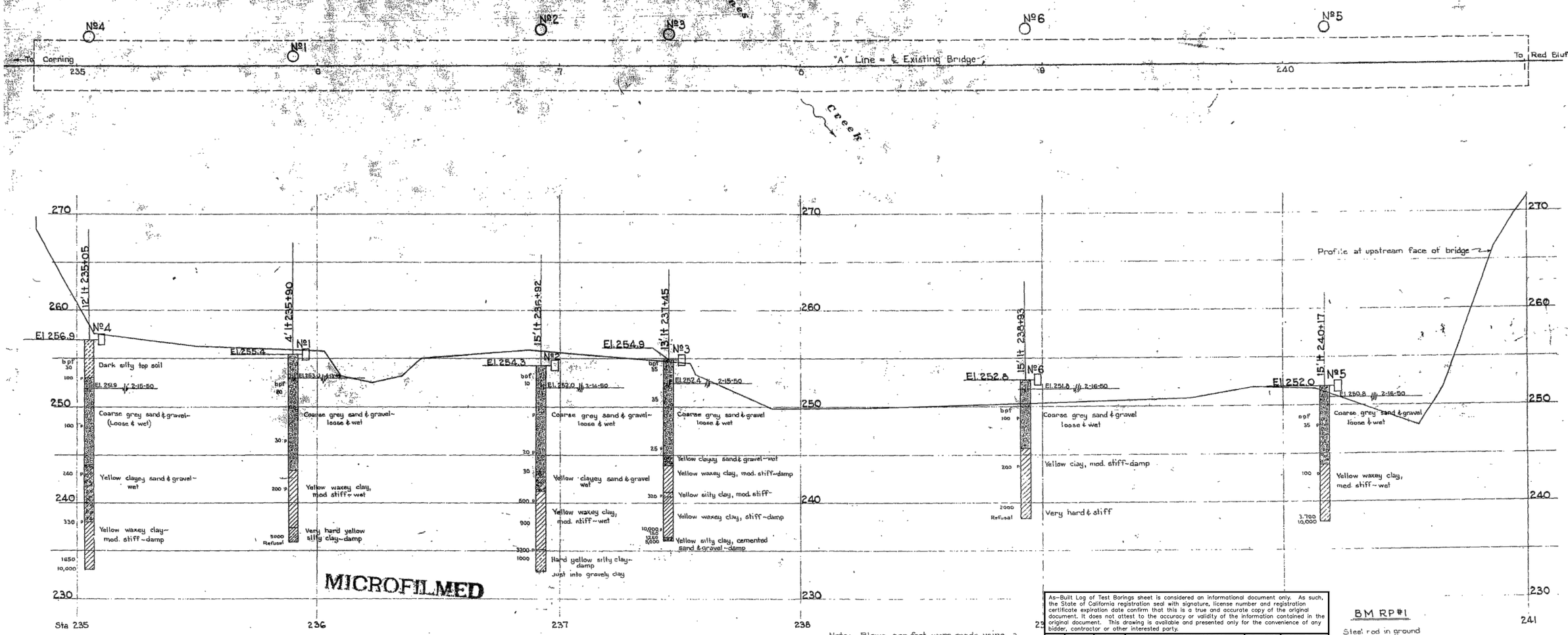
99W at THOMES CREEK BRIDGE PROJECT (REPLACE)
LOG OF TEST BORINGS 5 OF 6

PROJECT NO.: 370861

SHEET NO. S-27 | PAGE NO.

District	County	Route	Post Miles
03	Teh	7	A 14.21

March 6, 1950



MICROFILMED

Note: Blows per foot were made using a 28 lb hand hammer with a 12" drop

BM RP#1
Steel rod in ground
56.35' rt sta 241+75
Elev 258.47

95% SUBMITTAL

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	POST MILES–TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
03	Teh	99 (Old)		–	–

REGISTERED ENGINEERING GEOLOGIST DATE

99W at THOMES CREEK BRIDGE, PROJECT (REPLACE)

LOG OF TEST BORINGS 6 OF 6

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

CU:
EA:

BRIDGE NO. 08C–0377

SHEET NO. S-28 PAGE NO.

Revisions made to this Log of Test Borings from the original 1963 Log of Test Borings are the addition of the following table and notes.

Boring	Station	Offset from "99W" LINE
No. 1	102+31.42	03.54 ft Lt
No. 2	103+83.79	14.38 ft Lt
No. 3	104+36.81	12.42 ft Lt
No. 4	101+97.23	11.87 ft Lt
No. 5	107+07.21	14.66 ft Lt
No. 6	105+83.98	14.27 ft Lt

REGISTERED PROFESSIONAL ENGINEER
DAVID J. MORRELL
No. 60578
Exp. 12/31/14
CIVIL
STATE OF CALIFORNIA

NOTES

[illegible] DIRECTED TO SECTION 2, ARTICLE [illegible] OF SPECIFICATIONS AND TO THE SPECIAL PROVISIONS [illegible] AND [illegible]ATION OF EARTH MATERIAL AS SHOWN ON THIS SHEET [illegible] [illegible]TERN AND IS NOT TO BE CONSTRUED TO IMPLY MECHANICAL

AS BUILT (No corrections)
CORRECTIONS BY J. Hilborn
DATE: 2-5-51

CLASSIFICATION OF MATERIAL BASED ON STANDARD GRADE SIZE LIMITS

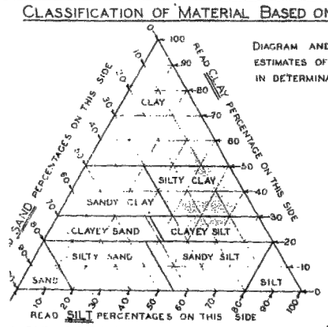


DIAGRAM AND TABLE SHOWING THE BASIS FOR ESTIMATES OF GRADE SIZE DISTRIBUTION USED IN DETERMINATION OF CLASS NAMES.

GROUPING USED IN TRIANGULAR CLASSIFICATION			
CLASS NAME	PERCENTAGE OF SIZES PRESENT		
	SAND	SILT	CLAY
SAND	80-100	0-20	0-20
SILTY SAND	45-80	0-55	0-20
SANDY SILT	0-45	35-80	0-20
SILT	0-20	80-100	0-20
CLAYEY SAND	38-80	0-42	20-30
CLAYEY SILT	0-38	32-80	20-30
SANDY CLAY	30-70	0-40	30-50
SILTY CLAY	0-30	20-70	30-50
CLAY	0-50	0-50	50-100

GRAVEL IS PRESENT IN APPRECIABLE AMOUNTS THE TERM "GRAVELLY" MAY BE ADDED TO THE CLASS NAME, IS. "GRAVELLY SAND." THE TERMS "COARSE," "MEDIUM" AND "FINE" WHEN USED TO DESCRIBE GRAVEL, SAND AND SILT REFER TO STANDARD GRADE SIZE LIMITS.

LEGEND OF BORING OPERATIONS

- PLAN OF ANY BORING
- 1" SAMPLER BORING
- ROTARY WASH BORING
- 1" CLOSED SAMPLER DRIVEN
- CORE BORING
- 2½" PENETROMETER DRIVEN
- 2" SAMPLER BORING
- 2" TO 5" AUGER BORING
- 6" TO 20" AUGER BORING
- CASING DRIVEN
- JET BORING
- (S) SAMPLE TAKEN

THE APPROPRIATE BORING SYMBOLS DESIGNATING THE METHOD OF OPERATION ARE SHOWN AT THE UPPER RIGHT-HAND CORNER OF THE RESPECTIVE BORING. WHERE TOOL CHANGES WERE MADE DURING THE BORING OPERATION SYMBOLS ARE SHOWN AT THE POINT OF CHANGE.

LEGEND OF EARTH MATERIALS

- GRAVEL – G
- SAND – S
- SILT – SI
- CLAY – C
- SILTY SAND – SI S
- CLAYEY SAND – C S
- SANDY SILT – S SI
- CLAYEY SILT – C SI
- SANDY CLAY – SC
- SILTY CLAY – SI C
- PEAT AND ORGANIC CLAY – O
- SANDSTONE – SS
- SHALE – SH
- BROKEN ROCK (FRAGMENTS) – BR
- ROCK – R

THOMES CREEK
LOG OF TEST BORINGS

Horizontal: 1" = 20'
Vertical: 1" = 5'

8-40 08C 0110

APPENDIX B

Laboratory Test Results

Laboratory Test Summary

Thomes Creek Bridge (Replace) at 99W

BCI File No. 1274.2

Boring	Sample	Depth (feet)	Unified Soil Classification	Dry Density (pcf)	Natural Moisture (%)	Plastic Limit	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Pocket Pen (tsf)	Unconfined Compressive Strength (tsf)	U-U Triaxial	Corrosivity			
														Su (psf)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
R-09-B1	1b	5.5-6.0	CL	100	23													
R-09-B1	1c	6.0-6.5	CL			26	44	18	0	7	93							
R-09-B1	2	10.0-11.5	SP-SC						38	51	11							
R-09-B1	3	15.0-16.5	SP-SC						29	60	11							
R-09-B1	5	25.0-26.5	CL	102	23	16	28	12	0	50	50							
R-09-B1	7	36.0-36.5	CL	100	26								0.6					
R-09-B1	8	40.0-41.5	CL	90	33	19	37	18	3	33	63							
R-09-B1	9	45.0-46.5	SP-SC						0	88	12							
R-09-B1	11	55.0-56.5	ML	109	23													
R-09-B1	12	60.0-61.5	CL	107	21	16	36	20	9	41	51							
R-09-B1	13	65.0-66.5	CL	101	26													
R-09-B1	14	70.0-71.5	CL	98	28													
R-09-B1	16	80.0-81.5	SP						6	76	18							
R-09-B1	17	85.0-86.5	SC	115	18													
R-09-B1	18	90.5-91.5	CL	108	22													
R-09-B1	20	110-111.5	SP		14													
R-09-B1	21	120.0-121.5	SP		14													
R-09-B2	1	10.0-11.5	SP-SM		12				35	55	11							
R-09-B2	2	15.0-16.5	CL	96	30	18	37	19	1	15	84							
R-09-B2	3c	21.0-21.5	CL	93	32				1	25	75							
R-09-B2	4	25.0-26.5	CL	97	30	16	34	18	0	14	86							
R-09-B2	5	30.0-31.5	CL	95	29													
R-09-B2	6b	35.5-36.0	CH	84	41									*1141				
R-09-B2	6c	36.0-36.5	CH			28	66	38	7	3	90	1.3						
R-09-B2	7	40.0-41.5	CL												7.97	1770	31.2	3.5
R-09-B2	8	45.0-46.5	CL	94	32				0	4	96							
R-09-B2	9c	51.0-51.5	CL	87	32													
R-09-B2	10	55.0-56.5	CL	95	24				14	36	50				7.97	2360	22.0	3.1
R-09-B2	12b	65.5-66.0	CL			22	44	22	15	25	60							
R-09-B2	12c	66.0-66.5	CL	104	24							2.0						
R-09-B2	13	70.0-71.5	CL												7.42	1630	26.0	5.2
R-09-B2	14	75.0-76.5	CL	100	26				0	43	57							
R-09-B2	15c	80.4-80.9	CL	100	18													
R-09-B2	16	85.0-86.5	ML	98	25													
R-09-B2	17b	90.5-91.0	ML						5	16	79							
R-09-B2	17c	91.0-91.5	CL	109	21													
R-09-B2	20	105.0-106.5	ML	94	31							2.0						

Laboratory Test Summary

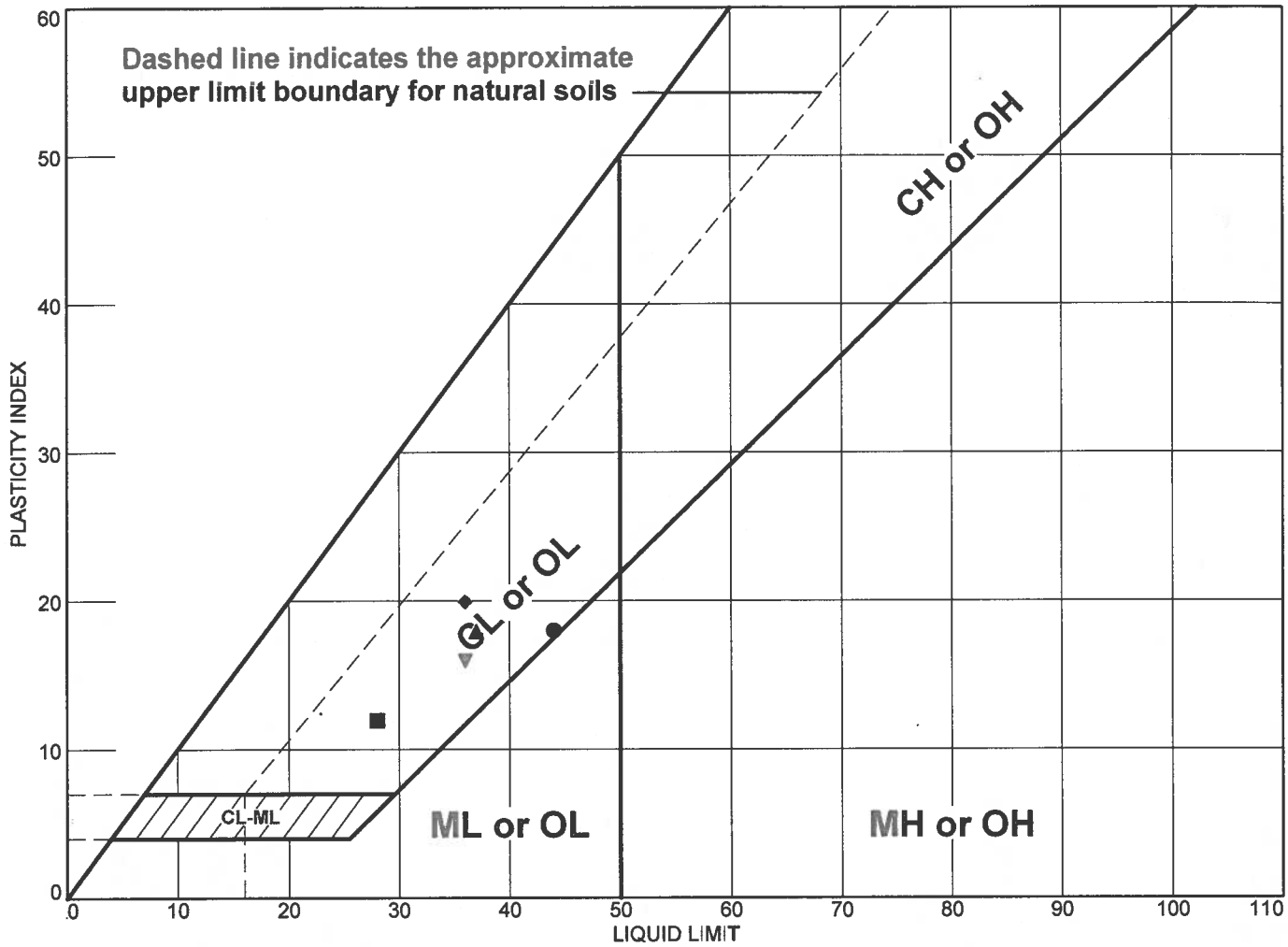
Thomes Creek Bridge (Replace) at 99W

BCI File No. 1274.2

Boring	Sample	Depth (feet)	Unified Soil Classification	Dry Density (pcf)	Natural Moisture (%)	Plastic Limit	Liquid Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)	Pocket Pen (tsf)	Unconfined Compressive Strength (tsf)	U-U Triaxial	Corrosivity			
														Su (psf)	pH	Minimum Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
A-09-B3	3	15.0-16.5	CL	107	24	20	32	12				2.5						
A-09-B3	6	30.0-31.5	SP-SC						16	71	13							
A-09-B3	7c	36.0-36.5	CL	107	22							2.5		*1332				
A-09-B3	8	40.0-41.5	CL			20	36	16	0	5	95							
A-09-B3	10c	51.0-51.5	CL	98	26							3.5		*1410				
A-09-B3	12	60.0-61.5	CL			20	37	17							7.49	1370	15.1	2.9
A-09-B3	13c	66.0-66.5	CL	115	19							>4.5						
A-09-B3	15	75.0-76.5	CL			18	28	10	0	32	68							
A-09-B3	16c	80.9-81.4	SC	128	12							2.5						
A-09-B3	17	85.0-86.5	SC						29	53	17							
A-09-B3	19c	96.0-96.5	CL	94	31							2.5						
A-09-B4	2	15.0-16.5	GC						50	37	13							
A-09-B4	4c	26.0-26.5	CL	89	31							2.5		*1954				
A-09-B4	6	35.0-36.5	CL			22	38	16	0	8	92							
A-09-B4	7c	41.0-41.5	CL	103	22							1.5		*886				
A-09-B4	10c	56.0-56.5	CL	111	20							4.5						
A-09-B4	12	65.0-66.5	CL			15	26	11	0	44	56							
A-09-B4	13c	71.0-71.5	CL	108	22							3.0		*1476				
A-09-B4	15	80.0-81.5	SM						0	59	41							
A-09-B4	16c	85.25-85.75	SW-SM	131	11				37	55	8	2.5						

*Samples appeared disturbed.

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Yellowish Brown Lean CLAY	44	26	18	99.4	92.9	CL
■	Light Olive Brown Sandy Lean CLAY	28	16	12	95.7	50.1	CL
▲	Light Yellowish Brown Sandy Lean CLAY	37	19	18	84.0	63.3	CL
◆	Olive Brown Sandy Lean CLAY	36	16	20	68.6	50.5	CL
▼	Olive Brown Lean CLAY	36	20	16	98.4	95.3	CL

Project No. 1274.1 **Client:** HDR Inc.

Project: SR 99 @ Thomes Creek

● **Depth:** 6.0-6.5 **Sample Number:** B1-1
■ **Depth:** 25.0'-26.5' **Sample Number:** B1-5
▲ **Depth:** 40.0'-41.5' **Sample Number:** B1-8
◆ **Depth:** 60.0'-61.5' **Sample Number:** B1-12
▼ **Depth:** 40.0-41.5 **Sample Number:** B3-8

Remarks:

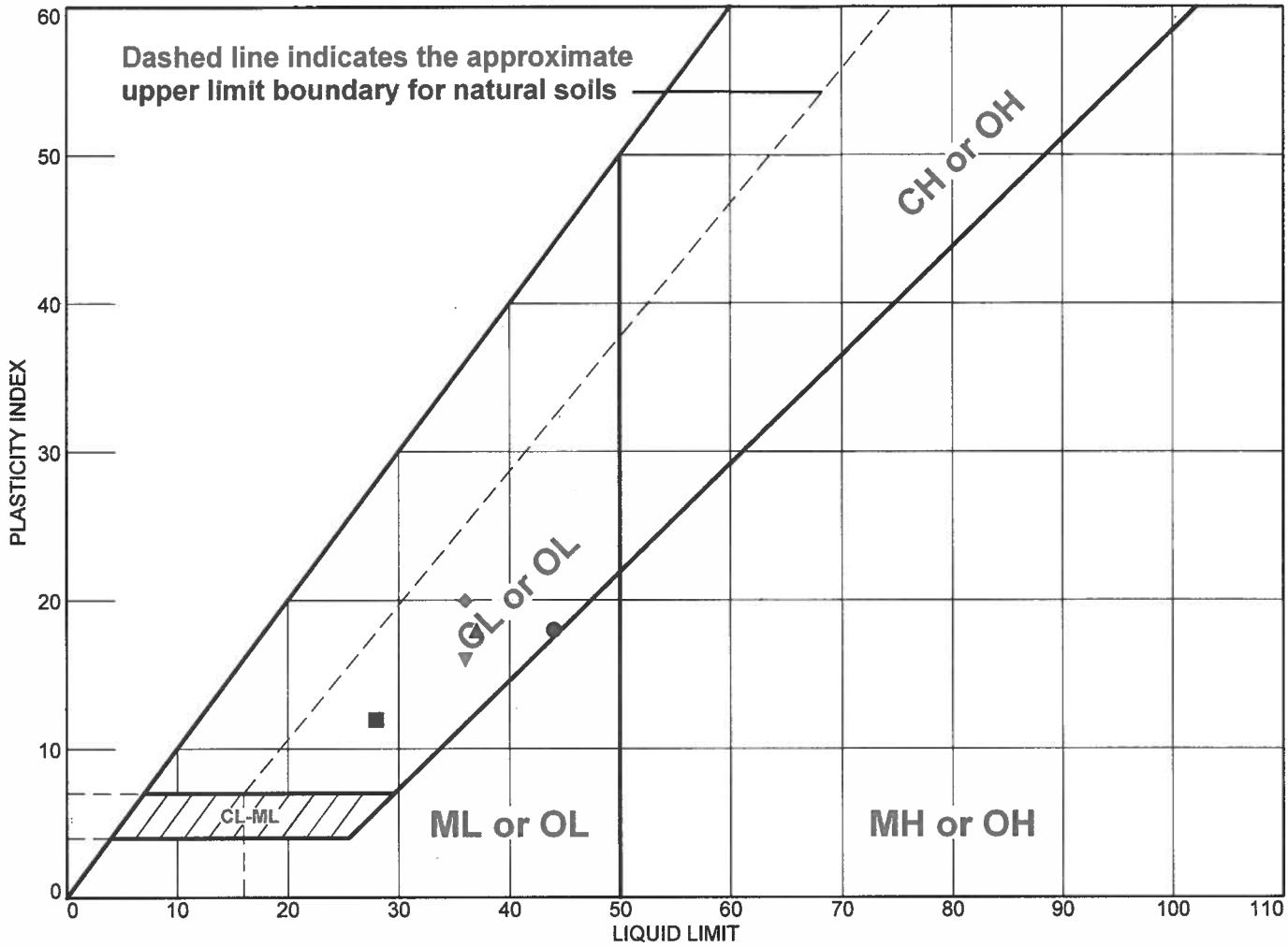
Blackburn Consulting

Auburn, CA

Figure

Tested By: KLC **Checked By:** KLC

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Yellowish Brown Lean CLAY	44	26	18	99.4	92.9	CL
■	Light Olive Brown Sandy Lean CLAY	28	16	12	95.7	50.1	CL
▲	Light Yellowish Brown Sandy Lean CLAY	37	19	18	84.0	63.3	CL
◆	Olive Brown Sandy Lean CLAY	36	16	20	68.6	50.5	CL
▼	Olive Brown Lean CLAY	36	20	16	98.4	95.3	CL

Project No. 1274.1 **Client:** HDR Inc.

Project: SR 99 @ Thomes Creek

● **Depth:** 6.0-6.5 **Sample Number:** B1-1
■ **Depth:** 25.0'-26.5' **Sample Number:** B1-5
▲ **Depth:** 40.0'-41.5' **Sample Number:** B1-8
◆ **Depth:** 60.0'-61.5' **Sample Number:** B1-12
▼ **Depth:** 40.0-41.5 **Sample Number:** B3-8

Remarks:

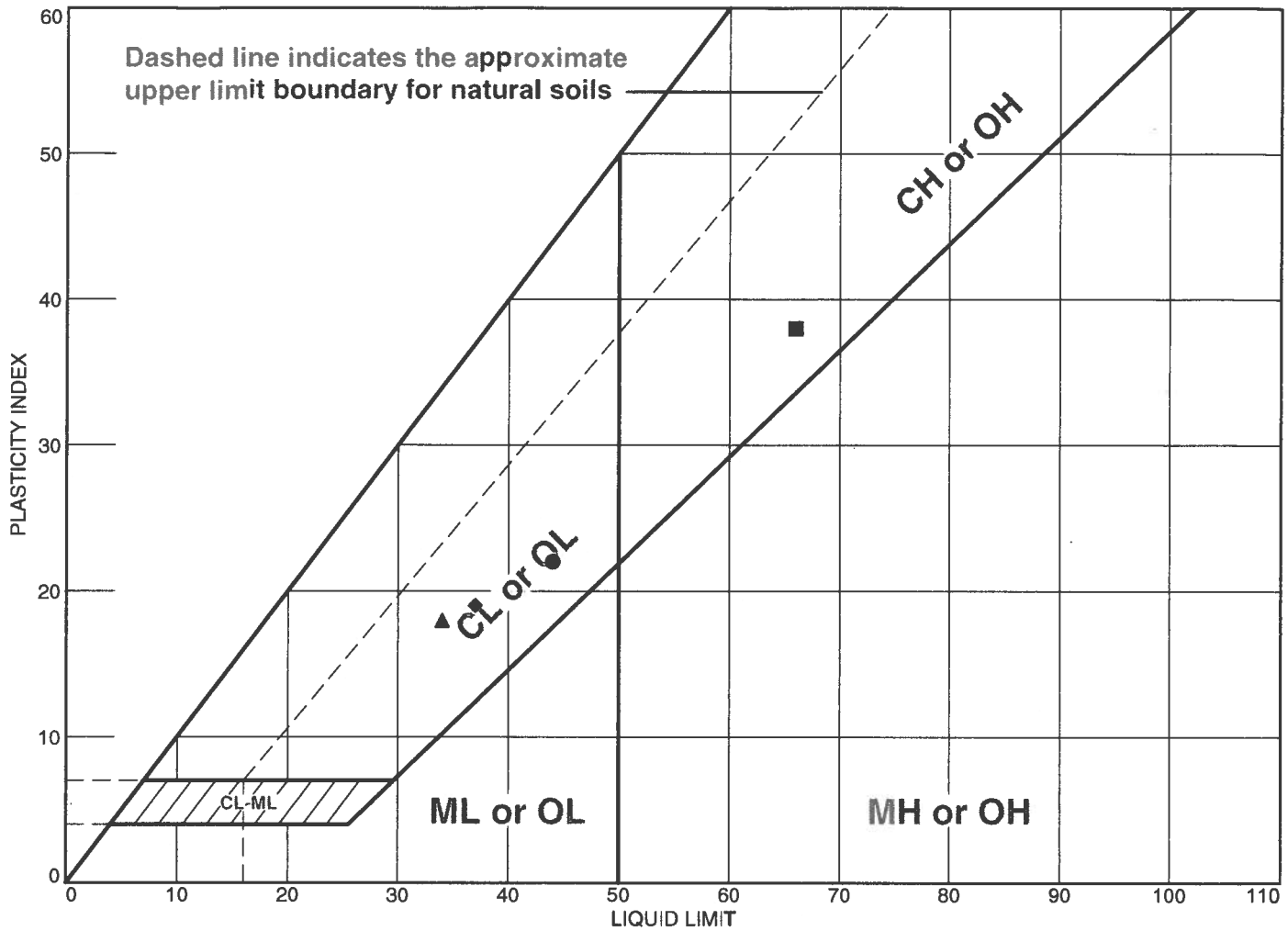
Blackburn Consulting

Auburn, CA

Figure

Tested By: KLC **Checked By:** KLC

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Olive Brown Sandy Lean CLAY	44	22	22	67.6	59.9	CL
■	Light Yellowish Brown Fat CLAY	66	28	38	91.8	90.0	CH
▲	Light Olive Brown Lean CLAY	34	16	18	97.5	85.6	CL
◆	Light Olive Brown	37	18	19	97.7	84.1	CL

Project No. 1274.1 **Client:** HDR Inc.

Project: SR 99 @ Thomes Creek

● **Depth:** 65.5'-66.0' **Sample Number:** B2-12
■ **Depth:** 36.0'-36.5' **Sample Number:** B2-6
▲ **Depth:** 25.0'-26.5' **Sample Number:** B2-4
◆ **Depth:** 15.0'-16.5' **Sample Number:** B2-2

Remarks:

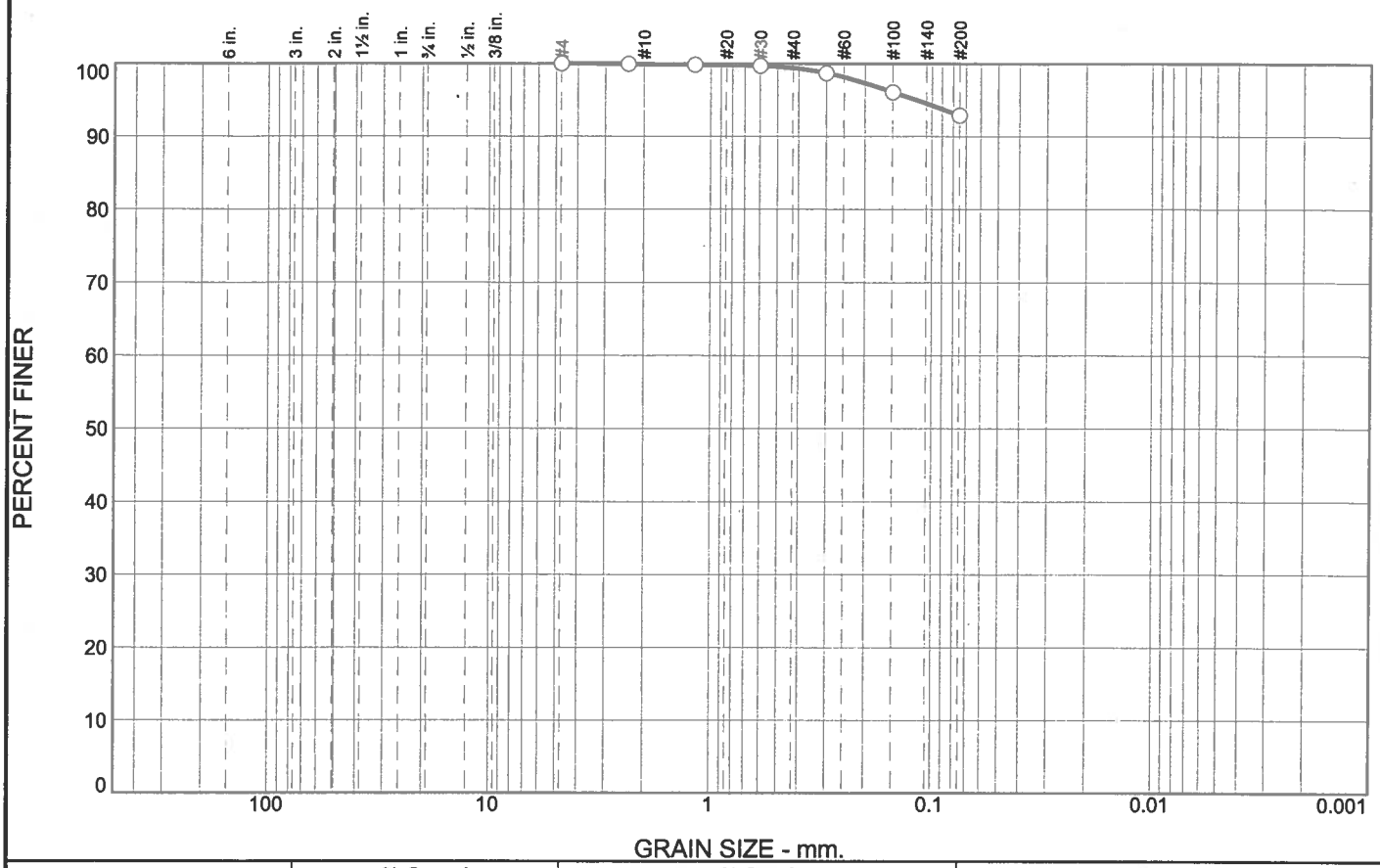
Blackburn Consulting

Auburn, CA

Figure

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.5	6.5	92.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.8		
#30	99.7		
#50	98.7		
#100	96.1		
#200	92.9		

* (no specification provided)

Material Description

Light Yellowish Brown Lean CLAY

Atterberg Limits

PL= 26 LL= 44 PI= 18

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(19)

Remarks

Sample Number: B1-1 **Depth:** 6.0-6.5

Date: 10-14-09

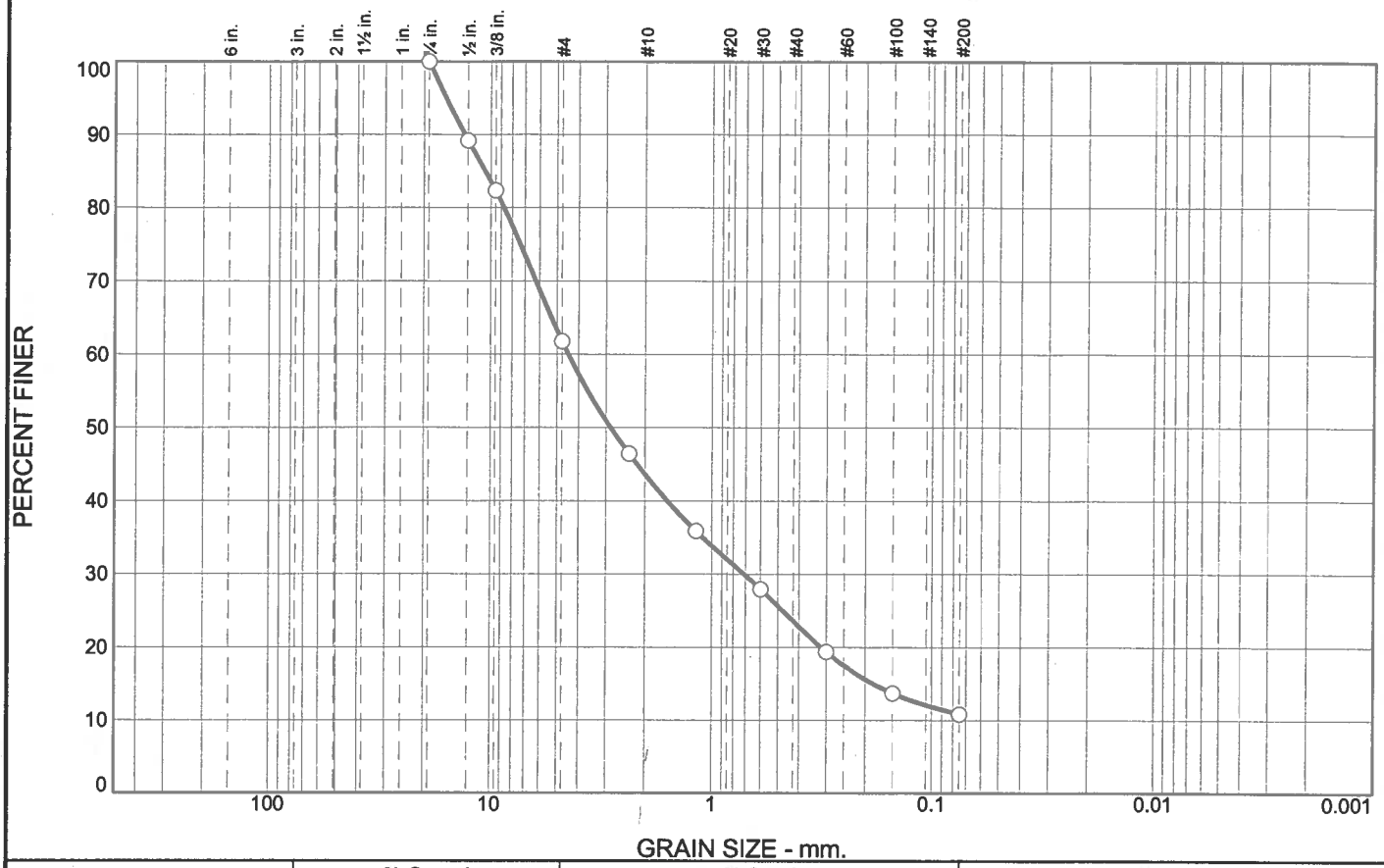
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	38.2	18.2	20.0	12.8	10.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	89.2		
3/8"	82.3		
#4	61.8		
#8	46.5		
#16	35.9		
#30	27.9		
#50	19.4		
#100	13.7		
#200	10.8		

* (no specification provided)

Material Description

Olive Brown poorly graded SAND with Clay and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.1181 D_{85}= 10.6200 D_{60}= 4.4491
D_{50}= 2.8575 D_{30}= 0.7139 D_{15}= 0.1841
D_{10}= C_u= C_c=

Classification

USCS= SP-SC AASHTO=

Remarks

Sample Number: B1-2 **Depth:** 10.0-11.5"

Date: 10-14-09

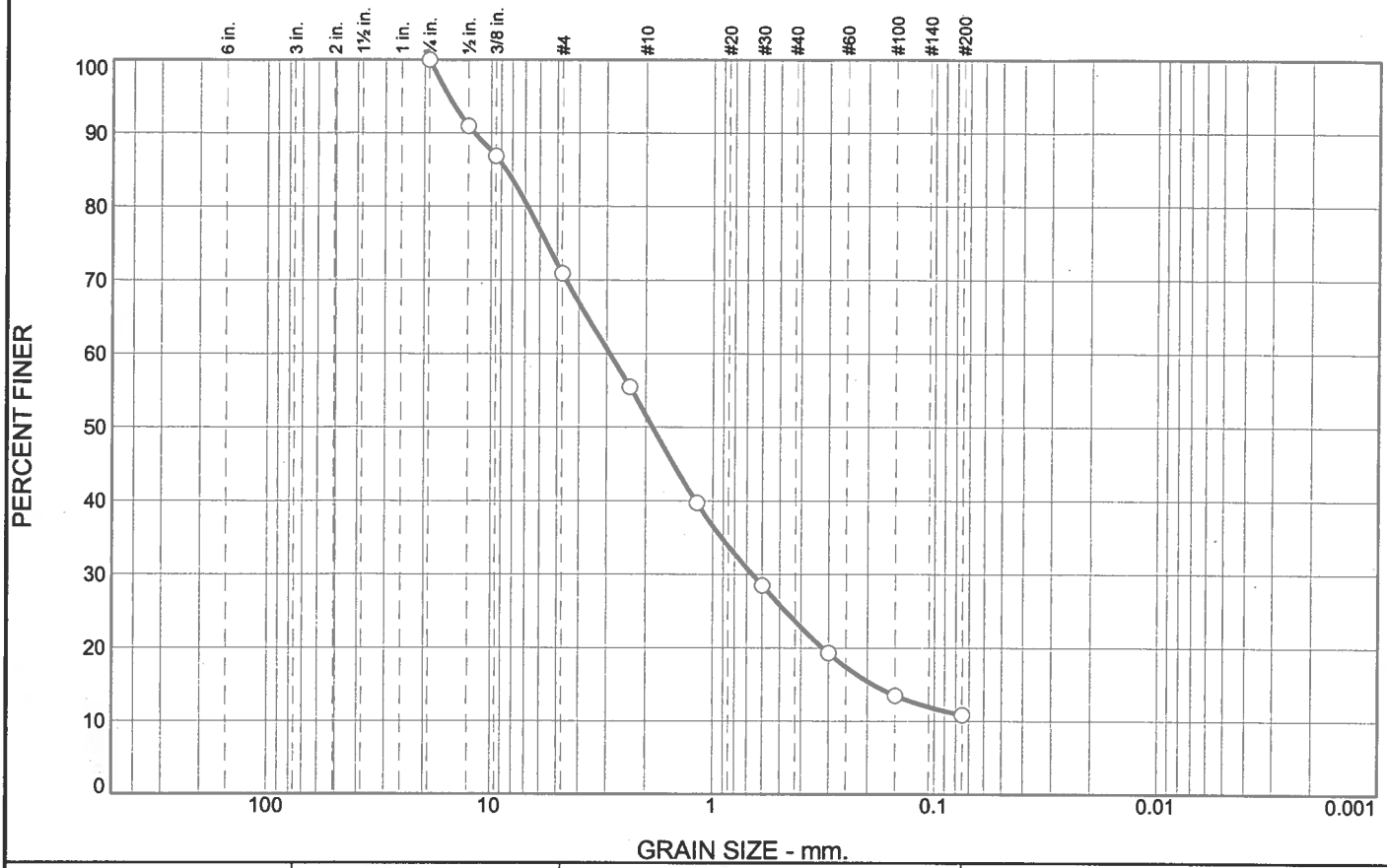
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	29.1	19.2	28.1	12.8	10.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	90.9		
3/8"	86.9		
#4	70.9		
#8	55.5		
#16	39.7		
#30	28.5		
#50	19.3		
#100	13.5		
#200	10.8		

* (no specification provided)

Material Description

Light Olive Brown Poorly Graded SAND with Clay and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.9470 D_{85}= 8.5709 D_{60}= 2.9012
D_{50}= 1.8618 D_{30}= 0.6651 D_{15}= 0.1893
D_{10}= C_u= C_c=

Classification

USCS= SP-SC AASHTO=

Remarks

Sample Number: B1-3 **Depth:** 15.0'-16.5'

Date: 10-14-09

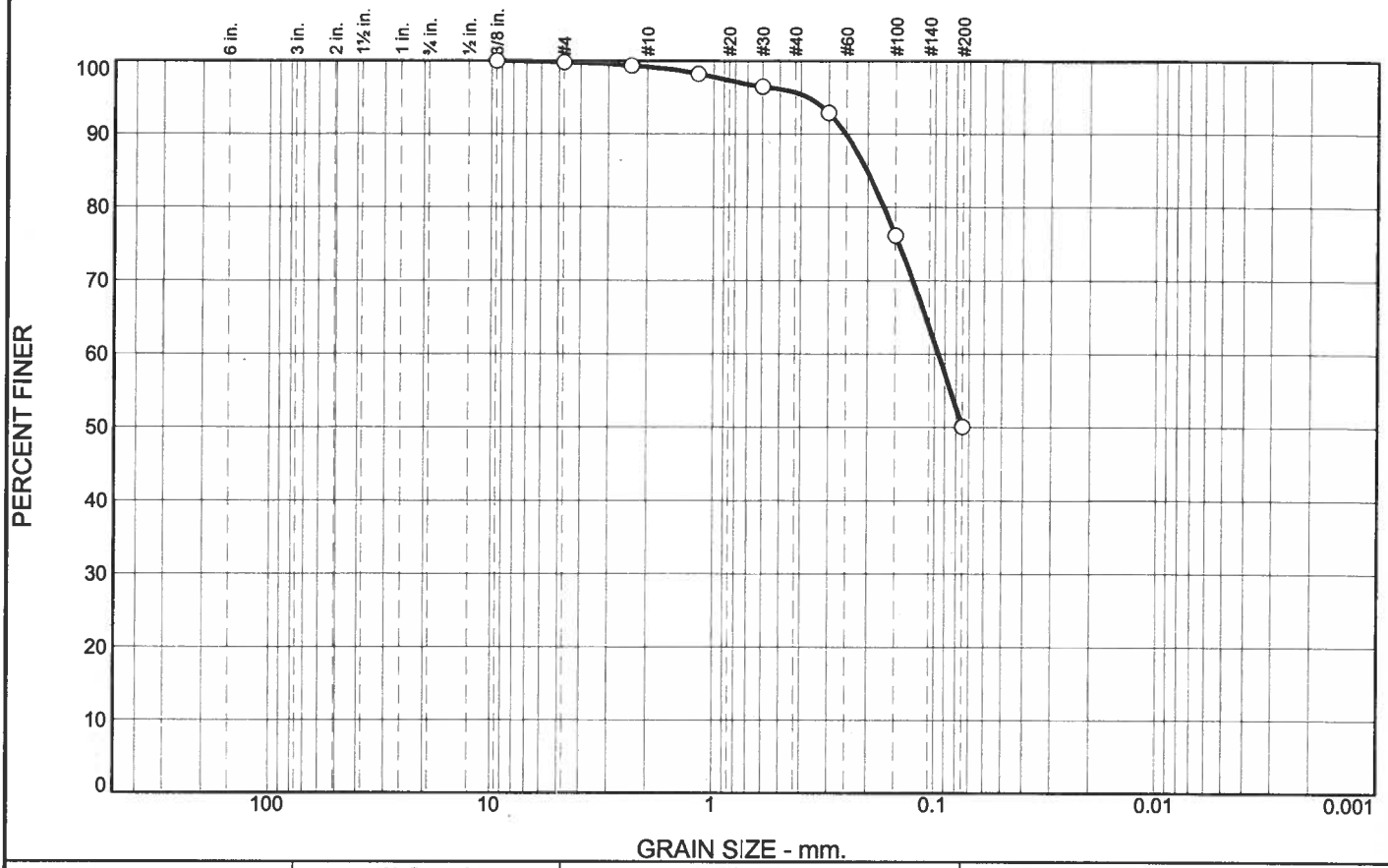
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.6	3.5	45.6	50.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.8		
#8	99.4		
#16	98.2		
#30	96.4		
#50	92.9		
#100	76.2		
#200	50.1		

* (no specification provided)

Material Description

Light Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= 16 LL= 28 PI= 12

Coefficients

D_{90}= 0.2514 D_{85}= 0.2020 D_{60}= 0.0963
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(3)

Remarks

Sample Number: B1-5 **Depth:** 25.0'-26.5'

Date: 10-14-09

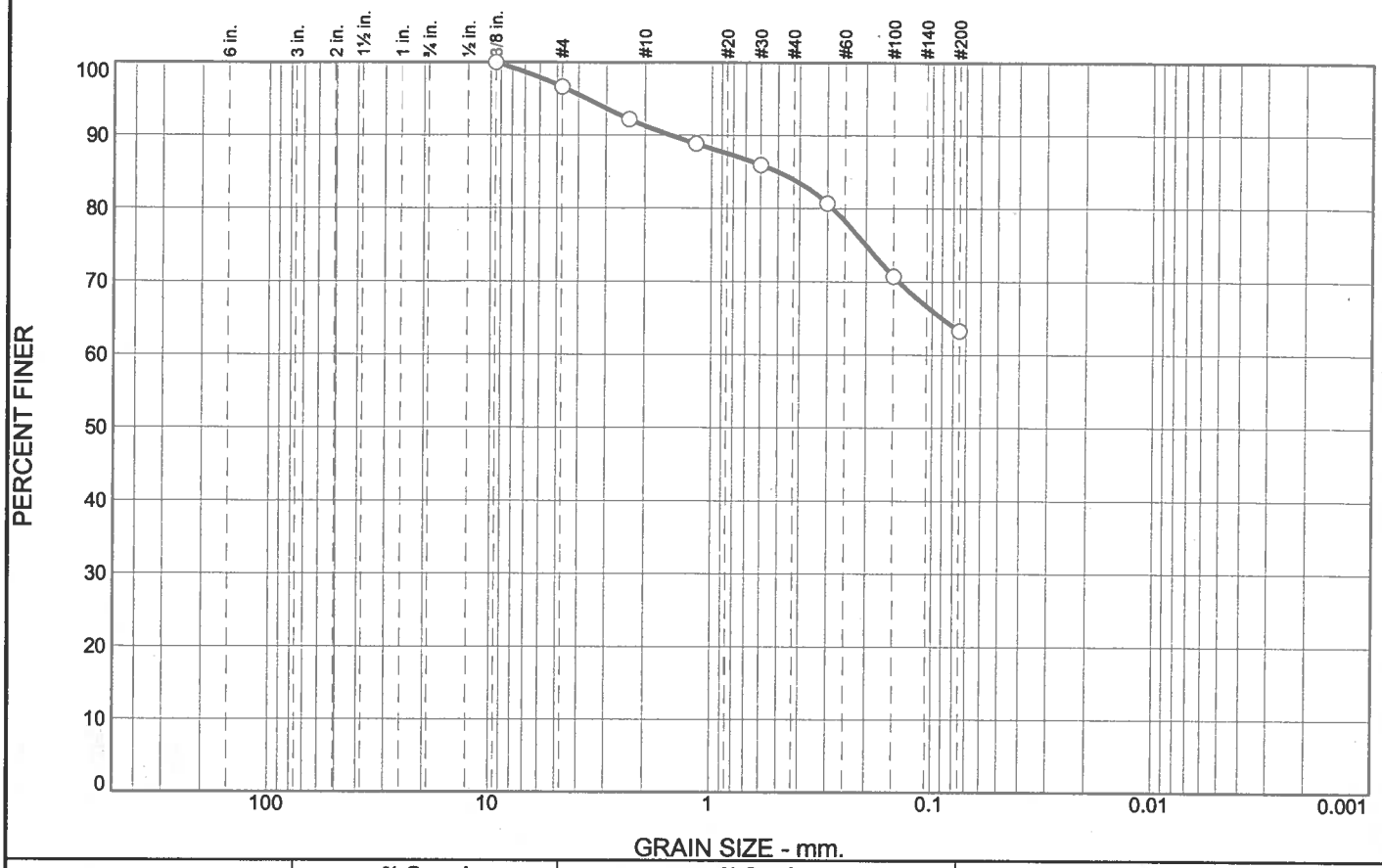
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.4	5.3	7.3	20.7	63.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	96.6		
#8	92.2		
#16	88.8		
#30	86.0		
#50	80.7		
#100	70.7		
#200	63.3		

* (no specification provided)

Material Description

Light Yellowish Brown Sandy Lean CLAY

Atterberg Limits

PL= 19 LL= 37 PI= 18

Coefficients

D_{90}= 1.5406 D_{85}= 0.5003 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(9)

Remarks

Sample Number: B1-8 **Depth:** 40.0'-41.5'

Date: 10-16-09

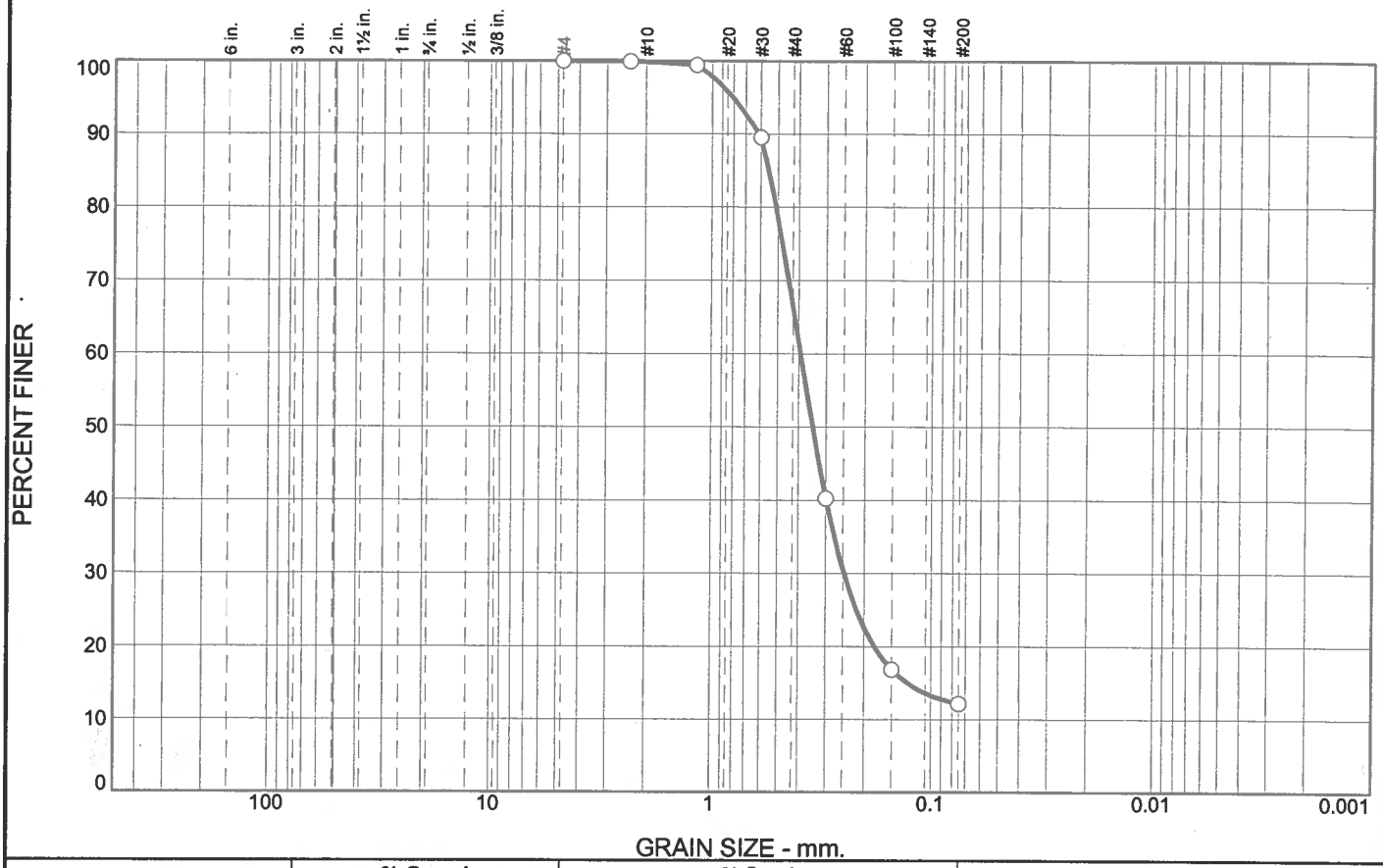
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	33.6	54.1	12.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	100.0		
#16	99.4		
#30	89.5		
#50	40.3		
#100	16.9		
#200	12.2		

* (no specification provided)

Material Description

Dark Grayish Brown Poorly Graded SAND with CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6140 D_{85}= 0.5522 D_{60}= 0.3925
D_{50}= 0.3450 D_{30}= 0.2472 D_{15}= 0.1278
D_{10}= C_u= C_c=

Classification

USCS= SP-SC AASHTO=

Remarks

Sample Number: B1-9 **Depth:** 45.0'-46.5'

Date: 10-15-09

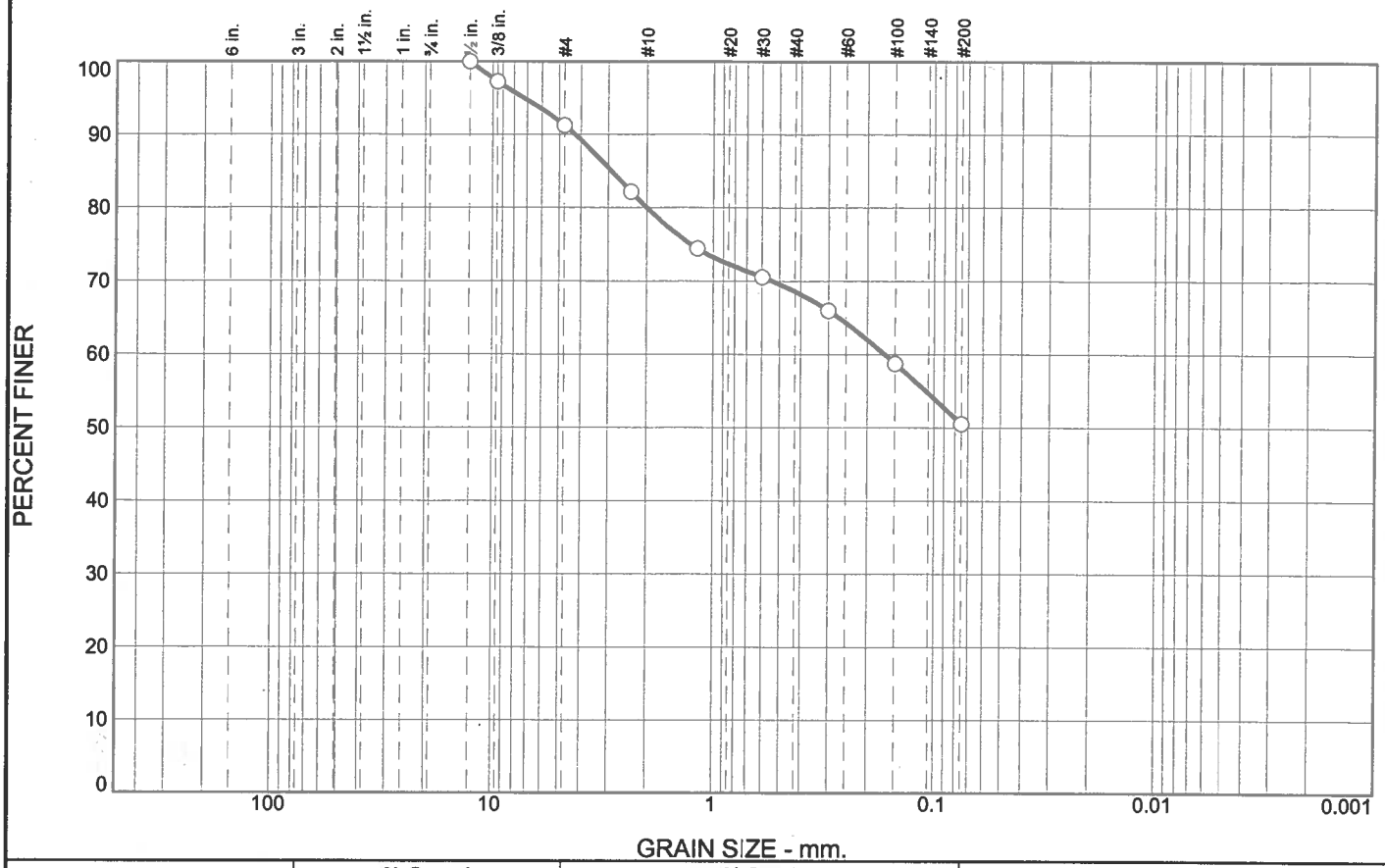
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	8.7	11.3	11.4	18.1	50.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	97.2		
#4	91.3		
#8	82.2		
#16	74.4		
#30	70.6		
#50	66.0		
#100	58.8		
#200	50.5		

* (no specification provided)

Material Description

Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= 16 LL= 36 PI= 20

Coefficients

D_{90}= 4.2492 D_{85}= 2.8991 D_{60}= 0.1670
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(6)

Remarks

Sample Number: B1-12 **Depth:** 60.0'-61.5'

Date: 10-17-09

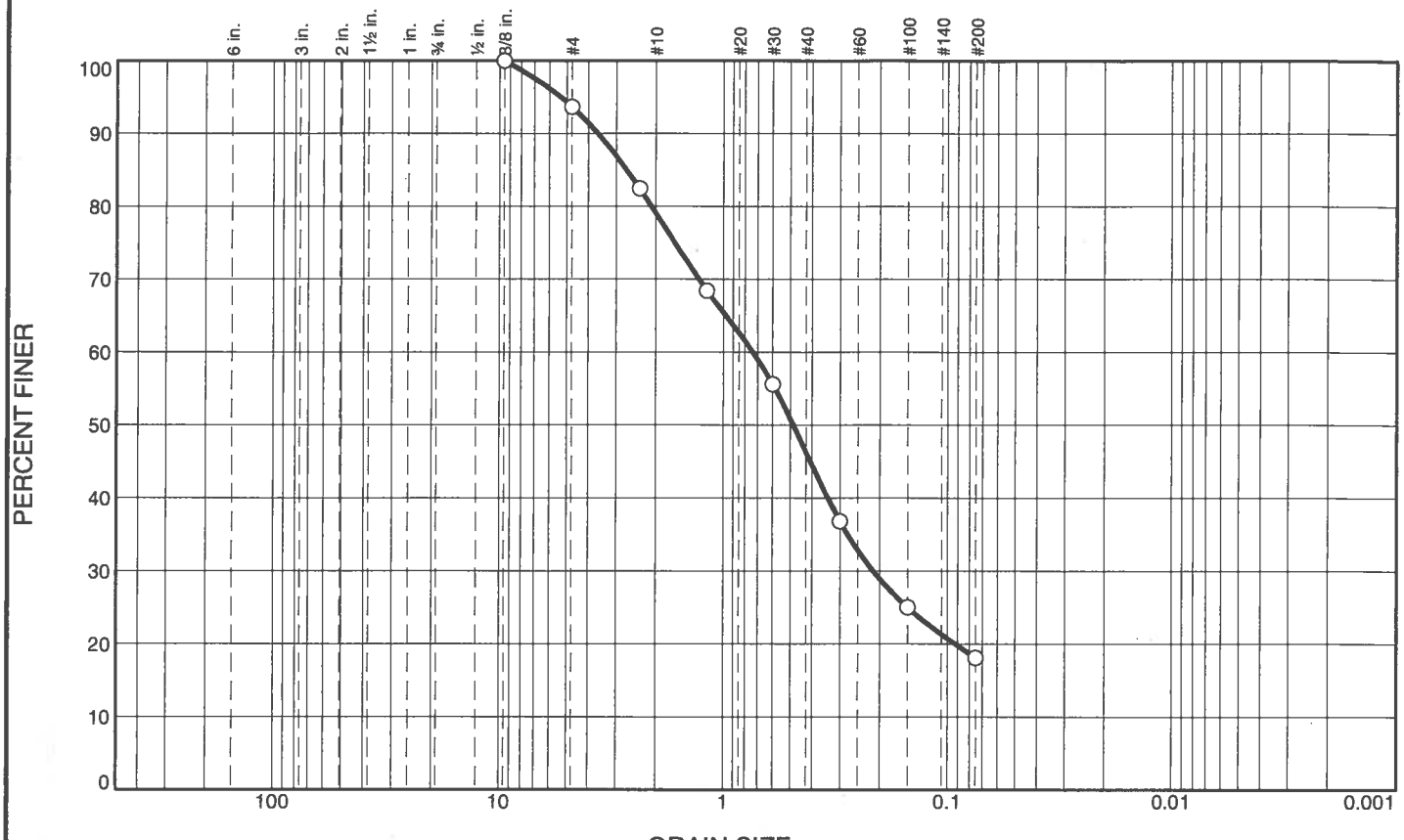
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.3	14.6	32.8	28.2	18.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	93.7		
#8	82.4		
#16	68.4		
#30	55.6		
#50	36.8		
#100	25.0		
#200	18.1		

* (no specification provided)

Material Description

Olive Brown Clayey SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.6323 D_{85}= 2.7026 D_{60}= 0.7365
D_{50}= 0.4844 D_{30}= 0.2139 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample Number: B1-16 **Depth:** 80.0'-81.5'

Date: 10-17-09

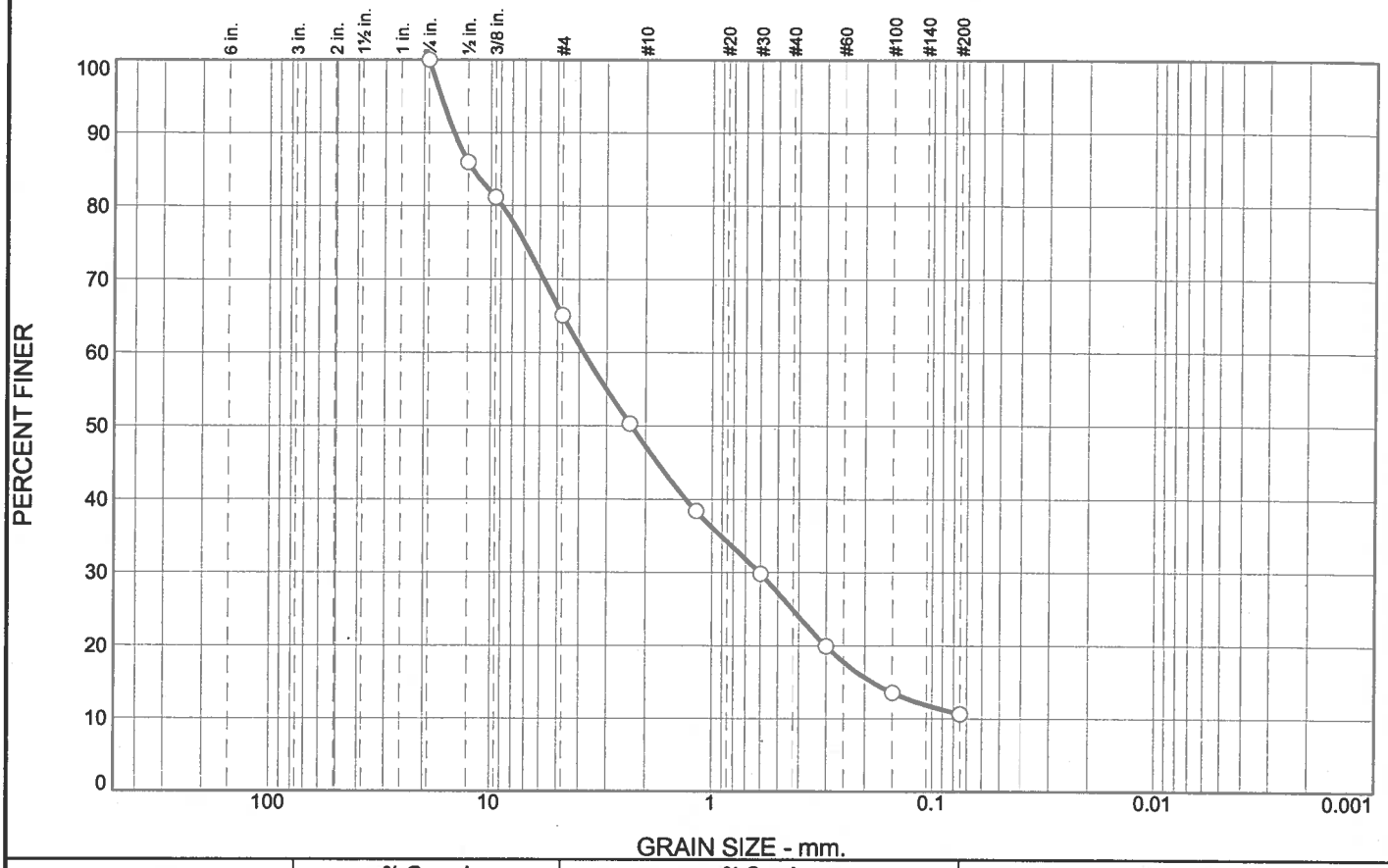
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	34.9	17.9	22.4	14.2	10.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	85.9		
3/8"	81.2		
#4	65.1		
#8	50.3		
#16	38.4		
#30	29.8		
#50	19.9		
#100	13.5		
#200	10.6		

* (no specification provided)

Material Description

Olive Brown Poorly Graded SAND with silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 14.6457 D_{85}= 12.1480 D_{60}= 3.8132
D_{50}= 2.3219 D_{30}= 0.6082 D_{15}= 0.1850
D_{10}= C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sample Number: B2-1 **Depth:** 10.0'-11.5'

Date: 10-14-09

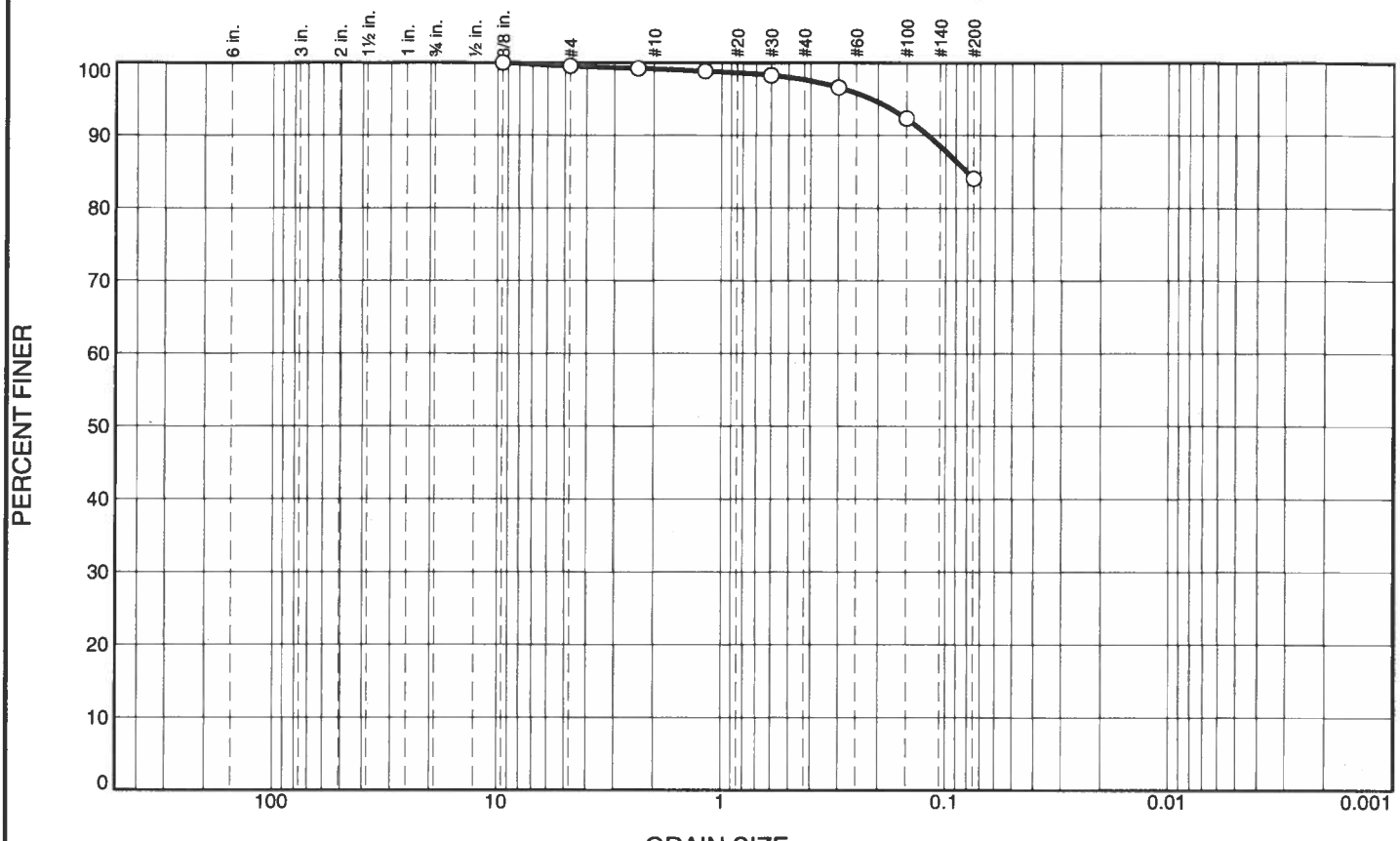
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	0.4	1.4	13.6	84.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.5		
#8	99.2		
#16	98.8		
#30	98.3		
#50	96.6		
#100	92.3		
#200	84.1		

* (no specification provided)

Material Description

Light Olive Brown Lean CLAY with Sand

Atterberg Limits

PL= 18 LL= 37 PI= 19

Coefficients

D_{90}= 0.1201 D_{85}= 0.0805 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(15)

Remarks

Sample Number: B2-2 **Depth:** 15.0'-16.5'

Date: 10-15-09

Blackburn Consulting

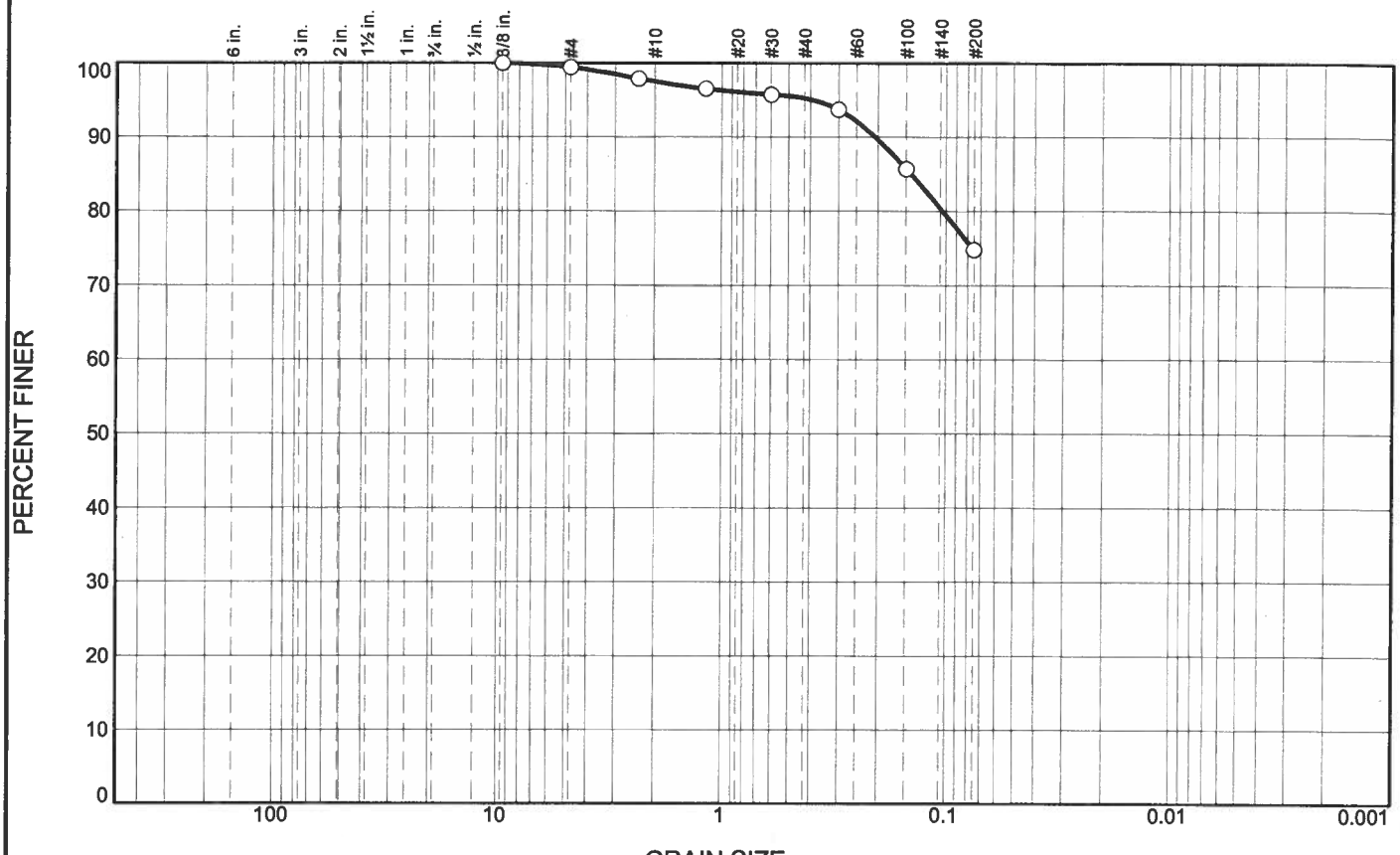
Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek

Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.6	1.9	2.3	20.4	74.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.4		
#8	97.9		
#16	96.5		
#30	95.8		
#50	93.7		
#100	85.7		
#200	74.8		

* (no specification provided)

Material Description

Light Olive Brown Lean CLAY with Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2061 D_{85}= 0.1432 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B2-3 **Depth:** 21.0'-21.5'

Date: 10-14-09

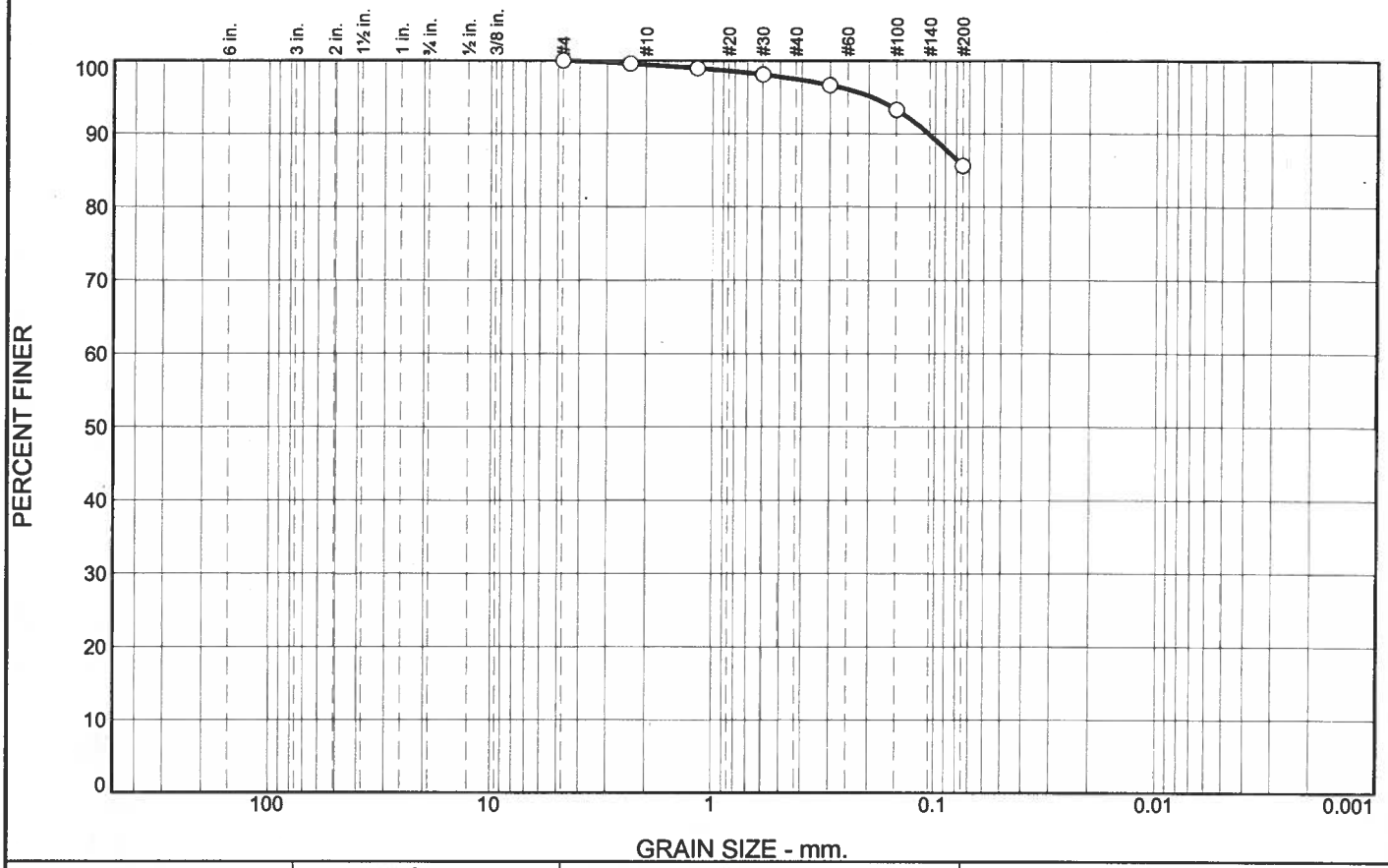
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.5	2.0	11.9	85.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.6		
#16	98.9		
#30	98.1		
#50	96.6		
#100	93.3		
#200	85.6		

* (no specification provided)

Material Description

Light Olive Brown Lean CLAY

Atterberg Limits

PL= 16 LL= 34 PI= 18

Coefficients

D_{90}= 0.1075 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(14)

Remarks

Sample Number: B2-4 **Depth:** 25.0'-26.5'

Date: 10-14-09

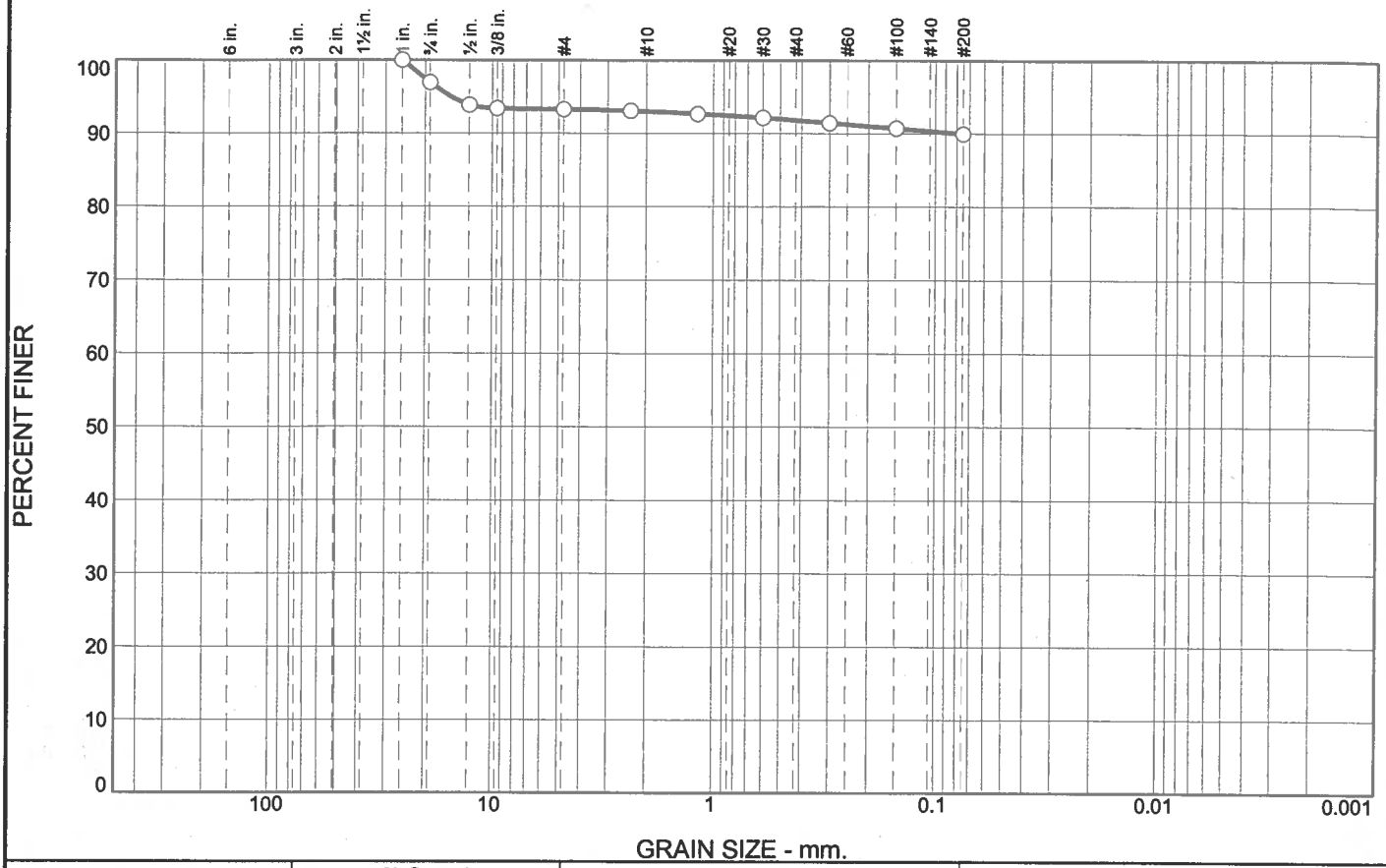
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.0	3.7	0.3	1.2	1.8	90.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0"	100.0		
3/4"	97.0		
1/2"	93.9		
3/8"	93.4		
#4	93.3		
#8	93.1		
#16	92.7		
#30	92.2		
#50	91.4		
#100	90.7		
#200	90.0		

* (no specification provided)

Material Description

Light Yellowish Brown Fat CLAY

Atterberg Limits

PL= 28 LL= 66 PI= 38

Coefficients

D_{90}= 0.0769 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(39)

Remarks

Sample Number: B2-6 **Depth:** 36.0'-36.5'

Date: 10-16-09

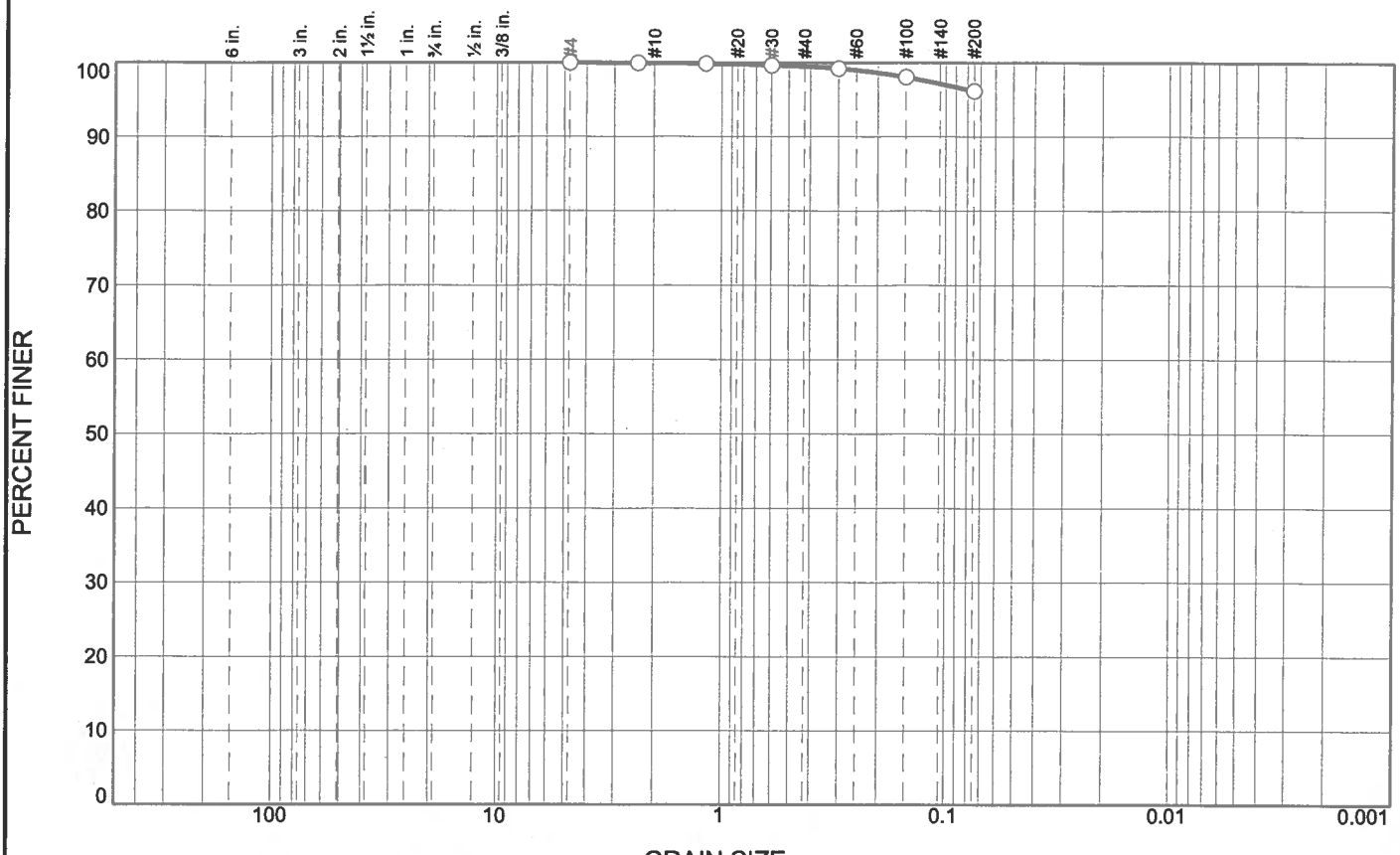
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.4	3.3	96.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#16	99.9		
#30	99.7		
#50	99.2		
#100	98.0		
#200	96.2		

* (no specification provided)

Material Description

Pale Olive Lean CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B2-8 **Depth:** 45.0'-46.5'

Date: 10-16-09

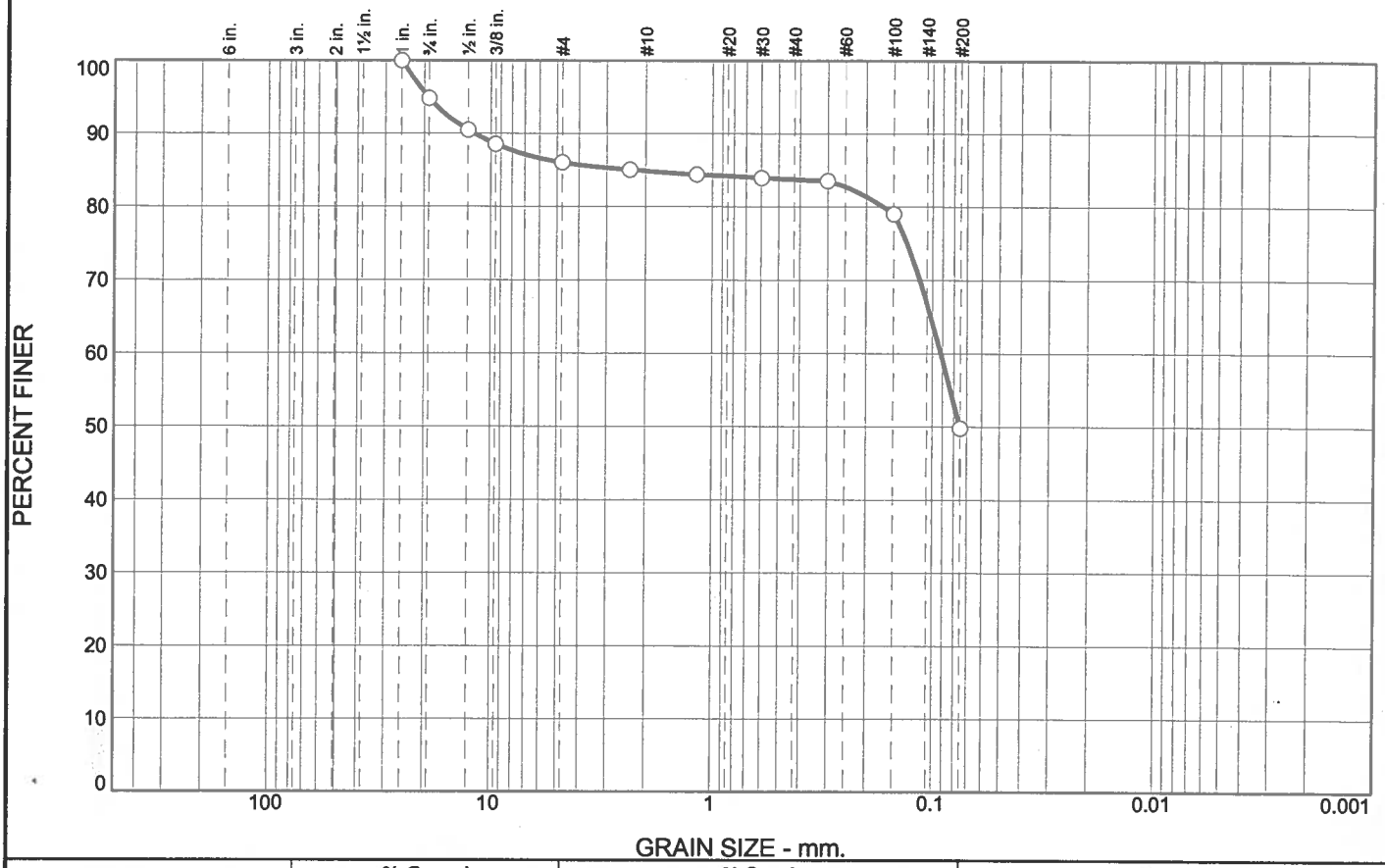
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.2	8.8	1.1	1.2	33.9	49.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0"	100.0		
3/4"	94.8		
1/2"	90.5		
3/8"	88.6		
#4	86.0		
#8	85.1		
#16	84.4		
#30	84.0		
#50	83.5		
#100	79.0		
#200	49.8		

* (no specification provided)

Material Description

Light Yellowish Brown Sandy Lean CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.8961 D_{85}= 2.2253 D_{60}= 0.0918
D_{50}= 0.0753 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B2-10 **Depth:** 55.0'-56.5'

Date: 10-16-09

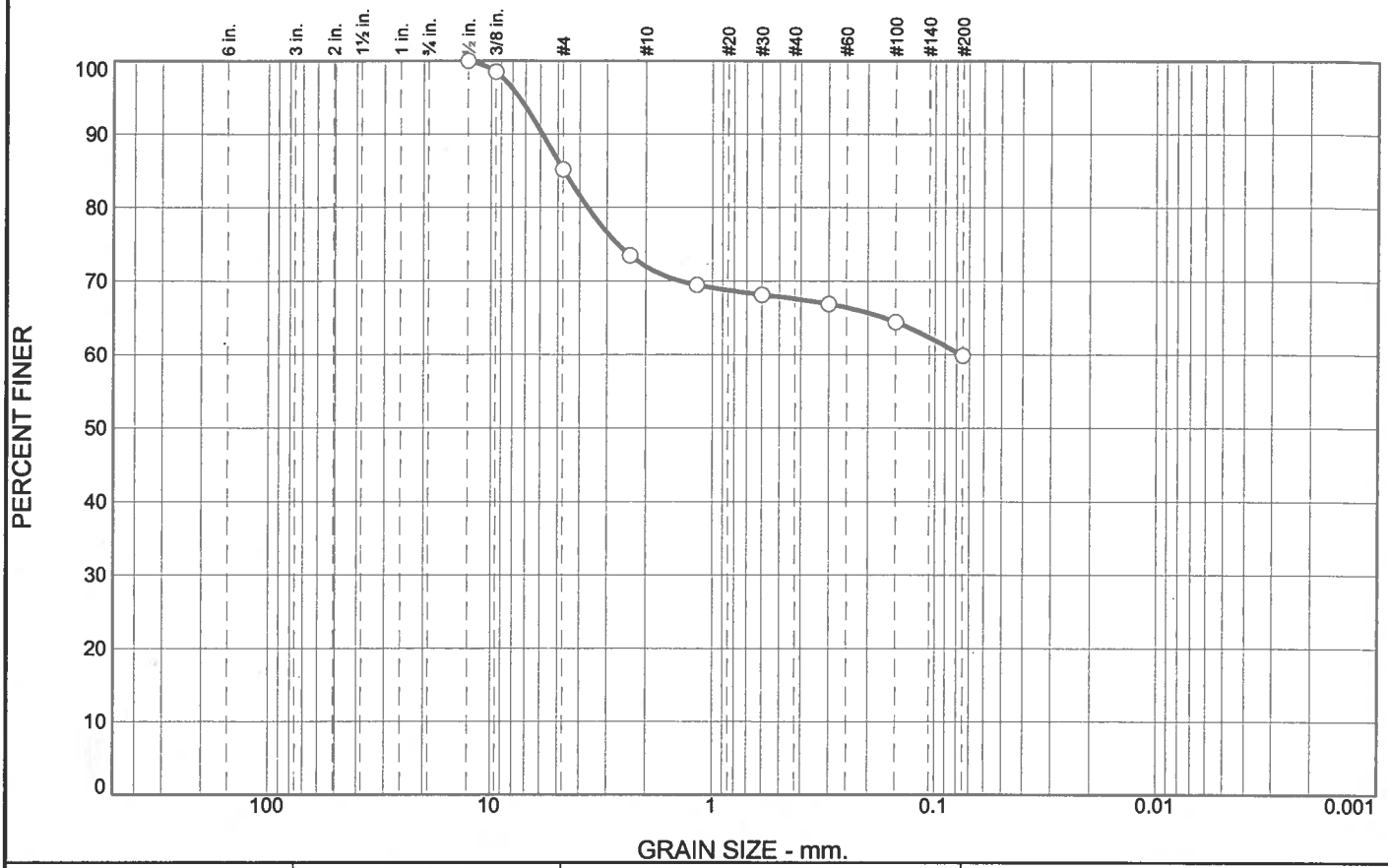
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	14.8	13.2	4.4	7.7	59.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2"	100.0		
3/8"	98.5		
#4	85.2		
#8	73.5		
#16	69.5		
#30	68.2		
#50	66.9		
#100	64.4		
#200	59.9		

* (no specification provided)

Material Description

Light Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= 22 LL= 44 PI= 22

Coefficients

D_{90}= 5.8877 D_{85}= 4.7064 D_{60}= 0.0764
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(11)

Remarks

Sample Number: B2-12 **Depth:** 65.5'-66.0'

Date: 10-16-09

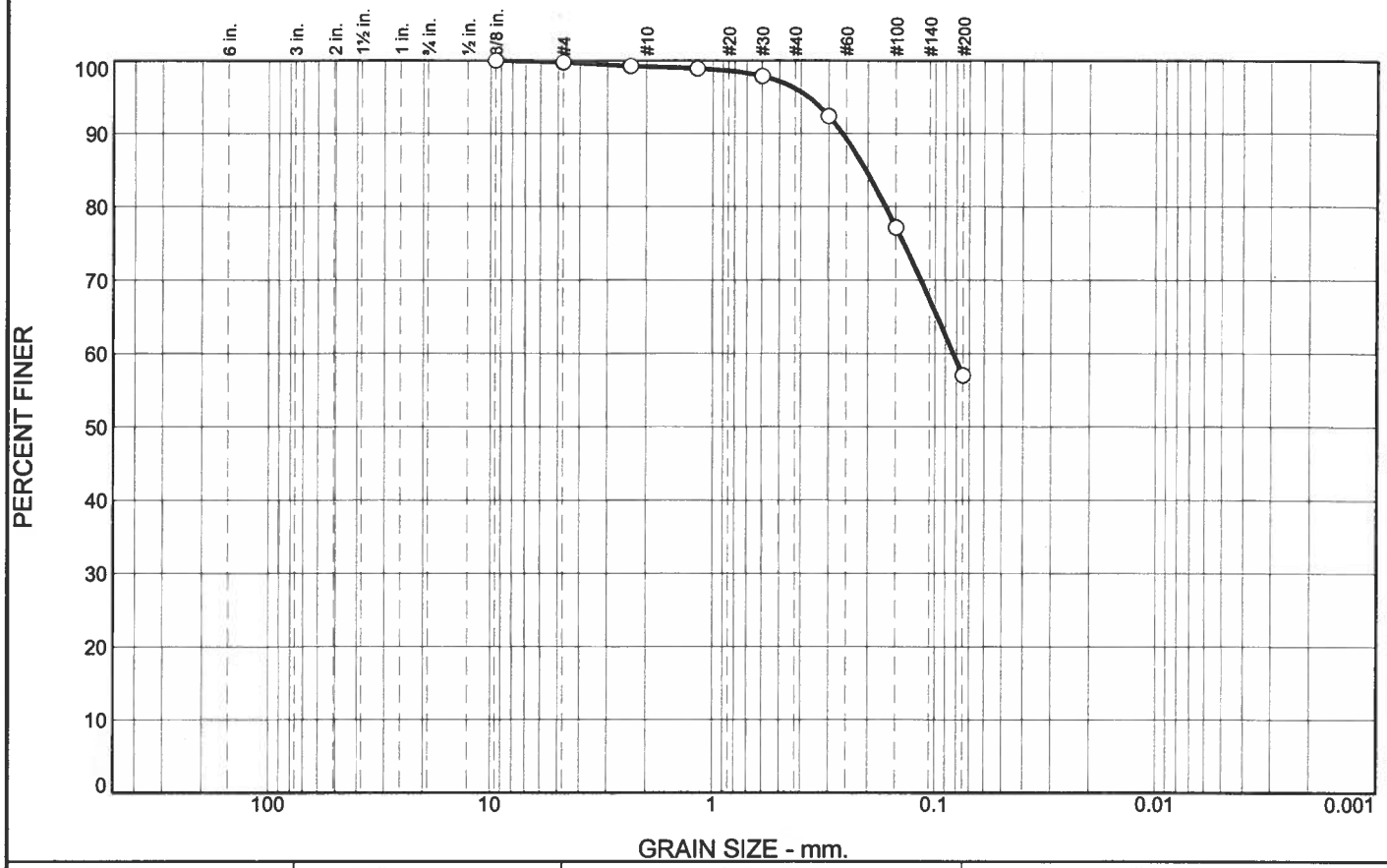
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.3	0.5	3.1	39.1	57.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	99.7		
#8	99.3		
#16	98.9		
#30	97.8		
#50	92.4		
#100	77.2		
#200	57.0		

* (no specification provided)

Material Description

Light Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2598 D_{85}= 0.2044 D_{60}= 0.0828
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B2-14 **Depth:** 75.0'-76.5'

Date: 10-16-09

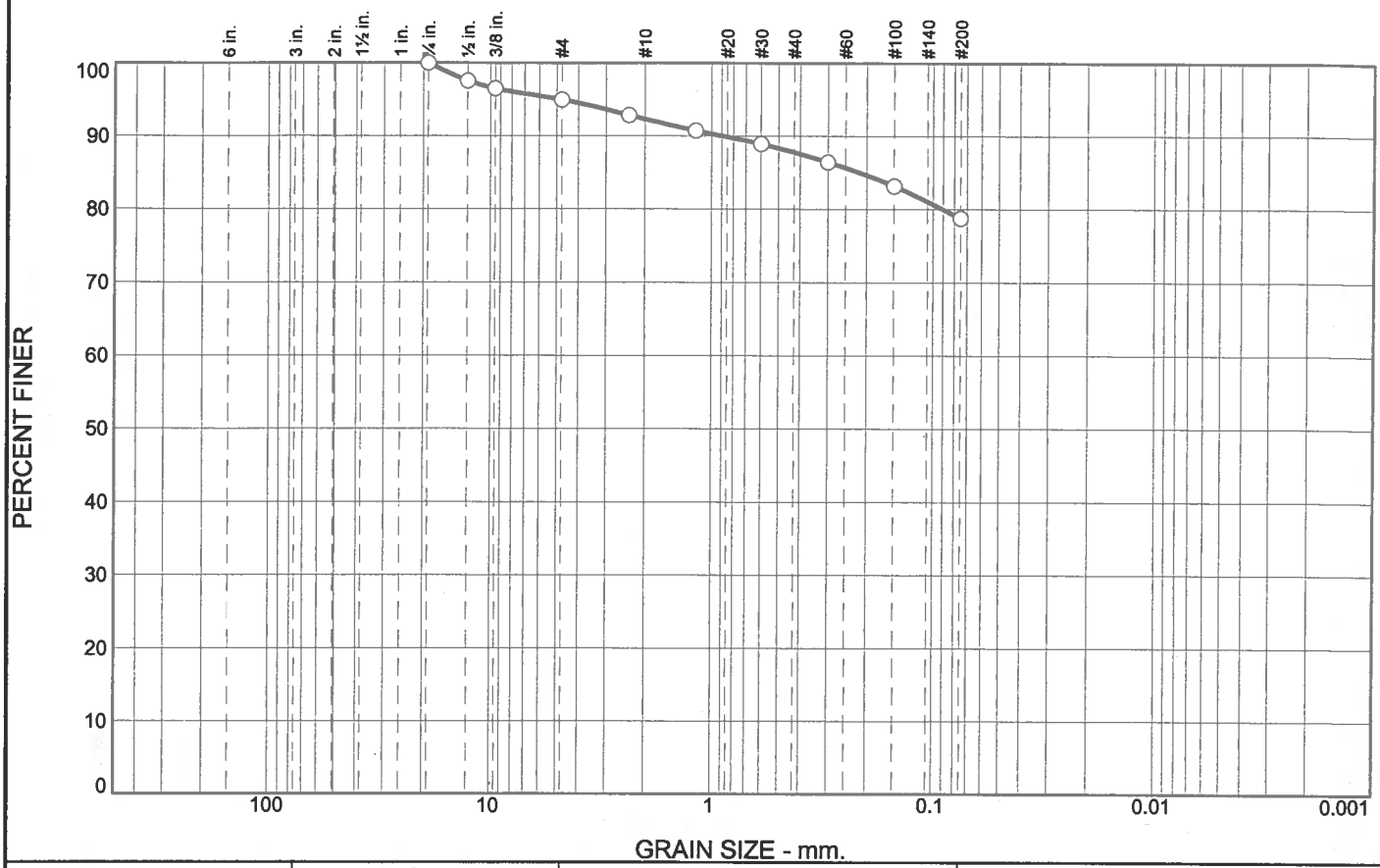
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.0	2.7	4.5	9.0	78.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
1/2"	97.6		
3/8"	96.5		
#4	95.0		
#8	92.8		
#16	90.7		
#30	88.9		
#50	86.4		
#100	83.2		
#200	78.8		

* (no specification provided)

Material Description

Yellowish Brown SILT with Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.8931 D_{85}= 0.2150 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sample Number: B2-17 **Depth:** 91.0'-91.5'

Date: 10-16-09

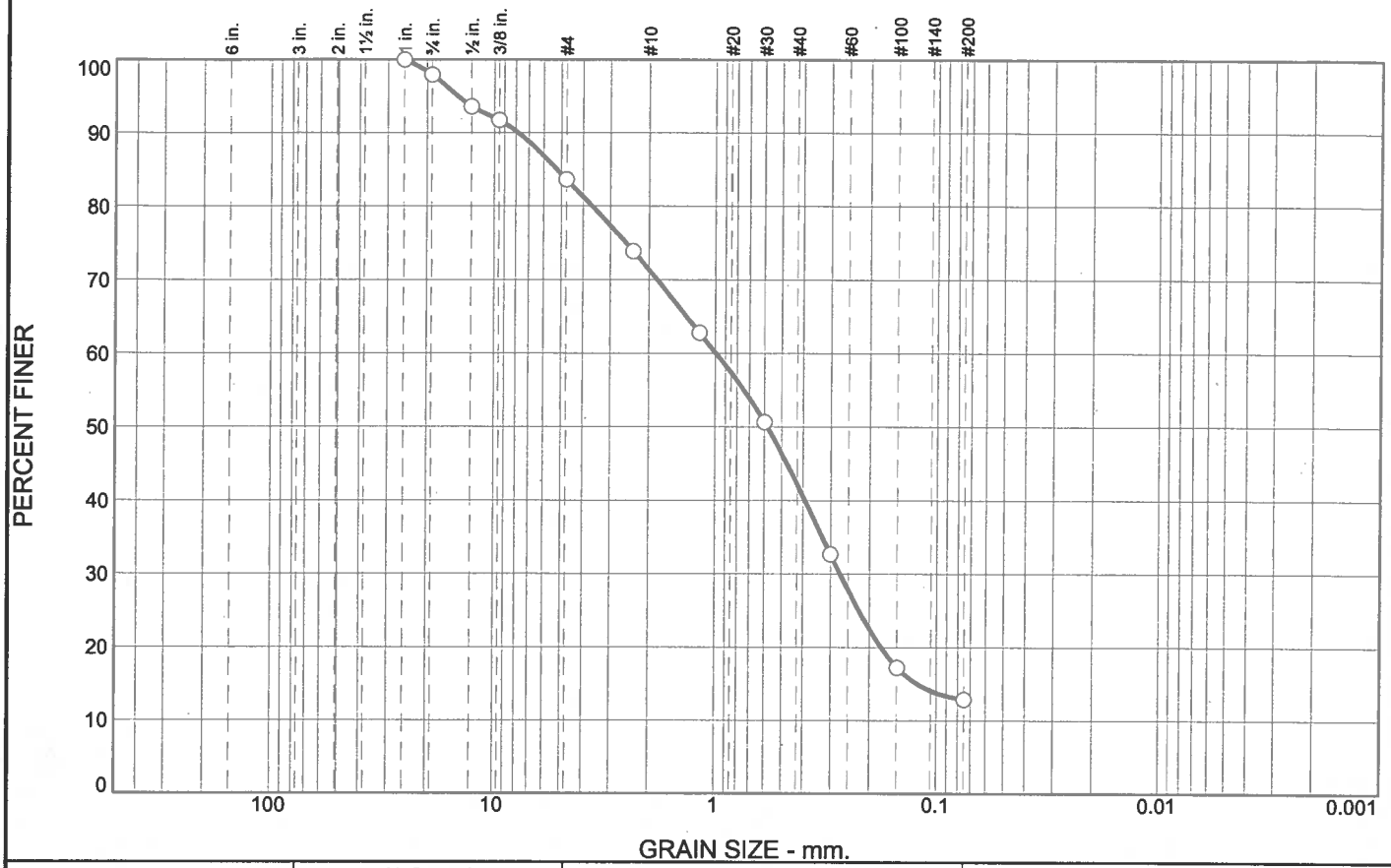
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: ECH **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.1	14.3	12.3	29.2	29.2	12.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0"	100.0		
3/4"	97.9		
1/2"	93.6		
3/8"	91.7		
#4	83.6		
#8	73.9		
#16	62.8		
#30	50.6		
#50	32.7		
#100	17.2		
#200	12.9		

* (no specification provided)

Material Description

Light Olive Brown Poorly Graded SAND with Clay and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.8371 D_{85}= 5.2466 D_{60}= 0.9920
D_{50}= 0.5828 D_{30}= 0.2716 D_{15}= 0.1222
D_{10}= C_u= C_c=

Classification

USCS= SP-SC AASHTO=

Remarks

Sample Number: B3-6 **Depth:** 30.0'-31.5'

Date: 10-21-09

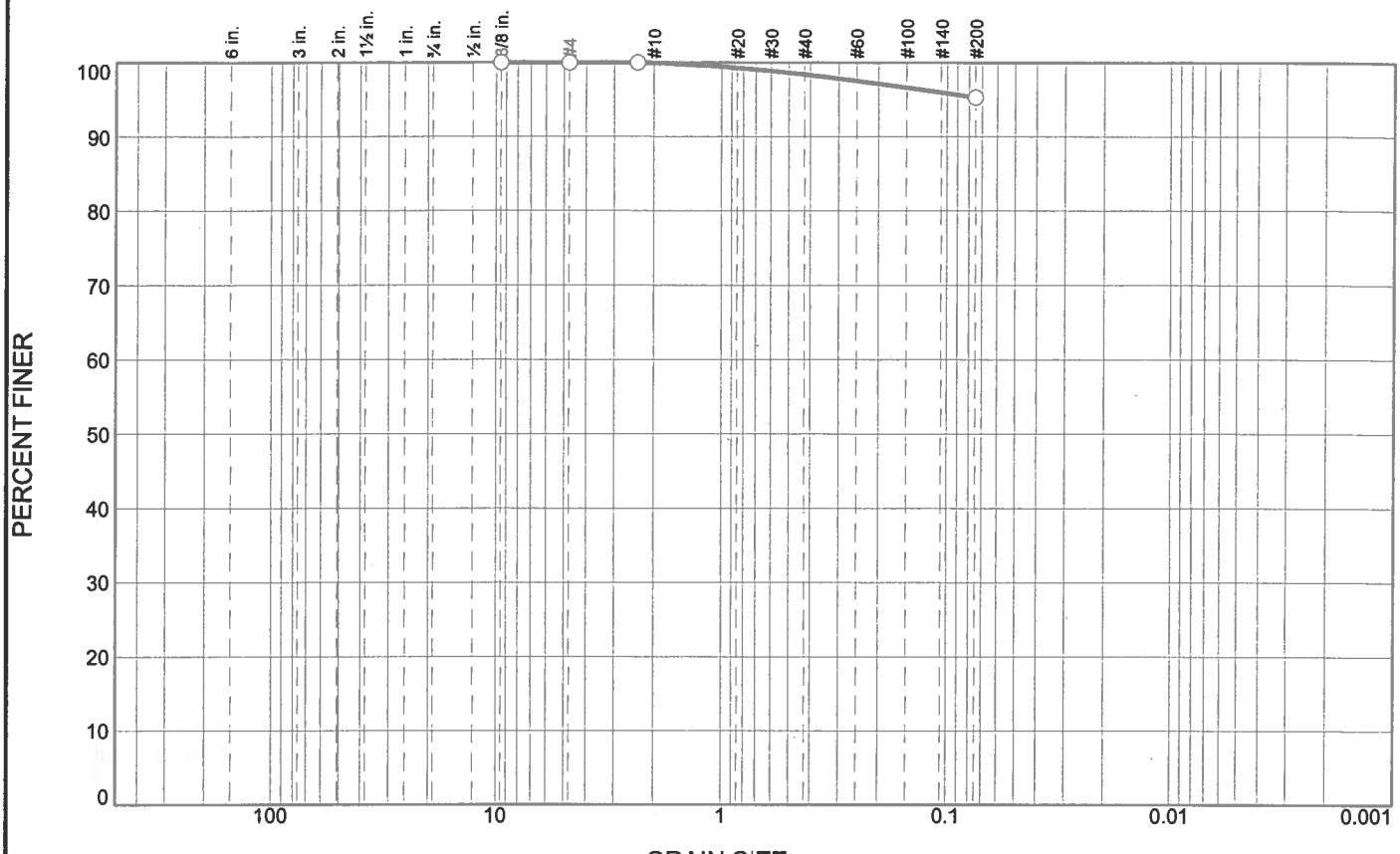
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.5	3.1	95.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8"	100.0		
#4	100.0		
#8	100.0		
#200	95.3		

* (no specification provided)

Material Description

Olive Brown Lean CLAY

Atterberg Limits

PL= 20 LL= 36 PI= 16

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(16)

Remarks

Sample Number: B3-8 **Depth:** 40.0-41.5

Date: 10-21-09

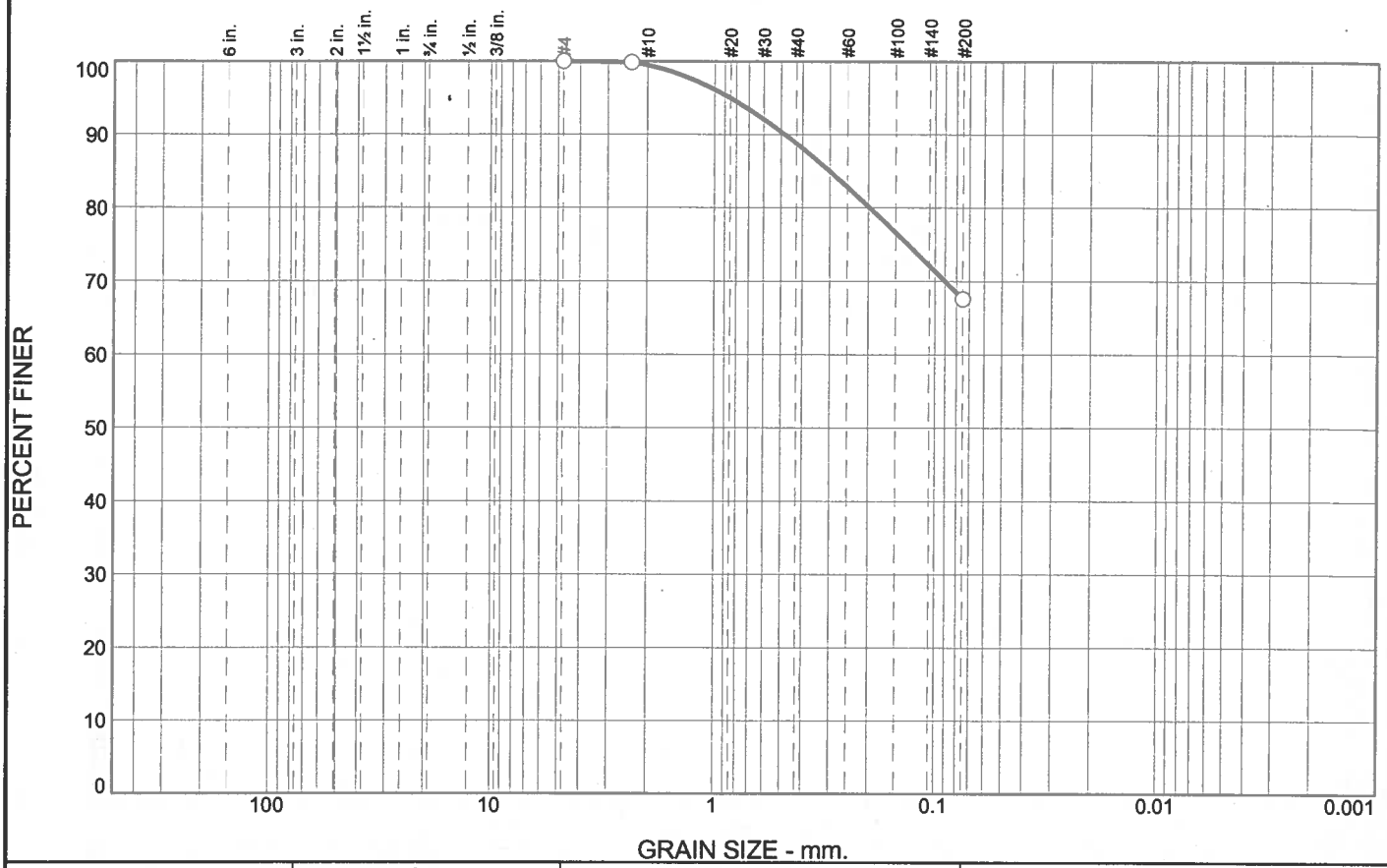
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.5	10.8	21.1	67.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.9		
#200	67.6		

* (no specification provided)

Material Description

Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= 18 LL= 28 PI= 10

Coefficients

D_{90}= 0.4818 D_{85}= 0.3008 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-4(5)

Remarks

Sample Number: B3-15 **Depth:** 75.0'-76.5'

Date: 10-21-09

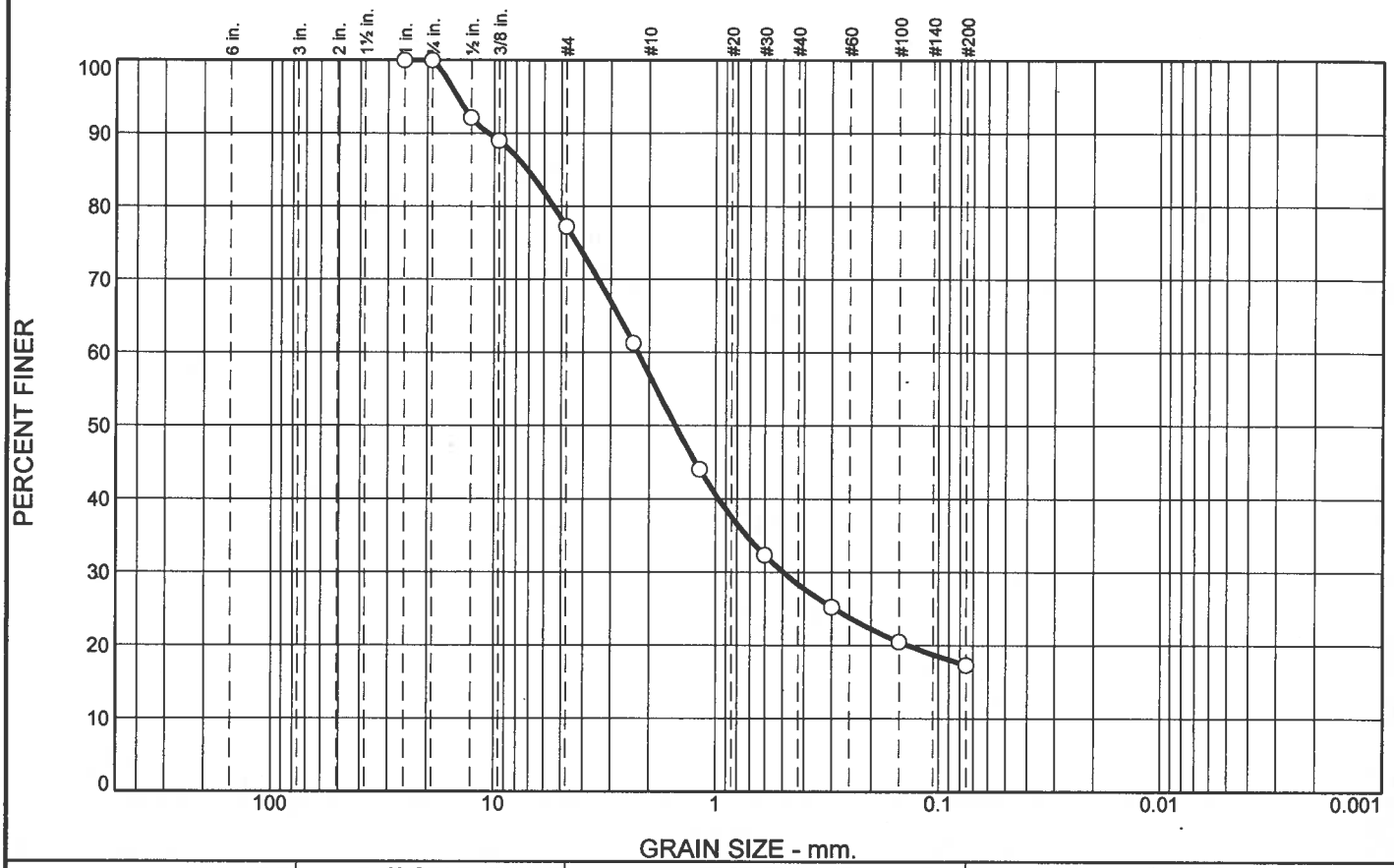
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	22.8	20.1	28.7	11.1	17.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.0"	100.0		
3/4"	100.0		
1/2"	92.1		
3/8"	89.0		
#4	77.2		
#8	61.3		
#16	44.1		
#30	32.4		
#50	25.3		
#100	20.5		
#200	17.3		

* (no specification provided)

Material Description

Olive Brown Clayey SAND with Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.6285 D_{85}= 7.1007 D_{60}= 2.2425
D_{50}= 1.5159 D_{30}= 0.4936 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SC AASHTO=

Remarks

Sample Number: B3-17 **Depth:** 85.0'-86.5'

Date: 10-21-09

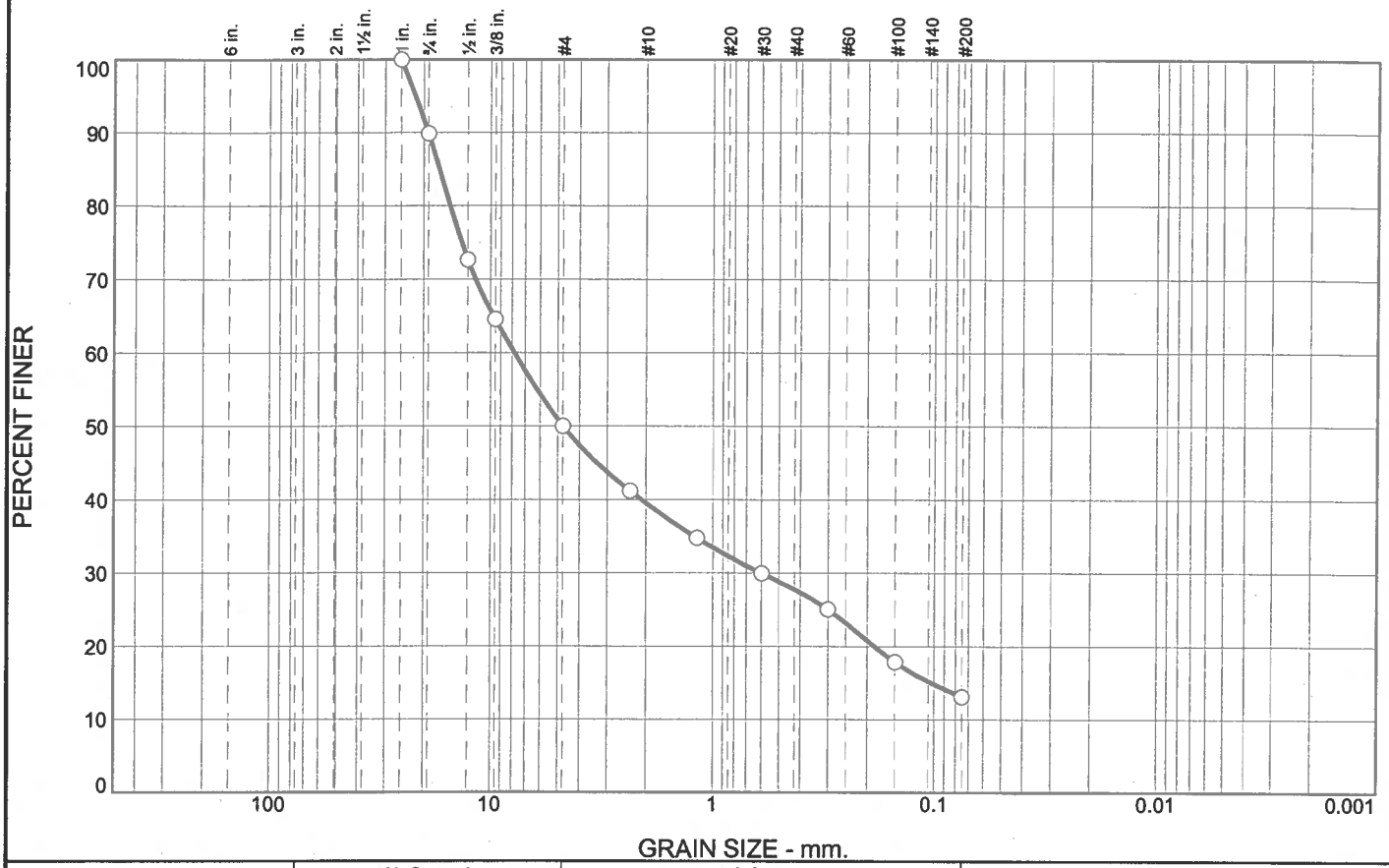
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	10.1	39.9	10.5	11.7	14.7	13.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1"	100.0		
3/4"	89.9		
1/2"	72.6		
3/8"	64.6		
#4	50.0		
#8	41.2		
#16	34.8		
#30	30.0		
#50	25.0		
#100	17.9		
#200	13.1		

* (no specification provided)

Material Description

Brown Clayey GRAVEL with Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 19.1172 D_{85}= 17.0029 D_{60}= 7.8358
D_{50}= 4.7425 D_{30}= 0.6045 D_{15}= 0.1031
D_{10}= C_u= C_c=

Classification

USCS= GC AASHTO=

Remarks

Sample Number: B4-2 **Depth:** 15.0'-16.5

Date: 10-21-09

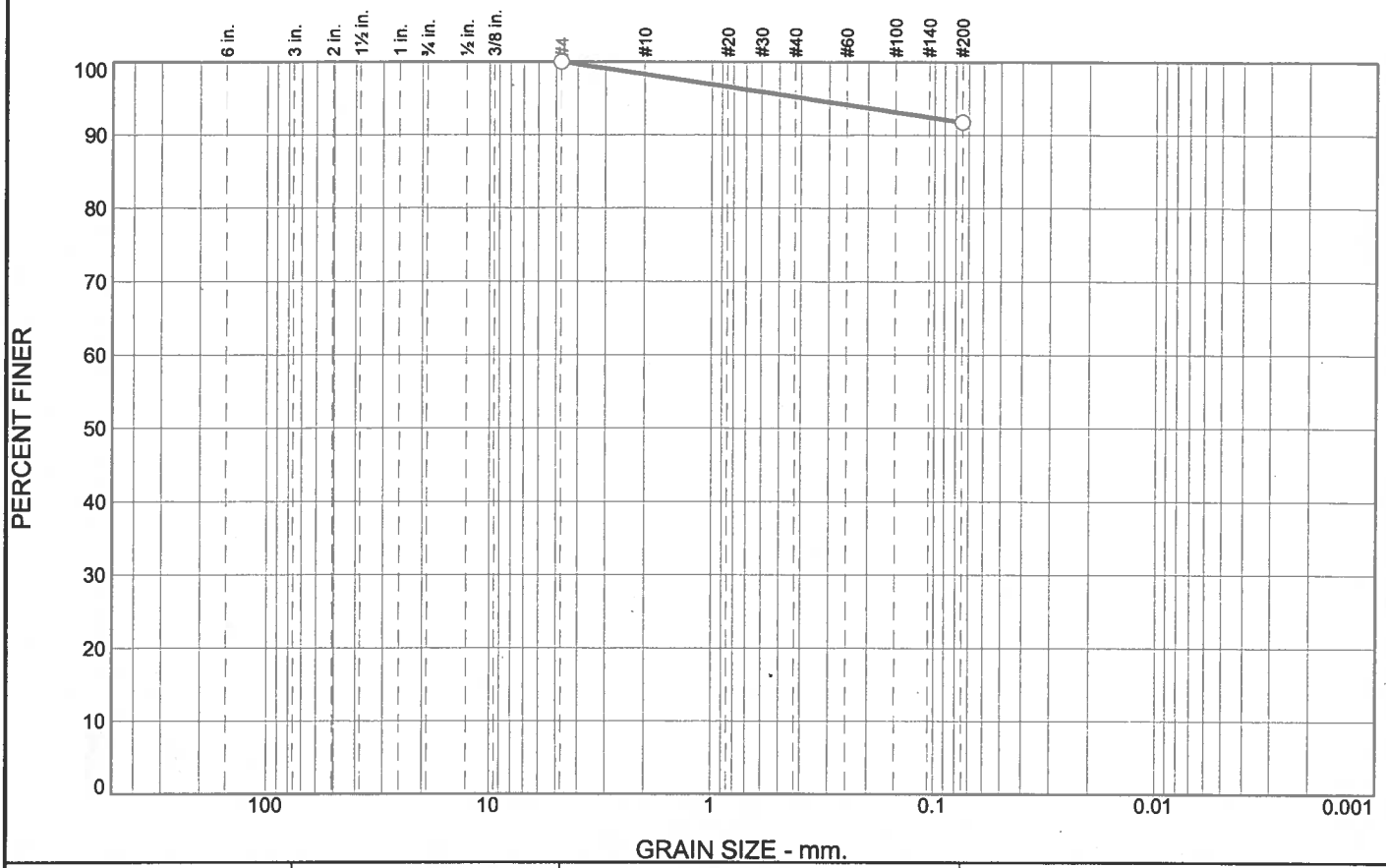
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.7	3.1	3.4	91.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#200	91.8		

* (no specification provided)

Material Description

Olive Brown Lean CLAY

Atterberg Limits

PL= 22 LL= 38 PI= 16

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(15)

Remarks

Sample Number: B4-6 **Depth:** 35.0'-36.5'

Date: 10-22-09

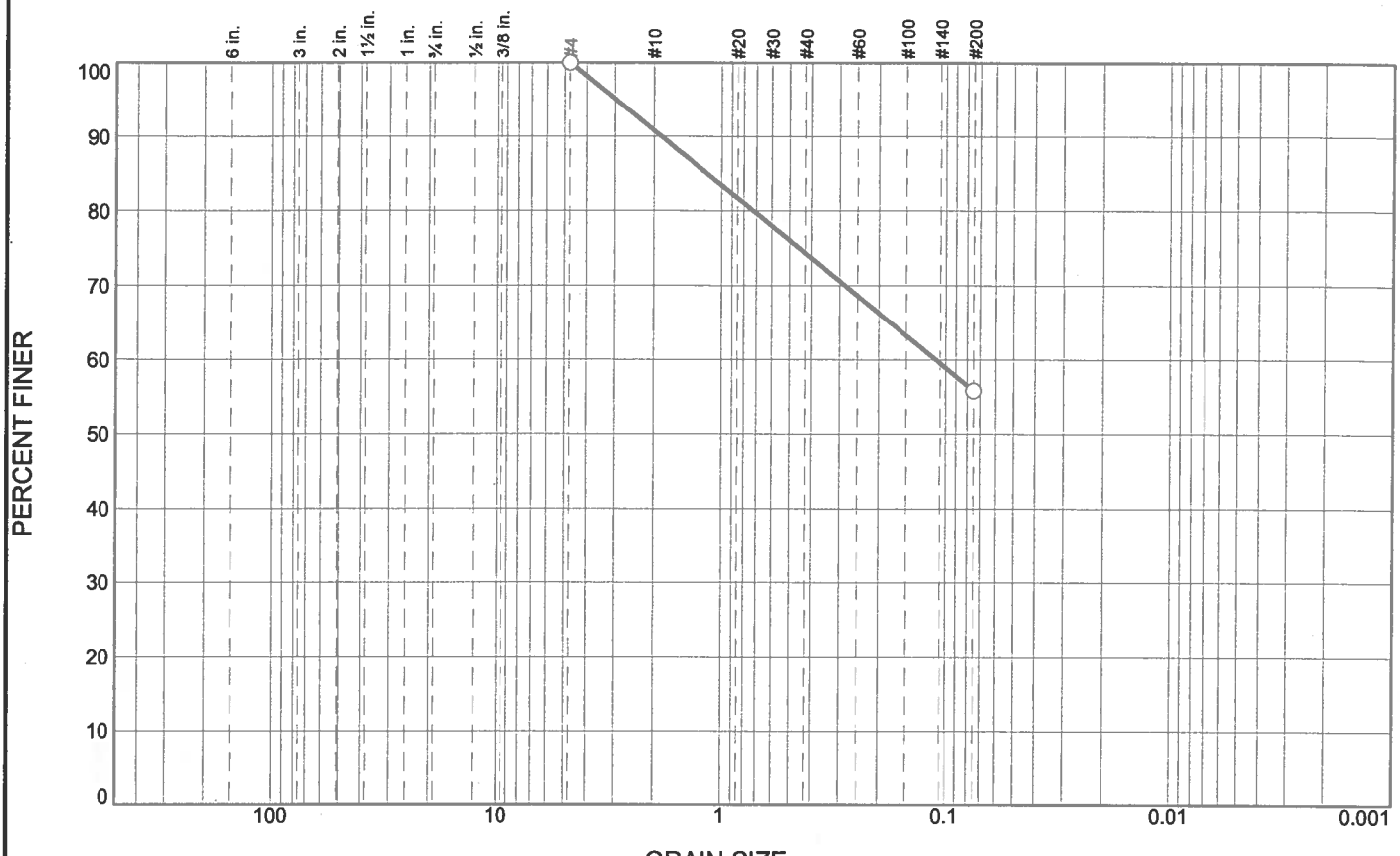
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	9.2	16.5	18.5	55.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#200	55.8		

* (no specification provided)

Material Description

Olive Brown Sandy Lean CLAY

Atterberg Limits

PL= 15 LL= 26 PI= 11

Coefficients

D_{90}= 1.8569 D_{85}= 1.1610 D_{60}= 0.1109
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(3)

Remarks

Sample Number: B4-12 **Depth:** 65.0'-66.5'

Date: 10-22-09

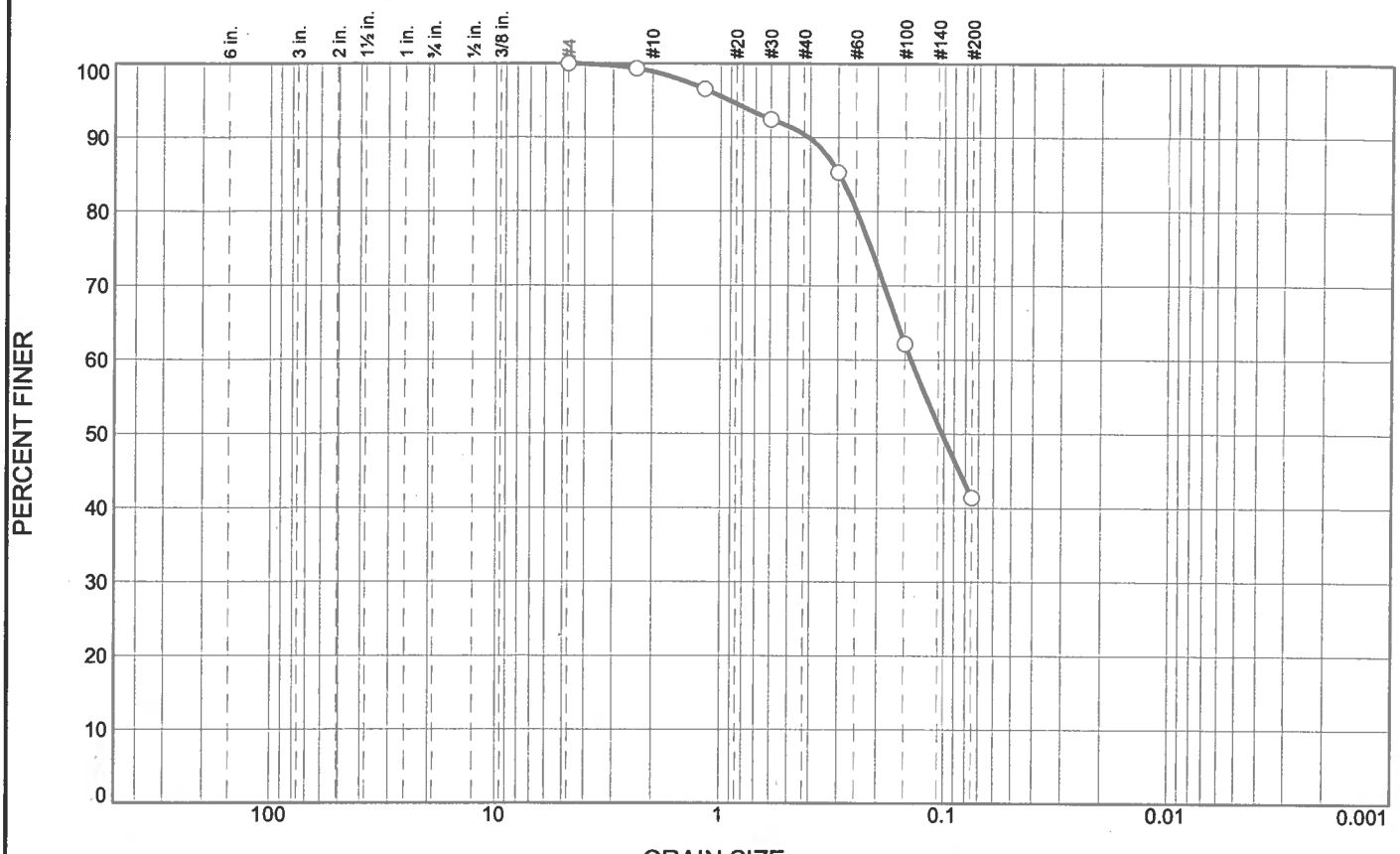
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.1	8.5	49.0	41.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#8	99.3		
#16	96.5		
#30	92.4		
#50	85.3		
#100	62.2		
#200	41.4		

* (no specification provided)

Material Description

Brown Silty SAND

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.4072 D_{85}= 0.2963 D_{60}= 0.1408
D_{50}= 0.1020 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Sample Number: B4-15 **Depth:** 80.0'-81.5'

Date: 10-22-09

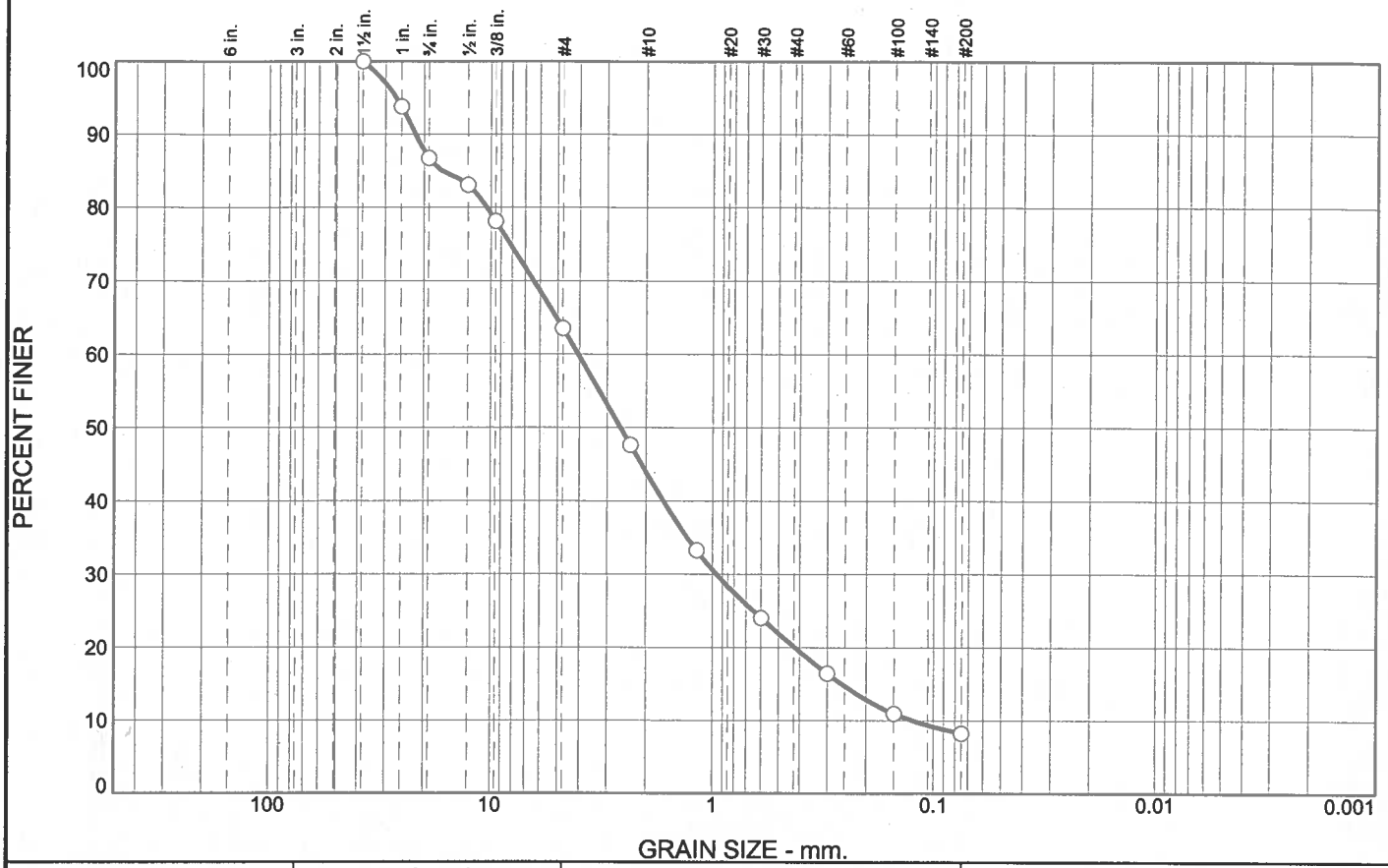
Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	13.3	23.2	19.6	23.8	11.9	8.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.5"	100.0		
1.0"	93.8		
3/4"	86.7		
1/2"	83.1		
3/8"	78.1		
#4	63.5		
#8	47.7		
#16	33.3		
#30	24.0		
#50	16.4		
#100	10.9		
#200	8.2		

* (no specification provided)

Material Description

Brown Well Graded SAND with Silt and Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 21.9455 D_{85}= 16.5922 D_{60}= 4.0526
D_{50}= 2.6135 D_{30}= 0.9594 D_{15}= 0.2580
D_{10}= 0.1246 C_u= 32.52 C_c= 1.82

Classification

USCS= SW-SM AASHTO=

Remarks

Sample Number: B4-16 **Depth:** 85.25'-85.75'

Date: 10-23-09

Blackburn Consulting

Auburn, CA

Client: HDR Inc.
Project: SR 99 @ Thomes Creek
Project No: 1274.1 **Figure**

Tested By: KLC **Checked By:** KLC

Unconfined Compression Test
ASTM D 2166-00

Project Name	SR 99 @ Thomes Creek		
Project Number	1274.1		
Sample	B1-7c	Depth	36.0-36.5
Sample Description	Olive Brown Silty CLAY (CL-ML)		
Date	10/14/2009		
Tested By:	KISB		

Original Sample Length	5.46
Original Diameter (in)	2.37
Sample Area (in^2)	4.43
Sample Weight (g)	800.80

axial strain	**5.0%**
Average cross-sectional area (in^2)	**4.66**
Average cross-sectional area (ft^2)	0.032
Peak Reading	0.295
Maximum Load(lb)	39
Compressive Strength (tsf)	**0.60**

Moisture Density

Wet Sample Weight (g)	903.0
Tare Number	NN
Tare Weight (g)	103.8
Dry Sample Weight (g)	737.8
Dry Weight (g)	634.0
Water Weight (g)	165.2
Percent Moisture (%)*	26.1
Wet Density (pcf)	126.2
Dry Density (pcf)	100.1

Remarks:

* % moisture taken after test.

Compression Tests

Dial reading @ 0 lb	0.021

Rate of Strain=0.056in/min

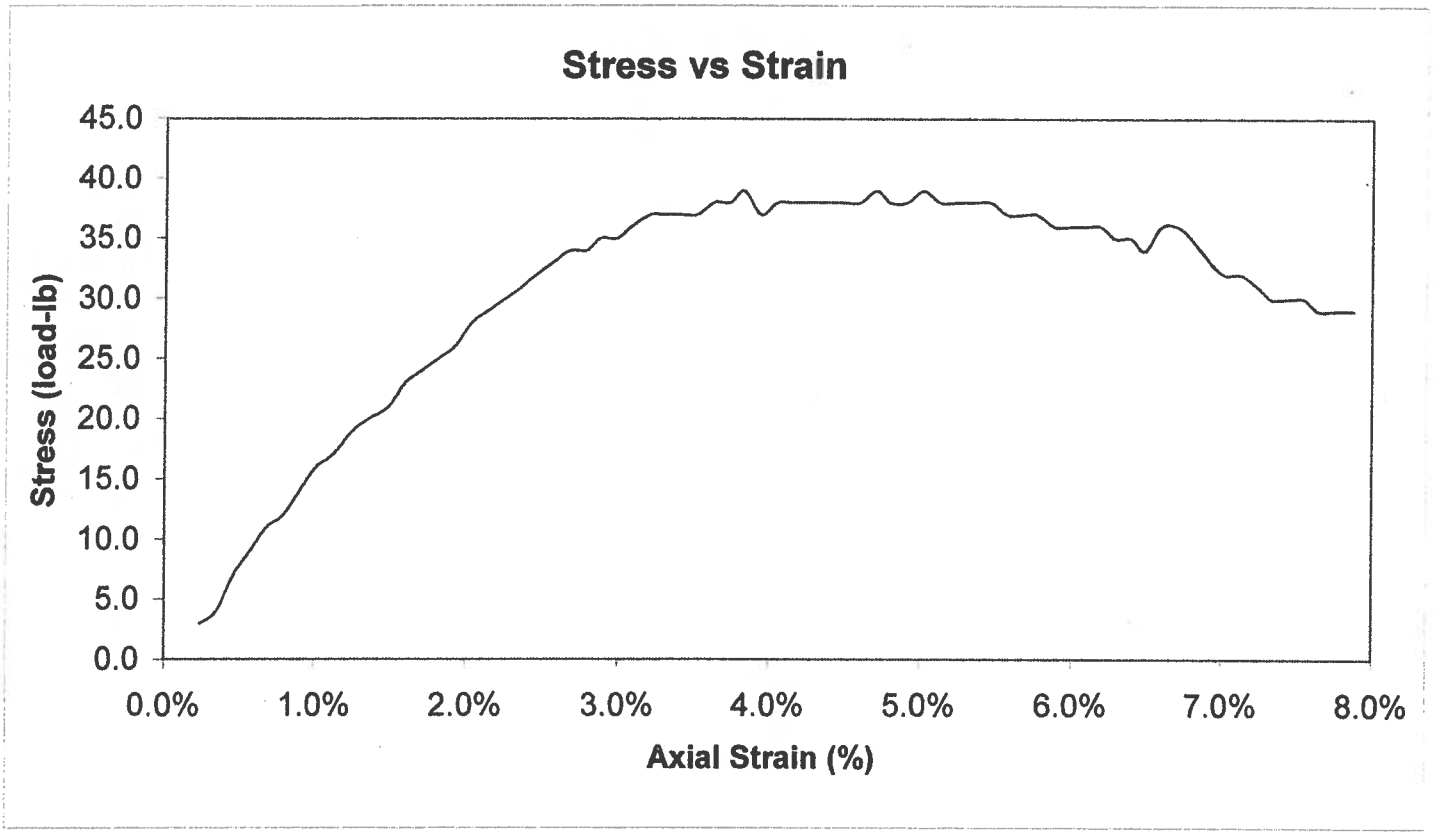
Unconfined Compression Test Readings

Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb	Dial Reading	Lb
0.034	3.000	0.132	28.000	0.225	38.000	0.313	38.000	0.404	32.000
0.040	4.000	0.138	29.000	0.230	39.000	0.319	38.000	0.410	32.000
0.046	7.000	0.144	30.000	0.236	37.000	0.325	37.000	0.416	31.000
0.052	9.000	0.150	31.000	0.242	38.000	0.331	37.000	0.421	30.000
0.058	11.000	0.155	32.000	0.248	38.000	0.336	37.000	0.427	30.000
0.064	12.000	0.161	33.000	0.255	38.000	0.342	36.000	0.433	30.000
0.070	14.000	0.167	34.000	0.260	38.000	0.348	36.000	0.438	29.000
0.076	16.000	0.173	34.000	0.266	38.000	0.353	36.000	0.444	29.000
0.082	17.000	0.178	35.000	0.272	38.000	0.359	36.000	0.450	29.000
0.089	19.000	0.184	35.000	0.278	39.000	0.364	35.000	0.451	29.000
0.095	20.000	0.189	36.000	0.283	38.000	0.370	35.000		
0.102	21.000	0.196	37.000	0.289	38.000	0.375	34.000		
0.108	23.000	0.201	37.000	0.295	39.000	0.381	36.000		
0.114	24.000	0.207	37.000	0.301	38.000	0.387	36.000		
0.120	25.000	0.213	37.000	0.307	38.000	0.392	35.000		
0.126	26.000	0.219	38.000	0.313	38.000	0.399	33.000		

blackburn consulting

Project
SR 99 @ Thomes Creek
Project Number
1274.1
Sample Number
B1-7c
Material Description
Olive Brown Silty CLAY (CL-ML)
Tested By
KISB

ASTM D 2166-00



Wet Density (pcf)	126.2
Dry Density (pcf)	100.1
% Moisture	26.1

Unconfined Compressive Strength (tsf) 0.6

Triaxial Compression UU no pore pressure
ASTM D 4767

blackburn consulting

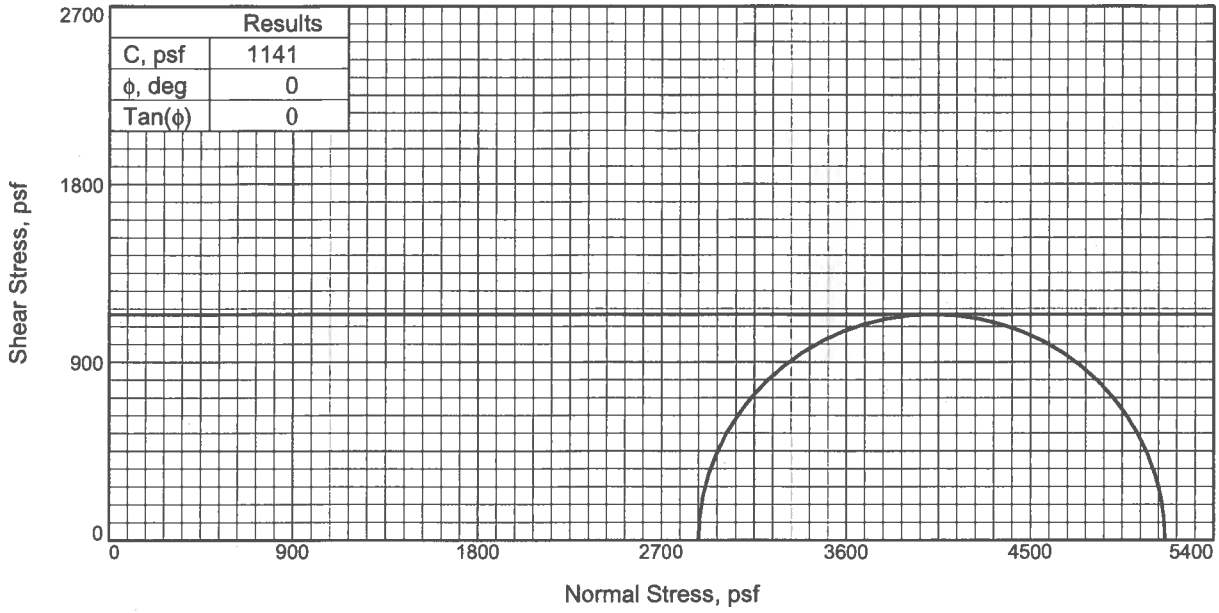
Project Name: SR 99 @ Thomes Creek
Project Number: 1274.1
Sample ID: B2-6b **Depth (ft):** 35.5-36.0
Sample Description: Light Yellowish Brown Fat CLAY (CH)
Date: 10/15/09 **Tested By:** KISB

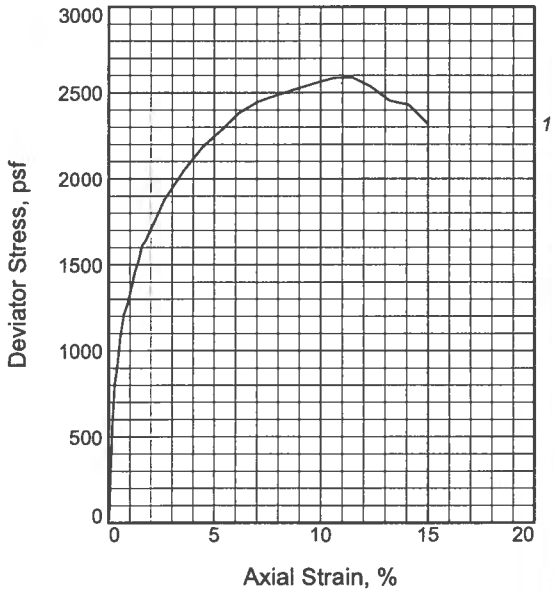
UU	X	
CU		Staged

Specimen			
Initial		Final	
Tare No.	B2	Tare No.	PP
Tare =	46.5	Tare =	105.3
Wet Soil + Tare =	73.9	Wet Soil + Tare =	893.1
Dry Soil + Tare =	66.1	Dry Soil + Tare =	665.5
Water Content (%)	39.8%	Water Content (%)	40.6%

Length of Sample =	**5.693**	**in**	**5%**	**0.28**
Diameter of Sample =	**2.387**	**in**	**10%**	**0.57**
Area=	**4.48**	**in^3**	**15%**	**0.85**
Weight of Sample =	**786.90**	**gm**		
Specific Gravity=	**2.70**			
Initial Wet Density =	**117.4**	**pcf**		
Initial Dry Density =	**84.0**	**pcf**		
Desired Cell Pressure=	**20**	**psi**		
Disp. after CP Applied=	**0.014**	**in.**		

NOTES:





Specimen No.		1
Initial	Water Content, %	40.6
	Dry Density, pcf	83.7
	Saturation, %	99.8
	Void Ratio	1.2010
	Diameter, in.	2.387
	Height, in.	5.693
At Test	Water Content, %	40.6
	Dry Density, pcf	83.8
	Saturation, %	100.0
	Void Ratio	1.1985
	Diameter, in.	2.389
	Height, in.	5.679
Strain rate, in./min.		0.005
Back Pressure, psf		0
Cell Pressure, psf		2880
Fail. Stress, psf		2282
Ult. Stress, psf		2323
σ_1 Failure, psf		5162
σ_3 Failure, psf		2880

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Light Yellowish Brown Fat CLAY (CH)
LL= 66 **PL=** 28 **PI=** 38
Assumed Specific Gravity= 2.95
Remarks: Failure chosen at 5% Strain

Figure _____

Client: HDR

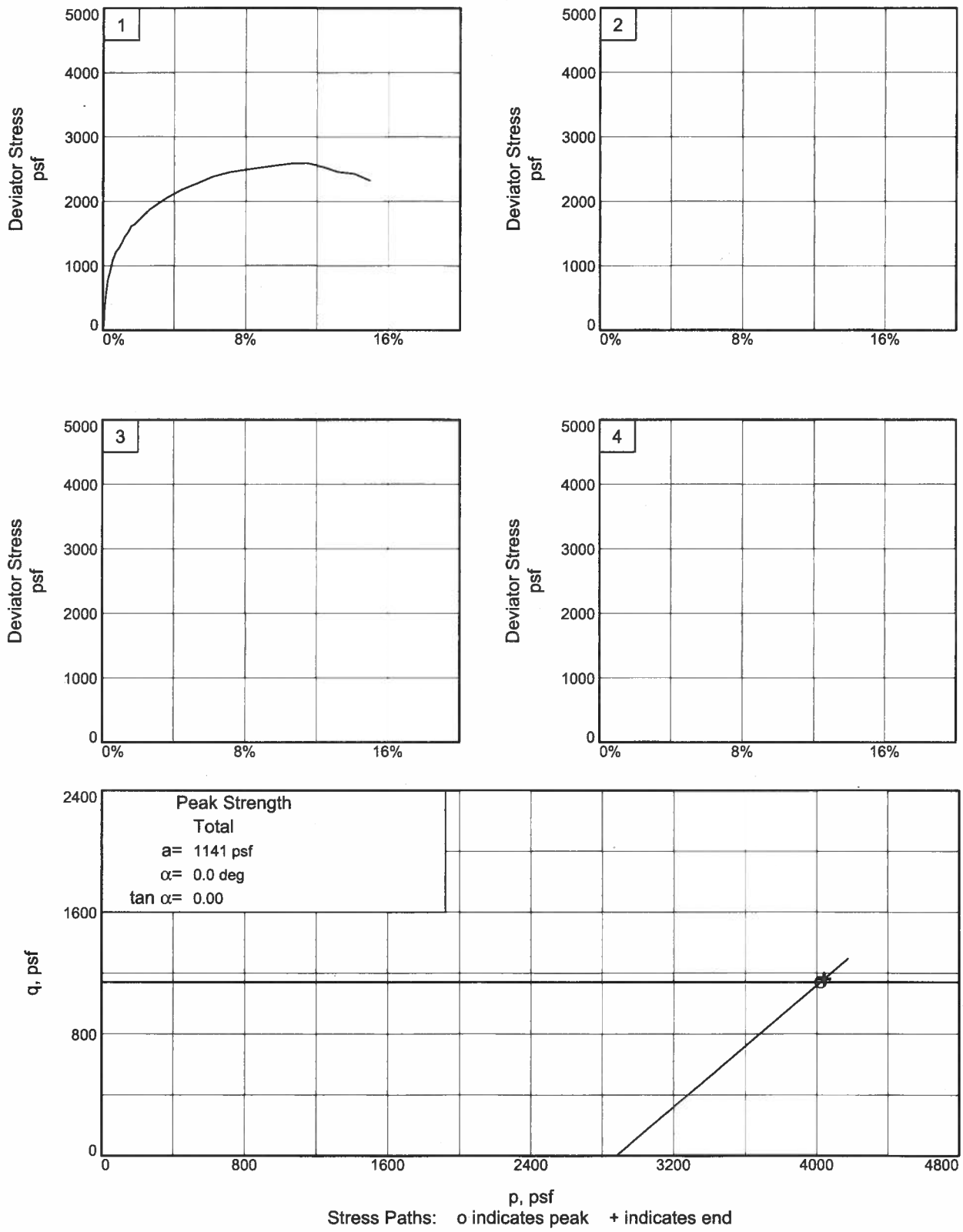
Project: SR 99 @ Thomes Creek

Source of Sample: B2 **Depth:** 35.5-36.0
Sample Number: 6b
Proj. No.: 1274.1 **Date Sampled:** 10-8-09

TRIAXIAL SHEAR TEST REPORT

Blackburn Consulting

Tested By: KISB **Checked By:** RBL



Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B2 **Depth:** 35.5-36.0 **Sample Number:** 6b
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

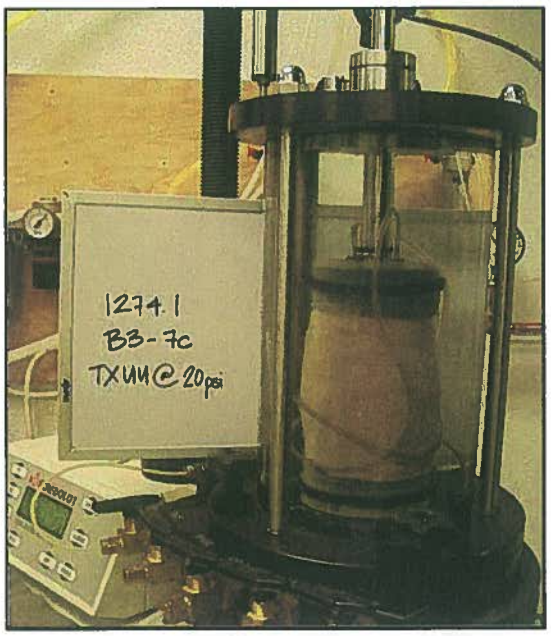
Triaxial Compression UU no pore pressure
ASTM D 4767

blackburn consulting

Project Name: SR 99 @ Thomes Creek
Project Number: 1274.1
Sample ID: B3-7c **Depth (ft):** 36.0-36.5
Sample Description: Olive Brown Lean CLAY (CL)
Date: 11/2/09 **Tested By:** KISB

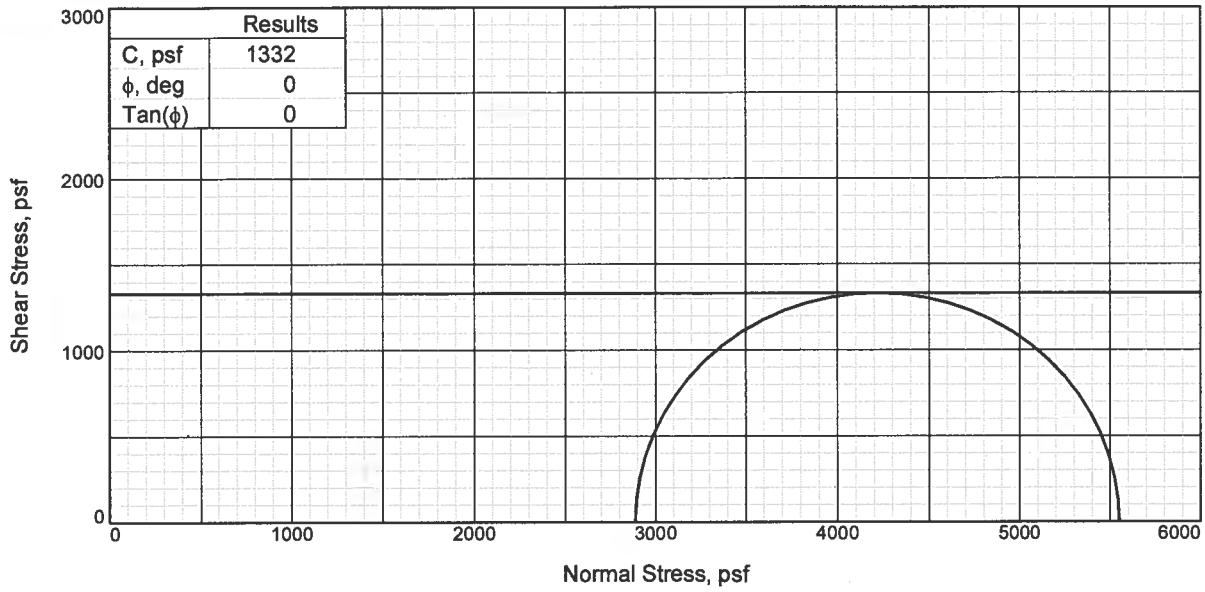
UU	X	
CU		Staged

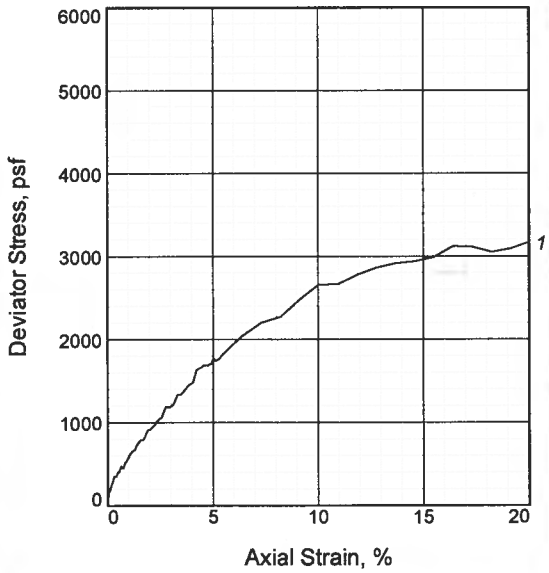
Specimen			
Initial		Final	
Tare No.	B2	Tare No.	VV
Tare =	46.5	Tare =	105.0
Wet Soil + Tare =	111.1	Wet Soil + Tare =	944.3
Dry Soil + Tare =	98.7	Dry Soil + Tare =	791.7
Water Content (%)	23.8%	Water Content (%)	22.2%



Length of Sample =	5.507	in	5%	0.28
Diameter of Sample =	2.382	in	10%	0.55
Area=	4.46	in^3	15%	0.83
Weight of Sample =	839.00	gm		
Specific Gravity=	2.70			
Initial Wet Density =	130.0	pcf		
Initial Dry Density =	105.0	pcf		
Actual Cell Pressure=	20.05	psi		
Disp. after CP Applied=	0.021	in.		

NOTES:





Specimen No.		1
Initial	Water Content, %	22.2
	Dry Density, pcf	106.6
	Saturation, %	100.0
	Void Ratio	0.6111
	Diameter, in.	2.382
	Height, in.	5.507
At Test	Water Content, %	22.2
	Dry Density, pcf	106.6
	Saturation, %	100.0
	Void Ratio	0.6111
	Diameter, in.	2.387
	Height, in.	5.486
Strain rate, in./min.		0.467
Back Pressure, psf		0
Cell Pressure, psf		2887
Fail. Stress, psf		2664
Strain, %		10.0
Ult. Stress, psf		3169
Strain, %		
σ_1 Failure, psf		5551
σ_3 Failure, psf		2887

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Olive Brown Lean CLAY (CL)

Assumed Specific Gravity= 2.75
Remarks: Failure chosen at 10% Strain
Ultimate at 20% Strain

Figure _____

Client: HDR

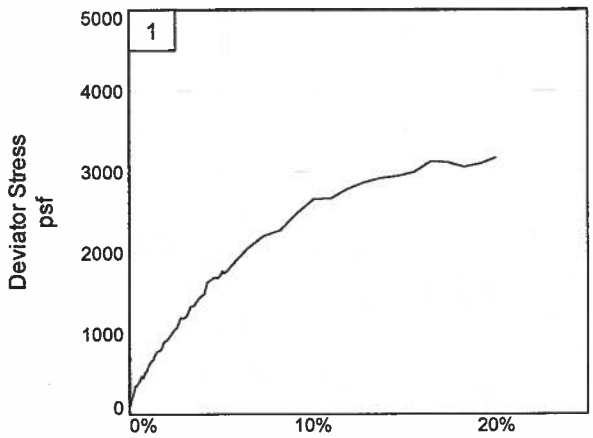
Project: SR 99 @ Thomes Creek

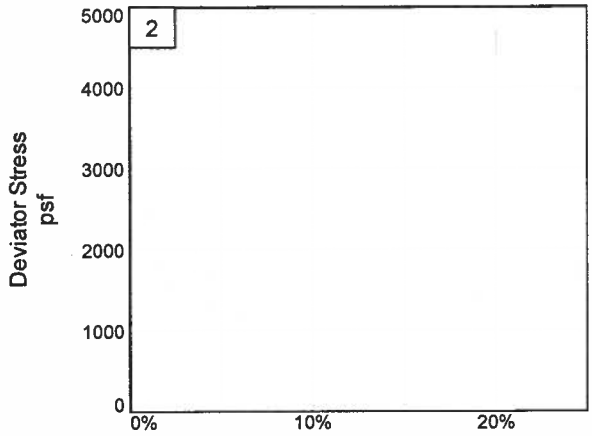
Source of Sample: B3 **Depth:** 36.0-36.5
Sample Number: 7c
Proj. No.: 1274.1 **Date Sampled:**

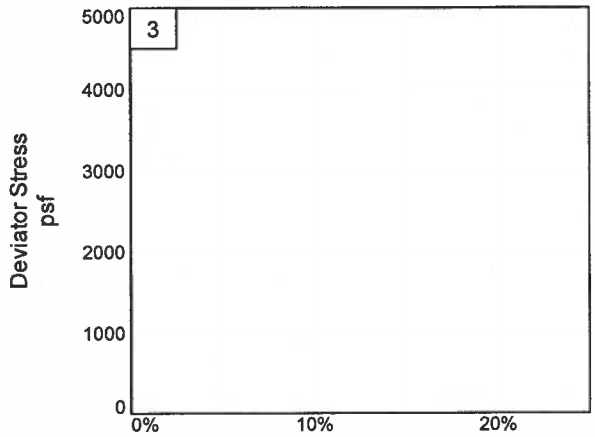
TRIAXIAL SHEAR TEST REPORT

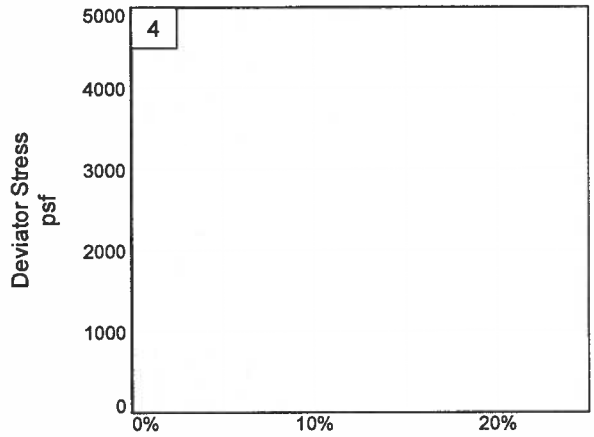
Blackburn Consulting

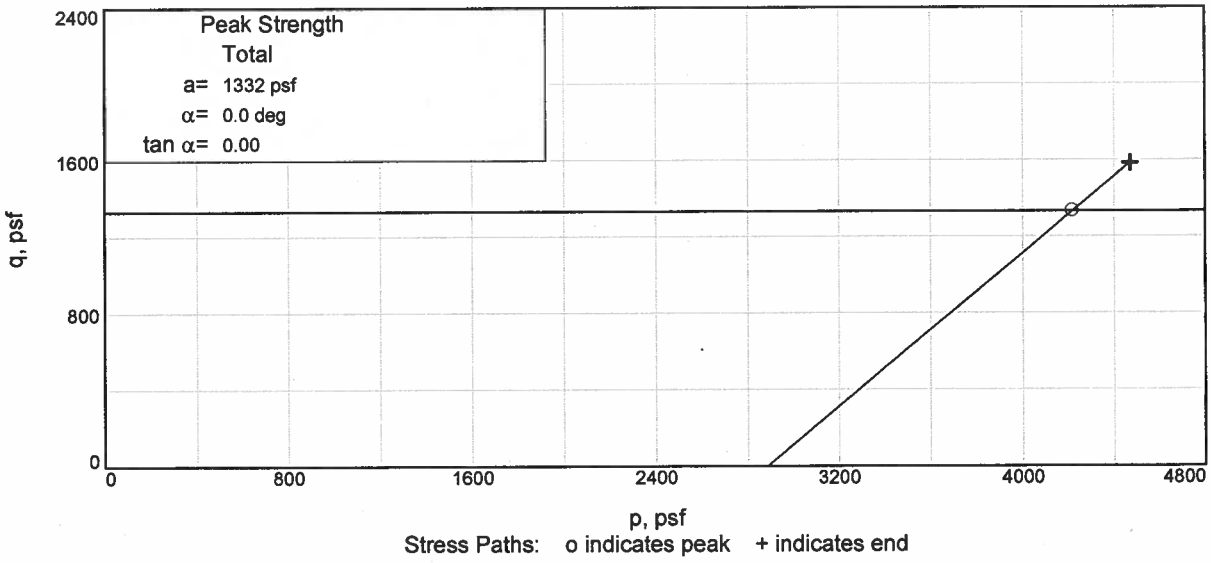
Tested By: KISB **Checked By:** RBL











Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B3 **Depth:** 36.0-36.5 **Sample Number:** 7c
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

Triaxial Compression UU no pore pressure
ASTM D 4767

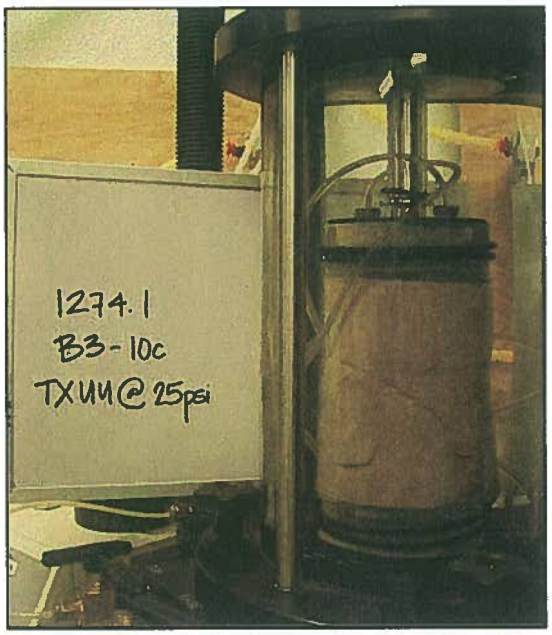
blackburn consulting

Project Name: SR 99 @ Thomes Creek
Project Number: 1274.1
Sample ID: B3-10c **Depth (ft):** 51.0-51.5
Sample Description: Light Yellowish Brown Lean CLAY (CL)
Date: 11/2/09 **Tested By:** KISB

UU	X
CU	

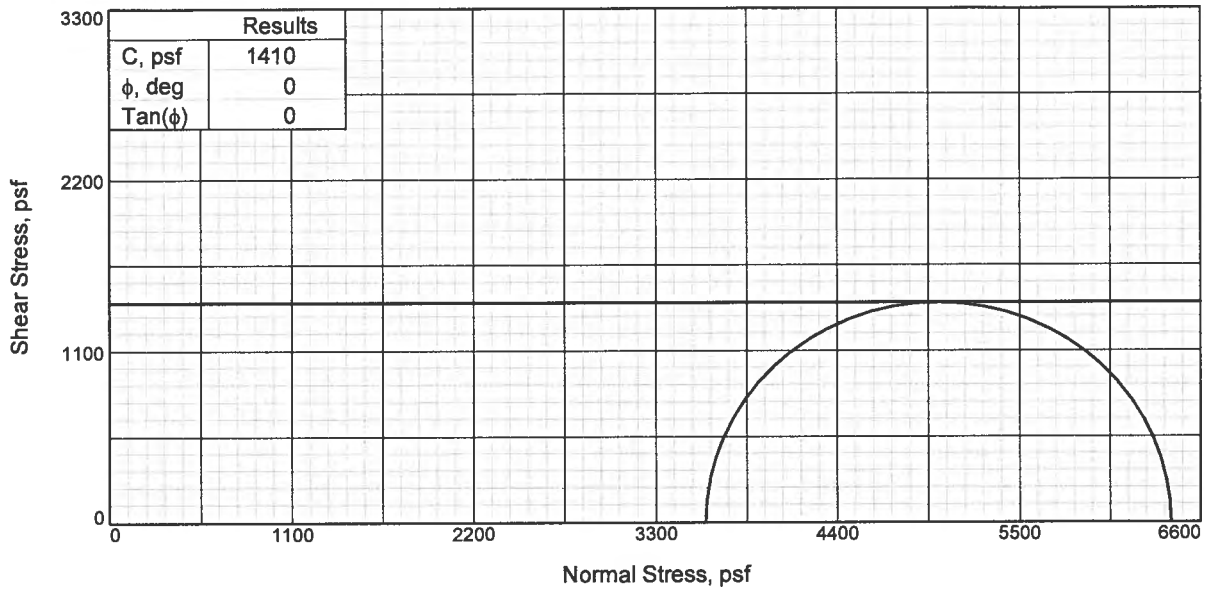
Staged

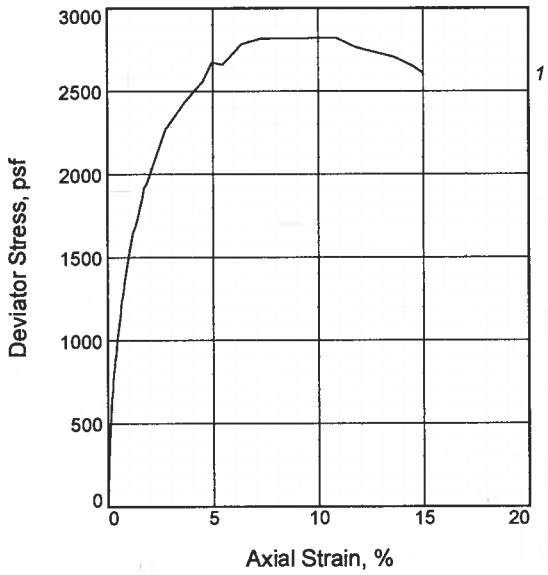
Specimen			
Initial		Final	
Tare No.	B4	Tare No.	NN
Tare =	44.6	Tare =	103.8
Wet Soil + Tare =	76.7	Wet Soil + Tare =	916.4
Dry Soil + Tare =	69.3	Dry Soil + Tare =	747.1
Water Content (%)	30.0%	Water Content (%)	26.3%



Length of Sample =	5.539	in	5%	0.28
Diameter of Sample =	2.395	in	10%	0.55
Area=	4.51	in^3	15%	0.83
Weight of Sample =	811.70	gm		
Specific Gravity=	2.70			
Initial Wet Density =	123.6	pcf		
Initial Dry Density =	95.1	pcf		
Acutal Cell Pressure=	25.00	psi		
Disp. after CP Applied=	0.024	in.		

NOTES:





Specimen No.		1
Initial	Water Content, %	26.3
	Dry Density, pcf	98.1
	Saturation, %	98.9
	Void Ratio	0.7182
	Diameter, in.	2.395
	Height, in.	5.539
At Test	Water Content, %	26.3
	Dry Density, pcf	98.5
	Saturation, %	100.0
	Void Ratio	0.7106
	Diameter, in.	2.395
	Height, in.	5.515
Strain rate, in./min.		0.463
Back Pressure, psf		0
Cell Pressure, psf		3600
Fail. Stress, psf		2820
Strain, %		10.0
Ult. Stress, psf		2609
Strain, %		
σ_1 Failure, psf		6420
σ_3 Failure, psf		3600

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Light Yellowish Brown Lean CLAY (CL)

Assumed Specific Gravity= 2.70
Remarks: Failure chosen at 10% Strain
Ultimate at 15% Strain

Figure _____

Client: HDR

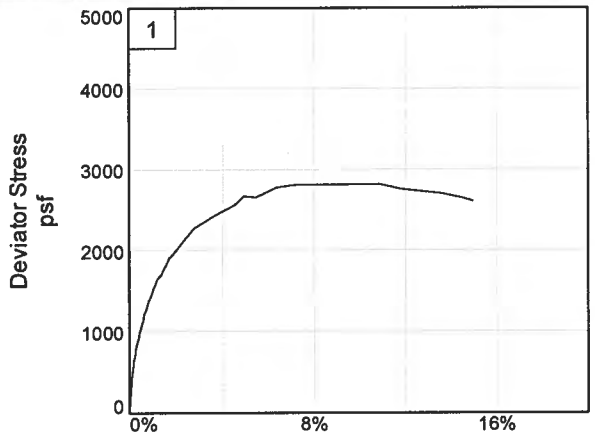
Project: SR 99 @ Thomes Creek

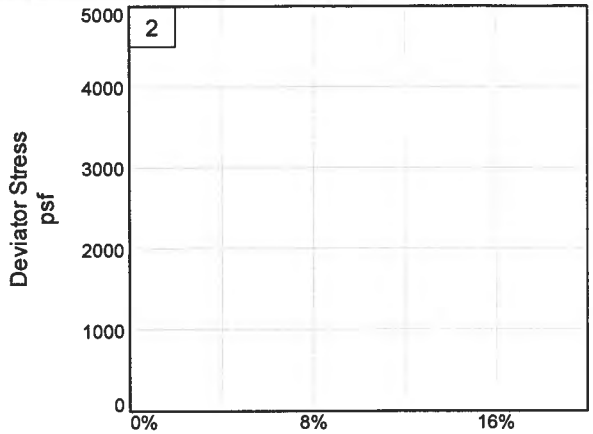
Source of Sample: B3 **Depth:** 51.0-51.5
Sample Number: 10c
Proj. No.: 1274.1 **Date Sampled:**

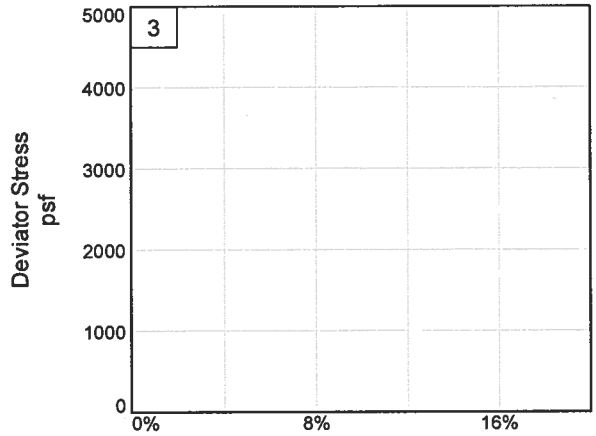
TRIAXIAL SHEAR TEST REPORT

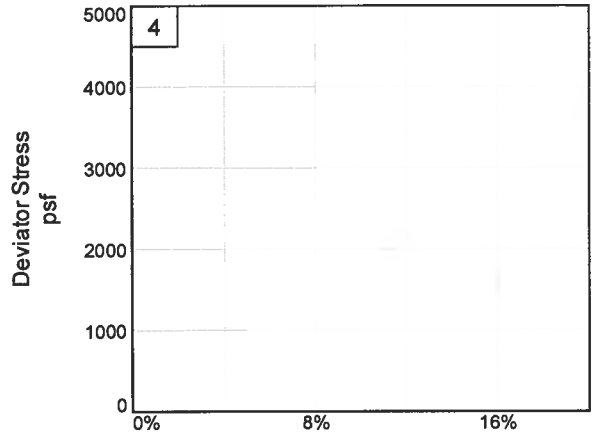
Blackburn Consulting

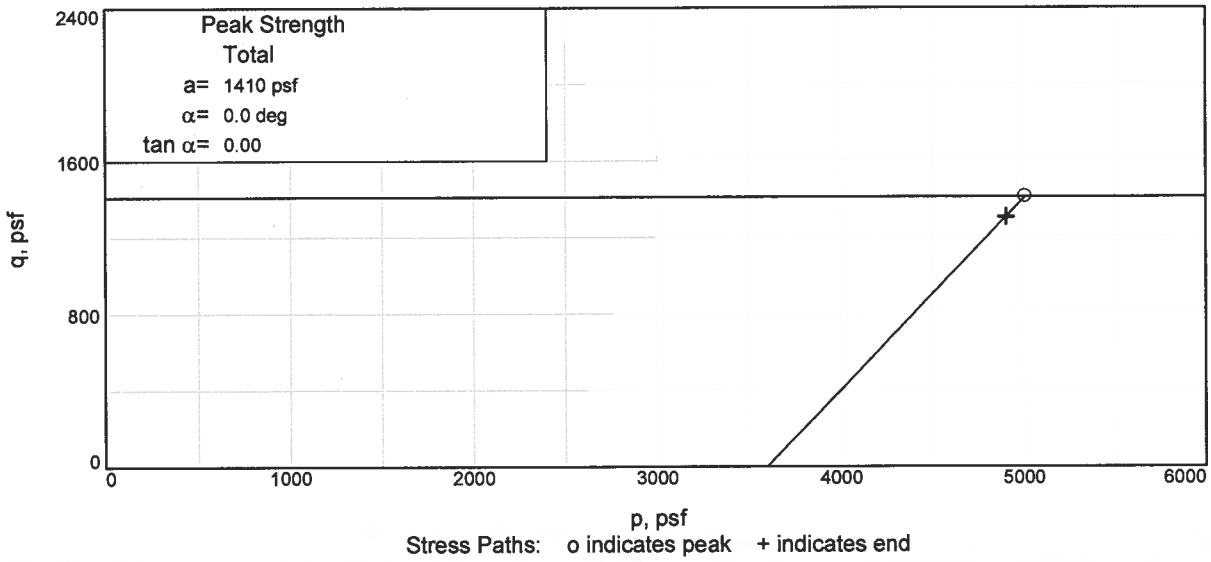
Tested By: KISB **Checked By:** RBL











Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B3 **Depth:** 51.0-51.5 **Sample Number:** 10c
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

Triaxial Compression UU no pore pressure
ASTM D 4767

blackburn consulting

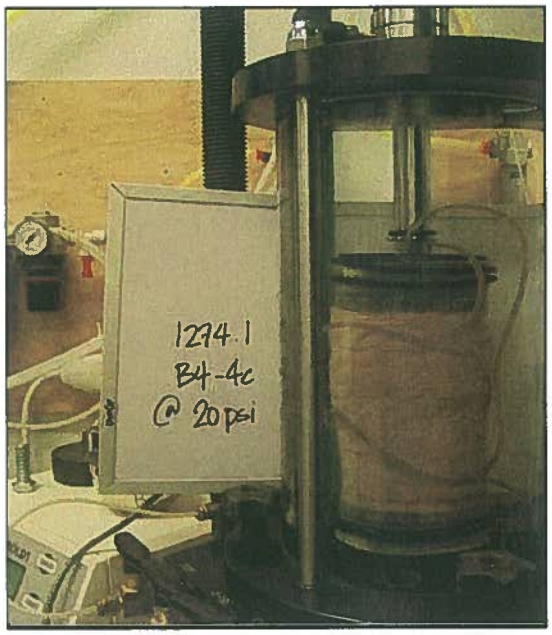
Project Name: SR 99 at Thomes Creek
Project Number: 1274.1
Sample ID: B4-4c **Depth (ft):** 26.0-26.5
Sample Description: Yellowish Brown Silty CLA Y (CL-ML)
Date: 11/6/09 **Tested By:** KISB

UU	X
CU	

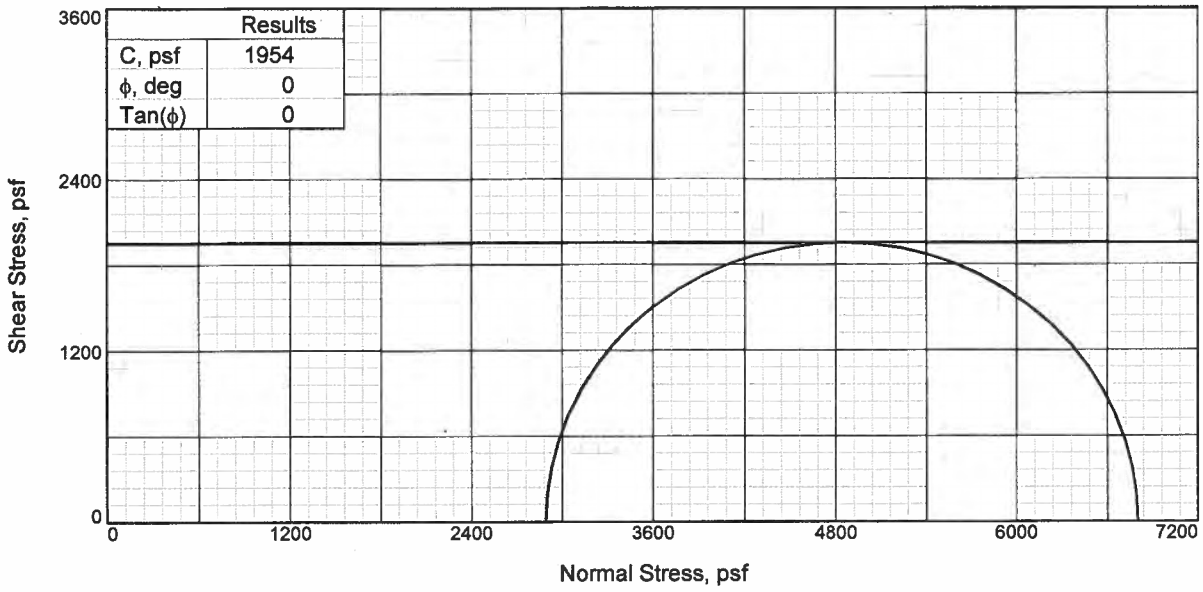
Staged

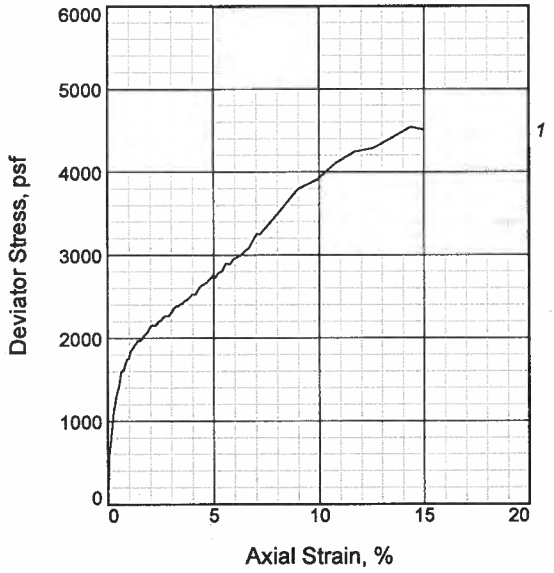
Specimen			
Initial		Final	
Tare No.	B5	Tare No.	NN
Tare =	46.9	Tare =	103.7
Wet Soil + Tare =	84.3	Wet Soil + Tare =	488.7
Dry Soil + Tare =	76.0	Dry Soil + Tare =	398.3
Water Content (%)	28.5%	Water Content (%)	30.7%

Length of Sample =	5.635	in	5%	0.28
Diameter of Sample =	2.369	in	10%	0.56
Area=	4.41	in^3	15%	0.85
Weight of Sample =	759.80	gm		
Specific Gravity=	2.70			
Initial Wet Density =	116.3	pcf		
Initial Dry Density =	90.5	pcf		
Actual Cell Pressure=	20.09	psi		
Disp. after CP Applied=	0.069	in.		



NOTES:





Specimen No.		1
Initial	Water Content, %	30.7
	Dry Density, pcf	89.2
	Saturation, %	93.1
	Void Ratio	0.8902
	Diameter, in.	2.369
	Height, in.	5.635
At Test	Water Content, %	30.7
	Dry Density, pcf	92.2
	Saturation, %	100.0
	Void Ratio	0.8285
	Diameter, in.	2.344
	Height, in.	5.566
Strain rate, in./min.		0.463
Back Pressure, psf		0
Cell Pressure, psf		2893
Fail. Stress, psf		3908
Strain, %		9.9
Ult. Stress, psf		4510
Strain, %		
σ_1 Failure, psf		6801
σ_3 Failure, psf		2893

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Yellowish Brown Silty CLAY (CL-ML)

Assumed Specific Gravity= 2.70
Remarks: Failure chosen at 10% Strain
Ultimate at 15% Strain

Figure _____

Client: HDR

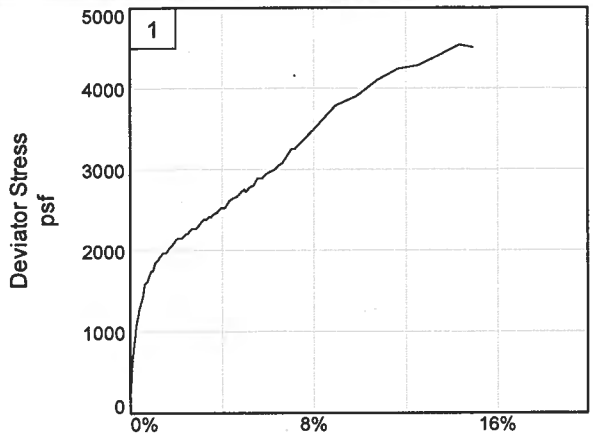
Project: SR 99 @ Thomes Creek

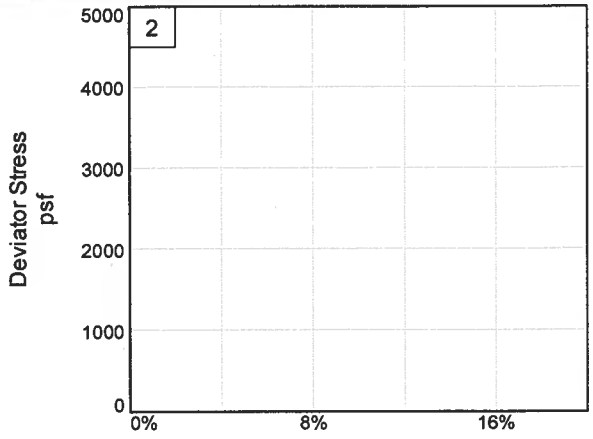
Source of Sample: B4 **Depth:** 26.0-26.5
Sample Number: 4c
Proj. No.: 1274.1 **Date Sampled:** 10-13-09

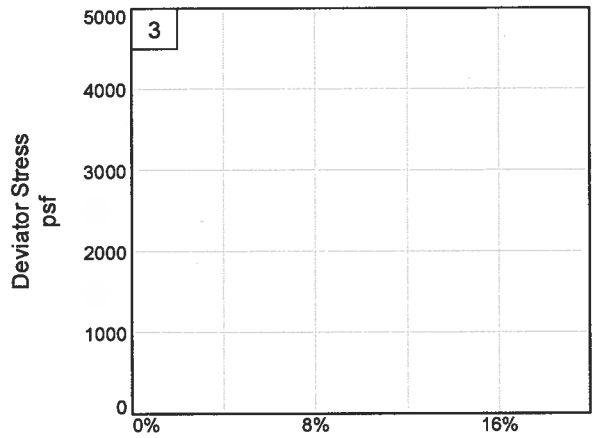
TRIAXIAL SHEAR TEST REPORT

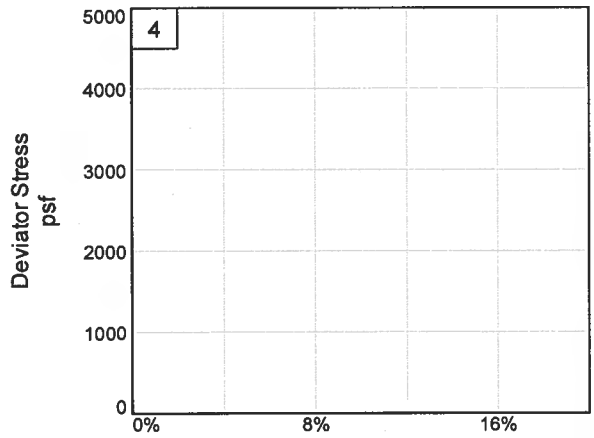
Blackburn Consulting

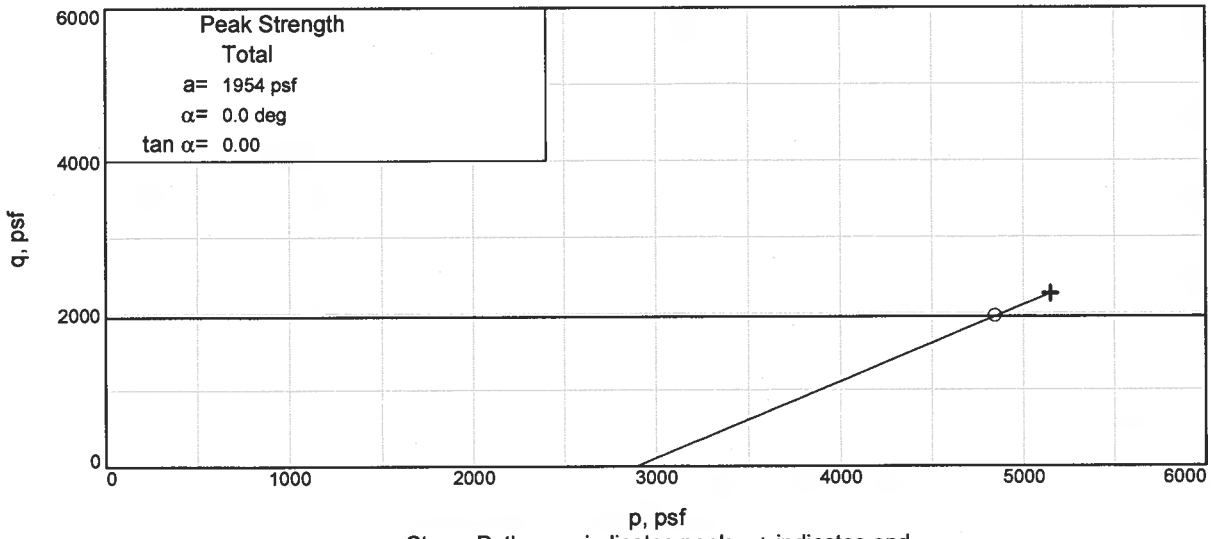
Tested By: KISB **Checked By:** RBL











Stress Paths: o indicates peak + indicates end

Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B4 **Depth:** 26.0-26.5 **Sample Number:** 4c
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

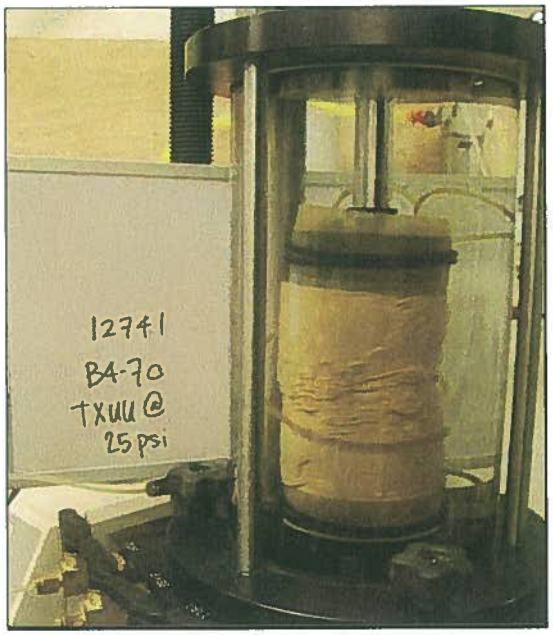
Triaxial Compression UU no pore pressure
ASTM D 4767

blackburn consulting

Project Name: SR 99 at Thomes Creek
Project Number: 1274.1
Sample ID: B4-7c **Depth (ft):** 41.0-41.5
Sample Description: Light Yellowish Brown Silty CLAY (CI-ML)
Date: 11/9/09 **Tested By:** KISB

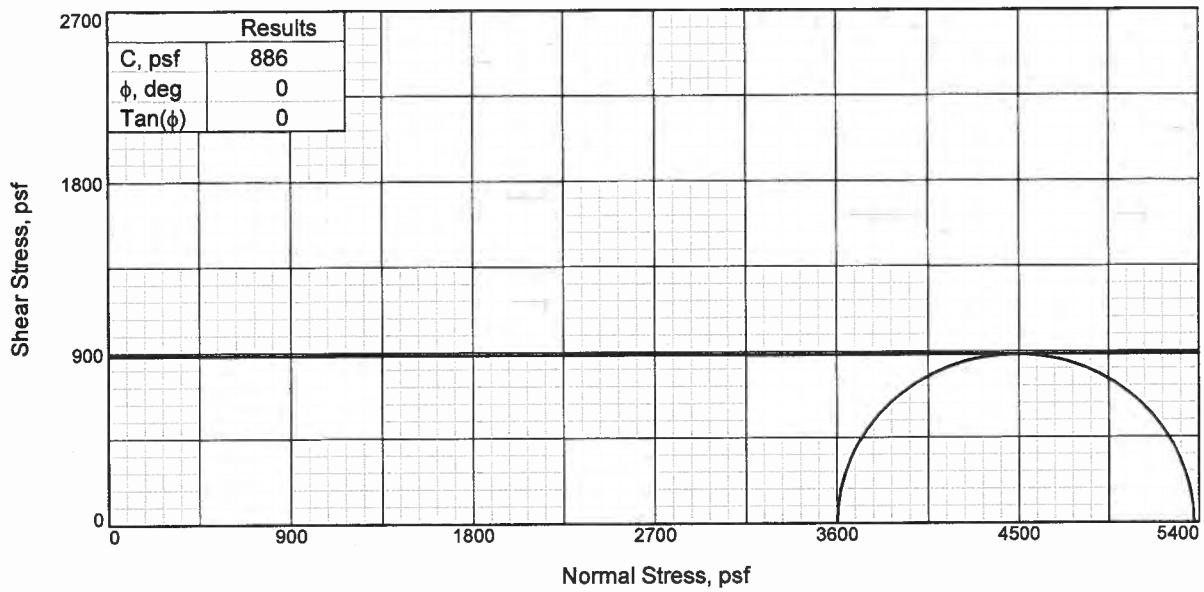
UU	X	
CU		Staged

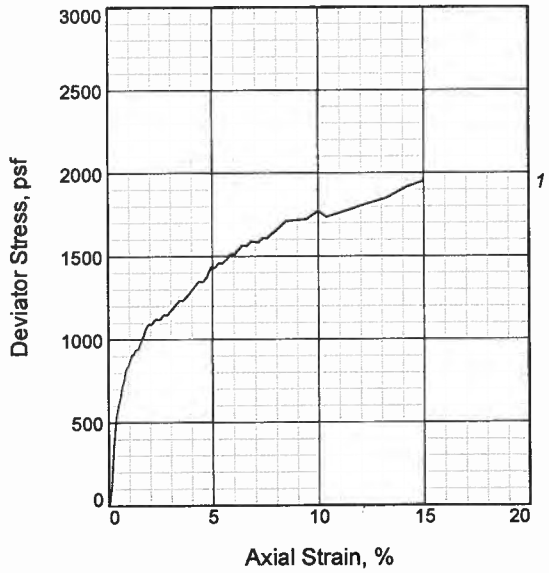
Specimen			
Initial		Final	
Tare No.		Tare No.	RR
Tare =		Tare =	105.0
Wet Soil + Tare =		Wet Soil + Tare =	496.6
Dry Soil + Tare =		Dry Soil + Tare =	425.5
Water Content (%)	#DIV/0!	Water Content (%)	22.2%



Length of Sample =	5.347	in	5%	0.27
Diameter of Sample =	2.380	in	10%	0.53
Area=	4.45	in^3	15%	0.80
Weight of Sample =	786.00	gm		
Specific Gravity=	2.70			
Initial Wet Density =	125.7	pcf		
Initial Dry Density =	#DIV/0!	pcf		
Actual Cell Pressure=	25.01	psi		
Disp. after CP Applied=	0.050	in.		

NOTES: 1.4" pocket of cemented soil in test specimen





Specimen No.		1
Initial	Water Content, %	22.2
	Dry Density, pcf	103.0
	Saturation, %	94.2
	Void Ratio	0.6361
	Diameter, in.	2.380
	Height, in.	5.347
At Test	Water Content, %	22.2
	Dry Density, pcf	105.4
	Saturation, %	100.0
	Void Ratio	0.5990
	Diameter, in.	2.364
	Height, in.	5.297
Strain rate, in./min.		0.473
Back Pressure, psf		0
Cell Pressure, psf		3601
Fail. Stress, psf		1772
Strain, %		10.0
Ult. Stress, psf		1952
Strain, %		
σ_1 Failure, psf		5373
σ_3 Failure, psf		3601

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Light Yellowish Brown Silty CLAY (CL-ML)

Assumed Specific Gravity= 2.70
Remarks: Failure chosen at 10% Strain
Ultimate at 15% Strain
1.4" pocket of cemented soil in test specimen

Figure _____

Client: HDR

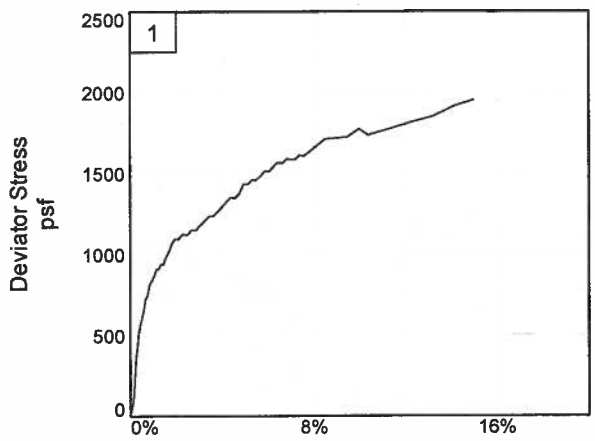
Project: SR 99 @ Thomes Creek

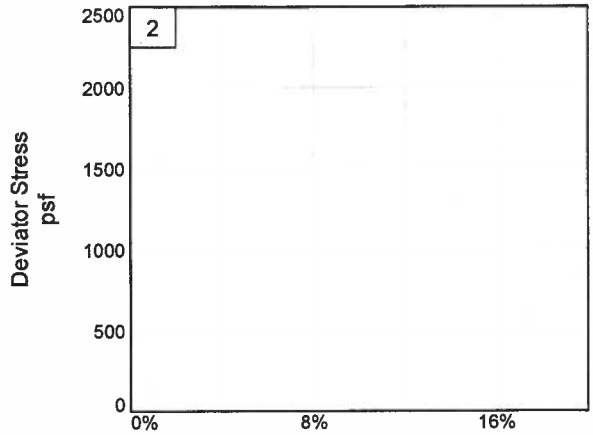
Source of Sample: B4 **Depth:** 41.0-41.5
Sample Number: 7c
Proj. No.: 1274.1 **Date Sampled:** 10-13-09

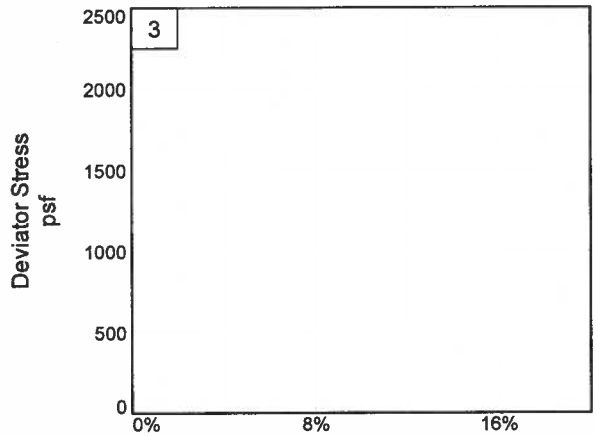
TRIAXIAL SHEAR TEST REPORT

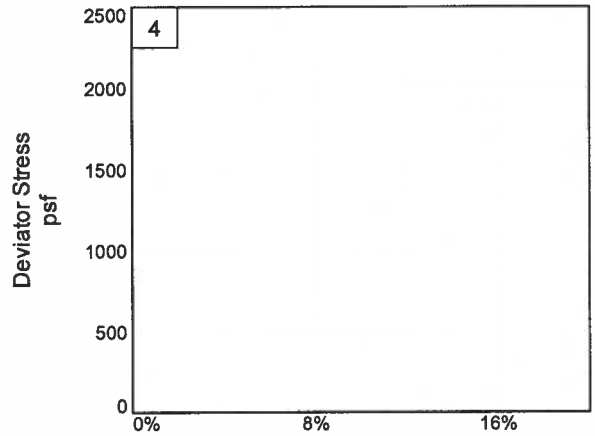
Blackburn Consulting

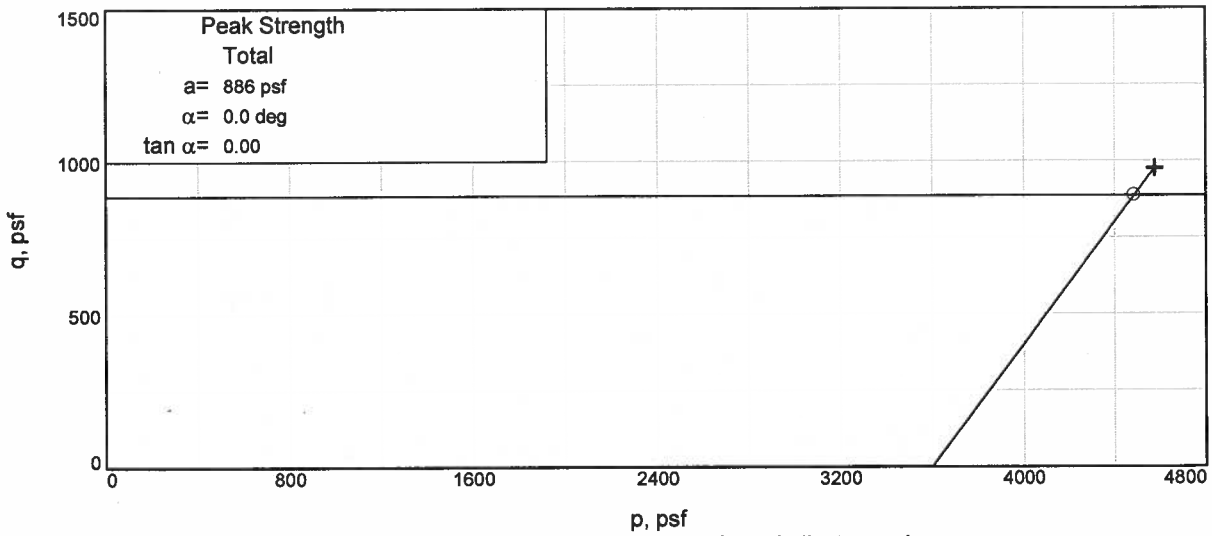
Tested By: KISB **Checked By:** RBL











Stress Paths: o indicates peak + indicates end

Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B4 **Depth:** 41.0-41.5 **Sample Number:** 7c
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

Triaxial Compression UU no pore pressure
ASTM D 4767

blackburn consulting

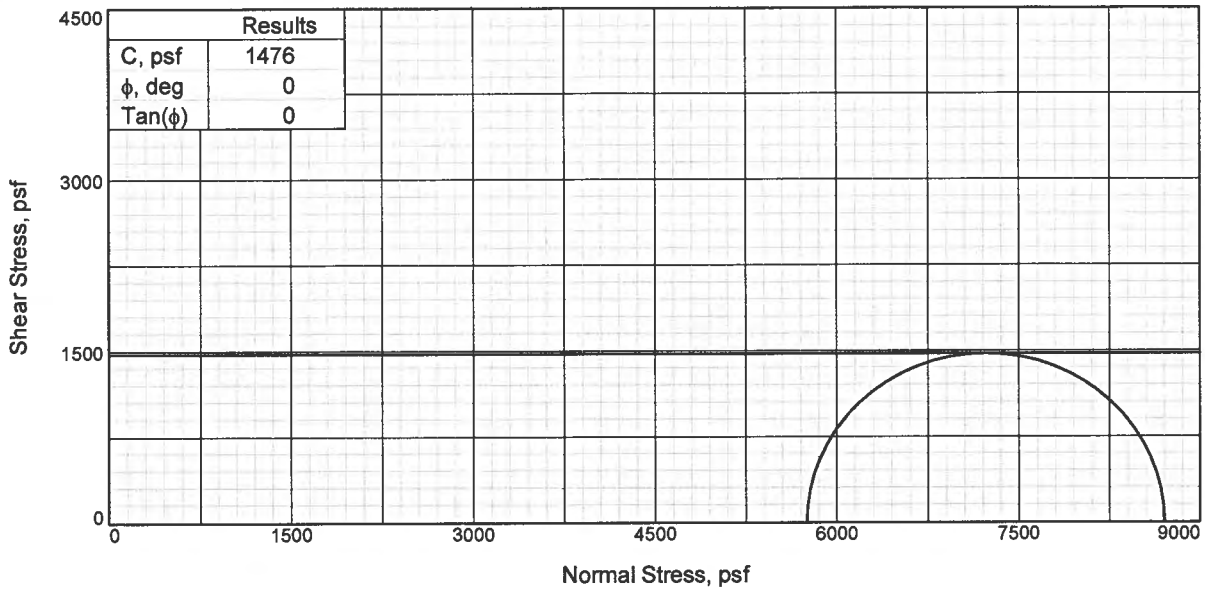
Project Name: SR 99 at Thomes Creek
Project Number: 1274.1
Sample ID: B4-13c **Depth (ft):** 71.0-71.5
Sample Description: Olive Brown Sandy SILT (ML)
Date: 11/9/09 **Tested By:** KISB

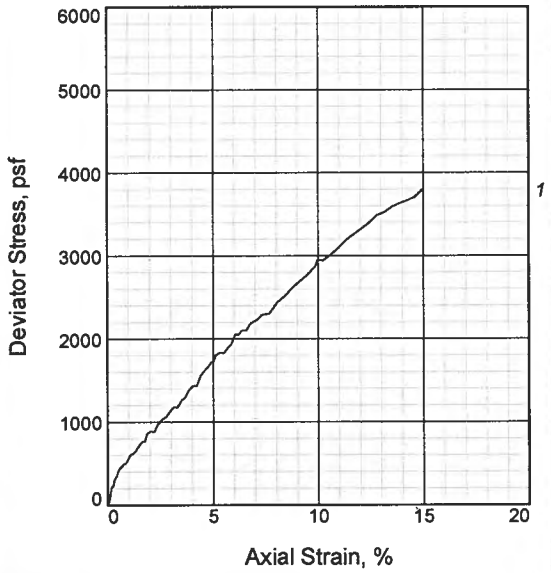
UU	X	
CU		Staged

Specimen			
Initial		Final	
Tare No.	B3	Tare No.	PP
Tare =	48.4	Tare =	105.2
Wet Soil + Tare =	78.1	Wet Soil + Tare =	602.8
Dry Soil + Tare =	72.5	Dry Soil + Tare =	512.3
Water Content (%)	23.2%	Water Content (%)	22.2%

Length of Sample =	5.493	in	5%	0.27
Diameter of Sample =	2.422	in	10%	0.55
Area=	4.61	in^3	15%	0.82
Weight of Sample =	877.60	gm		
Specific Gravity=	2.70			
Initial Wet Density =	131.8	pcf		
Initial Dry Density =	107.0	pcf		
Actual Cell Pressure=	39.95	psi		
Disp. after CP Applied=	0.01	in.		

NOTES:





Specimen No.		1
Initial	Water Content, %	22.2
	Dry Density, pcf	108.1
	Saturation, %	99.7
	Void Ratio	0.6288
	Diameter, in.	2.422
	Height, in.	5.493
At Test	Water Content, %	22.2
	Dry Density, pcf	108.2
	Saturation, %	100.0
	Void Ratio	0.6269
	Diameter, in.	2.423
	Height, in.	5.483
Strain rate, in./min.		0.455
Back Pressure, psf		0
Cell Pressure, psf		5753
Fail. Stress, psf		2953
Strain, %		10.0
Ult. Stress, psf		3799
Strain, %		
σ_1 Failure, psf		8705
σ_3 Failure, psf		5753

Type of Test:
Unconsolidated Undrained
Sample Type: 2.4" Mod Cal
Description: Light Yellowish Brown Sandy SILT (ML)

Assumed Specific Gravity= 2.82
Remarks: Failure chosen at 10% Strain
Ultimate at 15% Strain

Figure _____

Client: HDR

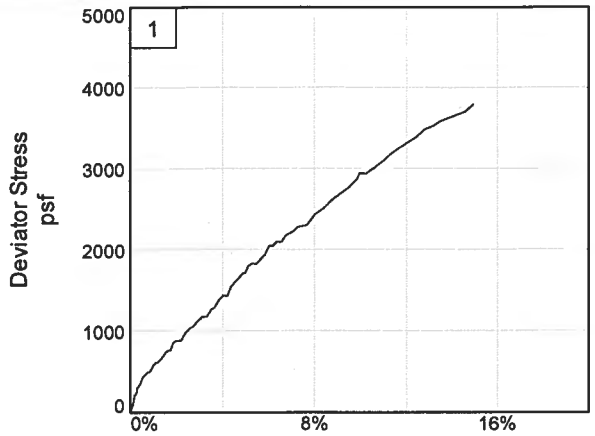
Project: SR 99 @ Thomes Creek

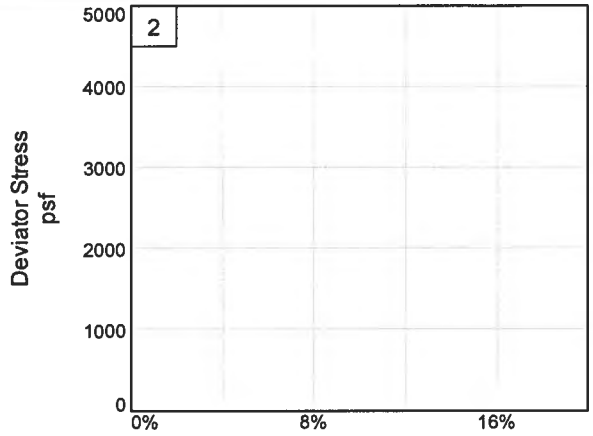
Source of Sample: B4 **Depth:** 71.0-71.5
Sample Number: 13c
Proj. No.: 1274.1 **Date Sampled:** 10-13-09

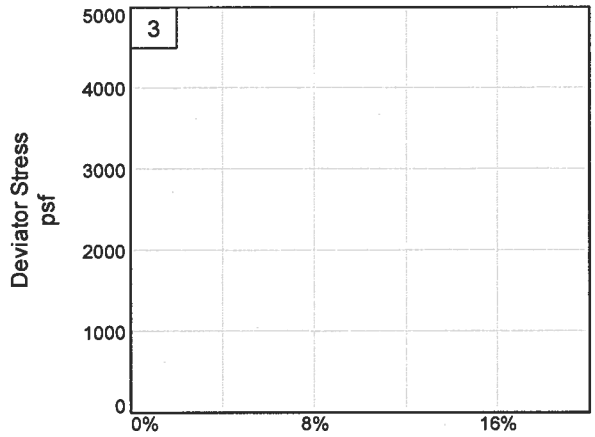
TRIAXIAL SHEAR TEST REPORT

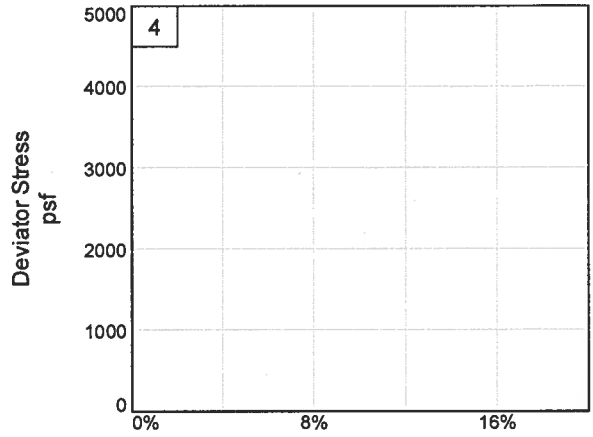
Blackburn Consulting

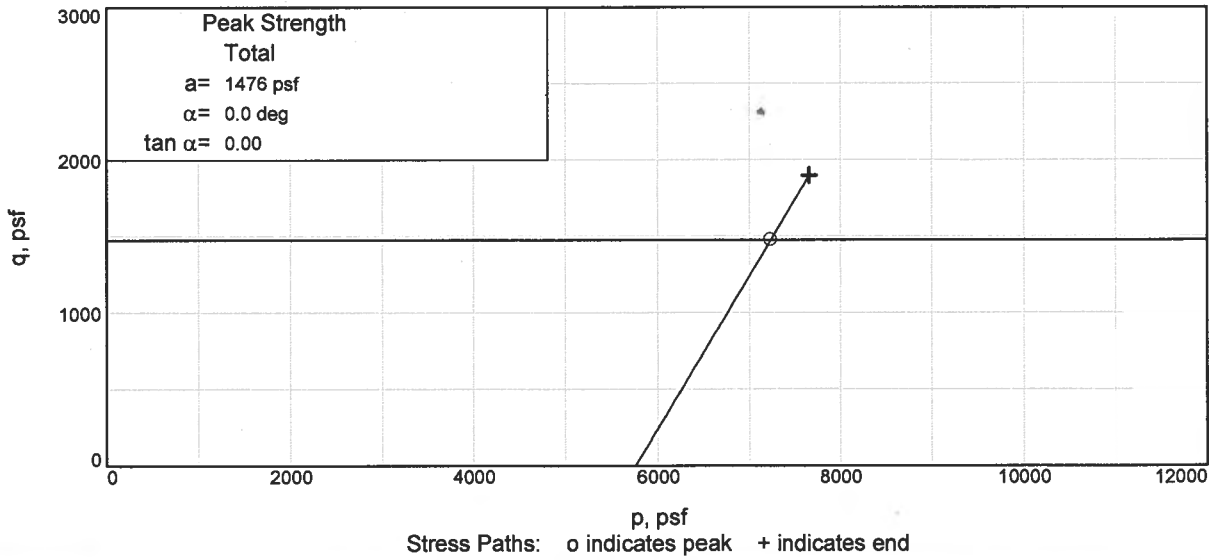
Tested By: KISB **Checked By:** RBL











Client: HDR
Project: SR 99 @ Thomes Creek
Source of Sample: B4 **Depth:** 71.0-71.5 **Sample Number:** 13c
Project No.: 1274.1 **Figure** _____

Blackburn Consulting

Tested By: KISB **Checked By:** RBL

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

Date Reported 10/16/2009
Date Submitted 10/13/2009

To: Ken Colburn
Blackburn Consulting
11521 Blocker Dr. Ste. 110
Auburn, CA 95603

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

The reported analysis was requested for the following location:
Location : 1274.1/SR99 @ THOMAS Site ID : F2-7.
Your purchase order number is 1274.1.
Thank you for your business.

* For future reference to this analysis please use SUN # 56894-115151.

EVALUATION FOR SOIL CORROSION

Soil pH	7.97	
Minimum Resistivity	1.77 ohm-cm (x1000)	
Chloride	31.2 ppm	00.00312 %
Sulfate	3.5 ppm	00.00035 %

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 10/16/2009
Date Submitted 10/13/2009

To: Ken Colburn
Blackburn Consulting
11521 Blocker Dr. Ste. 110
Auburn, CA 95603

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

The reported analysis was requested for the following location:
Location : 1274.1/SR99 @ THOMAS Site ID : F2-10.
Your purchase order number is 1274.1.
Thank you for your business.

* For future reference to this analysis please use SUN # 56894-115152.

EVALUATION FOR SOIL CORROSION

Soil pH	7.97		
Minimum Resistivity	2.36 ohm-cm (x1000)		
Chloride	22.0 ppm	00.00220	%
Sulfate	3.1 ppm	00.00031	%

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 10/16/2009
Date Submitted 10/13/2009

To: Ken Colburn
Blackburn Consulting
11521 Blocker Dr. Ste. 110
Auburn, CA 95603

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

The reported analysis was requested for the following location:
Location : 1274.1/SR99 @ THOMAS Site ID : F2-13.
Your purchase order number is 1274.1.
Thank you for your business.

* For future reference to this analysis please use SUN # 56894-115153.

EVALUATION FOR SOIL CORROSION

Soil pH	7.42	
Minimum Resistivity	1.63 ohm-cm (x1000)	
Chloride	26.0 ppm	00.00260 %
Sulfate	5.2 ppm	00.00052 %

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 10/23/2009
Date Submitted 10/20/2009

To: Ken Colburn
Blackburn Consulting
11521 Blocker Dr. Ste. 110
Auburn, CA 95603

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

The reported analysis was requested for the following location:
Location : 1274.1/SR99@THOMES Site ID : B3-12.
Your purchase order number is 1274.1.
Thank you for your business.

* For future reference to this analysis please use SUN # 56919-115188.

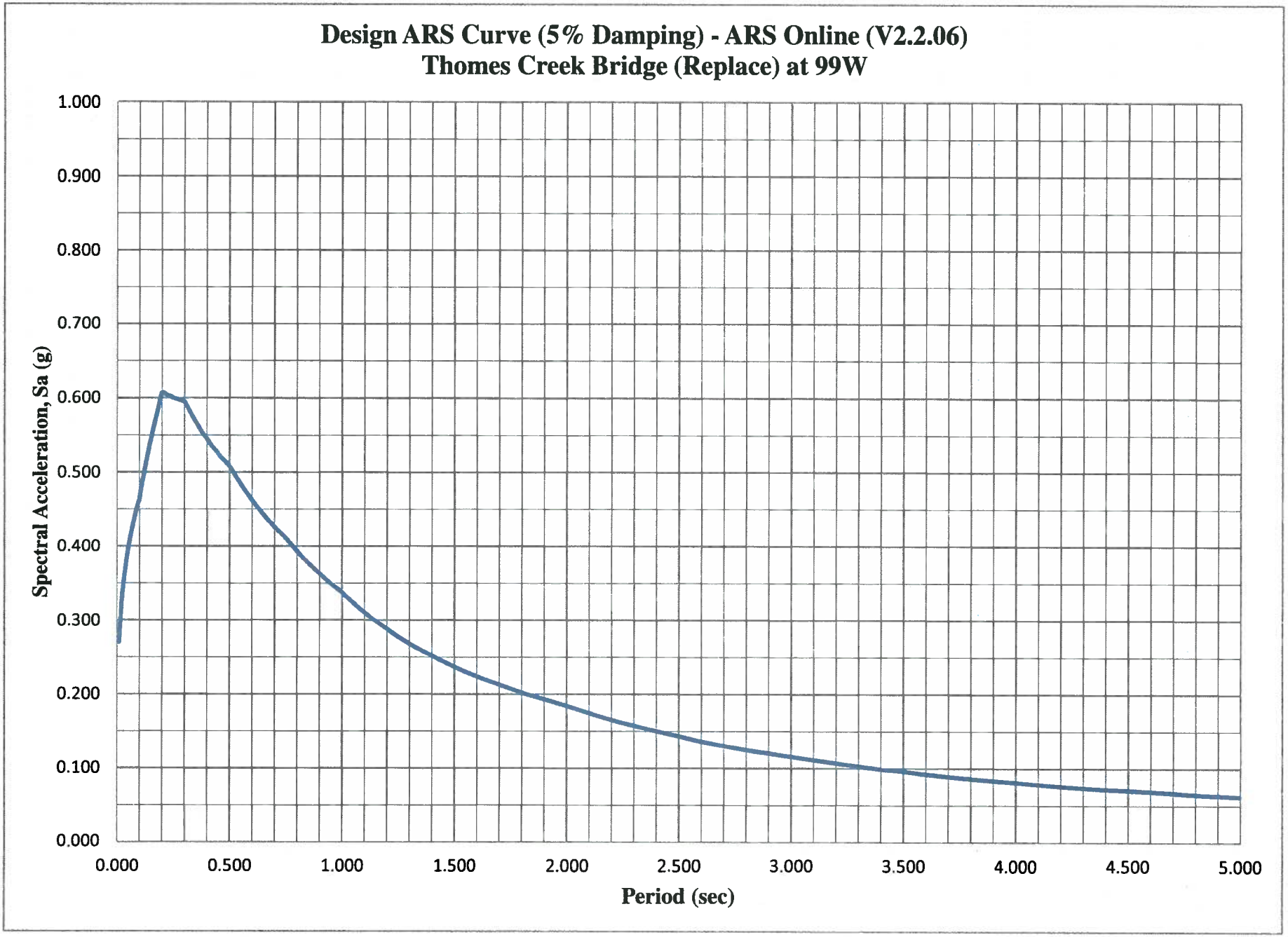
EVALUATION FOR SOIL CORROSION

Soil pH	7.49	
Minimum Resistivity	1.37 ohm-cm (x1000)	
Chloride	15.1 ppm	00.00151 %
Sulfate	2.9 ppm	00.00029 %

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

APPENDIX C

Design Response Spectrum and Tabular Data


Design ARS Curve (5% Damping) - ARS Online (V2.2.06)
Thomes Creek Bridge (Replace) at 99W
Spectral Acceleration, Sa (g)
1.000
0.900
0.800
0.700
0.600
0.500
0.400
0.300
0.200
0.100
0.000
0.000
0.500
1.000
1.500
2.000
2.500
3.000
3.500
4.000
4.500
5.000
Period (sec)

Thomes Creek Bridge (Replace) at 99W
BCI No. 1274.2
ARS Online Design Envelope Data (V2.2.06)
09/16/13

Period (sec.)	Spectral Accel. (g)	Period (sec.)	Spectral Accel. (g)	Period (sec.)	Spectral Accel. (g)
0.010	0.270	0.260	0.599	2.800	0.125
0.020	0.318	0.280	0.597	3.000	0.116
0.022	0.325	0.290	0.596	3.200	0.107
0.025	0.335	0.300	0.595	3.400	0.099
0.029	0.347	0.320	0.584	3.500	0.096
0.030	0.350	0.340	0.573	3.600	0.092
0.032	0.355	0.350	0.568	3.800	0.086
0.035	0.363	0.360	0.563	4.000	0.081
0.036	0.365	0.380	0.553	4.200	0.076
0.040	0.374	0.400	0.545	4.400	0.072
0.042	0.379	0.420	0.536	4.600	0.069
0.044	0.383	0.440	0.529	4.800	0.065
0.045	0.385	0.450	0.525	5.000	0.062
0.046	0.387	0.460	0.521		
0.048	0.391	0.480	0.515		
0.050	0.395	0.500	0.508		
0.055	0.404	0.550	0.484		
0.060	0.412	0.600	0.462		
0.065	0.420	0.650	0.443		
0.067	0.423	0.667	0.437		
0.070	0.427	0.700	0.426		
0.075	0.434	0.750	0.411		
0.080	0.441	0.800	0.393		
0.085	0.447	0.850	0.377		
0.090	0.453	0.900	0.363		
0.095	0.459	0.950	0.349		
0.100	0.464	1.000	0.337		
0.110	0.482	1.100	0.310		
0.120	0.498	1.200	0.288		
0.130	0.514	1.300	0.268		
0.133	0.518	1.400	0.252		
0.140	0.529	1.500	0.237		
0.150	0.543	1.600	0.224		
0.160	0.557	1.700	0.213		
0.170	0.570	1.800	0.202		
0.180	0.582	1.900	0.193		
0.190	0.595	2.000	0.184		
0.200	0.607	2.200	0.165		
0.220	0.604	2.400	0.150		
0.240	0.602	2.500	0.143		
0.250	0.600	2.600	0.136		

APPENDIX D

Driven 1.2 Input Files and
Compressive Resistance Graphs

LPILE Output Graphs

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\AB1HP.DVN
Project Name: Thomes Cr. Br. - Abut 1 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	9.00 ft
	- Driving/Restrike	9.00 ft
	- Ultimate:	9.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

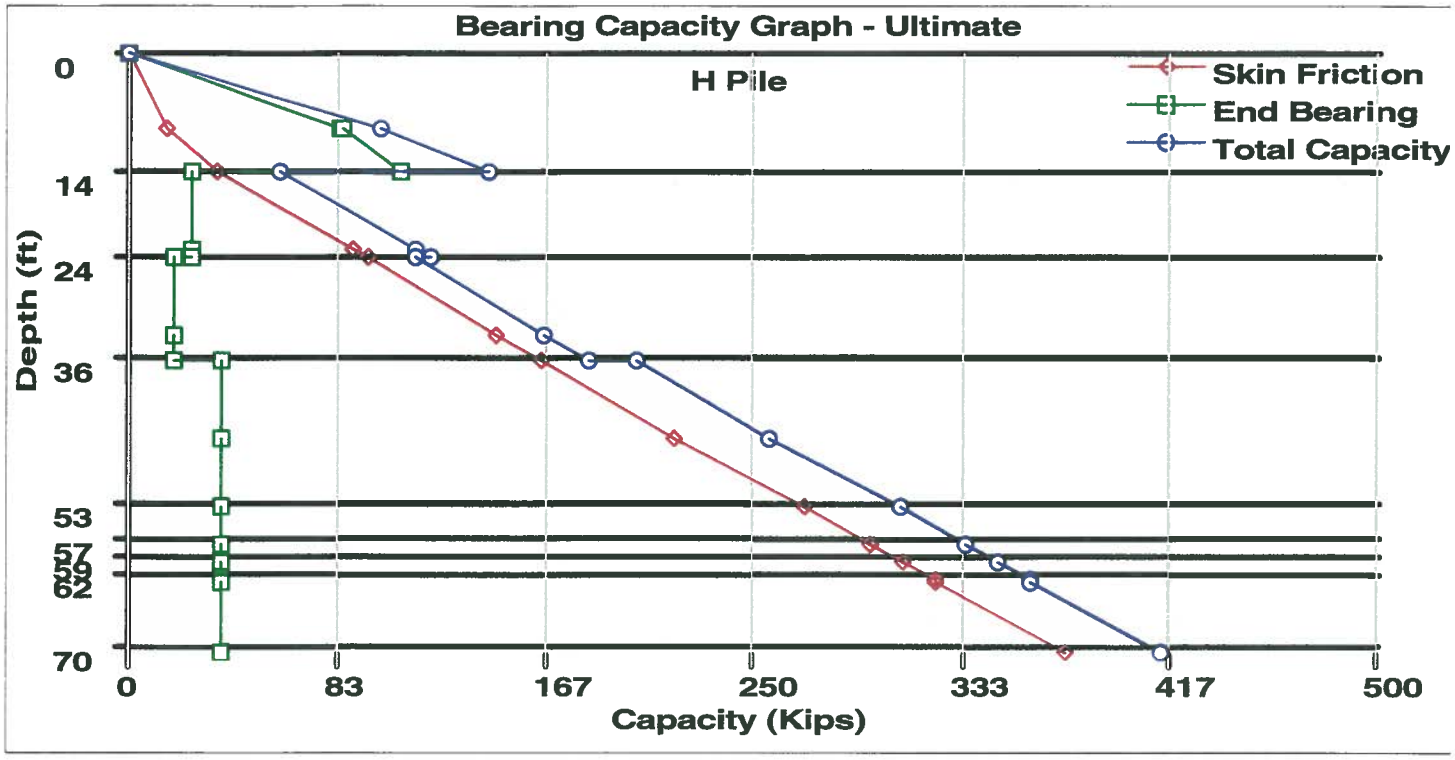
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	14.00 ft	0.00%	125.00 pcf	36.0/36.0	Nordlund
2	Cohesive	10.00 ft	0.00%	115.00 pcf	2000.00 psf	T-79 Steel
3	Cohesive	12.00 ft	0.00%	115.00 pcf	1500.00 psf	T-79 Steel
4	Cohesive	17.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
5	Cohesive	4.25 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
6	Cohesive	2.12 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
7	Cohesive	2.12 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
8	Cohesive	8.50 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.09 Kips	0.09 Kips
8.99 ft	15.58 Kips	85.33 Kips	100.91 Kips
9.01 ft	15.65 Kips	85.47 Kips	101.12 Kips
13.99 ft	35.33 Kips	109.14 Kips	144.47 Kips
14.01 ft	35.43 Kips	25.40 Kips	60.83 Kips
23.01 ft	89.67 Kips	25.40 Kips	115.07 Kips
23.99 ft	96.02 Kips	25.40 Kips	121.42 Kips
24.01 ft	96.14 Kips	19.05 Kips	115.19 Kips
33.01 ft	147.18 Kips	19.05 Kips	166.23 Kips
35.99 ft	165.16 Kips	19.05 Kips	184.22 Kips
36.01 ft	165.28 Kips	38.11 Kips	203.38 Kips
45.01 ft	218.55 Kips	38.11 Kips	256.65 Kips
52.99 ft	270.23 Kips	38.11 Kips	308.34 Kips
53.01 ft	270.35 Kips	38.11 Kips	308.46 Kips
57.24 ft	296.50 Kips	38.11 Kips	334.60 Kips
57.26 ft	296.62 Kips	38.11 Kips	334.73 Kips
59.37 ft	309.63 Kips	38.11 Kips	347.74 Kips
59.38 ft	309.75 Kips	38.11 Kips	347.86 Kips
61.49 ft	322.76 Kips	38.11 Kips	360.87 Kips
61.51 ft	322.89 Kips	38.11 Kips	360.99 Kips
69.99 ft	375.30 Kips	38.11 Kips	413.40 Kips

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\AB1HP.DVN



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR2HP.DVN
Project Name: Thomes Cr. Br. - Pier 2 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	15.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

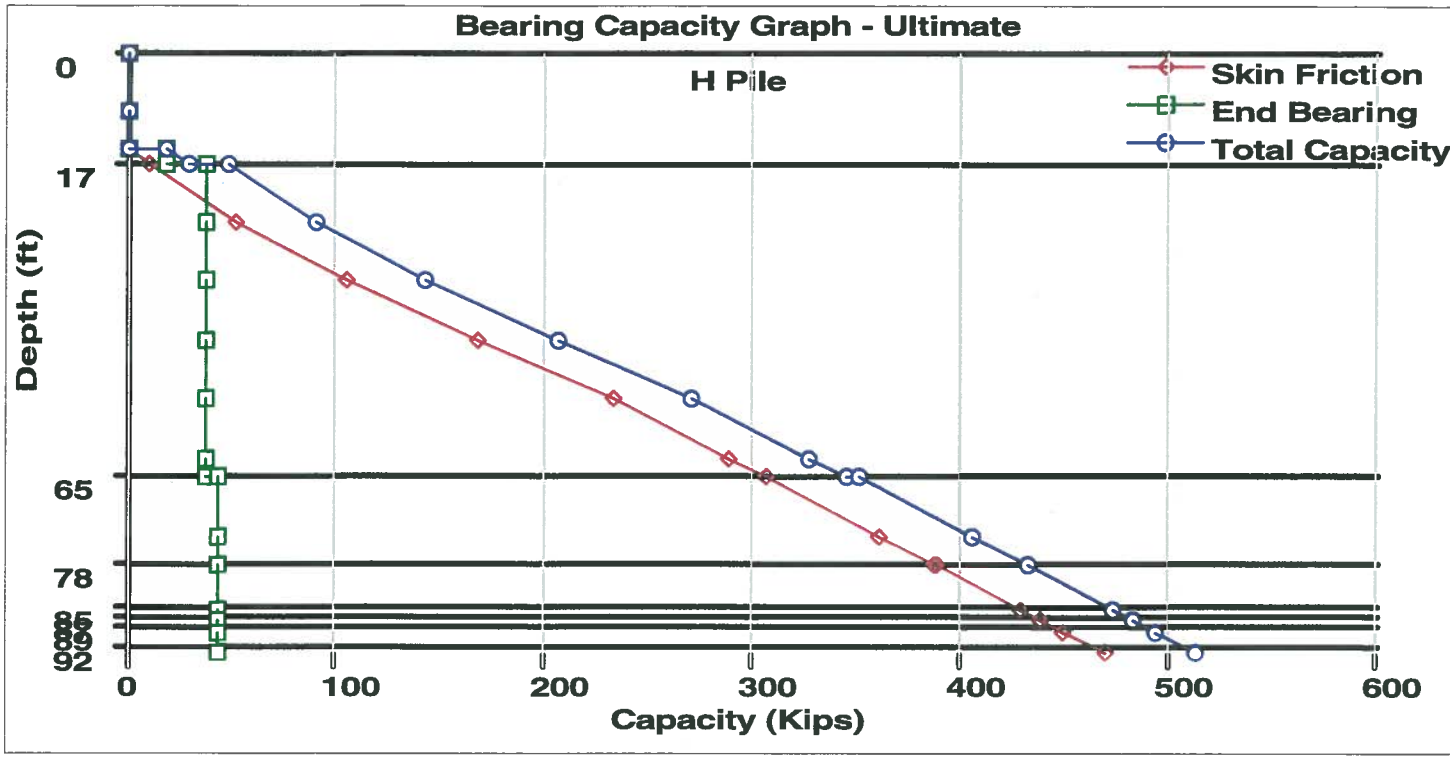
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	17.00 ft	0.00%	115.00 pcf	1500.00 psf	T-79 Steel
2	Cohesive	48.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
3	Cohesive	13.50 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
4	Cohesive	6.75 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
5	Cohesive	1.69 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
6	Cohesive	1.69 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
7	Cohesive	3.38 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
14.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
15.00 ft	0.00 Kips	19.05 Kips	19.05 Kips
16.99 ft	10.32 Kips	19.05 Kips	29.37 Kips
17.01 ft	10.41 Kips	38.11 Kips	48.51 Kips
26.01 ft	52.39 Kips	38.11 Kips	90.50 Kips
35.01 ft	105.07 Kips	38.11 Kips	143.17 Kips
44.01 ft	168.44 Kips	38.11 Kips	206.54 Kips
53.01 ft	232.92 Kips	38.11 Kips	271.03 Kips
62.01 ft	288.55 Kips	38.11 Kips	326.65 Kips
64.99 ft	306.97 Kips	38.11 Kips	345.07 Kips
65.01 ft	307.09 Kips	44.46 Kips	351.54 Kips
74.01 ft	361.00 Kips	44.46 Kips	405.46 Kips
78.49 ft	387.84 Kips	44.46 Kips	432.29 Kips
78.51 ft	387.96 Kips	44.46 Kips	432.41 Kips
85.24 ft	428.27 Kips	44.46 Kips	472.73 Kips
85.26 ft	428.39 Kips	44.46 Kips	472.85 Kips
86.93 ft	438.38 Kips	44.46 Kips	482.84 Kips
86.95 ft	438.50 Kips	44.46 Kips	482.96 Kips
88.61 ft	448.49 Kips	44.46 Kips	492.94 Kips
88.64 ft	448.61 Kips	44.46 Kips	493.06 Kips
91.99 ft	468.70 Kips	44.46 Kips	513.16 Kips

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR2HP.DVN



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR3HP.DVN
Project Name: Thomes Cr. Br. - Pier 3 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	15.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

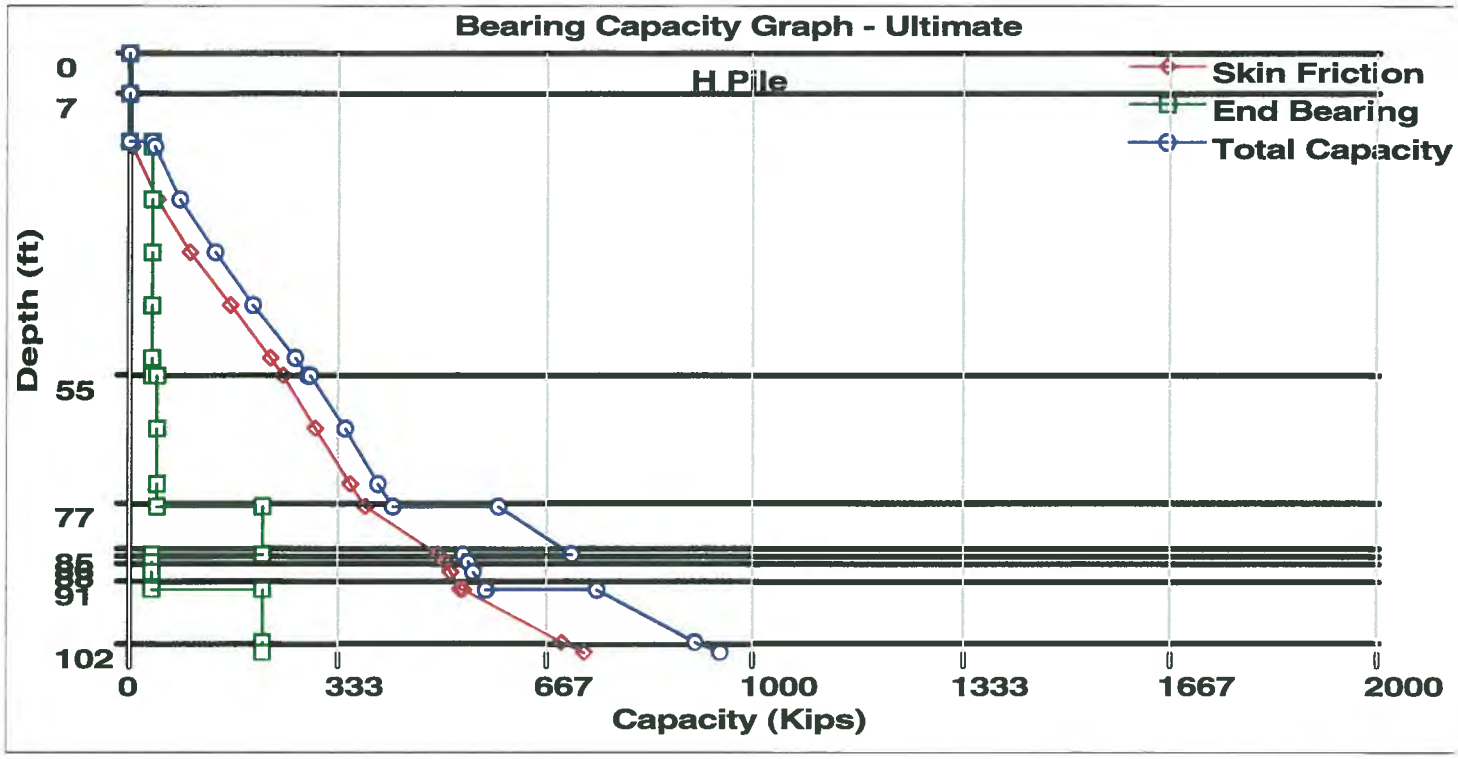
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	7.00 ft	0.00%	115.00 pcf	1500.00 psf	T-79 Steel
2	Cohesive	48.00 ft	0.00%	120.00 pcf	3000.00 psf	T-79 Steel
3	Cohesive	22.00 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
4	Cohesionless	8.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund
5	Cohesive	1.50 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
6	Cohesive	1.50 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
7	Cohesive	3.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
8	Cohesionless	11.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
6.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
7.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
14.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
15.00 ft	0.00 Kips	38.11 Kips	38.11 Kips
16.01 ft	4.04 Kips	38.11 Kips	42.15 Kips
25.01 ft	46.03 Kips	38.11 Kips	84.13 Kips
34.01 ft	98.70 Kips	38.11 Kips	136.81 Kips
43.01 ft	162.07 Kips	38.11 Kips	200.18 Kips
52.01 ft	228.74 Kips	38.11 Kips	266.84 Kips
54.99 ft	247.15 Kips	38.11 Kips	285.26 Kips
55.01 ft	247.28 Kips	44.46 Kips	291.73 Kips
64.01 ft	301.19 Kips	44.46 Kips	345.65 Kips
73.01 ft	355.10 Kips	44.46 Kips	399.56 Kips
76.99 ft	378.94 Kips	44.46 Kips	423.40 Kips
77.01 ft	379.14 Kips	213.96 Kips	593.10 Kips
84.99 ft	496.79 Kips	213.96 Kips	710.75 Kips
85.01 ft	497.01 Kips	38.11 Kips	535.12 Kips
86.49 ft	506.16 Kips	38.11 Kips	544.26 Kips
86.51 ft	506.28 Kips	38.11 Kips	544.39 Kips
87.99 ft	515.43 Kips	38.11 Kips	553.54 Kips
88.01 ft	515.55 Kips	38.11 Kips	553.66 Kips
90.99 ft	533.97 Kips	38.11 Kips	572.08 Kips
91.01 ft	534.20 Kips	213.96 Kips	748.16 Kips
100.01 ft	693.28 Kips	213.96 Kips	907.24 Kips
101.99 ft	730.55 Kips	213.96 Kips	944.51 Kips

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR3HP.DVN



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR4HP.DVN
Project Name: Thomes Cr. Br. - Pier 4 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	15.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

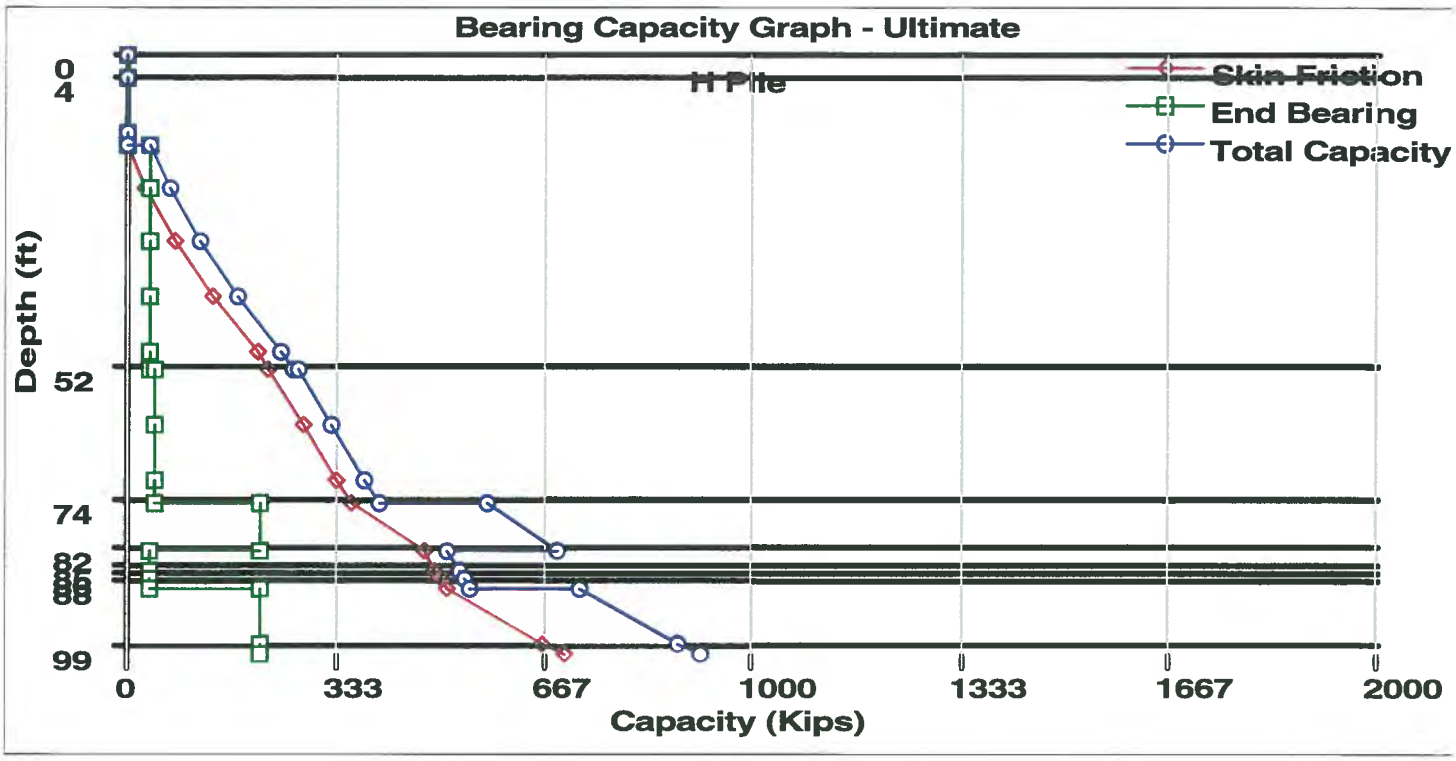
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	4.00 ft	0.00%	115.00 pcf	1500.00 psf	T-79 Steel
2	Cohesive	48.00 ft	0.00%	120.00 pcf	3000.00 psf	T-79 Steel
3	Cohesive	22.00 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
4	Cohesionless	8.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund
5	Cohesive	3.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
6	Cohesive	1.50 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
7	Cohesive	1.50 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
8	Cohesionless	11.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund

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
4.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
13.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
14.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
15.00 ft	0.00 Kips	38.11 Kips	38.11 Kips
22.01 ft	30.85 Kips	38.11 Kips	68.95 Kips
31.01 ft	79.96 Kips	38.11 Kips	118.06 Kips
40.01 ft	139.76 Kips	38.11 Kips	177.87 Kips
49.01 ft	210.20 Kips	38.11 Kips	248.30 Kips
51.99 ft	228.61 Kips	38.11 Kips	266.72 Kips
52.01 ft	228.74 Kips	44.46 Kips	273.19 Kips
61.01 ft	282.65 Kips	44.46 Kips	327.10 Kips
70.01 ft	336.56 Kips	44.46 Kips	381.02 Kips
73.99 ft	360.40 Kips	44.46 Kips	404.86 Kips
74.01 ft	360.60 Kips	213.96 Kips	574.55 Kips
81.99 ft	474.36 Kips	213.96 Kips	688.32 Kips
82.01 ft	474.58 Kips	38.11 Kips	512.68 Kips
84.99 ft	493.00 Kips	38.11 Kips	531.10 Kips
85.01 ft	493.12 Kips	38.11 Kips	531.22 Kips
86.49 ft	502.27 Kips	38.11 Kips	540.37 Kips
86.51 ft	502.39 Kips	38.11 Kips	540.50 Kips
87.99 ft	511.54 Kips	38.11 Kips	549.64 Kips
88.01 ft	511.76 Kips	213.96 Kips	725.72 Kips
97.01 ft	666.47 Kips	213.96 Kips	880.42 Kips
98.99 ft	702.77 Kips	213.96 Kips	916.72 Kips

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR4HP.DVN



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR5HP.DVN
Project Name: Thomes Cr. Br. - Pier 5 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	15.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

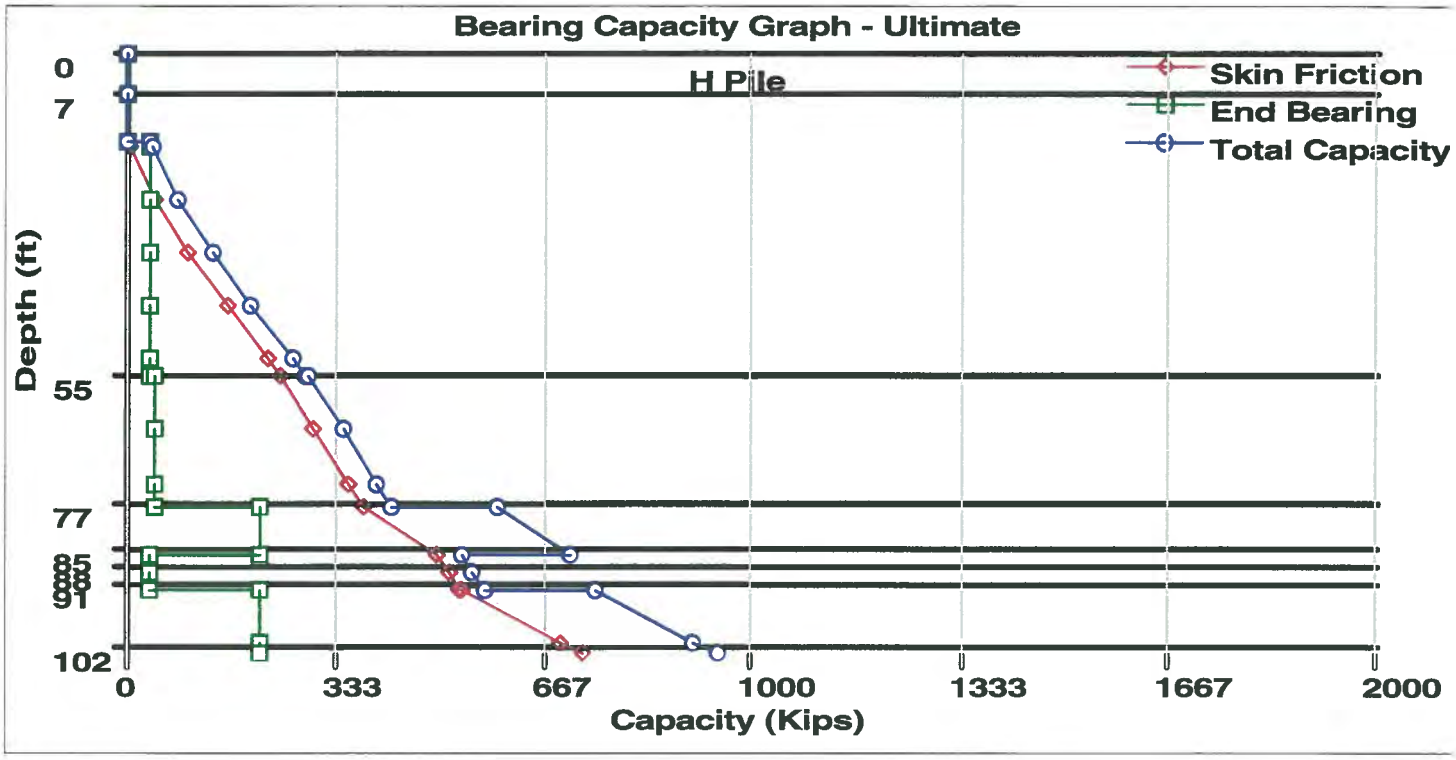
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	7.00 ft	0.00%	115.00 pcf	1500.00 psf	T-79 Steel
2	Cohesive	48.00 ft	0.00%	120.00 pcf	3000.00 psf	T-79 Steel
3	Cohesive	22.00 ft	0.00%	125.00 pcf	3500.00 psf	T-79 Steel
4	Cohesionless	8.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund
5	Cohesive	3.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
6	Cohesive	3.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel
7	Cohesionless	11.00 ft	0.00%	130.00 pcf	36.0/36.0	Nordlund

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
6.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
7.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
14.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
15.00 ft	0.00 Kips	38.11 Kips	38.11 Kips
16.01 ft	4.04 Kips	38.11 Kips	42.15 Kips
25.01 ft	46.03 Kips	38.11 Kips	84.13 Kips
34.01 ft	98.70 Kips	38.11 Kips	136.81 Kips
43.01 ft	162.07 Kips	38.11 Kips	200.18 Kips
52.01 ft	228.74 Kips	38.11 Kips	266.84 Kips
54.99 ft	247.15 Kips	38.11 Kips	285.26 Kips
55.01 ft	247.28 Kips	44.46 Kips	291.73 Kips
64.01 ft	301.19 Kips	44.46 Kips	345.65 Kips
73.01 ft	355.10 Kips	44.46 Kips	399.56 Kips
76.99 ft	378.94 Kips	44.46 Kips	423.40 Kips
77.01 ft	379.14 Kips	213.96 Kips	593.10 Kips
84.99 ft	496.79 Kips	213.96 Kips	710.75 Kips
85.01 ft	497.01 Kips	38.11 Kips	535.12 Kips
87.99 ft	515.43 Kips	38.11 Kips	553.54 Kips
88.01 ft	515.55 Kips	38.11 Kips	553.66 Kips
90.99 ft	533.97 Kips	38.11 Kips	572.08 Kips
91.01 ft	534.20 Kips	213.96 Kips	748.16 Kips
100.01 ft	693.28 Kips	213.96 Kips	907.24 Kips
101.99 ft	730.55 Kips	213.96 Kips	944.51 Kips

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\PR5HP.DVN



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\AB6HP.DVN
Project Name: Thomes Cr. Br. - Abut 6 Project Date: 02/14/2014
Project Client: HDR
Computed By: DJM
Project Manager: DJM

PILE INFORMATION

Pile Type: H Pile - HP14X89
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	2.50 ft
	- Driving/Restrike	2.50 ft
	- Ultimate:	2.50 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

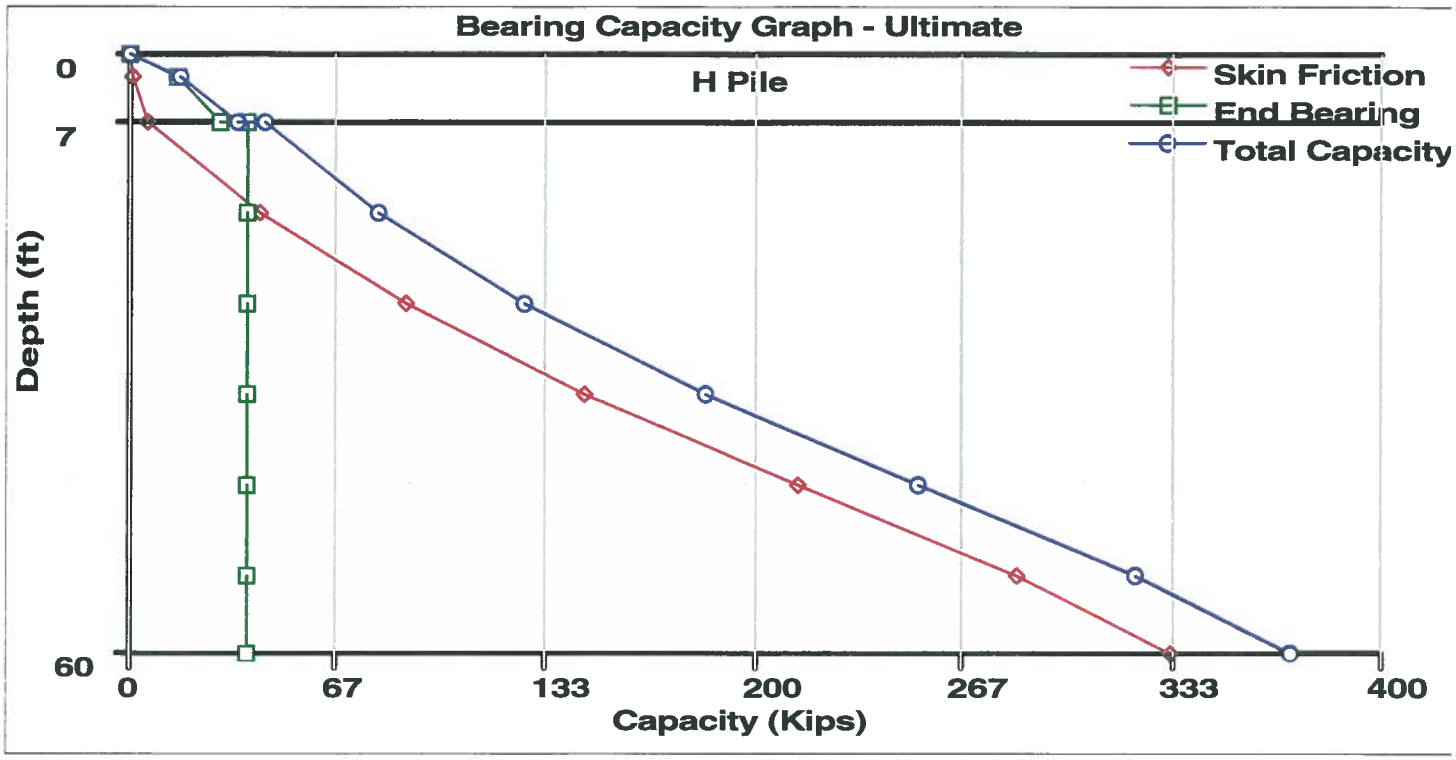
ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	7.00 ft	0.00%	120.00 pcf	34.0/34.0	Nordlund
2	Cohesive	53.00 ft	0.00%	125.00 pcf	3000.00 psf	T-79 Steel

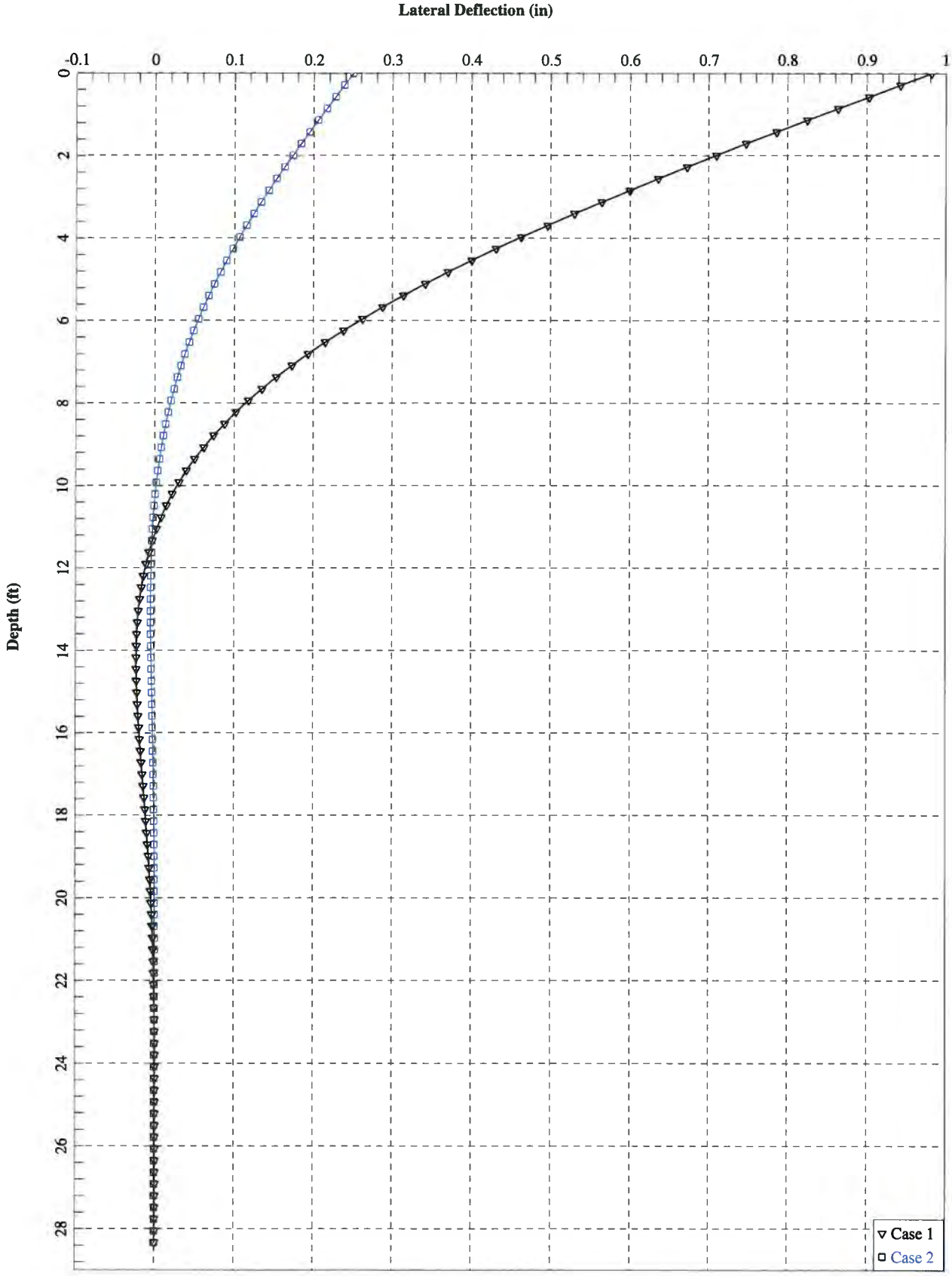
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.06 Kips	0.06 Kips
2.49 ft	0.93 Kips	15.54 Kips	16.47 Kips
2.51 ft	0.95 Kips	15.63 Kips	16.58 Kips
6.99 ft	5.77 Kips	29.05 Kips	34.82 Kips
7.01 ft	5.82 Kips	38.11 Kips	43.93 Kips
16.01 ft	41.86 Kips	38.11 Kips	79.97 Kips
25.01 ft	88.60 Kips	38.11 Kips	126.70 Kips
34.01 ft	146.02 Kips	38.11 Kips	184.13 Kips
43.01 ft	214.14 Kips	38.11 Kips	252.25 Kips
52.01 ft	283.96 Kips	38.11 Kips	322.07 Kips
59.99 ft	333.28 Kips	38.11 Kips	371.39 Kips

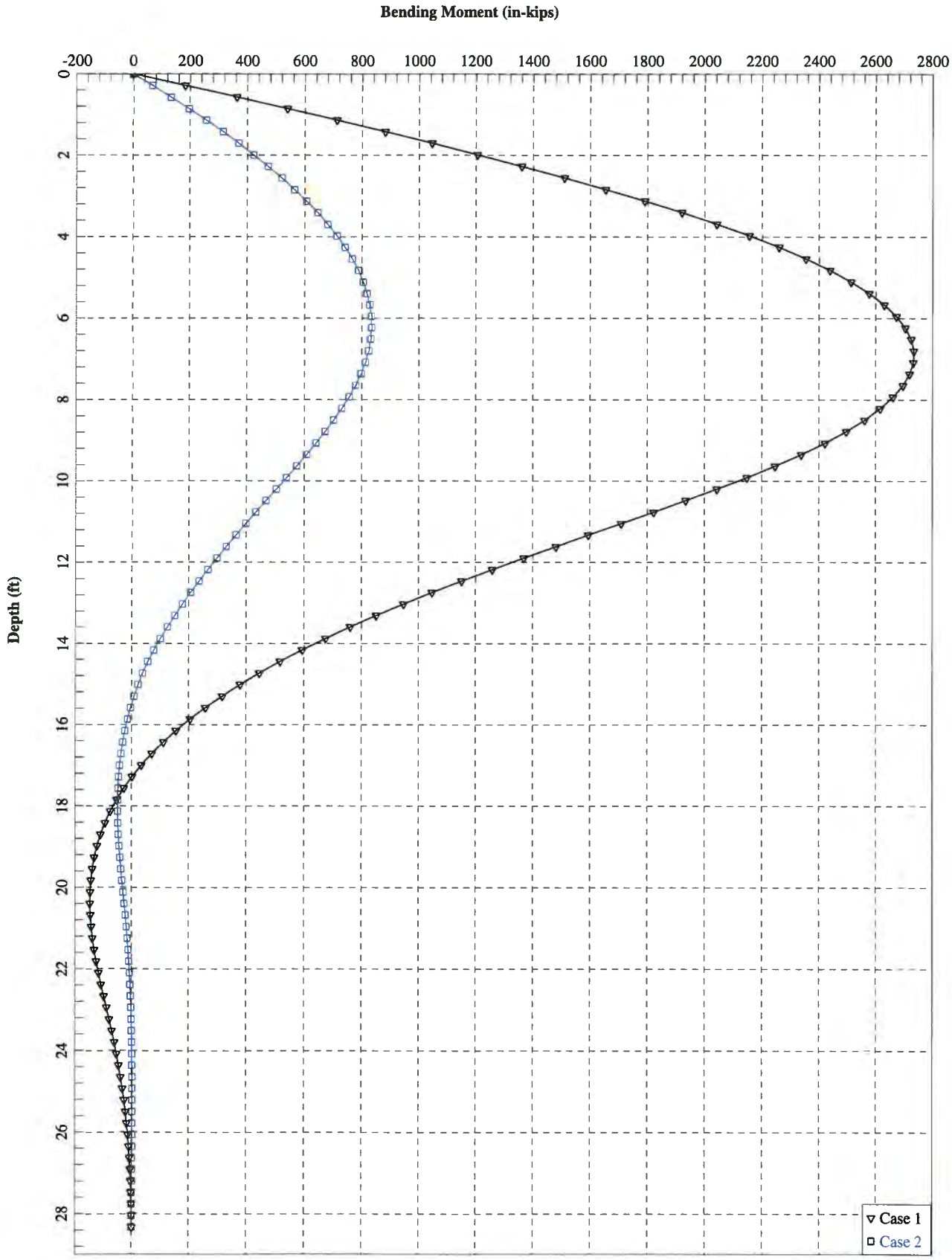
Filename: Z:\ACTIVE~1\1274~1.1RO\ENGINE~1\DRAFTF~1\AB6HP.DVN



Thomes Creek Bridge - Abut 1 Longitudinal HP14x89 Strong Axis

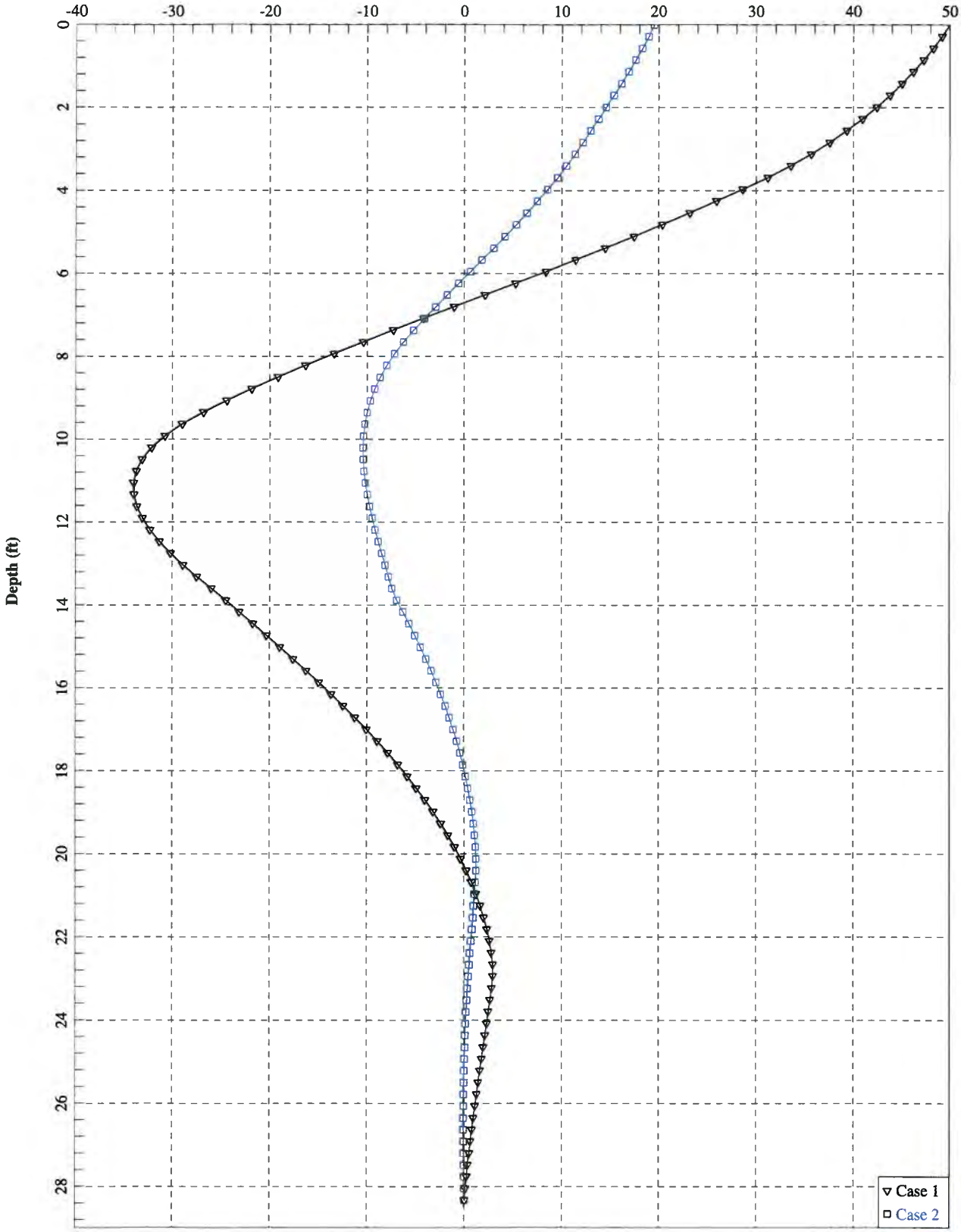


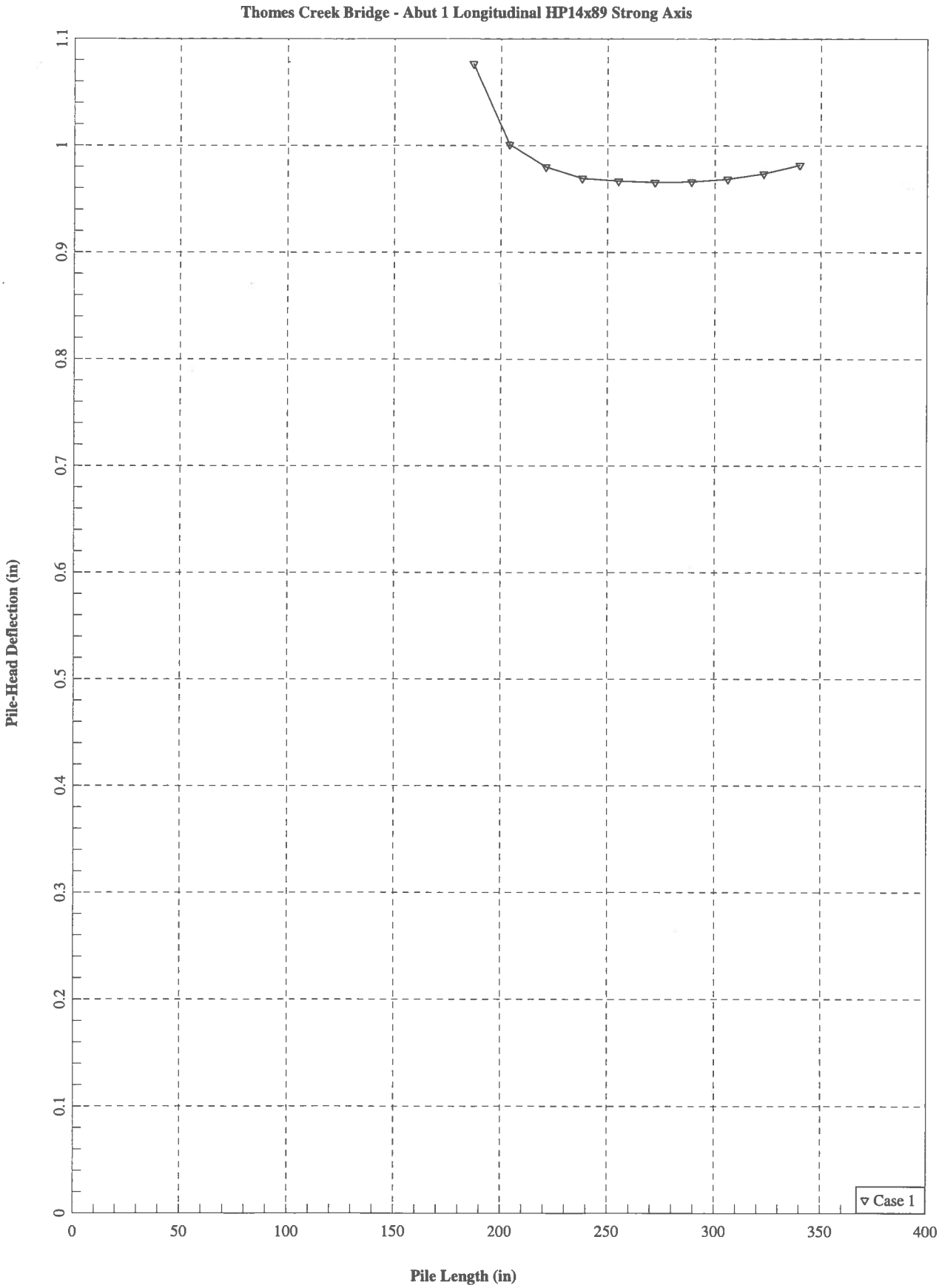
Thomes Creek Bridge - Abut 1 Longitudinal HP14x89 Strong Axis



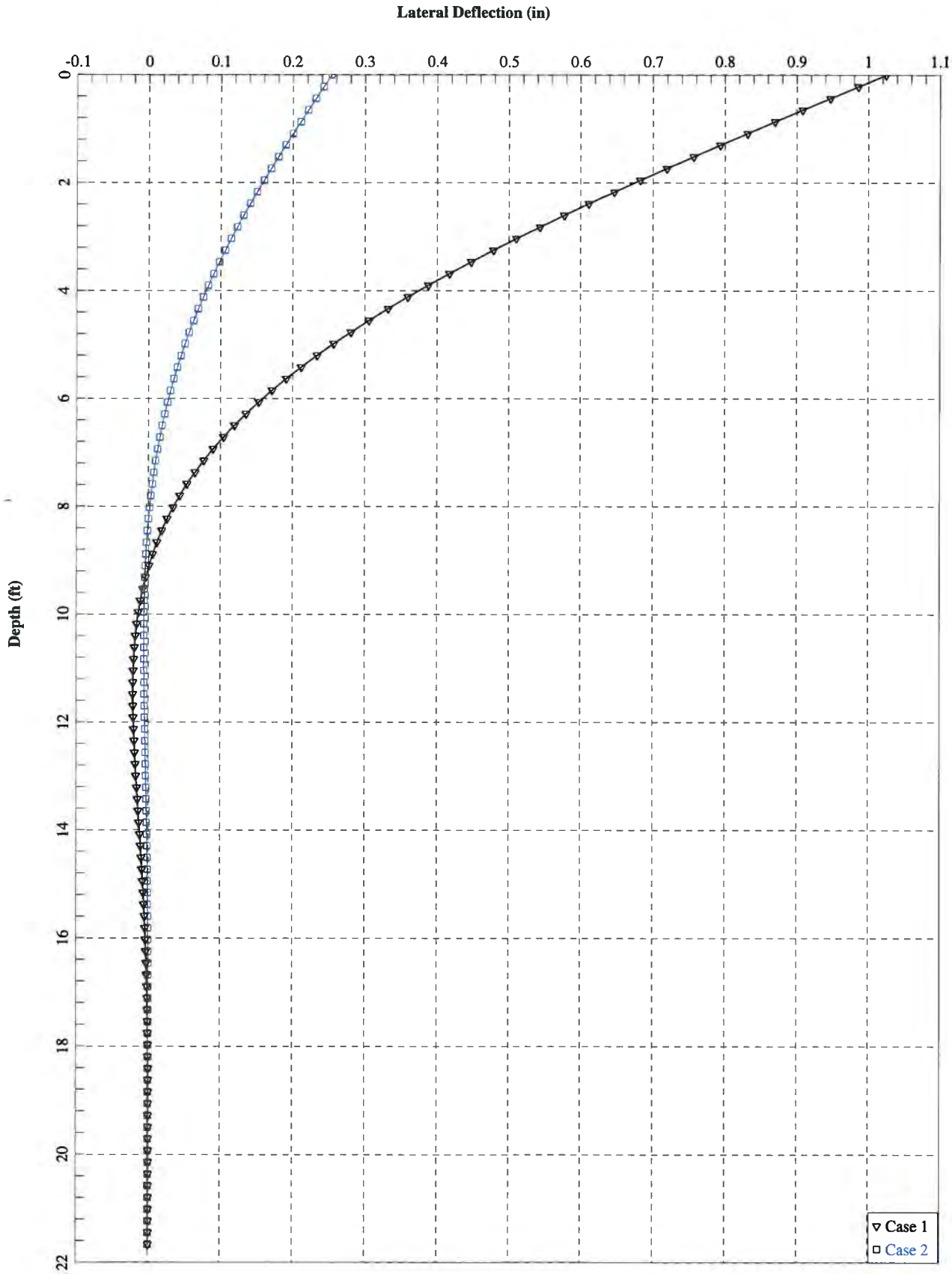
Thomes Creek Bridge - Abut 1 Longitudinal HP14x89 Strong Axis

Shear Force (kips)



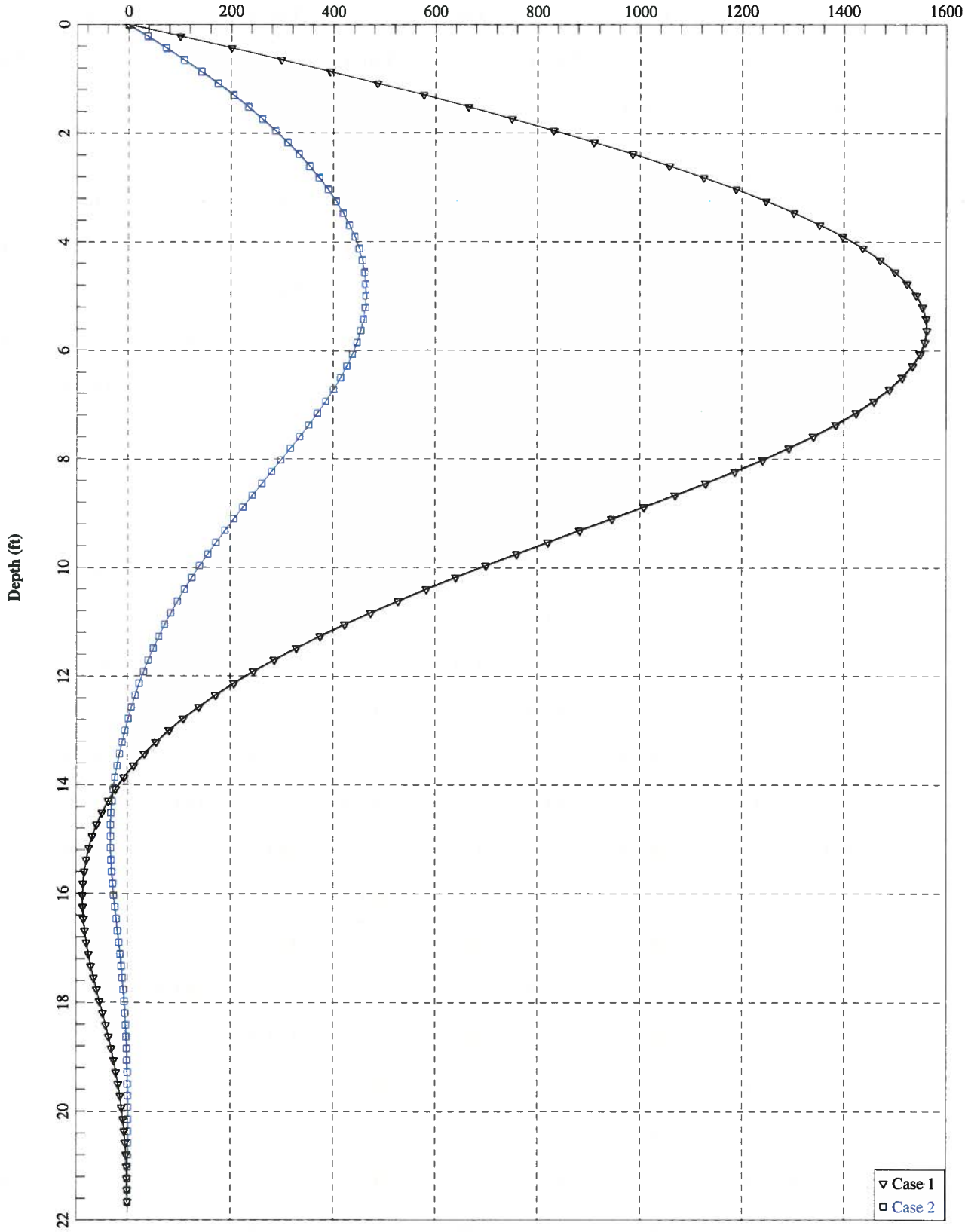

Thomes Creek Bridge - Abut 1 Longitudinal HP14x89 Strong Axis
Pile-Head Deflection (in)
Pile Length (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
0
50
100
150
200
250
300
350
400
Case 1

Abut 1 Transverse HP14x89 Weak Axis



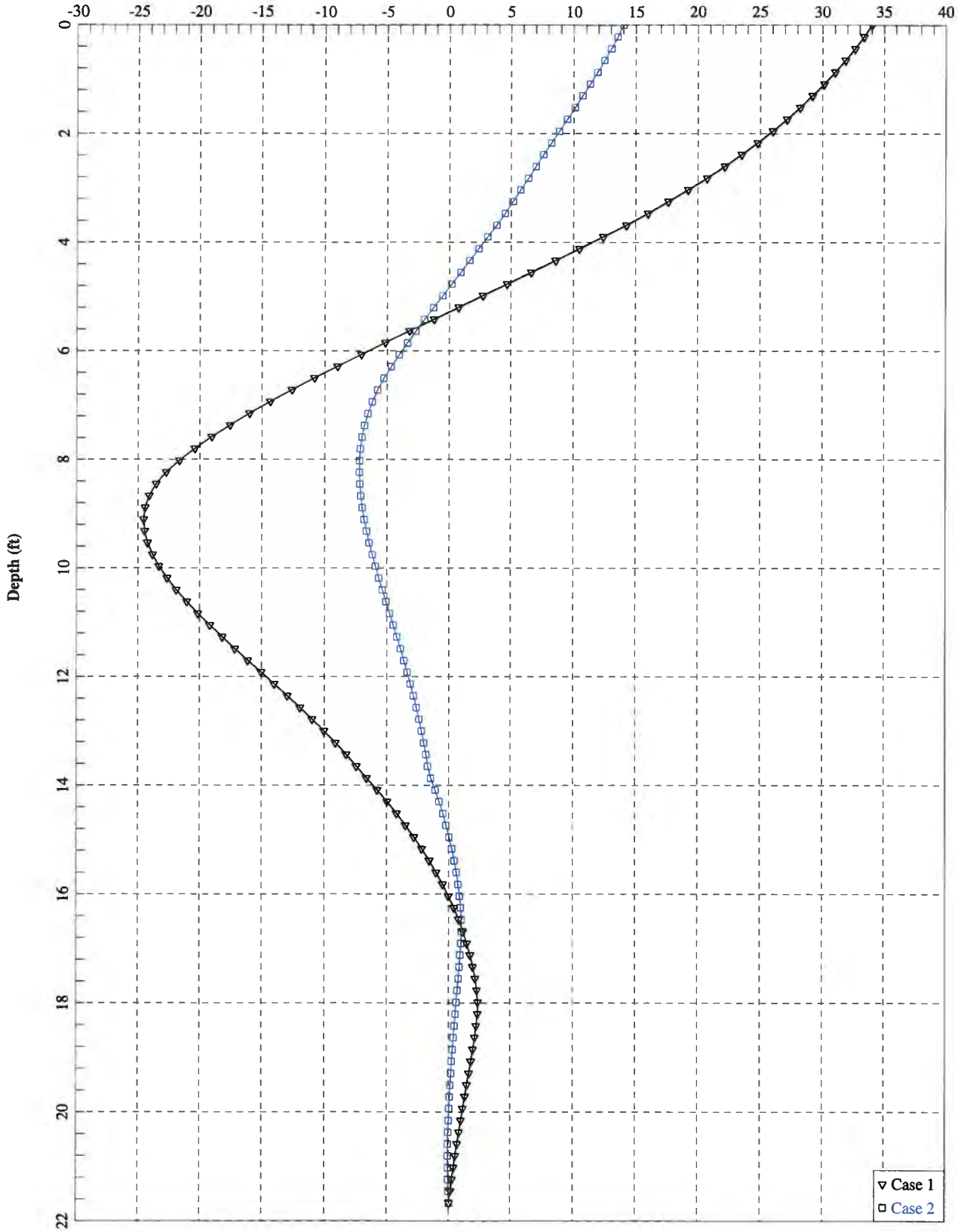
Abut 1 Transverse HP14x89 Weak Axis

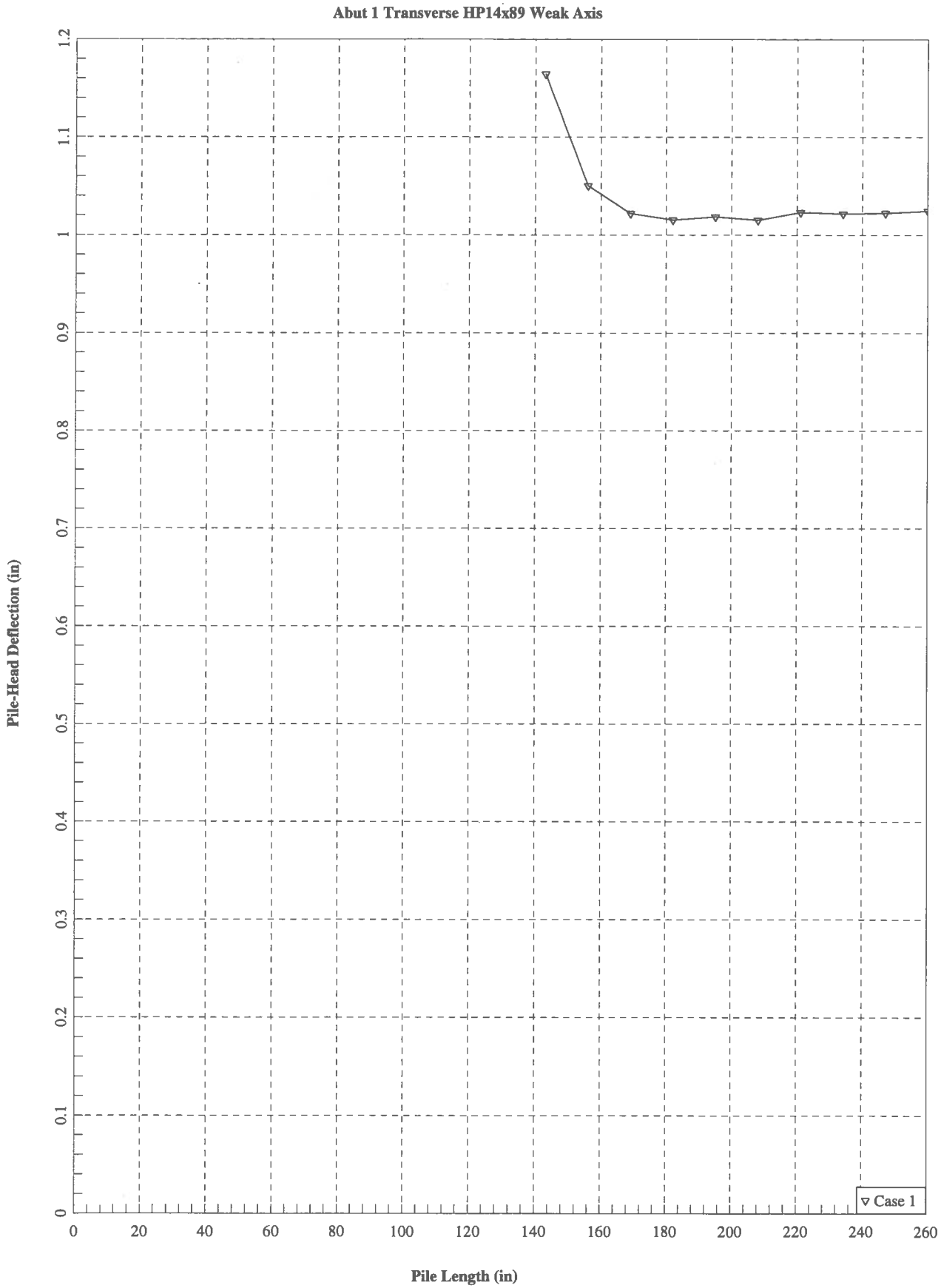
Bending Moment (in-kips)



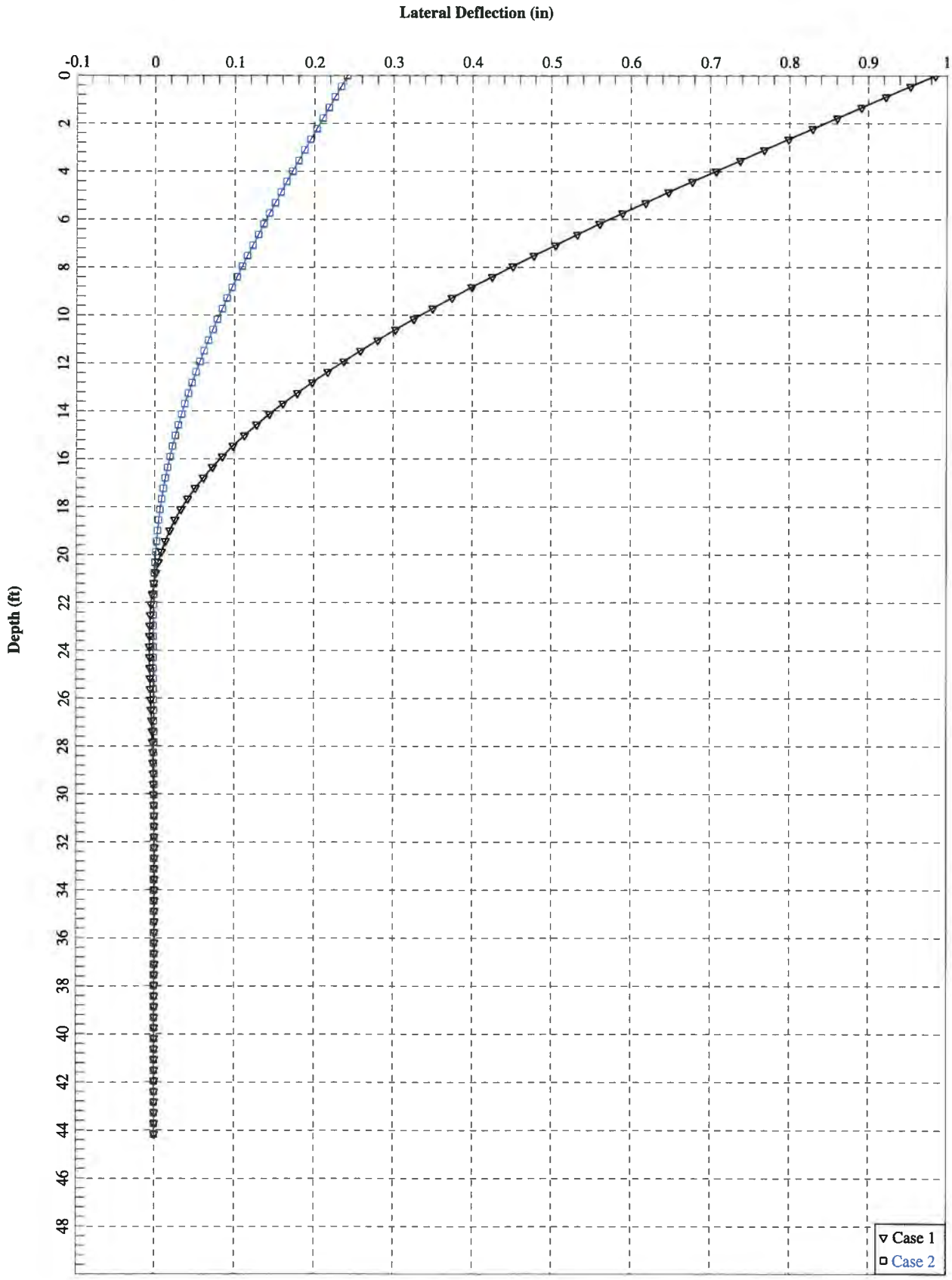
Abut 1 Transverse HP14x89 Weak Axis

Shear Force (kips)



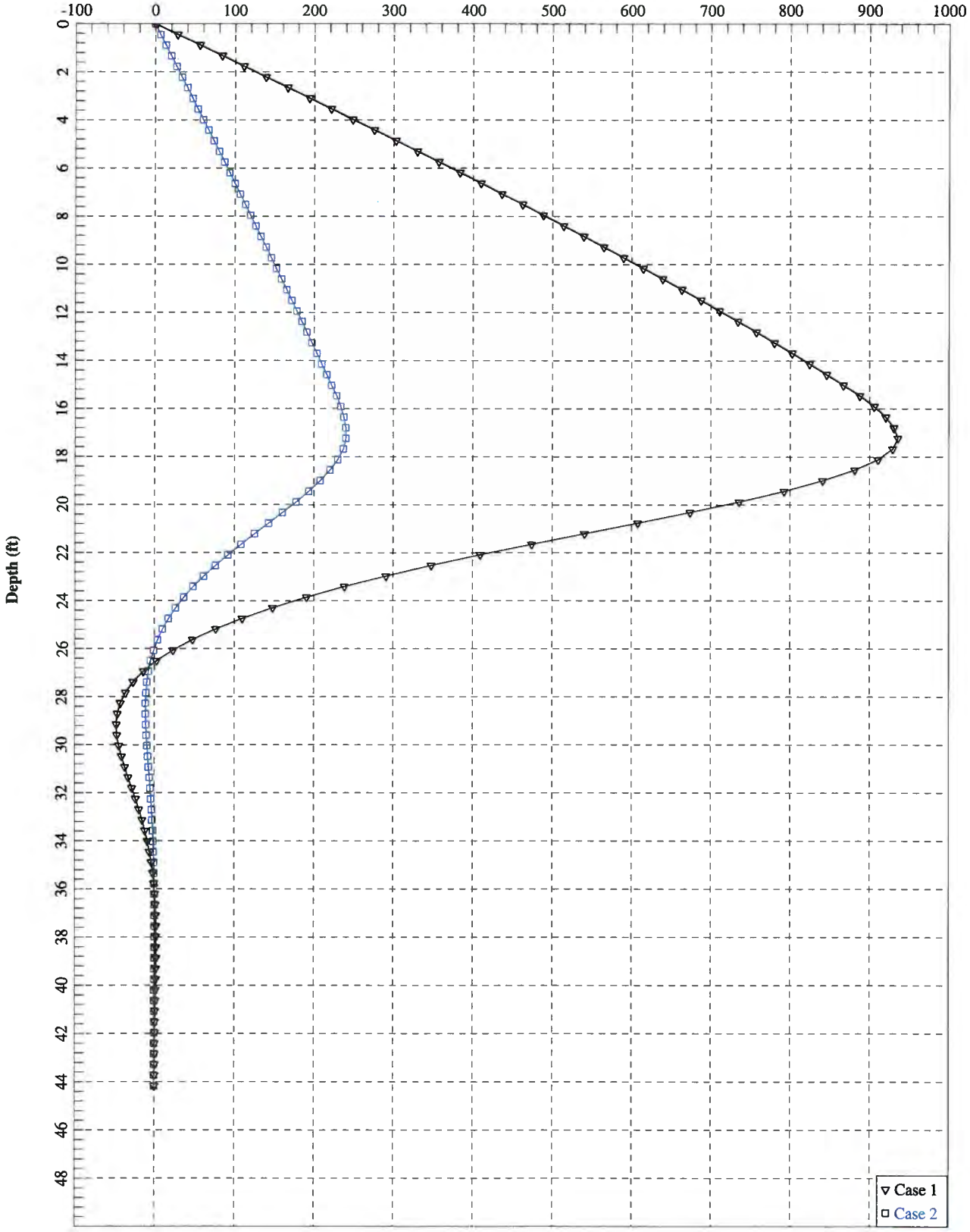

Abut 1 Transverse HP14x89 Weak Axis
Pile-Head Deflection (in)
Pile Length (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
0
20
40
60
80
100
120
140
160
180
200
220
240
260
Case 1

Thomes Creek Bridge - Piers 2-5 Longitudinal HP14x89 Strong Axis



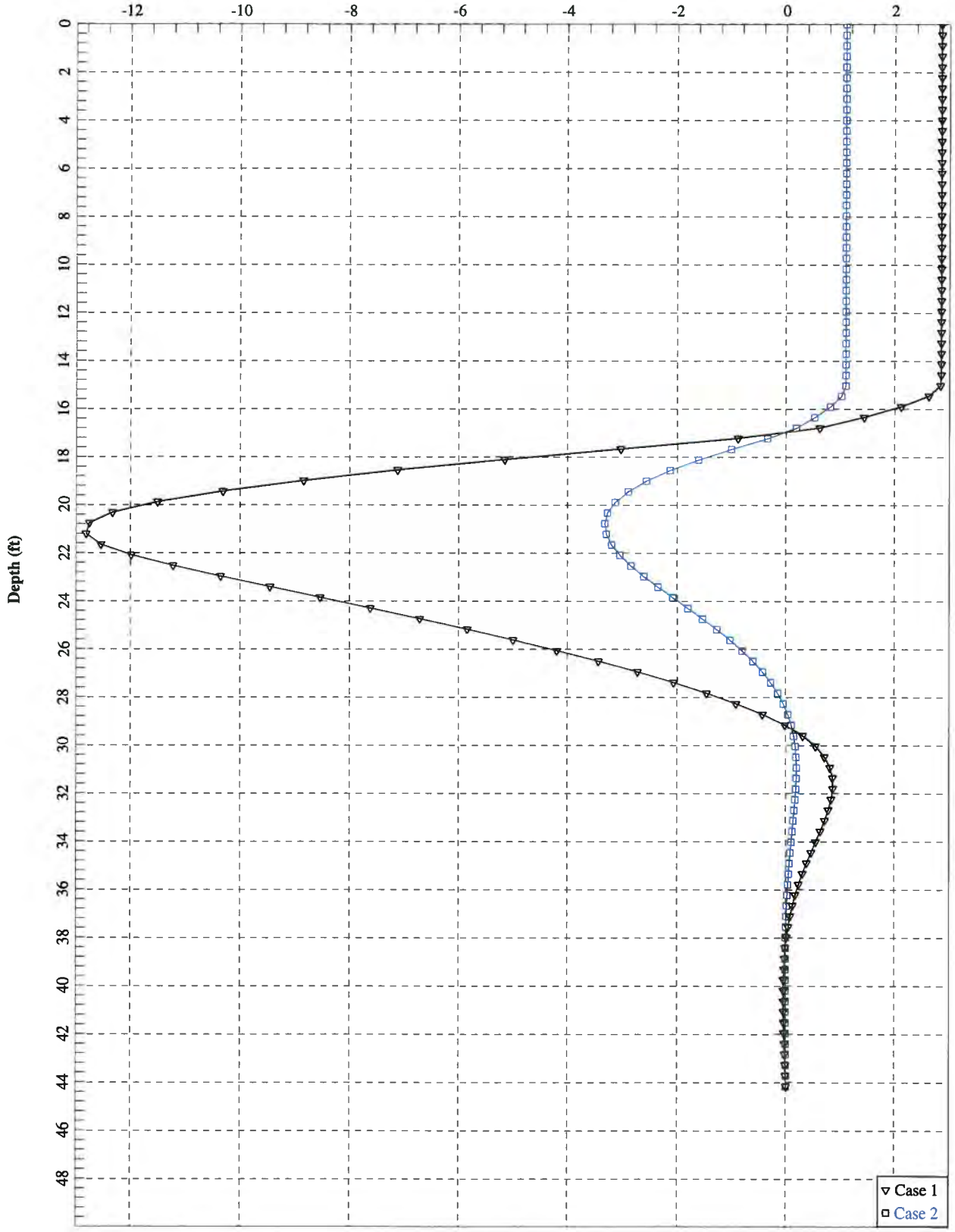
Thomes Creek Bridge - Piers 2-5 Longitudinal HP14x89 Strong Axis

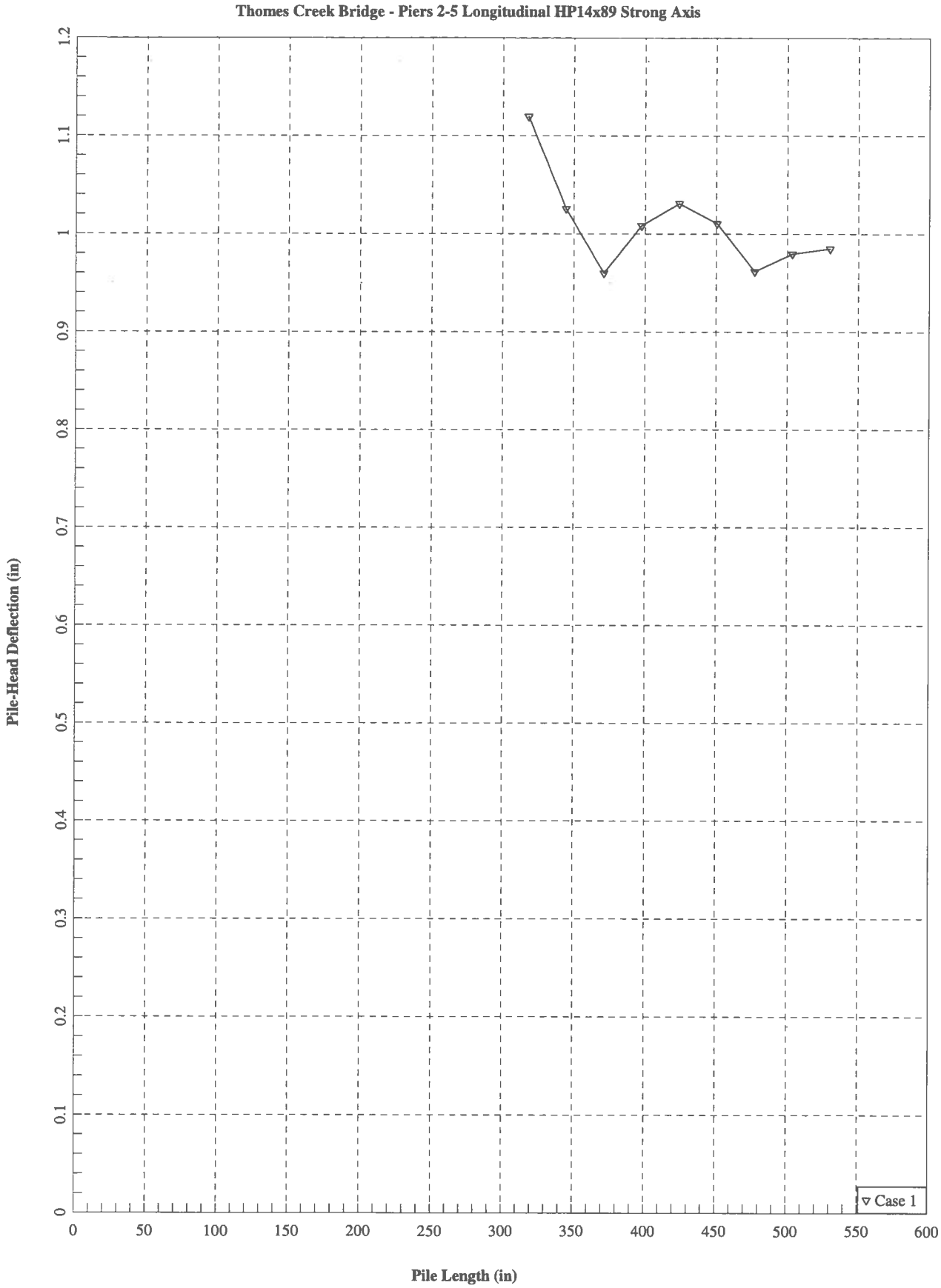
Bending Moment (in-kips)



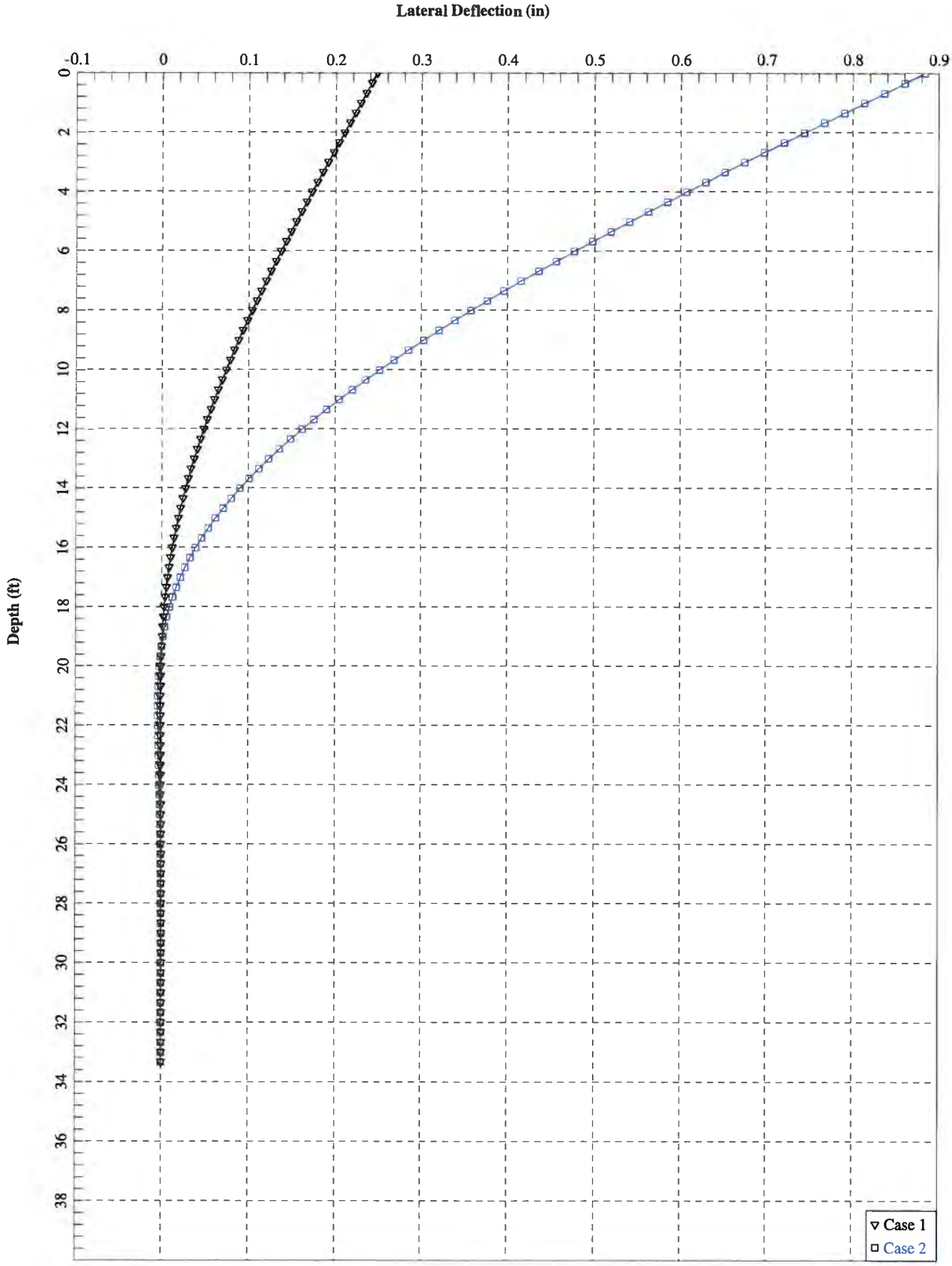
Thomes Creek Bridge - Piers 2-5 Longitudinal HP14x89 Strong Axis

Shear Force (kips)



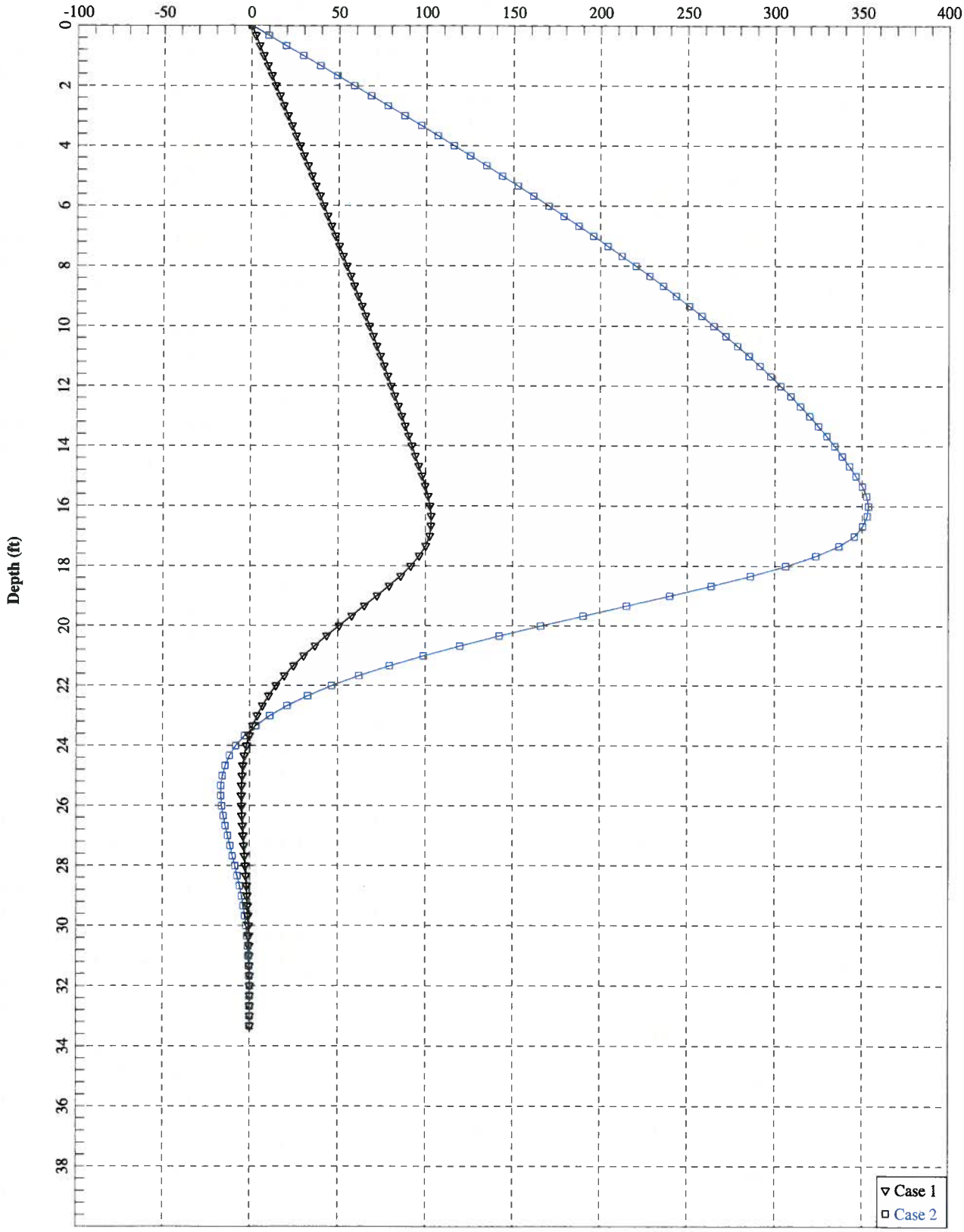

Thomes Creek Bridge - Piers 2-5 Longitudinal HP14x89 Strong Axis
Pile-Head Deflection (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
0
50
100
150
200
250
300
350
400
450
500
550
600
Pile Length (in)
Case 1

Thomes Creek Bridge - Piers 2-5 Transverse HP14x89 Weak Axis



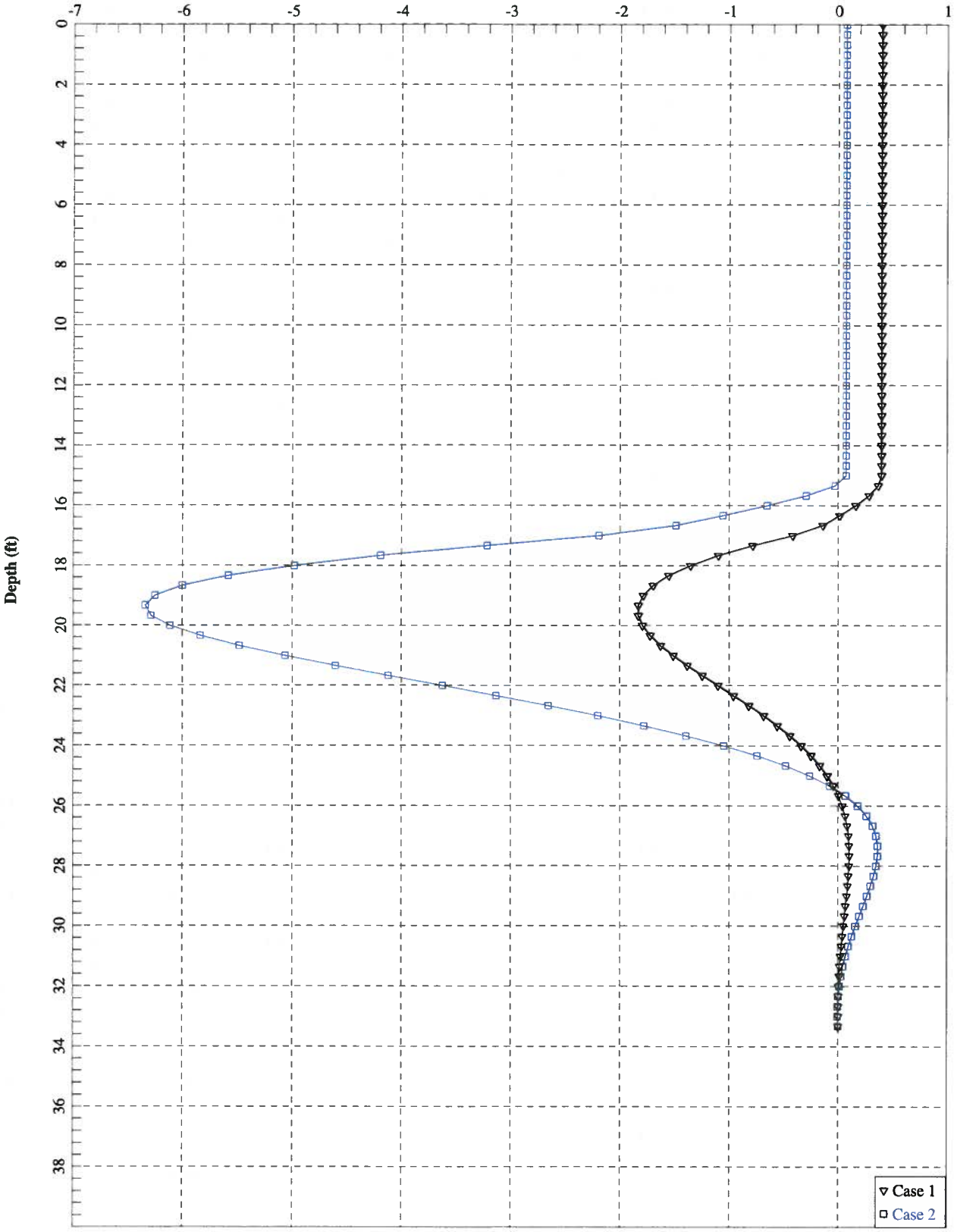
Thomes Creek Bridge - Piers 2-5 Transverse HP14x89 Weak Axis

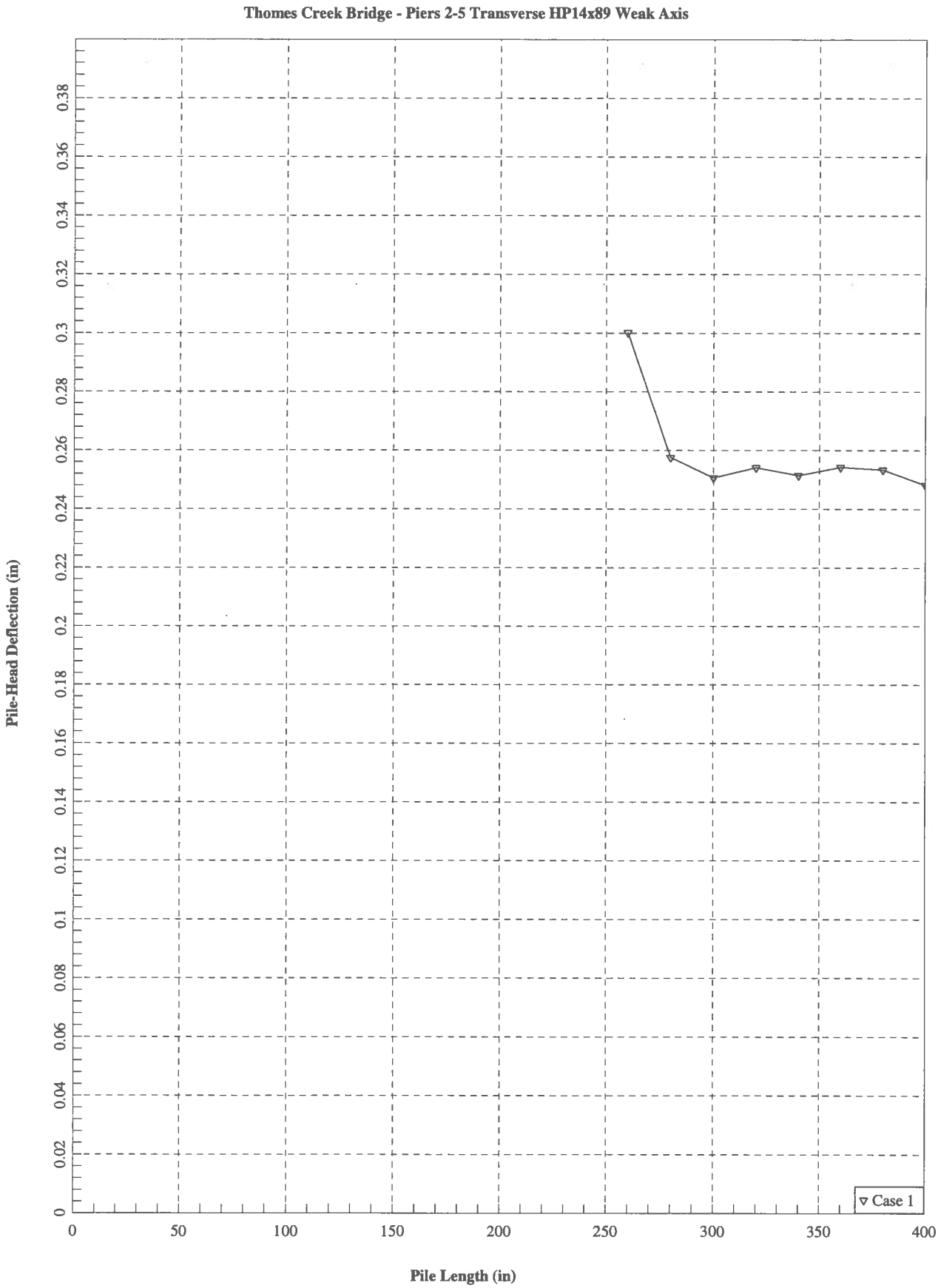
Bending Moment (in-kips)



Thomes Creek Bridge - Piers 2-5 Transverse HP14x89 Weak Axis

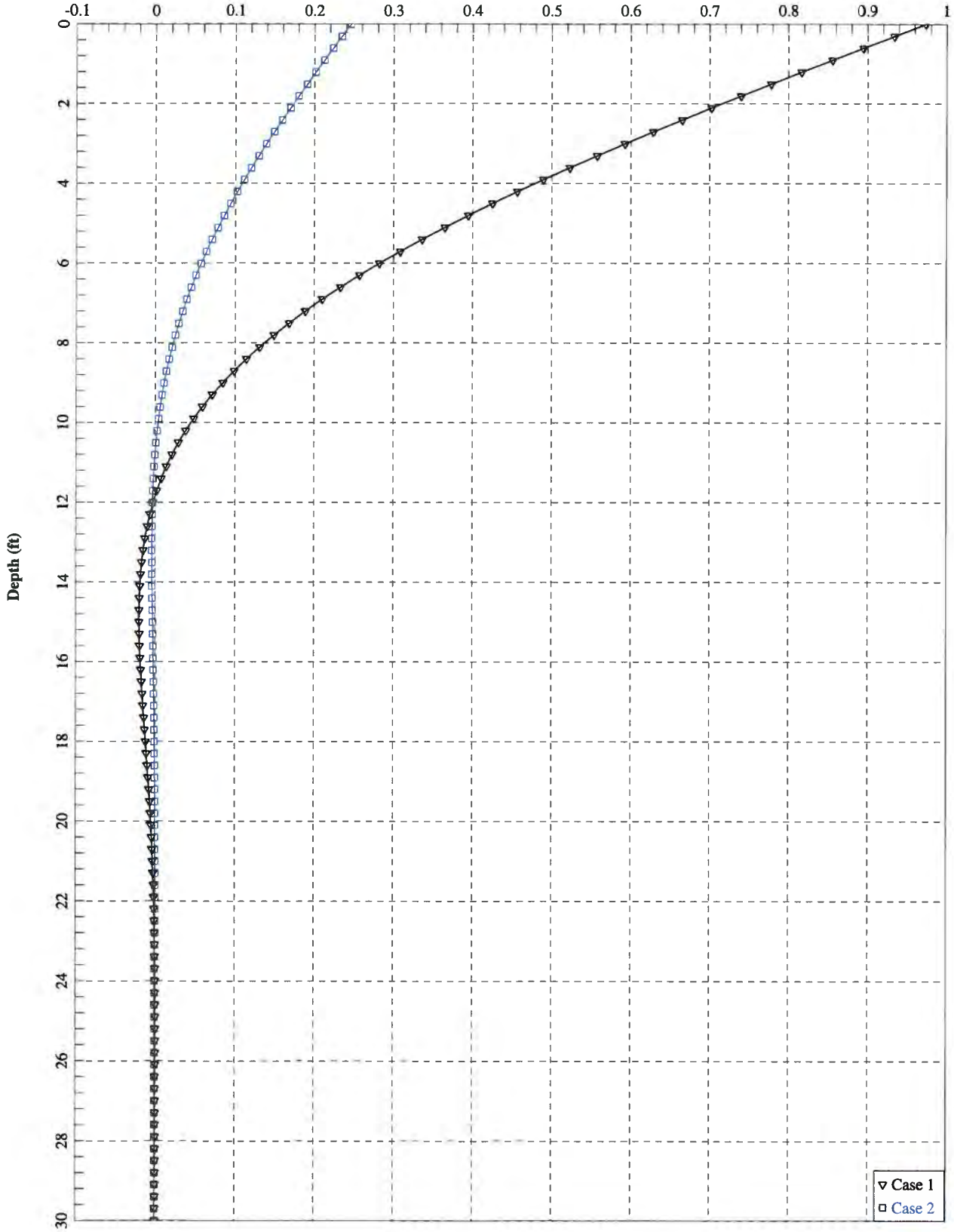
Shear Force (kips)




Thomes Creek Bridge - Piers 2-5 Transverse HP14x89 Weak Axis
Pile-Head Deflection (in)
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
0.16
0.18
0.2
0.22
0.24
0.26
0.28
0.3
0.32
0.34
0.36
0.38
0
50
100
150
200
250
300
350
400
Pile Length (in)
Case 1

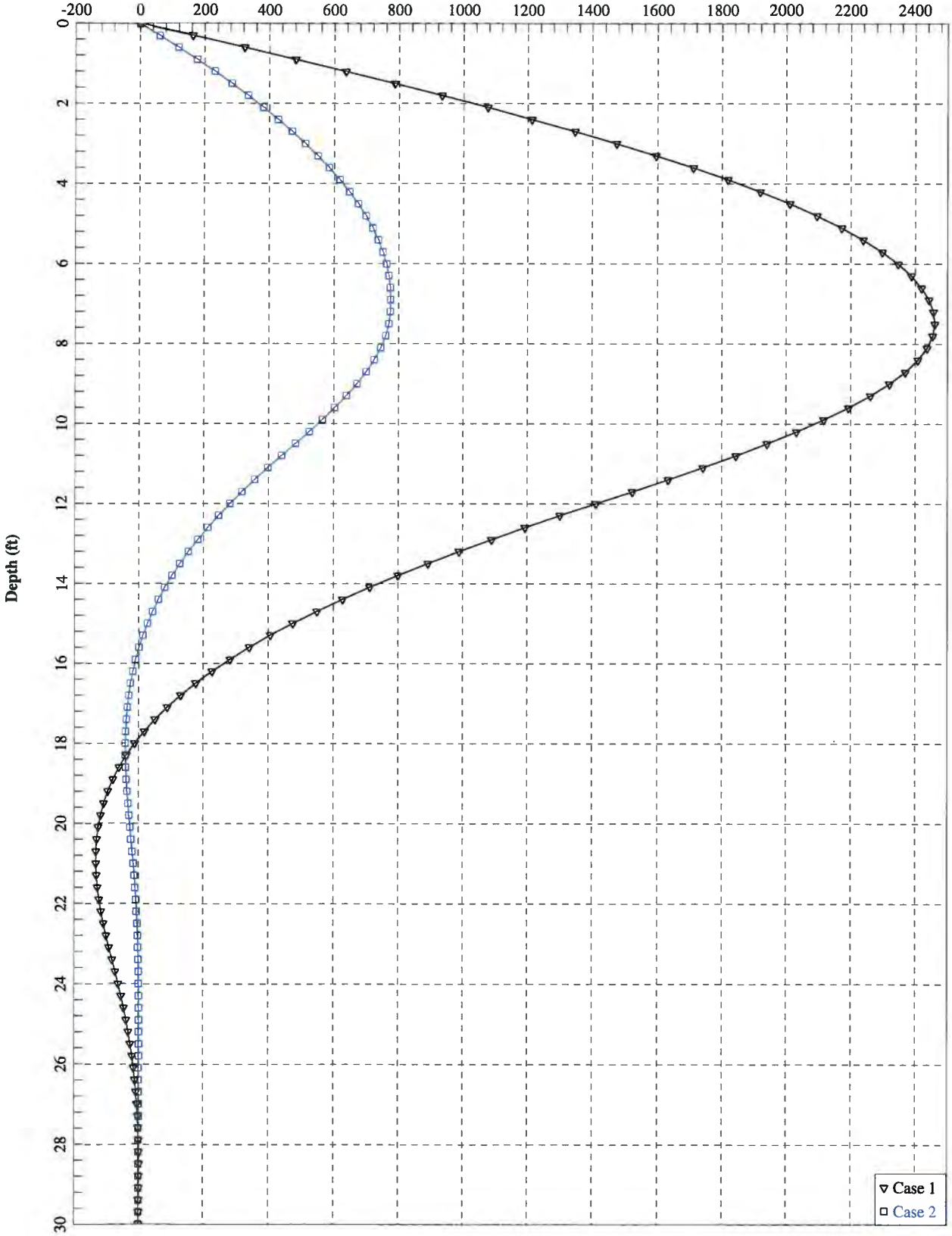
Thomes Creek Bridge -Abut 6 Longitudinal HP14x89 Strong Axis

Lateral Deflection (in)



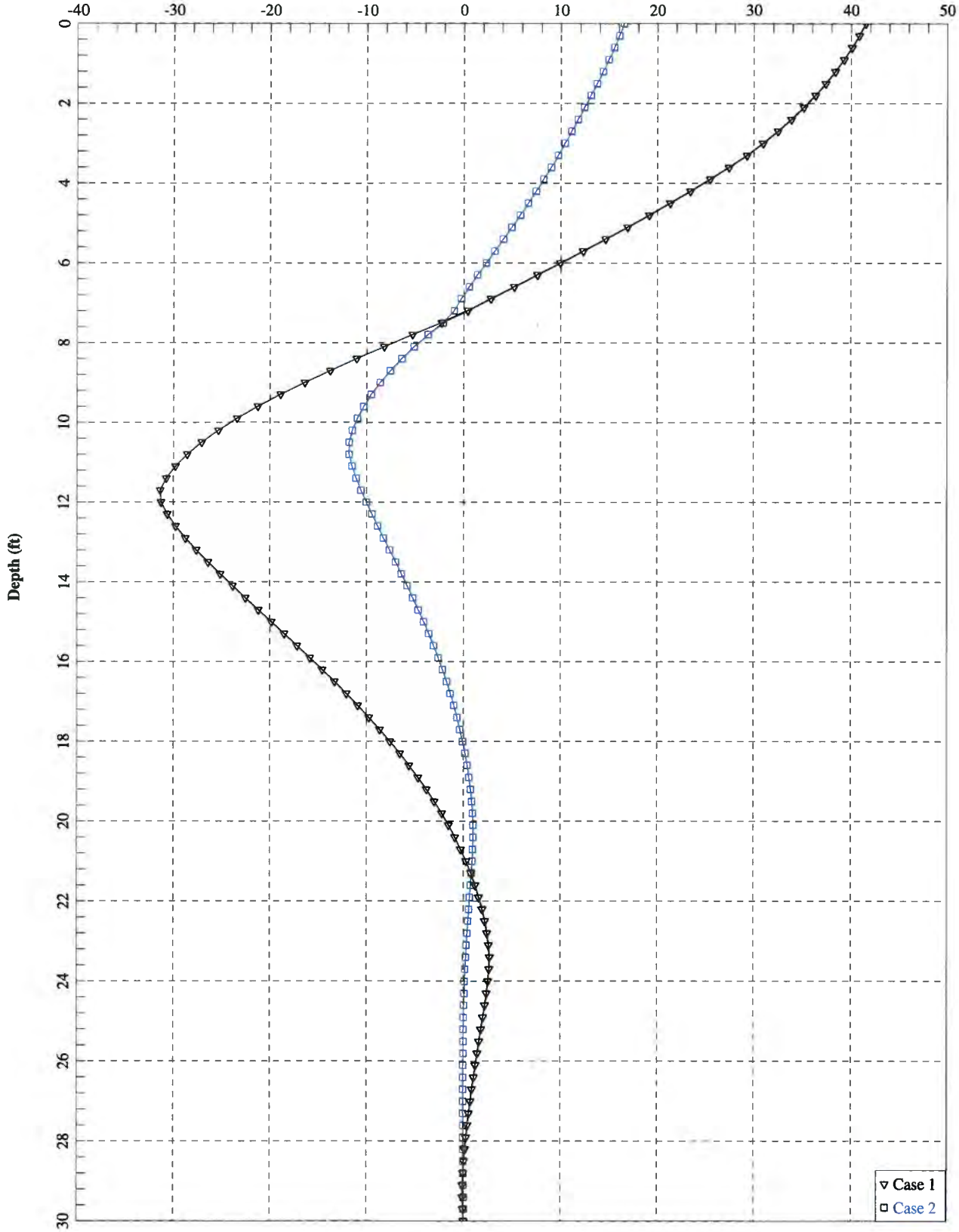
Thomes Creek Bridge -Abut 6 Longitudinal HP14x89 Strong Axis

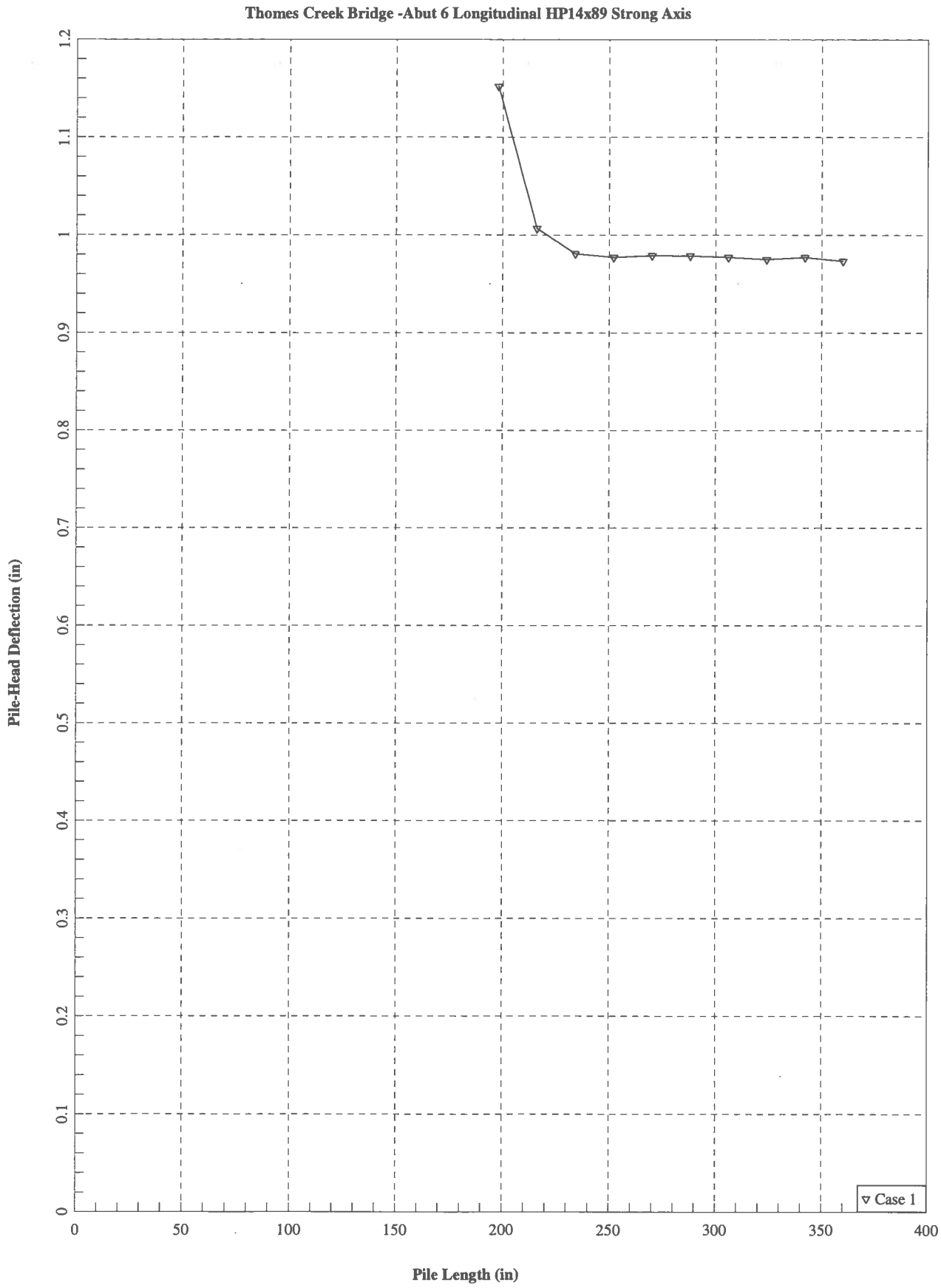
Bending Moment (in-kips)



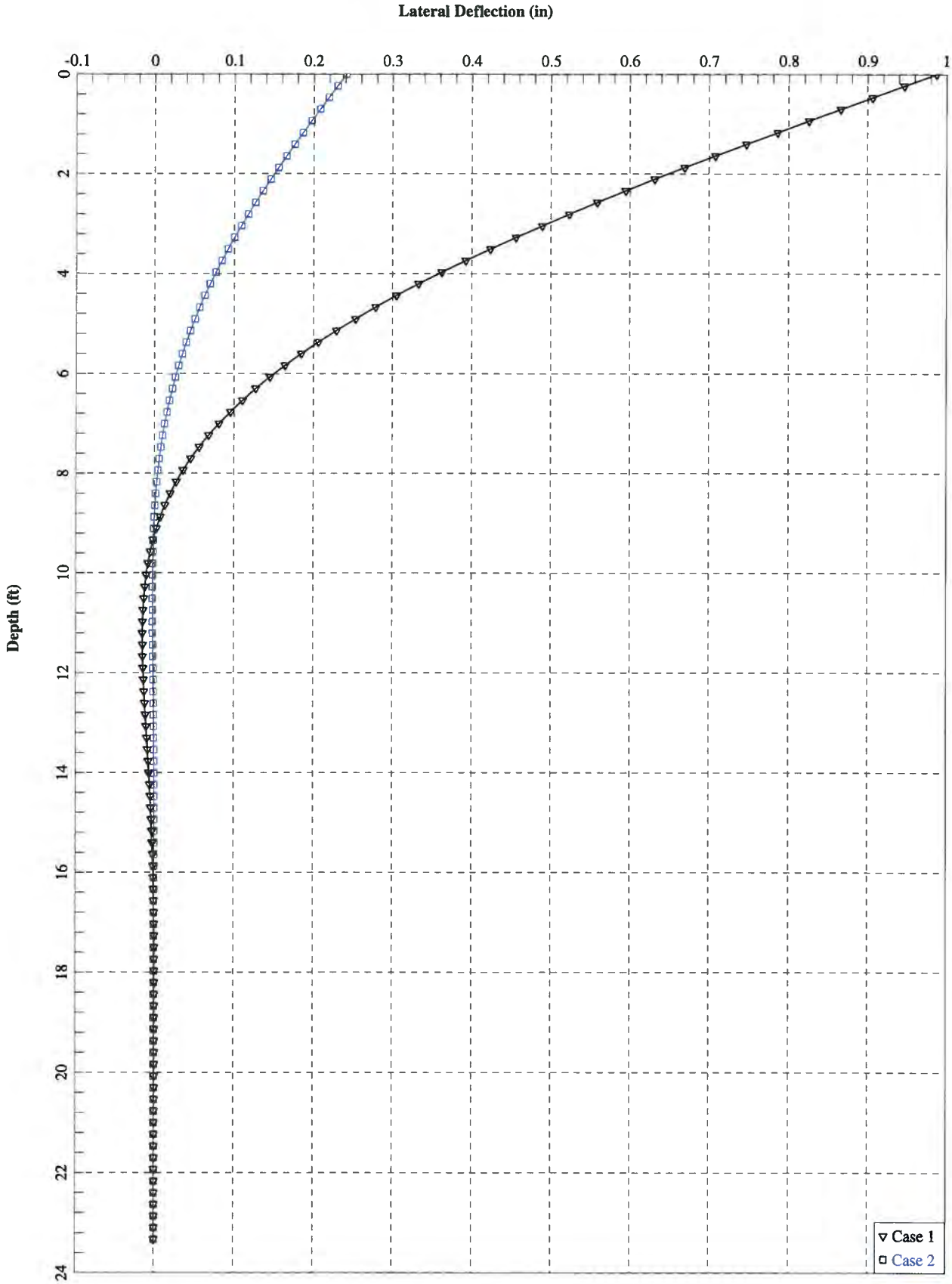
Thomes Creek Bridge -Abut 6 Longitudinal HP14x89 Strong Axis

Shear Force (kips)



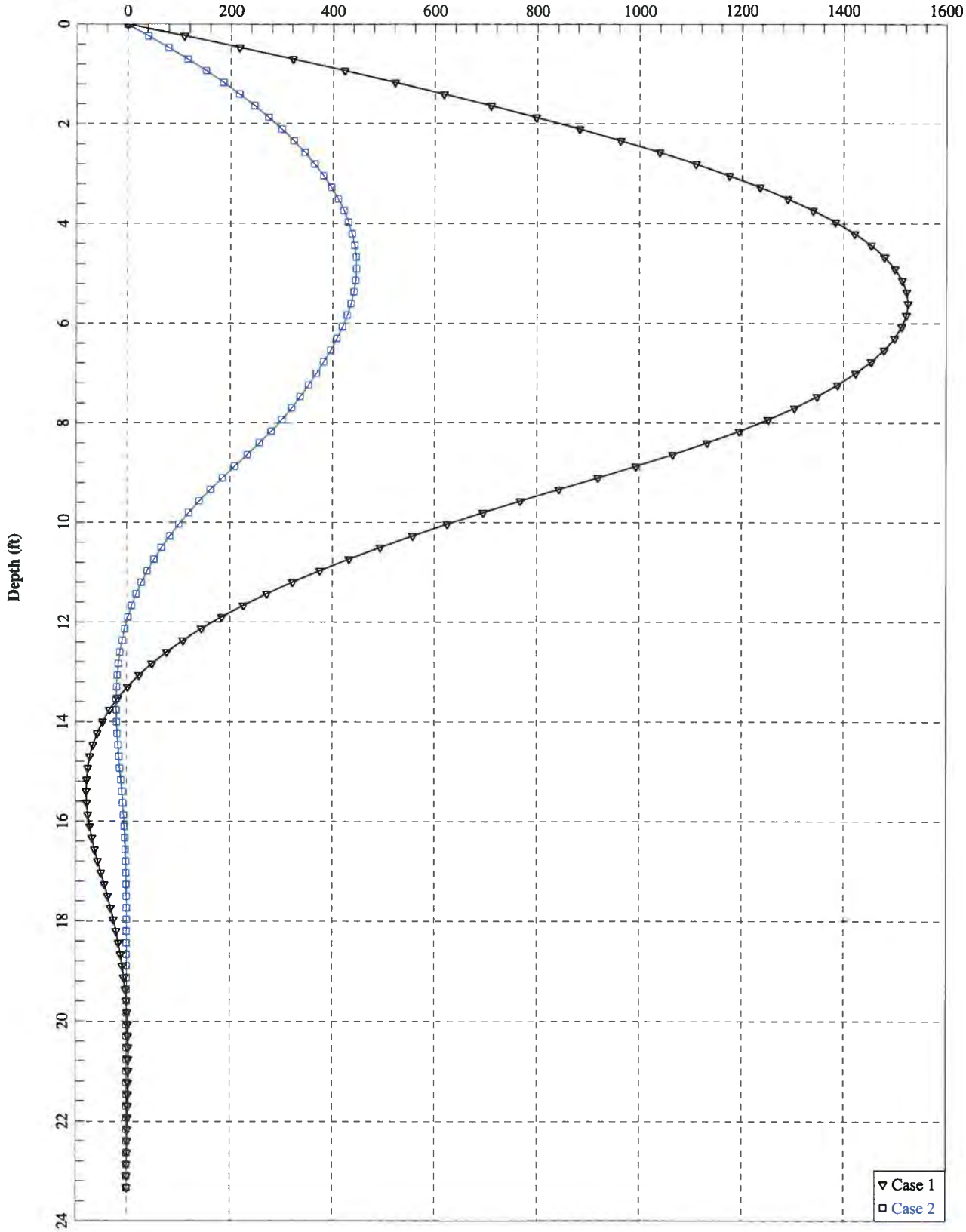

Thomes Creek Bridge -Abut 6 Longitudinal HP14x89 Strong Axis
Pile-Head Deflection (in)
Pile Length (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
1.2
0
50
100
150
200
250
300
350
400
Case 1

Thomes Creek Bridge - Abut 6 Transverse HP14x89 Weak Axis



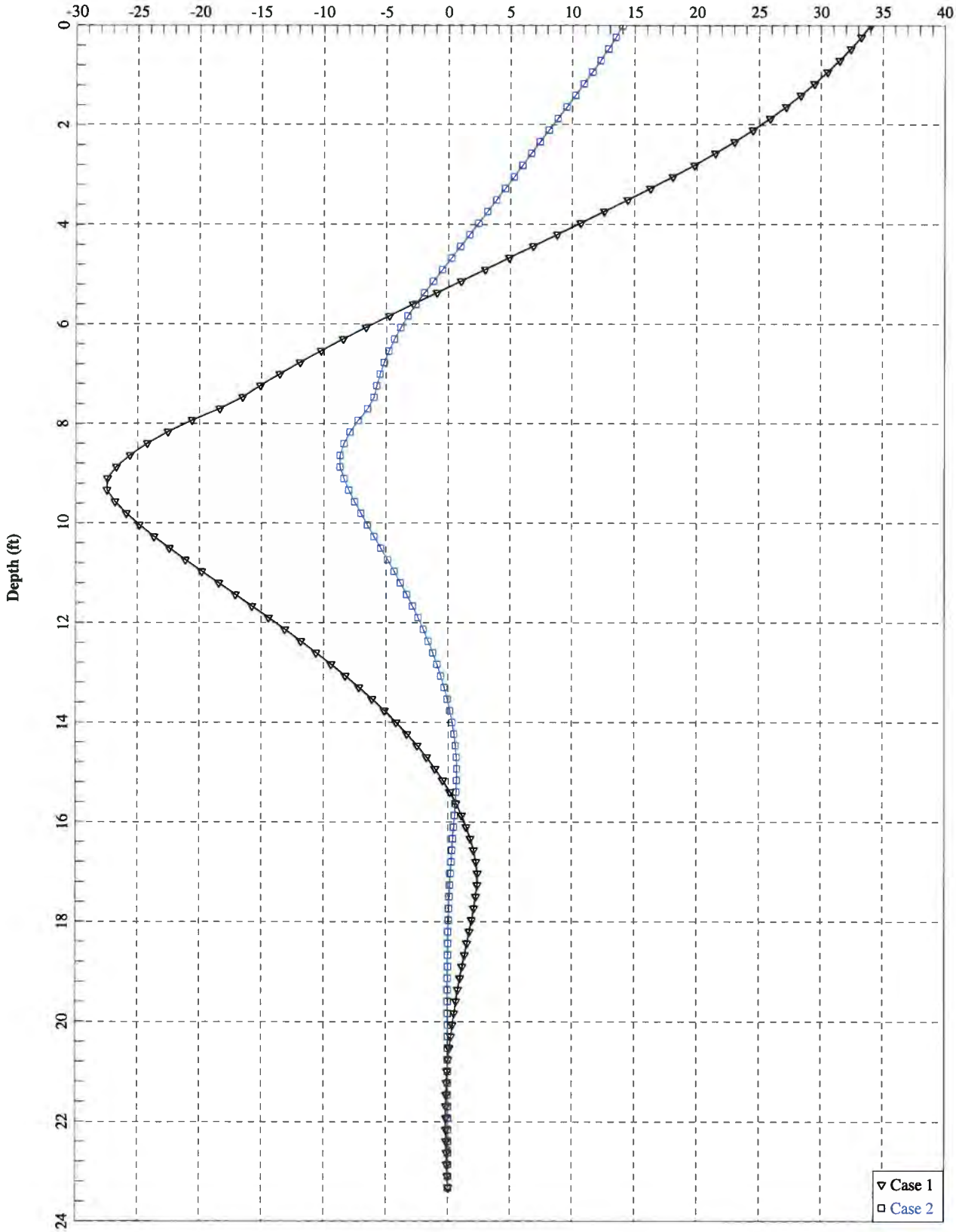
Thomes Creek Bridge - Abut 6 Transverse HP14x89 Weak Axis

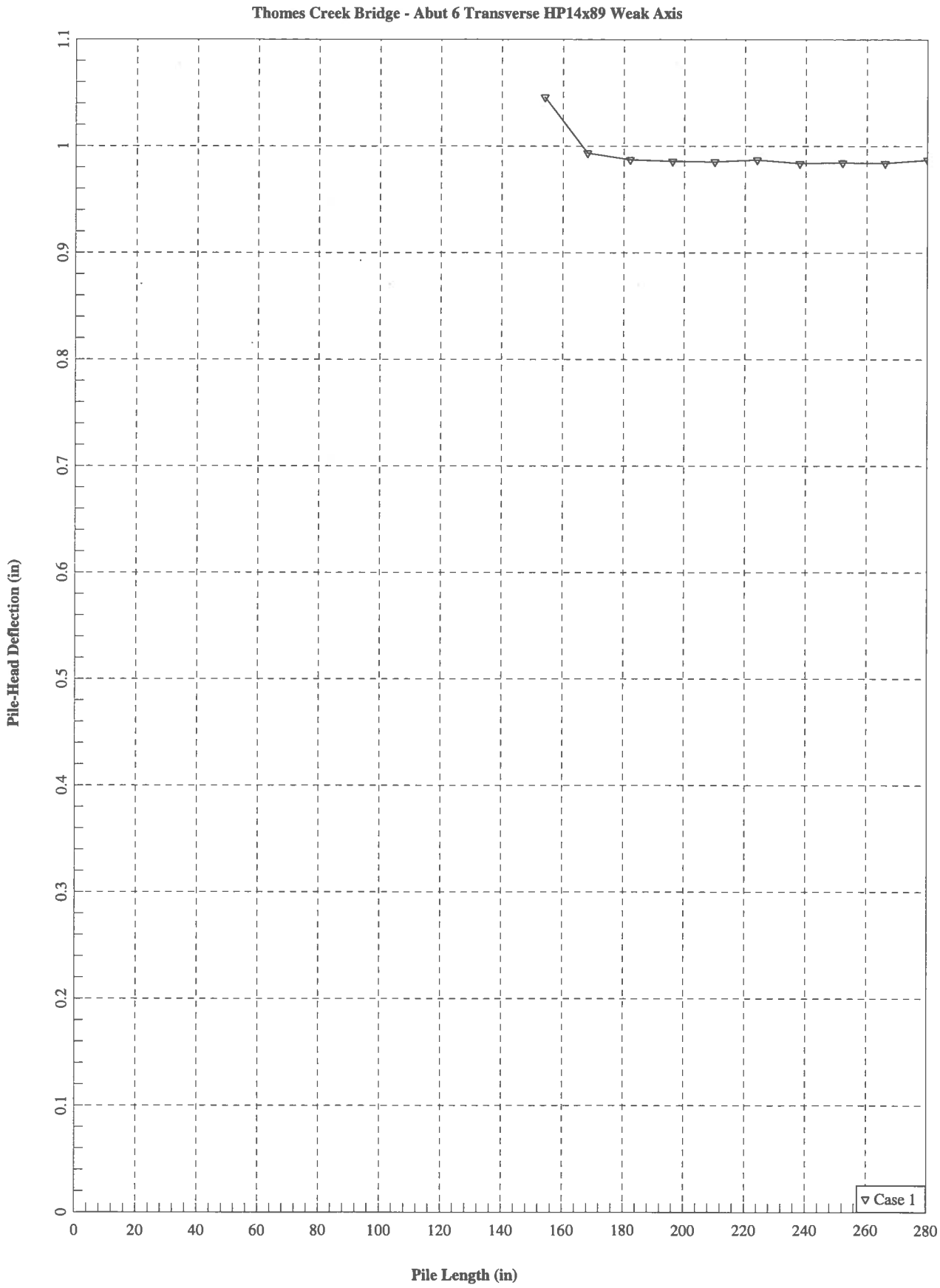
Bending Moment (in-kips)



Thomes Creek Bridge - Abut 6 Transverse HP14x89 Weak Axis

Shear Force (kips)




Thomes Creek Bridge - Abut 6 Transverse HP14x89 Weak Axis
Pile-Head Deflection (in)
Pile Length (in)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
Case 1