

FOUNDATION REPORT

N. State Street Bridge at Ackerman Creek

Bridge No. 10C-0065
Mendocino County, CA

November 2023

Prepared for:



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File No. 2103.1
November 9, 2023

Mr. Maxwell Katt, PE
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Subject: **FOUNDATION REPORT**
North State Street Bridge at Ackerman Creek
Bridge No. 10C-0065
Mendocino County, California

Dear Mr. Katt:

Blackburn Consulting prepared this Foundation Report for the North State Street bridge project in Mendocino County, California. This report addresses Consor's comments on our Draft Foundation Report. We did not update our analysis to current Caltrans standards based on Consor's direction. Blackburn prepared this report in accordance with our June 14, 2011 agreement between Blackburn and Consor Engineers.

Thank you for the opportunity to be part of your design team. Please call if you have questions or require additional information.

Sincerely,

BLACKBURN CONSULTING



Rob Pickard, P.G., C.E.G.
Senior Engineering Geologist



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1 INTRODUCTION

1.1 Purpose

Blackburn Consulting (Blackburn) prepared this Foundation Report for the North State Street Bridge project at Ackerman Creek in Mendocino County, California. This report contains our findings and recommendations for design of new bridge foundations.

This report is for Mendocino County and Consor Engineers (Conсор) to use during design and construction of the bridge replacement. Do not use or rely upon this report for different locations or improvements without the written consent of Blackburn.

1.2 Scope of Services

To prepare this report, Blackburn:

- Discussed the project with Consor and Mendocino County Department of Transportation (DOT).
- Reviewed available Bridge Inspection Reports (BIR) and construction drawings for the existing North State Street bridge and the upstream SR101 bridge at Ackerman Creek.
- Reviewed Advanced Planning Study (APS) drawings prepared by Consor.
- Reviewed published maps and literature related to site geologic and seismic conditions.
- Conducted a preliminary geologic site reconnaissance within the project limits.
- Drilled and sampled two borings to maximum depth of 90 feet below existing road grade.
- Performed laboratory testing on soil samples retrieved from the borings.
- Performed engineering and seismic analysis to provide recommendations for structure foundations and approaches.

2 PROJECT DESCRIPTION

2.1 Project Location

The project is located on the north side of Ukiah, California on North State Street, approximately 1,200 feet north of Orr Springs Road. Site coordinates are approximately latitude 39.1793°N and longitude 123.2099°W. The site is about 800 feet downstream of the SR101 bridge across Ackerman Creek. Figure 1 shows the bridge location.

2.2 Site Description

Ackerman Creek, at this location, flows east with the active channel presently centered near the north pier of the existing bridge.

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The existing bridge was built in 1965 and is indicated in a Bridge Inspection Report (BIR) dated 3/22/66 to have replaced a 1913 bridge that was “severely damaged by scour in the floods of December 1964”.

The existing bridge is:

- A 3-span, reinforced concrete slab structure.
- 146 feet long and 34 feet wide.
- Supported with solid (thin-wall) piers and seat abutments.
- Supported on reinforced concrete piles cast in driven steel casings.
- At approximate elevation 628 feet (deck) with channel bottom at about elevation 610-615 feet (approximately 15 feet below bridge deck).

Scour has exposed the top of the north pier (Pier 3) footing and the south pier (Pier 2) footing is covered by a gravel bar.

A log check-dam, held in place with driven H-piles, is located approximately 50 feet downstream (east) of the bridge. Large boulders have been placed on the downstream face of the check dam for erosion/scour protection. A fish ladder is positioned near the center of the check dam.

2.3 Project Description

The County proposes to replace the existing bridge with a structure that is:

- 64 feet wide and 146 feet long.
- Single span, cast-in-place, concrete box girder bridge.
- Bridge deck at approximate elevation 626.5 feet.
- Supported by driven Class 200 Pipe Piles.

Both alternatives will require widened approach fills and pavement sections. The project will remove the fish ladder and replace the existing check-dam with a series of step-pools along a restored, natural channel. Blackburn’s scope did not include evaluation or recommendations for channel modifications.

3 SUBSURFACE EXPLORATION

Previous subsurface work was performed in 1965¹ for the existing bridge and for the SR101 bridge over Ackerman Creek in 1957². We attach the Log of Test Borings (LOTB’s) for this previous work in Appendix A.

To characterize the subsurface conditions and obtain samples for laboratory testing, Blackburn retained Taber Drilling to drill and sample four exploratory borings (R-13-01, R-13-02, A-13-03, and A-13-04) in August 2013. Taber Drilling used a CME 75 truck mounted drill rig to:

¹ Hans Feibusch, Consulting Engineer, drilled in June 1965

² Caltrans Division of Highways, drilled in July 1957

- Advance borings to maximum depth of ± 90 feet below the ground surface (bgs) with 4-inch O.D. solid-stem augers and 94 mm wireline drilling methods.
- Obtain relatively undisturbed samples with Modified California Samplers (equipped with 2.4-inch I.D. brass liners). Samplers were driven into the ground with a 140-pound, automatic trip hammer falling 30 inches.
- Backfill the borings with cement-grout.

Blackburn planned the general location and depth of the borings based on the proposed improvements and existing site conditions. Blackburn's project geologist logged the borings consistent with the Caltrans 2010 Logging Manual. We made groundwater observations in the augered portion of the borings.

Figure 2 shows the boring locations. We include the LOTB and Boring Logs in Appendix A. The LOTB and Boring Logs for this study provides soil descriptions and an explanation of the descriptive terms used to log the soil samples.

4 LABORATORY TESTING

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

- Moisture Content - Dry Density (ASTM D2216 / D2937)
- Particle Size Analysis (ASTM D422)
- Atterberg Limits (ASTM D4318)
- Sulfate/Chloride Content (CTM 417/422)
- pH/Minimum Resistivity (CTM 643)
- Unconsolidated Undrained Triaxial (ASTM 2850)
- Resistance Value (R-value; CTM 301)

We present the laboratory test results on the LOTB and in Appendix B.

5 SITE GEOLOGY

5.1 Site Geology

The project site is located on the north side of Ukiah Valley within the Coast Ranges Province. The Coast Ranges are northwest trending mountain ranges and valleys that consist of thick sequences of Mesozoic and Cenozoic sedimentary rocks.

Mapped geology³ in the project vicinity consists of:

- Quaternary alluvium and terrace deposits, typically sand, silt and clay with variable gravel.
- Pliocene and Pleistocene-age sedimentary rocks of the Cache Formation (typically pebbly sandstone, conglomerate, siltstone and tuff)
- Mesozoic-age sedimentary rocks of the Franciscan Formation (typically sandstone, shale and conglomerate)

We did not observe any bedrock in the channel or immediate vicinity of the bridge. Figure 3 shows a Regional Geologic Map.

The Ackerman Creek channel at this location contains primarily bedload deposits comprised of loose sand and gravel. These materials, to approximately 5 to 10 feet below the channel bottom, appear subject to channel migration.

No mapped faults are known to cross the site. The nearest mapped fault is the Maacama Fault approximately 0.8 miles east of the site. Figure 4 shows faults in the region.

6 SUBSURFACE CONDITIONS

6.1 Subsurface Soil Conditions

Historic Log of Test Borings

LOTB drawing for the existing bridge⁴ shows a general sequence of:

- Loose silty sand and gravel with minor clay in the upper 10 feet (generally to elevation 610 to 605 feet) which is about 5-10 feet below the existing channel bottom.
- Compact (cemented) sand/gravel and stiff clay and clayey sand to a maximum depth of 40 feet (using rotary bucket auger drilling method and penetration tests).

The LOTB for the SR101 bridge at Ackerman Creek⁵, located approximately 800 feet upstream, shows a similar soil sequence. The uppermost 10 feet is described as generally loose sand and gravel, underlain by compact sand/gravel and stiff clay. Maximum depth of drilling at this location (penetrometer tests) was about 55 feet.

Appendix A contains the LOTB drawings for the existing North State Street bridge and the upstream SR101 bridge.

Blackburn 2013 Investigation at Bridge

Blackburn drilled and sampled two borings to a maximum depth of 90 feet below the existing ground surface (bgs) on August 6 and 7, 2013. We summarize the subsurface conditions in the table below and include our LOTB in Appendix A.

³ California Geological Survey, "Geologic Map of California – Ukiah Sheet", scale 1:250,000, 1960

⁴ Hans Feibusch, Consulting Engineer, drilled in June 1965

⁵ California Division of Highways, drilled in July 1957

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**TABLE 1: SUBSURFACE CONDITIONS**

Unit	Depth (ft) (R-13-01/R-13-02)	Elevation (ft) (R-13-01/R-13-02)	Subsurface Conditions
1	Surface to 26.5/27.0	628.0/623.8 to 601.5/596.8	Medium Dense to Very Dense Silty Sand, Poorly graded Sand, Poorly graded Gravel, and Clayey Gravel
2	26.5/27.5 to 59.25/54.75	601.5/596.8 to 568.75/569.1	Stiff to Hard Lean Clay with minor seams of Clayey Gravel and Silty Sand
3	59.25/54.75 to 90.0/79.0	568.75/569.1 to 538.0/544.8	Dense to Very Dense Clayey Gravel

Blackburn 2013 Investigation at Approaches

Blackburn drilled and sampled two borings to a maximum depth of 6.5 feet bgs on August 7, 2013 at the bridge approaches. At these borings we observed 3 to 4 inches of asphalt overlying 6 to 29 inches of aggregate base rock (Borings A-13-04 and A-13-03 respectively). In Boring A-13-03 the aggregate base rock was underlain by medium dense, poorly graded gravel with sand to the total depth of exploration. In Boring A-13-04 the aggregate base rock was underlain by stiff to very stiff, sandy silt to the full depth of exploration. We summarize the subsurface conditions in the table below and include our boring logs in Appendix A.

6.2 Groundwater

Groundwater in previous studies (1965) showed groundwater within about 1 to 5 feet of ground surface (shown at approximate elevations of 611 to 617 feet. The Ackerman Creek channel contained minor flow during our site review in November 2010 and was dry at the time of a review in September 2011 and drilling in August 2013. Groundwater was encountered at approximate elevation 603 feet in our August 2013 borings.

The channel will be re-graded as part of the project and based on discussions with Consor, typical flows will be at approximate elevation of 612 feet or lower. For design we assume a ground water elevation of 612 feet. Unit 1 granular soils below elevation 612 feet will be saturated and will be capable of transmitting significant volumes of water into excavations. Groundwater may reach higher elevations during and shortly after flood or significant rain events. See Section 17.3 Shoring Considerations and Section 17.4 Dewatering Considerations for recommendations regarding the impact of groundwater on design and construction.

7 SCOUR EVALUATION

As noted above, severe scour was reported in the December 1964 flood that led to the construction of this bridge in 1965. A 1978 Bridge Inspection Report (BIR) contains the following observation: *“There has been severe degradation of the channel which exposed the bent footings and threatened to undermine them. This condition has been alleviated by a check-dam constructed a short distance downstream from the bridge.”*

BIR’s from 1978 through 2008 note the accumulation of heavy drift but no significant scour. A February 2010 BIR describes the full length of the north pier (Pier 3) footing as exposed and a scour hole at the upstream end that undermined the 2.5 foot thick footing. Our site observation in September 2011 noted this scour hole and the top of the underlying steel-cased concrete piles.

Conсор indicates that to allow for fish passage and to reduce the potential for scour, the following channel improvements are proposed:

- Removal of the downstream check dam.
- Construction of a series of 8 rock weirs to prevent channel degradation and allow fish passage.
- Placement of Rock Slope Protection (RSP).

Avila & Associates prepared a January 26, 2016 Design Hydraulic Study Report for the project. Table 2 presents a summary of their abutment scour analysis results.

Table 2: Scour Analysis Results		
	Abutment 1	Abutment 2
Long Term Degradation Scour Elevation (ft.)	N/A	N/A
Local Abutment Scour Elevation (ft.)	603	603

Avila & Associates’ Report indicates local scour could only occur at the abutments if the channel improvements are unsuccessful during an extreme state hydraulic event.

8 CORROSION EVALUATION

Caltrans Corrosion Guidelines⁶ (2021) 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 1,500 ppm or greater,
- pH is 5.5 or less.

We summarize the corrosion testing results in Table 3.

⁶ Caltrans, Division of Engineering Services, Materials Engineering and Testing Services Corrosion Branch, May 2021; Corrosion Guidelines, Version 3.2.

TABLE 3: LABORATORY SOIL CORROSIVITY RESULTS					
Boring	Sample No./ Depth (ft)	pH	Minimum Resistivity (ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)
R-13-01	7/34.0-34.5	8.2	1,420	12.3	15.0
R-13-02	8/39.0-39.5	8.2	1,580	16.8	5.9
R-13-02	14/69.0-69.5	7.8	4,020	10.4	15.2
R-13-04	1/3.0-3.5	7.1	1,930	11.6	11.4

The test results indicate the site soil is not considered corrosive to structural foundation elements per the Caltrans guidelines referenced above. Appendix B contains the test results.

9 SEISMIC DATA AND EVALUATION

9.1 Ground Motions

Blackburn used the Caltrans ARS Online Version 2.3.06 (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. Consor informed us that the ground motions did not have to be updated using Caltrans ARS Online Version 3.1.0 (web-based tool) or the current Caltrans Seismic Design Criteria Version 2.0 (April 2019).

Table 4 provides a summary of our ground motion study results.

TABLE 4: GROUND MOTION STUDY RESULTS	
Latitude/Longitude	39.179339/-123.209948
Peak Ground Acceleration (PGA)	0.798g
*V ₅₃₀ (Small Strain Shear Wave Velocity)	300 meters per second
Near Fault Factor	Yes with USGS Deaggregation
Basin Amplification Factor	N/A
Controlling Deterministic Scenario	Maacama fault zone (Fault ID: 66) ➤ Style: strike-slip ➤ Maximum Moment Magnitude (M_{max}) = 7.4 ➤ Site to fault distance (R_{RUP}) = 0.8 miles
RECOMMENDED 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. PGA= 0.798 Mmax= 7.4

*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.

Figure 5 contains a graphical display of our recommended Design Response Spectrum along with the tabular data used to create the spectrum.

9.2 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)⁷, 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 using data from the 2012 Caltrans Fault Database. We consider the potential for fault rupture at the site to be low.

9.3 Liquefaction Evaluation

Liquefaction can occur when saturated, loose to medium dense, granular soils (generally within 50 ft of the surface), or specifically defined cohesive soils, are subjected to ground shaking. Our analysis shows liquefaction can occur in Unit 1 silty sands encountered at Abutment 2 (north abutment, Boring R-13-01) between approximate elevation 606.5 to 601.5 feet. Liquefaction will cause both vertical and lateral movement. The analysis shows potential settlement of 0.18 feet and unrestrained (no pile resistance) lateral spread⁸ (horizontal displacement) of up to 53 feet at Abutment 2.

Note that our liquefaction analysis for Boring R-13-02 (Abutment 1) indicated a thin approximately 1-foot layer of liquefiable soil between approximate elevations 612.0 to 611.0. Since this layer is very thin and located well above the bottom of the abutment pile cap (pile cutoff elevation of 607.9 feet), we do not anticipate it will impact design or performance of the abutment.

See Section 13 for discussion of lateral spread and pile foundations.

9.4 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 approaches to be low when structure backfill is placed in accordance with construction specifications.

9.5 Seismic Slope Instability

We consider seismic slope instability from lateral spreading may occur at the north bank. We provide pile design analysis for abutment stability in Section 13.

⁷ Fault Rupture Hazard Zones in California, Special Publication 42, Interim Revision; California Geological Survey

⁸ Idriss, I. M. and Boulanger, R. W., "Residual Shear Strength of Liquefied Soils," Proceedings, 27th USSD Annual Meeting and Conference, Modernization and Optimization of Existing Dams and Reservoirs, Philadelphia, Pennsylvania, March 5-9, 2007.

10 AS-BUILT FOUNDATION DATA

County bridge plans ("As-Built", dated 6/21/65) show concrete piles in steel casings with 50 ton design capacity under all supports. Specified tip elevations are 589.0 feet at the abutments and 585.0 feet at the bents (approximately 25-30 feet below channel bottom). The As-Built plans indicate the piles were driven to within about one foot of specified tips.

11 FOUNDATION RECOMMENDATIONS

Based on subsurface conditions, suitable structure support can be achieved by either steel pipe piles (driven with open or closed end) or steel H-piling. Steel H piles would achieve resistance mostly through side friction with some end-bearing contribution and, therefore, would likely drive longer than steel pipe piles for the same design loading. Steel pipe piles would achieve resistance mostly in end-bearing with side friction contribution. Based on our discussions with Consor, Class 200 Steel Pipe Piles (PP16 x 0.50) are preferred.

We do not recommend Caltrans standard pre-cast concrete piles (Alternative "X" and "Y") due to the potential for hard driving within local dense layers above specified tip elevations that may result in damage to concrete piles.

Due to the granular nature of the subsurface materials and high groundwater conditions, CIDH piles would require special installation measures, including casing, slurry drilling methods and the use of minimum 24-inch diameter CIDH piles for tremie concrete placement. Therefore, the use of CIDH piles is not preferred.

We do not recommend the use of spread footings due to the potential for excessive settlement, scour, and limited bearing capacity.

11.1 Foundation Data and Loading

To evaluate Alt "W" Class 200 Steel Pipe Piles, Blackburn used the following information provided by Consor:

- Working Stress Design (WSD) Method.
- 190 kip Service Design compression load per pile.
- 380 kip Nominal Resistance compression load per pile.
- No tension demand.
- Pile cut-off at elevation 607.9 feet (at both abutments).
- Two rows of piles at each abutment. The back row contains 10 piles spaced at approximately 6 to 8 feet (center-to-center) and the front row contains 15 battered piles spaced at approximately 4.5 feet (center-to-center).

- Scour depths:
 - 3.5 feet below the bottom of the pile cap for service condition.
 - 1.75 feet below the bottom of the pile cap for strength condition.
 - We accounted for the above scour depths in our lateral pile analysis but did not account for loss of axial compressive resistance in our pile analysis since the reduced skin friction is negligible at the strength limit state (upper 1.75 feet of pile exposed).
- Permissible settlement of 2.0-inch at Service level loads.

Consort informed us that the pile analysis using the information above and subsequent recommendations did not need to be updated using current AASHTO LRFD 8th Edition and Caltrans Amendments.

11.2 Foundation Recommendations and Pile Data Table

Blackburn evaluated abutment foundations using Caltrans November 2003 Bridge Design Specifications (updated November 2008) for foundations using Working Stress Design methods. Table 5 presents our recommended pile data table. We present our engineering analysis in the following sections.

TABLE 5: PILE DATA TABLE						
Support No.	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft.)	Specified Tip Elevation (ft.)	Nominal Driving Resistance (kips)
		Compression	Tension			
Abut 1	Class 200, Alt "W" PP16 x 0.50	380	0	565.5(a), 587.5 (b), 578.0 (c)	565.5	380
Abut 2	Class 200, Alt "W" PP16 x 0.50	380	0	565.5(a), 585.5 (b), 578.9 (c)	565.5	390

Notes: 1. Design tip elevations for Abutments are controlled by (a) Compression, (b) Lateral Load no scour, (c) Lateral load at Strength with 1.75 feet of abutment scour
 2. Do not raise specified tip elevation unless approved by Consort and Blackburn.

11.3 Engineering Parameters

Blackburn developed engineering parameters for this project based on the following:

- Average unit weight values based on our laboratory tests.
- Average cohesion values based on pocket penetrometer tests and unconsolidated undrained triaxial test results.
- Friction angles based on published blow count correlations and material type.
- Engineering experience and judgment.
- Design groundwater at elevation 612.0 ft.

Blackburn used the generalized soil parameters shown in Table 6 for our analysis of pile compressive resistance, lateral resistance and settlement.

TABLE 6: GENERALIZED SOIL PARAMETERS

Elevation (ft)	General Soil Type	Total Unit Weight (lb/ft ³)	Buoyant Unit Weight (lb/ft ³)	Friction Angle (degrees)	Cohesion (psf)	Modulus, k (lb/in ³)
607.9 to 596.8 (Abut 1)	Dense Clayey Gravel	146	83.6	35	--	125
607.9 to 601.5 (Abut 2)	Medium Dense Silty Sand	129	66.6	33	--	60
596.8 to 569.1 (Abut 1) 601.5 to 568.8 (Abut 2)	Stiff to Hard Lean Clay	130	67.6	--	3,000	1,000
569.1 to 544.8 (Abut 1) 568.8 to 538.0 (Abut 2)	Very Dense Clayey Gravel	143	80.6	38	--	125

12 ABUTMENT PILES

12.1 Compressive Resistance

Blackburn determined compressive resistance for the Alt “W” Class 200 Steel Pipe Piles using the AllPile v7.19d (2015) computer program developed by CivilTech Software. Blackburn multiplied the Service-1 Limit Load (190 kips) by a Factor of Safety of 2.0 to determine the required nominal resistance 380 kips. We include static pile calculations in Appendix C. The specified pile tip elevations are based on calculated resistance, variation of materials, and anticipated plugging of the pipe piles during driving.

12.2 Settlement

We expect that settlement will be less than the permissible 2-inch settlement specified for the single span structure with seat type abutments due to the presence of dense to very dense soil at and below the specified tip elevations. We do not anticipate significant long-term settlement at this site.

12.3 Lateral Load Analysis

We used LPILE Plus Version 5.0 software to evaluate lateral pile capacity. Blackburn determined the allowable lateral pile design loads that would produce approximately ¼-inch and 1-inch top-of-pile deflection assuming a pinned (free-head) condition. Blackburn used a p-multiplier of 0.76 in the longitudinal direction and 0.52 in the transverse direction to correct for group effects. We determined these values based on the pile spacing discussed in Section 7.1. For our analysis, we applied a minimum axial compression of 200 kips to the top of the pile modeled. Blackburn presents our lateral pile analysis for the Service (¼-inch deflection) and Strength (1-inch deflection) scour conditions (3.5 and 1.75 feet of scour below the bottom of the pile caps respectively) considered for both the longitudinal and transverse bridge directions in Tables 7 and 8.

TABLE 7: LATERAL PILE CAPACITY (SCOUR CASE) (Longitudinal Bridge Direction)		
Support	Top-of-Pile Deflection	Lateral Resistance (kips)
Abutment 1	¼ inch	6.1
	1 inch	21.5
Abutment 2	¼ inch	10.0
	1 inch	28.0

TABLE 8: LATERAL PILE CAPACITY (SCOUR CASE) (Transverse Bridge Direction)		
Support	Top-of-Pile Deflection (inches)	Lateral Resistance (kips)
Abutment 1	¼ inch	4.8
	1 inch	18.3
Abutment 2	¼ inch	8.7
	1 inch	24.5

Blackburn calculated a minimum lateral tip elevation of 579.9 feet at Abutment 1 and 582.9 feet at Abutment 2 using a factor of safety of 1.5.

Tables 9 and 10 present the results of our lateral pile analysis for both the longitudinal and transverse bridge directions considering no scour at the piles.

TABLE 9: LATERAL PILE CAPACITY (NO SCOUR CASE) (Longitudinal Bridge Direction)		
Support	Top-of-Pile Deflection	Lateral Resistance (kips)
Abutment 1	¼ inch	53.5
	1 inch	141.5
Abutment 2	¼ inch	11.8
	1 inch	33.5

TABLE 10: LATERAL PILE CAPACITY (NO SCOUR CASE) (Transverse Bridge Direction)		
Support	Top-of-Pile Deflection (inches)	Lateral Resistance (kips)
Abutment 1	¼ inch	38.0
	1 inch	100.0
Abutment 2	¼ inch	10.0
	1 inch	27.0

Blackburn calculated a minimum lateral tip elevation of 578.0 feet at Abutment 1 and 578.9 feet at Abutment 2 using a factor of safety of 1.5 for the controlling scour case (Strength Limit).

Refer to the LPILE output graphs in Appendix C for additional information. Lateral deflection at Abutment 2 does not account for loads from lateral spread that could occur during a seismic event.

12.4 Negative Skin Friction

We do not anticipate negative skin friction at Abutment 1. At Abutment 2 we anticipate minor negative skin friction due to liquefaction-induced settlement of the thin liquefiable Silty Sand layer. We added a nominal 10 kips to the required nominal driving resistance to account for any pile downdrag loads due to liquefaction induced negative skin friction at Abutment 2.

13 LATERAL SPREAD SOIL MOVEMENT VS. PILE STRENGTH SHEAR RESISTANCE

Lateral spread of the approach backfill may occur at Abutment 2 during a seismic event and will increase the lateral loads from the soil on the structure. To evaluate the increased loads on the structure and the pinning effects of the piles we used the method in the Caltrans Memo to Designers 20-15 (May 2017) to evaluate Lateral Spread Soil Movement vs. Pile Strength Shear Resistance at Abutment 2. To perform our analysis we:

- Step 1: Determined the residual shear strength of the liquefiable layer (Silty Sand)
- Step 2: Performed slope stability analysis using SLIDE 6.034, using the residual shear strength to determine the post liquefaction yield acceleration.
- Step 3: Used the residual shear strength and yield acceleration to determine the approximate lateral movement using the Newmark procedure as outlined by Bray and Travasarou (2007)⁹.
- Step 4: Increased the residual shear strength of the liquefiable layer by steps, determined the post liquefaction yield acceleration, and the lateral movement.

⁹ Bray, J.D. and Travasarou, T., Simplified procedure for estimating earthquake induced deviatoric slope displacements, Journal of Geotechnical and Environmental Engineering, 133 (4): 381-392, 2007

- Step 5: Calculated the shear resistance provided by the piles by subtracting the actual residual shear strength from the increased residual strength values used in Step 4. This value represents the pile shear resistance per foot of embankment.
- Step 6: Plotted the calculated lateral spread vs the calculated shear resistance provided by the piles (Step 5).

We include the plot of Lateral Spread Movement vs Required Pile Shear Resistance in Appendix C. When this method is used to calculate the pile resistance to reduce lateral displacement to an acceptable value. Use an effective failure mass width of 1.5 times the foundation width in accordance with Caltrans Memo to Designers (MTD).

14 LATERAL EARTH PRESSURES

Use the equivalent fluid weights (EFWs) shown in Table 11 to design the retaining walls assuming level backfill conditions:

TABLE 11: EQUIVALENT FLUID WEIGHTS		
Condition	Static EFW (pcf)	Seismic EFW (pcf)
Active	34	91
Passive	280	176

For static design, apply the resultant of the static active earth pressure (34 lb/ft³) 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 91 lb/ft³ 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 120 pcf, a minimum angle of internal friction of 34°, and that drainage behind walls is placed in accordance with Caltrans “Standard Plans and Specifications.”

To limit wall deflection to acceptable levels, Blackburn applied a factor of safety of 1.5 to the ultimate passive pressure to generate the allowable passive pressures provided above.

Blackburn estimated the EFWs for seismic loading using the Mononobe-Okabe equation for active and passive lateral coefficients K_a and K_p . In the Mononobe-Okabe equation, Blackburn used a horizontal seismic acceleration coefficient (k_h) of 0.56 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 1.0-inches during the design seismic event. Blackburn 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 0.45 to resist sliding for concrete placed on native undisturbed soil or compacted fill.

15 APPROACH ROADWAY EMBANKMENT

15.1 Roadway and Approach Fill

With the exception of abutment backfill, we do not expect significant depths of new fill for this project. Construct and compact approach fills in accordance with 2023 Caltrans Standard Specifications.

Compact all approach fill within 150 feet of the bridge deck to a minimum 95% relative compaction based on Caltrans Test Method (CTM) 216. Refer to Section 17.5 for import fill material requirements.

Within the proposed embankment/roadway areas to receive structural fill, strip and dispose of concentrations of organics. After stripping, scarify and recompact the upper 12 inches of existing subgrade prior to placement of embankment fill. Extend the foundation preparation limits to a minimum of 3 feet beyond the toe of embankment. The geotechnical engineer shall determine final depth of subgrade preparation.

15.2 Cuts and Excavations

Prepare all slopes and shore temporary excavations in accordance with current Cal OSHA requirements. Where the use of excavation sloping and/or shoring is required, a competent person must classify each soil deposit in accordance with OSHA procedures. At abutment locations, we classify the upper native soils and fill as Type C, which requires a temporary slope gradient of 1.5H:1V or flatter.

Temporary shoring may be required to maintain traffic. Acceptable shoring methods that may be considered include (but are not limited to) sheet piles, soldier pile, or deep soil mixing walls. Driven and/or vibratory installed sheet piling elements may encounter difficult installation through the dense to very dense sands from approximate elevation 609-597 feet at Abutment 1 and approximate elevation 615 to 606 feet at Abutment 2. Additional difficult sheet pile installation may be encountered below approximate elevation 570 feet. Jetting or drilling may be required to advance sheet piles into the dense to very dense gravels. The use of jetting or drilling must be approved by Consor and Blackburn.

The contractor is responsible for designing and constructing suitable excavation sloping and/or shoring based on the actual soil conditions exposed.

15.3 Settlement

We do not anticipate significant immediate or long-term embankment settlement. No waiting period is necessary prior to construction of bridge abutment foundations.

16 STRUCTURAL SECTION AND ROADWAY SUBGRADE

16.1 New Pavement Section

R-value test results (CTM 301) on samples of anticipated roadway subgrade show values of 44 and 68. Based on these results and observed soil/rock conditions along the roadway, we recommend a minimum R-value of 40 for use in pavement structural section design.

Table 12 shows recommended pavement sections based on the computer program CalFP (Caltrans pavement design software, V1.1) and on a Traffic Index of 9 (provided by Consor).

TABLE 12: PAVEMENT SECTION			
Design R-Value	Traffic Index	Dense Graded HMA (ft)	Class 2 Aggregate Base (ft)
40	9	0.85	0
		0.45	0.8

Construct and compact roadway subgrade in accordance with 2023 Caltrans Standard Specifications, Sections 19-2 and 19-5, and County of Mendocino Standard Road Specifications. Class 2 aggregate base shall have a minimum R-value of 78, and shall be compacted to 95 percent relative compaction (CTM 216).

Imported material used for roadway subgrade and within 4 feet of the surface shall have a minimum R-value equal to the design R-Value of 40.

Slope all areas adjacent to pavement at a gradient of 2% or greater to allow for positive surface drainage. Positive surface slope and uniform compaction are necessary for proper pavement performance. To prevent premature pavement failure, design the roadway drainages such that water does not flow beneath the paved areas.

17 CONSTRUCTION CONSIDERATIONS

Where referenced below, "Caltrans Standard Specifications" refers to the 2023 Caltrans Standard Specifications.

17.1 Existing Bridge Foundation Conflicts

Conсор provided a pile layout plan to minimize potential conflicts between the existing piles and pile installation for the replacement structure.

The existing piles are at cut off elevation 614.0 feet. Existing piles will need to be cut-off at or below the new pile cut-off elevation. The contractor shall not remove the existing piles as it will disturb the upper soil layers and reduce the pile capacity. Consor's pile layout has taken care to limit the conflicts between existing piles and new piles based on the as-built information. The location of existing bridge piles should be confirmed during construction and the layout for new piles modified if conflicts are noted. Any modifications to the contract pile layout must be approved by Consor and Blackburn.

17.2 Class 200 Alt. "W" Piles (PP16x0.50)

Piles shall conform to Section 49 of the Caltrans Standard Specifications (CSS).

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

Difficult pile installation with hard driving is anticipated through locally dense to very dense gravel layers above the specified tip elevation. 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. The drivability evaluation (wave equation analysis) shall be performed using Boring R-13-002 (Abutment 1) data.

Verify pile capacity during driving per CSS 49-2.01A(4)(c). A pile load test is not necessary. Do not raise specified tip elevations without the approval of Blackburn and Consor.

Piles that do not attain the required nominal driving resistance at specified tip elevations 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. Select piles with the lowest nominal driving resistance (EOD) at each support for restrike. The pile penetration during restrike must be at least 2 inches. If all 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 Blackburn for additional recommendations which could include an extended "setup period".

17.3 Shoring

Temporary shoring may be required to maintain traffic over the period of the project (2 years). Shoring design must consider surcharge loads and the project timeline. Acceptable shoring methods that may be considered include (but are not limited to) sheet piles, soldier pile, or deep soil mixing walls. Driven shoring elements may encounter difficult driving in the dense to very dense sands from approximate elevation 609-597 feet at Abutment 1 and approximate elevation 615 to 606 feet at Abutment 2. Additional hard driving will be encountered below approximate elevation 570 feet. Jetting or drilling may be required to advance piles into the dense to very dense gravels. The use of jetting or drilling must be approved by Consor and Blackburn.

The contractor is responsible for design and construction of excavation sloping and shoring in accordance with CalOSHA Standards, and to protect existing structures, utilities, and other facilities during construction.

17.4 Excavation Dewatering

Excavations that extend below the creek bed or surface water level will require dewatering using sump pumps, diking/diversion of the creek and/or possible cofferdam construction. The need for an abutment pile cap seal course should be evaluated by the Engineer during pile cap excavation and will depend in part on the flow rate of water into the bottom of the excavation. The contractor is responsible for design and construction of dewatering systems, including seal courses, in accordance with Caltrans guidelines and the 2023 Caltrans Standard Specifications.

To reduce dewatering construction impacts and associated costs, we strongly recommend scheduling foundation construction during the late Summer to early Fall months.

17.5 Import Fill

General import fill (not including structure backfill) shall meet the following requirements:

- No concentrations of organics, debris, and other deleterious materials.
- Maximum particle size of 2 inches, with at least 75 percent passing the No. 4 Sieve and at least 20 percent passing the No. 200 Sieve.
- Plasticity Index (ASTM D4318) less than 20.
- R-value (California Test Method, CTM 301) greater than or equal to 40 within the upper 24 inches of finish pavement subgrade.
- Where structure backfill is required, it shall meet the requirements of Section 19 of the CSS.

18 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, retain Blackburn to:

- Review and provide written comments on the (civil, structural) plans and specifications prior to construction.
- Monitor construction to check and document our report assumptions. At a minimum, Blackburn should observe pile installation.
- Update this report if design changes occur, 2 years lapse between this report and construction, or site conditions change.

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

19 LIMITATIONS

Use this foundation report only for the design and construction of the North State Street Bridge at Ackerman Creek bridge replacement, as described herein.

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

Blackburn based this report on the current site and project conditions. We assumed the soil and groundwater conditions we observed in our borings are representative of the subsurface conditions on the site. Actual conditions between borings could be different.

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

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.

FOUNDATION REPORT

N. State Street Bridge at Ackerman Creek

Bridge No. 10C-0065
Mendocino County, CA

November 2023

FIGURES

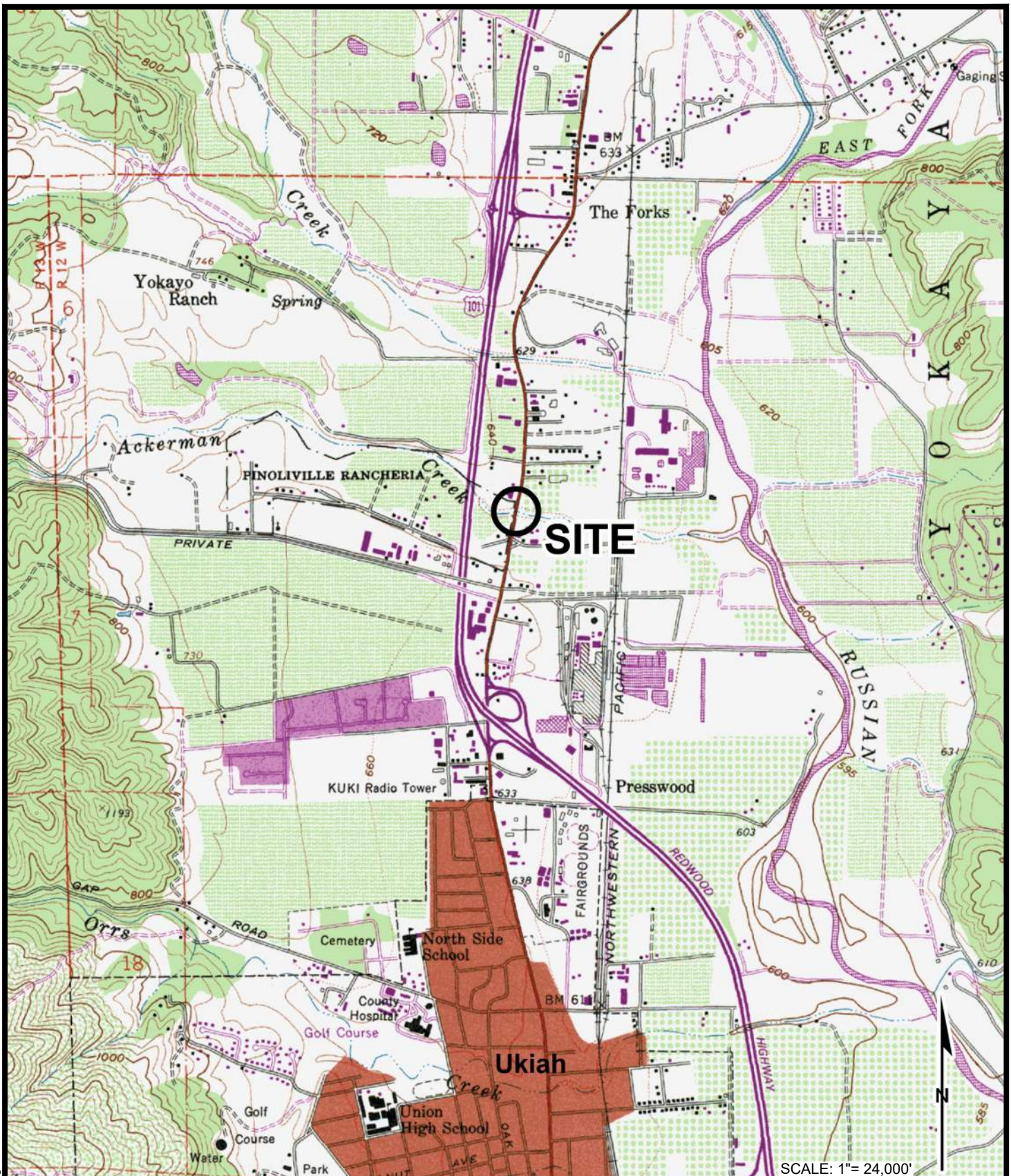
Figure 1 – Vicinity Map

Figure 2 – Site Map

Figure 3 – Geologic Map

Figure 4 – Regional Fault Map

Figure 5 – ARS Response Spectra



Source: MAPTECH Terrain Navigator Pro, v. 8.0, USGS
topographic 7.5 minute quadrangle, Ukiah, 1958 (revised 1975)

SCALE: 1"= 24,000'
DISTANCE, FEET
0 24000 48000



VICINITY MAP

N. State Street Bridge at Ackerman Creek
Bridge No. 10C-0065
Mendocino County, California

File No. 2103.1

November 2023

Figure 1



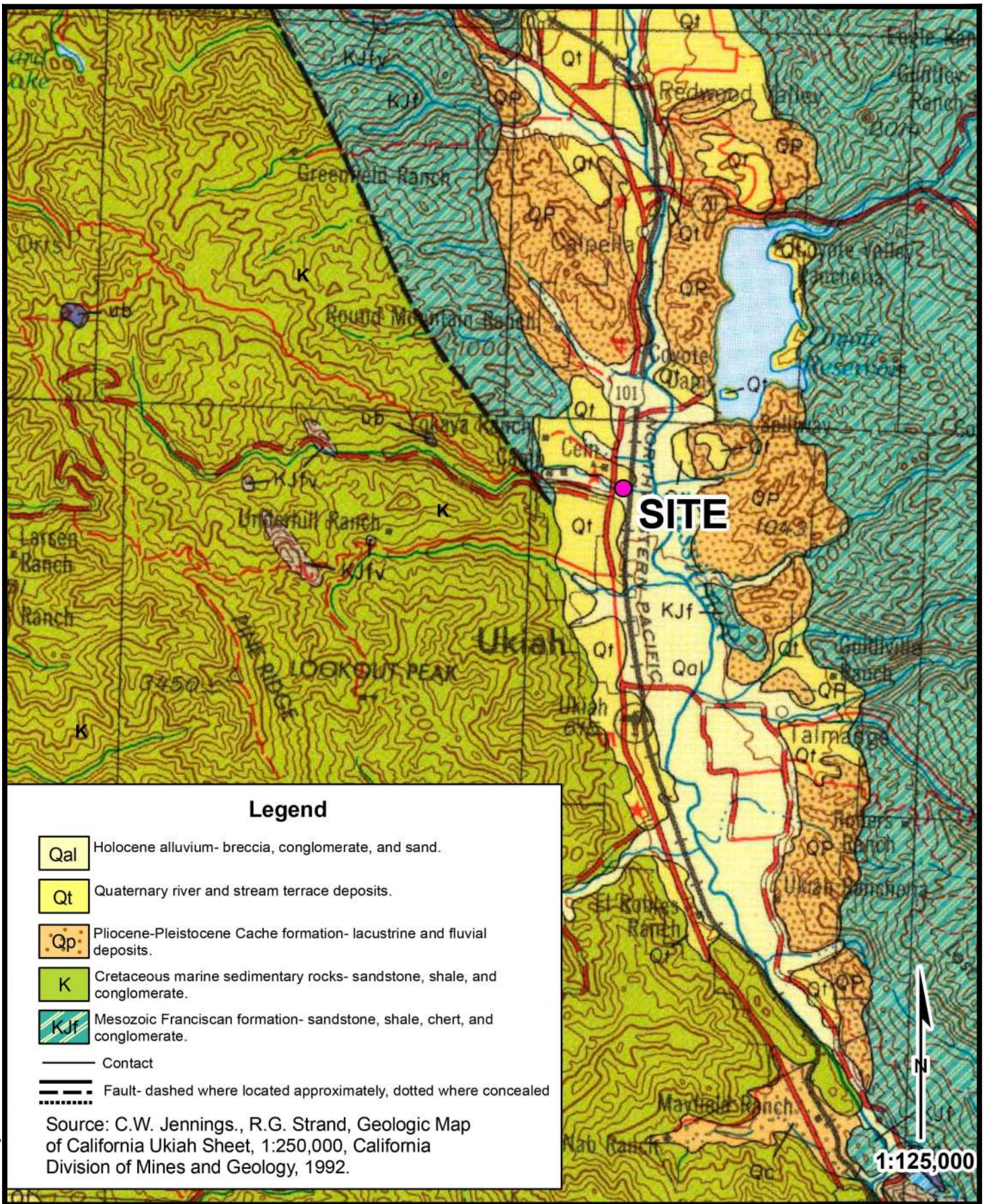
11521 Blocker Drive, Suite 110
Auburn, CA 95603
(530) 887-1494
(530) 887-1495-Fax
www.blackburnconsulting.com

SITE MAP
N. State Street Bridge at Ackerman Creek
Bridge No. 10C-0065
Mendocino County, California

File No. 2103.X

November 2023

Figure 2

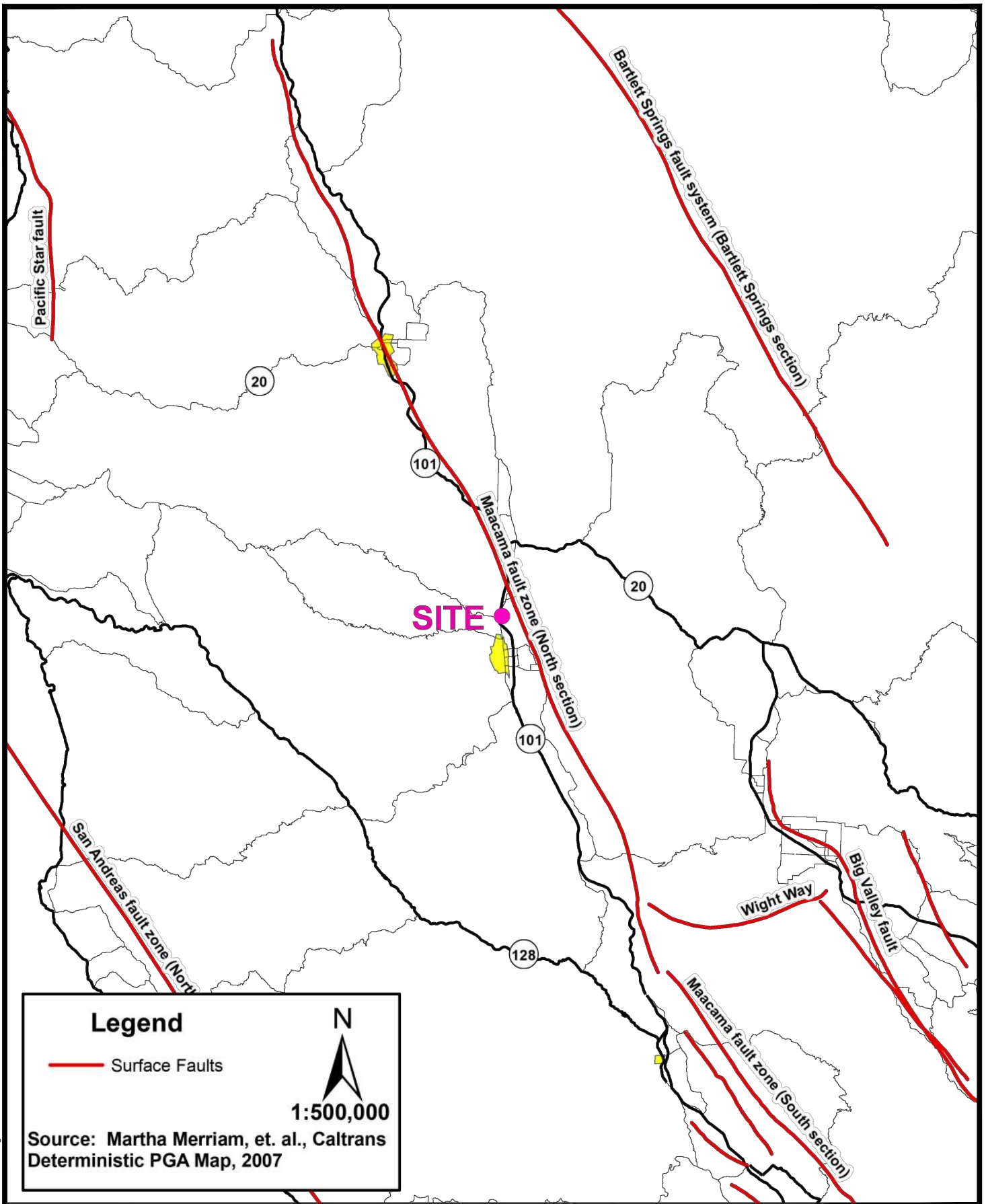


GEOLOGIC MAP
N. State Street Bridge at Ackerman Creek
 Bridge No. 10C-0065
 Mendocino County, California

File No. 2103.1

November 2023

Figure 3

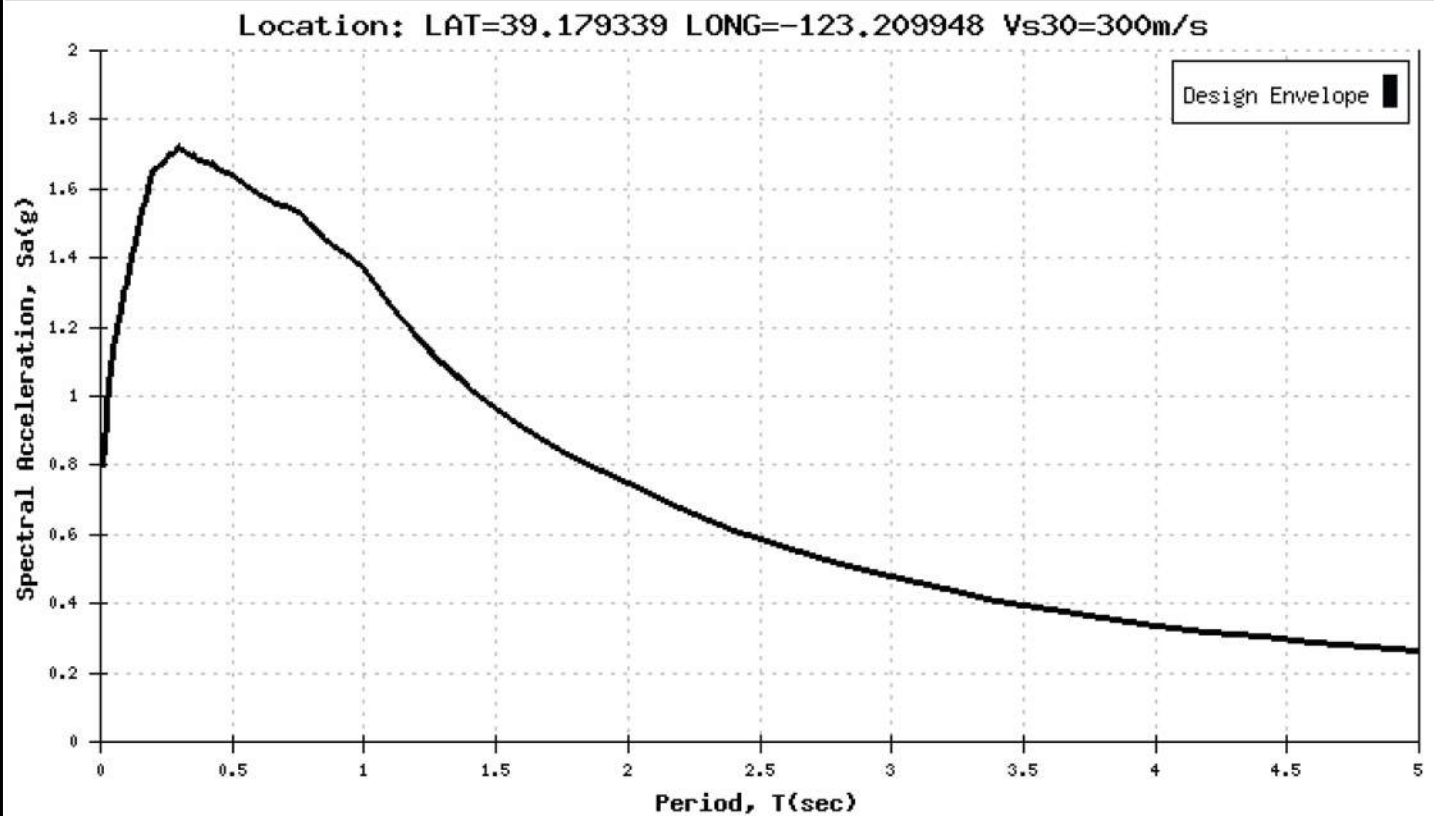


11/3/2023 2:03.1 Fig-1-5 Ackerman Creek.dwg



REGIONAL FAULT MAP
N. State Street Bridge at Ackerman Creek
Bridge No. 10C-0065
Mendocino County, California

File No. 2103.1
November 2023
Figure 4



Caltrans ARS Online (V 2.3.06)



ARS RESPONSE SPECTRA
N. State Street Bridge at Ackerman Creek
Bridge No. 10C-0065
Mendocino County, California

File No. 2103.1

November 2023

Figure 5

Period	SA	Period	SA	Period	SA	Period	SA
0	0.798	0.085	1.28	0.35	1.694	1.4	1.024
0.01	0.798	0.09	1.297	0.36	1.69	1.5	0.965
0.02	0.93	0.095	1.312	0.38	1.682	1.6	0.912
0.022	0.95	0.1	1.327	0.4	1.675	1.7	0.865
0.025	0.977	0.11	1.368	0.42	1.668	1.8	0.823
0.029	1.01	0.12	1.405	0.44	1.661	1.9	0.785
0.03	1.018	0.13	1.441	0.45	1.658	2	0.751
0.032	1.032	0.133	1.452	0.46	1.655	2.2	0.675
0.035	1.053	0.14	1.475	0.48	1.649	2.4	0.612
0.036	1.059	0.15	1.507	0.5	1.644	2.5	0.585
0.04	1.084	0.16	1.538	0.55	1.613	2.6	0.56
0.042	1.096	0.17	1.568	0.6	1.587	2.8	0.515
0.044	1.107	0.18	1.596	0.65	1.566	3	0.477
0.045	1.113	0.19	1.623	0.667	1.559	3.2	0.441
0.046	1.118	0.2	1.65	0.7	1.548	3.4	0.41
0.048	1.129	0.22	1.665	0.75	1.533	3.5	0.396
0.05	1.139	0.24	1.679	0.8	1.493	3.6	0.383
0.055	1.163	0.25	1.686	0.85	1.458	3.8	0.358
0.06	1.186	0.26	1.692	0.9	1.427	4	0.337
0.065	1.207	0.28	1.704	0.95	1.398	4.2	0.32
0.067	1.215	0.29	1.71	1	1.373	4.4	0.304
0.07	1.227	0.3	1.716	1.1	1.264	4.6	0.29
0.075	1.246	0.32	1.707	1.2	1.171	4.8	0.277
0.08	1.263	0.34	1.698	1.3	1.093	5	0.266

FOUNDATION REPORT

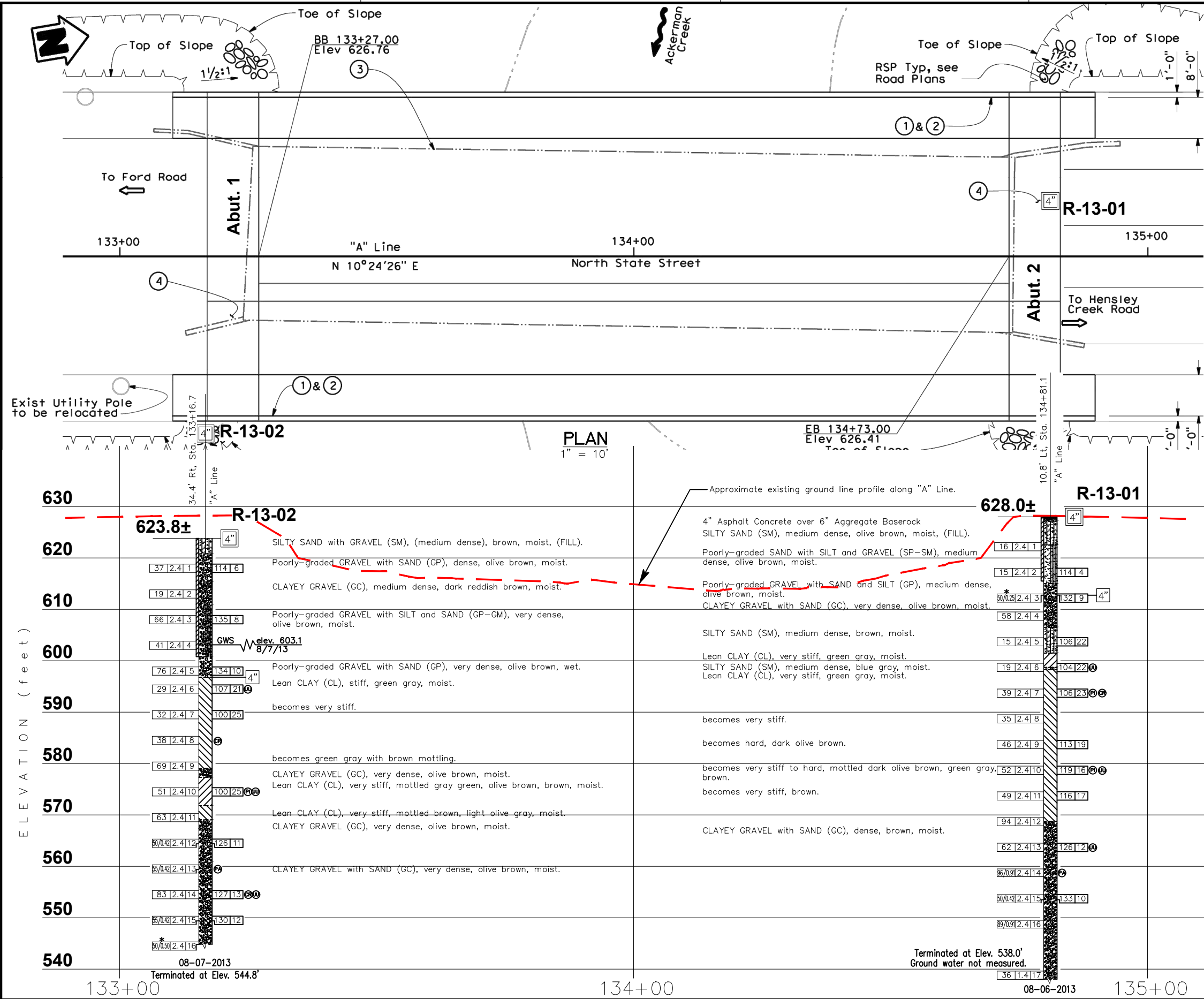
N. State Street Bridge at Ackerman Creek

Bridge No. 10C-0065
Mendocino County, CA

November 2023

APPENDIX A

Log of Test Borings (2015)
Boring Logs and Legend (2015)
North State Street @ Ackerman Creek (1965)
SR101 @ Ackerman Creek (1957)



DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
01	Men	CR 104	XX.X/XX.X		

PROFESSIONAL SEAL
REGISTERED PROFESSIONAL GEOLOGIST
ROBERT C. PICKARD
No. 2508
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

PLANS APPROVAL DATE _____

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

QUINCY ENGINEERING
3247 RAMOS CIRCLE
SACRAMENTO, CA 95827

BLACKBURN CONSULTING
11521 BLOCKER DRIVE, SUITE 110
AUBURN, CA 95603

FILE No. 2103.1

DESIGN OVERSIGHT	DRAWN BY	FIELD INVESTIGATION BY:	PREPARED FOR THE MENDOCINO COUNTY DEPARTMENT OF TRANSPORTATION	BRIDGE NO.	LOG OF TEST BORINGS 1 OF 3
SIGN OFF DATE	CHECKED BY	DATE:	PROJECT ENGINEER	POST MILE	
GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)			UNIT: PROJECT NUMBER & PHASE:	CONTRACT NO.:	DISREGARD PRINTS BEARING EARLIER REVISION DATES
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS			FILE => \$REQUEST		REVISION DATES
					SHEET OF

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

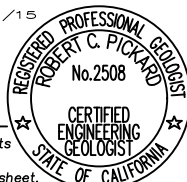
CEMENTATION	
Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
01	Men	CR 104	XX.X/XX.X		



CERTIFIED GEOTECHNICAL ENGINEER

03/30/15



PLANS APPROVAL DATE _____

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of scanned copies of this plan sheet.

QUINCY ENGINEERING
3247 RAMOS CIRCLE
SACRAMENTO, CA 95827

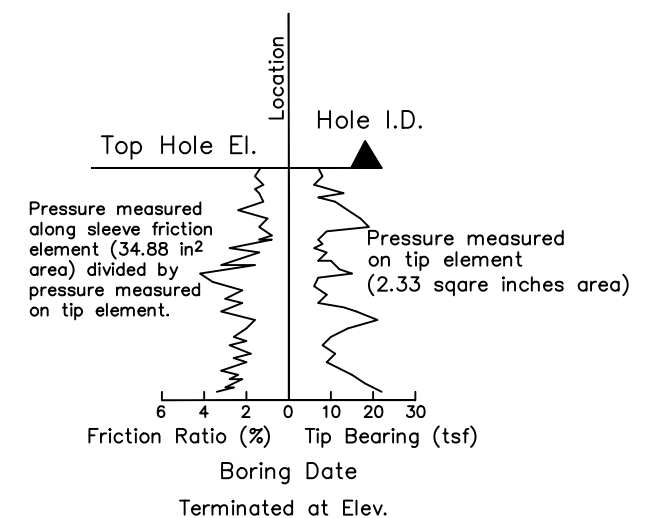
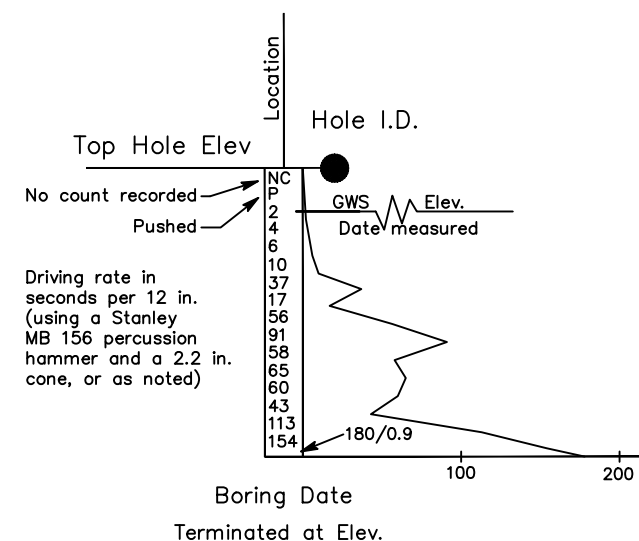
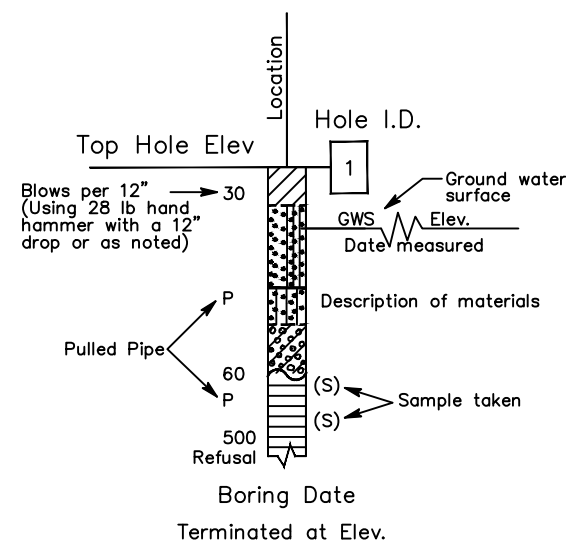
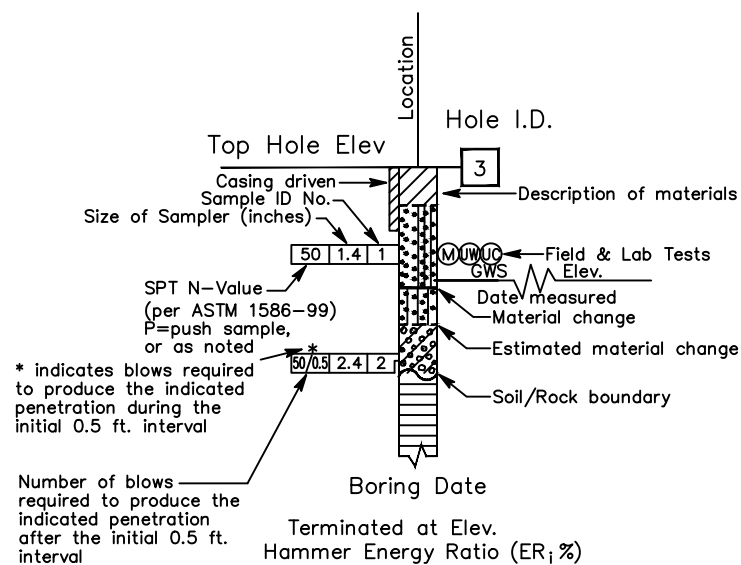
BLACKBURN CONSULTING
11521 BLOCKER DRIVE, SUITE 110
AUBURN, CA 95603

FILE No. 2103.1

BOREHOLE IDENTIFICATION		
Symbol	Hole Type	Description
	A	Auger Boring (hollow or solid stem bucket)
	R	Rotary drilled boring (conventional)
	RW	Rotary drilled with self-casing wire-line
	RC	Rotary core with continuously-sampled, self-casing wire-line
	P	Rotary percussion boring (air)
	R	Rotary drilled diamond core
	HD	Hand driven (1-inch soil tube)
	HA	Hand Auger
	D	Dynamic Cone Penetration Boring
	CPT	Cone Penetration Test (ASTM D 5778)
	O	Other (note on LOTB)

NOTE: Size in inches.

Consistency of Cohesive Soils				
Description	Shear Strength (tsf)	Pocket Penetrometer Measurement, PP, (tsf)	Torvane Measurement, TV, (tsf)	Vane Shear Measurement, VS, (tsf)
Very Soft	Less than 0.12	Less than 0.25	Less than 0.12	Less than 0.12
Soft	0.12 – 0.25	0.25 – 0.50	0.12 – 0.25	0.12 – 0.25
Medium Stiff	0.25 – 0.5	0.50 – 1	0.25 – 0.5	0.25 – 0.5
Stiff	0.5 – 1	1 – 2	0.5 – 1	0.5 – 1
Very Stiff	1 – 2	2 – 4	1 – 2	1 – 2
Hard	Greater than 2	Greater than 4	Greater than 4	Greater than 4



ROTARY BORING

HAND BORING

DYNAMIC CONE PENETRATION BORING

CONE PENETRATION TEST (CPT) SOUNDING

SOIL LEGEND

DESIGN OVERSIGHT _____ SIGN OFF DATE _____		DRAWN BY _____	CHECKED BY _____	FIELD INVESTIGATION BY: _____ DATE: _____	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER _____	BRIDGE NO. _____ POST MILE _____	LOG OF TEST BORINGS 2 OF 3				
GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)					ORIGINAL SCALE IN INCHES FOR REDUCED PLANS	0 1 2 3	UNIT: PROJECT NUMBER & PHASE: _____	CONTRACT NO.: _____	DISREGARD PRINTS BEARING EARLIER REVISION DATES →	REVISION DATES 	SHEET	OF

11/3/2023 21031 Ackerman Creek LOTB 2023.dwg

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (2010)

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

FIELD AND LABORATORY TESTING	
Ⓒ	Consolidation (ASTM D 2435)
ⒸL	Collapse Potential (ASTM D 5333)
ⒸP	Compaction Curve (CTM 216)
ⒸR	Corrosivity Testing (CTM 643, CTM 422, CTM 417)
ⒸU	Consolidated Undrained Triaxial (ASTM D 4767)
ⒹS	Direct Shear (ASTM D 3080)
ⒺI	Expansion Index (ASTM D 4829)
Ⓜ	Moisture Content (ASTM D 2216)
ⒼC	Organic Content-% (ASTM D 2974)
Ⓟ	Permeability (CTM 220)
ⓅA	Particle Size Analysis (ASTM D 422)
ⓅI	Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
ⓅL	Point Load Index (ASTM D 5731)
ⓅM	Pressure Meter
ⓅP	Pocket Penetrometer
Ⓡ	R-Value (CTM 301)
ⓈE	Sand Equivalent (CTM 217)
ⓈG	Specific Gravity (AASHTO T 100)
ⓈL	Shrinkage Limit (ASTM D 427)
ⓈW	Swell Potential (ASTM D 4546)
ⓈV	Pocket Torvane
ⓈC	Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
ⓈU	Unconsolidated Undrained Triaxial (ASTM D 2850)
ⓈW	Unit Weight (ASTM D 2937)

APPARENT DENSITY OF COHESIONLESS SOILS	
Description	SPT N ₆₀ -Value (Blows / 12 inches)
Very Loose	0 – 5
Loose	5 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	Greater than 50

MOISTURE	
Description	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visable free water

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

PARTICLE SIZE		
Description		Size
Boulder		Greater than 12"
Cobble		3" – 12"
Gravel	Coarse	3/4" – 3"
	Fine	1/5" – 3/4"
Sand	Coarse	1/16" – 1/5"
	Medium	1/64" – 1/16"
	Fine	Less than 1/300"

SOIL LEGEND									
ACKERMAN CREEK BRIDGE									
LOG OF TEST BORINGS 3 OF 3									
DESIGN OVERSIGHT		DRAWN BY	M. ROBERTSON	R. C. PICKARD		PREPARED FOR THE		BRIDGE NO.	
SIGN OFF DATE		CHECKED BY	T. BLACKBURN	FIELD INVESTIGATION BY:		MAX KATT		10C0065	
				DATE: AUGUST 2013		PROJECT ENGINEER		POST MILE	
						DEPARTMENT OF TRANSPORTATION		X.X	
GS CIVIL LOG OF TEST BORINGS SHEET (ENGLISH) (REV. 7/16/10)						UNIT: X		CONTRACT NO.: 01-XCXXXX	
						PROJECT NUMBER & PHASE: X		DISREGARD PRINTS BEARING EARLIER REVISION DATES	
						FILE => \$REQUEST		REVISION DATES	
								SHEET	
								OF	

ORIGINAL SCALE IN INCHES
FOR REDUCED PLANS

0 1 2 3

UNIT: X
PROJECT NUMBER & PHASE: X

DISREGARD PRINTS BEARING
EARLIER REVISION DATES

REVISION DATES
SHEET
OF

TIME PLOTTED => \$DATE
USERNAME => \$USER

LOGGED BY RCP	BEGIN DATE 8-7-13	COMPLETION DATE 8-7-13	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID A-13-03
DRILLING CONTRACTOR Taber	BOREHOLE LOCATION (Offset, Station, Line) ~13.00' Rt Sta ~131+07		SURFACE ELEVATION	
DRILLING METHOD Solid-Stem Auger	DRILL RIG CME 75		BOREHOLE DIAMETER 4 in	
SAMPLER TYPE(S) AND SIZE(S) (ID) 2.5-inch Cal Mod	HAMMER TYPE Safety semi-automatic drop (140#/ 30")		HAMMER EFFICIENCY, ERI ~80	
BOREHOLE BACKFILL AND COMPLETION Backfilled with cement	GROUNDWATER READINGS	DURING DRILLING None	AFTER DRILLING (DATE) None	TOTAL DEPTH OF BORING 6.5 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (tsf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
0			4" ASPHALT														
			29" AGGREGATE BASE														
1																	
2					1	17	26										
						14											
3			Poorly graded GRAVEL with SAND; GP; medium dense; olive brown; moist	Bag A	12										R		
4																	
5					2	16	34										
						16											
6						18											
			Bottom of borehole at 6.5 ft bgs														
7			Backfilled with cement														
8																	
9																	
10																	

BCI LOG FOR SOIL 2103.X LOGS.GPJ BCI 2012 LOG.GLB 4/17/15



blackburn
consulting

Blackburn Consulting

11521 Blocker Drive, Suite 110

Auburn, CA 95603

Phone: (530) 887-1494

Fax: (530) 887-1495

PROJECT NAME Ackerman Creek Bridge		FILE NO. 2103.X	HOLE ID A-13-03
COUNTY MEN	ROUTE CR 104	POSTMILE	
CLIENT			
PREPARED BY RCP	CHECKED BY PFF	SHEET 1 of 1	

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Fax: (530) 887-1495

LOGGED BY RCP	BEGIN DATE 8-7-13	COMPLETION DATE 8-7-13	BOREHOLE LOCATION (Lat/Long or North/East and Datum)	HOLE ID A-13-04
DRILLING CONTRACTOR Taber	BOREHOLE LOCATION (Offset, Station, Line) ~9.00' Rt Sta ~136+31		SURFACE ELEVATION	
DRILLING METHOD Solid-Stem Auger	DRILL RIG CME 75		BOREHOLE DIAMETER 4 in	
SAMPLER TYPE(S) AND SIZE(S) (ID) 2.5-inch Cal Mod	HAMMER TYPE Safety semi-automatic drop (140#/ 30")		HAMMER EFFICIENCY, ERI ~80	
BOREHOLE BACKFILL AND COMPLETION Backfilled with cement	GROUNDWATER READINGS	DURING DRILLING None	AFTER DRILLING (DATE) None	TOTAL DEPTH OF BORING 6.5 ft

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION/REMARKS	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Moisture Content (%)	Dry Unit Weight (pcf)	% <200 Sieve	Plasticity Index	Phi Angle (°)	Shear Strength (tsf)	Unconfined Compressive Strength (tsf)	Additional Lab Tests	Drilling Method	Casing Depth
	0		3" ASPHALT														
			6" AGGREGATE BASE ROCK														
	1		SANDY SILT; ML; very stiff to stiff; olive brown; moist														
				Bag B											R		
	2				1	7	11							PP = 2.5-2.75	CR		
						5											
	3					6											
	4																
	5				2	2	11							PP = 1.5-1.75			
						4											
	6					7											
	7		Bottom of borehole at 6.5 ft bgs Backfilled with cement														
	8																
	9																
	10																

BCI LOG FOR SOIL 2103.X LOGS.GPJ BCI 2012 LOG.GLB 4/17/15

 blackburn consulting	Blackburn Consulting 11521 Blocker Drive, Suite 110 Auburn, CA 95603 Phone: (530) 887-1494 Fax: (530) 887-1495		PROJECT NAME Ackerman Creek Bridge		FILE NO. 2103.X	HOLE ID A-13-04
	COUNTY MEN		ROUTE CR 104		POSTMILE	
	CLIENT					
	PREPARED BY RCP		CHECKED BY PFF		SHEET 1 of 1	

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Fax: (530) 887-1495

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

FIELD AND LABORATORY TESTS

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

SAMPLER GRAPHIC SYMBOLS

	Standard Penetration Test (SPT)
	2.5" ID Sampler
	2" ID Sampler
	Shelby Tube
	Piston Sampler
	NX Rock Core
	HQ Rock Core
	Bulk Sample
	Other (see remarks)

DRILLING METHOD SYMBOLS

	Auger Drilling		Rotary Drilling		Dynamic Cone or Hand Driven		Diamond Core
--	----------------	--	-----------------	--	-----------------------------	--	--------------

WATER LEVEL SYMBOLS

	First Water Level Reading (during drilling)
	Static Water Level Reading (short-term)
	Static Water Level Reading (long-term)



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BORING RECORD LEGEND

CONSISTENCY OF COHESIVE SOILS				
Descriptor	Unconfined Compressive Strength (tsf)	Pocket Penetrometer (tsf)	Torvane (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	0.25 - 0.50	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	0.50 - 1.0	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	1.0 - 2.0	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	2.0 - 4.0	1.0 - 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS	
Descriptor	SPT N ₆₀ - Value (blows / foot)
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

MOISTURE	
Descriptor	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	
Descriptor	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%

SOIL PARTICLE SIZE		
Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS	
Descriptor	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; it 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.

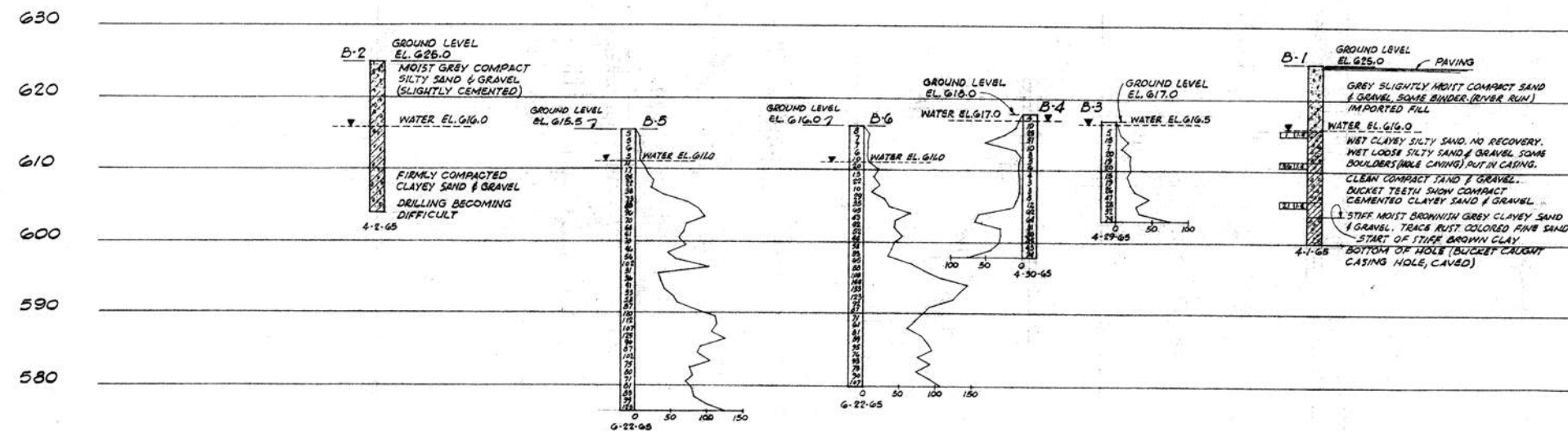
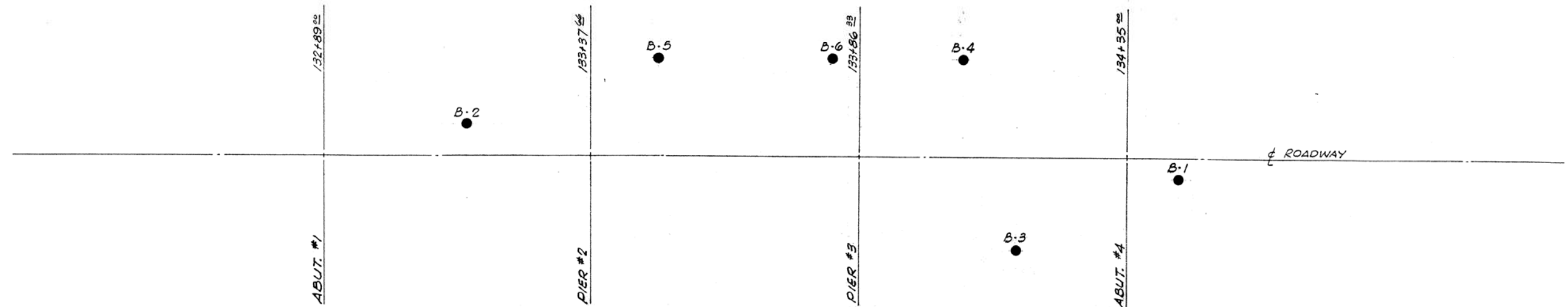
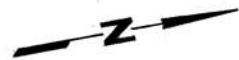
CEMENTATION	
Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptors and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (July 2007), Section 2, for tables of additional soil description components and discussion of soil description and identification.



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BORING RECORD LEGEND

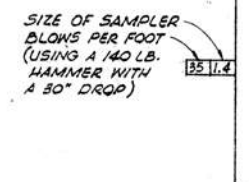


As Built
Corrections - None
By: G.W. Santken

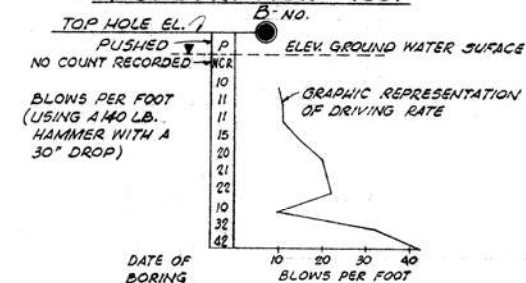
LEGEND OF EARTH MATERIALS

	GRAVEL		PEAT OR ORGANIC MATTER
	SAND		FILL MATERIAL
	SILT		SHALE
	CLAY		SANDSTONE
	SANDY CLAY OR CLAYEY SAND		LIMESTONE
	SANDY SILT OR SILTY SAND		METAMORPHIC ROCK
	SILTY CLAY OR CLAYEY SILT		IGNEOUS ROCK

ROTARY BORING



PENETRATION TEST



COUNTY OF MENDOCINO ACKERMAN CREEK BRIDGE LOG OF TEST BORINGS

HANS A. FEIBUSCH
consulting engineer
112 Market Street - San Francisco 11

Scale
1" = 10'
Date
6/23/65
Drawing No.
095-7

F-0052(6)

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
7	CAL.			61	161

DIST.	COUNTY	ROUTE	SECTION	SHEET NO.	TOTAL SHEETS
1	Men	1	6	12	162

DATE APPROVED: APR 1 1961

PLAN
Scale: 1" = 20'

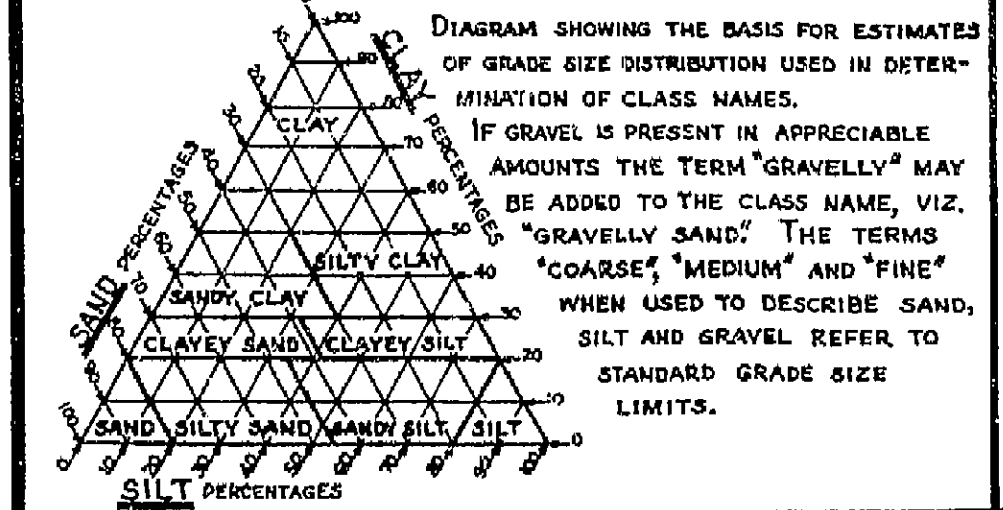
BM - Marked "T.S. 26" USC & GS Datum
brass cap set in concrete in open
field SW of De Bon Garage
208' Rt. Sta 143 + 10 "C"
Elev. 623.181

AS BUILT PLANS
Contract No. 62-147134
Date Completed
Document No. 10001062

PROFILE
Scale: 1" = 10'

AS BUILT
CORRECTIONS BY
CONTRACT NO. 62-147134-F
DATE

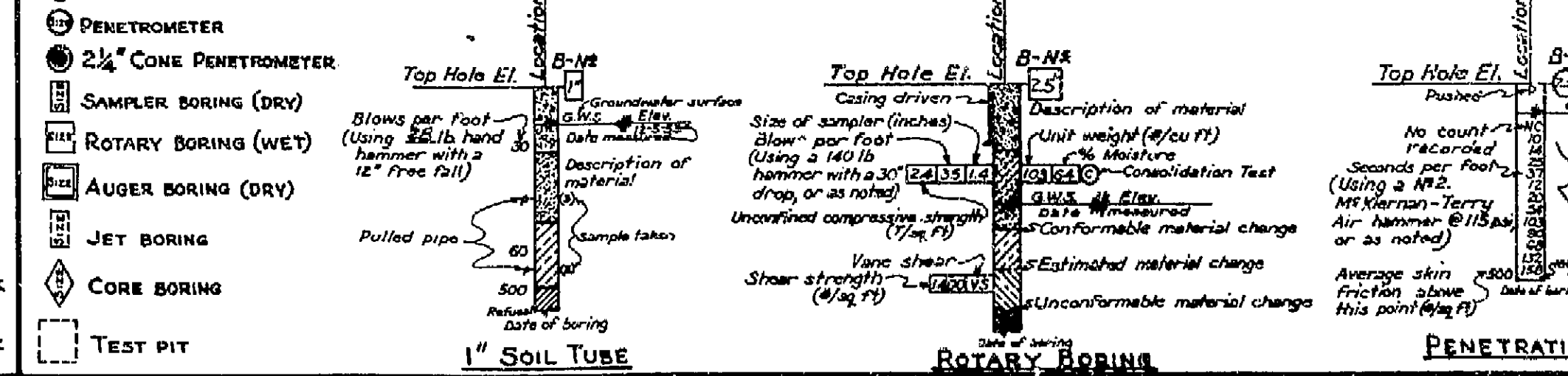
CLASSIFICATION OF MATERIAL BASED ON STANDARD GRADE SIZE LIMITS



LEGEND OF EARTH MATERIALS

GRAVEL	SILTY CLAY OR CLAYEY SILT
SAND	PEAT AND/OR ORGANIC MATTER
SILT	FILL MATERIAL
CLAY	IGNEOUS ROCK
SANDY CLAY OR CLAYEY SAND	SEDIMENTARY ROCK
SANDY SILT OR SILTY SAND	METAMORPHIC ROCK

LEGEND OF BORING OPERATIONS



NOTES

Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

ACKERMAN CREEK BRIDGE

LOG OF TEST BORINGS

SCALE: AS NOTED
BRIDGE 10-199 R/L
FILE E-30
DRAWING C-5601-12

I HEREBY CERTIFY THAT THIS IS A TRUE AND ACCURATE COPY OF THE ABOVE DOCUMENT TAKEN UNDER MY DIRECTION AND CONTROL ON THIS DATE IN SACRAMENTO, CALIFORNIA PURSUANT TO AUTHORIZATION BY THE DIRECTOR OF PUBLIC WORKS.

DATE 12-24-77 SIGNATURE James E. Cant TITLE Sr. RMC

FOUNDATION REPORT

N. State Street Bridge at Ackerman Creek

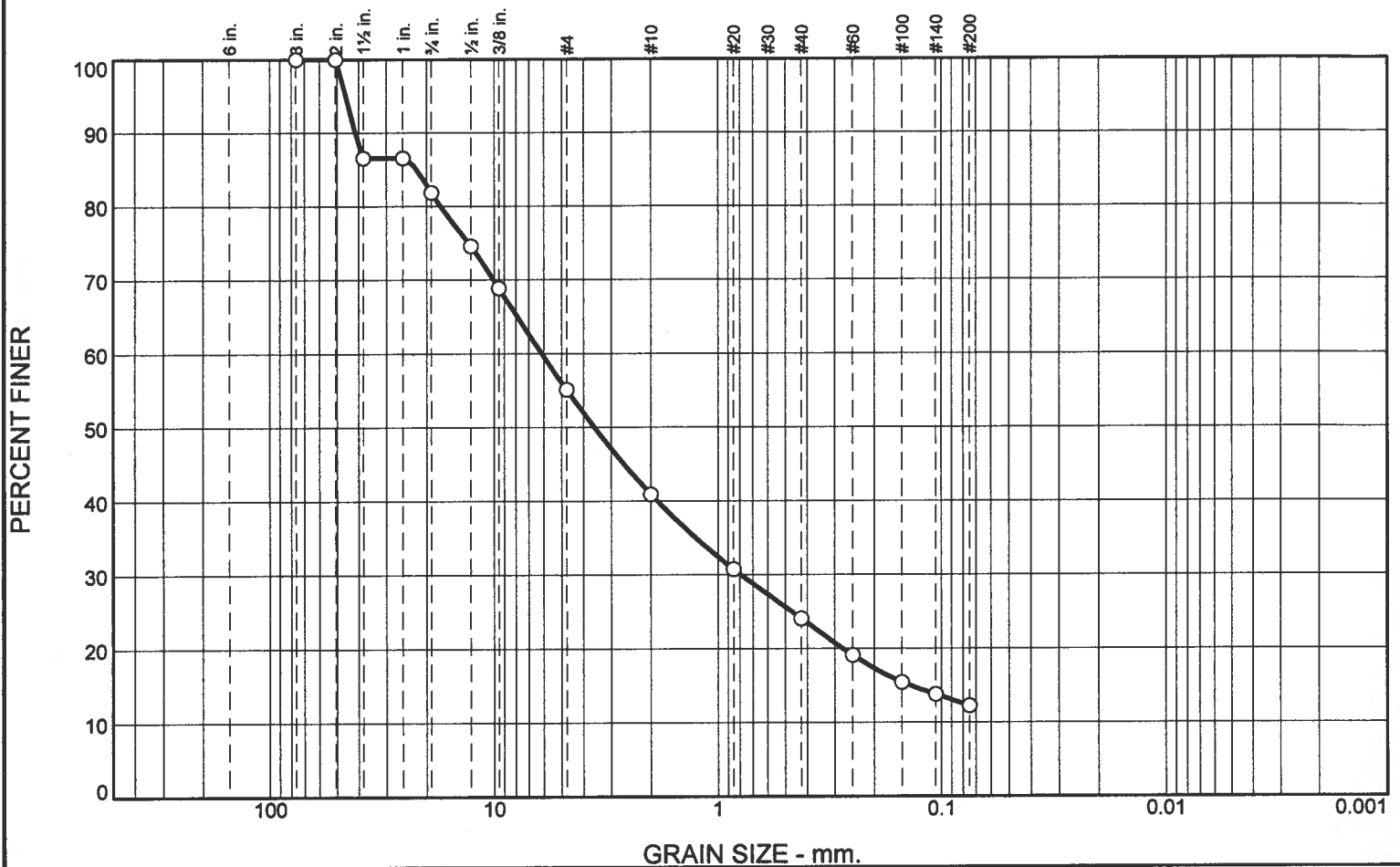
Bridge No. 10C-0065
Mendocino County, CA

November 2023

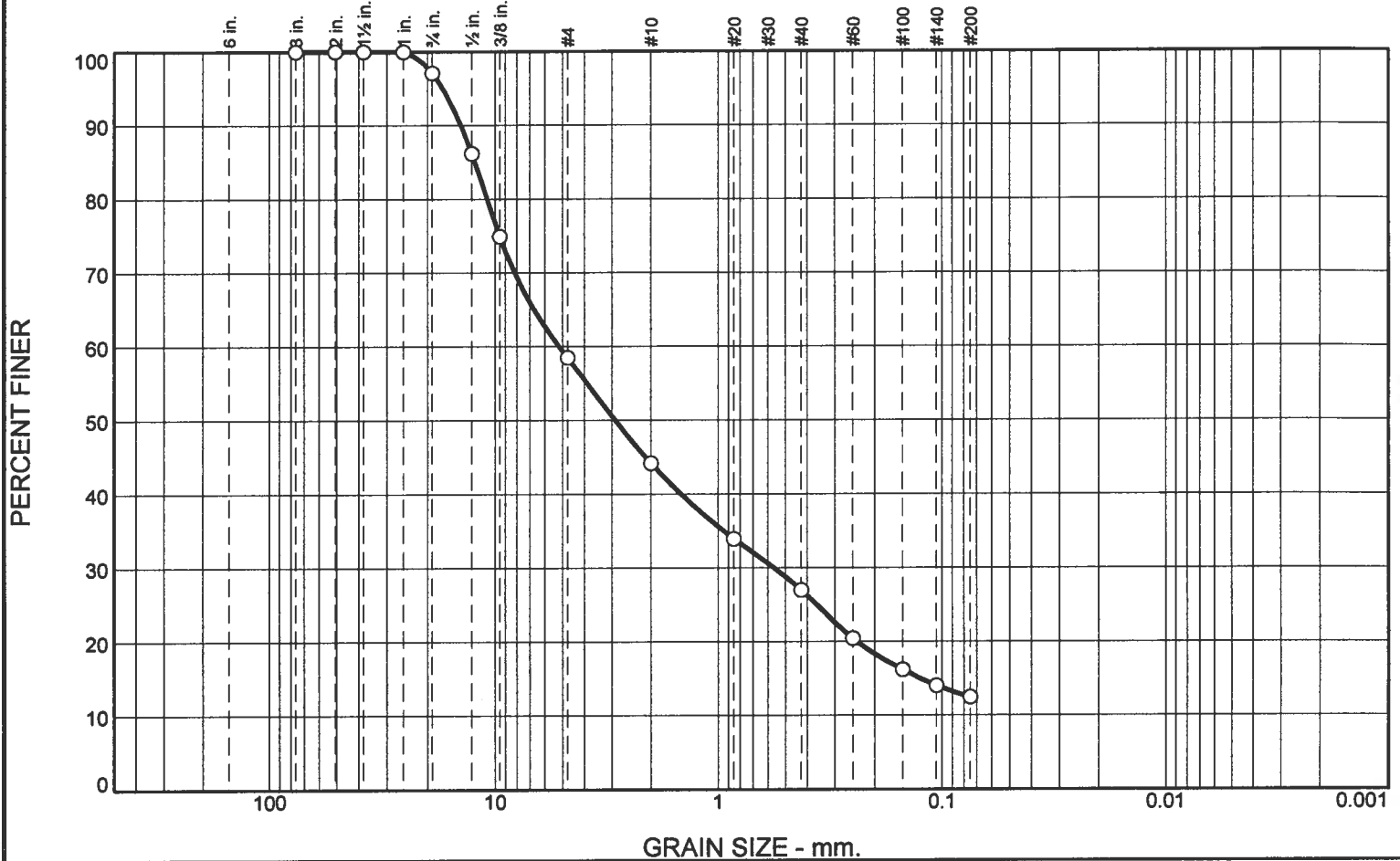
APPENDIX B

Laboratory Test Results

Particle Size Distribution Report - ASTM D 422



Particle Size Distribution Report - ASTM D 422



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	2.9	38.6	14.3	17.2	14.5	12.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3	100.0		
2	100.0		
1.5	100.0		
1	100.0		
.75	97.1		
.5	86.2		
.375	74.9		
#4	58.5		
#10	44.2		
#20	33.9		
#40	27.0		
#60	20.4		
#100	16.2		
#140	14.0		
#200	12.5		

* (no specification provided)

Soil Description		
CLAYEY GRAVEL with SAND, yellowish brown		
Atterberg Limits		
PL=	LL=	PI=
Coefficients		
D ₉₀ = 14.2048	D ₈₅ = 12.3174	D ₆₀ = 5.1801
D ₅₀ = 2.8981	D ₃₀ = 0.5626	D ₁₅ = 0.1255
D ₁₀ =	C _u =	C _c =
Classification		
USCS= GC	AASHTO=	
Remarks		

Source of Sample: R-13-02
Sample Number: 13c

Depth: 63.9-64.4'

Date:

Blackburn Consulting

W. Sacramento, CA

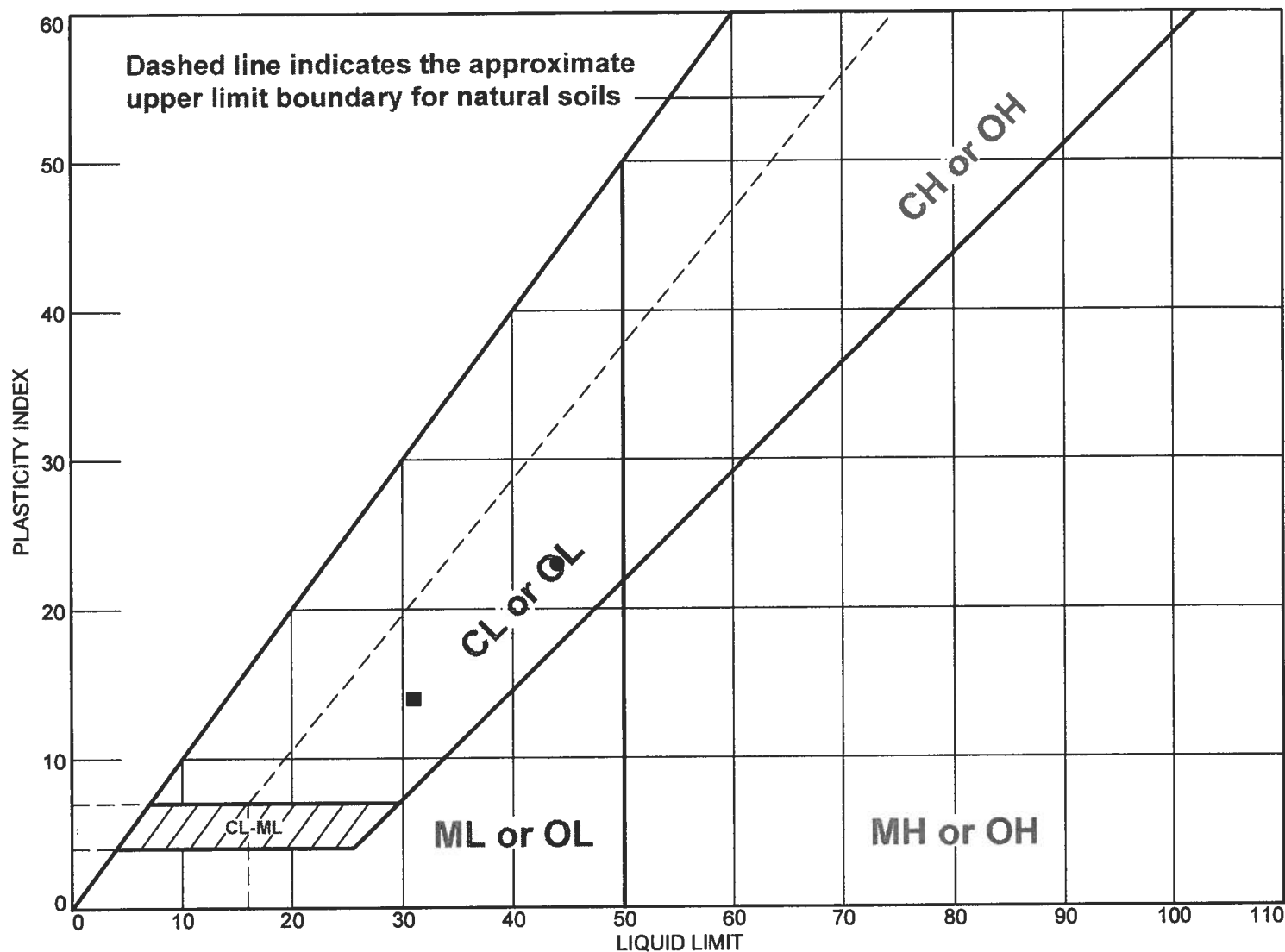
Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Project No: 2103.x

Figure

Atterberg Limits - ASTM D 4318



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean CLAY, very dark bluish gray	44	21	23			CL
■	Lean CLAY, olive brown	31	17	14			CL

Project No. 2103.x **Client:** Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

● **Source of Sample:** R-13-01

Depth: 34.5-35.0'

Sample Number: 7c

■ **Source of Sample:** R-13-01

Depth: 49.5-50.0'

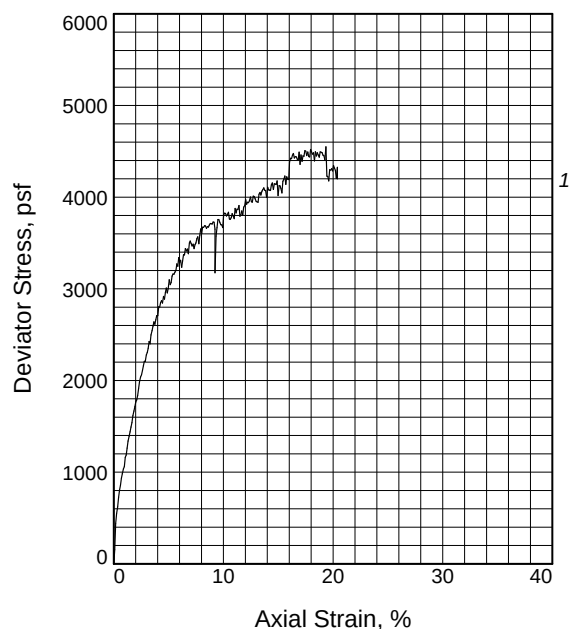
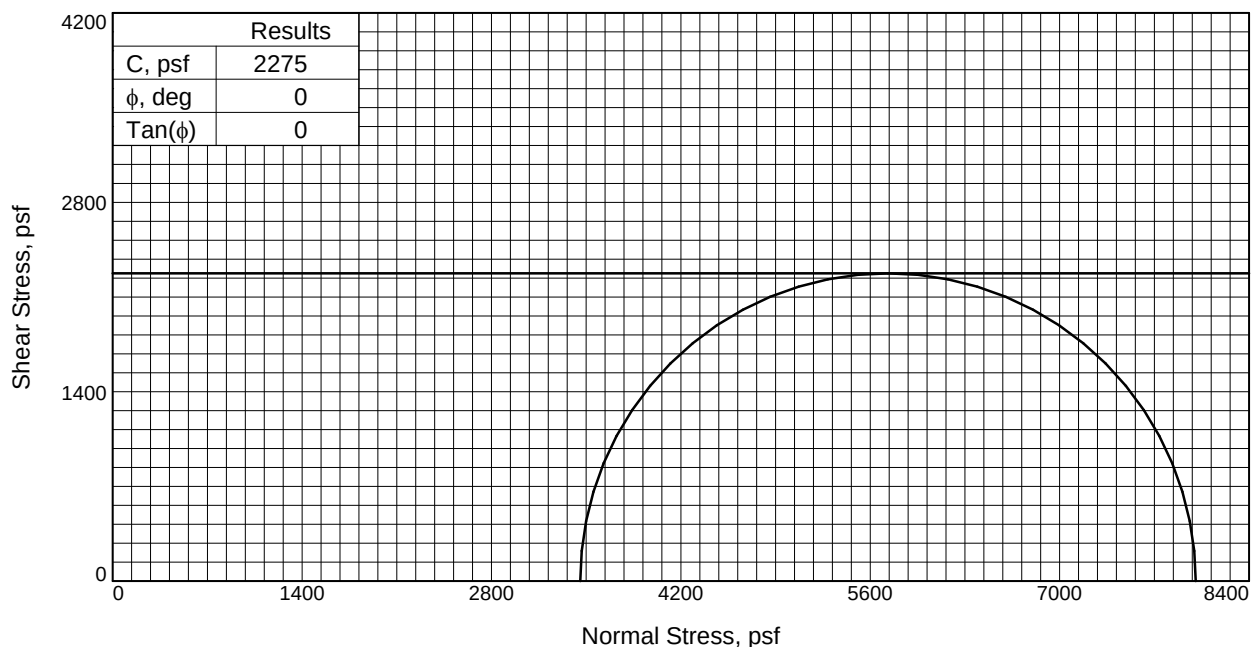
Sample Number: 10c

Remarks:

Blackburn Consulting

W. Sacramento, CA

Figure



Sample No.		1
Initial	Water Content, %	22.4
	Dry Density, pcf	104.0
	Saturation, %	97.3
	Void Ratio	0.6209
	Diameter, in.	2.41
	Height, in.	5.90
At Test	Water Content, %	22.4
	Dry Density, pcf	104.0
	Saturation, %	97.3
	Void Ratio	0.6209
	Diameter, in.	2.41
	Height, in.	5.90
Strain rate, in./min.		0.02
Back Pressure, psf		0
Cell Pressure, psf		3456
Fail. Stress, psf		4550
Strain, %		19.3
Ult. Stress, psf		
Strain, %		
σ_1 Failure, psf		8006
σ_3 Failure, psf		3456

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4"

Description: Lean CLAY, very dark greenish gray

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

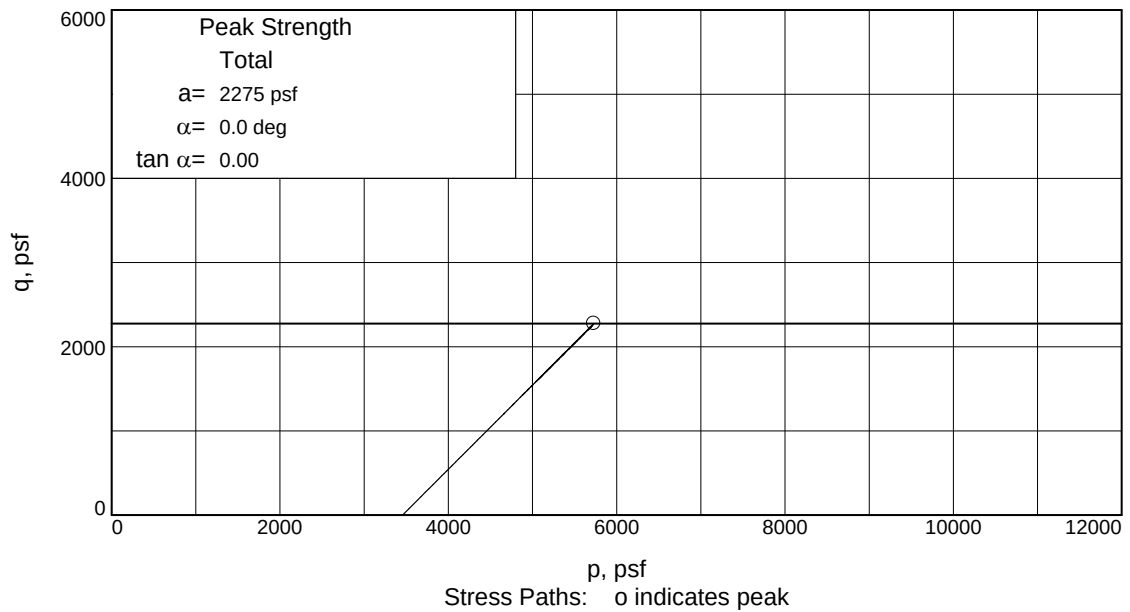
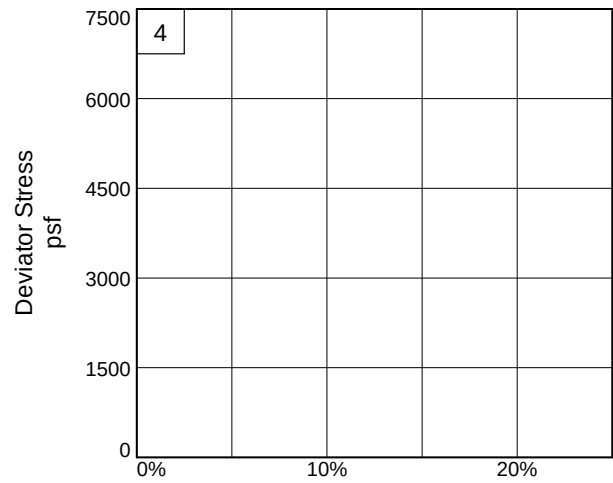
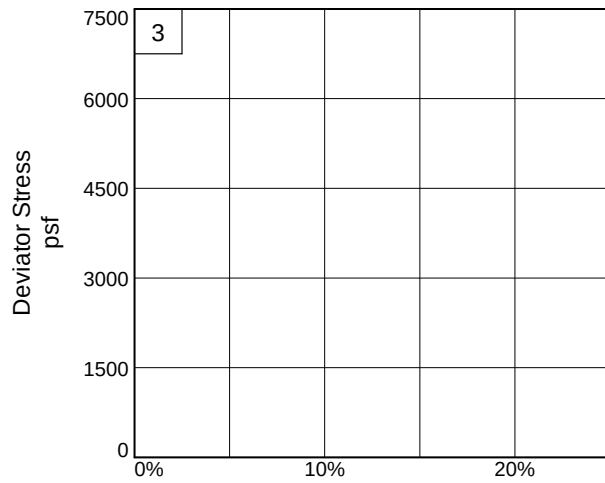
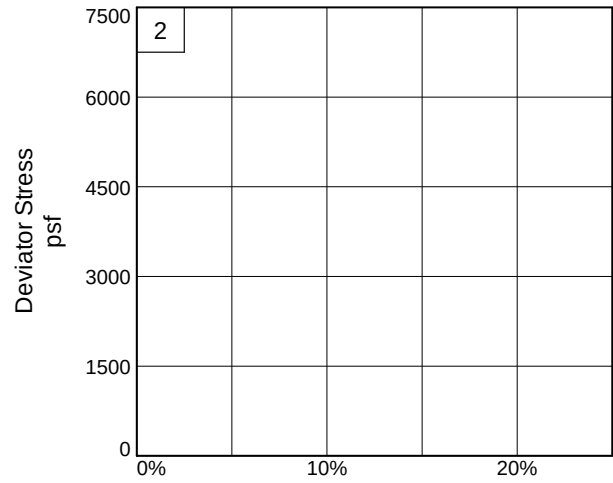
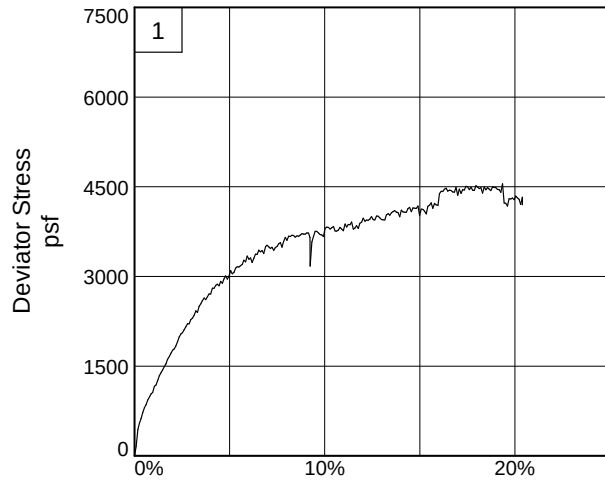
Source of Sample: R-13-01 **Depth:** 29.5-30.0'

Sample Number: 6c

Proj. No.: 2103.x

Date Sampled: 8/12/13

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-01

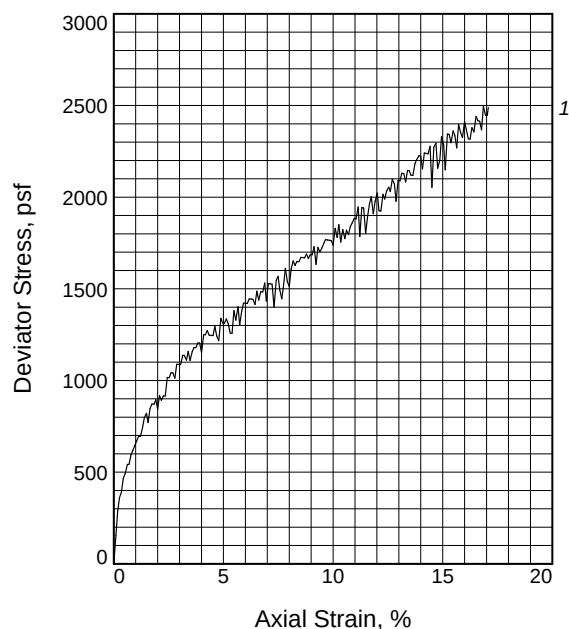
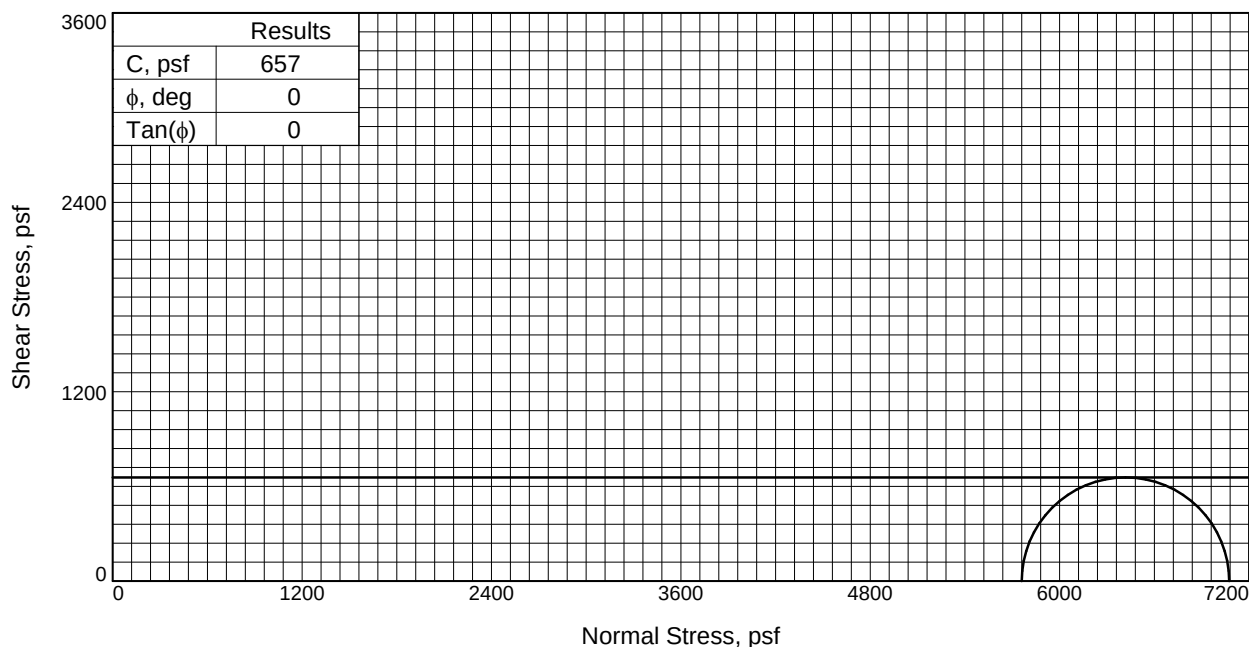
Depth: 29.5-30.0'

Sample Number: 6c

Project No.: 2103.x

Figure _____

Blackburn Consulting



Sample No.		1
Initial	Water Content, %	16.0
	Dry Density, pcf	119.3
	Saturation, %	98.5
	Void Ratio	0.4496
	Diameter, in.	2.41
	Height, in.	5.91
At Test	Water Content, %	16.0
	Dry Density, pcf	119.3
	Saturation, %	98.5
	Void Ratio	0.4496
	Diameter, in.	2.41
	Height, in.	5.91
Strain rate, in./min.		0.03
Back Pressure, psf		0
Cell Pressure, psf		5760
Fail. Stress, psf		1314
Strain, %		5.0
Ult. Stress, psf		
Strain, %		
σ_1 Failure, psf		7074
σ_3 Failure, psf		5760

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4" Cal Mod

Description: Lean CLAY with SAND, olive brown

Assumed Specific Gravity= 2.77

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-01

Depth: 49.049.5

Sample Number: 10b

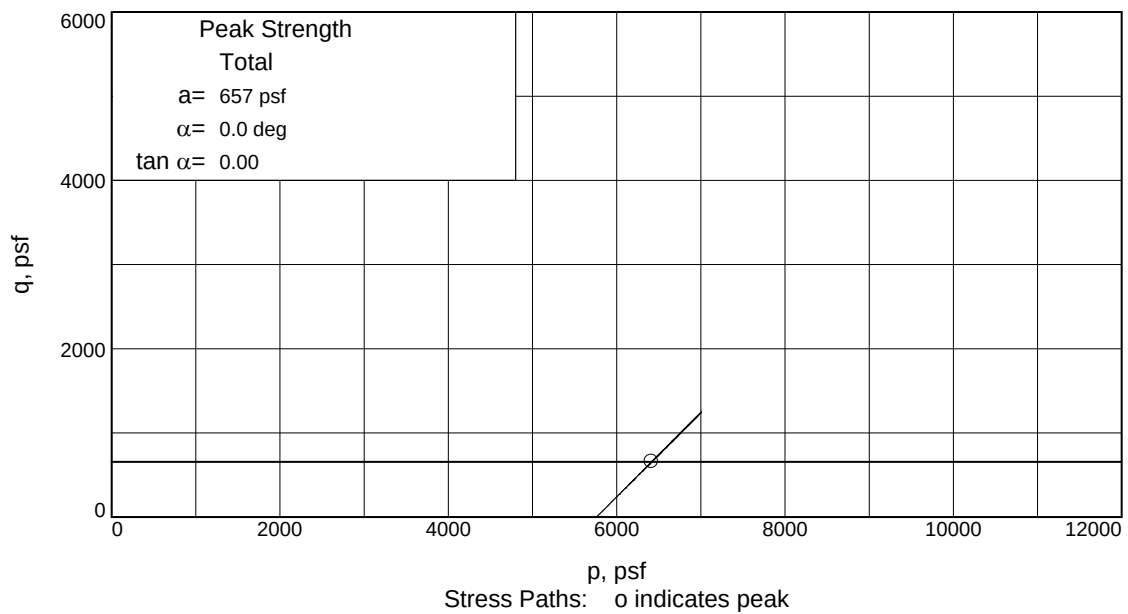
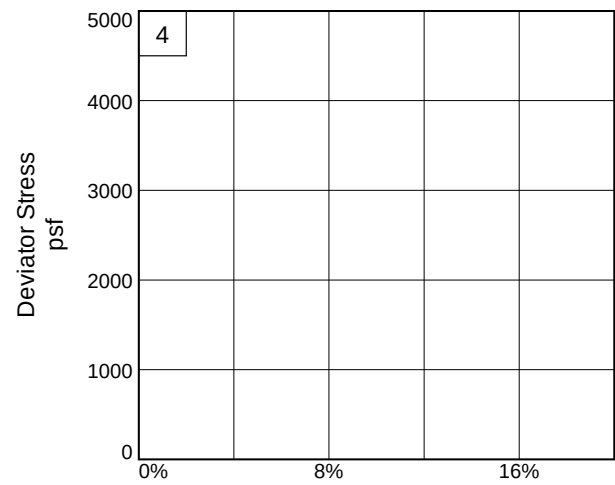
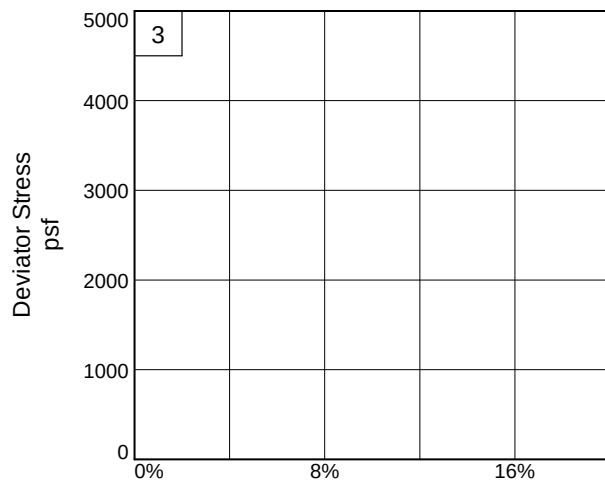
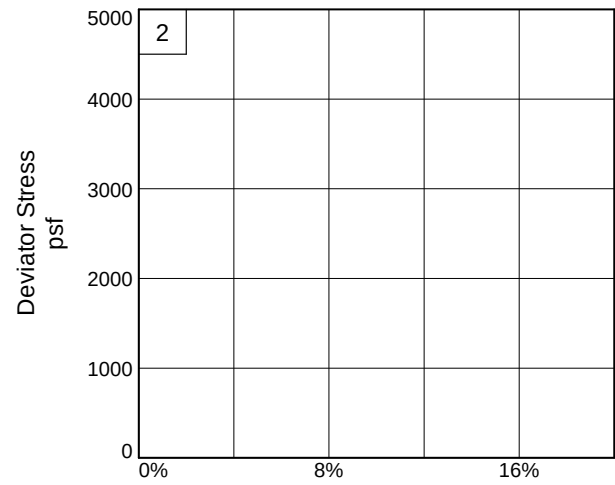
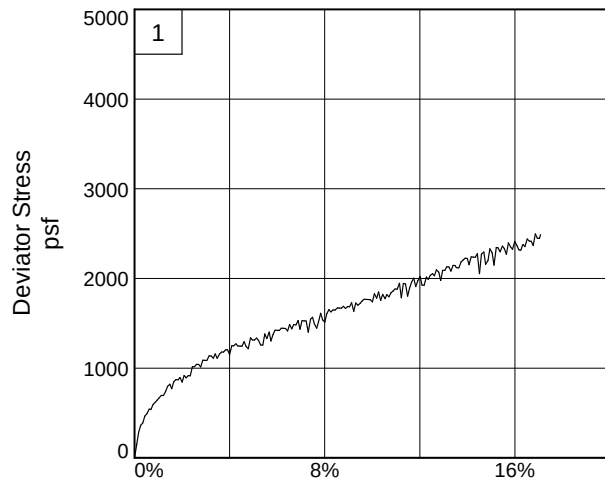
Proj. No.: 2103.x

Date Sampled:

TRIAXIAL SHEAR TEST REPORT

Blackburn Consulting

W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-01

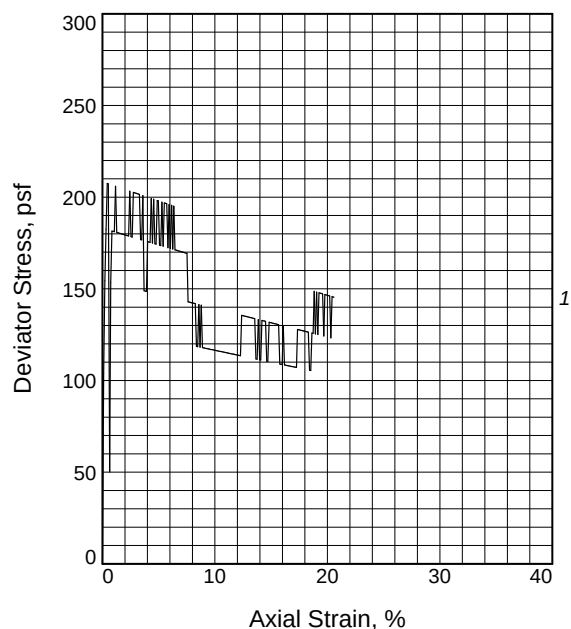
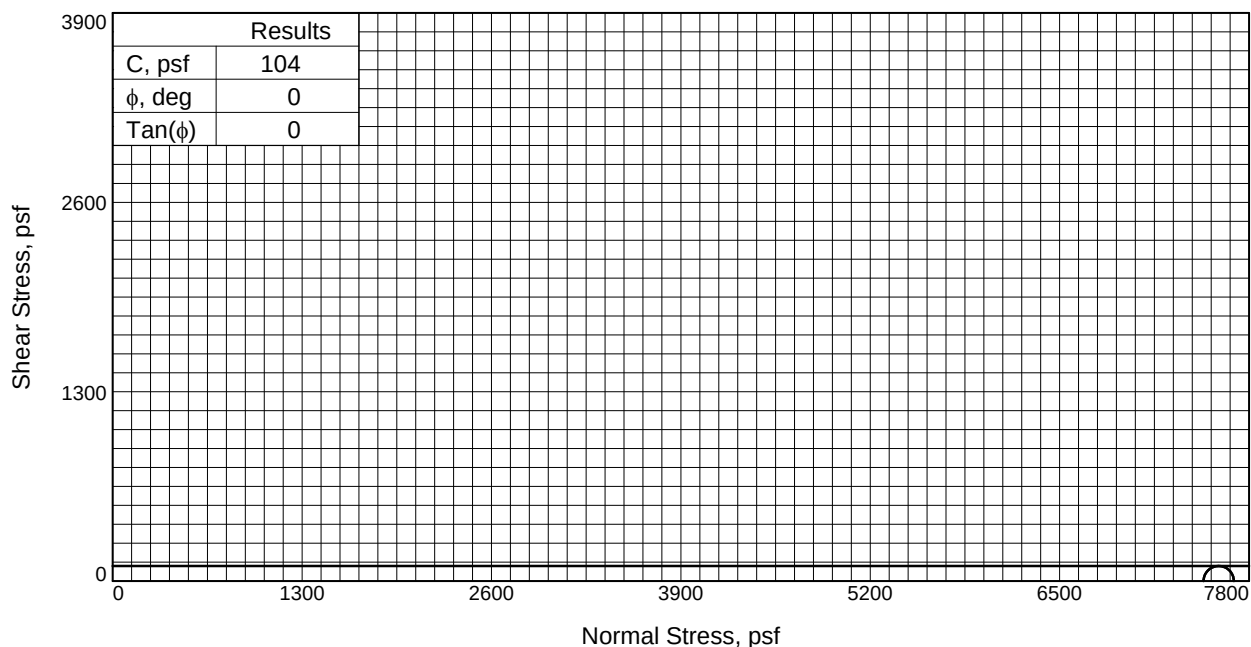
Depth: 49.049.5

Sample Number: 10b

Project No.: 2103.x

Figure _____

Blackburn Consulting



Sample No.		1
Initial	Water Content, %	16.2
	Dry Density, pcf	116.2
	Saturation, %	97.3
	Void Ratio	0.4509
	Diameter, in.	2.41
	Height, in.	4.86
At Test	Water Content, %	16.2
	Dry Density, pcf	116.2
	Saturation, %	97.3
	Void Ratio	0.4509
	Diameter, in.	2.41
	Height, in.	4.86
Strain rate, in./min.		0.05
Back Pressure, psf		0
Cell Pressure, psf		7488
Fail. Stress, psf		207
Strain, %		0.4
Ult. Stress, psf		
Strain, %		
σ_1 Failure, psf		7695
σ_3 Failure, psf		7488

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4" Cal Mod

Description: GRAVELLY CLAY, yellowish brown

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

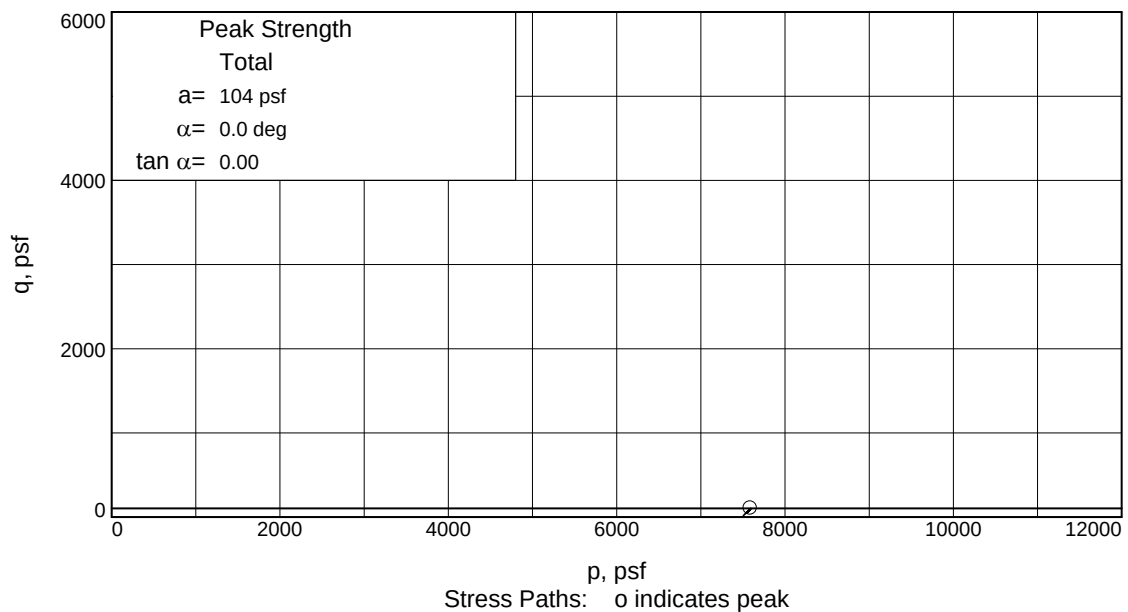
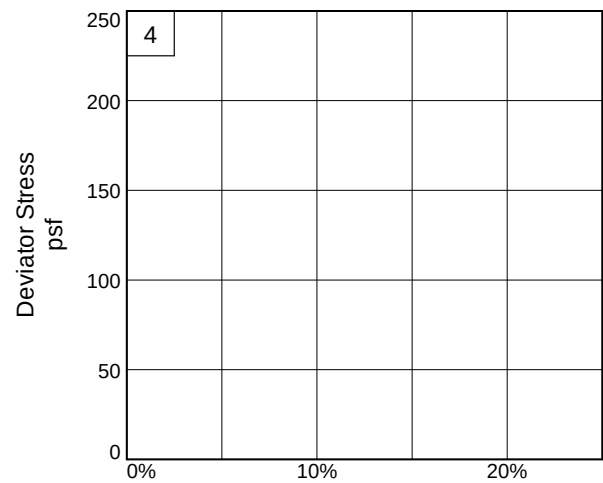
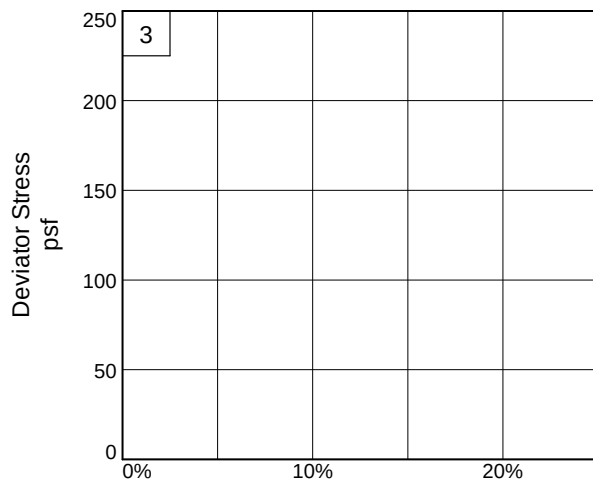
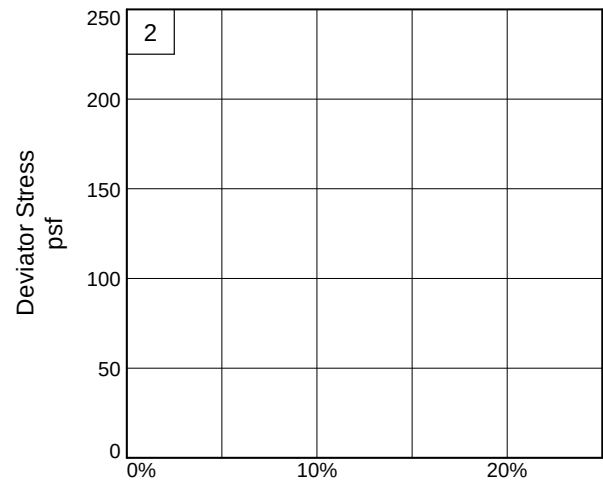
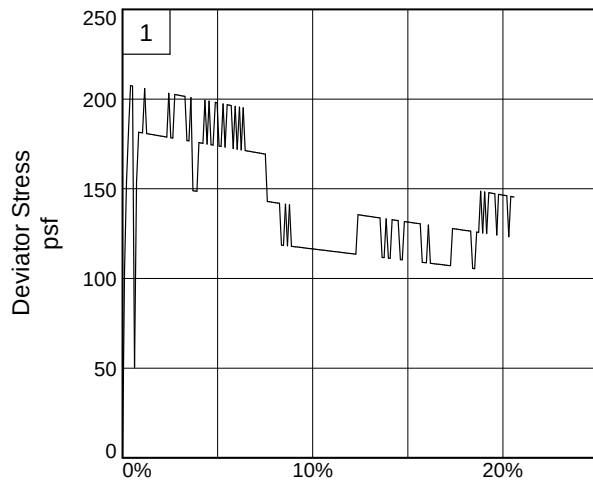
Source of Sample: R-13-01 **Depth:** 64.0-64.5'

Sample Number: 13b

Proj. No.: 2103.x

Date Sampled:

TRIAXIAL SHEAR TEST REPORT
 Blackburn Consulting
 W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-01

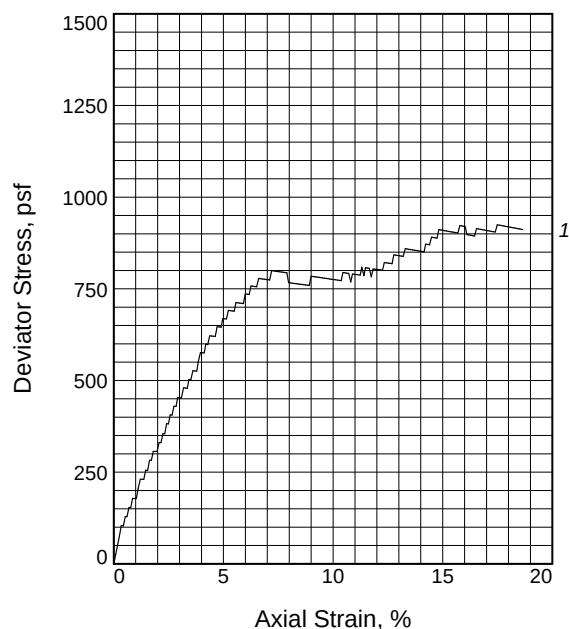
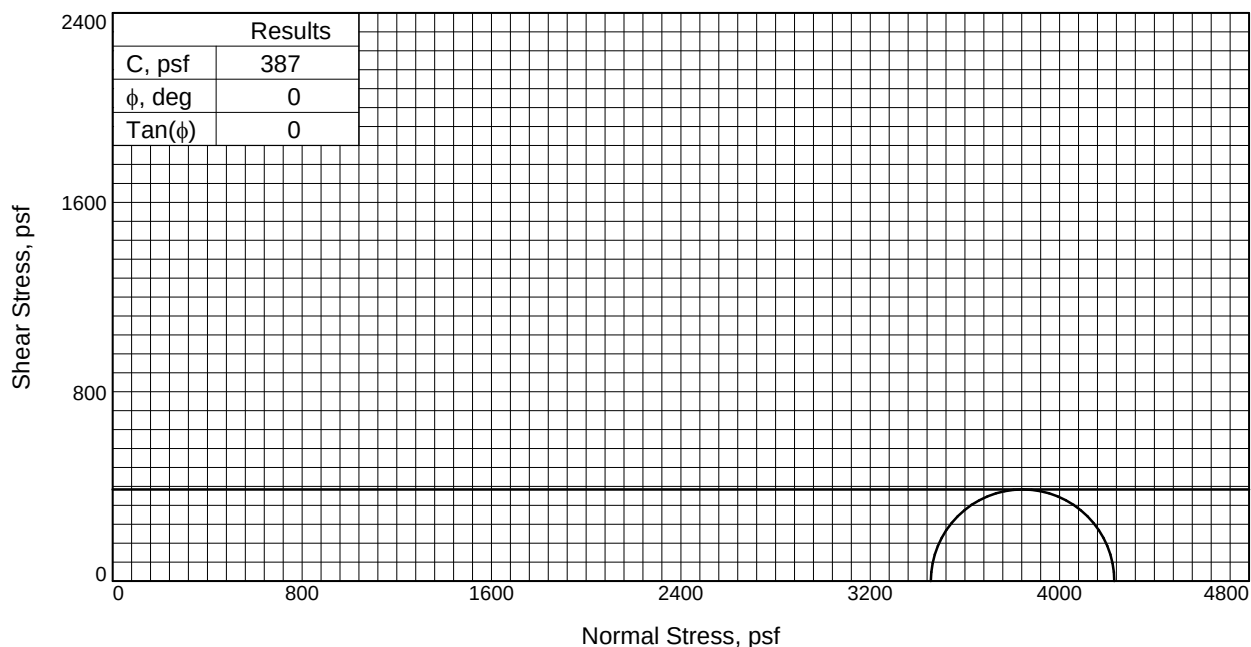
Depth: 64.0-64.5'

Sample Number: 13b

Project No.: 2103.x

Figure _____

Blackburn Consulting



Sample No.		1
Initial	Water Content, %	20.5
	Dry Density, pcf	107.3
	Saturation, %	97.1
	Void Ratio	0.5704
	Diameter, in.	2.41
	Height, in.	6.00
At Test	Water Content, %	20.5
	Dry Density, pcf	107.3
	Saturation, %	97.1
	Void Ratio	0.5704
	Diameter, in.	2.41
	Height, in.	6.00
Strain rate, in./min.		0.09
Back Pressure, psf		0
Cell Pressure, psf		3456
Fail. Stress, psf		775
Strain, %		7.0
Ult. Stress, psf		
Strain, %		
σ_1 Failure, psf		4231
σ_3 Failure, psf		3456

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4" Cal Mod

Description: Lean CLAY with SAND, dark greenish gray

Assumed Specific Gravity= 2.70

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

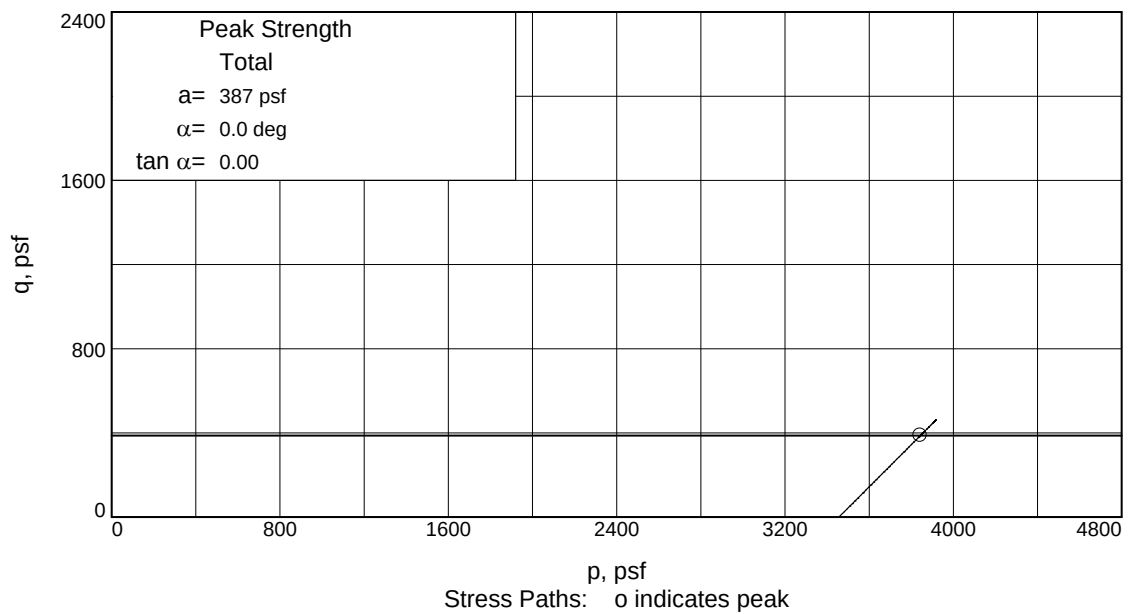
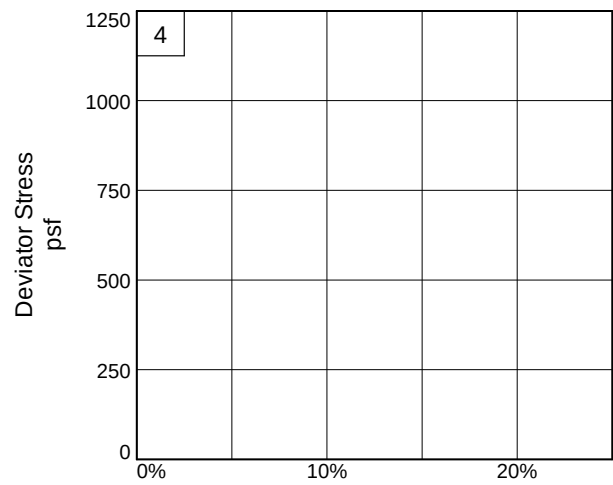
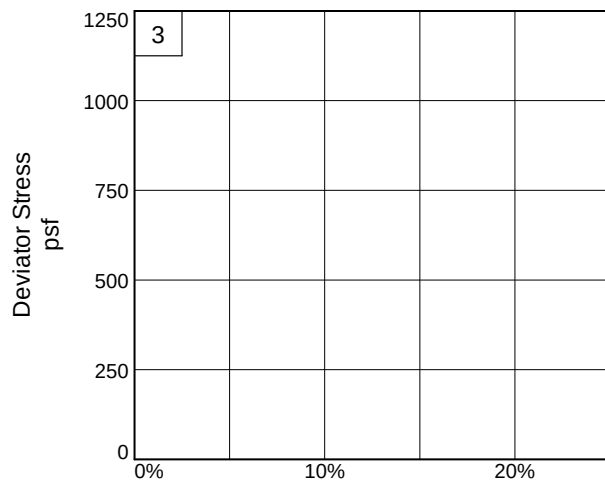
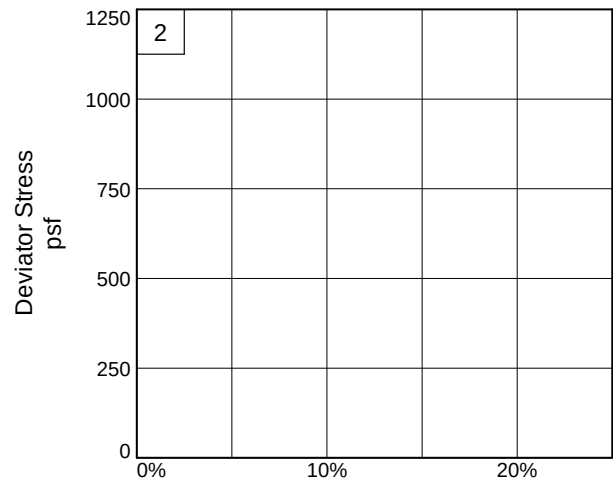
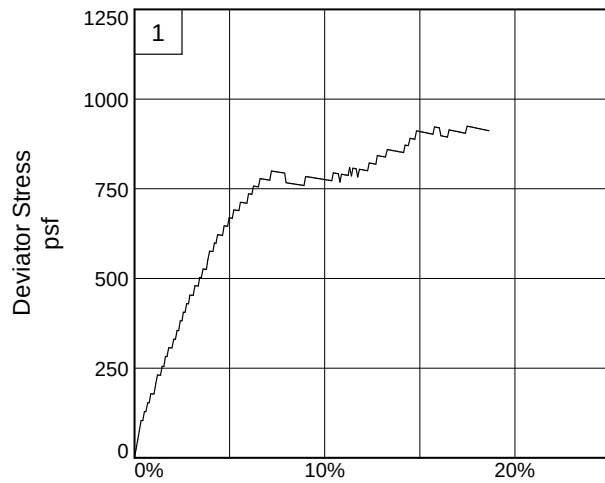
Source of Sample: R-13-02 **Depth:** 29.5-30.0'

Sample Number: 6c

Proj. No.: 2103.x

Date Sampled:

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-02

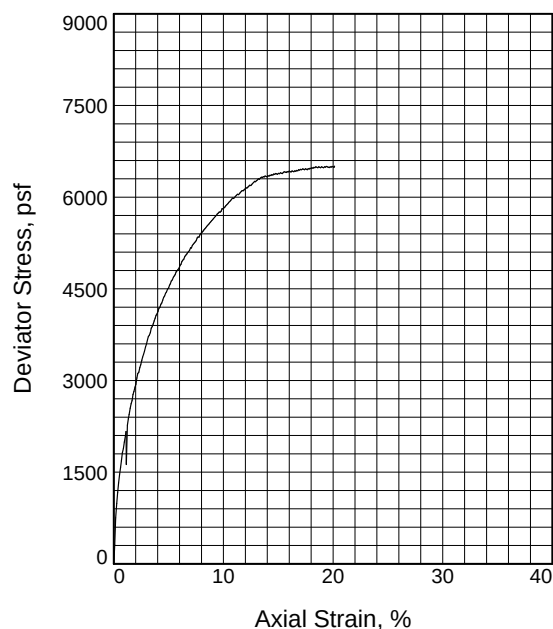
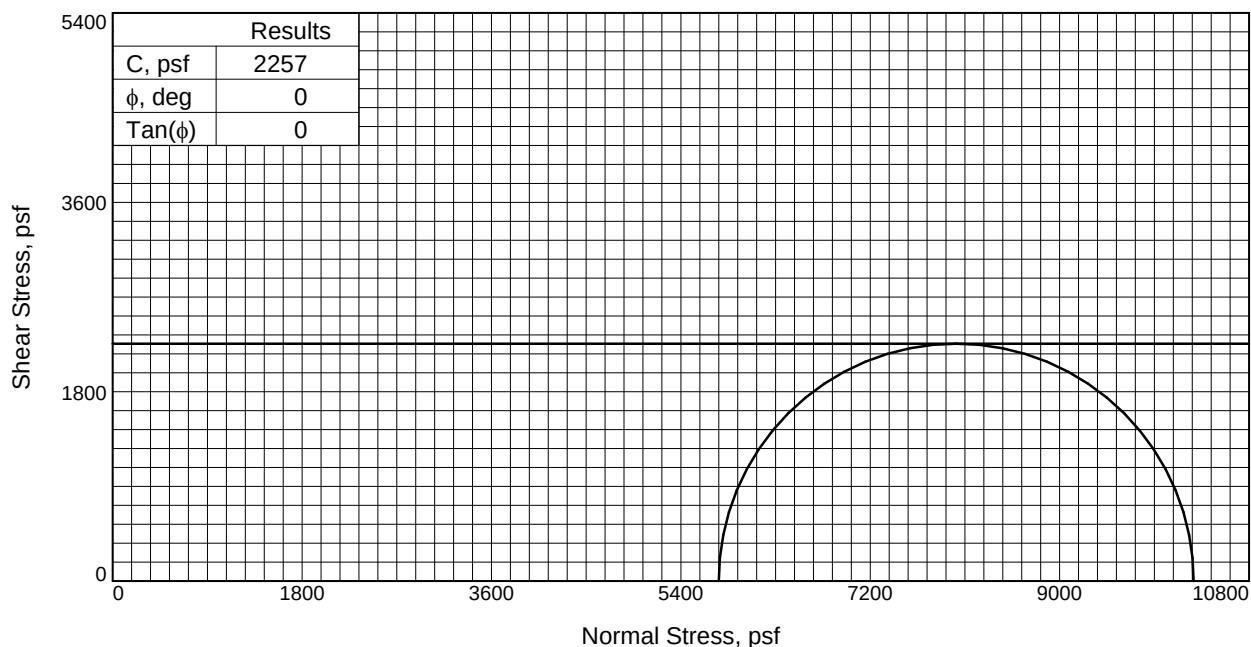
Depth: 29.5-30.0'

Sample Number: 6c

Project No.: 2103.x

Figure _____

Blackburn Consulting



Sample No. 1	
Initial	Water Content, % 20.0
	Dry Density, pcf 110.0
	Saturation, % 98.1
	Void Ratio 0.5612
	Diameter, in. 2.41
	Height, in. 5.89
At Test	Water Content, % 20.0
	Dry Density, pcf 110.0
	Saturation, % 98.1
	Void Ratio 0.5612
	Diameter, in. 2.41
	Height, in. 5.89
Strain rate, in./min. 0.08	
Back Pressure, psf 0	
Cell Pressure, psf 5760	
Fail. Stress, psf 4514	
Strain, % 5.0	
Ult. Stress, psf	
Strain, %	
σ_1 Failure, psf 10274	
σ_3 Failure, psf 5760	

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4" Cal Mod

Description: Lean CLAY, olive brown

Assumed Specific Gravity= 2.75

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

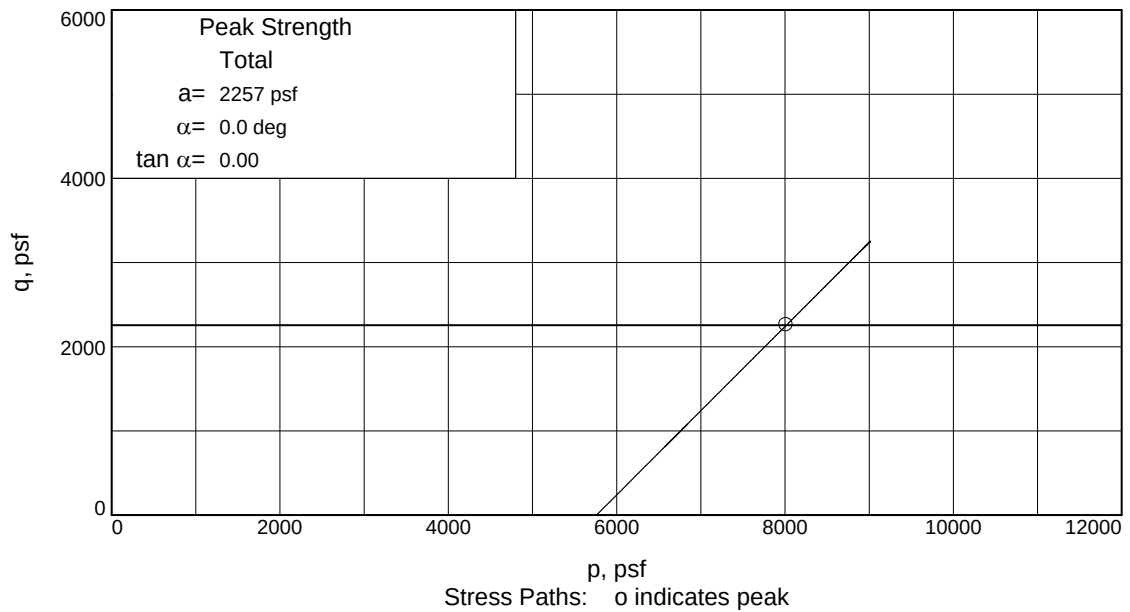
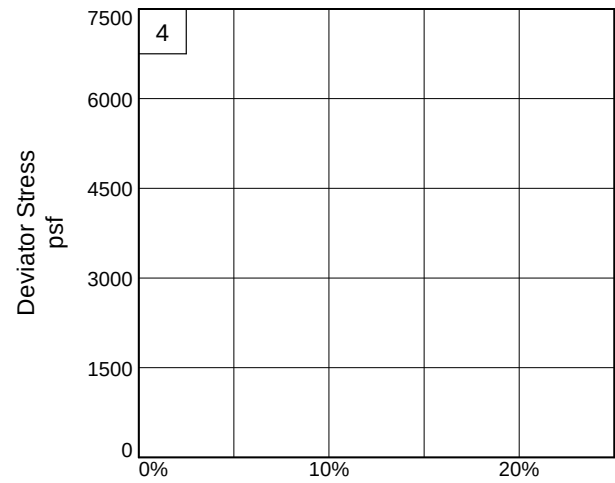
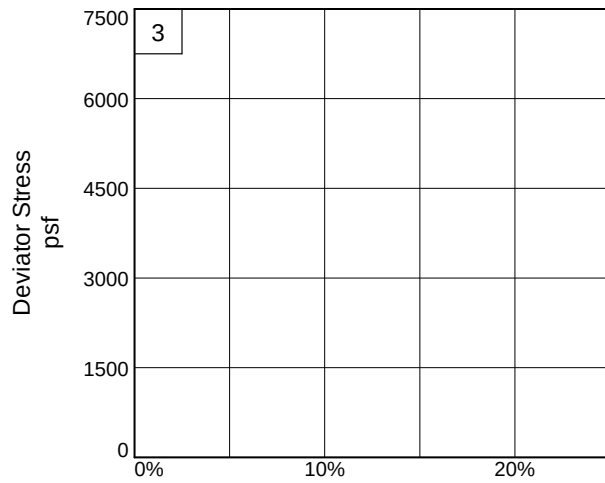
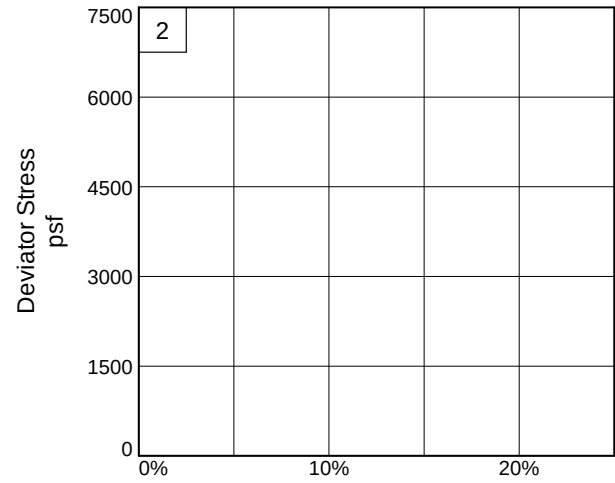
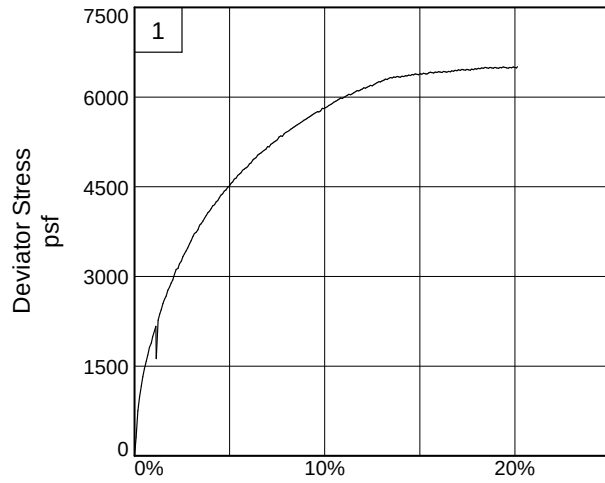
Source of Sample: R-13-02 **Depth:** 49.0-49.5'

Sample Number: 10b

Proj. No.: 2103.x

Date Sampled:

TRIAXIAL SHEAR TEST REPORT
Blackburn Consulting
W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-02

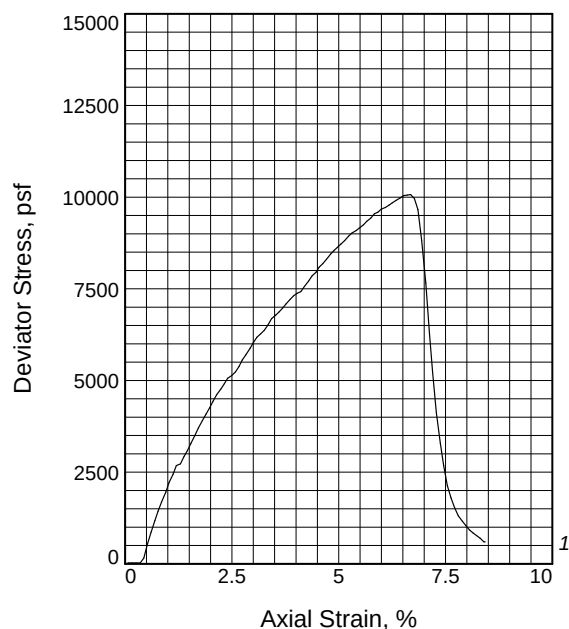
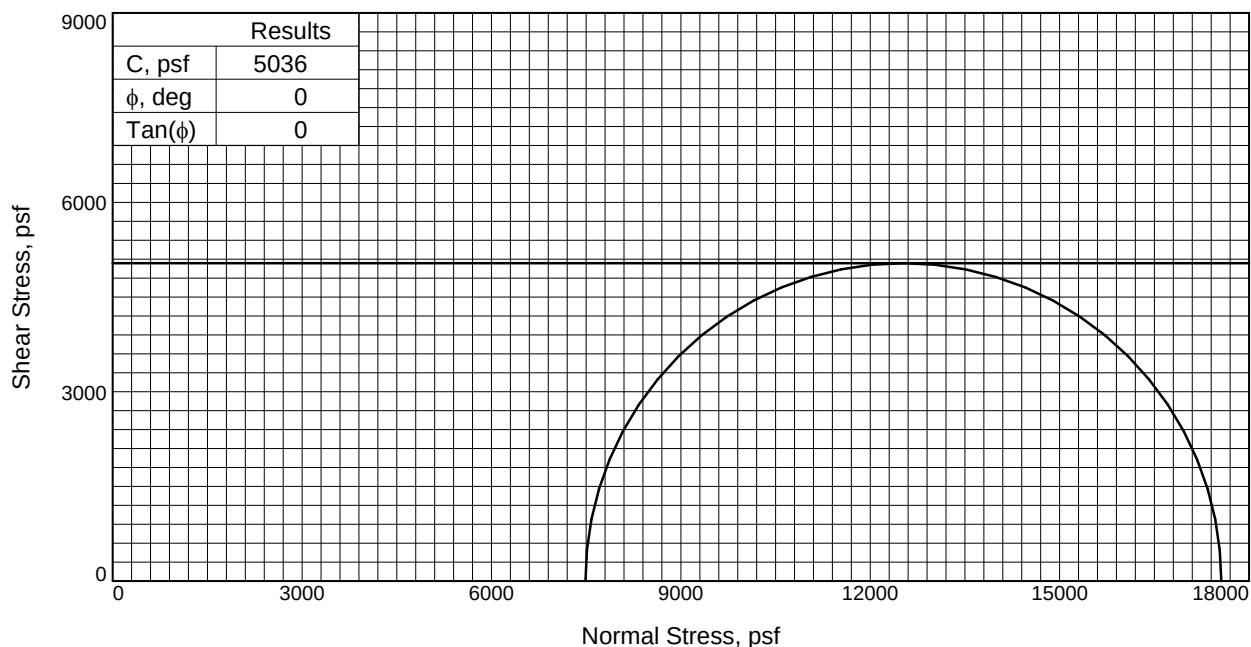
Depth: 49.0-49.5'

Sample Number: 10b

Project No.: 2103.x

Figure _____

Blackburn Consulting



Sample No.		1
Initial	Water Content, %	12.8
	Dry Density, pcf	127.1
	Saturation, %	98.4
	Void Ratio	0.3604
	Diameter, in.	2.40
	Height, in.	6.00
At Test	Water Content, %	12.8
	Dry Density, pcf	127.1
	Saturation, %	98.4
	Void Ratio	0.3604
	Diameter, in.	2.40
	Height, in.	6.00
Strain rate, in./min.		0.08
Back Pressure, psf		0
Cell Pressure, psf		7488
Fail. Stress, psf		10072
Strain, %		6.7
Ult. Stress, psf		
Strain, %		
σ_1 Failure, psf		17560
σ_3 Failure, psf		7488

Type of Test:

Unconsolidated Undrained

Sample Type: 2.4" Cal Mod

Description: Lean CLAY with GRAVEL, olive brown

Assumed Specific Gravity= 2.77

Remarks:

Figure _____

Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

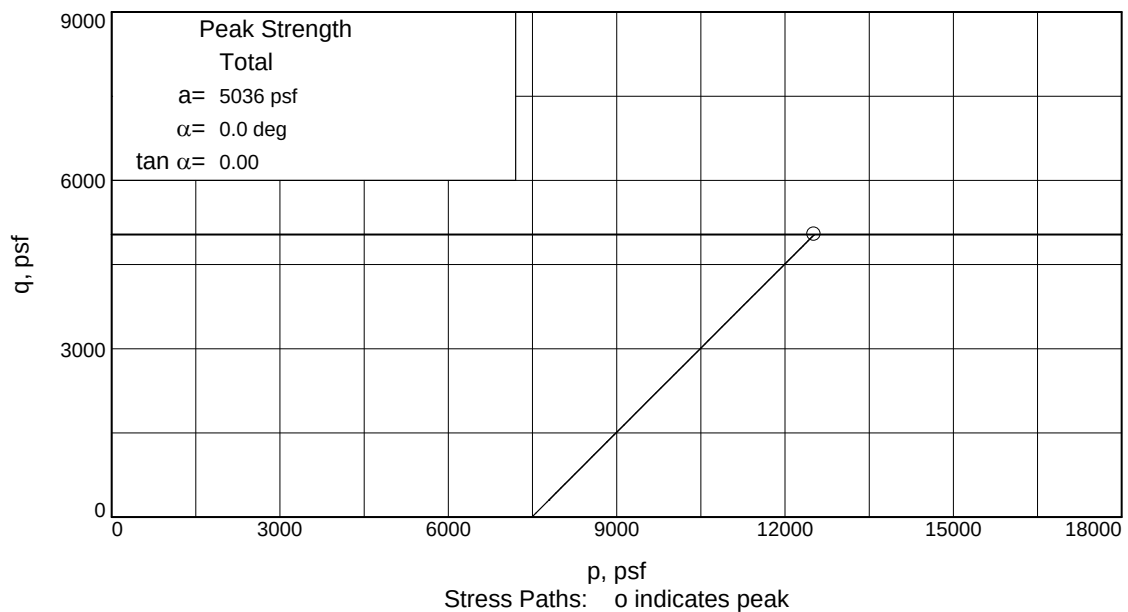
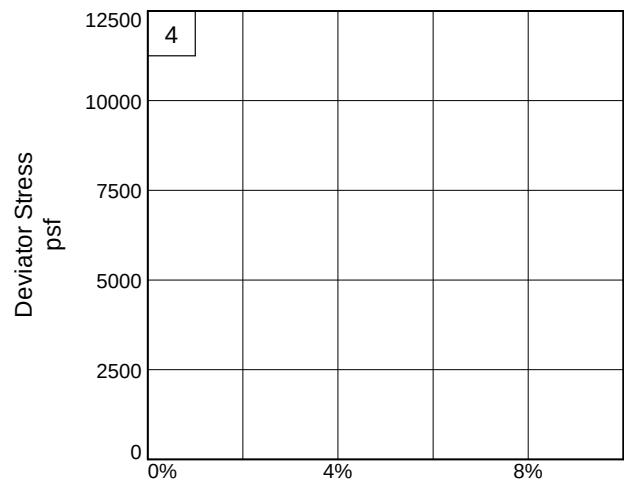
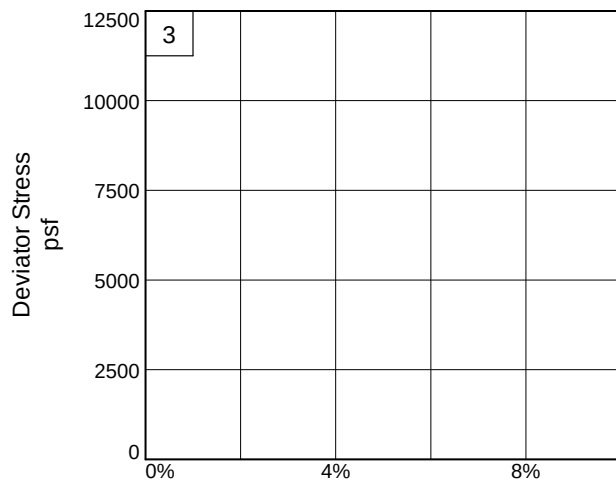
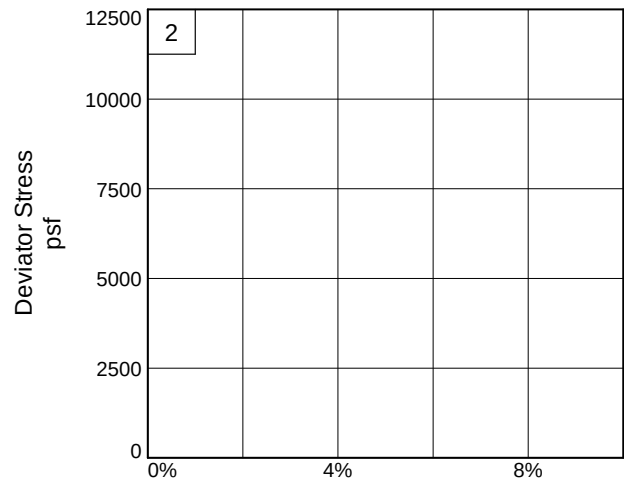
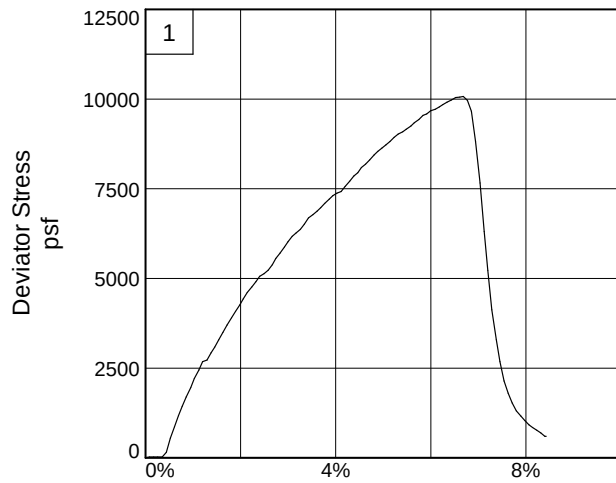
Source of Sample: R-13-02 **Depth:** 69.5-70.0'

Sample Number: 14c

Proj. No.: 2103.x

Date Sampled:

TRIAXIAL SHEAR TEST REPORT
 Blackburn Consulting
 W. Sacramento, CA



Client: Quincy Engineering

Project: Ackerman Creek Bridge at North State Street

Source of Sample: R-13-02

Depth: 69.5-70.0'

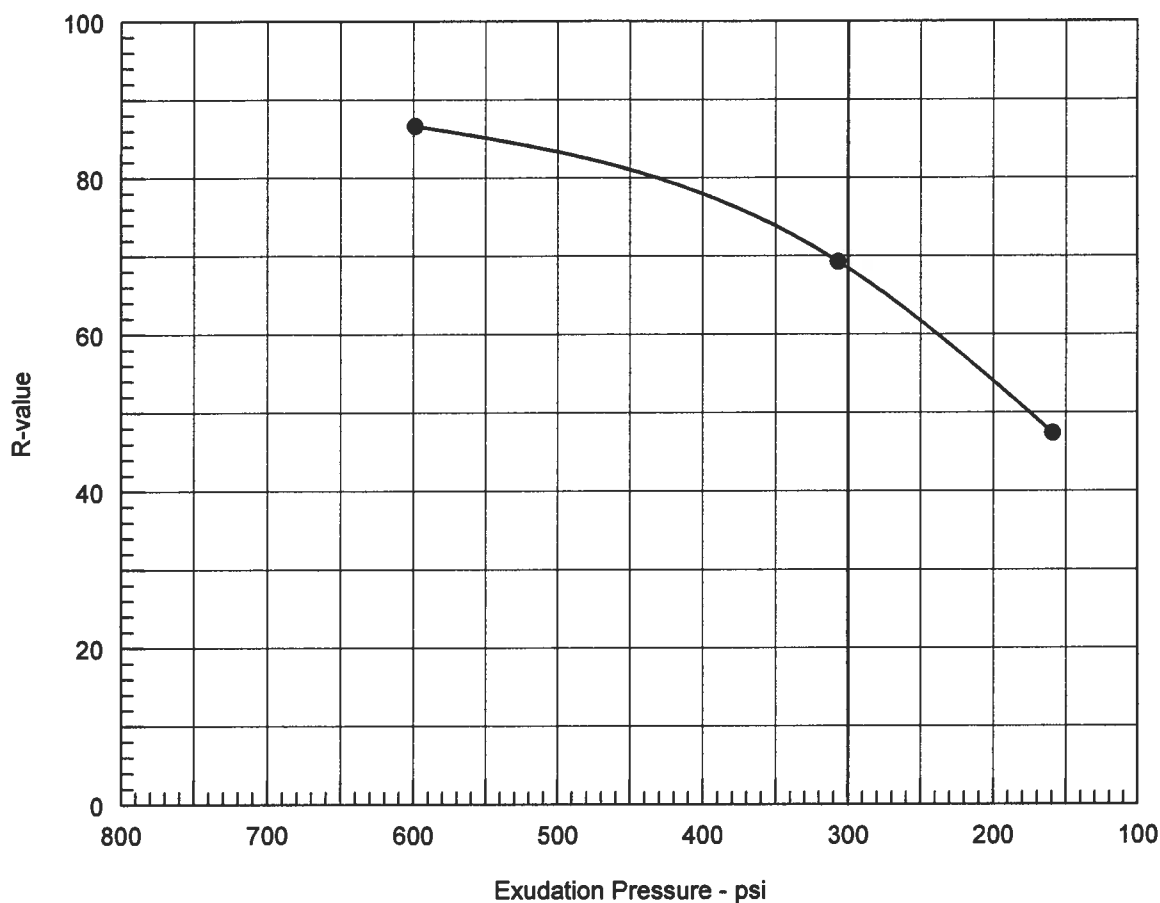
Sample Number: 14c

Project No.: 2103.x

Figure _____

Blackburn Consulting

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	140.8	6.4	0	13	2.44	599	87	87
2	350	141.0	6.9	0	32	2.43	307	71	69
3	350	140.1	7.4	0	58	2.34	159	52	47

Test Results

Material Description

R-value at 300 psi exudation pressure = 68

CLAYEY SAND with GRAVEL, dark brown

Project No.: 2103.1

Project: N. State Street Bridge at Ackerman Creek

Source of Sample: A-13-03

Depth: 2.75-6.0'

Sample Number: Bag A

Date: 8/26/2013

Tested by: MDR

Checked by: RBL

Remarks:

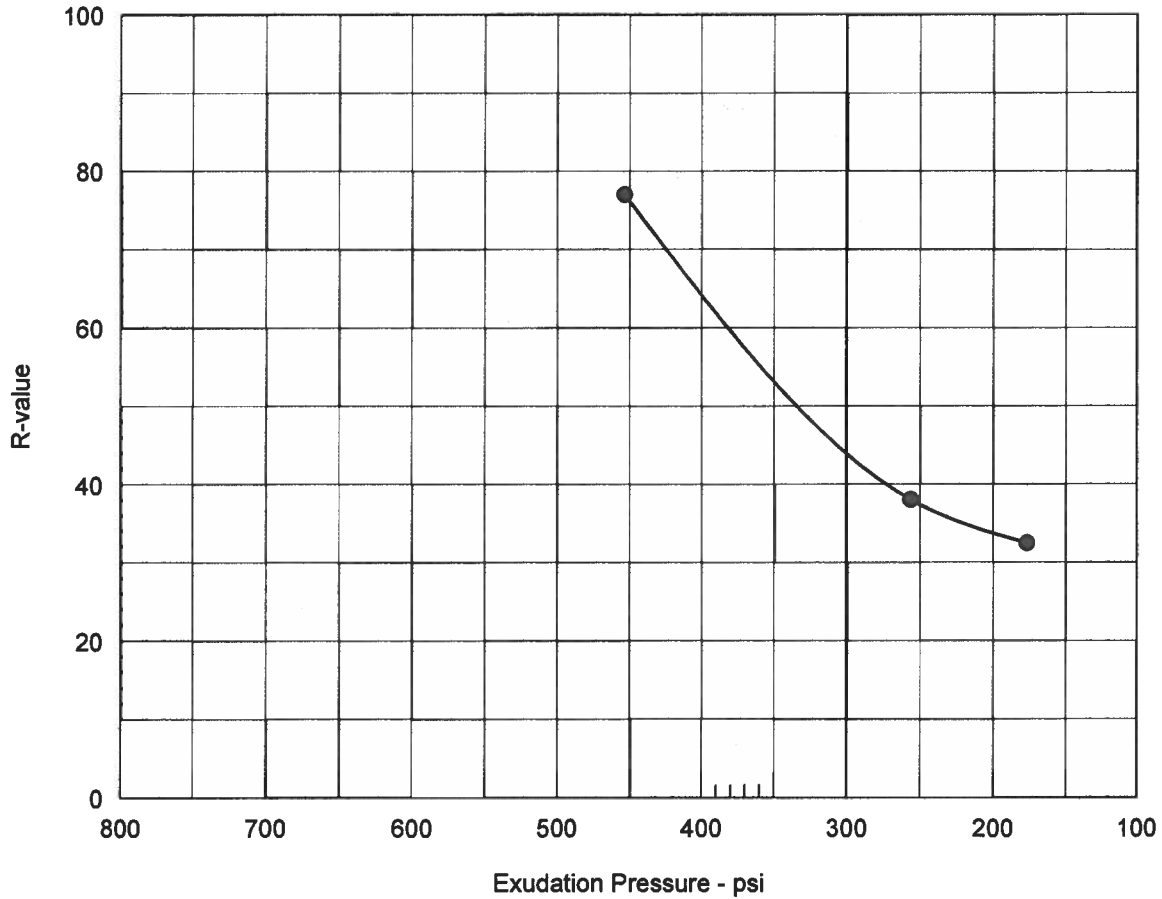
35.9% retained on No. 4 sieve, sample batched

R-VALUE TEST REPORT

Blackburn Consulting

Figure _____

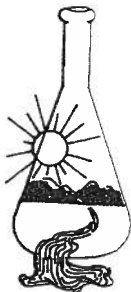
R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact Pressure psi	Density pcf	Moist. %	Expansion Pressure psf	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Value Corr.
1	350	133.1	8.4	70	27	2.49	454	77	77
2	350	131.4	8.9	9	76	2.53	256	38	38
3	350	130.8	9.5	0	87	2.48	177	32	32

Test Results	Material Description
R-value at 300 psi exudation pressure = 44	CLAYEY SAND with GRAVEL, dark brown
Project No.: 2103.1 Project: N. State Street Bridge at Ackerman Creek Source of Sample: A-13-04 Sample Number: Bag B Date: 8/26/2013	Tested by: MDR Checked by: RBL Remarks: 29.2% retained on #4 sieve, sample batched
R-VALUE TEST REPORT Blackburn Consulting	Figure _____



Sunland Analytical

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

Date Reported 08/23/2013

Date Submitted 08/19/2013

To: Bryce Moore
Blackburn Consulting
2491 Boatman Ave
W. Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 2013.1 ACKERMAN CRK. Site ID : R13-01#76 @ 34.
Thank you for your business.

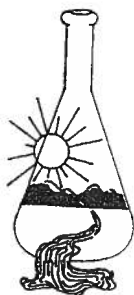
* For future reference to this analysis please use SUN # 65329-135200.

EVALUATION FOR SOIL CORROSION

Soil pH	8.20		
Minimum Resistivity	1.42	ohm-cm (x1000)	
Chloride	12.3 ppm	00.00123	%
Sulfate	15.0 ppm	00.00150	%

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 08/23/2013
Date Submitted 08/19/2013

To: Bryce Moore
Blackburn Consulting
2491 Boatman Ave
W. Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 2013.1 ACKERMAN CRK. Site ID : R13-02#8 @ 39.
Thank you for your business.

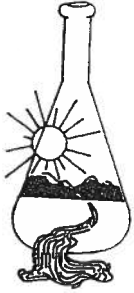
* For future reference to this analysis please use SUN # 65329-135201.

EVALUATION FOR SOIL CORROSION

Soil pH	8.20		
Minimum Resistivity	1.58 ohm-cm (x1000)		
Chloride	16.8 ppm	00.00168	%
Sulfate	5.9 ppm	00.00059	%

METHODS

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



Sunland Analytical

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

Date Reported 08/23/2013
Date Submitted 08/19/2013

To: Bryce Moore
Blackburn Consulting
2491 Boatman Ave
W. Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 2013.1 ACKERMAN CRK. Site ID : R13-02#14 @ 69.
Thank you for your business.

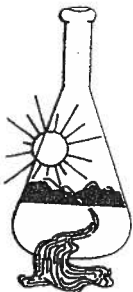
* For future reference to this analysis please use SUN # 65329-135198.

EVALUATION FOR SOIL CORROSION

Soil pH	7.77		
Minimum Resistivity	4.02 ohm-cm (x1000)		
Chloride	10.4 ppm	00.00104	%
Sulfate	15.2 ppm	00.00152	%

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 08/23/2013
Date Submitted 08/19/2013

To: Bryce Moore
Blackburn Consulting
2491 Boatman Ave
W. Sacramento, CA 95691

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

The reported analysis was requested for the following location:
Location : 2013.1 ACKERMAN CRK. Site ID : A13-04#1C @ 3.
Thank you for your business.

* For future reference to this analysis please use SUN # 65329-135199.

EVALUATION FOR SOIL CORROSION

Soil pH	7.10		
Minimum Resistivity	1.93	ohm-cm (x1000)	
Chloride	11.6 ppm	00.00116	%
Sulfate	11.4 ppm	00.00114	%

METHODS

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

FOUNDATION REPORT

N. State Street Bridge at Ackerman Creek

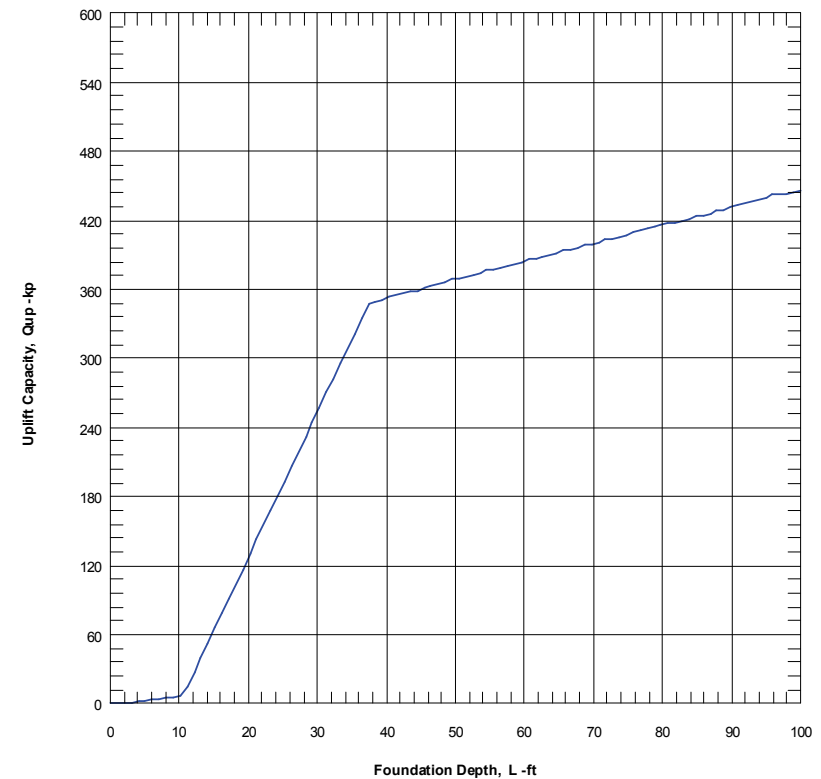
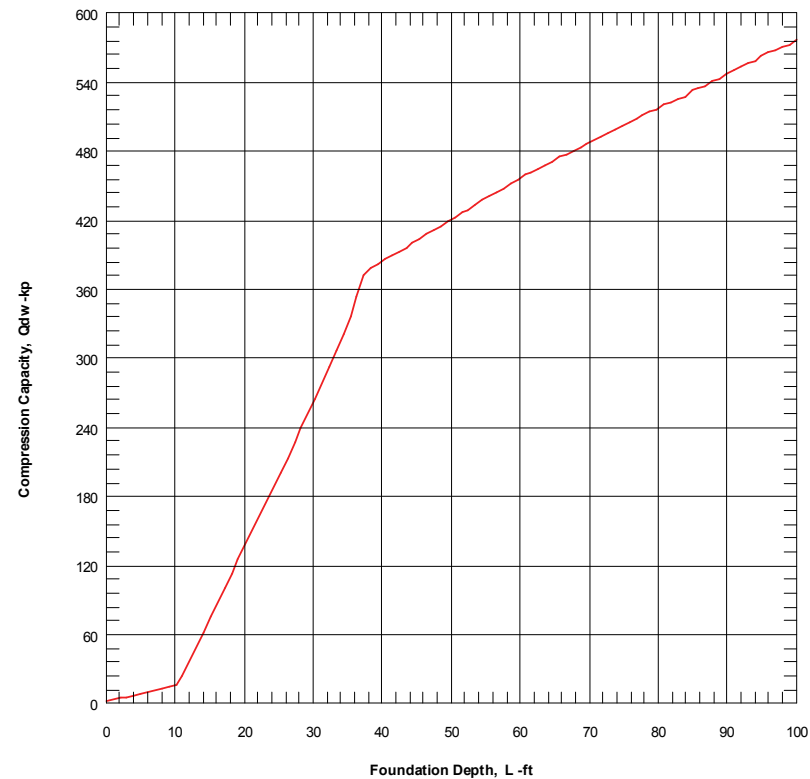
Bridge No. 10C-0065
Mendocino County, CA

November 2023

APPENDIX C

Calculations

ULTIMATE CAPACITY vs FOUNDATION DEPTH

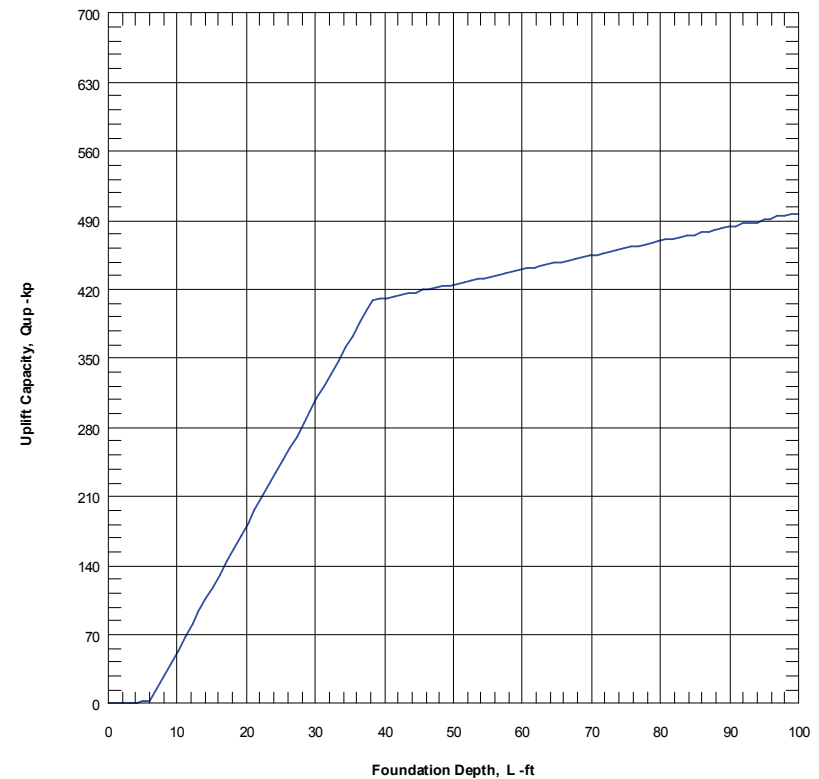
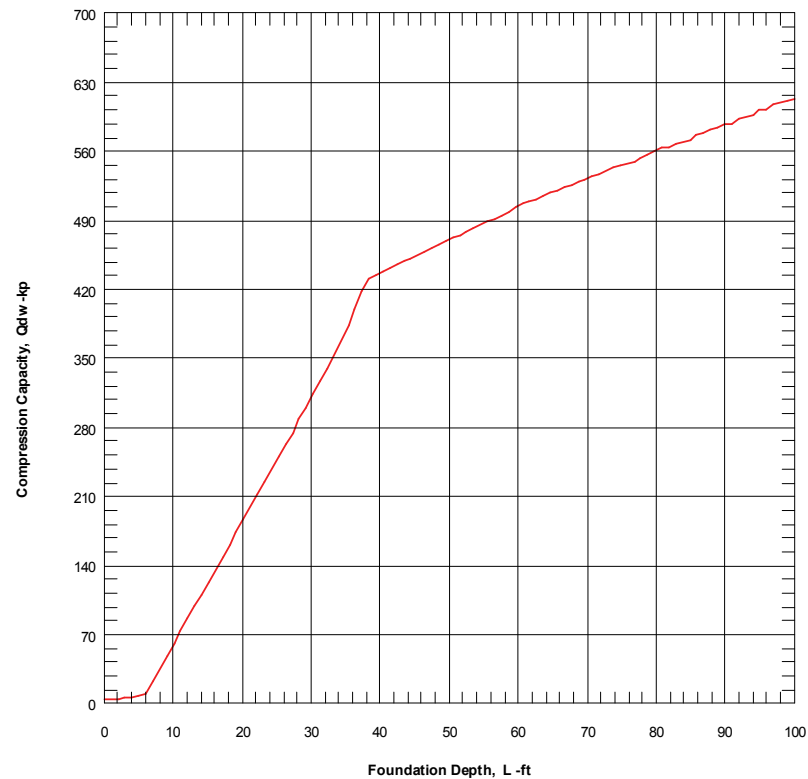


**CivilTech
Software**

Ackerman Creek Bridge Abut 1

Figure 1

ULTIMATE CAPACITY vs FOUNDATION DEPTH

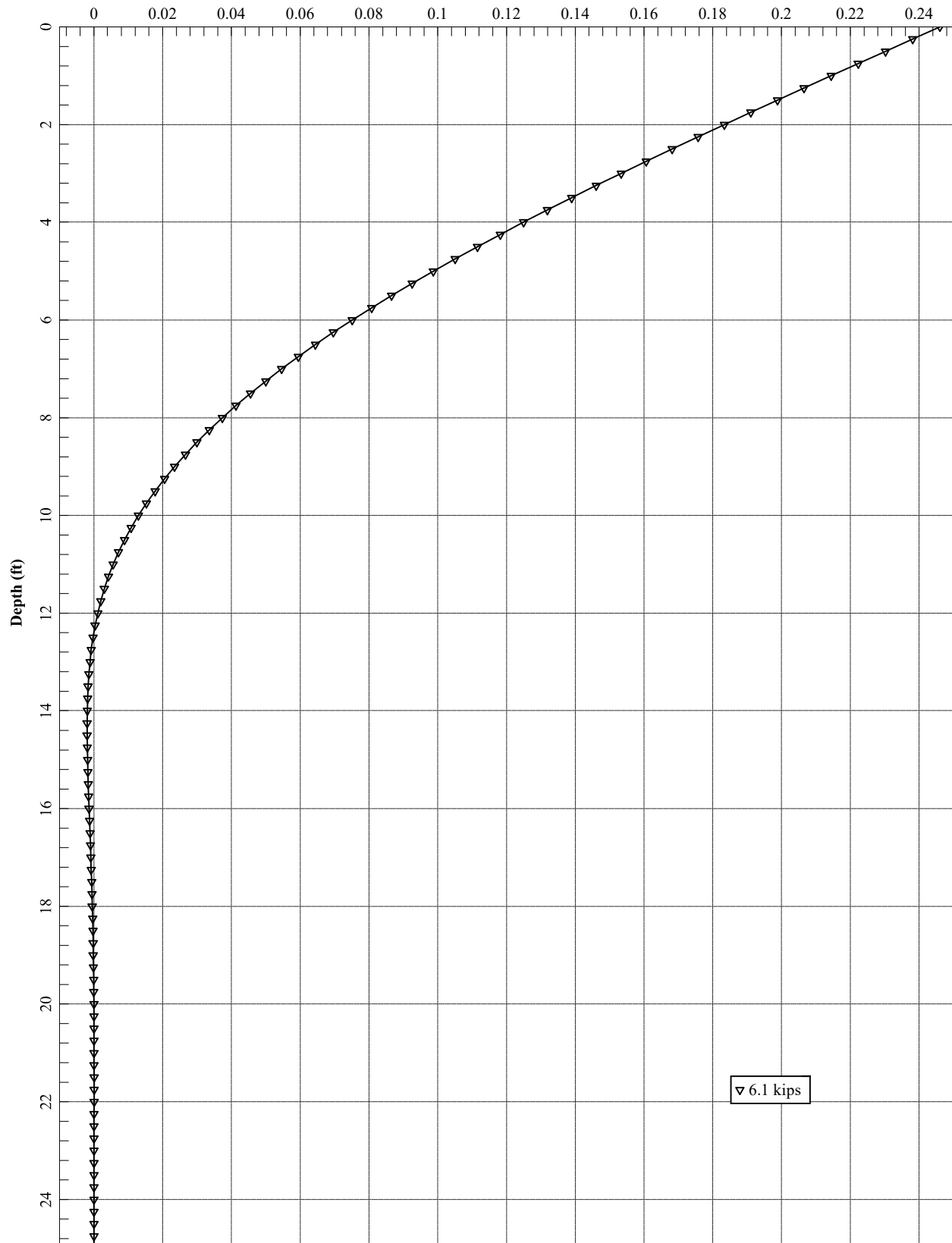


**CivilTech
Software**

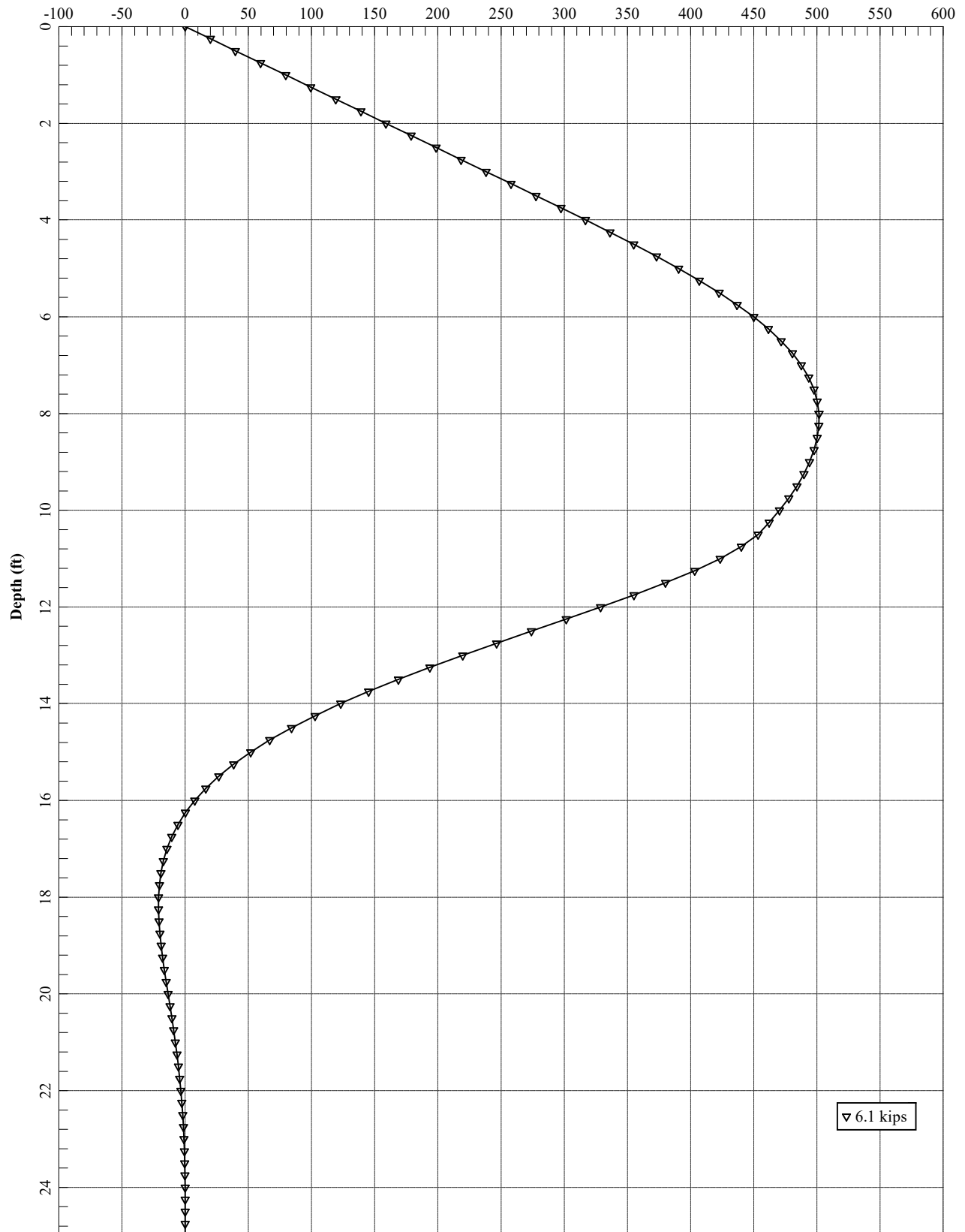
Ackerman Creek Bridge Abut 2

Figure 1

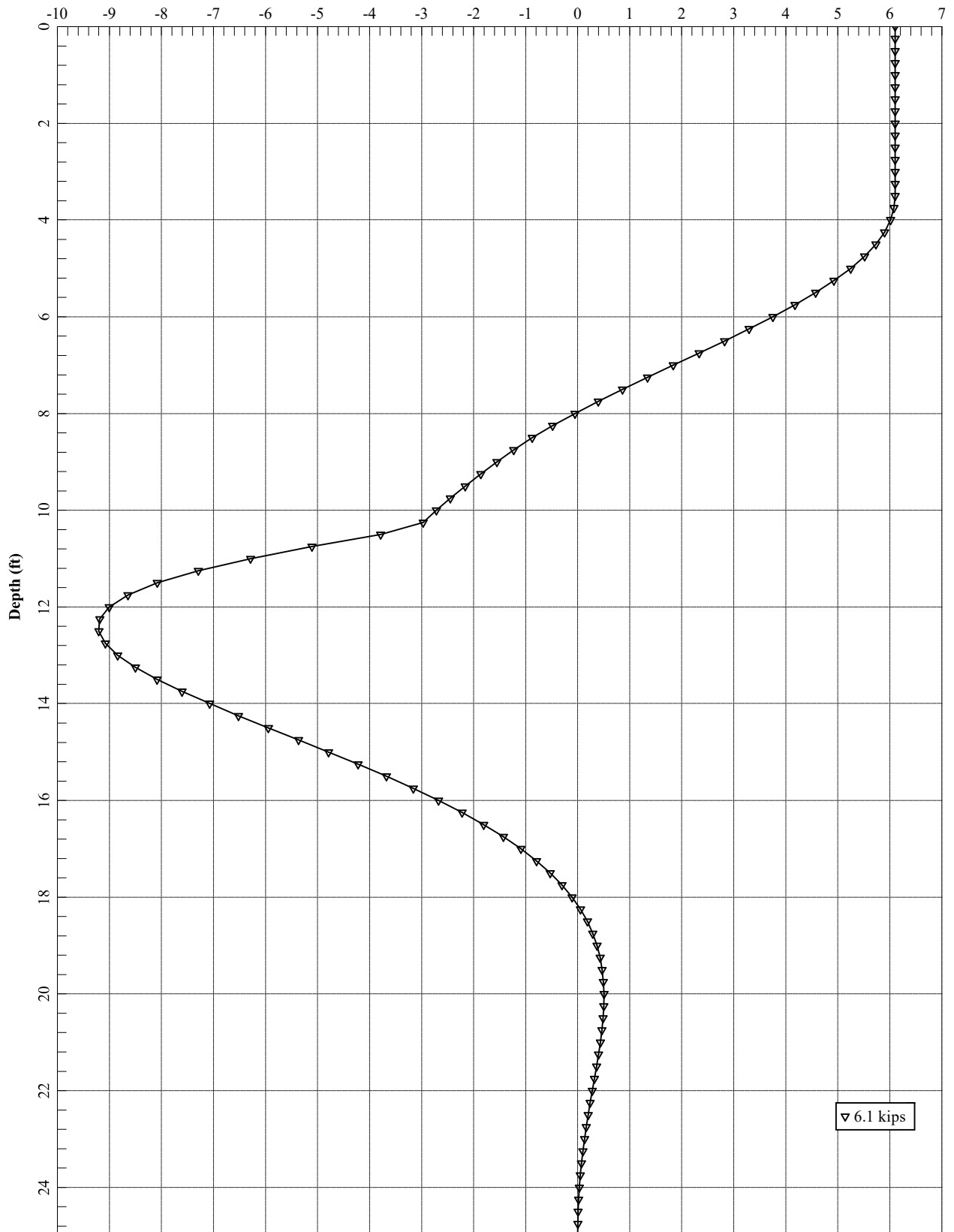
Ackerman Creek Bridge - Abutment 1 Longitudinal, Service (3.5' Scour)
Lateral Pile Deflection (inches)



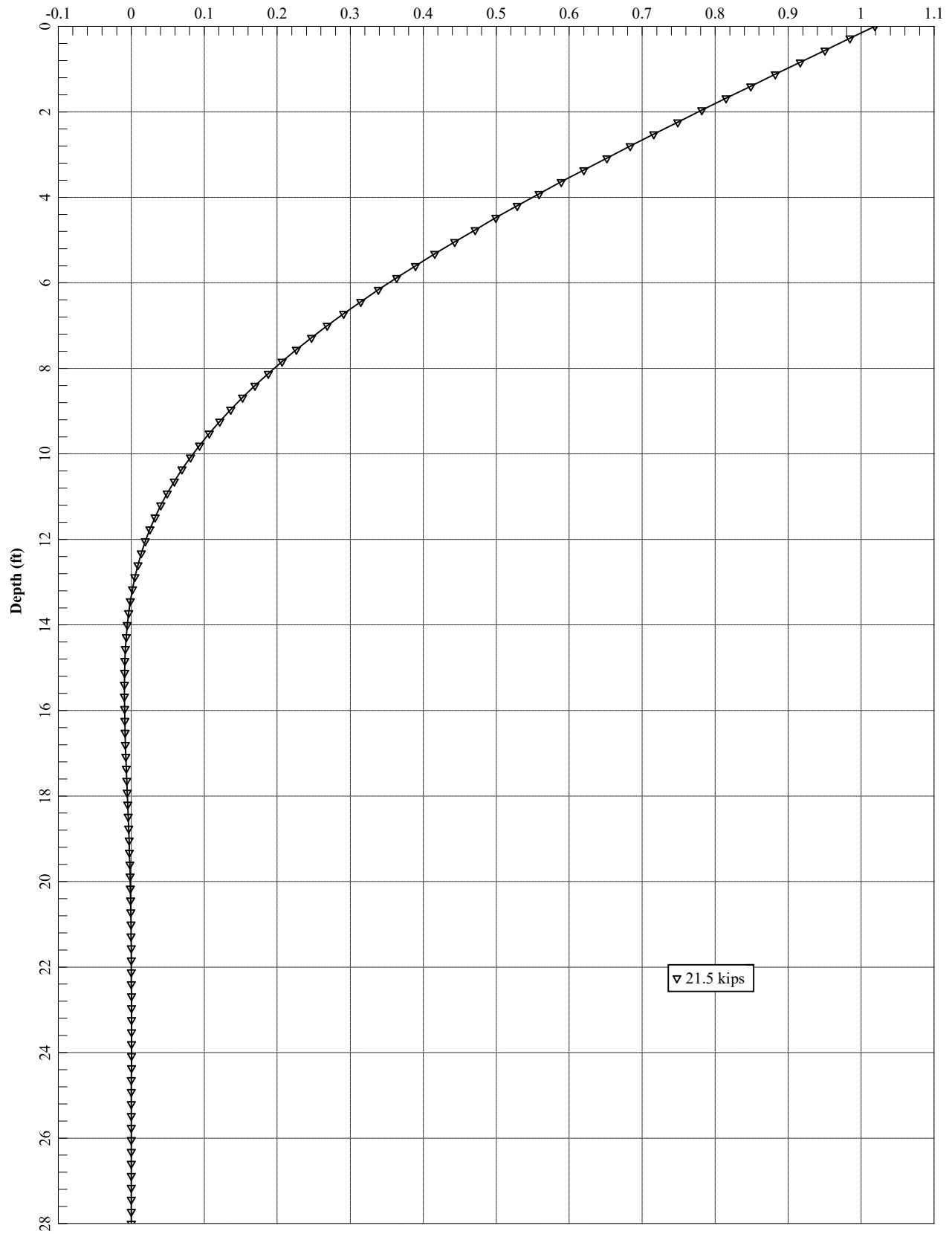
Ackerman Creek Bridge - Abutment 1 Longitudinal, Service (3.5' Scour)
Bending Moment (in-kips)



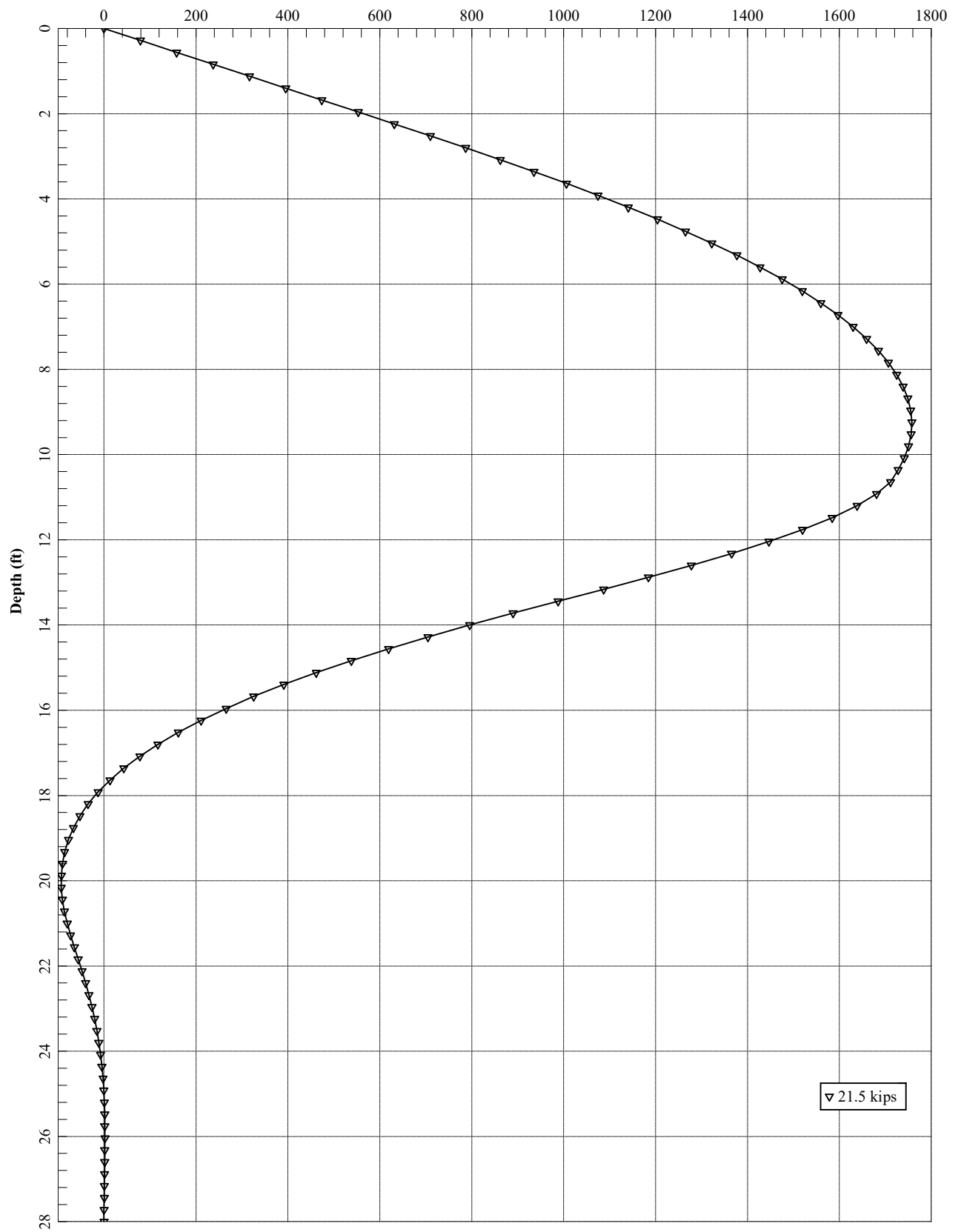
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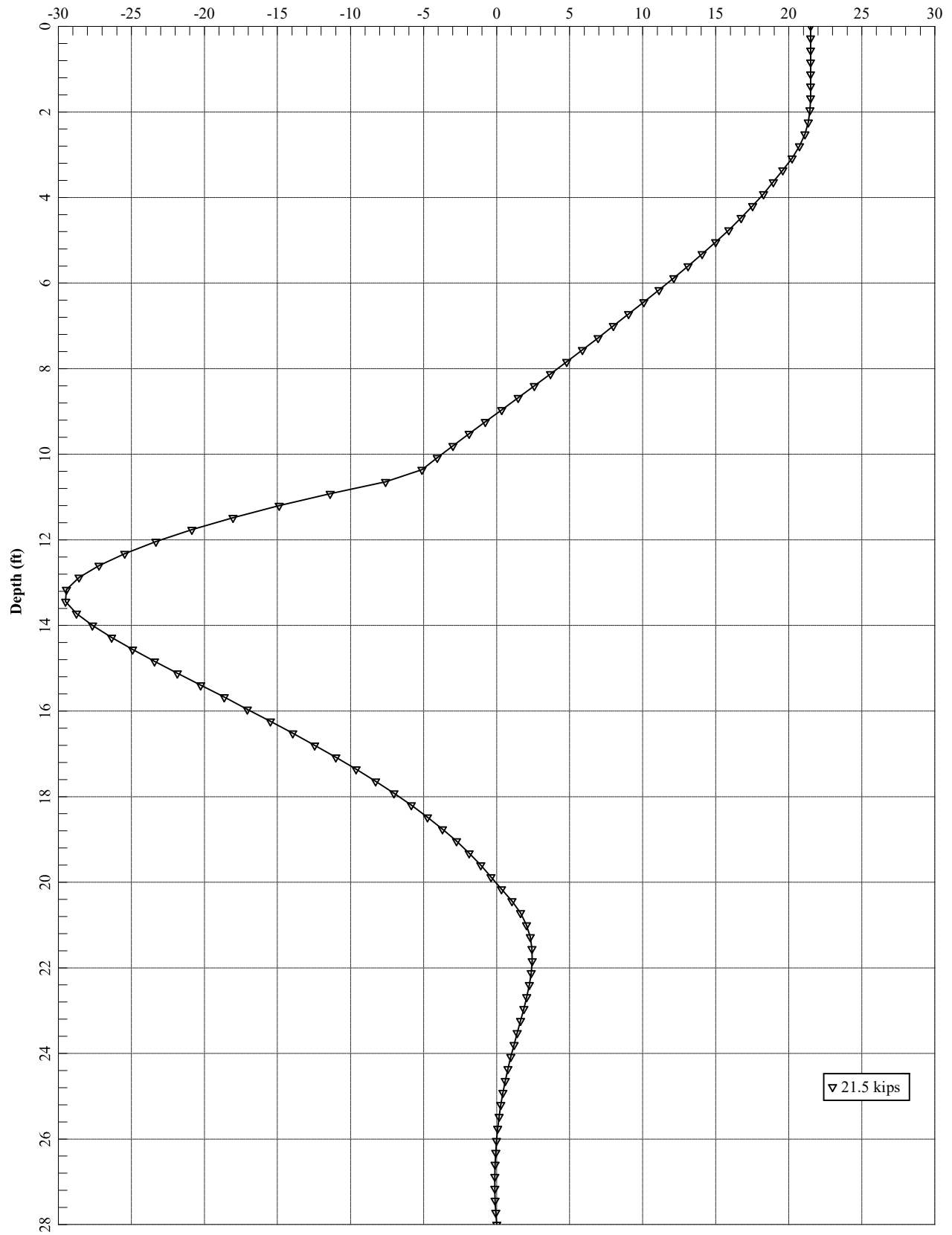
Ackerman Creek Bridge - Abutment 1 Longitudinal, Strength (1.75' Scour)
Lateral Pile Deflection (inches)



Ackerman Creek Bridge - Abutment 1 Longitudinal, Strength (1.75' Scour)
Bending Moment (in-kips)

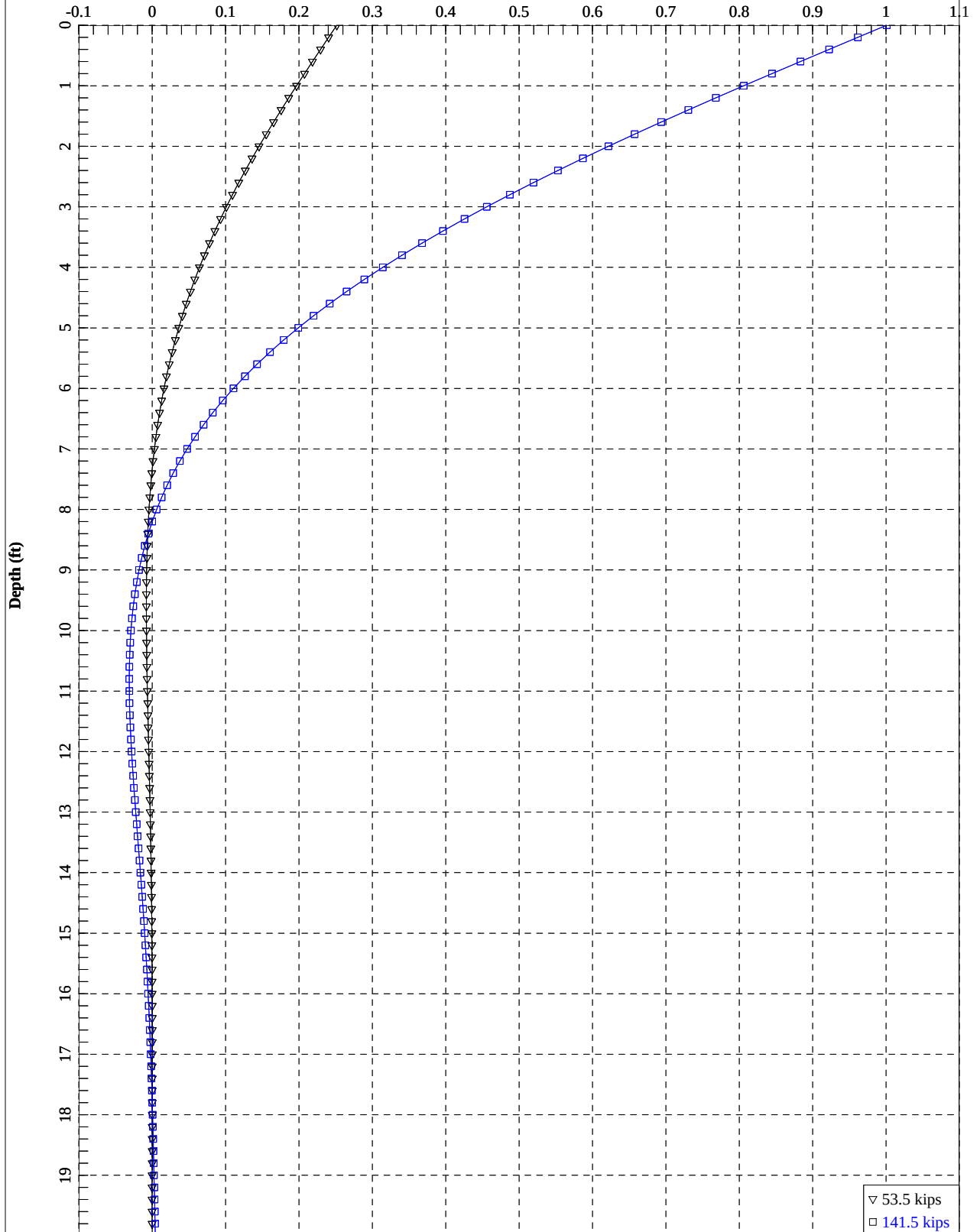


Ackerman Creek Bridge - Abutment 1 Longitudinal, Strength (1.75' Scour)
Shear Force (kips)



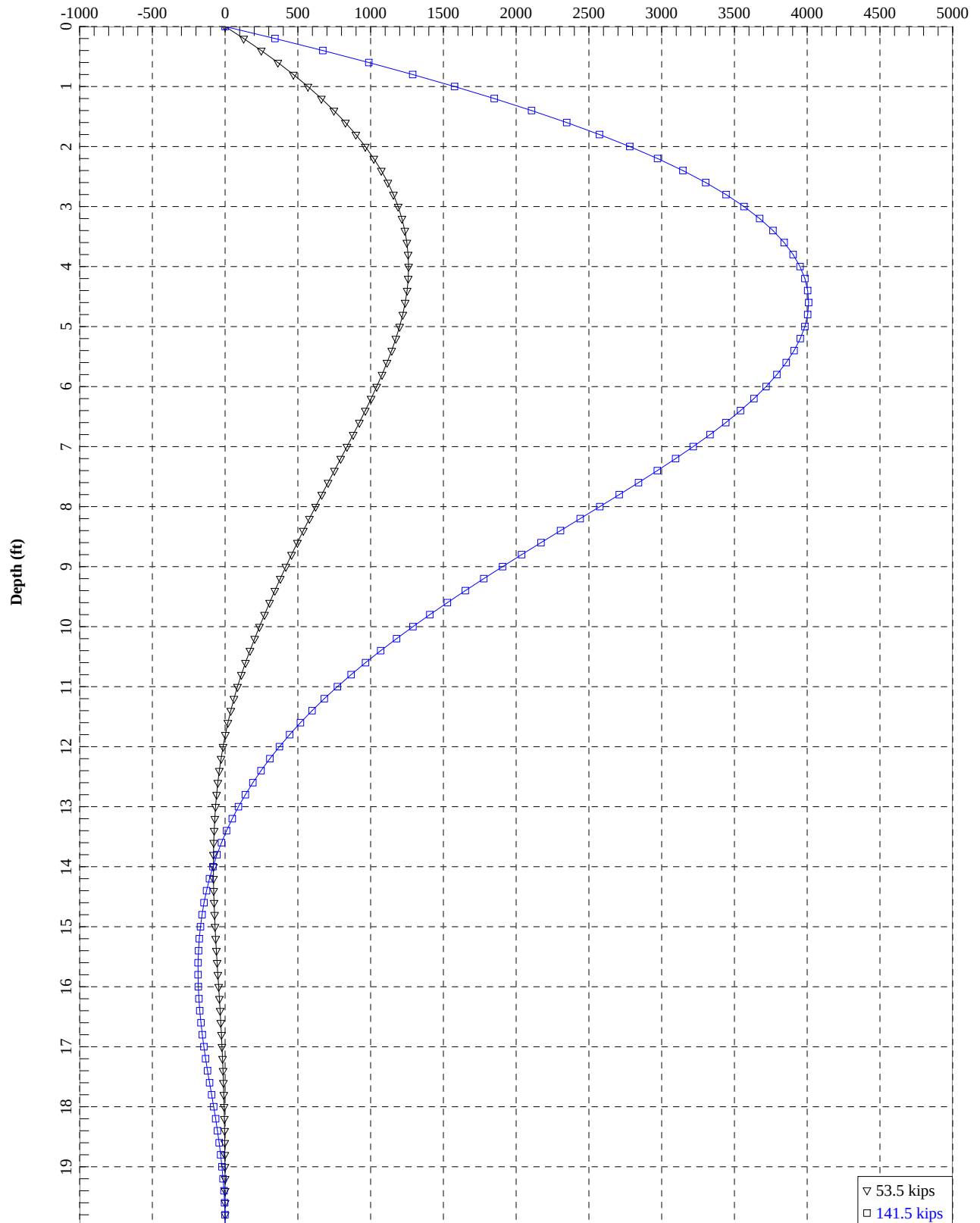
Ackerman Creek Bridge-Abut 1 Longitudinal (No Scour)

Lateral Deflection (in)



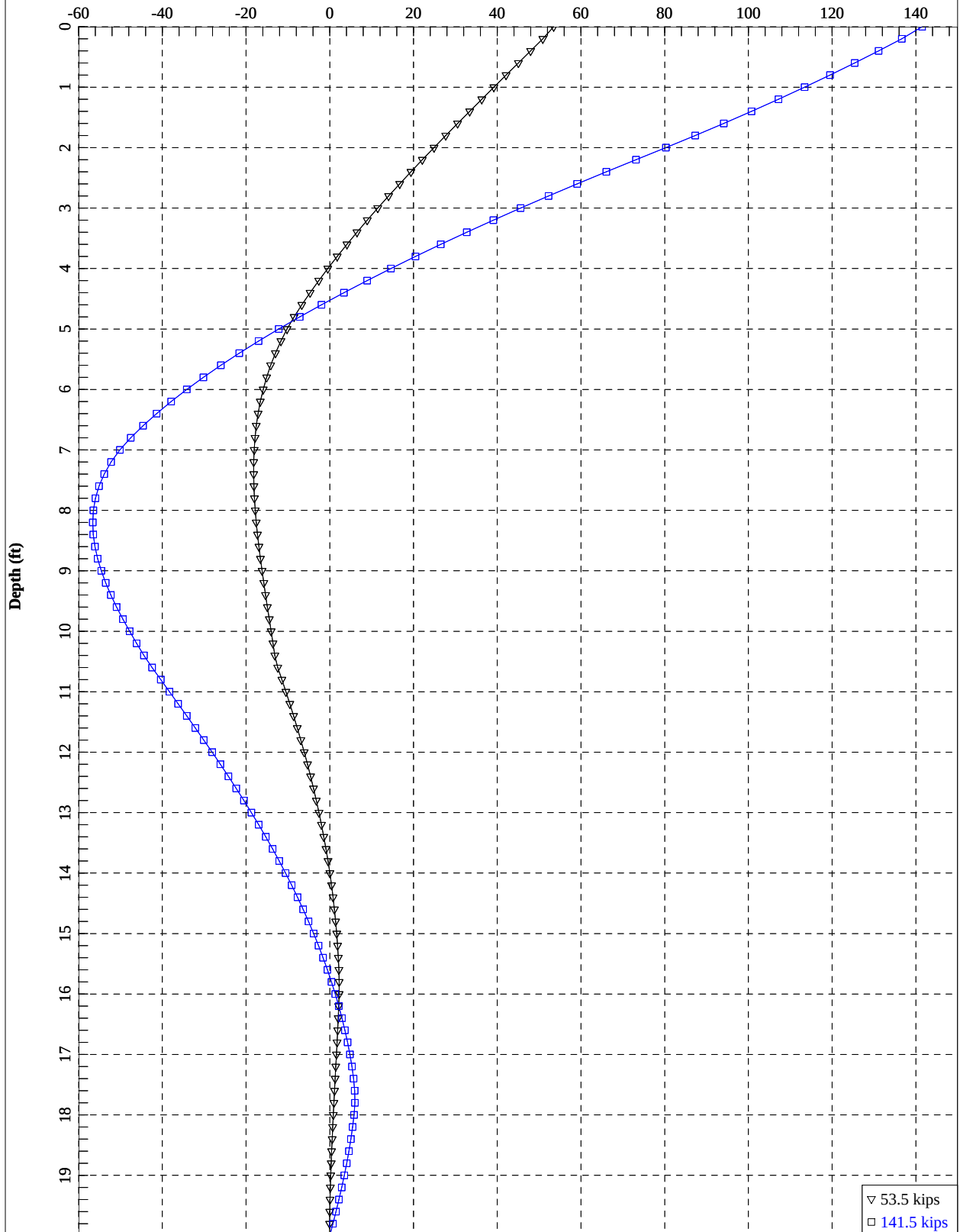
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Bending Moment (in-kips)

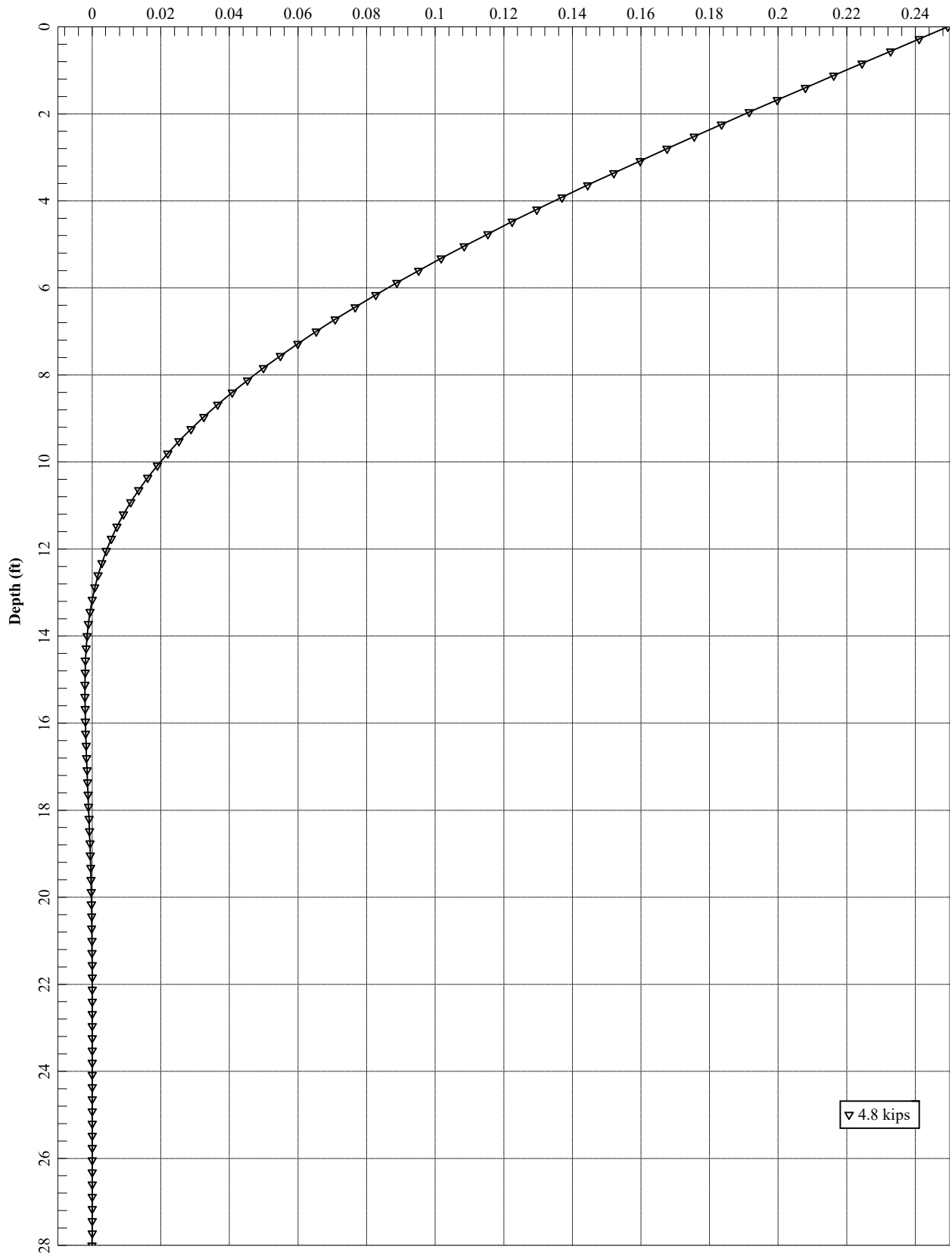


Ackerman Creek Bridge-Abut 1 Longitudinal (No Scour)

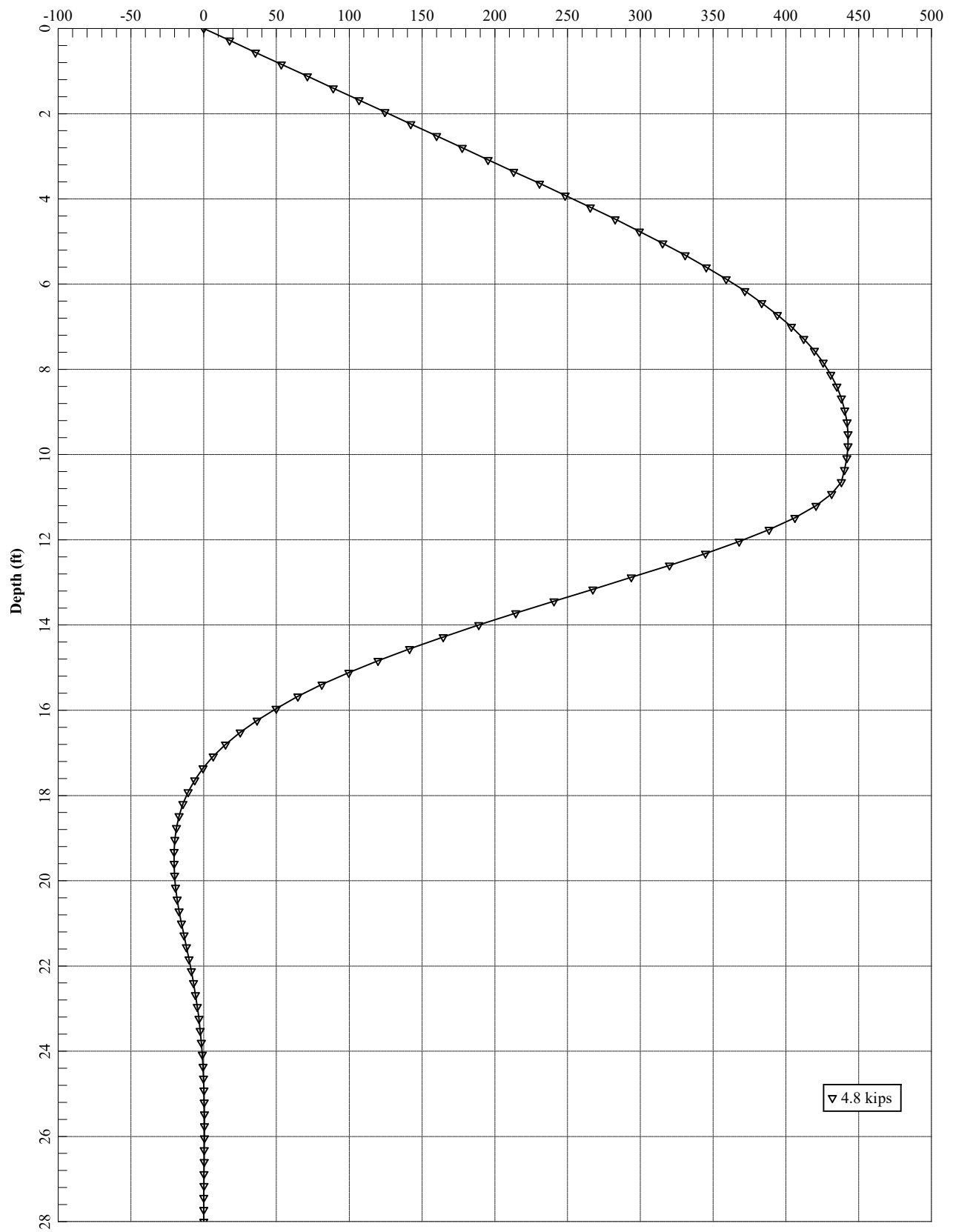
Shear Force (kips)



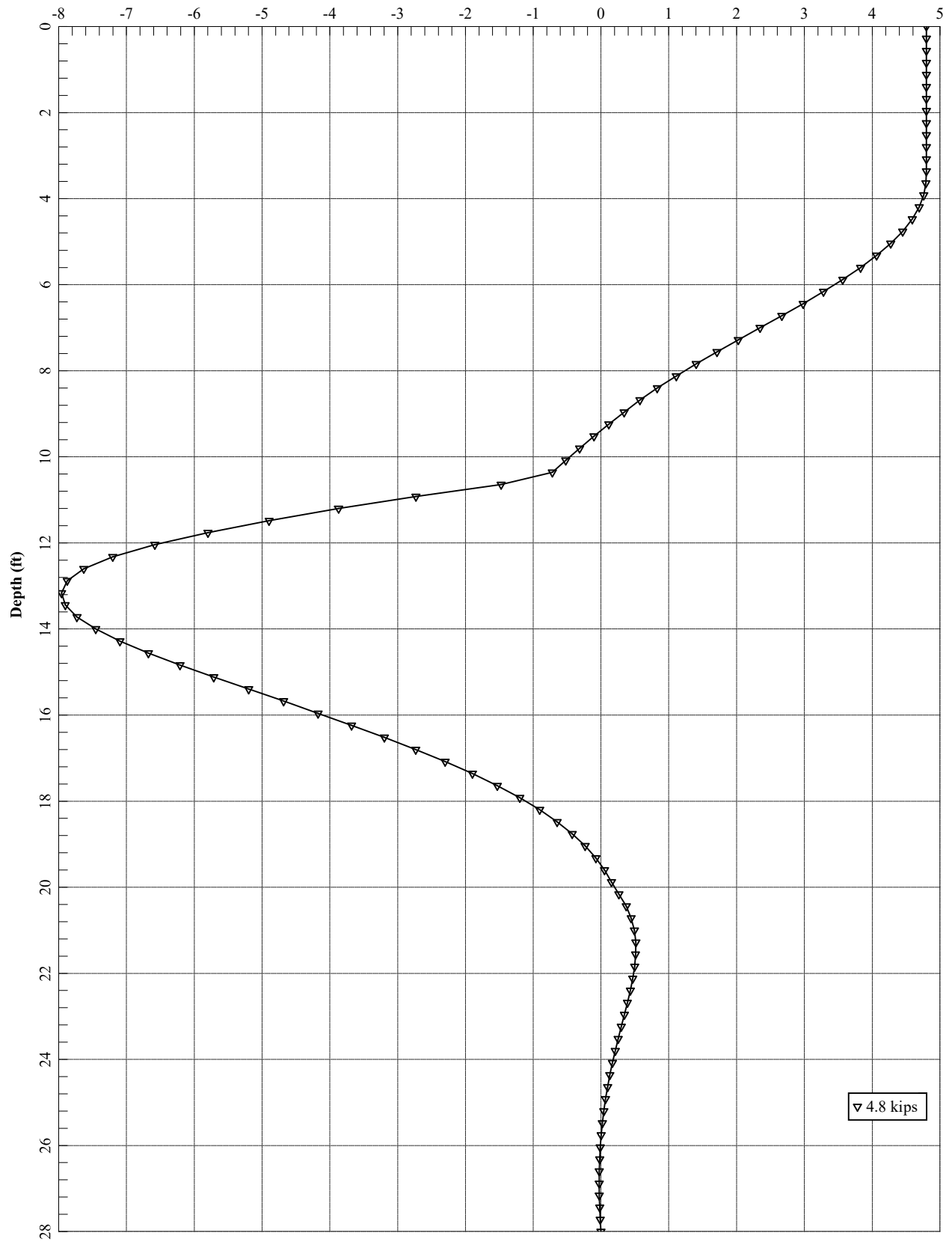
Ackerman Creek Bridge- Abutment 1 Transverse, Service (3.5' Scour)
Lateral Pile Deflection (inches)



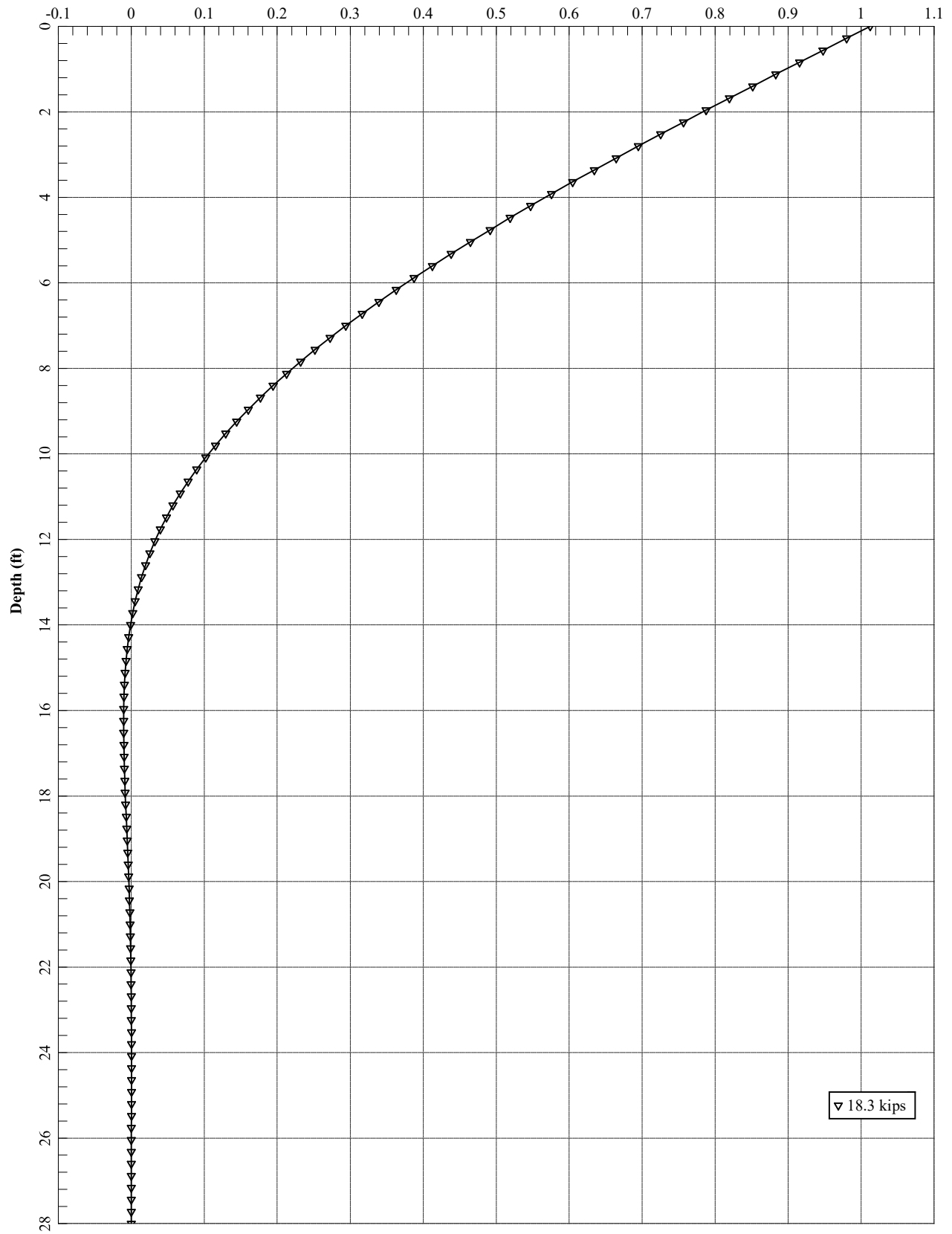
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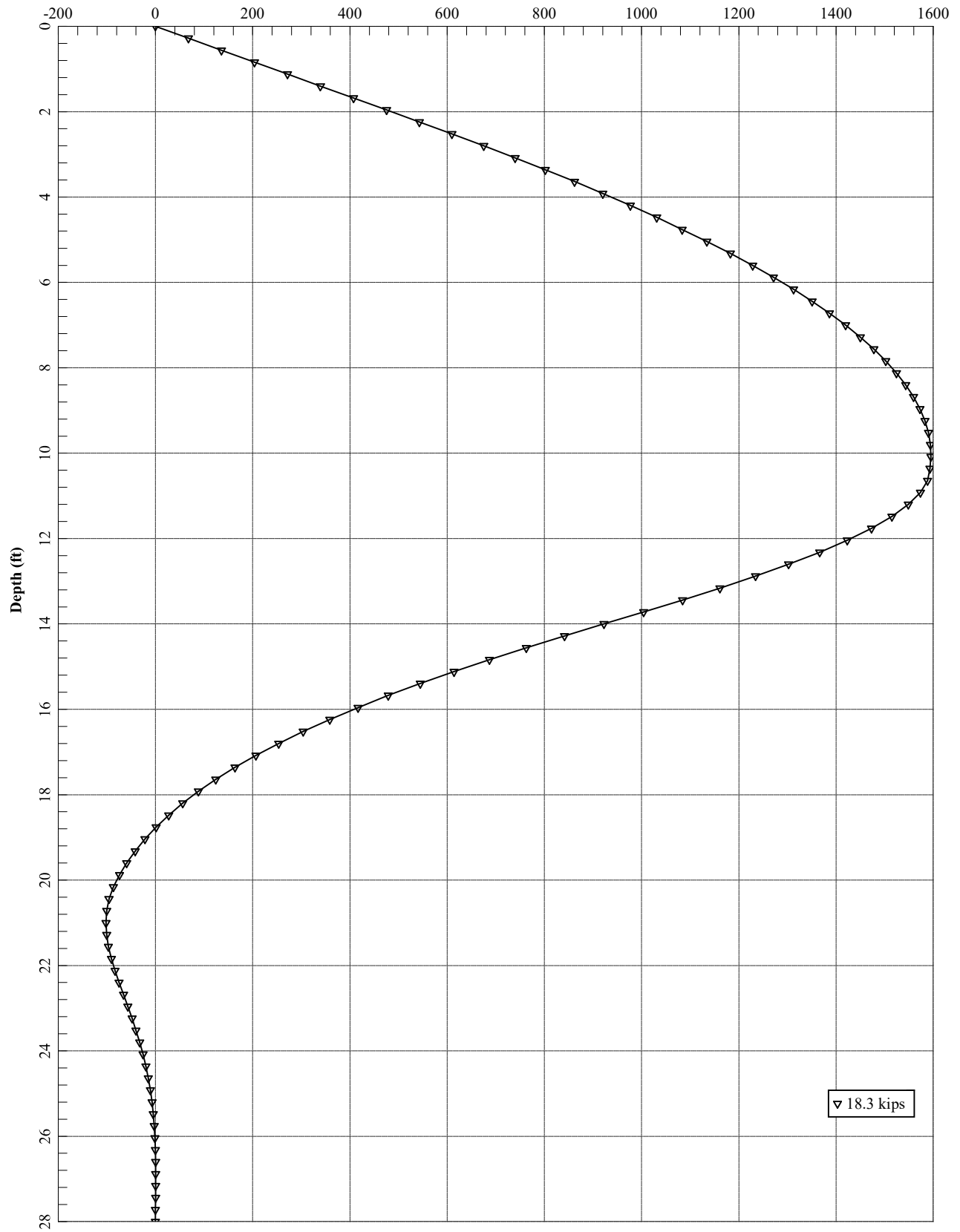
Ackerman Creek Bridge- Abutment 1 Transverse, Service (3.5' Scour)
Shear Force (kips)



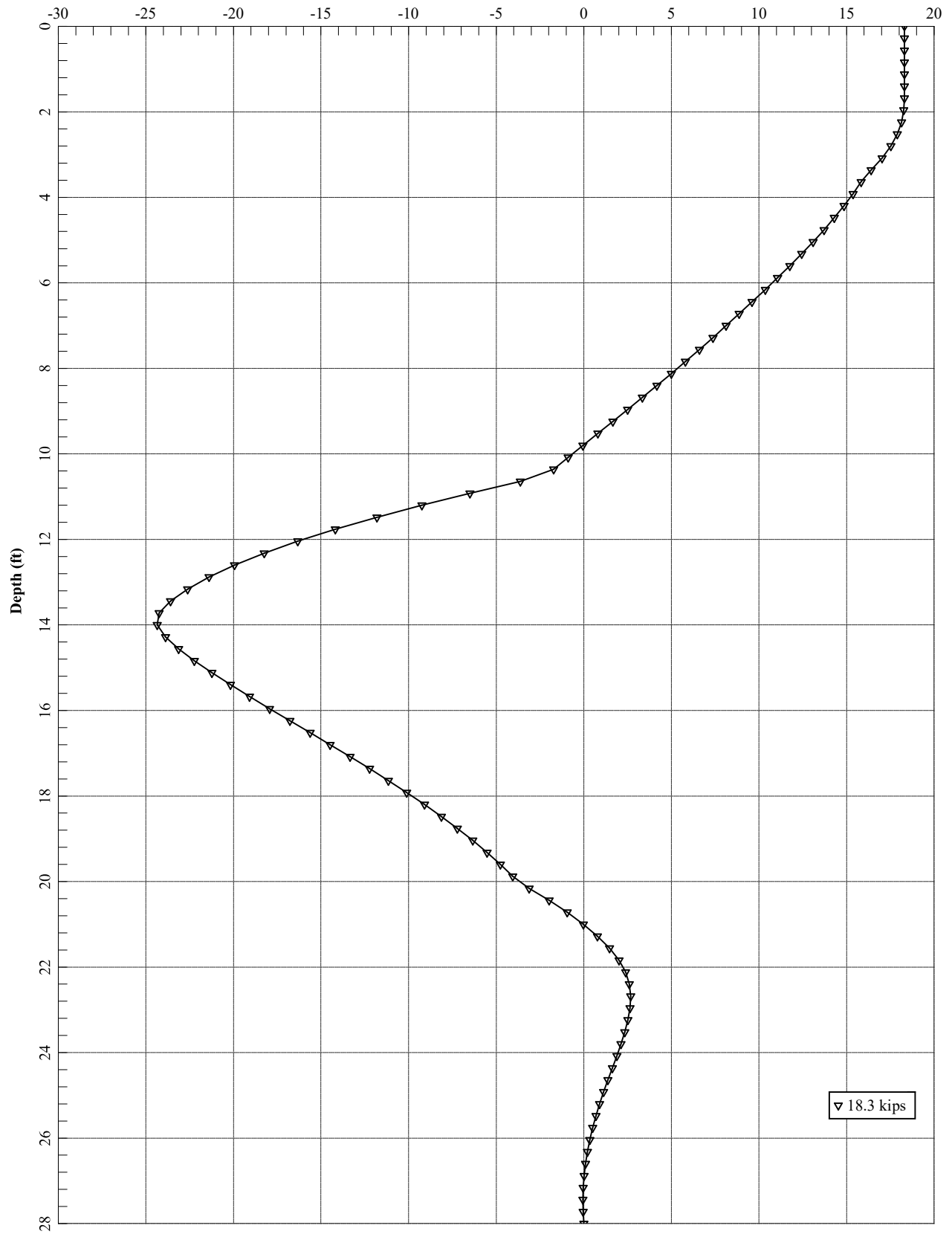
Ackerman Creek Bridge - Abutment 1 Transverse, Strength (1.75' Scour)
Lateral Pile Deflection (inches)



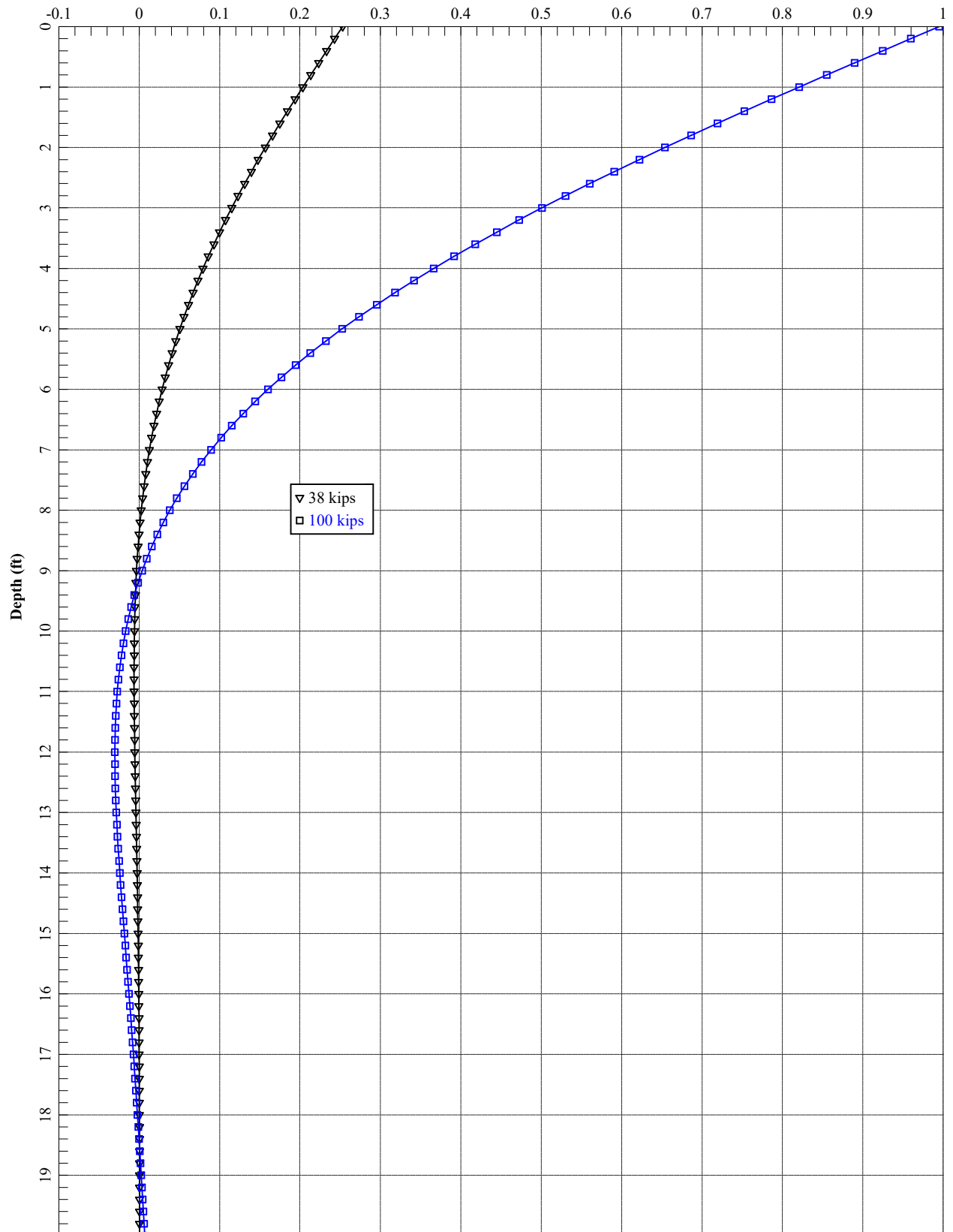
Ackerman Creek Bridge - Abutment 1 Transverse, Strength (1.75' Scour)
Bending Moment (in-kips)



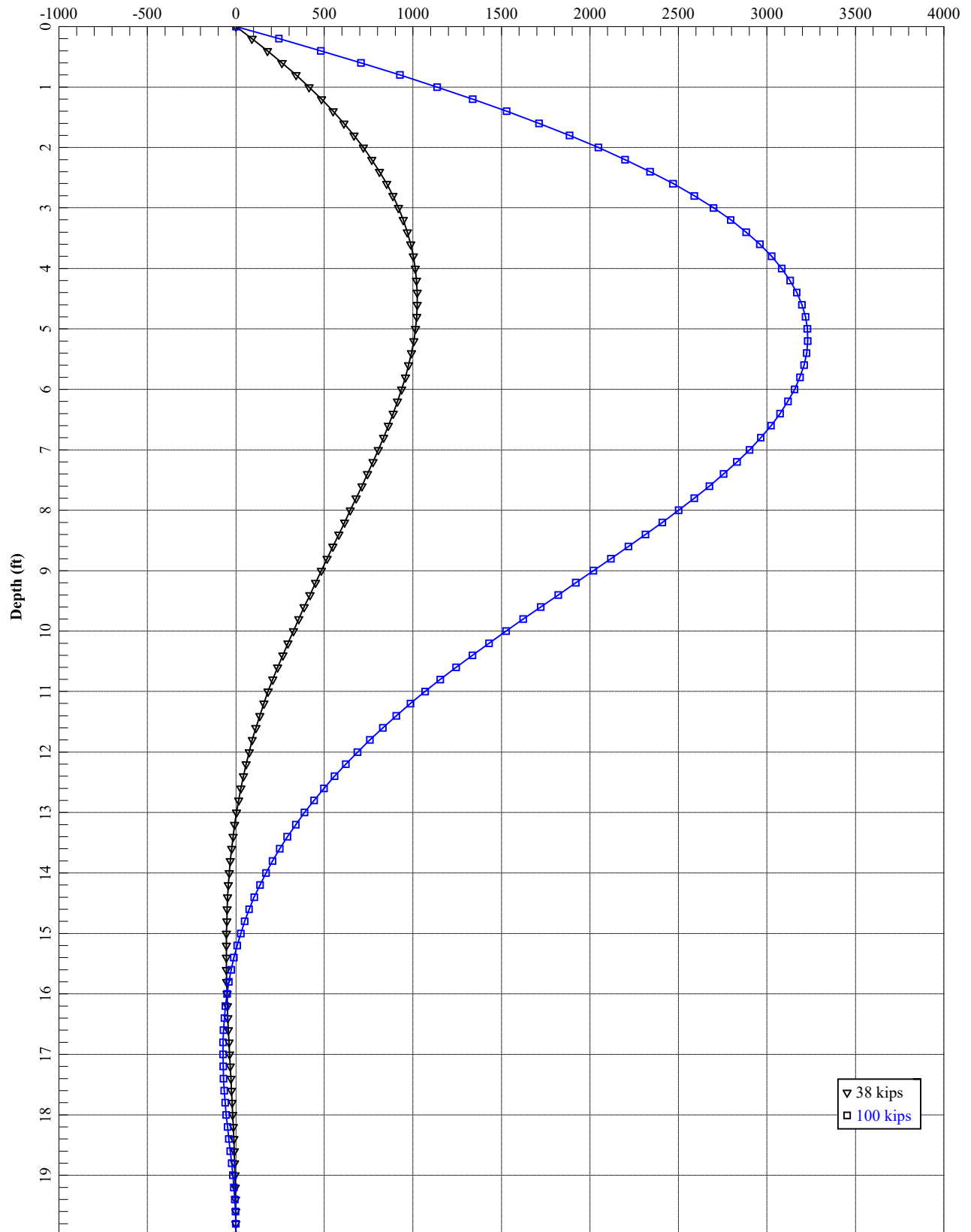
Ackerman Creek Bridge - Abutment 1 Transverse, Strength (1.75' Scour)
Shear Force (kips)



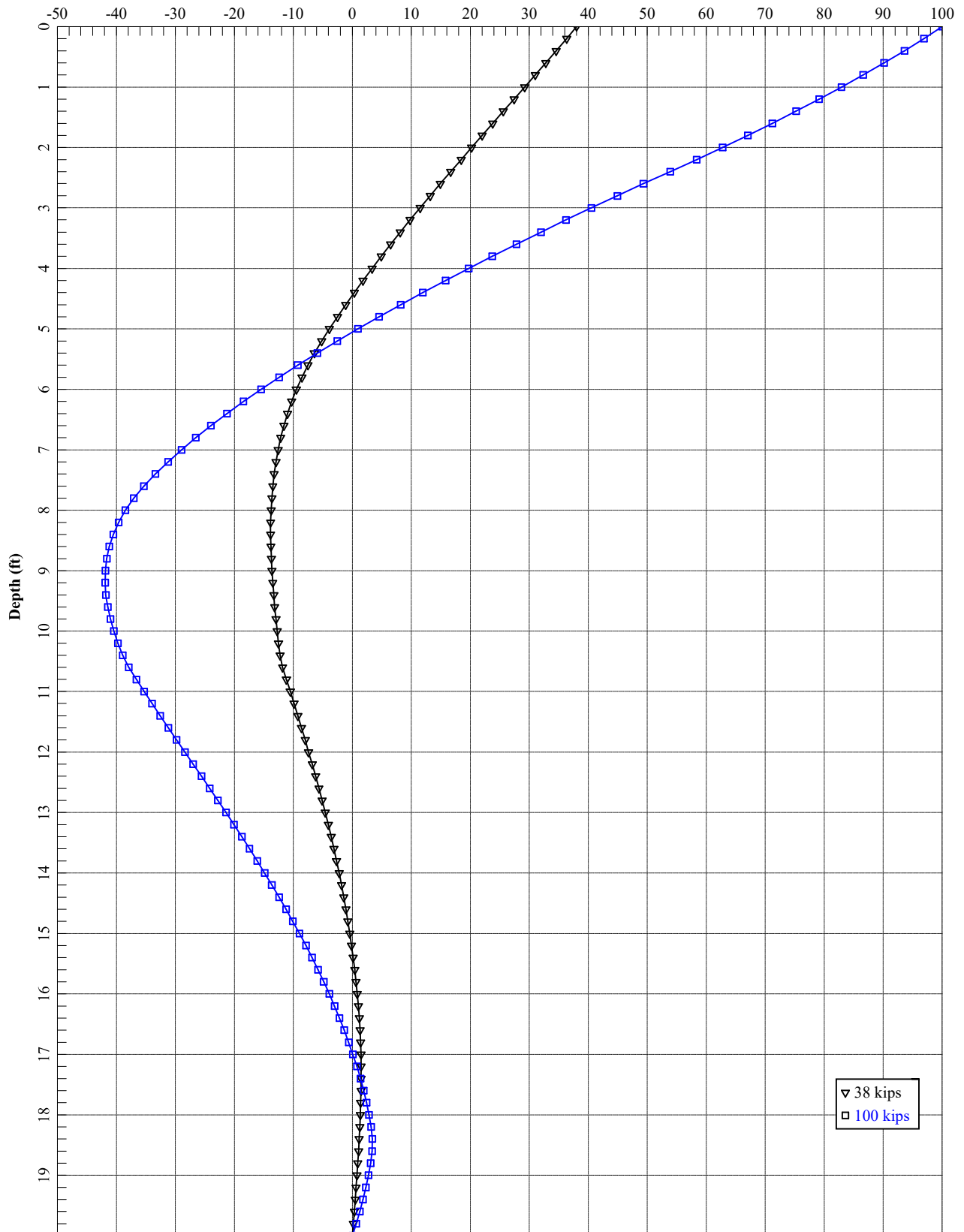
Ackerman Creek Bridge - Abutment 1 Transverse (No Scour)
Lateral Pile Deflection (inches)



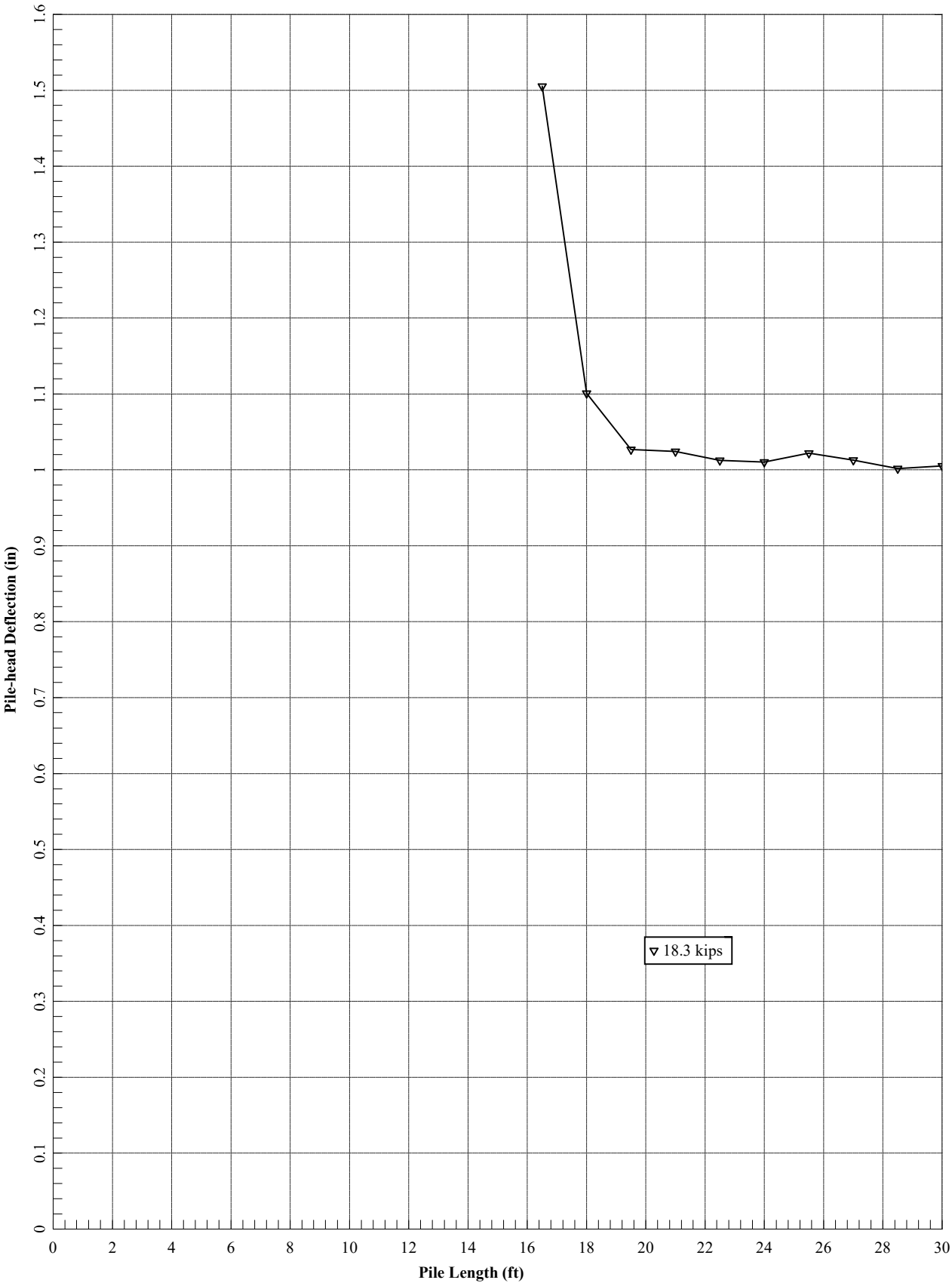
Ackerman Creek Bridge - Abutment 1 Transverse No Scour
Bending Moment (in-kips)



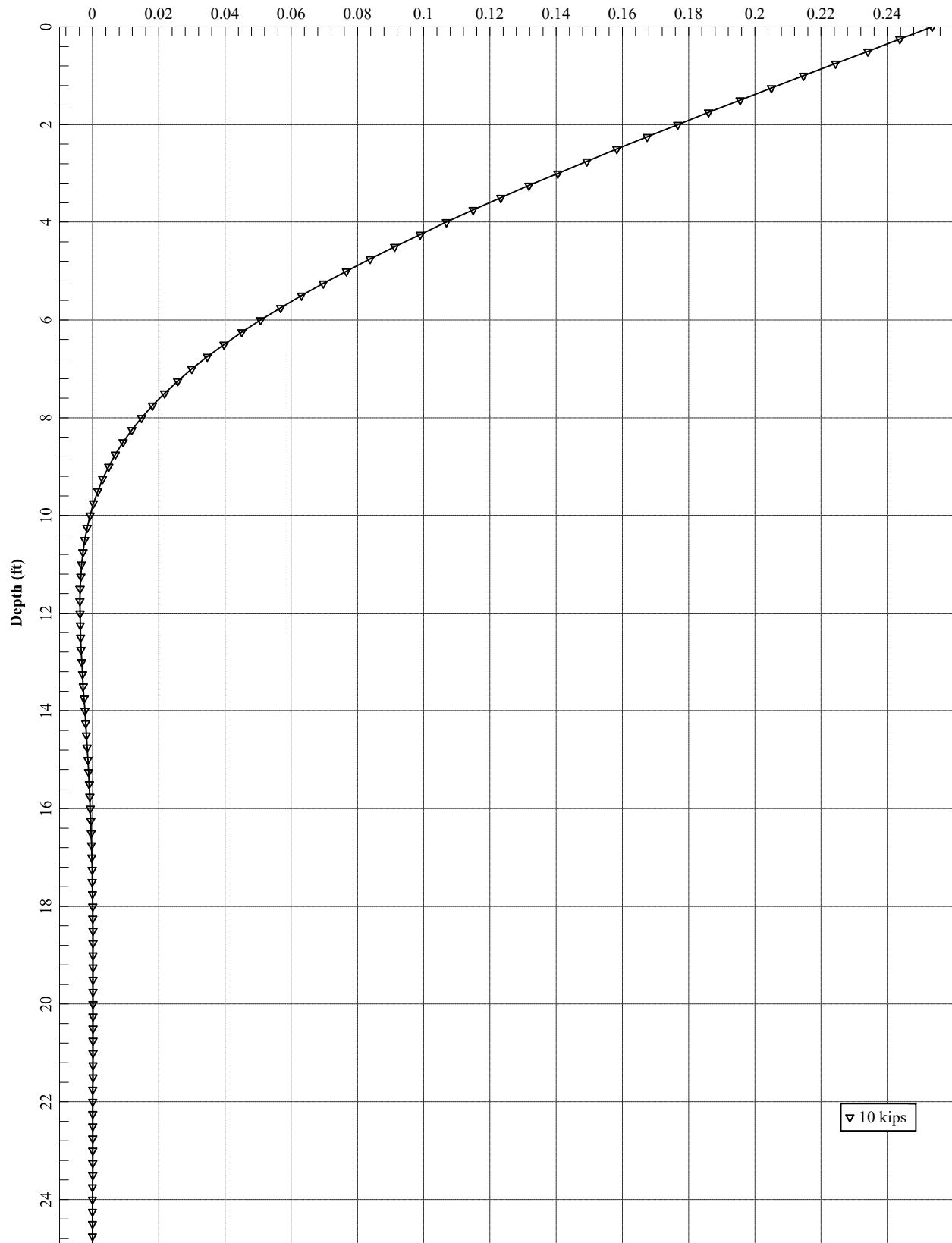
Ackerman Creek Bridge - Abutment 1 Transverse No Scour
Shear Force (kips)



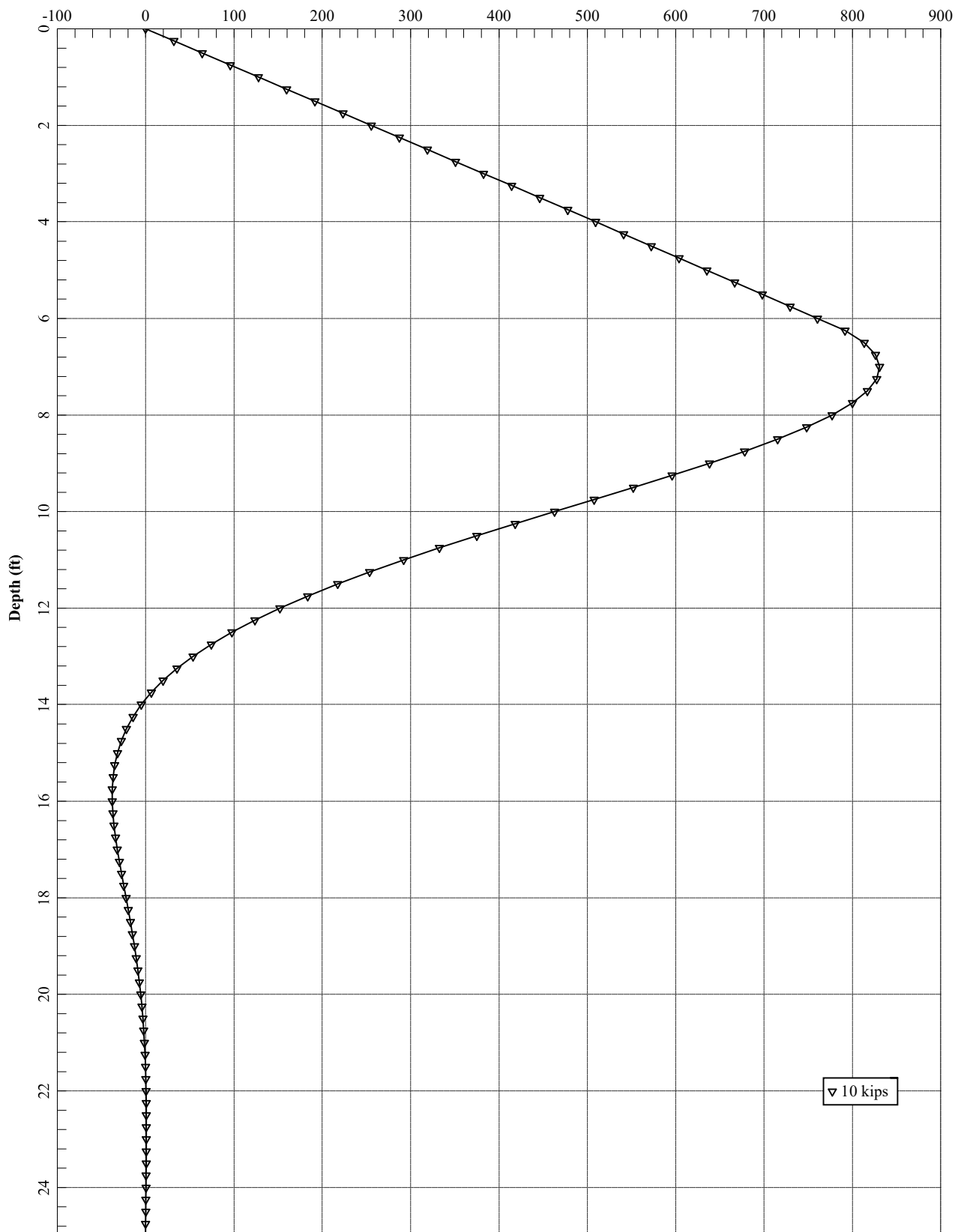
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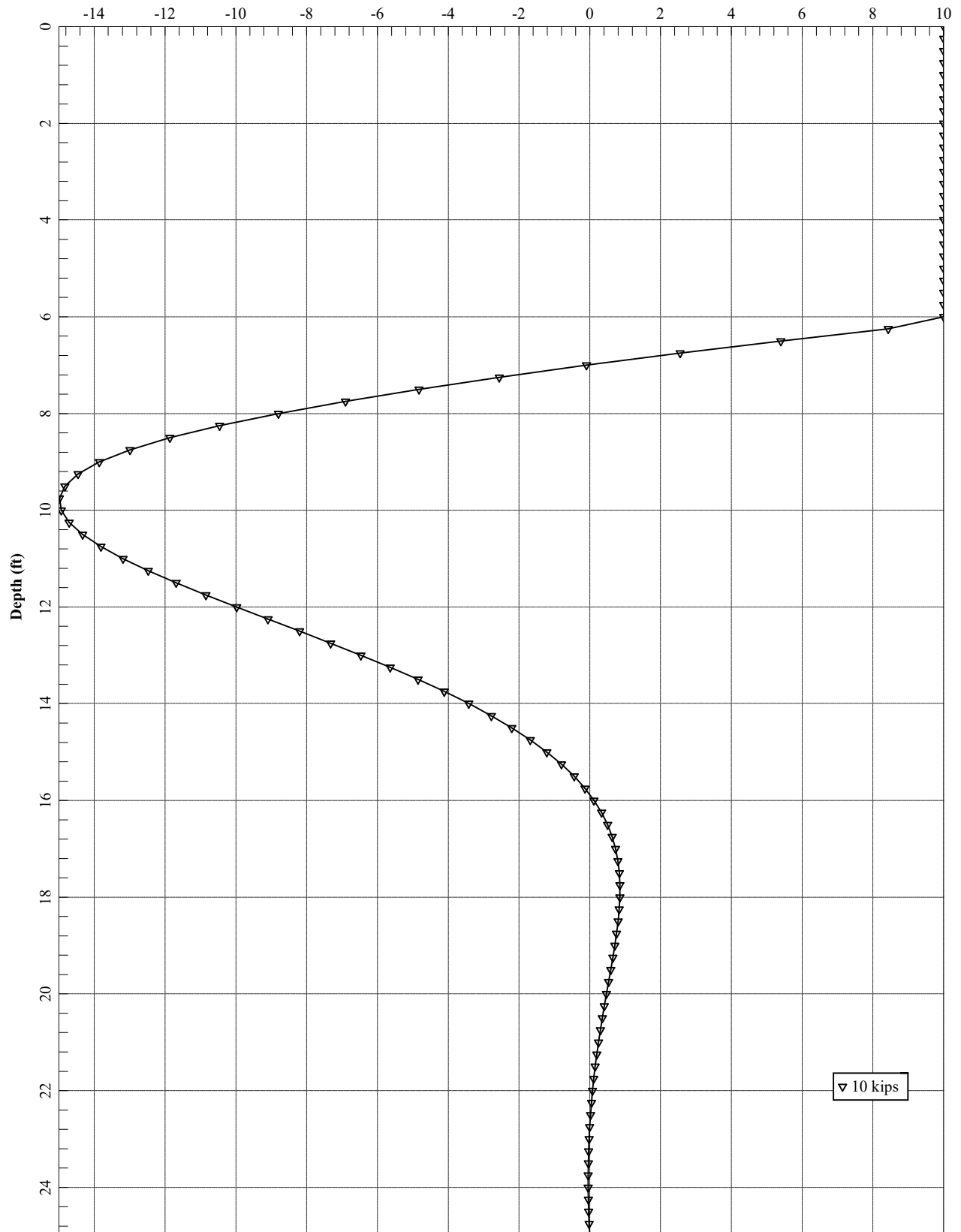
Ackerman Creek Bridge - Abutment 2 Longitudinal, Service (3.5' Scour)
Lateral Pile Deflection (inches)



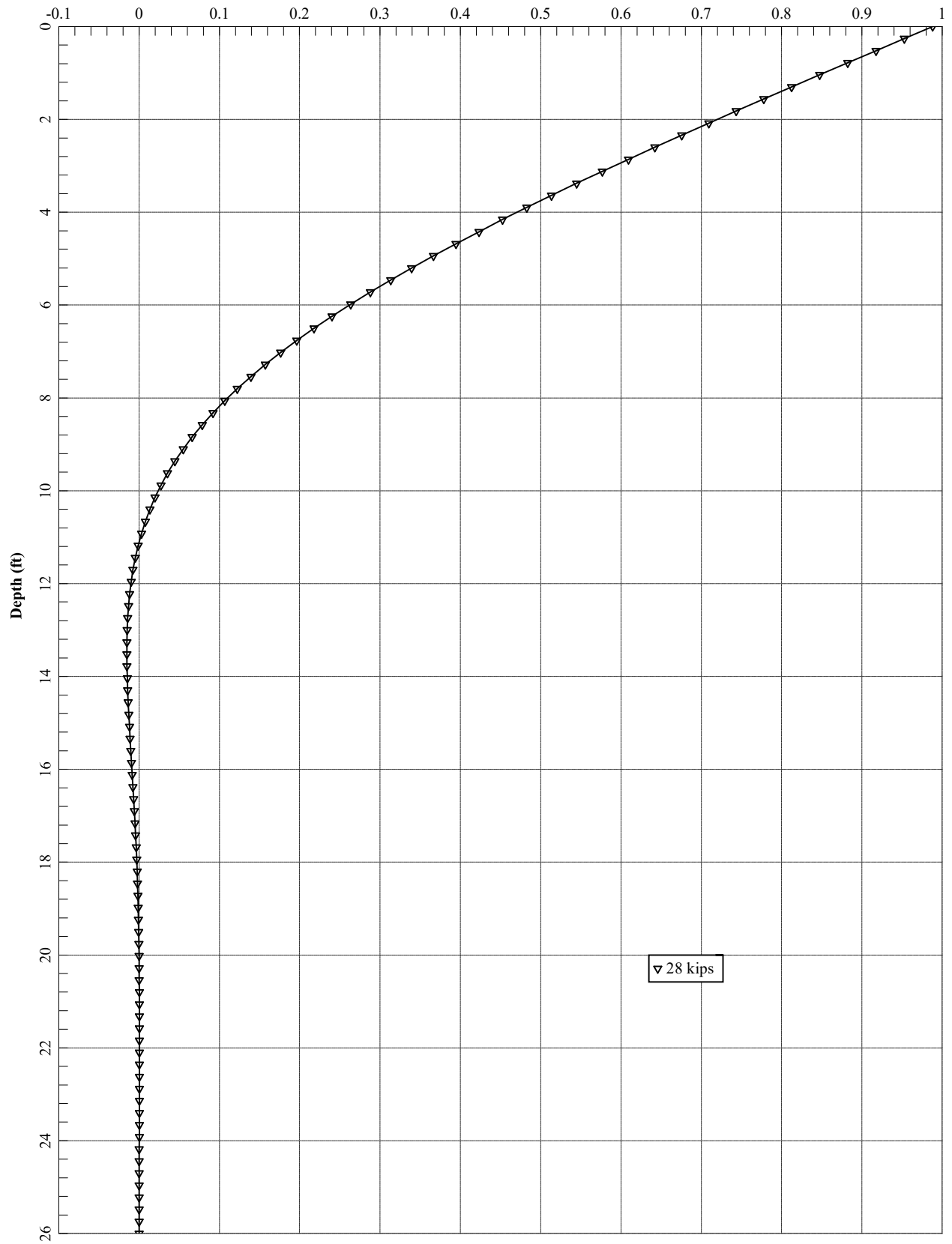
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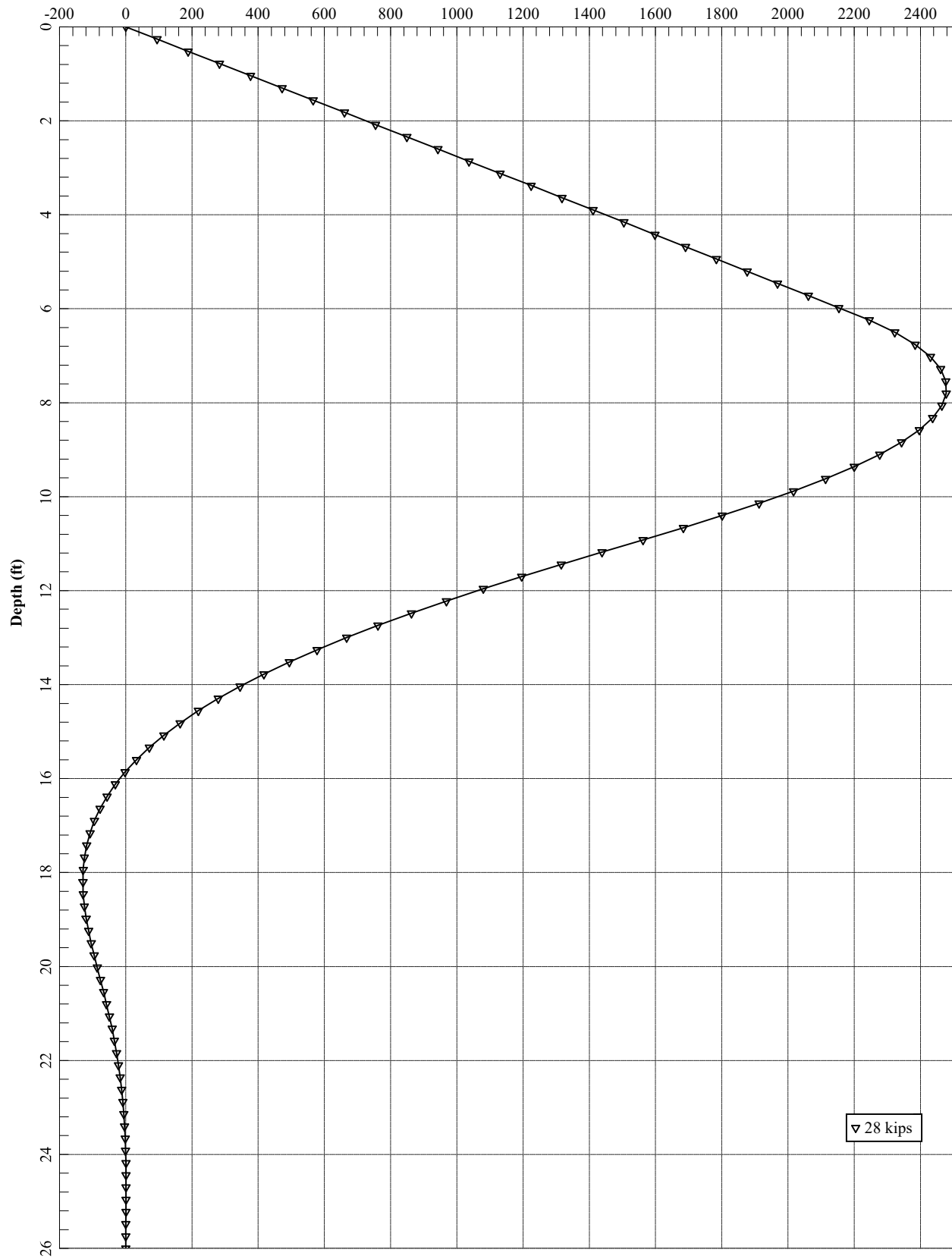
Ackerman Creek Bridge - Abutment 2 Longitudinal, Service (3.5' Scour)
Shear Force (kips)



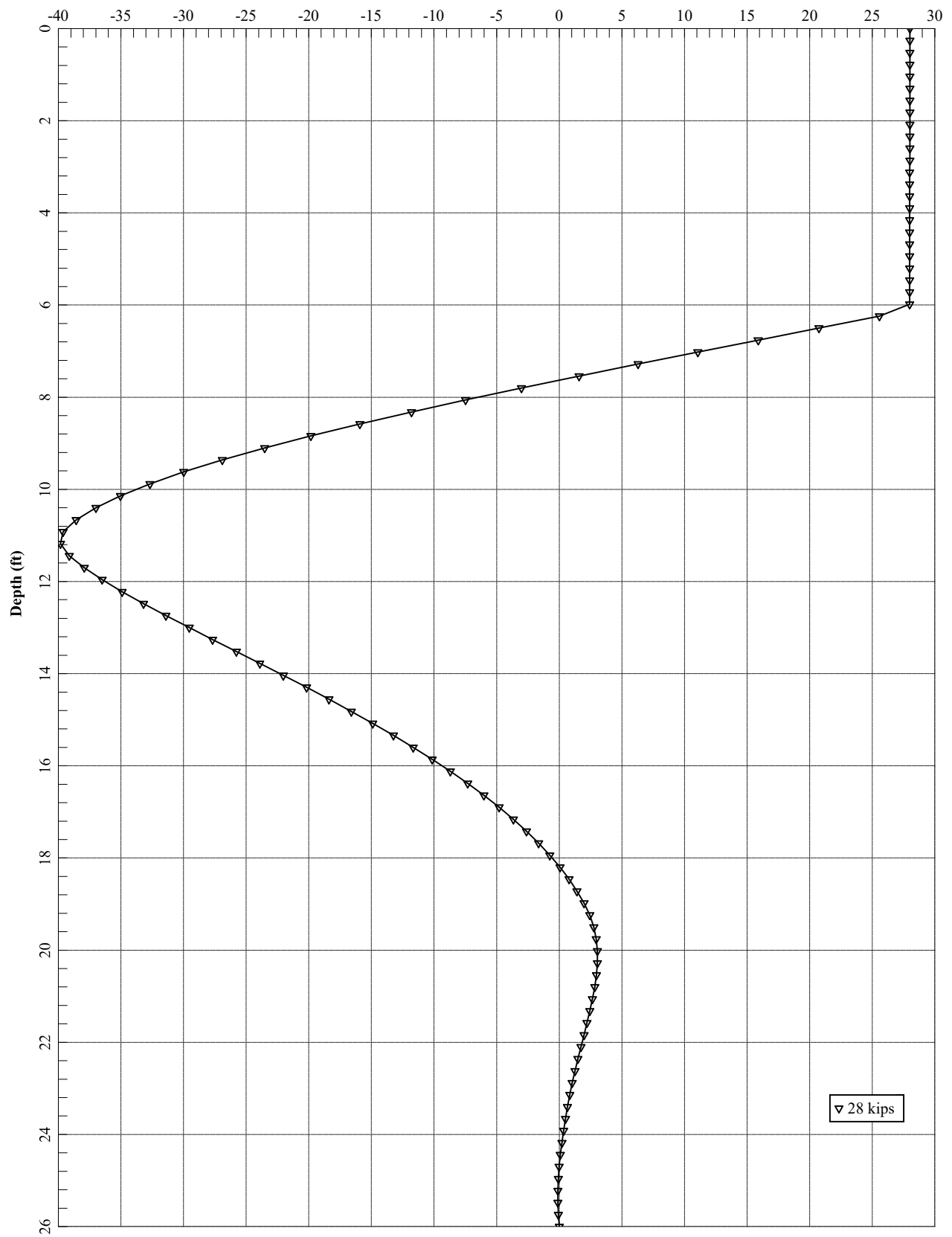
Ackerman Creek Bridge - Abutment 2 Longitudinal, Strength (1.75' Scour)
Lateral Pile Deflection (inches)

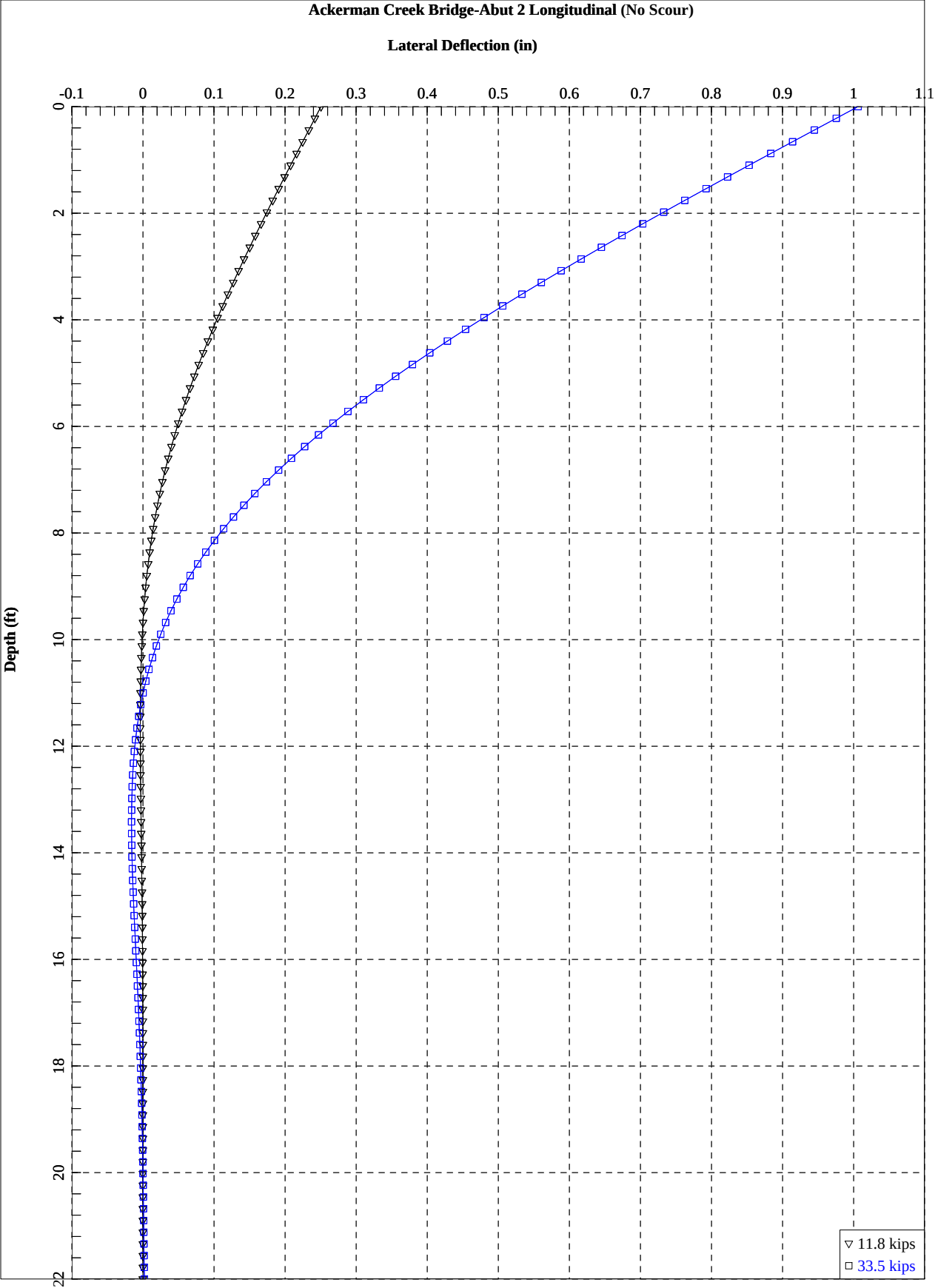


Ackerman Creek Bridge - Abutment 2 Longitudinal, Strength (1.75' Scour)
Bending Moment (in-kips)



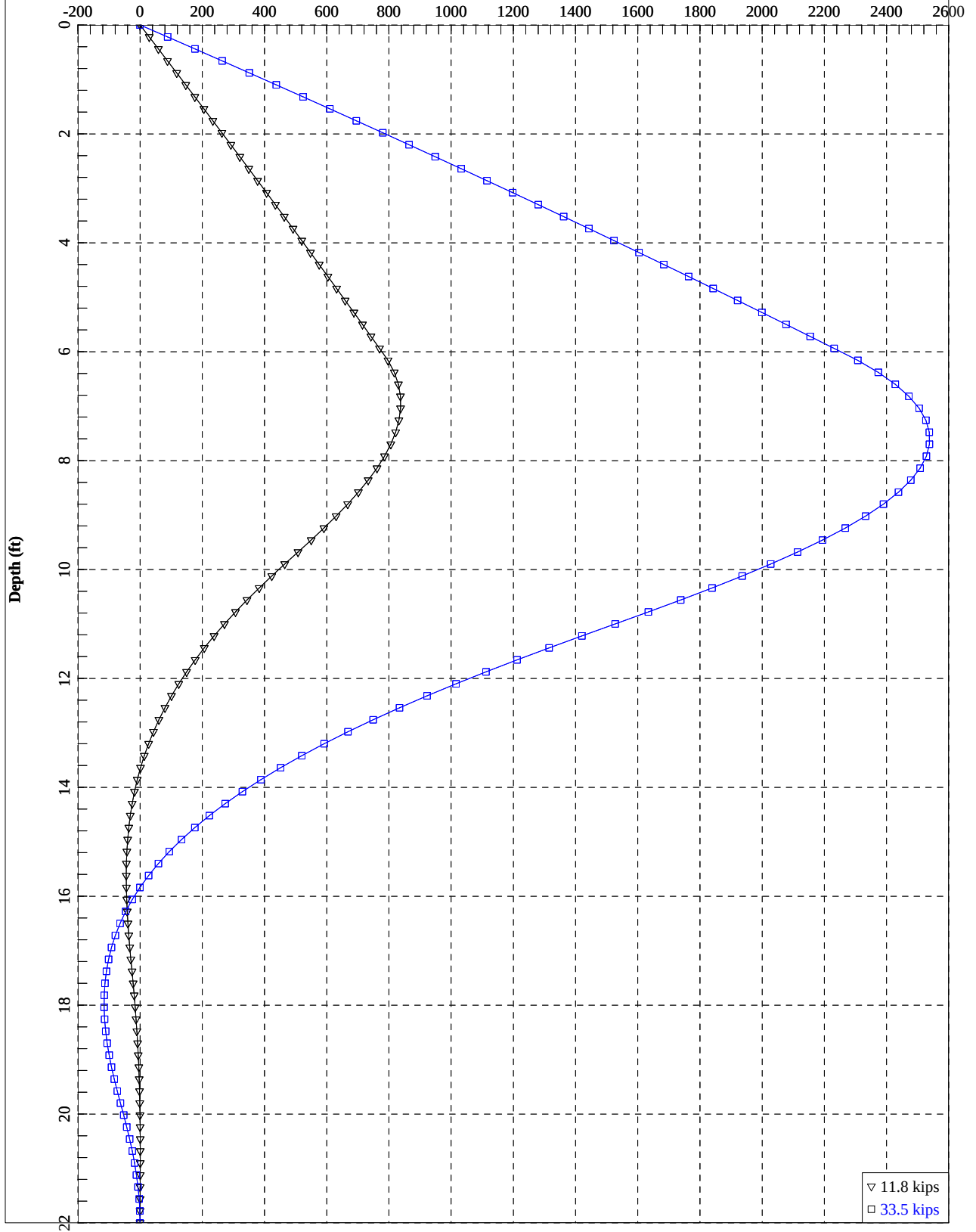
Ackerman Creek Bridge - Abutment 2 Longitudinal, Strength (1.75' Scour)
Shear Force (kips)

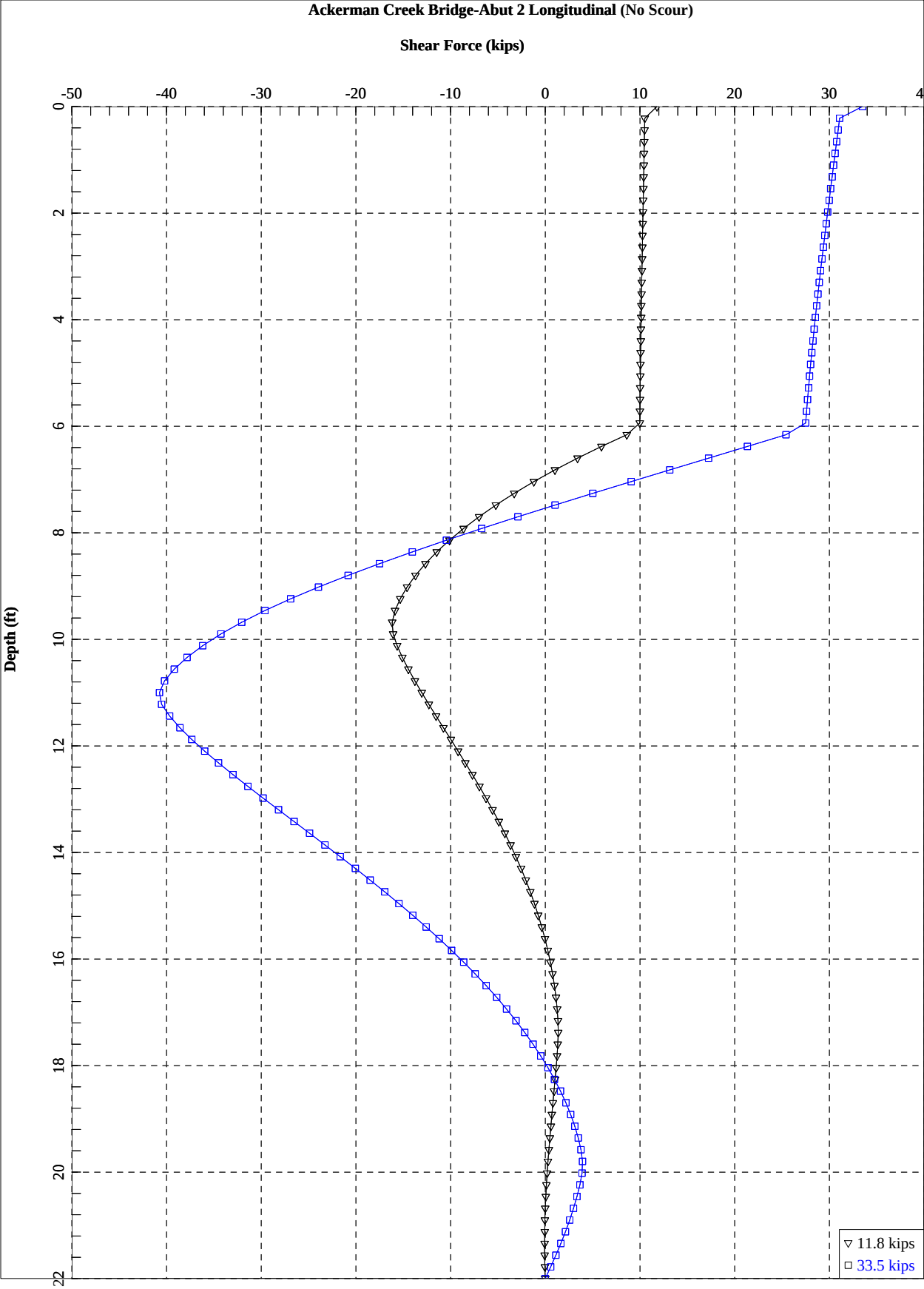




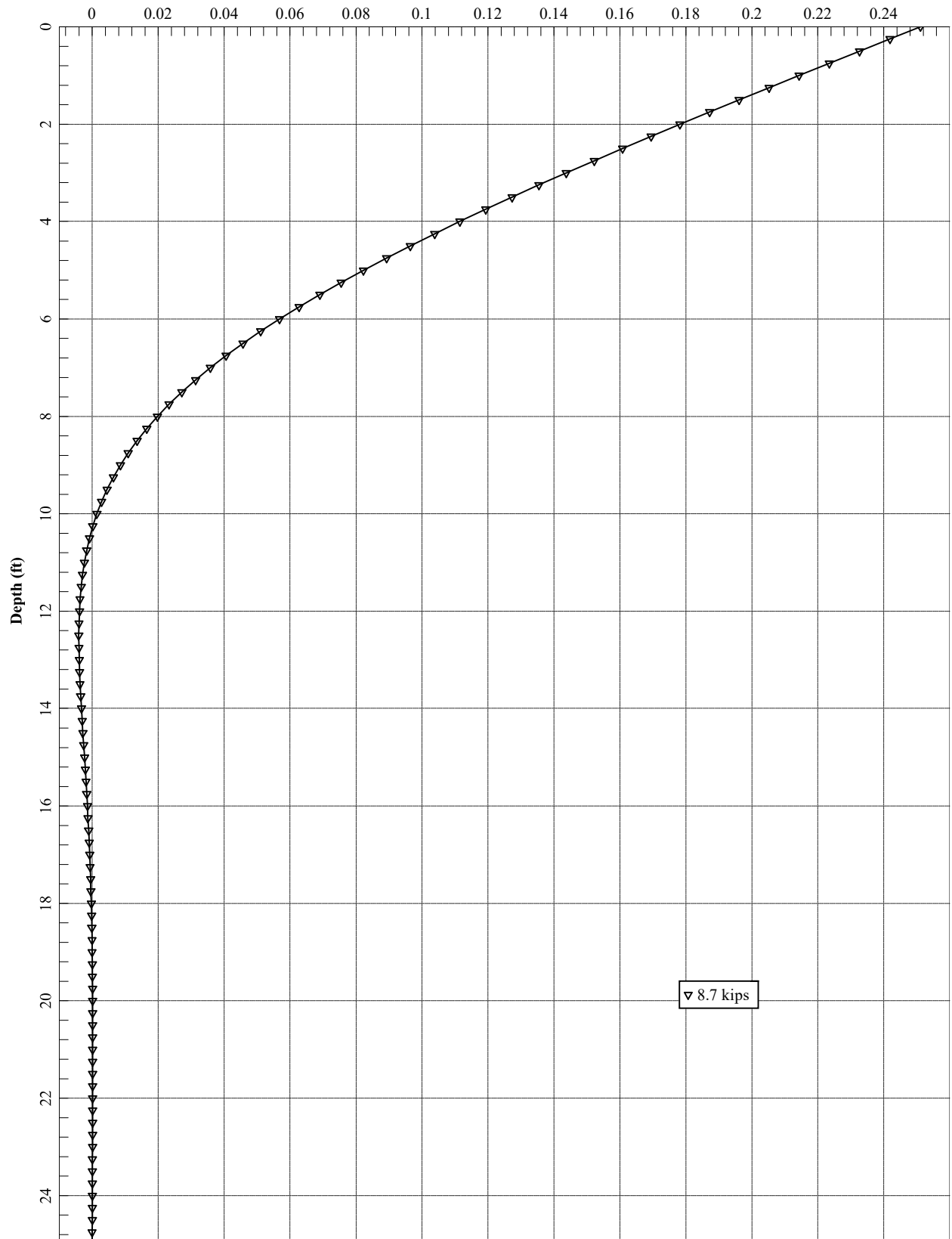
Ackerman Creek Bridge-Abut 1 Longitudinal (No Scour)

Bending Moment (in-kips)

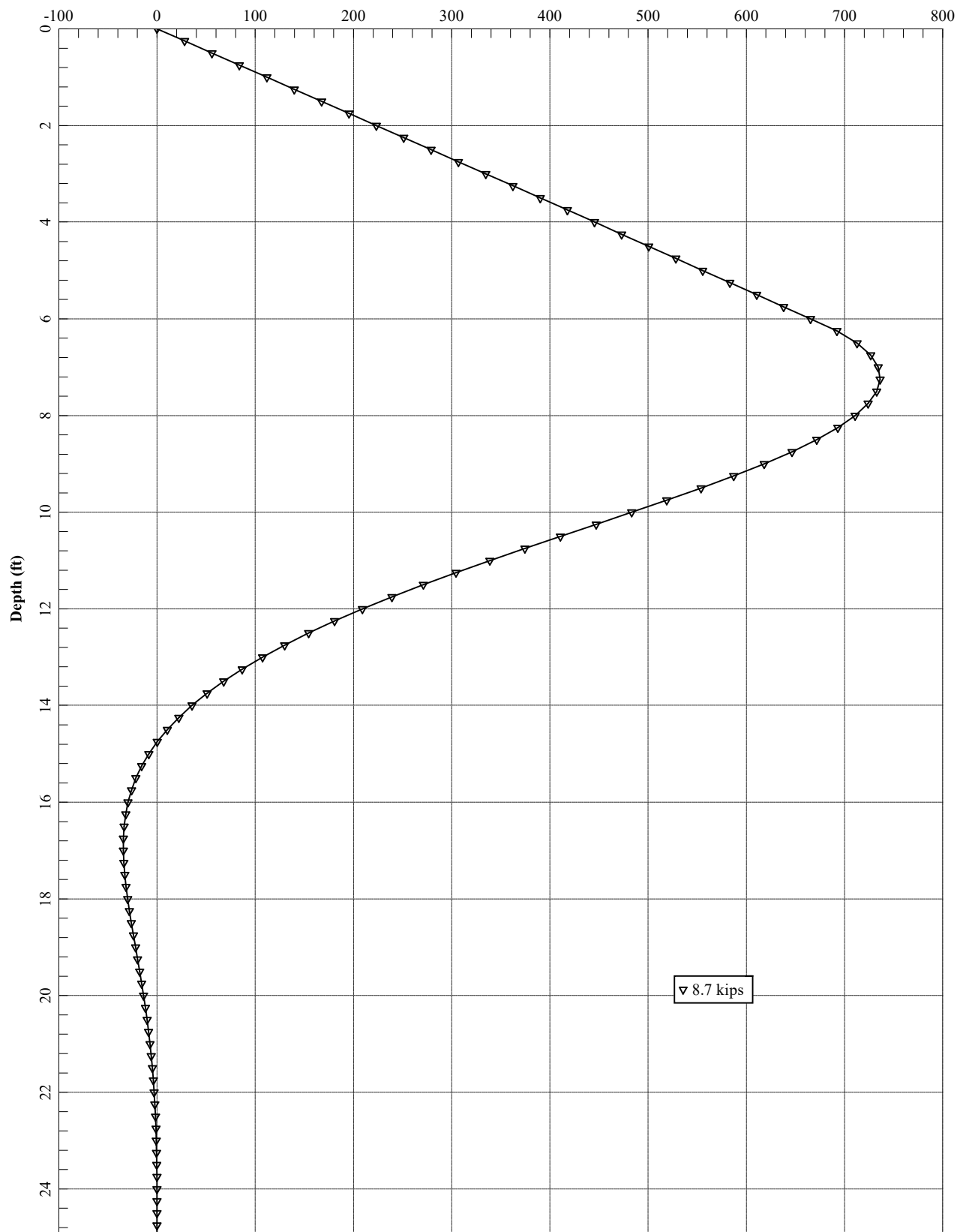




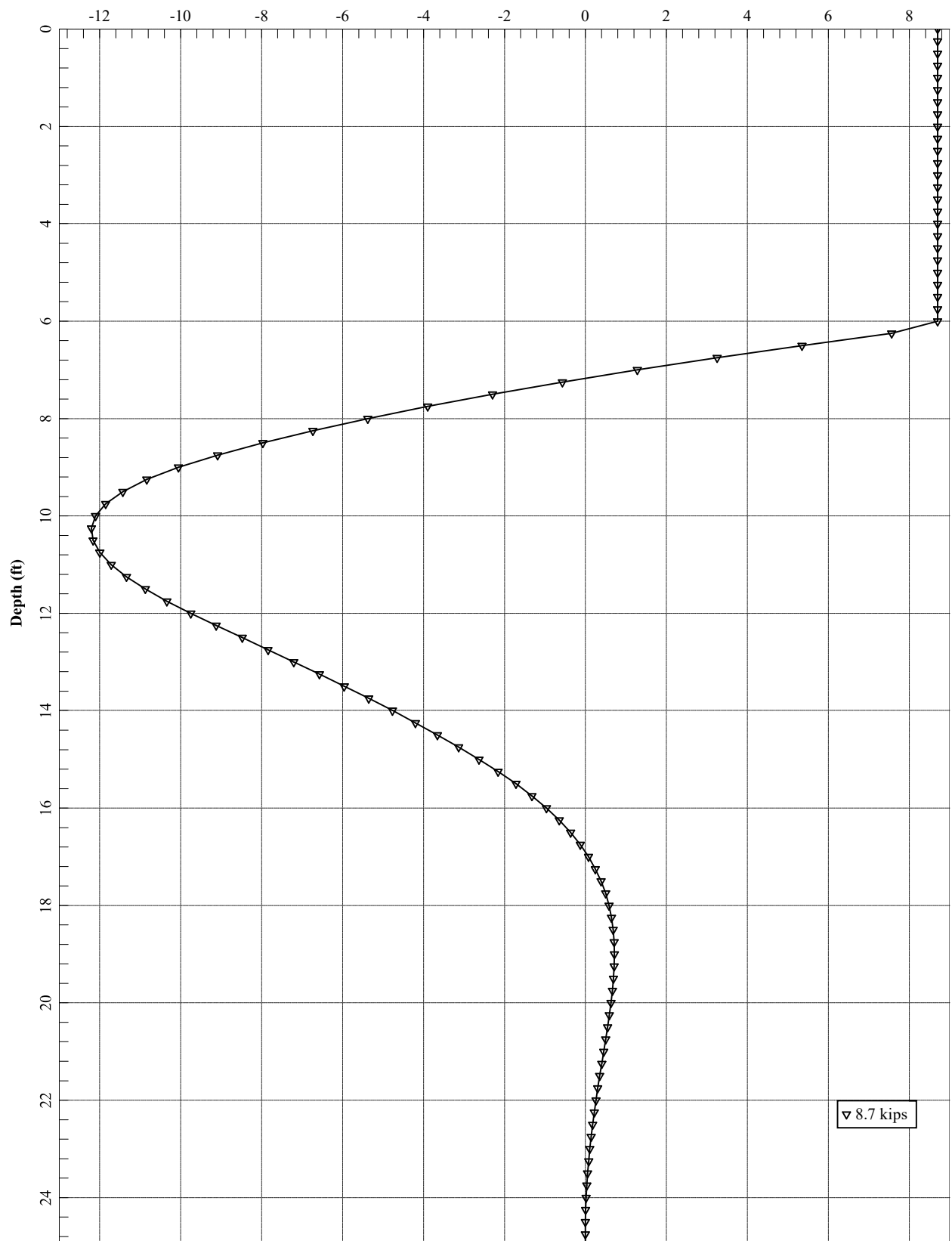
Ackerman Creek Bridge- Abutment 2 Transverse, Service (3.5' Scour)
Lateral Pile Deflection (inches)



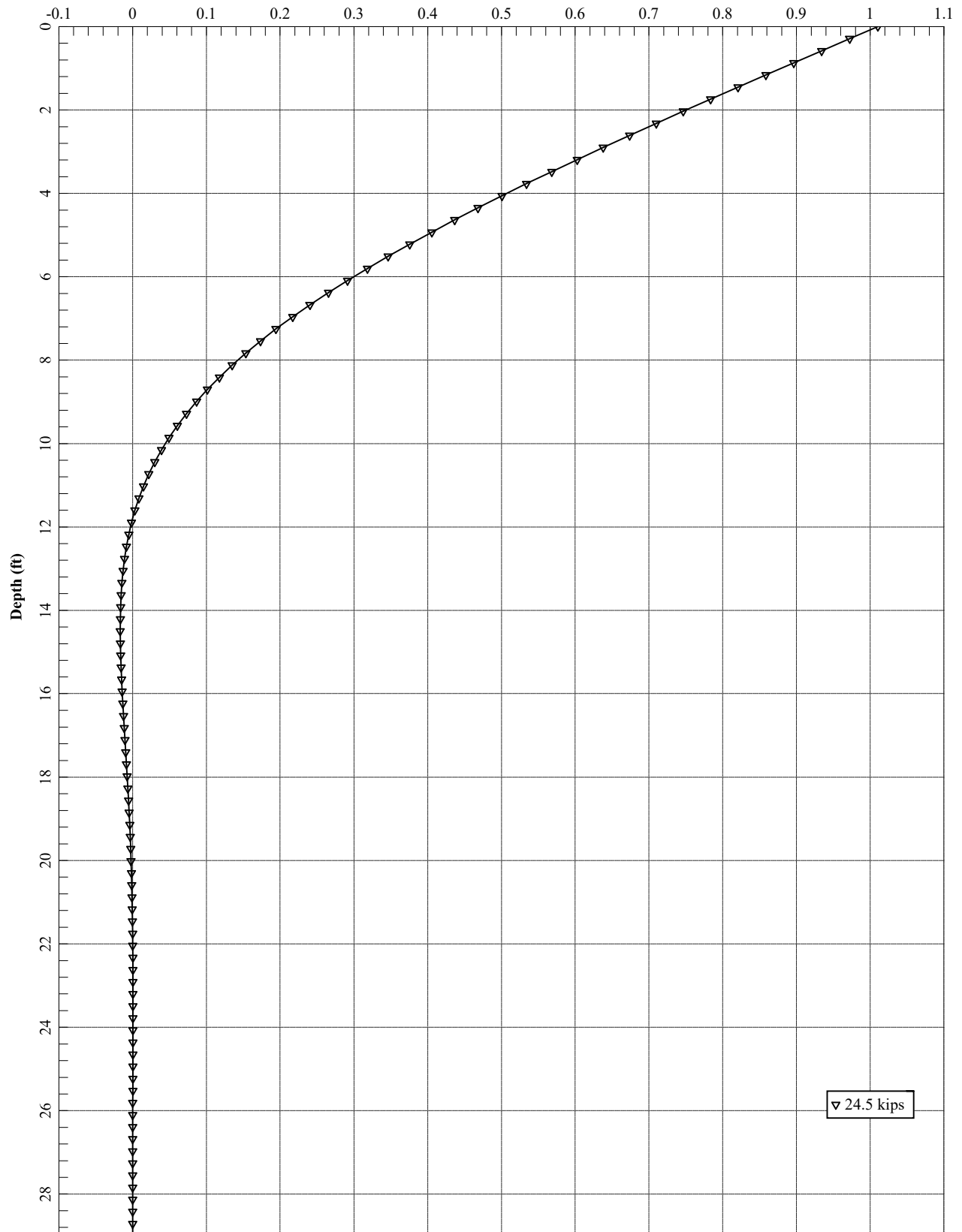
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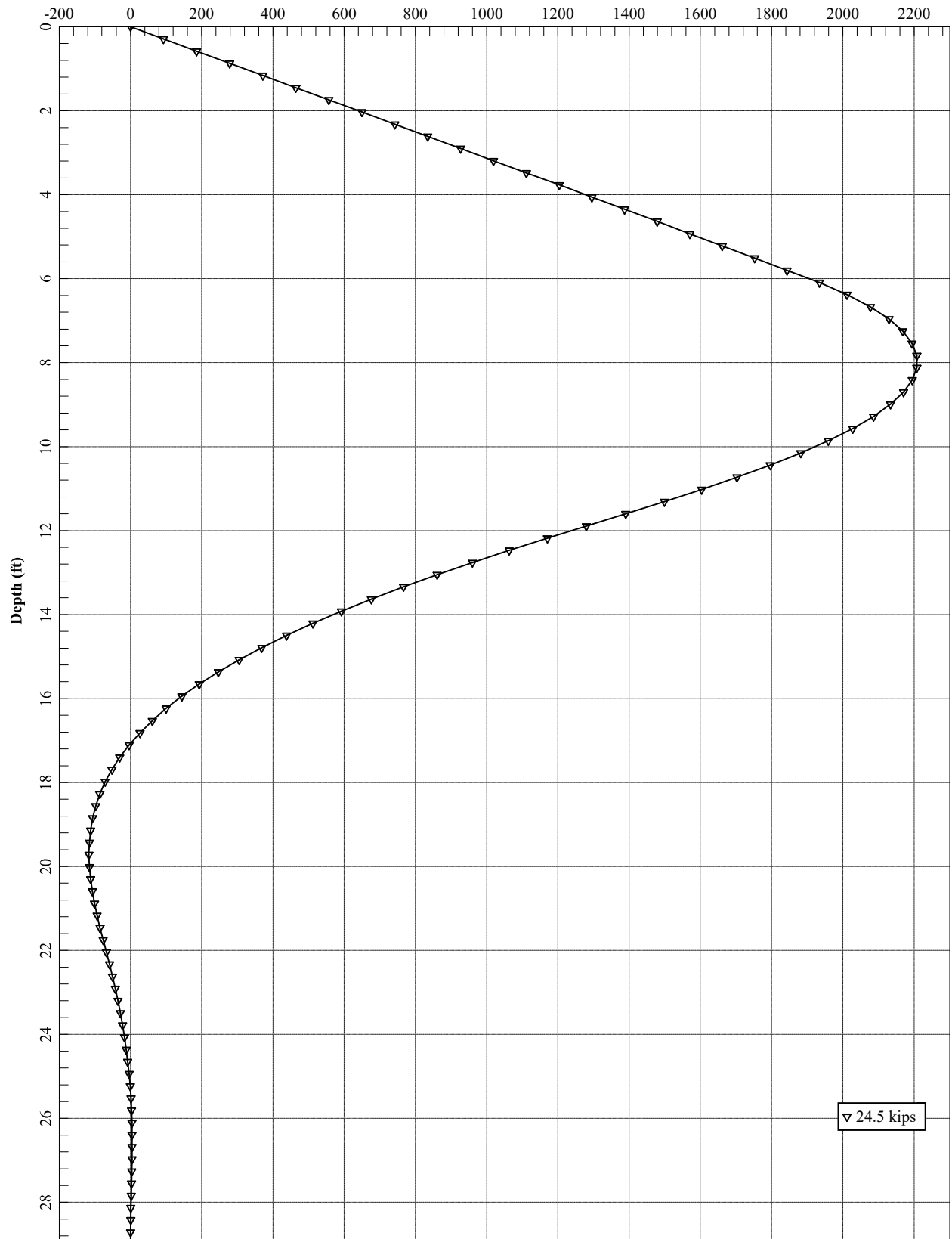
Ackerman Creek Bridge- Abutment 2 Transverse, Service (3.5' Scour)



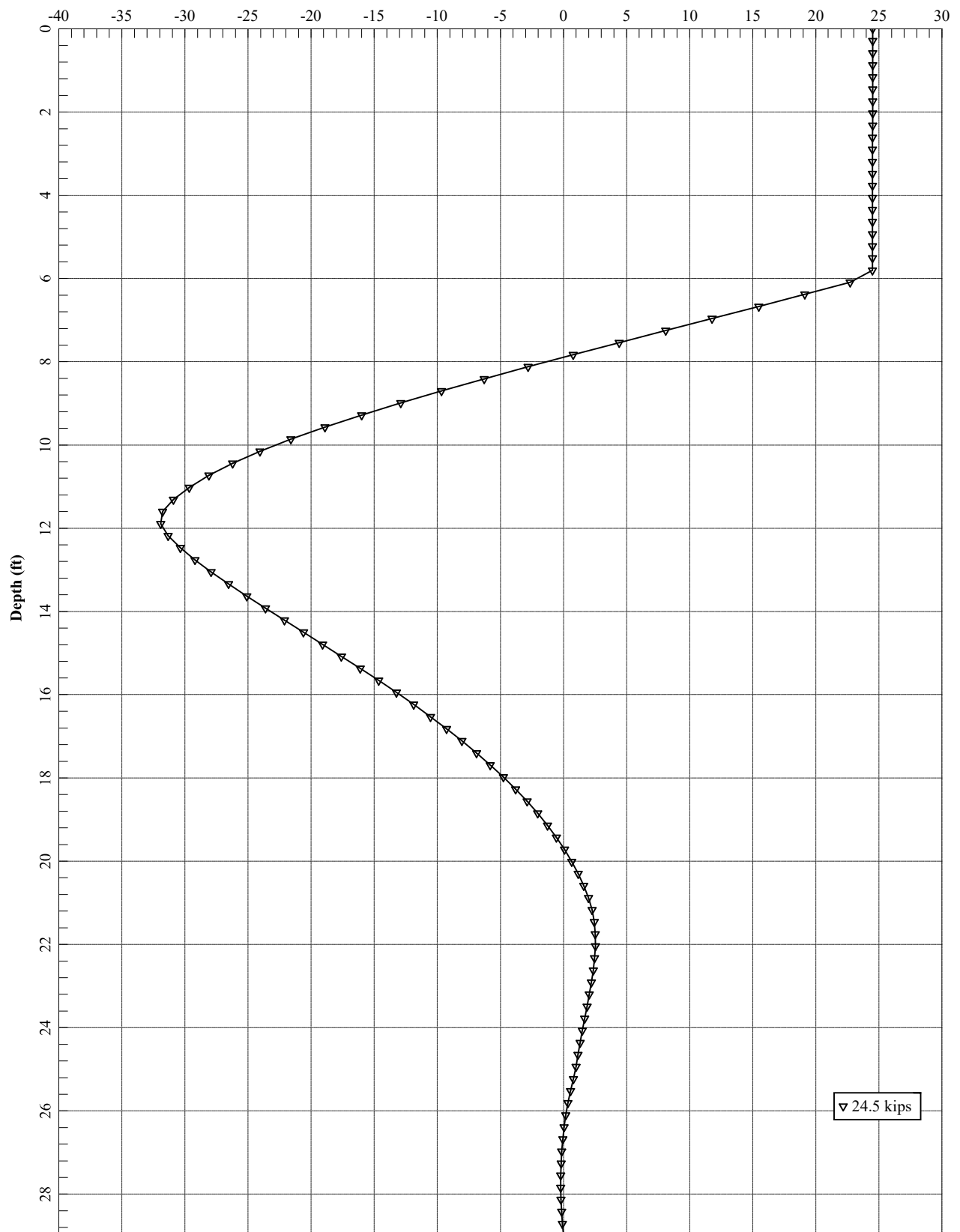
Ackerman Creek Bridge - Abutment 2 Transverse, Strength (1.75' Scour)
Lateral Pile Deflection (inches)



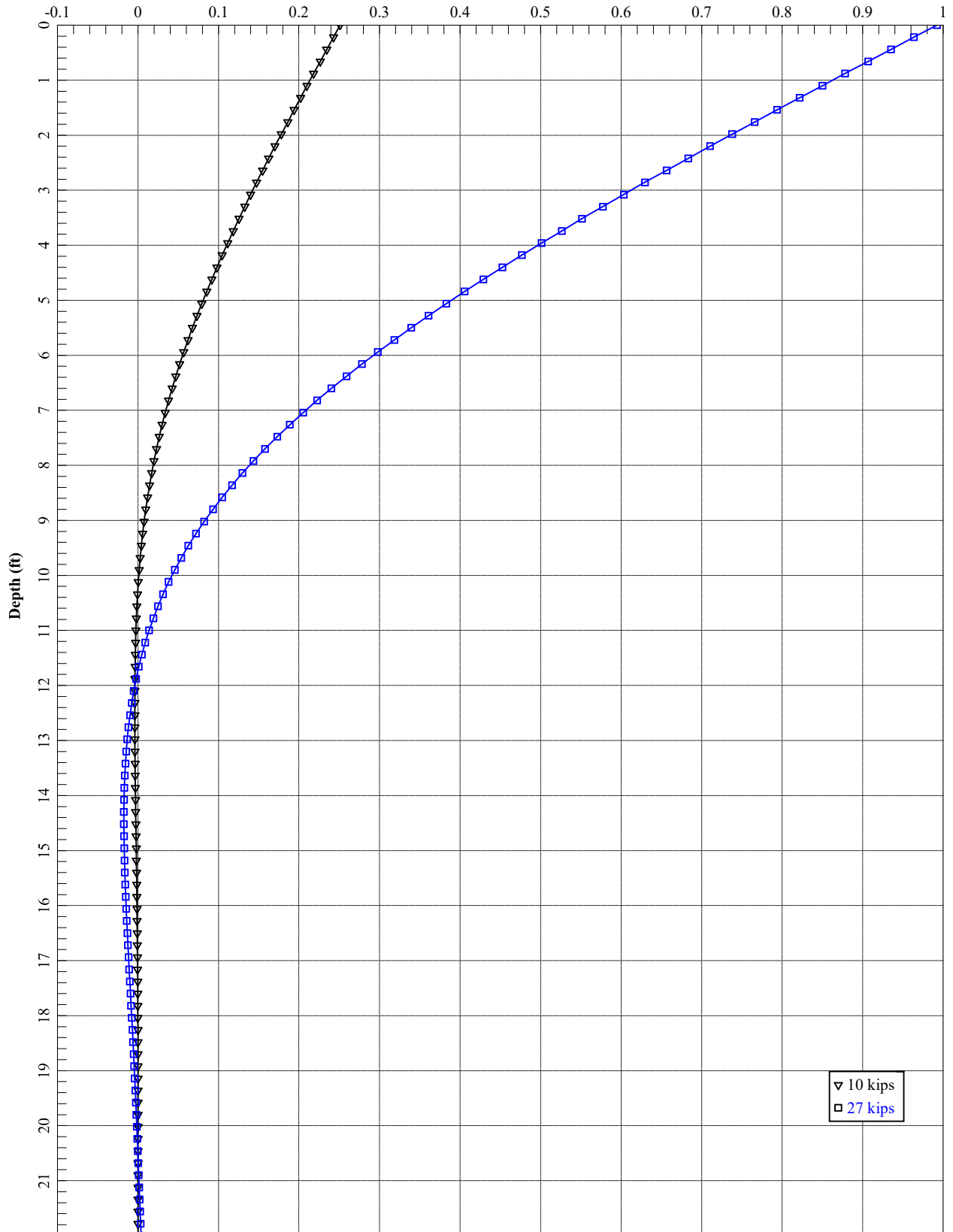
Ackerman Creek Bridge - Abutment 2 Transverse, Strength (1.75' Scour)
Bending Moment (in-kips)



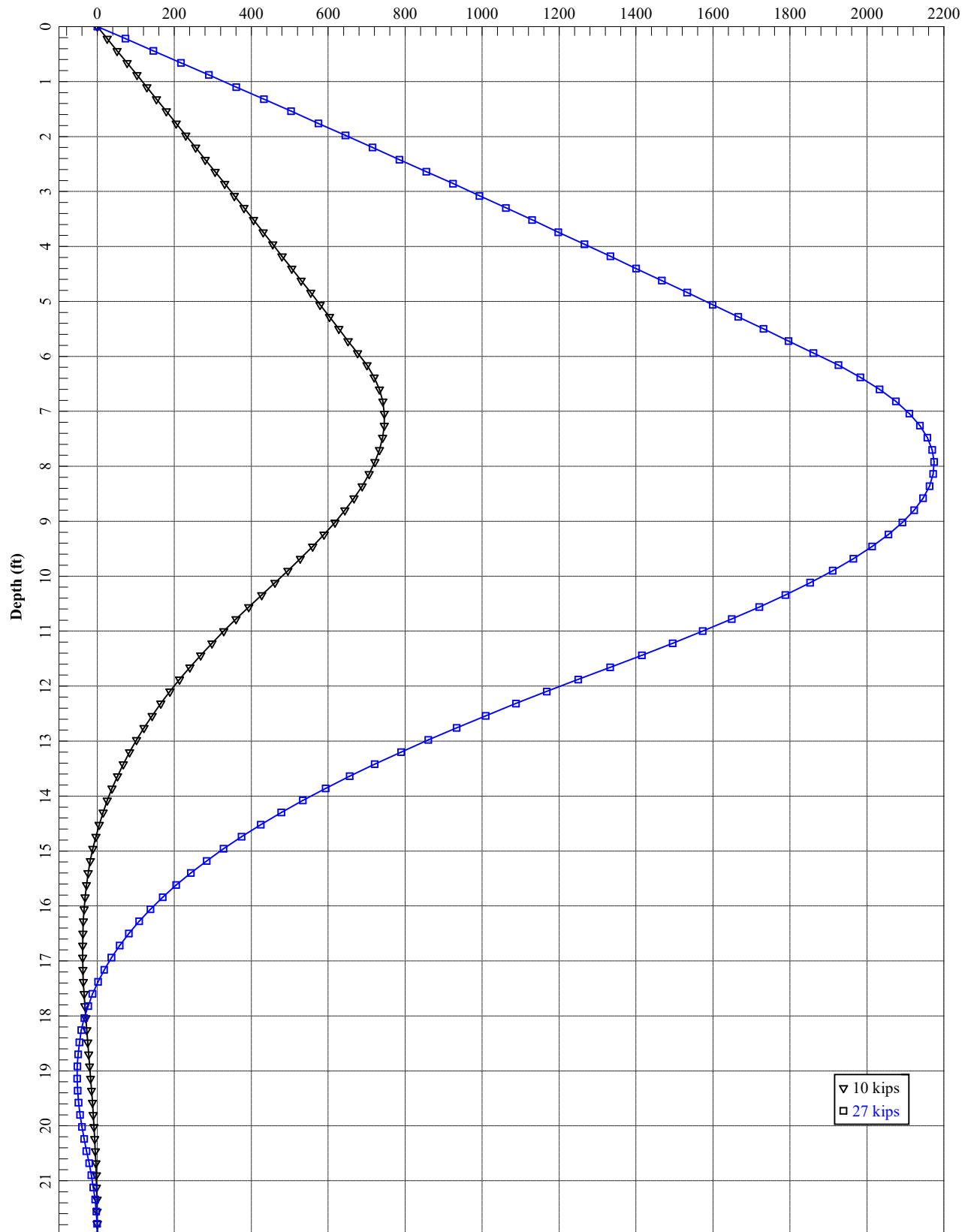
Ackerman Creek Bridge - Abutment 2 Transverse, Strength (1.75' Scour)



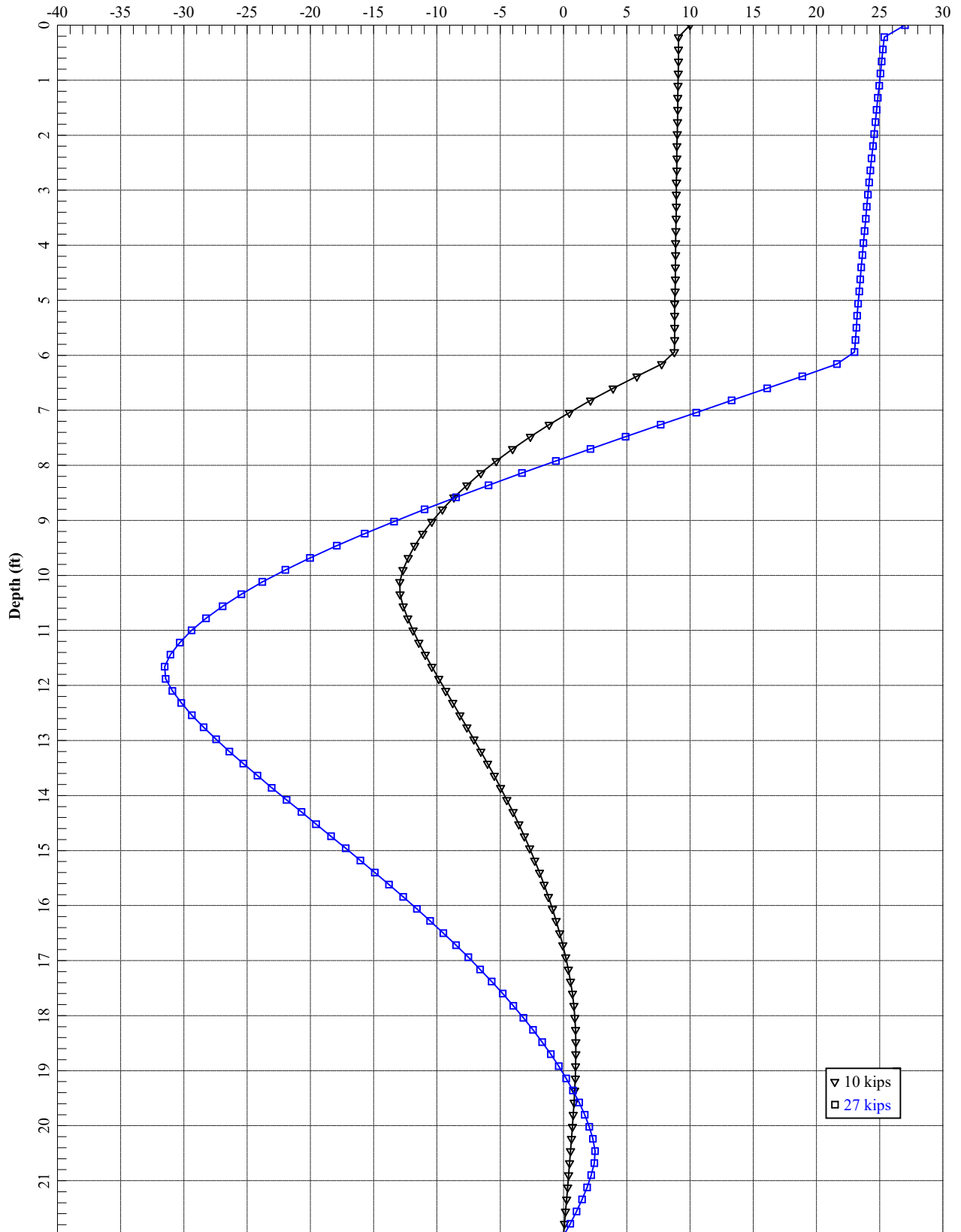
Ackerman Creek Bridge - Abutment 2 Transverse No Scour
Lateral Pile Deflection (inches)



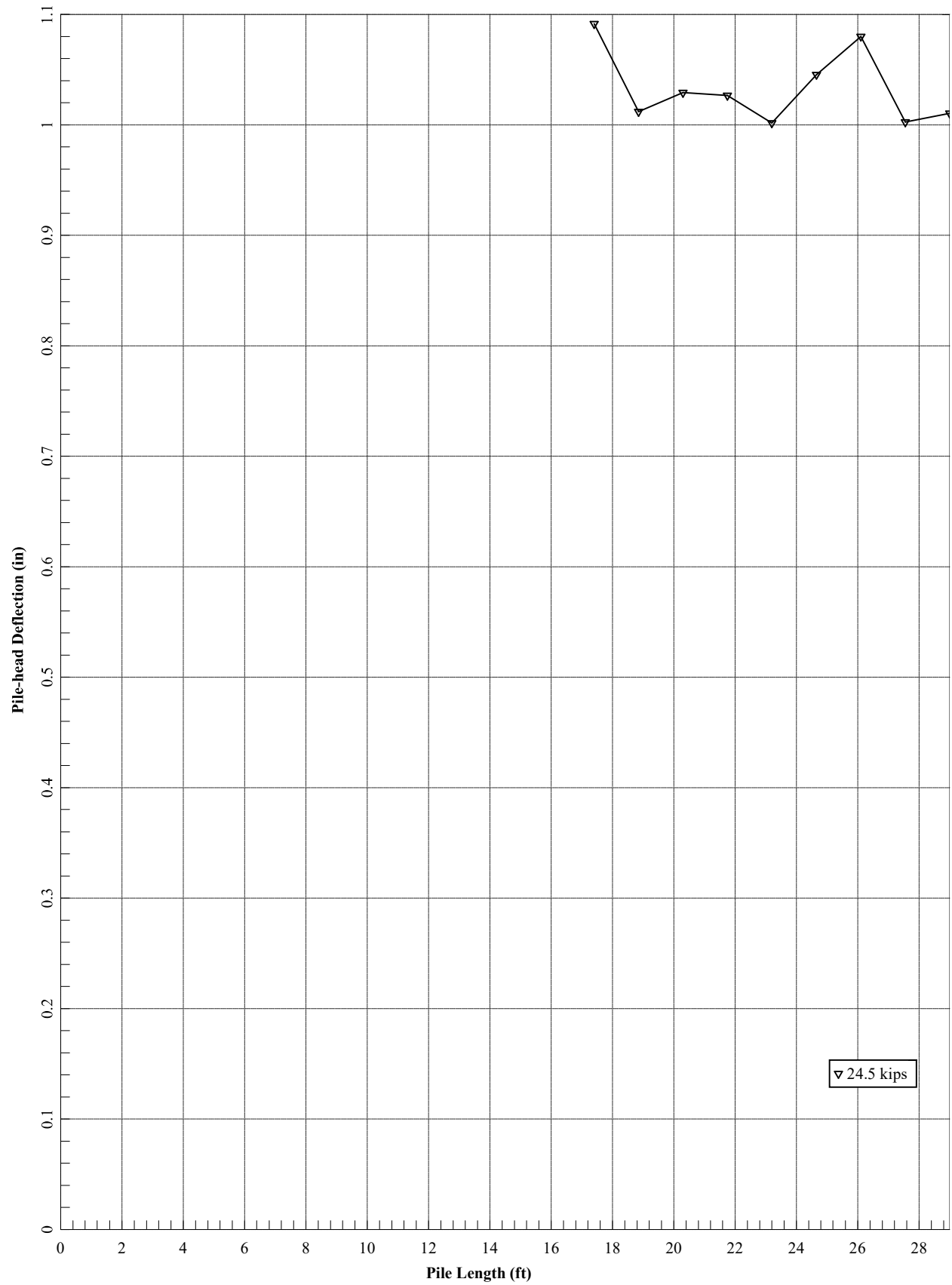
Ackerman Creek Bridge - Abutment 2 Transverse No Scour
Bending Moment (in-kips)



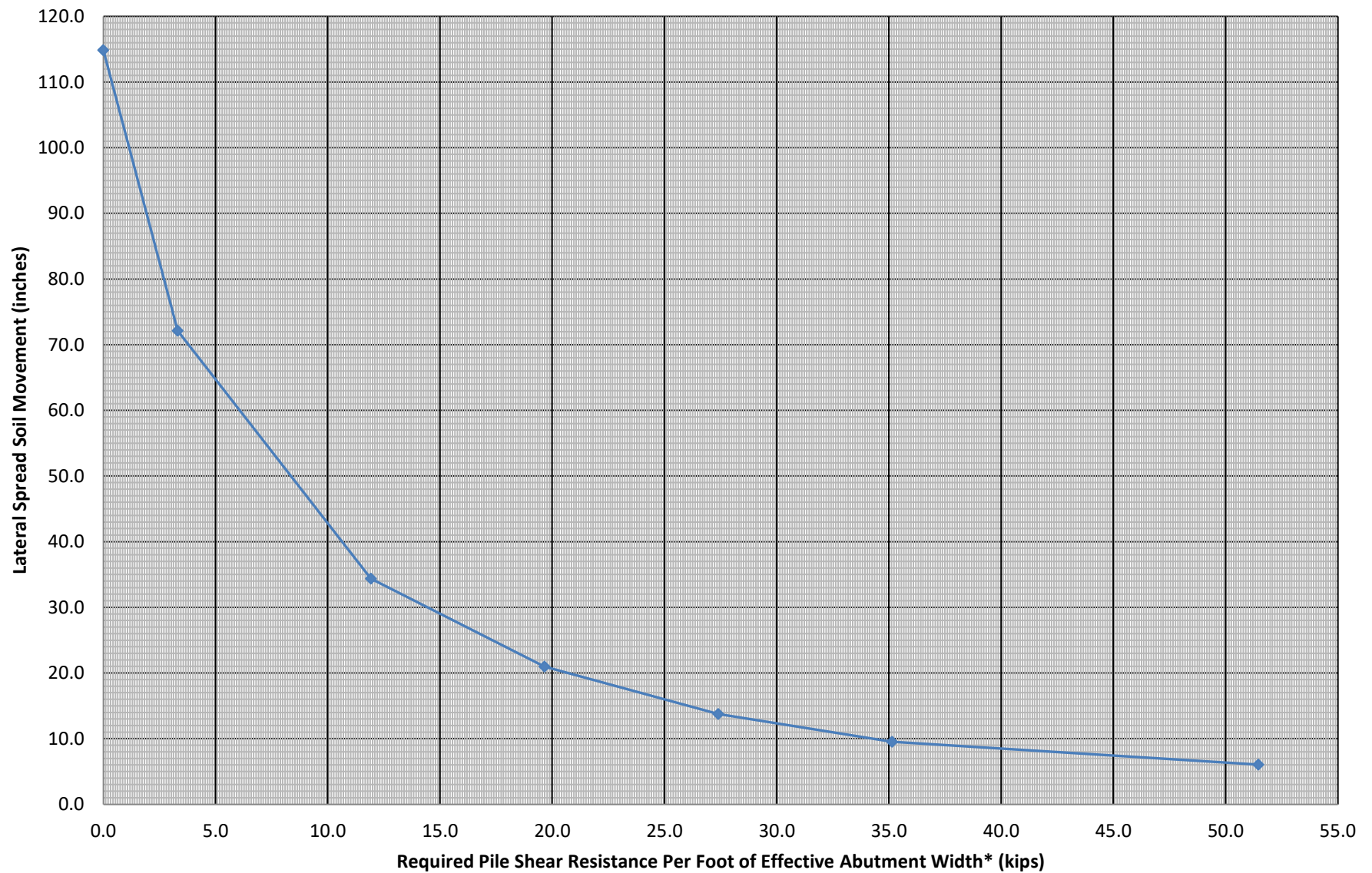
Ackerman Creek Bridge - Abutment 2 Transverse No Scour
Shear Force (kips)



Ackerman Creek Bridge - Abutment 2 Transverse, Strength (3.5' Scour)



Lateral Spread Soil Movement Vs. Required Pile Shear Resistance (Variable Failure Surface Length)



*1.5x the foundation width per Caltrans Memo To Designers 20-15 (2017)

Soil Shear Strength - Liquified Layer (psf)	Yield Acceleration Ky	*Required Pile Shear Resistance (kips per foot of **embankment width)	***Lateral Movement (inches)	Failure Surface Length within Liquefiable Layer (ft.)
296 (residual)	0.06	0.0	114.9	61.4
350	0.09	3.3	72.1	61.4
450	0.15	11.9	34.4	77.4
550	0.2	19.7	21.0	77.4
650	0.25	27.4	13.8	77.4
750	0.3	35.1	9.5	77.4
900	0.37	51.5	6.1	85.2

*Equal to the soil shear strength minus the residual soil strength multiplied by the length of the failure surface within the liquified layer.

**The width of the embankment should include the width of the abutment plus a portion of the side slope masses as outlined in Ashford, Boulanger, and Brandenburg (2011), Recommended Design Practice for Pile Foundations in Laterally Spreading Ground, PEER

*** Bray and Travasarou (2007) method, Caltrans Guidelines on Foundation Loading and Deformation Due to Liquefaction Induced Lateral Spreading (October 2013), using PGA = 0.798g, Moment Magnitude = 7.4, and above Ky values.

FOUNDATION REPORT
N. State Street Bridge at Ackerman Creek

Bridge No. 10C-0065
Mendocino County, CA

November 2023

APPENDIX D:

GBA-Important info about Geotech Report

Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual site-wide subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists.*



GEOPROFESSIONAL
BUSINESS
ASSOCIATION

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e-mail: info@geoprofessional.org www.geoprofessional.org