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December 14, 2006
File No.: 35352.GEO

Craig Drake, P.E.
TY Lin International
10365 Old Placerville Road, Suite 200
Sacramento, California 95827

SUBJECT: Preliminary Foundation Report
Main Street Bridge at Santa Rosa Creek
Cambria, San Luis Obispo County, California
State Bridge No. 49C-0337

Dear Mr. Drake:

The attached report presents geotechnical design criteria for the Main Street bridge replacement at Santa Rosa Creek in San Luis Obispo County, California. This report describes the study and provides conclusions and recommendations for use in project design.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to TY Lin International and San Luis Obispo County Department of Public Works. We trust this information meets your current needs. If there are any questions concerning the information presented in this report, please contact this office at your convenience.

Respectfully submitted,

KLEINFELDER, INC.

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December 14, 2006

PRELIMINARY FOUNDATION REPORT
MAIN STREET BRIDGE
AT SANTA ROSA CREEK
05-SLO-0-CR
BRLO-5949 (065)
05-927220
STATE BRIDGE NO. 49C-0337
CAMBRIA, CALIFORNIA

Prepared For:

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December 14, 2006

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Prepared For:
TY Lin International
10365 Old Placerville Road, Suite 200
Sacramento, California 95827

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COVER LETTER

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A	SITE VICINITY MAP AS BUILT LOG OF TEST BORINGS
B	LABORATORY TEST RESULTS
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1. INTRODUCTION

1.1. PURPOSE

This report presents the results of a geotechnical investigation for the replacement of the Main Street Bridge at Santa Rosa Creek (Br No 49C-0337), located southeasterly of the intersection of Main Street and Santa Rosa Creek Road, in Cambria, San Luis Obispo County, California. The location of the project site is illustrated on the Site Vicinity Map, Plate 1, Appendix A of this report.

The purpose of this investigation was to evaluate the general soil and foundation conditions, and provide geotechnical recommendations and opinions to aid in the structure design. Kleinfelder's Client was T. Y. Lin International. Geotechnical input has been provided by verbal and email communication during the Design Concept Approval and Type Selection process.

1.2. SCOPE OF SERVICES

The scope of services consisted of site reconnaissance and field exploration, laboratory testing, geotechnical engineering and analysis, and preparation of this report. Specific items that are addressed include:

- Pertinent data provided by Client and other members of Project Design Team;
- Results of field exploration;
- Comments on the regional geology and site engineering seismology, including the recommended peak bedrock acceleration (PBA) and Caltrans Seismic Design Criteria Version 1.4 ARS curve;
- Analysis and comments on liquefaction potential;
- Comments on bridge foundation types;
- Recommendations for alternative pile foundations, including design and specified tip elevations and available nominal capacities;

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- Parameters for use in LPILE;
- General comments on project grading;
- Recommended flexible pavements sections for roadway pavements;
- Comments on proposed roadway retaining walls;
- General comments on the corrosion potential of foundation soil;
- Log of Test Borings for inclusion into the contract drawings.

1.3. PROJECT DESCRIPTION

1.3.1. Bridge

The existing two-span bridge will be removed and replaced with a new single-span bridge 150 feet in length. The proposed alignment centerline for the replacement bridge is located approximately 36 to 39 feet upstream from the existing centerline (easterly of the existing bridge).

The Design Concept Approval and Type Selection process has determined a single span cast-in-place, pre-stressed concrete box girder type structure. The approach fill depth and Peak Bedrock Acceleration would require a Type EQ10 approach slab at each end of the bridge.

The planned width for the new structure is 37 feet, measured from outer edge of concrete barriers. Each lane will be 12 feet wide. The planned supports are noted in Table 1.3-1.

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TABLE 1.3-1
PLANNED BRIDGE SUPPORT DATA

Support	Location (Main Line)	Deck Elevation (ft)	Bottom of Pile Cap Elevation (ft)
Abutment 1	16 + 60	79.01	+56.0
Abutment 2	18 +10	78.22	+56.0

1.3.2. Roadway

The realignment will require road improvements along Main Street from about Station 13+50 to Station 21+25 and for about 100 feet northerly on Santa Rosa Creek Road. Fill is anticipated at both approaches to the bridge. Anticipated thickness of fill is about 16 to 18 feet. Final slopes beneath both abutments will have a gradient of 1½:1 (H:V).

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2. FIELD AND LABORATORY PROGRAMS

2.1. FIELD EXPLORATION

The geotechnical field exploration consisted of drilling two test borings, one boring at each end of the existing bridge. The test borings were performed using 6-inch diameter hollow-stem auger techniques. Boring B-1 was drilled to a depth of 91.5 feet (elevation -17.2 feet) and boring B-2 was drilled to a depth of 81.5 feet (elevation -7.0 feet). Samples were obtained from the test borings with either a 1.4-inch I.D. standard penetration sampler (SPT) or a 2.5-inch I.D. modified California sampler. The SPT sampler was used without liners. The sampler was driven into the bottom of the boring using a 140-pound hammer held by a rope on cathead, dropping 30 inches. The sampler is driven 18 inches and the number of blows taken to penetrate the final 12 inches is recorded. The blow counts listed on the Log of Test Borings have not been corrected for the effects of overburden pressure, rod length, sample size, or hammer efficiency. Bulk samples were also obtained from the auger cuttings of both borings.

A Kleinfelder field engineer logged the type of soils encountered during the drilling operation. Soil samples obtained were taken to the laboratory for geotechnical testing. The borings were backfilled with soil cuttings and covered with asphalt cold patch.

2.2. LABORATORY DATA

Laboratory tests were performed on selected samples to evaluate certain physical characteristics and engineering properties. The laboratory testing program was designed with emphasis on the evaluation of geotechnical properties of foundation materials as they pertain to the proposed construction. The laboratory testing program included performing the following tests:

- ❑ Sieve Analysis (ASTM C136)
- ❑ Unit Weight (ASTM D2937)
- ❑ Moisture Content (ASTM D2216)

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- Direct Shear (ASTM D3080)
- Soluble Sulfates (California Test Method No.417)
- Soluble Chlorides (California Test Method No.422)
- Resistivity and pH (California Test Method Nos. 532 and 643)
- Resistance Value (California Test Method No. 301)

The dry density and moisture content test results are shown on the Log of Test Borings, Appendix A. The soluble sulfate, soluble chloride, pH and minimum resistivity are discussed in the "Corrosion Potential" section (Section 5.7.) The remaining test results are provided in Appendix B.

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3. SITE CONDITIONS

3.1. SURFACE CONDITIONS

The project site consists of an existing bridge and connecting approaches paved with asphalt concrete (AC). This site is located easterly of the town of Cambria, where Main Street intersects Santa Rosa Creek. The project vicinity is generally surrounded by coastal hills. The area of the new alignment is mostly covered with relatively heavy vegetation consisting of vines, brush, and small to large trees.

The Santa Rosa Creek channel supports year round flow. Its general downstream direction is southerly.

3.2. EARTH MATERIALS

The earth material deposits along Santa Rosa Creek are comprised of recent alluvium and stream gravel (San Luis Obispo Sheet 1959, Geologic Map of California). Outside the general creek influence, the earth materials generally consist of Mesozoic Era Upper Cretaceous marine sedimentary rock and Cretaceous/Jurassic Melange. None of the bedrock material is within the influence of the planned bridge.

Earth materials encountered in the borings consisted of medium to very dense silty sand fill soils up to about 15 to 19 feet thick. The fill material contained trace of gravel. Based on these thicknesses, the bottom of fills corresponds to Elevation 60 feet (Boring B-1) and Elevation 56 feet (Boring B-2), respectively. These fill soils were underlain by interbedded, laterally discontinuous, medium dense to dense silty sand, medium dense to dense clayey sand, medium stiff to stiff sandy clay, medium dense sand and some medium dense to dense sandy gravel. Refer to the Log of Test Boring drawing in Appendix A for a detailed description of earth material profile encountered in borings B-1 and B-2.

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The test borings were drilled in the existing pavement for Main Street. In Boring B-1, the existing pavement section was approximately 1.25 feet of asphalt concrete (AC). About 1.5 feet of AC was encountered in Boring B-2. At both borings, the AC was placed directly on subgrade soil (no aggregate base).

3.3. GROUNDWATER CONDITIONS

Groundwater was encountered at about 28 feet (Elevation 46.5 feet) in both borings. This groundwater level was closely coincident with the water level in Santa Rosa Creek (about Elevation 48 to 49 feet) at the time of drilling.

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4. GEOLOGY & SEISMICITY

4.1. GEOLOGY

The project site lies on the western flank of the Santa Lucia Range, which is within the Coast Ranges geomorphic province of California. The geology of the Santa Lucia Range is relatively complex, consisting of en echelon zones of sedimentary bedrock and extruded mélange. There are numerous northwest-southwest trending faults within the Mesozoic Era rocks comprising the range.

4.2. LOCAL FAULTING AND SEISMICITY

The site is not located within an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 published by the CGS.

The site is in relatively close proximity to several potentially active faults. While not well defined, an apparent or logical extension of the Cambria Fault is situated about 1 km southwesterly of the site. The Cambria Fault is a reverse fault with a maximum credible earthquake (MCE) magnitude of 6.25 (Mualchin-Caltrans). The Oceanic-West Huasna Fault is a reverse oblique fault (MCE magnitude of 7.25) situated about 6 km northeasterly of the site. The Hosgri-San Simeon Fault is off-shore from Cambria and is estimated to be about 4.3 km southwesterly of the site. The assigned MCE for this fault is 7.5. Mualchin (1992) indicates the Hosgri-San Simeon Fault is a reverse-oblique fault. However, recent mapping of the San Simeon Fault (Hanson, 1994) and the San Luis Obispo County Seismic Safely Element (1999) categorize the fault as a right-lateral strike slip fault. Table 4.2-1 provides data on several local and regional faults.

The Oceanic-West Huasna Fault is suspected to be the source of the December 22, 2003 magnitude 6.5 San Simeon earthquake. This recent seismic event essentially produced a unidirectional (NW-SE) attenuation of significant motion. Measured ground acceleration transverse to the fault in San Simeon (6 to 7 miles from epicenter) was

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0.18g, while parallel to the fault in Templeton (about 29 miles from epicenter) the ground acceleration was 0.48g. This unusual ground motion attenuation is likely the result of the more competent geologic conditions longitudinal to the Coast Ranges faults.

Using Mualchin (1992), the estimated peak bedrock acceleration (PBA) at the site for the Hosgri-San Simeon Fault (based on a strike-slip mechanism) is 0.59g. To account for the faulting mechanism, an attenuation model by Sadigh (1997) was used to estimate a site PBA of 0.56g for the Oceanic-West Huasna Fault and 0.73g for the Cambria Fault. While the PBA is greater for the Cambria Fault, the difference in the MCE magnitudes results in a higher spectral acceleration over all periods for the Hosgri-San Simeon and Oceanic-West Huasna faults. Consequently, these faults are considered to be the governing design events for the Main Street Bridge.

4.3. SEISMIC DESIGN

The Hosgri-San Simeon Fault and Oceanic-West Huasna Fault would result in similar seismic response. The governing peak bedrock acceleration would be 0.6g. The soil profile is D. The ARS curve shown in Figure B.8 (Soil Profile D, M = 7.25 $\pm$ 0.25) of the Caltrans Seismic Design Criteria, Version 1.4, June 2006 is recommended for use. Due to the proximity of the governing design faults (directivity effects), an increase to the standard Caltrans ARS spectral accelerations will be necessary.

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TABLE 4.2-1
FAULT PARAMETERS

Fault Name	Site Distance (km)	Type of Displace-ment (1)	Slip Rate (mm/yr)	Maximum Magnitude (2)	Maximum Magnitude (3)	Peak Bedrock Acceleration Mean (4)
Cambria	1	RE	--	6 ¼	--	0.73
Hosgri-San Simeon	4.3	ST	2.5	7 ½	7.3	0.54
Oceanic-West Huasna	6	RO	--	7 ¼	--	0.56
Los Osos	23	RE	0.5	6 ¾	6.8	0.19
Rinconada (Jolon/San Marcos)	28	ST	1	7 ½	7.3	0.20
Point San Luis**	34	--	--	--	--	--
Santa Lucia**	34	--	--	--	--	--
Black Mountain**	35	--	--	--	--	--
Pecho*	45	--	--	--	--	--
Santa Lucia Bank***	47	--	--	--	--	--
Wilmar Ave.-S.L. Bay-Olson****	47	--	--	--	--	--
La Panza**	53	--	--	--	--	--
San Andreas (Parkfield)	66	ST	34	7 ½	7.8	0.19
San Juan	72	ST	1	7.0	7.0	0.05

Notes:

(1) ST-strike slip, RE-reverse including thrust, NO-normal oblique, RO-reverse oblique, XX-not known
(2) Mualchin (1996)
(3) California Division of Mines and Geology (CDMG, currently CGS-California Geological Survey), Peterson et, al. (1996)
(4) Mualchin & Jones (1992), modified by Sadigh (1997) for faulting mechanism

*Potentially active, entirely offshore

**Based on County of San Luis Obispo Local Hazard Mitigation Plan, there are blind thrust faults beneath the Central California Coast that could produce earthquakes with magnitudes in the range of magnitude 5.0 to 7.5. These faults have the potential to produce strong ground motion without surface fault rupture.

***Mapped as deformed marine platform. Studies indicate portions of the Santa Lucia Bank fault zone to be active.

****Wilmar Avenue Fault should be considered as potentially active. San Luis Bay fault is considered potentially active. Olson is considered inactive.

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4.4. LIQUEFACTION

The Youd et. al. (2001) method was utilized to estimate the potential for liquefaction at the site. Earthquake induced settlements at the boring locations have been estimated using the method proposed by Tokimatsu and Seed (1987). Attenuation relations by Sadigh (1997) determined a peak ground acceleration (PGA) of 0.48g, associated with a magnitude 7.5 earthquake. This PGA was used in the analyses. Analyses assumed a groundwater elevation of +46 feet.

The results of the liquefaction analyses for the average site conditions indicated that the subsurface soils are not likely to liquefy during a major seismic event. If analysis considers individual penetration rates, isolated liquefaction might occur just below the groundwater (elevation 46 to 41) at Boring 1. The maximum seismically induced settlement in this layer would be about 1 inch. Should this isolated settlement occur, the abutment piles would still have adequate bearing.

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5. DESIGN RECOMMENDATIONS

5.1. APPROACH EMBANKMENTS

Existing embankments along the proposed replacement bridge alignment will require significant amount of fill. Thickness of fills will be up to about 15 feet at Abutment 1 and up to about 19 feet at Abutment 2. Planned embankments will have slopes of 1½:1. Based on the relative consistency of the encountered alluvial deposits, it is estimated that the planned approach fill at Abutment 1 and Abutment 2 will result in less than 0.5 inch of settlement. Given the potential magnitude of settlement, no monitoring or construction delay is considered necessary.

5.2. BRIDGE FOUNDATIONS

5.2.1. Type Selection

Based on the approach fill thickness and design PBA, use of conventional shallow spread footings are inconsistent with Caltrans criteria and is not recommended. Sand and gravel layers in combination with relatively shallow groundwater would necessitate the use of casing or slurry to facilitate drilled piles. Consequently, driven piles were considered to be the most appropriate means of structure support.

This report presents preliminary data for pile evaluation of Caltrans Class 70 and Class 100 piles (12-inch and 14-inch reinforced pre-cast concrete), and HP 12x53 and 14x73 steel piles. Random, laterally discontinuous layers of relatively dense material (particularly at Abutment 1) may cause driving difficulty and inconsistency with the Class 70 and Class 100 piles. If structure performance can be satisfied, consideration should be given to the use of minimum displacement H-piles.

5.2.2. Axial Capacity

Table 5.2-1 provides the specified tip elevations for the piles. These tip elevations are the design tip elevations associated with compression design loads (Service Load) of

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140 kips/pile and 200 kips/pile. Design tip elevations associated with lateral loading are also provided in Table 5.2-1. It is understood no tension loads are anticipated on the piles.

Analysis for the axial capacity utilized an empirical method originally developed by Caltrans and used successfully for design of pile foundations for most San Luis Obispo County bridges. Piles should be spaced at a minimum distance of 3 pile widths, center to center.

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TABLE 5.2-1
PILE DATA TABLE

Location	Station (SL Line)	Pile Type	Footing Elevation (ft)	Design Load (kips)	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
					Compression	Tension		
Abutment 1	16+50	HP 12x53	56.0	140	280	0	+3 (1), 13 (3)	+3
		Class 70	56.0	140	280	0	+25 (1), 35 (3)	+25
		HP 14x73	56.0	200	400	0	-2 (1), 7 (3)	-2
		Class 100	56.0	200	400	0	+15 (1), 24 (3)	+15
Abutment 2	18+10	HP 12x53	56.0	140	280	0	-7 (1), 13 (3)	-7
		Class 70	56.0	140	280	0	+15 (1), 25 (3)	+15
		HP 14x73	56.0	200	400	0	-16 (1), 7 (3)	-16
		Class 100	56.0	200	400	0	+5 (1), 30 (3)	+5

Design Tip Elevation is controlled by the following demands

(1) Compression

(2) Tension

(3) Lateral

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5.2.3. Foundation Construction Considerations

It is recommended that project special provisions require pile-driving equipment capable of providing a minimum energy of 28,000 ft-lbs per blow. All piles should have design bearing based on Section 49-1.08 of the Caltrans Standard Specifications, May 2006 (CSS) at final tip elevation. It is anticipated that variations in resistance during pile driving may be encountered. The RE Pending File should note that pile driving could be terminated within 5 feet of specified tip on any bridge pile that has achieved twice design bearing in accordance with CSS Section 49-1.08.

5.2.4. Lateral Capacity

The lateral response of pile foundations can be evaluated using LPILE Plus Version 4.0 for Windows (computer software developed by *Ensoft Inc.*). The geotechnical parameters summarized in Tables 5.2-2 and 5.2-3 can be used for evaluation of lateral loading of piles at Abutment 1 and Abutment 2, respectively.

TABLE 5.2-2

SOIL INPUT PARAMETERS FOR LPILE AT ABUTMENT 1

Elevation (ft)		p-y	K	γ'	ϕ	c
From	To	Curve	(pci)	(pcf)	(°)	(psf)
74	56	Sand	140	118.2	35	0
56	46	Sand	140	102.7	35	0
46	41	Sand	140	47.6	35	0
41	26	Silt	430	79.0	33	500
26	21	Sand	140	65.0	35	0
21	1	Silt	430	67.3	33	500
1	-17	Sand	140	66.1	35	0

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TABLE 5.2-3

SOIL INPUT PARAMETERS FOR LPILE AT ABUTMENT 2

Elevation (m)		p-y Curve	K (pci)	γ' (psf)	ϕ (°)	C (psf)
From	To					
74	56	Sand	140	115.7	35	0
56	46	Sand	140	93.9	35	0
46	36	Sand	140	52.2	35	0
36	31	Silt	430	64.9	33	500
31	16	Silt	550	61.6	23	1500
16	10	Silt	430	64.9	33	500
10	-7	Silt	550	63.9	23	1500
-7	-16	Sand	140	66.0	35	0

It should be noted that LPILE denotes the P-Y curve, which considers both ϕ and c as "silt." In metric units, LPILE uses a density in mass units. Densities in the attached LOTB are force units.

Figures 5.2-1 through 5.2-4 present the anticipated deflection of the pile head in response to a lateral load applied at the top of the pile. Data are presented for both free and fixed head conditions for the piles presented in Table 5.2-1.

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Figure 5.2-1

Lateral Load Deflection Curves HP 12x53

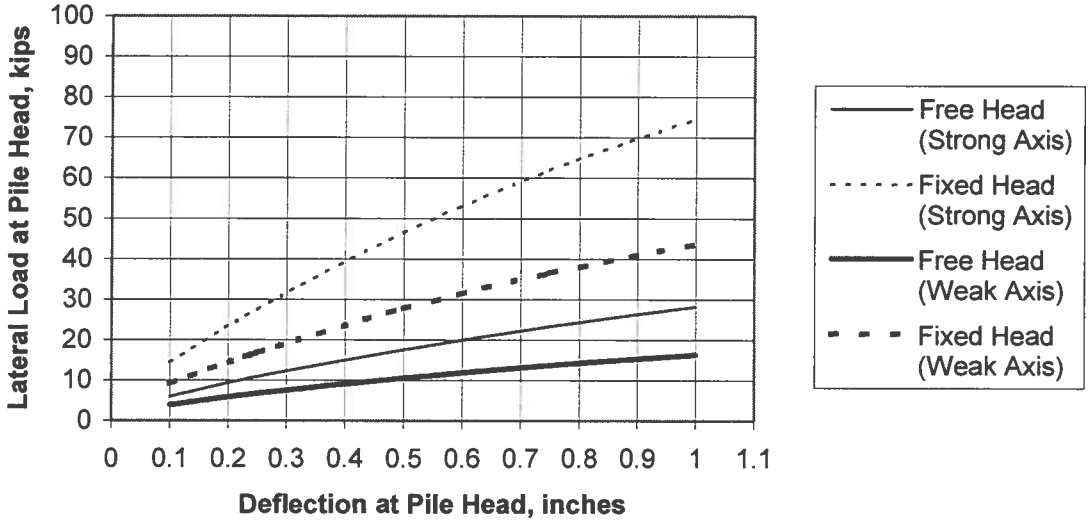

Figure 5.2-2

Lateral Load Deflection Curves HP 14x73

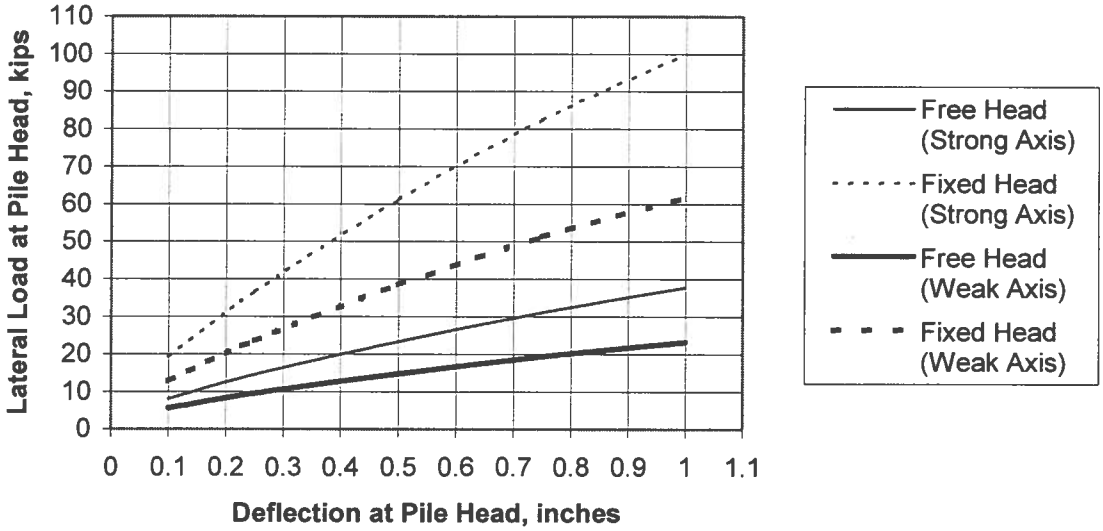

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Figure 5.2-3

Lateral Load Deflection Curves, Class 70 Pile

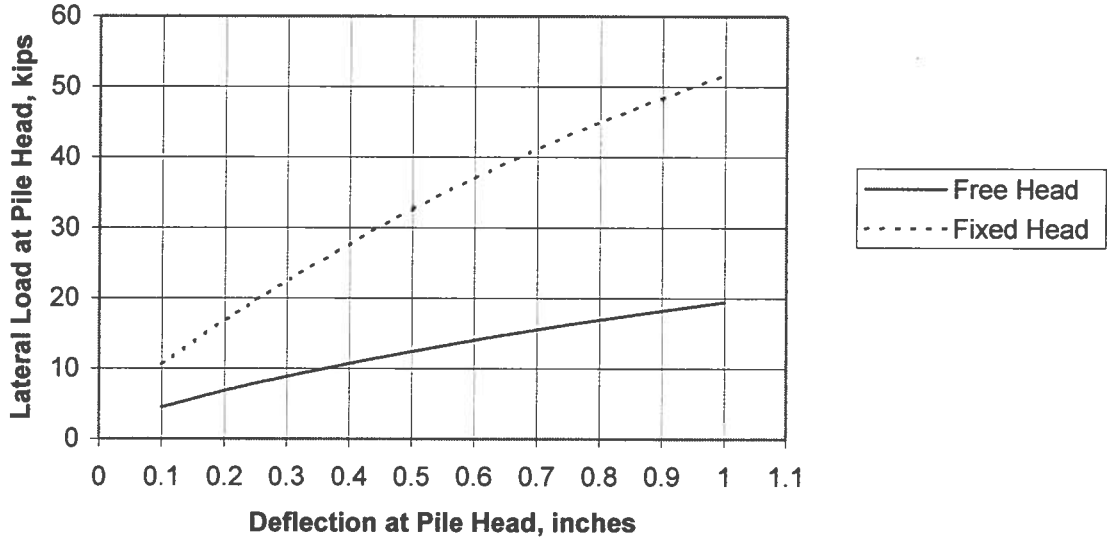
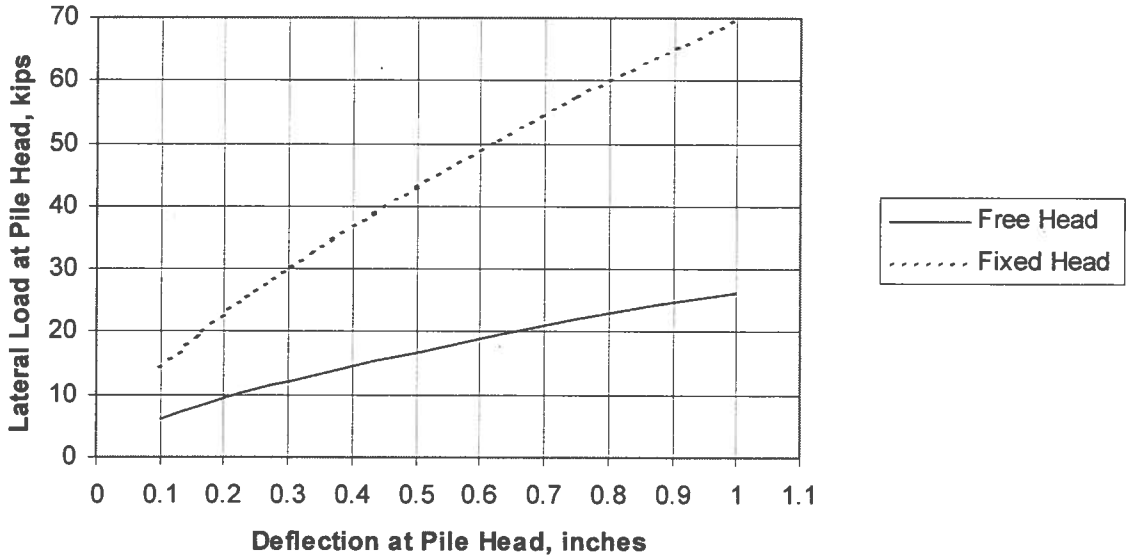


Figure 5.2-4

Lateral Load Deflection Curves, Class 100 Pile



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Curves displaying the moment distribution along the pile for free head and fixed head conditions are presented in Appendix C (Plates C-1 through C-24). The moment distribution data are for pile head deflections of 0.1, 0.25, 0.5, 0.75, and 1.0 inch.

When considering the lateral capacity of a pile group, it will be necessary to reduce the single pile capacity of trailing piles. The reduction in capacity due to the effects of shaft interaction will be dependent upon the center-to-center (CTC) pile spacing. It is recommended that the capacity of individual trailing piles in a laterally loaded group be reduced according to the data in Table 5.2-4.

TABLE 5.2-4

GROUP AFFECT FOR LATERALLY LOADED PILE

CTC Spacing (In-line Loading)	Ratio of Lateral Resistance of Trailing Pile in Group to Isolated Single Pile
2B	0.4
3B	0.6
4B	0.8
5B	1.0

NOTE: B IS PILE WIDTH

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5.3. ABUTMENT ENDWALL DYNAMIC LATERAL RESISTANCE

Backfill at abutments, when constructed in accordance with Caltrans Standard Specifications, should be of equal (or better) quality than the Caltrans "typical" abutment endwall backfill. An initial abutment soil stiffness of 20 kips/in/foot is recommended. The ultimate lateral resistance that may be applied against abutment endwalls to resist seismic loading will be dependent on the deflection that occurs (which mobilizes shear resistance in the soil). Figure 5.3-1 presents the ultimate equivalent uniform lateral soil resistance as a function of horizontal strain (deflection/height) for the abutments. The maximum resistance for strain in excess of 1.0% is 5.0 ksf, when the height of the wall is greater than or equal to 5.5 ft. When the abutment endwall height is less than 5.5 ft, the maximum equivalent uniform lateral soil resistance shall be reduced proportionately by H/5.5, where H is the endwall height in feet.

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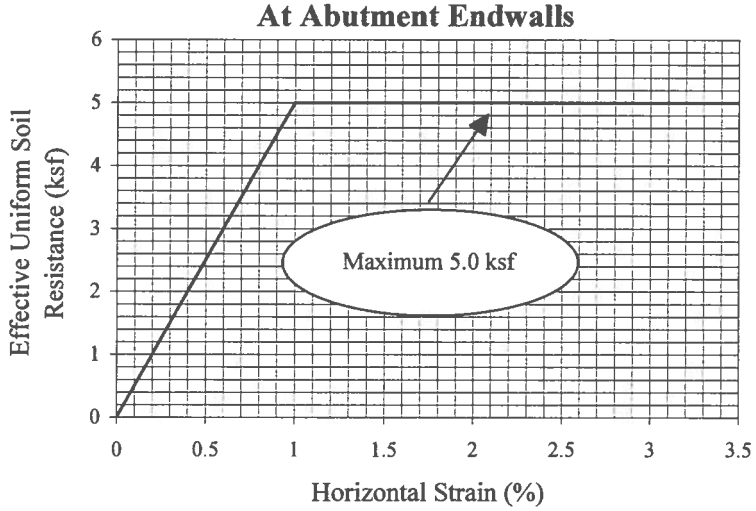
FIGURE 5.3-1
Allowable Lateral Bearing for Seismic Loading At Abutment Endwalls



5.4. SEISMIC DESIGN CRITERIA

According to Table B.1 of Caltrans Seismic Design Criteria (SDC) Version 1.4 (June 2006), the site can be classified as Soil Profile Type D. The peak bedrock acceleration (PBA) at the site is about 0.59g. According to the guidelines presented in Section 6.1.2.1 of the SDC and Section 3.7 of Caltrans Guidelines for Foundation Investigation and Reports, the ARS curve should be selected by rounding up the PBA value. For sites within 15 km of the fault, the spectral acceleration should be increased by 20% for periods above 1.0 second and, by linear interpolation, increased 0% to 20% for periods between 0.5 and 1.0 second. Consequently, design should use a modified 0.6g ARS curve from Figure B.8 (Soil Profile D, M=7.25±0.25) of the Caltrans Seismic Design Criteria, Version 1.4. The design ARS curve modified for proximity, is provided in Appendix D.

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5.5. ROADWAY DESIGN

5.5.1. Flexible Pavement

R-values of 34 and 22 have been determined for subgrade soil on north approach of Main Street (and Santa Rosa Creek Road) and south approach of Main Street, respectively. R-values were governed by expansion considerations.

Flexible pavement sections have been determined using Caltrans design procedures. San Luis Obispo County has provided Traffic Indexes (TI) of 7.5. The recommended pavement sections resulting from the analysis are provided in Table 5.5-1.

TABLE 5.5-1
RECOMMENDED PAVEMENT SECTIONS

Location	TI	R-value	Asphalt Concrete (AC)	Aggregate Base (AB)
Main Street & Santa Rosa Creek Road (North approach)	7.5	34	0.35 feet	0.80 feet
Main Street (South approach)	7.5	22	0.35 feet	1.05 feet

AC should conform to, and be placed in accordance with, Section 39 of the Caltrans Standard Specifications, May 2006 (CSS). AB should conform to Section 26 of the CSS. AB and at least the upper 8 inches of subgrade soils should be compacted to at least 95% of maximum density.

5.5.2. Earthwork

In general, all required earthwork should be performed in accordance with applicable items of Section 19 of the Caltrans Standard Specifications, May 2006. Areas to receive fill should be stripped of vegetation and other deleterious matter. Where the stripped ground surface has a gradient steeper than 5:1 (H:V), essentially level benches should be cut into approved soil as the fill is brought up. The local bedrock nearest to

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the bridge is mapped as undifferentiated sedimentary rocks. This type of rock is an unlikely source of naturally occurring asbestos (NOA). However, rock consisting of Franciscan formation which is considered a possible source of NOA, is present in areas near the sedimentary rocks.

5.6. CORROSION POTENTIAL

Based on the soluble sulphate content (238 ppm), chloride content (53.2 ppm), and pH of 7.5, determined from a soil sample obtained from Boring B-2 at 2 to 5 feet and the apparent performance of the existing bridge, the project site environment is anticipated to be non-corrosive with respect to foundation concrete. The minimum resistivity of 1780 ohm-cm is just outside the threshold of Caltrans general guidelines for a corrosive environment.

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6. CLOSURE

The conclusions and recommendations of this report are for the preliminary design of the proposed Main Street Bridge replacement at Santa Rosa Creek in San Luis Obispo County, California, as described in the text of this report. The findings, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No warranty, expressed or implied, is made. If any changes are implemented which materially alters the project, additional geotechnical services may be required along with revisions to the recommendations given herein.

This report may be used only by the County of San Luis Obispo and T.Y. Lin International, and their designated representatives, and only for the purposes stated. Noncompliance with any of the conditions or recommendations, or misuse of this report, will release Kleinfelder from any liability.

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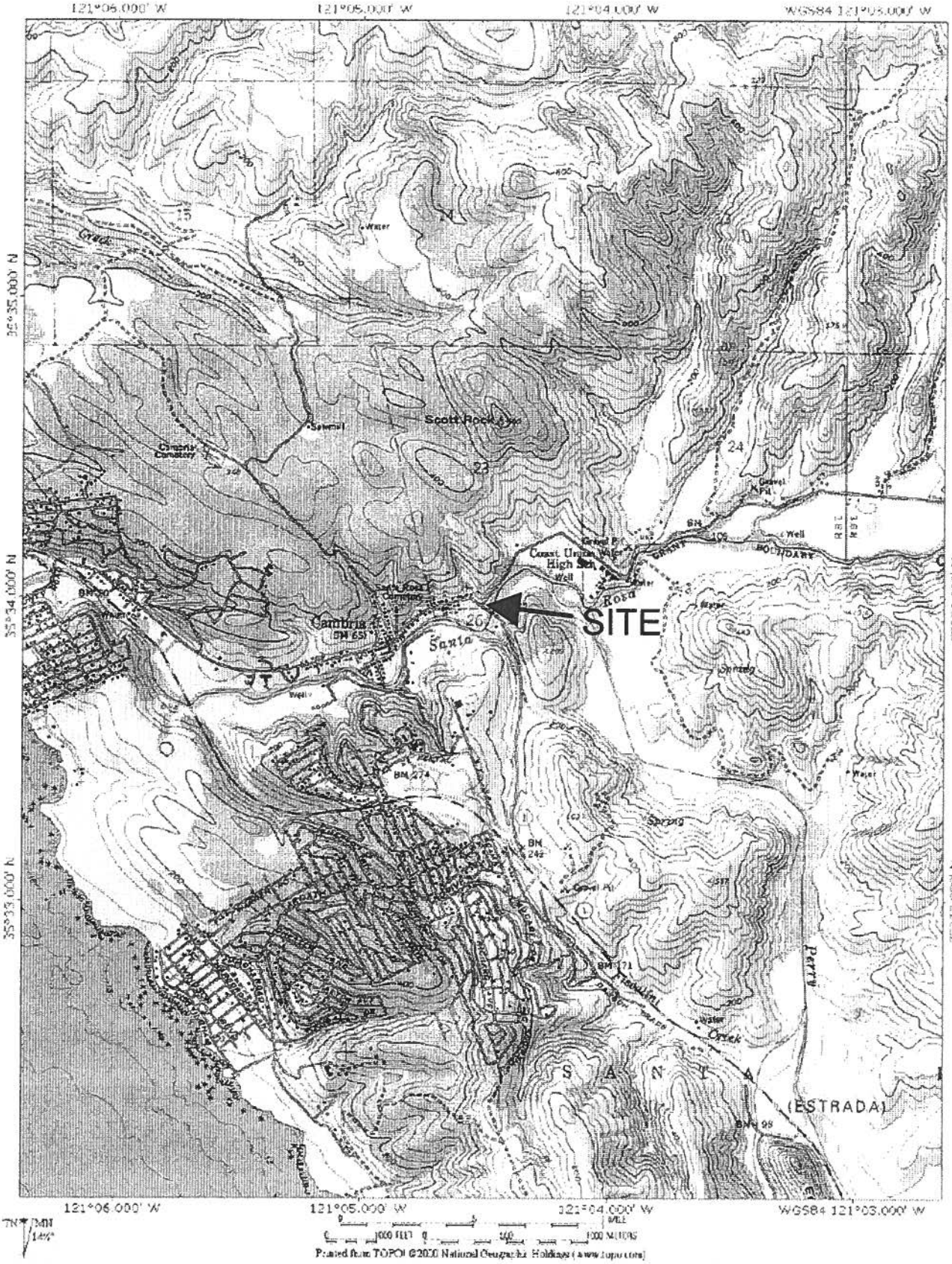
APPENDIX A



KLEINFELDER

PROJECT NO. 35352

DATE: 7-11-06

SITE VICINITY MAP
MAIN STREET BRIDGE
SANTA ROSA CREEK
SAN LUIS OBISPO COUNTY, CA

PLATE

1

DIST.	COUNTY	ROUTE	POST km TOTAL PROJECT	SHEET No.	TOTAL SHEETS
05	SLO	Co. Rd.			

REGISTERED GEOTECHNICAL ENGINEER

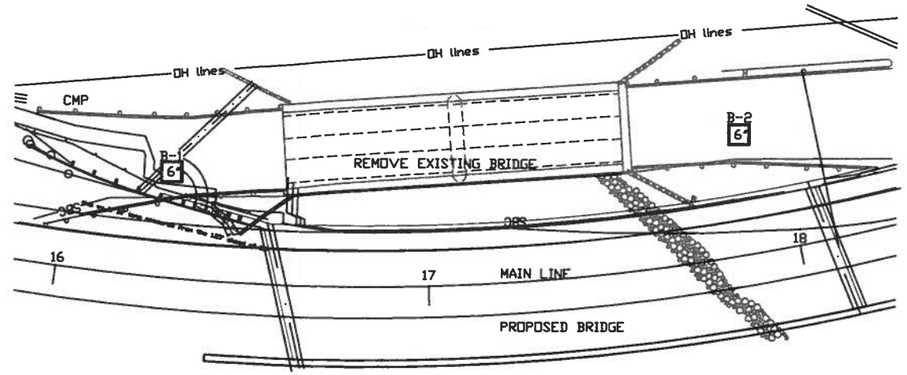
PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — David L. Pearson — No. 674 — Exp. 12/31/05 — GE — STATE OF CALIFORNIA

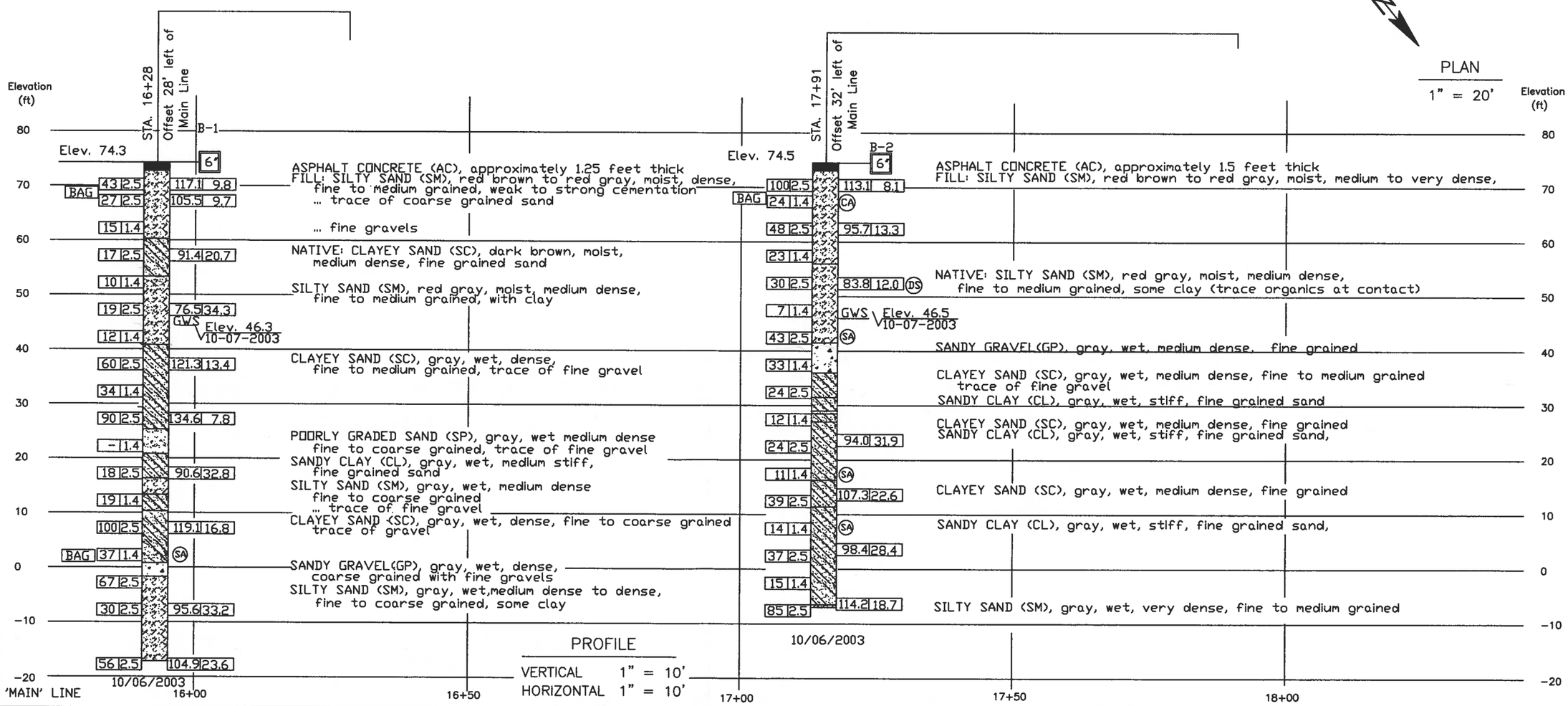
KLEINFELDER
1410 F STREET
FRESNO, CALIFORNIA 93706
(559) 486-0750

File No.: 35352.GEO

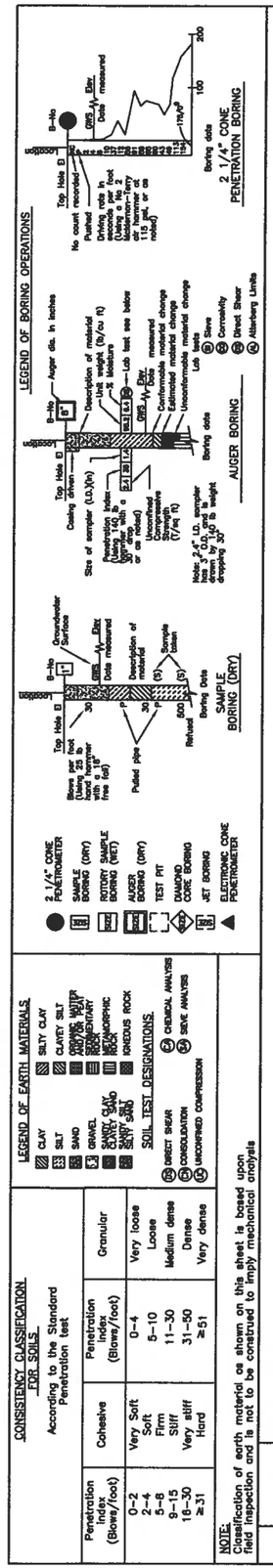
NOTES:
1. Bench mark – BM# Y693, Brass disk in N.E. wingwall of bridge Elev. 74.14 feet
2. Ground water was encountered within the depths of exploration as shown on the boring logs.
3. Hammer type – Rope & cathead with a 140 lb safety drop hammer dropping 30 inches



PLAN
1" = 20'



PROFILE
VERTICAL 1" = 10'
HORIZONTAL 1" = 10'



DESIGN OVERSIGHT			
SIGNOFF DATE			
FIELD ENGINEER :	NATHAN DAHLEN	10/2003	
DRAWN BY :	MICHAEL BELTRAN	04/2005	
CHECKED BY :	DAVID L. PEARSON	02/2006	

PREPARED FOR THE
COUNTY OF SAN LUIS OBISPO
DEPARTMENT OF PUBLIC WORKS

PROJECT ENGINEER :

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU
EA

BRIDGE No. 49C-0337
POST km

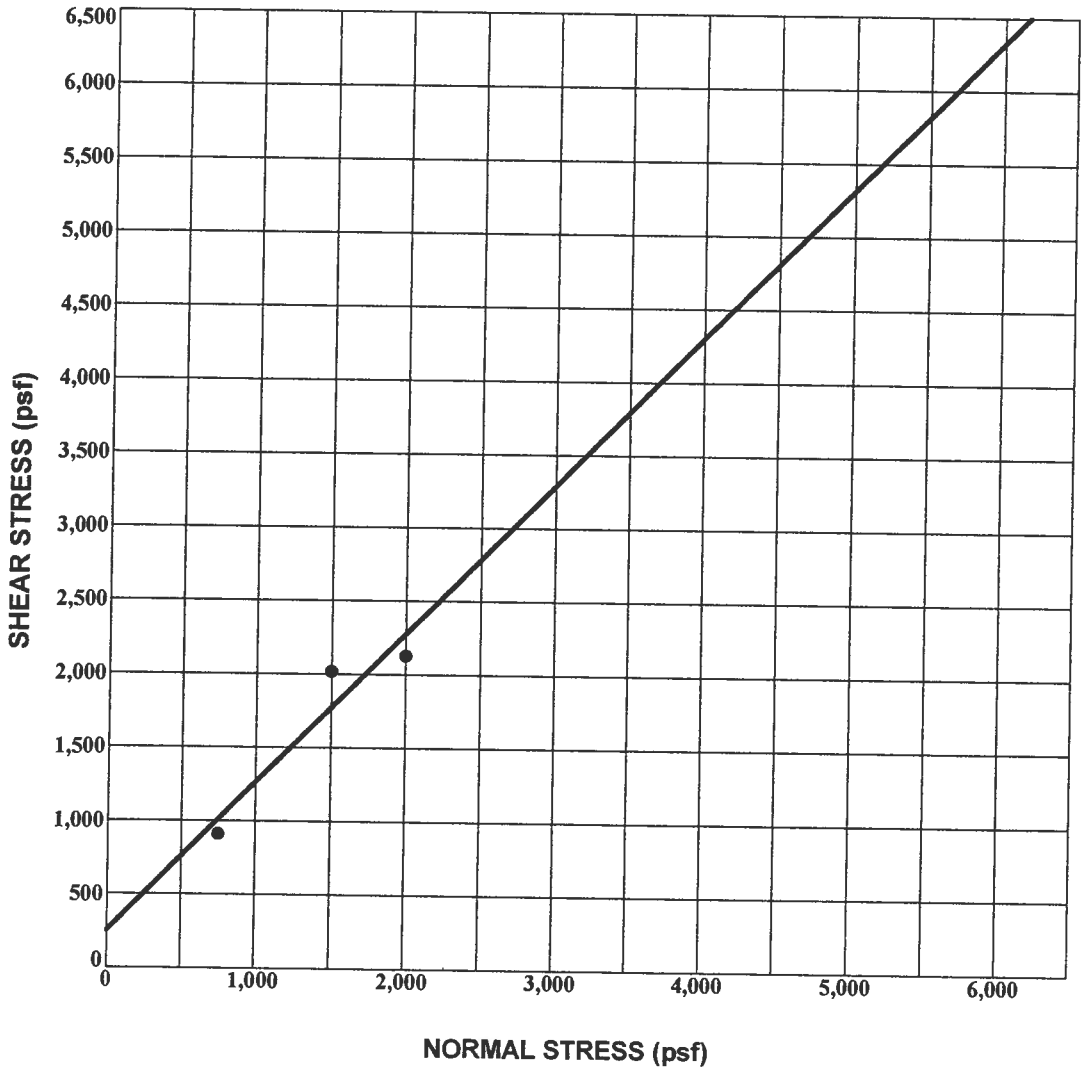
MAIN STREET BRIDGE
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET OF

APPENDIX B

DIRECT SHEAR



Source:	B-1 ; B-2	Friction Angle =	45.0 deg
Depth:	6.0 ft, 16 ft ; 11 ft	Cohesion =	250 psf
Test Type:	Consolidated - Drained		
Soil Description:	SILTY SAND (SM) ; CLAYEY SAND (SC)		

Dry Density (pcf)	100.1	96.9	91.1
Initial Water Content (%)	9.7	13.3	20.7
Final Water Content (%)	21.1	15.6	21.4
Normal Stress (psf)	750	1500	2000
Shear Stress(psf)	912	2019	2128

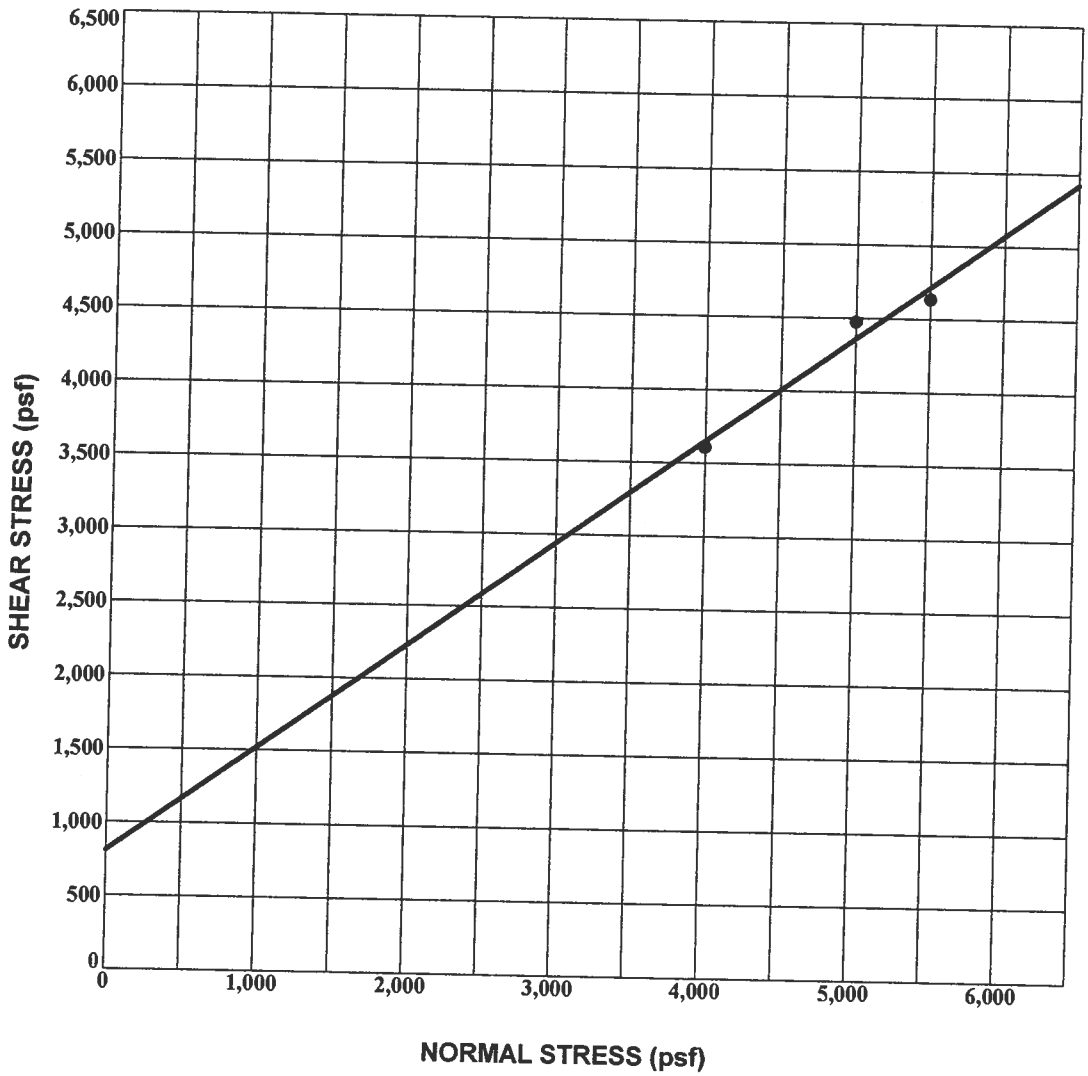
KLEINFELDER

PROJECT NO. 35352.GEO

DIRECT SHEAR TEST
MAIN STREET BRIDGE AT SANTA ROSA CREEK
MAIN STREET
CAMBRIA, CALIFORNIA

PLATE
B-1

DIRECT SHEAR



Source:	B-1	Friction Angle =	35.0 deg
Depth:	36.0 ft, 46.0 ft, 56.0 ft	Cohesion =	807 psf
Test Type:	Consolidated - Drained		
Soil Description:	CLAYEY SAND (SC), POORLY GRADED SAND (SP), SANDY CLAY (CL)		

Dry Density (pcf)	120.1	134.5	91.1
Initial Water Content (%)	13.4	7.8	22.8
Final Water Content (%)	14.7	9.9	23.1
Normal Stress (psf)	4000	5000	5500
Shear Stress(psf)	3610	4476	4637

KLEINFELDER

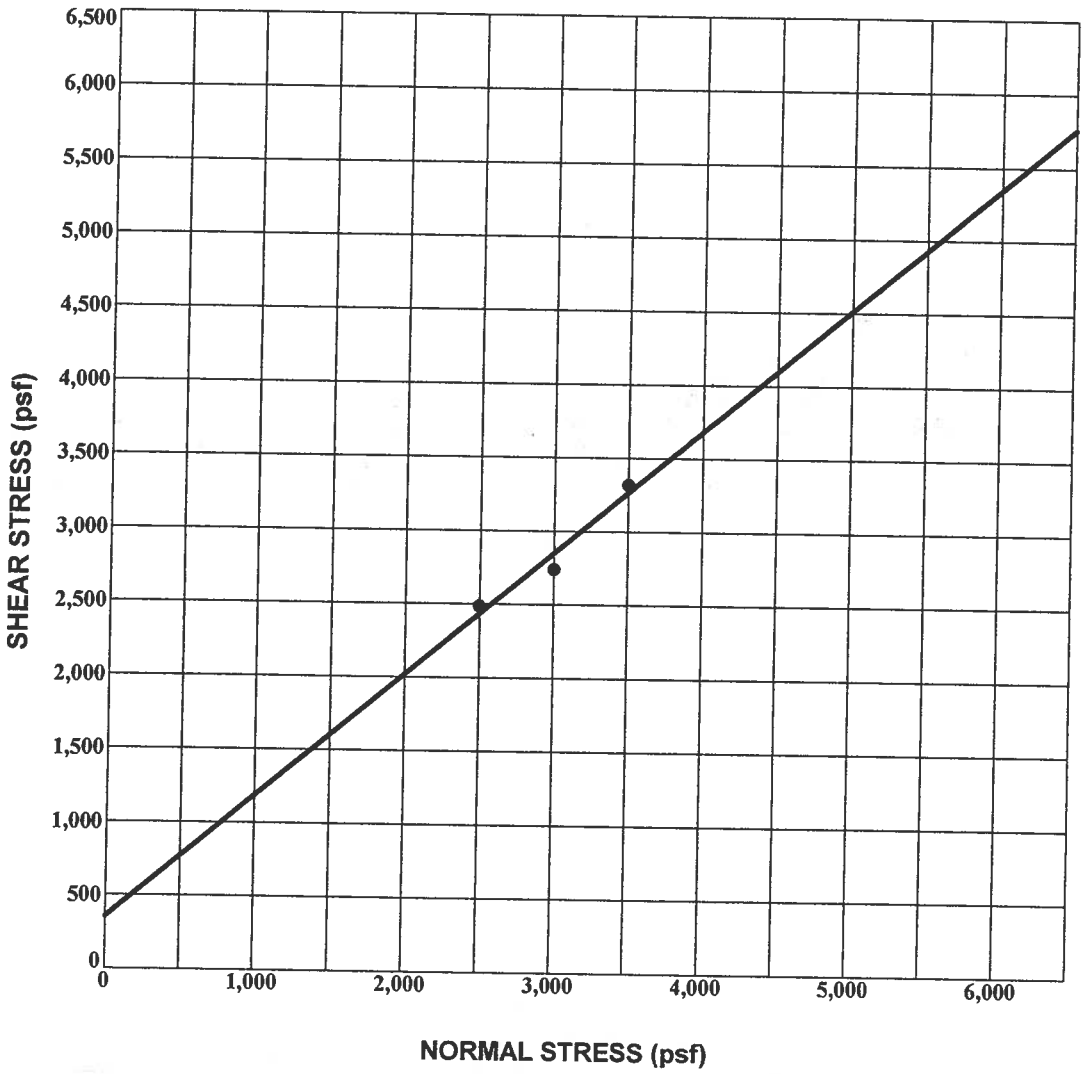
PROJECT NO. 35352.GEO

DIRECT SHEAR TEST
MAIN STREET BRIDGE AT SANTA ROSA CREEK
MAIN STREET
CAMBRIA, CALIFORNIA

PLATE

B-2

DIRECT SHEAR



Source:	B-2	Friction Angle =	40.0 deg
Depth:	21.0 ft	Cohesion =	350 psf
Test Type:	Consolidated - Drained		
Soil Description:	SILTY SAND (SM)		

Dry Density (pcf)	82.0	82.4	84.5
Initial Water Content (%)	12.0	12.0	12.0
Final Water Content (%)	12.1	11.0	11.7
Normal Stress (psf)	2500	3000	3500
Shear Stress(psf)	2487	2740	3318

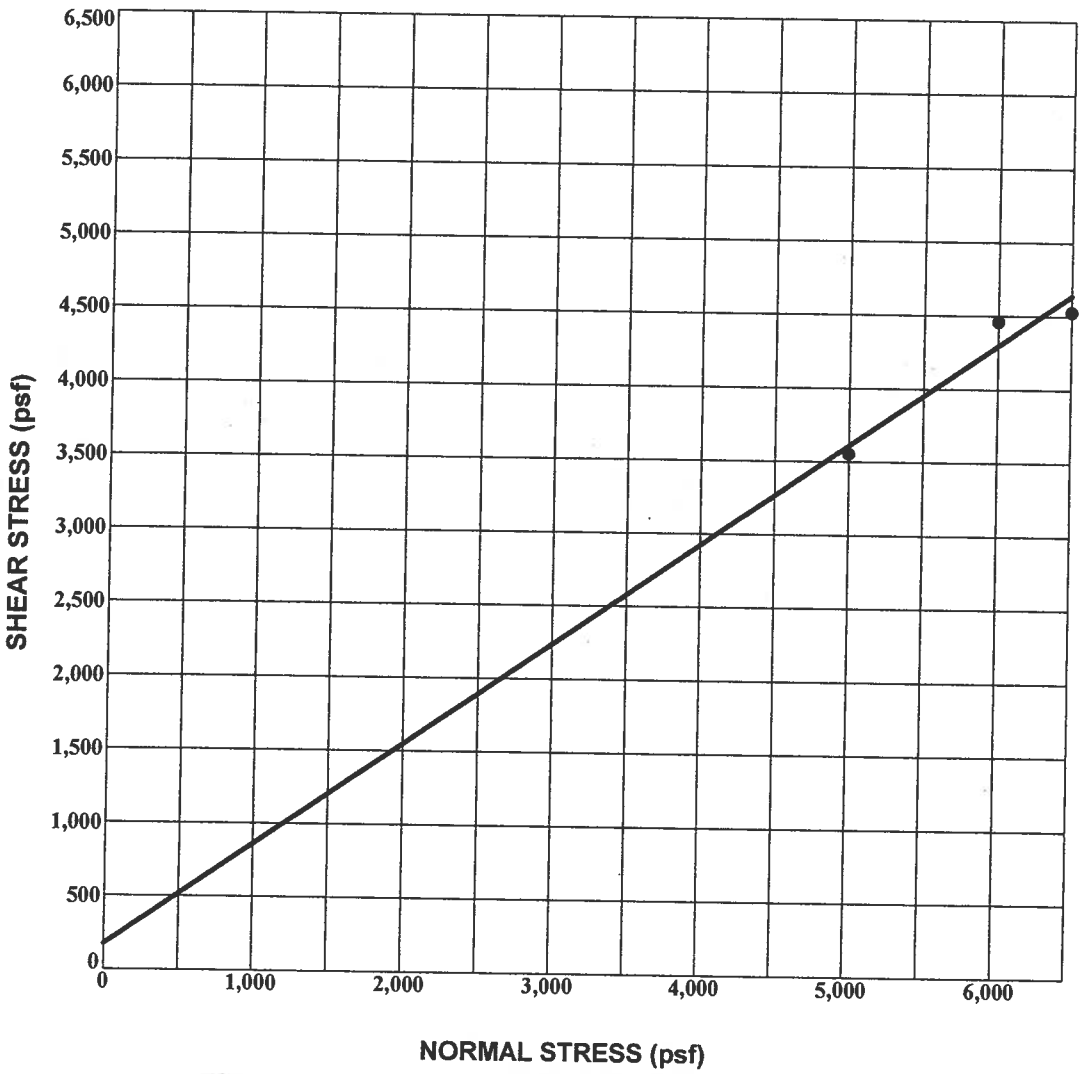
KLEINFELDER

PROJECT NO. 35352.GEO

DIRECT SHEAR TEST
MAIN STREET BRIDGE AT SANTA ROSA CREEK
MAIN STREET
CAMBRIA, CALIFORNIA

PLATE
B-3

DIRECT SHEAR



Source:	B-2	Friction Angle =	35.0 deg
Depth:	51.0 ft, 61.0 ft, 71.0 ft	Cohesion =	173 psf
Test Type:	Consolidated - Drained		
Soil Description:	SANDY CLAY (CL); SANDY CLAY (CL); SANDY CLAY (CL)		

Dry Density (pcf)	91.4	105.7	91.4
Initial Water Content (%)	29.3	22.6	28.4
Final Water Content (%)	30.5	22.0	28.8
Normal Stress (psf)	5000	6000	6500
Shear Stress(psf)	3554	4461	4530

KLEINFELDER

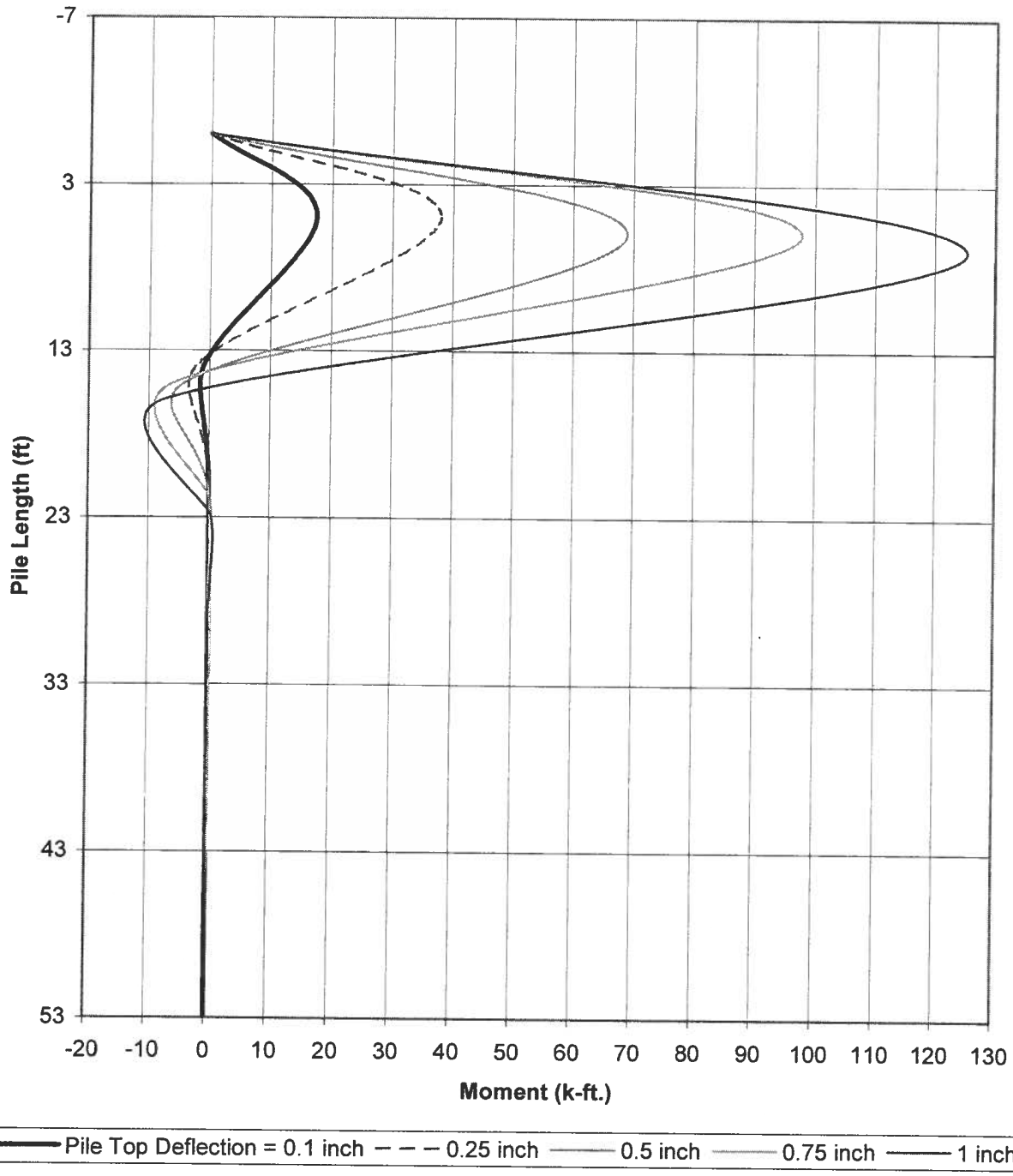
PROJECT NO. 35352.GEO

DIRECT SHEAR TEST
MAIN STREET BRIDGE AT SANTA ROSA CREEK
MAIN STREET
CAMBRIA, CALIFORNIA

PLATE
B-4

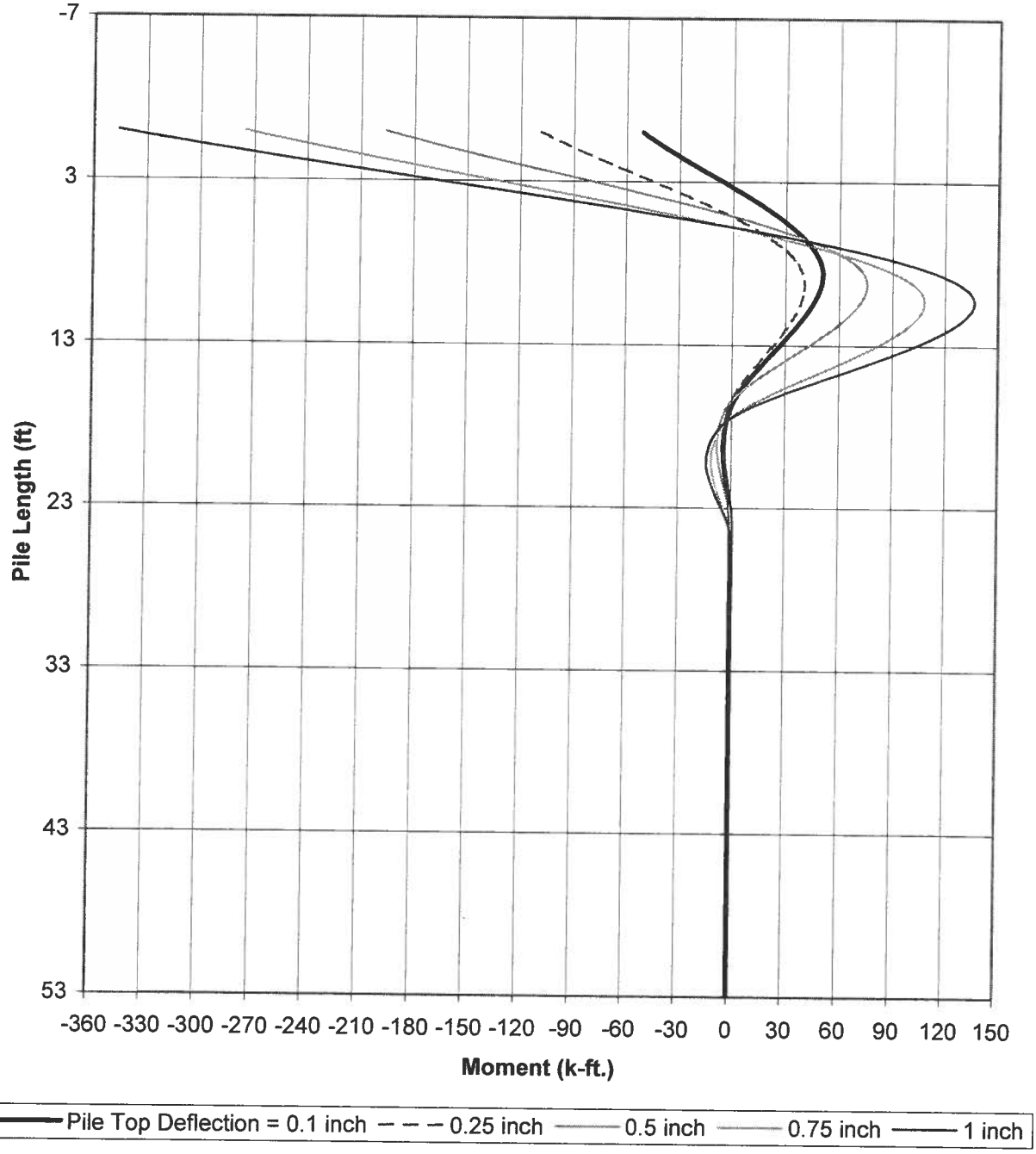
APPENDIX C

Moment Along Pile Length - HP 12x53
Abutment 1 - Free Head (Strong Axis)



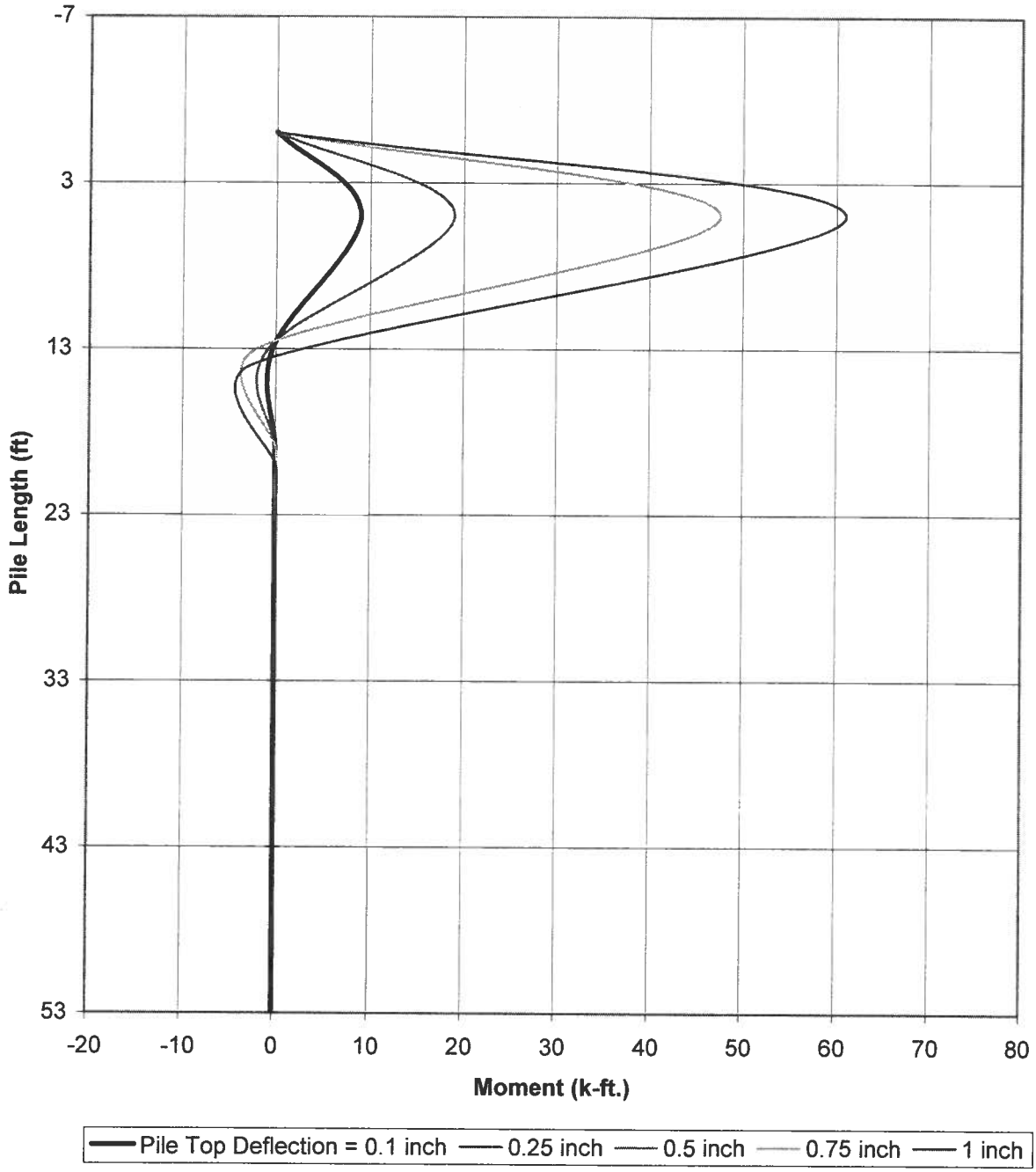
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-1**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 1 - Fixed Head (Strong Axis)



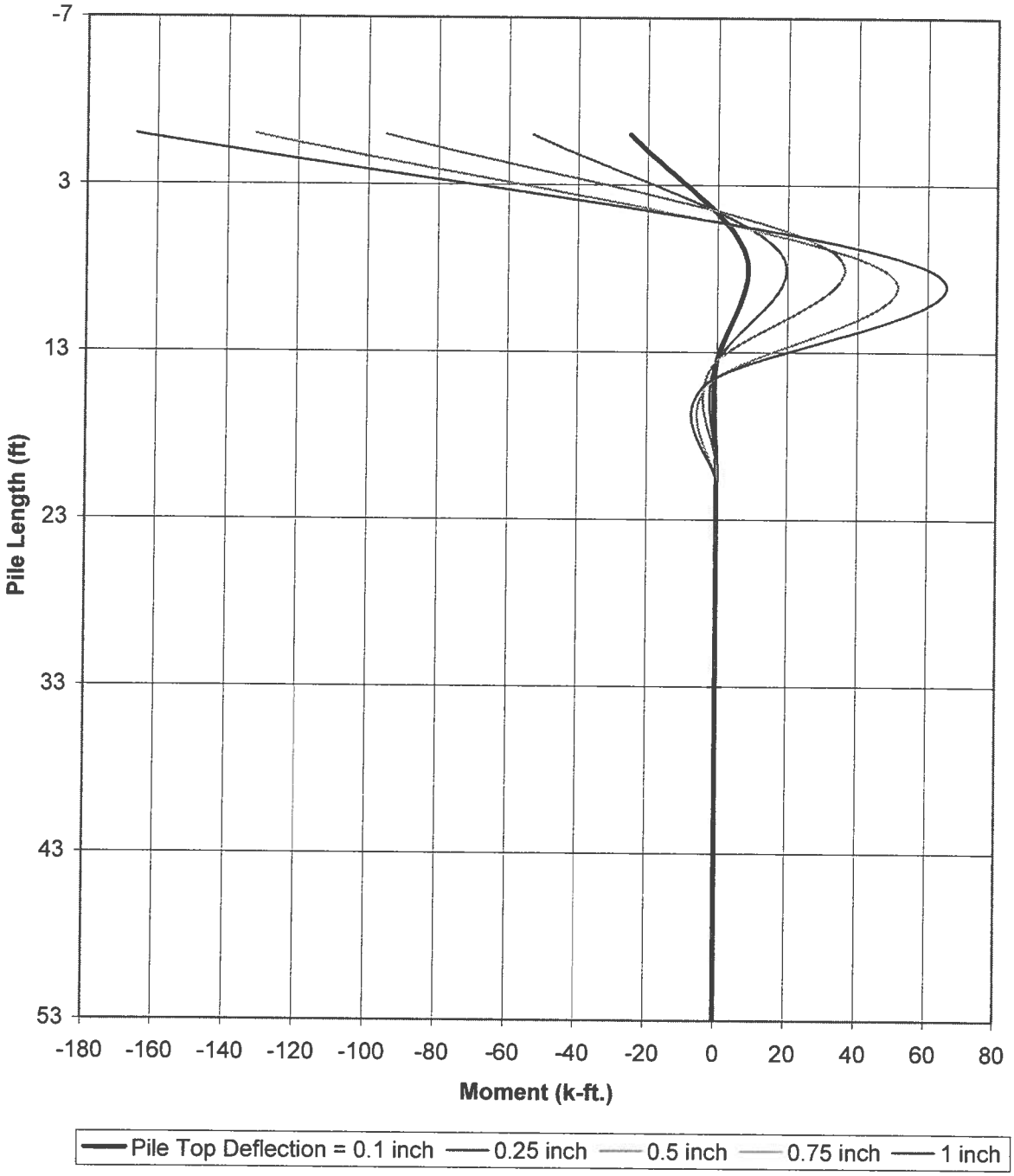
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-2**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 1 - Free Head (Weak Axis)



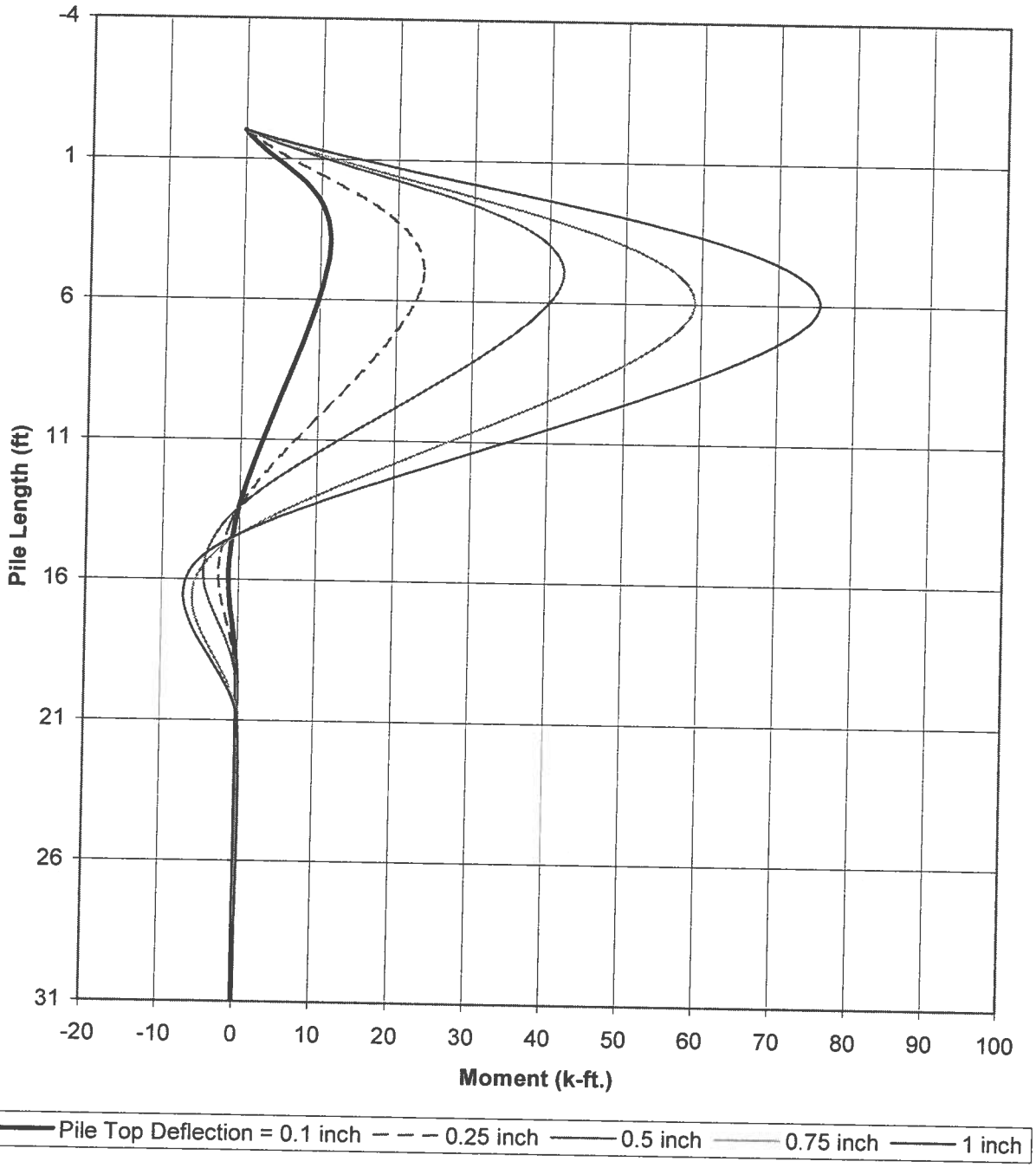
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-3**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 1 - Fixed Head (Weak Axis)



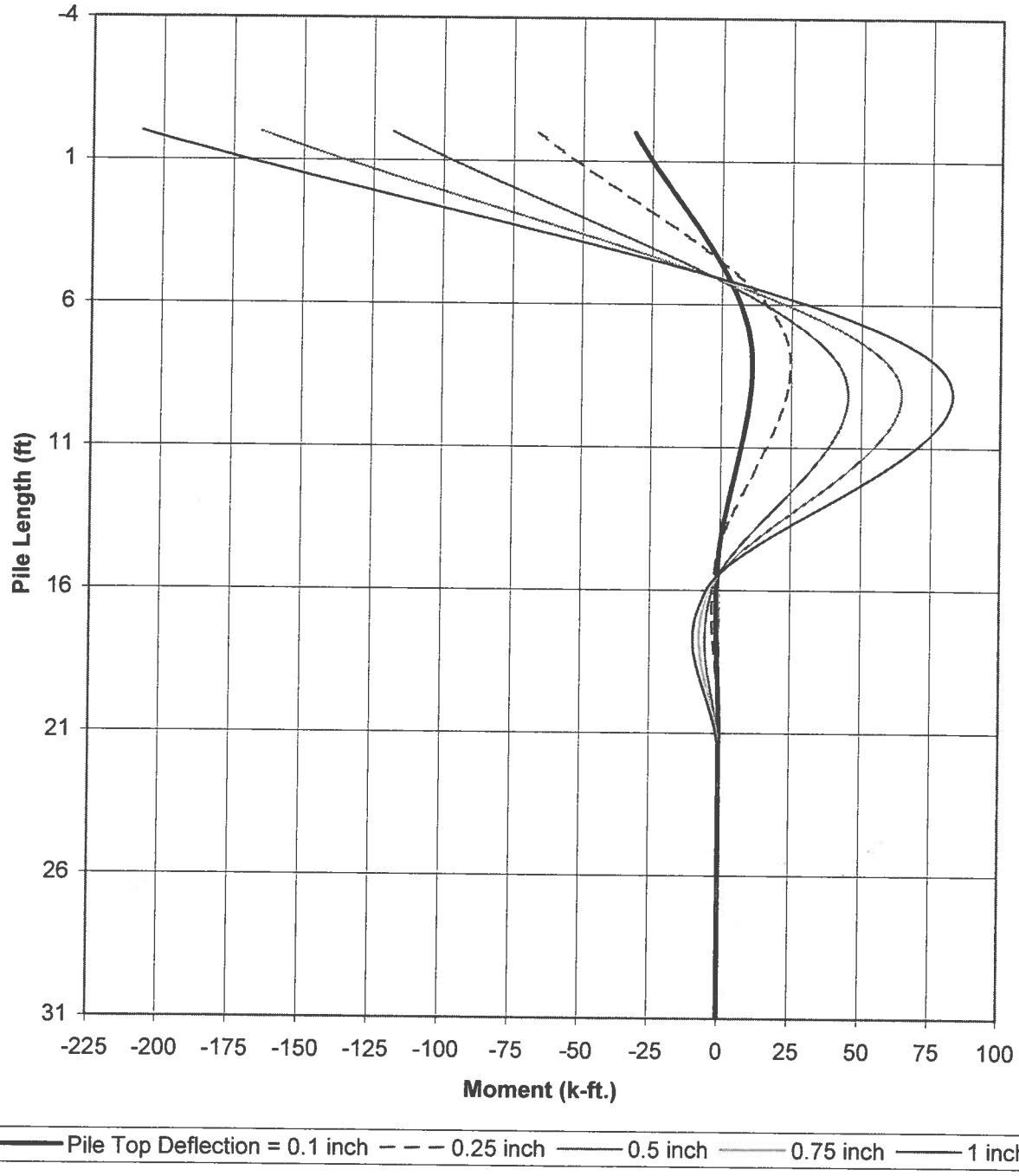
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-4**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - Class 70
Abutment 1 - Free Head



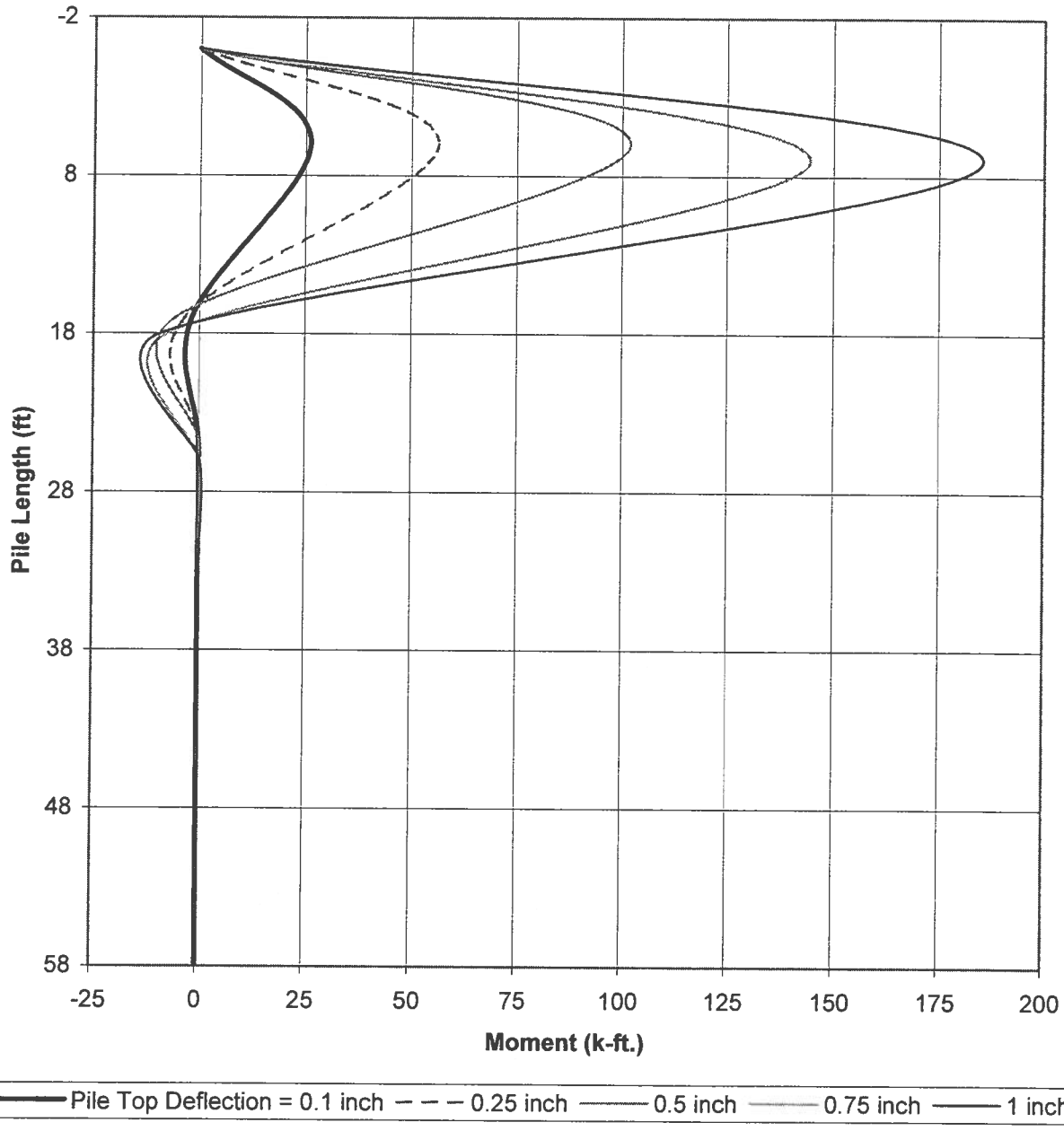
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-5**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - Class 70
Abutment 1 - Fixed Head

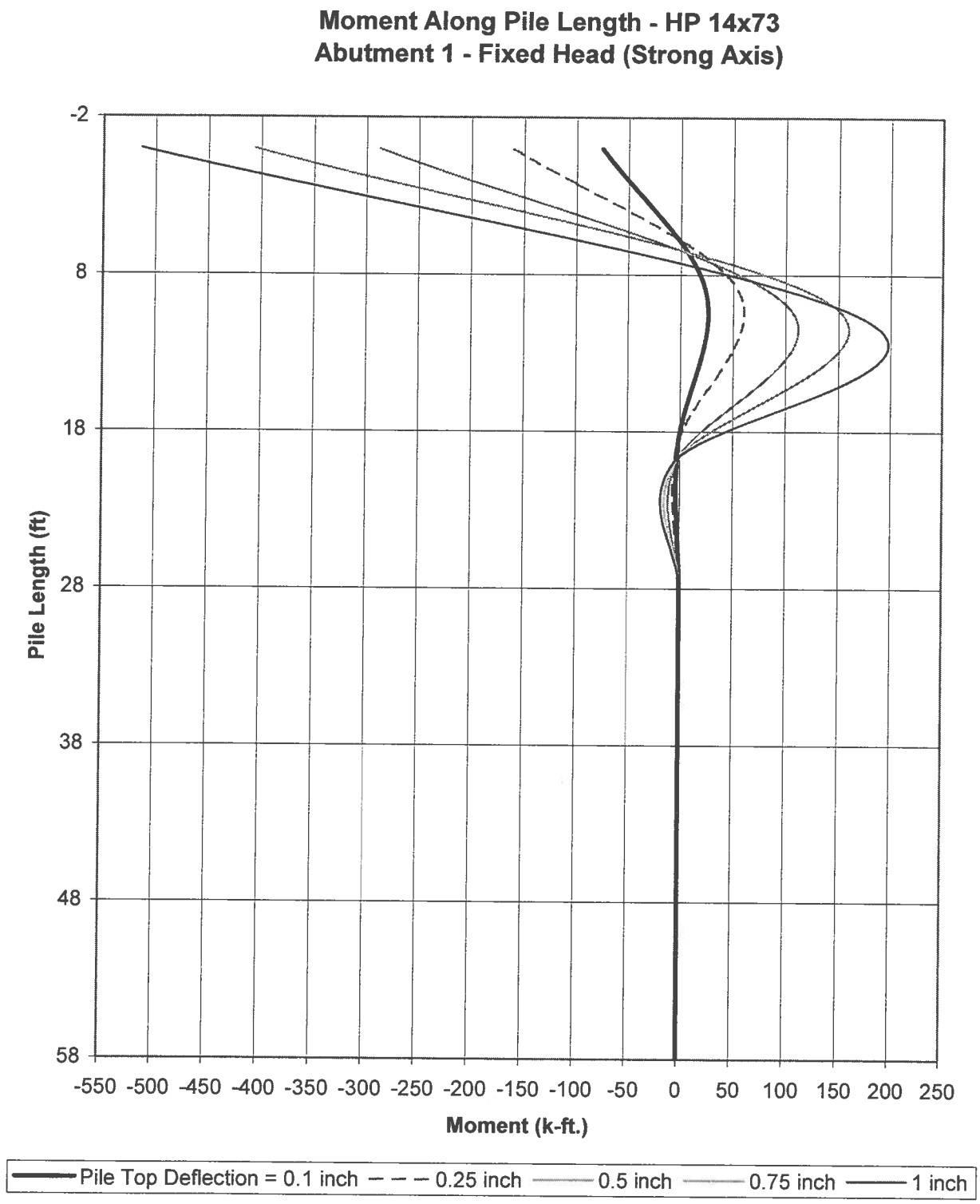


KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-6**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

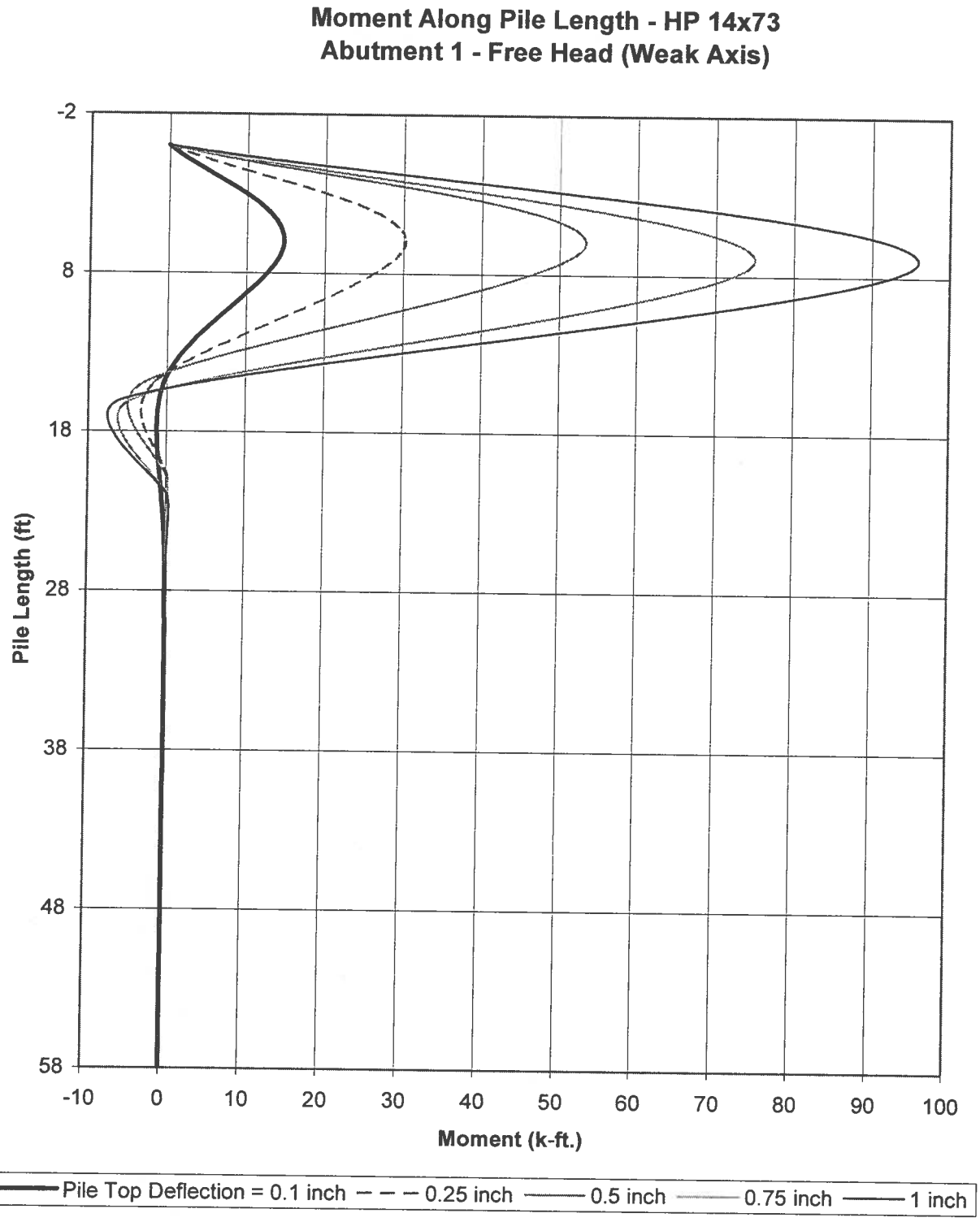
Moment Along Pile Length - HP 14x73
Abutment 1 - Free Head (Strong Axis)



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-7**
PROJECT NO. 35352.GEO | DATE: 11-01-06		



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-8**
PROJECT NO. 35352.GEO | DATE: 11-01-06		



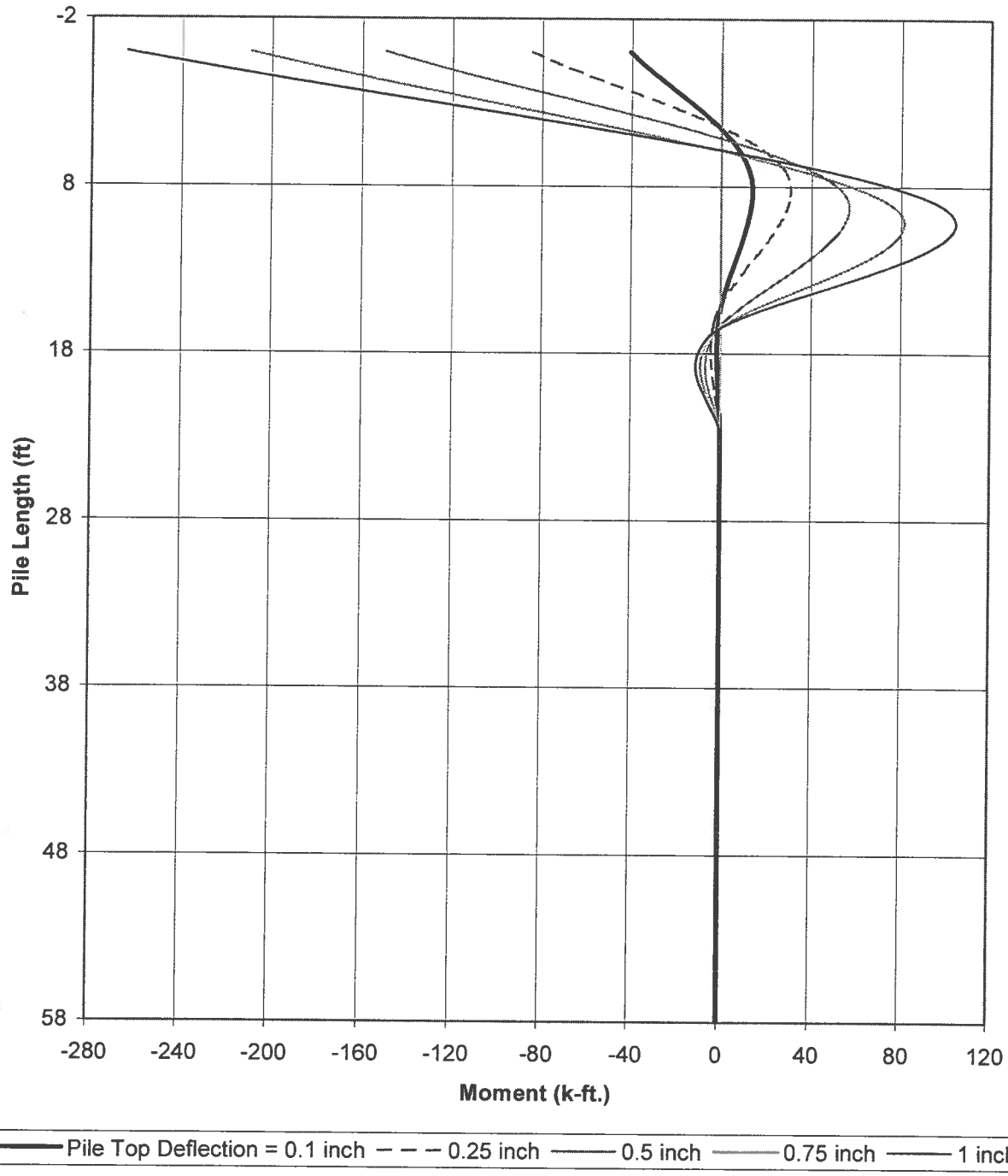
KLEINFELDER

PROJECT NO. 35352.GEO | DATE: 11-01-06

MAIN STREET BRIDGE
AT SANTA ROSA CREEK
CAMBRIA, CALIFORNIA

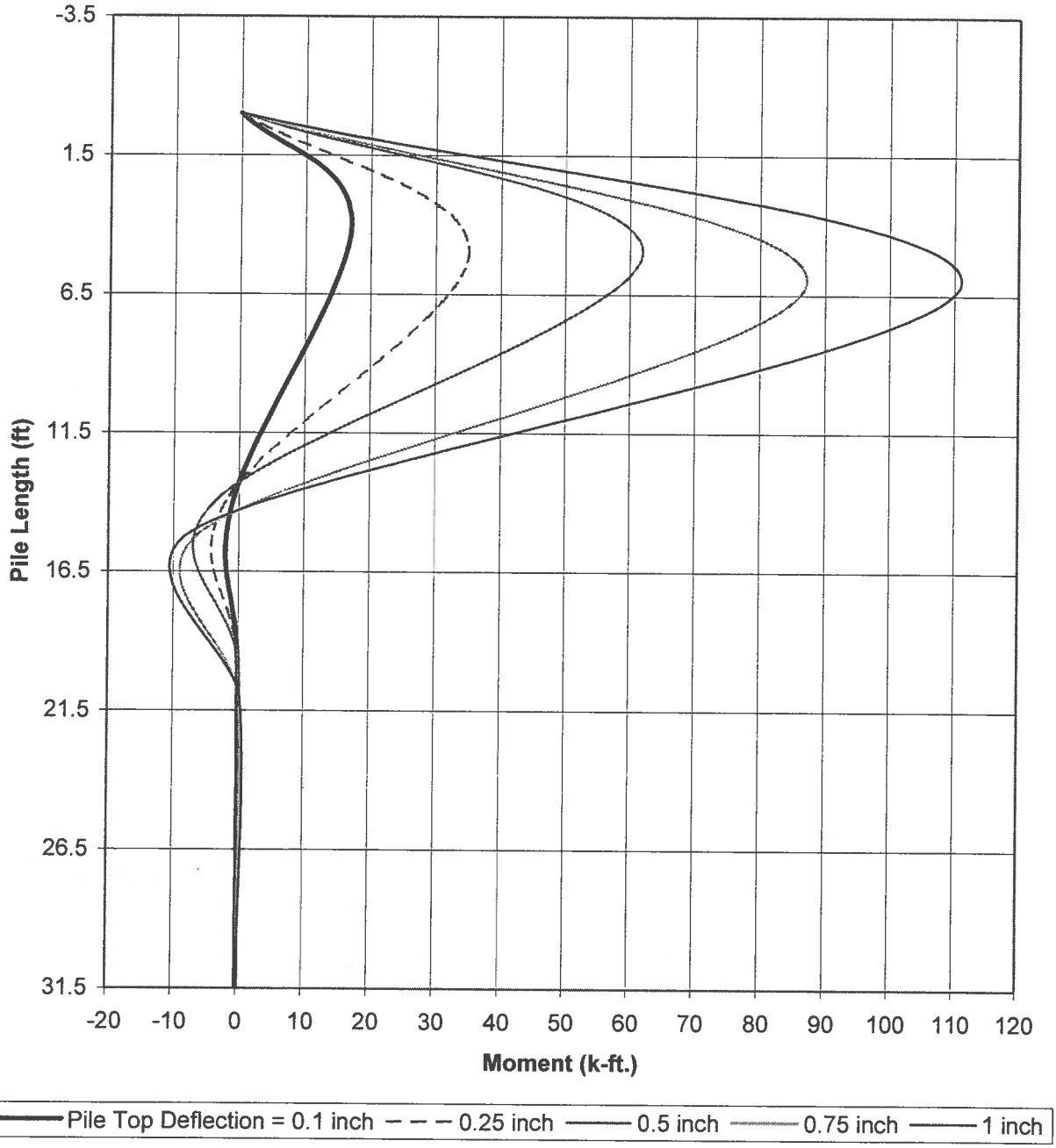
PLATE
C-9

Moment Along Pile Length - HP 14x73
Abutment 1 - Fixed Head (Weak Axis)



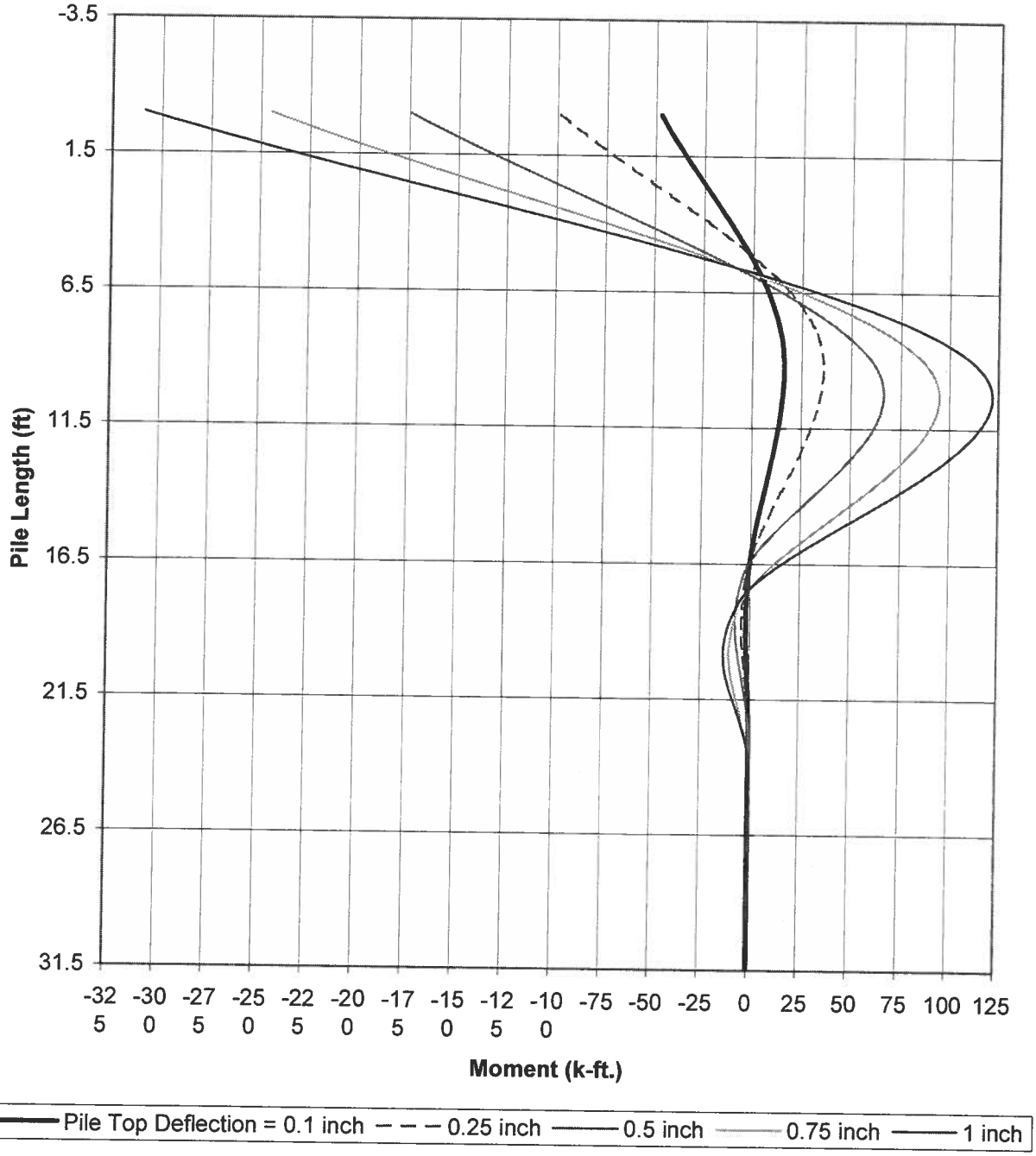
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-10**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - Class 100
Abutment 1 - Free Head



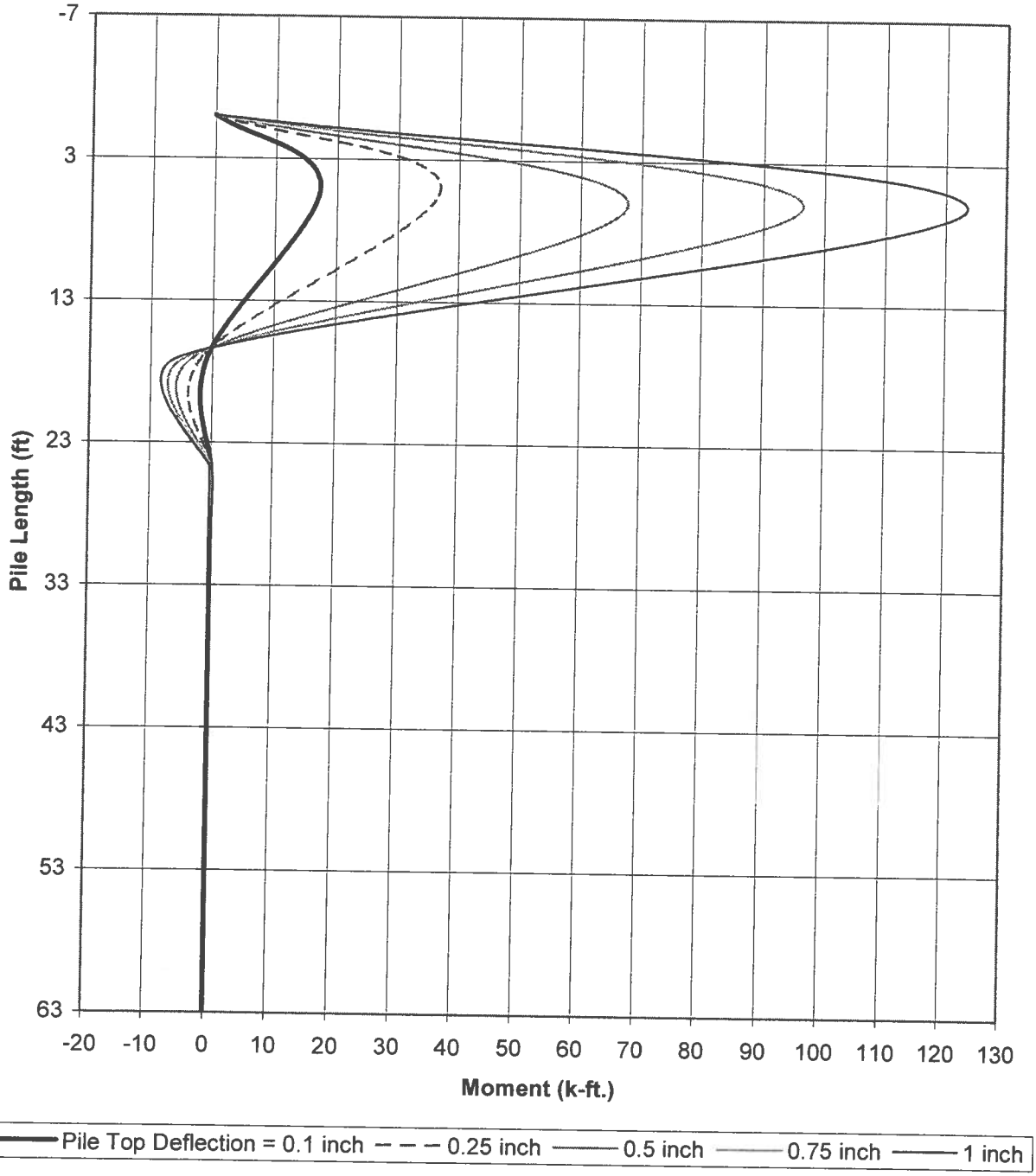
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE C-11
PROJECT NO. 35352.GEO / DATE: 11-01-06		

Moment Along Pile Length - Class 100
Abutment 1 - Fixed Head



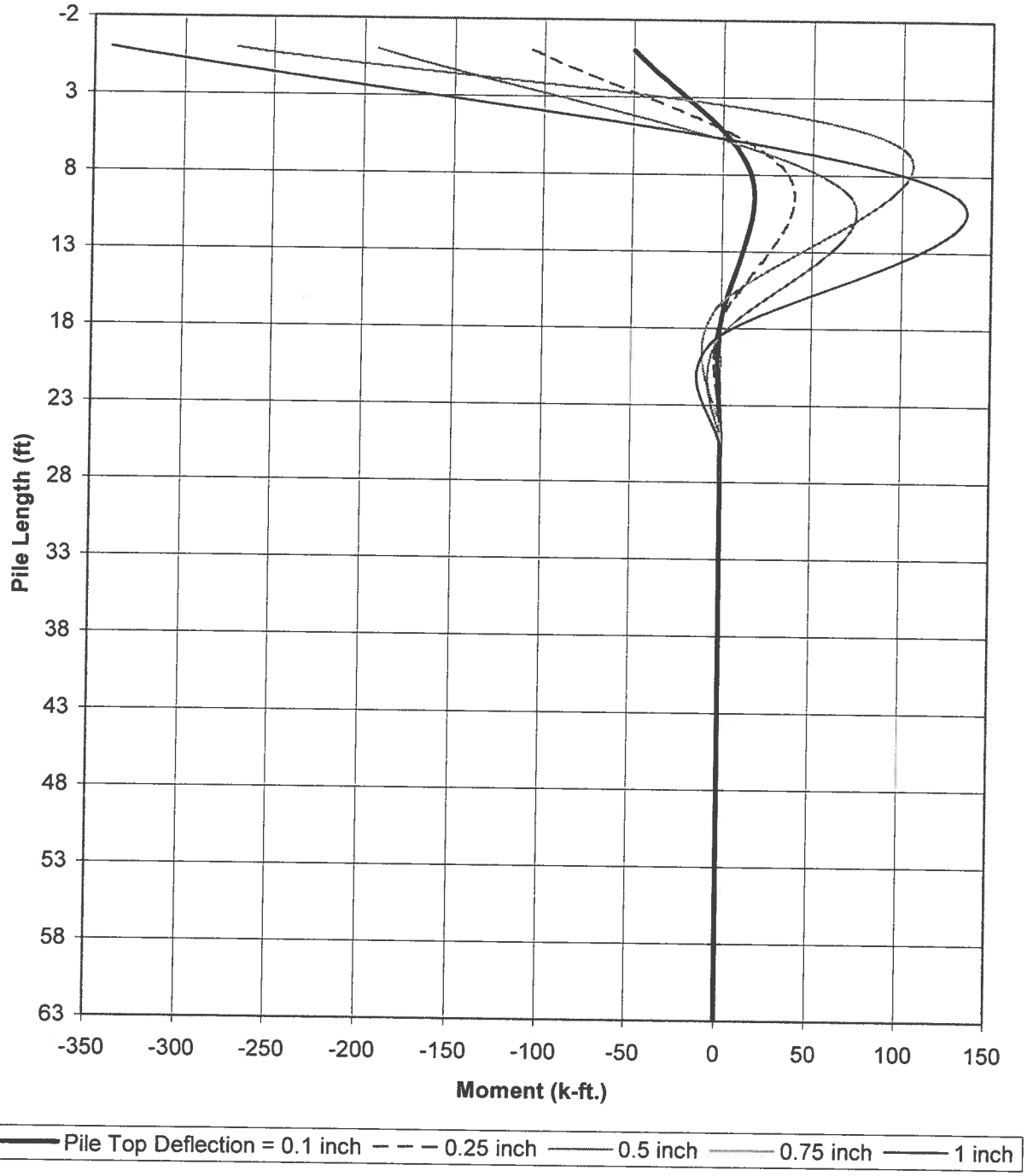
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE C-12
PROJECT NO. 35352.GEO / DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 2 - Free Head (Strong Axis)



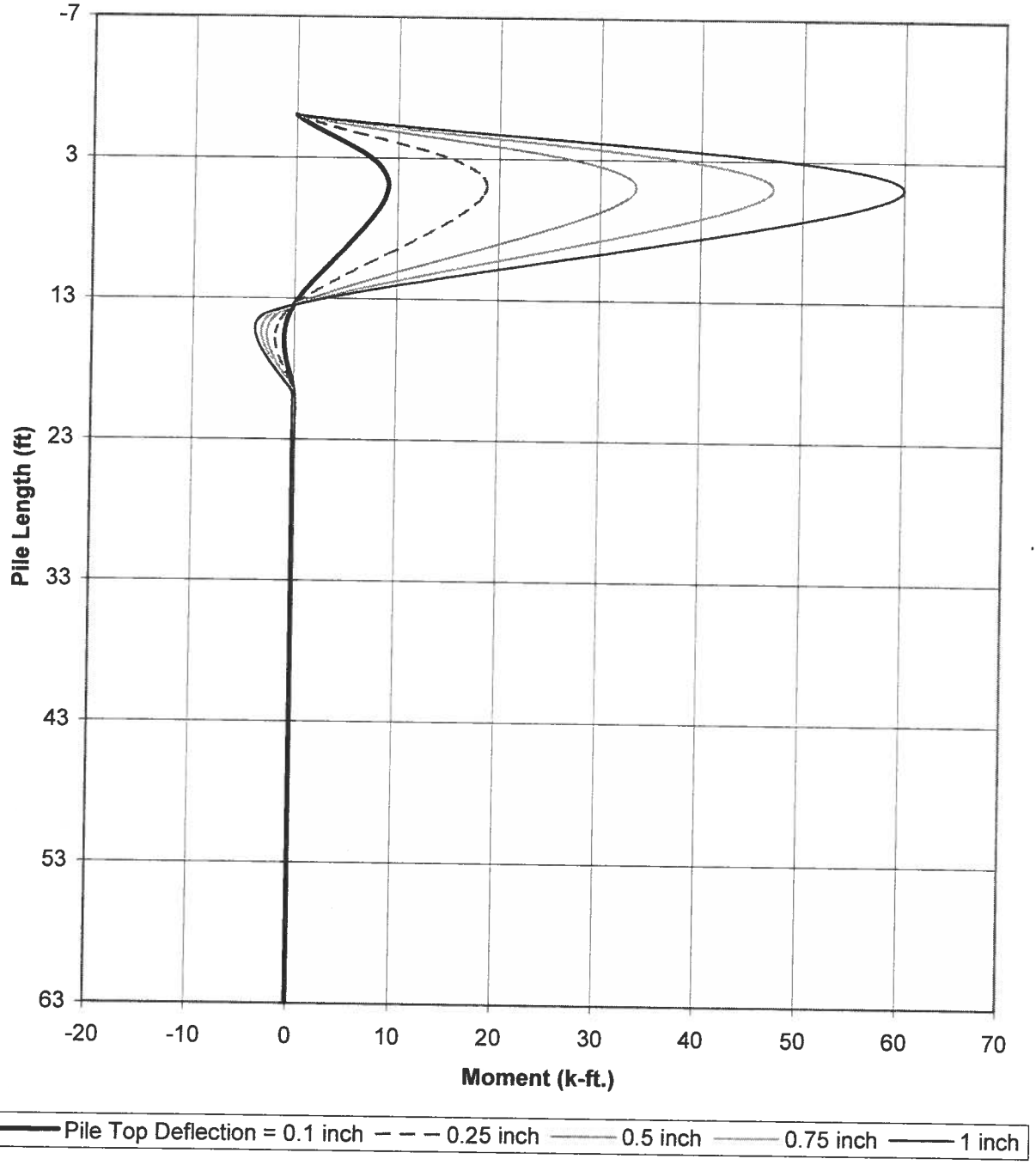
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-13**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 2 - Fixed Head (Strong Axis)



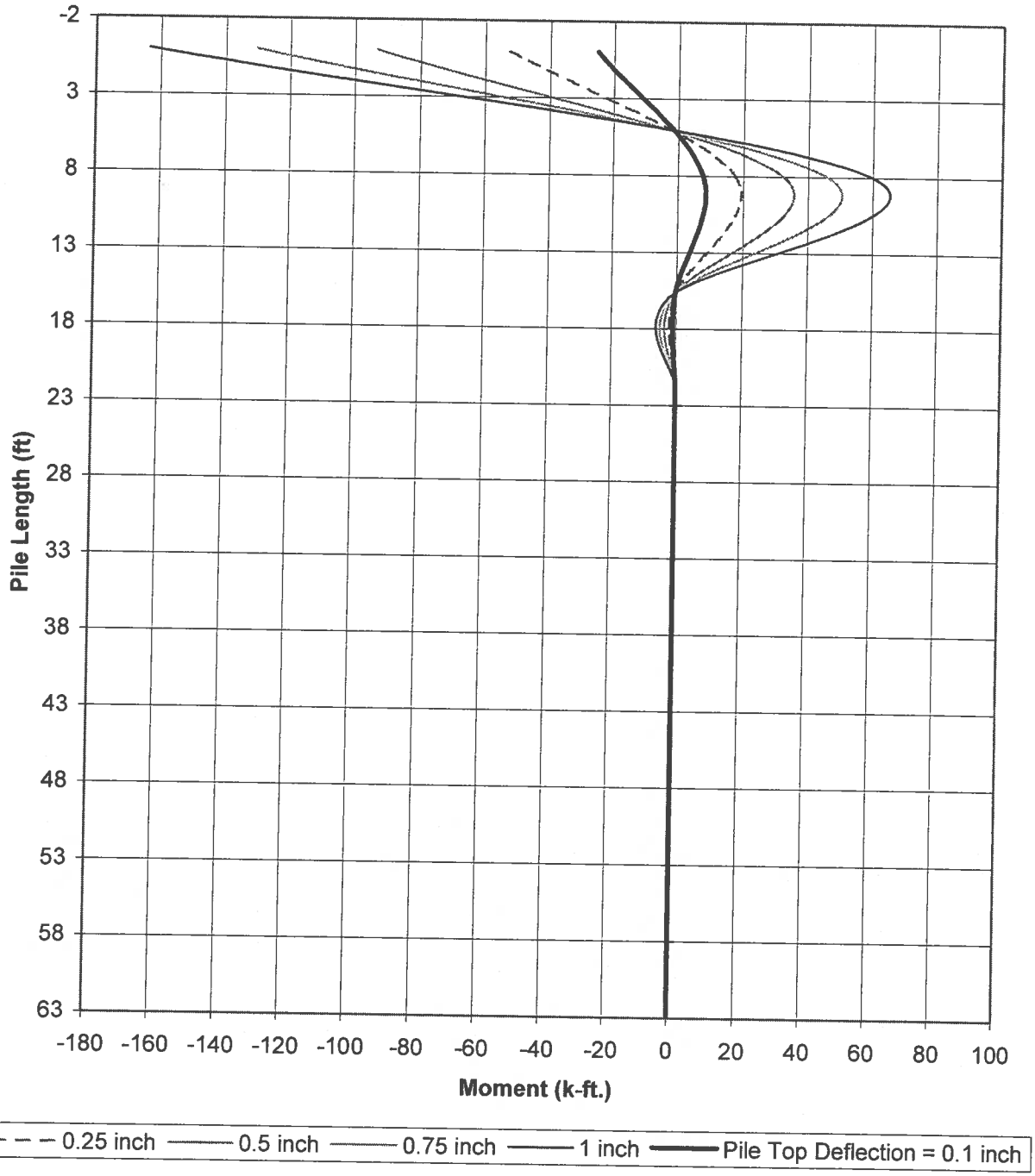
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-14**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 2 - Free Head (Weak Axis)

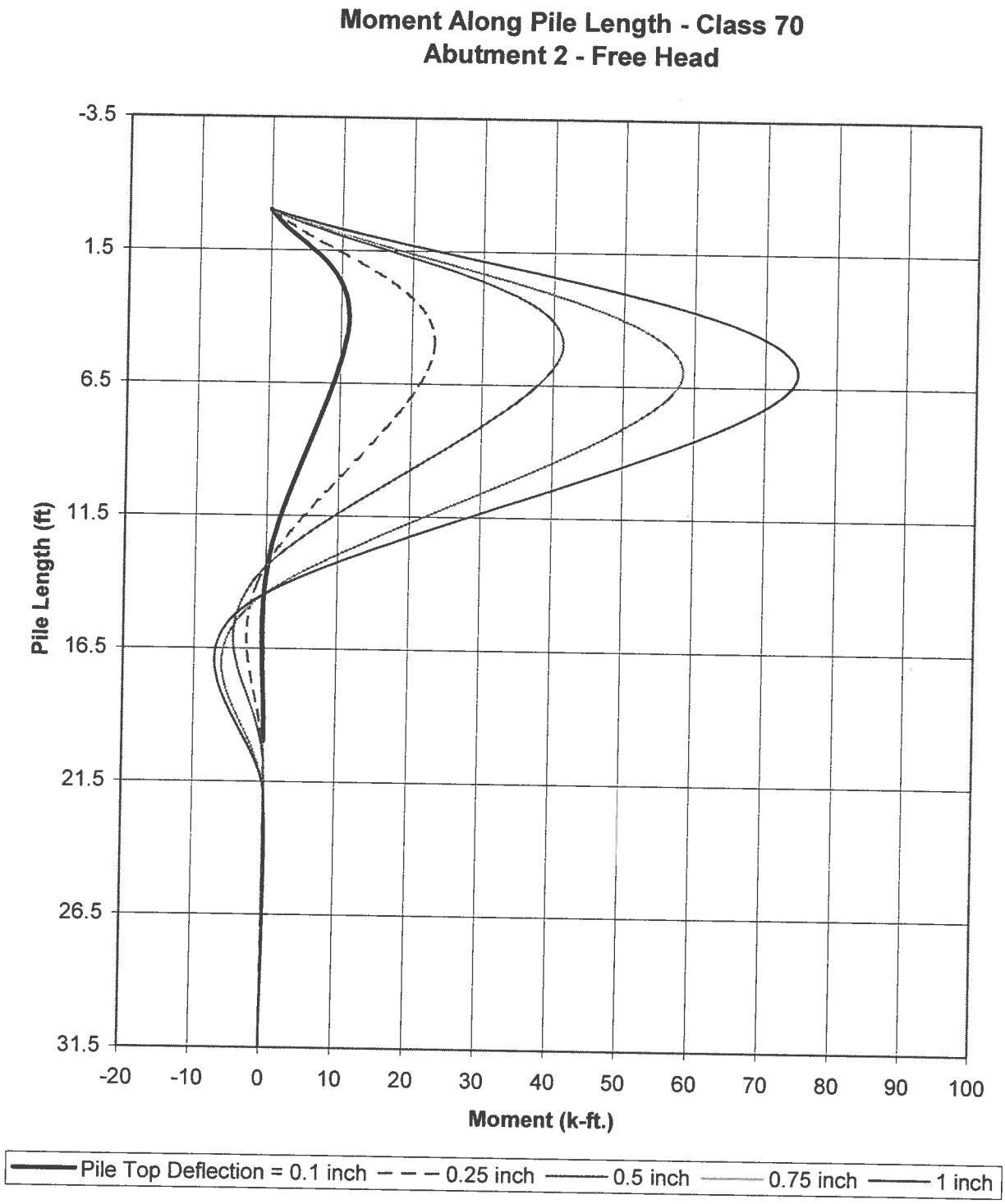


KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-15**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 12x53
Abutment 2 - Fixed Head (Weak Axis)



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-16**
PROJECT NO. 35352.GEO | DATE: 11-01-06		



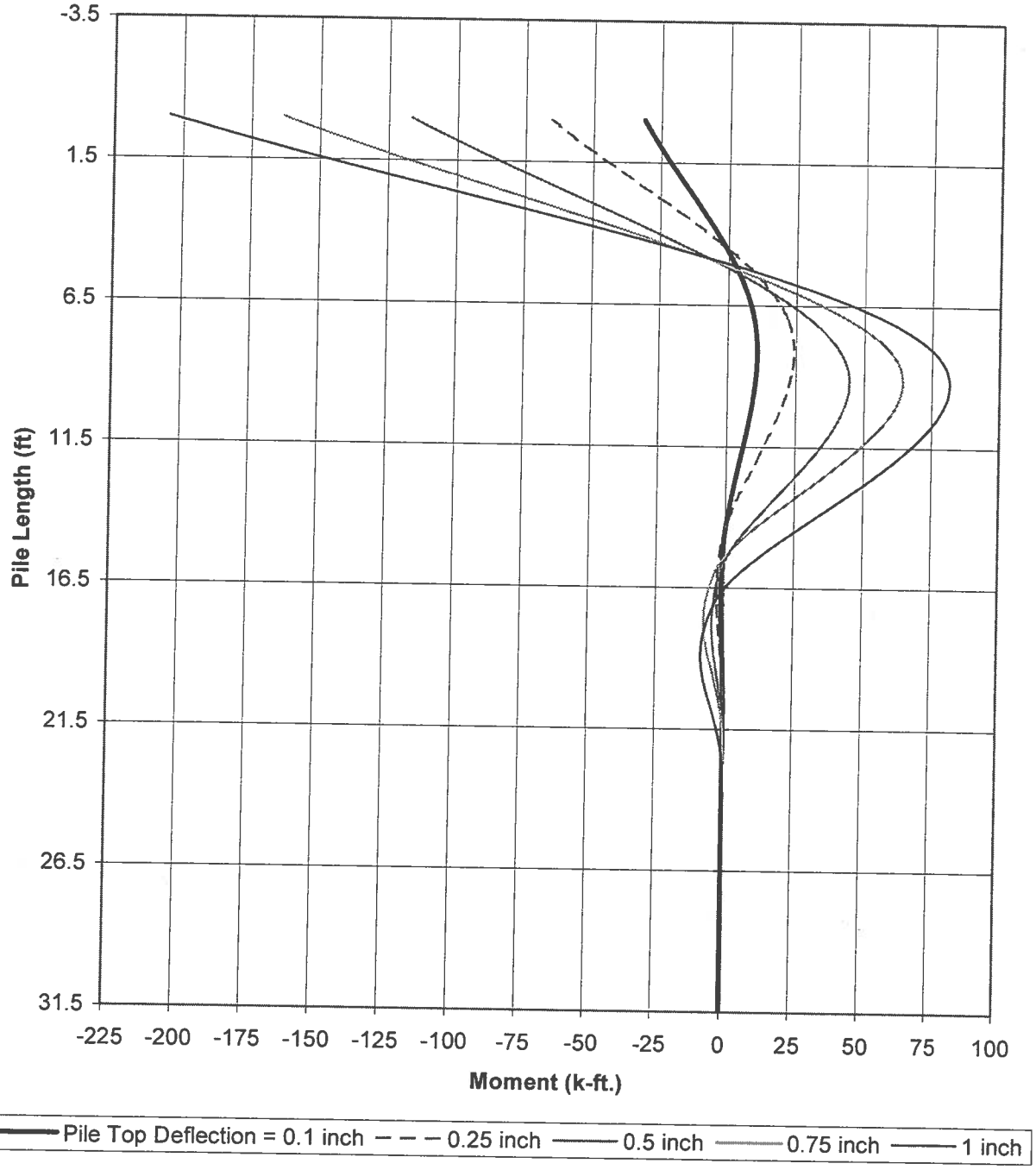
KLEINFELDER

PROJECT NO. 35352.GEO | DATE: 11-01-06

MAIN STREET BRIDGE
AT SANTA ROSA CREEK
CAMBRIA, CALIFORNIA

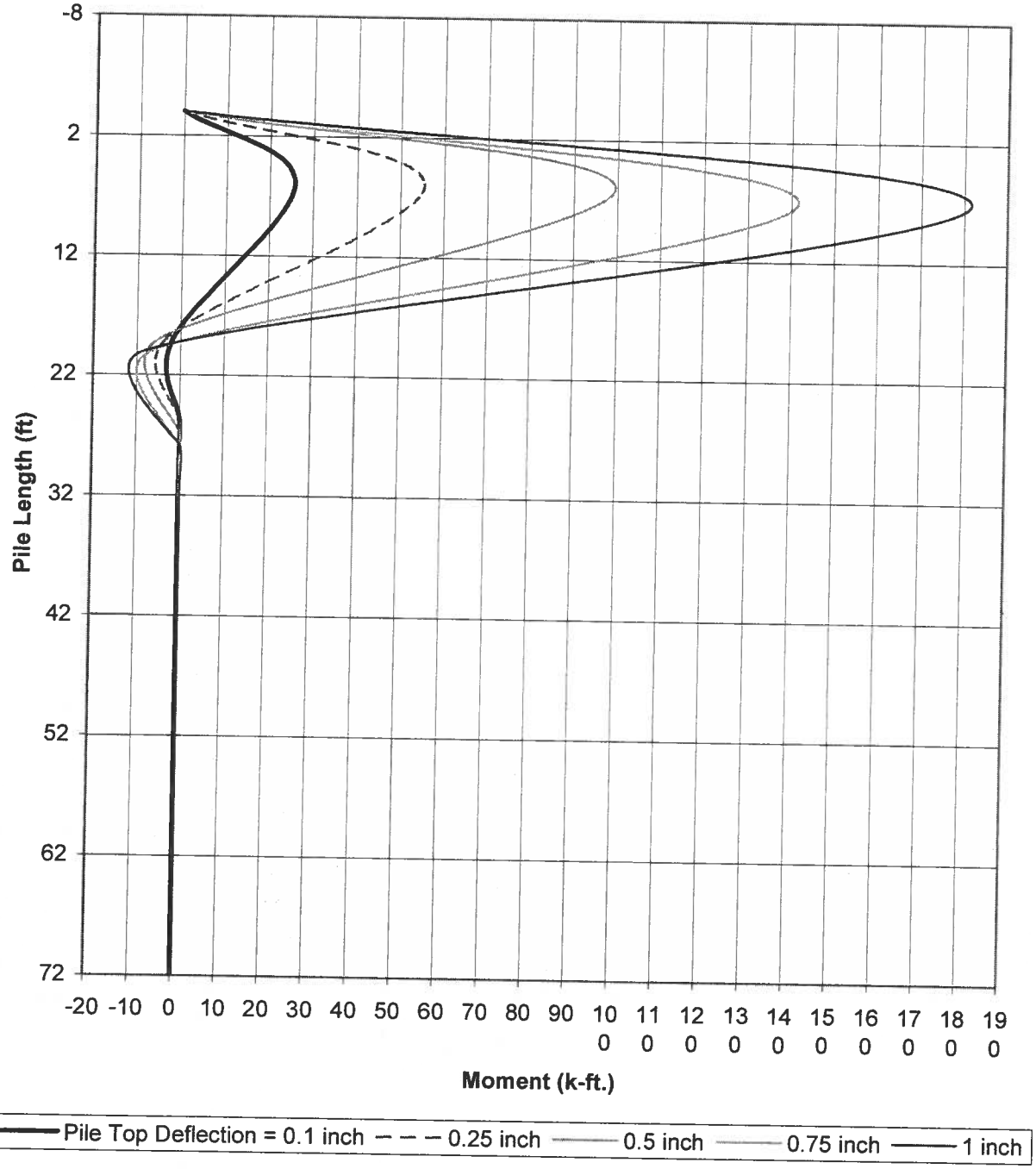
PLATE
C-17

Moment Along Pile Length - Class 70
Abutment 2 - Fixed Head



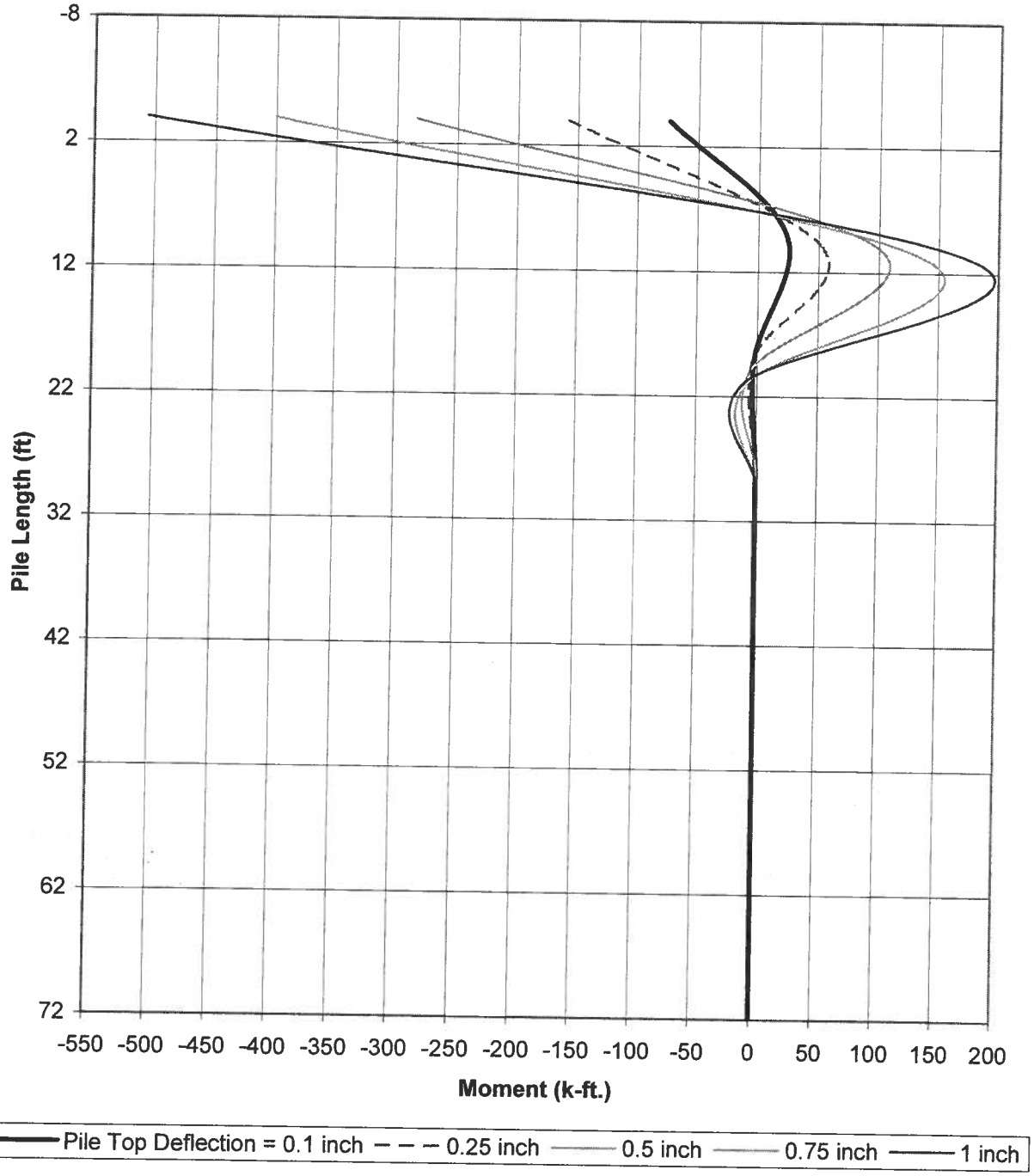
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE C-18
PROJECT NO. 35352.GEO / DATE: 11-01-06		

Moment Along Pile Length - HP 14x73
Abutment 2 - Free Head (Strong Axis)



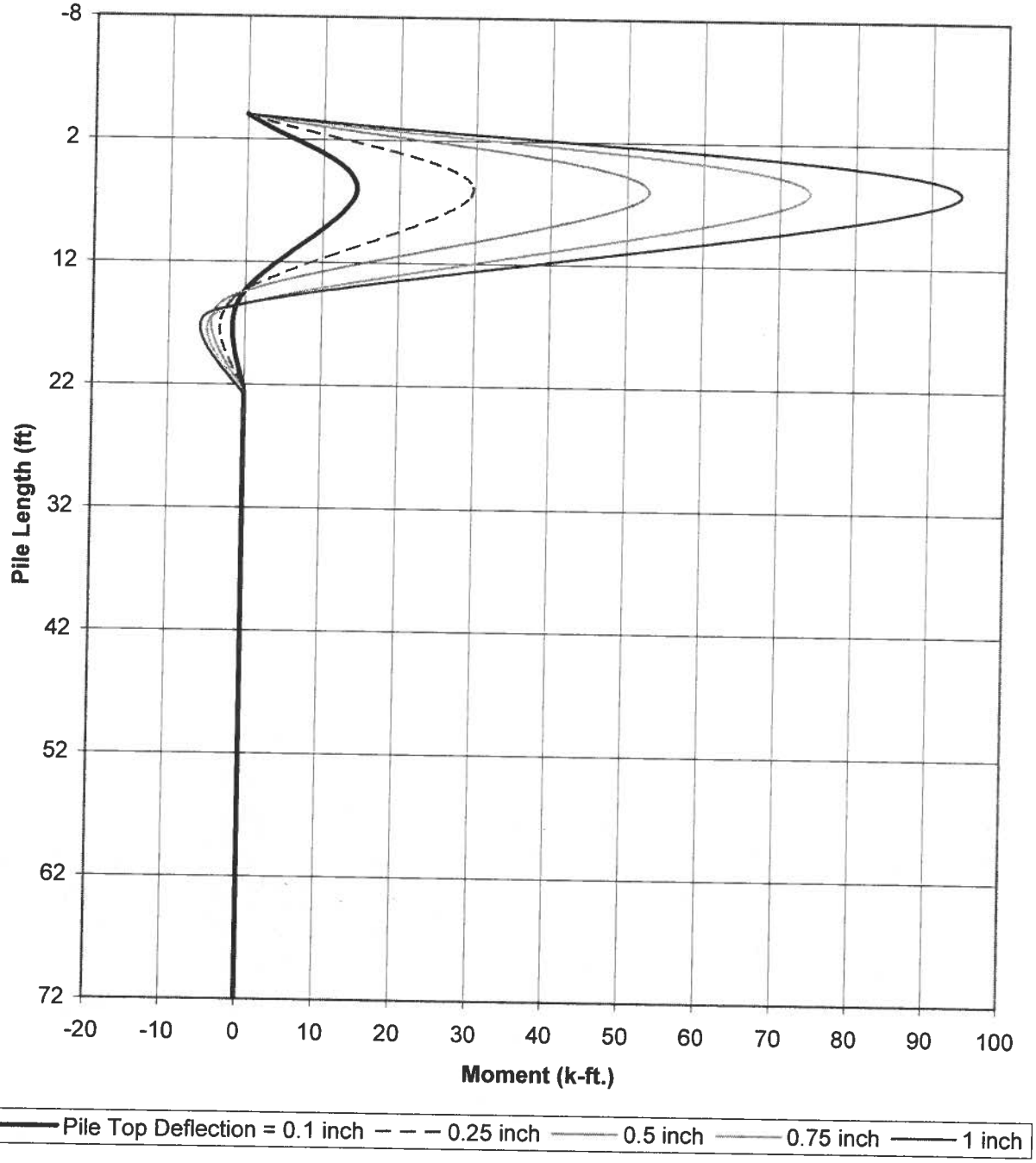
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE C-19
PROJECT NO. 35352.GEO / DATE: 11-01-06		

Moment Along Pile Length - HP 14x73
Abutment 2 - Fixed Head (Strong Axis)



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-20**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

Moment Along Pile Length - HP 14x73
Abutment 2 - Free Head (Weak Axis)



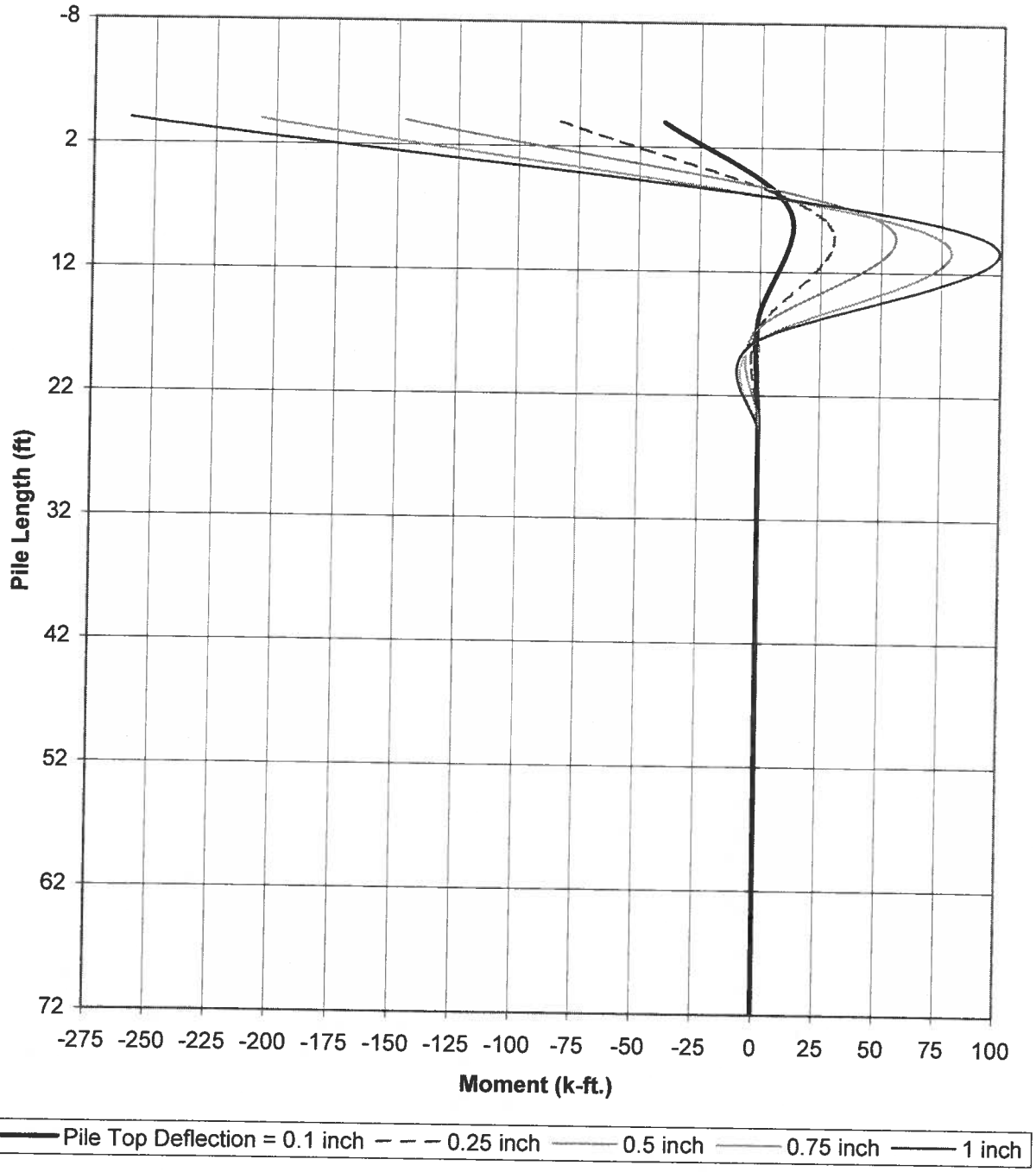
KLEINFELDER

PROJECT NO. 35352.GEO | DATE: 11-01-06

MAIN STREET BRIDGE
AT SANTA ROSA CREEK
CAMBRIA, CALIFORNIA

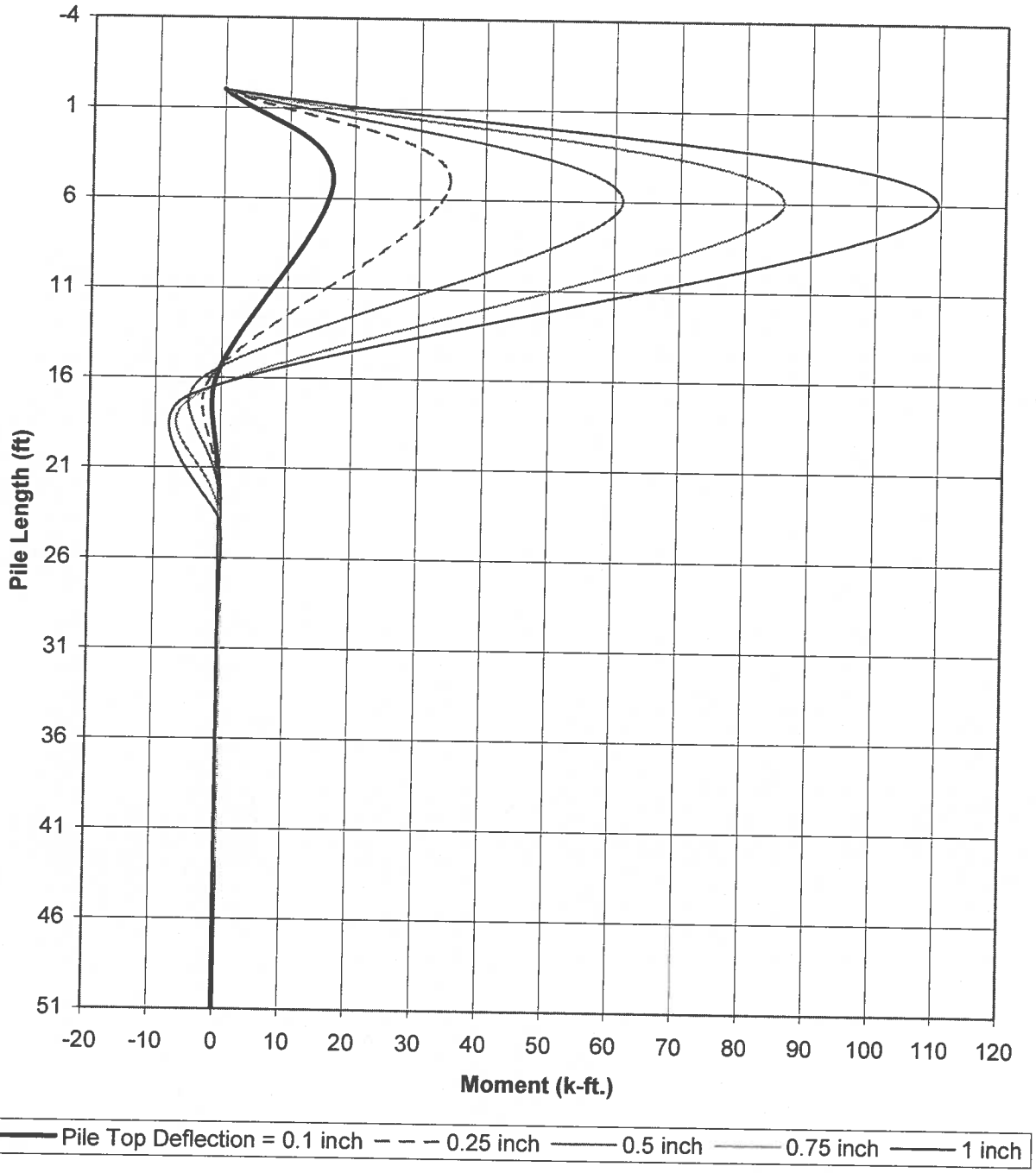
PLATE
C-21

Moment Along Pile Length - HP 14x73
Abutment 2 - Fixed Head (Weak Axis)



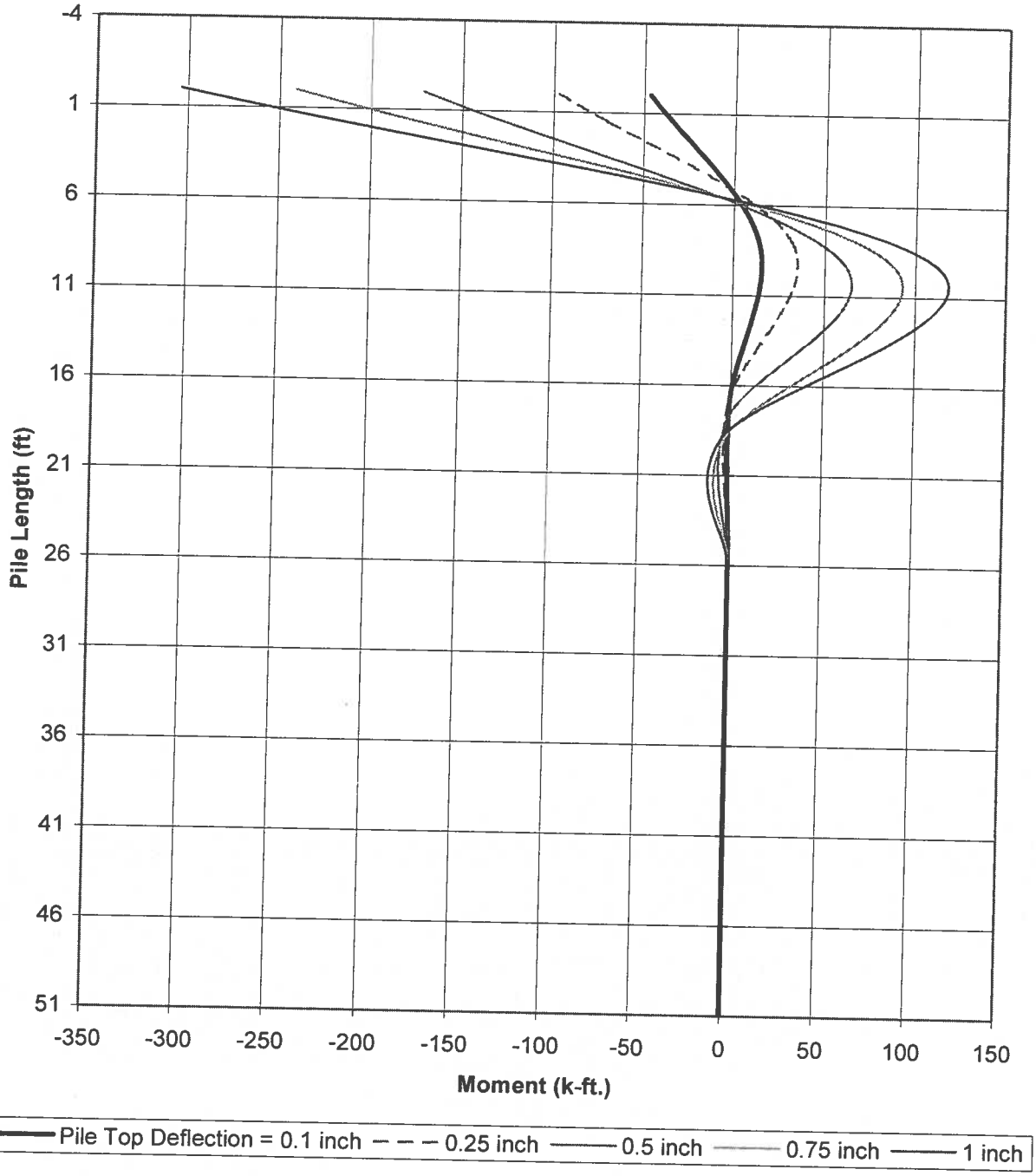
KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE C-22
PROJECT NO. 35352.GEO / DATE: 11-01-06		

Moment Along Pile Length - Class 100
Abutment 2 - Free Head



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-23**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

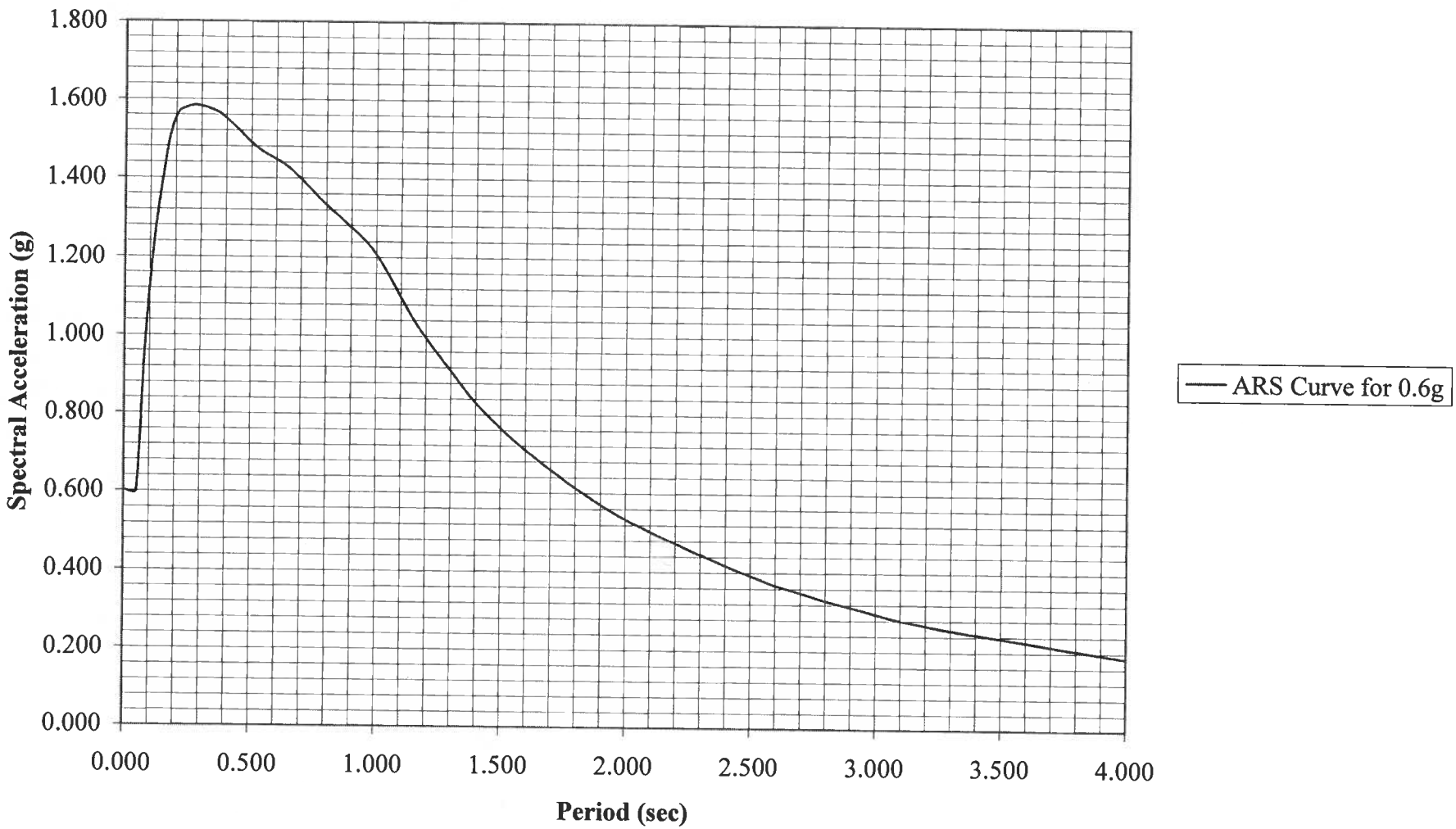
Moment Along Pile Length - Class 100
Abutment 2 - Fixed Head



KLEINFELDER	MAIN STREET BRIDGE AT SANTA ROSA CREEK CAMBRIA, CALIFORNIA	PLATE **C-24**
PROJECT NO. 35352.GEO | DATE: 11-01-06		

APPENDIX D

Caltrans Seismic Design Criteria Version 1.4 Figure B.8 ARS Curve for Soil Profile D (M=7.25+0.25) With Proximity Increase



mc@bengalengineering.com

CALTRANS SEISMIC DESIGN CRITERIA 2004 VERSION 1.4 (BASED ON ATC-32)
M7.25+0.25, Soil Type D With Proximity Increase

ARS Curve for 0.6g

Period (sec)	SA (g)	SA + 20% (g)
0.010	0.600	0.600
0.050	0.600	0.600
0.079	0.938	0.938
0.099	1.127	1.127
0.124	1.283	1.283
0.154	1.419	1.419
0.177	1.507	1.507
0.208	1.564	1.564
0.236	1.578	1.578
0.271	1.584	1.584
0.309	1.580	1.580
0.355	1.570	1.570
0.386	1.557	1.557
0.442	1.525	1.525
0.533	1.453	1.472
0.650	1.345	1.426
0.787	1.203	1.340
0.868	1.128	1.294
0.998	1.008	1.209
1.162	0.859	1.031
1.307	0.755	0.906
1.411	0.685	0.822
1.575	0.602	0.723
1.819	0.504	0.605
2.068	0.428	0.513
2.537	0.319	0.383
2.954	0.253	0.304
3.203	0.222	0.266
3.463	0.199	0.239
4.000	0.155	0.186
		0.000