October 6, 2015
Revised: April 3, 2020
File No.: 20210004.001A

**Quincy Engineering, Inc.**
11017 Cobblerock Road, Suite 100
Rancho Cordova, California 95670

Attention: Mr. Mario Quest

**SUBJECT: Foundation Report (95% Submittal)**
**El Camino Real Bridge Replacement**
**Santa Margarita Creek and El Camino Real**
**Bridge No.**
**San Luis Obispo County, California**

Mr. Quest:

The attached report presents the results of a geotechnical foundation evaluation for the El Camino Real Bridge Replacement over Santa Margarita Creek in San Luis Obispo County, California. This report describes the study and provides conclusions and recommendations for use in design for the bridge.

Kleinfelder appreciates the opportunity to provide geotechnical engineering services to Quincy Engineering, Inc. and County of San Luis Obispo. It is trusted this information will meet 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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**FOUNDATION REPORT (95% Submittal)**
**EL CAMINO REAL BRIDGE REPLACEMENT**
**SANTA MARGARITA CREEK AND**
**EL CAMINO REAL**
**BRIDGE NO.**
**SAN LUIS OBISPO COUNTY, CALIFORNIA**

A report prepared for:

**Quincy Engineering, Inc.**
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Report prepared by:

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October 6, 2015
Revised April 3, 2020
File No.: 20210004.001A

Prepared For:
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**FOUNDATION REPORT (95% Submittal)**
**EL CAMINO REAL BRIDGE REPLACEMENT**
**SANTA MARGARITA CREEK AND EL CAMINO REAL**
**BRIDGE NO. 48C-0029**
**SAN LUIS OBISPO COUNTY, CALIFORNIA**

Kleinfelder Job No.: 20210004.001A

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October 6, 2015
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**COVER LETTER**

# 1 INTRODUCTION

## 1.1 GENERAL

This report presents the results of a Foundation Report for the proposed El Camino Real Bridge Replacement over Santa Margarita Creek in San Luis Obispo County, California.

## 1.2 SCOPE OF WORK

The purpose of the Foundation Report is to provide geotechnical recommendations and opinions to aid in design of the project. The scope of services consisted of field exploration, laboratory testing, engineering analysis, and preparation of this written report. The report provides the following:

- A description of the proposed project;
- Discussion of the field and laboratory testing programs;
- Comments on the corrosion potential of foundation soil;
- Comments on the regional geology and site engineering seismology, including the recommended Caltrans Seismic Design Criteria Version 1.7 ARS curve;
- Comments on liquefaction potential;
- Design and specified tip elevations for cast in drilled hole (CIDH) piles;
- LPILE profile and comments on lateral pile capacity;
- Comments on initial soil stiffness and ultimate equivalent lateral pressure by Caltrans procedures for resisting dynamic loading of abutment endwalls;
- Recommended unfactored active earth pressures for static (Service and Strength/Construction Limit) and dynamic (Extreme Event) conditions;
- Comments on cut slope gradients; and
- Log of Test Borings

## 1.3 PROJECT DESCRIPTION

The proposed El Camino Real Bridge Replacement will consist of a three-span structure over the Santa Margarita Creek. Planning indicates the bridge will have a total length of 142 feet and bridge width of 53 feet. The bridge will be a CIP prestressed concrete slab. The bridge will be supported by 24-inch diameter CIDH piles at Abutments 1 and 4 and by 48-inch CIDH piles at Bents 2 and 3. Tables 1.3-1 and 1.3-2 provide foundation data on the bridge furnished by Quincy Engineering, Inc., the Project Structural Design Engineer. El Camino Real will follow the same alignment with construction performed in phases to maintain access across the bridge.

**TABLE 1.3-1**
**DEEP FOUNDATION DESIGN DATA SHEET**

| Support | Location (Sta. No.) | Pile Type | Finished Grade Elev. (ft) | Cut-off Elev. (ft) | Pile Cap Size (ft) | | $S_P$[1] | No. Piles per Support |
|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | | |
| Abutment 1 | 22+65.77 | 24” CIDH | 931 | 921.25 | 10 | 54 | 1” | 9 |
| Bent 2 | 23+08.77 | 48” CIDH | 918 | 915 | N/A | N/A | 1” | 5 |
| Bent 3 | 23+67.77 | 48” CIDH | 921 | 915 | N/A | N/A | 1” | 5 |
| Abutment 4 | 24+07.77 | 24” CIDH | 931 | 919.25 | 10 | 54 | 1” | 9 |

Notes: 1. Permissible settlement under service limit load
2. To allow for future degradation, design channel bottom is considered to be elevation 903 feet.

**TABLE 1.3-2**
**DEEP FOUNDATION DESIGN LOADS**

| Supp-ort | Pile Type | Service Limit State (kips) | | | Strength Limit State (kips) | | | | Extreme Event Limit State (kips) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Total Load | | Permanent Loads | Compression | | Tension | | Compression | | Tension | |
| | | Per Support | Max Per Pile | Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max Per Pile | Per Support | Max. Per Pile |
| Abut 1 | 24” CIDH | 1550 | 210 | 150 | N/A | 260 | N/A | N/A | N/A | 150 | N/A | N/A |
| Bent 2 | 48” CIDH | 1670 | 340 | 250 | N/A | 508 | N/A | N/A | N/A | 385 | N/A | N/A |
| Bent 3 | 48” CIDH | 1630 | 330 | 240 | N/A | 493 | N/A | N/A | N/A | 385 | N/A | N/A |
| Abut 4 | 24” CIDH | 1690 | 240 | 180 | N/A | 310 | N/A | N/A | N/A | 180 | N/A | N/A |

## 1.4 POLICY EXCEPTIONS

No known exceptions to Caltrans policy were made in the geotechnical evaluation for the foundations for this project.

# 2 FIELD AND LABORATORY PROGRAMS

## 2.1 FIELD INVESTIGATION AND TESTING

The field exploration for the project was conducted on April 14 and June 2, 2014 and consisted of drilling three (3) test borings at the proposed bridge crossings. The test borings were drilled with a CME 55 truck-mounted drill rig using hollow stem auger and rotary wash techniques. The borings were excavated to depths of 71.5 and 101.5 feet below the existing ground surface. The approximate locations of the test borings are indicated on the Log of Test Borings (Appendix A) of this report.

The earth materials encountered in the test borings were visually classified in the field and a continuous log was recorded. In-place samples of the soil unit were attempted in some of the test borings by driving a 1.4 and 2.5-inch I.D. split barrel sampler containing brass liners into the undisturbed soil with a 140-pound automatic safety hammer free falling a distance of 30-inches. Resistance to sampler penetration over the last 12-inches is noted on the Log of Test Borings. The penetration index listed on the Log of Test Borings has not been corrected for the effects of overburden pressure, sampler size, rod length, or hammer efficiency.

A Kleinfelder engineer logged the earth materials encountered during the drilling operation. Soil samples obtained were taken to the laboratory for geotechnical testing.

## 2.2 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples to evaluate certain 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:

- Unit Weight (ASTM D2937)
- Moisture Content (ASTM D2216)
- Direct Shear (ASTM D3080)
- Atterberg Limits (ASTM D4318)

- Material Finer than 75-micron (ASTM D1140)
- pH and Minimum Resistivity (California Test Method No. 643)
- Soluble Sulfates (California Test Method No.417)
- Soluble Chlorides (California Test Method No.422)

The soluble sulfate, soluble chloride, pH, and minimum resistivity results are presented in Section 4 ("Corrosion Evaluation"). The remaining test results are provided in Appendix B.

# 3 SITE GEOLOGY AND SUBSURFACE CONDITIONS

## 3.1 SURFACE CONDITIONS AND TOPOGRAPHY

The bridge site is located in the Santa Lucia Range of the Coast Ranges geomorphic province. The natural terrain in the project area is relatively flat with the exception of the creek channel. Santa Margarita Creek is a northeasterly flowing portion of the Salinas River system. The elevation of the project area ranges from about 920 feet to about 938 feet above sea level. The existing bridge is a 122-foot long four span bridge over Santa Margarita Creek. Exposed bedrock exists in the creek channel at the bridge site. Vegetation in the project area consists of sparsely scattered brush, grasses, and small trees.

## 3.2 REGIONAL GEOLOGY

The project site lies in 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.

## 3.3 SITE GEOLOGY

The material exposed in the Santa Margarita Creek channel is the late Miocene Age Santa Margarita Formation. The Santa Margarita Formation is essentially confined to the synclinal trough between the Rinconada and Nacimento fault zones. The site subsurface conditions were explored by performing three test borings which extended to depths of 71.5 feet and 101.5 feet below the existing road grade. The soil consisted primarily of late Miocene Age Santa Margarita Formation. The upper 13 to 15 feet, above the creek bed, consisted of silty sand (SM), which was then underlain by a sequence of beds of massive over-consolidated, somewhat friable sandstone. While not encountered in the field borings performed, it can be observed that the sandstone beds are interspersed with well cemented shell beds. These conglomeratic shell beds are locally about 2 to 3 feet thick.

The beds of the Santa Margarita Formation in the vicinity of the bridge dip southwesterly (upstream) at about 45 to 60 degrees. The conglomeratic shell beds trend nearly perpendicular to the creek bed and are spaced at about 40 to 80 foot intervals. These beds extend as far as

visible downstream and about 100 to 150 feet upstream. The conglomeratic shell beds are considerably more resistant to erosion than the massive sandstone. As a result, the well cemented shell beds protrude above the channel bottom.

Narrow lengths of the protruding conglomeratic shell beds within the channel have broken out, producing intervals of channel constriction. This constriction results in concentrated flows with significantly increased velocity and turbulence. This condition is very prominent in the shell bed directly upstream from the bridge. Downstream of the bridge, the cemented shell beds have functioned as "check-structures" which are producing 2 to 3 foot high steps in the channel bottom. The closest downstream shell bed has a vertical drop in excess of 8 feet (bottom could not be measured during site exploration). This bed essentially represents a significant "nickpoint" in the headward degradation of the Santa Margarita Creek channel.

A more detailed description of the materials encountered in the test borings is noted on the Log of Test Borings drawing in Appendix A.

## 3.4 GEOLOGIC HAZARDS

Landslides are not anticipated at the site due to a relatively flat topography and the relatively competent sedimentary bedrock in the creek channel. Jointing and fractures could result in localized rock-fall from sleeper slopes.

The project site and its vicinity are located in an area characterized by moderate to high seismic activity. Based on mapping by the Caltrans ARS Online website (Caltrans 2015), the Rinconada 2011 CFM fault (Fault ID No. 174) is mapped approximately 0.4 miles (0.6 km) northeast of the proposed bridge.

Based on the relatively shallow bedrock, subsidence, liquefaction or lateral spreading are not anticipated to be problematic to the structures.

The soils encountered at the site have a low expansion potential. The potential for heaving at the site is considered low.

## 3.5 GROUNDWATER CONDITIONS

Groundwater was encountered at a depth of approximately 33 feet in boring B-1. The drilling techniques did not allow for direct observation of groundwater seepage in the other borings.

# 4 CORROSION EVALUATION

A bulk soil sample obtained from test boring B-1 at a depth of 5 feet was tested to evaluate the pH, minimum resistivity, soluble sulfate content and soluble chloride content. Specific test results are presented in 4.4-1.

**TABLE 4.4-1**
**CORROSION RELATED TESTING**

| Boring No. | pH | Minimum Resistivity (ohm-cm) | Soluble Sulfate (mg/kg) | Soluble Chloride (mg/kg) |
|---|---|---|---|---|
| B-1 | 6.4 | 2,466 | 160 | 27 |

These laboratory tests indicate the resistivity and soluble sulfates and chlorides are all outside the Caltrans threshold limits. Consequently, normal portland cement concrete would be adequate for foundation concrete.

# 5 SEISMIC RECOMMENDATIONS

## 5.1 LOCAL FAULTING

There are no known faults which cut through the site. The project site is not located in an Alquist-Priolo Earthquake Fault Zone, as defined by Special Publication 42 (revised 2007) published by the California Geologic Survey (CGS).

## 5.2 SEISMIC DESIGN CRITERIA

Seismic design parameters were developed in accordance with the Seismic Design Criteria Version 1.7.

The project site is located in a region with the potential for relatively moderate seismic activity. The more significant faults that could influence the project site include the Rinconada 2011 CFM (Fault ID No. 209), the Oceanic – West Huasna (Fault ID No. 223), and the Los Osos 2011 Fault (Fault ID No. 232). According to the Caltrans fault database, the Rinconada 2011 CFM Fault is a strike slip fault with a dip angle of 82 degrees towards the west and assigned Maximum Magnitude ($M_{Max}$) of 7.4; the Oceanic – West Huasna Fault is a reverse fault with a dip angle of 58 degrees and assigned Maximum Magnitude ($M_{Max}$) of 6.9; and the Los Osos 2011 Fault is a reverse fault with a dip angle of 45 degrees and assigned Maximum Magnitude ($M_{max}$) of 6.9. The characteristics of these faults are summarized in Table 5.2-1.

Based on the data from the borings and per Caltrans SDC, the site can be classified as Soil Profile Type D. A $V_{s30}$ of 328 m/s was determined and used for the evaluation. The site is not located within a California deep soil basin region, as defined by Caltrans, so $Z_{1.0}$ and $Z_{2.5}$ were considered not applicable. Site characteristics and governing deterministic faults are summarized in Table 5.2-1 below.

## TABLE 5.2-1
## SITE CHARACTERISTICS AND GOVERNING DETERMINISTIC FAULTS PARAMETERS

| | |
|---|---|
| **Site Coordinates** | Lat = 35.4287 deg, Long = -120.6058 deg |
| **Shear Wave Velocity** | 328 m/s |
| **Depth to $V_s$=1.0 km/s, $Z_{1.0}$** | N/A |
| **Depth to $V_s$=2.5 km/s, $Z_{2.5}$** | N/A |
| | |
| **Fault Name and ID Number** | Rinconada 2011 CFM, No. 209 |
| **Maximum Magnitude ($M_{Max}$)** | 7.4 |
| **Fault Type** | Strike Slip |
| **Fault Dip** | 82 degrees |
| **Dip Direction** | West |
| **Bottom of Rupture Plane** | 10 km |
| **Top of Rupture Plane ($Z_{tor}$)** | 0 km |
| **$R_{RUP}$[1]** | 0.641 km |
| **$R_{jB}$[2]** | 0.000 km |
| **$R_X$[3]** | 0.647 km |
| **$F_{norm}$ (1 for normal, 0 for others)** | 0 |
| **$F_{rev}$ (1 for reverse, 0 for others)** | 0 |
| | |
| **Fault Name and ID Number** | Oceanic – West Huasna, No. 223 |
| **Maximum Magnitude ($M_{Max}$)** | 6.9 |
| **Fault Type** | Reverse |
| **Fault Dip** | 58 degrees |
| **Dip Direction** | Southwest |
| **Bottom of Rupture Plane** | 7 km |
| **Top of Rupture Plane ($Z_{tor}$)** | 0 km |
| **$R_{RUP}$[1]** | 9.948 km |
| **$R_{jB}$[2]** | 9.948 km |
| **$R_X$[3]** | 9.948 km |
| **$F_{norm}$ (1 for normal, 0 for others)** | 0 |
| **$F_{rev}$ (1 for reverse, 0 for others)** | 1 |
| | |
| **Fault Name and ID Number** | Los Osos 2011, No. 232 |
| **Maximum Magnitude ($M_{Max}$)** | 6.9 |
| **Fault Type** | Reverse |
| **Fault Dip** | 45 degrees |
| **Dip Direction** | Southeast |
| **Bottom of Rupture Plane** | 9.9 km |
| **Top of Rupture Plane ($Z_{tor}$)** | 0 km |
| **$R_{RUP}$[1]** | 14.061 km |
| **$R_{jB}$[2]** | 9.985 km |
| **$R_X$[3]** | 19.184 km |
| **$F_{norm}$ (1 for normal, 0 for others)** | 0 |
| **$F_{rev}$ (1 for reverse, 0 for others)** | 1 |

**Notes:**
[1]$R_{RUP}$ = Closest distance from the site to the fault rupture plane.
[2]$R_{JB}$ = Joyner-Boore distance; the shortest horizontal distance to the surface projection of the rupture area.
[3]$R_X$ = Horizontal distance from the site to the fault trace or surface projection of the top of the rupture plane.

### 5.2.1 Deterministic Response Spectrum

The deterministic response spectrum was developed using ARS Online as required by Caltrans. The deterministic response spectrum from the Rinconada CFM 2011 Fault governed.

### 5.2.2 Probabilistic Response Spectrum

The probabilistic response spectrum was developed using the ARS Online, and checked with the USGS Deaggregations tool.

### 5.2.3 Preliminary Design Response Spectrum

The upper envelope of the deterministic and probabilistic spectral values determines the design response spectrum. The deterministic response spectra were found to govern at this site for all periods up to approximately 5 seconds. The recommended acceleration and displacement design response spectra are presented graphically and numerically in Appendix C.

### 5.2.4 References


Caltrans. Caltrans ARS Online, http://dap3.dot.ca.gov/shake_stable/v2/.

Caltrans. Geotechnical Services Manual.

Caltrans. Seismic Design Criteria, Appendix B Design Spectrum

Caltrans. Website http://dap3.dot.ca.gov/shake_stable/v2/technical.php


## 5.3 LIQUEFACTION POTENTIAL

In order for liquefaction of soils due to ground shaking to occur, it is generally accepted that four conditions will exist:

- The subsurface soils are in a relatively loose state,
- The soils are saturated,
- The soils are non-plastic, and

- Ground motion is of sufficient intensity to act as a triggering mechanism.

Caltrans allows the use of the PHGA based on a probabilistic analysis for liquefaction when the site is near a fault. The PHGA based on a probabilistic analysis is 0.40g for this site. Based on the bridge foundation supporting material (bedrock), the potential for liquefaction, and associated seismically induced settlement, is nil.

# 6 FOUNDATION RECOMMENDATIONS

## 6.1 GENERAL

Based on the topographic data and the bedrock elevations encountered in the test borings, subsurface bedrock elevations have been estimated. Table 6.1-1 provides the estimated bedrock elevation at the support locations.

**TABLE 6.1-1**
**ESTIMATED BEDROCK ELEVATIONS**

| Support | Location (A-line) | Estimated Bedrock Elevation (feet) |
|---|---|---|
| Abutment 1 | Station 22+65.77 | 924 |
| Bent 2 | Station 23+08.77 | 920 |
| Bent 3 | Station 23+67.77 | 922 |
| Abutment 4 | Station 24+07.77 | 922 |

The bedrock is exposed in the creek channel. The values are approximate and are provided to allow for a general estimate of excavation and concrete quantities. The final bearing and tip elevations and final quantities will have to be determined based on field observation during construction.

## 6.2 CREEK BED SCOUR

The material below about elevation 922 to 920 feet and exposed in the channel is a massive sandstone with laterally interspersed steeply inclined conglomeratic shell beds. The test borings indicated the sandstone consistency stays relatively similar with depth.

As noted in the Draft Foundation Report, dated October 7, 2014, long-term degradation is the more significant form of scour at the site. With failure of the downstream conglomeratic shell bed, which is presently a stationary nick point, long-term degradation will likely accelerate at the bridge site. Design is considering the long-term degraded channel elevation at 903 feet (about 15 feet below the present channel).

The sandstone is a competent “soft to moderately strong” rock. Exposure results in a relatively slow weathering (chemical alteration) of the sandstone surface. The weathering process is likely fractions of an inch each year. The weakened and softened weathered surface of the sandstone is the material which will be subject to potential scour. As the more weathered material is transported away by channel flow, fresher sandstone which is more resistant to scour is exposed. Consequently, single event localized scour due to pier obstruction will be a function of event regularity.

Considering the apparent relative consistency of the sandstone with depth, the potential for localized pier scour would not be expected to differ as the channel degrades from the present elevation to the future design elevation. The greatest potential for localized single event pier scour would be after a prolonged drought. This would allow for the thickest development of weathered material. It is believed a very conservative estimation of a single event pier scour depth would be two (2) feet. Generally, pier scour is a function of pier width. However, at this site, the incremental increase in impinging velocity for wider piers is not going to have any significant impact on fresher sandstone. Pier scour will remove only the significantly weathered sandstone material. Consequently, the same pier scour depth should be used, regardless of pier width. As a note, pile capacity is essentially derived from deeper sandstone. Therefore, deviations in actual pier scour will not have significant impact on the available pile capacity.

The alluvial soil overlying the sandstone at abutments will be protected by rock slope protection (RSP). RSP designed (size and thickness) for the anticipated design flood velocity and RSP slope angle should be placed on fresh sandstone. It is recommended a toe bench be excavated two (2) feet below the alluvial/sandstone contact prior to placing the RSP.

## 6.3 PILE FOUNDATIONS

### 6.3.1 Axial Capacity

Table 6.3-1 provides the estimated design and specified tip elevations for the 24-inch and 48-inch diameter CIDH piles. Pile capacity is based only on side friction. As a note, the specified tip elevation is based on the lowest estimated bedrock elevation at the pile perimeter. The elevation has been rounded down to the whole foot. Final pile tip elevations will be based on geologic confirmation of the bedrock elevation and rock competency at specific pile borings.

**TABLE 6.3-1**
**FOUNDATION RECOMMENDATIONS**

| Support | Pile | Cut-off Elevation (ft) | LRFD Service-I Limit State Load per Column (kips) | | Sp[1] | Required Factored Nominal Resistance per Pile (kips) | | | | | | Design Tip Elevation (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Total | Permanent | | Service Limit | | Strength Limit | | Extreme Event | | | |
| | | | | | | Comp. ($\phi$=0.5) | Tens. ($\phi$=0.5) | Comp. ($\phi$=0.7) | Tens. ($\phi$=0.7) | Comp. ($\phi$=1.0) | Tens. ($\phi$=1.0) | | |
| Abut 1 | 24” CIDH | 921.25 | 210 | 150 | 1” | 210 | 0 | 260 | 0 | 150 | 0 | 873 (a), 876 (a-I), 888 (a-II), 894 (c) | 873 |
| Bent 2 | 48” CIDH | 915 | 340 | 250 | 1” | 340 | 0 | 508 | 0 | 385 | 0 | 877 (a), 876 (a-I), 886 (a-II), 895 (c) | 876 |
| Bent 3 | 48” CIDH | 915 | 330 | 240 | 1” | 330 | 0 | 493 | 0 | 385 | 0 | 878 (a), 877 (a-I), 886 (a-II), 895 (c) | 877 |
| Abut 4 | 24” CIDH | 921.25 | 240 | 180 | 1” | 240 | 0 | 310 | 0 | 180 | 0 | 870 (a), 872 (a-I), 886 (a-II), 893 (c) | 870 |

Notes:
1. Total permissible support settlement.
2. Design tip elevations are controlled by: (a) Compression (Service Limit), (b) Tension (Service Limit), (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, (d) Lateral Load – to be determined by designer.
3. The specified tip elevation shall not be raised above the design tip elevations for tension, lateral, and tolerable settlement.
4. The design tip elevations assume long term degradation channel elevation of 903 feet.

### 6.3.2 Lateral Capacity

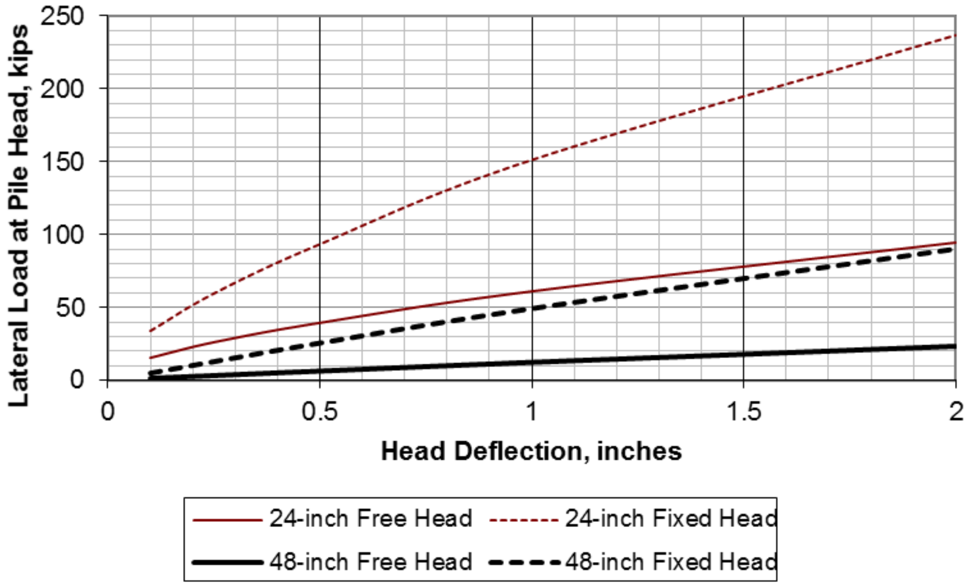
The lateral response of the foundations was evaluated using LPILE Plus Version 5.0 for Windows (computer software developed by Ensoft Inc.). Geotechnical parameters summarized in Table 6.3-2 is recommended for evaluation of lateral loading of piles at the abutments and bents. This data can be used by designers, if LPILE evaluation is to be performed.

**TABLE 6.3-2**
**SOIL INPUT PARAMETERS FOR PILE**

| Elevation (feet) | | p-y Curve | K (psi) | γ' (pci) | ϕ (˚) |
|---|---|---|---|---|---|
| From | To | | | | |
| 937 | 920 (Sandstone) | Sand | 90 | 0.072 | 35 |
| Below 920 (Sandstone) | | Sand | 200 | 0.036 | 42 |

Figure 6.3-1 presents the anticipated deflection of the pile head in response to a lateral load applied at the bridge deck connection elevation for the 24- and 48-inch CIDH piles. Data are presented for both free and fixed head conditions. The bent piles were analyzed using the long-term degraded channel elevation of 903 feet. The abutment piles were analyzed with the pile head in the pile cap which is within the sandstone.

**FIGURE 6.3-1**
**LATERAL LOAD DEFLECTION CURVES**
**24- AND 48- INCH CIDH PILES**




## 6.3.3 Construction Considerations

It is anticipated the drilling of the CIDH piles in sandstone can be accomplished with normal drilling techniques. If conglomeratic shell beds are encountered, rock augers, core buckets, gads, or air-impact tools may be necessary.

If creek flow or ground water is encountered above the rock surface at the time of drilling, it will be necessary for the contractor to seal a segment of casing into the hole and pump out water. It is anticipated any seepage through the sandstone could be sufficiently managed (i.e., pumped out immediately prior to concrete pour) to allow for concrete placement. Alternatively, wet pile installation procedures could be used.

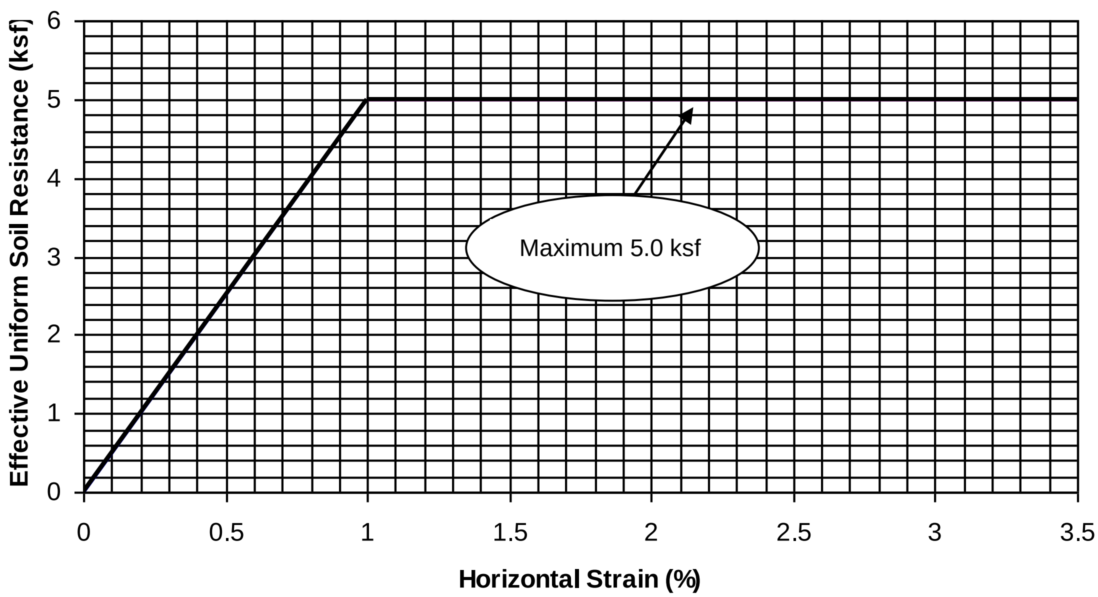
Any required open excavation of the material above the bedrock will encounter silty sand to sand. It is anticipated these materials should stand at an average gradient of about 1 to 1.5: 1 (H:V). If steeper gradients are necessary, initial planning by the contractor should anticipate some form of shoring. The contractor will have to assess the safety of final unsupported excavation side slopes based on the materials encountered and the exposure to workers, equipment or traffic.

### 6.4 ABUTMENT EVALUATION

#### 6.4.1 Abutment Dynamic Lateral Resistance

For backfill at abutments constructed in accordance with applicable provisions of the Caltrans Standard Specifications, an initial abutment soil stiffness of 50 kip/in/ft is recommended. The ultimate lateral resistance that may be applied against abutment to resist seismic loading will be dependent on the deflection that occurs (which mobilizes shear resistance in the soil). Figure 6.4-1 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 kips per square foot (ksf), when the height of the wall that is buried below the horizontal ground surface is equal to, or greater than, 5.5 feet. When the abutment height is less than 5.5 feet, the maximum equivalent uniform lateral soil resistance shall be reduced proportionately by H/5.5, where H is the endwall height in feet.

**FIGURE 6.4-1**
**UNFACTORED NOMINAL LATERAL BEARING FOR SEISMIC LOADING AT ABUTMENTS**



### 6.4.2 Active Lateral Earth Pressures

The alluvial soil above the sandstone at abutments is considered to be comparable to conditions associated with Caltrans standard design. Consequently Caltrans Standard Plan walls could be used at the site.

If site specific design is desired, Table 6.4-1 provides the recommended lateral earth pressures acting against cantilevered abutments and wing walls. The bottom of abutments are anticipated to be above potential ground water levels. Consequently, recommended values do not include hydrostatic considerations. Wall backfill should be adequately drained.

**TABLE 6.4-1**
**ABUTMENT LATERAL LOAD PARAMETERS**

| | |
|---|---|
| Estimated Friction Angle | 35˚ |
| Uniform Surcharge Coefficient ($k_a$) | 0.27 |
| Unfactored Active Earth Pressure | 34 psf/ft of depth |
| Unfactored Dynamic Increment | 14 psf/ft of depth |

Appropriate load factors should be applied to the active pressure. The factored dynamic increment would be added to the factored active pressure for Extreme Event consideration. A horizontal ground acceleration of 0.20g (one half the probabilistic PHGA) was used for calculation of the unfactored dynamic increment. The Caltrans approach to seismic design considers the distribution of the dynamic increment to be an upright triangle (similar to the active pressure). Consequently, the resultant load for the active pressure and dynamic increment would both be applied at 0.33 H from the base, where H is the total retained height.

# 7 CLOSURE

The conclusions and recommendations in this report are for the design of the El Camino Real Bridge Replacement over Santa Margarita 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, express or implied, is made. The field exploration program and this report were based on the proposed project information provided to Kleinfelder. If any change (i.e., structure type, location, etc.) is implemented which materially alters the project, additional geotechnical services may be required, which could include revisions to the recommendations given herein.

This report is intended for use by San Luis Obispo County, Quincy Engineering, Inc., and their subconsultants, within a reasonable time from its issuance. Noncompliance with the recommendations of the report or misuse of the report will release Kleinfelder from any liability.

The scope of the geotechnical services did not include an environmental site assessment for the presence or absence of hazardous/toxic materials in the soil, groundwater or atmosphere, or the presence of wetlands.

# PLATES

PLOTTED: 28 Sep 2015, 11:41pm, MGriffin

CAD FILE: C:\CADD\2015\20143900\SantaMargaritaCrk-Bridge_9-2015\ LAYOUT: 1

Images: Aerial-Image_SantaMargarita_3000_8-23-13.jpg


SOURCE: GOOGLE EARTH PRO 2014, DATED 8/23/13.

N

2,000 1,000 0 2,000

APPROXIMATE SCALE (feet)

The information included on this graphic representation has been compiled from a variety of sources and is subject to change without notice. Kleinfelder makes no representations or warranties, express or implied, as to accuracy, completeness, timeliness, or rights to the use of such information. This document is not intended for use as a land survey product nor is it designed or intended as a construction design document. The use or misuse of the information contained on this graphic representation is at the sole risk of the party using or misusing the information.

ATTACHED IMAGES:
ATTACHED XREFS:
LONG BEACH, CA

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| | |
|---|---|
| PROJECT NO. | 20143900 |
| DRAWN: | 9/2015 |
| DRAWN BY: | MRG |
| CHECKED BY: | NP |
| FILE NAME: | 20143900p1_SVM.dwg |

SITE VICINITY MAP

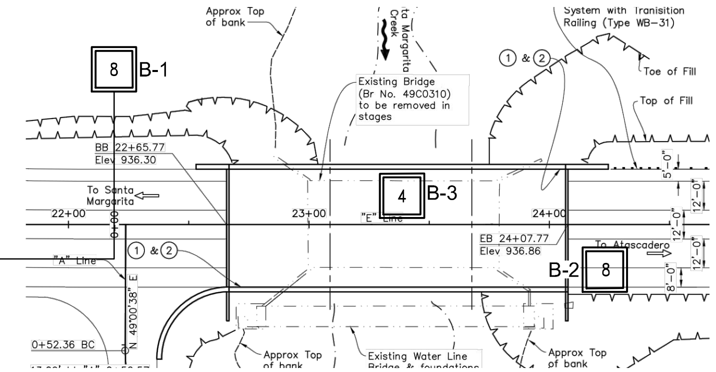
SANTA MARGARITA CREEK BRIDGE
EL CAMINO REAL
ATACADERO, CALIFORNIA

PLATE

1

## NOTES:

1. 1.4-INCH DIAMETER SAMPLES WERE TAKEN USING A STANDARD PENETRATION TEST (SPT) SPLIT BARREL SAMPLER WITH AN INSIDE DIAMETER (ID) OF 1.4 INCHES AND AN OUTSIDE DIAMETER (OD) OF 2.0 INCHES.
2. 2.4-INCH DIAMETER RING SAMPLES WERE TAKEN USING A CALIFORNIA SPLIT BARREL SAMPLER WITH AN ID OF 2.4 INCHES AND AN OD OF 3.0 INCHES.
3. ALL DRIVE SAMPLES WERE DRIVEN WITH 140 LB HAMMER WITH A FALLING HEIGHT OF 30 INCHES.




## PLAN

SCALE: 1"=40'

| DIST | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET No | TOTAL SHEETS |
|---|---|---|---|---|---|
| X | X | X | | | |

REGISTERED CIVIL ENGINEER X DATE

PLANS APPROVAL DATE

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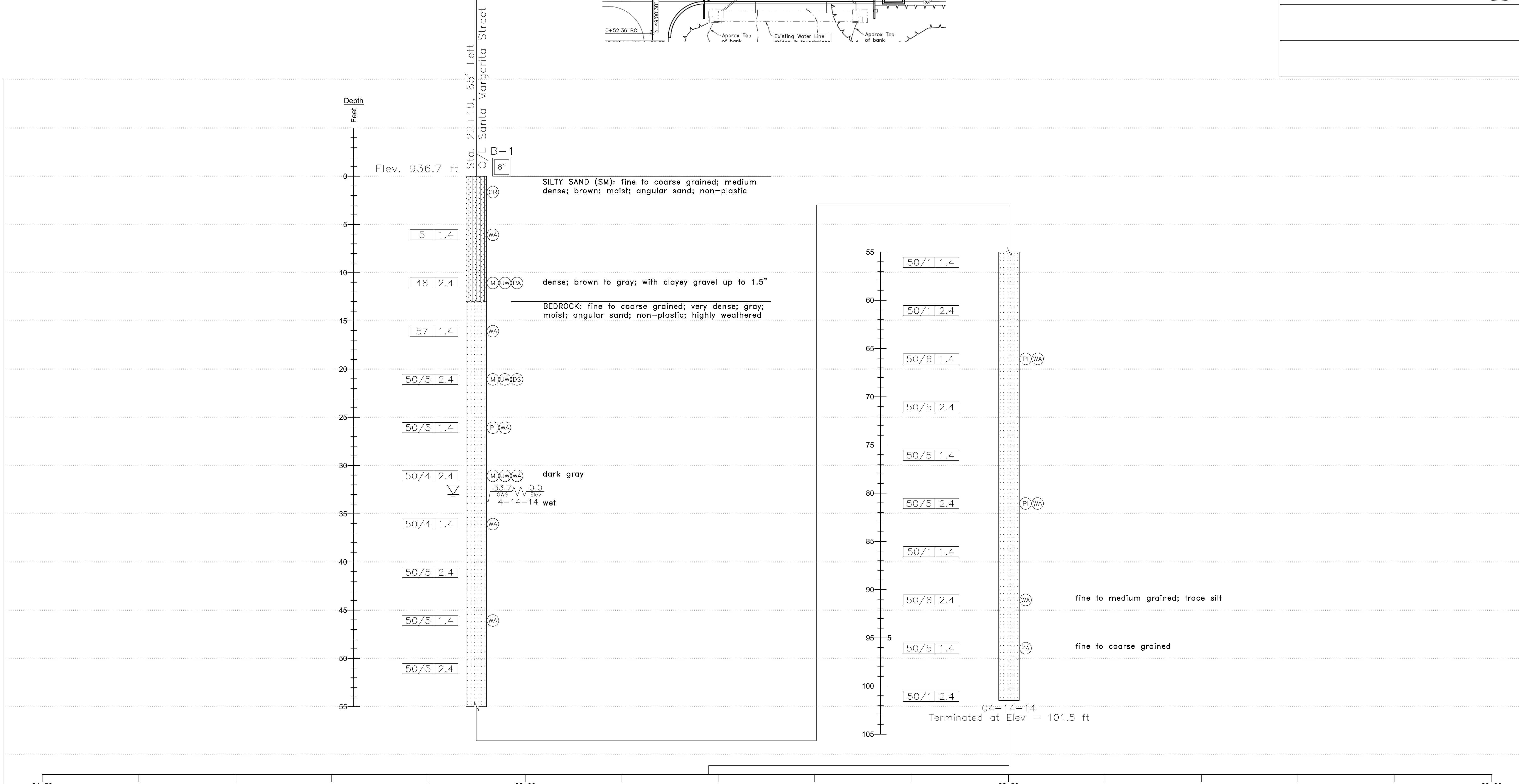
REGISTERED PROFESSIONAL ENGINEER Stephen P. Paulson No. 2731 Exp. 09/30/21 GEOTECHNICAL STATE OF CALIFORNIA




SANTA MARGARITA STREET

## PROFILE

SCALE: 1"=10' HORIZONTAL
SCALE: 1"=5' VERTICAL

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| | |
|---|---|
| PROJECT NO. | 20210004 |
| DRAWN: | 04/2020 |
| DRAWN BY: | MRG |
| CHECKED BY: | TD |
| FILE NAME: | 20143900p2-3_LOTB-SM.dwg |

**LOG OF TEST BORINGS**

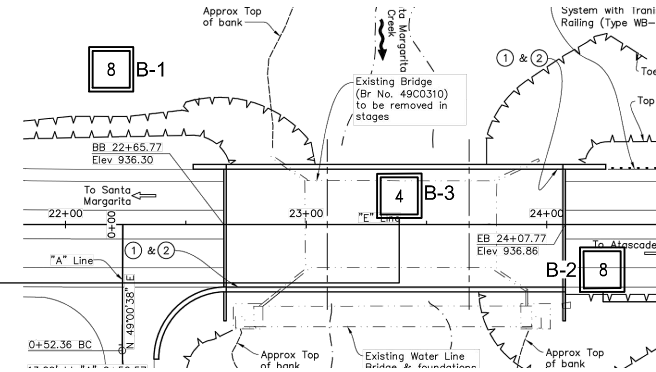
SANTA MARGARITA CREEK BRIDGE
EL CAMINO REAL
ATACADERO, CALIFORNIA

PLATE 2

PLOTTED: 03 Apr 2020, 9:01am, DFahrney
CAD FILE: U:\Projects\CADD\CADD 2021\20210004\SantaMargaritaCrk-Bridge 9-2015\ LAYOUT: 2
ATTACHED IMAGES: Images: Draft_PlanBridgeView_9-28-15.png
ATTACHED XREFS:
RIVERSIDE, CA

NOTES:

1. 1.4-INCH DIAMETER SAMPLES WERE TAKEN USING A STANDARD PENETRATION TEST (SPT) SPLIT BARREL SAMPLER WITH AN INSIDE DIAMETER (ID) OF 1.4 INCHES AND AN OUTSIDE DIAMETER (OD) OF 2.0 INCHES.
2. 2.4-INCH DIAMETER RING SAMPLES WERE TAKEN USING A CALIFORNIA SPLIT BARREL SAMPLER WITH AN ID OF 2.4 INCHES AND AN OD OF 3.0 INCHES.
3. ALL DRIVE SAMPLES WERE DRIVEN WITH 140 LB HAMMER WITH A FALLING HEIGHT OF 30 INCHES.




## PLAN

SCALE: 1"=40'

| DIST | COUNTY | ROUTE | POST MILES TOTAL PROJECT | SHEET No | TOTAL SHEETS |
|---|---|---|---|---|---|
| X | X | X | | | |

REGISTERED CIVIL ENGINEER X DATE

PLANS APPROVAL DATE

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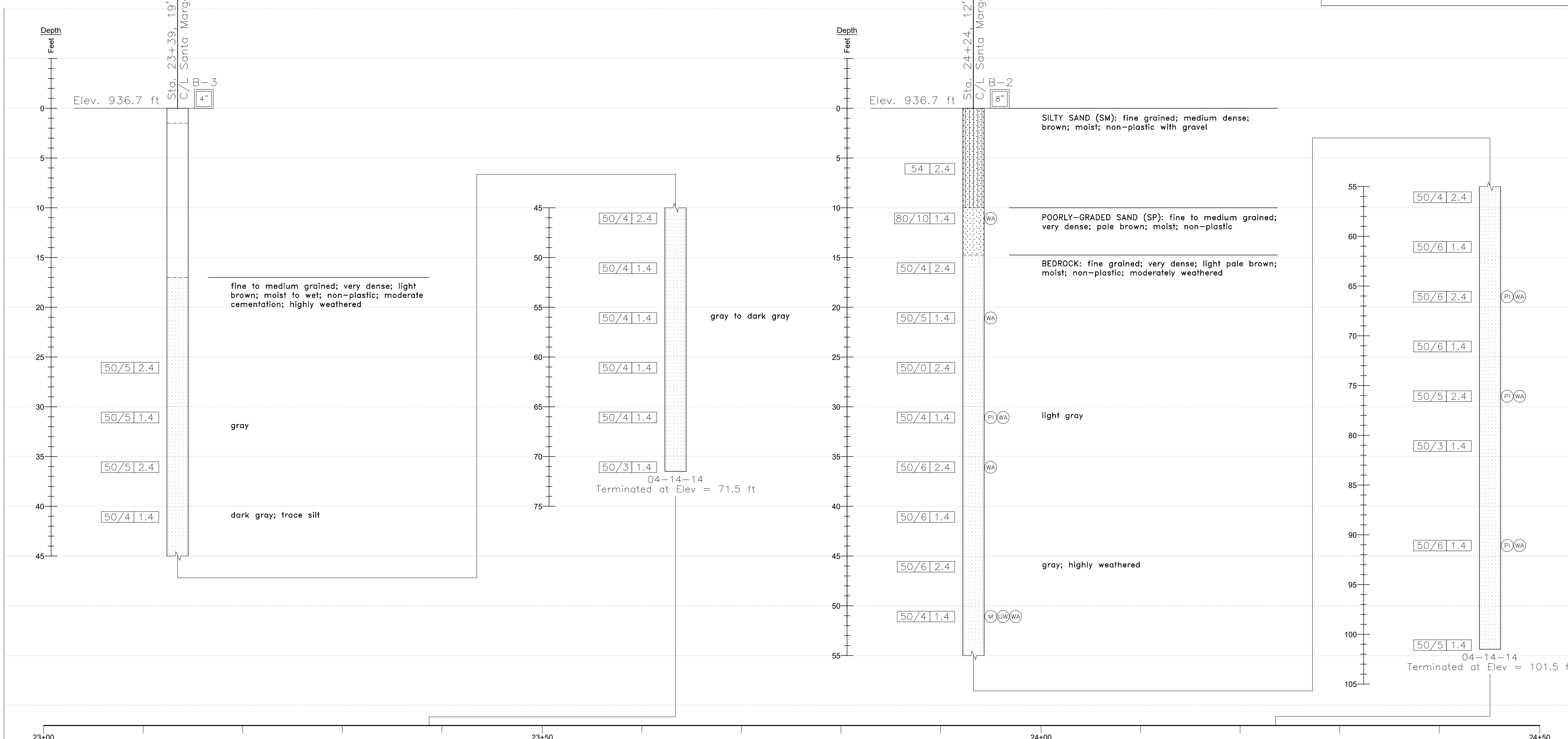
REGISTERED PROFESSIONAL ENGINEER Stephen P. Paulson No. 2731 Exp. 09/30/21 GEOTECHNICAL STATE OF CALIFORNIA




SANTA MARGARITA STREET

## PROFILE

SCALE: 1"=10' HORIZONTAL
SCALE: 1"=5' VERTICAL

PLOTTED: 03 Apr 2020, 9:01am, DFahrney LAYOUT: 3

CAD FILE: U:\Projects\CADD\CADD 2021\20210004\SantaMargaritaCrk-Bridge 9-2015\

ATTACHED IMAGES: Images: Draft_PlanBridgeView_9-28-15.png
ATTACHED XREFS:
RIVERSIDE, CA

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| | |
|---|---|
| PROJECT NO. | 20210004 |
| DRAWN: | 04/2020 |
| DRAWN BY: | MRG |
| CHECKED BY: | TD |
| FILE NAME: | 20143900p2-3_LOTB-SM.dwg |

**LOG OF TEST BORINGS**

SANTA MARGARITA CREEK BRIDGE
EL CAMINO REAL
ATACADERO, CALIFORNIA

PLATE
3

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# APPENDIX A

| Exploration ID | Depth (ft.) | Sample Description | Water Content (%) | Dry Unit Wt. (pcf) | Sieve Analysis (%) | | | Atterberg Limits | | | Additional Tests |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Passing 3/4" | Passing #4 | Passing #200 | Liquid Limit | Plastic Limit | Plasticity Index | |
| B-2A | 0.0 | SILTY SAND (SM) | | | | | | | | | **pH=** 6.39 |
| | | | | | | | | | | | **Resistivity=** 2466.2 |
| | | | | | | | | | | | **Sulfates=** 160 |
| | | | | | | | | | | | **Chlorides=** 27 |
| B-2A | 5.0 | SILTY SAND (SM) | | | | | 30 | NP | NP | NP | |
| B-2A | 10.0 | SILTY SAND (SM) | 9.4 | 123.2 | | 90 | 24 | | | | |
| B-2A | 15.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 18 | | | | |
| B-2A | 20.0 | BEDROCK: SANTA MARGARITA SANDSTONE | 14.0 | 110.1 | | | | | | | **Direct Shear=** |
| | | | | | | | | | | | Peak Cohesion: 200 psf |
| | | | | | | | | | | | Peak Friction Angle: 42.0° |
| B-2A | 25.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 23 | 32 | 19 | 13 | |
| B-2A | 30.0 | BEDROCK: SANTA MARGARITA SANDSTONE | 12.9 | | | | 14 | | | | |
| B-2A | 35.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 26 | | | | |
| B-2A | 45.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 23 | | | | |
| B-2A | 65.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 33 | 32 | 15 | 17 | |
| B-2A | 80.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 25 | 26 | 18 | 8 | |
| B-2A | 90.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 38 | | | | |
| B-2A | 95.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | 100 | 24 | | | | |
| B-2B | 10.0 | POORLY-GRADED SAND (SP) | | | | | 19 | | | | |
| B-2B | 20.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 34 | | | | |
| B-2B | 30.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 42 | 25 | 20 | 5 | |
| B-2B | 35.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 23 | | | | |
| B-2B | 50.0 | BEDROCK: SANTA MARGARITA SANDSTONE | 10.9 | 123.3 | | | 22 | NP | NP | NP | |
| B-2B | 65.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 20 | 25 | 23 | 2 | |
| B-2B | 75.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 22 | 26 | 23 | 3 | |
| B-2B | 90.0 | BEDROCK: SANTA MARGARITA SANDSTONE | | | | | 25 | 25 | 18 | 7 | |

Refer to the Geotechnical Evaluation Report or the supplemental plates for the method used for the testing performed above.
NP = NonPlastic

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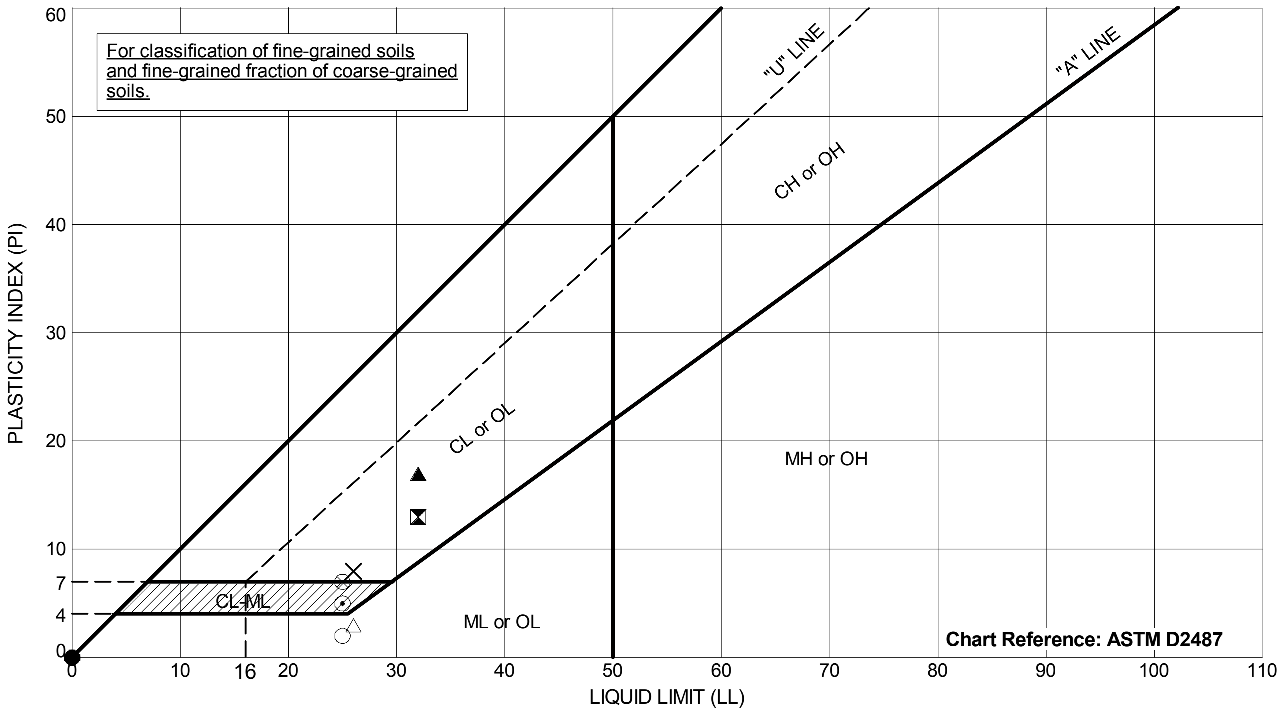
PROJECT NO.: 20143900
DRAWN BY: T. DESOUZA
CHECKED BY: N. POPENOE
DATE: 7/30/2014
REVISED: -

LABORATORY TEST RESULT SUMMARY

EL CAMINO REAL BRIDGE OVER SANTA MARGARITA CREEK BRIDGE
SAN LUIS OBISPO COUNTY, CALIFORNIA

TABLE
A-1




| | Exploration ID | Depth (ft.) | Sample Description | Passing #200 | LL | PL | PI |
|---|---|---|---|---|---|---|---|
| ● | B-2A | 5 | SILTY SAND (SM) | 30 | NP | NP | NP |
| ⧗ | B-2A | 25 | BEDROCK: SANTA MARGARITA SANDSTONE | 23 | 32 | 19 | 13 |
| ▲ | B-2A | 65 | BEDROCK: SANTA MARGARITA SANDSTONE | 33 | 32 | 15 | 17 |
| ✕ | B-2A | 80 | BEDROCK: SANTA MARGARITA SANDSTONE | 25 | 26 | 18 | 8 |
| ⊙ | B-2B | 30 | BEDROCK: SANTA MARGARITA SANDSTONE | 42 | 25 | 20 | 5 |
| ✿ | B-2B | 50 | BEDROCK: SANTA MARGARITA SANDSTONE | 22 | NP | NP | NP |
| ○ | B-2B | 65 | BEDROCK: SANTA MARGARITA SANDSTONE | 20 | 25 | 23 | 2 |
| △ | B-2B | 75 | BEDROCK: SANTA MARGARITA SANDSTONE | 22 | 26 | 23 | 3 |
| ⊗ | B-2B | 90 | BEDROCK: SANTA MARGARITA SANDSTONE | 25 | 25 | 18 | 7 |

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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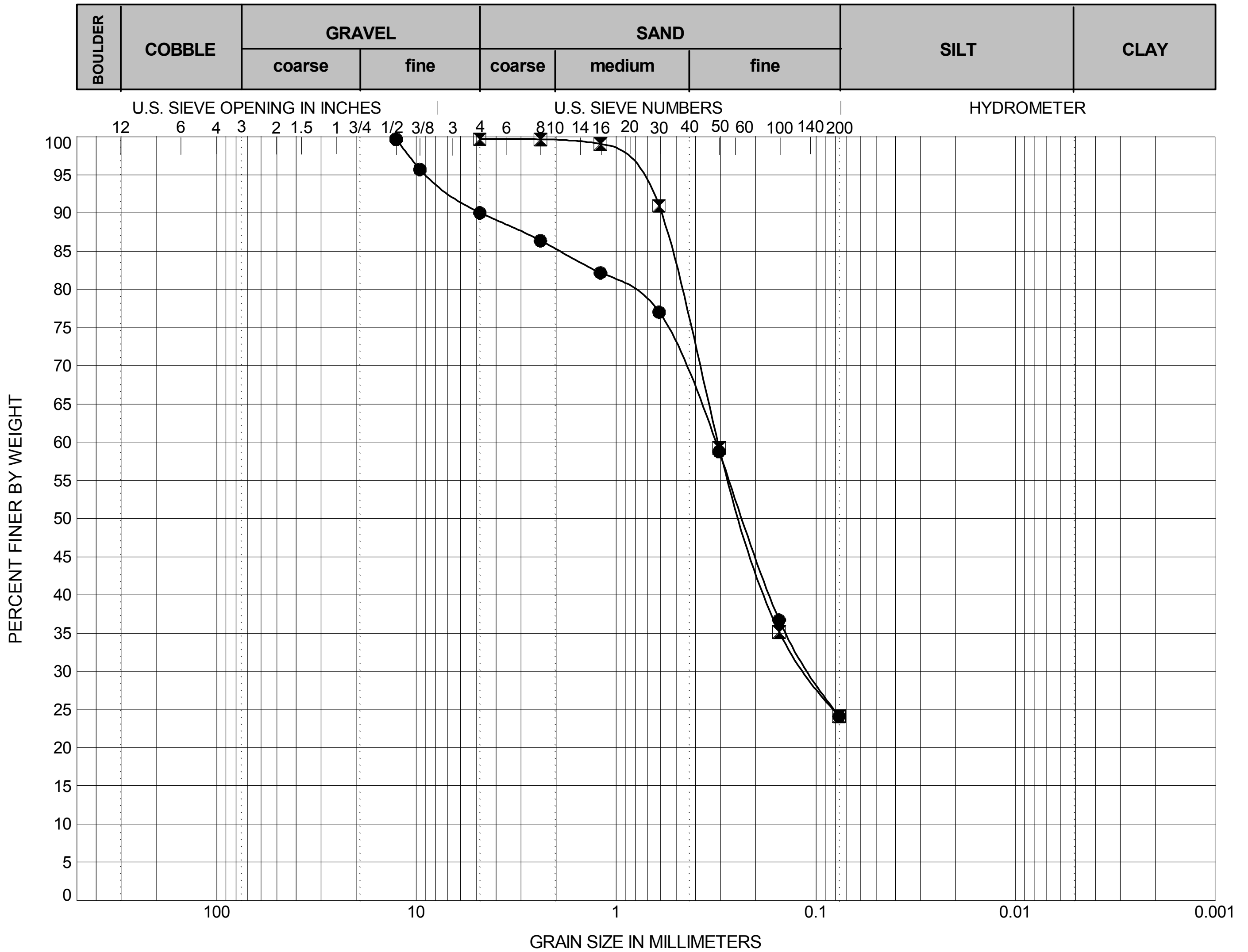
PROJECT NO.: 20143900
DRAWN BY: TD
CHECKED BY: RS
DATE: 5/14/2014
REVISED: -

ATTERBERG LIMITS

EL CAMINO REAL BRIDGE OVER
SANTA MARGARITA CREEK BRIDGE
SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
A-2




| Exploration ID | | Depth (ft.) | Sample Description | LL | PL | PI |
|---|---|---|---|---|---|---|
| ● | B-2A | 10 | SILTY SAND (SM) | NM | NM | NM |
| ⧗ | B-2A | 95 | BEDROCK: SANTA MARGARITA SANDSTONE | NM | NM | NM |

| Exploration ID | | Depth (ft.) | $D_{100}$ | $D_{60}$ | $D_{30}$ | $D_{10}$ | Cc | Cu | Passing 3/4" | Passing #4 | Passing #200 | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ● | B-2A | 10 | 12.5 | 0.311 | 0.102 | NM | NM | NM | | 90 | 24 | NM | NM |
| ⧗ | B-2A | 95 | 4.75 | 0.303 | 0.107 | NM | NM | NM | | 100 | 24 | NM | NM |

Coefficients of Uniformity - $C_u = D_{60} / D_{10}$
Coefficients of Curvature - $C_C = (D_{30})^2 / D_{60} D_{10}$
$D_{60}$ = Grain diameter at 60% passing
$D_{30}$ = Grain diameter at 30% passing
$D_{10}$ = Grain diameter at 10% passing

Sieve Analysis and Hydrometer Analysis testing performed in general accordance with ASTM D422.
NP = Nonplastic
NM = Not Measured

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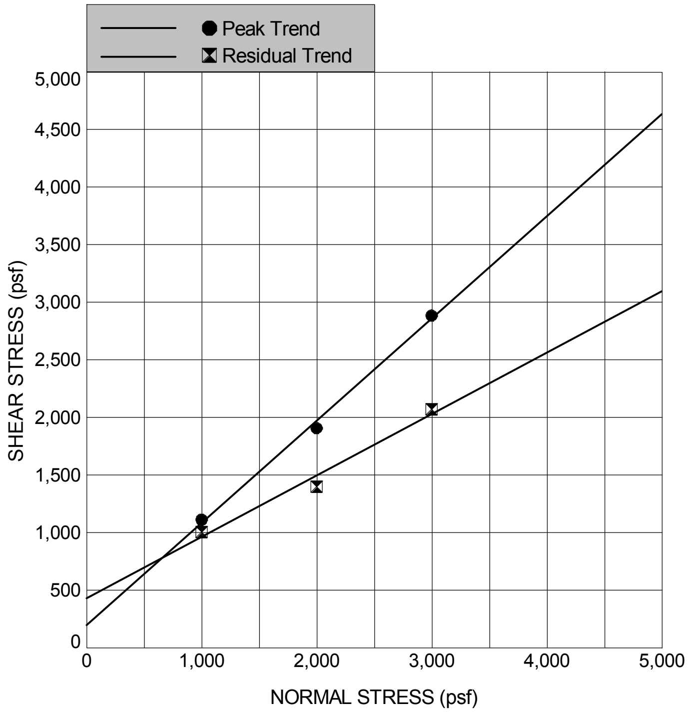
| | |
|---|---|
| PROJECT NO.: | 20143900 |
| DRAWN BY: | TD |
| CHECKED BY: | RS |
| DATE: | 5/14/2014 |
| REVISED: | - |

SIEVE ANALYSIS

EL CAMINO REAL BRIDGE OVER
SANTA MARGARITA CREEK BRIDGE
SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
A-3




| Exploration ID | Depth (ft.) | Sample Description |
|---|---|---|
| B-2A | 20 | BEDROCK: SANTA MARGARITA SANDSTONE |

| Passing #4 (%) | Passing #200 (%) | Liquid Limit | Plastic Limit | Plasticity Index | Specific Gravity |
|---|---|---|---|---|---|
| NM | NM | NM | NM | NM | |

| | Specimen No. | Water Content (%) | Dry Unit Weight (pcf) | Saturation (%) | Void Ratio | Area ($in^2$) | Height (in) |
|---|---|---|---|---|---|---|---|
| Initial | 1 | 14.0 | 105.8 | | | 4.60 | 1.00 |
| | 2 | 14.0 | 105.3 | | | 4.60 | 1.00 |
| | 3 | 14.0 | 107.7 | | | 4.60 | 1.00 |

| | Specimen No. | Water Content (%) | Dry Unit Weight (pcf) | Saturation (%) | Void Ratio | Area ($in^2$) | Height (in) |
|---|---|---|---|---|---|---|---|
| At Test | 1 | | | | | | |
| | 2 | | | | | | |
| | 3 | | | | | | |

| Specimen No. | Peak Shear Stress (psf) | Residual Shear Stress (psf) | Horizontal Displacement (in) | Normal Stress (psf) | Strain Rate (in/min) |
|---|---|---|---|---|---|
| 1 | 1123.08 | 1018.82 | 0.4800 | 1000 | 0.005 |
| 2 | 1919.66 | 1412.46 | 0.4800 | 2000 | 0.005 |
| 3 | 2897.6 | 2085.7 | 0.4800 | 3000 | 0.005 |

| Results | Cohesion (psf) | Friction ϕ (deg) | Tan ϕ (deg) |
|---|---|---|---|
| **Peak** | 200 | 42 | |
| **Residual** | | | |

Testing perfomed in general accordance with ASTM D3080.
NP = Nonplastic
NM = Not Measured

| | | | |
|---|---|---|---|
| KLEINFELDER Bright People. Right Solutions. | PROJECT NO.: 20143900<br>DRAWN BY: TD<br>CHECKED BY: RS<br>DATE: 5/14/2014<br>REVISED: - | DIRECT SHEAR<br><br>EL CAMINO REAL BRIDGE OVER SANTA MARGARITA CREEK BRIDGE SAN LUIS OBISPO COUNTY, CALIFORNIA | PLATE<br>A-4 |



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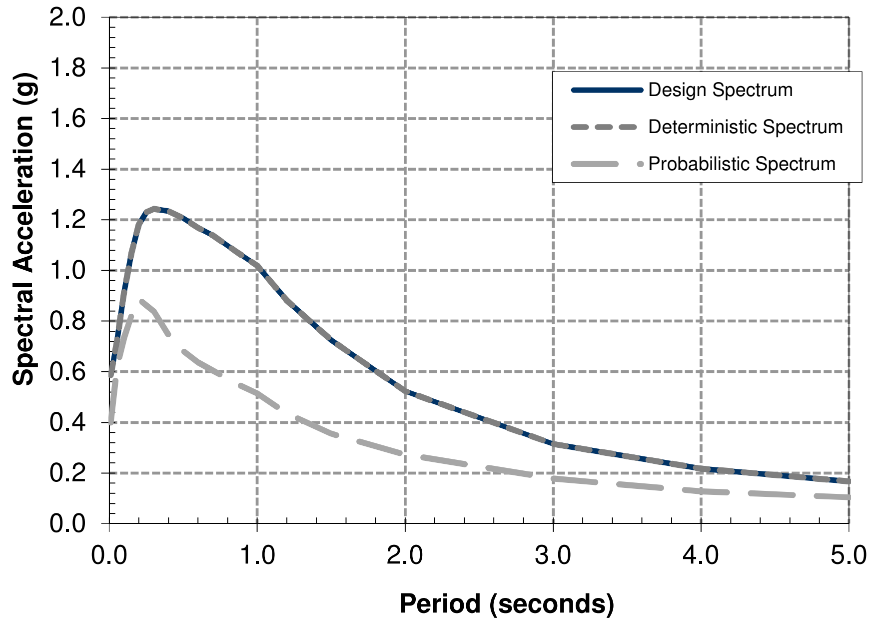
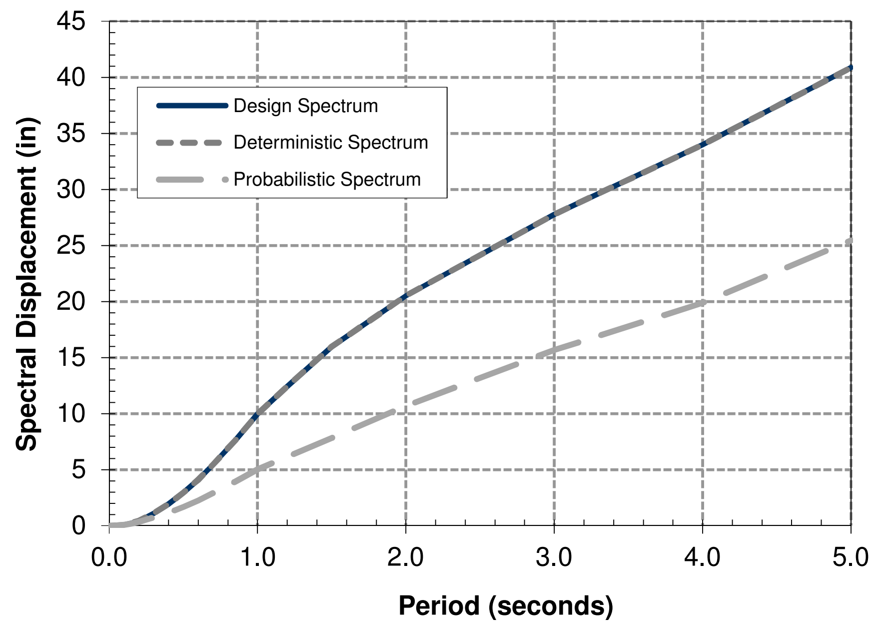
# APPENDIX B

## SITE DATA

| | | | |
|---|---|---|---|
| Latitude: | 35.4287 | Shear Wave Velocity | 328 m/s |
| Longitude: | -120.6058 | Depth to Vs = 1.0 km/s: | N/A |
| | | Depth to Vs = 2.5 km/s: | N/A |

| Period (s) | SA (g) | SD (in) |
|---|---|---|
| 0.01 (PGA) | 0.587 | 0.00 |
| 0.05 | 0.714 | 0.02 |
| 0.1 | 0.914 | 0.09 |
| 0.15 | 1.069 | 0.24 |
| 0.2 | 1.181 | 0.46 |
| 0.25 | 1.230 | 0.75 |
| 0.3 | 1.243 | 1.09 |
| 0.4 | 1.234 | 1.93 |
| 0.5 | 1.206 | 2.95 |
| 0.6 | 1.168 | 4.12 |
| 0.7 | 1.138 | 5.46 |
| 0.85 | 1.078 | 7.62 |
| 1 | 1.019 | 9.97 |
| 1.2 | 0.881 | 12.42 |
| 1.5 | 0.725 | 15.97 |
| 2 | 0.524 | 20.51 |
| 3 | 0.315 | 27.75 |
| 4 | 0.217 | 33.98 |
| 5 | 0.167 | 40.86 |

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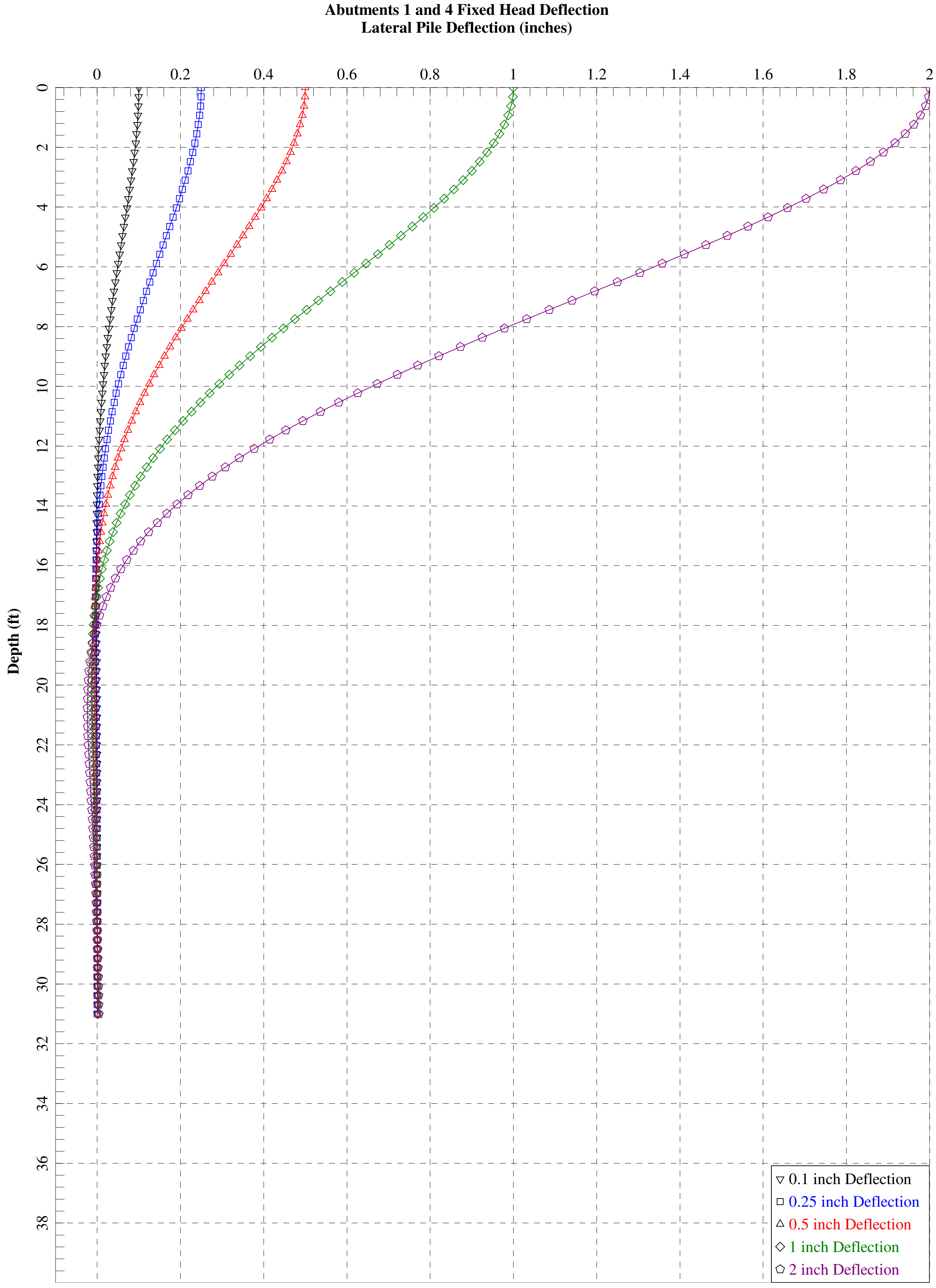
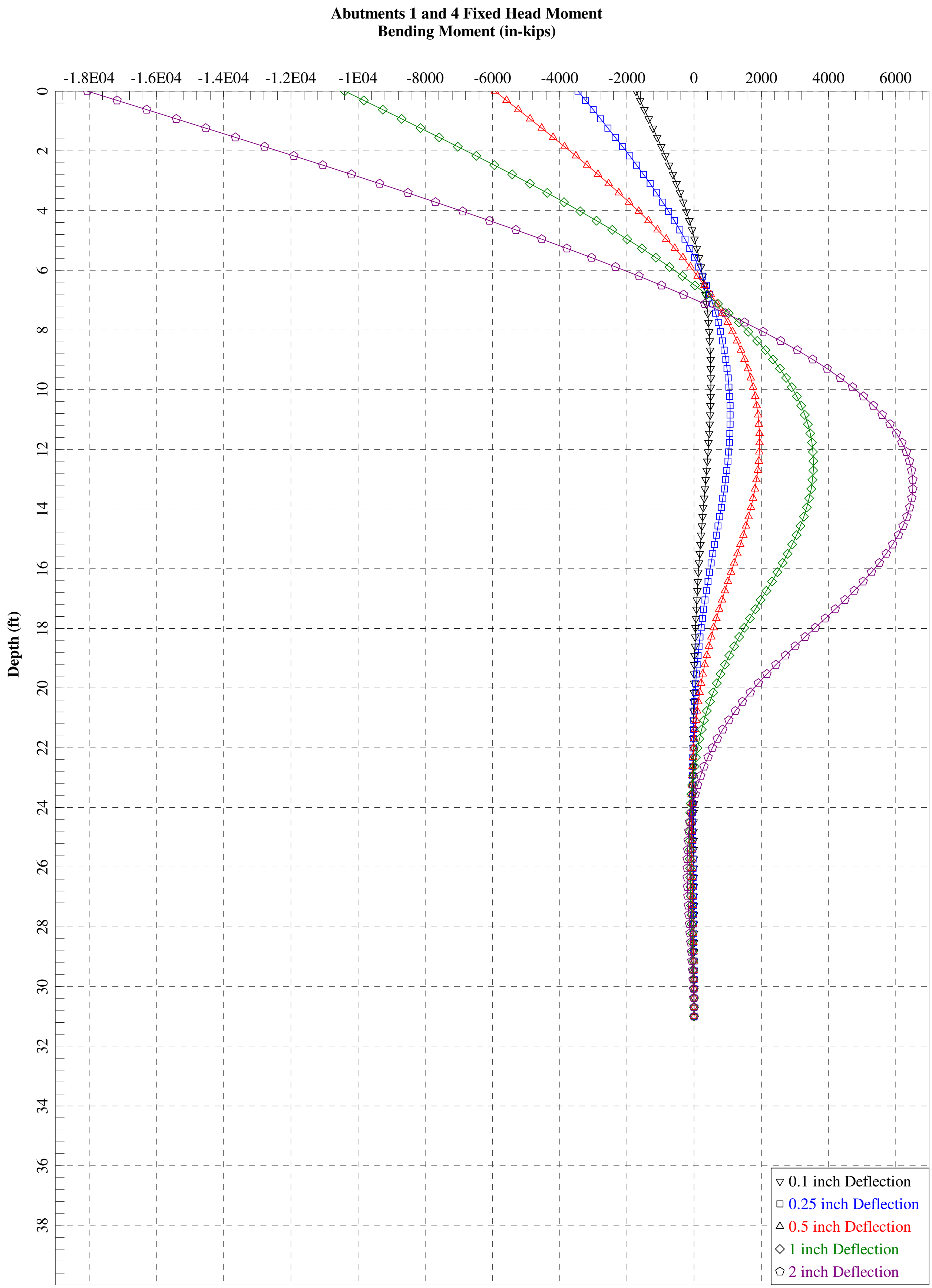
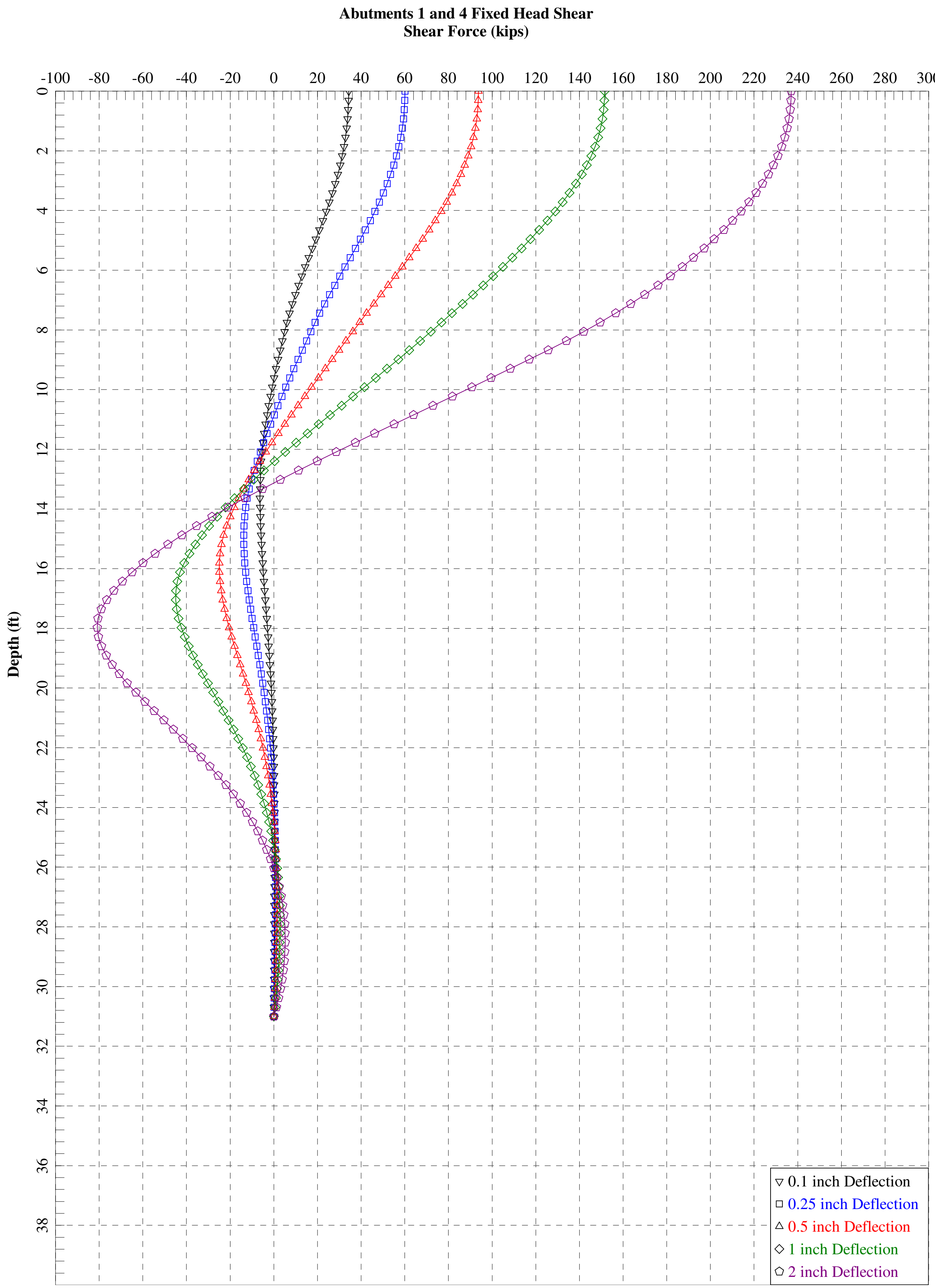
| | | | | |
|---|---|---|---|---|
| PROJECT NO | 20143900 | **CALTRANS ARS CURVES** | PLATE |
| DRAWN: | 9/9/15 | | |
| DRAWN BY: | B. Campbell | **SANTA MARGARITA CREEK BRIDGE**<br>EL CAMINO REAL<br>SAN LUIS OBISPO COUNTY, CA | **B-1** |
| CHECKED BY: | | | |
| FILE NAME: | | | |

# APPENDIX C

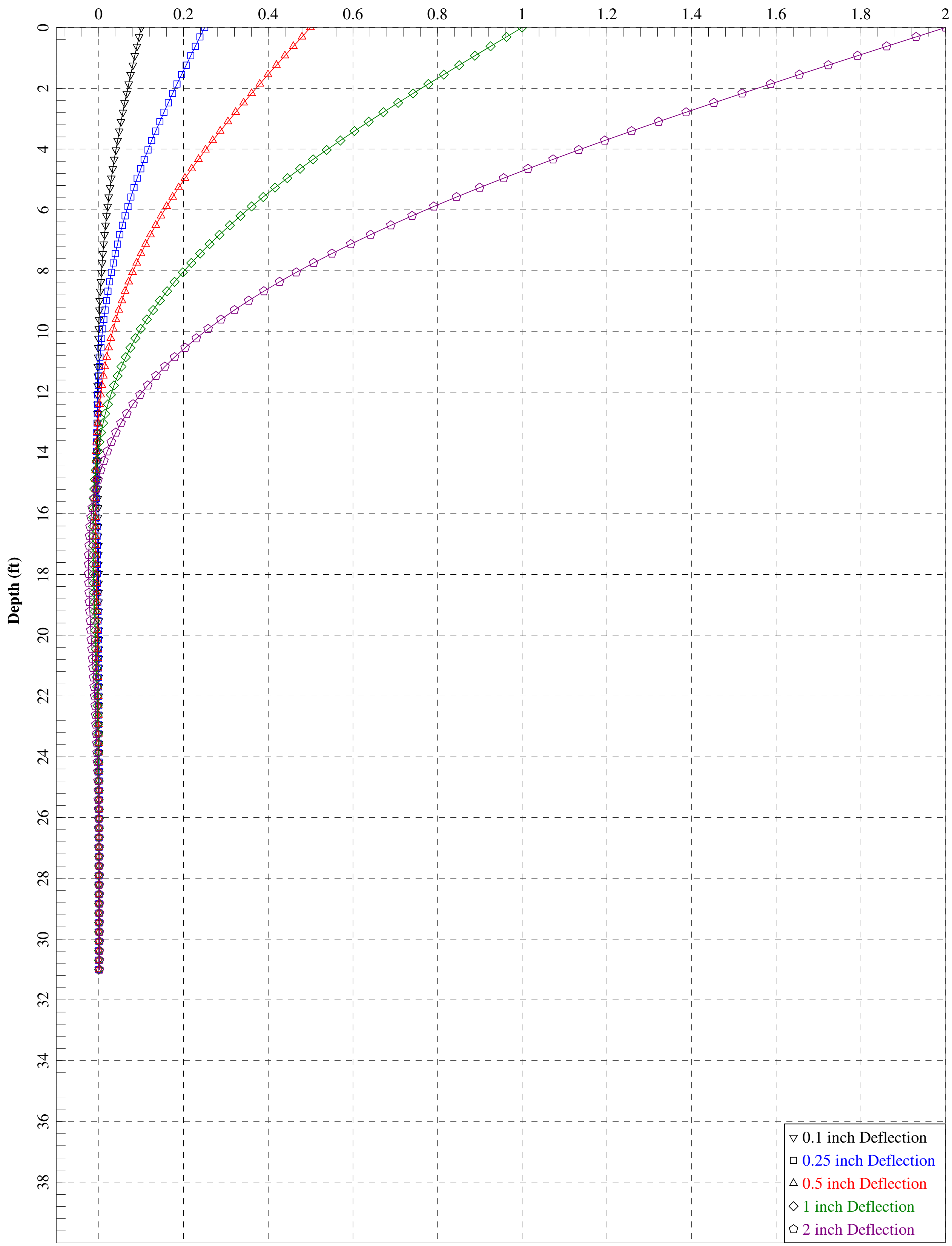

Abutments 1 and 4 Fixed Head Deflection
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

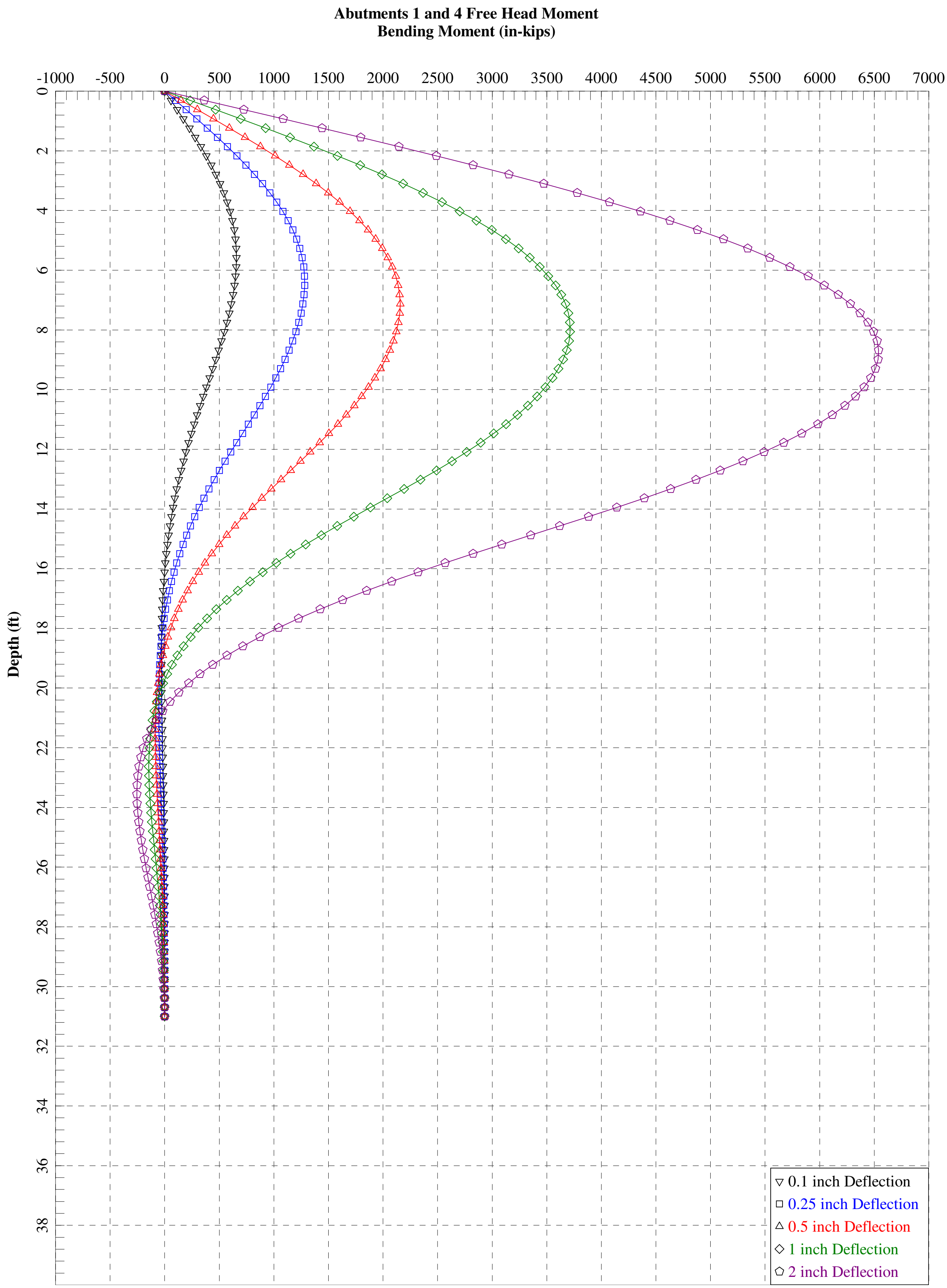

Abutments 1 and 4 Fixed Head Moment
Bending Moment (in-kips)
-1.8E04
-1.6E04
-1.4E04
-1.2E04
-1E04
-8000
-6000
-4000
-2000
0
2000
4000
6000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

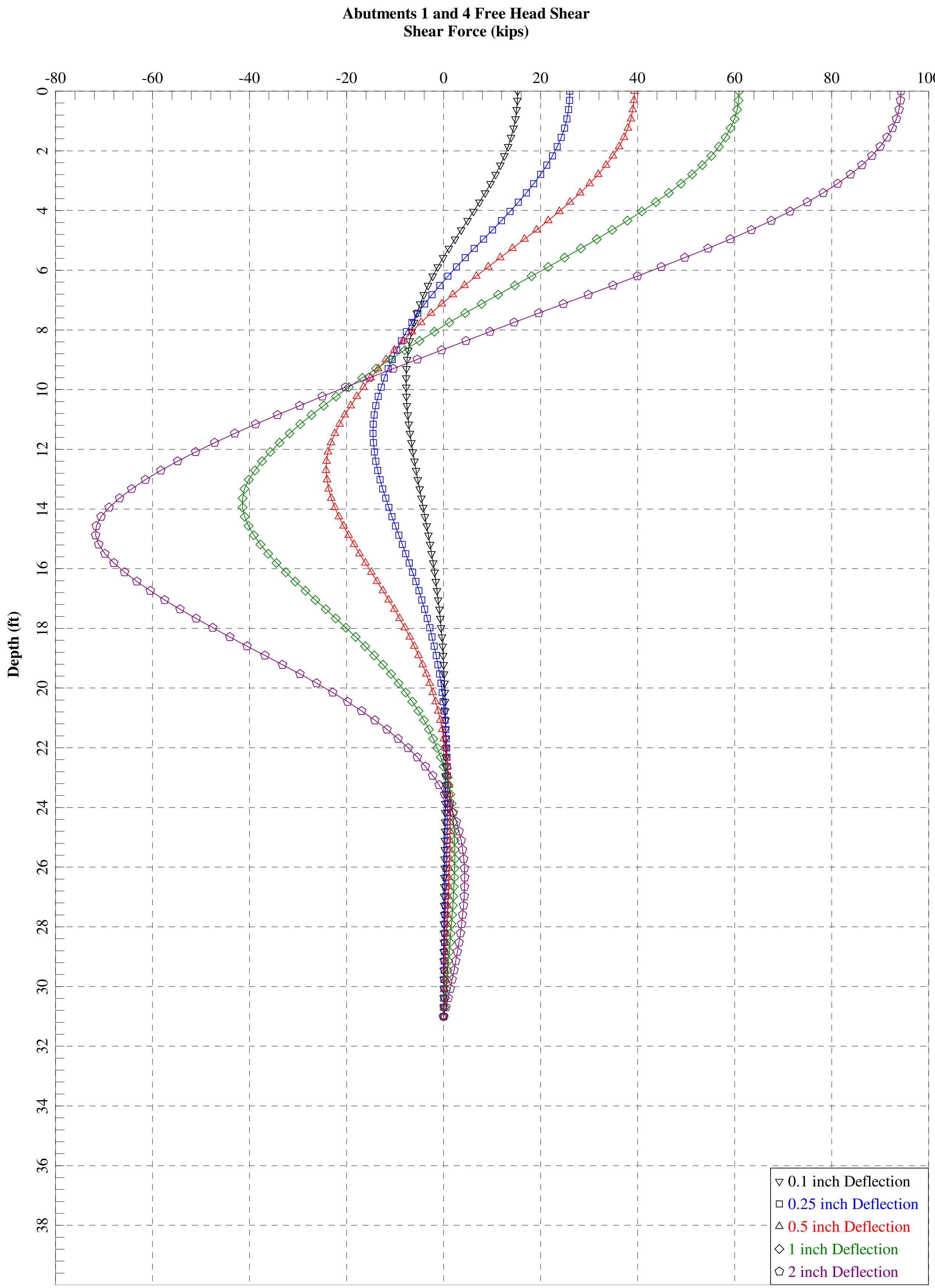


Abutments 1 and 4 Fixed Head Shear
Shear Force (kips)
-100 -80 -60 -40 -20 0 20 40 60 80 100 120 140 160 180 200 220 240 260 280 300
Depth (ft)
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

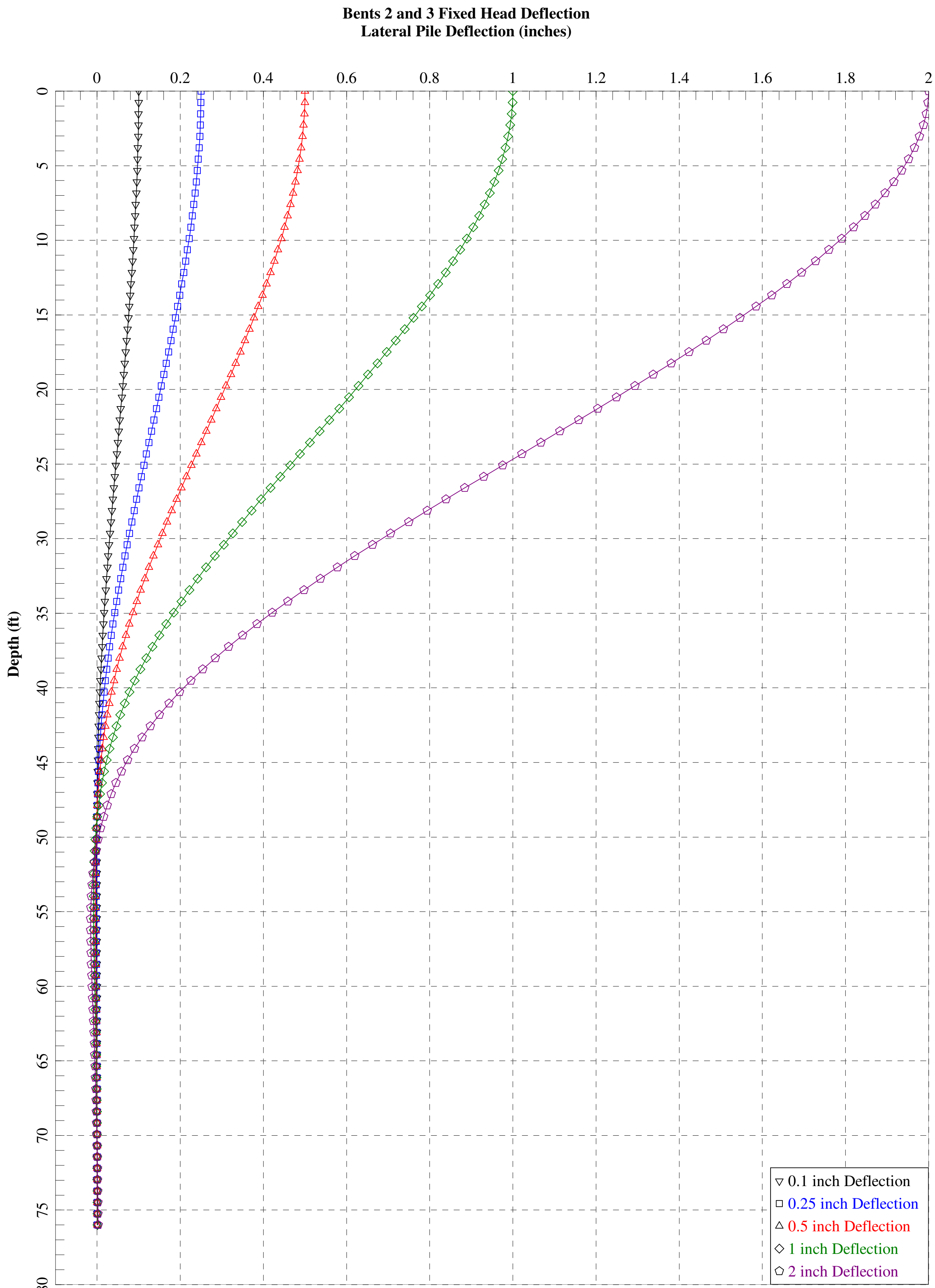
Abutments 1 and 4 Free Head Deflection
Lateral Pile Deflection (inches)

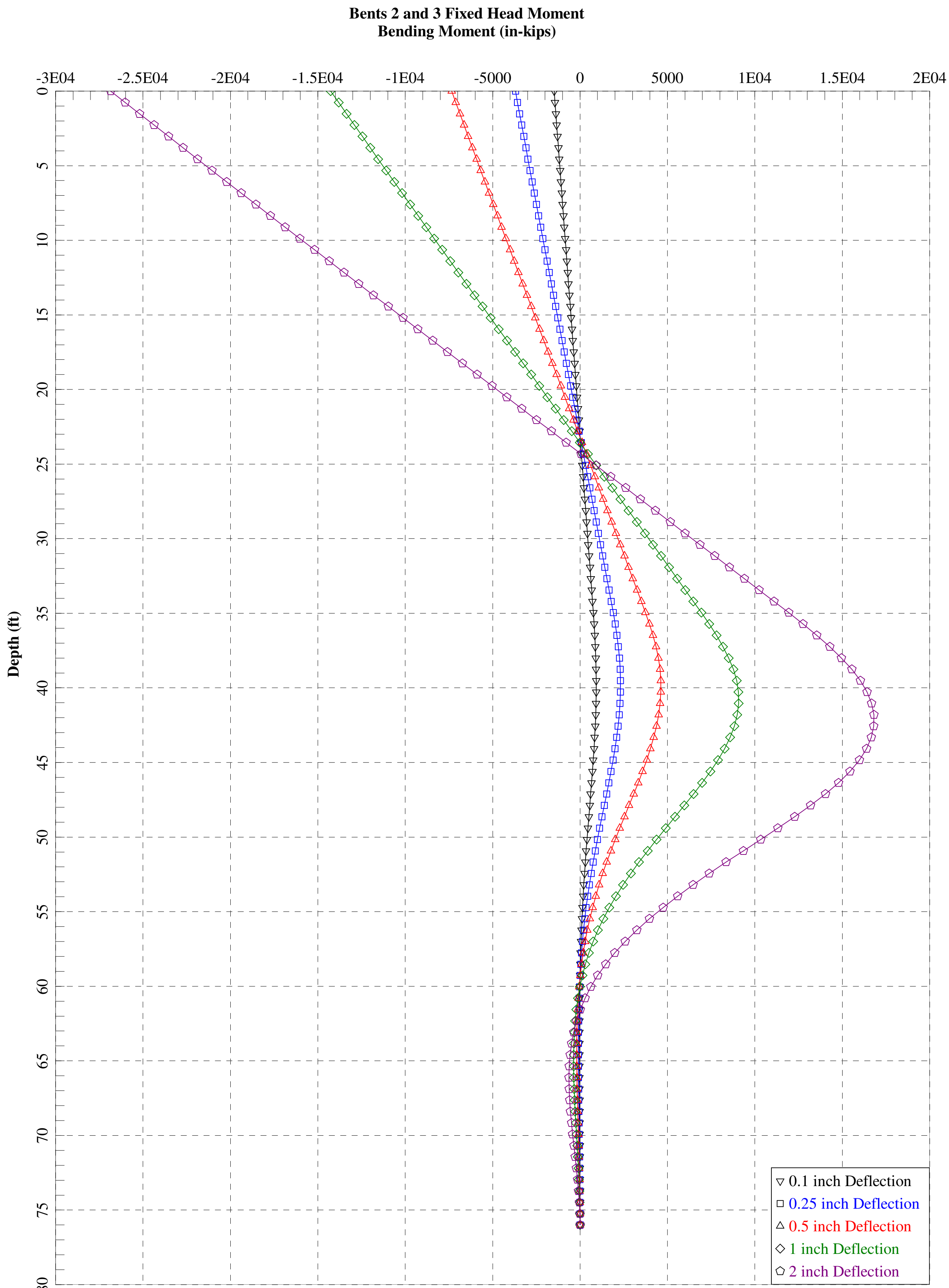



Abutments 1 and 4 Free Head Moment
Bending Moment (in-kips)
-1000
-500
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
5500
6000
6500
7000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

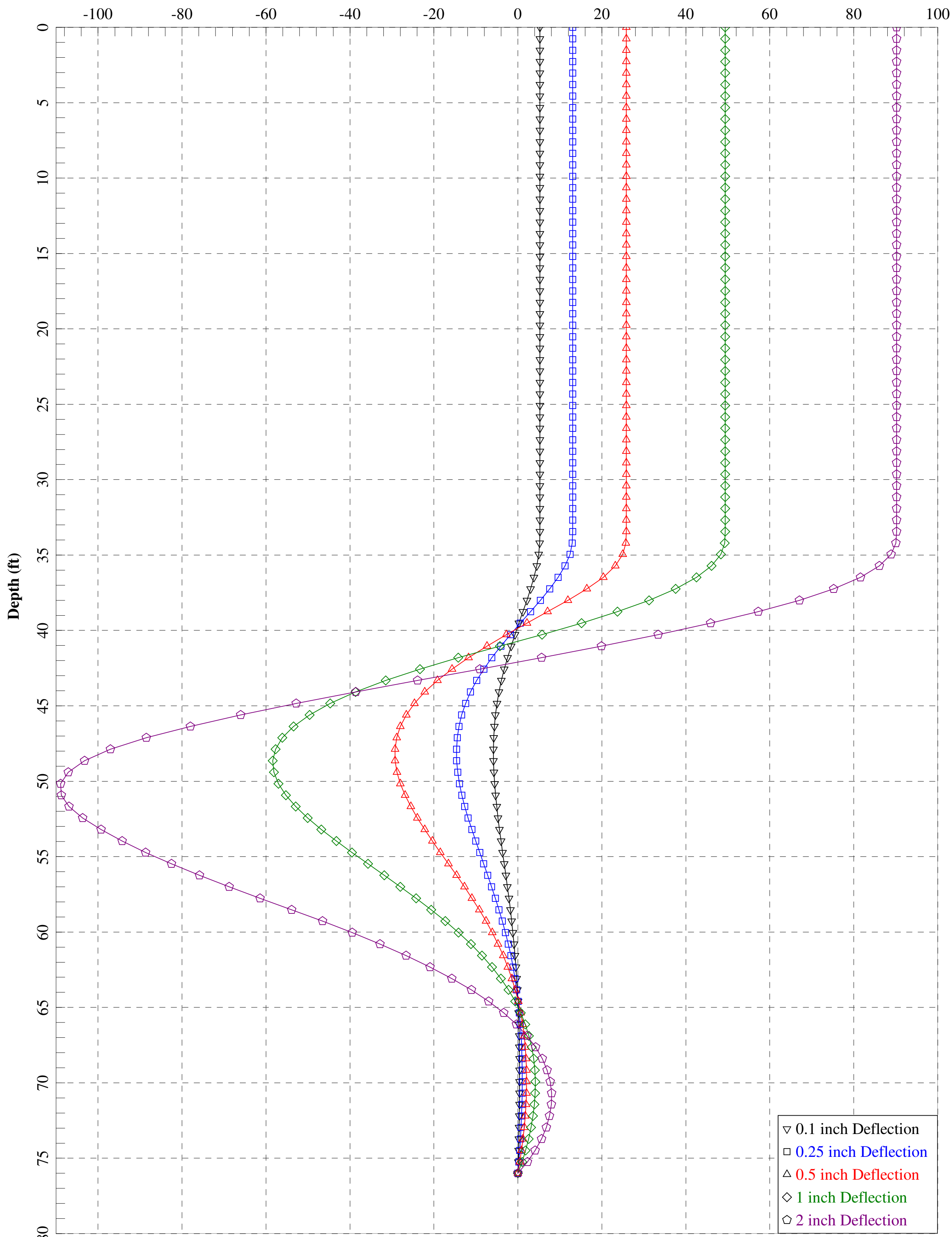


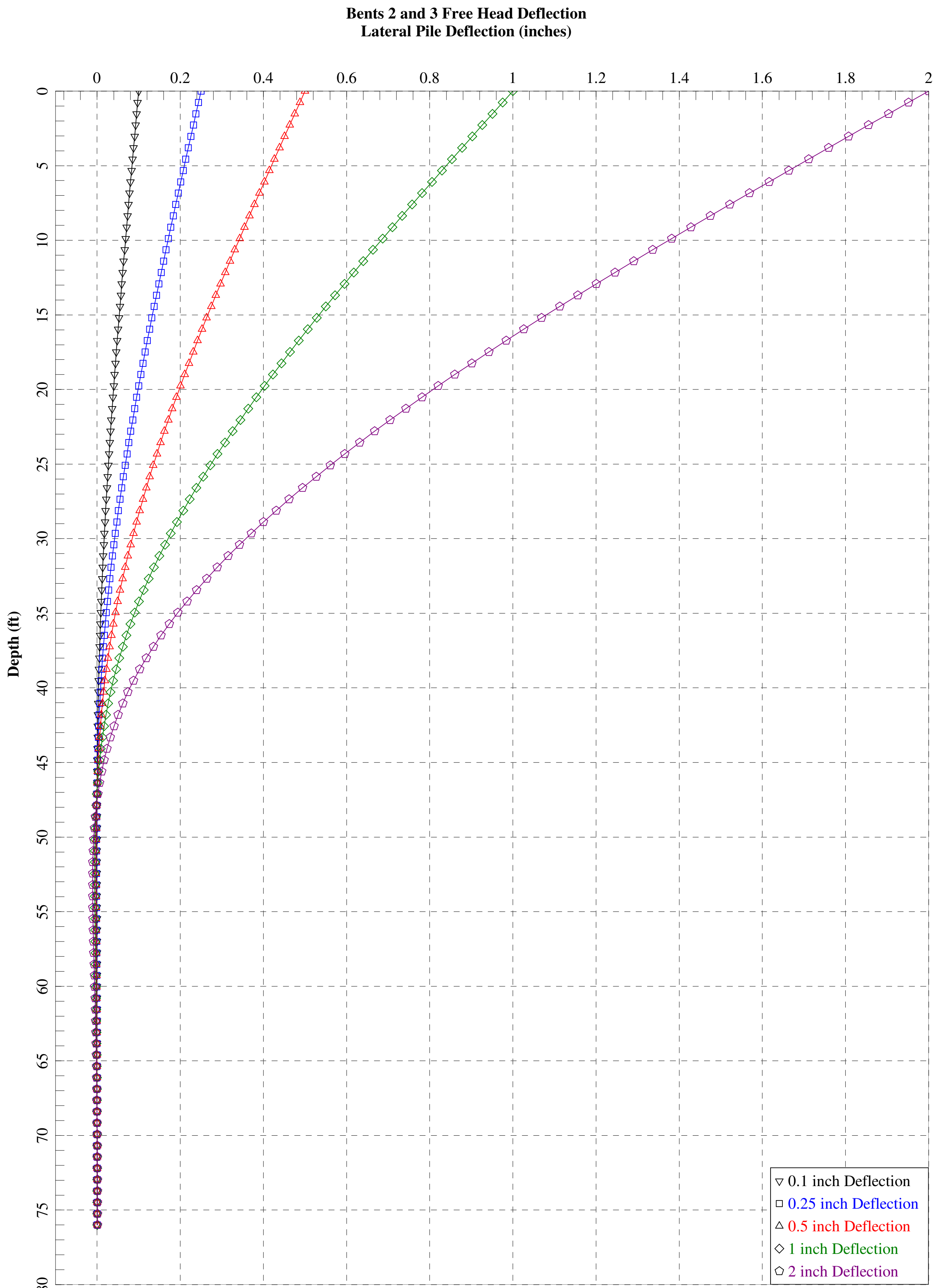
Abutments 1 and 4 Free Head Shear
Shear Force (kips)
-80
-60
-40
-20
0
20
40
60
80
100
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

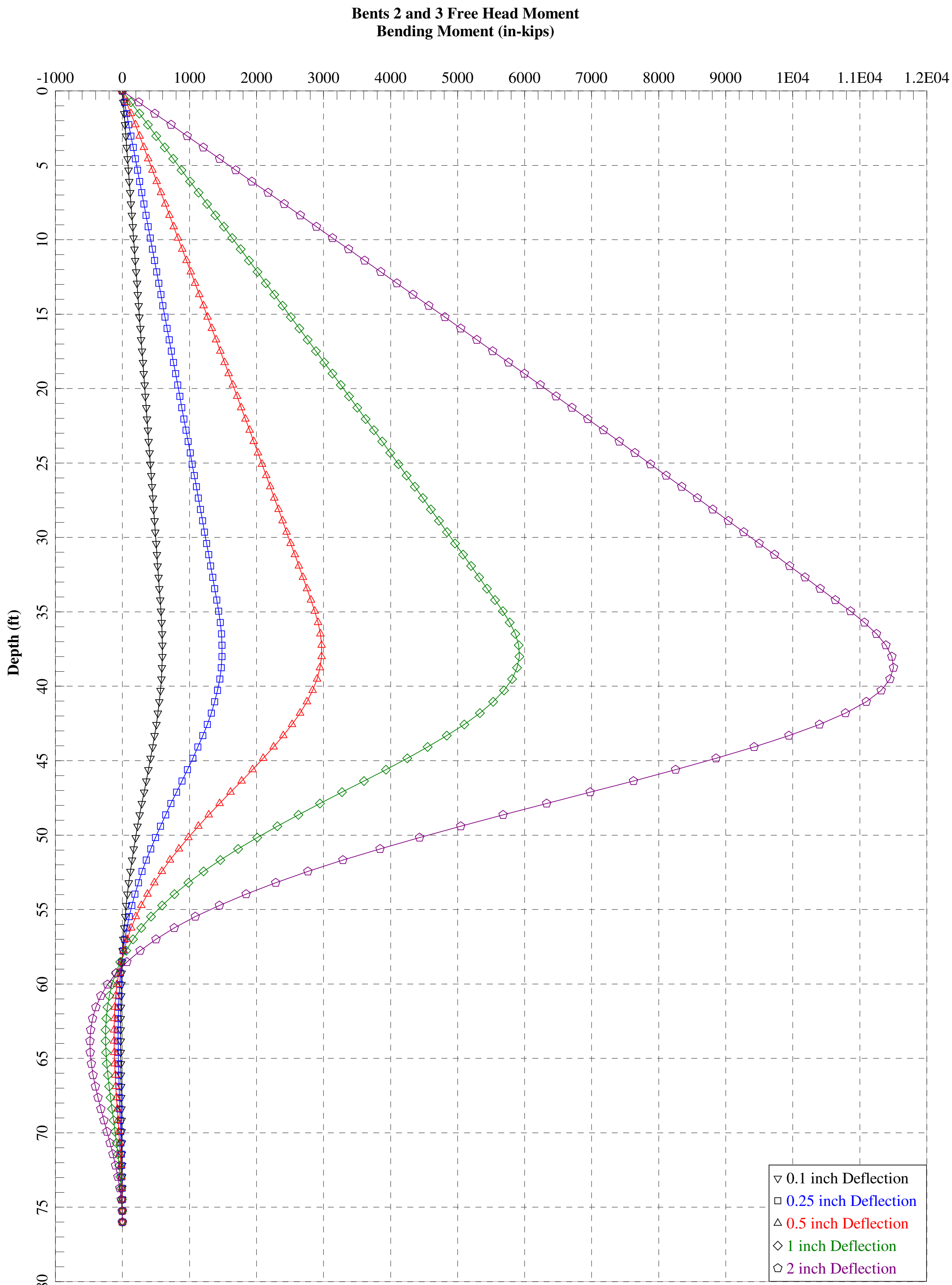

Bents 2 and 3 Fixed Head Deflection
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

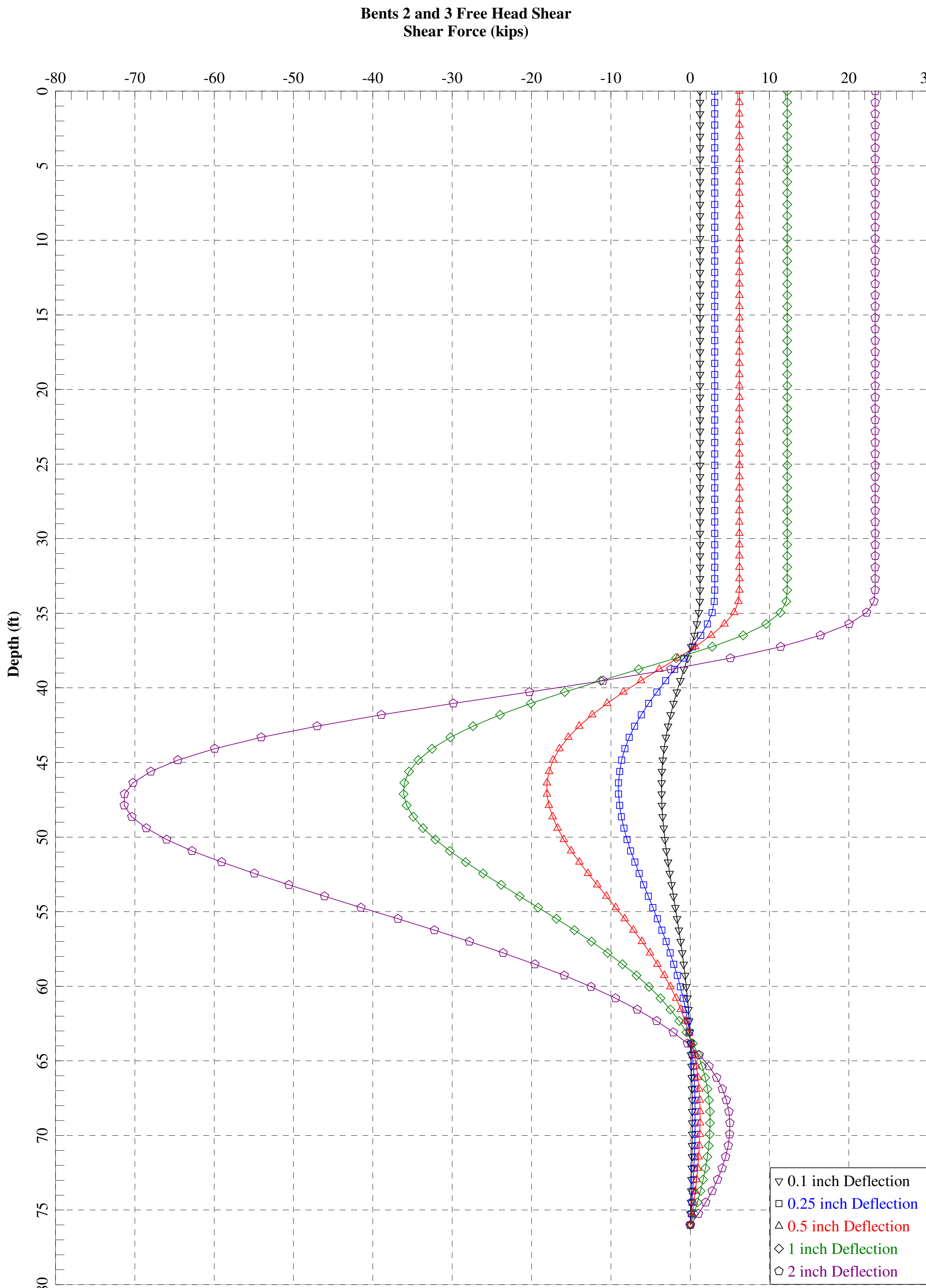

Bents 2 and 3 Fixed Head Moment
Bending Moment (in-kips)
-3E04
-2.5E04
-2E04
-1.5E04
-1E04
-5000
0
5000
1E04
1.5E04
2E04
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection

**Bents 2 and 3 Fixed Head Shear**
**Shear Force (kips)**




Bents 2 and 3 Free Head Deflection
Lateral Pile Deflection (inches)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection


Bents 2 and 3 Free Head Moment
Bending Moment (in-kips)
-1000
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
1E04
1.1E04
1.2E04
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection


Bents 2 and 3 Free Head Shear
Shear Force (kips)
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
0.1 inch Deflection
0.25 inch Deflection
0.5 inch Deflection
1 inch Deflection
2 inch Deflection