FOR CONTRACT NO: 04-0A1844
PROJECT ID: 0400020004

INFORMATION HANDOUT

MATERIALS INFORMATION

SUMMARY OF MATERIALS REPORT FOR HIGHWAY 101 H O V LANE PROJECT

EXCERPTS OF SUMMARY OF HAZARDOUS MATERIALS/ HAZARDOUS WASTE

FOUNDATION REPORT
WILLOW BROOK BRIDGE (WIDEN) (BRIDGE NO. 20-0161 R/L) PETALUMA, CALIFORNIA

STORM WATER INFORMATION HANDOUT

ROUTE: 04-Son-101-7.1/8.9

04-SON-101-PM7.1/8.9
EA04-0A1841
JUNE 3, 2010

MATERIALS HANDOUT
HIGHWAY 101 HOV LANE PROJECT
PETALUMA AND COTATI, CALIFORNIA

Prepared for
Sonoma County Transportation Authority
520 Mendocino Avenue, Suite 240
Santa Rosa, CA 95401

URS

100 W. San Fernando Street, Suite 200
San Jose, California 95113

June 3, 2010
Project 28645047.07000

File No.04-Son-101-PM7.1/8.9
EA04-0A1841

Mr. Guy Preston
Project Manager
Sonoma County Transportation Authority
520 Mendocino Avenue, Suite 240
Santa Rosa, CA 95401

Subject: **Materials Handout Report**
Segment B
Highway 101 HOV Lane Project
Petaluma and Cotati, California

Dear Mr. Preston:

URS has completed the geotechnical investigation to provide the Materials Handout Report for the proposed Highway 101 HOV Lane Project in Petaluma and Cotati, California. The enclosed report describes the available materials and the material recommendations for the subject project.

No conclusions or opinions as to the quality of materials from the potential borrow sources have been stated or implied. During our review of available borrow sources there were no site inspections or laboratory tests performed as part of this investigation; however, we understand that site inspection and laboratory tests will be performed by others during construction. The information presented in the enclosed report is based on limited telephone conversations with the potential sources. Consequently, upon selection of a borrow source, representative samples should be collected and tested in the laboratory to confirm that the borrow materials meet the material specifications. In addition where appropriate, representative samples should be tested for hazardous substances and for corrosion potential. All units presented in this report are shown in metric.

The recommended materials specifications presented in this Materials Handout Report were developed with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either express or implied, as to the professional advice included in this report.

If any questions should arise, or if we can be of further service, please contact the undersigned at (408) 297-9585.

Sincerely,

S. Stephen Huang
Geotechnical Project Leader, G.E. 2150

MATERIALS HANDOUT REPORT
HIGHWAY 101 HOV LANE PROJECT
PETALUMA AND COTATI, CALIFORNIA

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MATERIALS HANDOUT REPORT
HIGHWAY 101 HOV LANE PROJECT
PETALUMA AND COTATI, CALIFORNIA

INTRODUCTION

This Materials Handout Report presents the results of a geotechnical investigation for the proposed addition of HOV lanes in each direction of Highway 101 in Sonoma County for approximately 1-¾ miles. The project limits (PM 7.1/8.9) are from half a mile south of Old Redwood Highway interchange in Petaluma to just north of Pepper Road onramp in Cotati.

The results of our materials available studies are presented in this report, and are consistent with the format presented in "Topic 111 – Materials Sites and Disposal Sites" of the Caltrans Highway Design Manual (1995). Background information of the project, previous investigations near the project site and the scope of work are included in the introduction.

Purpose and Scope of Geotechnical Services

The purpose of this investigation is to locate potential borrow sources for use in the construction of roadway widening and new embankment fill for the subject project.

The scope of work was developed to meet guidelines for Material Sites as presented in "Topic 111 – Materials Sites and Disposal Sites" of the Caltrans Highway Design Manual (1995).

Available Reports

URS has prepared the following reports pertinent to the available materials along the project alignment:

- "Geotechnical Design Report (GDR) and Materials Report, Highway 101 HOV Lane Project, Petaluma, Cotati and Rohnert Park, California," dated July 31, 2008;
- "Addendum, Volume I of III, Geotechnical Design Report (GDR) and Materials Report, Segment B, Highway 101 HOV Lane Project, Petaluma and Cotati, California," dated March 17, 2010.

Both reports were prepared in accordance with Caltrans GDR Guidelines dated December 2006. As part of this investigation, URS reviewed the following existing Foundation Investigation data provided by Caltrans:

Bridge Number	Name	Caltrans File No.	Date
20-0161 R/L	Willow Brook Creek Bridge	54-4TC63F 4000 2280	December 24, 1956
		Bridge Inspection Report	September 23, 2004*
	Cattle Pass Undercrossing	54-4TC63F 4000 1442	

*Inspection Date

The Caltrans Materials Report for the original design of this segment of Highway 101 was not available.

GENERAL INFORMATION

Description of Project

General

The proposed project consists of adding HOV lanes in each direction by widening Highway 101 in Sonoma County for approximately 1¾ miles. As shown on Figure 1, the project limits are from half a mile south of Old Redwood Highway interchange in Petaluma to just north of Pepper Road onramp in Cotati. The widening will primarily take place in the median. Standard shoulders will be provided by widening outside the existing highway.

The project also includes interchange modifications to meet current design standards, such as ramp re-alignments and other improvements. Ramp metering will be installed and, where feasible preferential HOV bypass lanes.

Most of these improvements can be accommodated within the existing right-of-way; however, some improvements will require acquisition of additional right-of-way.

Coordination will be required between projects to the immediate north and south of the project limits. The project to the south is the Marin-Sonoma Narrows project. The project to the north is Segment A of the HOV Lanes project. Both projects are in various stages of development to add HOV lanes.

Mainline Improvements

The mainline improvements include widening Highway 101 from four to six lanes by adding one high occupancy vehicle (HOV) lane in each direction. Shoulders will be upgraded to the standard 10 feet. A concrete median barrier will be constructed to address drainage issues, except in the vicinity of the Old Redwood Interchange to address drainage issues. In general, the freeway mainline will be widened symmetrically about the existing centerline. The vertical alignment will typically follow the existing profile.

These improvements will be designed to be consistent with current highway geometric standards: travel lanes will be 12 feet wide, inside shoulders will be 10 feet wide, and outside shoulders will be 10 feet wide.

One bridge will be modified to accommodate the mainline improvements. At Willow Brook, the existing parallel bridges carrying Highway 101 will be widened into the median and joined.

Old Redwood Highway – Petaluma Boulevard North Interchange Improvements

The existing partial cloverleaf interchange configuration at Old Redwood Highway will remain, but the diagonal ramps will be reconstructed to accommodate the Highway 101 widening. The diagonal on-ramps will be realigned to improve safety and transition to the highway by increasing the acceleration distance on the ramp and improving sight distance for safer merges. The northbound diagonal on-ramp from Old Redwood Highway will be reconstructed with a larger radius curve and to accommodate one mixed-flow ramp-metered lane, an HOV preferential lane and a CHP enforcement area. The southbound diagonal on-ramp from Petaluma Boulevard North will be reconstructed with a larger radius curve and will accommodate one mixed-flow, ramp-metered lane. The entrance taper on both the northbound and southbound loop on-ramps will be improved to current standards. The northbound off-ramp to Old Redwood Highway will be reconstructed to provide standard deceleration distance. The southbound off-ramp to Petaluma Boulevard North will be reconstructed to provide standard deceleration distance and a two-lane exit ramp.

Soundwalls

A new sound wall (SW 417) will be installed along the southwest side of the Highway 101 corridor northwest of the Old Redwood Highway Overcrossing (OC). SW 417 is approximately 1,183 feet long. The wall begins about 83 feet left of "ML" Station 413+20, and extends along the northeast boundary of the adjacent mobile home park, and then turns southwesterly to extend about 86 feet along the northeast side of Denman Road. Design height is 14.33 feet. It will be placed on a Type 736SV concrete barrier and supported on 16 feet long, 16-inch diameter CIDH concrete piles.

Earthwork and Pavements

Existing Highway 101 will be widened, resulting in cuts and fills.

The proposed project will require approximately 31,200 cubic yard of roadway excavation and 6,810 cubic yard of embankment. The proposed project will require pavement materials with approximate quantities as presented below. Borrow pits and disposal sites within the project limits are shown on the project plans.

Material Type	Approximate Quantities	
Hot Mixed Asphalt (HMA)	24,300	Ton
HMA (Open Graded)	6,400	Ton
Rubberized HMA (Gap Graded)	14,800	Ton
Lean Concrete Base	14,500	cubic yard
Aggregate Base (Class 3)	70	cubic yard
Aggregate Subbase (Class 4)	29,100	cubic yard

MATERIALS AVAILABLE

Potential borrow sources were located for use in the construction of the roadway embankment. This review does not include any site inspections or testing programs. Viable local commercial sources were identified and are listed in Table 1. Also included in this table is a summary of the approximate haul distance to the project site and available material.

Before placement, representative samples should be collected from the proposed borrow source and laboratory tests should be performed. The program should include performing sand equivalent, plasticity index, sieve analysis, corrosion, and R-Value tests. In addition, testing for hazard substances should be considered. Information presented above is based on telephone conversations with the potential borrow sources. No site visits or inspections of the material from the potential sources were performed. There may be other potential borrow sources in the project vicinity that have not been identified.

In addition, more detailed soil/bedrock exploration and test results are available in both July 31, 2008 Materials Report and March 17, 2010 Addendum Report.

TABLE 1
MATERIALS AVAILABLE

Source:		Stony Point Rock Quarry Cotati	Soils Plus Sonoma	Mark West Quarry Santa Rosa	Blue Rock Forestville	Canyon Rock, Inc Forestville	Sebastopol Ready Mix Santa Rosa	Superior Supplies, Inc Santa Rosa	Northern California Ready Mix Santa Rosa	Asphalt & Asphalt Products (Boddan Co. Inc) Santa Rosa
		707-795-1775	707-996-3400	707-576-8205	707-576-8205	707-887-2207	707-823-1067	707-546-7864	707-546-9422	707-576-8205 (707-576-8209x115)
Haul Distance (one way) Miles:		3	14	11	23	23	18	11	18	11
Quarry:		Stony Point		Mark West	Blue Rock					
Product:	Units:									
Portland Cement Concrete: Class A, 6 Sack 7 Sack, no CaCl 7 Sack, 2% CaCl	Cubic Yard						$128.50 $132.50	$130.00 $136.00 $150.00	$126.50 $132.00 $143.00	
Asphalt Concrete: Type A Open Graded* Rubberized (RAC-G)	Ton	- -	- -	- -	- -	- -	- -	- -	- -	$64.00 - $84.00
Cement Treated Base: Plant Mix	Ton									
Cement Treated Base Aggregate (Class A)	Ton									
Asphalt Treated Permeable Base	Ton									
Aggregate Base Rock, Class 2	Ton	$13.95	$11.00	$13.45	$11.95	$8.00				
Aggregate Sub-Base rock Class 4	Ton	$7.25	$5.75	$11.75	$10.25					
Structural Backfill	Ton									
Permeable Material Class 1, Type A	Ton									
Permeable Material Class 2	Ton		$18.65	$17.25	$17.00					
Import Borrow	Ton									
Gabion Rock	Ton	$31.50	$25.00	$25.00	$20.00					

* Only Supplier for Open Graded Asphalt Concrete is Granite Rock Co located in Redwood City, 80 miles from project site (@ $ 94 / Ton)
** Requires bringing in and setting up a mobile RAC plant in Santa Rosa adjacent to an existing AC plant.

RECOMMENDED MATERIALS SPECIFICATIONS

Specifications

Earthwork

Earthwork shall conform to the applicable portions of Section 19 of the Standard Specifications and to current Special Provisions.

Structure Backfill

Structure backfill shall conform to the provisions in Section 19-3 of the Standard Specifications.

Embankment Construction

Embankment material for at least 1.2 m below the grading plane shall conform to the requirements noted in the Special Provisions. All imported borrow shall conform to the provisions of Section 19-7.02 of the Standard Specifications and to current Special Provisions.

Structural Pavement Sections

Asphalt Concrete (AC)
Asphalt concrete shall be Type A, 19-mm maximum coarse graded and shall conform to the provisions in Section 39 of the Standard Specifications and to the requirements noted in the Standard Special Provisions.

Asphalt Treated Permeable Base (ATPB)
Asphalt treated permeable base shall conform to the provisions in Section 29 of the Standard Specifications and to the requirements noted in the Standard Special Provisions.

Aggregate Base (AB)
Aggregate base shall be Class 3, 37.5-mm or 19-mm maximum grading and shall conform to the provisions in Section 26 of the Standard Specifications and to the requirements noted in the Standard Special Provisions.

Aggregate Subbase (AS)
Aggregate subbase shall be Class 4 and shall conform to the provisions in Section 25 of the Specifications and to the requirements noted in the Standard Special Provisions.

Lean Concrete Base
Lean concrete base shall conform to the provisions in Section 28 of the Standard Specifications and to the requirements in the Standard Special Provisions.

Pavement Reinforcing Fabric
Pavement reinforcing fabric shall conform to the provisions in Section 88-1.02 of the Standard Specifications.

Culverts And Drains

Edge Drains

Edge drains shall conform to the provisions in Section 68-3 of the Standard Specifications.

Filter Fabric

Filter fabric shall conform to the provisions in Section 88-1.03 of the Standard Specifications.

Standard Special Provisions

Aggregate Base (AB)

Aggregate base shall be Class 3, 37.5-mm or 19-mm maximum grading and shall conform to the provisions in Section 26 of the Standard Specifications and to these Special Provisions.

The aggregate grading for Class 3 aggregate base is revised from the Class 2 aggregate base by changing "Percentage Passing" the No. 200 screen to:

Sieve	**37.5-mm Maximum**		**19-mm Maximum**	
	Operating Range	**Contract Compliance**	**Operating Range**	**Contract Compliance**
No. 200	2-11	0-14	2-11	0-14

The quality requirements shall conform to the quality requirements shown on the following table.

Tests	California Test Method Number	Requirements	
		Operating Range	Contract Compliance
Sand Equivalent	217	25 Min.	22 Min.
Resistance (R-value)	301	-	78 Min.
Durability Index	229	-	35 Min.

Aggregate Subbase (AS)

Aggregate subbase shall be Class 4 and shall conform to the provisions in Section 25 of the Standard Specifications and to these Special Provisions.

Class 4 aggregate subbase shall be clean and free from vegetable matter and other deleterious substances.

The percentage composition by weight of Class 4 aggregate subbase shall conform to the following grading as determined by California Test Method No. 202.

Sieve Size	**Percentage Passing**	
	Moving Range	**Individual Test Results**
63-mm	100	100
No. 4	30-65	25-70
No. 200	0-15	0-18

Class 4 aggregate subbase shall also conform to the quality requirements shown on the following table.

Tests	California Test Method Number	Requirements	
		Moving Average	Individual Test Results
Sand Equivalent	217	21 Min.	18 Min.
Resistance (R-value)	301	--	50 Min.

Special Provisions

Embankment Construction

The upper 4 feet of embankment fill below the grading plane shall have a minimum R-value of 15.

Settlement monitoring devices shall be installed prior to filling where new fill height exceeds 3 feet. These devices shall not be damaged by the Contractor. Settlement shall be measured once a week, plotted and submitted to URS for review.

INDEX OF PLANS

STATE OF CALIFORNIA

DEPARTMENT OF TRANSPORTATION

PROJECT PLANS FOR CONSTRUCTION ON

STATE HIGHWAY

IN SONOMA COUNTY

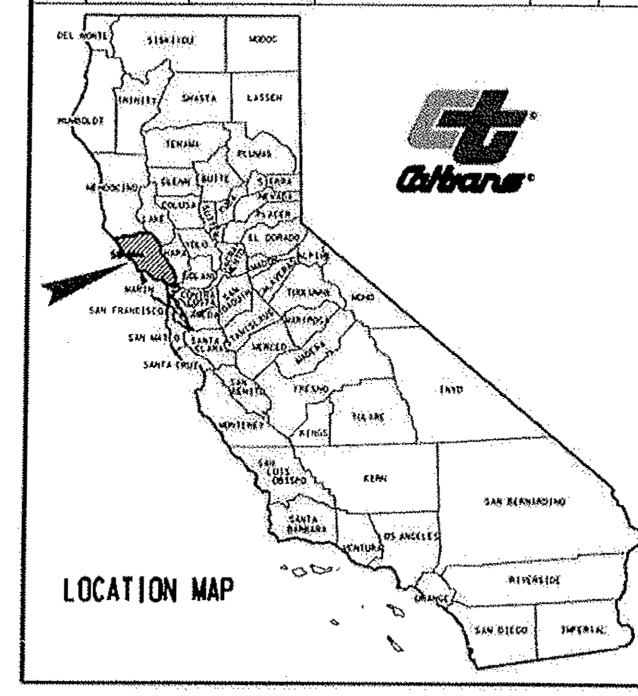
IN AND NEAR PETALUMA

FROM 0.5 MILE SOUTH OF OLD REDWOOD HIGHWAY OVERCROSSING

TO 0.1 MILE NORTH OF PEPPER ROAD

TO BE SUPPLEMENTED BY STANDARD PLANS DATED MAY 2006

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		



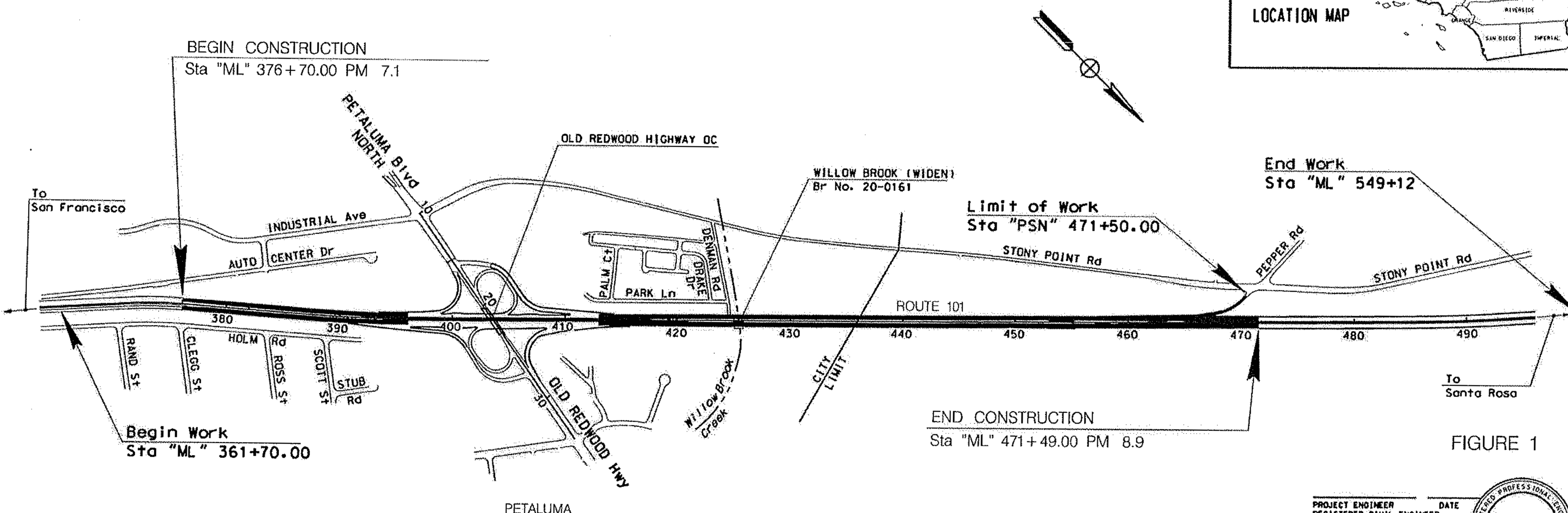


FIGURE 1

APPROVED AS TO IMPACT ON STATE FACILITIES AND CONFORMANCE WITH APPLICABLE STATE STANDARDS AND PRACTICES AND THAT TECHNICAL OVERSIGHT WAS PERFORMED.

CONSULTANT DESIGN ENGINEER	CALTRANS DESIGN OVERSIGHT APPROVAL	REGISTRATION No.	LICENSE Exp DATE	DATE SIGNED
RAMSEY HISSEN	JONATHAN LEE	47520	12/31/2011	2/23/2010

PROJECT ENGINEER DATE
REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
No.
Exp.
CIVIL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

URS CORPORATION
100 W. SAN FERNANDO St. SUITE 200
SAN JOSE, CA 95113

SCTA
520 MENDOCINO AVENUE, SUITE 240
SANTA ROSA, CA 95041

CONTRACT No. 04-0A1844

LAST REVISION 4-9-10 | DATE PLOTTED $DATE | TIME PLOTTED $TIME

THE CONTRACTOR SHALL POSSESS THE CLASS (OR CLASSES) OF LICENSE AS SPECIFIED IN THE "NOTICE TO BIDDERS."

FINAL PS&E SUBMITTAL
FEBRUARY 23, 2010

NO SCALE

BORDER LAST REVISED 8/1/2008 | CALTRANS WEB SITE IS: HTTP://WWW.DOT.CA.GOV/ | RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3 | USERNAME => $USER DGN FILE => $REQUEST | CU 04276 | EA 0A1841

SUMMARY OF HAZARDOUS MATERIALS/HAZARDOUS WASTE (PART II)

MEMORANDUM

URS
100 West San Fernando Street,
Suite 200
San Jose, CA 95113
Telephone: (408) 297-9585
Facsimile: (408) 297-6962

Re: Highway 101 Central HOV Lanes Project – Segment B
Sonoma County, California
Subject: Summary of Hazardous Materials/Hazardous Waste

URS' Site Investigation Report dated July 29, 2010, describes soil reuse and disposal restrictions, as well as worker Health and Safety requirements, for the above-referenced Project. The following is a general summary of this document provided for information purposes only; the full report must be adhered to during construction.

Aerially Deposited Lead - Soil Reuse Restrictions
For the purpose of this discussion, the following definitions apply:

- "Y1" soil may only be reused at the Site if placed beneath at least 1 foot of clean soil (soil not classified as Y1, Y2, or Z3) and at least five feet above the water table.

Approximately 6120 cubic yards of soil is classified as "Y1", which is located only within the top 1.0 foot of soil, within the central and northbound areas of the median (from the edge of the median on the northbound roadway, to the midway point between the center of the median and the edge of the median on the southbound roadway). See the Site Investigation Report for the precise limits of Y1 soil in the topmost 1.0 foot of soil.

All other soil (except that identified as Y1, described above), may be reused without restriction.

Aerially Deposited Lead - Soil Disposal Restrictions
Due to concentrations of Aerially Deposited Lead, soil to be excavated from the top 1.0 foot within the central and northbound median areas, if removed from the Project Limits, would qualify as a California Hazardous Waste and would require disposal at a permitted facility. The current design of the Project makes it likely that all excavated soil may be reused on-Site even under highly restricted conditions.

Groundwater
Groundwater was not encountered during the soil sampling effort performed in preparation for the Site Investigation Report. Soil samples were collected to the expected maximum depth of excavation within the Project Limits. Therefore, URS does not expect groundwater to be encountered during construction. If groundwater is encountered, URS recommends

groundwater sampling be performed to evaluate worker Health and Safety concerns and prior to any discharge to creeks or publically owned treatment works.

Based on URS' geotechnical investigation, which was performed in the summer of 2006, depth to groundwater varied throughout the southern project segment. At locations south of Station 455+00, groundwater was typically encountered at a depth of between 3 and 9 feet bgs, while north of Station 455+00, groundwater was typically encountered at a depth of between 8 and 27 feet. URS will work with the design team to select a fill location which is consistent with the Caltrans variance requirements.

Worker Health & Safety Plan

Based on the concentrations of lead detected in soil, a site-specific health and safety plan may be required to protect workers and the public from exposure risks due to lead. This health and safety plan may require workers to receive HAZWOPER 24 or 40 hour training prior to beginning field work. This health and safety plan may also need to be address arsenic and cobalt, which are present at background concentrations for the region, which are greater than PRGs. One way to address these issues would be to control and monitor exposure to dust (which may contain constituents including lead and arsenic) in accordance with Cal-OSHA guidelines.

0 4 - S O N - 1 0 1 , P M 7 . 1 / 8 . 9
E A – 0 A 1 8 4 1 C U 0 4 2 7 6

S E P T E M B E R 2 5 , 2 0 0 9

FOUNDATION REPORT

WILLOW BROOK BRIDGE (WIDEN)
(BRIDGE NO. 20-0161 R/L)
PETALUMA, CALIFORNIA

Prepared for
Sonoma County Transportation Authority
520 Mendocino Avenue, Suite 240
Santa Rosa, CA 95401

URS

55 South Market Street, Suite 1500
San Jose, California 95113

September 25, 2009
Project 28645047

File No. 04-SON-101-PM 7.1/8.9
EA-0A1841 CU 04276

Mr. Guy Preston
Sonoma Transportation Authority
520 Mendocino Avenue, Suite 240
Santa Rosa, California 95401

Subject: **Foundation Report**
Willow Brook Bridge (Widen)
Caltrans Bridge No. 20-0161 R/L
Petaluma, California

Dear Mr. Preston:

URS has completed the accompanying Foundation Report for the Willow Brook Bridge (Widen) in Petaluma, California. The report was prepared in accordance with Caltrans Guidelines for Foundation Investigation Reports, Version 2.0, dated March 2006.

The Foundation Report presents our engineering opinions and recommendations regarding the geotechnical factors influencing the design and construction of the proposed structure widen. The opinions and recommendations have been based upon the results of our field investigation, laboratory testing, engineering judgment and local experience. Mr. Madhu Thummuluru, P.E., performed engineering calculations. Ms. Anne-Marie Moore, G.E. assisted with the preparation of this report. Mr. David Simpson, Certified Engineering Geologist, prepared the earthquake analysis portion of this report. Mr. Michael Larson, G.E., provided review, consultation and guidelines for the project.

Our responses to comments received from Caltrans, dated July 29, 2009, on the Foundation Report are incorporated into this report.

If any questions should arise, or if we can be of further service, please contact the undersigned at (408) 297-9585.

Sincerely,

REGISTERED PROFESSIONAL ENGINEER
S. STEPHEN HUANG
No. GE 002150
03/31/2010
GEOTECHNICAL
STATE OF CALIFORNIA

Stephen Huang, G.E. 2150
Geotechnical Task Leader

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1.1 GENERAL

The Sonoma 101 Central HOV Lanes, Segment B Project (EA 0A1831) consists of improvements to Highway 101 from Old Redwood Highway (Station 376+70.00) in Petaluma to Pepper Road (Station 471+49.10), Sonoma County, California, to provide for increased travel demand in this north-south transportation corridor. Improvements to Highway 101 include widening of Willow Brook Bridge (Bridge No. 20-0161 R/L) and constructing sound walls. Figure 1-1 presents the location of this structure.

One sound wall is planned between “ML” Line Station 424+77.46 to 424+59.00 along the Southwest edge of the widened Willow Brook Bridge.

1.2 SCOPE OF INVESTIGATION

The geotechnical services performed for the Willow Brook Bridge (Widen) were as follows:

- Review existing subsurface information and as-built plans
- Geotechnical field investigation including one exploratory boring, NB05, and one cone penetration test (CPT), CT06
- Laboratory testing to estimate pertinent engineering properties
- Design recommendations and opinions were developed for the following topics:
  - Pile foundation design recommendations
    - Vertical capacity
    - Tip elevations
  - Resistance to lateral loads
  - Pile foundation and approach fill settlement
  - Abutment grading and approach fill construction
  - Earthquake information consistent with Caltrans Response Spectra Design Techniques
  - Assessment of the potential for earthquake induced liquefaction, settlement, and lateral spreading
  - Corrosion testing and analysis
  - Address construction issues, including:
    - Earthwork for abutments and new bridge approaches
    - Installation of pile foundations, as applicable

Review related to environmental and hazardous waste issues was performed during a concurrent investigation, with findings presented in a separate report, titled “Soils Investigation Report – Highway 101 Central HOV Lane Project.”

1.3 CALTRANS REVIEW COMMENTS

We reviewed comments prepared by Caltrans, dated July 29, 2009, for the Willow Brook Bridge Widening. Our responses to these comments are incorporated into this report. Copies of the review comments and our responses are presented in Appendix A.

2.1 PROJECT DESCRIPTION

According to as built structure plans in Appendix B, the two existing 70 foot long bridges are supported on concrete piles with approximate pile tip at Elevation-6 feet. There are 2 bents and 2 abutments.

The proposed improvement of Highway 101 from Old Redwood Highway in Petaluma to Rohnert Park Expressway in Rohnert Park will include regrading the mainline to meet current vertical alignment and clearance standards, and widening the highway for construction of HOV lanes in each direction and to provide standard shoulder widths. At Willow Brook, the existing three-span parallel bridges carrying Highway 101 will be widened into the median and joined. Widening at the southwest shoulder with new wingwall construction is also planned. The profile, elevation, and typical section for planned improvements at the Willow Brook Bridge are shown on Figure 2-1. The Foundation Plan is shown as Figure 2-2. A sound wall is currently planned to be constructed along the southwest side of this bridge.

The bridge is to begin at ‘ML’ Line Station 424+90.11 and end at ‘ML’ Line Station 425+60.13 and have a deck elevation ranging from Elevation 44.3 to 44.5 feet. We understand that 18-inch diameter cast-in-steel shell (CISS) piles are currently proposed to support new bridge foundation elements. Bottom of pile cap is expected to be positioned at about Elevation 36.31 to 36.45 feet within the median and Elevation 35.24 to 35.25 feet at the wingwalls. Sliver fills ranging to about 1½ feet thick are currently planned to widen the highway shoulder embankments, with median fills ranging to less than 3 feet at the bridge approaches. Embankment slopes of 2:1 or flatter (horizontal:vertical) are currently planned along the highway embankment.

3.1 GEOLOGIC SETTING

3.1.1 Regional Geology

The Sonoma Highway 101 HOV expansion project is located in the Coast Range physiographic province, near the southern end of the Coast Range Thrust. The Coast Range province is characterized by north to northwest trending elongated mountain ranges and intervening valleys. This physiography reflects the influence of the San Andreas fault system, a domain of north-northwest oriented right-lateral strike-slip faulting that accommodates the majority of the plate motion between the Pacific and North American plates. In addition to the right-lateral strike-slip deformation, a component of convergence oriented normal to the plate boundary is accommodated by a series of folds and thrust faults, including the faults of the Coast Range-Sierran Block Boundary zone, oriented sub-parallel to the faults of the San Andreas system.

Late Cenozoic (last 30 million years) deformation associated with the transpressional plate boundary is reflected in the Coast Range geology, which typically consists of intensely folded and faulted Upper Jurassic (150 million years old) and younger rocks of the Franciscan Complex, a complex assemblage of metamorphosed oceanic crustal rocks and marine sediments. In the Neogene, compressional basins of deposition, *en echelon* folds, northwest-trending strike-slip faults, and lesser east-west-trending thrust faults that dip both east and west were formed. The region is now characterized by elongate topographic regions comprising fault-bounded slivers of different rock types. The majority of the reverse faults now appear to be either inactive or significantly less active than the northwest-striking, strike-slip faults of the San Andreas system, which offset them.

Information regarding the actual depth of the bedrock at the site is not available. Based on published information on geology of the site, the bedrock consists of rocks of the Pliocene age (1 to 13 million years old) Sonoma Volcanics and older marine siltstones, sandstones, and conglomerates of the Petaluma formation.

3.1.2 Site Geology

The geology along the Sonoma Highway 101 HOV project alignment has been mapped by Fox *et al.* (1973) and reproduced in this report as Figure 3-1. The Quaternary (recent to 2 million years old) deposits in the project area include interfluvial marshlike basin sediments and alluvial fan deposits. These overlie Tertiary units including marine deposits of the Petaluma formation and Sonoma Volcanics.

The Pliocene age Sonoma Volcanics are characterized by ryolitic, basaltic and andesitic flows overlying tuff and agglomerate, though only andesitic and basaltic lava flows outcrop in the study area (Fox *et al*, 1973).

The Petaluma formation consists primarily of claystone and siltstone with thick lenses of sandstone and pebble conglomerate. Layers of tuff or tuffaceous siltstone and lenses of diatomite occur as interbeds. The diatomite is known to contain fresh-water and brackish-water mollusks as well as rare mammalian remains. The siliceous shale deposits are originally derived from the Franciscan assemblage as well as detritus from the Sonoma volcanics (Fox *et al*, 1973).

The sediments in the Willow Brook crossing along Highway 101 are younger Quarternary alluvial fan deposits grading headward to terrace deposits (Qyf). The unit consists of moderately sorted fine sands and silts with gravel becoming more abundant toward fan heads (Fox *et a l.*, 1973).

3.2 SITE AND SUBSURFACE CONDITIONS

3.2.1 Site Topography

Development of the freeway, bridges, and on-ramps and off-ramps has required the placement of fill embankments and cuts along the alignment. Current elevations along the proposed improvements in the vicinity of Willow Brook range from a low of about 30 feet at the toe of the creek embankment to about 45 feet on the bridge. Residential development is located immediately southwest of the bridge.

3.2.2 Field Exploration

Subsurface investigation for design of the existing bridge was performed in June 1952. At that time, subsurface information was obtained from one rotary wash boring and one penetration boring. The approximate locations of these borings are shown on the Site and Boring Location Plan, Figure 3-2.

To supplement available data, URS drilled one exploratory boring, NB05, to a depth of 69½ feet and advanced one CPT, CT06, to a depth of approximately 59½ feet. The field exploration was performed on August 10 and 24, 2006. The locations of the explorations are shown on Figure 3-2, as well as on the Log of Test Borings (LOTBs), Figures 3-4a and 3-4b. The LOTBs also present descriptions of the soils encountered. The new boring was initially drilled with a 4-inch diameter solid flight auger so that the depth to groundwater could be measured. Thereafter, the drilling method was switched to 5-inch rotary wash.

A representative of URS supervised the drilling operations and soil sampling. Visual classifications of the soils encountered were made from cuttings and soil samples. The soil samples collected from the boring were sealed and labeled immediately to preserve their natural moisture content. At completion of the exploration, samples were delivered to the laboratory for further examination and testing. The boring was then backfilled with a mixture of cement and bentonite in accordance with the requirements of the Sonoma County Health Department. A detailed discussion of the field exploration program is presented in Appendix C.

The Unified Soil Classification System, as well as guidelines summarizing soil consistency and relative density, are presented on the LOTB legend, Figures 3-3a and 3-3b. The logging method is consistent with Caltrans Soil & Rock Logging Classification Manual (Field Guide), dated August 1996. These logs also illustrate the notation used for the size of samplers and the methods of advancing them. Description of the soils encountered in the borings are presented on Figures 3-4b and 3-4c.

3.2.3 Laboratory Testing

The water content, dry density, Plasticity Index (PI), grain size distribution, and unconfined compressive strength were determined for selected samples to estimate the strength and compressibility of the underlying soils. The results of these tests, together with the resistance to penetration of the sampler are shown at the corresponding locations on the LOTBs. The results of sieve analysis and PI tests are graphically shown in Appendix C.

3.2.4 Soil Conditions

The soils encountered during the 1952 investigation consisted predominantly of interbedded soft to stiff clays and compact sand to approximately Elevation 0 feet, underlain by cemented coarse sand and gravel to the maximum depth explored at Elevation –24.6 feet.

The recent boring encountered fill consisting of approximately 1½ feet of clayey sand underlain by approximately 7 feet of very stiff sandy lean clay. Beneath the fill, the boring encountered native alluvial soils consisting of medium to very stiff lean and fat clays and medium dense to very dense silty and clayey sands to a depth of 69½ feet, the maximum depth of exploration. The sand interbeds typically range from 6 to 10 feet thick. The native soils encountered in the CPT can generally be characterized as silty clay and clayey silt with sandy silt, silty sand, and gravelly sand interbeds corresponding to similar thickness and elevations encountered in Boring NB05. Based on Boring NB05 and CPTCT06, the generalized soil profile is presented in Figure 3-5.

3.2.5 Groundwater

Free groundwater was encountered in Boring NB05 at approximately Elevation 22.6 feet. In addition, during this study a pore pressure dissipation record was compiled in CT07 (located approximately 138 feet north of Willow Brook along the ‘ML’ Line, to measure the groundwater level. This record is included in Appendix C and indicates a groundwater depth of approximately 14¾ feet (Elevation 29 feet). Groundwater was reportedly encountered at a depth of 8½ feet (Elevation 27.5 feet) in penetration Boring B-2 performed during the 1952 investigation.

Because the site is located at a creek crossing, groundwater elevation likely corresponds to water levels in the creek. We believe it is reasonable to assume a design groundwater level at the maximum historical level obtained in 1952 at Elevation 27.5 feet.

3.3 GEOLOGIC HAZARDS

3.3.1 Geologic Resources

Resources consulted for geologic hazard assessments included:

- Geologic maps of the California Geological Survey (CGS, formerly California Division of Mines and Geology).
- Alquist-Priolo Earthquake Fault Zone maps.

- California Geological Survey, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Special Bulletin 117, updated version May 28, 2002.
- Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M. and Helley, E.J., 2000, Preliminary maps of Quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region: A digital database, U.S. Geological Survey, Open-File Report 00-444, 60 p.
- Preliminary Geologic Map of Eastern Sonoma County and Western Napa County, CA, U.S. Geological Survey, Miscellaneous Field Studies Map, MF-483, 4 plates.
- Mualchin, L., 1996, California seismic hazard map 1996 based on maximum credible earthquakes (MCE), California Department of Transportation.
- Index to detailed maps of landslides in the San Francisco Bay region, California**,** U.S. Geological Survey, Open-File Report 97-745-D, 20 p.

3.3.2 Fault-Related Ground Rupture

Surface fault rupture tends to recur along existing fault traces. The highest potential for surface faulting is along existing fault traces that have had Holocene fault displacement. The California Geological Survey (formerly Division of Mines and Geology) has produced maps showing Alquist-Priolo Earthquake Fault Zones along faults with known Holocene activity that pose a potential surface faulting hazard. There are no Alquist-Priolo zones mapped in the vicinity of the project site. The closest active fault to the site is the Rodgers Creek fault, approximately 4½ to 5½ miles to the east. The San Andreas fault is located 14¼ to 15½ miles to the west. The potential for surface fault rupture at the site is considered remote.

3.3.3 Landslide and Slope Failure

Based on the relatively flat topography at the site, landsliding is not considered a hazard at the Willow Brook crossing.

3.3.4 Scour

The channel beneath the Willow Brook Bridge is not lined. Based on the bridge inspection report prepared by Caltrans dated September 23, 2004 (included in Appendix B), scour within the channel has exposed approximately 3 feet of pile shell at both abutments on the right bridge, and the face of Abutment 1 at the bridge has been undermined at the mid section for a length of 10 feet. This abutment has been undermined to a depth of approximately ⅓ foot. The report recommends placement of rock slope protection at both abutments for this right bridge. At the left bridge, the steel shell pile extensions were vertically exposed to depths of 1 foot at Bent 2 and 3 feet at Bent 3. Rock slope protection was not recommended in the report for the left bridge.

Based on the Willow Brook Creek project bridge design hydraulic study report (Wreco, March 2009), the channel is not lined. Wreco states "In order to avoid structural damage, and/or undermining, the additional pier foundations for the proposed bridge widening should be constructed below the estimated total scour depth or to the bedrock layer. Furthermore, any plans for the modification or replacement of existing piers and pier foundations should provide scour protection by placing scour countermeasures such as appropriately sized rip-rap." In addition their hydraulic model results predict that the 100-year flood flow in Willow Brook Creek will pass beneath both the existing and proposed bridges without overtopping. Other details are provided in the project hydraulic report. The executive summary section of hydraulic report by Wreco, dated March 2009 is presented in Appendix D.

3.3.5 Flooding

Based on a flood hazard map generated from the Association of Bay Area Governments (ABAG) geographic information systems (GIS) and reproduced in this report as Figure 3-6 (primarily based on FEMA Q3 flood data), flooding at the site is a potential hazard. The site is located within the FEMA 100-year flood zone (*i.e.*, the region that has approximately a 1% annual probability of flooding).

3.3.6 Subsidence and Seismic Compaction

Subsidence typically occurs as a result of subsurface fluid extraction (e.g. groundwater, petroleum) or compression of soft, geologically young sediments. Groundwater extraction for high volume municipal and agricultural use has the potential to cause future ground subsidence in the region. However, we are not aware of subsidence in the area; there are a number of plugged and abandoned dry petroleum holes and some completed gas holes about 3 miles east of the project site (California Division of Oil, Gas, and Geothermal Resources, 2001).

Compaction settlement, or seismic densification, occurs when loose granular soils above the groundwater table increase in density as a result of earthquake shaking. This soil densification can result in differential settlement because of variations in soil composition, thickness, and initial density. For design, we estimated the potential post-earthquake settlement at CT06, using the computer program LIQUEFY PRO. In our analyses we used a peak ground acceleration (PGA) of 0.5g and design earthquake moment magnitude (Mw) of 7.0. At Boring NB05, the soils above groundwater table are primarily lean clay and fat clay. The results of these studies indicated that the magnitude of seismic compaction settlement at abutments ranges from nil to ⅛ inch. We believe that compaction settlement of the native clay soils due to seismic densification during strong ground shaking at bents to be negligible. Copies of these calculations are included in Appendix E.

3.3.7 Liquefaction Potential

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. In extreme cases, the soil particles can be suspended in groundwater, resulting in the deposit becoming mobile and fluid-like. Three conditions are generally required for liquefaction to occur: 1) a cohesionless soil of loose to medium dense relative density; 2) a saturated condition; and 3) rapid, large strain cyclic loading normally induced by earthquake ground shaking. Liquefaction can result in loss of foundation bearing capacity, differential settlements, and lateral spreading. Traditionally, a depth of 50 feet has been used as the depth of analysis for the evaluation of liquefaction.

Based on a liquefaction susceptibility map generated from the Association of Bay Area Governments (ABAG) geographic information systems (GIS) and reproduced in this report as Figure 3-7 (primarily based on Knudsen, et al, 2000 data), the northern portion of the project alignment, just south of Laguna de Santa Rosa Creek to Rohnert Park, is underlain by soils that reportedly have a high potential for liquefaction. The southern portion of the alignment, between the Old Redwood Highway Interchange and the Pepper Road on-ramp, which includes Willow Brook, is also mapped as an area of "high" liquefaction susceptibility. The remaining central portion of the alignment is mapped as having a "very low" level of susceptibility to liquefaction.

Based on a review of the boring and CPT completed for this bridge, the post-liquefaction ground surface settlement was estimated and summarized in the following Table 3-1.

Table 3-1: Estimated Post Liquefaction Ground Surface Settlement (GSS) at Willow Brook Bridge

Boring / CPT Number	Depth to Top of Layer (feet)	Layer Thickness (feet)	Estimated Ground Surface Settlement (inch)
NB05	23	7	3/4
	40	6	1¼
	46	3	< 1/4
CT06	22	4	3/4
	30	2	1/2
	39	1	< 1/4
	47	1	< 1/4

As shown in the above table, GSS is about 2¼ inches at NB05 and 1¾ inches at CT06. Since GSS exceeds ½ inch, downdrag on the CISS is a design concern at these locations. In summary, potentially liquefiable sands were encountered in recent exploration NB05 at depths of approximately 23, 40 and 46 feet; potentially liquefiable sands and silts were identified in CT06 at depths of approximately 22, 30, 39 and 47 feet. These deposits range in thickness from 1 to 7 feet.

We estimated post-liquefaction settlement at CT06 using the computer program LIQUEFY PRO for a peak ground acceleration (PGA) of 0.5g and design earthquake moment magnitude, Mw, of 7.0. Our analysis suggests that total post-liquefaction settlement on the order of 1¾ inches could occur. In addition, we analyzed post-liquefaction settlement of the medium dense sand layers encountered in Boring NB05 for the same PGA and design earthquake, correcting the measured driving resistance (blow counts) in the field for hammer type, sampler size, overburden pressure, rod length, and fines content; the magnitude of settlement at this boring location is estimated to be about 2¼ inches. Copies of these calculations are included in Appendix E. Based on the depth and extent of these deposits, the likely consequence of liquefaction will be settlement; lateral spreading or other types of slope instability are unlikely.

3.4 GEOLOGICAL PROFILES AND ENGINEERING PARAMETERS

The LOTBs presented in Figures 3-4b and 3-4c are based on our review of previous investigations and current field information and laboratory testing from our exploratory borings at the proposed foundation support locations. Engineering soil parameters were selected from laboratory test results as well as engineering judgment and local experience.

The undrained shear strength, internal friction angle (for granular soil), relative density, dry unit weight and moisture content are the engineering soil parameters used in our foundation design and analysis. Atterberg limits tests were performed for classification of soils. In general, unconfined compression tests were performed on cohesive soil samples to estimate the undrained shear strength. Some disturbance may occur while sampling cohesive soils; therefore unconfined compressive strengths in localized areas can be lower than the insitu field conditions. Consequently, engineering judgment and local experience were applied in our interpretation of the laboratory test results. The relative density of cohesionless soils was estimated from vertical effective stress and Standard Penetration Resistance, N blows per 1 foot based on correlations developed by Gibbs and Holtz (DM7.1 – 87, 1986). Where non-standard sampler sizes were used, such as the modified California sampler (2½ inches outside diameter), a correction factor was applied to the observed blows per 1 foot to estimate the Standard Penetration Resistance, N values.

A generalized soil profile is presented in Figure 3-5 illustrating the layering of the various soil strata and summarizing the corresponding geotechnical parameters. It should be noted that this profile was developed based on extrapolation of available data from the borings drilled for this investigation and previous explorations by others and, therefore, may differ from actual conditions.

4.1 CORROSION EVALUATION

An assessment of the potential for corrosion of various buried foundation and pipe structures was performed by V& A Consulting Engineers (V&A). The results of their investigation are presented in Appendix F. The following paragraphs include their summary, conclusions, and recommendations.

4.2 SUMMARY

V&A was retained by URS Corporation to perform a Soil Corrosivity Investigation for the Highway 101 HOV Lane Project, 0.8 kilometer south of the Old Redwood Highway in Petaluma, California, to the Rohnert Park Expressway Overcrossing in Rohnert Park, California. The objective of this investigation is to measure various soil parameters and evaluate the results to determine the corrosivity of the soil to the proposed Willow Brook Bridge (widen) for the Highway 101 HOV Lane project. Corrosivity was determined for materials of the proposed project structures to depths ranging from 0 to 20 feet below grade.

The California Department of Transportation, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch has published "Corrosion Guidelines" (Guidelines) to define corrosive soil. The Guidelines consider soil to be corrosive to structural elements (steel reinforced concrete) if one or more of the following conditions exists for water or soil samples:

> "Chloride concentration is 500 ppm (mg/kg) or greater, sulfate concentration is 2,000 mg/kg or greater, or the pH is 5.5 or less."

A wide variety of soluble salts is typically found in soils. Two soils having the same resistivity may have significantly different corrosion characteristics, depending on the specific ions available. The major constituents that accelerate corrosion are chlorides, sulfates and the acidity (pH) of the soil. Chloride ions tend to break down otherwise protective surface deposits, and can result in corrosion of reinforcing steel in concrete structures. Sulfates in soil can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. High concentrations of bicarbonates tend to decrease soil resistivities. Although bicarbonates are not aggressive to concrete, lower resistivity environments can promote corrosion activity.

Acidity, as indicated by the pH value, is another important factor of soil. The lower the pH (the more acidic the environment), the higher will be the corrosivity with respect to buried metallic and concrete structures. As pH increases above 7 (the neutral value), conditions become increasingly more alkaline and passive to buried structures.

Evaluation of the in-situ soil environment was made in terms of potential damage to structures due to corrosion. Soil resistivity measurements were conducted in the field during the initial stages of the work. In addition, soil samples collected by URS during the geotechnical investigation were forwarded to Cooper Testing Laboratory in Palo Alto, California for chemical analysis. The soil samples were analyzed for minimum resistivity, pH, water-soluble chloride ion concentration, and water-soluble sulfate ion concentration, in accordance with the Guidelines. The results are summarized in the following table.

Table 4-1: Soil Corrosion Test Results Summary

Sample ID	Minimum Resisitivity (ohm-cm)	pH	Chloride Content (mg/kg)	Sulfate Content (mg/kg)
NB05, 2	1,041	7.9	9	<5
NB05, 3	581	7.6	6	<5

As shown above, the pH values of the soil samples analyzed range from 7.6 to 7.9. The water-soluble chloride ion concentrations of the soil samples analyzed range from 6 milligrams per kilogram (mg/kg) to 9 mg/kg. The water soluble sulfate ion concentrations of the soil samples analyzed were below the minimum detection level of 5 mg/kg.

4.3 CONCLUSIONS

Soil samples were obtained for corrosion analysis at Boring Location NB05. Based on the results, the following conclusions can be drawn:

- The chloride ion concentration for soil samples collected from Boring NB05 ranges from 6 milligrams per kilogram (mg/kg) to 9 mg/kg and is considered non-corrosive.
- The sulfate ion concentration for soil samples collected from Boring NB05 is less than the minimum detection level of 5 mg/kg and is considered non-corrosive.
- The pH of soil samples collected from NB05 ranges from 7.6 to 7.9 and is considered non-corrosive.

Also the proposed structure is not within 1,000 feet of salt or brackish water. Therefore, the site can be classified as *non-corrosive*.

4.4 RECOMMENDATIONS

4.4.1 Buried Reinforced Concrete Structures, Prestressed Concrete Piles, and Cast-in-Place Concrete Piles

Buried concrete structures should be constructed of durable concrete as described in Section 8.22 of the Caltrans Bridge Design Specifications and ACI Standards 201.2R and 222R. These recommendations include, but are not limited to, the following:

- The water/cement ratio should not exceed 0.50.
- A concrete cover of a minimum of 2 inches should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- Type II modified cement should be used.
- The concrete should be allowed to cure according to the manufacturer’s recommendations.

4.4.2 Steel Pipe Piles

Based on the results of the corrosion analyses, the site is considered non-corrosive to steel pipe piles. Fill materials should be tested for corrosivity, and if they are found to be corrosive, then the recommendations in the Guidelines for steel pipe piles in corrosive soil should be followed.

A conservative approach for the design of pipe piles would be to assume that corrosion will occur at a rate of 0.001 inch per year between grade and a depth 3 feet below the water table, in accordance with the Guidelines. The renewable supply of oxygen typically available in this region of soil sustains corrosion and the corrosion allowance in accordance with the Guidelines is recommended. It is important to note that structural considerations for pipe piles may require increases in metal wall thickness to withstand both installation and structural loadings, as they have not been considered in this analysis.

5.1 SEISMIC DESIGN METHODOLOGY AND RESOURCES

The seismic design methodology adopted for this project is based on the following current Caltrans standards:

1. Seismic Design Criteria (SDC), v 1.3, February 2004
2. Guidelines for Foundation Investigations and Reports, v 2.0, dated March 2006
3. California Seismic Hazard Map (Mualchin, 1996)

5.2 PEAK BEDROCK ACCELERATION

The closest active fault to this section of Highway 101 is a portion the Rodgers Creek fault (RCF). This fault is designated with a Maximum Credible Earthquake (MCE) moment magnitude of 7.0 by the Working Group on Northern California Earthquake Probabilities (WGNCEP, 2003). The location of this fault and associated peak bedrock acceleration (PBA) contours were obtained from the California Seismic Hazard Map (Mualchin, 1996). The horizontal distance from the site to the Rodgers Creek fault is 5 miles, with a corresponding PBA contour of 0.4g based on the California Seismic Hazard Map and 0.45g based on work by others (Sadigh, et al, 1997).

5.3 FAULT TYPE, NEAR-FIELD, AND SPECTRAL ACCELERATION INCREASES

The technical report that accompanies the California Seismic Hazard Map (Mualchin, 1996) indicates that the controlling fault has strike-slip displacement. Therefore, in accordance with Caltrans design procedures referenced above, an increase in design spectral accelerations is not required for fault type. However, since the project site is less than 9.3 miles from the nearest active fault, design spectral accelerations should be modified to account for near-field effects as follows:

Table 5-1: Spectral Acceleration Increase for Near Field Effects

Period (sec)	Increase in Spectral Acceleration (%)
< 0.5	0
0.5 – 1.0	0 – 20 (determined by linear interpolation)
≥1	20

Since the fundamental periods of the structures at the site are unknown at this time, the spectral accelerations have not been adjusted for the effect of fundamental period. If the fundamental period of the structure is determined to be greater than 1.5 seconds, then an adjustment should be made in accordance with Caltrans design procedures.

5.4 SOIL PROFILE TYPE AND DEPTH TO RCOK-LIKE MATERIALS

As-built LOTBs for Willow Brook indicate interbedded soft to stiff clays and compact sands to a depth of approximately 33 feet, underlain by cemented sand and gravel to a depth of 60 feet. The 2006 exploratory data reveal that stiff to very stiff clays and medium dense to very dense sands extend to a depth of at least 70 feet. Although the maximum depth of exploration did not reach 100 feet, regional geology suggests the soils become stiffer and denser with increasing depth. Accordingly, this bridge site can be classified as a stiff soil site or Soil Profile Type D pursuant to the guidelines given in Table B.1 in SDC, Version 1.3, February 2004.

5.5 DESIGN ACCELERATON RESPONSE SPECTRA

The design seismic response spectrum for the bridge structure was developed following the guidelines presented in the Caltrans SDC, Version 1.3 and can be summarized as follows:

- Soil profile type
- Maximum Credible Earthquake (MCE) Magnitude
- Peak bedrock acceleration
- ARS increase for fault-type
- ARS increase for near-field effect

The standard Acceleration Response Spectrum (ARS) corresponding to moment magnitude of controlling event (**Mw** 7.25 $\pm$ 0.25), and a PBA of 0.5g, was obtained from SDC for a soil site class D (Figure B.8). We believe the standard ARS is appropriate since the highway structure will be founded on soil. No modifications were required for fault type and long-period structure as described above. However, the spectrum was adjusted to include the effect of near-field as the controlling fault is less than 9.3 miles from the site. The resulting acceleration response spectrum is presented in Figure 5-1 and the spectral values are provided in Table 5-2 below.

Table 5-2: Recommended Spectral Acceleration Values for Willow Brook Bridge

T (sec)	Sa * (g)	Sa ** (g)
0.010	0.5003	0.5003
0.020	0.5003	0.5003
0.030	0.5002	0.5002
0.050	0.5002	0.5002
0.075	0.7192	0.7192
0.100	0.9186	0.9186
0.120	1.0367	1.0367
0.150	1.1564	1.1564
0.170	1.2091	1.2091
0.200	1.2591	1.2591
0.240	1.2660	1.2660
0.300	1.2467	1.2467
0.400	1.1898	1.1898
0.500	1.1249	1.1249

Table 5-2: Recommended Spectral Acceleration Values for Willow Brook Bridge

T (sec)	Sa * (g)	Sa ** (g)
0.750	0.9438	1.0382
1.000	0.7722	0.9267
1.500	0.4839	0.5807
2.000	0.3213	0.3856
3.000	0.1698	0.2038
4.000	0.1064	0.1277

* From Caltrans Seismic Design Criteria

** Modified values for near fault effects

6.1 AS-BUILT FOUNDATION DATA

Originally constructed in 1956, the Willow Brook Bridge is a 70-foot long, three-span, reinforced concrete continuous deck slab structure as shown on the as-built drawings (Contract No. 54-4TC-63F). The plans indicate that the two abutments and two bents are supported on circular reinforced concrete pile extensions. As shown on the Standard Pile Details sheet, Cast-in-Place Concrete Piles, Alternative "Z," were used for the foundation piles with column extensions supporting the bridge deck and abutments. The concrete piles were installed by first driving a steel shell extending from 1 foot below ground surface to the pile tip elevation. Then, the steel shell was filled with concrete to form the pile; concrete extension continued to the cutoff elevations to complete the column. The steel shell tapered from a diameter of 15½ inches at the butt to a minimum diameter of 8 inches at the tip. The concrete extension was 15½ inches in diameter. Recorded pile tip elevations range from Elevation –3 feet to –1 foot at the abutments (Abut 1 and Abut 4) and from Elevation –7.8 feet to –4 feet at the bents (Bent 2 and Bent 3). These piles were designed for a load of 32 tons; we assume this refers to an axial, compressive load for the service condition.

The survey datum on the project profiles, Sheets 5 to 26 of the 1954 plan set, is listed as C.H.C. (3.4 feet above MLLW datum).

Copies of pertinent as-built structure plans are presented in Appendix B.

The proposed bridge is underlain predominantly by medium to very stiff lean and fat clay and medium dense to very dense silty and clayey sand. The principal geotechnical issues at the site are:

- Selection of the type and depth of foundation that will be compatible with the underlying soils,
- Construction issues associated with the proximity of proposed new piles to existing piles,
- Post-liquefaction settlement of potentially liquefiable soils encountered near the bridge, and
- Stability of embankment slopes.

We understand that the project Structural Engineer has elected to use 18-inch diameter CISS piles at the abutments, with pile extensions at the bents.

Because pockets of potentially liquefiable soils were encountered in both explorations, consistent with Caltrans requirements, we have included the estimated average downdrag force down to approximately Elevation 0 feet associated with post-liquefaction settlement in the design pile tip elevations presented in Section 7.1. The maximum downdrag force is estimated to be approximately 115 tons at Abutments 1 and 4, and approximately 49 tons at the bents.

7.1 PILE DESIGN CAPACITY AND TIP ELEVATION

7.1.1 Axial Pile Capacity Analysis

Based on our review of the subsurface conditions encountered in the current borings and 1954 as-built borings, in our opinion, the proposed structure widening can be supported on cast-in-steel-shell (CISS) closed end piles as planned. This pile type is appropriate, considering the relatively shallow groundwater (about 20 feet below ground surface) and deeper saturated silty sand and sand layers that are prone to cave-in.

The specified tip elevations presented in the following Table 7-1: "Pile Data Table" are based on end-bearing resistance developed in the dense sand stratum below Elevation -10 feet. The estimated pile embedment included additional penetration requirements to resist potential downdrag loads that could act on the piles due to post-construction liquefaction induced settlement.

Table 7-1: Pile Data Table

Location	Pile Type	Design Loading Service (kips)	Nominal Resistance Compression (kips)	Nominal Resistance Tension (kips)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
Abutment 1	CISS PP 18x0.5	90	180	0	-14 (1) -3 (2)	-14
Bent 2	CISS PP 18x0.5	NA	280	0	-16 (1) -5 (2)	-16

Location	Pile Type	Design Loading Service (kips)	Nominal Resistance Compression (kips)	Nominal Resistance Tension (kips)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
Bent 3	CISS PP 18x0.5	NA	280	0	-16 (1) -5 (2)	-16
Abutment 4	CISS PP 18x0.5	90	180	0	-14 (1) -3 (2)	-14

Design tip elevation is controlled by the following demands: (1) Compression, (2) Lateral

The results of our axial pile capacity analysis, which form the basis of our selection of the design tip elevations, are presented in Appendix G.

No group reduction factor needs to be applied to the single pile compression load capacities presented above provided a center to center spacing of at least three pile diameters is used.

7.1.2 Lateral Load Capacity

The driven pile foundations are capable of resisting lateral loads. Resistance to lateral loads can be developed by bending of the pile and by pile-soil interaction. The magnitude of the lateral load resistance that can develop depends upon several factors such as the pile size, the physical properties of the surrounding soils, and the structural design of the pile. We used LPILE 4.0 (Reese et al., October 2000) to assist in estimating the lateral load resistance of 18-inch CISS piles. The program models the soil response in the form of load-deflection (p-y) curves.

Output files for laterally loaded piles at the abutments and bents are presented in Appendix H and include deflection versus depth, bending moment versus depth, shear versus depth, slope versus depth, and total stress versus depth.

Based on the foundation plan provided, it appears that piles will be installed in single row; therefore, no reduction would be required to the group efficiency provided that a minimum three pile widths spacing is used between piles.

If additional lateral capacity is needed beyond the lateral load capacity of the piles, passive resistance against the abutment walls can be utilized. For abutments, Caltrans limits the soil resistance at the back wall to a 5.5 feet wall height and a maximum uniform soil pressure of 5,000 pounds per square foot. For wall heights less than 5.5 feet, the average unit pressure should be reduced linearly in proportion to the height. The recommended values presented above are ultimate values, and should be used with an appropriate factor of safety.

7.2 PILE INSTALLATION

All piles should be installed under the direct observation of the Geotechnical Engineer and in accordance with Section 49 of the Caltrans Standard Specifications, “Piling.” Specific additions and modifications to these requirements are discussed below. However, using water jets and pumps to achieve the specified tip elevation should not be permitted.

Some hard driving could be experienced between Elevation 5 and -5 where a relatively dense sand layer was encountered. The Contractor should be prepared to predrill an undersized hole, where necessary, and as approved by the Geotechnical Engineer, to enhance the installation of piles to the specified tip elevations. An undersized hole is not greater than the least dimenions of the pile. All equipment used for predrilling should be in accordance with Standard Specifications, Section 49-1.05, Driving Equipment. In addition, the maximum depth of predrilling should not extend deeper than 10 feet above the specified tip elevation. Each pile should be driven immediately after the hole is predrilled to minimize cave-in problems.

The Contractor should submit evidence of compatibility of the proposed pile hammer with the pile type and soil conditions at the site. The Contractor's hammer submittal should include, as a minimum, a dynamic analysis of the pile driving system that is based on wave equation analysis using computer programs such as WEAP. Acceptance criteria for driven piles should follow Caltrans Standard Specifications Section 49-1.08, Pile Driving Acceptance Criteria as well as Special Provisions 49-228, Redriving. Driven piles reaching refusal within 10 feet of the specified tip elevation and meeting the acceptance criteria may be cut-off above the tip elevation required by the compression loads. This assumes that the design lateral load and tension load tip elevations have been reached (see Pile Data Table). Preliminarily, we recommend that the refusal criteria be two times the minimum required blowcount. However, refusal criteria will be defined later based on the pile driving system proposed by the Contractor.

7.3 APPROACH EMBANKMENT SETTLEMENT

We evaluated settlement of the approach embankment due to placement of less than 3 feet of new fill to the grades shown on the 'ML' Line profile. Based on our analysis, we estimate that settlement on the order of less than ½ inch could occur along the approach centerline at the abutments. We estimate that approximately 90% of this settlement should occur within one month of fill placement assuming the full embankment width is constructed at one time. In our opinion, surcharging is not necessary.

7.4 SLOPE STABILITY

Based on topographic data, it appears that the existing slopes along Willow Brook Creek at the proposed widened bridge are inclined at about 2:1 (horizontal:vertical). The General Plan (Figure 2-1) for the structure indicates that no rock slope protection is to be placed along the slope, inclined at 2:1. The hydraulic report (Wreco, March 2009) presented in Appendix D states "In order to avoid structural damage, and/or undermining, the additional pier foundations for the proposed bridge widening should be constructed below the estimated total scour depth or to the bedrock layer. Furthermore, any plans for the modification or replacement of existing piers and pier foundations should provide scour protection by placing scour countermeasures such as appropriately sized rip-rap."

The embankment soils consist primarily of stiff to very stiff sandy lean clay, lean clay and fat clay with a medium dense silty sand layer down to about Elevation 3; in our opinion, these soils are sufficiently strong to support the proposed 2:1 slopes.

Based on our analyses, the computed minimum (critical) factor of safety at Abutments 1 and 4 are 3.3 and 3.2 respectively, for static stability. The computed factor of safety (FOS) values exceed the minimum required value of 1.5 mentioned in Section 5.2.2.3 of Bridge Design Specifications (August 2004). The results of slope stability analyses are presented in Appendix I as Figures I-1 and I-2.

For earthquake loading conditions, we performed a pseudo-static analysis to determine the failure surface for a seismic coefficient equal to one-third the peak horizontal bedrock acceleration of 0.5g (for Rodgers Creek fault, see Section 5.2 "Peak Bedrock Acceleration") anticipated at the site. As shown in Figures H-3 and H-4 (Appendix I), the computed FOS values for earthquake loading are both 1.9 for Abutments 1 and 4; this value exceeds the minimum required value of 1.0 for design of walls.

In addition, a medium dense sand layer (N=23 blows per foot) was encountered in boring NB05 between depths of 23 and 30 feet. As discussed previously in Section 3.3.7 "Liquefaction Potential", this layer is potentially liquefiable. Due to its close proximity to the channel bottom, consideration was given to the possibility of lateral spreading. Consequently, we performed post-liquefaction slope stability analyses using residual strengths estimated for the potentially liquefiable sand layer. Both circular and wedge shaped slip surfaces were included in the analyses and the results are summarized in Table 7-2. The graphical presentation of the results of these analyses are presented in Appendix I (Figures I-5 through I-8).

Table 7-2: Summary of Post Earthquake Stability Analyses

Figure No.	Abutment No.	Factory of Safety		Residual Undrained Shear Strength (psf)	Slip Surface
		Minimum* Required	Estimated		
H-5	1	1.3	3.0	950	Circle
H-6	1	1.3	2.2	950	Wedge
H-7	4	1.3	2.9	950	Circle
H-8	4	1.3	2.4	950	Wedge

*Assumed

The FOS against undrained loading simulating conditions immediately after liquefaction are all above 1.3. Therefore, we conclude that lateral spreading is unlikely at Abutments 1 and 4 of the Willow Creek Bridge.

7.5 APPROACH FILL EARTHWORK

All earthwork should be completed in accordance with the applicable sections of the Caltrans Standard Specifications and as described in the URS companion Geotechnical Design and Materials Report for the Highway 101 HOV Widening Project.

7.6 ABUTMENT EXCAVATION

Footing/pile cap areas should be excavated as required to bring those areas to their finish subgrade elevations. All loose soil should be removed from the exposed subgrade prior to footing construction. Because shallow groundwater is not expected to be encountered during excavation of the pile cap footings at the abutments, we recommend the type of excavation be classified as Structure Excavations (Bridge) in accordance with the Bridge Design Aids, March 2005.

8.1 TEMPORARY CONSTRUCTION EXCAVATION

We anticipate that excavation into the embankment fills or native soils for construction of the abutments will result in temporary near vertical unsupported soil faces as high as about 7 feet. Safety standards set by OSHA limit the height of unshored vertical excavations to 5 feet if construction personnel will be working in the excavations. The set of guidelines published by OSHA (Department of Labor, Occupational Safety and Health Administration, 1989), classifies soils in detail as Type A, B, or C. In general, Type A soils are stronger, Type B soils are intermediate, and Type C soils are weaker. Based on the soil type, depth, duration the excavation is open, and sequence of soils exposed in the excavation, OSHA recommends maximum allowable slopes. For example, for excavations 20 feet or less in depth through homogeneous soils, they state that maximum allowable slopes (horizontal to vertical) should be ¾ to 1, 1 to 1, and 1½ to 1 for Type A, B, and C soils, respectively. Based on the strengths of the soils encountered in our recent borings, the existing embankment fills in the vicinity of the abutments are considered to be OSHA Types A and B, while the underlying native soils are considered to be OSHA Type A.

The guidelines provided by OSHA are for trench excavations; they state that there is uncertainty as to when and to what degree an employer must slope, shore, or otherwise protect employees in a “non-trench” excavation. In consideration of these factors, we recommend that temporary cut slopes in the existing embankment fills and native soils not exceed 1 to 1 during construction.

For locations where excavation with sloping sides is not viable because of space limitations or in areas where temporary slopes steeper than 1:1 are planned, shoring will be required. The Contractor should retain an experienced Registered Civil Engineer to design the shoring system.

8.2 CONSTRUCTION DEWATERING

The bottom of footing at the abutments is expected to be about 10 feet above the highest measured groundwater level; therefore it is unlikely that groundwater will be encountered in footing excavations. However, if some surface water infiltration is encountered during construction, we anticipate that dewatering can be accomplished with standard sumping procedures.

8.3 PILE CUTOFF

When driven piles develop the required compressive capacities before reaching the specified tip elevation, the Contractor may be given the option, with the Geotechnical Engineer’s approval, to stop driving and cut off the piles. Pile cut-off should be approved only if the piles also have satisfied the tension and lateral demand requirements, and the structural capacity has not been compromised. For maximum pile cut-off length, refer to the Standard Plans (Caltrans, 2004).

8.4 EFFECTS OF CONSTRUCTION WORK ON ADJACENT STRUCTURES

Efforts shall be made to minimize effects of construction work on adjacent structures. These situations may result from pile-driving vibration, or settlement due to dewatering or excavation. A monitoring program may be required for pile driving at, or adjacent to, existing structures that are susceptible to damage or sensitive to noise and/or vibration to assure a presumptive threshold will not be exceeded.

The opinions, conclusions and recommendations presented in this Foundation Report are based on information obtained from new and previous explorations made at widely separated locations, site reconnaissance, review of available topographic information and historic data, and upon local experience and engineering judgment.

The recommendations presented in this report are based on the assumption that the soil and geologic conditions do not deviate substantially from those encountered in the exploratory boring and CPT. If any variations are encountered during construction, URS should be contacted so that supplementary recommendations can be made.

If the planned construction is changed from that presently conceived, URS should be retained to review the changes and make modifications to the original recommendations presented in this report in order to meet the project needs.

The Geotechnical Engineer should review the final specifications and drawings to verify that these documents are consistent with the intent of the geotechnical recommendations. Geotechnical issues may arise during construction that are not apparent at this time. URS should be retained during construction to review the soil conditions encountered and the construction procedures. All earthwork and testing should be done under the direct observation of a representative of our firm.

The elevations shown on the new LOTBs are based on interpolation from spot and contour elevations shown on available topographic maps.

As-built drawings pertinent only to the geotechnical investigation are included.

Specific review and investigation for environmental issues and subsurface environmental contamination were beyond the scope of our services.

The opinions and recommendations presented in this Foundation Report were developed with the standard of care commonly used as state of the practice in the profession. No other warranties are included, either express or implied, as to the professional advice provided in this report.

Association of Bay Area Governments, 2004, Liquefaction Susceptibility Map.

FEMA, 1996, National Flood Insurance Program, (www.fema.gov).

Knudsen, K.L., Sowers, J.M., Witter, R.C., Wentworth, C.M. and Helley, E.J., 2000, Preliminary maps of Quaternary deposits and liquefaction susceptibility, nine-county San Francisco Bay region: A digital database, U.S. Geological Survey, Open-File Report 00-444, 60 p.

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Sadigh, K., Change, C.Y., Egan, J.A., Makdisi, F., and Youngs, R.R., 1997, Attenuation relationships for shallow crustal earthquakes based on California strong motion data, Seismology Research Letters, v. 68, p. 180-189.

Wagner, D.L., Bortugno, E.J., and McJunkin, R.D., 1990, Geologic map of the San Francisco-San Jose quadrangle: California Division of Mines and Geology Regional Geologic Map No. 5A, scale 1: 250,000.

Wells, D.L. and Coppersmith, K.J., 1994, New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement, Bulletin of the Seismological Society of America, v. 84, p. 974-1002.

Working Group for California Earthquake Probabilities, 2003, Earthquake probabilities in the San Francisco Bay region, 2003-2031, US Geological Survey Open File Report 03-214.

Fox, K.F., Jr. Sims, J.D., Bartow, J.A., and Helley, E.J., 1973, Preliminary geologic map of eastern Sonoma County and western Napa County, California, U.S. Geological Survey, Miscellaneous Field Series Map MF-483, 1:62,500.

INDEX OF PLANS

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

STATE OF CALIFORNIA

DEPARTMENT OF TRANSPORTATION

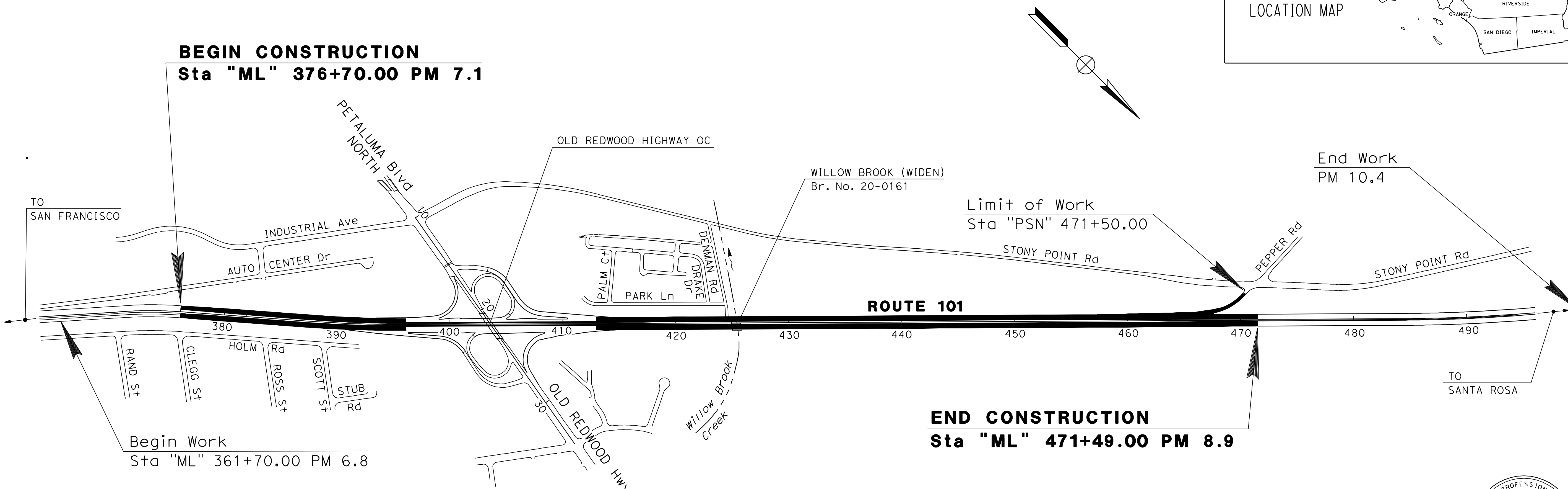
PROJECT PLANS FOR CONSTRUCTION ON

STATE HIGHWAY

IN SONOMA COUNTY
IN PETALUMA
FROM 0.5 MILE SOUTH OF OLD REDWOOD HIGHWAY OVERCROSSING
TO 0.1 MILE NORTH OF PEPPER ROAD

TO BE SUPPLEMENTED BY STANDARD PLANS DATED MAY 2006

LOCATION MAP



APPROVED AS TO IMPACT ON STATE FACILITIES AND CONFORMANCE WITH APPLICABLE STATE STANDARDS AND PRACTICES AND THAT TECHNICAL OVERSIGHT WAS PERFORMED.

DATE SIGNED | LICENSE Exp DATE | REGISTRATION No. | CALTRANS DESIGN OVERSIGHT APPROVAL: JONATHAN LEE | CONSULTANT DESIGN ENGINEER: RAMSEY HISSEN

PROJECT ENGINEER DATE
REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER
No.
Exp.
CIVIL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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55 S. MARKET STREET, SUITE 1500
SAN JOSE, CA 95113

SCTA
520 MENDOCINO AVENUE, SUITE 240
SANTA ROSA, CA 95041

CONTRACT No. **04-0A1844**

DATE PLOTTED => 9/25/2009
TIME PLOTTED => 8:15:58 AM
LAST REVISION 00-00-00

FINAL PS&E SUBMITTAL
JULY 27, 2009

NO SCALE

THE CONTRACTOR SHALL POSSESS THE CLASS (OR CLASSES) OF LICENSE AS SPECIFIED IN THE "NOTICE TO BIDDERS."

FIGURE 1-1

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

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REGISTERED PROFESSIONAL ENGINEER Jan M. Hueser No. C050215 Exp. 6/30/11 CIVIL STATE OF CALIFORNIA

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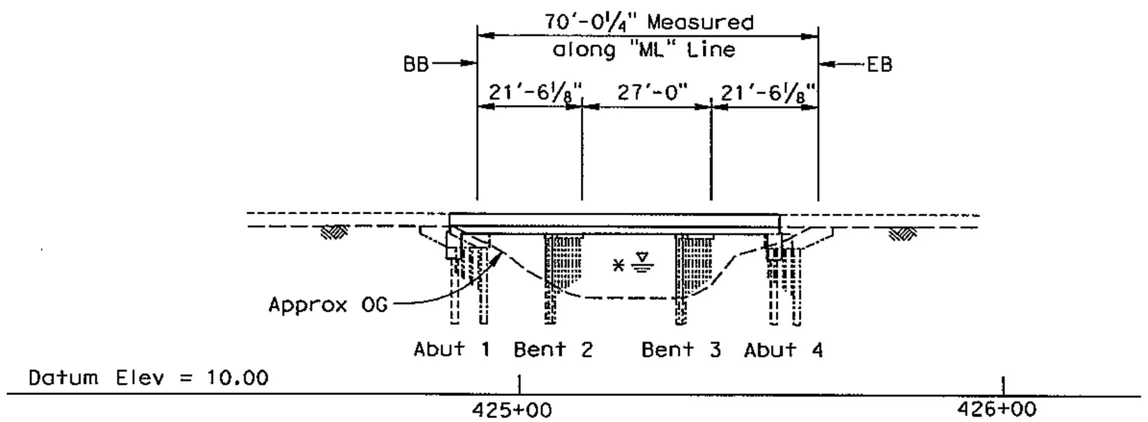
520 MENDOCINO AVENUE, SUITE 240
SANTA ROSA, CA 95401

URS CORPORATION
1380 LEAD HILL BLVD, SUITE 100
ROSEVILLE, CA 95661-2997

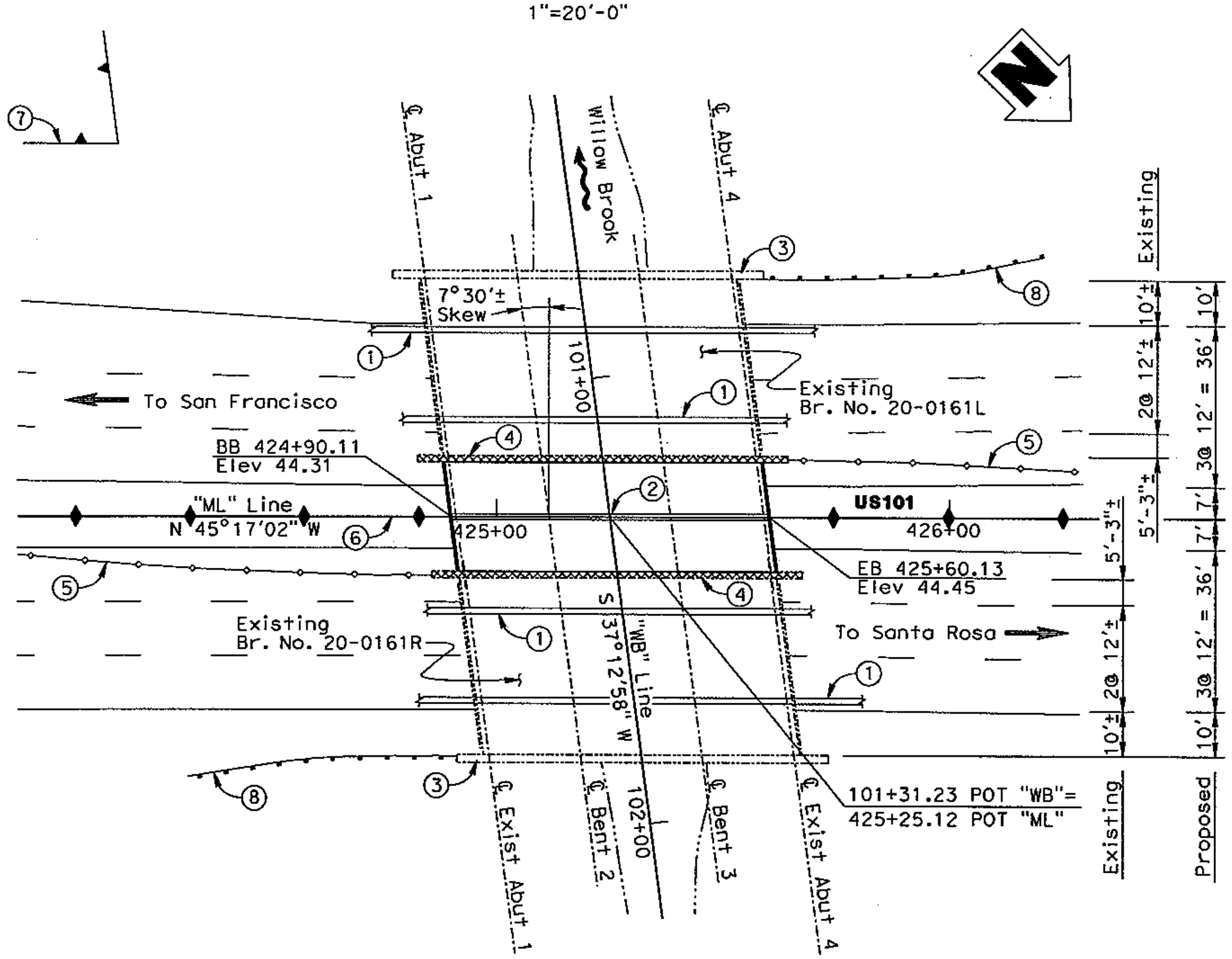


SECTION

1"=20'-0"

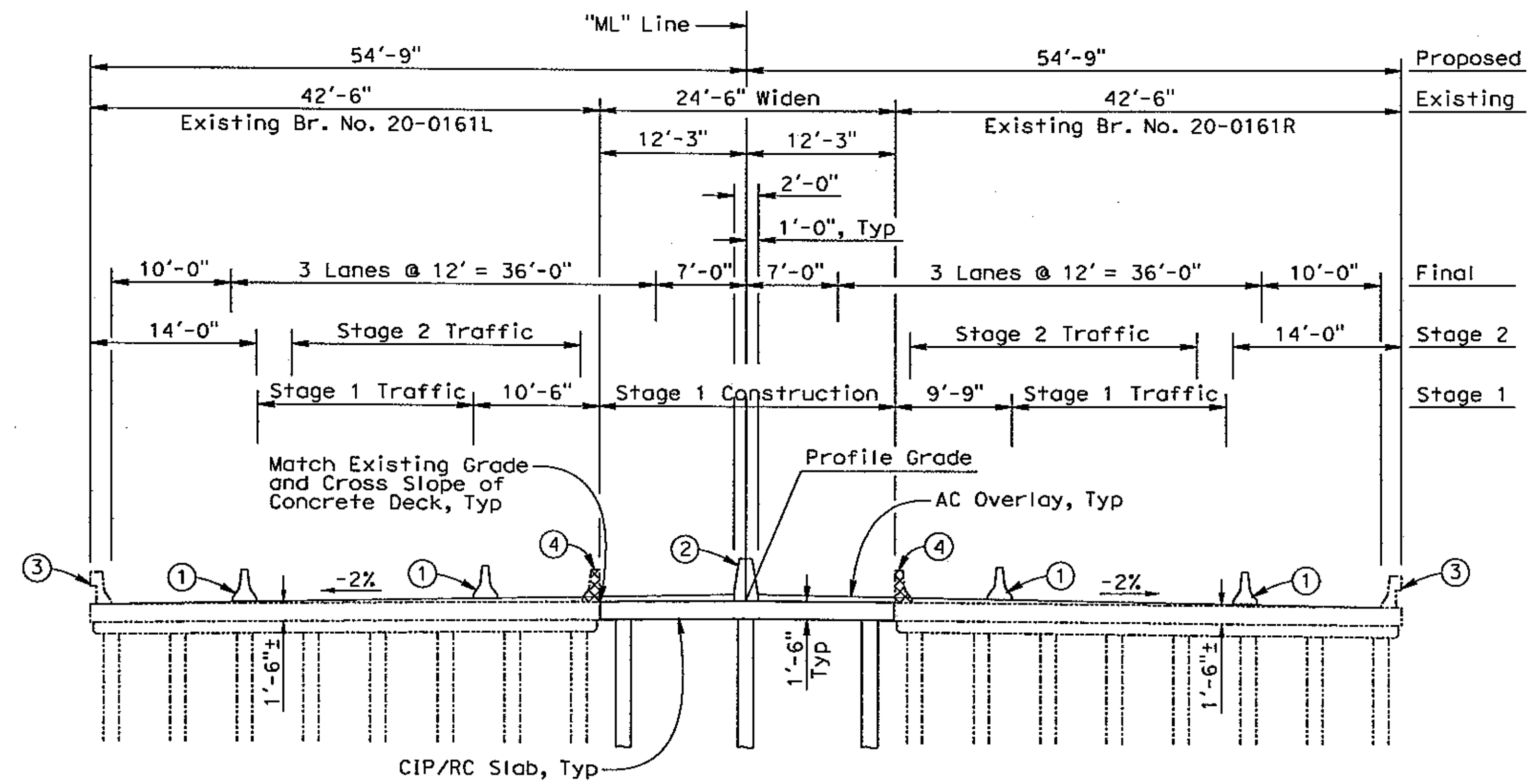
LEGEND:

- Indicates existing structure
- Indicates direction of traffic
- Indicates direction of flow
- Indicates Bridge Removal (Portion)
- * For Hydrologic Summary, see "Foundation Plan"



PLAN

1"=20'-0"



TYPICAL SECTION

⅛"=1'-0"

NOTES:

1. Temporary Railing (Type K), see "Road Plans"
2. Concrete Barrier (Type 60A Modified)
3. Existing Concrete Barrier Type 25R
4. Remove existing Concrete Barrier
5. Remove existing Median Barrier, see "Road Plans"
6. Concrete Barrier, see "Road Plans"
7. Sound Wall on Type 736 SV Barrier, see "Road Plans"
8. Remove existing MBGR and place new MBGR, see "Road Plans"

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

FIGURE 2-1

DESIGN	BY Jan Hueser	CHECKED George Rowe	LOAD FACTOR DESIGN	LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD
DETAILS	BY L. Davis	CHECKED George Rowe	LAYOUT	BY George Rowe / CHECKED Jan Hueser
QUANTITIES	BY	CHECKED	SPECIFICATIONS	BY Dave Harnagel / PLANS AND SPECS COMPARED

DESIGN OVERSIGHT

SIGN OFF DATE

DESIGN GENERAL PLAN SHEET (METRIC) (REV. 10/27/05)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

Walt LaFranchi
PROJECT ENGINEER

BRIDGE NO. 20-0161
POST MILE 8.05

WILLOW BROOK (WIDEN)
GENERAL PLAN

CU 04276
EA 0A1831

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 5-14-07 4-17-09 6-08-09

SHEET 1 OF 13

FILE => J:\Br CADD\Segment B\Willow Brook 20-0161\20-0161-a-ap01.dgn

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER
Jon M. Hueser
No. C050215
Exp. 6/30/11
CIVIL
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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URS CORPORATION
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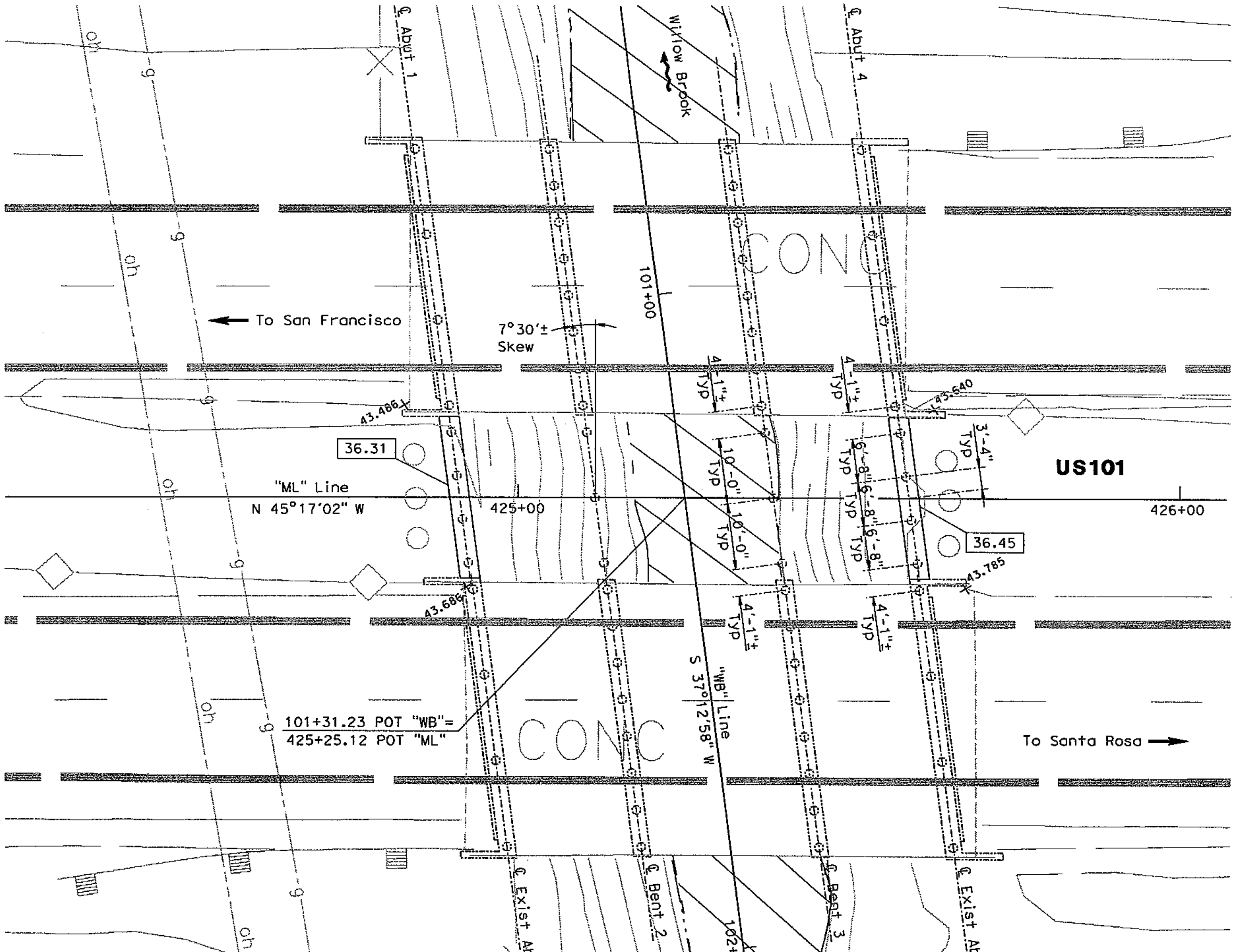
ROSEVILLE, CA 95661-2997

PILE DATA TABLE

Location	Pile Type	Design Loading (kips)	Nominal Resistance Compression (kips)	Nominal Resistance Tension (kips)	Ultimate Geotechnical Capacity (kips)	Cut-Off Elev (ft)	Design Tip Elev* (ft)	Specified Tip Elev (ft)
Abut 1	CISS	90	180	0	180		-14 (1) -3 (2)	-14
Bent 2	CISS	N/A	280	0	280		-16 (1) -5 (2)	-16
Bent 3	CISS	N/A	280	0	280		-16 (1) -5 (2)	-16
Abut 4	CISS	90	180	0	180		-14 (1) -3 (2)	-14

* Design tip elevation is controlled by the following demands: (1) Compression, (2) Tension, (3) Lateral Loads



PLAN
1"=10'-0"

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

HYDROLOGIC SUMMARY

Drainage Area: 28.9 Square Miles	Design Flood	Base Flood
Frequency (years)	50	100
Discharge (cubic feet/second)	2,560	2,560
Water Surcharge (elevation at bridge)	40.14	40.28

LEGEND:
- Indicates existing structure
- Indicates direction of traffic
- Indicates direction of flow
- Indicates Bottom of Footing Elevation
- Indicates Existing Pile
- Indicates CISS Pile
- XX.XXX Indicates Elevation along existing roadway

BENCH MARK

Bench Mark	N	E	Elev	Description
#824	1863213.554	6367780.936	43.225	1" Iron Pipe easterly side of NB101 10' northerly of Willow Brook Creek
#883	1863839.616	6367153.213	40.554	1" Iron Pipe easterly side of NB101 900' northerly of Willow Brook Creek

Survey Control
Horizontal control for this survey is based on the California Coordinate System, Zone II, Us Survey feet, North American Datum of 1983 (NAD 83), Epoch 2007.00.
Vertical control is the North American Vertical Datum of 1988 (NAVD 88).

FIGURE 2-2

GEOTECHNICAL PROFESSIONAL APPROVAL DATE

DESIGN OVERSIGHT	SCALE: 1"=10'	VERT.DATUM NAVD 88	HORZ.DATUM NAD 83	DESIGN	BY Jon Hueser	CHECKED George Rowe
	PHOTOGRAMMETRY AS OF:		ALIGNMENT TIES	DETAILS	BY L. Davis	CHECKED George Rowe
	SURVEYED	BY Caltrans	DRAFTED BY Caltrans	QUANTITIES	BY	CHECKED
SIGN OFF DATE	FIELD CHECKED	BY Caltrans	CHECKED BY Caltrans			

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

Walt LaFranchi
PROJECT ENGINEER

BRIDGE NO. 20-0161
POST MILE 8.05

WILLOW BROOK (WIDEN)
FOUNDATION PLAN

FOUNDATION PLAN SHEET (METRIC) (REV. 10/27/05)

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 04276
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DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 5-11-07 6-17-09 6-08-09

SHEET 3 OF 13

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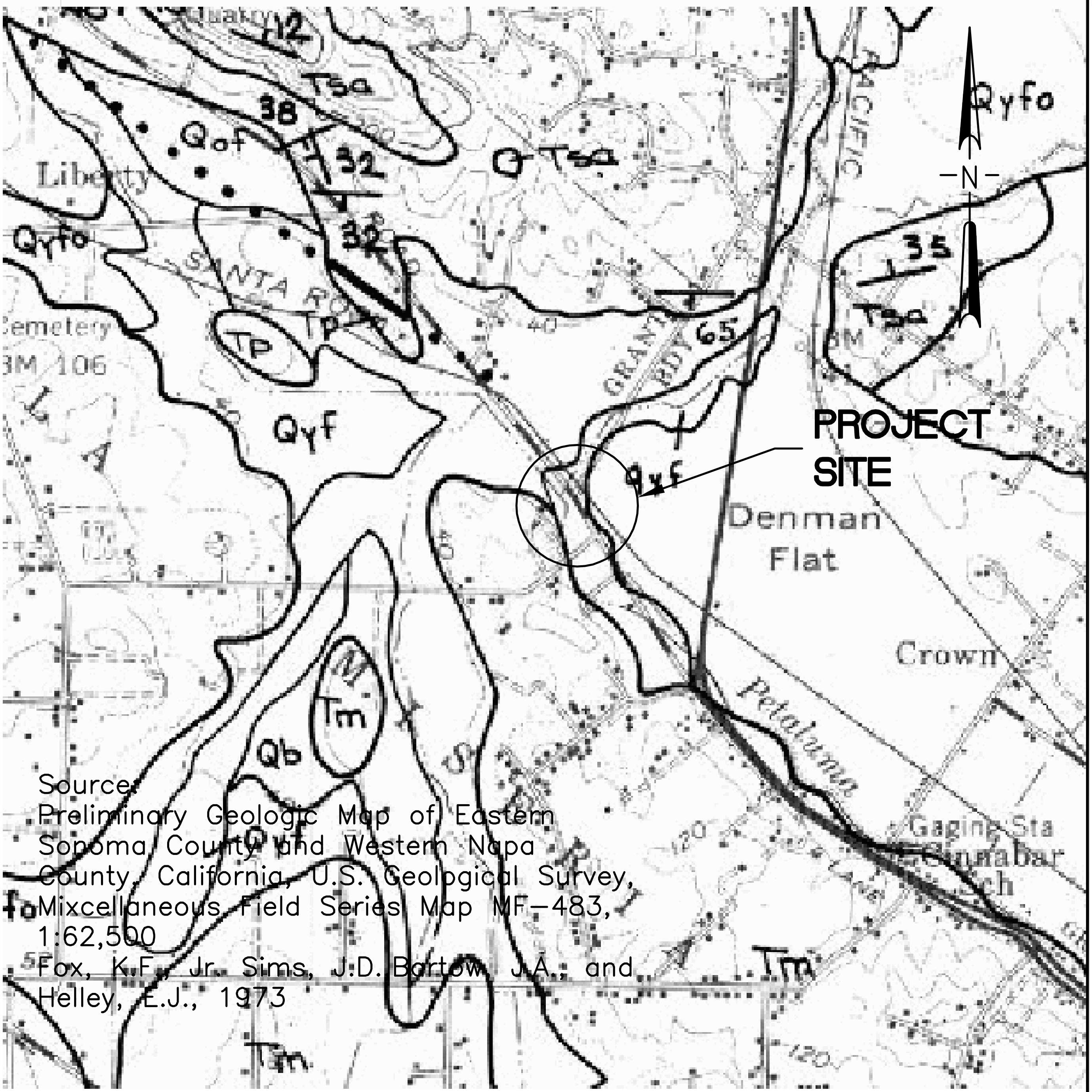
Qal,	alluvium, sand, silt, clay, and gravel
Qf,	fan deposits; gravel, sand, silt, and clay
Qt,	terrace deposits; gravel, sand, silt, and clay
Qoal,	older alluvium; sand, silt, clay, and gravel
Qyf,	alluvial fan deposits grading headward to terrace deposits incised in unit Qof: consist of moderately sorted fine sand and silt, with gravel becoming more abundant toward fan heads
Qyfo,	fluvial deposits at the outer edge of alluvial fans (Qyf); forms levees between basin deposits (Qb); characterized by fine, but variable, grain size; composed mainly of fine sand, silt, and silty clay
Qb,	interfluvial marsh-like basin deposits; mainly poorly sorted dark clay and silty clay, both rich in organic matter
Qbm,	bay mud
Qof,	alluvial fan deposits bordering uplands; heads of fans incised by channels partly filled by terraced deposits of younger alluvium (Qb, Qyfo, and Qyf); outer margins of fans overlapped by younger alluvial deposits (Qb, Qyfo, and Qyf); also includes deposits on stream terraces in narrow canyons cut into uplands; mainly deeply weathered poorly sorted coarse sand and gravel
Qodf,	alluvial fan deposits, moderately to highly dissected; consist of coarse to very coarse, highly weathered gravels



URS	Highway 101 HOV Lane Segment B Sonoma County, California	REGIONAL GEOLOGY MAP	1" = 2000' Figure 3-1 09/24/09

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER RAMSEY J. HISSEN No. C-39608 Exp. 12-31-09 CIVIL STATE OF CALIFORNIA

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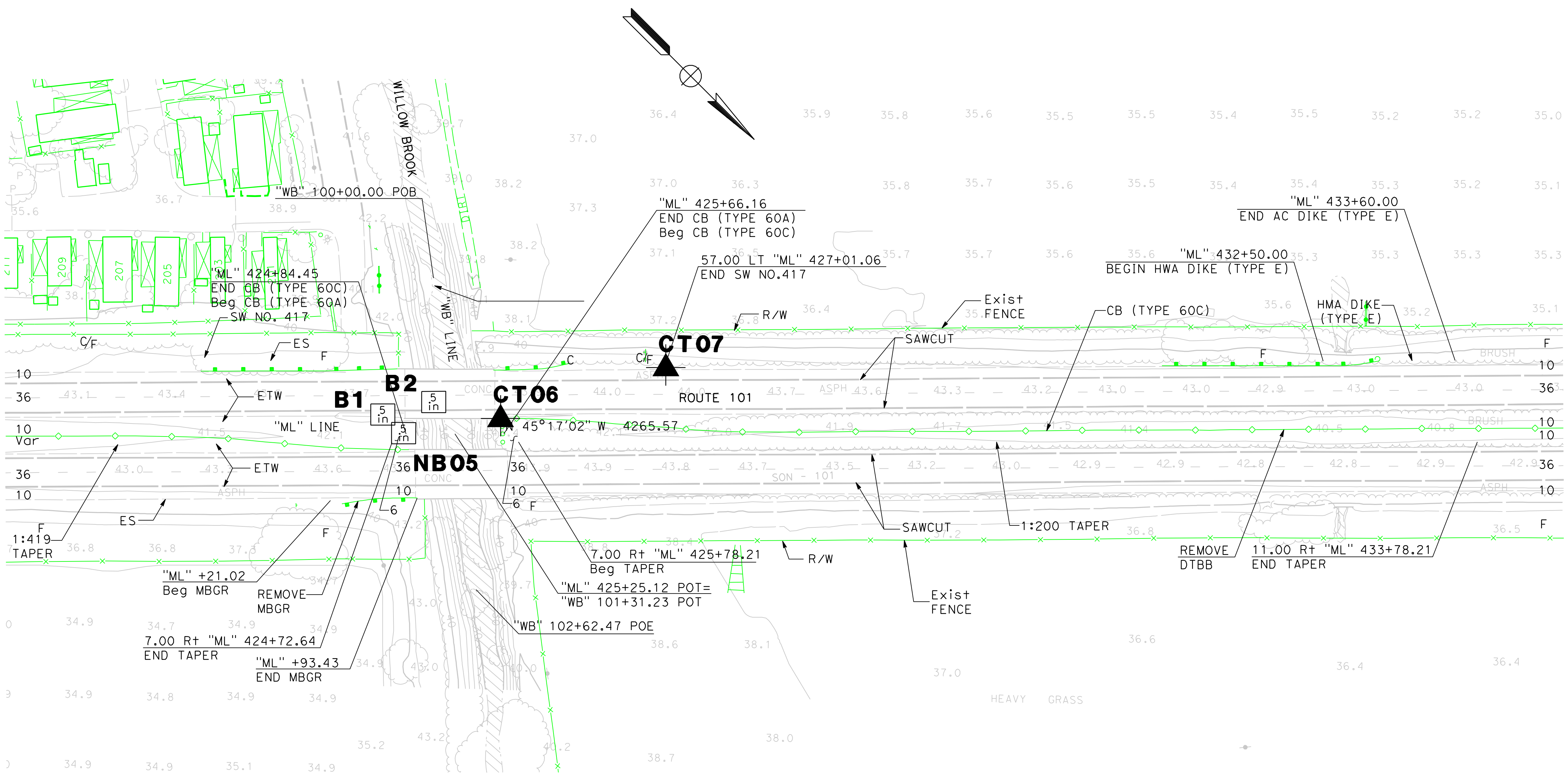
SANTA ROSA, CA 95041

URS CORPORATION
55 S. MARKET STREET
SUITE 1500
SAN JOSE, CA 95113

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



LEGEND:

CT06 ▲ EXPLORATIONS FOR BRIDGE WIDENING

NB05 [5 in]

B1, B2 [5 in] BORING BY OTHERS (1952)

SITE AND BORING LOCATION PLAN
FIGURE 3-2

SCALE 1"=50'

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: RAMSEY HISSEN

CALCULATED-DESIGNED BY: DUNFA XI

CHECKED BY

REVISED BY

DATE REVISED

DATE PLOTTED => 9/25/2009
TIME PLOTTED => 7:33:28 AM

LAST REVISION 00-00-00

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => .
DGN FILE => ...\OC-Geotech\Report\Fig4.dgn

CU 04276

EA 0A1841

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	SON	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

Stephen Huang No. C 42289 Exp. 03/31/2010 CIVIL STATE OF CALIFORNIA REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

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ALAMEDA COUNTY TRANSPORTATION IMPROVEMENT AUTHORITY (ACTIA)
1333 Broadway, Suite 300
Oakland, CA 94612

URS CORPORATION
1333 Broadway,
Suite 800
Oakland, CA 94612

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

CEMENTATION

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

CONSISTENCY OF COHESIVE SOILS

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

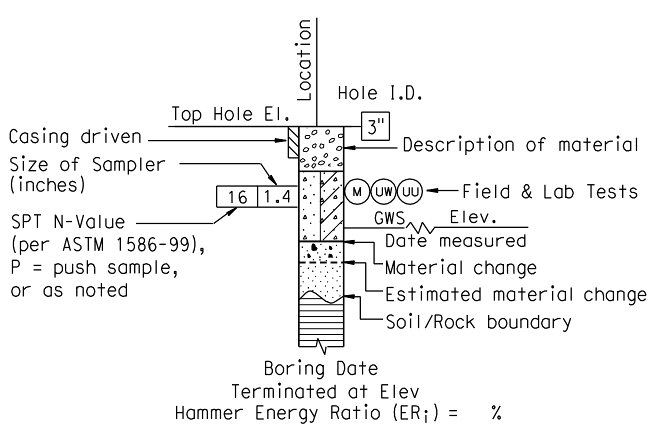
BOREHOLE IDENTIFICATION

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

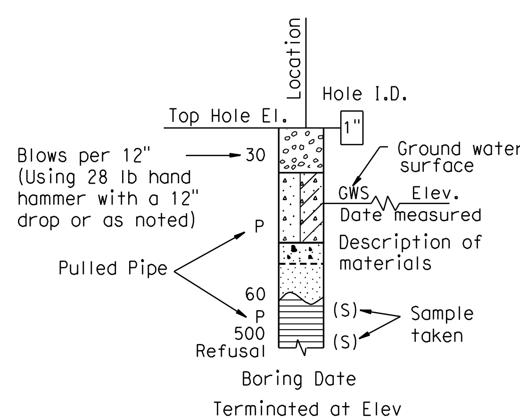
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

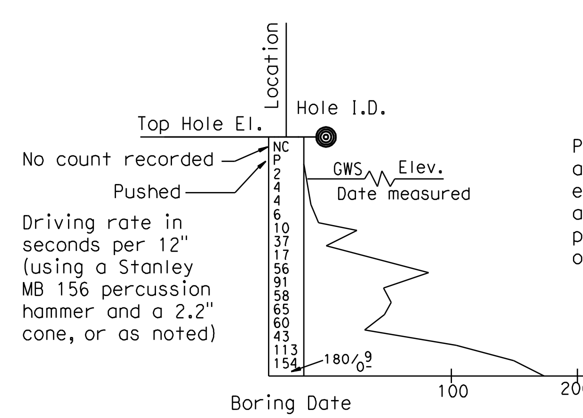
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be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



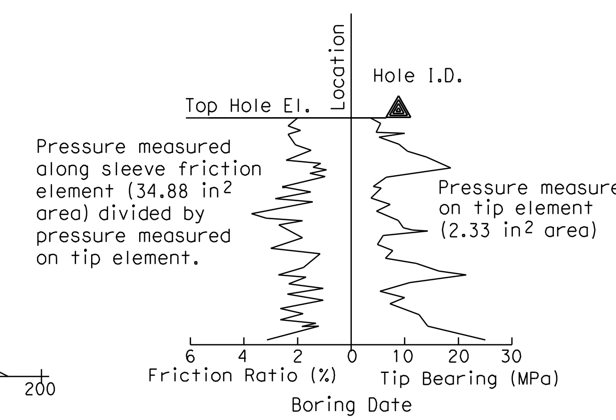
ROTARY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

WILLOW BROOK BRIDGE (WIDEN)
FIGURE 3-3a

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN	BRIDGE NO. 20-0161	SOIL LEGEND
FUNCTIONAL SUPERVISOR NAME: STEPHEN HUANG	DRAWN BY: E. GARNICA CHECKED BY: A. MOORE	FIELD INVESTIGATION BY: C.RAMBO		DESIGN BRANCH	POST MILES 19.31	LOG OF TEST BORINGS (1 of 5)
OGS CIVIL LOG OF TEST BORINGS SHEET		ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3		CU 04 276 EA 0A 1841	DISREGARD PRINTS BEARING EARLIER REVISION DATES	REVISION DATES · SHEET 1 OF 5

FILE => ...\Report\legend-soil1.dgn

USERNAME => madhu_thummalu DATE PLOTTED => 9/25/2009 TIME PLOTTED => 10:09:46 AM

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

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	SON	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER Stephen Huang No. C 42289 Exp. 03/31/2010 CIVIL STATE OF CALIFORNIA

PLANS APPROVAL DATE

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URS CORPORATION
1333 Broadway,
Suite 800
Oakland, CA 94612

GROUP SYMBOLS AND NAMES

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

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

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

APPARENT DENSITY OF COHESIONLESS SOILS

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

MOISTURE

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

PERCENT OR PROPORTION OF SOILS

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

PARTICLE SIZE

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

WILLOW BROOK BRIDGE (WIDEN)
FIGURE 3-3b

ENGINEERING SERVICES		GEOTECHNICAL SERVICES
FUNCTIONAL SUPERVISOR	DRAWN BY: E. GARNICA	FIELD INVESTIGATION BY: C.RAMBO
NAME: STEPHEN HUANG	CHECKED BY: A. MOORE	

STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN

DESIGN BRANCH

BRIDGE NO. 20-0161

POST MILES 19.31

SOIL LEGEND
LOG OF TEST BORINGS (2 of 5)

OGS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 04 276
EA 0A 1841

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES

SHEET 2 OF 5

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	SON	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Stephen Huang No. C 42289 Exp. 03/31/2010 CIVIL STATE OF CALIFORNIA

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Oakland, CA 94612

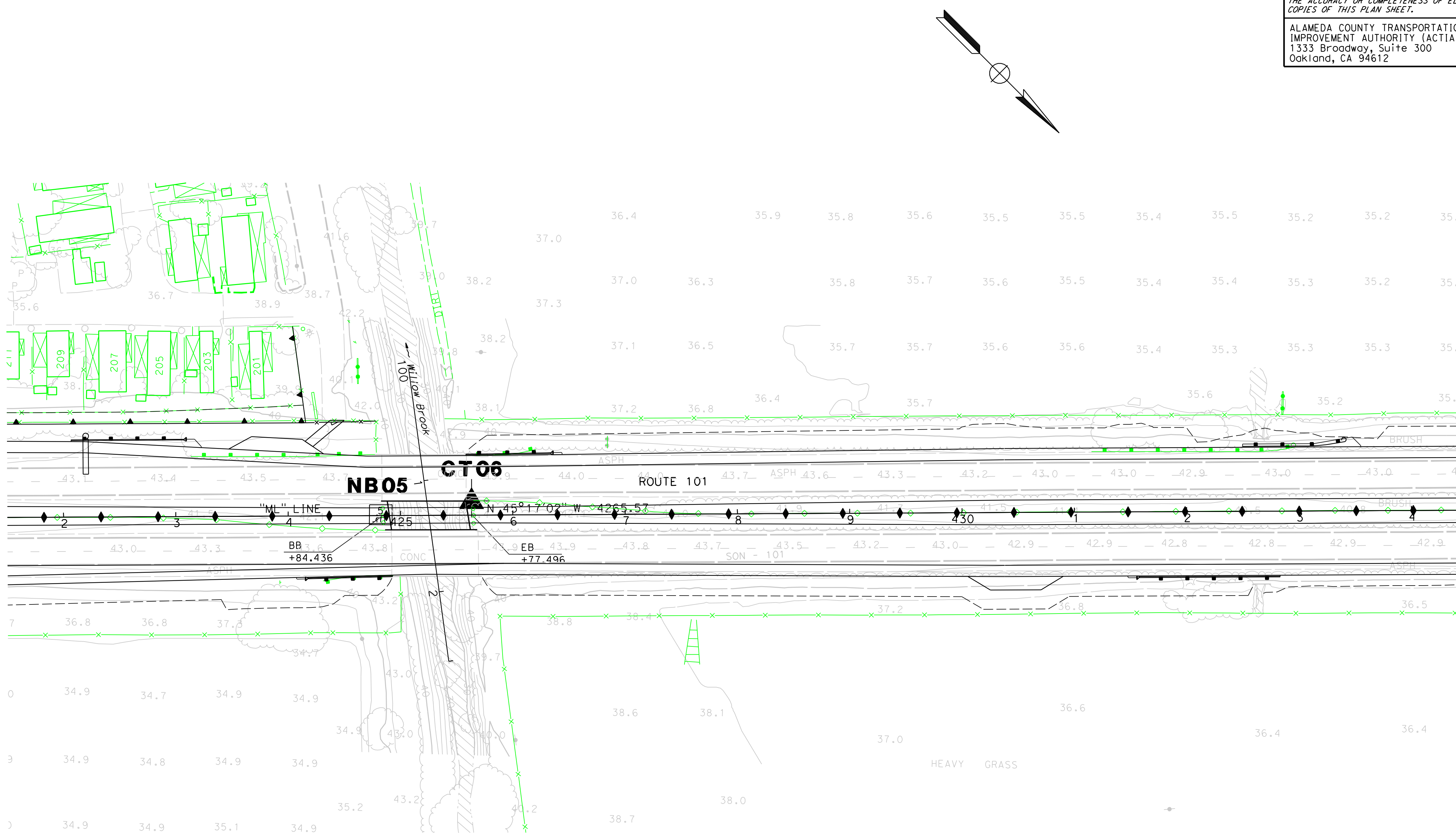

FIGURE 3-4a

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN / DESIGN BRANCH	BRIDGE NO. 20-0161 / POST MILES 19.31	WILLOW BROOK BRIDGE (WIDEN) LOG OF TEST BORINGS (3 of 5)
FUNCTIONAL SUPERVISOR NAME: STEPHEN HUANG	DRAWN BY: E. GARNICA CHECKED BY: A. MOORE	FIELD INVESTIGATION BY: C.RAMBO				

OGS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 04 276
EA 0A 1841

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES

SHEET 3 OF 5

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	SON	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Stephen Huang No. C 42289 Exp. 03/31/2010 CIVIL STATE OF CALIFORNIA

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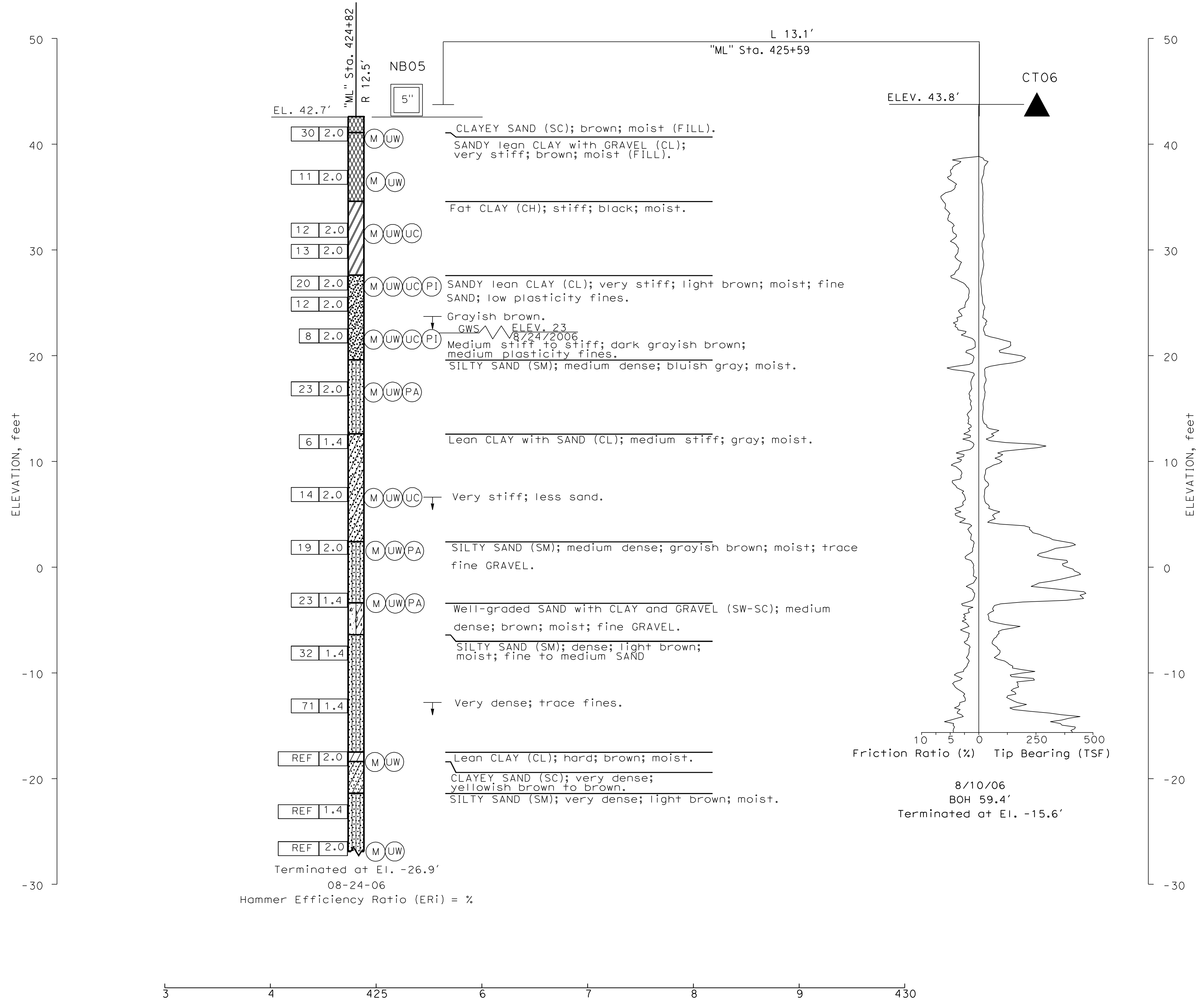

NOTES:

1. 2 inch diameter samples were retrieved using a Modified California Sampler with an inside diameter (ID) of 2 inches and an outside diameter (OD) of 2½ inches.
2. 1⅜ inch diameter samples were retrieved using a Standard Penetration Test Sampler with an ID of 1⅜ inch and an OD of 2 inches.
3. 3 inch diameter samples were retrieved using a Shelby Tube Sampler with an ID of 2⅞ inches and a OD of 3 inches. The sampler was hydraulically pushed at sample depth intervals in soft clays and silts. For samples in soft rock, the Shelby Tube formed the inner tube of a Pitcher Barrel sampler. A Pitcher Barrel sampler combines a Shelby Tube (protruding out the bottom) with the ability for overcoring.
4. Blow counts shown in boring logs are actual field blow counts; no adjustment for sampler type was made.
5. Sample penetration was based on a safety hammer weighing 140 pounds, falling 30 inches, raised and released by the rope and pulley technique.
6. The CPT soundings were performed in accordance with ASTM Standard D 5778-95. The cone has a tip area of 2.325 square inches and friction sleeve area of 34.875 square inches. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.85.
7. Design groundwater level is assumed to be El. 27.5 feet.

FIGURE 3-4b

ENGINEERING SERVICES		GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN	BRIDGE NO. 20-0161	WILLOW BROOK BRIDGE
FUNCTIONAL SUPERVISOR	DRAWN BY: E. GARNICA	FIELD INVESTIGATION BY: C.RAMBO		DESIGN BRANCH	POST MILES 19.31	LOG OF TEST BORINGS (4 of 5)
NAME: STEPHEN HUANG	CHECKED BY: A. MOORE					
OGS CIVIL LOG OF TEST BORINGS SHEET		ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3		CU 04 276 EA 0A 1841	DISREGARD PRINTS BEARING EARLIER REVISION DATES	REVISION DATES SHEET 4 OF 5

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F-0145 (2)

DIST.	STATE			
2	CAL			85

IV	Son	101	A3-73

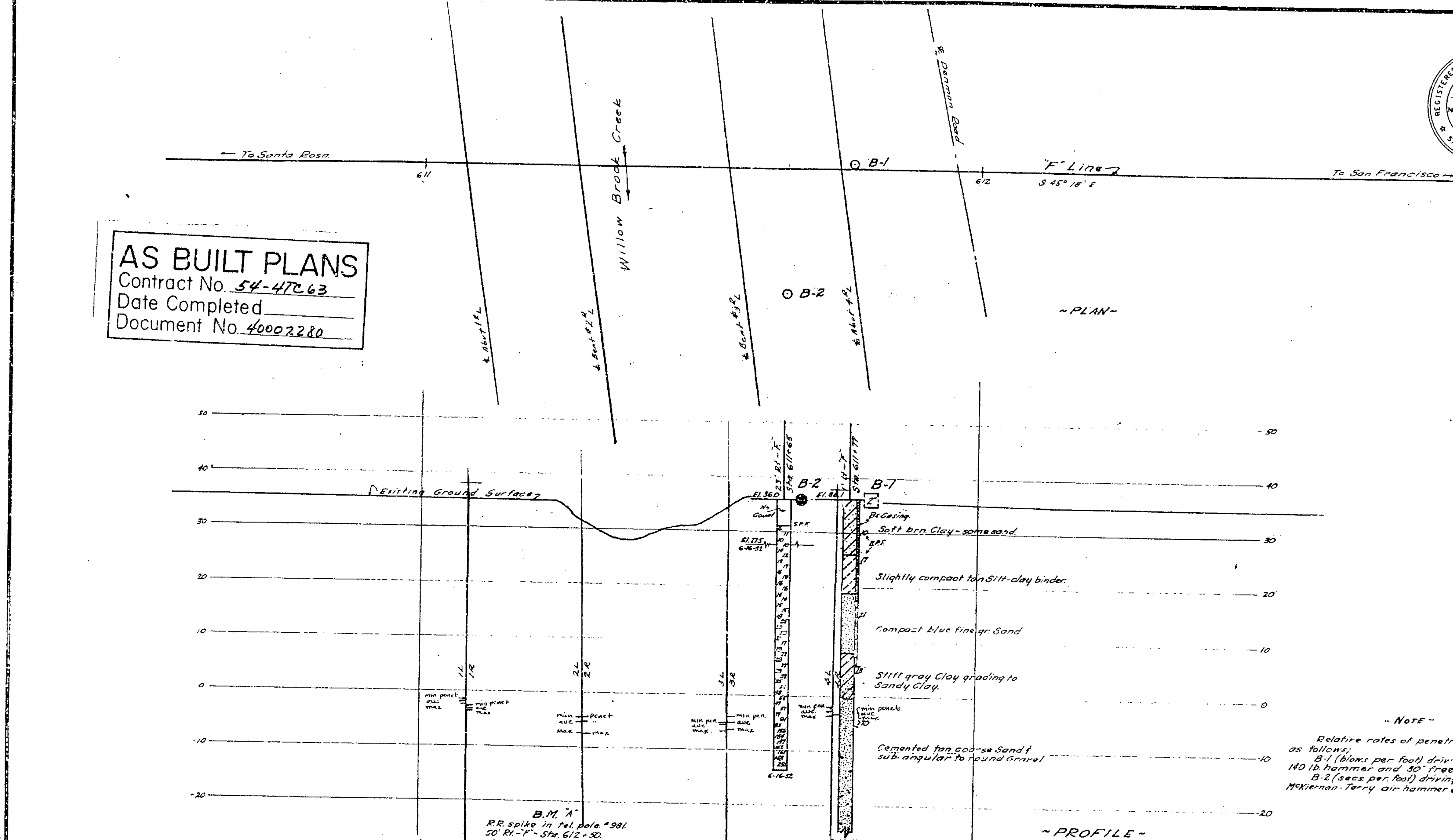
Stephen Huang
No. C 42289
Exp. 03/31/08
REGISTERED PROFESSIONAL ENGINEER
GEOTECHNICAL
STATE OF CALIFORNIA

Caltrans Metric

March 15, 1954

AS BUILT PLANS

Contract No. 54-4TC63
Date Completed
Document No. 40007280



- NOTE -

Relative rates of penetration were obtained as follows;
B-1 (blows per foot) driving 1.4" Sampler with 140 lb. hammer and 30" free fall.
B-2 (secs per foot) driving 1⅝" A-Rod with No. 2 McKiernan-Terry air hammer @ 115 p.s.i.

Notes:

1. See the plans dated February 8, 1955 for stationing. For example F Line Sta 366+20 (English) = ML Line Sta 95+59.29 (Metric).
2. Structure Design produced the data presented in the table below. The data are the metric locations for the As-Built Test Borings referenced to the proposed new structure location. This table is presented on the As-Built Log of Test Boring sheet for the convenience of any bidder, contractor or other interested party.

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DIVISION OF ENGINEERING SERVICES - GEOTECHNICAL SERVICES

As-Built Log of Test Borings sheet is considered an informational document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date confirm that this is a true and accurate copy of the original document. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

DIST.	COUNTY	ROUTE	KILOMETER POST-TOTAL PROJECT	Sheet No.	Total Sheets
04	Son	101	11.5/22.4		

REGISTERED GEOTECHNICAL ENGINEER — 06/29/07 DATE

DATE PLOTTED => 9/24/2009

COPELAND CREEK BRIDGE (WIDEN)

LOG OF TEST BORINGS 3 OF 3

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

CU: 04 276
EA: 0A1800

BRIDGE No. 20-0161

Sheet of

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

Boring	Station	Offset from ML Line (101)
B-1	20+74	0 m Rt
B-2	20+78	7.0 m Lt

CONTRACT No. 54-4TC63-F
DATE ACCEPTED DEC 24 1956
AS BUILT
RESIDENT ENGINEER
REVISIONS BY R.E.B. DATE 12/15/55

BRIDGE DEPARTMENT

CLASSIFICATION OF MATERIAL BASED ON STANDARD GRADE SIZE LIMITS

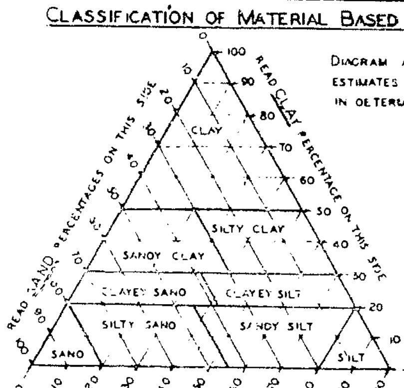


DIAGRAM AND TABLE SHOWING THE BASIS FOR ESTIMATES OF GRADE SIZE DISTRIBUTION USED IN DETERMINATION OF CLASS NAMES.

GROUPING USED IN TRIANGULAR CLASSIFICATION

CLASS NAME	PERCENTAGE OF SIZES PRESENT: SAND	SILT	CLAY
SAND	80-100	0-20	0-20
SILTY SAND	45-80	0-55	0-20
SANDY SILT	0-45	35-80	0-20
SILT	0-20	80-100	0-20
CLAYEY SAND	30-80	0-42	20-30
CLAYEY SILT	0-30	32-80	20-30
SANDY CLAY	30-70	0-40	30-50
SILTY CLAY	0-30	20-70	30-50
CLAY	0-50	0-50	50-100

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

LEGEND OF BORING OPERATIONS

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

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

LEGEND OF EARTH MATERIALS

- GRAVEL – G
- SAND – S
- SILT – Si
- CLAY – C
- SILTY SAND – Si S
- CLAYEY SAND – C S
- SANDY SILT – S Si
- CLAYEY SILT – C Si
- SANDY CLAY – SC
- SILTY CLAY – Si C
- PEAT and/or ORGANIC CLAY – O
- SANDSTONE – SS
- SHALE – Sh
- BROKEN ROCK (FRAGMENTS) – Br
- ROCK – R
- FILL MATERIAL

ABBREVIATIONS

EL. 69.4	ELEVATION OF GROUND AT TEST HOLE
bpf	BLOWS PER FOOT – (SEE NOTE ABOVE)
P	PULLED PIPE
M	MOISTURE AS % DRY WEIGHT
EL. 64.3 6-12-47	ELEVATION OF GROUND WATER AND DATE

NOTES

THE CONTRACTOR'S ATTENTION IS DIRECTED TO SECTION 2, ARTICLE (c) OF THE STANDARD SPECIFICATIONS AND TO THE SPECIAL PROVISIONS ACCOMPANYING THIS SET OF PLANS.

CLASSIFICATION OF EARTH MATERIAL AS SHOWN ON THIS SHEET IS BASED UPON FIELD INSPECTION AND IS NOT TO BE CONSTRUED TO IMPLY MECHANICAL ANALYSIS.

FIGURE 3-4C

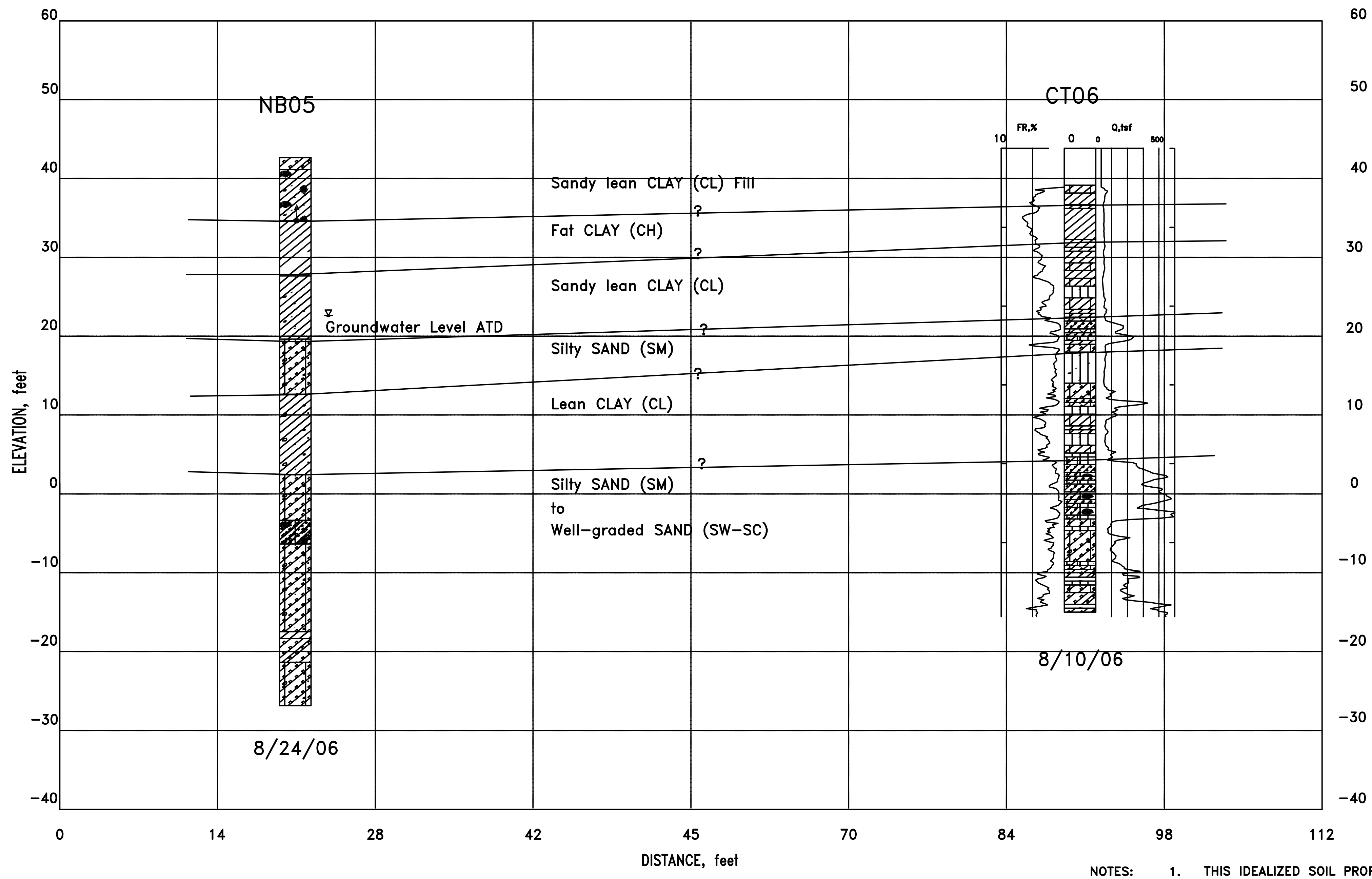
BRIDGE ACROSS WILLOW BROOK

LOG OF TEST BORINGS

SCALE 1" = 10'

BRIDGE No. 20-161 R/L | DRAWING No. C-2967-3

PREL. DRAWING NO. P-2987



Soil Layer	Dry Unit Weight (pcf)	Moisture Content (%)	Undrained Shear Strength (psf)	Relative Density (%)	Internal Friction Angle (Degrees)
Sandy Lean CLAY Fill	95–115	10–18 5–30	940–5850	NA	NA
Fat CLAY	108	20	1460–3970	NA	NA
Sandy Lean CLAY	108	20	1460–3970	NA	NA
Silty SAND	115–127		NA	70–90	38–40
Lean CLAY	115	14	1980–4390	NA	NA
Silty SAND to Well-graded SAND	115–127	10–18	NA	70–90	38–40

NOTES:

1. THIS IDEALIZED SOIL PROFILE HAS BEEN CONSTRUCTED BY DIRECT INTERPOLATION BETWEEN BORINGS DRILLED AT VARYING SPACINGS AND PROJECTED TO THE PROFILE LINE. THE LINES CONNECTING THE VARIOUS LAYERS AT EACH BORING LOCATION ARE FOR SCHEMATIC ILLUSTRATION PURPOSES ONLY AND SHOULD NOT BE CONSTRUED TO REPRESENT THE ACTUAL CONDITIONS IN THE FIELD.
2. FOR DETAILED DESCRIPTIONS OF THE MATERIALS ENCOUNTERED IN EACH OF THE BORINGS DRILLED FOR THIS INVESTIGATION, SEE CORRESPONDING LOG OF TEST BORING SHEETS.
3. THE EXISTING GROUND SURFACE INDICATED IS APPROXIMATE AND BASED UPON TOPOGRAPHIC DATA PRESENTED ON PROJECT LAYOUT PLANS.
4. DESIGN HIGH GROUNDWATER IS AT ELEVATION 27.5 feet.

Project: SCTA SONOMA 101 HOV LANE Project Number: 28649739	GENERALIZED SOIL PROFILE WILLOW BROOK CREEK (WIDEN)	Figure 3-5

4/27/07 2CONEL_04B_FENCE 9739E.GPJ

URS Corporation

FEMA Flood Hazard Areas

Flood Hazard Areas

- Zone V- (100 yr. Flood Zone)
- Zone A- (100 yr. Flood Zone)
- Zone X500- (500 yr. Flood Zone or other concerns)
- Urbanized Area

Shaded to show topographical relief

Detailed FEMA Explanation

Flood Zone	Description
Zone V	This code identifies an area inundated by 1% annual chance flooding with velocity hazard (wave action).
Zone A	This code identifies an area inundated by 1% annual chance flooding.
Zone X500	This code identifies an area inundated by 0.2% annual chance flooding; an area inundated by 1% annual chance flooding with average depths of less than 1 foot or with drainage areas less than 1 square mile; or an area protected by levees from 1% annual chance flooding.

N

Scale: 1 inch equals 1.17 miles

Source: FEMA Q3 Flood Data and ABAG.
The Q3 Flood Data do not replace the existing hardcopy FIRM, or, if one exists, Digital FIRM product. The product has been designed to support planning activities.
A more detailed version of this map is available at http://quake.abag.ca.gov

ABAG Geographic Information Systems

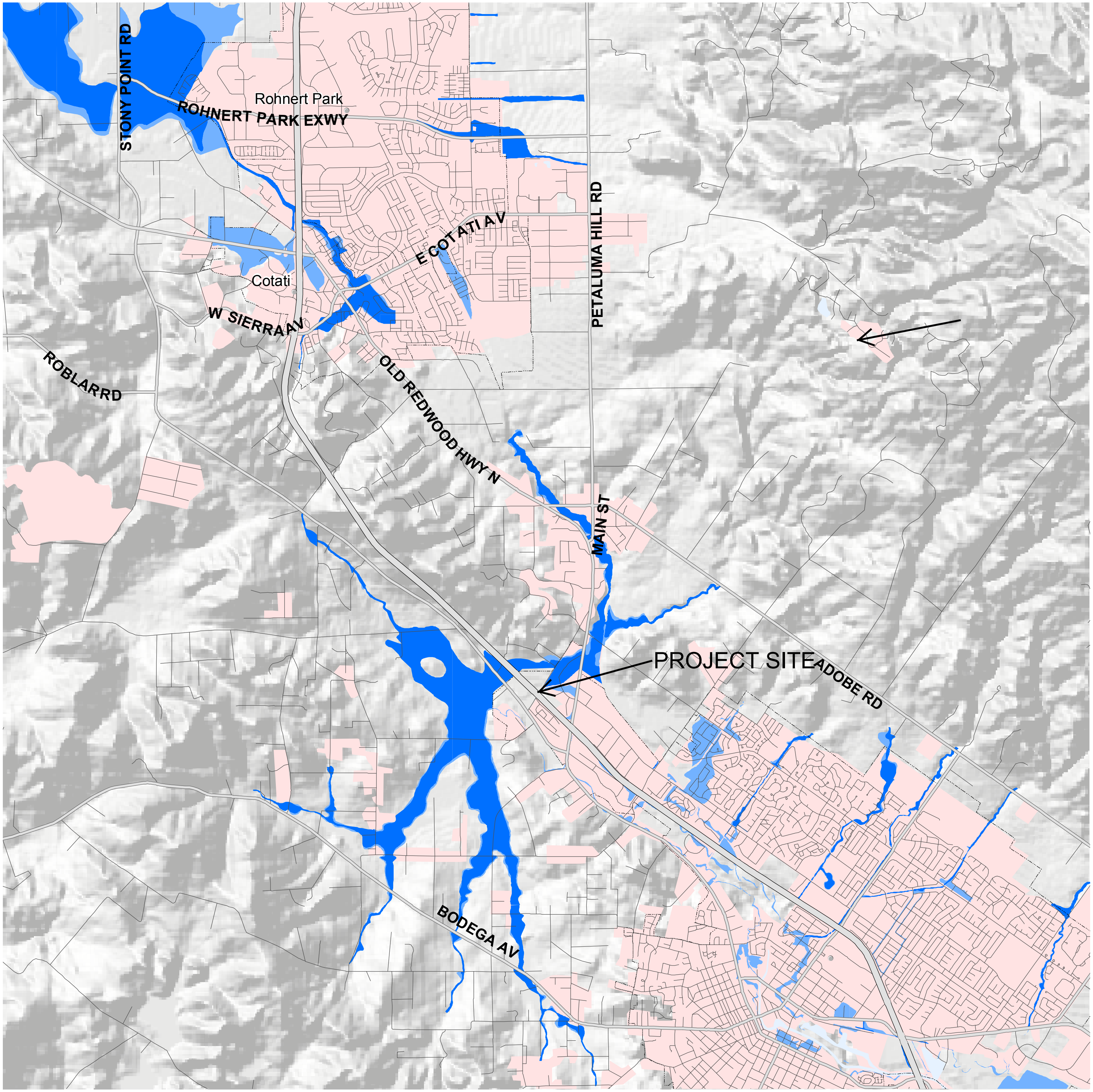


FIGURE 3-6

Liquefaction Susceptibility Map

Susceptibility Level

- Very High
- High
- Moderate
- Low
- Very Low
- Major Roads
- Local Roads

Scale: 1 inch equals 1.25 miles

This map is intended for planning use only and is not intended to be site-specific. Rather, it depicts the general risk within neighborhoods and the relative risk from community to community. More detailed maps are needed for site development decisions.

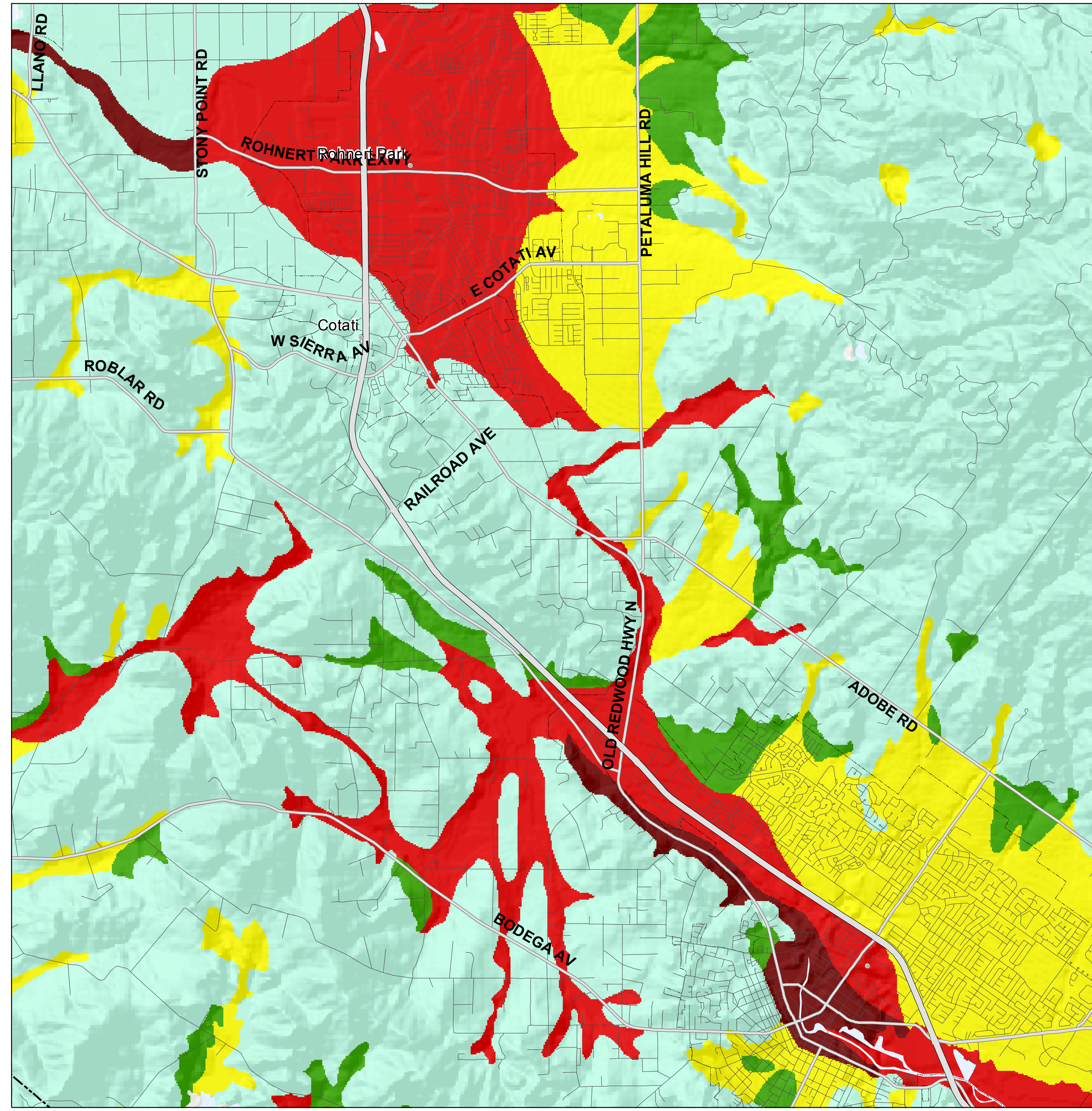
This map is available at http://quake.abag.ca.gov

Source:
This map is based on work by William Lettis & Associates, Inc. and USGS. USGS Open-File Report 00-444, Knudsen & others, 2000

For more information visit:
http://geopubs.wr.usgs.gov/open-file/of00-444/

Map Prepared by the ABAG Earthquake Program.
April 2004.

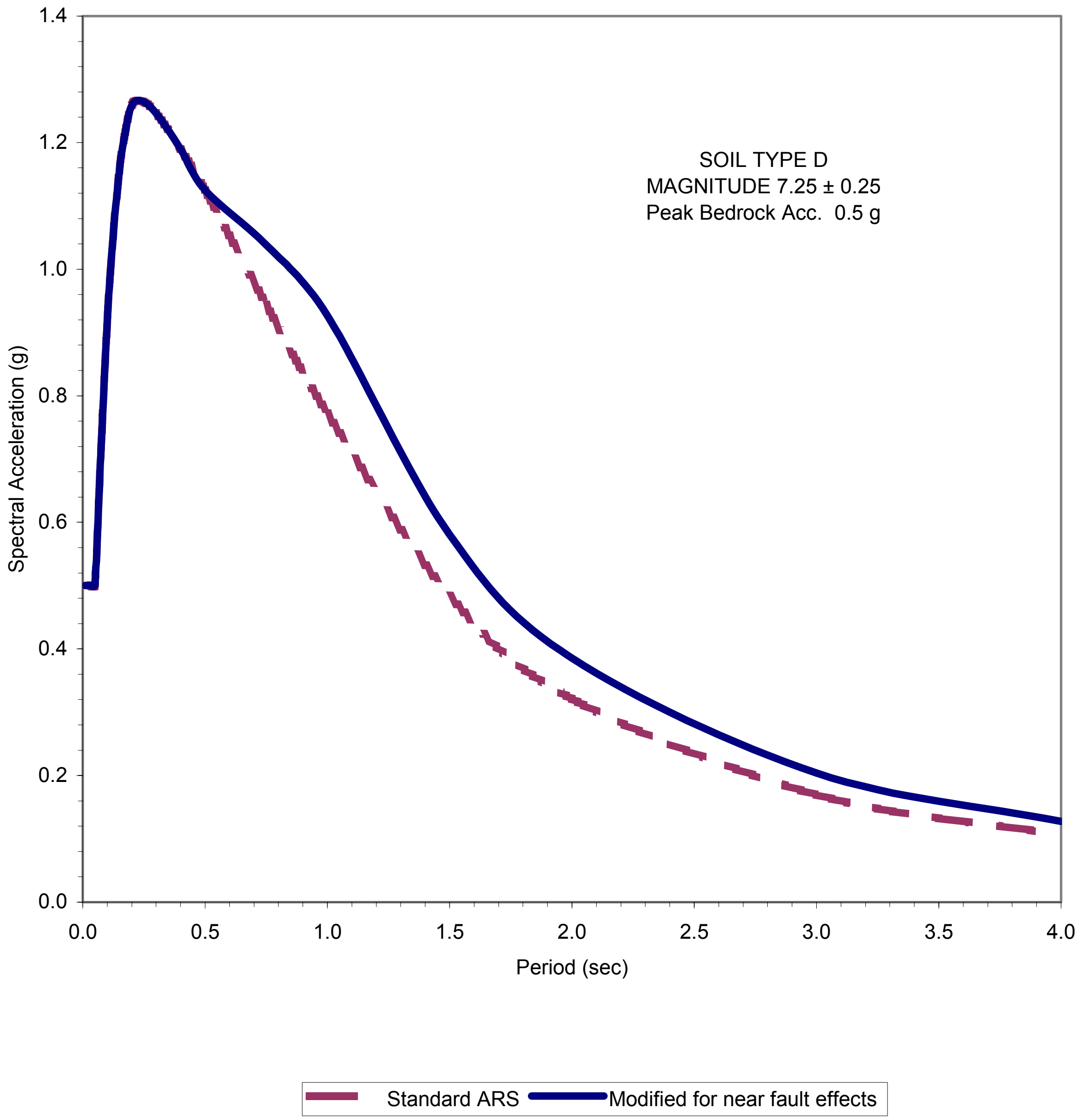
ABAG Geographic Information Systems



Note:

Street names added for clarity.

URS	Highway 101 HOV Lane Project Project No. 28649739	Liquefaction Susceptibility Map Sonoma County, California	Figure 3-7



URS	Project No. 28649739 101 HOV WIDENING PROJECT	RECOMMENDED ACCELERATION RESPONSE SPECTRA	Figure 5-1

Office of Special Funded Projects
Comment & Response Form

General Project Information	Review Phase	Reviewer Information
Dist: _ EA: 0A1831 Project Name: Willow Brook Bridge OSFP Liaison: T. Bertram Phone: 916-227-8397 e-mail: tracy_bertram@dot.ca.gov	☐ PSR/PDS (Review No. _) ☐ APS/PSR (Review No. _) ☐ APS/PR (Review No. _) ☐ Type Selection ☐ 65% PS&E Unchecked Details ☒ PS&E (Review No. 1) ☐ Construction Support ☐ Other:_____ **Structure Information** (Use when necessary to document comments by individual structure) Structure Name: Median widening of Willow Brook Bridge Br No: 20 0161	Reviewer Name: S.Awad/S.Yang Functional Unit: 327 Cost Center: 59 Phone Number: 510-622-5443/510-286-4808 e-mail: samuel_awad@dot.ca.gov Date of Review: 07/29/09

Consultant Information (to be filled in by Consultant)

Consultant Structure Lead (First and Last Name)	Structure Consultant Firm	Phone Number	e-mail	Response Date
_____	URS	408-297-9585	_____	September 24, 2009

#	Doc. (See Note 1)	Page, Section, or SSP	Review Comments	Consultant Responses	
1	90% Foundation Report Bridge No. 20 0161	3.1.2 Site Geology	The Foundation Report references a geologic map and report written by "…Fox et al. 1973." There is no map attached or the report is not reference in the reference section. Please attach a geologic map and reference the geologic report in the reference section.	Will include the geologic map by Fox et al. 1973 in the report as Figure 3-1.	
2	90% Foundation Report Bridge No. 20 0161	3.3.1 Geologic Resources	Caltrans Seismic Hazard Map has been updated in September 2007, which is based on the New Generation Attenuation Relationships developed for California. The consultant should contact Caltrans Office of Earthquake Engineering for the latest information.	Per the implementation memo of 8/6/09, the new Seismic Design Procedures are effective September 30, 2009 for all projects without previous Type Selection Approval. This project had its Type Selection Meeting for this bridge in September 2006 (three years ago), and is therefore exempt from the new Seismic Design Procedures.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer's use)

3	90% Foundation Report Bridge No. 20 0161	3.3.2 Fault-Related Ground Rupture	The dominant faults and fault distances should be updated based on the updated Caltrans Seismic Hazard Map.	See response to comment 2.	
4	90% Foundation Report Bridge No. 20 0161	3.3.6 Subsidence and Seismic Compaction	Seismic compaction should be evaluated based on the updated Caltrans Seismic Hazard Map.	See response to comment 2.	
5	90% Foundation Report Bridge No. 20 0161	3.3.7 Liquefaction Potential	Liquefaction induced settlements should be evaluated based on the updated Caltrans Seismic Hazard Map.	See response to comment 2.	
6	90% Foundation Report Bridge No. 20 0161	3.3.5 Flooding	The Foundation Report references "… FEMA 100-years flood zone," but there is no map attached. Please attach map showing the 100-years flood zone.	Will include the flood zone map in the report as Figure 3-6.	
7	90% Foundation Report Bridge No. 20 0161	4.3 Conclusions	The statement " The proposed structure is not within 1000 ft of salt water". This statement is not derived from laboratory tests; rather, it is a general statement.	We agree. See revised section 4.3 "Conclusions".	
8	90% Foundation Report Bridge No. 20 0161	5.1 Seismic Design Methodology and Resources	Use updated Caltrans Seismic Hazard Map.	See response to comment 2.	
9	90% Foundation Report Bridge No. 20 0161	5.2 Peak Bedrock Acceleration	Please re-evaluate PBA based on updated Caltrans Seismic Map.	See response to comment 2.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer's use)

10	90% Foundation Report Bridge No. 20 0161	5.5 Design Acceleration Response Spectra	Please re-evaluate ARS curve based on updated Caltrans Seismic Hazard Map.	See response to comment 2.	
11	90% Foundation Report Bridge No. 20 0161	Appendix F, Bents 2 and 3	Bottom of pile cap elevation conversion is wrong from Metric to English. 10 m converts to about 33 feet not 40 feet. Please correct it.	Will revise the pile cap elevation to 12.2 m (40 feet).	
12	90% Plans (Initial PS&E)	General Plan, page 1 of 13	The proposed bridge widening does not match with the proposed widening in the submitted 100 % PS&E from design (General Plan page 1 of 16) including widening into the median and joined widening at the southwest shoulder with new Soundwall on concrete barrier. Please revise it.	Subsequent to the submittal of the 100% PS&E “Road Plans” in April, the District approved a design exception that eliminated the outside widening and relocated the sound wall. Next submittal of “Road Plans” will contain revised structural plans.	

Note 1: Abbreviations for Typical Documents (if Abbr. is not below, type in the document type)					
P=Structure Plans	SP=Special Provisions	FR=Foundation Rpt	DC=Design Calcs	TS=Type Sel. Report	QCC=Quant. Check Calcs
RP=Road Plans	E=Estimate	H=Hydraulics Rpt	CC=Check Calcs	QC=Quant. Calcs	

✓= Comment Resolved
(for Reviewer’s use)

California Department of Transportation
Division of Maintenance

Structure Maintenance and Investigations

BRIDGE

INSPECTION

RECORDS

INFORMATION

SYSTEM

The requested documents have been generated by BIRIS.

These documents are the property of the California Department of Transportation and should be handled in accordance with Deputy Directive 55 and the State Administrative Manual.

Records for "Confidential" bridges may only be released outside the Department of Transportation upon execution of a confidentiality agreement.

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number	: 20 0161L
Facility Carried:	U.S. ROUTE 101
Location	: 04-SON-101-8.05-PET
City	: PETALUMA
Inspection Date	: 04/24/2008

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: WILLOW BROOK

CONSTRUCTION INFORMATION

Year Built : 1956
Year Widened: N/A
Length (m) : 21.3

Skew (degrees): 7
No. of Joints : 0
No. of Hinges : 0

Structure Description: Continuous RC slab superstructure on RC (4) pile open "U" type abutments and RC (8) pile bents. All founded on RC piles.

Span Configuration : 1 @ 6.1 m, 1 @ 8.2 m, 1 @ 6.1 m

LOAD CAPACITY AND RATINGS

Design Live Load: MS-18 OR HS-20
Inventory Rating: 39 metric tons — Calculation Method: LOAD FACTOR
Operating Rating: 65.3 metric tons — Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A — Type 3S2 N/A — Type 3-3 N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.2 m br, 0.6 m cu, 11.3 m, 0.6 m cu, 0.2 m br
Total Width: 13.0 m — Net Width: 11.3 m — No. of Lanes: 2
Rail Description: Concrete barrier Type 27 — Rail Code : 1111
Min. Vertical Clearance: Unimpaired

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel

CONDITION TEXT

CONDITION OF STRUCTURE

The maximum water depth in the channel was 18" in Span 2 at the time of investigation. A complete inspection was performed.

There are no significant defects on this structure.

SCOUR

The steel shell pile extensions are vertically exposed up to 10" at Bent 2 and 18" at Bent 3 (no increase in exposure from previous reports). The bottom of columns at Bent 2 were inspected by method of wading and probing.

ELEMENT INSPECTION RATINGS

F#Elem	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
101 39	Concrete Slab - Unprotected w/ AC Overlay	2	280	sq.m.	280	0	0	0	0
101 205	Reinforced Conc Column or Pile	2	16	ea.	16	0	0	0	0

Printed on: Monday 07/14/2008 07:51 AM — 20 0161L/AAAI/13693

F#	Elem	Element Description	Env	Total Qty	Units	Qty in each Condition State St. 1	St. 2	St. 3	St. 4	St. 5
		Extension								
101	215	Reinforced Conc Abutment	2	26	m.	26	0	0	0	0
101	331	Reinforced Conc Bridge Railing	2	67	m.	67	0	0	0	0
101	361	Scour	2	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS - NONE

Inspected By : B.Nekaien

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
BARI A. NEKAIEN
No. C 69959
Exp. 09/30/ 98
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1)	STATE NAME- CALIFORNIA	069
(8)	STRUCTURE NUMBER	**20 0161L**
(5)	INVENTORY ROUTE(ON/UNDER)- ON	121001010
(2)	HIGHWAY AGENCY DISTRICT	04
(3)	COUNTY CODE 097 (4) PLACE CODE	56784
(6)	FEATURE INTERSECTED-	WILLOW BROOK
(7)	FACILITY CARRIED-	U.S. ROUTE 101
(9)	LOCATION-	04-SON-101-8.05-PET
(11)	MILEPOINT/KILOMETERPOINT	8.05
(12)	BASE HIGHWAY NETWORK- PART OF NET	1
(13)	LRS INVENTORY ROUTE & SUBROUTE	000000010101
(16)	LATITUDE	38 DEG 16 MIN 42 SEC
(17)	LONGITUDE	122 DEG 40 MIN 30 SEC
(98)	BORDER BRIDGE STATE CODE % SHARE	%
(99)	BORDER BRIDGE STRUCTURE NUMBER	

******** STRUCTURE TYPE AND MATERIAL *********

(43)	STRUCTURE TYPE MAIN:MATERIAL-	CONCRETE CONT
	TYPE- SLAB	CODE 201
(44)	STRUCTURE TYPE APPR:MATERIAL-	NOT APPLICABLE
	TYPE- NOT APPLICABLE	CODE
(45)	NUMBER OF SPANS IN MAIN UNIT	3
(46)	NUMBER OF APPROACH SPANS	0
(107)	DECK STRUCTURE TYPE- CIP CONCRETE	CODE 1
(108)	WEARING SURFACE / PROTECTIVE SYSTEM:	
A)	TYPE OF WEARING SURFACE- BITUMINOUS	CODE 6
B)	TYPE OF MEMBRANE- NONE	CODE 0
C)	TYPE OF DECK PROTECTION- NONE	CODE 0

*************** AGE AND SERVICE ***************

(27)	YEAR BUILT	1956
(106)	YEAR RECONSTRUCTED	0000
(42)	TYPE OF SERVICE: ON- HIGHWAY	1
	UNDER- WATERWAY	5
(28)	LANES:ON STRUCTURE 02 UNDER STRUCTURE	00
(29)	AVERAGE DAILY TRAFFIC	32100
(30)	YEAR OF ADT 1997 (109) TRUCK ADT	7 %
(19)	BYPASS, DETOUR LENGTH	2 KM

*************** GEOMETRIC DATA ***************

(48)	LENGTH OF MAXIMUM SPAN	8.2 M
(49)	STRUCTURE LENGTH	21.3 M
(50)	CURB OR SIDEWALK: LEFT 0.0 M	RIGHT 0.0 M
(51)	BRIDGE ROADWAY WIDTH CURB TO CURB	11.3 M
(52)	DECK WIDTH OUT TO OUT	13.0 M
(32)	APPROACH ROADWAY WIDTH (W/SHOULDERS)	11.3 M
(33)	BRIDGE MEDIAN- NO MEDIAN	0
(34)	SKEW 7 DEG (35) STRUCTURE FLARED	NO
(10)	INVENTORY ROUTE MIN VERT CLEAR	99.99 M
(47)	INVENTORY ROUTE TOTAL HORIZ CLEAR	11.3 M
(53)	MIN VERT CLEAR OVER BRIDGE RDWY	99.99 M
(54)	MIN VERT UNDERCLEAR REF- NOT H/RR	0.00 M
(55)	MIN LAT UNDERCLEAR RT REF- NOT H/RR	0.0 M
(56)	MIN LAT UNDERCLEAR LT	0.0 M

*************** NAVIGATION DATA ***************

(38)	NAVIGATION CONTROL- NOT APPLICABLE	CODE N
(111)	PIER PROTECTION-	CODE
(39)	NAVIGATION VERTICAL CLEARANCE	0.0 M
(116)	VERT-LIFT BRIDGE NAV MIN VERT CLEAR	M
(40)	NAVIGATION HORIZONTAL CLEARANCE	0.0 M

****

SUFFICIENCY RATING =	94.0
STATUS	
HEALTH INDEX	100.0
PAINT CONDITION INDEX =	N/A

************* CLASSIFICATION ************* CODE

(112)	NBIS BRIDGE LENGTH- YES	Y
(104)	HIGHWAY SYSTEM- ROUTE ON NHS	1
(26)	FUNCTIONAL CLASS- PRIN ART FWY/EXP URBAN	12
(100)	DEFENSE HIGHWAY- STRAHNET	1
(101)	PARALLEL STRUCTURE- LEFT STRUCTURE	L
(102)	DIRECTION OF TRAFFIC- 1 WAY	1
(103)	TEMPORARY STRUCTURE-	
(105)	FED.LANDS HWY- NOT APPLICABLE	0
(110)	DESIGNATED NATIONAL NETWORK - NOT ON NET	0
(20)	TOLL- ON FREE ROAD	3
(21)	MAINTAIN- STATE HIGHWAY AGENCY	01
(22)	OWNER- STATE HIGHWAY AGENCY	01
(37)	HISTORICAL SIGNIFICANCE- NOT ELIGIBLE	5

**************** CONDITION **************** CODE

(58)	DECK	7
(59)	SUPERSTRUCTURE	7
(60)	SUBSTRUCTURE	6
(61)	CHANNEL & CHANNEL PROTECTION	8
(62)	CULVERTS	N

********* LOAD RATING AND POSTING ********* CODE

(31)	DESIGN LOAD- MS-18 OR HS-20	5
(63)	OPERATING RATING METHOD- LOAD FACTOR	1
(64)	OPERATING RATING-	65.3
(65)	INVENTORY RATING METHOD- LOAD FACTOR	1
(66)	INVENTORY RATING-	39.0
(70)	BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS	5
(41)	STRUCTURE OPEN, POSTED OR CLOSED-	A
	DESCRIPTION- OPEN, NO RESTRICTION	

**************** APPRAISAL **************** CODE

(67)	STRUCTURAL EVALUATION	6
(68)	DECK GEOMETRY	5
(69)	UNDERCLEARANCES, VERTICAL & HORIZONTAL	N
(71)	WATER ADEQUACY	8
(72)	APPROACH ROADWAY ALIGNMENT	8
(36)	TRAFFIC SAFETY FEATURES	1111
(113)	SCOUR CRITICAL BRIDGES	5

********** PROPOSED IMPROVEMENTS **********

(75)	TYPE OF WORK-	CODE
(76)	LENGTH OF STRUCTURE IMPROVEMENT	M
(94)	BRIDGE IMPROVEMENT COST	
(95)	ROADWAY IMPROVEMENT COST	
(96)	TOTAL PROJECT COST	
(97)	YEAR OF IMPROVEMENT COST ESTIMATE	
(114)	FUTURE ADT	90345
(115)	YEAR OF FUTURE ADT	2028

*************** INSPECTIONS ***************

(90)	INSPECTION DATE 04/08 (91) FREQUENCY	24 MO		
(92)	CRITICAL FEATURE INSPECTION:			(93) CFI DATE
A)	FRACTURE CRIT DETAIL-	NO	MO	A)
B)	UNDERWATER INSP-	NO	MO	B)
C)	OTHER SPECIAL INSP-	NO	MO	C)

DEPARTMENT OF TRANSPORTATION
Structure Maintenance & Investigations

Caltrans

Bridge Number	: 20 0161R
Facility Carried:	U.S. ROUTE 101
Location	: 04-SON-101-8.05-PET
City	: PETALUMA
Inspection Date	: 04/24/2008

Bridge Inspection Report

Inspection Type

Routine	FC	Underwater	Special	Other
X				

STRUCTURE NAME: WILLOW BROOK

CONSTRUCTION INFORMATION

Year Built : 1956
Year Widened: N/A
Length (m) : 21.3

Skew (degrees): 7
No. of Joints : 0
No. of Hinges : 0

Structure Description: RC continuous deck slab structure on RC (4) pile open "U" type abutments and RC (8) pile bents.

Span Configuration : 1 @ 6.1 m, 1 @ 18.2 m, 1 @ 6.1 m

LOAD CAPACITY AND RATINGS

Design Live Load: MS-18 OR HS-20
Inventory Rating: 39 metric tons Calculation Method: LOAD FACTOR
Operating Rating: 65.3 metric tons Calculation Method: LOAD FACTOR
Permit Rating : PPPPP
Posting Load : Type 3 N/A Type 3S2 N/A Type 3-3 N/A

DESCRIPTION ON STRUCTURE

Deck X-Section: 0.2 m br, 0.6 m cu, 11.3 m, 0.6 m cu, 0.2 m br
Total Width: 13.0 m Net Width: 11.3 m No. of Lanes: 2
Rail Description: Concrete barrier rails Rail Code : 1111
Min. Vertical Clearance: Unimpaired

DESCRIPTION UNDER STRUCTURE

Channel Description: Sand and gravel.

CONDITION TEXT

CONDITION OF STRUCTURE

The channel had 3' of water in Span 2 at the time of the inspection and a complete inspection of all the bridge elements was performed. Columns at both bents were inspected by wading and probing.

There are 1/32" wide longitudinal soffit cracks up to 6' long with no efflorescence.

There is heavy vegetation growth around the structure which hinders the inspection of the wingwalls and abutments.

SCOUR

All 16 columns have up to 15' of exposed pile shell.

The embankment at both abutments has eroded severely and needs to be protected. The face of Abutment 1 is beginning to be undermined at the mid section for a length of 10'. It has been undermined vertically for 1' and horizontally up to 6".

The footing at Abutment 4 is exposed for 20' and is vertically exposed 2" and horizontally for 2".

CONDITION TEXT

ELEMENT INSPECTION RATINGS

F#Elem	Element Description	Env	Total Qty	Units	St. 1	St. 2	St. 3	St. 4	St. 5
101 39	Concrete Slab - Unprotected w/ AC Overlay	2	280	sq.m.	280	0	0	0	0
101 205	Reinforced Conc Column or Pile Extension	2	16	ea.	16	0	0	0	0
101 215	Reinforced Conc Abutment	2	26	m.	26	0	0	0	0
101 331	Reinforced Conc Bridge Railing	2	67	m.	67	0	0	0	0
101 359	Soffit of Concrete Deck or Slab	2	1	ea.	0	1	0	0	0
101 361	Scour	2	1	ea.	0	1	0	0	0

WORK RECOMMENDATIONS

RecDate: 09/06/2000
Action : Remove Vegetation
Work By: BRIDGE CREW
Status : PROPOSED
EstCost: $1,300
StrTarget: 2 YEARS
DistTarget:
EA:

Remove the vegetation growing around the structure, since the vegetation hinders a complete inspection.

RecDate: 09/21/1993
Action : Sub-Scour Mitigate
Work By: MAINT. CONTRACT
Status : PROPOSED
EstCost: $11,500
StrTarget: 2 YEARS
DistTarget:
EA:

Place rock slope protection at both abutments.

Inspected By : Armin Groess

Registered Civil Engineer

REGISTERED PROFESSIONAL ENGINEER
ARMIN G. GROESS
No. C 65930
Exp. 6/30/2010
CIVIL
STATE OF CALIFORNIA

STRUCTURE INVENTORY AND APPRAISAL REPORT

**************** IDENTIFICATION ***************

(1) STATE NAME- CALIFORNIA 069
(8) STRUCTURE NUMBER **20 0161R**
(5) INVENTORY ROUTE(ON/UNDER)- ON 121001010
(2) HIGHWAY AGENCY DISTRICT 04
(3) COUNTY CODE 097 (4) PLACE CODE 56784
(6) FEATURE INTERSECTED- WILLOW BROOK
(7) FACILITY CARRIED- U.S. ROUTE 101
(9) LOCATION- 04-SON-101-8.05-PET
(11) MILEPOINT/KILOMETERPOINT 8.05
(12) BASE HIGHWAY NETWORK- PART OF NET 1
(13) LRS INVENTORY ROUTE & SUBROUTE 000000010101
(16) LATITUDE 38 DEG 16 MIN 42 SEC
(17) LONGITUDE 122 DEG 40 MIN 30 SEC
(98) BORDER BRIDGE STATE CODE % SHARE %
(99) BORDER BRIDGE STRUCTURE NUMBER

******** STRUCTURE TYPE AND MATERIAL *********

(43) STRUCTURE TYPE MAIN:MATERIAL- CONCRETE CONT
TYPE- SLAB CODE 201
(44) STRUCTURE TYPE APPR:MATERIAL- NOT APPLICABLE
TYPE- NOT APPLICABLE CODE
(45) NUMBER OF SPANS IN MAIN UNIT 3
(46) NUMBER OF APPROACH SPANS 0
(107) DECK STRUCTURE TYPE- CIP CONCRETE CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE- BITUMINOUS CODE 6
B) TYPE OF MEMBRANE- NONE CODE 0
C) TYPE OF DECK PROTECTION- NONE CODE 0

*************** AGE AND SERVICE ***************

(27) YEAR BUILT 1956
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON- HIGHWAY 1
UNDER- WATERWAY 5
(28) LANES:ON STRUCTURE 02 UNDER STRUCTURE 00
(29) AVERAGE DAILY TRAFFIC 32100
(30) YEAR OF ADT 1997 (109) TRUCK ADT 7 %
(19) BYPASS, DETOUR LENGTH 2 KM

*************** GEOMETRIC DATA ***************

(48) LENGTH OF MAXIMUM SPAN 8.2 M
(49) STRUCTURE LENGTH 21.3 M
(50) CURB OR SIDEWALK: LEFT 0.0 M RIGHT 0.0 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 11.3 M
(52) DECK WIDTH OUT TO OUT 13.0 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 11.3 M
(33) BRIDGE MEDIAN- NO MEDIAN 0
(34) SKEW 7 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 99.99 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 11.3 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 99.99 M
(54) MIN VERT UNDERCLEAR REF- NOT H/RR 0.00 M
(55) MIN LAT UNDERCLEAR RT REF- NOT H/RR 0.0 M
(56) MIN LAT UNDERCLEAR LT 0.0 M

*************** NAVIGATION DATA ***************

(38) NAVIGATION CONTROL- NOT APPLICABLE CODE N
(111) PIER PROTECTION- CODE
(39) NAVIGATION VERTICAL CLEARANCE 0.0 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR M
(40) NAVIGATION HORIZONTAL CLEARANCE 0.0 M

****

SUFFICIENCY RATING = 94.0
STATUS
HEALTH INDEX 100.0
PAINT CONDITION INDEX = N/A

************* CLASSIFICATION ************* CODE

(112) NBIS BRIDGE LENGTH- YES Y
(104) HIGHWAY SYSTEM- ROUTE ON NHS 1
(26) FUNCTIONAL CLASS- PRIN ART FWY/EXP URBAN 12
(100) DEFENSE HIGHWAY- STRAHNET 1
(101) PARALLEL STRUCTURE- RIGHT STRUCTURE R
(102) DIRECTION OF TRAFFIC- 1 WAY 1
(103) TEMPORARY STRUCTURE-
(105) FED.LANDS HWY- NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - NOT ON NET 0
(20) TOLL- ON FREE ROAD 3
(21) MAINTAIN- STATE HIGHWAY AGENCY 01
(22) OWNER- STATE HIGHWAY AGENCY 01
(37) HISTORICAL SIGNIFICANCE- NOT ELIGIBLE 5

**************** CONDITION **************** CODE

(58) DECK 6
(59) SUPERSTRUCTURE 7
(60) SUBSTRUCTURE 6
(61) CHANNEL & CHANNEL PROTECTION 8
(62) CULVERTS N

********* LOAD RATING AND POSTING ********* CODE

(31) DESIGN LOAD- MS-18 OR HS-20 5
(63) OPERATING RATING METHOD- LOAD FACTOR 1
(64) OPERATING RATING- 65.3
(65) INVENTORY RATING METHOD- LOAD FACTOR 1
(66) INVENTORY RATING- 39.0
(70) BRIDGE POSTING- EQUAL TO OR ABOVE LEGAL LOADS 5
(41) STRUCTURE OPEN, POSTED OR CLOSED- A
DESCRIPTION- OPEN, NO RESTRICTION

**************** APPRAISAL **************** CODE

(67) STRUCTURAL EVALUATION 6
(68) DECK GEOMETRY 5
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL N
(71) WATER ADEQUACY 8
(72) APPROACH ROADWAY ALIGNMENT 8
(36) TRAFFIC SAFETY FEATURES 1111
(113) SCOUR CRITICAL BRIDGES 5

********** PROPOSED IMPROVEMENTS **********

(75) TYPE OF WORK- CODE
(76) LENGTH OF STRUCTURE IMPROVEMENT M
(94) BRIDGE IMPROVEMENT COST
(95) ROADWAY IMPROVEMENT COST
(96) TOTAL PROJECT COST
(97) YEAR OF IMPROVEMENT COST ESTIMATE
(114) FUTURE ADT 90345
(115) YEAR OF FUTURE ADT 2028

*************** INSPECTIONS ***************

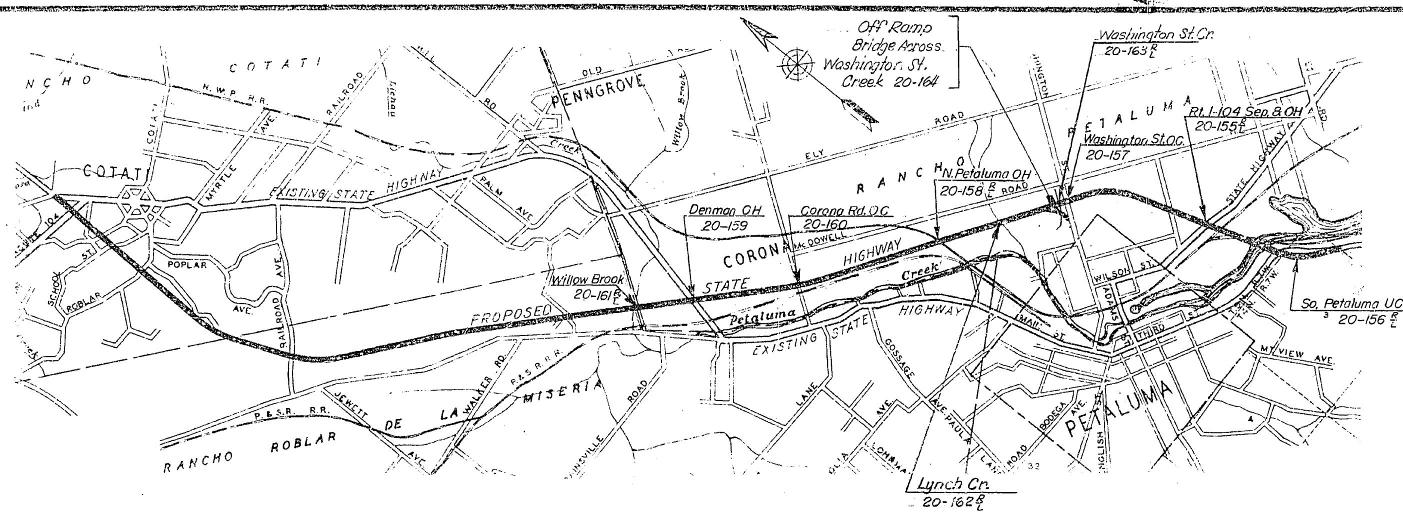
(90) INSPECTION DATE 04/08 (91) FREQUENCY 24 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL- NO MO A)
B) UNDERWATER INSP- NO MO B)
C) OTHER SPECIAL INSP- NO MO C)

F-0145 (2) 764

Fed. Road Dist. No.	State	Fed. Aid Proj. No.	Fiscal Year	Sheet No.	Total Sheets
7	CAL.			82	154

Dist.	County	Route	Section	Sheet No.
IV	Son	1	F	73

Civil and Street Engineer
March 15, 1954
State Highway Engineer
Civil Engineer - License



BRIDGE DEPARTMENT

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B4	Bent 2
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B6	Girder Layout
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C5	Girder Layout
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D7	Log of Test Borings

Sh. No.	Title
"E"	Bridge Across Lynch Creek No. 20-162 R L
E1	General Plan
E2	Foundation Plan
E3	Details.
E4	Log of Test Borings
"F"	Bridge Across Washington St. Creek No. 20-163 R L
F1	General Plan
F2	Foundation Plan.
F3	Deck Details
F4	Log of Test Borings
"G"	Washington St. Overcrossing No. 20-157
G1	General Plan
G2	Foundation Plan
G3	Abutments
G4	Bent 2
G5	Girder Layout & Typical Sections
G6	Girder Reinforcement
G7	Log of Test Borings
G8	Electrical Plan
"H"	Off Ramp Bridge Across Washington St. Creek No. 20-164
H1	General Plan
H2	Abutments
H3	Typical Section & Bents.
H4	Girder Details
H5	Log of Test Borings

Sh. No.	Title
"I"	Route 1/104 Separation & Overhead No. 20-155 R L
I1	General Plan
I2	Foundation Plan
I3	Abutments
I4	Piers
I5	Girder Layout-Spans 1 to 5 Rt. Br.
I6	Girder Reinforcement-Spans 1 to 5 Rt. Br.
I7	Typical Section & Girder Layout-Spans 6 to 8 Rt. Br.
I8	Girder Reinforcement-Spans 6 to 8 Rt. Br.
I9	Girder Layout-Spans 1 to 6 Lt. Br.
I10	Girder Reinforcement-Spans 1 to 6 Lt. Br.
I11	Girder Layout & Reinforcement-Spans 7 & 8 Lt. Br.
I12	Log of Test Borings.
"J"	South Petaluma Undercrossing No. 20-156 R L
J1	General Plan.
J2	Foundation Plan
J3	Abutments
J4	Bents
J5	Typical Section
J6	Girder Layout
J7	Girder Reinforcement - Rt. Br.
J8	Girder Reinforcement - Lt. Br.
J9	Log of Test Borings.
"S"	Standards
S1	Hinge & Bearings
S2	Pile Details
S3	Miscellaneous Details
S4	Lighting Electrical Details - Bridge

GENERAL NOTES

Specifications:
Design: A.A.S.H.O. dated 1949 with subsequent revisions, and Bridge Department Supplement dated 1949.

Construction: Standard Specifications, Division of Highways, dated January 1949 and the Special Provisions.

Live Loading: H20-S16-44

Unit Stresses:
Reinforced Concrete: fs=20,000 p.s.i., fc=1000 p.s.i., n=10
Structural Steel: fs=18,000 p.s.i.

Footing Pressure: 4 tons p.s.f. for Bents Br. No. 20-156 R L and Abut 1 & 4 Br. No. 20-156 L

Pile Loading: 32 tons. Type: Concrete

Reinforcement: Embedment is clear to outside of bar and is 2" to main reinforcement, except as noted. Hooks shall conform to the Manual of Standard Practice, A.C.I. Backing for hooks is four diameters, except as noted. Bar areas are based on rounds for less than 1" and squares for over 1".

Bar deformations shall conform to ASTM A305-49.

Where reinforcing bars are spliced they shall have a 20 diameter lap unless otherwise called for on the plans.

AS BUILT PLANS
Contract No. 54-4TC63
Date Completed
Document No. 40002280

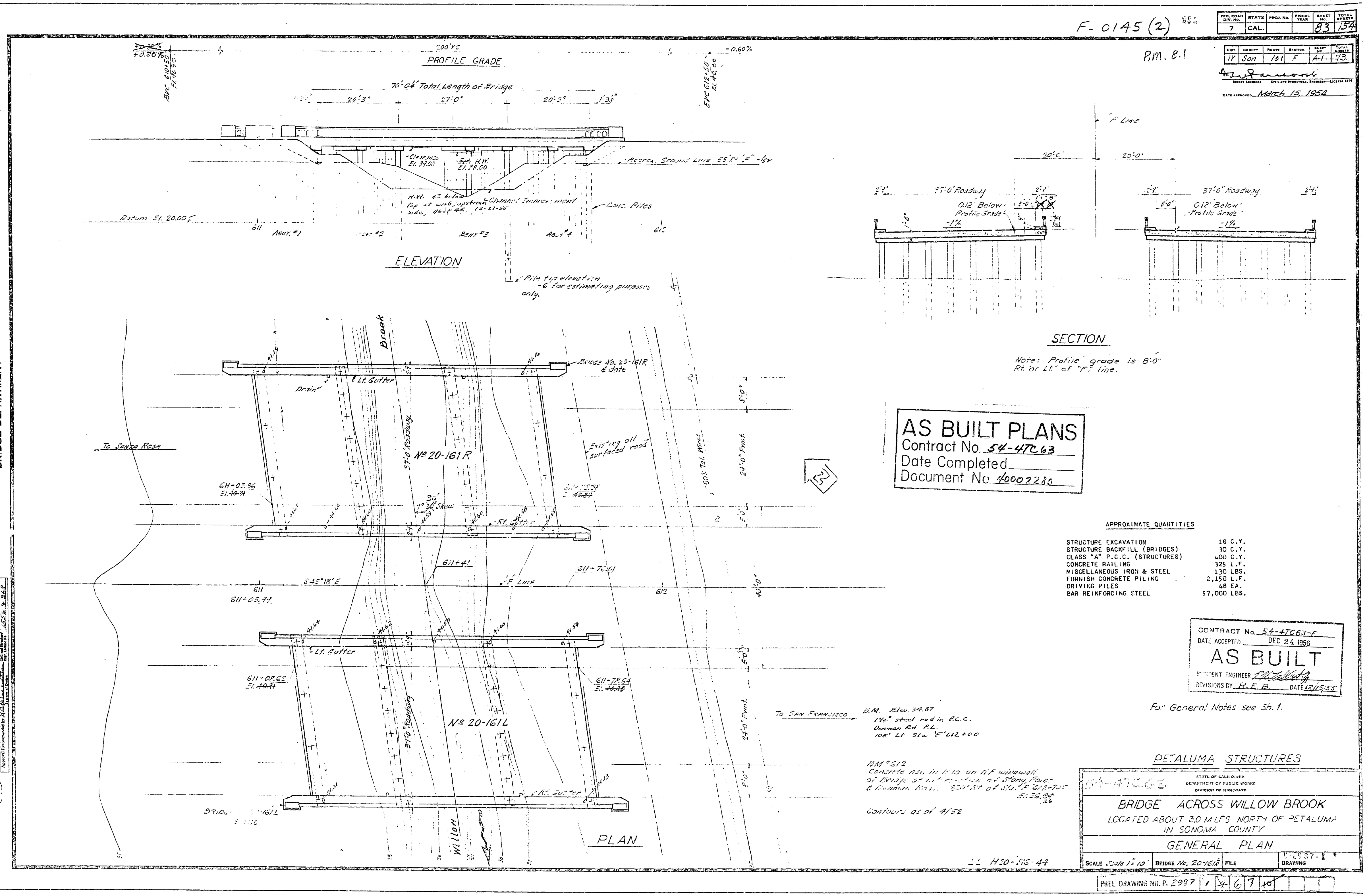
PETALUMA STRUCTURES
54-4TC63
INDEX TO PLANS

SCALE NONE | BRIDGE 20-155 to 20-164 incl.

PREL. DRAWING NO. P-2892 | 14 | 35 | 39

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

DATE ____ SIGNATURE ____ TITLE ____



APPROXIMATE QUANTITIES

STRUCTURE EXCAVATION	18 C.Y.
STRUCTURE BACKFILL (BRIDGES)	30 C.Y.
CLASS "A" P.C.C. (STRUCTURES)	400 C.Y.
CONCRETE RAILING	325 L.F.
MISCELLANEOUS IRON & STEEL	130 LBS.
FURNISH CONCRETE PILING	2,150 L.F.
DRIVING PILES	48 EA.
BAR REINFORCING STEEL	57,000 LBS.

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

DATE ______ SIGNATURE ______ TITLE ______

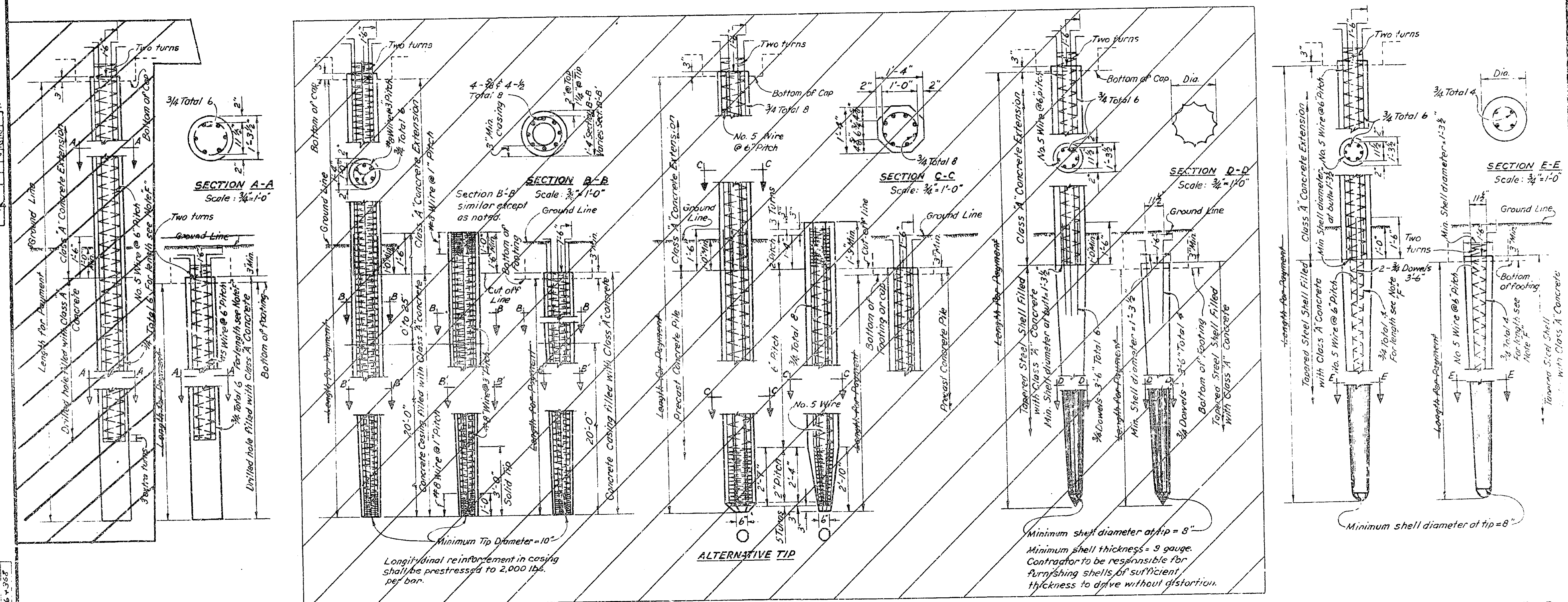
F-0145 (2) 152, 154

March 15, 1954

STANDARD DWG.	Initial	Date
Design		6-52
Detail	R.A.	6-52
Check	R.H.H.	6-52
Quant		
Check		

Supervised:

BRIDGE DEPARTMENT



CAST-IN-DRILLED-HOLE CONCRETE PILE ALTERNATIVE "V"

For Bridge 20-156 R only

CAST-IN-PLACE CONCRETE PILE ALTERNATIVE "W"

PRECAST CONCRETE PILE ALTERNATIVE "X"

Note: Steel "H" Piles used at Br. No. 20-155 R/L (See Plans)

CAST-IN-PLACE CONCRETE PILE ALTERNATIVE "Y"

CAST-IN-PLACE CONCRETE PILE ALTERNATIVE "Z"

Note F: Length of 3/4 round bars, measured below ground line or bottom of footing, to be either 1/3 of estimated penetration or 12', whichever is the greater. For total length of bars, add extension or splice.

AS BUILT PLANS
Contract No. 54-4TC63
Date Completed
Document No. 40002280

CONTRACT No. 54-4TC63-F
DATE ACC. PTED DEC 24 1956
AS BUILT
REVISIONS BY R.E.B. DATE 12/24/56

PETALUMA STRUCTURES

STANDARD

PILE DETAILS

Except as Noted SCALE: 1/2" = 1'-0" | BRIDGE | FILE | DRAWING F-2892-15

PREL. DRAWING NO. P-2892 30

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

DATE ______ SIGNATURE ______ TITLE ______

FIELD EXPLORATION PROGRAM

The geotechnical field investigation consisted of one geotechnical boring and one cone penetration test (CPT), extending to depths of 69½ feet and 59½ feet below the existing ground surface, respectively. The explorations were performed between August 10 and 24, 2006, by Taber Consultants of Sacramento, California (rotary wash boring), and Gregg Drilling and Testing, Inc. of Martinez, California (CPT).

Boring and CPT locations were carefully selected to obtain supplemental subsurface information to provide geotechnical design recommendations for the proposed structure while avoiding underground utilities and subsurface obstructions. Layout of the explorations was performed by representatives of URS, and exploration locations were checked for conflict with underground utilities by contacting Underground Service Alert (USA) Network. USA, in turn, alerted the various municipalities and utility companies that a subsurface investigation was to be conducted near their utilities.

After underground utility clearance, URS obtained permits from the County of Sonoma, Permit and Resource Management Department, Well and Permit Section, and coordinated with appropriate personnel to accommodate the required inspection during and following exploration at each location.

Rotary Wash Boring (NB05)

The rotary wash boring was drilled to provide the necessary information to evaluate the subsurface stratigraphy and to allow acquisition of high-quality soil samples for laboratory testing. The boring was drilled and sampled at the location indicated on the Site and Boring Location Plan, Figure 3-1. The boring was advanced to a depth of 69½ feet below existing ground surface using a truck-mounted drill rig under the supervision of a URS engineer who maintained a record of all field activities, classified the soils encountered using the Unified Soil Classification System (USCS), and prepared a log of the boring.

The drilling operation proceeded carefully, with particular attention to potential interference with utilities or other buried structures. During drilling, both disturbed and undisturbed samples were obtained for identification and laboratory testing. Soil samples were generally obtained at 5 foot intervals and at changes in strata. Samples were obtained using the Modified California (MC) sampler and Standard Split Spoon sampler (SPT). A brief description of each of these samplers follows:

- **Modified California Sampler (MC):** The Modified California Sampler was used to obtain relatively undisturbed samples of embankment fill and stiff to hard alluvial clays and silts. This sampler consists of a tube-lined barrel sampler with a nominal 2 inch inside diameter and 2½ inch outside diameter. A 140 pound hammer falling through a distance of 30 inches was used to drive the MC sampler. The blow count recorded on the boring logs adjacent to the sample depth is the number of blows required to drive the sampler for the final 1 foot of a maximum 1.5 foot drive.

- **Standard Split Spoon Sampler (SPT):** The Standard Split Spoon Sampler was used to obtain disturbed samples of sand and gravel layers. The sampler consists of a split barrel with a nominal $1^{3}/_{8}$ inch inside diameter and a 2 inch outside diameter. The standard penetration resistance of the soil is determined by the number of blows required to drive the sampler 1.5 feet into the soil with a 140 pound hammer falling through a distance of 30 inches. The blow count recorded on the boring logs adjacent to the sample depth is the number of blows required to drive the sampler for the final 1 foot of a maximum 1.5 foot drive.

One of the objectives of the field investigation was to obtain high-quality undisturbed samples for laboratory testing. Effort was made to minimize sample disturbance during sample handling and transportation. After careful withdrawal from the ground, the sample was placed upright and the ends of the sample were cleaned of disturbed soil. Where feasible, pocket penetrometer tests were performed on the bottom end of cohesive soil samples. Both ends of the samples were covered with plastic caps, and carefully transported to URS' laboratory.

Disposal of Cuttings

All drill cuttings and fluids generated during drilling of rotary wash boring were collected in drums for disposal in accordance with applicable local, state, and federal regulations.

Cone Penetration Test (CT06)

We also performed one CPT at the location shown on the Site and Boring Location Plan, Figure 3-1. The CPT was advanced to a depth of 59½ feet below existing ground surface under the supervision of a URS representative.

The Cone Penetration Test (CPT) consists of pushing a cone-tipped probe into the soil while simultaneously recording the cone tip resistance and side friction resistance to penetration. The CPT was conducted in general accordance with ASTM specification (ASTM D3441-79) using an electric cone penetrometer. The CPT equipment consists of a cone assembly mounted at the end of a series of hollow rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed all-wheel drive truck is used to transport and house the test equipment and to provide a 20 ton reaction to the thrust of the hydraulic rams.

The cone penetrometer assembly consists of a conical tip and a cylindrical friction sleeve. The conical tip has a 60-degree apex angle and a projected cross-sectional area of 2.325 square inches. The cylindrical friction sleeve has a surface area of 34.875 square inches. Both the conical tip and the cylindrical friction sleeve have outer diameters of 1¾ inches. The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurement of cone tip and friction sleeve resistance during penetration. Continuous electric signals from the strain gauges are transmitted by a cable in the sounding rods to analog and digital data recorders in the CPT truck.

Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations that utilize the CPT data. Soil interpretation presented on the CPT logs from this investigation was based on recent correlations developed by Robertson, 1990, presented on Figure B-1.

LABORATORY TESTING PROGRAM

A laboratory testing program was carried out to determine the index and engineering properties of the major subsurface strata encountered at the site. The laboratory testing program included conventional tests to confirm the existing information on the engineering characteristics of the major strata and to refine some of the engineering parameters. These tests were performed at the URS' laboratory.

Index Tests

Index tests were performed on both cohesive and cohesionless soil samples to aid in soil classification and in correlation with other engineering parameters. Index tests included Atterberg Limits, moisture content, dry density, and grain size distribution determinations. Atterberg Limits tests were performed in accordance with ASTM D 4318. The moisture content tests were performed in accordance with ASTM D 2216. Dry density was determined in accordance with ASTM D 2937. Gradation analyses were performed in accordance with ASTM D 422. The locations of these tests are indicated on the Logs of Test Borings adjacent to the appropriate sample depths. The results are summarized in Table C-1.

A plasticity chart graphically presenting the results of the Atterberg Limits tests is presented on Figure C-2. Grain size distribution curves are shown on Figure C-3.

Unconfined Compression Tests

Unconfined compression tests were performed on select cohesive soil samples to assist in determining shear strength parameters. These tests were performed in accordance with ASTM D 2166. The results of these tests are indicated on the Logs of Test Boring Sheet adjacent to the appropriate sample depths.

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT.* It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

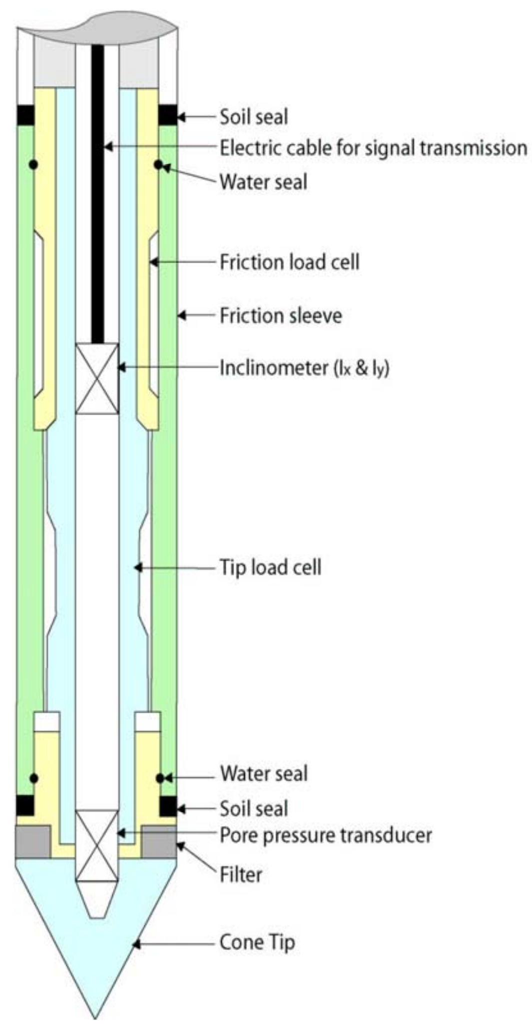


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 10 cm^2 and a friction sleeve area of 150 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

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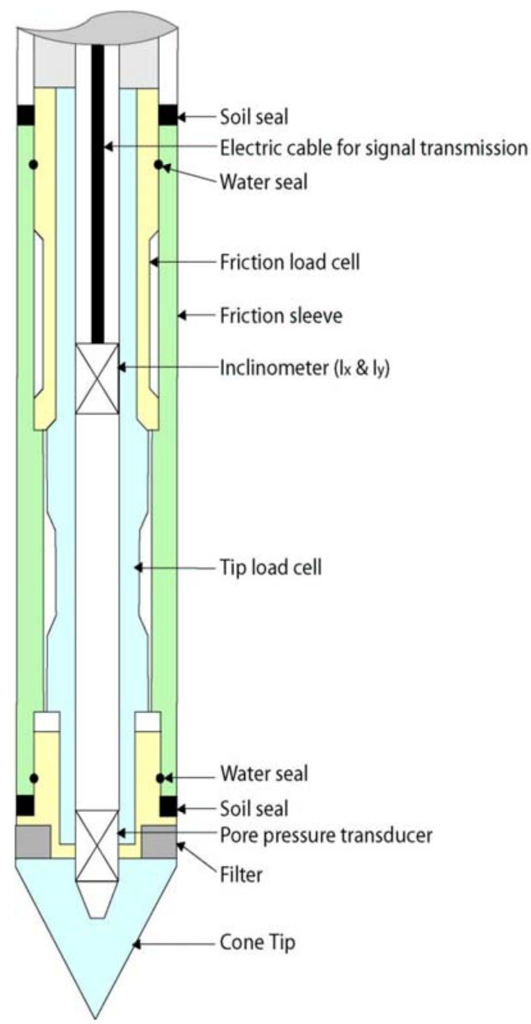


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

Pore Pressure Dissipation Tests (PPDT)

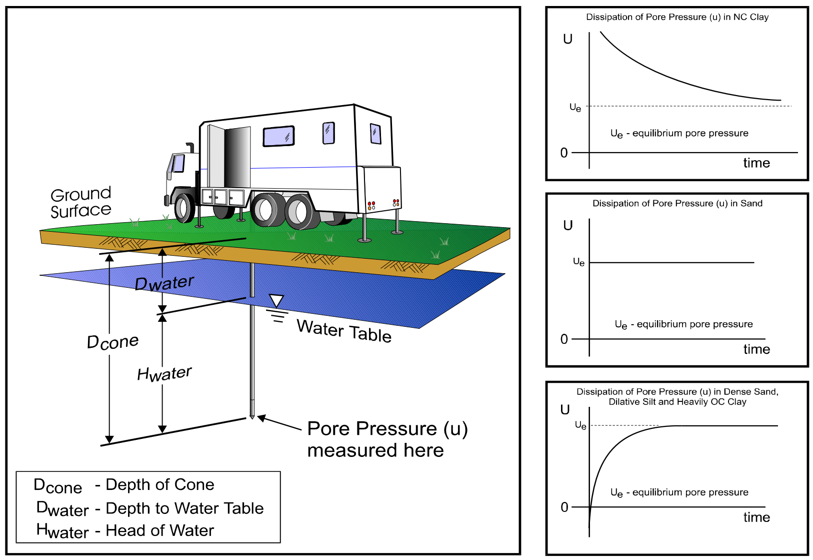
Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992.

A summary of the pore pressure dissipation tests is summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Cone Penetration Test Data & Interpretation

Soil behavior type and stratigraphic interpretation is based on relationships between cone bearing (q_c), sleeve friction (f_s), and pore water pressure (u_2). The friction ratio (R_f) is a calculated parameter defined by $100 f_s/q_c$ and is used to infer soil behavior type. Generally:
Cohesive soils (clays)

- High friction ratio (R_f) due to small cone bearing (q_c)
- Generate large excess pore water pressures (u_2)

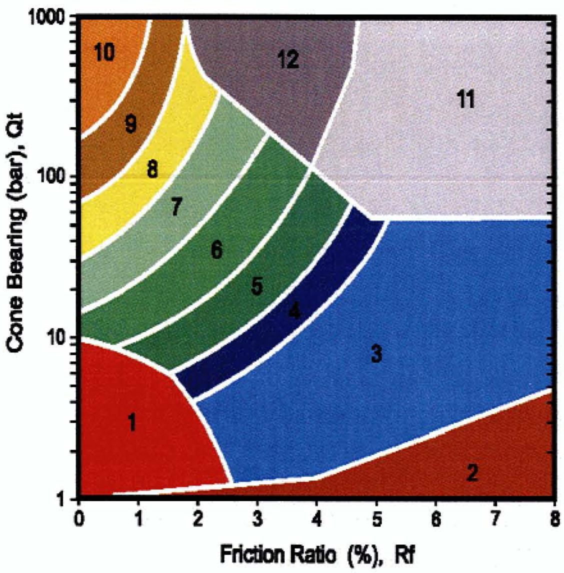
Cohesionless soils (sands)

- Low friction ratio (R_f) due to large cone bearing (q_c)
- Generate very little excess pore water pressures (u_2)

A complete set of baseline readings are taken prior to and at the completion of each sounding to determine temperature shifts and any zero load offsets. Corrections for temperature shifts and zero load offsets can be extremely important, especially when the recorded loads are relatively small. In sandy soils, however, these corrections are generally negligible.

The cone penetration test data collected from your site is presented in graphical form in Appendix CPT. The data includes CPT logs of measured soil parameters, computer calculations of interpreted soil behavior types (SBT), and additional geotechnical parameters. A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Soil interpretation for this project was conducted using recent correlations developed by Robertson, 1990, *Figure SBT*. Note that it is not always possible to clearly identify a soil type based solely on q_c, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.



ZONE	Qt/N	SBT
1	2	Sensitive, fine grained
2	1	Organic materials
3	1	Clay
4	1.5	Silty clay to clay
5	2	Clayey silt to silty clay
6	2.5	Sandy silt to clayey silt
7	3	Silty sand to sandy silt
8	4	Sand to silty sand
9	5	Sand
10	6	Gravely sand to sand
11	1	Very stiff fine grained*
12	2	Sand to clayey sand*

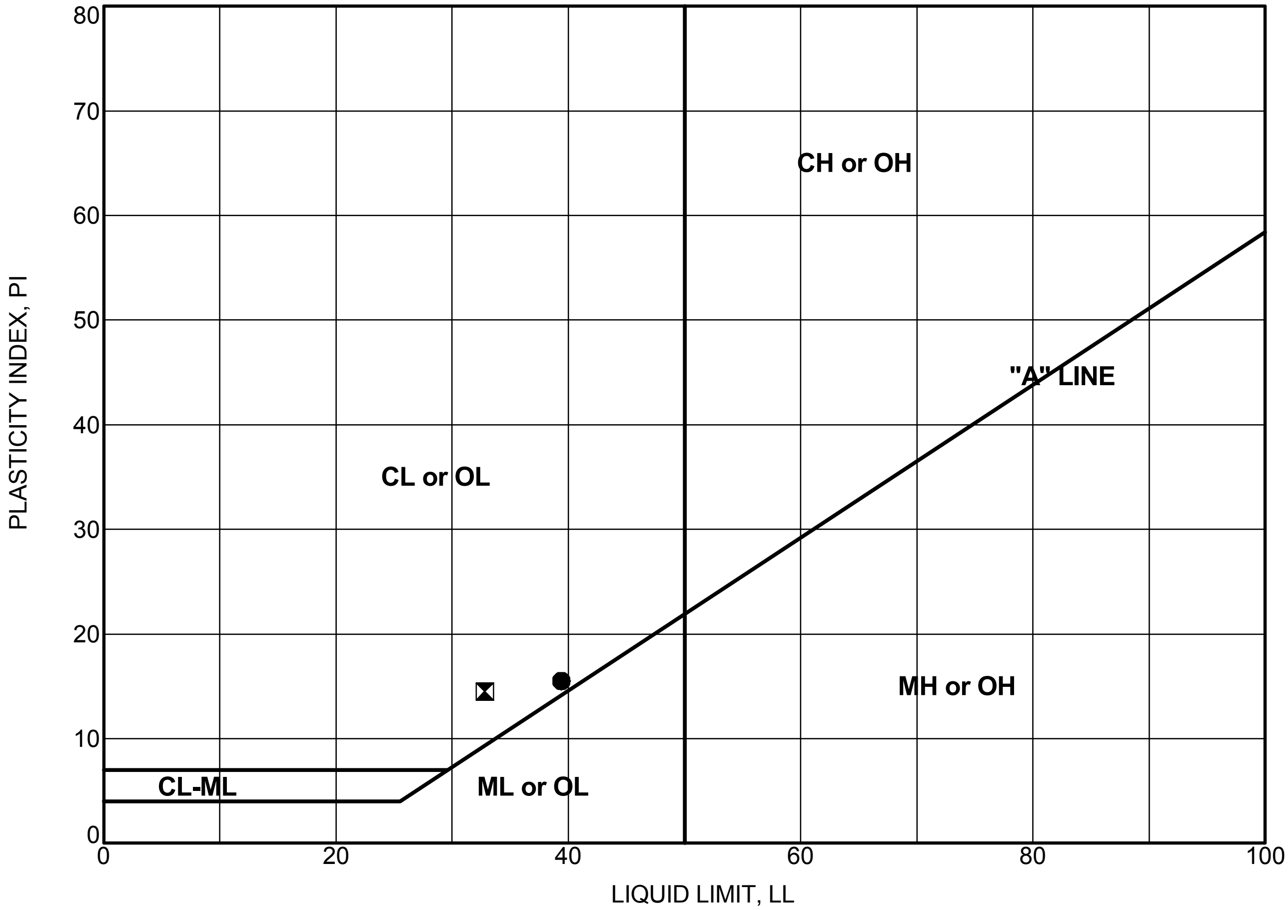
*over consolidated or cemented

Figure SBT

Figure C-1

TABLE C-1
SUMMARY OF LABORATORY TEST RESULTS

Boring or Sample No.	Date Sampled	Depth (feet)	Sieve Analysis Results (Percent Passing)												Atterberg Limits			Unconfined Compressive Strength (psf)	In-Place Conditions	
			%<1.5-in	%<1-in	%<3/4-in	%<1/2-in	%<3/8-in	%<#4	%<#8	%<#16	%<#30	%<#50	%<#100	%<#200	Liquid Limit	Plastic Limit	Plasticity Index		Water Content (%)	Dry Unit Weight (pcf)
NB05	24-Aug-06	1																	9.8	111
NB05		5																	17.4	107
NB05		10																3600	32.6	86
NB05		15													39	24	15	5630	27.3	95
NB05		20													33	18	15	1730	23.7	101
NB05		25	100	100	100	100	100	100	100	99.9	99.8	92.8	48.8	28.6					19.9	108
NB05		35																5090	18.0	108
NB05		40	100	100	100	100	100	99.1	97.7	92.4	79.2	43.4	21.6	17					19.3	
NB05		45	100	100	100	93.6	90.1	70.6	50.2	36	27.8	21	14	11.3					11.8	
NB05		60																	18.1	112
NB05		68.5																	23.0	



Boring Number	Sample Number	Depth (feet)	Test Symbol	Moisture Content (%)	LL	PL	PI	Description
NB05	5	15	●	27	39	24	15	Lean CLAY (CL)
NB05	7	20	⧗	24	33	18	15	Sandy lean CLAY (CL)

Project: SCTA SONOMA 101 HOV LANE SEG B **Project Number: 28645047**	**PLASTICITY CHART**	**Figure C-2**

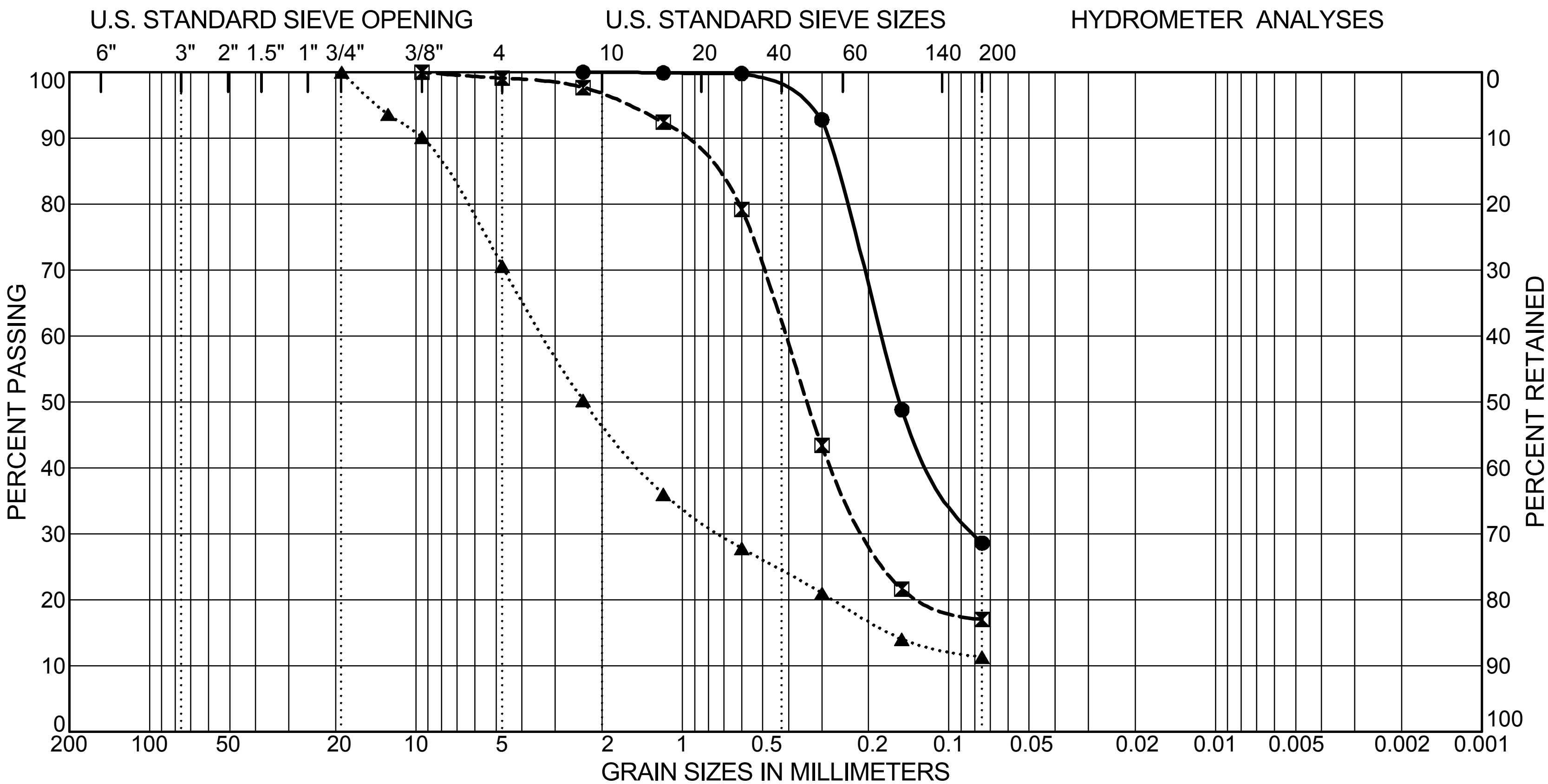
Project : SCTA SONOMA 101 HOV LANE SEG B
Project No. 28645047

9/24/09 USIEVE 9739E.GPJ

GRAIN SIZE DISTRIBUTION CURVES

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT AND CLAY
	coarse	fine	coarse	medium	fine	



Boring Number	Sample Number	Depth (feet)	Symbol	LL	PI	ClassificationC
NB05	8	25	●			Silty SAND (SM)
NB05	11	40	⊠			Silty SAND (SM)
NB05	12	45	▲			Well-graded SAND (SW-SC) with clay and gravel

URS

Figure C-3

04-Son-101
PM 7.1/8.9
EA04-0A1831

DRAFT
Design Hydraulic Study Report
For the Widening of Highway 101 Bridge over Willow Brook Creek (No. 20-0161 L/R) in the City of Petaluma, Sonoma County, California

Prepared for:

Caltrans District Four
111 Grand Avenue
Oakland, California 94612

Sonoma County Transportation Authority
520 Mendocino Avenue, Suite 240
Santa Rosa, California 95401

URS Corporation
55 South Market Street, Suite 1500
San Jose, California 95113

Prepared by:

WRECO
1814 Franklin, Suite 608
Oakland, California 94612

March 2009

Executive Summary

This Bridge Design Hydraulic Study Report presents hydraulic and scour analyses of the Highway 101 (Hwy 101) Bridge over Willow Brook Creek (Bridge Number 20-0161R on the northbound and 20-0161L on the southbound) in the City of Petaluma. This Project is part of the Sonoma County Transportation Authority's (SCTA) Highway 101 HOV Lanes Project Segment B (Project).

The existing Willow Brook Creek Bridge was built in 1956. It spans 70 feet in length and consists of two 42.5 feet wide sections, one for the northbound lane and one for the southbound lane, respectively. For the existing bridge, the soffit elevation at the upstream face of the bridge span for the southbound lane is approximately 43.12 feet.[1]

The proposed modification consists of widening the bridge deck to effectively join the two spans into a single continuous span, modifying the existing pier foundations, and installing new piers. The new deck will include a concrete barrier between the northbound and southbound lanes and a permanent soundwall along the southbound outside shoulder (i.e., the downstream face of the bridge).

Hydraulic Analysis

WRECO analyzed the hydraulic conditions in the study area using HEC-RAS, version 4.0 Beta, which is a hydraulic modeling system developed by the US Army Corps of Engineers, Hydrologic Engineering Center. HEC-RAS is a one-dimensional energy balancing model that uses a standard-step backwater procedure to estimate water surface elevations and flow velocity at user-defined cross sections. Results are based on design flows provided by the user.

The model scenarios assumed Manning's roughness values of:

- 0.03 for the stream channel, which has a mud bottom and brush of low-to-medium height growing in the channel and banks
- 0.05 for all overbank areas.

To determine the downstream boundary conditions, WRECO used the design storm water surface elevations in Petaluma Creek at its confluence with Willow Brook Creek as listed in the Flood Insurance Study (FIS) for the City of Petaluma published by the Federal Emergency Management Agency (FEMA). The flows for the 2% and 1% storm reoccurrence interval were also taken from the FIS. All water surface elevations were taken at the upstream face of each bridge span. The following tables summarize the water surface elevations and flow velocities from the hydraulic analysis.

[1] All elevations in this report refer to the 1988 North American Vertical Datum.

Table ES1. Hydrology and Hydraulics Summary—Willow Brook Creek Bridge Existing Condition

Location	1% Design Storm WS (ft)	1% Design Storm Freeboard (ft)	1% Design Storm Flow Velocity (ft/s)	2% Design Storm WS (ft)	2% Design Storm Freeboard (ft)	2% Design Storm Flow Velocity (ft/s)
Hwy 101–NB Bridge (Upstream Face)	40.30	2.82	4.61	40.15	2.97	4.68
Hwy 101–NB Bridge (Downstream Face)	40.01	3.11	5.85	39.85	3.27	5.98
Hwy 101–SB Bridge (Upstream Face)	39.97	3.15	5.77	39.8	3.32	5.89
Hwy 101–SB Bridge (Downstream Face)	39.96	3.16	7.76	39.8	3.32	5.27
Stony Point Road Bridge (Upstrem Face)	38.56	0.71	5.78	38.21	1.06	6.07
Stony Point Road Bridge (Downstream Face)	38.31	0.96	6.47	38.64	0.63	6.86

Table ES2. Hydrology and Hydraulics Summary—Willow Brook Creek Bridge Proposed Condition

Location	1% Design Storm WS (ft)	1% Design Storm Freeboard (ft)	1% Design Storm Flow Velocity (ft/s)	2% Design Storm WS (ft)	2% Design Storm Freeboard (ft)	2% Design Storm Flow Velocity (ft/s)
Hwy 101–Proposed Bridge (Upstream Face)	40.20	2.92	4.66	40.05	3.07	4.74
Hwy 101–Proposed Bridge (Downstream Face)	39.95	3.17	5.2	39.79	3.33	5.3
Stony Point Road Bridge (Upstrem Face)	38.59	0.68	5.59	38.24	1.03	5.86
Stony Point Road Bridge (Downstream Face)	38.31	0.96	6.47	37.91	1.36	6.86

The hydraulic model results predict that the 100-year flood flow in Willow Brook Creek will pass beneath both the existing and proposed bridges without overtopping. Under the existing condition, the minimum 50-year design storm freeboard for both the northbound and southbound bridges will be greater than 2 feet. The proposed widening will increase that freeboard slightly thus it will still be greater than the required 2 feet. The proposed condition, therefore, meets the Federal Highway Administration's basic hydraulic criteria for bridges.

Scour Analysis

The analysis included estimation of potential scour for all of the abutments and piers as well as long term bed elevation change. The analysis predicted slightly more contraction scour under the proposed condition as compared to both the northbound existing condition and the southbound existing condition. The predictions indicated that potential pier scour would increase in the proposed condition compared to the northbound and southbound existing conditions by approximately six to 9 inches. The local abutment scour decreased compared to the existing northbound condition by six inches at the left abutment, abutment 1, and more than one and one half foot at the right abutment, abutment 4. The increase in predicted scour potential is due primarily to the longer equivalent pier length (25.8 feet, compared to 10.32 feet under the existing condition). The analysis estimated a long term bed elevation change of 0.08 feet per year; this would result in 4 feet of bed elevation degradation over a 50 year lifespan.

Table ES3. Summary of Bridge Scour Analysis: Existing Willow Brook Creek Bridge: Northbound

	Local Scour (ft)	Contraction Scour (ft)	Long Term Scour (ft)	Total Scour (ft)
South Abutment, Abutment 1 (HEC-RAS Left)	0.66	0.39	4.00	5.05
South Piers, Bent 2 (HEC-RAS Pier 1)	5.66	0.39	4.00	10.05
North Piers, Bent 3 (HEC-RAS Pier 2)	5.71	0.39	4.00	10.11
North Abutment, Abutment 4 (HEC-RAS Right)	3.03	0.39	4.00	7.42

Table ES4. Summary of Bridge Scour Analysis: Existing Willow Brook Creek Bridge: Southbound

	Local Scour (ft)	Contraction Scour (ft)	Long Term Scour (ft)	Total Scour (ft)
South Abutment, Abutment 1 (HEC-RAS Left)	-	0.38	4.00	4.38
South Piers, Bent 2 (HEC-RAS Pier 1)	5.58	0.38	4.00	9.95
North Piers, Bent 3 (HEC-RAS Pier 2)	5.75	0.38	4.00	10.13
North Abutment, Abutment 4 (HEC-RAS Right)	-	0.38	4.00	4.38

Table ES5. Summary of Bridge Scour Analysis: Proposed Willow Brook Creek Bridge Widening

	Local Scour (ft)	Contraction Scour (ft)	Long Term Scour (ft)	Total Scour (ft)
South Abutment, Abutment 1 (HEC-RAS Left)	-	0.46	4.00	4.46
South Piers, Bent 2 (HEC-RAS Pier 1)	6.23	0.46	4.00	10.69
North Piers, Bent 3 (HEC-RAS Pier 2)	6.29	0.46	4.00	10.75
North Abutment, Abutment 4 (HEC-RAS Right)	1.38	0.46	4.00	5.84

Long term bed elevation change, contraction scour, and local scour—including both pier and abutment scour—must be considered when setting pier foundation depths in and near the main channel of Willow Brook Creek. In order to avoid structural damage, and/or undermining, the additional pier foundations for the proposed bridge widening should be constructed below the estimated total scour depth or to the bedrock layer. Furthermore, any plans for the modification or replacement of existing piers and pier foundations should provide scour protection by placing scour countermeasures such as appropriately sized rip-rap.

APPENDIX E Liquefaction Analysis/Seismic Compaction Calculations

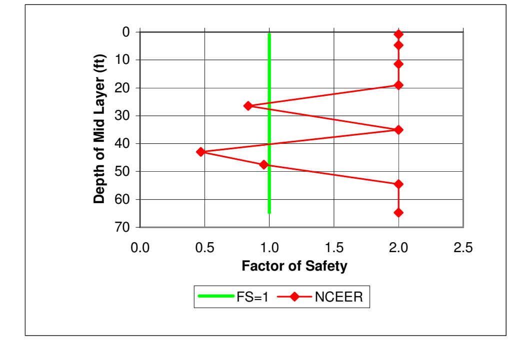
LIQUEFACTION ANALYSIS

Project	Highway 101 HOV Project
Project No.	28649739
Boring No.	NB05
Site	Willow Brook

PGA =	0.5	
M_w =	7.25	
GWT during EQ	15	ft
Sampler:	Symbol	Corr. Factor
SPT	S	1
Mod.Cal	M	0.8
Cal	C	0.65
Mag. Weighting Factor($CSR_M/CSR_{M=7.5}$)		0.91

Note: Cells in Red must be input

Hammer type :	Type		C_E	Code	C_E
	Donut		0.75	1	3
	Safety		1.00	2	
	Auto		1.16	3	
Borehole Diameter:	(inch)	(mm)	C_B		C_B
	2.5-4.5	65-115	1	1	1
	6	150	1.05	2	
	8	200	1.15	3	
Rod Length:	(feet)	(m)	C_R		
	Default			999	
Sampling Method:	Type		Cs		
	Stand		1	(Mod. Cal)	
	No liners		1.2	(SPT)	



Note: F.S. greater than 2 is set to be equal to 2

Layer Number	Soil Type (USCS)	γ_m pcf	At Bottom of Layer: Depth ft	At Bottom of Layer: Depth m	At Center of Layer: Depth ft	At Center of Layer: Depth m	EQ σ_o psf	EQ u psf	EQ σ'_o psf	r_d	EQ CSR	N raw*	Sampler	N SPT	Fines Content %
1	Fill	125	1.5	0.46	0.75	0.23	94	0	94	1.000	0.325			0	
2	CL	120	8.0	2.44	4.8	1.45	578	0	578	0.991	0.322			0.0	
3	CH	115	15.0	4.57	11.5	3.51	1,370	0	1,370	0.976	0.317			0	
4	CL	120	23.0	7.01	19.0	5.79	2,253	250	2,003	0.959	0.351			0	
5	SM	128	30.0	9.14	26.5	8.08	3,181	718	2,463	0.936	0.393	23	M	18.4	29
6	CL	125	40.0	12.19	35.0	10.67	4,254	1,248	3,006	0.891	0.410			0	
7	SM	128	46.0	14.02	43.0	13.11	5,263	1,747	3,515	0.823	0.401	19	M	15.2	17
8	SW-SC	128	49.0	14.94	47.5	14.48	5,839	2,028	3,811	0.778	0.388	26	S	26	11
9	SM	130	60.0	18.29	54.5	16.61	6,746	2,465	4,281	0.708	0.363	50	S	50	12
10	SM	130	69.5	21.18	64.8	19.74	8,078	3,104	4,974	0.624	0.329	100	S	100	12
11															
12															
13															
14															
15															
16															
17															
18															
19															
20															

Layer Number	Blow Count Corrections: $N_{60\text{-}cs}$ FC< 5%	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60\text{-}cs}$	$CRR^{**}_{7.5,1tsf}$	Magnitude Scaling Factor NCEER K_m Lower	Upper	Middle	Overburden Correction K_σ	Above GWT?	F.S. NCEER	Liquefy ? NCEER	F.S=1.0
1	0.0	2.00	1.16	1.00	0.75	1.00	0		1.09	1.12	1.10	1.000	Y	2.00	N	1
2	0.0	1.91	1.16	1.00	0.75	1.00	0		1.09	1.12	1.10	1.000	Y	2.00	N	1
3	0.0	1.24	1.16	1.00	0.75	1.00	0		1.09	1.12	1.10	1.000	Y	2.00	N	1
4	0.0	1.03	1.16	1.00	0.85	1.00	0		1.09	1.12	1.10	1.000	N	2.00	N	1
5	25.7	0.93	1.16	1.00	0.95	1.00	26	0.305	1.09	1.12	1.10	0.977	N	0.84	Liquef	1
6	0.0	0.84	1.16	1.00	1.00	1.00	0		1.09	1.12	1.10	0.945	N	2.00	N	1
7	19.1	0.78	1.16	1.00	1.00	1.00	17	0.186	1.09	1.12	1.10	0.918	N	0.47	Liquef	1
8	27.9	0.75	1.16	1.00	1.00	1.20	29	0.372	1.09	1.12	1.10	0.903	N	0.96	Liquef	1
9	53.1	0.70	1.16	1.00	1.00	1.20	52	0.459	1.09	1.12	1.10	0.879	N	2.00	N	1
10	104.7	0.65	1.16	1.00	1.00	1.20	95	0.459	1.09	1.12	1.10	0.848	N	2.00	N	1
11																
12																
13																
14																
15																
16																
17																
18																
19																
20																

Layer Number	Depth to Bot. Layer	Thickness ft	σ_o psf	u psf	σ'_o psf	r_d	$(N_1)_{60\text{-}cs}$	Liquefy ? NCEER	τ_{cyc} psf	$CSR_{M=7.5}$ τ_{cyc}/σ'_o	CSR_M τ_{cyc}/σ'_o	Volumetric Strain*** %	Settl. in
1	1.5	1.5	94	0	94	1.000	0.0	N	30.5	0.325	0.29	0.0	0.00
2	8.0	6.5	578	0	578	0.991	0.0	N	186.0	0.322	0.29	0.0	0.00
3	15.0	7.0	1,370	0	1,370	0.976	0.0	N	434.6	0.317	0.29	0.0	0.00
4	23.0	8.0	2,253	250	2,003	0.959	0.0	N	702.4	0.351	0.32	0.0	0.00
5	30.0	7.0	3,181	718	2,463	0.936	26.3	Liquef	967.8	0.393	0.36	1.1	0.92
6	40.0	10.0	4,254	1,248	3,006	0.891	0.0	N	1231.2	0.410	0.37	0.0	0.00
7	46.0	6.0	5,263	1,747	3,515	0.823	17.2	Liquef	1408.1	0.401	0.36	1.8	1.30
8	49.0	3.0	5,839	2,028	3,811	0.778	28.9	Liquef	1476.8	0.388	0.35	0.2	0.07
9	60.0	11.0	6,746	2,465	4,281	0.708	52.0	N	1552.3	0.363	0.33	0.0	0.00
10	69.5	9.5	8,078	3,104	4,974	0.624	95.1	N	1637.7	0.329	0.30	0.0	0.00
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													

Total Settlement (inch) = 2.29

* Note: Blowcounts above 100 blows/foot (bpf) met refusal criteria. The blowcount value of 100 bpf is assumed in the calculation for these cases.

** The equation used to obtain $CRR_{7.5}$ is not valid for corrected $(N_1)_{60}$ higher than 30. According to NCEER 1997, soil layers with corrected $(N_1)_{60}$ higher than 30 blows per foot have a low potential for liquefaction.

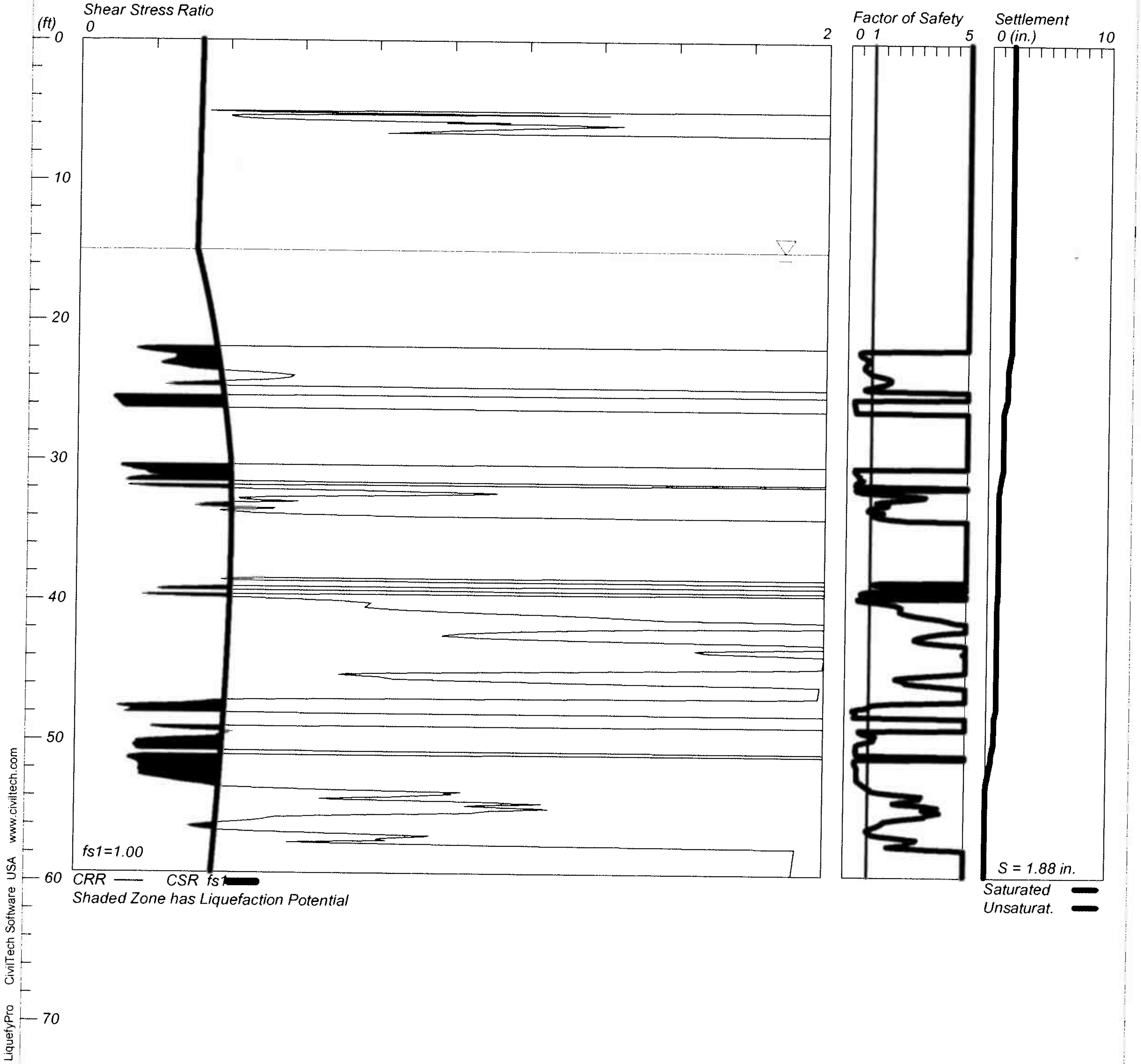
*** These values were taken from a chart for estimation of volumetric strain in saturated sands from cyclic stress ration and standard penetration resistance (after Tokimatsu and Seed, 1987).
See figure 9.53 from "Geotechnical Earthquake Engineering" by S. Kramer.

LIQUEFACTION ANALYSIS

Sonoma 101 HOV Project

Hole No.=CT06 Water Depth=15 ft

Magnitude=7.0
Acceleration=0.5g



URS

HIGHWAY 101 HOV LANE PROJECT

IN PETALUMA, COTATI & ROHNERT PARK
FROM 0.8 KM SOUTH OF OLD REDWOOD HIGHWAY OVERCROSSING TO ROHNERT PARK EXPRESSWAY OVERCROSSING

Willow Brook Creek Bridge (Widening)
FOUNDATION INVESTIGATION REPORT

Prepared for:

URS Corporation
1333 Broadway, Suite 800
Oakland, CA 94612

Prepared By:

V&A
Lake Merritt Plaza
1999 Harrison Street, Suite 975
Oakland, CA 94612

November 2006

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EXECUTIVE SUMMARY

V&A Consulting Engineers (V&A) was retained by URS Corporation to perform a Soil Corrosivity Investigation for the proposed Highway 101 HOV Lane Project 0.8 kilometers South of the Old Redwood Highway in Petaluma, CA, to the Rohnert Park Expressway Overcrossing in Rohnert Park, CA. The objective of this investigation is to measure various soil parameters and evaluate the results to determine the corrosivity of the soil to the proposed Willow Brook Creek Bridge Widening for the Highway 101 HOV project. Corrosivity was determined at the bridge site to depths ranging from 0 to 6 meters below grade.

The California Department of Transportation, Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch has published "Corrosion Guidelines" (Guidelines) to define corrosive soil. The Guidelines consider soil to be corrosive to structural elements (steel reinforced concrete) if one or more of the following conditions exist for water or soil samples:

> ***"Chloride concentration is 500 ppm (mg/kg) or greater, sulfate concentration is 2,000 mg/kg or greater, or the pH is 5.5 or less."***

A wide variety of soluble salts is typically found in soils. Two soils having the same resistivity may have significantly different corrosion characteristics, depending on the specific ions available. The major constituents that accelerate corrosion are chlorides, sulfates and the acidity (pH) of the soil. Chloride ions tend to break down otherwise protective surface deposits, and can result in corrosion of reinforcing steel in concrete structures. Sulfates in soil can be highly aggressive to portland cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. High concentrations of bicarbonates tend to decrease soil resistivities. Although bicarbonates are not aggressive to buried concrete, bicarbonates will lower the soil resistivity which can promote corrosion activity.

Acidity, as indicated by the pH of the soil, is another important factor of soil. The lower the pH (the more acidic the environment), the higher the corrosivity will be with respect to buried metallic and concrete structures. As pH increases above 7 (the neutral value), conditions become increasingly more alkaline and passive to buried structures.

This report provides recommendations for corrosion control for the various structural foundation materials under consideration. This foundation investigation is in accordance with Caltrans Corrosion Guidelines dated September 2003 (Guidelines). The materials considered as part of this investigation include buried reinforced concrete, cast-in-place concrete piles, prestressed concrete piles, and steel pipe piles.

Evaluation of the in-situ soil environment is reported in terms of potential damage to the structures due to corrosion. Soil resistivity measurements were conducted in the field during the initial stages of the work. In addition, soil samples collected by URS during the geotechnical investigation were forwarded to Cooper Testing Laboratory in Palo Alto, CA, for testing. The soil borings were analyzed for minimum resistivity, pH, water soluble chloride ion concentration, and water soluble sulfate ion concentration.

Based on the results of the corrosion analyses, the site is considered *non-corrosive*.

The pH of the soil samples analyzed ranged from 7.6 to 7.9, and the site is considered *non-corrosive*.

The water soluble chloride ion concentration ranged from 6 milligrams per kilogram (mg/kg) to 9 mg/kg, and the water soluble sulfate ion concentration of the soil samples was less than the minimum detection level of 5 mg/kg. The chloride ion and sulfate ion concentrations at the site are considered *non-corrosive*.

CONCLUSIONS

- Soil samples were obtained for corrosion analysis at Boring Location NB05. Based on the results of the corrosion analyses, the site is considered *non-corrosive*.
- The proposed structure is not within 300 meters (1,000 feet) of salt or brackish water.
- The chloride ion concentration for Boring NB05, Samples 2 and 3, ranged from 6 milligrams per kilogram (mg/kg) to 9 mg/kg and is considered *non-corrosive*.
- The sulfate ion concentration for Boring NB05, Samples 2 and 3, was less than the minimum detection level of 5 mg/kg and is considered *non-corrosive*.
- The pH of Boring NB05, Samples 2 and 3, ranged from 7.6 to 7.9 and is considered *non-corrosive*.

RECOMMENDATIONS

Based on the test data and review of the project requirements, specific recommendations for each structural foundation material alternative are listed below:

Buried Reinforced Concrete Structures, Prestressed Concrete Piles, and Cast-in-Place Concrete Piles

Buried concrete structures should be constructed of durable concrete as described in Section 8.22 of the Caltrans Bridge Design Specifications and ACI Standards 201.2R and 222R. These recommendations include, but are not limited to, the following:

- The water/cement ratio should not exceed 0.50.
- A concrete cover of a minimum of 5.1 centimeters should be applied over all steel reinforcement.
- Salt-free sand and potable water should be used.
- Type II modified cement should be used.
- The concrete should be allowed to cure according to the manufacturer's recommendations.

Steel Pipe Piles

Based on the results of the corrosion analyses, the site is considered *non-corrosive* to steel pipe piles. Fill materials should be tested for corrosivity, and if they are found to be corrosive, then the recommendations in the Guidelines for steel pipe piles in corrosive soil should be followed.

A conservative approach for the design of pipe piles would be to assume that corrosion will occur at a rate of 0.025 millimeters per year between grade and a depth 1 meter below the water table, in accordance with the Guidelines. The renewable supply of oxygen typically available in this region of soil sustains corrosion and the corrosion allowance in accordance with the Guidelines is recommended. It is important to note that structural considerations for pipe piles may require increases in metal wall thickness to withstand both installation and structural loadings, as they have not been considered in this analysis.

TEST METHODS

In attempting to predict corrosion problems associated with a particular type of structure prior to installation, it is necessary to investigate the soil conditions the structure will encounter. Since corrosion is an electrochemical process accompanied by current flow, the electrochemical characteristics of a soil are of primary importance when evaluating corrosivity. Test methods utilized during this investigation reflect the most practical methods of evaluating corrosivity.

In-Situ Soil Resistivity

In-situ resistivity of the soil was measured at two locations at the Willow Brook Creek Bridge site. Figure 1 shows the project vicinity map. Figure 2 shows the in-situ soil resistivity and soil boring locations.

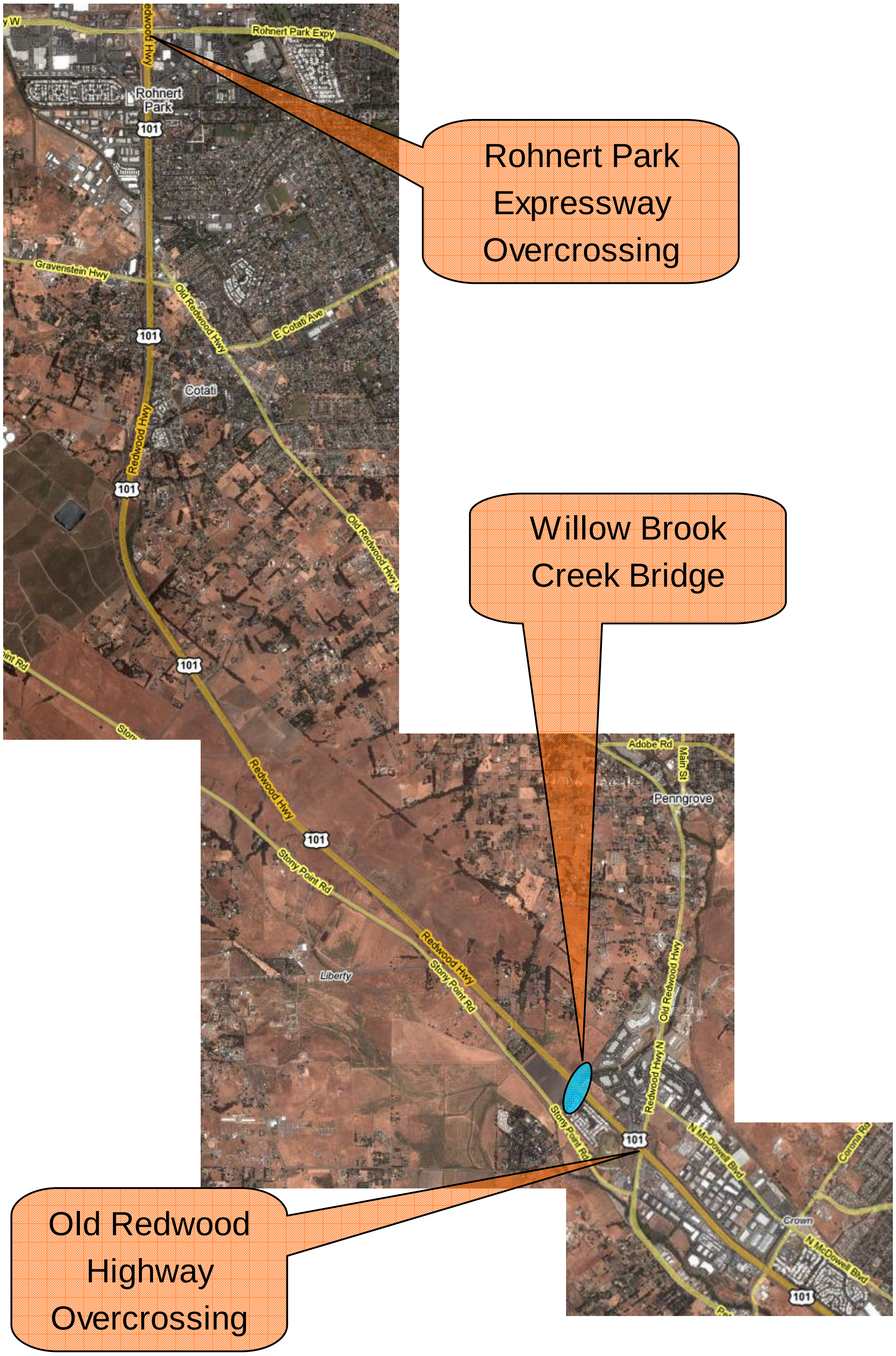


Figure 1 - Vicinity and Project Site Map

Figure 2 - Soil Resistivity Testing and Soil Sample Locations[1]

In-situ soil resistivity was measured using the Wenner 4-Electrode Method, with a Model 400 Soil Resistance Meter, manufactured by Nilsson Electrical Laboratory, Inc. The Wenner Method involves the use of four metal probes or electrodes, driven into the ground along a straight line, equidistant from each other, as shown in Figure 3.

[1] Maps courtesy of the USGS and the TerraServer Team

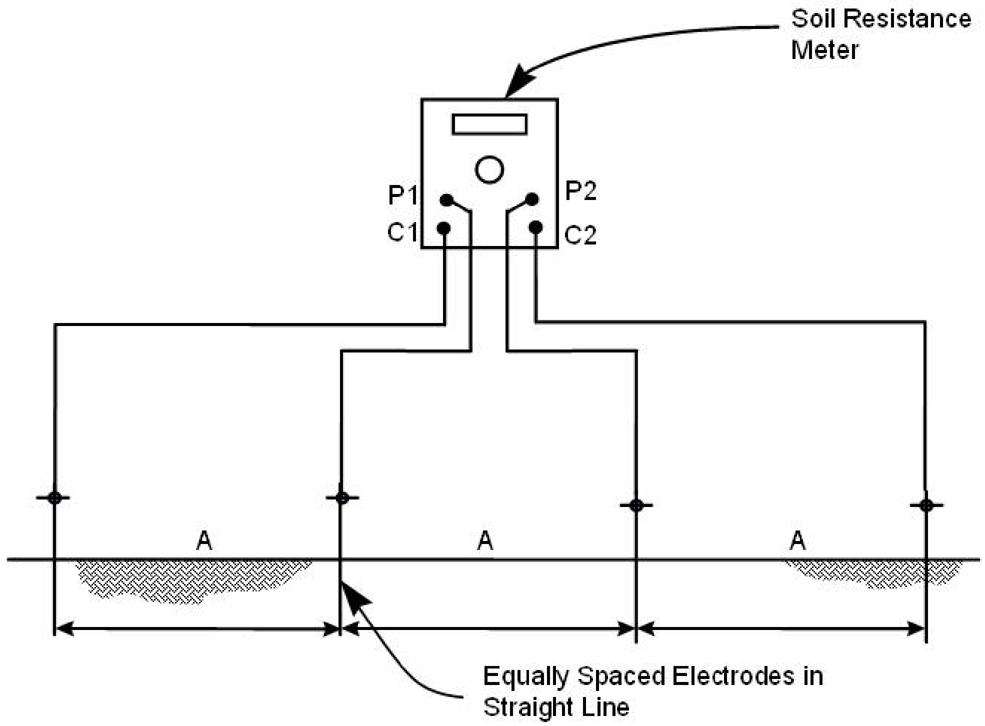


Figure 3 - Soil Resistivity Measurement Using the Wenner 4-Electrode Method

An alternating current from the Soil Resistance Meter causes a current to flow through the soil between the outside electrodes, C1 and C2. Due to the resistance of the soil, the current creates a voltage gradient, which is proportional to the average resistance of the soil mass to a depth equal to the distance between electrodes. The voltage drop is then measured across pins, P1 and P2. Resistivity of the soil is then computed from the instrument reading according to the following formula:

$$\rho = 2 \cdot \pi \cdot A \cdot R$$

Where:	ρ	=	soil resistivity (ohm-cm)
	A	=	distance between electrodes (cm)
	R	=	soil resistance, instrument reading (ohms)
	π	=	3.14 (approx.)

Soil resistivity measurements were conducted with electrodes spaced at 1.5, 3, 4.5, and 6 meters. The resistivity values obtained represent the average resistivity of the soil to a depth equal to the electrode spacing. An additional method of calculating the soil resistivity using the data from the 4-Electrode Method is the Barnes-Layer Resistivity Calculation. The Barnes Layer Calculation is used to determine the resistivity of the soil for each soil layer. While the 4-Electrode Method at 3-meter electrode spacing will consider all 3 meters of soil below the surface, the Barnes-Layer method will consider the resistivity of only the layer of soil between 1.5 and 3 meters below the surface. This method assumes the soil layers are a uniform thickness and parallel to the surface.

$$\rho_{b-a} = KR_{b-a}$$

and

$$\frac{1}{R_{b-a}} = \frac{1}{R_a} - \frac{1}{R_b}$$

Where:

ρ_{b-a}	=	Soil resistivity of layer depth b-a (ohm-cm)
a	=	Soil depth to top of layer (cm)
b	=	Soil depth to bottom of layer (cm)
R_a	=	Soil resistance read at depth a (ohms)
R_b	=	Soil resistance read at depth b (ohms)
R_{b-a}	=	Resistance of soil layer from a to b (cm)
K	=	Layer constant (cm)
	=	$2\ \pi$ (b-a)

Soil resistance is measured at electrode spacings of 1.5, 3, 4.5, and 6 meters. Barnes Layer resistivity is calculated for layers 0 to 1.5, 1.5 to 3, 3 to 4.5, and 4.5 to 6 meters below grade. The results of both methods are listed on Table 1.

Laboratory Analysis of Soil

To supplement the resistivity data obtained during field testing, soil samples from the Willow Brook Creek Bridge site were obtained from URS. The soil samples were forwarded to Cooper Testing Laboratory in Palo Alto, CA, for testing. The samples were tested for pH, minimum resistivity, and water soluble chloride and water soluble sulfate ion concentrations.

TEST RESULTS

Data obtained during this investigation has been summarized in tabular form for analysis and presentation. Table 1 lists the results of the in-situ soil resistivity measurements conducted at the site. Table 2 lists the minimum soil resistivity for each sample collected as well as the chemical analysis of the sample. The locations of the soil samples and the in-situ resistivity test locations are shown on Figure 2.

In-Situ Soil Resistivity

Since corrosion is a natural electrochemical process accompanied by the flow of electrical current, it is important to understand how easily current will travel through the soil surrounding the steel or steel reinforced concrete structure. Resistivity is a measure of the ability of a soil to conduct an electric current. Soil resistivity is primarily dependent on the chemical and moisture content of the soil. As the concentrations of chemical constituents increases, the soil resistivity will decrease. Additional moisture generally decreases the soil resistivity up to the point where the maximum solubility for the dissolved ionic chemicals is achieved. Beyond the maximum solubility, an increase in moisture generally increases the resistivity as the chemicals become more and more diluted. Since corrosion rate depends on current flow through the soil to and from the metal, soil corrosivity normally increases as soil resistivity decreases.

Table 1 - In-Situ Soil Resistivity Data

Site	Approx. Sta.	Depth (meters)	Resistivity (ohm-cm)	Layer Resistivity (ohm-cm)	Layer (meters)
1	20+90 RT	1.5	40,217	40,217	0 – 1.5
		3.0	76,605	804,348	1.5 – 3.0
		4.5	215,450	82,076	3.0 – 4.5
		6.0	1,417,185	90,075	4.5 – 6.0
2	20+90 LT	1.5	1,341	1,341	0 – 1.5
		3.0	1,513	1,736	1.5 – 3.0
		4.5	2,585	6,189	3.0 – 4.5
		6.0	575	172	4.5 – 6.0

Laboratory Analysis of Soil Samples

Soil samples were obtained for corrosion analyses and the results are shown in Table 2.

Table 2 - Soil Resistivity and Chemical Data

Item No.	Boring No.	Station	Sample No.	Depth (meters)	Soil Resistivity (ohm-cm) Minimum	Chemical Data		
						pH	Sulfate (mg/kg)	Chloride (mg/kg)
1	NB05	20+75	2	0.5	1,041	7.9	<5	9
2	NB05	20+75	3	1.5	581	7.6	<5	6

Based on the results of the corrosion analyses, the site is considered *non-corrosive* to buried reinforced concrete, prestressed concrete piles, and steel pipe piles.

Appendix A – Soil Laboratory Chemical Analysis Results

Corrosivity Test Summary

CTL # 632-002 **Date:** 11/3/2006 **Tested By:** PJ **Checked:** PJ

Client: V & A Engineering **Project:** Caltrans, Highway 101 HOV Lane **Proj. No:** 06-088

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	ASTM D2216	
NB-05, 2	-	-	-	1,041	-	9	<5	<0.0005	7.9	-	19.4	Olive Brown Clayey SAND
NB-05,3	-	-	-	581	-	6	<5	<0.0005	7.6	-	33.4	Dark Gray CLAY
NB-09, 3	-	-	-	1,425	-	14	<5	<0.0005	7.6	-	18.9	Brown Clayey SAND w/ Gravel
NB-10, 3	-	-	-	2,534	-	5	<5	<0.0005	7.3	-	21.4	Brown Clayey SAND w/ Gravel (silty)
NB-13, 3	-	-	-	1,288	-	10	<5	<0.0005	6.7	-	24.1	Yellowish Brown Clayey SAND w/ Gravel
NB-20, 3	-	-	-	1,220	-	8	<5	<0.0005	7.6	-	20.0	Yellowish Brown Clayey SAND w/ Gravel
NB-21, 3	-	-	-	1,328	-	9	<5	<0.0005	7.2	-	31.5	Olive Brown Clayey SAND, trace Gravel
NB-32, 3	-	-	-	4,127	-	7	<5	<0.0005	6.8	-	7.8	Brown SAND w/ Silt & Gravel
NB-37, 3	-	-	-	925	-	51	164	0.0164	7.4	-	18.3	Yellowish Brown Sandy CLAY
NB-41, 3	-	-	-	1,472	-	<2	<5	<0.0005	7.3	-	17.0	Brown Clayey SAND
NB-43, 3	-	-	-	1,343	-	11	<5	<0.0005	8.5	-	32.7	Brown Sandy CLAY
NB-48, 3	-	-	-	926	-	<2	8	0.0008	7.5	-	21.2	Gray Sandy CLAY
NB-54, 3	-	-	-	1,259	-	<2	<5	<0.0005	7.1	-	22.8	Grayish Brown CLAY w/ Sand
NB-56, 2	-	-	-	658	-	12	<5	<0.0005	7.7	-	30.5	Dark Brown CLAY w/ Sand
NB-56, 3	-	-	-	678	-	3	<5	<0.0005	7.9	-	23.2	Dark Brown CLAY w/ Sand
NB-62, 2	-	-	-	629	-	26	108	0.0108	7.8	-	29.0	Dark Brown CLAY w/ Sand
NB-62, 3	-	-	-	550	-	27	<5	<0.0005	8.2	-	30.1	Dark Brown CLAY w/ Sand
NB-74, 3	-	-	-	1,729	-	42	5	0.0005	7.5	-	36.2	Reddish Brown GRAVEL w/ Silt & Sand
NB-79, 2	-	-	-	2,139	-	8	<5	<0.0005	8.3	-	29.0	Reddish Brown Clayey SAND

SONOMA 101 HOV Project

9/24/2009

Pile Design Spreadsheet

Abutments 1 & 4

NAVFAC Design Method For a Single Pile

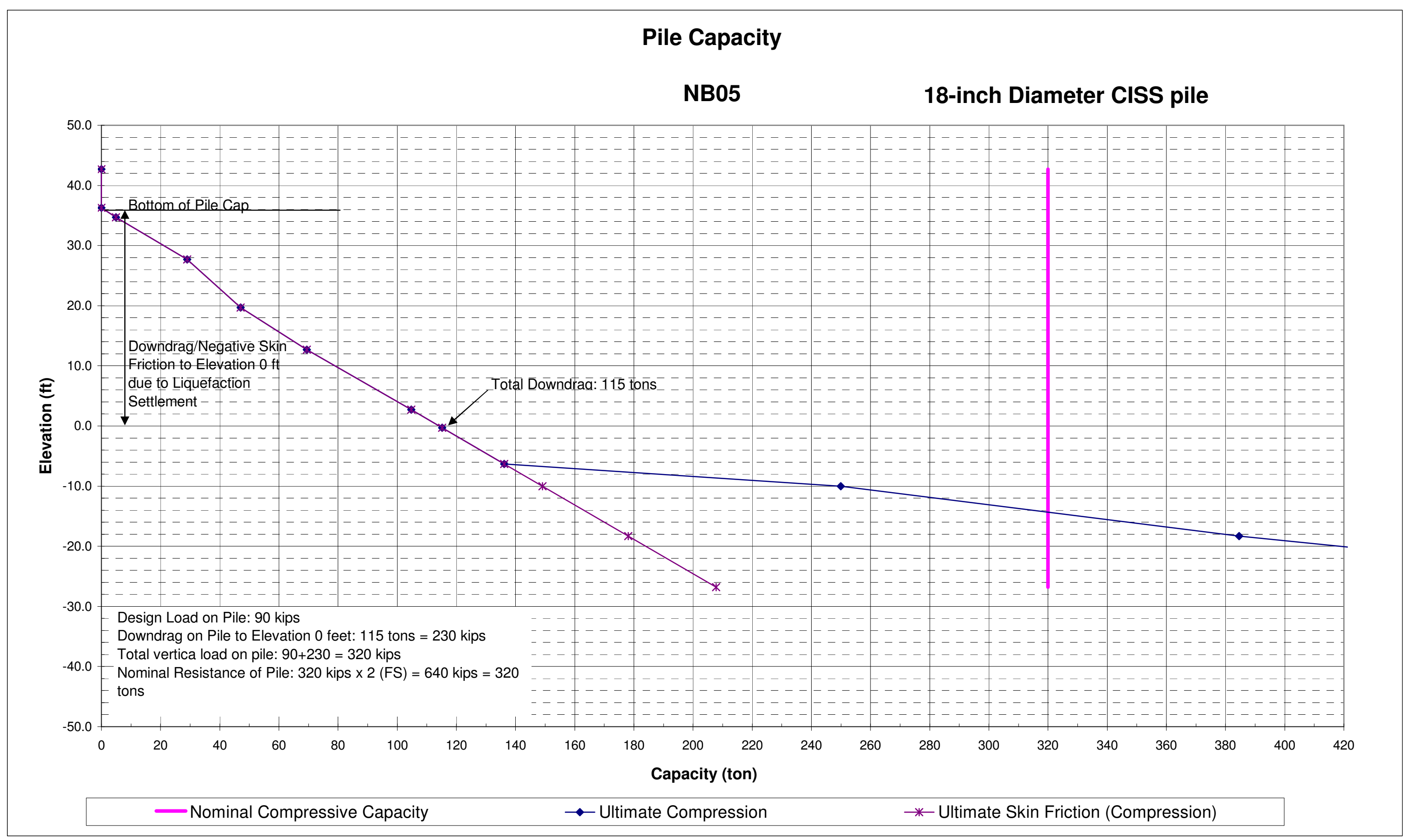
Project Name:	Sonoma 101 HOV	Nominal axial compress. Capacity (ult.)	320	ton	(including downdrag)					
Project Number:	28645047	Nominal axial compress. Capacity (ult.)	2848	kN						
Bridge Name:	Willow Brook Creek	Pile Diameter	18	inch	Pile Shape	Circular	(Square, Circular or Octogonal)			
Boring Surface El. (m)	13.0	Pile Diameter	457	mm	Pile Surface Area	1.44	sq. m/m	Pile Tip Area	0.16	sq. m
Boring Surface El. (ft)	42.7	Pile Diameter	1.50	ft	Pile Surface Area	4.71	sq. ft/ft	Pile Tip Area	1.77	sq. ft.
Design GWS El. (ft)	27.5	Crit depth ratio	20	-						
Unit Weight of Water (pcf)	62.4	Crit depth	30.0	ft						
Depth with no strength		Crit. Depth. Elev	12.7	ft						
Convert. Factor	3.281	Crit. Depth. Elev	3.9	m						
		Crit. Eff. Vert. Str.	2716	psf						
Bottom of Pile Cap El. (m)	11.1	Skin Resistance (Friction) of Pile in Granular Material:								
Bottom of Pile Cap El. (ft)	36.3	Type (Steel=S,Concrete=C,Timber=T)	S							
		Earth Press. Coefficient * (Comp.)	1.5							
		Earth Press. Coefficient * (Tens)	1.0							
		Minimum Adhesion	500	psf						

Note: Soils encountered above Elevation 0 feet are anticipated to have downdrag forces on the pile due to liquefaction settlement.

Layer No.	Layer Elevation Top		Bot.		Lateral Resists?	Cohesion		Adhesion Ratio	Adhesion	Friction Angle		Layer Thickness
						c	c	c_a/c	c_a	ϕ	δ	
(-)	(m)	(feet)	(m)	(feet)		(kPa)	(psf)		(psf)	(deg)	(deg)	(feet)
			13.01	42.7								
1	13.01	42.7	11.06	36.3	n	72	1500	0.87	0	0	0	6.4
2	11.06	36.3	10.58	34.7	y	72	1500	0.87	1305	0	0	1.6
3	10.58	34.7	8.44	27.7	y	86	1800	0.81	1458	0	0	7.0
4	8.44	27.7	6	19.7	y	48	1000	0.96	960	0	0	8.0
5	6	19.7	3.87	12.7	y	0	0	0.00	0	32	20	7.0
6	3.87	12.7	0.82	2.7	y	120	2500	0.60	1500	0	0	10.0
7	0.82	2.7	-0.09	-0.3	y	0	0	0.00	0	32	20	3.0
8	-0.09	-0.3	-1.92	-6.3	y	0	0	0.00	0	32	20	6.0
9	-1.92	-6.3	-3.05	-10.0	y	0	0	0.00	0	34	20	3.7
10	-3.05	-10.0	-5.58	-18.3	y	0	0	0.00	0	38	20	8.3
11	-5.58	-18.3	-8.17	-26.8	y	0	0	0.00	0	40	20	8.5
Bottom	-8.17	-26.8										

Layer No.	Internal Depth to Layer Top	Effective Vert. Stress (Bot. Lay.)	Design Effective Vert. Stress (Bot. Lay.)	delta Lat. Res. Cohesive	Friction (comp)	Friction (tensile)	Nq (granular)	z/B	Nc (cohesive)	Soil Type G= gran. C=cohes.	Bearing Capacity (B.C.)	Include B.C.?	Elevation to midlayer
(-)	(feet)	(psf)	(psf)	(ton)	(ton)	(ton)	(-)	(-)	(-)		(ton)		(ft)
		0	0					0.0	6.25	c	0	n	42.7
1	0.0	720	720	0.0	0.0	0.0	0	2.1	8.45	c	11	n	39.5
2	6.4	960	960	4.9	0.0	0.0	0	4.8	9.00	c	12	n	35.5
3	8.0	1730	1730	24.0	0.0	0.0	0	7.7	9.00	c	14	n	31.2
4	15.0	2243	2243	18.1	0.0	0.0	0	12.7	9.00	c	8	n	23.7
5	23.0	2716	2716	0.0	22.3	14.9	29	17.7	9.00	g	70	n	16.2
6	30.0	3362	2716	35.3	0.0	0.0	0	23.3	9.00	c	20	n	7.7
7	40.0	3565	2716	0.0	10.5	7.0	29	27.7	9.00	g	70	n	1.2
8	43.0	3971	2716	0.0	21.0	14.0	29	30.7	9.00	g	70	n	-3.3
9	49.0	4189	2716	0.0	12.9	8.6	42	33.9	9.00	g	101	y	-8.2
10	52.7	4842	2716	0.0	29.0	19.3	86	37.9	9.00	g	206	y	-14.2
11	61.0	5001	2716	0.0	29.7	19.8	145	43.5	9.00	g	348	y	-22.6

Layer No.	Compression: Cumulative Axial Res. (skin)		Cumulative Axial Res. (Tot)		Nominal Axial Capacity		Tension: Cumulative Axial Res. (Skin)		Nominal Axial Capacity	
(-)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)
	0	0	0	0	320	2848	0	0	160	1424
1	0.0	0	0.0	0	320	2848	0.0	0	160	1424
2	4.9	44	4.9	44	320	2848	4.9	44	160	1424
3	29.0	258	29.0	258	320	2848	29.0	258	160	1424
4	47.1	419	47.1	419	320	2848	47.1	419	160	1424
5	69.4	618	69.4	618	320	2848	61.9	551	160	1424
6	104.7	932	104.7	932	320	2848	97.3	866	160	1424
7	115.2	1025	115.2	1025	320	2848	104.3	928	160	1424
8	136.2	1212	136.2	1212	320	2848	118.3	1052	160	1424
9	149.1	1327	249.9	2224	320	2848	126.9	1129	160	1424
10	178.1	1585	384.5	3422	320	2848	146.2	1301	160	1424
11	207.8	1850	555.9	4947	320	2848	166.0	1477	160	1424



18-inch Diameter CISS Pile with 45 ton Allowable Bearing Capacity

SONOMA 101 HOV Project

Pile Design Spreadsheet

Bents 2 and 3

NAVFAC Design Method For a Single Pile

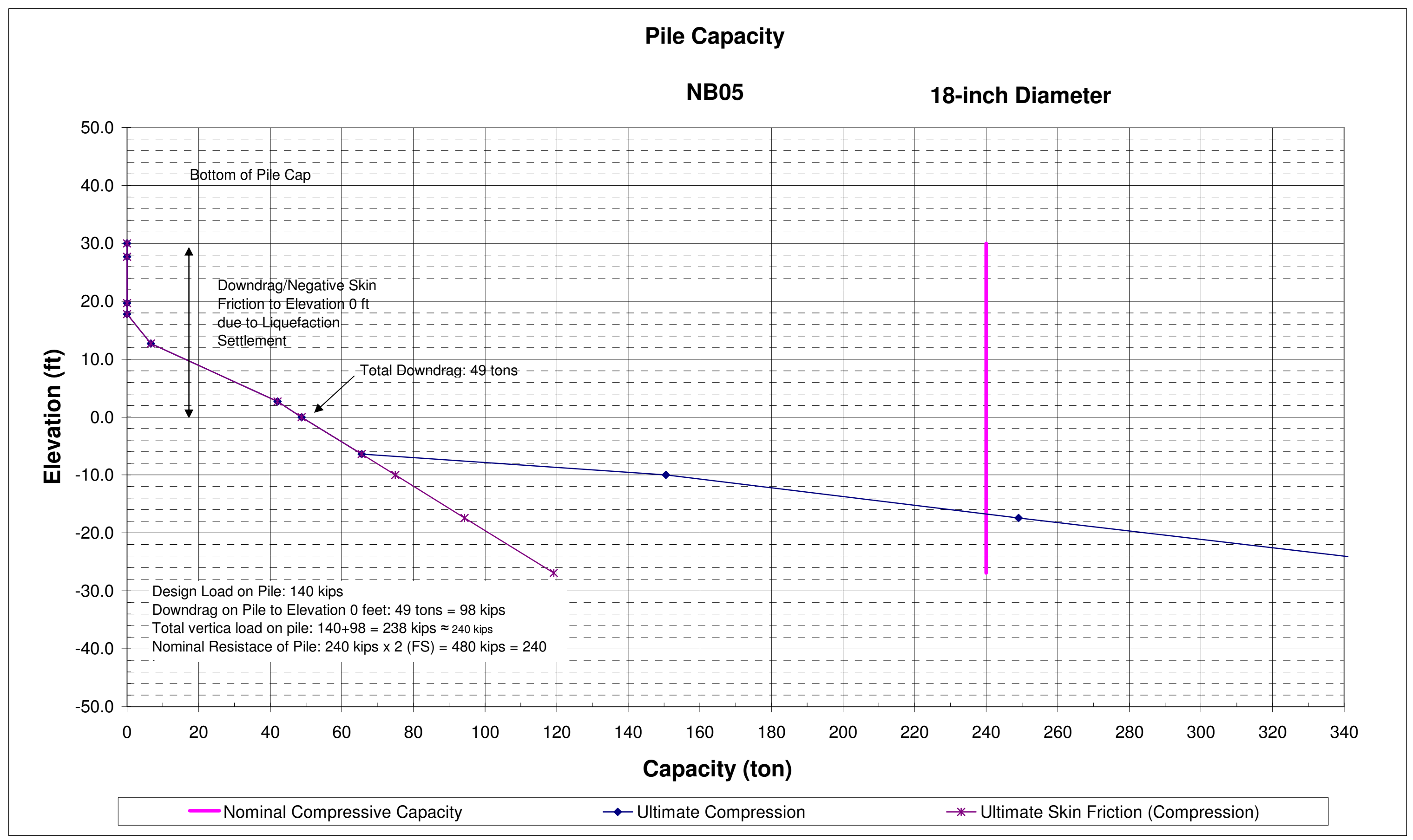
		Nominal axial compress. Capacity (ult.)	240	ton	(including downdrag)
Project Name:	Sonoma 101 HOV	Nominal axial compress. Capacity (ult.)	2136	kN	
Project Number:	28645047	Pile Diameter	18	inch	
Bridge Name:	Willow Brook Creek	Pile Diameter	457	mm	
Boring Surface El. (m)	9.1	Pile Diameter	1.50	ft	
Boring Surface El. (ft)	30.0	Crit depth ratio	20	-	
Design GWS El. (ft)	27.5	Crit depth	30.0	ft	
Unit Weight of Water (pcf)	62.4	Crit. Depth. Elev	0.0	ft	
Depth with no strength		Crit. Depth. Elev	0.0	m	
Convert. Factor	3.281	Crit. Eff. Vert. Str.	2036	psf	

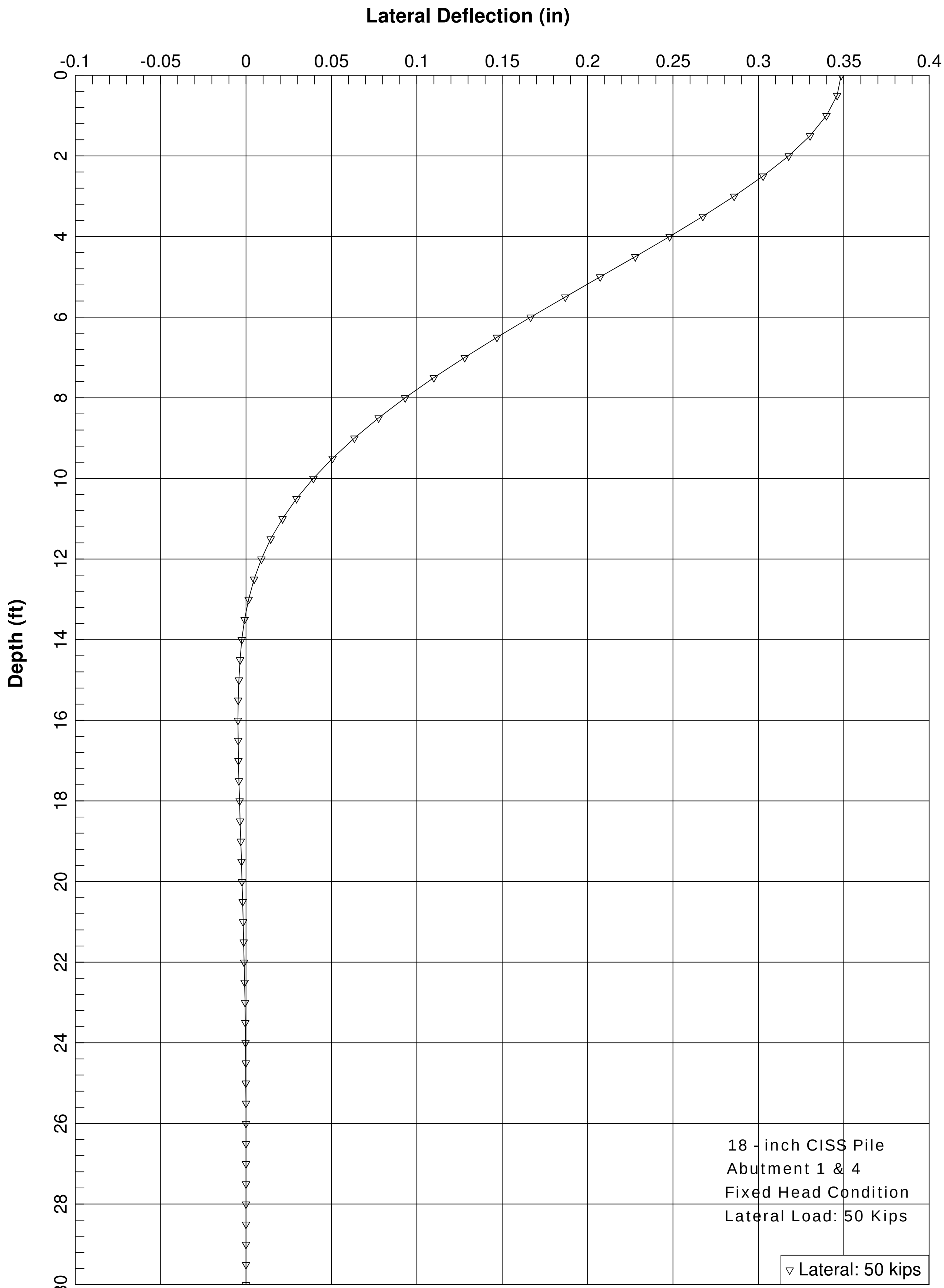
Pile Shape	Circular	(Square, Circular or Octogonal)			
Pile Surface Area	1.44	sq. m/m	Pile Tip Area	0.16	sq. m
Pile Surface Area	4.71	sq. ft/ft	Pile Tip Area	1.77	sq. ft.

Bottom of Pile Cap El. (m)	12.2	Skin Resistance (Friction) of Pile in Granular Material:			
Bottom of Pile Cap El. (ft)	40.0	Type (Steel=S,Concrete=C,Timber=T)		S	
Scour Elevation (ft)	17.8	Earth Press. Coefficient *	(Comp.)	1.5	
		Earth Press. Coefficient *	(Tens)	1.0	
		Minimum Adhesion		500	psf

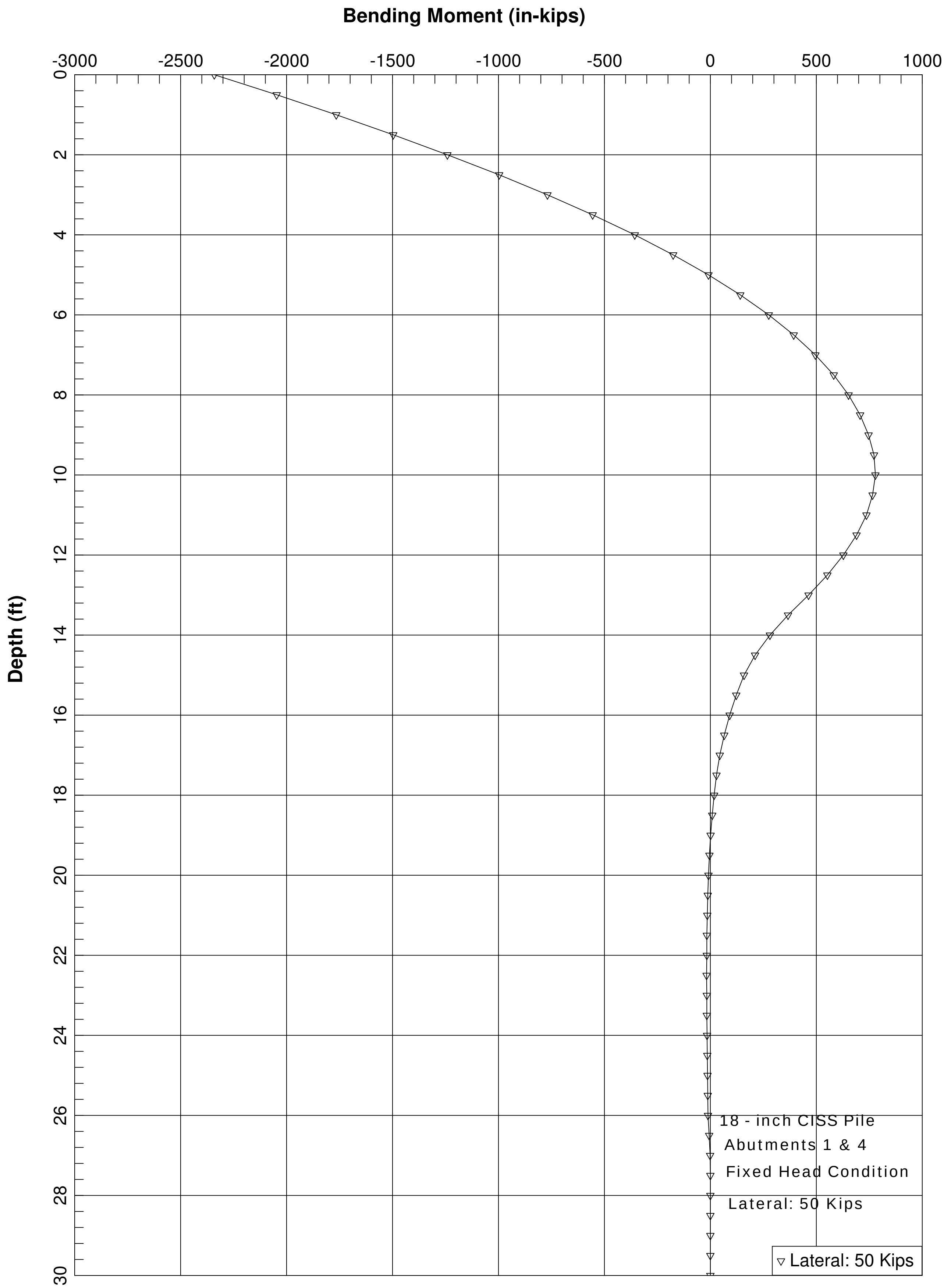
* Note: Soils in the potential scour zone are not neglected in this analysis and doesn't provide any suppport to pile; soils encountered above Elevation 0 feet are anticipated to have downdrag forces on the pile due to liquefaction settlement.

Layer No.	Layer Elevation Top		Layer Elevation Bot.		Lateral Resists?	Cohesion		Adhesion Ratio	Adhesion	Friction Angle		Layer Thickness	Internal Depth to Layer Top	Effective Vert. Stress (Bot. Lay.)	Design Effective Vert. Stress (Bot. Lay.)	delta Lat. Res. Cohesive	delta Lat. Res. Friction		Nq (granular)	z/B	Nc (cohesive)	Soil Type G= gran. C=cohes.	Bearing Capacity (B.C.)	Include B.C.?	Elevation to midlayer	Compression Cumulative Axial Res. (skin)		Compression Cumulative Axial Res. (Tot)		Compression Nominal Axial Capacity		Tension Cumulative Axial Res. (Skin)		Tension Nominal Axial Capacity	
						c	c	c_a/c	c_a	ϕ	δ						(comp)	(tensile)																	
(-)	(m)	(feet)	(m)	(feet)		(kPa)	(psf)		(psf)	(deg)	(deg)	(feet)	(feet)	(psf)	(psf)	(ton)	(ton)	(ton)	(-)	(-)	(-)		(ton)		(ft)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)	(ton)	(kN)
			9.14	30.0										0	0					0.0	6.25	c	0	n	30.0	0	0	0	0	240	2136	0	0	120	1068
1	9.14	30.0	8.44	27.7	n	86	1800	0.81	0	0	0	2.3	0.0	220	220	0.0	0.0	0.0	0	0.8	7.25	c	12	n	28.9	0.0	0	0.0	0	240	2136	0.0	0	120	1068
2	8.44	27.7	6	19.7	n	48	1000	0.96	0	0	0	8.0	2.3	716	716	0.0	0.0	0.0	0	4.2	9.00	c	8	n	23.7	0.0	0	0.0	0	240	2136	0.0	0	120	1068
3	6	19.7	5.43	17.8	n	0	0	0.00	0	32	20	1.9	10.3	849	849	0.0	0.0	0.0	29	7.5	9.00	g	22	n	18.8	0.0	0	0.0	0	240	2136	0.0	0	120	1068
4	5.43	17.8	3.87	12.7	y	0	0	0.00	0	32	20	5.1	12.2	1187	1187	0.0	6.7	4.5	29	9.8	9.00	g	30	n	15.3	6.7	59	6.7	59	240	2136	4.5	40	120	1068
5	3.87	12.7	0.82	2.7	y	120	2500	0.60	1500	0	0	10.0	17.3	1834	1834	35.3	0.0	0.0	0	14.9	9.00	c	20	n	7.7	42.0	374	42.0	374	240	2136	39.8	354	120	1068
6	0.82	2.7	0	0.0	y	0	0	0.00	0	32	20	2.7	27.3	2036	2036	0.0	6.7	4.5	29	19.1	9.00	g	52	n	1.4	48.7	434	48.7	434	240	2136	44.3	394	120	1068
7	0	0.0	-1.95	-6.4	y	0	0	0.00	0	32	20	6.4	30.0	2442	2036	0.0	16.8	11.2	29	22.1	9.00	g	52	n	-3.2	65.5	583	65.5	583	240	2136	55.5	493	120	1068
8	-1.95	-6.4	-3.05	-10.0	y	0	0	0.00	0	34	20	3.6	36.4	2730	2036	0.0	9.4	6.3	42	25.5	9.00	g	76	y	-8.2	74.9	667	150.5	1339	240	2136	61.7	549	120	1068
9	-3.05	-10.0	-5.3	-17.4	y	0	0	0.00	0	38	20	7.4	40.0	3246	2036	0.0	19.4	12.9	86	29.1	9.00	g	155	y	-13.7	94.3	839	249.0	2216	240	2136	74.7	664	120	1068
10	-5.3	-17.4	-8.2	-26.9	y	0	0	0.00	0	40	20	9.5	47.4	3860	2036	0.0	24.9	16.6	145	34.8	9.00	g	261	y	-22.2	119.2	1061	380.0	3382	240	2136	91.2	812	120	1068
Bottom	-8.2	-26.9																																	

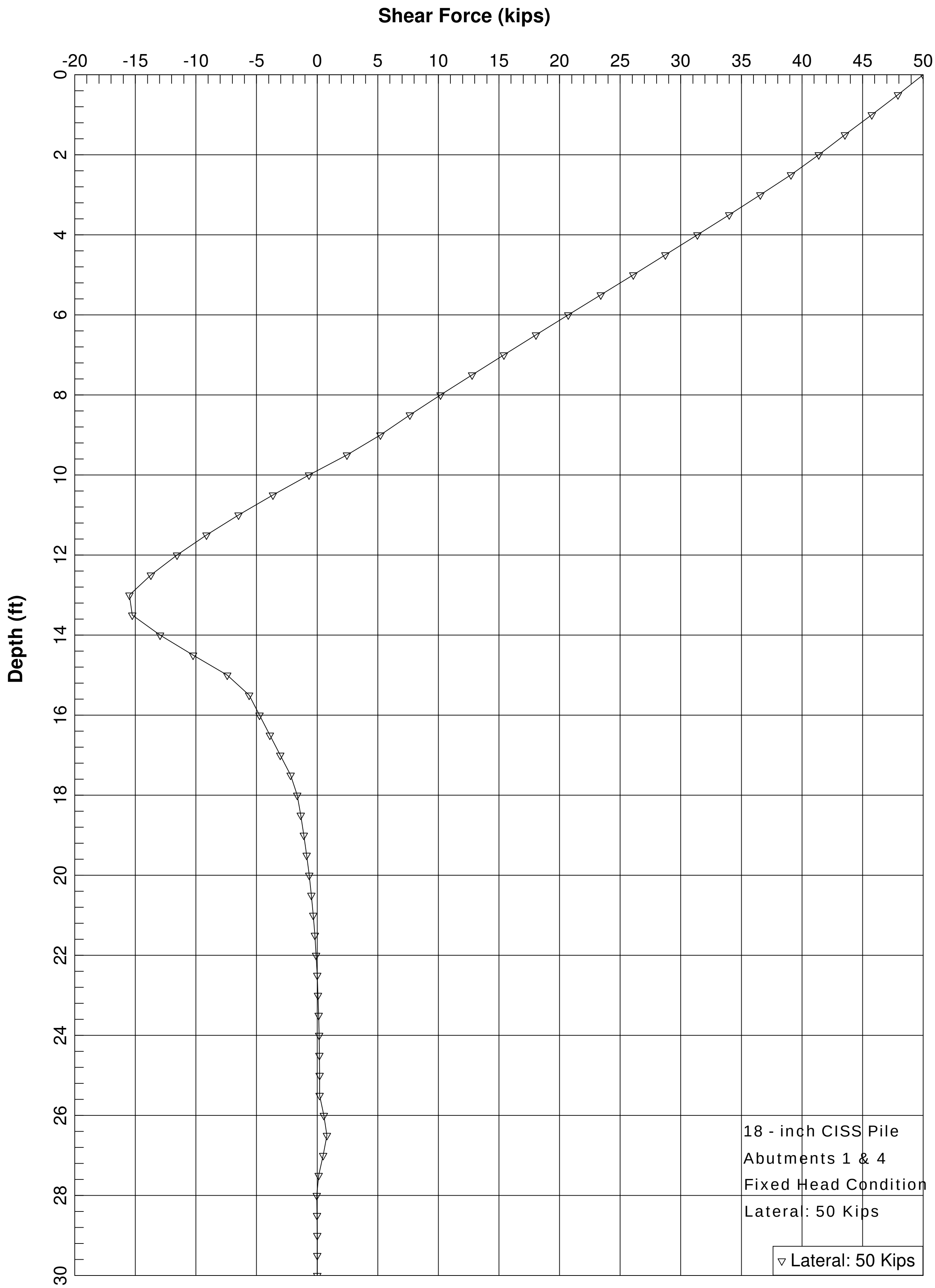


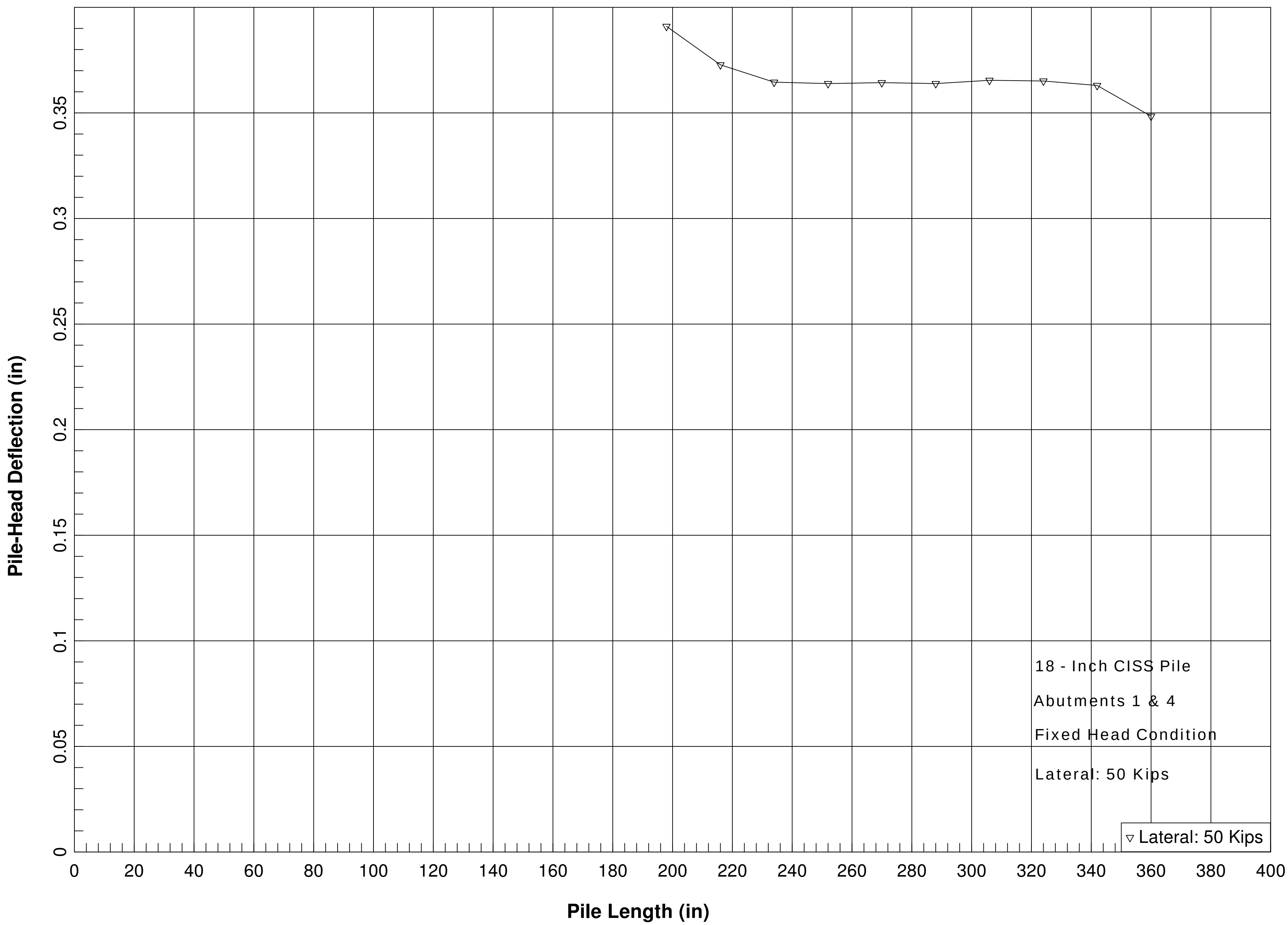


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutment 1 & 4
Fixed Head Condition
Lateral Load: 50 Kips
Lateral: 50 kips



Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutments 1 & 4
Fixed Head Condition
Lateral: 50 Kips
Lateral: 50 Kips


Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutments 1 & 4
Fixed Head Condition
Lateral: 50 Kips
Lateral: 50 Kips



Pile-Head Deflection (in)
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
Pile Length (in)
18 - Inch CISS Pile
Abutments 1 & 4
Fixed Head Condition
Lateral: 50 Kips
Lateral: 50 Kips

==

LPILE Plus for Windows, Version 4.0

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

==

This program is licensed to:

IT - San Jose
URS Corporation

Path to file locations: X:\101-Sonoma\geotech\Analysis\Pile Design\Lateral\For Bridge\New Folder\
Name of input data file: Willow Brook Creek_Abutment_ 18 inch CISS Pile_Fixed Head.lpd
Name of output file: Willow Brook Creek_Abutment_ 18 inch CISS Pile_Fixed Head.lpo
Name of plot output file: Willow Brook Creek_Abutment_ 18 inch CISS Pile_Fixed Head.lpp
Name of runtime file: Willow Brook Creek_Abutment_ 18 inch CISS Pile_Fixed Head.lpr

----
Time and Date of Analysis

Date: June 4, 2009 Time: 11:23:40

Problem Title

Sonoma 101 Segment B-Willow Brook Abutments 1 & 4 -18-in CISS Pile-Fixed

----
Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile

- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

Solution Control Parameters:
- Number of pile increments = 60
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 360.00 in
Depth of ground surface below top of pile = -72.00 in
Slope angle of ground surface = 26.60 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	18.000	5152.9000	254.5000	3605000.000
2	360.0000	18.000	5152.9000	254.5000	3605000.000

Soil and Rock Layering Information

The soil profile is modelled using 9 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = -72.000 in
Distance from top of pile to bottom of layer = 24.000 in
p-y subgrade modulus k for top of soil layer = 300.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 300.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 24.000 in
Distance from top of pile to bottom of layer = 108.000 in
p-y subgrade modulus k for top of soil layer = 300.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 300.000 lbs/in**3

Layer 3 is stiff clay without free water
Distance from top of pile to top of layer = 108.000 in
Distance from top of pile to bottom of layer = 180.000 in
p-y subgrade modulus k for top of soil layer = 625.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 625.000 lbs/in**3

Layer 4 is stiff clay without free water
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 216.000 in
p-y subgrade modulus k for top of soil layer = 72.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 72.000 lbs/in**3

```
Layer  5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      216.000 in
Distance from top of pile to bottom of layer =      312.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  6 is stiff clay without free water
Distance from top of pile to top of layer    =      312.000 in
Distance from top of pile to bottom of layer =      444.000 in
p-y subgrade modulus k for top of soil layer =      625.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      625.000 lbs/in**3

Layer  7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      444.000 in
Distance from top of pile to bottom of layer =      564.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  8 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      564.000 in
Distance from top of pile to bottom of layer =      720.000 in
p-y subgrade modulus k for top of soil layer =      100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      100.000 lbs/in**3

Layer  9 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      720.000 in
Distance from top of pile to bottom of layer =      828.000 in
p-y subgrade modulus k for top of soil layer =      125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      125.000 lbs/in**3


(Depth of lowest layer extends  468.00 in below pile tip)


-------------------------------------------------------------------------------
                   Effective Unit Weight of Soil vs. Depth
-------------------------------------------------------------------------------

Distribution of effective unit weight of soil with depth
is defined using 18 points
```

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-72.00	.07240
2	24.00	.07240
3	24.00	.06620
4	108.00	.06620
5	108.00	.03370
6	180.00	.03370
7	180.00	.03640
8	216.00	.03640
9	216.00	.03890
10	312.00	.03890
11	312.00	.03760
12	444.00	.03760
13	444.00	.04040
14	564.00	.04040
15	564.00	.04450
16	720.00	.04450
17	720.00	.04450
18	828.00	.04450

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 18 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-72.000	10.41700	.00	.00800	.0
2	24.000	10.41700	.00	.00800	.0
3	24.000	12.50000	.00	.00800	.0
4	108.000	12.50000	.00	.00800	.0
5	108.000	17.36100	.00	.00600	.0
6	180.000	17.36100	.00	.00600	.0
7	180.000	5.55600	.00	.01000	.0
8	216.000	5.55600	.00	.01000	.0
9	216.000	.00000	32.00	------	------
10	312.000	.00000	32.00	------	------
11	312.000	17.36100	.00	.00600	.0
12	444.000	17.36100	.00	.00600	.0
13	444.000	.00000	32.00	------	------
14	564.000	.00000	32.00	------	------
15	564.000	.00000	36.00	------	------
16	720.000	.00000	36.00	------	------
17	720.000	.00000	38.00	------	------
18	828.000	.00000	38.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Static loading criteria was used for computation of p-y curves

----
Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Shear force at pile head = 50000.000 lbs
Slope at pile head = .000 in/in
Axial load at pile head = 90000.000 lbs

(Zero slope for this load indicates fixed-head condition)

Output of p-y Curves at Specified Depths
----

p-y curves are generated and printed for verification at 1 depths.

Depth No.	Depth Below Pile Head in	Depth Below Ground Surface in
1	60.000	132.000

Depth of ground surface below top of pile = -72.00 in

p-y Curve Computed Using Criteria for Stiff Clay without Free Water

```
Soil Layer Number            =          2
Depth below pile head        =     60.000 in
Depth below ground surface =      132.000 in
Equivalent Depth             =    114.048 in
Diameter                     =     18.000 in
Undrained cohesion, c        =   12.50000 lbs/in**2
Avg. Undrained cohesion, c =     12.50000 lbs/in**2
Average Eff. Unit Weight     =     .07071 lbs/in**3
Epsilon-50                   =     .00800
Pct                          =   1021.452 lbs/in
Pcd                          =   2025.000 lbs/in
y50                          =       .360 in
p-multiplier                 =    1.00000
y-multiplier                 =    1.00000
```

y, in	p, lbs/in
.0000	.000
.0001	57.440
.0003	85.894
.0006	102.145
.0029	152.743
.0058	181.643
.0288	271.619
.0576	323.011
.1440	406.165
.2880	483.015
.4320	534.544
.5760	574.405
1.4400	722.275
2.8800	858.935
5.7600	1021.452
6.4800	1021.452
7.2000	1021.452

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Slope (BC Type 2)
Specified shear force at pile head = 50000.000 lbs
Specified slope at pile head = 0.000E+00 in/in
Specified axial load at pile head = 90000.000 lbs

(Zero slope for this load indicates fixed-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	.348431	-2.341E+06	50000.0000	5.551E-17	4442.2565	-340.8249
6.000	.346163	-2.047E+06	47918.0480	-7.086E-04	3928.6382	-353.1591
12.000	.339928	-1.765E+06	45765.4016	-.001324	3436.6020	-364.3898
18.000	.330272	-1.496E+06	43548.6908	-.001851	2966.9399	-374.5138
24.000	.317717	-1.241E+06	41387.8379	-.002293	2520.3705	-345.7704
30.000	.302757	-9.971E+05	39097.3272	-.002654	2095.1641	-417.7331
36.000	.285865	-7.685E+05	36566.5584	-.002939	1695.9199	-425.8565
42.000	.267484	-5.551E+05	33990.9485	-.003153	1323.2183	-432.6801
48.000	.248027	-3.572E+05	31378.4076	-.003301	977.5533	-438.1668
54.000	.227878	-1.750E+05	28737.0805	-.003386	659.3301	-442.2756
60.000	.207389	-8718.8021	26075.3691	-.003416	368.8627	-444.9616
66.000	.186883	1.416E+05	23401.9565	-.003395	600.8985	-446.1760
72.000	.166652	2.758E+05	20725.8334	-.003327	835.2932	-445.8651
78.000	.146956	3.939E+05	18056.3285	-.003219	1041.5691	-443.9699
84.000	.128022	4.959E+05	15403.1452	-.003075	1219.8095	-440.4246
90.000	.110050	5.820E+05	12776.4060	-.002901	1370.2061	-435.1552
96.000	.093206	6.524E+05	10186.7084	-.002702	1493.0640	-428.0774
102.000	.077626	7.072E+05	7645.1968	-.002482	1588.8069	-419.0932
108.000	.063416	7.468E+05	5215.2379	-.002248	1657.9830	-390.8931
114.000	.050654	7.722E+05	2451.0741	-.002002	1702.3532	-530.4949
120.000	.039388	7.784E+05	-669.7851	-.001752	1713.1322	-509.7915
126.000	.029631	7.661E+05	-3655.9673	-.001503	1691.6198	-485.6025
132.000	.021358	7.361E+05	-6485.0288	-.001260	1639.3408	-457.4180
138.000	.014512	6.896E+05	-9130.3524	-.001030	1558.0762	-424.3565
144.000	.009002	6.277E+05	-11557.4236	-8.169E-04	1449.9193	-384.6672
150.000	.004709	5.518E+05	-13713.5505	-6.264E-04	1317.3844	-334.0418
156.000	.001485	4.638E+05	-15482.8920	-4.624E-04	1163.6777	-255.7387
162.000	-8.41E-04	3.665E+05	-15256.6496	-3.283E-04	993.7496	331.1528
168.000	-.002456	2.811E+05	-12961.6289	-2.238E-04	844.5318	433.8541
174.000	-.003526	2.112E+05	-10234.7909	-1.443E-04	722.5080	475.0919
180.000	-.004187	1.584E+05	-7424.7209	-8.458E-05	630.2923	461.5981
186.000	-.004541	1.222E+05	-5612.1720	-3.927E-05	567.0523	142.5849
192.000	-.004658	91095.1678	-4753.9047	-4.821E-06	512.7404	143.5042
198.000	-.004599	65149.4740	-3894.2399	2.041E-05	467.4240	143.0508
204.000	-.004413	44342.2445	-3040.3188	3.809E-05	431.0823	141.5896
210.000	-.004141	28624.5062	-2197.4603	4.988E-05	403.6298	139.3633
216.000	-.003814	17918.8526	-1635.5333	5.740E-05	384.9315	47.9457
222.000	-.003453	8936.1198	-1357.7703	6.173E-05	369.2423	44.6419
228.000	-.003074	1558.9383	-1101.3018	6.343E-05	356.3574	40.8476
234.000	-.002692	-4348.0027	-868.5420	6.298E-05	361.2288	36.7390
240.000	-.002318	-8931.5801	-660.9045	6.083E-05	369.2344	32.4735
246.000	-.001962	-12344.5550	-478.9213	5.740E-05	375.1954	28.1876
252.000	-.001629	-14740.6239	-322.3661	5.302E-05	379.3804	23.9975
258.000	-.001325	-16270.2113	-190.3760	4.801E-05	382.0520	19.9992
264.000	-.001053	-17076.9910	-81.5710	4.263E-05	383.4611	16.2692
270.000	-8.14E-04	-17295.1018	5.8355	3.708E-05	383.8420	12.8663
276.000	-6.08E-04	-17047.0080	73.9341	3.153E-05	383.4087	9.8332
282.000	-4.35E-04	-16441.9458	125.0278	2.612E-05	382.3519	7.1980
288.000	-2.95E-04	-15574.8867	161.5520	2.095E-05	380.8375	4.9767
294.000	-1.84E-04	-14525.9496	186.0061	1.609E-05	379.0054	3.1747
300.000	-1.02E-04	-13360.1914	200.8961	1.159E-05	376.9693	1.7887
306.000	-4.50E-05	-12127.7112	208.6877	7.471E-06	374.8167	.8086
312.000	-1.19E-05	-10864.0076	555.3721	3.758E-06	372.6095	114.7529
318.000	9.99E-08	-5467.3043	796.2160	1.121E-06	363.1837	-34.4716
324.000	1.53E-06	-1310.6255	485.8474	2.594E-08	355.9237	-68.9846
330.000	4.11E-07	362.8368	126.8867	-1.271E-07	354.2683	-50.6690

336.000	4.96E-13	212.1523	-30.2382	-3.426E-08	354.0051	-1.7060
342.000	-1.50E-11	.015734	-17.6800	-4.135E-14	353.6346	5.8920
348.000	-1.41E-17	-.007739	-.001311	1.250E-12	353.6346	8.670E-04
354.000	3.48E-18	-1.085E-08	6.449E-04	1.171E-18	353.6346	-2.150E-04
360.000	9.77E-24	0.0000	0.0000	-5.808E-19	353.6346	-6.026E-10

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =      .34843137 in
Computed slope at pile head      =    5.55112E-17
Maximum bending moment           =   -2340917.763 lbs-in
Maximum shear force              =      50000.000 lbs
Depth of maximum bending moment  =          0.000 in
Depth of maximum shear force     =          0.000 in
Number of iterations             =             28
Number of zero deflection points =              5
```

Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
2	V= 50000.000	S= 0.000	90000.0000	.3484	-2.341E+06	50000.0000

Pile-head Deflection vs. Pile Length

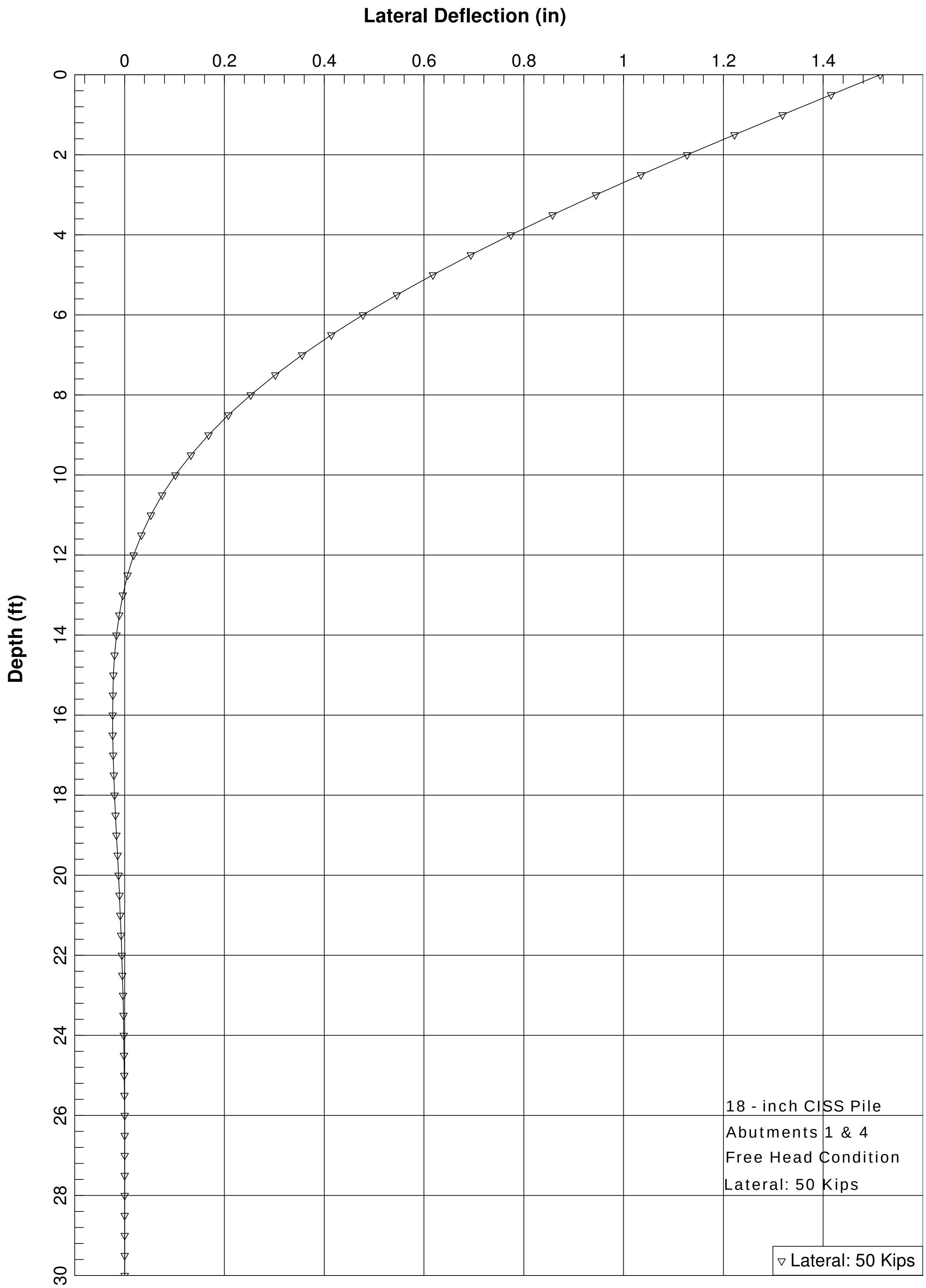
Boundary Condition Type 2, Shear and Slope

```
Shear      =          50000. lbs
Slope      =          .00000
Axial Load =          90000. lbs
```

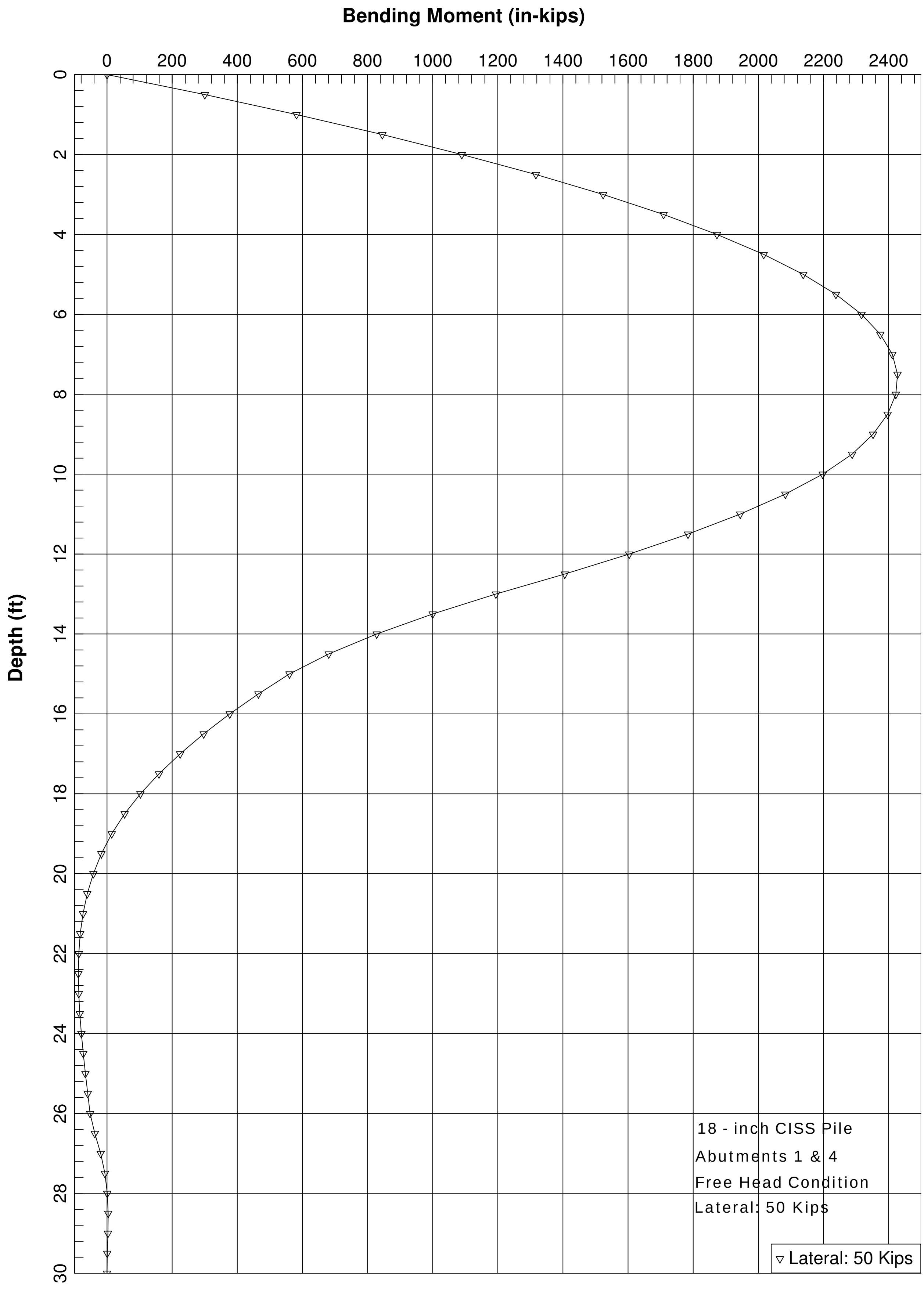
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
360.000	.34843137	-2340917.763	50000.000
342.000	.36303684	-2363403.300	50000.000

324.000	.36505965	-2365658.078	50000.000
306.000	.36539383	-2365710.225	50000.000
288.000	.36389376	-2363593.861	50000.000
270.000	.36431223	-2365030.544	50000.000
252.000	.36386771	-2365426.909	50000.000
234.000	.36451849	-2364160.349	50000.000
216.000	.37278514	-2371108.853	50000.000
198.000	.39098800	-2388768.776	50000.000

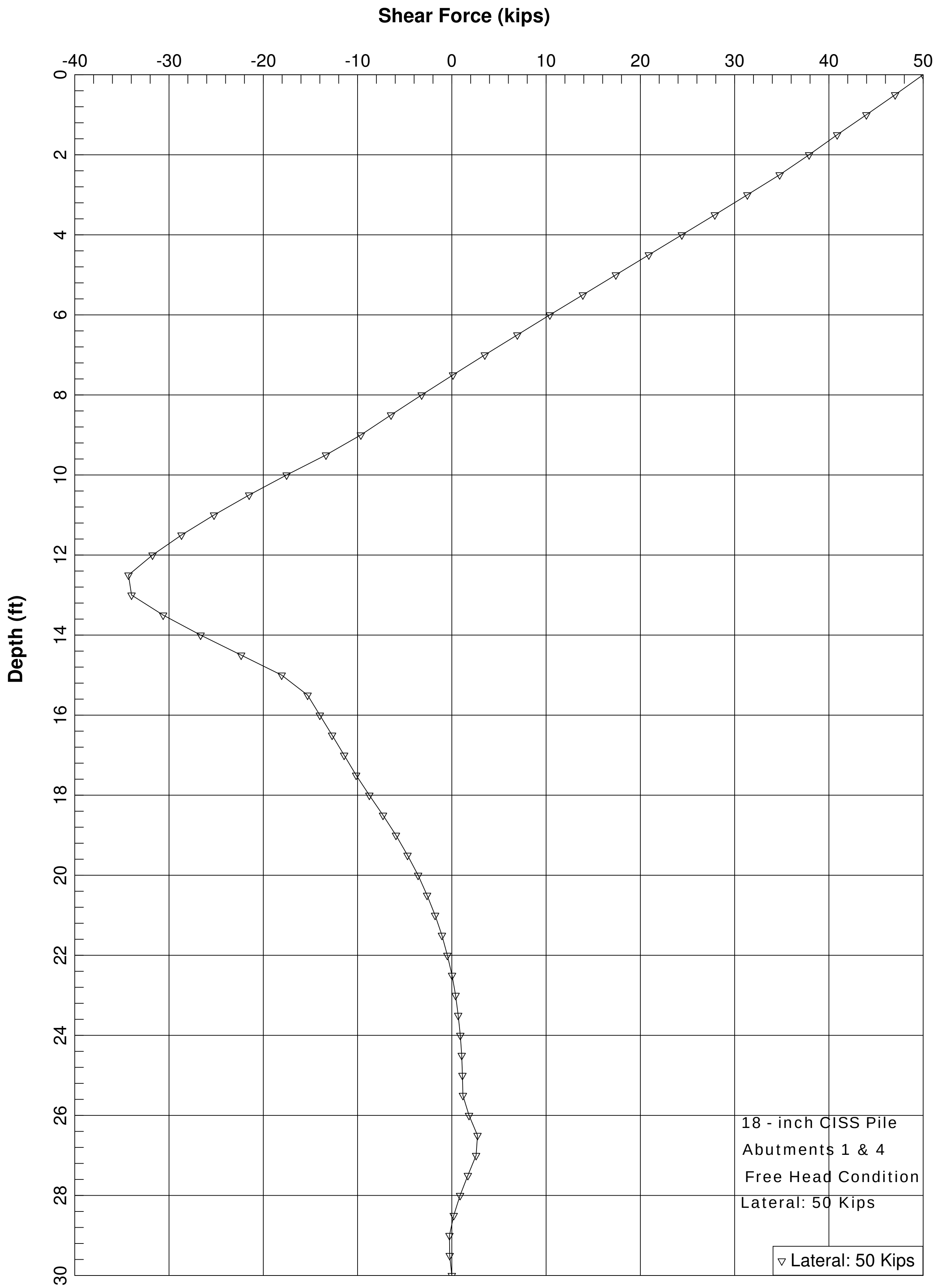
The analysis ended normally.



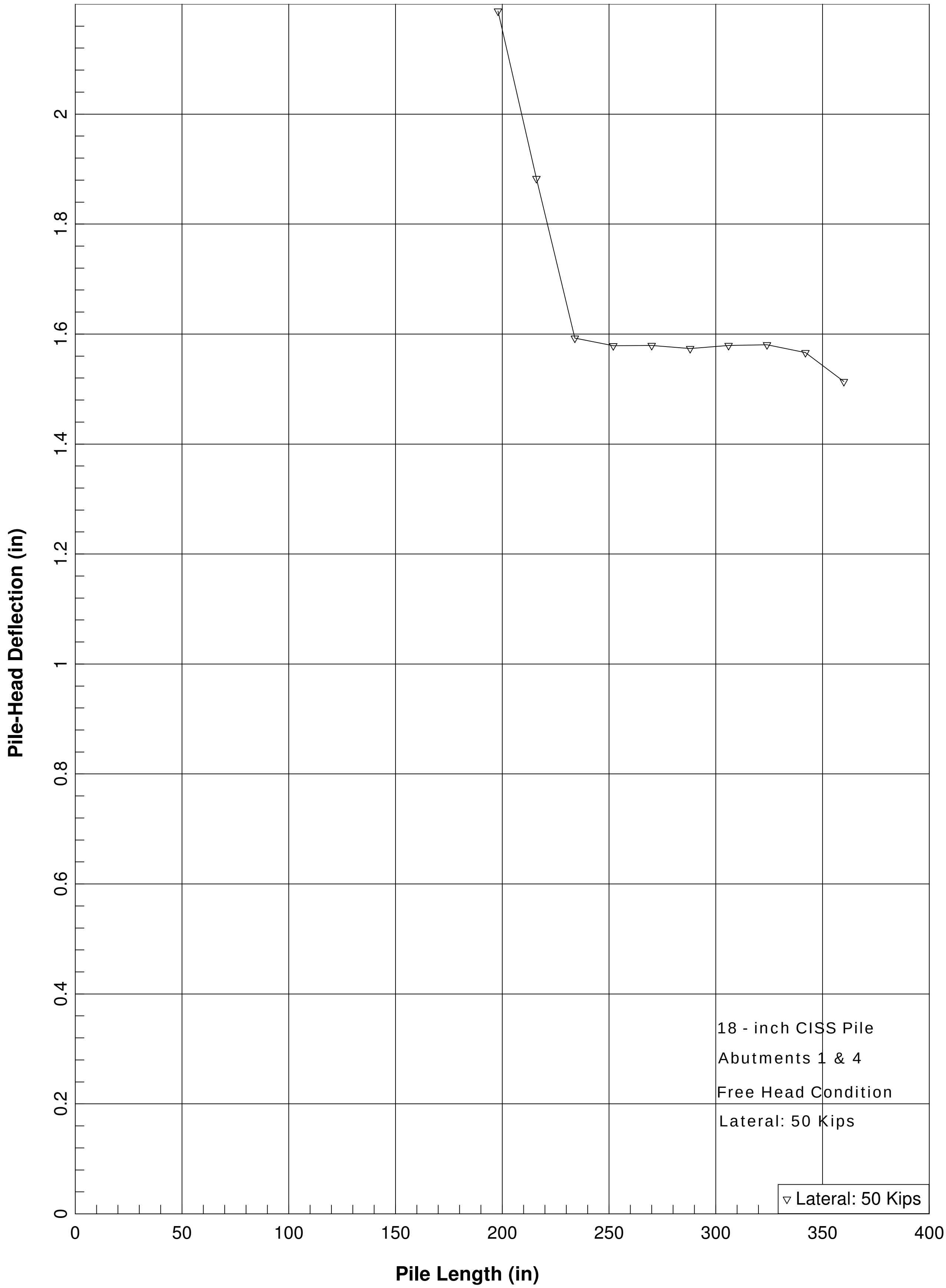
Lateral Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutments 1 & 4
Free Head Condition
Lateral: 50 Kips
Lateral: 50 Kips



Bending Moment (in-kips)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
2400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutments 1 & 4
Free Head Condition
Lateral: 50 Kips
Lateral: 50 Kips



Shear Force (kips)
-40
-30
-20
-10
0
10
20
30
40
50
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Abutments 1 & 4
Free Head Condition
Lateral: 50 Kips
Lateral: 50 Kips



Pile-Head Deflection (in)
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
0
50
100
150
200
250
300
350
400
Pile Length (in)
18 - inch CISS Pile
Abutments 1 & 4
Free Head Condition
Lateral: 50 Kips
Lateral: 50 Kips

```
===============================================================================

                   LPILE Plus for Windows, Version 4.0

               Analysis of Individual Piles and Drilled Shafts
              Subjected to Lateral Loading Using the p-y Method

                   (c) Copyright ENSOFT, Inc., 1985-2001
                            All Rights Reserved

===============================================================================


This program is licensed to:

IT - San Jose
URS Corporation

Path to file locations:      X:\101-Sonoma\geotech\Analysis\Pile Design\Lateral\For Bridge\New Folder\
Name of input data file:     Willow Brook Creek_Abutment_18 inch CISS Pile_Free Head.lpd
Name of output file:         Willow Brook Creek_Abutment_18 inch CISS Pile_Free Head.lpo
Name of plot output file:    Willow Brook Creek_Abutment_18 inch CISS Pile_Free Head.lpp
Name of runtime file:        Willow Brook Creek_Abutment_18 inch CISS Pile_Free Head.lpr


-------------------------------------------------------------------------------
                          Time and Date of Analysis
-------------------------------------------------------------------------------

               Date:  June  4, 2009     Time:  11:24:48


-------------------------------------------------------------------------------
                                Problem Title
-------------------------------------------------------------------------------

Sonoma 101 Segment B-Abutments 1 & 4-18-in CISS Pile-Free


-------------------------------------------------------------------------------
                               Program Options
-------------------------------------------------------------------------------

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
```

```
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

Solution Control Parameters:
- Number of pile increments            =          60
- Maximum number of iterations allowed =         100
- Deflection tolerance for convergence =   1.0000E-05 in
- Maximum allowable deflection         =   1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and
  soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) =  1


-------------------------------------------------------------------------------
                   Pile Structural Properties and Geometry
-------------------------------------------------------------------------------

Pile Length                                 =     360.00 in
Depth of ground surface below top of pile =     -72.00 in
Slope angle of ground surface               =      26.60 deg.

Structural properties of pile defined using  2 points

Point    Depth         Pile      Moment of       Pile      Modulus of
           X         Diameter     Inertia        Area      Elasticity
           in           in         in**4        Sq.in      lbs/Sq.in
-----  ---------   ------------  -----------  -----------  -----------
  1       0.0000       18.000    5152.9000     254.5000   3605000.000
  2     360.0000       18.000    5152.9000     254.5000   3605000.000


-------------------------------------------------------------------------------
                      Soil and Rock Layering Information
-------------------------------------------------------------------------------

The soil profile is modelled using  9 layers

Layer  1 is stiff clay without free water
Distance from top of pile to top of layer    =      -72.000 in
Distance from top of pile to bottom of layer =       24.000 in
p-y subgrade modulus k for top of soil layer =      300.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      300.000 lbs/in**3

Layer  2 is stiff clay without free water
Distance from top of pile to top of layer    =       24.000 in
Distance from top of pile to bottom of layer =      108.000 in
p-y subgrade modulus k for top of soil layer =      300.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      300.000 lbs/in**3

Layer  3 is stiff clay without free water
Distance from top of pile to top of layer    =      108.000 in
Distance from top of pile to bottom of layer =      180.000 in
p-y subgrade modulus k for top of soil layer =      625.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      625.000 lbs/in**3

Layer  4 is stiff clay without free water
Distance from top of pile to top of layer    =      180.000 in
Distance from top of pile to bottom of layer =      216.000 in
p-y subgrade modulus k for top of soil layer =       72.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       72.000 lbs/in**3
```

```
Layer  5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      216.000 in
Distance from top of pile to bottom of layer =      312.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  6 is stiff clay without free water
Distance from top of pile to top of layer    =      312.000 in
Distance from top of pile to bottom of layer =      444.000 in
p-y subgrade modulus k for top of soil layer =      625.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      625.000 lbs/in**3

Layer  7 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      444.000 in
Distance from top of pile to bottom of layer =      564.000 in
p-y subgrade modulus k for top of soil layer =       60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =       60.000 lbs/in**3

Layer  8 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      564.000 in
Distance from top of pile to bottom of layer =      720.000 in
p-y subgrade modulus k for top of soil layer =      100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      100.000 lbs/in**3

Layer  9 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer    =      720.000 in
Distance from top of pile to bottom of layer =      828.000 in
p-y subgrade modulus k for top of soil layer =      125.000 lbs/in**3
p-y subgrade modulus k for bottom of layer   =      125.000 lbs/in**3


(Depth of lowest layer extends  468.00 in below pile tip)


-------------------------------------------------------------------------------
                   Effective Unit Weight of Soil vs. Depth
-------------------------------------------------------------------------------

Distribution of effective unit weight of soil with depth
is defined using 18 points
```

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	-72.00	.07240
2	24.00	.07240
3	24.00	.06620
4	108.00	.06620
5	108.00	.03370
6	180.00	.03370
7	180.00	.03640
8	216.00	.03640
9	216.00	.03890
10	312.00	.03890
11	312.00	.03760
12	444.00	.03760
13	444.00	.04040
14	564.00	.04040
15	564.00	.04450
16	720.00	.04450
17	720.00	.04450
18	828.00	.04450

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 18 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	-72.000	10.41700	.00	.00800	.0
2	24.000	10.41700	.00	.00800	.0
3	24.000	12.50000	.00	.00800	.0
4	108.000	12.50000	.00	.00800	.0
5	108.000	17.36100	.00	.00600	.0
6	180.000	17.36100	.00	.00600	.0
7	180.000	5.55600	.00	.01000	.0
8	216.000	5.55600	.00	.01000	.0
9	216.000	.00000	32.00	------	------
10	312.000	.00000	32.00	------	------
11	312.000	17.36100	.00	.00600	.0
12	444.000	17.36100	.00	.00600	.0
13	444.000	.00000	32.00	------	------
14	564.000	.00000	32.00	------	------
15	564.000	.00000	36.00	------	------
16	720.000	.00000	36.00	------	------
17	720.000	.00000	38.00	------	------
18	828.000	.00000	38.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Static loading criteria was used for computation of p-y curves

---

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 50000.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = 90000.000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Output of p-y Curves at Specified Depths

---

p-y curves are generated and printed for verification at 1 depths.

Depth No.	Depth Below Pile Head in	Depth Below Ground Surface in
1	60.000	132.000

Depth of ground surface below top of pile =   -72.00 in

p-y Curve Computed Using Criteria for Stiff Clay without Free Water

```
Soil Layer Number           =          2
Depth below pile head       =      60.000 in
Depth below ground surface =      132.000 in
Equivalent Depth            =     114.048 in
Diameter                    =      18.000 in
Undrained cohesion, c       =    12.50000 lbs/in**2
Avg. Undrained cohesion, c =     12.50000 lbs/in**2
Average Eff. Unit Weight    =      .07071 lbs/in**3
Epsilon-50                  =      .00800
Pct                         =    1021.452 lbs/in
Pcd                         =    2025.000 lbs/in
y50                         =        .360 in
p-multiplier                =     1.00000
y-multiplier                =     1.00000
```

y, in	p, lbs/in
.0000	.000
.0001	57.440
.0003	85.894
.0006	102.145
.0029	152.743
.0058	181.643
.0288	271.619
.0576	323.011
.1440	406.165
.2880	483.015
.4320	534.544
.5760	574.405
1.4400	722.275
2.8800	858.935
5.7600	1021.452
6.4800	1021.452
7.2000	1021.452

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

```
Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head    =       50000.000 lbs
Specified bending moment at pile head =            .000 in-lbs
Specified axial load at pile head     =       90000.000 lbs
```

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	1.514	4.583E-07	50000.0000	-.016340	353.6346	-492.0596
6.000	1.416	3.000E+05	47017.1520	-.016291	877.5527	-502.2231
12.000	1.318	5.818E+05	43976.4172	-.016149	1369.8010	-511.3552
18.000	1.222	8.451E+05	40884.1042	-.015918	1829.7196	-519.4158
24.000	1.127	1.090E+06	37902.1932	-.015606	2256.7212	-474.5546
30.000	1.035	1.317E+06	34774.6026	-.015217	2653.5523	-567.9756
36.000	.944753	1.523E+06	31348.1502	-.014759	3014.2694	-574.1752
42.000	.857677	1.709E+06	27888.6791	-.014237	3338.4200	-578.9818
48.000	.773912	1.873E+06	24404.7051	-.013658	3625.6452	-582.3429
54.000	.693778	2.017E+06	20905.0698	-.013030	3875.6837	-584.2022
60.000	.617553	2.138E+06	17398.9639	-.012359	4088.3748	-584.4997
66.000	.545470	2.239E+06	13895.9547	-.011652	4263.6628	-583.1700
72.000	.477727	2.318E+06	10406.0222	-.010916	4401.6007	-580.1409
78.000	.414475	2.375E+06	6939.6050	-.010158	4502.3550	-575.3315
84.000	.355826	2.412E+06	3507.6613	-.009385	4566.2104	-568.6497
90.000	.301851	2.428E+06	121.7491	-.008604	4593.5759	-559.9877
96.000	.252581	2.423E+06	-3205.8622	-.007820	4584.9915	-549.2161
102.000	.208006	2.398E+06	-6462.0369	-.007042	4541.1358	-536.1754
108.000	.168077	2.353E+06	-9648.6868	-.006275	4462.8366	-526.0412
114.000	.132708	2.289E+06	-13357.5104	-.005525	4350.7447	-710.2333
120.000	.101774	2.198E+06	-17526.1407	-.004801	4193.2981	-679.3101
126.000	.075100	2.083E+06	-21493.6680	-.004109	3992.4686	-643.1990
132.000	.052463	1.945E+06	-25224.6300	-.003459	3750.5620	-600.4550
138.000	.033596	1.784E+06	-28670.7679	-.002856	3470.3079	-548.2576
144.000	.018187	1.604E+06	-31755.0087	-.002309	3155.0373	-479.8227
150.000	.005886	1.406E+06	-34302.0142	-.001823	2809.1082	-369.1792
156.000	-.003690	1.194E+06	-33967.9189	-.001403	2439.5378	480.5443
162.000	-.010952	9.998E+05	-30633.3477	-.001049	2099.8189	630.9795
168.000	-.016276	8.278E+05	-26650.2398	-7.537E-04	1799.4697	696.7231
174.000	-.019996	6.808E+05	-22359.4638	-5.101E-04	1542.6763	733.5356
180.000	-.022397	5.600E+05	-18052.0511	-3.097E-04	1331.7982	702.2687
186.000	-.023712	4.645E+05	-15298.4227	-1.442E-04	1164.9062	215.6075
192.000	-.024127	3.766E+05	-14001.9604	-8.370E-06	1011.4295	216.5466
198.000	-.023812	2.965E+05	-12704.8061	1.003E-04	871.4539	215.8382
204.000	-.022923	2.241E+05	-11415.9062	1.844E-04	744.9593	213.7951
210.000	-.021600	1.593E+05	-10142.5981	2.463E-04	631.8393	210.6409
216.000	-.019967	1.021E+05	-8753.0486	2.885E-04	531.9153	252.5423
222.000	-.018137	53936.4036	-7287.6411	3.137E-04	447.8393	235.9268
228.000	-.016203	14283.1348	-5930.0710	3.247E-04	378.5813	216.5965
234.000	-.014241	-17575.1505	-4693.8014	3.242E-04	384.3311	195.4934
240.000	-.012313	-42392.6088	-3586.9494	3.145E-04	427.6771	173.4573
246.000	-.010467	-60958.2108	-2612.9178	2.978E-04	460.1035	151.2199
252.000	-.008739	-74069.2643	-1771.0499	2.760E-04	483.0032	129.4027
258.000	-.007155	-82508.9007	-1057.2855	2.507E-04	497.7437	108.5187
264.000	-.005730	-87027.4718	-464.7986	2.233E-04	505.6358	88.9769
270.000	-.004474	-88327.6948	15.3995	1.950E-04	507.9068	71.0891
276.000	-.003390	-87053.3043	393.9019	1.667E-04	505.6809	55.0784
282.000	-.002474	-83780.9092	682.4026	1.391E-04	499.9654	41.0886
288.000	-.001721	-79014.7131	893.2512	1.128E-04	491.6408	29.1943
294.000	-.001120	-73183.7406	1039.0667	8.824E-05	481.4565	19.4109
300.000	-6.62E-04	-66641.2123	1132.4107	6.566E-05	470.0294	11.7038
306.000	-3.32E-04	-59665.7244	1185.5142	4.526E-05	457.8461	5.9974
312.000	-1.19E-04	-52463.9241	1814.2306	2.715E-05	445.2675	203.5747
318.000	-6.47E-06	-37924.2823	2720.2787	1.256E-05	419.8727	98.4413
324.000	3.21E-05	-19834.1397	2575.0336	3.228E-06	388.2767	-146.8563
330.000	3.23E-05	-7027.3650	1693.3665	-1.110E-06	365.9085	-147.0327

336.000	1.88E-05	487.4575	866.9019	-2.167E-06	354.4860	-128.4555
342.000	6.26E-06	3377.7983	188.6860	-1.542E-06	359.5342	-97.6165
348.000	2.77E-07	2753.3554	-239.0394	-5.522E-07	358.4436	-44.9586
354.000	-3.68E-07	509.9215	-229.4485	-2.522E-08	354.5252	48.1556
360.000	-2.55E-08	0.0000	0.0000	5.713E-08	353.6346	28.3273

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     1.51387987 in
Computed slope at pile head      =     -.01633977
Maximum bending moment           =    2427554.848 lbs-in
Maximum shear force              =      50000.000 lbs
Depth of maximum bending moment  =         90.000 in
Depth of maximum shear force     =          0.000 in
Number of iterations             =             36
Number of zero deflection points =              4
```

Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 50000.000	M= 0.000	90000.0000	1.5139	2.428E+06	50000.0000

Pile-head Deflection vs. Pile Length

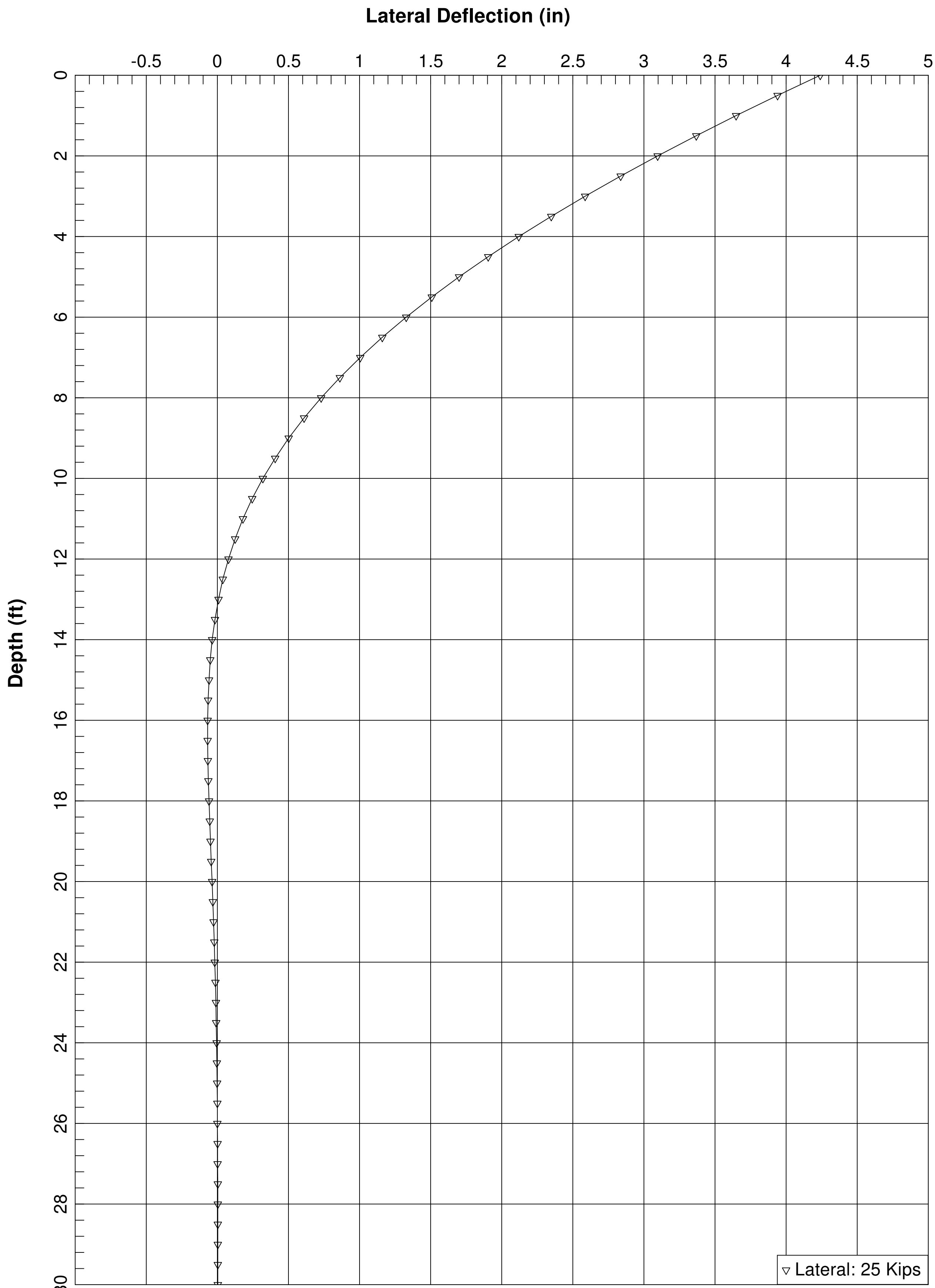
Boundary Condition Type 1, Shear and Moment

```
Shear      =          50000. lbs
Moment     =              0. in-lbs
Axial Load =          90000. lbs
```

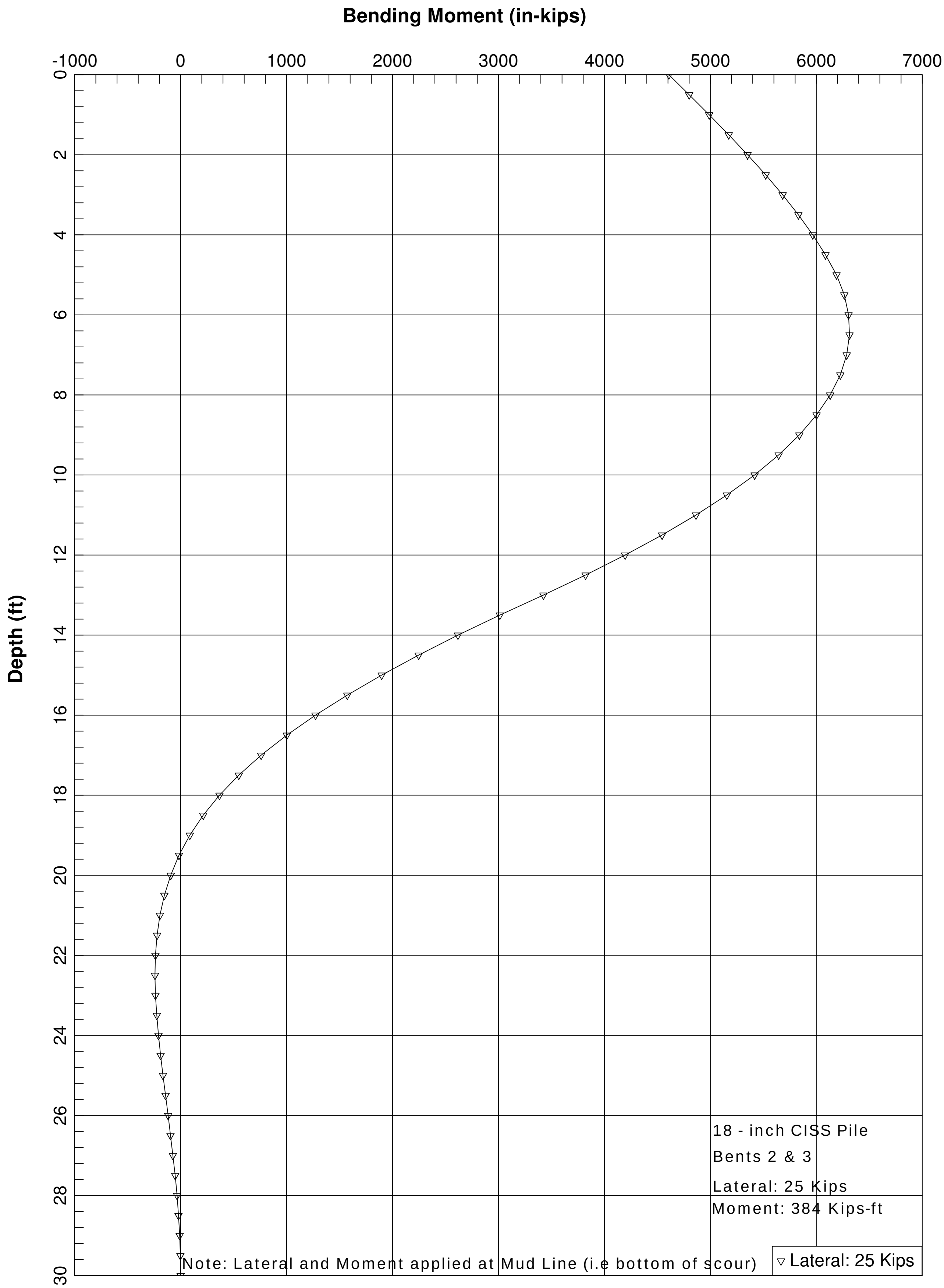
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
360.000	1.51387987	2427554.848	50000.000
342.000	1.56629896	2442059.571	50000.000

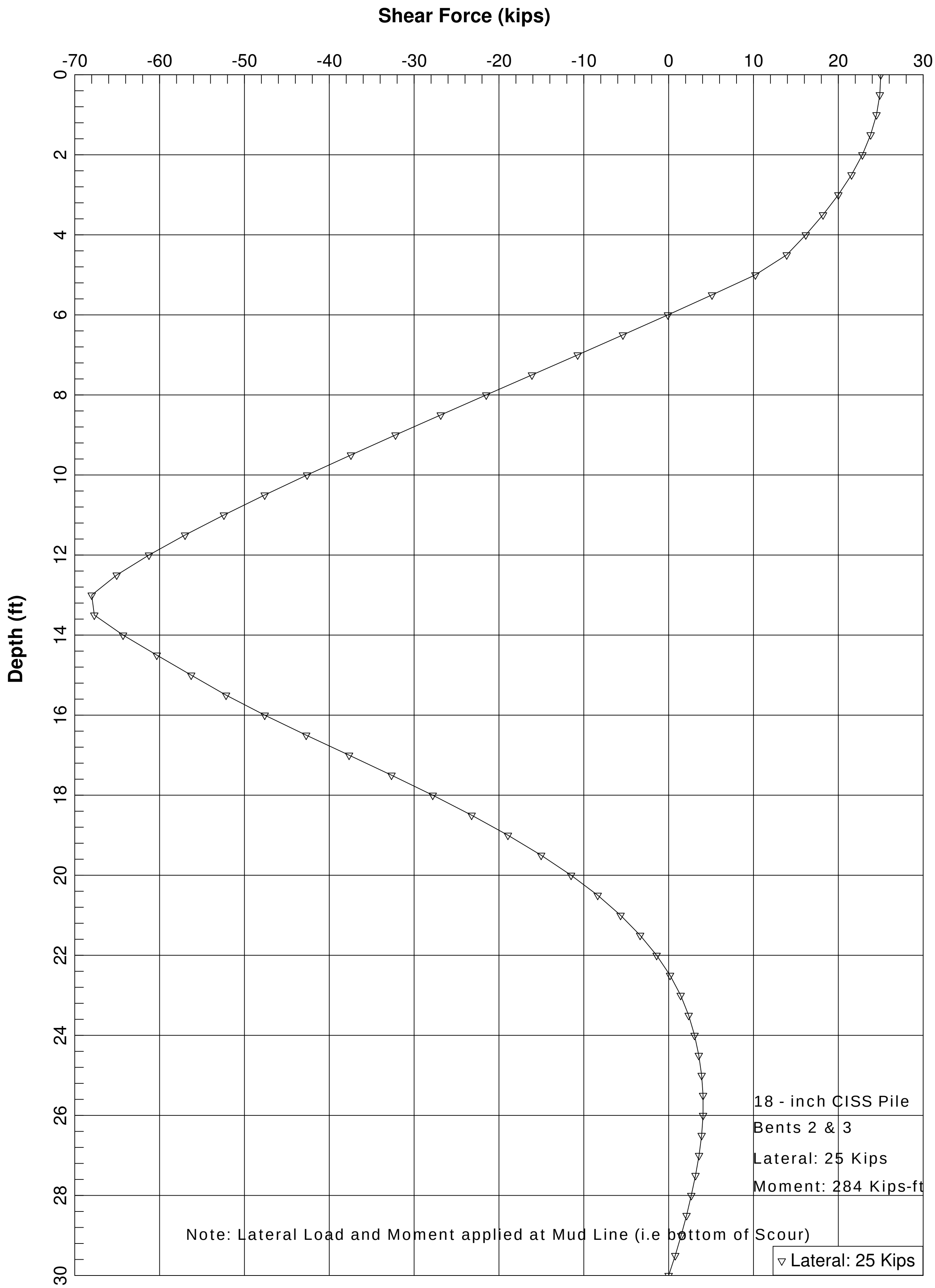
324.000	1.58054300	2437192.450	50000.000
306.000	1.57918199	2439178.524	50000.000
288.000	1.57364507	2437751.608	50000.000
270.000	1.57917037	2435911.049	50000.000
252.000	1.57901845	2438680.878	50000.000
234.000	1.59266184	2429681.271	50000.000
216.000	1.88283626	2332897.343	50000.000
198.000	2.18739576	2256929.586	50000.000

The analysis ended normally.

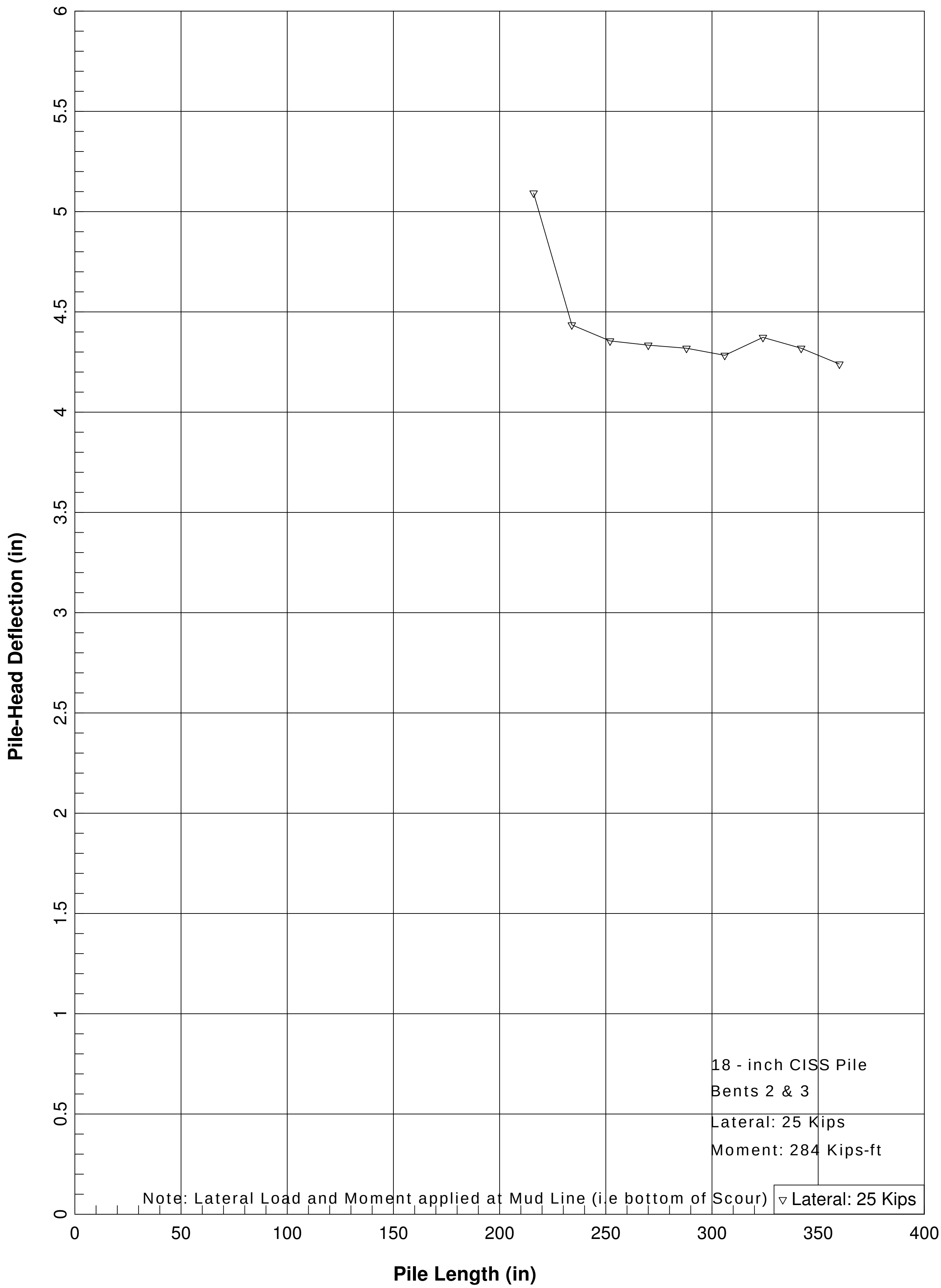


Lateral Deflection (in)
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Lateral: 25 Kips


Bending Moment (in-kips)
-1000
0
1000
2000
3000
4000
5000
6000
7000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Bents 2 & 3
Lateral: 25 Kips
Moment: 384 Kips-ft
Note: Lateral and Moment applied at Mud Line (i.e bottom of scour)
Lateral: 25 Kips



Shear Force (kips)
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
18 - inch CISS Pile
Bents 2 & 3
Lateral: 25 Kips
Moment: 284 Kips-ft
Note: Lateral Load and Moment applied at Mud Line (i.e bottom of Scour)
Lateral: 25 Kips



Pile-Head Deflection (in)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
0
50
100
150
200
250
300
350
400
Pile Length (in)
18 - inch CISS Pile
Bents 2 & 3
Lateral: 25 Kips
Moment: 284 Kips-ft
Note: Lateral Load and Moment applied at Mud Line (i.e bottom of Scour)
Lateral: 25 Kips

```
===============================================================================

                    LPILE Plus for Windows, Version 4.0

               Analysis of Individual Piles and Drilled Shafts
              Subjected to Lateral Loading Using the p-y Method

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                            All Rights Reserved

===============================================================================


This program is licensed to:

IT - San Jose
URS Corporation

Path to file locations:      X:\101-Sonoma\geotech\Analysis\Pile Design\Lateral\For
Bridge\New Folder\
Name of input data file:     Willow Brook Creek_Bent_18 in CISS-Load @ mud line.lpd
Name of output file:         Willow Brook Creek_Bent_18 in CISS-Load @ mud line.lpo
Name of plot output file:    Willow Brook Creek_Bent_18 in CISS-Load @ mud line.lpp
Name of runtime file:        Willow Brook Creek_Bent_18 in CISS-Load @ mud line.lpr


-------------------------------------------------------------------------------
                          Time and Date of Analysis
-------------------------------------------------------------------------------

               Date:  June  4, 2009     Time:  11:21:31


-------------------------------------------------------------------------------
                                Problem Title
-------------------------------------------------------------------------------

Sonoma 101 Segment B-Willow Brook - 18 in CISS-Loads @ mud line (scour)


-------------------------------------------------------------------------------
                               Program Options
-------------------------------------------------------------------------------

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 1:
- Computation of Lateral Pile Response Using User-specified Constant EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis includes automatic computation of pile-top deflection vs.
  pile embedment length
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- Additional p-y curves computed at specified depths

Solution Control Parameters:
```

- Number of pile increments = 60
- Maximum number of iterations allowed = 200
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 360.00 in
Depth of ground surface below top of pile = .00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	18.000	5152.9000	254.4000	3605000.000
2	360.0000	18.000	5152.9000	254.4000	3605000.000

Soil and Rock Layering Information

The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = .000 in
Distance from top of pile to bottom of layer = 60.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 2 is stiff clay without free water
Distance from top of pile to top of layer = 60.000 in
Distance from top of pile to bottom of layer = 180.000 in
p-y subgrade modulus k for top of soil layer = 625.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 625.000 lbs/in**3

Layer 3 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 180.000 in
Distance from top of pile to bottom of layer = 298.000 in
p-y subgrade modulus k for top of soil layer = 60.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 60.000 lbs/in**3

Layer 4 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 298.000 in
Distance from top of pile to bottom of layer = 442.000 in
p-y subgrade modulus k for top of soil layer = 100.000 lbs/in**3
p-y subgrade modulus k for bottom of layer = 100.000 lbs/in**3

Layer 5 is sand, p-y criteria by Reese et al., 1974
Distance from top of pile to top of layer = 442.000 in
Distance from top of pile to bottom of layer = 800.000 in
p-y subgrade modulus k for top of soil layer = 125.000 lbs/in**3

p-y subgrade modulus k for bottom of layer = 125.000 lbs/in**3

(Depth of lowest layer extends 440.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth
is defined using 10 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	.00	.03890
2	60.00	.03890
3	60.00	.03760
4	180.00	.03760
5	180.00	.04040
6	298.00	.04040
7	298.00	.04450
8	442.00	.04450
9	442.00	.04450
10	800.00	.04450

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 10 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	.000	.00000	32.00	------	------
2	60.000	.00000	32.00	------	------
3	60.000	17.36100	.00	.00600	.0
4	180.000	17.36100	.00	.00600	.0
5	180.000	.00000	32.00	------	------
6	298.000	.00000	32.00	------	------
7	298.000	.00000	36.00	------	------
8	442.000	.00000	36.00	------	------
9	442.000	.00000	38.00	------	------
10	800.000	.00000	38.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Static loading criteria was used for computation of p-y curves

---

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = 25000.000 lbs
Bending moment at pile head = 4608000.000 in-lbs
Axial load at pile head = 140000.000 lbs

Non-zero moment at pile head for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Output of p-y Curves at Specified Depths

---

p-y curves are generated and printed for verification at 1 depths.

Depth No.	Depth Below Pile Head in	Depth Below Ground Surface in
1	200.000	200.000

Depth of ground surface below top of pile = .00 in

p-y Curve in Sand Computed Using Reese Criteria

Soil Layer Number	=	3
Depth below pile head	=	200.000 in
Depth below ground surface	=	200.000 in
Equivalent Depth (see note)	=	208.818 in
Pile Diameter	=	18.000 in
Angle of Friction	=	32.000 deg.
Avg. Eff. Unit Weight	=	.03827 lbs/in**3
k	=	60.000 pci
A (static)	=	.8800
B (static)	=	.5000
Pst	=	4230.977 lbs/in
Psd	=	5071.938 lbs/in
Ps	=	4230.977 lbs/in
pu	=	3723.260 lbs/in
Cbar	=	4398.6661
n	=	1.6447
m	=	4287.3905
yk	=	.0692 in
ym	=	.3000 in
yu	=	.6750 in
p-multiplier	=	1.00000
y-multiplier	=	1.00000

If Psd <= Pst then actual depth is used in place of equivalent depth in computations.

y, in p, lbs/in

.0000	.000
.0250	313.227
.0500	626.454
.0750	910.664
.1000	1084.728
.1250	1242.345
.1500	1387.984
.1750	1524.362
.2000	1653.283
.2250	1776.020
.2500	1893.514
.2750	2006.482
.3000	2115.489
.6750	3723.260
18.6750	3723.260
36.6750	3723.260
54.6750	3723.260

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = 25000.000 lbs
Specified bending moment at pile head = 4608000.000 in-lbs
Specified axial load at pile head = 140000.000 lbs

Non-zero moment for this load case indicates the pile-head may rotate under the applied pile-head loading, but is not a free-head (zero moment)condition.

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Soil Res p lbs/in
0.000	4.240	4.608E+06	25000.0000	-.050734	8598.5980	0.0000
6.000	3.940	4.800E+06	24878.6540	-.049215	8933.9284	-40.4487
12.000	3.649	4.989E+06	24491.8727	-.047634	9264.4409	-88.4784
18.000	3.368	5.174E+06	23809.2888	-.045993	9587.0258	-139.0495
24.000	3.097	5.352E+06	22825.5022	-.044293	9898.4159	-188.8794
30.000	2.837	5.522E+06	21549.9554	-.042536	10195.3935	-236.3029
36.000	2.587	5.682E+06	20000.5238	-.040727	10474.8962	-280.1743
42.000	2.348	5.831E+06	18201.7074	-.038868	10734.0897	-319.4312
48.000	2.120	5.966E+06	16175.0017	-.036962	10970.4352	-356.1373
54.000	1.904	6.087E+06	13929.2895	-.035016	11181.5606	-392.4334
60.000	1.700	6.192E+06	10225.2403	-.033033	11365.1265	-842.2497
66.000	1.508	6.265E+06	5116.7023	-.031021	11492.7998	-860.5963
72.000	1.328	6.305E+06	-91.6936	-.028991	11563.3924	-875.5356
78.000	1.160	6.313E+06	-5379.0605	-.026953	11575.9458	-886.9200
84.000	1.004	6.286E+06	-10723.5686	-.024919	11529.7407	-894.5827
90.000	.861055	6.226E+06	-16102.3101	-.022898	11424.3078	-898.3311
96.000	.729700	6.131E+06	-21491.1182	-.020902	11259.4401	-897.9383
102.000	.610227	6.003E+06	-26864.3186	-.018943	11035.2069	-893.1285
108.000	.502388	5.841E+06	-32194.3715	-.017030	10751.9717	-883.5558
114.000	.405869	5.645E+06	-37451.3404	-.015175	10410.4130	-868.7672
120.000	.320290	5.417E+06	-42602.0646	-.013388	10011.5535	-848.1409

126.000	.245209	5.157E+06	-47608.7995	-.011681	9556.7983	-820.7708
132.000	.180121	4.865E+06	-52426.8160	-.010062	9047.9918	-785.2347
138.000	.124462	4.544E+06	-56999.7237	-.008543	8487.5063	-739.0678
144.000	.077610	4.196E+06	-61248.8973	-.007131	7878.3968	-677.3234
150.000	.038889	3.821E+06	-65042.3963	-.005836	7224.7108	-587.1763
156.000	.007573	3.425E+06	-68008.7634	-.004666	6532.2940	-401.6128
162.000	-.017105	3.013E+06	-67694.7056	-.003626	5813.0019	506.2987
168.000	-.035943	2.619E+06	-64296.1024	-.002717	5124.1167	626.5690
174.000	-.049707	2.246E+06	-60322.7928	-.001931	4473.3873	697.8675
180.000	-.059118	1.898E+06	-56287.9168	-.001262	3865.4736	647.0911
186.000	-.064850	1.573E+06	-52163.4218	-7.014E-04	3297.3477	727.7405
192.000	-.067535	1.273E+06	-47605.6347	-2.418E-04	2774.2348	791.5219
198.000	-.067752	1.002E+06	-42710.7435	1.257E-04	2300.2872	840.1085
204.000	-.066027	7.606E+05	-37661.1225	4.103E-04	1878.6885	843.0985
210.000	-.062828	5.493E+05	-32657.2177	6.219E-04	1509.7411	824.8698
216.000	-.058564	3.676E+05	-27812.6739	7.699E-04	1192.3988	789.9782
222.000	-.053588	2.143E+05	-23216.2887	8.639E-04	924.5540	742.1502
228.000	-.048197	87574.9946	-18935.3197	9.127E-04	703.2720	684.8395
234.000	-.042636	-14488.3282	-15017.2780	9.245E-04	575.6196	621.1744
240.000	-.037104	-94185.4398	-11491.9801	9.069E-04	714.8177	553.9248
246.000	-.031754	-1.539E+05	-8373.7586	8.668E-04	819.1420	485.4823
252.000	-.026702	-1.961E+05	-5663.7451	8.103E-04	892.8675	417.8555
258.000	-.022030	-2.232E+05	-3352.1501	7.426E-04	940.2265	352.6762
264.000	-.017791	-2.376E+05	-1420.4769	6.682E-04	965.3044	291.2149
270.000	-.014012	-2.414E+05	156.3812	5.908E-04	971.9590	234.4045
276.000	-.010701	-2.367E+05	1408.1998	5.136E-04	963.7604	182.8684
282.000	-.007849	-2.254E+05	2367.6645	4.390E-04	943.9514	136.9532
288.000	-.005433	-2.090E+05	3068.8125	3.688E-04	915.4244	96.7628
294.000	-.003423	-1.892E+05	3545.6828	3.045E-04	880.7141	62.1940
300.000	-.001779	-1.670E+05	3876.7315	2.470E-04	842.0036	48.1556
306.000	-4.59E-04	-1.431E+05	4059.3206	1.969E-04	800.1861	12.7074
312.000	5.83E-04	-1.186E+05	4047.9764	1.546E-04	757.5018	-16.4888
318.000	.001396	-94746.7017	3877.6064	1.202E-04	715.7980	-40.3012
324.000	.002026	-72294.5823	3577.6655	9.320E-05	676.5834	-59.6791
330.000	.002515	-51971.2963	3171.8275	7.313E-05	641.0870	-75.6002
336.000	.002903	-34355.5182	2677.9667	5.919E-05	610.3194	-89.0200
342.000	.003225	-19935.1397	2108.4332	5.043E-05	585.1330	-100.8245
348.000	.003508	-9139.0336	1470.6073	4.573E-05	566.2766	-111.7842
354.000	.003774	-2364.6787	767.7282	4.387E-05	554.4446	-122.5088
360.000	.004035	0.0000	0.0000	4.349E-05	550.3145	-133.4006

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

```
Pile-head deflection             =     4.23969737 in
Computed slope at pile head      =     -.05073413
Maximum bending moment           =    6312663.959 lbs-in
Maximum shear force              =     -68008.763 lbs
Depth of maximum bending moment  =         78.000 in
Depth of maximum shear force     =        156.000 in
Number of iterations             =             35
Number of zero deflection points =              2
```

Summary of Pile-head Response

Definition of symbols for pile-head boundary conditions:

y = pile-head displacment, in
M = pile-head moment, lbs-in
V = pile-head shear force, lbs
S = pile-head slope, radians
R = rotational stiffness of pile-head, in-lbs/rad

BC Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 25000.000	M= 4.61E+06	1.400E+05	4.2397	6.313E+06	-68008.7634

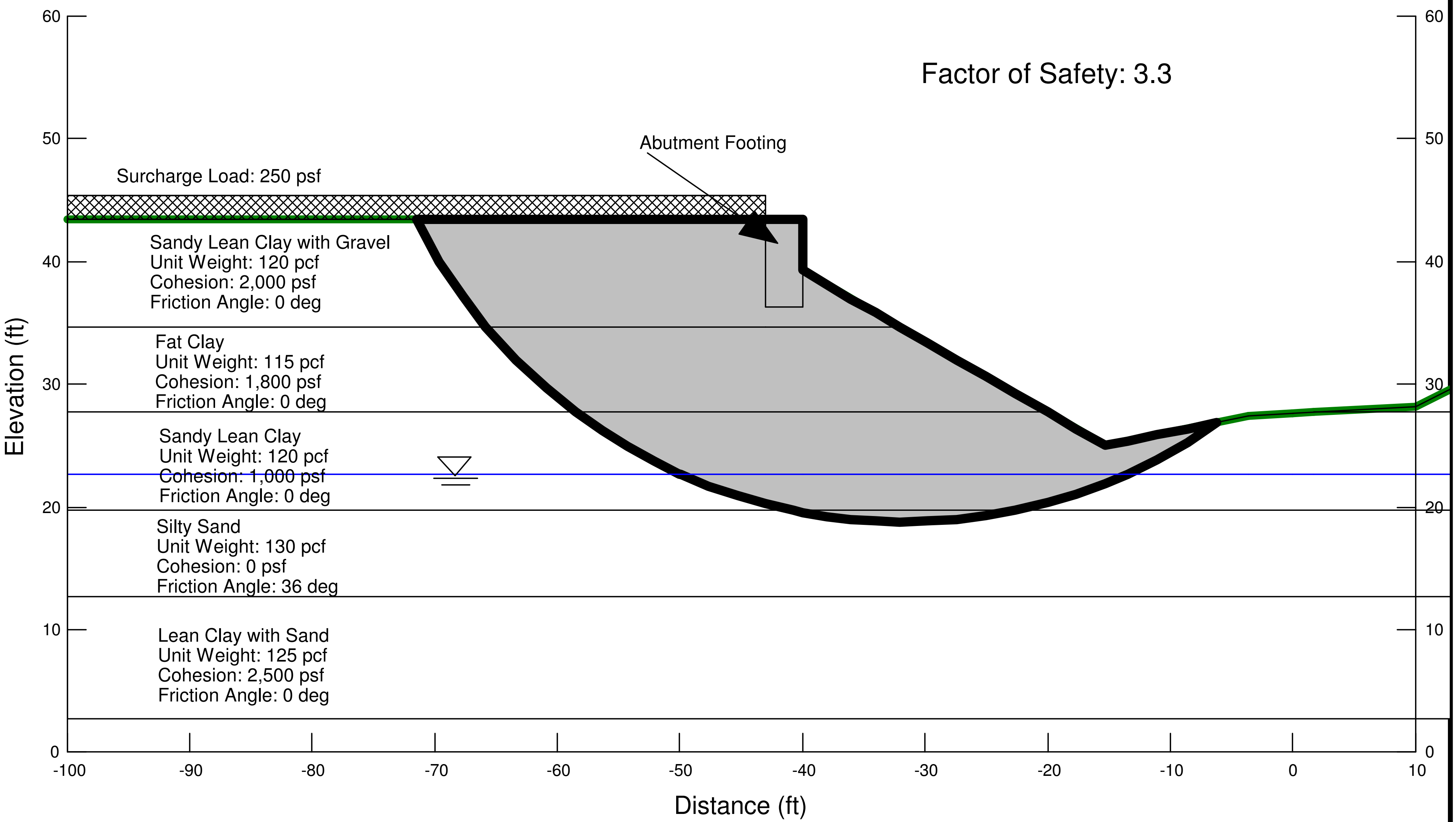
Pile-head Deflection vs. Pile Length

Boundary Condition Type 1, Shear and Moment

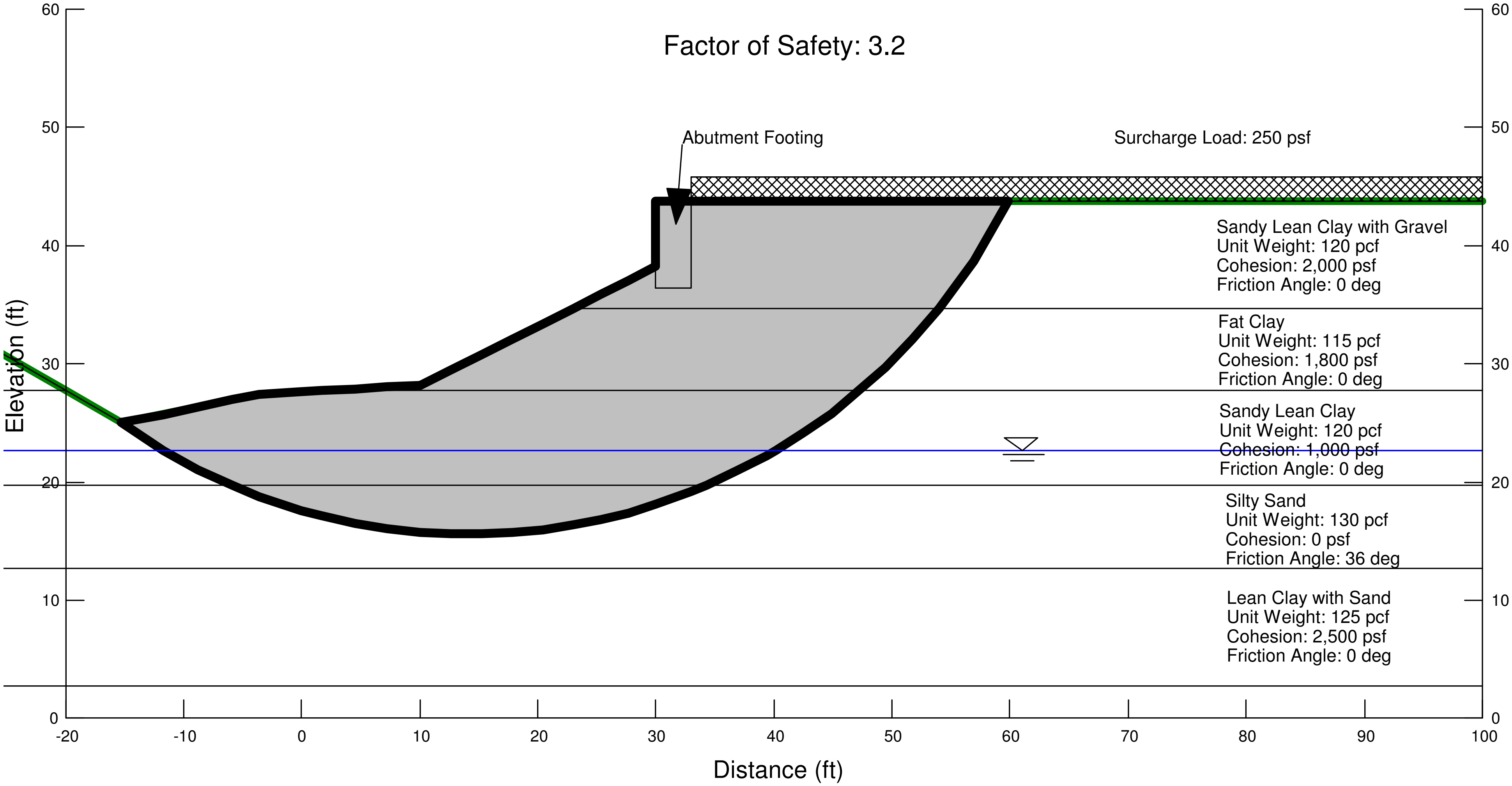
Shear = 25000. lbs
Moment = 4608000. in-lbs
Axial Load = 140000. lbs

Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
360.000	4.23969737	6312663.959	-68008.763
342.000	4.31921066	6336474.756	-68363.019
324.000	4.37272435	6354346.566	-68639.587
306.000	4.28361392	6327569.926	-68425.584
288.000	4.31857597	6338897.666	-68270.886
270.000	4.33366466	6344782.407	-68293.929
252.000	4.35566310	6344874.189	-68802.285
234.000	4.43479358	6347101.151	-71845.782
216.000	5.09346791	6374521.248	-80775.240

The analysis ended normally.

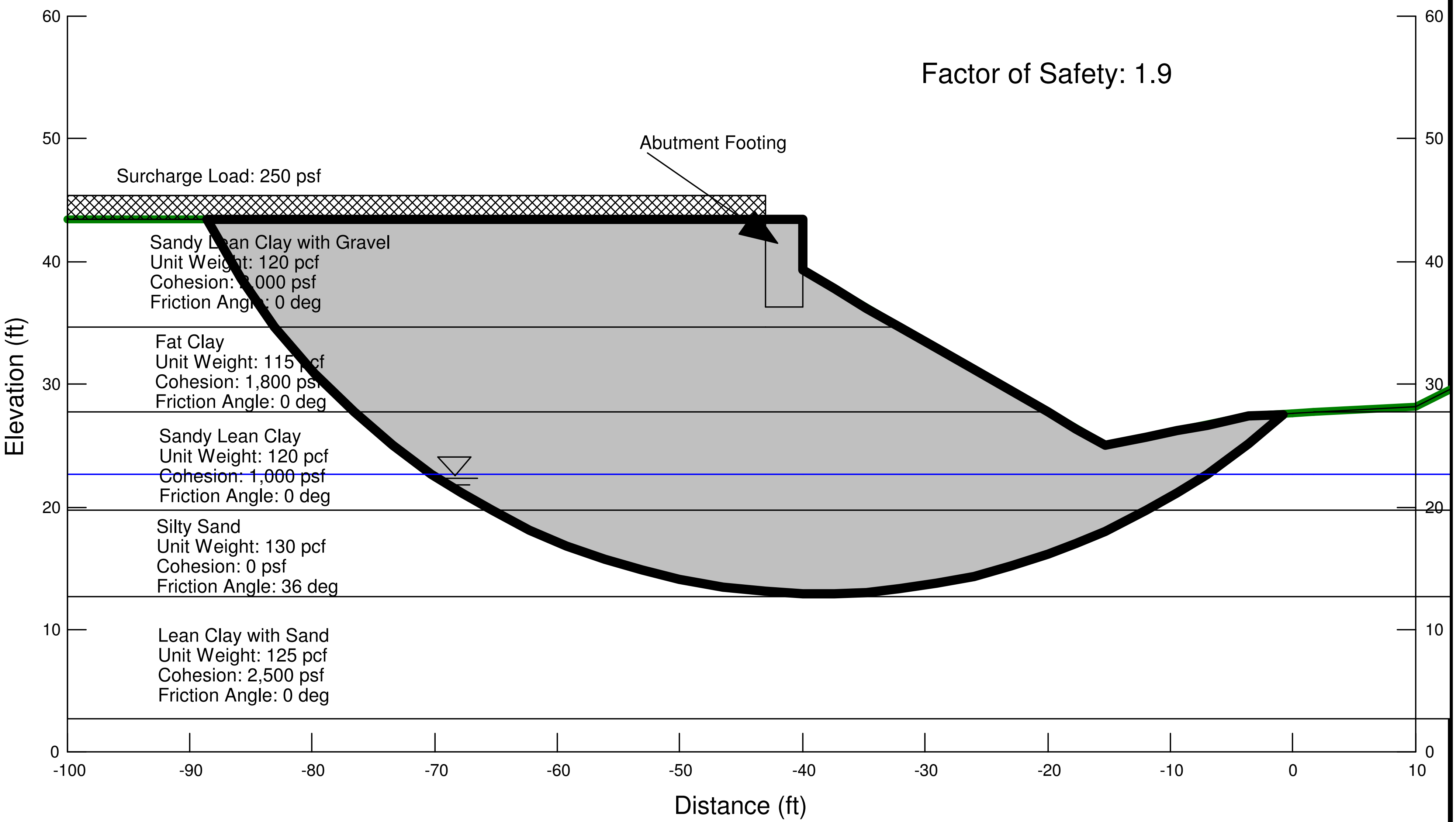


URS	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	101 SONOMA SEGMENT B SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING STATIC STABILITY ABUTMENT 1	FIGURE I-1
55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	Date May 4, 2009	CHECKED BY Michael Larson		

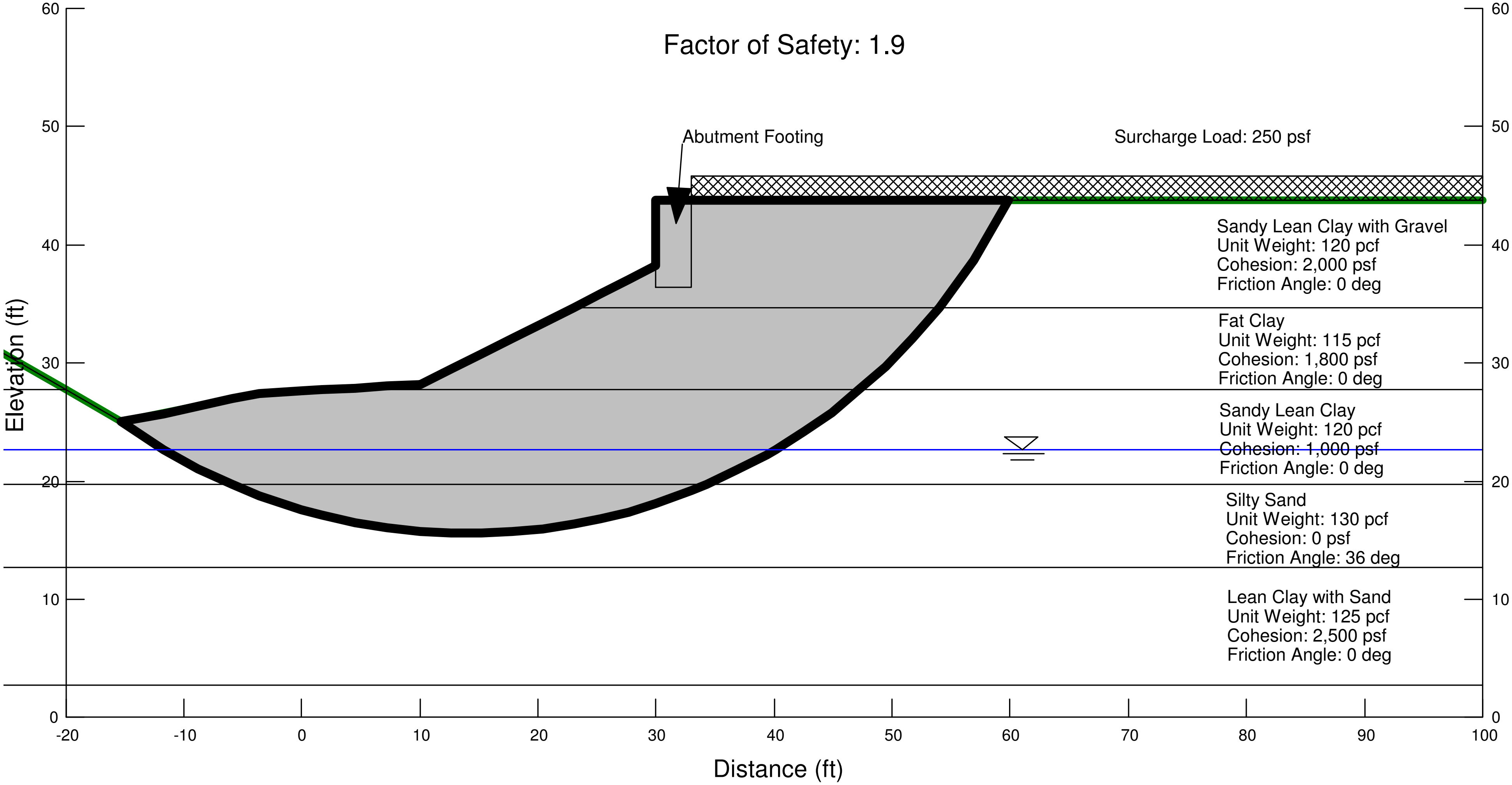


URS
55 South Market Street,
Suite 1500
San Jose, CA 95113
PHONE: (408) 297-9585
FAX: (408) 297-6962

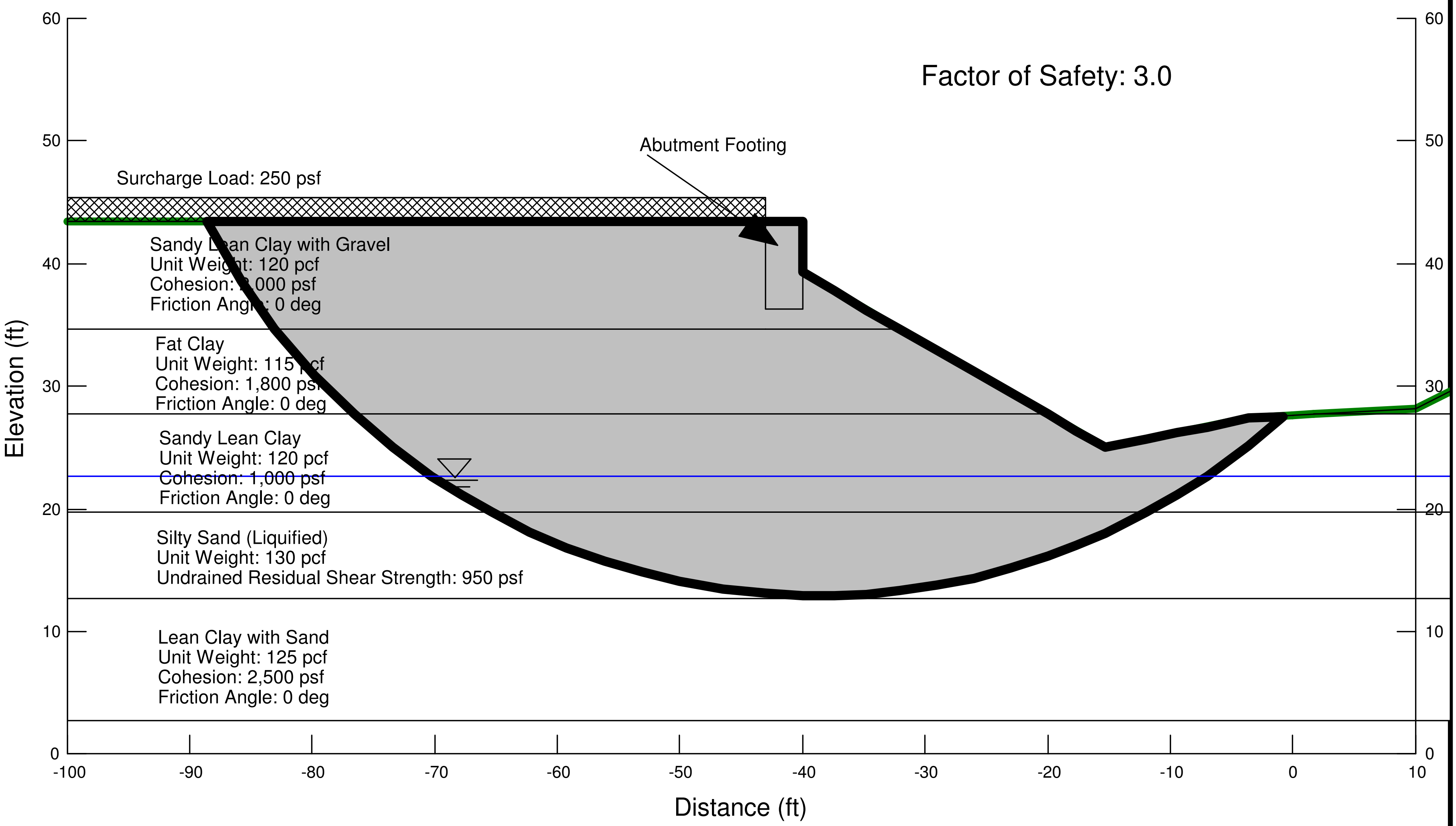
PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING STATIC STABILITY ABUTMENT 4	**FIGURE** **I-2**
Date May 4, 2009	CHECKED BY Michael Larson		



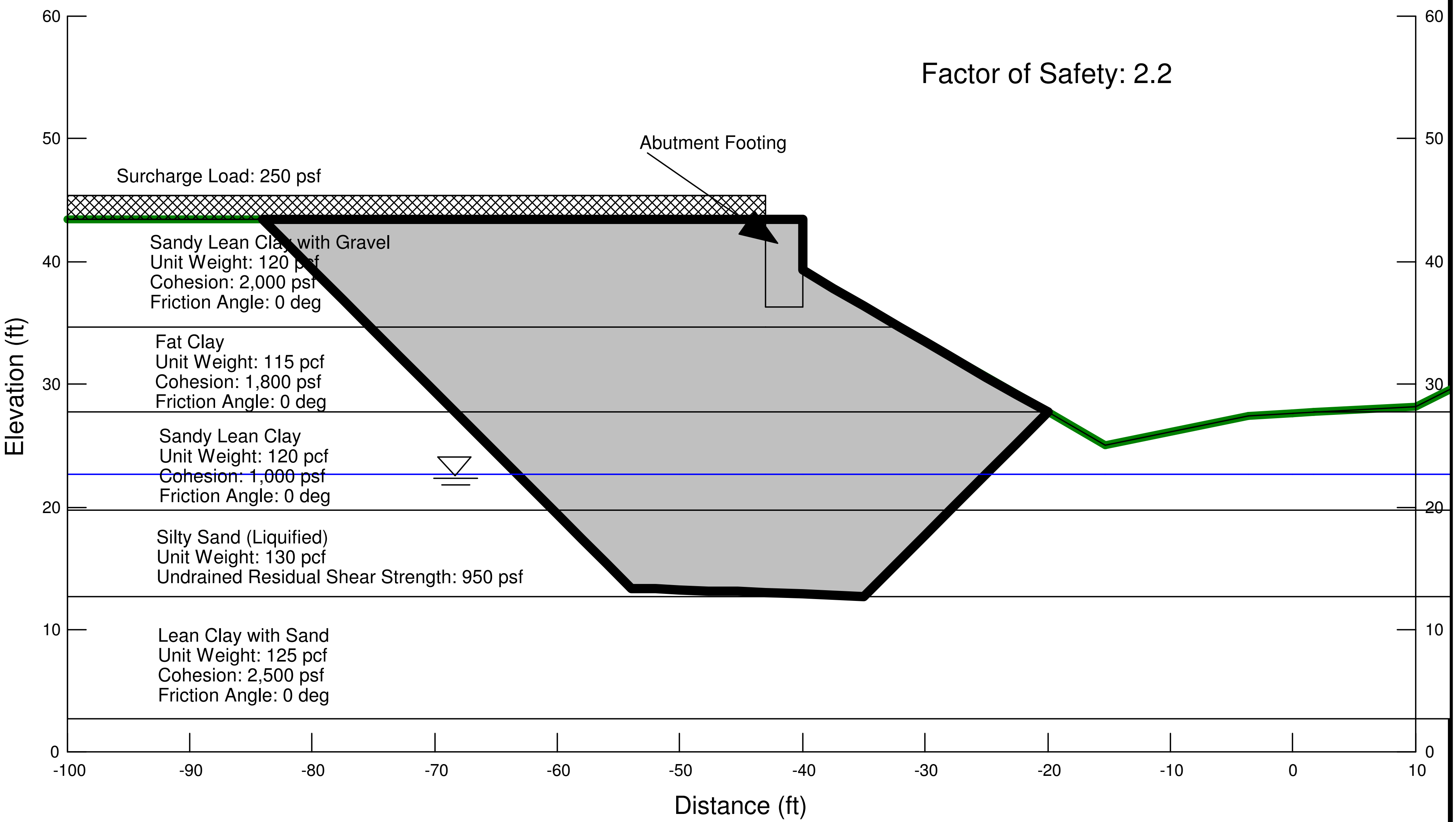
URS 55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING SEISMIC STABILITY ABUTMENT 1	**FIGURE** **I-3**
	Date May 4, 2009	CHECKED BY Michael Larson		



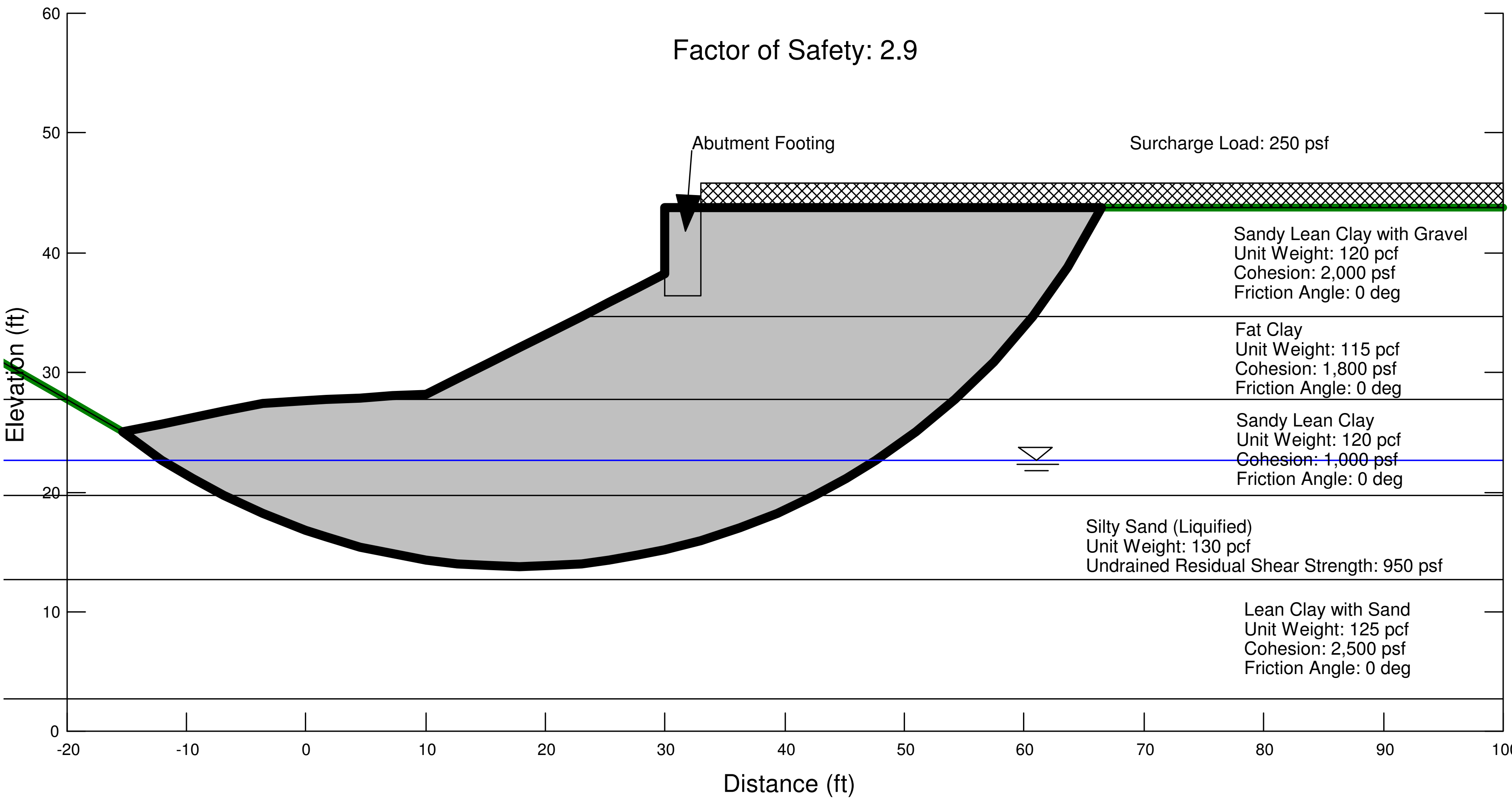
URS 55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING SEISMIC STABILITY ABUTMENT 4	**FIGURE** **I-4**
	Date May 4, 2009	CHECKED BY Michael Larson		



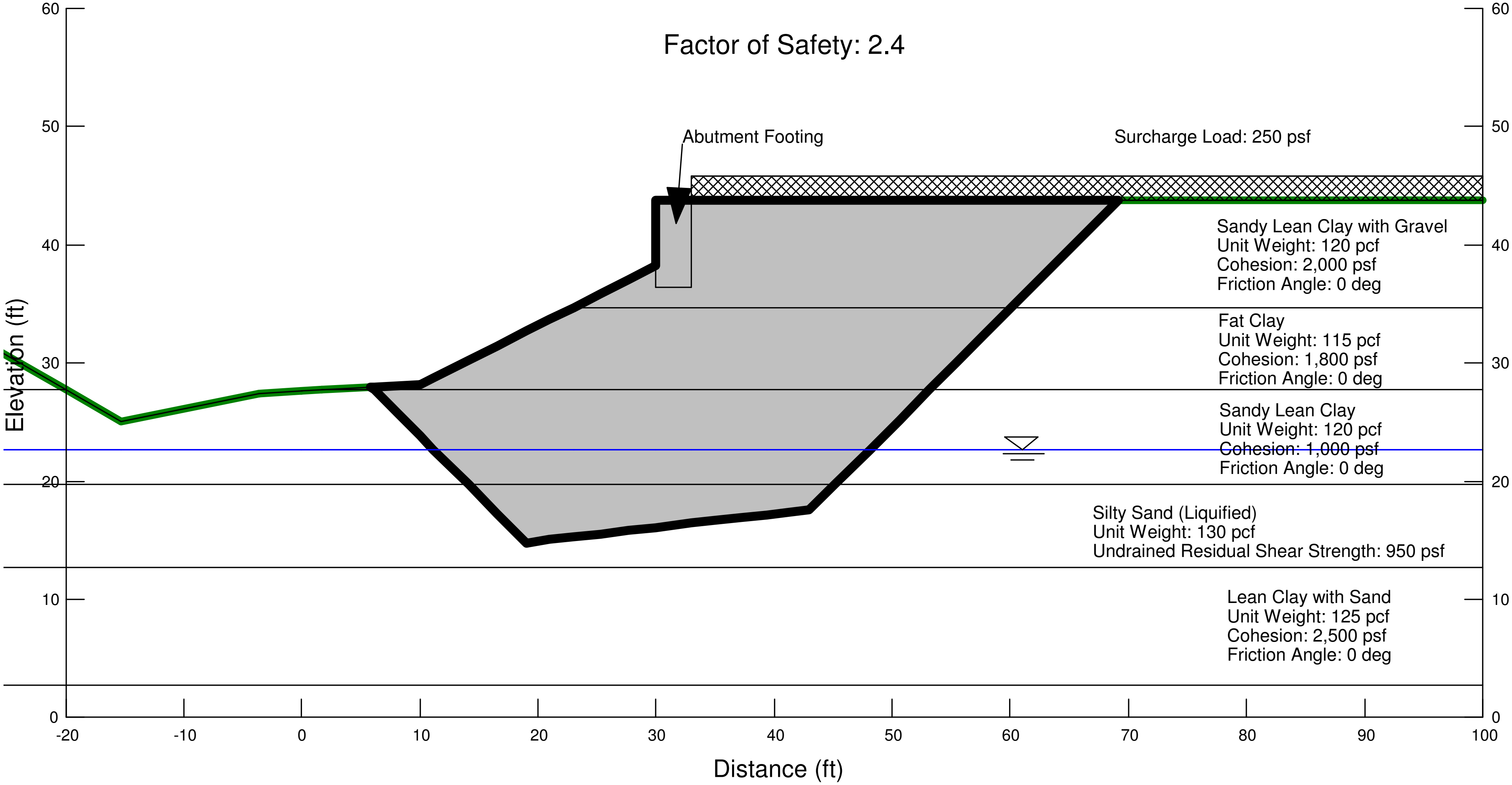
URS 55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	PROJECT No 28645047 Date May 4, 2009	CALCULATED BY Madhu Thummaluru CHECKED BY Michael Larson	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING POST EARTHQUAKE-CIRCULAR ABUTMENT 1	**FIGURE** **I-5**



URS 55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING POST EARTHQUAKE NON-CIRCULAR ABUTMENT 1	**FIGURE** **I-6**
	Date May 4, 2009	CHECKED BY Michael Larson		



URS 55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	**101 SONOMA SEGMENT B** SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING POST EARTHQUAKE-CIRCULAR ABUTMENT 4	**FIGURE** **I-7**
	Date May 4, 2009	CHECKED BY Michael Larson		



URS	PROJECT No 28645047	CALCULATED BY Madhu Thummaluru	101 SONOMA SEGMENT B SLOPE STABILITY ANALYSIS WILLOW BROOK BRIDGE WIDENING POST EARTHQUAKE NON-CIRCULAR ABUTMENT 4	FIGURE I-8
55 South Market Street, Suite 1500 San Jose, CA 95113 PHONE: (408) 297-9585 FAX: (408) 297-6962	Date May 4, 2009	CHECKED BY Michael Larson		

Supplemental Project Information

Storm Water Information Handout
04-SON-101, PM 7.1/8.9
EA: 04-0A1841
August 19, 2010

CONTENTS:
1. Notice of Intent
2. Site Maps
    a. Location Map
    b. Vicinity Map
3. Sampling Locations on Drainage Plans
4. Rainfall Data*
    a. IDF curves
    b. Runoff Coefficient
5. Shed map
6. Risk Level Assessment
7. 401 permit

Disclaimer
A "Disclaimer" is required specifying that the information provided in the Storm Water Information Handout is just a guideline and is to be used for information purposes only and should not be considered a sole source document to adhere to the requirements of the new National Pollutant Discharge Elimination System (NPDES) Construction General Permit (CGP), Number CAS000002, adopted on September 2, 2009. The contractor is required to provide water quality monitoring, sampling and implement best management practices (BMPs) based on standard industry operations, field conditions and conditions encountered based on the contractor's means and methods. The information in this handout is not to be construed in any way as a waiver of the provisions in the CGP. Bidders and contractors are cautioned to make independent investigations and examinations as they deem necessary to satisfy the conditions encountered in performance of work, with respect to the following: sampling and monitoring locations, distribution of watershed areas for sizing of BMPs, and selection of BMPs in order to conform to the requirement of the contract documents and the CGP.

*Rain fall Data:
a. Refer to Construction General Permit for Intensities
b. Refer to Highway Design Manual for Runoff Coefficients

Attachment 2

State Water Resources Control Board

NOTICE OF INTENT

TO COMPLY WITH THE TERMS OF THE
GENERAL PERMIT TO DISCHARGE STORM WATER
ASSOCIATED WITH CONSTRUCTION ACTIVITY (WQ ORDER No. 99-08-DWQ)

I. NOI STATUS (SEE INSTRUCTIONS)

MARK ONLY ONE ITEM 1. [X] New Construction 2. [ ] Change of Information for WDID#

II. PROPERTY OWNER

Name: California Dept of Transportation	Contact Person: Eric Schen		
Mailing Address: 111 Grand Ave	Title: Project Manager		
City: Oakland	State: CA	Zip: 94612	Phone: 510.286.4785

Owner Type (*check one*) 1.[ ] Private Individual 2.[ ]Business 3.[ ]Municipal 4.[X]State 5.[ ]Federal 6.[ ]Other

III. DEVELOPER/CONTRACTOR INFORMATION

Developer/Contractor	Contact Person		
Mailing Address	Title		
City	State	Zip	Phone

IV. CONSTRUCTION PROJECT INFORMATION

Site/Project Name: Sonoma 101 Central HOV lanes - SegB.	Site Contact Person			
Physical Address/Location: Sonoma County - Hwy 101 - PM 7.1/8.9	Latitude: 38.2633°	Longitude: 122.6585	County: Sonoma	
City (or nearest City): Petaluma	Zip	Site Phone Number	Emergency Phone Number	

A. Total size of construction site area: ________ Acres

B. Total area to be disturbed: 19.97 Acres (% of total ____)

C. Percent of site imperviousness (including rooftops):
Before Construction: ______%
After Construction: ______%

D. Tract Number(s): ________, ________

E. Mile Post Marker: 7.1/8.9

F. Is the construction site part of a larger common plan of development or sale?
[ ] YES [X] NO

G. Name of plan or development:

H. Construction commencement date: 12/01/2010

I. % of site to be mass graded: ________

J. Projected construction dates:
Complete grading: ___/___/___ Complete project: 12/31/2012

K. Type of Construction (Check all that apply):
1. [ ] Residential 2. [ ] Commercial 3. [ ] Industrial 4. [ ] Reconstruction 5. [X] Transportation
6. [ ] Utility Description: ________________ 7. [ ] Other (Please List): ________________

V. BILLING INFORMATION

SEND BILL TO: [ ] OWNER (as in II. above)	Name	Contact Person	
[ ] DEVELOPER (as in III. above)	Mailing Address	Phone/Fax	
[ ] OTHER (enter information at right)	City	State	Zip

VI. REGULATORY STATUS

A. Has a local agency approved a required erosion/sediment control plan? ☒ YES ☐ NO

Does the erosion/sediment control plan address construction activities such as infrastructure and structures? ☒ YES ☐ NO

Name of local agency: Caltrans Phone: 510.286.4785

B. Is this project or any part thereof, subject to conditions imposed under a CWA Section 404 permit of 401 Water Quality Certification? ☐ YES ☐ No

If yes, provide details: ______

VII. RECEIVING WATER INFORMATION

A. Does the storm water runoff from the construction site discharge to (Check all that apply):

1. ☐ **Indirectly to waters of the U.S.**
2. ☐ **Storm drain system - Enter owner's name:** ______
3. ☐ **Directly to waters of U.S. (e.g. , river, lake, creek, stream, bay, ocean, etc.)**

B. Name of receiving water: (river, lake, creek, stream, bay, ocean): Petaluma Creek and Willow Brook Creek

VIII. IMPLEMENTATION OF NPDES PERMIT REQUIREMENTS

A. STORM WATER POLLUTION PREVENTION PLAN (SWPPP) (check one)

☐ A SWPPP has been prepared for this facility and is available for review: Date Prepared: ___/___/___ Date Amended: ___/___/___

☐ A SWPPP will be prepared and ready for review by (enter date): ___/___/___

☐ A tentative schedule has been included in the SWPPP for activities such as grading, street construction, home construction, etc.

B. MONITORING PROGRAM

☐ A monitoring and maintenance schedule has been developed that includes inspection of the construction BMPs before anticipated storm events and after actual storm events and is available for review.

If checked above: A qualified person has been assigned responsibility for pre-storm and post-storm BMP inspections to identify effectiveness and necessary repairs or design changes ☐ YES ☐ NO

Name: ______ Phone: ______

C. PERMIT COMPLIANCE RESPONSIBILITY

A qualified person has been assigned responsibility to ensure full compliance with the Permit, and to implement all elements of the Storm Water Pollution Prevention Plan including:

1. Preparing an annual compliance evaluation ☐ YES ☐ NO

Name: ______ Phone: ______

2. Eliminating all unauthorized discharges ☐ YES ☐ NO

IX. VICINITY MAP AND FEE (must show site location in relation to nearest named streets, intersections, etc.)

Have you included a vicinity map with this submittal? ☐ YES ☐ NO

Have you included payment of the annual fee with this submittal? ☐ YES ☐ NO

X. CERTIFICATIONS

"I certify under penalty of law that this document and all attachments were prepared under my direction and supervision in accordance with a system designed to assure that qualified personnel properly gather and evaluate the information submitted. Based on my inquiry of the person or persons who manage the system, or those persons directly responsible for gathering the information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine or imprisonment. In addition, I certify that I have read the entire General Permit, including all attachments, and agree to comply with and be bound by all of the provisions, requirements, and prohibitions of the permit, including the development and implementation of a Storm Water Pollution Prevention Plan and a Monitoring Program Plan will be complied with."

Printed Name: ______

Signature: ______ **Date:** ______

Title: ______

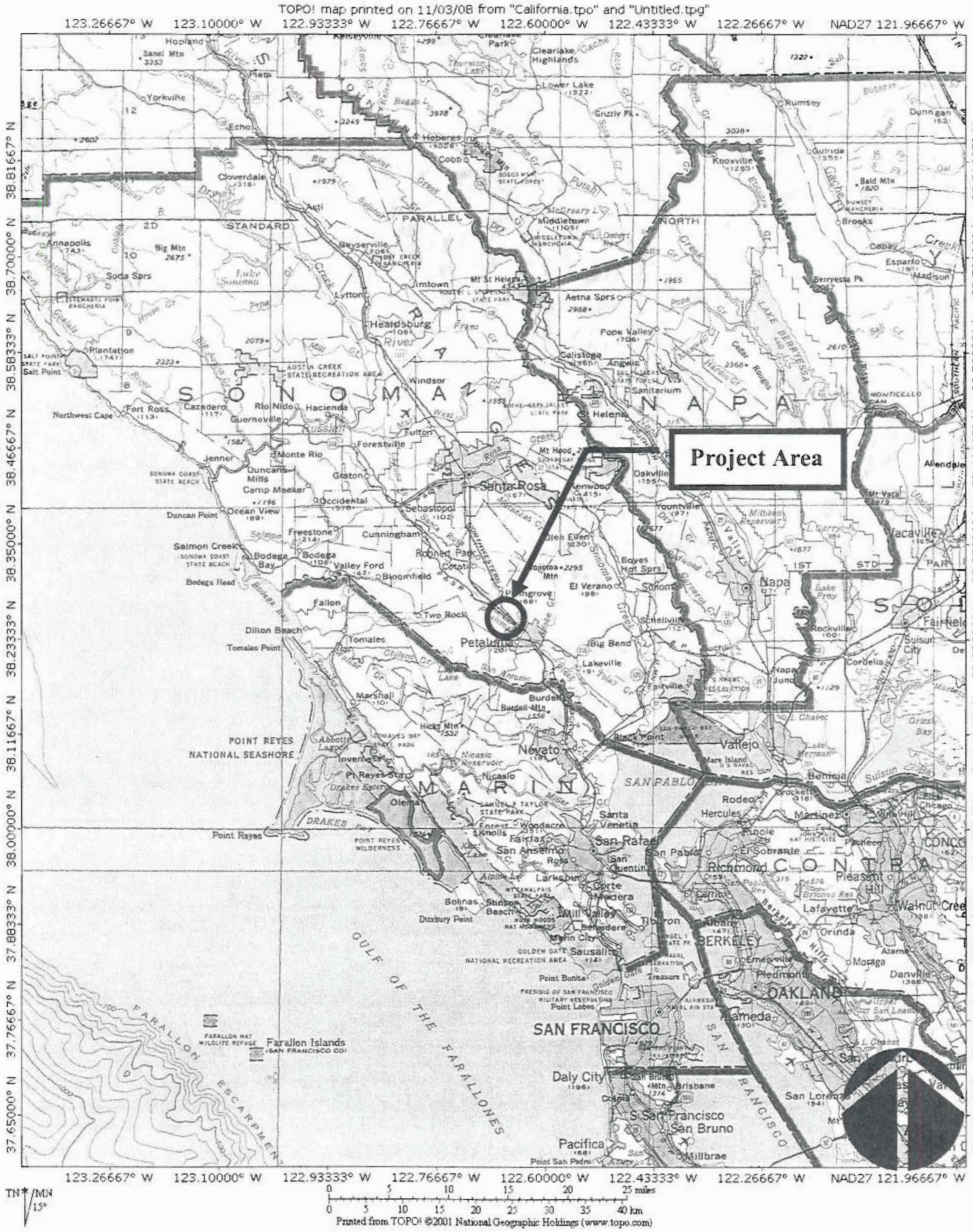


Figure 1. Location Map

Source: USGS

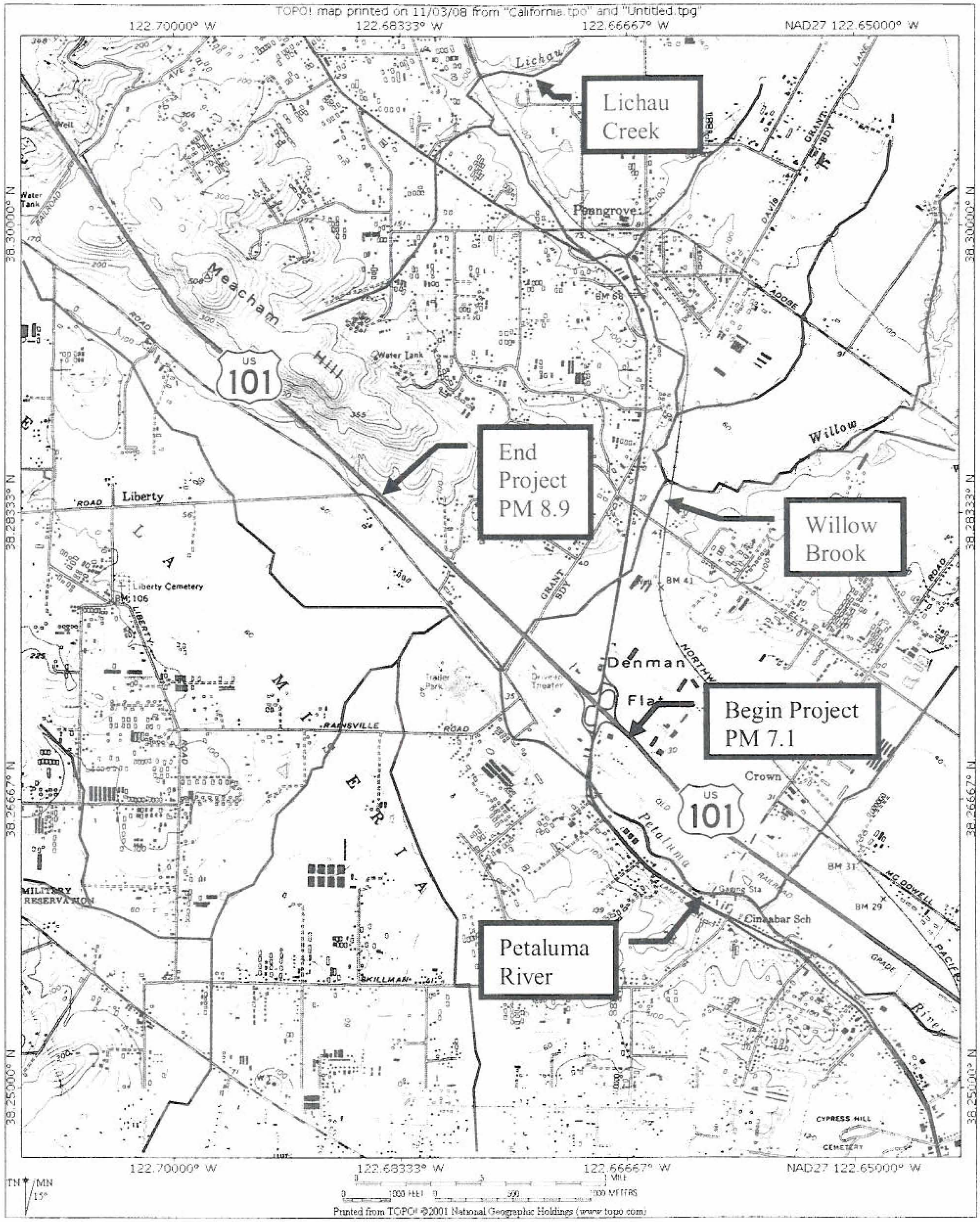
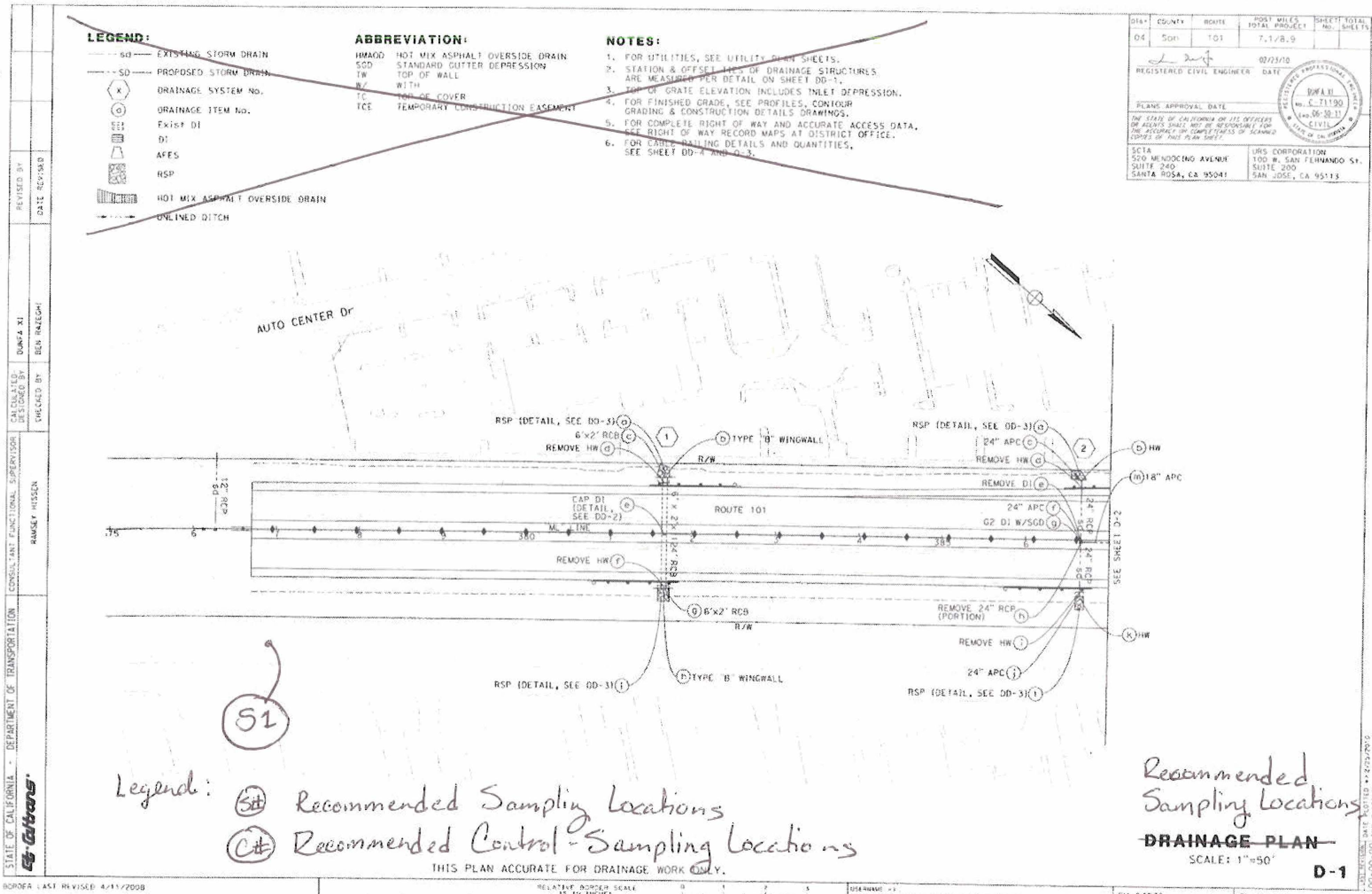


Figure 2. Vicinity Map

Source: USGS



Note: 1. Sample at lowest point.

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER 02/23/10 DATE

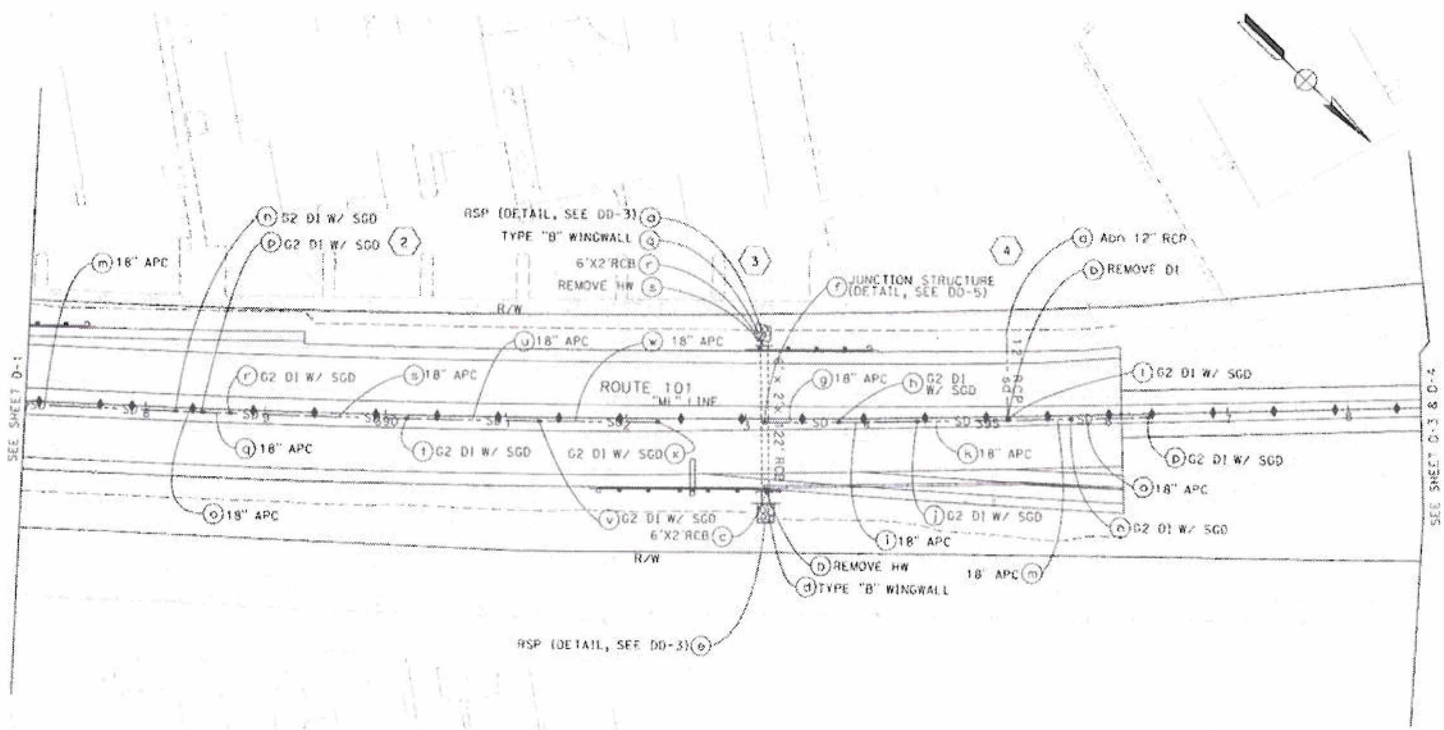
PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

SCTA
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95041

URS CORPORATION
100 W. SAN FERNANDO St.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-2

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY		REVISED BY
RAMSEY HISSEN	CHECKED BY	DUNFA XI / BEN RAZEGHI	DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => DGN FILE => ...\EA-Drainage40a1841d002.dgn

CU 04276 EA 0A1841

DATE PLOTTED => 2/25/2010 TIME PLOTTED => 9:10:03 AM

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER 02/23/10 DATE

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

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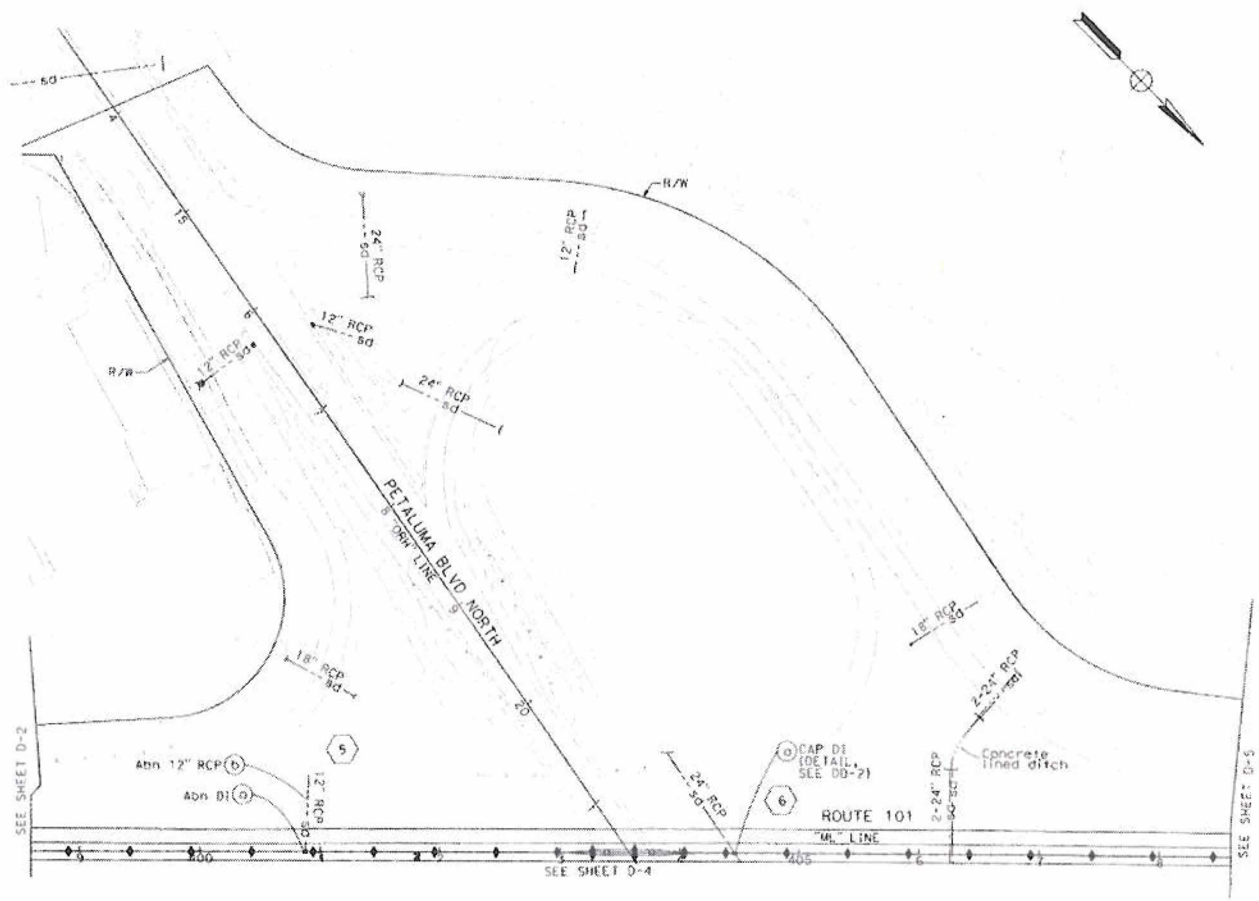
URS CORPORATION
100 W. SAN FERNANDO ST.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-3

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: RAMSEY HISSEN

CALCULATED-DESIGNED BY: DUNFA XI

CHECKED BY: BEN RAZEGHI

REVISED BY

DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

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CU 04276

EA 0A1841

DATE PLOTTED => 2/25/2010 TIME PLOTTED => 9:13:04 AM

LAST REVISION 00-00-00

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER 07/23/10 DATE

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

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SANTA ROSA, CA 95041

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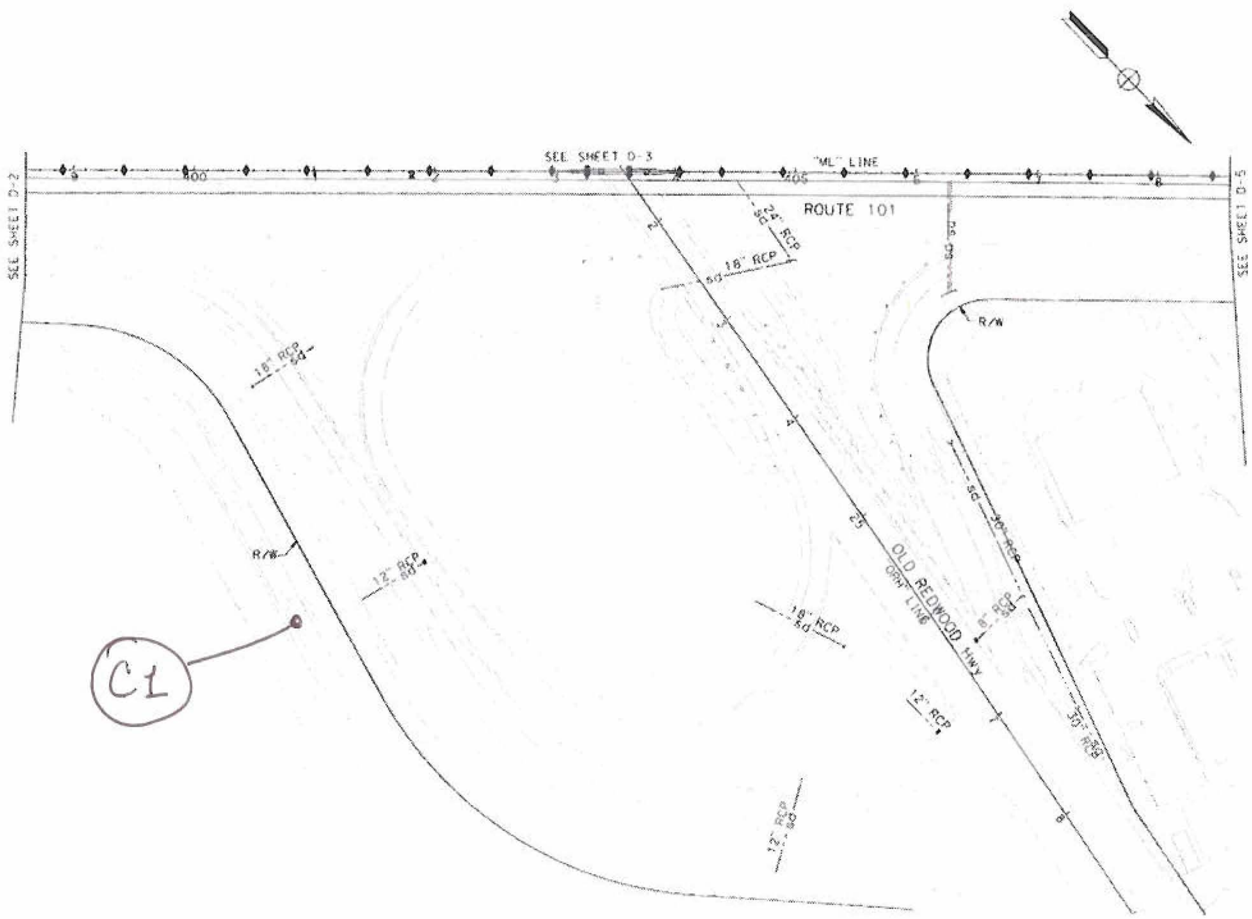
100 W. SAN FERNANDO St.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-4

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: RAMSEY HISSEN

CALCULATED-DESIGNED BY: DUNFA XI

CHECKED BY: BEN RAZEGHI

REVISED BY

DATE REVISED

BORDER LAST REVISED 4/11/2006

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

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CU 04276

EA 0A1841

DATE PLOTTED => 7/25/2010
TIME PLOTTED => 9:10:05 AM

00-00-00

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE 02/23/10

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER DUNFA KI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

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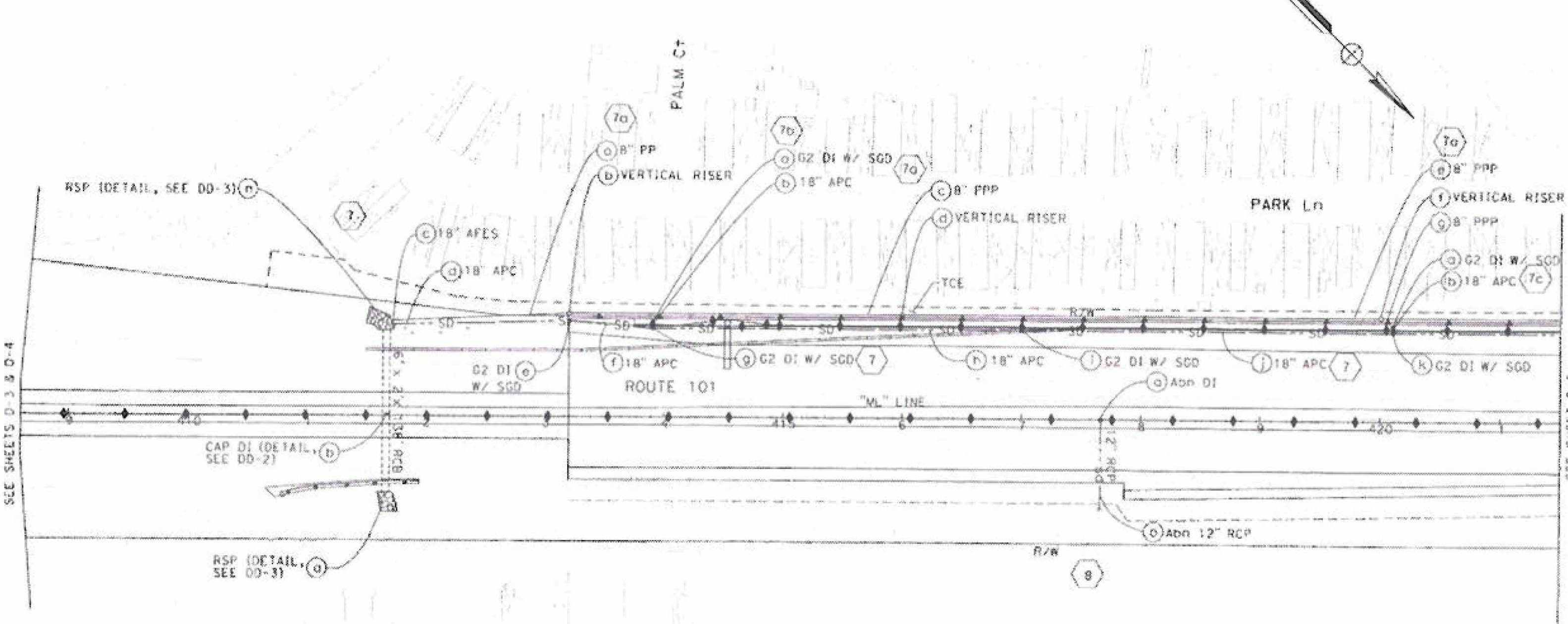
SANTA ROSA, CA 95041

URS CORPORATION
100 W. SAN FERNANDO ST.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-5

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	DUNFA KI	REVISED BY
Caltrans	RAMSEY HISSEN	CHECKED BY	BEN RAZEGHI	DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => ...
DGN FILE => ...

CU 04276

EA 0A1841

DATE PLOTTED => 2/25/2010 TIME PLOTTED => 9:10:06 AM

00-00-00

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE 02/23/10

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

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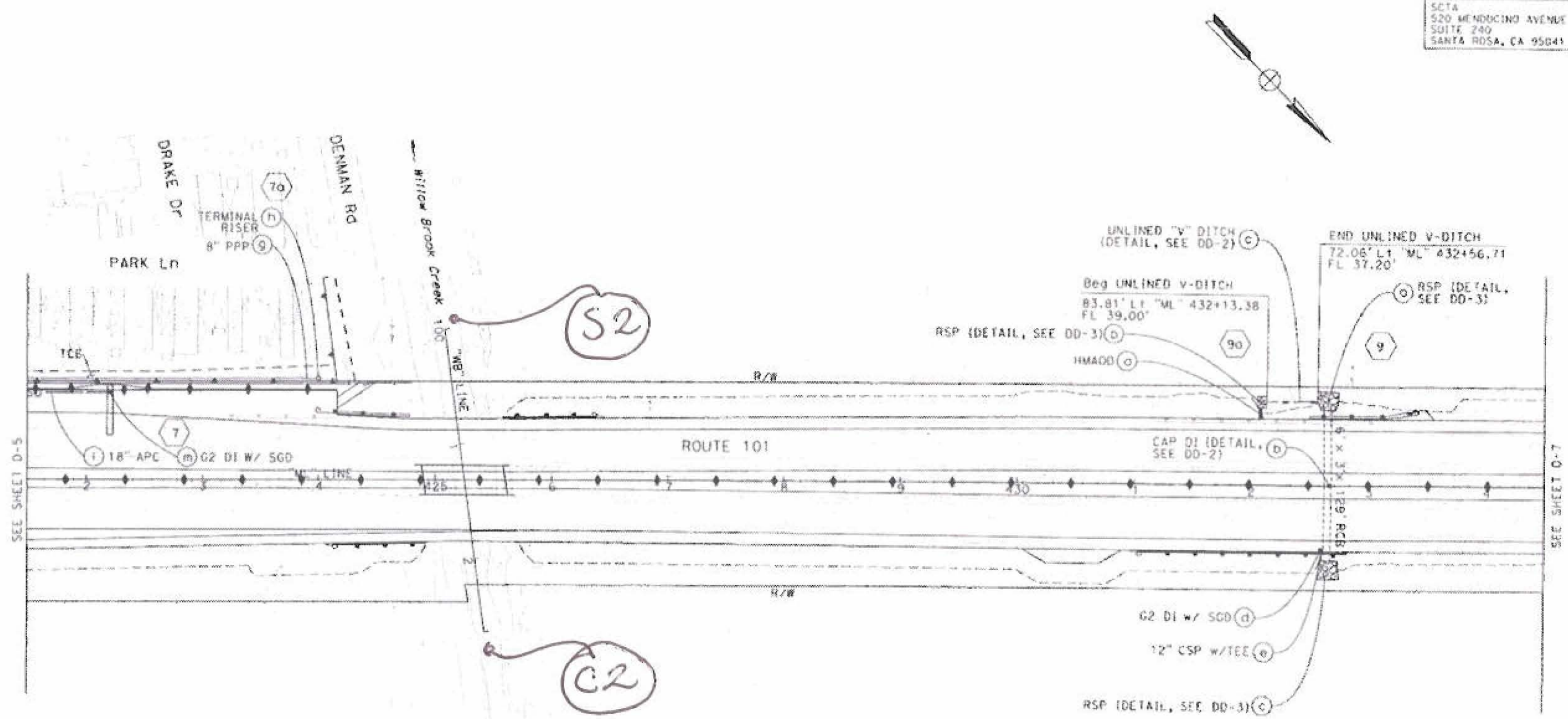
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95041

URS CORPORATION
100 W. SAN FERNANDO St.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-6

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	DUNFA XI	REVISED BY
Caltrans	RAMSEY NISSEN	CHECKED BY	BEN RAZEGHI	DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME =>
DGN FILE => ...sta-Drainage\40a1841d006.dgn

CU 04276

EA 0A1841

DATE PLOTTED => 2/25/2010
TIME PLOTTED => 9:10:07 AM

LAST REVISION 00-00-00

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER 02/23/10 DATE

PLANS APPROVAL DATE

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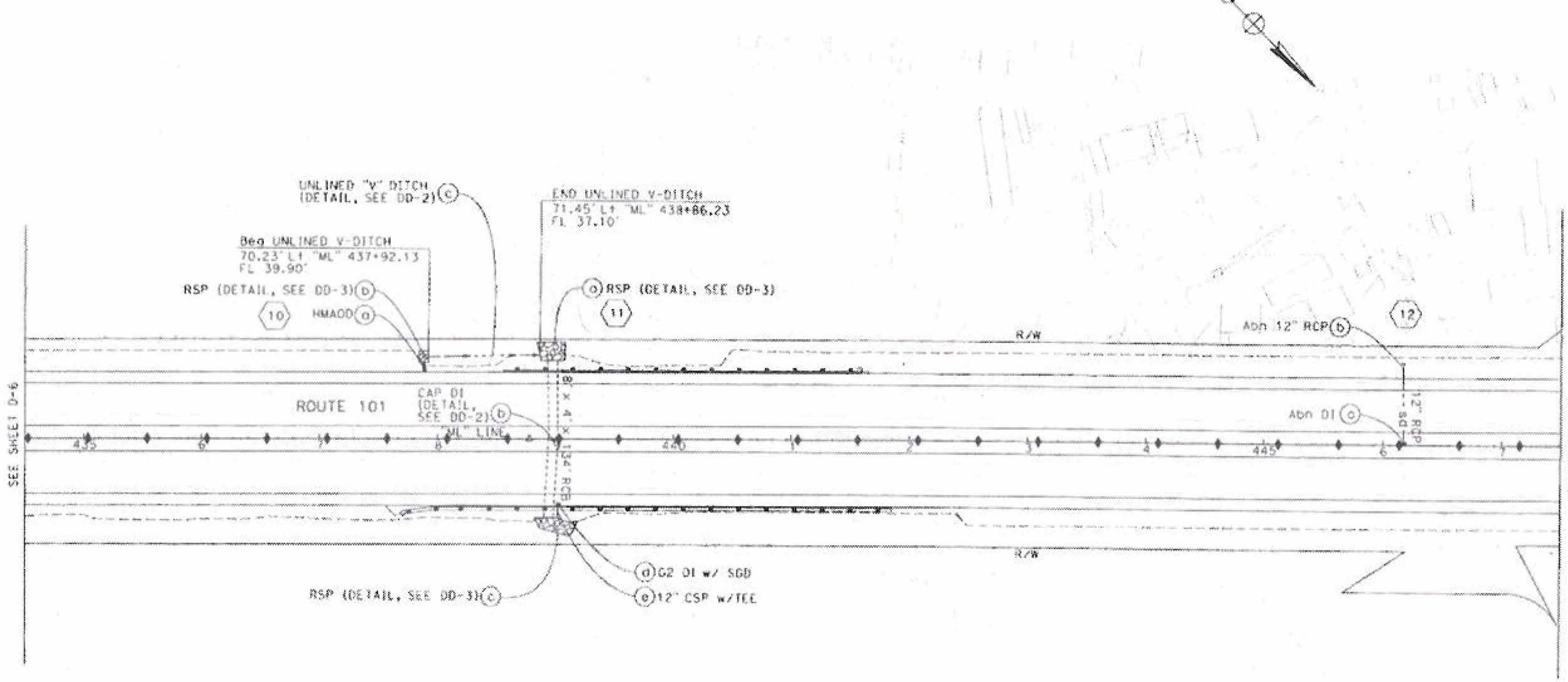
SUITE 240
SANTA ROSA, CA 95041

URS CORPORATION
100 W. SAN FERNANDO ST.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-7

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	DUNFA XI	REVISED BY
RAMSEY HISSEN	CHECKED BY	BEN RAZECHI	DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

CU 04276

EA 0A1841

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER — DATE 02/23/10

PLANS APPROVAL DATE

DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL — STATE OF CALIFORNIA — REGISTERED PROFESSIONAL ENGINEER

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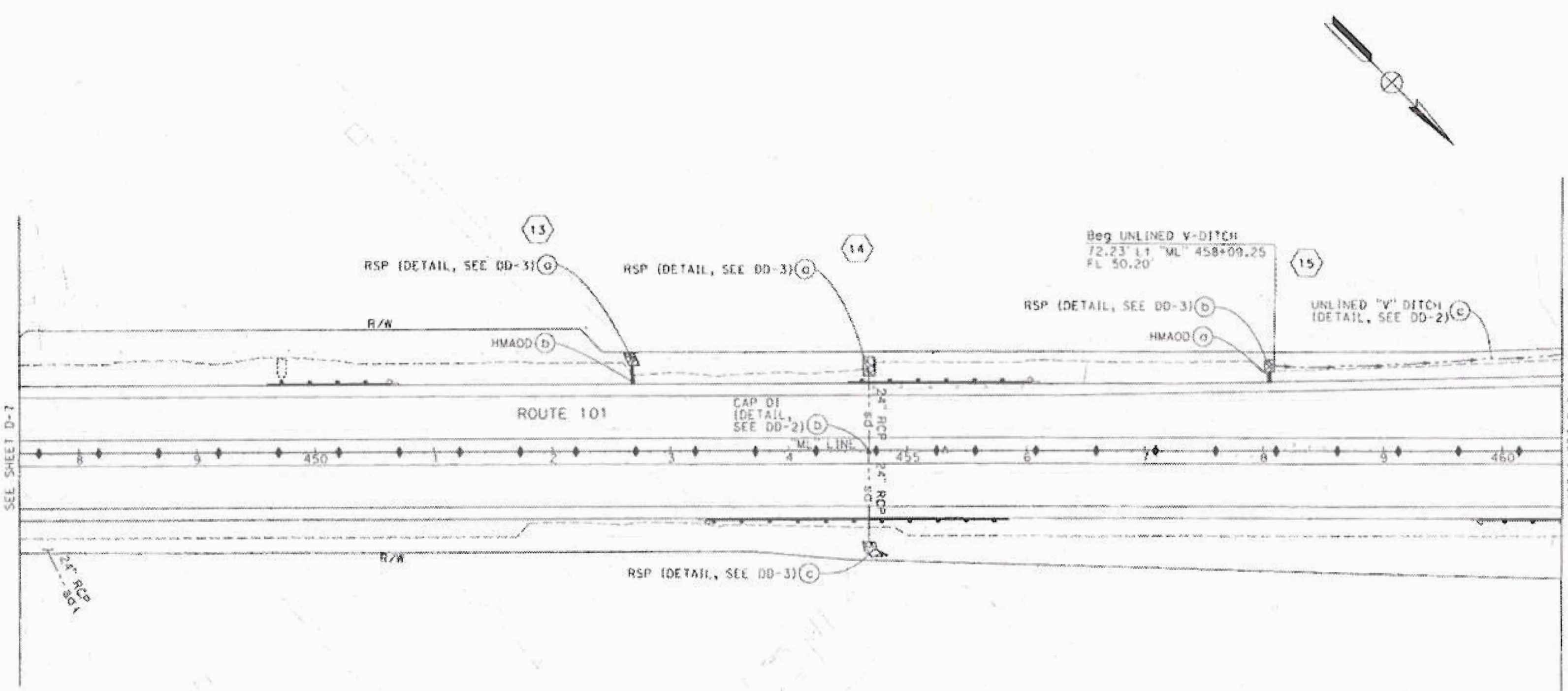
SUITE 240
SANTA ROSA, CA 95041

URS CORPORATION
100 W. SAN FERNANDO ST.
SUITE 200
SAN JOSE, CA 95113

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-8

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION — Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: RAMSEY HISSEN

CALCULATED-DESIGNED BY: DUNFA XI

CHECKED BY: BEN RAZEGHI

REVISED BY / DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => DGN FILE => ...\EA-Drainage\dad1841e008.dgn

CU 04226

EA 0A1841

DATE PLOTTED => 2/25/2010 TIME PLOTTED => 9:10:09 AM

NOTE: FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER 02/23/10 DATE

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER DUNFA XI No. C-71190 Exp. 06-30-11 CIVIL STATE OF CALIFORNIA

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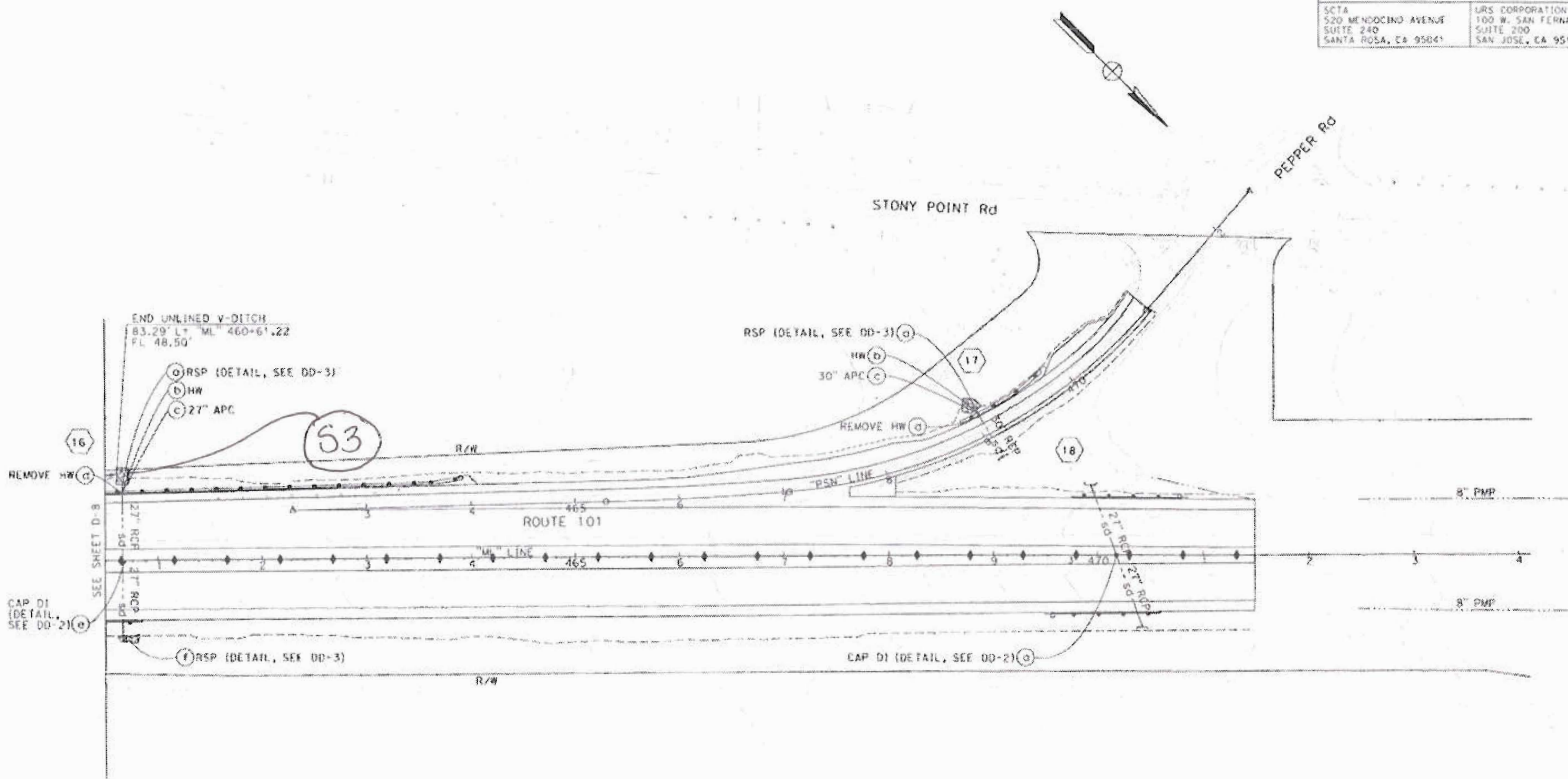
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95401

URS CORPORATION
100 W. SAN FERNANDO St.
SUITE 200
SAN JOSE, CA 95113



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET D-1.

THIS PLAN ACCURATE FOR DRAINAGE WORK ONLY.

DRAINAGE PLAN

SCALE: 1"=50'

D-9

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	CHECKED BY	REVISED BY	DATE REVISED
Caltrans	RAMSEY HISSEN	DUNFA XI	BEN RAZECHI		

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME =>
DGN FILE => ...

CU 04276

EA 0A1841

LAST REVISION 00-00-00 DATE PLOTTED => 2/25/2010 TIME PLOTTED => 9:10:10 AM

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

LEGEND:

- BIOFILTRATION STRIP
- TREATED IMPERVIOUS AREA

BMP No. / Treated Area (acres)

NOTE:

1. FOR ACTUAL LOCATIONS OF BIOFILTRATION STRIPS, SEE EROSION CONTROL SHEETS.

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER XXX No. X-XXXXX Exp. XX-XX-XX CIVIL STATE OF CALIFORNIA

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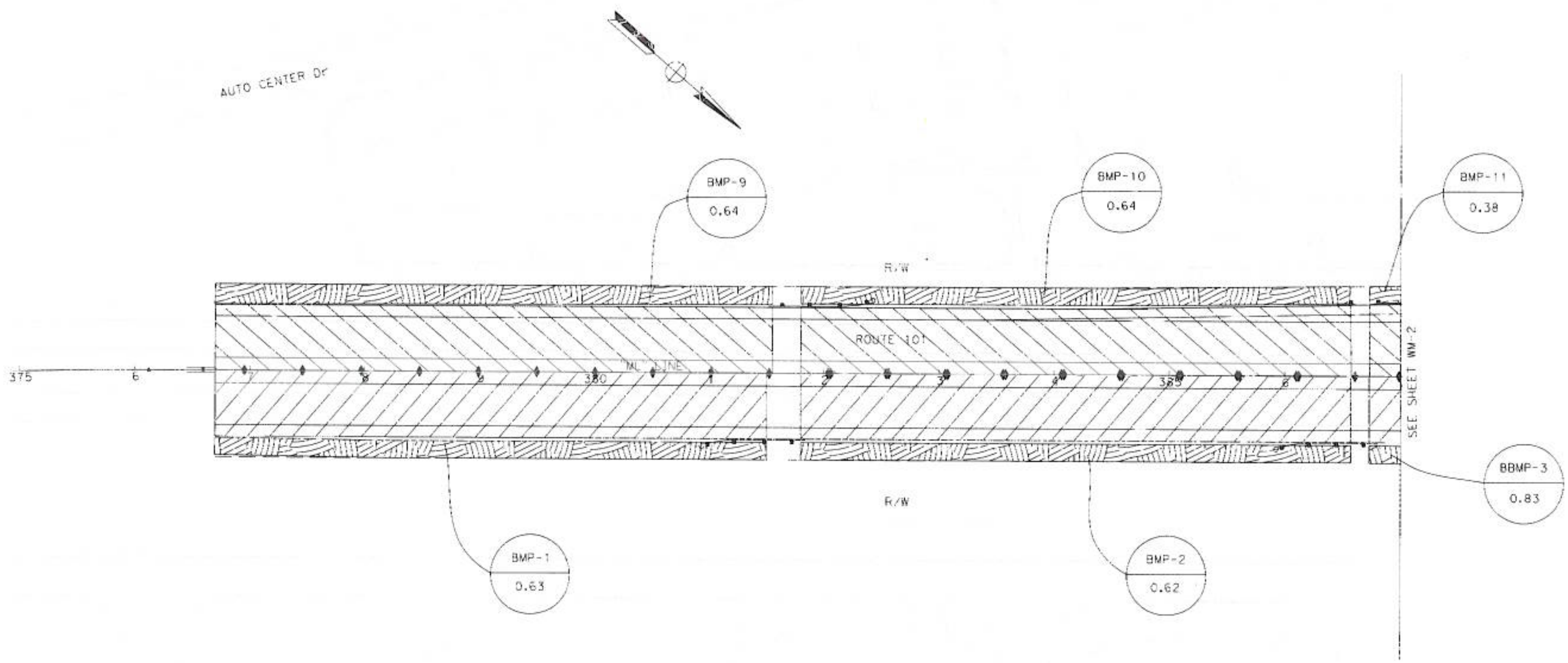
SCTA
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95041

WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612



THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50

WM-1

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	IRENE LIU	REVISED BY
HAN-BIN LIANG	CHECKED BY	ULYSSES HILLARD	DATE REVISED

LAST REVISION | DATE PLOTTED => 10/8/2009

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-01.dgn

CU 04276

EA 04-0A1831

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER XXX No. X-XXXXX Exp. XX-XX-XX CIVIL STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

SCTA
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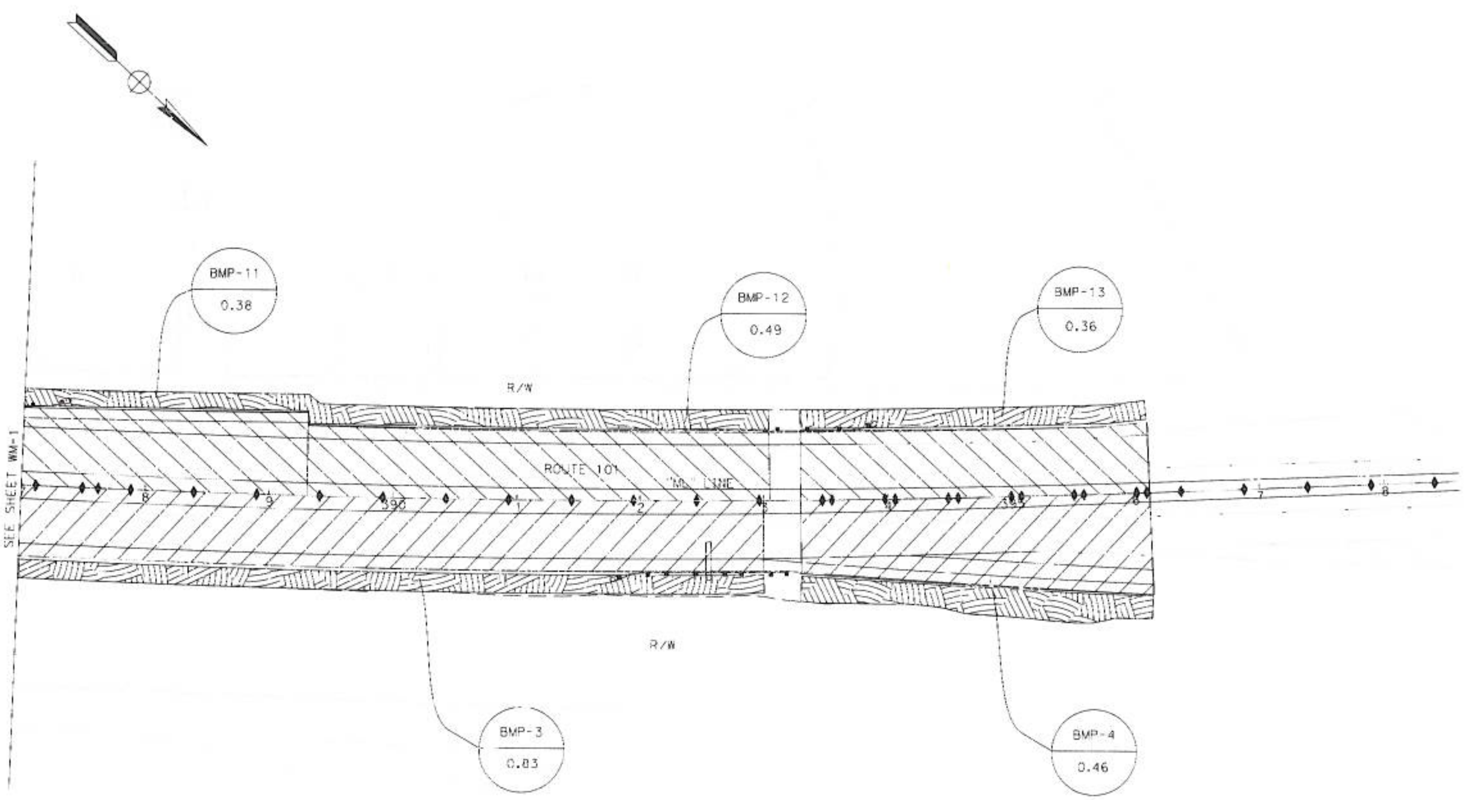
SUITE 240
SANTA ROSA, CA 95041

WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-2

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY	REVISED BY
HAN-BIN LIANG	IRENE LIU	
	CHECKED BY	DATE REVISED
	ULYSSES HILLARD	

LAST REVISION | DATE PLOTTED => 10/8/2009

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-02.dgn

CU 04276

EA 04-0A1831

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
XXX
No. X-XXXXX
Exp. XX-XX-XX
CIVIL
STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

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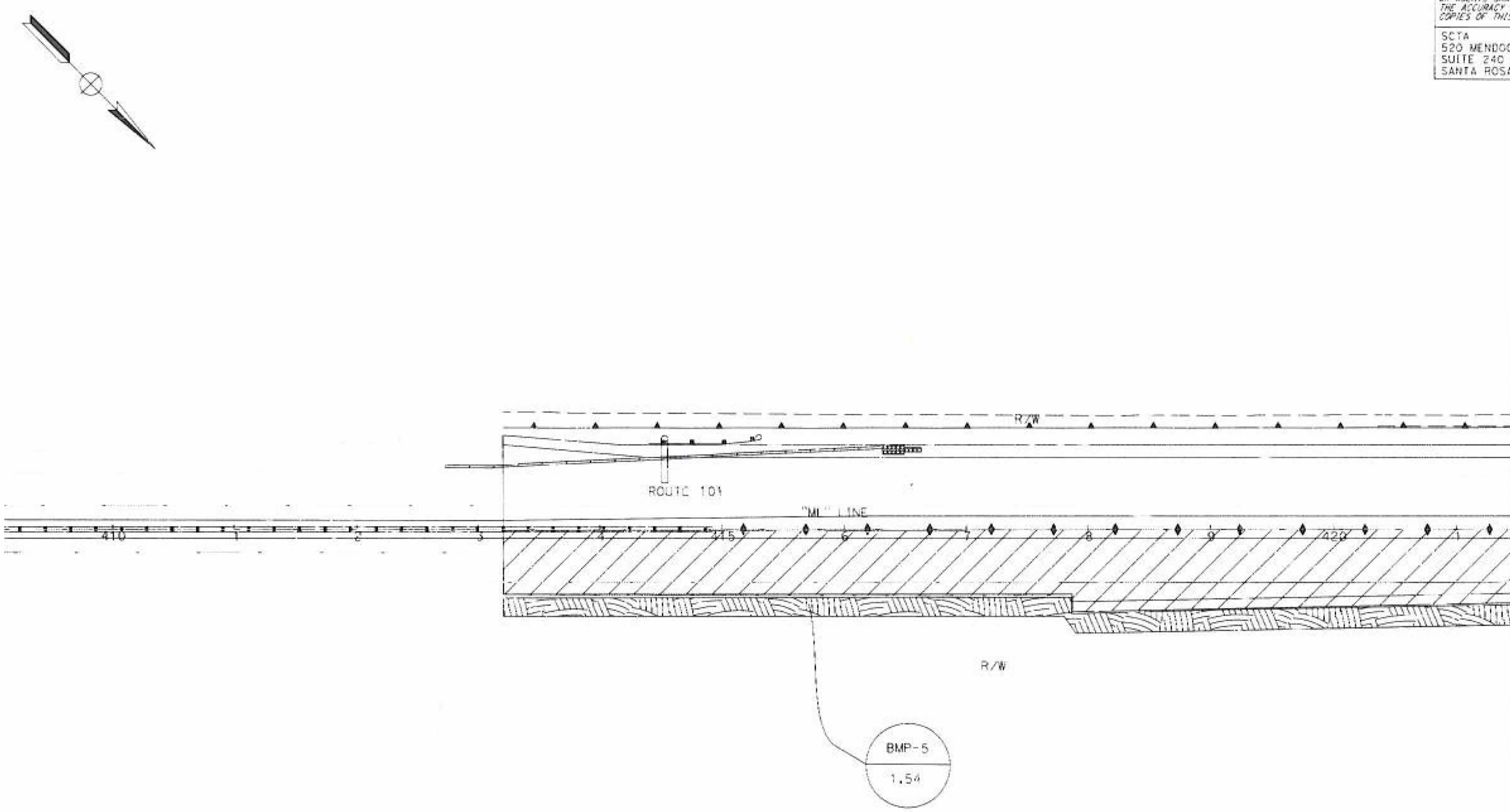
SUITE 240
SANTA ROSA, CA 95041

WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA,
SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-3

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR	HAN-BIN LIANG
CALCULATED-DESIGNED BY	IRENE LIU
CHECKED BY	ULYSSES HILLARD
REVISED BY	
DATE REVISED	

LAST REVISION | DATE PLOTTED => 10/8/2009

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-03.dgn

CU 04276

EA 04-0A1831

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

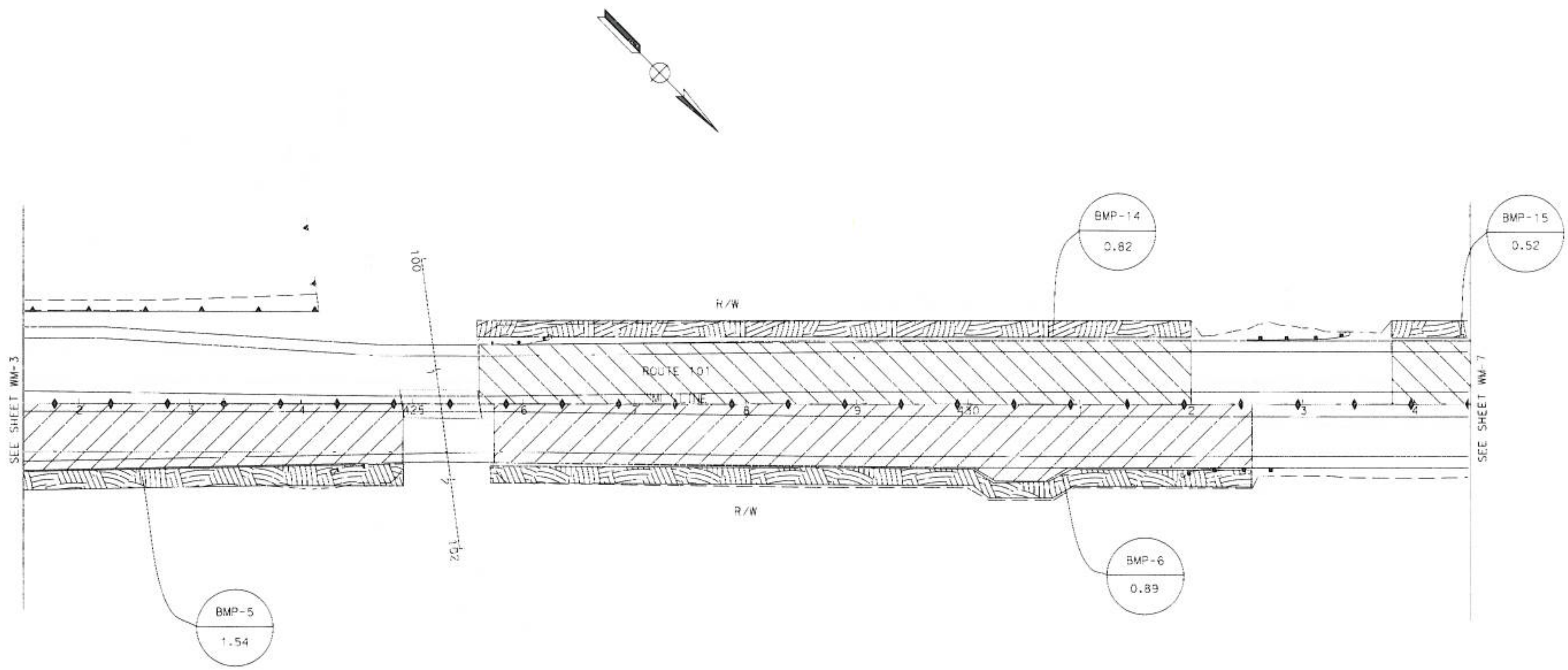
REGISTERED PROFESSIONAL ENGINEER
XXX
No. X-XXXXX
Exp. XX-XX-XX
CIVIL
STATE OF CALIFORNIA

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

SCTA
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95041

WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-4

CALCULATED-DESIGNED BY	IRENE LIU	REVISED BY
CHECKED BY	ULYSSES HILLARD	DATE REVISED

CONSULTANT FUNCTIONAL SUPERVISOR HAN-BIN LIANG

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

LAST REVISION | DATE PLOTTED => 10/8/2009

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

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CU 04276

EA 04-0A1831

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER XXX No. X-XXXXX Exp. XX-XX-XX CIVIL STATE OF CALIFORNIA

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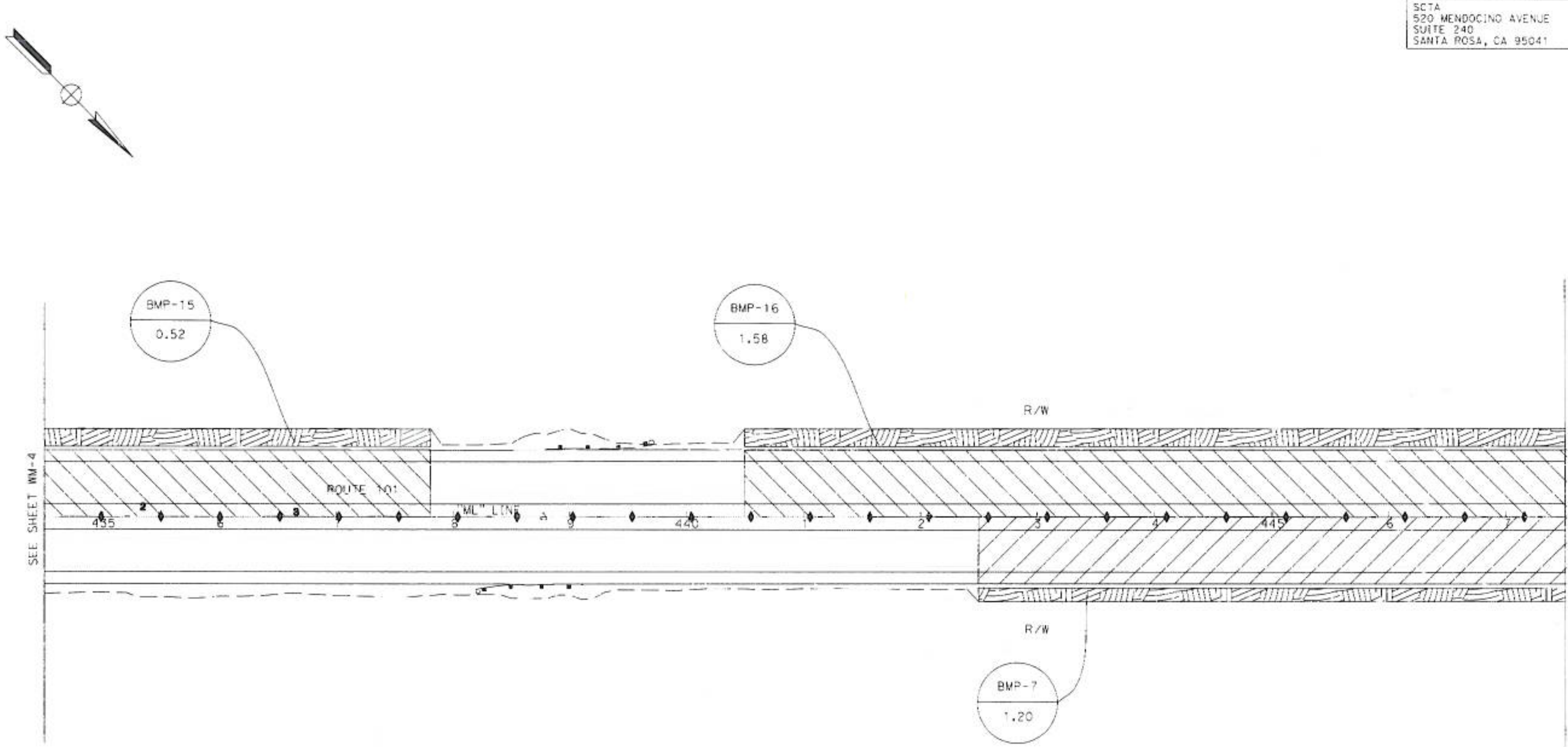
SUITE 240
SANTA ROSA, CA 95041

WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-5

REVISED BY	DATE REVISED

CALCULATED-DESIGNED BY	CHECKED BY
IRENE LIU	ULYSSES HILLARD

CONSULTANT FUNCTIONAL SUPERVISOR: HAN-BIN LIANG

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-05.dgn

CU 04276

EA 04-0A1831

LAST REVISION | DATE PLOTTED => 10/8/2009

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

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REGISTERED PROFESSIONAL ENGINEER XXX No. X-XXXXX Exp. XX-XX-XX CIVIL STATE OF CALIFORNIA

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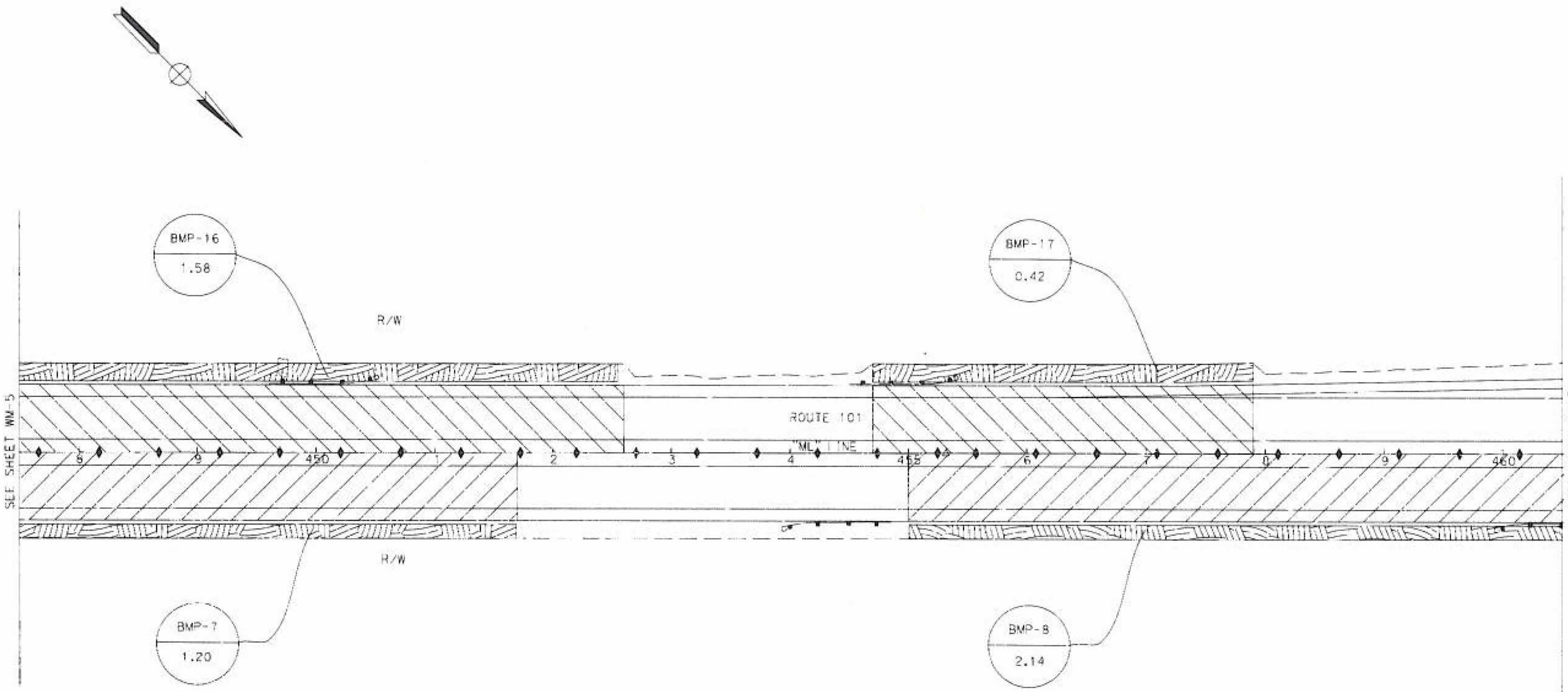
WRECO
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: HAN-BIN LIANG
CALCULATED-DESIGNED BY: IRENE LIU
CHECKED BY: ULYSSES HILLARD
REVISED BY
DATE REVISED

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-6

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-06.dgn

CU 04276

EA 04-0A1831

LAST REVISION | DATE PLOTTED => 10/8/2009

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER XXX No. X-XXXXX Exp. XX-XX-XX CIVIL STATE OF CALIFORNIA

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SANTA ROSA, CA 95041

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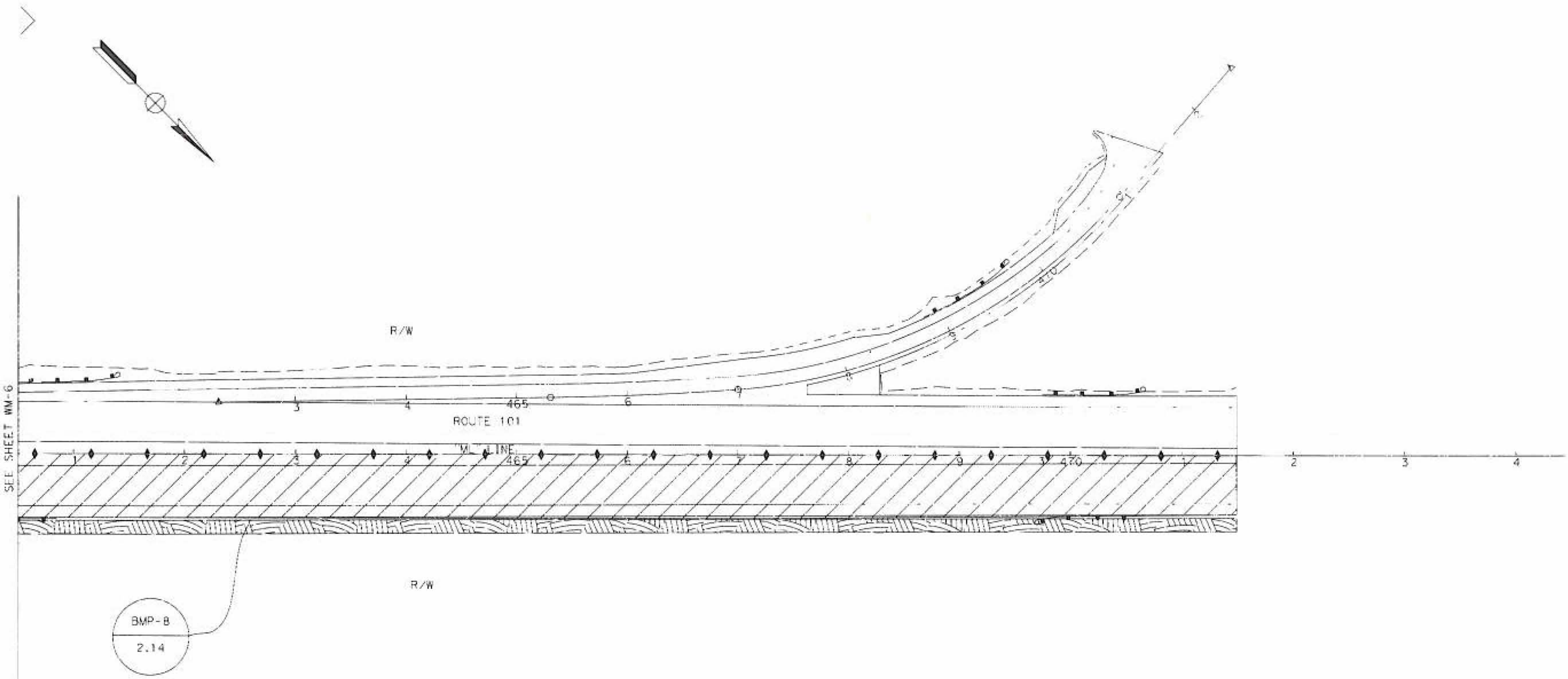
1814 FRANKLIN STREET
SUITE 608
OAKLAND, CA 94612

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.



FOR NOTES, ABBREVIATIONS AND/OR LEGEND SEE SHEET WM-1.

THIS PLAN ACCURATE FOR TREATMENT BMP WATERSHEDS ONLY.

WATERSHED MAP (TREATMENT BMP)

SCALE: 1"=50'

WM-7

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

CONSULTANT FUNCTIONAL SUPERVISOR: HAN-BIN LIANG
CALCULATED-DESIGNED BY: IRENE LIU
CHECKED BY: ULYSSES HILLARD
REVISED BY
DATE REVISED

LAST REVISION | DATE PLOTTED => 10/8/2009

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => andrew_chin
DGN FILE => ...\Dwg\WRECO\WM_BMP\WM-07.dgn

CU 04276

EA 04-0A1831

Combined Risk Level Matrix

Receiving Water Risk	Sediment Risk: Low	Sediment Risk: Medium	Sediment Risk: High
Low	Level 1	Level 2	Level 2
High	Level 2	Level 2	Level 3

Project Sediment Risk:	Medium
Project RW Risk:	High
Project Combined Risk:	Level 2

	A	B	C
1	**Sediment Risk Factor Worksheet**		**Entry**
2	**A) R Factor**		
3	Analyses of data indicated that when factors other than rainfall are held constant, soil loss is directly proportional to a rainfall factor composed of total storm kinetic energy (E) times the maximum 30-min intensity (I30) (Wischmeier and Smith, 1958). The numerical value of R is the average annual sum of EI30 for storm events during a rainfall record of at least 22 years. "Isoerodent" maps were developed based on R values calculated for more than 1000 locations in the Western U.S. Refer to the link below to determine the R factor for the project site.		
4	http://cfpub.epa.gov/npdes/stormwater/LEW/lewCalculator.cfm		
5	**R Factor Value**		97.02
6	**B) K Factor (weighted average, by area, for all site soils)**		
7	The soil-erodibility factor K represents: (1) susceptibility of soil or surface material to erosion, (2) transportability of the sediment, and (3) the amount and rate of runoff given a particular rainfall input, as measured under a standard condition. Fine-textured soils that are high in clay have low K values (about 0.05 to 0.15) because the particles are resistant to detachment. Coarse-textured soils, such as sandy soils, also have low K values (about 0.05 to 0.2) because of high infiltration resulting in low runoff even though these particles are easily detached. Medium-textured soils, such as a silt loam, have moderate K values (about 0.25 to 0.45) because they are moderately susceptible to particle detachment and they produce runoff at moderate rates. Soils having a high silt content are especially susceptible to erosion and have high K values, which can exceed 0.45 and can be as large as 0.65. Silt-size particles are easily detached and tend to crust, producing high rates and large volumes of runoff. Use Site-specific data must be submitted.		
8	Site-specific K factor guidance		
9	**K Factor Value**		0.24268
10	**C) LS Factor (weighted average, by area, for all slopes)**		
11	The effect of topography on erosion is accounted for by the LS factor, which combines the effects of a hillslope-length factor, L, and a hillslope-gradient factor, S. Generally speaking, as hillslope length and/or hillslope gradient increase, soil loss increases. As hillslope length increases, total soil loss and soil loss per unit area increase due to the progressive accumulation of runoff in the downslope direction. As the hillslope gradient increases, the velocity and erosivity of runoff increases. Use the LS table located in separate tab of this spreadsheet to determine LS factors. Estimate the weighted LS for the site prior to construction.		
12	LS Table		
13	**LS Factor Value**		0.644
14			
15	**Watershed Erosion Estimate (=RxKxLS) in tons/acre**	15.16285996	
16	**Site Sediment Risk Factor**	**Medium**	
17	Low Sediment Risk: < 15 tons/acre		
18	Medium Sediment Risk: >=15 and <75 tons/acre		
19	High Sediment Risk: >= 75 tons/acre		
20			

Receiving Water (RW) Risk Factor Worksheet	Entry	Score
A. Watershed Characteristics	yes/no	
A.1. Does the disturbed area discharge (either directly or indirectly) to a **303(d)-listed waterbody impaired by sediment**? For help with impaired waterbodies please check the attached worksheet or visit the link below: 2006 Approved Sediment-impared WBs Worksheet http://www.waterboards.ca.gov/water_issues/programs/tmdl/303d_lists2006_epa.shtml **OR** A.2. Does the disturbed area discharge to a waterbody with designated beneficial uses of SPAWN & COLD & MIGRATORY? http://www.ice.ucdavis.edu/geowbs/asp/wbquse.asp	**Yes**	**High**

Linda S. Adams
Agency Secretary

1515 Clay Street, Suite 1400, Oakland, California 94612
(510) 622-2300 • Fax (510) 622-2460
http://www.waterboards.ca.gov/sanfranciscobay

Arnold Schwarzenegger
Governor

January 27, 2010
CIWQS Place No. 726190 (BT)
401 Database Site No. 02-49-C0293

Sent via electronic mail: No hard copy to follow

California Department of Transportation
Attn: Mr. Eric Schen
Eric_Schen@dot.ca.gov
111 Grand Ave.
Oakland, CA 94612-3717

Subject: Water Quality Certification for the State Route 101 HOV Lanes Project, Segment B, City of Petaluma, Sonoma County

Department Project No.: EA 0A1831

Dear Mr. Schen:

We have reviewed and hereby issue water quality certification to the California Department of Transportation (Department) for the project referenced above (hereinafter Project). The Department has applied to the U.S. Army Corps of Engineers (Corps) for Nationwide Permit No. 14, *Linear Transportation Projects*, pursuant to Section 404 of the Clean Water Act (33 U.S.C. 1344). As such, the Department has applied to the Water Board for a Clean Water Act Section 401 water quality certification that the Project will not violate State water quality standards.

Project: The Department proposes to widen a 1.8-mile segment of State Route 101 (SR 101) from 0.51 miles south of the Old Redwood Highway overcrossing to 0.07 miles north of Pepper Road. High-occupancy vehicle lanes will be constructed in each direction, converting SR 101 from a four to six-lane highway. Project activities include widening two existing bridges across Willow Brook Creek, cut and fill earthwork, and culvert extensions.

The two existing bridges at Willow Brook creek will be widened and fill the existing approximately 26-foot median gap. The outside shoulders of the north and southbound travel lanes of SR 101 will each be widened by approximately 9.8 feet. The Pepper Road to southbound SR 101 on-ramp will be modified to accommodate a ramp metering system.

Impacts: Project implementation will result in the permanent fill of approximately 0.25 acres of jurisdictional seasonal freshwater wetlands, 26 linear feet (0.014 acres) of Willow Brook Creek, and 0.052 acres (1,129 linear feet) of jurisdictional roadside drainages. Approximately 0.10 acres (2,200 linear feet) of drainage ditches will be filled and re-built, in-kind, adjacent their former locations. Three to four willow trees will be removed from the banks of Willow Brook Creek due to bridge widening into the median.

Project implementation would result in approximately 13.3 acres of added impervious area. Stormwater runoff from impervious areas may contain hydrocarbons, metals, volatile organic compounds, trash, and sediment at levels that may significantly impact jurisdictional waters if left untreated.

Based upon a hydromodification susceptibility analysis prepared by WRECO and dated October 15, 2009, this Project will not result in hydromodification impacts due primarily to low-gradient receiving waters.

Mitigation: To mitigate for permanent impacts to jurisdictional seasonal freshwater wetlands, Willow Brook Creek, and the jurisdictional roadside drainage, the Department shall purchase 0.3 acres of wetland mitigation bank credits from Burdell Ranch Wetland Conservation Bank. The Department shall also plant and establish six willow trees on the southeastern bank of Willow Brook Creek. To mitigate for permanently impact roadside drainage ditches, all permanently impacted ditches shall be re-built, in-kind, adjacent their former locations (except for the drainage ditch between stations 411+71 and 425+07, which shall be mitigated for using mitigation credits at Burdell Ranch).

As mitigation for increased pollutant loads associated with impervious areas, the Department shall provide treatment of stormwater runoff from approximately 14.2 acres of impervious area using 16 compost-amended biofiltration strips. The following biofiltration strips and corresponding locations will mitigate water quality impacts resulting from Project implementation:

Strip No.	Northbound/ Southbound	From Post Mile	To Post Mile	Treated Impervious Area
1	NB	7.13	7.22	0.63
2	NB	7.23	7.32	0.62
3	NB	7.32	7.44	0.83
4	NB	7.44	7.50	0.46
5	NB	7.82	8.04	1.54
6	NB	8.06	8.19	0.89
7	NB	8.38	8.55	1.20
8	NB	8.62	8.93	2.14
9	SB	7.13	7.22	0.64
10	SB	7.23	7.32	0.64
11	SB	7.32	7.44	0.87
12	SB	7.44	7.50	0.36
13	SB	8.08	8.18	0.82
14	SB	8.21	8.29	0.52
15	SB	8.34	8.57	1.58

Strip No.	Northbound/ Southbound	From Post Mile	To Post Mile	Treated Impervious Area
16	SB	8.61	8.67	0.42
				total: 14.2 acres

The Department is proposing to treat stormwater runoff from approximately 14.2 acres of impervious area, approximately 0.9 acres above what is required by the Water Board. This surplus area of treatment may be applied as credit to a future Department Project in the Project watershed.

Wetland Tracking System: The Water Board tracks routine riparian repair and creek maintenance projects in an effort to detect potential systemic instabilities and document project performance in the creeks of the Bay Area. As such, the Applicant is required to submit a Riparian Repair and Maintenance Wetland Tracker short form describing Project size, type, and performance measures. An electronic copy of the short form and instructions can be downloaded at: http://www.waterboards.ca.gov/sanfranciscobay/certs.shtml.
Project information will be made available at the web link: http://wetlandtracker.org.

CEQA Compliance: The Project complied with the California Environmental Quality Act via the August 30, 2007, *Highway 101 HOV Lane Widening Project: Petaluma to Rohnert Park, Environmental Assessment/Final Environmental Impact Report.*

Certification: I hereby issue an order certifying that any discharge from the referenced project will comply with the applicable provisions of sections 301 (Effluent Limitations), 302 (Water Quality Related Effluent Limitations), 303 (Water Quality Standards and Implementation Plans), 306 (National Standards of Performance), and 307 (Toxic and Pretreatment Effluent Standards) of the Clean Water Act, and with other applicable requirements of State law. This discharge is also regulated under State Water Resources Control Board Order No. 2003 - 0017 – DWQ, "General Waste Discharge Requirements for Dredge and Fill Discharges That Have Received State Water Quality Certification" which requires compliance with all conditions of this Water Quality Certification. The following conditions are associated with this certification:

1. The Department shall adhere to the Standard conditions imposed by Nationwide Permit No. 14, issued to the Department by the Corps;

2. The Project shall be constructed in conformance with the Project Description described in this certification and certification application materials. Any change in the Project may require amendment of the certification and shall be reported to the Water Board. Any change in Project description must be accepted by the Water Board Executive Officer prior to implementation of said change in the Project;

3. Commencement of any Project element is prohibited until the Department has provided Water Board receipt of 0.3 acres of wetland mitigation bank credit from the Burdell Ranch Wetland Conservation Bank;

4. Except as expressly allowed in this certification, no equipment shall be operated in areas of flowing or standing water; no fueling, cleaning or maintenance of vehicles or equipment shall take place within jurisdictional waters or within any areas where an accidental discharge to waters of the State may occur;

5. All temporarily impacted areas shall be restored to pre-construction or enhanced conditions;

6. Except for the drainage ditch between stations 411+71 and 425+07, all permanently impacted roadside drainage ditches shall be re-built, in-kind, adjacent their former locations by Project completion;

7. Except as expressly allowed in this Certification, the discharge, or creation of the potential for discharge, to waters of the State of any construction wastes and/or soil materials including cement, fresh concrete, or washings thereof, silts, clay, sand, oil or petroleum products and other organic materials to waters of the State is prohibited;

8. The Department shall install biofiltration strips at the abovementioned locations by Project completion. All strips shall be compost-amended. To avoid damage to the strips from construction-related activities (e.g., compaction and sedimentation), the Department shall use order of work specifications to ensure the strips are constructed after major construction activities have been completed. If the Department cannot reserve strip construction until the final constructions stage, then Best Management Practices (BMPs) shall be detailed in the Stormwater Pollution Prevention Plan to either prevent and/or ameliorate damage to the BMPs. A BMP inspection report and description of any BMP repair measures shall be provided upon request by the Water Board;

9. The Department shall fully implement the "Willow Brook Creek Willow Tree Mitigation and Monitoring Plan (Plan)," dated November 2009, and included in this certification as Attachment A. The Department shall:

    a. Not deem willow plantings successful sooner than five growing seasons after planting, whereupon five of the six planted willows shall exhibit average or improved health and vigor from the previous two growing seasons;
    b. Provide additional planting, maintenance and monitoring until the success criteria is satisfied if the above success criteria is not met;
    c. Deem willow plantings successful before two full growing seasons have passed upon termination of supplemental watering; and,
    d. At a minimum, submit years 0, 1, 3 and 5 monitoring reports to the Water Board.

10. Willows planted as described in the Plan, and other native streamside vegetation growing on the southeastern bank of Willow Brook Creek, shall not be removed or trimmed at any time without authorization from the Water Board;

11. All work in Willow Brook Creek shall be conducted only between June 15 and October 15;

12. This certification does not allow for the take, or incidental take, of any special status species. The Department shall use the appropriate protocols, as approved by the California Department of Fish and Game and the U.S. Fish and Wildlife Service, to ensure that Project activities do not impact the Beneficial Use of the Preservation of Rare and Endangered Species;

13. The Department shall maintain a copy of this water quality certification at the Project site so as to be available at all times to site operating personnel. It is the responsibility of the Department to assure that all personnel (employees, contractors, and subcontractors) are adequately informed and trained regarding the conditions of this certification;

14. Not later than 30 days prior to the beginning of construction of any Project component, the Department shall submit, acceptable to the Executive Officer, a final SWPPP to address the Project's expected construction stage impacts, prepared pursuant to the State Water Resources Control Board Water Quality Order No. 99-06-DWQ, the NPDES Statewide Permit for Storm Water Discharges From the State of California City of Transportation Properties, Facilities, and Activities;

15. The Department shall submit, subject to acceptance by Water Board staff, a dewatering and/or diversion plan that appropriately describes how the work areas will be dewatered during construction. The dewatering and/or diversion plan shall be submitted no later than 30 days prior to the beginning of proposed dewatering or flow diversion. Information submitted shall include the area to be dewatered and/or diverted, timing of dewatering and/or diversion, and method of dewatering and/or diversion to be implemented. All temporary dewatering and/or diversion methods shall be designed to have the minimum necessary impacts to waters of the State to isolate the immediate work area. All dewatering and/or diversion methods shall be installed such that natural flow is maintained upstream and downstream of the project area. Any temporary dams or diversions shall be installed such that the diversion does not cause sedimentation, siltation, or erosion upstream or downstream of the project area. All dewatering methods shall be removed immediately upon completion of Project activities;

16. The Department is required to use the Riparian Repair and Maintenance Wetland Tracker short form to provide Project information within 14 days from the date of this certification. The completed short form and map showing the project boundaries shall be submitted electronically to wetlandtracker@waterboards.ca.gov or shall be submitted as a hard copy to both: 1) The Water Board (see the address on the letterhead), to the attention of Wetland Tracker; and 2) The San Francisco Estuary Institute, 7770 Pardee Lane, Oakland, CA 94621-1424, to the attention of Mike May;

17. The Resident Engineer shall hold on-site water quality permit compliance meetings (similar to tailgate safety meetings) to discuss permit compliance, including instructions on how to avoid violations and procedures for reporting violations. The meetings shall be held at least every other week, and particularly before forecasted storm events and when a new contractor or subcontractor arrives to begin work at the site. The contractors, subcontractors and their employees, as well as any inspectors or biological monitors assigned to the project, shall be present at the meetings. Caltrans shall maintain dated sign-in sheets for attendees at these meetings, and shall make them available to the Regional Water Board on request;

18. This certification action is subject to modification or revocation upon administrative or judicial review, including review and amendment pursuant to Section 13330 of the California Water Code (CWC) and Section 3867 of Title 23 of the California Code of Regulations(23 CCR);

19. This certification action does not apply to any discharge from any activity involving a hydroelectric facility requiring a Federal Energy Regulatory Commission (FERC) license or an amendment to a FERC license, unless the pertinent certification application was filed pursuant to California Code of Regulations (CCR) Title 23, Subsection 3855(b) and that application specifically identified that a FERC license or amendment to a FERC license for a hydroelectric facility was being sought; and,

20. Certification is conditioned upon total payment of the full fee required in State regulations (23 CCR Section 3833). Water Board staff received full payment of $640.00 on January 15, 2009.

We anticipate your cooperation in implementing these conditions. However, please be advised that any violation of water quality certification conditions is a violation of State law and subject to administrative civil liability pursuant to California Water Code (CWC) section 13350. Failure to respond, inadequate response, late response, or failure to meet any condition of this certification may subject you to civil liability imposed by the Water Board to a maximum of $5,000 per day per violation or $10 for each gallon of waste discharged in violation of this certification.

Conditions 3 and 14-16 are requirements for submission of reports. Any requirement for a report made as a condition to this action is a formal requirement pursuant to CWC section 13267, and failure or refusal to provide, or falsification of such required report is subject to civil liability as described in CWC section 13268.

We anticipate no further action on this request. Should new information come to our attention that indicates a water quality problem with this project, the Water Board may issue Waste Discharge Requirements pursuant to 23 CCR Section 3857.

If you have any question, please contact Brendan Thompson at (510) 622-2506, or via e-mail to BThompson@waterboards.ca.gov.

Sincerely,

Shin-Roei Lee acting for

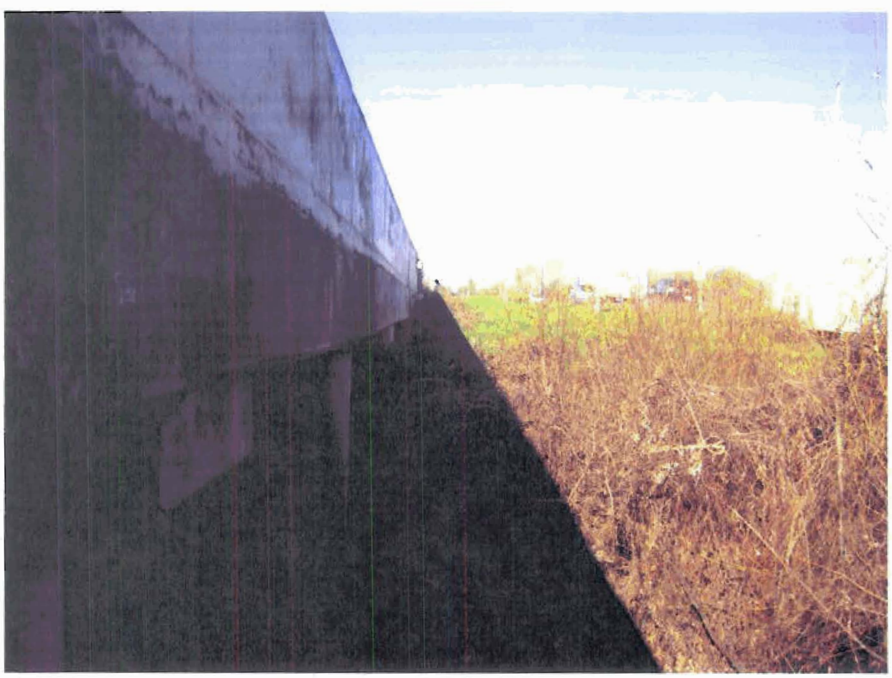
Bruce H. Wolfe
Executive Officer

cc (via e-mail):
Mr. Bill Orme SWRCB-DWQ
Mr. Hal Durio, Regulatory Branch, USACE
Ms. Jane Hicks, Regulatory Branch, USACE
Ms. Holly Costa, Regulatory Branch, USACE
Ms. Laurie Monarres, USACE
Mr. Cyrus Vafai, Caltrans
Mr. Dale Bowyer, Water Board
Ms. Melissa Escaron, Fish and Game, Yountville
Mr. Hardeep Takhar, Caltrans
Mr. David Smith, USEPA
Ms. Andrea Meier, USACE

Attachment A
Willow Brook Creek Willow Tree Mitigation Monitoring Plan

Willow Brook Creek Willow Tree Mitigation Monitoring Plan

101 Sonoma Segment B HOV Lanes Widening Project
Willow Brook Creek Bridge
04-0A1841
PM 7.1/8.9



November 2009

1. Introduction

This document presents a mitigation monitoring plan for planted willow trees within the Willow Brook Creek riparian corridor by the 101 Sonoma Segment B High Occupancy Vehicle (HOV) Lanes Widening and Improvement Project. The 101 Sonoma Segment B HOV Lanes Widening and Improvement Project extends from about a half a mile south of Old Redwood Highway crossing to just north of Pepper Road in Petaluma, California. The proposed part of the project at Willow Brook Creek for the 101 Segment B HOV Lane Widening Project consists of widening the 101 bridge into the median area between the northbound and southbound lanes and creating a complete deck across Willow Brook Creek. There is no proposed widening of the bridge to the east or west of the existing structure. The widened portion will match the existing structure with a deck slab and pile extensions at the two bents. The location of the Willow Brook Creek bridge is shown in **Figure 1**.

Two existing willow trees located within the Willow Brook riparian corridor and between the northbound and southbound spans of the existing Willow Brook bridge are proposed to be removed as a result of the bridge construction (see **Photo 1**). As a result of the removal of these trees from the riparian corridor, Caltrans has proposed to mitigate these potential impacts by planting six willows trees and implementing a willow tree mitigation monitoring plan.

2. Mitigation and Monitoring Details

The goal of the willow tree planting mitigation is to stabilize the stream bank, provide additional habitat or enhancement of existing habitat for fish and wildlife within the Willow Brook riparian corridor, to provide enhanced aesthetic qualities associated with the willow tree foliage and natural greenery, and to effectively maintain the "No Net Loss Policy" for riparian areas. To accomplish this goal, Caltrans proposes to plant willow tree cuttings taken from nearby willow trees. The willow tree cuttings are to be planted along the bank of Willow Brook Creek in close proximity to the Willow Brook Creek Bridge in the State Right of Way (the exact location is indicated by **Figure 1**).

The mitigation site shall be planted no later than the first winter following bridge construction using the willow tree cuttings from the nearby willow trees. The planted willow cuttings will be planted in a riparian area very close to where the trees were removed in a location where the trees are well suited to successfully grow and mature. The mitigation site was chosen on the east side of the bridge where there is sufficient space and access along the riparian corridor of Willow

Brook Creek. This allows for easier maintenance and monitoring of the mitigation site. The west side of the bridge was a less desirable location because of the confinement caused by the adjacent mobile home park and the abundance of existing vegetation. The recommended willow cuttings will be harvested and installed between January and February. Cuttings shall be reasonably straight and a minimum 24 inches long and ¾ to 1.5-inch in diameter. Cuttings shall be installed perpendicular to the soil surface such that approximately ¾ of the cutting length (~18 inches) is below ground and ¼ (~6 inches) is above ground.

Post construction monitoring of the site will occur each year following the plantings which will include a visual inspection of the plantings with photo documentation. Planted willow trees will be deemed successful and performance criteria met if after the fifth year 5 of the 6 planted willow trees exhibit average or better health and vigor and have observable growth from the last two years. If success criteria are not met, Caltrans will provide additional planting, maintenance and monitoring until the success criteria is satisfied.

The two existing willow trees to be removed are small trees at less then 5 inches in diameter at breast height. At least six willow tree cuttings will be planted to ensure that tree establishment is achieved.

The installation contractor will maintain the plantings for the first three year period and will also provide monthly monitoring updates to the Caltrans biologist. Maintenance of the plantings may require supplemental watering or installation of cages to prevent herbivory. Photo points will be established to document re-vegetation efforts. Montoring reports with photo-documentation will be provided and submitted to the agencies.at planting completion, 3-years after and 5-years after planting of willow tree cuttings. Monitoring reports shall summarize each year's monitoring results, compare data to previous years, describe progress towards meeting the final performance criteria and summarize the need for any remedial actions. Additional monitoring is not required if after three years of monitoring and no less than two years after supplemental watering has ceased, the planted willow trees are determined to be in good health. .

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
04	Son	101	7.1/8.9		

REGISTERED CIVIL ENGINEER DATE

RAMSEY J. HISSEN
No. C-39608
Exp. 12-31-09
CIVIL
REGISTERED PROFESSIONAL ENGINEER
STATE OF CALIFORNIA

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

SCTA
520 MENDOCINO AVENUE
SUITE 240
SANTA ROSA, CA 95041

URS CORPORATION
55 S. MARKET STREET
SUITE 1500
SAN JOSE, CA 95113

WILLOW TREES MITIGATION PLAN

SCALE 1"=50'

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	CONSULTANT FUNCTIONAL SUPERVISOR	CALCULATED- DESIGNED BY	DUNFA XI
Caltrans	RAMSEY HISSEN	CHECKED BY	
		REVISED BY	
		DATE REVISED	

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => .
DGN FILE => ...\Mitigation_WillowsLocation.dgn

CU 04276

EA 0A1841

LAST REVISION 00-00-00 DATE PLOTTED => 10/16/2009 TIME PLOTTED => 4:47:17 PM

Mitigation_WillowsLocation.dgn 10/16/2009 4:47:17 PM

Photo 1. Willow trees in between northbound and southbound Willow Brook Creek bridges that are to be removed.