F I N A L

PRELIMINARY GEOTECHNICAL REPORT

US 101 EXPRESS LANES PROJECT, SANTA CLARA COUNTY, CALIFORNIA

EA 2G7100
04-SCL-101, PM 16.0/52.55
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Prepared for

California Department of Transportation
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and

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June 2013

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Project 28645266

File No.04-SCl-101-PM 16.0/52.55
EA2G7100

Mr. Lam Trinh
Project Manager
Valley Transportation Authority (VTA)
3331 North First Street
Building B
San Jose, CA 95134-1927

Subject: **Preliminary Geotechnical Report (PGR)**
US 101 Express Lanes Project
Santa Clara County, California

Dear Mr. Trinh:

URS has completed a geotechnical study and Preliminary Geotechnical Report (PGR) for the proposed modifications to US 101 in Santa Clara County, California. The accompanying report has been prepared in accordance with Caltrans GDR Guidelines, dated December 2006.

The report presents our preliminary engineering opinions and recommendations regarding the geotechnical factors influencing the design and construction of the proposed roadway modifications and road signs. The opinions and recommendations have been based upon the results of our review of existing subsurface information, engineering judgment and local experience. Mr. Madhu Thummaluru, Professional Engineer, performed engineering calculations and assisted in preparation of this report. Ms. Sheri Janowski, Professional Geologist, contributed to the seismic source characterization and the evaluation of geologic conditions along the project alignment. Mr. Stephen Huang, Geotechnical Engineer, provided technical review and guidance to the URS project team over the course of the study. Mr. Paul Boddie, Principal Geotechnical Engineer, also provided overall peer review of this report in accordance with URS Corporation Quality Control Plan.

Since our September 2012, November 2012, December 2012, January 2013, and April 2013 reports were issued, we received review comments from Caltrans dated October 26, 2012, December 12, 2012, February 8, April 10, May 8 and 14, 2013. These comments have been 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
MICHAEL L. LARSON
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Exp. 06/30/2013
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This Preliminary Geotechnical Report presents a review of potential geologic, geotechnical, and seismic issues for the proposed United States Highway 101 (US 101) Express Lanes Project in Santa Clara County, California. Santa Clara Valley Transportation Authority (VTA), in cooperation with the California Department of Transportation (Caltrans), proposes to convert the existing High-Occupancy Vehicle (HOV) lanes along US 101 to High-Occupancy Toll (HOT) lanes (hereafter known as express lanes) and add a second express lane in each direction on northbound and southbound US 101 within the overall project limits of East Dunne Avenue interchange in Morgan Hill to the Santa Clara/San Mateo County line just north of the Oregon Expressway/Embarcadero Road interchange in Palo Alto. The express lanes will allow HOVs and eligible clean air vehicles continued use of the lanes for free and eligible Single-Occupant Vehicles (SOVs) to pay a toll. The project will also convert the US 101/State Route (SR) 85 HOV direct connectors in Mountain View to express lane connectors and restripe the northern 1.1 mile of SR 85 to introduce a buffer separating the mixed flow lanes from the express lane and connecting the SR 85 express lanes to the US 101 express lanes. A project Vicinity Map and Project Location Map are provided in Figures 1 and 2. The project length is 36.55 miles on US 101 and 1.1 miles on SR 85, for a total of 37.65 miles.

1.1 PURPOSE OF THIS STUDY

The purpose of this Preliminary Geotechnical Report is to document existing foundation conditions, review subsurface information, and provide potential geotechnical impacts of the project. The information in this document will provide input to the project's preliminary design and environmental document. A subsequent geotechnical study will be performed during final design.

1.2 PROJECT SCOPE OF WORK

The scope of work for this study included:

Review of available as-built bridge drawings, logs of test borings (LOTBs), laboratory test results, geologic maps, fault maps and geologic hazard maps, and other existing information.

Preparation of this report, which includes:

Development of seismic design criteria;

Description of site geology and evaluation of geologic hazards;

Assessment of subsurface conditions based on existing information;

Identification of potential geotechnical impacts on the project; and

Preliminary geotechnical recommendations regarding the roadway widening pavement design, new overhead sign structure foundations, and construction considerations.

1.3 PROJECT DESCRIPTION

1.3.1 Existing Facilities

US 101 in Santa Clara County is a 52.55-mile long freeway that connects Gilroy to Palo Alto. Within the project limits, US 101 passes through Gilroy, Morgan Hill, San Jose, Santa Clara, Sunnyvale, Mountain View and Palo Alto. US 101 intersects SR 85 in San Jose and in Mountain View, I-280/I-680, I-880, SR 87, and SR 237. US 101 typically has 4 lanes in each direction, including 3 mixed-flow lanes and 1 HOV lane with auxiliary lanes in some locations.

1.3.2 Proposed Project

The project consists of converting the existing HOV lanes along both northbound and southbound US 101 into express lanes and widening the freeway to add a second express lane for the majority of the corridor. The project also proposes to build new express lanes in the northbound direction between East Dunne Avenue and the existing HOV lane at Cochrane Road, and in the southbound direction between Burnett Avenue and Cochrane Road.

With these changes, there would be two express lanes on US 101 extending from approximately the Cochrane Road interchange in Morgan Hill to just south of the Oregon Expressway/ Embarcadero Road interchange in Palo Alto in the northbound direction, and from just south of the Oregon Expressway/Embarcadero Road interchange to just south of the Burnett Avenue overcrossing in the southbound direction.

1.3.2.1 Build Alternative

The addition of the second express lane will involve a combination of inside and outside widening. The majority of the inside widening will occur within the US 101 segments south of the SR 85/US 101 interchange in southern Santa Clara County where a wide unpaved median exists. The project proposes to widen and pave the median to accommodate the additional lanes. The outside widening will occur in the remainder of the corridor to accommodate the additional lanes where needed.

The express lanes facility would be separated from the adjacent mixed-flow lanes by a striped buffer. The buffer zone, delineated with solid stripes, will have designated openings to provide access into and out of the express lanes facility. The express lanes would allow HOVs to continue to use the lanes without cost and eligible single-occupant vehicles (SOVs) to pay a toll.

The project proposes to construct and operate the express lane system with some non-standard cross sectional elements which will minimize the need for new right-of-way, outside widening, and structure reconstruction. The proposed project maximizes the use of the existing pavement cross section with a combination of inside and outside widening to create the additional pavement needed to accommodate the second express lane.

1.3.2.1.1 Right of Way

It is anticipated that the project will require limited right-of-way and Temporary Construction Easements (TCE). Right of way activities are currently being coordinated based on the approval of design exceptions. Utility relocations are anticipated to accommodate the outside widening.

1.3.2.1.2 Construction Activities

In the section between the southern project limit and the SR 85 interchange in southern San Jose, where the median width varies between 46 and 86 feet, pavement widening would be constructed in the median to accommodate the dual express lane facility. A retaining wall in the median is required to accommodate the inside widening where a split profile exists between northbound and southbound US 101.

A dual express lane facility is proposed for the majority of the corridor, with the exception of short segments near the SR 85 express lane connectors where a single express lane is proposed. A single express lane is proposed between the SR 85 Interchange and the Blossom Hill Road Interchange in San Jose, and between the Mathilda Avenue interchange and the SR 85 interchange in Mountain View. Outside widening is proposed to accommodate dual express lanes between the Blossom Hill Road interchange and the Mathilda Avenue interchange.

Bridge widening will be required at a number of grade separations and undercrossings, as well as modifications to existing overcrossing abutments, which are summarized below in Tables 1-1 and 1-2. Widening of creek bridges is not anticipated at this time pending the approval of non-standard cross sectional features.

Table 1-1 Proposed Bridge Widening

Bridge No.	Post Mile	Bridge Name	Type of Work
37-344	21.25	Coyote Creek Golf Drive UC	Widen Bridge (Inside)
37-404	21.55	Utility Facility UC (Golf Course)	Widen Bridge (Inside)
37-347	27.01	Bernal Rd UC	Widen Bridge (Inside)
37-108	29.72	Coyote Rd UC	Widen Bridge (Inside and Outside)
37-409	31	Yerba Buena Rd UC	Widen Bridge (Inside and Outside)

Table 1-2 Proposed Modification to Bridge Abutments

Bridge No.	Post Mile	Existing	Bridge Name	Type of Work
37-668	33.03	4 Span 88.6' Width, 310.9' Length	Tully Rd OC	Modify Abutments
37-222	35.46	2 Span 45.9' Width, 236.2' Length	San Antonio St OC	Modify Abutments
37-48	35.76	2 Span 106.9' Width, 231.9'	Santa Clara St OC	Modify Abutments
37-123	36.12	2 Span 82' Width, 276.5' Length	Julian/McKee OC	Modify NB Abutments
37-115	37.99	2 Span 278.8' Length	North San Jose UP	Modify SB Abutments
37-118	38.09	4 Span 65.9' Width, 393.9' Length	10th St OC	Modify SB Abutment
37-403R	39.90	7 Span 39.4' Width, 945' Length	Route 87/101 SEP	Modify SB Abutments
37-183G	39.91	3 Span 41.7' Width, 521.5' Length	Route 87/101 SEP	Modify SB Abutment
37-390	42.73	2 Span 127.9' Width, 257.8'	Bowers Ave OC	Modify Abutments
37-152	43.85	2 Span 129.9' Width, 263.7'	Lawrence Expwy	Modify Abutments

The piles for the overhead signs are expected to be up to 6 feet in diameter and extend to approximately 30 feet below ground surface. The piles for the tolling devices are expected to be up to 2.5 feet in diameter and would extend to approximately 10 feet below ground surface. Some Traffic Operations Systems (TOS) equipment such as traffic monitoring stations, Closed Circuit Television cameras, cabinets, and controllers would be installed along the outside edge of pavement within the existing right-of-way.

Trenching would be conducted along the outside edge of pavement for installation of conduits. The depth of trenching would be 3 to 5 feet below the roadway surface. Conduits would be bored and jacked across the freeway to the median where needed to provide power and communication feeds to the new overhead signage and tolling equipment.

During construction, some lane and ramp closures would be required, but full freeway closures are not expected.

Biofiltration swales are proposed to provide storm water treatment for impervious areas that would be added or reworked as part of the project. These swales would be located within the existing right-of-way.

1.3.2.1.3 US 101/SR 85 Direct Connectors

At the south end of the project in southern San Jose, both the northbound and southbound HOV direct connectors from SR 85 to US 101 will be converted to express lane connectors by the SR

85 Express Lanes Project, allowing SOVs with valid FasTrak devices to use the direct connectors.

At the north end of the project in Mountain View, the US 101 Express Lanes Project will convert the existing HOV connectors to express lane connectors and will extend the buffer striping onto SR 85 to connect to the buffer constructed by the SR 85 Express Lanes Project (EA #04-4A7900). The combination of SR 85 and US 101 Express Lanes projects will provide a complete express lane system on both freeways that includes the direct connectors.

1.3.2.2 No Build Alternative

The No Build Alternative assumes no modifications would be made to the current US 101 corridor, including the continuous access HOV lane, other than routine maintenance and rehabilitation of the facility and any currently planned and programmed projects within the area.

2.1 REGIONAL GEOLOGY

The project alignment follows the Santa Clara Valley within the San Francisco Bay block, located in the central portion of the Coast Ranges geomorphic province of California. Northwest-southeast-trending valleys and ridges characterize the regional morphology of the Coast Ranges province. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent predominantly strike-slip faulting along the San Andreas fault system between the Pacific and North American plates, after the Farallon plate was subducted beneath the Pacific plate. The San Francisco Bay block is a relatively stable, aseismic block bounded by the San Andreas and Hayward faults to the west and east, respectively.

Geologic maps obtained from the U.S. Geological Survey (Helley et. al. 1994; Brabb and Graymer, 1998) and from the California Geological Survey (Jennings, 2010) indicate that the relatively level project alignment is underlain predominantly by thick, unconsolidated, interbedded alluvial and fluvial deposits of clay, silt, sand and gravel (see Figure 3A). The alluvial deposits were derived from a wide range of rock types that comprise the Franciscan Group, which is the principal bedrock geologic unit exposed in the nearby part of the Santa Cruz Mountains west of the alignment. Bay Mud deposits are also present at the northern end of the alignment along US 101 in the vicinity of Charleston Slough. Bedrock is exposed near the surface in the southeastern portion of the project along US 101. In areas where the bedrock is not exposed, it is covered with alluvium that varies from approximately 20 to 150 feet thick at the Coyote Creek Bridge for northbound US 101 (URS, 2011). On the northeast side of US 101, from the SR 85/US 101 Interchange to about ¼ mile south, Plio-Pleistocene-age loosely consolidated sandstone, shale, and gravel deposits (Jennings, 2010) are overlain by Jurassic- to Cretaceous-age rocks of the Franciscan Complex and serpentinite of unknown age. The Franciscan rocks and serpentinite have been thrust over the younger Santa Clara Formation by activity along the Coyote Creek fault, which parallels the freeway (see Figure 3A). This low-angle, northeastward dipping thrust fault is exposed in the existing highway cut (URS, 2011). A regional and site-specific discussion of faulting and seismicity is presented in Section 4.3. The reported distribution of the alluvial units in the vicinity of the route is delineated on Figure 3A and 3B, and descriptions of the units are presented below.

Bay Mud - Although not shown on Figure 3A due to the regional scale of the map, Bay Mud deposits have been mapped by Helley et al. (1994) along US 101 between about San Antonio Road and the Oregon Expressway. These materials consist of soft elastic silt and clay with thin silty sand interbeds.

Alluvium Deposits (Q) - Alluvium deposits as characterized by Brabb and Graymer (1998) consist of clay, silty clay, sandy clay, with minor gravel and sand. The results of recent field observations revealed that reworked gravel and cobbles are particularly concentrated in the various drainage channels that cross the route. These deposits are of Holocene geologic age.

Older Alluvium and Fan Deposits (Qoa) - Older alluvium and alluvial fan deposits consist mainly of coarse sand and gravel with interbeds of silt and clay, which are overlapped by young alluvial deposits. They are generally of Pleistocene geologic age. Between Burnett Avenue and the southern extent of the alignment, US 101 is underlain by thick older alluvial fan and fluvial deposits of Pliestocene age (see Figure 3A).

2.2 SITE AND SURFACE CONDITIONS

The total project area extends a distance of 37.65 miles (includes 36.55 miles on US 101 and 1.1 miles on SR 85) and is located south of San Francisco Bay. The profile along the project alignment varies from depressed sections as much as 20 feet below surrounding development to embankments as high as 34 feet. Development in the project area includes the freeway, numerous overcrossings and undercrossings, roadway interchanges and freeway interchanges and bridges over creeks, rivers and railroad crossings.

2.3 SUBSURFACE CONDITIONS

2.3.1 Previous Studies

Geotechnical information was available for a majority of the US 101 project alignment. The Materials Reports, Geotechnical Design Investigations and Foundation Investigation Reports reviewed or used for this study are listed in Table 2-1. Section 2.3.2 presents a discussion about subsurface conditions based on review of this existing data. Readily available LOTBs and Standard Soil Survey Sheets from these sources are included in Appendix A, as well as vicinity maps for Materials Reports. Table A-1 in Appendix A lists LOTBs and some vicinity maps for 25 projects along US 101; these LOTBs and vicinity maps are assigned Project Numbers (PN) PN01 through PN25. PN01 begins at the south end of the project alignment at Dunne Avenue, with PN increasing numerically in a northerly direction. Readily available Materials Reports (in lieu of individual Foundation Investigation Reports) were consulted to develop the description of subsurface conditions in Section 2.3.2.

Table 2-1 Previous Studies

PM	Report	Author	Date
US 101 PM R27.0/34.8	Route 101 Widening Project Materials Report, Blossom Hill Road to I 280/680 Interchange, MSA No. 203A	Terratech, Inc.	November 25, 1987
SR 85 PM 0.0/1.4	Route 85 Materials Report, Route 85/101/Bernal Road I.C. (PM 0.0) to Miyuki Drive (PM 1.4), Volume 1A, Route 85 Transportation Corridor, MSA No. 100/Contract No. 155-085, Santa Clara County, California	Terratech, Inc.	October 12, 1988 and October 20, 1989
US 101 PM 34.8/40.7	US 101 Materials Report, 0.1 Mile North of Route 880 to 0.1 Mile South of Trimble Road/De La Cruz Boulevard Overcrossing; 0.2 Mile South of Story Road Overcrossing to Silver Creek Bridge; Julian Street/McKee Road Overcrossing to 0.2 Mile South of East Hedding Street Overcrossing; and South of Union Pacific Railroad to 0.1 Mile North of Julian Street/McKee Road Overcrossing, Respectively. MSA 202-4, 202-7, 202-10 and 202-11.	Associated Geotechnical Engineers, Inc.	February 7, 1990

Table 2-1 Previous Studies (Continued)

PM	Report	Author	Date
US 101 PM 37.2/38.1, PM 38.1/38.8	Route 101 Widening Materials Report, Santa Clara County. MSA 202-2 and 202-3.	Dames & Moore	March 12, 1990
	Foundation Investigation, Platform Stations, Tasman Corridor LRT, Santa Clara County, California	Woodward – Clyde Consultants	May 17, 1994
	Foundation Investigation, Moffett Field Overhead Tieback Wall, Bridge No. 37-72, Mountain View, California	Woodward – Clyde Consultants	February 3, 1995
	Final Report, Geotechnical Investigation Moffett Field Depressed Track, Tasman West Light Rail Project, Mountain View, California	Woodward – Clyde Consultants	January 30, 1997
US 101 KP 28.7/40.7 (PM) 17.83/25.29	Geotechnical Design & Materials Report, Route 101 South Widening from 0.1 km North of Cochrane Road to 0.1 km North of Metcalf Road, Santa Clara County, California	Parikh Consultants, Inc.	June 11, 2001
US 101 PM 48.3/49.2	Geotechnical Design Report, Route 85/101 Interchange Project, State Routes 85 and 101, Santa Clara County, California	Kleinfelder, Inc.	October 21, 2002
US 101 PM 48.7/51.9	Geotechnical Design and Materials Report US 101 Auxiliary Lanes from Embarcadero to SR 85, Palo Alto, Mountain View, Santa Clara County, California	URS	November 5, 2010
US 101 PM Varies	Plans, including LOTBs for East Dunne Avenue OC (37-334), 1968, 1997 and 1998; East Main Avenue OC (37-335), 1968; Cochrane Road OC (37-341), 1968; Route 82/101 Separation (37-348 R and L); Lafayette Street (Santa Clara – Alviso Road) OC (37-170), 1956; San Tomas Aquino Creek Bridge Widen (37-41 R/L), 1977; Bowers Avenue OC (37-390), 1973; Lawrence Expressway OC Replace (37-152), 1995 and 1964; Fair Oaks Avenue OC (37-168), 1958.	Caltrans	See "Report" column

Horizontal and vertical datums were not readily available in these studies or LOTBs.

2.3.2 Subsurface and Groundwater Conditions

Localized embankments and approach fills are present along the entire length of US 101 in the southern project limit. The embankments and approach fills, which range in thickness from about 2 to 30 feet, generally are composed of moderately to well compacted gravelly and sandy clay to clayey gravel derived of local borrow from the surrounding area.

Beginning at Dunne Avenue (south end) and continuing northward to about Metcalf Road, available boring logs reviewed (studies at East Dunne Avenue, East Main Avenue, Cochrane

Avenue and Materials Report, Parikh, June 11, 2001) typically reveal several feet of surficial silt underlain by granular layers to depths on the order of 50 feet. In general the granular layers include clayey sand, silty sand, well graded sand, poorly graded sand, silty gravel and sandy gravel, all ranging from dense to very dense in relative density. A few interbeds of lean clay and fat clay were also encountered. These granular layers and cohesive interbeds are older alluvium and alluvial fan deposits discussed in Section 2.1.

Continuing northward from Metcalf Road to about Coyote Creek (near Hellyer Avenue), bedrock is exposed in existing cuts along the eastern foothills, south of Coyote Creek Bridge. (This reach includes US 101/SR 85 Interchange.) In these cuts Santa Clara Formation claystone, siltstone, conglomerate, and altered tuff deposits are overlain by Franciscan Complex sandstone and intrusive serpentinite. The Franciscan rocks and serpentinite have been thrust over the younger Santa Clara Formation by activity along the Coyote Creek fault, which parallels US 101 and is exposed near the top of an existing cut slope. Santa Clara Formation siltstone and claystone were encountered below alluvium in borings 1360/1 and BR 65/1 near Coyote Creek at a depth of approximately 20 feet (Materials Report, Terratech, October 20, 1989, see Figure A-1 in Appendix A). According to another Materials Report (Terratech, November 25, 1987), (1) two borings (76 and 141) in Package "B" encountered serpentinite bedrock at depths of 4 and 16 feet bgs, (2) "the serpentinite bedrock was encountered at shallow depths" (apparently 0 to 8 feet bgs) in Package "C" and (3) "the serpentinite bedrock will be encountered at variable depths ranging from 0 to 15 feet" in Package "D". As shown in Figure A-2 in Appendix A, the limits of Construction Packages "B," "C," and "D" include:

- "B" – Bernal Road to south of Coyote Creek;
- "C" – South of Coyote Creek to near Hellyer Avenue; and
- "D" – North of Hellyer Avenue to I-280.

These construction packages were planned for widening US 101 from four to six lanes to eight lanes after 1989.

Continuing northward, native soils within the US 101 study area are alluvial and fluvial deposits consisting predominantly of soft to very stiff lean clay overlying interbeds and discontinuous lenses of medium dense to very dense, silty and clayey sand and gravel, and firm to very stiff, lean clay and sandy clay. In particular, the available boring logs west of Guadalupe River to SR 237 occasionally reveal soft silt and fat clay layers in the upper 10 feet; west of SR 237 to Oregon Expressway these deposits of soft silt and fat clay in the upper 10 feet (and deeper) become more common. As discussed previously in Section 2.1, Bay Mud deposits occur between San Antonio Road and Oregon Expressway (URS, Materials Report, November 5, 2010).

A more detailed discussion of subsurface conditions based on existing LOTBs is presented below. The discussion is divided into reaches, beginning at the south end of the project.

2.3.2.1 Dunne Avenue to Cochrane Avenue

LOTBs (see PN01, PN02 and PN03 in Appendix A) were reviewed at three structure locations, including Dunne Avenue, East Main Avenue and Cochrane Avenue. Each structure location is discussed below.

At the Dunne Avenue overcrossing, four borings were advanced: B-1 (June 25, 1968, total depth 50 feet), B-1 (September 15, 1997, in 26 feet of embankment fill, total depth 72 feet), B-2

(September 12, 1997, in 26 feet of embankment fill, total depth 75 feet) and P-1 (August 20, 1998, disturbed auger sampling; total depth 40 feet). Within the embankment in the upper portion of borings B-1 (1997) and B-2 (1997), dense to very dense clayey sand with gravel fill occurred. Underlying the fill at these two locations and beginning at ground surface in borings B-1 (1968) and P-1 (1998) generally are granular layers ranging from dense to very dense clayey sand with gravel (up to 2 inches) and compact to very dense sandy gravel to the terminal depth of borings. An exception to this was a surficial layer 4 feet thick of clayey silt in boring B-1 (1968). Groundwater was measured at a depth of 34 feet bgs (Elevation 326 feet) in P-1 on August 20, 1998.

At the East Main Avenue overcrossing, boring B-1 (June 26, 1968, total depth 48 feet) was drilled in native soil consisting of 2 feet of clayey silt with gravel underlain by dense silty sand to 6 feet below ground surface (bgs) with deeper compact and dense sandy gravel extending to 37 feet bgs. Occurring below the gravel is slightly compact clayey silt and hard silty clay to the terminal depth of 48 feet. Although groundwater was not measured in B-1, it was measured in an onsite probe at Elevation 327.9 feet on June 27, 1968; this corresponds to a water depth of 44 feet bgs.

At the Cochrane Road overcrossing, two borings were advanced: B-1 (October 2, 1968, total depth 53 feet) and B-5 (October 3, 1968, total depth 48 feet). Underlying a surficial sandy silt layer 2 feet thick in both borings, were numerous compact to very dense silty gravel, sandy gravel and clayey gravel layers. A 2 foot thick cemented gravel layer was encountered in B-5 at a depth of 36 feet bgs. In boring B-1, two interbeds were encountered ranging from dense silty sand (8 feet deep) to compact silt (35 feet deep). Groundwater was not measured.

2.3.2.2 Cochrane Avenue to Metcalf Road

Based on the boring data (see PN04 in Appendix A) and laboratory test results (Parikh Consultants, Geotechnical Design & Materials Report, June 11, 2001), this reach is generally underlain by localized fill overlying native alluvial soils and/or Santa Clara Formation. Localized fill was encountered in borings B-5, B-9 to B-11, B-13 to B-17, B-20, B-26 to B-30, CC-l, CC-2, UC-1, UC-2, CG-1 and CG-2. US 101 in this reach appears to be constructed on a 6.5 to 13-foot high embankment. The embankment fill encountered in the borings mainly consists of stiff sandy lean to fat clays containing sand and gravel, clayey sands with gravel and occasionally silty sands with gravel. Cobbles were encountered in the fill in the vicinity of borings B-5 and B-9. The embankment fill is generally stiff to hard and medium dense to dense. It should be noted that other fills, not encountered during this investigation may be located on the site.

Native, alluvial soils were encountered below the localized fill in borings drilled within the valley floor. These alluvial soils generally consist of alternating layers of clays and sands/gravels. The thickness of this unit generally varies from a few feet at higher elevations to about 20 feet at Coyote Creek. The alluvial unit generally consists of interbedded sands, gravels and clays. The clays encountered in this unit have low to high plasticity and contain considerable amounts of sand and gravel. The sand and gravel layers are randomly dispersed and appear to be discontinuous. The sands and gravels encountered in the borings are medium dense to dense.

The alluvial deposits at the site are underlain by the Santa Clara Formation. This formation is also exposed in the cuts located at higher elevations, where the highway is aligned within the hillside slopes. The unit encountered in the borings consists of alternating layers of weakly cemented siltstone, sandstone and claystone. Sampler refusal was met at some locations within the unit. Since the formation layers are weakly cemented or poorly lithified (i.e., the properties are more like soil than rock), they have been classified in accordance with the Unified Soils Classification System on the LOTBs. Based on this classification system, the Santa Clara Formation generally consists of siltstone, sandstone and claystone with a consistency comparable to hard clays or a relative density comparable to very dense sands.

Groundwater was encountered in borings RW-3 to RW-7, CC-l, CC-2, UC-1, UC-2, CG-l, CG-2, EB-l, EB-3, EB-5, EB-6, and EB-8 during drilling. The groundwater level measured at the site varies from Elevations 239 to 344 feet. It is anticipated that groundwater level will vary due to seasonal groundwater fluctuations, surface and subsurface flow, ground surface run-off and other factors.

Thirteen R-value tests were conducted on samples collected at subgrade level. The R values of on-site soils range from 8 to 36. An R-value of 8 is in a localized area in the vicinity of B-3. The R-value of existing soils mostly ranges from 11 to 22. Fill is anticipated under most of the proposed pavement sections. An R-value of 15 was selected for pavement design.

A portion of the mainline was constructed with asphalt concrete pavement section for conformance with the existing section, whereas the remainder was constructed with a portland cement concrete (PCC) pavement section.

2.3.2.3 Metcalf Road to Blossom Hill Road

For this reach the following discussion has been divided into Existing Embankments and Approach Fills, Native Soils and Bedrock, and Bridge Locations.

Existing Embankments and Approach Fills

Soils along the proposed embankments and approach fills were explored by means of 59 borings ranging in depth from 26 feet to 126 feet bgs (Terratech, Materials Report, October 12, 1988 and October 20, 1989). See Figure 1 in PN05 in Appendix A, for location of this reach of US 101.

Localized embankments and approach fills are present along the length of US 101 that was studied for this Materials Report (between Station E4 1330+20 at Metcalf Road and E4 1471+56 south of Blossom Hill Road). The embankments and approach fills, which range in thickness from 4 to 25 feet, generally are composed of gravelly and sandy clay to clayey gravel derived from the surrounding area. Imported borrow fill materials for subgrade soils appear to consist generally of poorly graded gravel. Based on Terratech’s study of the embankment fill north of the Tenant Road undercrossing (Bernal Road), the embankment fills were found to have relative compaction between 85 and 96 percent.

Native Soils and Bedrock

Bedrock is exposed in existing cuts between Station E4 1356+40 (Right) 100 feet and E4 1370+00 RT 120 feet. In these cuts Santa Clara Formation claystone, siltstone, conglomerate, and altered tuff deposits are overlain by Franciscan Complex sandstone and intrusive serpentinite. The Franciscan rocks and sepentinite have been thrust over the younger Santa Clara Formation by activity along the Coyote Creek fault, which parallels the freeway and is exposed near the top of an existing cut slope. Santa Clara Formation siltstone and claystone were encountered below alluvium in borings 1360/1 (between two of the existing cuts) and in BR-65/1 (at Coyote Creek) at a depth of approximately 20 feet. According to information published by the California Division of Mines and Geology (Rogers and Williams, 1974), depth to bedrock in this study area west of Bernal Road is estimated to be approximately 300 feet bgs.

Native soils within the study area are alluvial and fluvial deposits consisting predominantly of soft to very stiff lean clay overlying interbeds and discontinuous lenses of medium dense to very dense, silty and clayey sand and gravel, and firm to very stiff, silty and sandy clay.

The upper stratum of silty clay ranges in thickness from about 20 to 36 feet and has low to medium plasticity. Clay in the upper several feet of this layer is generally very stiff to hard, probably as a result of dessication. Below this surface "crust," the clay generally becomes progressively softer with depth and ranges in consistency from very stiff to soft. Below the upper silty clay, soils consist predominantly of medium dense to very dense sands and gravels interbedded with lenses or layers of firm to hard clay with low to medium plasticity. Isolated lenses or layers of loose clayey or silty sand were encountered in some of the drill holes within the study area. Groundwater was encountered at the time of subsurface exploration from 23 to 78 feet bgs, corresponding to Elevations 119 to 196 feet. However, perched groundwater may be encountered at shallower depths during construction.

R-value tests were not performed on soil samples for this reach of US 101.

Bridge Locations

The following discussion describes subsurface conditions at pertinent bridge locations.

Subsurface conditions at the Coyote Creek Bridge (No. 37-346 L) site were explored by two borings (BL-60/1 and BL-65/1) ranging in depth from about 82 to 100 feet bgs. Soils encountered to a depth of about 19 feet in boring BL 60/1 (Elevation 219 feet), which was drilled near the south abutment, consisted primarily of stiff sandy gravelly clay fill with medium plasticity placed in the area of past quarrying (possibly a gravel pit). Between 19 and 24 feet bgs, loose silty sand was encountered. Soils below this depth consisted of stiff to hard clay with interbeds and lenses of medium dense sand and gravel. In boring BL-65/1 (Elevation 220 feet), which was located near the north abutment, dense to very dense sand and gravel were encountered to a depth of about 50 feet bgs. These materials were underlain by hard sandy clay from about 50 to 59 feet, by very dense sand from 59 to 69 feet, and by stiff sandy clay from 69 to at least 82 feet bgs. Groundwater was encountered at depths of 23 feet (Elevation 196 feet) in BL-60/1 and 34 feet (Elevation 186 feet) in BL-65/1. Groundwater levels for this area are controlled primarily by water levels in Coyote Creek. Historic records indicate groundwater levels have been as high as a few feet below original ground surface.

Subsurface conditions at the Coyote Creek Bridge (No. 37-346R) site were explored by three borings (BR-60/1, BR-65/1 and BR-70/1) ranging in depth from about 57 to 82 feet bgs. Soils encountered in boring BR-60/1 (Elevation 221 feet) consisted of approximately 10 feet of very stiff to hard clay fill underlain by 4 feet of medium dense silty sand over 6 feet of stiff to hard plastic clay. These soils are underlain by claystone of the Santa Clara Formation to a depth of 82 feet bgs. Soils encountered in boring BR-65/1 (Elevation 210 feet) consisted of 10 feet of dense to very dense sand and gravel underlain by hard clay with interbedded medium dense to dense sand to a depth of 20 feet bgs. Claystone of the Santa Clara Formation was encountered between a depth of 20 and 57 feet bgs where drilling refusal was met. In boring BR-70/1 (Elevation 222 feet) stiff to very stiff clay with interbedded sand and silt was encountered to a depth of 30 feet, underlain by 10 feet of medium dense clayey sand. Stiff to hard clay was encountered between 40 and 80 feet bgs, the maximum depth of exploration. Groundwater was encountered at depths of 32 feet (Elevation 189 feet) in BR-60/1, 23 feet (Elevation 187 feet) in BR-65/1, and 51 feet (Elevation 171 feet) in BR-70/1. Groundwater levels at the bridge site will be controlled primarily by water levels in Coyote Creek. Historic records indicate groundwater levels have been as high as a few feet below ground surface.

Below the Bernal Road undercrossing, two exploration borings (BR-85/1 and BL-90/1) were advanced to a depth of 101 feet bgs. Soils encountered to a depth of 74 to 84 feet in borings BR-85/1 (Elevation 209 feet) and BL-90/1 (Elevation 207 feet) consisted of 35 feet of firm to very stiff clay and stiff sandy silt overlying 45 to 50 feet of medium dense to very dense, clayey to silty sand with interbeds of clay and gravel. These soils were underlain by stiff to very stiff sandy clay to the maximum 101-foot depth. Groundwater was encountered at depths of 50 feet (Elevation 157 feet) in boring BL-90/1 and 75 feet (Elevation 134 feet) in boring BR-85/1. Historic groundwater levels for this section of highway were as shallow as about 15 to 20 feet bgs.

2.3.2.4 Blossom Hill Road (SR 82/US 101) Separation

At the Blossom Hill Road (SR 82/US 101) separation, field exploration (see PN06 in Appendix A), borings B-1 (November 19, 1970) and B-3 (November 20, 1970) were advanced to depths of 85 and 78 feet bgs, respectively. Subsurface soils consist of layers of loose to slightly compact silt, stiff silty clay, loose to dense fine to coarse sand, and silty sand with “pebble gravel.” Groundwater depths ranged from 15 to 25 feet bgs, corresponding to Elevations 183.3 and 175.2 feet, respectively.

2.3.2.5 Blossom Hill Road to I-280/I-680 Interchange

For this reach, the following discussion has been divided into Package “B,” Package “C,” and Package “D.” Subsurface conditions for the roadways and embankments within this project reach were explored by means of 169 borings ranging in depth from 2 feet in boring 206 to 44 feet in boring 1 (Terratech, Materials Report, November 25, 1987). See Figure 1 in PN07 in Appendix A for the location of this reach of US 101. Drilling was accomplished using a truck-mounted drill rig with 6-inch diameter continuous flight augers and 8-inch hollow stem continuous flight augers.

Package “B” Bernal Road to South of Coyote Creek

Subsurface conditions for the roadway improvements and embankments in Package “B” (near the Bernal Road undercrossing to south of the Coyote Road undercrossing) were explored by means of 37 drill holes ranging in depths from 11.5 feet to 16.5 feet.

In the southern portion of Package “B,” from Station E4 1432+12 to C5 369+00, drill holes were advanced in the median lane widening area. Based on drilling and field mapping, existing fill appears to vary in thickness from about 12 feet at the southern end of the lane widening (E4

1432+12) to about 3 feet at Station C5 369+00. Fill encountered generally consisted of stiff to very stiff gravelly clay with low to medium plasticity and medium dense clayey gravel.

These fills were underlain by interbedded alluvial deposits of firm to very stiff, clays and silts with medium to low plasticity to the maximum depth of exploration. Occasional lenses and layers of loose to medium dense sand and gravel generally less than 3 feet thick were encountered. Bedrock was encountered in borings 76 and 141 (between Stations C5 341+20, LT 75, and C5 340+50, LT 110) at depths of 4 and 16 feet, respectively. Near Station C5 340+50, serpentinite outcrops at the surface form the large rock slopes of the Edenvale Ridge at the northern end of Package “B” and at the southern end of Package “C.” Bedrock consists of a moderately to very severely weathered serpentinite. The serpentinite, which is highly variable, generally consisted of hard gravel and boulder size fragments in a clayey matrix. The serpentinite was judged to be easily rippable except in some of the harder zones where drilling and sampling refusal was encountered. These areas were judged to be rippable with conventional equipment, but with greater difficulty. Bedrock is overlain by layers of firm to stiff clay and gravelly clay with medium to high plasticity, and by medium dense sandy and clayey gravel.

A series of seven relative compaction tests (California Test 216 and 231) were performed on surficial layers of fill between Stations E4 1432+12 and C5 347+19. Testing indicated that relative compaction of the existing fill ranged from 86 to 96 percent, with an average of approximately 93 percent.

Seven R-value tests were performed on the surface soils between Stations E4 1444+02 and C5 347+19. R-values of the native clayey soils ranged from 9 to 14, whereas R-values for the gravelly clayey fills ranged from 21 to 34.

No free groundwater was encountered in the borings at the time of drilling.

Package “C” South of Coyote Creek to near Hellyer Avenue

Subsurface conditions for the roadway improvements and embankments in Construction Package “C” (between Stations C5 277+00 and C5 340+00, i.e. south of Coyote Road to north of Hellyer Avenue) were explored by means of 50 borings ranging in depth from 7.5 feet to 44 feet.

In the southern portion of Construction Package “C,” from Station C5 340+00 to C5 320+00, bedrock was encountered at shallow depth. The serpentinite bedrock was described in the subsurface conditions for Construction Package “B.” Borings made in the areas of the proposed roadway widening and retaining walls encountered between 0 and 8 feet of stiff to very stiff, gravelly clay fill with medium plasticity overlying bedrock.

In the vicinity of the Coyote Road undercrossing near Station C5 319+00, the serpentinite bedrock slopes down to the north toward Coyote Creek. Bedrock in this area is overlain by alluvial deposits from Coyote Creek or by fill from quarrying activities.

From Stations C5 316+55 to C5 309+00 embankments were constructed as part of another project in 1989 or later to support the widened section of US 101 between the Coyote Creek Bridge and the Coyote Road undercrossing.

From Station C5 309+00 to C5 277+00 sandy gravel/gravelly sand fills were encountered to a depth of 11.5 feet, the maximum depth of exploration for the pavements in this area. Based on information from drilling and field mapping, we estimate fill thickness reaches a maximum of approximately 16 feet along the roadway. These fills are underlain by interbedded alluvial deposits of firm to very stiff silts and clays and loose to medium dense sands overlying moderately to very severely weathered serpentinite bedrock.

A series of four relative compaction tests (California Test 216 and 231) were performed on the surficial fill between Stations C5 306+36 and C5 382+91. Testing indicated relative compaction of the fill ranged from 89 to 92 percent, with an average of approximately 90 percent.

Five R-value tests were performed on the surface soils between Stations C5 306+36 and C5 278+75. R-values of the sandy gravels and gravelly clayey fills ranged from 20 to 28.

Package "D" North of Hellyer Avenue to I-280

From Stations C5 277+00 to C5 240+00 in Construction Package "D" (north of Hellyer Avenue to I-280/I-680 interchange) serpentinite bedrock will be encountered at variable depths ranging from 0 to 15 feet. The overlying fill materials are composed of stiff to very stiff gravelly clay/clayey gravel, and medium dense sandy gravel and gravelly sand.

A series of six compaction tests (California Tests 216 and 231) were performed on the surficial fill between Stations C5 275+47 and C5 243+87. Testing indicates relative compaction of the fill ranges from 90 to 98 percent, with an average of approximately 94.5 percent.

Five R-value tests were performed on the surface soils between Stations C5 275+47 and C5 243+87. The R-value of a clay sample at Station C5 271+99 was 10. R-values for the remaining four tests on clayey gravels, sandy gravels and weathered bedrock ranged from 38 to 59.

Between Stations C5 243+87 and C5 49+00 maximum fill depths are approximately 4 feet. The fill varies from stiff to very stiff clay and gravelly clay with low plasticity to medium dense to dense clayey gravel.

Fill materials were underlain by alluvial deposits of soft to very stiff silts and clays with low plasticity to the maximum depth of exploration. Interbeds of loose to medium dense sand were also encountered.

Borings south of Station C5 168+00 encountered groundwater approximately 10 to 15 feet bgs. In this area loose sands interbedded with soft clays were encountered below the water table. Based on field and laboratory testing, the cohesionless soils appear to be potentially liquefiable. A few of the drill holes that were not drilled with hollow stem augers were prone to caving below groundwater.

A series of 15 relative compaction tests (California Tests 216 and 231) were performed on the surficial fill between Stations C5 228+97 and C5 62+94. Testing indicated relative compaction of the fill ranged from 92 to 100 percent, with an average of approximately 95 percent.

Fifteen R-value tests were performed on surface soils between Stations C5 228+97 and C5 62+94. Four low R-values were measured: 4 at Station C5 123+48; 5 at Station C5 142+86; 11 at Station V 92+99; and 12 at Station C5 163+10. The remaining 11 R-values in this reach of the highway ranged from 16 to 33.

2.3.2.6 I-280/680 Interchange to Hedding Street (Berryessa Road)

For this reach, the following discussion has been divided into Roadways and Embankments and then each of these has been subdivided into Master Service Agreement (MSA) numbers. See Figure 2 in PN08 in Appendix A for a breakdown of the MSA subreach locations. R-value test results are subsequently discussed.

Roadways

Subsurface conditions at each of the following two roadway locations are discussed below.

Below the bottom of the existing pavement structural section between north of I-280/680 and McKee Road (MSA 202-7, MSA 202-8 excluded), subsurface conditions as encountered in eight borings consisted generally of native soils of stiff silty clays to a depth of over 5 feet bgs (Associated Geotechnical Engineers, Inc., Materials Report, February 7, 1990).

Free groundwater was not encountered in the exploratory borings.

Below the existing pavement structural section between McKee Road and south of Berryessa Road (MSA 202-10 and MSA 202-11), subsurface conditions, as encountered in six borings, consisted generally of poorly compacted (with relative compaction of 85 percent or less) fill materials to a depth of 4 to over 5 feet bgs. The fills consisted generally of sands with various amounts of gravels and are situated in approximately the southern two-thirds of this reach. The remaining approximately northern one-third of the reach was underlain by native soils mainly of stiff silty clays to locally stiff clayey silts.

Free groundwater was not encountered in the exploratory borings.

Approximately 2,500 feet of the existing US 101 pavement between Silver Creek and McKee Road (MSA 202-10) were removed and reconstructed with new structural section after 1990. The various types of material, used to construct the existing structural section, consist of asphalt concrete, portland cement concrete (PCC), aggregate base, aggregate subbase, and underlying fill materials. These materials may potentially contain asbestos-bearing soils as discussed in a May 16, 1988 report, "Preliminary Geotechnical Findings, Technical Memorandum No. 1," prepared by Associated Geotechnical Engineers, Inc.

Asbestos identification tests were made to evaluate the presence of asbestos-bearing soils. The results of the tests indicate that they do not appear to exist in the materials used to construct the existing pavement structural section.

It should be noted, however, that asbestos-bearing soils may exist in the embankment fill. The existing San Antonio Street overcrossing is an example, as evidenced by a serpentine-bearing gravel sample obtained in boring EB-69, between depths of 3 to 7 feet (at approximately Elevations 104.7 to 108.7 feet). The embankment fill is believed to have been imported.

Embankments

Subsurface conditions at each of the following six embankment locations are discussed below.

Subsurface conditions at the Julian Street/McKee Road overcrossing (MSA 202-7, MSA 202-8 excluded), as encountered in borings EB-58, EB-61, EB-62, and EB-83, consist generally of poorly to moderately compacted fills to a depth varying from a low of 0 to 5 feet bgs in the proposed western embankment area to a high of 8 to 14 feet in the proposed eastern embankment area. The fill consists primarily of clayey to gravelly sands, underlain by stiff to very stiff, medium plasticity silty clays to the maximum depth explored of 44 and 80 feet. Locally, approximately 10 feet of thick soft, medium plasticity, and moderately compressible silty clays were encountered in two borings of the proposed new overcrossing alignment, EB-61 and EB-83, at a depth of 10 to 19 feet bgs.

It should be noted that an existing 24-inch diameter sewer pipe was encountered and damaged during the drilling of boring EB-58 on December 12, 1988. The top of the sewer pipe was approximately 10 feet bgs. At the time excavation was made to repair the damaged pipe, a strong odor of gasoline was noticed and later perched water mixed with gasoline was observed in the area around the damaged pipe as soon as the pipe was exposed on December 28, 1988. The repair work consisted of excavating an approximately a 5-lineal-foot section of the sewer on each side of the damaged area, placing sheet metal over the hole of the damaged sewer, and installing a reinforced concrete cap prior to backfilling with properly compacted soils. The observation of gasoline-contaminated water in the area around the damaged pipe was reported to Parsons Brinckerhoff Quade & Douglas, Inc., City of San Jose Neighborhood Maintenance, and the Santa Clara Valley Water District, in Terratech's December 30, 1988 letter.

Free groundwater was encountered in three of the four borings (EB-61, EB-62, and EB-83) at an average depth of 24 feet bgs (approximately Elevation 66.9 to 79.5 feet). Groundwater was not encountered in boring EB-58.

Subsurface conditions, as encountered in borings EB-68, EB-69, EB-71 and EB-72 at the San Antonio Street overcrossing (MSA 202-7, MSA 202-8 excluded), consisted generally of moderately compacted fills of gravelly sands to sandy gravels to depths varying from a low of approximately 4 to 9.5 feet bgs near both sides of the existing embankment approaches, to a high approximately averaging 23 feet in the area adjacent to both ends of the existing overcrossing structure. The fill is underlain generally by stiff to very stiff, medium plasticity silty clays to the maximum depth explored of 44 and 80 feet. A layer of soft, medium plastic and moderately silty to sandy clay was encountered at an average depth of approximately 15 feet below the bottom of the existing fill. This soft layer varies in thickness from 10 to 17 feet.

Free groundwater was encountered in all four borings at depths of approximately 24 to 38.5 feet bgs (approximately Elevation 66.4 to 76.4 feet).

Subsurface conditions at the Mabury Road/Taylor Street overcrossing (MSA 202-10 and MSA 202-11), as encountered in borings EB-32 through EB-35, consisted generally of poorly to moderately compacted structural fill to a maximum depth of approximately 23 feet bgs in borings EB-33 and EB-34, and approximately 5 to 15 feet in borings EB-32 and EB-35. The fill consists generally of gravelly sands, both fine to coarse grained. The fill is underlain by stiff to very stiff, medium plasticity silty clays, with zones of slightly compact fine sands and hard silty clays, to the maximum depth explored of 80 feet bgs.

Free groundwater was encountered in all four borings at depths of approximately 24 to 70 feet bgs (Elevation 35 to 57 feet).

Subsurface conditions at the Coyote Creek Bridge (MSA 202-10 and MSA 202-11), as encountered in borings EB-39 and EB-42, consist of reasonably well compacted structural fill consisting of silty to clayey fine sands to maximum depths of approximately 12 to 15 feet bgs.

The fill is underlain by stiff to hard, medium plasticity silty clays, with zones of slightly compact to compact clean to silty fine sands, to the maximum depth explored of 80 feet bgs.

Free groundwater was encountered in the two exploratory borings at a depth of approximately 50 to 53 feet bgs (Elevation 34 to 39 feet) at the time of drilling.

Subsurface conditions at the existing embankments near both ends of the existing Union Pacific Railroad underpass and shoofly structure, i.e. temporary detour railroad bridge, (MSA 202-10 and MSA 202-11) differed greatly in terms of consistency:

- Subsurface conditions at the eastern embankment, as encountered in borings EB-43 and EB-45, consisted generally of native soils of stiff to very stiff, medium plasticity silty clays to an average depth of approximately 9 feet bgs. These clays were underlain by slightly compact clayey fine sands to an average depth of approximately 14 feet, followed by stiff to very stiff, medium plasticity silty clays to the maximum depth explored of 30 feet.
- Subsurface conditions at the western embankment, as encountered in borings EB-44A, EB-46, and EB-46A, consisted generally of soft to very soft, low plasticity and moderate compressibility, silty clays to depths of 18 to 21 feet below the existing ground surface at approximately Elevation 73.6 feet. The top approximately 1 foot of soil in boring EB-46, consisted of loosely compacted clayey gravel fill. The silty clays were soft near the surface, but became stiff to very stiff to the maximum depths explored of 30 to 44 feet.

Free groundwater was encountered in the two exploratory borings in the eastern embankment area at a depth of 20 feet bgs (approximate Elevation 65 feet) at the time of drilling. In the western embankment, groundwater was encountered in borings EB-44A and EB-46A, at a depth of 24 to 26 feet bgs (approximate Elevations 47.6 to 49.6 feet).

Subsurface conditions at Silver Creek Bridge (MSA 202-10 and MSA 202-11), as encountered in borings EB-48 and EB-49, consisted generally of moderately compacted sandy fine gravel fill to an average depth of approximately 6 feet bgs. The fill was underlain by native soils of soft, moderate plasticity silty clays to an average depth of approximately 38 feet, followed by similar clays except that they became stiff to the maximum depth explored of 80 feet.

Free groundwater was encountered in the two exploratory borings at depths of 22 to 25 feet bgs (approximate Elevation 62 feet).

R-Value Test Results

Twenty R-Value tests were made on selected samples of subgrade supporting soils beneath existing pavement sections to provide data for pavement design. R-values ranged from about 10 to 26 in the clays and silts; the higher R-value of 26 was attributed to samples with gravel. For granular soil samples, R-Values ranged from about 27 to 72; an exception to this was a serpentinite aggregate sample with an R-Value of 78.

2.3.2.7 Hedding Street (Berryessa Road) to I-880

For this reach, the following discussion has been divided into Roadways, Embankments and R-value test results.

Roadways

Subsurface conditions at each of the following roadway locations are discussed below (Dames & Moore, Materials Report, March 12, 1990). See Sheet No. 1 in PN09 in Appendix A (Project Plans for Construction) which show station numbers and limits of MSA 202-2 and MSA 202-3.

An entire new traveled way was constructed after 1990 (in MSA 202-3, PM 38.8/38.0) from conform point B_{13} Station 395+00 to conform point Bo Station 422+40. Existing US 101 may be lowered as much as 4 feet to provide a 16.5 foot minimum vertical clearance under the I-880 separation. The proposed roadway widening between B_{13} Station 391+00 and the US 101/I-880 interchange, as well as the interchange loops and ramps, necessitated new pavements constructed on new embankment fills adjoining the existing roadway embankments. The existing shoulder sections between B_{13} Station 392+00 and B_{13} Station 395+00 and between B_o Station 422+40 and end of segment MSA 202-3 were also incorporated in the US 101 widening project after 1990. In addition, the proposed roadway widening between B_{13} Station 422+40 and end of segment MSA 202-3 may occupy the space that will be produced as a result of excavation into existing cut slopes. Subsurface conditions for this roadway segment are summarized below in Table 2-2.

Table 2-2 Subsurface Conditions in MSA 202-3

Station/Location	Comments	Roadway Subgrade Materials/Groundwater
B_{13} Station 395+00 to B_o Station 422+40	New traveled way to replace existing section	Stiff to very stiff silty clay (existing roadway embankment or native material) of low to medium plasticity, with occasional sand interbeds.
	Widening on the outside	New embankment
	Pavement grade: Elevation 51@ B_{13} Station 395+00 to Elevation 35@ B_o Station 422+40	Groundwater ranged from Elevation 25 @ B_{13} Station 395+00 to about Elevation 30 @ B_o Station 422+40
US 101/I-880 interchange ramps and loops		Existing stiff to very stiff silty clay fill of low to medium plasticity; and/or new embankment fill.
	Pavement grade: varies from Elevation 45 to 70	Groundwater ranged from about Elevation 21 to 34
B_o Station 422+40 to end of Segment MSA 202-3 at		North side – stiff silty clay of medium to high plasticity.

Table 2-2 Subsurface Conditions in MSA 202-3 (continued)

Station/Location	Comments	Roadway Subgrade Materials/Groundwater
B_o Station 428+60		South side – soft to stiff silty clay of medium to high plasticity.
	Pavement grade: Elevation 35 to 33	Groundwater @ about Elevation 31

The proposed roadway widening for the entire segment (in MSA 202-2, PM 38.0/37.2) will incorporate the existing shoulder sections and may occupy the space that will be produced as a result of excavation into existing cut slopes. Subsurface conditions for this roadway segment are summarized below in Table 2-3.

Table 2-3 Subsurface Conditions in MSA 202-2

Station/Location	Comments	Roadway Subgrade Materials/Groundwater
B_o Station 428 +60 to B_o Station 436+50		Stiff silty clay of medium to high plasticity.
	Pavement grade: Elevation 35 to 41	Groundwater ranged from Elevation 30 to 34.
B_o Station 436+50 to end of segment of MSA 202-2 at B_o Station 470+85		Stiff to very stiff silty clay/ clayey silt with fine sand.
	Pavement grade: Elevation 41 at B_o Station 436+50 to Elevation 80 at B_o Station 470+85	Groundwater ranged from Elevation 30 at B_o Station 436+50 to Elevation 50 at B_o Station 470+85

Embankments

Subsurface conditions at each of the following embankment locations are discussed below.

Between the Fourth Street on-ramp and US 101/I-880 interchange, the existing US 101 roadway was built on 6 to 8 feet of stiff silty/sandy clay fill. The embankment fills encountered at the US 101/I-880 interchange loops and ramps typically consist of silty to sandy clay with sand and gravel. These fills vary in height from a few feet to 25 feet; and are stiff to very stiff in consistency. From the North Fourth Street on-ramp to US 101/I-880 interchange, two layers of native silty clay are identified as potentially compressible layers in which consolidation settlements will take place under the new fill loads. The upper clay layer, encountered between approximately Elevation 45 and 30 feet, typically ranges from stiff to very stiff in consistency. These clays were damp to moist, ranging in moisture content from 15 to 25 percent, and are of low to medium plasticity. The lower clay layer, encountered between about Elevation 30 and 5 feet, ranges from soft to stiff in consistency. These clays are moist and fully saturated and range in moisture content from 30 to 40 percent, and are of medium to high plasticity. Below the clay, dense to very dense sand and gravel were revealed in the deep exploratory borings drilled near North Tenth Street. The groundwater level measured at the time of field exploration ranged from about Elevation 21 to 34 feet.

The subsurface soils encountered on the north and south sides of North Tenth Street overcrossing are quite similar. Dense sandy to clayey gravel fill was encountered to a depth of about 3 feet bgs (Elevation 55 feet). Beneath the fill, stiff silty clays of medium plasticity with occasional clayey/silty sand lenses were encountered to a depth of about 23 feet (Elevation 32 feet). Below Elevation 32 feet, silty clays of increasing moisture content and plasticity were encountered to a depth of about 43 feet (Elevation 12 feet). These clays were considered to be soft to stiff in consistency and moderately compressible. Below Elevation 12 feet, dense to very dense clayey and silty gravelly sands were encountered to the maximum depth of exploration at Elevation -25. No groundwater measurements were made because the rotary mud method of boring was used. Based on nearby borings where groundwater was measured, levels near North Tenth Street were approximately at Elevation 30 feet.

The profile grade on North Tenth Street will be raised a maximum of 4 feet to provide greater vertical clearance, keeping the existing US 101 profile grade unchanged.

Based on the proposed profile for the Old Oakland Road overcrossing, the approach embankment will be raised 4 to 5 feet. The on/off ramps at Old Oakland Road will be revised to facilitate the widening on US 101. Fill was encountered to a depth of about 4 to 8 feet bgs (Elevation 65 feet at bridge abutment). The fills encountered typically consist of stiff clayey silt with some gravelly and sandy interbeds. Below the surficial fills, stiff to very stiff silty clays of medium to high plasticity were encountered to the maximum depth of exploration of about 51 feet. These clays were dry to damp below the fill to moist below about Elevation 50 feet. Below Elevation 34 to 38 feet, the silty clays are more plastic and become fully saturated. It is anticipated that sand and gravel will be encountered below Elevation 5 feet based on deep exploratory borings drilled near North Tenth Street.

Groundwater was encountered from Elevation 35 feet on the south side to Elevation 28 feet on the north side.

Soils beneath the approach to the Hedding Street/Berryessa Road overcrossing (south abutment) are characterized by stiff clayey silts to silty clays to a depth of about 10 to 13 feet bgs (Elevation 77 feet). These clayey silts were dry to damp. Between 13 and 16.5 feet below grade (Elevation 64 to 60.5 feet) in boring 460 at B_o Station 460+42, an apparent void was encountered (Dames & Moore Information Memo dated January 4, 1989). Circulation of drilling fluid was interrupted and the drill stem fell freely approximately 3.5 feet. Samples taken at both 10 feet and 16.5 feet below grade showed no signs of an underground utility, and had no apparent odor.

In view of the close proximity to an existing 30-inch VCP sanitary sewer line, a video inspection of the line was made on March 28, 1989. No signs of distress or joint openings were noted in the vicinity of Station 460+42, which would account for the apparent void encountered during drilling. Details of the video inspection of the sanitary sewer line were presented in Dames & Moore's Information Memo dated March 28, 1989.

Below the void, stiff to very stiff silty clays of medium to high plasticity were encountered to the maximum depth of exploration. These clays typically increase in moisture content with depth. A clayey sand lens was encountered between depths of about 30 and 31 feet (Elevation 47 to 46 feet).

On the north side of US 101, the Berryessa Road overcrossing approach is underlain by gravelly clay fill from the surface to 3 feet bgs (Elevation 78 feet). Stiff clayey silts to silty clays were encountered below the fill to a depth of about 12 feet (Elevation 66 feet). Compact silty sand lenses were encountered between Elevation 63 and 62 feet. Below the clayey silts to silty clays, stiff to very stiff silty clays of medium to high plasticity were encountered to the maximum depth of exploration at 43.5 feet.

Groundwater is estimated at about Elevation 40 feet, based on nearby exploratory borings where groundwater measurements were made.

Based on the proposed profile for Berryessa Road, the approach embankment will be raised a maximum of 4 feet.

R-Value Test Results

Twelve R-value tests were performed on soil samples recovered from depths of 1 foot to 20 feet bgs. For predominately cohesive soils, the R-values ranged from a low of 13 to a high of 24, whereas granular soils ranged from 66 to 78. An R-value of 15 was recommended for use in design.

2.3.2.8 North Fourth Street to West of Guadalupe River

For this reach, the following discussion has been divided into Roadways, Embankments and R-value test results. See PN10 in Appendix A for a breakdown of the MSA subreach locations.

Roadways

Subsurface conditions at each of the following three roadway locations are discussed below.

Below the bottom of the existing pavement structural section between North Fourth Street and Guadalupe River (MSA 202-4 and MSA 202-5), subsurface conditions as encountered in 12 exploratory borings, consisted generally of moderately to "adequate" (with relative compaction of 90 percent or greater) compacted fill materials to a depth of 4 to 8.5 feet bgs (Associated Geotechnical Engineers, Inc., Materials Report, February 7, 1990). The fills consisted generally of sands with variable amounts of clays and silts in the northern portion of this segment (both sides of Guadalupe River) and in the southern portion between North First Street and North Fourth Street. The fills in the remaining area of the segment consisted generally of low to moderately compacted silty clays. It should be noted, however, that the upper 5 to 8.5 feet of fill materials in borings EB-4 and EB-21 were described as poorly compacted and moderately compacted (with relative compaction of 85 percent but less than 90 percent), respectively.

Free groundwater was not encountered in the exploratory borings.

Embankments

Subsurface conditions at each of the following embankment locations are discussed below.

Subsurface conditions, as encountered in borings EB-2 and EB-3 at the Guadalupe River Bridge (MSA 202-4 and MSA 202-5), consisted generally of poorly to moderately compacted clayey fine to coarse sand fill to a depth of approximately 4 to 6.5 feet bgs. Subsurface conditions below the fill differ greatly between the western and the eastern sides of the existing bridge.

The upper surface fill in the eastern side of the bridge was found to be underlain by alternating layers of soft, medium plasticity, and somewhat compressible silty clays and compact clean to silty fine sands to a depth of approximately 38.5 feet, and followed by a relatively thick layer of compact clean to silty fine sands to a depth of approximately 68 feet. Below 68 feet, hard sandy clays were encountered to the maximum depth explored of 80 feet.

Below the upper surface fill in the western side of the bridge, soils similar to those in the Eastern side were encountered except that the silty clays were stiff to very stiff and judged to be less compressible.

Free groundwater was encountered in the two exploratory borings, at the time of drilling, at an average depth of approximately 18.5 feet bgs (at approximately Elevation 19 feet).

Subsurface conditions at the SR 87/US 101 northbound ramp (MSA 202-4 and MSA 202-5), as encountered in borings EB-5 and EB-6, consisted generally of loosely to moderately compacted sandy clay fill to a depth of approximately 12 to 22 feet bgs. Subsurface conditions below the fill consist of 24 to 58 feet of predominantly medium to high plasticity silty clays interbedded with layers of compact, clean to silty fine sands. The silty clays were generally stiff except in EB-5 at depths between 17 and 32 feet bgs, where they were found to be soft and somewhat compressible.

At the time of drilling, free groundwater was encountered in both borings at a depth of 23 to 34 feet bgs (at approximately Elevation 13.5 to 24 feet).

The proposed US 101 embankments adjacent to the Brokaw Road undercrossing (MSA 202-4 and MSA 202-5) are underlain by rather anomalous subsurface conditions. Borings EB-10, EB-14, and EB-17 through EB-20A, encountered poorly to moderately compacted structural fill, 3 to 37 feet thick. This fill was the thinnest near the northern limit of the US 101 embankment, gradually increased in thickness to 16 feet at approximately Station B14 697+00, remained 16 feet thick approaching the Brokaw Road undercrossing, and then increased to 37 feet thick near the North First Street undercrossing. The fill consisted mainly of gravelly, fine to coarse sands, locally with layers of soft to stiff silty clays. The bottom of the fill was generally level, situated at about Elevation 30 feet.

The fill was underlain by soft, highly plastic, moderately compressible, normally consolidated silty clays. These silty clays had an average thickness of 19 feet, except in the general vicinity of the proposed Brokaw Road undercrossing which was underlain by approximately 29 feet of soft moderately compressible silty clays.

These soft compressible silty clays were underlain by slightly compact to dense, silty to clayey to clean fine-grained sands in most areas where clays average 19 feet thick, except in the localized area between the proposed Brokaw Road undercrossing and the northern limit of the existing North First Street undercrossing, which was underlain by stiff to very stiff silty clays, interbedded with layers of sands, to the maximum depth explored of 80 feet bgs in the exploratory borings, EB-5 and EB-18. This localized area was underlain by approximately 29 feet of compressible silty clays.

Free groundwater was encountered in the six exploratory borings at the time of drilling at an average Elevation 11 feet.

Boring EB-20A was a replacement for boring EB-20. EB-20A was located away from the toe of the existing northbound embankment slope adjacent to the southeast corner of the existing North First Street undercrossing (MSA 202-4 and MSA 202-5). EB-20 was advanced at the northbound shoulder of existing US 101 near the intersection of the existing North First Street undercrossing and the existing northbound on-ramp from North First Street.

PCC was encountered in boring EB-20 after the auger penetrated through the upper surface of approximately 6 inches of asphalt concrete. To avoid the PCC pavement, the boring location was moved three times along the existing northbound shoulder and toward the south, each spaced at approximately 50 feet away from the previous one. Similar sections were encountered at all boring locations. Each boring was extended to the maximum depth explored of approximately 12 inches below the top of the existing pavement surface.

The results of a ground penetrating radar survey and small diameter core barrel sampling indicated that: (1) the PCC was an approximate 10-foot wide continuous mass following approximately the eastern one-half of the existing northbound North First Street on-ramp, (2) the PCC mass was at least 7 inches thick, and (3) the PCC mass, which is not shown on the as-built drawings, appears to be part of the original 8-inch thick PCC pavement section constructed in the mid 1950s.

Except for boring EB-20A free groundwater was encountered in the exploratory borings at the time of drilling at approximately 21.5 to 43.5 feet bgs (approximate Elevation 10.2 to 14.5 feet).

Two embankments are required along the eastern and western US 101 rights-of-way between Station B_{13} 380+50 and Station B_{13} 409+00. A total of six exploratory borings were made by Dames & Moore in the area between Station B_{13} 391+00 and B_{13} 409+00, designated as 393, 396, 402, 404, 497, and 410/3. Subsurface conditions as encountered in these exploratory borings consisted generally of stiff to very stiff, low to moderate plasticity silty clays to a depth of approximately 34 to 39 feet bgs (approximately Elevation 8 to 13 feet).

Free groundwater was encountered in these borings at a depth of approximately 13 to 21 feet bgs (approximately Elevation 26 to 34 feet).

R-Value Test Results

Twenty R-Value tests were made on selected samples of subgrade soils beneath existing pavement sections to provide data for the pavement design. The results of R-value tests varied generally from a low of 10 to 19 for predominantly silty to sandy clays, to a medium of 26 to 35 for predominantly sands and silty to sandy clays with various amounts of gravels, to a high of 43 to 78 for predominantly gravelly sands.

2.3.2.9 West of Guadalupe River to SR 237

LOTBs (see PN11, PN12, PN13, PN14 and PN15 in Appendix A) were reviewed at five structure locations, including Lafayette Street overcrossing, San Tomas Aquino Creek, Bowers Avenue overcrossing, Lawrence Expressway and North Fair Oaks Avenue. Each structure location is discussed below.

At the Lafayette Street overcrossing (referred to on LOTB as Santa Clara – Alviso Road overcrossing), two borings: B-3 (June 25, 1956, total depth 72 feet) and B-4 (June 20, 1956, total depth 93 feet) were advanced. Boring B-3 encountered layers of stiff to very stiff silty clay, sandy silt and clayey silt to a depth of 65 feet bgs underlain by compact sand and gravel to the terminal boring depth. At boring B-4, alternating layers of stiff to soft clay and silt and slightly compact to compact sand extended to a depth of about 45 feet bgs; deeper granular deposits of very dense silty sand and pebble gravel and mostly compact silty and fine sand occurred to the terminal boring depth. Groundwater was measured in two nearby probes at Elevations 27.3 (depth of 9 feet) on June 21, 1956 and 26.7 (depth of 10 feet) on June 26, 1956.

At San Tomas Aquino Creek Bridge, three borings were advanced: B-1 (October 25, 1977, total depth 90 feet), B-2 (October 18, 1977, total depth 85 feet) and B-3 (date unknown, total depth 130 feet). Predominately stiff to very stiff clays and silts were encountered in the three borings; occasional soft or very soft clays and silts to depths on the order of 45 feet were recorded. Granular interbeds, ranging in thickness from a foot to 15 feet, consisted of compact to dense sand and gravel. Groundwater was measured in B-2 on October 18, 1977, at Elevation 13 feet, corresponding to a depth of 14 feet bgs.

At the Bowers Avenue overcrossing, three borings were advanced: B-1 (November 7, 1973, total depth 146 feet), B-2 (November 7, 1973, total depth 95 feet) and B-6 (November 6, 1973, total depth 92 feet). Predominantly stiff to very stiff silts and clays were encountered; however a soft layer of clay was found in the upper 6 feet of boring B-1. Granular interbeds, ranging in thickness from 2 to 11 feet, generally included compact to dense sand and dense to very dense gravel. Groundwater was measured in three nearby probes, ranging from Elevation 13.0 feet (November 7, 1973) to Elevation 15.4 feet (November 6, 1973), corresponding to depths of 13.5 to 12.5 feet bgs.

At the Lawrence Expressway overcrossing, four borings were advanced: B-1 (January 26, 1995, total depth 110 feet), B-2 (February 2, 1995, total depth 200 feet), B-1 (June 15, 1956, total depth 85 feet), and B-5 (June 22, 1956, total depth 100 feet). The soil profile in 1995 borings B-1 and B-2 included 4 to 20 feet of clayey silt, silty clay and sandy gravel fill underlain predominately by native layers of stiff to hard silty clay and silt to terminal boring depth; granular interbeds 2 to 4 feet thick were encountered including mostly compact to very dense sand and gravel. However, at 1956 borings B-1 and B-5, soft to hard silt and clay was encountered to depths on the order of 30 to 40 feet bgs; below these depths numerous granular soil layers of compact to very dense sand and gravel occurred with a few interbeds of stiff to very stiff silt and clay. Groundwater was measured in borings B-2 (1995), B-1 (1956) and B-5 (1956) ranging from Elevation 0 to 19.5 feet, corresponding to depths of 8.5 to 24 feet bgs.

At the Fair Oaks Avenue overcrossing, two borings were advanced: B-1 (April 1, 1958, total depth 107 feet) and B-5 (April 2, 1958, total depth 95 feet). Soft silt fill to a depth of 6 feet bgs was encountered in boring B-1. Underlying this fill and beginning at ground surface in B-5, were predominately fine grained layers of soft to very stiff silt and clay; below a depth of 45 feet bgs in B-5 these deposits are described as “clayey sand and sandy clay.” Granular interbeds, 5 to 6 feet thick, were encountered in both borings and ranged from slightly compact to very dense sand and gravel. Groundwater was measured in B-5 at Elevation 22.9 feet on April 4, 1958; this corresponds to a depth of about 3 feet bgs.

2.3.2.10 SR 237 to SR 85

For this reach, the following discussion has been divided into Depressed Track and Moffett Field Station and Moffett Field Overhead. See LOTBs in PN16, PN17 and PN18 in Appendix A for boring locations.

Depressed Track and Moffett Field Station

For this reach existing subsurface information was limited to three subsurface investigations performed by URS' predecessor firm, Woodward-Clyde Consultants, for the Tasman West Light Rail Project, including (1) Moffett Field Depressed Track (MFDT) dated January 30, 1997; (2) Platform Stations dated May 17, 1994, which included recommendations for Moffett Field Station (MFS) as well as other stations; and (3) Moffett Field Overhead Tieback Wall (MFOTW), Bridge No. 37-72 dated February 3, 1995. The MFDT is positioned parallel to and north of Manila Drive and US 101 just south of the southernmost runways and taxiways at Moffett Field. Since the Moffett Field Station is positioned at the west end of the MFDT, the subsequent discussion will be combined for these two reports. The MFOTW is located under the Moffett Field Overhead (US 101 overcrossing of Ellis Street) on the east side of Ellis Street. This approach embankment is about 27 feet high.

The subsurface conditions encountered along the project alignment for the MFDT and MFS generally consist of an upper clay layer, typically stiff, with high plasticity, and varying in thickness from 3 feet to 10 feet. However, this highly plastic clay was not encountered at boring P-11. A surface layer of clayey gravel fill was encountered to a depth of 3½ feet at this location. Highly variable fill, ranging from sand and gravel to lean clay, was encountered in borings F12, F13A and F13B to depths of 1 to 2 feet. Underlying the high plasticity clay, and below the fill in borings P-11, F12, F13A and F13B, is generally a medium to low plasticity silty clay, varying in consistency from soft to medium stiff. The silty clay layer extends to a maximum depth of 23 feet bgs at boring P-13. The silty clay layer is underlain by a 5 to 10-foot thick medium dense to dense granular layer consisting of silty sand, clayey sand, well graded sand, poorly graded sand, and gravelly sand. The granular stratum was encountered at a fairly consistent depth of about 15 feet along the western portion of the structure alignment, but varies in depth along the eastern portion from 5 feet bgs at boring F14 (approximate Station 168+00) to 23 feet bgs at boring P-13 (approximate Station 171+70). The sand layer was silty at the top of the stratum and became denser with less silt and some gravel content with depth.

To the east of P-14, a fairly consistent layer of loose clayey sand, 15 ± feet deep, and about 4 to 6 feet thick was encountered. This layer is discontinuous near CPTs P-14C and P-13C. The clayey sand graded to a medium dense silty sand westward towards F14. The top of this silty sand deposit was about 5 and 11 feet bgs at boring F14 and CPT-14A, respectively; this silty sand deposit was not encountered at CPT F14C. Apparently cleaner and denser sand and gravel deposits occur below the silty and clayey sands, except in CPT P-13C; in boring P-13 this sand layer is described as loose on the boring log. It should be noted that boring P-13 was drilled with a hollow stem auger which has a tendency to disturb the soil during drilling, prior to sampling. This apparent disturbance was confirmed when CPT P-13C subsequently was pushed adjacent to boring P-13 and revealed the sand layer was medium dense.

Underlying the sand stratum is a medium to stiff silty clay extending to a depth of approximately 50 feet; interbeds of dense to very dense sand were observed within the silty clay layer. Directly below the silty clay deposit are alternating layers of sandy silt, silty sand and gravelly sand deposits extending to the maximum depth of exploration of 120 feet.

During the field investigations, most of the samples retrieved from borings P-13, P-14 and P-15 and some of the samples retrieved from boring F14 were monitored for detectable levels of organic vapors using an Organic Vapor Meter (OVM). All readings were 0 parts per million (ppm; non-detectable.)

Groundwater was initially encountered during drilling at depths ranging between approximately 10 and 17.5 feet bgs. Groundwater soundings performed 5 to 7 days after drilling on July 15, 1992, at monitoring wells MW-1 and MW-2 indicated that the water level rose to a depth of about 9 feet bgs. After completion of monitoring well MW-3, groundwater depths measured on October 24, 1993, in all three wells indicated that the groundwater depths ranged from about 7.7 to 8.7 feet bgs. These depths correspond to approximately Elevation 26 to 27 feet, based on ground surface at Elevation 35 feet. In subsequent borings F13A (November 27, 1996) and F13B (December 10, 1996), groundwater was encountered at depths of 10 and 6.5 feet, respectively. These depths correspond to approximately Elevations 24.5 and 27.3 feet.

Additional readings were obtained from December 16, 1993 through August 12, 1986. The highest groundwater level readings observed were on February 24, 1994, when the depths ranged from 5.6 to 6.3 feet bgs. These depths correspond to approximately Elevations 29.4 to 28.7 feet, based on ground surface at Elevation 35 feet.

Moffett Field Overhead

Based on the conditions encountered in borings P10, F10 and P9, the eastern approach embankment consists primarily of very stiff to hard cohesive fill materials that are mottled gray, brown, and black silty and sandy clays with traces of fine gravel. In general, the fill materials encountered are low to moderate plasticity clays; an exception is a high plasticity clay layer revealed in boring P10 between approximately Elevation 40 and 50 feet. In boring F10, some debris was encountered at a depth of approximately 27 to 29 feet below grade, corresponding to Elevation 34 to 32 feet. The debris generally consisted of concrete, rocks, and wood. Some rotary fluid loss was noted at that depth interval during drilling.

Native soils were encountered beneath the approach embankment (borings P9, F10 and P10) at approximately Elevation 29 to 32 feet, and in borings completed adjacent to Ellis Street (F9 and F11), near the toe of the approach embankment. They can generally be described as alluvial soils with alternating layers of clay, silt, sand and gravel. The upper 4 feet (down to Elevation 30 feet) of native soils are generally stiff to very stiff, moist, dark gray or black highly plastic clays. They are underlain by soft to stiff, moist to saturated, silty clays which extend to a depth of approximately 13 feet (Elevation 21 feet). To a depth of about 8 to 9 feet (Elevation 26 to 25 feet), these clays are also highly plastic; whereas below a depth of 9 feet, the clays are moderately plastic. These clays are generally underlain by medium dense to dense sands and gravels to depths of 26 to 30 feet (Elevation 8 to 4 feet). Below a depth of 30 feet (Elevation 4 feet), interbeds of highly variable alluvial materials generally continued to the terminal depths of exploration at approximately Elevation -29.5. These materials ranged from medium dense to dense sands and gravels to medium to very stiff silts and clays.

By comparison, the logs from the original exploration of the site in 1956 indicate approximately 8 to 9 feet of silt at the surface, overlying layers of sand with various amounts of silt and gravel to the terminal depth of their borings at Elevation -53 feet (elevation adjusted to Caltrans 1985 datum).

The rotary wash method of drilling was used to advance the borings in the approach embankment. For this reason, groundwater measurements could not be taken at the time of drilling. However, borings located adjacent to the toe of the embankment, near the natural grade, were drilled with solid flight augers until groundwater was encountered. Groundwater was encountered in borings F9 and F11 at a depth of approximately 10 to 12.5 feet below grade at the time of drilling (approximately Elevation 21.5 to 24 feet). Readings at nearby monitoring wells (MW-1, MW-2 and MW-3), installed along the Light Rail Transit (LRT) alignment adjacent to Moffett Field for the Depressed Track structure indicate groundwater levels ranging from approximately Elevation 24 to 27 feet. These wells are located at distances ranging from about 1,750 feet to 3,100 feet from the planned retaining walls, corresponding to Stations 167+50 to 181+50. It is conceivable that groundwater in borings F9 and F11 might also be somewhat higher, and it is reasonable to expect that water levels in the retaining wall area might rise to a depth of about 7 feet, corresponding to approximately Elevation 27 feet.

Exploration completed for the US 101 overcrossing in 1955 and 1956 indicates groundwater depths ranged in Elevation from 13.7 to 15.4 feet. However, using the Caltrans 1985 datum, the elevations on the original as-built plans should be lowered by 5.0 feet to Elevation 8.7 to 10.4 feet.

2.3.2.11 SR 85 Interchange to North Shoreline Boulevard

For this reach, the following discussion has been divided into Ramps and Structures and R-value test results. See LOTBs in PN19 in Appendix A for boring locations.

Ramps and Structures

The subsections that follow are brief descriptions of the subsurface soil conditions at each site (Kleinfelder, Inc., Geotechnical Design Report, October 21, 2002).

The northbound US 101 off-ramp/US 101 Separation (MIR3) site is underlain by alluvial material, which consists of layers of clay and sand. In boring B-1, located in the area of Abutment 1, a pavement section consisting of approximately 6 inches of asphalt concrete over approximately 30 inches of base was encountered. Below the pavement section in boring B-1 and in boring B-3 (located in the area of Abutment 4), a layer of high plasticity, stiff to very stiff clay was encountered to Elevation 12 and 17 feet, respectively. This clay was underlain by layers of clay, sandy clay, and silty sand to the maximum explored depths of about 101.3 feet. These clay layers were stiff to very stiff and the sand layers were medium dense to very dense.

CPT-2 and CPT-3A were located in the area of Bent 2 and Bent 3, respectively. Interpretation of material type suggests predominantly clayey soils with interbedded layers of sandy soils extended to the maximum explored depths of 100 feet and 97 feet.

At the Stevens Creek Bridge site boring B-22, located in the right shoulder of southbound US 101, an asphalt concrete section 1 foot thick was encountered at the ground surface. The asphalt concrete was underlain by medium dense clayey sand that extended to a depth of about 5 feet.

This sand was underlain by a layer of very stiff to hard fat clay of high plasticity to a depth of about 13 feet. Below the clay were interbedded layers of clay, silty sand, and clayey sand to the maximum explored depth of 61.3 feet. The clayey soil layers were generally stiff to very stiff in consistency and the sandy soil layers were generally medium dense.

In CPT B-21, located in the right shoulder of northbound US 101, the interpreted soil behavior types consisted of very stiff clayey soil from ground surface to a depth of 12 feet, underlain by dense sandy soil to a depth of 25 feet, very stiff clayey soil to a depth of 68 feet, medium dense sandy soil to a depth of 78.5 feet, and very stiff clayey soil to the maximum exploration depth of 100 feet.

In CPT B-49, located in the right shoulder of the Moffett Boulevard off-ramp from southbound US 101 at Stevens Creek, the interpreted soil behavior types consisted of stiff to very stiff clay from ground surface to a depth of 15.5 feet, medium dense to dense sand to a depth of 24.2 feet, stiff to very stiff clay to a depth of 67.2 feet, underlain by dense sand to a depth of 70.5 feet, and stiff to very stiff clay to the maximum depth of 97 feet.

The subsurface soils at this site (SR 85-US 101/southbound US 101 HOV Connector Separation (HAR1) and northbound SR 85-US 101/US 101 Connector Separation (AR2)) are predominantly alluvium. Within existing pavement areas, a pavement section consisting of 1 foot to 2 feet of asphalt paving and associated aggregate base was encountered. The surficial layer of soil consists of 2 to 3 feet of stiff, brown sandy clay. A 10-foot to 15-foot thick layer of highly plastic, stiff to very stiff clay was encountered throughout the bridge site. The clay layer was underlain by sand and clay layers of varying thickness and gradations. The clay layers were generally stiff to very stiff. The sand layers were generally dense to very dense.

This bridge site (northbound SR 85 off-ramp/US 101 Separation (AR3)) is underlain by alluvial deposits consisting of interbedded layers of clays, silts and sands with small amounts of gravel, extending to the terminal depths of the borings and CPTs, ranging from 65.5 to 98.5 feet. Borings B-18 and B-20A were advanced near the abutments and CPT holes B-19 and B-20 were advanced near the bent locations. Boring B-18 was advanced at a location where the ground surface was elevated about 33 feet above the ground surface elevation at boring B-20A, and CPT holes B-19 and B-20. The subsurface strata encountered in the borings and CPT were similar at about the same elevations.

The soils encountered in B-18 and B-20A below the asphalt concrete pavement structural section consist of interbedded layers of gray, brown, and dark brown, stiff to hard fine sandy clay; green, gray, olive gray, brown, and olive brown, medium dense to locally very dense, silty, fine to coarse grained sand with gravel; gray, green, stiff to very stiff clay with low plasticity; dark brown, very stiff to hard fat clay; and olive brown, dense well graded sand. The thickness of the layers ranged between 1.5 feet and 19 feet.

This bridge site (southbound US 101 on-ramp/S101-SR 85 Separation (SR4)) is underlain by alluvial deposits consisting of interbedded layers of clays, silts and sands with small amounts of gravel, which extended to the termination depth of all borings and CPTs. Borings B-4 and B-8 were advanced near the abutments. In these borings, interbedded layers were encountered consisting of green brown to brown, medium dense to dense, clayey, fine to coarse grained sand; gray, green, brown and dark brown, stiff to very stiff clay to fat clay; brown, medium dense, silty fine sand; and green, gray, brown, medium stiff to hard sandy clay. The thickness of these soil layers ranged between 2.5 feet and 17.5 feet.

CPT holes B-5, B-6 and B-7 were advanced near the bents. The CPTs also indicated interbedded layers of medium stiff to hard clayey soils, and medium dense to dense sandy soils. The thickness of the layers ranged between 1 foot and 22.5 feet.

R-Value Test Results

A total of 21 near-surface soil samples were collected within the project limits, adjacent to selected borings; 12 samples were selected for untreated R-value tests. R-values ranged from less than 5 to 38. Based on discussions with Caltrans, an R-value of 5 was used to calculate the minimum structural pavement sections constructed on native soil and an R-value of 15 was used to calculate the minimum pavement sections constructed on fill. The portion of fill placed within 5 feet of the finished grades was specified to have an R-value of not less than 15.

2.3.2.12 North Shoreline Boulevard to Embarcadero Road

For this reach, the following discussion has been divided into Roadways and R-value test results. See LOTBs in PN20 in Appendix A for boring locations.

Roadways

All explorations (auger, rotary wash borings, and CPTs) in this reach were located in the paved shoulder, in the future traveled way, within bridge areas (existing or replace), or near future retaining walls. Alluvium was encountered in all of the borings either beginning at the ground surface or underlying the road base and fill materials (URS Corporation, Geotechnical Design and Materials Report, November 5, 2010). An existing pavement section typically consisting of asphalt concrete overlying aggregate base material was encountered in a number of borings. Borings at creek crossings (R-09-001, R-09-004, and R-09-005) encountered fill that ranged from about 3 feet thick to as much as 9.5 feet in thickness. Eight feet of fill was encountered in boring R-09-001 on the southern bridge abutment at Matadero Creek, 9.5 feet of fill was encountered in boring R-09-004 on the northern bridge abutment at Adobe Creek, and 3 feet of fill was encountered in boring R-09-005 on the southern end of box culvert at Permanente Creek. The fill consisted of locally derived clay and sand with gravel.

The alluvium consists of complexly interbedded lean and fat clay, clayey, silty sand, and well-graded sand with silt. In general, the clay alluvium is soft to very stiff and the sand interbeds are medium dense to dense. Fine grained soils encountered at shallow depths (less than 10 feet) from southern end of Permanente Creek northward to about San Antonio Road overcrossing were generally classified as lean clay. Fine grained soils encountered north of San Antonio Road overcrossing to the end of project at Oregon Expressway at shallow depths were mainly classified as fat clay. North of San Antonio Road overcrossing fine grained soils encountered at shallow depths are generally soft to stiff in consistency. Fine grained soils south of San Antonio Road overcrossing are generally medium to very stiff in consistency. Fine grained soils encountered between depths of 10 to 50 feet were generally classified as medium stiff to stiff lean clay and soils, whereas below depths of 50 feet they comprise alternating layers of medium stiff to very stiff lean and fat clay.

Groundwater was encountered in 28 out of 91 explorations (borings and CPTs). The groundwater depths encountered in 28 explorations are summarized in the following Table 2-4.

Table 2-4 Groundwater Measurements from North Shoreline Boulevard to Embarcadero Road

Boring Number	Water Depth (feet)	Total Boring Depth (feet)	Drilling Method	Comments
A-09-103	9	10.0	Hollow Stem Auger	
CPT-09-106	13.2	45	Cone Penetration Test	Groundwater depth calculated from dissipation test results
A-09-185	11	35.0	Hollow Stem Auger	
A-09-184	12	35.0	Hollow Stem Auger	
R-09-001	Not Measured	101.5	Rotary Wash	Dry to 7.5 feet
A-09-183	13.5	36.5	Hollow Stem Auger	
A-09-182	8	35.0	Hollow Stem Auger	
A-09-121	3	10.0	Hollow Stem Auger	
A-09-181	28	36.5	Hollow Stem Auger	
A-09-180	10.5	36.5	Hollow Stem Auger	
A-09-125	3	10.0	Hollow Stem Auger	
A-09-179	9.0	36.5	Hollow Stem Auger	
R-09-004	Not Measured	101.5	Rotary Wash	Dry to 9.0 feet
CPT-09-003	21.8	100	Cone Penetration Test	Groundwater depth calculated from dissipation test results
A-09-178	12	35.0	Hollow Stem Auger	
A-09-138	7.5	10.0	Hollow Stem Auger	
A-09-152	12	35.0	Hollow Stem Auger	
A-09-153	8	35.0	Hollow Stem Auger	
A-09-154	15	36.5	Hollow Stem Auger	
A-09-177	15	36.5	Hollow Stem Auger	
A-09-159	8	10.0	Hollow Stem Auger	
A-09-160	8	10.0	Hollow Stem Auger	
A-09-161	10	36.5	Hollow Stem Auger	
A-09-162	8	10.0	Hollow Stem Auger	
A-09-164	8	36.5	Hollow Stem Auger	
R-09-005	Not Measured	51.5	Rotary Wash	Dry to 8.5 feet
A-09-169	10	35.0	Hollow Stem Auger	
A-09-170	7.5	10	Hollow Stem Auger	
A-09-171	10.5	40	Hollow Stem Auger	
A-09-174	10	11.5	Hollow Stem Auger	
A-09-175	8	36.5	Hollow Stem Auger	

All of the deeper exploratory borings completed at the creek crossings were drilled with rotary wash drilling methods and immediately backfilled with a mixture of cement and bentonite grout at completion of drilling, in accordance with the Santa Clara Valley Water District guidelines; therefore, reliable groundwater depths could not be measured in the rotary wash borings. However, groundwater levels were measured in hollow stem auger borings as tabulated above in Table 2-4.

In the northern section of the project where ground surface elevations are lower, the groundwater level is expected to be around Elevation 2 to 4 feet, similar to the surface water level in Matadero and Adobe Creeks. Furthermore, on the upgrade south of Station 150+00 depth of groundwater varies considerably from about Elevation 4 to 15 feet. The groundwater level is probably subject to some influence of tidal fluctuation. No groundwater springs or seeps were observed on or near the project alignment. Groundwater levels may vary considerably in the area with seasonal rainfall or with tidal cycles.

Several of the exploratory borings completed by Kleinfelder (2000) also were drilled using hollow-stem auger drilling methods, which allowed for the groundwater level to be measured at time of drilling. The depth to groundwater ranged from about 10 to 25 feet bgs, or about Elevation 10 to 20 feet. Several of these borings were completed in clayey soils and the groundwater level measurements at time of drilling were likely not stabilized.

R-Value Test Results

R-value tests were performed on bulk samples obtained from the subgrade directly under the existing asphalt concrete and aggregate base structural pavement (shoulder) or on fill or native soil in the unpaved shoulder area at depths in the upper 2 to 5 feet. The results of the tests ranged from an R-value of 4 to 63 for the sixteen soil samples. Twelve R-value tests were performed by others as part of the SR 85/US 101 interchange project; those results ranged from less than 5 to 38. The subgrade R-values selected for design ranged from 5 to 10.

2.4 CALTRANS REVIEW COMMENTS

URS submitted a Preliminary Geotechnical Report (PGR) dated September 2012 for the US 101 Express Lanes Project. Caltrans Office of Geotechnical Design – West, Division of Structural Foundations, reviewed the PGR and summarized their comments in an email dated October 26, 2012. The responses to Caltrans email have been incorporated into this report. Copies of the Caltrans email and the VTA Highway Program Comment Review and Response matrix are presented in Appendix B.

Subsequently, Caltrans reviewed the November PGR and summarized their comments in an email dated December 12, 2012. The second set of responses to Caltrans email has been incorporated into this report. Copies of the second Caltrans email and the second VTA Highway Program Comment Review and Response matrix are presented in Appendix B.

Over 40 overcrossings, undercrossings and bridges provide grade separations for US 101 over and under city and county roadways, and rivers and creeks. Available as-built data compiled and reviewed from VTA, Caltrans, as well as URS' files, are presented in Appendix A.

The following describes potential geotechnical, geologic, and seismic impacts from the project and proposed mitigation measures.

4.1 OVERHEAD SIGN STRUCTURE FOUNDATIONS

The subsurface conditions along most of the southern alignment consist primarily of dense sand and gravels with interbeds of stiff clays and silts, and groundwater levels at locations other than creek crossings are generally more than 30 feet in depth. Therefore, Standard Plan foundation consisting of a single cast-in-drilled-hole (CIDH) pile is considered feasible. The design of CIDH piles (drilled piers) is based on granular soils above groundwater (unsubmerged). Some of the proposed locations of overhead signs could encounter groundwater within the Standard Plan pile depths of 25 feet. Site-specific conditions should be evaluated to confirm the Standard Plan assumptions are applicable for this portion of the project alignment.

Near the northern end of the project, between approximately SR 237 and Embarcadero Road, layers of soft to stiff silty clay were encountered from ground surface to depths in the order of 20 to 30 feet bgs. Groundwater was measured in this area at depths ranging from about 3 to 28 feet. In consideration of these soft to stiff clays, a non-standard foundation for overhead signs most likely will be required in this area, consisting of either driven piles or drilled piers.

4.2 PAVEMENT DESIGN CONSIDERATIONS

Based on review of available subsurface information, R-values (California Test 301) of the onsite native soils in the existing median and shoulders vary from 5 to 78. Based on URS' local experience, we believe a subgrade design R-value of 15 provides a reasonable basis for preliminary design of new pavement for the widening in the median or shoulders along most of the project alignment. However, the design R-value for the segment between SR 237 and Embarcadero Road should be expected to be on the order of 5.

4.3 SEISMIC AND GEOLOGIC HAZARDS

4.3.1 Regional Tectonic Setting and Seismicity

The project alignment lies between known active and potentially active geologic faults. In general, earthquakes occur as a result of movement along active faults. For the purpose of activity classification, faults are generally grouped into the following categories by the California Geological Survey (Jennings and Bryant, 2010):

- Holocene: displacement has occurred within the last 10,000 to 11,000 years.
- Late Quaternary: displacement has occurred within the last 700,000 years, but evidence of Holocene activity is lacking.
- Quaternary: evidence of displacement within the last 1.6 million years, but evidence of Holocene activity is lacking.
- Pre-quaternary: no recognized evidence of displacement in the last 1.6 million years.

Generally, faults with Holocene movement are considered to be "active" while faults with late Quaternary to Quaternary movement are considered to be "potentially active."

The closest active faults to the project alignment are the San Andreas, Hayward, Calaveras, Silver Creek, Cascade and Monte Vista faults (Caltrans Deterministic PGA Map, 2007). The project alignment crosses two faults, Palo Alto and San Jose, which are not described below due to their Early Quaternary age, inferred alignment location, and lack of inclusion on the Caltrans Deterministic PGA Map (2007). The Caltrans Seismic Map (Mualchin, 1996) divides the Monte Vista fault into west and east branches. In the 2007 Caltrans Deterministic PGA Map the Monte Vista East fault has been renamed as Cascade fault. The California Geological Survey (2010) has produced maps showing faults with known Holocene activity that pose a potential surface faulting hazard. The San Andreas, Hayward, and Calaveras faults are considered active faults; however the Monte Vista, Silver Creek, and Cascade faults are not.

For the purpose of this discussion, the project alignment has been divided into three segments: Segment 1: US 101 from Dunne Avenue in Morgan Hill to SR 85 in South San Jose; Segment 2: US 101 from SR 85 intersection to I-880; and Segment 3: US 101 from I-880 to Embarcadero Road in Palo Alto. Silver Creek fault is located about 1.9 miles northeast of the southern end of Segment 1. Silver Creek fault crosses the project alignment twice in the northern half of Segment 2. Monte Vista fault is located about 6.5 miles northwest of Segment 2. The San Andreas, Hayward, and Calaveras faults generally parallel the alignment in all three segments. The San Andreas fault is located about 14 miles west of Segment 2, and the Hayward and Calaveras faults are located 5.5 miles east and 8 miles east of Segment 2, respectively. Cascade fault crosses the project alignment at the southern section of Segment 2, and also at the intersection of US 101 and SR 237 in Segment 3. Cascade fault is located about 1.8 miles east of the northern end of Segment 3. Figure 4 shows active faults within the site region relative to the project. The project alignment does not cross any faults considered to be active by the California Geological Survey or USGS.

The following is a brief description of the nearby active faults.

4.3.2 San Andreas Fault Zone

The dominant active fault structure in this region is the San Andreas fault. The fault extends from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, a total distance of 746 miles. The San Andreas fault accommodates the majority of the motion between the Pacific and North American plates. This fault is the largest active fault in California and is responsible for the largest known earthquake in Northern California, the 1906 M 7.9 San Francisco earthquake (Wallace, 1990).

Movement on the San Andreas fault is right-lateral strike-slip, with a total offset of some 348 miles (Irwin, 1990). In northern California, the San Andreas fault is clearly delineated, striking northwest, approximately parallel to the vector of plate motion between the Pacific and North American plates. Over most of its length, the San Andreas fault is a relatively simple, linear fault trace. Immediately south of the Bay, however, the fault splits into a number of branch faults or splays, including the Calaveras and Hayward faults. In the Bay Area, the main trace of the San Andreas fault forms a linear depression along the Peninsula, occupied by the Crystal Springs and San Andreas Lake reservoirs. Geomorphic evidence of Holocene faulting includes fault scarps in Holocene deposits, right-laterally offset streams, shutter ridges, and closed linear depressions (Wallace, 1990). The 1906 earthquake resulted from rupture of the fault from San Juan Bautista north to Point Delgada, a distance of approximately 295 miles. The average amount of slip on the fault during this earthquake was 16.7 feet in the area to the north of the Golden Gate and 8.2 feet in the Santa Cruz Mountains (WGNCEP, 1996).

Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault is divided into a number of fault segments. Each of these segments is capable of rupturing either independently or in conjunction with adjacent segments. In the Bay Area, these segments include Santa Cruz Mountains, the Peninsula, and the North Coast segments. These fault segments have calculated maximum earthquakes of moment magnitude (**M**) 7.1, 7.2, and 7.5, respectively (WGCEP, 2008). The North Coast segment may also be subdivided into two shorter segments with a boundary at Point Arena. These northern and southern North Coast segments are capable of generating earthquakes of **M** 7.5 and 7.7, respectively (WGCEP, 2008).

South of the Golden Gate, the fault slip rate is 0.67 + 0.27 inch per year (Hall et al., 1999). North of the Golden Gate, the slip rate increases to 0.94 + 0.20 inch per year (Niemi and Hall, 1992). WGCEP (2008) assigns a recurrence interval of 361 years to **M** 8.0 1906-type event on the San Andreas fault, with a 21 percent probability of a **M** 6.7 or larger earthquake on San Andreas in northern California in the time period 2007 and 2036. Recent investigations indicate that the repeat time for large earthquakes on the North Coast segment may be less than 250 years.

4.3.3 Hayward Fault

The Hayward fault is a part of the San Andreas fault system and extends for about 62 miles from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay. The Hayward fault is an active, right-lateral, strike-slip fault, considered the most likely source of the next major earthquake in the Bay Area (Working Group on California Earthquake Probabilities, 2003). As well as moving during earthquake ruptures, the Hayward fault also moves by aseismic slip (creep). Measurements along the fault over the last two decades show that the creep rate is between 0.20 and 0.35 inch per year (Lienkaemper and Galehouse, 1997).

The last large earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This moment magnitude (**M**) 6.8 event caused toppling of buildings in Hayward and other localities within about 3.1 miles of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 18.6 miles, from Warm Springs to San Leandro, with a maximum reported displacement of about 39 inches.

Recent research of historical documents has led to the conclusion that an earthquake in 1836, previously thought to have occurred on the northern Hayward fault, occurred elsewhere (Toppozada and Borchardt, 1998), thereby increasing the time since the last earthquake on this segment of the fault. Recent paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper et al., 1997). This study also indicated at least four surface-rupturing earthquakes in the last 2,250 years. The WGCEP (2003) assigns mean maximum earthquakes of **M** 6.5 and 6.7, and mean recurrence intervals of 312 and 292 years, for the northern and southern segments of the Hayward fault, respectively. Rupture of the entire Hayward fault zone could generate an earthquake of **M** 6.9 (WGCEP, 2003). The maximum magnitudes estimated by the WGCEP (2008) are slightly lower than past values because of their use of a seismogenic scaling factor (R), which accounts for aseismic slip (creep) on the fault.

The WGCEP (2008) considers the Hayward-Rodgers Creek fault system the most likely source of the next **M** 6.7 or larger earthquake in the Bay Area, with a 31 percent probability of occurring in the time period 2007 to 2036. Their model also incorporates a scenario where the Hayward fault ruptures along with the Rodgers Creek fault. Rupture of the entire length of both faults would generate a mean maximum earthquake of **M** 7.3 (WGCEP, 2008). Rupture of the Rodgers Creek fault and the northern segment of the Hayward fault would generate a maximum event of **M** 7.1.

Maulchin (1996) assigns a maximum credible earthquake (MCE) of **M** 7.5 to the Hayward fault. Note a MCE is a conservative estimate of the maximum magnitude and is not related to the mean values computed by WGCEP (2003).

4.3.4 Calaveras Fault

The 80.8-mile-long ± 6.2 miles Calaveras fault traverses the Hollister Plain and the Diablo Range east of the Santa Clara Valley (Figure 4) and is a major structural boundary between the Diablo Range and the San Francisco Bay structural depression (Page, 1982). The Calaveras fault exhibits prominent geomorphic expression along its entire active length and has generated small and moderate earthquakes during the past 200 years of recorded history. Based on structural relations with other major faults, contemporary seismicity, rate of present-day creep and geodetic deformation, and geomorphic expression, the Calaveras fault consists of three primary sections (Kelson, 2001; WGCEP, 2003):

- Northern Calaveras fault (from Danville to Calaveras Reservoir);
- Central Calaveras fault (from Calaveras Reservoir to San Felipe Lake); and
- Southern Calaveras (from San Felipe Lake to the Paicines fault south of Hollister).

WGCEP (2008) assigns a 7 percent probability of an **M** 6.7 or larger earthquake on Calaveras fault in the time period 2007 and 2036.

4.3.5 Foothills Thrust Belt

The Foothills fault system is a series of southwest, dipping thrust faults located along the range front of the Santa Cruz Mountains (Bürgmann et al., 1994). The Monte Vista-Shannon, Cascade and Sargent faults are the main active faults in the Foothills thrust system. The Monte Vista-Shannon fault zone is approximately 27 miles long and dips at a moderate angle to the southwest, merging with the San Andreas fault at depth. The Cascade fault is approximately 22 miles long. The Sargent fault is approximately 35 miles long and merges with the San Andreas fault near Loma Prieta.

The southwestern margin of the Santa Clara Valley is bounded by the rugged, young southern Santa Cruz Mountains. Late Cenozoic uplift of the mountains has occurred, in part, along a series of northwest-striking reverse faults, known as either the Loma Prieta domain (Aydin and Page, 1984) or Foothills thrust belt (Bürgmann et al., 1994), bordering the northeastern margin of the range front. Bounded by the main trace of the San Andreas fault to the west, this sequence of southwest-dipping thrusts, associated with a restraining left bend in the San Andreas fault, has been responsible for the uplift of the Santa Cruz Mountains (Bürgmann et al., 1994). These faults offset the Pliocene and Pleistocene Santa Clara Formation, and locally offset and deform overlying Quaternary sediments and geomorphic surfaces within the range-front communities of

Palo Alto, Los Altos Hills, Cupertino, Saratoga, and Los Gatos, located along the southwestern margin of the Santa Clara Valley (Hitchcock and Kelson 1999; Hitchcock et al. 1994). The up-dip projection of the blind Loma Prieta fault, which is interpreted to have been the source of the 1989 **M** 6.9 Loma Prieta earthquake (Bürgmann et al., 1994), coincides with the Foothills thrust belt.

The Monte Vista fault is one of the primary range-front faults and probably the most extensively studied fault in the Foothills thrust belt. The exposed fault strikes northwest and places Franciscan, Santa Clara Formation, and Pleistocene alluvium over Pleistocene and older strata. To the south, the fault merges with the Shannon fault, while at its northern end it intersects the San Andreas, via the Hermit fault, between Woodside and Redwood City. Limited exploratory trenching indicates that the Monte Vista-Shannon fault has had late Quaternary and possibly Holocene displacement. Recent geomorphic mapping by Hitchcock et al. (1994) shows that late Pleistocene fluvial terraces flanking Stevens Creek are deformed. The style of late Quaternary deformation affecting these terrace surfaces is consistent with reverse faulting on the Monte Vista faults. Hitchcock and Kelson (1999) have estimated a very low late Pleistocene slip rate of 0.0067 ± 0.0035 inch per year for the Monte Vista-Shannon fault.

The Cascade fault traverses the coalescent alluvial-fan complex underlying the Santa Clara Valley approximately 1 to 4 miles northeast of the Santa Cruz Mountains range front. Hitchcock et al. (1994) show a strong correlation between the mapped trace of the Cascade fault and fault-related geomorphic features, including vegetation lineaments, closed depressions, linear drainages, stream profile convexities, and high-sinuosity stream reaches. These features are developed in late Pleistocene (and possibly Holocene) and displaced along the Cascade fault. Between Los Altos Hills and Los Gatos, most of the major streams show longitudinal profile convexities where they cross mapped trace of the Cascade fault. In general, the crests of the convexities coincide with the zone of lineaments. These relations indicate late Pleistocene uplift along this section of the Cascade fault (Hitchcock et al., 1994). Although this provides little or no information on the sense of slip and the amount and direction of the fault dip, it is likely that the Cascade fault is a southwest dipping, northeast vergent reverse fault similar to, but perhaps having a shallower dip in the near surface than the Monte Vista-Shannon fault (Fenton and Hitchcock, 2001).

4.3.6 Silver Creek Fault Zone

The Silver Creek fault is mapped as a steeply dipping northwest-trending strike-slip fault with an exposed well documented southern fault section and an inferred northern section buried beneath late Quaternary sediments of Santa Clara Valley. The southern section extends approximately 19 miles from the Anderson Reservoir, northeast of Morgan Hill, where it splays from the Coyote Creek fault to the mouth of the Silver Creek Valley, southwest of the Evergreen district of San Jose. The northern section is approximately 25 miles long beneath Santa Clara Valley, extending from Silver Creek Valley to Alameda Creek in Fremont (Fenton and Hitchcock, 2001).

Two **M** 6.1 earthquakes in 1903 were originally believed to have been sourced from the Silver Creek fault zone. Due to inaccuracy in locating the epicenter of these earthquakes at that time, and the lack of small earthquakes in recent years, it is believed that the 1903 events could be linked to other faults in the area. The potential for repeat earthquakes on this fault is believed to be small (Wentworth et. al, 2010).

4.4 PRELIMINARY SEISMIC DESIGN CRITERIA

4.4.1 Seismic Design Methodology

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

- Seismic Design Criteria (SDC), v 1.6, November 2010;
- Foundation Report Preparation for Bridge Foundations, dated December 2009; and
- 2007 Caltrans Deterministic PGA Map.

4.4.2 Peak Bedrock Acceleration

The active faults close to the US 101 Express Lanes Project alignment are the Hayward, Cascade, San Andreas, Calaveras, Monte Vista- Shannon, and Silver Creek faults. Silver Creek fault crosses the project alignment at the Tully Road and I-880 interchanges. Based on a review of Geotechnical Design and Materials Report for the US 101 Improvements Project (Parikh Consultants, Inc, dated April 14, 2009), it is our understanding that a separate study was performed regarding the Silver Creek Fault trace and potential rupture for Tully Road overcrossing structure design. The pertinent seismic source parameters are summarized below in Table 4-1.

Table 4-1 Seismic Source Parameters

Fault	Type	M_{Max}	Distance (miles)
Calaveras	Strike-slip	7.4	4.0[(1)]
Silver Creek	Reverse	7.1	<1[(1)] 1.1[(2)]
Cascade	Reverse	6.9	2.7[(3)]
Monte Vista-Shannon	Reverse	6.7	4.7[(3)]
San Andreas	Strike-slip	8.0	7.5[(3)]
Hayward	Strike-slip	7.5	10.7[(3)]

(1) From Segment 1
(2) From Segment 2
(3) From Segment 3

4.4.3 Site Soil Profile

To determine the Design Acceleration Response Spectrum (ARS), one location from each of the three project segments described in Section 4.3.1 was selected:

- Segment 1 – Coyote Creek Bridge (Golf Drive);
- Segment 2 – Tully Road Interchange; and
- Segment 3 – Matadero Creek Bridge.

Based on a review of subsurface data along US 101, soils in Segments 1 and 2 are categorized as stiff and very dense (Site Class D and C), whereas soils in Segment 3 are categorized as stiff to soft (Site Class D and E, respectively). Based on a review of subsurface data the soil becomes stiffer and denser towards the southern end of the alignment. Site specific investigations should be completed to confirm the subsurface conditions and selection of the soil profile type at the proposed structure locations.

Three seismic CPTs were performed for the US 101 Auxiliary Lanes Project by URS (November 5, 2010) with the intent of estimating the shear wave velocity profile. Shear wave velocity (Vs) was measured at 10 feet intervals starting at 10 feet bgs with values ranging from 591 to 990 feet per second (fps). For preliminary design of the US 101 Express Lanes Project structures, Soil Profile Type D was recommended based on the guidelines given in SDC Figure B.12. Vs values of 985, 885 and 820 fps, are estimated for Segments 1, 2 and 3, respectively. This Vs value was input for the Caltrans ARS online. It should be noted the estimated values for all segments fall within the soil profile Type D Vs range of 600 to 1,200 fps.

4.4.4 Fault Type and Near-Field Spectral Acceleration Increases

The 2007 Caltrans fault database indicates that the closest active faults listed above in Table 4-1 have strike-slip or reverse 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 alignment is less than 9.5 miles from most of the active faults, design spectral accelerations should be modified to account for near-fault effects as shown in Table 4-2.

Table 4-2 Increase in Spectral Acceleration from near Fault Effects

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

At the time of this study, no bridge structures with fundamental period of vibration greater than 1.5 seconds are anticipated; therefore no adjustments are required for long period effects.

4.4.5 Design Acceleration Response Spectrum

The preliminary design response spectrum for the site is estimated with spectral acceleration values generated using Caltrans ARS Online (2009). This method was developed by Caltrans

Geo Research Group in partnership with the USGS, Pacific Earthquake Engineering Research (PEER) and California Department of Conservation. This web-based tool calculates both deterministic and probabilistic acceleration response spectra for any location in California based on criteria provided in Appendix B of the Caltrans SDC.

The deterministic spectrum is determined as the average of median response spectra calculated using the Campbell-Bozorgnia (2008) and Chiou-Youngs (2008) ground motion prediction equations developed under the "Next Generation Attenuation" project coordinated through the PEER-Lifelines program. These equations are applied to all faults considered to be active in the last 750,000 years (late-Quaternary age) that are capable of producing a moment magnitude earthquake of 6.0 or greater. The probabilistic spectrum is obtained from the USGS (2008) National Hazard Map for 5 percent probability of exceedance in 50 years. Caltrans design spectrum is based on the larger of the deterministic and probabilistic spectral values. Both the deterministic and probabilistic spectra account for soil effects through incorporation of the parameter Vs30, the average shear wave velocity in the upper 30 meters (100 feet) of the soil profile.

The input values selected for Caltrans ARS Online included:

- V_{S30} of 985 fps (300 meters per second, i.e. mps) for Segment 1;
- V_{S30} of 885 fps (270 mps) for Segment 2;
- V_{S30} of 820 fps (250 mps) for Segment 3;
- No ARS increase for fault-type or long period structure; and
- ARS increase for near-field effects.

The selected design curve is based on the higher spectral value for the corresponding period calculated using either the deterministic or probabilistic methods. Tabulated spectral values of the design curves for the three project segments are presented in Tables 4-3, 4-4, and 4-5.

Table 4-3 Spectral Acceleration Values for Segment 1

(US 101 from Dunne Avenue to SR 85 Interchange)
(V_s= 300 mps, Lat. = 37.190695, Long. = -121.69322)

Period (seconds)	Sa (g)	Sa* (g)
0.010	0.667	0.667
0.100	1.145	1.145
0.200	1.331	1.331
0.300	1.383	1.383
0.500	1.292	1.292
1.000	0.869	1.043
2.000	0.464	0.556
3.000	0.291	0.350
4.000	0.204	0.245
5.000	0.163	0.195

Table 4-4 Spectral Acceleration Values for Segment 2

(US 101 from SR 85 Interchange to I-880 Interchange)
(V_s= 270 mps, Lat. = 37.318434, Long. = -121.831377)

Period (seconds)	Sa (g)	Sa* (g)
0.010	0.600	0.600
0.100	0.840	0.840
0.200	1.045	1.045
0.300	1.151	1.151
0.500	1.272	1.272
1.000	1.056	1.267
2.000	0.593	0.711
3.000	0.348	0.418
4.000	0.236	0.283
5.000	0.193	0.231

Table 4-5 Spectral Acceleration Values for Segment 3

(US 101 from I-880 Interchange to Oregon Expressway)
(V_s= 250 mps, Lat. = 37.416831, Long. = -122.08673)

Period (seconds)	Sa (g)	Sa* (g)
0.010	0.578	0.578
0.100	1.000	1.000
0.200	1.257	1.257
0.300	1.264	1.264
0.500	1.132	1.132
1.000	0.805	0.966
2.000	0.486	0.583
3.000	0.327	0.392
4.000	0.238	0.286
5.000	0.193	0.231

*Modified with near-fault factors outlined in Section 4.4.4.

4.5 SURFACE FAULT DISPLACEMENT AND GROUND SHAKING

The project alignment is not crossed by any known active faults (CGS, 2010). The alignment crosses the Silver Creek and Cascade faults, but available geologic data indicate the most recent episode of ground surface rupture on these faults predated Holocene time and may have been pre-late Pleistocene. The likelihood of ground surface rupture on these faults is considered low. Therefore, surface rupture due to faulting at the site is not expected to occur. However, the closest distance to the San Andreas fault (4.4 miles) and other more distant active faults creates a high risk for ground shaking from fault movement. The intensity of the ground shaking is dependent upon the size of the earthquake, the distance of the epicenter from the site, the direction that the earthquake propagates along the fault, and the site geologic conditions. Section 4.3.1 discusses the anticipated seismic shaking from these faults.

4.6 LANDSLIDES

US 101 is on relatively flat ground along the project alignment. Landsliding is not a potential hazard. The CGS has not mapped any area of the project alignment as an earthquake-induced landslide zone (CGS, 2002). It is conceivable that localized instability could occur where stream/channel banks have been oversteepened by erosion or scour. Such conditions should be reviewed on a site specific basis during final design.

4.7 LIQUEFACTION

Liquefaction is a phenomenon whereby sediments temporarily lose shear strength and collapse. This condition is caused by cyclic loading during earthquake shaking that generates high porewater pressures within the sediments. The soil type most susceptible to liquefaction is loose, cohesionless, granular soil below the water table and within about 50 feet of the ground surface. Liquefaction can result in loss of foundation support and settlement of overlying structures, ground subsidence and translation due to lateral spreading, lurch cracking, and differential settlement of affected deposits. Lateral spreading occurs when a layer liquefies at depth and causes horizontal movement or displacement of the overburden mass toward a free face such as a stream bank or excavation, or towards an open body of water.

In a regional study of the nine-county San Francisco Bay region for the USGS, Witter et al. (2006) mapped the liquefaction susceptibility of the soils in the project vicinity. The Association of Bay Area Governments (ABAG, 2004) has also published a liquefaction susceptibility map based on mapping in the USGS Open File Report 00-444 by Knudsen et al. (2000). A copy of this map is included as Figure 5. The map indicates the project alignment generally contains soils with moderate liquefaction susceptibility. Soils in the southern-most portion of the alignment in Morgan Hill are mapped with low liquefaction susceptibility.

High to very high liquefaction susceptibility has been mapped within younger fluvial deposits where larger drainages cross the alignment, such as Coyote Creek, Guadalupe River, and San Tomas Aquino Creek. Very high liquefaction susceptibility has also been mapped at the north end of the project alignment along US 101 between San Antonio Road and Oregon Expressway, where the alignment is underlain by unconsolidated to semi-consolidated alluvial deposits.

During final project design, a detailed liquefaction evaluation should be completed at planned foundation locations of the overhead sign structures, embankment locations and bridge locations. The potential for lateral spreading along the project alignment is considered low; nevertheless, during final design, lateral spreading should be evaluated. The potential for cyclic densification of the unsaturated fill soils should also be evaluated.

4.8 SUBSIDENCE AND SETTLEMENT

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 since the Santa Clara Valley Water District implemented groundwater recharge programs more than 50 years ago. No active petroleum wells are present within many miles of the project alignment (California Division of Oil, Gas, and Geothermal Resources, 2009). In addition, there was no reported subsidence in the area near a groundwater extraction system installed for mitigating subsurface contamination at the former Fairchild Semiconductor site in South San Jose. Settlement can occur quickly when soil is loaded by a structure or by the placement of fill on top of soil, and it can also occur gradually when soil pore pressures, increased by vertical loading, gradually dissipate over time. Since no extensive fill loads are expected for this project, the potential impact and hazards of consolidation settlement due to embankment loading are considered low.

Compaction settlement, or seismic densification, occurs when loose granular soils above the water table increase in density as a result of earthquake shaking. The soil densification can result in differential settlement because of variations in soil composition, thickness, and initial density. As previously mentioned, localized lenses and layers of loose to medium dense granular soils were encountered. These granular deposits may be subject to cyclic densification during strong ground shaking, resulting in compaction settlement. An analysis of the amount and location for compaction settlement to occur should be completed during the final design and be mitigated through appropriate foundation design or ground improvements.

4.9 FLOODING

Segments 1 and 2 of the project alignment, southeast of I-880 interchange, are mostly located in “urbanized area” according to the ABAG flood hazard map (2007); that is, it is outside any Federal Emergency Management Agency (FEMA) flood zones (Zones V, A or X500). Portions of the alignment where it crosses Coyote Creek are located within Zone X500 and Zone A. Segment 3, northwest of the I-880 interchange, is completely within Zone X500, except in minor sections that are within Zone A. The code X500 identifies areas (1) inundated by 0.2 percent annual chance flooding, (2) inundated by 1 percent annual chance flooding with average depths of less than 1 foot, (3) inundated by 1 percent annual chance flooding with drainage areas less than 1 square mile, or (4) that are protected by levees from 1 percent annual chance flooding. Zone A identifies an area inundated by 1 percent annual chance flooding. A copy of the ABAG flood hazard map is included as Figure 6. Flooding is addressed in more detail in this project’s Water Quality and Location Hydraulic Study reports.

4.10 EROSION

Throughout the project alignment, existing embankment inclinations are in general 2 to 1 (horizontal to vertical), and in special cases 1.5 to 1. Natural slopes along the project alignment also are relatively flat. The majority of the northbound roadway of the southernmost project segment between Dunne Avenue and Metcalf Road is positioned in well-vegetated (grasses) cuts; whereas the southbound roadway is located in both fills and well-vegetated (grasses) cuts.

A concrete barrier wall is located in the US 101 median; the ground surface on the west side of the median typically is well vegetated (grasses) whereas the east side is typically paved with PCC. Typically there is a ground surface differential height of several feet along this median wall. Since proposed express lanes are planned adjacent to the median, only a slight change of rate of erosion is expected from this new project.

Along most of the US 101 alignment between Metcalf Road and Embarcadero Road, the roadway surface is close to original grade. Only a few retaining walls were required and are mostly at interchanges. Since proposed express lanes are planned in the median and shoulders, only a slight change of rate of erosion is expected from this new project.

A large cut was made through a hillside near Hellyer Avenue; there was erosion of this sloped face during the latter 1990s and subsequent successful remediation. It would be prudent to minimize excavation and disturbance in this hillside during future construction.

Continuing northbound along US 101 between Alum Rock Avenue and De La Cruz Boulevard, a majority of the roadway is located in deep cuts (20 feet or deeper) retained by concrete retaining walls. A sloped soil toe was frequently observed at the base of the retaining walls; these slopes contain numerous shrubs, with limited grass cover. The exposed slopes revealed granular materials (sands and gravels); there are no apparent signs of erosion observed on these sloped soil toes. Sound walls are common and located on top of or in back of the retaining walls. The median consists of exposed soil (no vegetation) with a metal traffic barrier down the middle. Since the cut faces are mostly supported by reinforced concrete walls, no change in erosion rates are expected for this new project.

The northernmost portion of the alignment is along US 101 between De La Cruz Boulevard and Embarcadero Road. Both the northbound and southbound roadways in this segment are level, since they are located in relatively flat topography. Consequently, cuts and fills are small. The median is typically paved in this segment (URS, 2010). Approach embankment fill on US 101 was observed at a number of structure locations, including at Ellis Street.

4.11 CORROSION

Corrosion tests have been performed on onsite soils obtained during previous investigations along the US 101 project alignment. The results have been reported in several Materials Reports and are summarized in Table 4-6 below.

Table 4-6 Corrosion Test Results

References	Location	pH	Resistivity (ohm-cm)		Chloride Content (ppm)	Sulfate Content (ppm)	Comments
			Laboratory	Field			
Parikh, June 11, 2001	Cochrane Road to Metcalf Road	7.4-8.7	750-7,600		1.8-67.5	2.9-56.2	Non-corrosive

Table 4-6 Corrosion Test Results (continued)

References	Location	pH	Resistivity (ohm-cm)		Chloride Content (ppm)	Sulfate Content (ppm)	Comments
			Laboratory	Field			
Terratech, October 20, 1989	Metcalf Road to Blossom Hill Road	7.5-9.6	540-2,900+	1,628-8,043	<50	<100; except 200	Non-corrosive
Terratech, November 27, 1987	Blossom Hill Road to I-280	6.6-9.15	520-6,000	760-30,000	0-600	4-900	Corrosive
Dames & Moore, March 12, 1990	Hedding Street to I-880	7.6-8.5	500-1,595	1,341-27,576	11.0-670	64.0-2,000	Corrosive
Associated Geotechnical Engineers, Inc., February 7, 1990	I-880 to Guadalupe River	6.6-9.7	380-1,066,800	1,092-63,195	0.0-170.0	0-290.0	Mostly non-corrosive
URS Corporation, November 5, 2010	SR 85 to Oregon Avenue	8.6-10.3	421-11,098	296-15,560	Soil: <2-256 Water: 174-187	Soil: <5 - 203 Water: 12 - 21	Non-corrosive

According to Caltrans guidelines (California Department of Transportation's Division of Engineering Services, Materials Engineering and Testing Services, Corrosion Technology Branch *Corrosion Guidelines*, Version 1.0, dated September 2003), they consider representative soil or water samples to be corrosive to metallic or reinforced concrete structural elements if one or more of the following conditions exist:

1. The chloride concentration is 500 parts per million (ppm) or greater and/or
2. The sulfate concentration is 2,000 ppm or greater and/or
3. The pH is 5.5 or less.

Based on the site specific pH values and water-soluble chloride and sulfate concentrations at each of the locations listed above in Table 4-6, the soil samples are either non-corrosive or corrosive to buried steel and reinforced concrete structures (listed under "comments" column in Table 4-6) as defined by the Caltrans *Corrosion Guidelines.*

4.12 CLIMATE

Climatologically, the area is classified as Mediterranean, with dry summers and mild winters. The following discussion includes records at the San Jose, California weather station between the years of 1971 and 2000 (NCDC, 2004). This weather station is located approximately in the middle of the project alignment. The mean annual temperature is 61.3 degrees Fahrenheit (^{o}F). Extremes of temperatures during the study years range from an average daily maximum of

84.3°F in July to average daily low of 41.0°F in December. The highest and lowest temperatures on record in San Jose during the study years are 109° F on June 14, 2000 and 19+°F on December 23, 1990. Freezing temperatures at this station are highly infrequent, so freeze-thaw conditions should not be a factor.

Precipitation in San Jose averages 15.08 inches per year, primarily confined to the months of October through April. November, December, January, February and March usually have the most precipitation accumulation, averaging 1.73, 2.00, 3.03, 2.84 and 2.69 inches per month, respectively.

4.13 CONSTRUCTION DEWATERING CONSIDERATIONS

4.13.1 Five Bridge Locations

The potential need for construction dewatering at these five (5) bridge locations: 37-344, 37-404, 37-347, 37-108 and 37-409 is discussed in this section. Groundwater depths / elevations are based on information presented on the existing LOTBs. In addition existing foundation plans were also obtained from the as-built drawings: these include locations of abutments and bents, as well as elevations of bottom of footings. In our analyses, we have assumed that widening of existing bridges will result in extending abutments/bents at their same bottom of footing (BOF) elevations. The potential need for construction dewatering at each of the five bridge locations are discussed below.

4.13.1.1 Bridge 37-344 – Coyote Creek Road UC

The name of this structure, "Coyote Creek Road UC," is based on Caltrans Bridge Inspection Report of Bridge 37-344. This bridge was previously named Scheller Avenue UC; however, the current southbound US 101 overhead sign refers this exit as Coyote Creek Golf Drive. Two log of test boring (LOTB) sheets available for this UC are listed in Project Number (PN) 21 in Table A-1. The As-built LOTB sheet reveals four (4) penetration borings (conducted with 2¼ inch cone penetrometer) B-1 (04/28/71), B-3 (04/28/71), B-4 (04/28/71), and B-5 (04/29/71); groundwater was not recorded in these penetration borings. Furthermore the As-built LOTB shows five rotary borings including B-2 (04/28/1971), B-6 (04/29/1971), B-7 (06/15/1971), B-8 (06/16/1971) and B-9 (01/31/1975); maximum boring depth is 43 feet below ground surface (bgs). Groundwater was encountered only in B-7 at Elevation 317.5 feet (06/16/1971); this corresponds to a depth of about 33 feet bgs.

A second LOTB (as-built date of February 20, 2004) reveals two auger Borings 00-CG-1 (02/10/2000) and 00-CG-2 (02/08/2000) with measured groundwater at Elevations 320.8 and 316.5 feet, respectively.

The structure is supported on two abutments and one bent, with elevations listed in Table 4-7 below.

Table 4-7 Bridge 37-344 Footing Elevations

Location	Range of Elevations of Bottom of Existing Footing
Abut 1 (South)	359.9 to 370.9
Bent 2	341.6 to 348.8
Abut 3 (North)	358.6 to 369.9

The deepest BOF Elevation is 341.6 feet at Bent 2. The highest measured groundwater is Elevation 320.8 at 00-CG-1, which is 20.8 feet below (341.6-320.8) the deepest BOF. Therefore, assuming no significant change in groundwater conditions, construction dewatering is not anticipated during widening of Bridge 37-344.

4.13.1.2 Bridge 37-404 – Utility Facility UC (Golf Course)

Two LOTB sheets available for this UC are listed in PN22 in Table A-1.

The first As-built LOTB sheet dated 1979 reveals three penetration borings (conducted with 2¼ inch cone penetrometer) B-3 (06/21/1979), B-4 (06/21/1979) and B-5 (06/21/1979); groundwater was not recorded in these penetration borings. The same As-built LOTB shows two (2) rotary borings including B-1 (06/19/1979) and B-2 (06/20/1979); maximum boring depth is 90 feet bgs. Groundwater was not recorded in either boring.

A more recent LOTB sheet (as-built date of February 20, 2004) reveals two Borings 00-UC-1 (02/11/2000) and 00-UC-2 (02/08/2000) with groundwater encountered at approximate Elevations 302.1 and 316.5 feet, respectively. Maximum boring depth is 90 feet bgs.

The structure is supported on two abutments with elevations listed in Table 4-8 below.

Table 4-8 Bridge 37-404 Footing Elevations

Location	Range of Elevations of Bottom of Existing Footing (feet)
Abut 1 (Southeast)	346.1 to 354.7
Abut 2 (Northwest)	345.5 to 354

The deepest BOF Elevation is 345.5 feet, whereas the highest measured groundwater is Elevation 316.5 feet at 00-UC-2. Therefore, groundwater is 29 feet (345.5-316.5) below deepest BOF. Therefore, assuming no significant change in groundwater conditions, construction dewatering is not anticipated during widening of Bridge 37-404.

4.13.1.3 Bridge 37-347 – Bernal Road UC

Two LOTB sheets available for this UC are listed in PN23 in Table A-1.

This bridge was previously named Tennant Road UC. The As-built LOTB sheet reveals a sampler boring B-4 (11/16/1970) and four (4) penetration borings (with 2¼ inch cone penetrometer) B-2 (11/12/1970), B-3 (11/13/1970), B-5 (11/17/1970), B-6 (11/18/1970) and B-7 (11/17/1970). Groundwater was encountered in penetration borings B-5 and B-7 at Elevations

194.5 (11/17/1970) and 188.4 (11/17/1970), respectively. Furthermore the As-built LOTB shows two (2) rotary Borings B-1 (11/12/1970) and B-8 (11/17/1970); maximum boring depth is 80 feet bgs. Groundwater was not recorded in either rotary boring.

A second LOTB (as-built date of March 10, 2005) reveals one cone penetration test sounding (CPT) CPT-5 (02/29/2000) and one rotary wash Boring EB-6 (02/29/2000) with measured groundwater at Elevation 178.9 feet in EB-6.

The structure is supported on two abutments and one bent with elevations listed in Table 4-9 below.

Table 4-9 Bridge 37-347 Footing Elevations

Location	Range of Elevations of Bottom of Existing Footing (feet)
Abut 1 (East)	225.5
Bent 2	209.5 to 210
Abut 3 (West)	225.5 to 226.1

The deepest BOF Elevation is 209.5 feet, whereas the highest groundwater is Elevation 197 at B-5. The highest groundwater is 12.5 feet (209.5-197) below deepest BOF. Therefore, assuming no significant change in groundwater conditions, construction dewatering is not anticipated during widening of Bridge 37-347.

4.13.1.4 Bridge 37-108 – Coyote Road UC

One LOTB sheet available for this UC is listed in PN24 in Table A-1. The 1988 widening project LOTB sheet reveals one auger Boring DH-10 (10/12/87) and four (4) rotary Borings DH-7 (05/03/1987), DH-8 (04/15/1987), DH-9 (04/16/1987) and DH-11 (10/12/1987). Maximum boring depth was 55 feet bgs. Groundwater levels were recorded at Elevations 151, 158 (reportedly perched), and 145, respectively, at Borings DH-7 (05/03/1987), DH-8 (04/15/1987) and DH-11 (10/12/1987); shallowest measured groundwater was Elevation 158 in DH-8, corresponding to a depth of about 13 feet bgs.

The structure is supported on two (2) abutments and two (2) bents with elevations listed in Table 4-10 below.

Table 4-10 Bridge 37-108 Footing Elevations

Location	Range of Elevations of Bottom of Existing Footing (feet)
Abut 1 (North)	176.0 – 182.0
Bent 2	163.25 – 164.0
Bent 3	163.25 – 164.0
Abut 4 (South)	179.5 – 183.5

The deepest BOF Elevation is 163.25 feet at Bents 2 and 3. The highest measured groundwater (perched) is Elevation 158 at DH-8, which is 5.25 feet (163.25-158) below the deepest BOF at Bents 2 and 3.

At Abuts 1 and 4, we have conservatively assumed the highest groundwater is Elevation 158 (at DH-8). The deepest BOF at Abuts 1 and 4 is Elevation 176.0; consequently the measured groundwater is about 18 feet below (176-158) the deepest BOF.

In summary, assuming no significant change in groundwater conditions, construction dewatering is not anticipated during widening of Bridge 37-108.

4.13.1.5 Bridge 37-409 – Yerba Buena Road UC

One LOTB sheet available for this UC is listed in PN25 in Table A-1. The LOTB sheet (as-built date of August 22, 1989) reveals B-1 (10/24/1985), a 7-foot deep penetration boring (conducted with 2¼ inch cone penetrometer), and two rotary wash Borings B-2 (10/25/1985) and B-3 (10/23/1985) which were drilled to a maximum depth of about 28 feet bgs. The two rotary wash borings encountered serpentine to terminal depth. According to a note on this as-built LOTB, "No groundwater encountered during field investigation." The bottom of the deepest Boring B-3 is about Elevation 149 feet.

This structure is supported on two (2) abutments with elevations listed in Table 4-11 below.

Table 4-11 Bridge 37-409 Footing Elevations

Location	Range of Elevations of Bottom of Existing Footing (feet)
Abut 1 (South)	163.5 to 164.4
Abut 2 (North)	151.0 to 166.4

The deepest BOF at the two abutments is Elevation 151.0. If we assume the groundwater is no higher than Elevation 149 (bottom of deepest Boring B-3), groundwater is at least 2 feet (151.0-149) below BOF. Furthermore, serpentine may be relatively impervious. In consideration of both these observations, construction dewatering is not anticipated during widening of Bridge 37-409.

4.13.1.6 Summary

In summary, based on limited existing subsurface information, construction dewatering operations are not anticipated at the five bridge sites including 37-344, 37-404, 37-347, 37-108 and 37-409.

We understand a Structure Preliminary Geotechnical Report (SPGR) is being proposed for each of the five (5) bridges listed above. The construction dewatering discussion presented herewith can be included in each of the five SPGRs.

4.13.2 New Retaining Walls

New retaining walls are planned at a number of locations including:

- Between "A" Line Stations 310+00 to 390+00, Median
- Between "A" Line Stations 506+25 to 543+75, Median
- Between "A" Line Stations 549+25 to 562+50, Median
- Between "A" Line Stations 797+00 to 803+50, West Shoulder
- Between "A" Line Stations 865+80 to 867+90, East Ramp
- Between "A" Line Stations 977+00 to 978+50, Both Sides
- Between "A" Line Stations 1141+60 to 1143+90, NE Shoulder
- Between "A" Line Stations 1238+20 to 1242+00, South Shoulder
- Between "A" Line Stations 1244+50 to 1252+00, South Shoulder
- Between "A" Line Stations 1325+00 to 1331+00, North Shoulder
- Between "A" Line Stations 1340+00 to 1342+00, South Shoulder
- Between "A" Line Stations 1549+00 to 1551+30, Both Sides, 2 Walls

Wall heights range from 4 to 10 feet. At each retaining wall location we have estimated the groundwater elevations based on the highest groundwater shown on as-built LOTB(s) of nearby boring(s). The potential need for construction dewatering at each of the 13 retaining walls will now be discussed.

Table 4-12 summarizes the 13 retaining walls (RW), designated as A through M. In addition the closest boring with highest groundwater is tabulated, as well as estimated ground surface.

Based on Table 4-12, we conclude the following:

- RW A through F
  - Groundwater is deep in relation to ground surface (about 13 feet or deeper) and BOF (assumed to be 3 feet below finished grade)
  - Construction dewatering operations are therefore not anticipated at these RWs
- RW G, L and M
  - Groundwater is about 4 or 5 feet below BOF
  - Some construction dewatering operations could be anticipated
- RW H and I
  - Assuming widening occurs at the bottom of the existing side slopes in the vicinity of 10th Street, major construction dewatering operations should be anticipated
- RW J
  - SCVWD map indicates groundwater less than 10 feet deep.
  - In close proximity to Guadalupe River
  - Some construction dewatering operations could be anticipated
- RW K
  - In close proximity to Guadalupe River
  - One nearby boring is 10 feet deep
  - Boring apparently dry at time of drilling
  - Second nearby boring has groundwater depth of 8 feet.

- If BOF is 3 feet below ground surface, may be dry excavation; if BOF is deeper maybe wet excavation requiring dewatering

It should be understood that foundation plan and profile sheets of proposed RWs are not presently available. Therefore, BOF elevations and finished grades used in this report are, at best, approximations and should be reevaluated on a wall by wall basis during design.

Table 4-12 Summary of Proposed Retaining Walls and Measured Groundwater

Retaining Wall Designation	Station Locations From	Station Locations To	Wall Length (feet)	Wall Plan Location	Nearest Boring(s) Highest GWS Elevation (feet)	Estimated Ground Surface Elevation (feet)	Comments
A	310+00	390+00	8,000	Median	Coyote Creek Golf Drive (Near STA 339+00), B-7 (06/16/1971), GWS @ 317.5	350±	350' > 317.5' Dry Excavation
					Utility Facilities, (Near STA 354+00), 00-UC-2 (02/08/2000), GWS @ 316.5'	350±	350' > 316.5' Dry Excavation
B	506+25	543+75	3,750	Median	Metcalf Road (Near STA 552+00), B-1 (04/18/1979), GWS @ 215.1'	250±	Boring outside 250' > 215.1' Dry Excavation
C	549+25	562+50	1,325	Median	Metcalf Road (Near STA 552+00), B-1 (04/18/1979), GWS @ 215.1'	250±	Boring outside 250' > 215.1' Dry Excavation
D	797+00	803+50	650	West Shoulder	Coyote Road (Near STA 803+50), DH-8 (04/15/1987), GWS @ 158' (perched)	170±	170' > 158' Dry Excavation
E	865+80	867+90	210	East Ramp	Yerba Buena Road (Near STA 871+00), B-3 (10/23/1985), Dry to EL 149'	175±	175' > 149' Dry Excavation
F	977+00	978+50	150	Both Sides	Tully Road (Near STA 978+00), B-5 (1961) GWS @ EL 109'	125±	125' > 109' Dry Excavation

Table 4-12 Summary of Proposed Retaining Walls and Measured Groundwater (continued)

Retaining Wall Designation	Station Locations From	Station Locations To	Wall Length (feet)	Wall Plan Location	Nearest Boring(s) Highest GWS Elevation (feet)	Estimated Ground Surface Elevation (feet)	Comments
G	1141+60	1143+90	230	NE Shoulder	McKee Road (At STA 1143+00), B-3 (1955±) GWS @ EL 81'	88±	88' > 81' Only 7' - 3' = 4' (includes 3' Footing Depth); Maybe Dry Excavation
H	1238+20	1242+00	380	South Shoulder	N. 10th Street Pumping Plant (At STA 1243+00), WCC-W1 OW (10/30/1992) GWS @ EL 35.1'	56 (Top of Embankment); 25 Toe of Embankment	56' > 35' Dry Excavation if @ top If @ toe, wet conditions
I	1244+50	1252+00	750	South Shoulder	N. 10th Street Pumping Plant (At STA 1243+00), WCC-W1 OW (10/30/1992) GWS @ EL 35.1'	56 (Top of Embankment); 25 Toe of Embankment	56' > 35' Dry Excavation if @ top If @ toe, wet conditions
J	1325+00	1331+00	600	North Shoulder	Borings not available. SCVWD “Figure III-1 Depth to First Ground-water for the Santa Clara Basin” map indicates depth to first groundwater at RW J is 0 to 10 feet. Also RW J is in close proximity to Guadalupe River.	Not available	Maybe wet excavation.
K	1340+00	1342+00	200	South Shoulder	Guadalupe Parkway (SR 87), (At STA 1340+00), EB-8 (11/20/1988), Boring Dry at Time of Drilling to 10 foot depth; a piezometer set in second boring Number 5	34	One Shallow Boring, Dry to Terminal Depth of 10'; second boring GWS 8' deep. Maybe

Table 4-12 Summary of Proposed Retaining Walls and Measured Groundwater (continued)

Retaining Wall Designation	Station Locations		Wall Length (feet)	Wall Plan Location	Nearest Boring(s) Highest GWS Elevation (feet)	Estimated Ground Surface Elevation (feet)	Comments
	From	To					
					(12/21/1981) measured groundwater at depth of 8 feet.		wet excavation. Also RW K is in close proximity to Guadalupe River.
L and M	1549+00	1551+30	180	Both Shoulder (2 walls)	Lawrence Expressway (At STA 1550+00), B-1 (06/15/1956), GWS @ EL. 19.5'	28	28' >19.5' Only 8.5'-3' = 5.5' (Includes 3' Footing Depth); maybe dry excavation

Notes: GWS = groundwater surface
EL = elevation
Footing embedment depth included at RW G, L and M
SCVWD = Santa Clara Valley Water District

The extensive available geotechnical data along the project alignment will form the basis of the preliminary geotechnical recommendations for the foundation design of the overhead signs. However, site specific data may not be available at the subgrade level where pavement widening is planned or at the locations of the overhead signs. Caltrans Standard Plan overhead sign structures require cast-in-drilled-hole (CIDH) piles of 14 to 25 feet in depth. Therefore, additional explorations should be planned to supplement the existing available subsurface data during the design phase. Based on the Express Lane System access points, approximately one overhead sign will be required per exit point and three signs per entry point. For planning purpose, at locations where no existing data are available, one boring should be conducted at each sign location. In addition, the pavement subgrade conditions should be investigated using shallow borings (10 feet maximum depth) at about 1,500 feet intervals. On this basis, approximately 100 borings will be needed to supplement the existing available data. The borings should be logged and sampled at selected intervals for laboratory testing. Laboratory testing should be performed on soil samples to evaluate in-situ moisture content, dry density, gradation, Atterberg limits, R-value, soil corrosivity, and consolidation characteristics. Subsurface explorations and laboratory test data will serve as the basis for estimating the engineering parameters of the materials encountered.

After borings and laboratory testing of new soil samples are completed as described in the previous paragraph, recommendations for design of foundations for bridge widening, abutment modifications, non-standard retaining wall and non-standard overhead sign will be included in separate foundation reports in the PS&E phase.

This study is intended for preliminary design purposes only. The opinions, conclusions and recommendations presented herein are based on available subsurface information developed by URS, Caltrans, and others. The preliminary recommendations presented in this report are based on the assumption that the soil and geologic conditions do not deviate substantially from those presented on the available logs of test borings. Available site specific exploration and analysis should be completed prior to the development of final design recommendations. We understand no modifications to existing bridges (over streams or rivers) are planned.

Many of the borings at bridge and retaining wall locations encountered cohesive soils (silts and clays). In cohesive soils a fairly long time is required for the groundwater to seep into the borehole and attain an equilibrium position with the present hydrostatic groundwater table. Thus the immediate readings obtained at time of drilling may or may not be representative of the actual groundwater table level at that time.

Fluctuations in the location of the hydrostatic groundwater table should be anticipated throughout the year depending upon the variations in the amount of precipitation, evaporation and surface runoff. Consequently, since some of the borings were drilled many years ago (1970s, 1980s and 2000), the present groundwater levels may vary considerably from those drilled and used in this study.

When specific modifications to existing structures and new retaining walls are identified during design, construction dewatering requirements should be reevaluated on a site by site basis.

Existing facilities, utilities, soils/bedrock conditions, road/structure distress, slope distress or groundwater/seepage conditions other than those noted herein have not been considered in the preparation of this report. Locating utilities and evaluating potential utility interference is outside the scope of this report. Individuals utilizing this report should inform URS if they are aware of any additional facilities or site conditions so that their presence and impact upon the project (or vice-versa) can be properly evaluated and recommendations modified to address geotechnical issues as necessary.

Specific review and investigation for environmental issues and subsurface environmental contamination for this project were not considered in this study, and are addressed in separate technical reports.

The opinions and recommendations presented in this report were developed with the standard of care commonly used by other professionals practicing at the same time, within the same locality and under the same limitations. No other warranties are included, either express or implied, as to the professional advice included in this report.

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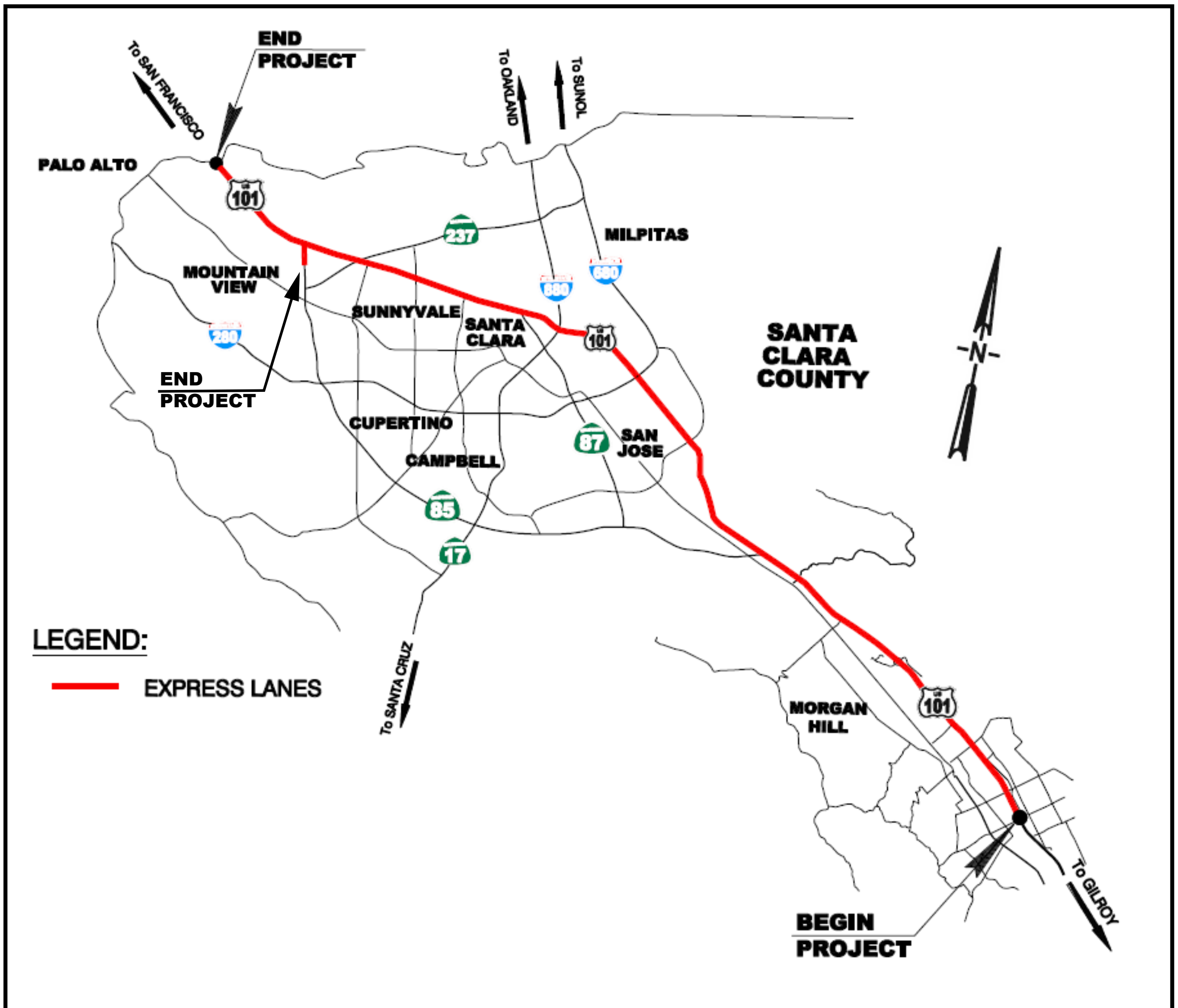
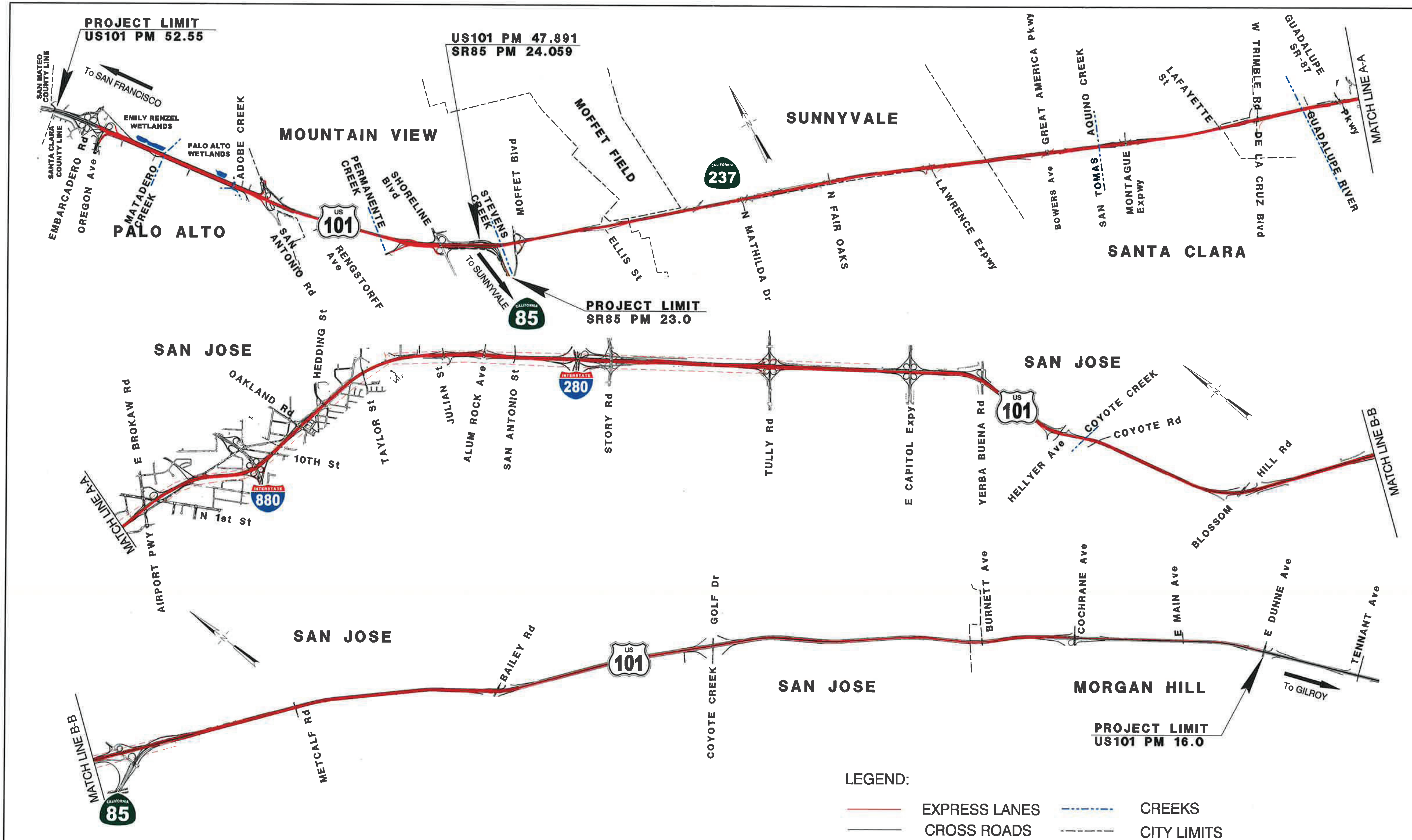
Figures



US 101 Express Lanes Project
Dunne Ave to San Mateo/Santa
Clara County Line

PROJECT LOCATION & VICINITY
Figure 1



URS

US 101 Express Lanes Project

DUNNE AVE TO SAN MATEO/SANTA CLARA COUNTY LINE

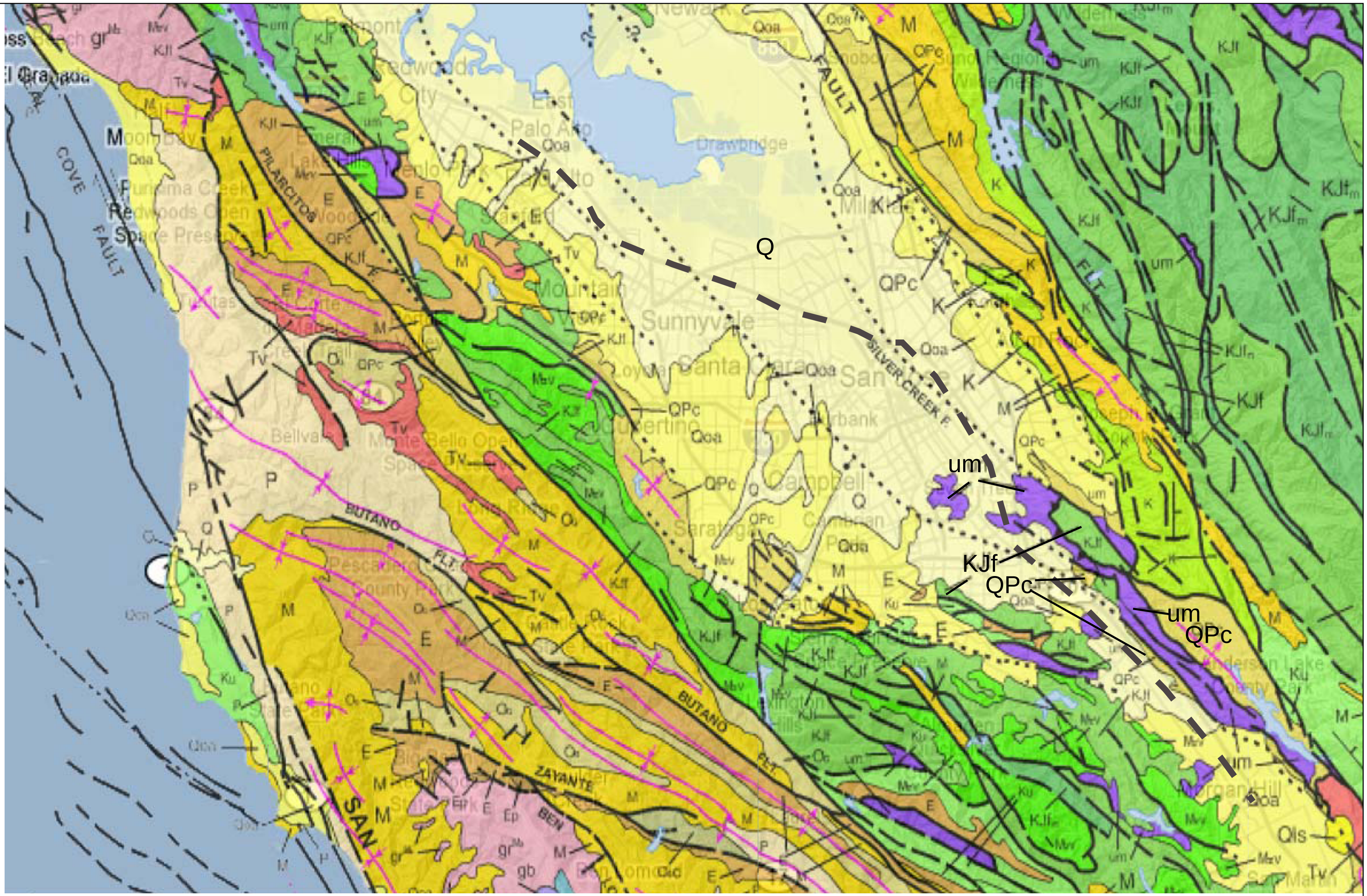
PROJECT LOCATION MAP

NOT TO SCALE

Figure 2

SEPTEMBER 2012

North



5 miles

Project Alignment

SOURCE: California Geological Survey - 2010 State Geologic Map of California

URS	28645266 US 101 Express Lanes Project	SITE AND GEOLOGIC MAP	Figure 3A

GEOLOGIC LEGEND

(GENERALIZED DESCRIPTION OF ROCK TYPES)

CENOZOIC

MARINE SEDIMENTARY ROCKS

QUATERNARY (Pleistocene - Holocene)

Qs — Extensive marine and nonmarine sand deposits, generally near the coast or desert playas.

Q — Alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated. Mostly nonmarine, but includes marine deposits near the coast.

Qoa — Older alluvium, lake, playa, and terrace deposits

TERTIARY

Pliocene: P — Sandstone, siltstone, shale, and conglomerate; mostly moderately consolidated.

Miocene: M — Sandstone, shale, siltstone, conglomerate, and breccia; moderately to well consolidated.

Oligocene: Ø — Sandstone, shale, conglomerate; mostly well consolidated.

Eocene: E — Shale, sandstone, conglomerate, minor limestone; mostly well consolidated.

Paleocene: Ep — Sandstone, shale, and conglomerate; mostly well consolidated.

NONMARINE (CONTINENTAL) SEDIMENTARY ROCKS

Qls — Selected large landslides, such as the Blackhawk Slide on the north side of San Gabriel Mountains; early to late Quaternary.

Qg — Glacial till and moraines. Found at high elevations mostly in the Sierra Nevada and Klamath Mountains.

QPc — Pliocene and/or Pleistocene sandstone, shale, and gravel deposits; mostly loosely consolidated.

Mc — Sandstone, shale, conglomerate, and fanglomerate; moderately to well consolidated.

Tc — Undivided Tertiary sandstone, shale, conglomerate, breccia, and ancient lake deposits.

Øc — Sandstone, shale, and conglomerate; mostly well consolidated.

Ec — Sandstone, shale, conglomerate; moderately to well consolidated.

VOLCANIC ROCKS

Qrv | Qrv^p

Qrv: Recent (Holocene) volcanic flow rocks; minor pyroclastic deposits.

Qrv^p: Recent (Holocene) pyroclastic and volcanic mudflow deposits.

Qv | Qv^p

Qv: Quaternary volcanic flow rocks; minor pyroclastic deposits.

Qv^p: Quaternary pyroclastic and volcanic mudflow deposits.

Tv | Tv^p

Tv: Tertiary volcanic flow rocks; minor pyroclastic deposits

Tv^p: Tertiary pyroclastic and volcanic mudflow deposits.

Ti — Tertiary intrusive rocks; mostly shallow (hypabyssal) plugs and dikes.

PLUTONIC ROCKS

gr^Cz — Cenozoic (Tertiary) granitic rocks - quartz monzonite, quartz latite, and minor monzonite, granodiorite, and granite; found in the Kingston, Panamint, Amargosa, and Greenwater Ranges in southeastern California.

MESOZOIC, PALEOZOIC, PRECAMBRIAN

MARINE SEDIMENTARY AND METASEDIMENTARY ROCKS

TERTIARY - CRETACEOUS: TK — Sandstone, shale, and minor conglomerate in coastal belt of northwestern California; included by some in Franciscan Complex. Previously considered Cretaceous, but now known to contain early Tertiary microfossils in places.

CRETACEOUS: Ku — Upper Cretaceous sandstone, shale, and conglomerate.

Kl — Lower Cretaceous sandstone, shale, and conglomerate.

K — Undivided Cretaceous sandstone, shale, and conglomerate; minor nonmarine rocks in Peninsular Ranges.

KJf | KJf$_m$ | KJf$_s$

KJf: Franciscan Complex: Cretaceous and Jurassic sandstone with smaller amounts of shale, chert, limestone, and conglomerate. Includes Franciscan melange, except where separated - see KJf$_m$.

KJf$_m$: Melange of fragmented and sheared Franciscan Complex rocks.

KJf$_s$: Blueschist and semi-schist of Franciscan Complex.

JURASSIC: J — Shale, sandstone, minor conglomerate, chert, slate, limestone; minor pyroclastic rocks

TRIASSIC: Ћ — Shale, conglomerate, limestone and dolomite, sandstone, slate, hornfels, quartzite; minor pyroclastic rocks.

sch — Schists of various types; mostly Paleozoic or Mesozoic age; some Precambrian.

PERMIAN: Pm — Shale, conglomerate, limestone and dolomite, sandstone, slate, hornfels, quartzite; minor pyroclastic rocks.

ls — Limestone, dolomite, and marble whose age is uncertain but probably Paleozoic or Mesozoic.

CARBONIFEROUS: C — Shale, sandstone, conglomerate, limestone, dolomite, chert, hornfels, marble, quartzite; in part pyroclastic rocks.

Pz — Undivided Paleozoic metasedimentary rocks. Includes slate, sandstone, shale, chert, conglomerate, limestone, dolomite, marble, phyllite, schist, hornfels, and quartzite.

DEVONIAN: D — Limestone and dolomite, sandstone and shale; in part tuffaceous.

SILURIAN - ORDOVICIAN: SO — Sandstone, shale, conglomerate, chert, slate, quartzite, hornfels, marble, dolomite, phyllite; some greenstone.

CAMBRIAN: € — Sandstone, shale, limestone, dolomite, chert, quartzite, and phyllite; includes some rocks that are possibly Precambrian.

PRECAMBRIAN: p€ — Conglomerate, shale, sandstone, limestone, dolomite, marble, gneiss, hornfels, and quartzite; may be Paleozoic in part.

MIXED ROCKS

gr-m — Granitic and metamorphic rocks, mostly gneiss and other metamorphic rocks injected by granitic rocks. Mesozoic to Precambrian.

m — Undivided pre-Cenozoic metasedimentary and metavolcanic rocks of great variety. Mostly slate, quartzite, hornfels, chert, phyllite, mylonite, schist, gneiss, and minor marble.

p€c — Complex of Precambrian igneous and metamorphic rocks. Mostly gneiss and schist intruded by igneous rocks; may be Mesozoic in part.

METAVOLCANIC ROCKS

Mzv — Undivided Mesozoic volcanic and metavolcanic rocks. Andesite and rhyolite flow rocks, greenstone, volcanic breccia and other pyroclastic rocks; in part strongly metamorphosed. Includes volcanic rocks of Franciscan Complex: basaltic pillow lava, diabase, greenstone, and minor pyroclastic rocks.

mv — Undivided pre-Cenozoic metavolcanic rocks. Includes latite, dacite, tuff, and greenstone; commonly schistose.

Pzv — Undivided Paleozoic metavolcanic rocks. Mostly flows, breccia, and tuff, including greenstone, diabase and pillow lavas; minor interbedded sedimentary rocks.

PLUTONIC ROCKS

gr^Mz — Mesozoic granite, quartz monzonite, granodiorite, and quartz diorite.

um — Ultramafic rocks, mostly serpentine. Minor peridotite, gabbro, and diabase; chiefly Mesozoic.

gb — Gabbro and dark dioritic rocks; chiefly Mesozoic.

gr — Undated granitic rocks.

gr^Pz — Paleozoic and Permo-Triassic granitic rocks in the San Gabriel and Klamath Mountains.

gr^p€ — Precambrian granite, syenite, anorthosite, and gabbroic rocks in the San Gabriel Mountains; also various Precambrian plutonic rocks elsewhere in southeastern California.

SYMBOLS

Geologic boundary

?
Fault traces, solid where well located; dashed where approximately located or inferred; and dotted where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Many concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trends only. For faults color-coded according to recency of movement, see FAULT ACTIVITY MAP OF CALIFORNIA, GEOLOGIC DATA MAP SERIES, MAP NO. 6 (2010).

?
Ball and bar on downthrown side (relative or apparent).

?
Arrows indicate direction of lateral movement (relative or apparent).

Thrust fault (barbs on upper plate), solid where well located; dashed where approximately located or inferred; and dotted where concealed by younger rocks or by lakes or bays. Fault surface generally dips less than 45 degrees, but locally may have been subsequently steepened.

Regional strike and dip of stratified rocks.

Regional strike and dip of stratified rocks (overturned).

Anticlinal fold.

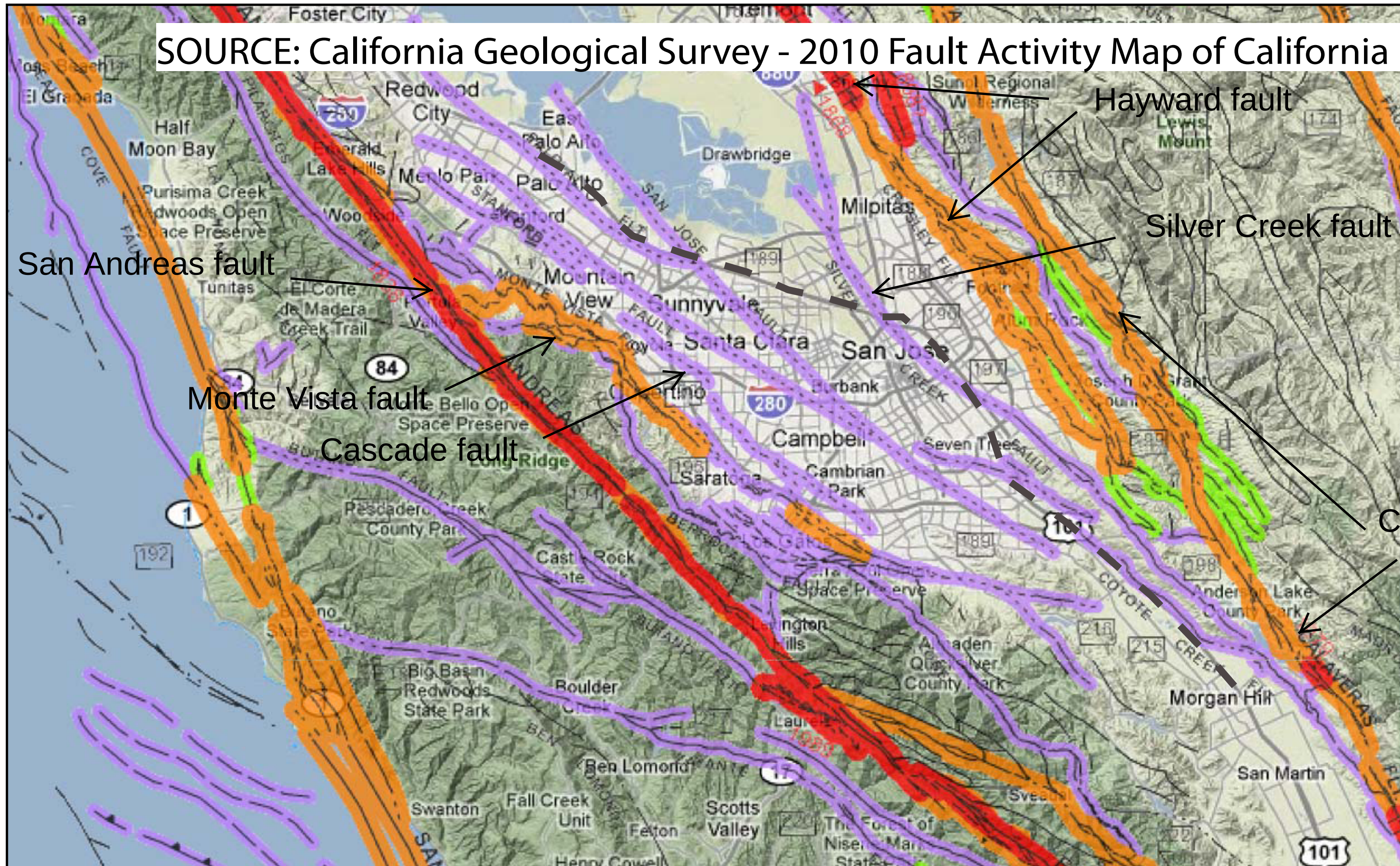
Synclinal fold.

Dotted offshore and where concealed under alluvium in the Great Valley and elsewhere. Arrow indicates direction of plunge. Concealed folds may be confined to certain units, and their location may be approximate.

Monoclinal fold.

Structural discontinuity in the offshore region.

Volcano or cinder cone.



North

5 miles

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	11,700			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	700,000			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		1,600,000*			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			4.5 billion (Age of Earth)			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

— — — Project Alignment

FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

URS	28645266 US 101 Express Lanes Project	FAULT ACTIVITY MAP	Figure 4

Liquefaction Susceptibility Map

Susceptibility Level

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

N

Scale: 1 inch = 6.59 miles

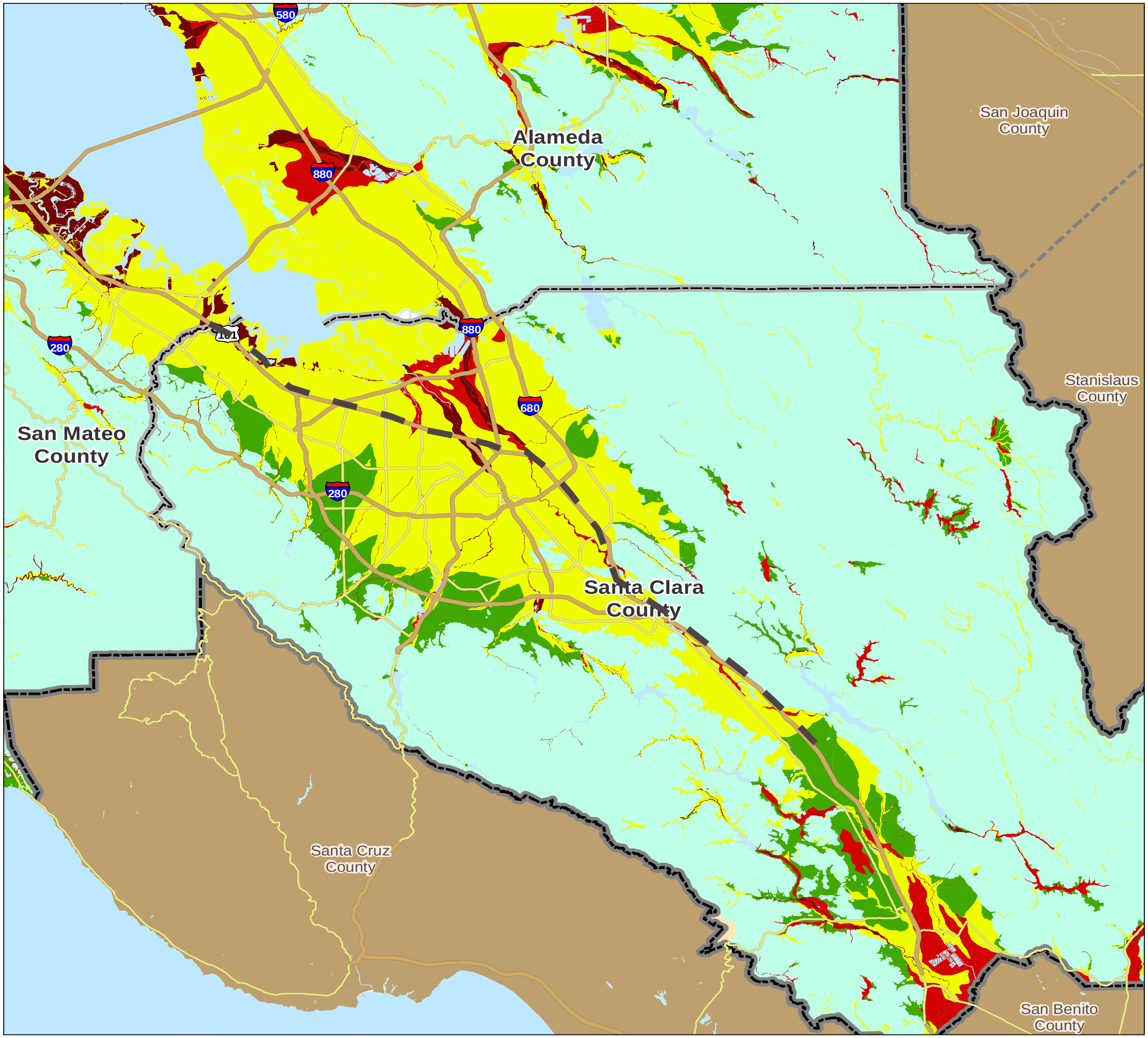
This map is intended for planning use only and is not intended to be site-specific. Rather, it depicts the general hazard level of a neighborhood and the relative hazard levels from community to community. Hazard levels are less likely to be accurate if your neighborhood is on or near the border between two zones. This information is not a substitute for a site-specific investigation by a licensed professional.

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

Sources:
This map is based on work by William Lettis & Associates, Inc. and USGS.
USGS Open-File Report 00-444, Knudsen & others, 2000 and
USGS Open-File Report 2006-1037, Witter & others, 2006

For more information visit:
http://pubs.usgs.gov/of/2000/of00-444/
http://pubs.usgs.gov/of/2006/1037/

ABAG Geographic Information Systems



URS	28645266 / 101 Express Lanes Project	LIQUEFACTION SUSCEPTIBILITY MAP	Figure 5

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.

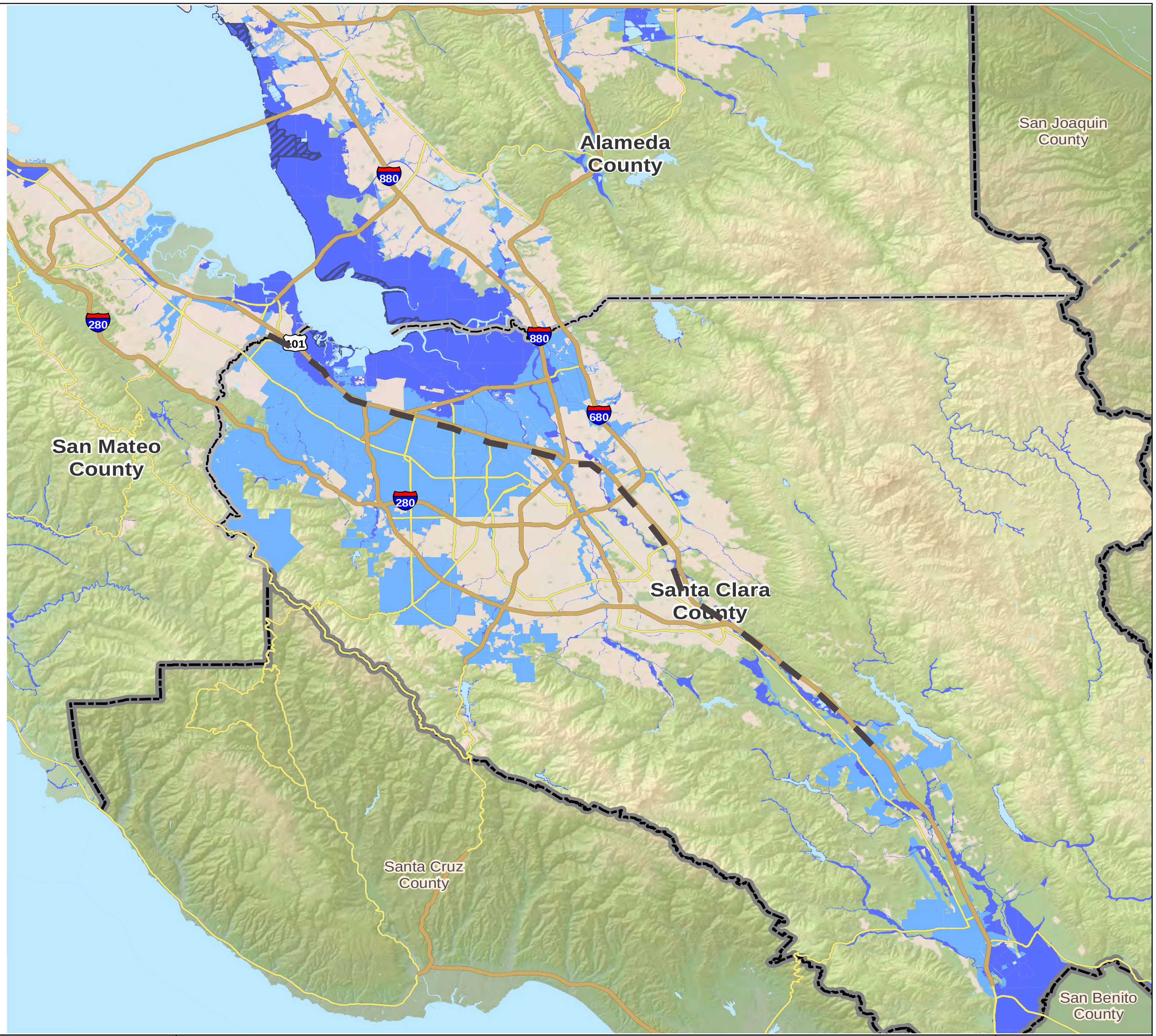
N

Scale: 1 inch = 6.58 miles

Project Alignment

Sources:
Flood Zones - FEMA Q3 (2003) and DFIRM (2009)
Base Data - TeleAtlas (2008)
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



URS	28645266 US 101 Express Lanes Project	FEMA FLOOD HAZARD AREA	Figure 6

Table A-1
Available As-built Bridge, Structure and Materials Report Plans

Project Number (PN)	Bridge/Street/Over-or Undercrossings	Post Miles	Bridge Number	As-built Contract Number	Date	Plan Type
PN01	East Dunne Avenue OC	16.0	37-334	04-117354	-	Structures
PN01	East Dunne Avenue OC	16.0	37-0592	04-232504	01/05/2001	Structures
PN02	East Main Avenue OC	16.8	37-335	04-117354	-	Structures
PN03	Cochrane Road OC	17.8	37-341	04-117354	-	Structures
PN04	From 0.06 mile N. of Cochrane to 0.06 mile N. of Metcalf	17.8-25.3	-	-	-	Materials Report
PN05	Metcalf Road to Blossom Hill Road, From PM 25.2 to PM 27.9	25.2-27.9	-	-	-	Materials Report
PN06	Blossom Hill Road (Route 82/101 Separation)	28.2	37-348 R/L	04-117324	-	Structures
PN07	Bernal Road to 280/680/101 IC Vicinity Map	R27.0-34.8	-	-	-	Materials Report
PN08	280/680/101 IC to near DeLa Cruz Blvd Vicinity Map	34.8-40.6	-	-	-	Materials Report
PN09	0.1 mile S. of N. 4th Street to 0.2 mile S. of Berryessa Road	37.2-38.8	-	-	-	Materials Report
PN10	San Antonio Street to Guadalupe River	34.8-40.7	-	-	-	Materials Report
PN11	Lafayette Street (Santa Clara-Alviso Road) OC	-	37-170	61-4TC10	-	Structures
PN12	San Tomas Aquino Creek Bridge (Widen)	42.3	37-41 R/L	04-487104	02/22/87	Structures
PN13	Bowers Avenue OC	42.7	37-390	04-428314	11/21/75	Structures
PN14	Lawrence Expressway OC (Replace)	43.9	37-152	04-125724	09/04/98	Structures
PN15	Fairoaks Avenue OC	-	168	61-4TC10	2/26/1992	Structures
PN16	Moffett Field Depressed Structure	-	-	-	-	SCVTA(1)
PN17	Moffett Field LRT Station	-	-	-	-	SCVTA(1)
PN18	Moffett Field Overhead	47.9	37-72	-	-	Structures(1)
PN19	Route 85/101 IC (GDR/MR)	76.9-79.2 (2)	37-054TK	-	-	Structures
PN20	U.S. 101 Auxiliary Lanes from Embarcadero to SR 85	48.7-51.9	-	-	-	Materials Report

Table A-1
Available As-built Bridge, Structure and Materials Report Plans

Project Number (PN)	Bridge/Street/Over-or Undercrossings	Post Miles	Bridge Number	As-built Contract Number	Date	Plan Type
PN21	Coyote Creek Golf Drive UC	19.2	37-344	04-409904	01/18/2004	Structures
PN22	Utility Facility UC	21.55	37-404	04-117324	08/18/1982	Structures
PN23	Bernal Road UC	27.01	37-347	04-437904	05/25/1995	Structures
PN24	Coyote Road UC	29.72	37-108	04-437404	03/29/1990	Structures
PN25	Yerba Buena Road UC	31.0	37-409	04-393844	08/22/1989	Structures

(1) For Light Rail Transit (LRT)
(2) KP

Review Discipline: Geotechnical – PGR – US 101 Express Lanes Project **Reviewer:** Meng-Hsi Hung, Caltrans
Comments Due By: **Meeting Date:**
Date Reviewed By: **Next Meeting Date:** TBD

TECHNICAL COMMENT REVIEW AND RESPONSE

Project Description: US 101 Express Lanes Project (EA 04-2G7100) Dunne Ave in Morgan Hill to the Santa Clara/San Mateo County Line in Palo Alto

VTA Contract No.: S06119 **Consultant Contract No.:** 28645266

Submittal Title: PGR

Updated: April 25, 2013

Project Phase/Milestone:

CODE: A-Will Comply; **B**-Caltrans to Evaluate; **C**-Will Not Incorporate; **D**-Agency (URS/VTA) to Evaluate; **E**-No Action Required; **F**-Will Incorporate Later Phase

AGCY	INIT	Item No.	Drawing or Page No.	Review Comments	Designer Responses	Revised by	JRT Disp.	Final Disp.	Verified By/Sign
CT	Meng-Hsi Hung	1		The bottom of footing (BOF) elevations for bridges 37-404 (Utility Facility UC) and 37-108 (Coyote Rd UC) are not shown in the attached PN22 and PN24 plans, respectively. Therefore, we are not able to comment on the accuracy of elevation descriptions.	Please see updated Appendix A.	SH	A	A	
		2		We have reviewed the attached files. The BOF (bottom of footing) elevations for Abut. 1 and 4 shown on '37-0108L Coyote Road-abut_retrofit2.pdf' file do not match the BOF elevations stated in Table 4-10, page 4-16, of the PGR. It seems that the BOF elevations presented in Table 4-10 were quoted from 'Abutment 1 Layout' and 'Abutment 4 Layout' as-built plans, dated 3-29-90, under EA 04-437404, for Coyote Rd UC (Widen) project (BIRIS: Bridge No. 37 0108R). The consultant needs to update the plans and clarify the statement for BOF elevations accordingly.	Please refer to updated Table 4-10 and Appendix A.	SH	A	A	

Review Discipline: Geotechnical – PGR – US 101 Express Lanes Project **Reviewer:** Meng-Hsi Hung, Caltrans

Comments Due By: **Meeting Date:**

Date Reviewed By: **Next Meeting Date:**

TECHNICAL COMMENT REVIEW AND RESPONSE

Project Description: US 101 Express Lanes Project (EA 04-2G7100) Dunne Ave in Morgan Hill to the Santa Clara/San Mateo County Line in Palo Alto

VTA Contract No.: S06119 **Consultant Contract No.:** 28645266

Submittal Title: PGR

Updated: April 10, 2013

Project Phase/Milestone:

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AGCY	INIT	Item No.	Drawing or Page No.	Review Comments	Designer Responses	Revised by	JRT Disp.	Final Disp.	Verified By/Sign
CT	MH	1	General	Page numbers shown in our review comments may not be referred correctly due to MS Word comment mark-ups and/or format glitches between different Word versions. A .pdf format is preferred for review.	Please refer to updated hard copy of report.	SH	A	A	MT
CT	MH	2	Sections 4.13 and 4.14/ pp. 4-14 to 4-21	Comparing the TOC and main content of the report, the list numbers for Sections 4.13 and 4.14 (including all sub-sections) in the main content may be misplaced. Please correct accordingly.	Please refer to updated hard copy of report.	SH	A	A	MT
CT	MH	3	Sections 4.14.1.4 (4.13.1.4?) and 4.14.1.5 (4.13.1.5?) /pp. 4-17 to 4-18	(1) ‘As-built’ should be mentioned in the description of log of test borings (LOTBs) if the LOTBs are not original; (2) PN No. for the referenced LOTBs needs to be stated in the report; (3) No foundation layout plan showing the BOF of bridge footings is attached in the revised submittal. Therefore, we are not able to comment on the BOF elevation conversions.	1) Have included “as-built” if LOTBs are not original. 2) Have included PN number for the referenced LOTBs. 3) Have included foundation plans in updated Appendix A of report.	SH	A	A	MT

Review Discipline: Geotechnical – PGR – US 101 Express Lanes Project **Reviewer:** Meng-Hsi Hung, Caltrans

Comments Due By: **Meeting Date:**

Date Reviewed By: **Next Meeting Date:** TBD

TECHNICAL COMMENT REVIEW AND RESPONSE

Project Description: US 101 Express Lanes Project (EA 04-2G7100) Dunne Ave in Morgan Hill to the Santa Clara/San Mateo County Line in Palo Alto **VTA Contract No.:** S06119 **Consultant Contract No.:** 28645266	**Submittal Title:** PGR **Updated:** January 16, 2013	**Project Phase/Milestone:**

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CT	Meng-Hsi Hung	1		The project EA should be read “2G7100”..	Have updated the EA.	SH	A	A	SC
CT	MH	2a	Table of Content/ page ii	Several tables in the report have been added/revised. Therefore, the ‘List of Tables’ needs to be updated accordingly.	Have updated the table of content.	SH	A	A	SC
CT	MH	3	Section 2.3.1/p. 2-3	The lower border line of rows 1, 2 and 3 on the first column of the continued Table 2-1 need to be eliminated if the listed reports have same post mile limits.	Have updated Table 2-1 format.	SH	A	A	SC
CT	MH	4	Sections 4.13.1.1 to 4.13.1.5/pp.4-14 to 4-18	‘As-built’ should be mentioned in the description of log of test borings (LOTBs) if the LOTBs are not original.	Have included “as-built” reference to LOTBs that consist of the As-Built Title Block.	SH	A	A	SC
CT	MH	5	Sections 4.13.1.1 to 4.13.1.5/pp.4-14 to 4-18	Most of the ‘bottom of footing’ (BOF) elevations appear to be incorrectly converted from meters into feet. Please re-check all BOF conversions and correct accordingly.	Have modified and updated elevations where referenced datum differed from original research results.	SH	A	A	SC

Review Discipline: Geotechnical – PGR – US 101 Express Lanes Project **Reviewer:** Meng-Hsi Hung, Caltrans

Comments Due By: **Meeting Date:**

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Project Description: US 101 Express Lanes Project (EA 04-2G7100) Dunne Ave in Morgan Hill to the Santa Clara/San Mateo County Line in Palo Alto

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CT	MH	6	Section 4.13.1.1/p. 4-14	Please contact Structure Design of Caltrans to confirm the official name of the Bridge 37-344.	According to Bridge Inspection Report (BIRIS), the name of Bridge 37-344 is "Coyote Creek Road Undercrossing." The section title has been revised to Coyote Creek Road Undercrossing. However, a reference to Coyote Creek Golf Drive has been added to reflect current exit ramp name on overhead sign and local street map.	SH	A	A	SC
CT	MH	7	Section 4.13.1.3/p. 4-16	Referring to LOTBs of Project Number (PN) 23 in Appendix A, the as-built LOTBs (05/25/95) mentioned in paragraph 3 is not for Bridge 7-347 but for Bridge 37-547 R/L. Please revise.	The as-built LOTB sheet for Bridge 37-547 has been removed from Appendix A, because it was for Bernal Road Undercrossing at Route 85 that is about 1000 feet south of the project alignment.	SH	C	A	SC
CT	MH	8	Sections 4.13.1.4 and 4.13.1.5/pp. 4-16 to 4-17	PN No. for the referenced LOTBs needs to be stated in the report. Also, no foundation layout plan showing the BOF of Bridge footings is attached. Please revise accordingly.	Have added the referenced PNs (including foundation plans) in the updated report.	SH	A	A	SC

Review Discipline: Geotechnical – PGR – US 101 Express Lanes Project **Reviewer:** Meng-Hsi Hung, Caltrans

Comments Due By: **Meeting Date:**

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TECHNICAL COMMENT REVIEW AND RESPONSE

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CT	MH	9	Section 4.13.2/p. 4-18	The name of the alignment needs to be noted. Also, retaining wall layout plans need to be included in the PGR.	Retaining wall alignment names have not been established at this phase of the project. All referenced station intervals are based on the mainline stations. Copies of the Pavement Delineation Plans with locations of retaining walls have been provided under a separate cover as reference. Retaining wall layout sheets will be developed for the PS&E phase.	SH	C	A	SC
CT	MH	10	Section 4.13.2/p. 4-21	The beginning station (1549+00) of retaining walls L and M shown in Table 4-12 is different from that presented in page 4-18 (1549+50). Please correct.	The beginning station of wall L should have been 1549+00, and have been modified in the report to be consistent with Table 4-12.	SH	A	A	SC