State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MOHSEN PAZOOKI
Project Manager, Ben/Mart Bridge
Division of Toll Bridge Program

Date: April 30, 2007

Attention: S. Pawar
H. Syed
W. Degui

File: 4-CC-680-KP 38.0/41.0
4-Sol-680 KP-0.0/0.8
4-Sol-780-KP L1.0/1.1
4225-006051
Marina Vista/I-680/680/780 I/C

H. Nikoui

From: HOOSHMAND NIKOUI
Chief, Branch A
Office of Geotechnical Design – West
Geotechnical Services
Division of Engineering Services

Subject: Geotechnical Design Report

This report presents the results of our geotechnical and foundation studies performed specifically for the proposed modification and diversion of the existing Route I-680 at Marina Vista Interchange from 0.5 km South of Mococo OH to Route I-680 at Bayshore I/C and on Route 780 to East 5th Street/780 I/C.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the roadway portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of changed site conditions.

Conclusions and recommendations presented in this report are based on our subsurface exploration and laboratory testing programs. Variation between anticipated and actual subsurface soil conditions may be found in localized areas during construction. Geotechnical Design West, Branch-A should be contacted for review and supplemental recommendations if significant variations in subsurface conditions are encountered during construction.

MR. MOHSEN PAZOOKI
Attn: Pawar/Syed/Degui
April 230, 2007
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This report is intended for use by the Project Design Engineer, Construction Personnel, Bidders and Contractors.

We appreciate the opportunity to provide these services to you. If you have any questions about this report, please do not hesitate to call me at 510-286-4811.

c: TPokrywka, HNikoui, Construction (3), Hydraulics (2), Daily File, Project File, Translab File

HNikoui/mm

REGISTERED PROFESSIONAL ENGINEER
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1. INTRODUCTION

This project will complete the overall Benicia-Martinez Project system. It proposes to divert the Route I-680 northbound traffic to the new Benicia Martinez Bridge, and the Route I-680 southbound traffic to the existing Benicia- Martinez Bridge. This Geotechnical Design Report (GDR) is specifically prepared for Contract EA #04-0060A1 and covers the following major activities of this project (there are no significant geotechnical issues on the remaining activities):

1) Reconstruction of existing Route I-680 south of the existing Benicia- Martinez Bridge including the Mococo OH to provide a uniform grade across all lanes.

2) Fixing of the undulations south of Mococo Overhead and on Marina Vista Road.

3) Construction of Pedestrian and Bike Path (PBP) from Park Road in the city of Benicia, Solano County to Vista Point, and continuing on to Marina Vista in the city of Martinez, Contra Costa County through the modified existing Benicia-Martinez Bridge.

1.1 Location of Project

The project is located in Contra Costa and Solano Counties in Martinez and Benicia on Route 680 from 0.5 mile south of Marina Vista I/C to 680 at Bayshore I/C and on Route 780 to East 5th Street 780 I/C. Refer to the attached Location Map Figure 1 for the exact limits of the project.

2. EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

2.1 Project Background / Existing Facilities

Route I-680, within the limits of this project (EA #04-0060A1), was constructed under Contract EA #04-61-4MBC-1 in the early 1960's. The portion of Route I-680 and the on and off-ramps just south of the existing Mococo Overhead was originally constructed on an approximately 3 m high engineered fill, across a tidal marshland. During the original fill construction, a substantial amount of fill material was allowed to sink into the very soft and highly compressible bay mud and peat (providing a working platform), enabling the construction crew to build the embankment to the as-built profile grade. Due to the variable thickness of soft foundation soil (approximately 3 meters to 18 meters of bay mud and peat) immediately beneath the roadway fill and the weight of the fill itself, the existing freeway has experienced severe differential settlement. The differential settlement ranges from a few cm to more than 2.75 m. The maximum settlement of the roadway fill has occurred within two distinct limits along the roadway (I-680) alignment. The so-called southern undulation starts from Station "H" 90+50

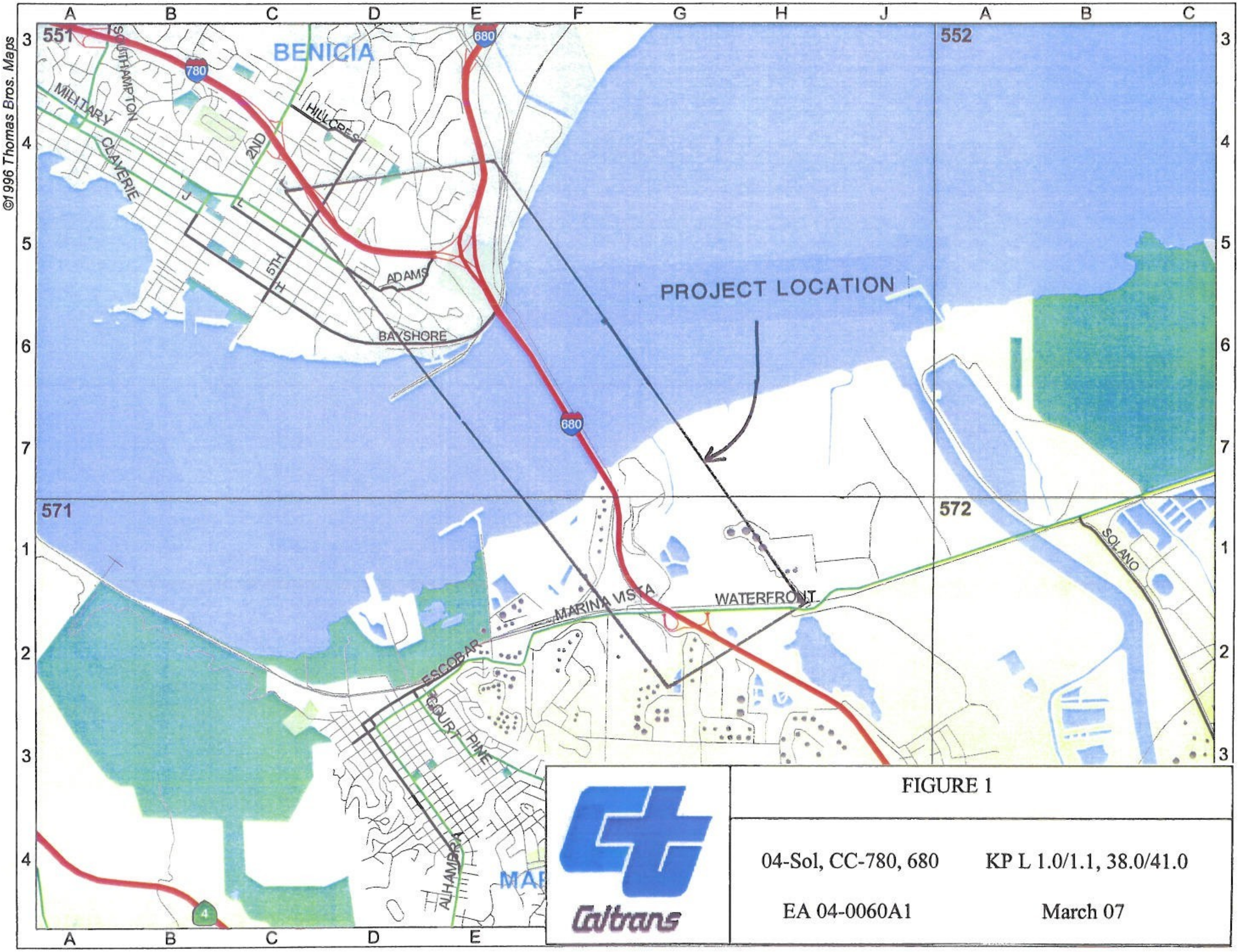


FIGURE 1

04-Sol, CC-780, 680 KP L 1.0/1.1, 38.0/41.0

EA 04-0060A1 March 07

and ends at Station "H" 93+20. The northern undulation starts from "H" Station 94+20 and ends at Station "H" 98+90. These two areas were subjected to severe flooding during heavy storms for years with periodic maintenance problems. In 1998, under emergency (Director Order) Contract EA #04-254504, the southern undulation was permanently repaired using lightweight fill (cellular concrete) and the northern undulation was temporarily fixed under the same contract by filling the undulation area with asphalt concrete. The northern undulation was not permanently fixed at the time because the existing traffic could not be detoured around the construction without widening the existing bridge at Mococo OH.

Now that the new Mococo OH Bridge has been constructed (completed in 2006) next to the existing Mococo bridge under Contract (04-006051), the so-called northern undulation can be fixed permanently under this project (04-0060A1). For details of the strategy used to construct the approach embankments to the new Mococo OH, Marina Vista/NB I-680 on –ramp approach bridge, I-680 NB off-ramp to Marina Vista, and Water Front Road using combinations of cellular concrete, polystyrene blocks and cutoff walls refer to all pertinent plans of contract plans for Project 04-006054 and our Geotechnical Design Report dated December 2000.

Exhibit A shows the soil profile within both the northern and southern undulation areas.

Under Contract EA #04-126014, prior to the repair of the undulation areas, an additional median lane and 3 m inside shoulders (including a continuous concrete median barrier) were constructed in each direction.

2.1.1 Existing Utilities

There is an existing 10' X 4' reinforced concrete box culvert crossing the I-680/Marina Vista on- and off-ramps at Station EL1 10+55.50. This box culvert appears to be on a pile foundation and is used for above ground Ultramar (formerly Tosco) oil pipes crossing. Settlement of the roadway surface around these culverts is clearly noticeable. There are numerous other utilities such as underground oil, water, telephone, gas, electrical and sewer lines that exist in Marina Vista/Waterfront Road within the project limits.

2.2 Proposed Improvements and Elements

The proposed project will:

1) Opens to traffic new bridge for northbound I-680,

2) Reconstruct the existing Route I-680 south of the existing Benicia Martinez Bridge including the Mococo OH to provide a uniform grade across all lanes,

3) Permanently correct the undulation south of Mococo Overhead (so-called northern undulation) and Marina Vista Road,

4) Repair the settlement area at the beginning of Southbound I-680 on-ramp at the intersection with the Marina Vista Road, and

5) Construct of (PBP) from Park Road in the city of Benicia, Solano County to Vista Point, and shall continue on to Marina Vista in the city of Martinez, Contra Costa County through the modified existing Benicia Bridge.

3. PERTINENT REPORTS AND INVESTIGATIONS

Reference is made to the following reports, which provided background information:

- As-Built Plans: 1961 Contract #04-61-4MBC1.
- As-Built Plans: 1998 Contract #04- 254504.
- Materials Report for the original project date and Contract # 61-4MBC-1.
- Caltrans Geotechnical Report for Environmental Study dated July 1991.
- Geotechnical Assessment Repot for Caltrans by AGS, INC. dated June 1991.
- "Final Geotechnical Design Report for Correcting Undulation Area" on Route I-680 Contract #04-254501 dated April 22, 1998.
- Materials Report for this project dated July 25, 2000 by Caltrans District 4 Materials Branch.
- Contract Plans for recently completed (2006) Project #04-006054.
- Geotechnical Design Report dated December 2000 for recently completed (2006) Project #04-006054

4. PHYSICAL SETTING

4.1 Climate

The climate of Contra Costa County is strongly influenced by its topography and proximity to the San Francisco and San Pablo Bays. These bodies of water act to mediate the temperature, cooling the region in the summer and warming it in the winter. The eastern part of the county, at some distance from the influential marine air and closer to the San Joaquin Valley, has warmer summers and cool winters. Rolling hills generally control the flow of air throughout the county. During the summer months, the warming of the Central Valley to the east draws air eastward creating strong afternoon winds. In the vicinity of the project area, the Carquinez Strait funnels westerly winds, and wind velocities range between 10 and 25 knots in the afternoon.

The average annual temperature in the project area is 15.1°C. The coolest month is January, with an average temperature of 8.0°C, and the warmest month is July with an average temperature of 21.3°C. Average annual precipitation is 444 mm with the most rainfall coming in the month of January (95.5 mm) and the least in July (0.25 mm). Humidity in Contra Costa County averages about 80 percent in the winter and 65 percent in the summer. Snow is rare, but does fall at the higher elevations within the county.

4.2 Topography and Drainage

Contra Costa County consists of four distinct physiographic regions: the Coast Ranges, the intermountain valleys, the San Francisco Bay region, and the Sacramento-San Joaquin Delta. Elevations range from sea level to 1173 m at the top of Mount Diablo.

Rolling hills and valleys of the Coast Range trend northwest with the most dominant feature being Mount Diablo. Valleys are generally young and V-shaped. The two largest valleys in the county, San Ramon and Ygnacio, separate the San Francisco Bay from Mount Diablo to the east. The San Francisco Bay borders the western-most edge of the county. The San Pablo and Suisun Bays frame the northern-county border and give way to the Sacramento-San Joaquin Delta to the east.

The largest stream in Contra Costa County, Marsh Creek, originates in the eastern half of the county and flows roughly north and east where it empties into the Sacramento-San Joaquin Delta. Mount Diablo dominates drainage patterns in the county. Sycamore, Alamo, and Tassajara Creeks in the southwestern corner of the county flow south into the Livermore Valley, while creeks in the eastern half of the county flow east into the Central Valley and Delta. In the project area, Grayson, Walnut, and Pine Creeks flow north into Suisun Bay.

4.3 Regional Geology and Seismicity

The project lies on the eastern edge of the central Coast Ranges geomorphic province. It is on the south side of the East End of the Carquinez Strait, an east-west trending gap in the northwest-trending Coast Range Mountains. The Carquinez Strait connects Suisun Bay to the east and San Pablo Bay to the west. The Carquinez Strait is thought to have been created by erosion about 600,000 to 650,000 years ago when drainage from the Central Valley spilled over the Colma Gap near San Francisco (Andre Sarna, personal communication).

Northwest-oriented ridges and valleys, faults, and folds characterize the regional geology of the area. Numerous west and southwest trending primarily dip slip faults have been mapped on regional geologic maps between the active Concord fault to the east of the project and the potentially active Franklin Fault to the west of the project (Sims and others, 1973). Cretaceous age sedimentary rocks of the

Great Valley sequence occur to the east and south of the project. Sandstone and pebble conglomerate with thin shale interbeds of the Paleocene age Martinez formation overlie the Great Valley Sequence in the area. These rocks occur in the low hills to the north and west of the project. The project is on the southwest side of a large embayment where tidal marsh deposits overlie the bedrock. These soft bay mud deposits are 2 m to 14 m thick at the project. Bedrock in the area has been folded. Sedimentary beds in the area dip steeply to the west and southwest. Refer to attached Figure 2, Geologic Map.

The project lies within the San Andreas Fault system, a seismically active transform tectonic plate boundary between the Pacific Plate to the west and the North American Plate to the east. The nearest trace of the active Concord-Green Valley Fault is 2.6 km to the northeast. It is capable of a maximum earthquake of moment magnitude of 6.9. The active Marsh Creek-Greenville Fault is 19.3 km to the southeast. It is also capable of a maximum earthquake of moment magnitude 6.9. The potentially active Franklin Fault, capable of a 6.25 magnitude earthquake, is 6.7 km to the southwest. Other major faults within 50 km of the site and capable of maximum earthquakes of magnitude 7.0 or greater are the Hayward Fault, the Calaveras Fault, the Rogers Creek Fault and the San Andreas Fault.

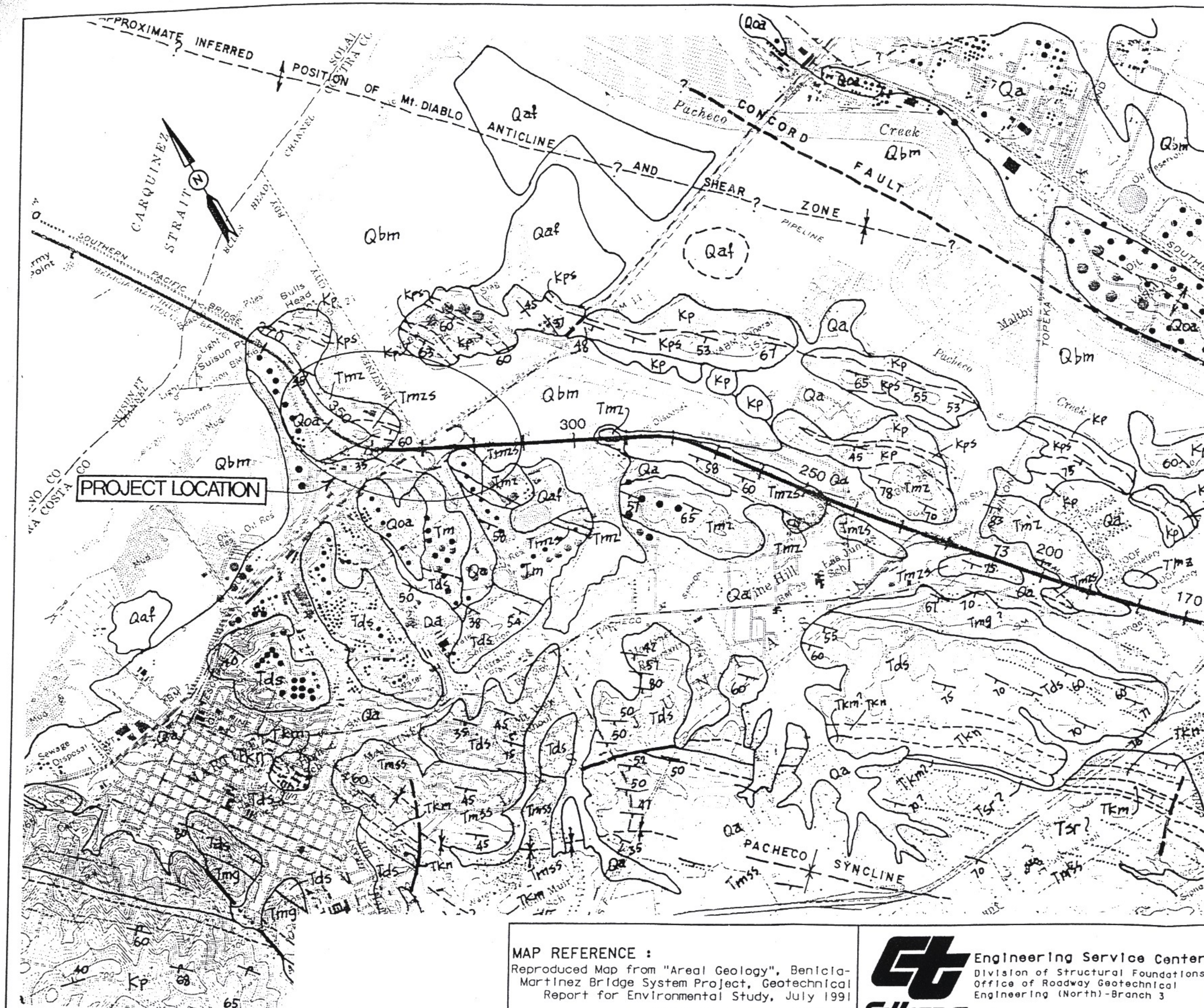
4.4 Soil Survey Mapping

Surficial soils in the project area are derived from colluvium and weathered bedrock. The Soil Survey of Contra Costa County, California by Lawrence E. Welch (1977) describes the surface soil in the project area as Omni silty clay. They are described as poorly drained soils that formed in alluvium from sedimentary rock on slopes less than 2 percent. It is classified as Lean Clay (CL) in the Unified Soil Classification (USC) system. Its shrink-swell potential is high and corrosivity is very high. Permeability is low and it has an available water capacity of 127-178 mm. The soil is subject to occasional ponding and the surface water runs off very slowly. There is no hazard of erosion. Refer to attached Figure 3, Soils Map.

5. EXPLORATION

5.1 Program

No foundation exploration for the undulation area was conducted specifically for since substantial foundation information is available from our previous investigations in 1960s (original construction of Route I-680) and between 1979 and 2000. The primary objectives of the subsurface exploration program were:



EXPLANATION

GEOLOGIC UNITS :
(Within or near the Project Limits)

Holocene		QUATERNARY
Qaf	Manmade fill	
Qbm	Bay mud	
Qa	Alluvium	
Qls	Landslide	
Pleistocene		
Qoa	Older Alluvium	
Pliocene		TERTIARY
Tsv	Andesitic flows/flow breccias	
Tst	Adesitic tuff, breccia	
Miocene		
Tmss	Buff, massive, arkosic sandstone	
Oligocene	San Ramon Formation:	
Tsr	Fine grained sand	
Eocene	Kreyenhagen Formation:	
Tkm	Markley Sandstone	
Tkn	Nortonville Shale	
Tds	Domengine Sandstone	
Tmg	Meganos Formation	
Paleocene	Martinez Formation	
Tmz	Clay, shale member	
Tmzs	Sandstone member	
	Panoche Formation:	CRETACEOUS
Kps	Arkosic sandstone	
Kp	Micaceous shale	

GEOLOGIC SYMBOLS :

Contact

Dashed where gradational or approximately located

Fault

Dashed where inferred, dotted where concealed, queried where uncertain

Sandstone bed

30° Inclined

Vertical

70° Overturned

Strike and dip of strata

anticline

syncline

MAP REFERENCE :
Reproduced Map from "Areal Geology", Benicia-Martinez Bridge System Project, Geotechnical Report for Environmental Study, July 1991

SCALE : 1 : 24000

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Office of Roadway Geotechnical Engineering (North)-Branch 3

FIGURE 2

04-CC-680	KP 38.5/40.1
04225-006051	OCTOBER 2000

GEOLOGIC MAP

SCALE 1 : 24000

MAP REFERENCE :
Soil Survey of Contra Costa County, Calif.
by Welch, L.E., 1975

SOIL LEGEND :

SYMBOL	NAME
AaE	Alo clay, 15 to 30 percent slopes
AbD	Altamont clay, 9 to 15 percent slopes
AbE	Altamont clay, 15 to 30 percent slopes
BdA	Botella clay loam, 0 to 2 percent slopes
BdE2	Briones loamy sand, 15 to 30 percent slopes, eroded
Bdf2	Briones loamy sand, 30 to 50 percent slopes, eroded
DeE	Dibble silty clay loam, 15 to 30 percent slopes
DeF	Dibble silty clay loam, 30 to 50 percent slopes
GbG	Gaviota sandy loam, 50 to 75 percent slopes
Ja	Joice muck
LcE	Lodo clay loam, 9 to 30 perecent slopes
LcF	Lodo clay loam, 30 to 50 perecnt slopes
LhE	Loss Osos clay loam, 15 to 30 percent slopes
LhF	Los Osos clay loam, 30 to 50 percent slopes
MeE	Millsholm loam, 15 to 30 percent slopes
MeF	Millsholm loam, 30 to 50 percent slopes
Oa	Omni clay loam
Ob	Omni silty clay
PkC	Positas loam, 2 to 9 percent slopes
Ra	Reyes silty clay
Tac	Tierra loam, 2 to 9 percent slopes
Ub	Urban land

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Engineering (North)-Branch 3

FIGURE 3

04-CC-680 KP 38.5/40.1
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SOILS MAP

Undulation Areas

1) To explore, and collect samples of the foundation soils along both southbound and northbound existing Route I-680 alignments and the recently constructed new roadway alignments to a reasonable depth that may significantly be influenced by the proposed fill and retaining wall construction.

2) To visually classify the various soil units encountered within the depths of exploration with field identification and in-situ testing.

3) To collect and recover both disturbed and undisturbed soil samples for laboratory testing to determine pertinent engineering characteristics such as index properties, consolidation and shear strength.

PBP (Pedestrian and Bike Path)

1) To explore, recover and collect soil/rock core samples along the proposed bike path to identify the material within the depth of the proposed excavation.

2) To perform seismic refraction along the pedestrian and bike path alignment to determine the rippability of the rock and the type of excavating equipment to be used by the contractor.

Trench excavation for the proposed Storm Drain Pipe along existing Northbound Route I-680.

1) To perform seismic refraction near the alignment of the proposed trench to determine the rippability of the rock and the type of excavating equipment to be used by the contractor.

5.2 Borings

Extensive foundation explorations have been conducted along the existing alignment of Route I-680 within the project limits both during the original construction in the early 1960s and during our recent investigation within the undulated and distressed roadway areas between 1979 and 1989. These investigations consisted of numerous exploratory soil test borings (including Modified California sampling and soil tubes borings) and Electronic Cone Penetration Test (CPT) soundings. To investigate the foundation for the new freeway alignment, upon our request, the Office of Structure Foundation (OSF) drilled several additional borings at various locations. The exploration by OSF consisted of several borings along the proposed Retaining Walls #1 and #2 between CCNB Station 96+20 and 98+60 and along the proposed structure for the northbound on-ramp (WL Line) where high embankments are proposed. In addition to the above available foundation information, Geotechnical Design West, conducted additional 8 CPT soundings (CPT-A through I) along the outside edge of the new Northbound I-680 alignment and the new NB off-ramp (WR Line). Locations, depths, detailed soil descriptions are shown graphically on the attached soil profiles (Exhibit A and Figures 5).

Two rotary wash borings with continuous coring of rock samples were drilled to appropriate depths to identify the type of soil/rock formation within the excavation limits (near Station BKS 3+75 and near Station BKS 4+50) for the proposed PBP (pedestrian and bike path).

5.3 Drilling and Sampling

All power borings were drilled using truck-mounted drill rigs. Relatively undisturbed 50-mm Modified California samples were obtained for laboratory testing. The materials encountered were logged in the field during drilling operations. Bulk and jar samples were taken for laboratory testing. For detailed descriptions of the foundation material within the undulation area refer to the attached Exhibit A and Figure 5.

Continuous rock core samples were collected during drilling (Borings B-1 and B-2) for the proposed bike path excavation, which is available for observation, by the bidders.

5.4 Instrumentation

No instrumentation was necessary for this project.

6. GEOTECHNICAL TESTING

6.1 In Situ Testing

No in situ testing other than Cone Penetration Test (CPT) and SPT blow counts was necessary for the purposes of our investigation.

6.2 Laboratory Testing

Upon completion of drilling, the samples were sealed and taken to the District and Transportation Laboratories for examination and testing. Visual soil classifications were made in the field based on Unified Soil Classification System. Laboratory tests such as moisture/density, gradation, corrosion, Atterberge Limits and consolidation tests were performed on selected samples. For test results, refer to the GDR prepared for Project EA# 04-006054.

7. GEOTECHNICAL CONDITIONS

7.1 Site Geology

Distinctly different rock types underlie the northern and southern halves of the project area. The geology of the northern part of the project area consists of Tertiary sedimentary rocks, while tidal marshes and mudflats characterize the southern portion. Paleocene marine shales and sandstones of the Martinez formation as well as Pleistocene age alluvium deposits of sand, silt, and clay comprise the small hills on the northern edge of the project area. To the south, unconsolidated bay mud and peat make up much of the subsurface. These have been deposited by the influx of fine sediments within tidal marshes and small deltas that border Suisun Bay and the Carquinez Strait. Refer to attached Figure 2, Geologic Map.

7.1.1 Lithology

Rocks that occur along the alignment include the Paleocene Martinez formation and Pleistocene alluvial and terrace deposits. The Martinez formation includes marine sandstones and shales that dip steeply to the southwest. Alluvial and terrace deposits include weakly cemented, poorly indurated, interbedded sands, shales, and clays (Dibblee, 1980). For a description of unconsolidated sediments throughout the rest of the project area refer to Section 7.2.

7.1.2 Existing Slope Stability

There are no signs of instability on any of the existing fill and cut slopes within the project limits. All existing fill and cut slopes are 1(V): 2(H) and flatter.

7.2 Subsurface Soil Conditions

Undulation Area

Borings drilled on both sides of Route I-680, and along its on- and off-ramps (within the project limits) generally describe the foundation soil as approximately 0.75 m to 6 m of embankment fill over 2.5 m to 13 m of very soft and compressible, highly saturated (moisture content of up to 300%) bay mud and peat. These materials are underlain by stiff sandy clay with silt. The fill material consists of light to dark brown silty clay with fine sandstone fragments. Refer to the attached Exhibit A and Soil Profiles Figures 5.

As mentioned in Section 2.1 of this report, the northern undulation was temporarily fixed in 1998/1999 under Contract # 04-254504 by filling the undulation area gradually with asphalt concrete (AC) up to 1.8 meters in thickness. Therefore, the thickness of the AC layer within the undulation area maybe more than 2.4 meters.

7.2.1 Bedrock

Undulation

Most of the borings drilled (by Roadway Geotechnical Branch) for this project were to identify the depth to the bottom of the soft bay mud deposits and were not deep enough to encounter bedrock. One boring drilled at near CCNB Station 92+00 to a depth of 14.6 m below the ground surface appeared to encounter soft sandstone bedrock. Borings drilled by Office of Structure Foundation (OSF) for the proposed Retaining Walls #1 and #2 encountered bedrock between elevation -12.5 m and -29 m (MSL). The bedrocks described by these borings are sandstone, siltstone and shale.

Trench Excavation for Storm Pipe

It should also be mentioned that during the excavation for construction (EA # 04-006054) of the Type 1 Retaining wall (RW #4) located along the northbound I-680, where it is proposed (under this contract) to excavate up to 4.5 m deep trench for drainage pipe, the contractor unexpectedly encountered very hard sandstone rock. The contractor therefore could not use conventional excavating equipment and used ripper to excavate the hard rock.

The seismic refraction conducted previously by our Geophysics and Geology Branch at the Toll Plaza, indicated the velocity of sandstone bedrock increased significantly where concretions exist. Our experience at the toll plaza was that rock velocity is highly variable due to presence of concretions, which tend to be limited in dimension and not easily detected by the seismic refraction method.

Due to close proximity of the proposed trench to traffic and the fact that we have sufficient foundation information, we did not take any borings along the proposed trench.

Pedestrian and Bike Path

Two borings (B-1 and B-2) were drilled on February 6, 2007 to identify the rocks within the excavation limits and depths for the pedestrian and bike path. They show soft to moderately soft sandstone and siltstone rock. Our Geophysics and Geology Branch conducted seismic refractions along the BKS line on March 2007 to better identify the rock formation along the proposed excavation. The results of the seismic refractions agreed with the two borings B-1 and B-2 we drilled along the pedestrian and bike path alignment in regards to the hardness of the sandstone and siltstone bedrock. The graphical results of the seismic refractions are attached to this report.

7.3 Water

7.3.1 Surface Water

The project area lies 1 km south of the Carquinez Strait, which separates San Pablo Bay to the west from Suisun Bay and the Sacramento Delta to the east. Drainage patterns in and around the project area flow north into the strait. These drainages commonly terminate in small deltas and marshes that may flood during very high tides or during periods of heavy rainfall. In the southern half of the project area, I-680 is underlain by fill placed over a marsh that drains into Suisun Bay. During reconstruction of I-680 to fix differential settlement through the project area, five 900-mm drain pipes were placed under the roadway near the southern edge of the project area to better move surface water from south to north. The marsh within the project limits is triangular in shape, broadening to the north into the bay. A single small drainage extends from the head of the marsh, travels underneath I-680 through a box culvert, and continues to the north where it coalesces with several other small drainage. It can be expected that this marsh will contain water during the winter months.

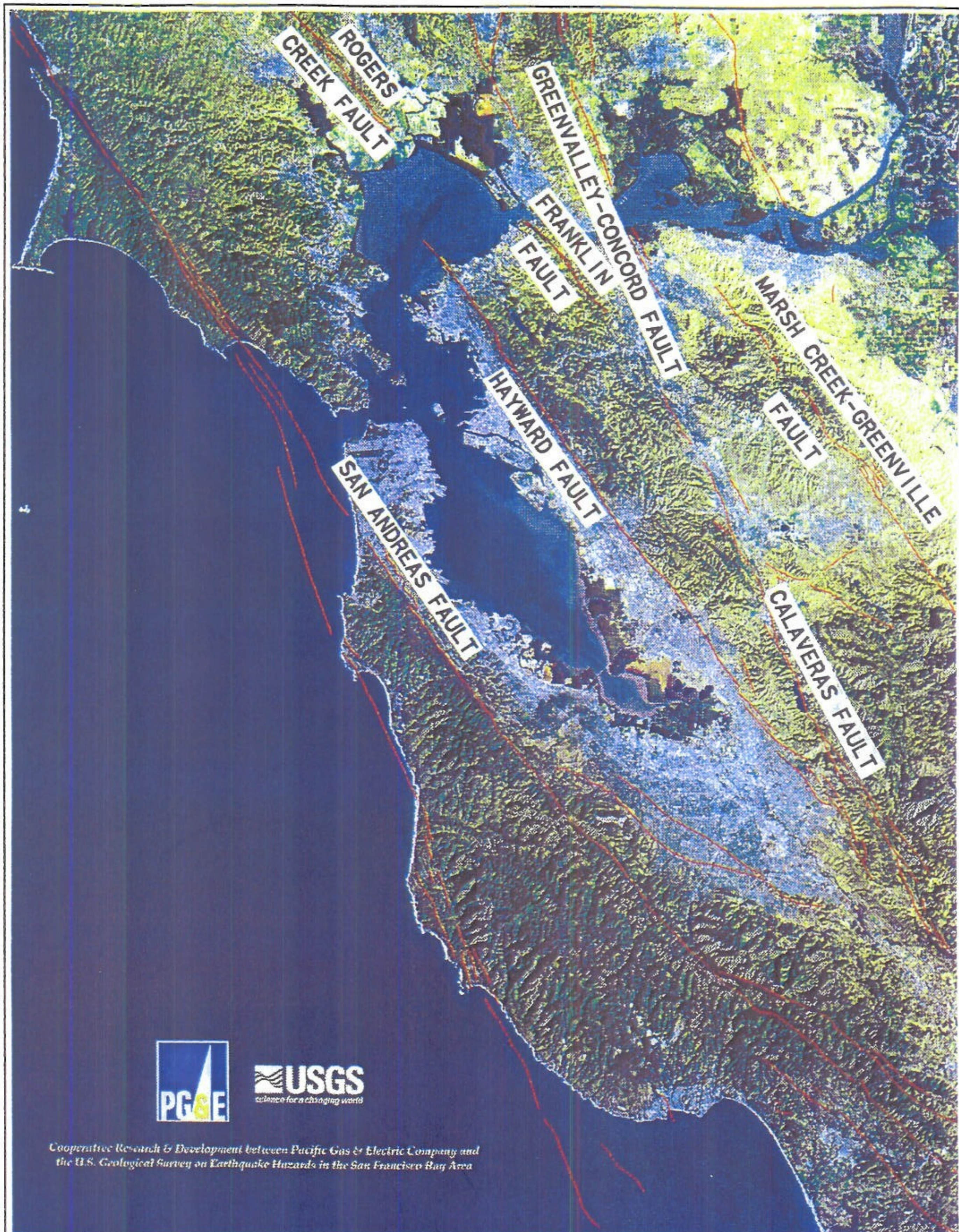
7.3.2 Groundwater

Groundwater in low lying and marshy areas fluctuates with bay water levels and range between elevation 0 m to –1.9 m. Refer to Section 8.3.3-B for groundwater mitigation during construction.

7.4 Project Site Seismicity

The project is located within the seismically active San Andreas Fault system. The San Andreas Fault system is a series of active faults that comprise a transpressional crustal plate boundary. The fault system separates the North American plate on the east from the Pacific plate on the west. Five active faults located near the project are capable of producing a major seismic event that could affect the project. These faults are the San Andreas, Hayward, Calaveras, Rogers Creek, Marsh Creek-Greenville and Concord-Green Valley. Active faults are those with most recent movement in the past 11,000 years. In addition to these active faults, the nearby Franklin Fault is potentially active and capable of producing an earthquake that could affect the project. Potentially active faults are those with most recent movement in the Quaternary period, 2 to 3 million years ago. The west to southwest trending dip slip faults in the area may not be seismogenic. They probably rupture sympathetically during large earthquakes on nearby strike slip faults of the San Andreas Fault System.

Table 1 below summarizes the nearby faults, their maximum credible earthquakes, the distance from the project and the estimated peak ground accelerations that can be expected at the project during a maximum credible earthquake. Refer to attached Figure 4 Regional Fault Map.



Caltrans	FIGURE 4
Engineering Service Center Division of Structural Foundations Office of Roadway Geotechnical Engineering (North)-Branch 3	04-CC-680 KP 38.5/40.1 04225-006051 OCTOBER 2000
	FAULT MAP

TABLE 1

Fault	*MCE*	*Distance from project*	*PGA*	*Activity*
San Andreas	8.0	48 km southwest	0.23g	Active
Hayward	7.5	19 km southwest	0.31g	Active
Calaveras	7.5	30 km south	0.22g	Active
Rogers Creek	7.0	31 km northwest	0.16g	Active
Marsh Creek-Greenville	6.9	19 km southeast	0.22g	Active
Concord-Green Valley	6.9	2.6 km east	0.58g	Active
Franklin	6.25	6.8 km southwest	0.33g	Potentially Active

A small segment of the series of east trending dip slip faults is shown on Sims and other maps (1973) crossing the project on the northwest end of the project at about CCNB Line Station 100+00. The fault is shown as concealed beneath Quaternary age bay mud. The fault is not considered active. It may rupture sympathetically during a large earthquake on one of the nearby active faults. Offsets are likely to be small, on the order of centimeters.

8. GEOTECHNICAL ANALYSIS AND DESIGN

8.1 General

The most significant geotechnical issues under this project are the existing undulation just south of Mococo OH, the proposed excavation for the Bike Path (BKS line) along the SB I-680 and the proposed trench excavation for the storm pipe along the existing NB I-680.

8.2 Cuts and Excavations

Proposed Pedestrian and Bike Path

According to the submitted cross-sections, cuts (up to 6.5 meters deep) into the adjacent existing hillside along I-680 southbound and adjacent to Bridgehead Road are required for the construction of the bike path. Within these limits (Station BKS 2+60 to BKS 5+25), we recommend the cut slope no steeper than 1V: 1.5H. Refer to the attached Typical X-Section X-5 and X-9 for limits of the proposed Bike Path (BKS Line).

Please refer to Section 8.2.2 of this report for rock rippability and type of excavation equipment requirement.

Construction de-watering may be required (Refer to Section 8.2.3 of this report).

Undulation Area

The other major cuts proposed in this project are the temporary excavations required for the foundation treatment to fix the undulation area just south of Mococo OH. Due to the proximity of the proposed excavation to the existing retaining wall and environmentally sensitive area and to maintain existing Marina Vista/I-680 SB On-ramp, shoring will be required. Please refer to the attached NSSP for "Shoring."

As mentioned in Section 2.1 of this report, the northern undulation was temporarily fixed in 1998/1999 under Contract # 04-254504 by filling the undulation area gradually with asphalt concrete (AC) up to 1.8 meters in thickness. Therefore, the contractor should be aware that within the limits of the undulation and the proposed excavation just south of the existing Mococo Overhead there exists more than 2.4 meters of asphalt concrete. ***The SSP for this project should contain statement specifying this fact.***

Construction de-watering will likely be required (Refer to Section 8.2.3 of this report).

Trench Excavation

The proposed maximum 4.5 meter deep trench excavation along the shoulder of the existing northbound I- 680 (between Station CCSB 103+05 and 105+35) will be entirely in hard sandstone rock. Due to the proximity of the proposed trench to the existing retaining wall, appropriate trench shoring will be required. ***We recommend that the length of proposed trench excavation and backfill be limited to 10 meter at a time during construction. A statement for this requirement should be provided in the contract SSP.*** Please also refer to Section 8.2.2 of this report for rock rippability and type of excavation equipment requirement.

Construction de-watering will likely be required (Refer to Section 8.2.3 of this report).

8.2.1 Cut Slope Stability

As stated above, the only major cuts planned for this project are cuts for the proposed bike path (BKS Line) and temporary excavation for the foundation treatment just south of Mococo OH. No cut slope stability problem is anticipated in this project, provided that the cut slope recommendations given in this section are followed.

8.2.2 Rippability

Excavation for the Proposed Pedestrian and Bike Path and Trench for Storm Drain Pipe

Two rotary wash power boring drilled along the proposed pedestrian and bike path alignment (BKS Line Station 2+60 to 5+25) where we will have substantial excavation during construction (up to 6.5 meters deep). Continuous coring of the rocks was obtained and is available for bidders to look at before bidding. Both borings show that the rock within the excavation depth is composed of soft to moderately soft silty sandstone. As stated in the previous section of this report, besides the two borings drilled to identify the rocks within the excavation limits, we have conducted seismic refractions along the BKS line to better identify the rock quality and rippability. The results of the seismic refractions indicate that the rock is rippable and conventional excavating equipment will be sufficed. However, ***we recommend that the SSP provide language to state that the contractor should consider the use of hoe rams or other rock-breaking equipment to deal with concretions that may be exposed during excavation.***

As stated in Section 7.2.1 of this report, during the excavation for construction (EA # 04-006054) of the Type 1 Retaining Wall (RW #4) located along the northbound I-680, **where it is proposed (under this contract) to excavate up to 4.5 m deep trench for drainage pipe**, the contractor unexpectedly encountered hard sandstone rock. Based on our experience in this area, ***we strongly believe that the major portion of the proposed trench will be excavated into hard sandstone and siltstone rocks and thus the contractor should use hoe rams or other rock-breaking equipment for the proposed trench excavation, which will be along the existing shoulder of NB I-680. Please provide this requirement in the SSP. Due to the close proximity of the proposed trench to the existing retaining wall, Shoring of the trench will be required.*** Please also refer to Section 8.2 of this report regarding recommendation for trench shoring.

8.2.3 Construction De-watering

The bottom of the excavation for the foundation treatment under the proposed lightweight fill will likely be below the groundwater level. The excavation needs to be relatively dry during the placement of lightweight fill. The recommendation for excavation de-watering is given in Section 8.3.3-B of this report.

It may be necessary to dewater all other excavations (bike path and trench for storm drain pipe), if it is performed during spring or winter. The contractor should be prepared to lower ground water level as necessary to maintain a dry and stable condition to support construction equipment.

8.2.4 Excavated Materials/Imported Borrow

The excavated material generated from the proposed excavation in this project will be the existing embankment, existing roadway pavements, existing materials from hillside excavation, and unsuitable material (bay mud and peat). The existing embankment and materials obtained from excavation of the hillsides may be used for proposed embankment construction and contour grading elsewhere in the project. The unsuitable material shall become the contractor's property and should be disposed outside of State Right-of-Way.

8.3 Embankment Fills

We have concluded that the use of lightweight fill is the most feasible and economical solution to permanently repair the undulation area just south of Mococo Overhead. By excavating the heavy asphalt concrete and existing fill material and possibly the bay mud deposits to a specific depth, backfilling the excavation with lightweight material and constructing the proposed embankment using lightweight fill, we will be able to balance the load and minimize or eliminate embankment settlement. This approach was implemented successfully in the southern undulation area on Route I-680 just south of this project, recently constructed new I-680 Northbound adjacent to this project as well as in the Cypress Project (Ala-I-880) in the City of Oakland. Base on the weight requirement and ease of construction, the most effective lightweight materials for the purpose intended is cellular concrete. Other lightweight fill materials such as natural lightweight aggregate (volcanic material), expanded shale, shredded tires and saw dust, require deeper excavation due to their heavier weight. Also, compaction requirements and availability in large quantity are the other disadvantages of these lightweight materials. Cellular concrete fill is briefly described in the following sub-sections.

8.3.1 Cellular Concrete Fill

Although, Caltrans has limited prior experience with cellular concrete, this product has been used for years as a lightweight fill material throughout the United States and Canada. Cellular concrete consists of Portland cement, water, foaming agent and accelerating set time admixtures. Specifications for construction of cellular concrete are attached to this report.

Cellular concrete will be job site batched, mixed, and placed with specialized equipment.

8.3.2 Foundation Recommendations

A. General

As stated above the most feasible and economical solution to settlement problems (undulation) at the south of Mococo Overhead is the employment of lightweight fill. The most suitable lightweight fills were determined to be cellular concrete. In order to balance the load, we made an assumption (based on the available data and the monitoring of the site) that the primary settlement due to the load of original fill including pavement placed over 30 years ago is completed. Only a negligible amount of secondary consolidation due to the decomposition of underlaying peat will continue to take place in the future. In addition a maximum 1.8- meter thick layer of asphalt concrete was placed over the entire undulated area south of Mococo Overhead as a temporary fix under Contract 04-254504 in 1998/1999. This additional load has further consolidated the foundation soil (bay mud) in this subject area. In order to balance the load, it is required to sub-excavate below the roadway surface within the limits and depths shown on the attached Plans Sheets L-3, L-4 and L-5, Profile Sheets C-1 and C-3 and Typical X-Sections (X-2, X-3 and X-8) and backfill with lightweight fill (cellular concrete).

B. Groundwater Problem/Uplift Pressure During and After Lightweight Fill Construction

All sub-excavated areas should be filled with cellular concrete (unit weight = 4.7 to 6.3 kN/m^3).

The main concern during the placement of the cellular concrete is the uplift pressure from high tides. We strongly recommend that at the bottom of excavation, a 200-mm thick layer of permeable material (PM-3) blanket be placed on filter fabric and covered with geomembrane (Type B) prior to placement of cellular concrete. For the locations of the permeable blanket refer to the attached Typical Cross-Sections (Sheet X-2, X-3 and X-8). The water collected by the permeable blanket should be directed to the end of excavation using a sump pump (Refer to Section 8.9.5 "Contaminated Groundwater" of this report).

In order to protect the roadway from buoyant uplift forces from a 100-year flood level, we recommend that the sheet piles along the southbound, used as shoring remain in place to act as a cutoff wall. Refer to the attached SSP for Shoring. Our seepage analysis indicated that the factor of safety against uplift pressure is less than 1.0 during 100-year flood, if a cutoff wall is not used.

C. Northbound and Southbound Existing I-680 Undulation – CCSB Line Station 95+40 to Station 98+60

Within the above limits the proposed roadway surface should be raised to its original elevation. The maximum settlement within these limits prior to placement of asphalt concrete for temporary fix was a little more than 2 m. To eliminate or substantially reduce the post construction settlement, we recommend the following steps (Refer to the attached Typical X-Section Sheet X-2 through X-3 and X-8):

Step 1. **Install Shoring Sheet piles--** Install shoring sheet piles as per stage construction plans along the existing northbound, center and southbound I-680 as shown on the attached Typical X-Sections (X-2 and X-3).

Step 2. **Excavation/Temporary Sheet piles**-- Excavate to elevation shown on the attached profile Sheet C-1 and C-3. This excavation also covers the entire width of the proposed Marina Vista Southbound on-ramp (including the gore area). For the proposed excavation and required stage construction, temporary shoring will be needed along the existing Retaining Wall #1 and concrete barrier, which separate future northbound freeway from southbound. All temporary excavation should be 1:1 or flatter. Existing Retaining Wall #1 is constructed on batter pile foundation. Therefore, care should be taken not to damage these batter piles during temporary sheet piles installation. Attached Typical X-Section X-2 and X-3 show the appropriate locations for the temporary sheet piles.

Step 3. **Permeable Blanket**—At the bottom of excavation, place a 200-mm thick (8 inches) Class 3 Permeable Material (PM-3) blanket over filter fabric and cover the PM-3 with geomembrane (Type B). De-water the excavation using sump pumps or any other methods chosen by the contractor.

Step 4. **Cellular Concrete**-- Backfill the sub-excavated area over the PM-3 blanket (geomembrane) with lightweight cellular concrete and construct the embankment up to the pavement sub-grade using lightweight cellular concrete. Refer to the attached Typical X-Section X-2 and X-3 and Profile Sheet C-1). Cellular concrete placed directly below the pavement sub-grade should have a minimum unit weight of about 6.3 kN/m^3 (40 pcf) but not to exceed 6.6 kN/m^3 (42 pcf) and should have a minimum compressive strength of 550 kPa (80 psi). The remaining cellular concrete below this level should have a maximum unit weight of 4.7 kN/m^3 (30 pcf) but not less than 3.9 kN/m^3 (25 pcf) and should have a minimum compressive strength of 276 kPa (40 psi). The cellular concrete used to construct the embankment (if there are any in this project) should be stepped with 1V:2H set back. The steps should then be backfilled with imported borrow or existing excavated fill material (fill slope of 1V: 2H) at 90% relative compaction. Please refer to the attached specifications for the cellular concrete.

Step 5. **Structural Section**-- Construct the roadway structural section over the cellular concrete.

D. Marina Vista/Waterfront Road MW Line Station 2+99.94 to Station 4+80

Within the above limits, the surface elevation of the Marina Vista/Waterfront Road needs to be raised in order to conform to the existing on- and off-ramp elevations at the intersection with Waterfront Road. Based on the submitted cross section, the maximum surface elevation increase is about 1.0 m at around station MW 3+85. In order to minimize future settlement, and at the same time keep the required excavation above the underground utilities, we recommend the following:

- Sub-excavate to a minimum of 0.60 m below the existing ground/roadway surface; refer to the attached Typical X-Section Sheet X-8 and Profile Sheet C-3.
- Place sub-grade enhancement fabric at the bottom of excavation (no permeable blanket will be required),
- Backfill the sub-excavated area and construct the fill all the way to the bottom of the roadway structural section using cellular concrete. The cellular concrete should have a minimum unit weight of about 6.3 kN/m^3 (40 pcf) but not to exceed 6.6 kN/m^3 (42 pcf) and should have a minimum compressive strength of 550 kPa (80 psi), and
- Construct the proposed structural section.

8.4 Liquefaction

Due to the clayey nature of the foundation soil no ground mitigation for liquefaction hazard is recommended.

8.5 Culvert Foundation

The preliminary drainage plans show that there are several new culverts planned for this project at various locations. The existing 3.03 mX1.2 m (10' X 4') RCP located near Station EL1 10+55.50 will stay in place and will neither be extended nor shortened due to the new roadway alignment. The following foundation treatment is recommended for all new culverts:

For those culverts that may be placed in the Marshland (if any), regardless of size, excavate all materials to a minimum elevation of 0.30 m below and 0.60 m on each side of the culvert. Place sub-grade enhancement fabric across the bottom of the excavation and replace the excavated materials with structure backfill.

For all other culverts, detailed foundation preparation as specified in the Caltrans Standard Plans will be sufficient. For any culverts placed within the cellular concrete, no foundation treatment will be necessary; place the culvert over the already cured cellular concrete and backfill around and over the culvert with cellular concrete. No plastic pipes should be placed in the cellular concrete because the temperature of the heat of hydration of cellular concrete exceeds the melting point of the plastic pipe.

8.6 Corrosion Investigation

8.6.1 Culvert Corrosion Study

Corrosion tests were conducted by the District Materials Branch on soil and water samples collected within the limits of the project, EA 006054. For culvert corrosion studies and recommendations regarding appropriate culverts for this project, consult with District Materials.

8.6.2 Sheet piles Corrosion and Recommended Design Sections

Based on the results of the corrosion tests conducted by the District Materials Branch and Office of Structure Foundations and our previous corrosion study, it was concluded that the site environment is moderately to highly corrosive. In order to prevent damage to the cellular concrete during sheet piles retrieval, we recommend that those portions of the temporary sheet piles below the ground surface remain in place permanently. Due to the highly corrosive environment, all sheet piles should be of sufficient thickness to resist the corrosive environment. The pH value of 7 water samples and 12 soil samples taken range from 4.0 to 7.9 and the resistivity values range from 50 to 1148 ohm-cm. The chloride concentrations in the marshland area range from 747 to as high as 17,600 PPM. The corrosion test results were sent to Headquarters Corrosion Department for the previous project (04-EA 006051) to determine the required thickness of sheet piles for a 50-year service life. Mr. Rick Carter of METS Corrosion Department concluded that since the entire sheet pile would be permanently below the water table (soil permanently saturated) that would lower the corrosion rate because of reduced availability of oxygen. He recommended that the minimum sheet pile thickness be 9.5 mm (3/8").

8.6.3 Contaminated Groundwater

Under the previous contract 04-006054, the test results of water samples (sampling and testing conducted by Geocon Environmental for the District Environmental Unit) showed the groundwater contains a small concentration of hydrocarbons that may adversely affect cellular concrete, which is proposed to be placed over a Class 3 permeable material blanket (PM-3). Therefore, we recommend that an impervious geomembrane be placed over the top of the proposed PM-3 instead of filter fabric. Please note that filter fabric is still required below the PM-3. Refer to the attached SSPs for Geomembrane Type B and PM-3.

9. MATERIAL SOURCES

Contact District Materials Branch for material sources.

10. MATERIAL DISPOSAL

Excess material shall be disposed of outside State R/W as per Section 19 of the Standard Specifications. All excavated material may be contaminated. Handling and disposal shall be in accordance with applicable regulations as recommended by the District 4 Environmental Branch.

11. CONSTRUCTION CONSIDERATIONS

11.1 Construction Advisory

Contaminated groundwater shall not be allowed to mix with the surface water because the combination is considered to be contaminated.

If soft soils are encountered during the construction of culverts, the existing soils should be replaced as recommended in the culvert foundation section of this report.

11.2 Construction Considerations that Influence Design

Considering the height of the proposed embankments and the presence of soft bay mud and peat deposits, excessive settlement is expected to occur if lightweight fills (cellular concrete) as recommended in this report are not used.

11.3 Differing Site Conditions

Section 5-1.116 of the Standard Specifications shall apply. If any variations, undesirable conditions, or other geotechnical issues, which are not apparent, are encountered during construction, Hooshmand Nikoui of Branch A, Geotechnical Design West, should be contacted so that supplemental recommendations can be made.

12. RECOMMENDED SPECIFICATIONS

Reference – Standard Specifications dated May 2006.

12.1 Earthwork

- Earthwork shall conform to the provisions of Section 19-1 of the Standard Specifications and to the appropriate Standard Special Provisions.
- Special Provisions for excavations in contaminated material, and for treatment/disposal of contaminated soil and water to be recommended by the District Environmental Branch should be used.

12.2 Cellular Concrete

Refer to the attached specifications for Cellular Concrete.

12.3 Geomembrane (Type B)

Refer to the attached specifications for Geomembrane (Type B).

12.4 Class 3 Permeable Material

Refer to the attached specifications for Class 3 Permeable Material (PM-3).

12.5 Sub-grade Enhancement Fabric

Refer to the attached specifications for Sub-Grade Enhancement Fabric.

12.6 Filter Fabric

Refer to Standard Specifications Section 68-1.028

10-1.__ SUBGRADE ENHANCEMENT GEOSYNTHETIC

Subgrade enhancement geosynthetic shall be fabric and shall be placed as shown on the plans and in locations as directed by the Engineer and in accordance with the special provisions.

Fabric

When shown on the plans or required by these specifications, subgrade enhancement geosynthetic shall be a fabric used for reinforcement, separation, and filtration applications. Subgrade enhancement fabric shall be manufactured from one or more of the following materials: polyester, nylon, or polypropylene.

Subgrade enhancement fabric shall conform to the following:

Specification	Requirement	
	Class A	Class B *
Wide Width Tensile Strength, min. in each direction, kN ASTM Designation: D 4595	35	70
Grab tensile strength (25-mm grip), min. in each direction, KN ASTM Designation: D 4632	1.4	2.0
CBR Puncture Strength, KN ASTM Designation: D 6241	2.5	5.5
Permittivity, sec^{-1} ASTM Designation: D4491	0.5	0.1
Apparent Opening Size, mm (max) ASTM Designation: D4751	0.60	0.22
Elongation at break, % (max.). ASTM Designation: D 4632	35	35

Subgrade enhancement geosynthetic shall be furnished in a cover capable of protecting it from ultraviolet radiation and abrasion due to shipping and handling, and shall remain in the cover until installation.

Subgrade enhancement geosynthetic shall be accompanied by a Certificate of Compliance conforming to the provisions in Section 6-1.07, "Certificate of Compliance," of the Standard Specifications. The subgrade to receive the geosynthetic shall conform to the compaction and elevation tolerance specified in Section 25-1.03, "Subgrade," of the Standard Specifications and these special provisions and shall be free of loose or extraneous material and sharp objects that may damage the fabric during installation.

Should the geosynthetic be damaged during placement, the damaged section shall be repaired by placing a new piece of geosynthetic over the damaged area. The repair piece of geosynthetic shall be large enough to cover the damaged area and to provide a minimum 600-mm overlap on all edges.

Subgrade enhancement geosynthetic shall be handled and placed in accordance with the manufacturer's recommendations and shall be positioned longitudinally along the alignment, and pulled taut to form a wrinkle-free mat.

Adjacent borders of geosynthetic shall be overlapped a minimum of 600 mm. All roll ends shall be overlapped a minimum of 600 mm in the direction of the spreading of the aggregate base material. On curves, the fabric may be folded or cut to conform to the curves. If cut, a minimum overlap of 600 mm shall be provided for adjacent fabric sides. The fold or overlap shall be held in place by staples, pins, or piles of fill of the materials to be placed on the fabric.

The amount of subgrade enhancement geosynthetic placed shall be limited to that which can be covered with aggregate base or other material, as shown on the plans, within 72 hours.

Damage to the subgrade enhancement geosynthetic resulting from the Contractor's vehicles, equipment, or operations shall be repaired at the Contractor's expense.

Vehicles and equipment shall not be driven directly on the subgrade enhancement fabric. Only rubber tired vehicles may be allowed directly on the subgrade enhancement geogrid at speeds less than 8 km/h if the underlying material is capable of supporting the loads. Turning or stopping of rubber tired equipment on the geogrid should be avoided if possible. A minimum cover of 150 mm of fill material shall be maintained between the geosynthetic and the equipment to prevent damage to the geosynthetic. Stockpiling of materials directly on the subgrade enhancement fabric will not be allowed. An initial layer of aggregate base shall be placed and compacted over the subgrade enhancement fabric using either smooth wheel (without vibratory action) or rubber tired rollers to form a working platform.

As ordered by the Engineer, subgrade enhancement geosynthetic shall be sampled during placement for testing to verify conformance with physical and mechanical properties requirements

The quantity of subgrade enhancement geosynthetic to be paid for will be measured by the square meter of area covered, not including additional fabric for overlap.The contract price paid per square meter for subgrade enhancement geosynthetic shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in placing the fabric, complete in place as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.

10-1.__ PERMEABLE MATERIAL

Permeable material shall conform with the details shown on the plans, to the provisions in Section 68-1, "Underdrains," of the Standard Specifications, and these special provisions.Class 3 permeable material shall conform to the following gradation:

Grading Requirements

Sieve Sizes	Percentage Passing
37.5-mm	100
25-mm	90-100
19-mm	40-100
9.5-mm	0-50
4.75-mm	0-15
2.36-mm	0-5

Class 3 permeable material shall have a Durability Index of not less than 40.

At least 90 percent by mass of Class 3 permeable material shall be crushed particles as determined by California Test 205.

Filter fabric for use with permeable material shall conform to the provisions for filter fabric for underdrains in Section 88, "Engineering Fabrics," of the Standard Specifications and the following:

A. The subgrade and trench to receive the filter fabric, immediately prior to placing, shall conform to the compaction and elevation tolerance specified for the material involved.
B. Filter fabric shall be handled and placed in conformance with the manufacturer's recommendations.
C. The fabric shall be aligned and placed in a wrinkle-free manner.
D. Within 72 hours after the filter fabric has been placed, the fabric shall be covered with the planned thickness of overlying material as shown on the plans.

10-1.39 GEOMEMBRANE (TYPE B)

Geomembrane (type B) shall be placed where shown on the plans in accordance with the Standard Specifications and these special provisions.

The geomembrane shall be flexible and, by its own weight, shall cover and conform closely to 90 degree edges and corners of subgrade materials at ambient temperatures of about 7.2 to 26.7 degrees Celsius, without additional heating of the geomembrane.

The geomembrane shall be a reinforced or unreinforced geomembrane. It shall be manufactured from tri-polymer consisting of polyvinyl chloride, ethylene interpolymer alloy, and polyurethane or a comparable polymer combination. It shall meet the following physical and chemical requirements, specified as minimum or maximum, not average roll properties.

Property	Test Method	Acceptance Value
Thickness, mm	ASTM D751	28 mm min.
Unleaded gasoline vapor Transmission rate, g/m^2 /day	ASTM D814	122 max.
Grab tensile strength, both machine and cross direction (15.4 mm grip; 101.6 mm x 203.2 mm sample)	ASTM D751	2.67 kN min
Shear, kg	ASTM D751 (Modified per National Sanitation Foundation Std. No. 54) Fail in base geomembrane material	145 min.
Cold crack -- pass degrees Celsius, (25.4 mm mandrel, 4 hours)	ASTM D2136	-34
Elongation @ break (%)	ASTM D751	20 min.
Toughness, grab tensile times % Elongation (e.g., 2.67 x 20% = 53.4)		53.4 min.
Puncture Resistance	ASTM D751 (ball tip)	3.6 kN min.
Cold crack, pass degree Celsius (using 25 mm mandrel, 4 hours)	ASTM D2136	-34
Factory seams		5 cm min.

A Certificate of Compliance shall be furnished in accordance with the provision of Section 6-1.07, "Certificates of Compliance," of the Standard Specifications. It shall also state that the selected geomembrane has been tested, meets the above requirements, and is:

A. Free from pinholes, tears, and other defects which would cause leakage of fluids through the geomembrane. At the Contractor's option, geomembrane shall be either of the following types or equal:

Manufacturer or Supplier	Product
Colorado Lining Co (303) 841-2022	Hytrel
Cooley, Inc (800) 223-9419	Hytrel
MCP Containment Systems (312) 927-4120	Petrogard VI
Seaman Corporation (615) 691-9476 8030	8030 GCL

The subgrade shall conform to the provisions in the section, "Earthwork" elsewhere in these special provisions. The subgrade shall be free of sharp objects prior to placement of the geomembrane.

Adjacent section of the geomembrane shall be overlapped a minimum of 0.45 m.

No construction equipment shall be driven directly on the geomembrane. Damage to the geomembrance resulting from the Contractor's vehicles, equipment, or operations shall be replaced or repaired by the Contractor.

The quantities of geomembrane (type B) will be measured by the square meter.

The contract price paid per square meter for geomembrane (Type B) shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in furnishing and placing geomembrane, complete in place, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.

10-1. SHORING

This work shall consist of designing, and maintaining shoring, and later removing portion of shoring as shown on the plans, as specified in the Standard Specifications and these special provisions.

The material used to construct shoring shall be steel sheet piles with a minimum thickness of 9.5mm.

The Contractor shall be responsible for designing, constructing, and maintaining safe and adequate shoring which will support all loads imposed, including traffic loads.

The Contractor shall submit to the Engineer 5 sets of working drawings and one set of design calculations for shoring. The drawing and calculations shall be submitted to the Engineer at least 2 weeks in advance of the time the Contractor begins construction of the shoring. The working drawings shall be signed by an Engineer who is registered as a Civil Engineer in the State of California.

Attention is directed to Section 5-1.02, "Plans and Working Drawing," of the Standard Specifications.

When no longer required as determined by the Engineer, the top portion of the shoring shall be removed to a depth of not less than 0.9 meters below finished grade. The remainder of the shoring shall be abandoned in place. Removed shoring shall be disposed of outside the highway right of way as provided in Section 7-1.13, "Disposal of Material Outside the Highway Right of Way," of the Standard Specifications.

Shoring will be measured by the meter along the exposed face of the completed shoring.

The contract price paid per meter of the shoring shall include full compensation for furnishing all labor, materials, tools, equipment, and incidentals, and for doing all the work involved in shoring, complete in place, including maintaining shoring and removing and disposing of shoring, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.

10-1.__ LIGHTWEIGHT EMBANKMENT MATERIAL (CELLULAR CONCRETE)

The work shall consist of constructing a lightweight embankment material (cellular concrete) to the lines, grades, and dimensions shown on the plans, in accordance with the Standard Specifications, these special provisions and as directed by the Engineer.

The Contractor shall furnish two mix designes, one of which will produce a cast density (at point of placement) of 6.3 to 6.6 kN/m^3 with a minimum compressive strength of 550 kPa at 28 days, and one mix design which will produce a cast density (at point of placement) of 3.9 to 4.7 kN/m^3 with a minimum compressive strength of 276 kPa at 28 days. The Contractor shall provide the Engineer with a Work Plan of the equipment and procedures proposed at least 10 working days prior to placement; items in the submittal shall include:

1. Material list of items; manufacturer's specifications;
2. Mix designs, including laboratory data using the mix design verifying mass and strength requirements.

Admixtures for accelerating the set time may be used in accordance with the manufacturer's recommendations. A foaming agent shall be used and shall be tested in accordance with ASTM C 796. Mixing water shall be potable and free of deleterious amounts of acids, alkali, salts, oils, and organic materials which would adversely affect the setting or strength of the Lightweight Embankment Material (cellular concrete).

Portland Cement shall comply with ASTM C150, Types I, II, or III. Pozzolans and other cementitious materials may be used when specifically approved by the manufacturer of the foaming agent.

At the point of placement, the density shall be in accordance with the specified cast density. A single cast density test shall represent the lesser of 230 cubic meters or one day's production.

The compressive strength shall be tested in accordance with ASTM C 495 except as follows:

1. Unless otherwise approved by the Engineer, the specimens shall be 76.2 mm (3-inch) by 152.4 mm (6-inch) cylinders. During molding, place the concrete in two approximately equal layers, and raise an drop the cylinders approximately 25 mm three times on a hard surface after placing each layer; no rodding shall be allowed. Specimens shall be covered and protected immediately after casting to prevent damage and loss of moisture.

2. Specimens shall be moist cured in the molds for a period of seven (7) days prior to the 28-day compressive strength test. Specimens shall not be oven dried.

Lightweight embankment material (cellular concrete) shall be placed to the designated dimensions as specified in Sections 19-1.03, "Grade Tolerance,".

Lift thickness for lightweight embankment material (cellular concrete) shall not exceed 0.6 m. If more than one lift is required, the layer to receive the next lift shall be scarified with a broom or rake to provide surface roughness. After curing for 12 hours, any crumbling area on the surface should be removed and scarified before the next layer is placed. Surface stepping shall be limited to 125 mm. Grades of up to 5 percent may be made by adding a thickening agent to the mix, in conformance with the manufacturer's recommendations.

If more than one lift is required, the layer to receive the next lift shall be scarified with a broom or rake to provide surface roughness. After curing for 12 hours, any crumbling area on the surface should be removed and scarified before the next layer is placed. Surface stepping shall be limited to 125 mm. Grades of up to 5 percent may be made by adding a thickening agent to the mix, in conformance with the manufacturer's recommendations.

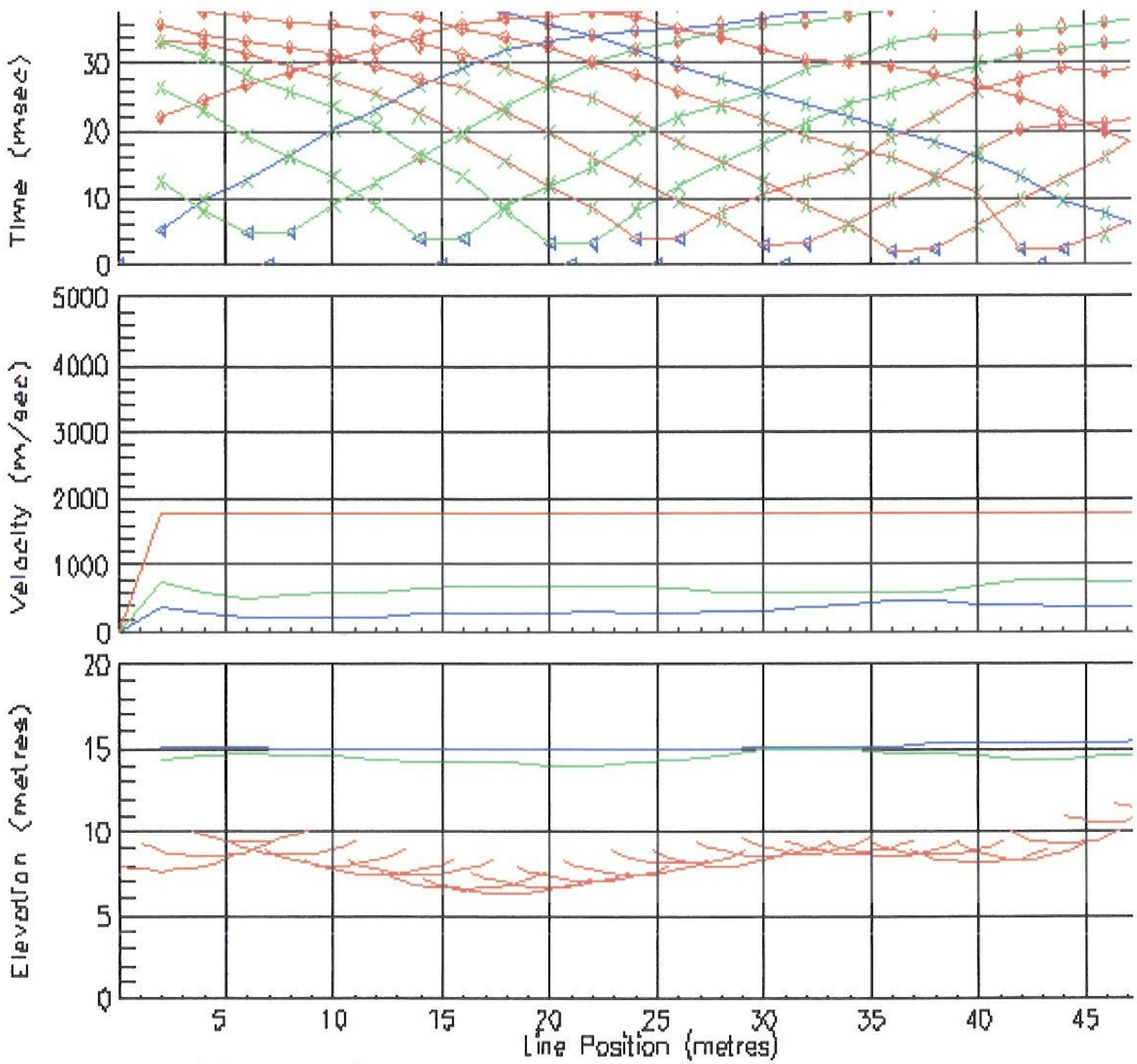
A minimum 12-hour waiting time between lifts shall be required. If ambient temperatures are anticipated to be below 4.5 C within 24 hours after placement, the mixing water should be heated when specifically approved by the manufacturer of the foaming agent, or placement shall be prohibited during such period. Placement shall not be allowed on frozen ground.

Lightweight embankment material (cellular concrete) shall be job site batched, mixed with the foaming agent, and placed with specialized equipment certified by the manufacturer. Cement and water may be premixed and delivered to the site; and foam shall be added at the site. Slurry coats and multilayer casting are acceptable methods of installation. Subgrade to receive lightweight embankment material (cellular concrete) shall be free of all loose and extraneous material. Subgrade shall be uniformly moist, and any excess water standing on the surface shall be removed prior to placing lightweight embankment material (cellular concrete).

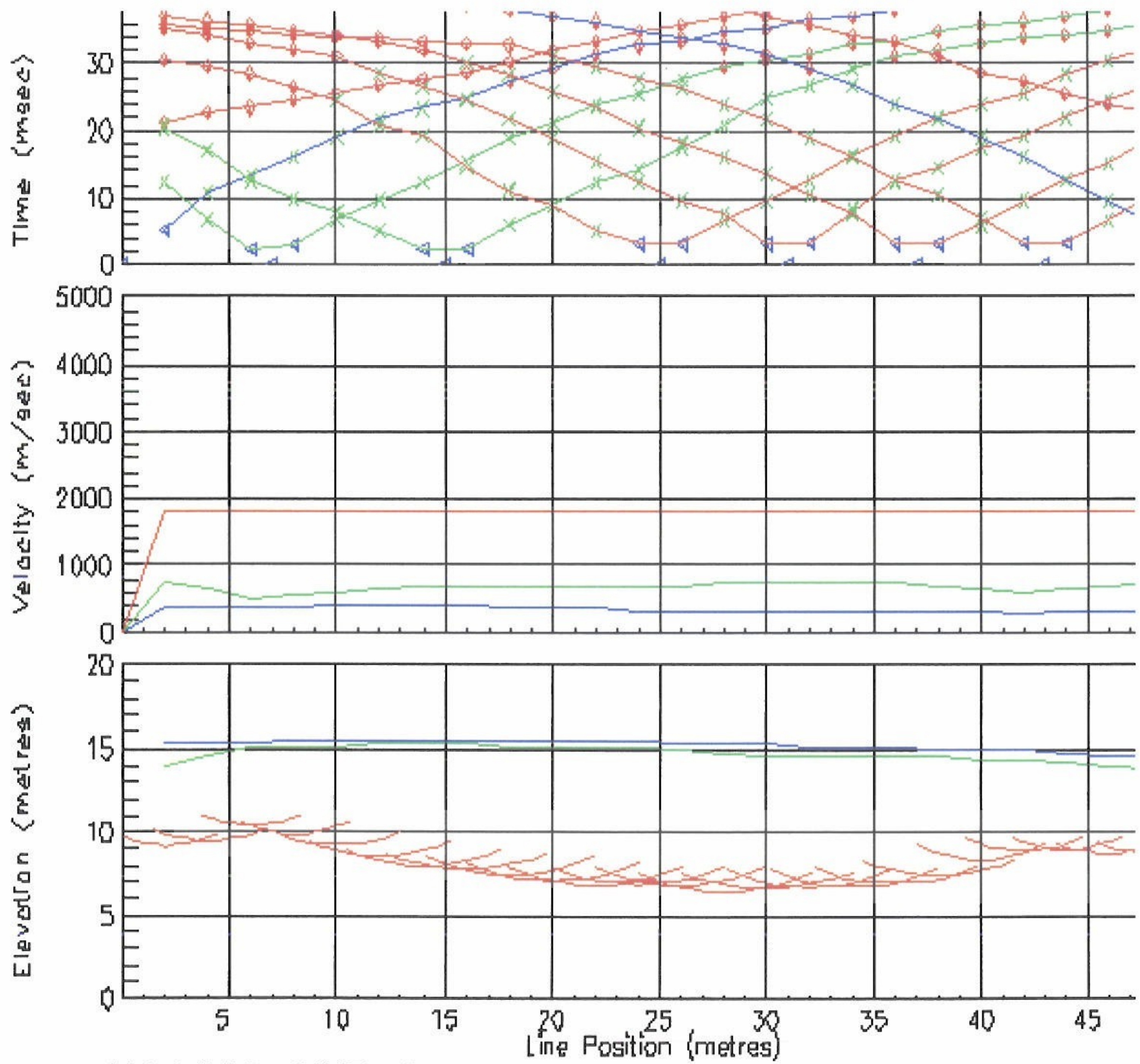
After placing the final lift of lightweight embankment material (cellular concrete), the exposed surface of shall be covered with a prime coat. The prime coat shall conform to the requirements in Section 94, "Asphaltic Emulsions," of the Standard Specifications. A prime coat of SS-1 shall be applied uniformly at a rate of between 0.68 and 1.13 liters per square-meter, with the exact rate determined by the Engineer.

Pay quantities for lightweight embankment material (cellular concrete) will be measured by the cubic meter, to the lines and grade shown on the plans and as directed by the Engineer.

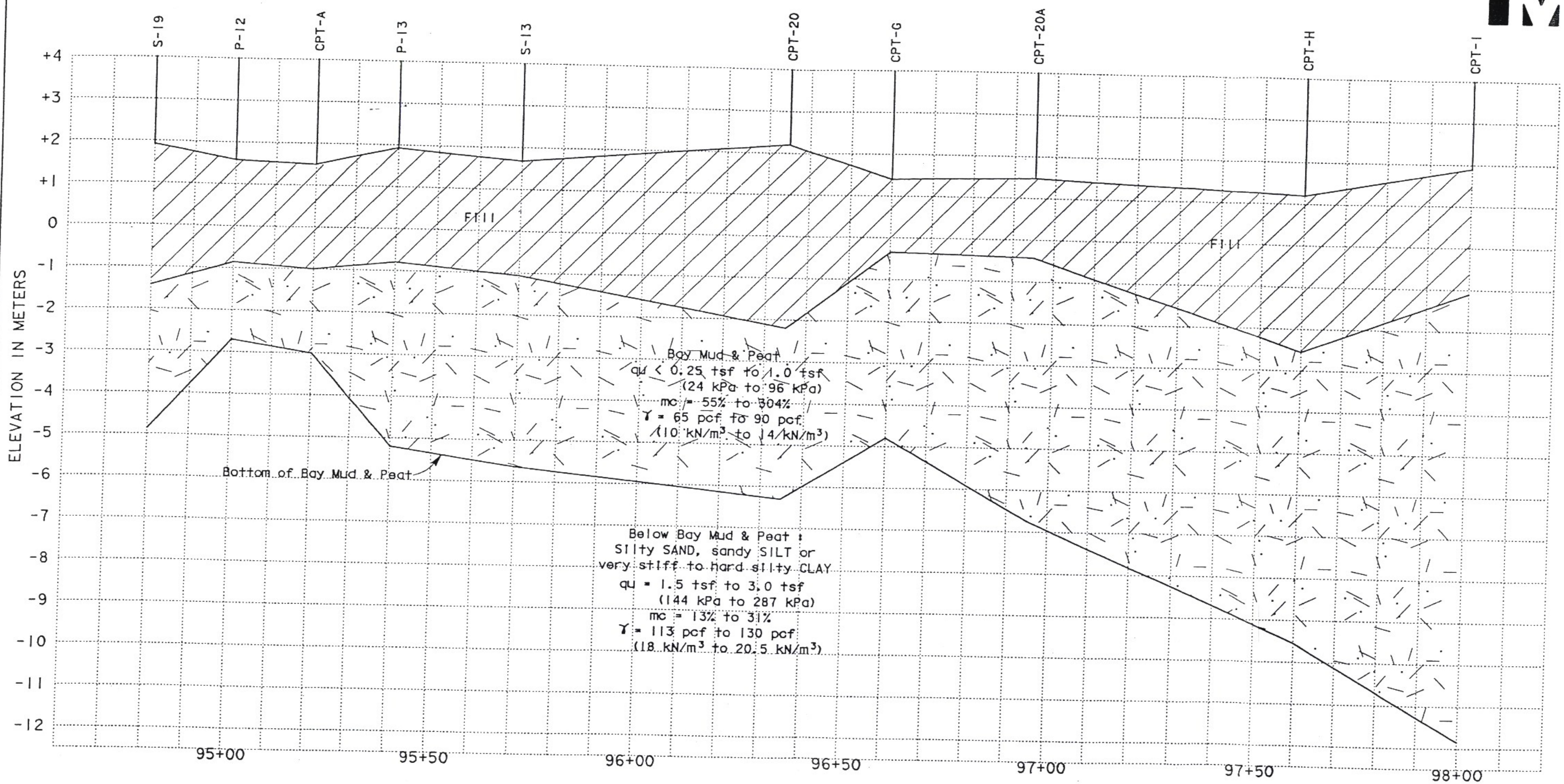
The contract price paid per cubic meter for lightweight embankment material (cellular concrete) shall include full compensation for furnishing all labor, materials (including furnishing and applying the prime coat), tools, equipment, and incidentals, and for doing all the work involved in furnishing and placing, the lightweight embankment material (cellular concrete), complete in place, as shown on the plans, as specified in the Standard Specifications and these special provisions, and as directed by the Engineer.



MOCOCO LINE 1



MOCOCO LINE 2



Soil Profile Along The East Edge
of New NB I-680 Freeway
"CCNB" Line Sta. 94+80± to 98+00±

NO SCALE

Caltrans

Engineering Service Center
Division of Structural Foundations
Office of Roadway Geotechnical
Engineering (North)-Branch 3

FIGURE 5	
04-CC-680	KP 38.5/40.1
04225-006051	OCTOBER 2000
SOIL PROFILE	

04	CC	680	38.0/39.0

4-29-98

REGISTERED CIVIL ENGINEER

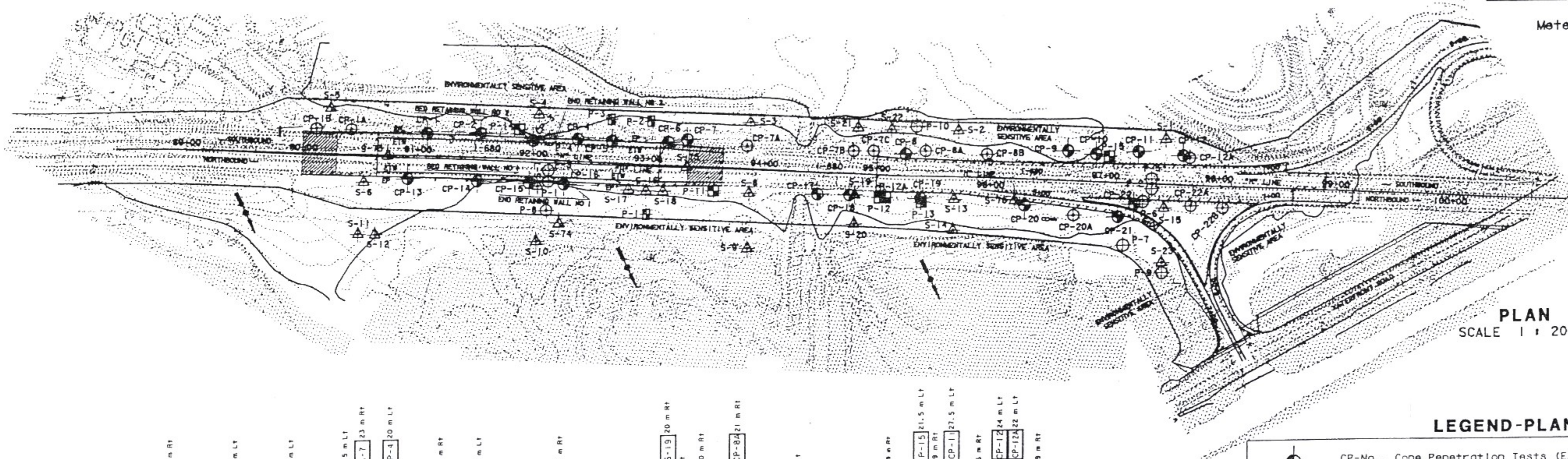
H. Nikoui-G
No. 42698
Exp. 3-31-00
CIVIL
REGISTERED PROFESSIONAL ENGINEER
STATE OF CALIFORNIA

PLANS APPROVAL DATE

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

California Metric

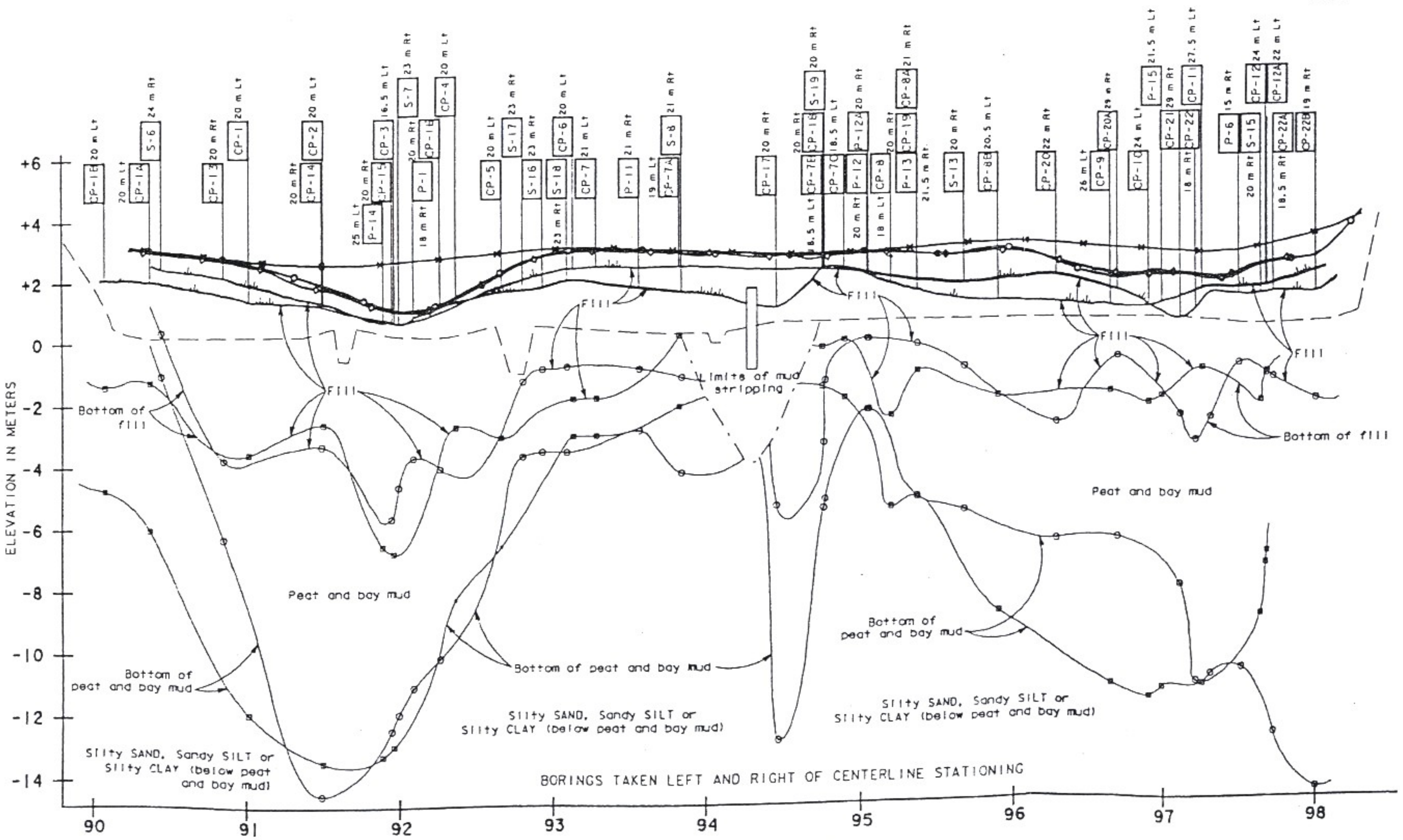
All Dimensions are in Meters unless otherwise shown



PLAN
SCALE 1 : 2000

LEGEND-PLAN

- CP-No., Cone Penetration Tests (EA# 126011-1988)
- P-No., Power Borings (EA# 120611-1988) (P-2 to P-5 are 6" augers for AC thickness determination)
- CP-NO., Cone Penetration Test (EA# 910054-1989)
- S-No., Soil Tubes (EA# 030701-1979)
- P-No., Power Borings (EA# 030701-1979)
- S-No., Soil Tubes (EA# 61-4MBCI-1956)
- P-No., 2" Soil Sampler (EA# 910054-1989)
- Individual boring records are available upon request



PROFILE
SCALE HORIZ. 1 : 4000
VERT. 1 : 200

LEGEND-PROFILE

- Original Ground
- NB Outside EP-1960
- NB Outside EP-1980
- NB Outside EP-1988
- Boring Plots 20 m± Rt of ℄
- Boring Plots 20 m± Lt of ℄

UNDULATION AREAS ON ROUTE I-680
LOCATION OF BORINGS
AND SOIL PROFILE

EXHIBIT A

SCALE AS SHOWN

DATE PLOTTED -> 31-Oct-2000

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

PROJECT ENGINEER

CALCULATED/DESIGNED BY | CHECKED BY | REVISED BY | DATE REVISED