GEOTECHNICAL DESIGN & MATERIALS REPORT
MARE ISLAND/ROUTE 37 PROJECT
04-SOL-37/KP 11.4-11.9
SOLANO COUNTY, CA
EA 284700, CU 04277

For

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"Approved as to impact on State facilities and conformance with applicable State standards and practices, and the technical oversight were performed as described in the California Department of Transportation A&E Consultant Services Manual."

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1. INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Mare Island/Route 37 Project, in Solano County, California. The attached Title Sheet, shows the location of the project vicinity. The proposed project includes an addition to the existing bridge structure and realignment/widening of Railroad Avenue and existing ramps.

This report addresses design of pavement sections, corrosion investigation, and bridge embankment recommendation for the project. The foundation design recommendations for the bridge is submitted in a separate report. The investigation included review of readily available soils and geologic literature pertaining to the site including as-built information, obtaining representative samples and logging soil materials encountered in exploratory borings, laboratory testing of the representative samples, performing engineering analyses, and preparation of this report.

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, if any.

The report is intended for use by the project roadway design engineer, construction personnel, bidders and contractors for information and reference purposes only and should not be construed directly as project specifications.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain a properly constructed project. We, therefore, recommend that a contingency fund be provided to accommodate any additional charges resulting from technical services that may be required during construction.

2. EXISTING FACILITIES AND PROPOSED IMPROVEMENTS

Existing structure at site is Walnut Avenue Overcrossing (Bridge No. 23-109).

The State Route 37-Mare Island interchange improvement project requires the reconstruction of the southern end of the interchange to accommodate changes that are being made to the Mare Island local roadway system. The scope of work includes reconstruction of the EB Off-ramp and EB On-ramp to align with Railroad Avenue. An HOV bypass lane is being added to the EB On-ramp. The project also widens Railroad Avenue to provide 6-lanes and a raised median to a future intersection approximately 140 meters south of the interchange. Lastly, the project requires that the southern end of the Walnut Avenue overcrossing structure be widened and a retaining wall be constructed to accommodate the new ramp alignments. The retaining wall will be about 3.6 m high at the maximum point and will be supported on piles. The height of fill behind the wall is about 3.5 m.

3. PERTINENT REPORTS AND INVESTIGATION

- Preliminary Geotechnical Engineering Study and Consolidation Evaluation for Proposed North Mare Island Business Park, Vallejo, CA, dated July 20, 2001.

- As-built Log of Test Borings plan for the Walnut Avenue Overcrossing. (Bridge No. 23-109, Caltrans)

- As-built roadway plans and pavement sections for Railroad Avenue and ramps.

4. PHYSICAL SETTING

4.1 Climate

The project area is characterized with moderate climatic conditions. This consists of mild winters, warm summers, small daily and seasonal temperature ranges and mild humidity. Extremes of temperature would range from 3°C to 14°C in December/January and 12°C to 28°C in July.

Based on statistics from National Weather Service, average rainfall precipitation is about 110 mm in the Vallejo area and is principally during the months of November through March. January usually has the most precipitation accumulation with 140 mm as an average.

4.2 Topography and drainage

The topography along Route 37 is relatively level. The surface drainage will be collected in the local storm drains. The surface drainage generally is not influenced by the hills or valleys.

4.3 Man-Made and Natural Features of Engineering and Construction Significance

The subject was considered and was determined to be not applicable to the project.

4.4 Regional Geology and Seismicity

Based on California Geomorphic Provinces, the geologic study area is located in Coastal Range province. The Coast Ranges are mountain ranges (600-1200, occasionally 1800 m elevation above sea level) and valleys. The ranges and valleys trend northwest, subparallel to the San Andreas Fault. The province terminates on the east where strata dip beneath alluvium of the Great Valley; on the west by the Pacific ocean with mountains rising sharply from uplifted and terraced, wave-cut coast; on the north by South Fork Mountain, which has the characteristic trend of the Coast Ranges, and on the south by the Transverse Ranges. The Coast Ranges are composed of thick late Mesozoic and Cenozoic sedimentary strata. The northern and southern ranges are separated by a depression containing the San Francisco Bay. Offshore, the continental shelf is transected by submarine canyons. The Monterey submarine canyon, 3000 m deep, is apparently a submerged river canyon. The northern coast ranges are dominated by irregular, knobby, landslide topography of the Franciscan Formation. The eastern border is characterized by strike-ridges and valleys in Upper Mesozoic strata. In several areas, Franciscan rocks are overlain by volcanic cones and flows of the Quien Sabe, Sonoma, and Clear Lake volcanic fields. The Coast Ranges are subparallel to the rift valley of the active San Andreas Fault. The San Andreas is more than 965 km long, extending from Pt. Arena to the Gulf of California. The Salinian block to the west of the San Andreas has a granitic core, extending from the southern extremity of the Coast Ranges to north of the Farallon Islands.

5. EXPLORATION

5.1 Drilling and Sampling

Based on the preliminary plans, discussions with the design team, and readily available geotechnical data in the area, 9 shallow borings were drilled at selected locations to maximum depths of 1.5 m below the existing ground surface. Due to existence of soft clay (Bay Mud), two additional borings were drilled to a depth of 23 m for evaluating potential settlement in the area. The Site Plan, Plate 2, show the approximate locations of these borings. All borings were advanced with truck-mounted drill rig using 203 mm rotary wash and hollow stem auger.

The borings were drilled under the technical supervision of one of our engineers, who classified and continuously logged the soils encountered during drilling and supervised the collection of soil samples at various depths for visual examination and laboratory testing. The soil samples were obtained during drilling and by driving a 64 mm I.D. Modified California sampler into the subsurface soils under the impact of a 63.5 kg hammer falling through 76 cm. The blow counts required to drive the sampler for the last 30 cm are presented on the "Log of Test Borings", Appendix A. After visual examination, the collected samples were sealed and transported to our laboratory for further evaluation and testing. The boring locations, stations, and relevant information are summarized in Table 1.

The descriptions of the soils encountered and relevant boring information are presented on the Logs of Test Borings attached in Appendix A. The laboratory test methods and results are presented in Appendix B. The logs presented in Appendix A were prepared from the field logs which were edited after visual re-examination of the soil samples in the laboratory and results of classification tests on selected soil samples as indicated on the logs. The abrupt stratum changes shown on these logs may be gradational and relatively minor changes in soil types within a stratum may not be noted on the logs due to field limitations.

TABLE 1 - SUMMARY OF BORINGS

Boring No.	Station (m)	Offset (m)	Ref. Line	Boring Depth (m)	Approx. Ground Surface Elev. (m)	Approx. Groundwater Elev. (m)
R-1	116+95	41, Rt.	Route 37	1.5*	1.8	-
R-2	117+65	69 Rt.	"	"	1.5	0.3
R-3	118+19	103 Rt.	"	"	1.1	-
R-4	118+46	119 Rt.	"	"	2.1	-
R-5	118+89	183 Rt.	"	"	2.8	-
R-6	119+20	217 Rt.	"	"	3.0	-
R-7	118+84	60 Rt.	"	"	2.9	-
R-8	119+74	21 Rt.	"	"	3.5	-

TABLE 1 - SUMMARY OF BORINGS (Cont.)

Boring No.	Station (m)	Offset (m)	Ref. Line	Boring Depth (m)	Approx. Ground Surface Elev. (m)	Approx. Groundwater Elev. (m)
R-9	120+18	26 Rt.	"	"	4.0	-
BM-1	117+94	72.5 Rt.	"	23.0	1.0	**
BM-2	118+79	26 Rt.	"	23.0	2.8	**

Note: *The 1.5 m borings are mainly R-value borings. ** Groundwater was not measured due to rotary wash method of drilling.

5.2 Geologic Mapping

The site consists of surficial deposits (Bay Mud). The subject was considered and was determined to be not significant for the project.

5.3 Geophysical Studies

The subject was considered and was determined to be not applicable to the project.

5.4 Instrumentation

The subject was considered and was determined to be not applicable to the project.

5.5 Exploration Notes

The exploratory borings encountered surficial deposits predominantly consisting of soft clay (Bay Mud). Groundwater was encountered in the boring R-2 but it was not measured in deep borings due to rotary wash method of drilling. Drilling conditions using rotary wash and hollow stem augers were considered normal.

6. Geotechnical Testing

6.1 In-Situ Testing

In-situ testing consists of recording blow counts during sampling in the field. The soil samples were obtained during drilling by driving a 64 mm I.D. Modified California sampler into the subsurface soils under the impact of a 63.5 kg hammer falling through 76 cm. Based on our previous experience, when correlating Standard Penetration Test data in similar soils, the blow counts for the Modified California Sampler can be taken as roughly 2 times that for the Standard Penetration Test in similar soils.

6.2 Laboratory Testing

Laboratory tests performed for the study include the following: Laboratory determination of Moisture-Density (California Test Method 226), Consolidation Test (California Test Method 219), Unconfined Compression Test (California Test Method 221), R-value Test (California Test Method 301), and Corrosion Test (California Test Method 643). The laboratory test results are attached in Appendix B. Moisture-Density test and Unconfined Compression Strength test results are summarized on the LOTBs attached in Appendix A. The stress stain curve for unconfined compression test is attached in Appendix B.

In general, the natural moisture contents of the clayey soils are in the range of 50% and higher. Laboratory unconfined compression test results are presented on the Log of Test Borings at the appropriate sample depths.

7. Geotechnical Conditions

7.1 Site Geology

The geology of the site is referenced from the "Preliminary Geologic Map of Mare Island

Quadrangle, Solano and Contra Costa Counties, California". The project site is generally underlain by very stiff to hard clay near surface and soft clay (Bay Mud) to the maximum depth drilled. The geologic map is shown on Plate 3.

7.1.1 Lithology

The subject was considered and was determined to be not applicable for the project.

7.1.2 Structure

The subject was considered and was determined to be not applicable for the project.

7.1.3 Existing Slope Stability

The existing Mare Island/Route 37 embankment is constructed on fill of approximately 2 m high. The side slopes range from 1V: 2H are vegetated and generally appear to be in good conditions.

7.2 Subsurface Soil Conditions

The deep borings for the bridge embankment mostly encountered very stiff to hard clay followed by soft clay with occasional peat layers (Bay Mud). The thickness of very stiff clay was on the order of 0.6 m to 1.5 m. The thickness of soft clay (Bay Mud) was in the range of 10 m to 13.5 m. The shallow borings generally encountered hard clay to 1.5 m, the maximum depth drilled.

Detailed descriptions of the materials encountered in the exploratory borings are presented in Appendix A, "Log of Test Borings". It should be noted that these descriptions and related information depict subsurface conditions only at the locations indicated on logs and on the particular date noted on the logs. Because of the variability from place to place within soil strata in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. The abrupt stratum changes shown on the logs may be gradational and relatively minor changes in soil types within a stratum may not be noted due to field limitations. Also, the passage of time may result in a change in the soil conditions at the locations due to environmental changes.

7.3 Water

7.3.1 Surface Water

The terrain along Route 37 in the vicinity of the project is generally level. Walnut Avenue overcrossing is on embankment fill. The gradient difference along Route 37 and the overcrossing is about 2 m. The surface water/drainage generally follows the ground topography and drains away from the subject site and is collected in local storm drain.

7.3.1.1 Scour

The subject was considered and was determined to be not applicable for the roadway project.

7.3.1.2 Erosion

It is our understanding that a small portion of the roadway embankment will be placed at about 2.4 m high slope and at 1V:1.5H gradient. This section is along the pedestrian path that climbs up the roadway embankment to connect to the existing Napa River Bridge. The slope surface should be protected against possible washouts.

7.3.2 Groundwater

Groundwater was encountered at Elev. 0.3 m in the shallow borings (R-2) but it was not measured in the deep borings due to rotary wash method of drilling. As per preliminary investigation performed by others in the proximate area (July 2001) and based on the as-built LOTBs by Caltrans (1956), groundwater was encountered at between 0.5 m and 1.5 m below ground surface. It is anticipated that groundwater level will vary with the passage of time due to seasonal groundwater fluctuations, surface and subsurface flow, ground surface run-off, water fluctuation in the bay, and other factors that were not existent at the time of investigation.

7.4 Project Site Seismicity

7.4.1 Ground Motions

Faults in the vicinity of the site with a moderate to high potential for surface rupture include the Franklin Fault, Green Valley Fault, and Rodgers Creek – Healdsburg Fault. Earthquake data including the magnitude of Maximum Credible Earthquakes (MCE) and distance to fault are summarized in Table 2.

TABLE 2
EARTHQUAKE DATA

Fault	Estimated Distance From Project Site (km)	Maximum Credible Earthquake
FRA – Franklin fault	6.8	6.50
GVY – Green Valley Fault	13.9	6.75
RCH – Rodgers Creek – Healdsburg Fault	14.6	7.00

7.4.2 Ground Rupture

Since no active faults pass through the site, the potential for fault rupture is low.

8. Geotechnical Analysis and Design

8.1 Dynamic Analysis

8.1.1 Parameter Selection

For selection of the seismic design parameters, we have adopted a Peak Bedrock Acceleration (PBA) of 0.40 g and an associated Peak Ground Acceleration (PGA) of 0.25 g. The PBA and PGA were based on the attenuation relationships proposed by Mualchin and Jones (1992) and Seed and Idriss (1982), respectively.

8.1.2 Analysis

Based on the layout plans provided, fill embankment is proposed for the bridge addition. About 2 m fill material will be placed over native ground at the bridge embankment. The gradient for the slope should be 1:2 (V:H) or flatter. It is our understanding that a small portion of the roadway embankment will be placed at about 2.4 m high slope and at 1V:1.5H gradient. This section is along the pedestrian path that climbs up the roadway embankment to connect to the existing Napa River Bridge.

8.1.3 Liquefaction Potential

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing, cyclic shear stresses associated with earthquake shaking. Submerged cohesionless sands and silts of low relative density are the type of soils, which usually are susceptible to liquefaction.

Clays are not generally susceptible to liquefaction.

Based on the field boring logs and available data, the soils encountered during our investigation were mainly soft clay (Bay Mud) overlaying stiff clay at depth. In our opinion, the liquefaction potential at the site is anticipated to be low.

8.2 Cuts and Excavations

Based on the plans and profiles provided no major cuts and excavation is planned for the project.

8.2.1 Stability

Based on the plans and profiles provided no major cuts and excavation is planned for the project.

8.2.2 Rippability

The majority of the project will include fill material. Based on the investigation, rippability does not appear to be a concern for construction.

8.2.3 Grading Factor

The on-site native soil meeting the project specifications may be used as engineered fill. For preliminary estimate, a grading factor of 0.9 may be assumed based on previous experience.

8.3 Embankments

8.3.1 Evaluation of Embankment Settlements

There is about 2 m and 3.6 m high new embankment fill planned for the roadway and bridge embankment, respectively. Consolidation settlement evaluation under the new fill was based on the consolidation test results on the soft bay mud samples. For the analysis purposes, groundwater was assumed at Elev. –1 m. The preconsolidation pressure and modified compression index of the bay mud were derived from the laboratory consolidation data. For the other relatively stiff cohesive soil, the preconsolidation pressure was estimated using the Su/p (undrained shear strength over effective overburden stress) relationship proposed by Skempton. The modified compression index ($Cc/1+e_o$) correlated to natural moisture content of the soil as established by Lambe and Whitman (1969) was adopted. For the bay mud, the P_p and $Cc/1+e_o$ obtained from the laboratory consolidation tests appear to correlate well with that estimated from the empirical approach (Su/p ratio and $Cc/1+ e_o$).

Assumption of one-dimensional consolidation is adopted for the new fill at roadway embankment. The Skempton-Bjerrum factor for adjusting the calculated settlement for three-dimensional effects is not used in the analyses in order to compensate for the effects of immediate settlement. The settlement analysis results are contained in Appendix C of this report.

The estimated settlement for the new fill along the roadway and at bridge embankment is anticipated to be significant and is expected to be completed over a relatively long period of time. This order of magnitude of settlement is generally unacceptable due to its impact on the surface pavement and bridge embankment settlement. Two options are feasible for mitigating the settlement concerns. Option one is to install wick drain system and expedite the settlement period and, option two is to reduce the impact of the new fill by replacing the material with lightweight fill.

Wick Drains

Option one uses prefabricated drain installed in regular pattern (wick drain system). The wick drain will accelerate the process of settlement through dissipation of the pore water pressure. It is our understanding that wick drains will be utilized at bridge embankment and along Route 37 off-ramp to Mare Island. The limits of the wick drains at the off-ramp ("CM" Line) will be between Stations 417+40 and 418+94. The wick drains in the bridge area ("HM" Line) will be between Stations 216+30 and 216+49. The estimated settlement at the maximum point of fill at the bridge and along the road is anticipated to be about 150 mm and 260 mm, respectively. The estimated time for settlement after installation of wick drains is about 25 days (rounding-off 30). The actual time may vary and settlement monitoring should be used to confirm this. We recommend that wick drains be utilized where new embankments exceed 0.6 m in height. This system will generate groundwater at the surface. The length of the wick drains should be at least 13.7 m deep from finish subgrade. This water should be collected and be tested at the time of construction to see if it meets with the environmental requirements. Additional treatment may be necessary to comply with the disposal requirements. At the bridge abutment, where batter piles are planned, pile driving should not commence until settlement is completed.

Settlement monitoring is required wherever wick drains are installed. The settlement monitoring should be as per Caltrans Test Method 112. For every 150 m one settlement plate is required along the roadway where wick drains are installed. Settlement plates are not required along on-ramp where wick drains are not installed. For the bridge embankment two to three settlement plate would be enough. Settlement readings should be surveyed every five to seven days. The settlement monitoring data should be submitted to our office on a timely bases for review.

Lightweight Fill

Second option to control settlement is to use lightweight fill. Lightweight fill should be utilized where thickness of fill reaches 0.3 m and up. The lightweight fill should include 0.6 m over excavation to the finish subgrade where fill thickness reaches 0.3 m and more. As per the layout plans provided by the designer, the lightweight fill will be along Mare Island to Route 37 on-ramp ("EM" Line) and along off-ramp ("CM" Line) between Stations 418+48 and 417+40. At the bridge embankment ("HM" Line) lightweight fill is planned between Stations 216+09 and 216+30. By using lightweight fill, the settlement along the on-ramp ("EM" Line) is estimated to be on the order of 150 mm over 30+ years.

Based on the layout plans, lightweight fill is planned adjacent to wick drains along off-ramp and bridge embankment. Different rate of settlement between lightweight fill and wick drains will cause bump at the transition zone in the long run. This will require frequent maintenance.

8.3.2 Evaluation of Embankment Stability

The embankment slope stability analysis was performed using Program Winstabl and PCSTABL5M. The stability under seismic condition was analyzed using pseudo-static approach.

Per Caltrans Guidelines for Foundation Investigations and Reports, pseudo-static analyses may be performed using a seismic factor equal to one third of the horizontal peak acceleration and not exceeding 0.2 g. A pseudo-static factor of safety equal to or greater than 1.1 is considered adequate.

It is our understanding that a small portion of the roadway embankment will be placed at about 2.4 m high slope and at 1V:1.5H gradient. This section is along the pedestrian path that climbs up the roadway embankment to connect to the existing Napa River Bridge. Based on the PCSTABL5M analyses, the factor of safety was found to be 2.16 and 1.13 for the static and pseudo-static case, respectively. In this analysis it is assumed that the slope embankment is drained and no hydrostatic pressure is allowed to build up within the embankment. Based on these results, the stability of the proposed fill slope is adequate. The results of the slope stability analyses are shown in Appendix C of this report.

TABLE 3 - SLOPE STABILITY RESULTS

Location	Station	Static Condition	Pseudo-static condition (0.2g)
S3-M Line	120+60	2.16	1.13

8.4 Earth Retaining Systems

It is our understanding that due to right-of-way and other geometric constraints, the project will require construction of one retaining wall.

- **Retaining wall No. 1– 216+09 to 216+45, 9.086 Lt. "HM" Line, (Case I, Type 1)**

The anticipated total wall length is about 36 m, with a maximum height of about 3.6 m. Based on the boring data (BM-1 & BM-2) in the vicinity of the proposed wall, the material anticipated at the footing subgrade of this wall may range from very soft to hard clay depending on the footing base elevation. Caltrans Standard Type 1 wall on concrete pile is planned to be used. In our opinion, Class 400C (355 mm square PC/PS concrete) piles may be used. The pile driving should not commence until the settlements have completed. With the use of wick drain system, we anticipated that a waiting period of 25 to 30 days might be required after earthwork construction. The pile data table for the retaining wall is presented below and a sample calculation is attached in Appendix C.

TABLE 4
PILE DATA TABLE - ALT. 'X' PC/PS CONCRETE PILES (RETAINING WALL)

Location	Pile Type	Design Loading	Nominal Resistance		Footing Elev. (m)	Design Tip Elev. (m)	Specified Tip Elev. (m)
			Compression	Tension			
Retaining Wall	355 mm Class 400C	400 kN	800 kN	400 kN	1.135	-18.68 (1), -15.63 (2), -14.11 (3)	-18.68
	355 mm Class 400C	400 kN	800 kN	400 kN	1.745	-18.07 (1), -15.02 (2), -13.50 (3)	-18.07

Design tip elevations are controlled by the following demands: (1) Compression, Tension (2), and (3) Lateral Load.

8.5 Culverts

8.5.1 Corrosion Investigation

The corrosion investigation for this project was performed in general accordance with the provisions of California Test Method 643. Representative native soil samples at the anticipated pipe subgrade were obtained for corrosion tests. A summary of the corrosion test results is presented in Table 4. Based on the results obtained, corrosion analyses were carried out using Caltrans CULVERT 5 program. The analysis results and design for culverts are presented in Table 6.

TABLE 5
SUMMARY OF CORROSION TEST RESULTS

Boring Number	Station & Offset (m)*	Sample Depth (m)	pH	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)
R-1	116+95, 41 Rt.	0 – 1.5	8.4	1600	13.5	25.9
R-7	118+84, 60 Rt.	0 – 1.5	8.49	4000	4.5	14.4
R-8	119+74, 21 Rt.	0 - 1.5	8.23	1600	3.0	11.5

* Reference Line, Route 37.

Corrosion test results indicate that near surface soil is not corrosive. However, based on our experience in Bay Mud we recommend that the concrete substructure be designed for a corrosive environment. Based on CULVERT 4 analysis, Standard reinforced concrete pipe design is suitable with Type IP (MS) modified cement or Type II Modified cement, minimum required by Caltrans Std. Specs 90-1.01. For corrugated steel pipes, the recommended pipe thickness is 1.3 mm (25-yr., Galv. 57g), 2.0 mm (50-yr., Galv. 57g), or 1.3 mm (50-yr., Galv. with bituminous coating on the soil side). For steel spiral rib pipes, the recommended thickness is 1.3 mm (50-yr. with bituminous coating on the soil side). Thermoplastic pipe can be used as an alternative and should not have any corrosion concerns. However, the types of thermoplastic pipe that can be used will depend on the height of fill, available sizes and manufacturer's specifications.

TABLE 6
RECOMMENDED MINIMUM THICKNESS AND PROTECTIVE MEASURES FOR CULVERTS

							Alternative Design (also see note below)			
Location		Culvert Type		Corrugated Steel (Galv.) (mm)			Reinforced Concrete	Corrugated Aluminum (mm)	Corrugated Aluminized Steel (Type 2, mm)	Steel Spiral Rib Pipe (Gal., mm)
		Est. Service Life (yr.)		25	50	50		50	50	50
Boring Number	**Station & Offset**	Resistivity (ohm-cm)	pH	Galv.(57 g)		Bit. Coat. (Soil Sides)				Bit. Coat. (Soil Sides)
R-1	116+95, 41 Rt.	1600	8.4	1.3	2.0	1.3	see note (2)	1.5	1.6	1.3
R-7	118+84, 60 Rt.	4000	8.49	1.3	1.6	1.3	see note (2)	1.5	1.6	1.3
R-8	119+74, 21 Rt.	1600	8.23	1.3	2.0	1.3	see note (2)	1.5	1.6	1.3

Note (1):Thermoplastic pipe can be used as an alternative and should not have any corrosion concerns. However, the types of thermoplastic pipe can be used will depend on the height of fill, available sizes and manufacturer's specifications.
Note (2): Standard reinforced concrete pipe design is suitable with Type IP (MS) modified cement or Type II Modified cement, minimum required by Caltrans Std. Specs 90-1.01.

8.6 Minor Structures

There is no soundwall or other minor structures proposed for this project.

9. Structural Pavement

R-value tests were conducted on representative samples collected at subgrade level. The test results are summarized in Table 7.

TABLE 7
SUMMARY OF R-VALUE TEST RESULTS

Sample No.	Description	R-Value
R-1	Brown sandy clay with gravel	-*
R-2	Brown with bluish mottling lean clay	24
R-3	Brown sandy lean clay	-*
R-4	Brown lean clay, trace sand	13
R-5	Brown lean clay, trace sand	-*
R-6	Light brown silty gravel	64
R-7	Light brown silty gravel	68
R-8	Brown silty clay	20
R-9	Brown silty clay	-*

* Not tested

Base on the test results and as-built information obtained from the designer, an R-value of 10 is selected for pavement design. The Traffic Indices (TIs) provided by Caltrans are for 10 and 20-year design. Utilizing State of California Department of Transportation design procedures (Highway Design Manual- Section- 608), the structural pavement section data are tabulated in Table 8.

TABLE 8
STRUCTURAL PAVEMENT SECTIONS

Location	R-value	TI	Structural Pavement Sections (mm)					
			Option 1			Option 2		Option 3
			AC	AB	AS	AC	AB	Full depth AC
Mare Island/Rte 37	10	10	165	225	330	165	525*	375
		12	195	285	390	195	650*	465
	10	10	150	180	315	-	-	-
		12	180	225	390	-	-	-
Temporary Detour Road	10	6	-	-	-	90	300	-
	10	7	-	-	-	105	355	-

AC (Type A) Asphalt Concrete, (ATPB) Asphalt Treated Permeable Base, (AB) Class 2 or 3 Aggregate Base with R-value equal to 78, (AS) Class 4 Aggregate Sub-base with the R-value equal to 50.
Based on Caltrans comments (65% PS&E Review), 150 mm AS (Cl. 4) should be applied as a working platform for full depth AC.

As-built and proposed pavement sections are shown on Plates K-1, L-1, L-2, & X-1 through X-3.

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	Sol	37	11.4/11.9		

Caltrans Metric

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
No.
Exp.
CIVIL
STATE OF CALIFORNIA

CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583

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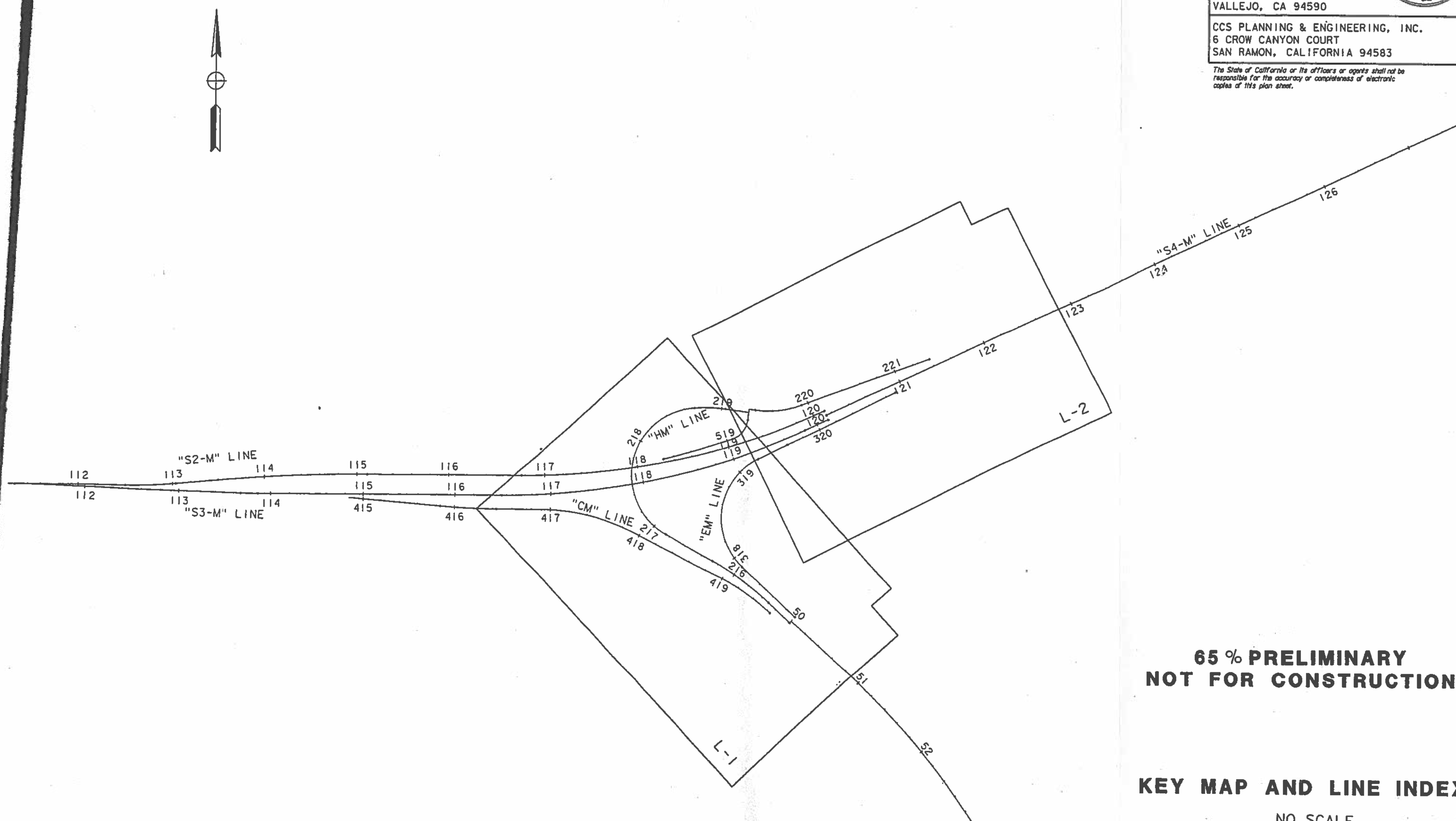

65 % PRELIMINARY
NOT FOR CONSTRUCTION

KEY MAP AND LINE INDEX

NO SCALE

K-1

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DATE: 28-Aug-02 18:49

LAST REVISION

FOR REDUCED PLANS ORIGINAL SCALE IS IN MILLIMETERS 0 20 40 60 80

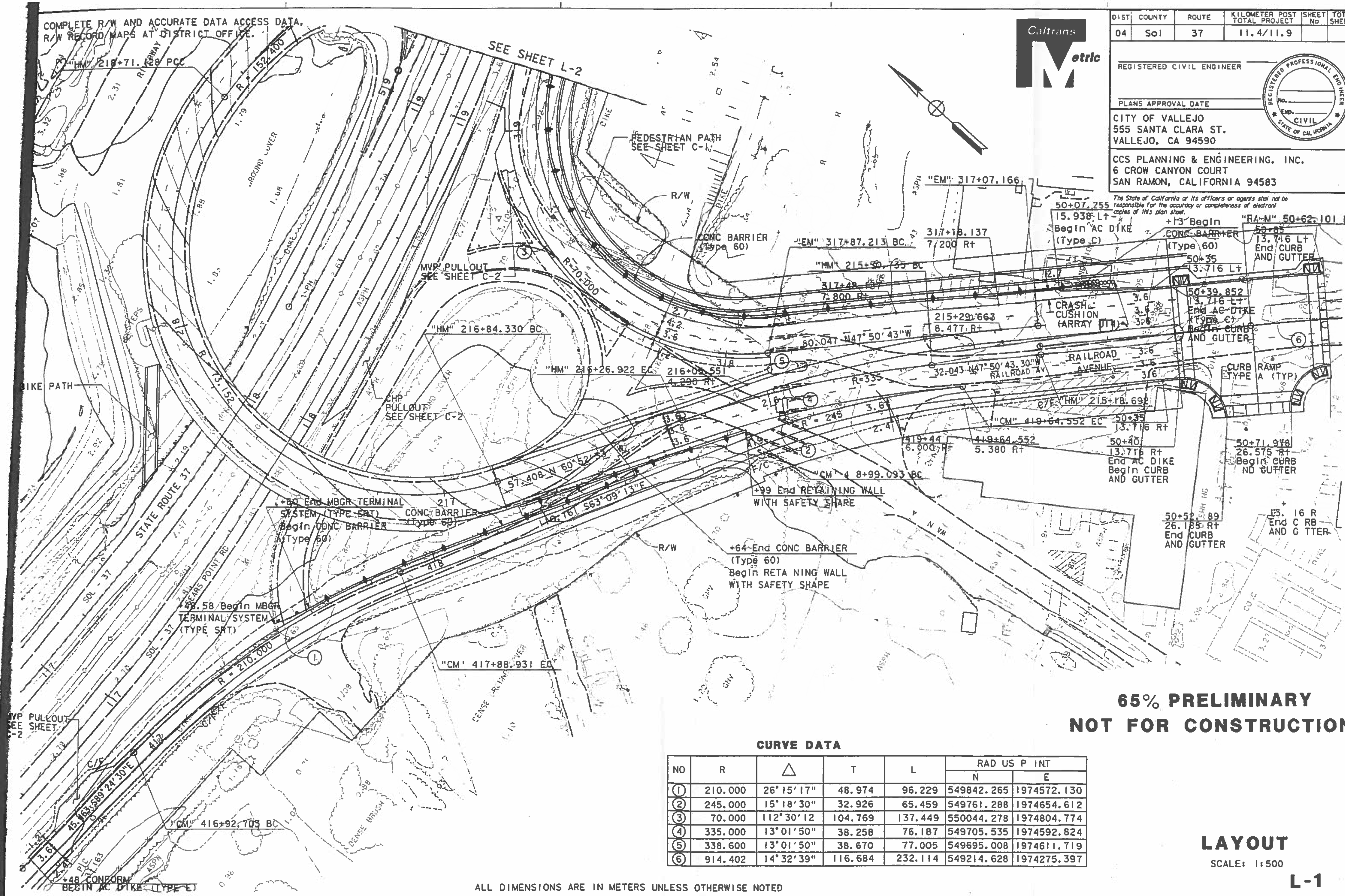
DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	Sol	37	11.4/11.9		

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583



CURVE DATA

NO	R	△	T	L	RAD US P INT N	RAD US P INT E
1	210.000	26°15'17"	48.974	96.229	549842.265	1974572.130
2	245.000	15°18'30"	32.926	65.459	549761.288	1974654.612
3	70.000	112°30'12	104.769	137.449	550044.278	1974804.774
4	335.000	13°01'50"	38.258	76.187	549705.535	1974592.824
5	338.600	13°01'50"	38.670	77.005	549695.008	1974611.719
6	914.402	14°32'39"	116.684	232.114	549214.628	1974275.397

65% PRELIMINARY
NOT FOR CONSTRUCTION

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE NOTED

LAYOUT

SCALE: 1:500

L-1

FOR REDUCED PLANS ORIGINAL 0 20 40 60 80

FILE: P:\01018\Sheets\Layout\L1.dgn DATE: 04-Sep-02 10:38

COMPL[...] R/W AND ACCURATE DATA ACCESS DATA,
R/W R[...]RD MAPS AT DISTRICT OFFICE.

DIST	COUNTY	ROUTE	TOTAL PROJECT	NO	SHEE[T]
04	Sol	37	11.4/11.9		

Caltrans Metric

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER
No.
Exp.
CIVIL
STATE OF CALIFORNIA

CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583

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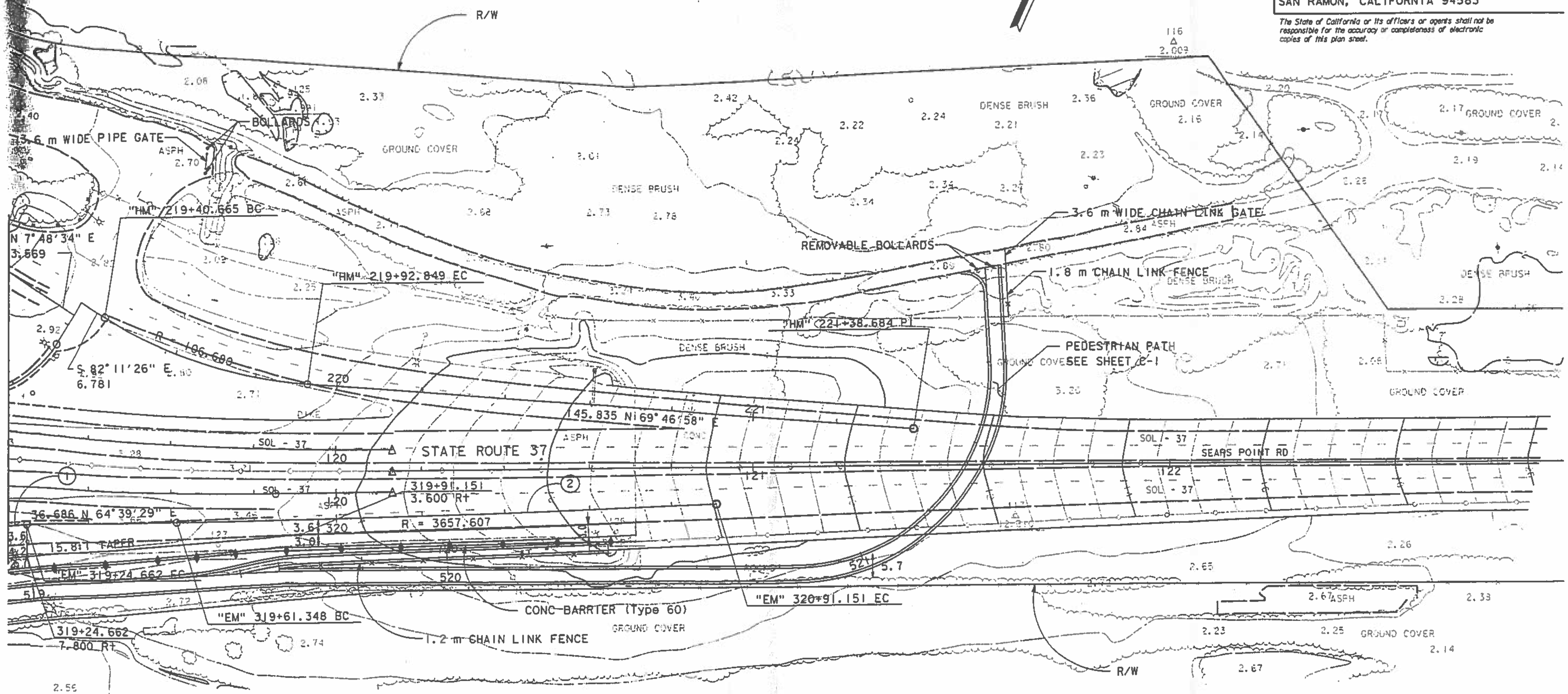

CURVE DATA

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2	3657.607	2°02'00	64.908	129.802	553428.879	1973272.406

65% PRELIMINARY
NOT FOR CONSTRUCTION

LAYOUT

SCALE: 1:500

L-2

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

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE NOTED

FOR REDUCED PLANS ORIGINAL SCALE IS IN MILLIMETERS 0 20 40 60 80

DGN FILE =>

CU 04277

EA 284700

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No
04	Sol	37	11.4/11.9	

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583

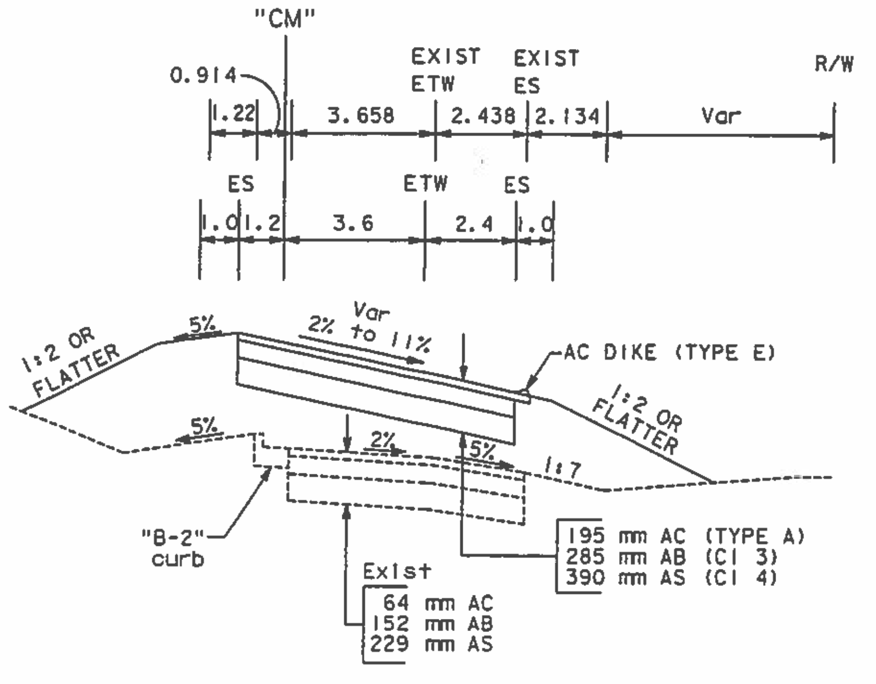
Metric

NOTES:

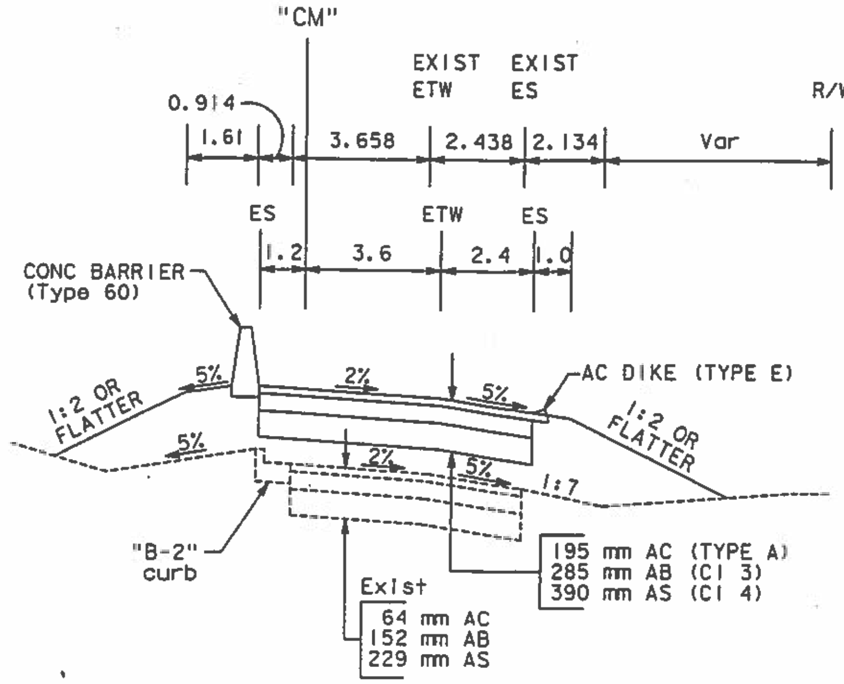
1. DIMENSIONS OF THE STRUCTURAL SECTIONS ARE SUBJECT TO TOLERANCES SPECIFIED IN THE STANDARD SPECIFICATIONS.
2. SUPERELEVATION AS SHOWN OR AS DIRECTED BY THE ENGINEER.

ABBREVIATIONS:

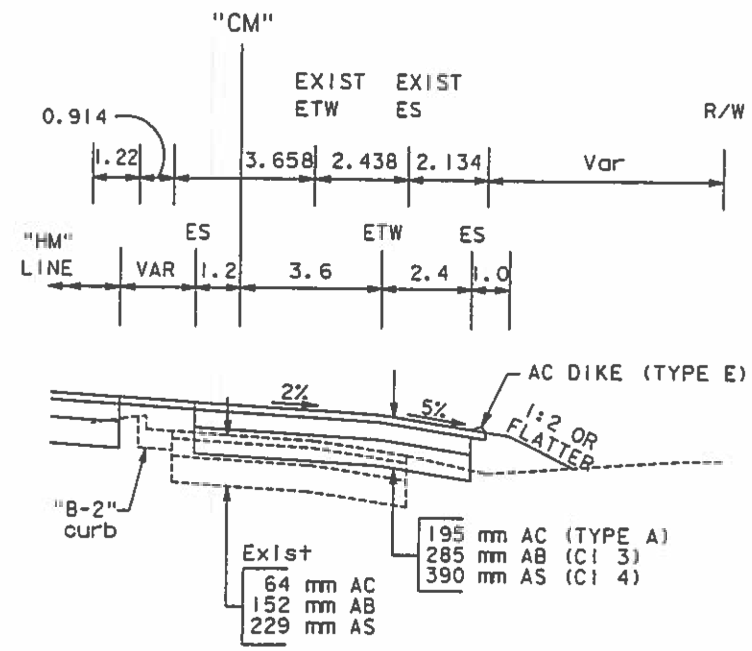
DG DECOMPOSED GRANITE



EASTBOUND DIAGONAL OFF-RAMP
STA 416+61 TO 418+00



EASTBOUND DIAGONAL OFF-RAMP
STA 418+21 TO 419+00



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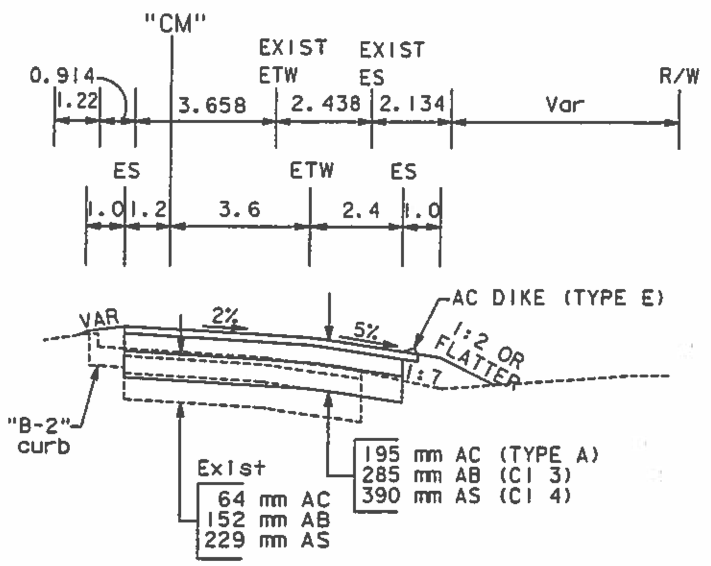
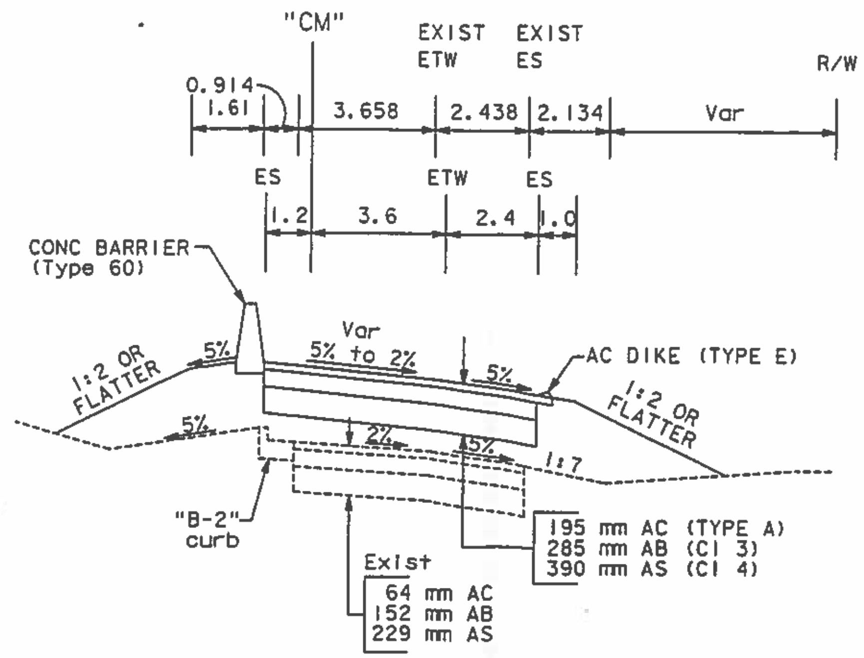
STA 419+00 TO 419+64



EASTBOUND DIAGONAL OFF-RAMP
STA 416+48 TO 416+61



EASTBOUND DIAGONAL OFF-RAMP
STA 418+00 TO 418+21

65% PRELIMINARY
NOT FOR CONSTRUCTION

TYPICAL CROSS SECTIONS

NO SCALE

X-1

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE NOTED

DATE REVISED

CHECKED BY

STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION

Caltrans

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	Sol	37	11.4/11.9		

Caltrans Metric

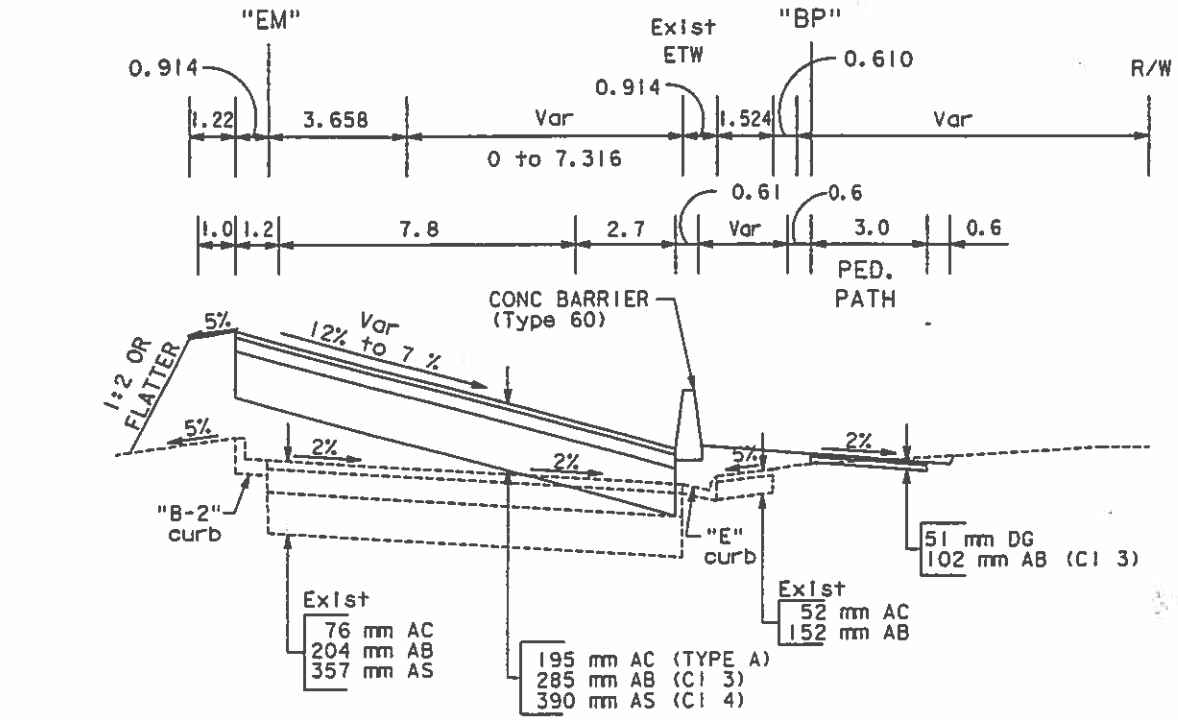
REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

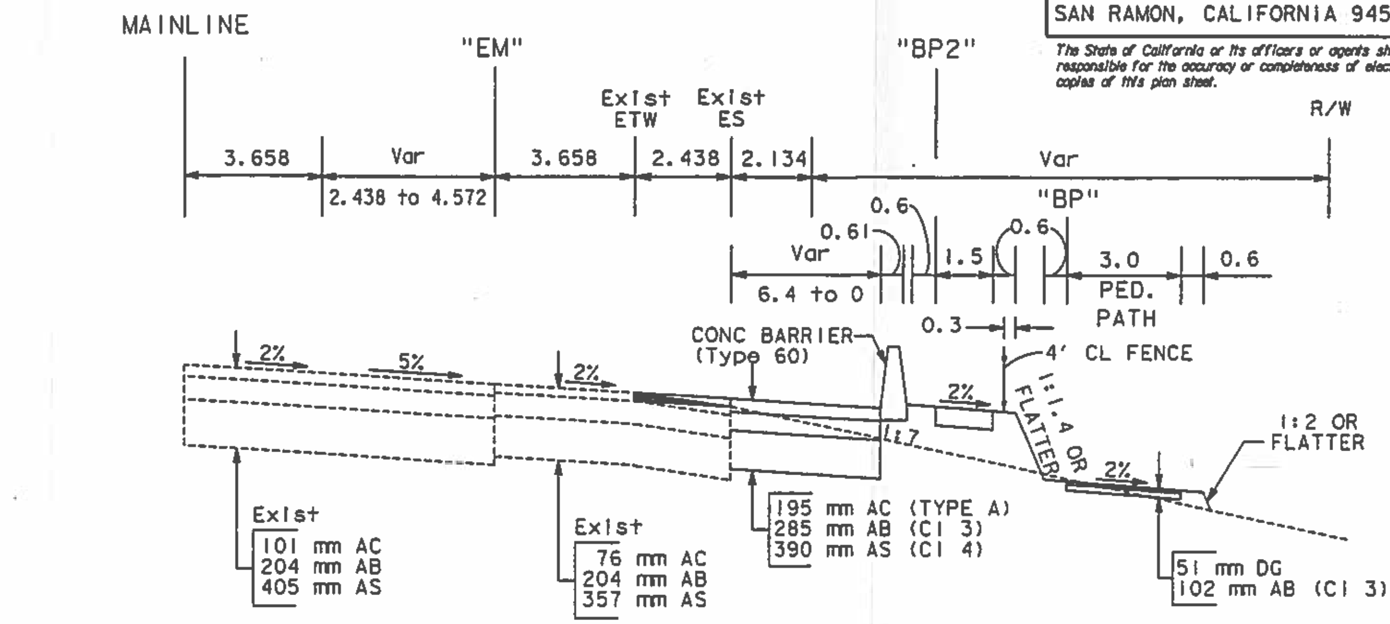
CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583



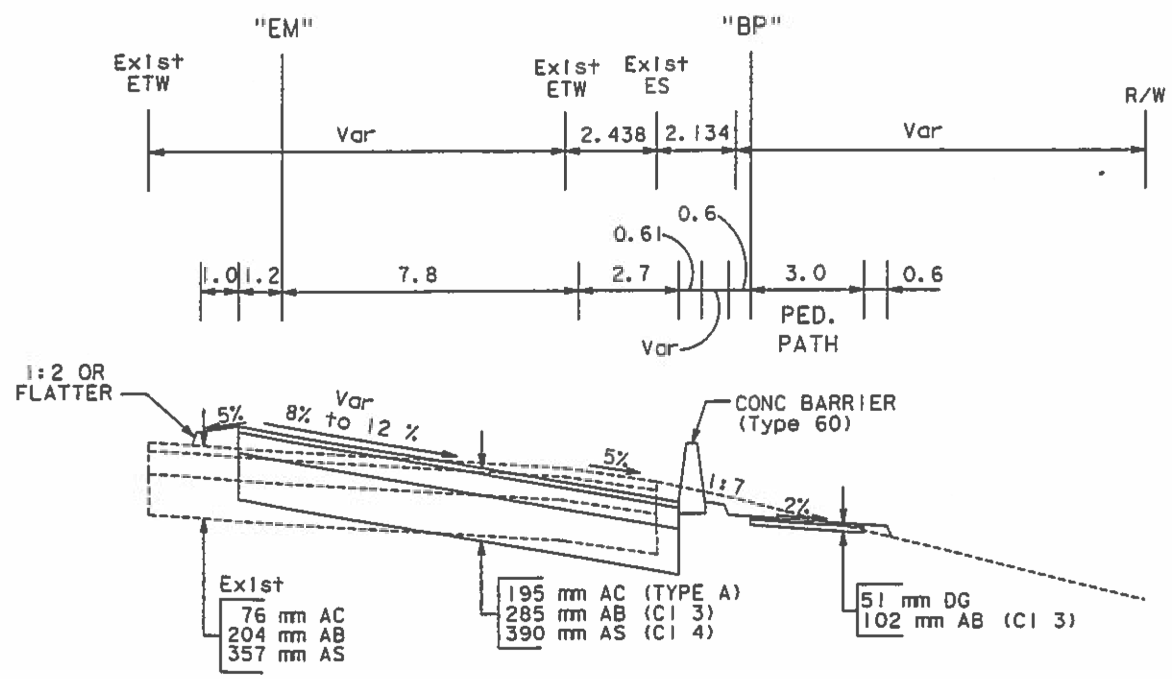
EASTBOUND DIAGONAL ON-RAMP

STA 318+33 TO 319+15



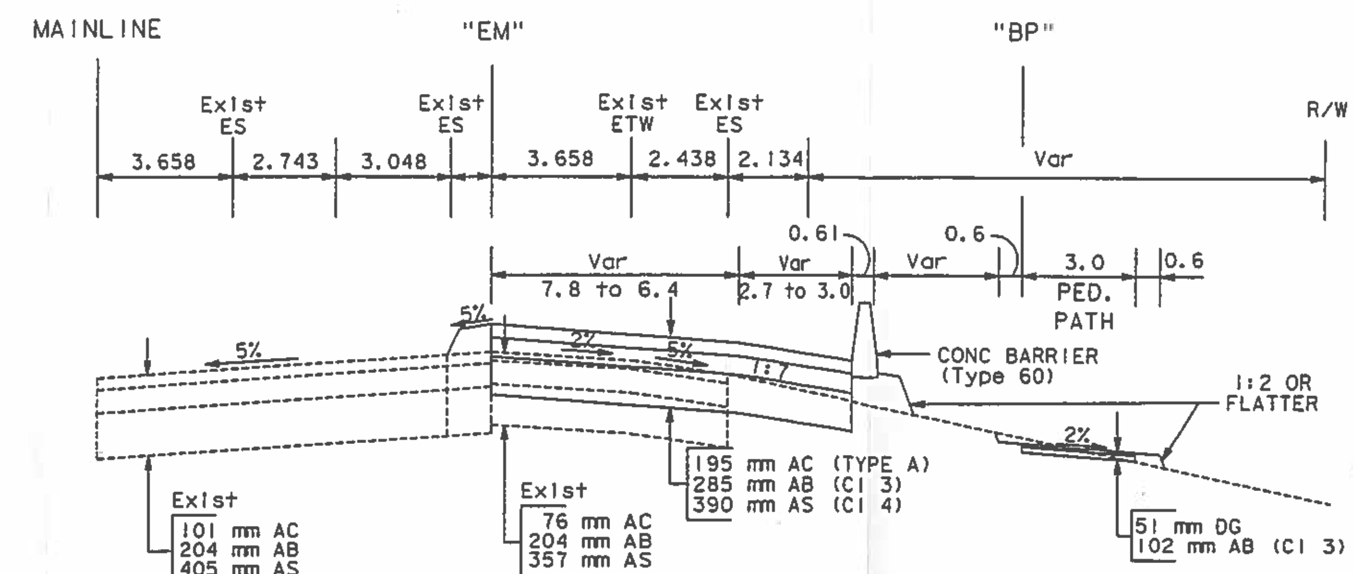
EASTBOUND DIAGONAL ON-RAMP

STA 319+45 TO 320+71



EASTBOUND DIAGONAL ON-RAMP

STA 317+87 TO 318+33



EASTBOUND DIAGONAL ON-RAMP

STA 319+15 TO 319+45

65% PRELIMINARY
NOT FOR CONSTRUCTION

TYPICAL CROSS SECTIONS

NO SCALE

X-2

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

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE NOTED

FILE: P:\01018\Sheets\nor-X2.dgn DATE: 28-Aug-02 18:12
LAST REVISION

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No	TOTAL SHEETS
04	Sol	37	11.4/11.9		

Caltrans Metric

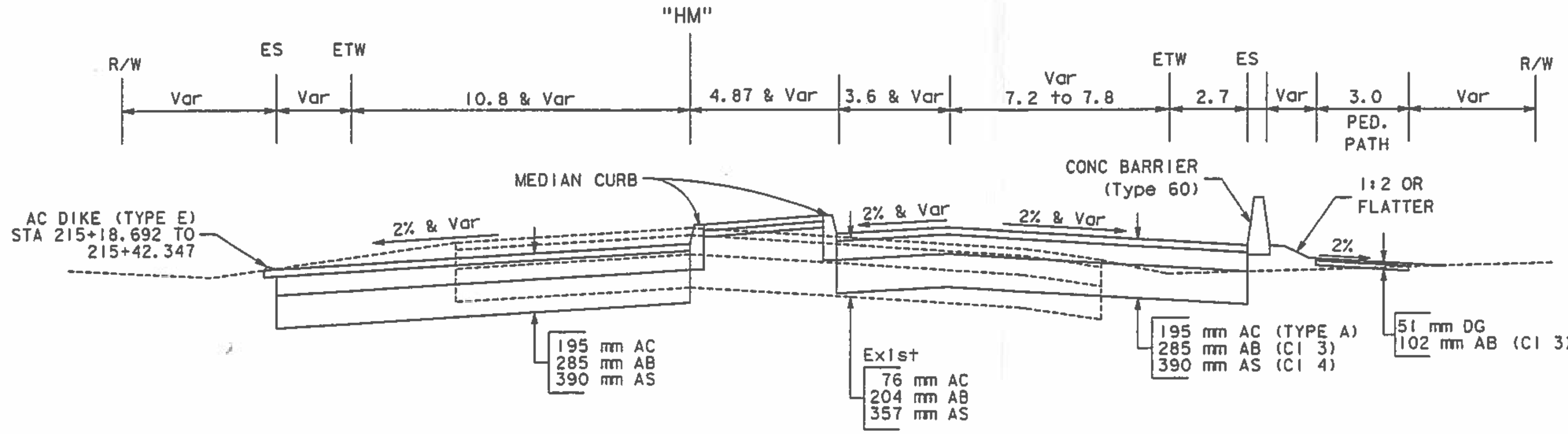
REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

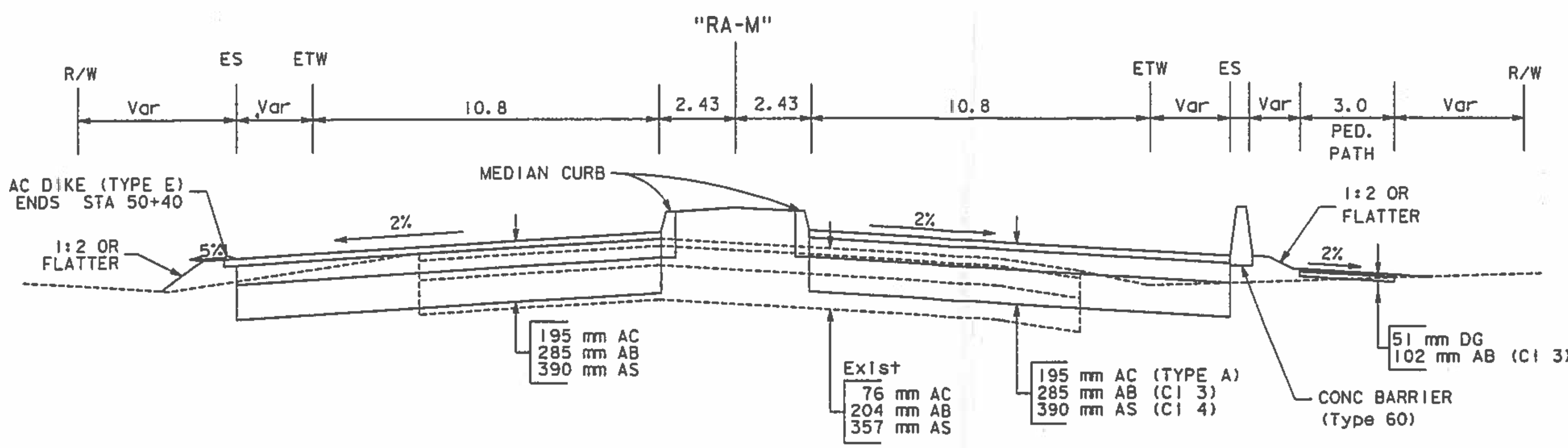
REGISTERED PROFESSIONAL ENGINEER No. Exp. CIVIL STATE OF CALIFORNIA

CITY OF VALLEJO
555 SANTA CLARA ST.
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING, INC.
6 CROW CANYON COURT
SAN RAMON, CALIFORNIA 94583



OVERCROSSING RAMP
STA 215+18 TO 216+09



RAILROAD AVENUE
STA 50+00 TO 50+84

65% PRELIMINARY
NOT FOR CONSTRUCTION

TYPICAL CROSS SECTIONS
NO SCALE

X-3

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

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE NOTED

LAST REVISION | FILE: P:\01018\Sheets\Vo\rX3.dgn DATE: 28-Aug-02 19:47

10. Material Sources

There are several commercial sources of asphalt, concrete, and aggregate products in the area. Table 9 lists available commercial suppliers in the area.

TABLE 9
SOURCES OF IMPORTED BORROW

Source	Location	Approx. Haul Dist. (one way, km)
Solano Concrete Co.	Dixon, Tremont Rd.	18
Solano Concrete Co. Inc.	Fairfield, Cement Hill Rd.	24
Granite Construction Co.	Suisun City, Lozano Ln.	25
Don Pridmore & Son Constr. Co.	Napa, Capell Valley Rd.	54

11. Material Disposal

Majority of the project will require fill for the proposed widening. Based on our understanding, the project will not require disposal of the excess materials.

12. Construction Considerations

12.1 Construction Advisories

The following sections of the report include comments related to excavation, dewatering, temporary excavation and shoring, foundation construction, earthwork and other geotechnical aspects of the proposed construction. The sections are written primarily for the engineer responsible for the preparation of plans and specifications. Since these sections identify potential construction issues related to the project, it may also be of use to the Agency's representatives involved in monitoring of construction activity. The field investigation performed by us primarily addresses design issues and was not planned specifically to identify construction issues.

Prospective contractors for the project must evaluate construction-related issues on the basis of their own knowledge and experience in the local area, on the basis of similar projects in other localities, or on the basis of field investigation on the site performed by them, taking into account their proposed construction methods and procedures. In addition, construction activities related to excavation and lateral earth support must conform to safety requirements of OSHA and other applicable municipal and State regulatory agencies.

Groundwater should be expected during construction, specially during overexcavation and placement of lightweight fill and retaining wall construction. If soft clay (Bay Mud) is exposed during overexcavation it can be treated by using a working platform. The suggested working platform is discussed below:

1. Place a layer of subgrade enhancement geofabric such as Mirafi 600x or equivalent at the subgrade.

2. Place 0.6 m of lightweight fill over the geofabric. The fill should be compacted to 95% relative compaction per ASTM 1557-91. The subgrade enhancement geofabric should then be wrapped on top of the compacted lightweight fill for a minimum of 0.6 m from the edge. The 0.6 m thick layer of the lightweight fill can serve as a "reinforced mat" for reducing load imposed on the underlying materials and provide a working platform. The groundwater should be maintained at least 1 m below the bottom of the excavation at all times.

12.2 Hazardous Waste Considerations

The project environmental study report (if any) should be referred to for further details at this site. Groundwater sampling and testing has been conducted and is provided under a separate cover by others.

12.3 Differing Site Conditions

The soil conditions described in this report are based on available boring data. It should be noted that these borings depict subsurface conditions only at the locations drilled. Because of the variability from place to place within soils in general, and the nature of geologic depositions, subsurface conditions could change between the explored locations.

Early communication should be made between the Resident Engineer, the Contractor and the Geotechnical Engineer as soon as conditions that differ from those established in this report are recognized by any of the parties. Additional recommendations could be provided if such conditions arise.

13. Recommendations and Specifications

13.1 Summary of Recommendations

If the designer has questions or concerns with any of these recommendations, or, if conditions are found to be different during construction, the Geotechnical Engineer who prepared this report should be contacted. Additional fieldwork, analysis or changes in recommendations may be required. These services may be provided under a separate authorization, as necessary. A concise summary of the geotechnical recommendations is presented below:

- As per preliminary investigation performed by others in the proximate area (July 2001) and based on the as-built LOTBs by Caltrans (1956), groundwater was encountered at between 0.5 m and 1.5 m below ground surface.
- Design peak bedrock acceleration (PBA) = 0.4 g. Design peak ground acceleration (PGA) = 0.25 g. (Ref.: Section 8.1)

- The boring data indicates that the subsoils generally consist of very stiff to hard lean clay and soft fat clay (Bay Mud). Due to rotary wash method of drilling, groundwater was not measured during current investigation. The liquefaction potential is considered low. (Ref.: Section 8.1.3)

- Proposed is new embankment at the bridge abutment. Based on the analysis the estimated settlement is expected to be significant. Use of wick drain is recommended at bridge embankment.

- Proposed is a pile-supported retaining wall (Type 1, Case 1) along "HM" Line between stations 216+09 and 216+45 (Section 8.4).

- Groundwater was encountered at Elev. 0.3 m in the boring R-2.

- Pavement Sections (Ref: Section 9).

13.2 Recommended Materials Specifications

13.2.1 Standard Specifications

Unless otherwise stated in the special provisions, all materials specifications should conform to Caltrans Standard Specifications, July 1999 edition, including but not limited to the following: Earthwork, Structure Backfill, Pervious Backfill Material, Reinforcing Geofabric, Thermoplastic Pipes, Asphalt Concrete, Aggregate Base, Aggregate Subbase, Cement Treated Base, etc.

13.2.2 Special Provisions

Imported Borrow:

Imported material should be in accordance with the specifications set forth in Caltrans Section 19. In particular, for new embankment/roadway construction, the material placed within 1.5 m of the finish pavement subgrade should meet the following requirements:

1. Free of organic or other deleterious materials.
2. An R-value of no less than 10 for ramps.

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

Class 4 aggregate subbase shall be clean and free from organic matter and other deleterious substances. The percentage composition by weight of Class 4 aggregate subbase shall conform to the following grading as determined by California Test Method No. 202.

Gradation Requirement (Percent Passing)

Sieve Sizes	Operating Range	Contract Compliance
63 mm	100	100
4.75 mm	30 - 65	25 - 70
75-µm	0 - 15	0 – 18

Class 4 aggregate subbase shall also conform to the quality requirements given on the following table:

Quality requirements

California Test Method	Operating Range	Contract Compliance
Sand Equivalent (217)	21 Min.	18 Min.
Resistance (R-value) (301)	-	50 Min.

Aggregate Base: Class 3 aggregate base shall conform to the provisions in Section 26 of the Standard Specifications and to these Special Provisions. It shall also be clean and free from organic matter and other deleterious substances. The percentage composition by weight of Class 3 aggregate base shall conform to the following grading as determined by California Test Method No. 202.

Gradation Requirement (Percent Passing)

Sieve Sizes	19 mm Maximum	
	Operating Range	Contract Compliance
25 mm	100	100
19 mm	90 - 100	87 - 100
4.75 mm	35 - 60	30 - 65
600 µm	10 - 30	5 - 35
75 µm	2 - 11	0 - 14

Gradation Requirement (Percent Passing)

Sieve Sizes	37.5 mm Maximum	
	Operating Range	Contract Compliance
50 mm	100	100
37.5 mm	90 - 100	87 – 100
25 mm	------	------
19 mm	50 - 85	45 – 90
4.75 mm	25 - 45	20 – 50
600 µm	10 - 25	6 – 29
75 µm	2 - 11	0 – 14

Quality requirements

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

14. INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our field exploration and the assumption that the soil conditions do not deviate from observed conditions.

No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. The scope of our services did not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in structures, soil, surface water, groundwater or air, below or around this site. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed project as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our findings and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the soil conditions can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge.

Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

Manny Saleminik, P.E. C60597
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
GAURANG R. PARIKH
No. 666
EXP. 12-31-06
GEOTECHNICAL
STATE OF CALIFORNIA

Gary Parikh, P.E., G.E. 666
Project Manager

MS/GP/201126.GDR {2B}

REFERENCES

1. Applied Technology Council, June 1996, Improved Seismic Design Criteria for California Bridges: provisional Recommendations, Report No. ATC-32.

2. California Department of Transportation, July 1999, Standard Plans, 284p.

3. California Department of Transportation, July 1999, Standard Specifications, Sections 1 through 95.

4. California Department of Transportation, Highway Design Manual, fifth edition, July 1995.

5. California Department of Transportation, April 1992, Information and Procedures Guide.

6. Department of the Navy, Navy Facilities Engineering Command, May 1982, Foundation and Earth Structures, Design Manual 7.2, 244p.

7. Department of the Navy, Navy Facilities Engineering Command, May 1982, Soil Mechanics, Design Manual 7.1, 348p.

8. Department of the Navy, Navy Facilities Engineering Command, April 1983, Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction, Design Manual 7.3, 102p.

9. Dibblee, Jr., Thomas, "Preliminary Geologic Map of The Mare Island Quadrangle, Solano and Contra Costa Counties, California, 1981.

10. Earthquake Engineering Research Institute, May 1990, Loma Prieta Earthquake Reconnaissance Report, Earthquake Spectra, Supplement to Volume 6.

11. Mualchin, L., A Technical Report To Accompany the Caltrans California Seismic Hazard Map (1996).

12. Mualchin, L., 1996, The Caltrans California Seismic Hazard Map.

13. National Research Council, Washington, DC, 1985, Liquefaction of Soils during Earthquakes, PB86-163110, November 1985.

14. Preliminary Geotechnical Engineering Study and Consolidation Evaluation for Proposed North Mare Island Business Park, Vallejo, CA, by LFR Levine-Fricke, dated July 20, 2001.

15. Sadigh, K., Chang, C.Y., Egan, J.A., Makdisi, and Youngs, R.R., 1997, "Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data", Seismological Research Letters, Volume 68, No. 1, January/February 1997.

REFERENCES(Continued)

16. Seed, R. Bolton and Dickinson, S., 1993, Site-Dependent Seismic Site Response: Proceedings, Second Annual Seismic Research Workshop, Caltrans, March 1993.

17. Seed, H. B. and I. M. Idriss, "Ground Motions and Soil Liquefaction during Earthquakes", 1982.

18. Seed, H.B. and Martin, G.R., 1966, "The Seismic Coefficient of Earth Dam Design," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol 92, No. SM3, Proceedings Paper 4824, pp. 25-48.

19. Terzaghi, Karl and Peck, Ralph B., 1967, Soil Mechanics in Engineering Practice; John Wiley & Sons, New York; 729p.

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Sol	37	11.4/11.9	1	

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INDEX OF SHEETS

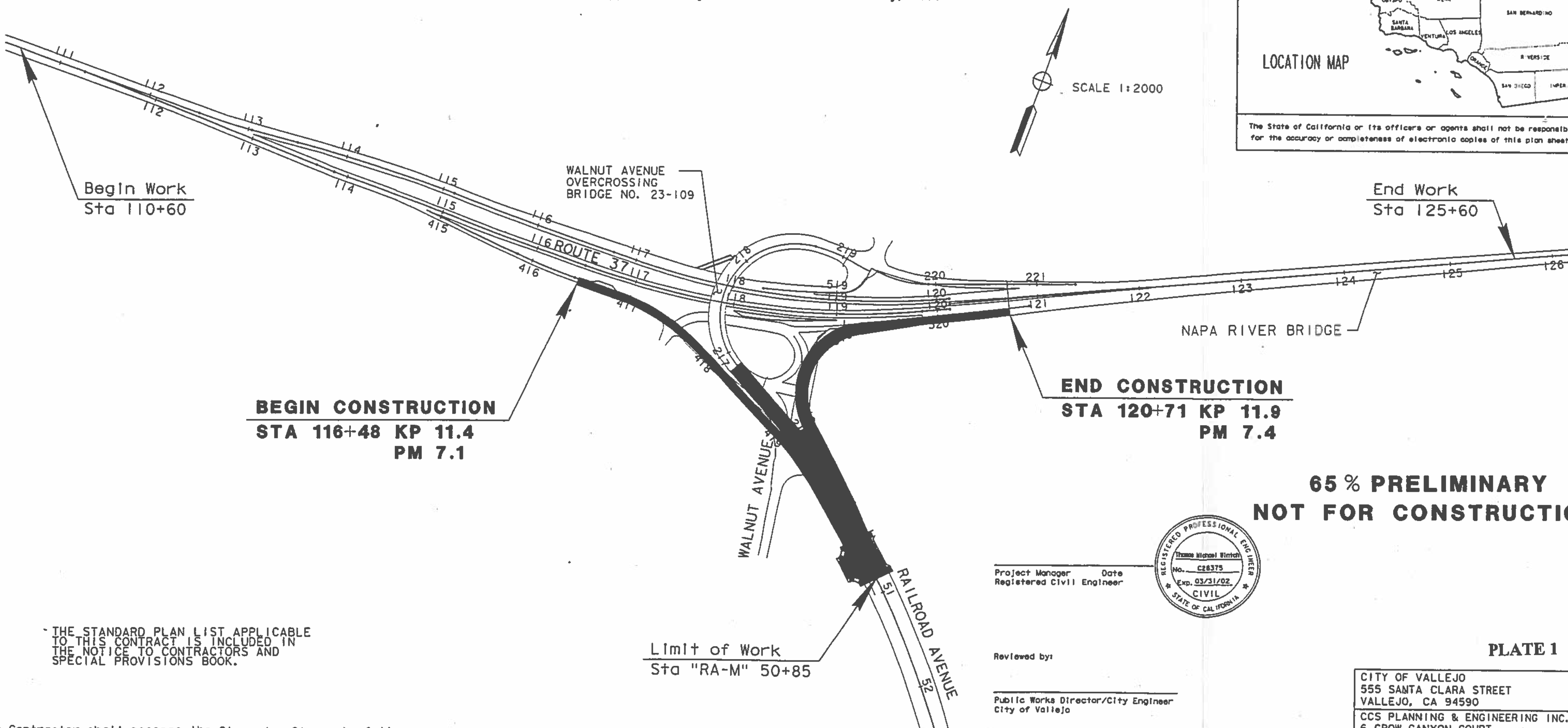
SHEET NO.

CITY OF VALLEJO

DEPARTMENT OF PUBLIC WORKS
PROJECT NO. 9966

PROJECT PLANS FOR CONSTRUCTION ON
STATE HIGHWAY
IN SOLANO COUNTY
FROM 0.2 KM WEST OF WALNUT AVENUE OVERCROSSING TO THE WEST ABUTMENT OF NAPA RIVER BRIDGE

To be supplemented by Standard Plans dated July, 1999



65 % PRELIMINARY
NOT FOR CONSTRUCTION

REGISTERED PROFESSIONAL ENGINEER
Thomas Michael Rintch
No. C28375
Exp. 03/31/02
CIVIL
STATE OF CALIFORNIA

Project Manager Date
Registered Civil Engineer

Reviewed by:

Public Works Director/City Engineer
City of Vallejo

Date

- THE STANDARD PLAN LIST APPLICABLE TO THIS CONTRACT IS INCLUDED IN THE NOTICE TO CONTRACTORS AND SPECIAL PROVISIONS BOOK.

The Contractor shall possess the Class (or Classes) of license as specified in the "Notice to Contractors".

PLATE 1

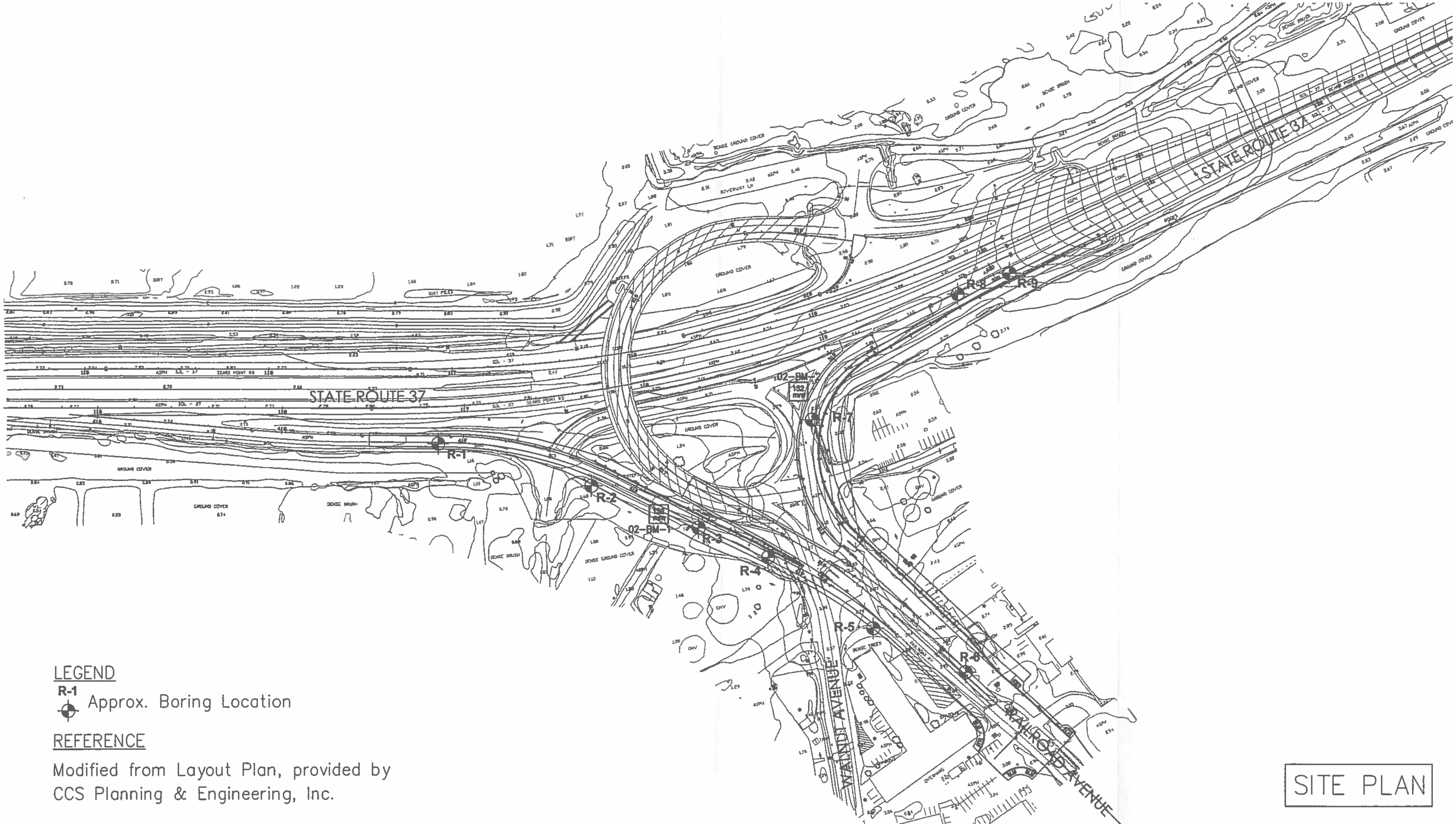
CITY OF VALLEJO
555 SANTA CLARA STREET
VALLEJO, CA 94590

CCS PLANNING & ENGINEERING INC.
6 CROW CANYON COURT
SAN RAMON, CA 94583

Approved as to impact on State facilities and conformance with applicable State standards and practices and that technical oversight was performed as described in the California Department of Transportation A & E Consultant Service Manual.

PROJECT ENGINEER
LARRY TAYLOR

DESIGN OVERSIGHT APPROVAL
PRINT NAME: SIGNATURE:
REGISTRATION NO. DATE



LEGEND

R-1 Approx. Boring Location

REFERENCE

Modified from Layout Plan, provided by CCS Planning & Engineering, Inc.

SCALE

1:1000

SITE PLAN

PARIKH CONSULTANTS INC.
GEOTECHNICAL CONSULTANTS AND MATERIALS TESTING

MARE ISLAND/ROUTE 37
SOLANO COUNTY, CALIFORNIA

JOB NO.: 201126.GDR

PLATE NO.: 2

Legend:

Qbm - Surficial deposites, Bay Mud

Qa - Surficial deposites, Alluvium

Kp - Panoche Formation, micaceous shale, minor thin sandstone beds

Qoa - Older Alluvium

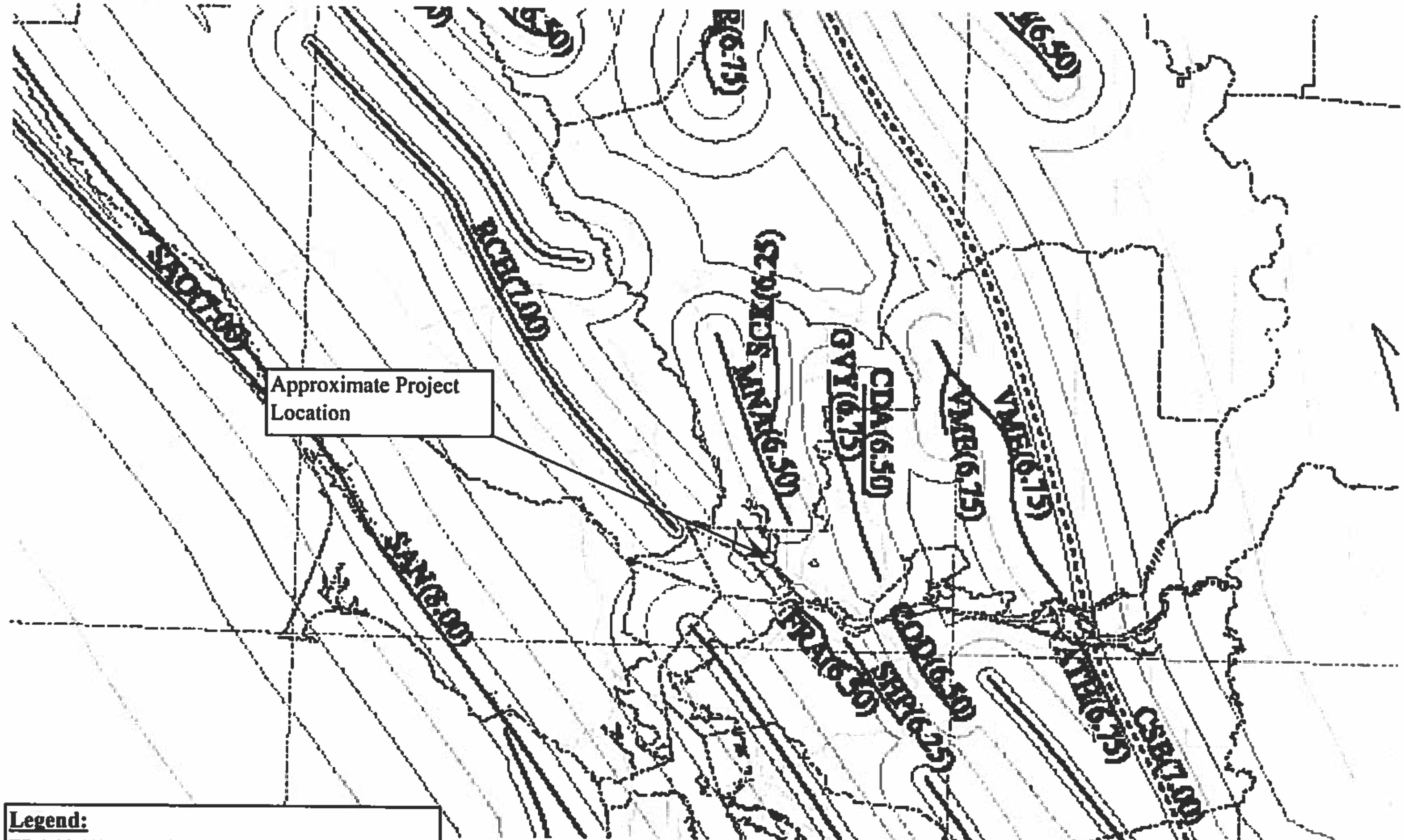
Source:

Preliminary Geologic Map of The Mare Island Quadrangle, Solano and Contra Costa Counties, California
by Thomas W. Dibblee, Jr., 1981,
USGS Open-File Report 81-234
Scale: 1 : 24,000

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS TESTING

MARE ISLAND/ROUTE 37
SOLANO COUNTY, CALIFORNIA

JOB NO.: 201126.GDR | PLATE NO.: 3



Legend:
FRA(6.50) - Franklin
GVY(6.50) - Green Valley
RCH(7.00) - Rodgers Creek-Healdsburg
SAN(8.00) - San Andreas

Source: Modified from "California Seismic Hazard Map 1996", by L. Mualchin

FAULT MAP

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	MARE ISLAND/ROUTE 37 SOLANO COUNTY, CA	
	JOB NO.: 201126.GDR	PLATE NO.: 4

APPENDIX A

APPENDIX A

LIST OF LOGS OF TEST BORINGS

		Boring Nos.
Plate A-1	-	BM-1, BM-2
Plate A-2	-	R-1, R-2, R-3, R-4
Plate A-3	-	R-5, R-6, R-7, R-8

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SOL	37	11.4/11.9		

REGISTERED ENGINEER-GEOTECHNICAL

PLANS APPROVAL DATE

GARY PARIKH No. G.E. 666 Exp. 12/31/05 GEOTECHNICAL STATE OF CALIFORNIA

PARIKH CONSULTANTS, INC.
356 S. MILPITAS
MILPITAS CA 95035

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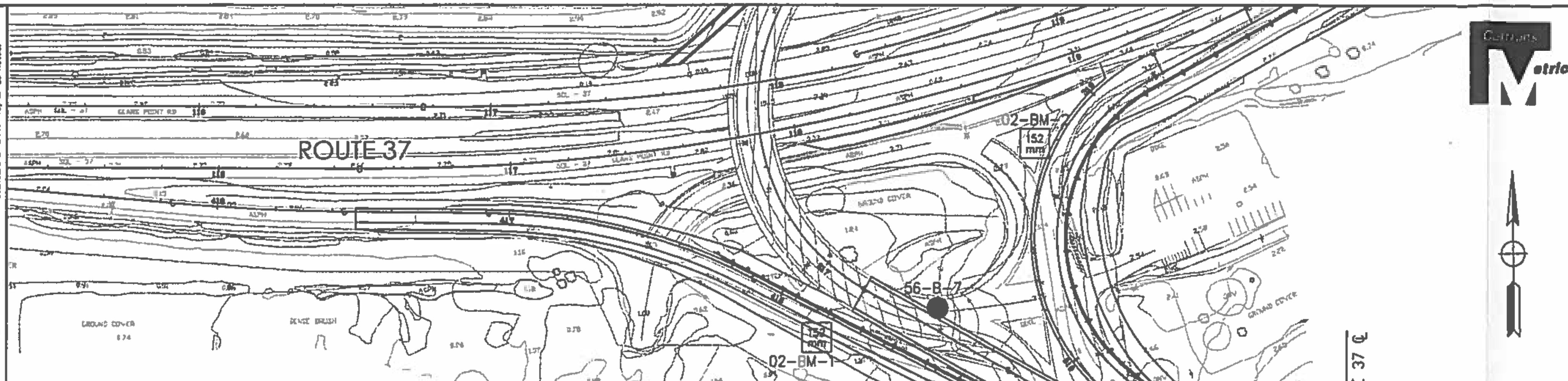
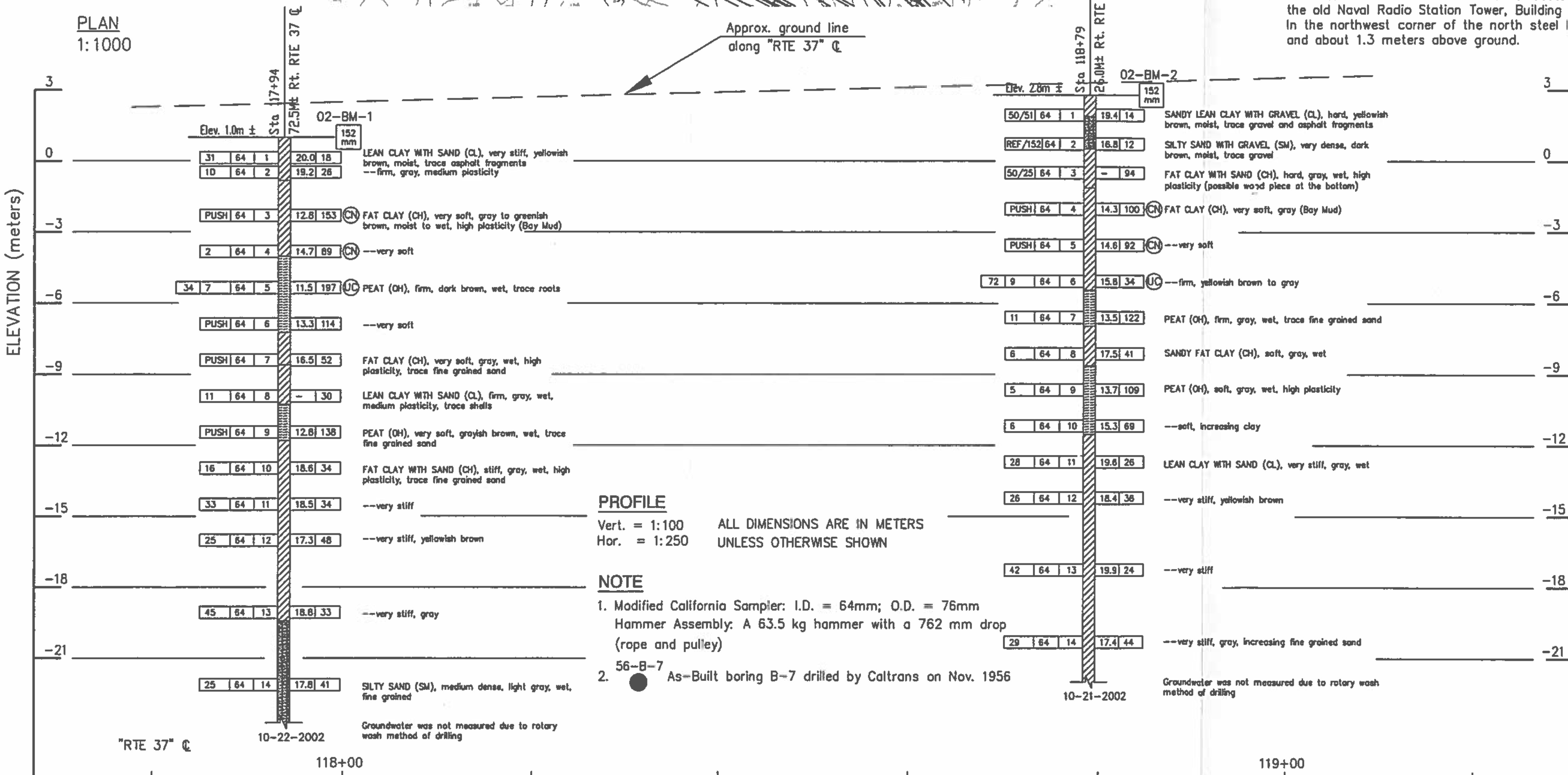
Caltrans Metric



PLAN
1:1000

BENCH MARK NO. X552

Elev. 3.980 m
First Order Class II standard NGS Brass Disk in concrete base of a radio tower base located at the old Naval Radio Station Tower, Building 505. In the northwest corner of the north steel leg and about 1.3 meters above ground.



PROFILE
Vert. = 1:100
Hor. = 1:250

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

NOTE
1. Modified California Sampler: I.D. = 64mm; O.D. = 76mm Hammer Assembly: A 63.5 kg hammer with a 762 mm drop (rope and pulley)
2. 56-B-7 ● As-Built boring B-7 drilled by Caltrans on Nov. 1956

c:\ocad\201126\LOTB\LOTB.dwg

LEGEND OF BORING OPERATIONS · IN-SITU, LAB & FIELD TEST DESIGNATIONS · TYPES OF BORINGS · LEGEND OF EARTH MATERIALS (USCS) BASED ON ASTM D2487, D2488 · CONSISTENCY CLASSIFICATION FOR SOILS According to the Standard Penetration Test (ASTM D-1586)

DESIGN OVERSIGHT	DESIGN	By: F. WANG	Field investigation by:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	M. SALEMINIK PROJECT ENGINEER	BRIDGE NO 23-109	MARE ISLAND/ROUTE 37
SIGN OFF DATE	CHECKED	By: M. SALEMINIK	L. BHANGOO			KILOMETER POST 11.6	LOG OF TEST BORINGS

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04268
EA 284701

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) ~~12/3/2002~~ ~~12/20/2002~~ ~~1/23/2003~~ 3/18/2003

SHEET OF

PLATE A-1

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Sol	37	11.4/11.9		

REGISTERED ENGINEER-GEOTECHNICAL

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
356 S. MILPITAS BLVD
MILPITAS CA 95035

GARY PARIKH No. G.E. 666 Exp. 12/31/05 GEOTECHNICAL STATE OF CALIFORNIA

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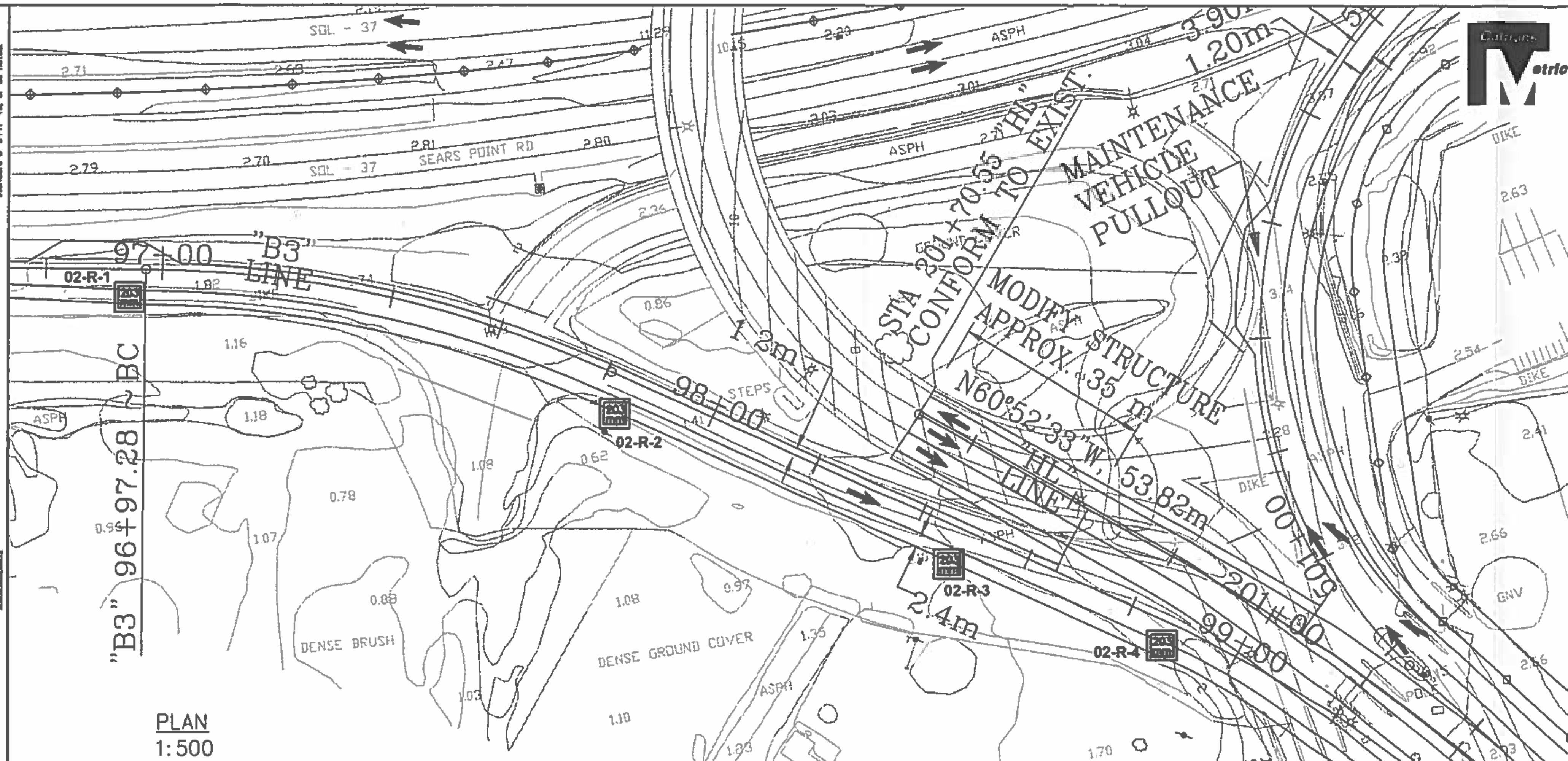

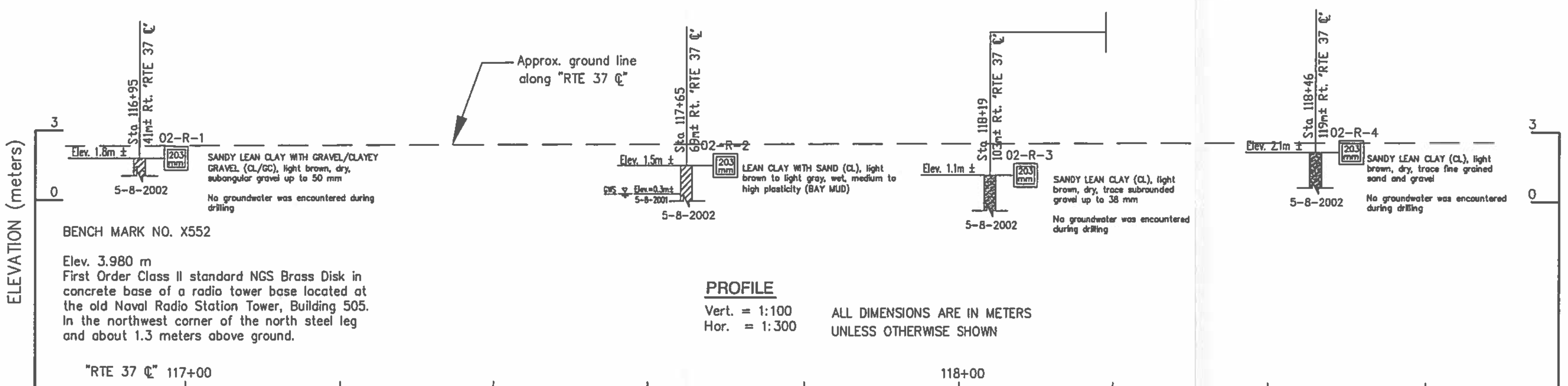

BENCH MARK NO. X552

Elev. 3.980 m
First Order Class II standard NGS Brass Disk in concrete base of a radio tower base located at the old Naval Radio Station Tower, Building 505. In the northwest corner of the north steel leg and about 1.3 meters above ground.

PROFILE
Vert. = 1:100
Hor. = 1:300

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

LEGEND OF BORING OPERATIONS

IN-SITU, LAB & FIELD TEST DESIGNATIONS

TYPES OF BORINGS

LEGEND OF EARTH MATERIALS (USCS) BASED ON ASTM D2487, D2488

CONSISTENCY CLASSIFICATION FOR SOILS According to the Standard Penetration Test (ASTM D-1586)

NOTE: Visual classifications of earth materials are based on field inspection and are confirmed or revised with laboratory test results as necessary.

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DESIGN OVERSIGHT		DESIGN	By: F. WANG	Field investigation by:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	M. SALEMINK PROJECT ENGINEER	BRIDGE NO	MARE ISLAND/RTE 37
SIGN OFF DATE		CHECKED	By: M. SALEMINK	F. WANG			KILOMETER POST	LOG OF TEST BORINGS

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04277
EA 284700

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) 6/4/2002 12/20/2002 3/16/2003 SHEET OF

PLATE A-2

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	Sol	37	11.4/11.9		

REGISTERED ENGINEER-GEOTECHNICAL

PLANS APPROVAL DATE

PARIKH CONSULTANTS, INC.
356 S. MILPITAS BLVD
MILPITAS CA 95035

GARY PARIKH No. G.E. 666 Exp. 12/31/03 GEOTECHNICAL STATE OF CALIFORNIA

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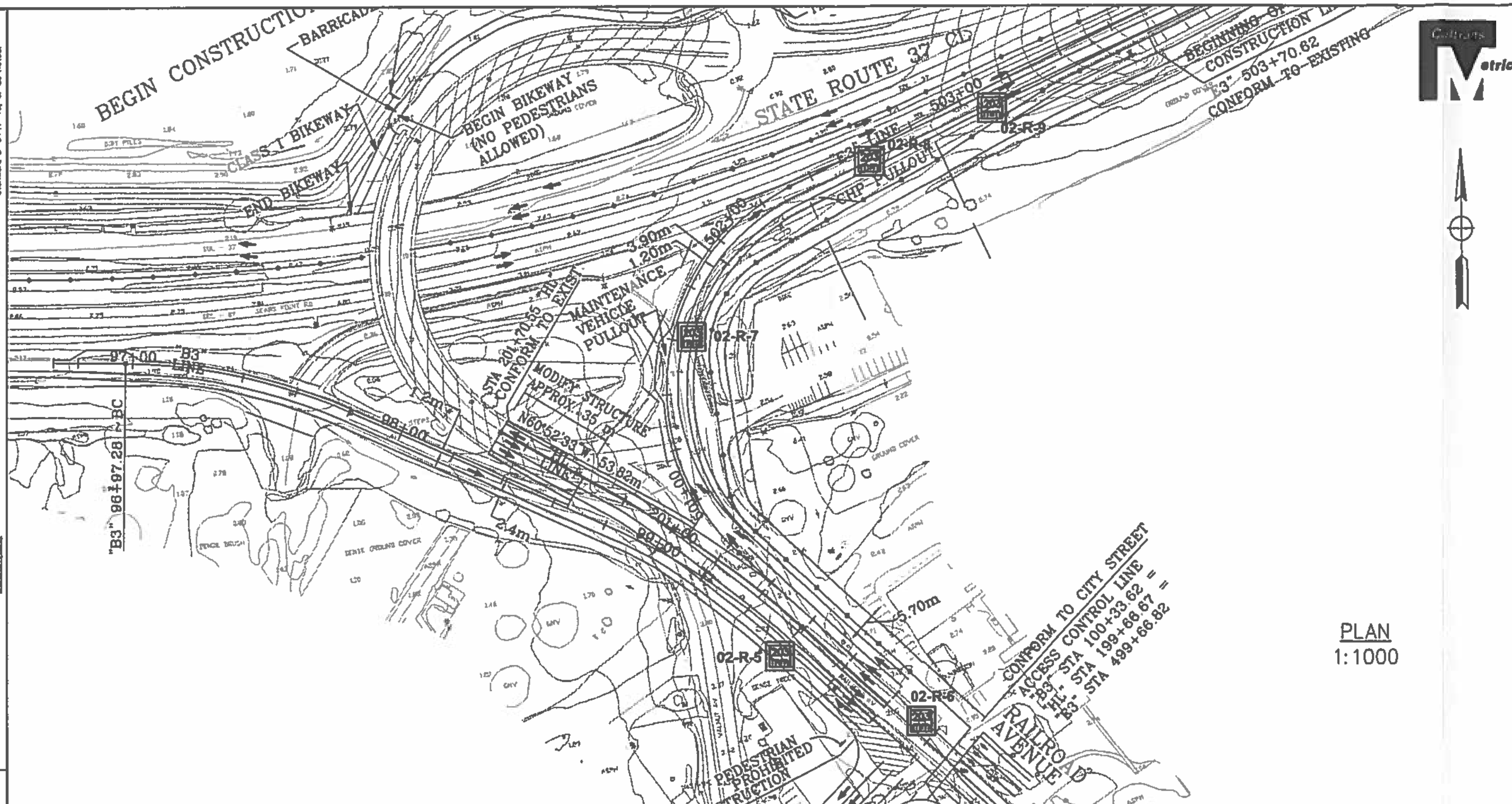
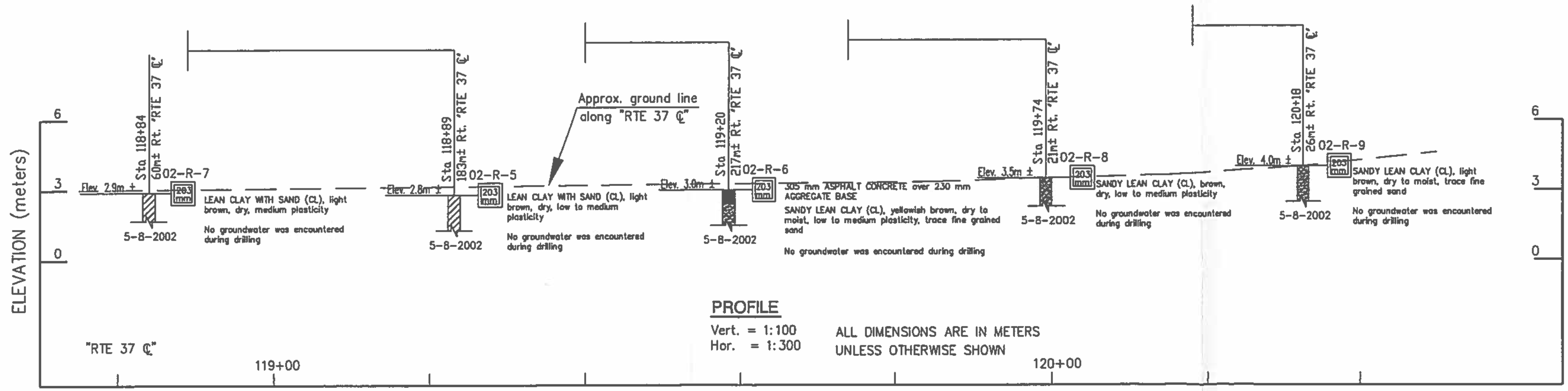
BENCH MARK NO. X552

Elev. 3.980 m
First Order Class II standard NGS Brass Disk in concrete base of a radio tower base located at the old Naval Radio Station Tower, Building 505. In the northwest corner of the north steel leg and about 1.3 meters above ground.





PROFILE
Vert. = 1:100
Hor. = 1:300

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

c:\acad\20017S\LOTBs\A-4.dwg

LEGEND OF BORING OPERATIONS; IN-SITU, LAB & FIELD TEST DESIGNATIONS; TYPES OF BORINGS; LEGEND OF EARTH MATERIALS (USCS) BASED ON ASTM D2487, D2488; CONSISTENCY CLASSIFICATION FOR SOILS According to the Standard Penetration Test (ASTM D-1586)

DESIGN OVERSIGHT	DESIGN	By: F. WANG	Field investigation by:	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	M. SALEMINIK PROJECT ENGINEER
SIGN OFF DATE	CHECKED	By: M. SALEMINIK	F. WANG		

ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 10 20 30 40 50 60 70 80 90 100

CU 04277
EA 284700

BRIDGE NO.
KILOMETER POST

MARE ISLAND/RTE 37
LOG OF TEST BORINGS

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY) 8/4/2002 12/20/2002
SHEET OF

PLATE A-3

As-Built Log of Test Borings

As-Built Log of Test Borings sheet is considered an information document only. As such, the State of California registration seal with signature, license number and registration certificate expiration date will not be required. This drawing is available and presented only for the convenience of any bidder, contractor or other interested party.

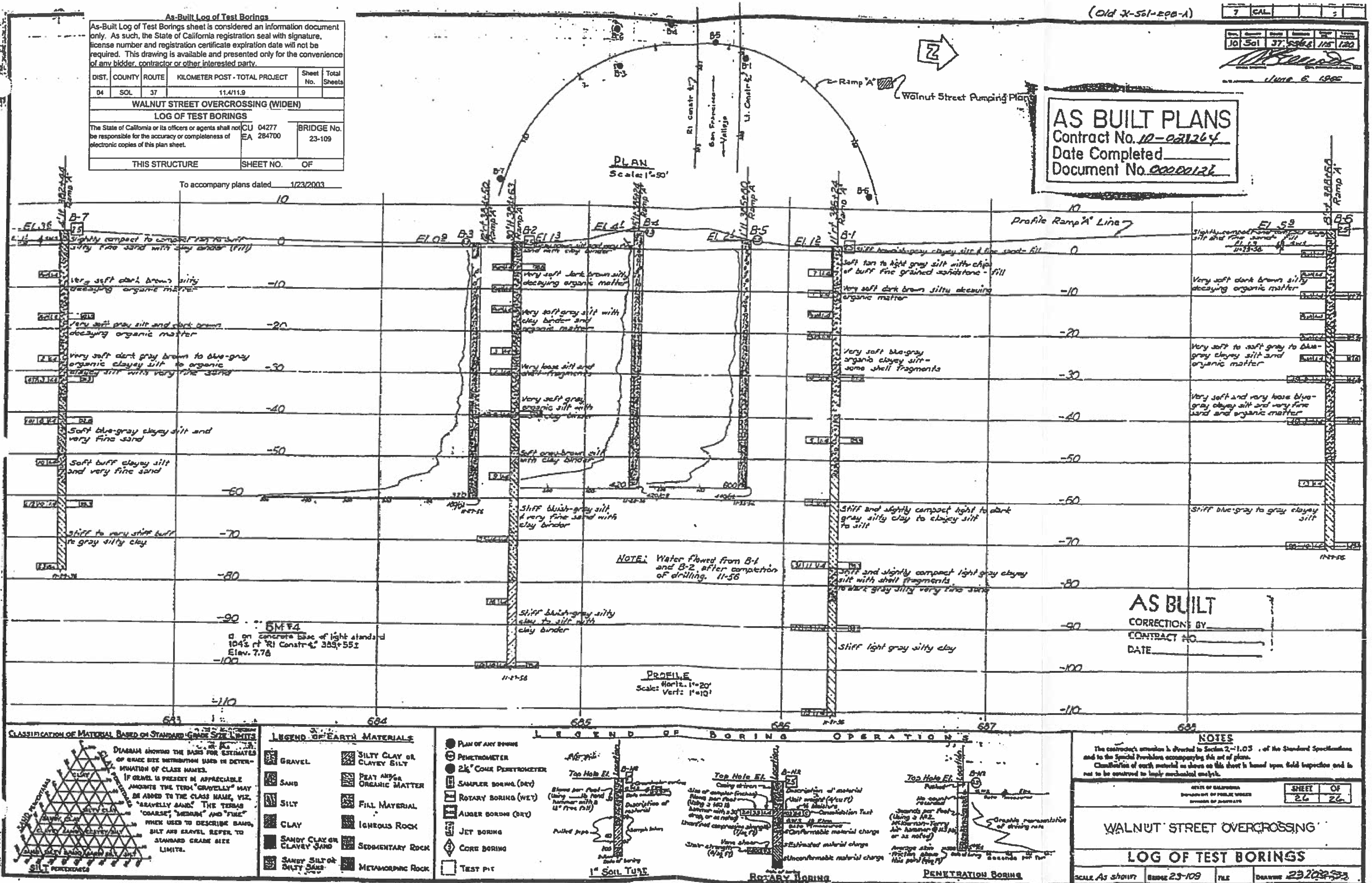
DIST.	COUNTY	ROUTE	KILOMETER POST - TOTAL PROJECT	Sheet No.	Total Sheets
04	SOL	37	11.4/11.9		

WALNUT STREET OVERCROSSING (WIDEN)

LOG OF TEST BORINGS

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.	CU 04277 EA 284700	BRIDGE No. 23-109
THIS STRUCTURE	SHEET NO.	OF

To accompany plans dated 1/23/2003



APPENDIX B

APPENDIX B

LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory according to the Unified Soil Classification System. The results are presented in, Appendix A.

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils in general accordance with ASTM Test Method D 2216-92. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the "Log of Borings", Appendix A.

Unconfined Compression Tests

Strength tests were performed on selected undisturbed sample using unconfined compression machine. Unconfined compression test was performed in general accordance with ASTM Test Method D 2166-91. The results are presented on "Log of test borings", Appendix A.

Consolidation

Consolidation tests were performed on selected samples in accordance with California Test Method 219. Test results are presented on Plates B-2.

R-value Tests

R-value tests were performed on representative bulk samples for pavement design. The tests were performed by Parikh Consutltants, Inc., as per California Test Method 301. The results are presented on Plates B-3.

LABORATORY TESTS
(Continued)

Corrosion Tests

Corrosion tests were performed on selected samples to determine the corrosion potential of the soils. The pH and minimum resistively tests were performed according to California Test Method 643. Sulfate and chloride tests were performed by AnaCon Testing Laboratory. The test results for water-soluble sulfate and chloride contents are presented on Plates B-4.

CONSOLIDATION TEST (DATA)

ASTM D - 2435

(408)945-1011

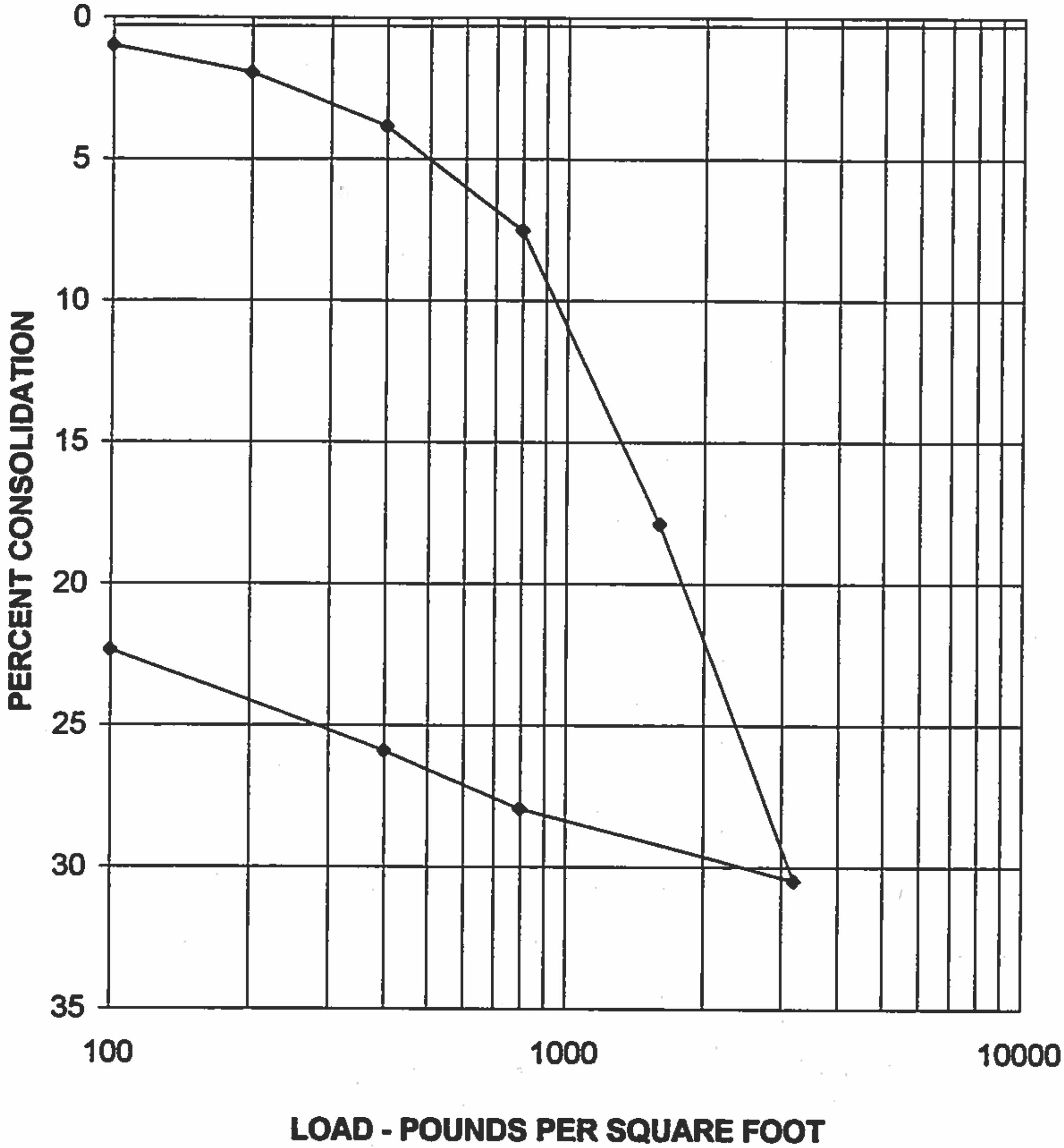
PROJECT NAME:	Mare Island Route 37			PROJECT #:	201126.BR
BORING #:	BM-1	SAMPLE #:	3	DEPTH:	10'
LAB #:	G298			DATE:	11/6/2002
MATERIAL DESCRIPTION	Organic clay, gray			TESTED BY:	PD

SQRT (Time) Min.	Deformation 10**-3 in
0.5	6.6
0.7	8.3
1	11.1
1.4	15
2	21.3
3	31.6
4	41.8
5	52
6	61.2
7	69.4
8	76.8
9	82.8
10	87.8
11	92.3
12	96
13	99.4
14	102.4
15	104.6
16	106.7
17	108.6
18	110.4
19	112
20	113.4
21	114.7
22	116
23	117.2
38	125.9

		Dial Reading
Dial Reading @ 0.0 ksf		0.0506
POUNDS PER SQUARE FOOT	% CONSOLIDATION	
100	0.99	0.0605
200	1.94	0.07
400	3.85	0.0891
800	7.53	0.1259
1600	17.89	0.2295
3200	30.48	0.3554
800	27.96	0.3302
400	25.91	0.3097
100	22.34	0.274

	BEFORE	AFTER
Wt. Wet Soil + Ring	145.69	
Wt. Of Ring	46.43	
Wt. Of Container		85.74
Wt. Of Ring + Container		132.17
Wt of Wet Soil + Ring + Cont.		215.71
Wt. Of Dry Soil + Ring + Cont.		171.48
Wt of Wet Soil	99.26	
Wt. Of Dry Soil	39.31	39.31
Wt. Of Water	59.95	44.23
Height of Sample (inch)	1	0.7766
% Moisture Content	152.5	112.5
Dry Density (pcf)	32.4	41.8

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	152.5	32.4	1.0000	2.416
FINAL	112.5	41.8	0.7766	2.416

BORING NO.	BM-1	SAMPLE NO.	3	ELEV. OR DEPTH	10'
DESCRIPTION	Organic clay, gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Mare Island Route 37 CCS PLANNING & ENGINEERING	
DATE 11/6/2002	JOB NO: 201126.BR
Reported by: Prav Dayah	

PLATE B-2B

CONSOLIDATION TEST - TIME CURVE

JOB #: 201126.BR

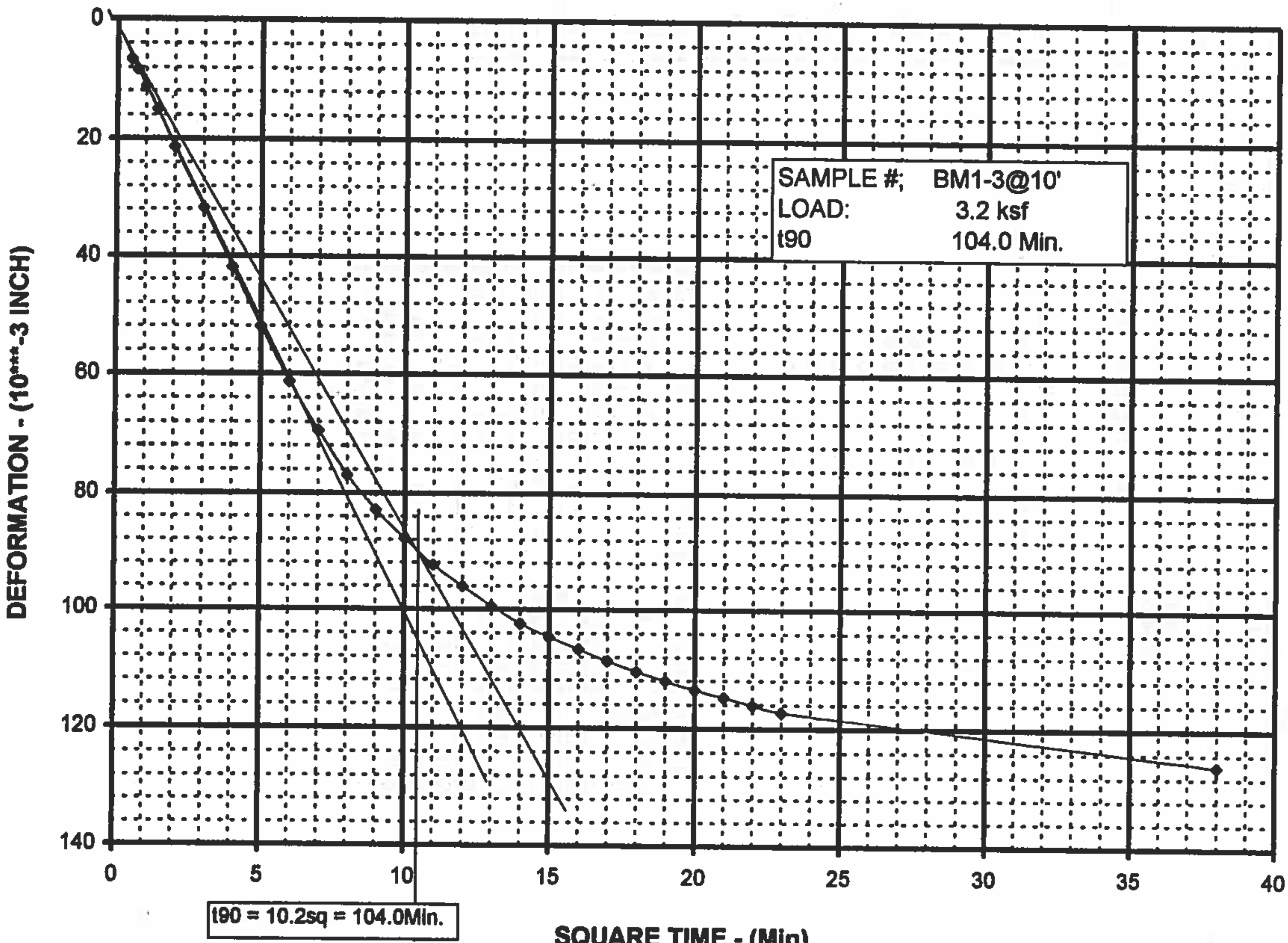


PLATE B-2C

CONSOLIDATION TEST (DATA)

ASTM D - 2435

(408)945-1011

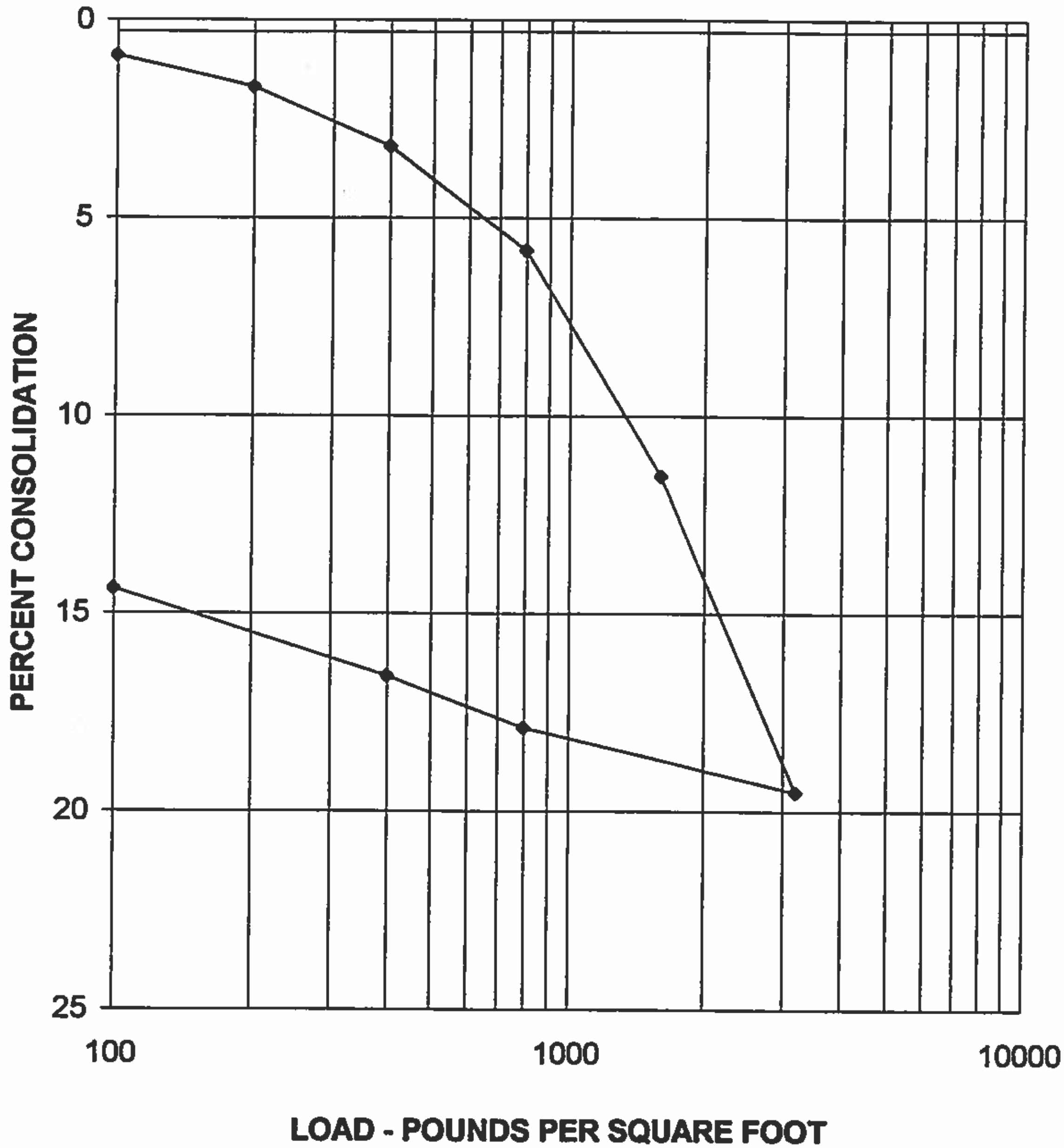
PROJECT NAME:	Mare Island Route 37			PROJECT #:	201126.BR
BORING #:	BM-1	SAMPLE #:	4	DEPTH:	15'
LAB #:	G298			DATE:	11/6/2002
MATERIAL DESCRIPTION	Organic clay, gray			TESTED BY:	PD

SQRT Time) Min.	Deformation 10**-3 in
0.5	4.1
0.7	5.6
1	7.7
1.4	10.9
2	15.5
3	23.7
4	31.5
5	38.2
6	44.2
7	49.2
8	53.4
9	56.8
10	59.5
11	61.8
12	63.7
13	65.4
14	66.8
15	68
16	69.2
17	70.4
18	71.1
19	72
20	72.8
21	73.6
22	74.3
23	75
38	79.8

		Dial Reading
Dial Reading @ 0.0 ksf		0.0461
POUNDS PER SQUARE FOOT	% CONSOLIDATION	
100	0.91	0.0552
200	1.69	0.063
400	3.19	0.078
800	5.83	0.1044
1600	11.53	0.1614
3200	19.51	0.2412
800	17.89	0.225
400	16.58	0.2119
100	14.38	0.1899

	BEFORE	AFTER
Wt. Wet Soil + Ring	159.1	
Wt. Of Ring	45.57	
Wt. Of Container		83.62
Wt. Of Ring + Container		129.19
Wt of Wet Soil + Ring + Cont.		232.74
Wt. Of Dry Soil + Ring + Cont.		189.42
Wt of Wet Soil	113.53	
Wt. Of Dry Soil	60.23	60.23
Wt. Of Water	53.3	43.32
Height of Sample (inch)	1	0.8562
% Moisture Content	88.5	71.9
Dry Density (pcf)	49.7	58.0

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	88.5	49.7	1.0000	2.416
FINAL	71.9	58.0	0.8562	2.416

BORING NO.	BM-1	SAMPLE NO.	4	ELEV. OR DEPTH	15'
DESCRIPTION	Organic clay, gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Mare Island Route 37 CCS PLANNING & ENGINEERING	
DATE 11/6/2002	JOB NO: 201126.BR
Reported by: Prav Dayah	

PLATE B-2E

CONSOLIDATION TEST - TIME CURVE

JOB #: 201126.BR

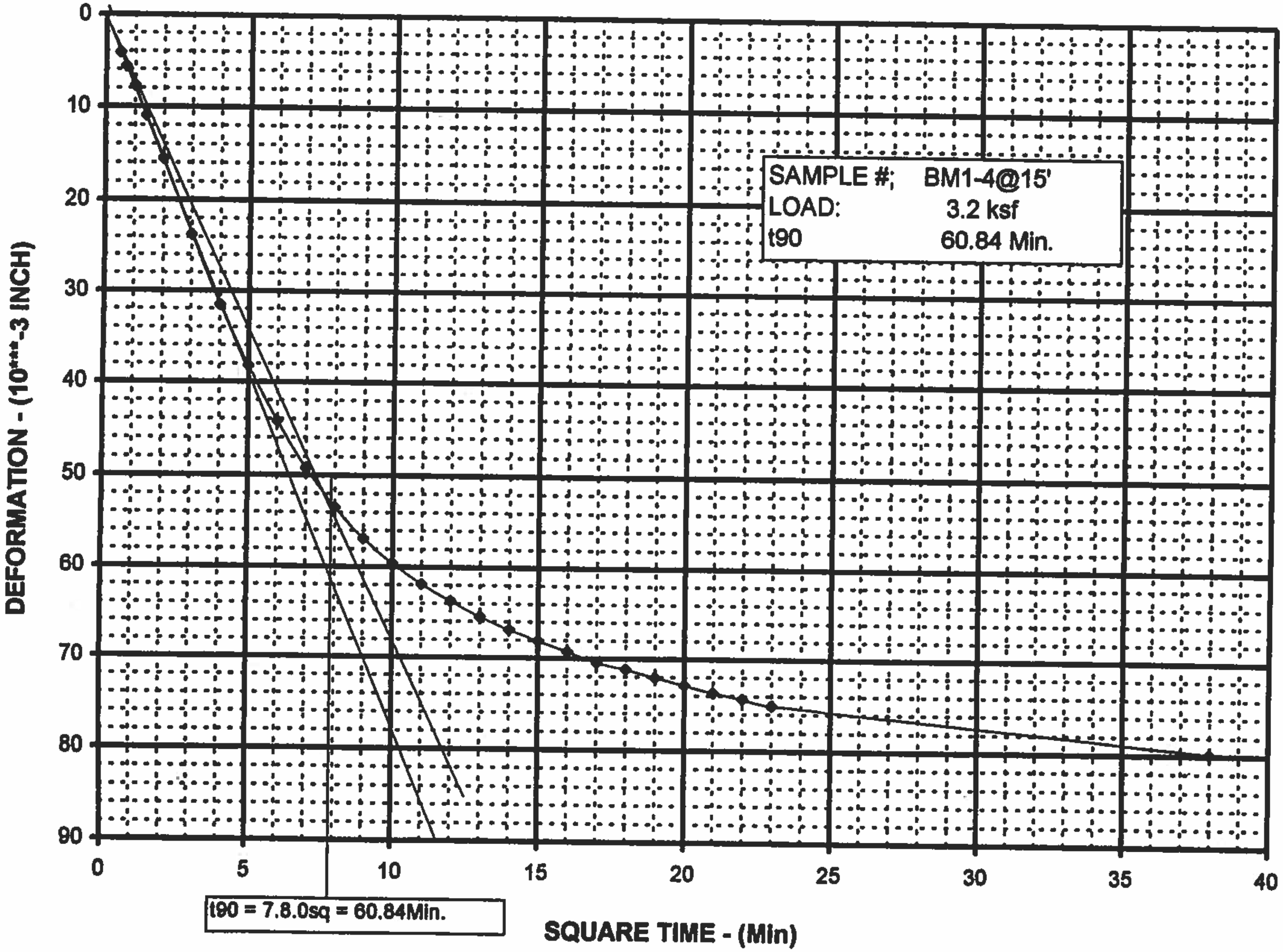


PLATE B-2F

CONSOLIDATION TEST (DATA)
ASTM D - 2435

(408)945-1011

PROJECT NAME: Mare Island Route 37		PROJECT #:	201126.BR
BORING #: BM-2	SAMPLE #: 4	DEPTH:	15'
LAB #: G298		DATE:	11/4/2002
MATERIAL DESCRIPTION	Organic clay, gray	TESTED BY:	PD

SQRT (Time) Min.	Deformation 10**-3 in
0.5	5
0.7	6.5
1	8.8
1.4	11.8
2	16.2
3	24
4	31.9
5	39.1
6	45
7	50
8	54.2
9	57.8
10	60.7
11	63
12	65
13	66.8
14	68.3
15	69.8
16	71
17	72.2
18	73.4
19	74.4
20	75.5
21	76.4
22	77.4
23	78.3
38	84.3

Dial Reading @ 0.0 ksf — Dial Reading 0.075

POUNDS PER SQUARE FOOT	% CONSOLIDATION	Dial Reading
100	0.96	0.0846
200	1.64	0.0914
400	3.07	0.1057
800	5.21	0.1271
1600	9.7	0.172
3200	18.13	0.2563
6400	27.26	0.3476
1600	25.35	0.3285
400	22.08	0.2958
100	19.47	0.2697

	BEFORE	AFTER
Wt. Wet Soil + Ring	157.16	
Wt. Of Ring	46.43	
Wt. Of Container		84.07
Wt. Of Ring + Container		130.50
Wt of Wet Soil + Ring + Cont.		227.61
Wt. Of Dry Soil + Ring + Cont.		185.85
Wt of Wet Soil	110.73	
Wt. Of Dry Soil	55.35	55.35
Wt. Of Water	55.38	41.76
Height of Sample (inch)	1	0.8053
% Moisture Content	100.1	75.4
Dry Density (pcf)	45.7	56.7

CONSOLIDATION TEST - TIME CURVE

JOB #: 201126.BR

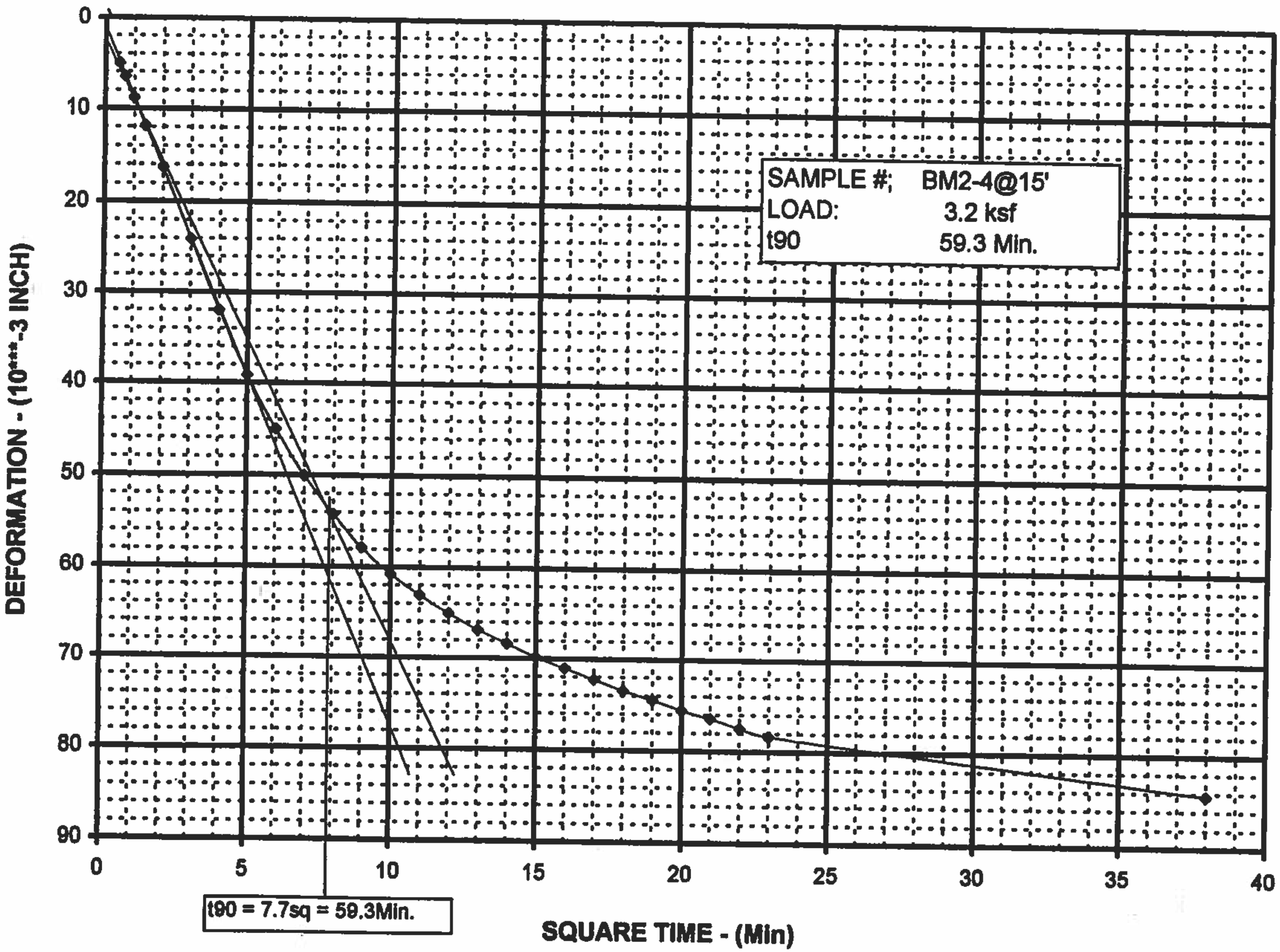
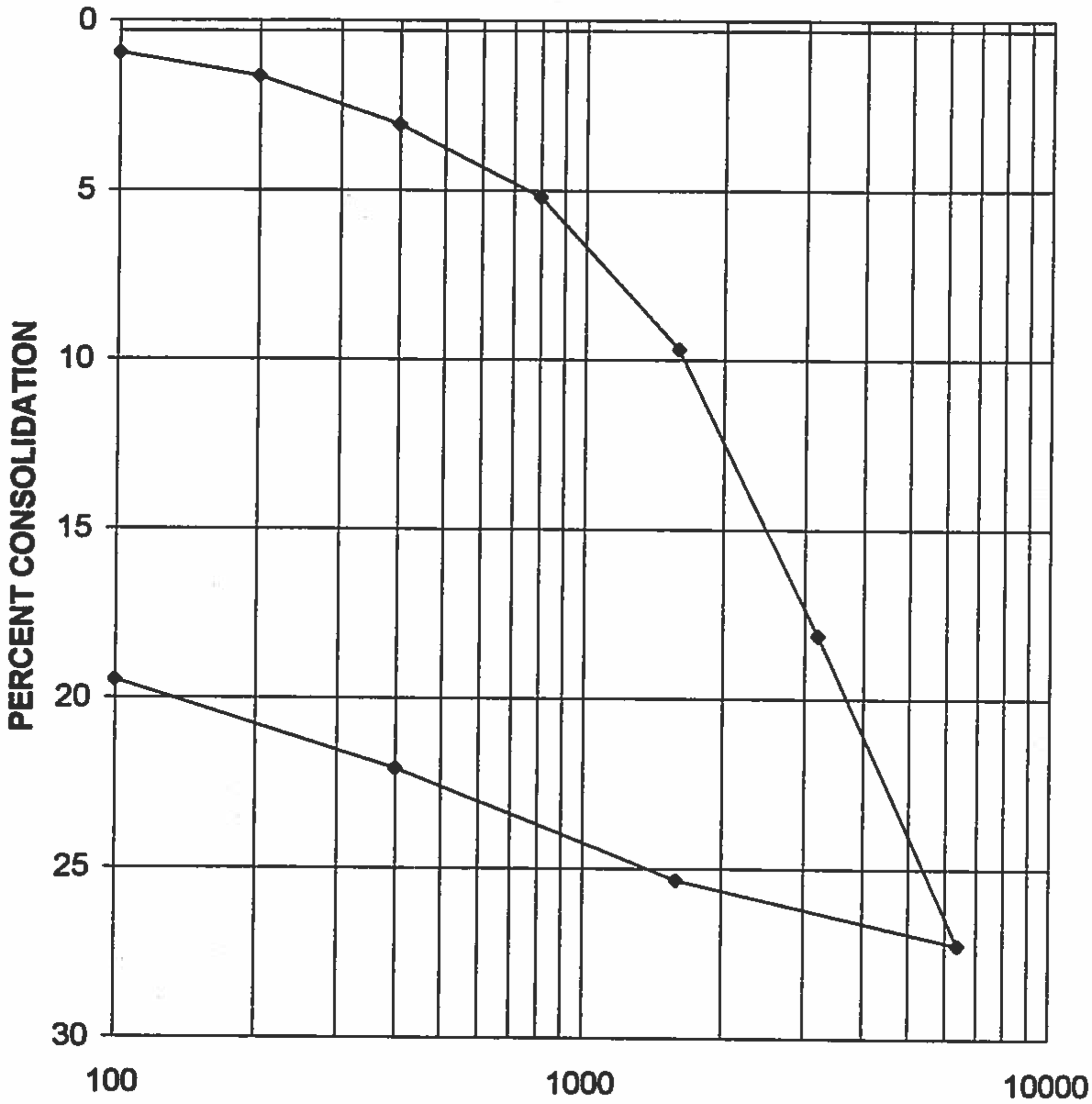


PLATE B-2H

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	100.1	45.7	1.0000	2.416
FINAL	75.4	56.7	0.8053	2.416

BORING NO.	BM-2	SAMPLE NO.	4	ELEV. OR DEPTH	15'
DESCRIPTION	Organic clay, gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

Mare Island Route 37 CCS PLANNING & ENGINEERING	
DATE 11/4/2002	JOB NO: 201126.BR
Reported by: Prav Dayah	

PLATE B-2I

CONSOLIDATION TEST (DATA)
ASTM D - 2435

(408)945-1011

PROJECT NAME:	Mare Island Route 37			PROJECT #:	201126.BR
BORING #:	BM-2	SAMPLE #:	5	DEPTH:	20'
LAB #:	G298			DATE:	11/4/2002
MATERIAL DESCRIPTION	Organic clay, gray			TESTED BY:	PD

SQRT (Time) Min.	Deformation 10**-3 in
0.5	4.2
0.7	5.6
1	7.7
1.4	10.9
2	15.3
3	23
4	30.9
5	38.1
6	44.4
7	49.3
8	54.1
9	57.9
10	61
11	63.7
12	65.9
13	67.9
14	69.7
15	71.2
16	72.7
17	74
18	75.4
19	76.6
20	77.7
21	78.7
22	79.7
23	80.7
38	86.9

	POUNDS PER SQUARE FOOT	% CONSOLIDATION	Dial Reading
Dial Reading @ 0.0 ksf			0.0496
	100	1.13	0.0609
	200	2	0.0696
	400	3.65	0.0861
	800	5.65	0.1061
	1600	10.28	0.1524
	3200	18.97	0.2393
	6400	28.07	0.3303
	1600	26.42	0.3138
	400	23.64	0.286
	100	21.35	0.2631

	BEFORE	AFTER
Wt. Wet Soil + Ring	158.99	
Wt. Of Ring	45.64	
Wt. Of Container		84.88
Wt. Of Ring + Container		130.52
Wt of Wet Soil + Ring + Cont.		229.69
Wt. Of Dry Soil + Ring + Cont.		189.62
Wt of Wet Soil	113.35	
Wt. Of Dry Soil	59.1	59.1
Wt. Of Water	54.25	40.07
Height of Sample (inch)	1	0.7865
% Moisture Content	91.8	67.8
Dry Density (pcf)	48.8	62.0

CONSOLIDATION TEST - TIME CURVE

JOB #: 201126.BR

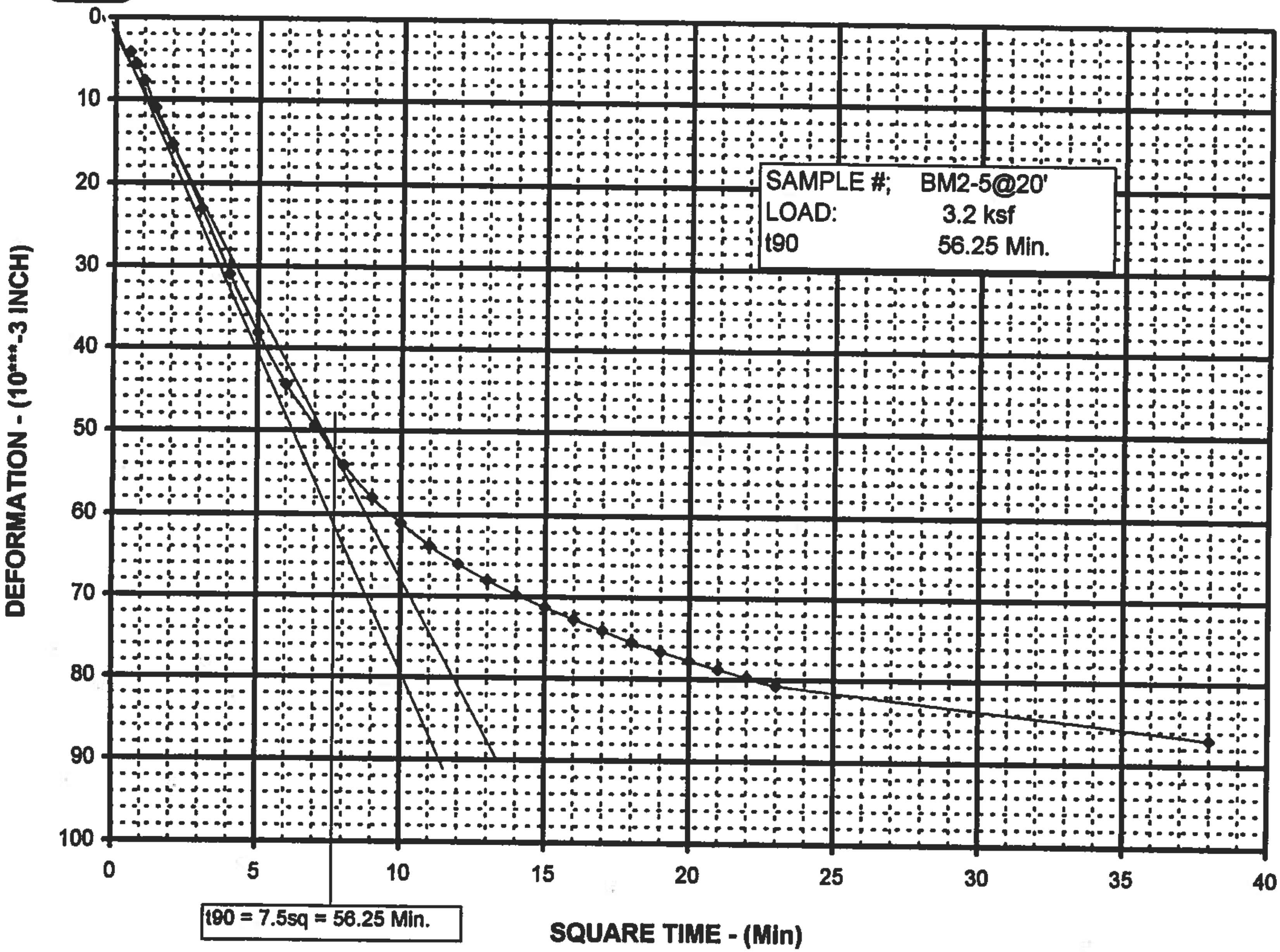
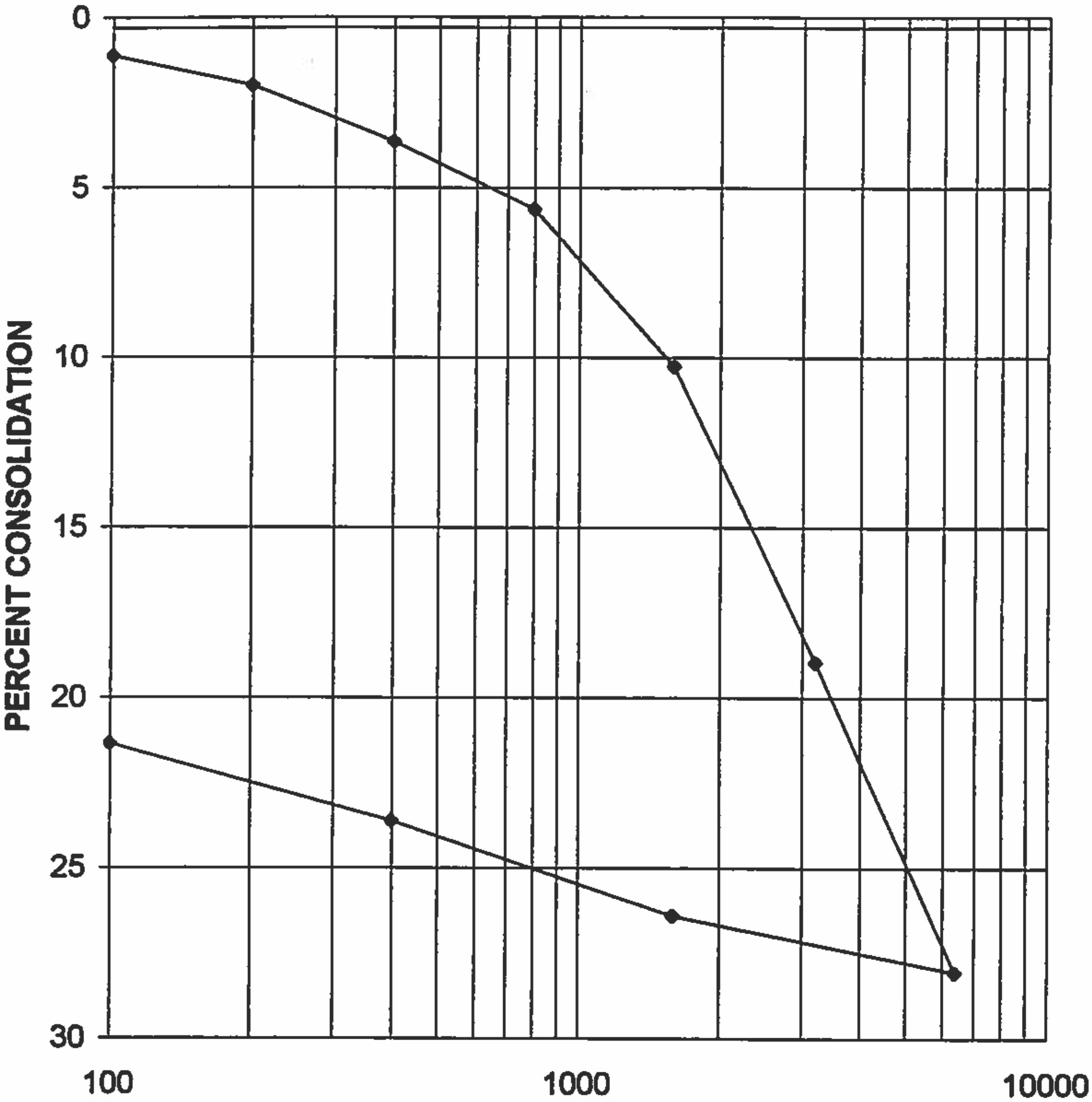


PLATE B-2K

CONSOLIDATION TEST RESULTS



	MOISTURE CONTENT %	DRY DENSITY PCF	HEIGHT (INCHES)	DIAMETER (INCHES)
INITIAL	91.8	48.8	1.0000	2.416
FINAL	67.8	62.0	0.7865	2.416

BORING NO.	BM-2	SAMPLE NO.	5	ELEV. OR DEPTH	20'
DESCRIPTION	Organic clay, gray				

PARIKH CONSULTANTS, INC.
GEOTECHNICAL CONSULTANTS
MATERIALS ENGINEERING

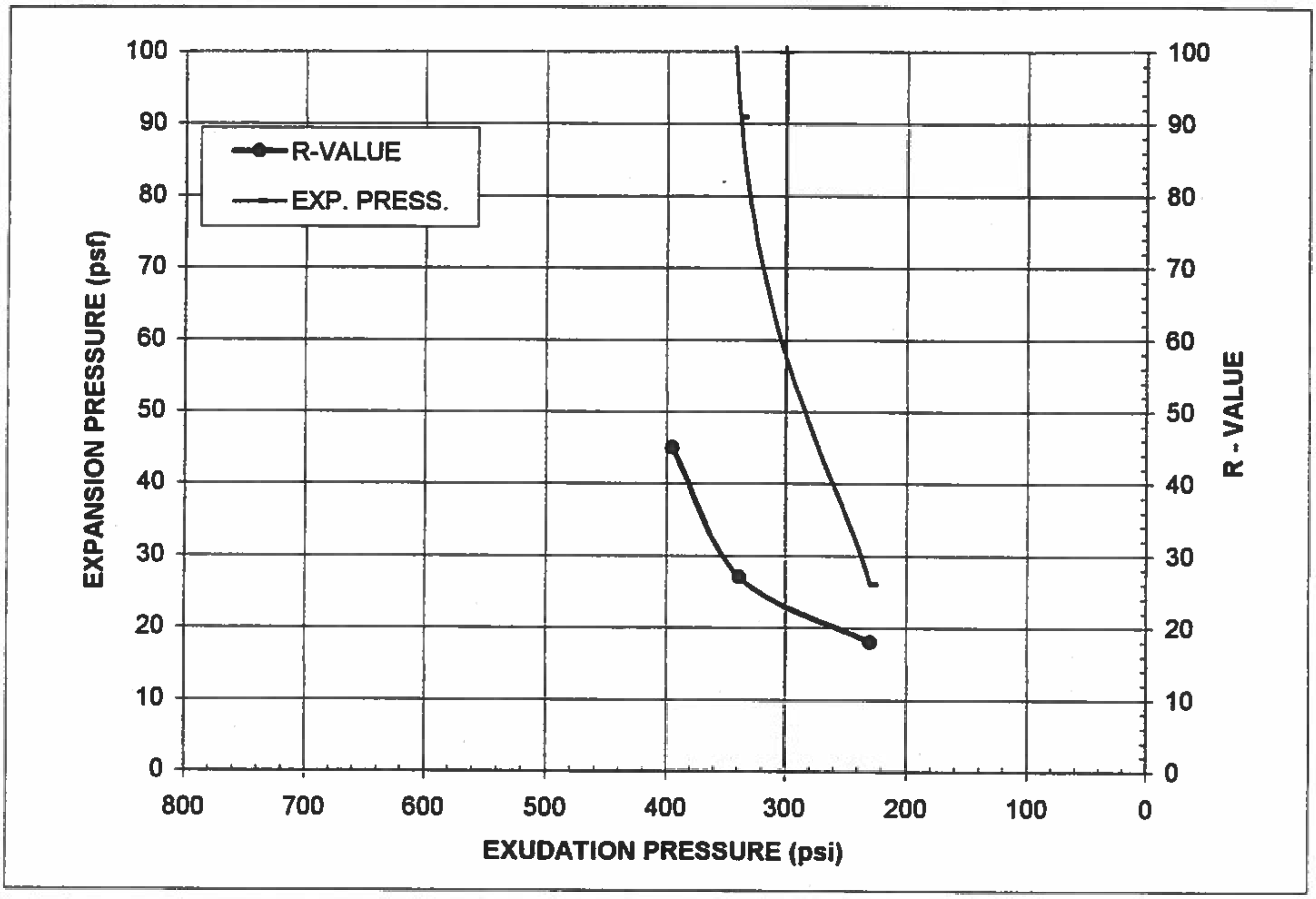
Mare Island Route 37 CCS PLANNING & ENGINEERING	
DATE 11/4/2002	JOB NO: 201126.BR
Reported by: Prav Dayah	

PLATE B-2L

R-VALUE REPORT

Parikh Consultants, Inc. | ASTM D2844 or CTM 301 | (408) 945-1011

Project Name:	Mare Island, Rte 37	Date:	05/21/02
Client:	CCS Planning & Engineering	Project #:	201126.GDR
Sample #: R-2	Depth: 0' - 5'	Lab #:	M311
Location / Source:	Native / Sta 417+75	Sample Date:	
Material :	Sandy clay with gravel, dark brown	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	229	338	395
Expansion Pressure, psf	26	91	355
R-Value	18	27	45
Moisture Content at Test, %	29.2	27.1	25.0
Dry Density at Test, pcf	88.6	91.3	93.9

R-Value @ 300 psi Exudation Pressure =	**24**	Expansion Pressure @300 psi Exudation, psf =	58
Minimum R-Value Requirement:			

Comments:

Report By: Prav Dayah

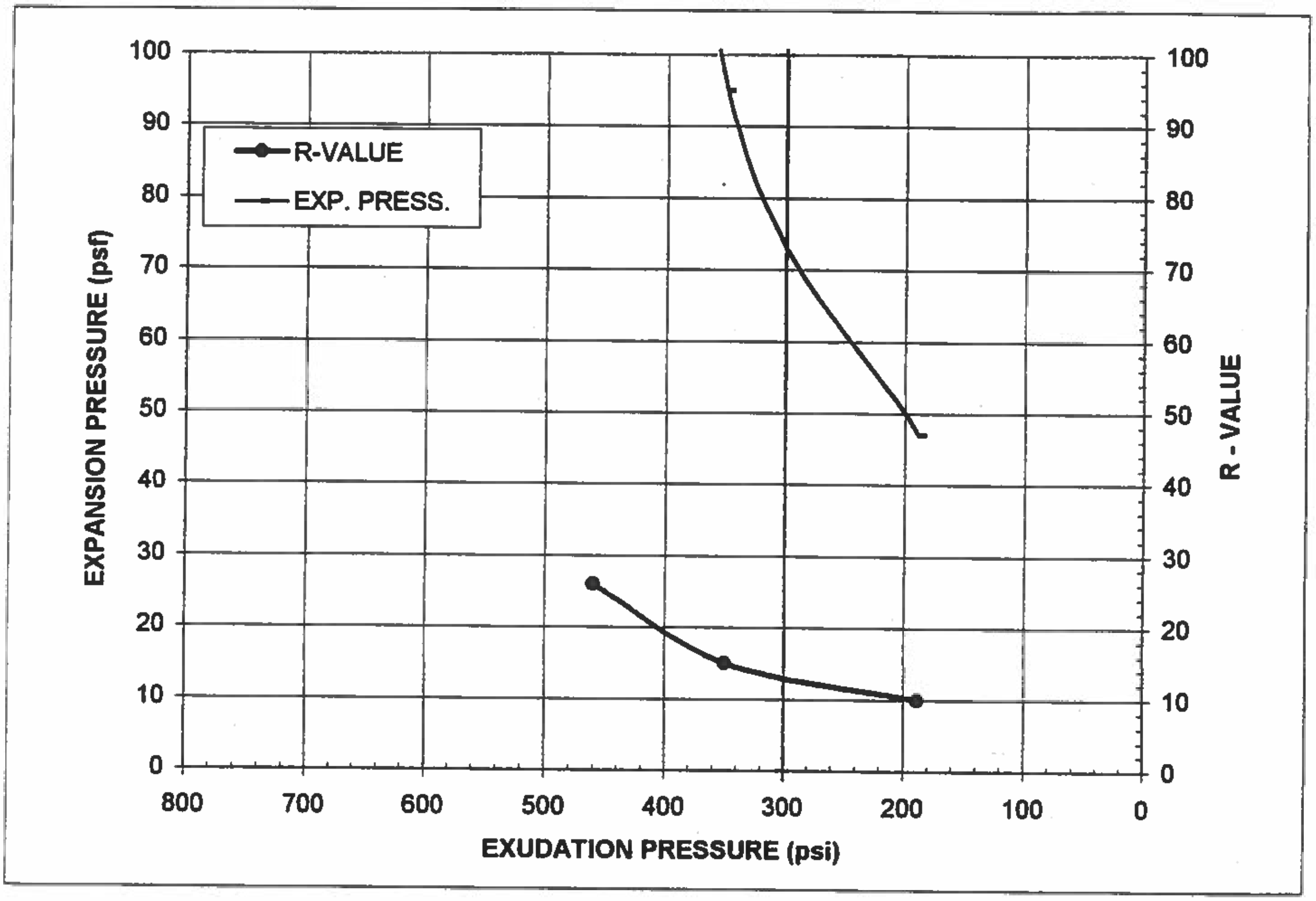
RVALUE with calcs pdp

PLATE B-3A

R-VALUE REPORT

Parikh Consultants, Inc. | ASTM D2844 or CTM 301 | (408) 945-1011

Project Name:	Mare Island, Rte 37	Date:	05/21/02
Client:	CCS Planning & Engineering	Project #:	201126.GDR
Sample #:	R-4 Depth: 0' - 5'	Lab #:	M311
Location / Source:	Native / Sta 418+80	Sample Date:	
Material :	Sandy clay, brown	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	188	350	460
Expansion Pressure, psf	47	95	242
R-Value	10	15	26
Moisture Content at Test, %	17.6	15.7	13.9
Dry Density at Test, pcf	104.5	108.9	112.0

R-Value @ 300 psi Exudation Pressure =	13	Expansion Pressure @300 psi Exudation, psf =	72
Minimum R-Value Requirement:			

Comments:

Report By: Prav Dayah

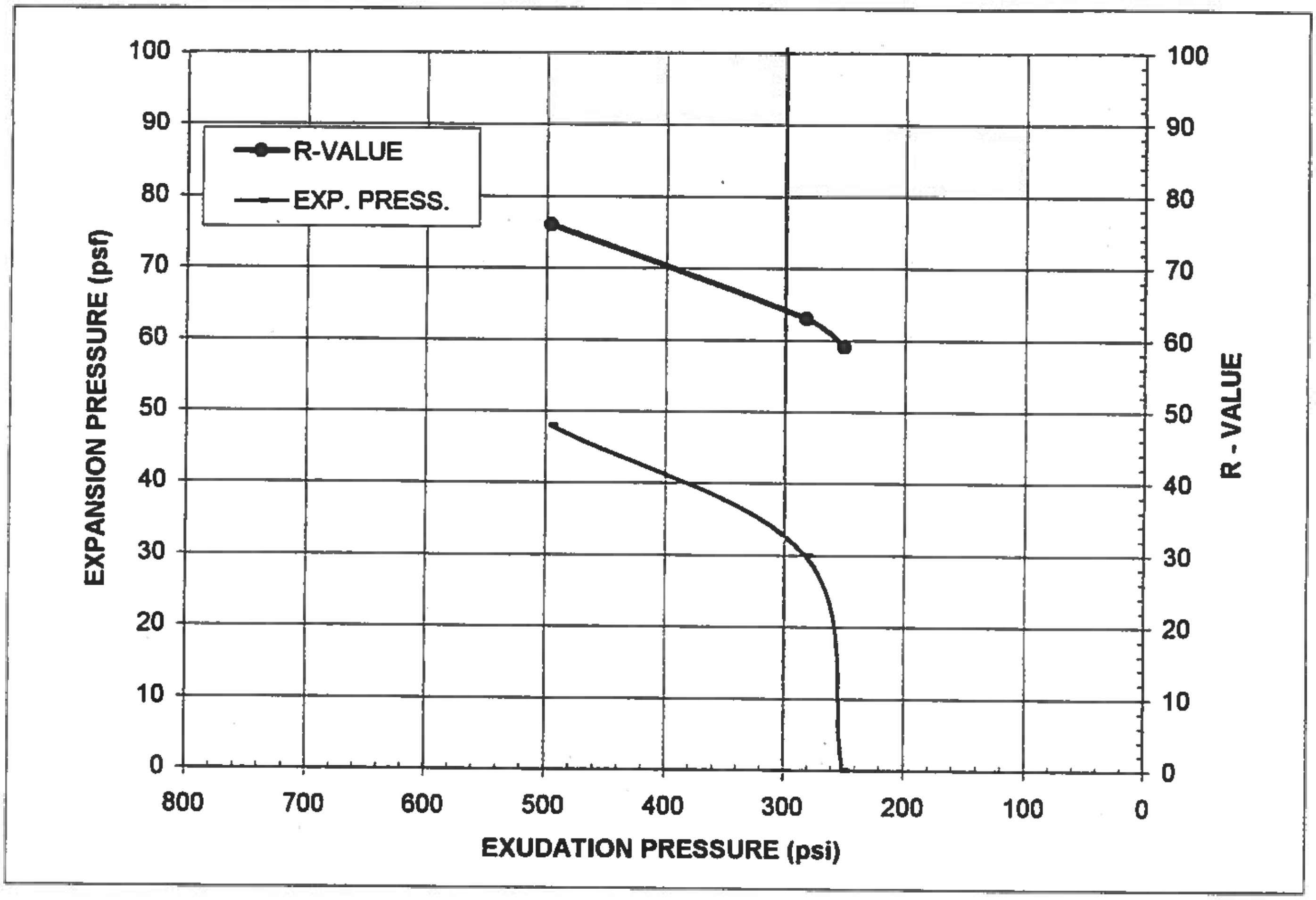
RVALUE with calcs pdp

PLATE B-3B

R-VALUE REPORT

Parikh Consultants, Inc. ASTM D2844 or CTM 301 (408) 945-1011

Project Name:	Mare Island, Rte 37			Date:	05/21/02
Client:	CCS Planning & Engineering			Project #:	201126.GDR
Sample #:	R-6	Depth:	0' - 5'	Lab #:	M311
Location / Source:		Native / Sta 316+86		Sample Date:	
Material :	Silty gravel, yellowish brown			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	250	282	496
Expansion Pressure, psf	0	30	48
R-Value	59	63	76
Moisture Content at Test, %	12.9	12.5	12.0
Dry Density at Test, pcf	118.3	119.0	121.5

R-Value @ 300 psi Exudation Pressure = 64

Expansion Pressure @300 psi Exudation, psf = 34

Minimum R-Value Requirement:

Comments:

Report By: Prav Dayah

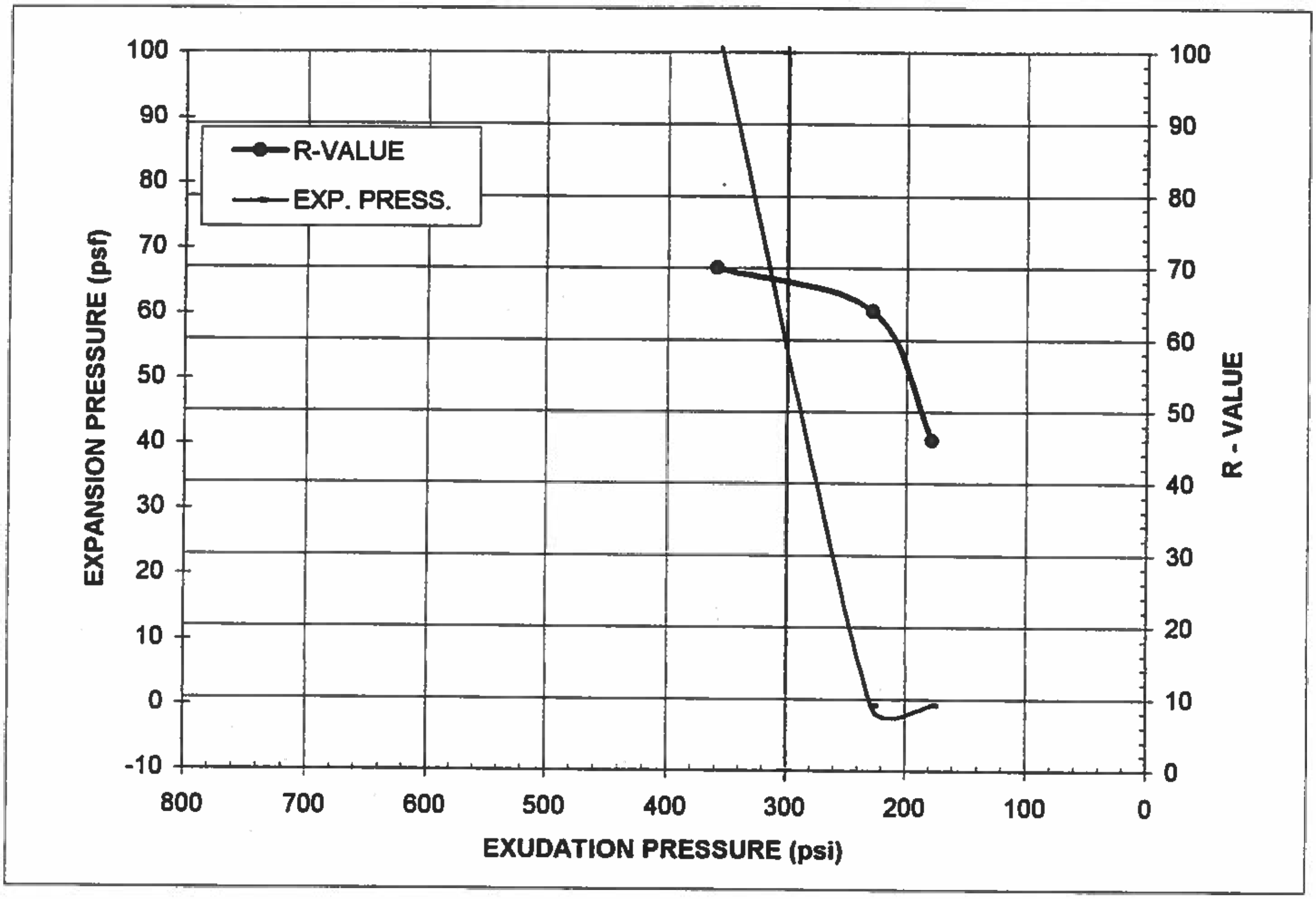
RVALUE with calcs pdp

PLATE B-3C

R-VALUE REPORT

Parikh Consultants, Inc. ASTM D2844 or CTM 301 (408) 945-1011

Project Name:	Mare Island, Rte 37			Date:	05/21/02
Client:	CCS Planning & Engineering			Project #:	201126.GDR
Sample #:	R-7	Depth:	0' - 5'	Lab #:	M311
Location / Source:		Native / Sta 318+60		Sample Date:	
Material :	Clayey gravel, brown			Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	178	228	359
Expansion Pressure, psf	0	0	103
R-Value	46	64	70
Moisture Content at Test, %	12.6	11.4	10.6
Dry Density at Test, pcf	117.5	119.6	120.6

R-Value @ 300 psi Exudation Pressure = 68

Expansion Pressure @300 psi Exudation, psf = **54**

Minimum R-Value Requirement:

Comments:

eport By: Prav Dayah

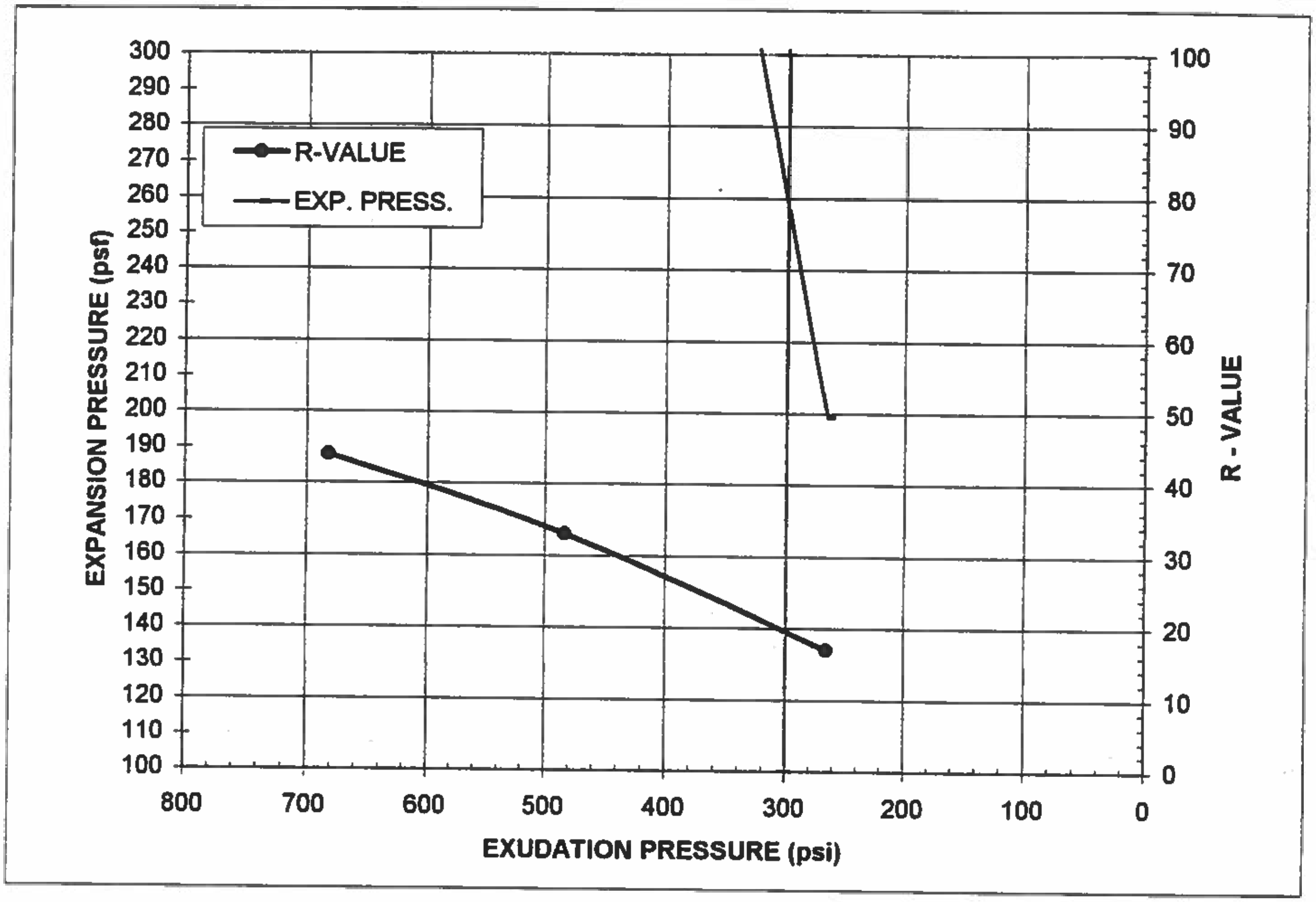
RVALUE with calcs pdp

PLATE B-3D

R-VALUE REPORT

Parikh Consultants, Inc. ASTM D2844 or CTM 301 (408) 945-1011

Project Name:	Mare Island, Rte 37	Date:	05/21/02
Client:	CCS Planning & Engineering	Project #:	201126.GDR
Sample #:	R-8 Depth: 0' - 5'	Lab #:	M311
Location / Source:	Native / Sta 319+73	Sample Date:	
Material :	Sandy clay with gravel, light brown	Sampled By:	



Specimen No.	A	B	C
Exudation Pressure, psi	265	483	682
Expansion Pressure, psf	199	567	818
R-Value	17	33	44
Moisture Content at Test, %	17.7	15.8	14.9
Dry Density at Test, pcf	113.6	117.5	120.2

R-Value @ 300 psi Exudation Pressure = 20

Expansion Pressure @300 psi Exudation, psf = **260**

Minimum R-Value Requirement:

Comments:

Report By: Prav Dayah

RVALUE with calcs pdp

PLATE B-3E

AnaCon Testing Laboratories, Inc.

415 Fairchild Drive
Telephone: (650) 335-1233

Mountain View, California 94043
Facsimile: (650) 335-1076

June 12, 2002/ ld

Parikh Consultants, Inc.
356 South Milpitas Boulevard
Milpitas, California 95035

ATL No.: 0036.01
Lab No.: 42513.1.6

Attention: Prav Dayah

Service: **CHLORIDE & SULFATE TESTS**

Project No.: 201126.GDR (Mare Island, Route 37)

Composites: R-1
R-7
R-8

Date Received: June 11, 2002

Sample Identification:	Water Soluble Chloride* Found	Water Soluble Sulfate** Found
R-1	13.5	25.9
R-7	4.5	14.4
R-8	3.0	11.5

*Water Soluble Chloride, mg Cl/Kg Soil: Requirement - 500 max. Calif. Test Method 422

**Water Soluble Sulfate, mg SO_4/Kg Soil: Requirement - 2000 max. Calif. Test Method 417

Respectfully submitted,
AnaCon Testing Laboratories, Inc.

Louis Davis
Chemistry Laboratory

Sample No.	Depth (m)	USCS *	Grain Size Analysis			Atterberg Limits			Strength Parameters	Dry Density	Moisture Content	Remarks
			gravel %	sand %	fines %	LL %	PL %	PI %	Unconf. Compression (kPa)	(kN/m^3)	%	
BM 1-1	0.6	CL								17.0	18	
BM 1-2	1.2	CH								15.3	26	
BM 1-3	3.0	SC								5.1	153	C
BM 1-4	4.6	ML								7.8	89	C
BM 1-5	6.1	OH							34.8	3.9	197	U.C. - See Plate
BM 1-6	7.6	OH								6.2	114	
BM 1-7	9.1	ML								10.9	52	
BM 1-8	10.7	ML								-	30	
BM 1-9	12.2	OH								5.4	138	
BM 1-10	13.7	CH								13.8	34	
BM 1-11	15.2	CH								13.8	34	
BM 1-12	16.8	CH								11.7	48	
BM 1-13	19.8	CH								14.1	34	
BM 1-14	22.9	SM								12.6	41	

SUMMARY OF LABORATORY TEST RESULTS
MARE ISLAND/ ROUTE 37
MARE ISLAND, CALIFORNIA

PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

Notes:
* Unified Soil Classification System
C. = Consolidation   D.S. = Direct Shear
G. = Gradation   P.I. = Plasticity Index
R. = R-Value   U.C. = Unconf. Compression
Corr. = Corrosion

Date: 12/06/02   Job No.: 201126.GDR

Plate B-5A

Sample No.	Depth (m)	USCS *	Grain Size Analysis gravel %	sand %	fines %	Atterberg Limits LL %	PL %	PI %	Strength Parameters Unconf. Compression (kPa)	Dry Density (kN/m^3)	Moisture Content %	Remarks
BM 2-1	0.6	CL								17.1	14	
BM 2-2	1.8	SM								15.8	12	
BM 2-3	3.0	CL								19.5	94	
BM 2-4	4.6	OH								7.2	100	C
BM 2-5	6.1	OH								7.7	92	C
BM 2-6	7.6	OH							70.5	11.8	34	U.C. - See Plate
BM 2-7	9.1	OH								6.1	122	
BM 2-8	10.7	CL								12.4	41	
BM 2-9	12.2	OH								6.6	109	
BM 2-10	13.7	OH								9.1	69	
BM 2-11	15.2	CH								15.6	26	
BM 2-12	16.8	CL								13.5	36	
BM 2-13	19.8	CL								16.1	24	
BM 2-14	22.9	CL								12.1	44	
R-1	1.5	CL										Corr.
R-2	1.5	CL										R
R-4	1.5	CL										R
R-6	1.5	CL										R
R-7	1.5	CL										R, Corr.
R-8	1.5	CL										R, Corr.

SUMMARY OF LABORATORY TEST RESULTS
MARE ISLAND/ROUTE 37
MARE ISLAND, CALIFORNIA

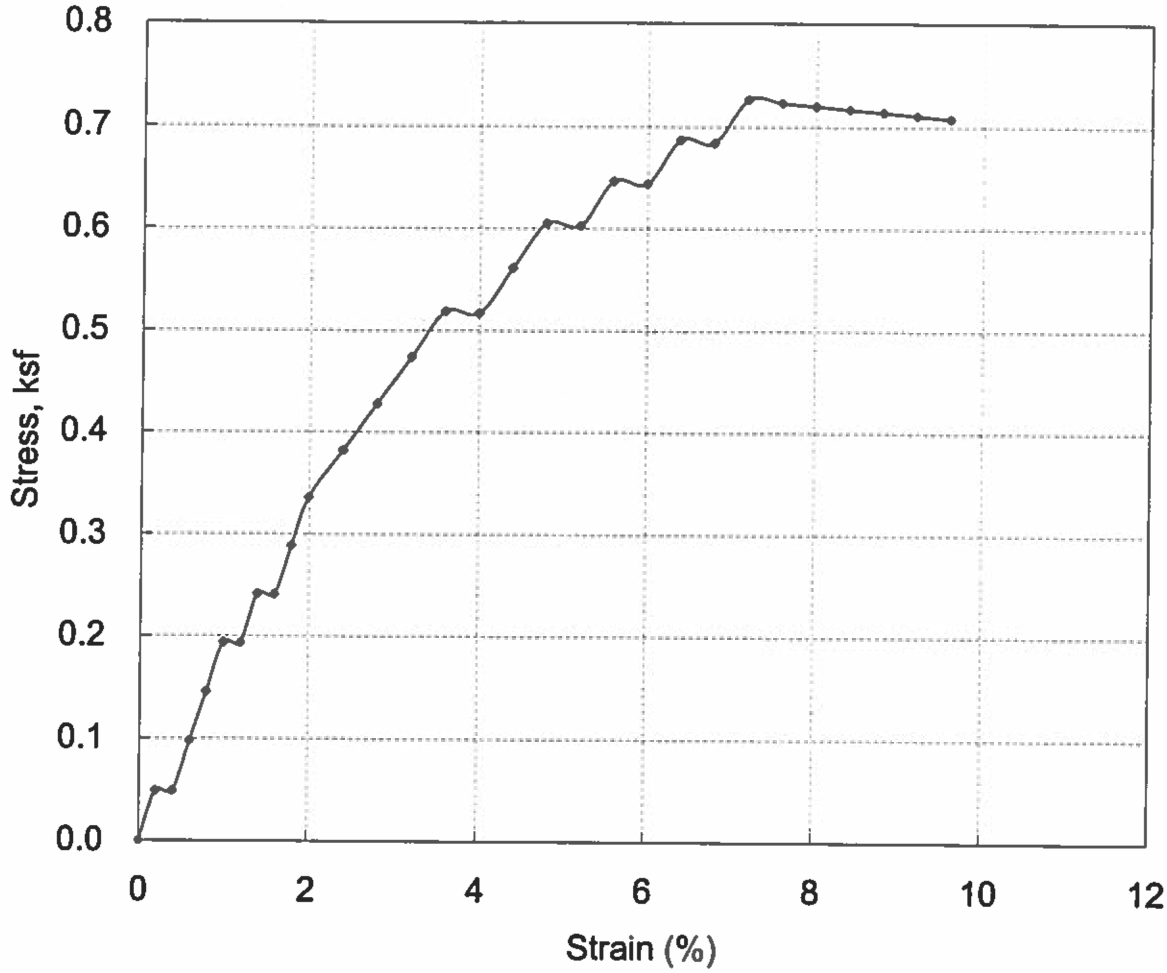
PARIKH CONSULTANTS, INC.
Geotechnical & Materials Engineering

Notes:
* Unified Soil Classification System

C. = Consolidation
D.S. = Direct Shear
G. = Gradation
P.I. = Plasticity Index
R. = R-Value
U.C. = Unconf. Compression
Corr. = Corrosion

Date: 12/06/02
Job No.: 201126.GDR

UNCONFINED COMPRESSION TEST



Boring No.: BM-1

Sample No. : MC-5

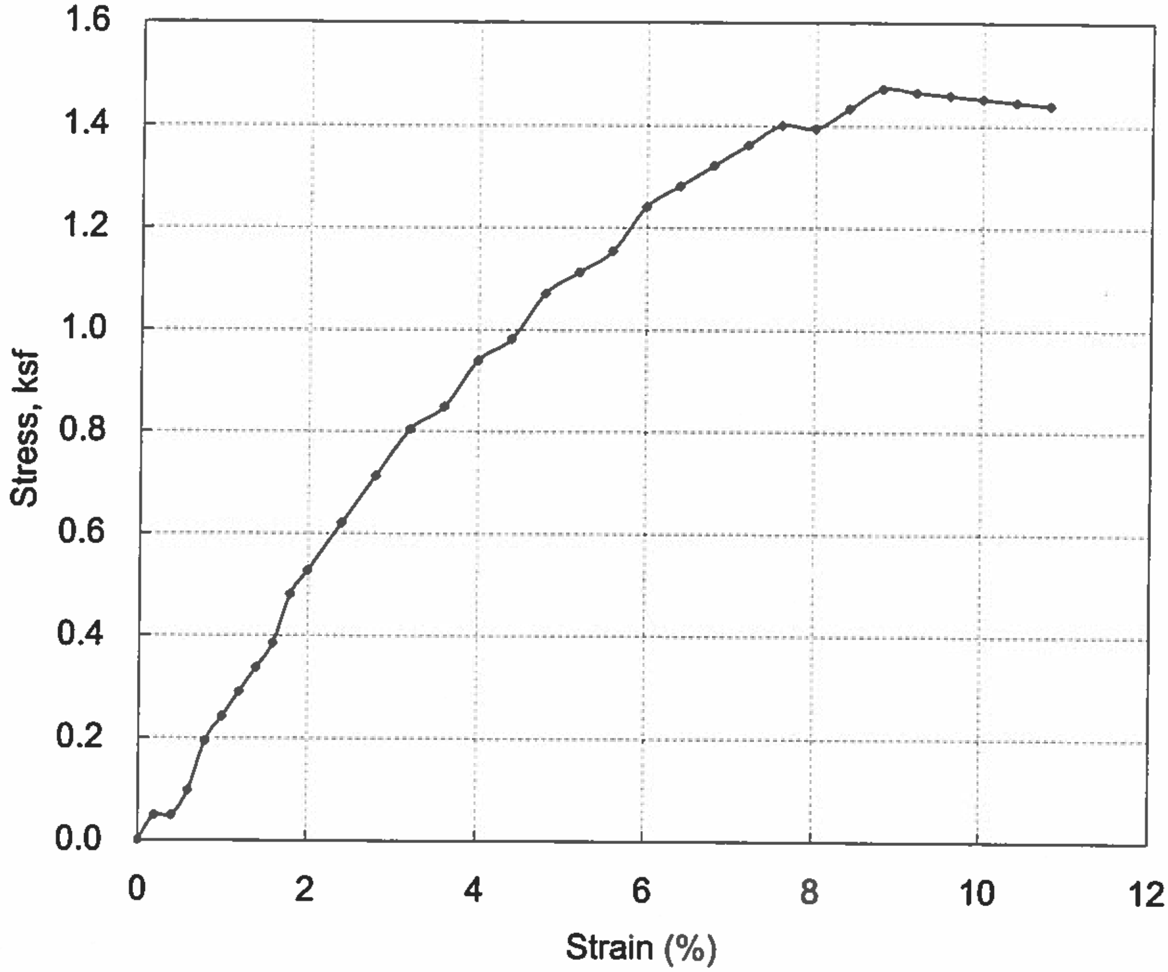
Depth (m): 6.1

Material Description:
ORGANIC SILT, very soft, black

Test Results
Stress of 0.7 ksf at 7.2% strain

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	MARE ISLAND/ROUTE 37 SOLANO COUNTY, CALIFORNIA	
	JOB NO.: 201126.10	PLATE NO.: 6A

UNCONFINED COMPRESSION TEST



Boring No.: BM-2

Sample No. : MC-6

Depth (m): 7.6

Material Description:
Peat, very soft, dark brown

Test Results
Stress of 1.5 ksf at 8.8% strain

PARIKH CONSULTANTS, INC. GEOTECHNICAL CONSULTANTS MATERIALS TESTING	MARE ISLAND/ROUTE 37 SOLANO COUNTY, CALIFORNIA
JOB NO.: 201126.10	PLATE NO.: 6B

APPENDIX C

RTE 37, Sta 120+60, "S3-M", Static Condition

Ten Most Critical. D:MARE_S2.PLT By: M.S. 03-13-03 11:52am

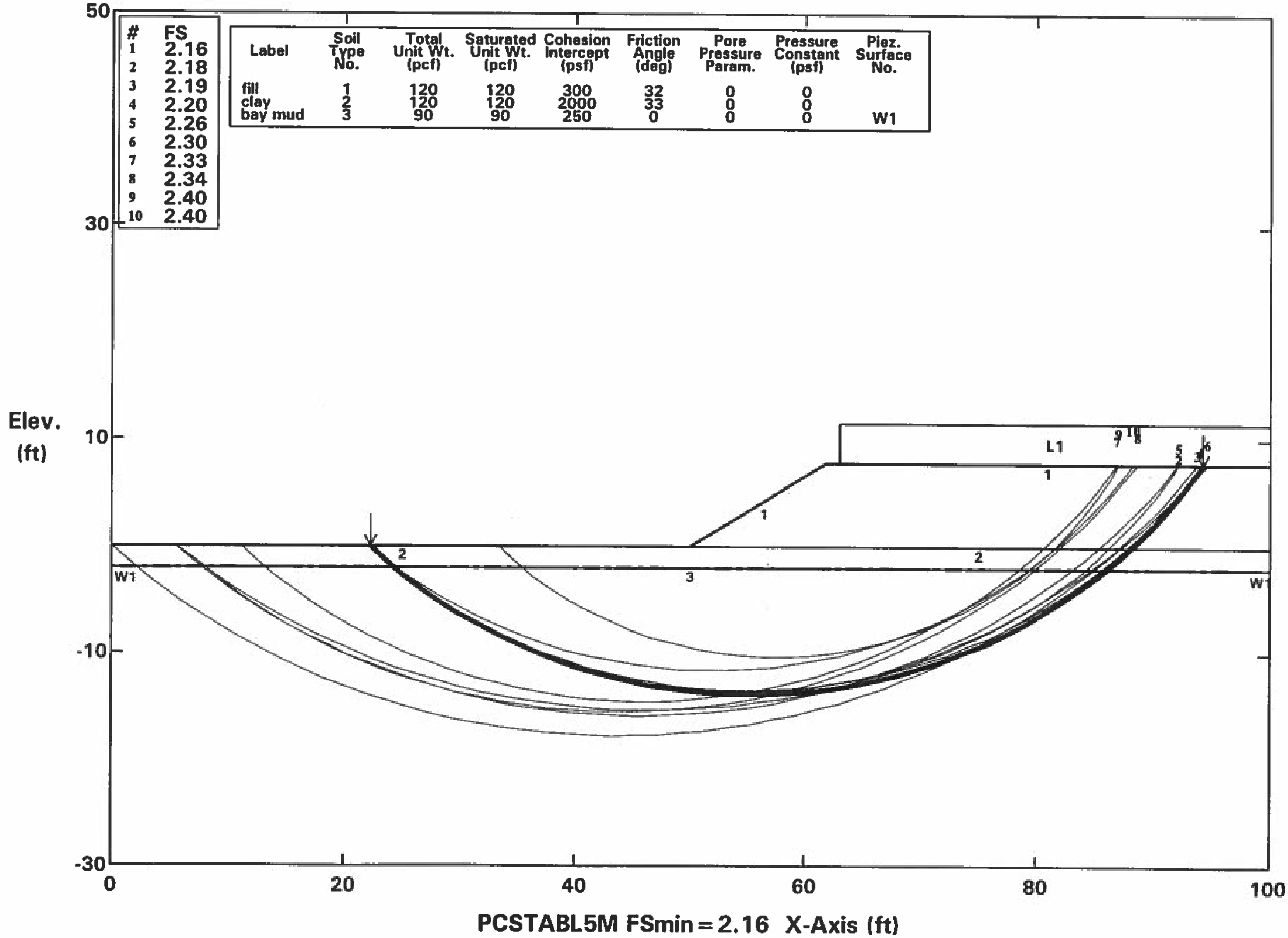


PLATE C-1A

RTE 37, Sta 120 + 60, "S3-M", Pseudostatic Condition (g = 0.2)

Ten Most Critical. D:MARE_D2.PLT By: M.S. 03-13-03 11:51am

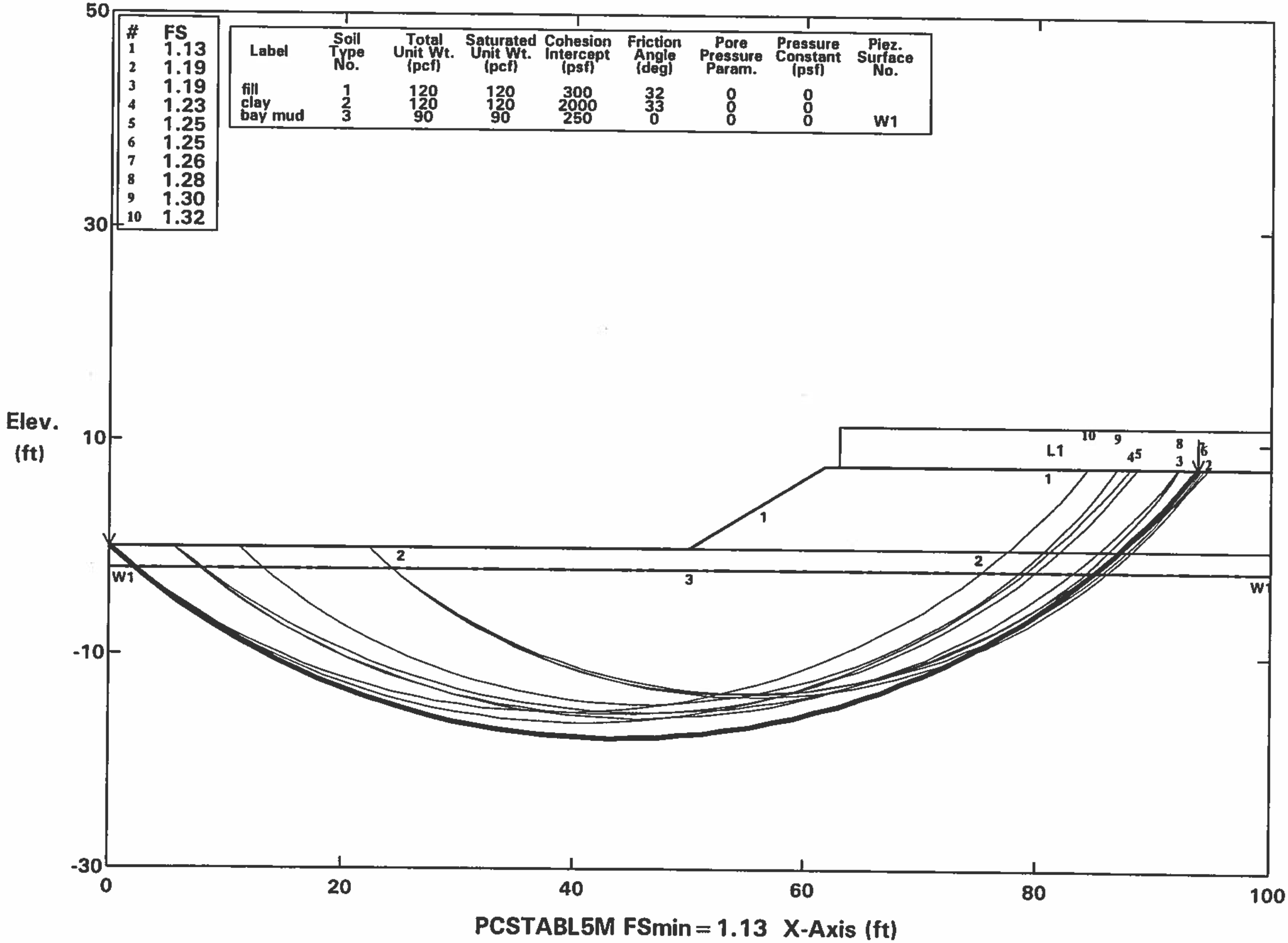


PLATE C-1B

SETTLEMENT ANALYSES (using wick drain)

PROJECT: Mare Island/Rte/37
JOB NO: 201126.gdr

LOCATION: Eastbound off ramp to Rte 37
REF. BORING: BM-1

DATE: 1/24/03

Height of fill: 6 ft
Width of fill: 22 ft
Total unit weight of fill: 120 pcf

Soil Type; Sand/silt(1), Clay(2)
Type of Sampler; MC(1), SPT(2)

Layer No.	Depth of the Layer From (ft)	To (ft)	Soil Type	Blow Counts SPT-N	Type of Sampler	γ' (psf)	W(%)	σ' (psf)	Δσ (psf)	Su (psf)	Pc Lab.	Pc Calculated	$Cc/1+e_0$	$Cr/1+e_0$	Consolidation Settlements (inches) OC	NC	SUM
1	0	2	2	31	1	120	18	120	689	2325		10568	0.0477	0.0057	0.11	0.00	0.11
2	2	4	2	10	1	120	26	360	634	750		3409	0.0823	0.0099	0.10	0.00	0.10
3	4	10	2	0	1	20	153	540	546	0	900	0	0.3370	0.0404	0.69	1.98	2.67
4	10	15	2	2	1	30	89	675	459	150	1000	469	0.2582	0.0310	0.44	0.85	1.29
5	15	20	2	7	1	11	197	777.5	401	525		1641	0.3737	0.0448	0.49	0.00	0.49
6	20	25	2	0	1	23	114	862.5	356	0		0	0.2942	0.0353	0.00	2.65	2.65
7	25	30	2	0	1	43	52	1027.5	320	0		0	0.1800	0.0216	0.00	1.27	1.27
8	30	35	2	11	1	50	30	1260	291	825		2578	0.1000	0.0120	0.06	0.00	0.06
9	35	40	2	0	1	50	138	1510	266	0		0	0.3219	0.0386	0.00	1.36	1.36
10	40	45	2	16	1	20	34	1685	246	1200		5455	0.1182	0.0142	0.05	0.00	0.05
11	45	50	2	33	1	55	34	1872.5	228	2475		11250	0.1182	0.0142	0.04	0.00	0.04
12	50	55	2	25	1	55	45	2147.5	213	1875		8523	0.1590	0.0191	0.05	0.00	0.05
13	55	60	2	45	1	55	33	2422.5	199	3375		15341	0.1139	0.0137	0.03	0.00	0.03
14	60	65	1	25	1	55	41	2697.5	187	1875		8523	0.1454	0.0175	-	-	-

TOTAL SETTLEMENT (inches) = 2.06 8.11 10.17

O.C. RANGE SETTLEMENT = 2.06 inches
N.C. RANGE SETTLEMENT = 8.11 inches

RATE OF SETTLEMENT:
Height of lab sample = 1 inch
Thickness of clay interval = 2 ft
t_{90} = 60 min. (from lab)
$Cv = TH^2/t_{90}$ = 0.03533 ft^2/day
$t = TH^2/Cv$ = 24 days (for 90% consolidation using wick drain 2 ft apart)

Note:
1) The undrained shear strength (S_u) is correlated with SPT-N values based on Terzaghi and Peck (1967), or obtained from laboratory test results.
2) Approximate estimation of preconsolidation pressure is based on S_u/P ratio per Skempton (1957).
3) The modified compression index ($C_c/1+e_o$) is correlated with natural moisture content based on Lambe and Whitman (1969).
4) The recompression index is adopted as 12% of the virgin compression index, which is typically 5% to 10% of the compression index (Holtz and Kovacs, 1982)
5) The applied pressure is estimated from 2V:1H distribution of the embankment load.

SETTLEMENT ANALYSES (using light weight fill)

PROJECT: Mare Island/Rte/37
JOB NO: 201126.gdr

LOCATION: Eastbound on ramp to Rte 37
REF. BORING: BM-2

DATE: 1/24/03

Height of fill: 6 ft
Width of fill: 39 ft
Total unit weight of fill: 60 pcf

Soil Type; Sand/silt(1), Clay(2)
Type of Sampler; MC(1), SPT(2)

Layer No.	Depth of the Layer From (ft)	To (ft)	Soil Type	Blow Counts SPT-N	Type of Sampler	γ' (psf)	W(%)	σ' (psf)	Δσ (psf)	Su (psf)	Pc Lab.	Pc Calculated	$Cc/1+e_0$	$Cr/1+e_0$	Consolidation Settlements (inches) OC	NC	SUM
1	0	2	1	50	1	120	14	120	351	3750		17045	0.0423	0.0051	-	-	-
2	2	4	1	50	1	120	12	360	334	3750		17045	0.0389	0.0047	-	-	-
3	4	10	2	50	1	30	94	570	305	3750		11719	0.2661	0.0319	0.43	0.00	0.43
4	10	15	2	0	1	30	100	735	273	0		0	0.2751	0.0330	0.00	2.26	2.26
5	15	20	2	0	1	30	92	885	248	0		0	0.2630	0.0316	0.00	1.70	1.70
6	20	25	2	9	1	30	34	1035	228	675		2109	0.1182	0.0142	0.07	0.00	0.07
7	25	30	2	11	1	30	122	1185	211	825		2578	0.3040	0.0365	0.16	0.00	0.16
8	30	35	2	6	1	30	41	1335	196	450		1406	0.1454	0.0175	0.04	0.32	0.37
9	35	40	2	5	1	30	109	1485	184	375		1172	0.2876	0.0345	0.00	0.87	0.87
10	40	45	2	6	1	30	69	1635	172	450		2045	0.2211	0.0265	0.07	0.00	0.07
11	45	50	2	28	1	30	26	1785	162	2100		9545	0.0823	0.0099	0.02	0.00	0.02
12	50	60	2	26	1	55	36	2135	149	1950		8864	0.1265	0.0152	0.05	0.00	0.05
13	60	70	2	42	1	55	24	2685	135	3150		14318	0.0724	0.0087	0.02	0.00	0.02
14	70	80	2	29	1	55	44	3235	123	2175		9886	0.1557	0.0187	0.04	0.00	0.04

TOTAL SETTLEMENT (inches) = 0.90 5.15 6.06

O.C. RANGE SETTLEMENT = 0.90 inches
N.C. RANGE SETTLEMENT = 5.15 inches

RATE OF SETTLEMENT:

Height of lab sample = 1 inch
Thickness of clay interval = 45 ft
t_{90} = 60 min. (from lab)
$Cv = TH^2/t_{90}$ = 0.03533 ft^2/day
$t = TH^2/Cv$ = 12150 days (for 90% consolidation using light weight fill)

Note:
1) The undrained shear strength (S_u) is correlated with SPT-N values based on Terzaghi and Peck (1967), or obtained from laboratory test results.
2) Approximate estimation of preconsolidation pressure is based on S_u/P ratio per Skempton (1957).
3) The modified compression index ($C_c/1+e_o$) is correlated with natural moisture content based on Lambe and Whitman (1969).
4) The recompression index is adopted as 12% of the virgin compression index, which is typically 5% to 10% of the compression index (Holtz and Kovacs, 1982)
5) The applied pressure is estimated from 2V:1H distribution of the embankment load.

Settlement Analyses for Bridge Embankment

Job Name: Mare Island
Job No.: 201126.gdr
Ref. Boring: BM-2
Structure Type: Brdg. Embank.
Date: 3/12/03

INPUT

Load (lbs.) = 3307086
B (ft) = 40
L (ft) = 60
Submerged Layer = 3
Depth of the Footing (ft) = 0

Rate of Settlement;y/n = n
Starting Settle. Layer No. =
Ending Settle. Layer No. =
LL =
% of Desired Consolidation =
2-way/1-way Drainage Layer =

Soil Layer	Soil Type	Sampler Type	Layer Thick.(ft)	Blow Counts	γ (pcf)	Wn %	q_u (tsf)	PI	γ' (pcf)	Su(psf)	P_0 (psf)	Z (ft)	ΔP_0 (psf)	Pc (psf)	ΔH (in.)
1	1	1	2	50	120	14			120	-	120	1	1322	-	-
2	1	1	3	50	120	12			120	-	420	3.5	1197	-	-
3	2	1	5	50	90	94			30	3780	675	7.5	1031	17182	0.733
4	2	1	5	0	90	100			30	0	825	12.5	869	0	4.921
5	2	1	5	0	90	92			30	0	975	17.5	742	0	3.676
6	2	1	5	9	90	34			30	680.4	1125	22.5	641	3093	0.135
7	2	1	5	11	90	122			30	831.6	1275	27.5	560	3780	0.335
8	2	1	5	6	90	41			30	453.6	1425	30	525	2062	0.000
9	2	1	5	5	90	109			30	378	1575	0	1378	1718	0.000
10	2	1	5	6	90	69			30	453.6	1725	0	1378	2062	0.000
11	2	1	5	28	90	26			30	2116.8	1875	0	1378	9622	0.000
12	2	1	5	26	90	36			30	1965.6	2025	0	1378	8935	0.000
13	2	1	5	42	90	24			30	3175.2	2175	0	1378	14433	0.000
14	2	1	5	29	90	44			30	2192.4	2325	0	1378	9965	0.000
														Sum:	9.799

Notes:
1) Soil type: sand (1), clay (2); Sampler type: MC (1), SPT (2).
2) The undrained shear strength (S_u) is correlated with SPT-N values based on Terzaghi and Peck (1967), or obtained from laboratory test results.
3) Approximate estimation of preconsolidation pressure is based on S_u/P ratio per Skempton (1957).
4) The modified compression index ($C_c/1+e_o$) is correlated with natural moisture content based on Lambe and Whitman (1969).
5) The recompression index is adopted as 12% of the virgin compression index, which is typically 5% to 10% of the compression index (Holtz and Kovacs, 1982)
6) The applied pressure is estimated from 2V:1H distribution of the embankment load.

MAINTENANCE-FREE SERVICE DESIGN ESTIMATES FOR DRAINAGE FACILITIES USING:
CALIFORNIA CULVERT CRITERIA AND CULVERT4.EXE, (RELEASE DATE 04-16-98)

PROJECT LOCATION...Mare Isalnd

PROJECT ACCOUNT NO.201126.gdr

SAMPLE LOCATION....R-1

TEST SAMPLE NO.....0-1.5 m

OPERATOR..........DPD

TEST DATE..........6-10-02

************ A DATA VALUE OF ZERO INDICATES NO DATA INPUT *****************

CSP SITE pH = 8.4 , WATER pH = 0.0 , SOIL pH = 8.4

MINIMUM RESISTIVITY, OHM-CM: CSP SITE = 1600 , WATER = 0 , SOIL = 1600

****

ESTIMATED SERVICE LIFE OF CSP CULVERTS, YEARS
SEE CALTRANS HIGHWAY DESIGN MANUAL CHAPTER 850

CSP THICK age & mm		GALV. 57 g	GALV.+ BIT COAT. (WATER SIDE)	GALV.+ BIT COAT & PAVED INV. (ABRASION)	GALV.+ BIT COAT (SOIL SIDE)	GALV.+ POLYMER 90 DEG INVERT
18	1.3	30	38	45	55	80
16	1.6	39	47	54	64	89
4	2.0	48	56	63	73	98
12	2.8	66	74	81	91	116
10	3.5	84	92	99	109	134
8	4.3	102	110	117	127	152

FLOW VEL. <1.5 m/s WITH NON-ABRASIVE CONDITIONS, (DEFAULT VALUES)
CAP, 18 GAGE (1.3 mm) CSP AND CASP MAY BE USED WITH THESE FLOW VELOCITIES

STANDARD REINFORCED CONCRETE PIPE DESIGN SHOULD BE
SUITABLE FOR THIS USER DEFINED LEVEL OF CHLORIDES

CONCRETE AND RCP MITIGATION MEASURES FOR pH
TYPE IP (MS) MODIFIED CEMENT OR TYPE II MODIFIED CEMENT
MINIMUM REQUIRED BY CALTRANS STD. SPECS. 90-1.01

A CORRUGATED ALUMINUM PIPE, CAP, MAY BE USED
IF ABRASIVE CONDITIONS DO NOT EXIST
SITE CONDITIONS MEET CORROSION REQUIREMENTS

A CORRUGATED ALUMINIZED STEEL PIPE, CASP, MAY BE USED
SITE CONDITIONS MEET CORROSION REQUIREMENTS

PLASTIC PIPE IS APPROVED FOR 50 YEARS SERVICE LIFE FOR
CORROSIVE CONDITIONS. ABRASION MUST BE EVALUATED. ALSO,
CONSIDER CONCRETE HEADWALLS AND CONCRETE OR METAL END
TREATMENT WHERE HIGH FIRE POTENTIAL EXISTS.

MAINTENANCE-FREE SERVICE DESIGN ESTIMATES FOR DRAINAGE FACILITIES USING:
CALIFORNIA CULVERT CRITERIA AND CULVERT4.EXE, (RELEASE DATE 04-16-98)

PROJECT LOCATION...Mare Isalnd

PROJECT ACCOUNT NO.201126.gdr

SAMPLE LOCATION....R-7

TEST SAMPLE NO.....0-1.5 m

OPERATOR...........DPD

TEST DATE..........6-10-02

************ A DATA VALUE OF ZERO INDICATES NO DATA INPUT *****************
CSP SITE pH = 8.5 , WATER pH = 0.0 , SOIL pH = 8.5
MINIMUM RESISTIVITY, OHM-CM: CSP SITE = 4000 , WATER = 0 , SOIL = 4000

ESTIMATED SERVICE LIFE OF CSP CULVERTS, YEARS
SEE CALTRANS HIGHWAY DESIGN MANUAL CHAPTER 850

CSP THICK Gage	& mm	GALV. 57 g	GALV.+ BIT COAT. (WATER SIDE)	GALV.+ BIT COAT & PAVED INV. (ABRASION)	GALV.+ BIT COAT (SOIL SIDE)	GALV.+ POLYMER 90 DEG INVERT
18	1.3	44	52	59	69	94
16	1.6	57	65	72	82	107
14	2.0	70	78	85	95	120
12	2.8	96	104	111	121	146
10	3.5	123	131	138	148	173
8	4.3	149	157	164	174	199

FLOW VEL. <1.5 m/s WITH NON-ABRASIVE CONDITIONS, (DEFAULT VALUES)
CAP, 18 GAGE (1.3 mm) CSP AND CASP MAY BE USED WITH THESE FLOW VELOCITIES

STANDARD REINFORCED CONCRETE PIPE DESIGN SHOULD BE
SUITABLE FOR THIS USER DEFINED LEVEL OF CHLORIDES

CONCRETE AND RCP MITIGATION MEASURES FOR pH
TYPE IP (MS) MODIFIED CEMENT OR TYPE II MODIFIED CEMENT
MINIMUM REQUIRED BY CALTRANS STD. SPECS. 90-1.01

A CORRUGATED ALUMINUM PIPE, CAP, MAY BE USED
IF ABRASIVE CONDITIONS DO NOT EXIST
SITE CONDITIONS MEET CORROSION REQUIREMENTS

A CORRUGATED ALUMINIZED STEEL PIPE, CASP, MAY BE USED
SITE CONDITIONS MEET CORROSION REQUIREMENTS

PLASTIC PIPE IS APPROVED FOR 50 YEARS SERVICE LIFE FOR
CORROSIVE CONDITIONS. ABRASION MUST BE EVALUATED. ALSO,
CONSIDER CONCRETE HEADWALLS AND CONCRETE OR METAL END
TREATMENT WHERE HIGH FIRE POTENTIAL EXISTS.

MAINTENANCE-FREE SERVICE DESIGN ESTIMATES FOR DRAINAGE FACILITIES USING:
CALIFORNIA CULVERT CRITERIA AND CULVERT4.EXE, (RELEASE DATE 04-16-98)

PROJECT LOCATION...Mare Isalnd

PROJECT ACCOUNT NO.201126.gdr

SAMPLE LOCATION....R-8

TEST SAMPLE NO.....0-1.5 m

OPERATOR..........DPD

TEST DATE..........6-10-02

************ A DATA VALUE OF ZERO INDICATES NO DATA INPUT *****************
CSP SITE pH = 8.2 , WATER pH = 0.0 , SOIL pH = 8.2
MINIMUM RESISTIVITY, OHM-CM: CSP SITE = 1600 , WATER = 0 , SOIL = 1600
****

ESTIMATED SERVICE LIFE OF CSP CULVERTS, YEARS
SEE CALTRANS HIGHWAY DESIGN MANUAL CHAPTER 850

CSP THICK Gage	& mm	GALV. 57 g	GALV.+ BIT COAT. (WATER SIDE)	GALV.+ BIT COAT & PAVED INV. (ABRASION)	GALV.+ BIT COAT (SOIL SIDE)	GALV.+ POLYMER 90 DEG INVERT
18	1.3	30	38	45	55	80
16	1.6	39	47	54	64	89
14	2.0	48	56	63	73	98
12	2.8	66	74	81	91	116
10	3.5	84	92	99	109	134
8	4.3	102	110	117	127	152

FLOW VEL. <1.5 m/s WITH NON-ABRASIVE CONDITIONS, (DEFAULT VALUES)
CAP, 18 GAGE (1.3 mm) CSP AND CASP MAY BE USED WITH THESE FLOW VELOCITIES

STANDARD REINFORCED CONCRETE PIPE DESIGN SHOULD BE
SUITABLE FOR THIS USER DEFINED LEVEL OF CHLORIDES

CONCRETE AND RCP MITIGATION MEASURES FOR pH
TYPE IP (MS) MODIFIED CEMENT OR TYPE II MODIFIED CEMENT
MINIMUM REQUIRED BY CALTRANS STD. SPECS. 90-1.01

A CORRUGATED ALUMINUM PIPE, CAP, MAY BE USED
IF ABRASIVE CONDITIONS DO NOT EXIST
SITE CONDITIONS MEET CORROSION REQUIREMENTS

A CORRUGATED ALUMINIZED STEEL PIPE, CASP, MAY BE USED
SITE CONDITIONS MEET CORROSION REQUIREMENTS

PLASTIC PIPE IS APPROVED FOR 50 YEARS SERVICE LIFE FOR
CORROSIVE CONDITIONS. ABRASION MUST BE EVALUATED. ALSO,
CONSIDER CONCRETE HEADWALLS AND CONCRETE OR METAL END
TREATMENT WHERE HIGH FIRE POTENTIAL EXISTS.

APPENDIX D

PARIKH CONSULTANTS, INC.
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• Geotechnical
• Environmental
• Materials Testing
• Construction Inspection

CCS PLANNING & ENGINEERING
6 Crow Canyon Court
San Ramon, CA 94583

Job No.: 201126.GDR
March 18, 2003

Attn: Mr. Larry Taylor

Sub: Response to Caltrans Office of Geotechnical Design Comments

Ref: GEOTECHNICAL DESIGN AND MATERIALS REPORT
MARE ISLAND/ROUTE 37 PROJECT
04-SOL-37/KP 11.4-11.9
SOLANO COUNTY, CA
EA 284700, CA 04277

Dear Mr. Taylor:

This letter is a response to the comments by Caltrans on the above reference report. The comments and respective responses are listed below for convenience.

Office of Geotechnical Design

1. "Existing Facilities and Improvements" section of the report should include a more detailed description of the magnitudes of fills and heights of retaining walls planned for the project.

Response: Will comply.

2. Section 4.1 Climate. There appears to be an error in rainfall amounts since the January precipitation average is greater than the annual rainfall amount.

Response: As per reference, January has an average precipitation of 140 mm. The average of November through March is about 110 mm. (Nov. 100 mm, Dec. 100 mm, Jan. 140 mm, Feb. 100 mm, & March 90 mm).

3. There are two sections numbered 4.2.

Response: Report will be revised.

4. Section 4.4 Regional Geology. The project location is located in the Coastal Range Geomorphic Province, rather than the Great Valley Province.

Response: Will revise.

5. Section 6.2 Laboratory Testing. The report indicates that Grain Size Analysis (California Test Method 202) tests were performed, but the results were not included in Appendix B. Also, stress strain curves for laboratory unconfined compression tests should be included in Appendix B.

Response: Grain size analysis has not been performed. Stress stain curves for laboratory unconfined compression tests will be shown in Appendix B.

6. Section 7.2 "Subsurface Soil Conditions" should include a description of the thickness of subsurface materials encountered.

Response: Will comply.

7. Section 8.1.1, Parameter Selection. The references cited should be included in the References Section of the report.

Response: Will comply.

8. Section 8.3.1 recommends wick drains for embankment over 0.6 m high. Detailed recommendations should be provided, including methods to monitor settlement.

Response: Will comply.

9. Section 8.3.1 also recommends lightweight fill for roadway portions where no structural foundations are involved. It was not clear if all roadway portions would apply or only roadway portions up to a certain height of new fill. Specific/detailed recommendations, including lightweight fill specifications should be provided.

Response: Roadways with 0.3 m fill and up will need light weight fill. Detailed recommendations will be addressed in our report.

10. Section 8.3.2, Evaluation of Embankment Stability only provides stability results for what appears to be shorter portions of fill embankments. Slope stability analyses for the worst-case embankments, including steeper portions of the pedestrian path embankment should be included. A table summarizing the conditions evaluated should be included in the text.

Response: Will comply.

11. Section 8.4, Earth Retaining System. It was not clear if the pile recommendations were based on soil strengths after the settlement period was complete. Also, it was not clear if the pile foundation had lateral load or tension demands. The pile data table should conform to Section 3-1 of the Caltrans Memo to Designers.

Response: The soil strength data are based on the laboratory tests performed on the samples retrieved in the soil borings. To obtain strength data after consolidation is complete, one should drill new soil boring and perform a set of lab tests to get new soil strength data. However, using soil strength parameters before consolidation is completed, as in this case, will result in a more conservative design. Based on the designer there is 13 kips lateral load and no tension demand on the piles. However, the length of the piles are governed by compression load. The pile data table will be revised per section 3-1 of the Caltrans Memo to Designers.

12. Oversized Sheets K-1, L-1, L-2, X-1, X-2 and X-3 were included in the report after page 26. Their purpose is not clear and is not mentioned in the text of the report.

Response: These sheets are the as-built data of existing pavement sections. Some past reviewers have recommended attaching them in the report.

13. Section 12, Construction Consideration. Possible groundwater is not mentioned for removal and replacement of lightweight fill or retaining wall construction.

Response: Due to rotary wash method of drilling, groundwater depth could not be measured. Based on the LOTBs by others and as-built Log of Test Borings (1956), groundwater level is recorded at Elev. 1.5 ft. (about 0.5 m below ground surface). For the purpose of analyses, we have assumed the groundwater table at about the same elevation. It will be clarified in the Construction Considerations.

14. Section 13.1 Summary of Recommendations. The pile-supported retaining wall is not included in the summary.

Response: Will comply.

15. Site Plan, Plate 2. Add boring locations BM-1 and BM-2 to the site Plan.

Response: Will comply.

16. Geologic Map, Plate 3. The map is not legible. Provide a legible Geologic Map.

Response: Will comply.

17. Fault Map, Plate 4. The map is not legible. Provide a legible Fault Map.

Response: Will comply.

18. Appendix B, provide Laboratory data for Grain Size Analysis and Unconfined Compression test results.

Response: No Grain Size Analysis has been performed. The Unconfined Compression test results will be attached in Appendix C.

Response: Will comply.

19. Appendix D includes Caltrans plans for another project for stone columns. Since stone columns are not recommended in the report, they should be removed from the appendix. Also, Appendix C includes layout details for a different wick drain project at another location. A layout specific for the project should be included.

Response: Appendix D was attached as a sample. Since the designer will have a detailed description in the Specification, tailored for this job, this appendix will be deleted.

20. Specifications for lightweight fill should be included.

Response: This will be shown in the Specifications prepared by the designer.

Materials

1. Page 26, Structural Pavement Section. For R value of 10 and TI of 12, the option 1 section should be 195 AC(A), 285 AB(3), 405 AS(4).

   Response: Will comply.

2. In the same table, delete the ATPB option since it is no longer recommended for new structural section by Materials. ATPB is recommended only for widening sections where adjacent pavement consists of ATPB.

   Response: Will comply.

Respectfully submitted,
PARIKH CONSULTANTS, INC.

M. Sal. ml

Manny Saleminik, P.E. C60597
Project Engineer

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GAURANG R. PARIKH
No. 666
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STATE OF CALIFORNIA

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