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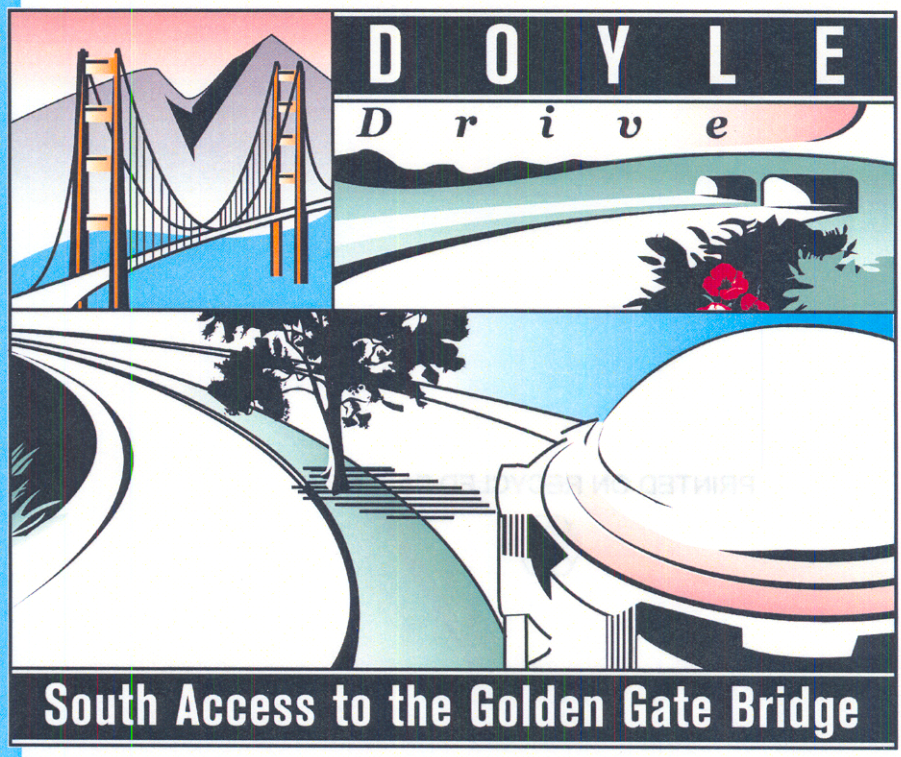

FINAL PRELIMINARY GEOTECHNICAL REPORT

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Prepared By:

Partha Sircar, Parsons Brinckerhoff

Reviewed By:

Matthew Fowler, Parsons Brinckerhoff

Approved By:

Leroy L. Saage, Project Manager
San Francisco County Transportation Authority

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SECTION 1: EXECUTIVE SUMMARY

The San Francisco County Transportation Authority (SFCTA) is proposing to replace Doyle Drive, a stretch of Highway 101 extending to the southern approach of the Golden Gate Bridge. Doyle Drive lies entirely within the Presidio of San Francisco (the Presidio), which is presently part of the Golden Gate National Recreation Area (GGNRA) and lies entirely within the Trust managed lands. The existing stretch includes a 463-meter (1,520-foot) long high-viaduct near the western end and an approximately 1,137-meter (3,730-foot) long low-viaduct at the eastern end.

The purpose of the project is to replace Doyle Drive to improve the seismic, structural, and traffic safety of the roadway within the setting and context of the Presidio and its purpose as a National Park. The new facility would provide wider lanes, a median barrier and continuous outside shoulders. Three alternatives are being considered: No-build, Replace and Widen, and Presidio Parkway. The Replace and Widen alternative would replace the high- and low-viaducts in generally the same location. The Presidio Parkway alternative involves reconstruction of the high-viaduct, and the construction of four short tunnels (two cut-and-cover and two at-grade concrete box construction) and two low causeway structures. The new facility will generally follow the same alignment as the existing roadway.

This Final Preliminary Geotechnical Report has been prepared as a part of the Doyle Drive Environmental and Design Study (Design Study) with the specific aim of:

- Providing sufficient geotechnical information from existing sources to allow, in conjunction with other project reports (e.g., 'Hydrology and Water Resources' and 'Natural Environmental Study (NES)'), the assessment of environmental impacts, constructability and preliminary construction costs for the subsurface structural elements (structure foundations, tunnel section designs, limits of hazardous materials, etc.);
- Identifying areas or conditions that will require additional subsurface investigations for final design;
- Addressing the technical content and topics required for Preliminary Geotechnical Reports by the State of California Department of Transportation (Caltrans); and
- Providing preliminary design recommendations for deep foundations and tunnel structures.

This report presents all of the pertinent background information presented earlier in the Preliminary Geotechnical Report, Final (April, 2002). However, as noted earlier, the several replacement alternatives considered in the prior report but since then rejected are not presented in this report. Separate design reports will be produced for each structure for the Final Foundation Reports, as appropriate, once the project is finalized.

The draft version of this report (Draft Preliminary Materials and Foundation Report, July 2000) was based solely on available existing reports, boring logs, as-built drawings and a reconnaissance of the project site. A limited preliminary subsurface investigation program was performed in February 2001 to obtain additional groundwater and geotechnical data to allow a more accurate estimate of bedrock depths, liquefaction potential and hydrologic conditions in the Tennessee Hollow and East Bluff areas for purposes of alternative evaluations and cost estimating. The results of the preliminary investigations were incorporated in: 'Preliminary Geotechnical Report, Revision 0, May 2001. The report was finalized with the incorporation of all comments from the various agencies and published as Preliminary Geotechnical Report, Final (April, 2002).

Additional subsurface investigations will be required to complete the final design, including detailed investigations to aid the design of the viaduct foundations and tunnels and the design of appropriate systems to minimize impacts to the hydrologic system and the vegetation and habitats it supports. All structures pertaining to this project (viaducts, tunnels, and road sections) have been addressed in this Preliminary Report. Separate reports will be produced for each structure for the Final Foundation Reports once a preferred alternative is identified.

Cast-In-Drilled-Hole (CIDH) piles are recommended for the High-viaduct foundations where the bedrock is shallow as is expected for the western half of the high-viaduct. Cast-In-Steel Shell (CISS) or CIDH piles are recommended for the low-viaduct, Replace and Widen – No Detour Alternative. Driven piles are recommended for the rest of the viaduct foundations. However, CIDH piles or other non-standard piles may be opted for in other areas to allay concerns regarding potentially unacceptable levels of noise or vibrations generated by pile driving. Estimated required pile penetrations are provided for the CIDH piles and for a range of the most common sizes of Cast-In-Steel-Shell (CISS) and pre-stressed, precast concrete piles.

Seismic design for the structures should be base on a Magnitude 8 (Moment Magnitude) earthquake on the San Andreas Fault. A peak ground acceleration of 0.5g should be used. Design response spectra consistent with the subsurface conditions are provided in Section 12.

Preliminary design and construction recommendations are provided for the four tunnels, retaining walls, pavements and earthwork. The following findings are judged to be particularly significant from the standpoints of final design and construction:

- Liquefaction potential exists in the area covered by the 'soft soils' of the historic tidal marsh all through the corridor east of Station 20+00. This will impact the low causeway structures, the Main Post Tunnels and the depressed roadway east of the Main Post Tunnels. Large post-liquefaction settlements may occur in these areas.
- There is some indication for potential for liquefaction in the area of the proposed high-viaduct (between stations 9+50 and 11+30). Future soil investigations are required to address the issue more definitively.
- Due to the ongoing remedial actions at various locations at the Presidio Army Base, major portion of the hazardous materials at the site are expected to be removed prior to start of construction. Nevertheless, some hazardous/contaminated materials are still likely to be encountered:
    - Friable asbestos and other chemicals from demolition of buildings
    - Naturally occurring asbestos in the serpentine at the western end of the site likely to be encountered during excavation for foundations for the high-viaduct and other structures
    - Elevated levels of lead contamination near the area of the high-viaduct
- The proximity of the project site to San Francisco Bay and the past existence of a tidal marsh strongly suggest a highly corrosive environment.
- There is a slide repair immediately north of the west abutment of the low-viaduct.

SECTION 2: INTRODUCTION

The San Francisco County Transportation Authority (SFCTA) is proposing to replace Doyle Drive, a 2.4-kilometer (1.5-mile) stretch of Highway 101 extending from Marina Boulevard and Lombard Streets to the southern approach of the Golden Gate Bridge. As shown in Figure 1, Doyle Drive lies entirely within the Presidio, which in 1994 was transferred from the U.S. Army to the National Park Service and is now part of the Golden Gate National Recreation Area (GGNRA). In July 1998, management of the non-coastal portions of the Presidio (known as Area B) was transferred to the Presidio Trust (the Trust). Doyle Drive lies predominantly within the Trust managed lands. The existing structure includes an approximately 463-meter (1,520-foot) long high-viaduct near the western end and an approximately 1,137-meter (3,730-foot) long low-viaduct at the eastern end.

2.1 OBJECTIVE AND SCOPE OF REPORT

This Final Preliminary Geotechnical Report has been prepared as a part of the Doyle Drive Environmental and Design Study (Design Study) with the specific aim of:

- Update the geotechnical analysis to reflect the current alternatives, including the Presidio Parkway that was introduced after the publication of the previous report (April, 2002).
- Providing sufficient geotechnical information from existing sources to allow the assessment of the environmental impacts, constructability and preliminary construction costs for the subsurface structural elements (structure foundations, tunnel section designs, limits of hazardous materials, etc.) for the acceptable alternatives;
- Identifying areas or conditions that will require additional subsurface investigations for final design;
- Addressing the technical content and topics required by the State of California Department of Transportation (Caltrans); and
- Providing preliminary design recommendations for deep foundations, tunnel structures, retaining walls and other pertinent geotechnical issues. Two of the tunnels will be shallow cut-and-cover and two other tunnels will be at-grade concrete box construction.

A Draft Preliminary Materials and Foundation Report, based solely on available existing reports, boring logs, as-built drawings and a reconnaissance of the project site was presented in July, 2000. Subsequently, a limited subsurface investigation program was performed in February 2001 to obtain additional groundwater and geotechnical data to allow a more accurate estimate of bedrock depths, liquefaction potential and hydrologic conditions in the Tennessee Hollow and East Bluff areas for purposes of alternative evaluations and cost estimating. The investigations at Tennessee Hollow assumed particular significance for the alternatives being considered at that time, since the potential for disturbance of the hydrogeologic regime at Tennessee Hollow is a very sensitive issue. A Final Preliminary Geotechnical Report was then presented in April, 2002, which incorporated the field investigation results. The report also presented the several alternatives being considered at that time (involving bridges and deeper tunnels, including across Tennessee Hollow), which have subsequently been dropped in favor of the now preferred Presidio Parkway alternative.

Additional subsurface investigations will be required to complete the final design.

2.2 BACKGROUND

Doyle Drive is located in the Presidio of San Francisco (the Presidio), in the northern part of the City of San Francisco at the southern approach to the Golden Gate Bridge (see Figure 1). In 1994, when the US Army transferred jurisdiction of the Presidio to the National Park Service (NPS), it became part of the National Park system and Golden Gate National Recreation Area (GGNRA). In 1998, management of the Presidio was divided between two federal agencies: The Presidio Trust (the Trust), the agency responsible for oversight of 80 percent of the Presidio delineated as Area B; and the NPS, which is responsible for management of the coastal portions of the park (the remaining 20 percent) that are delineated as Area A. Doyle Drive lies

predominately within the Area B lands managed by the Trust with a small portion at the western end located in Area A on land operated by the Golden Gate Bridge, Highway and Transportation District (GGBHTD). The Presidio has also been designated a National Historic Landmark District (NHLD) since 1962 with the Doyle Drive roadway determined to be a contributing element to that landmark.

Doyle Drive, the southern approach of US 101 to the Golden Gate Bridge, is 2.4 kilometers (1.5 miles) long with six traffic lanes. There are three San Francisco approach ramps which connect to Doyle Drive: one beginning at the intersection of Marina Boulevard and Lyon Street; one at the intersection of Richardson Avenue and Lyon Street; and one where Park Presidio Boulevard (State Route 1) merges into Doyle Drive approximately 1.6 kilometers (one mile) west of the Marina Boulevard approach (see Figure 1). Doyle Drive passes through the Presidio on an elevated concrete viaduct (low-viaduct) and transitions to a high steel truss viaduct (high-viaduct) as it approaches the Golden Gate Bridge Toll Plaza.

Doyle Drive is nearly 70 years old and it is approaching the end of its useful life, although regular maintenance, seismic retrofit, and partial rehabilitation activities are keeping the structure safe in the short term. However, further structural degradation caused by age and the effects of heavy traffic and exposure to salt air will cause the structures to become seismically and structurally unsafe in the coming years. In addition, the eastern portion of the aging facility is located in a potential liquefaction zone identified on the State of California Seismic Hazard Zones map dated August 2000.

Currently, Doyle Drive has nonstandard design elements, including travel lanes from 2.9 to 3.0 meters (9.5 to 10.0 feet) in width, no fixed median barrier, no shoulders and exit ramps that have tight turning radii. During peak traffic hours, plastic pylons are manually moved to provide a median lane as well as to reverse the direction of traffic flow of several lanes (Project Study Report: Doyle Drive Reconstruction, 1993).

2.3 CURRENT STUDY

A number of alternatives for Doyle Drive have been identified through a review of previous studies, public and agency input at scoping meetings and consideration by the study team. The following are the objectives and criteria adopted for the project:

- The rebuilding of the Doyle Drive will generally be along the current alignment;
- The setting for the project will conform to the Presidio as a national historic district;
- The new route will be designed as a scenic parkway with aesthetic features that will complement the park environment;
- Important historic buildings along the project route will be protected, where feasible;
- The lanes and the shoulders will typically be widened to meet current traffic safety standards
- The high-viaduct and the low-viaduct will be replaced;
- Tunnel(s) will be constructed to integrate the upper and lower sections of the Presidio
- Measures will be taken to minimize the impacts to the hydrogeologic regime in the Eastern Bluff and the Tennessee Hollow areas; and
- A Transit Center will be constructed on or adjacent to the new Doyle Drive Parkway to increase transit service levels within the park and provide a hub for the proposed Presidio shuttle service.

SECTION 3: PERTINENT REPORTS AND INVESTIGATIONS

This report has been primarily compiled from soils reports produced for construction of the existing Doyle Drive, from several more recent reports prepared for the U.S. Army for buildings in the Presidio, and from soils investigations performed for the Crissy Field restoration. The most recent data presented here is derived from a supplemental subsurface investigation program performed by Taber Consultants in February 2001 for the Doyle Drive Environmental and Design Study to obtain groundwater and soils data needed to better assess bedrock depths, liquefaction potential and hydrologic conditions in the Tennessee Hollow and East Bluff areas. The following documents have been used in preparing this report:

Apex Environmental Recovery, Inc., 1994, "Report, Site Investigation, Route 101 Presidio Viaduct/Doyle Drive, Route 1 Presidio Viaduct Ramp, Marina Viaduct, San Francisco, California", prepared for California, Dept. of Transportation, District 4, August, 1994

Baseline Environmental Consulting, "Revised Hydrology and Water Resources Technical Report, Doyle Drive Reconstruction Project", for Parsons Brinckerhoff, September, 2004

Baseline Environmental Consulting, "Revised Preliminary Site Investigation, Doyle Drive Reconstruction Project", for Parsons Brinckerhoff, September, 2004

Dames & Moore, 1997, "Final Remedial Investigation Report, Presidio Main Installation, Presidio of San Francisco" submitted to U.S. Army Environmental Center, Maryland, Contract No. DAAA15-90-D-0018, January, 1997

Environmental Science Associates (ESA), 2004, "Final Natural Environmental Study Doyle Drive Reconstruction Project", for Parsons Brinckerhoff, September, 2004

Golden Gate Bridge and Highway District of California, 1933, "Presidio Road Boring Diagram (Sheet A-11)", September 23, 1933

Golden Gate National Recreation Area, California (GGNRA), 1994, "Creating a Park for the 21st Century from military post to national park", Final General Management Plan Amendment, Presidio of San Francisco (GMPA), July, 1994

Harlan Tait Associates (HTA), 1998, "Geotechnical Investigation, Crissy Field, San Francisco, California", prepared for Golden Gate National Parks Association, June, 1998

IT Corporation, 1997, "Site Investigation Report, Buildings 207/231, Presidio of San Francisco, California", submitted to U.S. Army Corps of Engineers, Sacramento, Contract No. DACW05-95-D-0001, December, 1997

Montgomery Watson, 1994, "Draft, Building 637, Additional Site Investigation Report, Presidio of San Francisco, San Francisco, California", prepared for U.S. Army Corps of Engineers, Sacramento, California, Contract No. DACA05-93-0069, May 1994

Ove Arup & Partners California Ltd., 2004, "South Access to the Golden Gate Bridge, Doyle Drive SPUR Alternative Feasibility Study", Final Report to San Francisco County Transportation Authority, March 2004.

San Francisco Guideway Associates, 1996a, "Final Report, Doyle Drive Intermodal Study", prepared for SFCTA, November, 1996

San Francisco Guideway Associates, 1996b, "Technical Appendix, Doyle Drive Intermodal Study", prepared for SFCTA, November, 1996

Taber Consultants, 2001, "Summary of Exploration and Testing, Doyle Drive Reconstruction Project, San Francisco, CA", for Parsons Brinckerhoff Quade & Douglas, Inc., March, 2001

Treadwell and Rollo, 2001, "Geotechnical Investigation, Letterman Digital Center at the Presidio, San Francisco, California", Report to Conversion Management Associates, Inc., San Francisco, California, August, 2001.

U.S. Army Corps of Engineers (Sacramento District), 1971, "Presidio of San Francisco, California, Western Medical Institute of Research, (As Built) – Sheet G4 (Site Area Map) and Sheets C3&C4 (Exploration Logs)", May 1971

U.S. Army Corps of Engineers (Sacramento District), 1987, "Presidio of San Francisco, Commissary Building, Boring Log (Sheets B1.1 through B1.5)", June 1987

SECTION 4: PROJECT ALTERNATIVES AND EXISTING FACILITIES

4.1 EXISTING FACILITIES

The proposed area of construction principally consists of a corridor along the existing alignment of Doyle Drive. It also includes the Highway 1/Highway 101 Interchange and ramps (both inbound and outbound) to Richardson Avenue and Marina Boulevard. This portion of Highway 101 is a 6-lane highway with three lanes in each direction and no median. Plastic pylons have been used since 1968 during commute hours to reverse the direction of one of the center lanes, thereby adding an extra lane in the major commute direction at the time.

The 2.4-kilometer (1.5-mile) stretch includes an approximately 463-meter (1,520-foot) long high-viaduct. The high-viaduct has a concrete deck supported on steel trusses; the trusses in turn are supported on 18 concrete bents, supported at the ground surface. The maximum distance between the deck and ground surface is about 27 meters (90 feet). There is also a low-viaduct, approximately 1,137 meters (3,730 feet) long, near the east end of the route. The maximum deck distances from the ground surface are about 15 meters (50 feet) for the low-viaduct. Both of the viaducts were retrofitted in 1995-96 to satisfy the current seismic design requirements for Caltrans. Presently, there are no tunnels along the stretch of interest.

The entire route of interest to this project passes essentially through the Presidio. As has been noted earlier, the Presidio was an Army Base from 1848 to 1994. Old structures and other facilities once belonging to the U.S. Army are present all along the subject route, as may be seen from the map in Figure 3. Of particular significance to this project are the San Francisco National Military Cemetery and the Main Post buildings.

The project area is generally about 305 meters (1,000 feet) from the San Francisco Bay shoreline. Most of the area north of the alignment is occupied by what is known as Crissy Field. Crissy Field was an Army airfield until 1974. A major project has recently been completed in the Crissy Field area, which included, among other improvements, the restoration of a historic tidal marsh that originally occupied the area.

4.2 PROJECT ALTERNATIVES

The build alternatives for the Doyle Drive Project were developed with input from public scoping and reflected the parkway concept that evolved from previous studies. Through the screening analysis, six alternatives were selected for consideration in the Administrative DEIS/DEIR: Alternative 1, No-Build; Alternative 2, Replace and Widen; Alternatives 3a and 3b, Long Tunnels; and Alternatives 4a and 4b, Short Tunnels.

Subsequent to the Administrative DEIS/DEIR in 2002, a fifth alternative, the Presidio Parkway, was added to the list of alternatives for more detailed study. In comparison to the tunnel alternatives it was determined that Alternative 5, Presidio Parkway, would provide all the benefits and functions of Alternatives 3a, 3b, 4a, and 4b with less cost, construction duration and environmental impact. Hence, in November 2003 the four tunnel alternatives were recommended to be removed from further consideration and analysis in the DEIS/DEIR.

At a public meeting held in February 2004, the public agreed with the decision to drop Alternatives 3a, 3b, 4a, and 4b and retain Alternative 1, No-Build, Alternative 2, Replace and Widen, and Alternative 5, Presidio Parkway for consideration in the DEIS/DEIR.

This section describes the build alternatives in terms of physical and operating characteristics and a No-Build Alternative. As shown in Figure 1, the project limits are from Merchant Road, just south of the Golden Gate Bridge Toll Plaza, to the intersection of Richardson Avenue/Francisco Street and Marina Boulevard/Lyon Street. During the screening process, all alternatives were evaluated for their ability to meet the project's Purpose and Need. Detailed drawings showing the plan and profile of each alternative are presented in Figures 2A through 2E.

4.2.1 Alternative 1: No-Build Alternative

The No-Build Alternative represents the future year conditions if no other actions are taken in the study area beyond what is already programmed by the year 2020. The No-Build Alternative provides the baseline for existing environmental conditions and future travel conditions against which all other alternatives are compared.

Doyle Drive would remain in its current configuration, with six traffic lanes ranging in width from 2.9 to 3.0 meters (9.5 to 10 feet) and an overall facility width of 20.4 meters (67 feet) (see Figure 2A). There are no fixed median barriers or shoulders. The lane configuration is changed by manually moving plastic pylons to increase the number of lanes in the peak direction of traffic. The facility passes through the Presidio on a high steel truss viaduct and a low elevated concrete viaduct with lengths of 463 meters (1,519 feet) and 1,137 meters (3,730 feet), respectively. This alternative does not improve the seismic, structural, or traffic safety of the roadway.

Vehicular access to the Presidio is available from Doyle Drive via the off-ramp to Merchant Road at the Golden Gate Bridge Toll Plaza. Presidio access at the east end of the project will be provided for southbound traffic via a right turn from Richardson Avenue to Gorgas Avenue. Presidio access for northbound traffic will be provided by a slip ramp from Richardson Avenue to Gorgas Avenue, which is currently under construction.

4.2.2 Alternative 2: Replace and Widen Alternative

The Replace and Widen Alternative would replace the 463-meter (1,519-foot) high-viaduct and the 1,137-meter (3,730-foot) low-viaduct with wider structures that meet the most current seismic and structural design standards (see Figures 2B and 2C). The new facility would be replaced on the existing alignment and widened to incorporate improvements for increased traffic safety.

This alternative would include either six 3.6-meter (12-foot) lanes and a 3.6-meter (12-foot) eastbound auxiliary lane with a fixed median barrier or six 3.6-meter (12-foot) lanes with a moveable median barrier. The new facility would have an overall width of 38.0 meters (124 feet). The fixed median barrier option would require localized lane width reduction to 3.3 meters (11 feet) to avoid impacts to the historic batteries and Lincoln Boulevard, reducing the facility width to 32.4 meters (106 feet). Both options would include continuous outside shoulders along the facility. At the Park Presidio interchange, the two ramps connecting eastbound Doyle Drive to Park Presidio Boulevard and the ramp connecting westbound Doyle Drive to southbound Park Presidio Boulevard would be reconfigured to accommodate the wider facility. The Replace and Widen Alternative would operate similar to the existing facility except that there would be a median barrier and shoulders to accommodate disabled vehicles.

The Replace and Widen Alternative includes two options for the construction staging:

No Detour Option (Figure 2B) – The widened portion of the new facility would be constructed on both sides and above the existing low-viaduct and would maintain traffic on the existing structure. Traffic would be incrementally shifted to the new facility as it is widened over the top of the existing structure. Once all traffic is on the new structure, the existing structure would be demolished and the new portions of the facility would be connected. To allow for the construction staging using the existing facility, the new low-viaduct would be constructed two meters (six feet) higher than the existing low-viaduct structure.

With Detour Option (Figure 2C) - A 20.4-meter (67-foot) wide temporary detour facility would be constructed to the north of the existing Doyle Drive to maintain traffic through the construction period. Access to Marina Boulevard during construction would be maintained on an elevated temporary structure south of Mason Street. On and off ramps to the mainline detour facility would be located near the Post Exchange (PX) building.

Vehicular access to the Presidio is available from Doyle Drive via the off-ramp to Merchant Road at the Golden Gate Bridge Toll Plaza. Presidio access at the east end of the project will be provided for southbound traffic via a right turn from Richardson Avenue to Gorgas Avenue. There would be no Presidio access for northbound traffic at the east end of Doyle Drive due to geometric constraints and concerns for traffic safety.

4.2.3 Alternative 5: Presidio Parkway Alternative

The Presidio Parkway Alternative would replace the existing facility with a new six-lane facility and an eastbound auxiliary lane between the Park Presidio interchange and the new Presidio access at Girard Road (see Figures 2D and 2F). The new facility would have an overall width of up to 45 meters (148 feet), and would incorporate wide landscaped medians and continuous shoulders. To minimize impacts to the park, the footprint of the new facility would include a large portion of the existing facility's footprint east of the Park Presidio interchange. A 450-meter (1,476-foot) high-viaduct would be constructed between the Park Presidio interchange and the San Francisco National Cemetery. Shallow cut-and-cover tunnels would extend 240 meters (787 feet) past the cemetery to east of Battery Blaney. The facility would then continue towards the Main Post in an open depressed roadway with a wide, heavily landscaped median. From Building 106 (Band Barracks) cut-and-cover tunnels up to 310 meters long (984 feet) would extend to east of Halleck Street. The facility would then rise slightly on a low level causeway 160 meters (525 feet) long over the site of the proposed Tennessee Hollow restoration and a depressed Girard Road. East of Girard Road the facility would return to existing grade north of the Gorgas warehouses and connect to Richardson Avenue.

The Presidio Parkway Alternative would include an underground parking facility at the eastern end of the project corridor between the Mason Street Warehouses, Gorgas Street Warehouses and Palace of Fine Arts. The parking garage would supply approximately 500 spaces to maintain the existing parking supply in the area and improve pedestrian and vehicular access between the Presidio and the Palace of Fine Arts.

At the intersection with Merchant Road, just east of the toll plaza, a design option has been developed for a Merchant Road slip ramp. This option would provide an additional new connection from westbound Doyle Drive to Merchant Road. This ramp would provide direct access to the Golden Gate Visitors' Center and alleviate the congested weaving section where northbound Park Presidio Boulevard merges into Doyle Drive.

The Park Presidio interchange would be reconfigured due to the realignment of Doyle Drive to the south. The exit ramp from eastbound Doyle Drive to southbound Park Presidio Boulevard would be replaced with standard exit ramp geometry and widened to two lanes. The loop of the westbound Doyle Drive exit ramp to southbound Park Presidio Boulevard would be improved to provide standard exit ramp geometry. The northbound Park Presidio Boulevard connection to westbound Doyle Drive would be realigned to provide standard entrance ramp geometry. There are two options for the northbound Park Presidio Boulevard ramp to an eastbound Doyle Drive connection:

Option 1: Loop Ramp - Replace the existing ramp with a loop ramp to the left to reduce construction close to the Calvary Stables and provide standard entrance and exit ramp geometry.

Option 2: Hook Ramp - Rebuild the ramp with a similar configuration as the existing ramp with a curve to the right and improved exit and entrance geometry.

The Presidio Parkway Alternative includes two options for direct access to the Presidio and Marina Boulevard at the eastern end of the project:

Diamond Option (Figure 2E) – Direct access to the Presidio and Marina Boulevard in both directions is provided by the access ramps from Doyle Drive connecting to a grade-separated interchange at Girard Road. East of the new Letterman garage, Gorgas Avenue is a one-way street and connects to Richardson Avenue with access to Palace Drive via a signalized intersection at Lyon Street.

Circle Drive Option (Figure 2F) – The Circle Drive Option provides direct access to the Presidio and Marina Boulevard for eastbound traffic by access ramps connecting to a grade-separated interchange of Girard Road. Westbound traffic from Richardson Avenue would access the Presidio and Palace Drive through a jug handle intersection with Gorgas Avenue.

SECTION 5: PHYSICAL SETTING

5.1 CLIMATE

The climate of San Francisco is characterized as dry-summer subtropical, with cool wet winters and relatively warmer dry summers. The average maximum temperature is 62.1 degrees Farenheit and the average minimum temperature is 49.3 degrees Farenheit. The mean annual rainfall in the vicinity of the project site, for the period between 1914 and 2000, is approximately 50 centimeters (20 inches) (Western Regional Climate Center). Analysis of long-term precipitation records indicates that wetter and drier cycles lasting several years are common in the region.

5.2 TOPOGRAPHY AND DRAINAGE

The area occupied by the Presidio is generally hilly, sloping down northwards to the Crissy Field area. Crissy Field is generally flat, with elevations ranging from 2.7 to 3.7 meters (9 to 12 feet) above mean sea level. In the western half of the project route, two bluffs rise steeply to about 25 meters (80 feet). The two bluffs are separated by an 18-meter (60-foot) deep, 460-meter (1,500- foot) wide valley, which is spanned by the high-viaduct. The eastern half of the route is relatively flat.

Along the 2.4-kilometer (1.5-mile) project route, ground surface elevations range from less than 6 meters (20 feet) near the eastern end at the Marina Boulevard to about 60 meters (200 feet) just west of the Hwy 1/Hwy 101 interchange. At the western end, the project route is on high ground, south of the bluffs, depressing to the lower elevations after crossing the eastern bluff.

The western half of the alignment is on natural terrain. The flat eastern half of the route is generally located on fill. Extensive fill placement along the shoreline occurred in the 1960s (Harlan Tait, 1998).

A map of 'Seismic Hazard Zones' for the City and County of San Francisco (CDMG, 2000) indicates landslide potential at two locations of potential interest to the project: (a) on the Western Bluff, starting from near the western end of the high-viaduct, going westward; and (b) a narrow zone along the northern edge of the Eastern Bluff, about 61 meters (200 feet) from the present Doyle Drive alignment. Neither of these two locations is close enough to the proposed alignment to be of significance to the project. Fractured rock is evident in the slope face in the Eastern Bluff down slope from the National Cemetery. Also evident in the same general area is a slide repair immediately north of the west abutment of the low-viaduct.

Except for the Crissy Field area on the edge of the Bay, the rest of the area, unless built up, is vegetated, including many tall trees and wooded areas.

There are three main watersheds within the Presidio comprising an area of approximately 550 hectares (1,360 acres): the Lobos Creek watershed; the western coastal watershed; and the San Francisco Bay watershed. The 354-hectare (875-acre) San Francisco Bay watershed drains to the San Francisco Bay and is of particular interest to the present project (Baseline, 2004). Open spaces in the project generally drain by overland flow and open channels; in urbanized areas, the storm water and sewer are carried by the Presidio Storm Drain System. The Doyle Drive corridor crosses all of the 14 drainage basins that make up the San Francisco Bay watershed. The Doyle Drive alignment crosses Tennessee Hollow, the largest of the basins, in the northern portion of the watershed. The current flow estimates for 100-year storms in the Tennessee Hollow basin are in the range of 240 to 289 cfs. The Presidio Trust Management Plan (PTMP) directs the Trust to:

"Restore a functioning stream ecosystem with associated riparian and wetland habitats; improve the quality of freshwater flows into Crissy Marsh; improve management practices in the surrounding watershed; protect and enhance cultural and archaeological resources; provide recreational, educational, and interpretive opportunities; and adapt existing infrastructure to support the restoration".

5.3 PRIOR LAND USAGE

The Presidio site was initially established in 1776 by the Spanish as a military installation, but was only slightly used for a long time. The site came under Mexican rule in 1822 but was not significantly developed until U.S. occupation in 1848. From 1848 to 1994 the Presidio served as a mobilization and embarkation point during several overseas conflicts. It has also served as a medical debarkation center and for coastal defense for the San Francisco Bay Area. Maintenance and repair of vehicles, aircraft and base facilities are some of the industrial operations formerly associated with the Presidio. The Crissy Field area was originally a marshy area and was used as a dumping ground for debris and refuse from the Presidio and other sources. It was filled in 1912, used as a racetrack and as the site for the Panama Pacific Exposition in 1915 and finally as an army airfield until 1974.

The construction of the Doyle Drive connection to the Golden Gate Bridge (presently part of Hwy 101) was completed in 1936.

In 1994 the U.S. Army closed its operations at the Presidio and transferred the property to the National Park Service. Per the Base Closure and Realignment Act, the site was required to undergo contaminated site investigations. Several extensive studies have been conducted for various areas at the site to address the potential contamination of the soils and the groundwater in these areas.

5.4 NATURAL AND MAN-MADE FEATURES OF ENGINEERING AND CONSTRUCTION SIGNIFICANCE

5.4.1 Natural Features

The major natural feature of significance to the project is the Tennessee Hollow drainage, which crosses the alignment in the area of Halleck Street. As noted earlier in Section 5.2, Tennessee Hollow is the largest basin crossed by the proposed alignment. It is also one of the Presidio's primary drainages. In addition, Tennessee Hollow serves as the upland drainage system for the former wetlands at Crissy Field, which, as part of the PTMP has been reestablished into what is now known as Crissy Marsh.

The PTMP also calls for the riparian corridor along Tennessee Hollow to be restored to reestablish a more natural drainage system to feed the restored wetlands at Crissy Field. This will involve fill removal, storm drain removal and revegetation with native species. Historic vistas will be restored by clearing non-historic forested areas.

The restoration of Tennessee Hollow takes on added significance due to its association with the former wetlands at Crissy Field. The wetlands restoration is part of the plan to upgrade the Crissy Field Area, to its original appearance, rehabilitation of historic buildings, restoration and preservation of native plant communities in the serpentine outcrop and extension of the native dune system along the bayshore.

Another feature of importance is the Eastern Bluff. Per the PTMP, "The coastal Bluffs will be preserved as the wildest part of the Presidio landscape, where geology, wind and water shape the land and make the forces felt." The PTMP also states: "The Presidio contains some fragile geologic resources, including the Colma Formation dunes, and the serpentine outcrops and bluffs at Inspiration Point and south of Crissy Field. The Presidio Trust would protect and monitor unique geologic and subsurface hydrologic resources and functions …..". Removal of the subsurface materials is a required activity for tunnel construction for Alternative 5 – the impact is significant, but unavoidable. The Natural Environmental Study for the project (ESA, 2001) indicates that the existing plants on the bluff slopes include coast rock cress, Franciscan thistle, San Francisco wallflower and San Francisco gumplant. All of these plants are considered 'Special Status Species' in the report. In addition, the gumplant and the thistle occur in serpentine soils; thereby they are marked for resource management by the PTMP by minimizing disturbance.

A further factor deserving attention is the vegetation on the slopes of the bluff downslope from the National Cemetery. According to the project Hydrology Report (Baseline, 2001), the vegetation is 'indicative of near-

surface high moisture and/or saturated conditions'. It is thus inferred in the Hydrology and Water Resources Report (Baseline, 2004) that the bluffs must be supported by a continuous source of groundwater which may be disrupted by the construction of the tunnel unless special design measures are taken.

5.4.2 Man-Made Features

Besides the two existing viaducts, other man-made features of significance in close proximity to the project are:

- The Main Post Buildings 104, 105, 106, 107 near the Main Parade Ground;
- The San Francisco National Military Cemetery;
- The Batteries on the Eastern Bluff
- The Commissary and PX Buildings
- The Gorgas Warehouses
- The Mason Street Warehouses
- The Crissy Center
- Halleck Street Buildings 201, 204

Most of the features are shown in Figure 3.

SECTION 6: GEOLOGY

6.1 REGIONAL GEOLOGY

The project site location falls in the Coast Range geomorphic province, which is characterized by a series of nearly parallel mountain ranges that trend obliquely to the coast in a northwesterly direction. This trend in direction is emphasized by the alignment of the fault zone, the fold axes and geologic units.

The San Francisco Bay Area as it is known today came into existence in mid- to late Pleistocene era. Like all other areas of California, a long record of seismic activity characterizes the geologic history of the San Francisco area. In addition, the area has been strongly influenced by changes related to the Pleistocene glaciers and the presence of the San Francisco Bay trough, resulting in variable thicknesses of recent deposits of soft to medium stiff Holocene clays (Bay Mud), an older, stiffer Pleistocene clay (Old Bay Clay) and sand deposits.

6.2 SITE SPECIFIC GEOLOGIC CONDITIONS

Within the Doyle Drive corridor, the varying topography along the 2.4-kilometer (1.5-mile) project route is the result of a diverse range of geologic conditions. Shallow bedrock of the Franciscan Formation, a heavily folded and sheared assemblage of graywacke, shale, sandstone, chert, and serpentinite generally dominate the higher elevations at the western end. Overburden soils in these regions are made up of artificial fill, slope debris and ravine fill and/or by the Colma Formation, an unconsolidated to weakly consolidated fine- to medium-grained sand unit with clay beds. The lower elevations on the eastern side reflect an estuarine deposition environment, such as in the Crissy Field area, where the bedrock is significantly deeper. The surficial soils in this area are dune and beach sands and soft clayey silt layers. These soils are generally underlain by the Colma Formation, which generally overlies the bedrock. Significant thicknesses of Bay Mud and Old Bay Clay have not been found in this area (Goldman, 1969). A site geological map excerpted from the U.S. Geological Survey San Francisco North Quadrangle by Schlocker (1974) is provided in Figure 4.

A particularly significant feature of this area noted by Harlan Tait Associates (1998) is the past presence of an extensive tidal marsh separated from the Bay by a beach and dunes. The area, which extends from Crissy Field in a southeasterly direction towards Lombard Street and underlies Doyle Drive east of the Post Commissary Building, was filled in 1912. The approximate extents of this “Historic Marsh Area” were identified from old photographs by Harlan Tait Associates (1998) as part of the investigations performed for the Crissy Field Restoration Project and are presented in their report as Figure 1. Soil borings within the area show very soft to soft-greenish gray clayey silts heterogeneously interlayered with thicker Beach/Dune Sand layers. This is attributed to the active tidal marsh and beach depositional environment. The hydraulically placed fill consists of loose sands with variable amounts of silt and clay.

6.2.1 Bedrock

The basement rocks in the project area belong to the Franciscan Formation. Lithologically, the Franciscan Formation is dominated by graywackes, derived from rapid erosion of volcanic highland and deposited in deep marine basins in the late Jurassic to mid-Cretaceous period. The graywackes are composed mainly of quartz and plagioclase feldspar, with a chlorite-mica matrix, which gives it the dark greenish color. The graywackes are interbedded with dark shale and occasional limestone.

All Franciscan rocks have been intruded by ultrabasic igneous rocks like serpentinized peridotite (serpentine). The bedrocks revealed by the borings (see Chapter 7.0) indicate exclusively serpentine west of Station 10+84. Sandstones and shale were encountered east of that point.

6.2.2 Overburden Soils

As may be expected, a wide variety of materials occur above the bedrock, based on the geologic setting. The soils are described below.

6.2.2.1 Artificial Fill (Qaf)

Predominantly dune sand, but includes silt, clay, rock wastes from excavations, manmade debris and organic waste. Hydraulically placed loose sand with variable amounts of finesis present at the site of the former tidal marsh. Hydraulic fill is overlain with gravelly, clayey sand in most of the former airfield areas. The existing fill often contains construction debris, generally supported in a sand matrix.

6.2.2.2 Colluvium

Per the USGS geologic map shown in Figure 4 (Schlocker, 1974), these materials (designated by Schlocker as Slope Debris and Ravine Fill) are present on the surface at the western end of the route to about the Hwy 1/Hwy 101 interchange. The descriptions presented in that report indicate that this formation consists of rock fragments in a sand, silt and clay matrix; generally light yellow to reddish brown. They generally are designated as CL, SM, or SC under the Unified Soil Classification system. The descriptions above generally agree with the findings in the soil borings.

6.2.2.3 Beach and Dune Sand

These materials occur at shallow depths in the lower elevations, in particular, in the Tidal Marsh area. They are generally poorly sorted (e.g., SP), well rounded and medium to coarse grain size. The Beach Sand is divided into Older and Modern units, according to the time of deposition. The Modern unit is generally loose and is present in the upper 3 to 6 meters (10-20 feet). The underlying Older unit is medium dense to dense.

6.2.2.4 Bay Mud

The materials classified as Bay Mud along the project route are not the thick, soft, medium-to high plasticity clays found elsewhere along the Bay and traditionally referred to as Young Bay Mud. The Bay Muds encountered in the project area are typically clayey silts, greenish gray to dark gray, and generally soft, becoming medium stiff with depth. The Bay Mud generally is not expected to occur outside the limits of the Historic Tidal Marsh and where present, is limited to 1.5 to 3-meter (5 to 10-foot) layers near the top 9 meters (30 feet).

6.2.2.5 Colma Formation

The geologic map by Schlocker indicates the presence of the Colma Formation at the surface over some sections of the project route, particularly in the area of the western tunnel, above the bluff. They appear to have been deposited mostly by water and gravity in a variety of coastal environments. Per Schlocker, the Colma Formation materials are light brown to orange fine to medium sand with minor amounts of clay (mostly SP). Subsurface investigations at the present site discussed in the next chapter, however, indicates a significant presence of clayey sands in several borings (GB-4, GB-5, HGB-2).

6.2.3 Groundwater

Groundwater data as encountered in the soil borings both from available data and from the recent Preliminary Investigations are presented in Section 7.0, in the boring logs provided in the appendices, and in the Hydrology and Water Resources Technical Report for the project (Baseline, 2004). Per the Hydrology Report (Baseline, 2004), groundwater is present at depths ranging from near the surface (at El Polin Springs, which is 900 meters south of the alignment) to more than 15 meters (50 feet) below ground surface in the hilly uplands. Groundwater occurs in both unconsolidated sediments and underlying bedrock (essentially in the fractures).

The quality of the groundwater resulting from past activities at the site (e.g. hazardous materials releases) is presented in the Revised Preliminary Site Investigation, (Baseline, 2004). Saltwater intrusion into shallow groundwater appears to be restricted to the near-shore areas (Baseline, 2004).

Per the hydrology report (Baseline, 2004), the vegetation on the bluff slopes below Doyle Drive (between McDowell Avenue and the eastern edge of the cemetery) includes several species indicative of near-surface high moisture and/or saturated conditions. Also, per the report, several places on the slope currently support relatively dense growth of high water need plants, and indicate the presence of springs. The locations of the vegetations noted above would indicate near surface groundwater flow through the Franciscan bedrock.kb

Additional field investigations and associated laboratory testing were performed on the groundwater regime in the Tennessee Hollow and Crissy Field areas. An upward gradient was noted in the vicinity of Tennessee Hollow. There appears to be three distinct aquifers in the Tennessee Hollow area, separated by thin layers of aquitards. Other findings and additional details are described in detail in the project hydrology report (Baseline, 2004).

6.3 TECTONIC SETTING

According to plate tectonic theory, California's mountains have been produced chiefly by the collision of the east moving Pacific plate and the west drifting North American plate. In fact, interpretations of Coast Range geology have been greatly affected by plate tectonics. A dominant feature of the Coast Range province tectonic landscape is the San Andreas Fault. The known length of the San Andreas is about 1,200 kilometers (750 miles). It was the source of the great 1906 San Francisco earthquake and several significant earthquakes since then.

California's reputation as earthquake country is based on the earthquakes generated by the movements along the numerous faults across the state. It is believed that the major faults mapped in the Bay Area are all parts of the San Andreas Fault system. This system consists of a number of northwest-trending faults that have developed in a similar tectonic stress regime: a right-lateral transform plate boundary. The activity of these faults (e.g., magnitude of earthquakes, types of ground motions produced, earthquake return periods) has been studied extensively over the years and serves as the basis for the current methods of earthquake resistant design.

A Seismic Hazard Map for the state of California has been developed by Caltrans (Caltrans, 1996), with a companion report for use in seismic design, primarily for bridges. The map presents all of the known faults and their associated Maximum Credible Earthquakes (MCEs). The MCEs represent moment magnitudes (the older measured magnitudes are usually in the Richter scale). Also presented in the map are contours of Peak Rock Accelerations associated with the MCEs, for the entire state. The companion report provides additional information on the characteristics of the faults, selection criteria for the faults and other pertinent information. Per the Caltrans companion report (Caltrans, 1996), a deterministic approach based on MCEs is preferred, since 'earthquake cycles are conceptual and not fixed'. Geological and historical records have revealed that earthquakes do not occur in cycles like clockwork and the nature of the timing of future earthquakes is fundamentally unknown'. No attempt is thus made in the companion report to evaluate the

recurrence intervals of the earthquakes. Design recommendations are based on a deterministic approach using the MCE as is the current state-of-practice for standard Caltrans projects.

The portion of the Caltrans Hazard map that covers the project site is shown in Figure 4. The map shows the project site to be:

- 9 to 10.5 kilometers (5.5 to 6.5 miles) from the San Andreas Fault
- 13 to 14.5 kilometers (8.0 to 9.0 miles) from the San Gregorio Fault
- 19.5 to 21 kilometers (12.0 to 13.0 miles) from the Hayward Fault.

No known faults cross the project site. Referring to Figure 5, the Peak Rock Acceleration at the project site is shown as 0.5g.

A brief description of the three faults governing the seismic risks at the Presidio site is presented below.

6.3.1 San Andreas Fault

The massive San Andreas Fault is clearly the dominant presence from the project seismic design standpoint. Over 1300 kilometers (800 miles) long, the San Andreas Fault slices through California from the Mendocino coast to the desert near the Salton Sea. Four moderate to large earthquakes have occurred over the last 160 years (historical period):

Date	Magnitude (Richter)
June, 1838	7
10 October, 1865	6.3
18 April, 1906	8.3
17 October, 1989	6.9

In the great San Francisco earthquake of 1906, the San Andreas Fault is reported to have ruptured a total length of 430 kilometers (270 miles), from Shelter Cove to San Juan Bautista. An average fault displacement of 4.9 meters (16 feet) has been reported. Subsequent evaluations indicate a Moment Magnitude of 7.9 for the great San Francisco earthquake of 1906.

In the Seismic Hazard Map (Caltrans, 1996), the San Andreas Fault is shown as seven segments, each with a different associated MCE. The segment nearest to the site is designated as San Andreas/N. It is a strike-slip fault (as are all the other segments except one) and is associated with a maximum credible earthquake of 8 (Moment Magnitude). Additional information on this fault may be found in Green and Kennedy (1988), Mualchin and Jones (1992), and Jennings (1994).

6.3.2 Hayward Fault

The Hayward Fault is approximately 96 kilometers (60 miles) long. It branches off from Calaveras Fault near Mt. Misery, southeast of San Jose, trending northwesterly along the west side of the East Bay hills to San Pablo Bay. North of San Pablo Bay, the Rodgers Creek-Healdsburg Fault follows the same trend as the Hayward Fault, but is considered a separate fault in the Seismic Hazard Map. Per the Seismic Hazard Map, the Hayward Fault is a strike-slip fault and has a maximum credible earthquake of 7.5 (Moment Magnitude).

Three major earthquakes are attributed to the Hayward Fault:

Date	Magnitude (Richter)
10 June, 1836	6.8
26 November, 1858	6.1
21 October, 1868	6.8

Additional pertinent information on this fault may be obtained from Slemmons and Chung (1982), Mualchin and Jones (1992) and Jennings (1994).

6.3.3 San Gregorio Fault

The San Gregorio fault is another northwest trending fault, west of the San Andreas and approximately 260 kilometers (160 miles) long. In the Seismic Hazard Map, it is associated with an MCE of 7.5. The San Gregorio fault is also designated as a strike-slip fault. No major earthquake (e.g., greater than Magnitude 6.0) has been recorded on this fault in historical times. Additional pertinent information on the fault is available in Greene and Kennedy (1988), Mualchin and Jones (1992) and Jennings (1994).

6.3.4 Probabilistic Analyses

The probability of occurrence of a major earthquake at the project site is of interest. A comprehensive study by USGS (USGS, 1999), reports the findings of a Working Group (WG99) consisting of dozens of scientists representing a wide cross-section of the earth science community. The primary approach of this working group was to develop a comprehensive regional model for the long term occurrence of earthquakes, founded on geological and geophysicalobservations and constrained by plate tectonics. The study pertains to the San Francisco Bay Area extending from Healdsburg in the northwest to Salinas in the southeast and reports the probability of occurrence of a major earthquake within the next three decades (2000-2030).

For the purposes of the study, a major earthquake is defined as an earthquake with a moment magnitude of 6.7 or greater, It is postulated that based on the experiences of the Northridge (M=6.7) and Kobe (M=6.9) earthquakes that earthquakes of this size can have a disastrous impact on the social and economic fabric of densely urbanized areas. Probability of occurrence of a major earthquake is reported for seven major fault systems, which include the three faults discussed earlier. The probability of occurrence of a major earthquake before 2030 on the three faults is estimated as follows:

San Andreas Fault	0.21
Hayward Fault	0.32
San Gregorio Fault	0.10

It is also estimated that there is a 0.70 probability of a major earthquake occurring in the San Francisco Bay Area before 2030.

6.4 CONCLUSIONS

The following conclusions may therefore be made about the seismic characteristics of the project site, based on standard procedures (deterministic) used in Caltrans projects:

- There are no known faults crossing the project site;
- The San Andreas/N fault with an associated MCE of magnitude 8 is the controlling fault for design purposes; and
- The Peak Ground Acceleration to be used for the project is 0.5g.

The above conclusions have been utilized to develop the design and recommendations in Sections 12 and 13.

Probabilistic analyses indicate a 0.70 probability of a major earthquake (Magnitude greater than 6.7) occurring in the San Francisco Bay Area.

SECTION 7: SUBSURFACE GEOTECHNICAL CONDITIONS

7.1 GENERAL

As indicated earlier, preliminary subsurface investigations were performed for this present phase of the project. Pertinent available reports and boring logs were located and reviewed with the aim of establishing for initial design a basic understanding and framework of the soil and groundwater conditions at the site. This information is necessary to evaluate the constructability, cost and environmental impact of the proposed alternatives presented in Section 4.0. A review of the available information revealed significant gaps in information in a few areas where it was judged that additional information would lead to greater accuracy in the cost estimates thereby improving the evaluation of the alternatives. A limited subsurface investigation (Preliminary Investigations) was thus undertaken.

In this chapter, the existing available information and the information obtained in the Preliminary Investigations are discussed in detail, including the limitations. The need for and extent of additional subsurface data acquisition for final design are then discussed.

7.2 EXISTING SUBSURFACE INFORMATION

The locations of all borings for which boring logs were available and utilized for this study are presented in Figure 6 (Boring Location Plan). Applicable logs of test borings are provided in the Appendixes. The findings for each set of borings are discussed below.

7.2.1 Borings for Presidio Road

The most useful existing information is available from the 33 soil borings presented as part of the as-built drawings for the original Presidio Road construction. The drawings were prepared in 1933 for the Golden Gate Bridge and Highway District – they most likely were part of the construction of the Golden Gate Bridge. The borings are unevenly distributed across the project route between the westernmost boring at Station 2+24 and the easternmost at Station 25+54. Boring locations for the Presidio Road Borings are shown in Figure 6. This same set of test borings is included with the as-built drawings of the Presidio Viaduct (high-viaduct).

A major limitation in the Presidio Road boring logs is the absence of blow counts or of any indication in the descriptions of the ‘compactness’ or relative density of the granular soils or the strength and consistency of the cohesive soils. Another limitation is that there is no clear mention of the occurrence of groundwater. However, some assessments of the occurrence of groundwater may be made from the descriptions showing ‘water bearing sands’, which presumably indicate saturated sands.

The information clearly indicates the presence of predominantly serpentine bedrock at shallow depths (less than 9 meters [30 feet]) in the western end of the project route, from Station 2+24 to Station 9+19; the bedrock depth is 3 meters (10 feet) or less to Station 8+23. The ground surface in this zone slopes steeply along the road alignment at a slope of approximately 20 percent. The serpentine bedrock is typically weathered and decomposed near the top, hardening with depth. Also, there is some evidence of sandstone and shale underlying the serpentine layer between Stations 8+78 and 9+19. The overburden soils are generally clays to Station 8+08; between Stations 8+08 and 9+19 there is a preponderance of yellow sands and sandy clays. There is indication of a potential water table, possibly perched, in only one boring in the western end: water-bearing sands are indicated at a Station 8+37 boring at an elevation of 28 meters (91.0 feet).

From Stations 9+60 to 11+25, the terrain is relatively flat. Groundwater (noted as wet or water bearing sand or seepage) is indicated in the boring logs at elevations ranging from 3.2 to 5.1 meters (10.5 to 16.6 feet) or about 6 meters (20 feet) below the ground surface. This may be significant information for liquefaction

potential, depending on the in situ density or compactness of the sands. Water bearing sand layers 7.5 meters (25 feet) thick or more occur at three locations between Stations 9+60 to 11+25.

The depth to bedrock is at least 15 meters (50 feet) below ground surface in the western end of this section, getting deeper going eastward. There is no record of bedrock even to a depth of 29 meters (95 feet) below ground surface (elevation –18.3 meters (-60.2 feet)) in a boring on the northern edge of the road at Station 10+01. It is perhaps significant that 'hard sands' (possibly Colma Formation) are indicated around elevation –10.4 meters (-34 feet) in the only two borings that were drilled beyond this elevation. This may be a suitable bearing layer for piles.

Beyond Station 11+25 the ground surface rises at a slope of approximately 10 percent to Station 12+29. The depth to bedrock (shale) is 17.7 meters (58 feet) at Station 11+66 and only 3.4 meters (11 feet) (sandstone) at the top of the slope. A soil layer described as 'hardpan' was encountered at a depth of 6 meters (20 feet) in Boring 15 at Station 11+98. There are no indications of water bearing sands in the logs for any of the three borings in this zone.

Soil borings are conspicuously absent between Stations 12+29 and 15+55. Ground elevations in this area range from 35 meters (115 feet) in the west to 23 meters (75 feet) to the east. The boring at Station 15+55, which is at the top of the abutment for the low-viaduct, primarily shows clays and sandy clays to 18.8 meters (60 feet) depth, with possibly decomposed bedrock at the bottom of the boring.

There are four borings between Stations 18+90 and 25+30, which almost cover the length of the present low-viaduct. The borings are 15 to 18 meters (50 to 60 feet) deep and all with ground elevations within 1.5 meters (5 feet) of each other (2.7 to 4.2 meters [9 to 14 feet]. There is no indication of bedrock or groundwater in any of the borings. However, based on the occurrence of groundwater at shallow depths in the Commissary Building borings, which are within a short distance and at similar elevations, the presence of groundwater is likely. The soils as revealed by the borings are predominantly sands; there is some clay, sandy clay and loam layers also present.

7.2.2 Borings for Commissary Building (Building 610)

The Post Commissary Building is located approximately 30 meters (100 feet) north of the edge of the existing Highway 101, between Stations 17+00 and 18+00. As-built plans for the building dated June 1987 show logs of 10 soil borings, ranging in depth from 18.4 to 33.2 meters (60 to 109 feet).

Very soft-to soft silts and silty sands were generally encountered near the tops of the borings to depths ranging from 3 to 8.5 meters (10 to 28 feet) below ground surface. There is evidence of peat and other organic material within this soft material. The soft material layers are most likely representative of the active tidal marsh and beach depositional environment. The Commissary building is entirely within the boundary of the historic tidal marsh discussed earlier (Harlan Tait, 1998).

Dense to very dense sands and silty sands were encountered below the soft layers to depths of 18.3 to 21.3 meters (60 to 70 feet). Blue-gray silty/sandy clays were encountered below the sands, which in turn are underlain by shale, generally highly weathered, at depths ranging from 20 to 29 meters (65 to 95 feet). In two borings, sandstone was encountered below the shale, at depths ranging from 22 to 30 meters (72 to 100 feet). Groundwater was encountered 0.9 to 1.5 meters (3 to 5 feet) below ground surface (an artesian condition with water flowing to top was recorded in boring 2F-84-114, 30 meters (100 feet) away from the edge of the highway.

The Commissary Building is within 30 meters (100 feet) and at the same elevation as the part of the low-viaduct nearest to it. The soil conditions along the route should therefore be generally similar. This indicates that saturated soils will be encountered between Stations 17+00 and 18+00 at about elevation 3 +/- 1.5 meters (10 +/-5 feet). Also, a soft silt/silty sand layer is present in the top 3 to 6 meters (10 to 20 feet). Perhaps the soft layer is indicated by the 'black sandy loam' in the boring at Station 18+90 in the 'Presidio Road' borings. There is, however, no indication of groundwater in the log for that boring.

7.2.3 Boring for Buildings 207 and 231

A sizeable site investigation was carried out by IT Corporation in 1997 for tank removal activities in the vicinity of Buildings 207 and 231. The buildings are located on opposite sides of the Highway 101 ramps near Station 22+00. Thereby, the soil conditions at the project route are conveniently interpolated.

The investigations included visual examination of soil samples retrieved by hand auger, geoprobe and hollow stem augering, as well as Cone Penetrometer Testing (CPT). Groundwater measurements were made in the borings along with geoprobes as they occurred. Based on the above, the report identifies six hydrostratigraphic units. A summary of the six units as presented in the report is given below:

- A vadose zone approximately 0.9 to 2.6 meters (3 to 8.5 feet) thick consisting of fill material;
- A shallow, saturated sand layer underlying the vadose zone. This unit is laterally discontinuous across the site, approximately 0.3 to 3 meters (1 to 10 feet) thick, and consists of fill, sand, silty sands, and sandy silts;
- Saturated silt layer approximately 0.8 to 3.0 meters (2.5 to 10 feet) thick and consisting of soft silt, clayey silt, and sandy silt. The layer is classified as 'Bay Mud', though it is significantly silty compared to the standard Bay Mud;
- An intermediate sand layer underlying the Bay Mud, which is semiconfined and laterally continuous across the site. The layer is approximately 1.4 to 3.0 meters (4.5 to 10 feet) thick and consists of sands, silty sands, and sandy silts;
- An intermediate layer of a similar 'Bay Mud' as above underlying the intermediate sand. The unit is laterally continuous across the site, ranges in thickness from 0.8 to 3.0 meters (2.5 to 8.5 feet) and consists of soft silt, clayey silt, and silty sand; and
- A deep sand layer composed of well-graded sands and silty sands. The thickness of this unit was not determined because it extends beneath the depth of exploration (9 meters [30 feet]).

Groundwater was encountered 1.5 to 3.0 meters (5 to 10 feet) below the ground surface.

An examination of the CPT data indicates that the Bay Mud is predominantly medium stiff having shear strengths between 24 and 48 kPa (0.5 and 1.0 ksf). All of the sand layers are predominantly medium dense (standard penetration test [SPT] blow counts between 10 and 30 blows per foot). There is no obvious evidence of soft Bay Mud or loose sands in the investigation. However, this location is within the inferred limits of the historic tidal marsh: therefore, localized soft layers should be expected in the upper 6 to 9 meters (20 to 30 feet).

Borings under the low-viaduct indicate approximately 1.8 meters (6 feet) of fill and shallow sand, 3.7 meters (12 feet) of shallow bay mud, 1.5 meters (5 feet) of intermediate sand and 1.8 meters (6 feet) of intermediate bay mud overlying the deep sand.

Soil and groundwater samples collected at both the Building 207 and 231 sites indicate the presence of gasoline, diesel, fuel oil and benzene, toluene, ethylbenzene, and xylene (BTEX) compounds.

7.2.4 Borings for Letterman Army Institute of Research

Seven exploratory borings were drilled in 1971the Letterman area for construction of the U.S. Army Hospital and the Institute of Research. The boring locations and the soil descriptions are presented in the as-built drawings dated May 1971. The boring locations were 120 to 240 meters (400 to 800 feet) from the edge of Richardson Avenue around Station 27+70. The nearest three borings (120 to 180 meters [400 to 600 feet] away) were auger holes, drilled to 4.7 meters (15.5 feet) depths. The remaining four borings were Denison or Pitcher tube holes and ranged in depths from 20.4 to 53.9 meters (67 to 177 feet).

The soils in the three nearest borings are all sands, silty sands or clayey sands. Groundwater was encountered at 3.0 to 4.1 meters (10.0 to 13.5 feet) below ground surface. The same general soil conditions

are present in the borings farther away - the first occurrence of clay is at 39.6 meters (130 feet) depth. There are no obvious loose or soft layers anywhere in the borings. Also, bedrock was not encountered even at 54 meters (177 feet) depth.

Additional subsurface information has become available from the more recent geotechnical investigation report for the proposed Letterman Digital Center (Treadwell and Rollo, 2001). The report provides 13 new borings ranging in depth from 9.7 to 20 meters (32 to 61 feet). The subsurface information generally agrees with the earlier reports. One significant difference is the presence of very soft to soft interbedded sand and clay layers to a maximum depth of 5.1 meters (17 feet) in the nearest borings, about 60 meters (200 feet) from Highway 101. Groundwater was encountered 0.6 to 2 meters (2 to 6 feet) below ground surface in the borings nearest to the proposed alignment.

7.2.5 Borings in Building 637 Area

Several borings were drilled in the area of Building 637 (Montgomery Watson, 1994). The building has since been removed. All the borings were drilled below the bluff at elevations less than 4.6 meters (15 feet). In the two nearest borings (just at the toe of the bluff), groundwater was encountered 2.4 to 2.7 meters (8 to 9 feet) below the ground surface and serpentine bedrock was encountered 4.3 to 4.6 meters (14 to 15 feet) below ground surface. The overburden soils were predominantly sandy soils underlying 1.1 meters (3.5 feet) of gravel fill at the surface. The sands were loose to about 3 meters (10 feet) in depth, then medium dense. The adjacent edge of the highway is at an elevation of approximately 26 meters (85 feet), 120 meters (400 feet) away (near Station 13+70). The borings are therefore considered to be not representative of conditions along the project route.

7.2.6 Borings for Crissy Field Restoration

An extensive geotechnical investigation was performed by Harlan Tait Associates as part of the improvements at Crissy Field, including the tidal marsh restoration. Their findings and conclusions are presented in a comprehensive report (HTA, June, 1998). The focus of their study area is slightly outside the project site area. However, several important pieces of information and trends may be extracted to our advantage from this study.

The field investigations consisted of drilling and sampling 11 test borings, 12 hydraulically driven sample probes, and 15 multiple-aligned test pits/trenches; and the performance of hydraulically advanced CPT at 6 locations. Pertinent subsurface information (soil borings, CPT, etc.) from several other sources is included in the report. Also as part of the investigation, a review was made of historic maps, photographs, reports and construction documents of interest from various organizations, and stereoscopic aerial photographs covering the site.

The historic development of the site is covered in some detail in the report. It is noted that the Crissy Field site consisted of a tidal marsh separated from the Bay by a beach and dunes, which exposed at low tide primarily as sloughs and mudflats (see Section 6.2). The active tidal marsh and beach depositional environment resulted in heterogeneously interlayered units of Bay Mud and Beach/Dune Sands. This is well borne out in the Building 207 / 231 borings.

The borings nearest to the project route are 45 to 120 meters (150 to 400 feet) north of the project route, approximately from Station 18+50 to Station 22+10. The boring depths range from 6 to 9 meters (20 to 30 feet) and indicate loose sandy soils and soft clayey silts (Bay Mud) to up to 9 meters (30 feet) in depth. This is generally consistent with the fact that all of the borings are within or close to the inferred limits of the historic tidal marsh. The softer layers are underlain by medium dense to dense older beach/dune sands. The depths to groundwater as revealed by these borings ranged from 1.5 to 2.1 meters (5 to 7 feet) below ground surface.

7.3 PRELIMINARY INVESTIGATIONS

An examination of the available existing reports and boring logs revealed a few areas where limited additional information would contribute substantially to more definitive preliminary design considerations. The accuracy of construction cost estimates for the various alternatives would thereby be greatly improved. Three areas were identified for additional investigation:

1. Station 17+00 to 23+00 Available data indicated that there are heterogeneously interbedded layers of soft clayey silt and loose sand near the surface in the area east of Station 17+00. The characteristics of these deposits indicate strong potential for liquefaction. The depth to which these deposits extend cannot be established with confidence, though indications are that they do not occur beyond 6 to 9 meters (20 to 30 feet) depth. The Presidio Parkway Alternative would construct an at-grade concrete box tunnel through this section of the corridor. Therefore, as discussed later, in Section 13, the determination of the depth of the liquefiable zone help greatly in selecting a suitable construction procedure for the stabilization of these soils, resulting in more accurate bases of construction cost estimates;
2. East of Station 23+00 Pertinent existing subsurface information beyond Station 23+00 was scanty. Two of the replacement alternatives (2 and 5) involve structures (either mainline or ramps) in this portion of the corridor. Most of the available information was from borings 120 to 180 meters (400 to 600 feet) from the proposed alignment. Additionally, there was no information on sand relative densities or clay shear strengths even in these borings. Applicable additional information in this area would therefore be extremely valuable for a meaningful assessment of subsurface conditions as required for the pile foundations for the aerial structures or for tunnel design as appropriate; and
3. East Bluff Area There was essentially no subsurface information between Stations 12+29 and 15+55. The limited available information indicates construction of the tunnel proposed through this area in the Presidio Parkway Alternative will require cutting through rock for at least a portion near the western end. Information regarding the depth to bedrock as well as the quality of the rock encountered (e.g., weathering, strength, rippability) will help significantly in reducing the uncertainties in the construction cost estimates.

In response to the identified need for better data along the alignment in these areas, Taber Consultants[1] was retained to perform additional investigations (referred to hereafter as the Preliminary Investigations). Taber drilled nine soil borings with associated laboratory testing and performed seven CPTs. Three of the borings were selected for use as hydrologic borings and were fitted with piezometers under the direction of Baseline Environmental Consulting, Inc. CPT and boring logs from the Taber report are presented in Appendix B.

Findings from the Preliminary Investigations are the following:

1. Soft, potentially liquefiable layers were encountered to a maximum depth of 6 meters (20 feet) for most of the area in the corridor east of Station 20+00 (Boring Nos. GB1-GB3, HGB1-HGB3, and CPT1-CPT6). In the segment beyond Station 23+50 towards Marina Boulevard, the deposits may extend up to 11 meters (35 feet) deep (Boring HGB1). Some tendency towards deeper soft deposits to the north (towards the Bay) is apparent;
2. The liquefiable deposits are associated with cone tip resistances ranging from 0.3 to 1.5 megapascal (MPa) (3 to 15 ton/square foot [tsf]) and are mostly classified as ‘sensitive fine grained’. Corresponding samples from soil borings indicate SPT blow counts ranging from 0 (push) to 5 blows per foot. Grain size distribution curves generally show slightly clayey silts with some fine sand; and
3. The two borings drilled to investigate the depth and quality of the rock in the East Bluff area near the western end of the tunnel indicate:

1 “Summary of Exploration and Testing, Doyle Drive Reconstruction Project, San Francisco, CA,” Taber Consultant, March 2001.

- In Boring GB6 (Station 13+10), brown sandstone with siltstones is present 3.5 meters (11.5 feet) below ground surface. The rock is noted as being very intensely weathered and intensely fractured. Unconfined compression tests were not feasible for the samples recovered; point load tests performed instead on three samples ranging in depth from 3 to 5 meters (13 to 16 feet) below ground surface indicated uniaxial compressive strengths of intact rock ranging from 29 to 37 MPa (4,200 to 5,350 pounds/square inch [psi]). Rock mass strengths will be lower due to the weakening effects of joints and bedding discontinuities.
- In Boring GB5 (Station 14+30), bedrock is present at around 18 meters (59 feet) below ground surface. The rock is described as intensely weathered and intensely fractured 'metasedimentary' rock. Point load test performed on one sample from 18 meters (59 feet) below ground surface indicated a uniaxial compressive strength of 3.6 MPa (520 psi.)

The rock depths and rock types are generally consistent with the conditions logged in the Presidio Road borings east and west of the new borings.

7.4 ASSESSMENT OF AVAILABLE INFORMATION

The available geotechnical information presented above was reviewed in conjunction with the geological information discussed earlier in Section 6.0. The subsurface profile shown in Figure 7 was thus developed to summarize the subsurface conditions along the project route as input for preliminary design and cost estimates.

The following is an assessment of the general quality and reliability of the available geotechnical data as they pertain to the preliminary design of the proposed tunnel and foundation elements for the Doyle Drive Environmental and Design Study:

- The information on bedrock depths and rock types is generally adequate;
- The information on the depth to groundwater is generally adequate;
- The new information clearly confirms the presence of relatively thin layers of soft clayey silts and loose sands from around Station 17+00 to Station 28+00. Based on the boring and CPT locations, the soft and loose conditions are attributed to the depositional environment of the historic tidal marsh as delineated by HTA from old aerial photographs;
- The data appears to be quite definitive indicating that there are no thick soft Bay Mud deposits and no significant deposits of old bay clay along the project route. In fact, except for the top 6-11 m (20-35 ft) of historic tidal marsh deposition, there is a preponderance of sandy soils, generally dense but sometimes medium dense;
- There is no sand density information in the high-viaduct area between Stations 9+60 and 11+25; groundwater in this area was encountered about 6 meters (20 feet) below ground surface, and relatively thick sand layers exist just below the groundwater. The sands would be potentially liquefiable if they are found to be loose or even moderately dense.

7.5 ADDITIONAL SUBSURFACE INFORMATION FOR FINAL DESIGN

Further subsurface investigations will be required to complete the final design of the adopted configuration. Caltrans Guidelines for Foundation Investigations and Reports (Caltrans, 2000) require that the field and laboratory work performed for final design of a structure meet three basic requirements:

- Soil and rock information shall be adequate to cover each and every support location in both longitudinal and transverse directions;
- Depths of soil and rock investigation shall be sufficiently deeper than those of proposed pile tips and footing bottoms; and

- Soil and rock information shall be adequate for foundation design and construction.

The above implies adequate information is required for each pile cap and the two abutments for the viaduct(s). The criteria above do not specifically address the requirements for a tunnel. However, they are discussed herein to satisfy the requirements of adequacy for foundation design and construction. Specific locations of borings or CPT probes holes cannot be given until the viaduct bent locations and the tunnel alignments and elevations are finalized. Future geotechnical investigations performed for the tunnels should give particular consideration to gathering additional information for the groundwater regime in the area.

The recommended general scope and type of additional subsurface investigations work for each structure are presented below.

7.5.1 High-Viaduct and Park Presidio Interchange

A boring should generally be drilled at each bent of the northbound and southbound High-viaducts, roughly at the center of the pile cap. Additionally, 2 borings should be drilled for the Park Presidio Interchange overpasses: one in the area of the northbound travelway and one in the southbound area. The depths of borings should be at least 3 meters (10 feet) below the penetrations required by the piles as given in Section 13.0. Soil samples should be recovered by an accepted driven sampling method whose blow counts can be converted to equivalent SPT blow counts. Laboratory tests on select samples should include density, moisture content, grain size distribution for granular soils, and shear strength and Atterberg limits for cohesive soils. Where piles bear on bedrock, continuous rock cores to at least 3 meters (10 feet) into rock should be obtained and selected samples tested for compressive strength.

7.5.2 Low-Viaduct

If replacement of the low-viaduct is required, as in Alternative 2, the requirements for field investigation and laboratory testing for the low-viaduct (if applicable) will generally be the same as presented above. However, rock coring is not expected for the proposed stretch of the low-viaduct.

7.5.3 Battery Tunnels

Borings should be drilled at approximately every 30 meters (100 feet) along the length of the tunnels, from the start of the tunnels at approximately 12+70 to the end at approximately 15+30. The borings should be staggered: starting at the eastern edge of the southbound tunnel, followed by one at the western edge of the northbound tunnel, followed by one at the eastern edge. This will provide information of the depths of the potentially sloping rock surface. The borings should be drilled to at least 3 meters (10 feet) below the invert slab of the tunnel for rock and 6 meters (20 feet) for soil. Overburden soil samples and rock cores should be taken at 1.5-meter (5-foot) intervals. As a minimum, testing on overburden soils should be performed for index properties and corrosivity.

The rock cores should be examined for fracture and jointing characteristics (spacing, orientation, aperture, and infilling) and selected samples tested for compressive strength. Water bearing fractures and high conductivity zones should be identified so that the groundwater regime and the flow patterns in the area may be better understood. Other advisable tests include downhole video logging and packer testing at selected locations to assess the hydraulic conductivity characteristics of the rock mass. Selected exploratory boreholes should be fitted with piezometers to provide seasonal longer-term data on groundwater levels. The information thus obtained may be used in the selection of shoring systems and design of special drainage measures that may be required to minimize construction impacts to the pre-existing groundwater regime in the Eastern Bluffs to protect the vegetation on the bluff slopes. The examination of the jointing and fractures in the borehole walls and the collected rock cores thus assumes particular significance.

7.5.4 Main Post Tunnels

Borings for the Main Post Tunnels should be drilled at 60-meter (200-foot) intervals in the same staggered pattern as for the Battery tunnels. The boring program should be supplemented with cone penetrometer tests. CPT probe holes should be driven at approximately the midpoints of the boring locations and should be carried out to roughly the same depths as the adjacent borings.

A primary objective of the subsurface investigation for the Main Post Tunnels is to estimate the extent and depth of the 'soft layer'. Borings should therefore be drilled to at least 6 meters (20 feet) below the bottom of the 'soft layer'. Samples of soil should be taken at 1.5-meter (5 foot) intervals and fairly extensive testing performed on them, particularly to identify the soft zone. The tests contemplated are similar to those discussed for the high-viaduct.

7.5.5 Low Causeways and Girard Road Overpass

A boring or a CPT should be taken at approximately 30-meter intervals. The borings and the CPTs should be approximately 20 meters (65 feet) deep and should be distributed evenly between the northbound and southbound travelways and the Gorgas Avenue off-ramp. In addition, a boring should be drilled for each of the two proposed overpasses over Girard Road. Samples should be recovered every 1.5-meter (5-ft) intervals to a depth of 11 meters (35 feet) and at 3-meter intervals thereafter. Laboratory tests on selected samples should include index tests: density, in-situ moisture contents, Atterberg limits for cohesive samples, grain size analyses for cohesionless samples.

7.5.6 Other Areas

Borings should be drilled at 60 meter intervals along the alignment of the proposed depressed roadways between the Battery and Main Post Tunnels. Borings at 60 meter intervals should also be drilled east of the causeways along the proposed travelways for all the options being considered. The boring depths should extend to at least 1.5 meters (5 feet) below the proposed finish grades of the roadways. At least one sample shall be recovered within 1 ft of the finish grade and one at the bottom of the boring. Index tests and R-value tests should be performed on selected samples.

SECTION 8: CORROSION

Installations below ground surface are vulnerable to corrosion if the surrounding soils and/or groundwater offer a corrosive environment and appropriate measures to resist the corrosion are not taken. The proposed pile foundations, pile caps and tunnels for the project are therefore potentially subject to corrosion, depending on the chemical characteristics of the surrounding soil and groundwater. Caltrans, in their Corrosion Guidelines for Foundation Investigations, Version 1.0 (Caltrans, 2003), defines a corrosive area as one where the soil and/or water has a minimum resistivity of 1000 ohm-cm and additionally, contains more than 500 parts per million (ppm) of chlorides, or more than 2,000 ppm of sulfates, or a pH of less than 5.5. Applicable chemical test results for the project site are not available thereby precluding a ready assessment of potential vulnerability to corrosion for the various foundation elements. However, the proximity of the project site to the San Francisco Bay and the past existence of a tidal marsh strongly suggest a surrounding corrosive environment for the structures. Appropriate testing of the surrounding soil and groundwater is therefore necessary to select suitable corrosion mitigation measures during Final Design. Possible mitigation measures include epoxy coating of the reinforcing bar, design of appropriate concrete mix and stainless steel cladding.

Sampling and testing of site soils and groundwater shall be in conformance with the Corrosion Guidelines for Foundation Investigations (Caltrans, 2003) as described below. Samples for corrosion testing should be taken during the boring and sampling program discussed in Section 7.5 as follows:

From one soil boring between Stations 9+60 and 11+25:

- One sample between 0.3 and 1.5 meters (1 and 5 feet) below the ground surface
- One sample at the water table
- One sample below that at each distinct change of stratum (no more than 4 samples)

From one soil boring between Stations 17+00 and 23+00:

- One sample between 0.3 and 1.5 meters (1 and 5 feet) below the ground surface
- One sample at the water table
- One sample below that every 1.5 to 9 meters (5 ft to 30 feet) below ground surface
- One sample below that at each distinct change of stratum to bottom of boring (no more than 3 samples)

From one soil boring between Station 23+00 and end of route:

- One sample between 0.3 and 1.5 meters (1 and 5 feet) below ground surface
- One sample at water table
- One sample below that at each distinct change of stratum (no more than 4 samples)

All samples shall be at least 2.3 kilograms (5 pounds) of minus No. 8 sieve material. Each sample shall be tested for:

- Chloride content (California Test Method CTM 422)
- Sulfate content (CTM 417)
- pH of soil (CTM 643, Part 3)
- Minimum Resistivity (CTM 643, Part 4)

SECTION 9: SCOUR

Presently, there are no major open waterways or drainages in the vicinity of the project route. A few minor waterways or drainages do exist or may appear in the future due to the restoration of Tennessee Hollow and expansion of Crissy Marsh. However, these features are not expected to be of a scale that would affect the footings or underlying piles of the proposed Causeway. Therefore, any new, properly sited causeway footings or piles are not expected to be vulnerable to damage from scour.

SECTION 10: HAZARDOUS WASTE IMPACT

10.1 GENERAL

As part of the Doyle Drive Intermodal Study (San Francisco Guideway Associateskd, 1996b), Nolte and Associates compiled the report: Summary of Hazardous Materials Cleanup, Environmental Impact Mitigation Review. In the report, four types of potential hazardous materials cleanup work are identified:

1. Residual petroleum and other contaminants left over after completion of the planned Base Realignment and Closure (BRAC) environmental cleanup program;
2. Lead contaminants caused by sand blasting operations in connection with bridge repainting;
3. Hazardous waste material, such as asbestos, generated by demolition of any buildings that may be required for the Doyle Drive reconstruction; and
4. Asbestos from serpentine bedrock occurring in the western half of the high-viaduct.

10.2 RESIDUAL CONTAMINANTS AFTER BRAC CLEANUP

When the Presidio Army Base was proposed for closure by the BRAC task force in December 1988, the U.S. Army was responsible for the overall administration of environmental compliance programs for the Presidio. In 1999, an agreement was reached between the U.S. Army, National Park Service, and Presidio Trust to transfer responsibility for remaining environmental cleanup to the Presidio Trust. The remedial actions are presently ongoing at various locations within the Presidio Army Base. Per the most current Site Investigation Report (Baseline, 2004), it is expected that most remedial actions will be completed before start of construction work for the Doyle Drive Replacement Project. However, it is likely that some residual soil and groundwater contamination will remain, particularly in the areas of the proposed tunnels.

10.3 LEAD CONTAMINANTS CAUSED BY SANDBLASTING

Elevated levels of lead contamination in the soils below the high-viaduct have been reported in a previous study (Apex, 1994). The lead contamination is attributed primarily to windblown contaminants from sandblasting operations during repainting of the truss structure. The study area, extending the entire length of the high-viaduct, is divided into seven subregions. In six of the subregions, the soils were classed as hazardous to 0.15 to 0.3 meter (0.5 to 1.0 foot) below ground surface and contaminated below that to 0.6 meter (2 feet) below ground surface; in one subregion, the soils were contaminated to 1.5 meters (5 feet) in depth. All the soils were granular: sands, and clayey or silty sands and all were found to be above the water table at the time of the study.

The lead-contaminated soils under the high-viaduct have been remediated by Caltrans as part of their completed seismic retrofit work for the viaduct. However, elevated levels of lead contamination as described above are expected to occur in the area of the new alignment. Tests to delineate the level and extent of contamination in this area have not been performed. Nolte's report provides a preliminary estimate of 31,750 tonnes (35,000 tons) of lead-contaminated soil for excavation and disposal – the preferred remediation alternative per the Apex report. The estimate is based on extending the subregions of the Apex report to the area of the new alignment and assuming the depths of contamination for each subregion to remain unchanged.

10.4 HAZARDOUS MATERIAL FROM BUILDING DEMOLITION

The replacement of Doyle Drive will involve the demolition of several buildings. Hazardous material generated from this demolition work will have to be included for hazardous waste cleanup for the project. The primary hazardous material from building demolition is expected to be friable asbestos, which will require appropriate removal and disposal. It is estimated that 80 percent of the buildings within the project

area, including several historic buildings contain asbestos (Caltrans, 1993). Additionally, lead and chemicals such as polychlorinated biphenyls (PCBs), trichloroethylenes (TCEs), and trichloroethanes (TCAs) are known to be potentially present inside several buildings along Doyle Drive. Finally, some of the buildings may have active or inactive storage tanks for fuel oil for space and/or water heating.

10.5 ASBESTOS FROM SERPENTINE

Serpentine was encountered west of Station 9+26 in the old Caltrans soil borings for the high-viaduct at depths ranging from 0.3 to 7 meters (1 to 23 feet). The rock will therefore be encountered during the pile cap excavations and during CIDH pile drilling for the high-viaduct and the viaducts for the Park Presidio Interchange. Serpentine is a source of fibrous asbestos (chrysotile), which is a known carcinogen as well as having a potential for scarring of the lungs. The workers involved should therefore have appropriate training and equipment to detect and handle the material when encountered. All work involving handling and disposal of asbestos, as well as worker health and safety arising out of the serpentine should be performed with strict adherence to all applicable Caltrans, federal and local laws and regulations.

Volume of serpentine to be excavated and removed will depend on the type of bridge designed for the high-viaduct and its associated foundations. Preliminary estimates indicate the volume of serpentine to be approximately 12000 cubic meters. This estimate is deemed conservative, since the material overlying the serpentine has not been excluded and conservative (large) foundation dimensions are assumed.

10.6 ESTIMATED COSTS FOR HAZARDOUS WASTE CLEANUP

Nolte's report (Nolte, 1996) presents an estimated total cost for cleanup of $10 million of which the major portion ($6.6 million) is for the excavation of lead-contaminated soils near the high-viaduct and transportation and disposal at an appropriate landfill (Class I, Class II or Class III, based on the level of contamination). A cost of $136 per tonne ($150/ton) was assumed. The remaining $3.4 million was estimated for unidentified additional hazardous waste cleanup, as discussed in Subsections 10.2 and 10.4. These costs do not include costs for work involving excavation and drilling of serpentine bedrock for foundations in the area of the high-viaduct. Total costs for excavation, transportation and disposal of the serpentine were estimated as $6.5 million for Alternatives 2 and 5 based on a unit cost of $550 per cubic meter.

SECTION 11: OTHER POTENTIAL PROJECT IMPACTS

11.1 GROUNDWATER REGIME IN THE EASTERN BLUFF

The vegetation on the bluff slopes below the existing Doyle Drive (e.g., willows and calla lilies) are indicative of near surface high moisture conditions. Construction of the tunnel upgradient of the bluffs would likely alter or disrupt the groundwater flow downstream. Disruption of the flows could therefore cause potentially adverse impacts to the existing plants that depend on the emerging groundwater from the seeps and springs along the face of the bluffs. Specific mitigation measures may be incorporated into the tunnel design to minimize disruption to groundwater flow around the tunnel. However, even minor alterations to the groundwater flow pattern to the bluffs could have indirect impacts on the biotic resource on the bluffs. Subsurface information indicates that water from rainfall or due to watering of the cemetery infiltrates through the high permeability dune sands, through the Colma formation, and into the fractures of the San Franciscan bedrock. Groundwater conveyance to the fracture could be disrupted if the water table is substantially lowered or the fractures are sealed. The effects of tunnel construction on the groundwater flow in the area and possible mitigation measures are discussed in further detail in the Hydrology and Water Resources Report (Baseline, 2004). Since the water flow in this region is primarily through fractures in the bedrock, knowledge of the fracture patterns is important for designing the necessary mitigation measures to restore the flows.

11.2 TENNESSEE HOLLOW

As noted earlier, plans are underway to restore Tennessee Hollow. As reported in the Hydrology and Water Resources Report (Baseline, 2004), the groundwater flow pattern in the area of Tennessee Hollow is complex (upward gradient), possibly influenced by multiple aquitards within the aquifers. Available information on the aquifer characteristics and groundwater flow patterns are presently not sufficient for a complete assessment of the groundwater flow situation.

Some earlier alternatives (now withdrawn) included construction of deeper tunnels across Tennessee Hollow, causing disruption to groundwater flow in the region with potential adverse effects to the Crissy Field wetlands and restoration of the Tennessee Hollow Drainage. Since none of the alternatives presently being considered include a tunnel that extends well below the water table in the area, it is not expected that the new roadway construction will cause significant disruption to the groundwater flow regime.

11.3 PROTECTION OF STRUCTURES

The impact of construction on several important structures in the vicinity of the proposed construction corridor requires special consideration. Several of these structures are important cultural resources and need to be protected during construction. Construction plans should therefore be developed to adequately mitigate the impact of construction, wherever applicable.

A major cause of distress to structures due to adjacent excavations is differential settlements in the structures. The distress is exacerbated by horizontal strains in the ground associated with the settlement. The degree of damage depends on the size and type of structure and the associated foundation, the proximity and depth of the excavation, and the ground movement. Several investigators have examined the issue of damage to structures due to excavations in close proximity (Wahls (1981), Boscardin and Carding (1989), Clough and O'Rourke (1990)). Analytical procedures have been developed by these investigators, whereby the degree of potential damage may be quantified. Such methods, as applicable, are recommended to plan for adequate measures for protection of the sensitive buildings adjacent to the proposed tunnel construction.

Other causes of potential damage to structures due to construction activities in the proximity are vibrations caused by pile driving or other operations. Due consideration should therefore be given to choosing the pile type and installation procedures to reduce the potential vibrations to acceptable levels.

11.3.1 San Francisco National Military Cemetery

The Presidio Parkway Alternative (Alternative 5) would place the new roadway south of the existing Doyle Drive under Lincoln Boulevard and immediately adjacent to the boundary of the San Francisco National Military Cemetery for a short distance. Special shoring methods that do not require the placement of tiebacks under the cemetery will be required for construction along the cemetery. Alternative 5 will result in construction of the south Battery Tunnel with a minimum clearance of 0.3 meters (1 foot) to the National Cemetery fence.

11.3.2 Batteries

Both the build alternatives (Alternatives 2 and 5) would move the roadway to within 0.7 meters of Battery Slaughter.

11.3.3 Main Post Buildings

Under Alternative 5, the south retaining wall of the southbound roadway would pass within 3 meters of the northernmost corner of Building 106. This will probably require underpinning of the building and continuous monitoring of ground conditions before, during and after construction.

11.3.4 North Halleck Street Buildings

Buildings 201 and 230 are located within the footprint of Alternative 5. The proposed roadway is too close to the above buildings to protect them and thus, the buildings will be removed. Building 204 lies to the south of the Alternative 5 alignment, but will be removed to permit extension of the bluff over the Main Post Tunnels.

11.3.5 Gorgas Warehouses

The Gorgas warehouses will need to be protected in place during the construction of the retaining walls as part of the Girard Road extension..

11.3.6 Existing Viaducts

The existing high- and low-viaducts will require temporary protection, particularly for Alternative 2, when the construction will take place in very close proximity. Thus, staging and construction methods should be carefully considered for construction along the existing viaducts. Appropriate selection of pile type and installation procedures is particularly significant for these areas.

11.3 OTHER POTENTIAL PROJECT IMPACTS

Other potential project impacts include excavation and permanent removal of geologic resources – in-situ rock and soil – from the site, primarily due to excavation for the tunnels, high- and low-viaduct and causeway foundations, and the roadway between the two tunnels. The excavation and removal of serpentine for foundation construction was discussed in detail in Section 10.5. As noted in Section 13.7 (Earthwork), the impact will be mitigated to the extent feasible, by reusing suitable material as onsite backfill as required.

SECTION 12: SEISMIC DESIGN CONSIDERATIONS

12.1 GENERAL

The levels of seismic activity at the project site and the associated sources (faults) are discussed in Section 6.3. The potential consequences of this seismic activity (e.g., seismic hazards) are identified in this section and appropriate mitigation measures or design criteria are noted. The following effects of seismic activity are addressed:

- Fault rupture;
- Soil liquefaction;
- Excessive soil settlements and soil movements;
- Effect of ground motions on superstructure (Design Response Spectra for the viaduct(s)); and
- Effect of earthquake on tunnel design.

12.2 FAULT RUPTURE

The California Seismic Hazard Map (Caltrans, 1996) indicates that there are no identified active faults crossing the project route. Therefore the risk of surface fault rupture along the project route is considered negligible.

12.3 SOIL LIQUEFACTION

Saturated soils subjected to cyclic loading during an earthquake may develop pore water pressures that increase with each succeeding cycle of loading. Under certain combinations of circumstances (soil type and density, water table elevation, level of cyclic loading, number of cycles) sufficient pore pressure may develop, resulting in a consequent reduction of shear strength to essentially zero. This phenomenon is known as liquefaction. Liquefaction is most viable in loose, clean sands and, as may be expected, the potential for liquefaction increases with increasing levels of loading and the increasing number of load cycles (duration of earthquake.) In liquefied zones, foundation elements will lose vertical and lateral resistance. The phenomenon of liquefaction has been widely studied, starting with Professor H. B. Seed of U. C. Berkeley and his co-workers and is still on-going at various universities and research institutes. A comprehensive discussion on liquefaction is presented in Youd et al (2001a,b). A map showing potential areas of soil liquefaction is presented in CDMG (2000).

A preliminary assessment of the liquefaction potential along the project route was made using the procedures described in the FHWA geotechnical earthquake engineering guidelines (Kavazanjian et al, 1997). The following results were obtained, based on the procedure (and using somewhat conservative assumptions):

Depth	SPT Value Above Which Liquefaction Will Not Occur
1.5 meters (5 feet)	16
3 meters (10 feet)	17
6 meters (20 feet)	22
9 meters (30 feet)	26
12 meters (40 feet)	31
15 meters (50 feet)	36

Sand deposits with an SPT value lower than the indicated threshold value, will liquefy under the design earthquake. For sands with significant silt contents (greater than 15 percent), the threshold SPT values reduce by 8 blows per foot to reflect the beneficial effect of fines in reducing the tendency of soil to liquefy. SPT blow counts are not available for major portions of the project route precluding a complete assessment of the liquefaction potential for the entire alignment. Therefore, definitive statements on liquefaction potential are made wherever possible; in other areas, only qualitative statements are made, pointing out the trends (such as the presence of sandy soils at shallow depths and below water) and identifying the data required for a more definitive assessment.

The soils most susceptible to liquefaction are the soft deposits from the historic tidal marsh discussed in Sections 6.0 and 7.0. Additional and more definitive information on these soils was obtained during the Preliminary Investigations. The presence of these soils is noted essentially all through the corridor east of Station 20+00. The deposits are interbedded layers of silty sands, sandy silts and clayey silts. The SPT blow counts range from zero (push) to 5 blows per foot and the CPT cone resistances are generally below 1 MPa (10 tsf). The material is classified as 'sensitive fine grained' in a majority of cases and gradation tests show a predominantly silty material. The maximum depth of occurrence of these soft materials was found to be about 6 meters (20 feet), except for the well boring HGB-1 near the project limits on Marina Blvd, where soft clayey silt (0 blow count) was encountered to 10 meters (35 feet) in depth. This layer appears to extend deeper going north towards the Bay.

Another portion of the project route that merits attention is between Stations 9+50 and 11+30. Caltrans borings from before 1933 indicate 'water bearing sands' at shallow depths (i.e., ranging from 4.3 to 7.9 meters [14 to 26 feet] below ground surface). There is no other information for these soils on the boring log. This portion is close to the edge of the 'historic tidal marsh and the layering is somewhat indicative of a tidal deposition environment – thin peat layers are shown in two of the borings and 1.2 meters (4 feet) of 'black sandy mud' in a third boring. The high- viaduct will traverse this portion of the route. The depth and extent of the liquefiable soils, if present, will be determined based on the subsurface investigations for the Final Design. The information will then be incorporated in the design of the piles as appropriate – most significantly, piles in the liquefied zones will need to be longer and will require stronger sections on the upper portions to resist the lateral loads.

12.4 EXCESSIVE SETTLEMENTS AND SOIL MOVEMENTS

Surface settlements on the order of 25 millimeters (1 inch) or less may occur after an earthquake. Larger settlements are likely in the event of liquefaction occurring in the areas formerly covered by the historic tidal marsh, such as in the area of the Main Post Tunnels and the causeways. Such settlements are of minor consequence to the functioning of the causeways, if measures are taken in the design of the pile foundations by appropriately incorporating the effects of the liquefied zone. Similarly, appropriate measures will be required for the Main Post Tunnel foundations to mitigate the effects of the liquefied layer.

12.5 SLOPE STABILITY

Backfilled slopes above the tunnel box structures should generally not exceed a slope of 1 (vertical) on 2 (horizontal). The slopes should be constructed of compacted granular soils with clay contents not exceeding 10 percent – subsurface information indicates availability of suitable material onsite. The recommended slope is judged to be reasonable for the proposed fill material and with current standard practice. The slope surfaces should be at least one meter (3.3 feet) above the top of the tunnel and should be vegetated to blend in with the surroundings and irrigated. The vegetation, once mature, will further enhance the stability of the slopes.

The steeper slopes along the project route and vicinity are all natural slopes, generally underlain by rock or competent soil.

Liquefied soils (see previous section) may flow downslope in a sloping ground, particularly if the soil is not confined. The ground is essentially flat in the area between Stations 17+00 to 28+00, the area most prone to liquefaction; thus, large downslope movements generated by a liquefied soil mass are not likely.

12.6 DESIGN RESPONSE SPECTRA FOR VIADUCT STRUCTURES

The viaduct(s) will require appropriate seismic design as outlined in the Caltrans Seismic Design Criteria (Caltrans, 2001). The design criteria provide design response spectra (Acceleration Response Spectrum [ARS] curves) that reflect: (a) the magnitude of the design earthquake; (b) the peak bedrock/surface acceleration at the project site; and (c) the subsurface soil profile characteristics at the site. In addition, recommendations are provided in the design criteria to modify the ARS curves to incorporate the effects of the fault type (strike-slip, reverse, etc.); fault proximity; and structures with long periods in deep (greater than 76 meters [250 feet]) soil sites.

Per the design criteria, the maximum credible earthquake (MCE) should be used as the design earthquake – justifications are provided for this requirement as opposed to probabilistic methods based on earthquake return periods. The subsurface soil property variations are reflected in the six assigned soil profile types: five of them reflect ranges of average shear wave velocities over the top 30 meters (100 feet); the sixth one identifies soils that require site-specific evaluation (e.g., liquefiable soils or peat over 3 meters [10 feet] thick). The soil profile types are based on recommendations in ATC-32 (ATC, 1996). A simple method is presented there, whereby a profile type may be assigned to a soil profile with variable soil properties (not readily amenable to type assignment) by prorating the properties over a 30-meter (100-foot depth).

As described in Section 6.3, the San Andreas/N fault segment (MCE=8.0) is the governing fault for this project. It is 9 to 10.5 kilometers (5.5 to 6.5 miles) from the project site; therefore the standard ARS curves must be modified for the proximity of the fault to the site. Since it is a strike-slip fault, and since the rock surface is unlikely to be more than 76 meters (250 feet) deep, no other modifications are required. The peak rock acceleration at the site of 0.5g will be used in the development of the ARS curves.

12.6.1 High-Viaduct

The depth to rock along the high-viaduct alignment varies widely, from 3 meters (10 feet) below ground surface to over 27 meters (90 feet). Based on the procedure in ATC-32 and some simple, conservative assumptions, the soil profile types would be Type C for rock depths less than 15 meters (50 feet) and Type D for greater than 15 meters (50 feet). The recommended ARS Design Curves in the region of the high-viaduct alignment are as follows:

Between Stations 9+45 to 11+70 (See Figure 9):
Soil Profile Type D
ARS Design Curve R3-9 (ATC-32), M_w=8.0, Peak Rock Acceleration=0.5g

- Increase spectral accelerations 20 percent for structural periods >= 1 sec
- No change in spectral accelerations for structural periods <= 0.5 sec
- Linearly interpolate spectral accelerations between 0.5 and 1.0 sec

Rest of the High-viaduct alignment (see Figure 8):
Soil Profile Type C
ARS Design Curve R3-6 (ATC-32), M_w=8.0, Peak Rock Acceleration=0.5g

- Increase spectral accelerations 20 percent for structural periods >= 1 sec
- No change in spectral accelerations for structural periods <= 0.5 sec
- Linearly interpolate spectral accelerations between 0.5 and 1.0 sec

12.6.2 Low-Viaduct and Causeway Structures

For the proposed Causeway structure over the Tennessee Hollow area, the same ARS Design Response Curve shown in Figure 9 is recommended:

Soil Profile Type D

ARS Design Curve R3-9 (ATC-32), M_w=8.0, Peak Rock Acceleration=0.5g

- Increase spectral accelerations 20 percent for structural periods >= 1 sec
- No change in spectral accelerations for structural periods <= 0.5 sec
- Linearly interpolate spectral accelerations between 0.5 and 1.0 sec

12.7 SEISMIC DESIGN CONSIDERATIONS FOR TUNNELS

All tunneling presently contemplated for the project are 'shallow burial' tunnels to be generally constructed by 'cut-and-cover' methods. Some lengths of the tunnel(s) are expected to be above ground on one side during construction and to be covered later. An earthquake will induce additional lateral earth pressures and consequent additional displacements on the structure, induced primarily by the shear waves generated by the earthquake. Depending on the direction of the shear wave relative to the structure alignment, the structure will be subjected to axial and curvature deformation along the tunnel and most importantly to racking: deformations (sideways distortion), which place high ductility demands on the joints that require special reinforcing details. Additionally, the whole or a part of the tunnel alignment may be subjected to buoyancy forces in the event that liquefaction occurs in the surrounding soils along the alignment.

The Mononobe-Okabe method (described by Seed and Whitman, 1970) has been widely used for seismic design of shallow tunnels, though it is widely recognized that it is not strictly applicable for underground structures. The method, originally developed for aboveground earth retaining walls, calculates the dynamic earth pressure caused by the inertial force of the surrounding soils, which are based on the soil shear strength properties and the design seismic coefficient. Experience indicates that the Mononobe-Okabe method yields unrealistic results (e.g., causes a structure to rack at an amount that is greater than the surrounding ground); the unrealistic result is amplified as the depth of burial increases. Nevertheless, the method has been shown to serve as a reasonable design method for estimating seismic earth loads for tunnels buried at shallow depths (Wang, 1993).

Wang (1993) presents a simplified procedure for evaluating racking distortion effects on rectangular tunnels that may be used advantageously in the case of deeper burial. The procedure incorporates the effects of the relative stiffness of the tunnel structure with respect to the surrounding soil (flexibility ratio) under the conditions of the design earthquake. Relationships have been developed from the findings from a large number of finite element soil-structure interaction analyses, using earthquake time-history input, whereby the racking deformation may be estimated from the free field deformation and the flexibility ratio. The free field deformations are computed by inputting values of soil shear moduli or shear wave velocities for the soil profile to an appropriate ground response program (e.g., the SHAKE, FLAC or SPECTRA). The flexibility ratio is computed using the elastic moduli and section properties of the tunnel structure and the strain-compatible shear modulus of the surrounding soil compatible. The racking deformation thus computed is utilized to calculate the corresponding loads and moments in the structure as outlined in Wang (1993) for use in the structural design of the tunnel.

Battery Tunnels. The Battery Tunnels will be constructed along the top of the Eastern Bluff. Per the present plans, part of the tunnel will be in a cut (southern side) and part of it will be filled. The sidewalls of the tunnel where the tunnel is in soil should be designed using the Mononobe-Okabe theory and a seismic coefficient of 0.5g.

Main Post Tunnels. As presently contemplated, the Main Post Tunnels will be embedded 1-2 meters in 'soft soils' that have a high potential for liquefaction. The soil Liquefaction will induce additional loads on the walls as well as reduce the resistances. These effects will have to be incorporated in the design.

A shear wave velocity of 150 meters/second (500 feet/second) is recommended for use if the tunnel is surrounded by soils with SPT values of 15 blows per foot or less; for soils above 15 blows per foot, a value of 300 meters/second (1,000 feet/second) is recommended.

A detailed dynamic soil-structure interaction analysis using ground motion history input may be required for the project.

SECTION 13: PRELIMINARY DESIGN AND CONSTRUCTION RECOMMENDATIONS

13.1 DEEP FOUNDATIONS

The high-viaduct, low-viaduct and the causeway foundations will be required to resist compressive (downward), tensile (uplift) and lateral loads as imposed by the design dead and live loads and seismic activity, with an adequate factor of safety. In particular, the uplift and the lateral forces will be sizeable, due to the large design earthquake. Deep foundations will therefore be required. The most common form of deep foundations are piles, which merit due consideration.

It should be noted that for the High-viaduct, other innovative deep foundations, such as a 'Sheet Pile Foundation' as used in Japan using a ring of pipe piles with a central concrete core, appears to be viable. Such foundations generate relatively low levels of noise and vibration and high flexibility of construction. Details of the method are presented in Appendix C.

Suitability of a pile type (pile material, method of installation) for a particular situation depends on specific subsurface conditions and construction constraints. For the present project, high noise levels and vibrations to nearby structures due to pile installation are of particular concern. Locally, the most common pile types are:

- Driven Precast, Pre-stressed Concrete (PPC) piles;
- Driven Cast in Steel Shell (CISS) pipe piles, both open ended or closed ended; or
- Cast in Drill Hole (CIDH) piles.

Driven piles may potentially cause unacceptable noise or vibration levels close to existing structures, precluding their use in some situations. CIDH piles or other less common non-driven pile types may be considered (e.g., Tubex piles, screw piles, oscillating piles) if noise or vibrations become an issue. They are, however, unlikely to prove as cost-effective as driven piles for most situations.

Pile penetrations as required for downward and uplift loads depend on the pile type, the pile size (cross-section), the strength properties of the surrounding soil/rock and the method of pile installation. Several accepted algorithms exist for such computations. For this project, the program APILEplus (Reese and Wang, 1993b) may be used for driven piles and SHAFT (Reese, Wang and Alleraga, 1998) for CIDH piles. The soil property and load transfer algorithms used should be per the program recommendations. Similarly, the lateral load behavior may be determined using the program LPILEplus (Reese and Wang, 1993a) and associated soil property algorithms recommended in the program. The required pile penetration will be the highest of the penetrations required for downward, uplift and lateral load standpoints; thereby, all the three loads may be resisted adequately.

The advantages/disadvantages of the three pile most common pile types are presented below:

13.1.1 PPC Piles

Advantages:

- The least expensive of the three alternatives
- The most readily available (in 305 and 355 millimeter (12 and 14 inch) sizes) and widely used in this area
- More resistant to potential corrosion problems than steel piles

Disadvantages:

- Less ductile; therefore less resistant to lateral loads, particularly from seismic

- Potential driveability problems including refusal in thick, very dense sand layers
- Cannot be driven through rock – the pile tip will be damaged
- Premature refusal will require cutting off from top of pile, resulting in loss of required reinforcement for lateral loads
- More susceptible to damage during driving, due to excessive stresses from alignment problems
- Splicing of pile sections is not cost-effective
- Noise and vibration during pile driving

13.1.2 CISS Piles

Advantages:

- More ductile, and therefore less susceptible to damage from seismic lateral loads
- Less susceptible to damage from driving stresses
- For open-ended piles, driving is significantly easier and less uncertain; in the event of refusal, the soil plug may be drilled out and the pile driven deeper to satisfactory penetration
- Splicing of pile sections is relatively easy

Disadvantages:

- More expensive than PPC piles
- Prone to corrosion from corrosive soils
- Drilled out soil from inside open-ended pile needs to be disposed; contaminated soils may cause added problems
- Filling with concrete is an added effort and expense
- Noise and vibration during pile driving
- Closed-ended piles could encounter hard driving and possible refusal in thick, dense sand layers, similar to PCC piles

Note: A variant of the driven CISS pile is the ‘oscillating’ pile. Here, the steel shell is oscillated into the ground, instead of being driven in by a hammer. This results in significant reduction in noise and vibration levels during installation. All other pertinent advantages and disadvantages are similar to driven CISS piles.

13.1.3 CIDH Piles

Advantages:

- Significantly less noise generated during installation
- Can be drilled through dense sand or rock, unless the rock is very hard; the serpentines, shale and sandstones can most likely be drilled through
- There less potential for damage of adjacent structure or property due to vibrations, unlike with driven PPC and CISS piles

Disadvantages:

- The cost and time to install are both greater than for PPC and CISS piles
- Degree of difficulty of installation and associated costs increase significantly for shallow groundwater and presence of loose, non cohesive soils
- If slurry is used to keep the hole open, disposal of the excavated slurry mixed soil may incur additional costs and complications, depending on the chemical composition of the slurry

Based on the above, it may be inferred that from the standpoint of subsurface conditions, CIDH piles are appropriate for the areas of shallow rock, such as on the western half of the high-viaduct and a small portion at the eastern end - driven piles appear to be more suitable in the other areas. However, as noted earlier, driven piles may be precluded from use in several locations due to excessive noise and vibration or potential soil displacement (heave) close to existing structures. Under such conditions, consideration of CIDH piles should be given priority. Where driven piles may be used, open-ended CISS piles should be considered first. Also, for driven piles, it is recommended that driveability studies be performed prior to pile type selection to assess a suitable hammer-pile combination whereby the piles may be driven to the required penetrations without being damaged. For instance, there is some indication of the presence of dense to very dense sands at shallow depths in some areas, such as near the Letterman Hospital, which may indicate potential for hard driving and potential for premature refusal.

13.1.4 Pile Lengths

A preliminary assessment of the required pile lengths for piles was made for preliminary design and cost estimating for the piles most commonly used for projects in the area. They indicate (Caltrans standard terminology used):

Pile Type / Class	Capacity	Effective Pile Length
PPC Pile (Alt.“X”)-Class 625	625 kN (70 ton)	16.8 m (55 feet)
PPC Pile (Alt.“X”)-Class 900	900 kN (100 ton)	18.3 meters (60 feet)
CISS Pile (610-millimeter [24-inch] diameter)	900 kN (100 ton)	19.8 meters (65 feet)
CISS Pile (610-millimeter [24-inch] diameter)	1350 kN (150 ton)	25.9 meters (85 feet)
CIDH Pile (915-millimeter [36-inch] diameter)	1350 kN (150 ton)	16.8 meters (55 feet) in soil minimum 1.5 meters (5 feet) into rock

Note that the effective embedment lengths are below the pile cap at the bent or below original ground at the abutments.

For preliminary cost estimates, 915-millimeter (36-inch) diameter CIDH piles are proposed from the start of the high-viaduct at the western end to Station 9+30 and from Station 11+74 to the eastern end. These locations are based on estimated rock depths of 15.3 meters (50 feet) below ground surface or less. Driven piles should be used at all other locations.

A Pile Data Table providing pile types and sizes, nominal resistances, design loads and tip elevations should be prepared for the final design.

13.2 TUNNELS

13.2.1 Battery Tunnels

As presently envisioned, the northbound and southbound Battery Tunnels will be excavated through the eastern bluff. The planned elevations indicate that parts of the excavation will be through rock. The highly fractured Franciscan bedrock is expected to be amenable to being ripped or excavated with standard rock excavators. Groundwater was not encountered in the Franciscan Formation in the borings in the area, presumably due to the limited depths of drilling, however, the Hydrology and Water Resources Report (Baseline, 2001) suggests a strong likelihood for encountering groundwater in the formation. Water inflow during tunnel excavation will therefore need to be controlled as appropriate (e.g., dewatering). Portions of the tunnel structure on the north side that protrude above the present ground surface will need to be covered with fill on the top and the sides.

As noted in Section 11.2 the proposed southbound tunnel will pass within 0.3 meters (1 foot) from the fence of the National Cemetery. In this area, no components of the shoring system (e.g., tiebacks, soil nails or dowels) will be allowed to extend under the cemetery.

The preservation of natural groundwater flow to the north facing slopes of the Eastern Bluff is a priority that must be reflected in the final design and construction of the tunnel box structure. During excavation through the bedrock, particular attention will need to be paid to the rock fractures and jointing (spacing, width, attitude, apparent permeability), so as to allow the placement of drainage materials against the exposed bedrock to collect and convey groundwater around the tunnel box. This will require the excavation procedure to facilitate the inspection of the water bearing fractures discussed above. For instance, if top-down construction is employed, the shoring shall be limited to the top of rock and the rock face shall be contained by rock bolts.

One option for maintaining the hydraulic connection across the tunnels is to apply geocomposite drainage strips to exposed water-bearing fissures on the south sidewall of the cut to collect, transport, and redistribute groundwater to the north sidewall. Geocomposites are routinely used worldwide in underground and tunnel construction to passively collect and move groundwater away from concrete structures. The approach, therefore, appears conceptually viable provided that it is feasible to map the exposed water-bearing fissures and structures on both sides of the tunnel during construction. Manufacturers of geocomposites contacted for this study expressed confidence in the ability of their products to provide permanent drainage around the structure. However, none was aware of a similar application of geocomposite drainage strips to reintroduce water into bedrock around a tunnel. Piezometers placed well in advance of the tunnel construction should be used to establish background groundwater levels and to monitor of groundwater levels during and after construction.

13.2.2 Main Post Tunnels

As presently proposed in the Presidio Parkway Alternative, the northbound and southbound Main Post Tunnels require only a shallow excavation into the existing soils. Per present plans, a minimal excavation (1-meter or less) below groundwater table may be required. The invert slab of the tunnels will be constructed essentially on grade and the walls will then be formed and cast in place. The roof will consist of precast segments to minimize disruption to traffic operations. The tunnel boxes will then be covered with soil to the desired finish grades.

The profile of the Main Post tunnels indicate the presence of soft soils of the old Tidal Marsh below the bottom of the box-structures. The ‘soft soils’ are judged inadequate for supporting the tunnels and backfilled soil cover. It is therefore proposed that the tunnels be supported on piles penetrating adequately into the dense sandy soils underlying the ‘soft soils’. Alternatively, measures to improve the ‘soft soils’ should be investigated, whereby the need for the piles may be eliminated. Soil improvement, if opted for, may potentially cause irreversible adverse impacts to the geologic resources and the hydrogeologic regime. The impacts should be evaluated as part of the investigation for the appropriate soil improvement measure.

13.3 RETAINING WALLS

Major retaining structures are planned along the alignment at several locations. The maximum height of the walls is 9 meters with the majority of the walls being significantly lower. In general, the retaining walls can be standard Caltrans walls on spread footings. In some areas, where the ‘soft soils’ of the Old Marsh are present, pile foundations may be required.

Precast walls with soil reinforcement may be used at certain locations for aesthetic importance, such as at the Girard Street underpass and the north wall of the northbound travelway between the two tunnels.

At certain locations, such as near Building 106, secant pile walls should be considered to minimize construction impacts, particularly where important historical or other resources are immediately adjacent to the proposed roadway.

The new retaining walls should have minimal impact on the overall hydrogeologic regime, though minor localized effects cannot be ruled out.

13.4 SETTLEMENTS

As noted earlier, the bottom slab for the Main Post tunnels will be underlain by 'soft soils' of the historic tidal marsh. Excessive settlements would therefore occur if the backfilled box-structures were to be supported by these soils. Pile foundations other appropriate measures should therefore be taken to mitigate the potential settlements.

Current alternatives do not involve the use of shallow spread footings as foundations. Additionally, no deeper compressible layers have been found in the borings drilled to date that would cause significant settlements of the Battery tunnels or the pile-supported structures. Hence, settlements due to consolidation under static loads do not apply to these anticipated structures.

13.5 SLOPES

The highest cut slopes are for the depressed surface road west of the high-viaduct, which are about 4 meters (15 feet) high and primarily in rock. All other steep slopes along the project route and vicinity are all natural slopes, generally underlain by rock or competent soil. The locations of the potential landslides shown in the Seismic Hazard Zones map from the Division of Mines and Geology (CDMG, 2000) do not appear to adversely affect the design or choice of alternatives for the project. Potential for significant failures of the natural slopes along the proposed alignments for the new roadway is therefore considered small, even under a strong earthquake or heavy rainfall.

A slide repair on the Bluff sloes immediately north of the west abutment of the low-viaduct is noted earlier in Section 5. This is the only slide indicated in the available records (personal communications with George Ford of The Presidio Trust and Grant Wilcox of Caltrans). The slide occurred in 1998 in a manmade slope after a rainstorm and was reportedly caused by the failure of a storm drain. Uncontrolled dumping in the area may have been an additional contributing factor to the slide.

13.6 PAVEMENTS

For the Presidio Parkway Alternative, the major at-grade roadways are the depressed travelways between the Battery and Main Post tunnels and east of the causeways. For preliminary assessment, pavement sections for at-grade roadways should be designed using a Traffic Index (TI) of 11. An R-value of 10 may be used for preliminary design for most of the pavements, except as discussed in the next paragraph. R-vaues will be measured from the recovered soils as part of laboratory testing prior to Final Design.

As may be noted from Figure 7, portions of the at-grade roadways may be founded on the historic tidal marsh 'soft soils'. This would indicate a very low R-value. For preliminary purposes, a value of 5 may be used. This may result in a very thick pavement section. Alternatively, the pavement section may be designed using a geotextile fabric in-between the subgrade soils and the structural section.

13.7 EARTHWORK

All earthwork for the project should generally conform to the requirements of Section 19 (Earthwork) of the most current Caltrans Standard Specifications.

Soils excavated in one location should be reused as fill or backfill in another location to the extent possible, provided it meets the appropriate requirements. Unsuitable materials such as contaminated soils, soils with high plasticity or excessive organic content or soils such as serpentine will be appropriately disposed offsite.

Other excavated rock, properly processed should be usable as fill onsite. An earthwork management plan should be developed in coordination with NPS.

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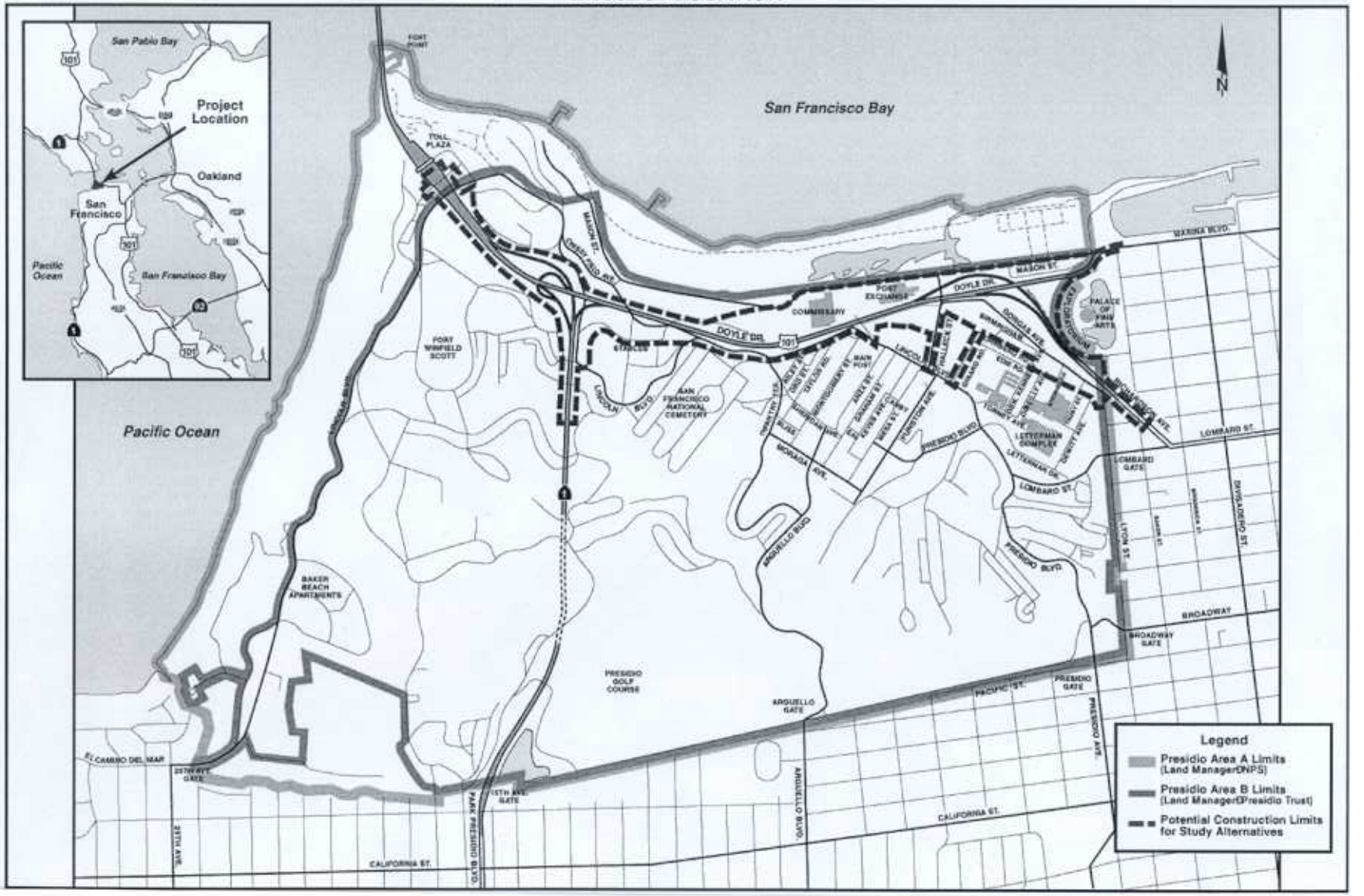
FIGURES

FIGURE 1
PROJECT LOCATION



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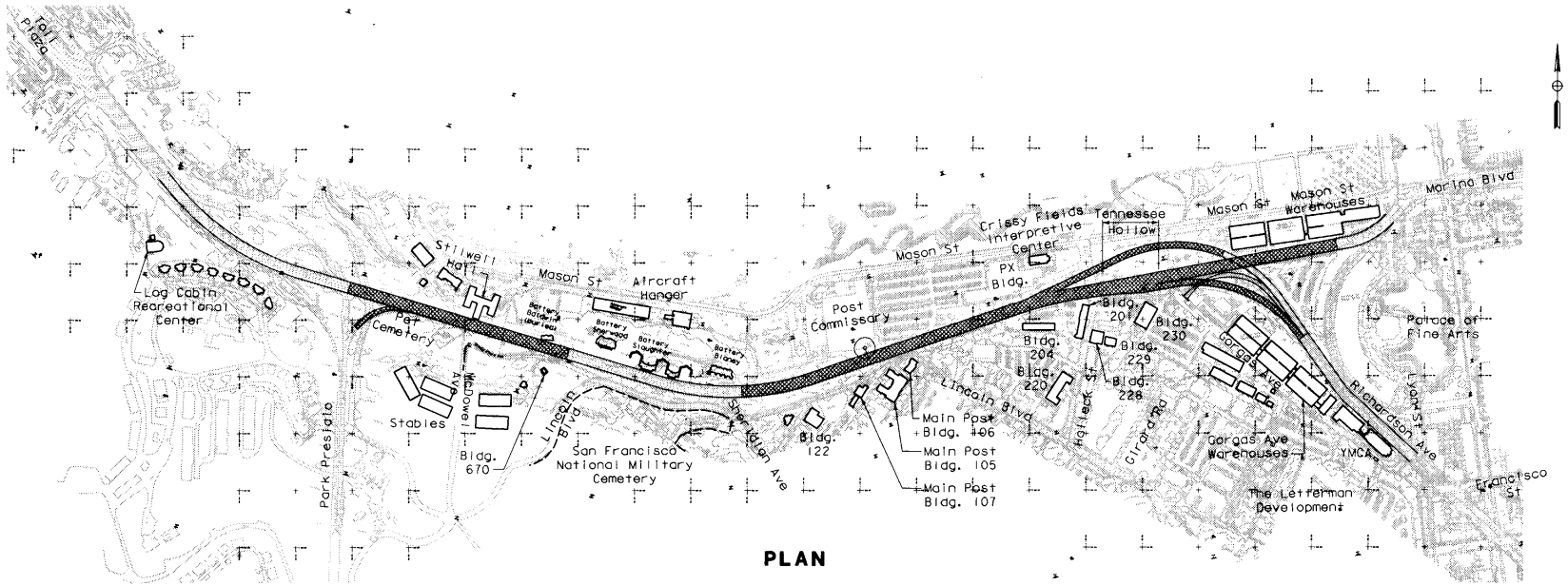
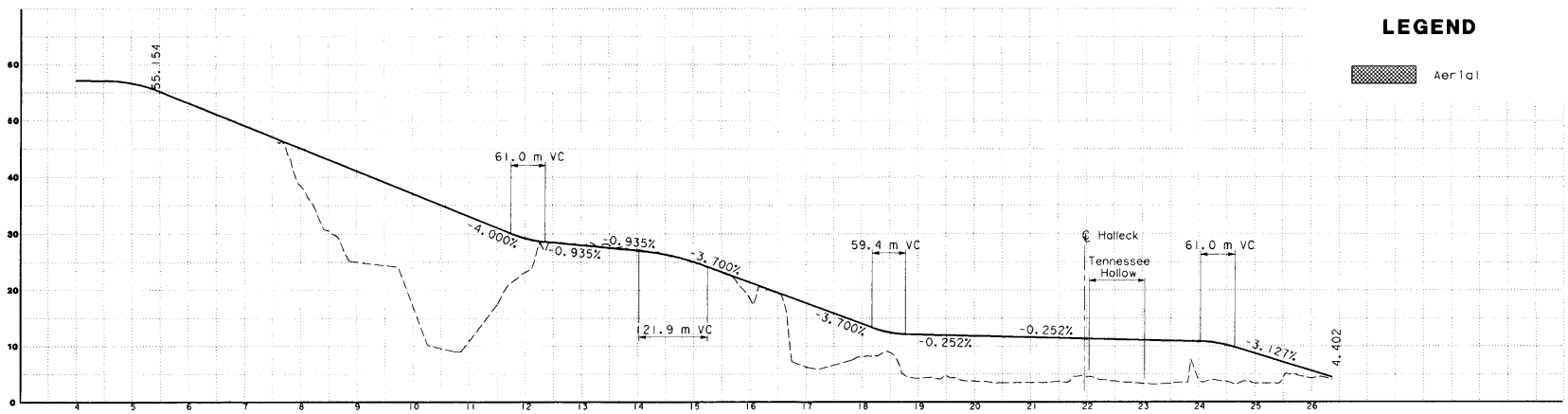


PLAN



PROFILE

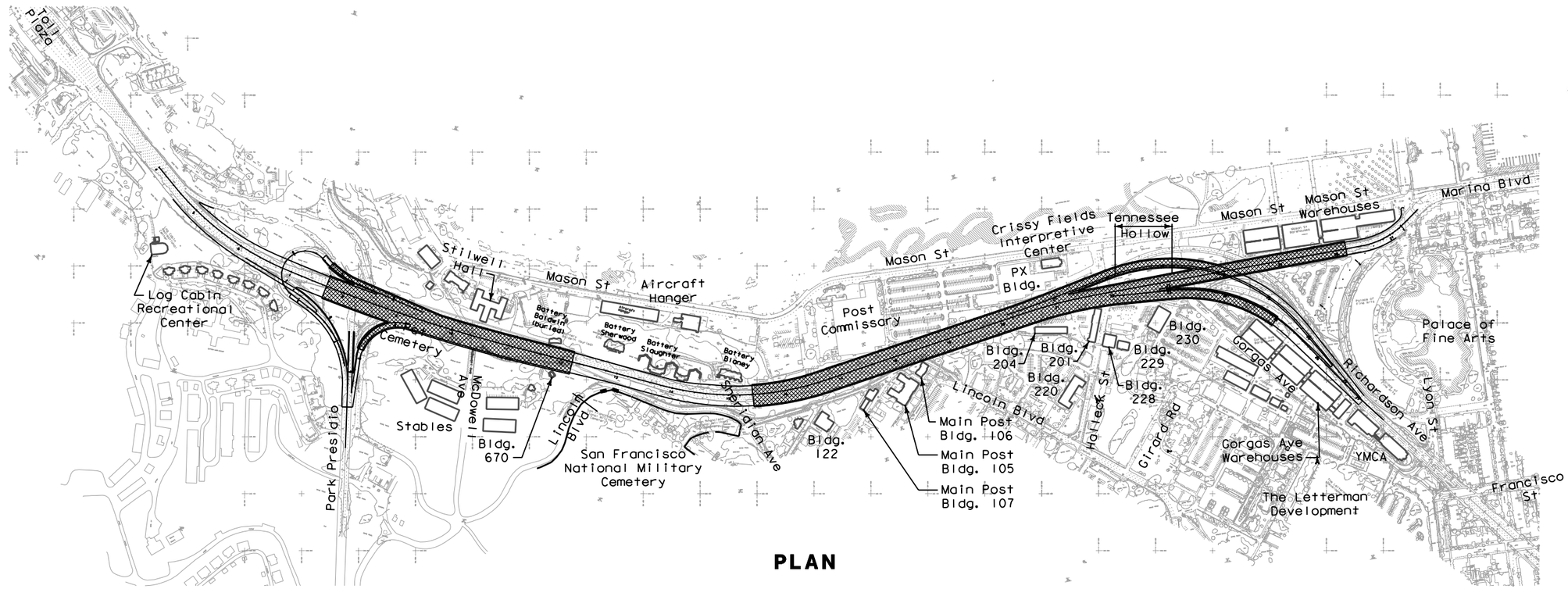
LEGEND

Aerial

0 75 150 300

Figure 2A

2a. Replace and Widen - No Detour



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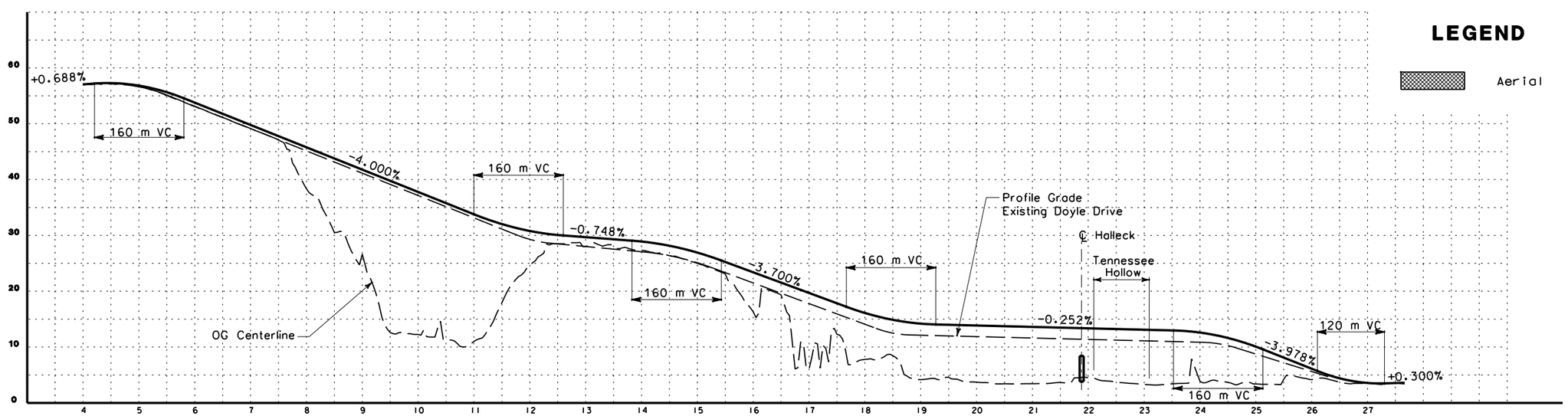

PROFILE

LEGEND

Aerial

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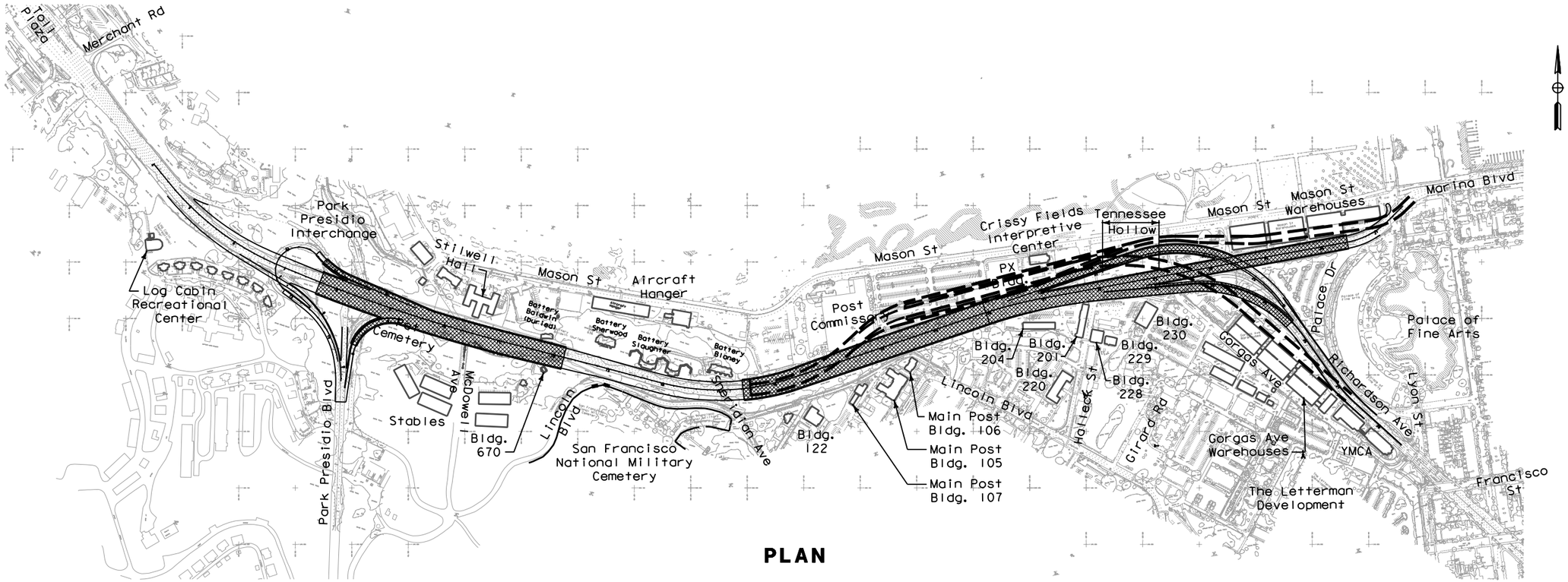
Figure 2B

2. Replace and Widen - With Detour



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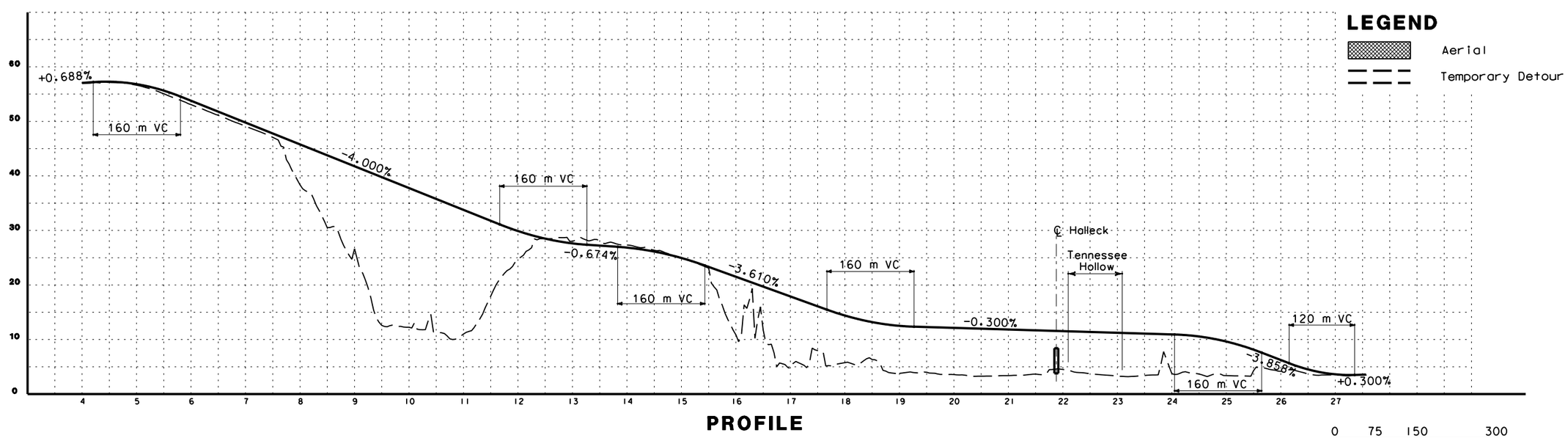

PROFILE

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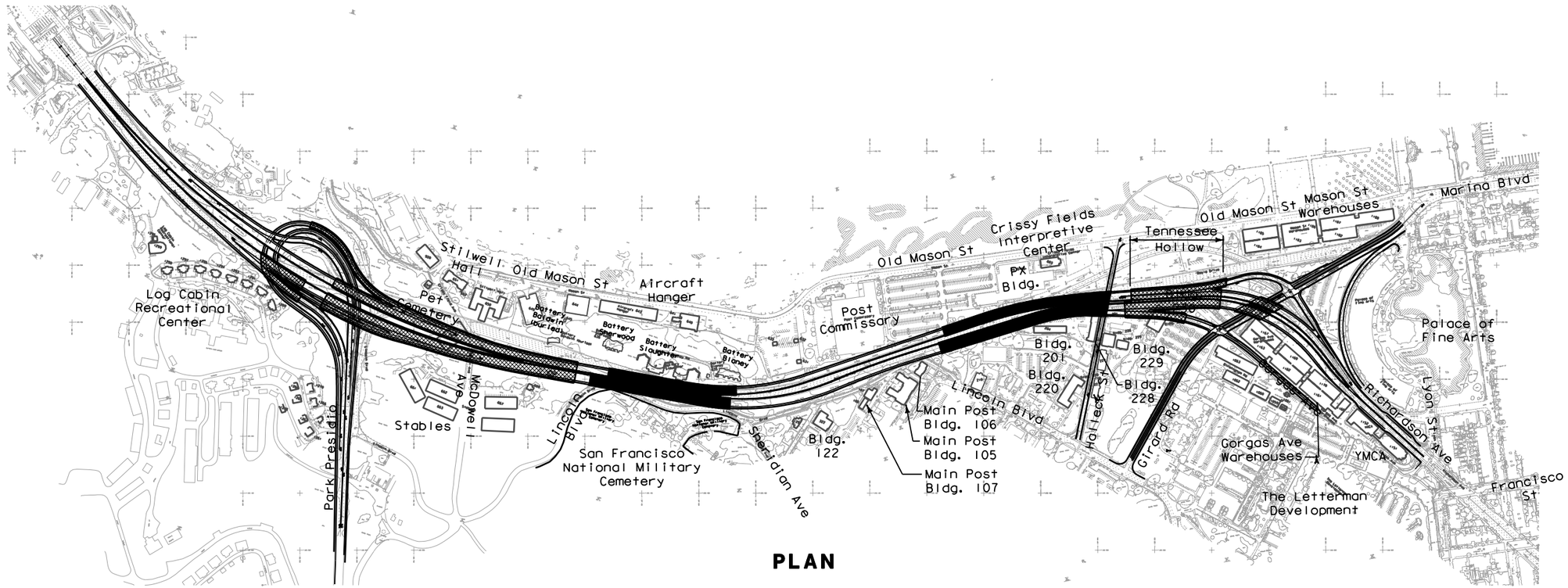
Figure 2C

5. Presidio Parkway



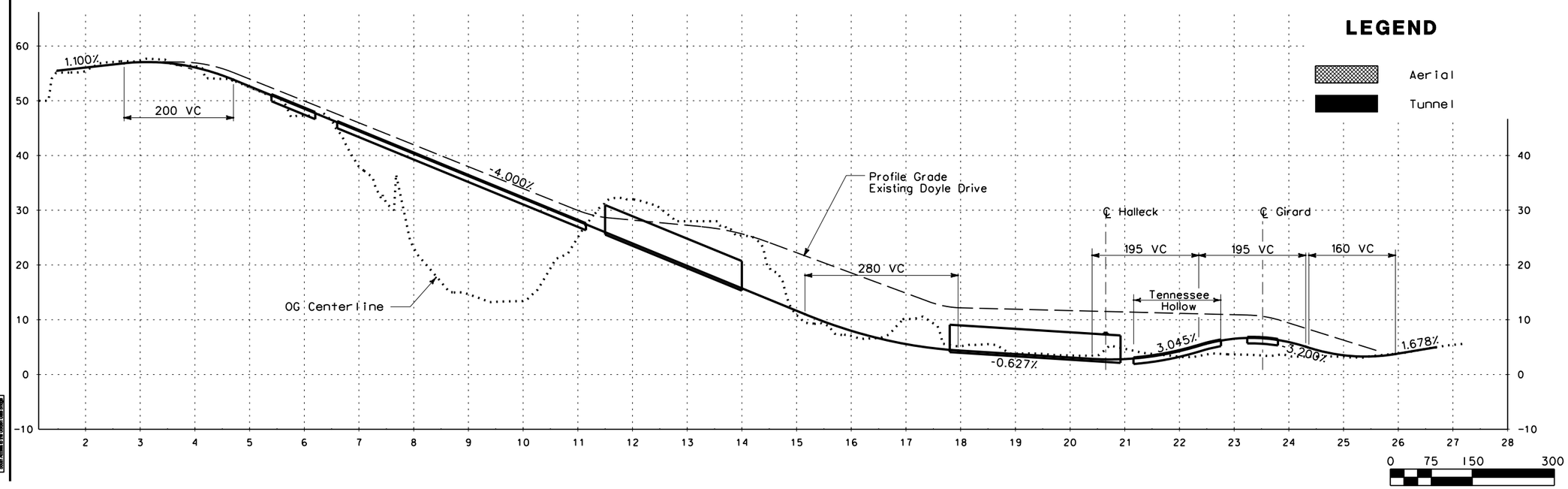


Figure 2D

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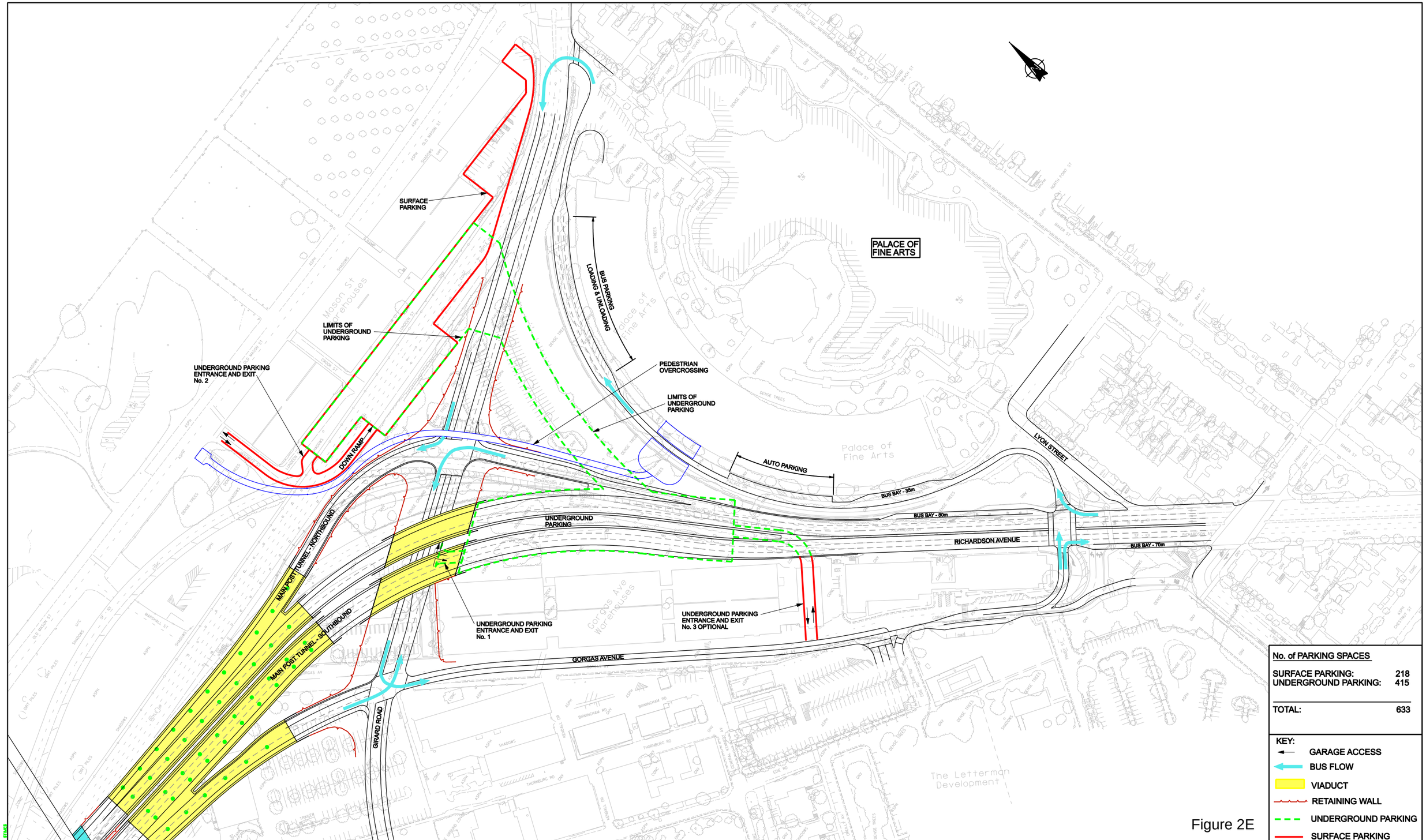

Figure 2E

No. of PARKING SPACES	
SURFACE PARKING:	218
UNDERGROUND PARKING:	415
TOTAL:	633

KEY:

- GARAGE ACCESS
- BUS FLOW
- VIADUCT
- RETAINING WALL
- UNDERGROUND PARKING
- SURFACE PARKING

NOTES.

1. AUTO PARKING ESTIMATE IS BASED ON THE GUIDLINE OF 32.5 m2 per VEHICLE.
2. VEHICLE CLEARANCE:

1m Structure

3m Min

Issue	Date	By	Chkd	Appd

Job Title: ALTERNATIVE 5 PRESIDIO PARKWAY

Drawing Title: LAYOUT PLAN DIAMOND OPTION EAST END PARKING & CIRCULATION

Scale: 1:1000

File Name: SFSK-052.DGN

Drawing Status: DRAFT

Job No	Drawing No	Issue
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19/64 Ove Arup Partnership Ltd

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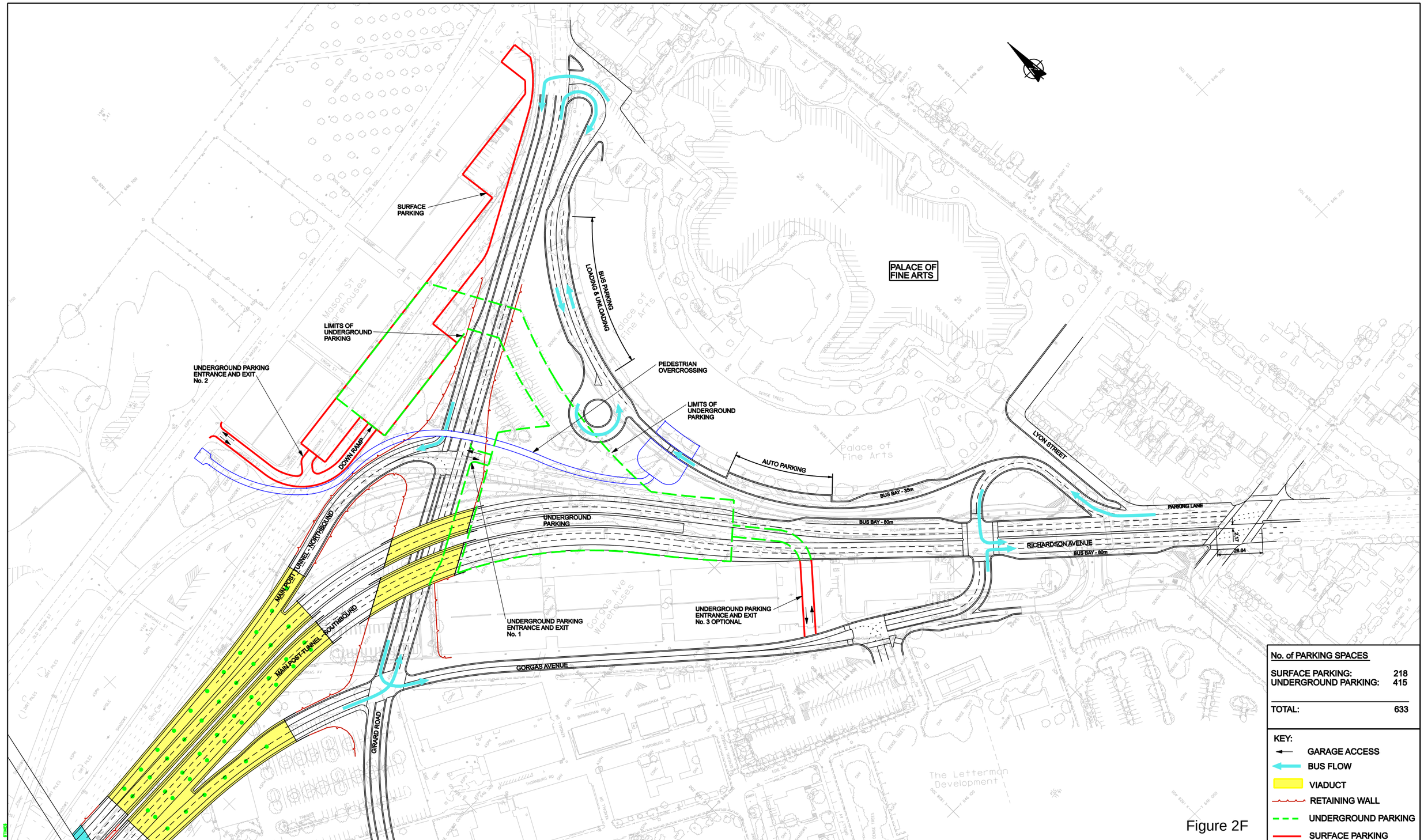

Figure 2F

No. of PARKING SPACES	
SURFACE PARKING:	218
UNDERGROUND PARKING:	415
TOTAL:	633

KEY:

- GARAGE ACCESS
- BUS FLOW
- VIADUCT
- RETAINING WALL
- UNDERGROUND PARKING
- SURFACE PARKING

NOTES.

1. AUTO PARKING ESTIMATE IS BASED ON THE GUIDLINE OF 32.5 m2 per VEHICLE.
2. VEHICLE CLEARANCE:

1m Structure

3m Min

Issue	Date	By	Chkd	Appd

Job Title: ALTERNATIVE 5 PRESIDIO PARKWAY

Drawing Title: LAYOUT PLAN CIRCLE DRIVE EAST END PARKING & CIRCULATION

Scale: 1:1000

File Name: SFSK-051.DGN

Drawing Status: DRAFT

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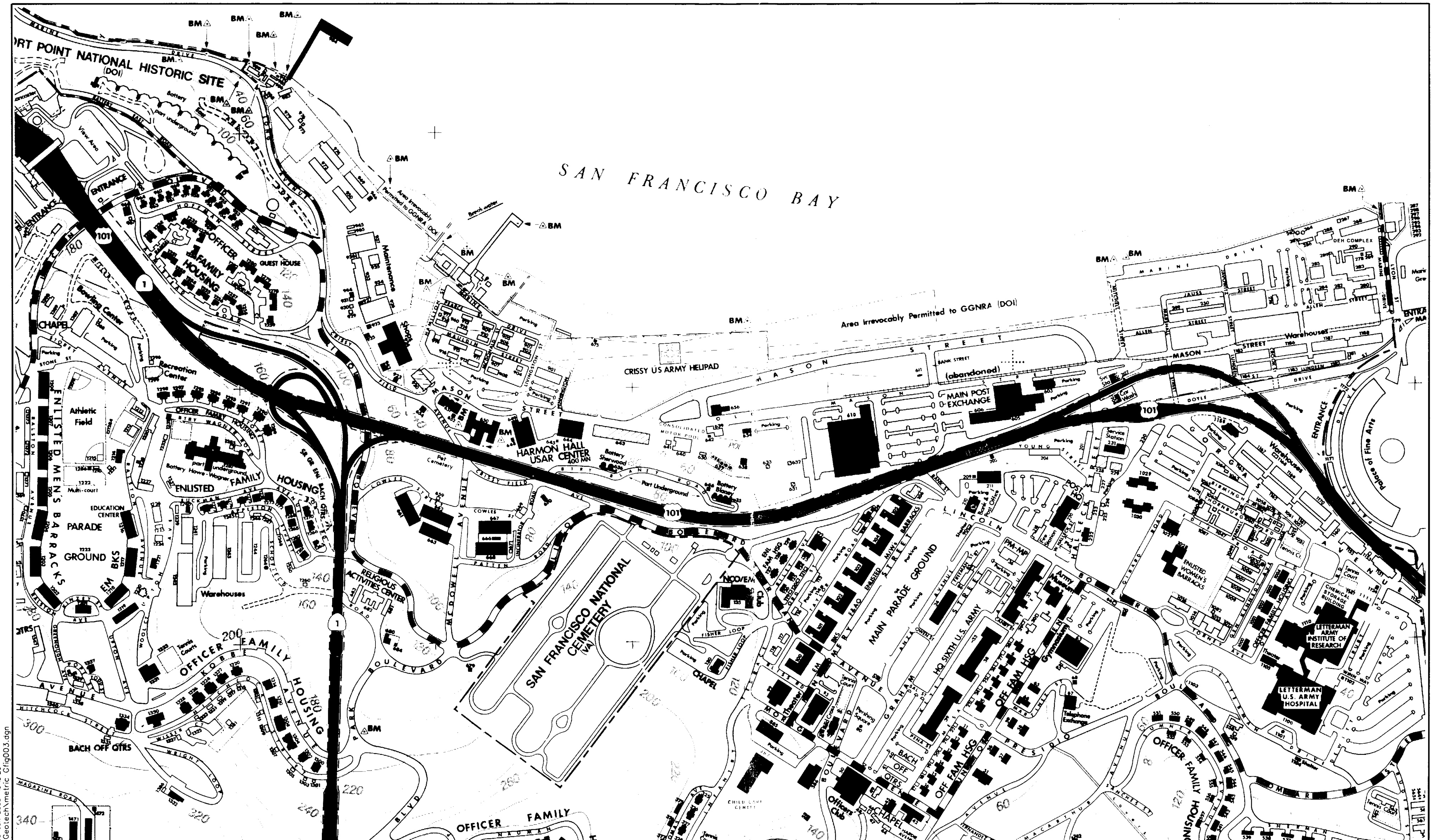

PARSONS BRINCKERHOFF 303 Second St., Suite 700N, San Francisco, CA 94107	Caltrans Metric	Caltrans District 4 Route 101 City and County of San Francisco SFCTA Contract no. 99/00-7

Doyle Drive Environmental & Design Study

STRUCTURES AND FACILITIES IN THE PROJECT VICINITY

June 2000 | Revision 0 | Figure 3

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SURFICIAL DEPOSITS

Qaf

Artificial fill

Predominantly dune sand but includes silt, clay, rock waste from excavations, manmade debris, and organic waste. Probable fill material indicated in parentheses. Maximum thickness approximately 60 feet

Qb

Modern beach deposits

Predominantly well sorted medium to coarse gray sand; coarse gravel and large cobbles in some places. Maximum thickness approximately 40 feet

Ql | **Qly** | **Qlo**

Landslide deposits

Generally unstratified mixtures of bedrock, sand, silt, and clay in varying proportions.

Qly, *younger landslide deposits*

Qlo, *older landslide deposits*

Qal

Alluvium

Predominantly clayey silty sand and clayey silt, some pebbles; locally clean medium sand, generally gray to brown. Maximum thickness approximately 25 feet; observed thickness 15 feet

Qsr

Slope debris and ravine fill

Angular rock fragments in sand, silt, and clay matrix; generally light yellow to reddish brown. Maximum thickness approximately 80 feet

Qd

Dune sand

Clean well-sorted fine to medium sand; yellowish brown to light gray. Maximum thickness approximately 150 feet

Qob

Older beach deposits

Predominantly well sorted medium to coarse gray sand. Maximum thickness approximately 30 feet

Qc

Colma Formation

Unconsolidated fine to medium sand with small to moderate amounts of silt and clay; in places includes clay beds, 6 inches to 5 ft thick; cobble-size rubble rare; commonly light brown to gray. Observed thickness 75 feet; probably maximum thickness approximately 300 feet

Qm

Bay mud and clay

Plastic gray silty clay; some lenses of sand, peat, and shell fragments; fluid to soft upper layers; moderately stiff clay at depth. Concealed except for exposures north of Corinthian Island and north of Sausalito Point. Maximum thickness approximately 140 feet

Qu

Surficial deposits, undivided

BEDROCK

Ks

Sheared rocks, undifferentiated

Coherent blocks and pieces of hard rock as much as several hundred feet in diameter, in a matrix of intensely sheared shale and serpentine. Clasts predominantly sandstone, shale, and serpentine, but also include all other rock types known in Franciscan Formation. Matrix gray to greenish gray, moderately firm to soft and clayey; generally expansive and plastic when wet

sph | **sp**

Serpentine

Mostly soft sheared rock containing hard knobs of unsheared serpentine, rodingite, and rocks of the Franciscan Formation. Derived from peridotites (mostly harzburgite, some dunite).

sph, *hard serpentine, slightly sheared. Shown separately on Potrero Hill only. Variously colored, but generally greenish gray, blue, or brown. Includes sheared rocks* (Ks) *where that unit is mostly serpentine*

gb

Gabbro

Fine- to coarse-grained gabbro; includes diabase where texture is subophitic. Occurs as inclusions or segregations in serpentine

KJss | **KJsh** | **KJs**

Clastic sedimentary rocks

KJss, *sandstone; thick-bedded and massive graywacke sandstone interbedded with thin layers of shale and fine-grained sandstone; some thick conglomerate lenses.*

KJsh, *shale and thin-bedded sandstone; predominantly interbedded and laminated shale and fine-grained graywacke sandstone; beds generally 2 to 5 inches thick.*

KJs, *sandstone and shale, undifferentiated; consists of units* KJss *or* KJsh *so poorly exposed that predominant lithology could not be determined.*

Sandstone and shale at Point Lobos and vicinity west of City College fault zone is probably Great Valley sequence of Bailey, Irwin, and Jones (1964, p. 123)

KJc

Radiolarian chert and shale

Reddish-brown; alternate beds of hard brittle chert, 1 to 5 inches thick, and firm brittle shale, ⅛ to ½ inch thick. Locally includes bodies of massive chert

KJg

Greenstone

Greenish-gray aphanitic to medium-grained altered volcanic rocks; predominantly basalt. Includes flows, agglomerate, and tuff; pillow lavas common; less commonly massive; interbedded with radiolarian chert and sandstone

KJm

Metamorphic rocks

Fine- to coarse-grained slate, schist, and granofels of the blueschist facies

Reference: Schlocker (1974)

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Caltrans Metric

Caltrans District 4	Doyle Drive Environmental & Design Study
Route 101	SITE GEOLOGICAL MAP EXPLANATION
City and County of San Francisco	
SFCTA Contract no. 99/00-7	June 2000 · Revision 0 · Figure 4A

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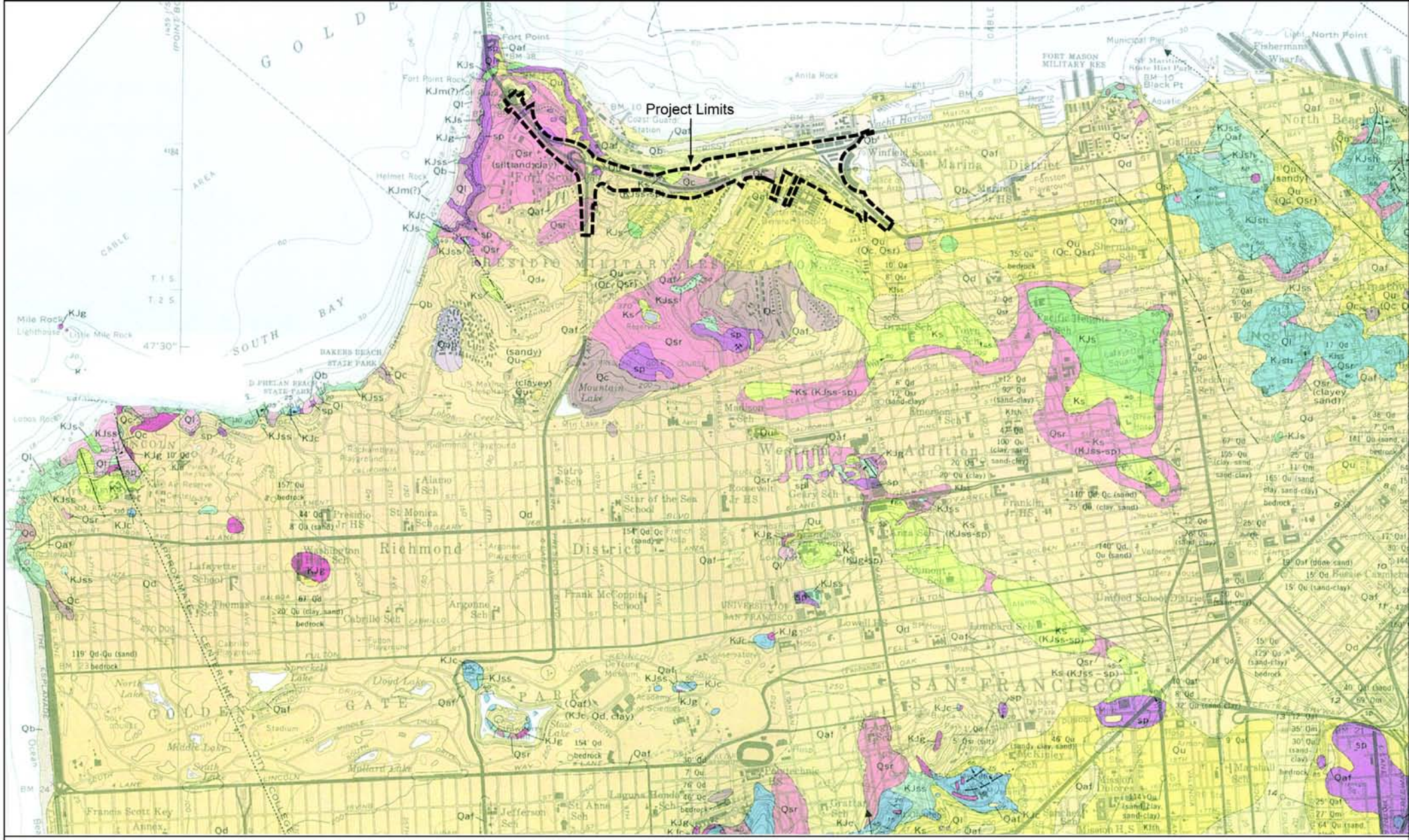

Reference: Schlocker (1974)

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PB PARSONS BRINCKERHOFF

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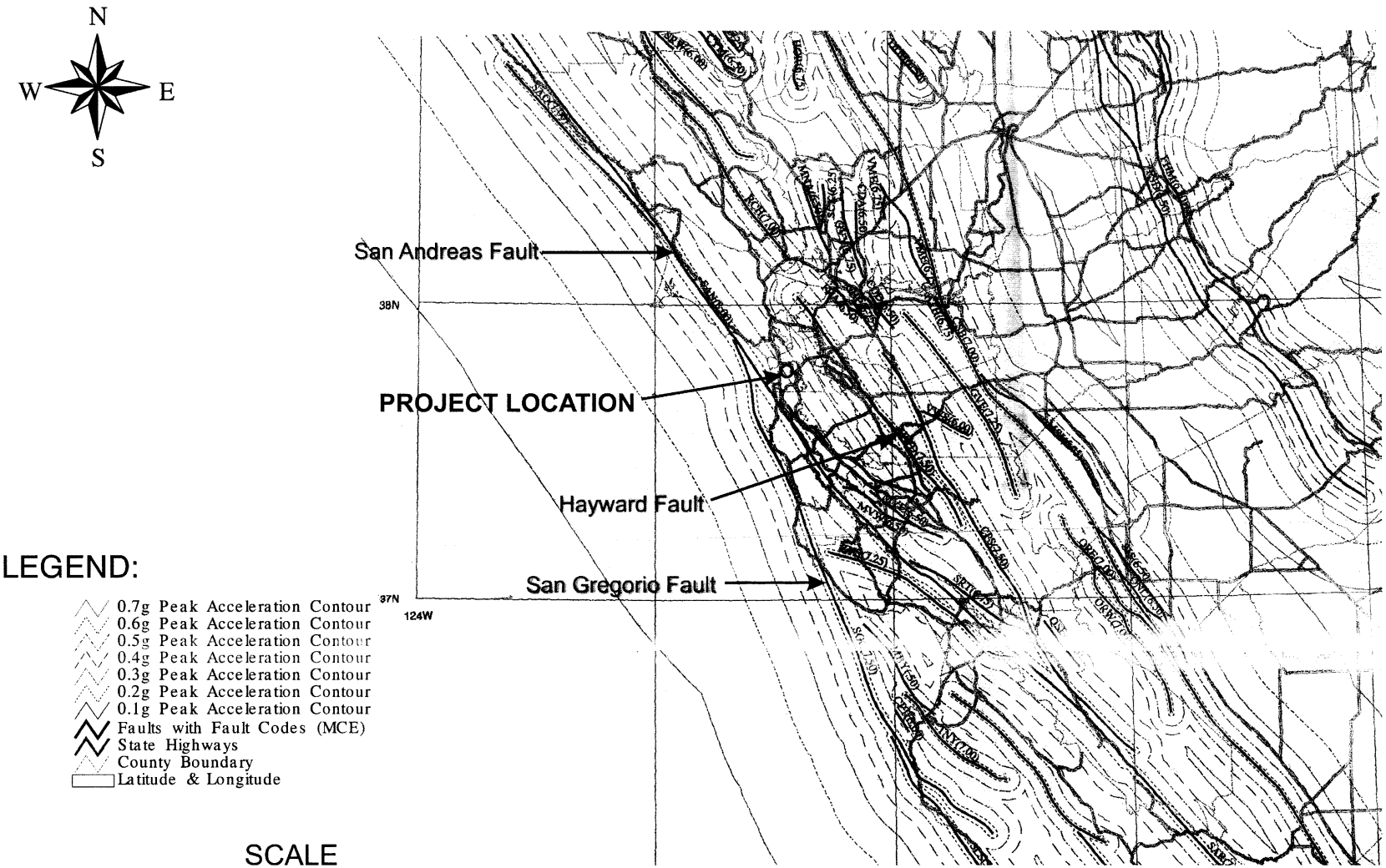
Caltrans Metric

Caltrans District 4	Doyle Drive Environmental & Design Study
Route 101	SITE GEOLOGICAL MAP
City and County of San Francisco	
SFCTA Contract no. 99/00-7	June 2000 · Revision 0 · Figure 4



LEGEND:

- 0.7g Peak Acceleration Contour
- 0.6g Peak Acceleration Contour
- 0.5g Peak Acceleration Contour
- 0.4g Peak Acceleration Contour
- 0.3g Peak Acceleration Contour
- 0.2g Peak Acceleration Contour
- 0.1g Peak Acceleration Contour
- Faults with Fault Codes (MCE)
- State Highways
- County Boundary
- Latitude & Longitude

SCALE

0 50 100 150 200 250 Kilometers

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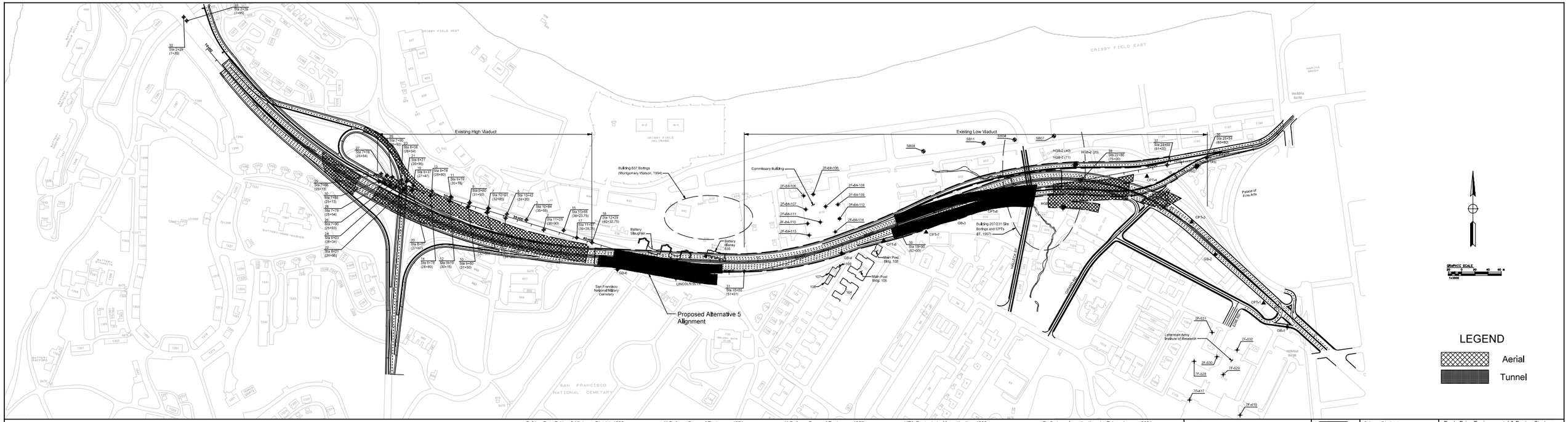
Caltrans District 4
Doyle Drive
City and County of San Francisco
SFCTA Contract no. 99/00-7

Doyle Drive Environmental & Design Study

SEISMIC HAZARD MAP OF SITE VICINITY

June 2000 | Revision 0 | Figure 5



Key to Borings:

Golden Gate Bridge & Highway District, 1933
Boring No. — 30
Approx.Hwy IQ Station In meters — Sta 7+73
1933 roadway stationing In feet — (25+13)
Approximate Boring Location

U.S. Army Corps of Engineers, 1971
Boring No. — 2F-531
Approximate Boring Location

U.S. Army Corps of Engineers, 1987
Boring No. — 2F-84-109
Approximate Boring Location

HTA Geotechnical Investigation, 1998
Boring No. — SB07
Approximate Boring Location

Preliminary Investigations by Taber, January 2001
Boring No. — GB-4, HGB-3 (74)
Approximate Boring/Well Location
Boring No. — CPT-8
Approximate CPT Location

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Metro

Caltrans District 4	Doyle Drive Environmental & Design Study	
Route 101	BORING LOCATION PLAN	
City and County of San Francisco		
SFCTA Contract no. 99/00-7	September 2004	Revision 0	Figure 6

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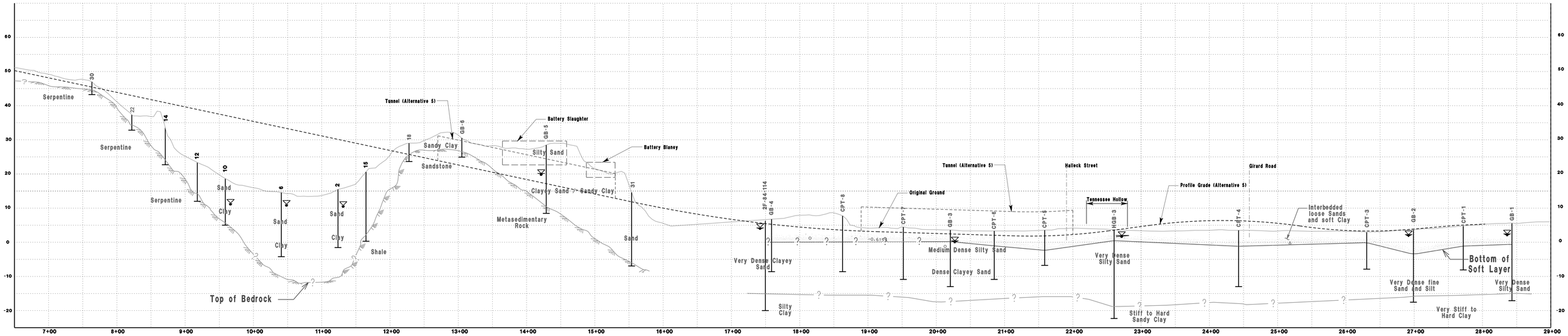

DRAFT

Note: *Profile represent the Alignment for Detour Construction, Tunnel under Halleck Alternative, Direct Marine Tunnel and Signalized Marine Connection (Alternative 6)*

Key: Boring — Water Table — *Presidio Road Borings, 1933 (Sheet No A-11)* — 22

Geotechnical Boring/Well (Tober, 2001) — GB-4, HGB-3
Cone Penetration Test Location (Tober, 2001) — CPT-3
US Army Corps of Engineers, 1987 — 2F-84-114

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Caltrans District 4
Route 101
City and County of San Francisco
SFCTA Contract no. 98/00-7

Doyle Drive Environmental & Design Study

PROJECT AND SOIL PROFILES

June 2004 | Revision 0 | Figure 7

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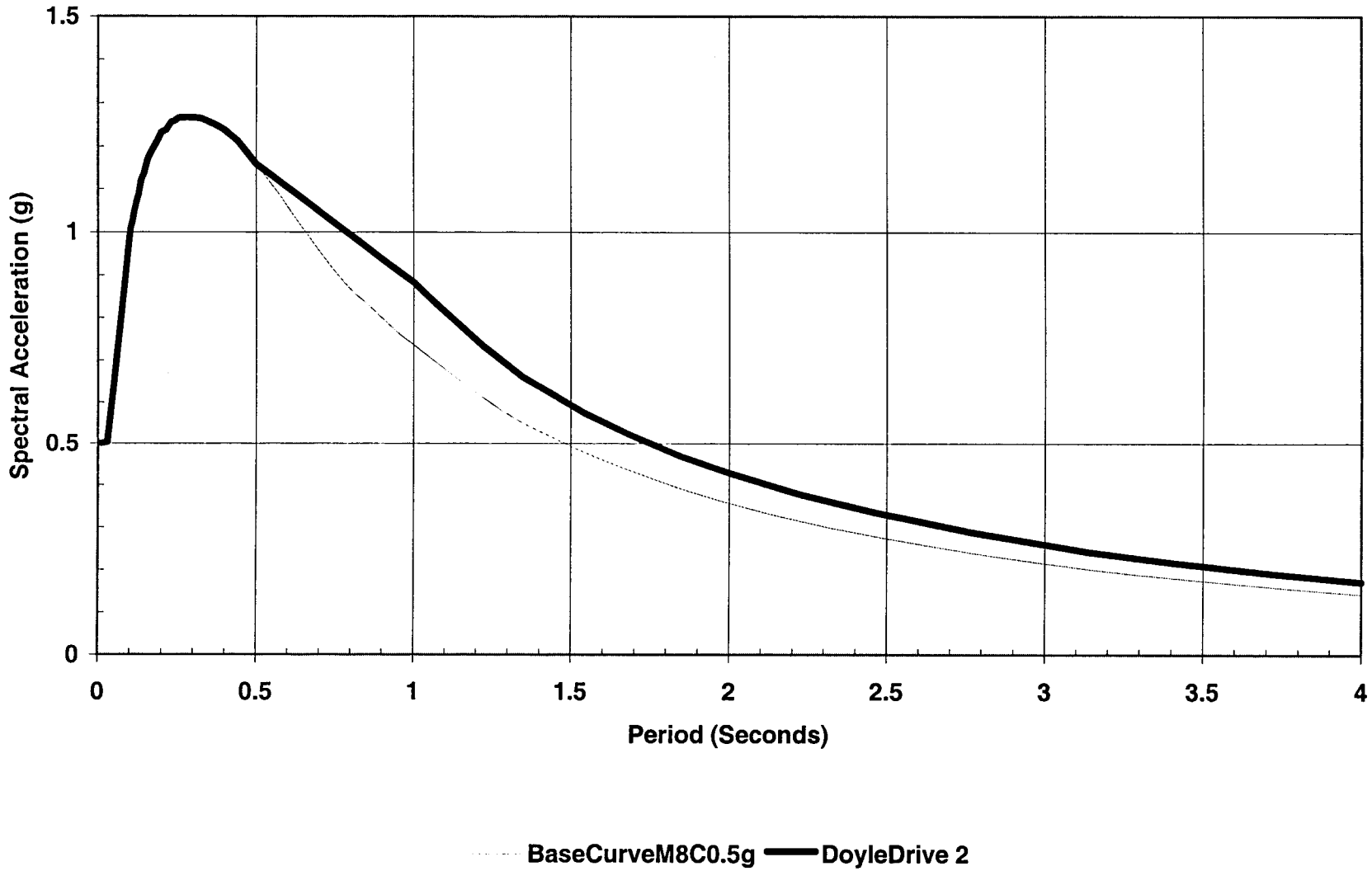

PARSONS BRINCKERHOFF 303 Second St., Suite 700N, San Francisco, CA 94107	Caltrans Metric	Caltrans District 4 Route 101 City and County of San Francisco SFCTA Contract no. 99/00-7	Doyle Drive Environmental & Design Study ARS DESIGN CURVE FOR SOIL PROFILE TYPE C	
			May 2001	Revision 0	Figure 8

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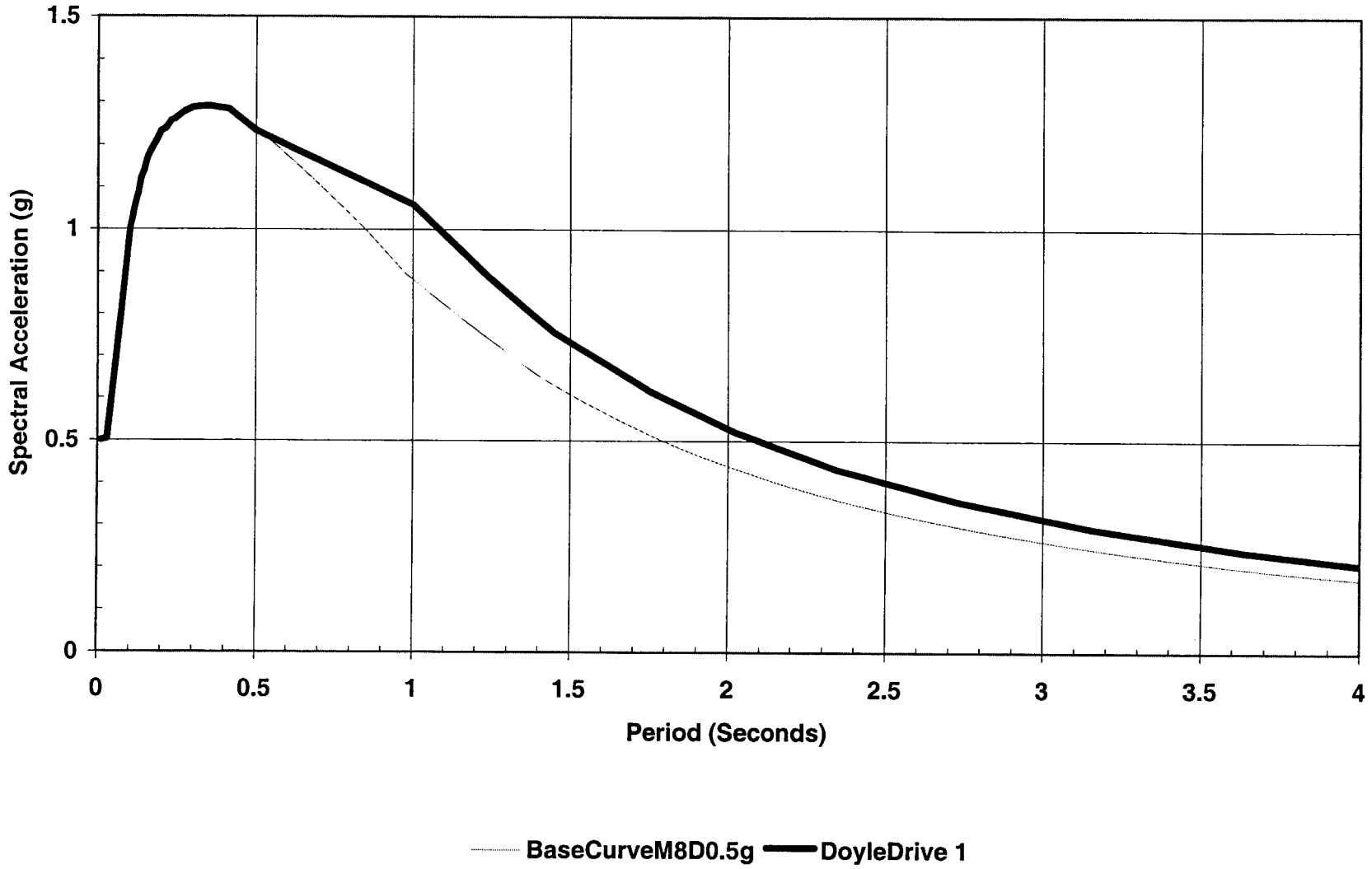

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Caltrans Metric

Caltrans District 4	Doyle Drive Environmental & Design Study		
Route 101	ARS DESIGN CURVE FOR SOIL PROFILE TYPE D		
City and County of San Francisco			
SFCTA Contract no. 99/00-7	May 2001	Revision 0	Figure 9

APPENDIX A

EXISTING AVAILABLE BORING LOGS

PRESIDIO BORING LOGS **A-1**

COMMISSARY BUILDING BORING LOGS ..... **A-2 TO A-6**

LETTERMAN ARMY HOSPITAL BORING LOGS ..... **A-7 & A-8**

CRISSY FIELD BORING LOGS ..... **A-9 TO A-13**

BUILDING 207/231 – CPT LOCATIONS **A-14**

BUILDING 207/231 – TYPICAL CPT RESULTS ..... **A-15 TO A-1**

The As-Built Log of Test Boring Sheets were used to prepare the geotechnical report.

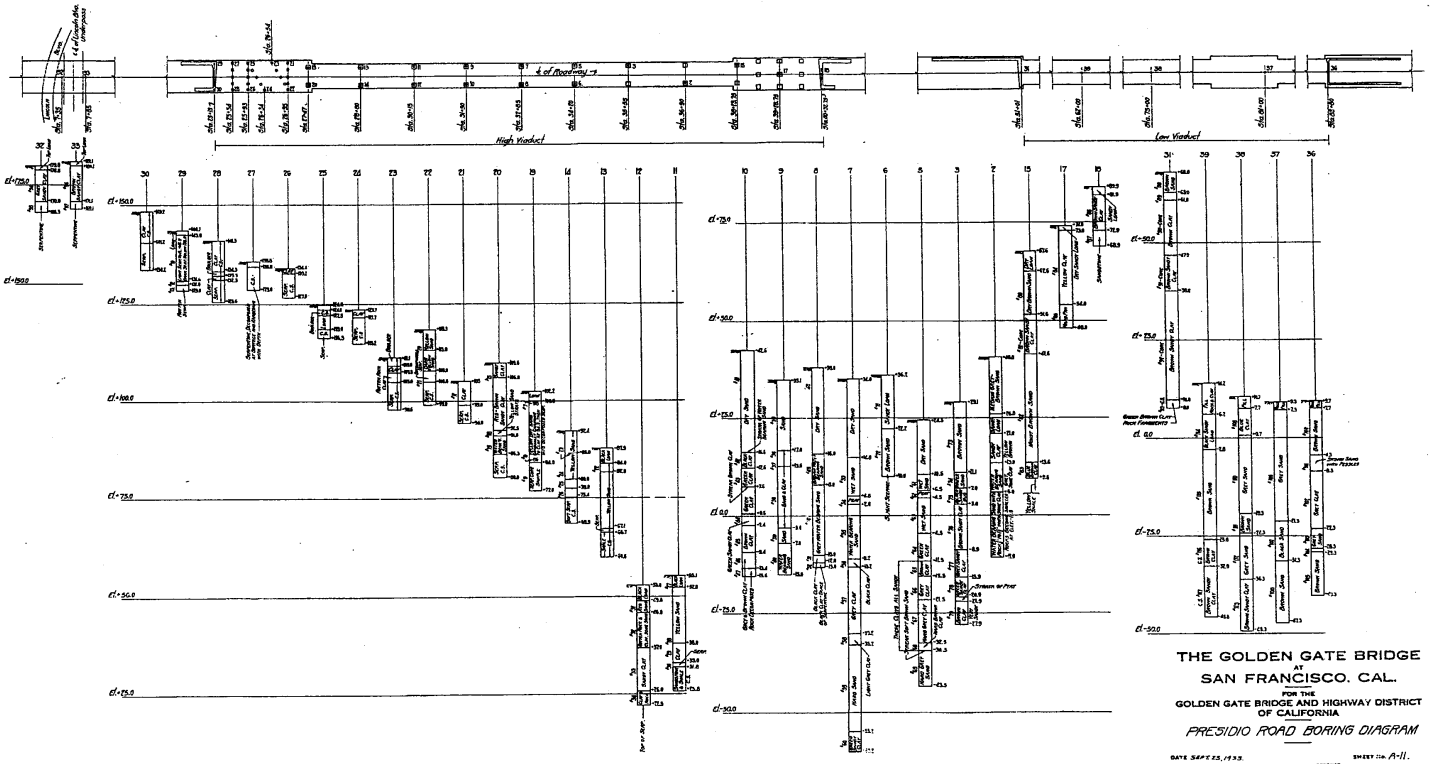
Ignatius Lam
GEOTECHNICAL PROFESSIONAL

REGISTERED PROFESSIONAL ENGINEER
IGNATIUS PO LAM
No. GE 2107
Exp. 3-31-96
GEOTECHNICAL
STATE OF CALIFORNIA

2-6-95
TO ACCOMPANY PLANS DATED

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SF	101	9.1/9.4	76	80

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.



GENERAL NOTES:-
--indicates location of boring. All stations given are for location of borings.
"CS" indicates core sample.
Number to left of hole indicates sample number.
Number to right of hole indicates elevation.

THE GOLDEN GATE BRIDGE
AT
SAN FRANCISCO, CAL.
FOR THE
GOLDEN GATE BRIDGE AND HIGHWAY DISTRICT
OF CALIFORNIA

PRESIDIO ROAD BORING DIAGRAM

DATE SEPT 23, 1933. SHEET No. A-11.
CHIEF ENGINEER
PLANS PREPARED BY STRAUSS ENGINEERING CORPORATION
CHIEF ENGINEER

NO AS-BUILT CHANGES

AS BUILT
CORRECTIONS BY T. BART
CONTRACT NO. 04-141704
DATE 3/31/97 MBT 3/11/98

M. CULLEN M J Cullen DESIGN OVERSIGHT 12-7-94 SIGNOFF DATE	NOTE: ADDITIONAL AS-BUILT FOUNDATION DATA MAY BE AVAILABLE AT THE OFFICE OF TRANSPORTATION MATERIALS AND RESEARCH 5900 FOLSOM BOULEVARD SACRAMENTO, CALIFORNIA 95819	DESIGN BY CHECKED DETAILS BY CHECKED QUANTITIES BY CHECKED	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	J. VINCENT PROJECT ENGINEER	BRIDGE NO. 34 - 19 POST MILE 9.1 - 9.4
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3			CU: 04 EA 141701	DISREGARD PRINTS BEARING EARLIER REVISION DATES 10/27/94 11/21/94	SHEET 65 OF 65

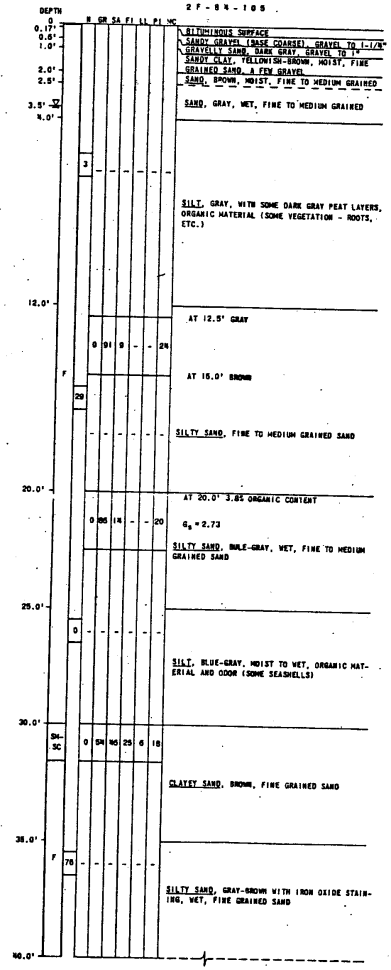
CONTRACT NO.: 590042

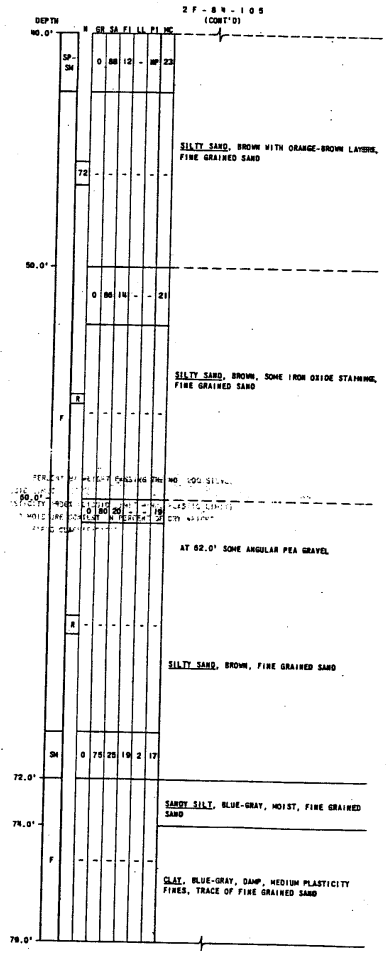
PRESIDIO VIADUCT
AS-BUILT LOG OF TEST BORINGS 1

DGN NAME : C:\ACAD\PRESIDIO\BLOGTB.DWG
PLOT SCALE : 1=1

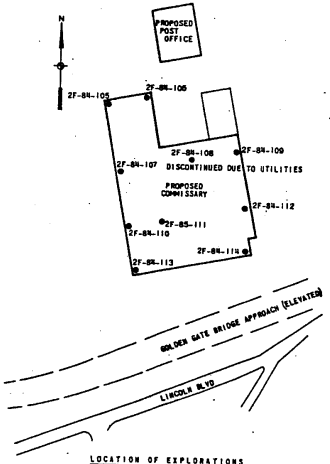
A-1

FUNCTIONAL ANALYSIS - VE PAYS

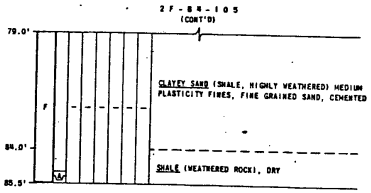




VERTICAL SCALE: 1" = 2'



LOCATION OF EXPLORATIONS
SCALE: 1" = 100'



NOTES: (CONT'D)

7. REFUSAL OF THE STANDARD PENETROMETER IS DEFINED AS ONE OF THE FOLLOWING:
   A. 25 BLOWS FOR 1" OR LESS ADVANCEMENT OF SAMPLER; OR
   B. 50 BLOWS FOR 1" TO 6" ADVANCEMENT OF SAMPLER.
8. ATTEMPT WITH THE STANDARD PENETROMETER IS DEFINED AS REFUSAL WITHIN THE FIRST 6"; SEATING PENETRATION.
9. WHERE BLOW COUNT EQUALS '0' (IN 'B' COLUMN) PENETROMETER ADVANCED UNDER ITS OWN WEIGHT.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS 50% OR MORE RETAINED ON THE NO. 200 SIEVE.	GRAVELS MORE THAN HALF OF COARSE FRACTION RETAINED ON THE NO. 4 SIEVE.	CLEAN GRAVELS	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND MIXTURES.
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES.
	SANDS MORE THAN HALF OF COARSE FRACTION PASSING THE NO. 4 SIEVE.	CLEAN SANDS	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.
			SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES.
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES.
FINE GRAINED SOILS MORE THAN 50% PASSING THE NO. 200 SIEVE.	SILTS AND CLAYS	LIQUID LIMIT BELOW 50%	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY FINE SANDS OR SILTS. PLASTICITY BELOW "A" LINE.
			CL	INORGANIC CLAYS, GRAVELLY CLAYS, SANDY CLAYS, LEAN CLAYS. PLASTICITY ABOVE "A" LINE.
			OL	ORGANIC SILTS AND ORGANIC CLAYS. PLASTICITY BELOW "A" LINE.
		LIQUID LIMIT 50% AND ABOVE	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS. ELASTIC SILTS. PLASTICITY BELOW "A" LINE.
			CH	INORGANIC FAT CLAYS. PLASTICITY ABOVE "A" LINE.
			OH	ORGANIC CLAYS OR ORGANIC SILTS. PLASTICITY BELOW "A" LINE.
HIGHLY ORGANIC SOILS			Pt	PEAT. ORGANIC CONTENT GREATER THAN 50%.

STANDARD PENETROMETER DESCRIPTIVE DATA

CONESIONLESS		CONESIVE	
BLOWS*	RELATIVE DENSITY	BLOWS*	CONSISTENCY
0-8	VERY LOOSE	0-1	VERY SOFT
9-10	LOOSE	2-4	SOFT
11-20	FIRM	5-8	FIRM
21-30	VERY FIRM	9-15	STIFF
31-50	DENSE	16-30	VERY STIFF
51+	VERY DENSE	31+	HARD

*BLOWS PER 1.0 FT. OF PENETRATION OF A 2-INCH O.D. AND 1 3/8-INCH I.D. SAMPLER DRIVEN BY A 140-LB. HAMMER, 30-INCH FREEFALL.

LEGEND:

- ●2F-BN-114 LOCATION OF EXPLORATION.
- GR GRAVELS, PERCENT BY WEIGHT PASSING 3-INCH SIEVE AND RETAINED ON THE NO. 4 SIEVE.
- SA SANDS, PERCENT BY WEIGHT PASSING THE NO. 4 SIEVE AND RETAINED ON THE NO. 200 SIEVE.
- FI FINES, PERCENT BY WEIGHT PASSING THE NO. 200 SIEVE.
- LL LIQUID LIMIT.
- PI PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
- MC FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
- F VISUAL FIELD CLASSIFICATION.
- * LABORATORY VISUAL CLASSIFICATION.
- N NUMBER OF BLOWS OF STANDARD PENETROMETER.
- R REFUSAL WITH STANDARD PENETROMETER.
- A ATTEMPT WITH STANDARD PENETROMETER.
- ▽ WATER LEVEL

NOTES:

1. CLASSIFICATIONS ARE IN ACCORDANCE WITH THE UNIFIED SOILS CLASSIFICATION SYSTEM (ASTM D-2487).
2. ALL SIEVE SIZES ON THE CHART ARE U.S. STANDARD.
3. THE TERMS "SILT" AND "CLAY" ARE USED RESPECTIVELY TO DISTINGUISH MATERIALS EXHIBITING LOWER PLASTICITY FROM THOSE WITH HIGHER PLASTICITY. THE MINUS NO. 200 SIEVE MATERIAL IS CLASSIFIED AS SILT IF THE LIQUID LIMIT AND PLASTICITY INDEX PLOT BELOW THE "A" LINE ON THE PLASTICITY CHART (ASTM D-2487) AND IS CLASSIFIED AS CLAY IF THE LIQUID LIMIT AND PLASTICITY INDEX PLOT ABOVE THE "A" LINE ON THE CHART.
4. BORDERLINE CLASSIFICATION: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS (FOR EXAMPLE, GW-GC, A WELL-GRADED GRAVEL-SAND MIXTURE WITH A CLAY BINDER).
5. BORINGS WERE DRILLED 17 THROUGH 28 JUNE 1988 TO REFUSAL IN WEATHERED BEDROCK (EXCEPT FOR BORINGS 2F-BN-112 & 114) WITH A FAILING 750 DRILL PRS UTILIZING A 5-1/2" DRAG BIT TO ADVANCE THE HOLE. BORINGS 2F-BN-107, 111, AND 117 USED A TRICONE BIT WITH HYDRAULIC PRESSURE TO ADVANCE THE HOLE IN THE WEATHERED BEDROCK. BORING 2F-BN-108 ENCOUNTERED A BURIED ELECTRICAL POWER LINE AT 4 FEET DEPTH WHICH CAUSED A PARTIAL POWER OUTAGE ON THE BASE. AN ATTEMPT WAS MADE TO RELOCATE 2F-BN-108 NEARBY, BUT FACILITIES PERSONNEL WOULD NOT GIVE PERMISSION. STANDARD PENETROMETER DATA WAS COLLECTED IN ALL BORINGS. UNDISTURBED SAMPLES WERE COLLECTED IN BORINGS 2F-BN-105, 107 AND 113. A COLLAR PIPE WAS REQUIRED IN EACH BORING.
6. GROUNDWATER WAS ENCOUNTERED DURING DRILLING AS INDICATED ON THE LOGS. BORING 2F-BN-114 ENCOUNTERED AN ARTESIAN WATER CONDITION. WATER FLOWED AT A RATE OF ABOUT 7 GALLONS/HOUR.

GRAPHIC SCALES

1" = 2' 0 1' 2' 4' 6' 8' 10'

1" = 100' 0 50' 100' 200' 300' 400' 500'

1" = 100' 100' 50' 0 100' 200'

REVISION	DATE	DESCRIPTION	BY	BY

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

DESIGNED: R.L.T.
DRAWN: A.E.B., M.H.E.
CHECKED: PL

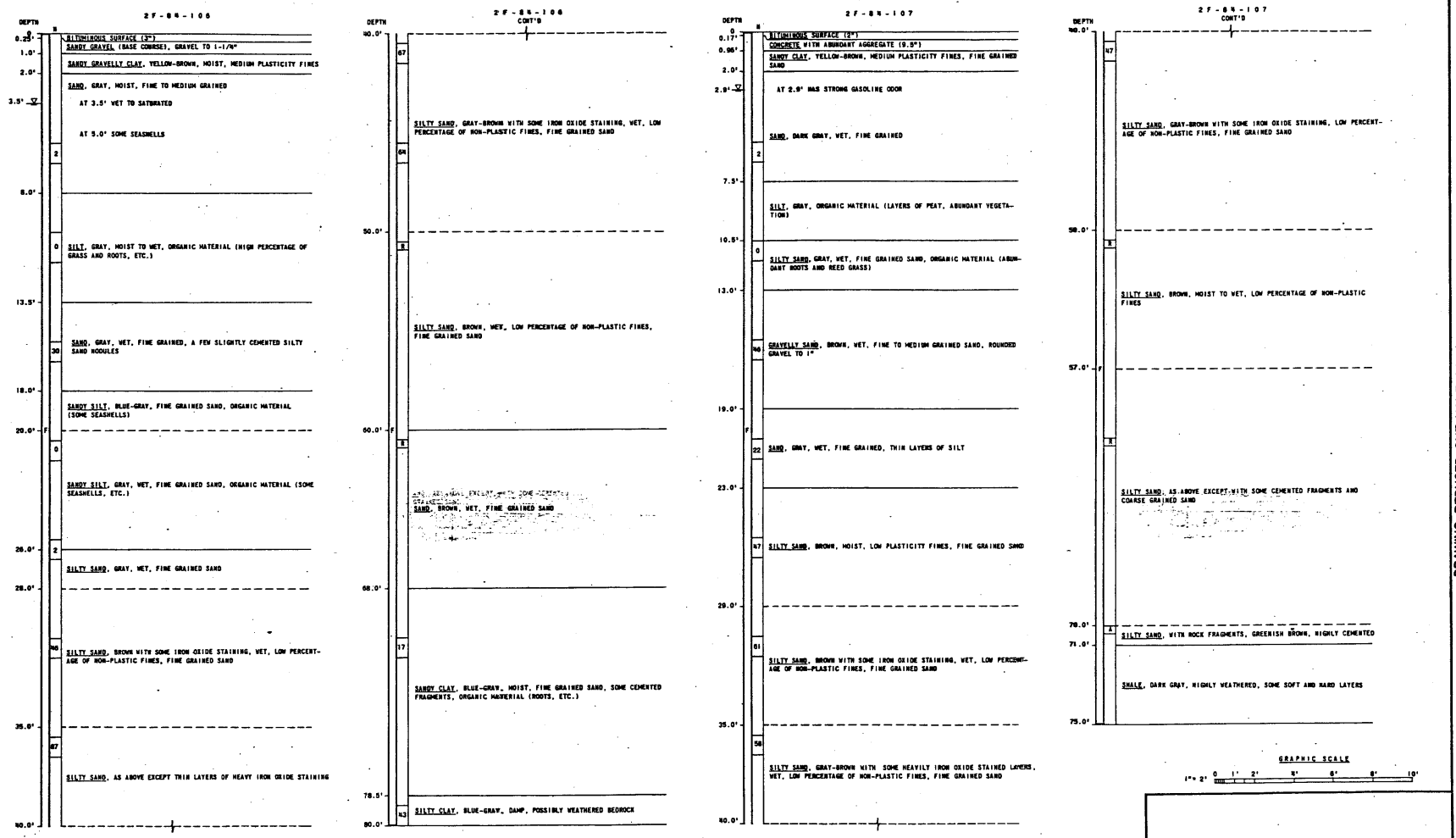
PRESIDIO OF SAN FRANCISCO CALIFORNIA

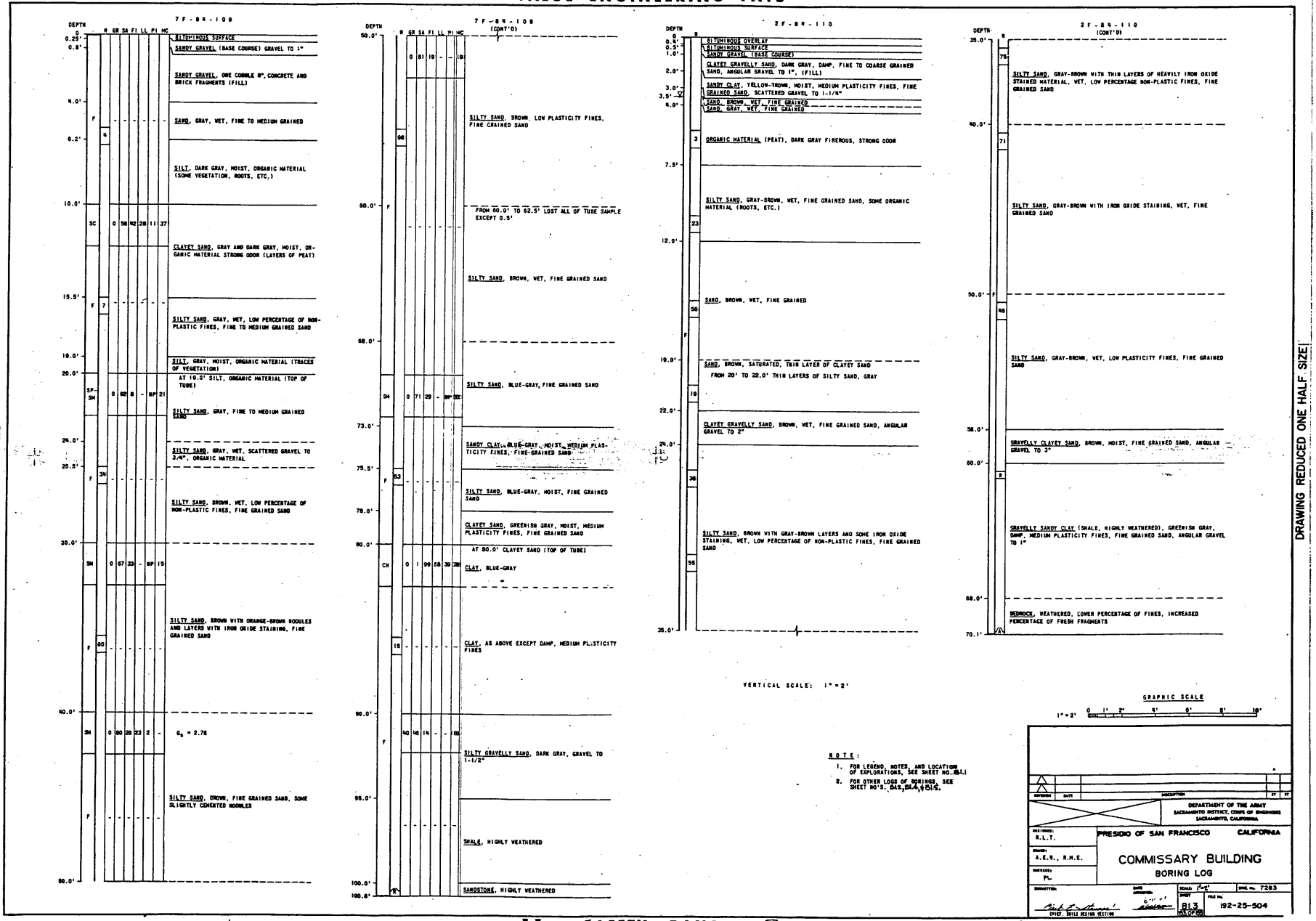
COMMISSARY BUILDING
BORING LOG

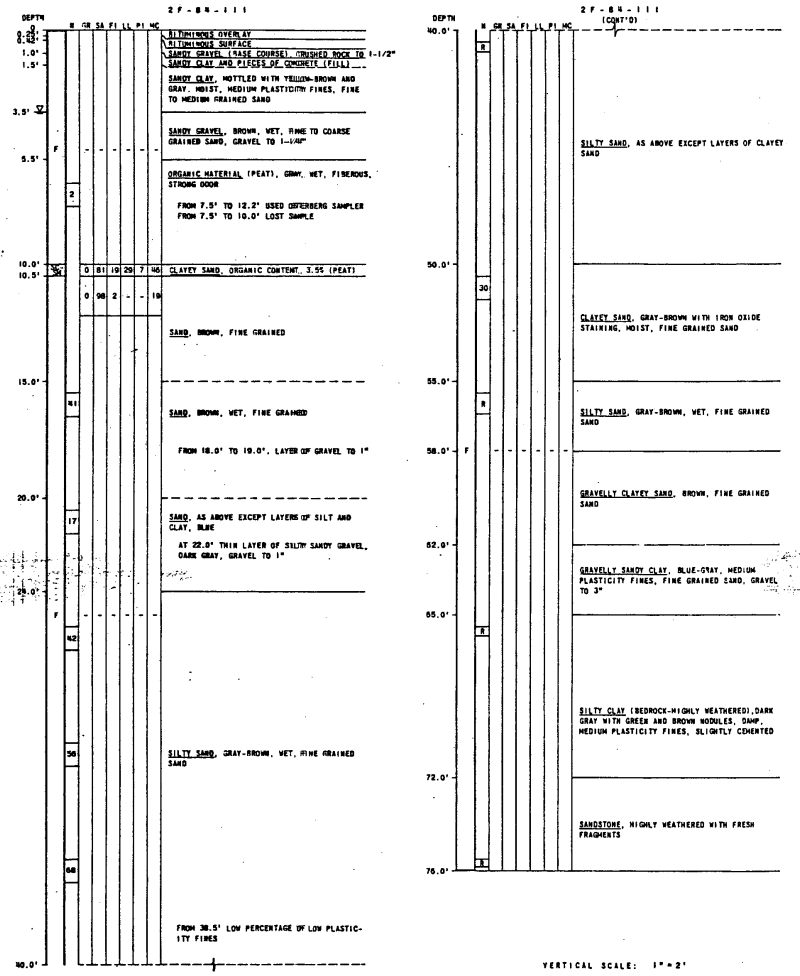
SUBMITTED: Chief, Soils Design Section
DATE APPROVED: 6-11-88
SCALE: 1" = 100'
SHEET: B1.1 OF 7
SPEC. No. 7283
FILE No. 192-25-504

DRAWING REDUCED ONE HALF SIZE

SAFETY PAYS


2F-84-105
DEPTH
N
0.25'
BITUMINOUS SURFACE (3")
1.0'
SANDY GRAVEL (BASE COURSE), GRAVEL TO 1-1/2"
2.0'
SANDY GRAVELLY CLAY, YELLOW-BROWN, MOIST, MEDIUM PLASTICITY FINES
SAND, GRAY, MOIST, FINE TO MEDIUM GRAINED
3.5'
AT 3.5' WET TO SATURATED
AT 5.0' SOME SEASHELLS
2
8.0'
0
SILT, GRAY, MOIST TO WET, ORGANIC MATERIAL (HIGH PERCENTAGE OF GRASS AND ROOTS, ETC.)
13.5'
SAND, GRAY, WET, FINE GRAINED, A FEW SLIGHTLY CEMENTED SILTY SAND NODULES
30
18.0'
SANDY SILT, BLUE-GRAY, FINE GRAINED SAND, ORGANIC MATERIAL (SOME SEASHELLS)
20.0'
F
0
SANDY SILT, GRAY, WET, FINE GRAINED SAND, ORGANIC MATERIAL (SOME SEASHELLS, ETC.)
26.0'
2
SILTY SAND, GRAY, WET, FINE GRAINED SAND
28.0'
46
SILTY SAND, BROWN WITH SOME IRON OXIDE STAINING, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
35.0'
67
SILTY SAND, AS ABOVE EXCEPT THIN LAYERS OF HEAVY IRON OXIDE STAINING
40.0'
2F-84-106
CONT'D
40.0'
67
SILTY SAND, GRAY-BROWN WITH SOME IRON OXIDE STAINING, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
64
50.0'
8
SILTY SAND, BROWN, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
60.0'
F
8
SAND, BROWN, WET, FINE GRAINED SAND
68.0'
17
SANDY CLAY, BLUE-GRAY, MOIST, FINE GRAINED SAND, SOME CEMENTED FRAGMENTS, ORGANIC MATERIAL (ROOTS, ETC.)
78.5'
43
SILTY CLAY, BLUE-GRAY, DAMP, POSSIBLY WEATHERED BEDROCK
80.0'
2F-84-107
DEPTH
N
0.17'
BITUMINOUS SURFACE (2")
0.96'
CONCRETE WITH ABUNDANT AGGREGATE (9.8")
2.0'
SANDY CLAY, YELLOW-BROWN, MEDIUM PLASTICITY FINES, FINE GRAINED SAND
2.9'
AT 2.9' HAS STRONG GASOLINE ODOR
SAND, DARK GRAY, WET, FINE GRAINED
2
7.5'
SILT, GRAY, ORGANIC MATERIAL (LAYERS OF PEAT, ABUNDANT VEGETATION)
10.5'
0
SILTY SAND, GRAY, WET, FINE GRAINED SAND, ORGANIC MATERIAL (ABUNDANT ROOTS AND REED GRASS)
13.0'
46
GRAVELLY SAND, BROWN, WET, FINE TO MEDIUM GRAINED SAND, ROUNDED GRAVEL TO 1"
19.0'
F
22
SAND, GRAY, WET, FINE GRAINED, THIN LAYERS OF SILT
23.0'
47
SILTY SAND, BROWN, MOIST, LOW PLASTICITY FINES, FINE GRAINED SAND
29.0'
61
SILTY SAND, BROWN WITH SOME IRON OXIDE STAINING, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
35.0'
58
SILTY SAND, GRAY-BROWN WITH SOME HEAVILY IRON OXIDE STAINED LAYERS, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
40.0'
2F-84-107
CONT'D
40.0'
47
SILTY SAND, GRAY-BROWN WITH SOME IRON OXIDE STAINING, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
50.0'
8
SILTY SAND, BROWN, MOIST TO WET, LOW PERCENTAGE OF NON-PLASTIC FINES
57.0'
F
8
SILTY SAND, AS ABOVE EXCEPT WITH SOME CEMENTED FRAGMENTS AND COARSE GRAINED SAND
70.0'
A
SILTY SAND, WITH ROCK FRAGMENTS, GREENISH BROWN, HIGHLY CEMENTED
71.0'
SHALE, DARK GRAY, HIGHLY WEATHERED, SOME SOFT AND HARD LAYERS
75.0'
GRAPHIC SCALE
1" = 2'
0 1' 2' 4' 6' 8' 10'
DRAWING REDUCED ONE HALF SIZE
VERTICAL SCALE: 1" = 2'
NOTES:
1. FOR LEGEND, NOTES, AND LOCATION OF EXPLORATIONS, SEE SHEET NO. B11.
2. FOR OTHER LOGS OF BORINGS, SEE SHEET NO'S. B13, B14 & B15
REVISION
DATE
DESCRIPTION
BY
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA
DESIGNED: R.L.T.
DRAWN: R.L.T., V.L.H.
CHECKED: FL
PRESIDIO OF SAN FRANCISCO
CALIFORNIA
COMMISSARY BUILDING
BORING LOG
SUBMITTED:
CHIEF, SOILS DESIGN SECTION
DATE APPROVED: 6-11-87
SCALE: 1" = 2'
SHEET: B12
SHEETS OF 15
SPEC. No. 7283
FILE No. 192-25-504


7F-8N-109
DEPTH
N GR SA FI LL PI MC
0.25' BITUMINOUS SURFACE
0.8' SANDY GRAVEL (BASE COURSE) GRAVEL TO 1"
SANDY GRAVEL, ONE COBBLE 8", CONCRETE AND BRICK FRAGMENTS (FILL)
4.0'
F 4 SAND, GRAY, WET, FINE TO MEDIUM GRAINED
6.2'
SILT, DARK GRAY, MOIST, ORGANIC MATERIAL (SOME VEGETATION, ROOTS, ETC.)
10.0'
SC 0 56 42 26 11 37
CLAYEY SAND, GRAY AND DARK GRAY, MOIST, ORGANIC MATERIAL STRONG ODOR (LAYERS OF PEAT)
15.5'
F 7 SILTY SAND, GRAY, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE TO MEDIUM GRAINED SAND
19.0'
SILT, GRAY, MOIST, ORGANIC MATERIAL (TRACES OF VEGETATION)
20.0'
AT 19.0' SILT, ORGANIC MATERIAL (TOP OF TUBE)
SP-SM 0 92 8 - NP 21
SILTY SAND, GRAY, FINE TO MEDIUM GRAINED SAND
24.0'
SILTY SAND, GRAY, WET, SCATTERED GRAVEL TO 3/4", ORGANIC MATERIAL
25.5'
F 34
SILTY SAND, BROWN, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
30.0'
SM 0 67 33 - NP 15
F 80
SILTY SAND, BROWN WITH ORANGE-BROWN NODULES AND LAYERS WITH IRON OXIDE STAINING, FINE GRAINED SAND
40.0'
SM 0 80 20 23 2 -
G_s = 2.78
F
SILTY SAND, BROWN, FINE GRAINED SAND, SOME SLIGHTLY CEMENTED NODULES
50.0'
7F-8N-109 (CONT'D)
50.0'
0 81 19 - - 19
SILTY SAND, BROWN, LOW PLASTICITY FINES, FINE GRAINED SAND
F 98
60.0'
FROM 60.0' TO 62.5' LOST ALL OF TUBE SAMPLE EXCEPT 0.5'
SILTY SAND, BROWN, WET, FINE GRAINED SAND
68.0'
SM 0 71 29 - NP
SILTY SAND, BLUE-GRAY, FINE GRAINED SAND
73.0'
SANDY CLAY, BLUE-GRAY, MOIST, MEDIUM PLASTICITY FINES, FINE-GRAINED SAND
75.5'
F 83
SILTY SAND, BLUE-GRAY, MOIST, FINE GRAINED SAND
78.0'
CLAYEY SAND, GREENISH GRAY, MOIST, MEDIUM PLASTICITY FINES, FINE GRAINED SAND
80.0'
AT 80.0' CLAYEY SAND (TOP OF TUBE)
CH 0 1 99 68 30 38
CLAY, BLUE-GRAY
F 15
CLAY, AS ABOVE EXCEPT DAMP, MEDIUM PLASTICITY FINES
90.0'
40 46 14 - - 55
F
SILTY GRAVELLY SAND, DARK GRAY, GRAVEL TO 1-1/2"
95.0'
SHALE, HIGHLY WEATHERED
100.0'
SANDSTONE, HIGHLY WEATHERED
100.8'
2F-8N-110
DEPTH
N
0 0.4' BITUMINOUS OVERLAY
0.5' BITUMINOUS SURFACE
1.0' SANDY GRAVEL (BASE COURSE)
2.0' CLAYEY GRAVELLY SAND, DARK GRAY, DAMP, FINE TO COARSE GRAINED SAND, ANGULAR GRAVEL TO 1", (FILL)
3.0' SANDY CLAY, YELLOW-BROWN, MOIST, MEDIUM PLASTICITY FINES, FINE GRAINED SAND, SCATTERED GRAVEL TO 1-1/4"
3.5' ▽
4.0' SAND, BROWN, WET, FINE GRAINED
SAND, GRAY, WET, FINE GRAINED
3 ORGANIC MATERIAL (PEAT), DARK GRAY FIBEROUS, STRONG ODOR
7.5'
SILTY SAND, GRAY-BROWN, WET, FINE GRAINED SAND, SOME ORGANIC MATERIAL (ROOTS, ETC.)
23
12.0'
SAND, BROWN, WET, FINE GRAINED
50
F
19.0'
SAND, BROWN, SATURATED, THIN LAYER OF CLAYEY SAND
FROM 20' TO 22.0' THIN LAYERS OF SILTY SAND, GRAY
19
22.0'
CLAYEY GRAVELLY SAND, BROWN, WET, FINE GRAINED SAND, ANGULAR GRAVEL TO 2"
24.0'
38
SILTY SAND, BROWN WITH GRAY-BROWN LAYERS AND SOME IRON OXIDE STAINING, WET, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
55
35.0'
2F-8N-110 (CONT'D)
DEPTH
N
35.0'
75
SILTY SAND, GRAY-BROWN WITH THIN LAYERS OF HEAVILY IRON OXIDE STAINED MATERIAL, WET, LOW PERCENTAGE NON-PLASTIC FINES, FINE GRAINED SAND
40.0'
71
SILTY SAND, GRAY-BROWN WITH IRON OXIDE STAINING, WET, FINE GRAINED SAND
50.0'
F 45
SILTY SAND, GRAY-BROWN, WET, LOW PLASTICITY FINES, FINE GRAINED SAND
58.0'
GRAVELLY CLAYEY SAND, BROWN, MOIST, FINE GRAINED SAND, ANGULAR GRAVEL TO 3"
60.0'
8
GRAVELLY SANDY CLAY (SHALE, HIGHLY WEATHERED), GREENISH GRAY, DAMP, MEDIUM PLASTICITY FINES, FINE GRAINED SAND, ANGULAR GRAVEL TO 1"
68.0'
BEDROCK, WEATHERED, LOWER PERCENTAGE OF FINES, INCREASED PERCENTAGE OF FRESH FRAGMENTS
70.1'
VERTICAL SCALE: 1" = 2'
GRAPHIC SCALE
1" = 2'
0 1' 2' 4' 6' 8' 10'
NOTE:
1. FOR LEGEND, NOTES, AND LOCATIONS OF EXPLORATIONS, SEE SHEET NO. B1.1
2. FOR OTHER LOGS OF BORINGS, SEE SHEET NO'S. B1.2, B1.4, & B1.5.
DRAWING REDUCED ONE HALF SIZE
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA
PRESIDIO OF SAN FRANCISCO CALIFORNIA
COMMISSARY BUILDING
BORING LOG
R.L.T.
A.E.G., R.M.E.
PL
CHIEF, SOILS DESIGN SECTION
B1.3
192-25-504
7283



NOTES:

1. FOR LEGEND, NOTES, AND LOCATION OF EXPLORATIONS, SEE SHEET NO. B&I
2. FOR OTHER LOGS OF BORINGS, SEE SHEET NO'S. B1.2, B1.3, & B1.5

GRAPHIC SCALE

1" = 2' 0 1' 2' 4' 6' 8' 10'

DRAWING REDUCED ONE HALF SIZE

REVISION	DATE	DESCRIPTION	BY	CK

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

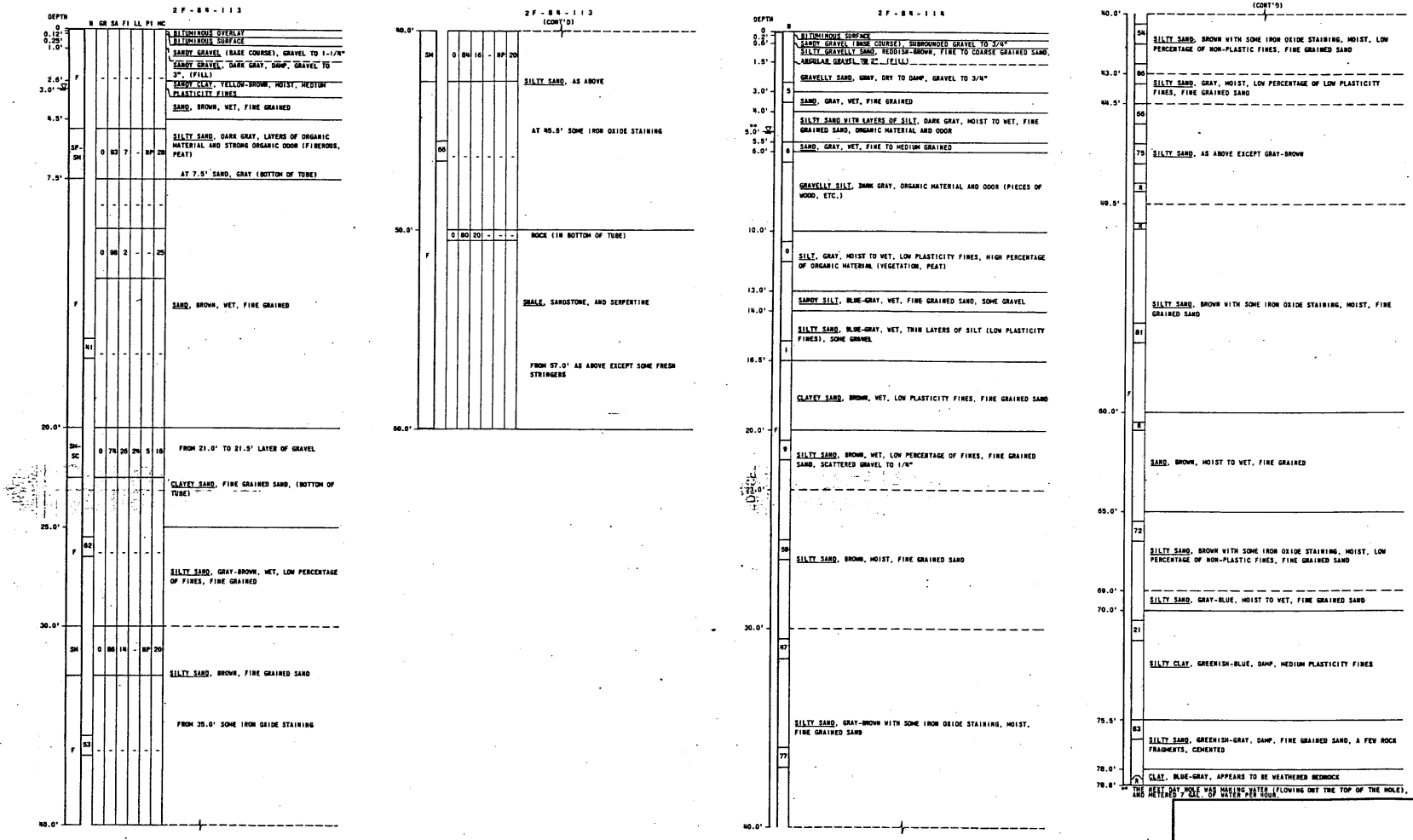
DESIGNED: R.L.T.
DRAWN: A.E.B., R.H.E.
CHECKED: PL
SUBMITTED: CHIEF, SOILS DESIGN SECTION
DATE APPROVED: 6-11-81

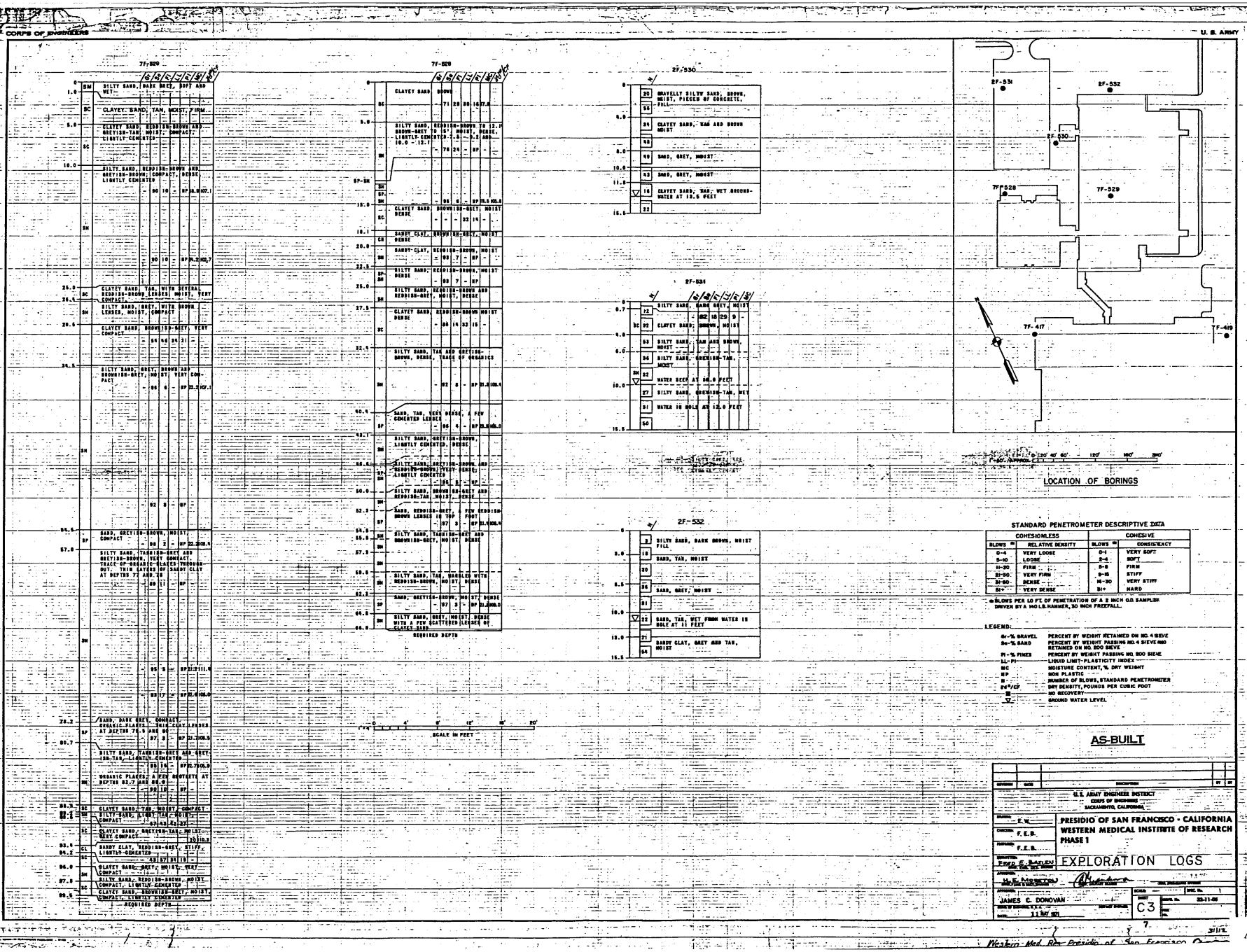
PRESIDIO OF SAN FRANCISCO CALIFORNIA

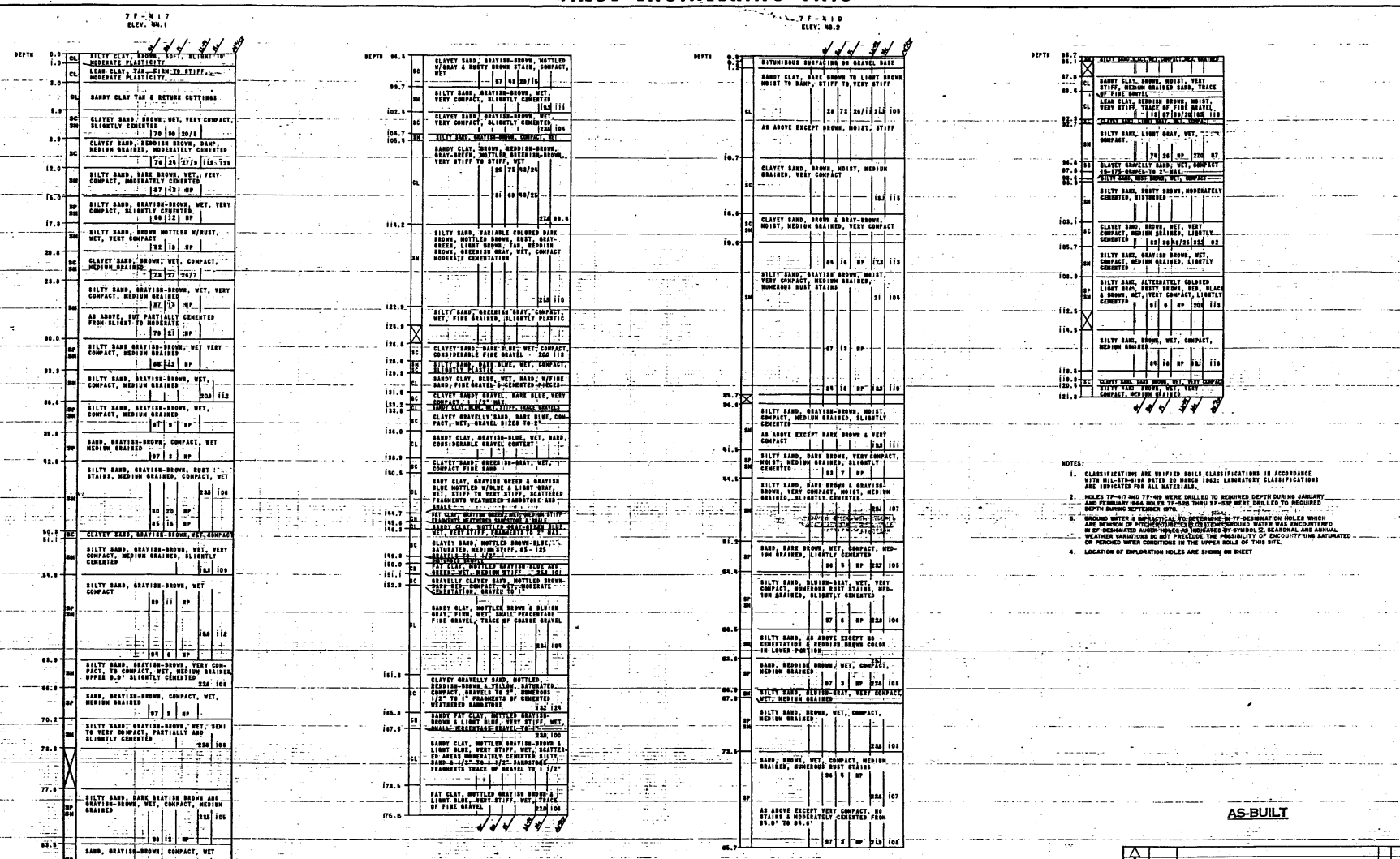
COMMISSARY BUILDING

BORING LOG

SCALE: 1/2" SHEET 8 OF 10 | DWG. No. 7283 | FILE No. 192-25-504


2F-B4-113
DEPTH
N GR SA FI LL PI MC
0.12'
0.25'
1.0'
2.6'
3.0'
4.5'
7.5'
20.0'
25.0'
30.0'
40.0'
BITUMINOUS OVERLAY
BITUMINOUS SURFACE
SANDY GRAVEL (BASE COURSE), GRAVEL TO 1-1/4"
SANDY GRAVEL, DARK GRAY, DAMP, GRAVEL TO 3", (FILL)
SANDY CLAY, YELLOW-BROWN, MOIST, MEDIUM PLASTICITY FINES
SAND, BROWN, WET, FINE GRAINED
SILTY SAND, DARK GRAY, LAYERS OF ORGANIC MATERIAL AND STRONG ORGANIC ODOR (FIBEROUS, PEAT)
AT 7.5' SAND, GRAY (BOTTOM OF TUBE)
SAND, BROWN, WET, FINE GRAINED
FROM 21.0' TO 21.5' LAYER OF GRAVEL
CLAYEY SAND, FINE GRAINED SAND, (BOTTOM OF TUBE)
SILTY SAND, GRAY-BROWN, WET, LOW PERCENTAGE OF FINES, FINE GRAINED
SILTY SAND, BROWN, FINE GRAINED SAND
FROM 35.0' SOME IRON OXIDE STAINING
2F-B4-113 (CONT'D)
40.0'
50.0'
60.0'
SILTY SAND, AS ABOVE
AT 45.5' SOME IRON OXIDE STAINING
ROCK (IN BOTTOM OF TUBE)
SHALE, SANDSTONE, AND SERPENTINE
FROM 57.0' AS ABOVE EXCEPT SOME FRESH STRINGERS
2F-B4-114
DEPTH
0.2'
0.6'
1.5'
3.0'
4.0'
5.0'
5.5'
6.0'
10.0'
13.0'
14.0'
16.5'
20.0'
22.0'
30.0'
40.0'
BITUMINOUS SURFACE
SANDY GRAVEL (BASE COURSE), SUBROUNDED GRAVEL TO 3/4"
SILTY GRAVELLY SAND, REDDISH-BROWN, FINE TO COARSE GRAINED SAND, ANGULAR GRAVEL TO 2", (FILL)
GRAVELLY SAND, GRAY, DRY TO DAMP, GRAVEL TO 3/4"
SAND, GRAY, WET, FINE GRAINED
SILTY SAND WITH LAYERS OF SILT, DARK GRAY, MOIST TO WET, FINE GRAINED SAND, ORGANIC MATERIAL AND ODOR
SAND, GRAY, WET, FINE TO MEDIUM GRAINED
GRAVELLY SILT, DARK GRAY, ORGANIC MATERIAL AND ODOR (PIECES OF WOOD, ETC.)
SILT, GRAY, MOIST TO WET, LOW PLASTICITY FINES, HIGH PERCENTAGE OF ORGANIC MATERIAL (VEGETATION, PEAT)
SANDY SILT, BLUE-GRAY, WET, FINE GRAINED SAND, SOME GRAVEL
SILTY SAND, BLUE-GRAY, WET, THIN LAYERS OF SILT (LOW PLASTICITY FINES), SOME GRAVEL
CLAYEY SAND, BROWN, WET, LOW PLASTICITY FINES, FINE GRAINED SAND
SILTY SAND, BROWN, WET, LOW PERCENTAGE OF FINES, FINE GRAINED SAND, SCATTERED GRAVEL TO 1/4"
SILTY SAND, BROWN, MOIST, FINE GRAINED SAND
SILTY SAND, GRAY-BROWN WITH SOME IRON OXIDE STAINING, MOIST, FINE GRAINED SAND
2F-B4-114 (CONT'D)
40.0'
43.0'
44.5'
49.5'
60.0'
65.0'
69.0'
70.0'
75.5'
78.0'
78.8'
SILTY SAND, BROWN WITH SOME IRON OXIDE STAINING, MOIST, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
SILTY SAND, GRAY, MOIST, LOW PERCENTAGE OF LOW PLASTICITY FINES, FINE GRAINED SAND
SILTY SAND, AS ABOVE EXCEPT GRAY-BROWN
SILTY SAND, BROWN WITH SOME IRON OXIDE STAINING, MOIST, FINE GRAINED SAND
SAND, BROWN, MOIST TO WET, FINE GRAINED
SILTY SAND, BROWN WITH SOME IRON OXIDE STAINING, MOIST, LOW PERCENTAGE OF NON-PLASTIC FINES, FINE GRAINED SAND
SILTY SAND, GRAY-BLUE, MOIST TO WET, FINE GRAINED SAND
SILTY CLAY, GREENISH-BLUE, DAMP, MEDIUM PLASTICITY FINES
SILTY SAND, GREENISH-GRAY, DAMP, FINE GRAINED SAND, A FEW ROCK FRAGMENTS, CEMENTED
CLAY, BLUE-GRAY, APPEARS TO BE WEATHERED BEDROCK
** THE NEXT DAY HOLE WAS MAKING WATER (FLOWING OUT THE TOP OF THE HOLE), AND METERED 7 GAL. OF WATER PER HOUR.
VERTICAL SCALE: 1" = 2'
GRAPHIC SCALE
1" = 2'
0 1' 2' 4' 6' 8' 10'
NOTES:
1. FOR LEGEND, NOTES, AND LOCATION OF EXPLORATIONS, SEE SHEET NO. B4I .
2. FOR OTHER LOGS OF BORINGS, SEE SHEET NO'S. B12, B13, & B14.
DRAWING REDUCED ONE HALF SIZE
REVISION DATE DESCRIPTION BY CK
DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA
DESIGNED: R.L.T.
DRAWN: A.E.S., R.A.V.
CHECKED: PL
PRESIDIO OF SAN FRANCISCO CALIFORNIA
COMMISSARY BUILDING
BORING LOG
SUBMITTED: CHIEF, DATA BASES SECTION
DATE APPROVED: 3-57
SCALE: 1" = 2'
SPEC. No. 7283
SHEET B15 OF 15
FILE No. 192-25-504


U. S. ARMY
CORPS OF ENGINEERS
7F-529
7F-528
2F-530
2F-531
2F-532
SILTY SAND, DARK GREY, SOFT AND WET
CLAYEY SAND, TAN, MOIST, FIRM
CLAYEY SAND, REDDISH-BROWN AND GREYISH-TAN, MOIST, COMPACT, LIGHTLY CEMENTED
SILTY SAND, REDDISH-BROWN AND GREYISH-BROWN, COMPACT, DENSE, LIGHTLY CEMENTED
CLAYEY SAND, TAN, WITH SEVERAL REDDISH-BROWN LENSES, MOIST, VERY COMPACT
SILTY SAND, GREY, WITH BROWN LENSES, MOIST, COMPACT
CLAYEY SAND, BROWNISH-GREY, VERY COMPACT
SILTY SAND, GREY, BROWN AND BROWNISH-GREY, MOIST, VERY COMPACT
SAND, GREYISH-BROWN, MOIST, COMPACT
SILTY SAND, TANNISH-GREY AND GREYISH-BROWN, VERY COMPACT, TRACE OF ORGANIC PLACES THROUGHOUT. THIN LAYERS OF SANDY CLAY AT DEPTHS 7 AND 78
SAND, DARK GREY, COMPACT, ORGANIC FLAKES, THIN CLAY LENSES AT DEPTHS 75.5 AND 80
SILTY SAND, TANNISH-GREY AND GREYISH-TAN, LIGHTLY CEMENTED
ORGANIC FLAKES; A FEW SEAMETTS AT DEPTHS 82.7 AND 86.9
CLAYEY SAND, TAN, MOIST, COMPACT
SILTY SAND, LIGHT TAN, MOIST, COMPACT
CLAYEY SAND, GREYISH-TAN, MOIST, VERY COMPACT
SANDY CLAY, REDDISH-GREY, STIFF, LIGHTLY CEMENTED
CLAYEY SAND, GREY, MOIST, VERY COMPACT
SILTY SAND, REDDISH-BROWN, MOIST, COMPACT, LIGHTLY CEMENTED
CLAYEY SAND, BROWNISH-GREY, MOIST, COMPACT, LIGHTLY CEMENTED
REQUIRED DEPTH
CLAYEY SAND, BROWN
SILTY SAND, REDDISH-BROWN TO 12.7 BROWN-GREY TO 15.7, MOIST, DENSE, LIGHTLY CEMENTED 7.5 - 9.2 AND 10.0 - 12.7
SANDY CLAY, BROWNISH-GREY, MOIST, DENSE
SANDY CLAY, REDDISH-BROWN, MOIST
SILTY SAND, REDDISH-BROWN, MOIST, DENSE
SILTY SAND, REDDISH-BROWN AND REDDISH-GREY, MOIST, DENSE
CLAYEY SAND, REDDISH-BROWN MOIST DENSE
SILTY SAND, TAN AND GREYISH-BROWN, DENSE, TRACE OF ORGANICS
SAND, TAN, VERY DENSE, A FEW CEMENTED LENSES
SILTY SAND, GREYISH-BROWN, LIGHTLY CEMENTED, DENSE
SILTY SAND, GREYISH-BROWN AND REDDISH-BROWN, VERY DENSE, LIGHTLY CEMENTED
SILTY SAND, BROWNISH-GREY AND REDDISH-TAN, MOIST, DENSE
SAND, REDDISH-GREY, A FEW REDDISH-BROWN LENSES IN TOP FOOT
SILTY SAND, TANNISH-GREY AND BROWNISH-GREY, MOIST, DENSE
SAND, GREYISH-BROWN, MOIST, DENSE
SILTY SAND, GREY, MOIST, DENSE WITH A FEW SCATTERED LENSES OF CLAYEY SAND
GRAVELLY SILTY SAND, BROWN, MOIST, PIECES OF CONCRETE, FILL
CLAYEY SAND, TAN AND BROWN MOIST
SAND, GREY, MOIST
SAND, GREY, MOIST
CLAYEY SAND, TAN; WET GROUND-WATER AT 12.5 FEET
SILTY SAND, DARK GREY, MOIST
CLAYEY SAND, BROWN, MOIST
SILTY SAND, TAN AND BROWN, MOIST
SILTY SAND, GREYISH-TAN, MOIST
WATER SEEN AT 26.0 FEET
SILTY SAND, GREYISH-TAN, WET
WATER IN HOLE AT 12.0 FEET
SILTY SAND, DARK BROWN, MOIST FILL
SAND, TAN, MOIST
SAND, GREY, MOIST
SAND, TAN, WET FROM WATER IN HOLE AT 11 FEET
SANDY CLAY, GREY AND TAN, MOIST
SCALE IN FEET
2F-531
2F-532
2F-530
7F-528
7F-529
7F-417
7F-419
LOCATION OF BORINGS
STANDARD PENETROMETER DESCRIPTIVE DATA
COHESIONLESS
RELATIVE DENSITY
VERY LOOSE
LOOSE
FIRM
VERY FIRM
DENSE
VERY DENSE
COHESIVE
CONSISTENCY
VERY SOFT
SOFT
FIRM
STIFF
VERY STIFF
HARD
BLOWS PER LO FT OF PENETRATION OF A 2 INCH O.D. SAMPLER DRIVEN BY A 140 LB. HAMMER, 30 INCH FREEFALL.
LEGEND:
Gr-% GRAVEL — PERCENT BY WEIGHT RETAINED ON NO. 4 SIEVE
Sa-% SAND — PERCENT BY WEIGHT PASSING NO. 4 SIEVE AND RETAINED ON NO. 200 SIEVE
Fi-% FINES — PERCENT BY WEIGHT PASSING NO. 200 SIEVE
LL-PI — LIQUID LIMIT-PLASTICITY INDEX
MC — MOISTURE CONTENT, % DRY WEIGHT
NP — NON PLASTIC
N — NUMBER OF BLOWS, STANDARD PENETROMETER
γd — DRY DENSITY, POUNDS PER CUBIC FOOT
NO RECOVERY
GROUND WATER LEVEL
AS-BUILT
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA
PRESIDIO OF SAN FRANCISCO - CALIFORNIA
WESTERN MEDICAL INSTITUTE OF RESEARCH
PHASE I
EXPLORATION LOGS
FRED E. BAXLEY
JAMES C. DONOVAN
C3
Western Med Res Presidio of San Francisco



NOTES:

1. CLASSIFICATIONS ARE UNIFIED SOILS CLASSIFICATIONS IN ACCORDANCE WITH MIL-STD-619B DATED 20 MARCH 1962; LABORATORY CLASSIFICATIONS ARE INDICATED FOR ALL MATERIALS.
2. HOLES 7F-417 AND 7F-418 WERE DRILLED TO REQUIRED DEPTH DURING JANUARY AND FEBRUARY 1964. HOLES 7F-535 THRU 7F-538 WERE DRILLED TO REQUIRED DEPTH DURING SEPTEMBER 1970.
3. GROUND WATER IS PRACTICAL TO DETERMINE IN 7F-DESIGNATION HOLES WHICH ARE VENTED OR PITCHED WITH 2" CASING. GROUND WATER WAS ENCOUNTERED IN 2F-DESIGNATED AUGER HOLES AS INDICATED BY SYMBOL. SEASONAL AND ANNUAL WEATHER VARIATIONS DO NOT PRECLUDE THE POSSIBILITY OF ENCOUNTERING SATURATED OR PERCHED WATER CONDITIONS IN THE UPPER SOILS OF THIS SITE.
4. LOCATION OF EXPLORATION HOLES ARE SHOWN ON SHEET

AS-BUILT

U. S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
SACRAMENTO, CALIFORNIA

PRESIDIO OF SAN FRANCISCO - CALIFORNIA
WESTERN MEDICAL INSTITUTE OF RESEARCH
PHASE I

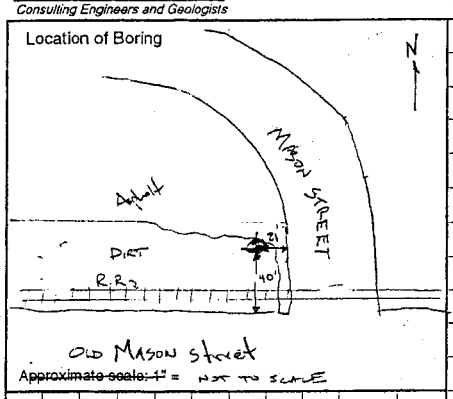
EXPLORATION LOGS

DESIGNED BY: T.S.G.
DRAWN BY: T.S.G.
CHECKED BY: F.E.B.
APPROVED BY: F.E.B.
FRED E. BAGLIEN
CHIEF, CON. DIV.
H.E. MORETON
CHIEF, ENGINEERING DIVISION
JAMES C. DONOVAN
CHIEF, DESIGN BRANCH
DATE: 21 MAY 1971
FILE NO.
SHEETS C4
SHEET NO. 8

Presidio of San Francisco WMIR C-2010

HTA Harlan Tait Associates
Consulting Engineers and Geologists

Field Log of Boring CF97-SB04



Logged by	KBC
Weather	Clear
Conditions	Flat Asphalt surface
Drilling Contractor	RNL
Drilling Equipment	B-53
Driller's Name	Bob, Reuben
Drilling Method	8" Rotary Wash
Sampling Methods	spt, shelby tube
Hammer Weight	140 lbs — Drop 30'
Start Time	1:00 pm — Date 10/16/97
Completion Time	3:20 — Date 10/16/97
Time of Backfilling	3:30 — Date 10/16/97 — By Cullinane
Boring Depth, feet	35.0'
Casing Depth, feet	—
Water Depth, feet	~5.0 ft
Time	
Date	
Surface Elevation	7 ft — Datum

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	Description
		SET	5'	CASING	(HSA)					GP	Brown Sandy Gravel (GP) - loose, dry to moist, (Fill)
		Smear	Corney	wash					1		
									2	SM	Brown Silty Sand (SM) - loose, dry, trace gravel (Fill)
									3		
									4	ML	Brown Sandy Silt (ML) - stiff, wet to saturated, organics, (Fill)
	SPT	1	6	6					5		
		2	6	6	Bag	5.0-6.5			6		SIEVE at 5.0 feet, USDA = LOAMY SAND
		3	6	6							
									7	SP	Grey Sand (SP) - loose, saturated, trace clam shells, (Fill?)
									8		
											increasing silt content below 8.5 ft
	SPT	0	6	6					9		
		1	12	6	Bag	9.0-10.5			10	ML	Dark Grey Clayey Silt (ML) - soft, saturated, organic smell, peat. (Bay Mud)
			6	6							

Project Name Crissy Field Project No. 1317.010 Sheet 1 of 3

FIG A14

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Field Log of Boring CF97-SB04 (continued)

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	Description
	Shelby		24	30	G	10.5-12.5			11	ML	γ_d = 45 pcf, w = 97.7%
									12		LL = 94, PI = 57 at 12.0 feet.
											TXUU = 337 (750) at 12.0 feet.
									13	SP	Grey Sand (SP) - med. dense, saturated, medium grained, well sorted, trace fines, (BEACH/DUNE SAND)
									14		
	SPT	7	6	6					15		
		11	6	6	Bag	15.0-16.5			16		
		10	6	6					17		
									18		
									19		
									20	ML	Dark Grey Clayey Silt (ML) - soft, saturated, organics, trace fine sand (Bay mud)
									21		
									22		
	SPT	1	6	6					23		
		1	6	6	Bag	23.0-24.5			24		W_n = 61.8% at 23.5 feet
		2	6	6					25		
									26		
									27		
									28		
									29		
	SPT	Auger	24	30		29.0-31.0			30		TXUU = 1697 (1500) at 30.0 feet. γ_d = 100 pcf w = 22.7%
									31		

FIG A15

Project Name Crissy Field Project No. 1317.010 Sheet 2 of 3

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log
									31	ML
									2	
									3	
									4	
									35	
									6	
									7	
									8	
									9	
									0	
									1	
									2	
									3	
									4	
									5	
									6	
									7	
									8	
									9	
									0	

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Field Log of Boring CP97-SB04
(continued)

- Increasing sand content with depth

BOH - 35.0'
GW encountered @ ~5 ft.
Hole backfilled w/ cuttings upon completion

Fig A16

Project Name Cassini Field Project No. 1317.010 Sheet 3 of 3

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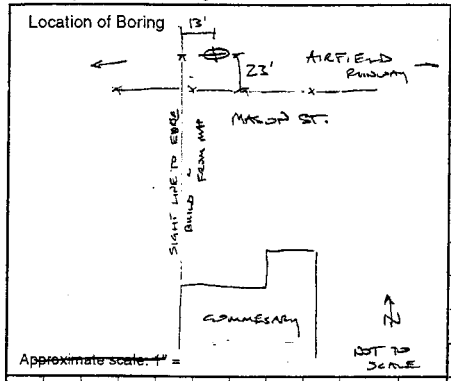
Field Log of Boring CF97-SB08

Location of Boring



~~Approximate scale: 1" =~~

Logged by	T. MATHISON
Weather	SUNNY
Conditions	ASPHALT PERIMETER
Drilling Contractor	BAY LANDS
Drilling Equipment	CME 75
Driller's Name	KIRT
Drilling Method	H.S.A.
Sampling Methods	STANDARD PEN. - SPT. MODIFIED CALIF.
Hammer Weight	AUTOMATIC TRIP 140# CHAIN. Drop 30 in.
Start Time	4 pm Date 6/15/97
Completion Time	Date
Time of Backfilling	Date By
Boring Depth, feet	
Casing Depth, feet	
Water Depth, feet	
Time	
Date	
Surface Elevation	7.0 ± Datum NGS NGVD TRUNING INC. TOPO 1997

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log
									0	
									1	
	BULK	SAMPLE				1.5 TP 4.0			2	
									3	
									4	
	MC	4	6	6	P	4.5			5	
		5	6	2	F	4.7				
		9	6	0				10	6	
									7	
	SPT	0	6	6		8.0			8	
		0	6	6			0.175			
		0	6	6		9.0	0.2	0	9	
									10	

ASPHALT 2½ INCHES THICK
BROWN GRAVELLY SAND ~8 INCHES THICK (BASE MATERIAL)
BROWN SAND (SP) - loose, dry, trace to some gravel in top 1 to 2 feet (FILL)
GREENISH GREY SAND (SP) - loose, saturated (HYDRAULIC FILL)
γ_{dry} = 108.9 pcf ; W_n = 19.5% at 4.7 feet.
W_n = 77.4% at 8.0 feet
GREY SILTY CLAYEY SILT (ML) - soft, saturated, some organics, hydrogen sulfide smell. (BAY MUD)

Project Name GEOTECHNICAL INVESTIGATION CRISSY FIELD SAN FRANCISCO CA Project No. _____ Sheet 1 of 2

FIG A.24

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Field Log of Boring CF97-SB08 (continued)

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log
									11	
	SHELBY TUBE	0	12		?	11.0			2	
		600							3	
									4	
									5	
									6	
									7	
	SPT	—	—		GRAB	~13			8	
									9	
									0	
									1	
									2	
									3	
									4	
									5	
									6	
									7	
									8	
									9	
									0	

γ_{dry} = 93.6 pcf ; W_n = 14.9% at 11.1 feet.
GREENISH GREY SAND (SP) - saturated (Olser Beach / Dune Sand)
- flowing into hollow stem prevents sample
- loose at 18.0 feet.
BOTTOM OF HOLE 18.0 FEET.
BACKFILLED WITH CUTTINGS UPON COMPLETION

FIG A.

Project Name CRISSY FIELD Project No. 1317.010 Sheet 2 of 2

HTA Harlan Tait Associates
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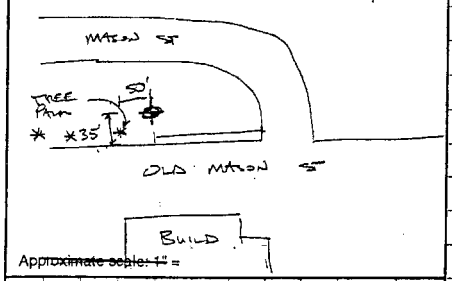
Field Log of Boring CF97-SB11

Location of Boring



Approximate scale: 1" =

Logged by	T. MATHISON (@ PROPOSED PIR LOC)
Weather	SUNNY
Conditions	ASPHALT LOADING RAMP
Drilling Contractor	RJL
Drilling Equipment	MOBIL B61
Driller's Name	RUESEN
Drilling Method	HOLLOW STEAM AUGER
Sampling Methods	MC, SPT.
Hammer Weight	140 lbs — Drop 30 inches (ROPE & CATHEAD)
Start Time	PM. — Date 10/17/97
Completion Time	Date 10/17/97
Time of Backfilling	Date — By
Boring Depth, feet	
Casing Depth, feet	
Water Depth, feet	
Time	
Date	
Surface Elevation 7.5	Datum 1929 NGVD (BUILD INC. 1997 TOPO)

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	
											ASPHALT
									1		BASE ROCK ~ 6 INCHES THICK
									2		REDDISH BROWN GRAVELLY SANDY CLAY (CL) stiff, moist (FILL)
									3		
	MC	18	6	6	9	3.0					γdry = 121 pcf; w/m = 12.4 % at 4.0 feet
		14	6	6	5	3.5			4		GREENISH GREY SAND (SP) -
		16	6	6	4	4.0		20			medium dense, moist, trace
									5		silty lenses (HYDRAULIC FILL)
											- Shells at 4.5 feet
	MC	2	6	6					6		LBROWN SILTY SAND (FILL)
		4	6	6	6.3						
		6	6	3	6.5			7	7		DARK GREY SILTY GRAVELLY SAND (SM) loose, saturated (FILL)
									8		SIEVE at 6.0 feet, USDA: SANDY LOAM
									9		
									10		GREENISH GREY CLAYEY SILT (ML) - soft, saturated (BAY MUD)

Project Name CRISSY FIELD Project No. 1317.010 Sheet 1 of 2

FIG A30

HTA Harlan Tait Associates
Consulting Engineers and Geologists

Field Log of Boring CF97 SB11 (continued)

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	
									1		
									2		GREENISH GREY CLAYEY SILT (ML) very soft, saturated, trace grasses. (BAY MUD)
	SPT	1	6	6	BAG	12.0			3		
		2	6	6				8			
		6	6	6	BAG	13.0			4		GREENISH GRAY SAND (SP) - loose to medium dense, saturated, trace to some thin lenses clayey silt. (BEACH / DUNE SAND)
									5		
									6		
									7		
									8		
		MATERIAL HEAVE IN AUGER			10 only				9		
	SPT	2	6	6	BAG				0		
		2	6	6				N/A			
		2	6	0					1		- clayey silt loss at 19.5 feet.
									2		BOTTOM OF HOLE 20.5 feet BACKFILLED WITH CUTTINGS UPON COMPLETION.
									3		
									4		
									5		
									6		
									7		
									8		
									9		
									0		

FIG A.

Project Name CRISSY FIELD Project No. 1317.010 Sheet 2 of 2

HTA Harlan Tait Associates
Consulting Engineers and Geologists

Field Log of Boring CF97-SB07 (continued)

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	
	SHELBY	0	30	30	G	10.0			11		- TXUU = 114 (700) at 10.8 feet. γ_d=59 pcf w=68.1%
		PST							2		
		100							3		DARK GREY SAND (SP) - medium dense, saturated, with lenses of medium stiff silty clay. (Old Beach/Dune Sand)
	MC	4	6	6	F	14.0		ANALY	4		
		10	6	6		14.5					
		25	6	6	G	15.0		24	5		- silty clay lens at 15.0 feet. - γ_{dry}=61.6 pcf. w=61.0% - SIEVE ANALYSES at 15.0 feet.
									6		
									7		
									8		
									9		
									20		
									1		GREY SAND (SP) - medium dense saturated with lenses of medium stiff silty clay (BAY MUD LENSES)
									2		
									3		
	SAMPLE FOR MAT. ID. only								4		
	HEAVE IN DRILL STEM										
	WHEN PUMP PUMPS!!								2.5		
	SPT	2	6	6					6		
		1	6	6	G	25.5					
		2	6	6	G	26.0			7		BOTTOM OF HOLE 26.5 feet. Backfilled upon completion with soil cuttings.
									8		
									9		
									0		

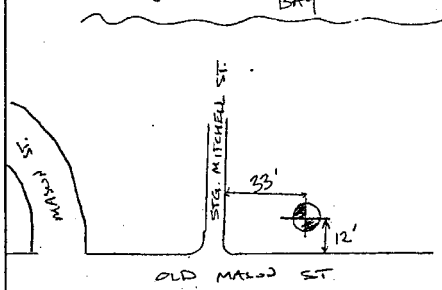
FIG A23

Project Name Geotechnical Inv. Crissy Field S.F. CA Project No. 1317.010 Sheet 2 of 2

HTA Harlan Tait Associates
Consulting Engineers and Geologists

Field Log of Boring CF97-SB07

Location of Boring



Approximate scale: 1" =

Logged by	T. MATHISON.
Weather	CLEAR
Conditions	BARE GROUND
Drilling Contractor	BAYLANDS
Drilling Equipment	CME 75
Driller's Name	KIET
Drilling Method	HOLLOW STEM AUGER.
Sampling Methods	SPT MODIFIED CALIFORNIA = MC
Hammer Weight	AUTOMATIC 140 lbs — Drop 30 inch
Start Time	9.50 AM — Date 10/15/97
Completion Time	Date
Time of Backfilling	Date — By
Boring Depth, feet	
Casing Depth, feet	
Water Depth, feet	
Time	
Date	
Surface Elevation 6.6'	Datum NGVD 1929 TOWILL INC 1997 TOPO

Drilling Rate (minutes per foot)	Sampler Type	Blows per 6 inches	Inches Driven	Inches Recovered	Sample Condition	Sample No./Depth	Penetrometer (tsf)	Standard Penetration Blow Count	Depth in Feet	Graphic Log	
									1		BROWN SAND (SP) - loose to medium dense, dry trace gravel (Fill)
									2		
					BULK	2-7'			3		
									4		
	MC	6	6	6	4.0	ANLY TEST			5		γ_{DRY} = 104 pcf w = 23.3% at 5.0 feet - greenish grey below 5.5 feet (HYDRAULIC FILL?) γ_{DRY} = 105 pcf w = 21.8 at 5.5 feet
		6	6	6	4.5						
		7	6	6	5.0			9	6		
									7		
		wt. of rods.							8		
		0	6	6		8.0			9		GREENISH GREY CLAYEY SILT (ML) Soft, saturated, (BAY MUD) γ_{DRY} = 55.4 pcf w = 74.1% at 8.5 feet
	MC	0	6	6		8.5				ML	
		0	6	6		9.0	1.5 1.75	0	10		

FIG A

Project Name GEOTECHNICAL INVESTIGATION CRISSY FIELD SAN FRANCISCO CA Project No. 1317.010 Sheet 1 of 2

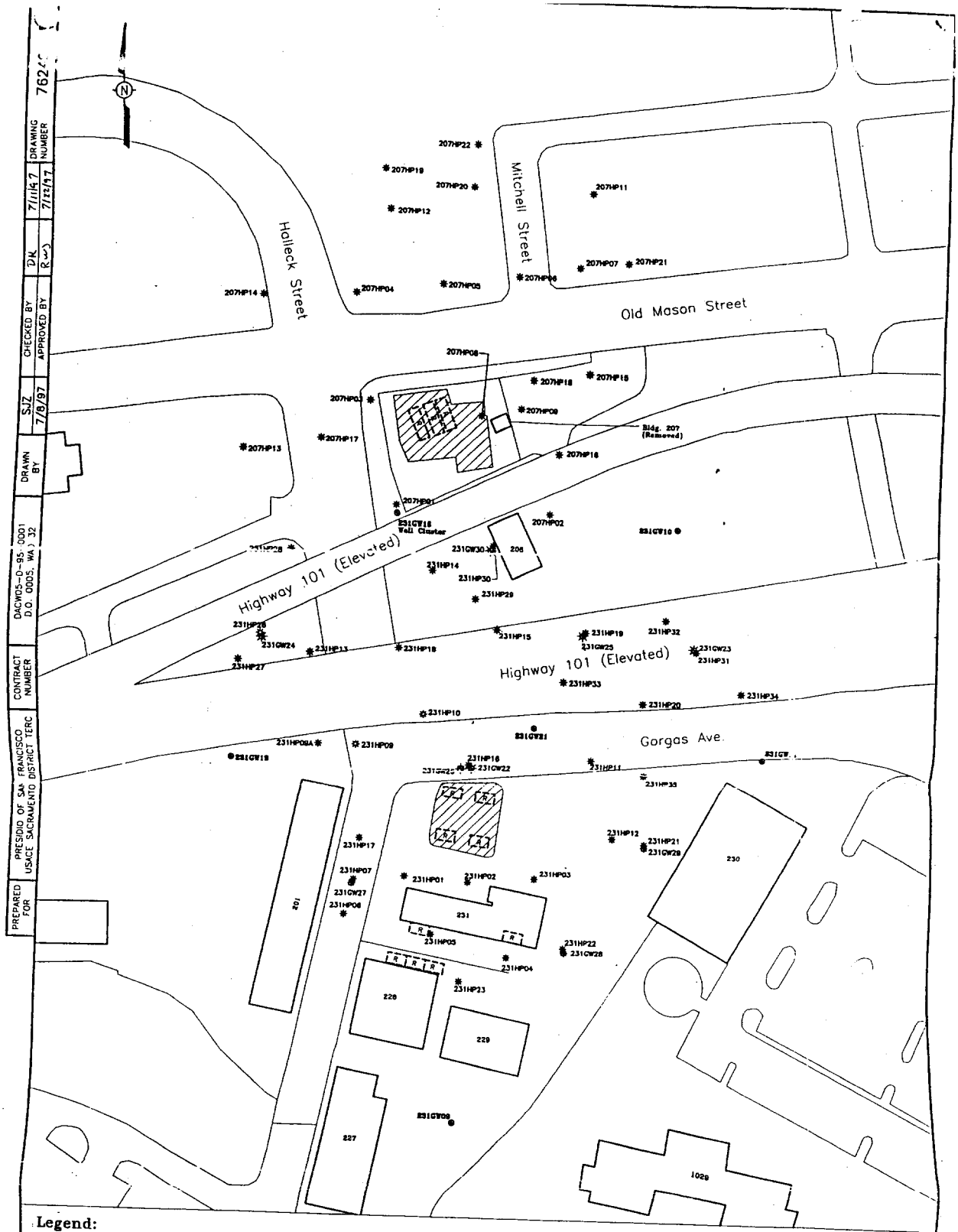
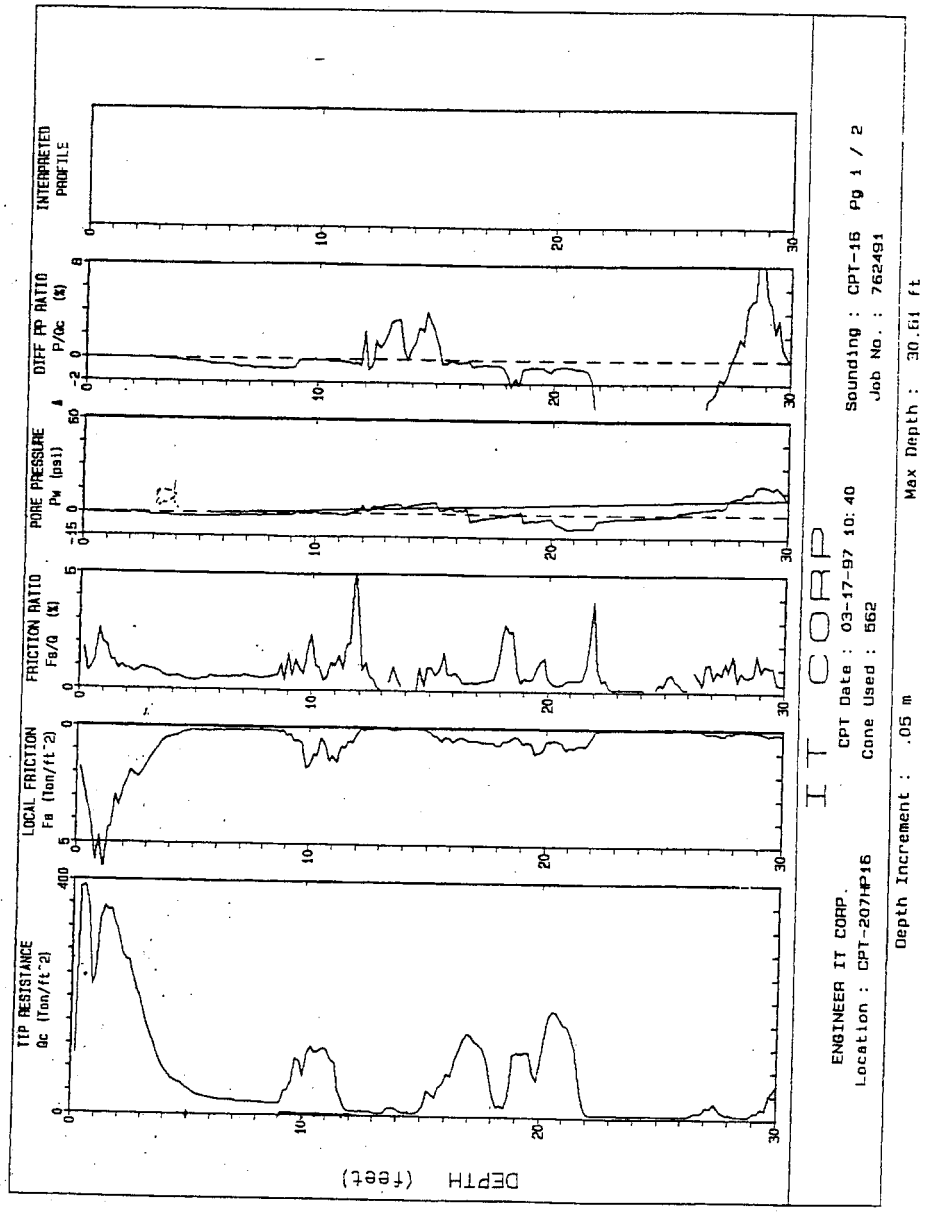


FIGURE 4
BUILDING 207/231 SITE
GROUNDWATER INVESTIGATORY
LOCATIONS

PRESIDIO OF SAN FRANCISCO



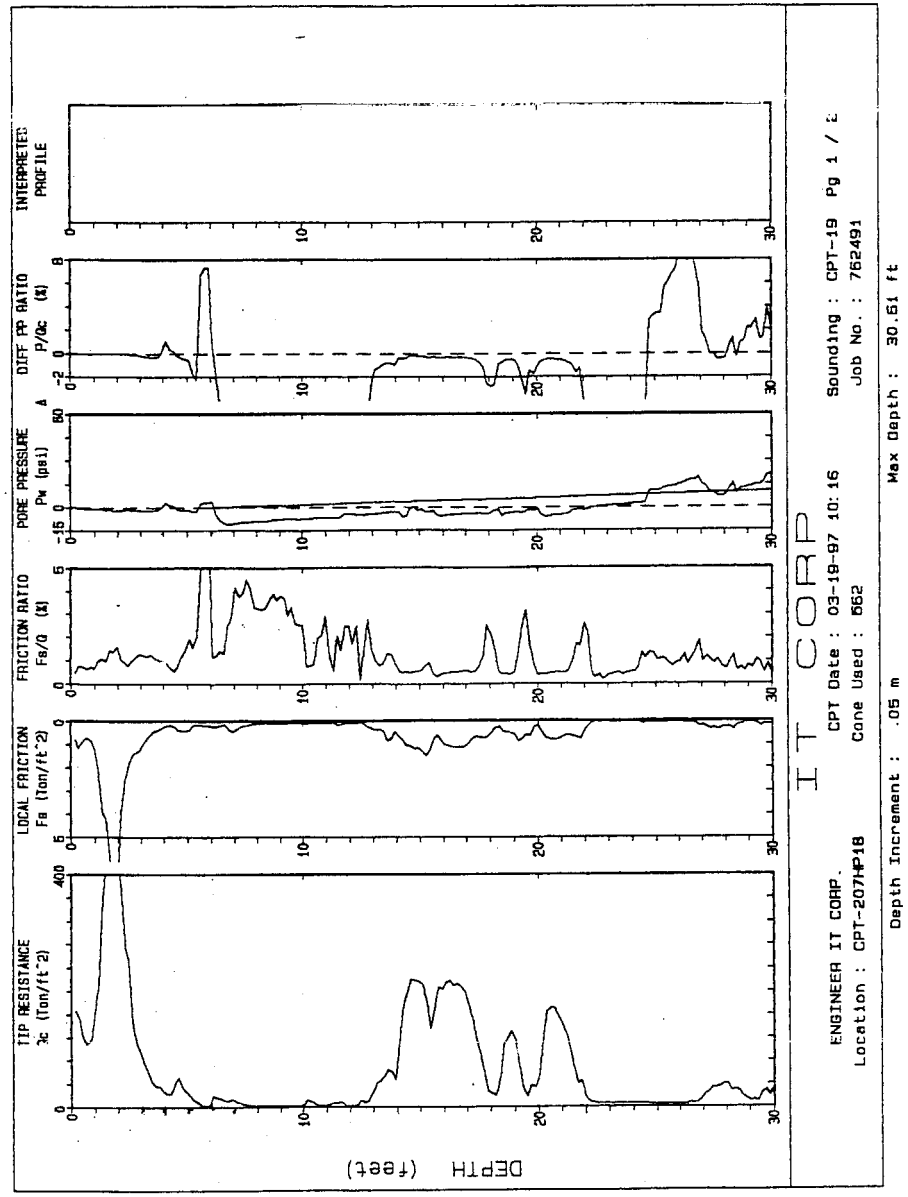
ENGINEER IT CORP.
On Site Loc:CPT-207HP16
Job No. :762491
Tot. Unit Wt. (avg) : 115 pcf

CPT Date 03-17-97 10:40
Cone Used :562
Water table (feet) : 5.479003

DEPTH (meters)	DEPTH (feet)	Qc (avg) (tsf)	Fs (avg) (tsf)	Rf (avg) (%)	SIGV' (tsf)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su tsf
0.30	1	282.08	3.86	1.37	0.03	sand	>90	>48	>50	UNDEFINED
0.60	2	335.67	3.98	1.19	0.08	sand	>90	>48	>50	UNDEFINED
0.95	3	230.24	1.94	0.84	0.15	sand	>90	>48	44	UNDEFINED
1.25	4	104.32	0.68	0.65	0.21	sand to silty sand	>90	46-48	25	UNDEFINED
1.55	5	52.80	0.24	0.45	0.26	sand to silty sand	60-70	44-46	13	UNDEFINED
1.85	6	31.85	0.17	0.52	0.32	silty sand to sandy silt	50-60	40-42	10	UNDEFINED
2.15	7	25.30	0.15	0.57	0.34	silty sand to sandy silt	40-50	38-40	8	UNDEFINED
2.45	8	22.22	0.12	0.55	0.37	silty sand to sandy silt	<40	38-40	7	UNDEFINED
2.75	9	21.05	0.18	0.84	0.40	sandy silt to clayey silt	UNDFND	UNDFD	8	2.05
3.05	10	67.27	0.91	1.35	0.42	silty sand to sandy silt	60-70	42-44	21	UNDEFINED
3.35	11	112.78	1.00	0.89	0.45	sand to silty sand	80-90	44-46	27	UNDEFINED
3.65	12	46.38	0.76	1.64	0.47	silty sand to sandy silt	50-60	40-42	15	UNDEFINED
3.95	13	7.77	0.04	0.56	0.50	sensitive fine grained	UNDFND	UNDFD	4	.70
4.25	14	9.82	0.03	0.32	0.52	sandy silt to clayey silt	UNDFND	UNDFD	4	.90
4.55	15	7.15	0.02	0.32	0.55	sensitive fine grained	UNDFND	UNDFD	3	.63
4.85	16	40.25	0.36	0.88	0.58	silty sand to sandy silt	40-50	38-40	13	UNDEFINED
5.15	17	104.37	0.47	0.45	0.60	sand	70-80	42-44	20	UNDEFINED
5.45	18	114.32	0.59	0.52	0.63	sand	70-80	42-44	22	UNDEFINED
5.75	19	39.22	0.48	1.22	0.65	silty sand to sandy silt	40-50	38-40	13	UNDEFINED
6.05	20	97.88	0.80	0.81	0.68	sand to silty sand	70-80	42-44	23	UNDEFINED
6.40	21	157.91	0.61	0.39	0.71	sand	80-90	44-46	30	UNDEFINED
6.70	22	86.42	0.64	0.74	0.74	sand to silty sand	60-70	40-42	21	UNDEFINED
7.00	23	6.05	0.02	0.40	0.76	sensitive fine grained	UNDFND	UNDFD	3	.47
7.35	24	5.56	0.01	0.12	0.79	sensitive fine grained	UNDFND	UNDFD	3	.42
7.65	25	5.45	0.01	0.24	0.82	sensitive fine grained	UNDFND	UNDFD	3	.40
8	26	5.25	0.02	0.31	0.84	sensitive fine grained	UNDFND	UNDFD	3	.37
8.25	27	11.48	0.09	0.78	0.87	sandy silt to clayey silt	UNDFND	UNDFD	4	.99
8.55	28	16.22	0.15	0.95	0.90	sandy silt to clayey silt	UNDFND	UNDFD	6	1.46
8.85	29	7.07	0.07	0.93	0.92	sensitive fine grained	UNDFND	UNDFD	3	.54
9.15	30	33.42	0.18	0.53	0.95	silty sand to sandy silt	<40	34-36	11	UNDEFINED

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 10

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****



IT CORP.

ENGINEER IT CORP.
On Site Loc:CPT-207HP18
Job No. :762491
Tot. Unit Wt. (avg) : 115 pcf

CPT Date 03-19-97 10:16
Cone Used :562
Water table (feet) : 5.675853

DEPTH (meters)	(feet)	Qc (avg) (tsf)	Fs (avg) (tsf)	Rf (avg) (%)	SIGV' (tsf)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su tsf
0.30	1	138.75	0.91	0.66	0.03	sand	>90	>48	27	[illegible]
0.60	2	389.27	4.98	1.28	0.08	sand	>90	>48	>50	UNDEFINED
0.95	3	191.60	1.92	1.00	0.15	sand	>90	>48	37	UNDEFINED
1.25	4	38.73	0.43	1.10	0.21	silty sand to sandy silt	60-70	42-44	12	UNDEFINED
1.55	5	32.68	0.32	0.97	0.26	silty sand to sandy silt	50-60	40-42	10	UNDEFINED
1.85	6	8.03	0.18	2.29	0.32	silty clay to clay	UNDFND	UNDFD	5	.77
2.15	7	13.15	0.28	2.11	0.35	clayey silt to silty clay	UNDFND	UNDFD	6	1.27
2.45	8	4.18	0.16	3.83	0.38	clay	UNDFND	UNDFD	4	.37
2.75	9	2.65	0.09	3.58	0.40	clay	UNDFND	UNDFD	3	.21
3.05	10	2.82	0.08	2.91	0.43	clay	UNDFND	UNDFD	3	.22
3.35	11	6.18	0.07	1.16	0.45	sensitive fine grained	UNDFND	UNDFD	3	.55
3.65	12	5.48	0.08	1.51	0.48	sensitive fine grained	UNDFND	UNDFD	3	.48
3.95	13	9.80	0.15	1.49	0.50	clayey silt to silty clay	UNDFND	UNDFD	5	.90
4.25	14	51.93	0.55	1.05	0.53	silty sand to sandy silt	50-60	40-42	17	UNDEFINED
4.55	15	192.07	1.00	0.52	0.56	sand	>90	46-48	37	UNDEFINED
4.85	16	185.32	1.04	0.56	0.58	sand	>90	44-46	36	UNDEFINED
5.15	17	210.10	1.09	0.52	0.61	sand	>90	46-48	40	UNDEFINED
5.45	18	104.12	0.76	0.73	0.63	sand to silty sand	70-80	42-44	25	UNDEFINED
5.75	19	75.42	0.43	0.57	0.66	sand to silty sand	60-70	40-42	18	UNDEFINED
6.05	20	50.63	0.61	1.21	0.69	silty sand to sandy silt	50-60	38-40	16	UNDEFINED
6.40	21	137.79	0.64	0.46	0.71	sand	80-90	42-44	26	UNDEFINED
6.70	22	66.12	0.62	0.94	0.74	sand to silty sand	50-60	40-42	16	UNDEFINED
7.00	23	7.10	0.06	0.79	0.77	sensitive fine grained	UNDFND	UNDFD	3	.58
7.35	24	6.36	0.03	0.47	0.80	sensitive fine grained	UNDFND	UNDFD	3	.50
7.65	25	5.90	0.07	1.13	0.82	sensitive fine grained	UNDFND	UNDFD	3	.44
7.95	26	5.18	0.05	0.95	0.85	sensitive fine grained	UNDFND	UNDFD	2	.37
8.25	27	9.87	0.12	1.24	0.88	clayey silt to silty clay	UNDFND	UNDFD	5	.83
8.55	28	34.50	0.31	0.90	0.90	silty sand to sandy silt	<40	36-38	11	UNDEFINED
8.85	29	21.47	0.18	0.83	0.93	sandy silt to clayey silt	UNDFND	UNDFD	8	1.98
9.15	30	21.37	0.14	0.66	0.95	sandy silt to clayey silt	UNDFND	UNDFD	8	1.96

Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 10

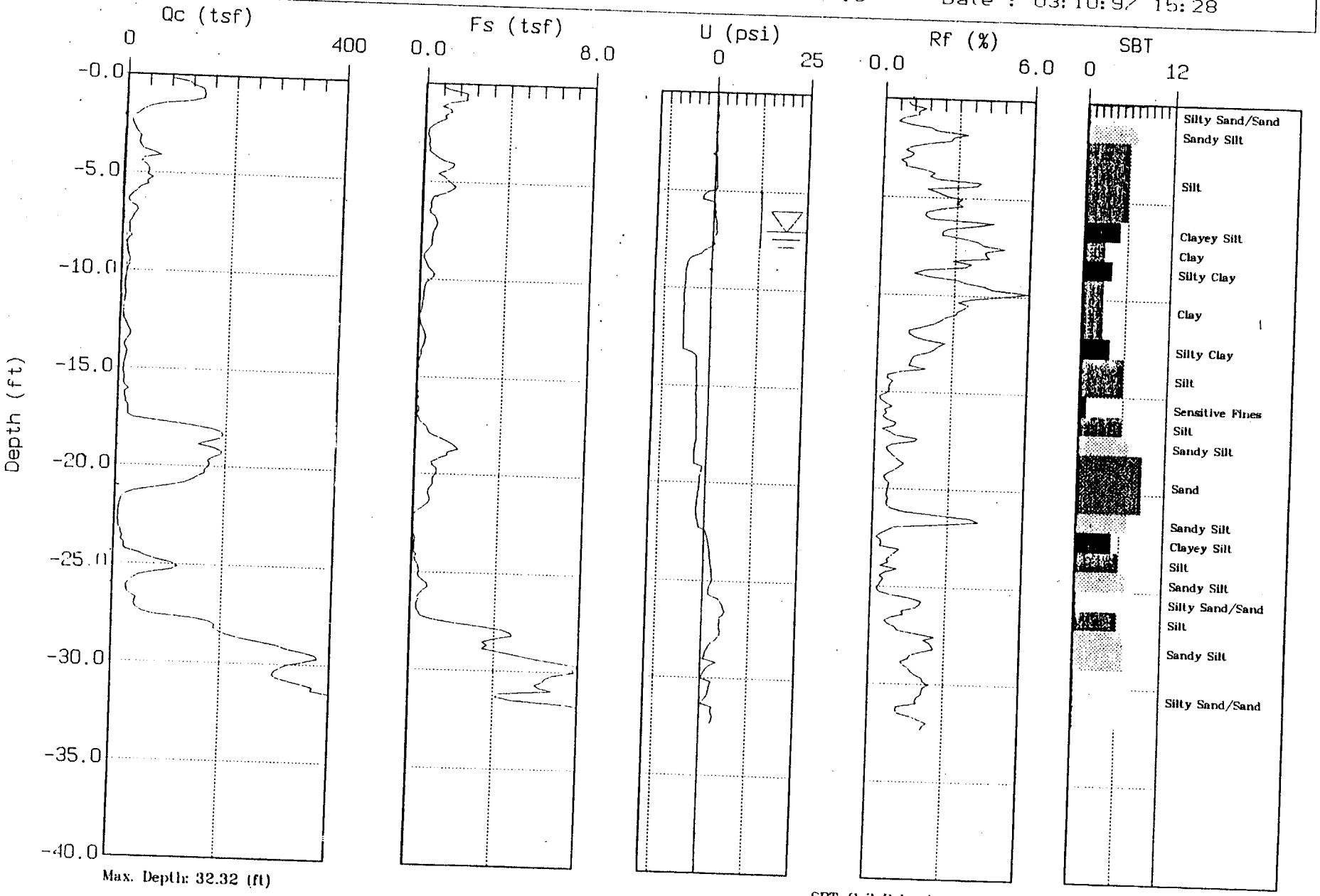
**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

GREGG

IT CORP: PRESIDIO

Site : BLDG. 231
Location : 23:HP15

Engineer : J. SELTZER
Date : 03:10:97 15:28

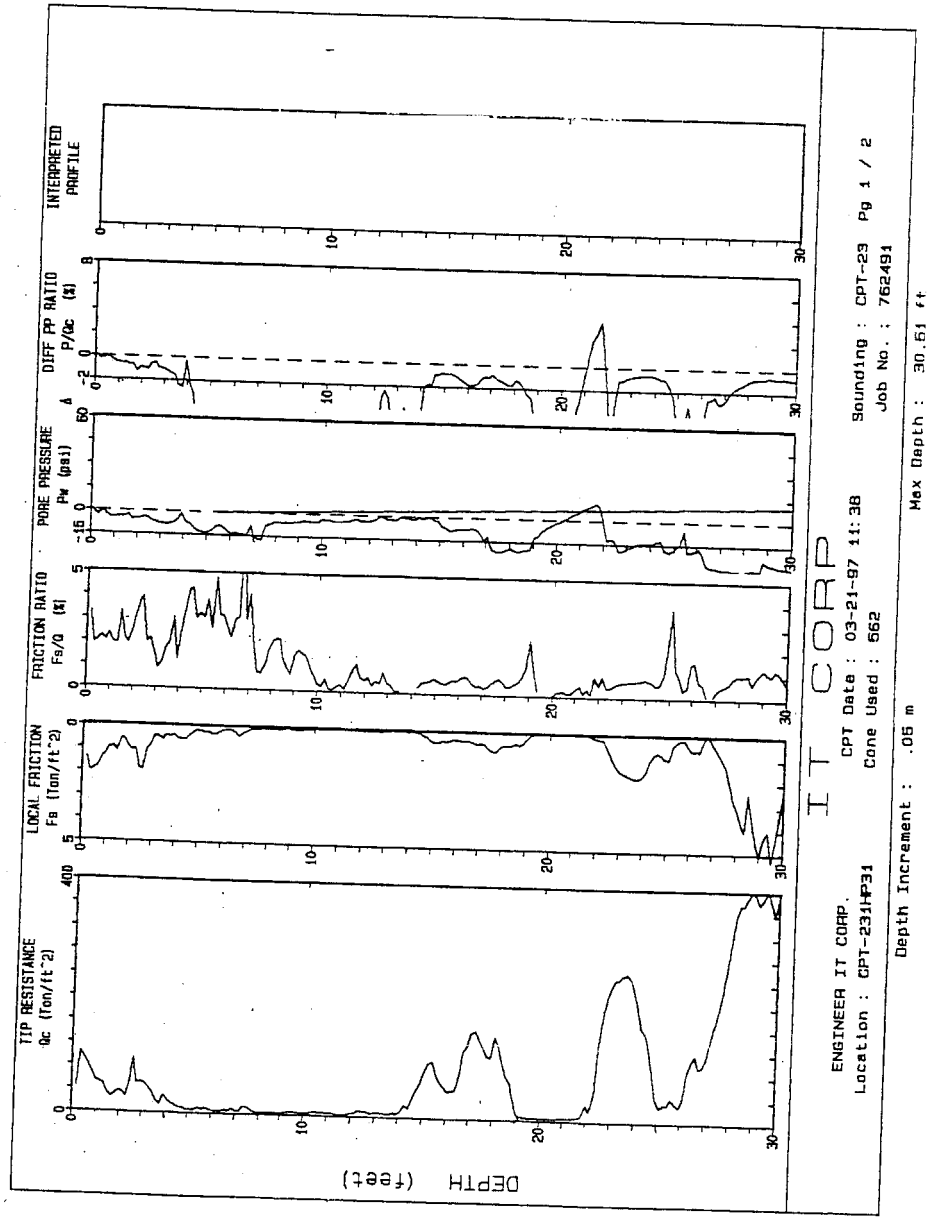


Max. Depth: 32.32 (ft)

Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson and Campanella 1988)

Estimated Phreatic Surface



IT CORP.

ENGINEER IT CORP.
On Site Loc:CPT-231HP31
Job No. :762491
Tot. Unit Wt. (avg) : 115 pcf

CPT Date 03-21-97 11:38
Cone Used :562
Water table (feet) : 5.577428

DEPTH (meters)	(feet)	Qc (avg) (tsf)	Fs (avg) (tsf)	Rf (avg) (%)	SIGV' (tsf)	SOIL BEHAVIOUR TYPE	Eq - Dr (%)	PHI deg.	SPT N	Su tsf
0.30	1	73.40	1.63	2.21	[illegible]	silty sand to sandy silt	>90	>48	23	UNDEFINED
0.60	2	38.10	0.85	2.23	0.08	sandy silt to clayey silt	UNDFND	UNDFD	15	3.80
0.95	3	51.20	1.17	2.28	0.15	sandy silt to clayey silt	UNDFND	UNDFD	20	5.10
1.25	4	27.77	0.48	1.73	0.21	sandy silt to clayey silt	UNDFND	UNDFD	11	2.75
1.55	5	10.72	0.39	3.62	0.26	clay	UNDFND	UNDFD	10	1.04
1.85	6	7.73	0.26	3.34	0.32	clay	UNDFND	UNDFD	7	.74
2.15	7	6.40	0.24	3.68	0.35	clay	UNDFND	UNDFD	6	.60
2.45	8	7.65	0.09	1.13	0.37	undefined	UNDFND	UNDFD	UDF	UNDEFINED
2.75	9	4.02	0.06	1.46	0.40	sensitive fine grained	UNDFND	UNDFD	2	.35
3.05	10	4.98	0.06	1.25	0.42	sensitive fine grained	UNDFND	UNDFD	2	.44
3.35	11	5.72	0.01	0.26	0.45	sensitive fine grained	UNDFND	UNDFD	3	.51
3.65	12	4.48	0.03	0.58	0.48	sensitive fine grained	UNDFND	UNDFD	2	.38
3.95	13	8.55	0.04	0.49	0.50	sensitive fine grained	UNDFND	UNDFD	4	.78
4.25	14	7.03	0.01	0.20	0.53	sensitive fine grained	UNDFND	UNDFD	3	.62
4.55	15	32.95	0.16	0.50	0.55	silty sand to sandy silt	40-50	38-40	11	UNDEFINED
4.85	16	77.68	0.40	0.52	0.58	sand to silty sand	60-70	42-44	19	UNDEFINED
5.15	17	76.92	0.39	0.51	0.61	sand to silty sand	60-70	42-44	18	UNDEFINED
5.45	18	130.45	0.64	0.49	0.63	sand	80-90	44-46	25	UNDEFINED
5.75	19	92.32	0.49	0.53	0.66	sand to silty sand	70-80	42-44	22	UNDEFINED
6.05	20	6.37	0.05	0.82	0.68	sensitive fine grained	UNDFND	UNDFD	3	.52
6.40	21	5.54	0.01	0.18	0.71	sensitive fine grained	UNDFND	UNDFD	3	.43
6.70	22	11.32	0.05	0.48	0.74	sandy silt to clayey silt	UNDFND	UNDFD	4	1.00
7.00	23	135.75	0.78	0.57	0.76	sand	70-80	42-44	26	UNDEFINED
7.35	24	238.46	1.71	0.72	0.79	sand	>90	44-46	46	UNDEFINED
7.65	25	99.05	0.90	0.91	0.82	sand to silty sand	60-70	40-42	24	UNDEFINED
7.95	26	34.93	0.34	0.97	0.85	silty sand to sandy silt	<40	36-38	11	UNDEFINED
8.25	27	101.73	0.32	0.32	0.87	sand	60-70	40-42	19	UNDEFINED
8.55	28	219.93	1.82	0.83	0.90	sand	>90	44-46	42	UNDEFINED
8.85	29	383.43	3.87	1.01	0.92	sand	>90	46-48	>50	UNDEFINED
9.15	30	384.57	4.00	1.04	0.95	sand	>90	46-48	>50	UNDEFINED

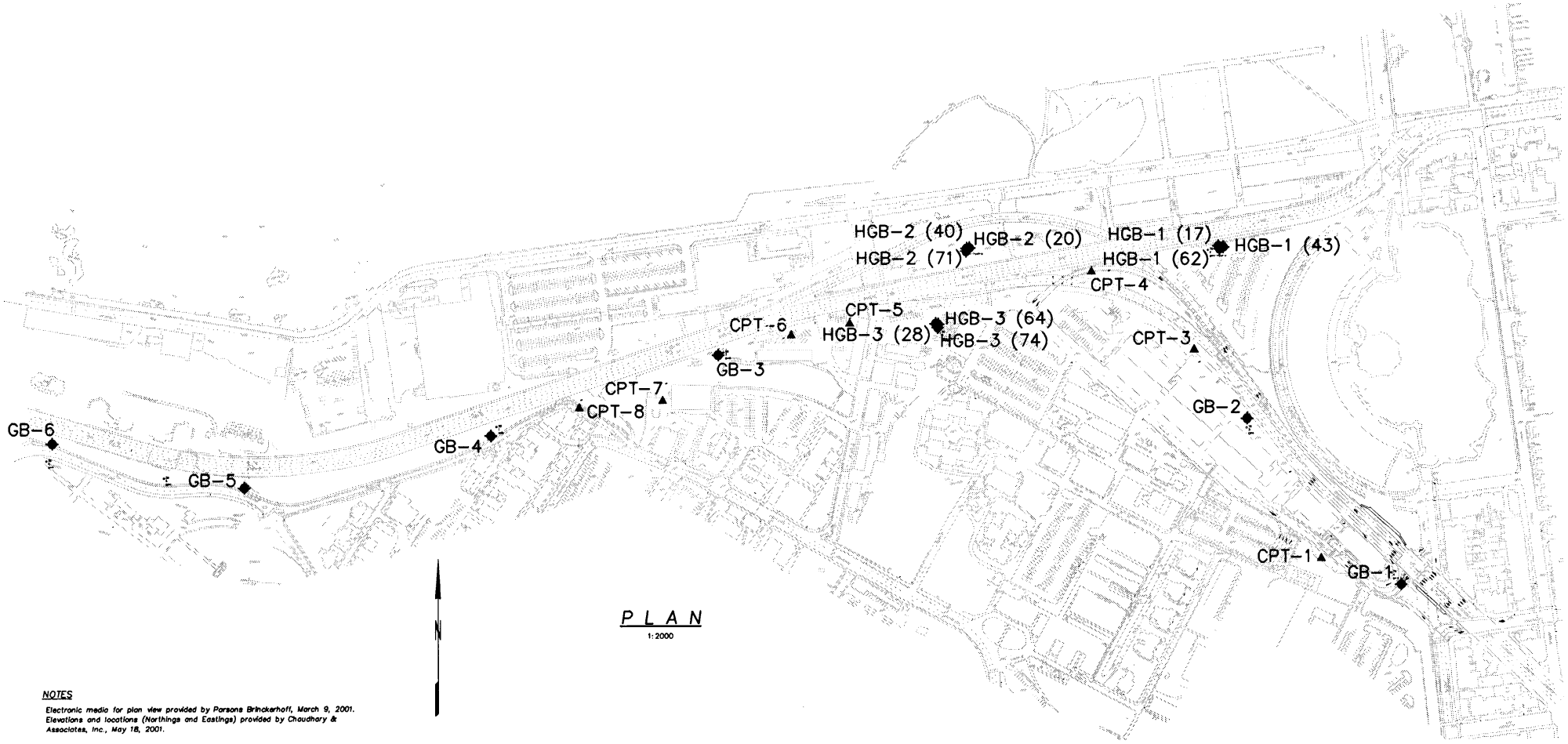
Dr - All sands (Jamiolkowski et al. 1985) PHI - Robertson and Campanella 1983 Su: Nk= 10

**** Note: For interpretation purposes the PLOTTED CPT PROFILE should be used with the TABULATED OUTPUT from CPTINTR1 (v 3.04) ****

APPENDIX B

CPT AND BORING LOGS

Source:
"Summary of Exploration and Testing, Doyle Drive Reconstruction Project, San Francisco, CA", Taber Consultants, March 2001.



PLAN
1:2000

NOTES

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001.
Elevations and locations (Northings and Eastings) provided by Choudhary & Associates, Inc., May 18, 2001.

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER	DOYLE DRIVE RECONSTRUCTION PROJECT
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE			KP	LOG OF TEST BORINGS No. 1

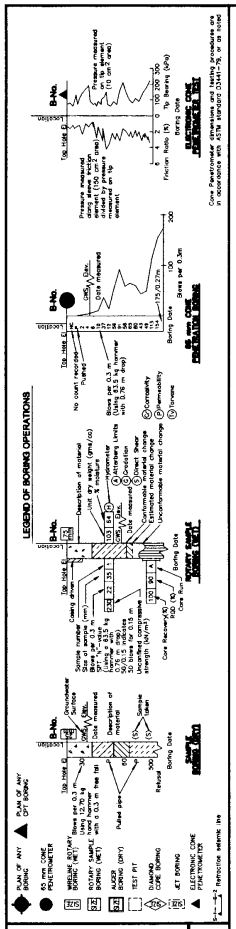
Taber Consultants Job No. 202/300/133 file id: 202300133q.dwg date: 05/24/01

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

CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET



WEATHERING DESCRIPTORS

Modified from United States Bureau of Reclamation, Engineering Geology Field manual.

Descriptors	Diagnostic features: Chemical weathering–Discoloration and/or oxidation: Body of rock	Chemical weathering–Discoloration and/or oxidation: Fracture surfaces†	Mechanical weathering–Grain boundary conditions (disaggregation) primarily for granitics and some coarse-grained sediments	Texture and solutioning: Texture	Texture and solutioning: Solutioning	General characteristic (strength, excavation, etc.)¶
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck. Almost always rock excavation except for naturally weak or weakly cemented rocks such as siltstones or shales.
Slightly weathered to fresh*						
Slightly weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened. With few exceptions, such as siltstones or shales, classified as rock excavation.
Moderately to slightly weathered*						
Moderately weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty", feldspar crystals are "cloudy".	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened. Depending on fracturing, usually is rock excavation except in naturally weak rocks such as siltstones or shales.
Intensely to moderately weathered*						
Intensely weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation, see grain boundary conditions.	All fracture surfaces are discolored or oxidized, surfaces friable	Partial separation, rock is friable; in semiarid conditions granitics are disaggregated	Texture altered by chemical disintegration (hydration, argillation)	Leaching of soluble minerals may be complete	Dull sound when struck with hammer, usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures, or veinlets. Rock is significantly weakened. Usually common excavation.
Very intensely weathered*						
Decomposed	Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay.		Complete separation of grain boundaries (disaggregated)	Resembles a soil, partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Always common excavation. Resistant minerals such as quartz may be present as "stringers" or "dikes".

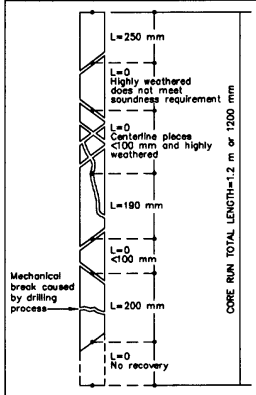
NOTE: This chart and its horizontal categories are more readily applied to rocks with feldspars and mafic minerals. Weathering in various sedimentary rocks, particularly limestones and poorly indurated sediments, will not always fit the categories established. This chart and weathering categories may have to be modified for particular site conditions or alterations such as hydrothermal effects; however, the basic framework and similar descriptors are to be used.

* Combination descriptors are permissible where equal distribution of both weathering characteristics are present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, dual descriptors should not be used where significant, identifiable zones can be delineated. When given as a range, only two adjacent terms may be combined (i.e., decomposed to slightly weathered or moderately weathered to fresh) are not acceptable.

† Does not include directional weathering along shears or faults and their associated features. For example, a shear zone that carried weathering to great depths into a fresh rock mass would not require the rock mass to be classified as weathered.

¶ These are generalizations and should not be used as diagnostic features for weathering or excavation classification. These characteristics vary to a large extent based on naturally weak materials or cementation and type of excavation.

RQD LOGGING



$$RQD = \frac{\sum \text{Length of sound core >100 mm pieces}}{\text{Total core run length}}$$

$$RQD = \frac{250+190+100}{1200} \times 100\%$$

RQD= 53% (fair)

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0 - 25%	VERY POOR
25 - 50%	POOR
50 - 75%	FAIR
75 - 90%	GOOD
90 - 100%	EXCELLENT

After Deere & Deere, 1989

FRACTURE DENSITY

Modified from United States Bureau of Reclamation, Engineering Geology Field manual.

FRACTURE DENSITY–Based on the spacing of all natural fractures in an exposure or core recovery lengths in boreholes; excludes mechanical breaks, shears, and shear zones; however, shear-distributed zones (fracturing outside the shear) are included. Descriptors for fracture density apply to all rock exposures such as tunnel walls, dozer trenches, outcrops, or foundation cut slopes and inverts, as well as boreholes. Descriptive criteria presented below are based on borehole cores where lengths are measured along the core axis, for other exposures the criteria is distance measured between fractures (size of blocks).

UNFRACTURED (FD0): No fractures.

VERY SLIGHTLY FRACTURED (FD1): Core recovered mostly in lengths greater than 1 m.

SLIGHTLY TO VERY SLIGHTLY FRACTURED (FD2) *

SLIGHTLY FRACTURED (FD3): Core recovered mostly in lengths from 300 to 1000 mm, with few scattered lengths less than 300 mm or greater than 1000 mm.

MODERATELY TO SLIGHTLY FRACTURED (FD4) *

MODERATELY FRACTURED (FD5): Core recovered mostly in 100 to 300 mm lengths with most lengths about 200 mm.

INTENSELY TO MODERATELY FRACTURED (FD6) *

INTENSELY FRACTURED (FD7): Lengths average from 30 to 100 mm with scattered fragmented intervals. Core recovered mostly in lengths less than 100 mm.

VERY INTENSELY TO INTENSELY FRACTURED (FD8) *

VERY INTENSELY FRACTURED (FD9): Core recovered mostly as chips and fragments with a few scattered short core lengths.

* Combinations of fracture densities (e.g. very intensely to intensely fractured or moderately to slightly fractured) are used where equal distribution of both fracture density characteristics are present over a significant interval or exposure, or where characteristics are "in between" the descriptor definitions.

ROCK HARDNESS/STRENGTH DESCRIPTORS

Alphanumeric Descriptor	Descriptor	Criteria
H1	Extremely hard	Core, fragment, or exposure cannot be scratched with knife or sharp pick; can only be chipped with repeated heavy hammer blows.
H2	Very hard	Can be scratched with knife or sharp pick. Core or fragment breaks with repeated heavy hammer blows.
H3	Hard	Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.
H4	Moderately hard	Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.
H5	Moderately soft	Can be grooved 1/16 inch (2 mm) deep by knife or sharp pick with moderate or heavy pressure. Core or fragment breaks with light hammer blow or heavy manual pressure.
H6	Soft	Can be grooved or gouged easily by knife or sharp pick with light pressure, can be scratched with fingernail. Breaks with light to moderate manual pressure.
H7	Very soft	Can be readily indented, grooved or gouged with fingernail, or carved with a knife. Breaks with light manual pressure.

Any bedrock unit softer than H7, very soft, is to be described using ASTM D-2488 consistency descriptors.

NOTE: Although "sharp pick" is included in these definitions, descriptions of ability to be scratched, grooved or gouged by a knife is the preferred criteria.
Modified from United States Bureau of Reclamation, Engineering Geology Field manual.

NOTES

1. Standard Penetration Tests (SPT) were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pulley. Drill rods were 1.61 inch diameter "A"-rods; sampler was driven without brass liners.
2. Rock classification according to Caltrans "Soil & Rock Logging Classification Manual (Field Guide)", August 1996.
3. Rock Quality Designation (RQD), Weathering, Rock Hardness/Strength, Bedding, and Fracture Density, as shown on this sheet, were used to describe all rock core from borings. Descriptors were determined in the field.
4. REC = Percent Core Recovery.
5. RQD = Percent Rock Quality Designation.
6. Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001.

Source: U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual".

ENGINEERING GEOLOGY FIELD DESCRIPTORS

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 2

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. A. Krause & M. W. Heublein (FIELD INVESTIGATOR)
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001

METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

PROJECT ENGINEER

KP

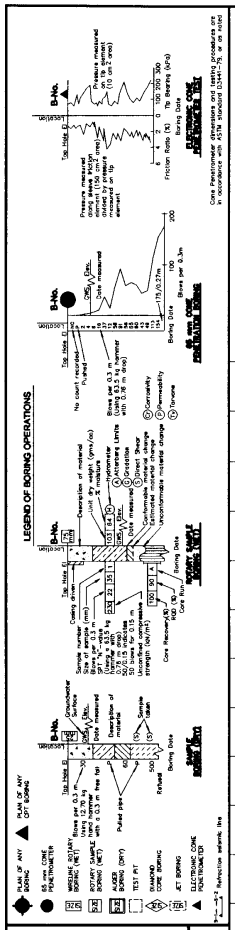
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

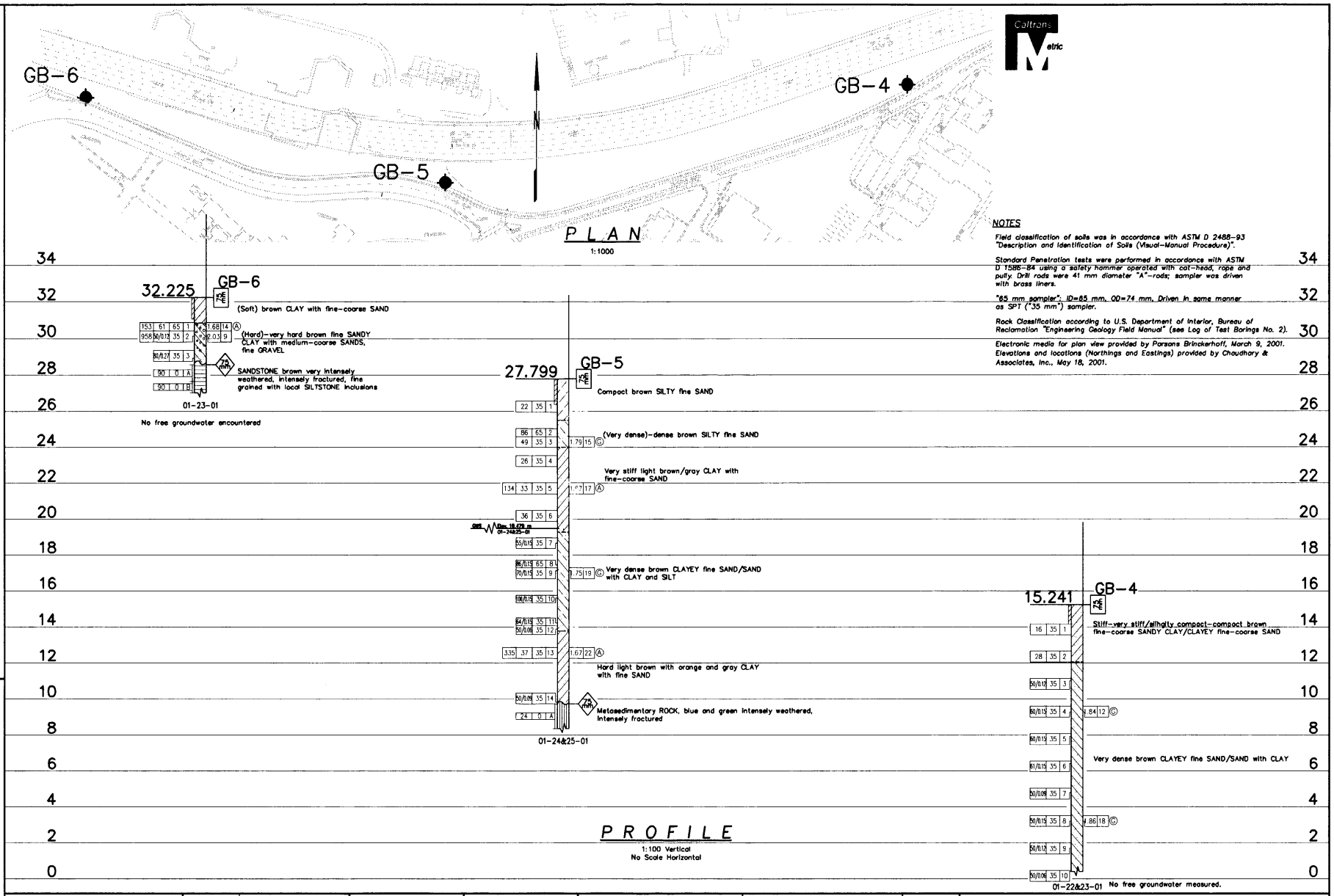
CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET

Taber Consultants Job No. 202/300/133 file Id: 202300133r.dwg date: 03/22/01





NOTES

Field classification of soils was in accordance with ASTM D 2488-93 "Description and Identification of Soils (Visual-Manual Procedure)".

Standard Penetration tests were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pully. Drill rods were 41 mm diameter "A"-rods; sampler was driven with brass liners.

"65 mm sampler": ID=65 mm, OD=74 mm. Driven in same manner as SPT ("35 mm") sampler.

Rock Classification according to U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual" (see Log of Test Borings No. 2).

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Choudhary & Associates, Inc., May 18, 2001.

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	M. W. Heublein FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001		

PROJECT ENGINEER

KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 3

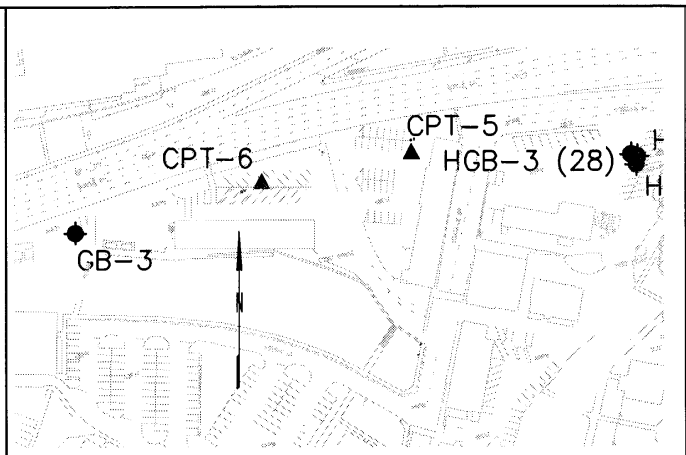
Tabor Consultants Job No. 202/300/133 file id: 202300133a.dwg date: 05/24/01

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

CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET



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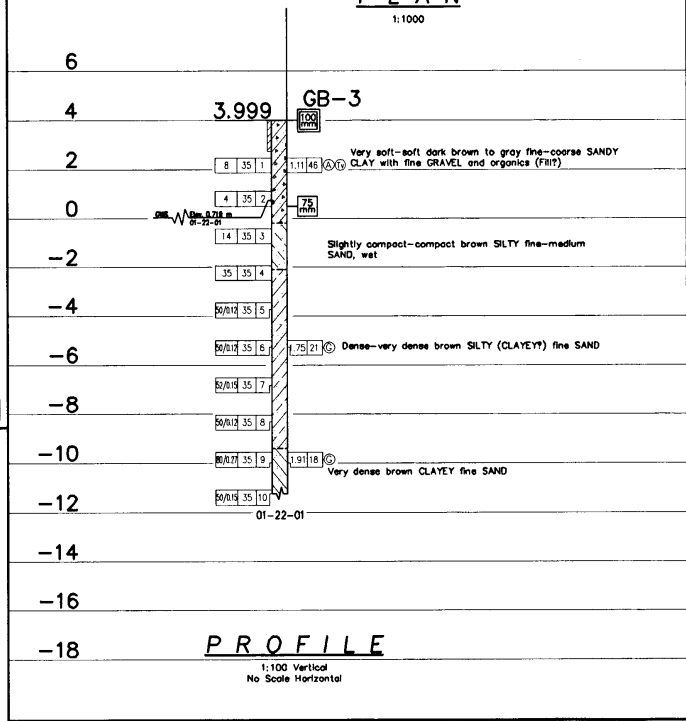

PROFILE
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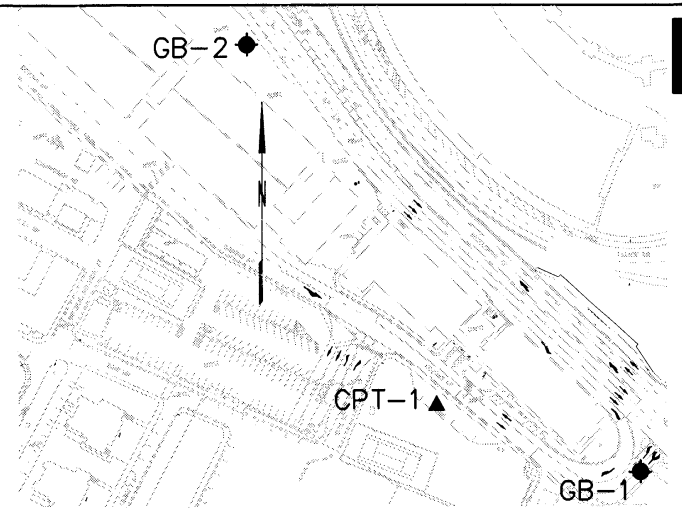
No Scale Horizontal



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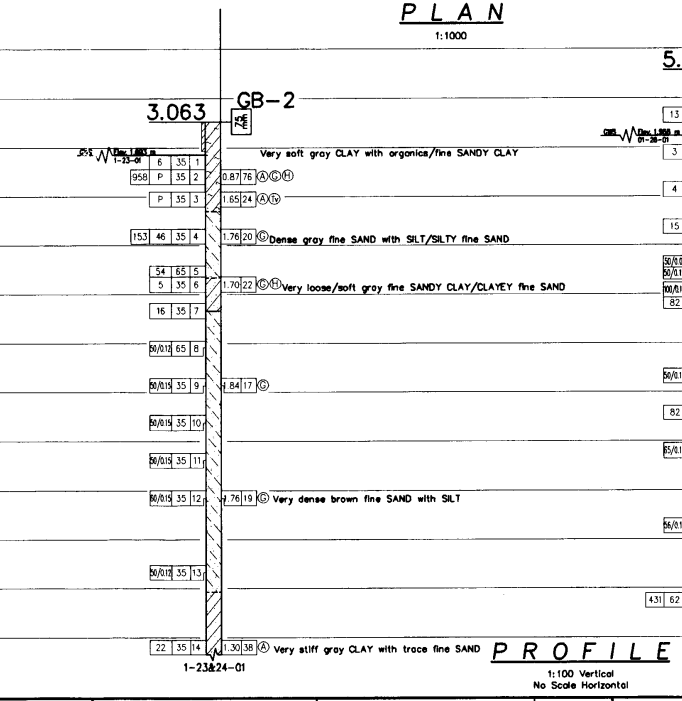

PROFILE
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No Scale Horizontal

NOTES

Field classification of soils was in accordance with ASTM D 2488-93 "Description and Identification of Soils (Visual-Manual Procedure)".

Standard Penetration tests were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pully. Drill rods were 41 mm diameter "A"-rods; sampler was driven with brass liners.

"65 mm sampler": ID=65 mm, OD=74 mm. Driven in same manner as SPT ("35 mm") sampler.

Rock Classification according to U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual" (see Log of Test Borings No. 2).

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

LEGEND OF BORING OPERATIONS

PUSH OF ANY CPT BORING — 63 mm CONE PENETROMETER — ROTARY SAMPLE BORING (WET) — MUDDUMP ROTARY BORING (WET) — AUGER BORING (DRY) — TEST PIT — DIAMOND CORE BORING — JET BORING — ELECTRONIC CONE PENETROMETER — Remotely seismic line

LEGEND OF EARTH MATERIALS

GRAVEL — SAND — SILT — CLAY — ORGANIC CLAY or CLAYEY SAND or SANDY SILT or SILTY SAND — SILTY CLAY — CLAYEY SILT — ORGANIC WATER (DRAINABLE) — FILL MATERIAL — CHERTS — CALCEOUS ROCK — SEDIMENTARY ROCK — METAMORPHIC ROCK

CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

Standard Penetration N'-Value	Granular	Cohesive
0-4	Very loose	Very soft
5-9	Loose	Soft
10-19	Slightly Compact	Stiff
20-34	Compact	Very stiff
35-70	Dense	Very hard
>70	Very dense	

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

DESIGN OVERSIGHT	DRAWN BY M. D. Robertson	M. W. Heublein FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"
SIGN OFF DATE	CHECKED BY K. R. Dahl & M. W. Heublein	DATE January 2001	ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION

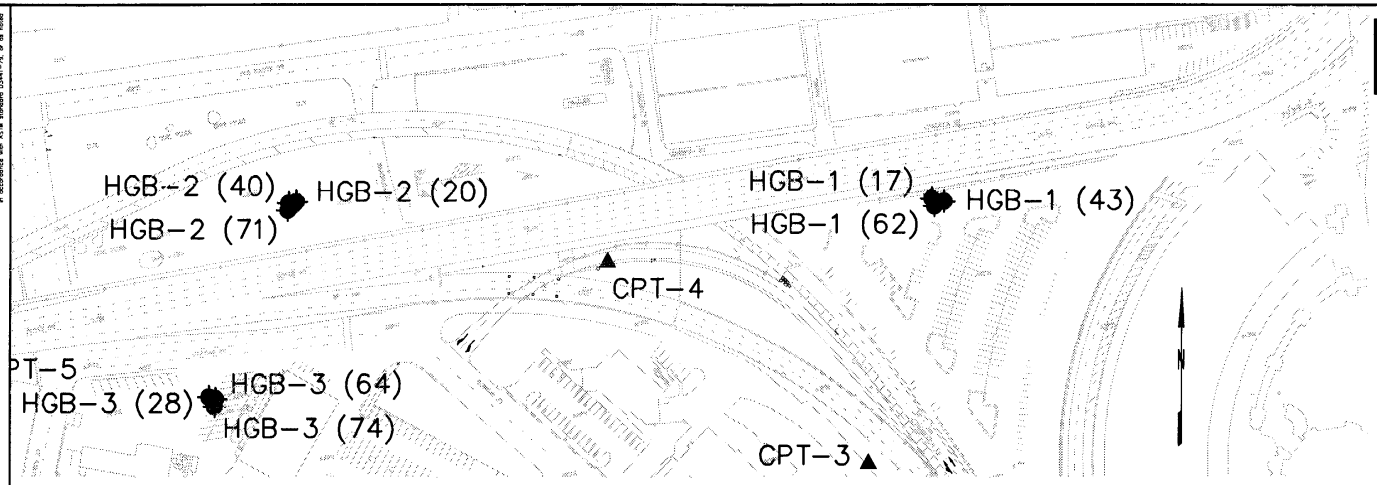
PROJECT ENGINEER — CU EA — KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 4

DISREGARD PRINTS BEARING EARLIER REVISION DATES — REVISION DATES (PRELIMINARY STAGE ONLY) — SHEET

Tober Consultants Job No. 202/300/133 file id: 202300133aa.dwg date: 05/24/01



PLAN

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NOTES

Field classification of soils was in accordance with ASTM D 2488-93 "Description and Identification of Soils (Visual-Manual Procedure)".

Standard Penetration tests were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pully. Drill rods were 41 mm diameter "A"-rods; sampler was driven with brass liners.

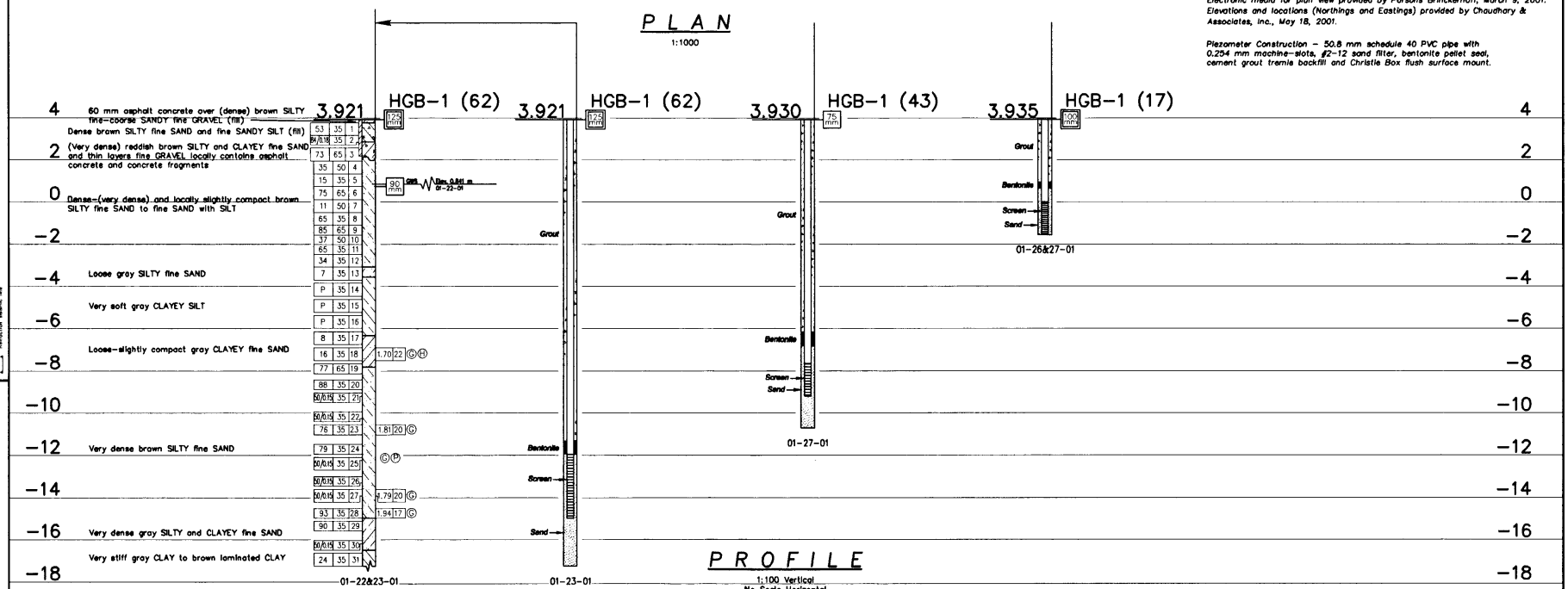
"65 mm sampler": ID=65 mm, OD=74 mm. Driven in same manner as SPT ("35 mm") sampler.

"50 mm sampler": ID=50 mm, OD=65 mm. Driven in same manner as SPT ("35 mm") sampler.

Rock Classification according to U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual" (see Log of Test Borings No. 2).

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

Piezometer Construction - 50.8 mm schedule 40 PVC pipe with 0.254 mm machine-slots, #2-12 sand filter, bentonite pellet seal, cement grout tremie backfill and Christie Box flush surface mount.



PROFILE

1:100 Vertical
No Scale Horizontal

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. A. Krause & M. W. Heublein FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
SIGN OFF DATE	CHECKED BY	K. R. Dohl & M. W. Heublein	DATE January 2001		

PROJECT ENGINEER

KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 5

Tober Consultants Job No. 202/300/133 file id: 20230013 3b1.dwg date: 05/24/01

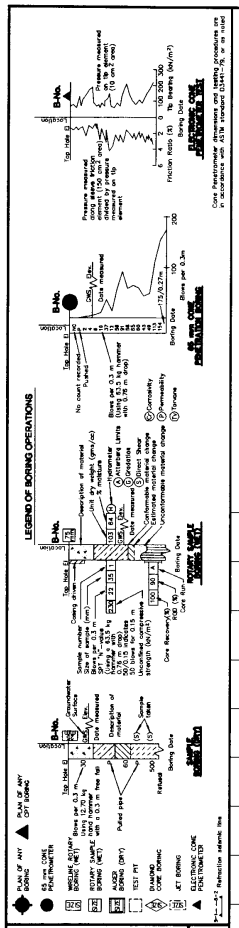
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

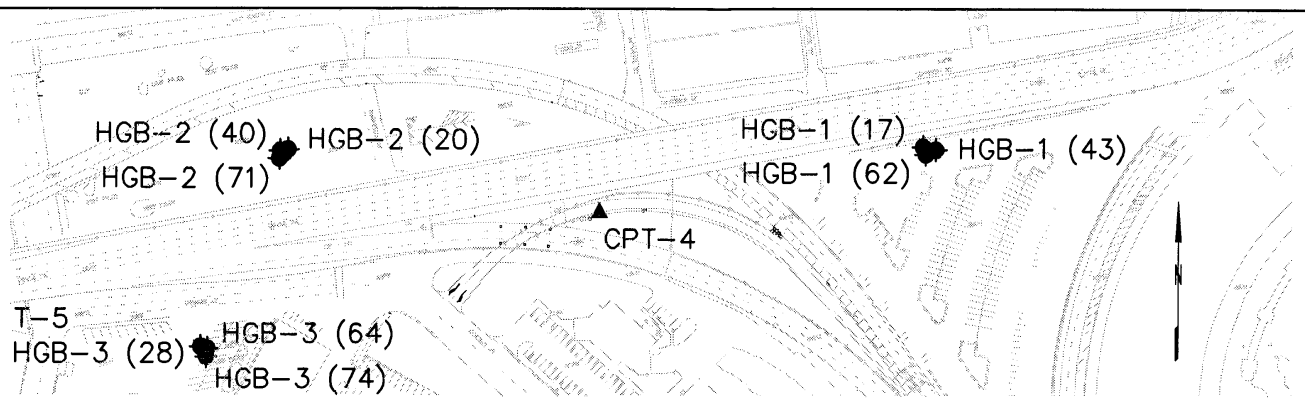
CU
EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET





Caltrans Metric

NOTES

Field classification of soils was in accordance with ASTM D 2488-93 "Description and Identification of Soils (Visual-Manual Procedure)".

Standard Penetration tests were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pully. Drill rods were 41 mm diameter "A"-rods; sampler was driven with brass liners.

"65 mm sampler": ID=65 mm, OD=74 mm. Driven in same manner as SPT ("35 mm") sampler.

Electronic media for plan view provided by The Spink Corporation. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

Piezometer Construction - 50.8 mm schedule 40 PVC pipe with 0.254 mm machine-slots, #2-12 sand filter, bentonite pellet seal, cement grout tremie backfill and Christie Box flush surface mount.

PLAN

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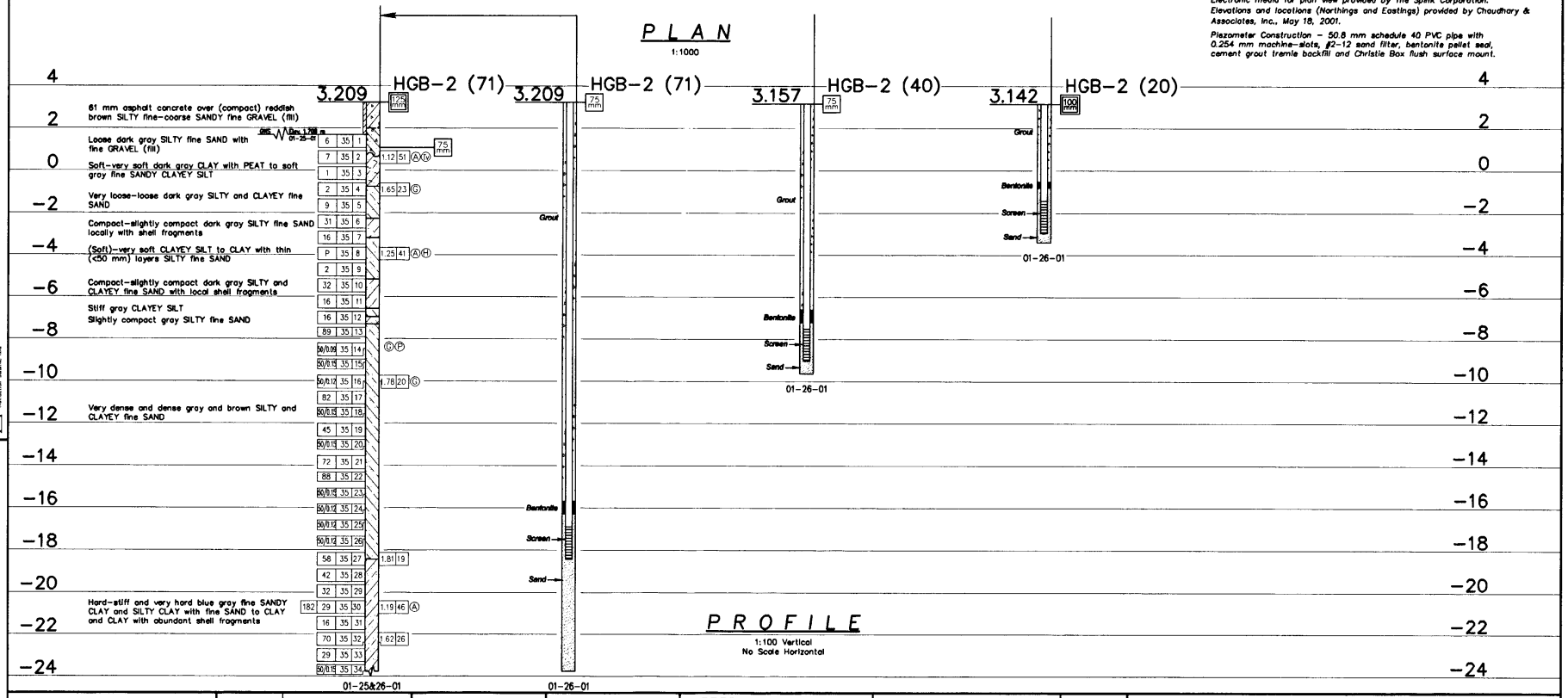


PROFILE

1:100 Vertical
No Scale Horizontal

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. A. Krause FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001			

KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 6

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

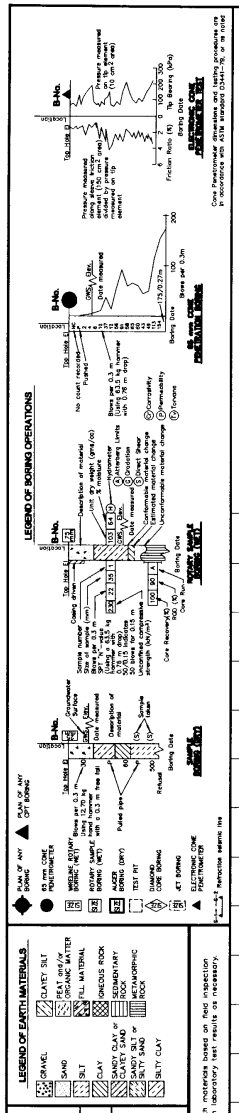
CU
EA

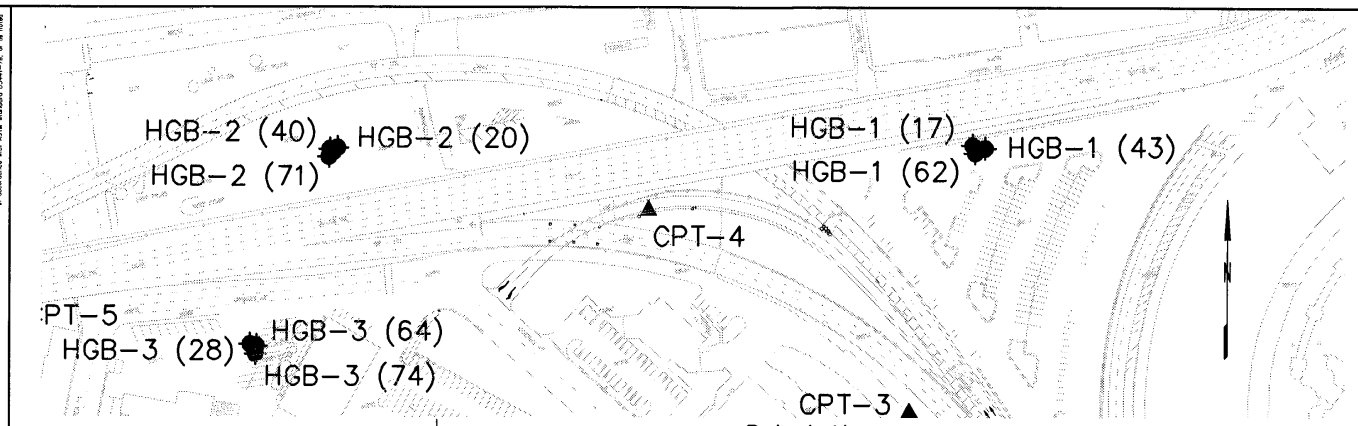
DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET

Tober Consultants Job No. 202/300/133 file id: 20230013 3b2.dwg date: 05/24/01





PLAN

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NOTES

Field classification of soils was in accordance with ASTM D 2488-93 "Description and Identification of Soils (Visual-Manual Procedure)".

Standard Penetration tests were performed in accordance with ASTM D 1586-84 using a safety hammer operated with cat-head, rope and pully. Drill rods were 41 mm diameter "A"-rods; sampler was driven with brass liners.

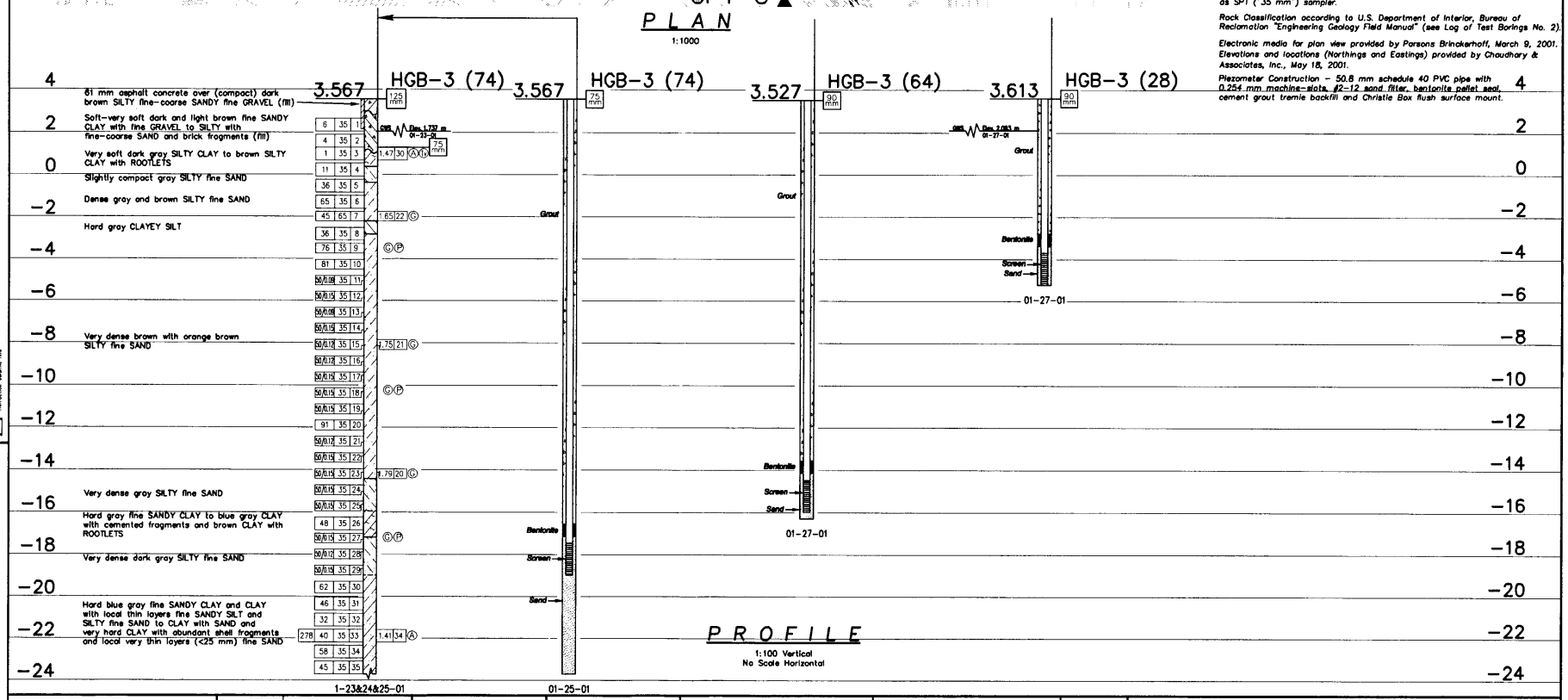
"65 mm sampler": ID=65 mm, OD=74 mm. Driven in same manner as SPT ("35 mm") sampler.

"50 mm sampler": ID=50 mm, OD=65 mm. Driven in same manner as SPT ("35 mm") sampler.

Rock Classification according to U.S. Department of Interior, Bureau of Reclamation "Engineering Geology Field Manual" (see Log of Test Borings No. 2).

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

Piezometer Construction - 50.8 mm schedule 40 PVC pipe with 0.254 mm machine-slots, #2-12 sand filter, bentonite pellet seal, cement grout tremie backfill and Christie Box flush surface mount.



PROFILE

1:100 Vertical
No Scale Horizontal

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. A. Krause & M. W. Heublein FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER	DOYLE DRIVE RECONSTRUCTION PROJECT
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001			KP	LOG OF TEST BORINGS No. 7

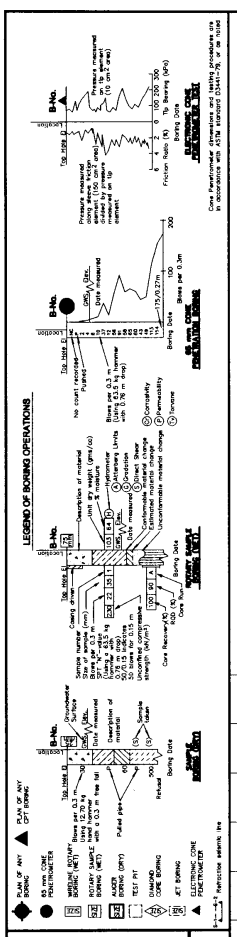
Tober Consultants Job No. 202/300/133 file id: 202300133b3.dwg date: 05/24/01

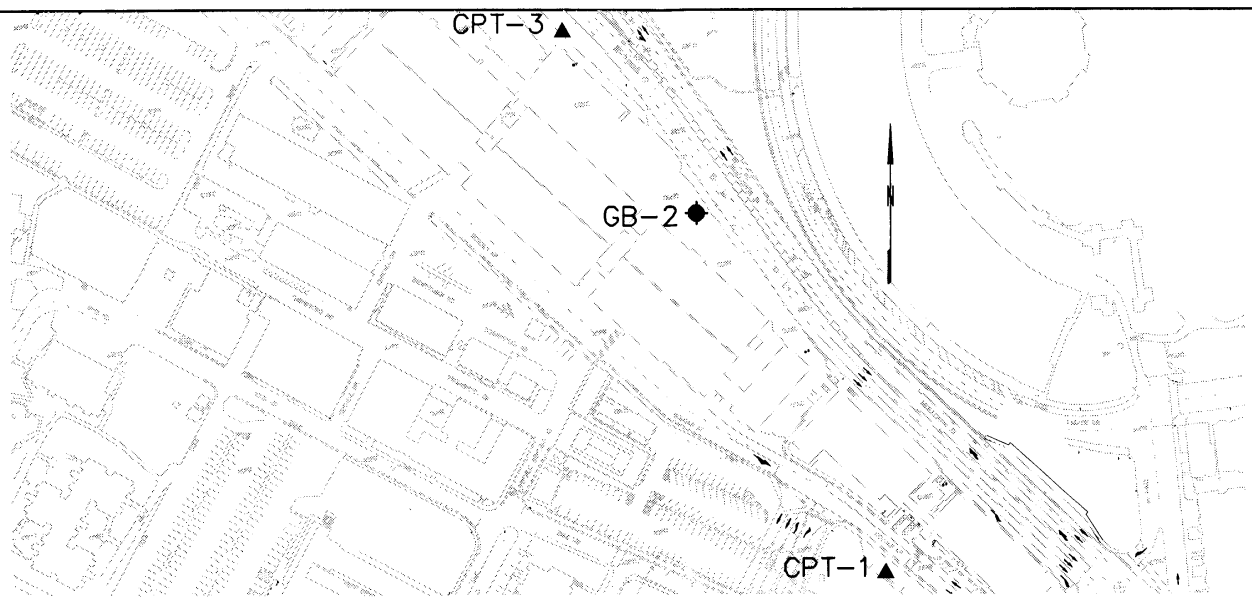
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET





PLAN

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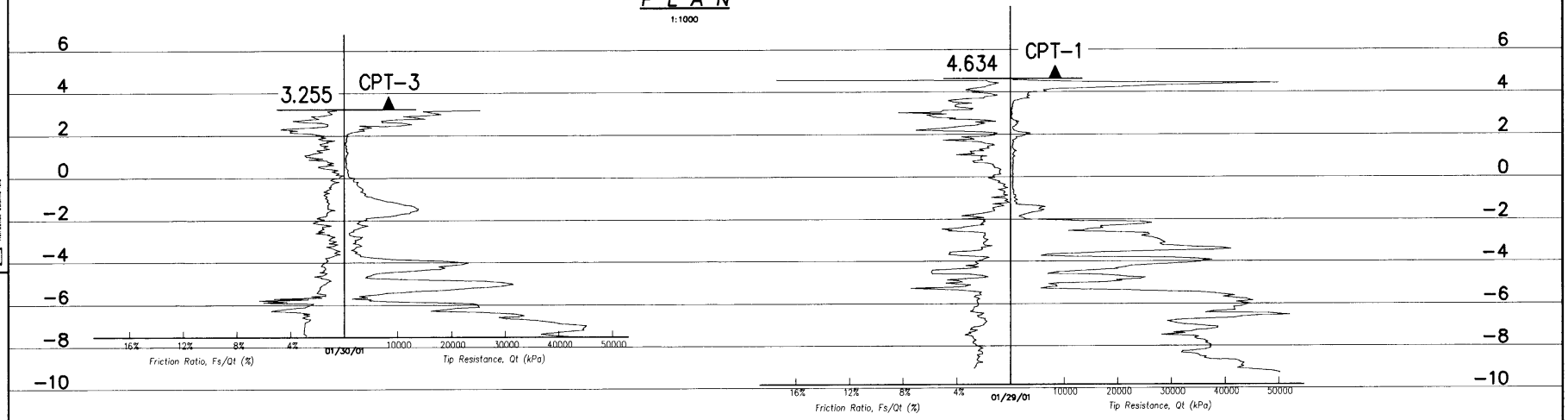
NOTES

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Choudhary & Associates, Inc., May 18, 2001.

CPT data reported for CPT-4 through CPT-8 was digitized from original metric data plot.



PROFILE

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DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. d'Arcy FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER	DOYLE DRIVE RECONSTRUCTION PROJECT
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001			KP	LOG OF TEST BORINGS No. 8

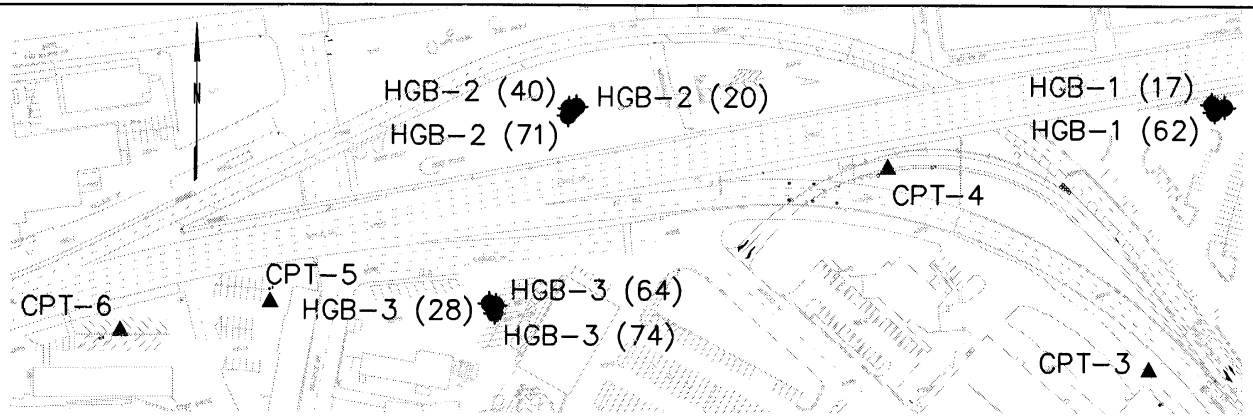
Tober Consultants Job No. 202/300/133 file id: 202300133a.dwg date: 05/24/01

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

CU EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET



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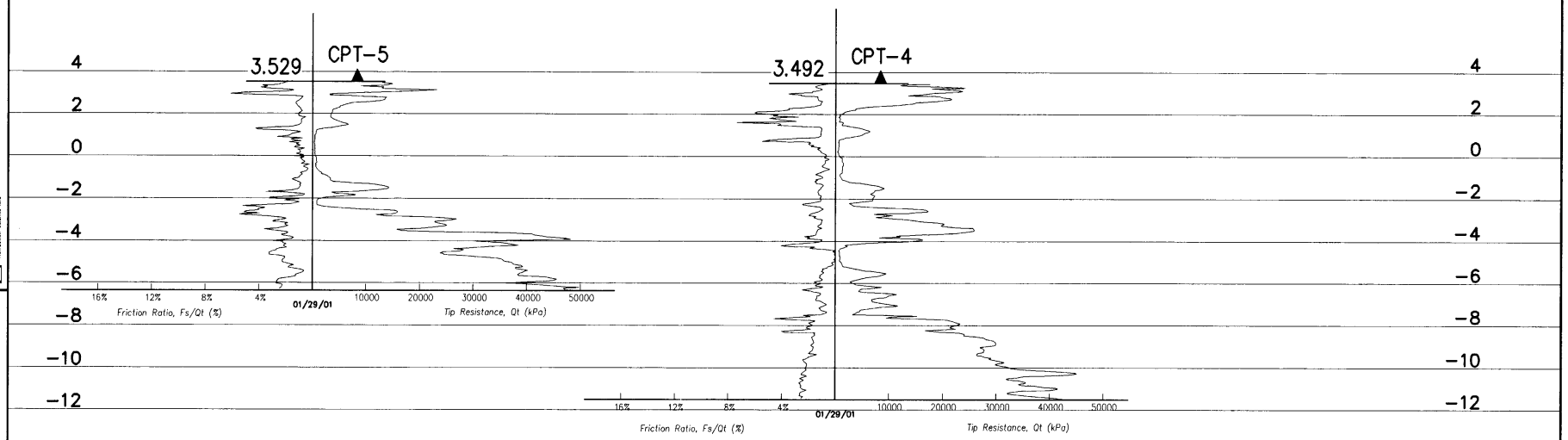
NOTES

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

CPT data reported for CPT-4 through CPT-8 was digitized from original metric data plot.



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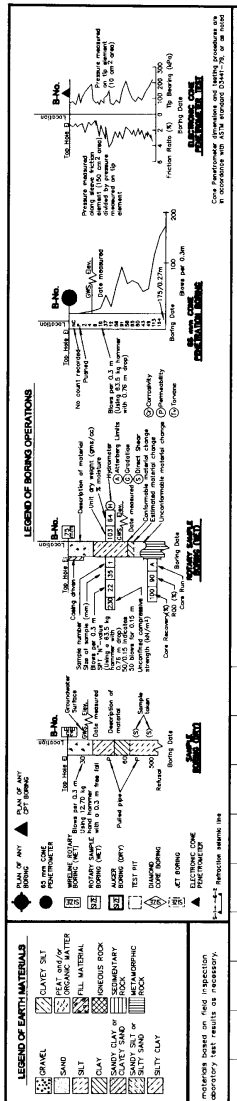


DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. d'Arcy FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001			

DOYLE DRIVE RECONSTRUCTION PROJECT

KP

LOG OF TEST BORINGS No. 9

Tober Consultants Job No. 202/300/133 file Id: 202300133t.dwg date: 05/24/01

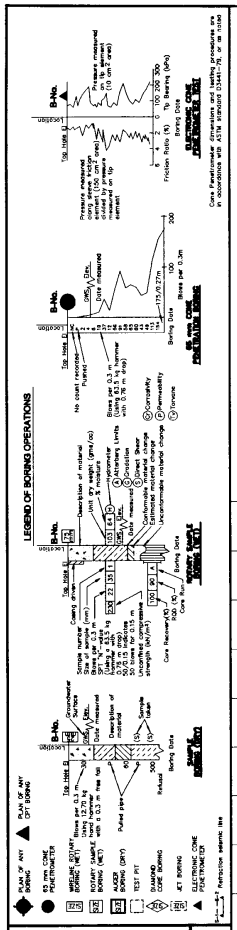
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

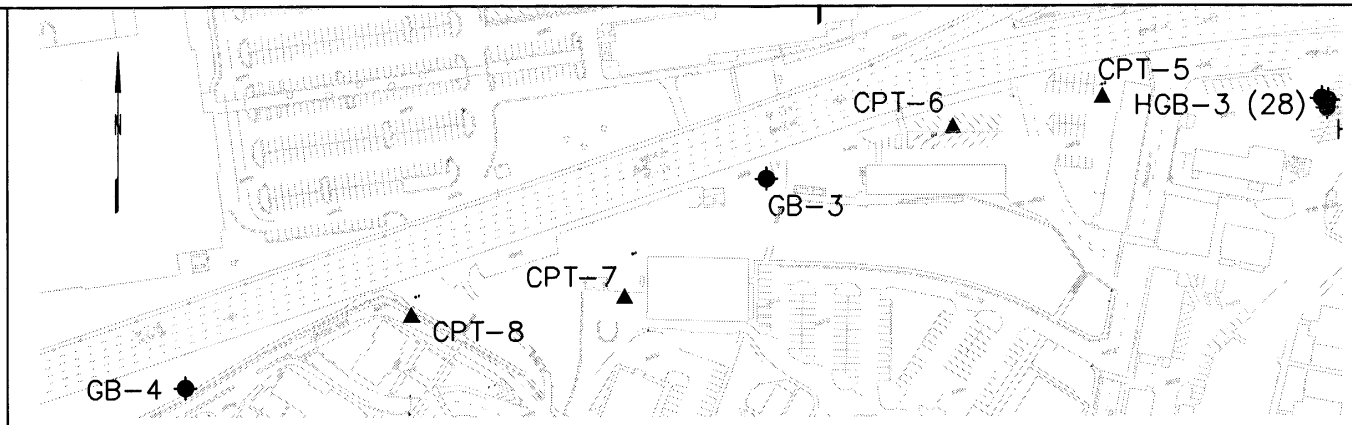
CU
EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET





PLAN

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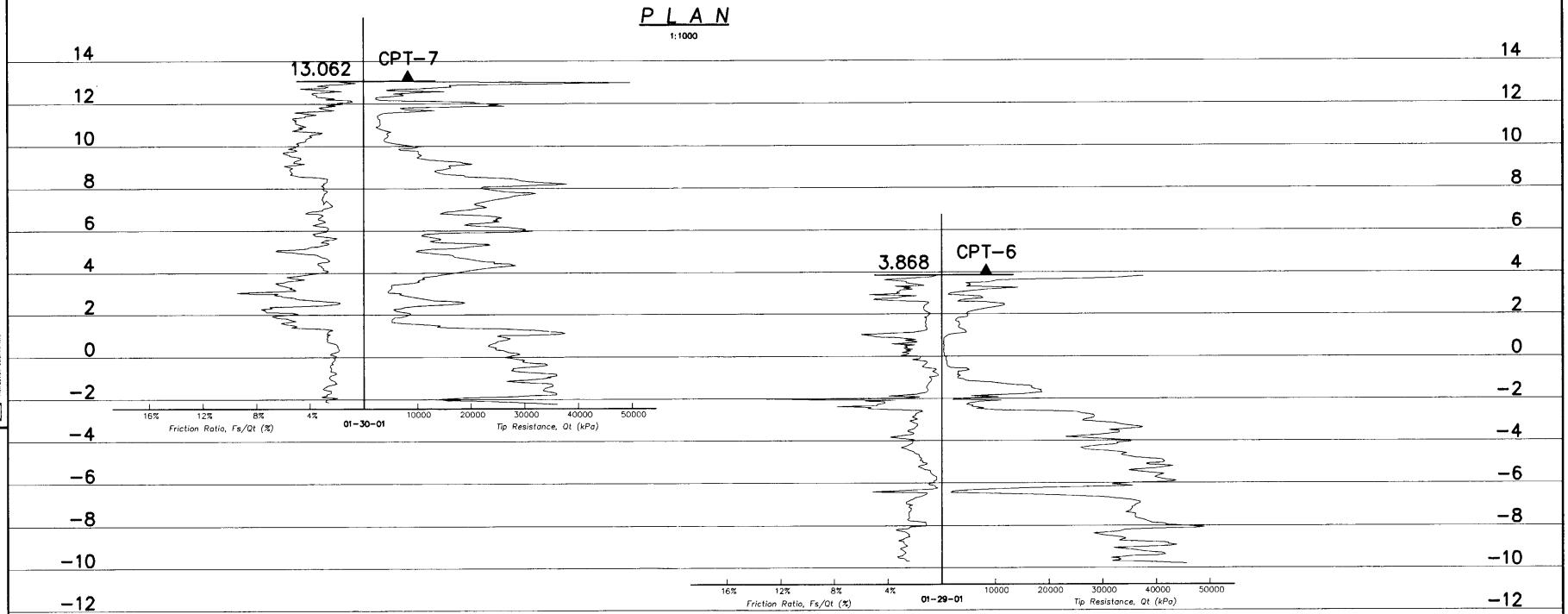
Caltrans Metric

NOTES

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

CPT data reported for CPT-4 through CPT-8 was digitized from original metric data plot.



PROFILE

1:100 Vertical
No Scale Horizontal

DESIGN OVERSIGHT	DRAWN BY	M. D. Robertson	T. d'Arcy FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
SIGN OFF DATE	CHECKED BY	K. R. Dahl & M. W. Heublein	DATE January 2001		

PROJECT ENGINEER

CU
EA

KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 10

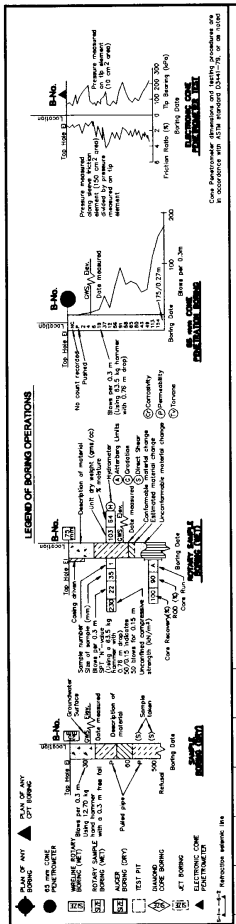
ORIGINAL SCALE IN MILLIMETERS FOR REDUCED PLANS 0 20 40 60 80

DISREGARD PRINTS BEARING EARLIER REVISION DATES

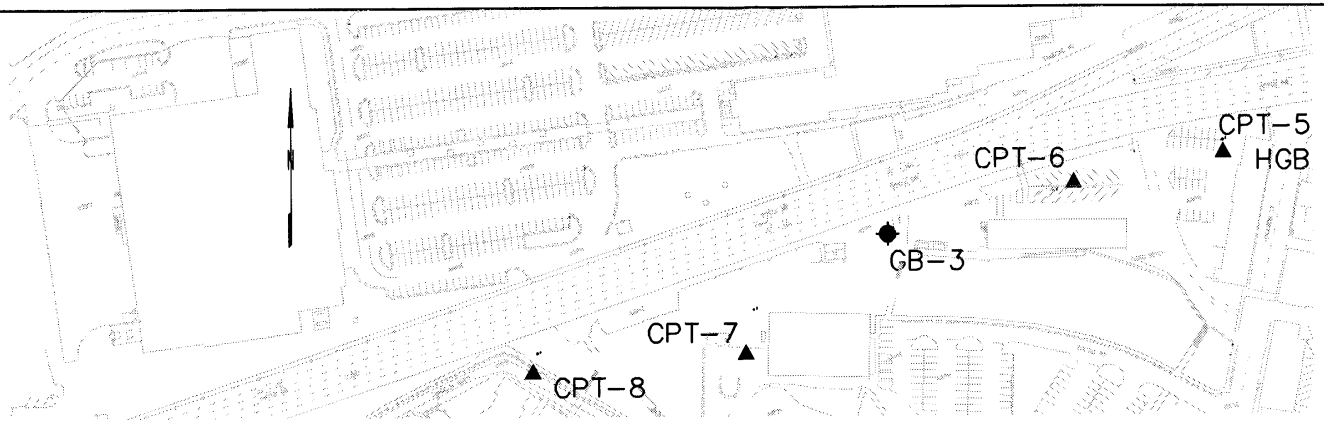
REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET

Taber Consultants Job No. 202/300/133 file Id: 202300133u.dwg date: 05/24/01



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



PLAN

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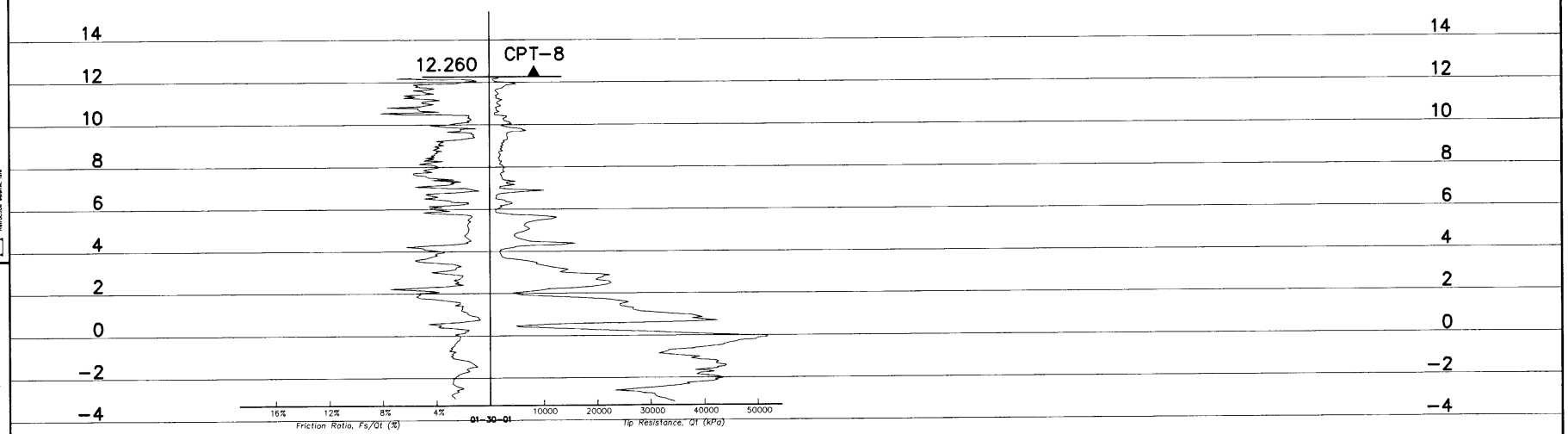
NOTES

Electronic media for plan view provided by Parsons Brinckerhoff, March 9, 2001. Elevations and locations (Northings and Eastings) provided by Chaudhary & Associates, Inc., May 18, 2001.

CPT data reported for CPT-4 through CPT-8 was digitized from original metric data plot.



PROFILE

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No Scale Horizontal

DESIGN OVERSIGHT	DRAWN BY: M. D. Robertson	T. d'Arcy, FIELD INVESTIGATOR	METRIC "ALL DIMENSIONS AND ELEVATIONS ARE IN METRIC (m)"	PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	PROJECT ENGINEER
SIGN OFF DATE	CHECKED BY: K. R. Dahl & M. W. Heublein	DATE: January 2001			KP

DOYLE DRIVE RECONSTRUCTION PROJECT

LOG OF TEST BORINGS No. 11

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

CU
EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) SHEET

Taber Consultants Job No. 202/300/133 file id: 202300133v.dwg date: 05/24/01

APPENDIX C

ALTERNATE INSTALLATION TECHNOLOGIES WITH MINIMAL IMPACT

Overview

Traditional piling installations rely on percussion, vibration, digging operations which are disruptive to the site. New methods of installation have been developed and are mature in the European and Japanese markets. These markets have developed foundation installation systems which can minimize impacts to the environment by reducing:

1. Noise
2. Vibrations
3. Impacts to adjacent structures
4. Space required to install the foundation system

Installation Methods

1. Press-In Method
2. Hydraulic Drilling Method
3. Hydraulic Oscillator Methods

Press-In Method

Hydraulic jacking system which uses fully installed piles as the reaction force for pile installation. All moving, pitching and pressing-in of piles is done from the top of fully installed piles. No reliance on the self-weight and base of the piling machine for stability and reaction of counter-weighting, as required by normal piling operations.

Pile types:

- tubular steel piles
- U-sheet piling
- Z-sheet piling
- Zero-sheet piling
- H-piles
- concrete piles

Piling may be installed in adverse conditions, including:

- Non-Staging, narrow access
- Lmited overhead clearance
- Zero clearance
- Adjacent structures safety
- Difficult subsoil conditions

Developer: Geiken Seisakusho Co., Ltd.

General Information: www.giken-smp.com

American Contact: Giken America Corporation
5802 Hoffner Ave., #707
Orlando, FL 32822
Phone: 407.380.3232
Fax: 407.380.9411
Email: info@gikenamerica.com

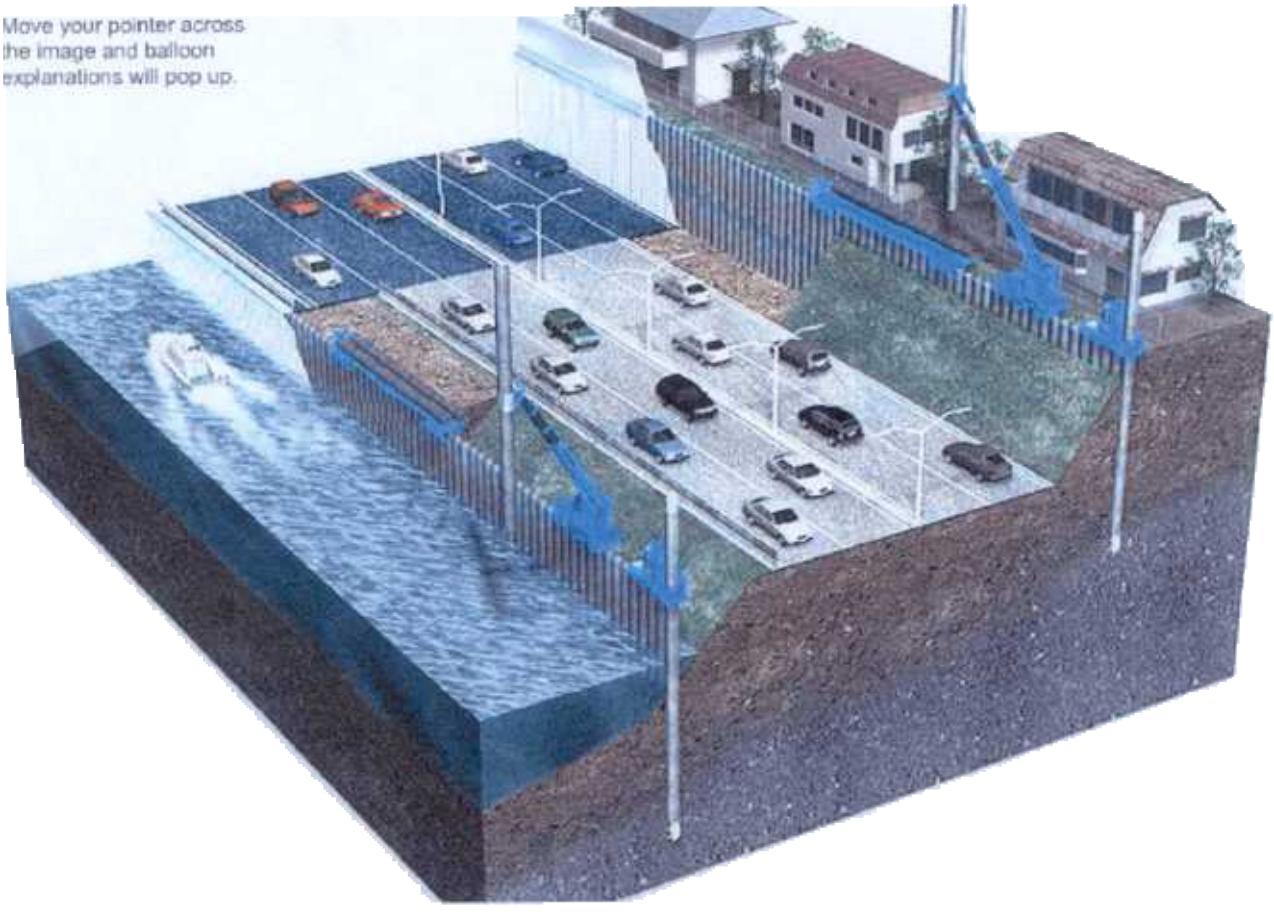
Figures C1, C2, and C3 show applications and example projects of the Press-In Method.

Marvelous Solutions **for Road Widening Construction**

Maintain Existing Traffic !! **Save Overall Cost !!** **Minimize Environmental Impact !!**

Proposals | Advantages | Projects | Operations

Attaining the primary aim of your road widening projects, the integrated GIKEN Wall system provides the best solution out of a range of Silent Piling Technologies considering the type and functions of a structure, site restrictions, soil conditions, quality of work and environmental impact. The system was developed by state of the art technology in the field of geotechnical and mechanical engineering. And One-step Approach allows the GIKEN Wall to be constructed in the shortest possible period of time at the lowest cost incorporating the environmentally friendly Press-in Method and non-staging GRB System, which is free from temporary works. It has redefined *"Impossible"* in urban construction.

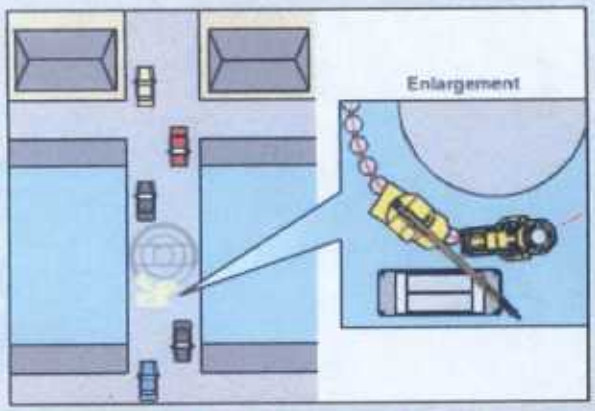


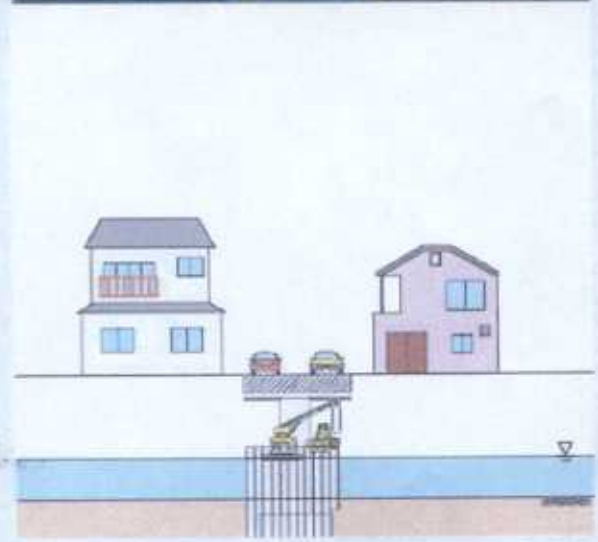
In the above construction situation, is there any possible alternative existing ?

Piling Work without disturbing Overhead facilities

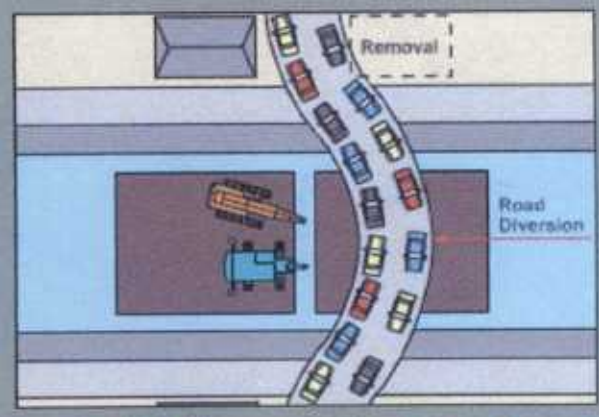
Overhead Clearance Method provides you with the ultimate solution under strict overhead restrictions.

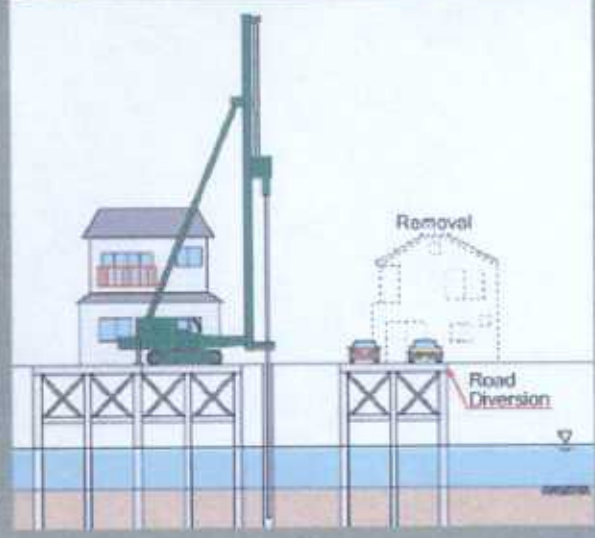
Overhead Clearance Method





Conventional Method





Marvelous Solutions for Road Widening Construction

Maintain Existing Traffic !! Save Overall Cost !! Minimize Environmental Impact !!

These are two samples of achievement of GIKEN Wall system for road widening construction. Both projects have enjoyed the benefits of the system. For further information of the projects, please contact your nearest Giken office.

Hodogaya Bypass Road Widening for Approach to New Junction in Tokyo

Safe Operation with Compact Equipment

Pressing-in Tubular Sheet Piles with simultaneous augering into hard layers (max. SPT value =167)

The **GIKEN WALL** as seen after excavation work.

Noise barrier panels attached to the **GIKEN WALL**

Road Widening of Long Island Expressway in New York

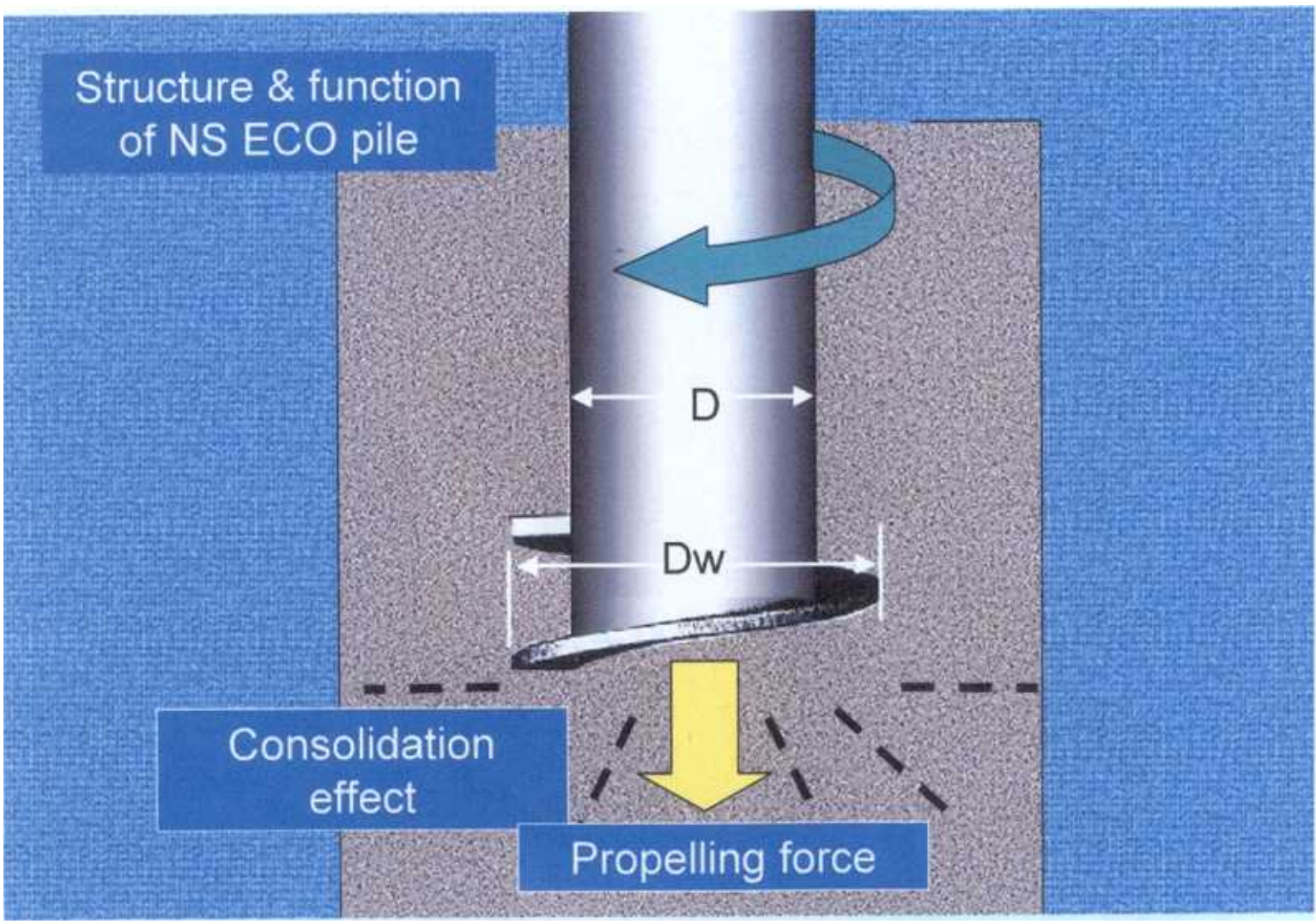
Construction of Road Revetment

Press-in Operation

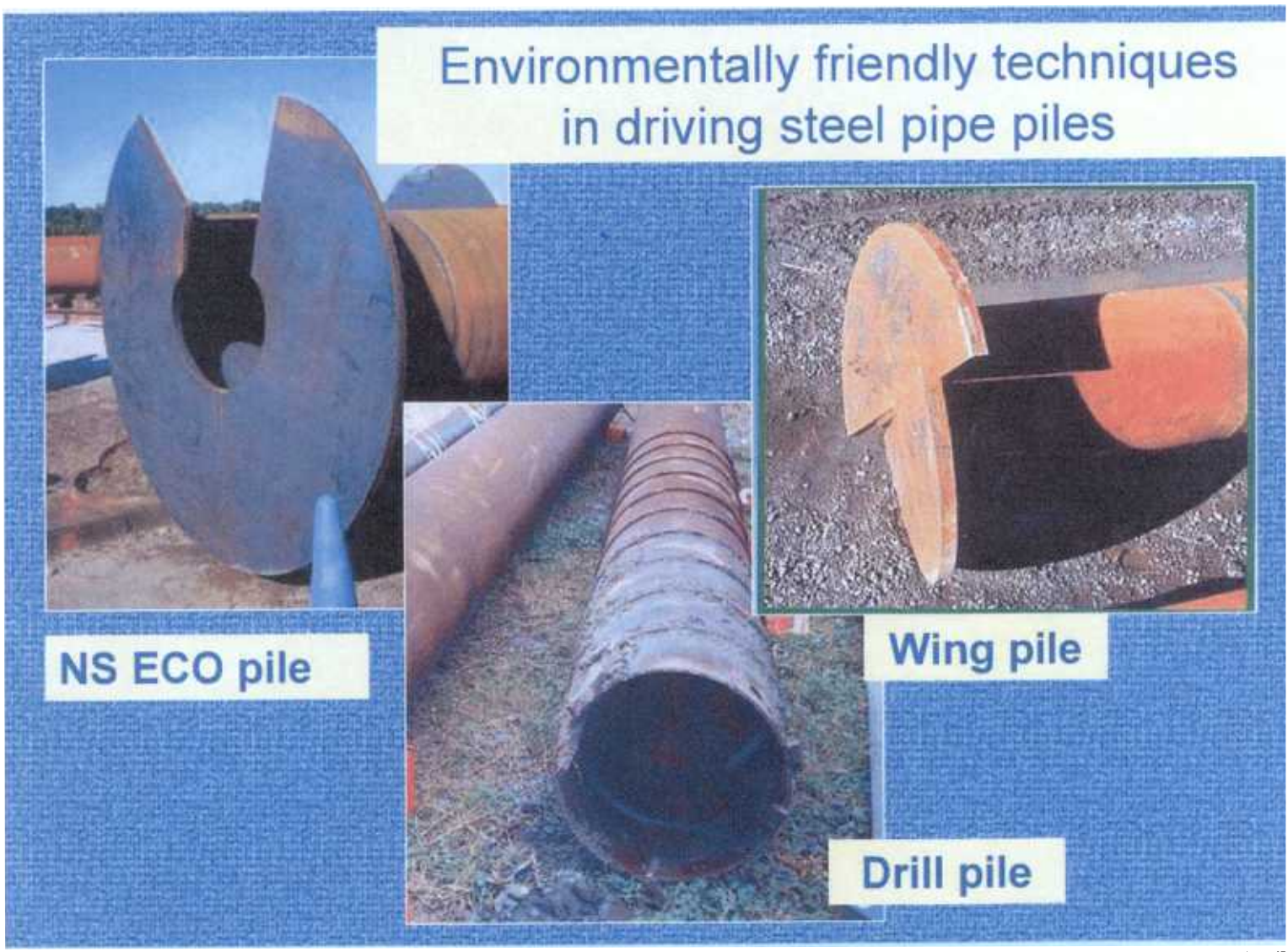
After Excavation

Renovation of Marathon Bridge

Piling Work beside Traffic



When the pile is rotated, a certain propelling force is generated by the spiral wing which is effective to drive the pile down into soil. The pile can be driven down into comparatively soft soil without any pressing force applied. A consolidation effect of the soil can be expected around the pile end.



These 3 techniques which are practically employed as [ENVIRONMENTALLY FRIENDLY TECHNIQUE-no bentnite/cement required]

Hydraulic Oscillator Method

Hydraulic oscillating methods use high torque and large oscillation of casings to install piles and reduce noise and vibration.

Pile types:

- Single and double-walled steel-cased piles

Piling may be installed in adverse conditions, including:

- Difficult subsoil conditions with risk of collapse of the excavation
- Noise restrictive areas

Equipment: Casagrande GCL Casing Oscillator, GCP Heavy Duty Oscillator

General Information: www.casagrandegroup.com

American Contact: International Construction Equipment, Inc.
301 Warehouse Dr.
Matthews, NC 28104
Phones: 888-ICEUSA1 and 704.821.8200
Faxes: 704.821.8201 and 704.821.2781
Website: www.iceusa.com
Email: info@iceusa.com

Figure C6 shows the Casagrande Casing Oscillator.

Casagrande casing oscillator

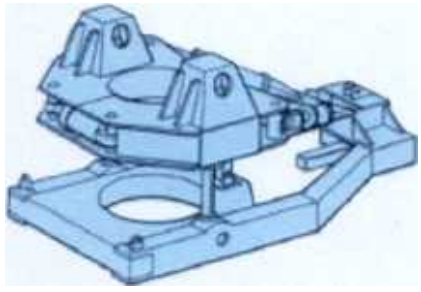
In difficult soils where there is a risk of collapse of the excavation, the use of a steel casing to guarantee the stability of the wall of the borehole is a technique commonly employed by many contractors.

Casagrande produces a complete range of machines, equipment and tools necessary for this drilling technique.

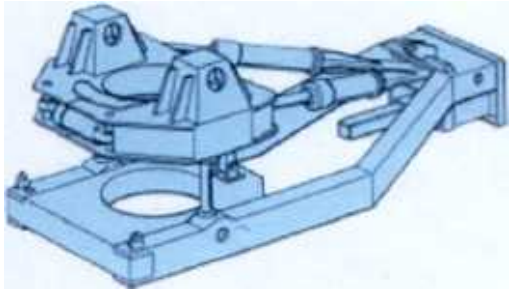
- hydraulic casing oscillators to drive and extract casings
- power packs, when necessary, to drive the casing oscillators
- single and double wall casing elements, fitted with special quick coupling joints

Very high torque and large oscillation angle make the insertion very easy. Different options are available such as:

- telescopic attachment
- vertically control device
- hydraulically adjustable casing positioning cylinder
- lower casing locking device
- reduction sets



The **GCL Casing oscillator** is used in combination with hydraulic rotary rigs.



The **GCP Heavy duty oscillator** is used in combination with a grab

Figures C7 and C8 demonstrate bridge foundations which can be created from tubular sheet piles. The examples shown are from bridge structures in Japan. This type of foundation could utilize the alternate foundation installation methods described in this document.

THE BRIDGE AND FOUNDATION ENGINEERING

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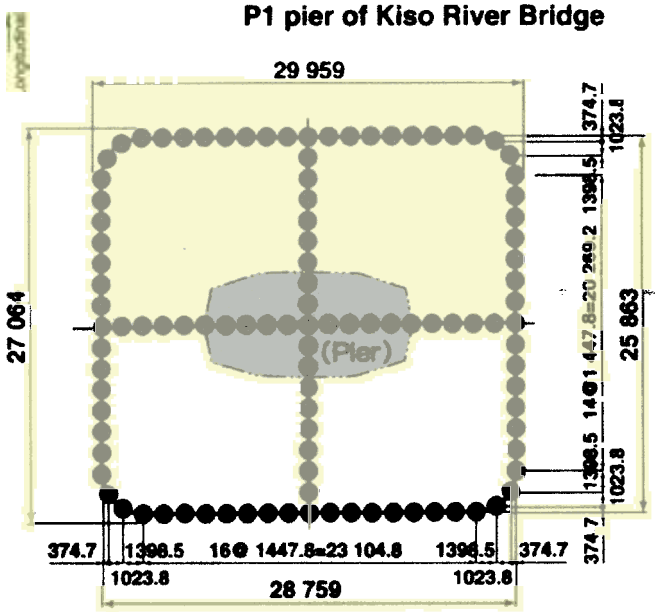
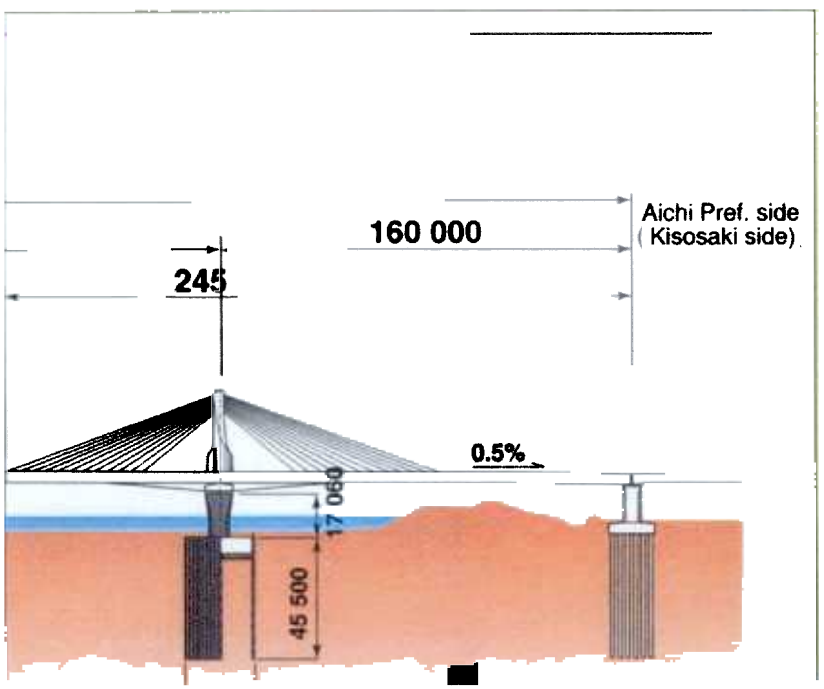
施工　東亜港湾工業株式会社

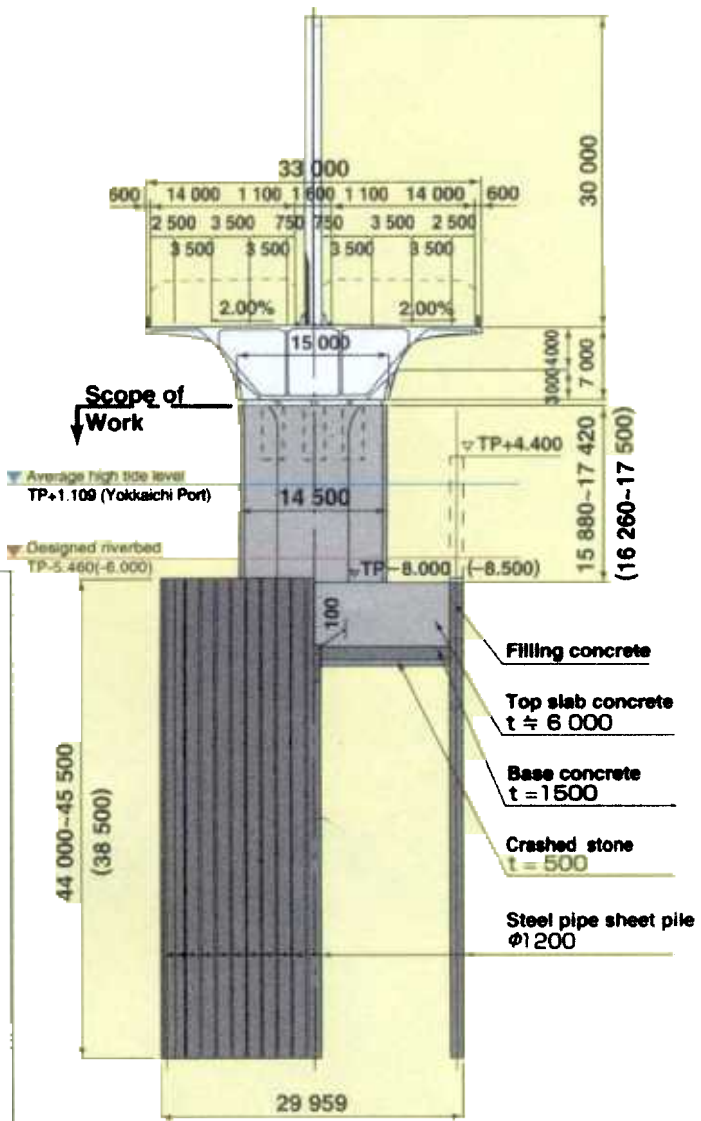
Layout of Steel Pipe Sheet Piles

P1 pier of Kiso River Bridge





Section View of Pier



* Values in ( ) are for the Ibi River Bridge.
Unit : mm unless otherwise specified.

Hydraulic Drilling Method

Hydraulic drilling system which uses large torque to push steel pipe piles into the ground. Pipe piles generally have spiral wing at end to facilitate torque push into ground. Pile is rotated and propelled downward by spiral wing at tip. Consolidation occurs at pile end.

Proprietary Pile types, Makers, and Sizes:

- NS ECO Pile – Nippon Steel – 200 - 1200 mm diameter
- Kawatetsu Drill Pile – Kawasaki Steel - 300 – 600 mm diameter
- Wing Pile – Nippon Steel Pipe – 318 to 508 mm diameter

Advantages:

- No excavated soil
- Large end bearing capacity – consolidation effect at end of pile
- Short Construction period
- Extraction Capability (can be un-screwed and re-used in another location)

Contact Information:

Nippon Steel: www0.nsc.co.jp/shinnihon_english/

Kawasaki Steel

Figures C4 and C5 show examples of the ECO pile system.