Prepared for: West Yost Associates

Geotechnical Engineering Investigation Report

City of Sausalito Priority 1 Sewer Replacement Project

Sausalito, California

September 2009

DCM | GeoEngineers

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September 8, 2009
File: 18337-001-00

Mr. Patrick Fuss
West Yost Associates
2020 Research Park Drive, #100
Davis, CA 95618

Subject: Geotechnical Engineering Investigation Report
City of Sausalito
Priority 1 Sewer Replacement Project
Sausalito, California

Dear Mr. Fuss:

We are pleased to submit our geotechnical engineering investigation report for the City of Sausalito's Priority 1 Sewer Replacement Project in Sausalito, California.

This report is divided into two sections. Section I (Geotechnical Data) contains all the technical data generated by the geotechnical investigation. Section II (Geotechnical Design Summary) contains our interpretation of the technical data, including specific conclusions and recommendations for design and construction of the project.

We appreciate the opportunity to be of service to the City and West Yost Associates on this project. If you have any questions regarding this report, please contact us.

Respectfully submitted,

DCM | GeoEngineers

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Enclosures

GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
CITY OF SAUSALITO
PRIORITY 1 SEWER REPLACEMENT PROJECT
SAUSALITO, CALIFORNIA

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SECTION I

GEOTECHNICAL DATA

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GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
CITY OF SAUSALITO
PRIORITY 1 SEWER REPLACEMENT PROJECT
SAUSALITO, CALIFORNIA

SECTION I - GEOTECHNICAL DATA

1.0 INTRODUCTION

This geotechnical engineering investigation report is for the City of Sausalito's Priority 1 Sewer Replacement Project. The project is located in the Gate 5 Road and Spinnaker Areas of Sausalito, California. The locations of these project areas are illustrated on Plate I-1. The project consists of replacing existing small diameter (6- to 8-inch) gravity-flow sanitary sewer pipelines with a new 8-inch diameter gravity-flow pipeline and a new 4-inch diameter force main pipeline. A description of segments in each project area is summarized in Tables I-1 and I-2.

<u>Table I-1</u> – Sanitary Sewer Replacement - Gate 5 Road Area (see Plate I-2)

<table>
<tr><th rowspan="2">Segment Location</th><th rowspan="2">Length (ft)</th><th rowspan="2">Invert Depth (ft)</th><th colspan="2">Existing Gravity Pipeline</th><th rowspan="2">New Replacement Pipeline Size and Construction[1]</th></tr>
<tr><th>Material</th><th>Condition</th></tr>
<tr><td>Gate 5 Road & southeast extension</td><td>1,638</td><td>6 to 14</td><td>8″ VCP lined with HDPE</td><td rowspan="3">contains sags</td><td rowspan="3">8″ by open-cut trenching same alignment and depth as existing</td></tr>
<tr><td>Coloma Street</td><td>316</td><td>9 to 13</td><td>VCP lined with HDPE[2]</td></tr>
<tr><td>Harbor Drive</td><td>417</td><td>6 to 9</td><td>8″ VCP and/or ACP</td></tr>
</table>

[1]The material type of the new pipeline is not known to us at this time.
[2]The diameter of the existing VCP is not known to us at this time.

<u>Table I-2</u> – Sanitary Sewer Replacement - Spinnaker Area (see Plate I-3)

<table>
<tr><th rowspan="2">Segment Location</th><th rowspan="2">Length (ft)</th><th rowspan="2">Invert Depth (ft)</th><th colspan="2">Existing Gravity Pipeline</th><th rowspan="2">New Replacement Pipeline Size and Construction[1]</th></tr>
<tr><th>Material</th><th>Condition</th></tr>
<tr><td>Humboldt Avenue</td><td>282</td><td>9 to 12</td><td rowspan="3">6″ Cast Iron</td><td rowspan="3">contains sags and is severely corroded</td><td>8″ by open-cut trenching same alignment and depth as existing</td></tr>
<tr><td>Bay Road and parking lot</td><td>593</td><td>5 to 9</td><td>slip 4″ force main into existing 6″[2]</td></tr>
<tr><td>Bay Road parking lot to Spinnaker Restaurant</td><td>208</td><td>4 to 5</td><td>8″ by open-cut trenching same alignment and depth as existing[3]</td></tr>
</table>

[1]The material type of the new pipeline is not known to us at this time.
[2]Includes a new sanitary sewer pump station at the east end of the force main. We have no plan or profile information pertaining to the pump station at this time.
[3]Includes a new, 6- to 8-foot deep grease interceptor at the east end of the segment for the Spinnaker Restaurant.

The new sanitary sewer replacement pipelines will be routed through existing sanitary sewer manholes. Groundwater is infiltrating into the existing sanitary sewer pipeline through some of these manholes. Leaky manholes will be rehabilitated as part of this project (e.g., possibly by manhole lining, chemical grouting, etc. to prevent groundwater infiltration). As illustrated on Plates I-2 and I-3, the existing gravity sanitary sewer pipelines to be replaced by this project, slope to City pump stations.

This report contains a description of geotechnical conditions along the alignment of planned new sanitary sewer replacement pipelines in the project areas described in Table I-1. All descriptions provided in this report pertaining to existing and new sanitary sewer replacement pipelines and related project structures (e.g., location, depth, size, length, material type, condition, and construction methods, etc.) are based on in-progress design plans by West Yost Associates dated August 21, 2009 (West Yost Associates, 2009).

2.0 GEOTECHNICAL DATA

Geotechnical data for design and construction of the project is provided in Section I.2. An interpreted summary of the geotechnical data is provided in Section II.2.

2.1 Test Borings

Seven (7) project test borings (Borings B-1, B-2, B-3A, B-3B, B-4, B-5 and B-6) and nine (9) reference test borings (Reference Borings RB-1 through RB-9) were completed in the Gate 5 Road and Spinnaker Areas of the project. Selected data from these test borings is provided in Table I-3.

<u>Table I-3</u> – **Selected Data from Test Borings**

Test Borings[a]									Project Pipe[b]	
Boring No.	Date (m-d-y)	Top El. (ft)[b]	Total Depth (ft)	Projected Station[b] (ft)	Ground water Depth (ft)[c]	Fill Depth (ft)	Pavement Section		Invert El. (ft.)	Invert Depth (ft)
							AC (in)	AB (in)		
Gate 5 Road Area										
B-1	7-6-09	10	19	15+95	6½	>19	10[d]	20[d]	2	8
RB-1	1-6-77	7	32	-	2	18	-	-	-	-
RB-2	1-5-77	8½	22	-	2	15	-	-	-	-
B-2	7-6-09	8	18½	21+00	4	>18½	6[d]	24[d]	-5	13
RB-3	9-26-79	6	33	-	-	28	-	-	-	-
RB-4	9-25-79	6	117	-	8	15	-	-	-	-
RB-5	9-25-79	4	17	-	6	13	-	-	-	-
B-3A	7-6-09	7½	20	46+60	7	>20	6[d]	18[d]	1	6½
B-3B	7-9-09	7½	3½	46+60	>3½	>3½	6[d]	16[d]	1	6½
B-4	7-6-09	11	20	42+60	6	>20	4[d]	22[d]	1½	9½
RB-6	1959	4½	133[e]	-	-	13½	-	-	-	-
RB-7	1959	5½	122[e]	-	-	13½	-	-	-	-
RB-8	1-12-84	-	26	-	1½	-	-	-	-	-
Spinnaker Area										
B-5	7-9-09	8	21	20+75	7	11½+	9[d]	5[d]	4	4
B-6	7-9-09	12½	21½	12+65	10½	17	6[d]	11[d]	2½	10
RB-9	2-3-95	7	130[e]	-	2	8	-	-	-	-

[a] See complete logs of project test borings in Appendix B and reference test borings in Appendix D.
[b] Planned new sanitary sewer replacement pipeline projected and rounded from in-progress design plans (West Yost Associates, 2009).
[c] Depth to free groundwater measured in the boring at the time of drilling.
[d] B-1 & B-2 = Gate 5 Rd; B-3A, B-3B & B-4 = Harbor Dr; B-5 = Humboldt Ave; B-6 = Spinnaker Rest. Parking.
[e] The bottom of Bay Mud was encountered at a bgs depth of < 117' (RB-4), 130' (RB-6), 110' (RB-7) and 105' (RB-8).

2.1.1 Project Test Borings

Seven (7) test borings (Borings B-1, B-2, B-3A, B-3B, B-4, B-5 and B-6) were drilled and logged for the project at the locations illustrated on Plates I-2 and I-3. All the project test borings, except Boring B-3B, were drilled off the alignment of the existing sanitary sewer pipeline. Boring B-3B was intentionally drilled into the trench backfill located above the existing sanitary sewer pipeline. The logs of project borings are provided in Appendix B. Project test borings were drilled with a truck-mounted Mobile B-24 drill using 5-inch diameter continuous flight solid-stem augers. Soil and groundwater conditions were logged and representative soil samples were obtained from each boring. Relatively undisturbed soil samples were obtained by pushing a 3.0-inch outside diameter, 2.9-inch inside diameter Shelby Tube Sampler (STS) or by driving a 2.5-inch inside diameter, 3.0-inch outside diameter Modified California Sampler (MCS) containing brass liners, into the bottom of the boring at the depths indicated on the logs. Disturbed soil samples were obtained by driving a 1.4-inch inside diameter, 2.0-inch outside diameter Standard Penetration Test (SPT) sampler (ASTM D1586) or a 2.0-inch inside diameter, 2.5-inch outside diameter Split Spoon Sampler (SSS) into the bottom of the boring.

All STS samplers were pushed into the bottom of the borehole using the weight of the drill rig. A 140-pound hammer falling 30 inches per blow was used to drive all other samplers. The number of blows required to drive samplers the last 12 inches of an 18-inch drive are recorded on the boring logs as penetration resistance (blows/ft). The penetration resistance values (blows/ft) recorded for SPT sampler drives on the boring logs are actual American Society for Testing and Materials (ASTM) D1586 N-values. The penetration resistance values recorded on boring logs for MCS sampler drives are actual field blow counts for the MCS sampler and have not been reduced to approximate SPT N-values. Samples retrieved from the borings were examined for field classification and logging, and sealed to preserve their natural moisture content for laboratory testing. Classification systems used to log soil samples are provided in Appendix A. Descriptions of soils provided on the boring logs are based on observations during drilling and sampling and on the results of laboratory tests.

At the end of drilling each boring, the depth to which groundwater had accumulated in the boring was measured and the boring was backfilled with cement grout. The static equilibrium groundwater level may be higher or lower than the depth of accumulated groundwater measured in the test borings at the time of drilling.

2.1.2 Reference Test Borings

Nine (9) test borings (Reference Borings RB-1 through RB-9) were drilled by others for other nearby past projects at the locations illustrated on Plates I-2 and I-3. The logs of these borings are provided in Appendix D for reference only. The methods used to drill and sample the reference borings are not known to us at this time except as specifically indicated on the reference logs.

2.2 Laboratory Testing

Moisture content, unit weight, Atterberg limits (plasticity index), grain size analysis, unconfined compressive strength, direct shear strength, and soil corrosivity laboratory tests were performed on soil samples retrieved from project test borings to evaluate their physical characteristics and engineering properties. The results of the tests are included on the logs of project borings in Appendix B and/or on laboratory test result plates in Appendix C.

2.3 Geologic Mapping

A geologic map of the project areas, by the U.S. Geological Survey (Blake, 2002), is provided on Plate I-4. This map shows that the ground surface of project areas is made of artificial fill over Bay Mud (Qmf). A soil map by the U.S. Soil Conservation Service (Kashiwagi, 1985) shows the project areas as tidelands or bay areas covered with artificial fill. Artificial fill is a man-made accumulation of various materials including soil (e.g., clay, silt, sand, and gravel) and rock fragments (e.g., cobbles and boulders), organic material (e.g., peat), concrete, asphalt, debris and rubbish (e.g., steel, rubber tires, etc.). Bay Muds are typically very soft, lightweight, organic-rich, highly compressible and weak silty clay estuarine deposits (with occasional sand lenses and stringers) that are corrosive to concrete and steel and which have been accumulating within the limits of the San Francisco Bay (including Richardson Bay) for several thousands of years.

The fill and native soils encountered in test borings for the project (see the logs of project test borings provided in Appendix B and the logs of reference test borings in Appendix D) are consistent with these mapped descriptions. A contour map of the base of Bay Mud by the California Division of Mines and Geology (Goldman, 1969; now known as the California Geological Survey) is provided on Plate I-5. This map is consistent with reference borings that show the base of Bay Mud in the project areas (see Reference Borings RB-6, RB-7 and RB-9) to be on the order of at least 100 feet below ground surface.

2.4 Historic Topographic Maps and Aerial Photos

Historic topographic maps and aerial photos of the project areas are provided on Plates I-6 (Gate 5 Area) and I-7 (Spinnaker Area). These maps and photos document that prior to the 1900s the project areas were below sea level in Richardson Bay. Urban development in the project areas since the early 1900s include (1) artificial infilling to raise the project areas above sea level (Hitchcock, 2008), (2) construction of piers, ferry depots, wharfs, warehouses, railways and spur lines (e.g., see the Northwestern Pacific Railroad shown partially overwater on the 1897 maps on Plates I-6 and I-7) and other structures, and (3) drainageway modifications, including concentration of storm drainage in a culvert beneath Gate 5 Road near Coloma Street.

Infilling of the project areas occurred in 1942 when Bechtel Corporation began developing the Marinship Shipyard. Additionally, approximately 30,000 wooden piles were reportedly driven into the ground to provide foundations for shipyard structures. The locations of these piles, relative to the planned new sanitary sewer replacement pipelines and related project structures, are not known to us at this time. The shipyard was eventually turned over to the U.S. Army Corps of Engineers (USACE) in the late 1940s. The USACE removed many of the shipyard structures and the area has since been redeveloped with marinas, boat yards, commercial/office properties, light industrial warehouses and other structures. The date of construction of the existing sanitary sewer pipeline to be replaced by this project is not known to us at this time but probably coincided with this area-wide redevelopment about 60 years ago.

2.4.1 Gate 5 Road Area

As part of the Marinship Shipyard activities, the area of Gate 5 Road southeast of Harbor Drive was used as a staging and parking area and may have been covered with concrete paving. During demolition of the Marinship Shipyard, the area appears to have been used to store salvageable materials (e.g., steel beams, lumber, etc.). Approximately 4 feet of fill was placed over Gate 5 Road southeast of Harbor Drive for flood protection improvement purposes within the last few years. As part of Marinship Shipyard activities, a portion of Gate 5 Road northwest of Harbor Drive contained a railroad spur (see 1942 photo on Plate I-6). It appears that portions of Gate 5 Road northwest of Harbor Drive were last raised with fill in the 1970s, and that some of these areas still become flooded during periods of rainfall and high tide.

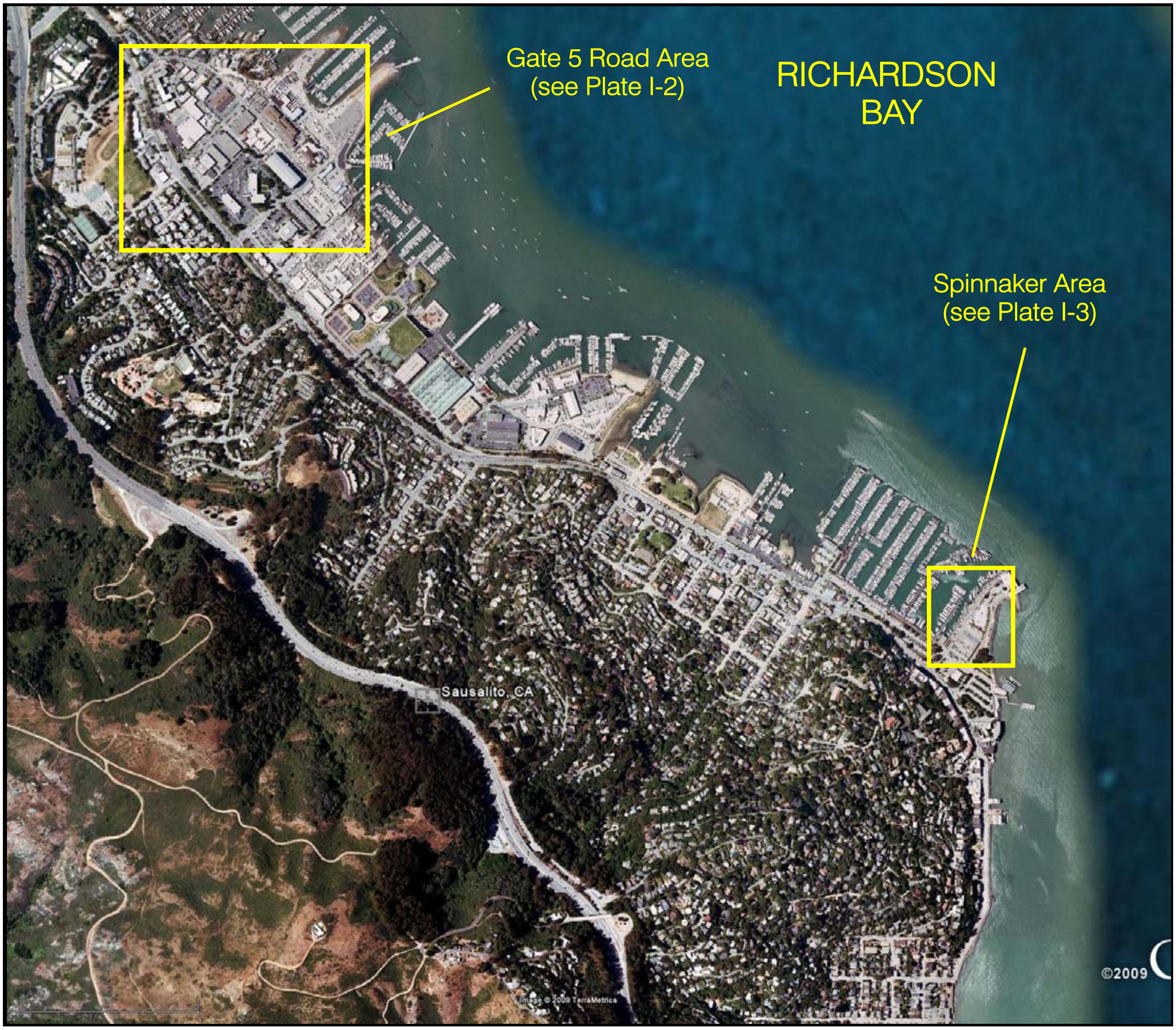
2.4.2 Spinnaker Area

The original structure, now occupied by the Spinnaker Restaurant, was built in the 1960s (the footprint of the original structure is visible at the east end of the Spinnaker Area alignment in the 1970 photo on Plate I-7). An addition onto the north side of the original Spinnaker Restaurant structure was constructed in the 1990s (compare the 1970 photo with the 2009 photo on Plate I-7). A geotechnical evaluation by Geoengineering (1995) for this addition indicates the following:

- The Spinnaker Restaurant overlies pre-1960s fill.
- In order to contain this fill, a steel barge about 100 feet long and 25 feet wide was sunk into Richardson Bay at a location immediately north of the footprint of the original Spinnaker Restaurant.
- The Spinnaker Restaurant addition was constructed over a portion of this sunken barge.
- A yacht harbor is located north of the Spinnaker Restaurant and the sunken barge. This yacht harbor is sustained by a timber bulkhead that was reportedly anchored to the sunken barge by underground cables. The locations of these anchors were not known to Geoengineering (1995) and are not known to us at this time.

The project alignment in the Spinnaker Area is west of the Spinnaker Restaurant. Based on Geoengineering (1995) descriptions, it appears that the east end of the project alignment in the Spinnaker Area is located at least several tens of feet west of the sunken barge. Therefore, there is a low likelihood that the project alignment encroaches across underground cables used to anchor the sunken barge to the yacht harbor's timber bulkheads. It appears that a railroad spur formerly occupied a portion of Humboldt Avenue in the Spinnaker Area (see the 1942 photo on Plate I-7).

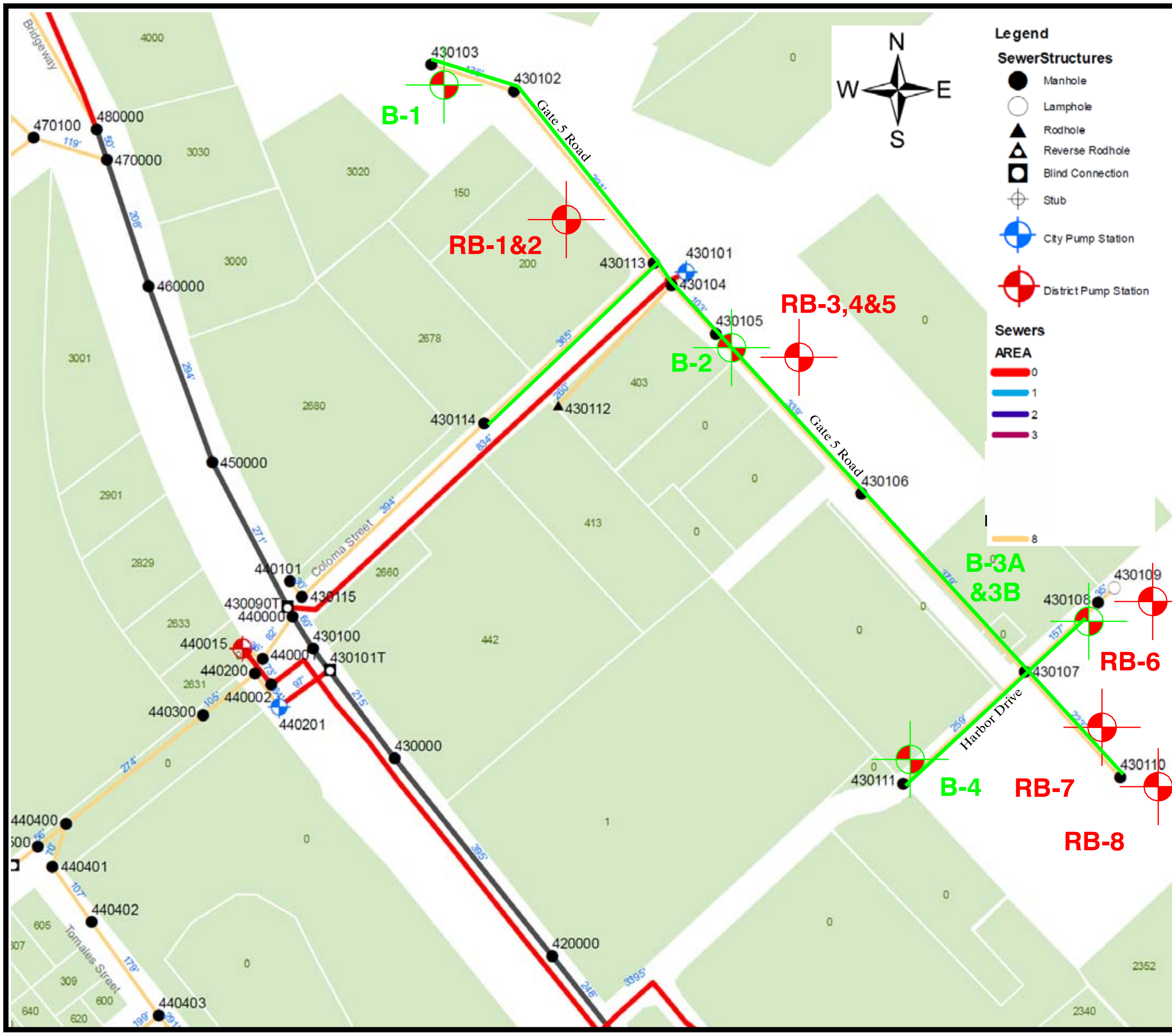
PRIORITY 1 SEWER REPLACEMENT



Base image from Google.

DCM | GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California		PLATE NO. I-1 (COLOR PLATE)
FILE NO. 18337-001-00	AUGUST 2009	PROJECT MAP	

See Plate I-1



Modified from 2007 City drawings and in-progress project design plans (West Yost Associates, 2009).

- Planned New Gravity Sanitary Sewer Pipeline (shown offset from existing pipeline for illustration purposes only)

B-1 Project Test Boring

RB-1 Reference Test Boring

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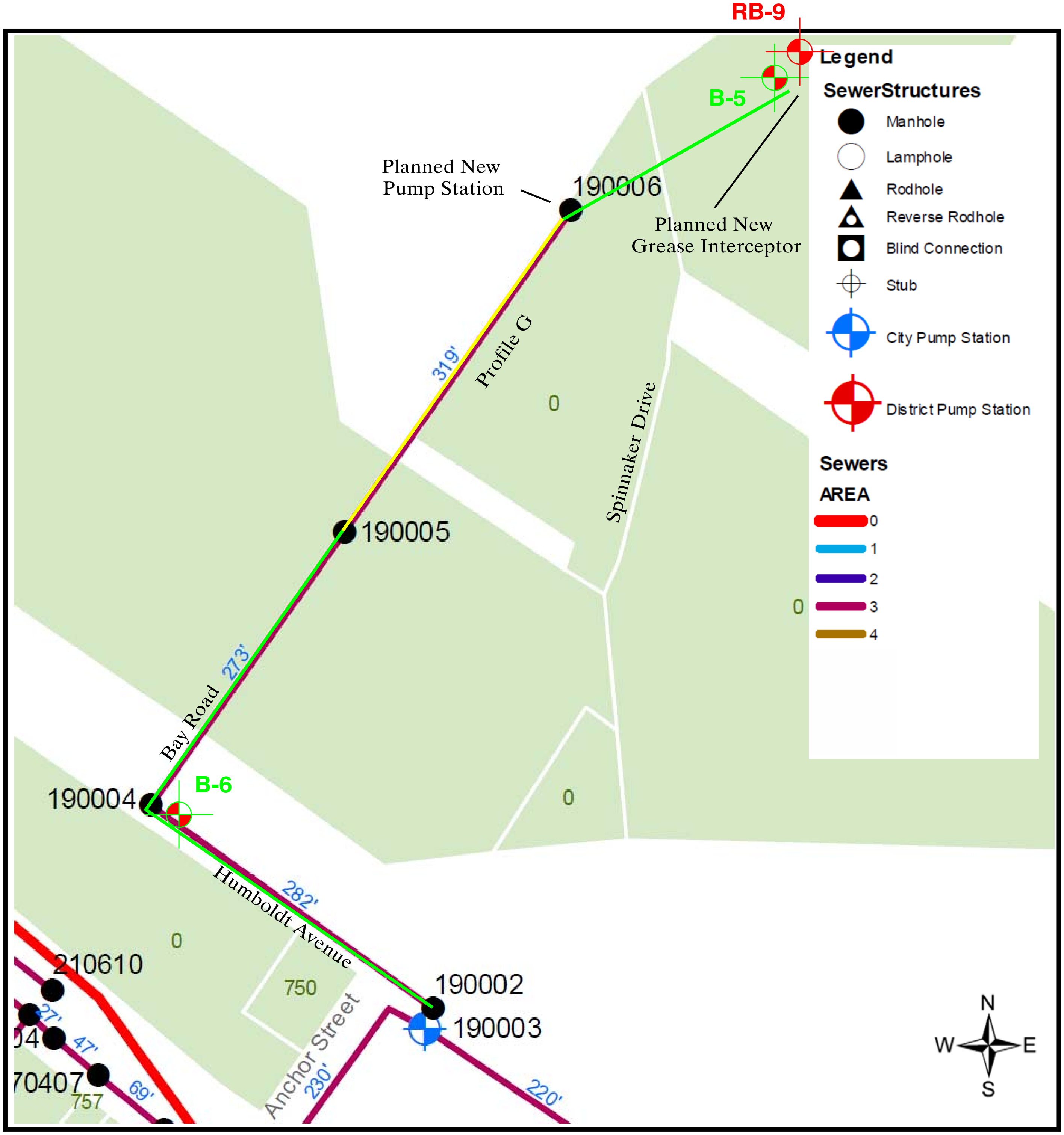
FILE NO. 18337-001-00 | AUGUST 2009

WEST YOST ASSOCIATES
City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California
BORING LOCATION MAP - GATE 5 ROAD AREA

PLATE NO.
I-2
(COLOR PLATE)

See Plate I-1



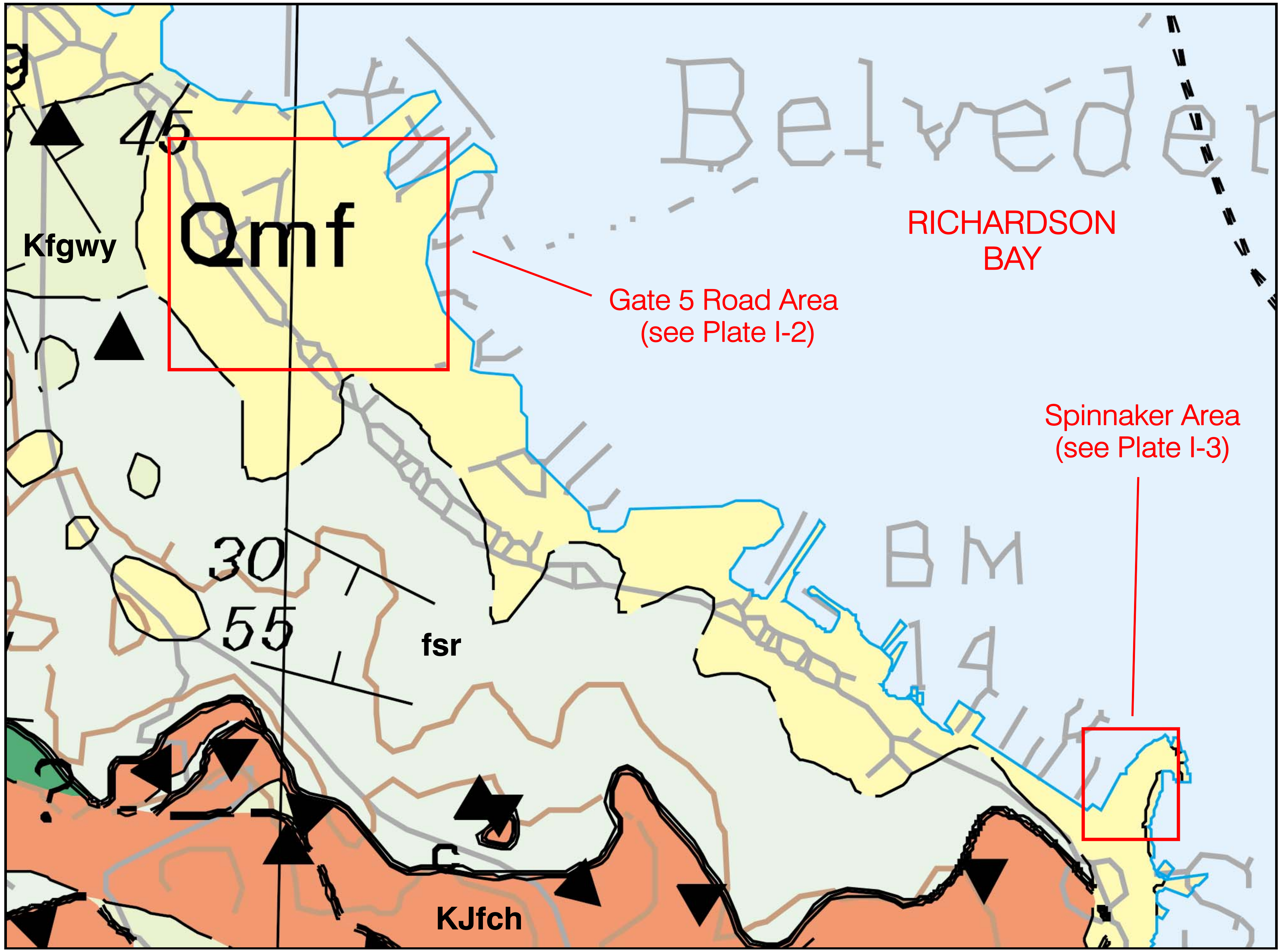
Modified from 2007 City drawings and in-progress project design plans (West Yost Associates, 2009).

gravity : force main - Planned New Sanitary Sewer Pipeline (shown offset from ex. pipeline for iillustration purposes only)

RB-1 Reference Test Boring

B-5 Project Test Boring

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California BORING LOCATION MAP - SPINNAKER AREA	PLATE NO. I-3 (COLOR PLATE)
FILE NO. 18337-001-00 / AUGUST 2009		



Modified from Blake (2002)

TYPICAL DESCRIPTION:

Fill and Native Soils

Qmf — **Artificial fill over marine and marsh deposits (Quaternary) -** Bay Mud overlain by artificial fill of varing character, consisting of clay, silty sand, rock fragments, organic material and man-made debris.

Franciscan Bedrock

Kfgwy — **Graywacke (Cretaceous) -** Moderately hard to hard graywacke sandstone, shale, and some metagraywacke.

fsr — **Melange -** A tectonic mixture of variably sheared shale and sandstone containing hard tectonic inclusions and resistant masses of greenstone, chert, graywacke, and their metamorphised equivalents plus serpentinite and minor discrete masses of limestone too small to be shown on map. Blocks and resistant masses have survived the extensive shearing evident in the melange matrix, and range in abundance from less than 1 to 50 percent or more of the rock mass.

Jfgs — **Greenstone (Jurassic) -** Moderately hard to hard well-bedded lava and minor intrusive diabase. Smaller masses are hard and relatively unfractured, but larger masses are closely fractured or sheared, softened by weathering, and bear distinctive red soil.

KJfch — **Chert (Cretaceous and Jurassic) -** Chert with shale interbeds. Chert is hard, thin bedded, closely fractured, parts along bedding planes, and crops out as irregularly-shaped bodies.

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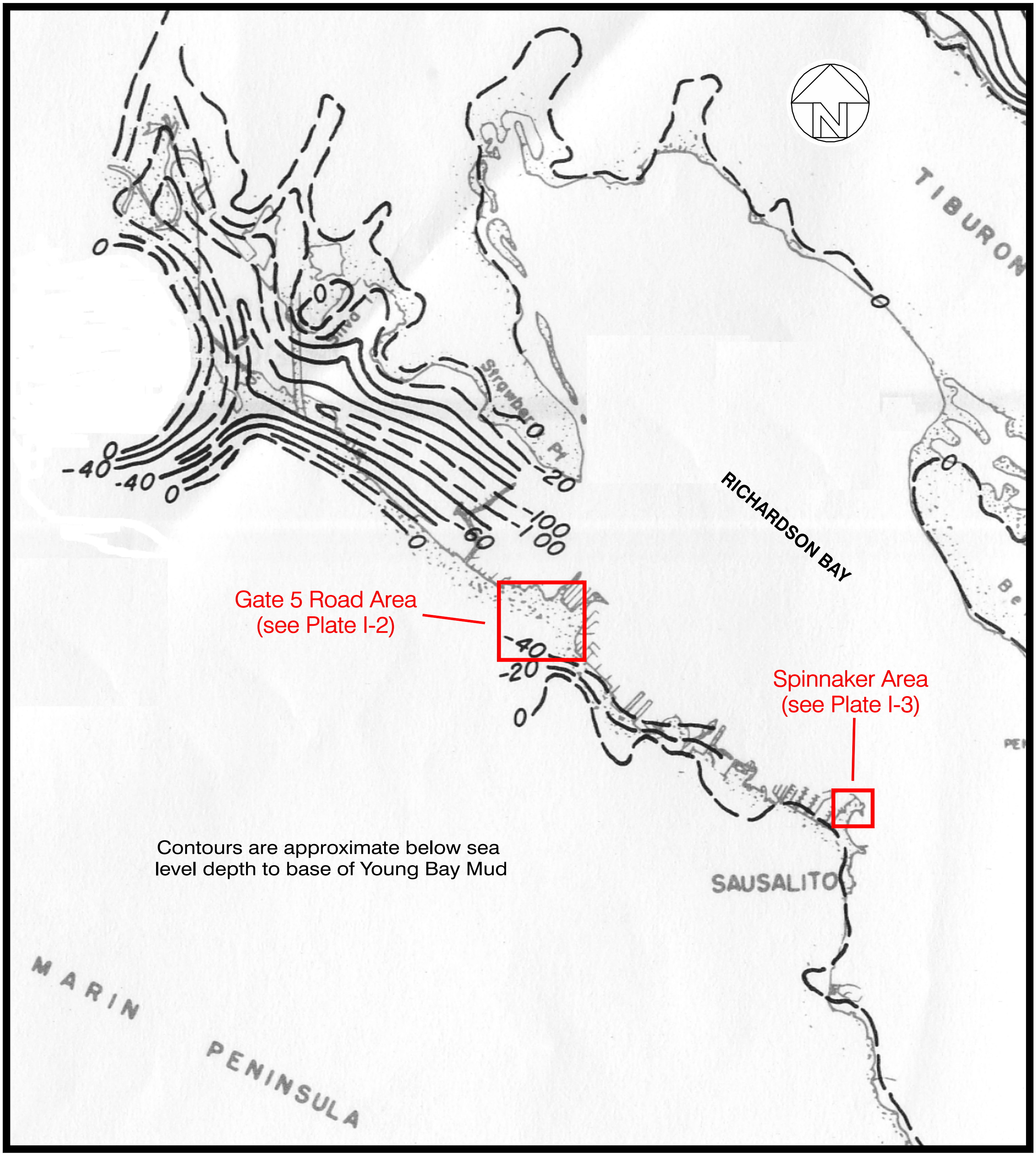
City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

GEOLOGIC MAP

PLATE NO.
I-4
(COLOR PLATE)



Modified from CDMG (1969).

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

BAY MUD MAP

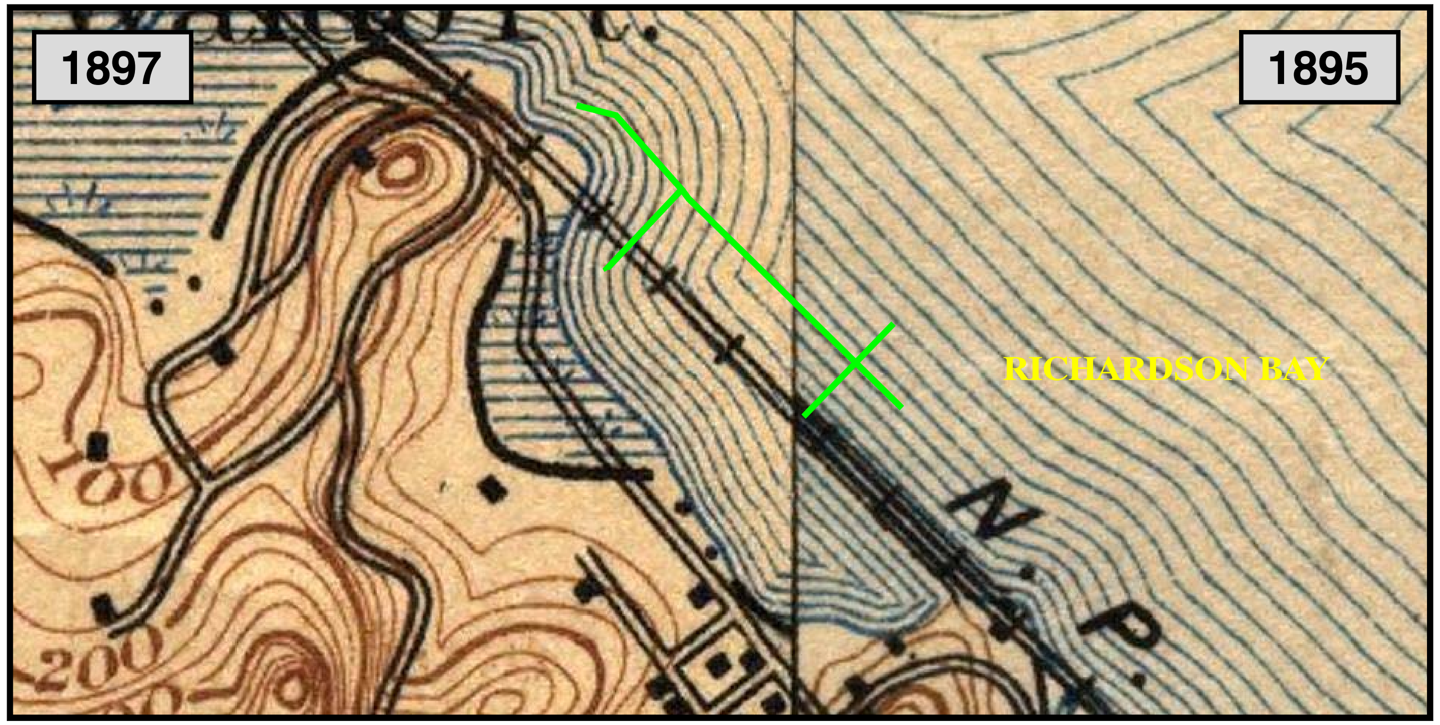
PLATE NO.

I-5

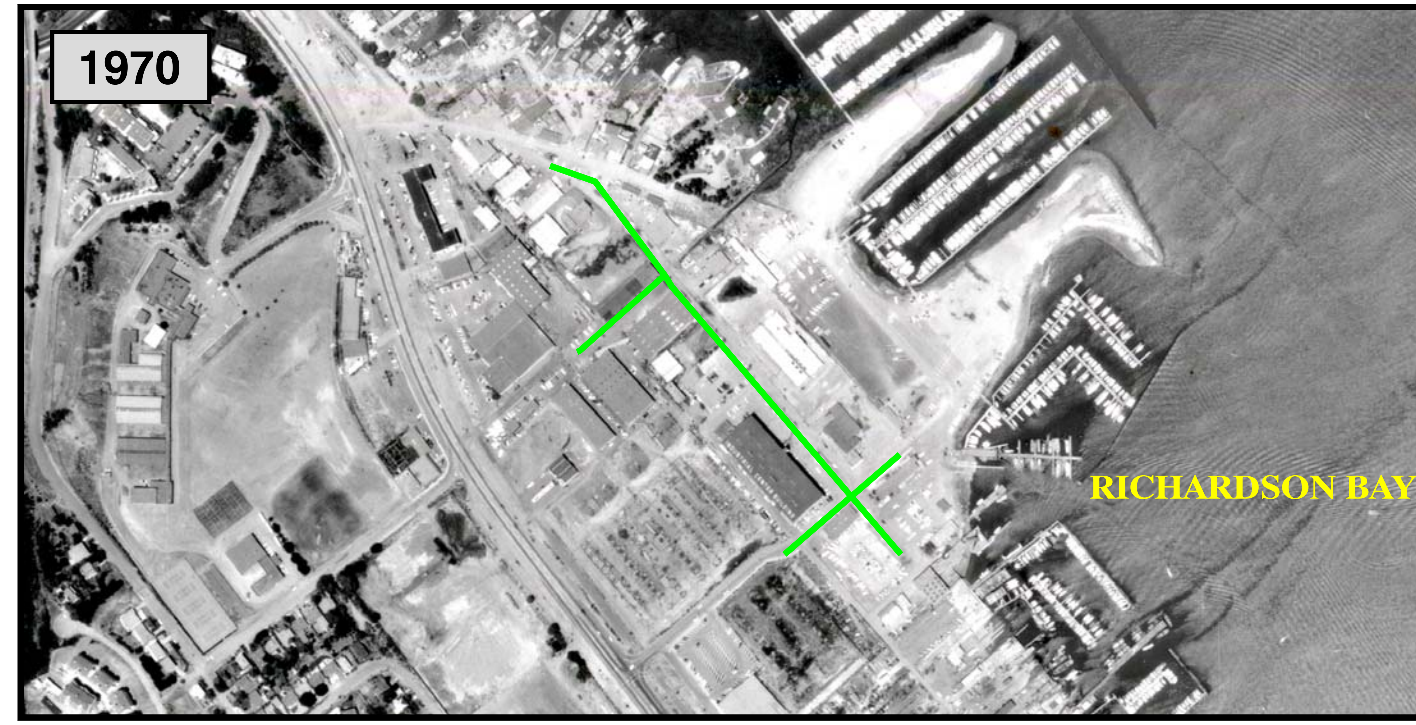
(COLOR PLATE)

FILE NO. 18337-001-00

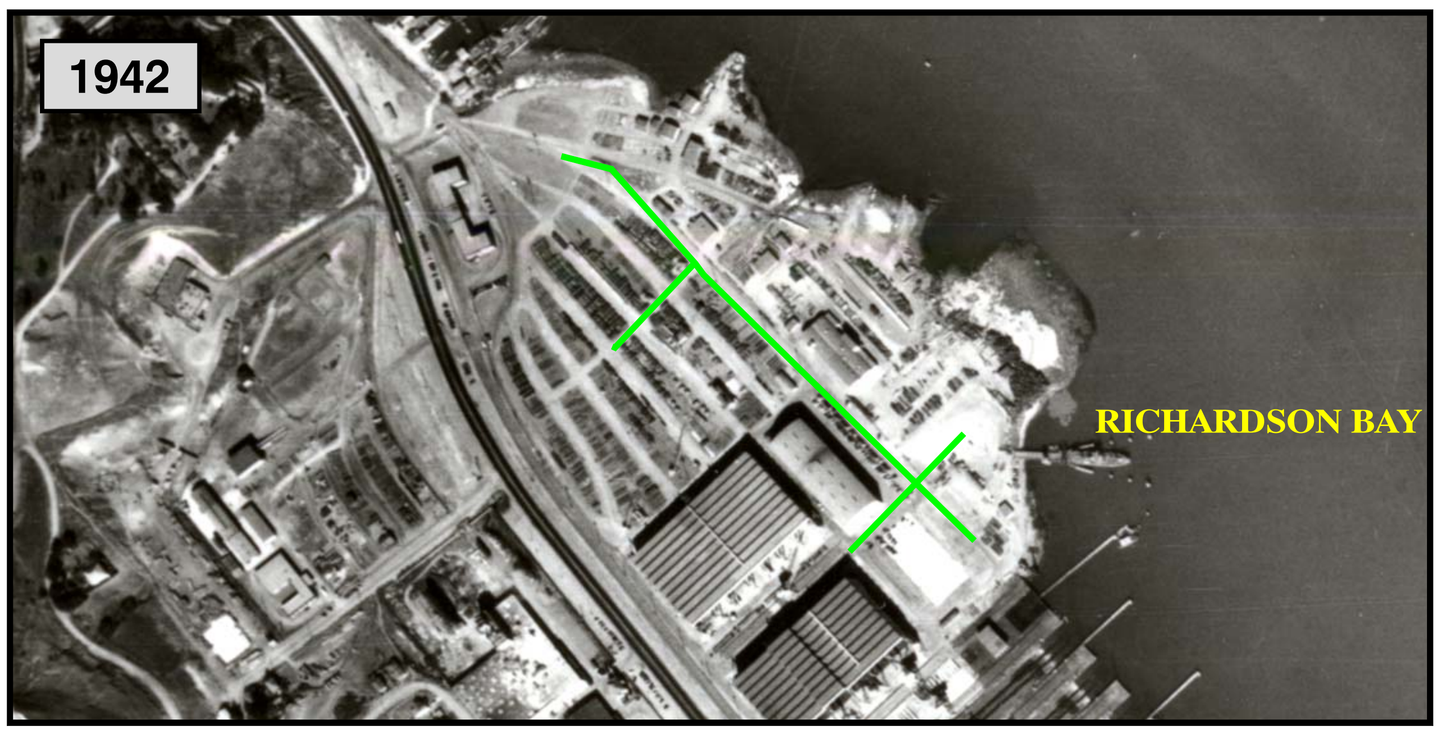
AUGUST 2009



U.S.G.S. Tamalpais (1897) & San Francisco (1895) 15 Min. Quadrangles.



Pacific Aerial Surveys, AV-957-07-25, flown 6-12-70.



Pacific Aerial Surveys, AV 9-5-3, flown 10-28-46.

- Project Alignment

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California	PLATE NO. I-6 (COLOR PLATE)
FILE NO. 18337-001-00 AUGUST 2009	HISTORIC TOPO MAPS & AERIAL PHOTOS - GATE 5 ROAD AREA	

U.S.G.S. San Francisco (1895) 15 Min. Quadrangle.

Pacific Aerial Surveys, AV-957-07-25, flown 6-12-70.

Pacific Aerial Surveys, AV 9-5-3, flown 10-28-46.

- Project Alignment

DCM GeoEngineers		WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California HISTORIC TOPO MAPS & AERIAL PHOTOS - SPINNAKER AREA	PLATE NO. I-7 (COLOR PLATE)
FILE NO. 18337-001-00	AUGUST 2009		

APPENDIX A

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KEY TO PROJECT BORING LOGS IN APPENDIX B

- Grab sample
- 1.4" I.D./2" O.D. Standard Penetration Test (ASTM D1586) sampler (SPT)
- 2.5" I.D./3" O.D. Modified California sampler (MCS) with brass liners
- 2.9" I.D./3" O.D. Shelby tube sample
- Water level measured at end of drilling
- Projected project pipeline

RELATIVE DENSITY		CONSISTENCY		
SANDS AND GRAVELS	SPT, N	SILTS AND CLAYS	SPT, N	UNCONFINED COMPRESSIVE STRENGTH, tsf
VERY LOOSE	0-4	VERY SOFT	0-2	0-0.25
LOOSE	4-10	SOFT	2-4	0.25-0.50
MEDIUM DENSE	10-30	MEDIUM STIFF	4-8	0.50-1.00
DENSE	30-50	STIFF	8-15	1.00-2.00
VERY DENSE	50+	VERY STIFF	15-30	2.00-4.00
		HARD	30+	>4.00

Reference: Terzaghi, K. and Peck, R., SOIL MECHANICS IN ENGINEERING PRACTICE, 2nd ed., John Wiley and Sons, New York, 1967. Page 341 Table 45.1 and pp. 347 Table 45.2.

CONSTITUENT DESCRIPTIONS

DESCRIPTION	CRITERIA
TRACE	less than 5%
FEW	5% to 10%
LITTLE	15% to 25%
SOME	30% to 45%
MOSTLY	50% to 100%

Reference: ASTM D2488, Note 15

MOISTURE CONDITION

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp but no visible water
WET	Visible free water, usually soil is below water table

Reference: ASTM D2488, Table 3 - Criteria for Describing Moisture Condition

GROUND BEHAVIOR	CLASSIFICATION
Ground that can be excavated without initial support and where shoring can be installed before the ground starts to move. For example, unfissured hard clay when not highly overstressed.	Firm
Ground of which chunks or flakes begin to fall off excavation walls. If raveling starts within a few minutes of excavation then it is "fast" raveling; otherwise, it is "slow" raveling. Silts and sands with clay binder may be fast raveling. Stiff fissured clays may be slow or fast raveling depending upon the degree of overstress.	Raveling
Ground that squeezes or plastically extrudes into excavations without visible fracturing. Can occur at shallow to medium depth in very soft to medium stiff clay, and can occur in stiff to hard clay under high overstress.	Squeezing
Ground consisting of clean dry granular material (e.g., sand and gravel) that moves by gravity to its angle of repose.	Running
Ground in a fluid-like condition (e.g., a disturbed mixture of predominantly silt, sand and/or gravel with water), that flows across pressure gradients.	Flowing
Ground that expands in volume due to the absorption of water (e.g., clays).	Swelling

Reference: Modified from Heuer, R.E., 1974, Important ground parameters in soft ground tunneling, Subsurface exploration for underground excavation and heavy construction, New England College, Henniker, New Hampshire, American Society of Civil Engineers, New York, P. 41-55.

NOTES:

1. Lines separating strata in the logs represent approximate boundaries and are dashed where strata change depth is less certain. Strata change may be gradual.
2. Penetration Resistance (blows/ft.) are the last 12" of an 18" drive using a 140-pound cathead hammer falling 30 inches per blow unless noted otherwise. The Penetration Resistance values noted on the logs are actual blows per foot of penetration for the respective sampler type (i.e., MCS sampler penetration resistance blow counts have not been reduced to approximate SPT sampler "N" values).
3. All borings were made with a Mobile B-24 drill rig using 5-inch diameter continuous flight solid stem augers.
4. See plates in Appendix C for grain size definitions and nomenclature.

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PLATE NO.
A-1

FILE NO. 18337-001-00 | AUGUST 2009

BORING LOG LEGEND

(1 of 2)

KEY TO PROJECT BORING LOGS IN APPENDIX B (Cont'd)

UNIFIED SOIL CLASSIFICATION SYSTEM

CRITERIA FOR ASSIGNING GROUP SYMBOLS AND GROUP NAMES [A]				SOIL CLASSIFICATION: GROUP SYMBOL	SOIL CLASSIFICATION: GROUP NAME [B]
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	**GRAVELS** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels** < 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]	**GW**	Well-graded gravel [F]
			$Cu < 4$ and/or $1 > Cc > 3$ [E]	**GP**	Poorly graded gravel [F]
		Gravels with Fines > 12% fines [C]	Fines classify as ML or MH	**GM**	Silty gravel [F,G,H]
			Fines classify as CL or CH	**GC**	Clayey gravel [F,G,H]
	SANDS 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands** < 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]	**SW**	Well-graded sand [I]
			$Cu < 6$ and/or $1 > Cc > 3$ [E]	**SP**	Poorly graded sand [I]
		Sands with Fines > 12% fines [D]	Fines classify as ML or MH	**SM**	Silty sand [G,H,I]
			Fines classify as CL or CH	**SC**	Clayey sand [G,H,I]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	**SILTS AND CLAYS** Liquid limit ≤ 50	**Inorganic**	PI > 7 plots on or above "A" line [J]	**CL**	Lean clay [K,L,M]
			PI < 4 plots below "A" line [J]	**ML**	Silt [K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	**OL**	Organic Clay [K,L,M,N]
					Organic Silt [K,L,M,O]
	SILTS AND CLAYS Liquid limit > 50	**Inorganic**	PI plots on or above "A" line	**CH**	Fat clay [K,L,M]
			PI plots below "A" line	**MH**	Elastic silt [K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	**OH**	Organic Clay [K,L,M,P]
					Organic Silt [K,L,M,Q]
HIGHLY ORGANIC SOILS		**Primarily organic matter, dark color and organic odor**		**PT**	Peat

NOTES:

- **A** Based on the material passing the 3-in. (75mm) sieve.
- **B** If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
- **C** Gravels with 5% to 12% fines require dual symbols:
  - GW-GM well-graded gravel with silt
  - GW-GC well-graded gravel with clay
  - GP-GM poorly graded gravel with silt
  - GP-GC poorly graded gravel with clay
- **D** Sands with 5% to 12% fines require dual symbols:
  - SW-SM well-graded sand with silt
  - SW-SC well-graded sand with clay
  - SP-SM poorly graded sand with silt
  - SP-SC poorly graded sand with clay
- **E** $Cu = \frac{D_{60}}{D_{10}}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
- **F** If soil contains ≥ 15% sand, add "with sand" to group name.
- **G** If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
- **H** If fines are organic, add "with organic fines" to group name.
- **I** If soil contains ≥ 15% gravel, add "with gravel" to group name.
- **J** If Atterberg limits plot in hatched area, soil is a CL-ML (silty clay).
- **K** If soil contains 15% to 29% plus No. 200,add "with sand" or "with gravel", whichever is predominant.
- **L** If soil contains ≥ 30% plus No.200, predominantly sand, add "sandy" to group name.
- **M** If soil contains ≥ 30% plus No.200, predominantly gravel, add "gravelly" to group name.
- **N** PI ≥ 4 and plots on or above "A" line.
- **O** PI < 4 or plots below "A" line.
- **P** PI plots on or above "A" line.
- **Q** PI plots below "A" line.

APPENDIX B

LOG OF BORING B-1 ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 10'
PROJECTED STATION: 15 + 95
PLANNED PIPELINE INVERT ELEVATION: 2'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	% MOISTURE	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
					GATE 5 ROAD: 10 inches asphalt concrete over 20 inches aggregate base - base layer contains few to some coarse gravel										
	1				**CLAYEY SAND WITH GRAVEL (SC) - FILL** - dark yellowish brown - coarse-grained angular gravel - fine sand - silty - dry										
5															
	2		6		**CLAYEY GRAVEL WITH SAND (GC) - FILL** - dark gray - few to little medium and coarse sand - very loose to loose grading to medium dense - wet - Sandy Lean Clay (CL) from 8 - 8½' Borehole sloughing below ~5' (see table below) ⑤	10	129			45	39	16			
	3		4												
10															
	4		7												
	5		10												
15					**CLAYEY SAND WITH GRAVEL (SC) - FILL** - dark gray - medium to coarse sand - loose - wet										
	6		8			12									
20					**BOTTOM OF BORING AT 19 FEET**										
25															

SLOUGH DEPTHS ON SAMPLING

Sample No.	Slough Depth*
6	8'

*Slough depth measured up from intended sample depth.

NOTES

① Drilled 7/6/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 6½ feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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Priority 1 Sewer Replacement
Sausalito, California

PLATE NO. B-1

FILE NO. 18337-001-00 | AUGUST 2009

BORING LOG B-1

LOG OF BORING B-2 ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 8'
PROJECTED STATION: 21 + 00
PLANNED PIPELINE INVERT ELEVATION: -5'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	Sand (#4 to #200 sieve) %	Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	Internal Friction Angle
	1				**GATE 5 ROAD:** 6 inches asphalt concrete over 24 inches aggregate base										
	2		12	▼	**GRAVELLY LEAN CLAY WITH SAND (CL) - FILL** - brownish yellow to dark brownish yellow, becoming reddish with depth - angular gravel - fine sand - medium to low plasticity - stiff - dry to moist			40	18						
5															
	3		8		**POORLY GRADED GRAVEL (GP) - FILL** - dark brown - angular gravel - very loose to loose - wet					86	10	4			
	4		4												
10					Borehole sloughing below 5½' (see table below)										
	5		2		⑤ **POORLY GRADED SAND (SP) - FILL** - brown - trace gravel - fine to medium sand - loose - wet	20									
15					**CLAYEY GRAVEL WITH SAND (GC) - FILL** - dark grayish brown - subangular coarse gravel - loose - wet										
	6		5			14									
					BOTTOM OF BORING AT 18½ FEET										
20															
25															

SLOUGH DEPTHS ON SAMPLING

Sample No.	Slough Depth*
3	1½'
4	3'
5	6½'

*Slough depth measured up from intended sample depth.

NOTES

① Drilled 7/6/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 4 feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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PLATE NO. B-2

FILE NO. 18337-001-00 | AUGUST 2009 | BORING LOG B-2

LOG OF BORING B-3A①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 7½'
PROJECTED STATION: 46 + 60
PLANNED PIPELINE INVERT ELEVATION: 1'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	Sand (#4 to #200 sieve) %	Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	Internal Friction Angle
					HARBOR DR: 6 inches asphalt concrete over 18 inches aggregate base										
					LEAN CLAY WITH SAND (CL) - FILL - yellowish brown - trace gravel - medium grained sand - medium plasticity - moist										
5				▼	**LEAN CLAY WITH SAND (CL) - FILL** - dark yellowish brown and dark gray - few to some jumbled pockets of sand and gravel - dry to moist ⑤										
	1		9		**LEAN CLAY (CL) - FILL** - light olive brown - trace medium sand-sized to coarse gravel-sized pieces of brick - few fine sand - medium plasticity - medium stiff - moist	21	105							480	16°
	2		5							CORROSION TEST Samples 1 & 2 See Plate C-5					
10					**LEAN CLAY (CL) TO FAT CLAY (CH) - FILL** - dark gray to dark greenish gray and few dark yellowish brown - trace organics - trace shells - trace gravel - fine to medium sand; content varies throughout from trace to little - medium to high plasticity - soft to medium stiff - moist to wet										
	3		6			21	109						1.40		
	4		6					82	49						
15					**ORGANIC CLAY (OH) - FILL** - black - organic odor - high plasticity - medium stiff - moist to wet					LL RATIO $\frac{O.D.}{N.D.}$ =0.67					
	5		pushed		**LEAN CLAY (CL) - FILL** - dark grayish brown - trace to few fine gravel - trace to few fine sand - medium plasticity - medium stiff - wet										
	6		8												
20					**SANDY LEAN CLAY WITH GRAVEL (CL) - FILL** - dark greenish gray to dark bluish gray - coarse sand - angular gravel - low to medium plasticity - medium stiff - wet										
					BOTTOM OF BORING AT 20 FEET										
25															

NOTES

① Drilled 7/6/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 7 feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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PLATE NO. B-3A

FILE NO. 18337-001-00 | AUGUST 2009

BORING LOG B-3A

LOG OF BORING B-3B ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 7½'
PROJECTED STATION: 46 + 60
PLANNED PIPELINE INVERT ELEVATION: 1'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	% MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	Sand (#4 to #200 sieve) %	Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	Internal Friction Angle
					HARBOR DR: 6 inches asphalt concrete over 16 inches aggregate base										
	1	X			**LEAN CLAY WITH SAND (CL) - TRENCH BACKFILL** - yellowish brown - trace gravel - medium grained sand - medium plasticity - moist	21		47	21						
5					**BOTTOM OF BORING AT 3½ FEET**										
					⑤										
10															
15															
20															
25															

NOTES

① Drilled 7/9/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level was not encountered during drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Existing Priority pipeline to be replaced with planned 8" ID pipeline.

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PLATE NO. B-3B

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BORING LOG B-3B

LOG OF BORING B-4 ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 11'
PROJECTED STATION: 42 + 60
PLANNED PROFILE D INVERT ELEVATION: 1½'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (>#4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (<#200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
	1				**HARBOR DR:** 4 inches asphalt concrete over 22 inches aggregate base										
	2		25		**LEAN CLAY (CL) - FILL** - very dark brown - trace sand and fine gravel - medium plasticity - dry										
5	3		10		**LEAN CLAY WITH SAND (CL) - FILL** - dark gray - angular gravel - fine gravel to cobble at 3½'; broken by sampler - medium plasticity - moist - very stiff					38	44	18			
	4		10		**CLAYEY SAND WITH GRAVEL (SC) - FILL** - brown with yellowish brown splotches - all sizes sand and gravel - very loose to loose - moist ⑤	12	124			54	28	18			
10	5		4		**CLAYEY GRAVEL WITH SAND (GC) - FILL** - brown - angular to subrounded gravel - ~2½" gravel near 9' - very loose to loose - wet					47	32	21			
	6		7		**SILTY SAND (SM) - FILL** - very dark gray - trace fine angular gravel - coarse sand - loose - wet										
15															
	7		4		**SILTY SAND WITH GRAVEL (SM) - FILL** - brown - fine angular gravel - coarse sand - very loose to loose - wet										
20					**BOTTOM OF BORING AT 20 FEET**										
25															

Borehole sloughing below 7' (see table below)

SLOUGH DEPTHS ON SAMPLING

Sample No.	Slough Depth*
4	¾'
6	6'

*Slough depth measured up from intended sample depth.

NOTES
① Drilled 7/6/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 6 feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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Sausalito, California

PLATE NO. B-4

FILE NO. 18337-001-00 | AUGUST 2009 | BORING LOG B-4

LOG OF BORING B-5 ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 8'
PROJECTED STATION: 20 + 75
PLANNED PIPELINE INVERT ELEVATION: 4'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
					SPINNAKER DR: 9 inches asphalt concrete over 5 inches aggregate base										
	1		30		**LEAN CLAY (CL) - FILL** - grayish brown - broken fine gravel to cobble broken at 3¾' - trace fine sand and gravel - cemented from 3-3½' - medium plasticity - very stiff - moist ⑤	12	118	43	18						
5	2		5		**LEAN CLAY (CL) - FILL** - dark brown with yellowish brown and grayish green mottles - trace to few coarse sand - trace to few fine gravel - low plasticity - medium stiff - moist					48	34	18			
	3		6	▼	**CLAYEY GRAVEL WITH SAND (GC) - FILL** - very dark gray - angular to subangular gravel - all grades sand - loose	21	105						1.24		
10	4		7		**LEAN TO FAT CLAY WITH SAND (CL/CH) - FILL** - mostly dark greenish gray, little very dark gray and little green - soil colors layerd at arbitrary angles - green layer has trace to few fines - trace broken shells at 9' - mostly grayish green below 9' with trace coarse sand and trace coarse gravel - medium to low plasticity - soft to medium stiff - wet - wood found at 10½'										
	5		1			48	73								
	6		pushed												
15	7		11		**ORGANIC CLAY (OH) - FILL & POSSIBLY BAY MUD** - dark greenish gray - trace shells - trace fine sand - high plasticity - very soft - wet - Sample 6 stopped by wood found from 14½ to 16' in Sample 7 - Sample 8 contains some to mostly clam shell fragments										
20	8		pushed												
25					**BOTTOM OF BORING AT 21 FEET**										

CORROSION TEST
Samples 3 & 4
See Plate C-5

NOTES
① Drilled 7/9/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 7 feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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PLATE NO. B-5

FILE NO. 18337-001-00 | AUGUST 2009 | BORING LOG B-5

LOG OF BORING B-6 ①

LOCATION: See Plate I-1

④ BORING SURFACE ELEVATION: 12½'
PROJECTED STATION: 12 + 65
PLANNED PROFILE G INVERT ELEVATION: 2½'

DEPTH feet	SAMPLE NO.	TYPE	PENETRATION RESISTANCE blows/ft.	GROUNDWATER ③	DESCRIPTION ②	MOISTURE %	DRY DENSITY lbs./ft.³	LIQUID LIMIT	PLASTICITY INDEX	GRAIN SIZE Gravel (> #4 sieve) %	GRAIN SIZE Sand (#4 to #200 sieve) %	GRAIN SIZE Fines (< #200 sieve) %	UNCONFINED COMPRESSIVE STRENGTH kips/ft.²	DIRECT SHEAR Cohesion p.s.f.	DIRECT SHEAR Internal Friction Angle
					HUMBOLDT AVE: 6 inches asphalt concrete over 11 inches aggregate base										
					CLAYEY SAND (SC) - FILL - brown - few gravel - dry										
	1		31		**WELL-GRADED GRAVEL WITH SILT AND SAND (GW-GM) - FILL**	6	120								
5	2		8		- brownish yellow - localized pockets of angular gravel - loose to medium dense - dry					57	36	7			
	3		10		**WELL-GRADED GRAVEL WITH SILT AND SAND (GW-GM) - FILL** - brownish yellow - gravels mostly rounded - loose - moist ⑤	8	113						0.63		
10	4		8	▼						58	34	8			
	5		10		**WELL-GRADED GRAVEL WITH SAND (GW) - FILL** - yellowish brown - angular to rounded grains - loose - wet - few to little fines at 14½' (Borehole sloughing below 12' (see table below))					77	19	4			
15	6		5												
	7		2		**FAT CLAY (CH) - BAY MUD** - very dark brown - trace fine angular gravel - trace sand - medium to high plasticity - very soft to soft - wet	44									
20															
25					**BOTTOM OF BORING AT 21.5 FEET**										

SLOUGH DEPTHS ON SAMPLING

Sample No.	Slough Depth*
6-5	1'
6-6	2½'

*Slough depth measured up from intended sample depth.

NOTES
① Drilled 7/9/09 using a Mobile B-24, 5" diameter solid stem augers, and a 30" drop by 140 lb. cathead sampling hammer.
② See report text in Section I and plates in Appendices A and C for definitions, lab test results, and additional soil descriptions.
③ Free groundwater level measured in boring at depth of 10½ feet after drilling. Static equilibrium groundwater depth is unknown.
④ Rounded from in-progress design plans (West Yost Associates, 2009).
⑤ Projected planned 8" ID pipeline.

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PLATE NO. B-6

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BORING LOG B-6

APPENDIX C

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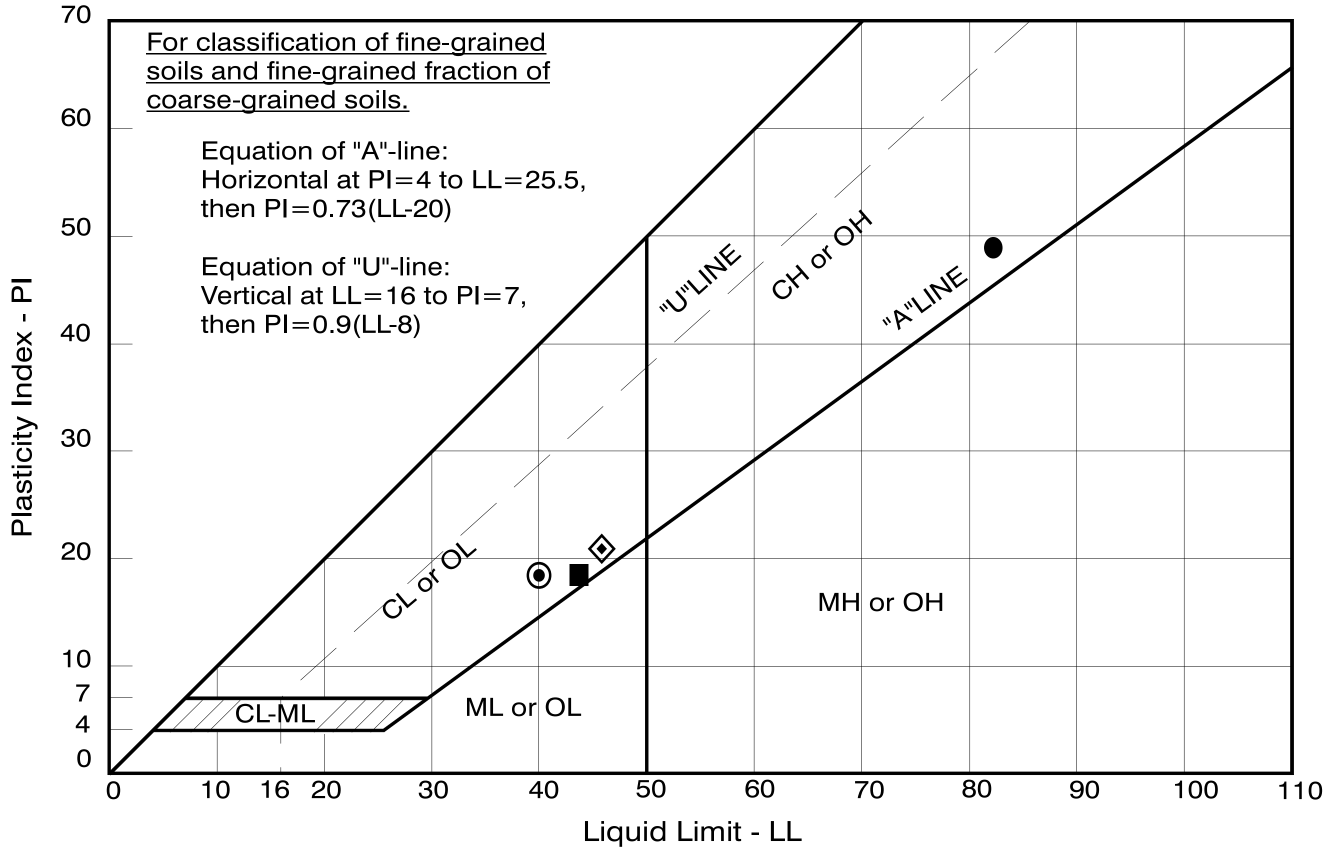

TEST SYMBOL	BORING SAMPLE NO.	DEPTH (ft)	LIQUID LIMIT - LL (not dried)	PLASTICITY INDEX - PI	RATIO*	GROUP SYMBOL**
⦿	B-2-2	2-4	40	18	NA	CL
●	B-3A-4	13¾-14½	82	49	0.67	OH
◈	B-3B-1	2½-3½	47	21	NA	CL
■	B-5-1	3½-4	43	18	NA	CL

* LL oven dried to LL not dried. Where ratio is <0.75 then sample is organic.
** Classification of fines < 0.425mm

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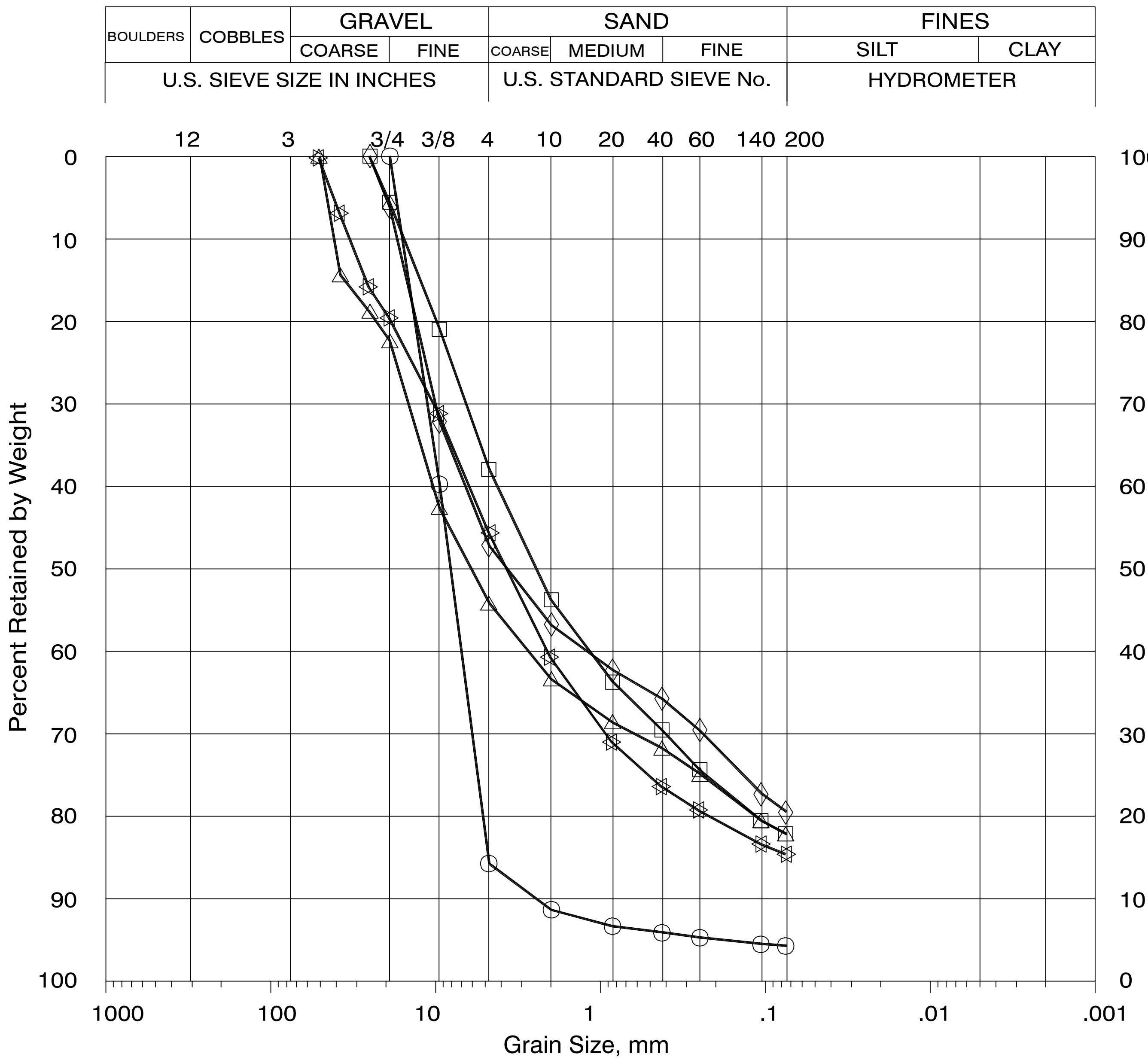
PLASTICITY INDEX

PLATE NO.
C-1



TEST SYMBOL	BORING SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL	DESCRIPTION (based on grain size)
✡	B-1-2	6½-7	GC	clayey gravel with sand
○	B-2-3	7½-8½	GP	poorly graded gravel
□	B-4-3	4½-6	SC	clayey sand with gravel
△	B-4-4	9-9½	GC	clayey gravel with sand
◇	B-4-5	9½-11	GC	clayey gravel with sand

NOTE: The largest particle (grain) size that could have been sampled is a function of the inside diameter of the sample barrel used (see Plate A-1). Therefore, there may be larger particles (e.g., cobbles) in the soils sampled than reflected on the boring logs and grain size distribution curves provided in this report.

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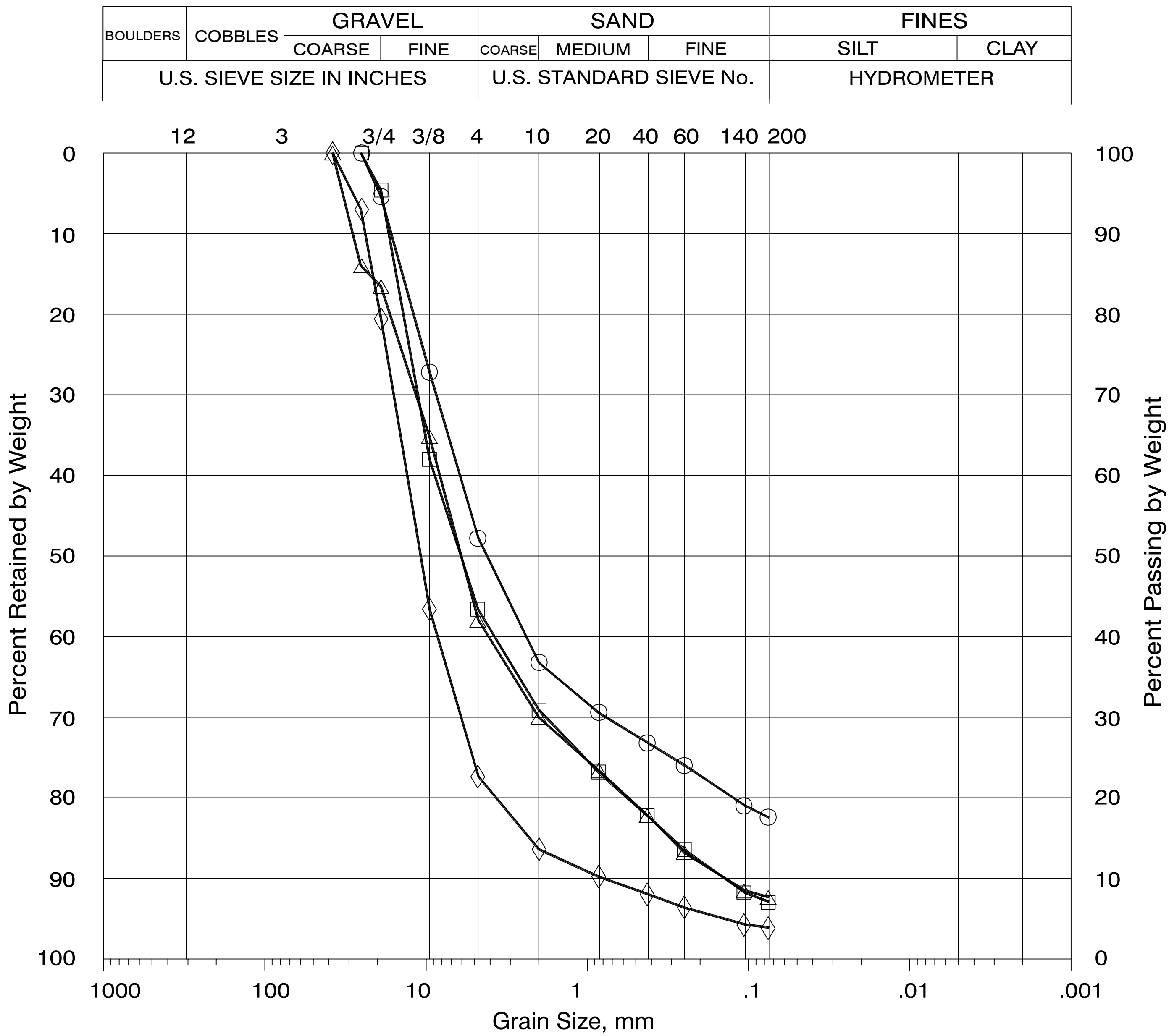
GRAIN SIZE ANALYSIS

PLATE NO.

C-2

(1 of 2)



TEST SYMBOL	BORING SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL	DESCRIPTION (based on grain size)
○	B-5-2	5¼-5½	GC	clayey gravel with sand
□	B-6-2	4½-6	GW-GM	well-graded gravel with silt and sand
△	B-6-4	9½-11	GW-GM	well-graded gravel with silt and sand
◊	B-6-5	13½-14½	GW	well-graded gravel with sand

NOTE: The largest particle (grain) size that could have been sampled is a function of the inside diameter of the sample barrel used (see Plate A-1). Therefore, there may be larger particles (e.g., cobbles) in the soils sampled than reflected on the boring logs and grain size distribution curves provided in this report.

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Priority 1 Sewer Replacement
Sausalito, California

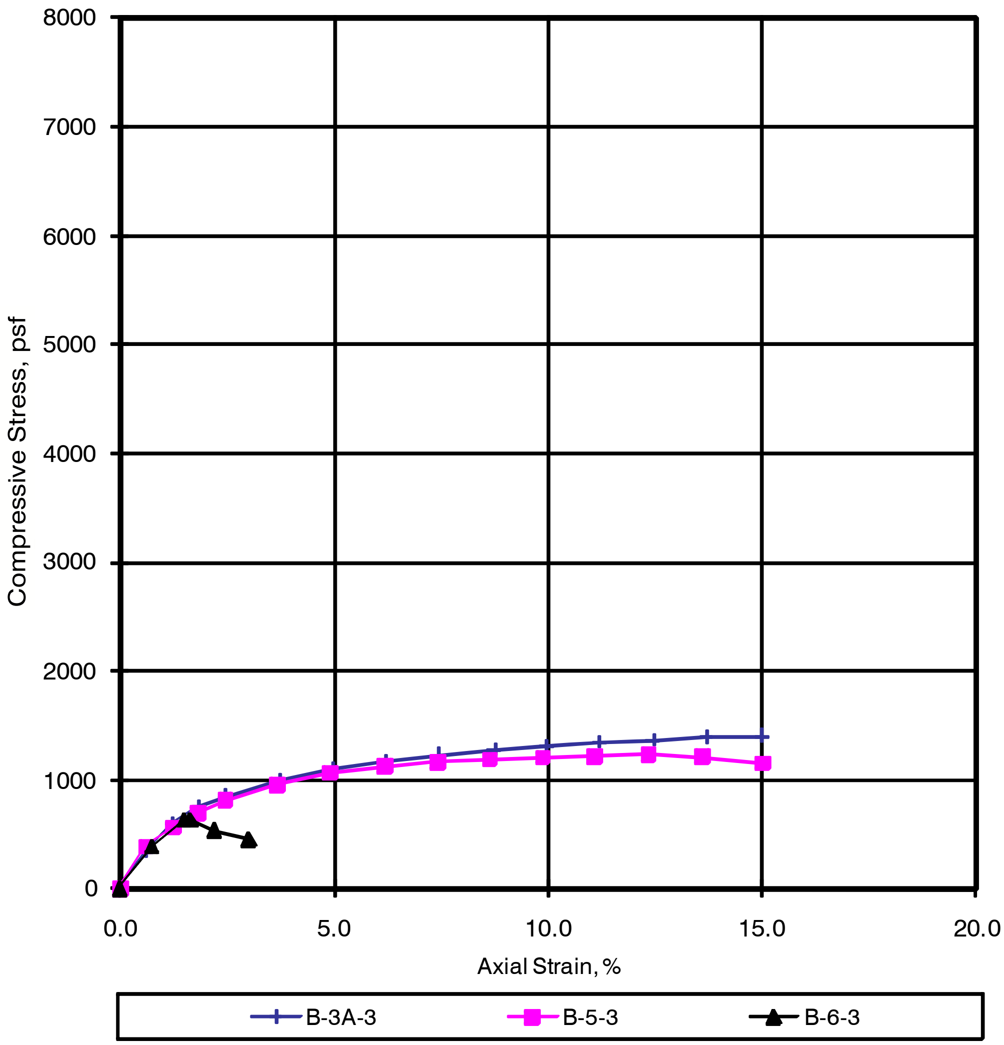
GRAIN SIZE ANALYSIS

PLATE NO.

C-2

(2 of 2)

UNCONFINED COMPRESSION TEST



BORING SAMPLE NO.	B-3A-3	B-5-3	B-6-3
MAXIMUM UNCONFINED STRESS, psf	1,402	1,235	634
% STRAIN @ PEAK STRESS	15.0	12.3	1.7
DEPTH, ft.	13-13½	8½-9	8½-9
WATER CONTENT, %	21	21	8
DRY DENSITY, pcf	109	105	113
SATURATION, %	107	95	45

Maximum Unconfined Stress cut-off = 15% strain
Average Strain Rate = 0.07 in/min.

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Sausalito, California

PLATE NO.

C-3

FILE NO. 18337-001-00

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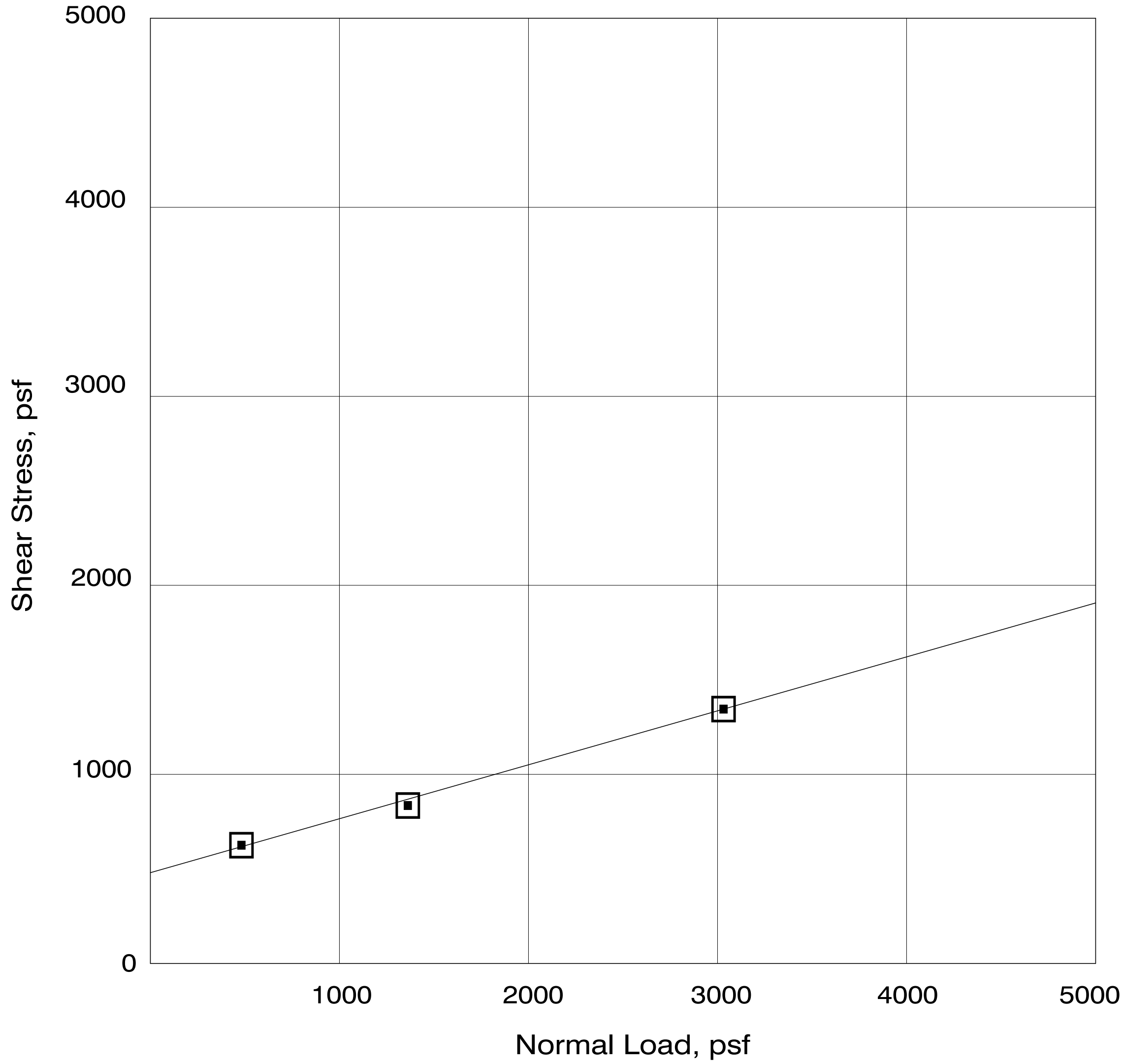
UNCONFINED COMPRESSION



TEST SYMBOL	GRAPH LINE	BORING SAMPLE NO.	DEPTH (ft)	COHESION (p.s.f.)	INTERNAL FRICTION ANGLE (degrees)	AVE. DRY DENSITY (pcf)/ MOISTURE CONTENT (%)	
						BEFORE TEST	AFTER TEST
⊡	———	B-3A-1	7½-8	480	16	105/21	105/23

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Sausalito, California

PLATE NO.

C-4

FILE NO. 18337-001-00

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DIRECT SHEAR

CORROSION TESTS and RESULTS

SAMPLE NO.	RESISTIVITY saturated (ohm-cm)	REDOX (mv)	pH	SULFATE (mg/kg)	CHLORIDE (mg/kg)
B-3A (8-10.5')	140	450	7.6	520	2,100*
B-5 (7-9.5')	320	450	7.5	N.D.	670
Test Method	ASTM G57	ASTM D1498	ASTM D4972	ASTM D4327	ASTM D4327
Detection Limit	-	-	-	15	15

Test Notes:

1. The above tests (excluding redox and sulfides) were performed in accordance with the following ASTM Methods:

   a. ASTM G57: STANDARD TEST METHOD FOR FIELD MEASUREMENT OF SOIL RESISTIVITY USING THE WENNER FOUR-ELECTRODE METHOD

   b. ASTM D1498: STANDARD TEST METHOD FOR OXIDATION-REDUCTION POTENTIAL OF WATER

   c. ASTM D4972: TEST METHOD FOR pH OF SOILS

   d. ASTM D4327: STANDARD TEST METHOD FOR ANIONS IN WATER BY CHEMICALLY SUPPRESSED ION CHROMATOGRAPHY

2. Testing provided by Cerco Analytical.

3. N.D. = Not Detected

* Detection limit elevated to 75 mg/kg due to dilution.

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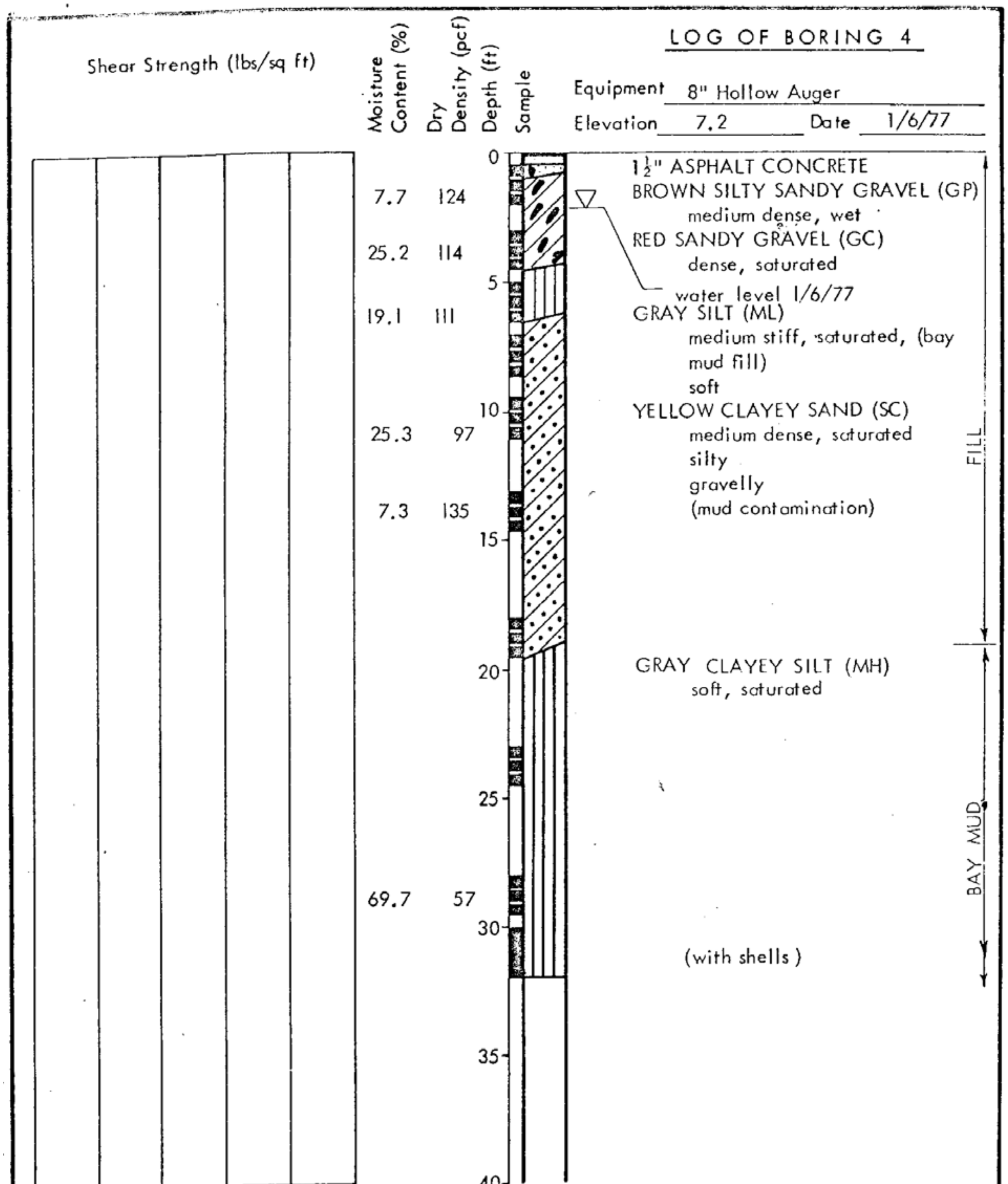
Priority 1 Sewer Replacement
Sausalito, California

SOIL CORROSIVITY

PLATE NO. C-5

FILE NO. 18337-001-00 | AUGUST 2009

APPENDIX D



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Consulting Engineers and Geologists

Job No. 2830,013.01 Appr. Date 1/14/77

LOG OF BORING 4

COMMERCIAL BUILDING

100 GATE 5 ROAD

PLATE

5

Modified from Harding Lawson and Associates (1977)

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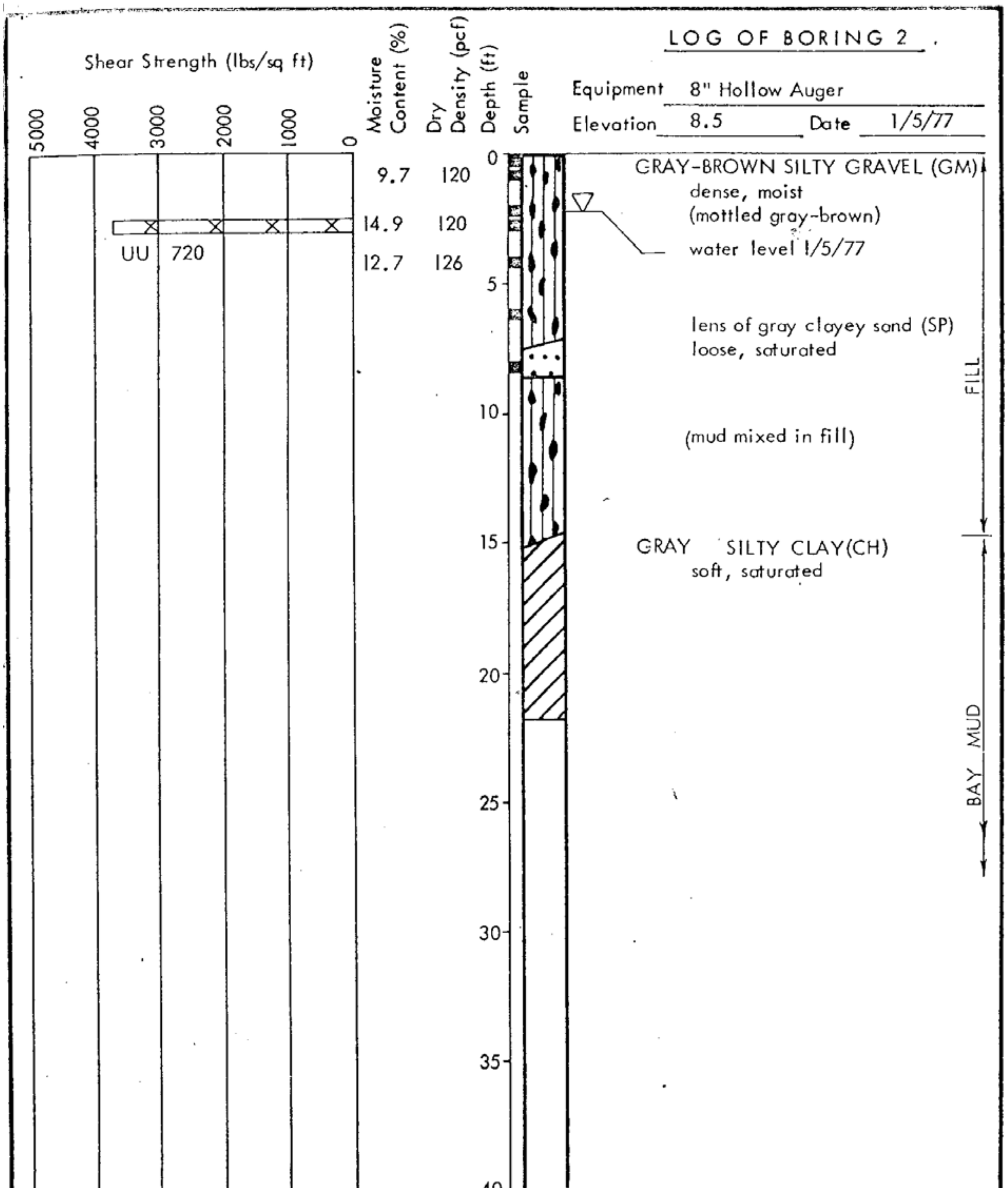
City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

PLATE NO.

D-1

FILE NO. 18337-001-00 | AUGUST 2009

REFERENCE BORING RB-1



HARDING - LAWSON ASSOCIATES

Consulting Engineers and Geologists

Job No 2830,013.01 Appr [signature] Date 1/14/77

LOG OF BORING 2

COMMERCIAL BUILDING
100 GATE 5 ROAD

PLATE
3

Modified from Harding Lawson and Associates (1977)

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-2

PLATE NO.
D-2

LOG OF BORING 6

Equipment 8" Hollow Auger

Elevation +6.0 Ft. Date 9/26/79

Blows/foot	Moisture Content (%)	Dry Density (pcf)	Depth (ft)	
11			0	RED-BROWN SANDY CLAY (CL) soft to moderately stiff, dry
14				GRAY SILTY SAND (SM) moderately dense, dry, with pebbles soft at 4 feet
			5	RED-BROWN SANDY CLAY (CL) stiff, moist
19				BROWN-GRAY SANDY SILT (ML) moderately stiff, moist
12			10	saturated increase in gravel
14			15	GRAY SILTY GRAVEL (GM) moderately dense, saturated
			20	
			25	very coarse gravel
21			30	GRAY SILT (MH) medium stiff, saturated
Push			35	water level not observed
			40	

FILL (0 to approximately 29 ft)

Harding Lawson Associates
Engineers Geologists
& Geophysicists

LOG OF BORING 6
Federal Ornamental Building
Sausalito, California

PLATE 8

DRAWN P. Anderson | JOB NUMBER 2830,017.01 | APPROVED KHB | DATE 12/3/81 | RE. SED | DATE

Modified from Harding Lawson and Associates (1981)

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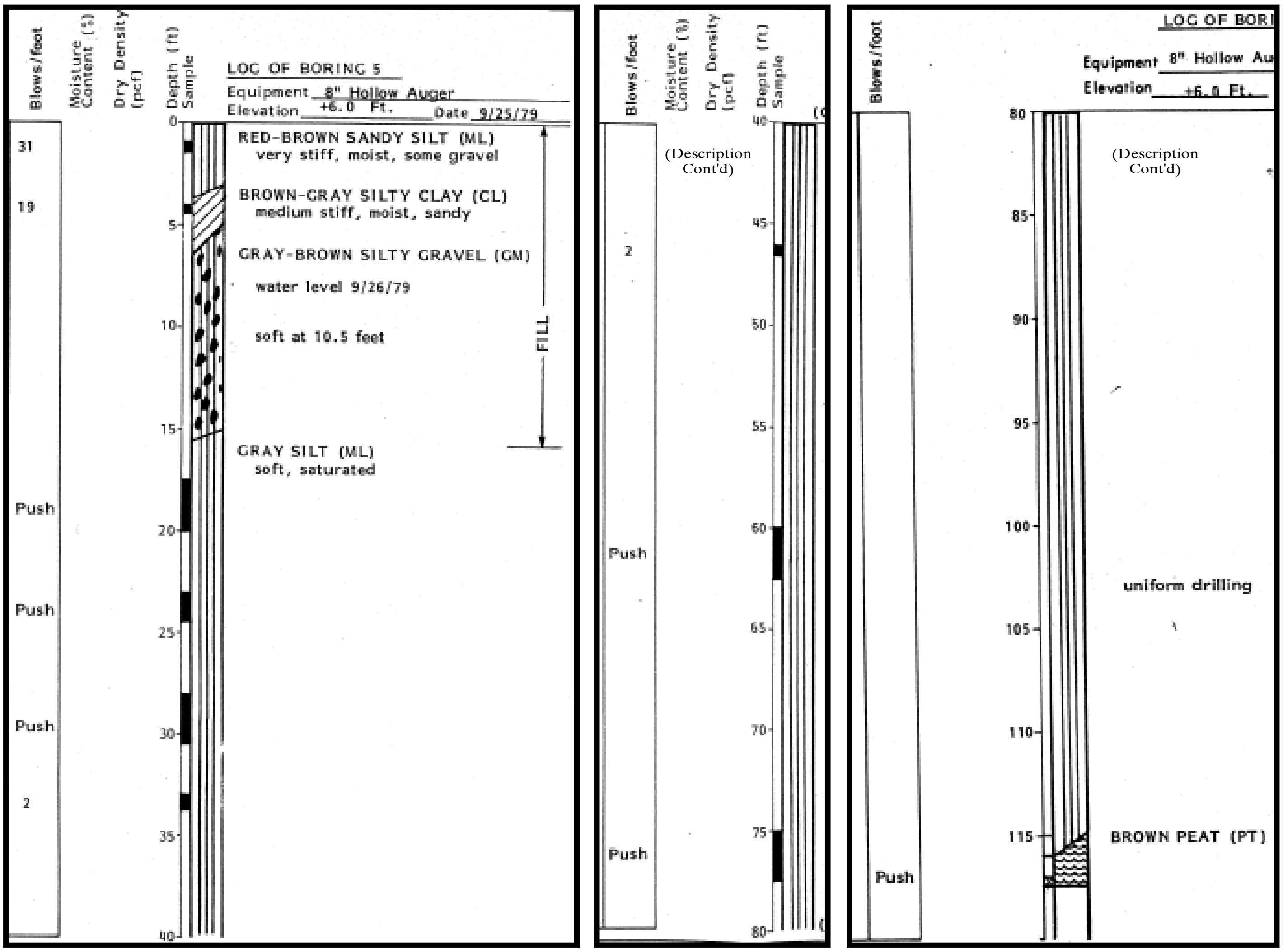
City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-3

PLATE NO. D-3

FILE NO. 18337-001-00 | AUGUST 2009



Modified from Harding Lawson and Associates (1981)

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California	PLATE NO. D-4
FILE NO. 18337-001-00 / AUGUST 2009	REFERENCE BORING RB-4	

LOG OF BORING 4

Equipment 8" Hollow Auger

Elevation +4.0 Ft. Date 9/25/79

Blows/foot	Moisture Content (%)	Dry Density (pcf)
22		
16		
27/6"		
8		
Push		

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35, 40

Sample

BROWN SANDY SILT (ML)
soft to medium dense, dry
with pebbles

▽ water level 9/26/79

YELLOW-BROWN SANDY CLAY (CL)
medium stiff, saturated

concrete block at 8.5 feet
(moved 5 feet west and redrilled)

GRAY SANDY SILT (ML)
soft, saturated, with
occasional small pebbles

GRAY SILT (ML)
soft, saturated

FILL

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& Geophysicists

LOG OF BORING 4
Federal Ornamental Building
Sausalito, California

PLATE 5

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
P. Anderson	2830,017.01	KAB	12/3/81		

Modified from Harding Lawson and Associates (1981)

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-5

PLATE NO. D-5

FILE NO. 18337-001-00 | AUGUST 2009

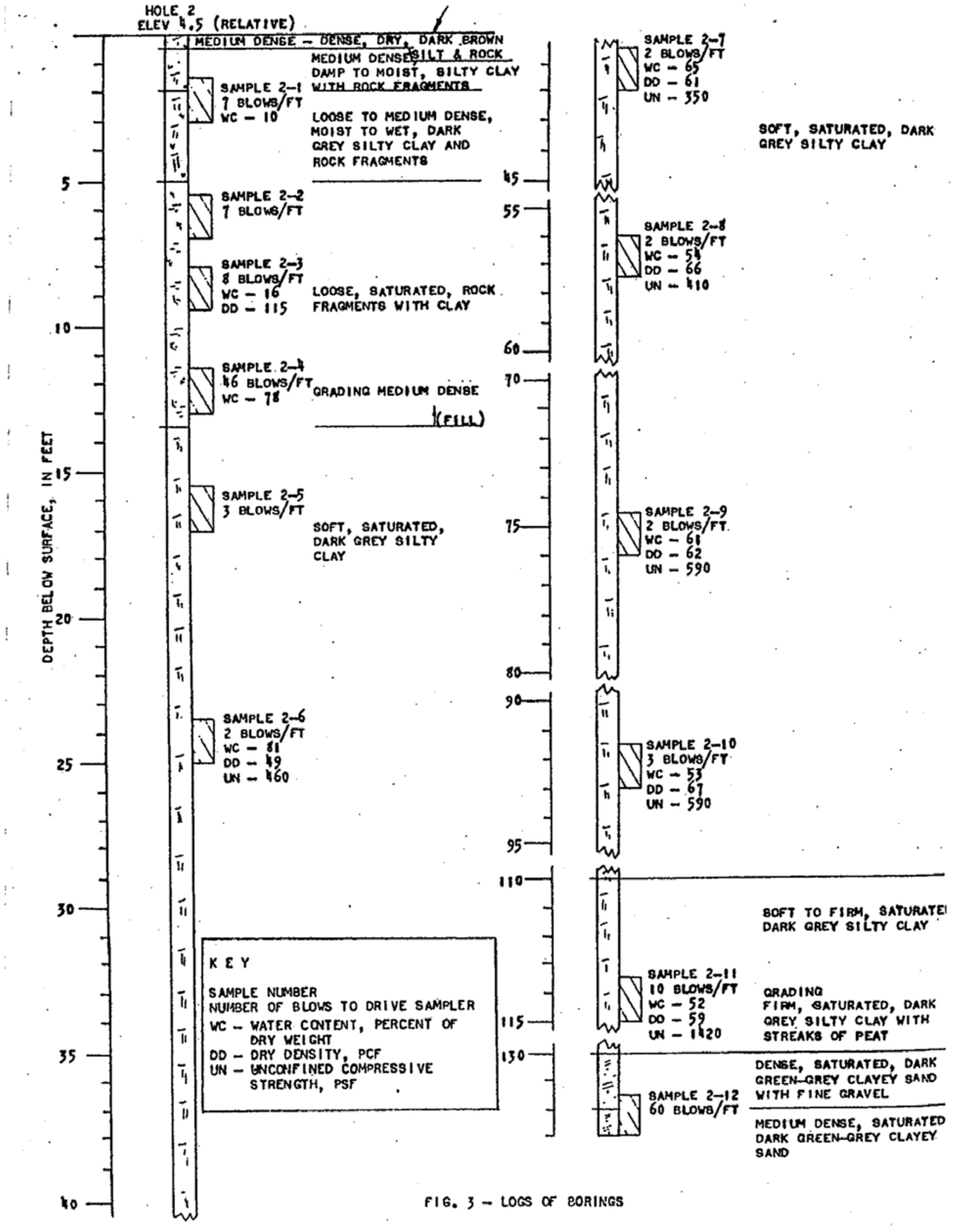

FIG. 3 – LOGS OF BORINGS

Modified from Woodward-Clyde-Sherard & Associates (1959)

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-6

PLATE NO.
D-6

FILE NO. 18337-001-00 | AUGUST 2009

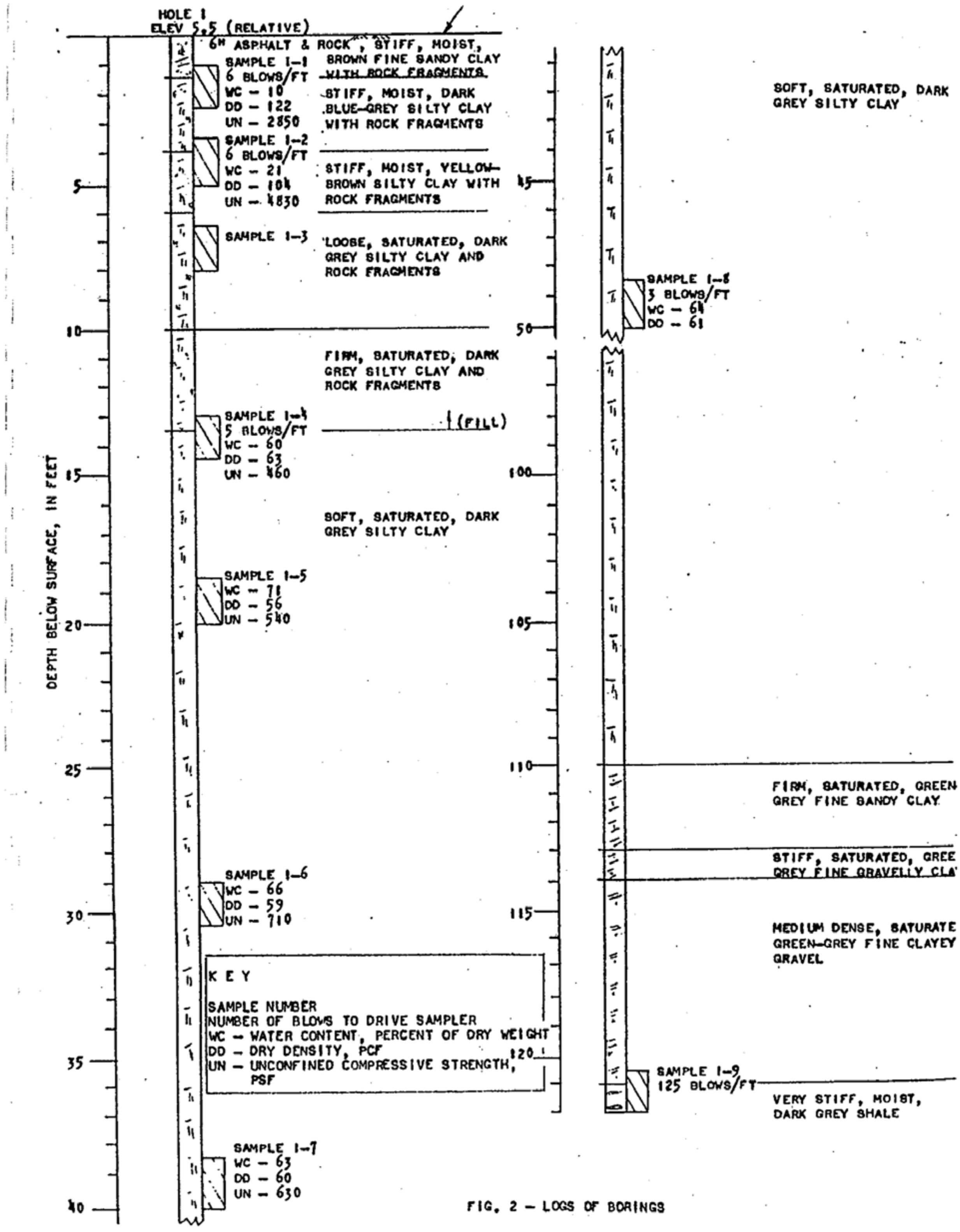

FIG. 2 — LOGS OF BORINGS

Modified from Woodward-Clyde Consultants (1959)

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WEST YOST ASSOCIATES

City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-7

PLATE NO.

D-7

Project: CLIPPER YACHT HARBOR
Sausalito, California

Log of Boring No. 3

Date Drilled: January 12, 1984
Type of Boring: 5" Auger
Hammer Weight: 140 lbs.

Remarks:

(See Legend Sheet for sampler types and hammer weights)

Depth, Ft.	Samples	Blows/Ft.	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, pcf	Unconfined Compressive Strength, psf
			Surface Elevation:			
			ASPHALT			
			CLAYEY SANDY GRAVEL (GC): Medium dense to dense, moist, gray (ATD)			
			Varying sand and gravel content, saturated, gravel to 2" in diameter			
5	1	5	SILTY CLAY (CL) Soft, brown and blue-gray mottled	16	117	608
			Becomes gravelly			
			SANDY GRAVELLY CLAY (CL) Soft to medium stiff, tan			
	2	16	SILTY CLAY (CL): Medium stiff, green-gray	10	131	2382
10			GRAVELLY CLAY (CL) Stiff, red-brown to light brown			
			Gravel to 1" in diameter			
			SILTY CLAY (CL/CH) Soft to medium stiff, gray, (Bay Mud)			
15			CLAYEY GRAVEL (GC) Loose to medium dense, gray, gravel to 1" in diameter			
	3	8				
			Varying clay content			
20						
25			SILTY CLAY (CL/CH) Soft, gray, (Bay Mud)			
			BOTTOM OF BORING @ 26'			
30						

Proj. No. 16155A | Woodward-Cly... | Figure 4

Modified from Woodward-Clyde Consultants (1984)

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

PLATE NO.
D-8

FILE NO. 18337-001-00 | AUGUST 2009

REFERENCE BORING RB-8

GEOENGINEERING, INC.

5-52-wm

BORING No. A
Date Drilled 2/3/95
Station
Approx Elev. +7.2 mllw

2½" OF AC

OTHER TEST DATA	PERCENT SILT CLAY	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	BLOW COUNTS (PER FOOT)	SAMPLE INTERVAL	CLASSIFICATION	0 (DEPTH FEET)		
MEASURED GROUNDWATER (2:30) (9:30)				4		SP	0	BRN & GRY FN-CL SAND & GRAVEL	FILL
				1		SM		RED-BRN FN-MED SAND, LOOSE	FILL
.19 TXUU P1=1.0 P3=1.64	93%	67 73	55 47	SHELBY TUBE PUSH 2		CH	10	GRN GRY CLAY W/S SHELLS & SLIGHT ODOR, SOFT	BAY MUD
	80%	76	44	3		CH		SMALL QUANTITIES OF CREOSOTED WOOD (APPARENT OLD PILE)	BAY MUD
.24/.20 DSUU N=1.8 S=0.34		89	33	2		CH	20	NUMEROUS SHELLS	BAY MUD
.25/.24		63 67	64 55	1		CH		(THIN LENSES OF VY FN SA SILT)	BAY MUD
.22/.27		77	70	0		CH	30		BAY MUD
DSSU N=2.7 S=0.41 .22/.27		58 64	72 59	1		CH	40		BAY MUD
.29/.27		67	64	0		CH	50		BAY MUD
							60	CONTINUED ON FIGURE A2	

Hand Penetrometer/TORVANE READINGS

NOTES:

1. To be interpreted according to Figure A and description in Appendix A & Text.

ABBREVIATION KEY:

SI—Silty	VY—Very	GRY—Gray
SA—Sandy	MED—Medium	GRN—Green
CL—Clayey	STF—Stiff	BRN—Brown
FN—Fine	DNS—Dense	DK—Dark
W—With	LOS—Loose	LT—Light

LOG OF TEST BORING No.

FIG A1

Modified from Geoengineering (1995)

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City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-9

PLATE NO. D-9 (1 of 2)

FILE NO. 18337-001-00 | AUGUST 2009

GEOENGINEERING, INC.

BORING No. A (SHEET 2 OF 2)

5-52-wm

OTHER TEST DATA	PERCENT SILT CLAY	DRY DENSITY (PCF)	MOISTURE CONTENT(%)	BLOW COUNTS (PER FOOT)	SAMPLE INTERVAL	CLASSIFICATION	(DEPTH FEET)	
						CH	60	GRN GRY CLAY, SOFT
.22/.22		63	62	0			70	BAY MUD
						CH	80	
.35/.25		57	76	0			90	
							100	
								GRY-BRN SI CLAY (MED STIFF)
.95/.33		66	64	9			110	
							120	
1.1/.45		55	76	11			130	BOTTOM OF BORING

Hand Penetrometer Readings/TORVANE READINGS

SEE NOTES ON FIGURE A1)

FIG A2

Modified from Geoengineering (1995)

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WEST YOST ASSOCIATES

City of Sausalito
Priority 1 Sewer Replacement
Sausalito, California

REFERENCE BORING RB-9

PLATE NO.

D-9

(2 of 2)

FILE NO. 18337-001-00

AUGUST 2009

SECTION II

GEOTECHNICAL DESIGN SUMMARY

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GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
CITY OF SAUSALITO
PRIORITY 1 SEWER REPLACEMENT PROJECT
SAUSALITO, CALIFORNIA

SECTION II - GEOTECHNICAL DESIGN SUMMARY

1.0 INTRODUCTION

This geotechnical engineering investigation report is for the City of Sausalito's Priority 1 Sewer Replacement Project. The project is located in the Gate 5 Road and Spinnaker Areas of Sausalito, California. The locations of these project areas are illustrated on Plate I-1 Section I. The project consists of replacing existing small-diameter (6- to 8-inch) gravity-flow sanitary sewer pipelines with a new 8-inch diameter gravity-flow pipeline and a new 4-inch diameter force main pipeline. A description of segments in each project area is summarized in Tables II-1 and II-2.

Table II-1 - Sanitary Sewer Replacement - Gate 5 Road Area (see Plate I-2)

Segment Location	Length (ft)	Invert Depth (ft)	Existing Gravity Pipeline		New Replacement Pipeline Size and Construction[1]
			Material	Condition	
Gate 5 Road & southeast extension	1,638	6 to 14	8″ VCP lined with HDPE	contains sags	8″ by open-cut trenching same alignment and depth as existing
Coloma Street	316	9 to 13	VCP lined with HDPE[2]		
Harbor Drive	417	6 to 9	8″ VCP and/or ACP		

[1]The material type of the new pipeline is not known to us at this time.
[2]The diameter of the existing VCP is not known to us at this time.

Table II-2 - Sanitary Sewer Replacement - Spinnaker Area (see Plate I-3)

Segment Location	Length (ft)	Invert Depth (ft)	Existing Gravity Pipeline		New Replacement Pipeline Size and Construction[1]
			Material	Condition	
Humboldt Avenue	282	9 to 12	6″ Cast Iron	contains sags and is severely corroded	8″ by open-cut trenching same alignment and depth as existing
Bay Road and parking lot	593	5 to 9			slip 4″ force main into existing 6″[2]
Bay Road parking lot to Spinnaker Restaurant	208	4 to 5			8″ by open-cut trenching same alignment and depth as existing[3]

[1]The material type of the new pipeline is not known to us at this time.
[2]Includes a new sanitary sewer pump station at the east end of the force main. We have no plan or profile information pertaining to the pump station at this time.
[3]Includes a new, 6- to 8-foot deep grease interceptor at the east end of the segment for the Spinnaker Restaurant.

The new gravity sanitary sewer replacement pipelines will be routed through existing sanitary sewer manholes. Groundwater is infiltrating into the existing sanitary sewer pipeline through some of these manholes. Leaky manholes will be rehabilitated as part of this project (e.g., possibly by manhole lining, chemical grouting, etc. to prevent groundwater infiltration). As illustrated on Plates I-2 and I-3 Section I, the existing gravity sanitary sewer pipelines to be replaced by this project, slope to City pump stations.

This report contains a description of geotechnical conditions along the alignment of planned new sanitary sewer replacement pipelines in the project areas (Section I) and geotechnical conclusions and recommendations for design, construction, and useful long-term performance of the new replacement pipelines and related project structures as described herein (Section II). All descriptions provided in this report pertaining to existing and new sanitary sewer replacement pipelines and related project structures (e.g., location, depth, size, length, material type, condition, and construction methods, etc.) are based on in-progress design plans by West Yost Associates dated August 21, 2009 (West Yost Associates, 2009).

2.0 GEOTECHNICAL DATA SUMMARY

Section II.2 is an interpreted summary of the geotechnical data provided in Section I.

2.1 Roadway Pavements, Fills and Bay Mud

Materials encountered in test borings in the project areas (boring logs are provided in Appendices B and D) consisted of roadway pavements, fills and native soils. Summary descriptions of these materials are provided in the following sections.

2.1.1 Roadway Pavements

Roadway pavements encountered in project test borings are described on the respective logs in Appendix B. A summary of pavement section types and thicknesses encountered in project test borings is provided in Table I-3, Section I. Pavement sections consisting of asphalt concrete over aggregate base rock were encountered in project test borings. Pavement sections encountered in test borings may not be the same type and thickness of pavement sections located in other portions of roadways in the project areas. Composite pavements (i.e., original pavement plus subsequent pavement overlays) are generally thickest in the center of roadways and generally thin laterally to the edges of roadways. Roadways in the Gate 5 and Spinnaker Areas have existed since the 1940s or earlier and may have been repaved multiple times with differing materials. For example, the area of Gate 5 Road south of Harbor Drive was used as a staging and parking area during the 1940s Marinship Shipyard activities (see Section I.2.4) and may have been paved with concrete.

2.1.2 Utility Trench Backfill

Boring B-3B was intentionally drilled into trench backfill above the existing project sanitary sewer pipeline. Boring B-3B encountered lean clay with sand trench backfill. The new sanitary sewer replacement pipeline alignment parallels and crosses other backfilled trench excavations made for the installation of existing utilities. The geometry of these utility excavations (i.e., vertical or side-sloped), and the type of bedding and backfill materials used (i.e., granular vs. cohesive), are not known to us at this time. However, backfill specified by agencies with utilities in the project area (e.g., PG&E, City of Sausalito, Marin County, etc.) is often granular (i.e., sand and gravel), pervious, non-cohesive and with little to no clay content.

2.1.3 Areal Fills

Areal fills were encountered in each of the seven (7) project test borings, except Boring B-3B, to the depths of the invert of the planned new sanitary sewer replacement pipelines in both the Gate 5 Road and Spinnaker Areas of the project. Boring B-3B was intentionally drilled in trench backfill above the existing sanitary sewer pipeline to be replaced. Areal fills encountered in project test borings are described on the respective logs in Appendix B. Areal fills encountered in project test borings included organic clay (OH), lean clay with sand (CL), lean clay (CL), fat clay (CH), gravelly lean clay with sand (CL), clayey sand with gravel (SC), silty sand (SM), silty sand with gravel (SM), poorly graded sand (SP), clayey gravel with sand (GC), well-graded gravel with silt and sand (GW-GM), well-graded gravel with sand (GW), and poorly graded gravel (GP). Similar areal fills were encountered in each of the nine (9) reference test borings. Areal fills encountered in reference test borings are described on the respective logs in Appendix D. A summary of the thicknesses of areal fills encountered in test borings is provided in Table I-3, Section I.

Most of the areal fills in the project areas date back to the early 1900s and were placed to fill in portions of Richardson Bay for the 1940s development of the Marinship Shipyard (see Section I.2.4). Much of the fill material is crushed rock derived from quarries and excavations in the hills along the west side of Sausalito. As illustrated on Plate I-4, these hills are formed by a variety of resistant bedrock types of the Franciscan Bedrock.

Areal fills placed before the 1960s were typically not engineered and consisted solely of end-dump placement (i.e., not compacted). Fills of this type may contain debris and rubbish (see Section I.2.3). Selected parameters and typical engineering properties of areal fills encountered in project test borings (does not include data from reference test borings) and tested in the laboratory are as follows:

- Thickness = 11½ to >20 feet
- Moisture Content = 10 to 21% (12 tests)
- Dry Unit Weight = 105 to 129 pcf (8 tests)
- Average In-Situ Total Unit Weight = 131 pcf (8 tests)
- Standard Penetration Blow Count (N) = 2 to 12 (average = 6 for 18 tests)
- Liquid Limit = 40 to 82* (clayey samples, 3 tests)
- Plasticity Index = 18 to 49* (clayey samples, 3 tests)
- Unconfined Compressive Strength = 0.63 to 1.4 ksf (3 tests)
- Direct Shear Cohesion = 480 psf (clayey sample, 1 test)
- Direct Shear Phi Angle = 16 degrees (clayey sample, 1 test)

- % Passing No. 200 Sieve = 4 to 18 (9 project tests)
- % Passing Retained on No. 4 Sieve = 0 to 77 (9 project tests)

*Sand and gravel areal fill encountered in project test borings were nonplastic.

As noted by references to borehole sloughing in the logs of project test borings (see Borings B-1, B-2, B-4 and B-5 in Appendix B, Section I), much of the granular, non-cohesive areal fill in the project areas demonstrates flowing or fast-raveling behavior where below groundwater and vertically cut in an unshored condition, even within a 5-inch diameter borehole. The tendency for areal fill to flow will be even greater when and where exposed in vertical excavations for the project.

2.1.4 Bay Mud

Bay Mud was encountered in project test Borings B-5 and B-6 below the depth of the invert for the planned sanitary sewer replacement in the Spinnaker Area of the project. Project test borings in the Gate 5 Road Area of the project were drilled to depths of 18½ to 20 feet below ground surface and did not penetrate through the entire thickness of areal fill and into the underlying Bay Mud. Bay Mud encountered in project test borings (see Borings B-5 and B-6) is described on the respective logs in Appendix B and included organic clay (OH) and fat clay (CH). Bay Mud is widely known for being very corrosive to steel and concrete.

Bay Mud was encountered in each of the nine (9) reference test borings that were drilled in the project areas (including Reference Borings RB-1 through RB-8 in the Gate 5 Road Area and Reference Boring RB-9 in the Spinnaker Road Area). Bay Mud encountered in the reference test borings is described on the respective logs in Appendix D. Reference Borings RB-6, RB-7, and RB-9 were the only borings to penetrate through the entire Bay Mud thickness and into underlying alluvial soils and/or bedrock. The thickness of Bay Mud as logged in Reference Borings RB-4, RB-6, RB-7 and RB-9 is approximately 100 feet.

Selected parameters and typical engineering properties of Bay Mud encountered in project test borings and reference borings and tested in the laboratory are as follows:

- Thickness = approximately 100 feet
- Moisture Content = 33 to 81% (26 tests)
- Dry Unit Weight = 49 to 89 pcf (26 tests)
- Average In-Situ Total Unit Weight = 103 pcf (26 tests)
- Standard Penetration Blow Count (N) = 2 (1 test)
- Unconfined Compressive Strength = 0.35 to 0.71 ksf (reference tests)

Reference borings indicate that Bay Mud contains horizons of peat.

Loads on compressible Bay Mud cause the Bay Mud to consolidate (settle). In a steady state, the rate of Bay Mud settlement will slowly decrease over time. For a Bay Mud thickness of 100 feet, it will take hundreds of years to reach "ultimate" consolidation (see Plate II-1).

Studies of Bay Mud in or near the project areas by others have concluded the following:

- Settlement from consolidation of the Bay Mud due to new fills or from lateral movement of the bulkhead or barge in the Spinnaker Area is still possible (Geoengineering, 1995).
- The 15-year settlement estimate of Bay Mud consolidation in the Gate 5 Road Area near Harbor Drive, assuming no additional loading, would be approximately ½ foot between 1999 and 2014 (BAGG, 1999).
- The 15-year settlement estimate of Bay Mud consolidation in the Gate 5 Road Area near Harbor Drive, assuming three to four feet of new areal fill, would be approximately 1 foot between 1999 and 2014 (BAGG, 1999).
- Terrain elevation difference analysis of the Sausalito area by Towill (2009) shows that portions of the project areas in 2007 were generally 3 to 4 feet lower in elevation than in 1968. Towill (2009) reported that areas with large settlements are typically on unimproved land near the present shoreline and areas with large fills where additional fill and improvements have been added since 1968.

2.2 Groundwater

The measured depth to which groundwater accumulated in project test borings on completion of drilling is recorded on the individual boring logs in Appendix B and summarized in Section I, Table I-3. The project test borings were backfilled with grout immediately upon drill completion to minimize traffic disruption; therefore, the groundwater levels measured on completion of drilling do not represent static (i.e., equilibrium) groundwater levels. Equilibrium groundwater levels can take several hours to days to be established in an open borehole. Equilibrium groundwater levels will likely be higher (i.e., closer to the ground surface) than the groundwater levels measured on completion of drilling. In addition, groundwater levels in the project areas will fluctuate based on factors such as tides, seasonal rainfall, water levels in nearby drainages, and possibly other factors not evident at the time of writing this report. Portions of the both the Gate 5 Road and Spinnaker Areas of the project are shown within FEMA's 100-year flood hazard areas (ABAG, 2009).

2.3 Faulting

No active fault (where active fault is defined by the State of California as one with known surface displacement within the last 10,000 years, see Hart and Bryant, 1997) is known to cross the project areas. The nearest active fault to the project areas is the San Andreas Fault, located between 6 and 7 miles to the southwest. The location of the San Andreas Fault, and other seismogenic faults relative to the project areas are shown on Plate II-2.

2.4 Ground Shaking

The project areas will be subject to strong ground shaking during earthquakes on nearby faults, including those identified on Plate II-2. It is estimated that the peak firm rock ground acceleration in the project areas, based on 10% probability of exceedance in 50 years (equivalent to a seismic recurrence interval of one event every 475 years), is 0.5g (see Plate II-3). The actual ground shaking that will occur in the project areas during an earthquake will be dependent upon the earthquake magnitude, its distance, surrounding topography, and the geometric relationships and seismic response of the underlying soil and bedrock. Earthquake shaking in the Bay Area has been amplified in areas underlain by Bay Muds during historic earthquakes (e.g., the 1989 Loma Prieta earthquake). Bolt (1993) indicates that average peak ground accelerations greater than 0.5g results in ground cracks and breakage of underground pipes (see Plate II-4).

2.5 Liquefaction

Liquefaction is a phenomenon in which soils lose internal strength as a result of increased pore pressure generated by cyclic loading. This behavior is commonly induced by ground shaking during earthquakes. Soils prone to liquefaction are saturated (below groundwater), non-cohesive silts and sands of low to medium density. Liquefaction-prone soils encountered in project test borings include the loose, poorly graded sand fill at depths of 12½ to 15½ feet below ground surface in Boring B-2, and the loose, silty sand fill at depths of 12 to 20 feet below ground surface in Boring B-4. The Association of Bay Area Governments has identified all areas of Sausalito having old fills over Bay Mud (i.e., both project areas) as very highly susceptible to liquefaction (ABAG, 2009). Historically, we are aware of only one instance of earthquake-caused liquefaction occurring in the Sausalito area (Youd and Hoose, 1978). This instance occurred during the 1906 San Francisco earthquake near the Sausalito Ferry Building (the Ferry Building was located at that time part-way between the Gate 5 Road and Spinnaker Areas). We are not aware of any reports of liquefaction occurring in the Sausalito area during the 1989 Loma Prieta earthquake (Knudsen, 2000).

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the findings of our geotechnical investigation and our understanding of the project, it appears that the planned new sanitary sewer replacement pipeline and related project structures will be entirely within old areal fills placed over Bay Mud. Approximately 30,000 wooden piles were driven into the ground in the area to provide foundations for construction of the 1940s Marinship Shipyard (see Section I.2.4). Uneven consolidation settlement of the underlying Bay Mud (and possibly the effects of the old driven piles) has resulted in the development of sags (and possibly hogs) in the existing sanitary sewer pipeline in the project areas. As described in

Section II.2.1.4, the Bay Mud in the project areas will continue to consolidate for hundreds of years. This consolidation settlement of the Bay Mud will be uneven over the lengths of the pipelines as a result of variations in composition, consistency and thickness of the Bay Mud, variations in fill thickness and possibly underlying remnant pile foundations. These factors will lead to differential settlement of the new pipelines. Therefore, unless founded at depth (e.g., on piles), the new sanitary sewer replacement pipeline will develop sags in the future as a consequence of continued ongoing Bay Mud consolidation settlement. Supporting the new sanitary sewer replacement pipeline at depth is economically unfeasible for small-diameter sanitary sewer pipelines. Pipeline design and long-term expectations for the pipeline must therefore include continuing, on-going consolidation and settlement of the thick underlying Bay Mud.

Assuming no significant new loading on the Bay Mud in the project areas (i.e., no new fills), the rate of future total and differential settlement in the new replacement sanitary sewer pipeline will be less than that which has developed in the existing pipeline. As described in Section I.2.4.1, portions of the Gate 5 Road Area south of Harbor Drive have been raised with about 4 feet of fill within the last few years. The resulting rise in ground surface elevation due to this new fill is visible in Towill's 1968 to 2007 terrain elevation difference analysis of Sausalito (2009). We anticipate that the rate and amount of future total and differential settlement in the new sanitary sewer replacement pipeline in the Gate 5 Road Area south of Harbor Drive will be significantly greater than in other areas of the project due to this relatively new areal fill loading. Based on an underlying Bay Mud thickness on the order of 100 feet beneath all project pipelines, we estimate that between 2 and 3 feet of total Bay Mud consolidation settlement will occur in the next 50 years in the newly fill-raised area of the project (i.e., Gate 5 Road Area south of Harbor Drive). Elsewhere in the project areas, we estimate that between 1 and 2 feet of total Bay Mud consolidation settlement will occur in the next 50 years given the existing long-term loading conditions that date back to the 1970s and earlier (see Section I.2.4).

Over the long-term, the new pipelines will most likely develop sags from differential settlement in the same locations as the existing pipeline. To the greatest extent possible and practical, the new pipeline design should incorporate past performance into new design. For example, increase pipe slopes where possible to off-set on-going settlement. In addition, a jointless pipe such as HDPE or fusible PVC will eliminate problems with settlement-induced joint spreading and inflow and infiltration (these pipe materials will also eliminate problems associated with soil corrosivity).

Notwithstanding the effects of ongoing, long-term consolidation settlement of the underlying Bay Mud (i.e., the same conditions that have affected the existing sanitary sewer pipeline since its construction), we conclude that the geotechnical conditions in the project areas are suitable for construction of the replacement project (i.e., there are no fatal flaws). In addition to ongoing, long-term consolidation settlement of the underlying Bay Mud, our geotechnical investigation identified a variety of other conditions in the project areas that will require attention by designers and contractors in order to successfully design and construct the project in a safe and economic manner and to ensure its useful long-term performance. A summary of some of the critical geotechnical conditions described in this report include the following:

- High groundwater and proximity to tidally-influenced sea level in Richardson Bay;
- Old uncompacted areal fills of variable composition and consistency that include highly-porous and permeable non-cohesive granular crushed bedrock materials (i.e., materials subject to flowing behavior where unsupported below groundwater) that will have little to no stand-up time and are capable of storing and transmitting large amounts of groundwater;
- Existing and/or abandoned sanitary sewers and utilities with variable bedding and backfill geometries (vertical or side-sloped), types, and consistencies. Some of the bedding and backfill may be porous, non-cohesive and granular (i.e., materials subject to flowing behavior where unsupported below groundwater) with little to no stand-up time and capable of storing and transmitting large amounts of groundwater;
- Thick (approximately 100 feet) native, soft and compressible Bay Mud that is and will continue to consolidate and cause total and differential settlement (sags) in overlying fills, utilities (including the new sanitary sewer replacement pipeline), pavements and structures;

- The unknown location of old (1940s), wooden, driven piles relative to the planned new sanitary sewer replacement pipeline and related structures;
- Former railroads, pavements and structures dating back to the 1800s;
- Corrosive soils;
- Construction vibrations; and
- Seismic shaking.

Soil and groundwater contamination is a potentially critical condition for the project; however, its study is outside the scope of our work for this geotechnical investigation.

The following sections contain our specific geotechnical conclusions and recommendations for the design, construction and useful long-term performance of the project with respect to these and other geotechnically-related conditions.

3.1 Existing Pipeline Abandonment

All existing pipeline to be replaced and/or abandoned should be completely removed or completely filled with CLSM (see Section II.3.4.4) to prevent open conduits from collecting and/or conducting drainage waters and/or collapsing in the future. Accurate and complete as-built documentation of abandonment work should be kept. Removal and/or in-place abandonment of AC pipe, if any is encountered in the project areas, requires special procedures and handling in accordance with regulations of the Environmental Protection Agency and Bay Area Air Quality Management District.

3.2 Design Groundwater Levels

The depth to groundwater encountered in test borings is summarized in Table I-3, Section I. The shallowest depth to groundwater recorded in project test borings drilled in July 2009 (summertime) was in Boring B-2 where groundwater was measured at a depth of 4 feet below ground surface immediately after drilling and before the boring was grouted. The shallowest depth to groundwater recorded in reference test borings was in Reference Boring RB-8 where groundwater was measured at a depth of 1½ feet in January 1984 (wintertime). Groundwater was also encountered at a depth of 2 feet below ground surface on January 5 and 6, 1977, in reference test borings RB-1, RB-2, and on February 3, 1995, in RB-9. Portions of both the Gate 5 Road and Spinnaker Areas of the project are mapped within FEMA's 100-year flood hazard areas (ABAG, 2009).

3.2.1 Temporary Construction

Assuming a summertime construction period, the contractor should use a design groundwater depth of 4 feet below ground surface for preliminary design of temporary shoring and dewatering systems. However, the project specifications should require that the contractor's final shoring system design and implementation, and the contractor's final dewatering system design and implementation, be based on the actual groundwater depth at the time of construction, including from any perched groundwater encountered above static equilibrium groundwater depths.

3.2.2 Permanent Project Elements

A long-term groundwater level equivalent to the ground surface in the project areas should be used for the design of permanent subsurface project structures (for example, with respect to lateral pressures and buoyancy of the pump station and grease interceptor).

3.3 Temporary Excavations

Temporary excavations consisting of vertical-walled trench excavations for open-cut installation will be required for (1) new sanitary sewer replacement pipeline with invert depths on the order of 4 to 13 feet below ground surface, (2) a grease interceptor with an invert depth on the order of 6 to 8 feet below ground surface, and (3) a new sanitary sewer pump station at an unknown invert depth (see Table I-3, Section I). Based on our test borings and research, project excavations will extend below the groundwater level and will be entirely within trench backfill of the existing sanitary sewer to be replaced and areal fills over Bay Mud as described in Section II.2.1. All excavations will require shoring and dewatering and/or ground improvement. The project specifications should make the contractor solely responsible for the design, installation, performance and removal of all shoring and related items (e.g., dewatering and ground improvement systems where used). The contractor should be required to submit his proposed shoring, dewatering and ground improvement systems to the owner for review prior to their implementation. The submittal should contain alternative and contingent systems that the contractor will be prepared to implement should the initial systems not achieve the minimum performance requirements described herein.

3.3.1 Excavatibility

Project excavations in trench backfill and areal fills as encountered in project test borings (see logs in Appendix B) can generally be made with appropriately-sized conventional excavators. Project excavations through (1) pavements of varying age, thickness and materials (Section II.2.1.1), (2) old abandoned and currently concealed railroads or driven wood piles (Section I.2.4) or (3) where hard debris or rubbish are encountered in areal fill (e.g., see concrete block encountered at a depth of 8½ feet in Reference Test Boring RB-5), if any, may require special excavation equipment and methods (e.g., hoe-rams, jack hammers). Contractors must independently evaluate the excavatibility of the subsurface materials to be encountered during project construction and choose appropriate excavation equipment and methods.

3.3.2 Dewatering

All construction in project excavations should be performed in the dry. Dewatering and/or "water-tight" shoring will be a critical component to successful construction of the project. The groundwater level within open bore holes is above the invert of the planned new sanitary sewer replacement pipeline at 4 of the 6 project test boring locations (i.e., at locations of Borings B-1, B-2, B-4 and B-6). Based on our groundwater findings and the anticipated project excavation depths (summarized at the test boring locations in Table I-3, Section I), dewatering or water-tight shoring should be planned for all excavations greater than 4 feet in depth. The contractor should be made solely responsible for the design, construction, and effects of temporary dewatering systems, and the contractor should be required to submit dewatering plans to the owner for review prior to implementation.

The design of the dewatering systems should be based on the actual groundwater inflow into excavations at the time of construction and the type of shoring used (e.g., interlocking driven sheet piles with adequate toe embedment can reduce or eliminate external dewatering requirements). For short-term excavations (i.e., trench excavations open less than 24 hours) and where the groundwater level is at or below the invert of the planned new sanitary sewer replacement pipeline, a stable trench bottom may be maintained by an internal dewatering system consisting of regularly-spaced, rock-filled sumps excavated below the trench bottom. Submersible pumps within the rock-filled sumps will remove collected groundwater. The spacing and depth of these sumps and the foundation rock between sumps should be such that the trench bottom is relatively dry and stable, and capable of supporting compaction of pipe bedding material.

Where the invert of the planned new sanitary sewer replacement pipeline is below the groundwater level, external dewatering efforts such as dewatering wells, well points, trench drains, or the installation of a water-tight shoring system will be required. Water-tight shoring typically consists of continuous, pre-driven interlocking sheet piles which have been driven with sufficient toe embedment to prevent groundwater flow to and boiling (i.e., piping) in the excavation bottom. The depth of shoring toe embedment will be dependent upon the difference in elevation between the trench bottom and groundwater level (that is, unbalanced hydraulic head).

Even where water-tight shoring is used, we anticipate that limited internal dewatering (i.e., pumping from rock-filled sumps inside the excavation) will be required to remove nuisance water and minor seeps.

Dewatering methods will need to vary within the project areas to account for variations in subsurface conditions, proximity to drainageways, groundwater depth, required excavation depths, and dewatering method limitations related to the grain size of the soils being dewatered. The limitations of various methods of dewatering relative to the particle (grain) size of the water-bearing soils are illustrated on Plate II-5. Grain size distributions for project soils to be dewatered are plotted on Plate C-2 in Section I, Appendix C. Based on a comparison of these plots with Plate II-5, there is a potential for high rates of groundwater inflow into project excavations through the granular areal fills in the project areas.

Collectively, the contractor's project dewatering system(s), together with his project shoring and/or ground improvement systems, are to preserve the undisturbed bearing capacity of the existing subgrade soils at the bottom of excavations and meet all of the following minimum performance requirements:

- Provide stable excavation walls and bottom;
- Provide a reasonably dry base of excavation;
- Filter native soil and prevent loss of ground from dispersion and erosion;
- Prevent piping (boiling) of the excavation bottom;
- Draw down the groundwater level to 3 feet below and beyond the excavation bottom and sidewalls where shoring is not designed to resist hydrostatic pressures;
- Prevent damaging settlement to nearby structures, utilities and/or pipelines;
- Be installed and removed in accordance with governing (e.g., County and State) requirements; and
- Allow for controlled release of groundwater to its static level in a manner that prevents disturbance of the bottom soils and prevents flotation or movement of structure or pipelines.

The project specifications should require that the contractor's dewatering, shoring and ground improvement submittals contain alternative contingent systems, and that the contractor be prepared to implement alternative systems should the initial systems not achieve these minimum performance requirements. Uncontrolled seepage of groundwater through excavation sidewalls or bottom will cause the excavations to be unstable and unsuitable for pipeline and related structural support. Consequently, the contractor should be prepared to locally dewater or modify (e.g., by ground improvement) construction excavations, if and where needed, to provide stable and reasonably dry excavations.

Prolonged dewatering will cause an increase in effective stress on the underlying Bay Mud which will lead to consolidation and area subsidence. Settlement monitoring points should be provided between the locations of dewatering wells and on nearby critical structures, utilities, and pipelines. These settlement monitoring points should be regularly monitored during active dewatering to measure related ground settlement, if any. Modifications to the contractor shoring and dewatering systems should be required if settlements are measured, or if damaging settlements are likely to occur, given anticipated future rates of dewatering and the location of dewatering relative to the existing critical structures, utilities, and pipelines.

3.3.3 Shoring

The contractor should be required to shore all project excavations in accordance with Cal/OSHA regulations. The contractor should be made solely responsible for the selection, design, construction, removal and effects of shoring noting the following:

- Project excavations will be located (1) within backfill of the existing pipeline to be replaced, (2) parallel to and/or across backfill for other existing utilities, and (3) within areal fills all of which will be over soft compressible Bay Mud (see Sections II.2.1.2 and II.2.1.3). Project excavations will therefore encounter various types of fill including granular, non-cohesive materials that will tend to run or ravel when dry or flow when saturated with groundwater (i.e. have little to no stand-up time in unshored vertical excavations). Unsupported vertical excavations in flowing, running or raveling ground will most likely experience excavation wall loss and related undermining of adjacent pavements, utilities, and structures. Therefore, excavations into these types of materials must (1) have water-tight shoring (i.e., continuous interlocked steel sheet piles with toe embedment), (2) be externally dewatered and fully shored (e.g., full excavation face coverage with plywood or steel plate backing of trench boxes or speed shores), and/or (3) improve the ground (e.g., permeation grouting) prior to excavation.

- Active shoring systems (e.g., braced driven sheet piles with toe embedment) are preferred to minimize surface settlement and roadway and adjacent utility damage. Aluminum hydraulic speed shores with full solid sheet backing may only be used for excavations where the external groundwater level has been drawn down to at least 3 feet below the depth of excavation base and where the soils have sufficient stand-up time for its safe and complete installation (i.e., not in running, flowing or fast-raveling soils).

- Passive shoring systems such as trench boxes should only be used for excavations where (1) the external groundwater level has been drawn down to at least 3 feet below the depth of excavation base, (2) excavation occurs from within the box as it is lowered incrementally into place and in step with the deepening excavation (i.e., so as to provide continuous full-face excavation side-wall support), and (3) any gaps between the outside face of the trench box and vertical soil cut is immediately filled with sand or gravel. Driving steel backer plates alongside and below the trench box excavation should be performed to provide additional base stability. Excavation below and prior to trench box shoring installation should not be permitted where soils have insufficient strength and stand-up time (e.g., in flowing, running or fast-raveling ground conditions) to safely and completely install the trench box.

- Where shoring systems are not used in conjunction with external dewatering systems designed to draw down the groundwater level a minimum of 3 feet below the excavation bottom and beyond the excavation sidewalls, the shoring systems must be designed to resist hydrostatic pressures and to extend below the base of the excavation to sufficient depths to (1) provide lateral stability at the base of the shoring system and (2) to prevent heave and/or piping (boiling) through the base of the excavation. The shoring designer should determine the minimum required toe embedment based on the depth of the excavation, the specific shoring and dewatering systems used, and the soil and groundwater conditions encountered in the field at the time of construction. For the purposes of sheet pile design, the average buoyant unit weight of area fill and Bay Mud soils in the project areas, to depths of the invert of the planned new sanitary sewer replacement pipeline, can be taken as 70 pcf and 40 pcf, respectively, with a critical hydraulic gradient of 1.0 and 0.8, respectively. We recommend that a minimum safety factor of 2.0 be used for design of project shoring and dewatering systems against base failure.

- Shoring systems that do not provide positive support to excavation walls (i.e., passive shoring like trench boxes that allow inward movement of the trench wall) may cause surface settlement and related damage to nearby roadways, utilities and structures. A summary of the potential surface settlement of passively-shored excavations is provided in Table II-3. Unrestricted flowing, running, or raveling ground conditions will result in surface settlements significantly greater than that indicated in Table II-3.

Table II-3 - Potential Surface Settlement of Passively-Shored Excavations

Soil Type	Surface Settlement (% of Excavation Depth)	Lateral Zone of Disturbance (Multiples of Excavation Depth)
Sand	0.5%H	H
Soft to Medium Stiff Clay	1-2%H	3-4H
Stiff Clay	<1%H	2H

[1] From Suprenant and Basham (1993).

- Preliminary design of braced and cantilever shoring may be based on the preliminary lateral earth shoring pressure diagrams provided on Plates II-6 and II-7, respectively. These diagrams represent soil conditions encountered in project test borings. Final earth pressures and pressure diagrams for the contractor’s design and implementation of individual shoring systems will be dependent on (1) the actual soil and groundwater conditions encountered during construction, (2) the contractor’s shoring type, design, and installation method, and (3) surcharge pressures, including those from stockpiling, construction equipment, vehicle traffic, and existing structures within a 1.5H:1V plane projected upward from the excavation bottom (see Plate II-8 for minimum surcharge pressures).

A professional Structural or Civil Engineer licensed in the State of California and with experience in the design of shoring systems should design, sign, and stamp the contractor’s proposed shoring plans. The plans should be required to be submitted to the owner for review prior to construction. The shoring plans should indicate interrelationships with dewatering and ground improvement systems. The shoring plans should contain alternative contingent systems, and the contractor should be prepared to implement these alternative systems should the initial plans not achieve the following minimum performance requirements:

- Protect personnel that enter the excavation.
- Comply with all governing regulations pertaining to excavation safety (e.g., the most current edition of Cal/OSHA Construction Safety Orders, Article 6).
- Be compatible with the surface and subsurface soil and groundwater conditions encountered in project test borings, and/or mapped in the project areas, and resist lateral earth pressures and hydrostatic pressures.
- Protect existing utilities, pavements, and structures.
- Excavation and installation of shoring must occur in a manner and sequence that does not damage existing structures, pavements, and utilities including through settlement, heave, or vibrations.
- Prevent caving (i.e., raveling, running, or flowing) or lateral movement of excavation walls and associated loss of adjacent ground and adjacent ground surface settlement, even when subjected to construction vibrations.
- Provide stable excavation walls and bottom (e.g., prevent bottom heave).
- Allow for removal or abandonment of shoring in a manner and sequence that (1) is in step with the backfilling sequence (i.e., shoring should not be removed ahead of backfilling), (2) does not cause disturbance (i.e., loosening) of pipe bedding and pipe embedment material, and (3) does not damage the existing pipeline or structures, pavements, and utilities including through settlement, heave, or vibrations (contractor to address removal/abandonment concerns specific to the type of shoring proposed in the shoring submittal). Any void space created by shoring removal should be completely filled with CLSM (see Section II.3.4.4) or approved equivalent.
- Resist lateral earth pressures including those from lateral loads from vehicular traffic, construction equipment and spoils, and hydrostatic pressures, if and where applicable.

Soil conditions can vary widely over short lateral and vertical distances in the project areas; therefore, project excavations should be continually monitored and documented by the contractor's Cal/OSHA approved "competent person", and the contractor should be prepared to make changes and modifications to shoring requirements in response to these changes and consistent with governing regulations (e.g., the most current edition of Cal/OSHA Construction Safety Orders) pertaining to excavation safety. Cal/OSHA soil classifications include the following:

Stable Rock: Natural solid mineral matter that can be excavated with vertical sides and remain intact when exposed.

Type A Soil: Excludes material that is part of a sloped or layered system dipping into the excavation at a slope ≥ 4H:1V, but includes cohesive soil with an unconfined compressive strength of ≥ 1.5 tsf that is:

- Not fissured,
- Not subject to vibration from heavy traffic, pile driving, or similar effects, and
- Not been previously disturbed.

Type B Soil: Excludes material that is part of a sloped or layered system dipping into the excavation at a slope ≥ 4H:1V, but includes the following:

- Cohesive soil with unconfined compressive strength between 0.5 and 1.5 tsf,
- Angular gravel and silt,
- Previously disturbed soil, except that is otherwise classified as Type C,
- Soil fissured or subject to vibration and not otherwise Type C soil, or
- Dry rock that is not stable.

Type C Soil: Excludes material that is part of a sloped or layered system dipping into the excavation at a slope ≥ 4H:1V, but includes the following:

- Cohesive or disturbed soils with unconfined compressive strength ≤ 0.5 tsf,

- Sand and non-angular gravel,
- Submerged soil or soil from which water is freely seeping, or
- Submerged rock that is not stable.

The subsurface soils encountered in test borings and mapped in the project areas were consistent with a Cal/OSHA soil classification Type C.

The contractor should be required to provide special shoring design for owner review in cases where excavations will be in close proximity (below an imaginary plane projected downward at an inclination of 1.5H:1V from the nearest foundation or utility edge) to critical structures or utilities in order to minimize potential excavation-related damage. Special shoring should account for surcharge pressures and should be designed to maintain positive lateral support for adjacent structures and utilities. Areas requiring special shoring should also receive preconstruction condition surveys to establish a baseline against which any claimed third-party damages can be compared.

3.3.4 Ground Improvement

Granular, non-cohesive areal fills and trench backfill soils capable of raveling, running, or flowing ground behavior (see definitions on Plate A-1, Appendix A) will be encountered in project excavations. These types of soils will have little to no stand-up time in unshored vertical excavations and may need to be stabilized by ground improvement (e.g., grouting) where not completely and continuously shored and/or dewatered. Inadequately stabilized and/or shored excavations will allow existing nearby utilities, structures and roadways to be damaged by loss of support, undermining, or vibration-induced settlement. Stabilization of these types of soils can be accomplished through grout stabilization (e.g., displacement, permeation or jet grouting) by a specialized and experienced grouting contractor. The contractor should be made solely responsible for design and implementation of grout stabilization systems, and should require that the proposed systems be submitted to the owner for review prior to implementation.

3.4 Foundation, Pipe Embedment and Trench Backfill Materials

Foundation, Pipe Embedment and Trench Backfill Materials should conform to the requirements of this section where not exceeded by the City or governing agency or pipe manufacturer requirements, and so long as they will not cause damage or deformation to the pipeline or its coatings, if any. Refer to Plate II-9 for trench backfill details.

The recommendations provided in the following sections will need to be modified if and where the planned new sanitary sewer replacement pipeline or related project structures are found to be over old driven wood piles described in Section I.2.4. Whether or not portions of the planned new sanitary sewer replacement pipeline or related project structures are over old driven wood piles may not be known unless encountered in excavations during project construction. Modifications to the recommendations provided in the following sections will need to be addressed on a case by case basis at that time depending on the number, spacing and location of the piles encountered. If and where driven wood piles are encountered, we anticipate that the modifications to the construction of the planned new sanitary sewer replacement will include the following:

- Removing the top of the wood pile to a depth of at least 5 feet below the invert of the planned new sanitary sewer replacement pipeline, and
- Backfilling the removed pile space, up to the bottom of planned Foundation Material, with lightly-compacted trench excavation material so as to not create a hard point below the pipeline (such a hard point could eventually result in a hog in the overlying pipeline).

3.4.1 Foundation Material

Where the trench bottom at the planned excavated grade is soft, loose, or disturbed by construction activity, or otherwise unstable (e.g., pumping subgrade under foot load, boiling, etc.), overexcavation should be required until either (1) a firm material is reached or (2) a firm base can be created by the placement of a layer of Foundation Material. Based on the loose condition of areal fills encountered at the invert depth of the planned new sanitary sewer replacement pipelines in project test borings (see boring logs in Appendix B), we anticipate that a layer of Foundation Material will be required for all excavations. The Foundation Material layer should consist of clean, natural, durable 1½-inch crushed (i.e., angular) rock that is graded within the requirements provided in Table II-4 and wrapped with a 12-inch minimum overlap of geotextile filter fabric. The thickness of the Foundation Material layer should not be less than 12 inches thick.

<u>Table II-4</u> - 1½-inch Crushed Rock

Sieve Size	Percent Passing
2"	100
1½"	90 – 100
3/4"	5 – 30
3/8"	5 – 20
No. 200	0 – 4

The geotextile fabric should be a non-woven material consisting of polyester, nylon or polypropylene filaments formed into a stable network and conforming to properties in Table II-5. The fabric should be permeable, inert to commonly encountered chemicals, rot-proof, resistant to ultra-violet light, and not act as a wicking agent. Mirafi 160N, Amoco Propex 4506, or similar geotextile filter fabrics which meet the criteria given in Table II-5 are acceptable.

<u>Table II-5</u> - Geotextile Fabric

Property	Test Value	ASTM Test Method
Weight	5.4 oz./yd.^{2} (min.)	D5261
Grab tensile strength	150 lb. (min.)	D4632
Elongation at break	50% (max.)	D4632
Puncture strength	80 lb. (min.)	D4833
Burst strength	300 psi (min.)	D3786
Apparent Opening Size	#70 (max.)	D4751
Permittivity	1.0 sec^{-1} (min.)	D4491
UV Resistance	70% (min.)	D4355

3.4.2 Pipe Embedment Material

Pipe Embedment Material should envelop the pipeline to the dimensions illustrated on Plate II-9. Pipe Embedment Material should consist of either (1) clean, durable, natural, crushed (i.e., angular) rock meeting the gradational and quality requirements for Caltrans Class 2 Aggregate Base (Class 2AB) provided in Table II-6 and compacted as recommended in Section II.3.4.6, or

(2) CLSM as specified in Section II.3.4.4, particularly where compaction is not possible due to working space constraints.

Table II-6 - Class 2AB

Sieve Size	Percent Passing	
1"	100	
3/4"	90-100	
No. 4	35-60	
No. 30	10-30	
No. 200	2-9	
Test	**California Method No.**	**Requirement**
Resistance (R-Value)	301	78 min.
Sand Equivalent	217	22 min.

3.4.3 Trench Backfill Material

In paved areas, or areas to receive improvements, trench excavations should be backfilled above the pipe embedment zone with (1) Class 2AB (see Table II-6) and compacted as recommended in Section II.3.4.6, or (2) CLSM as specified in Section II.3.4.4.

3.4.4 Controlled Low Strength Material (CLSM)

Controlled low strength material (CLSM) should consist of the following:

- A hand-excavatable mixture of cement, pozzolan, coarse and fine aggregate, and water that has been mixed in accordance with ASTM C94 and is in a flowable state during placement;
- A maximum in-place density of 150 pcf;
- A minimum 28-day compressive strength of no less than 50 psi and a maximum 28-day compressive strength of no more than 150 psi;
- A minimum 12-hour compressive strength of no less than 20 psi;
- Physiochemical properties that do not damage the pipeline; and
- Placed in appropriate lifts or with methods to prevent movement of the pipe, including by flotation.

Placement of backfill on top of CLSM should not be allowed until the CLSM passes the ball drop test of ASTM D6024.

Where CLSM is used as pipeline embedment material, the pipeline should be elevated off of the trench bottom or foundation material using cradles, sandbags, or other approved supports prior to CLSM placement. Spacing of these supports is dependent on the pipeline material, diameter and structural properties, as well as the permissible amount of sagging which can be allowed between supports.

Pipelines backfilled using CLSM have a tendency to float. This tendency can be mitigated by the use of pipe anchors/weights and/or sequential backfilling (where the CLSM is poured in stages, and allowed to set in between stages). For sequential backfilling, the height to which the CLSM can be initially poured is a function of the buoyant forces imposed on the pipeline, and the amount of resistance provided by the pipeline anchoring/weighting system (if used). Sequential backfilling will require the trench excavation to remain open for a longer period of time, which may not be practical where the project alignment is within the traveled portion of a roadway.

3.4.5 Compaction

The project specifications should make the contractor solely responsible for excavation backfill compaction, and solely responsible to protect the new sanitary sewer replacement pipeline from damage at all times, including during placement and compaction of Pipeline Embedment and Trench Backfill Materials. Project excavations should be shored so that vibrations from construction activities (e.g., compaction equipment) will not cause raveling or running from the excavation sidewalls. Additionally, all water that accumulates in the bottom of the excavation should be removed so that project work can be done in the dry. No jetting of backfill should be allowed. The following recommendations assume that the planned pipeline can support mechanical compaction of Pipe Embedment Material and/or Trench Backfill Material as recommended herein. Where this is not the case, then the Pipe Embedment Material and/or Trench Backfill Material should consist of CLSM (see Section II.3.4.4). All references to relative compaction are in accordance with laboratory maximum density/optimum moisture content by ASTM D1557.

Foundation Material should be densified in place (using a Vibra-plate compactor or equal) to provide a stable trench bottom capable of supporting mechanical compaction of the Pipe Embedment Material. Pipe Embedment Material should be compacted to a minimum of 90% relative compaction at a moisture content at or above optimum. The Pipe Embedment Material at the bottom of the pipe (i.e., pipe subgrade) should be compacted to a smooth, uniform plane to match the desired pipe slope. Where applicable, flange or bell holes should be excavated out at each pipe joint to ensure uniform pipe support to proper line and grade over the full length of each pipe segment.

After the pipe is laid in the trench, Pipe Embedment Material should be uniformly placed in maximum 8-inch thick lifts on each side of the pipe and hand-shovel sliced around the pipe haunches to support the sides of the pipe and to prevent pipe displacement, and then compacted to 90% relative compaction at or above optimum moisture condition. Compacting and testing Class 2AB below the pipe springline will be dependent on the trench width selected for installation of the pipeline, shoring and dewatering systems. It may not be practical to test the Class 2AB below the springline with less than 12 inches of side clearance between the pipe and trench/shoring wall. Above the springline of the pipe, the Pipe Embedment Material should be placed in maximum 8-inch thick loose lifts and compacted to a minimum of 90% relative compaction at or above optimum moisture content. Removal of shoring must not cause disturbance (i.e., loosening) of the compacted pipe embedment material.

Trench Backfill Material should be placed in maximum loose lifts of 8 inches above the Pipe Embedment Material. Trench Backfill Material should be compacted to a minimum of 90% relative compaction to within 3 feet of the pavement subgrade and to a minimum of 95% relative compaction within the upper 3 feet of backfill.

Inadequate compaction of utility trench backfill (i.e., less than that recommended herein) may cause excessive settlements resulting in damage to the pavement and other surface improvements.

3.4.6 Trench Dams

Trench dams, like that illustrated on Plate II-10, can be incorporated into the project design to minimize lateral subsurface flow of groundwater within permeable Foundation Material, Pipe Embedment Material, and Trench Backfill (e.g., to isolate any areas of known groundwater contamination). Trench dams will also minimize the amount of dewatering that would otherwise be required to access the pipeline during future maintenance and point repair excavations.

3.5 External Pipeline Loads

The type of pipe to be used for the planned new sanitary sewer replacement pipeline is not known to us at this time. Dead loads from soil on rigid and flexible pipeline are described below. Additionally, design criteria for live loads on the pipeline from vehicular traffic (H20 loading) are provided on Plate II-11. The total unit weight of CLSM (see Section II.3.4.4) or compacted Class 2AB (see Table II-6) may be taken as 150 pcf.

3.5.1 Rigid Pipe

Design criteria for dead loads on rigid pipe under trench conditions are presented on Plate II-12.

3.5.2 Flexible Pipe

Dead loads due to backfill soil overburden on a flexible pipeline assuming trench conditions can be estimated using the following Prism Method based formula (Moser, 2001):

$$W = D \gamma H$$

where: W = vertical soil load on a flexible pipeline due to trench backfill/overlying soil (pounds/linear foot),

D = pipe outside diameter (feet),

γ = unit weight of trench backfill (pcf), and

H = height of trench backfill above the pipeline (feet).

3.6 Composite Modulus of Soil Reaction

The composite modulus of soil reaction (E'_c) is useful for estimating the passive soil resistance that will develop upon vertical loading of flexible pipelines. E'_c is a function of the soil modulus of the pipe zone material (E'_{pz}), the soil modulus of the trench wall material (E'_{tw}), trench width, pipeline depth of cover, and pipeline diameter (see Plate II-13). E'_{pz} and E'_{tw} are in turn functions of the strength of each material. Where the new sanitary sewer replacement pipeline is bedded in well-compacted Class 2AB as recommended in Section II.3.4.2, E'_{pz} will be constant at approximately 1,500 psi. It is imperative that properly-compacted pipe zone material not be disturbed or loosened by shoring removal in order to maintain this 1,500 psi E'_{pz} value.

E'_{tw} varies in proportion to the consistency/density of the soils forming the trench walls. The soils encountered in project test borings in the project areas at the invert depth of the new sanitary sewer replacement pipeline were loose to medium stiff (average SPT blow count of N = 5, for 8 project tests). Typical (1) E′tw values for a range of soil consistency/density, and (2) corresponding E′tw:E′pz ratios are provided in Table II-7.

Table II-7 - E′c Input Values

E'_{tw} Value[1]		$E'_{tw} : E'_{pz}$ Ratio (E'_{pz} = 1,500 psi)
Soft soil (N=2)	125 psi	0.08
Medium stiff or loose soil (N<8)	250 psi	0.17

[1]N = standard penetration blow count.

Using the chart provided on Plate II-13 and based on an appropriate E′tw: E′pz ratio, the soil support combining factor Sc can be determined for the actual trench width to pipeline diameter ratio used in design. The Sc factor can then be used to calculate E′c based on the formula

E′c=ScE′pz. For example, assuming E′pz = 1,500 psi and E′tw= 250 psi, then E′tw:E′pz = 0.17. From Plate II-13, and assuming a B:D ratio of 1.5 (the actual B:D ratio to be used is not known to us at this time), then a E′tw :E′pz = 0.17 corresponds to a Sc = 0.25. Completing the equation for E′c=ScE′pz gives E′c= 0.25(1,500) = 375 psi.

3.7 Thrust Blocks

Thrust forces from internal pressure within the 4-inch force main portion of new sanitary sewer replacement pipeline may be resisted by thrust blocks. Project plans (West Yost Associates, 2009) show the force main to be an approximately 593-foot long straight segment between existing manholes that will connect to gravity portions of the new sanitary sewer replacement pipeline. Thrust block capacity is a function of soil type, depth below ground surface, allowable deflection and direction of force application (e.g., upward vs. downward vertical component of thrust). Thrust block design should occur in the following three steps:

- Preliminary design based on minimum depths, anticipated soil and bedrock type and presumptive allowable horizontal soil and bedrock bearing capacity;
- Plan design based on the geotechnical engineer's review of depth of embedment and direction of thrust application; and
- Final design with field adjustments during construction based on actual in-field subsurface conditions (e.g., adjustment for the presence of adjacent utility trenches, perched groundwater, localized changes in soil or bedrock condition, etc.).

For purposes of preliminary design, thrust blocks may be sized using a presumptive allowable soil bearing capacity of 400 psf for pipeline thrust blocks in loose areal fill as encountered in the project test borings near the force main (see logs of Borings B-5 and B-6 in Appendix B, Section I). This presumptive allowable soil thrust block bearing capacity is based on horizontal or downward thrusting only (do not use for upward thrust) using a thrust block having a minimum width of 12 inches and a minimum height of 24 inches. The maximum height of the thrust block must be less than one-half the depth from the ground surface to the base of the thrust block. Based on this presumptive thrust block design, the thrust block deflection will be limited to less than ½ inch. Final thrust block capacity should be evaluated in the field during construction prior to pouring concrete and should be based on final thrust block depth, configuration, and the strength and safe bearing capacity of the exposed soils.

3.8 Permanent Subsurface Structures

Permanent subsurface structures planned for the project consist of a new manhole at Station 20+77, a new grease interceptor adjacent to this new manhole, and a new pump station at Station 18+89 (at an existing manhole); all near the east end of the Spinnaker Area of the project. Except at Station 20+77, the project plans (West Yost Associates, 2009) show the new sanitary sewer replacement pipeline to flow through existing manholes. The planned invert of the new manhole is about El. 4 (about 4 feet deep) and the planned invert of the new grease interceptor is about El. 0 (between 6 to 8 feet deep). Details of the new sanitary sewer pump station at Station 18+89 (plan and profile dimensions) are not known to us at this time. The invert depth of the new manhole and new grease interceptor corresponds in nearby project test Boring B-5 to loose to soft to medium stiff areal fill overlying soft compressible Bay Mud.

3.8.1 Foundations

Foundations for the planned new permanent subsurface structures that are underlain by areal fill over soft Bay Mud may be designed based on load compensation (i.e., applied structural loading equal to or less than the weight of the soil removed to accommodate the structure) using an the allowable bearing capacity for mats of 500 psf for an 8-foot deep structure.

This loading can be increased by one-third where needed to resist transient loading on the structures (e.g., seismic forces). The new structures contribution to total long-term underlying Bay Mud consolidation settlement of the area will be negligible since their load compensation design constitutes no new loads. However, compensated structure excavations in Bay Muds will be subject to immediate recompression settlements of as much as 1 inch, which should occur quickly upon load application. The maximum differential undisturbed soil recompression across the base of the structures should be less than ½ inch.

These foundation recommendations are based upon an undisturbed base of excavation on which the structures will bear (i.e., dependent upon the performance of the contractor). For example, excavations that are not fully dewatered ahead of time or where groundwater has not been completely cut off (e.g., interlocking sheet pile with sufficient toe embedment) prior to excavating, will be subject to piping and boiling of the base of excavation and to consequent amounts of structure settlement significantly greater than that anticipated by solely undisturbed soil recompression effects.

Where the subgrade for permanent subsurface structures is found to be soft, loose, disturbed by construction activity, or otherwise unstable (e.g., pumping subgrade under foot load, boiling, etc.), we recommend that the subgrade be overexcavated to such a depth that a firm, stable base can be created by the placement of a layer of Foundation Material (see Section II.3.4.1). The thickness of the Foundation Material should not be less than 12 inches.

Subsurface structures including pre-cast elements or cast-in-place elements should be placed on a 6-inch layer of compacted Class 2AB (see Table II-6) overlying the geotextile-fabric-wrapped pipeline foundation material or undisturbed subgrade soils. This layer of Class 2AB should be compacted to a minimum of 90% relative compaction at a moisture content at or above optimum. The structure foundation concrete can then be poured directly on top of the compacted Class 2AB layer.

3.8.2 Structure Backfill Material and Compaction

Structure backfill material should consist of Class 2AB (see Table II-6) or CLSM (see Section II.3.4.4). Class 2AB structure backfill should be placed in maximum 8-inch thick loose lifts and compacted to a minimum of 90% relative compaction (ASTM D1557) at a moisture content at or near optimum to within 30 inches of the pavement subgrade. Within the upper 30 inches of backfill, Class 2AB structure backfill material should be compacted to a minimum of 95% relative compaction (ASTM D1557) at a moisture content at or near optimum. The project specifications should clearly state that at all times during the construction of project structures and placement/compaction of structure backfill material, it is the contractor's responsibility to protect the project structures from damage (e.g., overstressing the structures with heavy equipment, etc.).

3.8.3 Resistance to Hydrostatic Forces

All permanent subsurface structures should be designed to resist buoyant uplift and lateral hydrostatic forces assuming a long-term groundwater level at the ground surface. Buoyant uplift can be resisted by the dead weight of the structure, friction between the exterior structure walls and the backfill soils, and/or by the weight of backfill soils above an exterior perimeter lip added to the foundation mat. In all cases, a minimum factor of safety of 1.5 should be used for design against hydrostatic uplift. Frictional forces that will resist buoyant uplift of subsurface structures may be calculated using the at-rest earth pressures in Table II-8.

A sketch illustrating design parameters for hydrostatic uplift resistance is presented on Plate II-14. Friction between subsurface structure walls and Class 2AB structure backfill may be calculated using the at-rest earth pressures on Table II-8 and an ultimate friction factor of 0.35. If an exterior perimeter lip is added to the foundation mat, hydrostatic uplift can be resisted by the weight of backfill soils within the prism shown on Plate II-14. A buoyant unit weight of 80 pcf can be used for Class 2AB.

3.9 Lateral Earth Pressures

Lateral earth pressures will be imposed on all subsurface structures for the project. Subsurface structures for the project are not free to deflect and therefore should be designed to resist at-rest earth pressures. Lateral earth pressures provided in Table II-8, expressed as equivalent fluid densities, are for permanent below-ground structures based on the composition and consistency of planned structural backfill, and the soils encountered in project test borings (see logs in Appendix B).

Table II-8 - Lateral Earth Pressures[1]

Ultimate Static Lateral Earth Pressures[1] in Project Areas Expressed as Equivalent Fluid Density (psf/ft in a triangular distribution)		
Soil Type & Condition	**At-Rest[2]**	**Passive[3]**
Areal Fill	50	125
Bay Mud	35	60
Class 2AB Structure Backfill (Table II-6)	35	300

1All values are based on buoyant unit weights with design groundwater at the ground surface. Appropriate safety factors should be applied. Assumes structures less than 15 feet deep. See text for additional applicable pressures.
[2]Add hydrostatic component (+62.4 pcf).
[3]For passive pressures, a safety factor of at least 2.0 should be applied to avoid the lateral movement of the structure that would be necessary in order to reach full ultimate passive soil strength mobilization.

The following modifications to design lateral earth pressures should be made to the static lateral earth pressures provided in Table No. II-8, where applicable:

- Lateral surcharge from vehicles (Plate II-8),
- Lateral surcharge from adjacent fills or structures where an imaginary 1.5H:1V plane projected downward from an existing or planned new structure projects above or intersects the side of the planned new adjacent structure,
- Dynamic pressures (Pe) from seismic shaking. A dynamic earth pressure of Pe = 14 x H, expressed as pounds per square foot, should be applied as a rectangular distribution over a depth of H (where H = depth of wall embedment below grade in feet) assuming peak ground accelerations of 0.5g from Plate II-3 and a buoyant unit weight of 90 pcf (buoyant unit weight of Class 2AB used as structural backfill). The resultant should be applied at a distance of 0.6H from the bottom of the structure. The design ground surface acceleration has been factored for an acceleration taken as 80% of the peak (i.e., 0.5g x 0.80 = 0.4g).

Overcompaction of structure backfill is to be avoided because increasing the compactive effort can result in damaging lateral pressures that are higher than those provided in Table II-8. In addition to lateral earth pressures, an ultimate coefficient of sliding friction value of 0.35 between concrete structures and compacted Class 2AB (see Table II-6) can be used in calculations for lateral force resistance.

3.10 Settlement

From a practical viewpoint, and with the exception of ongoing, long-term, area-wide Bay Mud consolidation settlement (see Section II.2.1.4), the amount of settlement caused by the new sanitary sewer replacement pipelines and related project structures (e.g., the grease interceptor) will be minimal since no new loads will be applied. Localized settlement of the pipelines and project structures will depend mostly on the condition of the excavation bottom (i.e., mostly determined by the contractor's performance in achieving the minimum recommendations for trench bottom stability provided in this report). Therefore, it is imperative that stable excavation bottoms are maintained at all times and that loose, disturbed or otherwise softened soils are not allowed in excavation bottoms. Backfill loading upon such soils can produce random settlements creating pipeline sags greater than 1 inch that can be abrupt and localized over short sections of pipeline.

3.10.1 Recompression Settlement

Project excavations will be backfilled to their original grade and compacted backfill will exert no significant additional loads onto the underlying undisturbed soil deposits. Therefore, elastic deformation (i.e., recompression) of the native materials induced by backfill placement and compaction should occur quickly upon load application. For example, the maximum recompression of undisturbed trench bottoms on the order of 10 to 15 feet deep in loose or medium stiff soils should be less than ½ inch and should occur upon backfilling. The maximum differential recompression between differing soil consistencies/densities along the pipeline should also be less than ½ inch.

3.10.2 Backfill Compression

Excavation backfill placed within excavations will compress (settle) by self-weight even when well compacted. We estimate settlement of Foundation Material, Pipeline Embedment Material, Trench Backfill Material and Structure Backfill Material compacted as recommended in this report, to be less than 0.2 to 0.4 percent of their vertical thickness. Pavement sections overlying the sewer pipeline will reflect this long-term backfill settlement.

3.10.3 Vibration-Induced Settlement

Settlements damaging to the sanitary sewer replacement pipeline, related project structures and to adjacent improvements (e.g., existing underground utilities and street pavement) can occur as a result of soil densification upon vibration. Vibration-induced settlements occur as a result of localized liquefaction and densification of saturated, uniformly-graded, non-cohesive soils (e.g., nonplastic silts and sands). The loose granular areal fills, as encountered in project test borings, are particularly sensitive to vibration-induced settlements. Case histories cited by Lacy & Gould (1985) indicate several inches of pipeline settlement upon vibratory sheet pile extraction in these soil types. Therefore, all shoring extending into granular, non-cohesive areal fills should be installed and extracted with caution relative to the generation of vibrations; noting that static or "vibration less" shoring installation and removal may be required. The project specifications should require that shoring removal be performed in a manner that does not cause settlement of the new sanitary sewer replacement pipeline or any nearby surface or subsurface structure. The new sanitary sewer replacement pipeline and related project structures (grease interceptor, force main pump station, etc.) should be monitored for settlement when shoring is installed and removed. If settlement is observed during shoring installation or extraction, the contractor should be required to immediately stop and revise his methods of installation or extraction

3.11 <u>Construction Vibrations</u>

The planned project will be constructed in areal fills and existing utility and pipeline trench backfill over Bay Mud that will transmit construction vibrations to existing nearby surface (e.g., existing residential and commercial buildings) and subsurface structures (including utilities and pipelines). Therefore, the type and operation of equipment to be used during project construction should be selected by the contractor to limit construction vibrations (a function of frequency and peak particle velocity) to levels that will not damage existing surface structures and improvements, or existing subsurface structures including utilities and pipelines.

A commonly-accepted damage threshold criteria for high frequency peak particle velocity vibrations at existing surface structures and improvements is on the order of 1.0 to 2.0 inches per second (USBM RI-8507). High frequency peak particle velocities above these values can cause cosmetic damage to structures (e.g., cracking of plaster and drywall). Typical attenuation curves for vibratory pile driving indicate peak particle velocities are generally less than 1.0 inch per second at distances greater than 20 feet. Pile driving into obstructions or through coarse granular materials (e.g., gravelly fills, granular pipe embedment material and utility trench backfill) may generate higher than typical peak particle velocity vibrations with greater attenuation distances.

Construction vibrations should be monitored and documented by qualified technicians with approved vibration measuring equipment (seismographs) located at the residential and/or commercial structure nearest the site of actual ongoing construction. Vibration levels during construction should not exceed a 1.0 inch per second peak particle velocity. Vibration levels exceeding this value within 25 feet of the source or at nearby surface structures, whichever is closer, will require modification of the contractor’s construction procedures to reduce vibration levels. Photographic precondition surveys of the residential and commercial structures located adjacent to the planned pipeline alignment should be performed to establish baseline conditions prior to project construction and to aid assessing construction damage claims, if any.

3.12 Roadway Pavements

Roadway pavements in, and which provide access to, the project areas will most likely be damaged by heavy construction traffic loads that are typically required for construction of this type of project. Therefore, repair of pavement damaged by construction traffic should be included as a bid item for contractors in the project documents. Pre- and post-construction pavement condition surveys should be performed prior to the start of construction in order to document baseline conditions. Pavement section replacement or repair should at least match the existing section. Additionally, pavement section replacement or repair should be designed for appropriate traffic indices and meet the requirements of local jurisdictions.

3.13 Seismic Design

The hazard of fault rupture in the project areas is extremely low to nil based on the absence of active faults (see Section II.2.3). The hazard of ground shaking in the project areas is high (see Section II.2.4). The effects of ground shaking on the project may be mitigated by design and construction detailing in accordance with the foundation and seismic provisions of the 2006 International Building Code (IBC)/2007 California Building Code (CBC) including the parameters provided in Table II-9.

Table II-9 - 2006 IBC/2007 CBC Seismic Site Categorization and Design Coefficients

Categorization/Coefficient	Design Values
0.2s Spectral Response Accel. (S_S) for Site Class B (Figure 1613.5(3))	1.5g
1.0s Spectral Response Accel. (S_1) for Site Class B (Figure 1613.5(4))	0.68g
Site Class (Table 1613.5.2)	F
Long-period Transition Period, T_L (Figure 22-6)[1]	12

[1] From ASCE/SEI 7-05 (2006).

The hazard of liquefaction in the project areas is high based on the shallow depth to groundwater and the presence of silts and loose sands in the areal fills located over soft compressible Bay Mud (see Section II.2.5). If liquefaction were to occur in the project areas, the manifestation of liquefaction would be settlement of the ground surface and localized settlement or buoyant floatation of pipeline and subsurface structures. Liquefaction in the project areas could also cause lateral spreading of the ground down gentle slopes into Richardson Bay. Liquefaction in the project areas could also cause lateral spreading of the ground down gentle slopes into Richardson Bay. For critical life-line facilities, mitigation should be undertaken to address the potential for soil liquefaction. However, the most common approach for small-diameter sanitary sewer pipelines and related subsurface structures is to repair any damage caused by liquefaction after the fact, if they occur.

3.14 Corrosion Design

The results of corrosion tests (i.e., resistivity, redox, pH, sulfates and chlorides) on samples of areal fill soil taken from the project areas at depths of 8 to 10½ and 7 to 9½ feet below ground surface in Borings B-3 and B-5, respectively, are presented on Plate C-5 in Section I, Appendix C. These results should be incorporated into design of project elements (e.g., new structures, pipelines, valves, etc.), noting that portions of the existing sanitary sewer pipeline to be replaced by this project is severely corroded and that Bay Mud is widely known to be highly corrosive to concrete and steel.

4.0 ADDITIONAL SERVICES AND LIMITATIONS

We recommend that DCM/GeoEngineers be given the opportunity to provide the following additional services through the completion of project construction:

- Review of final plans and specifications prior to bid for conformance with geotechnical conditions and recommendations;
- Review of contractor submittals (e.g., shoring, dewatering, ground improvement, etc.) for conformance with geotechnical findings described herein;
- Review and response to contractor requests for information that relate to geotechnical issues; and
- Periodic construction observations during excavations to verify conformance of exposed surface conditions with the findings of this report.

We have prepared this report for the exclusive use of the City of Sausalito, West Yost Associates, and their authorized agents for the Sausalito Priority 1 Sewer Replacement Project in Sausalito, California. Field work for this geotechnical engineering investigation report was planned and completed based on project information provided to us at the time of our subsurface investigation. This geotechnical engineering investigation report was formulated based on findings from our field work and the project information provided to us by the time this report was prepared.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, expressed or implied, should be understood. During the course of our investigation, we reported limited information regarding soil corrosivity and soil and groundwater contamination in the project areas. Studies of, and design recommendations related to soil corrosivity and soil and groundwater contamination in the project areas, and the mitigation thereof, is not part of our scope of services for this geotechnical investigation.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by DCM/GeoEngineers, Inc. and will serve as the official document of record.

Please refer to the appendix titled "Report Limitations and Guidelines for Use" (Appendix E) for additional information pertaining to use of this report.

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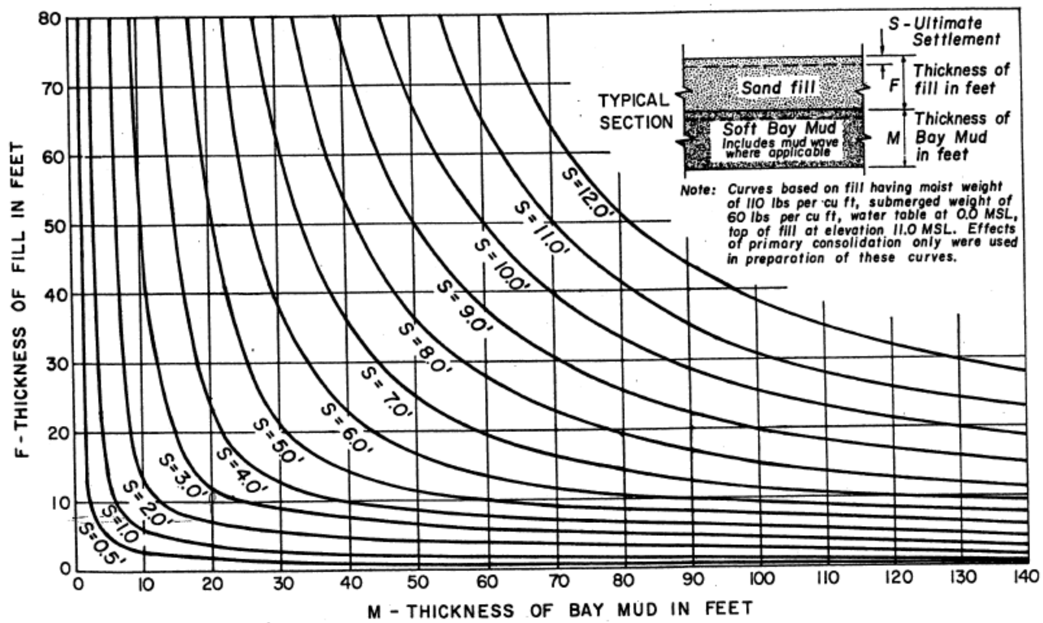
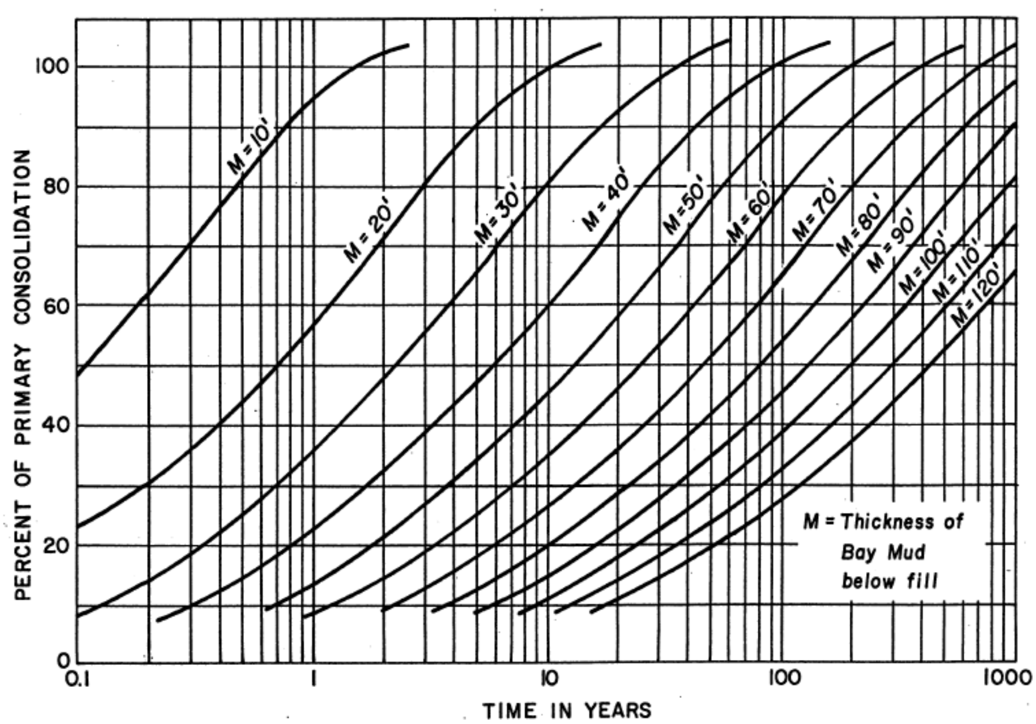
ULTIMATE AMOUNT OF SETTLEMENT OF FILLS ACCORDING TO THICKNESS OF FILL AND THICKNESS OF UNDERLYING BAY MUD.



Modified from CDMG (1969)

PERCENT SETTLEMENT OF FILLS OVER TIME ACCORDING TO THICKNESS OF MUD.



Modified from CDMG (1969)

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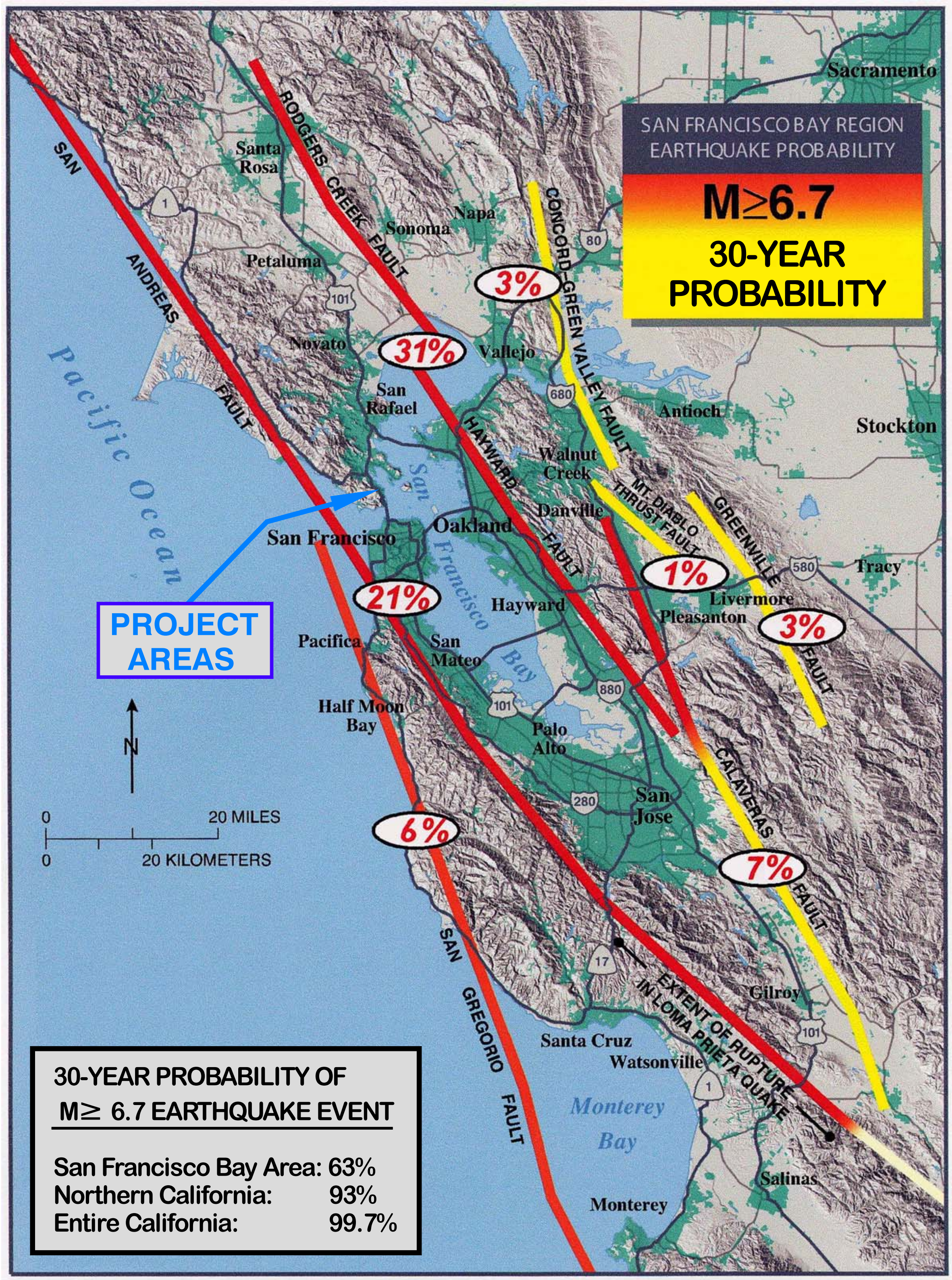
Priority 1 Sewer Replacement
Sausalito, California

BAY MUD SETTLEMENT AND TIME RATE

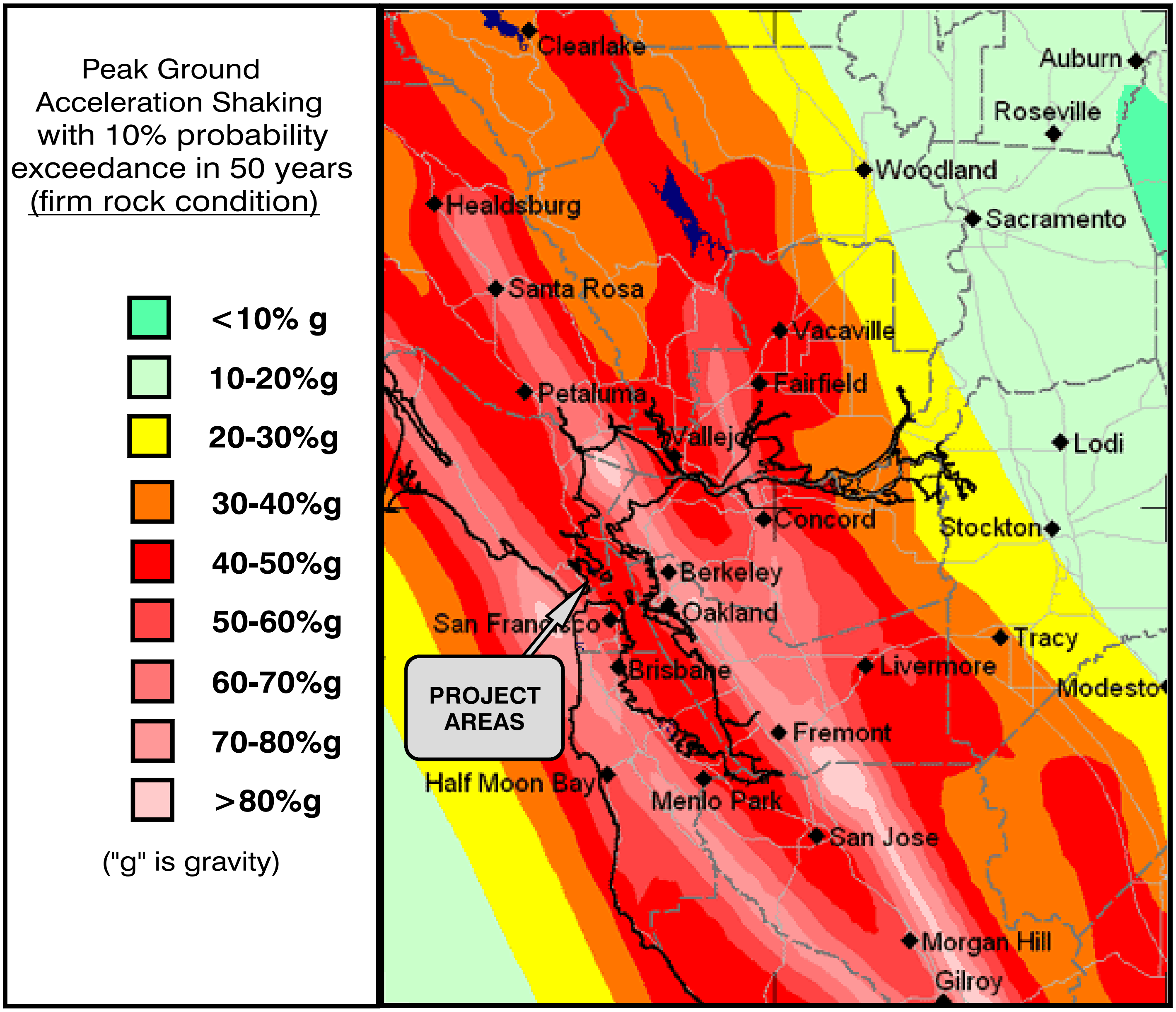
PLATE NO.
II-1

FILE NO. 18337-001-00 | AUGUST 2009



Modified from WGCEP (2003 and 2007)

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replaceement Sausalito, California	PLATE NO. II-2
FILE NO. 18337-001-00 / AUGUST 2009	BAY AREA FAULT MAP	(COLOR PLATE)



Modified from USGS/CGS 2002 Probabilistic Seismic Hazards Assessment Model (Cao and others 2003).

Latitude/Longitude:	**N37.8685°/W122.4985°**
Peak Ground Acceleration:	**0.5g**

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SEISMIC SHAKING MAP

PLATE NO.
II-3
(COLOR PLATE)

FILE NO. 18337-001-00 | AUGUST 2009

AVERAGE PEAK VELOCITY (CENTIMETERS PER SECOND)	MODIFIED MERCALLI INTENSITY VALUE AND DESCRIPTION	AVERAGE PEAK ACCELERATION ("g" is gravity - 9.80 metres per second squared)
	I. Not felt except by a very few under especially favorable circumstances.	
	II. Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.	
	III. Felt quite noticeable indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing vehicles may rock slightly. Vibration like passing of a truck. Duration estimated.	
1-2	IV. During the day felt indoors by many, outdoors by few. At night some awakened. Rattling of dishes, windows, and doors; walls make creaking sounds. Hanging objects swing. Sensation like a heavy truck passing. Standing vehicles rocked noticeably.	0.015g-.02g
2-5	V. Felt by nearly everyone, many awakened. Some dishes, windows and so on broken; cracked plaster in a few places; unstable objects overturned. Disturbances of trees, poles and other tall objects sometimes noticeable. Pendulum clocks may stop. Buildings trembled throughout.	0.03g-0.04g
5-8	VI. Felt by all, many frightened and run outdoors. Some moderately heavy furniture moved; a few instances of fallen plaster and damaged chimneys. Trees, bushes, shaken slightly to moderately. Damage slight in poorly constructed buildings. Broken dishes, glassware and some windows. Moved furnishings and overturned furniture.	0.06g-0.07g
8-12	VII. Everybody runs outdoors. Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; chimneys cracked to considerable extent. Noticed by persons driving vehicles. Waves on ponds, lakes, running water. Broke numerous windows, heavy furniture overturned. Dislodged bricks and stones.	0.10g-0.15g
20-30	VIII. Damage slight in specially designed structures; considerable in ordinary substantial buildings with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving vehicles disturbed.	0.25g-0.30g
45-55	IX. Damage considerable in specially designed structures; well-designed frame structures thrown out-of-plumb; great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken. Reservoirs threatened.	0.50g-0.55g
More than 60	X. Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Railroad rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed, slopped over banks. Reservoirs greatly damaged. Open cracks in cement pavements and asphalt road surfaces.	More than 0.60g
	XI. Few, if any, (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly. Dams, dikes, embankments severly damaged. Destroyed large well-built bridges.	
	XII. Damage total. Practically all works of construction damaged greatly or destroyed. Landslides, falls of rock, slumping of river banks extensive. Fault slips in firm rock, with notable horizontal vertical off-set displacements. Water channels, surface and underground disturbed and modified greatly. Waves seen on ground surfaces.	

REFERENCE ; Compiled from "Earthquakes & Volcanoes," Volume 21, Number 1, 1989, and "Earthquakes A Primer," Bruce A. Bolt, W.H. Freeman and Company, San Francisco, Copyright 1993.

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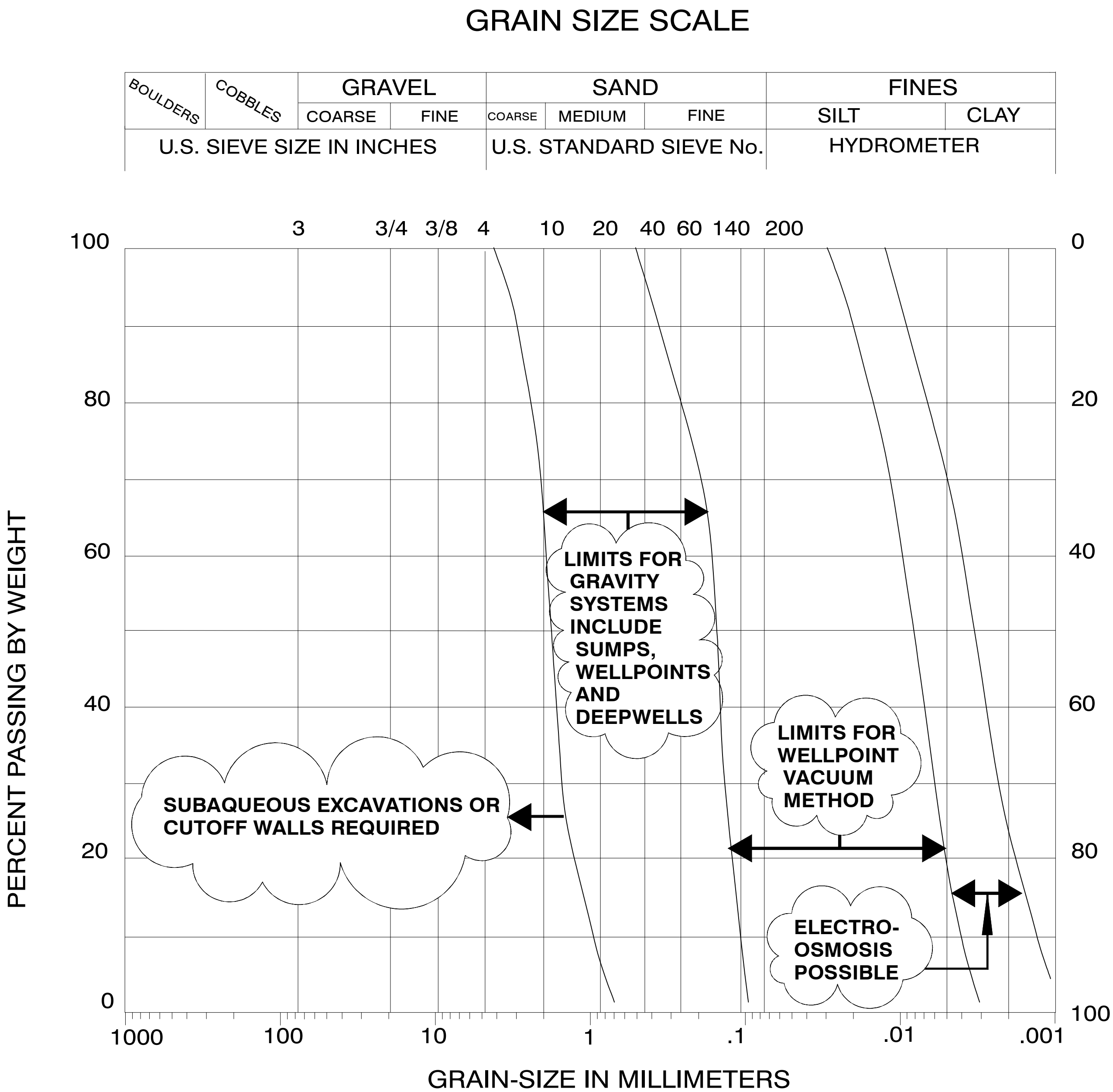
City of Sausalito
Priority 1 Sewer Replaceement
Sausalito, California

PLATE NO. II-4

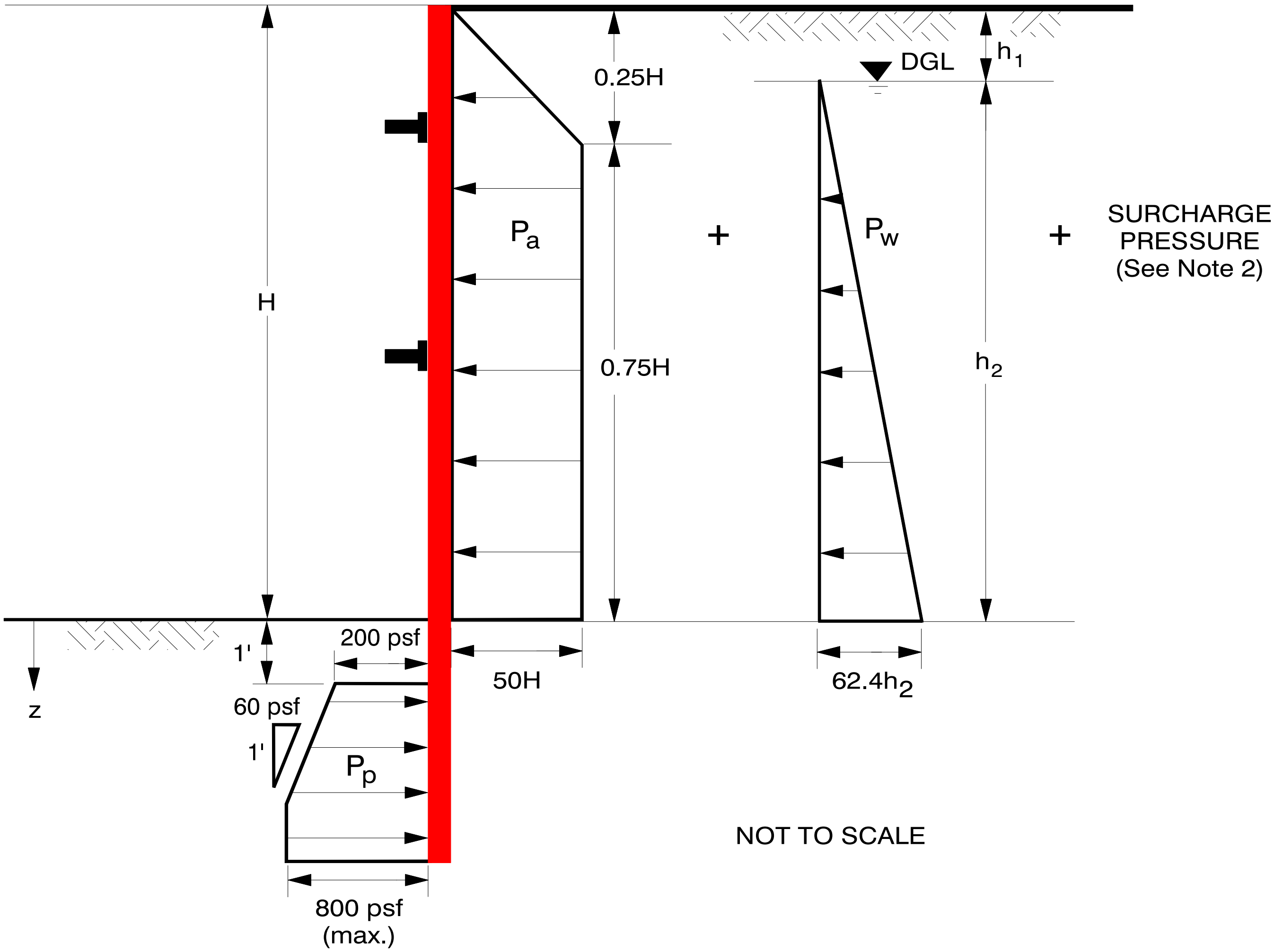
FILE NO. 18337-001-00 | AUGUST 2009

MODIFIED MERCALLI SCALE



From: Naval Facilities Engineering Command, 1986, Design Manual 7.02 Foundations and Earth Structures, Fiqure 14.

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replaceement Sausalito, California DEWATERING LIMITATIONS VS. GRAIN SIZE	PLATE NO. II-5
FILE NO. 18337-001-00 / AUGUST 2009		



NOT TO SCALE

LEGEND:

H = Total Excavation Height

h_1 = Height of shoring above design groundwater level (DGL)

h_2 = Height of shoring below design groundwater level (DGL)

z = depth below bottom of excavation

DGL = Design Groundwater Level (for Shoring Design)

P_a = Active Braced Earth Pressure (psf): equals 50H

P_p = Passive Earth Pressure (psf): equals 200 psf + 60 psf/ft of depth to a maximum of 800 psf

P_w = Hydrostatic Pressure (psf): equals 62.4h_2

NOTES:

1. Passive earth pressures are ultimate values and an appropriate factor of safety of at least 2 should be used in calculation of embedment depth.
2. This preliminary pressure diagram has been constructed without consideration for surcharge loads such as stockpiled soil, traffic, structural loads, etc. Surcharge loads must be added to this pressure diagram where applicable. Minimum shoring pressures for traffic and equipment surcharge are provided on Plate II-8.

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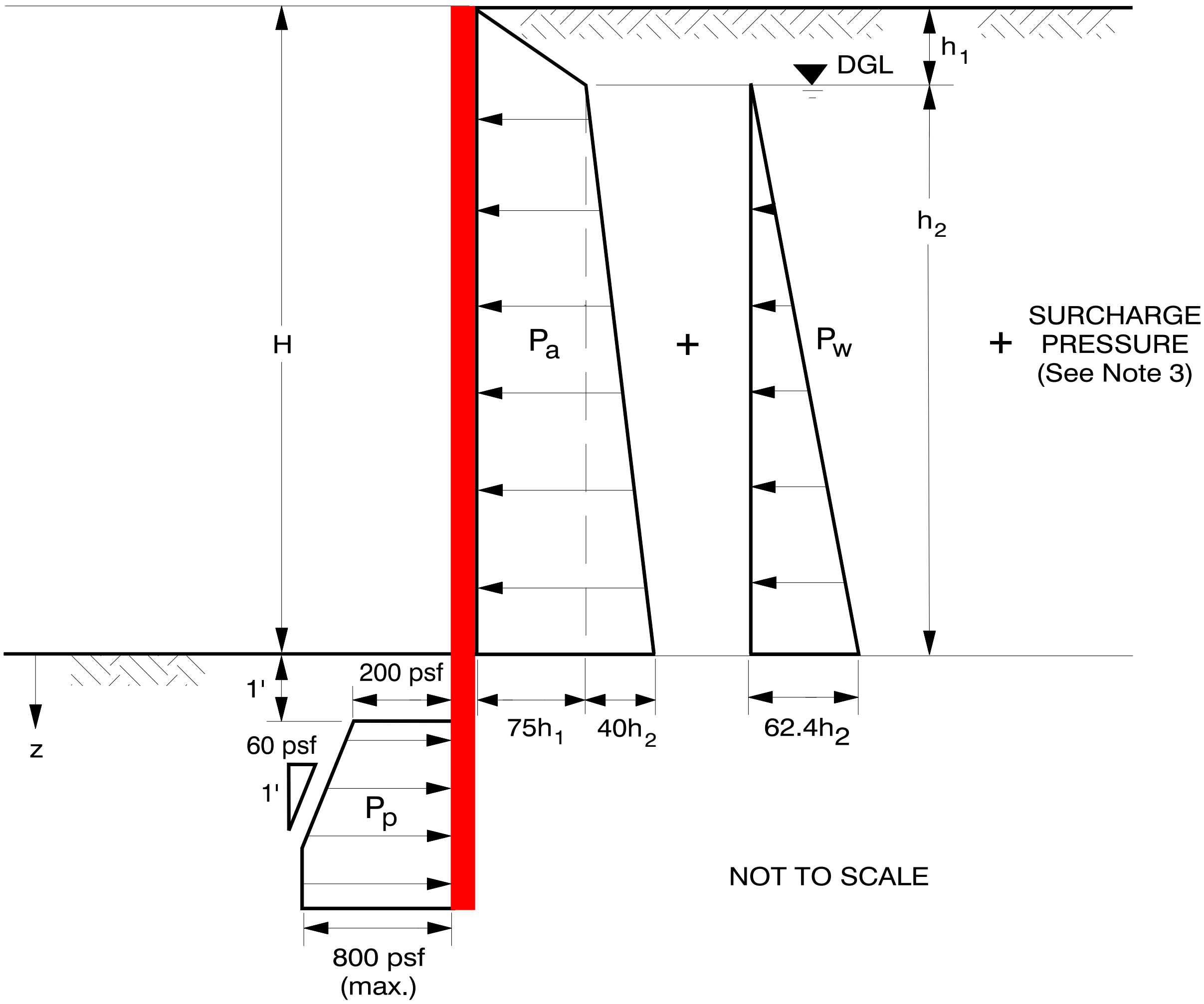
Priority 1 Sewer Replaceement
Sausalito, California

PRELIMINARY EARTH PRESSURE DIAGRAM FOR BRACED SHORING

PLATE NO.
II-6



LEGEND:

H = Total Excavation Height

h_1 = Height of shoring above design groundwater level (DGL)

h_2 = Height of shoring below design groundwater level (DGL)

z = depth below bottom of excavation

DGL = Design Groundwater Level (for Shoring Design)

P_a = Active Earth Pressure (psf): equals $75h_1$ above DGL, $75h_1 + 40h_2$ below DGL

P_p = Passive Earth Pressure (psf): equals 200 psf + 60 psf/ft of depth to a maximum of 800 psf

P_w = Hydrostatic Pressure (psf): equals $62.4h_2$

NOTES:

1. Passive earth pressures are ultimate values and an appropriate factor of safety of at least 2 should be used in the calculation of embedment depth.
2. Deflection of cantilevered shoring should be calculated on a case by case basis and limited as necessary to avoid damage to existing structures and utilities.
3. This preliminary pressure diagram has been constructed without consideration for surcharge loads such as stockpiled soil, traffic, structural loads, etc. Surcharge loads must be added to this pressure diagram where applicable. Minimum shoring pressures for traffic and equipment surcharge are provided on Plate II-8.

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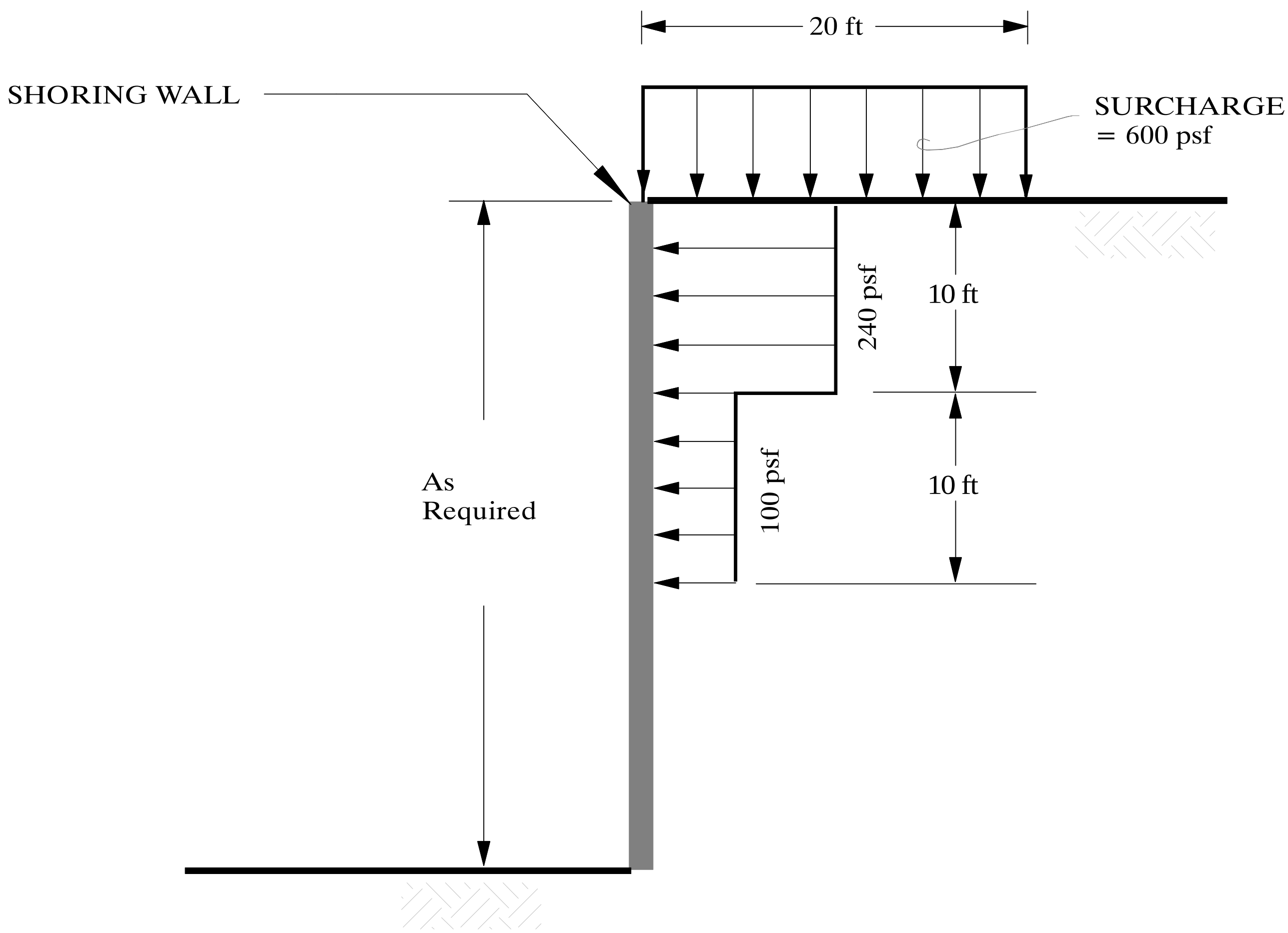
WEST YOST ASSOCIATES
City of Sausalito
Priority 1 Sewer Replaceement
Sausalito, California
PRELIMINARY EARTH PRESSURE
DIAGRAM FOR CANTILEVER SHORING

PLATE NO.
II-7



Notes:

These are minimum shoring pressures to be used for traffic and equipment surcharges. Shoring pressures from existing structures, construction activities or equipment that produce larger or different surcharge loading patterns than that shown should be determined by the shoring designer using geotechnical computational methods.

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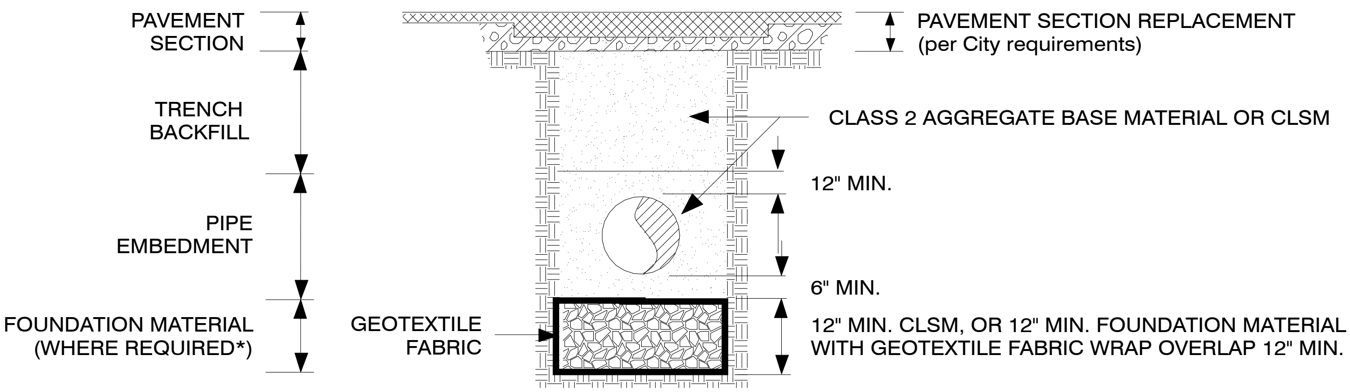
Priority 1 Sewer Replaceement
Sausalito, California

MINIMUM SHORING PRESSURES FOR
TRAFFIC AND EQUIPMENT SURCHARGE

PLATE NO.

II-8

TRENCH BACKFILL

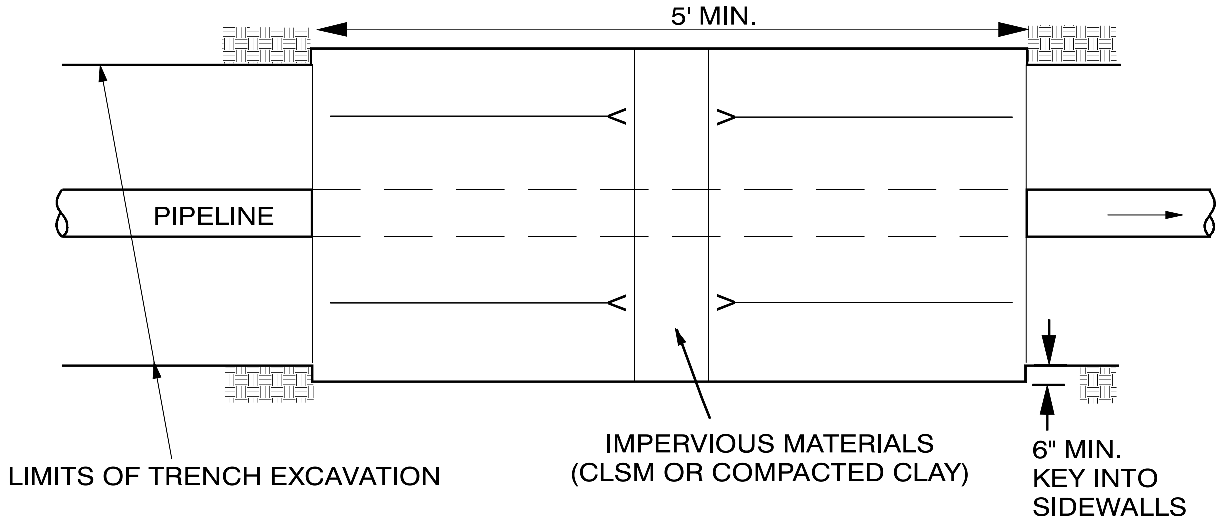


*Foundation Material should be required where trench bottoms are unstable (e.g., pumping subgrade under foot load, groundwater seepage, boiling, etc.)

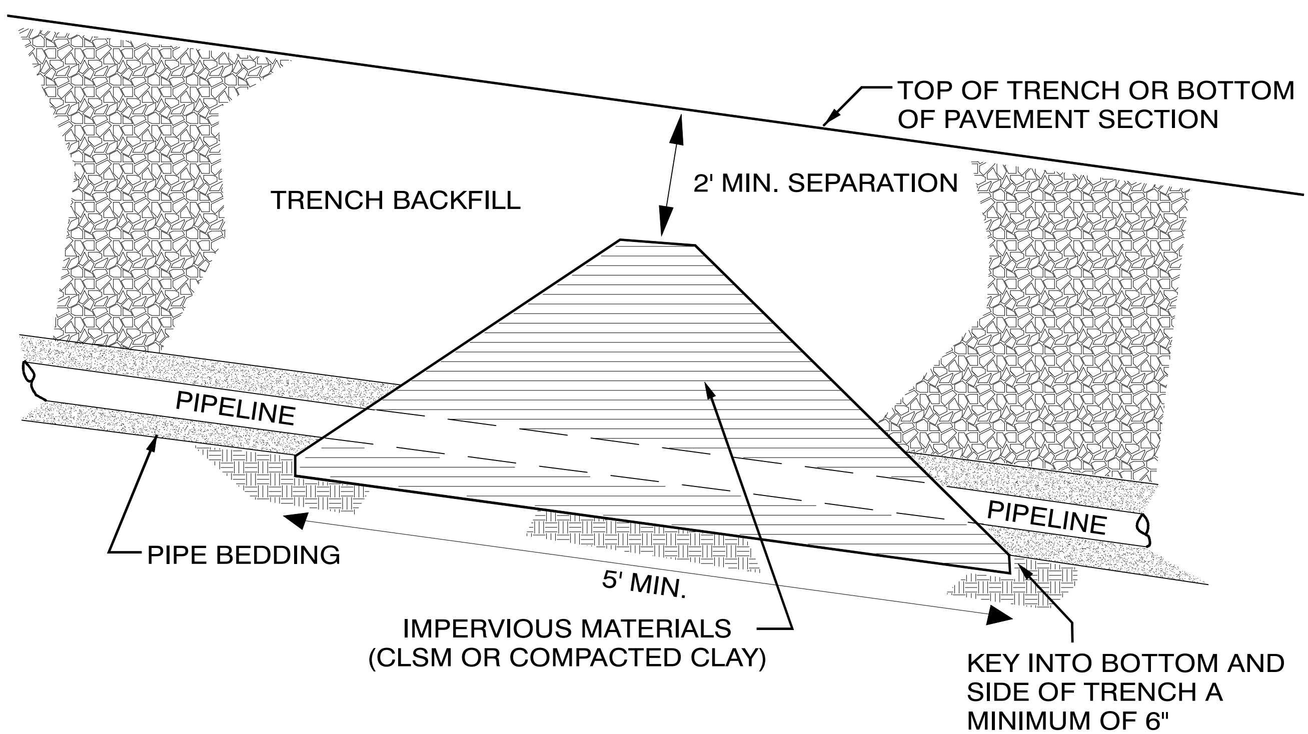
NOTES:

1. Not to scale.
2. See report text for related material recommendations and compaction requirements.

DCM GeoEngineers		WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replaceement Sausalito, California	PLATE NO. II-9
FILE NO. 18337-001-00	AUGUST 2009	TRENCH BACKFILL DETAILS	



PLAN VIEW



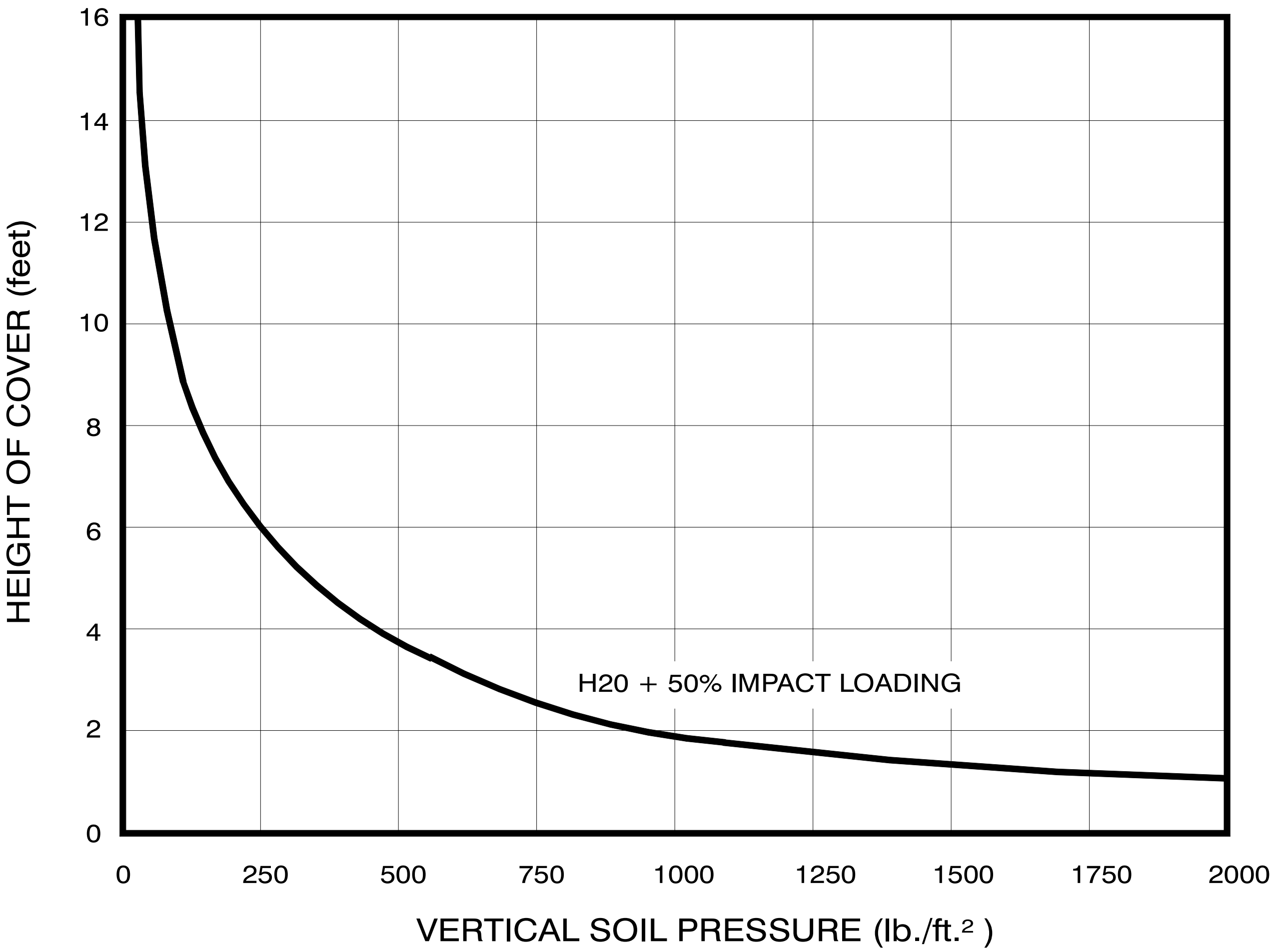
NOTE:

1. See text of report for material descriptions and for compaction requirements.

2. Trench dams to be constructed as shown on the plans a minimum of 3 feet from any pipe joint.

SECTION VIEW

DCM GeoEngineers	WEST YOST ASSOCIATES City of Sausalito Priority 1 Sewer Replacement Sausalito, California	PLATE NO. II-10
FILE NO. 18337-001-00 AUGUST 2009	TRENCH DAM	



NOTE: Apply vertical soil pressure to diameter of pipeline (horizontal projection) to calculate vertical pipe load (Moser, 2001).

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Priority 1 Sewer Replaceement
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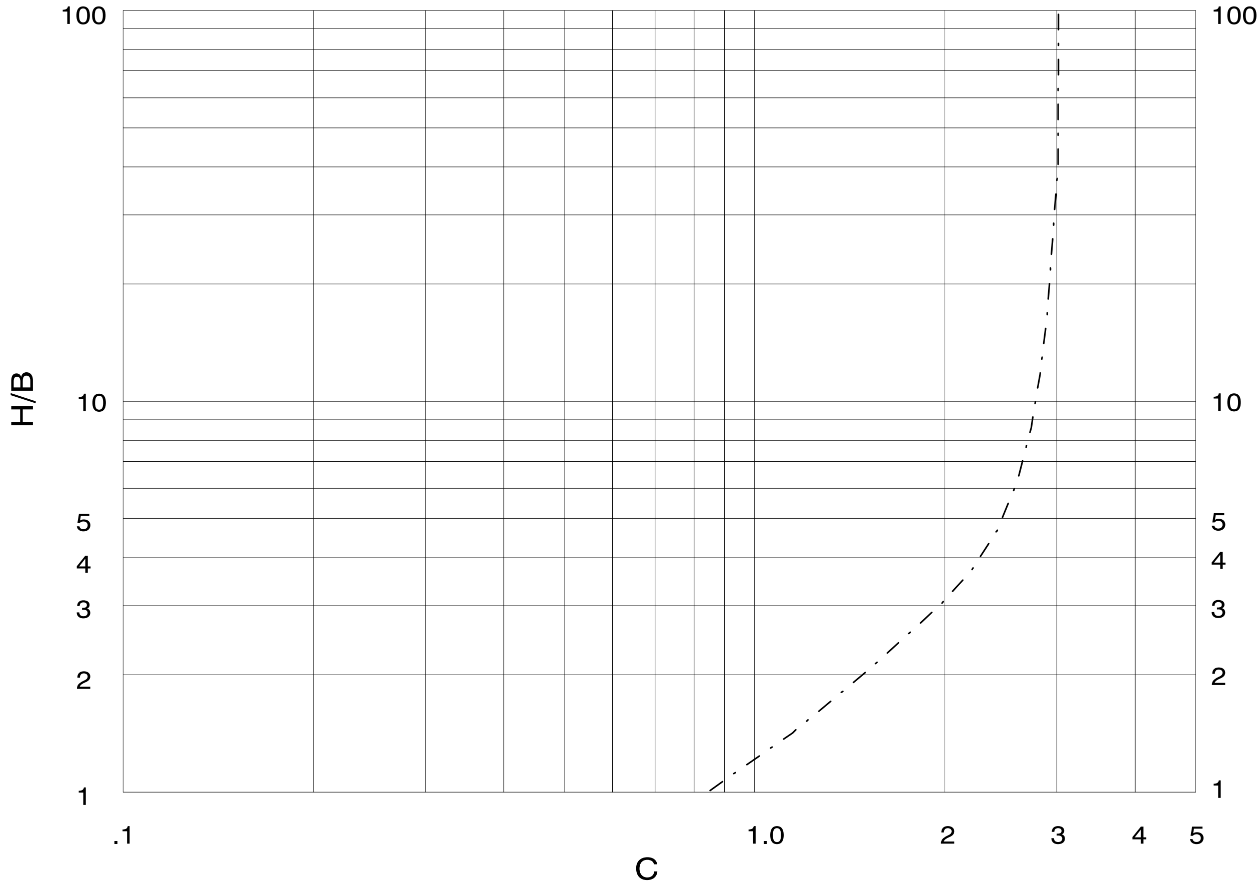
VERTICAL SOIL PRESSURE DUE TO LIVE LOADS

PLATE NO.

II-11



LEGEND:

— · — · — · — = Compacted Granular Backfill (Class 2AB)

$$W = C \gamma B^2$$

where: W = Vertical soil load on rigid pipe due to overburden (pounds/foot)

γ = Unit weight of overlying materials (pounds/cubic foot)

H = Height/depth of overlying materials (feet)

B = Trench width

C = Load coefficient

NOTE: Marston's load coefficients are used to calculate vertical soil loads on rigid pipes installed by open cut trenching (Moser, 2001).

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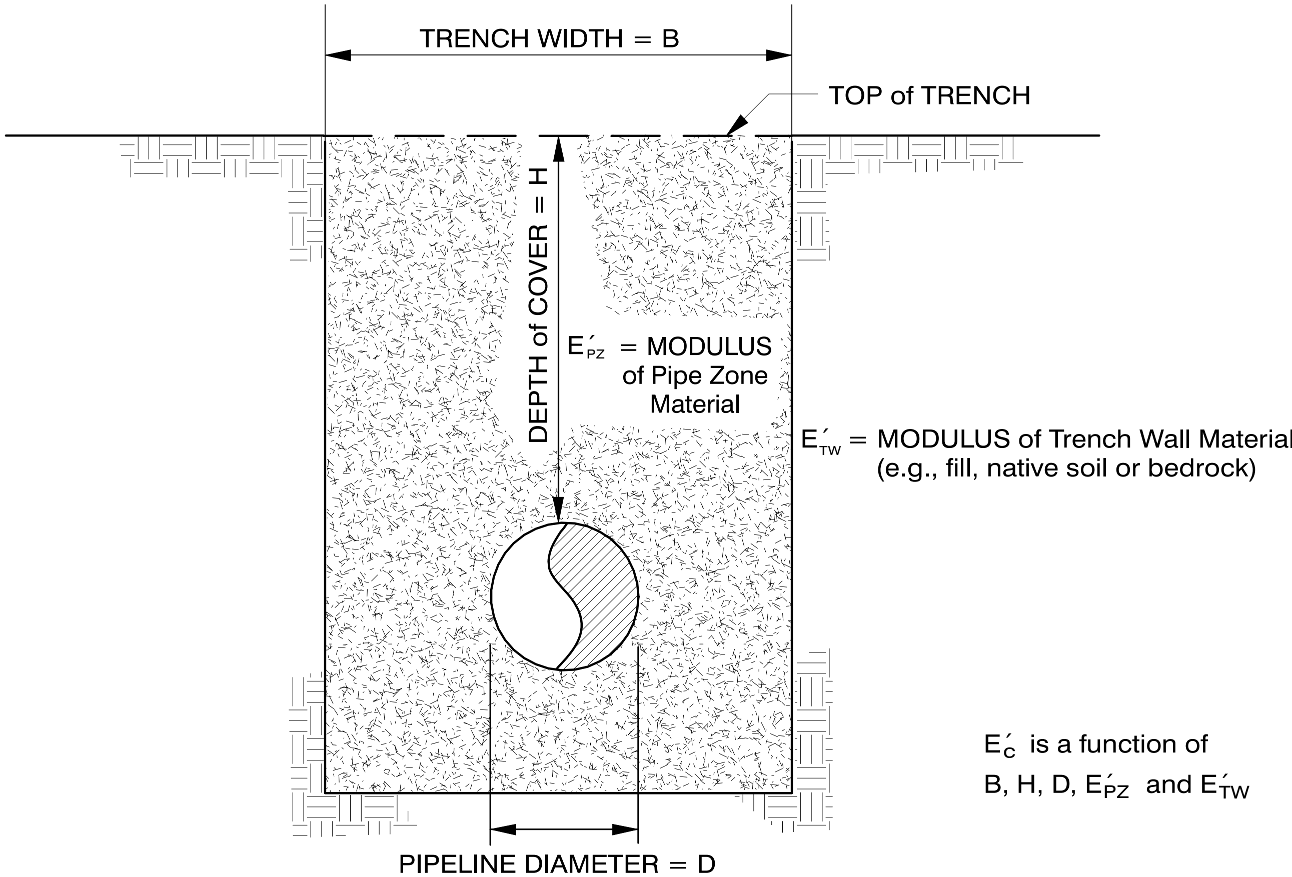
MARSTON'S LOAD COEFFICIENTS
TRENCH CONDITIONS

PLATE NO.
II-12

FILE NO. 18337-001-00

AUGUST 2009



E'_C is a function of B, H, D, E'_{PZ} and E'_{TW}

E'_{TW} / E'_{PZ}	S_C for various B:D ratios					
	1.5	2.0	2.5	3.0	4.0	5.0
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.30	1.15	1.10	1.05	1.00	1.00
2.0	1.50	1.30	1.15	1.10	1.05	1.00
3.0	1.75	1.45	1.30	1.20	1.08	1.00
>=5.0	2.00	1.60	1.40	1.25	1.10	1.00

Modified from Jeyapalan (2001)

$$E'_C = S_C E'_{PZ}$$

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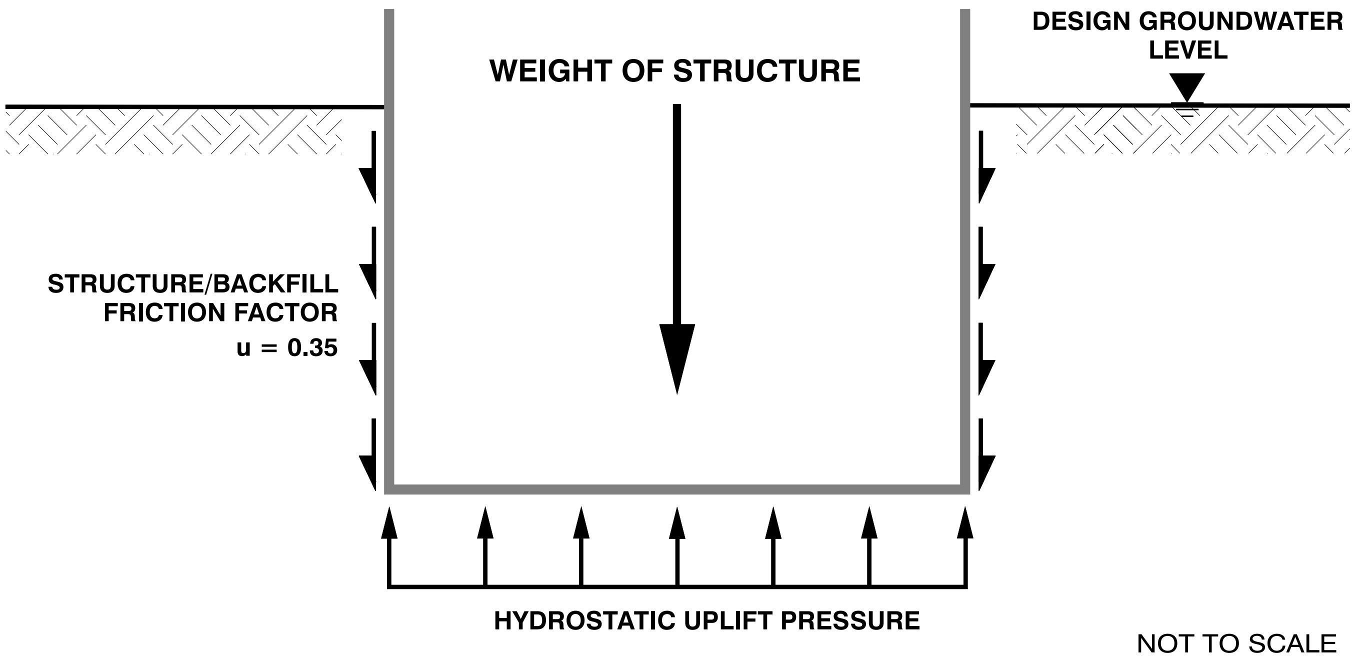
Priority 1 Sewer Replaceement
Sausalito, California

COMPOSITE MODULUS OF SOIL REACTION - E'c

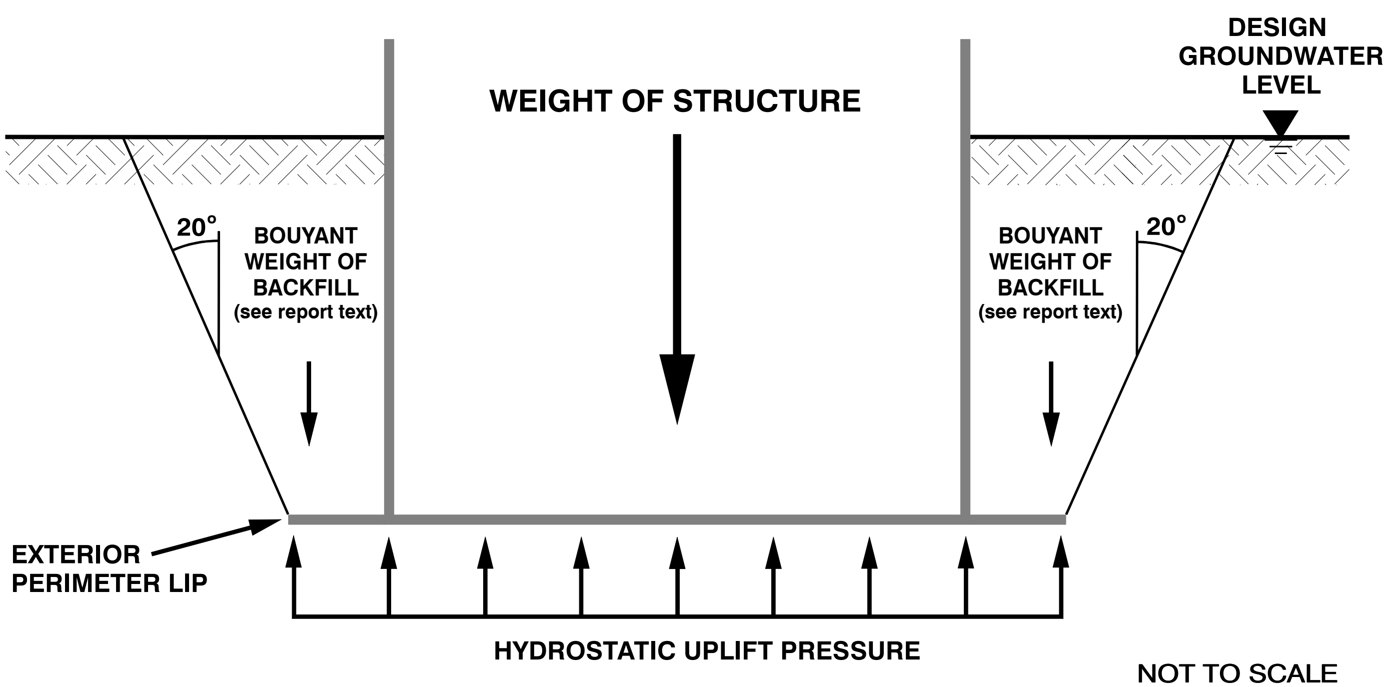
PLATE NO.

II-13

STRUCTURE WITHOUT EXTERIOR PERIMETER LIP



STRUCTURE WITH EXTERIOR PERIMETER LIP



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Priority 1 Sewer Replaceement
Sausalito, California

UPLIFT RESISTANCE

PLATE NO.
II-14

FILE NO. 18337-001-00 | AUGUST 2009

APPENDIX E

APPENDIX E
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for the exclusive use of West Yost Associates and City of Sausalito and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree to such reliance in advance and in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and the generally accepted geotechnical practices in this area at the time this report was prepared. Use of this report is not recommended for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for City of Sausalito's Priority 1 Sewer Replacement Project. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed pipelines;
- elevation, configuration, location, orientation or size of the proposed pipelines;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, we recommend that GeoEngineers be given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

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[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Please contact GeoEngineers before applying a report for its intended purpose so that we can evaluate whether changed conditions affect the continued reliability of the report.

MOST GEOTECHNICAL AND GEOLOGIC FINDINGS ARE PROFESSIONAL OPINIONS

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an informed opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

The construction recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers is unable to assume responsibility for the recommendations in this report without performing construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation. Please contact GeoEngineers if you want us to provide these additional services.

DO NOT REDRAW THE EXPLORATION LOGS

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

To help prevent costly problems associated with unanticipated subsurface conditions, we recommend giving contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited. In addition, encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

CONTRACTORS ARE RESPONSIBLE FOR SITE SAFETY ON THEIR OWN CONSTRUCTION PROJECTS

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

READ THESE PROVISIONS CLOSELY

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) are less exact than other engineering and natural science disciplines. Without this understanding, there may be expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

BIOLOGICAL POLLUTANTS

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.