Geotechnical Interpretive Report

Walnut Creek Water Treatment Plant

2201 Larkey Lane

Walnut Creek, California

Woodard & Curran

2175 N. California Boulevard | Walnut Creek, California 94596

June 9, 2022 | Project No. 403982001



Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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CELEBRATING 35 YEARS

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1 INTRODUCTION

This Geotechnical Interpretive Report (GIR) has been prepared by Ninyo & Moore as subconsultant to Woodard & Curran and presents the findings from recent subsurface exploration performed by Ninyo & Moore, a compilation of geotechnical and geologic information from regional studies and previous investigations at the Walnut Creek Water Treatment Plant (WCWTP), and geotechnical recommendations for the WCWTP Pretreatment Project (Project). The geotechnical recommendations are provided for consideration during project planning and should be reviewed by the future Geotechnical Engineer-of-Record (GEOR) and modified, as appropriate, to reflect actual structure dimensions, updated loads, or additional information from supplemental investigations.

2 SITE DESCRIPTION

The WCWTP located at 2201 Larkey Lane in Walnut Creek (Figure 1) is bounded to the east by single-family residential properties, and to the north, south, and west by open space. The WCWTP is situated on the northeastern flank of a northwest trending ridge atop a smaller, northerly-trending spur. Existing facilities at the WCWTP include a 16-million-gallon (MG) clearwell tank, a 4-MG chlorine contact chamber, dual-media filters, filter backwash water reclamation system, solids processing facilities, chemical building, Larkey Pumping Plant (PP), and the Leland Number 2 PP.

3 PROJECT DESCRIPTION

The proposed improvements associated with the Project under consideration (EBMUD, 2020) are summarized in Table 1. The proposed locations for the improvements are noted on Figure 2. The anticipated finish floor elevations for several of the proposed facilities are significantly above or below the existing grade. The anticipated finish pad elevations for the proposed facilities based on preliminary cut/fill estimates in the Basis of Design Report (BODR; EBMUD, 2020) are summarized in Table 2 along with the estimated grade differential between the existing grade and finish pad elevation assuming a finish pad elevation approximately 1 foot below finish floor. The assumed pad elevations listed in Table 2 are ordered geographically beginning with facilities at the western margin of the WCWTP and proceeding northeast then south. The alternate shading in the table is used to group facilities within close proximity.

Table 1 – Summary of Proposed Facilities

Facility ID	Name	Type	Quantity	Approximate Dimensions (ft)
1	Ballasted Flocculation	Concrete, Buried	4	81x32x27
2	Intermediate Ozone PP	Concrete, Buried	2	50x40x20
3	Intermediate Ozone Contactors	Concrete, Buried	2	80x44x26
4	Ozone Generator Building	Concrete	1	200x50x15
5	LOX Tanks & Vaporizers	Concrete	1	83x40x27
6	Pre-Ozone PP	Concrete	2	50x30x20
7	Combined Reclaim Metering Vault	Concrete, Buried	1	40x25x18
8	Ozone Gas Destruct	Canopy	5	20x10x12
9	Dewatering Building	Concrete	1	100x45x27
9A	Truck Loading Canopy	Prefabricated	1	45x30x20
10	Thickened Solids Blending Tanks	Concrete	2	20x20x16
11	Gravity Thickeners	Concrete, Buried	4	80’ Diameter x17
12	Thickened Solids PP	Concrete, Buried	2	50x30x27
13	Solids Transfer PP	Concrete	1	20x15x17
14	Electrical Facilities	Prefabricated	1	100x45x18
15	Maintenance Building	Concrete	1	120x60x27
16	Ozone Quenching Vault	Concrete, Buried	2	30x25x20
17	Chemical Storage	Canopy	1	93x23x20
18	Hydrogen Peroxide Station	Canopy	2	40x30x15

Note:
Dimensions are Length x Width x Height in feet unless otherwise noted

Reference: BODR (EBMUD, 2020)

Table 2 – Assumed Pad Elevations			
Facility ID	**Name**	**Finish Pad Elevation**	**FPE Relative to EGSE**
7	Combined Reclaim Metering Vault	323 feet	18 feet below
6	Pre-Ozone PP – East	339 feet	1 foot below to 3 feet above
6	Pre-Ozone PP – West	340 feet	1 to 3 feet below
8	Ozone Gas Destruct- West	346 feet	5 to 14 feet below
18	Hydrogen Peroxide Station – West	346 feet	1 to 14 feet below
15	Maintenance Building	336 feet	2 feet below to 7 feet above
14	Electrical Facilities	333 feet	2 feet to 6 feet above
4	Ozone Generator Building	361 feet	1 foot below to 17 feet above
5	LOX Tanks & Vaporizers	365 feet	2 feet below to 5 feet above
11	Gravity Thickener – Northeast	290 feet	4 to 20 feet below
11	Gravity Thickener – Northwest	290 feet	18 to 25 feet below
11	Gravity Thickener – Southwest	293 feet	24 to 27 feet below
11	Gravity Thickener – Southeast	293 feet	8 to 25 feet below
12	Thickened Solids PP – North	286 feet	22 to 28 feet below
12	Thickened Solids PP – South	290 feet	26 to 29 feet below
9	Dewatering Building	317 feet	3 to 7 feet below
9A	Truck Loading Canopy	322 feet	1 to 4 feet below
10	Thickened Solids Blending Tank – North	319 feet	0 to 5 feet below
10	Thickened Solids Blending Tank – South	319 feet	4 to 6 feet below
13	Solids Transfer PP	320 feet	1 foot below to 1 foot above
1	Ballasted Flocculation – Northeast	360 feet	24 feet below
1	Ballasted Flocculation – Northwest	360 feet	24 feet below
2	Intermediate Ozone PP – North	370 feet	14 feet below
3	Intermediate Ozone Contactor – North	357 feet	21 to 26 feet below
8	Ozone Gas Destruct	On North Intermediate Ozone Contactor (3)	
16	Ozone Quenching Vault – North	338 feet	10 to 22 feet below
8	Ozone Gas Destruct	On North Ozone Quenching Vault (16)	
18	Hydrogen Peroxide Station – East	382 feet	1 foot below
1	Ballasted Flocculation – Southeast	360 feet	24 feet below
1	Ballasted Flocculation – Southeast	360 feet	24 feet below
2	Intermediate Ozone PP – South	370 feet	14 feet below
3	Intermediate Ozone Contactors – South	364 feet	20 to 37 feet below
8	Ozone Gas Destruct	On South Intermediate Ozone Contactor (3)	
16	Ozone Quenching Vault – South	361 feet	20 to 31 feet below
8	Ozone Gas Destruct	On South Ozone Quenching Vault (16)	
17	Chemical Storage	380 feet	1 foot below to 1 foot above

Notes:
Finish Pad Elevation (FPE) estimated from preliminary cut/fill estimates in the BODR (EBMUD, 2020) and Existing Ground Surface Elevation (EGSE) from Figure 2.

New permanent retaining walls associated with the Project are under consideration at eight locations (EBMUD, 2020). The retaining walls will retain cut at four locations and fill at four locations. The retaining wall locations are listed in Table 3 along with a description of the retained grade differential across the wall. In addition, three new staging areas have been proposed for construction (Figure 2) with retaining walls along the southern margins of the staging areas so that the grade in the staging areas may be cut down to match the grade on the adjacent roadway. The anticipated grade differential of the retaining walls at the staging areas are also listed in Table 3.

Table 3 – Summary of Proposed Retaining Walls

Wall Location	Retains
West, south, and east of western Hydrogen Peroxide Station (18) and western Ozone Gas Destruct (8)	Up to 18 feet of cut
East and south of Electrical Facilities (14) and south of Dewatering Building (9) and Maintenance Building (15)	Up to 8 feet of fill
North and west of Ozone Generator Building (4)	Up to 22 feet of fill
Northern corner of LOX Tanks and Vaporizers (5)	Up to 10 feet of fill
West, north, and east of Gravity Thickeners (11) and Thickened Solids PP (12)	Up to 22 feet of fill
Eastern portion of northern Ozone Quenching Vault (16)	Up to 12 feet of cut
West, south, and east of southern Ozone Quenching Vault (16)	Up to 14 feet of cut
North, east, and south of southern Intermediate Ozone Contactor (3)	Up to 18 feet of cut
West and south of Staging Area 1	Up to 18 feet of cut
South and east of Staging Area 2	Up to 12 feet of cut
South and east of Staging Area 3	Up to 10 feet of cut

Reference: BODR (EBMUD, 2020)

To evaluate and develop appropriate foundation recommendations, Ninyo & Moore estimated distributed interior loads, sustained column and wall loads, and gross mat foundation bearing pressures that includes the distributed interior and wall/column loads for the proposed facilities, where applicable. The estimated foundation loads are summarized in Table 4.

Table 4 – Sustained Foundation Loads Assumed for Analysis					
Facility ID	Facility Name	Distributed Interior Load	Column Load	Wall Load	Gross Mat Bearing Pressure
1	Ballasted Flocculation Basins	2,700 psf	--	4.6 kips/foot	3.1 ksf
2	Intermediate Ozone PP Vault	1,000 psf	--	3.6 kips/foot	1.3 ksf
3	Intermediate Ozone Contactor Tanks	2,600 psf	--	4.6 kips/foot	2.9 ksf
4	Ozone Generator Building	1,000 psf	38 kips	2.5 kips/foot	1.1 ksf
5	LOX Tanks & Vaporizers Building	1,000 psf	28 kips	3.4 kips/foot	1.3 ksf
6	Pre-Ozone PP Building	1,000 psf	19 kips	2.5 kips/foot	1.3 ksf
7	Combined Reclaim Metering Vault	1,000 psf	--	--	1.4 ksf
8	Ozone Gas Destruct Canopy	1,000 psf	3 kips	--	1.1 ksf
9	Dewatering Building	1,000 psf	38 kips	3.5 kips/foot	1.2 ksf
9A	Truck Loading Canopy	--	15 kips	--	--
10	Thickened Solids Blending Tanks	1,600 psf	--	2.7 kips/foot	2.1 ksf
11	Gravity Thickener Tanks	1,700 psf	--	4.8 kips/foot	1.9 ksf
12	Thickened Solids PP Vault	1,000 psf	--	4.5 kips/foot	1.5 ksf
13	Solids Transfer PP Building	1,000 psf	--	1.9 kips/foot	1.5 ksf
14	Electrical Facilities	1,000 psf	38 kips	1.2 kips/foot	1.1 ksf
15	Maintenance Building	100 psf	16 kips	2.1 kips/foot	0.4 ksf
16	Ozone Quenching Vault	1,000 psf	--	3.3 kips/foot	1.5 ksf
17	Chemical Storage Canopy	1,000 psf	18 kips	--	--
18	Hydrogen Peroxide Station Canopy	1,000 psf	15 kips	--	1.1 ksf

Source: Ninyo & Moore estimates

4 FIELD EXPLORATION AND TESTING

EBMUD prepared a geotechnical investigation report in 2001 (EBMUD, 2001) that presented the results of geophysical surveys performed in 1997 and 1998, and the logs and laboratory test results for borings performed by EBMUD in 1997 as well as the logs and laboratory test results for borings performed by Olivia Chen Consultants in 2001. As part of this GIR, Ninyo & Moore performed a geophysical survey, subsurface exploration, and laboratory testing to supplement the findings from the previous investigations. The subsurface exploration, geophysical surveys, and geotechnical testing performed for the GIR and previous studies are documented in the Appendices to this report and summarized in the following subsections.

4.1 Borings

EBMUD logged 22 borings drilled in 1997 using mud rotary wash techniques with a Failing 750 rig. Olivia Chen Consultants logged 13 borings drilled in 2001 using mud rotary wash techniques with a Failing 1500 rig. The top of boring elevation, approximate depth explored, and date of completion for the borings logged by EBMUD and Olivia Chen Consultants are summarized in Table 5.

Table 5 – Summary of Previous Borings

Boring ID	Logged By	Date Completed	Elevation	Depth Explored
MM-1	EBMUD	10-28-1997	377 feet	76.0 feet
MM-2	EBMUD	11-5-1997	359 feet	33.0 feet
MM-3	EBMUD	11-3-1997	376 feet	65.5 feet
MM-4	EBMUD	11-5-1997	376 feet	34.0 feet
MM-5	EBMUD	11-6-1997	315 feet	14.0 feet
MN-2	EBMUD	11-4-1997	380* feet	6.5 feet
MN-3	EBMUD	10-30-1997	380* feet	23.5 feet
MN-4	EBMUD	10-27-1997	380* feet	32.5 feet
MN-8	EBMUD	11-4-1997	380* feet	13.0 feet
MN-9	EBMUD	10-31-1997	380* feet	24.0 feet
MN-10	EBMUD	10-31-1997	380* feet	18.0 feet
MN-11	EBMUD	11-6-1997	380* feet	9.0 feet
MN-13	EBMUD	10-27-1997	335* feet	22.5 feet
MN-14	EBMUD	10-28-1997	310* feet	20.0 feet
MN-15	EBMUD	10-23-1997	320* feet	47.5 feet
MN-16	EBMUD	10-23-1997	330* feet	42.5 feet
MN-17	EBMUD	10-23-1997	345* feet	42.5 feet
MN-18	EBMUD	10-21-1997	330* feet	82.0 feet
MN-19	EBMUD	10-22-1997	350* feet	40.7 feet
MN-20	EBMUD	11-5-1997	375* feet	7.0 feet
MN-21	EBMUD	10-24-1997	380* feet	35.0 feet
MN-22	EBMUD	10-30-1997	380* feet	32.0 feet
MN-23	Olivia Chen	2-12-2001	375* feet	43.3 feet
MN-23A	Olivia Chen	1-4-2001	380* feet	40.3 feet
MN-24	Olivia Chen	1-4-2001	380* feet	33.0 feet
MN-25	Olivia Chen	1-3-2001	375* feet	48.0 feet
MN-26	Olivia Chen	1-2-2001	335* feet	30.5 feet
MN-27	Olivia Chen	1-9-2001	320* feet	30.4 feet
MN-28A	Olivia Chen	1-3-2001	375* feet	31.5 feet
MN-29A	Olivia Chen	1-9-2001	365* feet	30.3 feet
MN-30	Olivia Chen	1-2-2001	340* feet	31.5 feet
MN-31	Olivia Chen	1-8-2001	320* feet	41.3 feet
MN-32	Olivia Chen	1-5-2001	335* feet	23.0 feet
MN-33	Olivia Chen	1-5-2001	330* feet	23.0 feet
MN-34	Olivia Chen	1-5-2001	315* feet	14.3 feet

Reference: EBMUD, 2001 except for Top of Boring elevation estimated from previous topography (EBMUD, 2001) where marked with asterisk

As part of this GIR, Ninyo & Moore retained Geo-Ex Subsurface Exploration of Dixon, California to drill eleven borings to supplement the information from the previous investigations. A sonic dual tube system on a Geoprobe 8140LS rig was used to recover continuous core in 5-foot intervals for two of the borings. A CME 75 rig was used to drill the remaining borings with solid stem auger, mud rotary wash techniques, or HQ wireline coring. The top of boring elevation, approximate depth explored, method of drilling, and date of completion for the borings logged by Ninyo & Moore are summarized in Table 6. Prior to drilling the borings, Ninyo & Moore obtained a boring permit from the Contra Costa County Environmental Health Division. A representative of Ninyo & Moore logged the subsurface conditions exposed in the borings and collected bulk and relatively undisturbed soil samples from the borings for laboratory testing. The borings were backfilled in accordance with the boring permit requirements shortly after drilling. The approximate locations of the borings for the current study and previous investigations are noted on Figure 2. Detailed logs of the borings drilled for this GIR are presented in Appendix A along with descriptions of the methods used for sample collection. Photographs of the recovered core samples are provided in Appendix A. Boring logs from the previous investigations are included in Appendix B.

Table 6 – Summary of Borings Logged by Ninyo & Moore

Boring ID	Method	Date Completed	Elevation	Depth Explored
SB-1	6-inch dual tube sonic	9-10-2021	363 feet	28.0 feet
SB-2	6-inch dual tube sonic	9-10-2021	355 feet	45.0 feet
NMB-1	Mud rotary & HQ wireline coring	9-27-2021	387 feet	26.5 feet
NMB-2	4-inch solid stem auger	9-27-2021	383 feet	9.4 feet
NMB-3	4-inch solid stem auger	9-27-2021	377 feet	28.7 feet
NMB-4	4-inch solid stem auger	9-28-2021	363 feet	40.0 feet
NMB-5	4-inch solid stem & mud rotary	9-28-2021	319 feet	38.8 feet
NMB-6	4-inch solid stem auger	10-8-2021	352 feet	25.0 feet
NMB-7	4-inch solid stem auger	10-8-2021	331 feet	35.0 feet
NMB-8	4-inch solid stem auger	10-8-2021	303 feet	10.8 feet
NMB-9	4-inch solid stem auger	10-8-2021	382 feet	25.9 feet

4.2 Geotechnical Testing

Geotechnical laboratory testing performed for this GIR included tests to evaluate in-situ soil moisture content and density, organic matter percentage, percentage passing the No. 200 sieve, particle size distribution, Atterberg limits, direct shear strength, consolidation characteristics, expansion index, unconfined compressive strength of soil and rock cores, unconsolidated undrained (UU) triaxial shear strength, consolidated undrained (CU) triaxial shear strength, and corrosivity of site soil. The results of the in-situ moisture content and dry density tests are

presented on the boring logs in Appendix A. The results of the other laboratory tests are presented in Appendix C along with descriptions of the test methods used.

Geotechnical laboratory testing performed as part of previous investigations included tests to evaluate in-situ soil moisture content and density, specific gravity, Atterberg limits, fines content, UU triaxial shear strength, and CU triaxial shear strength. The test results are presented on the boring logs in Appendix B and tabulated in Appendix D. Geomatrix (2001) also presented the results of particle size distribution and Atterberg limits tests on samples collected from Borings MM-1, MM-2, and MM-3 (Appendix D).

4.3 Geophysical Surveys

Ninyo & Moore performed a geophysical survey at the WCWTP using seismic refraction and refraction microtremor (ReMi) techniques. The survey was performed along five lines. The survey line locations are noted on Figure 2. The survey results are presented in Appendix E along with a description of the methods used.

Portola Geophysics performed downhole seismic surveys in 1997 at Boring MM-1 to a depth of approximately 54 feet and at Boring MM-3 to a depth of approximately 64 feet. The results of the downhole seismic surveys and a description of the methods used are provided in Appendix B. In 1998, Portola Geophysics performed a seismic refraction survey along three lines at the proposed location for a chemical building near the southeastern edge of the WCWTP, and along seven lines for two proposed 9-MG tanks at the current location of the 4-MG chlorine contact chamber. The results of the seismic refraction surveys and a description of the methods used are presented in Appendix F.

4.4 Piezometers

Several of the borings drilled by Olivia Chen Consultants were developed into piezometers. Details of piezometer construction are summarized in Table 7. Geomatrix (2001) reports groundwater measurements in four piezometers (WC-39 to WC-42) installed in the embankment of the open cut clearwell that was demolished to make room for the circular concrete 16-MG clearwell constructed in 2004 and 2005. Records concerning the specific location and other details regarding these four piezometers are not in the documents provided for review because the piezometers were excavated and removed as part of the open cut clearwell demolition.

Table 7 – Summary of Piezometer Construction			
Boring ID	Top of Casing Elevation	Seal Interval Depth	Screen Interval Depth
MN-23	375 feet	0-13 feet	22-42 feet
MN-23A	380 feet	0-11 feet	30-40 feet
MN-24	380 feet	0-11 feet	22.5-32.5 feet
MN-25	375 feet	0-11 feet	27.5-47.5 feet
MN-26	335 feet	0-11 feet	20-30 feet
MN-27	320 feet	0-11 feet	20-30 feet
MN-28A	375 feet	0-11 feet	20-30 feet
MN-29A	365 feet	0-11 feet	20-30 feet
MN-30	340 feet	0-11 feet	20-30 feet
MN-31	320 feet	0-11 feet	21-41 feet
MN-32	335 feet	0-11 feet	12.5-22.5 feet
MN-33	330 feet	0-11 feet	12.5-22.5 feet

Reference: EBMUD, 2001 except for Top of Casing elevation estimated from previous topography (EBMUD, 2001)

5 GEOLOGIC AND SUBSURFACE CONDITIONS

5.1 Regional Geologic Setting

The WCWTP site is located near the northwestern end of the Diablo Range, which is part of the Coast Ranges geomorphic province of California. The Coast Ranges are comprised of several mountain ranges and structural valleys formed by tectonic processes commonly found around the Circum-Pacific belt. Basement rocks have been sheared, faulted, metamorphosed, and uplifted, and are separated by thick blankets of Cretaceous and Cenozoic sediments that fill structural valleys and line continental margins. The San Francisco Bay Area has several ranges that trend northwest, parallel to major strike-slip faults such as the San Andreas, Hayward, and Calaveras (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Figure 3 presents the location of the site relative to the epicenters of historic earthquakes with magnitudes of 5.5 or more from 1800 to 2000.

The WCWTP site is situated on the eastern flank of the Diablo Range overlooking Ygnacio Valley and Pleasant Hill to the east. There are several regional geologic maps that cover this area including those by Dibblee and Minch (2005); Graymer et al. (1994); Helley and Graymer (1997); Knudsen et al. (2000); Nilsen (1975); and Witter et al. (2006). The geologic information provided on the referenced geologic maps varies slightly, but, in general, indicates that the WCWTP site is underlain by Miocene age marine sedimentary rocks consisting of sandstone, siltstone, and shale. Quaternary age alluvial sediments, colluvial soils, and shallow landslide deposits are also present

in the drainage channels and slopes. A regional geologic map from Dibblee and Minch (2005) is provided as Figure 4.

Geologic formations depicted on the regional maps include the Miocene age Monterey Formation and Eocene age Domengene Formation of Dibblee and Minch (2005), which is mapped as Miocene age Neroly Formation, Briones Formation, Hambre Sandstone, and Eocene age Las Juntas Formation by Graymer et al. (1994). Both regional maps indicate that the WCWTP site is on the eastern limb of a northwesterly trending syncline, with bedding orientations striking to the northwest and dipping southwest toward the axis of the syncline. A branch of the Franklin fault, also referred to as the Southhampton fault, transects the northeast boundary of the site. The Eocene formations are exposed east of the Franklin fault beneath the adjoining residential neighborhood.

5.2 Site Geology

Previous geotechnical and seismic evaluations of the site include, but are not limited to, those by EBMUD (2001) and Geomatrix (1995, 2001a, and 2001b). Those evaluations indicate that the WCWTP site is underlain by sedimentary bedrock consisting of sandstone and siltstone with interbeds of shale and claystone described as Miocene age Neroly, Briones, and Hambre formations and the Eocene age Meganos Formation. Hardness and weathering of the rock units varies with depth and location. Surficial deposits present at the WCWTP site include alluvial and colluvial soils, and artificial fills placed during previous construction projects. A geologic map for the WCWTP site is provided as Figure 5. Figure 5 is based on mapping by Geomatrix (1998) with modifications to show additional fill depicted in historic aerial imagery (Google, 2021) that was placed near the proposed locations for Facilities 4, 5, 9, 14, and 15 during construction of the 4-MG chlorine contact chamber in 2002 and 2003. Historic aerial imagery also indicates that a temporary 80-foot-diameter above ground storage tank was constructed at the proposed location for the gravity thickeners, an area that was previously undeveloped, in 2002 and fill was stockpiled around and north of this tank beginning in 2003 during construction of the 16-MG clearwell. The temporary tank was demolished and the stockpiled fill was largely removed by 2007. The approximate extents of the area that was disturbed by construction activities between 2002 and 2007 around the proposed location for the gravity thickeners are noted on Figure 5. The formation names from the Geomatrix (1998) geologic map were used for this GIR. The correlation of formation names used in this report with the formation names from previous mapping is listed in Table 8.

Table 8 – Geologic Unit Correlations				
Unit Description	This GIR	Geomatrix (1998)	Dibblee & Minch (2005)	Graymer et al. (1994)
Artificial Fill	Fill	Fill	--	--
Alluvium (Holocene)	Qal	Qal	Qa	Qu
Terrace Deposits (Quaternary)	Qt	Qt	Qoa	Qu
Sandstone (Miocene)	Tmnu (Upper Neroly)	Tmnu (Upper Neroly)	Tms (Monterey)	Tn (Neroly)
Sandstone & Shale (Miocene)	Tmnl (Lower Neroly)	Tmnl (Lower Neroly)	Tmc (Monterey)	Tn (Neroly)
Shale (Miocene)	Tmbu (Upper Briones)	Tmbu (Upper Briones)	Tmc (Monterey)	Tbr (Briones)
Sandstone (Miocene)	Tmbl (Lower Briones)	Tmbl (Lower Briones)	Tmc (Monterey)	Tbr (Briones)
Fine-Grained Sandstone (Miocene)	Tmh (Hambre)	Tmh (Hambre)	Tms (Monterey)	Th (Hambre)
Claystone (Eocene)	Tem (Meganos)	Tem (Meganos)	Tds (Domengene)	Tli (Las Juntas)

5.3 Landslides

Regional landslide mapping by Nilsen (1975) shows several landslides on the slopes along the easterly, western, and northern portions of the site (Figure 6). The approximate limits of some of the landslides are shown on Figure 5. Ninyo & Moore observed some of the Nilsen landslides on historical aerial photographs. The photographs reviewed by Ninyo & Moore are listed in Section 9. The landslides mapped by Nilsen appear to be shallow debris flows, mudflows, and soil slumps. A small slump located on an east facing slope in the northern portion of the Project site was identified during the aerial photograph review. No new facilities are located within approximately 300-ft of the small slump shown on Figure 5.

Geomatrix (1998) also identified a landslide along the western side of the site on the natural slope that ascends from the perimeter access road (Figure 5). The slide appears to be a relatively shallow soil slump.

With the exception of the southern Intermediate Ozone Contactor Tank (Facility 3), the mapped landslides do not overlap with the proposed facilities. The southern Intermediate Ozone Contactor partially overlaps with the top of a mapped landslide extending down the adjacent slope to the southeast, but with the estimated pad grade for the concrete structure at 20 to 27 feet below the existing grade, it is anticipated that the structure will be on formational material, below the colluvium prone to sliding.

5.4 Faulting

The WCWTP site is transected by fault traces associated with the Franklin fault (Figures 3 and 4) that have been evaluated by Geomatrix (1995 and 2001a) and reviewed by William Lettis & Associates (WLA, 1999). The Franklin fault traces identified as the West, Central, and East faults (WLA, 1999) are considered to be active per dam safety standards established by the California Division of Safety Dams (DSOD), which classifies an active fault as one that has been active in the last 35,000 years. The approximate locations of the West, Central, and East faults are shown on Figure 5. The development of structures designed for human occupancy near active faults in California is regulated by the Alquist-Priolo Earthquake Fault Zoning Act (A-P Act). The A-P Act tasks the California Geological Survey (CGS) with establishing Earthquake Fault Zones to delineate regions of potential ground surface rupture adjacent to faults determined by CGS to be sufficiently active and well-defined as to constitute a potential hazard to structures from surface fault rupture or fault creep. The WCWTP site is not located within an Earthquake Fault Zone (CDMG, 1993). For purposes of the A-P Act, active faults are defined by CGS as faults that have caused surface displacement within Holocene time, or within approximately the last 11,700 years (CGS, 2018). The Franklin fault traces identified as the West, Central, and East faults (WLA, 1999) are not considered to be active per standards established by the A-P Act and the CGS.. The closest, mapped Earthquake Fault Zone is the one associated with the Concord fault, which is located 4 miles from the site to the northeast (CDMG, 1993).

5.5 Naturally Occurring Asbestos

Naturally occurring asbestos is more likely to be encountered in, and immediately adjacent to, outcrops of serpentinite and ultramafic rocks. Serpentinite and ultramafic rocks are not present at the WCWTP site. Regional mapping by Churchill and Hill (2000) indicate that no ultramafic rocks have been mapped in the general vicinity of the WCWTP site. Therefore, it is unlikely that significant concentrations of naturally occurring asbestos are present at the WCWTP site.

5.6 Subsurface Conditions

Subsurface investigations at the site encountered fill, native soil, and bedrock below existing pavement, where present. The fill, as encountered, generally consisted of stiff to hard, lean to fat clay, stiff to hard sandy silt, and medium dense to very dense silty sand. The native soil, described as colluvium and residual soil, generally consisted of stiff to hard, lean to fat clay, stiff sandy silt, and very dense clayey sand. The bedrock encountered in the borings generally consisted of friable to moderately strong, weathered, and weakly to strongly cemented sandstone, siltstone, shale, and claystone of the Hambre and Briones Formations.

The subsurface conditions encountered in borings from previous investigations are summarized in Table 9. The subsurface conditions encountered in the borings drilled for the GIR are summarized in Table 10. Detailed descriptions are presented on the logs in Appendix A and Appendix B. Anticipated subsurface conditions at the locations for the proposed facilities are described in the following subsections.

Table 9 – Summary of Subsurface Conditions in Previous Borings

Boring ID	Pavement Section	Depth Interval Below Ground Surface		
		Fill	Native Soil	Bedrock
MM-1	--	0.0-69.0 feet	--	69.0-76.0 feet
MM-2	--	0.0-24.0 feet	--	24.0-33.0 feet
MM-3	--	0.0-51.5 feet	--	51.5-65.5 feet
MM-4	--	0.0-31.0 feet	--	31.0-34.0 feet
MM-5	--	0.0-9.5 feet	--	9.5-14.0 feet
MN-2	--	--	0.0-1.8 feet	1.8-6.5 feet
MN-3	--	--	0.0-3.3 feet	3.3-23.5 feet
MN-4	3”AC/3”AB	--	0.5-3.0 feet	3.0-32.5 feet
MN-8	3”AC/3”AB	--	--	0.5-13.0 feet
MN-9	4”AC/6”AB	0.8-5.0 feet	5.0-13.5 feet	13.5-24.0 feet
MN-10	3”AC/3”AB	0.5-5.0 feet	--	5.0-18.0 feet
MN-11	3”AC/3”AB	--	0.5-6.0 feet	6.0-9.0 feet
MN-13	--	--	--	0.0-22.5 feet
MN-14	--	0.0-2.0 feet	2.0-4.0 feet	4.0-20.0 feet
MN-15	4”AC/4”AB	0.7-8.0 feet	8.0-22.5 feet	22.5-47.6 feet
MN-16	--	--	0.0-2.5 feet	2.5-42.5 feet
MN-17	--	--	0.0-4.0 feet	4.0-42.5 feet
MN-18	--	0.0-20.0 feet	--	20.0-82.0 feet
MN-19	--	0.0-5.0 feet	--	5.0-40.7 feet
MN-20	--	0.0-2.5 feet	--	2.5-7.0 feet
MN-21	12”AB	--	1.0-3.5 feet	3.5-35.0 feet
MN-22	--	0.0-20.0 feet	--	20.0-32.0 feet
MN-23	--	0.0-5.0 feet	5.0-35.0 feet	35.0-43.3 feet
MN-23A	--	0.0-20.0 feet	--	20.0-40.3 feet
MN-24	4”AC	--	0.3-10.0 feet	10.0-33.0 feet
MN-25	--	0.0-10.0 feet	--	10.0-48.0 feet
MN-26	4”AC/6”PCC	--	--	0.8-30.5 feet
MN-27	--	0.0-10.0 feet	10.0-20.0 feet	20.0-30.4 feet
MN-28A	--	0.0-13.5 feet	--	13.5-31.5 feet
MN-29A	2”AC	--	--	0.2-30.3 feet
MN-30	--	0.0-10.0 feet	--	10.0-31.5 feet
MN-31	--	--	0.0-5.0 feet	5.0-41.3 feet
MN-32	--	--	0.0-10.0 feet	10.0-23.0 feet
MN-33	--	--	--	0.0-20.0 feet
MN-34	--	--	--	0.0-14.3 feet

Reference: EBMUD, 2001

Table 10 – Summary of Subsurface Conditions in Borings by NInyo & Moore				
Boring ID	Pavement Section	Depth Interval Below Ground Surface		
		Fill	Native Soil	Bedrock
SB-1	--	0.0-12.0 feet	12.0-15.0 feet	15.0-28.0 feet
SB-2	--	0.0-38.0 feet	--	38.0-45.0 feet
NMB-1	--	--	--	0.0-26.5 feet
NMB-2	--	--	--	0.0-9.4 feet
NMB-3	4½"AC/5½"AB	0.8-28.0 feet	--	28.0-28.7 feet
NMB-4	--	0.0-39.0 feet	--	39.0-40.0 feet
NMB-5	--	0.0-6.0 feet	--	6.0-38.8 feet
NMB-6	--	0.0-18.0 feet	--	18.0-25.0 feet
NMB-7	--	0.0-18.0 feet	18.0-23.5 feet	23.5-35.0 feet
NMB-8	--	0.0-4.0 feet	--	4.0-10.8 feet
NMB-9	--	--	0.0-12.0 feet	12.0-26.0 feet

Source: Boring Logs in Appendix A

5.6.1 Ballasted Flocculation Basins (Facility 1)

The subsurface conditions exposed in Borings MN-8 and MN-2 indicate that the proposed locations for the Ballasted Flocculation Basins are underlain by soil consisting of silty clay and by sandstone and siltstone of the Hambre Formation.

5.6.2 Intermediate Ozone PP Vaults (Facility 2)

The subsurface conditions exposed in Boring MN-9 indicate that the proposed location for the northern Intermediate Ozone PP Vault is underlain by fill consisting of sandy clay over soil consisting of sandy silt, over sandstone, siltstone, and claystone of the Hambre Formation. The subsurface conditions exposed in Boring MN-2 indicate that the proposed location for the southern Intermediate Ozone PP Vault is underlain by soil consisting of silty clay, over siltstone of the Hambre Formation.

5.6.3 Intermediate Ozone Contactor Tanks (Facility 3)

The subsurface conditions exposed in Boring NMB-3 indicate that the proposed location for the northern Intermediate Ozone Contactor Tank is underlain by fill consisting of sand and clay over sandstone and siltstone of the Hambre Formation. Historic grading plans (EBMUD, 1964) indicate that the ground surface elevation at the proposed location for the northern Intermediate Ozone Contactor Tank was previously about 8 to 23 feet below the existing ground surface elevation.

The subsurface conditions exposed in Boring NMB-1 indicate that the proposed location for the southern Intermediate Ozone Contactor Tank is underlain by siltstone of the Hambre Formation. Historic grading plans (EBMUD, 1964) indicate that the ground surface elevation at the proposed location for the southern Intermediate Ozone Contactor Tank was previously about 1 to 14 feet above the existing ground surface elevation.

5.6.4 Ozone Generator Building (Facility 4)

The subsurface conditions exposed in Borings NMB-4 and NMB-6 indicate that the proposed location for the Ozone Generator Building is underlain by fill consisting of sand, clay, and gravel over sandstone and siltstone of the Briones Formation.

5.6.5 LOX Tanks & Vaporizers (Facility 5)

The subsurface conditions exposed in Borings NMB-4, SB-2, and MN-26 indicate that the proposed location for the LOX Tanks & Vaporizers is underlain by fill consisting of sand, clay, and gravel over sandstone, siltstone, and claystone of the Briones Formation.

5.6.6 Pre-Ozone PPs (Facility 6)

Geologic mapping of the WCWTP site (Figure 5) and the subsurface conditions exposed in Boring MN-30 indicate that the proposed locations for the two Pre-Ozone PPs are underlain by fill consisting of sandy lean clay over soil consisting of lean clay, over claystone and siltstone of the Briones Formation.

5.6.7 Combined Reclaim Metering Vault (Facility 7)

Geologic mapping of the WCWTP site (Figure 5) indicates that the proposed location for the Combined Reclaim Metering Vault is underlain by fill over shale of the Briones Formation.

5.6.8 Ozone Gas Destruct Canopy (Facility 8)

Geologic mapping of the WCWTP site (Figure 5) indicates that the proposed location for the western Ozone Gas Destruct Canopy is underlain by soil over shale of the Briones Formation. The other four Ozone Gas Destruct Canopies will be constructed over the Intermediate Ozone Contactors (Facility 3) or the Ozone Quenching Vaults (Facility 16).

5.6.9 Dewatering Building (Facility 9)

The subsurface conditions exposed in Boring NMB-5 indicate that the proposed location for the Dewatering Building is underlain by fill consisting of clay and clayey sand, over siltstone and sandstone of the Hambre Formation.

5.6.10 Truck Loading Canopy (Facility 9A)

The subsurface conditions exposed in Boring NMB-5 indicate that the proposed location for the Truck Loading Canopy abutting the south side of the Dewatering Building is underlain by fill consisting of clay and clayey sand, over siltstone and sandstone of the Hambre Formation.

5.6.11 Thickened Solids Blending Tanks (Facility 10)

The subsurface conditions exposed in Boring NMB-5 indicate that the proposed locations for the two Thickened Solids Blending Tanks abutting the west side of the Dewatering Building are underlain by fill consisting of clay and clayey sand, over siltstone and sandstone of the Hambre Formation.

5.6.12 Gravity Thickeners (Facility 11)

The subsurface conditions exposed in Borings NMB-5 and NMB-8 indicate that the proposed locations for the four Gravity Thickeners are underlain by fill consisting of sandy clay and clayey sand, over siltstone and sandstone of the Hambre Formation.

5.6.13 Thickened Solids PP Vaults (Facility 12)

The subsurface conditions exposed in Borings NMB-5 and NMB-8 indicate that the proposed locations for the two Thickened Solids PP Vaults are underlain by fill consisting of sandy clay and clayey sand, over siltstone and sandstone of the Hambre Formation.

5.6.14 Solids Transfer PP (Facility 13)

The subsurface conditions exposed in Boring MN-33 indicate that the proposed locations for the Solids Transfer PP is underlain by siltstone of the Hambre Formation.

5.6.15 Electrical Facilities (Facility 14)

The subsurface conditions exposed in Boring MN-15, drilled before fill was placed to raise the grade in the area during construction of the 4-MG chlorine contact chamber and 16-MG clearwell, indicate that the proposed location for the Electrical Facilities is underlain by fill consisting of sandy clay and clayey sand, over soil consisting of silty clay, over claystone, siltstone, and shale of the Briones Formation.

5.6.16 Maintenance Building (Facility 15)

The subsurface conditions exposed in Borings MN-16, NMB-7, and SB-1 indicate that the proposed location for the Maintenance Building is underlain by fill consisting of sandy clay and clayey sand, over soil consisting of fat clay, over siltstone of the Briones Formation.

5.6.17 Ozone Quenching Vaults (Facility 16)

Geologic mapping of the WCWTP site (Figure 5) indicates that the proposed location for the northern Ozone Quenching Vault is underlain by fill over sandstone of the Hambre Formation and the southern location of the Ozone Quenching Vault is underlain by sandstone of the Hambre Formation. Historic grading plans (EBMUD, 1964) indicate that the ground surface elevation at the proposed location for the northern Ozone Quenching Vault was previously about 21 to 25 feet below the existing ground surface elevation and the ground surface elevation at the proposed location for the southern Ozone Quenching Vault was previously about 9 to 14 feet above the existing ground surface elevation.

5.6.18 Chemical Storage Canopy (Facility 17)

The subsurface conditions exposed in Borings MN-21 and MN-28A indicate that the proposed location for the Chemical Storage Canopy is underlain by fill consisting of sandy clay, over siltstone and claystone of the Hambre Formation. Historic grading plans (EBMUD, 1964) indicate that the ground surface elevation at the proposed location for the Chemical Storage Canopy was previously about 15 to 25 feet below the existing ground surface elevation

5.6.19 Hydrogen Peroxide Station Canopies (Facility 18)

Geologic mapping of the WCWTP site (Figure 5) indicates that the proposed location for the western Hydrogen Peroxide Station Canopy is underlain by soil over shale of the Briones Formation. The subsurface conditions exposed in Boring NMB-2 indicate that the proposed location for the eastern Hydrogen Peroxide Station Canopy is underlain by siltstone of the Hambre Formation.

5.7 Groundwater

Groundwater levels were measured in seven of the piezometers developed by Olivia Chen Consultants (EBMUD, 2001). Where measured, the depth to groundwater ranged between approximately 4 and 39 feet. A summary of the groundwater levels measured in the piezometers over four days in 2001 is provided in Table 11. The borings included in Appendix A were drilled using mud rotary wash techniques which generally obscures and precludes evaluation of groundwater levels during drilling. Groundwater was not encountered in the solid stem auger borings performed for the current study. Monitoring records (Geomatrix, 2001) for the four piezometers (WC-39 to WC-42) installed in the embankment of the previous open cut clearwell are included in Appendix G. The records indicate that over a 10-year period ending in February of 2001, the groundwater levels in the piezometers generally ranged between approximately 35 and 70 feet below the water level in the reservoir which was generally at an elevation of about 370 feet.

Table 11 – Piezometer Measurements

Boring ID	Depth to Groundwater Below Ground Surface [Groundwater Elevation] in feet			
	1-22-2001	3-9-2001	4-5-2001	5-17-2001
MN-23	Not Available	31.0 [344.0]	39.0 [336.0]	dry
MN-23A	dry	dry	dry	dry
MN-24	dry	dry	dry	dry
MN-25	23.0 [352.0]	23.4 [351.6]	23.4 [351.6]	23.9 [351.1]
MN-26	6.0 [329.0]	21.0 [314.0]	4.4 [330.6]	5.7 [329.3]
MN-27	21.0 [299.0]	12.6 [307.4]	9.3 [310.7]	13.1 [306.9]
MN-28A	dry	25.5 [349.5]	24.8 [350.2]	25.4 [349.6]
MN-29A	11.5 [353.5]	3.9 [361.1]	5.3 [359.7]	7.6 [357.4]
MN-30	dry	dry	dry	dry
MN-31	dry	dry	dry	dry
MN-32	dry	dry	dry	dry
MN-33	dry	13.0 [317.0]	dry	dry

Reference: EBMUD, 2001 except for groundwater elevation estimated from previous topography (EBMUD, 2001)

Groundwater may rise to a higher elevation than was encountered in the exploratory borings due to the short time available for seepage of water into the borings. Furthermore, groundwater levels may fluctuate in response to seasonal variations in precipitation, nearby groundwater pumping or dewatering, changes in irrigation practices adjacent to or within the Project area, or other factors. In addition, seeps may be encountered at elevations above the observed groundwater levels due to perched groundwater conditions, leaking pipelines, preferential drainage, or other factors not evident at the time of exploration.

5.8 Liquefaction and Strain Softening

The strong vibratory motions generated by earthquakes can trigger a rapid loss of shear strength in saturated, loose, granular soils of low plasticity (liquefaction) or in wet, sensitive, cohesive soils (strain softening). Liquefaction and strain softening can result in a loss of foundation bearing capacity or lateral spreading of sloping or unconfined ground. Liquefaction can also generate sand boils leading to subsidence at the ground surface. Liquefaction (or strain softening) is generally not a concern at depths more than 50 feet below ground surface. The WCWTP site is in an area that has not yet been evaluated for liquefaction seismic hazard zones by the California Geological Survey. Regional studies of liquefaction susceptibility by Knudsen et al. (2000) and Witter et al. (2006) indicate that the WCWTP site is in an area where the liquefaction susceptibility is considered to be very low.

The subsurface exploration for the GIR and previous investigations encountered fill and native soil that generally consisted of medium to very dense silty and clayey sand, and stiff to hard clay. Saturated loose granular soil was not encountered, and the cohesive soil encountered was not wet or particularly sensitive. Based on these findings, liquefaction, seismic strain softening and related hazards such as dynamic settlement, lateral spreading, sand-boil-induced ground subsidence, or a reduction in foundation bearing capacity due to liquefaction, are not design considerations at the WCWTP site.

5.9 Corrosive/Deleterious Soil

An evaluation of the corrosivity of the on-site material was conducted to assess the impact to concrete and metals. The corrosion impact was evaluated using the results of limited laboratory testing on samples obtained during our subsurface study. Laboratory testing to quantify pH, redox potential, electrical resistivity at in-situ and saturated conditions, sulfide content, chloride content, and soluble sulfate content was performed on selected samples collected during the subsurface exploration for this study. The results of the corrosivity tests are presented in Appendix C.

The California Department of Transportation (Caltrans) has developed guidelines (Caltrans, 2018) to assess the need for special measures to mitigate corrosion in corrosive environments for structural elements exposed to soil. The American Concrete Institute (ACI) has developed criteria to classify the degree of sulfate exposure for concrete in contact with soil so that concrete can be specified in accordance with the California Building Code to mitigate concrete deterioration by sulfate attack.

Caltrans defines a corrosive environment for structures as an area where the soil has a chloride concentration of 500 parts per million (ppm) or greater, soluble sulfate concentration of 0.15 percent (1,500 ppm) or greater, or a pH of 5.5 or less (Caltrans, 2018). The ACI criteria used to evaluate the deleterious nature of sulfate in soil on concrete are listed in Table 12. Based on these criteria and the results of the testing, site soil does not meet the definition of a corrosive environment for structures, and the sulfate exposure to concrete is negligible to moderate with an exposure classification for sulfate of S0 and S1. The samples tested indicate that the site soil is corrosive to ferrous metals based on the resistivity test results for saturated conditions and slightly corrosive based on the redox potential as noted in Appendix C.

Table 12 – Criteria for Soil Deleterious to Concrete

Sulfate Content Percent by Weight	Sulfate Exposure	Exposure Class
0.0 to 0.1	Negligible	S0
0.1 to 0.2	Moderate	S1
0.2 to 2.0	Severe	S2
> 2.0	Very Severe	S3

Reference: American Concrete Institute (ACI) Committee 318 Table 19.3.1.1 (ACI, 2016)

5.10 Expansive Soils

Some clay minerals undergo volume changes upon wetting or drying. Unsaturated soils containing those minerals will shrink/swell with the removal/addition of water. The heaving pressures and differential movement associated with expansion and changes in soil moisture can damage structures and flatwork. Laboratory testing was performed on selected samples of the near-surface soil to evaluate the expansion index. The tests were performed in general accordance with the American Society of Testing and Materials (ASTM) Standard D 4829 (Expansion Index) on a sample of the near-surface Hambre formation at Boring NMB-2 and a sample of near-surface fill at Boring NMB-6. The results of the laboratory testing indicate that the Expansion Index of the near-surface soil at those locations ranges from 24 to 39, which is consistent with a low expansion characteristic. The results of the Expansion Index tests are presented in Appendix C.

Additional testing was performed as part of the GIR and previous investigations to evaluate Atterberg Limits and Plasticity Index in accordance with ASTM D 4318. Three of 54 tests performed on fill samples, 4 of 11 tests performed on soil samples, 6 of 8 tests performed on samples of Briones Formation, and 1 of 9 tests performed on samples of Hambre Formation indicated that the Plasticity Index was 35 or more which is consistent with a very high expansion characteristic. In addition, 69 percent of tests on fill, 91 percent of tests on soil, 100 percent of

tests on Briones Formation, and 44 percent of tests on Hambre Formation indicated that the Plasticity Index exceeded 15 which is indicative of a moderate to very high expansion characteristic. These results indicate that layers of moderately to very highly expansive soil are present at the WCWTP site which is a consideration for pad grading using on-site materials at structures located within a few feet of finish grade where seasonal variations in soil moisture are anticipated.

The Expansion Index testing performed as part of this study indicate that the existing subgrade conditions at the Ozone Generator Building (Facility 4) and the eastern Hydrogen Peroxide Station (Facility 18) consist of material with a low expansion characteristic. The Ozone Generator Building may require additional fill to achieve finish pad grade.

The findings from Borings NMB-4 and SB-2 indicate that the existing subgrade conditions at the LOX Tanks & Vaporizers (Facility 5) consists of clayey sand which is indicative of a low to moderate expansion characteristic, but it is expected that additional fill will be needed to achieve finish pad grade.

The findings from Borings MN-30, MN-16, and MN-21 indicate that the existing subgrade conditions at the Pre-Ozone PPs (Facility 6 both locations), Maintenance Building (Facility 15), and Chemical Storage Canopy (Facility 17) consist of clay which may be indicative of a high expansion characteristic. With the exception of the western Pre-Ozone PP, these locations are also expected to receive some additional fill to achieve finish pad grades.

The findings from Boring NMB-5 indicate that the subgrade conditions at the Dewatering Building (Facility 9), Truck Canopy (Facility 9A), and Thickened Solids Blending Tanks (Facility 10) may include clay soil and siltstone of the Hambre Formation with may have a moderate to very high expansion characteristic.

The subgrade conditions are not known, or the corresponding expansion characteristic is not discernable at the Solids Transfer PP (Facility 13), the Electrical Facilities (Facility 14), and the western Hydrogen Peroxide Station Canopy and Ozone Gas Destruct Canopy (Facilities 18 and 8, respectively). The Solids Transfer PP and the Electrical Facilities may require additional fill to achieve finish pad grades.

Due to the depth of foundation embedment, shrink and swell of expansive soil subgrade is not a concern for the other proposed facilities.

5.11 Seismic Ground Motion and Site Classification

Based on historical seismic activity, the potential for future seismic ground motion at the site is considered significant. Seismic design criteria to address ground shaking are provided in Section 7.1. The peak ground acceleration (PGA) associated with the Maximum Considered Earthquake Geometric Mean (MCE_G) was calculated in accordance with the American Society of Civil Engineers (ASCE) 7-16 Standard and the 2019 California Building Code (CBC). The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.783g using the seismic design tool developed by the Structural Engineers Association of California in conjunction with the Office of Statewide Health Planning and Development (SEAOC/OSHPD, 2021). The PGA_M is based on a mapped MCE_G peak ground acceleration of 0.712g for the site and a site coefficient (F_{PGA}) of 1.1 for Site Class D. Site Class D was selected based on the results of the geophysical survey performed at the site using the Refraction Microtremor method (Appendix E) with an average shear wave velocity computed from the survey of 1,136 feet per second (fps) over a depth of 100 feet (Vs100).

5.12 Fill Suitability

Historic grading plans (EBMUD, 1964), geologic mapping of the WCWTP site (Figure 5), and subsurface exploration conducted for the GIR and previous investigations (EBMUD, 2001) indicates that several of the proposed facilities will be located over areas that are presently underlain by fill. The proposed facilities located over areas underlain by old fill include the Pre-Ozone PPs (Facility 6, both locations), the Maintenance Building (Facility 15), the Electrical Facilities (Facility 14), the Ozone Generator Building (Facility 4), the LOX Tanks & Vaporizers (Facility 5), the northern location of the Intermediate Ozone Contactor (Facility 3), the northern location of the Ozone Quenching Vault (Facility 16), and the Chemical Storage Canopy (Facility 17). Previous investigations (EBMUD, 2001) concluded that the existing fill derived from excavated overburden during WCWTP construction was unsuitable as a foundation material for the clearwell tank, chlorine contact chamber, chemical storage building, south filters, and blower building due to the potential for organic material in the fill.

As part of the GIR, Ninyo & Moore performed subsurface exploration that included sonic borings near the Maintenance Building (Facility 15) and the LOX Tanks (Facility 5), and auger borings near the Maintenance Building (Facility 15), LOX tanks (Facility 5), the Ozone Building (Facility 4), and the northern location of the Intermediate Ozone Contactor (Facility 3) to evaluate the suitability of the old fill and the need for mitigation by removal and replacement. The continuous cores recovered from the sonic borings were reviewed. The cores included scattered rootlets, wood chips, and grasses, but did not include continuous layers of organic material. Bulk soil

samples collected from the cores were tested to evaluate the organic content of the soil. Relatively undisturbed soil samples collected from the auger borings were tested to evaluate consolidation characteristics. The results of the organic content tests indicate that the organic content of the fill is less than 3 percent which indicates that the organic content is not significant and the existing on-site fill may be considered a mineral soil instead of an organic soil (15 percent organic content) or highly organic soil (30 percent organic content). The results of the consolidation tests indicate that the fill is relatively stiff and exhibits little swelling or compression on wetting. Based on these findings, the existing fill near the proposed locations for the Maintenance Building (Facility 15), LOX Tanks (Facility 5), and Ozone Building (Facility 4) may be considered a suitable bearing material for the proposed structures supported on footings or mat foundations. The surface topography and subsurface conditions encountered in Boring NMB-3 at the northern location of the Intermediate Ozone Contactor (Facility 3) indicate that the old fill will be essentially removed by excavation to get to finish pad elevation for the facility.

Subsurface exploration performed by Ninyo & Moore for this GIR encountered fill in Borings NMB 5 and NMB-8 indicating that the proposed locations for the Dewatering Building (Facility 9), the Truck Loading Canopy (Facility 9A), the Thickened Solids Blending Tanks (Facility 10), the Gravity Thickener Tanks (Facility 11), and the Thickened Solids PP Vaults (Facility 12), in an area not evaluated by EBMUD in the 2001 investigation, are also underlain by old fill. Historic aerial imagery (Google, 2021) indicates that the proposed locations for these facilities are in an area that was disturbed by construction activities between 2002 and 2007 (Figure 5). Documentation that the existing fill in this area was placed as engineered fill is not available. Due to the potential for inclusion of deleterious materials and variable or deficient support characteristics in the undocumented fill, and uncertainties regarding the thickness of the fill, the existing fill in this area should be considered, based on the available information, an unsuitable bearing material in the present state. It is anticipated, based on the assumed pad elevations and the subsurface conditions encountered in Borings NMB 5 and NMB-8, that the old undocumented fill will be removed by the excavation to achieve the assumed pad elevations for the Gravity Thickener Tanks (Facility 11) and the Thickened Solids PP Vaults (Facility 12) but some undocumented fill will remain below the pad elevations for the Dewatering Building (Facility 9), Truck Loading Canopy (Facility 9A), and Thickened Solids Blending Tanks (Facility 10).

Additional exploration may be performed to re-evaluate the suitability of the existing fill at the proposed locations for the Pre-ozone PPs (Facility 6, both locations), the Dewatering Building (Facility 9), the Truck Loading Canopy (Facility 9A), the Thickened Solids Blending Tanks (Facility 10), the Electrical Facilities (Facility 14), the northern location of the Ozone Quenching Vault (Facility 16), and the Chemical Storage Canopy (Facility 17). If the additional exploration

indicates that the fill is indeed suitable, these facilities may be supported on footings or mat foundations. If additional exploration is not performed or if the fill is found to be unsuitable, these facilities may be supported on deep footings or drilled pier foundations with a structural slab or additional piers to support tanks or other equipment. Alternatively, these facilities may be supported on footings or mat foundations provided that the existing fill is removed and replaced with compacted, engineered fill.

5.13 Slope Stability

Ninyo & Moore performed an analysis to evaluate the impact of the proposed construction on existing slopes. The analysis included an evaluation of the following slopes:

1. Global stability of the wall retaining the cut slope south of Staging Area 1 based on the conditions encountered in Boring NMB-9.

2. Global stability of the wall retaining fill on the slope northeast of northeast Gravity Thickener Tank (Facility 11) based on the conditions encountered in Boring NMB-8.

3. Stability of the slope northeast of the Maintenance Building (Facility 15) under surcharge from the new building and proposed fill based on the conditions encountered in Boring SB-1.

Slope stability and global wall stability were evaluated using GSTABL7 (Gregory, 2003), a two-dimensional slope stability analysis program, to compute the factor of safety, based on the Modified Bishop limit equilibrium method, for circular failure surfaces. The parameters assumed for the analysis are summarized in Table 13. The slopes were evaluated for static conditions. The slope northeast of the Maintenance Building (Facility 15) was also evaluated for stability under seismic conditions using a pseudo-static analysis with a seismic coefficient (k_h) of 0.29. The seismic coefficient was evaluated in accordance with the screening analysis procedure developed by the Southern California Earthquake Center (SCEC, 2002) for a displacement threshold of 2 inches and a maximum horizontal acceleration with a 10 percent probability of exceedance in 50 years, corrected for soft rock site conditions (MHA_r), of 0.645g corresponding to an earthquake magnitude and rupture distance of 6.49 and 7.07 kilometers, respectively. The Unified Hazard Tool (USGS, 2021) was used to evaluate the MHA_r and to perform a deaggregation analysis to evaluate the corresponding earthquake magnitude and rupture distance.

The results of the analysis, presented on Figures 7A through 7D and summarized in Table 13, indicate that the stability of the analyzed slopes, as modified by the proposed construction, is adequate based on a computed factor of safety meeting or exceeding 1.5 for static conditions and meeting or exceeding 1.0 for pseudo-static (seismic) conditions.

Table 13 – Summary of Stability Analysis	
Analysis	**Factor of Safety**
Wall supporting 18-foot high cut in 2.5:1 (horizontal:vertical) slope south of Staging Area 1. Slope consists of colluvium (ϕ' = 26°, c' = 200 psf) 12-feet thick over siltstone of the Briones Formation (ϕ' = 32°, c' = 450 psf)	2.2 static
Wall supporting 17-foot of new fill (ϕ' = 32°, c' = 200 psf) on 3:1 over 2:1 (H:V) slope northeast of gravity thickener tanks. Slope consists of old fill/colluvium (ϕ' = 26°, c' = 200 psf) 12-feet thick over sandstone of the Hambre Formation (ϕ' = 32°, c' = 450 psf)	1.6 static
New fill embankment (ϕ' = 32°, c' = 200 psf) 9 feet high with 7.5:1 (H:V) side slope and 100 psf surcharge for Maintenance Building on old fill embankment (ϕ' = 32°, c' = 200 psf) that is 25 feet high with a 1.8:1 (H:V) side slope northeast of the Maintenance Building Wall. Old fill embankment on 5 feet of colluvium (ϕ' = 26°, c' = 200 psf) over siltstone of the Briones Formation (ϕ' = 32°, c' = 450 psf) inclined down to northeast at 6:1 (H:V).	1.5 static
New fill embankment (ϕ' = 32°, c' = 200 psf) 9 feet high with 7.5:1 (H:V) side slope and 100 psf surcharge for Maintenance Building on old fill embankment (ϕ' = 17°, c' = 1700 psf) that is 25 feet high with a 1.8:1 (H:V) side slope northeast of the Maintenance Building Wall. Old fill embankment on 5 feet of colluvium (ϕ' = 0°, c' = 1500 psf) over siltstone of the Briones Formation (ϕ' = 32°, c' = 450 psf) inclined down to northeast at 6:1 (H:V).	1.2 seismic (k_h = 0.29)

6 CONCLUSIONS

The information presented in this GIR demonstrates that the proposed improvements are geotechnically feasible provided that the improvements are designed to accommodate the geotechnical conditions at the improvement locations. Conclusions regarding the geotechnical conditions at the WCWTP site, based on the information presented in this GIR, are as follows:

- Geologic mapping at the WCWTP site (Figure 5) and subsurface exploration from the GIR and previous investigations indicate that the proposed facilities will be underlain by fill, native soil described as colluvium, alluvium, or residual soil, and bedrock of Miocene age. As encountered in the borings, the fill generally consisted of stiff to hard, lean to fat clay, stiff to hard sandy silt, and medium dense to very dense silty sand; the native soil generally consisted of stiff to hard, lean to fat clay, stiff sandy silt, and very dense clayey sand; and the bedrock generally consisted of friable to moderately strong, weathered, and weakly to strongly cemented sandstone, siltstone, shale, and claystone of the Hambre and Briones Formations.

- Previous investigations (EBMUD, 2001) concluded that existing fill at WCWTP site was unsuitable as a foundation material due to the potential for organic material in the fill. As part of the GIR, Ninyo & Moore performed subsurface exploration that included sonic borings near the Maintenance Building (Facility 15) and the LOX Tanks (Facility 5), and auger borings near the Maintenance Building (Facility 15), LOX Tanks (Facility 5), Ozone Building (Facility 4), and the northern location of the Intermediate Ozone Contactor (Facility 3) to evaluate the suitability of the old fill and the need for mitigation (Figure 2). The findings from this subsurface exploration indicate the old fill at the northern location of the Intermediate Ozone Contactor (Facility 3) will be essentially removed by excavation to get to finish pad elevation for the facility and the existing fill near the proposed locations for the Maintenance Building (Facility 15), LOX Tanks (Facility 5), and Ozone Building (Facility 4) may be considered a

suitable bearing material for foundations. The suitability of the existing fill at the proposed locations for the Pre-ozone PPs (Facility 6, both locations), the Electrical Facilities (Facility 14), the northern location of the Ozone Quenching Vault (Facility 16), and the Chemical Storage Canopy (Facility 17) was not specifically evaluated.

- Ninyo & Moore encountered fill in Borings NMB-5 and NMB-8 near the proposed locations for the Dewatering Building (Facility 9), the Truck Loading Canopy (Facility 9A), and the Thickened Solids Blending Tanks (Facility 10) in an area that was not evaluated by EBMUD in the 2001 investigation. Historic aerial imagery (Google, 2021) indicates that the proposed locations for these facilities are in an area that was disturbed by construction activities between 2002 and 2007 (Figure 5). The fill in this area lacks documentation of placement as an engineered fill and is unsuitable as a foundation material under these facilities due to the potential for inclusion of deleterious materials, variable or deficient support characteristics, and uncertainties regarding the thickness of the fill. The proposed locations for the Gravity Thickener Tanks (Facility 11) and Thickened Solids PP Vaults (Facility 12) are also in the area disturbed by construction activities between 2002 and 2007 but it is anticipated that the old undocumented fill at the proposed locations for those facilities will be removed by excavation for the below grade structures based on the assumed finish pad elevations and subsurface conditions encountered in Borings NMB-5 and NMB-8.

- Groundwater was not encountered in the borings for this GIR which extended to depths of approximately 10 to 45 feet below the ground surface. Geomatrix (2001) reported groundwater levels at the WCWTP site that ranged between elevations of 300 and 335 feet or about 35 to 70 feet below the water level in the previous open cut clearwell that was removed in 2004 based on piezometers installed in the embankments of the clearwell. EBMUD (2001) reported groundwater levels that ranged between approximately 4 and 39 feet below the ground surface corresponding to elevations between approximately 299 and 361 feet based on piezometers developed from Borings MN-23, MN-25, MN-26, MN-27, MN-28A, MN-29A, and MN-33 (Figure 2). Variation and fluctuation in groundwater levels should be anticipated as discussed in Section 5.7.

- Based on historic seismicity (Figure 3), the WCWTP site could experience significant ground shaking during an earthquake. All new facilities should be designed per applicable Seismic Design Criteria presented in Section 7.1 of this GIR.

- The WCWTP site is not within an A-P Act Earthquake Fault Zone established by CGS to delineate regions of potential ground surface rupture adjacent to faults determined by CGS to be sufficiently active and well-defined as to constitute a potential hazard to structures designed for human occupancy. The WCWTP site is transected by fault traces associated with the Franklin Fault but the Franklin Fault is not considered Holocene active (Jennings & Bryant, 2010) with displacement in the last 11,700 years and therefore does not meet the standard established by CGS for a fault that is sufficiently active to constitute a hazard to structures designed for human occupancy by surface fault rupture or fault creep.

- The WCWTP site is in an area characterized by very low susceptibility to liquefaction based on regional studies (Knudsen et al., 2000; Witter et al., 2006) and soil susceptible to liquefaction was not encountered during subsurface exploration for the GIR and previous investigations indicating the liquefaction and related hazards (e.g., lateral spreading, sand-boil-induced ground subsidence, dynamic settlement, and reduction in foundation bearing capacity due to liquefaction) are not considerations for the proposed improvements.

- Several shallow landslides interpreted from aerial photographs, included on regional landslide maps (Nilsen, 1975), or identified during previous geologic mapping at the WCWTP site (Geomatrix, 1998) are noted on Figure 5. With the exception of the southern Intermediate Ozone Contactor Tank (Facility 3), the mapped landslides do not overlap with the proposed facilities. The southern Intermediate Ozone Contactor overlaps with a mapped landslide extending down the adjacent slope to the southeast but with the estimated pad grade for the tank at 20 to 27 feet below the existing grade, it is anticipated that the tank will be on formational material, below the colluvium prone to sliding.

- Ninyo & Moore performed an analysis to evaluate the impact of the proposed construction on existing slopes. The analysis included an evaluation of the wall retaining the cut slope south of Staging Area 1, the wall retaining fill on the slope northeast of the Gravity Thickener Tank (Facility 11), and the slope northeast of the Maintenance Building (Facility 15) under surcharge from the new building and proposed fill embankment. The results of the analysis, presented on Figures 7A through 7D, indicate that the stability of the analyzed slopes, as modified by the proposed construction, is adequate under static and seismic conditions.

- Significant concentrations of naturally occurring asbestos (NOA) are unlikely to be encountered at the WCWTP site because rocks that typically contain NOA are not present at the site based on regional mapping and the exploration conducted for this GIR.

- Laboratory testing performed for the GIR and previous investigations indicates that expansion characteristic of the on-site fill, soil, and formational material ranges from very low to very high. These results indicate that layers of moderately to very highly expansive soil are present at the WCWTP site which is a consideration for pad grading using on-site materials at structures located within a few feet of finish grade.

- Laboratory corrosion testing of soil samples collected during the subsurface exploration for this GIR indicates that the site does not meet the definition of a corrosive environment for structures (Caltrans, 2018) and the sulfate exposure to concrete is negligible to moderate with an exposure classification for sulfate of S0 and S1. The test results indicate that he samples tested may be considered corrosive to ferrous metals based on the electrical resistivity for saturated conditions and slightly corrosive based on the redox potential testing as noted in Appendix C. Buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel should be appropriately protected against corrosion depending on the importance or expected service life of the element. A corrosion engineer may be consulted to provide recommendations to mitigate corrosion.

7 RECOMMENDATIONS

The following sections present geotechnical recommendations for consideration during project planning. These recommendations should be reviewed by the Geotechnical Engineer-of-Record (GEOR) and modified, as appropriate, to reflect actual structure dimensions, updated loads, or additional information from supplemental investigations.

7.1 Seismic Design Criteria

Table 14 presents the Risk-Targeted, Maximum Considered Earthquake (MCE_R) spectral response accelerations consistent with the 2019 California Building Code and corresponding site-adjusted and design level spectral response accelerations based on the USGS seismic design maps (SEAOC/OSHPD, 2021). As discussed in Section 5.11, a geophysical survey using the Refraction Microtremor method (Appendix E) was performed as part of this GIR to evaluate seismic site classification. The values provided in the table may be used for seismic design provided that the seismic response coefficient is calculated from equation 12.8-2 of ASCE Standard 7-16 for structures with a fundamental period of 0.87 seconds or less in accordance with Exception 2 in Section 11.4.8 of ASCE 7-16 and presuming that the proposed structures do not include seismic-isolation or damping systems.

Table 14 – California Building Code Seismic Design Criteria

Seismic Design Parameter Evaluated for 37.9136ºNorth latitude, 122.0829ºWest longitude	Value
Site Class	D
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.7
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.751g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.600g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.751g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.020g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.167g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.680g
Seismic Design Category for Risk Category I, II, III, or IV	D

Reference: 2019 California Building Code and SEAOC/OSHPD, 2021

7.2 Foundations

Recommended foundations for the proposed facilities are provided in the following subsections. The estimated foundation loads assumed for the analysis, based on the facility type descriptions and dimensions in the BODR (EBMUD, 2020), are summarized in Table 4. Foundation recommendations should be re-evaluated if the actual foundation loads differ significantly from the assumed values.

7.2.1 Ballasted Flocculation Basins (Facility 1)

For a finish pad elevation of approximately 24 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings MN-8 and MN-2, that the Ballasted Flocculation Basins will be underlain by formational material and these four structures may be supported on mat foundations.

7.2.2 Intermediate Ozone PP Vaults (Facility 2)

For a finish pad elevation of approximately 14 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings MN-9 and MN-2, that the Intermediate Ozone PP Vaults will be underlain by formational material and these two structures may be supported on mat foundations.

7.2.3 Intermediate Ozone Contactor Tanks (Facility 3)

For a finish pad elevation of approximately 21 to 26 feet below the existing ground surface at the location for the northern Intermediate Ozone Contactor Tank, it is anticipated, based on historic grading plans (EBMUD, 1964) and the subsurface conditions exposed in Boring NMB-3, that the northern tank will be underlain predominantly by formational material with a thin wedge of old fill up to approximately 3 feet thick under the western corner of the tank.

For a finish pad elevation of approximately 20 to 37 feet below the existing ground surface at the location for the southern Intermediate Ozone Contactor Tank, it is anticipated, based on historic grading plans and the subsurface conditions exposed in Boring NMB-1, that the southern tank will be underlain by formational material. Based on the anticipated bearing conditions, these two structures may be supported on mat foundations.

7.2.4 Ozone Generator Building (Facility 4)

For a finish pad elevation that is approximately 1 foot below to 17 feet above the existing ground surface at the location for the Ozone Generator Building, it is anticipated, based on the subsurface conditions exposed in Borings NMB-4 and NMB-6, that the building will be underlain by approximately 0 to 17 feet of new fill over approximately 8 to 20 feet of old fill and 5 feet of soil with a depth to formational material of approximately 15 to 42 feet. As part of the GIR, Ninyo & Moore performed subsurface exploration and laboratory testing to evaluate the suitability of the old fill at the Ozone Generator Building and concluded that the old fill is a suitable bearing material for foundations based on the results of the exploration and testing. Based on the anticipated bearing conditions, the Ozone Generator Building may

be supported on footings. Alternatively, the Ozone Generator Building may be supported on a mat foundation. The mat foundation may be preferred where concrete pads for tanks or other equipment would be needed over large portions of the floor area. The new fill at the northern corner of the pad will be supported by a retaining wall. The retaining wall at the northern corner of the pad may be eliminated if the foundation wall for the Ozone Generator Building is designed to resist the lateral earth pressures from the fill used to build the building pad.

The ground conditions encountered in Borings NMB-4 and NMB-6, indicate that the existing subgrade material at the proposed location for the Ozone Generator Building consists of clayey sand which is indicative of a low expansion characteristic. To mitigate concerns related to expansive soil, new fill placed within 2 feet of finish pad grade should consist of select site fill or import fill with a low expansion characteristic. Chemical treatment of suitable on-site material with quicklime can be performed to create fill with a low expansion characteristic as an alternative to identifying select site fill or using import fill.

7.2.5 LOX Tanks & Vaporizers (Facility 5)

For a finish pad elevation that is approximately 2 foot below to 5 feet above the existing ground surface at the location for the LOX Tanks & Vaporizers, it is anticipated, based on the subsurface conditions exposed in Borings NMB-4, SB-2, and MN-26, that the facility will be underlain by approximately 0 to 5 feet of new fill over approximately 28 to 37 feet of old fill and 6 to 7 feet of soil with a depth to formational material of approximately 34 to 49 feet. As part of the GIR, Ninyo & Moore performed subsurface exploration and laboratory testing to evaluate the suitability of the old fill at the LOX Tanks & Vaporizers and concluded that the old fill is a suitable bearing material for foundations based on the results of the exploration and testing. Based on the anticipated bearing conditions, the facility may be supported on footings. Alternatively, the facility may be supported on a mat foundation. The mat foundation may be preferred where concrete pads for tanks or other equipment would be needed over large portions of the floor area. The new fill at the northern corner of the pad will be supported by a retaining wall. The retaining wall at the northern corner of the pad may be eliminated if the foundation wall for the facility is designed to resist the lateral earth pressures from the fill used to build the building pad.

The ground conditions encountered in Borings NMB-4, SB-2, and MN-26, indicate that the existing subgrade material at the proposed location for the LOX Tanks & Vaporizers consists of clayey sand which is indicative of a low expansion characteristic. To mitigate concerns related to expansive soil, new fill placed within 2 feet of finish pad grade should consist of select site fill or import fill with a low expansion characteristic. Chemical treatment of suitable on-site material with quicklime can be performed to create fill with a low expansion characteristic as an alternative to identifying select site fill or using import fill.

7.2.6 Pre-Ozone PPs (Facility 6)

For finish pad elevations that are approximately 3 feet below to 3 feet above the existing ground surface at the two locations for the Pre-Ozone PPs, it is anticipated, based on geologic mapping of the site (Figure 5) and the subsurface conditions exposed in Boring MN-30, that the PPs will be underlain by new fill (at the east location), old fill (at both locations) and soil over formational material. Approximately 5 feet of sandy clay fill was encountered at Boring MN-30 over 5 feet of lean clay soil on claystone of the Briones Formation. Ninyo & Moore did not evaluate the suitability of the old fill at the Pre-Ozone PPs as part of the GIR because the existing untreated water pipelines, that are adjacent to the proposed locations for the PPs could not be located by ground penetrating radar or electromagnetic scanning so proposed exploration at the PPs was abandoned to avoid potential damage to the untreated water pipelines. Previous investigations (EBMUD, 2001) concluded that the old fill was unsuitable as a foundation material due to the potential for organic material in the old fill. Based on the anticipated bearing conditions, the two Pre-Ozone PPs may be supported on footings or a mat foundation provided that the pad is over-excavated to remove the old fill before new fill is placed and compacted in lifts to achieve finish pad grade. The mat foundation may be preferred where concrete pads for pumps or other equipment would be needed over large portions of the floor area.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of select site fill or import fill with a low expansion characteristic. Chemical treatment of suitable on-site material with quicklime can be performed to create a 2-foot thick layer with low expansion characteristics as an alternative to identifying select site fill or using import fill. The ground conditions encountered in Boring MN-30 indicate that the existing subgrade material consists of clay and sandy clay which is consistent with a moderate to high expansion characteristic so additional removal or in-place chemical treatment may be needed where the old fill is within 2 feet below the finish pad grade.

7.2.7 Combined Reclaim Metering Vault (Facility 7)

For a finish pad elevation of approximately 18 feet below the existing ground surface, it is anticipated, based on geologic mapping of the WCWTP site (Figure 5), that the Combined Reclaim Metering Vault will be underlain by formational material and may be supported on a mat foundation.

7.2.8 Ozone Gas Destruct Canopies (Facility 8)

For a finish pad elevation of approximately 5 to 14 feet below the existing ground surface, it is anticipated, based on geologic mapping of the WCWTP site (Figure 5), that the western Ozone Gas Destruct Canopy will be underlain by soil or formational material. Based on the anticipated bearing conditions, the western Ozone Gas Destruct Canopy may be supported on footings. Alternatively, where concrete equipment pads are needed over large portions of the canopy area, a mat foundation may be used to support the canopy and facility equipment on a common foundation.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime.

The other four Ozone Gas Destruct Canopies will be constructed over the Intermediate Ozone Contactors (Facility 3) or the Ozone Quenching Vaults (Facility 16).

7.2.9 Dewatering Building (Facility 9)

For a finish pad elevation of 317 feet that is approximately 3 to 7 feet below the existing ground surface at the location for the Dewatering Building, it is anticipated, based on the subsurface conditions exposed in Boring NMB-5, that the Dewatering Building will be underlain by approximately 0 to 2 feet of old fill over siltstone and sandstone of the Hambre Formation. The old fill is undocumented and unsuitable as a foundation material based on existing information. The Dewatering Building may be supported on footings or a mat foundation provided that the old fill below the finish pad elevation is removed and replaced with engineered fill. The mat foundation may be preferred where concrete pads for tanks or other equipment would be needed over large portions of the floor area.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime. The ground conditions encountered in Boring NMB-5 indicate that the existing subgrade material includes clay soil and siltstone of the Hambre Formation which may have a moderate to very high expansion characteristic.

7.2.10 Truck Loading Canopy (Facility 9A)

For an assumed finish pad elevation of 322 feet that is approximately 1 to 3 feet below the existing ground surface at the location for the Truck Loading Canopy, it is anticipated, based on the subsurface conditions exposed in Boring NMB-5, that the canopy will be underlain by approximately 3 to 5 feet of old fill over siltstone and sandstone of the Hambre Formation. The old fill is undocumented and unsuitable as a foundation material based on existing information. Based on the anticipated bearing conditions and thickness of the old fill, the Truck Loading Canopy may be supported on footings that are extended to bear on the formational material below the old fill.

7.2.11 Thickened Solids Blending Tanks (Facility 10)

For an assumed finish pad elevation of 319 feet that is approximately 0 to 6 feet below the existing ground surface at the locations for the two Thickened Solids Blending Tanks, it is anticipated, based on the subsurface conditions exposed in Boring NMB-5, that the tanks will be underlain by approximately 0 to 5 feet of old fill over siltstone and sandstone of the Hambre Formation. The old fill is undocumented and unsuitable as a foundation material based on existing information. Based on the anticipated bearing conditions, the tanks may be supported on mat foundations provided that the old fill below the finish pad elevation is removed and replaced with engineered fill.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime. The ground conditions encountered in Boring NMB-5 indicate that the existing subgrade material includes clay soil and siltstone of the Hambre Formation which may have a moderate to very high expansion characteristic as discussed in Section 5.10.

7.2.12 Gravity Thickener Tanks (Facility 11)

For a finish pad elevation of approximately 4 to 27 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings NMB-5 and NMB-8, that the Gravity Thickener Tanks will be underlain by siltstone and sandstone of the Hambre Formation and these four structures may be supported on mat foundations.

7.2.13 Thickened Solids PP Vaults (Facility 12)

For a finish pad elevation of approximately 22 to 29 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings NMB-5 and NMB-8, that the Thickened Solids PP Vaults will be underlain by siltstone and sandstone of the Hambre Formation and these two structures may be supported on mat foundations.

7.2.14 Solids Transfer PP (Facility 13)

For a finish pad elevation of approximately 1 foot below to 1 foot above the existing ground surface at the proposed location of the Solids Transfer PP, it is anticipated, based on the subsurface conditions exposed in Boring MN-33, that the PP will be underlain by new fill and siltstone of the Hambre Formation. Based on the anticipated bearing conditions, the Solids Transfer PP may be supported on footings. Alternatively, the Solids Transfer PP may be supported on a mat foundation. The mat foundation may be preferred where concrete pads for pumps or other equipment would be needed over large portions of the floor area.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime. The expansion characteristic of the existing subgrade at the Solids Transfer PP is not known.

7.2.15 Electrical Facilities (Facility 14)

For an assumed finish pad elevation of 333 feet that is approximately 2 to 6 feet above the existing ground surface at the location for the Electrical Facilities, it is anticipated, based on the subsurface conditions exposed in Boring MN-15, that the Electrical Facilities will be underlain by approximately 2 to 6 feet of new fill over approximately 20 to 25 feet of old fill and 13 to 18 feet of soil with a depth to formational material of approximately 40 to 50 feet. Ninyo & Moore did not evaluate the suitability of the old fill at the Electrical Facilities as part of the GIR. Previous investigations (EBMUD, 2001) concluded that the old fill was unsuitable as a foundation material due to the potential for organic material in the old fill. Additional exploration may be performed to re-evaluate the suitability of the existing fill at the proposed

location for the Electrical Facilities. If the additional exploration indicates that the fill is indeed suitable, this facility may be supported on footings or mat foundations. If additional exploration is not performed or if the fill is found to be unsuitable, these facilities may be supported on deep footings or drilled pier foundations with a structural slab or additional piers to support equipment. Alternatively, this facility may be supported on footings or mat foundations provided that the existing fill is removed and replaced with compacted, engineered fill.

7.2.16 Maintenance Building (Facility 15)

For an assumed finish pad elevation of 336 feet that is approximately 2 feet below to 7 feet above the existing ground surface at the location for the Maintenance Building, it is anticipated, based on the subsurface conditions exposed in Borings MN-16, NMB-7, and SB-1, that the building will be underlain by approximately 0 to 7 feet of new fill over approximately 3 to 19 feet of old fill and 3 to 17 feet of soil with a depth to formational material of approximately 20 to 33 feet. As part of the current study, Ninyo & Moore performed subsurface exploration and laboratory testing to evaluate the suitability of the old fill at the Maintenance Building and concluded that the old fill is a suitable bearing material for foundations based on the results of the exploration and testing. Based on the anticipated bearing conditions, the Maintenance Building may be supported on footings.

To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime. The ground conditions encountered in Boring MN-16 indicate that the existing subgrade material includes clay soil which may be indicative of a high expansion characteristic.

7.2.17 Ozone Quenching Vaults (Facility 16)

For a finish pad elevation of approximately 20 to 31 feet below the existing ground surface, it is anticipated, based on historic grading plans (EBMUD, 1964) and geologic mapping of the WCWTP site (Figure 5), that the southern Ozone Quenching Vault will be underlain by formational material and may be supported on a mat foundation.

For a finish pad elevation of approximately 10 to 22 feet below the existing ground surface, it is anticipated, based on historic grading plans (EBMUD, 1964) and geologic mapping of the WCWTP site (Figure 5), that the northern Ozone Quenching Vault will be underlain by approximately 3 to 11 feet of old fill over soil and formational material. Ninyo & Moore did not evaluate the suitability of the old fill at the northern Ozone Quenching Vault as part of the current study. Previous investigations (EBMUD, 2001) concluded that the old fill was

unsuitable as a foundation material due to the potential for organic material in the old fill. Based on the anticipated bearing conditions and extent of unsuitable old fill below the existing ground surface, the northern Ozone Quenching Vault may be supported on drilled piers bearing on formational material.

7.2.18 Chemical Storage Canopy (Facility 17)

For a finish pad elevation that is approximately 1 foot below to 1 foot above the existing ground surface at the location for the Chemical Storage Canopy, it is anticipated, based on historic grading plans (EBMUD, 1964) and the subsurface conditions exposed in Borings MN-21 and MN-28A, that the Chemical Storage Canopy will be underlain by approximately 0 to 1 foot of new fill over approximately 15 to 25 feet of old fill with a depth to formational material of approximately 16 to 25 feet. Ninyo & Moore did not evaluate the suitability of the old fill at the Chemical Storage Canopy as part of the current study. Previous investigations (EBMUD, 2001) concluded that the old fill was unsuitable as a foundation material due to the potential for organic material in the old fill. Based on the anticipated bearing conditions, thickness of unsuitable old fill, and proximity to the Chemical Building, the Chemical Storage Canopy may be supported on drilled piers that extend below the old fill.

7.2.19 Hydrogen Peroxide Station Canopies (Facility 18)

For a finish pad elevation of approximately 1 to 14 feet below the existing ground surface at the proposed location of the western Hydrogen Peroxide Station Canopy, it is anticipated, based on geologic mapping of the WCWTP site (Figure 5), that the western canopy will be underlain by soil or formational material. Based on the anticipated bearing conditions, the western Hydrogen Peroxide Station Canopy may be supported on footings. Alternatively, where concrete equipment pads are needed over large portions of the canopy area, a mat foundation may be used to support the canopy and facility equipment on a common foundation. To mitigate concerns related to expansive soil, the top 2 feet below finish pad grade at the western canopy should consist of a material with low expansion characteristics using select site fill or import fill, or by chemically treating suitable on-site material with quicklime.

For a finish pad elevation of approximately 1 foot below the existing ground surface at the proposed location of the eastern Hydrogen Peroxide Station Canopy, it is anticipated, based on the subsurface conditions exposed in Boring NMB-2, that the eastern canopy will be underlain by siltstone of the Hambre Formation. Based on the anticipated bearing conditions, the eastern Hydrogen Peroxide Station Canopy may be supported on footings. Alternatively,

where concrete equipment pads are needed over large portions of the canopy area, a mat foundation may be used to support the canopy and facility equipment on a common foundation. Laboratory testing performed for this study indicated that existing subgrade at the location of the eastern Hydrogen Peroxide Station Canopy has a low expansion characteristic.

7.3 Footings

The parameters listed in Table 15 may be used to design footings to support the listed facilities presuming that the structure loads conform with the assumptions in Table 4, that the footings for the Truck Loading Canopy (Facility 9A) are extended to bear on formational material below the old fill, and that the old fill is removed from below the finish pad elevation for the Pre-Ozone PP (Facility 6) and the Dewatering Building (Facility 9) and replaced with engineered fill. If the proposed retaining walls at the north corner of the Ozone Generator Building (Facility 4) and the north corner VOX Tanks & Vaporizers (Facility 5) are eliminated, the foundation walls of these facilities at those locations should be designed as restrained retaining walls in accordance with the criteria for footings and lateral earth pressures in Section 7.9. A lateral bearing pressure of 330 psf per foot of depth may be used to evaluate the resistance of footings to lateral loads. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads. The weight of the material above a plane rising up and away from the bottom edges of the footings at 20 degrees off plumb may be considered, along with the weight of the footing and the material over the footing, when evaluating footing resistance to uplift. A unit weight of 120 pounds per cubic foot (pcf) for soil or aggregate and 150 pcf for normal weight concrete may be assumed for this evaluation.

Table 15 – Design Parameters for Footings					
ID	Facility	Bearing Depth[1]	Footing Width	Allowable Bearing Capacity[2]	Static Settlement[3]
4	Ozone Generator Building	36 inches	1½ to 4 feet	3,000 psf	1½ inch total
5	LOX Tanks & Vaporizers	36 inches	1½ to 4 feet	2,500 psf	1½ inch total
6	Pre-Ozone PP	36 inches	1½ to 4 feet	4,000 psf	1 inch total
8	Ozone Gas Destruct (west)	36 inches	2 to 4 feet	5,300 psf	1 inch total
9	Dewatering Building	36 inches	1½ to 4 feet	4,400 psf	⅔ inch total
9A	Truck Loading Canopy	36 inches	2 to 4 feet	6,000 psf	½ inch total
13	Solids Transfer PP	36 inches	1½ to 4 feet	6,000 psf	¼ inch total
15	Maintenance Building	36 inches	1½ to 4 feet	3,000 psf	1 inch total
18	Hydrogen Peroxide Station (west)	36 inches	2 to 4 feet	5,300 psf	1 inch total
18	Hydrogen Peroxide Station (east)	36 inches	2 to 4 feet	6,000 psf	¼ inch total

Notes:
1 Below the adjacent finish grade.
2 Gross allowable bearing capacity in pounds per square foot with Safety Factor of 3 or more. Allowable bearing capacity may be increased by one-third for wind or seismic load combinations.
3 Total static settlement due to sustained loads. Differential settlement is 50 percent of total over 20-foot span.

7.4 Mat Foundations

The parameters listed in Table 15 may be used to design mat foundations for the listed facilities presuming that the structure loads conform with the assumptions in Table 4 and that old fill is removed from below the finish pad elevation for the Pre-Ozone PP (Facility 6), the Dewatering Building (Facility 9), and the Thickened Solids Blending Tanks (Facility 12) and replaced with engineered fill.

Table 16 – Design Parameters for Mat Foundations					
ID	Facility	Plan Dimensions (feet)	Bearing Depth[1]	Allowable Bearing Capacity[2]	Static Settlement[3]
1	Ballasted Flocculation	81x32x27	24 feet	11,700 psf	¼ inch total
2	Intermediate Ozone PP	50x40x20	14 feet	8,700 psf	¼ inch total
3	Intermediate Ozone Contactors	80x44x26	20 feet	10,100 psf	¼ inch total
4	Ozone Generator Building	200x50x15	0.5 feet	1,500 psf	1¼ inch total
5	LOX Tanks & Vaporizers	83x40x27	0.5 feet	1,500 psf	1 inch total
6	Pre-Ozone PP	50x30x20	0.5 feet	3,000 psf	½ inch total
7	Combined Reclaim Metering Vault	40x25x18	18 feet	11,700 psf	¼ inch total
8	Ozone Gas Destruct (West)	20x10x12	0.5 feet	3,800 psf	⅓ inch total
9	Dewatering Building	100x45x27	0.5 feet	3,400 psf	⅓ inch total
10	Thickened Solids Blending Tanks	20x20x16	0.5 feet	4,000 psf	½ inch total
11	Gravity Thickeners	80’ Diameter x17	17 feet	7,500 psf	¼ inch total
12	Thickened Solids PP	50x30x27	22 feet	11,000 psf	¼ inch total
13	Solids Transfer PP	20x15x17	0.5 feet	4,000 psf	¼ inch total
15	Maintenance Building	120x60x27	0.5 feet	1,500 psf	½ inch total
16	Ozone Quenching Vault (south)	30x25x20	12 feet	10,300 psf	¼ inch total
18	Hydrogen Peroxide Station (east)	40x30x15	0.5 feet	5,100 psf	¼ inch total
18	Hydrogen Peroxide Station (west)	40x30x15	0.5 feet	3,100 psf	⅓ inch total

Notes:
1 Below the adjacent finish grade.
2 Gross allowable bearing capacity in pounds per square foot with Safety Factor of 3 or more. Allowable bearing capacity may be increased by one-third for wind or seismic load combinations.
3 Total static settlement due to sustained loads. Differential settlement is 50 percent of total over 20-foot span.

A lateral bearing pressure of 330 psf per foot of depth may be used to evaluate the resistance of mat foundations and embedded foundation walls to lateral loads. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure for foundation walls on ground sloping more than 5 percent should be reduced by 9 psf per foot depth per degree of inclination. For complex slopes, an equivalent ground slope may be evaluated as the slope between the ground surface above the edge of the mat foundation to a point on the ground surface at a lateral distance from the edge of foundation equivalent to three times the embedment depth of the foundation. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. The lateral bearing pressure may be increased by one-third when considering loads of short duration such as wind or seismic forces. A friction coefficient of 0.35 may be assumed for evaluating frictional resistance to lateral loads for mat foundations bearing on soil or formational material. The coefficient of friction may be increased to 0.50 where the mat is constructed over 6 inches of compacted

aggregate base. The coefficient of friction should be reduced to 0.20 where the mat is underlain by a plastic moisture vapor retarding membrane.

7.5 Drilled Piers

Drilled piers may be used to support the Electrical Facilities (Facility 15), the northern Ozone Quenching Vault (Facility 17), and the Chemical Storage Canopy (Facility 17) where the old fill underlying these facilities has not been found to be suitable and removal is not feasible due to the thickness of the fill or the proximity to adjacent improvements or slopes. As noted above, additional borings at the locations of the facilities listed above could provide data that indicates that drilled piers would not be needed.

The drilled piers may be designed for an allowable axial side resistance of 500 psf in soil below the old fill and 1,500 psf in formational material for downward loading with a safety factory of 2. The piers may be designed for an allowable tip resistance of 6,000 psf in soil below the old fill and 13,000 psf in formational material with a safety factor of 3. To reduce potential for downdrag due to settlement of the old fill, the embedment depth of the pier should be adjusted so that the sum of the allowable tip and side resistance below the old fill exceeds the sustained load on the pier plus a drag load computed from a negative skin friction of 750 psf on the pier in the old fill.

An allowable axial side resistance of 750 psf in the old fill may be considered in addition to the tip resistance and the side resistance in the soil and formational material below the old fill when evaluating resistance for downward loading under seismic or other transient loading conditions. An allowable axial side resistance of 500 psf in the old fill, 330 psf in the soil below the old fill, and 1,000 psf in the formational material may be assumed for upward loading with a safety factor of 3.

A lateral load corresponding to ½ inch head deflection of approximately 19, 28, and 51 kips for 18-, 24-, and 36-inch diameter piers, respectively, for pinned head conditions may be assumed for preliminary assessment of lateral load resistance. A lateral bearing pressure of 330 psf per foot of depth acting on embedded grade beams or pile caps may also be considered when evaluating foundation resistance to lateral loads. The recommended lateral bearing pressure is for level and gently sloping ground conditions where the ground slope adjacent to the foundation is 5 percent or less. The lateral bearing pressure should be neglected to a depth of 12 inches where the ground adjacent to the foundation is not covered by a slab or pavement. For a preliminary assessment of group effects, the lateral load resistance for a trailing pier parallel to the direction of the lateral load may be neglected where the center-to-center pier spacing is less than 8 diameters and the lateral load resistance per pier assumes that the center-to-center spacing perpendicular to the lateral load direction is no less than three diameters.

7.6 Below Grade Walls

Below grade walls for tanks, vaults, or basements should be designed to resist at-rest lateral earth pressures. Recommended values of at-rest lateral earth pressure, for drained and undrained conditions, and for level and sloping backfill, are provided in Table 17 for design of below grade walls under static or sustained loading. At-rest earth pressures are provided for below grade walls in formational material where less than 2 lateral feet of backfill is need to fill the annular space between the below grade wall and the edge of the excavation in formational material, and for below grade walls in site soil or where more than 2 lateral feet of general fill is needed to backfill the annular space. Below grade walls may be designed for drained conditions where a subdrain that discharges by gravity to a sump or a suitable outlet is provided near the base of the below grade wall. Table 17 also provides equivalent fluid lateral earth pressures for combined seismic and active lateral earth pressures to evaluate earth pressures on below grade walls under drained and undrained seismic conditions.

Table 17 – Lateral Earth Pressures for Below Grade Walls

Material	Drained Conditions		Undrained Conditions	
	At-Rest Lateral Earth Pressure (psf/foot depth)	Seismic Active Earth Pressure (psf/foot depth)	At-Rest Lateral Earth Pressure (psf/foot depth)	Seismic Active Earth Pressure (psf/foot depth)
Site soil or general backfill	67 + 1.2/degree	96 + 1.9/degree	96 + 1.2/degree	110 + 1.9/degree
Formational Material	49 + 1.2/degree	82 + 0.5/degree	87 + 1.2/degree	103 + 0.5/degree

Note:
Equivalent fluid lateral earth pressures presented for level backfill conditions plus additional lateral earth pressure per degree of backfill slope inclination when measured from the horizontal

Below grade walls that retain level ground should be designed to resist construction or live load surcharges on the backfill. The uniform lateral earth pressure due to a backfill surcharge of 240 psf may be calculated as the product of the appropriate at-rest lateral earth pressure and the surcharge equivalent backfill thickness of 2 feet. An additional backfill surcharge and lateral earth pressure for adjacent foundations or other surcharge should be considered, as applicable, where the adjacent foundation bears above an imaginary plane that rises up and away from the bottom edge of the below grade wall at a 1½:1 (horizontal to vertical) gradient.

7.7 Uplift Resistance

The resistance of partially buried tanks and vaults to buoyancy and seismic uplift may be evaluated by considering the weight of the material above a plane rising up and away from the bottom edges of the mat foundation at 20 degrees off plumb, along with the weight of the tank or vault and the material over the tank or vault (if buried). A unit weight of 120 pounds per cubic foot (pcf) for soil or aggregate above groundwater or 60 pcf below groundwater may be assumed for this evaluation. A subdrain around the perimeter or a drainage blanket under the tank or vault may be considered as a measure to reduce uplift due to buoyancy provided that the subdrain or drainage blanket can be drained by gravity or by a sump pump when the tank or vault is empty.

Tiedown anchors consisting of a bar tendon installed in a drilled hole backfilled with grout, may be used to provide additional resistance to uplift from buoyancy or seismic loading. A smooth plastic sleeve should be provided over the tendon to create an unbonded zone that extends no less than 10 feet for bar tendons, and no less than 15 feet below the ground surface. Gravity-grouted tiedown anchors with a bonded length of no less than 15 feet may be designed for an allowable grout-to-ground bond strength of 1,000 psf in formational material with a safety factor of 3. Pressure grouting during initial grout placement or during one or more post-grouting operations, may be performed below a depth of 10 feet from the ground surface to enhance pullout resistance. The allowable grout-to-ground bond strength may be increased to 2,000 psf, with a safety factor of 3, where pressure grouting is performed to enhance pullout resistance with injection pressures of 150 pounds per square inch (psi) or more.

7.8 Retaining Walls

New permanent retaining walls associated with the proposed improvements are under consideration for nine locations. The retaining walls will retain cut at four locations and new fill at four locations. In addition, three walls retaining cut have been proposed to create three staging areas. Retaining wall recommendations are presented in the following subsections with the anticipated wall length and retained grade differential across the wall based on the existing topography (Figure 2) and preliminary cut/fill estimates in the BODR (EBMUD, 2020).

7.8.1 Hydrogen Peroxide Station & Ozone Gas Destruct (Facilities 18 & 8)

The proposed retaining wall is approximately 150 feet long (EBMUD, 2020) and retains up to 16 feet of cut (EBMUD, 2020) along the western, southern, and eastern edge of the level pad for the western Hydrogen Peroxide Station (Facility 18) and western Ozone Gas Destruct Canopy (Facility 8). Retained soil slopes up to the south at a gradient of approximately 2.4:1 (horizontal to vertical). Soil nails with a shotcrete facing may be used to retain the cut face.

Soldier piles with treated timber or precast concrete lagging and tieback ground anchors may be considered as an alternative to soil nail walls to reduce the degree of wall movement and mitigate the potential reduction in slope stability due to excavation at the toe for the pad. Based on geologic mapping of the site (Figure 5) it is anticipated that the proposed wall will retain soil and shale of the Briones Formation.

7.8.2 Electrical Facilities & Maintenance Building (Facilities 14 & 15)

The proposed retaining wall at the Maintenance Building (Facility 15) and Electrical Facilities (Facility 14) is approximately 450 feet long (EBMUD, 2020) supporting up to approximately 8 feet of fill (EBMUD, 2020) along the southern and eastern edges of the new fill embankment below the Electrical Facilities and Maintenance Building. The proposed retaining wall location is along the toe of a slope descending from the south at a gradient of approximately 2:1 (horizontal to vertical). The proposed retaining wall will retain level backfill and a potential surcharge from the Maintenance Building and Electrical Facilities depending on facility location relative to the wall. The southern and eastern edges of the proposed fill embankment may be graded to slope down at gradients of 2:1 (horizontal to vertical) or flatter to eliminate the proposed wall. Where a 2:1 side slope cannot be accommodated due to adjacent improvements or features (e.g., structures, pipelines or utilities, drainage swales, or subdrain outlets), a semi-gravity wall such as an L-shaped or inverted tee precast or cast-in-place concrete wall on footings may be used to retain the edge of the fill embankment. Based on the subsurface conditions encountered in Borings MN-16, SB-1, and MN-15, it is anticipated that the proposed retaining wall will bear on old fill.

7.8.3 Ozone Generator Building (Facility 4)

The proposed retaining wall at the Ozone Generator Building (Facility 4) is approximately 250 feet long (EBMUD, 2020) supporting up to approximately 24 feet of fill (EBMUD, 2020) along the western and northern edges of the new fill pad for the building. The proposed retaining wall location is on a slope that descends to the north away from the wall at a gradient of approximately 2:1 (horizontal to vertical). The proposed retaining wall will retain level backfill and surcharge from the Ozone Generator Building. A semi-gravity retaining wall such as an L-shaped or inverted tee precast or cast-in-place concrete wall may be used to retain the fill for the building pad. The retaining wall may be eliminated if the foundation wall for the Ozone Generator Building is designed to resist the lateral earth pressures from the fill used to build the building pad. Based on the subsurface conditions encountered in Borings NMB-6 and NMB-4, it is anticipated that the proposed retaining wall will bear on old fill.

7.8.4 LOX Tanks & Vaporizers (Facility 5)

The proposed retaining wall at the LOX Tanks & Vaporizers (Facility 5) is approximately 55 feet long (EBMUD, 2020) supporting up to approximately 8 feet of fill (EBMUD, 2020) at the northern corner of the new fill pad for the building. The proposed retaining wall location is on a slope that descends to the north away from the wall at a gradient of approximately 2:1 (horizontal to vertical). The proposed retaining wall will retain level backfill and surcharge from the LOX Tanks & Vaporizers. A semi-gravity retaining wall such as an L-shaped or inverted tee precast or cast-in-place concrete wall may be used to retain the fill for the building pad. The retaining wall may be eliminated if the foundation wall for the building is designed to resist the lateral earth pressures from the fill used to build the building pad. Based on the subsurface conditions encountered in Borings NMB-4 and SB-2, it is anticipated that the proposed retaining wall will bear on old fill.

7.8.5 Gravity Thickeners & Thickened Solids Pumping Plant (Facilities 11 & 12)

The proposed retaining wall at the Gravity Thickeners (Facility 11) and Thickened Solids Pumping Plants (Facility 12) is approximately 660 feet long (EBMUD, 2020) supporting up to approximately 24 feet of fill (EBMUD, 2020) along the western, northern, and eastern edges of the pad for Gravity Thickeners. The proposed retaining wall location is on sloping ground that descends away from the wall to the west, north, and east at gradients of up to approximately 2:1 (horizontal to vertical). The proposed retaining wall will retain level backfill. No surcharge from the below grade Gravity Thickeners and Thickened Solids Pumping Plants is expected based on the anticipated bearing elevations for these facilities. A semi-gravity retaining wall such as an L-shaped or inverted tee precast or cast-in-place concrete wall may be used to retain the fill for the pad around the Gravity Thickeners. Based on the subsurface conditions encountered in Borings NMB-5 and NMB-8, it is anticipated that the proposed retaining wall will bear on siltstone and sandstone of the Hambre Formation.

7.8.6 Northern Ozone Quenching Vault (Facility 16)

The proposed retaining wall at the northern Ozone Quenching Vault (Facility 16) is approximately 65 feet long (EBMUD, 2020) and retains up to 5 feet of cut (EBMUD, 2020) along the northern, eastern, and southern edge of the level pad for the northern Ozone Gas Destruct (Facility 8) constructed over the below grade Ozone Quenching Vault. Retained soil slopes up to the east at a gradient of approximately 2:1 (horizontal to vertical). An internally braced soldier pile and lagging system extending up the proposed cut may be used to shore the excavation for construction of the vault. The solder piles retaining the proposed cut with treated timber or precast concrete lagging may be left in place after the vault is constructed

to retain the cut face on the eastern portion of the Ozone Gas Destruct pad. Soil nails with a shotcrete facing may be considered as an alternative method to retain the cut face. Based on historic grading plans (EBMUD, 1964) and geologic mapping of the WCWTP site (Figure 5), it is anticipated that the proposed retaining wall forming the pad for the northern Ozone Gas Destruct Canopy at the northern Ozone Quenching Vault will retain old fill.

7.8.7 Southern Ozone Quenching Vault (Facility 16)

The proposed retaining wall at the southern Ozone Quenching Vault (Facility 16) is approximately 100 feet long (EBMUD, 2020) and retains up to 12 feet of cut (Figure 2) along the western, southern, and eastern edge of the level pad for the southern Ozone Gas Destruct (Facility 8) constructed over the below grade Ozone Quenching Vault. Retained soil slopes parallel to wall alignment but is relatively level adjacent to the wall in a direction perpendicular to the wall alignment. The proposed retaining wall at the southern Ozone Quenching Vault may be eliminated by cutting the face back to slope at 2:1 (horizontal to vertical) above the excavation shoring for the vault. Alternatively, an internally-braced soldier pile and lagging system extending up the proposed cut may be used to shore the excavation for construction of the vault. The solder piles retaining the proposed cut with treated timber or precast concrete lagging may be left in place after the vault is constructed to retain the cut face on the southern portion of the Ozone Gas Destruct pad. Soil nails with a shotcrete facing may also be considered as an alternative method to retain the cut face. Based on geologic mapping of the WCWTP site (Figure 5), it is anticipated that the proposed retaining wall forming the pad for the southern Ozone Gas Destruct Canopy at the southern Ozone Quenching Vault will retain soil and sandstone of the Hambre Formation.

7.8.8 Southern Intermediate Ozone Contactor (Facility 3)

The proposed retaining wall at the southern Intermediate Ozone Contactor (Facility 3) is approximately 210 feet long (EBMUD, 2020) and retains up to 17 feet of cut (EBMUD, 2020) along the northern, eastern, and southern edges of the level pad for the southeastern Ozone Gas Destruct (Facility 8) constructed over the below grade Intermediate Ozone Contactor. Retained soil slopes parallel to wall alignment and down slightly in a direction perpendicular to the wall alignment. The proposed retaining wall at the southern Intermediate Ozone Contactor may be eliminated by cutting the face back to slope at 2:1 (horizontal to vertical) above the excavation shoring for the contactor tank. Alternatively, an internally-braced soldier pile and lagging system extending up the proposed cut may be used to shore the excavation for construction of the contactor tank. The solder piles retaining the proposed cut with treated timber or precast concrete lagging may be left in place after the tank is constructed to retain

the cut face on the eastern portion of the Ozone Gas Destruct pad. Soil nails with a shotcrete facing may also be considered as an alternative method to retain the cut face. Based on geologic mapping of the WCWTP site (Figure 5), it is anticipated that the proposed retaining wall forming the pad for the southeastern Ozone Gas Destruct Canopy at the southern Intermediate Ozone Contactor will retain soil and sandstone of the Hambre Formation.

7.8.9 Staging Area 1

The proposed retaining wall at Staging Area 1 is approximately 300 feet long and retains up to 18 feet of cut (Figure 2) along the western and southern edges of the near level staging area. Retained soil slopes up to the south at a gradient of up to approximately 2.5:1 (horizontal to vertical). Soil nails with a shotcrete facing may be used to retain the cut face. Soldier piles with treated timber or precast concrete lagging and tieback ground anchors may be considered as an alternative to soil nail walls to reduce the degree of wall movement and mitigate the potential reduction in slope stability due to excavation at the toe for the staging area. Based on the subsurface conditions encountered in Boring NMB-9, it is anticipated that the proposed retaining wall will retain soil and on siltstone of the Briones Formation.

7.8.10 Staging Area 2

The proposed retaining wall at Staging Area 2 is approximately 80 feet long and retains up to 12 feet of cut (Figure 2) along the southern and eastern edges of the near level staging area. Retained soil slopes up to the southeast at a gradient of up to approximately 3.5:1 (horizontal to vertical). Soil nails with a shotcrete facing may be used to retain the cut face. Soldier piles with treated timber or precast concrete lagging and tieback ground anchors may be considered as an alternative to soil nail walls to reduce the degree of wall movement and mitigate the potential reduction in slope stability due to excavation at the toe for the staging area. Based on geologic mapping of the site (Figure 5) it is anticipated that the proposed retaining wall will retain soil and shale of the Briones Formation.

7.8.11 Staging Area 3

The proposed retaining wall at Staging Area 3 is approximately 180 feet long and retains up to 10 feet of cut (Figure 2) along the southern and eastern edges of the near level staging area. Retained soil slopes up to the south at a gradient of up to approximately 2:1 (horizontal to vertical). Soil nails with a shotcrete facing may be used to retain the cut face. Soldier piles with treated timber or precast concrete lagging and tieback ground anchors may be considered as an alternative to soil nail walls to reduce the degree of wall movement and mitigate the potential reduction in slope stability due to excavation at the toe for the staging

area. Based on geologic mapping of the site (Figure 5) it is anticipated that the proposed retaining wall will retain soil and sandstone of the Hambre Formation.

7.9 Semi-Gravity Retaining Walls

Concrete inverted tee and L-shaped semi-gravity retaining walls may be used to retain embankment fill at the Maintenance Building and Electrical Facilities (Facilities 14 and 15), and pad fill at the Ozone Generator Building (Facility 4), the LOX Tanks & Vaporizers (Facility 5), and around the Gravity Thickeners (Facility 11). Semi-gravity retaining walls may be designed for drained conditions and hydrostatic pressures may be neglected presuming that weep holes are provided near the base of the wall stem and a subdrain is provided near the heel of the wall footing. Retaining walls that yield or deflect may be designed for active earth pressures. Retaining walls that are restrained against movement should be designed for at-rest earth pressures. Recommended values of active and at-rest lateral earth pressures for design of semi-gravity retaining walls backfilled with structural fill or select, chemically-treated fill are provided in Table 18. Structural fill used to backfill retaining walls should have a sand equivalent value of no less than 20 with 100 percent by dry weight passing the 3-inch sieve, 35 to 100 percent passing the No. 4 sieve, and 20 to 100 percent passing the No. 30 sieve. On-site material that meets these criteria, after stockpiling and testing, may be used as structural fill. On-site material with a sand equivalent value less than 20 may be used to backfill retaining walls provided the soil is chemically treated with quicklime to reduce the expansion characteristic and is processed, as needed to meet the gradation criteria. On-site material that meets the sand equivalent and gradation criteria for structural fill after chemical treatment and processing may be used as structural fill to backfill retaining walls. On-site material that meets the gradation criteria for structural fill but does not have a sand equivalent value more than 20 after chemical treatment and processing may be used to backfill retaining walls as select, chemically-treated fill provided that expansion index is not more than 50 or the plasticity index is not more than 15 after chemical treatment. The estimated retaining wall deflection needed to reduce at-rest earth pressure to active earth pressure is approximately ½ percent of wall height for walls backfilled with select, chemically-treated fill and approximately ¼ percent of wall height for walls backfilled with structural fill. An additional equivalent fluid pressure of 55 psf per foot depth, for consideration with the active earth pressure, may be used to evaluate seismic earth pressure on retaining walls as appropriate.

Table 18 – Lateral Earth Pressures for Semi-Gravity Walls				
Material	**Backfill or Ground Slope**	**Active Lateral Earth Pressure**	**At-Rest Lateral Earth Pressure**	**Passive Lateral Earth Pressure[1]**
Structural Fill	Level Backfill	29 psf/foot depth	53 psf/foot depth	--
Select, Chemically-Treated Fill	Level Backfill	35 psf/foot depth	60 psf/foot depth	--
Old Fill or Soil	Level Ground	--	--	330 psf/foot depth
	3H:1V Ground	--	--	134 psf/foot depth
	2H:1V Ground	--	--	78 psf/foot depth

Note:
1 Passive lateral earth pressure listed is 50 percent of nominal passive lateral earth pressure and corresponds to wall deflection equivalent to approximately 1 percent of footing embedment depth.

Walls retaining level ground should be designed to resist construction or live load surcharges on the backfill. The uniform lateral earth pressure due to a backfill surcharge of 240 psf equivalent to 2 feet of soil, may be calculated as the product of the appropriate at-rest or active lateral earth pressure and the surcharge. An additional backfill surcharge and lateral earth pressure for adjacent foundations or other surcharge should be considered, as applicable, where the adjacent foundations bear above an imaginary plane that rises up and away from the bottom edge of the retaining wall footing at a 1½:1 (horizontal to vertical) gradient.

The parameters listed in Table 19 may be used to design footings for the proposed semi-gravity retaining walls. Lateral forces may be resisted by friction at the base of the retaining wall footing, and passive earth pressure acting on the embedded wall, wall footing, or wall key, if present. Recommended values of passive lateral earth pressure for design of semi-gravity retaining walls are provided in Table 18 for level and sloping ground conditions. For complex slopes, an equivalent ground slope may be evaluated as the slope between the ground surface above the toe of the wall footing to a point on the ground surface at a lateral distance from the toe of the wall footing equivalent to three times the embedment depth of the footing bottom at the toe. The allowable bearing capacity and passive lateral earth pressure may be interpolated from the values provided in the tables for an intermediate slope. Passive earth pressure should be neglected to a depth of 1 foot below the ground surface when evaluating lateral load resistance where the ground surface is not covered by pavement or flatwork. Semi-gravity retaining walls may be designed for a coefficient of friction of 0.35 to resist lateral loads. The coefficient of friction may be increased to 0.50 where the footing is constructed over 6 inches of compacted aggregate base.

Table 19 – Design Parameters for Semi-Gravity Wall Footings

Wall	Ground Slope	Bearing Depth[1]	Footing Width	Allowable Bearing Capacity[2]	Static Settlement[3]
West, north, & east of Gravity Thickeners (Facility 11) on Hambre Formation	Level 3H:1V 2H:1V	36 inches	4 to 20 feet	12,700 psf 8,600 psf 7,000 psf	½ inch total
South of Facilities 14 & 15; North of Facility 4; North of Facility 5 on fill	Level 3H:1V 2H:1V	36 inches	4 to 20 feet	8,600 psf 5,300 psf 4,000 psf	1½ inch total

Notes:
1 Vertical distance from bottom of footing to finish grade. Evaluated at toe of footing.
2 Gross allowable bearing capacity in pounds per square foot with Safety Factor of 3 or more. Allowable bearing capacity may be increased by one-third for wind or seismic load combinations.
3 Total static settlement due to sustained loads. Differential settlement is 50 percent of total over 20-foot span.

7.10 Excavation Stabilization

Anticipated conditions and appropriate measures to stabilize excavations for the proposed vaults and below grade structures associated with the project improvements are discussed in the following subsections.

7.10.1 Ballasted Flocculation Basins (Facility 1)

The proposed locations for the Ballasted Flocculation Basins are near existing structures, utilities, and a paved access road. For a finish pad elevation of approximately 24 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings MN-8 and MN-2, that excavations for the Ballasted Flocculation Tanks will encounter a couple feet of clay soil over deeply to moderately weathered sandstone and siltstone of the Hambre Formation. To reduce impacts to adjacent improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavation.

7.10.2 Intermediate Ozone PP Vaults (Facility 2)

The proposed locations for the Intermediate Ozone PP Vaults are near existing structures, utilities, and a paved access road. For a finish pad elevation of approximately 14 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings MN-9 and MN-2, that excavations for the Intermediate Ozone Pumping Plant Vaults will encounter a couple feet of clay soil over deeply to moderately weathered siltstone of the Hambre Formation in Boring MN-2 and about 5 feet of sandy clay fill over 9 feet of sandy silt over deeply weathered sandstone of the Hambre Formation. To reduce impacts to adjacent

improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavation.

7.10.3 Intermediate Ozone Contactors (Facility 3)

The proposed locations for the Intermediate Ozone Contactors are near existing structures, utilities, and a paved access road. For a finish pad elevation of approximately 21 to 26 feet below the existing ground surface, it is anticipated, based on historic grading plans (EBMUD, 1964) and the subsurface conditions exposed in Borings NMB-3 and NMB-1, that excavations for the Intermediate Ozone Contactors will encounter intensely to moderately weathered and weakly to strongly cemented siltstone of the Hambre Formation in Boring NMB-1 and approximately 28 feet of sandy and clayey fill over weathered sandstone and siltstone of the Hambre Formation in Boring NMB-3. To reduce impacts to adjacent improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavation.

7.10.4 Combined Reclaim Metering Vault (Facility 7)

The proposed location for the Combined Reclaim Metering Vault is near an existing pipeline on near level ground. For a finish pad elevation of approximately 18 feet below the existing ground surface, it is anticipated, based on geologic mapping of the WCWTP site (Figure 5), that excavations for the Combined Reclaim Metering Vault will encounter fill over shale of the Briones Formation. Due to the relative lack of adjacent constraints, excavation side slopes may be cut back at 1½:1 (horizontal to vertical) to stabilize the side slopes of the excavation, or the side slopes may be partially cut back at 1½:1 (horizontal to vertical) with the lower portion of the excavation retaining by a cantilever soldier-pile-and-lagging wall.

7.10.5 Gravity Thickener Tanks (Facility 11)

The proposed locations for the Gravity Thickener Tanks are on gently sloping ground, away from existing improvements or active utilities. For finish pad elevations between approximately 4 and 27 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings NMB-5 and NMB-8, that excavations for the Gravity Thickener Tanks will encounter approximately 4 to 6 feet of sandy clay and clayey sand fill over weathered siltstone and sandstone of the Hambre Formation. To reduce impacts to adjacent improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavation. Due to the relative lack of adjacent constraints, excavation side slopes may be cut back at 1½:1 (horizontal to vertical) to stabilize the side slopes of the excavation.

7.10.6 Thickened Solids PP Vaults (Facility 12)

The proposed locations for the Thickened Solids PP Vaults are on gently sloping ground, away from existing improvements or active utilities. For finish pad elevations between approximately 22 and 29 feet below the existing ground surface, it is anticipated, based on the subsurface conditions exposed in Borings NMB-5 and NMB-8, that excavations for the Thickened Solids PP Vaults will encounter approximately 4 to 6 feet of sandy clay and clayey sand fill over weathered siltstone and sandstone of the Hambre Formation. To reduce impacts to adjacent improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavation. Due to the relative lack of adjacent constraints, excavation side slopes may be cut back at 1½:1 (horizontal to vertical) to stabilize the side slopes of the excavation.

7.10.7 Ozone Quenching Vault (Facility 16)

The proposed location for the southern Ozone Quenching Vault is near existing utilities and a paved access road and the location for the northern Ozone Quenching Vault is on the mid-slope bench of a 2:1 (horizontal to vertical) slope near existing utilities. For a finish pad elevation of approximately 10 to 36 feet below the existing ground surface, it is anticipated, based on historic grading plans (EBMUD, 1964) and geologic mapping of the WCWTP site (Figure 5), that excavations for the northern Ozone Quenching Vault will encounter fill and excavations for the southern Ozone Quenching Vault will encounter weathered sandstone and siltstone of the Hambre Formation. To reduce impacts to adjacent improvements, an internally-braced soldier-pile-and-lagging wall or slide rail shoring system may be used stabilize the sides of the excavations.

7.11 Excavation Shoring

Recommended values of active and passive lateral earth pressures for cantilever soldier-pile-and-lagging walls, and apparent lateral earth pressures for internally-braced soldier-pile-and-lagging walls, slide-rail-shoring, or similar systems are provided in Table 17 for consideration in the design and selection of shoring systems to stabilize excavations for constructions of the proposed facilities. The ratio of the center-to-center spacing of the soldier to the width of the pile (or width of the drilled hole for piles installed in drilled holes with lean concrete backfill) should not be less than 3. The estimated wall deflection needed to develop active earth pressure is approximately 1 percent of wall height for retaining walls retaining site soil and old fill, and approximately ¼ percent of wall height for walls retaining bedrock formational material.

Excavation shoring that retains level ground should be designed to resist construction or live load surcharges on the backfill. The uniform lateral earth pressure due to a backfill surcharge of 240 psf is presented in Table 20. An additional backfill surcharge and lateral earth pressure for adjacent foundations or other surcharge should be considered, as applicable, where the adjacent foundation bear above an imaginary plane that rises up and away from the bottom edge of the below grade wall at a 1½:1 (horizontal to vertical) gradient.

Table 20 – Lateral Earth Pressures for Excavation Shoring

Material	Active Lateral Earth Pressure[1] (psf/foot depth)	Surcharge Lateral Earth Pressure[2] (psf)	Passive Lateral Earth Pressure[3] (psf/foot depth)	Apparent Lateral Earth Pressure[4] (psf)
Site soil or old fill	41 + 1.9/degree	82	160	48H
Formational Material	27 + 0.5/degree	54	243	24H

Notes:
1 Equivalent fluid active lateral earth pressure acting on lagging and soldier pile width below lagging or on drilled hole width below lagging where backfilled with lean concrete. Active earth pressure presented for level backfill conditions plus additional lateral earth pressure per degree of backfill slope inclination.
2 Lateral pressure due to surcharge on level backfill of 240 psf.
3 Equivalent fluid passive earth pressure acting on an area equivalent to 300 percent of the soldier beam width below the bottom of lagging or 300 percent of the drilled hole diameter for soldier piles installed in drilled holes with lean concrete backfill. The passive pressure should be neglected to a depth of one foot below the bottom of excavation. Passive pressure listed is 50 percent of nominal passive lateral earth pressure.
4 Peak apparent lateral earth pressure occurring between 25 and 75 percent of exposed wall height (H) in trapezoidal pressure distribution with zero apparent earth pressure at top and bottom of exposed wall height.

7.12 Subgrade Preparation

Subgrade preparation at the Pre-Ozone PPs (Facility 6), the Dewatering Building (Facility 9), and the Thickened Solids Blending Tanks (Facility 12), where footings or mat foundations are anticipated and the proposed facility locations are underlain by old fill not previously found to be suitable, should include removal of the old fill and replacement with engineered fill. Removals should extend outside the building footprint a lateral distance equivalent to depth of removal below the finish pad grade. Excavated soil that contains less than 3 percent organic matter by dry weight and conforms with EBMUD's Standard Specifications may be reused as fill.

The expansion characteristic of existing subgrade within 2 feet of finish pad grade is not known at the western Ozone Gas Destruct Canopy (Facility 8), the Solids Transfer PP (Facility 13), and the western Hydrogen Peroxide Station (Facility 18). Existing subgrade at the Dewatering Building (Facility 9) and Thickened Solids Blending Tanks (Facility 10) may include clay and siltstone with a moderate to very high expansion characteristic. Subgrade preparation at these locations should include chemical treatment of the existing subgrade with quicklime to reduce the expansion

characteristic within 2 feet of finish pad grade or removal of the existing subgrade within 2 feet of finish pad grade and replacement with import or select site fill with a low expansion characteristic unless the existing subgrade is found to have a low expansion characteristic.

New fill will be needed to achieve finish pad grades at the Ozone Generator Building (Facility 4), the LOX Tanks & Vaporizers (Facility 5), the Pre-Ozone PPs (Facility 6), the western Ozone Gas Destruct Canopy (Facility 8), the Solids Transfer PP (Facility 13), the Maintenance Building (Facility 14), and the Hydrogen Peroxide Stations (Facility 18). Finish subgrade preparation at these locations should include placing import or select site soil or placing site soil and chemically treating the site soil with quicklime to create a zone with low expansion characteristics extending down to 2 feet below finish pad elevation. Import fill, select site soil, or chemically-treated site soil should have an expansion index of no more than 50 or a plasticity index of no more than 15 with 100 percent or more by dry weight passing the 4-inch sieve.

8 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this GIR have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the geotechnical conditions described in this GIR. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction or future exploration. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. The geotechnical conditions at a site could change with time as a result of natural processes or human activities at the WCWTP site or nearby sites.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This GIR is intended for planning purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the Project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

The conclusions, recommendations, and opinions in this GIR based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this GIR are encountered, Ninyo & Moore should be notified and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or human activities at the WCWTP or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This GIR has been prepared for the exclusive use of EBMUD and Woodard & Curran for the Project. Any use or reuse of the findings, conclusions, and/or recommendations in this GIR by parties other than EBMUD and Woodard & Curran is undertaken at said parties' sole risk.

9 REFERENCES

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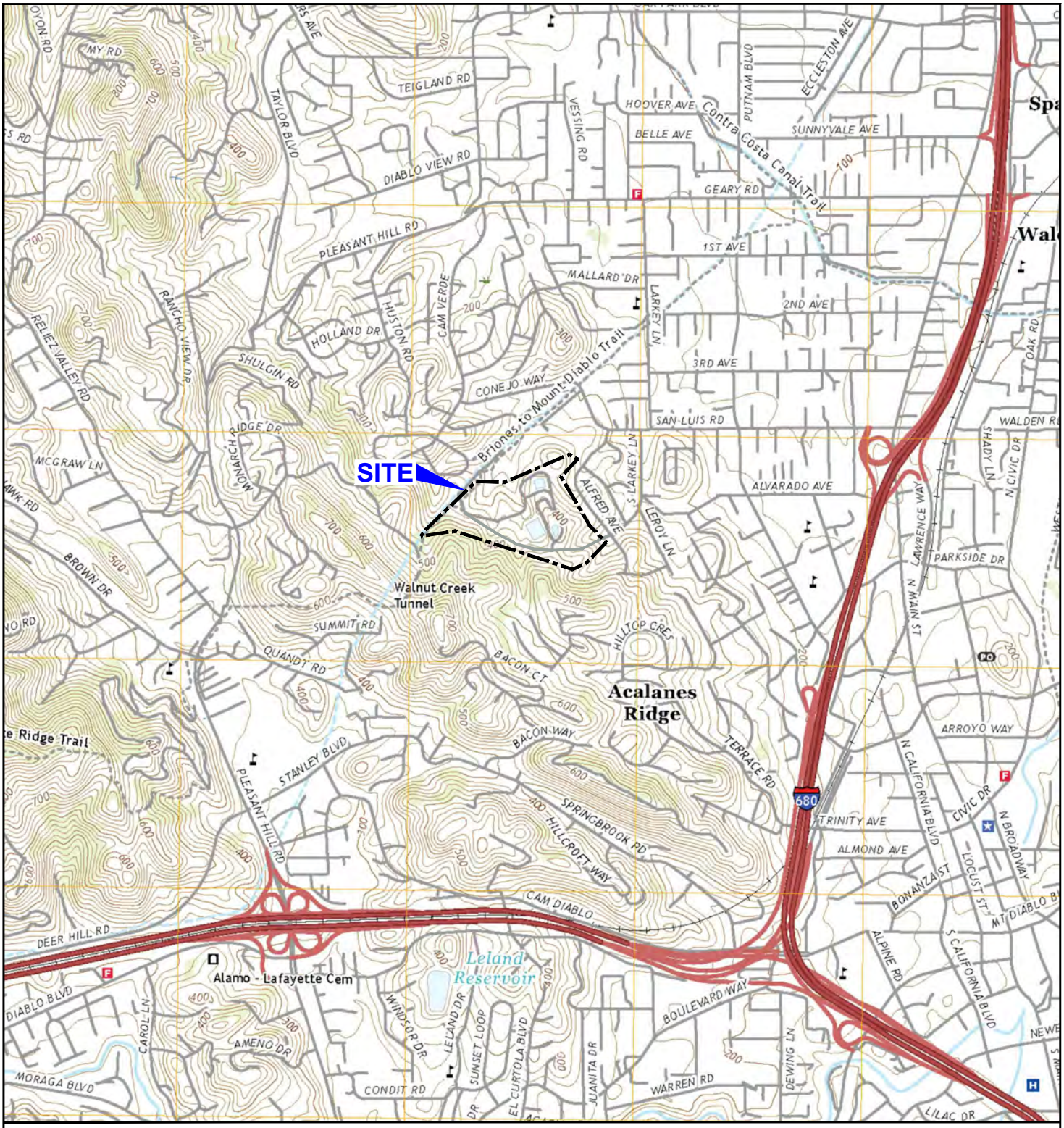
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List of Aerial Photographs Reviewed

Date	ID	Line	Frame	Scale
7-2-1959	AV334	22	2,3	1:9600
5-28-1969	AV905	11	11,12	1:12000
6-6-1978	AV1515	9	11,12	1:12000
4-20-1986	AV2862	7	1,2	1:12000
7-17-1990	AV3845	16	16,17	1:12000
10-8-1996	AV5200	18	14,15	1:12000
4-21-1999	AV6100	118	11,13	1:12000
4-2-2004	AV8660	19	12,13	1:15000

Sources:
Pacific Aerial Surveys, Novato, California

FIGURES



NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 2018

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FIGURE 1

SITE LOCATION

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22

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LEGEND

- NMB-1 — SOIL BORING (NINYO & MOORE, 2022)
- SB-1 — SONIC BORING (NINYO & MOORE, 2022)
- SL-1 — SEISMIC LINE (NINYO & MOORE, 2022)
- RETAINING WALL
- (1) — STAGING AREA
- MN-1 — SOIL BORING (CHEN, 2001)
- MM-1 — SOIL BORING (EBMUD, 1997)

PROPOSED FACILITIES					
ID	NAME	QTY	AREA (SF)	TYPE	DIMENSIONS LxWxD (FT)
1	BALLASTED FLOCCULATION	4	2592	CONC,BURIED	81x32x27
2	INTERMEDIATE OZONE PP	2	2000	CONC,BURIED	50x40x20
3	INTERMEDIATE OZONE CONTACTORS	2	3520	CONC,BURIED	80x44x26
4	OZONE GENERATOR BUILDING	1	10000	CONC	200x50x15
5	LOX TANKS AND VAPORIZERS	1	3320	CONC	83x40x27
6	PRE-OZONE PP	2	1500	CONC	50x30x20
7	COMBINED RECLAIM METERING VAULT	1	1000	CONC,BURIED	40x25x18
8	OZONE GAS DESTRUCT	5	200	COVERED CANOPY	20x10x12
9	DEWATERING BUILDING	1	4500	CONC	100x45x27
9A	TRUCK LOADING CANOPY	1	1350	PREFAB	45x30x20
10	THICKENED SOLIDS BLENDING TANKS	2	400	CONC	20'x20'x16
11	GRAVITY THICKENERS	4	5026	CONC	80'DIAx17
12	THICKENED SOLIDS PP	2	1500	CONC	50x30x27
13	SOLIDS XFER PP	1	300	CONC	20x15x17
14	ELECTRICAL FACILITIES	1	4500	PREFAB	100x45x18
15	MAINTENANCE BUILDING	1	7200	CONC	120x60x27
16	OZONE QUENCHING VAULT	2	750	CONC,BURIED	30x25x20
17	CHEMICAL STORAGE	1	2139	COVERED CANOPY	93x23x20
18	HYDROGEN PEROXIDE STATION	2	1200	COVERED CANOPY	40x30x15

PROPOSED FACILITY
PROPOSED FACILITY OUTSIDE OF PROJECT SCOPE
PROPOSED ACCESS ROAD

403982001.dwg 04/26/2022 AEK

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCES: GEOMATRIX, 1998; EBMUD, 2020, 2022

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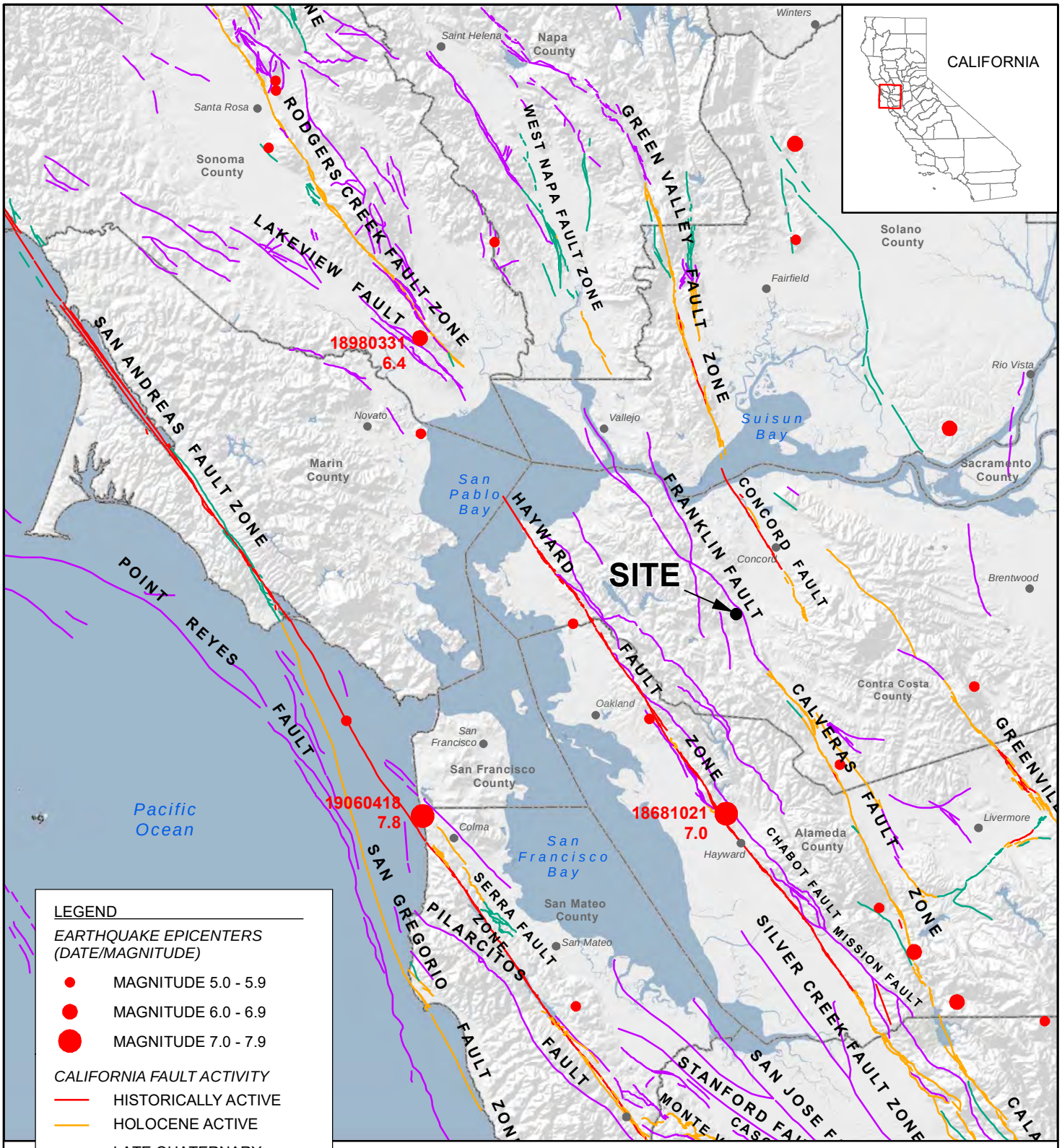
SCALE (FEET)
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FIGURE 2

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SITE PLAN AND EXPLORATION LOCATIONS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22



NOTE: DIRECTIONS, DIMENSIONS, AND LOCATIONS ARE APPROXIMATE

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, 2010, FAULT ACTIVITY MAP OF CALIFORNIA; CALIFORNIA GEOLOGICAL SURVEY, 2000, MAP SHEET MS 49

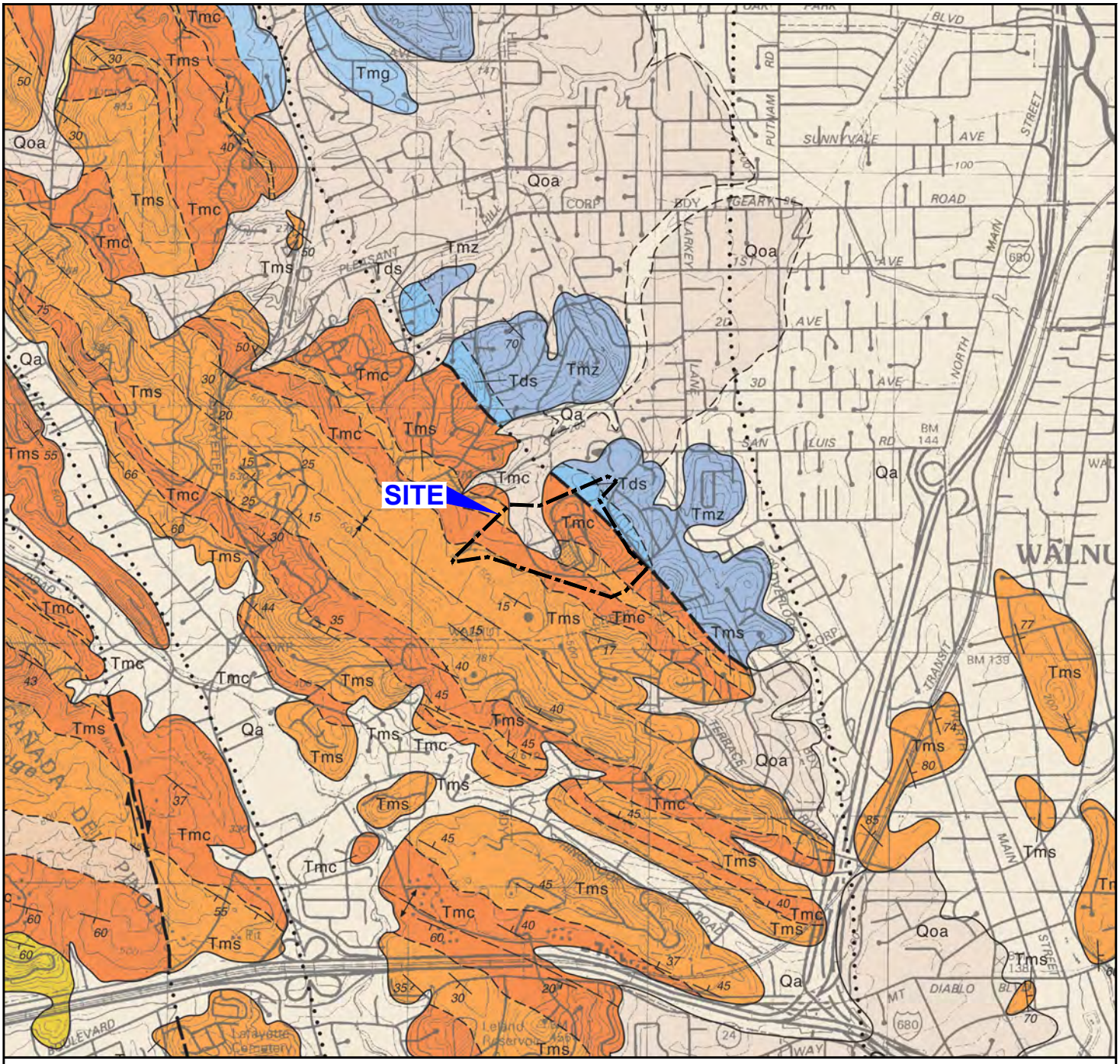
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FIGURE 3

FAULT LOCATIONS AND EARTHQUAKE EPICENTERS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 | 06/22

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LEGEND

Qa	ALLUVIAL GRAVEL, SAND, & CLAY OF VALLEY AREAS (HOLOCENE)	Tbr	BRIONES SANDSTONE (MIOCENE)	Tds	DOMENGE SANDSTONE (EOCENE)
Qoa	OLDER SURFICIAL SEDIMENTS (HOLOCENE)	Tms	MONTEREY FORMATION SANDSTONE (MIOCENE)	Tmz	MARTINEZ FORMATION CLAY SHALE/SILTSTONE (PALEOCENE)
Tor	ORINDA FORMATION: PEBBLE CONGLOMERATE, SANDSTONE, & CLAYSTONE (PLIOCENE)	Tmc	MONTEREY FORMATION CLAY SHALE/SILTSTONE (MIOCENE)		

THRUST FAULT

FAULT (DASHED WHERE INFERRED, DOTTED WHERE CONCEALED)

GEOLOGIC CONTACT

STRIKE AND DIP OF BEDDING

N

SCALE (FEET)

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: DIBBLEE, 2005

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FIGURE 4

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REGIONAL GEOLOGY

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22

LEGEND

- **NMB-1** — SOIL BORING (NINYO & MOORE, 2022)
- **SB-1** — SONIC BORING (NINYO & MOORE, 2022)
- **SL-1** — SEISMIC LINE (NINYO & MOORE, 2022)
- RETAINING WALL
- (1) — STAGING AREA
- **MN-1** — SOIL BORING (CHEN, 2001)
- **MM-1** — SOIL BORING (EBMUD, 1997)

LANDSLIDE APPROXIMATE LIMITS (QUERIED WHERE INFERRED, ARROWS SHOW DIRECTION OF MOVEMENT)

FAULT, DASHED WHERE APPROXIMATELY LOCATED, DOTTED WHERE CONCEALED, QUERIED WHERE INFERRED

AREA DISTURBED BY CONSTRUCTION (APPROXIMATE EXTENT, 2002-2007)

- Fill — FILL; SANDY TO SILTY CLAY (HISTORICAL)
- Qal — ALLUVIUM; SANDY CLAY, SAND, SANDY GRAVEL (HOLOCENE)
- Qt — ALLUVIAL GEOMORPHIC SURFACE; SANDY CLAY, SAND, SANDY GRAVEL (LATE PLEISTOCENE)
- Tmnu — UPPER NEROLY FORMATION; SANDSTONE (MIOCENE)
- Tmnl — LOWER NEROLY FORMATION; SANDSTONE & SHALE (MIOCENE)
- Tmbu — UPPER BRIONES FORMATION; SHALE MEMBER (MIOCENE)
- Tmbl — LOWER BRIONES FORMATION; SANDSTONE MEMBER (MIOCENE)
- Tmh — HAMBRE SANDSTONE (MIOCENE)
- Tem — MEGANOS FORMATION; CLAYSTONE MEMBER (EOCENE)

PROPOSED FACILITIES

ID	NAME	QTY	AREA (SF)	TYPE	DIMENSIONS LxWxD (FT)
1	BALLASTED FLOCCULATION	4	2592	CONC,BURIED	81x32x27
2	INTERMEDIATE OZONE PP	2	2000	CONC,BURIED	50x40x20
3	INTERMEDIATE OZONE CONTACTORS	2	3520	CONC,BURIED	80x44x26
4	OZONE GENERATOR BUILDING	1	10000	CONC	200x50x15
5	LOX TANKS AND VAPORIZERS	1	3320	CONC	83x40x27
6	PRE-OZONE PP	2	1500	CONC	50x30x20
7	COMBINED RECLAIM METERING VAULT	1	1000	CONC,BURIED	40x25x18
8	OZONE GAS DESTRUCT	5	200	COVERED CANOPY	20x10x12
9	DEWATERING BUILDING	1	4500	CONC	100x45x27
9A	TRUCK LOADING CANOPY	1	1350	PREFAB	45x30x20
10	THICKENED SOLIDS BLENDING TANKS	2	400	CONC	20'x20'x16
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12	THICKENED SOLIDS PP	2	1500	CONC	50x30x27
13	SOLIDS XFER PP	1	300	CONC	20x15x17
14	ELECTRICAL FACILITIES	1	4500	PREFAB	100x45x18
15	MAINTENANCE BUILDING	1	7200	CONC	120x60x27
16	OZONE QUENCHING VAULT	2	750	CONC,BURIED	30x25x20
17	CHEMICAL STORAGE	1	2130	COVERED CANOPY	93x23x20
18	HYDROGEN PEROXIDE STATION	2	1200	COVERED CANOPY	40x30x15

PROPOSED FACILITY
PROPOSED FACILITY OUTSIDE OF PROJECT SCOPE
PROPOSED ACCESS ROAD

FRANKLIN FAULT MAIN TRACE

EAST FAULTS

WEST FAULT

CENTRAL FAULT

LARKEY LANE

SCALE (FEET)
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NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCES: GEOMATRIX, 1998; EBMUD, 2020, 2022

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FIGURE 5

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SITE GEOLOGY

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22



LEGEND

Qaf — ARTIFICIAL FILL	LANDSLIDE	BEDROCK
Qal — ALLUVIAL DEPOSIT	COLLUVIUM	

NOTE: DIMENSIONS, DIRECTIONS, AND LOCATIONS ARE APPROXIMATE I REFERENCE: USGS, 1975

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SCALE (FEET)

0 2,000 4,000

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FIGURE 6

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REGIONAL LANDSLIDES

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22

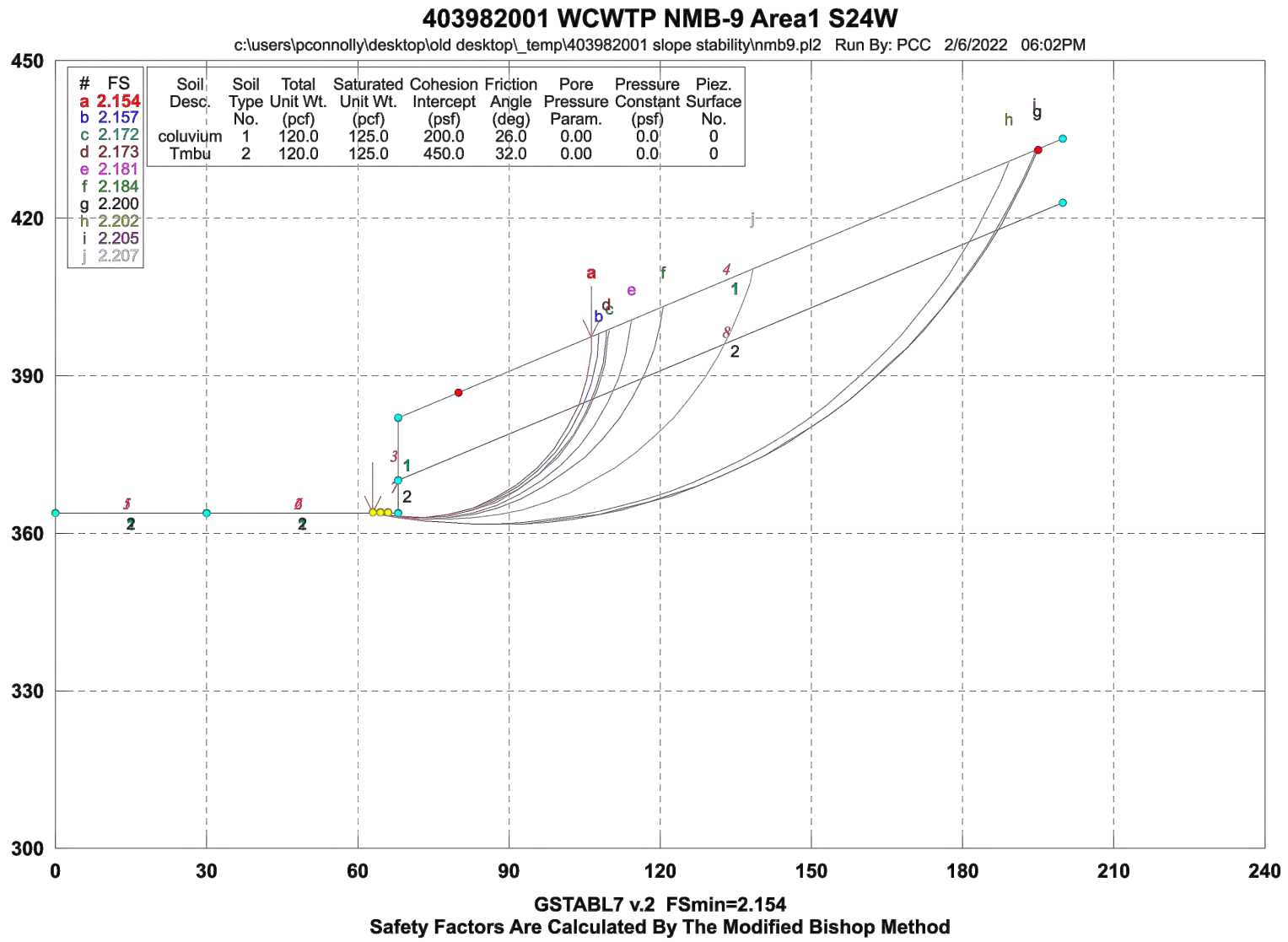


FIGURE 7A

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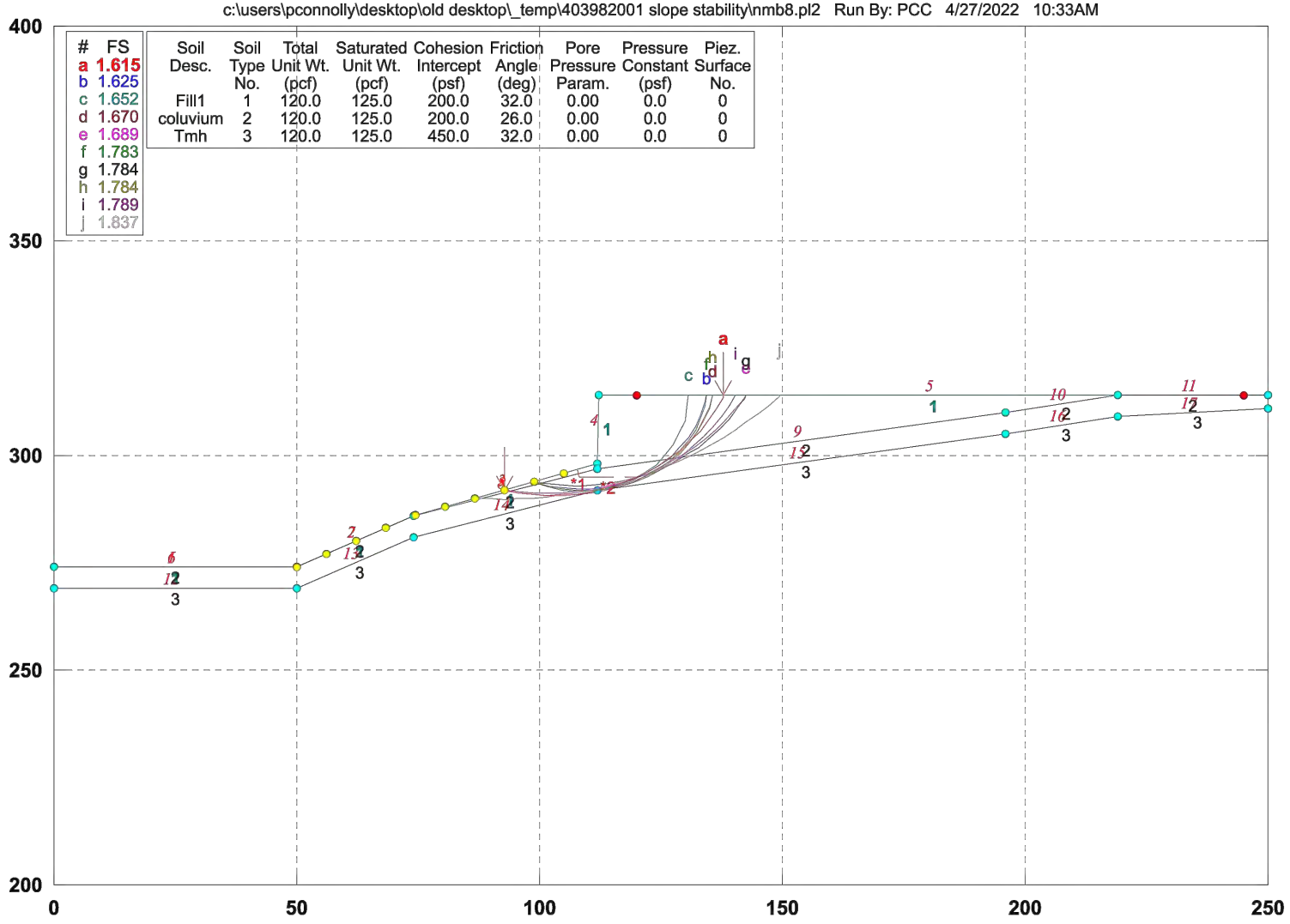
SLOPE STABILITY ANALYSIS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA
403982001 I 06/22

403982001.dwg 06/09/2022 AEK

403982001 WCWTP NMB-8 Item 11 S28W

c:\users\pconnolly\desktop\old desktop_temp\403982001 slope stability\nmb8.pl2 Run By: PCC 4/27/2022 10:33AM



GSTABL7 v.2 FSmin=1.615

Safety Factors Are Calculated By The Modified Bishop Method

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FIGURE 7B

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SLOPE STABILITY ANALYSIS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA

403982001 I 06/22

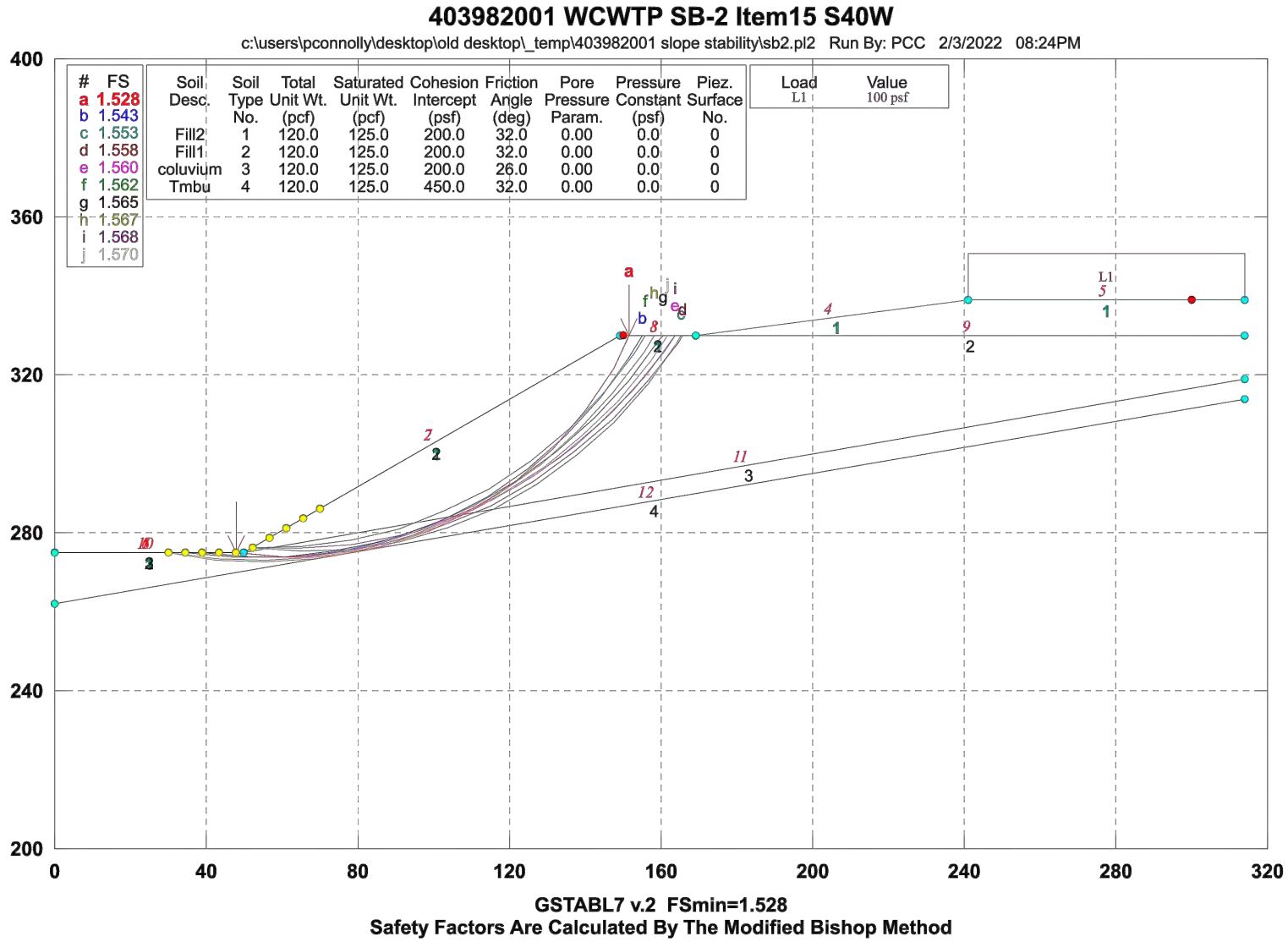


FIGURE 7C

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SLOPE STABILITY ANALYSIS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
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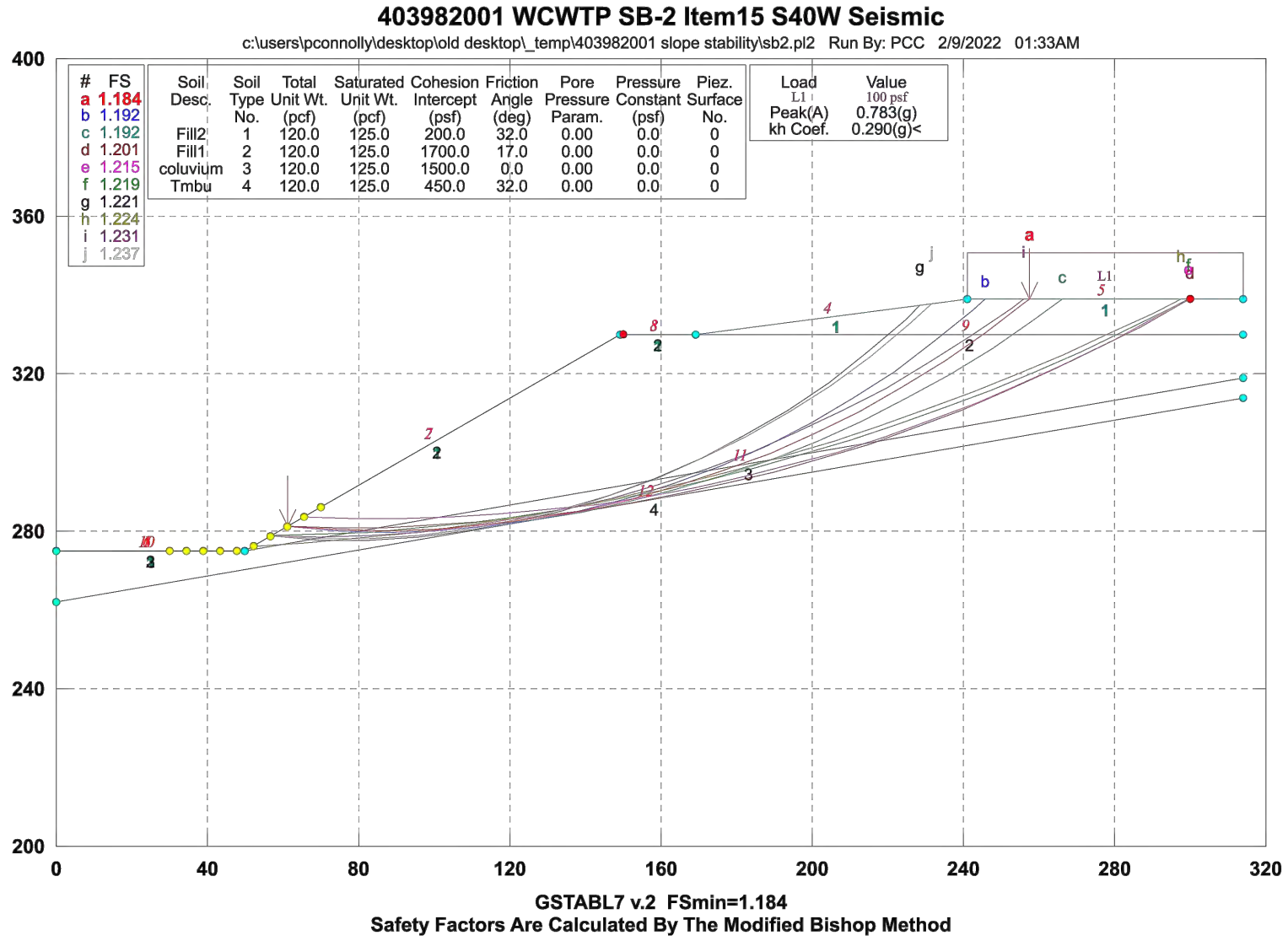


FIGURE 7D

SLOPE STABILITY ANALYSIS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following methods.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following methods.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3.0 inches, was lined with 6-inch long, thin brass liners with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring log as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass liners, sealed, and transported to the laboratory for testing.

Field Procedure for the Collection of Rock Core
Relatively undisturbed rock core was obtained in the field using a core barrel with a diamond bit on the outer barrel. The inner barrel was a HQ split tube core barrel. Rock core was retrieved from the exploratory borings, placed in rock core boxes and transported to the laboratory for further classification and evaluation.

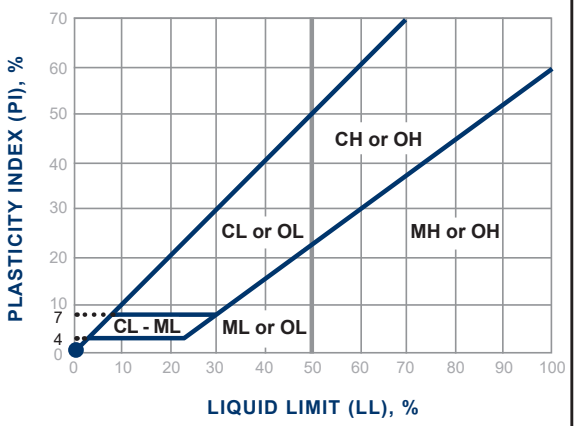
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

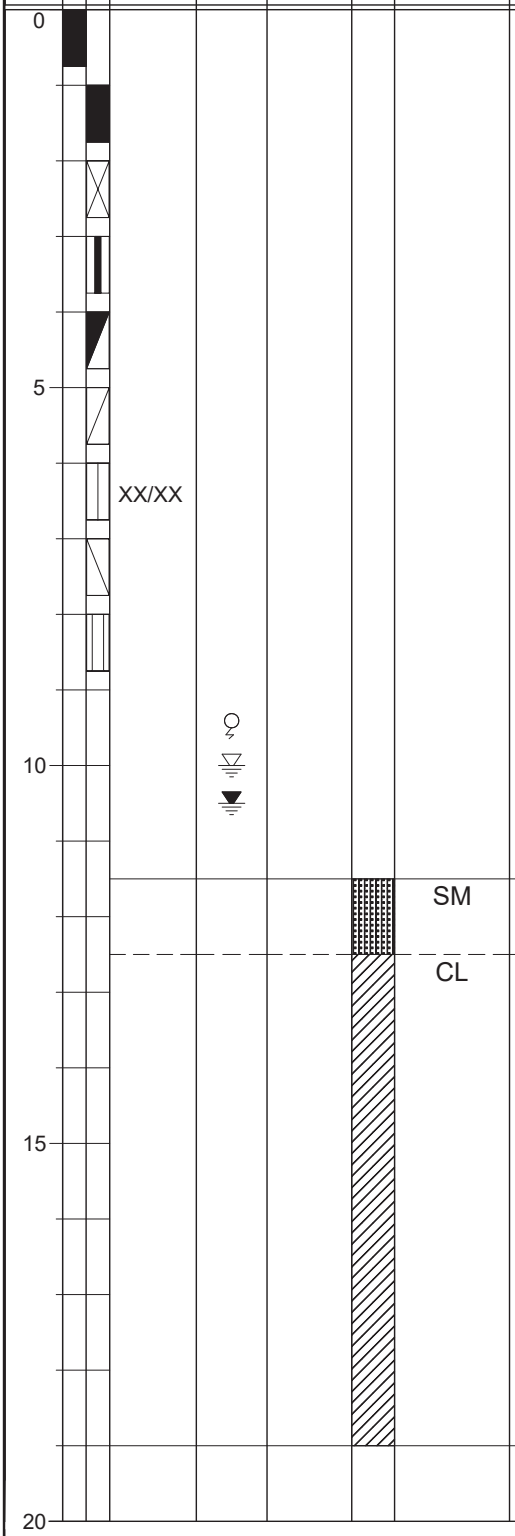
Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET



Bulk sample.

Modified split-barrel drive sampler.

No recovery with modified split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with a SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Seepage.

Groundwater encountered during drilling.

Groundwater measured after drilling.

SM

MAJOR MATERIAL TYPE (SOIL):

Solid line denotes unit change.

CL

Dashed line denotes material change.

Attitudes: Strike/Dip
b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Shear Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the boring.

LEGEND FOR ROCK CORE BORINGS

The following tables present the terminology used, along with definitions and abbreviations of the terminology used on the rock core boring logs presented in this appendix. The tables have been derived from the Engineering Geology Field Manual published by the U.S. Department of the Interior Bureau of Reclamation.

Tables A–1 through A–3: Rock Descriptions

Table A–1: Description of Rock Fracturing

	Unfractured	No Observed Fractures
1	Very slightly fractured	Core length recovery greater than 3’
2	Slightly to very slightly fractured	*
3	Slightly fractured	Core length recovery from 1’ to 3’
4	Moderately to slightly fractured	*
5	Moderately fractured	Core length recovery from 0.33’ to 1’
6	Intensely to moderately fractured	*
7	Intensely fractured	Core length recovery from 0.1’ to 0.33’
8	Very intensely to intensely fractured	*
9	Very intensely fractured	Core length recovery mostly chips and fragments
	*Note: Combinations of fracture densities are used where equal distribution of both fracture density characteristics are present over a significant core interval or exposure.	

Table A–2: Description of Rock Weathering

1	Fresh	No discoloration, surfaces not oxidized.
2	Slightly weathered	Discoloration and oxidation limited to surfaces.
3	Moderately weathered	Discoloration and oxidation extends from fractures, partial chemical alteration of some minerals.
4	Intensely weathered	Discoloration and oxidation throughout; chemical alteration of most minerals; Rock is friable.
5	Decomposed	Rock decomposed; resembles a soil.

Table A–3: Description of Rock Hardness

1	Extremely hard	Can’t be scratched with knife; can only be chipped with repeated heavy hammer blows.
2	Very hard	Can’t be scratched with knife; core can only be broken with repeated heavy hammer blows.
3	Hard	Can be scratched with a knife with difficulty; core can be broken with heavy hammer blow.
4	Moderately hard	Can be scratched with a knife with light to moderate pressure; core breaks with moderate hammer blow.
5	Moderately soft	Can be grooved 1/16 in. deep by knife with moderate to heavy pressure; core can be broken with light hammer blow.
6	Soft	Can be grooved easily with a knife with light pressure; core breaks with light to moderate manual pressure.
7	Very soft	Can be easily carved with a knife; core breaks with light manual pressure.

Tables A–4.1 through A–4.5: Descriptions and Abbreviations for Fractures

Table A–4.1: Surface Roughness

St	Stepped	Near normal steps and ridges occur on the fracture surface.
R	Rough	Large, angular asperities can be seen.
Mr	Moderately rough	Asperities are clearly visible and fracture surface feels abrasive.
Sm	Smooth	No asperities; smooth to the touch.

Table A–4.2: Surface Shape

Pl	Planar	Flat and uniform.
Un	Undulatory	Wavy and symmetrical.
Irr	Irregular	Non-uniform and symmetrical.

Table A–4.3: Aperture

T	Tight	No visible separation
So	Slightly Open	<1/32 inch
Mo	Moderately Open	1/32 inch – 1/8 inch
O	Open	1/8 inch – 3/8 inch
Mw	Moderately Wide	3/8 inch – 1.2 inches
W	Wide	>1.2 inches

Table A–4.4: Type of Infilling

Cl	Clay
Ca	Calcite
Ch	Chlorite
Fe	Iron Oxide
G	Gypsum
Mn	Manganese Oxide
Q	Quartz

Table A–4.5: Thickness of Infilling

Sta	Surface Staining	---
Vth	Very thin	<1/32 inch
Mth	Moderately thin	1/32 inch – 1/8 inch
Th	Thin	1/8 inch – 3/8 inch
Mtk	Moderately thick	3/8 inch – 1.2 inches
Tk	Thick	>1.2 inches

DATE DRILLED 9/10/2021 BORING NO. SB-1

GROUND ELEVATION 363' ± PD SHEET 1 OF 1

METHOD OF DRILLING 6" Sonic Dual Tube, Geoprobe 8140LS (Geo-Ex)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown, dry, clayey SAND; like gravel.
				30.8				
10								Dark brown, moist.
							CH	COLLUVIUM: Dark brown, moist, fat CLAY; little sand.
				25.1				
							ROCK	BRIONES FORMATION: Olive brown, moist, weakly cemented, SILTSTONE; highly weathered.
				24.7				
20				42.1				Moderately cemented.
				25.7				
30								Total depth = 28 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
40								

FIGURE A- 1

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 9/10/2021 BORING NO. SB-2

GROUND ELEVATION 355' ± PD SHEET 1 OF 2

METHOD OF DRILLING 6" Sonic Dual Tube, Geoprobe 8140LS (Geo-Ex)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Gray, dry, clayey SAND; little gravel.
						SM	Light brown, moist, silty SAND; little gravel.
						CL	Brown, moist, sandy lean CLAY; little gravel.
			27.4				
10			23.4				Dark gray.
			21.2			GC	Dark gray, moist, clayey GRAVEL; some sand.
						CL	Brown, moist, sandy CLAY.
20			27.1				Orange streaks.
30							Olive brown; trace sand. Trace sand.
			26.8				Tree root.
						ROCK	BRIONES FORMATION: Light gray, moist, weakly cemented, SANDSTONE and SILTSTONE; weathered.
40							

FIGURE A- 2

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 9/10/2021 BORING NO. SB-2

GROUND ELEVATION 355' ± PD SHEET 2 OF 2

METHOD OF DRILLING 6" Sonic Dual Tube, Geoprobe 8140LS (Geo-Ex)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40								BRIONES FORMATION: (Continued) Dark gray, moist, weakly cemented, SILTSTONE.
								Total depth = 45 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

DATE DRILLED 9/27/2021 BORING NO. NMB-1

GROUND ELEVATION 387' ± PD SHEET 1 OF 1

METHOD OF DRILLING 2.5" HQ Wireline Core, CME 75 Track Mounted Rig (Geo Ex)

DRIVE WEIGHT 140 lbs (Auto Trip) DROP 30 inches

SAMPLED BY TBG LOGGED BY DCS REVIEWED BY PCC

DEPTH (feet)	Samples: Bulk	Samples: Driven	BLOWS/FOOT	INTERVAL CORED (feet)	RECOVERY (%)	RQD (%)	FRACTURE FREQ. (per ft)	FRACTURE DIP (degrees)	DRILLING RATE (min/ft.)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0											ROCK	HAMBRE FORMATION: Light gray and brown, weakly to strongly cemented, SILTSTONE and sandy SILTSTONE; intensely to moderately fractured, intensely to moderately weathered, soft to moderately hard.
			78									
			50/5"									Moisture content = 25.8%; Dry Density = 77.2 pcf.
10												
			50/3"									
			50/5"									
				1.7	100	50	2	75	5.9			Core Run 1 - 14.8' to 16.5' - Recovery 100% - RQD=50. Mr, Pl, So, G, Vth
								65				Mr, Pl, So, G, Mth
							3					
							2	55				Core Run 2 - 16.9' to 21.5' - Recovery 93% - RQD=66.
				5	93	66	1		2.4			Mr, Pl, So, Ca, Mth
20												Moderately hard, carbonate cement, iron oxide staining on bioturbation structures, appears massive.
							2					
							2	75				Sm, Pl, So, G, Mth
							2					Core Run 3 - 21.5' to 26.5' - Recovery 98% - RQD=43.
				5	98	43	1	85	3.4			Mr, Pl, So, Ca, Mth
							3					
							2					
30												Total Depth = 26.5 feet. Boring was backfilled with cement grout, on 9/27/2021, after drilling. Groundwater was not measurable due to drilling fluid. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020) reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40												

FIGURE A- 4

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 9/27/2021 BORING NO. NMB-2

GROUND ELEVATION 383' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ROCK	HAMBRE FORMATION: Light gray, dry, weathered, silty SANDSTONE; iron oxide staining on fractures.
			50/3"					
			50/5"					
10								Total depth = 9.4 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

DATE DRILLED 9/27/2021 BORING NO. NMB-3

GROUND ELEVATION 377' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: 4.5 INCHES.
						SC	AGGREGATE BASE: 5.5 INCHES.
							FILL:
							Gray and brown, moist, medium dense, clayey SAND; trace gravel.
		38				CH	Mottled gray, brown, reddish brown, moist, hard, fat CLAY; some sand.
						SM	Gray and brown, moist, dense, silty SAND; pieces of sandstone in silty matrix.
10		51					
						CL	Brown, moist, hard, sandy lean CLAY; trace gravel composed of pieces of bedrock.
		44	24.0	94.0		SC	Gray and brown, moist, dense, clayey SAND; little gravel.
		45	26.5	91.5		SM	Gray, moist, dense, silty SAND; little gravel.
20							
		59					
		50/2"				ROCK	HAMBRE FORMATION:
							Gray, weathered, silty SANDSTONE and sandy SILTSTONE.
30							Total depth = 28.7 feet.
							Backfilled with cement grout and on-site soils after completion of drilling.
							Notes:
							Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
							The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
40							

FIGURE A- 6

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 9/28/2021 BORING NO. NMB-4

GROUND ELEVATION 363' ± PD SHEET 1 OF 2

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Gray and brown, dry, dense, clayey SAND; little gravel. Asphalt chunk.
		57	19.2	89.5			
		17	26.8	91.3		SM	Light to medium brown, moist, medium dense, silty SAND; little gravel.
		75	18.8	101.1		CL	Gray and brown, moist, sandy lean CLAY; trace gravel.
10							
		33	28.0	94.1		SC CL	Brown, moist, medium dense, clayey SAND; scattered rootlets; brick debris. Brown, moist, hard, lean CLAY.
		16	13.6	113.1		GC	Dark gray and brown, moist, medium dense, clayey GRAVEL with sand; scattered asphalt debris; scattered rootlets.
20							
		64					Medium brown to gray; scattered rootlets. Olive brown; dense.
		50				CL	Brown, moist, hard, sandy lean CLAY with gravel; asphalt chunks.
30							
		45	26.6	93.9			Trace gravel.
		56				ROCK	BRIONES FORMATION:
40							

FIGURE A- 7

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 9/28/2021 BORING NO. NMB-4

GROUND ELEVATION 363' ± PD SHEET 2 OF 2

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40								Gray, moist, weakly cemented, SANDSTONE; weathered; iron oxide staining on fractures.
								Total depth = 40 feet.
								Backfilled with cement grout and on-site soils after completion of drilling.
								Notes:
								Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

DATE DRILLED 9/28/2021 BORING NO. NMB-5

GROUND ELEVATION 319' ± PD SHEET 1 OF 2

METHOD OF DRILLING 4" Solid Stem Auger, 4" mud rotary CME-75 Track Mounted (Geo-

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0		50/5"	17.7	99.1		SC	FILL: Brown, dry, very dense, clayey SAND with gravel.
		50/6"	20.4	100.6		CL	Brown, moist, hard, lean CLAY.
						ROCK	HAMBRE FORMATION: Light brown and brown, dry, moderately cemented, sandy SILTSTONE and silty SANDSTONE; weathered.
		50/5"					
10		50/6"	18.2				
		50/3"					
20							Switched to rotary wash @20'.
		50/6"	36.4				
		50/5"					Gray.
30		50/2"					
		50/3"	29.8				
40							

FIGURE A- 9

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 6/22

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 9/28/2021 BORING NO. NMB-5

GROUND ELEVATION 319' ± PD SHEET 2 OF 2

METHOD OF DRILLING 4" Solid Stem Auger, 4" mud rotary CME-75 Track Mounted (Geo-

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY SSA REVIEWED BY PCC

40

Total depth = 38.8 feet.

Backfilled with cement grout and on-site soils after completion of drilling.

Notes:

Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.

50

60

70

80

DATE DRILLED 10/08/2021 BORING NO. NMB-6

GROUND ELEVATION 352' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Brown, moist, medium dense, clayey SAND with gravel; scattered asphalt.
10		19	23.0	98.6			Brown and olive brown.
		45	26.9	94.6			Orange and gray, dense. Grayish brown.
20		76	31.5	91.3		ROCK	BRIONES FORMATION: Gray, moist, weakly cemented, SILTSTONE; weathered; iron oxide staining on fractures; gypsum veins.
		69					Light brown.
30							Total depth = 25 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
40							

DATE DRILLED 10/08/2021 BORING NO. NMB-7

GROUND ELEVATION 331' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	FILL: Brown, moist, clayey SAND.
		46	25.1	96.7			Brown, moist, dense, clayey SAND; little gravel.
10		76	23.5	96.2			Very dense.
		35	30.7	82.7		CL	Brown, moist, hard, sandy lean CLAY.
20		36	22.1	91.3		CH	COLLUVIUM: Dark brown, moist, hard, fat CLAY; little sand.
		76	29.8	88.6		ROCK	BRIONES FORMATION: Olive brown, moist, weakly cemented, SILTSONE; weathered; iron oxide staining on fractures.
30		73					
		84	22.9	96.8			Moderately cemented.
							Total depth = 35 feet. Backfilled with cement grout and on-site soils after completion of drilling. See Figure A-1 for groundwater and elevation notes.
40							

FIGURE A- 12

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 10/08/2021 BORING NO. NMB-8

GROUND ELEVATION 303' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0			50/6"				CL	FILL: Brown, dry, hard, sandy lean CLAY.
			50/6"				ROCK	HAMBRE FORMATION: Gray, dry, moderately cemented, silty SANDSTONE; weathered. Light brown and gray; strongly cemented.
10			50/3"					
								Total depth = 10.8 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

FIGURE A- 13

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

DATE DRILLED 10/08/2021 BORING NO. NMB-9

GROUND ELEVATION 382' ± PD SHEET 1 OF 1

METHOD OF DRILLING 4" Solid Stem Auger, CME-75 Track Mounted (Geo-Ex)

DRIVE WEIGHT 140 lbs (automatic) DROP 30 inches

SAMPLED BY TBG LOGGED BY TBG REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SC	COLLUVIUM: Brown, dry, medium dense, clayey SAND.
						CL	Dark brown, moist, hard, sandy lean CLAY.
		29					
10		38	21.2	95.7			
						ROCK	BRIONES FORMATION: Gray, dry, weakly cemented, sandy SILTSTONE; weathered; iron oxide staining on fractures.
		69	27.6	90.7			
20		50/3"					Moderately cemented.
		50/5"					Dark gray; strongly cemented.
							Total depth = 25.9 feet. Backfilled with cement grout and on-site soils after completion of drilling. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of topographic information (EBMUD, 2020). It is not sufficiently accurate for preparing construction bids and design documents.
30							
40							

FIGURE A- 14

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



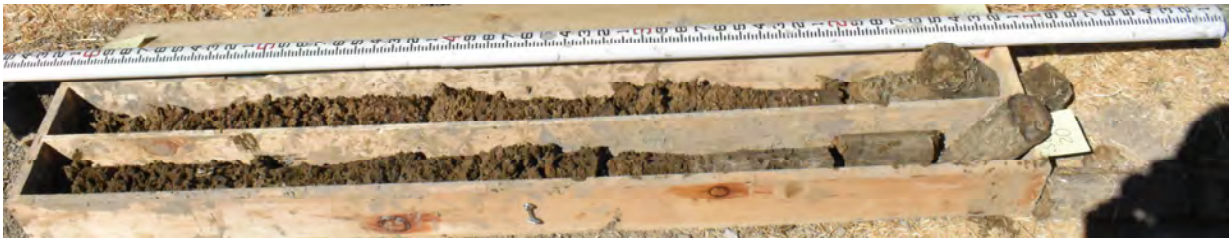
Photograph 1: **Boring SB-1 from depth 0 to 5 feet below ground surface**



Photograph 2: **Boring SB-1 from depth 5 to 10 feet below ground surface**



Photograph 3: **Boring SB-1 from depth 10 to 15 feet below ground surface**



Photograph 4: **Boring SB-1 from depth 15 to 20 feet below ground surface**



Photograph 5: **Boring SB-1 from depth 20 to 25 feet below ground surface**



Photograph 6: **Boring SB-1 from 25 to 28 feet below ground surface (bottom run)**



Photograph 7: Boring SB-2 from depth 1.5 to 5 feet below ground surface



Photograph 8: Boring SB-2 from depth 1.5 to 5 feet below ground surface



Photograph 9: Boring SB-2 from depth 8 to 10 feet below ground surface



Photograph 10: **Boring SB-2 from depth 10 to 15 feet below ground surface**



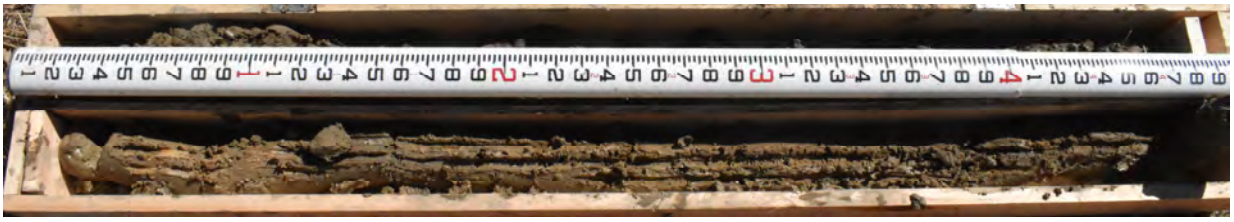
Photograph 11: **Boring SB-2 from depth 15 to 17 feet below ground surface**



Photograph 12: **Boring SB-2 from depth 17 to 20 feet below ground surface**



Photograph 13: Boring SB-2 from depth 20 to 25 feet below ground surface



Photograph 14: Boring SB-2 from depth 25 to 30 feet below ground surface



Photograph 15: Boring SB-2 from depth 30 to 35 feet below ground surface



Photograph 16: Boring SB-2 from depth 35 to 40 feet below ground surface



Photograph 17: **Boring SB-2 from depth 40 to 45 feet below ground surface**



Photograph 18: **Boring NMB-1 Run 1 from depth 14.8 to 16.5, Run 2 from depth 16.9 to 21.5, and Run 3 from depth 21.5 to 26.5 feet**

APPENDIX B

Boring Logs from Previous Investigations

WALNUT CREEK WATER TREATMENT PLANT
IMPROVEMENTS PROJECT

WALNUT CREEK, CALIFORNIA

GEOTECHNICAL INVESTIGATION REPORT

June 2001

by

East Bay Municipal Utility District
Engineering Services Division
Materials Engineering Section

Prepared by: _____

A. B. Yiadom
Associate Civil Engineer



EBMUD

Approved by: _____

F. M. Starr
Senior Civil Engineer

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE-GRAINED SOILS more than half is larger than No. 200 sieve	GRAVELS more than half coarse fraction is larger than No. 4 sieve size	CLEAN GRAVELS (less than 5% fines)	GW	well graded gravels, gravel-sand mixtures, little or no fines
			GP	poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS with fines	GM	silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS more than half coarse fraction is smaller than No. 4 sieve size	CLEAN SANDS (less than 5% fines)	SW	well graded sands, gravelly sands, little or no fines
			SP	poorly graded sand or gravelly sands, little or no fines
		SANDS with fines	SM	silty sands, sand-silt mixtures, non-plastic fines
			SC	clayey sands, sand-clay mixtures, plastic fines
FINE-GRAINED SOILS (fines) more than half is smaller than No. 200 sieve	SILTS AND CLAYS liquid limit is less than 50		ML	inorganic silts and very fine sands,rock flour, silty or clayey fine sands, or clayey silts with slight plasticity
			CL	inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS liquid limit is greater than 50		MH	inorganic silts, micaceous or diatomacious fine sandy or silty soils, elastic silts
			CH	inorganic clays of high plasticity, fat clays
			OH	organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		PT	peat and other highly organic soils

PENETRATION RESISTANCE

(RECORDED AS BLOWS/FOOT)

SANDS AND GRAVELS

RELATIVE DENSITY	BLOWS/FOOT *
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CLAYS AND SILTS

CONSISTENCY	BLOWS/FOOT *	STRENGTH **
VERY SOFT	0 - 2	0 - 1/4
SOFT	2 - 4	1/4 - 1/2
MEDIUM STIFF	4 - 8	1/2 - 1
STIFF	8 -16	1 - 2
VERY STIFF	16 - 32	2 - 4
HARD	OVER 32	OVER 4

* Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D (1-3/8 inch I.D.) split-barrel sampler (ASTM-1586 standard penetration test).

** Unconfined compressive strength in tons per square foot

DEFINITION OF TERMS

GRAIN SIZES

U.S. STANDARD SERIES SIEVE

#200 #50 #16 #4 3/4" 3" 12"

SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRADATION

Well-graded gravels and sands: Specimen with a ***wide*** range of grain sizes.
Poorly-graded gravels and sands: Specimen with a ***narrow*** range of grain sizes.

MOISTURE CONDITION

DRY SLIGHTLY DAMP DAMP MOIST VERY MOIST WET (SATURATED)

EBMUD

WALNUT CREEK WTP IMPROVEMENTS PROJECT

Date: 4/1/2001

Figure:

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LEGEND

EBMUD

SAMPLE #

MC	Modified California Sampler
SPT	Standard Penetration Test Sampler
1	First Sample Interval (2=second, etc.)
I	First Sample Tube Recovered (II=second, etc.)

TEST RESULTS

LL	Liquid Limit (%)	} ATTERBERG LIMITS
PL	Plastic Limit (%)	
PI	Plasticity Index (%)	
SG	Specific Gravity	
FINES	Percentage Passing #200 Sieve	
TX (1000)	Triaxial Shear Strength, psf (Confining Stress) (Strength taken at peak stress ratio)	

UU - Unconsolidated Undrained
CU - Consolidated Undrained

NOTES

$\underline{\nabla}_{10/27}$ Indicates approximate ground water level on October 27.

Elevations are referenced to National Geodetic Vertical Datum (1929) [redacted]

Blow counts shown are actual counts recorded for the 12 inches of penetration after the first 6 inches. 50/3" indicates 50 blows to drive the sampler 3" total. Each sampler was driven as follows:

Sampler Type	Hammer Wt. (pound)	Hammer Drop (inches)	Sampler penetration (inches)
MC	140	30	18
SPT	140	30	18

PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS

I. CONSOLIDATION OF SEDIMENTARY ROCKS: Usually determined from unweathered samples. Largely dependant on cementation.

U = Unconsolidated
P = Poorly consolidated
M = Moderately consolidated
W = Well consolidated

II. BEDDING OF SEDIMENTARY ROCKS

SPLITTING PROPERTY	THICKNESS (IN FEET)	STRATIFICATION
Massive	Greater than 4.0	Very thick-bedded
Blocky	2.0 to 4.0	Thick-bedded
Slabby	0.2 to 2.0	Thin-bedded
Flaggy	0.05 to 0.2	Very thin-bedded
Shaly or Platy	0.01 to 0.05	Laminated
Papery	Less than 0.01	Thinly laminated

III. FRACTURING

INTENSITY	SIZE OF PIECES (IN FEET)
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

IV. HARDNESS

1. SOFT - Reserved for plastic material alone.
2. LOW HARDNESS - Can be gouged deeply or carved easily with a knife.
3. MODERATELY HARD - Can be readily scratched by a knife; scratch leaves a heavy trace of dust and is readily visible after the powder is blown away.
4. HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visible.
5. VERY HARD - Cannot be scratched with knife blade; leaves a metallic streak.

V. STRENGTH

1. PLASTIC or very low strength
2. FRIABLE - Crumbles easily by rubbing with fingers.
3. WEAK - An unfractured specimen of such material will crumble under light hammer blows.
4. MODERATELY STRONG - Specimen will fracture with a few hammer blows.
5. STRONG - Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

VI. WEATHERING The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

D. DEEP - Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates, and/or clay or silt.
M. MODERATE - Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasional intense discoloration. Moderately coated fractures.
L. LITTLE - No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent or localized discoloration. Few stains on fracture surfaces.
F. FRESH - Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

WALNUT CREEK WTP
IMPROVEMENTS PROJECT

	DRAWN BY: A. B. YIADOM
	SCALE: NONE
	DATE: April 2001

LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 28, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -1

SURFACE ELEV. : 376.8

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown silty CLAY (CH), stiff (fill)
SG=2.75 LL=54, PI=33	27.4		19		MC-1	
FINES=74.4% LL=52, PI=30	32.2		17	5	SPT-2	
LL=40, PI=8	27.4		30		SPT-3	olive brown sandy SILT (ML), very stiff (fill)
TX-CU=2700 (1300) SG=2.69 FINES=35.8%	27.4	94.5		10	P-4	brown silty SAND (SM), dense, with gravel and a few cobbles. Gravel is crushed sandstone, siltstone and claystone (fill)
FINES=47.2%	30.4		34		SPT-5	
	27.6		38	15	SPT-6	
FINES=44.6%	29.6		26		SPT-7	
TX-CU=3100 (2600) FINES=34.9%	25.2	96.2		20	P-8	
FINES=42.6%	24.9		47		SPT-9	
	22.1		34	25	SPT-10	
FINES=44.3%	21.6		52		SPT-11	
				30		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 28, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -1

SURFACE ELEV. : 376.8

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				30	P-12	brown silty SAND (SM), dense, with gravel and a few cobbles. Gravel is crushed sandstone, siltstone, and claystone (fill)
FINES=38.8%	24.6		47		SPT-13	
	22.3		34	35	SPT-14	
FINES=42.9%	24.8		37		SPT-15	
TX-CU=6600 (5100)	22.1	97.2		40	P-16	
FINES=33.7%						
LL=41, PI=14	25.7		30		SPT-17	
				45		
FINES=27.9%	24.2		49		SPT-18	
TX-CU=6400 (5900)						
FINES=36.5%	24.1	92.7			P-19	
TX-CU=4950 (6600)				50	P-20	
FINES=44.8%	32.1	89.4			P-21	
LL=39, PI=9				55		
	30.3		56		SPT-22	dark brown sandy SILT (ML), hard (fill)
TX-CU=3150 (2400) 6100 (4900) 8550 (7100)	30.2	89.8			P-23	
				60		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 28, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -1

SURFACE ELEV. : 376.8

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
	30.5		35	60	SPT-24	mottled dark brown sandy SILT (ML), hard (fill)
			50	65	SPT-25	
				70	P-26	grey SANDSTONE, SILTSTONE and CLAYSTONE, friable to moderately strong, moderately weathered
			50/3"	75	SPT-27	Bottom of Boring at 76.0'
				80		
				85		
				90		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 5, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -2

SURFACE ELEV. : 359'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		dark brown sandy SILT (ML), stiff (fill)
FINES=52.7% LL=42, PI=25	17.9		20		SPT-1	
				5		
LL=53, PI=31	31.5		14		SPT-2	gravelly
						brown silty SAND (SM), medium dense (fill)
FINES=38.3% LL=54, PI=22	34.5		25		SPT-3	
						reddish brown silty SAND (SM) with gravel. Gravel is crushed sandstone, siltstone, and claystone (fill)
TX-CU=1400 (1100) SG=2.61 LL=40, PI=7	27.7	91.3		10	P-4	
FINES=30.4%	28.6		29		SPT-5	
				15		
	31.6		33		SPT-6	
FINES=32.2%	35.3		23		SPT-7	
				20		
					P-8	
FINES=36.3%	26.2		28		SPT-9	
						brown SANDSTONE, SILTSTONE, CLAYSTONE, friable to moderately strong, deep to moderately weathered
				25		
			50/5"		SPT-10	
			50/3"		SPT-11	
				30		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 5, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -2

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				30	P-12	orange-brown clayey SANDSTONE, weak, deeply weathered
			50/6"		SPT-13	
						Bottom of Boring at 33.0'
				35		
				40		
				45		
				50		
				55		
				60		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 3, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -3

SURFACE ELEV. : 376'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown clayey SILTY SAND (SM), medium dense with gravel. Gravel is crushed sandstone, siltstone and claystone (fill)
FINES=29%	24.3		49		SPT-1	
				5		
LL=48;PI=16	31.4		16		SPT-2	sandy CLAY layer
	36.1		24		SPT-3	
LL=45;PI=16						
TX-CU=1750 (2400)	24.5	98.5		10		
LL=30;PI=4 FINES=23.4%					P-4	
						sandy CLAY layer
			28		SPT-5	clayey SAND layer
				15		
	32.8		29		SPT-6	
	25.1		30		SPT-7	
				20		
TX-CU=5700 (4500)	36.4	81.8			P-8	
LL=40;PI=13 FINES=29.7%						
	35.4		26		SPT-9	
				25		
	34.5		22		SPT-10	
	22.3		48		SPT-11	
				30		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 3, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -3

SURFACE ELEV. : 376'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				30	P-12	brown silty SAND (SM), medium dense, with gravel. Gravel is crushed sandstone, siltstone and claystone (fill)
LL=36, PI=5	25.9		28		SPT-13	
TX-CU=4400 (4500)				35		
SG=2.65	25.8	95.2			P-14	
FINES=29.4% LL=42, PI=12	29.3		35		SPT-15	grey
FINES=27.3%				40	P-16	
LL=39, PI=8	31.2		27		SPT-17	
LL=37, PI=6	30.6		45	45	SPT-18	
TX-CU=6200 (6400) FINES=35.4% LL=44, PI=13	30.8		40		SPT-19	Orange-brown
FINES=34.7	29.8	90.0		50	P-20	
FINES=40.7% LL=39, PI=5	24.8		55		SPT-21	greyish-brown SANDSTONE and SILTSTONE, friable to moderately strong, deep to moderately weathered
			50	55	SPT-22	
			50/5"		SPT-23	
				60		

LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 3, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -3

SURFACE ELEV. : 376'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-CU=7550 (7610)	27.0	95.9		60	P-24	dark grey SILTSTONE, moderately strong, moderately weathered
			50/3"	65	SPT-25	Bottom of Boring at 65.5'
				70		
				75		
				80		
				85		
				90		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
LOCATION : Walnut Creek Clearwell
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 5, 1997
CONTRACTOR : Pitcher Drilling
EQUIPMENT : Failing 750
JOB NUMBER : 56248

EBMUD

BORING NO MM -4

SURFACE ELEV. : 376'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown sandy CLAY (CL), hard (fill)
LL=36, PI=16	12.6		36		SPT-1	
				5		tan silty SAND (SM), medium dense, with gravel (fill)
FINES=39.2%	23.8		22		SPT-2	
LL=45, PI=17	33.0		31		SPT-3	
				10		
TX-CU=1800 (1200)	32.9	84.0			P-4	
FINES=23.1%						
	32.9		27		SPT-5	reddish brown
				15		
LL=58, PI=29	35.3		27		SPT-6	reddish brown gravelly CLAY (CH) with sand, very stiff (fill)
LL=55, PI=28	31.9		25		SPT-7	silt layer
TX-CU=2650 (2600)	33.4	87.9		20		
SG=2.72 FINES=18.4% LL=55, PI=22					P-8	alternating layers of brown silty SAND (SM), medium dense; and grey sandy SILT (ML), stiff, both with gravel and some cobbles (fill)
	36.8		28		SPT-9	
				25		
FINES=51.2%	24.3		41		SPT-10	
LL=38, PI=9	27.3		35		SPT-11	
				30		

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
DATE : November 5, 1997
LOCATION : Walnut Creek Clearwell
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NUMBER : 56248

EBMUD

BORING NO MM -4

SURFACE ELEV. : 376'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-CU=4300 (4100) 7050 (5000) 9750 (7000)	25.3	98.1	50/5"	30 35 40 45 50 55 60	P-12 SPT-13	brown sandy CLAY (CL), hard (fill) tan SILTSTONE, weak to moderately strong, deeply weathered Bottom of Boring at 34.0'

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LOG OF BORING

PROJECT : Dam Seismic Evaluation
DATE : November 6, 1997
LOCATION : Walnut Creek Clearwell
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 1500
CHECKED BY : Derek Kawaii
JOB NUMBER : 56248

EBMUD

BORING NO MM -5

SURFACE ELEV. : 315'

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown gravelly CLAY (CL), very stiff, with some sand (fill)
			25		SPT-1	orange-brown sandstone COBBLES and GRAVEL (GP),medium dense (fill)
				5		
LL=47, PI=18	27.5		21		SPT-2	brown gravelly SILT (ML), very stiff (fill)
					SPT-3	
LL=45, PI=16	32.7		11			
				10	P-4	brown SANDSTONE, moderately strong, moderately weathered
					P-5	
			50/4"		SPT-6	
				15		Bottom of Boring at 14.0'
				20		
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : November 4, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NUMBER : 25692

EBMUD

BORING NO MN-2

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		tan, sandy, silty CLAY (CL)
LL = 36, PI = 5	14		78		SPT-1	tan, deeply to moderately weathered, sandy SILTSTONE
% fines = 74.2 LL = 34, PI = 1	17		80	5	SPT-2	
						Bottom of Boring at 6.5'
				10		
				15		
				20		
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 30, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NUMBER : 25692

EBMUD

BORING NO MN-3

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown, sandy CLAY (CL)
% fines = 35.9			40		SPT-1	
						gravelly, silty SAND (SC)
				5		brown, moderately to deeply weathered SILTSTONE
% fines = 60.4			42/6"		SPT-2	
TX-UU =1619 (1195) SG = 2.62	21.6	95.9		10	P-3	tan, moderately weathered and fractured SANDSTONE, very fine grained
				15		
				20		
TX-UU = 853 (2405) SG = 2.62	23.1	100.3			P-4	
				25		Bottom of Boring at 23.5'
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 27, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NUMBER : 25692

EBMUD

BORING NO MN-4

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		ASPHALT CONCRETE
						BASE ROCK
						dark brown CLAY
LL = 52, PI = 14	27		37		SPT-1	
						brown, deeply weathered and fractured CLAYSTONE with sand
SG = 2.55			30/4"	5	MC-2	brown to orange, deeply weathered and fractured SANDSTONE, with calcite seams, fine grained,
% fines = 45.7			40/4"	10	SPT-3	
TX-UU = 4449 (1800) SG = 2.57	22.1	100.8		15	P-4	very dark grey, slightly weathered SILTSTONE
				20	P-5	
				25		harder drilling at 29.5'
				30		

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LOG OF BORING

PROJECT :WC WTP Improvements
DATE : October 27, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR : Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NUMBER : 25692

EBMUD

BORING NO MN-4

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				30	P-6	very dark grey, slightly weathered SILTSTONE with organic and calcite seams
						Bottom of Boring at 32.5'
				35		
				40		
				45		
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : November 4, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-8

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		ASPHALT CONCRETE
						BASE ROCK
			50/4"		SPT-1	tan, friable, silty SANDSTONE, fine grained
			50/4"	5	SPT-2	tan, friable, deeply weathered SANDSTONE with a lot of oxidized seams
TX-UU = 9945 (1195) SG = 2.71	19.8	104.0		10	P-3	
			50/6"		SPT-4	
				15		Bottom of Boring at 13.0'
				20		
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 31, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-9

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		4" ASPHALT CONCRETE 6" BASE ROCK
LL = 38, PI = 18	20		25		SPT-1	brown, sandy CLAY (fill) (CL) bluish grey, gravelly
LL = 43, PI = 21	25		11	5	SPT-2	greyish brown, stiff, sandy SILT (ML), with clay
% fines = 65.6			15		SPT-3	
				10		with gravels
% fines = 44.2	22		63		SPT-4	mottled brown , deeply weathered, SANDSTONE, with organics, fine grained,
				15		
TX-UU = 382 (2203) SG = 2.57	25.6	96.4			P-5	brown, deeply weathered and fractured SILTSTONE cuttings with clay
				20		
			50/4"		SPT-6	grey, moderately weathered, CLAYSTONE, with calcite seams
				25		Bottom of Boring at 24.0'
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 31, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-10

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		3" ASPHALT CONCRETE
						BASE ROCK
			51		SPT-1	grey, very dense, silty SAND (SM) with cemented gravels, (fill)
	17.2		32	5	SPT-2	
	18.3		54	10	SPT-3	grey, friable, deeply weathered and friable, clayey SANDSTONE, with bluish black clay seams
SG = 2.61	16.0			15	P-4	harder drilling at 14.5'
						brown
			50/5"		SPT-5	with oxidized seams
				20		Bottom of Boring at 18.0'
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : November 6, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

TEST RESULTS

MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.
			0	
			5	
			10	
			15	
			20	
			25	
			30	

BORING NO MN-11

SURFACE ELEV. :

DESCRIPTION

ASPHALT CONCRETE
BASE ROCK
tan, very stiff, clayey SAND, gravelly

brown, deeply weathered SANDSTONE, with clay seams

grey, fine to medium grained

Bottom of Boring at 9.0'

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 27, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-13

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-UU = 3581 (605) SG = 2.63	17.8	109.3	50/5" 50/5"	0 5 10 15 20 25 30	SPT-1 P-2 P-3 SPT-4 P-4	tan-grey, moderately weathered and fractured SANDSTONE, fine grained, Bottom of Boring at 22.5'

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 28, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-14

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown, sandy CLAY (CL) (fill)
LL =52, PI = 33	15.0		41		SPT-1	dark brown, hard, sandy CLAY, (CH)
			50/4"	5	SPT-2	tan, deeply weathered SANDSTONE, fine grained,
SG = 2.63	19.8	112.7		10	P-3	slightly to moderately weathered
SG = 2.55	18.9	103.4		15	P-4	soft zone between 17.5' and 18.0'
				20		Bottom of Boring at 20.0'
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 23, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-15

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		4" ASPHALT CONCRETE
						4" BASE ROCK
						brown, sandy CLAY (CL) (fill)
	17		15		SPT-1	dark brown, stiff, sandy CLAY (CH)
						grey gravel (GP)
				5		brown, clayey SAND (SC) (fill)
LL = 43, PI = 26	23.6	104.3	25		MC-2	gravelly
						dark brown, stiff CLAY (CH)
LL = 58, PI = 29	23		16	10	SPT-3	
				15		
SG = 2.5	28.7	93.7	16		MC-4	dark greyish brown, with calcite
				20		
	19.3		8		SPT-5	brown, very stiff
						orange brown, deeply weathered silty CLAYSTONE
				25		
LL = 68, PI = 43	28		31		MC-6	
				30		

LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 23, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-15

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
	26.1		43	30	SPT-7	orange brown, deeply weathered and fractured, silty CLAYSTONE harder drilling between 31' and 34' dark green cuttings at 34'
TX-UU = 3316 (4205) SG = 2.6	25.1	99.7	66	35	MC-8	dark grey SILTSTONE / SHALE, vertical fractures started to use rock bit at 35'
TX-CU = 4617 (5400) SG = 2.63	13.5	116.8		45	P-9	Bottom of Boring at 47.5'
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 23, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-16

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		dark brown, sandy CLAY
LL = 62, PI = 34 SG = 2.71	28.8	92.1	29		MC-1	orange brown , deeply weathered and fractured CLAYSTONE
LL = 65, PI = 36	23		35	5	SPT-2	harder drilling at 7'
LL =66, PI = 36	29		29	10	SPT-3	greyish brown to orange brown
TX-CU = 7044 (1800)	19.9	103.4	16	15	MC-4	
	19.3		50/6"	20	SPT-5	mottled brown and orange brown, moderately weathered changed to rock bit at 20'
TX-UU = 3409 (2995)	18.9	107.3		25	P-6	very dark grey SILTSTONE / SHALE
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 22, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-16

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
			30/3"	30	SPT-7	very dark grey SILTSTONE/SHALE
			35	35		harder drilling at 38'
TX-CU = 4970 (2995) = 5900 (4003) = 7063 (4997) SG = 2.64	11.9	116.5		40	P-8	
				45		Bottom of Boring at 42.5'
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 23, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-17

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown, stiff, gravelly sandy CLAY (CL)
LL = 51, PI = 32	14		20		STP-1	
				5		orange brown, deeply weathered , silty CLAYSTONE
TX-CU = 857 (504) = 992 (994)	22.9	84.2	33		MC-2	
LL = 79, PI = 41	34		30	10	SPT-3	
TX-CU = 1853 (1800) SG = 2.53	32.7	87.2	42	15	MC-4	
	32.8		38	20	SPT-5	moderately weathered
TX-UU = 5564 (2995) SG = 2.5	30.4	88.8	70	25	MC-6	
				30		dark grey SILTSTONE/SHALE

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 23, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-17

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
	27.2		45/6"	30	SPT-7	very dark grey SILTSTONE/SHALE
				35		harder drilling at 35'
TX-CU = 13358 (4997) SG = 2.43	12.7	112.7		40	P-8	
				45		Bottom of Boring at 42.5'
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 21, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-18

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown, sandy, silty CLAY (CL) with few gravels (fill)
LL = 40, PI = 20			38		SPT-1	
						with sand
LL = 38, PI = 13	21		33	5	SPT-2	
						greenish grey
SG = 2.58	23.3	104.8	21	10	MC-3	
LL = 45, PI = 24	28		17	15	SPT-4	mottled grey
						stiffer drilling at 19.0'
TX-CU 2007 (2405) SG = 2.47	29.3	88.9	23	20	MC-5	greyish brown, deeply weathered and fractured CLAYSTONE
LL = 64, PI = 30	34		28	25	SPT-6	mottled dark tan and orange
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 21, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-18

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-UU = 1528 (3600)	33.1	86.9	61/16"	30	MC-7	dark grey and mottled orange, moderately weathered and fractured, silty CLAYSTONE
						light grey
	48.4		32	35	SPT-8	
						dark grey, slightly to moderately weathered SHALE
			50/4"	40	MC-9	
						changed to rock bit at 40'
	24.4		63	45	SPT-10	slightly clayey, fractured
TX-UU = 901 (6005) SG = 2.62	20.4	109.6		50	P-11	
						harder drilling at 53'
	18.9		50/5"	55	SPT-12	
						softer drilling and with brown rock fragments at 59'
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 21, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-18

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				60	P-13	harder drilling at 60.5'
TX-CU = 2508 (4997) = 3070 (6005) = 3270 (6998) SG = 2.58	20.3	105.4	50/5"	70	MC-14	
				75		
TX-UU = 1708 (8006)	18.1	110.1		80	P-15	dark grey to black silty CLAYSTONE and SHALE
						Bottom of Boring at 82.0'
				90		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 22, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-19

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		dark brown, sandy CLAY (CH) (fill)
LL = 85, PI = 54	24		9		SPT-1	
						orange brown, silty CLAY (CH) (fill)
TX-CU = 4384 (605) SG = 2.58	21.7	90.4	33	5	MC-2	mottled tan orange brown, deeply weathered SILTSTONE with calcite on fracture surfaces
LL = 68, PI = 36	34		28	10	SPT-3	greenish tan, deeply weathered CLAYTONE, with organic clay seams
TX-CU = 2215 (1800) = 4217 (2405) = 5347 (3600)	25.9	90.3	72	15	MC-4	greyish brown
	28.2		53	20	SPT-5	light brown
TX-UU = 6837 (2995) SG = 2.59	30.4	90.6	40/4"	25	MC-6	moderately weathered
				30		

U\data\borelogs\Walnut Creek\25692\MN-19.1.CDR

LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 22, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-19

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
	32		48	30	SPT-7	brown silty CLAYSTONE, with fractures at 45 degrees, deeply weathered
				35		
						dark grey, SHALE, with shell fragments
TX-CU = 5738 (4795) SG = question	28.7	93.0	50/5"	40	MC-8	Bottom of Boring at 40.7'
				45		
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : November 5, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-20

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		brown, gravelly, sandy CLAY (CL) (fill)
LL = 68, PI= 38	24		38		SPT-1	tan, deeply weathered SILTSTONE
LL = 47, PI = 17	17		43/6"	5	SPT-2	Bottom of Boring at 7'
				10		
				15		
				20		
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 24, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-21

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		BASE ROCK
LL = 78, PI = 52	21		25		SPT-1	brown, stiff, sandy CLAY (CH)
						orange brown, deeply weathered SILTSTONE
LL = 48, PI =27	19		13	5	SPT-2	
LL = 71, PI = 37 TX-UU = 2545 (1195) SG = 2.58	30.5	88.2	21	10	MC-3	orange brown, deeply weathered and fractured CLAYSTONE
						becomes harder drilling at 13.5'
LL = 64, PI = 29	34		24	15	SPT-4	rock fragments, calcite and carbonaceous materials
TX-UU 4235 (2390) SG = 2.66	22.3	100.1	58	20	MC-5	dark grey to tan orange, deeply weathered SILTSTONE / SHALE with oxidized and calcareous seams
	20.7		59/16"	25	SPT-6	
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 24, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-21

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-UU = 4328 (3499) SG = 2.71	26.5	98.1	33/6"	30	MC-7	brown, moderately weathered and fractured CLAYSTONE
						dark grey SILTSTONE chips in drilling fluid
				35		very hard drilling at 35'
						Bottom of Boring at 35'
				40		
				45		
				50		
				55		
				60		

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LOG OF BORING

PROJECT : WC WTP Improvements
LOCATION : Walnut Creek Filter Plant
LOGGED BY : Catherine Anderson
CHECKED BY : Derek Kawaii

DATE : October 30, 1997
CONTRACTOR: Pitcher Drilling
EQUIPMENT : Failing 750
JOB NO. : 25692

EBMUD

BORING NO MN-22

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
				0		tan, silty SAND (SM) (fill)
	10.5		35/6"		SPT-1	
	5.8		50/6"	5	STP-2	
TX-UU = 1085 (1195) SG =2.58	17.5	113.2		10	P-3	clay seam
				15		
TX-UU 9735 (2405) SG = 2.57				20	P-4	brown, moderately weathered and fractured SANDSTONE, fine grained
				25		
				30		

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LOG OF BORING

PROJECT : WC WTP Improvements
DATE : October 30, 1997
LOCATION : Walnut Creek Filter Plant
CONTRACTOR: Pitcher Drilling
LOGGED BY : Catherine Anderson
EQUIPMENT : Failing 750
CHECKED BY : Derek Kawaii
JOB NO. : 25692

EBMUD

BORING NO MN-22

SURFACE ELEV. :

TEST RESULTS	MOISTURE CONTENT %	DRY DENSITY (PCF)	BLOWS/FT*	DEPTH (FT)	SAMPLE NO.	DESCRIPTION
TX-UU = 2282* (3600) SG = 2.63 * sample stopped deforming at 0.7% strain	14.8	114.5		30	P-5	slightly to moderately weathered SANDSTONE, fine-grained
						Bottom of Boring at 32.0'
				35		
				40		
				45		
				50		
				55		
				60		

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APPENDIX B

LOGS OF BORINGS (Olivia Chen Consultants)

LOG OF BORING MN23	DRILLING DATE: 2-12-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: CT
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	87		LEAN CLAY with Sand (CL), light brownish gray, hard, dry to moist, gravel-sized sandstone fragments (fill).	83.5	18.0	41	19	72.7
5		2	39		SANDY LEAN CLAY (CL-ML), sandy.		20.9	40	15	69.4
10		3	52		SANDY LEAN CLAY, dark gray with rust striations, hard, dry, sandy.		20.3	35	9	54.2
15		4	37		LEAN CLAY with Sand (CL), dark greenish gray, hard, moist, medium to high toughness, plastic, organic odor (colluvium).		25.0	39	18	82.7
20		5	17				22.2			
25		6			Olive brown with rust striations, moist, strongly cemented.	106.3	21.4	37	17	83.9
30		7	50/2"		Yellowish brown, hard, weakly cemented.					
35		8			SANDSTONE, dark gray, moist, silty, closely fractured, slightly to moderately weathered, weak to moderately strong.		17.1			
		9	50/3"		Brownish beige with rust striations, dry, weakly cemented.					76.4
					SILTSTONE, dark gray, moist, closely to moderately fractured, strong.					

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 2	FIGURE 1

LOG OF BORING MN23	DRILLING DATE: 2-12-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: CT
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

Olivia Chen Consultants, Inc.
Civil • Environmental • Geotechnical

DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		10			SILTSTONE, dark gray, moist, closely to moderately fractured, strong. *(Continued)*		18.9			
		11	50/3"		Bottom of boring at 43.25' below ground surface.					
45										
50										
55										
60										
65										
70										
75										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 2 of 2	FIGURE 1

LOG OF BORING MN 23A	DRILLING DATE: 1-4-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: SI
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	50/ 5.5"		LEAN CLAY with Sand (CL), light brownish gray, very dense, dry, (fill).		12.6	45	22	84.3
5		2	49		Small rock fragments present.					
10		3	50/3"		LEAN CLAY with Sand (CL), dark brown, hard, moist, medium toughness, none to low dilatency (fill).					
15		4	50/3"							
20		5	50/3"		SANDSTONE, greenish gray, moist, silty, fine-grained, moderately weathered, friable to moderately strong, weakly cemented.		18.8			
25		6	50/3"							
30		7	50/2"				17.8			
35		8	50/3"							

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 2	FIGURE 1a

LOG OF BORING MN 23A	DRILLING DATE: 1-4-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: SI
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		9	50/2.5		SANDSTONE, greenish gray with rust striations, moist, weakly cemented.		20.1			
					Bottom of boring at 40.25' below ground surface. Piezometer installed on 1/4/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					
45										
50										
55										
60										
65										
70										
75										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 2 of 2	FIGURE 1a

LOG OF BORING MN 24	DRILLING DATE: 1-4-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: SI
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
					4" ASPHALT.					
		1	50/6"		SILTY SAND / CLAYEY SAND (SC-SM), light brownish beige some rust striations, dry, sandstone fragments.		13.4	33	9	47.9
5		2	50/4"				17.2			
10		3	50/4"		SANDSTONE, beige, dry, silty, fine-grained, moderate cementation, slight to moderately weathered, moderately strong.		21.0			45.0
15		4	50/4.5"		SILTSTONE, brownish orange, dry to moist, closely fractured, rust striations (iron stains), slight to moderately weathered, moderately strong.					
20		5	50/6"				25.8			
25		6	50/4"							
30		7			Moderately fractured.	102.6	21.2			
35					Bottom of boring at 33' below ground surface. Piezometer installed on 1/4/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 2

LOG OF BORING MN 25	DRILLING DATE: 1-3-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	46		LEAN CLAY with Sand (CL), grayish brown with white & orange mottles, hard, moist, (fill).		19.8	43	20	70.2
5		2	52		LEAN CLAY with Sand (CL), dark gray, hard, dry to moist, (fill).		20.2	40	18	74.1
10		3	25		CLAYSTONE, tannish beige, moist, deeply to moderately weathered, weak to moderately strong, moderate cementation.		28.6			
15		4	46		Tannish beige with orange striations.					
20		5	29				29.9			
25		6			Friable to plastic.	80.1	38.4			
30		7	68		CLAYSTONE, dark gray, moist, slightly to moderately weathered. moderately strong, moderate cementation.		20.9			
35		8	93		CLAYSTONE, silty, slightly weathered, moderately strong.					

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 2	FIGURE 3

LOG OF BORING MN 25	DRILLING DATE: 1-3-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		9	88		CLAYSTONE, tannish beige, moist, deeply to moderately weathered, weak to moderately strong, moderate cementation. *(Continued)*					
					CLAYSTONE, silty, slightly weathered, moderately strong.					
45		10				121.5	14.3			
					Bottom of boring at 48' below ground surface.					
50					Piezometer installed on 1/3/01; refer to completion diagram.					
					Groundwater encountered at 23 feet below ground surface during field visit on 1/22/01.					
55										
60										
65										
70										
75										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTW\PROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 2 of 2	FIGURE 3

LOG OF BORING MN 26	DRILLING DATE: 1-2-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
					4" ASPHALT.					
					6" CONCRETE.					
		1	27		CLAYSTONE, reddish brown with rust striations, dry to moist, silty, slightly to moderately fractured, weak to moderate cementation.		29.7			
5		2	36		Slightly to moderately weathered, weak to moderately strong. ▼		28.9			
10		3	52							
15		4				93.0	28.9			
20		5	80				28.6			
25		6	50/6"							
30		7	50/5.5"		Bottom of boring at 30.5' below ground surface. Piezometer installed on 1/2/01; refer to completion diagram. Groundwater encountered at 6 feet below ground surface during field visit on 1/22/01.					
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTSVFINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 4

LOG OF BORING MN 27	DRILLING DATE: 1-9-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	39		SANDY LEAN CLAY (CL), greenish brown with orange and gray mottles, dense, dry, (fill).		14.8	46	25	66.7
5		2	28		SANDY LEAN CLAY, medium dense, silty.		19.6	44	23	
10		3	9		FAT CLAY (CH), dark gray, stiff, moist, none to low dilatency, low to medium toughness, highly plastic (colluvium).		36.2	72	50	89.8
		4	17		FAT CLAY.	88.5	32.5	76	52	89.8
15		5	15			85.9	34.5			
20		6	62		CLAYSTONE, dark gray, moist, slightly to moderately weathered, rust striations, friable to weak.	91.1	30.7			
25		7			Weak to moderately strong.	97.9	26.7			
30		8	50/4.5		Bottom of boring at 30.4' below ground surface. Piezometer installed on 1/9/01; refer to completion diagram. Groundwater encountered at 21 feet below ground surface during field visit on 1/22/01.					
35										

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 5

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTW\PROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

LOG OF BORING MN 28A	DRILLING DATE: 1-3-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	26		LEAN CLAY with Sand (CL), brownish yellow, very stiff, dry, (fill).		14.0	38	17	71.6
5		2	45		LEAN CLAY with Sand (CL), brownish yellow with orange and white mottles, hard, dry, (fill).		17.6	46	21	77.2
10		3	19		NO RECOVERY, very stiff.					
15		4	30		CLAYSTONE, grayish brown with orange, gray, and white mottles, dry to moist, moderate cementation, moderately weathered, calcite veins, weak to moderately strong.		29.8			
20		5				87.3	35.4			
25		6	44		CLAYSTONE/SILTSTONE, grayish brown with orange, gray, and white mottles, rust striations, dry to moist, moderate cementation, moderately weathered, calcite seams, closely to intensely fractured, moderately strong.		24.3			
30		7	90		CLAYSTONE, silty.		24.5			
35					Bottom of boring at 31.5' below ground surface. Piezometer installed on 1/3/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 6

LOG OF BORING MN 29A	DRILLING DATE: 1-9-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
					2" ASPHALT.					
		1	22		CLAYSTONE, brownish yellow with rust striations, moist, silty, moderately weathered, moderate cementation, weak.		30.3			
5		2	38		SILTSTONE, brownish yellow with rust striations, dry to moist, clayey, moderately weathered, moderate cementation, weak.		30.7			
10		3	31		Closely fractured. ▼		29.3			
15		4	51				29.8			
20		5			CLAYSTONE, dark gray, moist, silty, closely fractured, moderately weathered, weak to moderately strong, weak cementation.	94.3	27.3			98.1
25		6	50/4"		Platy.		25.7			
30		7	50/3.5"		SILTSTONE, clayey, platy, slightly to moderately weathered, moderately strong.		25.1			
					Bottom of boring at 30.3' below ground surface. Piezometer installed on 1/9/01; refer to completion diagram. Groundwater encountered at 11.5 feet below ground surface during field visit on 1/22/01.					
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 7

LOG OF BORING MN 30	DRILLING DATE: 1-2-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	65		SANDY LEAN CLAY (CL), grayish brown with white mottles, dry to moist, sandstone fragments (fill).		12.8	47	26	67.5
5		2	39		LEAN CLAY with Sand (CL).		8.5	32	13	76.4
10		3	24		CLAYSTONE, brownish beige with rust striations, moist, moderately weathered, friable to weak, moderate cementation.		31.4			
15		4	26				32.0			
20		5	38		Closely fractured, moderately strong.		30.8			
25		6	52		Calcareous clay seams.		31.7			
30		7	57		SILTSTONE, clayey, slightly to moderately weathered, closely fractured, moderately strong.		29.9			
					Bottom of boring at 31.5' below ground surface. Piezometer installed on 1/2/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 8

LOG OF BORING MN 31	DRILLING DATE: 1-8-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: XS
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	50/3"		SANDY LEAN CLAY (CL), grayish brown, hard, moist.		21.6			56.2
5		2	50/4"		SANDSTONE, grayish brown with rust striations, moderately weathered, weak to moderately strong.		19.6			
10		3	50/3"		More gray in color.					
15		4	50/2"		Slightly weathered.		14.9			
20		5	50/2"		Moderately strong to strong.					
25		6			Closely fractured.	113.4	17.4			
30		7	50/4"							
35		8	50/3"		Rust color striations.		18.6			

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTW\PROJECTS\FINAL\1132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 2	FIGURE 9

Bottom of boring at 41.33' below ground surface.
Piezometer installed on 1/8/01; refer to completion diagram.
Groundwater not encountered during field visit on 1/22/01.

LOG OF BORING MN 31	DRILLING DATE: 1-8-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: XS
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		9	50/4"							
45										
50										
55										
60										
65										
70										
75										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 2 of 2	FIGURE 9

LOG OF BORING MN 32	DRILLING DATE: 1-5-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	50/1"		NO RECOVERY.					
5		2	50/4.5"		CLAYEY SAND (SC), dark brown, very dense, moist.		36.4			34.0
10		3	50/3.5"		SILTSTONE, fine grained, grayish yellow with rust striations, dry, silty, moderate to strong cementation, slightly weathered, moderately strong.		19.8			
15		4	50/4"		Closely fractured.		17.0			56.6
20		5			Strong.	108.5	18.0			
25					Bottom of boring at 23' below ground surface. Piezometer installed on 1/5/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					
30										
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 10

LOG OF BORING MN 33	DRILLING DATE: 1-5-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: KG
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	50/6"		SILTSTONE, fine grained, brownish yellow beige, dry to moist, moderate cementation, closely fractured, slightly weathered, moderately strong.		13.0			65.6
5		2	50/6"		Dark gray with rust striations, dry to moist.					
10		3	50/4.5"				22.0			52.5
15		4	50/3.5"							
20		5			Strong.	110.5	17.6			
25					Bottom of boring at 23' below ground surface. Piezometer installed on 1/5/01; refer to completion diagram. Groundwater not encountered during field visit on 1/22/01.					
30										
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\132001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 11

LOG OF BORING MN 34	DRILLING DATE: 1-5-01	SURFACE ELEV: N/A
	DRILLING METHOD: Mud Rotary	DATUM: Ground Surface
	DRILL RIG TYPE: Failing 1500	LOGGED BY: XS
	HAMMER TYPE: 140 LBS. SAFETY HAMMER	CHECKED BY: CBK

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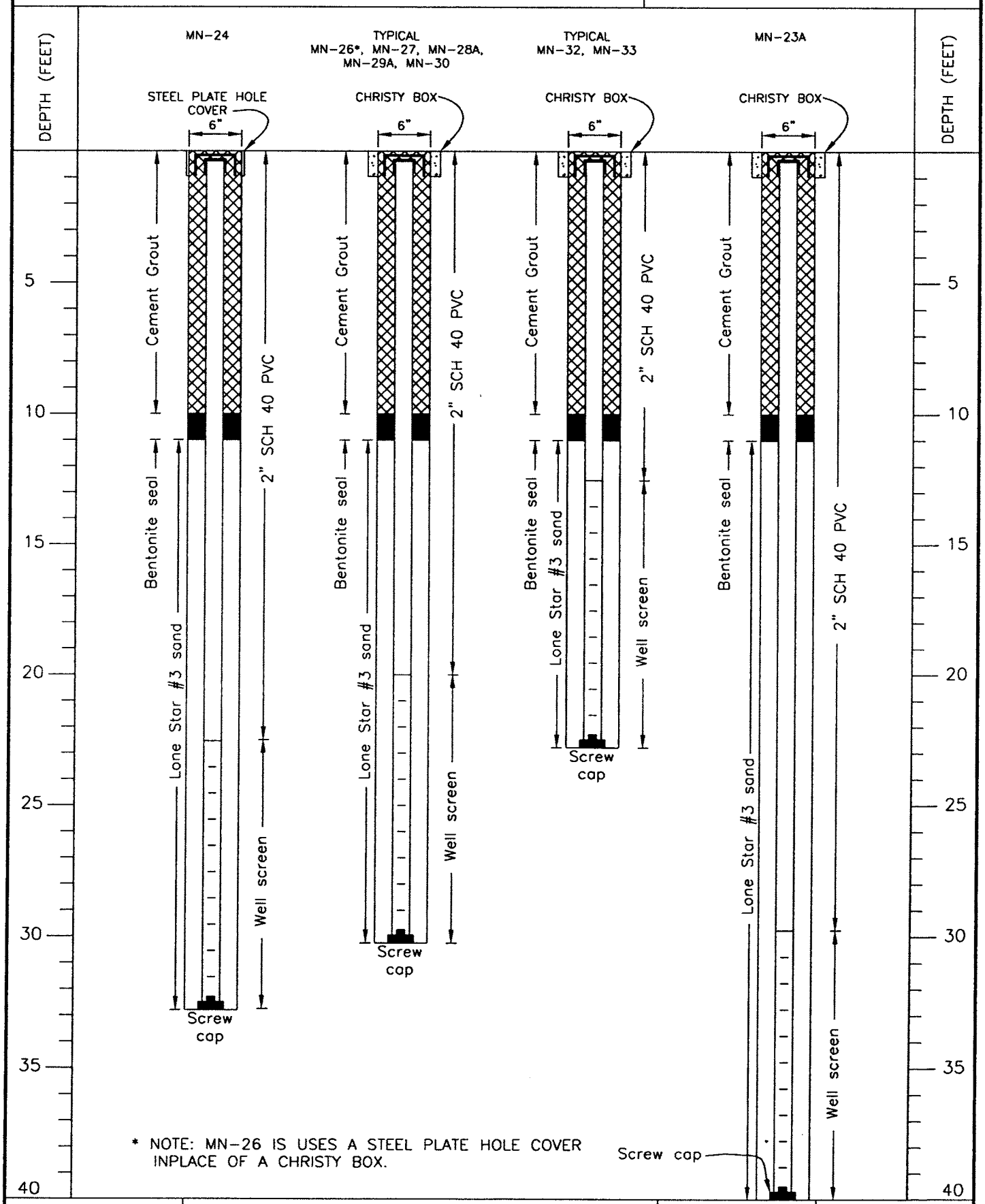
DEPTH (FEET)	SAMPLE TYPE	SAMPLE NO.	BLOW COUNT	GRAPHIC LOG	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
		1	76		SILTSTONE, fine grained, brownish yellow with rust striations, dry to moist, weak to moderate cementation, slightly to moderately weathered, moderate strong to strong.		16.2			60.2
5		2	50/2"				17.0			
10		3	50/5"				15.0			
		4	50/3"				11.3			
15					Bottom of boring at 14.25' below ground surface. Piezometer installed on 2/12/01; refer to completion diagram. Groundwater not encountered during drilling.					
20										
25										
30										
35										

LOG OF BORING L:\EBMUD\1320WA~1\BORINGS\GINTWPROJECTS\FINAL\1320001A.GPJ OLV_CHEN.GDT 4/10/01

JOB NO. 1320.01	PROJECT: Walnut Creek Water Treatment Plant	SHEET 1 of 1	FIGURE 12

PIEZOMETER COMPLETION DIAGRAM

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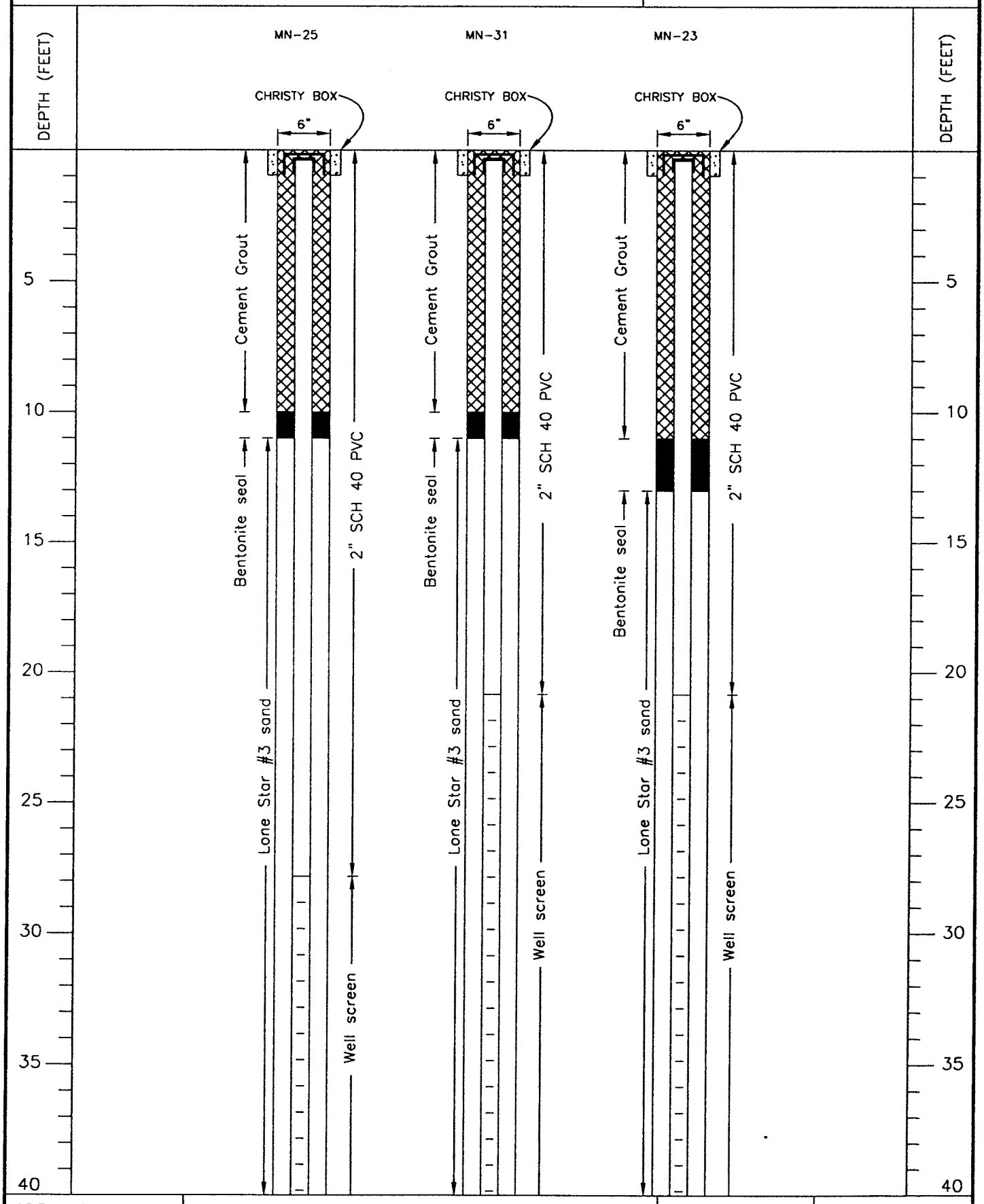


JOB NO. 1320	PROJECT: WALNUT CREEK WTP	SHEET 1 OF 1	FIGURE 13

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PIEZOMETER COMPLETION DIAGRAM

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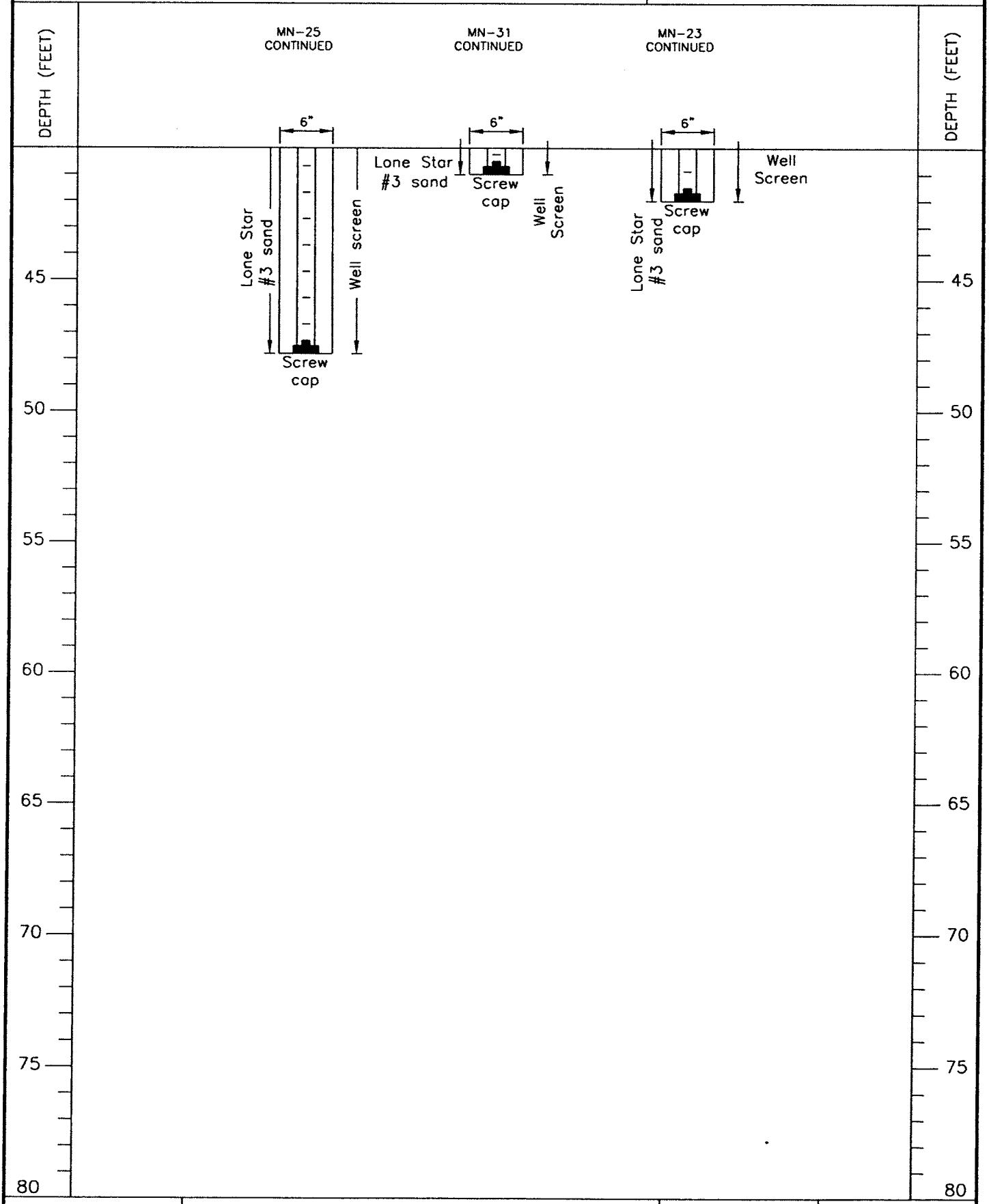


JOB NO. 1320	PROJECT: WALNUT CREEK WTP	SHEET 1 OF 2	FIGURE 14

L:\EBMUD\1320 WALNUT CREEK WTP\BORINGS\WELL DIAGRAM.DWG

PIEZOMETER COMPLETION DIAGRAM

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JOB NO. 1320	PROJECT: WALNUT CREEK WTP	SHEET 2 OF 2	FIGURE 14

L:\EBMUD\1320 WALNUT CREEK WTP\BORINGS\WELL DIAGRAM.DWG

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

NOTE: $Cu = \frac{D_{60}}{D_{10}}$; $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

BLOW COUNT

THE NUMBER OF BLOWS OF THE SAMPLING HAMMER REQUIRED TO DRIVE THE SAMPLER THE LAST 12 IN. OF AN 18-IN. DRIVE. THE NOTATION 100/9 INDICATES 9 IN. OF PENETRATION ACHIEVED IN 100 BLOWS. THE NUMBER "50+" INDICATES THAT >50 BLOWS WERE REQUIRED TO DRIVE THE SAMPLER THE LAST 12 INCHES.

CEMENTATION

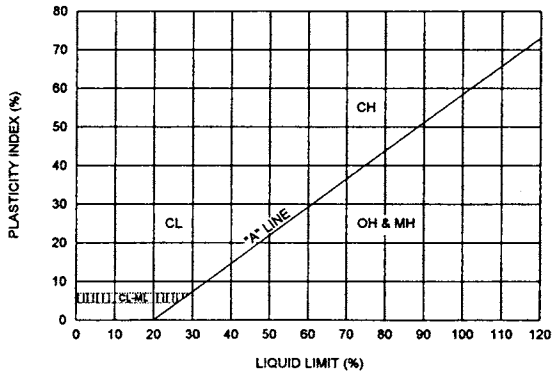
WEAK - CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE

MODERATE - CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE

STRONG - WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

SOURCE: U.S. DEPARTMENT OF INTERIOR, BUREAU OF RECLAMATION; PUB. YEAR, NOT AVAILABLE; ENGINEERING GEOLOGY FIELD MANUAL, P. 37.

PLASTICITY CHART



SAMPLE TYPES

- STANDARD PENETRATION
- ROCK CORE
- PITCHER SAMPLE
- MODIFIED CALIFORNIA
- PUSHED SHELBY TUBE

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	(200)	-	(WITH % PASSING NO. 200 SIEVE
CD	-	CONSOLIDATED DRAINED TRIAXIAL			
CN	-	CONSOLIDATION	SW	-	SWELL TEST
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TC	-	CYCLIC TRIAXIAL
DS	-	DIRECT SHEAR	TV	-	TORVANE SHEAR
PP	-	POCKET PENETROMETER (TSF)	UC	-	UNCONFINED COMPRESSION
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
RV	-	R-VALUE			
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
▽	-	WATER LEVEL (WITH DATE OF) MEASUREMENT	WA	-	WASH ANALYSIS
			(200%)	-	(WITH % PASSING NO. 200 SIEVE)

PENETRATION RESISTANCE
(RECORDED AS BLOWS / 0.5 FT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	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	FIRM	4 - 8	0.50 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

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Job No. 1320.01

LEGEND TO SOIL DESCRIPTIONS

FIGURE 15

LEGEND L:\EBMUD\1320WA~1\BORINGS\GINTW\PROJECTS\FINAL\1132001A.GPJ OLV_CHEN.GDT 4/10/01

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

Moisture Content
The moisture content of samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2216 by drying the samples in an oven at 110±5 degrees Celsius. The test results are presented on the logs of the exploratory borings in Appendix A.

In-Place Density
The dry density of relatively undisturbed samples obtained from the exploratory borings was evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Percent Organic Matter
The percentage of organic matter in selected soil samples obtained from the exploratory borings was evaluated in accordance with ASTM D 2974 by drying the samples in an oven at 440±40 degrees Celsius. The test results are presented on Figure C-3.

200 Wash Analysis
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The test results are presented on Figure C-4.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures C-5 through C-12. The test results were utilized in evaluating the soil classification in accordance with the Unified Soil Classification System (USCS).

Atterberg Limits
Tests were performed on selected soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure C-13.

Direct Shear Tests
Direct shear tests were performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. Test specimens were inundated during shearing to represent adverse field conditions. The results are shown on Figure C-14.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figures C-15 through C-23.

Expansion Index Tests
The expansion index of selected materials were evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1 inch thick by 4 inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The test results are presented on Figure C-24.

Unconfined Compression Tests
Unconfined compression tests were performed on relatively undisturbed samples in general accordance with ASTM D 2166. The test results are shown on Figure C-25.

Unconsolidated Undrained Triaxial Compression Tests
Triaxial compression tests were performed on selected relatively undisturbed samples in general accordance with ASTM D 2850. The test results are shown on Figures C-26 through C-28.

Rock Core Unconfined Compression
Unconfined compression tests were performed on intact rock core samples in general accordance with ASTM D 7012. The test results are shown on Figure C-29.

Consolidated Undrained Triaxial Compression Tests
Consolidated undrained triaxial compression tests were performed on selected relatively undisturbed samples in general accordance with ASTM D 4767. Cylindrical specimens with a length-to-diameter ratio of approximately 2:1 were saturated by back pressures until a minimum degree of saturation of 95 percent was achieved. Then the specimen was consolidated isotropically under a predetermined confining pressure. After completion of consolidation, the sample was sheared undrained under compression at a constant rate of axial deformation with pore pressure measurements. The test results are shown on Figures C-30 through C-33.

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
SB-1	5		Fill(SC)	30.8								%OM=1.7
	15		Soil(CH)	25.1								%OM=1.9
	20		Siltstone	24.7								
	21		Siltstone	42.1								
	28		Siltstone	25.7								
SB-2	7		Fill(CL)	27.4								%OM=2.2
	12		Fill(CL)	23.4								%OM=2.1
	16		Fill(GC)	21.2								%OM=2.4
	25		Fill(CL)	27.1								%OM=1.2
	38		Fill(CL)	26.8								%OM=2.2
NMB-1	5		Siltstone									
	6	46	Siltstone									
	9	71	Siltstone	25.8	77.2						Unsaturated UU: Su = 2.4 ksf (1.7 ksf)	
	14	135	Siltstone									
	15	153	Siltstone									
NMB-2	5		Sandstone									
	6	119	Sandstone									
	9	71	Sandstone									
NMB-3	5		Fill(SC)									
	6	30	Fill(CH)			59	36	0	39	61	Peak DS: c' = 0.33 ksf, ϕ ' = 46° Ultimate DS: c' = 0.20 ksf, ϕ ' = 36°	
	10	30	Fill(SM)									
	15	30	Fill(SC)	24.0	94.0			29	46	25	UC: Su = 1.0 ksf	
	20	34	Fill(SM)	26.5	91.5	43	14	27	49	24	Unsaturated UU: Su = 2.2 ksf (1.4 ksf) Su = 2.6 ksf (2.9 ksf) Su = 2.8 ksf (5.8 ksf)	
	25	44	Fill(SM)									
	29	227	Siltstone									
NMB-4	3	34	Fill(SC)	19.2	89.5			19	39	42	UC: Su = 0.6 ksf	
	6	10	Fill(SM)	26.8	91.3	43	16	14	45	42	Consolidation: CR = 0.075, RR = 0.005 σ,yield = 10.2 ksf	%Clay=12
	10	60	Fill(CL)	18.8	101.1							
	15	30	Fill(CL)	28.0	94.1						Saturated CU: Su = 3.5 ksf (3.0 ksf) Su = 4.9 ksf (5.8 ksf) Su = 7.1 ksf (11.6 ksf) c' = 0.65 ksf, ϕ ' = 29°	
	20	12	Fill(GC)	13.6	113.1	38	18	42	34	24	Consolidation: CR = 0.093, RR = 0.005 σ,yield = 6.2 ksf	
	25	48	Fill(GC)			38	19	36	36	29	Saturated CU: Su = 2.8 ksf (2.9 ksf) Su = 3.8 ksf (5.8 ksf) Su = 6.1 ksf (11.5 ksf) c' = 0.95 ksf, ϕ ' = 23°	
	30	50	Fill(CL)									
	35	48	Fill(CL)	26.6	93.9						Consolidation: CR = 0.120, RR = 0.003 σ,yield = 3.2 ksf	
	40	44	Sandstone									

FIGURE C-1

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

403982001 | 06/22

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
NMB-5	3	71	Fill(SC)	17.7	99.1			19	41	40		
	5		Fill(SC)									
	6	80	Fill(CL)	20.4	100.6	46	25	5	25	71	Consolidation: CR = 0.109, RR = 0.002 σ,yield = 9.7 ksf	%OM=1.9 %Clay=27
	9	135	Rock									
	14	128	Rock	18.2								
	19	255	Rock									
	24	143	Rock	36.4								
	29	171	Rock									
	35	450	Rock									
	39	300	Rock									
NMB-6	5		Fill(SC)					18	50	32		
	10	11	Fill(SC)	23.0	98.6	39	17	17	48	35	Consolidation: CR = 0.066, RR = 0.003 σ,yield = 10.4 ksf	
	15	30	Fill(SC)	26.9	94.6						Consolidation: CR = 0.088, RR = 0.007 σ,yield = 6.0 ksf	
	20	57	Siltstone	31.5	91.3						Unsaturated UU: Su = 3.7 ksf (1.4 ksf) Su = 3.2 ksf (2.9 ksf) Su = 4.1 ksf (5.8 ksf)	%OM=1.6
	25	52	Siltstone									
NMB-7	5		Fill(SC)									
	6	27	Fill(SC)	25.1	96.7	45	27	22	41	38	Consolidation: CR = 0.091, RR = 0.001 σ,yield = 10.0 ksf	
	10	45	Fill(SC)	23.5	96.3						Saturated CU: Su = 1.3 ksf (0.7 ksf) Su = 2.5 ksf (2.9 ksf) Su = 4.1 ksf (5.8 ksf) c' = 0.35 ksf, ϕ ' = 30°	
	15	32	Fill(CL)	30.7	82.7						Consolidation: CR = 0.117, RR = 0.012 σ,yield = 3.6 ksf	
	20	36	Soil(CH)	22.1	91.3	54	41	0	22	78	Consolidation: CR = 0.101, RR = 0.004 σ,yield = 7.4 ksf	%OM=1.6 %Clay=42
	25	57	Siltstone	29.8	88.6							
	30	55	Siltstone									
	35	67	Siltstone	22.9	96.8						Unsaturated UU: Su = 4.1 ksf (4.2 ksf)	
NMB-8	3	80	Fill(CL)									
	5	113	Sandstone									
	10	225	Sandstone									
NMB-9	6	23	Soil(CL)	21.2	95.7			0	45	55	UC: Su = 4.9 ksf	
	11	30	Soil(CL)					2	38	60		
	16	46	Siltstone	27.6	90.7	76	58	0	2	98	Saturated CU: Su = 1.3 ksf (0.6 ksf) Su = 2.8 ksf (2.7 ksf) Su = 4.5 ksf (5.6 ksf) c' = 0.45 ksf, ϕ ' = 32°	%OM=1.3 %Clay=50
	20	151	Siltstone									
	25	171	Siltstone									

FIGURE C-2

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	USCS SOIL TYPE	ORGANIC MATTER (percent by dry weight)	MOISTURE CONTENT w, (%)
SB-1	4.0-5.0	CL	1.7	30.8
SB-1	14.0-15.0	CH	1.9	25.1
SB-2	6.0-7.0	CL	2.2	27.4
SB-2	11.0-12.0	CL	2.1	23.4
SB-2	15.0-16.0	GC	2.4	21.2
SB-2	24.0-25.0	CL	1.2	27.1
SB-2	37.0-38.0	CL	2.2	26.8
B-4	19.5-20.0	CH	1.6	22.1
B-6	5.5-6.0	CL	1.9	20.4

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2974

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
NMB-3	14.5-15.0	Clayey SAND; little gravel	70.7	24.6	SC
NMB-3	19.5-20.0	Silty SAND; little gravel	73.5	24.2	SM
NMB-4	19.5-20.0	Clayey GRAVEL with sand	57.8	23.9	GC
NMB-4	24.5-25.0	Clayey GRAVEL with sand	64.0	28.5	GC
NMB-5	3.0-3.5	Clayey SAND with gravel	81.1	40.3	SC
NMB-6	9.5-10.0	Clayey SAND with gravel	83.5	35.4	SC
NMB-7	6.0-6.5	Clayey SAND; little gravel	78.3	37.8	SC
NMB-9	6.0-6.5	Sandy CLAY	99.7	54.9	CL

PERFORMED IN ACCORDANCE WITH 1140

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-4	6.0-6.5	43	27	16	0.003	0.040	0.552	191.8	1.0	42	SM

PERFORMED IN ACCORDANCE WITH ASTM D 422

FIGURE C-7

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

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GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-5	5.5-6.0	46	21	25	0.00	0.01	0.054	--	--	71	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422

FIGURE C-8

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-7	19.5-20.0	54	13	41	--	--	0.03	--	--	78	CH

PERFORMED IN ACCORDANCE WITH ASTM D 422

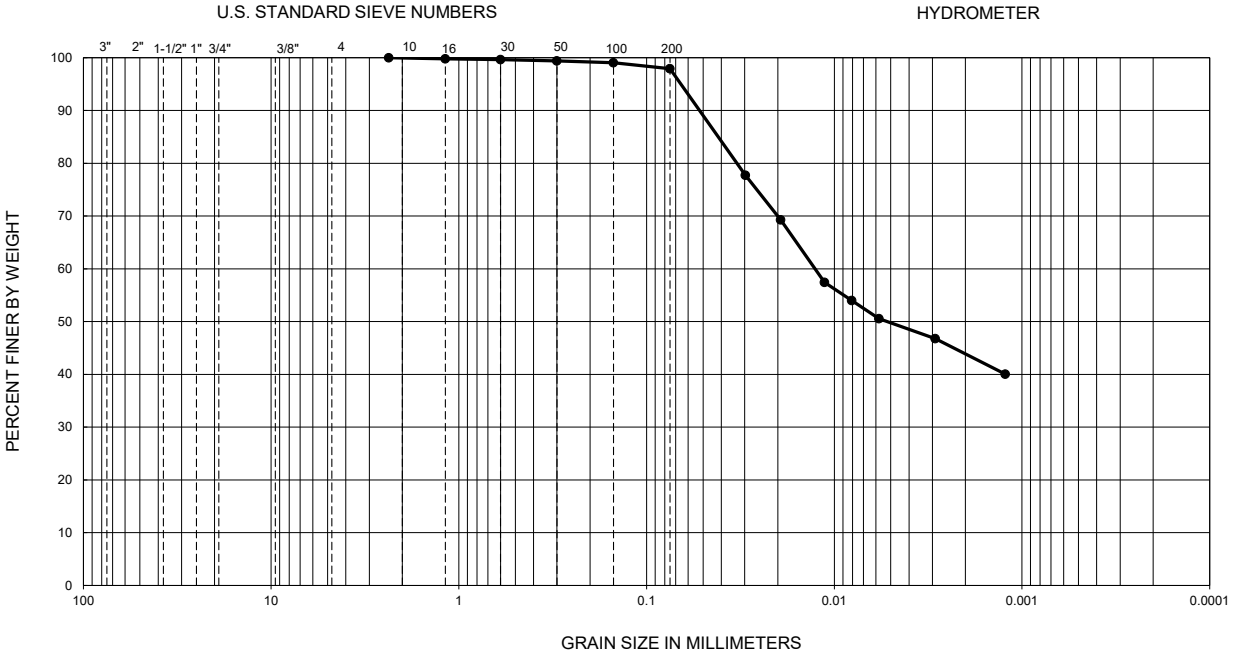
FIGURE C-10

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-9	15.0-15.5	76	18	58	--	--	0.013	--	--	98	CH

PERFORMED IN ACCORDANCE WITH ASTM D 422

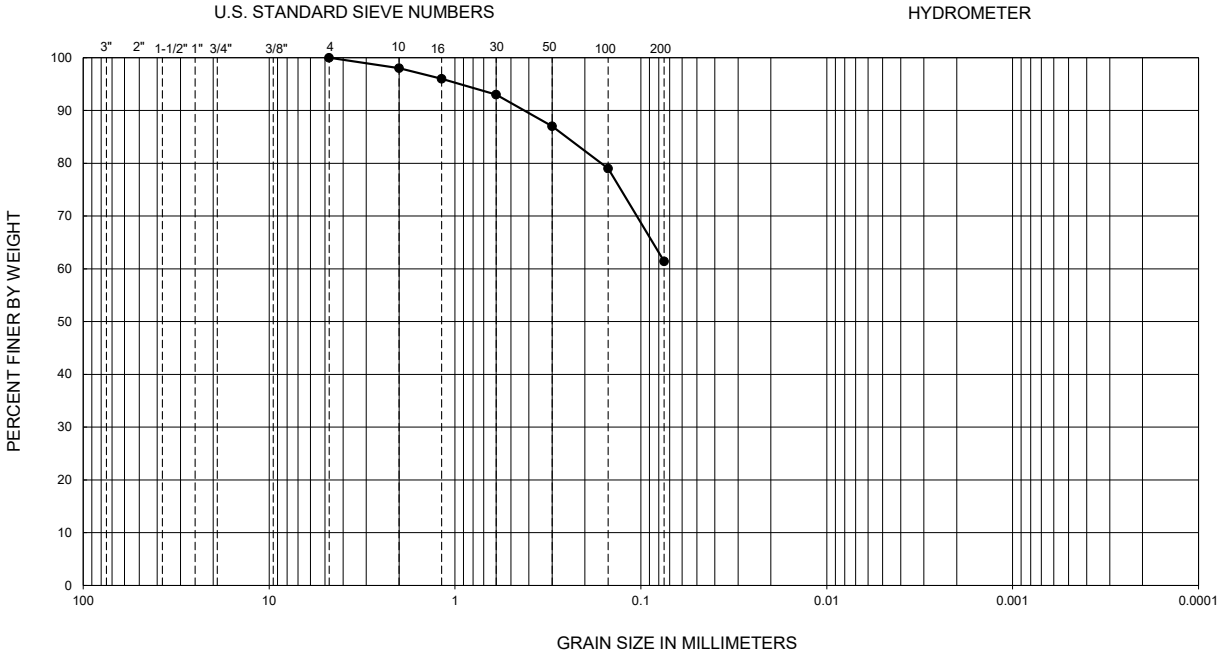
FIGURE C-12

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-3	6.0-6.5	59	23	36	--	--	0.07	--	--	61	CH

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

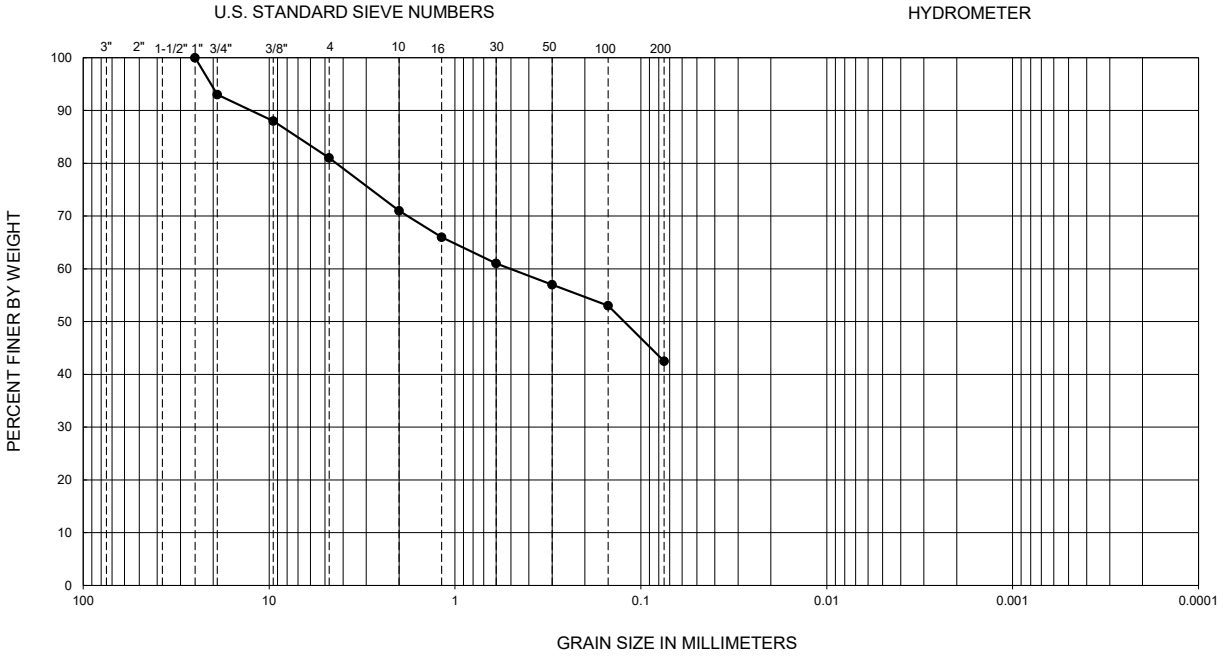
FIGURE C-5

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-4	3.0-3.5	--	--	--	--	--	0.53	--	--	42	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

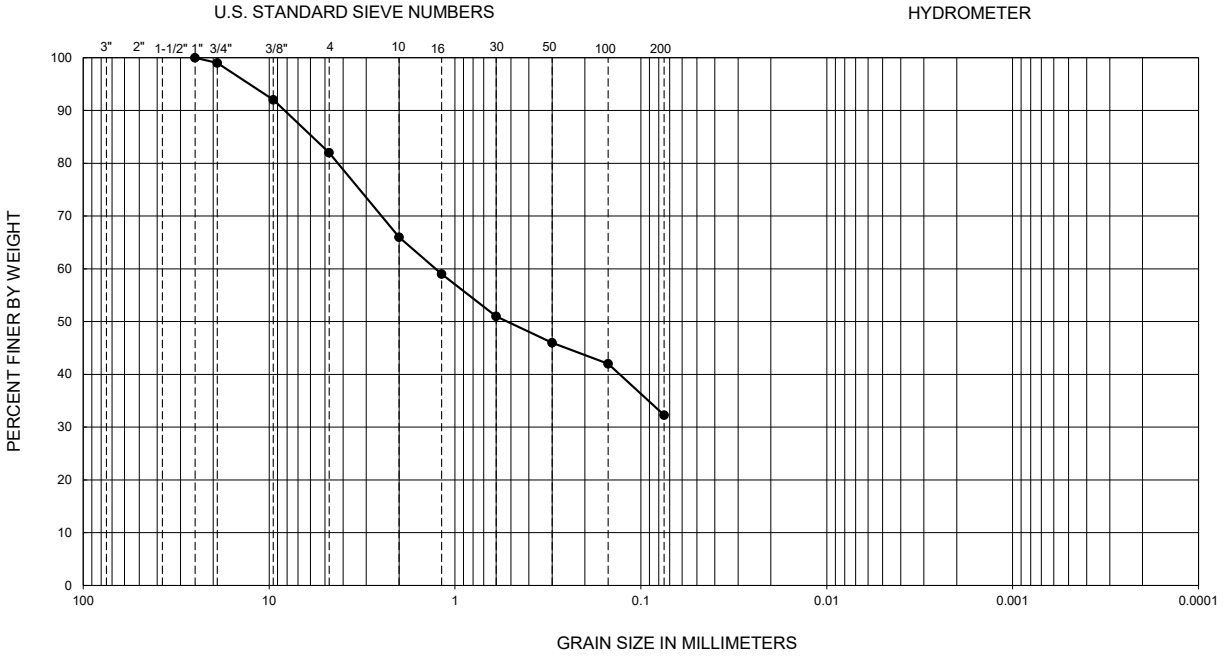
FIGURE C-6

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-6	0.0-5.5	--	--	--	--	--	1.30	--	--	32	SC

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

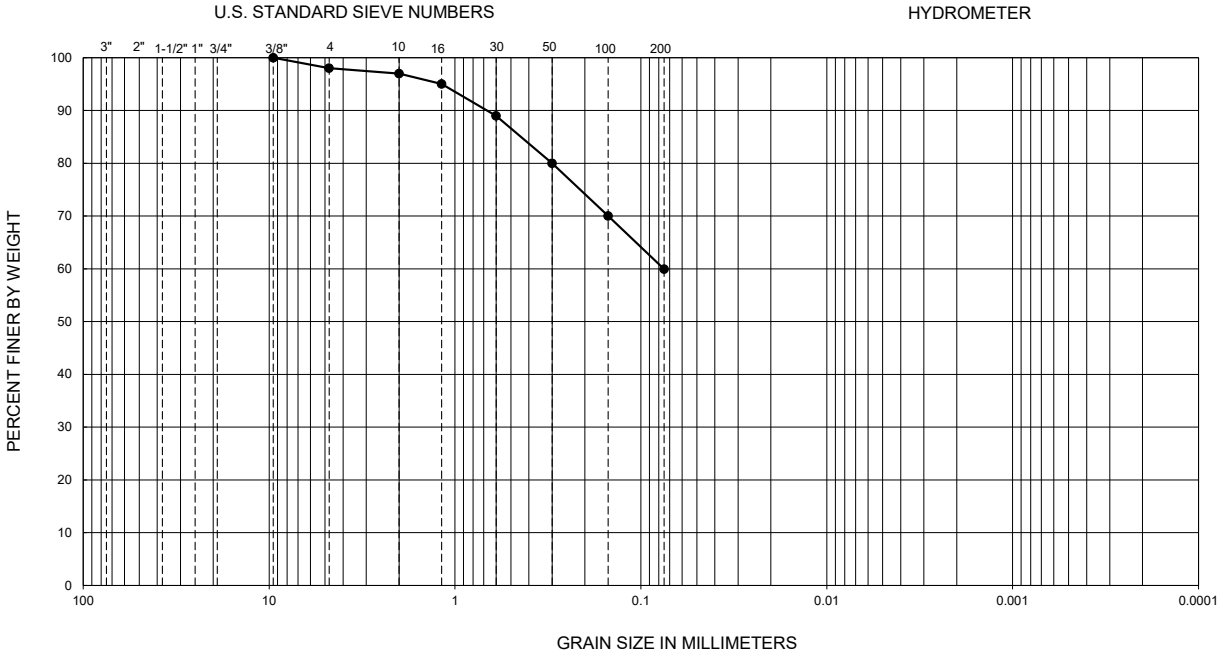
FIGURE C-9

GRADATION TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

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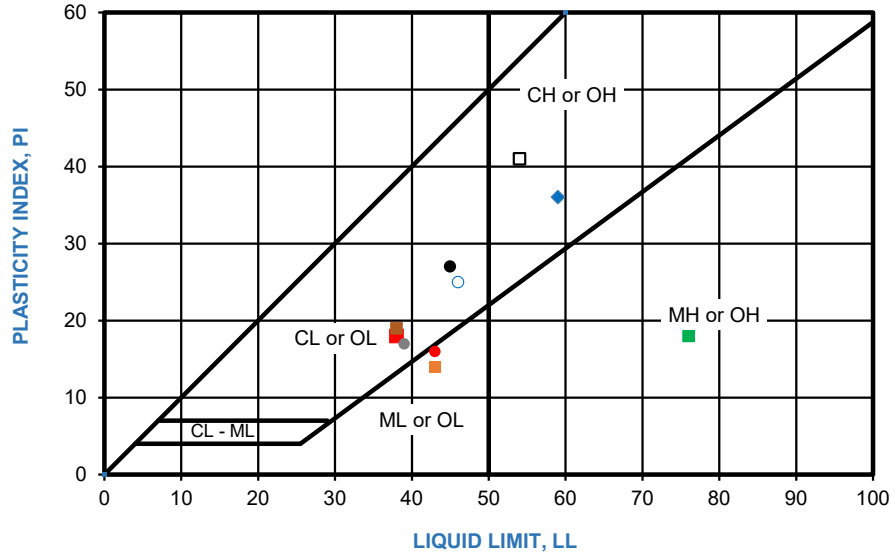
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	NMB-9	11.0-11.5	--	--	--	--	--	0.08	--	--	60	CL

PERFORMED IN ACCORDANCE WITH ASTM D 422 / D6913

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
◆	NMB-3	6.0-6.5	59	23	36	CH	CH
■	NMB-3	19.5-20.0	43	29	14	ML	SM
●	NMB-4	6.0-6.5	43	27	16	ML	SM
■	NMB-4	19.5-20.0	38	20	18	CL	GC
■	NMB-4	24.5-25.0	38	19	19	CL	GC
○	NMB-5	5.5-6.0	46	21	25	CL	CL
●	NMB-6	9.5-10.0	39	22	17	CL	SC
●	NMB-7	6.0-6.5	45	18	27	CL	SC
□	NMB-7	19.5-20.0	54	13	41	CH	CH
■	NMB-9	15.0-15.5	76	18	58	CH	CH



PERFORMED IN ACCORDANCE WITH ASTM D 4318

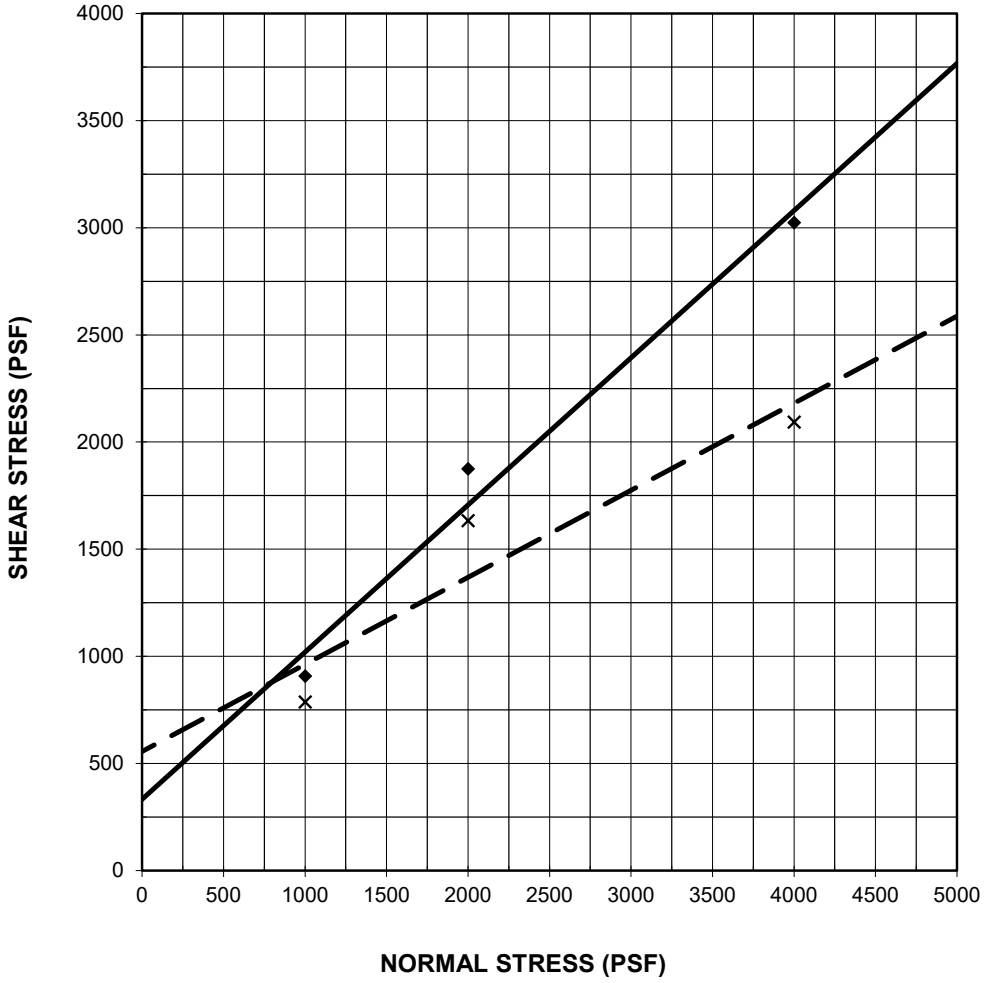
FIGURE C-13

ATTERBERG LIMITS TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Lab Master Excel_TBG2



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Fat clay	●—	NMB-3	6.0-6.5	Peak	325	46	CH
Fat clay	– –×– –	NMB-3	6.0-6.5	Ultimate	200	36	CH

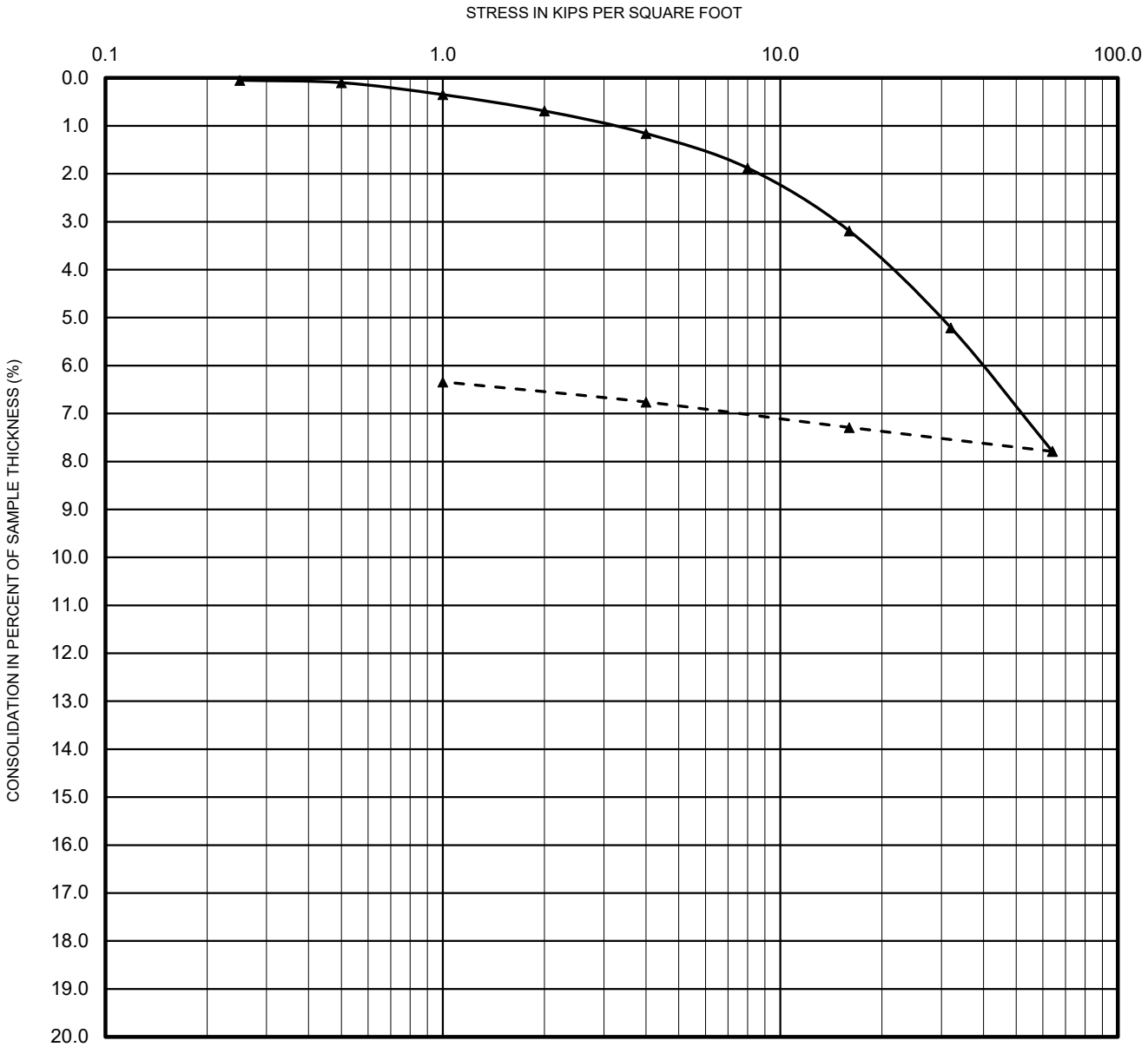
PERFORMED IN ACCORDANCE WITH ASTM D 3080

FIGURE C-14

DIRECT SHEAR TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA
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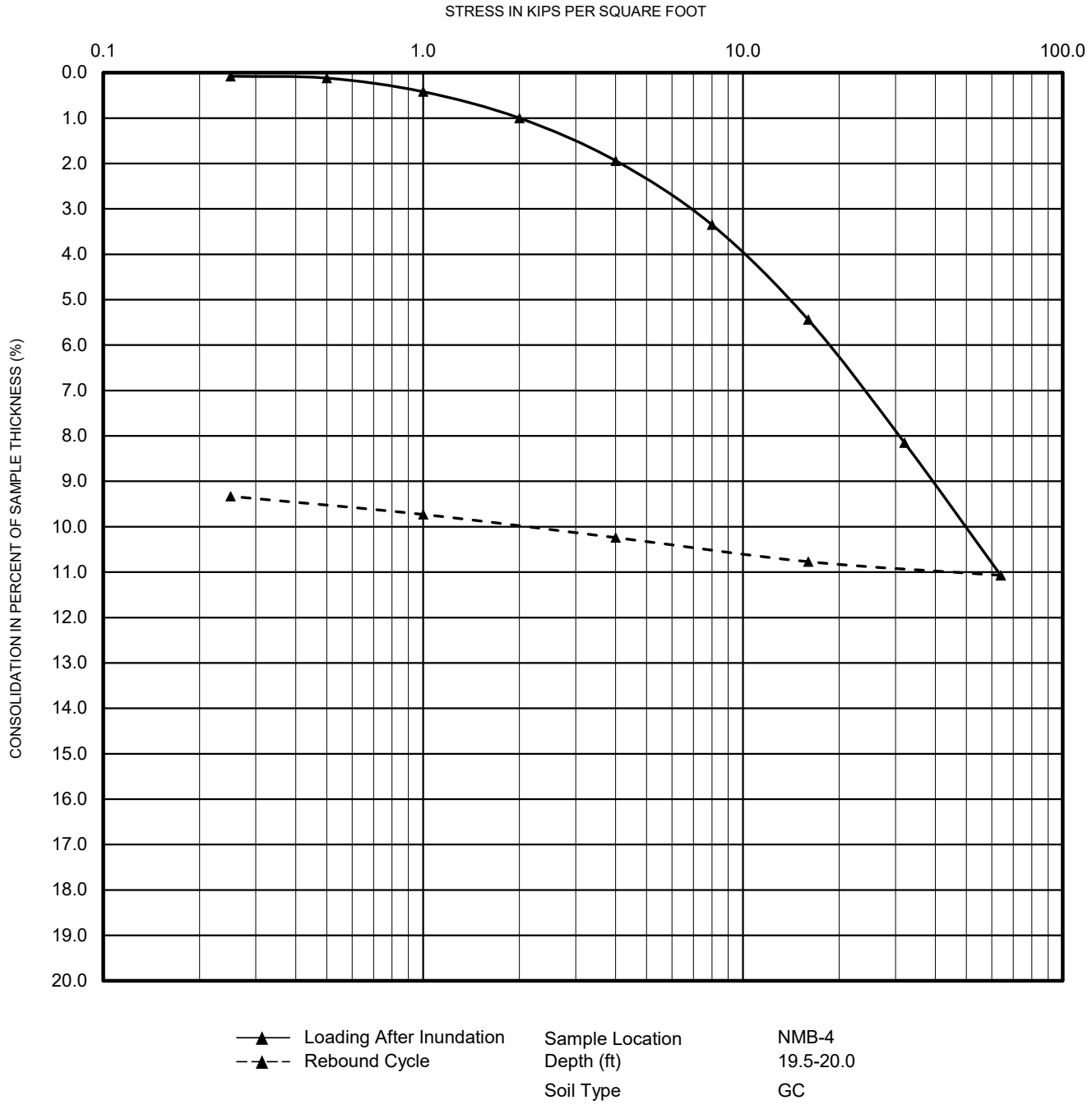
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-15

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



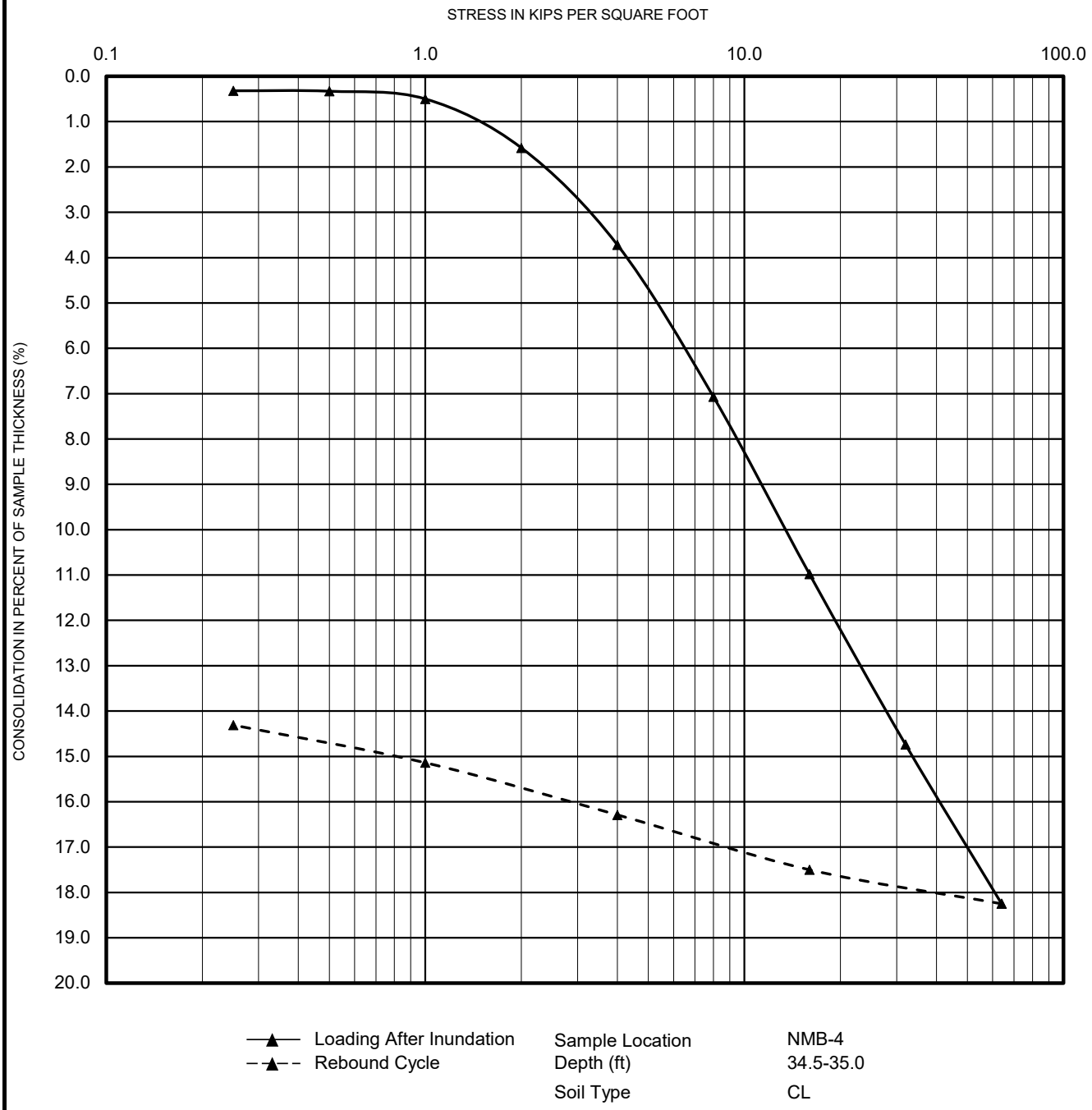
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-16

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



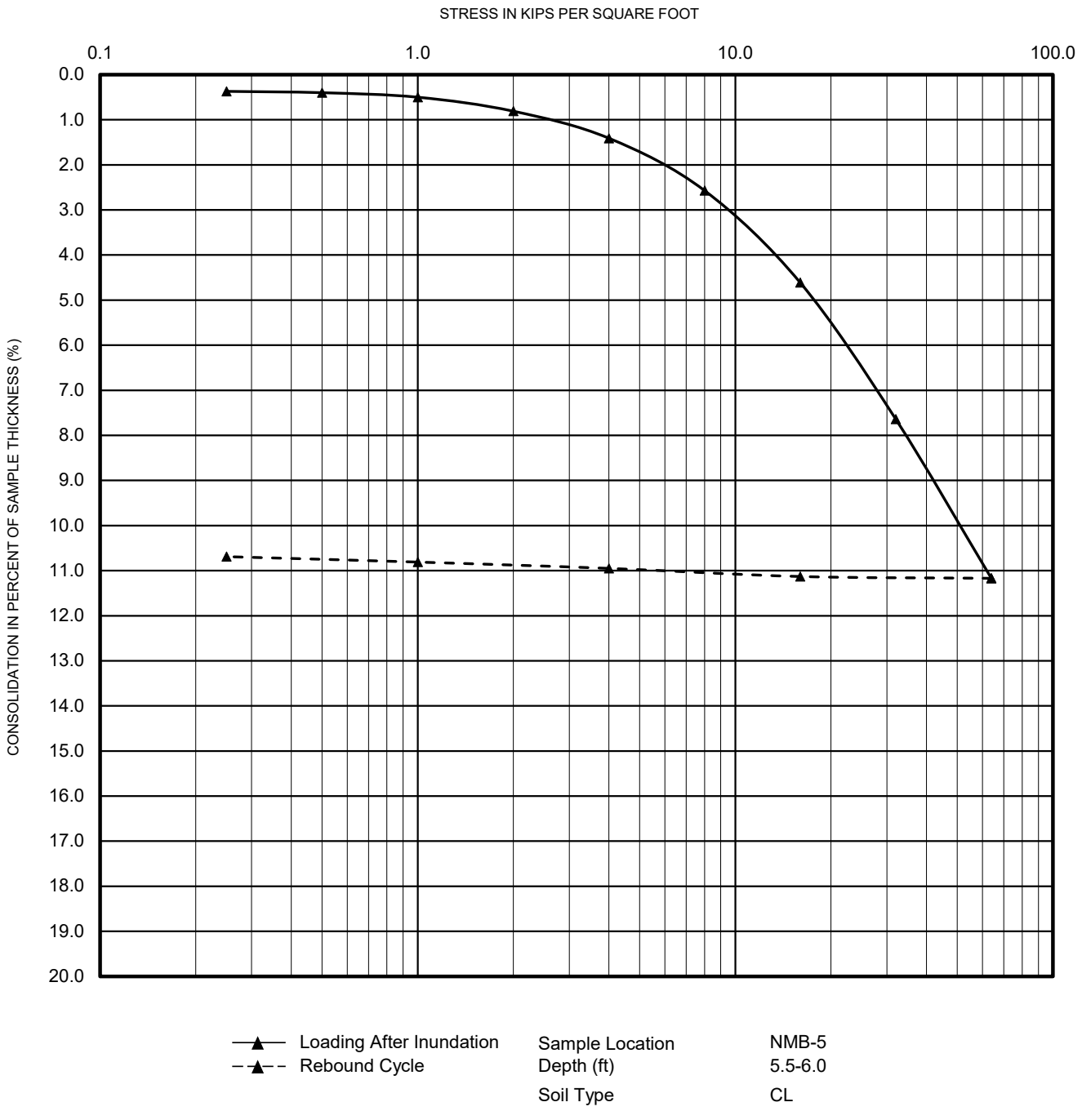
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-17

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



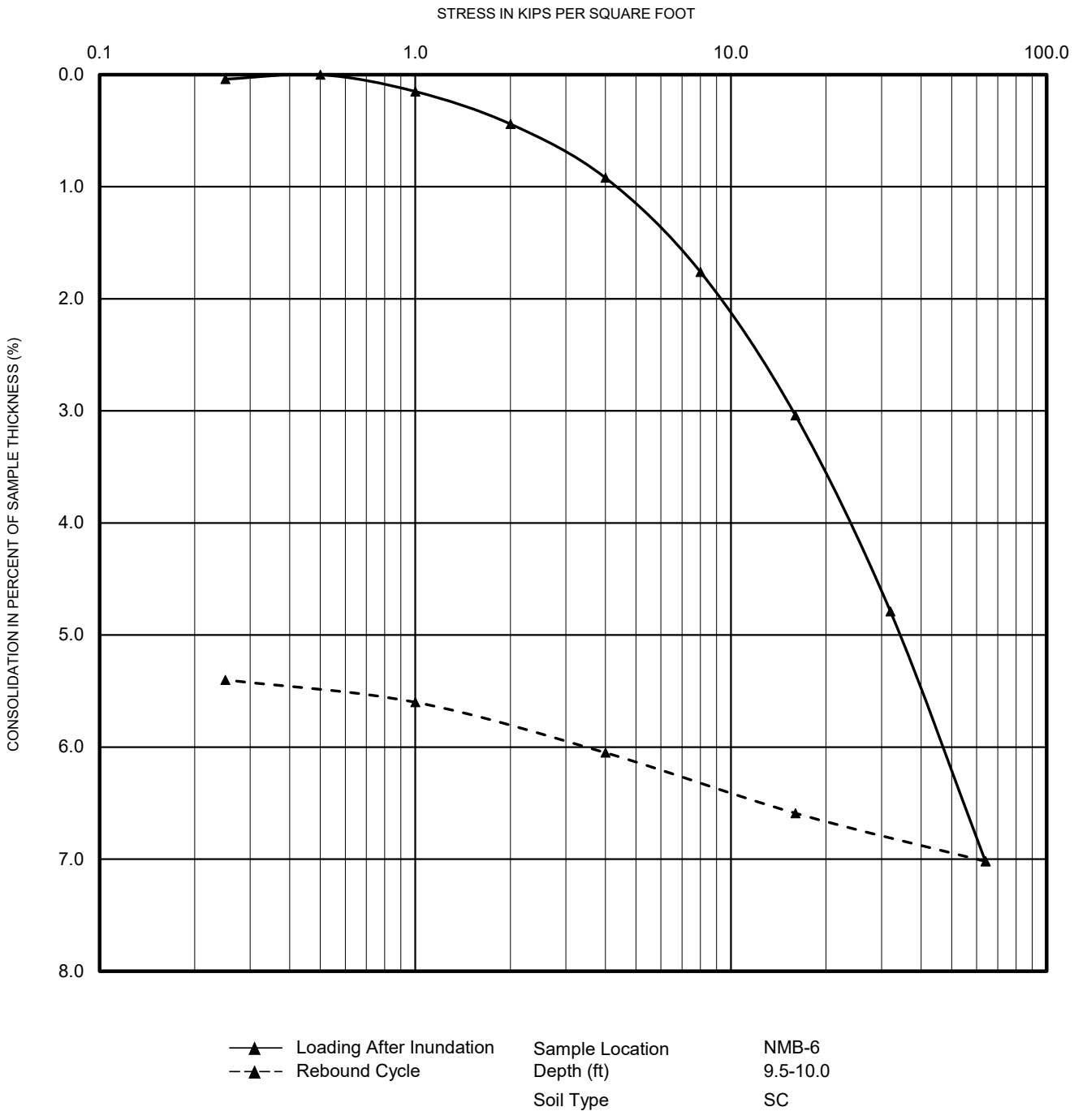
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-18

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



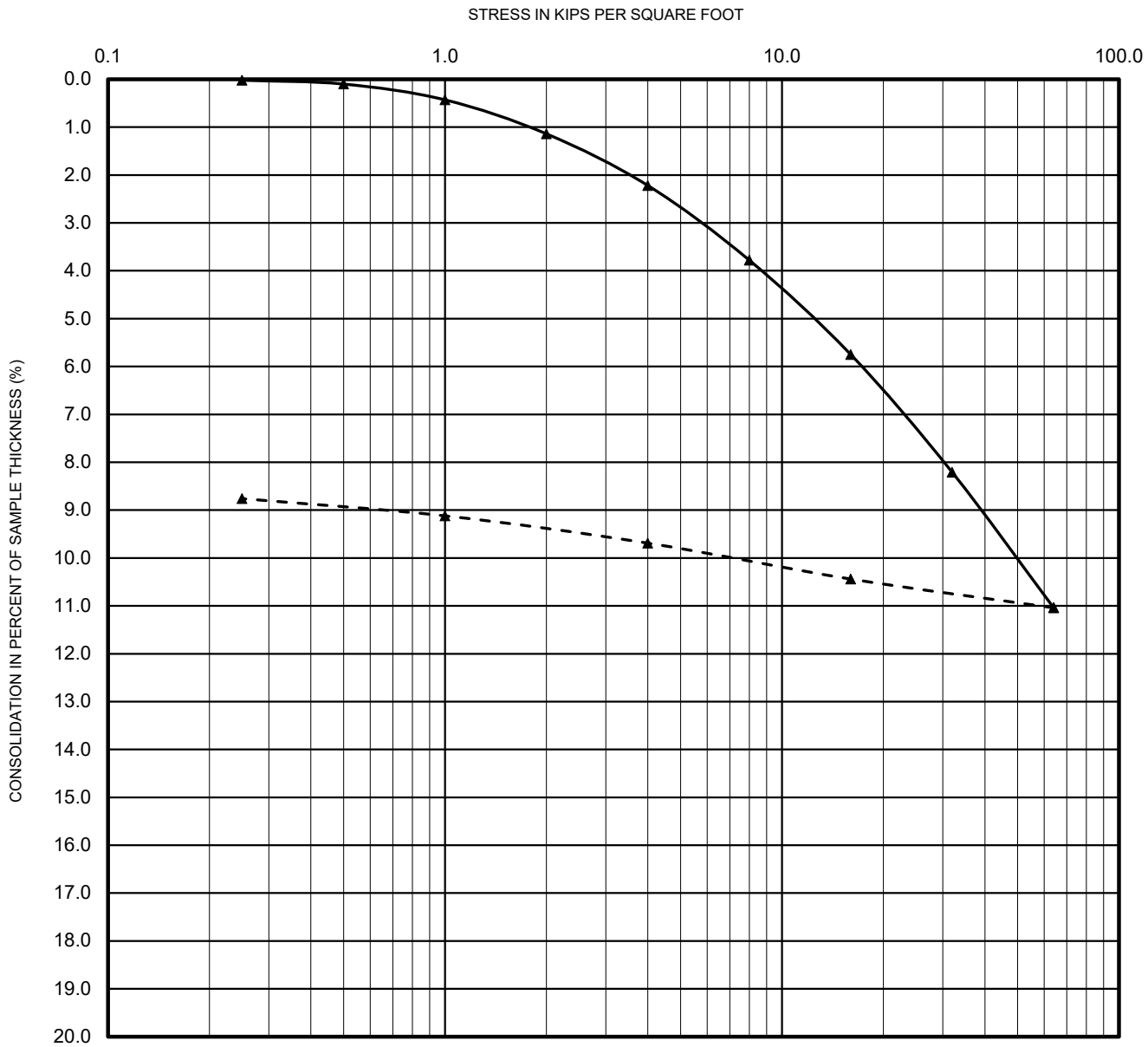
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-19

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



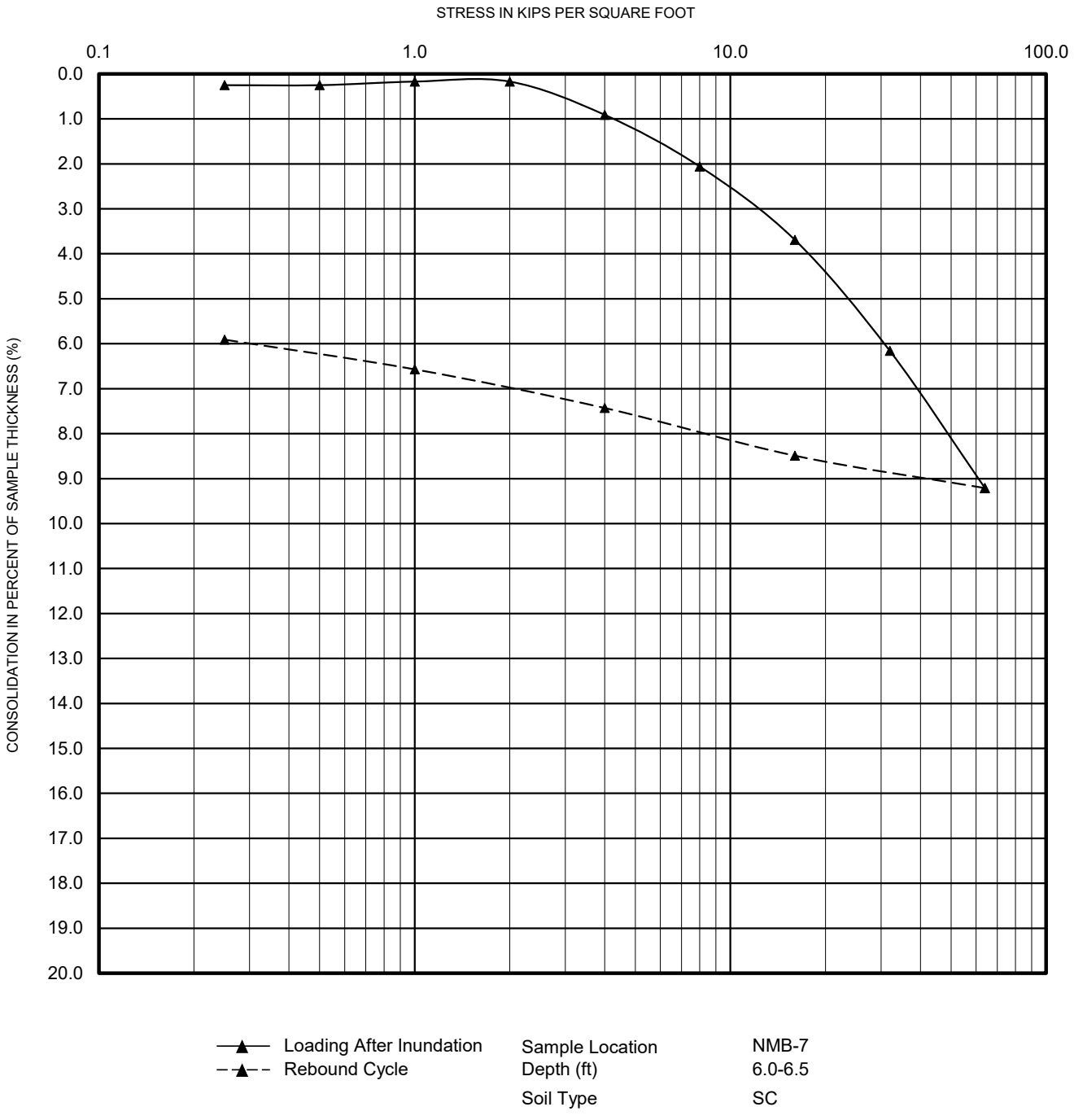
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-20

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



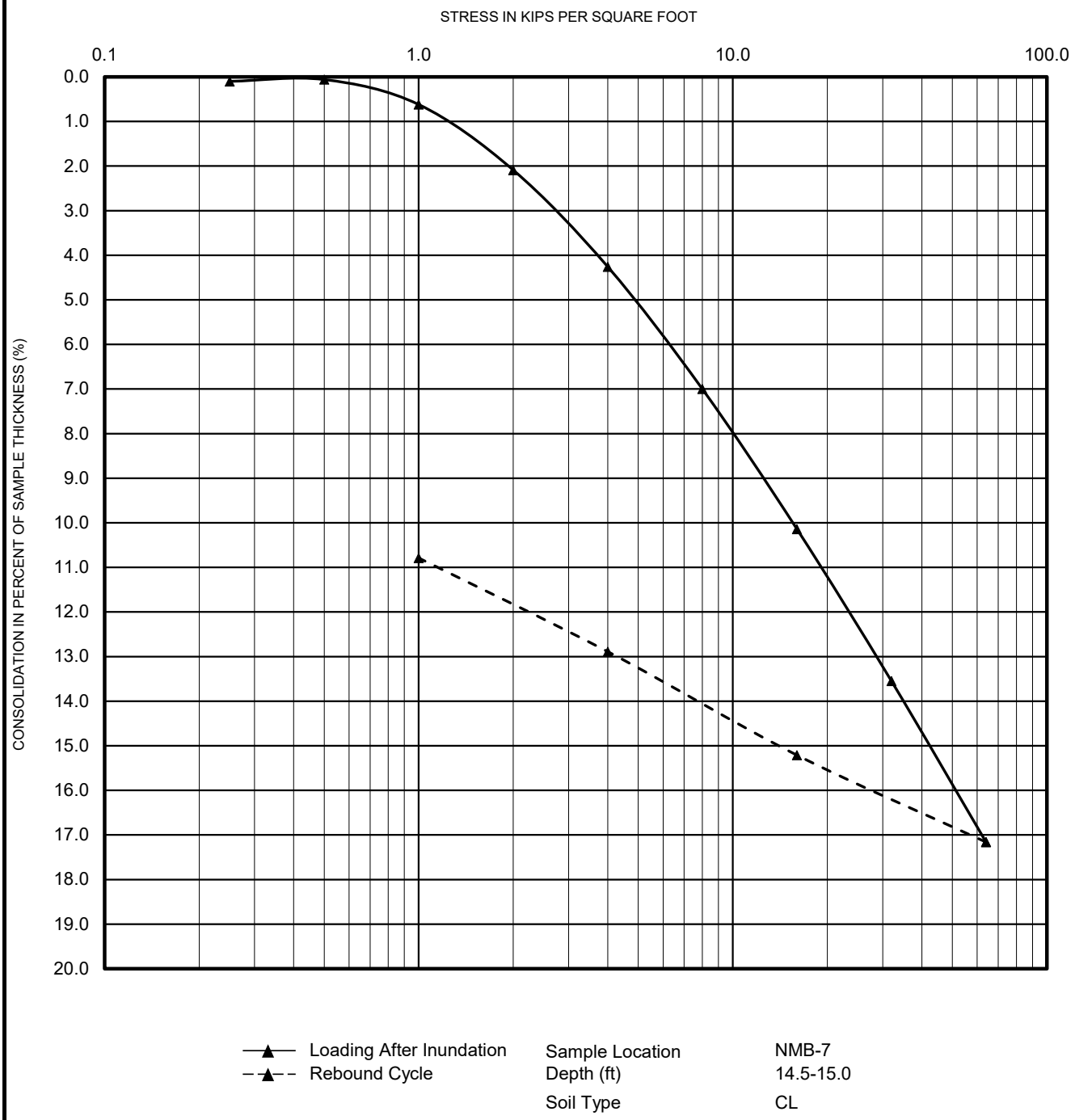
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-21

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



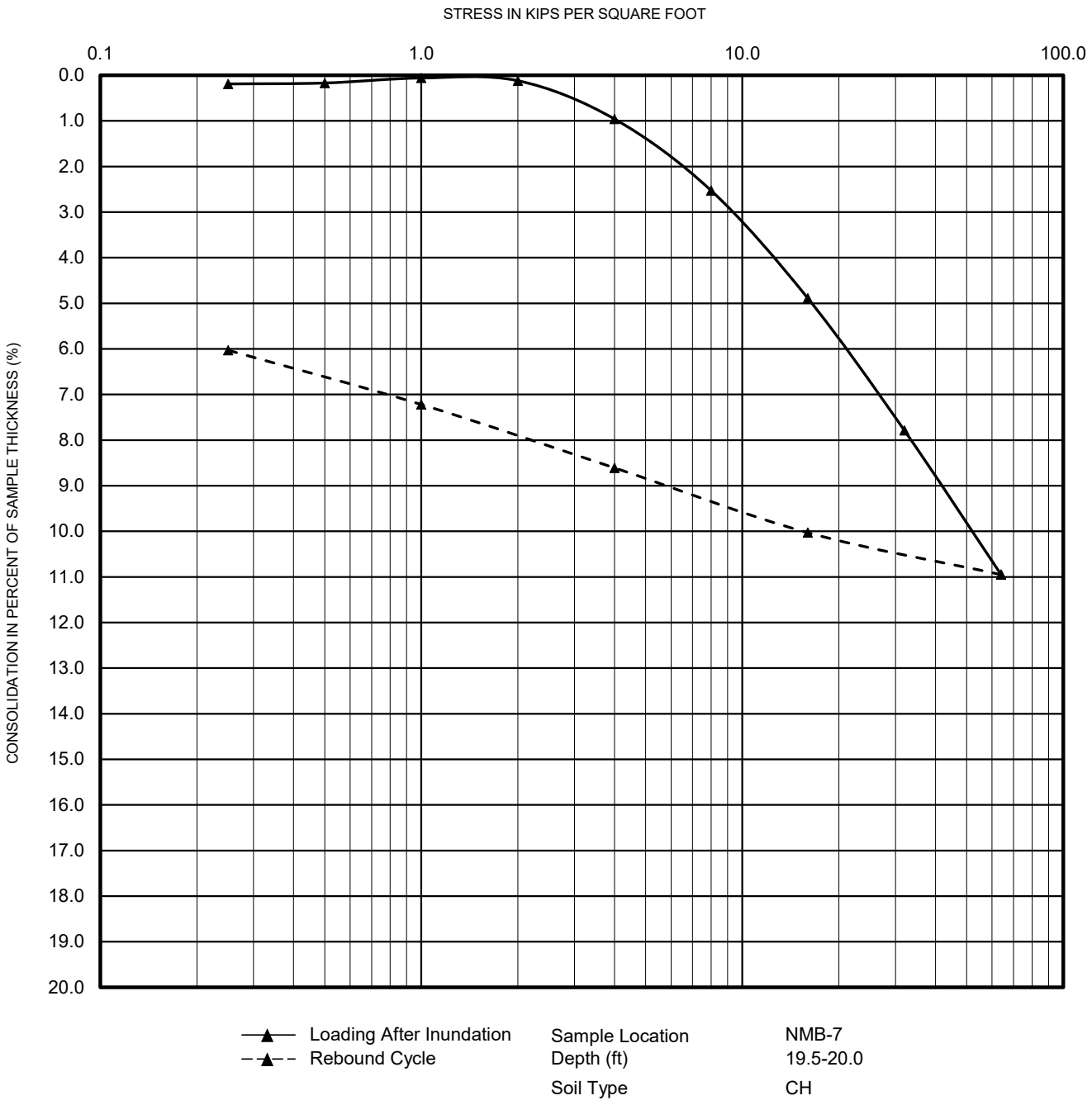
PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-22

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



PERFORMED IN ACCORDANCE WITH ASTM D 2435

FIGURE C-23

CONSOLIDATION TEST RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

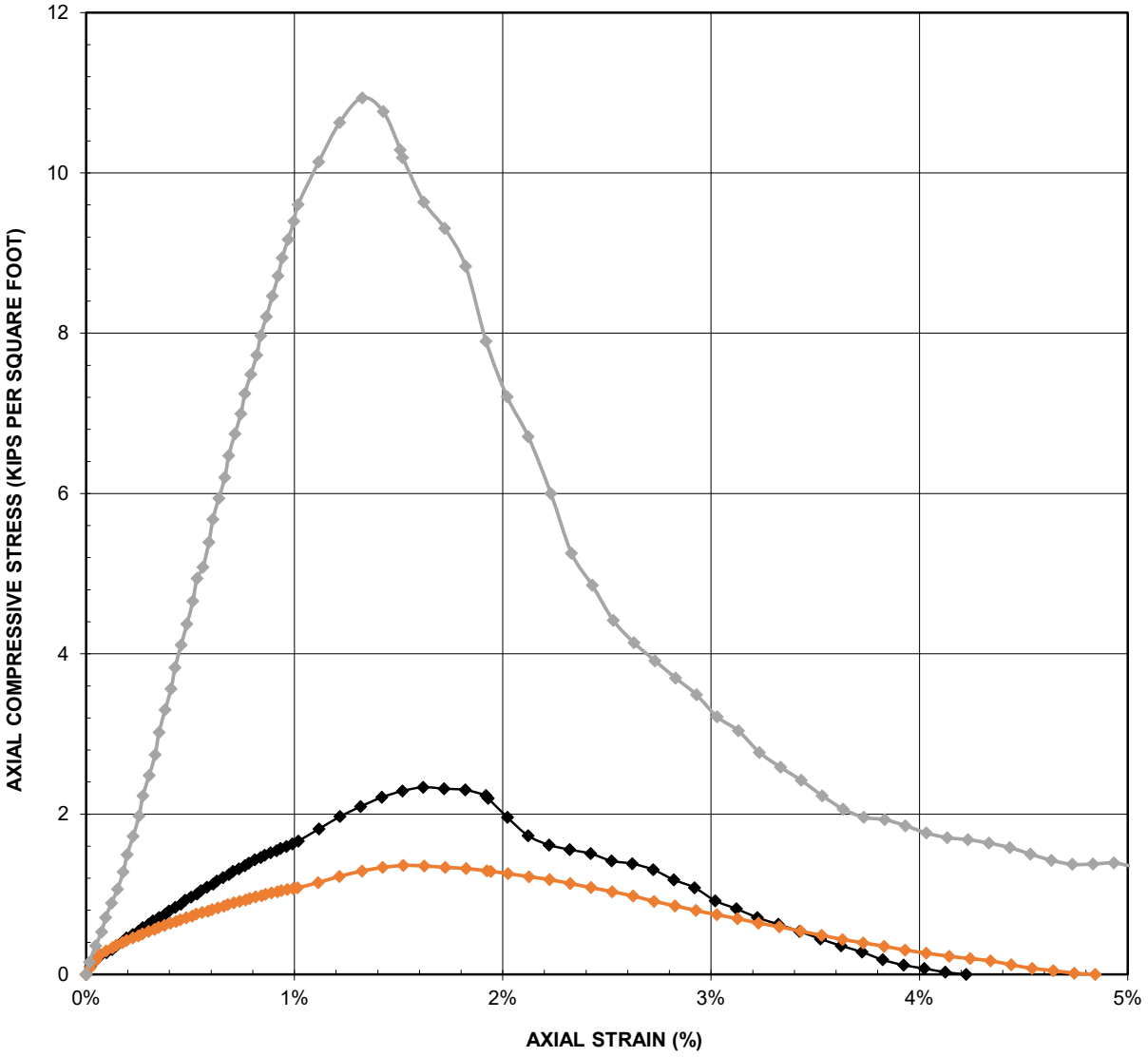
SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
NMB-2	0.0-5.0	15.0	92.8	30.0	0.039	39	Low
NMB-6	0.0-5.5	18.0	85.8	33.6	0.024	24	Low

PERFORMED IN ACCORDANCE WITH ASTM D 4829

FIGURE C-24

EXPANSION INDEX TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA



SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	STRAIN RATE (%/min.)	UNDRAINED SHEAR STR s_u, (ksf)
◆	Clayey SAND	SC	NMB-3	14.5-15.0	24.0	94.0	1.00	1.04
◆	Clayey SAND	SC	NMB-4	3.0-3.5	19.2	89.5	1.00	0.61
◆	Sandy CLAY	CL	NMB-9	11.0-11.5	21.2	95.7	1.00	4.89

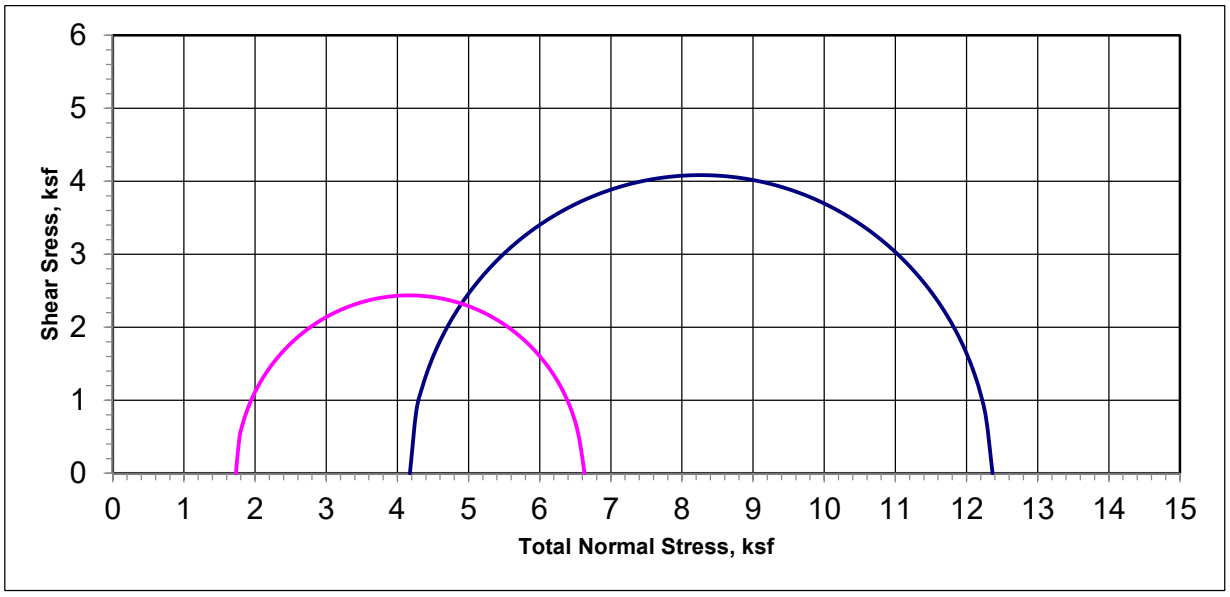
PERFORMED IN ACCORDANCE WITH ASTM D 2166

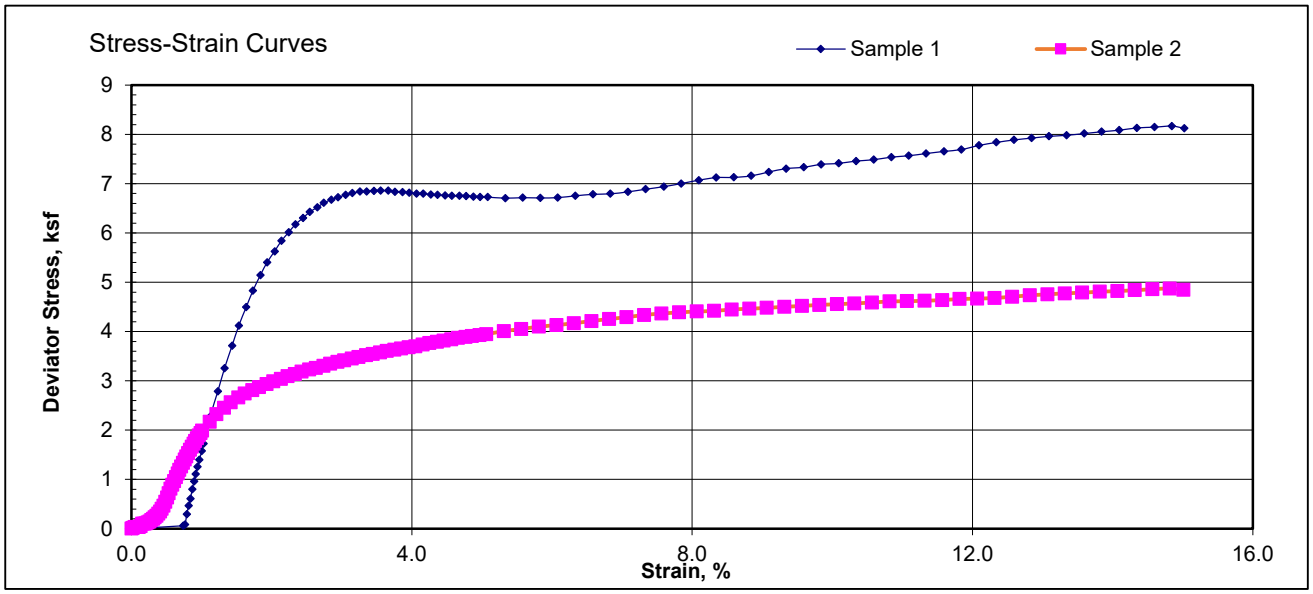
FIGURE C-25

UNCONFINED COMPRESSION RESULTS

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA
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SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH s_u, (ksf)
■	Hambre Siltstone	--	NMB-1	9.0-9.5	25.8	77.2	1.73	2.44
◆	Briones Siltstone	--	NMB-7	34.5-35.0	22.9	96.8	4.18	4.08

PERFORMED IN ACCORDANCE WITH ASTM D 2850

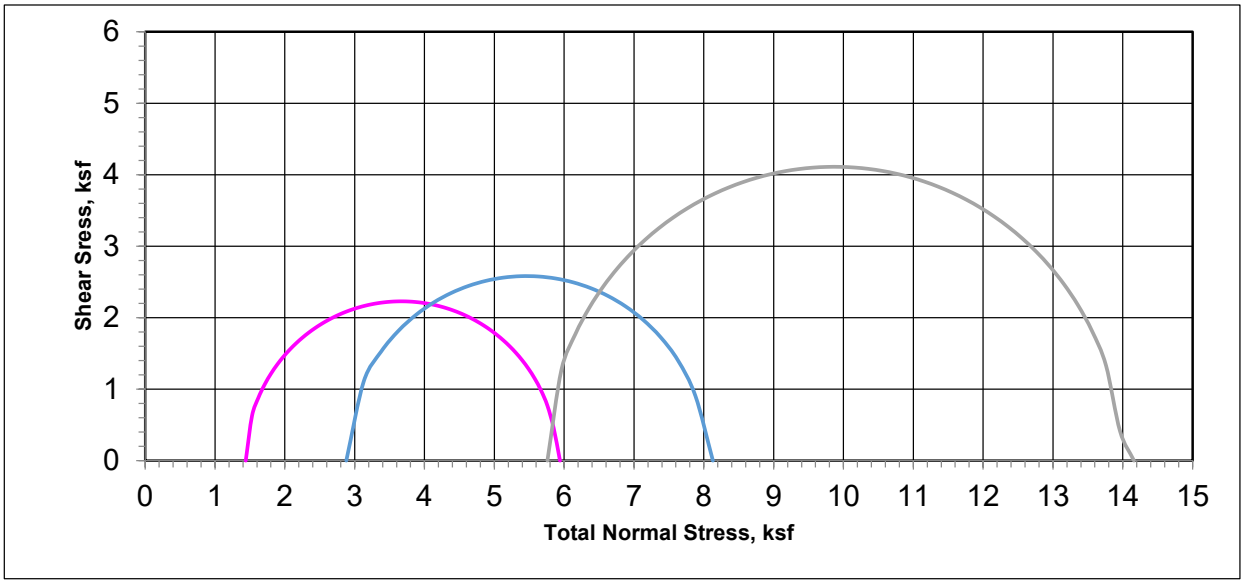
STRAIN RATE: 1.0%/MIN

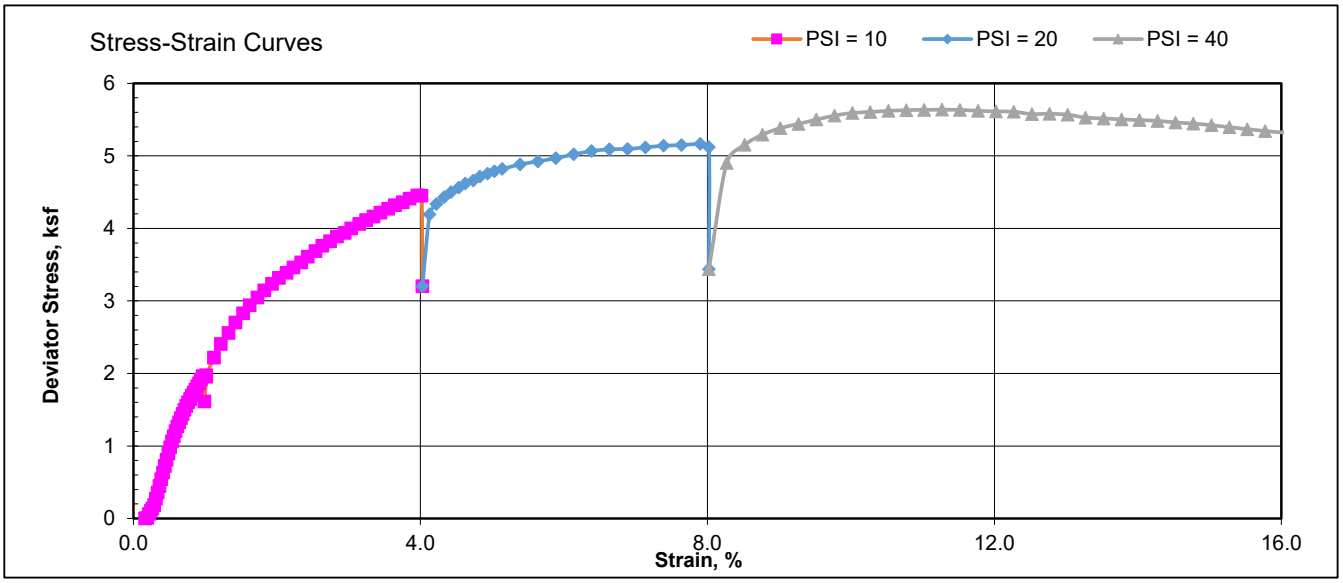
FIGURE C-26

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

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WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA
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SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH s_u, (ksf)
■	Silty SAND	SM	NMB-3	19.5-20	26.5	91.3	1.44	2.23
■	Silty SAND	SM	NMB-3	19.5-20	26.5	91.3	2.88	2.56
■	Silty SAND	SM	NMB-3	19.5-20	26.5	91.3	5.76	2.82

PERFORMED IN ACCORDANCE WITH ASTM D 2850

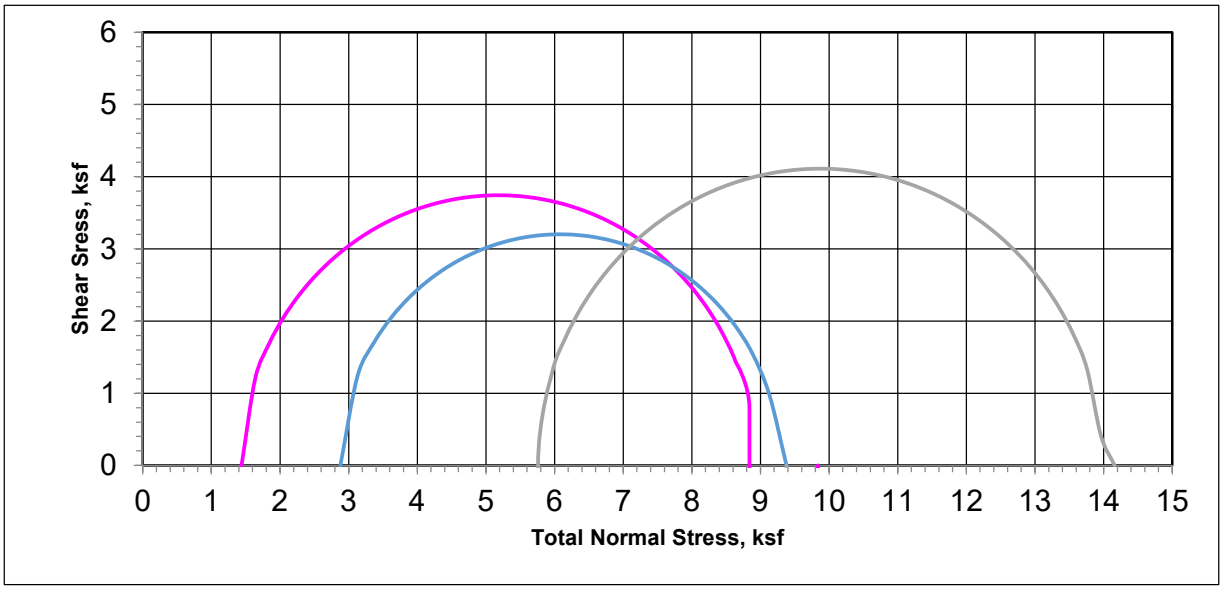
STRAIN RATE: 1.0%/MIN

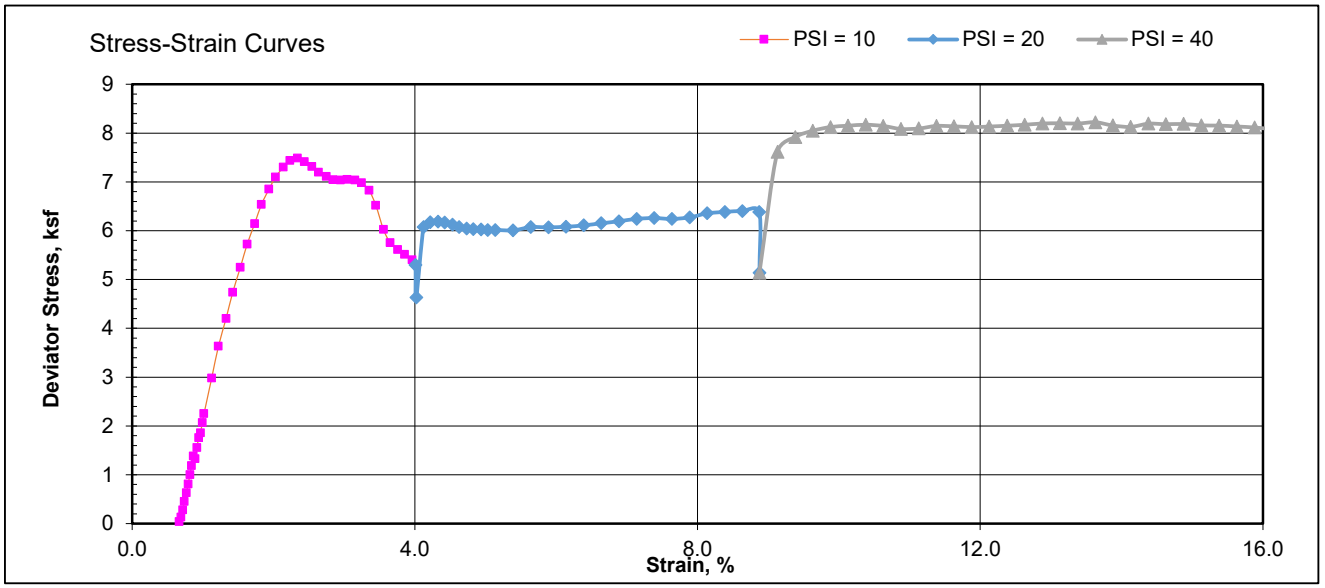
FIGURE C-27

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA
403982001 | 6/22





SYMBOL	DESCRIPTION	SOIL TYPE	SAMPLE LOCATION	SAMPLE DEPTH (ft.)	MOISTURE CONTENT w, (%)	DRY DENSITY γ_d, (pcf)	CELL PRESSURE (ksf)	UNDRAINED SHEAR STRENGTH s_u, (ksf)
■	Briones Siltstone	--	NMB-6	19.5-20	28.9	91.3	1.44	3.74
■	Briones Siltstone	--	NMB-6	19.5-20	28.9	91.3	2.88	3.20
■	Briones Siltstone	--	NMB-6	19.5-20	28.9	91.3	5.76	4.11

PERFORMED IN ACCORDANCE WITH ASTM D 2850

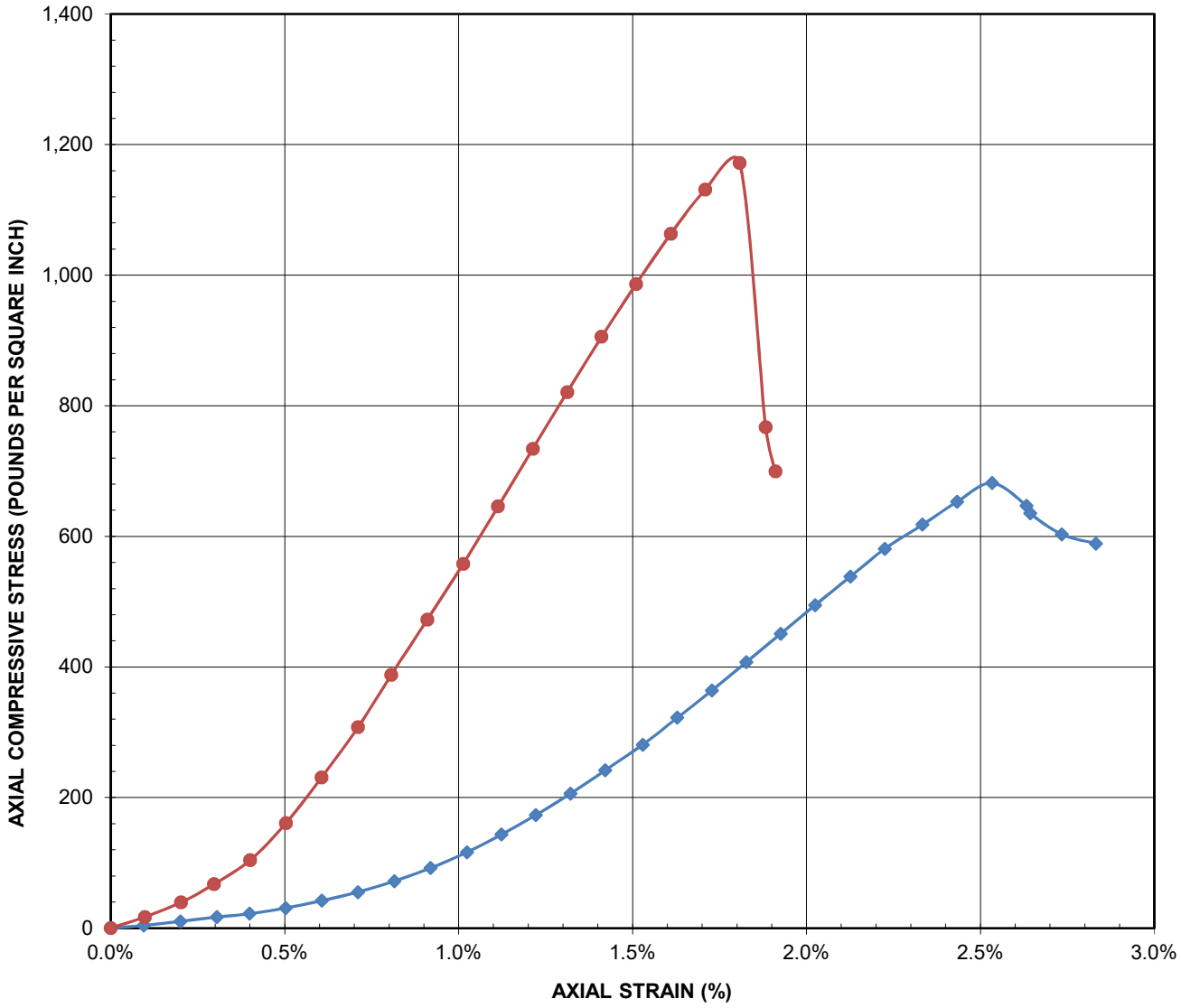
STRAIN RATE: 1.0%/MIN

FIGURE C-28

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA
403982001 | 6/22



SYMBOL	DESCRIPTION	SAMPLE LOCATION	SAMPLE DEPTH (feet)	SPECIMEN LENGTH (inches)	SPECIMEN DIAMETER (inches)	PEAK LOAD (lbs)	COMPRESS STRENGTH (psi)	YOUNGS MODULUS (ksi)
◆	Hambre Formation: Siltstone	NMB-1	20.6-21.0	4.04	2.27	2,754	680	38
●	Hambre Formation: Siltstone	NMB-1	21.0-21.5	4.47	2.31	4,909	1,170	80

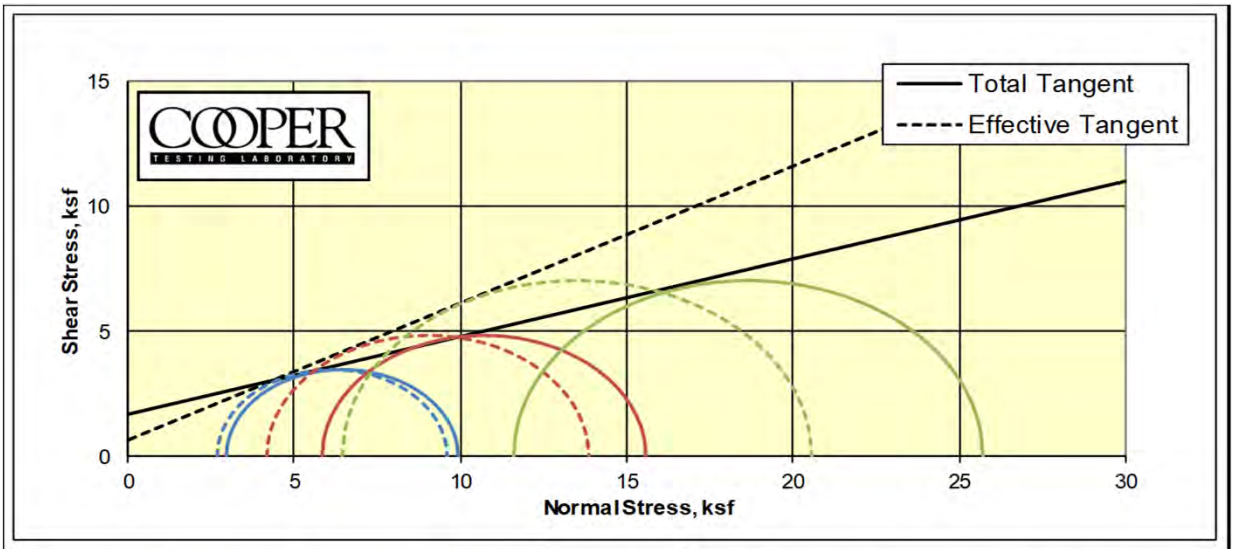
PERFORMED IN ACCORDANCE WITH ASTM D 7012

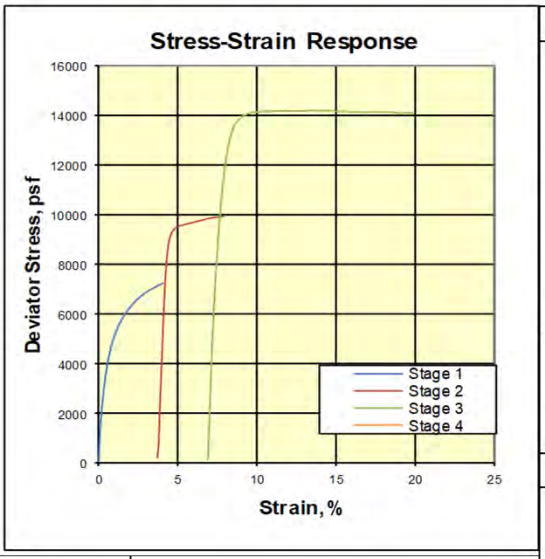
FIGURE C-29

ROCK CORE UNCONFINED COMPRESSION RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA





CTL Number:	357-131
Client Name:	Ninyo & Moore
Project Name:	2201 Larkey Lane
Project Number:	403982001
Date:	11/5/2021 By: MD/DC
Total C	1.700 ksf
Total phi	17.2 degrees
Eff. C	0.650 ksf
Eff. Phi	28.7 degrees ©

Remarks: Per the clients request the sample was consolidated to 80psi before rebounding to 20psi. Stage 1 was run at 20psi, and was strained to 4% strain. Stage 2 was run at 40 psi and was strained an additional 4% strain. Stage 3 was run at 80psi, and was strained an additional 14% strain. Strengths picked at the peak effective stress ratios.

Stage	1	2	3	4
Boring	NMB-4			
Sample				
Depth	14.5-15.0			
Visual Description	Olive Brown Sandy CLAY			
MC (%)	28.0			
Dry Density (pcf)	94.1			
Saturation (%)	93.5			
Void Ratio	0.824			
Diameter (in)	2.42			
Height (in)	5.03			
	Final			
MC (%)	26.7	26.2	25.2	
Dry Density (pcf)	99.0	99.8	101.4	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.735	0.720	0.693	
Diameter (in)	2.39	2.42	2.44	
Height (in)	4.92	4.74	4.58	
Cell Pressure (psi)	90.0	109.9	150.1	
Back Pressure (psi)	69.5	69.4	69.5	
	Effective Stresses At:			
Strain (%)	3.2	2.5	2.9	
Deviator (ksf)	6.953	9.728	14.115	
Excess PP (psi)	2.1	11.7	35.8	
Sigma 1 (ksf)	9.604	13.880	20.560	
Sigma 3 (ksf)	2.651	4.151	6.445	
P (ksf)	6.128	9.016	13.503	
Q (ksf)	3.476	4.864	7.057	
Stress Ratio	3.622	3.343	3.190	
Rate (in/min)	0.0005	0.0005	0.0005	

PERFORMED IN ACCORDANCE WITH ASTM D 4767

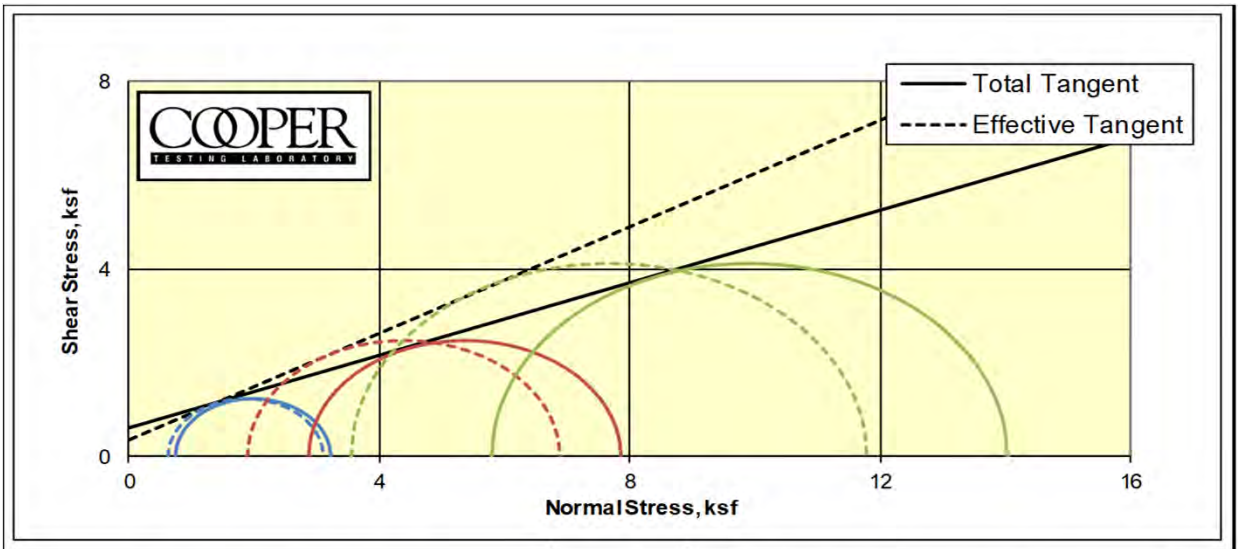
FIGURE C-30

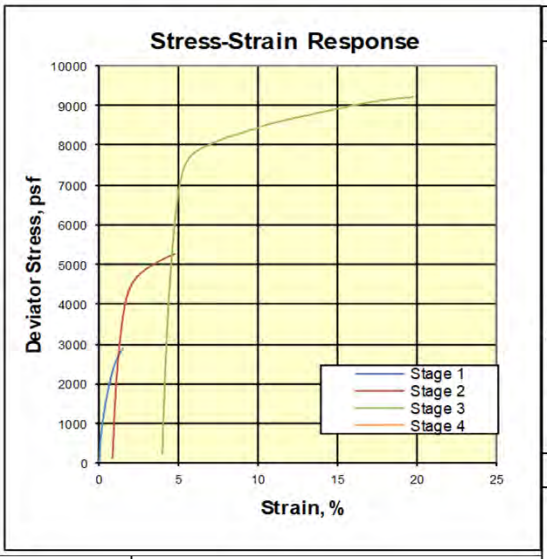
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants





CTL Number:	357-131
Client Name:	Ninyo & Moore
Project Name:	2201 Larkey Lane
Project Number:	403982001
Date:	11/5/2021 By: MD/DC
Total C	0.600 ksf
Total phi	21.1 degrees
Eff. C	0.350 ksf
Eff. Phi	29.6 degrees ©

Remarks: Per the clients request the sample was consolidated to 40psi before rebounding to 5psi. Stage 1 was run at 5psi, and was strained to 1.5% strain. Stage 2 was run at 20 psi and was strained an additional 4% strain. Stage 3 was run at 40psi, and was strained an additional 16.5% strain. Strengths picked at the peak effective stress ratios.

Stage	1	2	3	4
Boring	NMB-7			
Sample				
Depth	9.5-10.0			
Visual Description	Grayish Brown Clayey SAND			
MC (%)	23.5			
Dry Density (pcf)	96.3			
Saturation (%)	84.4			
Void Ratio	0.751			
Diameter (in)	2.41			
Height (in)	5.01			
	Final			
MC (%)	26.1	25.3	24.5	
Dry Density (pcf)	99.0	100.1	101.5	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.703	0.684	0.661	
Diameter (in)	2.39	2.38	2.41	
Height (in)	4.97	4.93	4.77	
Cell Pressure (psi)	85.1	99.8	119.8	
Back Pressure (psi)	80.0	79.8	79.6	
	Effective Stresses At:			
Strain (%)	1.0	2.5	4.9	
Deviator (ksf)	2.499	4.976	8.257	
Excess PP (psi)	0.7	6.8	15.6	
Sigma 1 (ksf)	3.125	6.886	11.803	
Sigma 3 (ksf)	0.626	1.910	3.546	
P (ksf)	1.875	4.398	7.674	
Q (ksf)	1.250	2.488	4.128	
Stress Ratio	4.993	3.605	3.329	
Rate (in/min)	0.0005	0.0005	0.0005	

PERFORMED IN ACCORDANCE WITH ASTM D 4767

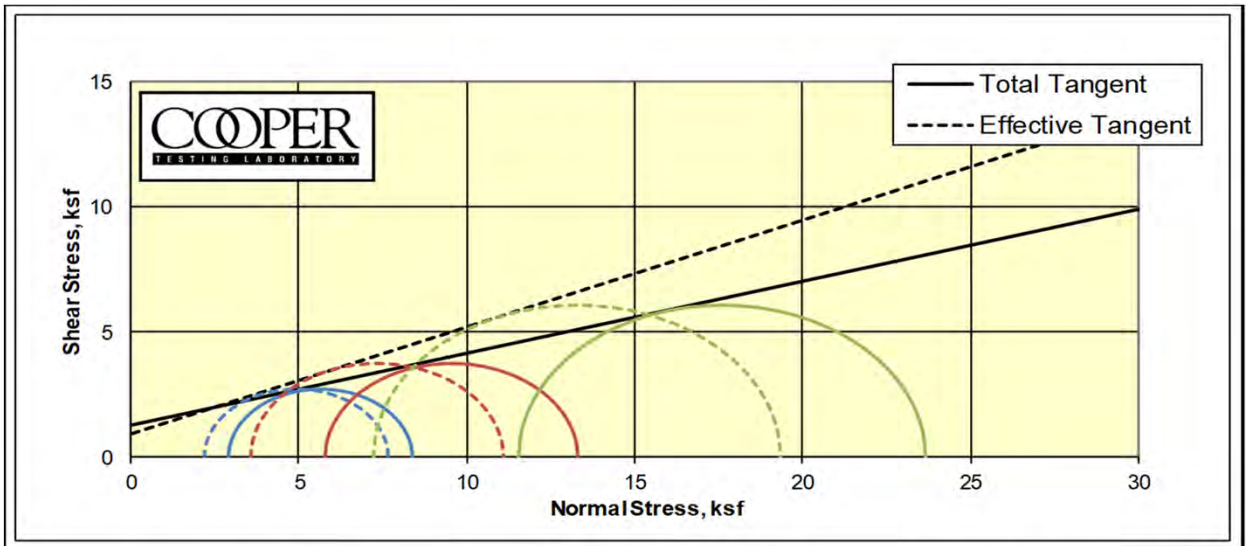
FIGURE C-31

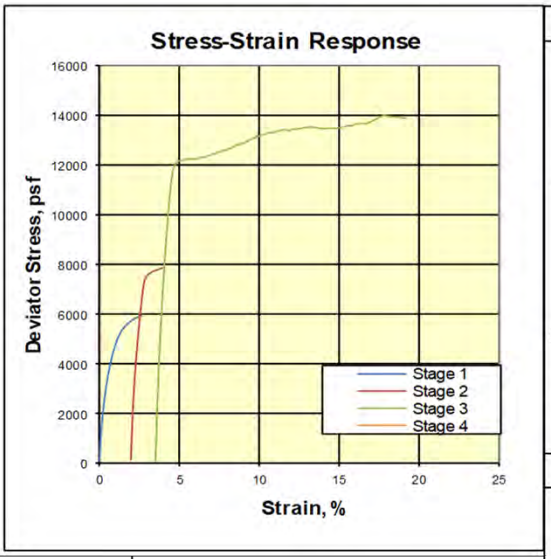
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants





CTL Number:	357-131
Client Name:	Ninyo & Moore
Project Name:	2201 Larkey Lane
Project Number:	403982001
Date:	11/5/2021 By: MD/DC
Total C	1.300 ksf
Total phi	16.0 degrees
Eff. C	0.950 ksf
Eff. Phi	23.1 degrees ©

Remarks: Per the clients request the sample was consolidated to 80psi before rebounding to 20psi. Stage 1 was run at 20psi, and was strained to 2.6% strain. Stage 2 was run at 40 psi and was strained an additional 2.1% strain. Stage 3 was run at 80psi, and was strained an additional 16.3% strain. Strengths picked at the peak effective stress ratios.

Stage	1	2	3	4
Boring	NMB-7			
Sample				
Depth	24.5-25.0			
Visual Description	Olive Brown Siltstone			
MC (%)	29.8			
Dry Density (pcf)	88.6			
Saturation (%)	85.5			
Void Ratio	0.974			
Diameter (in)	2.42			
Height (in)	5.03			
	Final			
MC (%)	33.4	32.7	31.6	
Dry Density (pcf)	90.3	91.2	92.7	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.936	0.916	0.885	
Diameter (in)	2.41	2.42	2.42	
Height (in)	4.99	4.89	4.82	
Cell Pressure (psi)	79.8	100.0	139.9	
Back Pressure (psi)	59.8	59.9	59.9	
	Effective Stresses At:			
Strain (%)	1.5	1.0	1.5	
Deviator (ksf)	5.500	7.551	12.125	
Excess PP (psi)	4.9	15.4	30.0	
Sigma 1 (ksf)	7.667	11.108	19.330	
Sigma 3 (ksf)	2.167	3.557	7.205	
P (ksf)	4.917	7.333	13.267	
Q (ksf)	2.750	3.776	6.063	
Stress Ratio	3.538	3.123	2.683	
Rate (in/min)	0.0005	0.0005	0.0005	

PERFORMED IN ACCORDANCE WITH ASTM D 4767

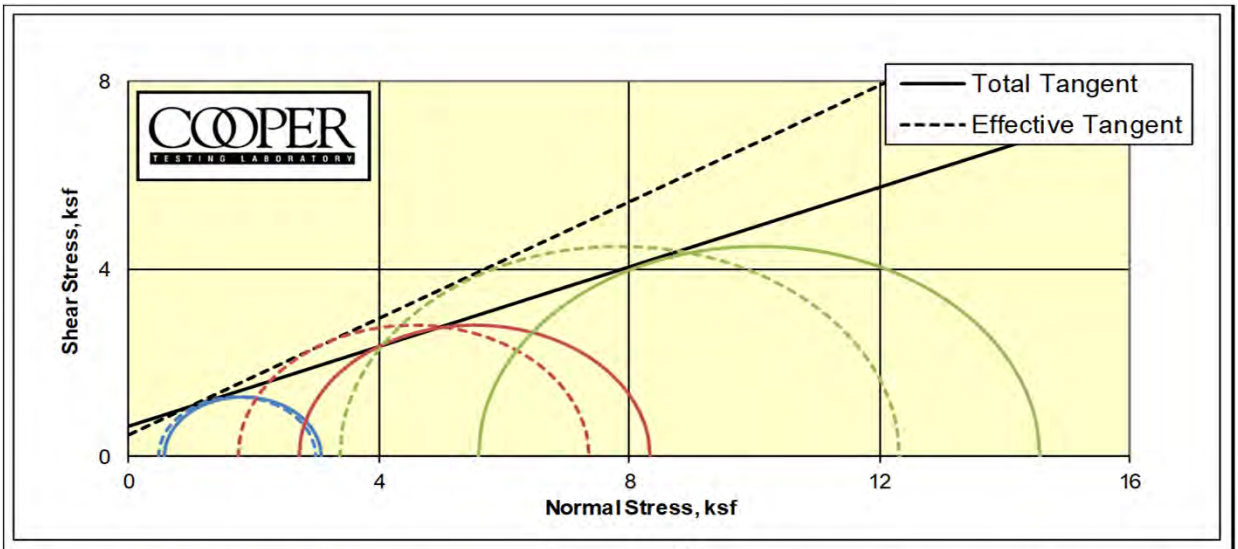
FIGURE C-32

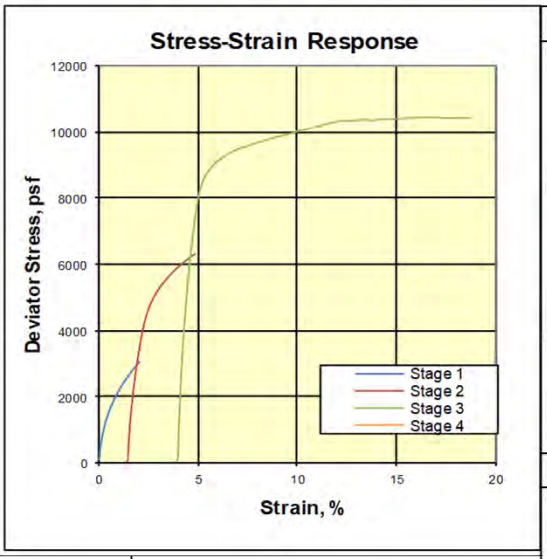
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants





CTL Number:	357-131
Client Name:	Ninyo & Moore
Project Name:	2201 Larkey Lane
Project Number:	403982001
Date:	11/11/2021 By: MD/DC
Total C	0.650 ksf
Total phi	22.9 degrees
Eff. C	0.450 ksf
Eff. Phi	31.8 degrees ©

Remarks: Per the clients request the sample was consolidated to 40psi before rebounding to 5psi. Stage 1 was run at 5psi, and was strained to 2% strain. Stage 2 was run at 20 psi and was strained an additional 3.5% strain. Stage 3 was run at 40psi, and was strained an additional 15.5% strain. Strengths picked at the peak effective stress ratios.

Stage	1	2	3	4
Boring	NMB-9			
Sample				
Depth	16.0-16.5			
Visual Description	Grayish Brown Siltstone			
MC (%)	20.5			
DryDensity(pcf)	90.7			
Saturation (%)	61.9			
Void Ratio	0.926			
Diameter (in)	2.42			
Height (in)	5.01			
	Final			
MC (%)	34.0	33.2	32.5	
DryDensity(pcf)	89.6	90.5	91.5	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.952	0.930	0.910	
Diameter (in)	2.44	2.44	2.46	
Height (in)	5.01	4.94	4.81	
Cell Pressure (psi)	104.9	119.8	139.7	
Back Pressure (psi)	100.9	100.9	100.8	
	Effective Stresses At:			
Strain (%)	1.3	2.1	1.9	
Deviator (ksf)	2.513	5.620	8.957	
Excess PP (psi)	0.6	6.8	15.5	
Sigma 1 (ksf)	3.005	7.363	12.327	
Sigma 3 (ksf)	0.492	1.744	3.369	
P (ksf)	1.749	4.554	7.848	
Q (ksf)	1.257	2.810	4.479	
Stress Ratio	6.111	4.223	3.658	
Rate (in/min)	0.0005	0.0005	0.0005	

PERFORMED IN ACCORDANCE WITH ASTM D 4767

FIGURE C-33

CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

21 January, 2022

Job No. 2201023
Cust. No.13270

Mr. Peter Connolly
Ninyo & Moore
2149 O'Toole Avenue, Suite 30
San Jose, CA 95131

Subject: Project No.: 403982001
Project Name: Walnut Creek Water Treatment Plant, 2201 Larkey Lane, Walnut Creek
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Connolly:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on January 20, 2022. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the 100% saturated resistivity measurements, all three of the samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations ranged from none detected to 90 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations ranged from 40 to 1,300 mg/kg and are determined to be sufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations. Therefore, concrete that comes into contact with this soil should use sulfate resistant cement such as Type II, in accordance with the California Building Code requirements with a maximum water-to-cement ratio of 0.50.

The sulfide ion concentrations reflect none detected with a detection limit of 50 mg/kg.

The pH of the soils ranged from 6.20 to 8.35 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 320 to 430-mV. Samples No.002 and No.003 are indicative of potentially "slightly corrosive" soils resulting from anaerobic soil conditions, and Sample No.001 is indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President
JDH/jdl

Enclosure

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Ninyo & Moore
Client's Project No.: 403982001
Client's Project Name: Walnut Creek Water Treatment Plant
Date Sampled: 8-Oct-21
Date Received: 20-Jan-22
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 21-Jan-2022

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2201023-001	NMB-2 0.0-50'	430	6.20	16,000	1,600	N.D.	N.D.	58
2201023-002	NMB-5 0.0-5.0'	300	7.80	770	560	N.D.	90	1,300
2201023-003	NMB-9 10.0-11.0'	320	8.35	2,700	990	N.D.	N.D.	40

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	21-Jan-2022	21-Jan-2022	20-Jan-2022	20-Jan-2022	21-Jan-2022	21-Jan-2022	21-Jan-2022

Cheryl McMillen for
Sherri Moore
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

Laboratory Testing from Previous Investigations

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
MM-1	3	14	Fill(CH)	27.4		54	33					SG = 2.75
	6	13	Fill(CH)	32.2		52	30			74		
	9	23	Fill(ML)	27.4		40	8					
	12		Fill(SM)	27.4	94.5					36	CU: Su=2.7 ksf (1.3 ksf)	SG=2.69
	14	29	Fill(SM)	30.4						47		
	16	32	Fill(SM)	27.6								
	19	22	Fill(SM)	29.6						45		
	22		Fill(SM)	25.2	96.2					35	CU: Su=3.1 ksf (2.6 ksf)	
	24	45	Fill(SM)	24.9						43		
	26	32	Fill(SM)	22.1								
	29	49	Fill(SM)	21.6						44		
	34	47	Fill(SM)	24.6						39		
	36	34	Fill(SM)	22.3								
	39	37	Fill(SM)	24.8				27	30	43		
	41		Fill(SM)	22.1	97.2					34	CU: Su=6.6 ksf (5.1 ksf)	
	42		Fill(GM)	9.2	112.0			29	26	45		pH=6.3
	44	30	Fill(SM)	25.7		41	14					
	47	49	Fill(GM)	24.2				45	27	28		
	50		Fill(SM)	24.1	92.7					37	CU: Su=6.4 ksf (5.9 ksf)	
	52		Fill(SM)		90.2						CU: Su=5.0 ksf (6.6 ksf)	
	54		Fill(SM)	29.5	93.3			18	56	26		pH = 5.7
	55		Fill(SM)	32.1	89.4					45		
	56	56	Fill(SM)	30.3		39	9					
	59		Fill(ML/GM)	30.2	89.8						CU: Su=3.2 ksf (2.4 ksf) CU: Su=6.1 ksf (4.9 ksf) CU: Su=8.6 ksf (7.1 ksf)	
	60		Fill(GM)	33.6	100.4	60	24	30	26	43		
	61	35	Fill(ML)	30.5								
	66	50	Fill(SM)			39	5	19	39	42		
	76	200	Sandstone									
MM-2	3	15	Fill(ML)	17.9		42	25			53		
	6	11	Fill(ML)	31.5		53	31					
	9	19	Fill(SM)	34.5		54	22			38		
	12		Fill(SM)	27.7	91.3	40	7				CU: Su=1.4 ksf (1.1 ksf)	SG=2.61
	14	25	Fill(SM)	28.6						30		
	16	28	Fill(SM)	31.6								
	19	20	Fill(SM)	35.3				25	42	32		
	24	27	Fill(SM)	26.2				27	36	36		
	26	114	Sandstone									
	28	190	Sandstone									
	33	100	Sandstone									
MM-3	4	37	Fill(SM)	24.3						29		
	6	12	Fill(CL)	31.4		48	16					
	9	18	Fill(SM)	36.1		45	16	17	40	43		
	12		Fill(SM)	24.5	98.5	30	4			23	CU: Su=1.8 ksf (2.4 ksf)	
	14	24	Fill(CL)									
	16	25	Fill(SM)	32.8								
	19	26	Fill(SM)	25.1								
	22		Fill(SM)	36.4	81.8	40	13			30	CU: Su=5.7 ksf (4.5 ksf)	
	24	25	Fill(SM)	35.4								
	26	21	Fill(SM)	34.5				33	40	27		
	29	46	Fill(SM)	22.3								
	34	28	Fill(SM)	25.9		36	5					
	37		Fill(SM)	25.8	95.2						CU: Su=4.4 ksf (4.5 ksf)	SG=2.65
	36		Fill(SM)	25.8	95.2							
	39	35	Fill(SM)	29.3		42	12			29		
	41		Fill(SM)					37	55	8		pH=5.6
	42		Fill(SM)							27		

FIGURE D-1

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

403982001 | 06/22

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
MM-3 (Cont)	44	27	Fill(SM)	31.2		39	8					
	46	45	Fill(SM)	30.6		37	6					
	49	40	Fill(SM)	30.8		44	13	21	44	35		
	51		Fill(SM)	29.8	90.0					35	CU: Su=6.2 ksf (6.4 ksf)	
	54	55	Sandstone	24.8		39	5	21	38	41		
	56	50	Sandstone									
	58	120	Sandstone									
	62		Siltstone	27.0	95.9						CU: Su=7.6 ksf (7.6 ksf)	
	65	200	Siltstone									
MM-4	4	27	Fill(CL)	12.6		36	16					
	6	17	Fill(SM)	23.8						39		
	9	23	Fill(SM)	33.0		45	17					
	12		Fill(SM)	32.9	84.0					23	CU: Su=1.8 ksf (1.2 ksf)	
	14	23	Fill(SM)	32.9								
	16	23	Fill(CH)	35.3		58	29					
	19	21	Fill(CH)	31.9		55	28					
	22		Fill(SM/ML)	33.4	87.9	55	22			18	CU: Su=2.7 ksf (2.6 ksf)	SG=2.72
	24	27	Fill(SM/ML)	36.8								
	26	39	Fill(SM/ML)	24.3						51		
	29	33	Fill(SM/ML)	27.3		38	9					
	32		Siltstone	27.3		38	9				CU: Su=2.7 ksf (2.6 ksf) CU: Su=7.1 ksf (5.0 ksf) CU: Su=9.8 ksf (7.0 ksf)	
	34	120	Siltstone									
MM-5	3	19	Fill(GP)									
	6	16	Fill(ML)	27.5		47	18					
	8	8	Fill(ML)	32.7		45	16					
	14	128	Sandstone									
MN-2	3	59	Siltstone	14.0		36	5					
	6	60	Siltstone	17.0		34	1			74		
MN-3	2	30	Soil(CL)							36		
	6	63	Siltstone							60		
	11		Sandstone	21.6	95.9						UU: Su=1.6 ksf (1.2 ksf)	SG=2.62
	21		Sandstone	23.1	100.3						UU: Su=0.9 ksf (2.4 ksf)	SG=2.62
MN-4	2	28	Claystone	27.0		52	14					
	6	36	Sandstone									SG=2.55
	11	90	Sandstone							46		
	16		Siltstone	22.1	100.8						UU: Su=4.4 ksf (1.8 ksf)	SG=2.57
MN-8	3	113	Sandstone									
	6	113	Sandstone									
	11		Sandstone	19.8	104.0						UU: Su=9.9 ksf (1.2 ksf)	SG=2.71
	13	75	Sandstone									
MN-9	3	19	Fill(CL)	20.0		38	18					
	6	8	Soil(ML)	25.0		43	21					
	9	11	Soil(ML)							66		
	14	54	Sandstone	22.0						44		
	20		Siltstone	25.6	96.4						UU: Su=0.4 ksf (2.2 ksf)	SG=2.57
	24	143	Claystone									
MN-10	3	38	Fill(SM)									
	6	24	Sandstone	17.2								
	11	41	Sandstone	18.3								
	16		Sandstone	16.0								SG=2.61
	18	102	Sandstone									
MN-13	2	90	Sandstone									
	7		Sandstone	17.8	109.3						UU: Su=3.6 ksf (0.6 ksf)	SG=2.63
	18	102	Sandstone									

FIGURE D-2

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
MN-14	3	31	Soil(CH)	15.0		52	33					
	5	113	Sandstone									
	12		Sandstone	19.8	112.7							SG=2.63
	20		Sandstone	18.9	103.4							SG=2.55
MN-15	3	11	Fill(CL/CH)	17.0								
	7	10	Fill(SC)	23.6	104.3	43	26					
	11	12	Soil(CH)	23.0		58	29					
	16	10	Soil(CH)	28.7	94.7							SG=2.50
	21	8	Soil(CH)	19.3								
	26	15	Claystone	28.0		68	43					
	31	41	Claystone	26.1								
	36	35	Siltstone	25.1	99.7						UU: Su=3.3 ksf (4.2 ksf)	SG=2.60
	47		Siltstone	13.5	116.8						CU: Su=4.6 ksf (5.4 ksf)	SG=2.63
MN-16	3	11	Claystone	28.8	92.1	62	34					SG=2.71
	6	26	Claystone	23.0		65	36					
	11	22	Claystone	29.0		66	36					
	16	7	Claystone	19.9	103.4						CU: Su=7.0 ksf (1.8 ksf)	
	20	95	Claystone	19.3								
	27		Claystone	18.9	107.3						UU: Su=3.4 ksf (3.0 ksf)	
	30	114	Siltstone									
	42		Siltstone	11.9	113.5						CU: Su=5.0 ksf (3.0 ksf) CU: Su=5.9 ksf (4.0 ksf) CU: Su=7.1 ksf (5.0 ksf)	SG=2.64
MN-17	3	15	Soil(CH)	14.0		51	32					
	6	13	Claystone	22.9	84.2						CU: Su=0.9 ksf (0.5 ksf) CU: Su=1.0 ksf (1.0 ksf)	
	11	23	Claystone	34.0		79	41					
	16	19	Claystone	32.7	87.2						CU: Su=1.9 ksf (1.8 ksf)	SG=2.53
	21	36	Claystone	32.8								
	26	35	Claystone	30.4	88.8						UU: Su=5.6 ksf (3.0 ksf)	SG=2.5
	31	86	Siltstone	27.2								
	42		Siltstone	12.7	112.7						CU: Su=13.4ksf (5.0 ksf)	SG=2.43
MN-18	3	29	Fill(CL)			40	20					
	6	25	Fill(CL)	21.0		38	13					
	11	11	Fill(CL)	23.3	104.8							SG=2.58
	16	14	Fill(CL)	28.0		45	24					
	21	11	Claystone	29.3	88.9						CU: Su=2.0 ksf (2.4 ksf)	SG=2.47
	26	27	Claystone	34.0		64	30					
	31	23	Claystone	33.1	86.9						UU: Su=1.5 ksf (3.6 ksf)	
	36	32	Claystone	48.4								
	40	80	Shale									
	46	63	Shale	24.6								
	51		Shale	20.4	109.6						UU: Su=0.9 ksf (6.0 ksf)	SG=2.62
	55	120	Shale	18.9								
	70	64	Shale	20.3	105.4						CU: Su=2.5 ksf (5.0 ksf) CU: Su=3.0 ksf (6.0 ksf) CU: Su=3.3 ksf (7.0 ksf)	SG=2.58
	82		Claystone	18.1	110.1						UU: Su=1.7 ksf (8.0 ksf)	
MN-19	3	7	Fill(CH)	24.0		85	54					
	6	13	Siltstone	21.7	90.4						CU: Su=4.4 ksf (0.6 ksf)	SG=2.58
	11	21	Claystone	34.0		68	36					
	16	32	Claystone	25.9	90.3						CU: Su=2.2 ksf (1.8 ksf) CU: Su=4.2 ksf (2.4 ksf) CU: Su=5.3 ksf (3.6 ksf)	
	21	50	Claystone	28.2								
	25	60	Claystone	30.4	90.6						UU: Su=6.8 ksf (3.0 ksf)	SG=2.59
	31	46	Claystone	32.0								
	40	64	Shale	28.7	93.0						CU: Su=5.7 ksf (4.8 ksf)	

FIGURE D-3

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
MN-20	3	29	Fill(CL)	24.0		68	38					
	6	65	Siltstone	17.0		47	17					
MN-21	3	19	Soil(CH)	21.0		78	52					
	6	10	Siltstone	19.0		48	27					
	11	8	Claystone	30.5	88.2	71	37				UU: Su=2.5 ksf (1.2 ksf)	SG=2.58
	16	20	Claystone	34.0		64	29					
	21	29	Siltstone	22.3	100.1						UU: Su=4.2 ksf (2.4 ksf)	SG=2.66
	26	42	Siltstone	20.7								
	31	33	Siltstone	26.5	98.1						UU: Su=4.3 ksf (3.5 ksf)	SG=2.71
MN-22	2	53	Fill(SM)	10.5								
	5	75	Fill(SM)	5.8								
	11		Fill(SM)	17.5	113.2						UU: Su=1.1 ksf (1.2 ksf)	SG=2.58
	22		Sandstone								UU: Su=9.7 ksf (2.4 ksf)	SG=2.57
	32		Sandstone	14.8	114.5						UU: Su=2.3 ksf (3.6 ksf)	SG=2.63
MN-23	4	46	Fill(CL)	18.0	83.5	41	19			73		
	6	29	Fill(CL-ML)	20.9		40	15			69		
	11	39	Fill(CL)	20.3		35	9			54		
	16	31	Soil(CL)	25.0		39	18			83		
	21	16	Soil(CL)	22.2								
	27		Soil(CL)	21.4	106.3	37	17			84		
	30	285	Soil(CL)									
	33	200	Siltstone	17.1						76		
	43	200	Siltstone	18.9								
MN-23A	4	82	Fill(CL)	12.6		45	22			84		
	6	37	Fill(CL)									
	11	150	Fill(CL)									
	16	170	Fill(CL)									
	21	190	Sandstone	18.8								
	26	190	Sandstone									
	31	285	Sandstone	17.8								
	36	200	Sandstone									
	40	240	Sandstone	20.1								
MN-24	4	75	Soil(SM/SC)	13.4		33	9			48		
	6	113	Soil(SM/SC)	17.2								
	11	113	Sandstone	21.0						45		
	16	113	Sandstone									
	21	95	Sandstone	25.8								
	26	143	Sandstone									
	33		Sandstone	21.2	102.6							
MN-25	4	34	Fill(CL)	19.8		43	20			70		
	6	39	Fill(CL)	20.2		40	18			74		
	11	19	Claystone	28.6								
	16	39	Claystone									
	21	28	Claystone	29.9								
	28		Claystone	38.4	80.1							
	31	65	Claystone	20.9								
	36	93	Claystone									
	41	88	Claystone									
	48		Claystone	14.3	121.5							
MN-26	4	20	Claystone	29.7								
	6	27	Claystone	28.9								
	11	39	Claystone									
	18		Claystone	28.9	93.0							
	21	76	Claystone	26.6								
	26	95	Claystone									
	30	104	Claystone									

FIGURE D-4

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Boring ID	Sample Depth (feet)	N60,spt bpf	Formation (USCS)	Percent Moisture Content	Dry Density (pcf)	Liquid Limit	Plasticity Index	Percent Gravel	Percent Sand	Percent Fines	Strength & Consolidation Results	Other Test Results
MN-27	4	29	Fill(CL)	14.8		46	25			67		
	6	21	Fill(CL)	19.6		44	23					
	11	7	Soil(CH)	36.2		72	50			90		
	13	9	Soil(CH)	32.5	88.5	76	52			90		
	15	9	Soil(CH)	34.5	85.9							
	20	31	Claystone	30.7	91.1							
	28		Claystone	26.7	97.9							
	30	126	Claystone									
MN-28A	4	20	Fill(CL)	14.0		38	17			72		
	6	34	Fill(CL)	17.6		46	21			77		
	11	14	Fill(CL)									
	16	26	Claystone	29.8								
	23		Claystone	35.4	87.3							
	26	42	Claystone	24.3								
	31	86	Claystone	24.5								
MN-29A	4	17	Claystone	30.3								
	6	29	Siltstone	30.7								
	11	23	Siltstone	29.3								
	16	43	Siltstone	29.8								
	23		Claystone	27.3	94.3					98		
	26	143	Claystone	25.7								
	30	162	Siltsone	25.1								
MN-30	4	49	Fill(CL)	12.8		47	26			68		
	6	29	Fill(CL)	8.5		32	13			76		
	11	18	Claystone	31.4								
	16	22	Claystone	32.0								
	21	36	Claystone	30.8								
	26	49	Claystone	31.7								
	31	54	Siltstone	29.9								
MN-31	4	150	Soil(CL)	21.6						56		
	6	113	Sandstone	19.6								
	11	150	Sandstone									
	16	255	Sandstone	14.9								
	21	285	Sandstone									
	28		Sandstone	17.4								
	31	143	Sandstone		113.4							
	36	200	Sandstone	18.6								
	41	150	Sandstone									
MN-32	2	450	Soil(SC)									
	5	100	Soil(SC)	36.4						34		
	10	128	Siltstone	19.8								
	15	128	Siltstone	17.0						57		
	23		Siltstone	18.0	108.5							
MN-33	3	75	Siltstone	13.0						66		
	5	75	Siltstone									
	10	100	Siltstone	22.0						53		
	15	145	Siltstone									
	23		Siltstone	17.6	110.5							
MN-34	4	57	Siltstone	16.2						60		
	5	225	Siltstone	17.0								
	10	90	Siltstone	15.0								
	14	170	Siltstone	11.3								

FIGURE D-5

SUMMARY OF TEST RESULTS

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Geotechnical Study for the Walnut Creek Water Treatment Plant Improvements Project

Walnut Creek, California

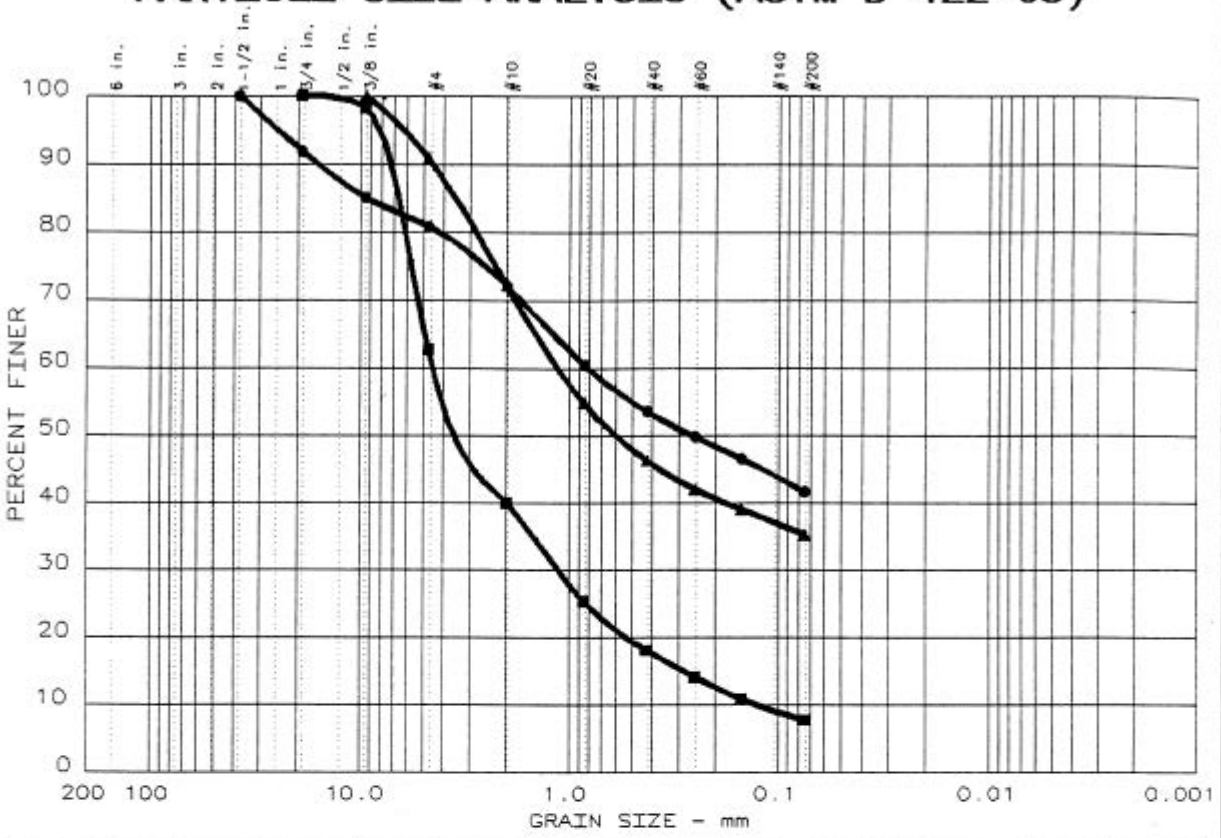
Prepared for:

Carollo Engineers
2700 Ygnacio Valley Road, Suite 300
Walnut Creek, California 94598

August 2001

Project No. 6967.001.0

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	19.2	39.2	41.6		SM		
▲	0.0	9.2	55.6	35.2		SM		
■	0.0	37.3	54.9	7.8		SW-SM		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0		
0.75	91.9		100.0
0.375	85.1	100.0	98.2
	GRAIN SIZE		
D_{60}	0.81	1.12	4.53
D_{30}			1.11
D_{10}			0.12
	COEFFICIENTS		
C_c			2.14
C_u			35.5

SIEVE number size	PERCENT FINER ●	▲	■
4	80.8	90.8	62.7
10	72.1	72.2	40.0
20	60.5	54.9	25.3
40	53.6	46.3	18.1
60	49.7	42.0	14.1
100	46.5	39.0	10.8
200	41.6	35.3	7.8

Sample information:

● MM1 ~~SPT~~-23&25 combined
Dark grey silty SAND w/gravel*

▲ MM2 SPT-7
Light grey silty SAND.

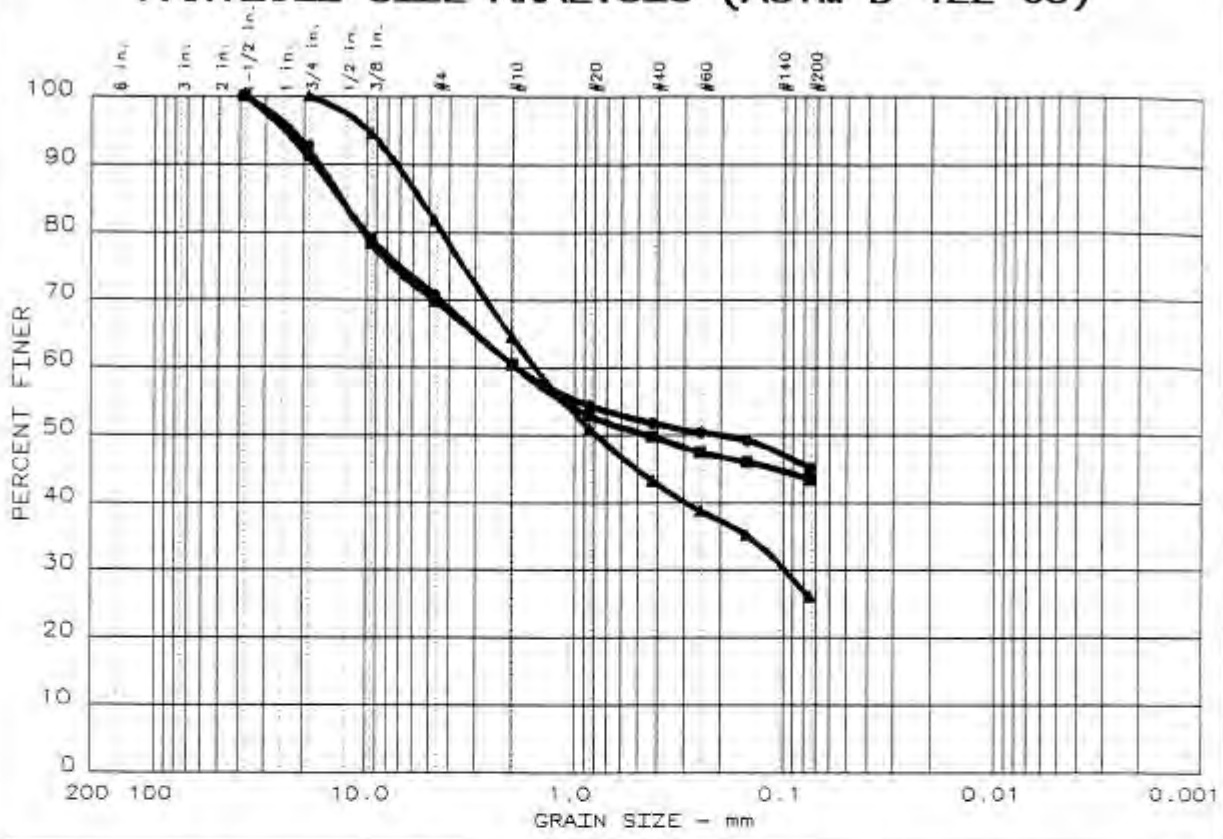
■ MM3 P-16 40-42.5'
V.dark grey f-c SAND w/gravel*

Remarks:
*Coarse agregates are friable & could break down to finer sizes during earth work.

Soil Mechanics Lab

Project No.: 6967.001
Project: Walnut Creek Clearwell
Date: 5-23-01 Data Sheet No. FIGURE 5-1

PARTICLE SIZE ANALYSIS (ASTM D 422-63)



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	LL	PI
●	0.0	29.3	25.6	45.1		GM		
▲	0.0	18.3	55.6	26.1		SM		
■	0.0	30.2	26.4	43.4		GM		

SIEVE inches size	PERCENT FINER ●	▲	■
1.5	100.0		100.0
0.75	91.2	100.0	92.5
0.375	79.1	94.4	78.4
GRAIN SIZE			
D_{60}	1.91	1.53	1.93
D_{30}		0.10	
D_{10}			
COEFFICIENTS			
C_c			
C_u			

SIEVE number size	PERCENT FINER ●	▲	■
4	70.7	81.7	69.8
10	60.4	64.5	60.4
20	54.4	50.9	52.8
40	51.7	43.3	49.7
60	50.4	38.8	47.4
100	49.2	35.1	46.0
200	45.1	26.1	43.4

Sample information:

● MM1 P-16 40-42.5' Lt. greyish brn.silty GRAVEL* w/sand

▲ MM1 P-21 52.5-55' Very dark grey silty SAND w/gravel*

■ MM1 P-23 57.5-60' V.dk.grey silty GRAVEL* w/sand.

Remarks:

*Coarse agregates are friable & could break down to finer sizes during earth work.

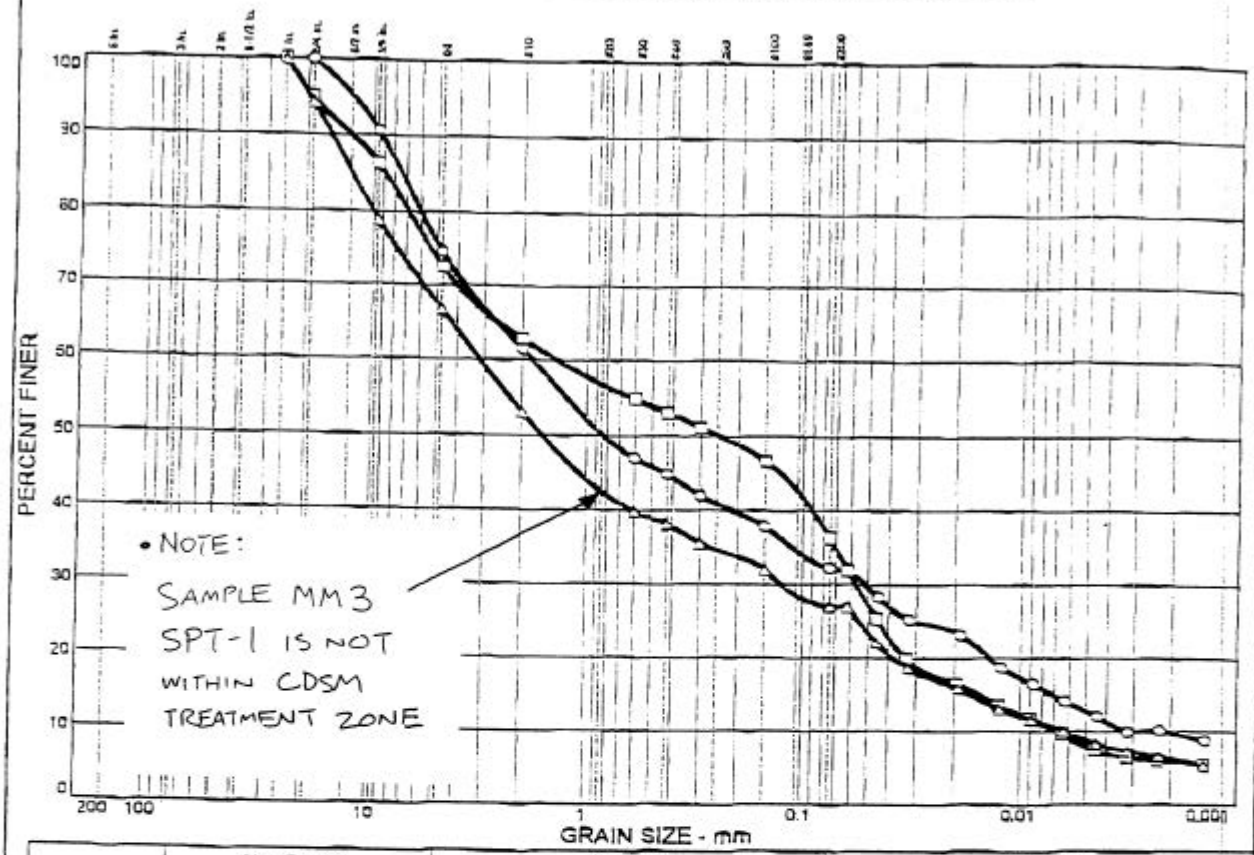
Soil Mechanics Lab

Project No.: 6967.001

Project: Walnut Creek Clearwell

Date: 5-23-01 Data Sheet No. FIGURE B.2

PARTICLE SIZE DISTRIBUTION CURVES



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	25.4	13.1	16.8	12.5	19.4	12.8
□	0.0	5.1	22.3	9.8	9.7	16.8	28.3	8.0
△	0.0	5.7	27.6	13.6	15.0	11.1	18.2	8.8

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			7.39	1.79	0.836	0.0514	0.0072	0.0022	0.67	814.27
□			8.82	1.38	0.246	0.0569	0.0155	0.0070	0.34	197.81
△			12.5	3.13	1.61	0.126	0.0169	0.0064	0.79	491.19

	MATERIAL DESCRIPTION	USCS	AASHTO
○	olive brown silty SAND w/gravel (weathered SILTSTONE)	SM	
□	olive brown silty SAND w/gravel (weathered SILTSTONE)	SM	
△	yellow brown mottled orange silty SAND w/gravel (weathered SILTSTONE)	SM	

Project No. 326-001 **Client:** EBMUD

Project: WCCW

Remarks:

○ **Source:** MM-2 **Sample No.:** SPT-7 **Elev./Depth:** 17.5'

□ **Source:** MM-2 **Sample No.:** SPT-9 **Elev./Depth:** 22.5'

△ **Source:** MM-3 **Sample No.:** SPT-1 **Elev./Depth:** 25'

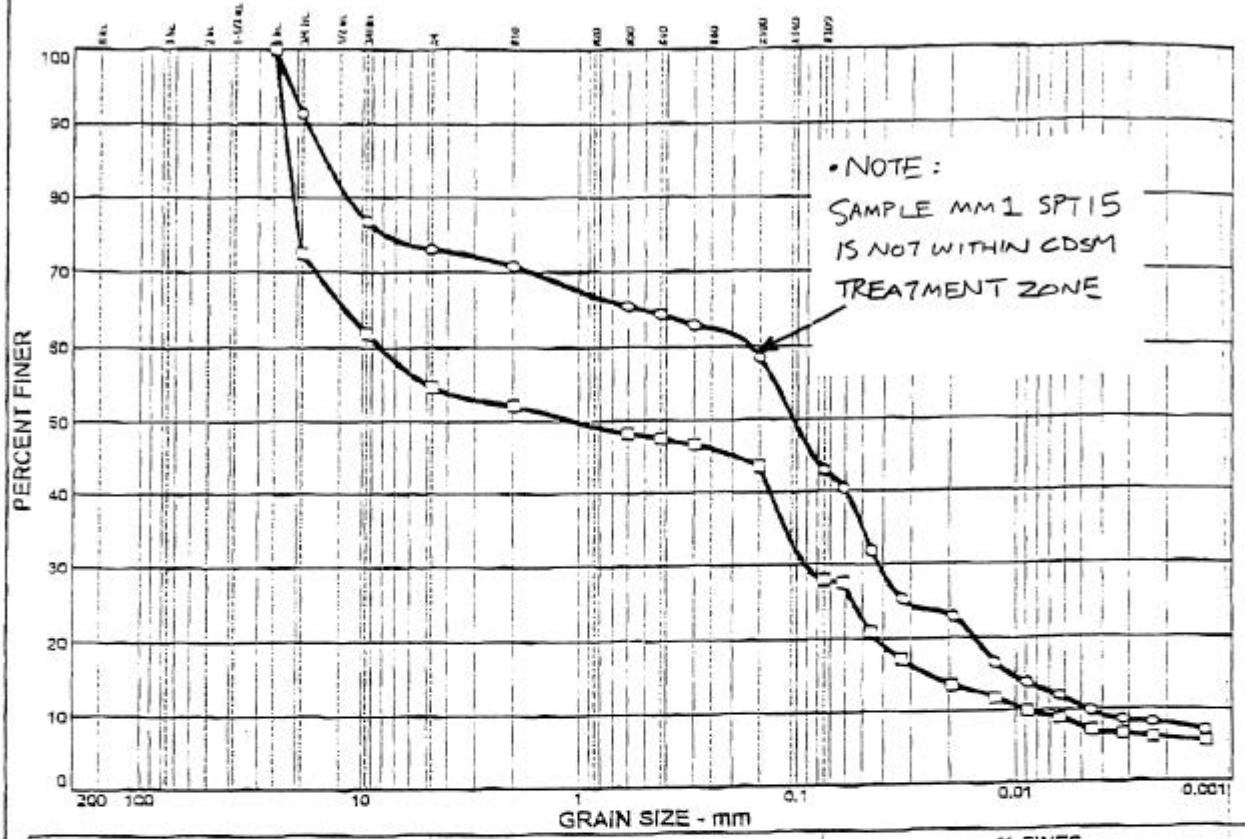
PARTICLE SIZE DISTRIBUTION CURVES

COOPER TESTING LABORATORY

FIGURE B-3

Plate

PARTICLE SIZE DISTRIBUTION CURVES



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	8.6	18.3	2.3	6.4	21.5	32.5	10.4
□	0.0	27.5	17.8	2.6	4.6	19.6	20.3	7.6

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			14.9	0.166	0.105	0.0423	0.0107	0.0047	2.30	35.32
□			22.0	8.23	1.06	0.0918	0.0250	0.0091	0.11	903.40

	MATERIAL DESCRIPTION	USCS	AASHTO
○	olive brown mixed w/dark brown silty SAND w/claystone	SM	
□	dark brown silty GRAVEL w/sand (sandstone gravel)	GM	

Project No. 326-001 **Client:** EBMUD
Project: WCCW

○ **Source:** MM-1 **Sample No.:** SPT-15 **Elev./Depth:** 37.5'
□ **Source:** MM-1 **Sample No.:** SPT-18 (to **Elev./Depth:** 42.5 or 46'

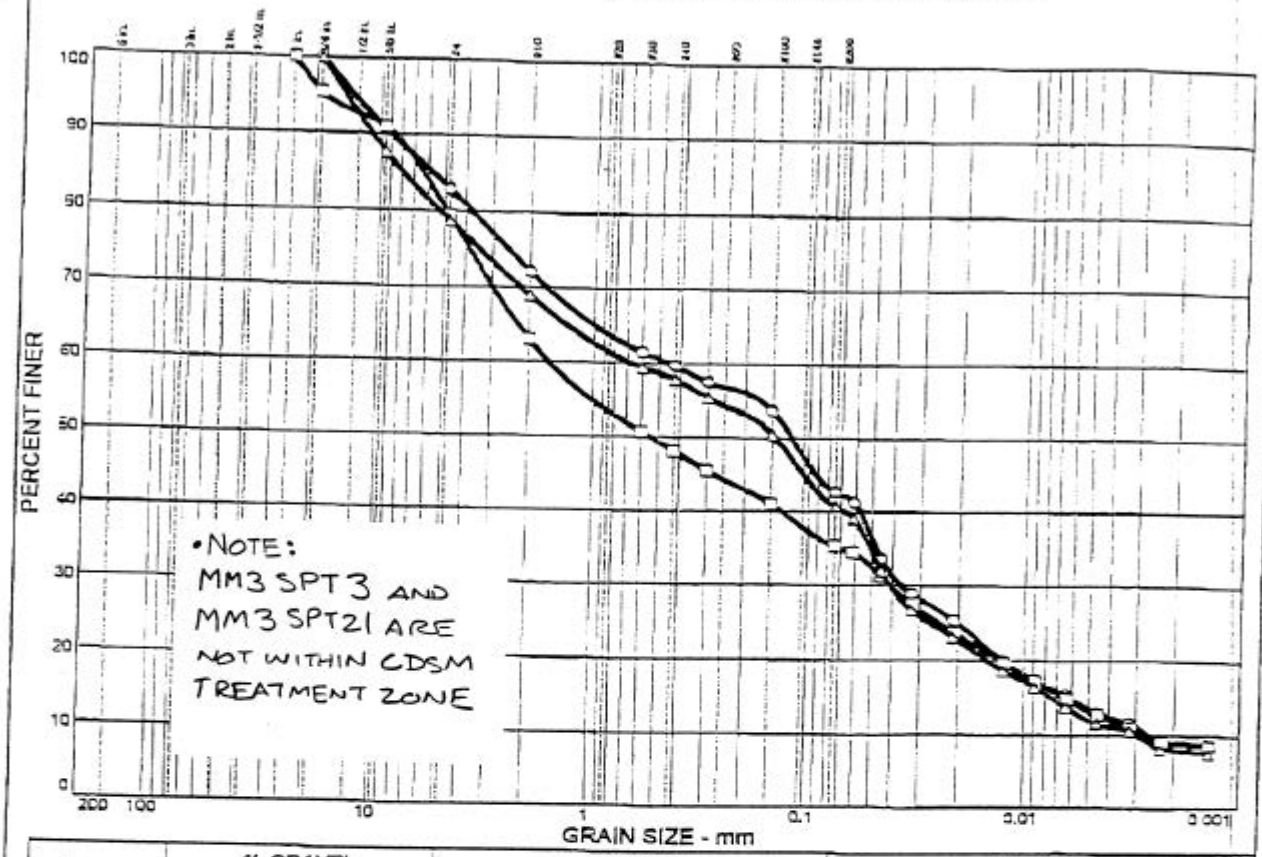
Remarks:
○
□

PARTICLE SIZE DISTRIBUTION CURVES

COOPER TESTING LABORATORY

FIGURE B-4
Plate

PARTICLE SIZE DISTRIBUTION CURVES



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	17.1	10.8	12.5	16.9	29.3	13.4
□	0.0	4.6	16.0	16.4	14.8	12.8	22.2	13.2
△	0.0	0.0	21.2	9.9	11.1	17.1	28.7	12.0

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	45	29	5.74	0.456	0.121	0.0362	0.0061	0.0026	1.09	172.47
□			6.31	1.62	0.544	0.0406	0.0064	0.0027	0.38	603.95
△			7.89	0.678	0.144	0.0410	0.0073	0.0029	0.84	230.31

	MATERIAL DESCRIPTION	USCS	AASHTO
○	olive brown silty SAND w/gravel (weathered SILTSTONE)	SM	
□	brown silty SAND w/gravel (weathered SILTSTONE)	SM	
△	olive brown silty SAND w/gravel	SM	

Project No. 326-001 **Client:** EBMUD
Project: WCCW

○ **Source:** MM-3 **Sample No.:** SPT-3 **Elev./Depth:** 7.5'
□ **Source:** MM-3 **Sample No.:** SPT-19 **Elev./Depth:** 47.5'
△ **Source:** MM-3 **Sample No.:** SPT-21 **Elev./Depth:** 52.5'

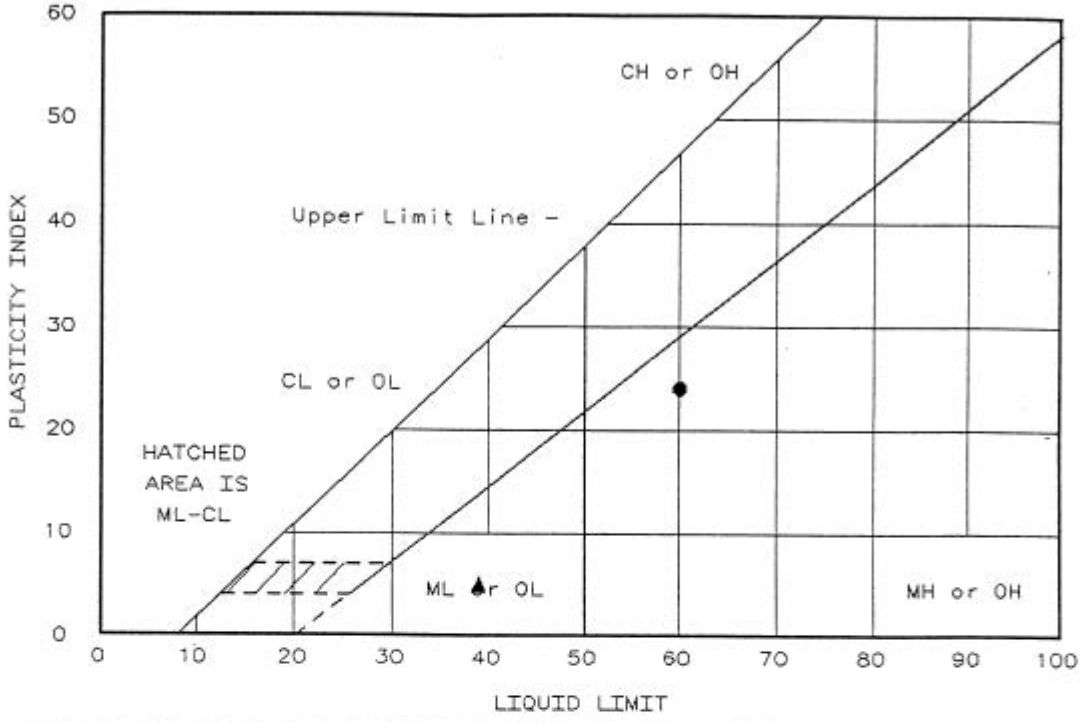
Remarks:
○
□
△

PARTICLE SIZE DISTRIBUTION CURVES

COOPER TESTING LABORATORY

FIGURE B-5
Plate

LIQUID AND PLASTIC LIMITS TEST REPORT



	Location + Description	LL	PL	PI	-200	ASTM D 2487-90
●	MM1/P-23 @ 57.5-60':Very dk. grey silty Gravel w/sand.	60	36	24	43.4	GM, Silty gravel with sand
▲	MM1 -23&25:Dark grey silty SAND w/gravel.	39	34	5	41.6	SM, Silty sand with gravel

Project No.: 6967.001
Project: Walnut Creek Clearwell

Client: Geomatrics Consultants,Inc.
Location:

Date: 5-23-01

LIQUID AND PLASTIC LIMITS TEST REPORT

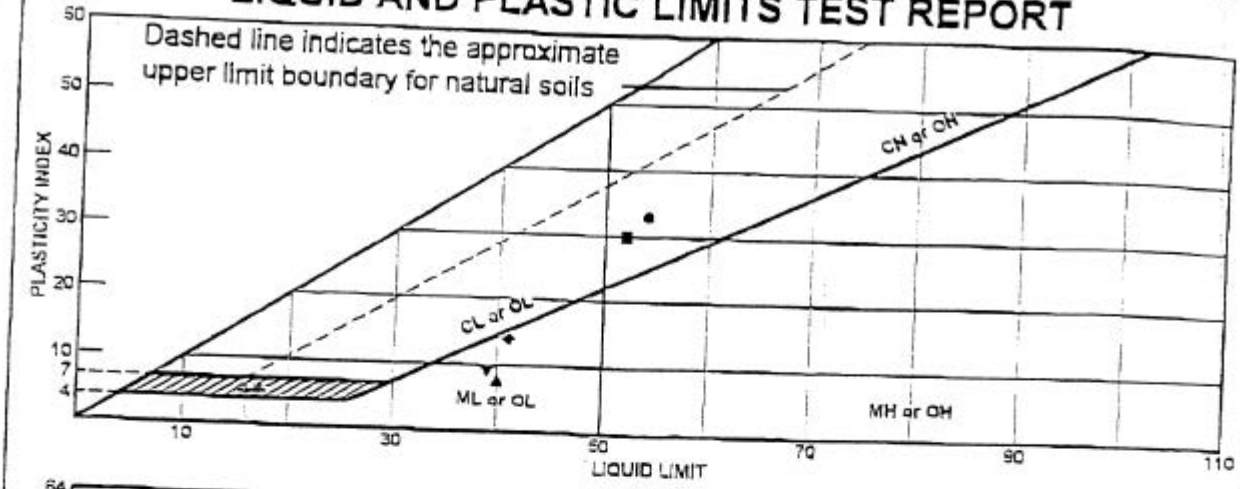
Soil Mechanics Lab

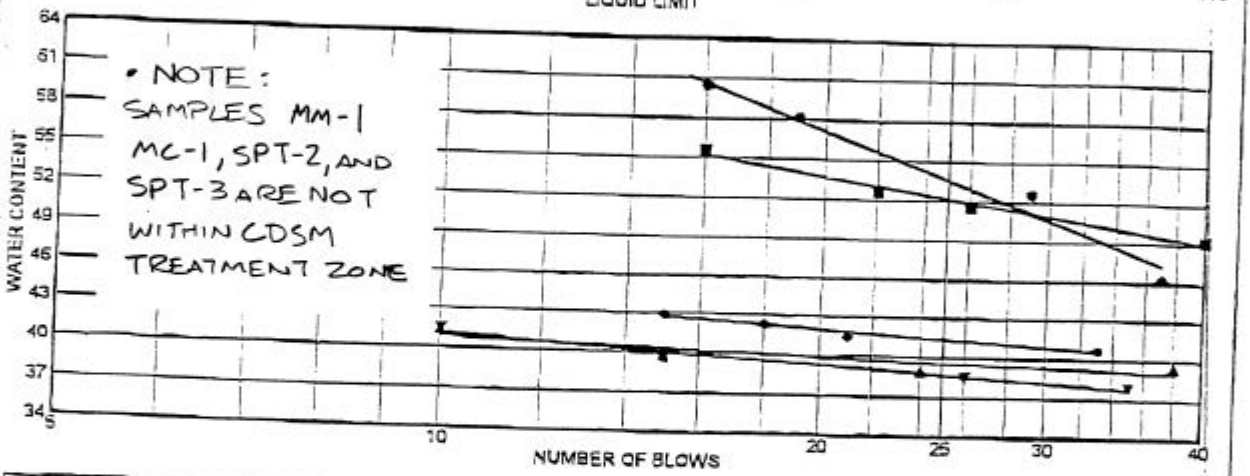
Remarks:

ASTM D 4318

Fig. No. B-6

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	olive brown mottled orange CLAY	54	21	33			
■	olive brown CLAY w/sand	52	22	30	84.9	74.4	CH
▲	olive brown SILT w/siltstone	40	32	8			
◆	brown sandy SILT w/siltstone	41	27	14			
▼	brown SILT w/siltstone	39	30	9			

Project No. 326-001 **Client:** EBMUD

Project: WCCW

- ● **Source:** MM-1 **Sample No.:** MC-1 **Elev./Depth:** 2'
- ■ **Source:** MM-1 **Sample No.:** SPT-2 **Elev./Depth:** 5'
- ▲ **Source:** MM-1 **Sample No.:** SPT-3 **Elev./Depth:** 7.5'
- ◆ **Source:** MM-1 **Sample No.:** SPT-17 (SPT-18 on sample)
- ▼ **Source:** MM-1 **Sample No.:** SPT-22 **Elev./Depth:** 55'

Remarks:

● ■ ▲ ◆ ▼

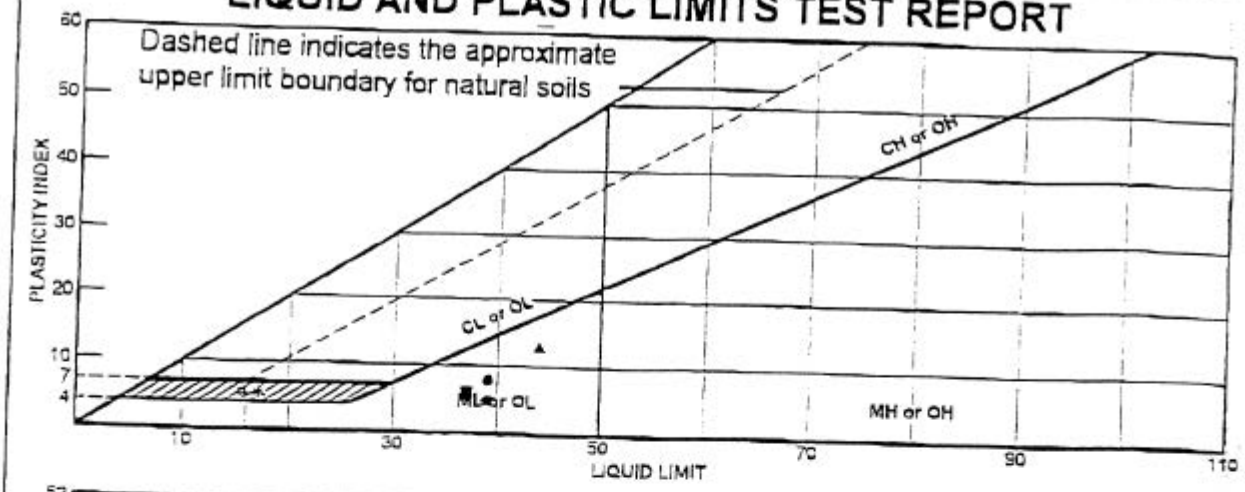
LIQUID AND PLASTIC LIMITS TEST REPORT

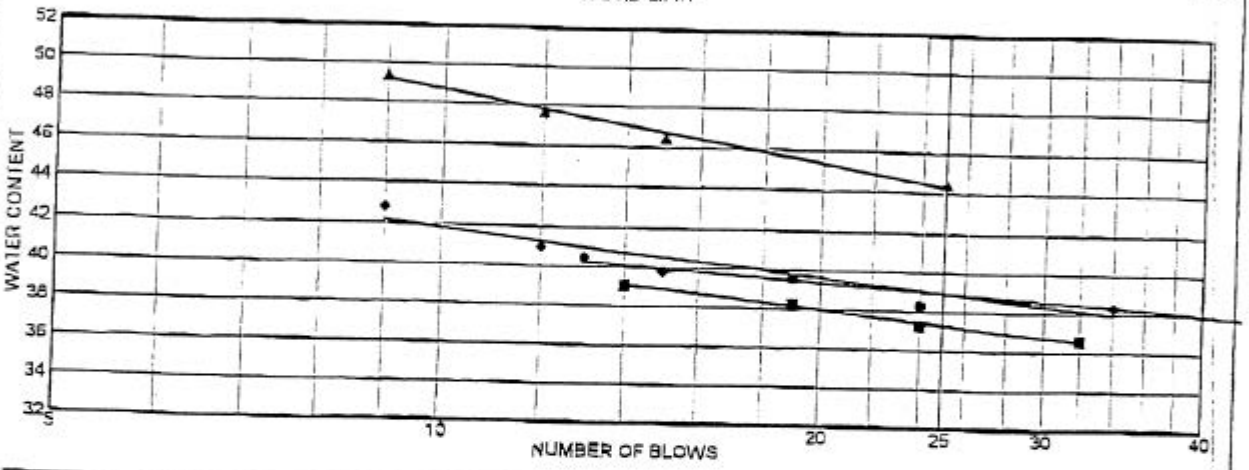
COOPER TESTING LABORATORY

FIGURE B-7

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	brown sandy CLAY w/claystone	39	31	8			
■	olive brown sandy SILT w/siltstone	37	31	6			
▲	brown silty SAND w/gravel (weathered SILTSTONE)	44	31	13	48.2	35.4	SM
◆	olive brown silty SAND w/gravel	39	34	5	57.8	40.7	SM

Project No. 326-001 **Client:** EBMUD

Project: WCCW

- ● **Source:** MM-3 **Sample No.:** SPT-17 **Elev./Depth:** 42.5'
- ■ **Source:** MM-3 **Sample No.:** SPT-18 **Elev./Depth:** 45'
- ▲ **Source:** MM-3 **Sample No.:** SPT-19 **Elev./Depth:** 47.5'
- ◆ **Source:** MM-3 **Sample No.:** SPT-21 **Elev./Depth:** 52.5'

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Remarks:

■

■

▲

●

•NOTE:
MM3 SPT-3 IS NOT WITHIN CDSM TREATMENT ZONE

FIGURE B-8

APPENDIX E

Geophysical Survey

APPENDIX E

GEOPHYSICAL SURVEY

Scope

Ninyo & Moore performed a geophysical survey at the site using seismic refraction and refraction microtremor (ReMi) techniques. The surveys were performed on September 8, 2021 and September 15, 2021 along five lines. The survey line locations are noted on Figure 2 of the report.

Refraction Microtremor Survey

The ReMi method was used to develop a one-dimensional profile of shear wave velocity and evaluate the characteristic shear wave velocity to a depth of 100 feet below the ground surface (V_s100) for seismic site classification. The survey consisted of collecting microtremor array measurements (MAM) from surface waves using ambient noise as a passive source and an array of geophones along one seismic line (SL-5). The following sections provide a summary of the methods and analyses used in the ReMi survey. The seismic model results are provided on Figure E-1.

Field Methods

A Geode 24–Channel Seismograph (Geometrics Inc., San Jose, California) was used for the MAM survey. 4.5 Hertz (Hz) vertical component geophone were placed every 10 feet for a total profile length of 230 feet. Approximately twenty-five to thirty records were collected, with a record length of 30 seconds (s) and a 2 millisecond (ms) sampling interval. The field data were digitally recorded in SEG2 format, reviewed in the field for data quality, saved to a hard disk, and documented.

Data Processing and Modeling

The MAM seismic data was processed using SeisImager (Geometrics Inc., San Jose, California) seismic processing software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (longer-period and lower-frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (shorter-period and higher-frequency) surface waves travel relatively shallow within the earth and thus contain more information about velocity closer to the surface. The dispersion is dependent on the material properties, such as surface wave velocity, relative material densities, and Poisson's ratio. An inversion is performed on the collected passive seismic shear wave records within SeisImager to produce a model of the variation in shear wave velocities with depth. The resulting data processing flow was used to calculate the average shear-wave velocity (AVS) to a depth of approximately 100 feet (Vs100) as the depth-interval-weighted, harmonic mean shear wave velocity.

- Collated records into list file and edited any bad channels or records,
- Applied 2D Spatial Auto Correlation (SPAC); using a linear array and 24 geophones at 10 feet spacing,
- Phase velocity frequency transformation from 2 to 25 Hz
- Automated velocity picks of raw phase velocity were calculated and updated manually,
- Created an initial model and carried out a non-linear Least Squares Method (LSM) inversion to produce a final shear wave velocity model; convergence of the inversion was judged whether the model achieved an RMS <5% within 5-7 iterations,
- Calculated V_s100 using final shear wave velocity model.

Results

Shear wave data resolution generally decreases with depth, due to the loss of sensitivity of the dispersion curve to changes in shear wave velocity as depth increases. The MAM seismic modeling results are provided on Figure E-1. The scaled figures indicate our interpretation of the approximate changes in shear wave velocity with depth at the surveyed location.

The characteristic shear wave velocity (Vs100) calculated from the model is approximately 1,136 feet per second which is consistent with a Class D seismic site classification for stiff soil (ASCE, 2016).

Seismic Refraction

Seismic refraction was used to develop two-dimensional profiles of seismic compressional (P-wave) velocity and evaluate the excavation characteristics or rippability of the earth materials along the profiles. The survey consisted of measuring the travel time of seismic energy from an active source with an array of geophones placed on the ground surface along five seismic lines (SL-1 through SL-5). The following sections provide a summary of the methods and analyses used in the survey and rippability assessment.

Field Methods

Ninyo and Moore personnel collected refraction data along five seismic lines denoted as SL-1, SL-2, SL3, SL-4, and SL-5. Lines SL-1, SL-4 and SL-5 were approximately 230 feet long with geophones spaced at 10-foot intervals. Lines SL-2 and SL-3 were approximately 115 feet long with geophones spaced at 5-foot intervals. The seismic data were collected using a 24-Channel Geometrics Geode, exploration seismograph coupled with 24 vertical components, and 10 Hertz geophones. A 10-pound hammer and aluminum plate were used as the seismic wave source for the refraction surveys. Field data acquisition included stacking multiple shots at each shot location in order to increase the quality of the data and reduce noise. The seismic refraction method uses recognition of first-arrival times of refracted seismic waves in units of milliseconds to evaluate the thickness and seismic velocities of subsurface layers. Seismic waves generated by the hammer impacting the ground surface at a given “shot” point are refracted at boundaries separating materials of contrasting material velocities. These refracted seismic waves are then detected by a series of surface geophones and recorded with a seismograph. Each hammer shot is recorded as time zero. The elapsed time, in milliseconds, that the seismic compressional wave (P-wave) signals take to travel to each geophone is recorded through the record length. This information is used in conjunction with the known shot-to-geophone horizontal distances to obtain the approximate thickness and velocity information about the subsurface materials.

Seismic Velocities and Rippability Correlations

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass for each detected layer. Possible areas of differing composition, texture, or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator. Table E-1 outlines the relationship between P-wave velocity and rippability.

The anticipated degree of rippability corresponding to the characteristic P-wave velocity of the material to be excavated by a Caterpillar D-9 dozer ripping with a single shank is listed in Table E-1. The limits of the velocity ranges corresponding to the various rippability categories in the table are approximate limits. Characteristics other than the material density and hardness corresponding to the seismic P-wave velocity, such as the spacing and orientation of fractures, joints, bedding planes or other discontinuities can influence rippability and rates of excavation. The seismic refraction method provides a measure of the average seismic wave velocity over a distance that is limited by the spacing of the

geophones. Wide variations in the degree of weathering or cementation over distances shorter than the geophone spacing or the inclusion of boulders or other obstructions, which may impact the rate of excavation, will not be detected by the seismic refraction survey. Hydraulic hammers may be used, as an alternative to blasting, for primary and secondary breaking of bedrock where rippability is difficult or not feasible.

Table E-1 – P-Wave Velocity and Rippability Correlation

P-Wave Velocity Range	Rippability
0 to 2000 feet/second	Easy Ripping
2000 to 4000 feet/second	Moderate Ripping
4000 to 5500 feet/second	Difficult Ripping, Possible Blasting
5500 to 7000 feet/second	Very Difficult Ripping, Probable Blasting
Greater than 7000 feet/second	Blasting Generally Needed

The rippability correlations presented in Table E-1, which are based on observations by Ninyo & Moore during excavation of similar materials, are slightly more conservative than those published in the Caterpillar Performance Handbook. Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids. It should also be noted that, as a general rule of thumb, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the refraction line. The approximate effective depths of the seismic lines ranged from 20 to 40 feet below ground surface.

Data Analysis

The obtained refraction data were processed using SeisImager/2D™, Version 3.3, which includes a suite of programs by Geometrics, Inc. Initially, data were grouped as all shot points in each line and first arrival picks were made manually in Pickwin v. 5.1.1.2. Once data were grouped as first arrival picks, a travel time curve was constructed and calculations were performed to derive approximate minimum and maximum velocities. Relative topography data collected in the field were incorporated with travel time data to account for topography effects. An initial velocity model of the surface was developed for each profile using a delay-time technique and velocity inversion in Plotrefa ™ v. 3.0.0.6. These models were used for each profile to develop a more detailed tomographic profile depicting approximate lateral and vertical changes in P-wave velocity across each seismic line.

Results

The tomographic models developed for this study indicate that earth materials within the effective depth of the profile generally consist of stiff soil, very dense soil, and soft rock. P-wave velocity profiles for the seismic lines SL-1, SL-3, SL-4, and SL-5 are presented on Figures E-2 through E-5. The quality of seismic refraction data is influenced by background noise, irregular terrain, and lateral inhomogeneity which can obscure the signal, introduce scatter, and limit the accuracy of interpretations. The signal data collected from seismic line SL-2 was not resolvable from background noise.

The refraction method generally necessitates that subsurface velocities (and therefore material densities and modulus of rigidity) increase with depth. A layer having a velocity lower than that of the layer above it will not be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequently deeper layers. This is known as a “velocity inversion” problem. In addition, relatively significant lateral variations in velocity, such as those which occur at open, or tightly spaced jointed or fractured rock, discontinuous caliche deposits, and nested subsurface cobbles and

boulders, can also result in misinterpretation of the subsurface conditions when using this method.

Refraction P-wave velocity data were used to assess rippability to depths of up to 40 feet below the ground surface. The anticipated rippability corresponding to the interpreted P-Wave velocity for selected depth intervals is summarized in Table E-2. Hydraulic hammers may be used, as an alternative to blasting, for primary and secondary breaking of bedrock where rippability is difficult or not feasible.

Table E-2 – Summary of Refraction (P) Wave Velocity Survey Results

Location	Depth (feet)	Approximate P-wave Velocity Range (feet/sec)	Comments
SL-1	0 - 5	950-1,500	Easy Ripping
	5 -10	1,500-2,700	Easy to Moderate Ripping
	10 – 20	2,700-4,000	Moderate to Difficult Ripping, Possible Blasting
SL-3	0 - 5	1,000-1,700	Easy Ripping
	5 -10	1,700-3,000	Easy to Moderate Ripping
	10 – 20	3,000-3,500	Moderate Ripping
SL-4	0 - 5	950-1,350	Easy Ripping
	5 -10	1,350-2,200	Easy to Moderate Ripping
	10 – 20	2,200-3,000	Moderate Ripping
	20-40	3,000-4,200	Moderate to Difficult Ripping, Possible Blasting
SL-5	0 - 5	950-1,350	Easy Ripping
	5 -10	1,350-2,200	Easy to Moderate Ripping
	10 – 20	2,200-3,000	Moderate Ripping
	20-40	3,000-4,200	Moderate to Difficult Ripping, Possible Blasting

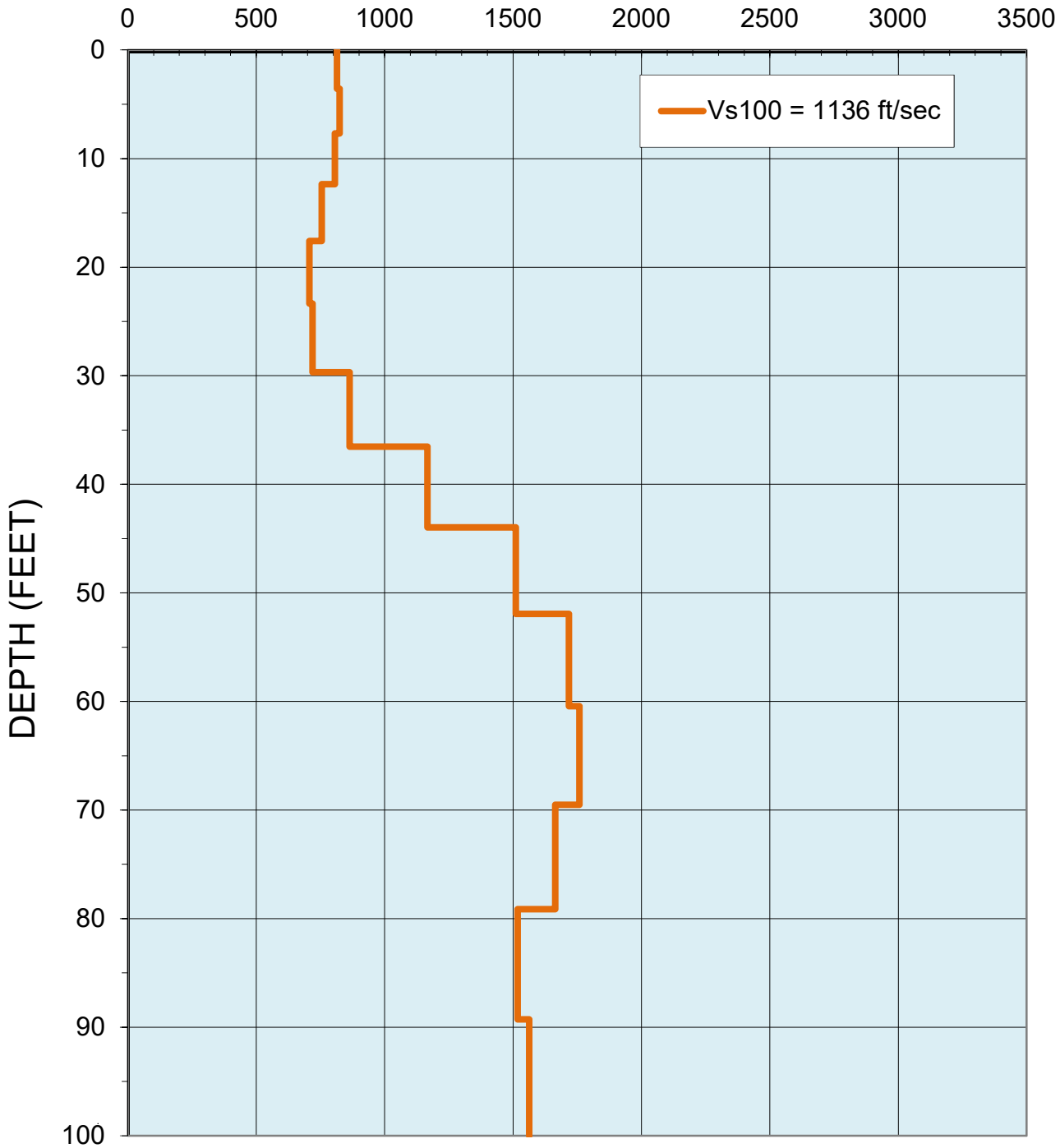


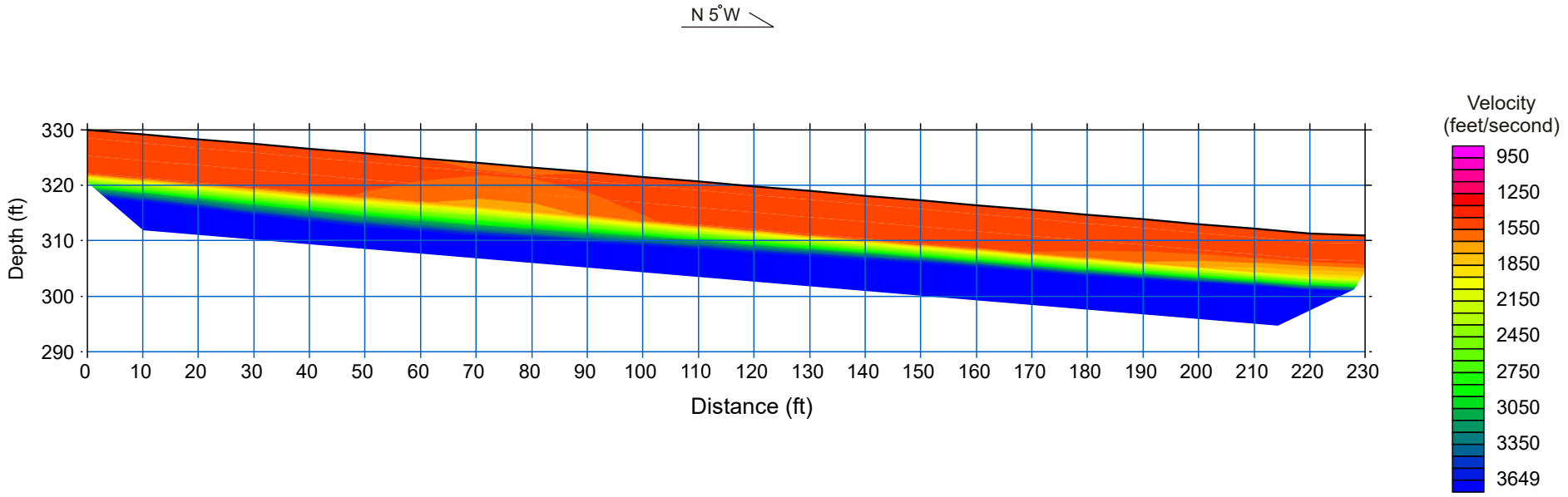
FIGURE E-1

LINE 5 SHEAR WAVE VELOCITY PROFILE

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE, WALNUT CREEK, CALIFORNIA

403982001 | 06/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTE: ELEVATIONS AND LINE ORIENTATION ARE APPROXIMATE.

FIGURE E-2

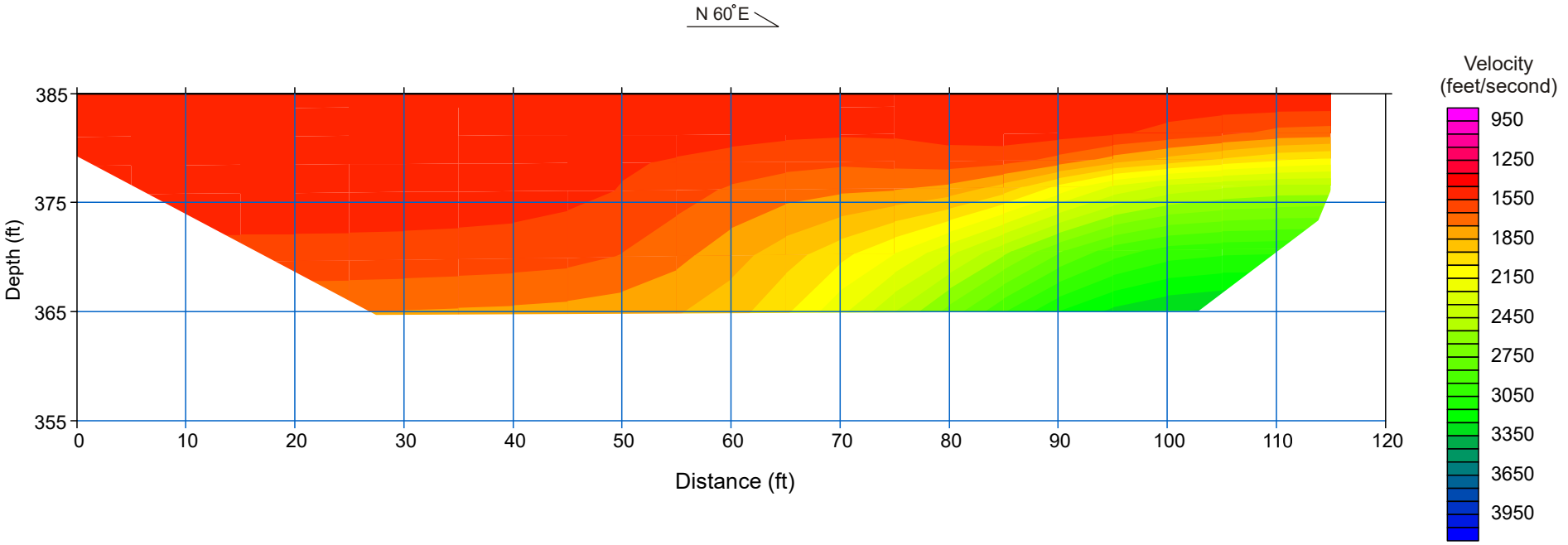
SEISMIC REFRACTION LINE 1

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA

403982001 | 6/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

bsm file no: 403982001sei1021a



NOTE: ELEVATIONS AND LINE ORIENTATION ARE APPROXIMATE.

FIGURE E-3

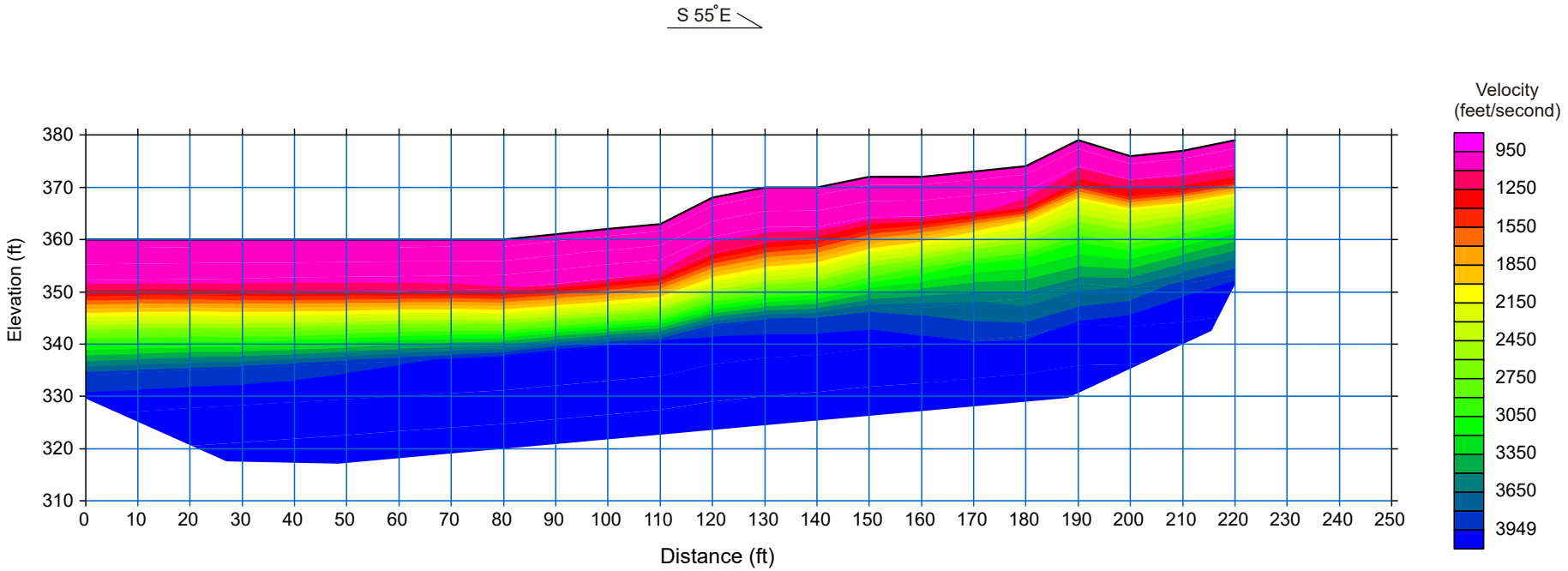
SEISMIC REFRACTION LINE 3

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA

403982001 | 6/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

bsm file no: 403982001sei1021d



NOTE: ELEVATIONS AND LINE ORIENTATION ARE APPROXIMATE.

FIGURE E-4

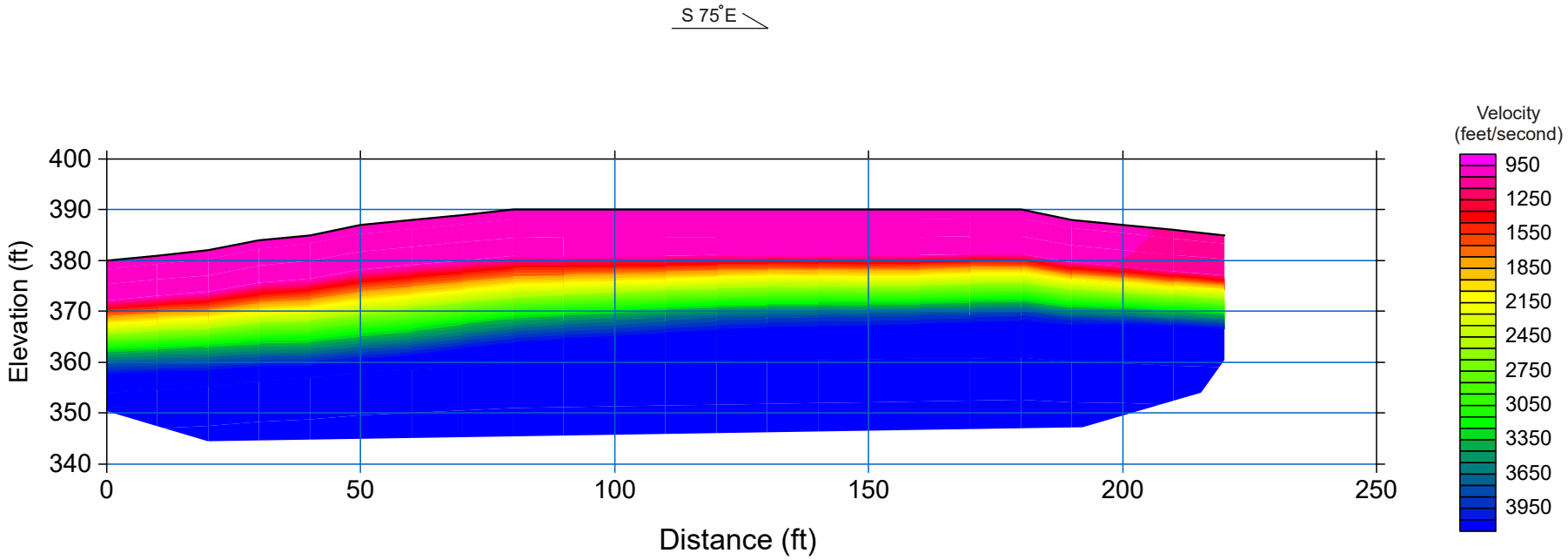
SEISMIC REFRACTION LINE 4

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA

403982001 | 6/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

bsm file no: 403982001sei1021b



NOTE: ELEVATIONS AND LINE ORIENTATION ARE APPROXIMATE.

FIGURE E-5

SEISMIC REFRACTION LINE 5

WALNUT CREEK WATER TREATMENT PLANT
2201 LARKEY LANE
WALNUT CREEK, CALIFORNIA

403982001 | 6/22

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

bsm file no: 403982001sei1021c

APPENDIX F

Geophysical Surveys from Previous Investigations

DOWNHOLE SEISMIC SURVEY

Walnut Creek Clearwell Seismic Evaluation Project

Walnut Creek, California

for

EAST BAY MUNICIPAL UTILITY DISTRICT

P.O. Box 24055

Oakland, California 94623

by

Portola Geophysics

900 Wayside Road

Portola Valley, California 94028

December 1997

Portola Geophysics
900 Wayside Road, Portola Valley, CA 94028 Phone & FAX (650) 851-5342

December 8, 1997
9707

Mr. Derek Kawaii, Project Engineer
Materials Engineering
EAST BAY MUNICIPAL UTILITY DISTRICT
P.O. Box 24055
Oakland, California 94623

SUBJECT: **Downhole Seismic Survey**
RE: Walnut Creek Clearwell Seismic Evaluation Project
Walnut Creek, California

Dear Mr. Kawaii:

The following report describes the findings and conclusions of our downhole seismic geophysical investigation of the East Bay Municipal Utility District's Walnut Creek Clearwell Seismic Evaluation Project in Walnut Creek, California. This geophysical investigation was conducted in accordance with our proposal to you dated October 31, 1997.

We appreciate the opportunity to have been of service to you on this project. If you have any questions regarding this report, any aspect of our investigation, or if you need additional services, please contact our office.

Very truly yours,

PORTOLA GEOPHYSICS

Patrick O. Shires
Principal Geophysicist
RGP 879

POS:st

DOWNHOLE SEISMIC SURVEY
Walnut Creek Clearwell Seismic Evaluation Project
Walnut Creek, California

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	2.2 Interpreted Results	2
3.0	INVESTIGATION LIMITATIONS	4
4.0	SUMMARY OF CONCLUSIONS	4

TABLES

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2	Velocity/Density Calculations, Boring MM-3	3

FIGURES

1 Downhole Seismic Results, Boring MM-1
2 Downhole Seismic Results, Boring MM-3

1.0 INTRODUCTION

This report describes the results of a downhole seismic survey conducted for the East Bay Municipal Utility District's Walnut Creek Clearwell Seismic Evaluation Project in Walnut Creek, California. This investigation was conducted in accordance with our proposal to you dated October 31, 1997, and consists of a downhole seismic (compression and shear wave) survey of two borings drilled to the north and west of the Walnut Creek Clearwell.

2.0 DOWNHOLE SEISMIC SURVEY

Downhole seismic (compression and shear wave) surveys were conducted in two borings (Boring MM-1 and Boring MM-3) cased to depths of approximately 55 and 65 feet, respectively, with downhole geophone refusal at depths of 54 and 64 feet, respectively.

2.1 Method and Equipment Used

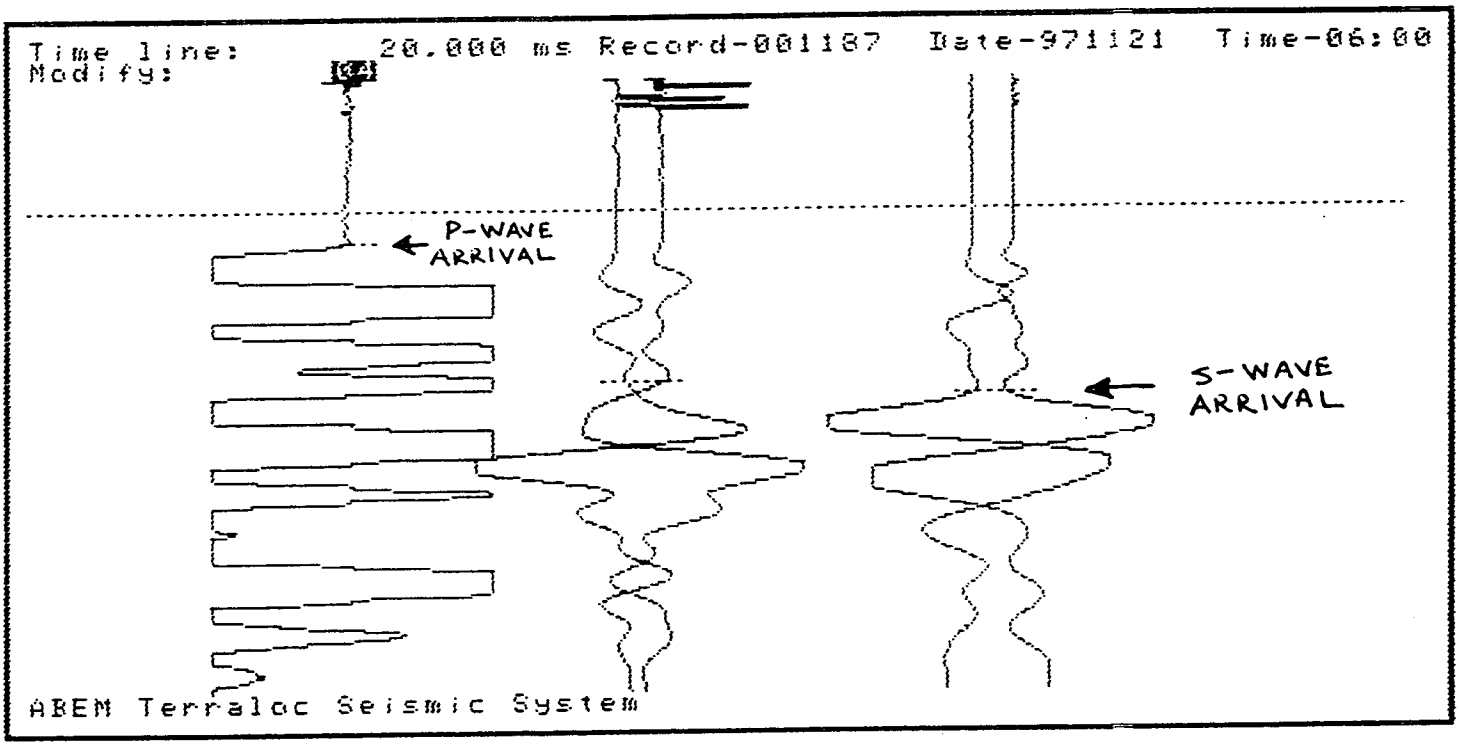
The downhole seismic survey procedure used for all recordings consisted of placing a triaxial geophone bundle at 5-foot depth intervals (except where restricted to a shorter interval at the bottom of the casings) within the cased (3.0-inch inside diameter PVC) and grouted borings.

The geophone bundle was clamped against the inside of the casing at successive 5-foot depth intervals by inflation of a pneumatic bladder. An 8.0-foot long wooden plank was held in place by the weight of the 4-wheel drive field vehicle at a distance of about 10 feet from the borings. A 16-pound sledge hammer equipped with seismograph/oscillograph triggering switch was impacted horizontally on a steel plate placed against both ends of the wooden plank to produce successive shear waves of reversed polarity. The hammer was also impacted vertically on the steel plate placed flat on the ground surface (also at about 10 feet from the boring) to produce compression waves. The hammer impacts generated shear and compression waves which travelled from the ground surface through the subsurface materials and were received by the deployed geophone bundle at successive depth intervals. The geophones were monitored digitally and displayed graphically in analog form on a portable 24-channel ABEM™ Terraloc Mark 3 Seismic System. Records were stored on magnetic disk and returned to the office for processing, printing, data reduction and interpretation.

Compression and shear wave velocity results and assumed bulk densities were used to calculate engineering parameters by spreadsheet.

The equipment used for the downhole seismic survey consisted of one triaxial geophone bundle of 4.5 Hz natural frequency with 300 feet of cabling and a pneumatic bladder (with 300 feet of supply tube). The geophone bundle was connected to a 24-channel ABEM™ Terraloc Mark 3 Seismic System using cabling with Mueller clips. The sweep traces for each horizontal (transverse and longitudinal) geophone were juxtaposed on the records so that successive shear waves of reversed polarity would make the first arrivals stand out (see representative record print-out below).

SAMPLE RECORD, MM-3 @ 60'



2.2 Interpreted Results

The downhole seismic recordings were conducted borings MM-1 and MM-3 drilled through fill embankments to the north and west of the Walnut Creek Clearwell, respectively. These holes were drilled vertically slightly offset to the north and west of the center of the embankment crest.

The borings were cased to depths of about 55 and 65 feet, respectively, and were downhole seismic surveyed to depths of 54 and 64 feet below ground surface (top of pavement just above top of casing), respectively. Survey results are presented on Figure 1 for Boring MM-1, and Figure 2 for Boring MM-3, in terms of time-distance plots of shear and compression wave arrivals (and velocities). The plots reflect actual distance from the offset energy source (sledge hammer on plate or plank) to the receiving geophone bundle. In order to correlate this information with boring logs, the assumption is made that velocities for the longer angular distances are representative for the velocities within the shorter vertical depth intervals surveyed. Tables 1 and 2, below, list shear and compression wave velocities as well as velocity/density calculations as a function of vertical depth interval for borings MM-1 and MM-3, respectively.

TABLE 1
Velocity/Density Calculations, Boring MM-1

Depth Interval (feet)	Compression Velocity (ft./sec.)	Shear Velocity (ft./sec.)	Bulk Density (pcf)	Poisson's Ratio (μ)	Compression Modulus (psf)	Shear Modulus (psf)
0.0-20.0	2,080	1,040	115	0.33	1.04E+07	3.89E+06
20.0-25.0	2,800	1,800	120	0.15	2.79E+07	1.22E+07
25.0-35.0	2,130	1,230	120	0.25	1.42E+07	5.67E+06
35.0-40.0	4,800	860	120	0.48	8.23E+06	2.77E+06
40.0-54.0	2,100	1,240	120	0.23	1.42E+07	5.77E+06

TABLE 2
Velocity/Density Calculations, Boring MM-3

Depth Interval (feet)	Compression Velocity (ft./sec.)	Shear Velocity (ft./sec.)	Bulk Density (pcf)	Poisson's Ratio (μ)	Compression Modulus (psf)	Shear Modulus (psf)
0.0-5.0	3,100	1,570	115	0.33	2.35E+07	8.86E+06
5.0-15.0	1,450	770	115	0.30	5.56E+06	2.13E+06
15.0-30.0	2,810	1,800	120	0.15	2.80E+07	1.22E+07
30.0-40.0	2,050	960	120	0.36	9.40E+06	3.46E+06
40.0-50.0	5,400	1,380	120	0.47	2.09E+07	7.14E+06
50.0-64.0	4,900	2,010	125	0.40	4.42E+07	1.58E+07

Using measured shear and compression wave velocities and assumed bulk densities, velocity/density calculations were made with the following formulae for Poisson's ratio (μ), compression (Young's) modulus (E) and

shear modulus (G), where √=bulk density, Vp=compression wave velocity, and Vs=shear wave velocity:

=[Vp/Vs) 2-2]/[2(Vp/Vs)2-2], E=2Vs2(√/32)(1+), and G=Vs2(√/32).

3.0 INVESTIGATION LIMITATIONS

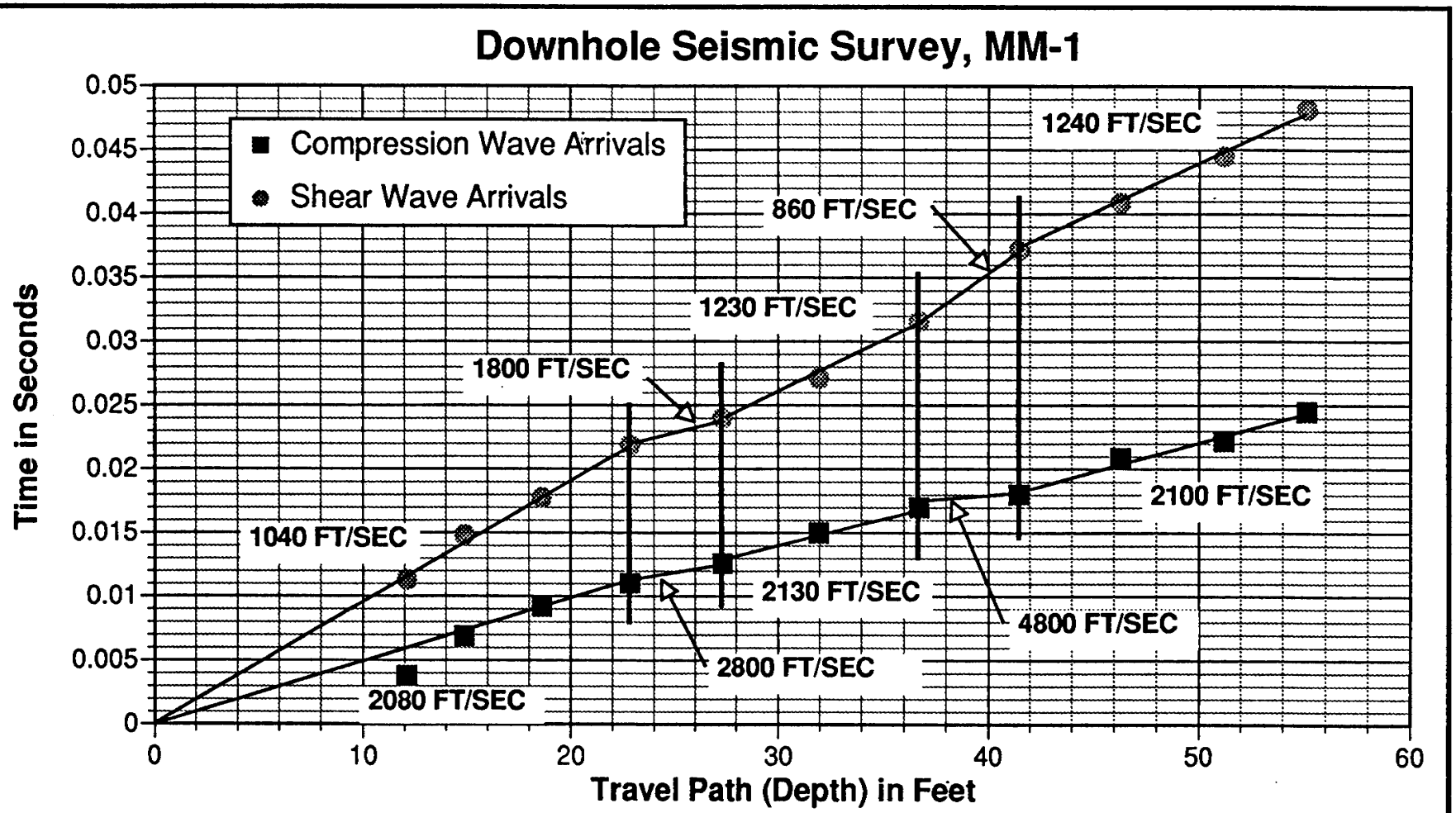
The downhole seismic results presented in this report represent the most reasonable interpretation of geophysical survey data based on our limited knowledge of the existing geologic conditions at the site. The results were not correlated with the logs for nearby exploratory borings, a correlation which, if completed, might affect interpretation of data. In addition, calculations were made using assumed densities for earth materials. Use of actual laboratory test data might also affect results. The results are presented for design information only and are not intended to serve as information for determining construction procedures. Interpretations were made in accordance with generally accepted geophysical methods and practices. This warranty is in lieu of all other warranties, express or implied.

4.0 SUMMARY OF CONCLUSIONS

The embankment surveyed is composed of artificial fill materials underlain by sandstone, siltstone and claystone bedrock. Compression and shear wave velocities are generally low to medium (1,450 to 5,400 ft/sec for compression waves and 770 to 2,010 ft/sec for shear waves) and consistant with what would be expected for compacted fill and weathered bedrock materials. Velocities vary within the fill and are generally higher in weathered bedrock and the data does not indicate that particularly loose soils are present.

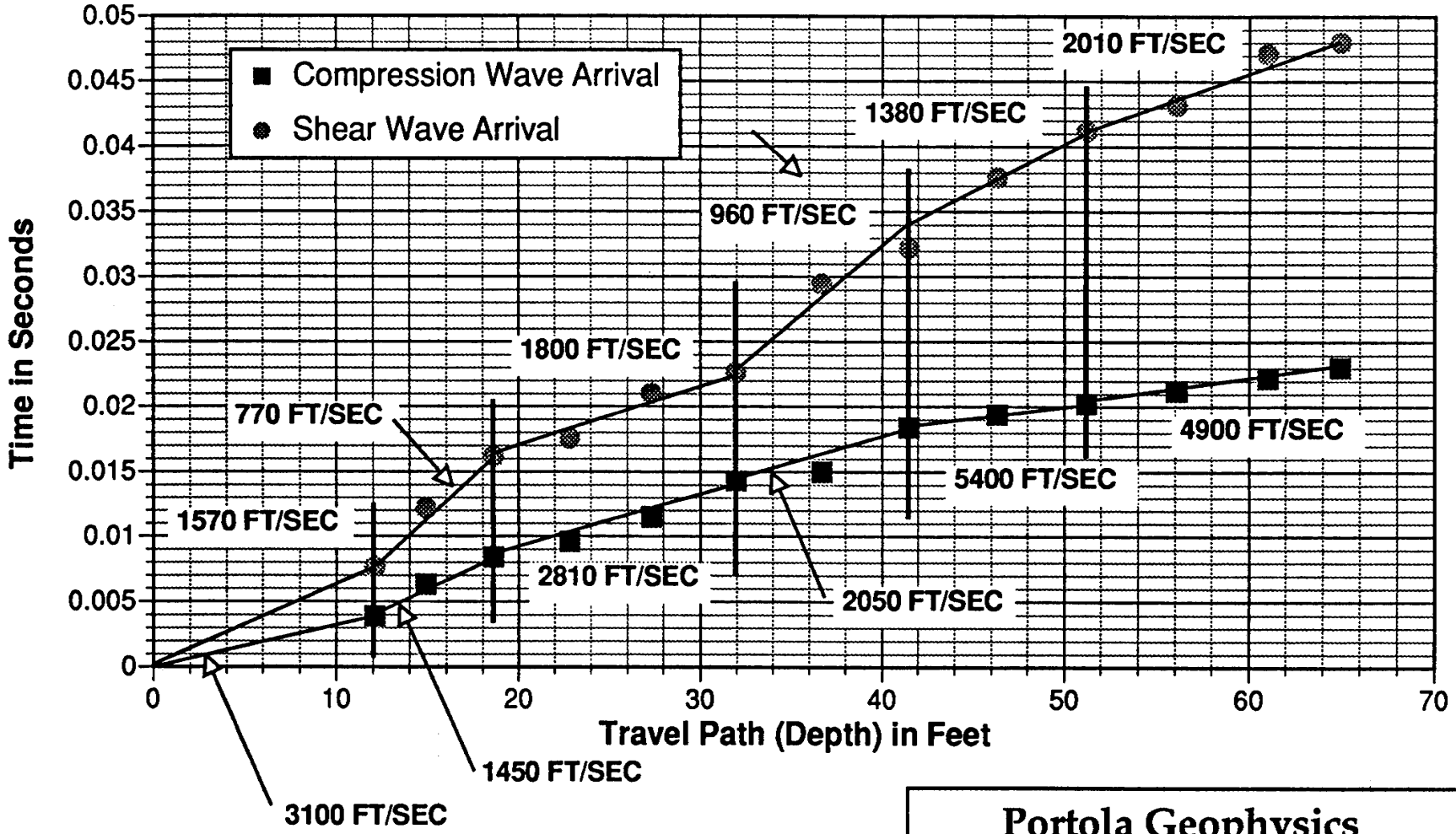
FIGURES

1 Downhole Seismic Results, Boring MM-1
2 Downhole Seismic Results, Boring MM-3



Portola Geophysics		
Downhole Seismic Results, Boring MM-1 Walnut Creek Clearwell Project Walnut Creek, California		
GEOPHYS. BY POS	SCALE AS SHOWN	PROJECT NO. 9707
APPROVED BY POS	DATE 11/21/97	FIGURE NO. 1

Downhole Seismic Survey, MM-3



Portola Geophysics		
Downhole Seismic Results, Boring MM-3 Walnut Creek Clearwell Project Walnut Creek, California		
GEOPHYS. BY POS	SCALE AS SHOWN	PROJECT NO. 9707
APPROVED BY POS	DATE 11/21/97	FIGURE NO. 2

SEISMIC REFRACTION SURVEY
EBMUD Walnut Creek Filter Plant Expansion
Walnut Creek, California

for

EAST BAY MUNICIPAL UTILITY DISTRICT
375 11th Street, MS-610
Oakland, California 94607

by

Portola Geophysics
900 Wayside Road
Portola Valley, California 94028

April 1998

Portola Geophysics
900 Wayside Road, Portola Valley, CA 94028 Phone & FAX (650) 851-5342

April 22, 1998
9802

Mr. Derek Kawaii, Project Engineer
Materials Engineering
EAST BAY MUNICIPAL UTILITY DISTRICT
P.O. Box 24055
Oakland, California 94623

SUBJECT: **Seismic Refraction Survey**
RE: EBMUD Walnut Creek Filter Plant Expansion Project
P.O. No. 562-57464-AX
Walnut Creek, California

Dear Mr. Kawaii:

The following report describes the findings and conclusions of our surface seismic refraction survey geophysical investigation of the EBMUD Walnut Creek Filter Plant Expansion Project in Walnut Creek, Contra Costa County, California. This geophysical investigation was conducted in accordance with our proposal to you dated March 6, 1998.

We appreciate the opportunity to have been of service to you on this project. If you have any questions regarding this report, any aspect of our investigation, or if you need additional services, please contact our office.

Very truly yours,

PORTOLA GEOPHYSICS

Patrick O. Shires
Principal Geophysicist
RGP 879

POS:st

SEISMIC REFRACTION SURVEY

EBMUD Walnut Creek Filter Plant Expansion

Contra Costa County, California

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7 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line Nos. S-9 and S-10
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1.0 INTRODUCTION

This report describes the methods and equipment used, interpreted results, limitations and conclusions of a seismic refraction geophysical survey of the East Bay Municipal Utility District (EBMUD) Walnut Creek Filter Plant Expansion Project site located in the Walnut Creek area of Contra Costa County, California. This investigation was conducted for EBMUD in accordance with our proposal dated March 6, 1998.

1.1 Project Description

The proposed project consists of expansion of the existing Walnut Creek Filter Plant by construction of two, new 236-foot diameter steel tank water reservoirs on a north-facing, moderately steep slope and a new chemical storage structure closer to the administration facilities of the plant in the Walnut Creek area of Contra Costa County, California.

1.2 Purpose and Scope of Investigation

The purpose of our investigation was to: 1) investigate subsurface conditions in the vicinity of the proposed dual reservoir site using the seismic refraction geophysical method, and to 2) evaluate the depth to and characteristics of the bedrock, including the rippability of materials in the vicinity of the proposed excavations.

The scope of work included a field seismic refraction survey followed by geophysical analysis of the acquired data, and preparation of this report.

1.3 Geologic Setting

The project area is characterized by moderately to steeply sloping, north-facing hillside terrain underlain by surficial soils and artificial fill over weathered Orinda Formation sedimentary bedrock.

2.0 SEISMIC REFRACTION SURVEY

A total of eleven (11) individual seismic refraction lines* with a combined spread length of 2,790 feet were recorded at two sites, the first three at the chemical building site near the existing plant administration facilities, and the remaining lines southwest of the existing plant in the vicinity of the proposed tanks. These seismic refraction lines were recorded on March 27 and 28, 1998, in the locations shown on figures 1 and 2. The interpreted results are presented on figures 3 through 8. Rippability correlations are presented on Figure 9. The method and equipment used, the interpreted results, limitations and a summary of pertinent conclusions are discussed in the following sections of this report.

2.1 Method and Equipment Used

The seismic refraction survey procedure used consisted of placing twelve (12)

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* For the purposes of this report, a seismic refraction line is defined as twelve geophones spaced at equal intervals of 25 feet along a straight line and monitored simultaneously while a sledge hammer is repeatedly impacted 12.5 feet off each end, and at the center, of the line.

geophones in as straight a line as practical (in plan) spaced at 10-foot or 25-foot intervals along as constant a slope as practical (in profile). A large sledge hammer was impacted at 10 or 12.5 feet off both ends of each line and at the center of the longer lines. The hammer impacts generated seismic compression waves which were refracted through subsurface materials and received by the deployed geophones. The signal from the energy source initiation (time break) started the instrument sweep as signals from the geophones were monitored (amplified, filtered and stacked) simultaneously by a digital seismograph with an on-board computer and displayed graphically in analog form on the built-in computer monitor. Digital records stored in the computer were field checked, stored on magnetic disk and returned to our office for printing, data reduction and interpretation.

Seismic refraction lines were marked for location and elevation using an existing base map combined with hand level, Brunton compass and measuring tape methods. Lines were marked with stakes and flags in the field and located on the base map. Locations and relative elevations should be considered approximate.

The data reduction and interpretation procedure consisted of the following sequence of tasks:

- computerized picking of first breaks of compression waves (P-waves) from the digital records of the seismic system computer,
- visual adjustment of first break picks by observing the analog record,
- plotting of time-distance graphs utilizing raw data,
- preliminary determination of apparent velocities,
- plotting of elevation data along the profiles,
- measurement of differences between actual geophone elevations and a constant slope profile,
- computer analysis of preliminary apparent velocities and elevation differences to determine travel-time corrections,
- adjustment of the time-distance graphs and refinement of apparent velocity determinations satisfying reciprocity,
- comparison of time-distance and velocity data with a catalog of subsurface structures to interpret an appropriate seismic refraction model,
- computer analysis (using computer program developed by Shires, 1983, involving principles published by Mooney, 1977, Handbook of Engineering Geophysics, satisfying the condition of reciprocity, travel-time = distance/velocity, and Snell's Law of Refraction) of apparent velocity and intercept time data to determine depths of refractors, true velocities, dips of refractors, and angles of wave incidence (seismic ray paths),
- measurement of time deviations from "best fit" apparent velocity slopes on the time-distance graphs,

- computer analysis of apparent velocities and time deviations to determine refractor profile corrections,
- adjustment of refractor depths to reflect time deviations,
- correlation of results with known geologic factors (from mapping and/or borehole logs), with adjacent or overlapping seismic refraction data, and
- final preparation of interpreted subsurface velocity profiles.

The equipment used for the seismic refraction survey consisted of twelve (12) geophones at one time of 10 Hz natural frequency. The geophones were connected to 10-foot or 25-foot take-out spacing cables using Mueller clips. The combination seismograph/oscillograph used was a 24-channel ABEM™ Terraloc Mark 3 Seismic System mounted on a pack frame for portability.

The energy source used consisted of a 16-pound sledge hammer equipped with a seismograph triggering mechanism. The sledge hammer was repeatedly impacted on a steel plate placed in a cleared area on the ground surface. Repeated impact signals were stacked for each seismic record.

2.2 Interpreted Results

Seismic refraction lines S-1, S-2 and S-3 were recorded at the proposed chemical building site in the locations shown on Figure 1. Results are presented on Figure 3. All three lines incorporated 12 geophones spaced at 10 feet with a spread lengths of 130 feet with shots at 10 feet off of both ends of each line.

Line S-1 is interpreted to be underlain by one velocity zone (refractor) to the depth surveyed. This zone is characterized by low to medium velocity (2020 ft/sec) material extending from the ground surface on down to the depth limit of information obtained (about 40 feet beneath the ground surface). This zone likely corresponds to weathered bedrock materials.

Lines S-2 and S-3 are interpreted to be underlain by two velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1110 to 1270 ft/sec) materials to a thickness of 3.5 to 10.7 feet beneath the ground surface. This upper zone corresponds to soil and deeply weathered bedrock materials that are relatively dry. The underlying zone consists of low to medium velocity (2950 to 3070 ft/sec) materials extending from 3.5 to 10.7 feet on down to the depth limit of information obtained (about 40 feet beneath the ground surface). This zone likely corresponds to weathered bedrock materials.

Based on geomorphology, the northeasterly end of Line S-3, where the low velocity zone thickens, may be underlain by landslide debris or thickening colluvium. Additional mapping and/or direct observation subsurface exploration should be carried out by your engineering geologist in this area to evaluate this feature.

Seismic refraction lines S-4, S-5, the northern one-half of Line S-6, and Line S-7 were recorded in a relatively flat area of artificial fill in the locations shown on Figure 2. Results are presented on figures 4 and 5. All lines incorporated 12 geophones spaced at

25 feet with a spread lengths of 300 feet with shots at 12.5 feet off of both ends and at the center of each line.

Lines S-4 through S-7 are interpreted to be underlain by two or three (2 or 3) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1150 to 1530 ft/sec) materials to a thickness of 7.7 to 29.9 feet beneath the ground surface. This upper zone corresponds to artificial fill, soil and deeply weathered bedrock materials that are relatively dry. The underlying zone consists of medium velocity (4120 to 5790 ft/sec) materials extending from 7.7 to 29.9 feet on down to 39.3 to 76.3 feet beneath the ground surface. This zone likely corresponds to weathered bedrock materials that may be saturated. The medium velocity materials are underlain in turn (in lines S-4, S-5 and S-7) by high velocity (8310 to 9240 ft/sec) materials to the depth limit of information obtained (about 96 feet beneath the ground surface). This zone likely corresponds to less weathered bedrock materials.

Seismic refraction lines S-8 through S-11 were recorded on the slope above the artificial fill area in the vicinity of the proposed twin tanks in the locations shown on Figure 2. Results are presented on figures 6 through 8. All lines incorporated 12 geophones spaced at 25 feet with a spread lengths of 300 feet with shots at 12.5 feet off of both ends and at the center of each line.

Lines S-8 and S-9 are interpreted to be underlain by three (3) velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (1140 to 1280 ft/sec) materials to a thickness of 9.0 to 21.3 feet beneath the ground surface. This upper zone corresponds to soil and deeply weathered bedrock materials that are relatively dry. The underlying zone consists of low to medium velocity (2710 to 4960 ft/sec) materials extending from 9.0 to 21.3 feet on down to 30.1 to 73.2 feet beneath the ground surface. This zone likely corresponds to weathered bedrock materials. The low to medium velocity materials are underlain in turn by medium to high velocity (5550 to 7980 ft/sec) materials to the depth limit of information obtained (about 96 feet beneath the ground surface). This zone likely corresponds to less weathered bedrock materials that may be saturated.

Lines S-10 and S-11 are interpreted to be underlain by two velocity zones (refractors) to the depth surveyed. The zone closest to the ground surface is characterized by low velocity (980 to 1130 ft/sec) materials to a thickness of 10.8 to 19.6 feet beneath the ground surface. This upper zone corresponds to soil and deeply weathered bedrock materials that are relatively dry. The underlying zone consists of medium velocity (5300 to 5600 ft/sec) materials extending from 10.8 to 19.6 feet on down to the depth limit of information obtained (about 96 feet beneath the ground surface). This zone likely corresponds to weathered bedrock materials that may be saturated.

2.3 Rippability

Rippability is strongly influenced by the physical condition of the rock masses to be ripped. Structural features in rock such as bedding planes, cleavage planes, joints, fractures and shear zones influence rippability. Rock masses tend to be rippable if they have closely-spaced fractures, joints, or other planes of weakness. Massive rock materials lacking discontinuities, even where partially weathered, may exhibit marginal rippability, requiring blasting for removal.

Seismic compression wave velocities can be related to both rock hardness and fracture density. Seismic refraction velocities have been related to rippability by Caterpillar Inc. (1990) as displayed on graphs relating seismic velocity for various rock types to rippability with various types of equipment (combinations of dozers and rippers). Two examples of these graphs are presented on Figure 9 for both D8L and D9N dozer/ripper combinations. In general, rocks such as the sedimentary claystone rocks present at this site become marginally rippable above about 7,400 feet/second and non-rippable above about 9,700 feet/second using either a D8 Dozer with a Multi or Single Shank No. 8 Ripper, or a D9 Dozer with a Multi or Single Shank No. 9 Ripper.

The charts of ripper performance should be considered as being only one indicator of rippability. The following precautions should be observed when evaluating the rippability of a given rock formation:

- Ripper tooth penetration is usually the key to successful ripping, regardless of seismic velocity. This is particularly true in finer-grained homogeneous materials and in tightly cemented formations.

- Although low seismic velocities in sedimentary rocks indicate probable rippability, if the fractures and bedding joints do not allow tooth penetration, the material may not be ripped effectively.

- Pre-blasting or "popping" may be required to induce sufficient fracturing to allow tooth penetration, but the economics of this should be checked carefully in the higher grades of sandstones, limestones and granites.

- Impact ripping may be used in marginal situations because significant boosts in production may be possible relative to conventional ripping by using an impact ripper mounted on a D10N or D11N dozer.

- Ripping success may well depend on the operator finding the proper combination of number of shanks used, length and depth of shank, tooth angle and direction and throttle position.

Based on the seismic velocities measured at this site, it appears that materials characterized by high velocities (i.e., greater than 7400 ft/sec) will be marginally to non-rippable with either the D8 dozer with No. 8 ripper combination, or a D9 dozer with No. 9 ripper. Based on seismic velocity correlation's, the surficial soil and weathered bedrock at this site (with velocities well under 7400 ft/sec) that are contemplated for removal should not pose significant problems to mass grading with heavy construction equipment.

3.0 INVESTIGATION LIMITATIONS

The subsurface velocity profiles presented in this report represent the most reasonable interpretation of geophysical survey data based on our limited knowledge of the existing geologic conditions at the site. The results are presented for design information only and are not intended to serve as information for determining construction procedures. Interpretations were made in accordance with generally accepted geophysical methods and practices. This warranty is in lieu of all other warranties, express or implied.

It is best to correlate seismic refraction data with direct subsurface exploration data and this was not accomplished for the chemical building lines and some of the water storage tank lines conducted for this investigation.

The quality of seismic refraction data for this survey was good, but in some cases affected by background noise, irregular terrain, and lateral inhomogeneity. These factors produced noise signals and/or scatter in the recorded data, limiting the accuracy of first break compression wave picks and interpretation. The seismic refraction method used has some inherent limitations such as the possibility for undetectable hidden layers, blind zones, and velocity inversions. The maximum depth of reliable seismic information obtained during this survey can be assumed to be approximately one-third of the length of the individual lines, with information at a maximum depth underlying the middle one-third of the lines. For example, a seismic refraction line 300 feet in length will typically yield reliable data on subsurface materials to a depth of about 100 feet beneath the middle 100 feet of the line.

4.0 SUMMARY OF CONCLUSIONS

The proposed chemical storage building site surveyed was generally underlain by one to two velocity zones to the depth limit surveyed (about 40 feet beneath the ground surface). These zones were generally characterized by lower velocities where surficial soils and deeply weathered bedrock were present (to depths of 3.5 to 10.7 feet) and low to medium velocities where weathered bedrock materials were present (from 3.5 to 10.7 feet up to at least about 40 feet deep). Based on velocity/rippability correlation's, surficial materials in the area of the chemical storage building should be rippable with heavy conventional excavation equipment. Based on geomorphology and a thicker low velocity zone, the northern end of Line S-3 may be underlain by landslide debris. This area should be investigated more thoroughly by engineering geologic mapping and direct observation subsurface exploration.

The proposed water tanks site was generally underlain by two to three velocity zones to the depth limit surveyed (about 96 feet beneath the ground surface). These zones were generally characterized by lower velocities where artificial fill, surficial soils and deeply weathered bedrock were present (to depths of 7.7 to 29.9 feet), low to medium velocities where weathered bedrock materials were present (from 7.7 to 29.9 feet down to 30.1 to at least about 96 feet), and medium to high velocities where less weathered bedrock materials were present (from 30.1 to greater than 96 feet down to greater than 96 feet). Based on velocity/rippability correlation's, surficial materials in the area of the proposed water tanks should be rippable with heavy conventional excavation equipment, provided that the higher velocity materials recorded at depth are not encountered. Based on geomorphology, there appears to be an earth slump/flow on the slope above the tank site (near the head of the mapped landslide) and there is a near-vertical, somewhat arcuate topographic lineament (scarp?) in the artificial fill slope downslope of the tanks pad. These areas should be investigated more thoroughly by engineering geologic mapping and/or direct observation subsurface exploration.

5.0 REFERENCES

ABEM, (1987), "ABEM Terraloc Mark 3 Operator's Manual".

Caterpillar, Inc., (1990), "Caterpillar™ Performance Handbook", a CAT™ Publication by Caterpillar, Inc., Peoria, Illinois, U.S.A.

Dobrin, M. B., (1960), "Introduction to Geophysical Prospecting", McGraw-Hill Book Company.

Shires, P. O., (1983), "A Seismic Refraction Interpretation Program for Multi-Dipping Layers".

FIGURES

1 Location Map (Chemical Building Site)

2 Location Map (Twin Water Tanks Site)

3 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line Nos. S-1, S-2, and S-3

4 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line Nos. S-4 and S-5

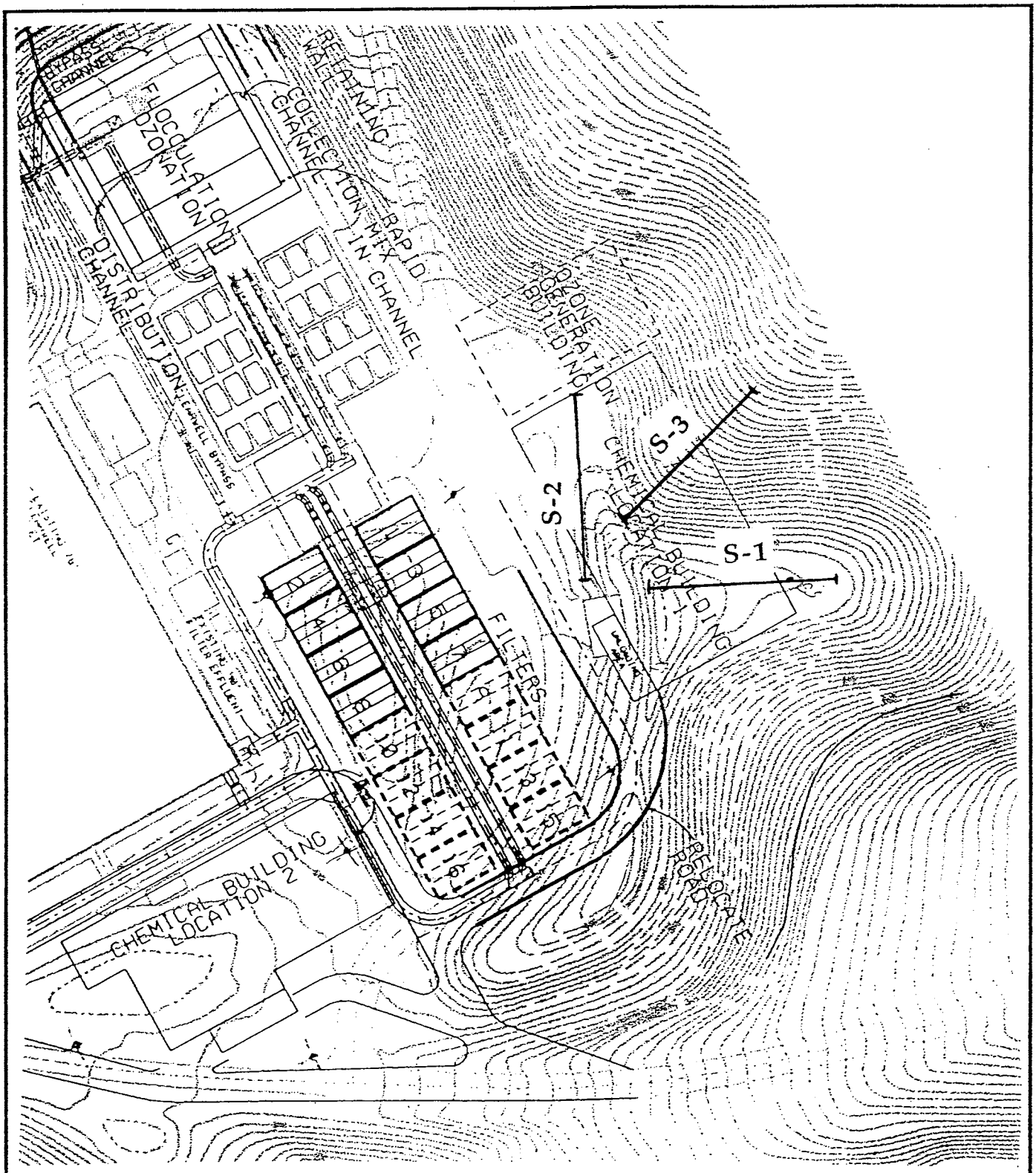
5 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profile, Seismic Line No. S-6

6 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line Nos. S-7 and S-8

7 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profiles, Seismic Line Nos. S-9 and S-10

8 Seismic Refraction Survey Data and Interpreted Subsurface Velocity Profile, Seismic Line No. S-11

9 Rippability Charts (Caterpillar, Inc., 1990)



N

0 100 200

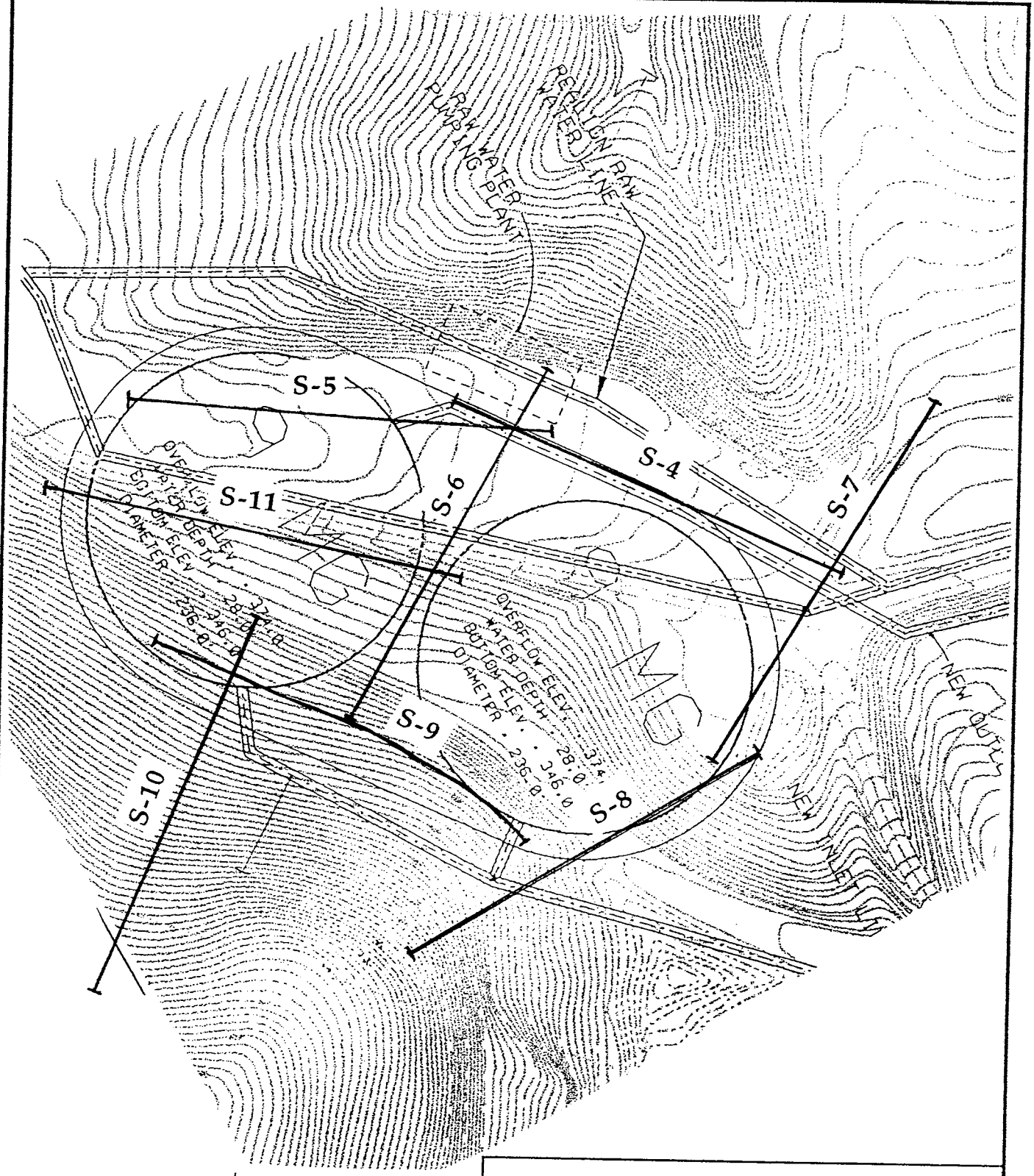
FEET

Portola Geophysics

LOCATION MAP (CHEMICAL BUILDING SITE)

SEISMIC REFRACTION SURVEY
EBMUD WALNUT CREEK FILTER PLANT EXPANSION
CONTRA COSTA COUNTY, CALIFORNIA

GEOPHYS. BY	SCALE	PROJECT NO.
POS	1"=100'	9802
APPROVED BY	DATE	FIGURE NO.
POS	4/22/98	1



N

0 100 200

FEET

Portola Geophysics

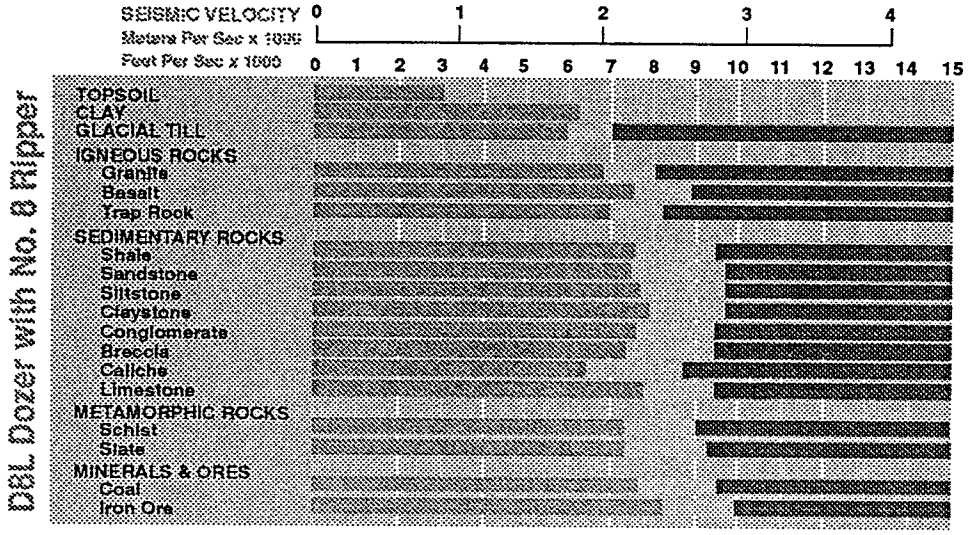
LOCATION MAP (TWIN WATER TANKS SITE)

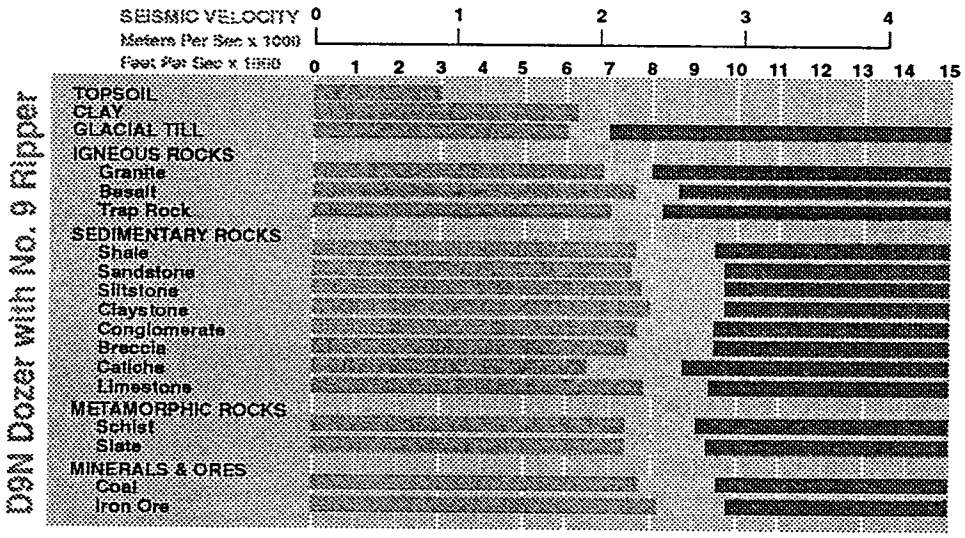
SEISMIC REFRACTION SURVEY
EBMUD WALNUT CREEK FILTER PLANT EXPANSION
CONTRA COSTA COUNTY, CALIFORNIA

GEOPHYS. BY	SCALE	PROJECT NO.
POS	1"=100'	9802
APPROVED BY	DATE	FIGURE NO.
POS	4/22/98	2

RIPPER PERFORMANCE

Multi or Single Shank Rippers
Estimated by Seismic Wave Velocities





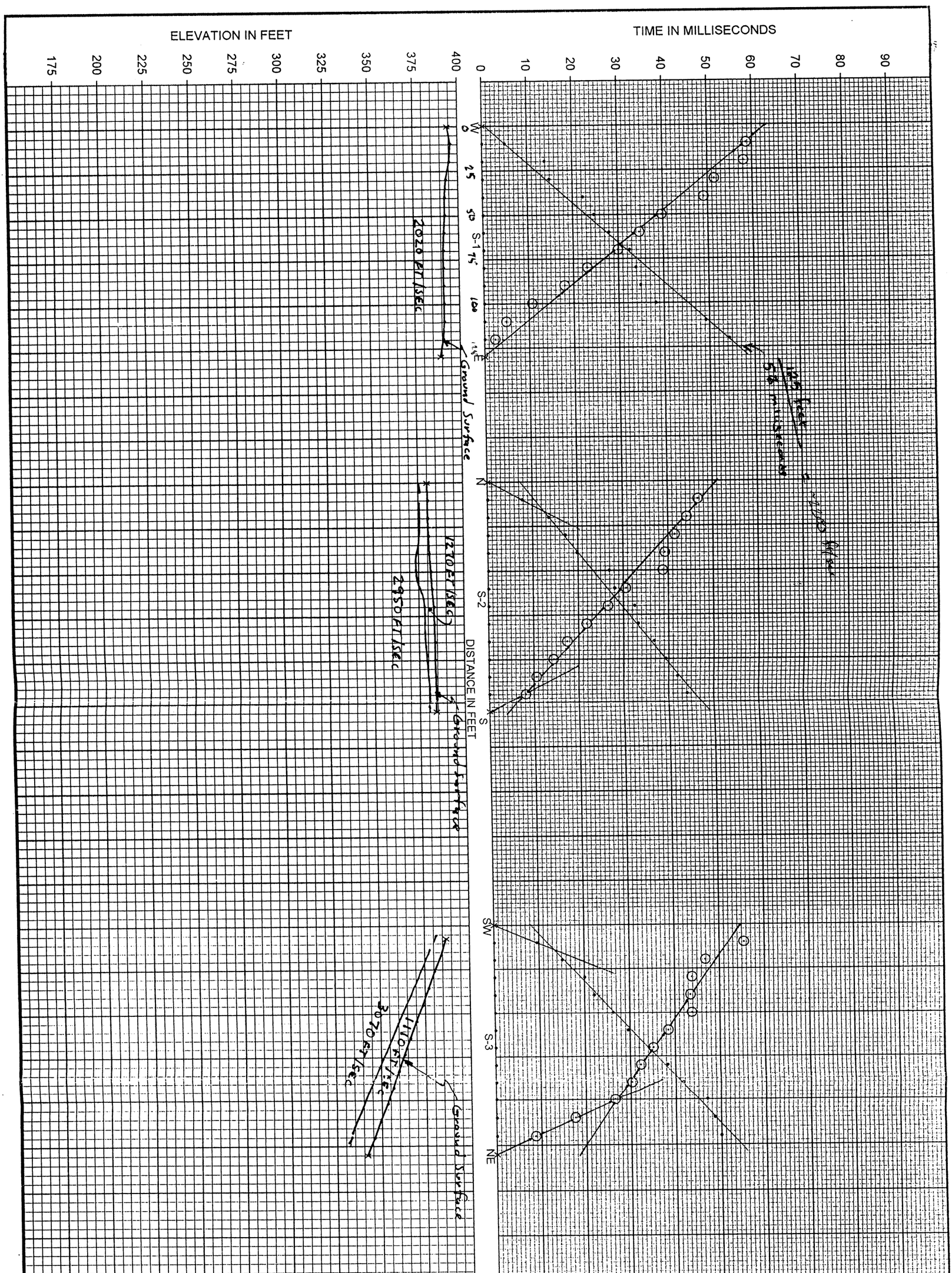
Rippable Marginal Non-Rippable

Source: Caterpillar™, 1990

Portola Geophysics

RIPPABILITY CHARTS (Caterpillar, Inc., 1990)
SEISMIC REFRACTION SURVEY
WALNUT CREEK FILTER PLANT EXPANSION
CONTRA COSTA COUNTY, CALIFORNIA

GEOPHYS. BY	SCALE	PROJECT NO.
POS	As Noted	9802
APPROVED BY	DATE	FIGURE NO.
POS	4/22/98	9



EXPLANATION

1. TIME-DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.
   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET
2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.
   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET
3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S)

REGISTERED GEOPHYSICIST
PATRICK O. SHIRES
No. 879
STATE OF CALIFORNIA

Portola Geophysics
900 Wayside Road Portola Valley,CA 94028

SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

EBMUD Walnut Creek Filter Plant Expansion
Contra Costa County, CA

FIGURE NO.: 3
PROJECT NO.: 9802
DATE: APRIL, 1998
SEISMIC LINE NO.(S): S-1, S-2 & S-3

TIME IN MILLISECONDS

90
80
70
60
50
40
30
20
10
0

W S-5 E W S-4 E

DISTANCE IN FEET

400
375
350
325
300
275
250
225
200
175

ELEVATION IN FEET

Ground Surface
Lines S-4 & S-6
cross

Boring
MW-18

Lines S-5 & S-6
cross

Line S-7
crosses

1200 FT/SEC
1150 FT/SEC
1530 FT/SEC
4930 FT/SEC
5560 FT/SEC
4460 FT/SEC
1530 FT/SEC
1470 FT/SEC
1250 FT/SEC
5650 FT/SEC
5790 FT/SEC
5660 FT/SEC
9090 FT/SEC

EXPLANATION

1. TIME-DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.
   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET
2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.
   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET
3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S)

Portola Geophysics
900 Wayside Road Portola Valley,CA 94028

SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

EBMUD Walnut Creek Filter Plant Expansion
Contra Costa County, CA

REGISTERED GEOPHYSICIST
PATRICK O. SHIRES
No. 879
STATE OF CALIFORNIA

SEISMIC LINE NO.(S):
S-4 & S-5

DATE: APRIL, 1998

PROJECT NO.: 9802

FIGURE NO.: 4

TIME IN MILLISECONDS

90
80
70
60
50
40
30
20
10
0

S
S-6
N

DISTANCE IN FEET

ELEVATION IN FEET

400
375
350
325
300
275
250
225
200
175

Ground Surface

Line S-11 Crosses

Boring MN-16

Lines S-4 & S-5 Cross

1380 FT/SEC

1220 FT/SEC

5050 FT/SEC

5550 FT/SEC

1240 FT/SEC

5050 FT/SEC

Portola Geophysics
900 Wayside Road Portola Valley,CA 94028

SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

EBMUD Walnut Creek Filter Plant Expansion
Contra Costa County, CA

REGISTERED GEOPHYSICIST
PATRICK O. SHIRES
No. 879
STATE OF CALIFORNIA

EXPLANATION

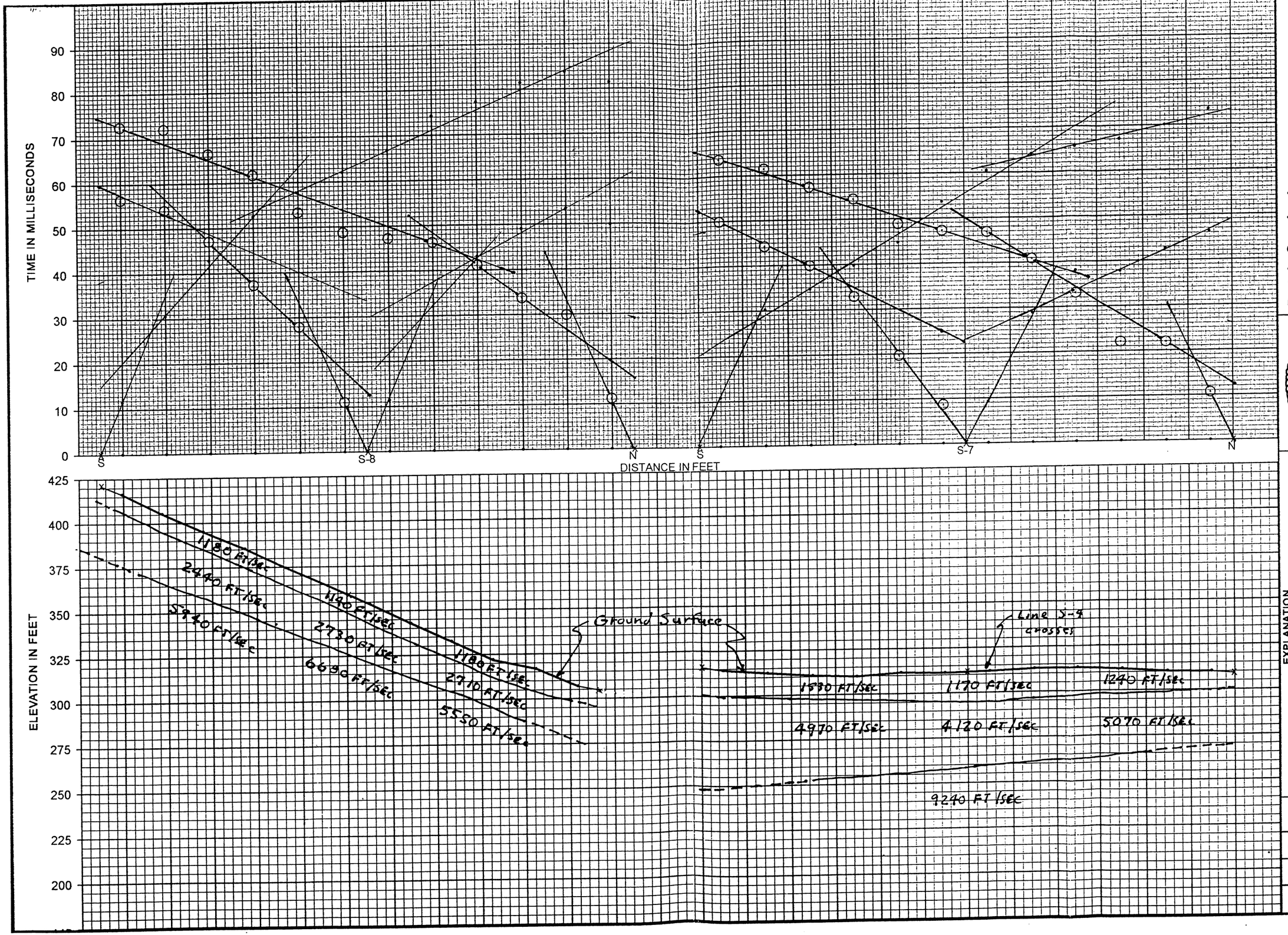
1. TIME-DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.
   VERTICAL SCALE: 1" = 20 MILLISECONDS
   HORIZONTAL SCALE: 1" = 50 FEET
2. SUBSURFACE VELOCITY PROFILES AT BOTTOM OF FIGURE REPRESENT INTERPRETATIONS OF SEISMIC REFRACTION DATA AND ARE INTENDED FOR DESIGN PURPOSES ONLY.
   VERTICAL AND HORIZONTAL SCALE: 1" = 50 FEET
3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S) 2

SEISMIC LINE NO.(S):
S-6

DATE: APRIL, 1998

PROJECT NO.: 9802

FIGURE NO.: 5



Portola Geophysics
900 Wayside Road Portola Valley,CA 94028

SEISMIC REFRACTION SURVEY
DATA AND INTERPRETED SUBSURFACE VELOCITY PROFILES

EBMUD Walnut Creek Filter Plant Expansion
Contra Costa County, CA

REGISTERED GEOPHYSICIST
PATRICK O. SHIRES
No. 879
STATE OF CALIFORNIA

EXPLANATION

1. TIME-DISTANCE GRAPHS AT TOP OF FIGURE REPRESENT SEISMIC REFRACTION SURVEY DATA; DOTS ON BOTTOM LINE OF GRAPHS REPRESENT GEOPHONE LOCATIONS, X'S REPRESENT SHOT POINT LOCATIONS.
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3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S) **2**

SEISMIC LINE NO.(S): S-7&S-8

DATE: APRIL, 1998

PROJECT NO.: 9802

FIGURE NO.: **6**

TIME IN MILLISECONDS

90
80
70
60
50
40
30
20
10
0

W
S-9
E
S
S-10
N

DISTANCE IN FEET

ELEVATION IN FEET

475
450
425
400
375
350
325
300
275
250

Ground Surface

Line S-10 crosses

1170 FT/SEC

1270 FT/SEC

1280 FT/SEC

4980 FT/SEC

4970 FT/SEC

4660 FT/SEC

7980 FT/SEC

Line S-9 crosses

1130 FT/SEC

5340 FT/SEC

1130 FT/SEC

5300 FT/SEC

Ground Surface

1120 FT/SEC

5490 FT/SEC

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EXPLANATION

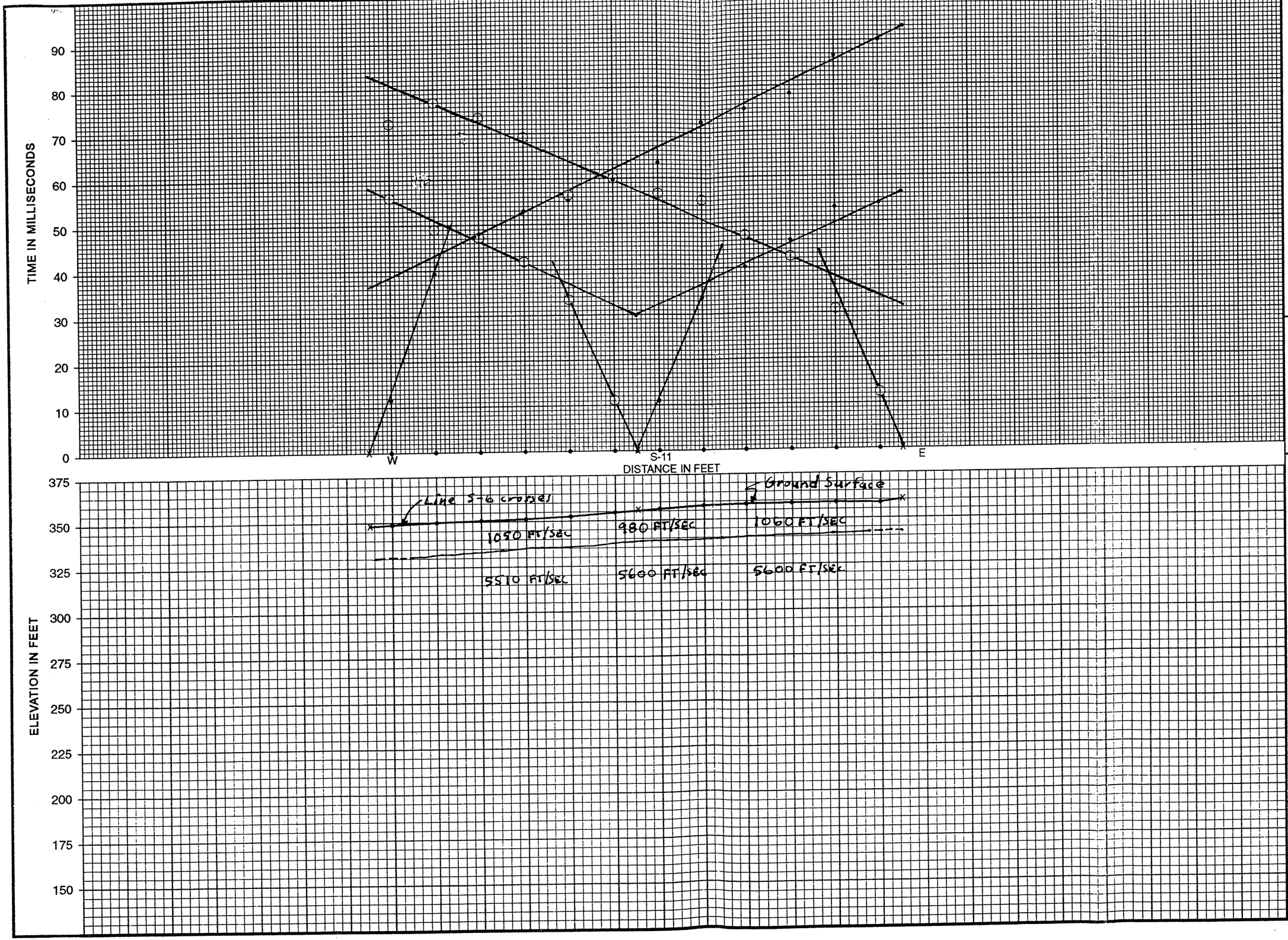
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3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S) 2

SEISMIC LINE NO.(S): S-9&S-10

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PROJECT NO.: 9802

FIGURE NO.: 7



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SEISMIC REFRACTION SURVEY
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EXPLANATION

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3. LOCATION OF SEISMIC LINES SHOWN ON FIGURE NO.(S) 2

SEISMIC LINE NO.(S): S-11

DATE: APRIL, 1998

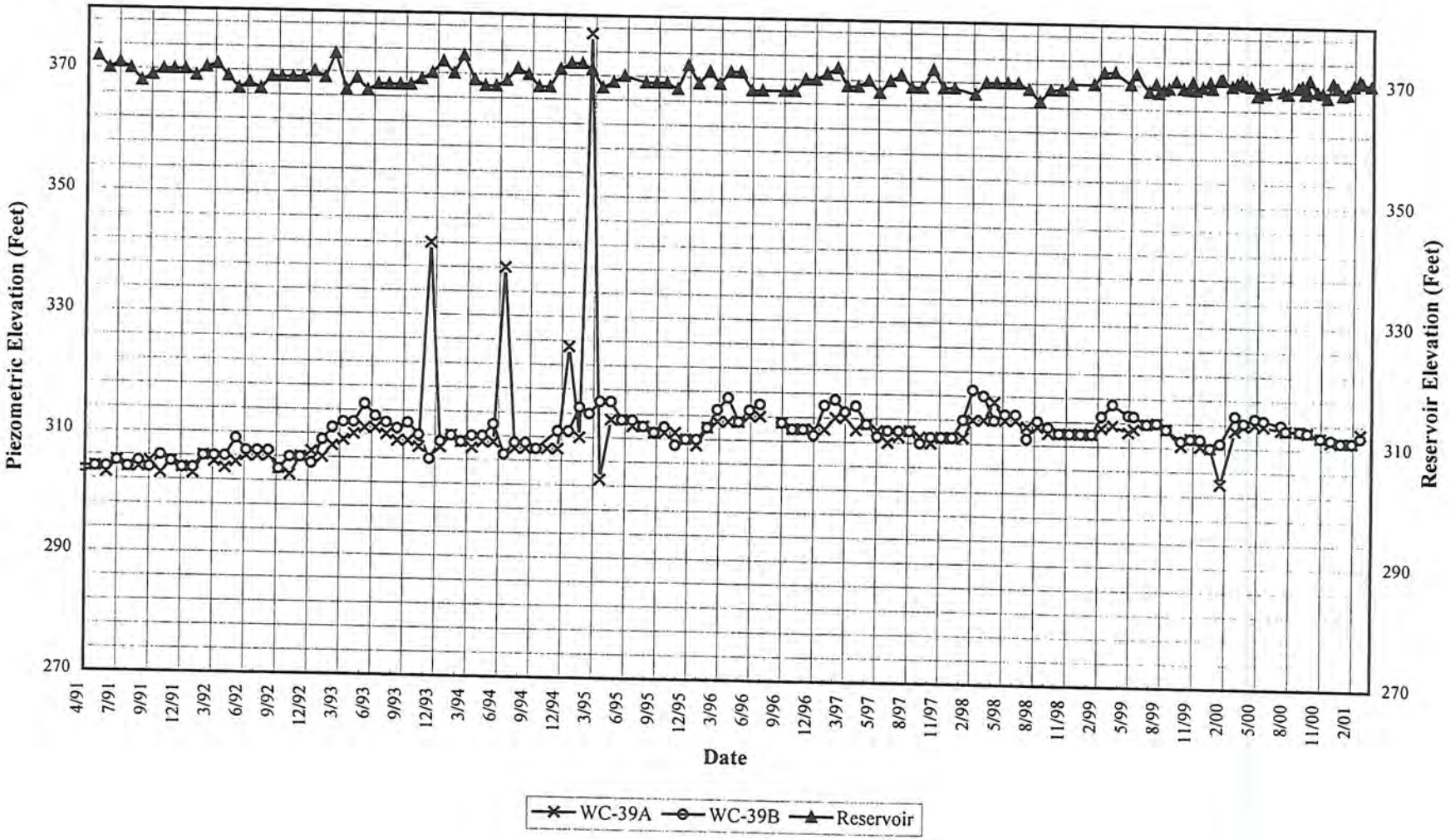
PROJECT NO.: 9802

FIGURE NO.: 8

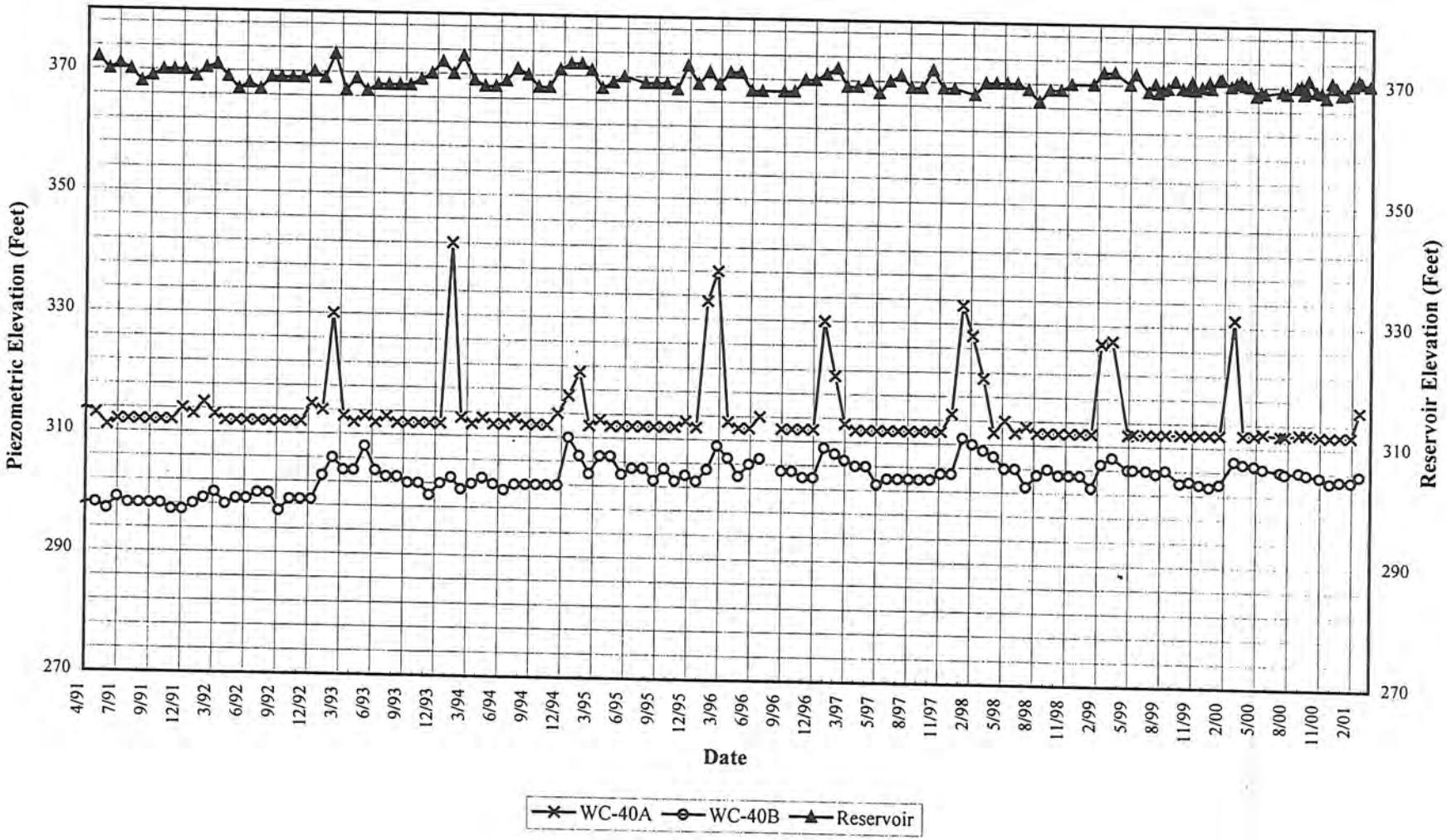
APPENDIX G

Clearwell Piezometer Monitoring Records

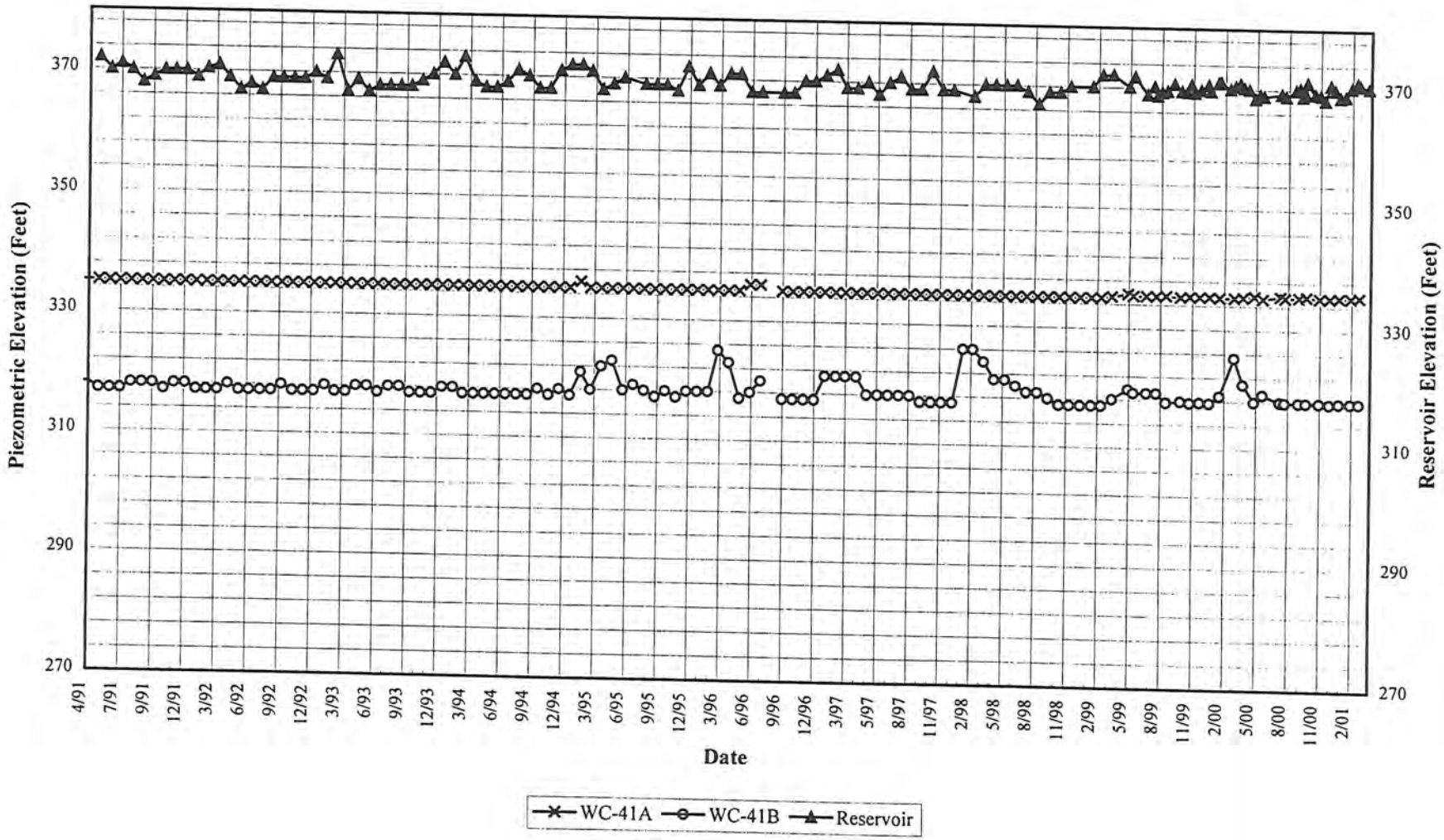
Walnut Creek Filter Plant Clearwell
Piezometer: WC-39
Instrument Number 311910010



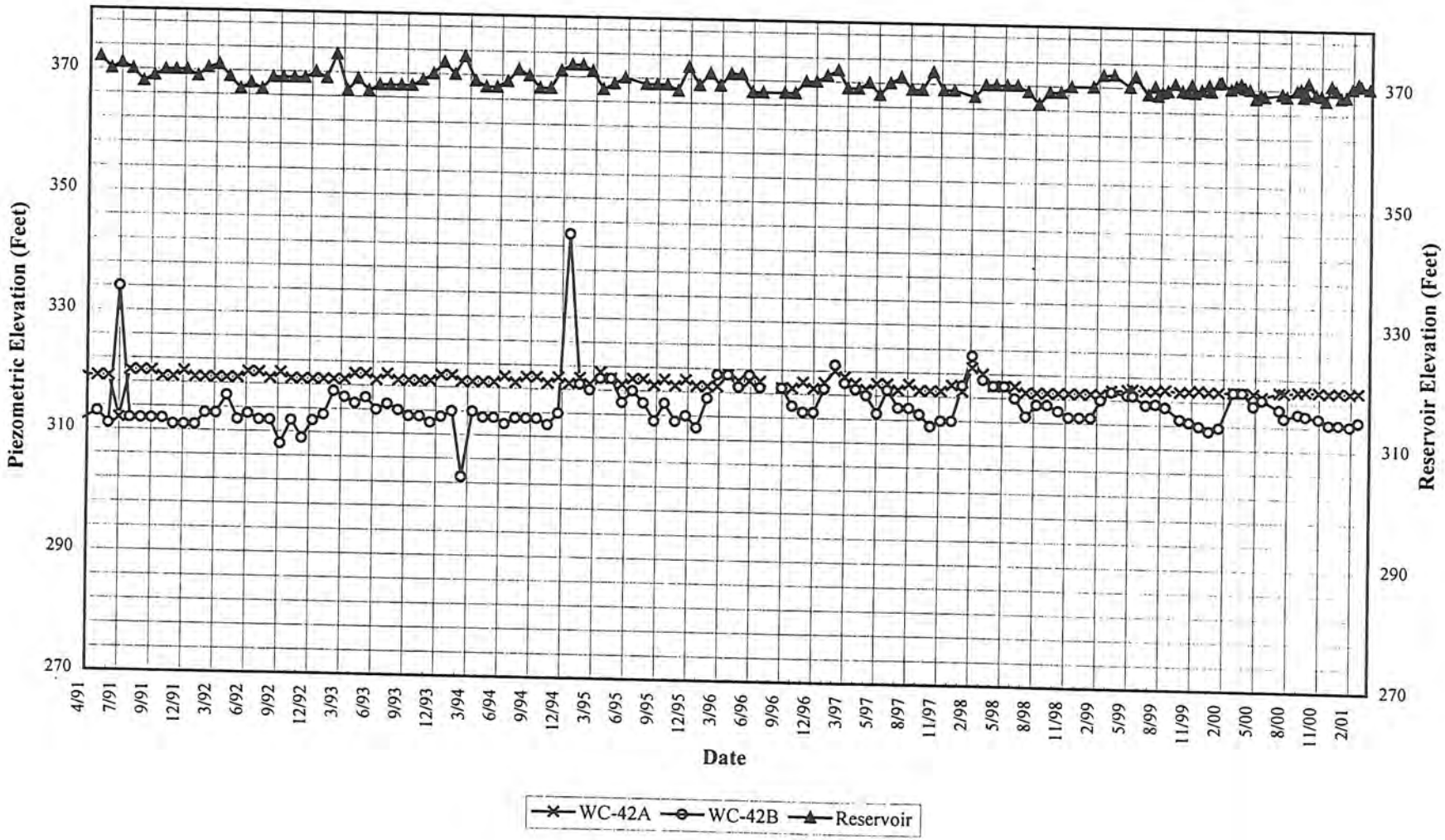
Walnut Creek Filter Plant Clearwell
Piezometer: WC-40
Instrument Number 311910020



Walnut Creek Filter Plant Clearwell
Piezometer: WC-41
Instrument Number 311910030



Walnut Creek Filter Plant Clearwell
Piezometer: WC-42
Instrument Number 311910040





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