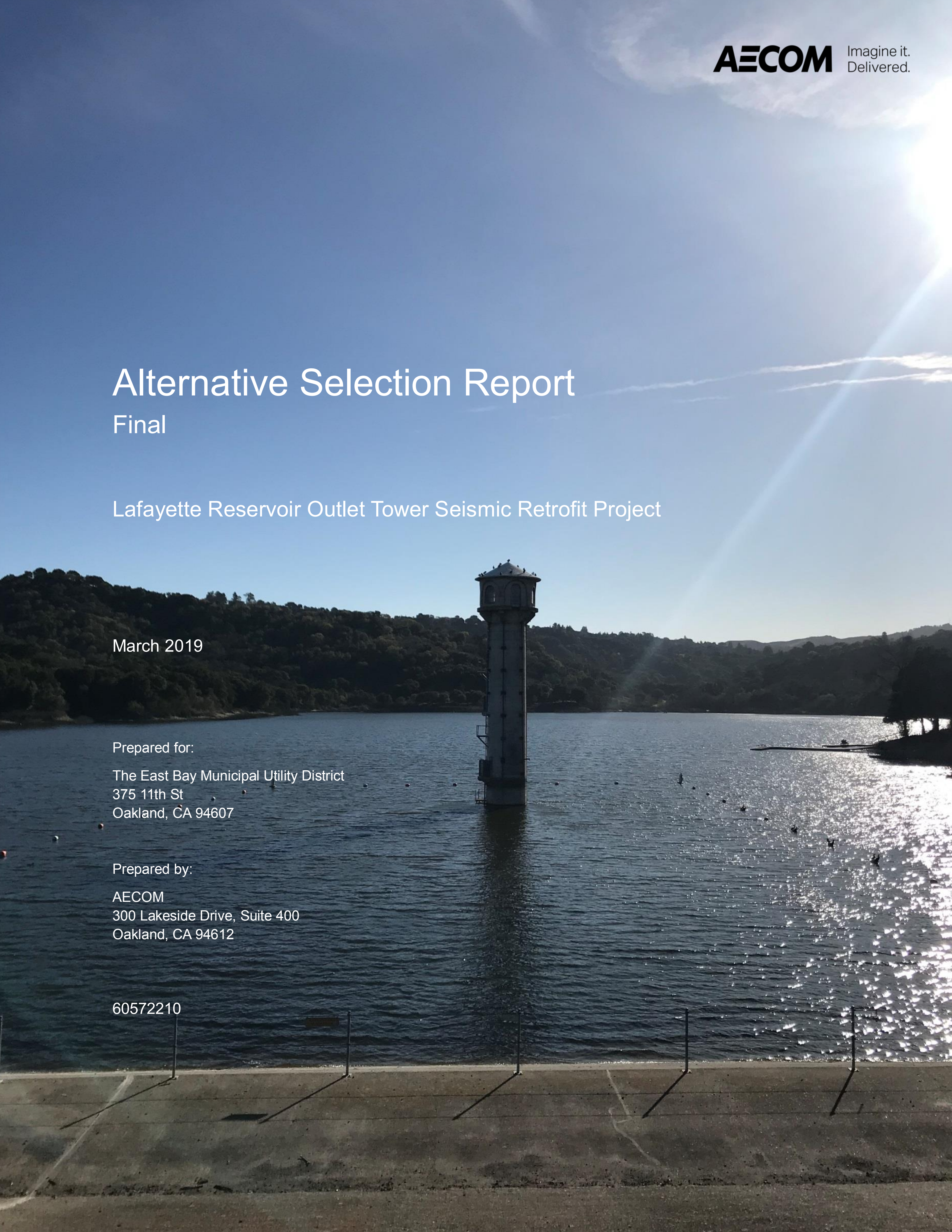

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Alternative Selection Report

Final

Lafayette Reservoir Outlet Tower Seismic Retrofit Project

March 2019

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Revision History

Revision	Revision date	Details	Authorized	Name	Position
01	08-28-2018	Ranking Updated		Mourad Attalla	Project Manager
02	12-14-2018	EBMUD comments incorporated, analysis updates included		Mourad Attalla	Project Manager
03	01-09-2019	Cost Estimates for Alternative 1 and 3B corrected		Mourad Attalla	Project Manager
04	02-15-2019	Addressed EBMUD comments. Added structural robustness comparison to Chapter 11.		Mourad Attalla	Project Manager
Final	03-27-2019	Addressed all remaining EBMUD comments.		Mourad Attalla	Project Manager

Distribution List

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Acronyms and Abbreviations

3-D	three-dimensional
ACI	American Concrete Institute
ARS	acceleration response spectrum
ASTM	American Society for Testing and Materials
CEQA	California Environmental Quality Act
CNDDB	California Natural Diversity Database
District	East Bay Municipal Utility District
DCR	demand-to-capacity ratio
DSHA	Deterministic Seismic Hazard Analysis
DSOD	California Department of Water Resources – Division of Safety of Dams
EL	Elevation
EM	Engineer Manual
FE	finite element
FPB	friction pendulum bearings
FRP	fiber-reinforced polymer
FS	factor of safety
g	unit of gravity
Hz	Hertz
ICEC	International Civil Engineering Consultants, Inc.
in./in.	inches per inch
kip	kilo pounds
kip-ft	kip feet
ksi	kips per square inch
M	moment magnitude
m/s	meters per second
MCE	Maximum Credible Earthquake
Mcr	Cracking Moment
MJA	McMillen Jacobs Associates
Mn	Nominal Moment Capacity
Mu	Moment Demand
NRHP	National Registry of Historic Places
NEHRP	National Earthquake Hazards Reduction Program
pcf	pounds per cubic feet
PGA	peak ground acceleration
Project	Seismic Retrofit Design of the Lafayette Reservoir Outlet Tower
PTI	Post Tensioning Institute
R_{JB}	Closest distance to surface projection of the rupture plane (Joyner-Boore distance)

R_{RUP}	Closest distance to rupture plane
R_X	Closest distance to the surface projection of the top edge of the rupture plane, measured perpendicular to its average strike
RSA	response-spectrum analysis
SOW	Scope of Work
SRSS	square root of the sum of squares
SSI	soil-structure interaction
TERRA/COWI	TERRA Engineers, Inc. and COWI
Tower	Lafayette Reservoir Outlet Tower
USACE	U.S. Army Corps of Engineers
Vn	Shear Capacity
Vu	Shear Demand
Vsf	shear friction capacity
V_{S30}	Time-averaged shear-wave velocity to 30-meter depth
W	Down-dip width of the rupture plane
$Z_{1.0}$	Depth to shear wave velocity of 1.0 km/sec
$Z_{2.5}$	Depth to shear wave velocity of 2.5 km/sec
Z_{hyp}	hypocentral depth
Z_{TOR}	Depth to top of coseismic rupture

1. Executive Summary

This report investigates seismic retrofit alternatives for the Lafayette Reservoir Outlet Tower (Tower) as part of AECOM's scope of work (SOW) to provide a retrofit design to address the Tower seismic deficiencies. The Tower serves a dual function acting as a spillway at Elevation 450, and as an outlet to control reservoir releases. The dam, reservoir and Tower are under the jurisdiction of the California Department of Water Resources, Division of Safety of Dams (DSOD). DSOD has determined that the Tower must be retrofitted to prevent uncontrolled release of the reservoir in the event of failure following a major earthquake and to maintain its spillway function.

This report provides the background information reviewed, structural analyses performed, and the rational approach followed in selecting a preferred retrofit alternative, taking into account the parameters of the Project objectives. Based on previous studies of the Tower and AECOM's assessment, four alternatives were further investigated in this report, namely:

Alternative 1 – Through-Wall Post-Tensioning

Alternative 2 – External Carbon-Fiber Wrapping

Alternative 3 – Tower Shortening

Alternative 4 – Mid-Height Base Isolation

The report includes a detailed comparison of the previous Tower studies and proposed retrofit alternatives. AECOM also provided its own assessment of the Tower using assumptions based on the most up-to-date design criteria and design standards. AECOM performed a dynamic response spectrum analysis of the Tower to conduct the assessment.

AECOM conducted on-site concrete testing in August 2018 to confirm the concrete compressive strength of the Tower and geotechnical investigation in September 2018 to obtain site-specific data for foundation properties and seismic ground motions. The geotechnical investigation was performed through an over-the water boring adjacent to the Tower. The results of the investigations are described in this report.

A parallel study, the Lafayette Reservoir Outlet Tower Historical Resource Evaluation (EBMUD 2018), found that the Lafayette Tower does not meet the eligibility criteria for listing in the National Register of Historic Places (NRHP), the California Register of Historical Resources (CRHR), nor as a Contra Costa County Historical Landmark, thus it does not qualify as a historical resource pursuant to California Environmental Quality Act (CEQA) guidelines.

AECOM performed structural analyses of the alternatives to determine their efficiency as seismic retrofits. The alternatives were also assessed in terms of CEQA (aesthetic and biological) requirements, constructability, cost, and life-cycle costs. Upon taking all these factors into consideration, Alternatives 1 and 3 were deemed to be the most appropriate to meet the overall Project and District objectives. AECOM performed a comparison between Alternatives 1 and 3 in terms of structural robustness and sensitivity to design assumptions. Alternative 3 was more effective in reducing the Tower vulnerabilities.

2. Introduction

2.1 Background

The East Bay Municipal Utility District (District) retained AECOM in May 2018 to provide engineering services for the Seismic Retrofit Design of the Lafayette Reservoir Outlet Tower (the Project). This Alternative Selection Report constitutes a deliverable under Task 4 of the Scope of Work (SOW) that serves to evaluate alternatives and present a recommended alternative for design.

The Lafayette Reservoir Outlet Tower (Tower) is in Contra Costa County, California (Figure 2-1). The Tower is owned and operated by the District. The Lafayette Reservoir Dam and Tower are under the jurisdiction of the California Department of Water Resources, Division of Safety of Dams (DSOD).

Figure 2-1 Lafayette Reservoir Outlet Tower Location

The Tower, constructed in 1927, is approximately 170 feet tall, with an operating house platform of elevation (EL) 500 feet[1] (Figure 2-2). The Tower functions as a multi-level outlet, as well as an overflow spillway at EL 450 feet. In addition to the spillway opening at EL 450 feet, there are three gates, at EL 384, EL 410, and EL 430 feet. The Tower has a reinforced-concrete circular shaft with an inner diameter of 8 feet, and an outside diameter varying from 11 feet, 2.5 inches at the top (EL 500 feet) to 13 feet, 7.5 inches at the bottom (EL 381 feet). The lower 43 feet of the Tower are embedded below ground from the bottom of the Reservoir at EL 388 feet to EL 345 feet. In the original design, a steel pipe channeled the flow from the spillway opening to a 60-inch-diameter conduit at the bottom of the Tower. In 1968, the Tower interior chamber was divided into two chambers by radial, reinforced concrete wall partitions at 45-degree rotations spiraling from EL 465 feet to EL 378 feet. The partitions separate the flow to the inlet/outlet conduit and the overflow spillway. The gate valve previously at EL 450 feet was removed to provide a spillway opening. The overflow spillway connects to a 60–inch-diameter reinforced-concrete conduit at EL 385 feet. Another 60–inch-diameter concrete inlet/outlet conduit passes through the Tower at EL 374 feet. The conduits run on top of each other downstream, and eventually turn side-by-side. See drawings in Appendix D.

The Tower serves as both an inlet/outlet conduit and an overflow spillway, and failure of the Tower would impede its ability to serve these functions. The Tower is important to the District because it provides the only means of

[1] All elevations are to EBMUD Aqueduct Datum (Standard Mean Sea Level Datum minus 0.52 foot)

controlling the reservoir water level and making releases in the event of a dam safety emergency. Seismic instability of the Tower would bring significant potential to compromise the ability to safety control the reservoir level following a large seismic event. Therefore, the DSOD has restricted the maximum allowable reservoir level, and has required the District to address the Tower's seismic deficiencies.

Figure 2-2 Photo of Lafayette Reservoir Outlet Tower

2.2 Purpose

The District's objective is to design a retrofit for the Lafayette Outlet Tower to address the seismic deficiencies of the Tower. This includes design of structural retrofits, mechanical retrofits to the gate valves to restore functionality, and associated electrical engineering services. The retrofit objectives must meet the DSOD dam safety objectives, which are to:

- Maintain ability to lower the reservoir following a major earthquake
- Reduce the risk of uncontrolled release of the reservoir due to a major earthquake

The earthquake design criterion is defined by DSOD as the 84th percentile earthquake, which is the uniform deterministic hazard earthquake representing the Maximum Credible Earthquake (MCE).

After review of the previous studies and analyses of the Tower, AECOM selected four alternatives for seismic retrofit of the Tower for further study and comparison to arrive at the most viable alternative that will be acceptable to the stakeholders:

Alternative 1 – Through-Wall Post-tensioning

Alternative 2 – External Fiber-wrapping

Alternative 3 – Tower Shortening

Alternative 4 – Mid-Height Base Isolation

These alternatives are discussed in detail in Sections 7 through 10 of this report. This purpose of this Alternative Selection Report is to present the background information reviewed, structural analyses performed, and the selection process followed to reach a viable retrofit alternative in light of the parameters contributing to the Project objectives, including considerations for cost, constructability, effectiveness in addressing the structural deficiencies, and the environmental (biological and aesthetic) considerations. Starting with an overview and evaluation of the previous analysis of the Tower, the report presents AECOM's assessment of the Tower's seismic performance. The retrofit criteria and objectives are then described in Section 6, and the alternatives are described in detail in Sections 7 through 10, along with details of the background information, analysis methods, analysis results, conclusions, and recommendations. The alternatives are evaluated and graded based on grading criteria devised to meet the Project objectives. The report concludes with presenting the recommended alternative for design, with the overall objective of engaging the District and other stakeholders in reaching a consensus on the most viable retrofit design alternative.

2.3 Background Information and Previous Studies

The primary sources of background information reviewed for the Lafayette Outlet Tower Retrofit Alternative Selection include the following:

- EBMUD Drawings DH 1064-7 and 1065-7, Lafayette Reservoir Operating Tower Conduit Details and Details of Reinforcing Steel (EBMUD, 1927); see Appendix D
- EBMUD Drawings 5450-G-1 and 5450-G-2, Lafayette Reservoir Outlet Tower Modifications (EBMUD, 1967); see Appendix D
- International Civil Engineering Consultants, Inc. (ICEC) Final Report *Seismic Evaluation of Lafayette Reservoir Outlet Tower* (ICEC, 1995)
- DSOD Memorandum of Design Review “Seismic Evaluation of Proposed Concrete Infill Retrofit,” Lafayette Tower, No. 31-2 (DSOD, 2011)
- McMillen Jacobs Associates (MJA) Report *Lafayette Outlet Tower Seismic Evaluation and Preliminary Retrofit Alternatives* (MJA, 2015)
- TERRA Engineers, Inc. and COWI (TERRA/COWI) Technical Memorandum “Conceptual Design of Base Isolator Retrofit Alternative for Lafayette Reservoir Tower” (TERRA/COWI, 2017)
- Technical Memorandum “Lafayette Reservoir Outlet Tower Historical Resource Evaluation” prepared by ESA for EBMUD (EBMUD, 2018)

The previous studies by ICEC (1995), DSOD (2011), MJA (2015), and TERRA/COWI (2017) identified a number of retrofit alternatives including the following:

<u>Shortening the Tower</u>

This alternative consists of demolishing the upper part of the Tower down to EL 455 to 460 feet (5 to 10 feet above the reservoir’s spillway elevation). It includes moving the gate house/control room down to EL 455 to 460 feet or placing a closure slab at the top and moving the gate controls on-shore. This alternative was considered by ICEC, DSOD, and MJA, and recommended as a potential solution by DSOD and MJA.

The previous studies have suggested that this is a relatively cost-effective alternative. A main disadvantage of this alternative is that shortening the Tower by approximately 40 feet would significantly change the appearance of the Tower. The Lafayette Reservoir Outlet Tower Historical Resource Evaluation (EBMUD 2018) concluded that the Tower does not qualify as a historical resource pursuant to California Environmental Quality Act (CEQA) Guidelines Section 15064.5(a). However, aesthetic factors need to be addressed since Tower shortening could affect the aesthetic values for the City of Lafayette, the local community, and patrons of the recreation area.

<u>Unreinforced Concrete-Infill</u>

This retrofit consists of infilling the lower portion of the Tower with unreinforced concrete, and was studied by DSOD and MJA; however, this alternative was not recommended by either. Although the concrete infill slightly improves the shear capacities of the lower sections, the bending capacities remain unimproved and the additional mass added to the Tower also increases the seismic demands.

<u>External Jacket</u>

An external jacket consisting of steel, reinforced-concrete, or carbon fiber-wrap, was discussed by MJA. An external jacket could be placed at only those elevations of the Tower with deficient shear and flexure capacities.

A main challenge would be installing the external jacket if sections of the Tower below the water elevation require retrofit. Installation of retrofit elements under the water line would require constructing a cofferdam, draining the reservoir, or installing elements by construction divers. For these reasons, MJA did not recommend this as a preferred retrofit alternative.

Post-tensioning

As proposed by MJA, this retrofit alternative consists of installing post-tensioning tendons from the top of the Tower, and anchoring them into the rock at the Tower's foundation. The tendons would provide a post-tensioning force increasing in the axial load within the Tower sections, thereby increasing the nominal moment and shear capacities.

MJA proposed external post-tensioning with the tendons attached to the outside of the Tower walls. This would present a visual change that would need to be reviewed as an aesthetic factor. Tensioned tendons require periodic maintenance and robust corrosion protection measures.

Base Isolation

This alternative, as proposed by TERRA/COWI, consists of cutting a horizontal joint in the Tower and installing friction pendulum bearings (FPB) within the structure approximately 10 feet above the spillway elevation. TERRA/COWI postulated that the FPBs would lengthen the natural period of the isolated structure to avoid the peak earthquake acceleration of the design response spectrum.

This alternative was presented at concept level only. DSOD had concerns about the validity and effectiveness of this alternative and requested physical modeling of the FPB to test its validity if selected for further study. However, due to concerns about its structural effectiveness and significant schedule impacts due to the required studies, the alternative was not further studied beyond the conceptual stage.

2.4 Scope of Work

The AECOM SOW for the retrofit design of the Tower includes the following main tasks:

Establish Geotechnical Data and Actual Concrete Strength

AECOM developed site-specific geologic and geotechnical properties based on a geotechnical investigation conducted as part of the SOW. AECOM conducted an exploratory over-the-water geotechnical boring near the Tower to obtain data to support developing foundation properties to confidently use in the alternatives selection analysis and in the design of the selected Tower retrofit alternative. The boring was used to provide shear and compression velocities of the bedrock. AECOM submitted a Geotechnical Investigation Report following the geotechnical exploration (report attached in Appendix E).

Additionally, AECOM submitted a Deterministic Seismic Hazard Analysis (DSHA) Technical Memorandum summarizing the analysis performed to develop site-specific ground motions. The site-specific acceleration response spectra were used in the alternative selection analysis presented in this report and the site-specific time histories will be used in the final analysis and design of the selected Tower retrofit alternative. The Final DSHA Technical Memorandum was submitted in December 2018 and is attached in Appendix F.

AECOM also performed in-situ testing to determine material properties of the Tower concrete. Concrete cores were extracted from the Tower and tested for compressive strength and elastic modulus. The results were analyzed, compressive strength and elastic modulus were established based on ACI 214 procedures, and the material properties were used for the alternatives analysis. The established properties will also be used in the final analysis and design of the selected retrofit alternative. The results of the concrete core testing are presented in Section 5 and attached in Appendix G.

Sections 3 and 4 of the report present the geotechnical properties and the ground motions based on the results of the geotechnical investigation.

Alternative Selection

This Alternative Selection Report summarizes structural analysis of the top alternatives, and ranks the alternatives. As part of the AECOM's SOW and presented in this Alternative Selection Report, AECOM reviewed previous analyses of the Tower and evaluated the remediation alternatives presented in those studies. The current alternatives proposed for consideration by AECOM in this phase are listed in the Section 2. AECOM performed an analysis of the existing Tower and four proposed alternatives. Detailed structural analyses of the existing Tower and the viable alternatives were performed to confirm the structural effectiveness of the retrofits. The existing Tower structural analysis and seismic evaluation are presented in Section 5. Section 5 summarizes the retrofit objectives and acceptance criteria used for the retrofit alternative.

Sections 7 through 10 present each retrofit alternative considered, including a description of the alternative, a summary of the structural analysis and seismic evaluation performed, and discussions of constructability, environmental, and cost considerations. Section 11 presents a summary of the pros and cons of each alternative and summarizes the alternative ranking process and results. As part of the alternative ranking process, AECOM attended an alternative selection review workshop with the District in September 2018, in which AECOM solicited input from the District on ranking of the alternative. AECOM presented the outcome of the alternative selection and preliminary design to Project stakeholders from the District on October 1st, 2018.

In coordination with the District, AECOM will present the results and recommendations to DSOD and, if needed, the City of Lafayette.

Design

Once the retrofit alternative is selected, AECOM will coordinate the design with mechanical and electrical requirements with a three-dimensional (3D) REVIT model. The model will provide an accurate way to prepare for constructability and cost estimating. Mechanical design, which will be led by AECOM's subconsultant YEI Engineers, includes replacing the Tower gate valves and operators with new gate valves.

Following alternative selection, AECOM will submit to the District a 50 Percent Design Review Report. AECOM will be prepared to attend a 50 percent design review meeting with the District and DSOD following submittal of the 50 Percent Design Review Report. AECOM will start preparation of the Plans, Specifications, and Estimates (PS&E) package upon review and approval of the 50 percent Design Review Report by the District and DSOD.

3. Geotechnical Review

AECOM has reviewed ICEC's geotechnical analysis (1995) of the soil-structure interaction (SSI) between he embedded sub-structure of the Tower (EL 345 to 388 feet) and the surrounding rock. ICEC ran an SASSI (Lysmer, *et al.*, 1982) model of the foundation system: the 43 feet of embedded Tower and the foundation. The results from the SASSI model were used to calculate the spring stiffness parameters at EL 388 feet. Three translational and two rocking stiffness parameters were generated. ICEC did not consider the torsional rotation because the configuration of the Tower above EL 388 feet is nearly axisymmetric. The spring stiffness values, calculated by ICEC, were used in the DSOD (2011), MJA (2015), and TERRA/COWI (2017) analyses.

AECOM conducted a geotechnical investigation at the Lafayette Reservoir, which consisted of an over-water boring adjacent to the Tower (about 30 feet from the center of the Tower) in September 2018. The purpose of the geotechnical investigation was to provide site-specific geologic and geotechnical data to update the geotechnical parameters to be used for analysis of the Tower. The full geotechnical report that includes the details and results of the investigation is attached in Appendix E.

ICEC developed geotechnical springs in 1995 using a Vs profile which assumed 25 feet of alluvium soil with a constant Vs equal to 550 feet/sec over bedrock with a constant Vs equal to 1,250 feet/sec. The boring drilled for this investigation encountered 44 feet of alluvium with Vs measurements between 723 and 903 feet/sec over Orinda formation bedrock with Vs measurements between 844 and 1,959 feet/sec. A comparison of the ICEC 1995 Vs profile and the Vs profile developed from this geotechnical study is shown in Figure 3-1.

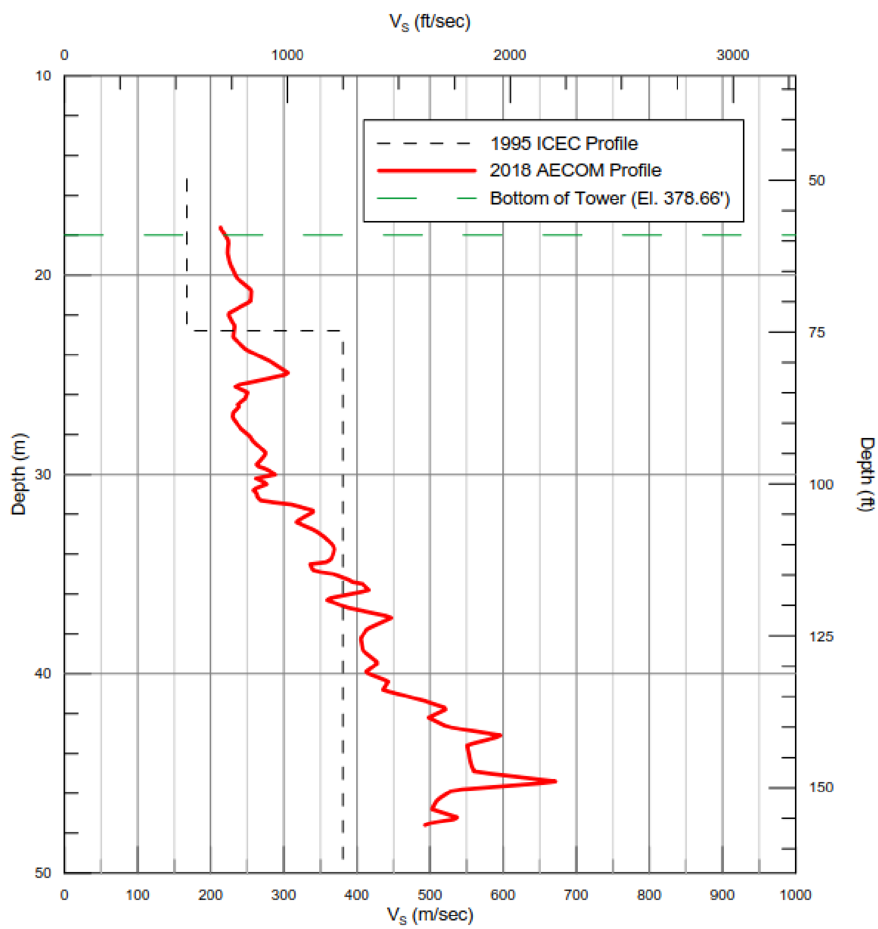


Figure 3-1 Comparison of ICEC 1995 Vs and Measured Vs Profile

The findings of the geotechnical investigation can be summarized as follows:

- The alluvium/bedrock contact was encountered deeper than ICEC assumed.
- The materials surrounding and supporting the Tower were found to be generally weaker than ICEC assumed, particularly in between EL 363 and EL 334 feet.
- The Vs profile in bedrock (measured in the current study) gradually increases with depth, whereas it was assumed to be constant in the 1995 ICEC profile.

For final design of the Tower retrofit alternative, AECOM will perform soil structure interaction (SSI) analysis and model the entire Tower from EL 345 feet with the measured geotechnical data. For alternatives analysis, AECOM used a stick model that only includes the superstructure of the Tower, which starts at EL 388 feet, similar to the approach used in ICEC's analysis. AECOM also used soil springs at EL 388 feet in the analysis or the existing Tower and the preliminary analysis of the alternatives. AECOM performed simplified calculation to adjust the foundation springs developed by ICEC to account for the measured Vs data. AECOM also performed SHAKE analysis for the ICEC 1995 profile and the measured profile based on current Vs measurements. The frequency of each profile is shown below in Figure 3-2. Based on the relation of f (frequency) and k (spring constant), AECOM reduced the ICEC spring constants by 33% based on the following calculations:

$$f \sim \sqrt{\frac{k}{m}} \rightarrow \frac{k_o}{k_{ICEC}} = \left(\frac{f_o}{f_{ICEC}}\right)^2 \rightarrow \frac{k_o}{k_{ICEC}} = \left(\frac{3.1}{3.8}\right)^2 \rightarrow k_o = 0.67k_{ICEC}$$

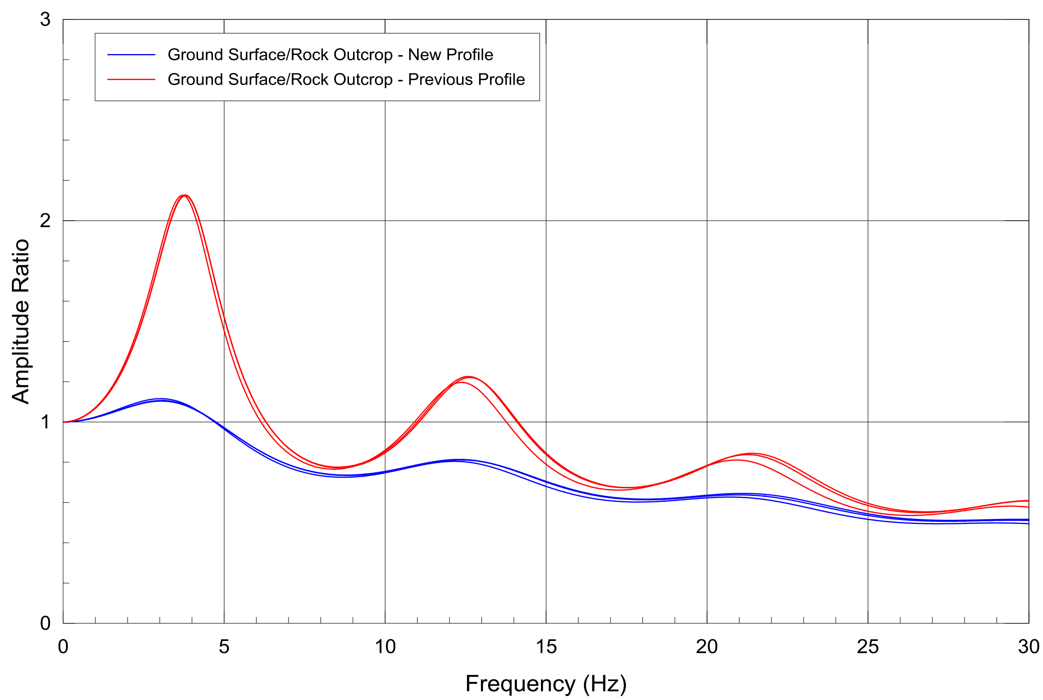


Figure 3-2 Comparison of Frequencies of ICEC Vs Profile and Measured Vs Profile

ICEC's analysis also calculated the ground motions at EL 388 feet and developed scaling factors for ground motions. These factors accounted for the ratio of the motion at the base of the Tower to the rock outcrop motion to approximate the SSI effect of the buried structure. For input ground motions, the Fault Normal (FN) direction is parallel to the global X axis in AECOM's computer model of the Tower and the Fault Parallel (FP) direction is parallel to the global Y axis. The computed scaling factors were 0.85 (0.85g/1.0g) for the direction parallel to the outlet conduit (Fault Normal/Global X) and 0.65 (0.65g/1.0g) for the direction normal to the conduit (Fault Parallel/Global Y). These values were also used in AECOM's analysis. Figure 5-6, in Section 5.7, shows the response spectrum curves in both X and Y directions, taking into account the scaling factors of 0.85 and 0.65.

Table 3-1 Spring Parameters at EL 388 feet with 0.67 factor

Direction	Stiffness Constants
X Translation	2.8×10^5 kip/ft
Y Translation	2.0×10^5 kip/ft
Z Translation	4.5×10^5 kip/ft
X Rotation	1.9×10^8 kip-ft/rad
Y Rotation	1.1×10^8 kip-ft/rad

A description of these spring parameters as boundary conditions is described in Section 5.4. Figure 3-3 and Figure 3-4 illustrate the model developed for ICEC's foundation system in plan and elevation views, respectively. This model was developed to calculate the coefficients in Table 3-1 above as well as the ground motion scaling factors of 0.85/0.65. Figure 3-5 shows the overall layout of the SASSI seismic model of the Tower and foundation structures from ICEC's analysis.

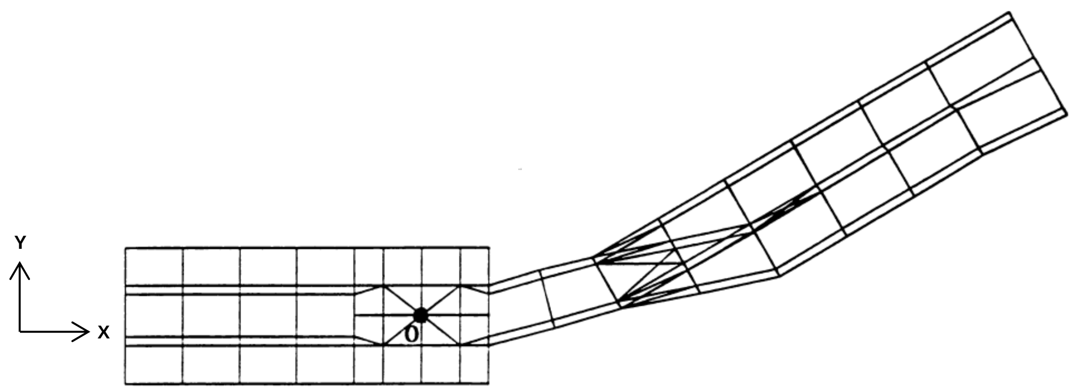


Figure 3-3 Plan View of SASSI Tower's Foundation Model in the X-Y Plane (ICEC, 1995)

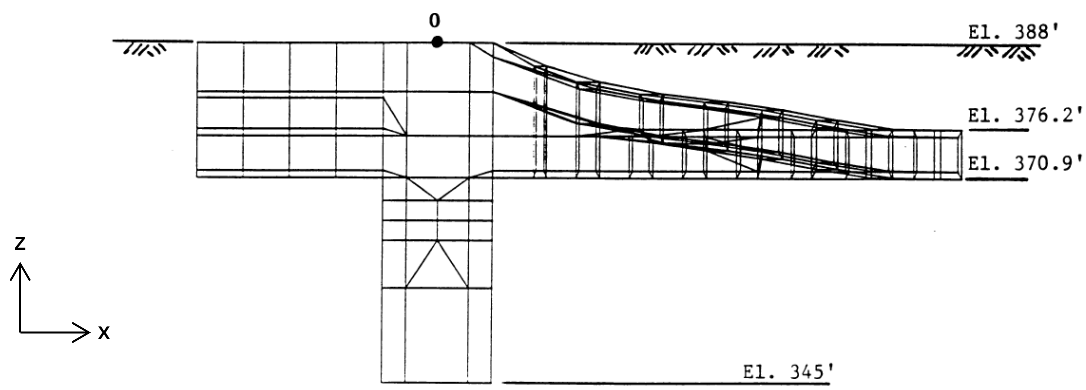


Figure 3-4 Elevation View of SASSI Tower's Foundation Model in the X-Z Plane (ICEC, 1995)

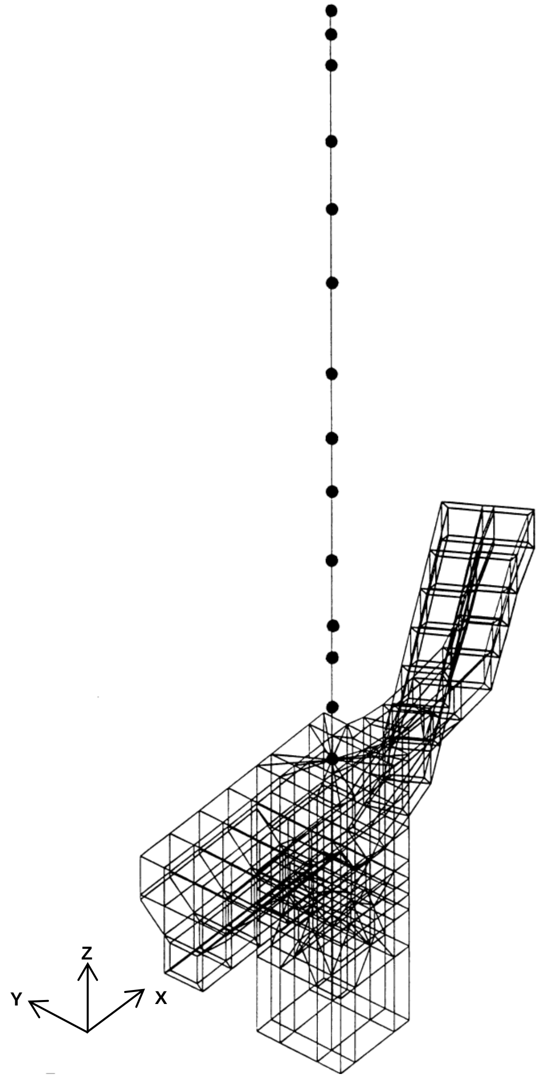


Figure 3-5 SASSI Model Evaluated by ICEC (ICEC, 1995)

4. Seismicity and Ground Motions

This section describes the Deterministic Seismic Hazard Analysis (DSHA) performed by AECOM under the scope of work of this Project. The results of the over-the-water borehole adjacent to the Tower described in Section 3 and illustrated in Figure 3-1 obtained a site-specific Vs30 of 320 m/sec.

Shear wave velocity data was acquired using the PS-wave suspension logging method. Vs30 is the time-averaged shear-wave velocity to 30-meter depth. The Vs profile is shown in Figure 3-1. To calculate the Vs30, the travel time for each layer for which there is a Vs measurement was calculated. Then the total thickness (30 m) is divided by the sum of the travel times to obtain the time-average shear-wave velocity for 30 m (or Vs30).

For structural analysis of the retrofit alternatives, an 84th percentile 5% damped horizontal acceleration response spectrum for design was developed for a moment magnitude (**M**) 7.25 event on the Hayward Fault at a rupture distance of 8.8 km using the NGA-West2 ground motion models. Additional input parameters are provided in Table 4-1. AECOM's technical memorandum describing the DSHA conducted for the Lafayette Tower is included in Appendix F.

Because the Tower is located at near-field distances of the Hayward Fault, forward directivity effects were incorporated to calculate the fault normal and fault parallel spectra. The 84th-percentile spectra adjusted for fault normal and fault parallel direction are listed in Table 4-2 and shown in Figure 4-1. The Fault Normal (FN) direction is nearly parallel to the conduit (and corresponds with the global X axis in the computer model), and the Fault Parallel (FP) direction is nearly perpendicular to the conduit (and corresponds with the global Y axis in the computer model).

Table 4-1 Parameters* used in DSHA for Hayward Fault

	Hayward Fault
M	7.25
Rupture Distance (km)	8.8
Joyner-Boore Distance (km)	8.8
R_X (km)	8.8
Sense of Slip	Right Lateral Strike-Slip
Z_{TOR} (km)	0
Dip angle of rupture plane (degrees)	90
Hanging Wall	No
$Z_{1.0}$ (km)	0.44
$Z_{2.5}$ (km)	1.63
Z_{hyp} (km)	default
W (km)	12
Vs30 (m/sec)	320

* Parameters are defined in Acronyms as well as in Appendix F

Table 4-2 84th-Percentile Horizontal Acceleration Response Spectra adjusted for Rupture Directivity

Period (sec)	**Fault Parallel**	**Fault Normal**
	SA (g)	
0.010	0.61	0.61
0.020	0.62	0.62
0.030	0.64	0.64
0.050	0.72	0.72
0.075	0.87	0.87
0.100	1.01	1.01
0.150	1.24	1.24
0.200	1.38	1.38
0.250	1.47	1.47
0.300	1.52	1.52
0.400	1.50	1.50
0.500	1.42	1.42
0.750	1.13	1.25
1.000	0.92	1.05
1.500	0.63	0.72
2.000	0.46	0.56
3.000	0.30	0.38
4.000	0.21	0.27
5.000	0.15	0.20
7.500	0.074	0.10
10.000	0.042	0.057

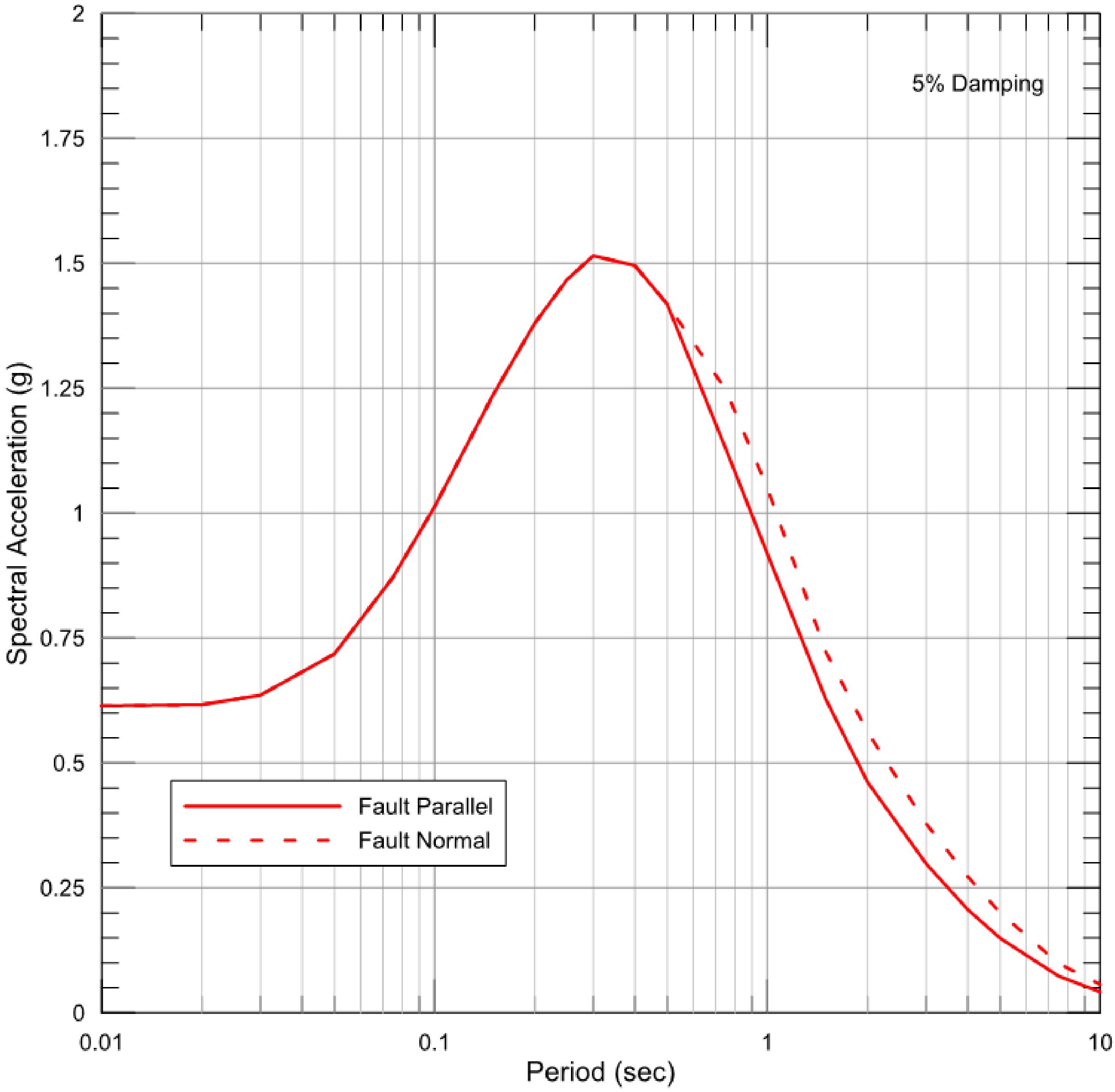


Figure 4-1 84th-Percentile Response Spectrum Adjusted for Directivity

In addition to the Hayward fault, other local faults were examined for the Lafayette Reservoir Outlet Tower that could potentially contribute to the seismic hazard based on their distance from the Outlet Tower and maximum magnitude. AECOM reviewed the data and developed deterministic response spectra for the Franklin fault, Contra Costa-Lafayette fault, Contra Costa Shear Zone Connector fault and the Moraga fault. The geometry for the faults was taken from the Third Uniform California Earthquake Rupture Forecast model (UCERF3, Field *et al.*, 2013), except the Moraga fault, which was not included in UCERF3, though is considered active by DSOD criteria. Maximum magnitudes were developed using the magnitude-area relationships utilized in UCERF3 (Field *et al.*, 2013). All input parameters are listed in Table 4-3 below.

Table 4-3 Fault Parameters used in DSHA for Additional Local Faults

	Hayward Fault	Moraga	Franklin	Contra Costa - Lafayette	Contra Costa Shear Zone Connector
M	7.25	6.75	6.8	6.2	6.7
Rupture Distance (km)	8.8	3.7	6.25	3.2	4.0
Joyner-Boore Distance (km)	8.8	3.7	6.25	3.2	4.0
R_X (km)	8.8	3.7	6.25	3.2	4.0
Sense of Slip	Right Lateral Strike-Slip	Reverse	Strike-Slip	Strike-Slip	Strike-Slip
Z_{TOR} (km)	0	0	0	0	0
Dip angle of rupture plane (degrees)	90	68	90	90	81
Hanging Wall	No	No	No	No	No
$Z_{1.0}$ (km)	0.44	0.44	0.44	0.44	0.44
$Z_{2.5}$ (km)	1.63	1.63	1.63	1.63	1.63
Z_{hyp} (km)	default	default	default	default	default
W (km)	12	320	320	320	320
Vs30 (m/sec)	320	0.44	0.44	0.44	0.44

Based on the DSOD Hazard Consequences Matrix (2018), for moderate slip-rate faults (1.0 to 0.01 mm/yr) and an extremely high hazard dam, the 67th to 84th percentile spectra can be selected. AECOM chose the 67th percentile for all faults, except the Hayward fault, due to the low slip rates and limited evidence for latest Quaternary activity.

As shown on Figure 4-2, the Hayward fault 84th percentile controls over most spectral periods. The Moraga fault 67th percentile controls at short periods (PGA) and at periods of 0.25 - 0.4 sec, but by no more than 3%. Considering that the Moraga fault is not included in the UCERF3 model, and a full rupture of the fault is considered unlikely (URS, 2011), a **M** 6.75 could be considered very conservative. For example, a **M** 6.25 was used for the Miller Creek fault in previous studies (URS, 2011) and the Thrust Fault Subgroup (1999) estimated that the Moraga, Miller Creek and Palomares faults are capable of generating earthquakes ranging in magnitude from about **M** 5.5 to **M** 6.5.

For the alternative analysis study, the Hayward fault 84th percentile spectrum is considered appropriate for the seismic analysis.

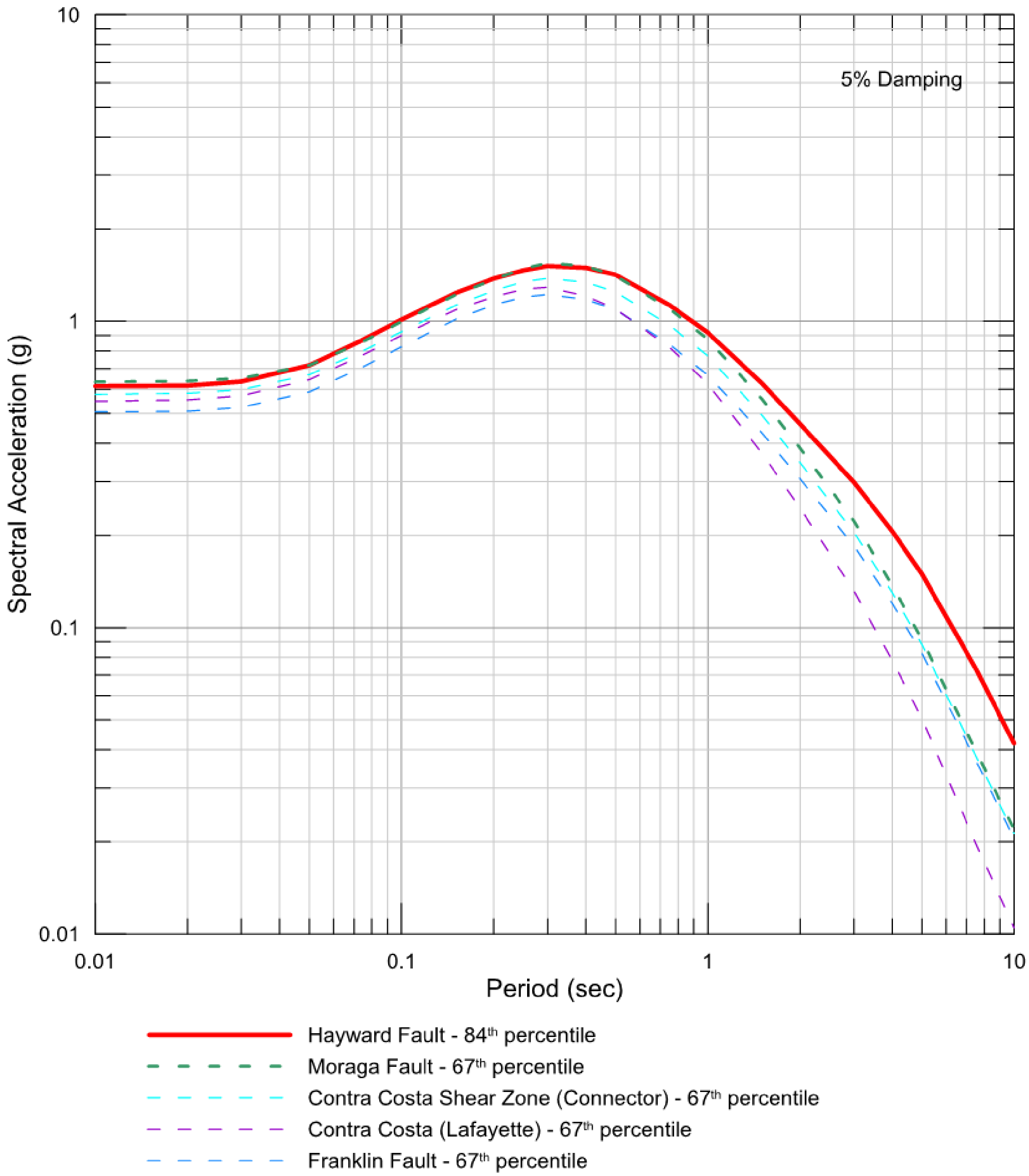


Figure 4-2 Comparison of Response Spectra for Hayward and Other Local Faults near Lafayette Reservoir

5. Tower Seismic Performance

5.1 Tower Seismic Deficiencies

The previous analyses listed in Section 2.3 concluded that the Tower cannot withstand the MCE loading. Internal forces in the Tower in the form of moment and shear demands from the MCE seismic loading are expected to exceed the Tower section capacities, but the predicted extent and severity of damage was different in each of the previous studies. AECOM attributes this variation in results to different assumptions in material properties, modeling, boundary conditions, load combinations, and shear and moment capacity calculations (refer to Table 5-1). For the purposes of the retrofit alternative selection, AECOM also performed an analysis of the Tower, and the assumptions made are also summarized in Table 5-1 and in the following sections.

In the following sections, the assumptions about modeling, material properties, boundary conditions, load combinations, and shear and moment capacity calculations used in the AECOM analysis are presented and compared with the previous studies. Similar to the previous studies, the AECOM analysis predicts that the Tower will experience high moment demands around the spillway elevation at EL 450, extending about 20 to 30 feet above and about 10 to 20 feet below the spillway elevation. High shear demands are predicted in the Tower portion just below the water surface. However, the high shear demands are only marginally at or above capacity. The retrofit must address the high moment demands; but for the high shear demands, a refined analysis is recommended during the design phase of the Project. For the purposes of the alternatives evaluation, shear reinforcement is assumed to be necessary.

In addition to the internal force (moment and shear) evaluation, the 2011 report by DSOD discussed overturning stability of the Tower, i.e., the potential for the Tower to rotate around its base and topple over. An analysis completed by the District in 2003 and submitted to DSOD assumed that the rock and soil around the embedded portion of the Tower between EL 345 to 363 feet would yield during a large earthquake. This could allow the Tower to rock on its base and increase the period while reducing the seismic loading. The EBMUD 2003 analysis also posited that the Tower would be able to resist the moment demands of a free-rocking condition, concluding that the Tower was incapable of overturning (DSOD, 2011). AECOM evaluated the buried portion of the Tower in LPILE (Ensoft Inc., 2018) to assess whether the surrounding soil/rock is expected to yield under seismic loads. AECOM also performed an analysis to assess the potential for overturning of the Tower in the event of failure of the surrounding soil and rock. The results are discussed in Section 5.9.

5.2 Analysis Method, Modeling, and Assumptions

For the purposes of alternatives analysis, AECOM calculated the internal forces in the Tower using a dynamic response-spectrum analysis (RSA) of a stick model of the Tower. The Acceleration Response Spectrum (ARS) curve used in the analysis to represent the MCE is described in Section 4 and shown in Figure 4-1. The input ground motions were adjusted by scaling factors of 0.85 (Fault Normal/Global X) and 0.65 (Fault Parallel/Global Y) based on the geotechnical review presented in Section 3. After alternative selection, AECOM will refine the analysis, and may use a more detailed model and analysis method to analyze the final design of the retrofit during the next phase of the Project.

AECOM used the general-purpose finite element (FE) program SAP2000 (CSI, 2017) to perform the analysis. The Tower was modeled from the top of the foundation at EL 388 feet to the top of the shaft (or the bottom of the operating house) at EL 500 feet as a series of frame elements. The operating house was modeled as an added mass at the top of the model. The frame element sections represent the cross sections of the Tower along the height. The stick model was analyzed in 3D space using 6 degrees of freedom at each node. Figure 5-2 shows an elevation view of the model. The extruded view is also shown to provide a proportional rendered view of the model. To account for stiffness degradation due to cracking during an earthquake, a reduced moment of inertia was used, as described in Section 5.6, below. The global X and Y coordinates are oriented such that the outlet conduit pipe is parallel to the X axis.

The various cross sections and their properties throughout the length of the Lafayette Outlet Tower were modeled using the “Section Designer” module in SAP2000. The SAP model includes 24 joints and 23 frames, with a total of 11 different section types reflecting the various geometries throughout the length of the Tower. These include the

Tower shaft thickness, vertical reinforcement layout, and interior partition wall rotations, based on drawings DH 1064-7 and DH 1065-7 (EBMUD, 1929), and 5450-G-2 (EBMUD, 1967). Figure 5-1 shows the typical sections of the Tower in SAP2000.

In the AECOM SAP model, the soil structure interaction (SSI) effects on the bottom part of the Tower were accounted for via the use of soil springs at the mudline at EL 388 feet. These soil springs account for the embedded portion of the Tower and the SSI effects between the embedded portion and surrounding soil. In the AECOM model, the soil springs were derived from the ICEC report (1995) and updated based on measured data from the geotechnical boring as described in Section 3.

The internal force demands were compared to the Tower's section capacity and demand-to-capacity ratios (DCRs) were calculated to measure the ability of the Tower to resist these demands. If the DCR value exceeds 1.0, the demand exceeds capacity. Acceptable values of the DCRs are determined based on the desired performance and the level of ductility that can be allowed. As described below, moment DCRs can be allowed to exceed 1.0 and acceptable behavior can be expected with moment DCRs of 2.0. Acceptable DCRs for shear cannot exceed 1.0.

The following sections outline the various assumptions made about the Tower's properties, in comparison with the properties used by ICEC (1995), DSOD (2011), MJA (2015), and TERRA/COWI (2017). The properties and assumptions used in these analyses are summarized in Table 5-1.

Table 5-1 Comparison of Assumptions and Analysis Methods of the Tower

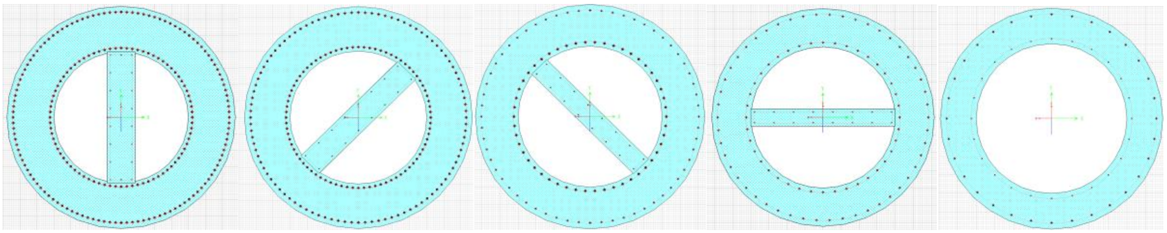
		AECOM (current study)	ICEC (1995)	MJA (2015)	DSOD (2011)	TERRA/COWI (2017)
Design Code		USACE EM 1110-2-6053 (2007)		USACE EM 1110-2-2400 (2003a)	USACE EM 1110-2-2400 (2003a)	
Material Properties • Concrete • Steel		• Static f'c = 4030 psi, cracked (I_E = [0.35Ig ~ 0.8Ig]) • Dynamic f'c = 4634 psi • fy = 33 ksi	• f'c = 2000 psi, uncracked (I_E = Ig) • No dynamic factor used • fy = 33 ksi	• f'c = 4000 psi, uncracked (I_E = Ig) • No dynamic factor used • fy = 33 ksi	• f'c = 2000 psi, uncracked (I_E = Ig) • No dynamic factor used • fy = 33 ksi	• f'c = 2000 psi, uncracked (I_E = Ig) • No dynamic factor used • fy = 33 ksi
FE modeling	Foundation substructure and Tower superstructure	No SASSI modeling of foundation Stick model from EL 388' to EL 500'	Base substructure modeled with SASSI Stick model from EL 388' to EL 509'	No SASSI modeling Stick model from EL 345' to EL 500' (substructure included)	No SASSI modeling Stick model from EL 345' to EL 500' (substructure included)	No SASSI modeling, stick model only Stick model from EL 345' to EL 500' (substructure included)
	Boundary conditions at base	Soil springs only	Soil springs only	Fixed base at EL 345'	Fixed base at EL 345'	Fixed base at EL 345'
	Soil springs	Springs adapted from ICEC assigned at EL 388', updated based on geotechnical boring data	Equivalent soil spring values from SASSI output assigned at EL 388' (see Table 3-1)	ICEC values assigned at EL 384'	ICEC values assigned at EL 378'	Soil springs added at EL 384'
	Gatehouse	Added point load of 71 kips at El 500'	Modeled as frame elements in stick model, corresponded to equivalent 71 kips	Added point load of 71 kips at El 500'	Added point load of 71 kips at El 500'	Added point load of 71 kips at El 500'
	Valves	Modeled as 3 kip point loads each	Not included	Not included	Not included	Not included
	Partition walls	Included in sections for shear capacity calculations. This will be investigated in more detail in the next phase.	Not included	Not included	Not included	Not included
Load combination		1. U = D + Ex + 0.4Ey 2. U = D + 0.4Ex + Ey	1. U = D + Eh + 0.4Ez Eh = max horizontal seismic load Ez = vertical seismic load	1. U = D + 1.1(Ex + 0.4Ey)/R 2. U = D + 1.1(0.4Ex + Ey)/R R = 1 See Note 1.	1. U = D + 1.1E/R E = max horizontal seismic load R = 2	(Not stated in TERRA/COWI report)
Seismic Input		(See Section 4) • Hayward fault • 84th percentile spectra • PGA = 0.61g • Envelope of Vs = 370, 420, and 470 m/s	• Calaveras fault • 84th percentile spectra • PGA = 0.65g • Rock Vs = 1,250 ft/s = 381 m/s	• Hayward fault • 84th percentile spectra • PGA = 0.66g • Vs = 392 m/s • Maximum rotated	Time History analysis • 84th percentile NGA motion • Corresponding PGA = 0.5g	Time History analysis • 84th percentile spectra • Corresponding PGA = 0.65g

Table 5-1 Comparison of Assumptions and Analysis Methods of the Tower (Cont.)

	AECOM (current study)	ICEC (1995)	MJA (2015)	DSOD (2011)	TERRA/COWI (2017)
	• Maximum rotated component included • 0.85 (X) and 0.65 (Y) scaling factors included (Section 3)	• 0.85g/1.0g (X) and 0.65g/1.0g (Y) scaling factors for scattering effects	component included		
Shear capacity	Per EM 6053: $V_c = 2\left[k + \frac{P}{2000A_g}\right]\left(\sqrt{f'_{ca}}\right)A_e$ **k = 0.5 to 1.0** (depends on moment DCR) f'ca = 4000 psi $V_s = \frac{\pi A_h f_y (0.8d)}{2s}$	Vn = Vc + Vs See Note 2.	Per EM 2400: $V_c = 2\left[k + \frac{P}{2000A_g}\right]\left(\sqrt{f'_{ca}}\right)A_e$ **k = 1 (constant)** f'ca = 4000 psi $V_s = \frac{\pi A_h f_y (0.8d)}{2s}$	See Note 3.	Used DSOD (2011) capacities
Moment capacity	P-M curves from SAP2000 Moment phi factor = 0.9	P-M curves from YIELD computer program Moment phi factor = 0.9	P-M curves from SAP2000 Moment phi factor = 0.9	"Per EM 2400"	Used DSOD (2011) capacities
Acceptance Criteria: Maximum allowable DCR	• Moment: **2.0** • Shear: 1.0	• Moment: 1.0 • Shear: 1.0	• Moment: 1.0 • Shear: 1.0	• Moment: 1.0 • Shear: 1.0	• Moment: 1.0 • Shear: 1.0

Notes:

1. Load combination for MJA not explicitly stated in report text; assumed from design code
2. Formulas and parameters used for shear capacity of concrete and steel reinforcement not stated in ICEC report. Report only indicates that shear strengths are calculated in accordance with ACI 318-89
3. Formulas and parameters used for shear capacity of concrete and steel reinforcement, as well as moment capacities, not stated in DSOD report. Report only indicates that shear strengths and moment capacities are calculated in accordance with EM 1110-2-2400. Based on the shear capacities provided, the evaluation likely uses the same formula and assumptions as MJA with f'c = 2,000 psi



Section 5 Section 6 Section 9 Section 11 Section 13

Figure 5-1 Typical Tower Sections

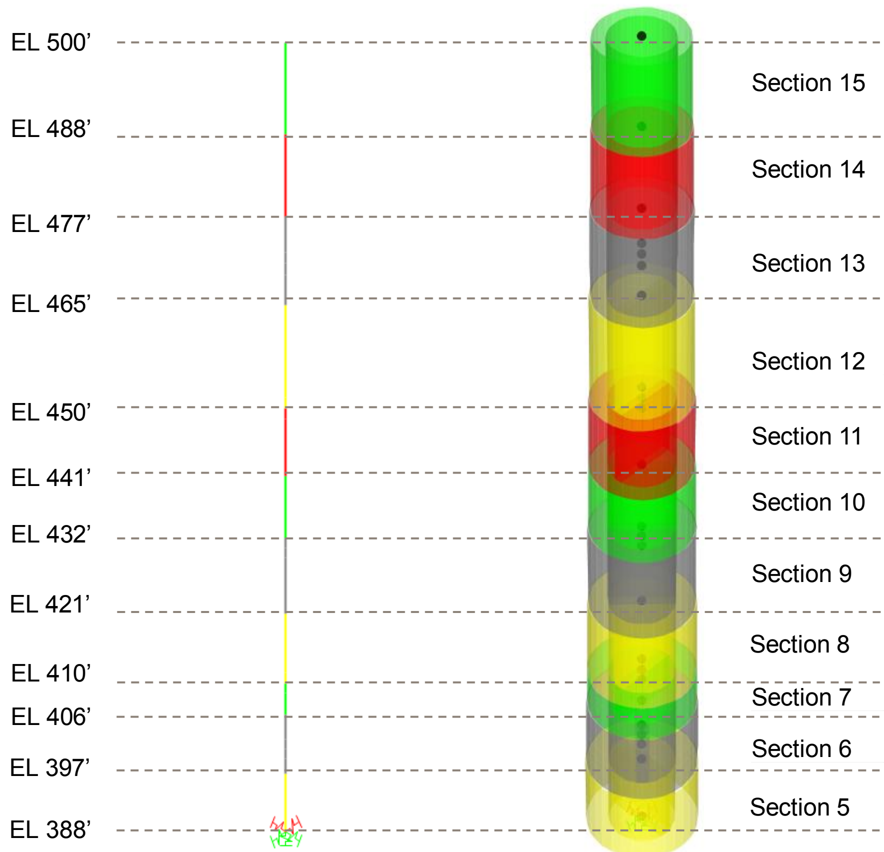


Figure 5-2 1D Stick and 3D Extruded SAP Model of Lafayette Tower (existing condition)

5.3 Material Properties

The concrete compressive strength (f'c) and elastic modulus (Ec) used by AECOM in this report in the analysis of the Tower and the retrofit alternatives are based on results of in-situ testing of the Tower concrete. In that regard, AECOM conducted concrete testing in August 2018 by extracting nine (9) concrete core samples from the exterior face of the Tower at three elevations above the water level – approximately EL 444, 464, and 484 feet. The extracted cores were then tested for compressive strength and modulus of elasticity. All tests were conducted in accordance with ASTM C469 Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression (ASTM, 2014), and ASTM C42 Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete (ASTM, 2018). Table 5-2 below summarizes the concrete testing results. The full report submitted by AECOM's sub-consultant Inspection Services, Inc. (ISI) is attached in Appendix G.

Table 5-2 Compressive Strengths of Concrete Cores

Core	Sample Elevation (ft)	Compressive Strength (psi)	Average Compressive Strength per EL (psi)
A	444	*	6,950
B	444	6,790	
C	444	7,110	
D	464	3,850	3,770
E	464	3,840	
F	464	3,610	
G	484	5,380	4,630
H	484	4,390	
I	484	4,110	

* No data available due to shearing of sample

An equivalent concrete strength from the concrete testing data was calculated in accordance with ACI 214.4R-10 Guide for Obtaining Cores and Interpreting Compressive Strength Results (ACI, 2010). The calculations are provided in Appendix G. Additionally, the concrete strength and elastic modulus were increased based on the relationship between static and dynamic properties in accordance with EM 1110-2-6053 Section 5-1d (USACE, 2007). These factors account for the effect of increase in material properties because of the dynamic nature of the earthquake loading. According to EM 1110-2-6053, the static compressive strength and elastic modulus are multiplied by a factor of 1.15 to obtain the dynamic compressive strength and elastic modulus. The static shear strength is multiplied by 1.1 to obtain the dynamic shear strength. The existing reinforcing steel's tensile strength (*fy*) is identified from the District's Specification No. 15 as A15-14 billet steel bars (EBMUD, 1927).

- Tower concrete
  - Density = 150 pounds per cubic feet (pcf)
  - Static f'_c = 4,030 pounds per square inch (psi)
  - Static E_c = 3,618,500 psi
  - Dynamic f'_c = 4,634 psi
  - Dynamic E_c = 4,161,000 psi
- Reinforcing Steel
  - Density = 490 pcf
  - f_y = 33 kilo-pounds (kips) per square inch (ksi)
  - E_s = 29,000 ksi
- Water
  - Density = 62.4 pcf

MJA's evaluation (2015) assumed a concrete compressive strength of 4,000 psi. Both the DSOD (2011) and ICEC (1995) studies apparently assumed a concrete compressive strength of 2,000 psi, based on as-built drawings of the Tower (EBMUD Drawing No. DH 1065-7, 1929).

5.4 Boundary Conditions

For the purposes of alternatives evaluation, the rock/soil spring stiffness parameters in the horizontal and vertical directions were adapted from ICEC (1995) and used in this analysis as described in Section 3 of this report. The values of the spring stiffnesses represent the effects of the soil/rock-structure interaction of the substructure (embedded Tower, foundation, and soil system) at EL 388 feet. The values of the foundation springs were scaled based on the ratio of the dominant frequencies of the ICEC Vs profile and the measured Vs profile. To be consistent with the assumptions made in ICEC's SASSI analyses, the set of five soil springs are attached to the bottom (EL 388 feet) of the stick model of the Tower where the foundation impedances and scattered foundation input motions were calculated (ICEC, 1995). The directions of the stiffness coefficients are the horizontal X (parallel to outlet conduit) and Y (normal to outlet conduit) directions, the vertical Z direction, and rocking about the X and Y directions.

5.5 Section Properties and Capacities

5.5.1 Shear Capacity

Section shear capacities of the Tower were calculated in accordance with equation (5-1) of Engineer Manual (EM) 1110-2-6053 (USACE, 2007):

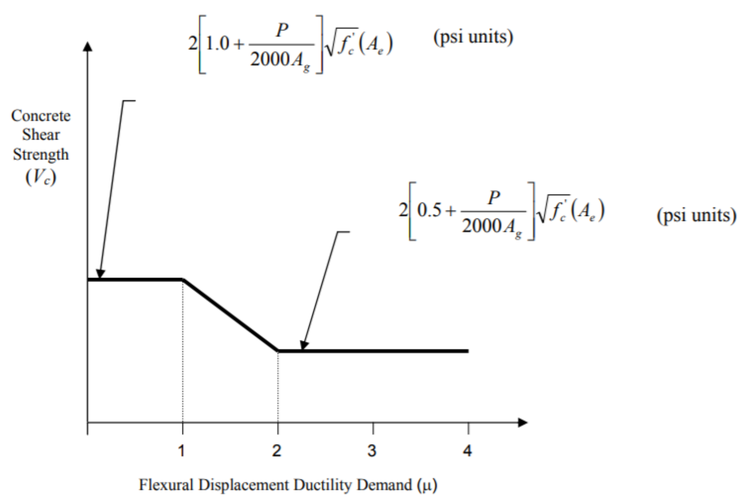
$$V_u = \phi(V_c + V_s)$$

where
ϕ = capacity reduction factor for shear = 0.85.

The concrete shear strength is increased by a factor of 1.1 based on the relationship between the dynamic shear strength to static shear strength provided by EM 1110-2-6053 (USACE, 2007). The concrete shear capacity (including the 1.1 factor for dynamic shear strength) is:

$$V_c = 2\left[k + \frac{P}{2000A_g}\right]\left(\sqrt{f'_{ca}}\right)A_e * 1.1$$

where
k = function of moment DCR, between 0.5 and 1; as shown in the following figure



(USACE, 2007)

f'_{ca} = actual concrete compressive strength = 4,030 psi (actual static strength based on core test results)
A_g = gross area, cross section area of the Tower, including inner partition walls
A_e = effective cross section area, defined as $0.8A_g$.

The contribution to the shear reinforcement is provided by:

$$V_s = \frac{\pi A_h f_y (0.8d)}{2s}$$

where,
A_h = horizontal reinforcement cross section
d = outside diameter of the Tower
s = spacing of reinforcement.

The calculated shear capacities of the Tower shaft are compared to those of the previous studies in Table 5-3 and are shown graphically in Figure 5-3 below. The capacities calculated for the current evaluation are lower than those calculated by DSOD (2011) and MJA (2017) because their analyses used EM 1110-2-2400 (USACE, 2003a) to calculate the shear capacity. In EM 1110-2-2400, the k value used to calculate the concrete shear capacity is a constant value of k = 1.0 or k = 0.5. It appears that both DSOD (2011) and MJA (2017) used k = 1.0. The shear capacities used in ICEC's study are much lower, because ICEC used ACI 318-89 (ACI, 1989) as its design criteria, which uses a lower shear reduction factor of 0.75. ICEC's assumption of 2,000 psi concrete strength also partially accounts for the lower shear capacity.

AECOM's calculated shear capacity is based on EM 1110-2-6053, which takes into account the moment DCR at the section and modifies the shear capacity accordingly (k ranges between 0.5 and 1.0 as shown in the graph above). According to this approach, when a high moment demand is expected at the section, the allowable shear capacity is reduced. This explains the drop in the shear capacity toward the middle of the Tower in AECOM's assessment. The rationale behind this approach in EM 1110-2-6053 in dealing with shear is to allow for ductile moment behavior while limiting brittle shear behavior.

Table 5-3 Summary and Comparison of Shear Capacities along Tower Height

Section[1]	Elevation	Calculated Section Shear Capacity (kips)			
		AECOM (current study) (f'c = 4,030 psi)[2]	MJA (f'c = 4,000 psi)	DSOD (f'c = 2,000 psi)	ICEC (f'c = 2,000 psi)
-	500	-	848	764	310
15	488	998	912	-	-
12	450	800	1,144	1,107	420
10	432	962	1,252	1,099	490
8	410	1,237	1,432	1,252	590
6	397	1,583	1,575	1,304	650
5	384	1,643	1,662	1,449	730

1. Corresponding section in AECOM FE model
2. Shear capacity listed is the dynamic shear capacity, which is equal to 1.1 times the static shear capacity

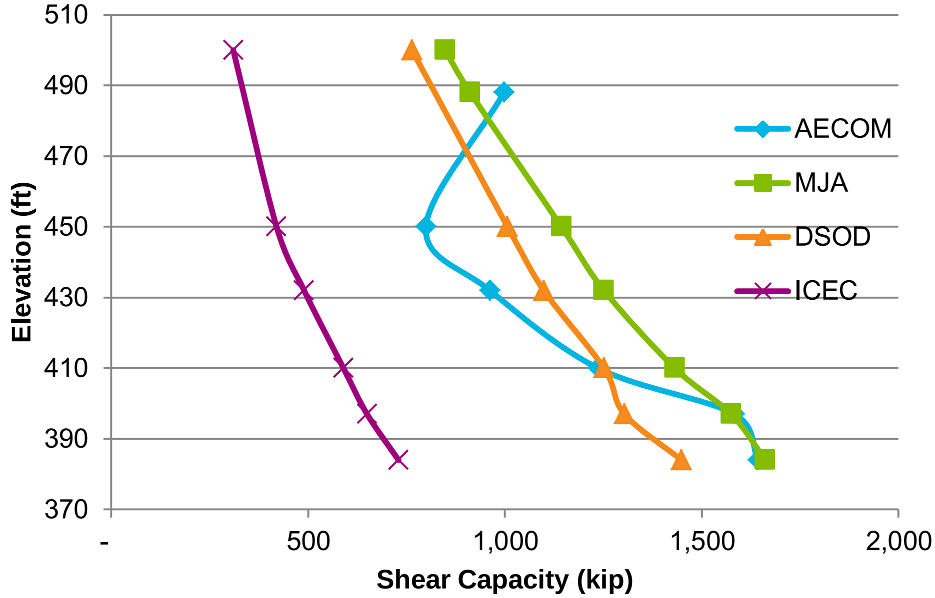


Figure 5-3 Comparison of Shear Capacities along Tower Height

5.5.2 Moment Capacity

Moment interaction curves were generated for the Tower sections using the SAP2000 Section Designer module. Strength reduction factors of 0.9 for moment and 0.65 for axial (compression) were used to generate the moment interaction (P-M) diagrams, per ACI 318-14 (ACI, 2014). The inner partition walls were neglected in the analysis of moment capacities. If fully effective, the additional partition walls would only increase the moment capacity for a given section by between 1 and 6 percent. Therefore, it was conservatively assumed that the partition walls do not contribute to the flexural strength of the Tower. Because the Tower is nearly symmetric, the nominal moment capacity in all directions is nearly identical. For all sections, the axial load is very small with respect to the moment-interaction diagram; at the base of the model, the maximum dead load is 1,380 kips. Figure 5-4 shows a typical moment interaction diagram. Moment interaction diagrams for Tower sections along the height are provided in Appendix C.

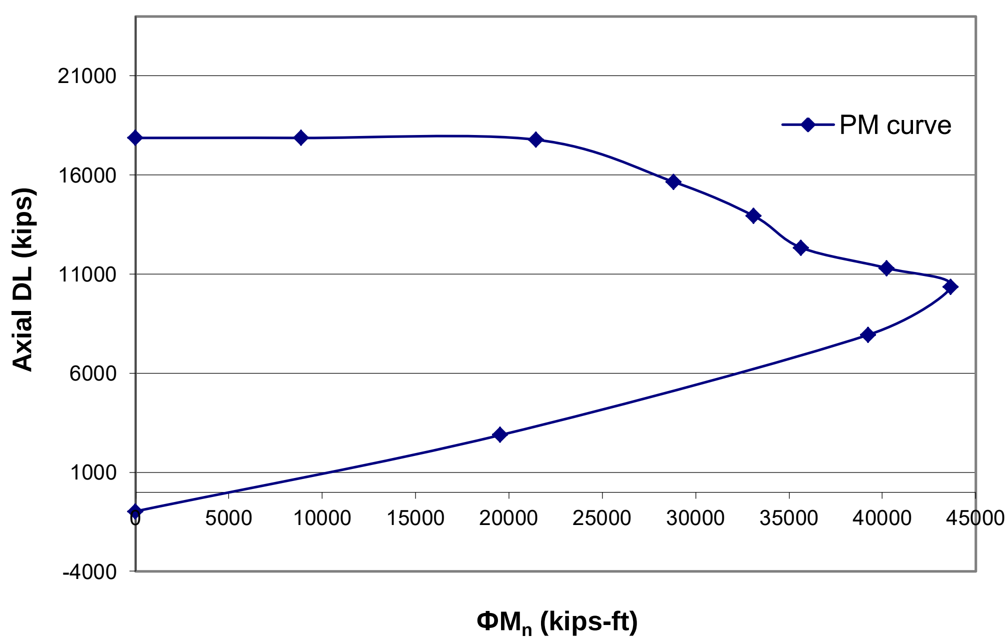


Figure 5-4 A Typical Moment Interaction Curve (for Section 13 at EL465 feet)

The calculated moment capacities of the Tower shaft are compared to the previous studies in Table 5-4, as well as graphically in Figure 5-5 below. Both show good comparison between AECOM's capacities and all other previous studies because moment capacities are not as sensitive to the analysis assumptions as shear capacities.

Table 5-4 Summary and Comparison of Moment Capacities along Tower Height

Section(1)	Elevation	Calculated Section Moment Capacity (kip-ft)			
		AECOM (current study) (f'c = 4,634 psi) (2)	MJA (f'c = 4,000 psi)	DSOD (f'c = 2,000 psi)	ICEC (f'c = 2,000 psi)
-	500	-	2,952	2,900	2,900
15	488	3,456	3,869	-	-
12	450	10,420	10,241	9,100	9,100
10	432	19,009	18,556	15,600	15,600
8	410	32,608	33,359	27,100	27,100
6	397	49,378	48,360	40,900	40,900
5	384	55,589	54,820	46,600	46,600

1. Corresponding section in AECOM FE model
2. Dynamic compressive strength per EM 1110-2-6053, obtained as 1.15 times the static compressive strength

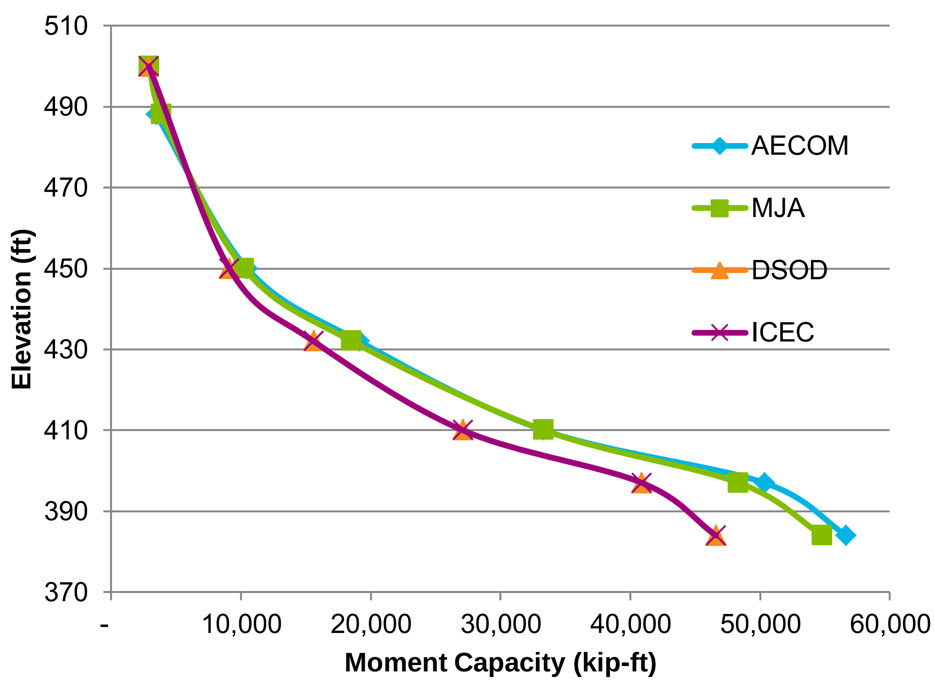


Figure 5-5 Comparison of Moment Capacities along Tower Height

5.6 Effective Moment of Inertia

During seismic shaking, the Tower concrete is expected to exhibit some level of cracking, as evidenced by the moment and shear DCRs. Cracking in the Tower will cause a reduction in stiffness. The reduced moment of inertia of the cracked structure becomes the effective moment of inertia I_E. Based on the expected seismic performance of the Tower, the ratio of the effective stiffness (I_E) to the gross stiffness (I_g) for each section was calculated in accordance with equation (4-4) of EM 1110-2-6053 (USACE, 2007):

$$\frac{I_E}{I_g} = 0.8 - 0.9\left(\frac{M_n}{M_{cr}} - 1\right)$$

where,

M_n = nominal moment capacity
M_{cr} = cracking moment

with an upper limit of 0.8 and lower limit of 0.35 for walls reinforced with grade-40 steel.

The previous studies assumed un-cracked concrete throughout the height of the Tower. However, using the effective stiffness for this study appears to be more reasonable, given the scale of the seismic loads and the level of the DCRs.

5.7 Loads and Load Combinations

The following types of loads were considered for the finite element analysis of the Tower:

- Dead load:

  Dead load is associated with the weight of all members, based on the specific weight of each member. Assumed values are outlined in Section 5.3 – Material Properties. The weight of the gatehouse was applied on the top node of the model (EL 500 feet) as an additional 71 kips (consistent with all previous models) under dead loading. The weight of each valve was applied at corresponding nodes as an assumed additional 3 kips. Self-weight and the mass of the concrete Tower walls are automatically calculated by SAP and implemented at each node.

- Hydrostatic load:

  The water elevation for the Tower was assumed at EL 450 feet. For the FE analysis of the stick model, hydrostatic loads were not included because they act in equal and opposite directions around all sides of the Tower, and therefore cancel out.

- Earthquake load:

  The Tower was evaluated for the 84^{th} percentile maximum credible earthquake, which has a peak ground acceleration (PGA) of 0.61 g, as discussed in Section 4. The acceleration response spectrum (ARS) curve used in the analysis for earthquake loading in both the X and Y directions is shown in Figure 5-6. The input ground motions were scaled by 0.85 in the X direction and 0.65 in the Y direction as the ratio of the motion at the base of the Tower to the rock outcrop motions, in order to approximate the SSI effect of the buried structure (ICEC, 1995). The analysis included loading in both X and Y directions simultaneously and the directions were combined based on the directional combinations described later in this section.

  Earthquake load includes seismic inertial load and hydrodynamic load. Seismic inertial load is automatically calculated by SAP2000 as part of the response spectrum analysis. Hydrodynamic effects of the water inside and outside the Tower were represented as added masses to corresponding joints in the SAP model, using the Goyal and Chopra (1989) method described in EM 1110-2-2400 (USACE, 2003a). In the SAP2000 model, hydrodynamic masses were added at each node. Comparison between these values and the hydrodynamic masses calculated by ICEC and MJA showed good agreement, as detailed in Table 5-5. Slight differences in added hydrodynamic masses are attributed to differences in modeling. The joints are assigned tributary masses, and therefore joints connected to longer frames elements will have higher values.

Table 5-5 Added Hydrodynamic Masses Comparison along Tower Elevation

EL (feet)	**AECOM** (current study) (kips)	**MJA (2015)** (kips)	**ICEC (1995)** (kips)
450	0	30.8	24.2
441	103.6	67.7	78.2
432	53.6	96.8	100.5
421	174.9	122.1	119.1
410	80.9	84.9	84.4
406	71	74.4	74.1
397	141.4	128.9	102.1
388	55.1	77.2	55.1
Sum Total	**680.5**	**682.8**	**637.7**

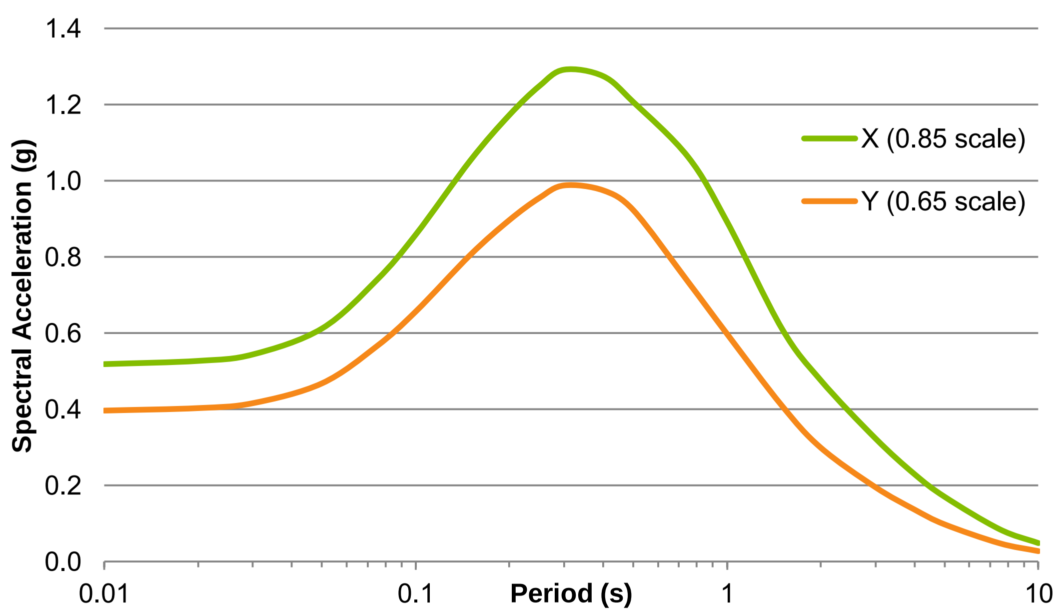


Figure 5-6 ARS Curve for 84th Percentile Loading with X and Y scaling factors

Load combinations used for the analysis were defined in accordance with USACE EM 1110-2-6053 (USACE, 2007) for the MCE as:

$$U = D + E$$

where
U = ultimate value of thrusts, shears, or moments due to the effects of dead load and earthquake load
D = internal forces from self-weight (Tower walls, valves, etc.)
E = internal forces from the MCE (includes mass acceleration due to water)

The potential for seismic loading in all directions must be evaluated. EM 1110-2-6053 provides guidance in combining the horizontal components of the response spectra to best characterize the Tower performance in a seismic event, irrespective of the direction.

Load combinations accounting for direction are: $E = \pm [E_x + \alpha E_y]$ and $E = \pm[\alpha E_x + E_y]$, where $\alpha = 0.4$ for circular Towers.

The total load combinations used for the analysis of the Tower were therefore taken as:

- $D + E_x + 0.4\ E_y$
- $D + 0.4\ E_x + E_y$

For damping, EM 1110-2-2400 recommends a value of 5 percent for the analysis of intake/outlet Towers under MCE loading (USACE, 2003a), and this value was adopted for the analysis.

5.8 Tower Modal Response

Table 5-6 lists the natural period and modal participation percentages of the first 10 significant modes in the X and Y directions of the AECOM model. Modal participation percentages for a specific mode are defined as the percentage of the structure mass excited by this mode in each direction. The modal participation percentages are indicators of the significance of each mode in the overall dynamic performance of the structure. The modal combination was performed in SAP2000 using the Complete Quadratic Combination method.

Table 5-6 Modal Analysis Summary of the Tower

Mode	Period (sec)	Frequency (Hz)	Participation Factors	
			% X	% Y
1	**0.868**	**1.15**	49	0
2	0.855	1.16	0	49
3	0.182	5.50	3	29
4	**0.180**	**5.55**	28	3
5	0.072	13.87	0	14
6*	0.070	14.38	0	0
7	**0.068**	**14.69**	13	0
8	0.043	23.12	0	4
9	0.041	24.20	5	0
10	0.027	36.97	0	1

*Mode 6 is dominant in the Z (vertical) direction

Table 5-7 shows a comparison between modal responses of the ICEC model, MJA model, DSOD model, and the current AECOM model of the Tower. Assuming a concrete strength of 2,000 psi with uncracked concrete, the period of the Tower is approximately 0.68 second, which is consistent with ICEC and MJA; but DSOD reports a longer first mode of 0.77 seconds. Assuming a concrete strength of 4,000 psi with uncracked concrete, the period of the Tower is approximately 0.58 second, which is consistent with MJA. The current AECOM evaluation uses a dynamic concrete strength of 4,634 psi with the corresponding dynamic elastic modulus of 4,161,000 psi. Additionally, the elastic modulus is reduced to account for cracking. The resulting period is approximately 0.868 seconds. The use of cracked concrete section results in a longer period. At the same time, the use of a higher concrete strength based on the actual tested concrete strength (as opposed to 2,000 psi) results in a stiffer Tower and shorter period. The net effect is still a longer period for the AECOM analysis than in previous studies.

Table 5-7 Modal Analysis Comparison with Previous Studies

Model	Period (sec)		
	Mode 1	Mode 2	Mode 3
ICEC Model (uncracked, 2000 psi)	0.667	0.161	0.067
MJA Model (uncracked, 2000 psi)	0.668	0.154	0.066
DSOD Model (uncracked, 2000 psi)	0.77	-	-
AECOM Model (uncracked, 2000 psi)	0.678	0.158	0.065
MJA Model (uncracked, 4000 psi)	0.58	0.134	0.059
AECOM Model (uncracked, 4000 psi)	0.58	0.136	0.057
AECOM Model (cracked, 4,634 psi) [used for current evaluation]	0.868	0.180	0.068

5.9 Tower Overturning Stability

Behavior of the buried portion of the Tower was evaluated using peak shear and moment demands calculated at EL 388 feet (base of the Tower) from the SAP2000 analysis. The buried portion of the Tower was modeled in LPILE as an idealized beam. LPILE p-y springs were calculated for a profile consisting of 25 feet of alluvium. The alluvium was modeled using the Stiff Clay without Free Water model with an undrained shear strength of 2,000 psf.

Using these properties, LPILE calculations indicate peak pile head displacements as large as several feet, and rotations greater than about 6 degrees. This suggests that the soil/rock surrounding the embedded portion of the Tower would yield and provide minimal lateral support to the Tower. However, the LPILE analysis includes several simplified assumptions. LPILE does not capture the effect of end-bearing at the base of the foundation. End-bearing effects may be significant because of the relatively wide aspect ratio of the buried portion of the Tower (length 43 feet, diameter 14 feet) and the large rotation. The effect of the outlet conduits connected to the base of the Tower is also not captured in the LPILE analysis. Therefore, AECOM will perform a more detailed soil-structure interaction analysis during the final design of the selected alternative.

For the current analysis, it was assumed that the soil/rock surrounding the embedded portion of the Tower will yield, and an assessment of the rotational stability (overturning stability) of the Tower under the MCE was performed by investigating whether the Tower will topple by rocking at the base (EL 345 feet). This evaluation considered an idealized single-degree-of-freedom system. This analysis uses Housner's Rigid Block Model (as shown in Figure 5-7) following the procedure in Appendix E of EM 1110-2-2400 (USACE, 2003a), which checks whether the conservation of kinetic and potential energies for slender rigid blocks is satisfied. Rocking and potentially overturning instability can occur if the overturning moment exceeds the restoring moment due to the weight of the Tower.

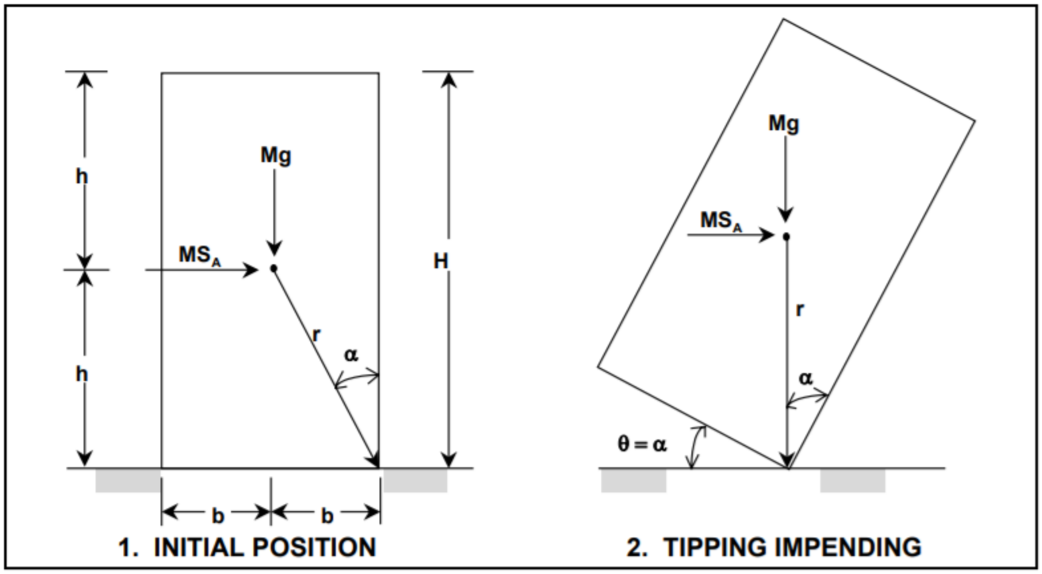


Figure 5-7 Housner's Model for Tipping of Slender Rigid Blocks (USACE, 2003a)

According to Housner, the equation for the critical angle of rotation (α_{cr}) "may be interpreted as stating that for a given spectral velocity S_V, a block that rocks through an angle α will have approximately a 50 percent probability of being overturned" (USACE, 2003a). To check for the potential of overturning of the Tower, scaling effects of the block—considering that a larger block will be more stable than a smaller block for two geometrically similar blocks—need to be taken into account.

To estimate the scaling effect for a damped structure, relationships for the pseudo-spectral velocity S_V, pseudo-spectral acceleration S_a, and the pseudo-spectral displacement S_d are used. The spectral displacement evaluation considers that if the spectral displacement is larger than one-half the base width of the Tower, overturning will likely occur. The foregoing analysis is based on the natural period of the Tower only. The fundamental period of the Tower is obtained from a stick model developed in SAP2000, from its base EL 345 feet to EL 500 feet. The total height of the rocking Tower is 155 feet, with a base width of 14 feet.

This evaluation conservatively assumes the Tower to have no cracking (I_E=Ig) because this scenario results in the lower-bound period and higher spectral acceleration. This case is also conservative because it assumes a damping of 5 percent, although the actual damping will be higher because of the surrounding soil.

Table 5-8 summarizes α_{cr} and S_d in comparison to α and half the base width. Factors of safety (FS) for each should be greater than 1.0 to indicate rotational stability. The results show that the critical angle is slightly less than α, with a FS of 1.05. This indicates that the Tower is stable against overturning but with a marginal FS. The spectral displacement S_d is much less than half the base width, with a FS greater than 11.

Table 5-8 Rocking Analysis Results for Lafayette Outlet Tower

T (sec)	α (rad)	α_{cr} (rad)	FS (α/α_{cr})	S_d (ft)	B/2 (ft)	FS ((B/2)/ S_d)
0.87	0.103	0.098	**1.05**	0.63	7.0	**11.11**

5.10 Flexural Moment Demands

Table 5-9 presents the values of the maximum moment demands, moment capacities, and Demand-Capacity Ratios (DCRs) along the height of the Tower for the current analysis. EM 1110-2-6053 allows a moment DCR of 2.0 if brittle failure modes are prevented; otherwise, the allowable moment DCR is equal to 1.0. If brittle failure modes do not exist, the section is considered to be ductile and can go through inelastic cycles during the earthquake shaking without failure. An allowable moment DCR of 2.0 is equivalent to reducing the DCR by a factor of 2.0. Section 5-12 summarizes the brittle failure mode evaluation and identifies the Tower sections that will have this potential. The allowable DCR for these sections is 1.0 as shown in Table 5-9. The same assumption was used in DSOD (2011), which describes the expected ductile moment performance and uses the R-factor to reduce the moment demands. DSOD goes further to describe that using an R-factor of 2.0 is conservative because ductility of the system would warrant an even higher R-factor most likely in the range of 4 to 6, further reducing the moment demands. Table 5-9 lists the “Effective DCR”, which is the ratio between the DCR and the allowable DCR based on EM 1110-2-6053 (1.0 for brittle sections and 2.0 for ductile sections). It is noted that ICEC (1995) and MJA (2015) did not use an R-factor.

The results indicate that the most critical sections of the Tower for flexure are between EL 440 to 480 feet, which is similar to the previous studies. SAP2000 outputs moment demands in the global Y-Y and global X-X directions. Because the Tower is circular, the maximum resultant moment demand was calculated as the vector sum of the moments in each direction. The maximum calculated DCR is 2.12 at EL 450, compared to the maximum allowable DCR of 1.0. This indicates that there will most likely be cracking of the concrete shaft, as well as plastic hinge formation.

As discussed by ICEC (1995) and DSOD (2011), the moment and shear demands below the first hinge are expected increase until the formation of another hinge occurs. This process will continue until a flexural yield hinge is formed at the Tower base, or until the Tower fails in shear. Therefore, based on the current analysis results, the Tower will likely be severely damaged under seismic loading from the MCE.

Table 5-9 Moment DCR Results for the Lafayette Outlet Tower under MCE Loading

Section	**EL**	M_u[1] (kip-ft)	ϕM_n[2] (kip-ft)	DCR_{act}[3]	DCR_{all}[4]	**Effective DCR**[5]
15	488	3,054	3,476	0.88	1.0	0.88
14	477	7,435	4,455	1.67	1.0	**1.67**
13	465	13,481	6,768	1.99	1.0	**1.99**
12	450	22,229	10,477	2.12	1.0	**2.12**
11	441	28,770	14,522	1.98	1.0	**1.98**
10	432	35,473	19,110	1.86	2.0	0.93
9	421	44,534	25,483	1.75	2.0	0.87
8	410	54,131	33,342	1.62	2.0	0.81
7	406	58,820	40,571	1.45	2.0	0.72
6	397	68,472	50,357	1.36	2.0	0.68
5	388	78,750	56,648	1.39	2.0	0.70

1. Resultant moment demand from SAP2000 outputs:
   Max $M_u = \text{Max}\{\sqrt{(M_{D3\text{-}3}^2 + \text{corresp.}M_{D2\text{-}2}^2)}, \sqrt{(M_{D2\text{-}2}^2 + \text{corresp.}M_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factors used to obtain moment-interaction curves: moment = 0.9 and axial = 0.65.
3. DCR_{act} is the *actual* DCR = $M_u/\phi M_n$.
4. DCR_{all} is the maximum *allowable* DCR. For sections that *do not* meet the criteria for brittle failure modes, the allowable DCR is equal to 1.0; otherwise, the allowable DCR is equal to 2.0; see Section 5.12.2
5. Effective DCR is the ratio of DCR_{act} to DCR_{all}. **BOLD red** values indicated DCRs exceeding acceptance criteria according to code standards.

A comparison between the moment DCRs of the current and prior studies is compared in Table 5-10, as well as graphically in Figure 5-8. The effective DCR as described above is reported for AECOM. The moment demands from AECOM's analysis are comparable to those from MJA (2015) and double the demands from DSOD (2011), who used a factor of 2.0 to reduce the moment demands to account for ductility.

Table 5-10 Comparison of Moment DCR Results for the Tower under MCE Loading

EL	AECOM		MJA (2015)		DSOD (2011)(3)		ICEC (1995)	
	M_u (kip-ft)	Effective DCR(1,2)	M_u (kip-ft)	DCR(1)	M_u (kip-ft)	DCR(1)	M_u (kip-ft)	DCR(1)
500	-	-	-	-	337	0.12	800	0.28
480	7,435	**1.67**	3,807	0.98	-	-	-	-
450	22,229	**2.12**	27,678	**2.70**	13,023	**1.43**	21,300	**2.34**
432	35,473	0.93	43,104	**2.32**	17,860	**1.14**	31,200	**2.00**
410	54,131	0.81	66,429	**1.99**	24,487	0.90	46,600	**1.72**
397	68,472	0.68	82,423	**1.70**	30,980	0.76	58,300	**1.43**
384	78,750	0.70	99,739	**1.82**	40,213	0.86	67,400	**1.45**

1. **BOLD red** values indicate DCRs exceeding acceptance criteria according to code standards
2. Effective DCR in AECOM's analysis is the ratio of DCR_{act} to DCR_{all}, where DCR_{act} is the *actual* DCR = $M_u/\phi M_n$. and DCR_{all} is the *allowable* DCR. See Table 5-9.
3. Moment demands in DSOD analysis are reduced by an R-factor of 2.0

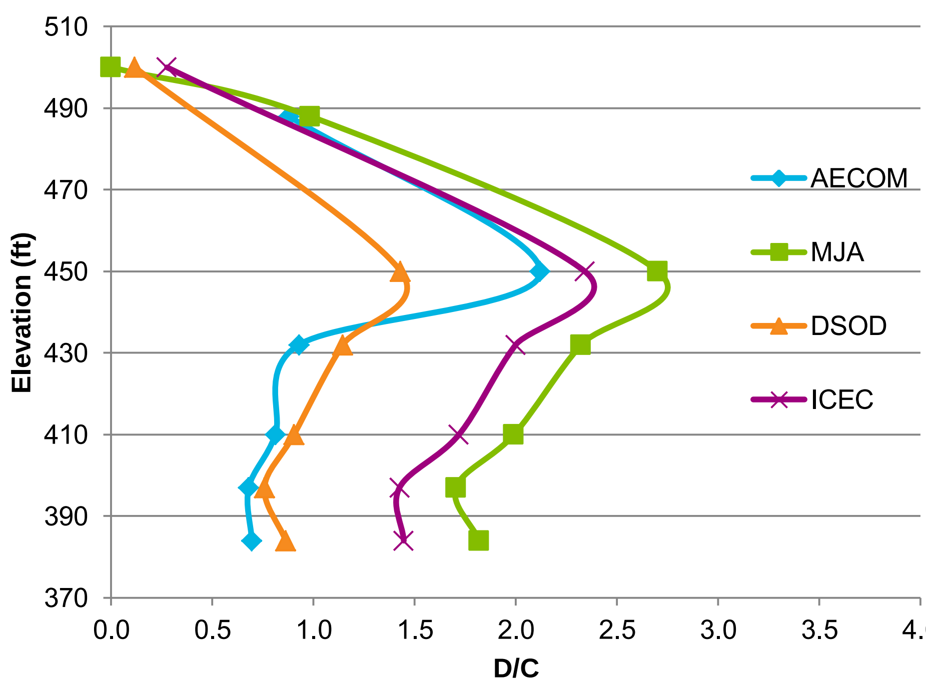


Figure 5-8 Comparison of Moment DCRs along the Height of the Tower

5.11 Shear Demands

Table 5-11 summarizes the shear demands, capacities, and DCRs for each section along the height of the Tower from the current analysis. Similar to the moment demands, the resultant shear demand was calculated as the maximum square root of the sum of squares (SRSS) of the shear demands in each direction. The shear capacity was calculated using the actual in-situ concrete strength obtained from the core tests and takes into account the dynamic factor of 1.1 per EM 1110-2-6053. The results indicate that all sections meet the required shear capacity with a maximum DCR for shear of 0.93 at EL 410. The results are consistent with MJA's (2015) results because MJA used a concrete compressive strength of 4,000 psi in calculating the shear capacity.

Table 5-11 Shear DCR Results for the Lafayette Outlet Tower under MCE Loading [current AECOM analysis]

Section	**EL**	**Max V_u**(1) (kips)	**ϕV_n**(2, 3,4) (kips)	**DCR**	**Max Allowable DCR**
15	488	254	998	0.25	1.0
14	477	399	755	0.53	1.0
13	465	544	675	0.81	1.0
12	450	662	800	0.83	1.0
11	441	716	848	0.85	1.0
10	432	824	962	0.86	1.0
9	421	983	1,092	0.90	1.0
8	410	1,151	1,237	0.93	1.0
7	406	1,191	1,390	0.86	1.0
6	397	1,263	1,583	0.80	1.0
5	388	1,293	1,643	0.79	1.0

1. Max $V_u = \text{Max}\{\sqrt{(V_{D3\text{-}3}^2 + \text{corresp.}V_{D2\text{-}2}^2)}, \sqrt{(V_{D2\text{-}2}^2 + \text{corresp.}V_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factor for shear = 0.85
3. Shear capacity is based on actual in-situ concrete strength
4. Shear capacity listed is the dynamic shear capacity, which is equal to 1.1 times the static shear capacity

A comparison between the shear DCRs of the various studies are presented in Table 5-12 as well as graphically in Figure 5-9.

Table 5-12 Comparison of Shear DCR Results for the Tower under MCE Loading

EL	AECOM		MJA (2015)		DSOD (2011)		ICEC (1995)	
	V_u (kip)	DCR(1)	V_u (kip)	DCR(1)	V_u (kip)	DCR(1)	V_u (kip)	DCR(1)
500	-	-	320	0.38	75	0.10	270	0.87
480	399	0.53	492	0.54	-	-	-	-
450	662	0.83	867	0.76	497	0.49	540	**1.29**
432	824	0.86	1,134	0.91	811	0.74	710	**1.45**
410	1,151	0.93	1,356	0.95	1,280	**1.02**	1,010	**1.71**
397	1,263	0.80	1,497	0.95	1,436	**1.10**	1,150	**1.77**
384	1,293	0.79	1,497	0.90	1,619	**1.12**	1,210	**1.66**

1. **BOLD red** values indicate DCRs exceeding acceptance criteria according to code standards

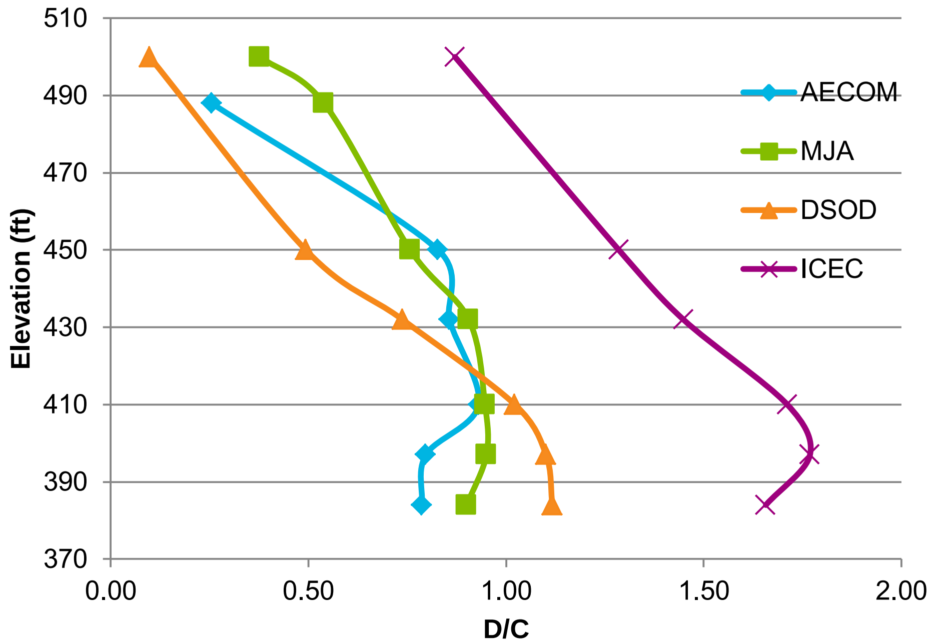


Figure 5-9 Comparison of Shear DCRs along the Height of the Tower

5.12 Brittle Failure Modes

Brittle failure modes outlined in EM 1110-2-6053 (USACE, 2007) and EM 1110-2-2400 (USACE, 2003a) include sliding shear, fracture of reinforcement, anchorage failure, splice failure, and compressive spalling failure. These effects would reduce the ability of the Tower to go through ductile cycles during earthquake shaking, and therefore have the potential to cause flexural failure if these brittle failure modes exist. If brittle failure modes do not exist, the Tower would have sufficient ductile behavior to go through inelastic cycles. Per EM 1110-2-6053, a maximum DCR of 2.0 for flexure may be allowed if it is demonstrated that brittle failures will not occur. Brittle failure modes were also calculated by previous consultants, but they were re-evaluated by AECOM to account for the updated material properties and different analysis assumptions. The acceptance criteria for each potential failure mode based on these codes is outlined in Section 6.3.

Also, appropriate concrete reinforcement confinement is needed in regions where large compressive strains will occur, and heavy confinement reinforcement is needed to improve cyclic performance of splices and anchorages.

5.12.1 Sliding Shear

The potential for sliding along a horizontal crack should be evaluated at all possible failure planes. Sliding shear is resisted by the frictional shear at a plane, as opposed to the diagonal shear presented in Section 5.5 and 5.11. The shear friction capacity (Vsf) should be based on by equation (5-5) of EM 1110-2-6053 (USACE, 2007):

$$V_{sf} = \mu_{sf}\left(P + 0.25A_s f_y\right)$$

where:

μ_{sf} = Sliding shear coefficient of friction, per ACI 318 (assumed 1.0)

P = Axial dead load

A_s = Area of longitudinal reinforcing steel across potential failure plane

f_y = Yield strength of reinforcing steel (33 ksi)

Table 5-13 summarizes the results of a sliding shear analysis performed for each section of the Tower. The sliding shear DCR is above 1.0 for only on section in the upper part of the Tower, with a maximum DCR of 1.08 at EL 477 feet. The retrofit alternatives should address the sliding shear capacity as needed.

Table 5-13 Sliding Shear DCRs for the Tower

Section	EL	P (kips)	A_S (in^2)	V_{SF} (kips)	Max V_u[1] (kips)	DCR[2]	Max Allowable DCR
15	488	161	18.4	313	254	0.81	1.0
14	477	251	21.5	429	399	0.93	1.0
13	465	360	32.5	628	544	0.87	1.0
12	450	515	50.3	930	662	0.71	1.0
11	441	630	70.7	1213	716	0.59	1.0
10	432	740	94.2	1518	824	0.54	1.0
9	421	885	125.2	1918	983	0.51	1.0
8	410	1028	171.9	2446	1,151	0.47	1.0
7	406	1097	218.8	2902	1,191	0.41	1.0
6	397	1236	281.3	3556	1,263	0.36	1.0
5	388	1380	312.5	3958	1,293	0.33	1.0

1. Max V_u from SAP2000, see: Table 5-11 from Section 5.11
2. **BOLD red** values indicate DCRs exceeding acceptance criteria according to code standards

5.12.2 Fracture of Reinforcement

To prevent fracture of tensile reinforcement, the nominal moment capacity (Mn) should equal or exceed the uncracked moment capacity by 20 percent. The cracking moment should be calculated using equation (4-9) of EM 1110-2-2400 (USACE, 2003a):

$$M_{cr} = \left(\frac{I_g}{C}\right)\left(\frac{P}{A_G} + f_r\right)$$

where:
C = distance from neutral axis to extreme fiber
P = axial load on Tower
A_g = gross section area
f_r = modulus of rupture = 7.5√f'c

In this design code, the nominal moment capacity is required to exceed the cracking moment by 20 percent (M_n/M_{cr} > 1.2) to ensure adequate ductility. Table 5-14 shows that the criteria are met only below EL 441 feet. The retrofit alternatives should address the lightly reinforced region at the top of the Tower above the spillway elevation with ratios between 0.43 and 1.17. Because this requirement is not met for these sections, the allowable DCR for flexure is reduced to 1.0.

Table 5-14 Ratio of Nominal to Cracked Moment in the Tower

Section	EL	M_N/M_{CR} (1)
15	488	**0.43**
14	477	**0.49**
13	465	**0.67**
12	450	**0.93**
11	441	**1.17**
10	432	1.44
9	421	1.77
8	410	2.14
7	406	2.48
6	397	2.88
5	388	3.02

1. **BOLD red** values indicate Mn/Mcr ratios less than the acceptance criteria according to code standards

5.12.3 Anchorage Failure

The flexural strength of a structure will deteriorate during a major earthquake if the flexural reinforcement is not adequately anchored. For straight bars, the anchorage length should be greater than equation (5-6) of EM 1110-2-6053 (USACE, 2007):

$$l_a = \frac{f_y(d_b)}{2000}\ [in]$$

Table 5-15 shows the minimum and provided anchorage length from Drawing No. DH 1065-7 (EBMUD, 1929). The anchorage lengths provided for each bar size exceed the minimum lengths, and therefore meet this requirement.

Table 5-15 Vertical Reinforcement Anchorage Lengths

EL	Bar Diameter (inches)	la required (inches)	la provided (inches)
470 to 500	0.625 (round)	10.3	25
420 to 470	1 (square)	16.5	40
356 to 420	1.25 (square)	20.6	50

5.12.4 Splice Failure

Splices in the flexural reinforcement may undergo strength deterioration if located in a plastic hinge region. When concrete compressive strains exceed 0.002 inch per inch (in./in.), the minimum area of transverse confinement steel provided at splice locations should be based on equation (4-13) of EM 1110-2-2400 (USACE, 2003a):

$$A_{tr} = \frac{sf_y}{l_s f_{yt}} A_b$$

where:
s = average spacing of transverse reinforcement over splice length
f_{yt} = yield stress of transverse reinforcement
A_b = area of spliced bar
l_s = splice length

The minimum required lap splice length is based on equation (5-7) of EM 1110-2-6053 (USACE, 2007):

$$l_s = \frac{A_b f_y}{11.31(\sqrt{f'_{ca}})(c + d_b)} \; [in]$$

where:

f'_{ca} = actual concrete compressive strength

c = the lesser of the clear cover over the reinforcing bars, or half the clear spacing between adjacent bars

A_b = area of reinforcing bars

Table 5-16 and Table 5-17 provide a comparison between the minimum and provided splice lengths and transverse areas of reinforcement. The provided areas and lengths exceed the minimum values, and therefore meet this requirement.

Table 5-16 Vertical Reinforcement Splice Lengths

EL	Bar Diameter (inches)	ls required (inches)	ls provided (inches)
470 – 500	0.625 (round)	3.4	25
420 – 470	1 (square)	11.5	40
356 – 420	1.25 (square)	27.5	50

Table 5-17 Transverse Reinforcement Areas

EL	Bar Diameter (inches)	Spacing (inches)	Atr required (inches)	Atr provided (inches)
450 to 500	0.625 (round)	12	0.147	0.307
430 to 450	0.75 (square)	16	0.314	0.44
400 to 430	0.75 (square)	14	0.275	0.44
369 to 400	0.75 (square)	12	0.375	0.44

5.12.5 Compressive Spalling Failure

Excessive compressive strains can cause spalling of the concrete cover and degradation of the transverse confining reinforcement. Compressive spalling failures can be prevented at ultimate load conditions if the concrete compressive strains are less than 0.4 percent, or if the location of the neutral axis is less than 15 percent of the effective depth to the centroid of reinforcement.

The moment-interaction curves were generated assuming a maximum concrete strain of 0.003. Because the moment demands are all within the moment-interaction curves (P-M diagrams), no spalling is expected to occur.

6. Tower Retrofit Design Criteria

6.1 Codes and Standards

The evaluation of the retrofit alternatives described in this report generally follows the procedures described in EM 1110-2-6053 (USACE, 2007). For design requirements of concrete not addressed in the EMs, ACI 318-11 Building Code Requirements for Structural Concrete (ACI, 2011) was taken into consideration. Data from the Post Tensioning Institute (PTI) and specific manufacturers listed below were used to design the retrofit alternatives, specifically for Alternative 1 (Post-tensioning) and Alternative 2 (Fiber-wrapping).

- U.S. Army Corps of Engineers (USACE) EM 1110-2-6053 Earthquake Design and Evaluation of Concrete Hydraulic Structures (2007)
- USACE EM 1110-2-2400 Structural Design and Evaluation of Outlet Works (2003a)
- USACE EM 1110-2-2104 Strength Design For Reinforced Concrete Hydraulic Structures (2016)
- USACE EM 1110-2-6050 Response Spectra and Seismic Analysis for Concrete Hydraulic Structures (1999)
- USACE EM 1110-2-6051 Time-History Dynamic Analysis of Concrete Hydraulic Structures (2003b)
- USACE EM 1110-2-2100 Stability Analysis of Concrete Structures (2005)
- American Concrete Institute (ACI) 318-14 (2014)
- PTI Post-Tensioning Manual (1990)
- Fyfe Co. TYFO SCH-41-2X COMPOSITE Brochure (2018)
- DYWIDAG Strand Anchors Systems Brochure (2018)

6.2 Seismic Retrofit Performance Objectives

Current analyses show that the Tower has seismic deficiencies that would result in unacceptable consequences, such as the potential for a catastrophic collapse of the upper portion of the Tower under the MCE earthquake, leading to a loss of ability to control releases from the reservoir. The seismic retrofit performance objectives of the Tower are based on the USACE EMs (2003a, 2005). The Tower retrofit should “accommodate extreme loads without experiencing a catastrophic failure, although structural damage which partially impairs the operation functions is tolerable, and major rehabilitation or replacement of the structure might be necessary” (USACE, 2005). This is also defined as “damage control performance” in EM 11102-6053 (USACE, 2007).

DSOD’s review of the ICEC study and re- evaluation of the Lafayette Outlet Tower (2011) included a risk assessment of the entire system, which included the dam, Tower, and pressurized conduit through the embankment. Because the Tower control gates are currently closed but experience leakage, the discharge is controlled through a valve at the downstream end of the outlet conduit near the downstream toe of the dam. Therefore, the outlet conduit is currently pressurized throughout the length. This poses another concern for DSOD that should be addressed. The DSOD risk assessment identified critical loading conditions considering the entire system, rather than just the Tower under seismic loads. DSOD evaluated three levels of ground motions — the median 50th percentile motion, 84th percentile motion (median plus one standard deviation), and a 10,000-year event.

DSOD found that the biggest risks resulted from ground motions at or exceeding the 84th percentile loads. The critical scenario included shearing of the spillway conduit, combined with failure of the Tower, resulting in significant uncontrolled outflow through the damaged spillway conduit. Given these findings, DSOD recommended the Tower should be retrofitted to withstand the 84th percentile loading, which is consistent with the loading criteria used in the current study. DSOD concluded that seismically retrofitting the Tower and addressing the functionality of the gate valves at the Tower would address the system failure modes by preventing pressurization of the outlet conduit.

6.3 Design Acceptance Criteria for Tower Retrofit

Design of the Lafayette Outlet Tower retrofit to resists MCE loading for shear and moment will be based on EM 1110-2-6053 (USACE, 2007). Maximum acceptable values for DCRs for each section are 1.0 for shear and 2.0 for moment for the retrofit objectives described in Section 6.2. To allow for a DCR of 2.0 for moment, ductile flexural performance must be checked through preventing brittle failure modes. Table 6-1 summarizes the design acceptance criteria for the Tower retrofit.

Table 6-1 Lafayette Outlet Tower Acceptance Criteria under MCE Loading

Action	Performance Objectives
Overturning Stability (FS)	≥ 1
Moment (DCR)	≤ 2.0
Shear (DCR)	≤ 1.0
Sliding Shear (DCR)	≤ 1.0
Fracture of Reinforcement	$M_n \geq 1.2M_{cr}$
Anchorage Failure	$l_a \geq (f_y\, d_b)/2000$
Splice Failure	$A_{tr} \geq (s\, f_y\, A_b)/(l_s\, f_{yt})$
Compressive Spalling	$\varepsilon \leq 0.004$ or $c/d \leq 0.15$

6.4 Summary of Retrofit Alternatives

Based on the previous studies and AECOM's current evaluation of the Tower's seismic performance, four seismic retrofit alternatives have been identified for study and comparison. These alternatives merited further evaluation and an alternative ranking was developed. The four alternatives considered are:

1. Through-Wall Post-Tensioning
2. External Fiber Wrapping
3. Tower Shortening
4. Mid-Height Base Isolation

Sections 7 through 10 describe each retrofit alternative.

6.5 Retrofit Alternatives Considerations

Each retrofit alternative was evaluated for its overall effectiveness in addressing structural deficiencies, environmental considerations, cost, and constructability. Comparison-level conceptual cost estimates are included in Appendix B. Biological and aesthetic considerations are discussed in the following paragraphs.

6.5.1 Biological and Aesthetic Considerations

The Lafayette Reservoir Outlet Tower was found to not qualify as a historical resource under CEQA (EBMUD 2018). However, the Tower does retain visual features that are identifiable to the local community and City of Lafayette, which may contribute to its aesthetic values. Following the CEQA Appendix G checklist, it will be necessary to evaluate if physical changes to the Tower result in a "substantial adverse effects on a scenic vista" or "substantially degrade the existing visual character or quality of the site and its surroundings." A final determination of the significance of the changes to the Tower's aesthetic values will be evaluated as part of the project's CEQA document.

The biological resources in the Tower area include the reservoir, the trees and vegetation surrounding the reservoir, and the interior of the operating house, which supports nesting/roosting habitat for birds and/or bats. No other special-status plant or wildlife species are anticipated to occur near the Project site. Any spillage or disposal of concrete, dust, waste water, or other material from the drilling would be considered a fill to waters of the U.S., waters of the State, and a potential impact to water quality. This could require permitting from the USACE and the Regional Water Quality Control Board (RWQCB) to authorize. Prior to impacting the operating house, the interior of the house will need to be investigated for nesting birds or roosting bats. Most, but not all, of the nesting birds in the region are protected under the federal Migratory Bird Treaty Act and state Fish and Game Code (Section 3500 et seq.). All the bat species that are anticipated to occur in the area are listed as California Species of Special Concern, which means they must be considered during the environmental review process for CEQA.

Under each retrofit alternative section, the environmental considerations list the pros and cons of the alternative in terms of the potential effects to the aesthetic considerations and biological resources. The potential for biological resources, including plants, animals, and regulated habitats (waters, wetlands, and reservoir), to consider within the Project have been evaluated using the California Natural Diversity Database (CNDDB) and the Sacramento Fish and Wildlife Service Office Information for Planning and Consultation website. Lastly, AECOM conducted a site visit, in coordination with the District in December 2018, to evaluate the potential for nesting birds or roosting bats to occupy the interior of the Tower or to be present around the proposed work area.

7. Alternative 1 – Through-Wall Post-Tensioning

7.1 Description

The Through-Wall Post-Tensioning (PT) Alternative consists of increasing the flexural and shear capacities of the Tower by installation of post-tensioned anchors. Current analysis indicates that six tendons would be sufficient to provide the required axial load to mitigate structural deficiencies and increase the nominal moment and shear capacities. The tendons would be installed in drilled holes in the walls through the height of the Tower (Figure 7-1). The holes would be centered through the wall thickness, and slightly angled vertically to maintain a centered position throughout the Tower height. The tendons would be anchored into the rock below the Tower, and grouted. Within the walls, the tendons will be un-bonded and will be encased in an individually greased plastic sheaths. Fully bonded tendons will also be considered in the final design if this alternative is selected to reduce maintenance and promote a more robust structural performance. In the bonded portion below the foundation, the tendons would be fully grouted and corrosion protected. The proposed six tendons would be positioned to avoid existing openings, including the two conduits at the bottom of the Tower. This alternative would maintain the Tower in its original height and appearance, which is important to the City of Lafayette. The control house roof and portions of its walls would likely be demolished and rebuilt to facilitate construction. See Figures 7-2 and 7-3. For more details of sketches of the retrofit alternatives, see Appendix A.

This alternative would require either a full or partial removal of the roof of the operating house to drill vertically through the concrete Tower walls, approximately 200 feet to install anchors from the operating floor to the base of the Tower, located below grade. The control room floor and the Tower walls immediately below would need to be strengthened to resist the additional forces induced by the tendons.

The conceptual design includes six tendons would likely consist of 15 steel strands of 0.6-inch-diameter high strength steel. Together, they would provide a total force of about 3,100 kips after lock-off losses. Alternatively, four tendons consisting of 27 steel strands would also achieve the same force level. Final determination of the tendon design will take place during final design in coordination with the specialty drilling subcontractor should this alternative be selected as the preferred alternative. AECOM performed a structural analysis of the Tower including the post-tensioning effects as discussed in detail in Section 7.2 below.

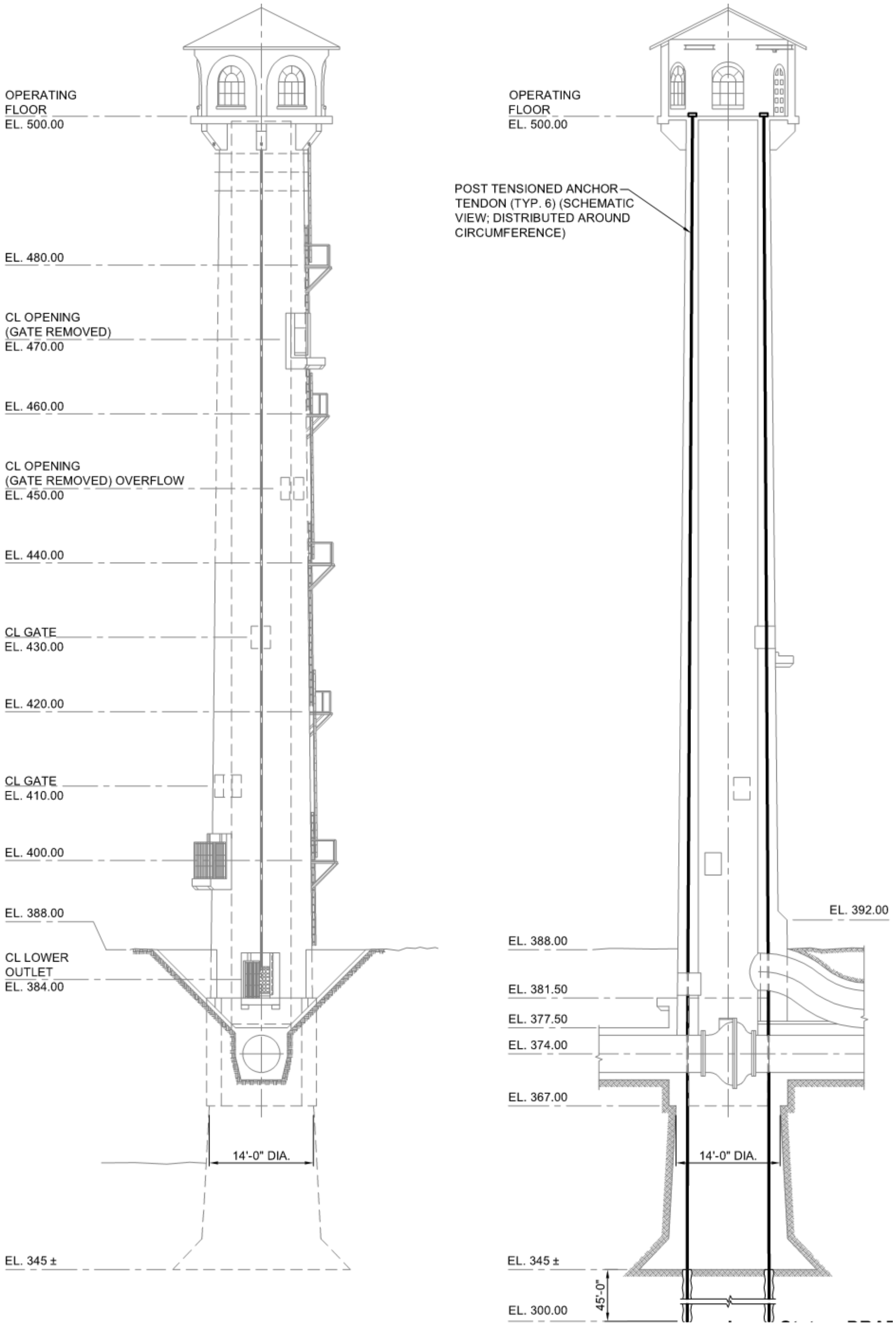


Figure 7-1 Elevation View of Alternative 1 – Through-Wall Post-Tensioning

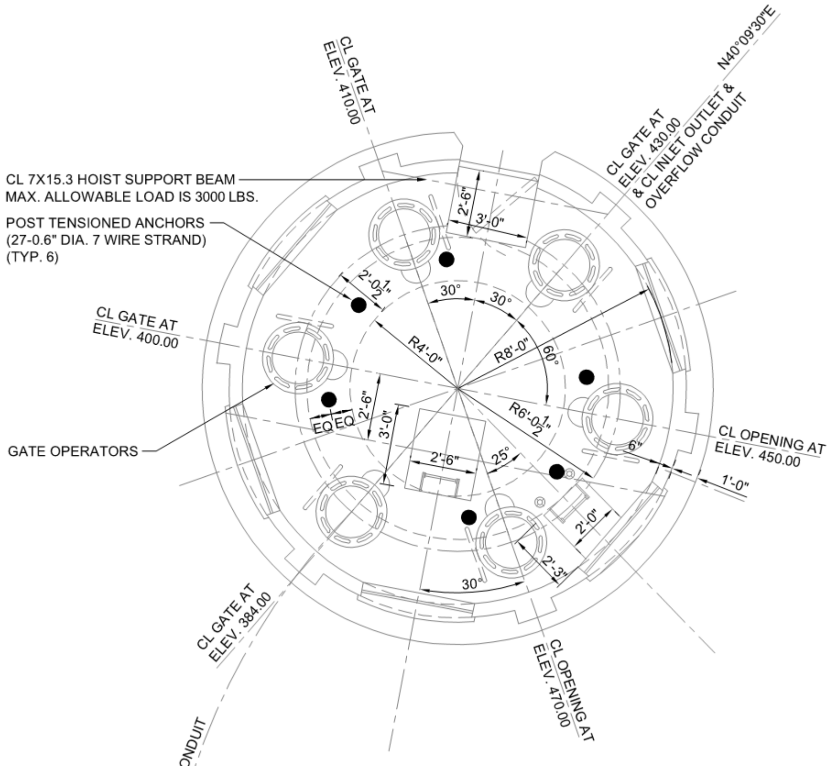


Figure 7-2 Through-Wall Post-Tensioning Anchors Plan Layout at Operating Floor

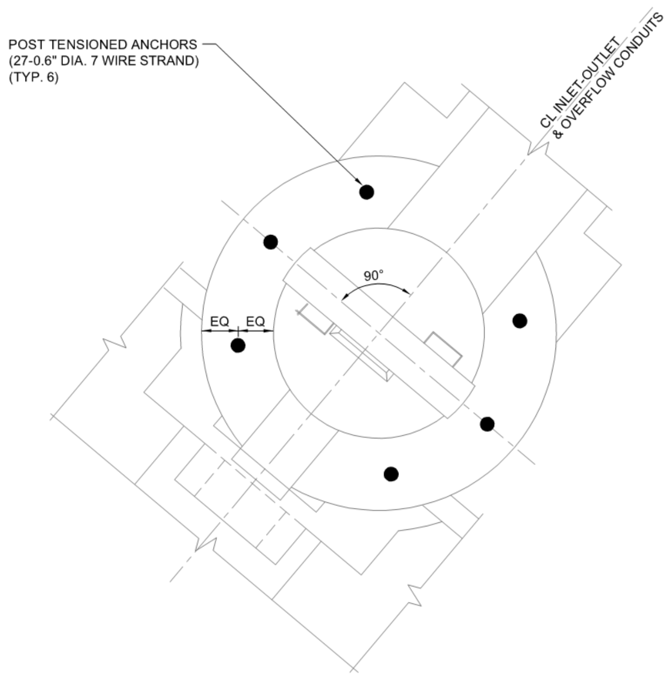


Figure 7-3 Through-Wall Post-Tension Anchors Plan Layout at Elevation 385 Feet

7.2 Structural Effectiveness

Post-tension tendons would be an effective retrofit option for the Lafayette Reservoir Outlet Tower. This alternative could mitigate structural deficiencies, such as moment and shear capacities at certain sections throughout its height.

Structural analysis of this alternative was performed by implementing the following changes to the model of the existing Tower described in Section 5:

- The effective moment of inertia I_E was conservatively updated to the gross moment of inertia ($1.0I_g$), because cracking is expected to be reduced with the additional axial load. In reality, an extreme event like the MCE would likely still cause some cracking, but the degree of cracking would be reduced due to post-tensioning.
- The nominal moment capacity increases due to the added axial force from post-tensioning, as indicated in the P-M interaction diagrams provided in Appendix C.
- The shear capacity also increases throughout the Tower height due to the added axial force from post-tensioning.

The analysis results are presented in Table 7-1 and Table 7-2. The moment DCR for sections throughout the Tower height are well within allowable limits, with a maximum moment DCR of 1.46. The maximum shear DCR is 0.89.

Table 7-1 Shear DCR Results for Through-Wall Post-Tensioned Tower

Section	EL	Max V_u (1) (kips)	ϕV_n (2,3) (kips)	DCR	Max Allowable DCR
15	488	288	1,087	0.26	1.0
14	477	458	1,155	0.40	1.0
13	465	641	1,225	0.52	1.0
12	450	801	1,412	0.57	1.0
11	441	874	1,408	0.62	1.0
10	432	1,007	1,413	0.71	1.0
9	421	1,174	1,432	0.82	1.0
8	410	1,334	1,499	0.89	1.0
7	406	1,372	1,599	0.86	1.0
6	397	1,444	1,751	0.82	1.0
5	388	1,477	1,795	0.82	1.0

1. Max $V_u = \text{Max}\{\sqrt{(V_{D3\text{-}3}^2 + \text{corresp.}V_{D2\text{-}2}^2)}, \sqrt{(V_{D2\text{-}2}^2 + \text{corresp.}V_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factor for shear = 0.85
3. Shear capacity listed is the dynamic shear capacity, which is equal to 1.1 times the static shear capacity

Table 7-2 Moment DCR Results for Through-Wall Post-Tensioned Tower

Section	EL	M_u[1] (kips-ft)	ϕM_n[2] (kips-ft)	DCR_{act}[3]	DCR_{all}[3]	Effective DCR[4]
15	488	3,451	18,102	0.19	2.0	0.10
14	477	8,486	19,380	0.44	2.0	0.22
13	465	15,593	21,824	0.71	2.0	0.36
12	450	26,134	25,359	1.03	2.0	0.52
11	441	34,255	29,098	1.18	2.0	0.59
10	432	42,838	33,105	1.29	2.0	0.65
9	421	54,559	38,127	1.43	2.0	0.72
8	410	66,757	46,138	1.45	2.0	0.72
7	406	72,576	53,433	1.36	2.0	0.68
6	397	84,315	63,295	1.33	2.0	0.67
5	388	96,595	69,761	1.38	2.0	0.69

1. Resultant moment demand from SAP2000 outputs:
   Max M_u = Max{$\sqrt{(M_{D3\text{-}3}^2 + \text{corresp.}M_{D2\text{-}2}^2)}$, $\sqrt{(M_{D2\text{-}2}^2 + \text{corresp.}M_{D3\text{-}3}^2)}$} ; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factors used to obtain moment-interaction curves:
   moment = 0.9 and axial = 0.65.
3. DCR_{act} is the *actual* DCR = $M_u/\phi M_n$. DCR_{all} is the maximum *allowable* DCR. For sections that *do not* meet the criteria for brittle failure modes, the allowable DCR is equal to 1.0; otherwise, the allowable DCR is equal to 2.0.
4. Effective DCR is the ratio of DCR_{act} to DCR_{all}.

The additional axial load from post-tensioning the Tower also reduces the possibility of other brittle failure modes. Adding a post-tensioning load of approximately 3,100 kips increases the nominal moment capacity; and consequently, the ratio of M_N to M_{CR} above the minimum value of 1.2. Other brittle failure criteria are met as for the original Tower. Table 7-3 presents these results.

Table 7-3 Ratio of Nominal to Cracked Moment in Post-Tensioned Tower

Section	EL	M_N/M_{CR}
15	488	1.23
14	477	1.24
13	465	1.31
12	450	1.41
11	441	1.52
10	432	1.65
9	421	1.80
8	410	2.05
7	406	2.30
6	397	2.59
5	388	2.71

The original Tower has a marginal factor of safety against toppling. Overturning stability will be enhanced by the post-tensioning by anchoring the post-tensioning cables into the rock below the Tower. The factor of safety will be increased. This evaluation will be performed in the final design should this alternative be selected.

7.3 Constructability Considerations

Constructing this alternative would require a specialty subcontractor with expertise in precision drilling and installation of post-tensioned grouted anchors or tendons. This technique has been implemented in past projects, including projects designed by AECOM (or AECOM predecessor firms), and therefore there is reasonable confidence in constructability.

Drilling would be conducted from the top of the Tower, about 70 feet above the reservoir. This would most likely require constructing a temporary working platform at the top of the Tower to facilitate construction. The roof of the control house, and potentially a portion of the perimeter walls, will likely be demolished to allow for drilling to take place; then replaced in-kind after construction. The platform would be supported on the Tower. One or more barges would be required to complete construction activities, such as supporting a crane, and haul materials. Transport and installation of the post-tensioned anchor tendons may be performed needed using a helicopter, as was done in a similar project. As with all other alternatives, a staging area will would be on-shore.

To further explore constructability of this alternative, AECOM consulted with two specialty drilling contractors, who visited the job site to discuss the feasibility of using post-tensioning to retrofit the Tower. Feedback from these contractors is summarized below.

For implementing post-tensioning to the Tower, feedback from the specialty contractors highlighted four primary work features: (A) the project infrastructure, (B) close-tolerance drilling, (C) post-tensioning, and (D) Tower restoration. Constructability considerations of these features are discussed below.

A. **Project Infrastructure** and support involves, among others, the following major activities:
   - Install environmental containment including apron fixed to Tower and floating boom
   - Anchor sectional barges to the Tower for staging support crane, drilling support and materials
   - Implement spoils control, containment and off-site disposal measures, including filter treatment of drill process water
   - Saw-cut the Tower's concrete roof and potentially a portion of the perimeter walls, brace for hoisting, remove and store for re-installation
   - Install 'hard-face' barrier shields to protect Tower windows throughout the project
   - Install super-structure work platform supported on the Tower shaft

B. **Close-tolerance drilling:**
   - Drill small diameter (3.5-in.) pilot holes using stabilized wireline core retrieval system
   - Run gyroscope surveys at 10-ft. intervals to verify hole alignment through Tower walls
   - Advance pilot holes several feet beyond Tower/bedrock interface for core specimen
   - Consider enlarging the pilot hole in successive stages with percussive drilling methods
   - Switch to rotary or percussive drilling techniques for advancing boreholes in bedrock
   - Water pressure test Tower/bedrock interface & bond zone; grout & re-drill as needed
   - Remove the super-structure work platform on completion of the drilling work scopes

C. **Post-Tensioning**
   - Using crane staged on support barge, install and two-stage grout the tendon anchors. Transport and installation of the post-tensioned anchor tendons may be performed using a helicopter, as was done in a similar project.
   - Load test anchors per specifications and in the sequence as directed by the Engineer
   - After lock-off, 2nd stage grout the anchor and complete the anchor heads as specified

D. **Tower Restoration**
- Reconstruct (in-kind) concrete roof, remove window barrier shields and, restore and clean Tower
- Remove environmental containment, dispose spoils, demobilize barges and equipment

7.4 Environmental Considerations

Aesthetic Considerations:

- Proposed strengthening work would be limited to the interior of the structure so there would be no significant changes to the aesthetic value of the Tower.

Biological Considerations:

- Proposed work would be limited to the interior of the structure, which would reduce the potential for impacts to the water quality of the reservoir.
- Working within the operating house will require exclusionary devices to restrict nesting birds/roosting bats access into the interior of the Tower. Exclusion could include netting, closed access doors, or sealed gaps between the windows/doors and the Tower.

7.5 Cost Considerations

Cost estimates for Alternative 1 are provided in Appendix B. The total estimated cost for Alternative 1 is $6.27M. The schedule and assumptions to arrive at this cost are:

a. Schedule: 9 months
b. Assumptions:
   1. Crane for 9 months
   2. New Sluice Gate at Tower base
   3. Dewater Tower to install new Sluice Gate

8. Alternative 2 – External Fiber Wrapping

8.1 Description

Alternative 2 proposes strengthening the Tower using a carbon fiber-reinforced polymer (FRP) system applied to the external surface of the Tower. Steel plates may be installed at the interior of the Tower, if needed for shear reinforcement. Based on estimates of the moment deficiencies described in Section 5.10, continuous vertical sheets of FRP extending from EL 430 feet to EL 480 feet would be required. This allows 5 feet of development length above and below the deficient areas. Although some obstructions cannot be avoided (i.e., valve openings), the vertical continuity of the FRP wrap will be sufficient to provide the necessary strength. The vertical sheets are typically 2 feet wide and would be wrapped circumferentially to provide additional confinement. The circumferential wrapping will also increase the Tower's shear strength and will anchor the vertical sheets. Ladders and other metal structures on the Tower exterior would be removed for FRP installation process, and then reinstalled after completion of the circumferential wrap. Custom widths of vertical sheets and additional layers might be required to run between obstructions. An exterior FRP would be viable for the exposed part of the Tower to address the high moment demands. For continuity in appearance of the Tower above the water, the FRP can be applied up to EL 500 feet. Because FRP installation in the wet is likely infeasible, this alternative includes installation of steel plates inside the Tower, assumed necessary to address shear deficiencies between the normal water level and the base. Conceptual sketches of the retrofit are shown in Figures 8-1 and 8-2, and in Appendix A.

According to the manufacturer, the FRP systems are designed based on a 50 to 70 year assumed service life. The concrete surface needs to be dry to the touch before wrapping. The time for the FRP to dry will depend on ambient conditions after installation. After the FRP layers are installed, a thickened coat of epoxy over all exposed surfaces, seams, and edges is applied; then two coats of exterior-grade texture and paint will be needed to improve the aesthetic value of the Tower. To reduce maintenance costs for re-painting, the design will also consider using colored epoxy resin to match the desired aesthetics. The system needs to cure for approximately 5 days prior to being exposed to the water; again, depending on the ambient temperatures.

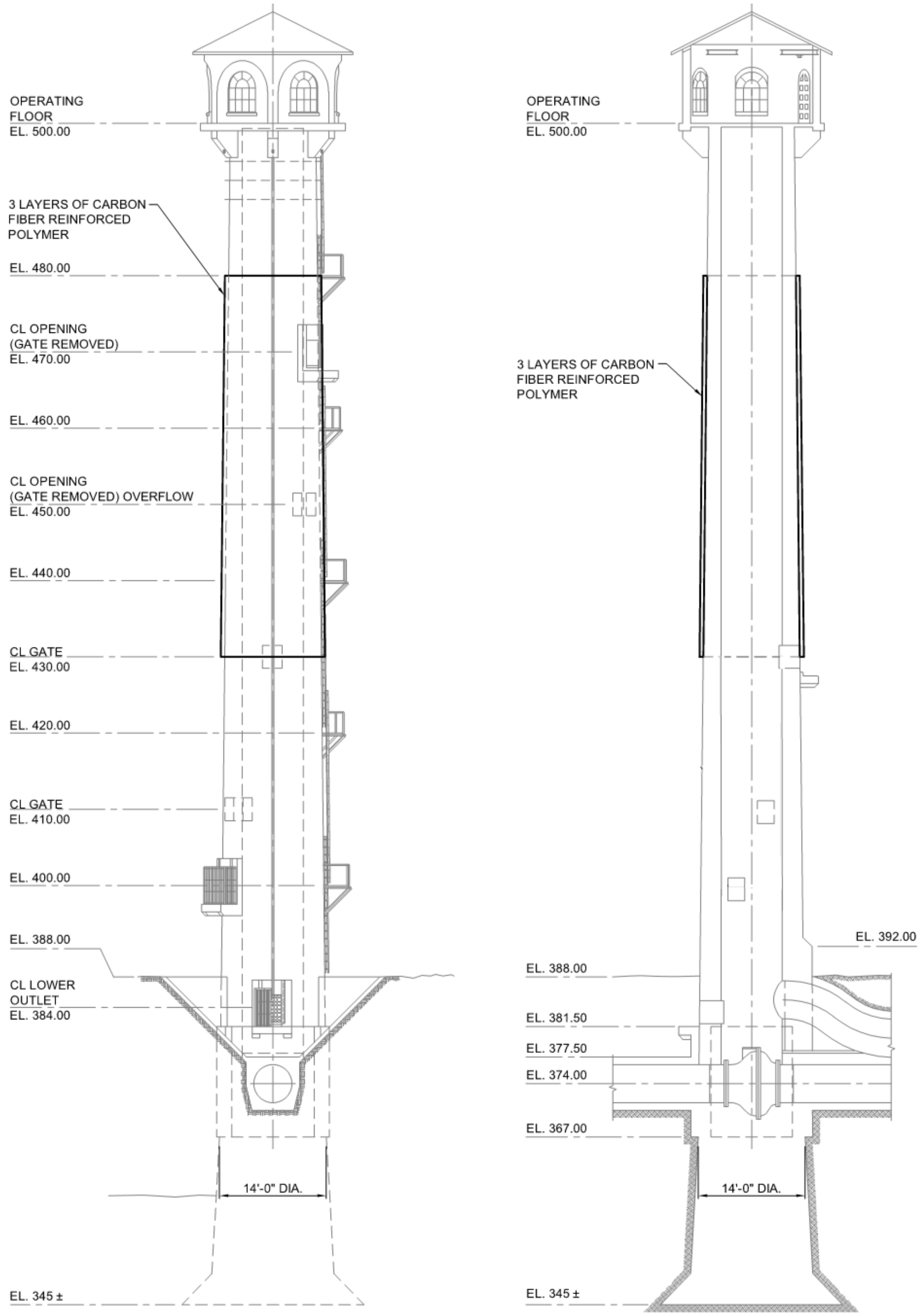


Figure 8-1 Elevation View of Alternative 2 – External Fiber Wrapping

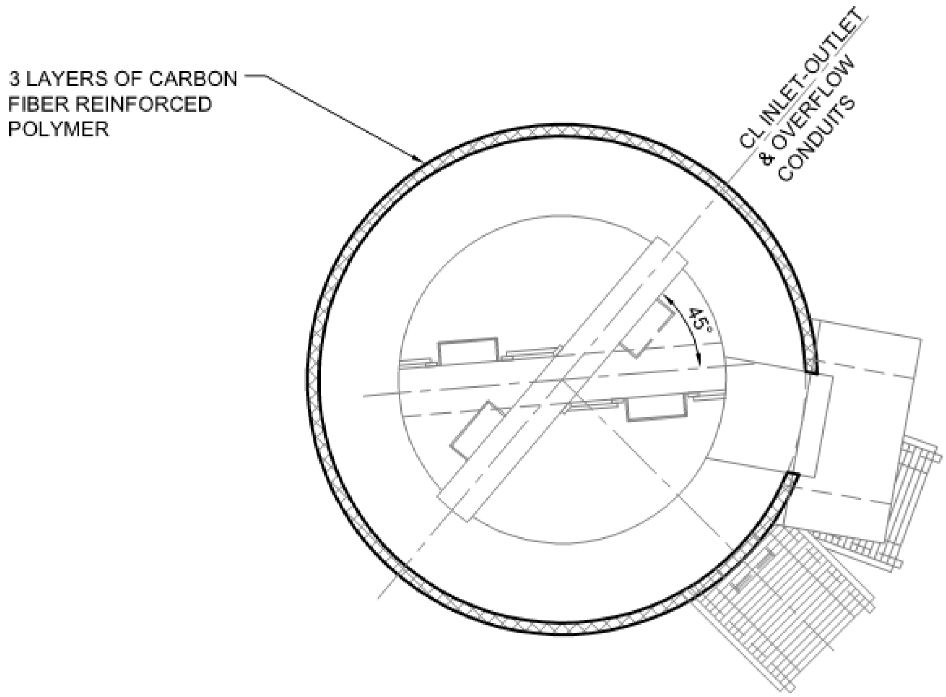


Figure 8-2 External Fiber Wrapping Section Cut at EL 450 feet

8.2 Structural Effectiveness

To evaluate the Tower retrofit alternative with external carbon-fiber wrapping, the sections in SAP2000 from EL 440 feet to EL 500 feet were updated in the Section Designer module. To account for the tensile force provided by the FRP layers, an equivalent steel reinforcement was calculated and added to the outer perimeter of the concrete shaft. The assumed strength values for the FRP were provided by Fyfe, a design and manufacturing company for FRP systems.

Assuming 2 layers of 0.08-inch-thick FRP material in vertical direction, straining to 0.004 in./in., each with 11,900 ksi tensile elastic modulus, the provided tensile force is:

(0.9 reduction factor) x (2 layers) x (0.08”) x (0.004 in/in) x (11,900 ksi) = 6.85 kips per inch along the outer perimeter.

To replicate the FRP properties applied to the structure, additional equivalent steel was added to the model to increase the moment capacities. #8 bars at 8-inch spacing were used:
(0.79 in^2) x (60 ksi) / (8 in) = 5.93 kips per inch < 6.85 kips per inch.

Once the sections were updated in SAP2000, updated moment-interaction diagrams were produced to obtain the moment capacities at each reinforced section. Note that one-layer in the transverse direction is also added to provide the anchorage/confinement.

Table 8-1 presents the results for the moment capacities and DCRs for the FRP alternative. All moment DCRs are less than 2.0 and the effective moment DCRs for Sections 13 and 12 decreased from 1.99 and 2.12 (of the existing Tower) to 0.34 and 0.47, respectively. The moment demands from the original Tower (Section 5) were used to calculate the DCRs.

Table 8-1 Moment DCR Results for Tower with External FRP

Section(1)	EL	M_u(2) (kips-ft)	ϕM_n(3) (kips-ft)	DCR_{act}(4)	DCR_{all}(4)	Effective DCR(5)
15	488	3,054	14,998	0.20	2.0	0.10
14	477	7,435	16,480	0.45	2.0	0.23
13	465	13,481	19,566	0.69	2.0	0.34
12	450	22,229	23,714	0.94	2.0	0.47
11	441	28,770	27,731	1.04	2.0	0.52
10	432	35,473	32,096	1.11	2.0	0.55
9	421	44,534	25,483	1.75	2.0	0.87
8	410	54,131	33,342	1.62	2.0	0.81
7	406	58,820	40,571	1.45	2.0	0.72
6	397	68,472	50,357	1.36	2.0	0.68
5	388	78,750	56,648	1.39	2.0	0.70

1. Sections 10 through 15 include additional FRP moment reinforcement
2. Resultant moment demand from SAP2000 outputs for (existing) Tower:
   Max $M_u = \text{Max}\{\sqrt{(M_{D3\text{-}3}^2 + \text{corresp.}M_{D2\text{-}2}^2)}, \sqrt{(M_{D2\text{-}2}^2 + \text{corresp.}M_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
3. ϕ is the strength reduction factors used to obtain moment-interaction curves: moment = 0.9 and axial = 0.65.
4. DCR_{act} is the *actual* DCR = $M_u/\phi M_n$. DCR_{all} is the maximum *allowable* DCR. For sections that *do not* meet the criteria for brittle failure modes, the allowable DCR is equal to 1.0; otherwise, the allowable DCR is equal to 2.0.
5. Effective DCR is the ratio of DCR_{act} to DCR_{all}

Per Table 5-11, the shear demands are under the capacity at all sections of the Tower, so no shear reinforcement is needed. The additional moment capacity and confinement provided by the FRP also reduce the potential of other brittle failure modes. Table 8-2 presents the ratio of nominal to cracked moment for Alternative 2 in which $M_N/M_{CR} > 1.2$ to ensure adequate ductility.

Table 8-2 Ratio of Nominal to Cracked Moment for FRP Tower

Section	EL	M_N/M_{CR}
15	488	1.97
14	477	1.95
13	465	2.08
12	450	2.21
11	441	2.35
10	432	2.53
9	421	1.84
8	410	2.23
7	406	2.59
6	397	3.01
5	388	3.15

8.3 Constructability Considerations

The installation process for external fiber wrapping would require scaffolding around the Tower. A moving platform, suspended from the top of the Tower at the control house, could also be used. One or more barges will be required to support the construction activities and potentially hold a crane. An on-shore staging area will be required to support construction activities.

8.4 Environmental Considerations

Aesthetic Considerations:

- The work proposed, assuming a final finish similar to the original finish, would protect and maintain the aesthetic features of the Tower.
- Alterative 2 requires removal of exterior features that contribute to the aesthetic character such as the metal ladder and platforms, and metal valve controls that would need to be reinstalled after completion of the wrap and painting to match the unfinished concrete exterior.
- Wrap material does not match the unfinished concrete exterior in terms of composition, design, color, and texture. Work would require application of an epoxy and/or epoxy paint over the installed carbon-fiber wrap to attempt to be visually compatible to retain distinctive materials, finishes, and construction techniques that may be part of the aesthetic value of the Tower.

Biological Considerations:

- Alternative 2 would avoid impacts to the potential nesting/roosting habitat in the operating house.
- Application of the carbon-fiber material and exterior treatment could result in discharge of fill to waters of the U.S./waters of the State. The Project Description would need to clearly identify avoidance and mitigation measures to reduce the chance for spillage or disposal.

8.5 Cost Considerations

With the relative ease of construction, minimal disruption and demolition required to apply the fiber-wrapping, and relatively short construction schedule, the fiber-wrapping alternative presented itself as a cost-effective solution to address seismic deficiencies of the Tower. However, the District was concerned with the shorter service life of the fiber-wrapping than the other alternatives presented so that the total project cost would be 1.5 to 2 times the estimated cost in order to be equivalent to the other alternatives. Although AECOM came up with an initial cost estimate for the fiber-wrapping alternative, it was decided upon further discussions with the District and through the alternative ranking process presented in Section 11 of this report that this alternative will not be further pursued and its cost estimate was not further refined.

9. Alternative 3 – Tower Shortening

9.1 Description

This alternative consists of demolishing the upper portion of the Tower from the top down to about EL 460 feet, which is 10 feet above the spillway elevation. This alternative has 2 options:

- Alternative 3A: New Operating Platform

  This option includes a new operating platform at EL 460 feet for the gate controls. This option would have the most notable change in appearance due to the loss of the operating house and open visual exposure of the valve operation, although aesthetic railing and beveled top could be added to enhance the appearance.

- Alternative 3B: New Lightweight Operating House

  This option includes a new replica operating house at EL 460 feet, using lightweight concrete that will be similar in appearance to the existing operating house. The gate controls would be housed inside of the new operating house and supported on a new 8-inch concrete slab.

In both options, the operating house could be preserved and relocated to a suitable on-shore site nearby, which will come at an additional cost that was not accounted for in the current cost estimate. Conceptual sketches of the two alternatives are shown in Figures 9-1, 9-2, and 9-3, and Appendix A.

A sensitivity study was performed to find the optimal height for the Tower, between EL 460 feet and EL 490 feet, with the lowest DCRs. AECOM performed structural analyses of The Tower using SAP2000 with the Tower shortened to EL 490 feet, EL 480 feet, EL 470 feet, and EL 460 feet. For each case, the Tower capacity was calculated and the seismic demands obtained from SAP2000 outputs. Shortening the Tower results in two competing effects in reducing or increasing the seismic demands. As the top elevation of the Tower decreases, the period of the structure decreases and thus increasing the corresponding spectral acceleration on the response spectra. However, the mass drops as the Tower shortens resulting in a drop in the seismic demands. The sensitivity study showed that cutting the Tower to EL 460 is the optimal height that produced the lowest DCRs.

The analysis for Alternative 3 considered the additional weight of approximately 50 kips at EL 460 feet. The effective moments of inertia I_E were calculated and assigned to the cut Tower following the same procedure in Section 5.6.

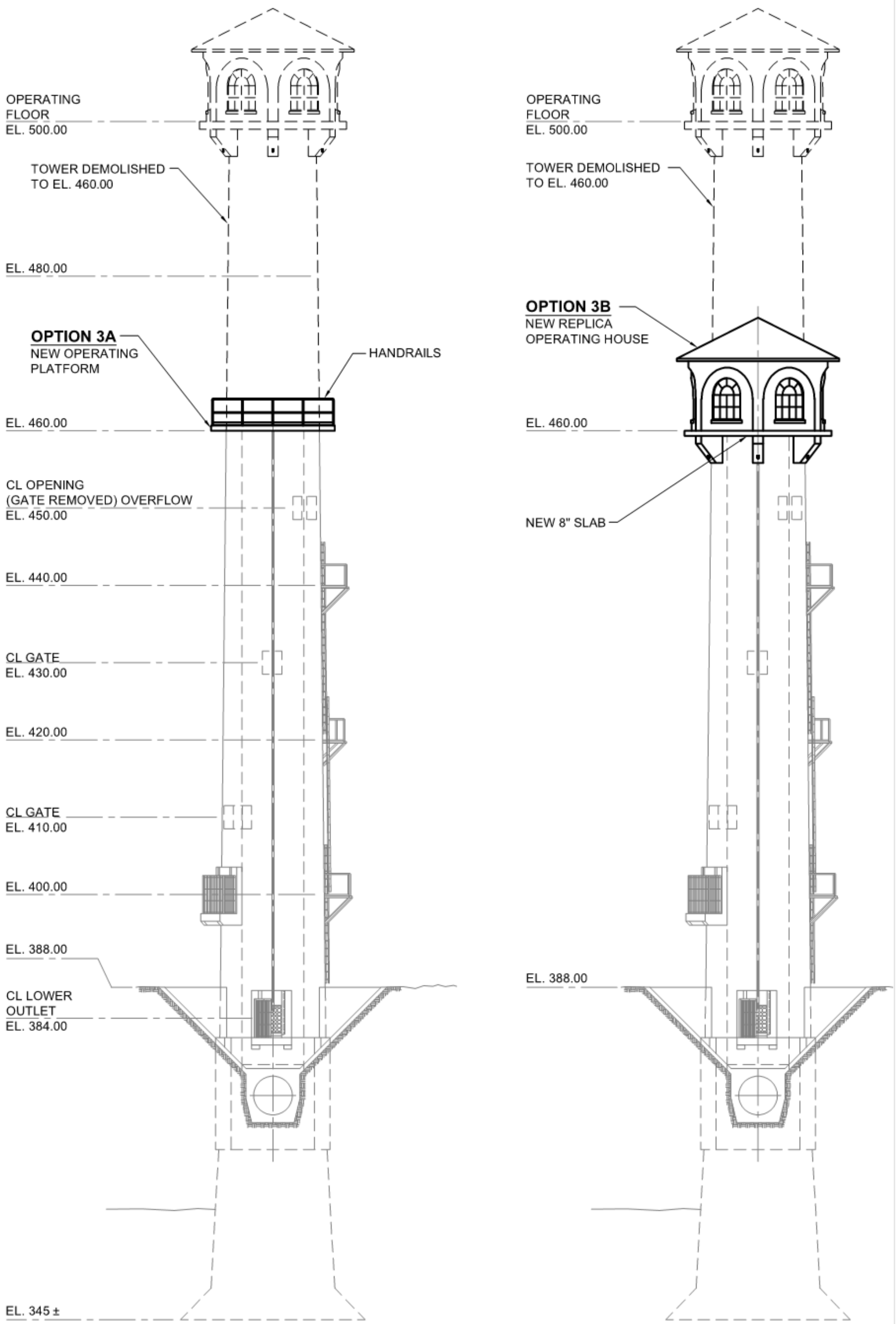


Figure 9-1 Elevation View of Alternative 3 – Tower Shortening (3A and 3B)

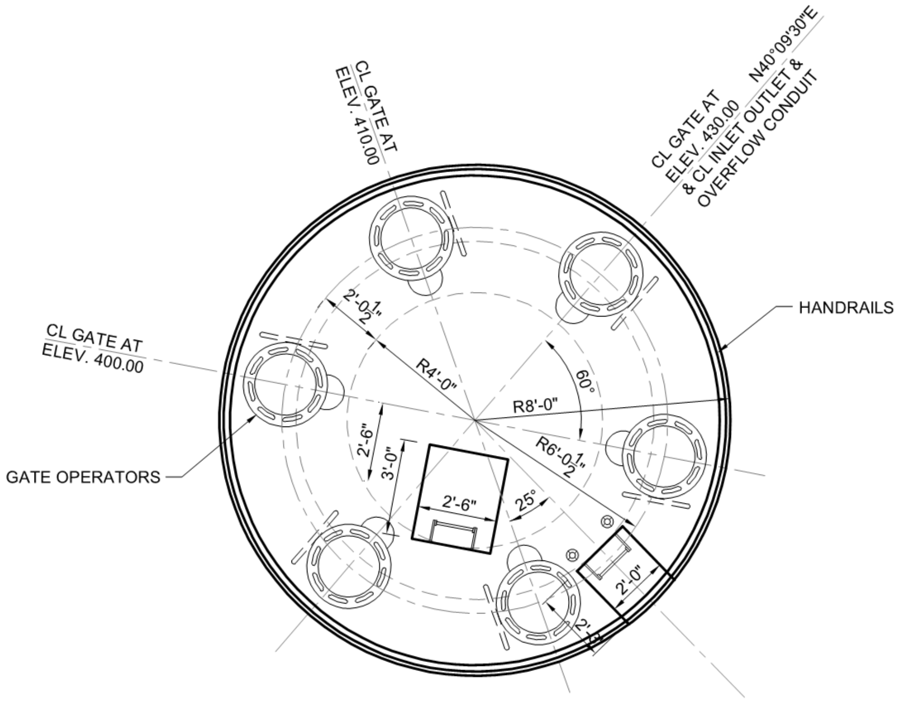


Figure 9-2 Operating Floor Plan at EL 460 feet for Alternative 3A – New Operating Platform

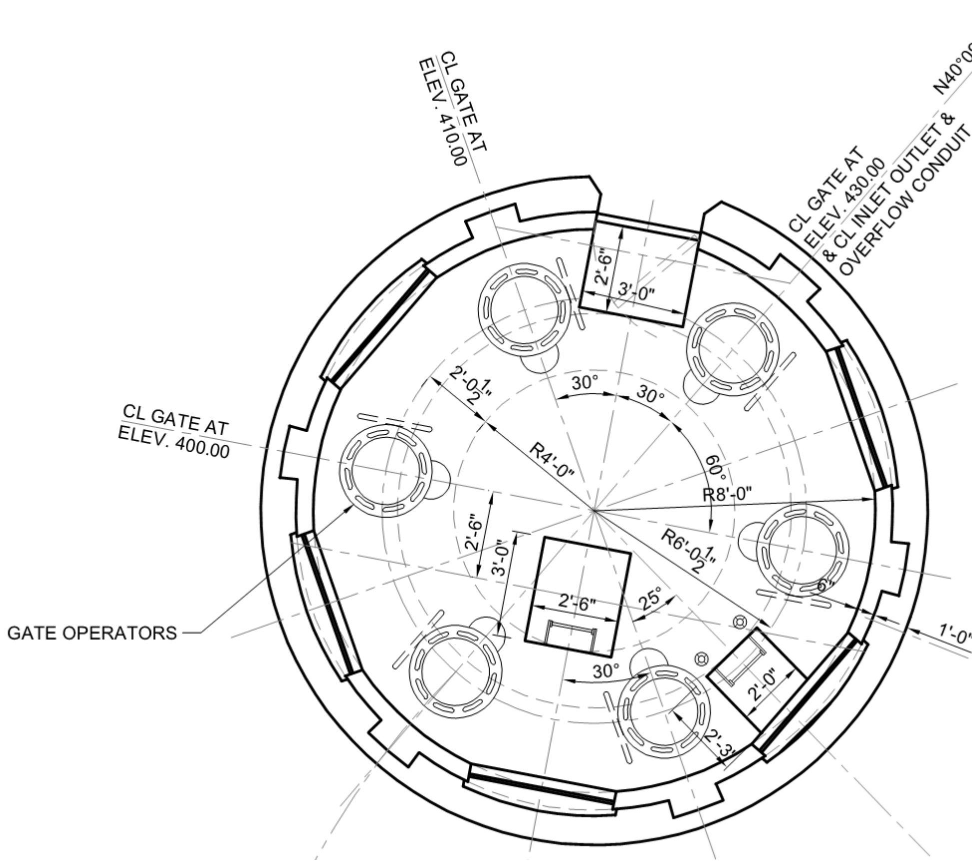


Figure 9-3 Operating Floor Plan at EL 460 feet for Alternative 3B – New Lightweight Operating House

9.2 Structural Effectiveness

The shear demands, capacities, and DCRs for each section along the elevation of the shortened Tower (top EL 460 feet) are presented in Table 9-1. The moment demands, capacities, and DCRs are presented in Table 9-2. The results show that flexural DCRs meet the requirements even for the upper portions whose allowable DCRs are lowered to 1.0 based on the brittle failure modes (Table 9-3). Shear DCRs are also all below the maximum allowable value of 1.0.

Table 9-1 Shear DCR Results for Shortened Tower (Top EL 460 feet)

Section	EL	Max V_u (1) (kips)	ϕV_n (2) (kips)	DCR	Max Allowable DCR
12	450	301	1,273	0.24	1.0
11	441	468	1,349	0.35	1.0
10	432	732	1,427	0.51	1.0
9	421	1,005	1,524	0.66	1.0
8	410	1,226	1,545	0.79	1.0
7	406	1,273	1,634	0.78	1.0
6	397	1,354	1,761	0.77	1.0
5	388	1,387	1,783	0.78	1.0

1. Max $V_u = \mathrm{Max}\{\sqrt{(V_{D3\text{-}3}^2 + \text{corresp.}V_{D2\text{-}2}^2)}, \sqrt{(V_{D2\text{-}2}^2 + \text{corresp.}V_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factor for shear = 0.85

Table 9-2 Moment DCR Results for Shortened Tower (Top EL 460 feet)

Section	EL	M_u (1) (kips-ft)	ϕM_n (2) (kips-ft)	DCR_{act} (3)	DCR_{all} (3)	Effective DCR(4)
12	450	2,033	8,279	0.25	1.0	0.25
11	441	6,350	12,261	0.52	1.0	0.52
10	432	12,891	16,793	0.77	2.0	0.38
9	421	23,385	22,857	1.02	2.0	0.51
8	410	35,174	30,926	1.14	2.0	0.57
7	406	40,890	38,050	1.07	2.0	0.54
6	397	52,459	47,677	1.10	2.0	0.55
5	388	64,522	53,865	1.20	2.0	0.60

1. Resultant moment demand from SAP2000 outputs:
   Max $M_u = \mathrm{Max}\{\sqrt{(M_{D3\text{-}3}^2 + \text{corresp.}M_{D2\text{-}2}^2)}, \sqrt{(M_{D2\text{-}2}^2 + \text{corresp.}M_{D3\text{-}3}^2)}\}$; where 2-2 and 3-3 are the local coordinate axes
2. ϕ is the strength reduction factors used to obtain moment-interaction curves: moment = 0.9 and axial = 0.65.
3. DCR_{act} is the *actual* DCR = $M_u/\phi M_n$. DCR_{all} is the maximum *allowable* DCR. For sections that *do not* meet the criteria for brittle failure modes, the allowable DCR is equal to 1.0; otherwise, the allowable DCR is equal to 2.0.
4. Effective DCR is the ratio of DCR_{act} to DCR_{all}.

Table 9-3 includes the evaluation for fracture of reinforcement. The results show that shortening the Tower does not meet the brittle failure checks for fracture of reinforcement for the lightly reinforced portion of the Tower above EL 440. For these sections, the allowable moment DCR is lowered to 1.0 (Table 9-2).

Table 9-3 Ratio of Nominal to Cracked Moment for Shortened Tower

Section	EL	M_N/M_{CR} (1)
12	450	**0.80**
11	441	**1.07**
10	432	1.36
9	421	1.70
8	410	2.16
7	406	2.52
6	397	2.94
5	388	3.08

1. **BOLD red** values indicate Mn/Mcr ratios less than the acceptance criteria according to code standards

The shortened Tower was analyzed for rotational stability using Housner's Rigid Block Model of EM 1110-2-2400 (USACE 2003a) described in Section 5.9. The analysis was performed using the reduced height of Tower after shortening from EL 345 feet to EL 460 feet, which is equal to 115 feet. The fundamental period is obtained from the stick model of the shortened Tower. Table 9-4 summarizes the results of the rocking analysis. Compared to the results listed in Table 5-8 for the original Tower, the shortened Tower has higher FS values than before shortening.

Table 9-4 Rocking Analysis Results for Shortened Tower

T (sec)	α (rad)	α_{cr} (rad)	Factor of Safety $= \alpha/\alpha_{cr}$	S_d (ft)	B/2 (ft)	Factor of Safety = (B/2) S_d
0.43	0.122	0.075	**1.61**	0.22	7.0	**31.42**

9.3 Constructability Considerations

Demolition of the top portion of the Tower will require measures to prevent the debris from falling into the reservoir. A crane supported on barges will be required to support the construction activities. To minimize the CEQA requirements during demolition and preserve the bottom portion of the Tower intact, it is envisioned that demolition would take place by sequentially saw-cutting to separate the portions to be demolished, then hauling them with a crane. Rebuilding a platform or replica house at the top would also require scaffolding, protection for the reservoir, and a crane.

9.4 Environmental Considerations

Aesthetic Considerations:

- Alternative 3A can potentially retain the original operating house, but at an on-shore location. Although of no historical value, preserving the operating house in another location at the Lafayette Reservoir Recreational Area may be welcomed by patrons using the park on a regular basis.
- Alternative 3B includes reconstruction of the operating house.
- Modification of the height of the Tower could result in a change to the aesthetic characteristics of the Tower..

Biological Considerations:

- Reconstruction of a lightweight operating house on the top of the Tower under Alternative 3B could provide potential nesting or roosting habitat if there are openings to the interior space from outside.
- Removal of the upper portion of the concrete Tower under both Alternative 3A and 3B would likely lead to impacts to water quality, requiring a permit from the USACE and RWQCB, and mitigation measures.

- The proposed platform on top of the Tower would not support nesting or roosting habitat.
- The removal of the operating house and relocation to a site on-shore would remove the potential nesting and roosting habitat. The placement of the structure on-shore is likely to render the space unsuitable to any bat roosting and would likely change the species of the birds that nest there.

9.5 Cost Considerations

Cost estimates for Alternative 3A – new operating platform – are provided in Appendix B. The total estimated cost for Alternative 3A is $4.65M. The schedule and assumptions to arrive at this cost are:

a. Schedule: 7 months
b. Assumptions:
    1. Crane for 7 months
    2. Demo Tower to El. 460'
    3. New operating platform
    4. New Sluice Gate at Tower base
    5. Dewater Tower to install new Sluice Gate

Based on discussions between EBMUD and AECOM, it was decided that Alternative 3A was a preferable retrofit over Alternative 3B, and updates to cost estimates for specifically Alternative 3B were not further investigated. Construction cost for Alternative 3B will likely be slightly higher than 3A, considering the operating platform would be replaced with a new lightweight control house.

10. Alternative 4 – Mid-Height Base Isolation

10.1 Description

The Base-Isolation retrofit alternative was proposed and presented in a technical memorandum by TERRA/COWI (2017). This alternative consists of four friction-bearing pendulums (FPBs) inside the Tower at EL 460 feet. The goal of the FPBs is to lengthen the period of the isolated structure (above EL 460 feet) and decrease the corresponding spectral loading from the design response spectrum. During an earthquake, the isolated upper portion of the structure would ideally move separately from the base where the seismic loads are applied. Figure 10-1 shows the location of the FPBs proposed by TERRA/COWI.

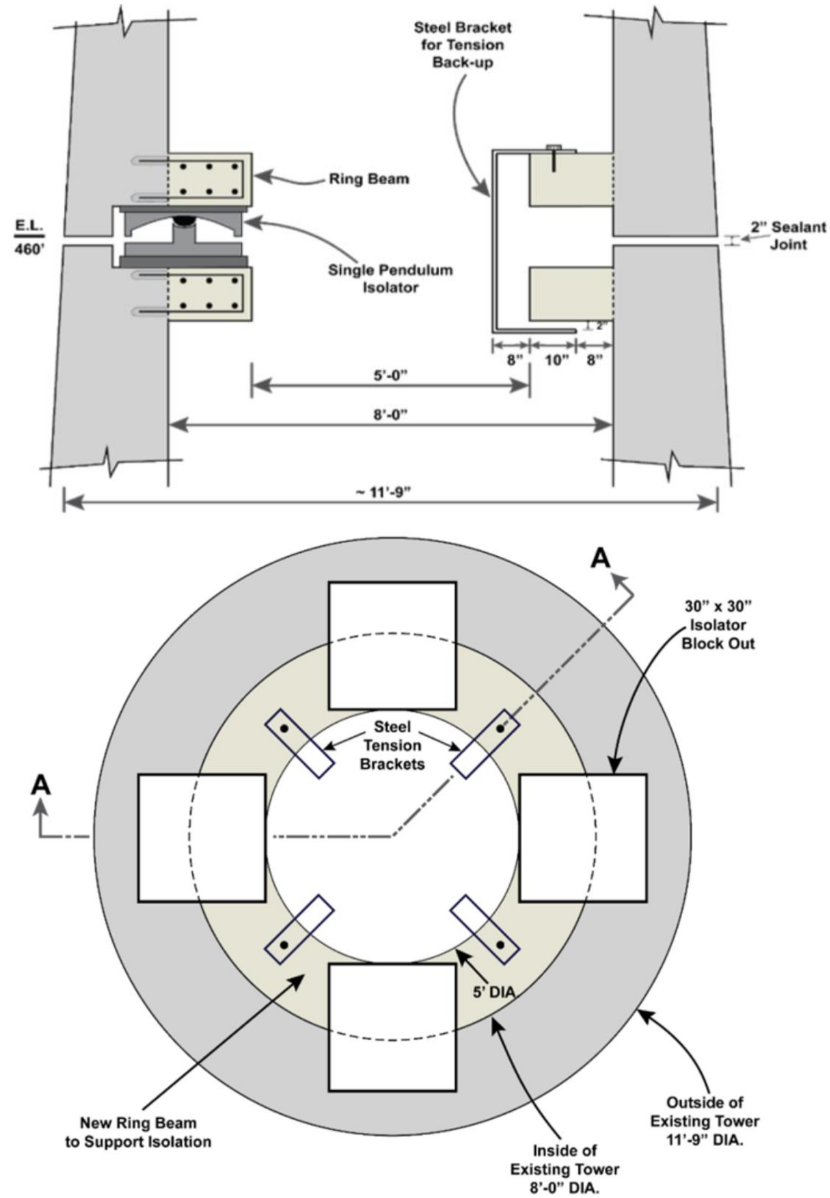


Figure 10-1 Section and Plan Views for Base-Isolation Alternative (TERRA/COWI, 2017)

10.2 Structural Effectiveness

Base isolation systems have been widely used at the foundations of buildings in high seismic zones to limit seismic forces going into the structure. Such base isolation systems would be installed between the building footings and the lowermost columns of the building. Typically, an elaborate grid of ground beams is required both above and below the base isolation system to tie all parts of the structure together. Significant relative displacements are accommodated at the base isolators, on the order of a few feet (depending on the level of seismicity). The base isolation systems normally have stoppers to limit relative movement above and below the isolators to a certain limit dictated by the acceptable deformation the building can accommodate. However, the implementation of this system for the Lafayette Tower proposed by TERRA/COWI (2017) is significantly different than usual practice.

TERRA/COWI's proposed isolators would be installed at EL 460 feet; that is, about 70 feet above the foundations of the Tower. Similar systems may have been designed but not constructed. It is considered that base isolation systems are most effective for isolating short-period structures from their foundations, thereby isolating the ground motions from the building. In at least one instance, the base isolators were installed at the top of the first floor, in a short-period building with a shear wall system. The building also included an elaborate system of tie beams, braces, and stoppers installed throughout the first floor to limit the deformations.

The following are concerns with the proposal for mid-height isolator installation:

1. The Lafayette Tower is not a short period structure; therefore, the implementation would need to be tested and verified for structural performance. Although the analysis presented by TERRA/COWI shows reduced demands, some of the assumptions used in the analysis need to be vetted and tested. The relative deformations between the top and bottom of the isolators may not be captured in the model.

2. Implementing the base isolations at almost mid-height of the 170-foot Tower is unprecedented and has not been proven. Base isolators installed at the foundations can experience large relative deformations. When implemented at mid-height of the Tower, there is no basis to accurately predict how much deformation the isolators may experience. In addition, the isolators will experience rotational displacements at this elevation, while normal base isolators installed at the foundation level experience only translational displacements. The performance of isolators under rotational displacements would need to be tested and verified.

3. The Tower would have to be cut at the isolator level. The conceptual design includes brackets to potentially limit free movement at the cut. However, a weak plane is introduced in the middle of the Tower that, even with mitigation could result in catastrophic failure or tipping of the top portion. Should the displacement exceed the capacity of the isolators, the isolators could fail, resulting in a "hinge" forming at mid-height of the Tower. Uncontrolled pounding between the top and bottom portions of the Tower may cause failure of the restraining brackets resulting in a potentially unstable system, which could allow the top of the Tower above the cut to topple.

For this system to have merit, extensive verification would need to take place. Large scale physical testing would be required to further study this alternative, since it has not been proven effective in the present context. The cost and schedule implications of such verification are incompatible with the directive to complete a timely retrofit of the Tower and therefore, would most likely render this alternative infeasible. Therefore, AECOM recommends no further investigation of this alternative.

11. Alternative Comparison and Ranking

11.1 Alternative Comparison

The four retrofit alternatives for the Lafayette Reservoir Outlet Tower described above in Sections 7 through 10 were compared and ranked in this section to select the most viable alternative for retrofit implementation. The comparison and ranking were based on the following criteria:

1. Structural effectiveness
2. Constructability Considerations
3. Environmental Considerations
    3.1. Biological Considerations
    3.2. Aesthetic Considerations
4. Cost Considerations
    4.1. Construction Cost
    4.2. Durability
    4.3. Life-cycle Cost

Table 11-1 provides a comparison of these considerations for each alternative. These considerations were discussed in more detail in Sections 7 through 10. Structural effectiveness is considered a prerequisite for each alternative, and the alternative will not be considered if it is not structurally sound and judged not effective at addressing the deficiencies in the Tower. In this regard, Alternative 4 – Mid-Height Base Isolation was not considered acceptable and was not included in the comparison for the other considerations.

Table 11-1 Lafayette Tower Retrofit Alternatives Comparison

Criteria	Factor	Alternative 1 Through-Wall Post Tensioning	Alternative 2 External-Fiber Wrapping	Alternative 3 Tower Shortening 3A – with platform only 3B – with new operating house	Alternative 4 Mid-Height Base Isolation
1	Structural effectiveness	• Reduces expected overall cracking in Tower due to increased compression from post-tensioning. • Increases the moment capacity on average by 133% throughout the Tower height. • Maximum moment DCR is reduced by 43% on average.	• Improves ductility by providing confinement in the high moment demand regions. • Increases the moment capacity on average by 180% where FRP is applied. • Maximum moment DCR is reduced by 50%. • Reduces brittle failure mode deficiencies to acceptable levels by	• Reduces demands by reducing overall mass. • Portion of Tower most prone to brittle failure modes is the part to be cut, thereby eliminating these modes. • Maximum moment DCR is reduced by 44%. • Maximum shear DCR is 0.79.	• Applications for long period structure and installation at mid-height of the Tower have not been verified. • Structural stability is questionable. • Will need physical testing to validate.

Criteria	Factor	Alternative 1 Through-Wall Post Tensioning	Alternative 2 External-Fiber Wrapping	Alternative 3 Tower Shortening 3A – with platform only 3B – with new operating house	Alternative 4 Mid-Height Base Isolation
		• Increases the shear capacity by an average of 38% throughout the height. • Maximum shear DCR is 0.89. • Reduces brittle failure mode deficiencies to acceptable levels.	increasing moment capacity.		
2	Constructability	• Requires precision drilling through the walls • Specialty sub-contractor for the drilling will be required. • Will most likely require a large temporary platform at the top of the Tower to facilitate through wall drilling • The roof and all or portions of the control house walls will need to be removed and replaced after construction. • A tall crane is required to lift the tendons into the drilled holes. A helicopter may be used also. • Will not require work close to the water or inside the reservoir	• Requires scaffolding around the Tower or a moving platform suspended from the top of the Tower.	• Requires measures to protect falling debris into the reservoir. • Will likely require a tall crane from a barge to lift demolished portions of the Tower and move them offshore. • For Alternative 3B, scaffolding and a platform will be required at the top of the shortened Tower to install the new control house.	• Not considered
3.1	Biological Considerations	• Reduces potential for impacts to the water quality of the reservoir. Working within of the operating house will require exclusionary devices to restrict	• Avoids impacts to the potential nesting/roosting in operating house. • Need to include mitigation measures to reduce the chance for spillage or discharge	• Alternative 3B could provide potential nesting or roosting habitat if there are openings to the interior space from outside. • Both 3A and 3B	• Not considered

Criteria	Factor	Alternative 1 Through-Wall Post Tensioning	Alternative 2 External-Fiber Wrapping	Alternative 3 Tower Shortening 3A – with platform only 3B – with new operating house	Alternative 4 Mid-Height Base Isolation
		nesting birds/ roosting bats access into the interior of the Tower.	to waters of the U.S./waters of the State.	would likely require a permit from USACE and RWQCB. • 3A platform would not support nesting or roosting habitat. • Placement of the structure onshore is likely to render the space unsuitable for any bat roosting.	
3.2	Aesthetic Considerations	• Interior work, would not change the aesthetics of the structure.	• External work would protect and maintain aesthetic features of the Tower. • Exterior features that contribute to the aesthetic value would need to be reinstalled after completion of the wrap and painting to match the unfinished concrete exterior. • Epoxy/paint will be needed to match existing materials to retain aesthetic features of the Tower.	• Alternative 3B can potentially reconstruct the operating house. • Modification of the height of the Tower could result in a change to the aesthetic characteristics of the Tower.	• Not considered
4.1	Construction Cost	• $6.27M	• Preliminary cost estimate developed but not further refined	• 3A: $4.65M (does not include relocating operating house on-shore) • 3B: not considered	• Not considered
4.2	Durability	• Lifespan is as good as the existing concrete structure. The post-tensioning tendons will be embedded in the concrete with double-corrosion protection.	• Lifespan of the carbon fiber-wrapping depends on exposure to sun and elements and the durability of the adhesive material to the concrete surface. Manufacturers are reporting lifespans ranging from 50-70 years. Painting may be helpful in increasing	• Lifespan is as good as the existing concrete structure.	• Not considered

Criteria	**Factor**	**Alternative 1** **Through-Wall Post Tensioning**	**Alternative 2** **External-Fiber Wrapping**	**Alternative 3** **Tower Shortening** 3A – with platform only 3B – with new operating house	**Alternative 4** **Mid-Height Base Isolation**
			the lifespan.		
4.3	Life-Cycle Cost	• Will require maintenance every 5-10 years.	• Will likely require re-painting every 5 years on average. Instead, colored epoxy resin may be used.	• No additional maintenance costs will be required.	• Not considered

11.2 Alternative Ranking

AECOM conducted a ranking process that engaged technical and project staff from AECOM and the District to perform the alternative ranking. Four technical and project staff from AECOM and four technical and project staff from the District entered their score independently and the average of all eight responses was used as the final score. Each of the criteria factors used in Section 11.1 to compare the alternatives was assigned a weight that is a function of its importance and effect on overall project objectives. Each of the participants voted on a weight for each factor such that the sum of the weights add to 100. Factors with higher weights meant higher influence of this factor to meet project objectives. Then each participant assigned a score from 1 to 10 to each alternative for each of the criteria factors, with 10 being the most desirable and 1 being least desirable. The votes were averaged, and the results are summarized in Table 11-2. As discussed above, Alternative 4 was not included in the ranking because it was judged to be lacking structural effectiveness without sufficient validation, which will likely involve lengthy and costly physical testing.

Table 11-2 Lafayette Tower Retrofit Alternatives Ranking Scores

		Ranking Score			
		Alternative 1	**Alternative 2**	**Alternative 3A**	**Alternative 3B**
Factor	**Weight**	Through-Wall Post-Tensioning	External Fiber-Wrapping	Tower Shortening With Platform only	Tower Shortening with New Operating House
1. Structural Effectiveness	25	7.6	6.0	7.6	7.7
2. Constructability	16	6.4	6.0	7.4	7.3
3. Environmental Considerations					
3.1 Biological Considerations	6	7.9	7.7	7.1	7.0
3.2. Aesthetic Considerations	13	7.8	7.0	5.3	6.0
4. Cost Considerations					
4.1. Construction Cost	16	5.8	6.0	8.0	7.4
4.2. Durability	13	7.0	4.3	8.4	8.4
4.3. Life-Cycle Cost	11	6.8	4.0	8.8	8.7
Total Weighted Score	**100**	**697.9**	**579.0**	**753.8**	**753.7**

Total Cost		**$6.27M**	**Not investigated further**	**$4.65M**	**Slightly higher than Alt 3A**

11.3 Structural Robustness Comparison

Alternatives 1 and 3A ranked highest amongst the other alternatives when all evaluation criteria are considered. Although the construction cost for Alternative 1 is 35% or $1.62M higher than Alternative 3A, Alternative 1 offers the benefit of retaining the height of the Tower in its original height. In order to present the District with another point of reference for comparing the two alternatives, AECOM performed a side-by-side comparison of the two alternatives in terms of structural robustness and effectiveness in addressing the Tower needs. AECOM also performed a sensitivity analysis of the two alternatives to two of the analysis assumptions deemed to impact the results the most, namely, the concrete strength, and the soil springs at the bottom. To that extent, Table 11-3 through Table 11-9 present a comparison between the results for Alternative 1 and 3A for moment and shear demands, capacities, and DCRs. Table 11-10 presents the results of the sensitivity analysis.

A comparison between the two alternatives and the original Tower is shown Table 11-3 for moment demands, Table 11-4 for moment capacities, and Table 11-5 for moment DCRs. Alternative 1 results in an increase in moment demands on the Tower by an average of 20 percent because of the reduced potential for cracking due to the added axial compression from post-tensioning. However, post-tensioning also increases the moment capacity by an average of 133 percent. The net effect is a significant reduction in moment DCRs by an average of 43%. Post-tensioning is most effective at the top portion of the Tower above EL 441, where the moment DCRs are highest. Above EL 441, the moment capacity increases by an average of 244 percent. Shortening the Tower is also effective in reducing the moment demands along the remaining Tower height by 48 percent on average; and while the capacity also slightly drops, the DCRs drops by an average of 44 percent. The 10 percent drop in moment capacity is attributed to the reduction in axial compression from the self-weight of the top of the Tower to be removed. The two alternatives have similar effectiveness in reducing moment DCRs.

A comparison between the two alternatives and the original Tower is shown in Table 11-6 for shear demands, Table 11-7 for shear capacities, and Table 11-8 for shear DCRs. While Alternative 1 increases the shear demands by an average of 17 percent, it increases the shear capacity by an average of 38 percent, resulting in a net average reduction in shear DCR of 13 percent. Alternative 3A reduces the shear demands by about 9 percent on average but the shear capacity increases by about 34 percent. The reduced moment demands and reduced cracking potential leads to higher shear capacity. Alternative 3A results in a net average reduction in shear DCRs by 28 percent. While both alternatives reduce the potential for shear failure, Alternative 3A is more effective than Alternative 1. The maximum shear DCRs are 0.79 for Alternative 3A as opposed to 0.89 for Alternative 1.

Table 11-9 presents a comparison of the nominal to cracking moment ratio along the Tower height, a measure of the potential for brittle failure at different sections along the height. Both alternatives address this issue in different ways. Alternative 1 increases the nominal moment along the Tower by increasing axial compression on the Tower sections through post-tensioning. Alternative 3A eliminates this potential by removing the top part. After removal, the criteria is not met for the top 10 feet of the Tower. However, the moment DCRs range from 0.25 to 0.52 and therefore high ductility is not required and brittle failure is not expected to occur.

Table 11-10 shows the results of the sensitivity analysis performed for the two alternatives. The goal is to identify the alternative that is possibly less sensitive to variability in design assumptions. To achieve that, AECOM investigated the sensitivity of both Alternatives 1 and 3A to changes in concrete material strength and soil spring constants. Variation in concrete strength was tested for a lower bound of 2,500 psi and an upper bound of 6,500 psi. Additionally, soil spring constants were tested for a lower bound of half the values used in the analyses presented in this report and an upper bound of double the values used in the analysis. The two alternatives were analyzed for these variations and the maximum moment and shear DCR results presented in the table. Both alternatives show little sensitivity to changes in soil spring constants. Variation in concrete strength has more of an effect on the DCRs. The shortening alternative has lower DCRs with these variations.

Table 11-3 Comparison of Moment Demands for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		M_u (kips-ft)	M_u (kips-ft)	% Change from Original	M_u (kips-ft)	% Change from Original
15	488	3,054	3,451	13%	-	-
14	477	7,435	8,486	14%	-	-
13	465	13,481	15,593	16%	-	-
12	450	22,229	26,134	18%	2,033	-91%
11	441	28,770	34,255	19%	6,350	-78%
10	432	35,473	42,838	21%	12,891	-64%
9	421	44,534	54,559	23%	23,385	-47%
8	410	54,131	66,757	23%	35,174	-35%
7	406	58,820	72,576	23%	40,890	-30%
6	397	68,472	84,315	23%	52,459	-23%
5	388	78,750	96,595	23%	64,522	-18%
Average Increase/Reduction				20%		-48%
Maximum Increase/Reduction				23%		-91%
Average Increase/Reduction below EL 450				22%		-48%

Table 11-4 Comparison of Moment Capacity for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		ϕM_n (kips-ft)	ϕM_n (kips-ft)	% Change from Original	ϕM_n (kips-ft)	% Change from Original
15	488	3,476	18,102	421%	-	-
14	477	4,455	19,380	335%	-	-
13	465	6,768	21,824	222%	-	-
12	450	10,477	25,359	142%	8,279	-21%
11	441	14,522	29,098	100%	12,261	-16%
10	432	19,110	33,105	73%	16,793	-12%
9	421	25,483	38,127	50%	22,857	-10%
8	410	33,342	46,138	38%	30,926	-7%
7	406	40,571	53,433	32%	38,050	-6%
6	397	50,357	63,295	26%	47,677	-5%
5	388	56,648	69,761	23%	53,865	-5%
Average Increase/Reduction				133%		-10%
Maximum Increase/Reduction				421%		-5%
Average Increase/Reduction below EL 450				61%		-10%

Table 11-5 Comparison of Moment DCRs for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		Effective DCR(1),(2)	Effective DCR (2)	% Change from Original	Effective DCR (2)	% Change from Original
15	488	0.88	0.1	-89%	-	-
14	477	**1.67**	0.22	-87%	-	-
13	465	**1.99**	0.36	-82%	-	-
12	450	**2.12**	0.52	-75%	0.25	-88%
11	441	**1.98**	0.59	-70%	0.52	-74%
10	432	0.93	0.65	-30%	0.38	-59%
9	421	0.87	0.72	-17%	0.51	-41%
8	410	0.81	0.72	-11%	0.57	-30%
7	406	0.72	0.68	-6%	0.54	-25%
6	397	0.68	0.67	-1%	0.55	-19%
5	388	0.7	0.69	-1%	0.60	-14%
Average DCR/Reduction			0.54	-43%	0.49	-44%
Maximum DCR/Reduction			0.72	-89%	0.60	-88%
Average DCR/Reduction below EL 450			0.66	-27%	0.49	-44%

1. **BOLD red** values indicate DCRs exceeding acceptance criteria according to code standards.
2. The effective DCR is the ratio of DCR over allowable DCR

Table 11-6 Comparison of Shear Demands for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		V_u (kip)	V_u (kip)	% Change from Original	V_u (kip)	% Change from Original
15	488	254	288	13%	-	-
14	477	399	458	15%	-	-
13	465	544	641	18%	-	-
12	450	662	801	21%	301	-55%
11	441	716	874	22%	468	-35%
10	432	824	1,007	22%	732	-11%
9	421	983	1,174	19%	1,005	2%
8	410	1,151	1,334	16%	1,226	7%
7	406	1,191	1,372	15%	1,273	7%
6	397	1,263	1,444	14%	1,354	7%
5	388	1,293	1,477	14%	1,387	7%
Average Increase/Reduction				17%		-9%
Maximum Increase/Reduction				22%		-55%
Average Increase/Reduction below EL 450				18%		-9%

Table 11-7 Comparison of Shear Capacity for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		ϕV_n (kip)	ϕV_n (kip)	% Change from Original	ϕV_n (kip)	% Change from Original
15	488	998	1,087	9%	-	-
14	477	755	1,155	53%	-	-
13	465	675	1,225	81%	-	-
12	450	800	1,412	77%	1,273	59%
11	441	848	1,408	66%	1,349	59%
10	432	962	1,413	47%	1,427	48%
9	421	1,092	1,432	31%	1,524	40%
8	410	1,237	1,499	21%	1,545	25%
7	406	1,390	1,599	15%	1,634	18%
6	397	1,583	1,751	11%	1,761	11%
5	388	1,643	1,795	9%	1,783	9%
Average Increase/Reduction				38%		34%
Maximum Increase/Reduction				81%		59%
Average Increase/Reduction below EL 450				35%		34%

Table 11-8 Comparison of Shear DCRs for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		DCR	DCR	% Change from Original	DCR	% Change from Original
15	488	0.25	0.26	4%	-	-
14	477	0.53	0.4	-25%	-	-
13	465	0.81	0.52	-36%	-	-
12	450	0.83	0.57	-31%	0.24	-71%
11	441	0.85	0.62	-27%	0.35	-59%
10	432	0.86	0.71	-17%	0.51	-41%
9	421	0.9	0.82	-9%	0.66	-27%
8	410	0.93	0.89	-4%	0.79	-15%
7	406	0.86	0.86	0%	0.78	-9%
6	397	0.8	0.82	2%	0.77	-4%
5	388	0.79	0.82	4%	0.78	-1%
Average DCR/Reduction			0.66	-13%	0.61	-28%
Maximum DCR/Reduction			0.89	-36%	0.79	-71%
Average DCR/Reduction below EL 450			0.76	-10%	0.61	-28%

Table 11-9 Comparison of Nominal to Cracking Moment Ratio for Post-Tensioned and Shortened Tower Alternatives

Section	EL	Original Tower	Alternative 1 (PT)		Alternative 3 (Shortening)	
		M_N/M_{CR} (1)	M_N/M_{CR}	% Difference	M_N/M_{CR} (1)	% Difference
15	488	**0.43**	1.23	186%	-	-
14	477	**0.49**	1.24	153%	-	-
13	465	**0.67**	1.31	96%	-	-
12	450	**0.93**	1.41	52%	**0.80**	-14%
11	441	**1.17**	1.52	30%	**1.07**	-9%
10	432	1.44	1.65	15%	1.35	-6%
9	421	1.77	1.80	2%	1.70	-4%
8	410	2.14	2.05	-4%	2.16	1%
7	406	2.48	2.30	-7%	2.52	2%
6	397	2.88	2.59	-10%	2.94	2%
5	388	3.02	2.71	-10%	3.08	2%
Average Ratio/%Difference			1.80	46%	1.95	-3%
Maximum Ratio/%Difference			2.71	186%	3.08	2%
Average Ratio/%Difference below EL 450			2.00	8%	1.95	-3%

1. **BOLD red** values indicate Mn/Mcr ratios less than the acceptance criteria according to code standards.

Table 11-10 Sensitivity of Analysis Results to Varying Modeling Assumptions

Dynamic Concrete Strength f'c (psi)	Spring Stiffness Factor	Alternative 1 (PT)		Alternative 3 (Shortening)	
		Maximum Moment DCR(1)	Maximum Shear DCR(2)	Maximum Moment DCR(1)	Maximum Shear DCR
4,634	1	0.72	0.89	0.60	0.79
Sensitivity to Soil Springs					
4,634	½	0.72	0.89	0.60	0.81
4,634	1	0.72	0.89	0.60	0.79
4,634	2	0.72	0.86	0.59	0.79
Sensitivity to Concrete Strength					
2,500	1	0.74	**1.07**	0.61	0.98
4,634	1	0.72	0.89	0.60	0.79
6,500	1	0.70	0.75	0.58	0.68

1. The effective DCR is the ratio of DCR over allowable DCR
2. **BOLD red** BOLD red values indicate DCRs exceeding acceptance criteria according to code standards.

12. Conclusions

This report presents an analysis of the potentially viable alternatives for seismic strengthening of the Lafayette Reservoir Outlet Tower. Retrofitting the Tower is necessary due to its critical function as a spillway and to prevent uncontrolled release of the reservoir in the event of Tower failure. Based on review of the previous studies and proposed retrofit alternatives of the Tower, four alternatives were selected by AECOM for further analysis and comparison in this study:

Alternative 1 – Through-wall Post-Tensioning

Alternative 2 – External Carbon-Fiber Wrapping

Alternative 3 – Tower Shortening

Alternative 4 – Mid-height Base Isolation

The alternatives were analyzed for structural effectiveness, comparative-level cost, constructability, and environmental considerations including potential effects on the aesthetic and biological conditions, life-cycle costs, and durability. Alternatives 2 and 4 are recommended for elimination from further consideration, for reasons of relatively short service life, and uncertain structural effectiveness, respectively. Alternatives 1 and 3 ranked highest considering all factors combined. While Alternative 1 has the appeal of maintaining the original height and look of the Tower, it will require a specialty subcontractor to perform precision drilling through the walls. Constructability considerations drive up construction cost. On the other hand, Alternative 3 does not require specialty subcontractor and is relatively easy to construct. Considering all factors combined (structural effectiveness, aesthetic and biological considerations, life cycle cost, and durability) both Alternatives 1 and 3 were further compared for structural effectiveness and for sensitivity to variations in design assumptions. The comparison showed that Alternative 3 was more effective in reducing the seismic demands and results in lower potential for shear failure. Either of these alternatives can resolve the Tower seismic deficiencies but Alternative 3 is recommended by AECOM based on structural efficiency, cost, and constructability considerations.

Statement of Limitations

The attached Report (the "Report") has been prepared by AECOM Technical Services, Inc. ("AECOM") for the benefit of the East Bay Municipal Utility District ("District") in accordance with the agreement between AECOM and the District, including the scope of work detailed therein (the "Agreement").

The information, data, recommendations, and conclusions contained in the Report (collectively, the "Information"):

- are subject to the scope, schedule, and other constraints and limitations in the Agreement and the qualifications contained in the Report (the "Limitations");
- represents AECOM's professional judgement in light of the Limitations and industry standards for the preparation of similar reports;
- may be based on information provided to AECOM that has not been independently verified;
- has not been updated since the date of issuance of the Report, and its accuracy is limited to the time period and circumstances in which it was collected, processed, made, or issued;
- must be read as a whole, and sections thereof should not be read out of such context;
- was prepared for the specific purposes described in the Report and the Agreement; and
- in the case of subsurface, environmental, or geotechnical conditions, may be based on limited testing and on the assumption that such conditions are uniform and not variable either geographically or over time.

AECOM has relied on the accuracy and completeness of information that was provided to it and has not been verified or updated such information. AECOM accepts no responsibility for any events or circumstances that may have occurred since the date on which the Report was prepared; and in the case of subsurface, environmental, or geotechnical conditions, is not responsible for any variability in such conditions, geographically or over time.

This represents professional judgement as described above. The Information has been prepared for the specific purpose and use described in the Report and the Agreement, but AECOM makes no other representations, or any guarantees or warranties whatsoever, whether express or implied, with respect to the Report, the Information, or any part thereof.

Without in any way limiting the generality of the foregoing, any estimates or opinions regarding probable construction costs or construction schedule provided by AECOM represent AECOM's professional judgement in light of its experience and the knowledge and information available to it at the time of preparation. Because AECOM has no control over market or economic conditions, prices for construction, equipment, or materials or bidding procedures, AECOM makes no representations, warranties, or guarantees whatsoever, whether express or implied, with respect to such estimates or opinions, or their variance from actual construction costs or schedules; and accepts no responsibility for any loss or damage arising therefrom or in any way related thereto.

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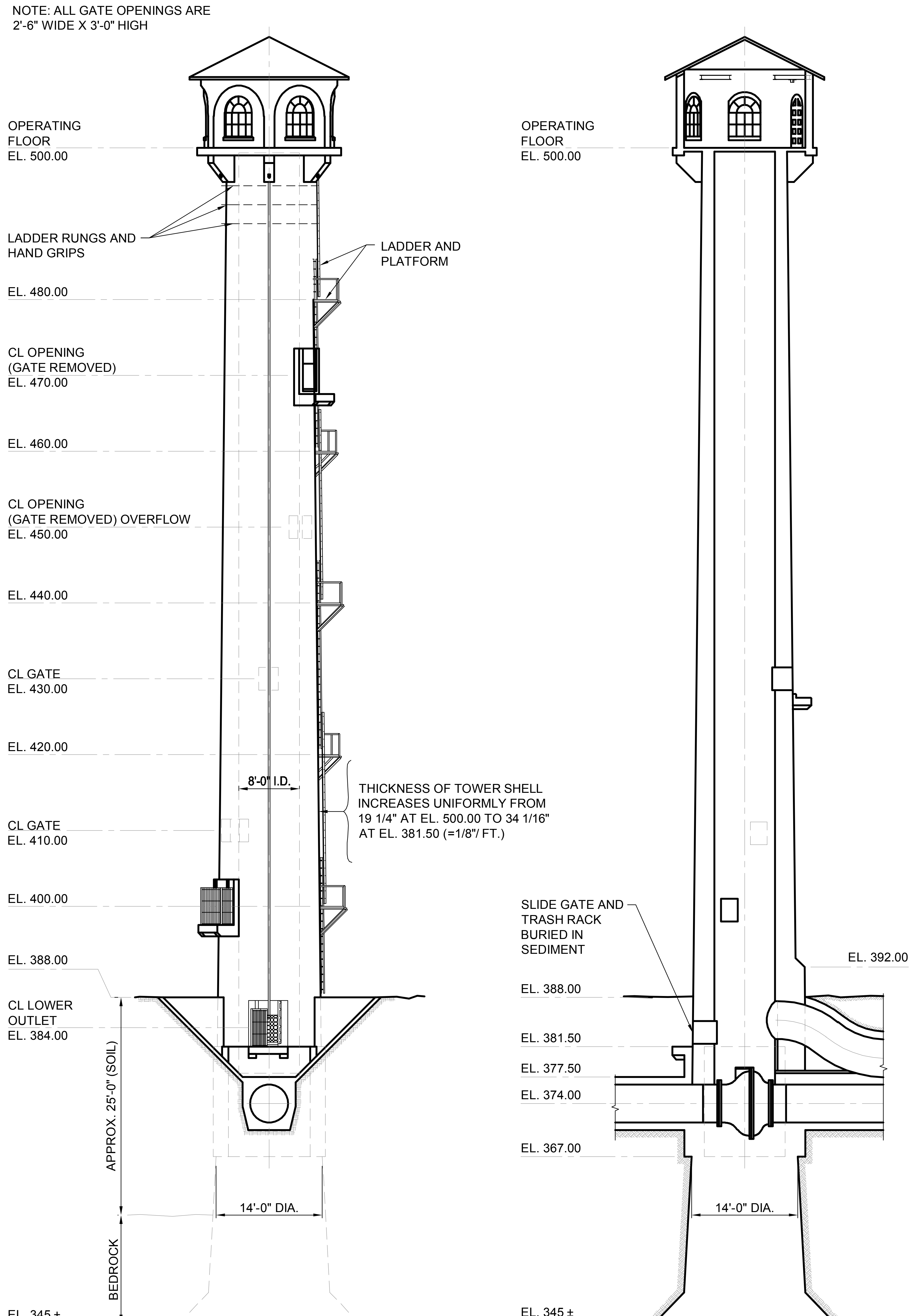
Appendix A

Retrofit Alternative Sketches



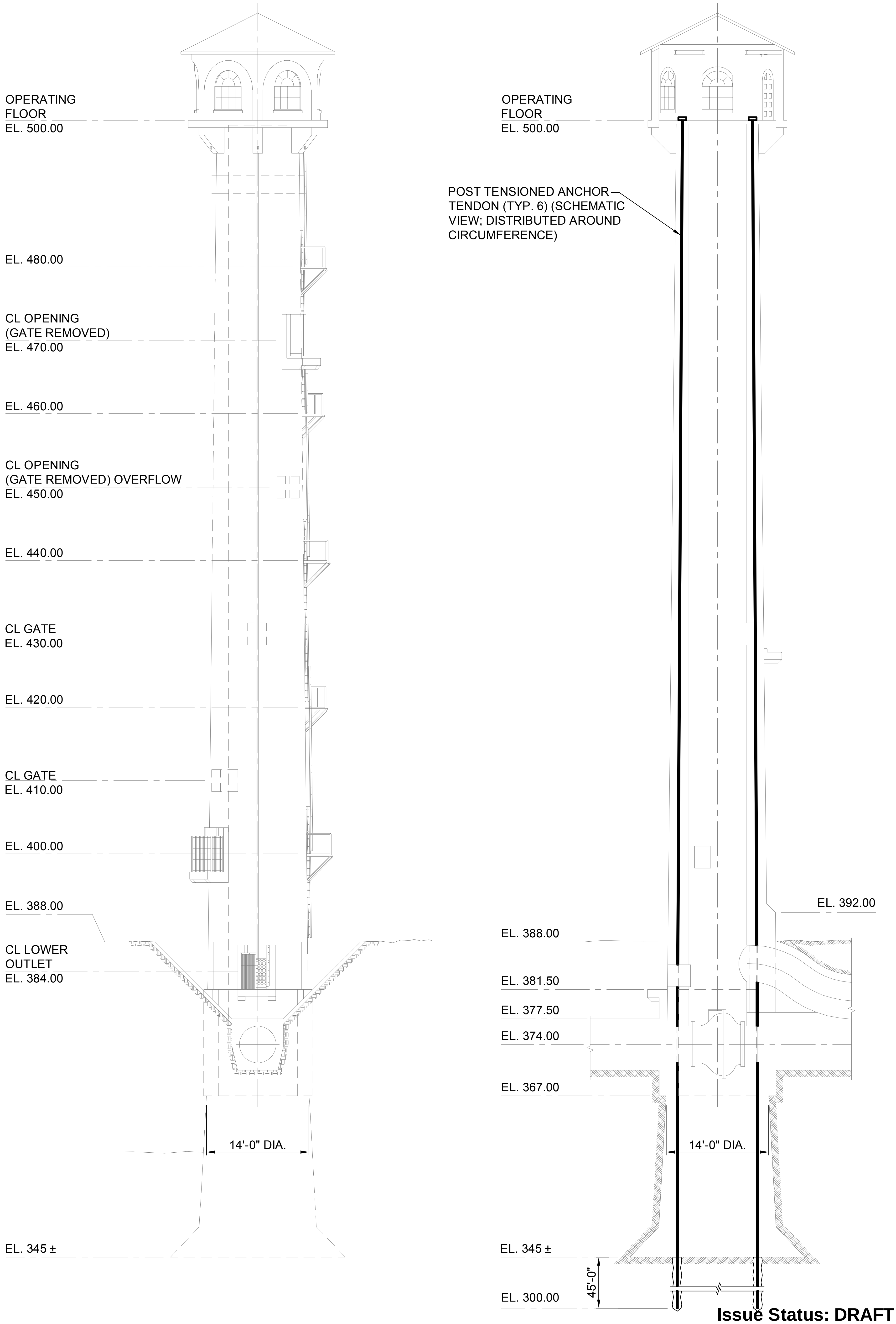
Issue Status: DRAFT

LAFAYETTE RESERVOIR OUTLET TOWER
RETROFIT PROJECT
EBMUD, LAFAYETTE, CA
Project No.: 60572210 Date: 2018-08-16

EXISTING CONDITIONS

AECOM

Figure: SSK-1



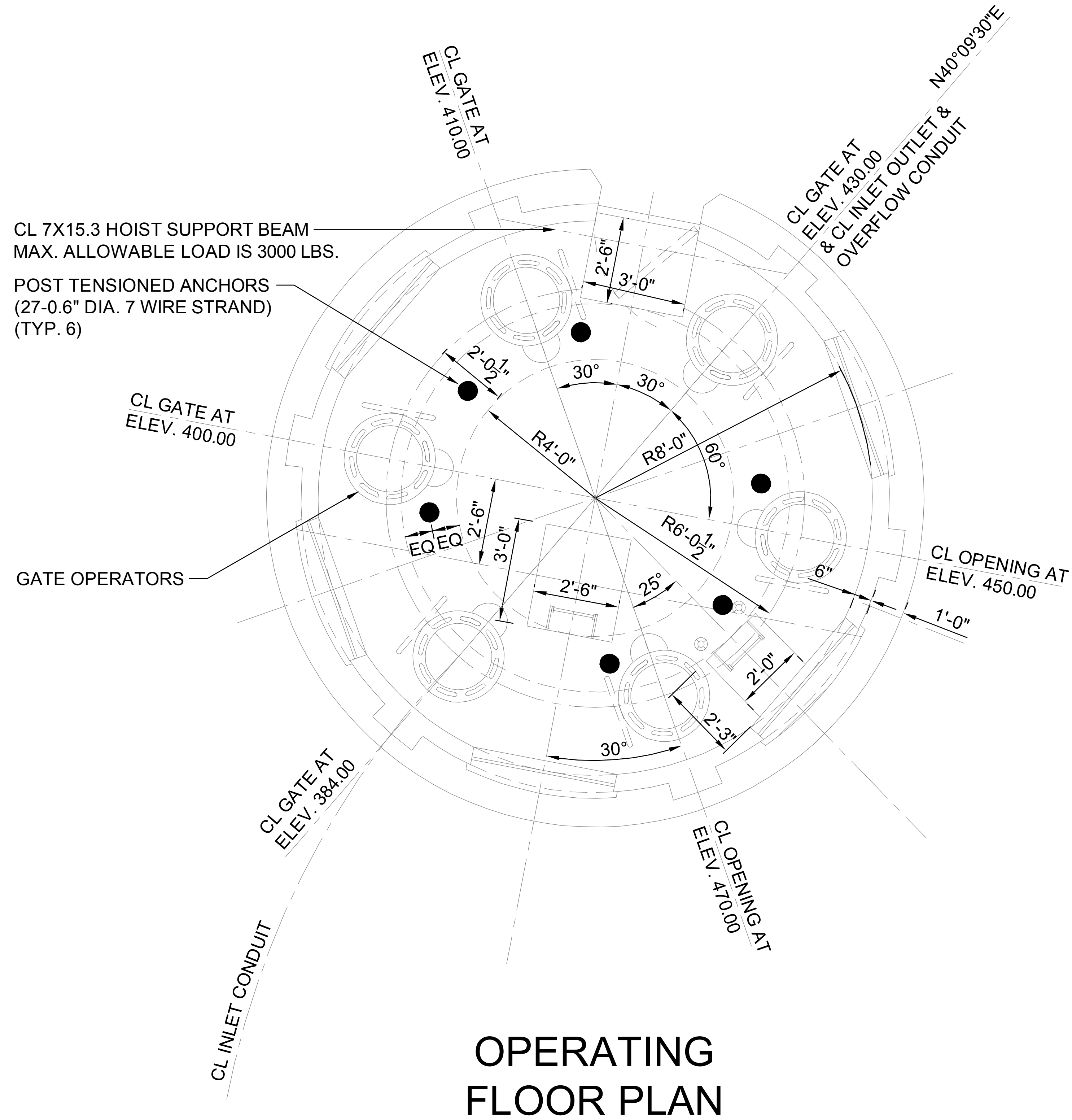
Issue Status: DRAFT

LAFAYETTE RESERVOIR OUTLET TOWER RETROFIT PROJECT
EBMUD, LAFAYETTE, CA
Project No.: 60572210 Date: 2018-08-16

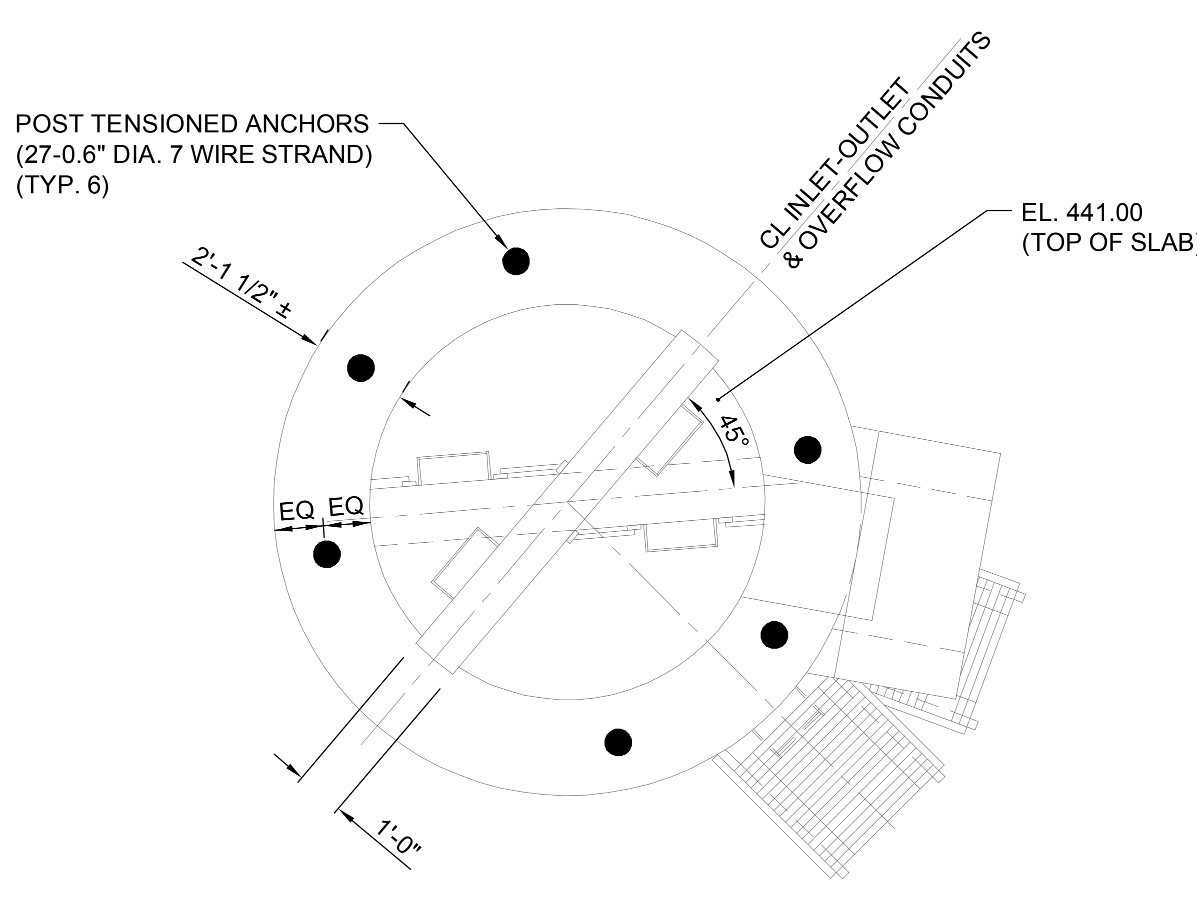
ALTERNATIVE 1 - THROUGH-WALL POST TENSIONING

AECOM

Figure: SSK-2



OPERATING FLOOR PLAN



PLAN AT EL. 450.00

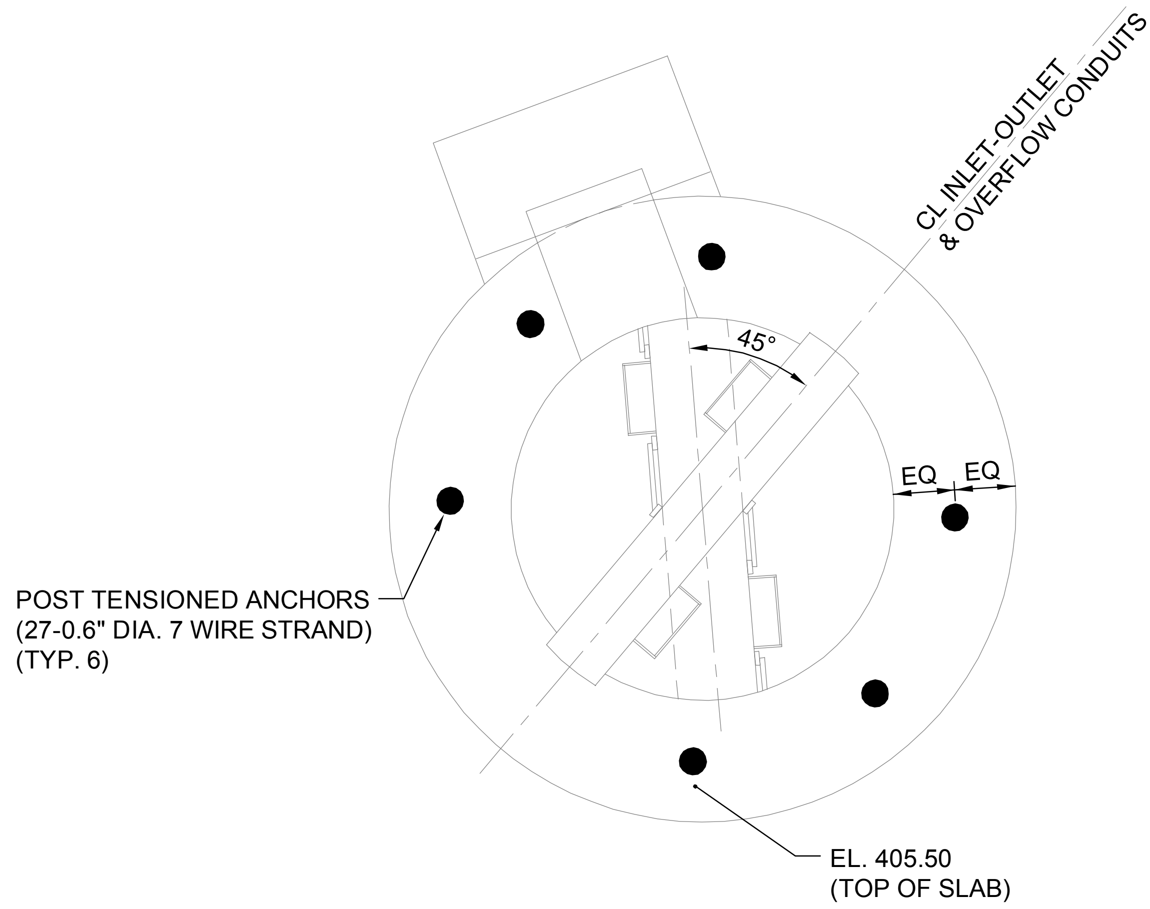
Issue Status: DRAFT

LAFAYETTE RESERVOIR OUTLET TOWER RETROFIT PROJECT
EBMUD, LAFAYETTE, CA
Project No.: 60572210 Date: 2018-08-16

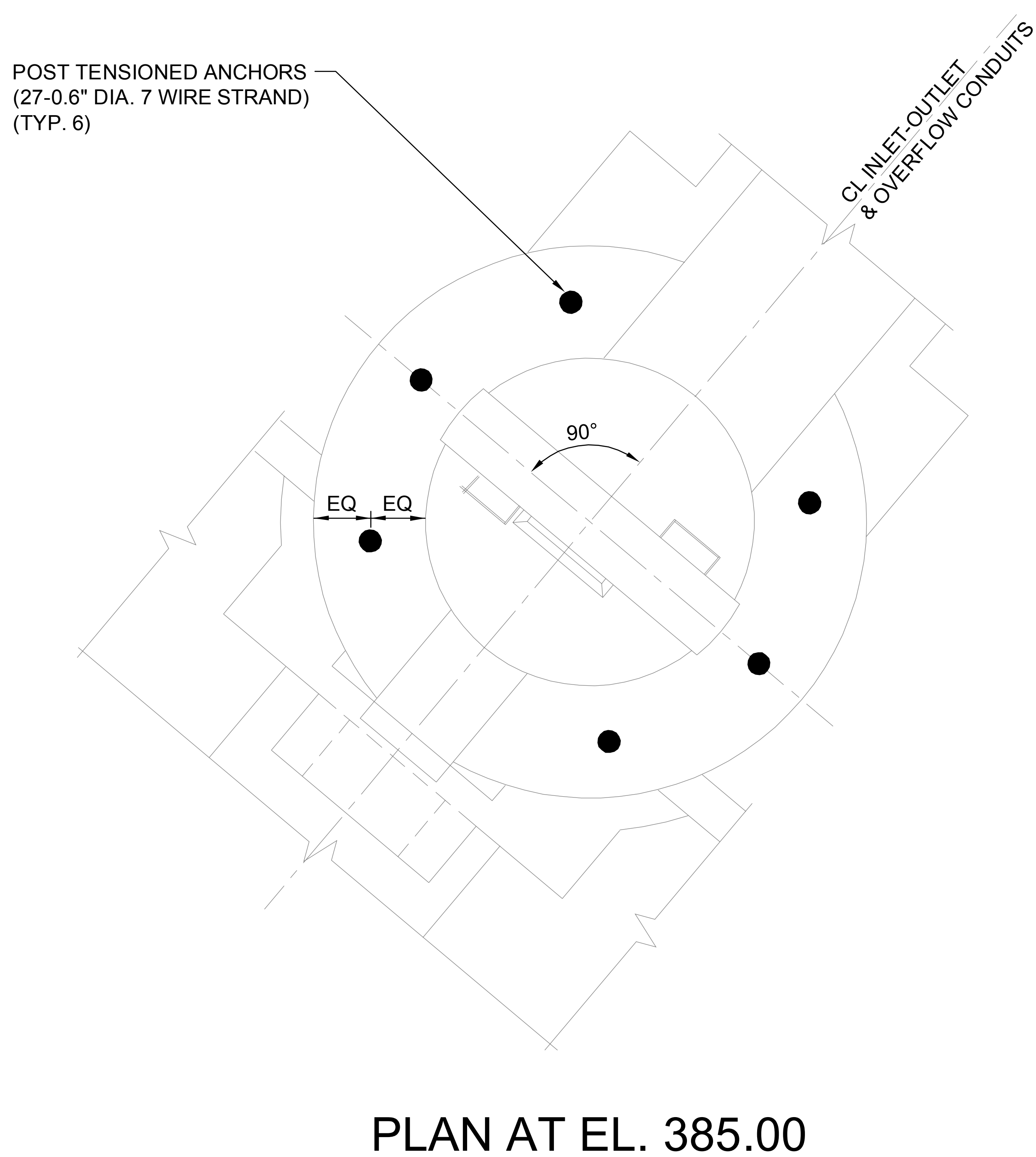
ALTERNATIVE 1 - THROUGH-WALL POST TENSIONING

AECOM

Figure: SSK-3

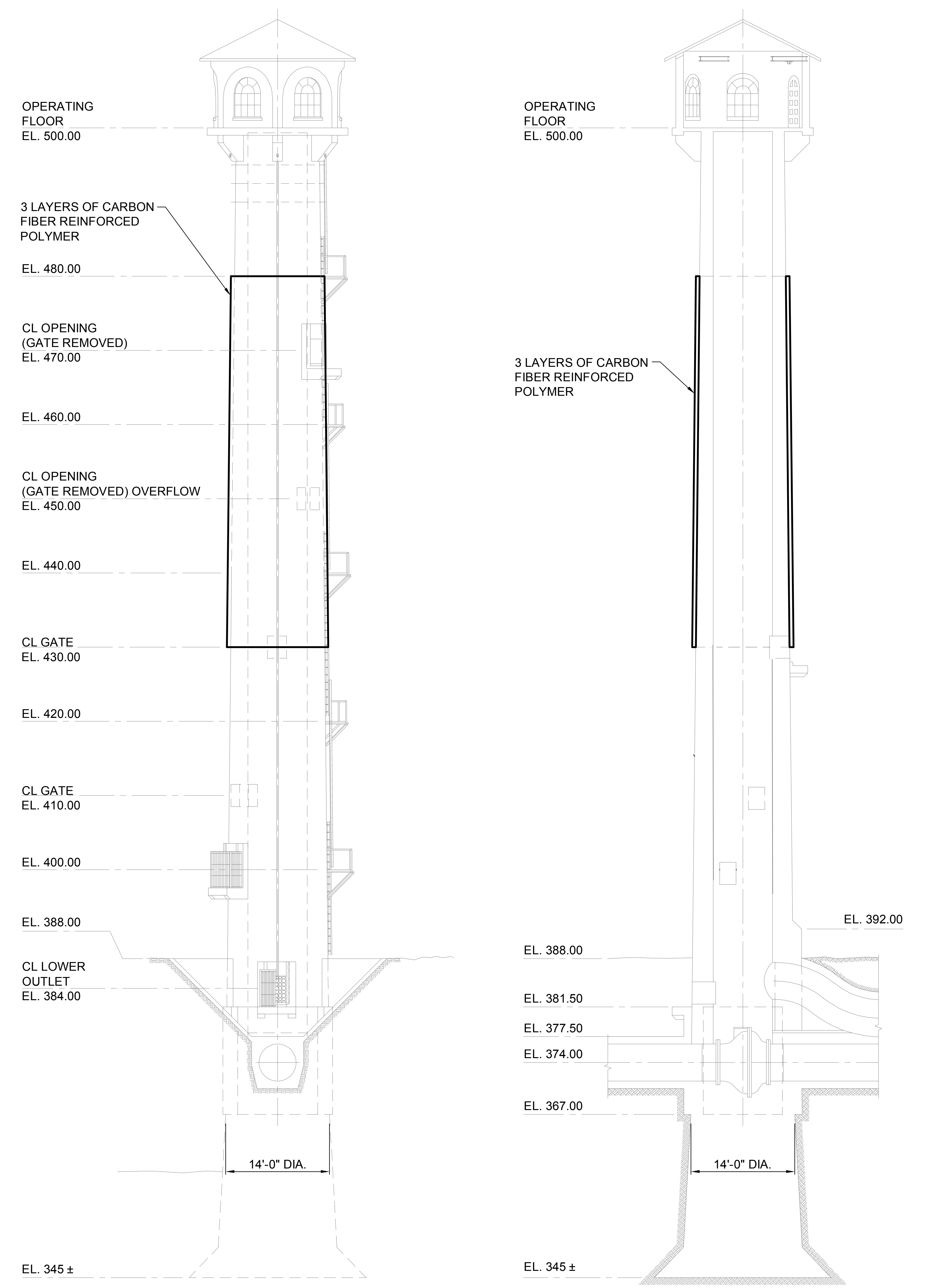


PLAN AT EL. 410.00



PLAN AT EL. 385.00

Issue Status: DRAFT



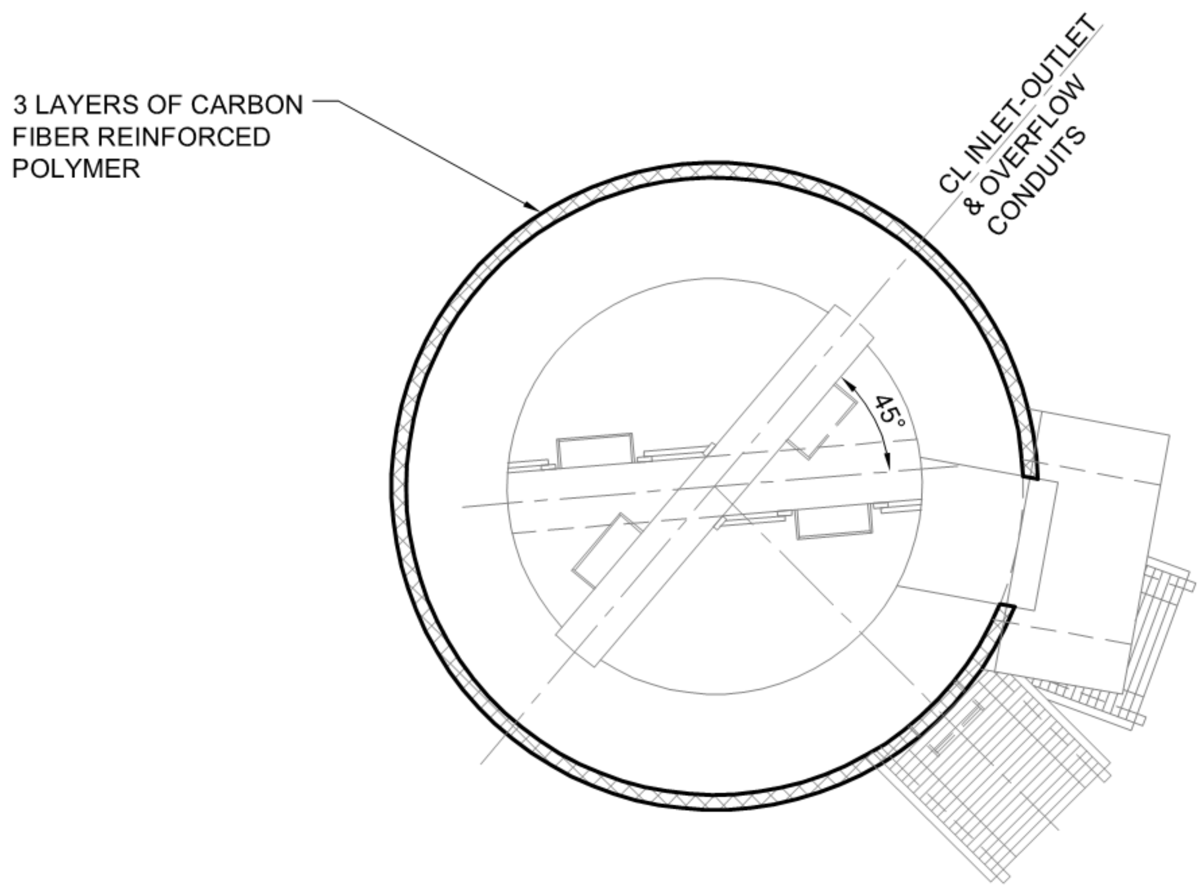
Issue Status: DRAFT

LAFAYETTE RESERVOIR OUTLET TOWER RETROFIT PROJECT
EBMUD, LAFAYETTE, CA
Project No.: 60572210 Date: 2018-08-16

ALTERNATIVE 2 - EXTERNAL FIBER-WRAPPING

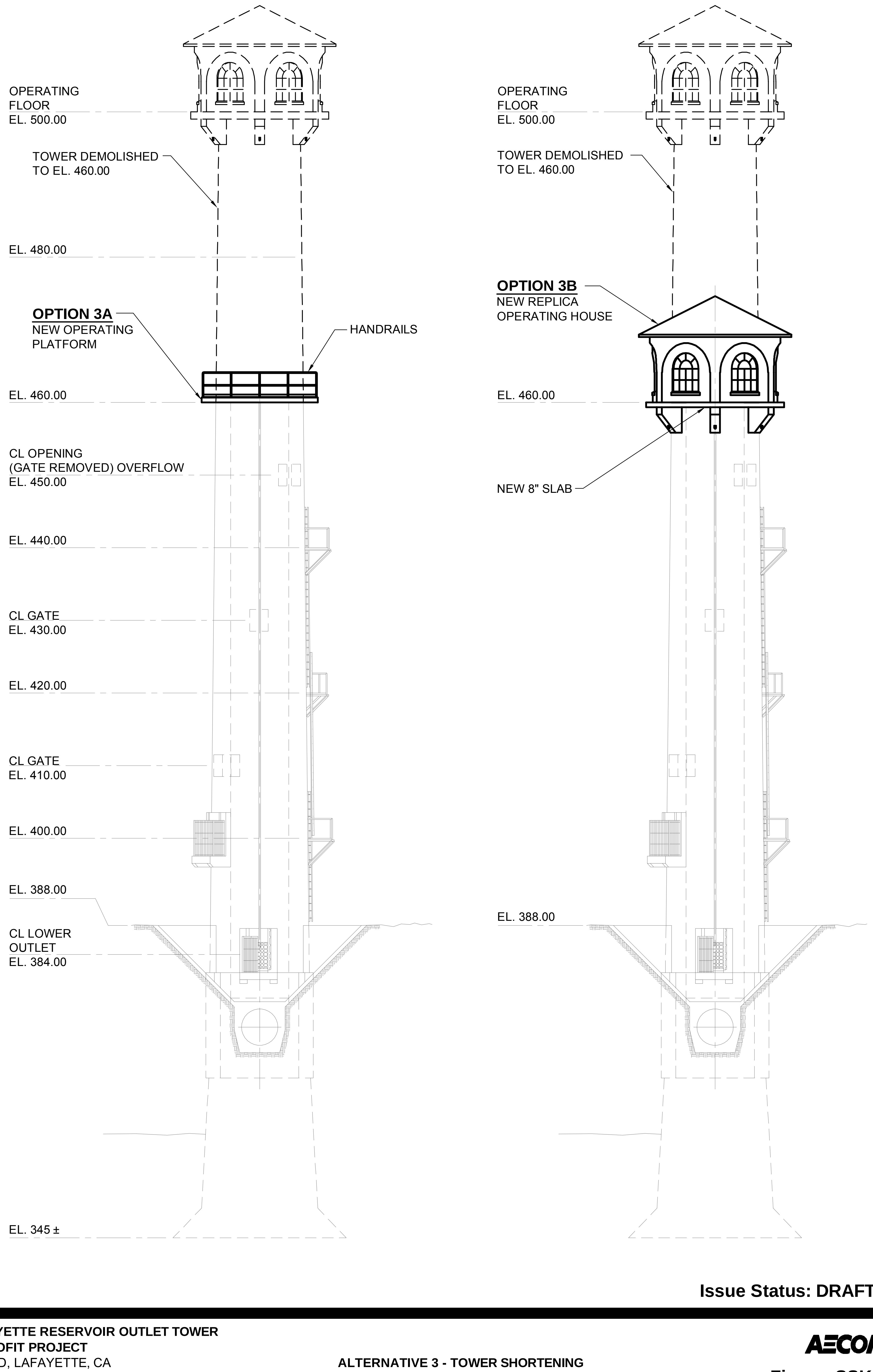
AECOM

Figure: SSK-5



PLAN AT EL. 450.00

Issue Status: DRAFT



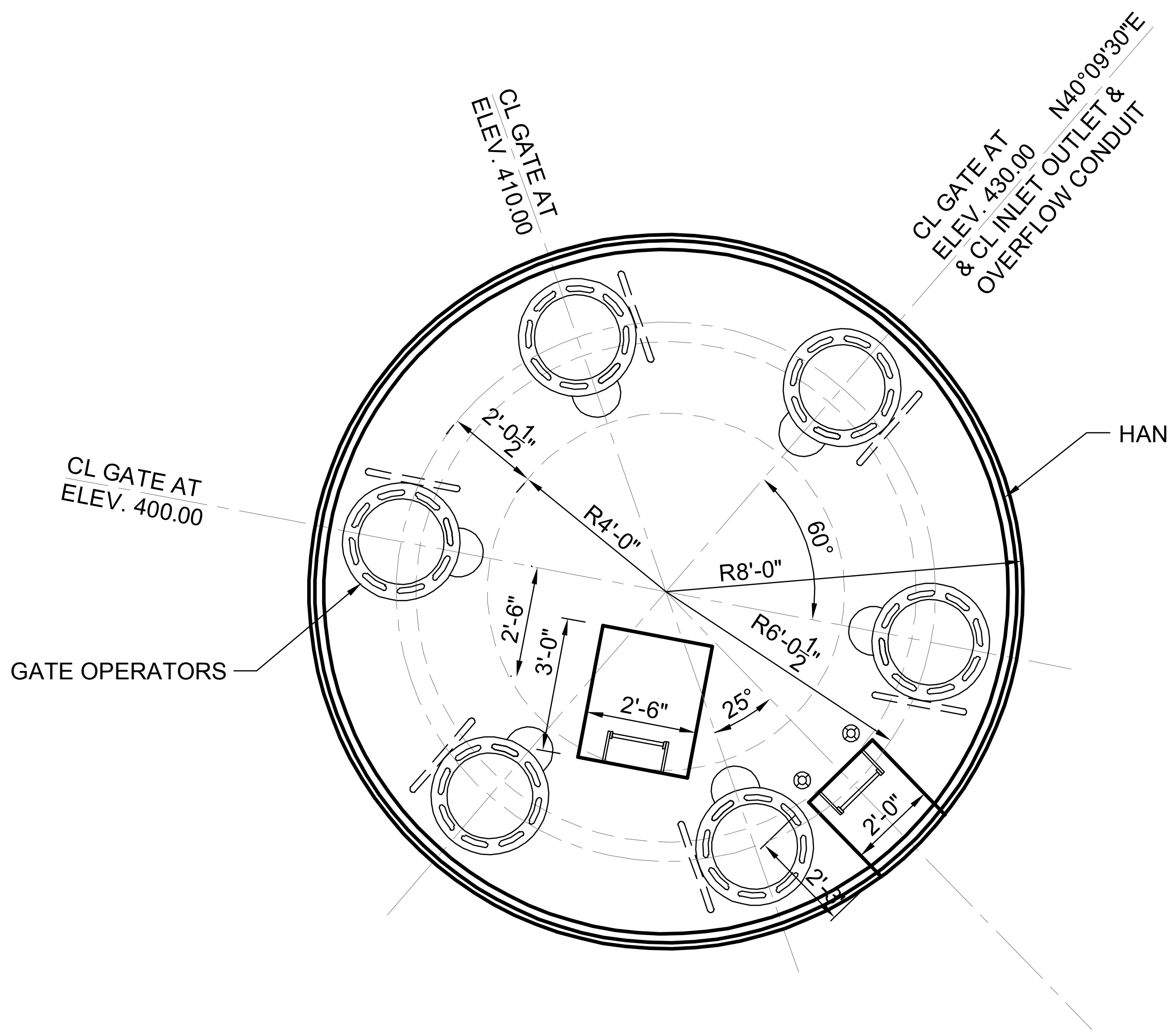
Issue Status: DRAFT

LAFAYETTE RESERVOIR OUTLET TOWER
RETROFIT PROJECT
EBMUD, LAFAYETTE, CA
Project No.: 60572210 Date: 2018-08-16

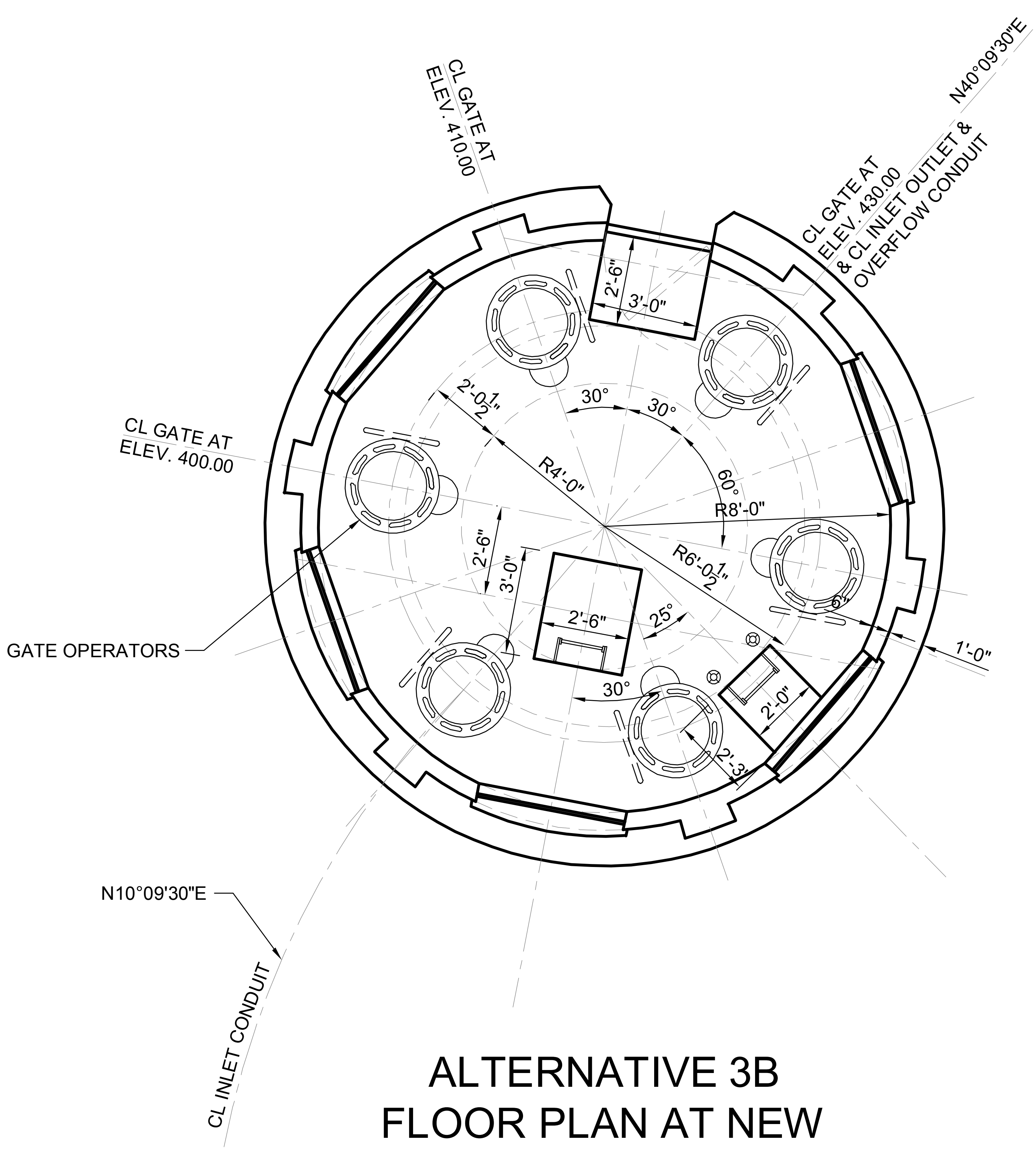
ALTERNATIVE 3 - TOWER SHORTENING

AECOM

Figure: SSK-7



OPTION 3A
FLOORPLAN AT NEW PLATFORM



ALTERNATIVE 3B
FLOOR PLAN AT NEW
OPERATING HOUSE

Issue Status: DRAFT

Appendix B

Cost Estimates of Retrofit Alternatives

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI Code	ITEM	DESCRIPTION	Quantity	Unit	Unit Price	Total
02 00 00		**DEMOLITION, STRUCTURAL EXCAVATION, AND GRADING**				$91,339.83
03 00 00		**CONCRETE**				$0.00
03 70 00		**DRILLED ANCHORS**				$359,174.59
05 00 00		**METALS**				$12,430.41
09 00 00		**FINISHES**				$0.00
23 00 00		**MECHANICAL**				$514,502.25
26 00 00		**ELECTRICAL**				$28,750.00
27 00 00		**COMMUNICATION (EEL)**				$0.00
31 00 00		**EARTH WORK**				$0.00
		SUBTOTAL, TRADE COST				**$1,006,197.09**
		MBE/DBE				**$100,619.71**
		GENERAL CONDITIONS SEE ATTACHED BREAKDOWN				$2,254,500.05
		NEW SUBTOTAL				**$3,361,316.85**
		DESIGN CONTINGENCY @ 50%				$1,680,658.42
		INSURANCE @5%				$252,098.76
		OVERHEAD @ 5%				$0.00
		PROFIT @ 15%				$794,111.11
		CONTRACTOR'S PAYMENT AND PERFORMANCE BOND @3%				$182,645.55
		TOTAL				**$6,270,830.70**
		EXCLUSIONS				
	1	Premium Time to Accelearte Construction Schedule				
	2	Removal of Underground Buried Structures				
	3	Contaminated Soils Removal				
	4	Sheet Piling				
	5	Rock Removal or Blasting				
	6	Shoring and sheeting				
	7	Exscalation Cost is not included in this cost				

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations
EBMUD Lafayette Outlet Tower
DD Development (Nine Months Duration)

CSI Code	ITEM	DESCRIPTION	Quantity	Unit	Unit Price	Total
02 00 00		**BUILDING DEMOLITION**				
02 60 00		**STRUCTURAL DEMOLITION**				
		Demolition of Gate House in section				
		Remove Doors	1	EACH	$3,003.55	$3,003.55
		Remove Windows	2	EACH	$2,766.78	$5,533.55
		Remove Actuators	1	EACH	$3,003.55	$3,003.55
		Relocate Conduit (See Electrical Work)			$0.00	$0.00
		Demo Roof (Concrete)	350	SQFT	$75.91	$26,569.87
		Demo Walls	540	SQFT	$75.91	$40,993.52
		Remove debries and take it to yard and dump	0	CUYD	$434.58	$0.00
					$0.00	$0.00
		Demo Tower Wall	0	CUYD	$434.58	$0.00
		Saw Cut	250	LNFT	$37.18	$9,295.43
		Demolition and off haul concrete	0	CUYD	$434.58	$0.00
		Salvage Gate House				
		Saw Cut at gate House	0	LNFT	$39.31	$0.00
		Hold Gate house with crane and Forklift	0	EACH	$6,007.10	$0.00
		Remove and Salvage Gate house	0	EACH	$6,007.10	$0.00
		Demo Platform and salvage for future use	3	EACH	$980.12	$2,940.36
02 21 15		**EXCAVATION, FILLING, AND BACKFILL FOR STRUCTURES (EST)**				
					$0.00	$0.00
					SUBTOTAL	**$91,339.83**
03 00 00		**CAST-IN-PLACE CONCRETE**				
03 10 00		**CONCRETE FORM WORK (EST)**				
		Form Work for new Gate House				
		Walls	704	SQFT	$30.14	$21,218.92
		Shoring for slab	278	SQFT	$74.99	$20,846.49
						$0.00
03 20 00		**CONCRETE REINFORCEMENT (EST)**				
		Reinforcement for wall	3250.00	LBS	$10.10	$32,812.89
		Reinforcement for Roof Slab	1017.00	LBS	$10.10	$10,267.91
					$0.00	$0.00
03 30 00		**CAST-IN-PLACE CONCRETE (EST & ARCH)**				
		Place Concrete at bend				
		Concrete for Wall	15	CUYD	$639.14	$9,587.10
		Concrete for Roof Slab	15	CUYD	$639.14	$9,587.10
					$0.00	$0.00
03 70 00		**DRILLED PIERS AT CONCRETE Wall**				
		Drill Hole up to 200 LNFT in Concrete Walls ($ 28,500/Hole)	6	EACH	$15,719.20	$94,315.20
		Furnish and Install new anchor	6	EACH	$16,298.55	$97,791.30
		Install coupler at 50 feet section	24	EACH	$1,751.78	$42,042.60
		Install A frame to hold Anchor for coupler installation	18	EACH	$763.96	$13,751.33
		Door	1	EACH	$2,271.25	$2,271.25
		Window	2	EACH	$2,341.25	$4,682.50
					$0.00	$0.00
					SUBTOTAL	**$359,174.59**

Cost Estimate Detail
Option-1 - PT Anchor at Six Locations
EBMUD Lafayette Outlet Tower
DD Development (Nine Months Duration)

CSI Code	ITEM	DESCRIPTION	Quantity	Unit	Unit Price	Total
05 00 00		**STRUCTURAL STEEL**				
05 55 00		**METAL FABRICATIONS**				
05 52 13		**PIPE AND TUBE BRACE**				
		Put Back Platform	3	EACH	$1,004.62	$3,013.86
		Put Anchor Bolt to Ladder	6	EACH	$537.38	$3,224.25
		Handrail	25	LNFT	$247.69	$6,192.30
					SUBTOTAL	**$12,430.41**
09 00 00		**FINISHES**				
					$0.00	$0.00
23 00 00		**MECHANICAL**				
		Design/shop drawings for sluice gate	1	EACH	$17,250.00	$17,250.00
		Fabricate/deliver sluice gate (Including Installation)	1	EACH	$398,362.50	$398,362.50
		Dewatering inside of tower	1	EACH	$95,929.25	$95,929.25
		Actuator Replacement	1	EACH	$2,960.50	$2,960.50
					SUBTOTAL	**$514,502.25**
					SUBTOTAL	**$0.00**
26 00 00		**ELECTRICAL**				
		Electrical Allowance	1	LSUM	$28,750.00	$28,750.00
					$0.00	$0.00
					SUBTOTAL	**$28,750.00**
27 00 00		**COMMUNICATION (EEL)**				
27 05 00		**COMMON WORK RESULTS FOR COMMUNICATIONS**				
					$0.00	$0.00
					SUBTOTAL	**$0.00**

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00008		LABORERS		WK		$89,280
00010		TEAMSTERS		WK		$0
00054		FIELD OFFICE		LS		$13,650
00056		FIELD OFFICE EQUIPMENT & FURNITURE		LS		$2,600
00060		TELEPHONE SETUP, INSTALLATION, AND USAGE		LS		$6,550
00062		WATER		LS		$3,600
00070		PRINTING COSTS		LS		$2,600
00072		SHIPPING / MESSENGER / POSTAGE		LS		$1,800
00076		SMALL TOOLS AND SUPPLIES		WK		$3,600
00078		TEMPORARY UTILITIES		LS		$2,050
00080		TEMPORARY HEAT / WEATHER PROTECTION		LS		$37,350
00082		SAFETY & PROTECTIVE EQUIPMENT		LS		$194,500
00084		SCAFFOLDING SETUP		LS		$0
00086		TEMPORARY BARRICADES		LS		$0
00088		FENCING		LS		$3,000
00090		SCHEDULING		LS		$3,750
00092		REPORTING / PHOTOS		LS		$6,300
00094		EXPEDITING / PERMITS		LS		$5,000
00096		DUMPSTERS		LS		$2,400
00098		RODENT CONTROL		LS		$0
00100		FINAL CLEAN-UP		SF		$2,500
00102		PROJECT ADMINISTRATION		LS		$210,200
00104		PROJECT SUPERVISION		LS		$594,630
00110		SURVEYING		LS		$18,600
00112		TESTING & INSPECTIONS		LS		$0
00114		SPECIAL EQUIPMENT / RENTAL		LS		$1,050,540
		TOTAL GENERAL CONDITIONS				**$2,254,500**

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00008		**LABORERS**				
	1	General for All Trades 1 Laborers	36	WK	2,480.00	89,280
					SUBTOTAL	**$89,280**
00010		**TEAMSTERS**				
	1	Teamster		HRS		0
					SUBTOTAL	**$0**
00054		**FIELD OFFICE**				
	1	Field Office Trailer Set-up	1	LS	2,200.00	2,200
	2	Field Office Trailer Rental	9	MOS	600.00	5,400
	3	Field Office Maintenance	9	MOS	200.00	1,800
	4	Temporary Toilets	9	MOS	250.00	2,250
	5	Storage Trailer	0	MOS	200.00	0
	6	Temporary utilities	1	LSUM	2,000.00	2,000
					SUBTOTAL	**$13,650.00**
00056		**FIELD OFFICE EQUIPMENT & FURNITURE**				
	1	Field Office Furniture, Desks, Chairs, Conference Table	1	LS	1,100.00	1,100
	2	Office Equipment / Fax - Copier -	1	LS	1,500.00	1,500
	3					0
					SUBTOTAL	**$2,600**
00060		**TELEPHONE EQUIPMENT & CHARGES**				
	1	Set-up Field Office Telephone, 1 Lines	1	EA	250.00	250
	2	Telephone charges	9	MOS	100.00	900
	3	Cell Phone	9	EA	600.00	5,400
					SUBTOTAL	**$6,550**
00062		**WATER**				
	1	Water Cooler Rental	9	MOS	250.00	2,250
	2	Water / Potable	9	MOS	150.00	1,350
	3					0
					SUBTOTAL	**3600**

Cost Estimate Detail
Option-1 - PT Anchor at Six Locations
EBMUD Lafayette Outlet Tower
DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00070		**PRINTING COSTS**				
	1	Record Set / Contract	1	LS	2,000.00	2,000
	2	Shop Drawings / Progress	1	MOS	300.00	300
	3	Blueprinting	1	LS	300.00	300
					SUBTOTAL	**$2,600**
00072		**SHIPPING / MESSENGER / POSTAGE**				
	1	Overnight Mail / Shipping	9	MOS	200.00	1,800
	2	Others				0
					SUBTOTAL	**$1,800**
00076		**SMALL TOOLS AND SUPPLIES**				
	1	Small Tools Allowance	36	WK	100.00	3,600
	2	Others				0
					SUBTOTAL	**$3,600**
00078		**TEMPORARY UTILITIES / CHARGES**				
	1	Temp Electric Utilities	9	MOS	200.00	1,800
	2	Temp Electric / Last Month During Testing	1	LS	250.00	250
	3	Others				0
					SUBTOTAL	**$2,050**
00080		**TEMPORARY HEAT / WEATHER PROTECTION**				
	1	Temporary Heating	9	MO	200.00	1,800
	2	Temporary Weather Protection	9	LS	200.00	1,800
	3	SWPPP	9	MO	3,750.00	33,750
					SUBTOTAL	**$37,350**
00082		**SAFETY & PROTECTIVE EQUIPMENT**				
	1	Safety Training	1	LS	5,500.00	5,500
	2	Safety Coordinator, 2 Hours Per Week	1,440	HRS	125.00	180,000
	3	Protective equipment	1	LSUM	1,500.00	1,500
	4	Added training	100	HRS	75.00	7,500
					SUBTOTAL	**$194,500**
00084		**SCAFFOLDING**				
	1	Hanging Scaffold Interior Lift	1	EA	0.00	0
	2	Scaffold Building One Time Erection and Takedown	1	LS	0.00	0
	3	Monthly Rental	1	MOS	0.00	0
	4	Others				0
					SUBTOTAL	**$0**

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00086		**TEMPORARY BARRICADES**				
	1	2" x 4" Wood Framing with Plywood Sheathing		LF		0
	2	Temporary Barricades / Elevator Openings, Stairs		ALLOW		0
	3	Temporary Barricades Building Perimeter		LF		0
					SUBTOTAL	**$0**
00088		**FENCING**				
	1	Temp. Chain Link Fence @ Site	100	LF	30.00	3,000
	2	Fence Gates 12 Feet Double Gate	0	EA	500.00	0
	3					
					SUBTOTAL	**$3,000**
00090		**SCHEDULING**				
	1	Set-up CPM Schedule	1	LS	1,500.00	1,500
	2	Update Schedule for Monthly Reporting	9	MOS	250.00	2,250
	3					0
					SUBTOTAL	**$3,750**
00092		**REPORTING / PHOTOS**				
	1	Stationary for Reporting	9	MOS	500.00	4,500
	2	Progress Photos	9	MOS	100.00	900
	3	Misc. Photos by Field Staff	9	MOS	100.00	900
	4					0
					SUBTOTAL	**$6,300**
00094		**EXPEDITING / PERMITS**				
	1	Expediting Service		ALLOW		0
	2	Permits / Fees	1	ALLOW	5,000.00	5,000
	3					0
					SUBTOTAL	**$5,000**
00096		**TRASH CONTAINERS / DUMPSTERS**				
	1	Allow 1 Container Per Month	2	EA	1,200.00	2,400
	2					0
					SUBTOTAL	**$2,400**

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00098		**RODENT CONTROL**				
	1	Exterminating Services, Initial Visit	0	LS	500.00	0
	2	Regular Maintenance	0	MOS	200.00	0
	3					0
					SUBTOTAL	**$0**
00100		**FINAL CLEAN-UP**				
	1	Final Clean-up Allowance	5,000	SF	0.50	2,500
					SUBTOTAL	**$2,500**
00102		**PROJECT ADMINISTRATION**				
	1	Project Manager, Buyout	160	HRS	100.00	16,000
	2	Project Manager, Part Time	1,782	HRS	100.00	178,200
	3	Project Manager, Closeout	160	HRS	100.00	16,000
	4					0
				HRS	**SUBTOTAL**	**$210,200**
00104		**PROJECT SUPERVISION**				
	1	Field Superintendent - 9 Mo Full Time	1,440	HRS	85.00	122,400
	2	Asst Superintendent	1,440	HRS	75.00	108,000
	3	Quality Control Engineer	1,440	HRS	85.00	122,400
	4	Testing and Inspection	720	HRS	85.00	61,200
	5	Project Engineer	1,782	HRS	65.00	115,830
	6	Ofice Administartion	720	HRS	45.00	32,400
	7	Payroll clark	720	HRS	45.00	32,400
					SUBTOTAL	**$594,630**
00110		**SURVEYING**				
	1	Survey and Layout Footings	0	EA	1,200.00	0
	2	Survey and Layout Column Lines	0	EA	1,200.00	0
	3	Survey and Layout Curbs		EA		0
	4	Survey and Layout Utilities		EA		0
	5	Survey and Layout Retaining Wall		EA		0
	6	Survey and Layout Drainage		EA		0
	6	Survey measurement and fabrictation	3	EA	1,200.00	3,600
	8	Final Survey for New Building	0	EA	1,200.00	0
	8	SWPPP Permit Cost	1	EA	15,000.00	15,000
					SUBTOTAL	**$18,600**

Cost Estimate Detail

Option-1 - PT Anchor at Six Locations

EBMUD Lafayette Outlet Tower

DD Development (Nine Months Duration)

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00112		**TESTING & INSPECTIONS**				
	1	Allowance For Testing Laboratory For Concrete	1	LS	0.00	0
	2	Allowance For Testing Laboratory For Subsurface Soils	1	LS	0.00	0
	3	Allowance For Testing Laboratory For Compaction		LS	0.00	0
	4	Allowance For Testing Laboratory For Welding	1	LS	0.00	0
	5	Allowance For Testing Laboratory Mechanical Systems	1	LS	0.00	0
	6	Allowance For Testing Laboratory Fire and Sprinkler Systems		LS		0
					SUBTOTAL	**$0**
00114		**SPECIAL EQUIPMENT / RENTAL**				
	1	Barge Mobilization (35' X 175')	1	EA	22,000.00	22,000
	1	Standby time for barge	0	MO	32,000.00	0
	2	Excavator	1	EA	1,500.00	1,500
	3	Manlift	1	EA	1,200.00	1,200
	4	Air Compressor	1	EA	500.00	500
	6	Compactor	1	EA	1,000.00	1,000
	7	Misc Tools	1	LS	1,000.00	1,000
	8	Standby time for Misc Equipment	1	LS	0.00	0
	9	Crane Rental for 9 Months (2 Cranes LINK-BELT HSP 8060) Rental rate is $136.20/Hour. Assuming crane usage is average 4 Hours/Day	9	MO	45,762.00	411,858
	10	Barge Rental Rate	9	MO	41,236.00	371,124
	11	Tug Boat	9	MO	16,457.00	148,113
	12	Manlift	9	MO	10,249.45	92,245
						0
						0
						0
						0
						0
					SUBTOTAL	**$1,050,540**

TOTAL GENERAL CONDITIONS===========>>>>> $2,254,500

Cost Estimate Detail
Option 3A
EBMUD Lafayette Outlet Tower
DD Development
Demo Gate House and put new 12" Concrete slab (Seven Month Duration)

CSI Code	ITEM	DESCRIPTION	Quantity	Unit	Unit Price	Total
02 00 00		**DEMOLITION, STRUCTURAL EXCAVATION, AND GRADING**				$110,103.22
03 00 00		**CONCRETE**				$23,459.77
03 70 00		**DRILLED PIERS**				$0.00
05 00 00		**METALS**				$0.00
09 00 00		**FINISHES**				$0.00
23 00 00		**MECHANICAL**				$497,252.25
26 00 00		**ELECTRICAL**				$25,000.00
27 00 00		**COMMUNICATION (EEL)**				$0.00
31 00 00		**EARTH WORK**				$0.00
		SUBTOTAL, TRADE COST				**$655,815.24**
		MBE/DBE				**$65,581.52**
		GENERAL CONDITIONS SEE ATTACHED BREAKDOWN				$1,770,246.15
		NEW SUBTOTAL				**$2,491,642.91**
		DESIGN CONTINGENCY @ 50%				$1,245,821.45
		INSURANCE @5%				$186,873.22
		OVERHEAD @ 5%				$0.00
		PROFIT @ 15%				$588,650.64
		CONTRACTOR'S PAYMENT AND PERFORMANCE BOND @3%				$135,389.65
		TOTAL				**$4,648,377.87**
		EXCLUSIONS				
	1	Premium Time to Accelearte Construction Schedule				
	2	Removal of Underground Buried Structures				
	3	Contaminated Soils Removal				
	4	Sheet Piling				
	5	Rock Removal or Blasting				
	6	Shoring and sheeting				
	7	Exscalation Cost is not included in this cost				

02 00 00		**BUILDING DEMOLITION**				
02 60 00		**STRUCTURAL DEMOLITION**				
		Demolition of Gate House				
		Demolition of Gate House in section				
		Remove Doors	1	EACH	$3,003.55	$3,003.55
		Remove Windows	2	EACH	$2,766.78	$5,533.55
		Remove Actuators	1	EACH	$3,003.55	$3,003.55
		Relocate Conduit (See Electrical Work)			$0.00	$0.00
		Demo Roof (Concrete)	350	SQFT	$75.91	$26,569.87
		Demo Walls	540	SQFT	$75.91	$40,993.52
		Remove debries and take it to yard and dump	0	CUYD	$434.58	$0.00
					$0.00	$0.00
		Demo Tower Wall	100	CUYD	$217.04	$21,703.75
		Saw Cut	250	LNFT	$37.18	$9,295.43
		Salvage Gate House				
		Saw Cut at gate House	0	LNFT	$39.31	$0.00
		Hold Gate house with crane and Forklift	0	EACH	$6,007.10	$0.00
		Remove and Salvage Gate house	0	EACH	$6,007.10	$0.00
					$0.00	$0.00
02 21 15		**EXCAVATION, FILLING, AND BACKFILL FOR STRUCTURES (EST)**				
					$0.00	$0.00
					SUBTOTAL	**$110,103.22**
03 00 00		**CAST-IN-PLACE CONCRETE**				
03 10 00		**CONCRETE FORM WORK (EST)**				
		Put new Concrete Slab 12" Thick			$0.00	$0.00
		Shoring for slab	185	SQFT	$74.99	$13,872.66
		Walls	0	SQFT	$30.14	$0.00
					$0.00	
03 20 00		**CONCRETE REINFORCEMENT (EST)**				
		Reinforcement for wall	0.00	LBS	$2.20	$0.00
		Reinforcement for Roof Slab	0.00	LBS	$2.20	$0.00
03 30 00		**CAST-IN-PLACE CONCRETE (EST & ARCH)**				
		Place Concrete at bend				
		Concrete for Wall	0	CUYD	$639.14	$0.00
		Concrete for Roof Slab Platform	15	CUYD	$639.14	$9,587.10
		Door	0	EACH	$2,271.25	$0.00
		Window	0	EACH	$2,341.25	$0.00
					$0.00	$0.00
					SUBTOTAL	**$23,459.77**
03 70 00		**DRILLED PIERS AT CONCRETE SLAB**				
					$0.00	$0.00
					SUBTOTAL	**$0.00**

05 00 00		**STRUCTURAL STEEL**				
05 55 00		**METAL FABRICATIONS**				
05 52 13		**PIPE AND TUBE BRACE**				
		Brackets	6	EACH	$1,261.33	$7,567.95
		Shear Reinforcement (Two Galvanized Steel Plate 1/2" X 8'1" X 21' Long with 3/4" dia epoxy stainless steel bolts	2	EACH	$47,228.00	$94,456.00
		3/4" dia epoxy stainless steel bolts	80	EACH	$391.70	$31,335.86
		3/4" X 12" Deep in Concrete Wall Drill Holes	80	EACH	$296.05	$23,683.86
		Install, Dewater, and Remove scaffolding				
		Scaffolding	1	LSUM	$39,771.00	$39,771.00
		Metal Railing	65	LNFT	$282.44	$18,358.44
		Put Back Platform	3	EACH	$1,004.62	$3,013.86
		Put Anchor Bolt to Ladder	6	EACH	$537.38	$3,224.25
					$0.00	$0.00
					SUBTOTAL	
09 00 00		**FINISHES**				
					$0.00	$0.00
					SUBTOTAL	**$0.00**
23 00 00		**MECHANICAL**				
		Design/shop drawings for sluice gate	1	EACH	$0.00	$0.00
		Fabricate/deliver sluice gate	1	EACH	$398,362.50	$398,362.50
		Dewatering inside of tower	1	EACH	$95,929.25	$95,929.25
		Actuator Replacement	1	EACH	$2,960.50	$2,960.50
					SUBTOTAL	**$497,252.25**
26 00 00		**ELECTRICAL**				
		Electrical Allowance	1	LSUM	$25,000.00	$25,000.00
					$0.00	$0.00
					SUBTOTAL	**$25,000.00**
27 00 00		**COMMUNICATION (EEL)**				
27 05 00		**COMMON WORK RESULTS FOR COMMUNICATIONS**				
					$0.00	$0.00
					SUBTOTAL	**$0.00**
31 00 00		**EARTH WORK**				
					$0.00	$0.00
					$0.00	$0.00
					$0.00	$0.00
					$0.00	$0.00
					SUBTOTAL	**$0.00**

Cost Estimate Detail

Option-3 - Cut Tower

EBMUD Lafayette Outlet Tower

DD Development (Seven Months Duration)

Demo Gate House and put new 12" Concrete slab

CSI CODE	ITEM	DESCRIPTION	QUANTITY	UNIT	PRICE	TOTAL
00008		LABORERS		WK		$69,440
00010		TEAMSTERS		WK		$0
00054		FIELD OFFICE		LS		$11,550
00056		FIELD OFFICE EQUIPMENT & FURNITURE		LS		$2,600
00060		TELEPHONE SETUP, INSTALLATION, AND USAGE		LS		$5,150
00062		WATER		LS		2800
00070		PRINTING COSTS		LS		$2,600
00072		SHIPPING / MESSENGER / POSTAGE		LS		$1,400
00076		SMALL TOOLS AND SUPPLIES		WK		$2,800
00078		TEMPORARY UTILITIES		LS		$1,650
00080		TEMPORARY HEAT / WEATHER PROTECTION		LS		$29,050
00082		SAFETY & PROTECTIVE EQUIPMENT		LS		$156,250
00084		SCAFFOLDING SETUP		LS		$0
00086		TEMPORARY BARRICADES		LS		$0
00088		FENCING		LS		$3,000
00090		SCHEDULING		LS		$3,250
00092		REPORTING / PHOTOS		LS		$4,900
00094		EXPEDITING / PERMITS		LS		$5,000
00096		DUMPSTERS		LS		$2,400
00098		RODENT CONTROL		LS		$0
00100		FINAL CLEAN-UP		SF		$2,500
00102		PROJECT ADMINISTRATION		LS		$145,400
00104		PROJECT SUPERVISION		LS		$476,775
00110		SURVEYING		LS		$18,600
00112		TESTING & INSPECTIONS		LS		$0
00114		SPECIAL EQUIPMENT / RENTAL		LS		$823,131

TOTAL GENERAL CONDITIONS **$1,770,246**

00008		**LABORERS**				
	1	General for All Trades 1 Laborers	28	WK	2,480.00	69,440
					SUBTOTAL	**$69,440**
00010		**TEAMSTERS**				
	1	Teamster		HRS		0
					SUBTOTAL	**$0**
00054		**FIELD OFFICE**				
	1	Field Office Trailer Set-up	1	LS	2,200.00	2,200
	2	Field Office Trailer Rental	7	MOS	600.00	4,200
	3	Field Office Maintenance	7	MOS	200.00	1,400
	4	Temporary Toilets	7	MOS	250.00	1,750
	5	Storage Trailer	0	MOS	200.00	0
	6	Temporary utilities	1	LSUM	2,000.00	2,000
					SUBTOTAL	**$11,550.00**
00056		**FIELD OFFICE EQUIPMENT & FURNITURE**				
	1	Field Office Furniture, Desks, Chairs, Conference Table	1	LS	1,100.00	1,100
	2	Office Equipment / Fax - Copier -	1	LS	1,500.00	1,500
	3					0
					SUBTOTAL	**$2,600**
00060		**TELEPHONE EQUIPMENT & CHARGES**				
	1	Set-up Field Office Telephone, 1 Lines	1	EA	250.00	250
	2	Telephone charges	7	MOS	100.00	700
	3	Cell Phone	7	EA	600.00	4,200
					SUBTOTAL	**$5,150**
00062		**WATER**				
	1	Water Cooler Rental	7	MOS	250.00	1,750
	2	Water / Potable	7	MOS	150.00	1,050
	3					0
					SUBTOTAL	**2800**
00070		**PRINTING COSTS**				
	1	Record Set / Contract	1	LS	2,000.00	2,000
	2	Shop Drawings / Progress	1	MOS	300.00	300
	3	Blueprinting	1	LS	300.00	300
					SUBTOTAL	**$2,600**
00072		**SHIPPING / MESSENGER / POSTAGE**				
	1	Overnight Mail / Shipping	7	MOS	200.00	1,400
	2	Others				0
					SUBTOTAL	**$1,400**

00076		**SMALL TOOLS AND SUPPLIES**				
	1	Small Tools Allowance	28	WK	100.00	2,800
	2	Others				0
					SUBTOTAL	**$2,800**
00078		**TEMPORARY UTILITIES / CHARGES**				
	1	Temp Electric Utilities	7	MOS	200.00	1,400
	2	Temp Electric / Last Month During Testing	1	LS	250.00	250
	3	Others				0
					SUBTOTAL	**$1,650**
00080		**TEMPORARY HEAT / WEATHER PROTECTION**				
	1	Temporary Heating	7	MO	200.00	1,400
	2	Temporary Weather Protection	7	LS	200.00	1,400
	3	SWPPP	7	MO	3,750.00	26,250
					SUBTOTAL	**$29,050**
00082		**SAFETY & PROTECTIVE EQUIPMENT**				
	1	Safety Training	1	LS	5,500.00	5,500
	2	Safety Coordinator, 2 Hours Per Week	1,134	HRS	125.00	141,750
	3	Protective equipment	1	LSUM	1,500.00	1,500
	4	Added training	100	HRS	75.00	7,500
					SUBTOTAL	**$156,250**
00084		**SCAFFOLDING**				
	1	Hanging Scaffold Interior Lift	1	EA	0.00	0
	2	Scaffold Building One Time Erection and Takedown	1	LS	0.00	0
	3	Monthly Rental	1	MOS	0.00	0
	4	Others				0
					SUBTOTAL	**$0**
00086		**TEMPORARY BARRICADES**				
	1	2" x 4" Wood Framing with Plywood Sheathing		LF		0
	2	Temporary Barricades / Elevator Openings, Stairs		ALLOW		0
	3	Temporary Barricades Building Perimeter		LF		0
					SUBTOTAL	**$0**
00088		**FENCING**				
	1	Temp. Chain Link Fence @ Site	100	LF	30.00	3,000
	2	Fence Gates 12 Feet Double Gate	0	EA	500.00	0
	3					
					SUBTOTAL	**$3,000**

00090		**SCHEDULING**				
	1	Set-up CPM Schedule	1	LS	1,500.00	1,500
	2	Update Schedule for Monthly Reporting	7	MOS	250.00	1,750
	3					0
					SUBTOTAL	**$3,250**
00092		**REPORTING / PHOTOS**				
	1	Stationary for Reporting	7	MOS	500.00	3,500
	2	Progress Photos	7	MOS	100.00	700
	3	Misc. Photos by Field Staff	7	MOS	100.00	700
	4					0
					SUBTOTAL	**$4,900**
00094		**EXPEDITING / PERMITS**				
	1	Expediting Service		ALLOW		0
	2	Permits / Fees	1	ALLOW	5,000.00	5,000
	3					0
					SUBTOTAL	**$5,000**
00096		**TRASH CONTAINERS / DUMPSTERS**				
	1	Allow 1 Container Per Month	2	EA	1,200.00	2,400
	2					0
					SUBTOTAL	**$2,400**
00098		**RODENT CONTROL**				
	1	Exterminating Services, Initial Visit	0	LS	500.00	0
	2	Regular Maintenance	0	MOS	200.00	0
	3					0
					SUBTOTAL	**$0**
00100		**FINAL CLEAN-UP**				
	1	Final Clean-up Allowance	5,000	SF	0.50	2,500
	2					0
					SUBTOTAL	**$2,500**
00102		**PROJECT ADMINISTRATION**				
	1	Project Manager, Buyout	160	HRS	100.00	16,000
	2	Project Manager, Part Time	1,134	HRS	100.00	113,400
	3	Project Manager, Closeout	160	HRS	100.00	16,000
	4					0
				HRS	**SUBTOTAL**	**$145,400**

00104		**PROJECT SUPERVISION**				
	1	Field Superintendent - 9 Mo Full Time	1,440	HRS	85.00	122,400
	2	Asst Superintendent	1,134	HRS	75.00	85,050
	3	Quality Control Engineer	1,134	HRS	85.00	96,390
	4	Testing and Inspection	567	HRS	85.00	48,195
	5	Project Engineer	1,134	HRS	65.00	73,710
	6	Ofice Administartion	567	HRS	45.00	25,515
	7	Payroll clark	567	HRS	45.00	25,515
					SUBTOTAL	**$476,775**
00110		**SURVEYING**				
	1	Survey and Layout Footings	0	EA	1,200.00	0
	2	Survey and Layout Column Lines	0	EA	1,200.00	0
	3	Survey and Layout Curbs		EA		0
	4	Survey and Layout Utilities		EA		0
	5	Survey and Layout Retaining Wall		EA		0
	6	Survey and Layout Drainage		EA		0
	6	Survey measurement and fabrictation	3	EA	1,200.00	3,600
	8	Final Survey for New Building	0	EA	1,200.00	0
	8	SWPPP Permit Cost	1	EA	15,000.00	15,000
					SUBTOTAL	**$18,600**
00112		**TESTING & INSPECTIONS**				
	1	Allowance For Testing Laboratory For Concrete	1	LS	0.00	0
	2	Allowance For Testing Laboratory For Subsurface Soils	1	LS	0.00	0
	3	Allowance For Testing Laboratory For Compaction		LS	0.00	0
	4	Allowance For Testing Laboratory For Welding	1	LS	0.00	0
	5	Allowance For Testing Laboratory Mechanical Systems	1	LS	0.00	0
	6	Allowance For Testing Laboratory Fire and Sprinkler Systems		LS		0
					SUBTOTAL	**$0**
00114		**SPECIAL EQUIPMENT / RENTAL**				
	1	Barge Mobilization (35' X 175')	1	EA	22,000.00	22,000
	1	Standby time for barge	0	MO	32,000.00	0
	2	Excavator	1	EA	1,500.00	1,500
	3	Manlift	1	EA	1,200.00	1,200
	4	Air Compressor	1	EA	500.00	500
	6	Compactor	1	EA	1,000.00	1,000
	7	Misc Tools	1	LS	1,000.00	1,000
	8	Standby time for Misc Equipment	1	LS	0.00	0
	9	Crane Rental for 7 Months (2 Cranes LINK-BELT HSP 8060) Rental rate is $136.20/Hour. Assuming crane usage is average 4 Hours/Day	7	MO	45,762.00	320,334
	10	Barge Rental Rate	7	MO	41,236.00	288,652
	11	Tug Boat	7	MO	16,457.00	115,199
	12	Manlift	7	MO	10,249.45	71,746
						0
						0
						0
						0
						0
					SUBTOTAL	**$823,131**

TOTAL GENERAL CONDITIONS ============>>>>> $1,770,246

[redacted]

INCLUDED?		SCOPE ITEM	NOTES
YES	NO		
*		Supervision	
*		Complete Set of Plans	
*		All Addenda/ Revisions/ A/E Clarifications	
*		Insurance	Standard Requirements
	*	Bond	Not Included, see detail estimate
*		Lower - Tier Subcontractors	
*		Shop Drawings / Submittals / Samples / Mockups	Subcontractor produced, GC checked
*		Schedule of Values	
*		Required Payment Forms (AIA,Etc.)	
	*	Union Labor	
	*	Certified Payrolls	Not Included, prepared by GC home office
	*	MBE/WBE/Apprenticeship Program	
*		Hazcom Plan/OSHA Requirements	
*		Schedule Requirements	
*		Phasing Requirements	
*		Liquidated Damages (______________/Day)	
*		Backcharge Rate(____500______/ Day)	Not included
*		Permits and Sign Offs	Allowance
*		Licenses	GC should have and maintain
*		Summer Start up	Assume job starts in summer
*		Winter start up	
	*	Winter protection	
	*	Overtime Included for Shut Downs / Tie Ins	
	*	Overtime Included for Contract work	Reasonable schedule - Not required
*		Clean Up for Own Trade to Container	
*		Layout	
*		Survey	Final survey for the building department
*		As- Built Drawings / O & M Manuals	
*		Warranties and Guarantees	
*		Testing and Inspection	
*		Jobsite Trailer	Deduct if office can be set up inside building
*		Jobsite Telephone	
*		Sales Tax	Included in each subcontract
*		Protection	
*		Temporary Heat	
	*	Unloading Materials & Equipment FBO	
	*	Permits for Temporary Heat	
	*	Attic Stock	

Appendix C

Moment-Interaction Diagrams

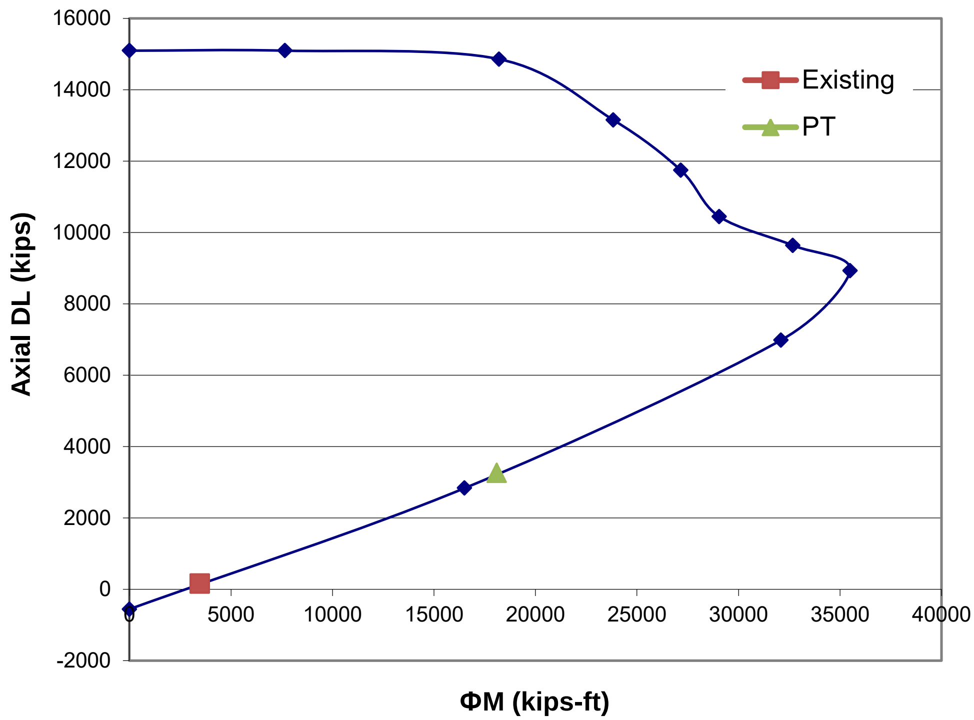


Figure C-1 P-M Curve for Section 15 (EL 488)

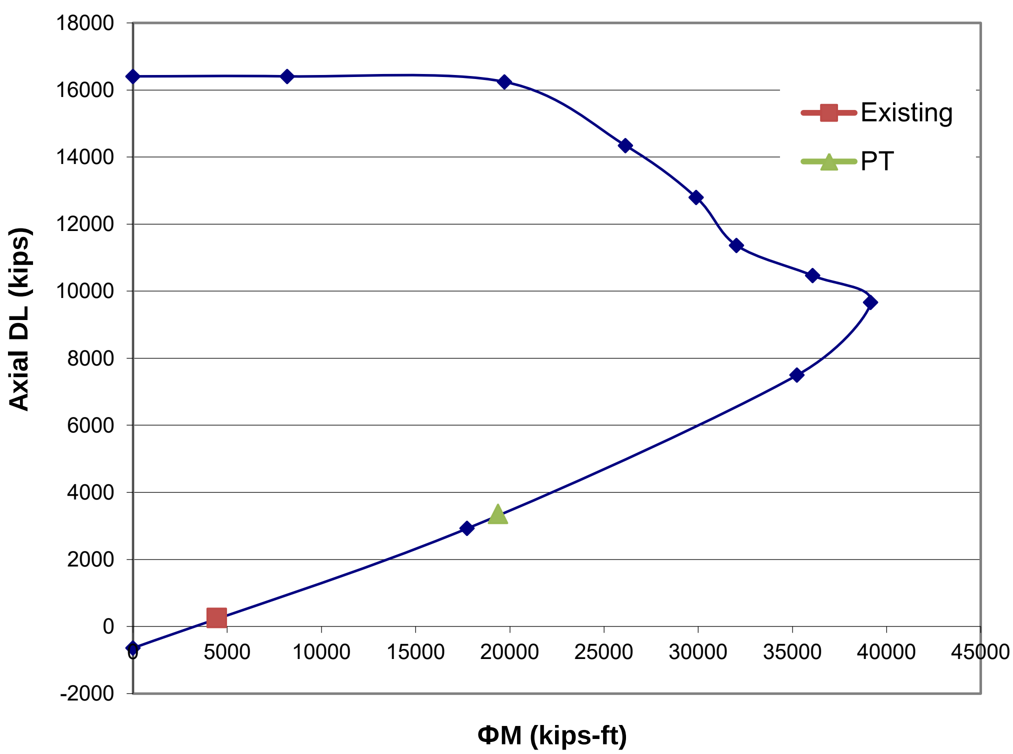


Figure C-2 P-M Curve for Section 14 (EL 477)

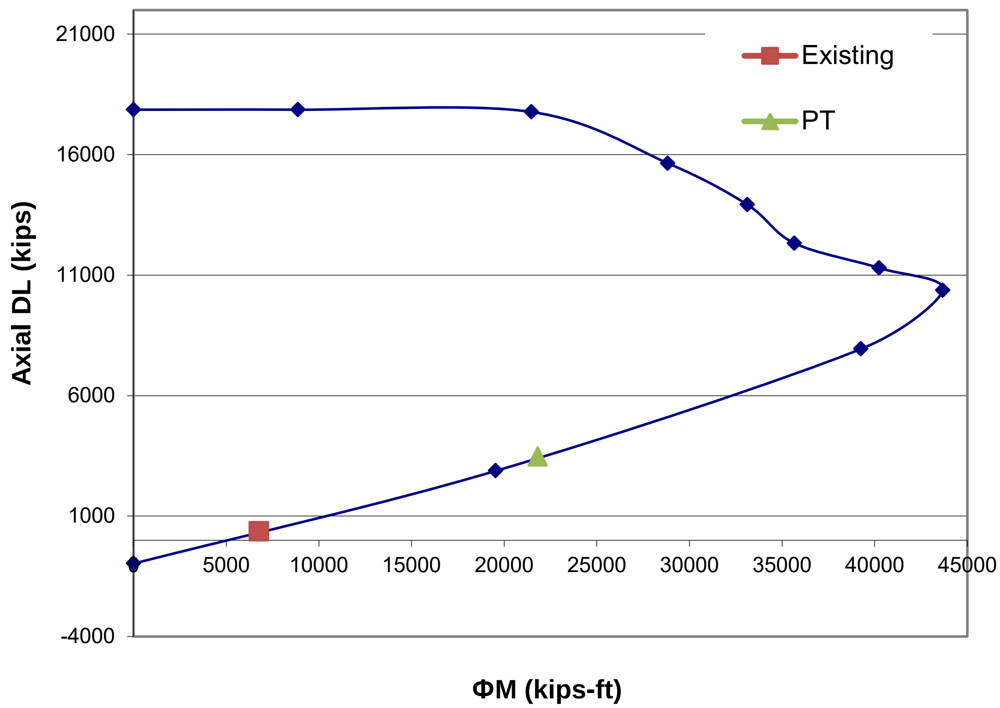


Figure C-3 P-M Curve for Section 13 (EL 465)

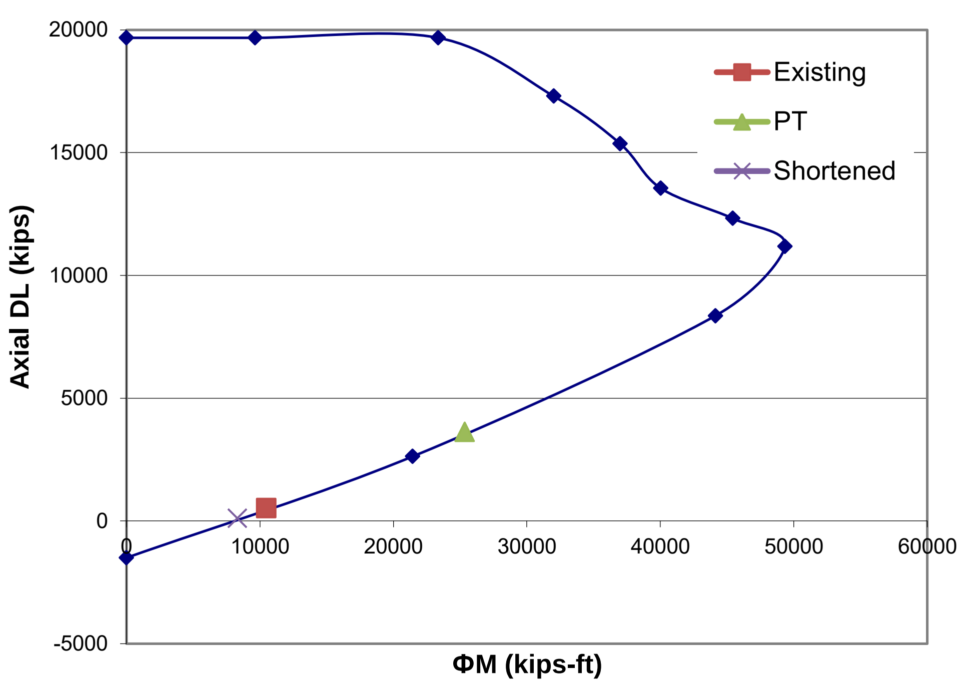


Figure C-4 P-M Curve for Section 12 (EL 450)

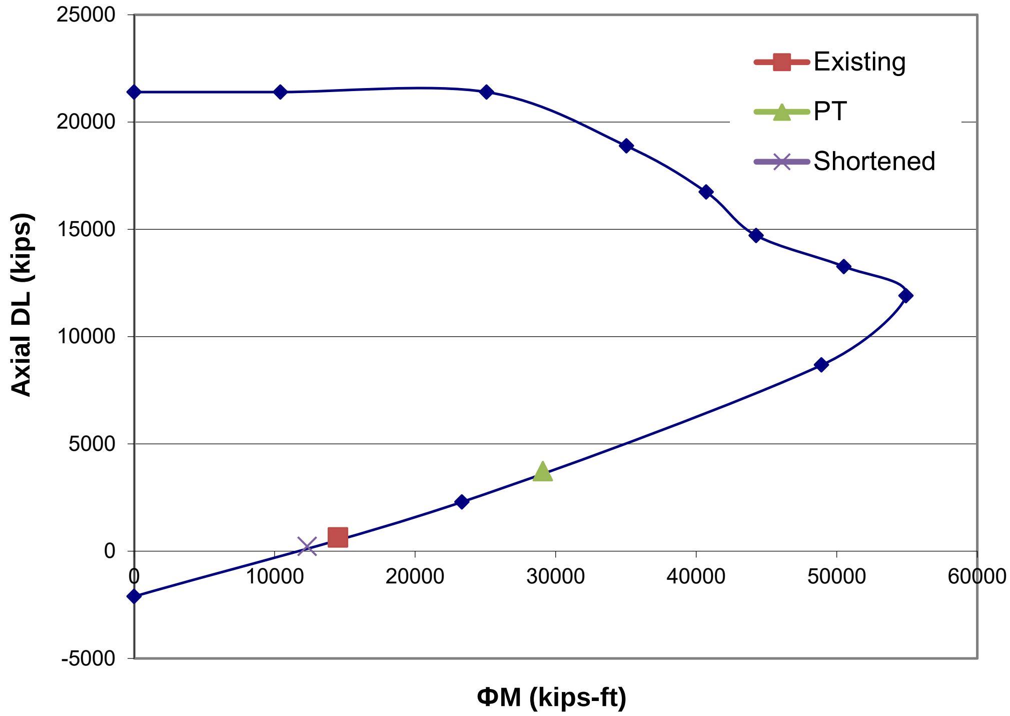


Figure C-5 P-M Curve for Section 11 (EL 441)

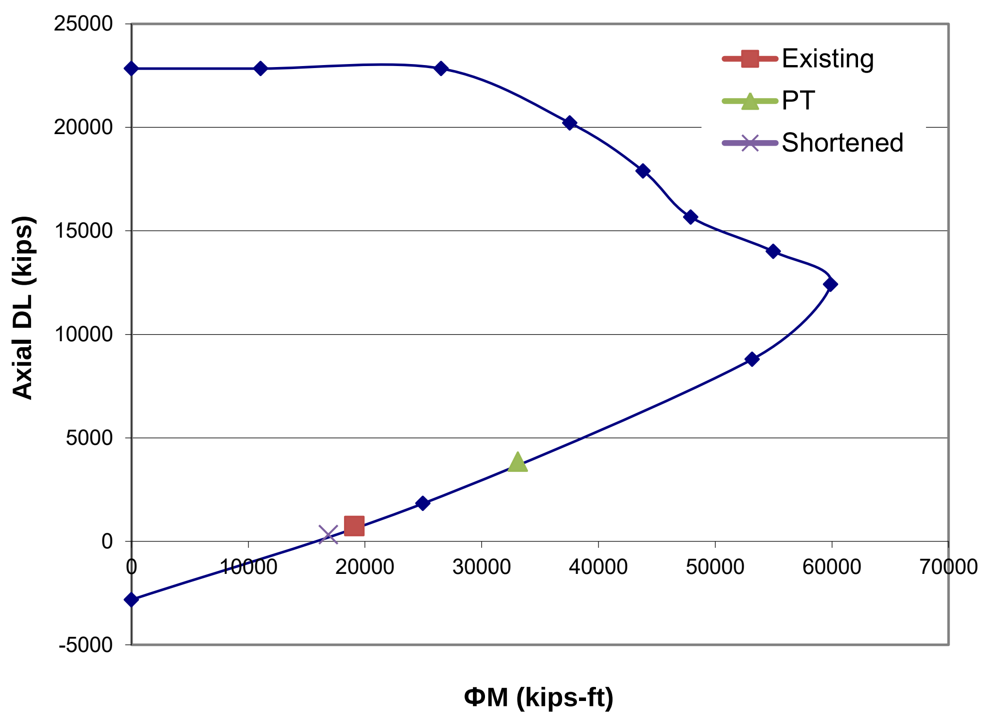


Figure C-6 P-M Curve for Section 10 (EL 432)

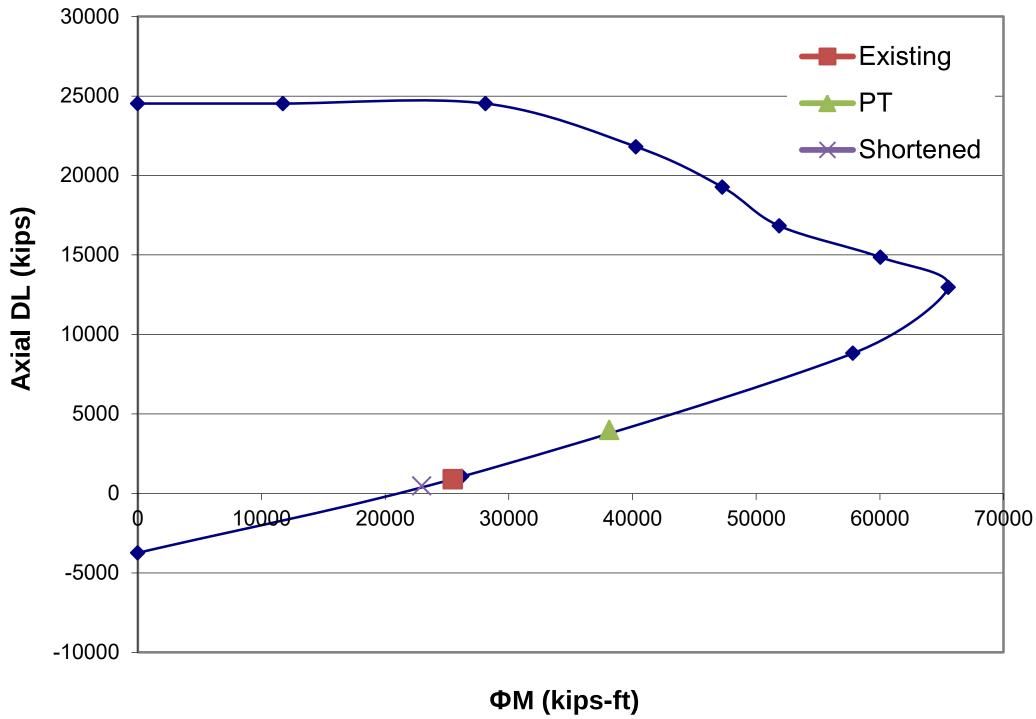


Figure C-7 P-M Curve for Section 9 (EL 421)

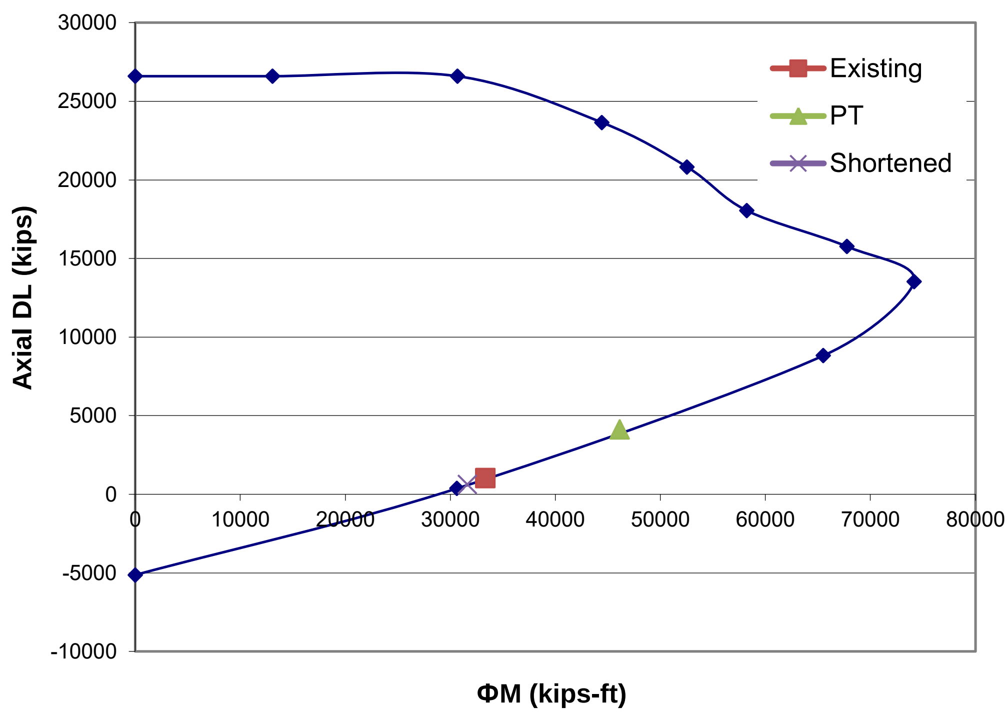


Figure C-8 P-M Curve for Section 8 (EL 410)

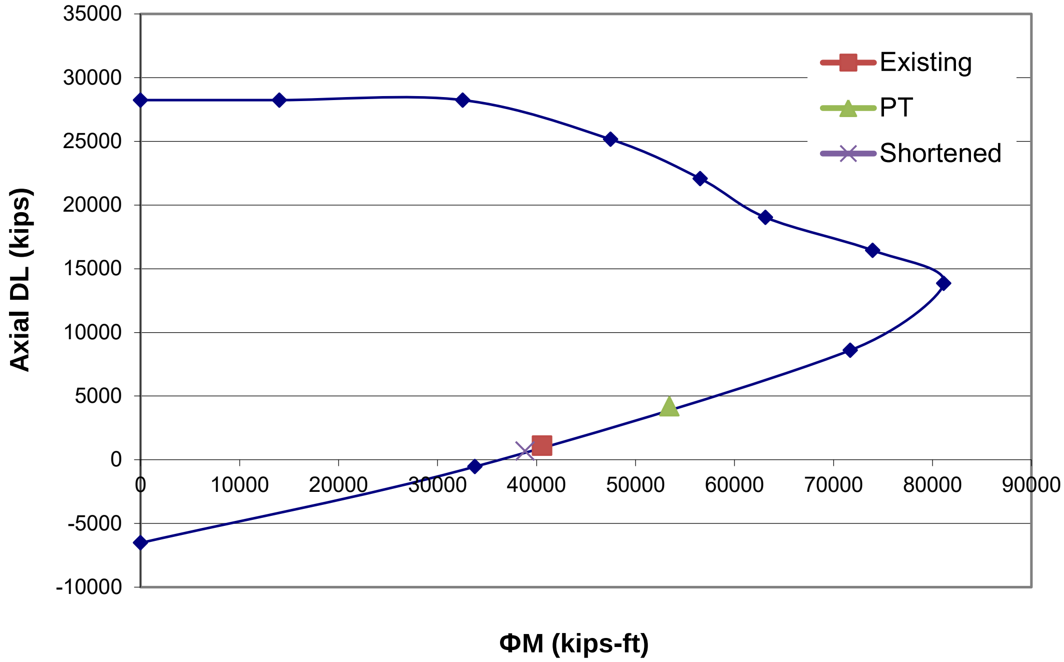


Figure C-9 P-M Curve for Section 7 (EL 406)

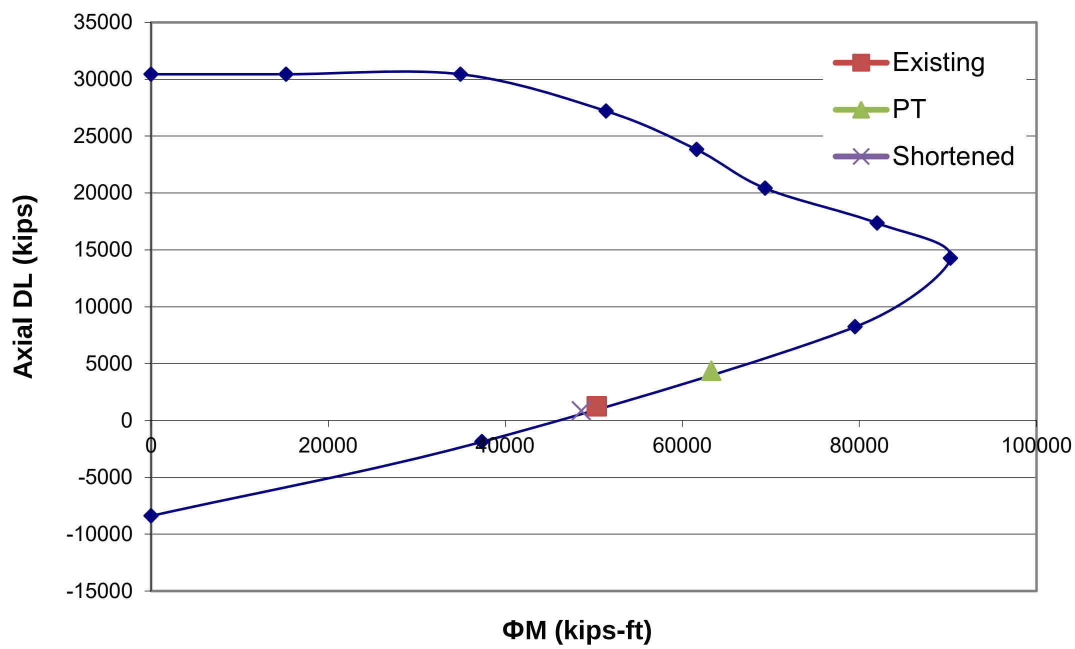


Figure C-10 P-M Curve for Section 6 (EL 397)

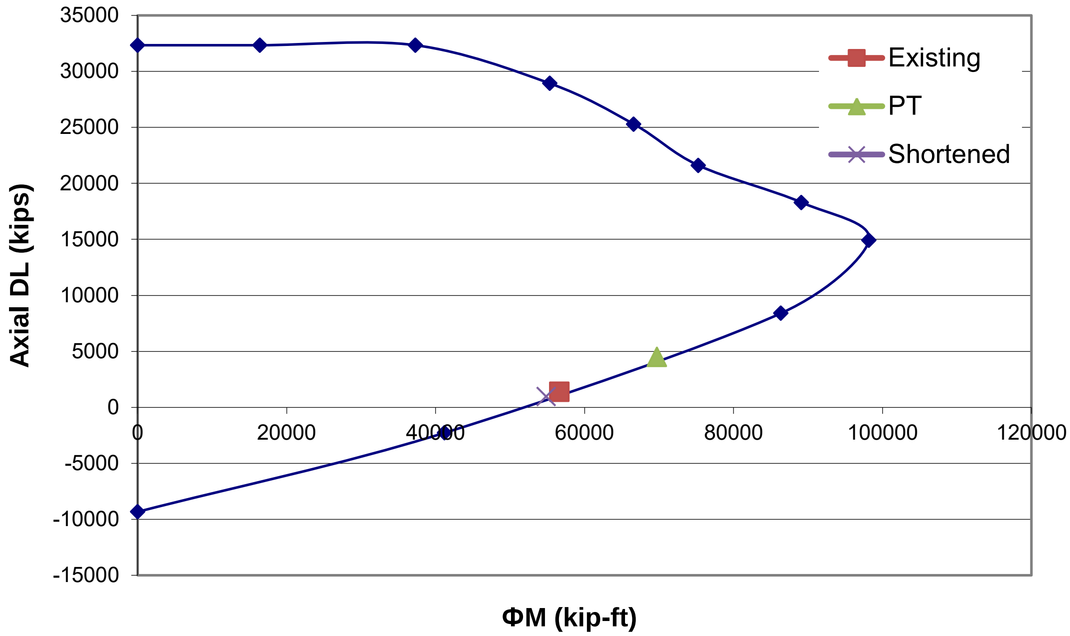


Figure C-11 P-M Curve for Section 5 (EL 388)

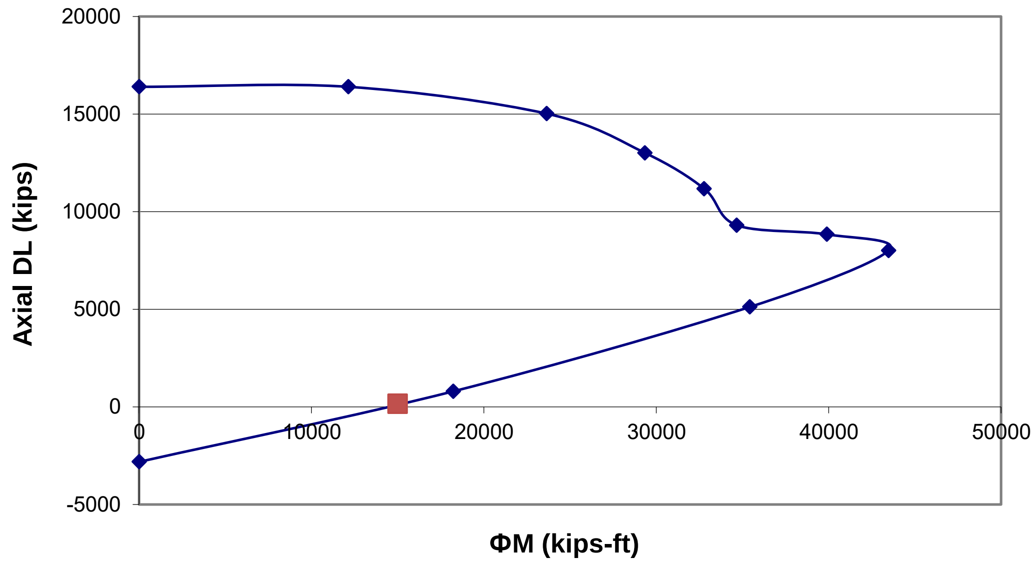


Figure C-12 P-M Curve for Section 15 (EL 488) with FRP

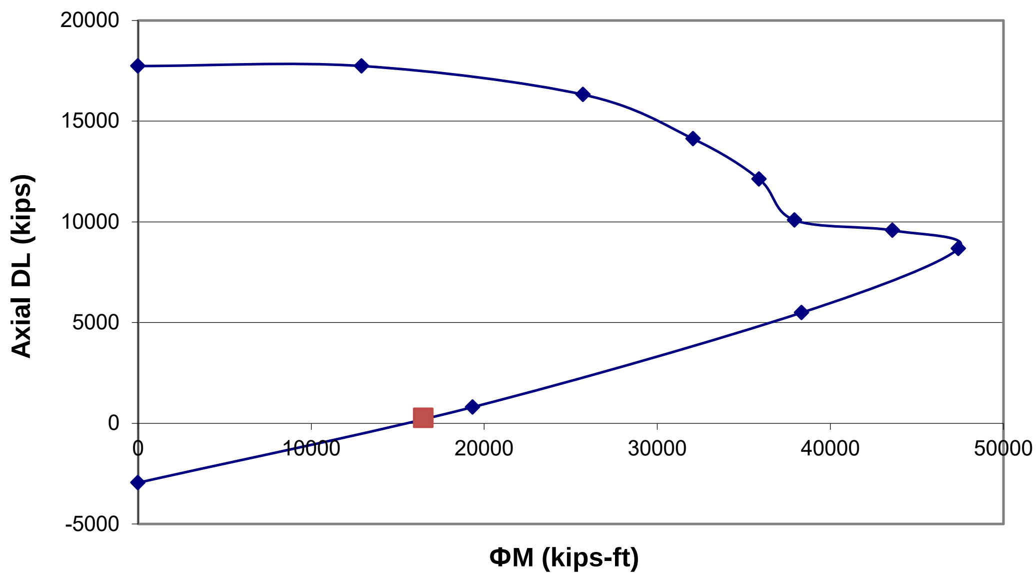


Figure C-13 P-M Curve for Section 14 (EL 477) with FRP

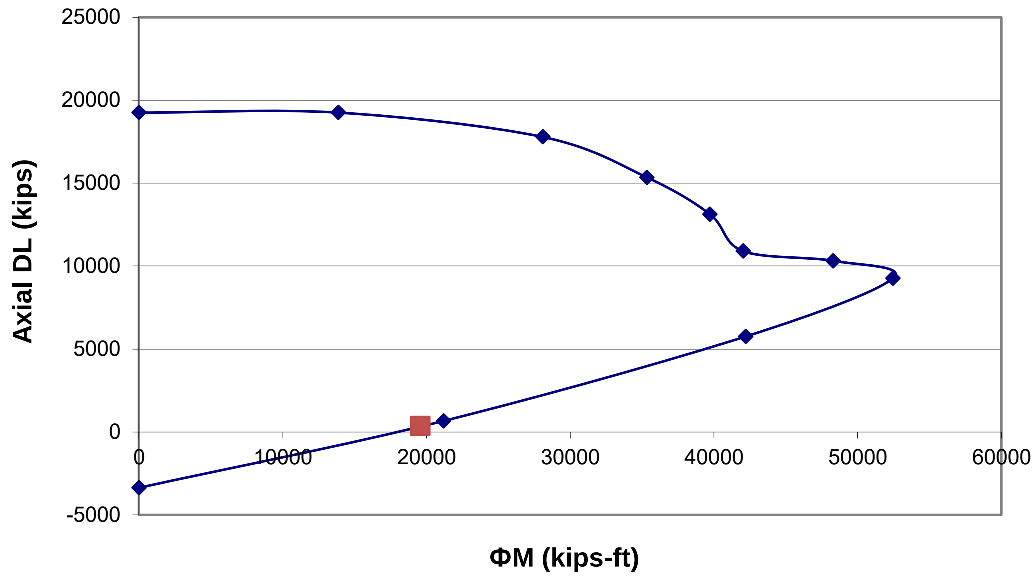


Figure C-14 P-M Curve for Section 13 (EL 465) with FRP

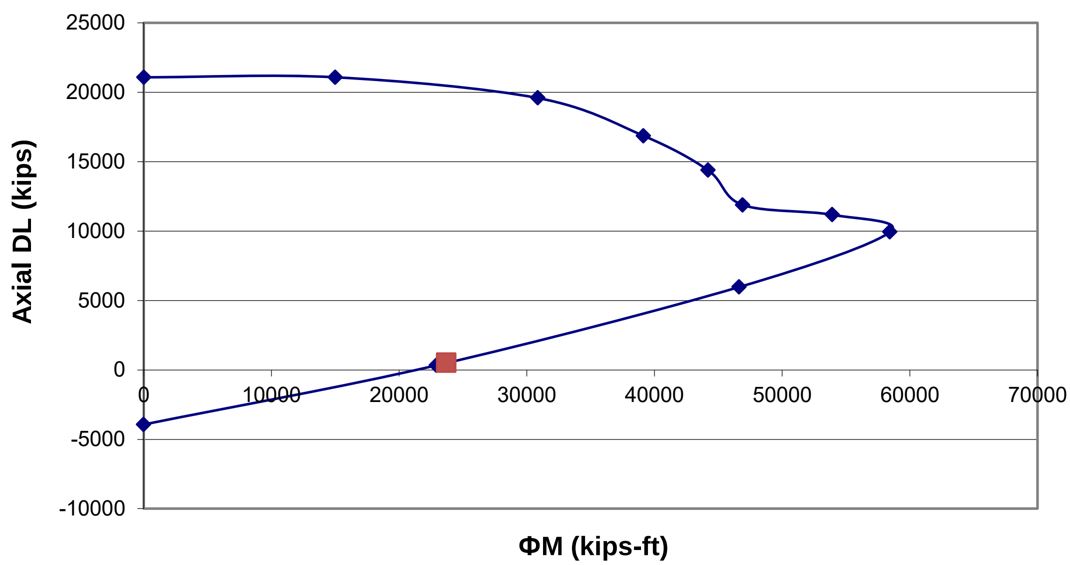


Figure C-15 P-M Curve for Section 12 (EL 450) with FRP

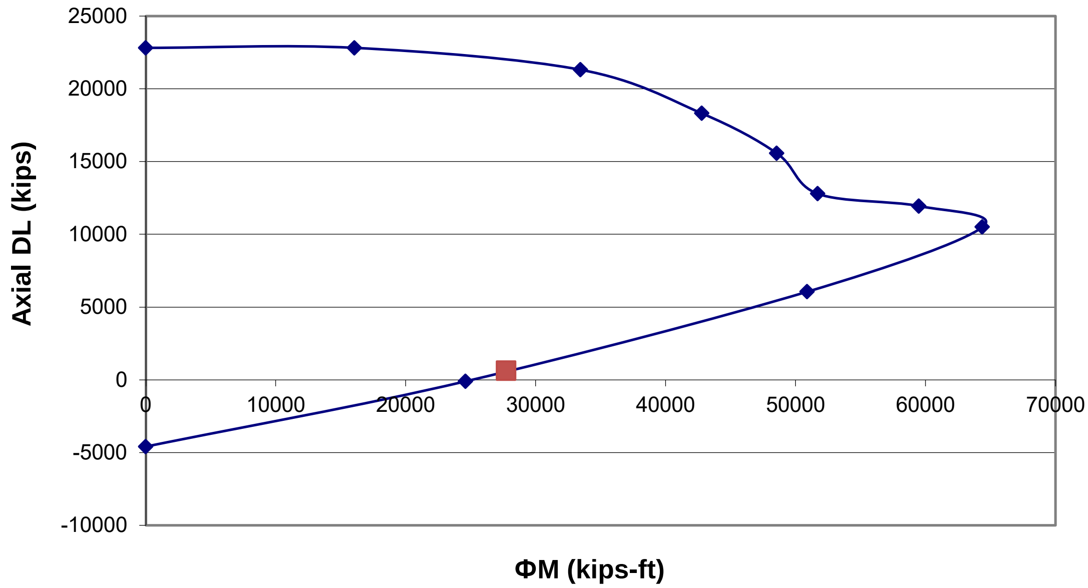


Figure C-16 P-M Curve for Section 11 (EL 441) with FRP

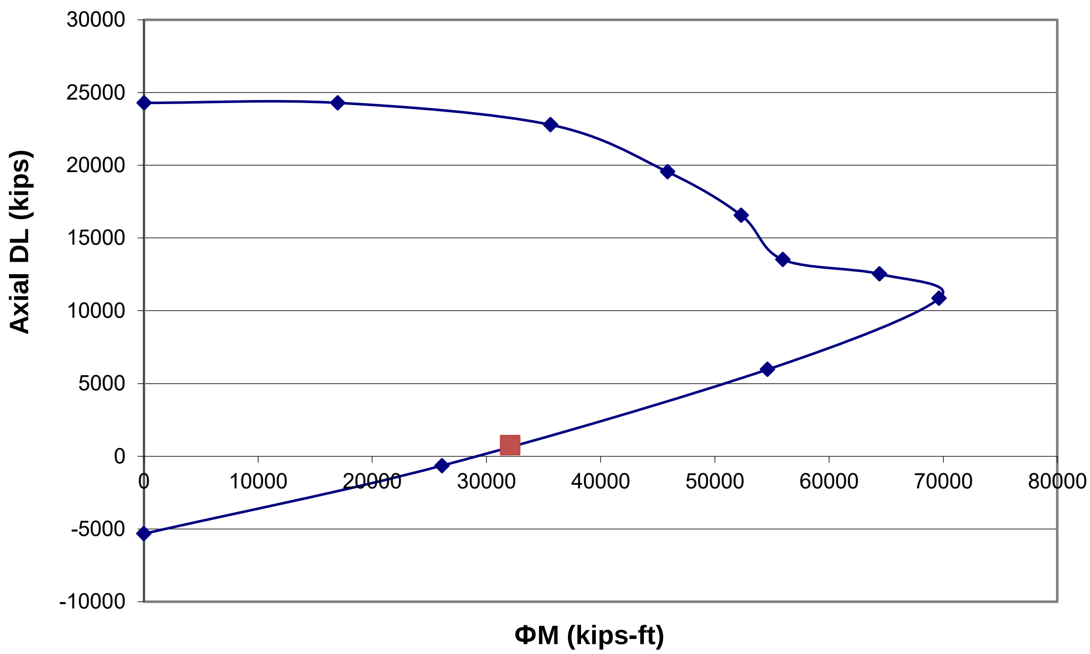
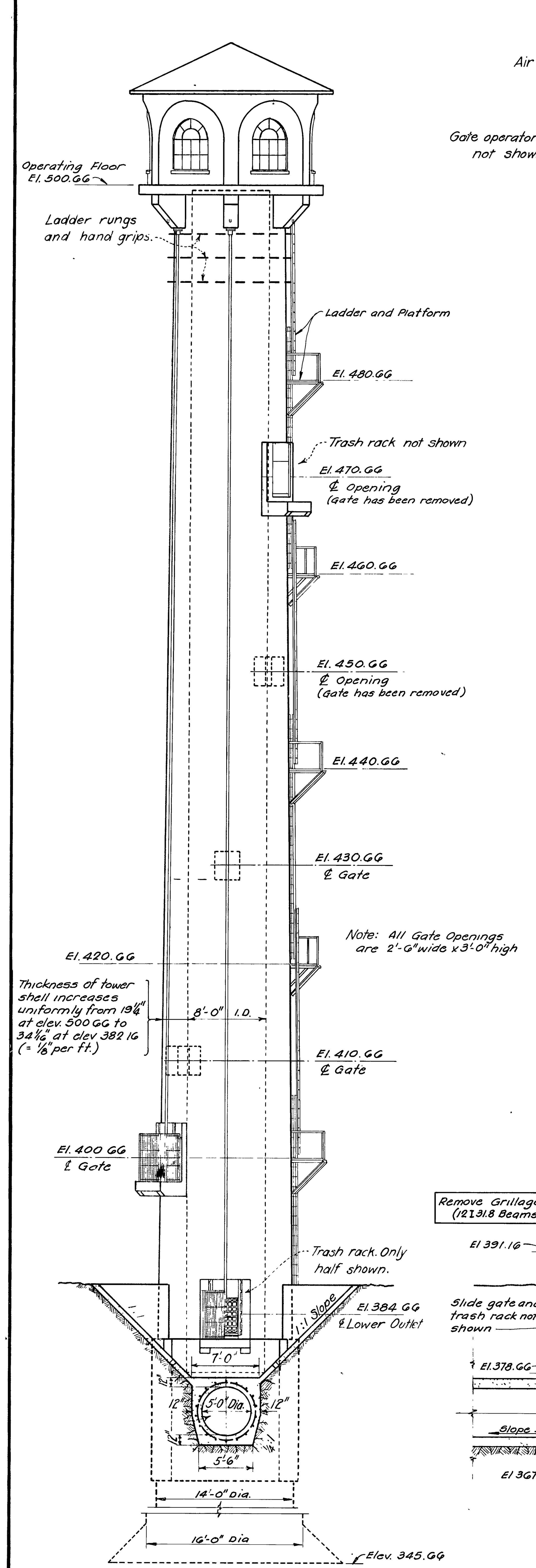


Figure C-17 P-M Curve for Section 10 (EL 432) with FRP

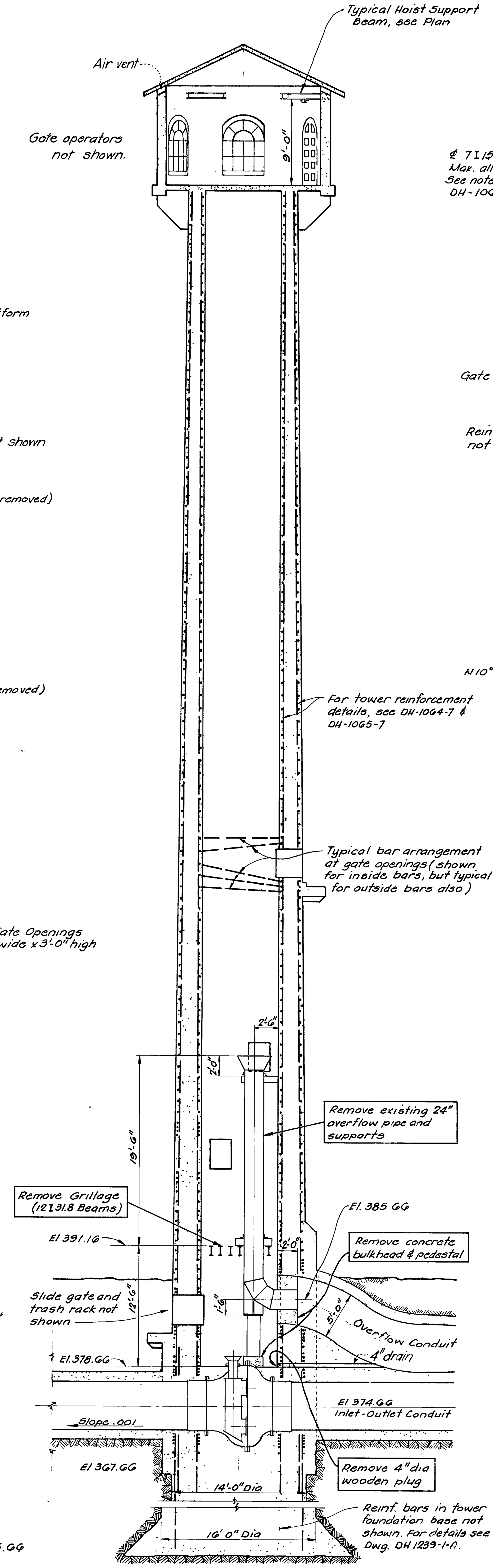
Appendix D

As-Built Drawings

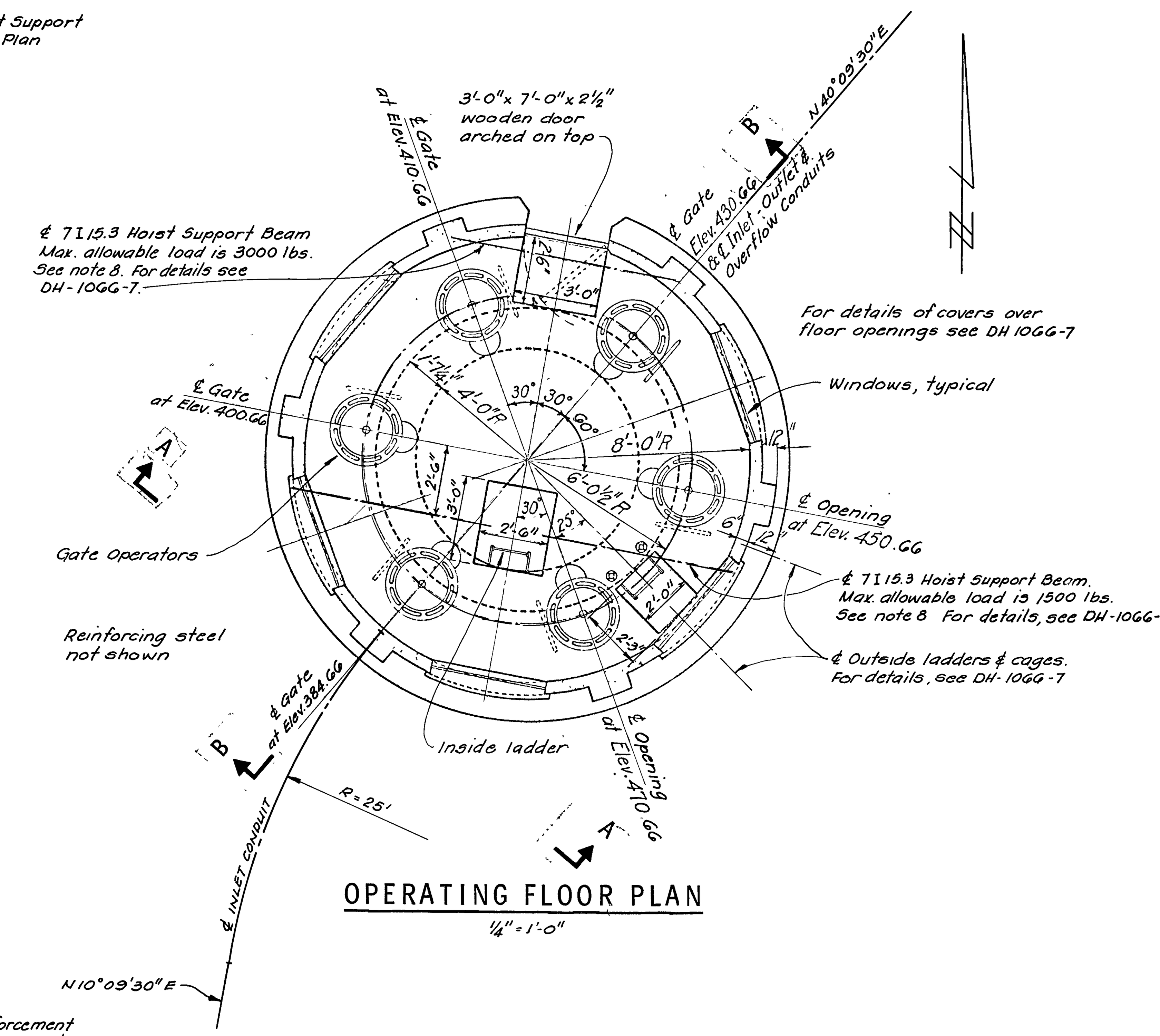


ELEVATION A-A

Scale: ⅛" = 1'-0"

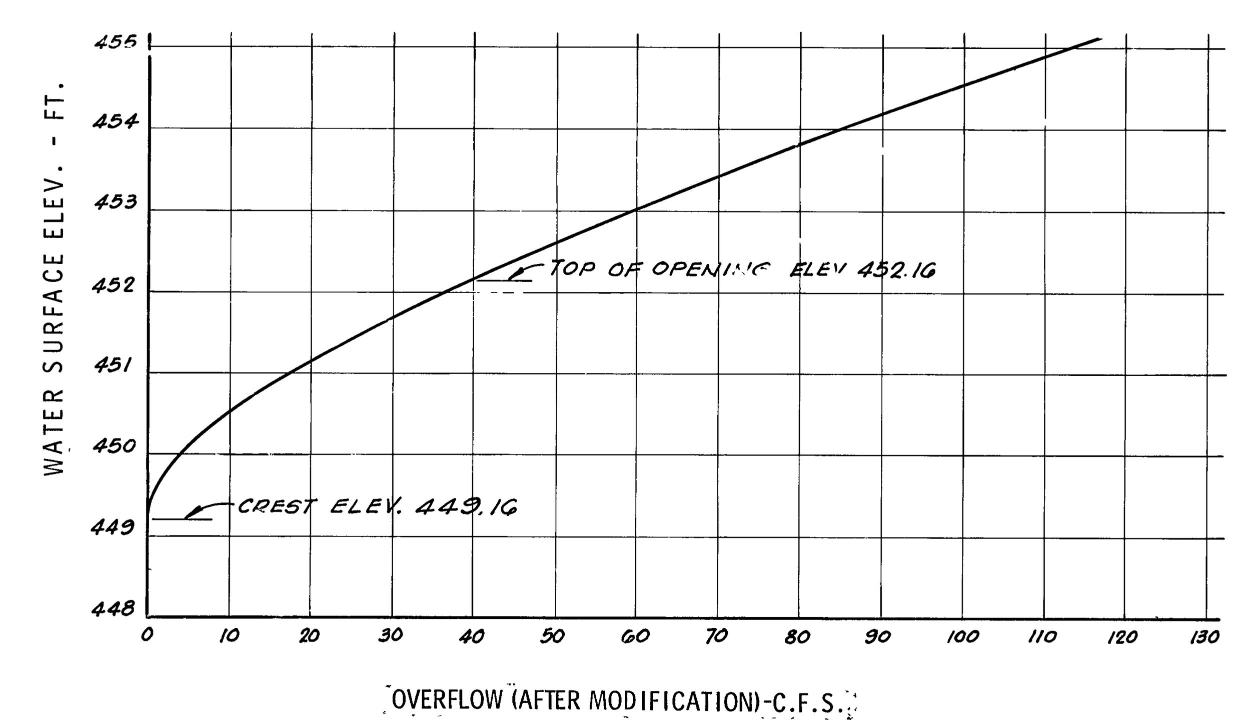


SECTION B-B



OPERATING FLOOR PLAN

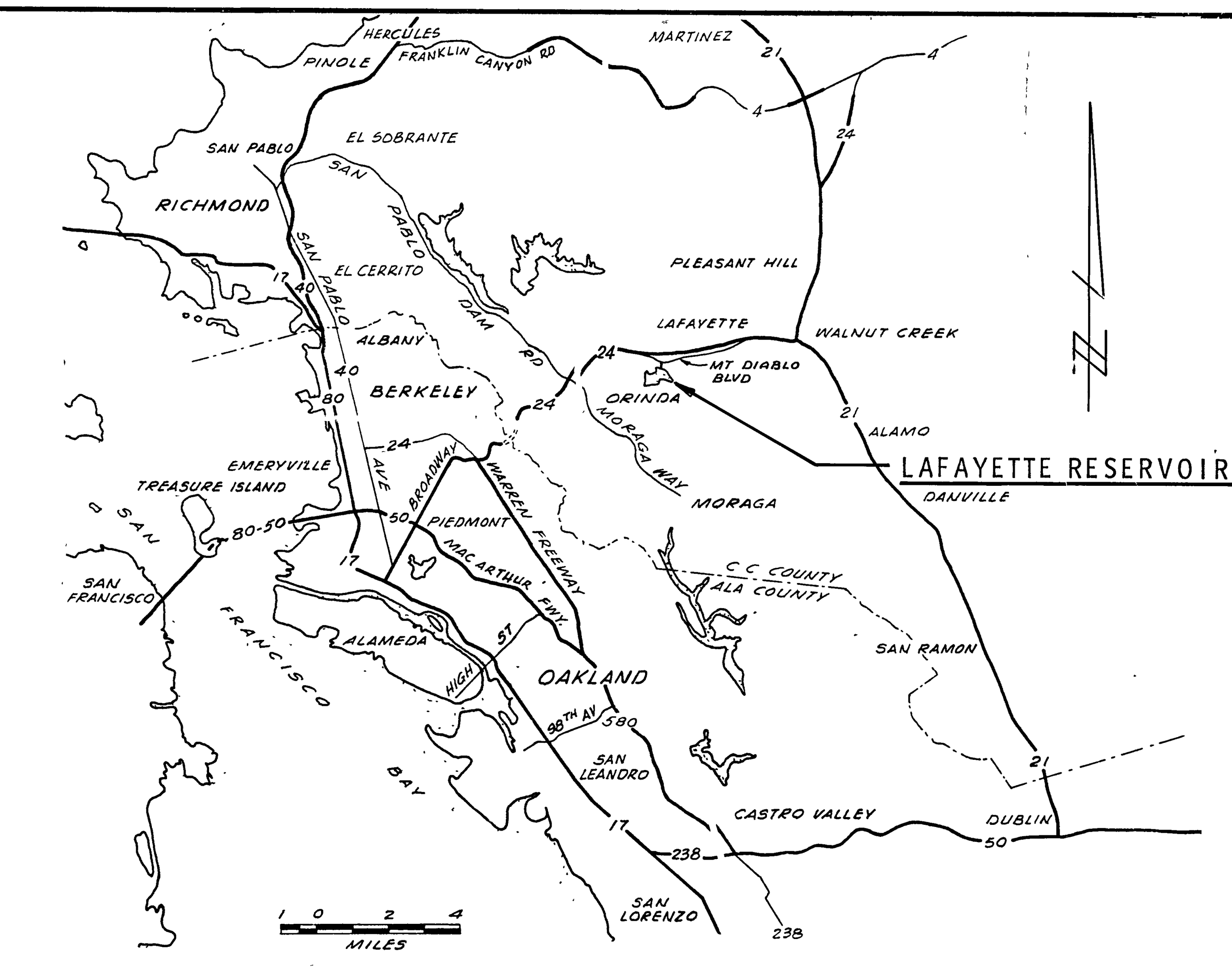
¼" = 1'-0"



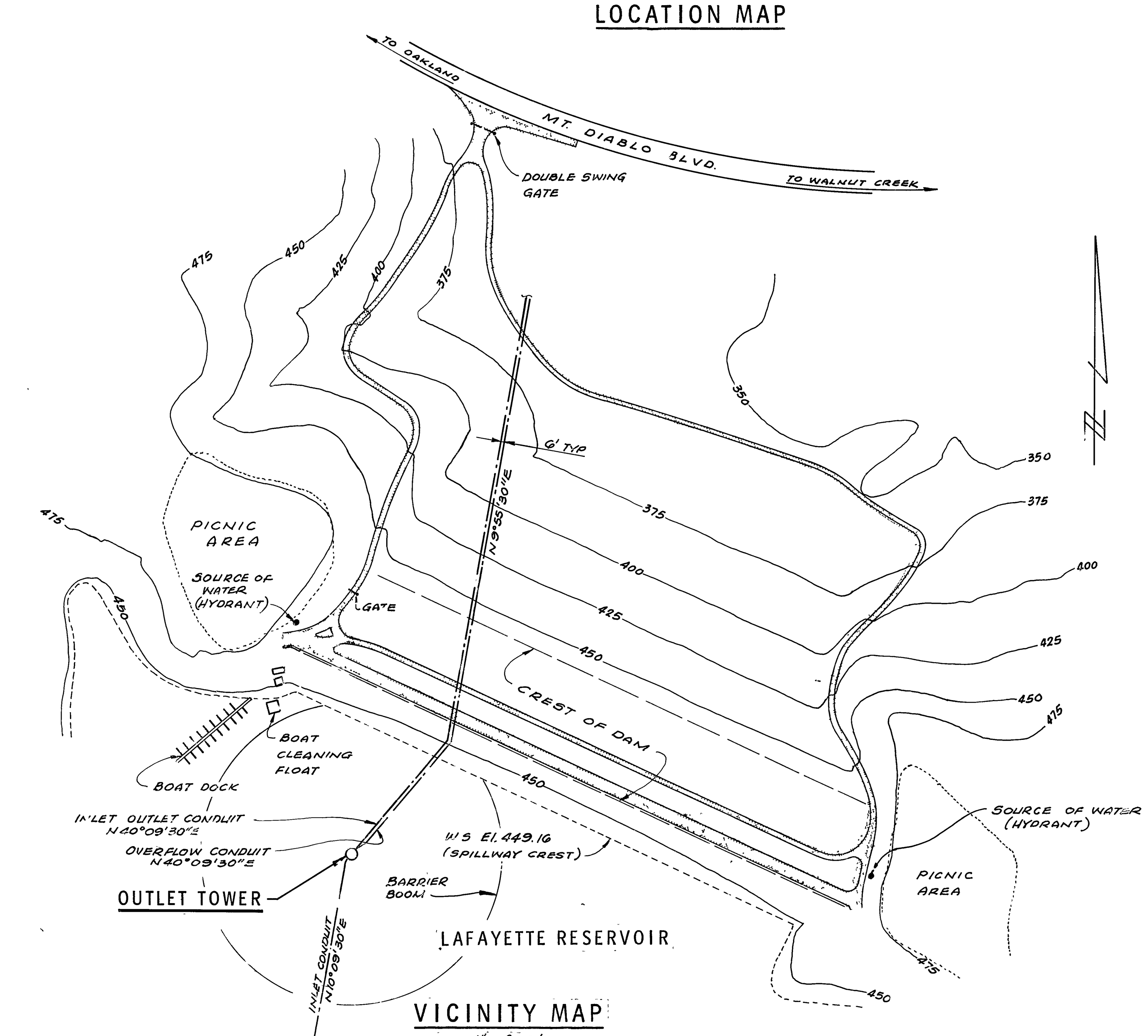
SPILLWAY OPENING AT ELEV. 450

NOTES

1. DETAILS OF EXISTING ELEMENTS SHOWN ON THIS SHEET AND ON G-2 ARE TAKEN FROM DESIGN DRAWINGS AND ACTUAL CONSTRUCTION MAY OR MAY NOT BE EXACTLY AS SHOWN. WHERE NECESSARY, THE CONTRACTOR SHALL VERIFY DIMENSIONS AND OTHER DETAILS IN THE FIELD PRIOR TO CONSTRUCTION. SEE LIST OF REFERENCE DRAWINGS ON THIS SHEET.
2. ELEVATIONS ARE BASED ON USC&GS MEAN SEA LEVEL DATUM OF 1929.
3. EXISTING ELEMENTS TO BE REMOVED ARE SHOWN ON THIS SHEET. NEW CONCRETE DIVIDING WALL TO BE CONSTRUCTED IS SHOWN ON G-2.
4. IN CHIPPING OUT EXISTING CONCRETE FOR INSTALLATION OF NEW DIVIDING WALL, DO NOT CUT EXISTING TOWER REINFORCEMENT. CHIP AROUND REINFORCEMENT, WHERE NECESSARY, TO OBTAIN MINIMUM KEY DEPTH. FOR DETAILS OF NEW CONCRETE DIVIDING WALL TO BE INSTALLED IN THE TOWER, SEE G-2.
5. SEE SPECIFICATIONS FOR TABLE SHOWING ANTICIPATED WATER SURFACE ELEVATION DURING THE PERIOD OF CONSTRUCTION ON THIS PROJECT.
6. RAILING, HAND HOLD, AND HANDRAIL JOINTS SHALL BE WELDED AND ALL WELDS SHALL BE GROUND FLUSH AND SMOOTH. EACH UNIT SHALL BE HOT-DIP GALVANIZED AFTER FABRICATION. EXPANSION ANCHOR BOLTS (½" DIA.) SHALL BE STAINLESS STEEL AND THE ANCHOR INSTALLATION SHALL HAVE A MINIMUM RATED "SAFE LOAD" OF 845 LBS. IN TENSION. BOLTS EMBEDDED IN CONCRETE SHALL BE STAINLESS STEEL.
7. STEP RUNGS, REQUIRED AS SHOWN ON G-2, ARE DETAILED ON DRAWING 1379-A.
8. THE MAXIMUM ALLOWABLE LOAD SHOWN FOR THE EXISTING HOIST BEAMS AT THE TOP OF THE TOWER ARE BASED ON THE EXISTING CONSTRUCTION. IF THE CONTRACTOR WISHES TO MODIFY THE BEAMS TO INCREASE THEIR LOAD CAPACITY, DETAILS OF SUCH MODIFICATIONS SHALL BE SUBMITTED TO THE ENGINEER FOR APPROVAL.



LOCATION MAP



VICINITY MAP

1" = 200'

REFERENCE DRAWINGS:

DH-1062-7 DH-1066-7 DH-1239-1-A S-937
DH-1064-7 DH-1067-7 DH-2240-7
DH-1065-7 DH-1068-7 DH-2874-41-A

FIELD BOOK REFERENCE:

F.B. 3402

SPECIFICATION 1105

NO.	DATE	REVISION	BY	APP.
1	17JUN69	IN SERVICE 31 MAY 68 AS BUILT INSP		

APPROVED [signature] CHIEF ENGINEER R C E NO 14167

DESIGNED BY	J. Tom
DRAWN BY	C. YAMASAKI
CHECKED BY	C. F. Yerby
CORROSION CHECK BY	AE Westerback
PROJECT ENGR.	C. F. Yerby
SUPERVISOR MECH & ELEC DESIGN	LBH
SUPERVISOR STRUCTURES DESIGN	[signature]
MANAGER DESIGN ENGRG.	[signature]
MANAGER WATER PROD & DIST.	[signature]

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA

LAFAYETTE RESERVOIR
OUTLET TOWER MODIFICATIONS

EXISTING ELEMENTS & GENERAL INFORMATION

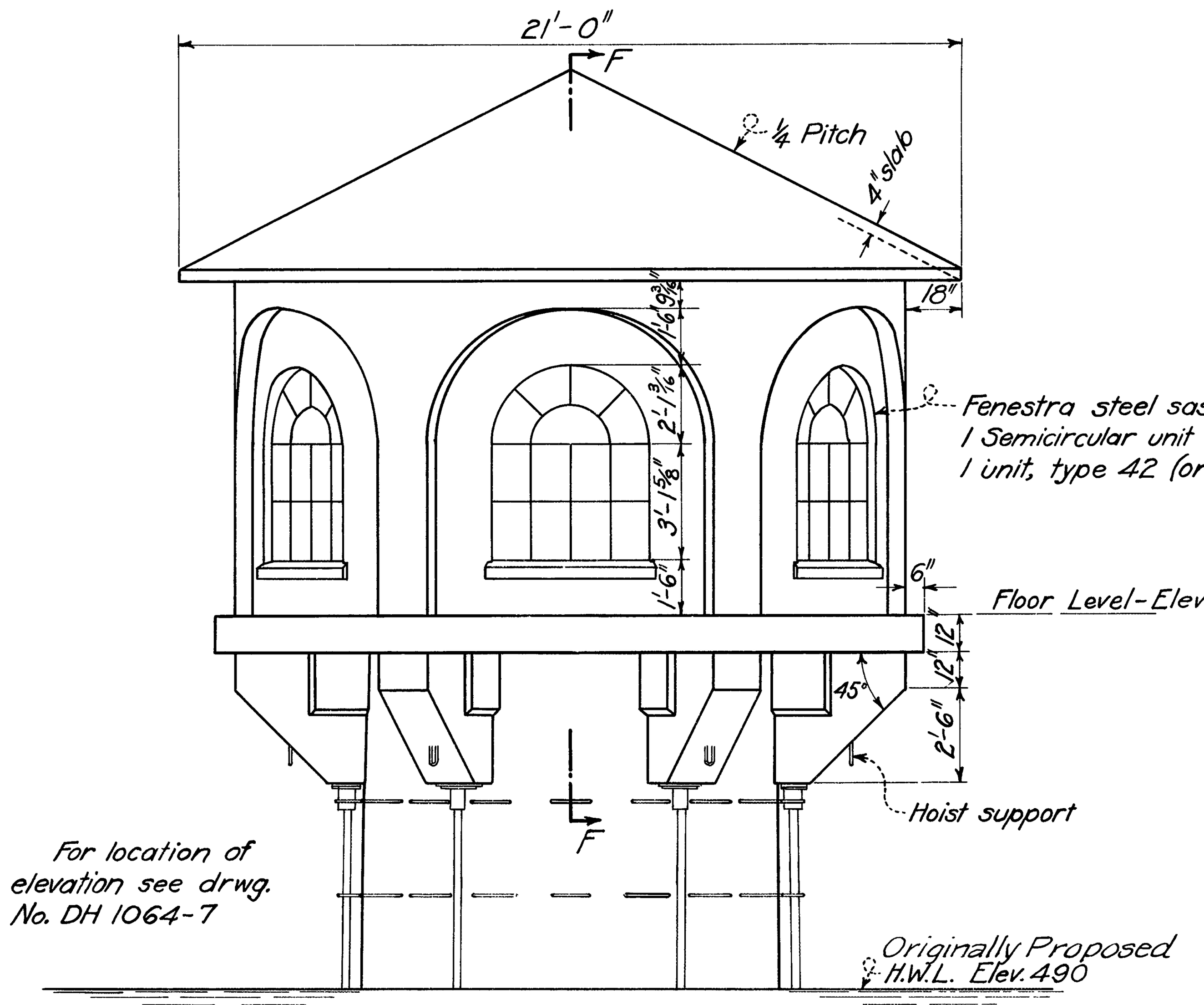
STRUCTURE OR ZONE DESIGNATION 527
SCALE AS NOTED
DATE 22 AUG 67

5450-G-1

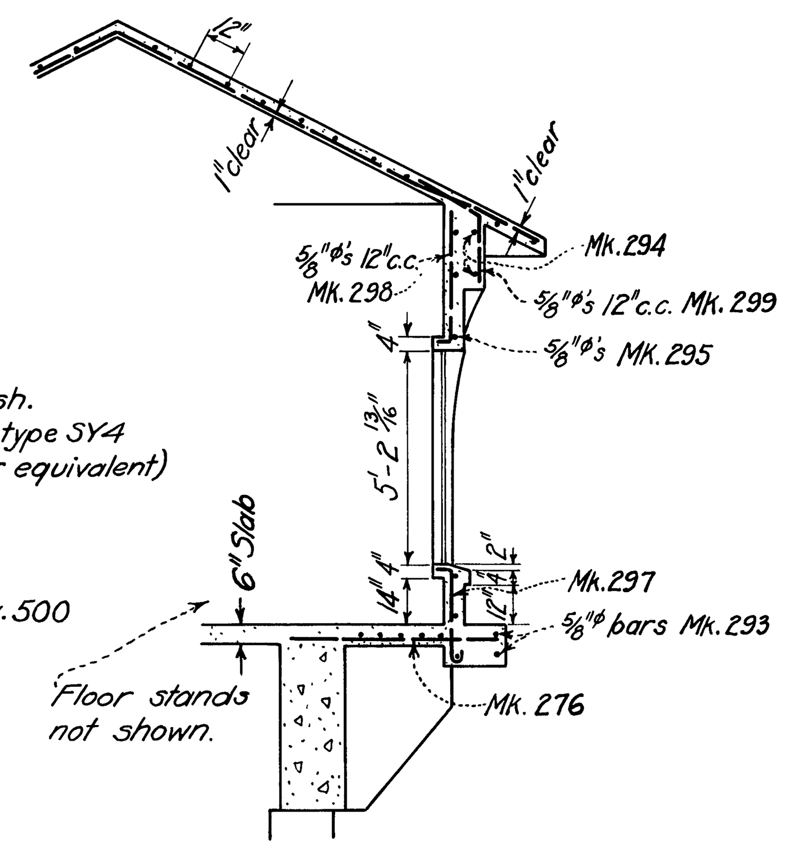
2


SECTION B-B
1/8" = 1'-0"
DETAIL 1
LADDER DETAIL
HANDRAIL DETAIL
1/2" = 1'-0"
SECTION E-E
AT ELEV. 400
SECTION H-H
AT ELEV. 450
SECTION M-M
1/2" = 1'-0"
SECTION L-L
1/2" = 1'-0"
HAND HOLD DETAIL
1/2" = 1'-0"
SECTION D-D
AT ELEV. 385
SECTION G-G
AT ELEV. 432
SECTION K-K
1/2" = 1'-0"
DETAIL 2
NOT TO SCALE
SECTION J-J
AT ELEV. 470
1/2" = 1'-0"
SECTION Q-Q
SECTION C-C
AT ELEV. 380
SECTION F-F
AT ELEV. 410
SECTION P-P
1/2" = 1'-0"
SECTION N-N
1/2" = 1'-0"
SECTION R-R
NOT TO SCALE
NOTE: EXCEPT FOR LANDING SLAB AT EL. 431.16 ARRANGEMENT OF REINF. IS TYPICAL FOR ALL SLABS BETWEEN EL. 398.16 AND 441.16 INCLUSIVE. STEP RUNGS AND HAND HOLDS NOT SHOWN
NOTES
1. FOR NOTES, SEE G-1
17 JUN 69 IN SERVICE 31 MAY 68 AS BUILT
13 NOV 67 ADDED TEMPORARY OPENING & PLUGGED EXISTING GATE OPENING.
NOV. 1-67 MISC. CORRECTIONS
NO. DATE REVISION BY APP.
DESIGNED BY J. Tom
DRAWN BY D. YAMASAKI
CHECKED BY C F Yerby
CORROSION CHECK BY A Westerback
PROJECT ENGR. C. F. Yerby
APPROVED CHIEF ENGINEER R.C.E. NO. 14167
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA
LAFAYETTE RESERVOIR
OUTLET TOWER MODIFICATIONS
CONCRETE DIVIDING WALL & MISCELLANEOUS DETAILS
STRUCTURE OR ZONE DESIGNATION 527
SCALE 1/4" = 1'-0" & AS NOTED
DATE 22 AUG 67
5450-G-2

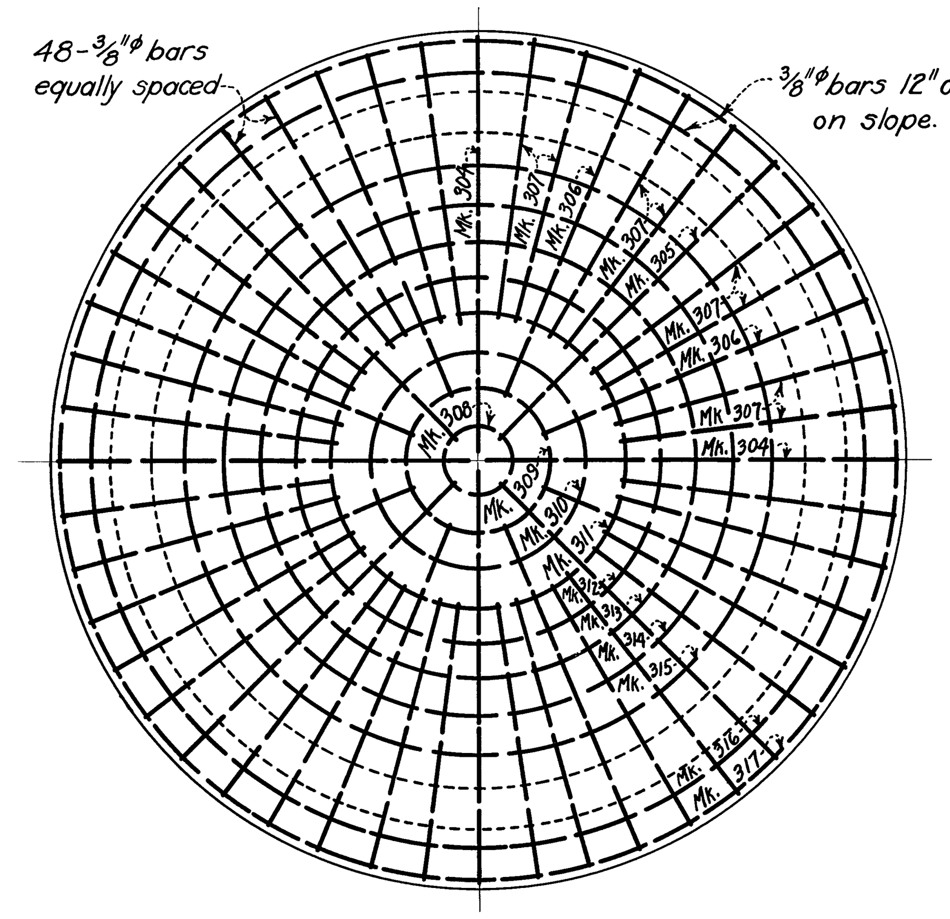

5.
ELEVATION A-A
Scale 1/8"=1'
For details of operating house see part Elev. E-E on drwg. DH 1065-7
Operating floor Elev. 500.
Ladder rungs and hand grips
Originally Proposed Normal F.W.L. Elev. 490
Elev. 480.00
Trash rack not shown
Elev. 470.00
℄ Outlet
Elev. 466
Top of Dam
Elev. 460.00
Elev. 450.00
℄ Outlet
Elev. 440.00
FOR MODIFICATIONS SEE 5573 G-1 & 2 1957
Elev. 428.5
Present H.W.L.
Elev. 430.00
℄ Outlet
Note: All outlets are to be 2'-6" wide x 3'-0" high (rectangular).
Elev. 420.00
8'-0" I.S. Diam.
Thickness of tower shell increases uniformly from 19¼" at elev. 500 to 34 7/16" at elev. 381.5 (=⅛" per ft.)
Elev. 410.00
℄ Outlet
Note: Stem guides and couplings not shown. For details see table on drwg. DH 1065-7
Elev. 400.00
℄ Outlet
Trash rack only half shown.
Elev. 384.00
℄ Lower Outlet
1:1 Slope
6" Slab reinforced with ⅜"ø bars 9" c.c. both ways
6" drain holes 10' c.c.
1"ø bars 6" c.c.
⅝"ø bars - 18 total
5 ft. diameter inlet conduit
5'-0" Dia.
5'-6"
14'-0" diameter
For foundation details see drwg. DH 1239-1-A
Elev. 345
Air vent
6"x16" Air vent covered on outside with copper screen (16 mesh, No 26 wire).
Gate hoists not shown.
℄'s Gate hoists
Operating chamber reinforcement not shown.
℄ 7"x15.3#x9'-0" I-Beam For detail see drwg. DH 1066-7
6-30" Chapman Motor Units Fig. 1302, Cat. 50 or equivalent, and suitable ball bearing floor stands.
Provide for tension of 50,000# in stem. 4-1½"x30" Anchor bolts for each floor stand. See detail on drwg. DH 1065-7
Bars Mk 201
Mk 206
Mk 205
Bars Mk 201
Typical inside diagonal bars
CONSTRUCTION JOINTS are to be located not nearer than 18" to openings into tower.
See Drg. Nº DH 2918-7 for Details of Outlet Gate
Elev. 392
See table on drwg. DH 1065-7 for size and spacing of tower reinforcement. Where horizontal and vertical reinforcement as shown in table would intersect openings in wall, bend all such steel around openings.
Slide gate and trash rack not shown.
Elev. 390
Elev. 385
Elev. 381.5
Elev. 377.5
Elev. 378.5
Elev. 374.00
Elev. 367
12'-6" Rad.
Manhole
4" drain
Ground surface
OUTLET
INLET
Conduit reinforcing not shown
For location of collars see drwg. DH 1062-7
Concrete collars 25 ft. c.c. from tower to d.s. toe of dam
Grade -.001
Grade +.001
19'-0"
50' (Transition section)
14'-0" diameter
Elev. 345
SECTION ON ℄ INLET CONDUIT
Scale 1/8"=1'
3'-0"x7'-0"x2½" wooden door arched on top.
℄ Outlet Elev. 430 & ℄ Inlet conduit
For details of covers over openings see drwg. DH 1066-7
℄ Outlet at Elev. 410
℄ Outlet at Elev. 450
℄ 7"x15.3#x16'-0" I-Beam For detail see drwg. DH 1066-7
Opening B
Opening C
Opening A
⅝"ø's Mk. 296
℄ Outlet at Elev. 400
℄ Outlet at Elev. 470
℄ Outlet at Elev. 384
All reinforcing steel not shown.
FLOOR PLAN OF OPERATING CHAMBER
Scale ¼"=1'
54 Ladder rungs and hand grips (¾"ø bars, galv.) Space as shown
℄ Inlet conduit
Ladder
Reinforcing steel not shown.
SECTIONAL PLAN AT ELEV. 494
Scale ¼"=1'
Stem guides not shown
SECTIONAL PLAN AT ELEV. 470
Scale ¼"=1'
Trash rack and gate not shown
18-1"ø bars in addition to bars shown in table on drwg. DH 1065-7 are to be placed as shown. These bars are to be ten feet long and extend from elev. 445 to elev. 455.
Trash rack and gate not shown.
For details of landings see drwg. DH 1066-7
SECTIONAL PLAN AT ELEV. 450
Scale ¼"=1'
30-1"ø bars in addition to bars shown in table on drwg. DH 1065-7 are to be placed as shown. These bars are to be ten feet long and extend from elev. 425 to elev. 435
℄'s of ladders and landings (which are not shown).
℄ Inside ladder
SECTIONAL PLAN AT ELEV. 430
Scale ¼"=1'
6-1¼"ø bars 12' long in addition to bars shown in table on drwg. DH 1065-7. Other reinforcement not shown.
SECTIONAL PLAN AT ELEV. 410
Scale ¼"=1'
Grillage at elev. 390 for breaking fall of water.
Trash rack and gate not shown.
5-12"x31.8# I-Beams 12" c.c. embedded in concrete
2-12"x31.8# I-Beams 12" c.c. (removable)
SECTIONAL PLAN AT ELEV. 400
Scale ¼"=1'
Note: Transition between section B-B & section C-C is to be made by varying angle between line connecting centers of conduits and horizontal, in proportion to distance from beginning of transition. Conduits in transition section are to be 6'-0" center to center at all points.
SECTION B-B
Reinforce section B-B same as shown in section C-C.
℄ Inlet conduit
Manhole
Trash rack and gate not shown
SECTIONAL PLAN AT ELEV. 385
Scale ¼"=1'
INLET CONDUIT
60" Class C C.I. Nipple
60" Rensselaer Valve Co., List No. 37 check valve, or equivalent (Shown in plan)
Reinforcing steel not shown.
℄ Tower
25'-0" Rad.
Central angle of curve.
SECTIONAL PLAN AT ELEV. 374
Scale ¼"=1'
SECTION D-D
TYPICAL SECTION OF INLET CONDUIT UPSTREAM FROM OUTLET CHANNEL
Scale ¼"=1'
Puddle backfill
1"ø bars 5" c.c.
⅝"ø bars
Lap bars 40 diameters
Slope to suit material
Rock line
Inlet
Outlet
Concrete collars and conduits, below ℄, are to be poured without forms, if material will stand to slopes shown.
SECTION C-C
Scale ¼"=1'
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND - CALIFORNIA
Arthur P. Davis, Chief Engineer and General Manager.
MOKELUMNE RIVER PROJECT
LAFAYETTE RESERVOIR
OPERATING TOWER AND CONDUIT DETAILS
Revised Sept. 29, 1927 Designation marks for reinforcing steel added. By L.D.W.
Revised Apr. 2, 1928 L.D.W.
Drawn by L. D. Wilbur
Traced by L. D. Wilbur
Checked by R. C. Kennedy
Approved by F. W. Hanna
Scale: 1/8" & ¼"=1'
Date: April 27, 1927
Nº DH 1064-7



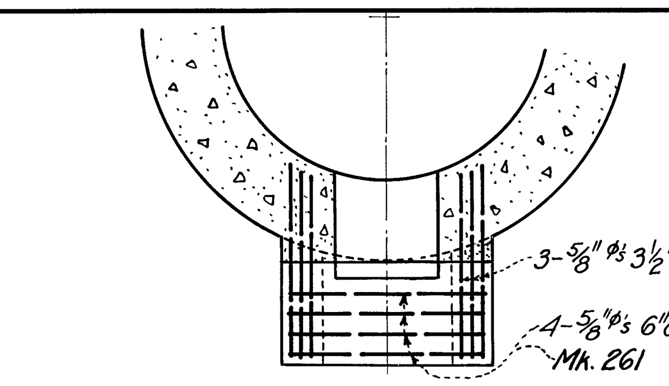
PART ELEVATION E-E
Scale ¼"=1'



SECTION F-F
Scale ¼"=1

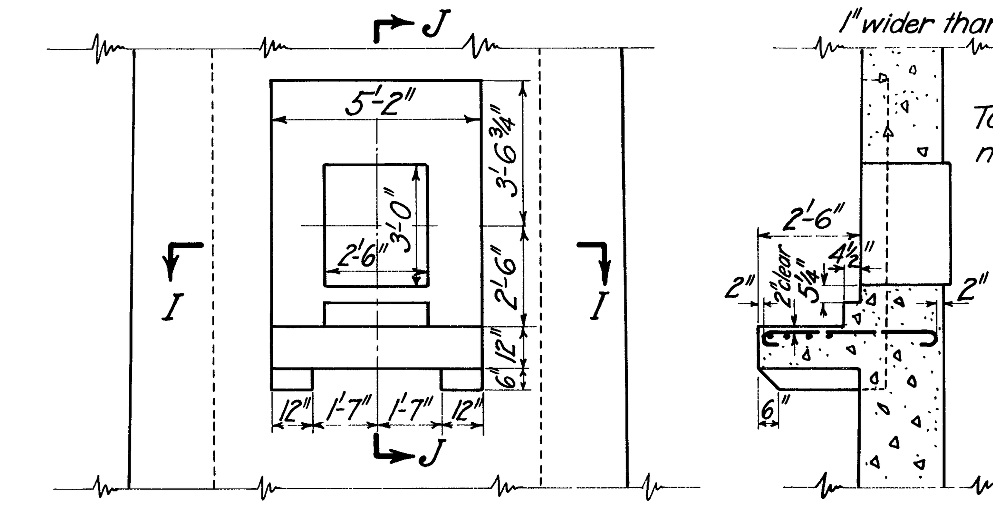


PLAN OF ROOF REINFORCEMENT
Scale ¼"=1'



SECTIONAL PLAN I-I

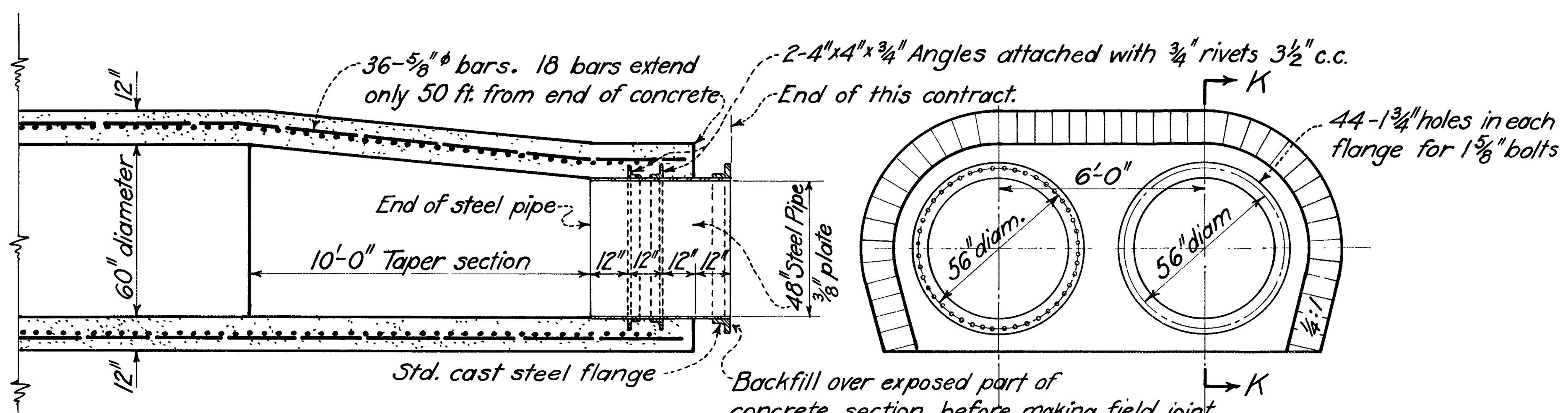
Note: See drwgs DH 1068-7 & DH 1067-7 for location of anchor bolts. If gate frames are set after tower is poured, temporary recesses in concrete must be neither more than 1" deeper nor 1" wider than frames.



ELEVATION SECTION J-J
DETAILS OF TRASH RACK SUPPORT
Scale ¼"=1'

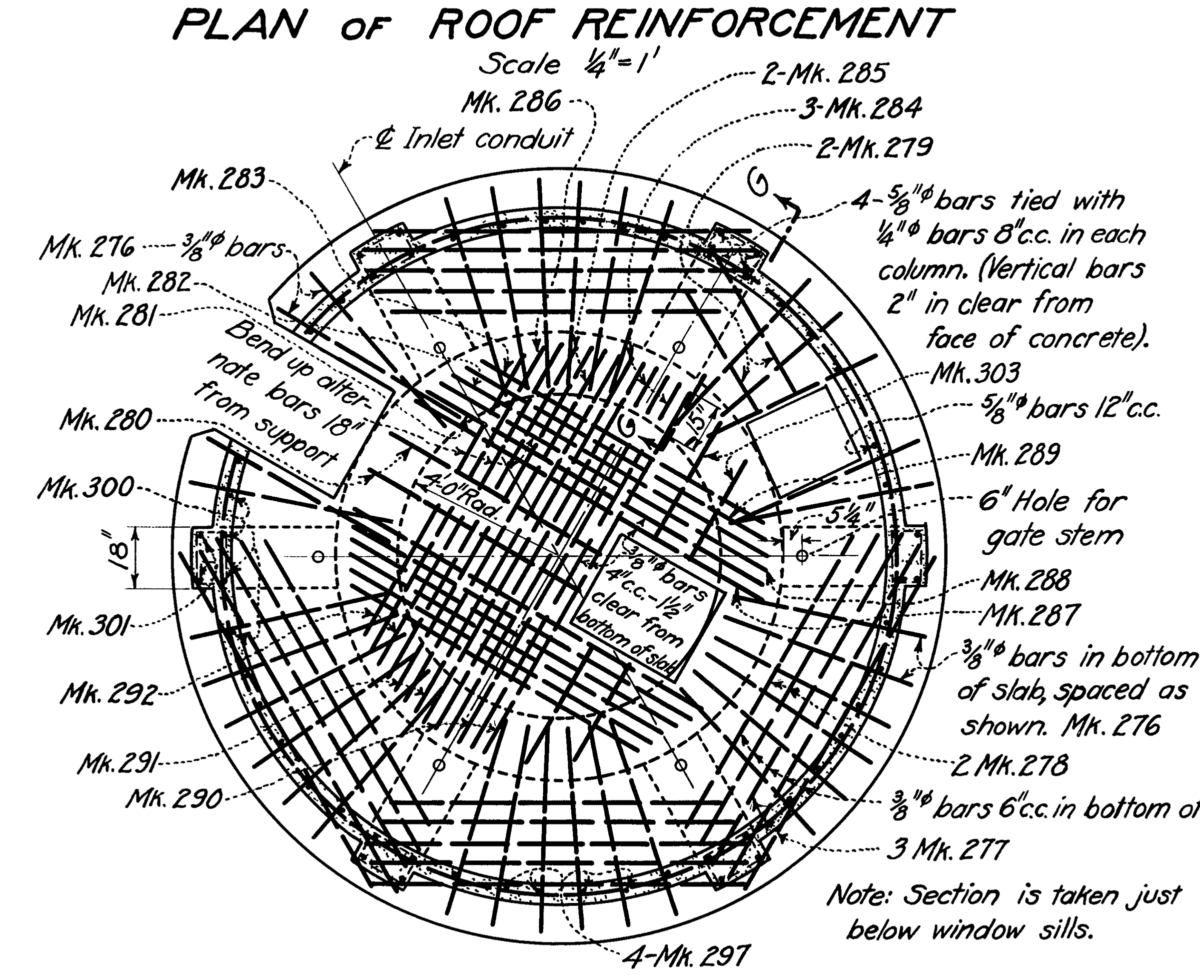
Note:
All concrete in operating tower and in conduits is to have a minimum compressive strength of 2000 #/□" when 28 days old.
The maximum size of coarse aggregate is to be as follows:

In tower - Below elev. 378	3"
Between elev. 378 and elev. 509.5	2"
Above elev. 509.5	¾"
In twin conduits	2"
In entrance channel and single conduit	1½"

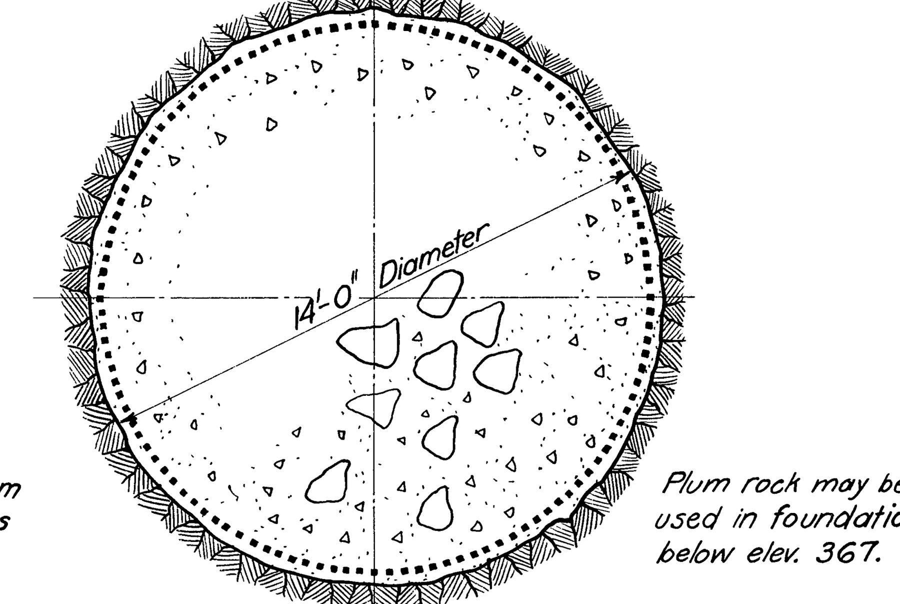


SECTION K-K
DETAIL OF CONNECTION
BETWEEN STEEL AND CONCRETE PIPES

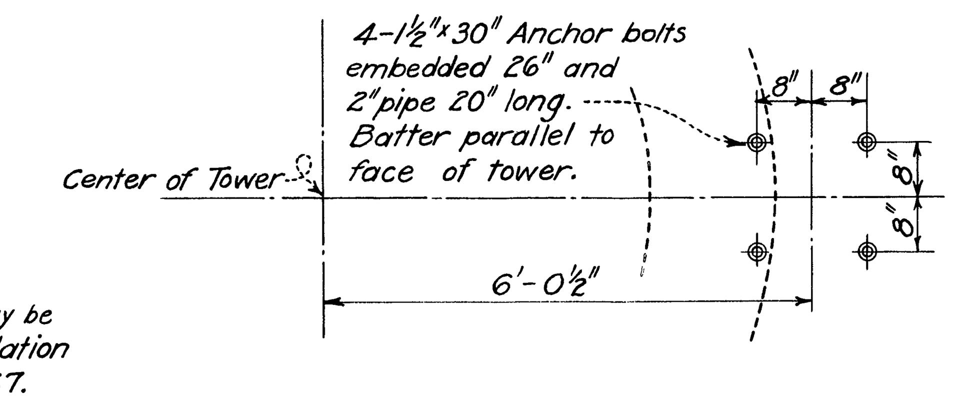
END ELEVATION



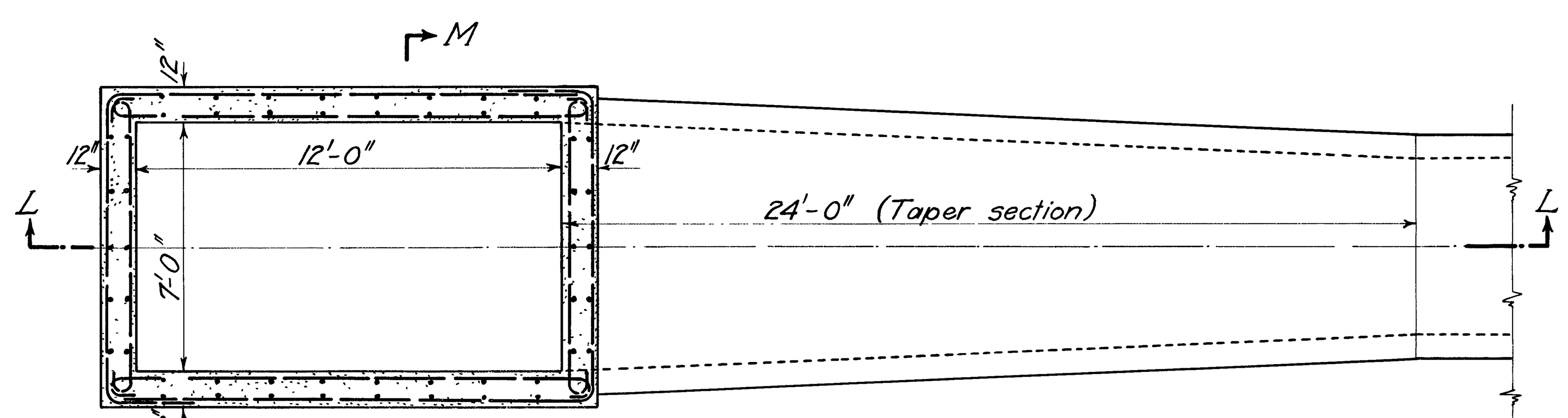
SECTIONAL PLAN SHOWING
OPERATING FLOOR AND WALL REINFORCEMENT
Scale ¼"=1'



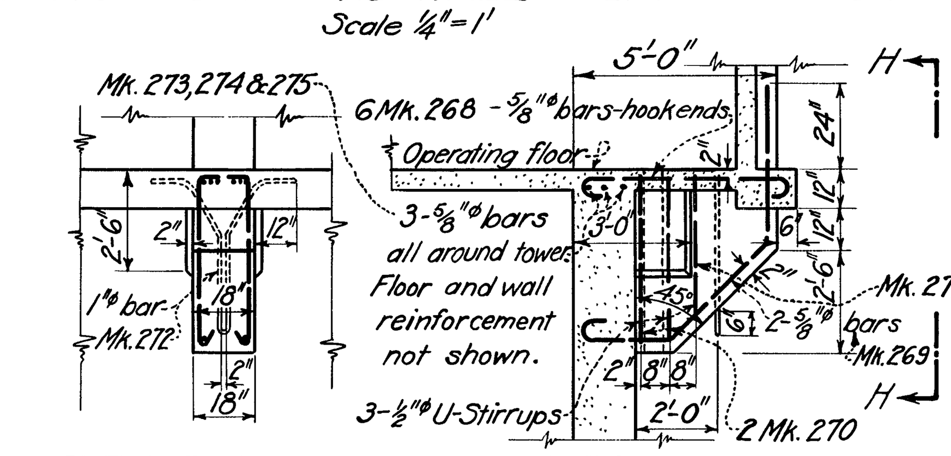
SECTIONAL PLAN
AT ELEVATION 358
Scale ¼"=1'



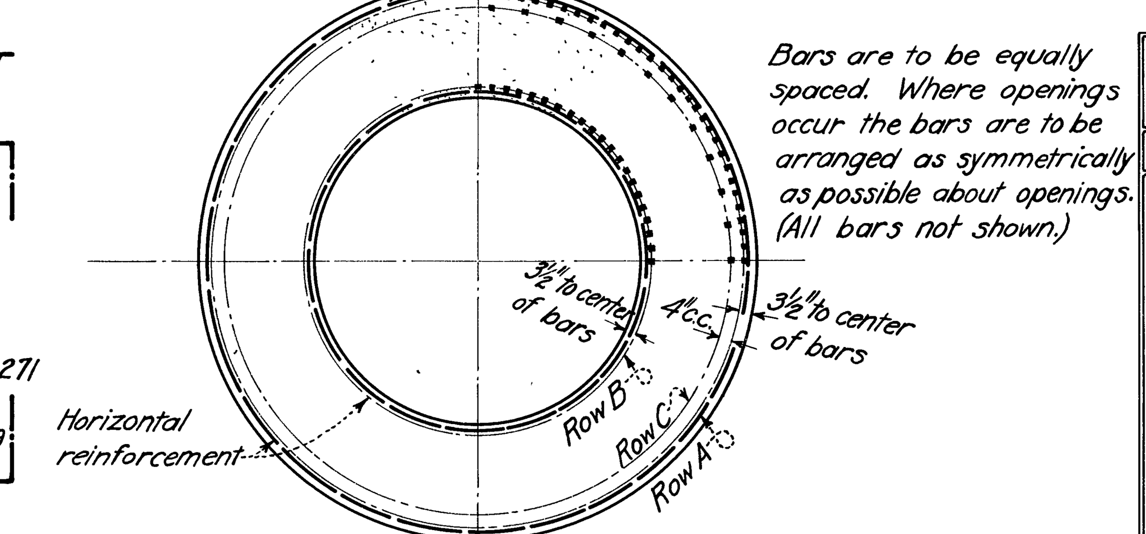
PLAN OF ANCHOR BOLTS
FOR GATE FLOOR STANDS



SECTIONAL PLAN N-N

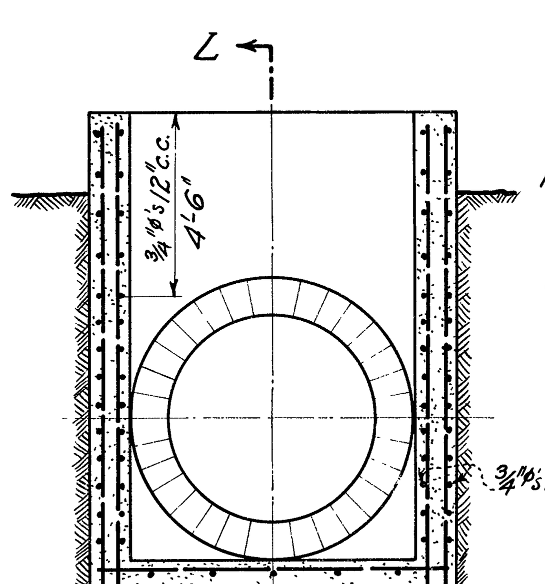


ELEVATION H-H SECTION G-G
BRACKET DETAILS
Scale ¼"=1'

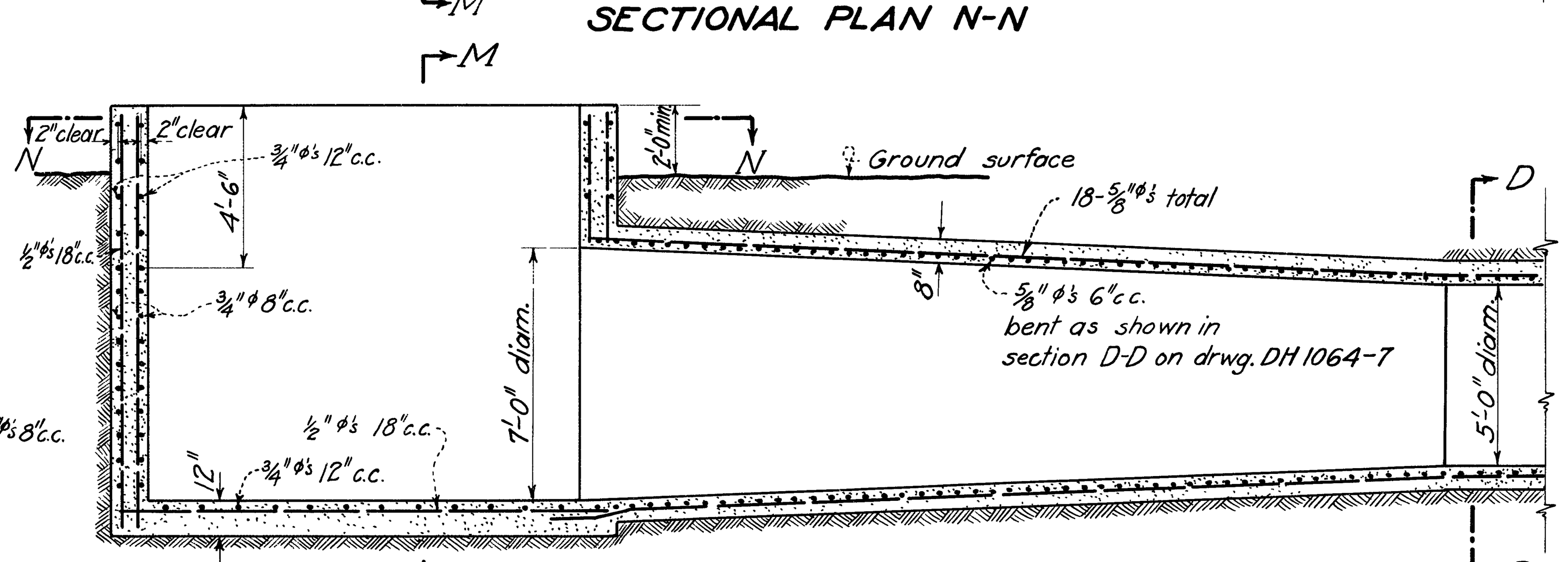


TYPICAL SECTION
SHOWING TOWER REINFORCEMENT

LOCATION OF STEM GUIDES & COUPLINGS						
GATE ELEVATIONS	384	400	410	430	450	470
ELEVATIONS OF TOPS OF STEM GUIDES	487.83	487.83	487.83	487.83	487.83	487.83
	480.83	480.83	480.83	480.83	480.83	480.83
	473.83	473.83	473.83	473.83	473.83	476.00
	466.83	466.83	466.83	466.83	466.83	
	459.83	459.83	459.83	459.83	459.83	
	452.83	452.83	452.83	452.83	456.00	
	445.83	445.83	445.83	445.83		
	438.83	438.83	438.83	438.83		
	431.83	431.83	431.83	436.00		
			424.83			
	427.83	427.83	417.83			
	420.83	420.83				
	413.83	413.83				
	406.83	406.83				
	399.79					
	392.75					
ELEVATIONS OF ℄'s OF STEM COUPLINGS (GATES CLOSED)	488.42	488.42	488.42	488.42	488.42	488.42
	468.42	468.42	468.42	468.42	468.42	
	448.42	448.42	448.42	448.42		
	432.42	432.42	432.42			
	422.42	422.42				
	402.42					



SECTION M-M



SECTION L-L

DETAIL OF END OF INLET CONDUIT
Scale ¼"=1'

VERTICAL REINFORCEMENT IN TOWER										
ELEVATION	ROW A Size	ROW A No. bars in row	ROW A Spacing c.c.	ROW B Size	ROW B No. of bars in row	ROW B Spacing c.c.	ROW C Size	ROW C No. of bars in row	ROW C Spacing c.c.	Total no. of bars
500	⅝"φ	30	13.2"	⅝"φ	30	10.7"	—			60
490	⅝"φ	40 a	10.3"	⅝"φ	30	10.7"	—			70
470				⅝"φ	24 b	13.4"	—			24
	1"φ	32	13.3"				—			32
460	1"φ	40	10.8"	1"φ	24	13.4"	—			64
450	1"φ	50	8.8"	1"φ	40	8.0"	—			90
440	1"φ	60	7.5"	1"φ	60 c	5.3"	—			120
430	1"φ	60 d	7.6"				—			60
				1¼"□	50	6.4"	—			50
420	1¼"□	60	7.7"	1¼"□	50	6.4"	—			110
410	1¼"□	80	5.9"	1¼"□	60	5.3"	—			140
400	1¼"□	100	4.8"	1¼"□	80	4"	—			180
390	1¼"□	120	4.1"	1¼"□	80	4"	—			200
380	1¼"□	120	4.1"	1¼"□	80	4"	1¼"□	45	10.4"	245
370	1¼"□	120	4.2"	1¼"□	80	4"	1¼"□	90	5.3"	290
365	1¼"□	120		—			—			120
356										

a. Start at elev. 468, to provide lap.
b. " " " 458, " " ".
c. " " " 426.5, " " ".
d. Start at elev. 416.5, to provide lap.
See typical section (above) for location of rows.
Splices must be staggered.

HORIZONTAL REINFORCEMENT IN TOWER			
ELEVATION	SIZE OF BAR	SPACING c.c.	NOTES
500			① Reinforcement is to be placed in pairs, each pair to be spaced as shown. ② Splices must be staggered.
	⅝"φ	12"	
450			
	¾"φ	16"	
430			
	¾"φ	14"	
400			
	¾"φ	12"	
369			

Revised by L.D.W.
Sept. 29, 1927.
Designation marks for reinforcing steel added.

NOTES:
Where splices in reinforcement are necessary, lap bars 40 diameters.
Radii of hooked bars are to be 4 diameters of bar.
Minimum spacing of bars shall be as follows:

⅜"φ - 1" c.c.	¾"φ - 1⅞" c.c.	½"□ - 1⅝" c.c.
½"φ - 1¼" c.c.	⅞"φ - 2¼" c.c.	1"□ - 3⅛" c.c.
⅝"φ - 1⅝" c.c.	1"φ - 2½" c.c.	1⅛"□ - 3½" c.c.
		1¼"□ - 4" c.c.

All bars are to be deformed.

Revised April 2, 1928. L.D.W.

EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND - CALIFORNIA
Arthur P. Davis, Chief Engineer and General Manager.

MOKELUMNE RIVER PROJECT

LAFAYETTE RESERVOIR
OPERATING TOWER
DETAILS OF REINFORCING STEEL

Drawn by L.D. Wilbur.	Scale: ¼"=1'
Traced by L.D. Wilbur.	Date: May 2, 1927.
Checked by R.C. Kennedy	№ DH 1065-7
Approved by F.W. Hanna	

Appendix E

Geotechnical Investigation Report

Geotechnical Investigation Report

Lafayette Reservoir Outlet Tower Seismic Retrofit Project
East Bay Municipal Utility District

SUBMITTED TO:

AECOM
c/o Mourad Attalla, PhD, PE, SE
Project Manager
300 Lakeside Drive, Suite 400
Oakland, CA 94612
Mourad.attalla@aecom.com

February 25, 2019

A3GEO

Revision History

Revision	Revision Date	Details	Name	Position
0	November 20, 2018	Initial Draft	Dona Mann	Geotechnical Engineer
1	February 25, 2019	Final Draft	Dona Mann	Geotechnical Engineer

February 25, 2019

Mourad Attalla, PhD, PE, SE
Project Manager
AECOM Technical Services, Inc.
300 Lakeside Drive, Suite 400
Oakland, California 94612

Geotechnical Investigation Report
Lafayette Reservoir Outlet Tower Seismic Retrofit Project
East Bay Municipal Utility District

Dear Mr. Attalla

This report presents the results of A3GEO's geotechnical investigation for the East Bay Municipal Utility District's (EBMUD) Lafayette Reservoir Outlet Tower Seismic Retrofit Project (Project) in Lafayette, California. A3GEO's services were provided as a subconsultant to AECOM, project design engineer and prime consultant for the project. A3GEO was authorized under a Master Consulting Services Subcontract with AECOM, Task Order No. 103033, dated February 1, 2016.

The purpose of this report is to provide site specific geologic and geotechnical data to be used in the retrofit design. A3GEO conducted an over-water, geotechnical boring near the existing Tower to explore subsurface conditions, performed laboratory testing from collected samples and rock cores and conducted downhole geophysical logging to measure compressional (P) and shear (S) and wave velocities of the alluvium and underlying bedrock. The results of our investigation will be used by AECOM to update the geotechnical parameters needed for final design.

A3GEO appreciates the opportunity to work with you on this exciting project. Should you have questions or concerns regarding the contents of this report, please do not hesitate to call.

Yours very truly,

A3GEO, Inc.

Sarah Khosravani, PE
Project Engineer
(650) 338-7205

Dona Mann, PE, GE
Principal Engineer
(415) 425-0247

TABLE OF CONTENTS

1. INTRODUCTION

1.01 General

This report presents the results of A3GEO's geotechnical investigation for the East Bay Municipal Utility District's (EBMUD's) Lafayette Reservoir Outlet Tower Seismic Retrofit Project (Project) in Lafayette, California. A list of references used in this study is presented in Section 7. Technical figures and appendices follow the reference list. Elevations in this report are in feet and reference the EBMUD Aqueduct Datum.

1.02 Site and Project Overview

As shown on the Site Plan, Figure 1, the Tower is located at the upstream toe of the embankment dam in Lafayette Reservoir. The Tower was built in approximately 1927 during the construction of the dam and serves as the dam's inlet/outlet conduit and houses the overflow spillway for the reservoir. The Tower is a 170-foot-high, reinforced concrete structure extending 43 feet below grade. The above-grade portion of the Tower is hollow with a constant inside diameter of about 8 feet and an outside diameter varying from about 11 feet at the top to about 14 feet at grade level. The below-grade portion of the Tower includes a solid concrete shaft (14 to 16-foot in diameter) supported on a 26-foot by 24-foot rectangular (4-foot-thick) footing. A more detailed description of the Tower is included in Section 4.02.

The Project involves designing a retrofit for the Lafayette Reservoir Outlet Tower to address seismic, mechanical and electrical deficiencies. Currently, the structural retrofit alternatives include: 1) adding post-tensioned anchors extending from the operating house down through the walls and into bedrock, and 2) shortening the Tower.

1.03 Purpose and Scope of Services

The primary purpose of our investigation was to investigate and characterize the geotechnical conditions in the vicinity of the Tower. The scope of our investigation consisted of the following:

- Reviewing pre-existing geotechnical investigation reports and available information;
- Conducting a geotechnical site reconnaissance;
- Exploring subsurface conditions with one high-quality geotechnical boring extending to approximately 100 to 120 feet below grade;
- Performing down-hole suspension geophysical logging;
- Conducting geotechnical laboratory tests;
- Characterizing the geotechnical, geologic and seismic conditions;
- Developing an interpretive geologic cross section at the Tower location;
- Developing geotechnical engineering properties of subsurface materials;
- Comparing new data with data used in previous studies; and
- Preparing this geotechnical investigation report.

Our scope was focused on collecting geotechnical data needed for AECOM to update the site-specific earthquake ground motions and structural model of the Tower. Our investigation did not include evaluations of potential geologic hazards such as faulting, liquefaction, landsliding and/or slope stability.

2. METHODS OF INVESTIGATION

2.01 Review of Existing Information

We reviewed pre-existing geotechnical investigation reports, seismic evaluation reports, geologic and historic maps, as-built drawings and available information provided by AECOM and/or EBMUD relevant to the project and the site. A list of selected items that we reviewed as part of this study is presented in Section 7, "References."

2.02 Site Reconnaissance

We conducted site reconnaissance visits at various times in May, June and September 2018. During these visits, we observed the surficial conditions at the site, verified site accessibility and selected a suitable location to drill a boring and stage equipment during drilling operations.

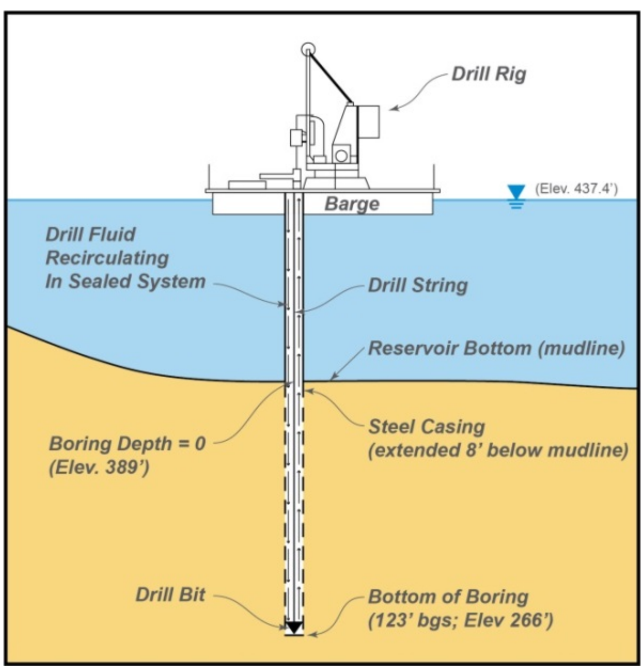
2.03 Subsurface Exploration

2.03.1 Drilling Preparation

Prior to drilling, A3GEO: 1) developed a detailed Geotechnical Field Investigation Work Plan which was incorporated into AECOM's memorandum to EBMUD dated July 10, 2018; 2) coordinated site access and drilling procedures with AECOM and EBMUD; and 3) notified Underground Service Alert (USA) of our intent to drill. In addition, our drilling subcontractor (Taber Drilling Company, Inc.) provided an affidavit to EBMUD confirming where the equipment (including barge, boats and motors) had been the month prior to arriving on site.

2.03.2 Test Boring

An over-water boring in close proximity to the Tower (versus an on-shore boring) was selected due to the lack of reliable geotechnical data within the vicinity of the Tower. The drilling operations took place between September 17th and September 21st, 2018. The approximate location of Boring B-1 (about 30 feet from the center of the Tower) is shown on the Site Plan, Figure 1. Taber Drilling Company, Inc. of West Sacramento drilled the boring with a CME 45 drill rig mounted on a barge using rotary wash method. A schematic of the drilling set-up is illustrated below. The elevations shown are relative to EBMUD Aqueduct Datum.



Schematic of Drilling Set-Up

As shown on the schematic above, the reservoir water elevation at the time of our investigation was recorded at 437.4 feet (https://www.ebmud.com/water/about-your-water/water-supply/water-supply-reports/daily-water-supply-report/). The ground surface elevation at the boring location was determined (by measuring from the deck of the barge) to be approximately 389 feet. A continuous steel casing extended from the deck of the barge into the subsurface soil to about 8 feet below the mudline creating a closed mud rotary system for drilling. A 4⅞ -inch diameter tri-cone bit was used to drill the upper 46.5 feet of the boring through the alluvium. Below 46.5 feet, an American Diamond Tool (ADT) CH-3 wireline core barrel system was used to core through the bedrock. An A3GEO geotechnical engineer directed the drilling, sampling and coring operations and prepared field logs of the subsurface conditions encountered.

Samples of the subsurface materials were obtained using the following equipment:

- 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) drive samplers without liners;
- 3-inch O.D. California Modified drive samplers with liners;
- Pitcher Barrel sampler equipped with 36-inch-long 3-inch O.D. thin-walled steel tubes;
- Shelby Tube sampler with 30-inch-long 3-inch O.D. thin-walled steel tubes; and,
- ADT CH-3 wireline core barrel (2.5-inch diameter).

The SPT and Modified California drive samplers were advanced using a 140-pound automatic-trip hammer falling 30 inches with an 80% average efficiency. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring log. Sampler blow counts presented on the logs are adjusted N-values. Blow counts have been adjusted for sampler type only. Rock Quality Designation (RQD) was determined in the field for each core run and is presented on the boring log. Photographs of the rock core recovered from the boring are included in Appendix B.

An A3GEO engineer reviewed samples in the laboratory to check field classifications and select suitable specimens for testing. Soils were classified in general accordance with ASTM D2488, which is based on the Unified Soil Classification System (USCS). The log of the boring is attached in Appendix A preceded by: 1) a Key to Exploratory Boring Logs that describes the USCS and the symbols used on the logs; and 2) a Key to Rock Descriptions. Generalized descriptions of the conditions encountered at the location of Boring B-1 can be found in Section 4.03, "Subsurface Conditions."

The attached boring log depicts interpreted subsurface conditions at the approximate location shown on the Site Plan (Figure 1) on the particular dates designated on the log; the passage of time may result in changes in the subsurface conditions. The approximate boring location indicated on the Site Plan was determined using a GPS coordinate tracker and was cross-checked with some of the existing improvements on-site.

2.03.3 Downhole Geophysical Logging

On September 20, 2018, NORCAL Geophysical Consultants, Inc. (NORCAL) conducted a downhole suspension logging investigation within Boring B-1. The downhole logging investigation utilizes an elongated tool equipped with a source and receivers, which is lowered into a fluid-filled borehole. The source generates a pressure wave, which is converted to seismic pressure and shear waves (P- and S-waves) at the borehole wall. The elapsed time between the arrivals of the waves at the receivers is used to evaluate average P- and S-wave velocities at stationary intervals within the column of soil/rock that surrounds the borehole. The results of the survey, including the shear wave velocity profile, are presented in NORCAL's Borehole Geophysical Logging Investigation Report which is included in Appendix C.

2.04 Geotechnical Laboratory Testing

Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical properties of the soils and rock that underlie the site. The following geotechnical laboratory tests were performed:

- Water content per ASTM D-2216;
- Dry density per ASTM D-7263;
- Atterberg Limits per ASTM D-4318;
- Particle size analysis per ASTM D-422;
- Percent minus #200 sieve per ASTM D-1140;
- Consolidated-undrained triaxial test with pore pressure measurements per ASTM D-4767; and
- Unconfined compression test on rock cores per ASTM D-2166.

Geotechnical laboratory tests were conducted by Inspection Services Inc (ISI) geotechnical laboratory in Berkeley, California. The preceding tests were conducted in general accordance with the current edition of the referenced ASTM standards at the time the tests were performed. The results of the tests are presented on the boring log presented in Appendix A at the appropriate sample depths, and the laboratory test data sheets are included in Appendix D.

3. GEOLOGIC AND TECTONIC SETTING

3.01 General

The geologic and tectonic setting of Lafayette Reservoir has been studied in depth by other consultants and further study was not specifically included in A3GEO's scope of work for this project. The most relevant available reports containing geologic and tectonic setting information with respect to the Tower retrofit project are listed below:

- GEI Consultants, Inc. (GEI), "Dynamic Stability Review of Lafayette Dam Report," dated August 16, 2005.
- International Civil Engineering Consultants, Inc. (ICEC), "Seismic Evaluation of Lafayette Reservoir Outlet Tower, Contra Costa County, California," dated April 1995.
- W.A. Wahler and Associates, Inc., "Seismic Stability Evaluation, Lafayette Dam, Contra Costa County, California," dated May 1976.
- Shannon and Wilson, Inc., "Review of Stability of Lafayette Dam," dated January 1966.
- EBMUD, "Lafayette Dam Foundation Investigation and Stability Analysis," dated November 1, 1957.

3.02 Geologic Setting

In summary, the project site is located in the East Bay Hills which consist of Tertiary age sedimentary and volcanic rocks which are highly folded and frequently faulted. Lafayette Dam blocks a short valley that has been carved into the folded sedimentary rocks of the Orinda formation. At the dam site, the old creek originally cut a deep trench which was later filled with alluvium to depths up to 90+ feet. The portion of the valley containing the Lafayette Reservoir is approximately 1,000 to 3,000 feet wide and is surrounded by moderately steep to very steep hills rising 350 feet or more above the valley floor (i.e., bottom of the reservoir).

The Regional Geologic Map (Dibblee, 2005) presented on Figure 2 shows the general geology of the area. At the location of the Tower, the area is mapped as being underlain by Holocene-aged alluvial surficial sediments (map symbol, Qa) over Pliocene to possibly late Miocene-aged Orinda formation bedrock (map symbol, Tor). Dibblee describes these deposits as follows:

- **Qa**: Surficial sediments, alluvial gravel, sand and clay of valley area.

- **Tor**: Orinda formation, Terrestrial pebble conglomerate of Franciscan detritus, sandstone and claystone interbedded, gray to greenish gray.

Two geologic cross sections (Figure 3) were developed by W.A. Wahler in 1976 to illustrate the geologic stratigraphy underlying Lafayette Dam. The locations of the cross sections (A-A' and B-B') are shown on the Site Plan, Figure 1, and the Regional Geologic Map, Figure 2. Cross Section A-A' intersects the dam at its maximum width. For reference, the projected location of the Tower is shown at the upstream toe of the dam on Section A-A'. Section B-B' passes through the longitudinal axis of the dam and illustrates the steep rise of Orinda formation bedrock on both sides of the valley blocked by Lafayette Dam.

Both sections illustrate that the dam is underlain by alluvium (Qal) over sedimentary rocks of Orinda formation (To). The Alluvium (Qal) is described as clayey soils with some sand and gravel, and the Orinda formation bedrock is described as moderately weathered siltstone/claystone beds, interfingered with moderately weathered clayey sandstone beds. The alluvium reaches a maximum depth of about 95 feet beneath the middle portion of the dam.

3.03 Tectonic Setting

The major active faults in the vicinity of project include the Hayward, Calaveras, Concord, Pleasanton, and San Andreas faults. Faults that are defined as active exhibit one or more of the following: (1) evidence of Holocene-age (within about the past 11,000 years) displacement, (2) measurable aseismic fault creep, (3) close proximity to linear concentrations or trends of earthquake epicenters, and (4) prominent tectonic-related aseismic geomorphology. The following table summarizes the active faults in the vicinity of the project.

Active Faults in the Vicinity of the Project Site (Jennings and Bryant 2010)

Fault System	Fault Type	Maximum Magnitude of Credible Earthquake. M_{max}	Distance from Fault to Project
Hayward	Strike Slip	7.25	5.5 miles
Calaveras (North)	Strike Slip	7.25	6 miles
Concord	Strike Slip	6.5	8 miles
Pleasanton	Strike Slip	6.5	9.4 miles
San Andreas (North)	Strike Slip	8.0	24 miles

As noted in the preceding table, the closest regional active fault to the site is the Hayward fault. The Hayward fault system is one of the primary active faults in the San Francisco Bay Region, and overall has the highest probability of generating a large-magnitude earthquake within the next 30 years (WGCEP, 2013). The Hayward fault system extends approximately 95 miles from Fremont to Healdsburg and is interpreted as stepping to the right beneath San Pablo Bay.

The greatest Maximum Magnitude of credible earthquake (M_{max}) belongs to San Andreas fault. The northern segment of San Andreas fault runs from Hollister, through the Santa Cruz Mountains up to San Francisco Peninsula and then offshore at Daly city. Despite its distance from the project site (approximately 24 miles), it may impose a significant seismic hazard, since it is capable of generating moderate to strong shaking with long duration (60 seconds or more).

Smaller faults, such as the Lafayette-Reliez Valley (LRV) at a distance of 1.9 miles, Franklin (4 miles) and Miller Creek (5.9 miles) could also generate significant motion at the site, although of shorter duration than upper-bound magnitude events along the Hayward or Calaveras (GEI, 2005).

As part of this project, AECOM performed a site-specific deterministic seismic hazard analysis (DSHA) and presented the results in a stand-alone Technical Memorandum dated November 26, 2018 (AECOM, 2018b). AECOM's technical memorandum provides a detailed discussion of the seismic sources considered in the analysis.

3.04 Mapped Landslides

GEI's 2005 report identifies numerous landslides along the margins of Lafayette Reservoir. The landslides were identified based on aerial photographs and site reconnaissance visits and are shown on the Regional Geologic Map, Figure 2. Excerpts from the GEI (2005) report follow:

> *An active landslide is present above the parking lot for the Visitor Center at the northwest corner of the reservoir. This landslide may have been triggered, in part, by grading for the reservoir facilities. We note the landslide is not visible in 1928 photographs, but appears to be fully developed by 1939.*
>
> *An older landslide is located adjacent to the right (east) abutment. The landslide appears to be old, based on a subdued landform, but still retains the distinctive remnants of a headscarp and landslide body. If the landslide actually underlies a corner of the embankment, and experiences movement in the future, it could potentially damage the downstream toe of the embankment. This would not affect, however, the overall safety of the dam.*

3.05 Liquefiable Deposits

Although a liquefaction evaluation was beyond the scope of this project, the materials encountered in Boring B-1 are not considered susceptible to liquefaction. The soils encountered were high in fines (i.e., minus #200 sieve ≥ 48 percent) and moderately to highly plastic (i.e., 17 ≤ PI ≤ 36).

4. SITE CONDITIONS

4.01 Dam Construction and 1928 Failure

The following information regarding the construction of Lafayette Dam was obtained from the following reports:

- Shannon and Wilson 1966 report titled, "Review of Stability of Lafayette Dam,"
- EBMUD 1957 report titled, "Lafayette Dam Foundation Investigation and Stability Analysis," and
- GEI 2005 report titled, "Dynamic Stability Review of Lafayette Dam Report."

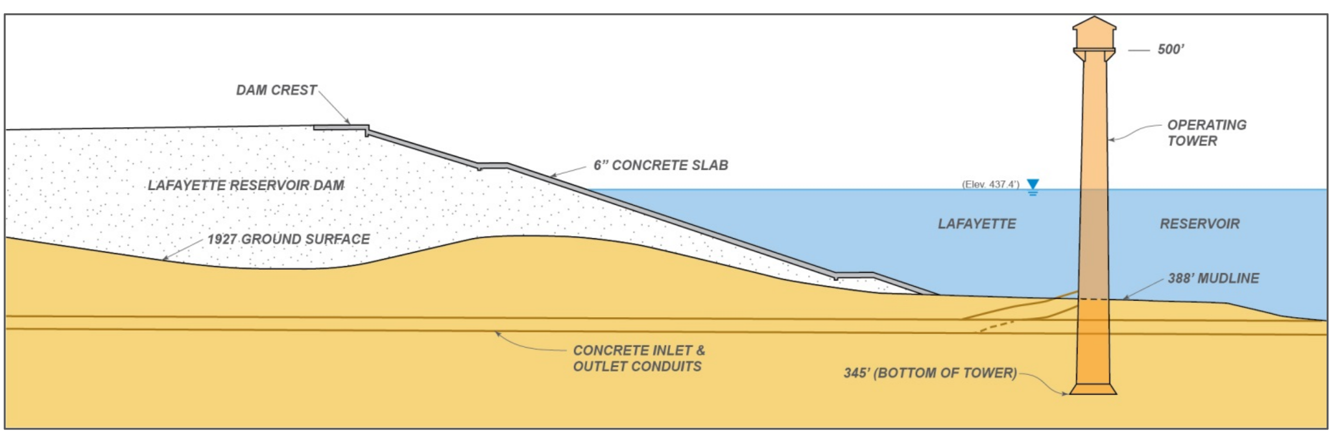
The information included below is for reference and is not intended to be a comprehensive description of the construction of the dam.

Construction of the dam began in August 1927 and was completed in 1933. The fill for the dam was obtained from the reservoir area and from side hills above the dam. A massive failure involving both the embankment and the foundation occurred during construction in 1928. The failure was characterized by cracking and subsidence of the entire crest of the dam over a width of about 525 feet. The crest settled approximately 24 feet, and the downstream toe rose about 20 feet. The upstream portion of the crest settled approximately 10 feet and the upstream toe moved outward about five feet. The concrete inlet-outlet conduit was apparently cracked and extended approximately four inches over a 100-foot length immediately upstream of the concrete cut-off wall, and a 24-inch pipe was laid in one of the barrels following the embankment failure (Shannon and Wilson, 1966). The Board of Consultants that convened after the failure concluded that the failure was apparently caused by excessively high pore pressure which developed within the foundation alluvium as a result of the rapid rate of construction.

Remedial work included filling cracks and scarps, removing bulged foundation soil at downstream toe and re-designing the dam to have a flatter downstream slope and a lower/wider crest, but the failed materials were essentially left in place. The concrete slab facing on the upstream face was repaired or replaced. The crest elevation was originally planned at El. 500 feet but was constructed at El. 467 feet (33 feet lower than originally planned). Because the Tower was constructed in proportion to the originally planned dam, it currently stands about 45 feet higher than the crest of the dam.

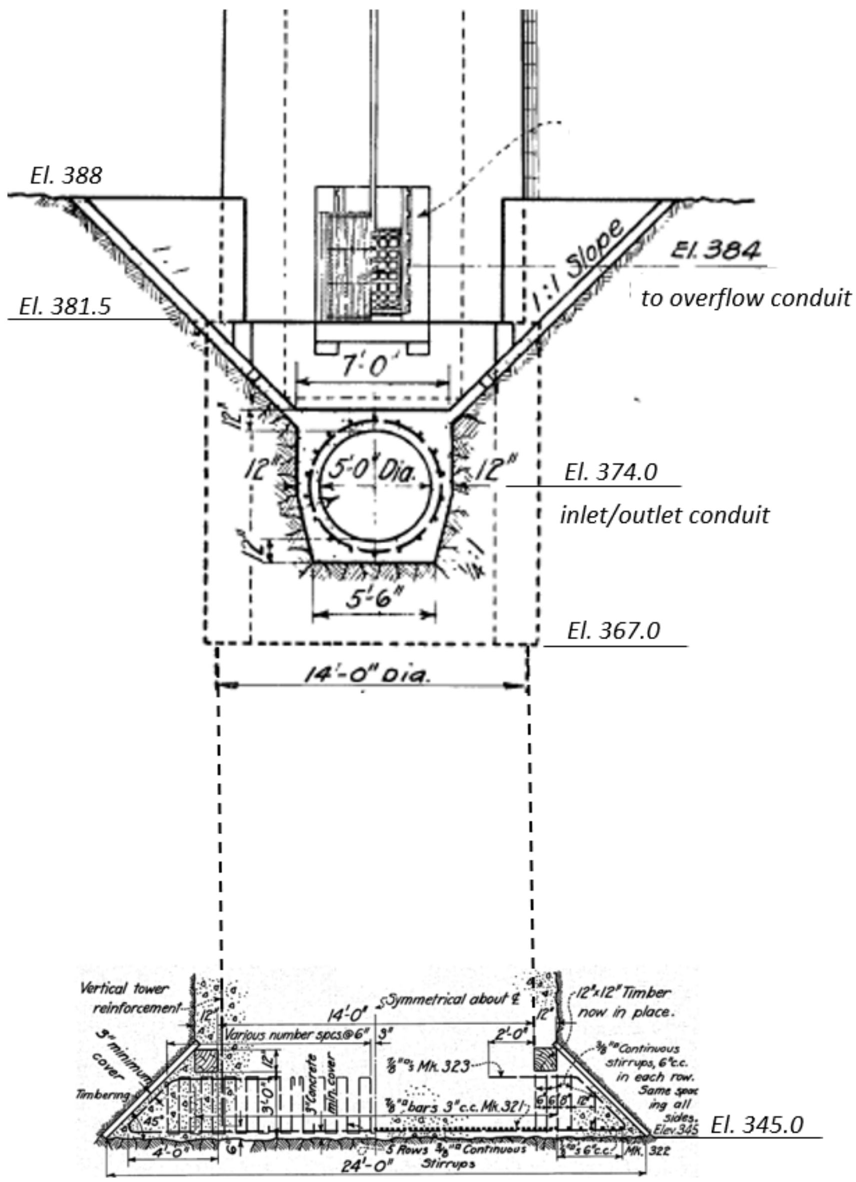
4.02 Tower Description

A schematic profile showing the Tower and the Tower's inlet/outlet conduits in relation to the dam is presented below. The ground surface elevation at the Tower is El. 388 feet and the elevation at the bottom of the foundation is El. 345 feet. The Tower is reported to be embedded 43 feet below grade.



The Tower, including the operating house and portion below grade, is approximately 170 feet high. The operating house sits on top of a hollow cylindrical structure with a constant inside diameter of about 8 feet and an outside diameter varying from about 11 feet at the top (El. 500 feet) to about 14 feet at grade level (El. 388 feet).

The below-grade portion of the Tower, as shown on the construction drawings (see below), includes a solid 14 to 16-foot diameter concrete shaft supported on a 26-foot by 24-foot rectangular (4-foot-thick) tapered footing. A 60-inch diameter inlet/outlet concrete conduit enters and exits the Tower at El. 374.0 feet. A 60-inch diameter overflow conduit exits the Tower at elevation 384.0 feet and then runs side by side with the inlet/outlet conduit forming a twin configuration. These concrete conduits extend horizontally under the reservoir bed and dam embankment as shown with yellow dashed lines on the Site Plan (Figure 1).



4.03 Subsurface Conditions

4.03.1 Overview

Boring B-1 drilled for this study encountered about 44 feet of alluvial deposits over bedrock. The alluvial deposits encountered generally consist of stiff to very stiff clays (CL and CH) with some medium dense layers of clayey sand (SC). The bedrock generally consists of relatively weak Orinda formation claystone, sandstone and siltstone. The subsurface materials encountered in Boring B-1 are discussed in more detail in the following section and also on the boring log included in Appendix A.

To further evaluate the subsurface conditions near the Tower, an interpretive Geologic Cross Section C-C' (Figure 4) was developed. The location of Cross Section C-C' is shown on the Site Plan, Figure 1, and on the Regional Geologic Map, Figure 2. Cross Section C-C' intersects the Tower, Boring B-1 drilled for this study (Appendix A), and two previous borings drilled by W.A. Wahler & Associates in 1976 (SS-25 and SS-27, Appendix E). All three of the borings included in Section C-C' are located outside the footprint of the dam embankment but are still within relatively close proximity to the Tower. Cross Section C-C' (Figure 4) illustrates how the Orinda formation bedrock rises steeply on both sides of the valley and that the alluvium is the thickest at the center of the valley floor about 400 feet east of the tower. A mapped landslide (GEI, 2005) is shown on the western hillside of the valley at the water's edge. The depth and downstream limits of this landslide are unknown.

Detailed descriptions of the subsurface materials interpreted to exist near the Tower (i.e., in Borings B-1, SS-25 and SS-27) are included in the following sections.

4.03.2 Alluvium

Alluvial deposits near the Tower (i.e., in Borings B-1, SS-25 and SS-27) generally consist of clay and sandy clay with moderate to high plasticity interbedded with layers of clayey sand. The clayey soils are typically characterized as stiff to very stiff with measured shear wave velocities between about 700 and 900 feet per second (ft/sec). The results of the laboratory tests performed on samples in alluvium are summarized in the table on the following page. The laboratory data sheets for this study are included in Appendix D; the laboratory data sheets for tests on samples from previous borings (SS-25 and SS-27) are included in Appendix F.

The effective and total shear strength parameters included in the table were determined from isotopically consolidated undrained triaxial tests (TXICU) with pore water pressure measurements performed on undisturbed samples from Boring B-1. The failure criterion of maximum effective stress obliquity (σ'_1/σ'_3) was used to obtain total and effective shear strength parameters.

Liquid Limits (LL) in the alluvium range from 31 to 53 percent and Plasticity Indices (PI) range from 16 to 36 with an average of 24 percent. Moisture contents vary between 20.4 and 35.7 percent and dry densities vary between 97.2 and 109.3 pounds per cubic foot (pcf). Unconfined compression strengths determined by either pocket penetrometer testing (PP) or unconfined compression testing (UC) ranged between 600 and 2,100 pounds per square foot (psf).

Summary of Laboratory Test Results on Alluvium

Boring ID	Sample Depth	Sample Elev.	USCS Soil Type	Water Content	Dry Density	Fines Content	Liquid Limit (LL)	Plasticity Index (PI)	Effective Stress Strength Parameters		Total Stress Strength Parameters		Unconfined Compression Strength
	feet	feet		%	pcf	%<#200	%	%	c' (psf)	Φ' (°)	c (psf)	Φ (°)	q_u (tsf)
B-1	3	386	CH	35.7									
B-1	6	383	CL	24.2	101.0								PP = 2.0
B-1	8	381	CL	20.5			47	30					
B-1	12.5	376.5	CH	20.4	108.2		53	36	218.7	31.4	430.5	17.9	
B-1	14.5	374.5	CH	25.2									
B-1	18	371	CL	22	106.1				285.2	27.7	491.3	16.9	PP = 1.7
B-1	19.5	369.5	CL	23.7		62	43	27					
B-1	23.5	365.5	CL	28.1									
B-1	25	364	CL	24.0									
B-1	27.5	361.5	CL	21.0	106.1		44	26	201.3	28.4	298.1	18.0	PP = 1.8
B-1	29.5	359.5	CL	21.4									
B-1	34	355	SC	22.3		48	38	22					
B-1	38.5	350.5	CL/SC	20.9	106.8	50	34	17					PP = 1.25
B-1	43.5	345.5	CL	20.8									
SS-25	13	364	CL/CH	26.1	97.2	85							UC = 0.6
SS-25	28	349	CL	23.7	104.2	72	48	28					
SS-25	53.5	323.5	CL	21.2	109.3	74	35	17					UC = 2.1
SS-25	87.5	289.5	CL	21.2	107.9	67	34	18					UC = 1.5
SS-27	21.5	347.5	CL	27.8	97.2	93	47	26					UC = 1.2
SS-27	45	324	CL	21.3	108.2	57	31	16					UC = 1.0

UC = Unconfined compression test (ASTM 2166)
PP = Pocket Penetrometer

4.03.3 Orinda Formation Bedrock

Sedimentary bedrock (Orinda Formation) was encountered in Boring B-1 at a depth of 44 feet below the ground surface (bgs). Based on our field characterization and shear wave velocity (Vs) measurements, the bedrock generally becomes less weathered and slightly stronger with depth. Based on Rock Quality Designation (RQD), Vs and field characterization, the bedrock can reasonably be separated into the following layers:

- **Depth 44 to 66 feet bgs (El. 345 to 323 feet.):** The bedrock mostly consists of friable to moderately strong, deeply weathered layers of claystone and siltstone with low hardness. The Rock Quality Designations (RQD) of the cored sedimentary rock for this section varied between 68 to 95 percent; however, due to deep weathering, the RQD soundness requirements were *not* met. Shear wave velocities were generally recorded between about 900 and 1,200 feet/sec.

- **Depth 66 to 84 feet bgs (El. 323 to 305 feet.):** The bedrock generally consists of low to moderately hard, moderately strong layers of claystone, siltstone and sandstone with little weathering. The Rock Quality Designations (RQD) for the cored sedimentary rock for this section varied between 54 to 100 percent (fair to excellent mass quality). Shear wave velocities were generally recorded between about 1,200 to 1,500 ft/sec.

- **Depth 84 to 123 feet bgs (El. 305 to 266 feet.):** The bedrock generally consists of moderately hard, weak to moderately strong layers of fresh sandstone and siltstone. The Rock Quality Designations (RQD) for the cored sedimentary rock varied between 37 and 100 percent (poor to excellent rock mass quality). Shear wave velocities were generally recorded between about 1,500 to 2,000 ft/sec.

The results of the laboratory tests performed on qualified rock cores are summarized in the table below. Moisture contents range between about 7 and 15 percent with dry densities between about 117 and 134 pcf. The RQD values and results of unconfined compression (UC) strength tests are presented on the boring log (B-1) at the corresponding depth (Appendix A). The laboratory data sheets are included in Appendix D.

Summary of Laboratory Test Results on Bedrock

Boring ID	Sample Depth	Sample El.	Water Content	Dry Density	Unconfined Compression (UC) Strength (psi)
	feet	feet	%	pcf	
B-1	51.5	337.5	14.7	118.3	71.4
B-1	60.5	328.5	15.2	118.2	47.6
B-1	69.7	319.3	12.8	124.2	25.1
B-1	82	307	14.7	117.1	37.6
B-1	87.5	301.5	6.7	131.3	158.0
B-1	102	287	9.5	134.2	18.1
B-1	116	273	14.9	116.7	17.3

In general, the unconfined compression test results are indicative of extremely to very weak rocks (ISRM, 1981). Based on ISRM methodology, rock with UC strengths less than 150 psi are considered extremely weak and rock with UC strengths between 150 and 725 psi are considered very weak. The UC strengths generally correlate with the low shear wave velocities measured in the rock (i.e., Vs between 900 and 2,000 ft/sec). It is worth mentioning that the UC tests were performed on core samples collected 4 weeks prior to testing and were not stored in completely sealed containers which may have affected the test results.

5. CONCLUSIONS AND RECOMMENDATIONS

5.01 Geotechnical Parameters

A primary objective of this investigation was to confirm the geotechnical parameters (e.g., shear wave velocity, Vs, profile) utilized by previous consultants in the structural analyses of the Tower. To date, all geotechnical input for structural analyses of the Tower have been based on ICEC's 1995 report titled, "Seismic Evaluation of Lafayette Reservoir Outlet Tower."

In ICEC's 1995 seismic evaluation of the *above-grade* portion of the Tower, the Tower's foundation/soil system (which was considered to be everything below El. 388 feet including the inlet/outlet and spillway conduits) was modeled as a set of springs (translational and rotational) at the ground surface (El. 388 feet). These springs were subsequently used by the Division of Safety of Dams in 2011, McMillen Jacobs Associates in 2015, and TERRA Engineers/COWI in 2017 in their analyses of the Tower's seismic capacity.

The geotechnical parameters used to develop the ICEC 1995 springs included a Vs profile which assumed 25 feet of alluvium soil with a constant Vs equal to 550 feet/second over bedrock with a constant Vs equal to 1,250 feet/second. The soil stratigraphy (i.e., 25 feet of alluvium over rock) was obtained from a poorly recorded boring drilled at the Tower location in 1927 prior to construction. The 1927 boring (included in Appendix E and shown for reference on Cross Section C-C', Figure 4) did not contain detailed descriptions of the materials encountered, blow counts or any other pertinent information to accurately characterize the materials encountered. In addition, the elevation noted at the top of the boring does not correspond with the current ground surface at the Tower. The 1995 ICEC Vs values were derived primarily from Vs measurements collected by Woodward Clyde Consultants in 1975 (WWC, 1975).

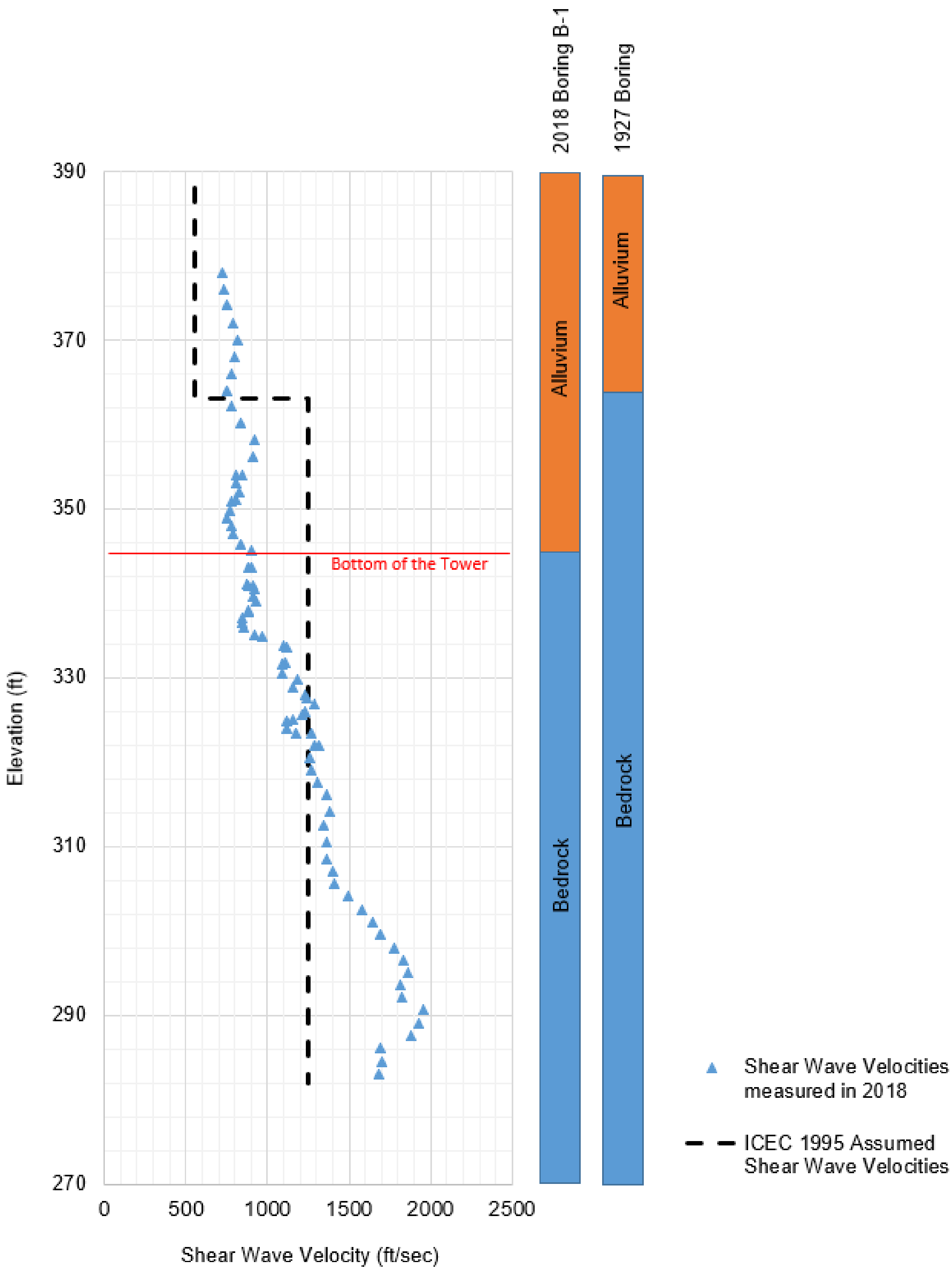
Boring B-1 drilled for this investigation, approximately 24 feet west of the Tower, encountered 44 feet of alluvium with Vs measurements between 723 and 903 feet/second over Orinda formation bedrock with Vs measurements between 844 to 1,959 feet/second.

A graphical comparison of the ICEC 1995 Vs profile and the Vs profile developed for this study is included on the following page. In summary, we conclude the following:

- The alluvium/bedrock contact was encountered deeper than ICEC assumed.
- The materials surrounding and supporting the Tower were found to be generally weaker than ICEC assumed, particularly between El. 363 and El. 334 feet.
- The Vs profile in bedrock (measured in Boring B-1 drilled for this study) gradually increases with depth, whereas it was assumed to be constant in the 1995 ICEC profile.

Base on the above, we judge that the springs developed by ICEC in 1995 are not representative of the Tower's foundation/soil system and therefore should not be used in future analyses.

In 1995, ICEC also performed a simplified evaluation of the seismic performance of the *below-grade* portion of the Tower assuming a soil-rock interface at El. 363 feet while deliberately eliminating the inlet/outlet and spillway conduits due to the added complexity. Considering the soil-rock interface was actually encountered significantly lower, and the integrity of the inlet/outlet and spillway conduits are critical to the performance of the Tower, we recommend re-evaluating the seismic performance of the entire below-grade portion of the Tower (including the inlet/outlet and spillway conduits).



Vs Profile Comparison

5.02 Tower Construction Considerations and Possibility of Artificial Fill

Based on the available information (i.e. construction drawings and photos), it is not known how the Tower was constructed below grade. For example, it is not known if the below-grade portion of the tower was constructed in a shored excavation or a sloped/benched excavation, or in some other way. If the excavation was shored or sloped and depending on where the shoring was installed or where the slopes began, it is possible the Tower is surrounded by artificial fill.

Based on our review and experience, we think it is probable that: 1) the below grade portion of the tower was actually constructed in a ±14-foot square-shaped excavation shored with timber, and 2) that the excavation was entirely filled with concrete with the timber shoring left in place.

None of the samples collected from Boring B-1 appear to contain artificial fill; however, it can be difficult to differentiate between native material and fill if onsite soils were used as fill. It is also possible, that Boring B-1 was drilled in native soil and that fill exists somewhere between Boring B-1 and the Tower.

6. <u>LIMITATIONS</u>

This report has been prepared for the exclusive use of AECOM, East Bay Municipal Utility District (EBMUD) and their consultants for specific application to the proposed Lafayette Reservoir Outlet Tower Seismic Retrofit Project in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

The findings of this report are valid as of the present date. However, the passage of time will likely change the existing conditions due to natural processes and works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by this office.

Our scope was focused on collecting geotechnical data needed for AECOM to update the site-specific earthquake ground motions and structural model of the Tower. Our investigation did not include evaluations of potential geologic hazards such as faulting, liquefaction, landsliding and/or slope stability.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require additional expenditure to be made during any construction to attain a properly constructed project. In the event, any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered, these subsurface data shall not be considered sufficient unless we are given the opportunity to review the nature of the variations or conditions in order to assess whether additional exploration will be required.

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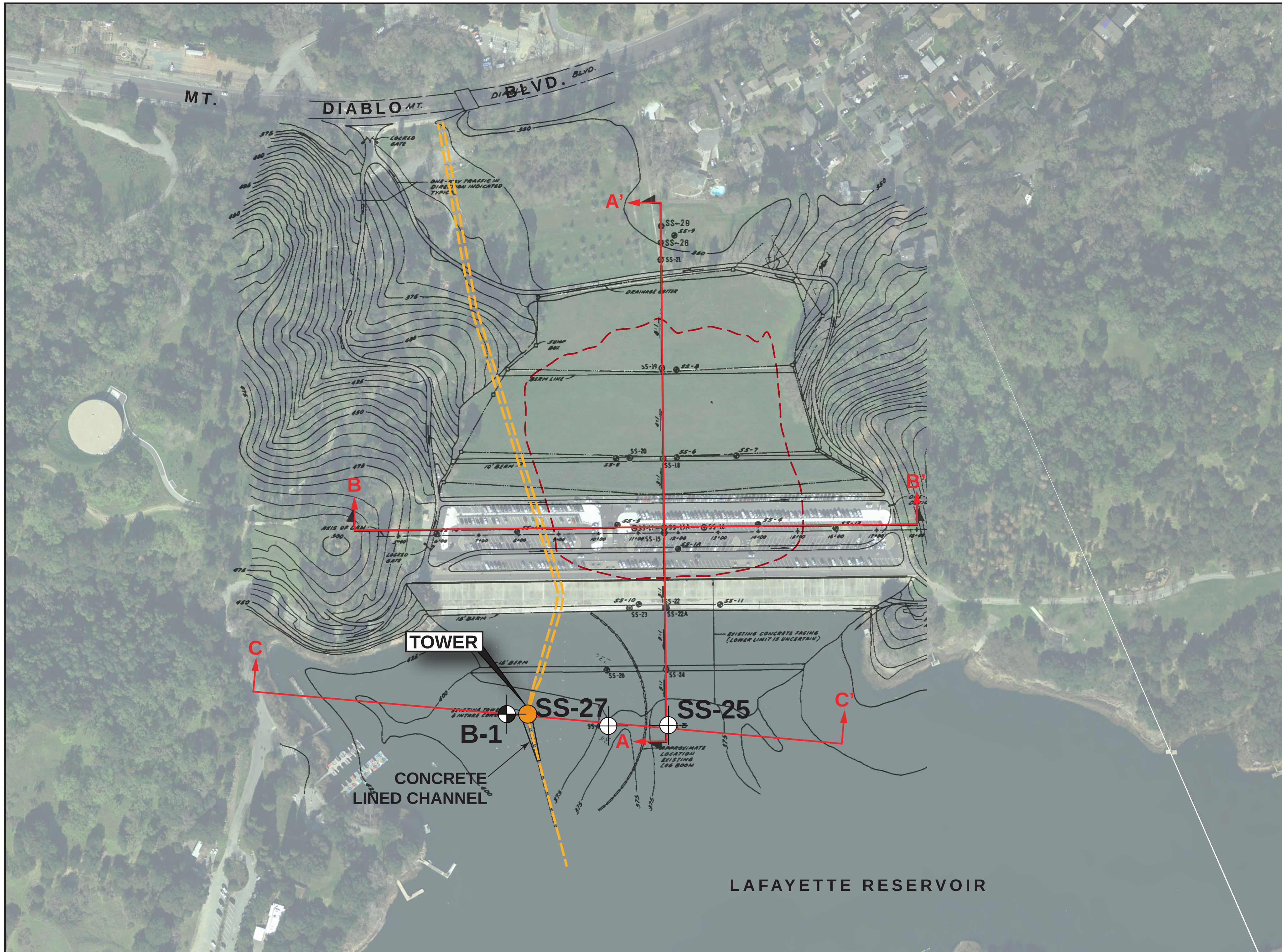
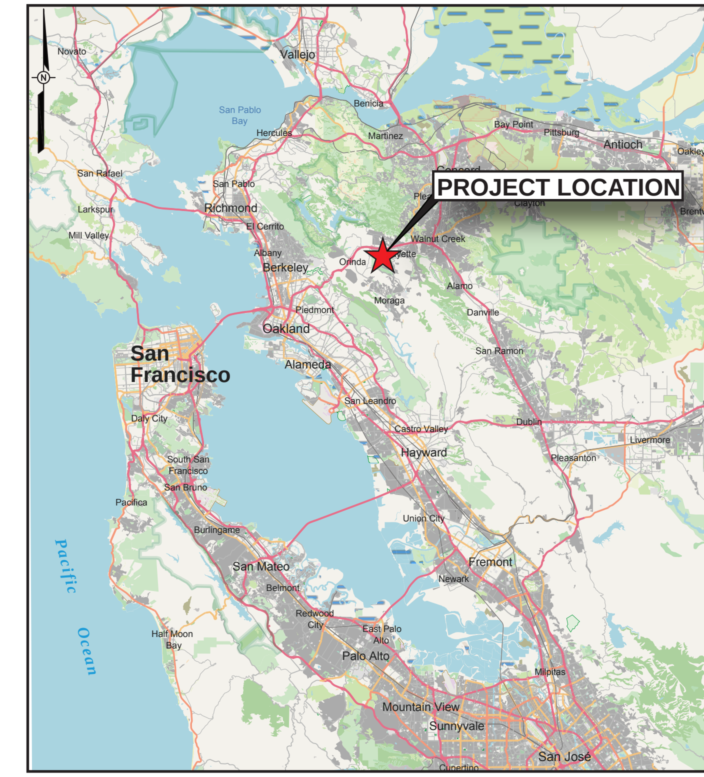
FIGURES



Base Map: USGS Aerial Photograph, 2015 & Topography from W.A. Wahler Associates, 1976.



LOCATION MAP

PHOTO LOOKING EAST

LEGEND:

B-1	APPROXIMATE LOCATION OF EXPLORATORY BORING DRILLED FOR THIS STUDY	C — C'	CROSS SECTION LOCATION
			APPROXIMATE LOCATION OF CONDUITS
SS-27	APPROXIMATE LOCATION OF EXPLORATORY BORING DRILLED BY WAHLER, 1973		APPROXIMATE LIMIT OF 1928 DOWNSTREAM SURFACE MOVEMENT

(SCALE IN FEET)

0 300 600

LAFAYETTE RESERVOIR
OUTLET TOWER RETROFIT
LAFAYETTE, CALIFORNIA

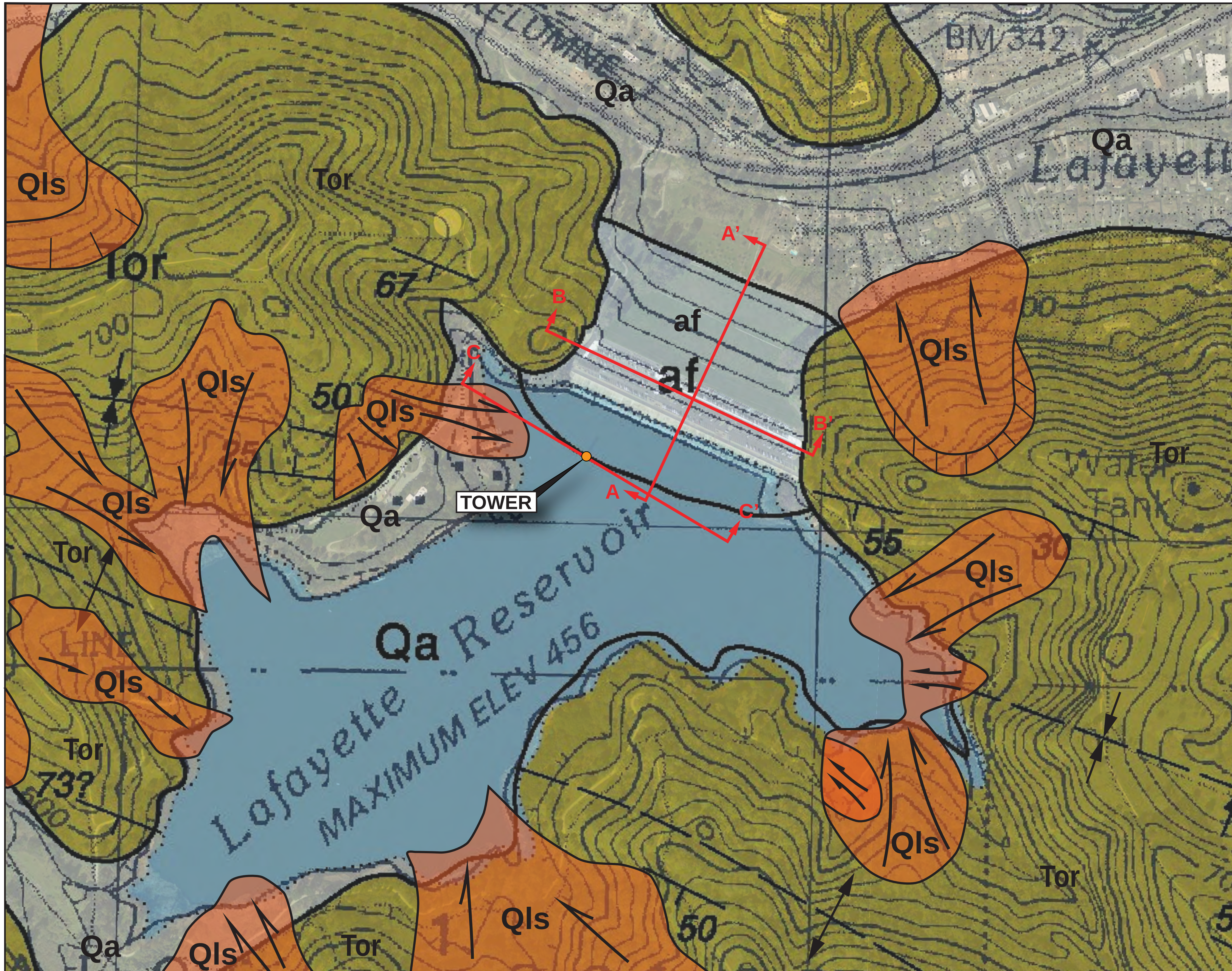
Project No. 1141-4A

SITE PLAN

A3GEO

FIGURE 1

Date: 10/31/2018



Base Maps: 1) USGS Site Aerial Photograph, 2015; 2) Dibblee, et al, 2005; 3) GEI, 2005.

Date: 111/9/2018

LEGEND:

SURFICIAL SEDIMENTS

af MAN MADE ARTIFICIAL FILL FOR DAM

Qls LANDSLIDE DEPOSIT (GEI, 2005)

Qa ALLUVIAL GRAVEL, SAND AND CLAY OF VALLEY AREAS

ORINDA FORMATION

Tor TERRESTRIAL PEBBLE CONGLOMERATE OF FRANCISCAN DETRITUS, SANDSTONE AND CLAYSTONE INTERBEDDED, GRAY TO GREENISH GRAY

MAP SYMBOLS

GEOLOGIC CONTACT

APPROXIMATE LOCATION OF MAPPED LANDSLIDE (GEI, 2005)

C C' CROSS SECTION LOCATION

N

(SCALE IN FEET)

0 500 1,000

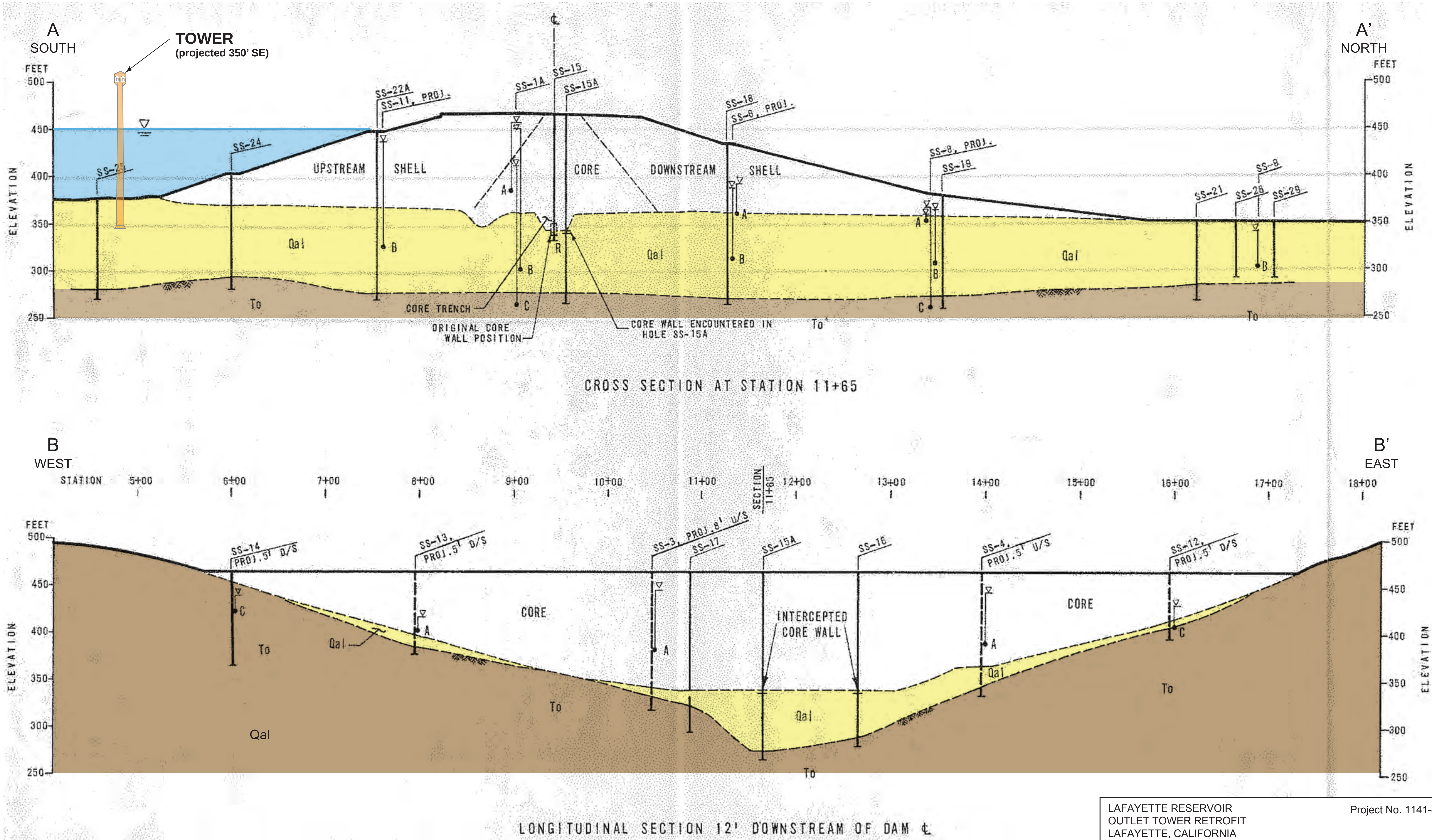
LAFAYETTE RESERVOIR
OUTLET TOWER RETROFIT
LAFAYETTE, CALIFORNIA

Project No. 1141-4A

REGIONAL GEOLOGIC MAP

A3GEO

FIGURE 2



LEGEND:

Qal	Alluvium
To	Orinda Formation

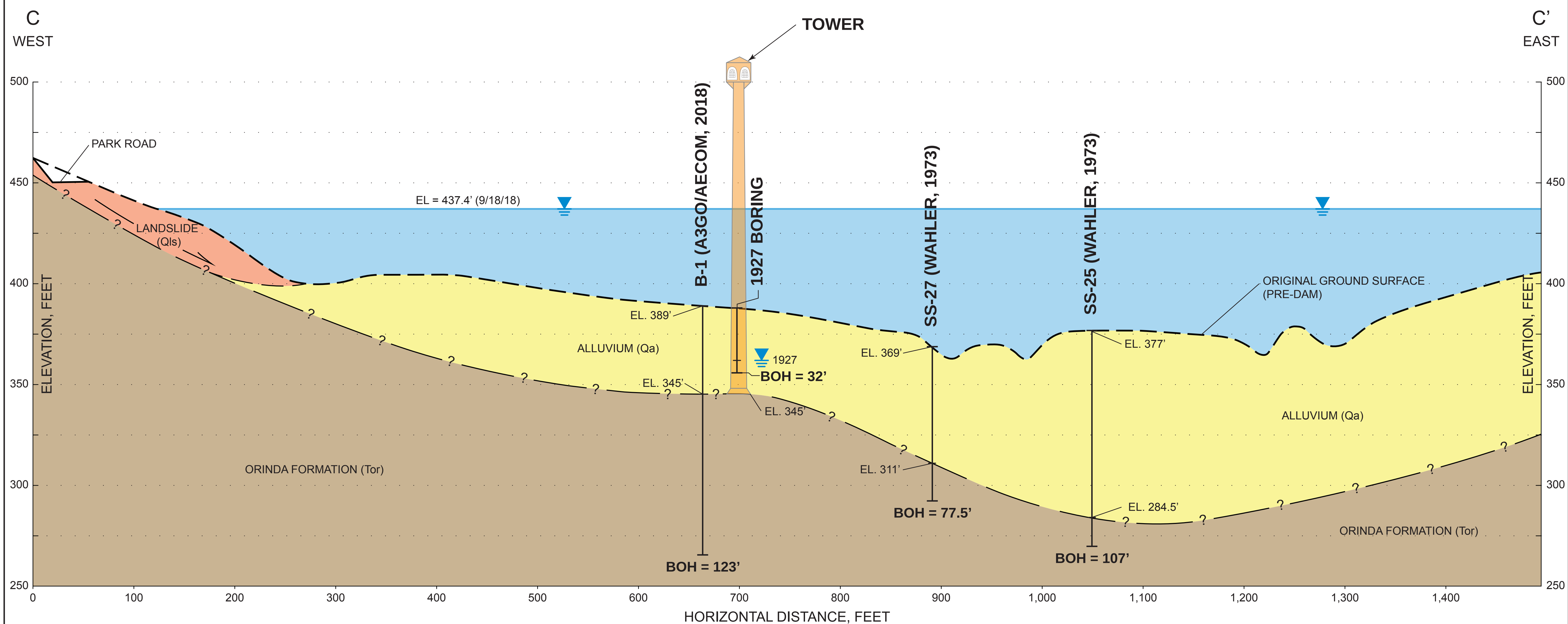
Date: 11/8/2018

LAFAYETTE RESERVOIR
OUTLET TOWER RETROFIT
LAFAYETTE, CALIFORNIA

Project No. 1141-4A

GEOLOGIC CROSS SECTIONS A & B
W.A. WAHLER, 1976

FIGURE 3



1" = 100' Horizontal
1" = 50' Vertical
BOH = Bottom of Hole

LAFAYETTE RESERVOIR
OUTLET TOWER RETROFIT
LAFAYETTE, CALIFORNIA

Project No. 1141-4A

GEOLOGIC CROSS SECTION C - C'

A3GEO

FIGURE 4

Date: 11/9/2018

APPENDIX A

A3GEO Boring Log

A3GEO

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS: more than 50% retained on No. 200 sieve	COARSE GRAINED SOILS: 50% or more of coarse fraction on No. 4 sieve	CLEAN GRAVELS	GW	Well graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH SAND	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	SANDS: more than 50% passing on No. 4 sieve	CLEAN SANDS	SW	Well graded sands and gravelly sand, little or no fines
			SP	Poorly graded sands and gravelly sand, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS: 50% or more passing No. 200 sieve	SILTS AND CLAY: Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAY: Liquid Limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic clays
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat, muck, and other highly organic soils

BOUNDARY CLASSIFICATION AND GRAIN SIZES

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

U.S. Standard Sieve Sizes: No. 200 (0.075 mm); No. 40 (0.425 mm); No. 10 (2 mm); No. 4 (3/16"); 3/4"; 3"; 12"

SYMBOLS

Modified California (MC) Sampler (3" O.D.)	HQ ROCK CORE (HQ)	No Recovery (NR)
Standard Penetration Test: SPT (2" O.D.)	Pitcher Tube (PT) ShelbyTube (ST)	Water Levels At time of drilling At end of drilling After drilling

ABBREVIATIONS

Item	Meaning
LL	Liquid Limit (%) (ASTM D 4318)
PI	Plasticity Index (%) (ASTM D 4318)
-200	Passing No. 200 (%) (ASTM D 1140)
TXICU	Laboratory consolidated undrained triaxial test of undrained shear strength (psf) (ASTM D 4767)
UC	Laboratory unconfined compression test (ASTM D 2166)
psf/tsf	pounds per square foot / tons per square foot
psi	pounds per square inch
OD	Outside Diameter
RQD	Rock-quality designation

NOTES

1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Recorded blow counts have not been adjusted for hammer energy.

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BEDDING OF SEDIMENTARY ROCK		
SPLITTING PROPERTY	THICKNESS	STRATIFICATION
Massive	Greater than 4.0 feet	Very Thick-Bedded
Blocky	2.0 to 4.0 feet	Thick-Bedded
Slabby	0.2 to 2.0 feet	Thin-Bedded
Flaggy	0.05 to 0.2 feet	Very Thin-Bedded
Shaly or Platy	0.01 to 0.05 feet	Laminated
Papery	Less than 0.01 feet	Thinly Laminated

FRACTURING	
INTENSITY	SIZE OF PIECES IN FEET
Very Little Fractured	Greater than 4.0 feet
Occasionally Fractured	1.0 to 4.0 feet
Moderately Fractured	0.5 to 1.0 feet
Closely Fractured	0.1 to 0.5 feet
Intensely Fractured	0.05 to 0.1 feet
Crushed	Less than 0.05 feet

HARDNESS	
Soft	Reserved for plastic material alone
Low Hardness	Can be gouged deeply or carved easily by a knife blade
Moderately Hard	Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away
Hard	Can be scratched by a knife blade with difficulty; scratch produces little powder and is often faintly visible
Very Hard	Cannot be scratched by a knife blade; leaves a metallic streak

STRENGTH	
Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen of such material will crumble under light hammer blows
Moderately Strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments
Very Strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

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	
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.
Moderate	Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little	No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
Fresh	Unaffected by weathering agents. No discoloration or disintegration. Fractures usually less numerous than joints.

KEY TO ROCK DESCRIPTIONS

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BORING NUMBER B-1

CLIENT AECOM/EBMUD
PROJECT NAME Lafayette Outlet Tower Seismic Retrofit
PROJECT NUMBER 1141-4A
PROJECT LOCATION Lafayette, CA
DATE STARTED 9/18/18 **COMPLETED** 9/19/18
GROUND ELEVATION 389 ft **HOLE SIZE** 5.5 inches
DRILLING CONTRACTOR Taber Drilling
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
AT TIME OF DRILLING --- Borehole located under reservoir water
LOGGED BY SK **CHECKED BY** DM
AT END OF DRILLING --- Borehole located under reservoir water
NOTES Boring was drilled using barge-mounted CME 45 rig
AFTER DRILLING --- Borehole located under reservoir water

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
0	Approximate Mud-line at Elev. 389', Auger sank for two feet							
	FAT CLAY (CH) - brown to grayish brown, soft to medium stiff, few silt pockets, trace rounded fine gravel, high plasticity, wet. [ALLUVIUM]	MC	8			36	67	Sample collected in a bag
	becomes medium stiff							
5		ST		2.0	101	24		
	LEAN CLAY WITH SAND (CL) - brown to grayish brown, stiff, some silt pockets, predominantly fine sand with trace coarse sand, medium plasticity, wet	SPT	25			21	89	LL=47, PI=30
10	**SANDY SILTY CLAY (CL-ML)** - brownish gray, stiff to very stiff, fine to coarse sand, medium to high plasticity, wet							
		PT			108	20		TXICU LL=53, PI=36
	FAT CLAY WITH SAND (CH) - yellowish brown, stiff to very stiff, predominantly fine sand with trace coarse sand, trace rounded gravel, trace orangish oxidation, medium to high plasticity, wet	SPT	22			25	94	
15								
	SANDY LEAN CLAY (CL) - grayish olive brown, stiff, fine to coarse sand, medium plasticity, wet	PT		1.7	106	22		TXICU
	trace coarse gravel							
20	predominantly fine sand with trace coarse sand	SPT	16			24	89	-#200=62% LL=43, PI=27
	SANDY LEAN CLAY (CL) to CLAYEY SAND (SC) - brown, loose to medium dense, fine to coarse sand, wet	NR				28		Disturbed sample collected with MC catcher.
25		SPT	11				78	

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DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
25	**LEAN CLAY WITH SAND (CL)** - brown to yellowish brown, stiff, predominantly fine sand with trace coarse sand, trace fine rounded gravel, trace black and orange oxidation, low to medium plasticity, wet*(continued)*					24		
		PT		1.8	106	21		TXICU LL=44, PI=26
30	**SANDY LEAN CLAY (CL)** - brown to yellowish brown, stiff, fine sand, trace to few fine rounded gravel	SPT	19			21	100	
35	**CLAYEY SAND (SC)** - olive to yellowish brown, medium dense, fine to coarse sand, few fine subrounded gravel up to ¾-inch in diameter, low plasticity fines, wet interbedded layers of Sandy Silty Clay below depth of 34'	MC	16			22	0	Disturbed sample collected with MC catcher. Gravel: 9% Sand: 43% -#200: 48% LL=38, PI=22
		MC	11				0	No Recovery
40	**SANDY LEAN CLAY (CL)** to **CLAYEY SAND (SC)** - grayish olive brown, stiff to medium dense, fine to coarse sand, few fine subrounded gravel, some silt pockets, few orangish oxidation, low plasticity fines	MC	14	1.25	107	21	100	MC catcher used initially Gravel: 5% Sand: 45% -#200: 50% LL=34, PI=17
	SANDY LEAN CLAY (CL) - light olive brown, hard, some gray silt pockets, medium plasticity, some orange oxidation, wet	SPT	41			21	89	
45	transferring to **CLAYSTONE** - deeply weathered, light brown, low hardness, some fine sand, rounded fine to coarse gravel, trace silt pockets, some orange to orangish light brown oxidation. [ORINDA FORMATION]							
50	**CLAYSTONE** - light brown with yellowish brown to orange oxidation, low hardness, weak to moderately strong, deep weathering, occasionally fractured	HQ					100 (95)	RQD soundness requirements have not been met

(Continued Next Page)

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BORING NUMBER B-1

CLIENT	AECOM/EBMUD	**PROJECT NAME**	Lafayette Outlet Tower Seismic Retrofit
PROJECT NUMBER	1141-4A	**PROJECT LOCATION**	Lafayette, CA
DATE STARTED	9/18/18 **COMPLETED** 9/19/18	**GROUND ELEVATION**	389 ft **HOLE SIZE** 5.5 inches
DRILLING CONTRACTOR	Taber Drilling	**GROUND WATER LEVELS:**	
DRILLING METHOD	Rotary Wash Drilling	**AT TIME OF DRILLING**	--- Borehole located under reservoir water
LOGGED BY	SK **CHECKED BY** DM	**AT END OF DRILLING**	--- Borehole located under reservoir water
NOTES	Boring was drilled using barge-mounted CME 45 rig	**AFTER DRILLING**	--- Borehole located under reservoir water

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
50		**CLAYSTONE** - light brown with yellowish brown to orange oxidation, low hardness, weak to moderately strong, deep weathering, occasionally fractured *(continued)*	HQ			118	15	93 (80)	RQD soundness requirements have not been met UC=71.4 psi @ 51.5'
		moderately fractured							
55		**SANDSTONE** - light brown, low to moderately hard, friable, deep weathering, closely fractured	HQ					90 (75)	RQD soundness requirements have not been met
		SILTSTONE - gray, low hardness, friable to weak, deep weathering, occasionally to moderately fractured, interbedded thin layers of sandstone							
		CLAYSTONE - light brown, low hardness, moderately strong, deep weathering, occasionally to moderately fractured							
60		**SILTSTONE** - light gray, low hardness, moderately strong, deep to moderate weathering, occasionally to moderately fractured	HQ			118	15	100 (68)	RQD soundness requirements have not been met UC=47.6 psi @ 60.5'
		SANDSTONE - gray, fine-grained, low hardness, weak, deep weathering, closely fractured							
		CLAYSTONE/SILTSTONE - light brownish gray with orangish oxidation, low hardness, weak, deep to moderate weathering, occasionally fractured							
65		becomes yellowish brown, deep weathering, pocket of fine to coarse sand at depth of 64.5' to 65'	HQ					90 (80)	RQD soundness requirements have not been met
		SILTSTONE - gray to light gray, low to moderately hard, moderately strong, moderate weathering, closely fractured							
		little weathering, very little to occasionally fractured							
70			HQ			124	13	100 (100)	UC=25.1 psi @ 69.7'
		moderately hard, little to fresh weathering, occasionally fractured							
75		**SANDSTONE** - gray, very fine-grained, moderately hard, moderately strong, little to fresh weathering, occasionally fractured	HQ					61 (54)	

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DRILLING CONTRACTOR Taber Drilling
GROUND WATER LEVELS:
DRILLING METHOD Rotary Wash Drilling
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DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
75		**SANDSTONE** - gray, very fine-grained, moderately hard, moderately strong, little to fresh weathering, occasionally fractured *(continued)*							No Recovery at depth of 75 to 76.5 feet. Disturbed-broken sample collected in the next core run
		CLAYSTONE - gray, low to moderately hard, moderately strong, little to fresh weathering, moderately to closely fractured	HQ					100 (54)	
		occasionally fractured							
80		**SANDSTONE** - gray and white, fine-grained, hard, moderately strong, little weathering, occasionally fractured	HQ					94 (79)	
		SILTSTONE - gray, low to moderately hard, moderately strong, little weathering, occasionally fractured				117	15		UC=37.6 psi @ 82'
85		**SANDSTONE** - gray and white with trace orange oxidation, fine grained, moderately hard, moderately strong, little weathering, closely fractured at depth of 84.5 to 85.5 occasionally fractured	HQ					100 (85)	
		fine to medium grained, trace orange oxidation, moderately hard to hard, moderately strong				131	7		UC=158 psi @ 87.5'
90			HQ					100 (94)	
		low hardness, very little to occasionally fractured							
95			HQ					100 (100)	
		moderately hard							
		low hardness, weak to moderately strong							
100								100	

(Continued Next Page)

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AFTER DRILLING --- Borehole located under reservoir water

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	RECOVERY % (RQD)	OTHER LAB TESTS / NOTES
100		**SANDSTONE** - gray and white with trace orange oxidation, fine grained, moderately hard, moderately strong, little weathering, closely fractured at depth of 84.5 to 85.5*(continued)*	HQ					(93)	
						134	10		UC=18.1 psi @ 102'
		moderately hard, moderately strong							
105			HQ					90 (83)	
		SILTSTONE - light gray, moderately hard, weak to moderately strong, little weathering, occasionally to moderately fractured, trace orange oxidation on surface moderately strong, little to fresh weathering, moderately to closely fractured low to moderately hard							
110			HQ					95 (37)	
		SANDSTONE - gray to light gray, very fine-grained, moderately hard, moderately strong, little to fresh weathering, moderately fractured							
		SILTSTONE - light gray to gray, moderately hard to hard, moderately strong, little to fresh weathering, occasionally to moderately fractured							
		hard, weak to moderately strong, fresh weathering							
115		intensely fractured at depth of 114.5' to 115.5' moderately hard, moderately strong, moderately fractured	HQ					100 (83)	
						117	15		UC=17.3 psi @ 116'
		occasionally fractured							
120			HQ					100 (64)	
		closely fractured							

Bottom of borehole at 123.0 feet.
Water elevation at reservoir was recorded as ~437.4 ft. Water depth was measured as ~48.4 ft.
Average SPT/MC hammer efficiency = 80%.

APPENDIX B

Rock Core Photos

Depth (from top of barge): 95.5 to 99 ft
Depth (from top of boring): 46.5 to 50 ft
Elevation: 342.5 to 339 ft

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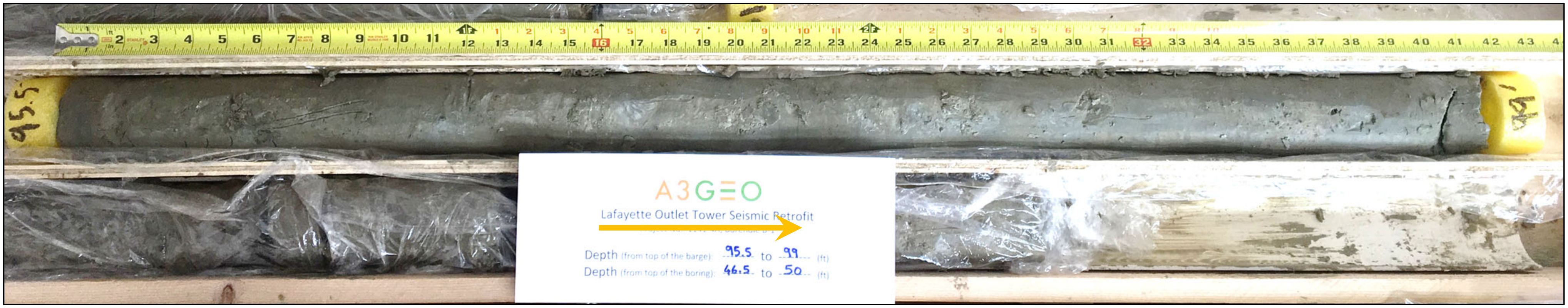
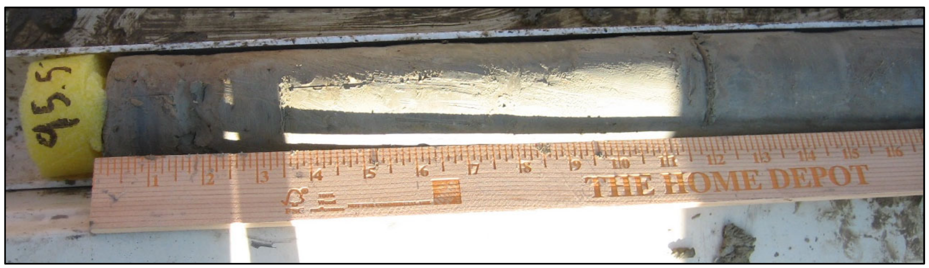
❖ **Core Photo Taken in A3GEO Laboratory**



❖ **Core Photos Taken On-Site**



Depth (from top of barge): 99 to 101.5 ft
Depth (from top of boring): 50 to 52.5 ft
Elevation: 339 to 336.5 ft

❖ Core Photo Taken in A3GEO Laboratory

❖ Core Photos Taken On-Site

Depth (from top of barge): 101.5 to 106.5 ft
Depth (from top of boring): 52.5 to 57.5 ft
Elevation: 336.5 to 331.5 ft

❖ Core Photo Taken in A3GEO Laboratory

❖ Core Photos Taken On-Site

Depth (from top of barge): 106.5 to 111.5 ft
Depth (from top of boring): 57.5 to 62.5 ft
Elevation: 331.5 to 326.5 ft

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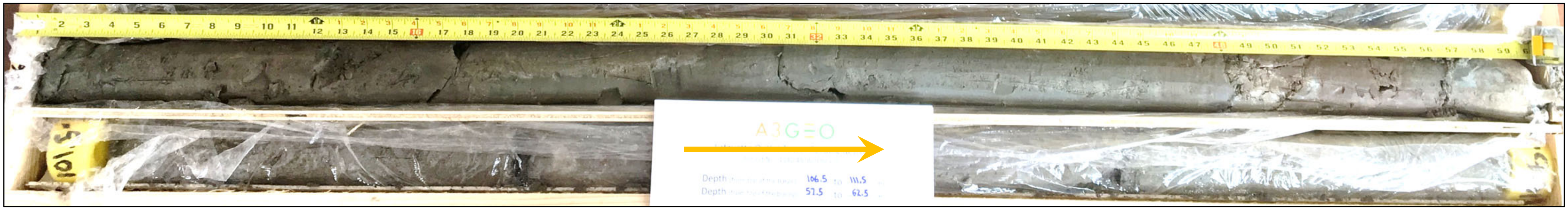
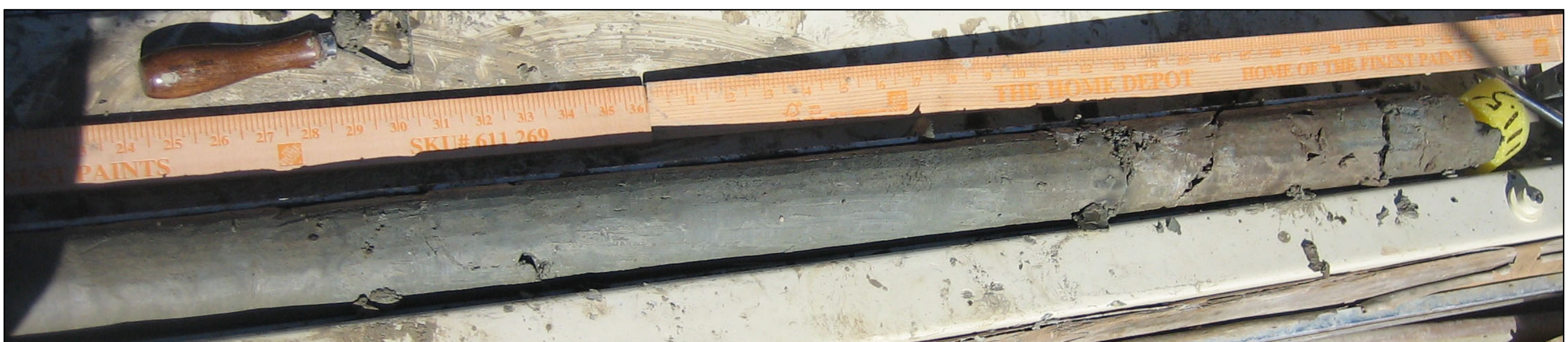
- **Core Photo Taken in A3GEO Laboratory**

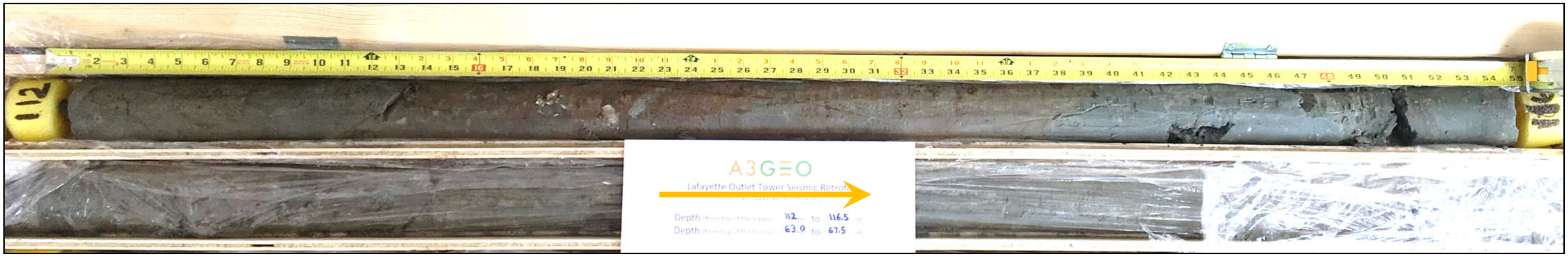


- **Core Photos Taken On-Site**



Depth (from top of barge): 112 to 116.5 ft
Depth (from top of boring): 63 to 67.5 ft
Elevation: 326 to 321.5 ft

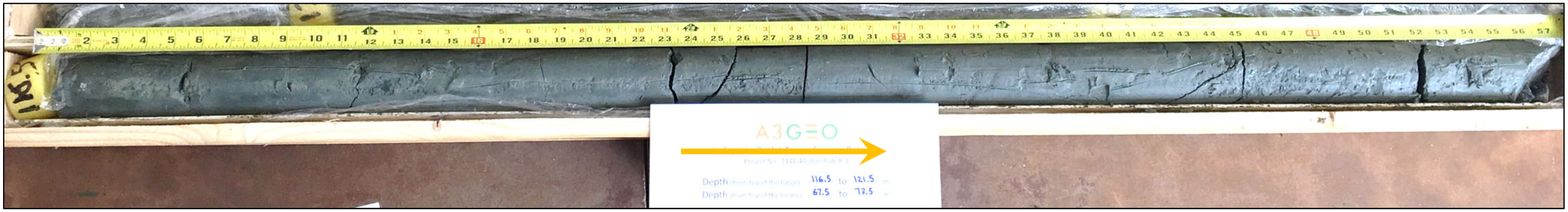
❖ Core Photo Taken in A3GEO Laboratory



❖ Core Photos Taken On-Site

Depth (from top of barge): 116.5 to 121.5 ft
Depth (from top of boring): 67.5 to 72.5 ft
Elevation: 321.5 to 316.5 ft

- **Core Photo Taken in A3GEO Laboratory**



- **Core Photos Taken On-Site**

Depth (from top of barge): 121.5 to 124 ft
Depth (from top of boring): 72.5 to 75 ft
Elevation: 316.5 to 314 ft

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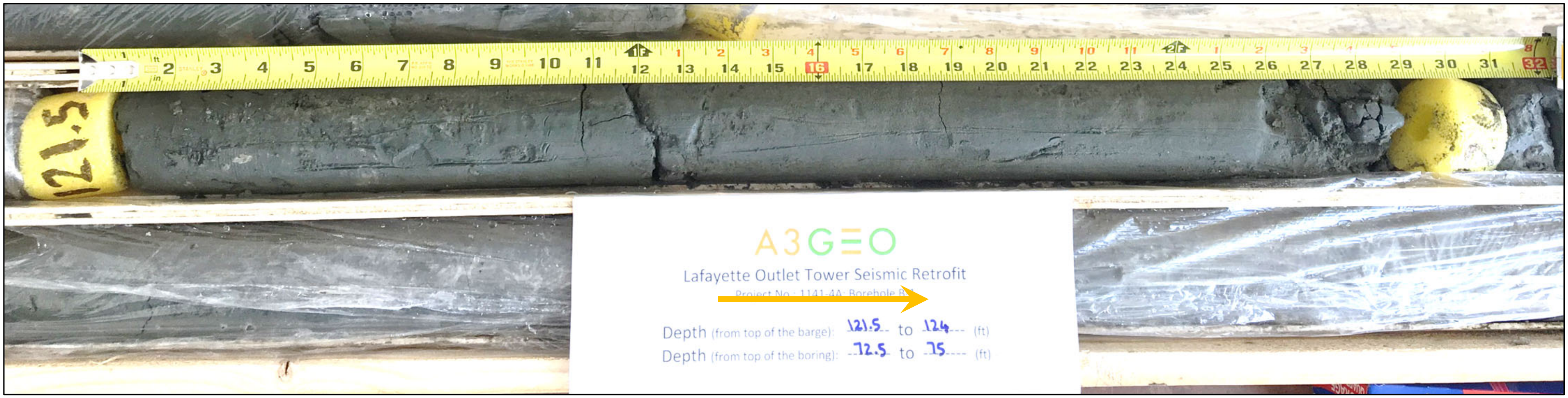
- **Core Photo Taken in A3GEO Laboratory**



- **Core Photos Taken On-Site**

Depth (from top of barge): 125.5 to 127.5 ft
Depth (from top of boring): 76.5 to 78.5 ft
Elevation: 312.5 to 310.5 ft

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❖ Core Photo Taken in A3GEO Laboratory

❖ Core Photos Taken On-Site

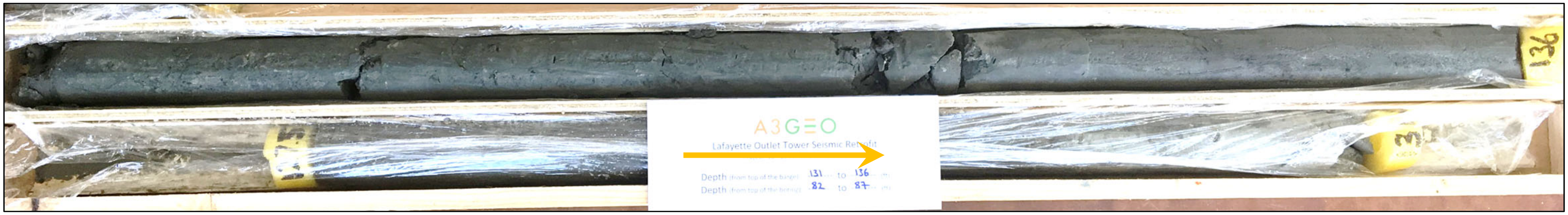
Depth (from top of barge): 127.5 to 131 ft
Depth (from top of boring): 78.5 to 82 ft
Elevation: 310.5 to 307 ft

❖ **Core Photo Taken in A3GEO Laboratory**

❖ **Core Photos Taken On-Site**

Depth (from top of barge): 131 to 136 ft
Depth (from top of boring): 82 to 87 ft
Elevation: 307 to 302 ft

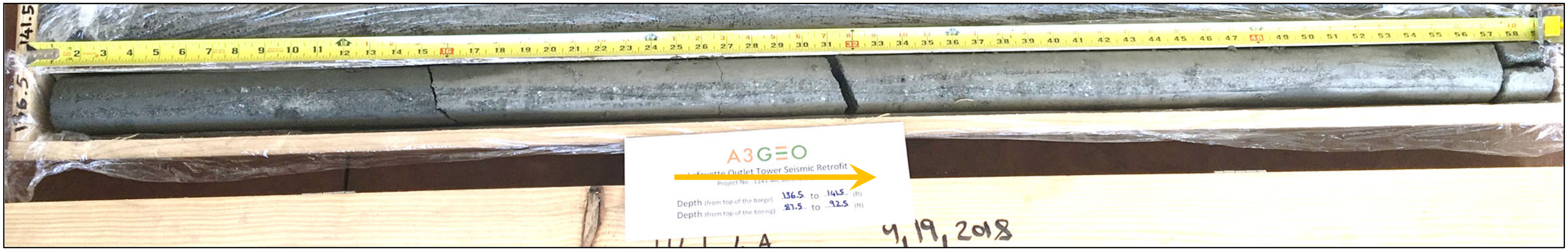
- **Core Photo Taken in A3GEO Laboratory**



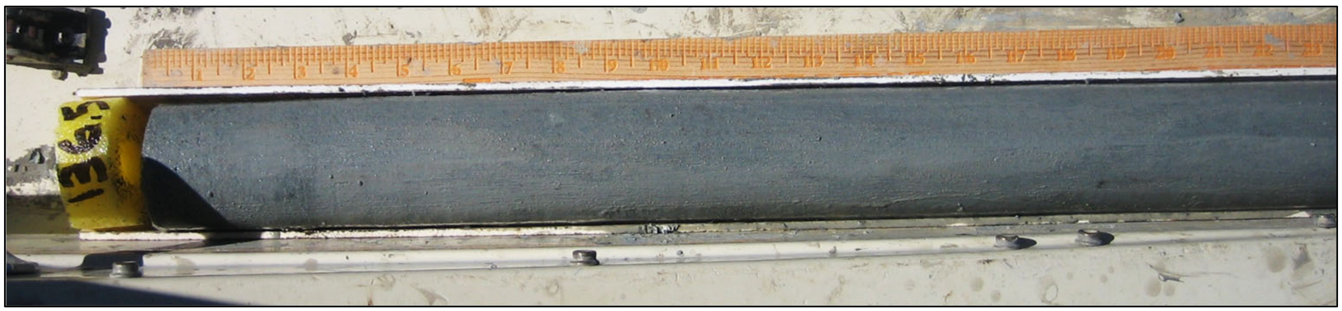
- **Core Photos Taken On-Site**

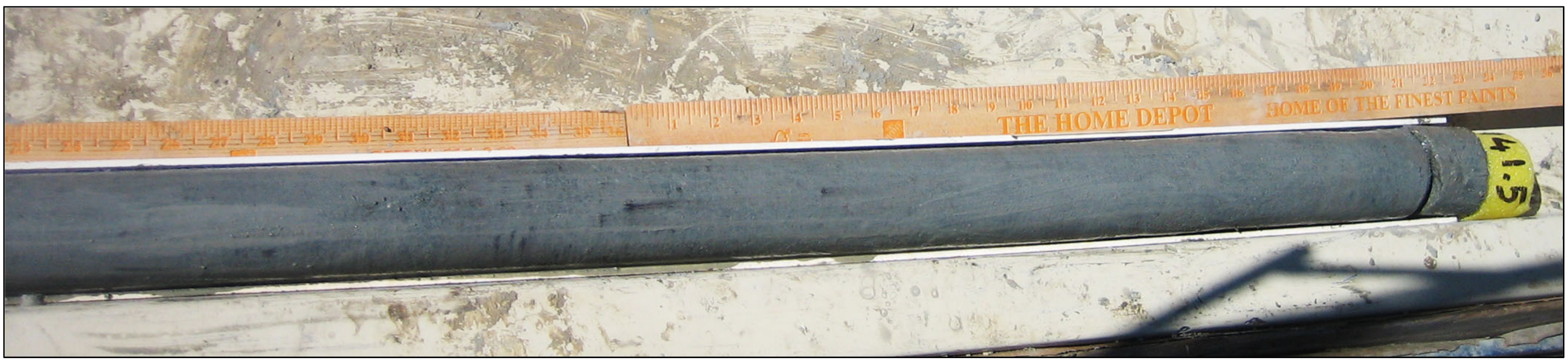
Depth (from top of barge): 136.5 to 141.5 ft
Depth (from top of boring): 87.5 to 92.5 ft
Elevation: 301.5 to 296.5 ft

- **Core Photo Taken in A3GEO Laboratory**



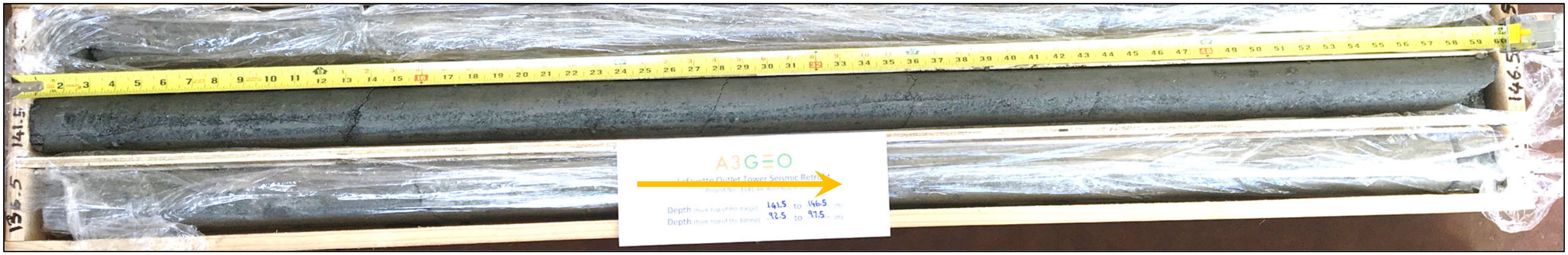
- **Core Photos Taken On-Site**





Depth (from top of barge): 141.5 to 146.5 ft
Depth (from top of boring): 92.5 to 97.5 ft
Elevation: 296.5 to 291.5 ft

- **Core Photo Taken in A3GEO Laboratory**



- **Core Photos Taken On-Site**

Depth (from top of barge): 146.5 to 151.5 ft
Depth (from top of boring): 97.5 to 102.5 ft
Elevation: 291.5 to 286.5 ft

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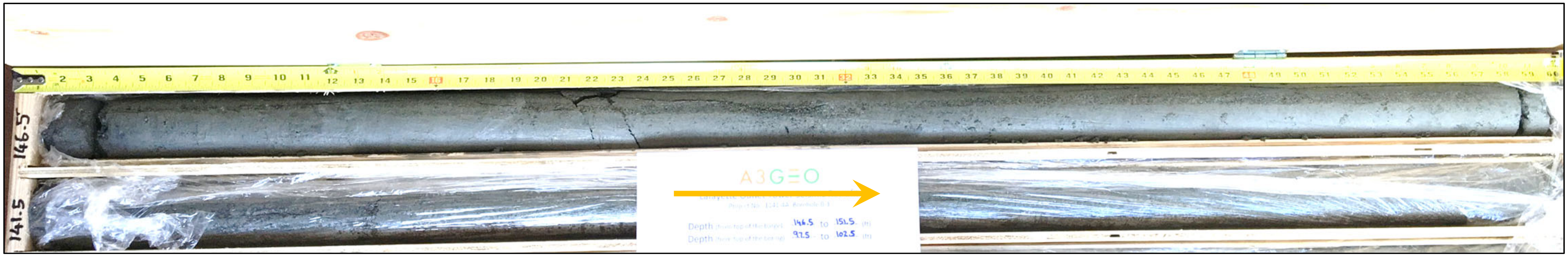
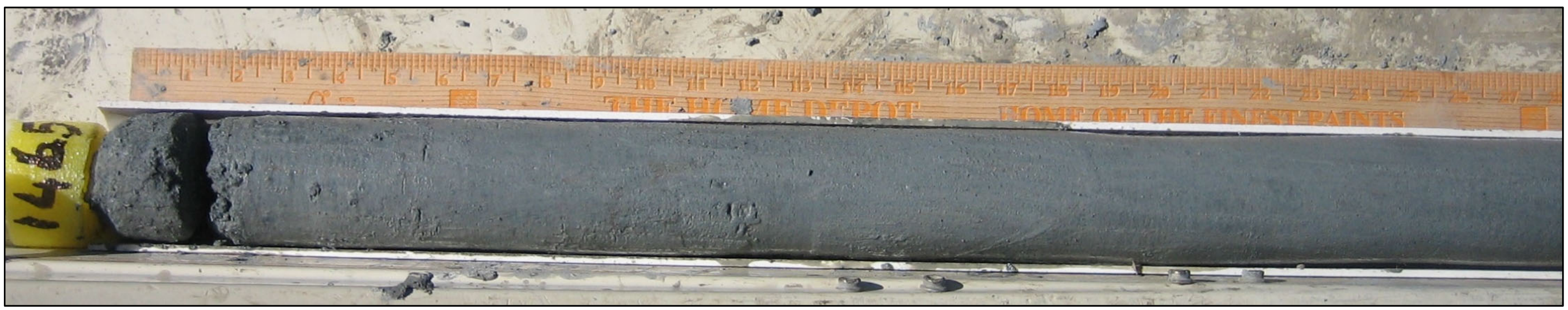
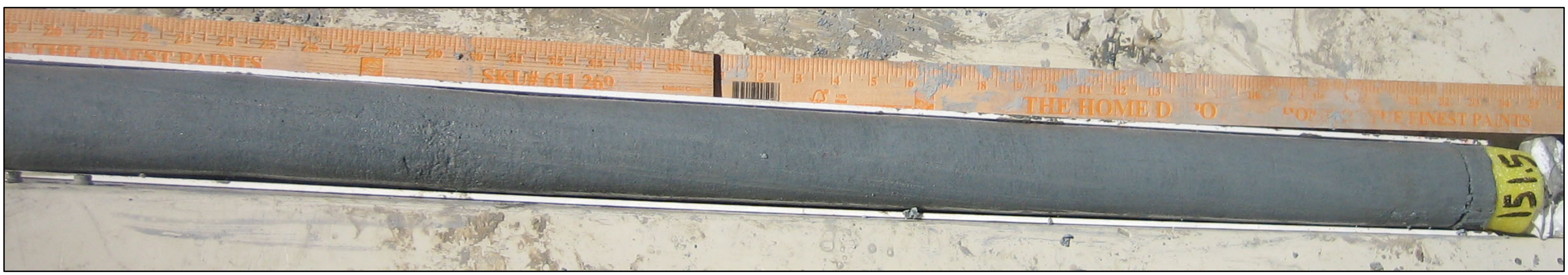
❖ **Core Photo Taken in A3GEO Laboratory**



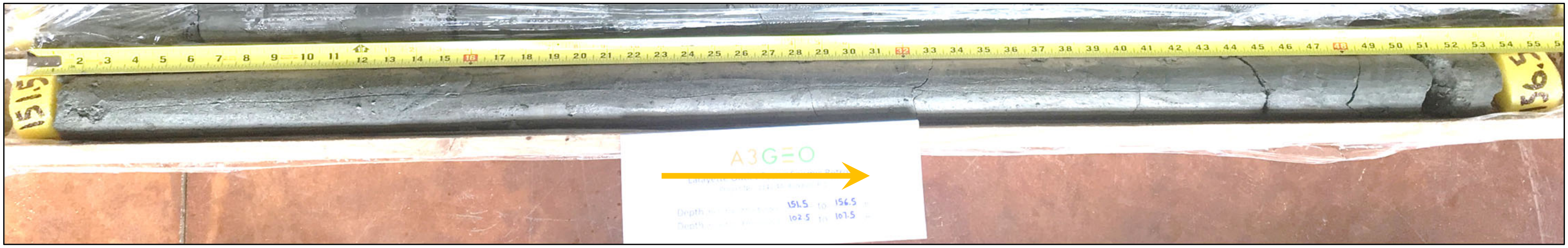
❖ **Core Photos Taken On-Site**



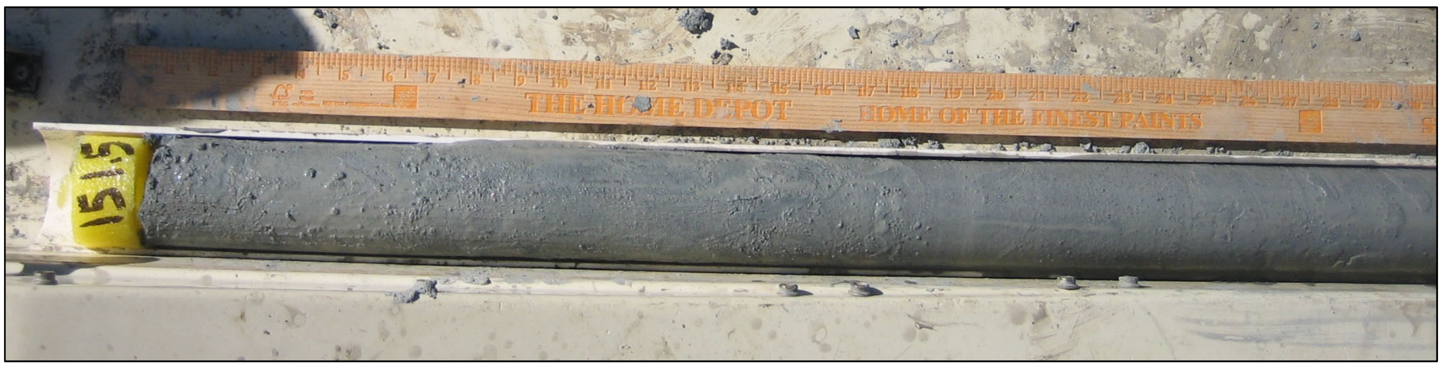


Depth (from top of barge): 151.5 to 156.5 ft
Depth (from top of boring): 102.5 to 107.5 ft
Elevation: 286.5 to 281.5 ft

- **Core Photo Taken in A3GEO Laboratory**

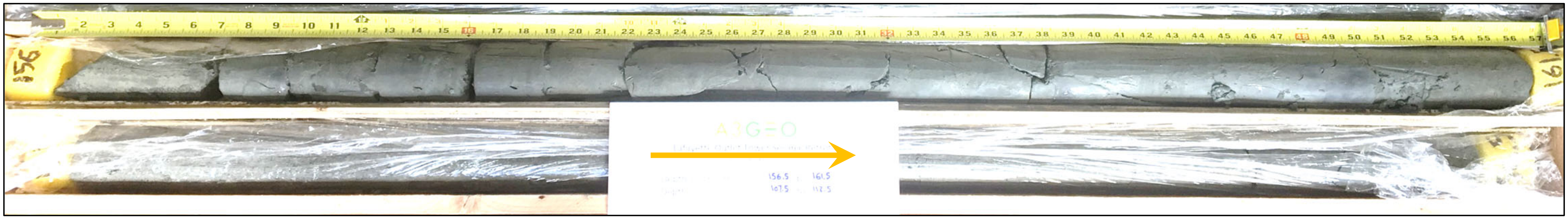


- **Core Photos Taken On-Site**



Depth (from top of barge): 156.5 to 161.5 ft
Depth (from top of boring): 107.5 to 112.5 ft
Elevation: 281.5 to 276.5 ft

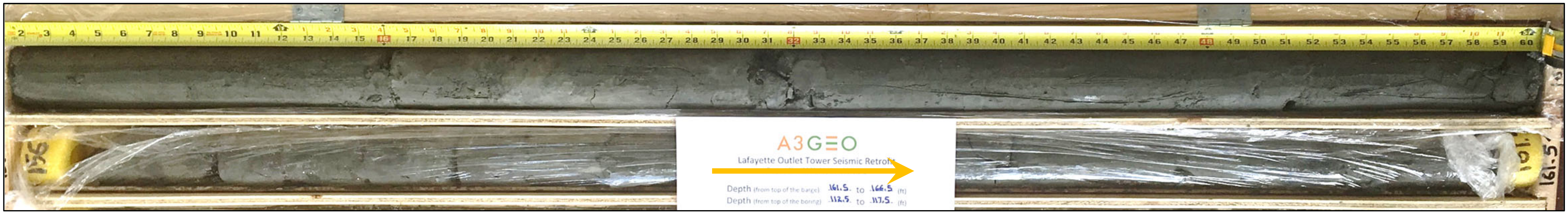
- **Core Photo Taken in A3GEO Laboratory**



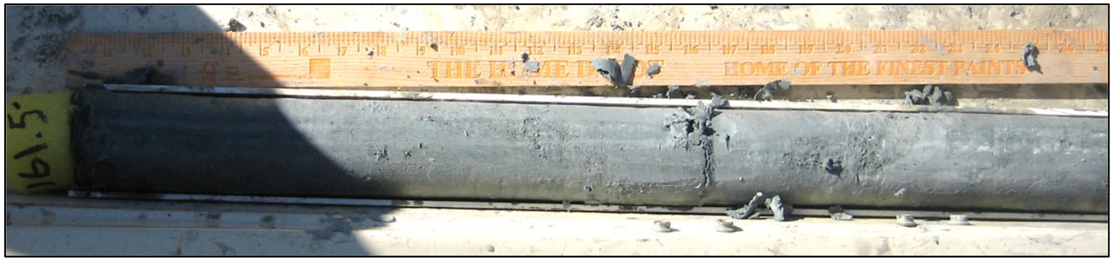
- **Core Photos Taken On-Site**

Depth (from top of barge): 161.5 to 166.5 ft
Depth (from top of boring): 112.5 to 117.5 ft
Elevation: 276.5 to 271.5 ft

- **Core Photo Taken in A3GEO Laboratory**



- **Core Photos Taken On-Site**



Depth (from top of barge): 166.5 to 172 ft
Depth (from top of boring): 117.5 to 123 ft
Elevation: 271.5 to 266 ft

- **Core Photo Taken in A3GEO Laboratory**

- **Core Photos Taken On-Site**

APPENDIX C

NORCAL Geophysical Logging Investigation Report

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

October 8, 2018

A3GEO
1331 Seventh Street, Unit E
Berkeley, CA 94710

Subject: Borehole Geophysical Logging Investigation
Lafayette Reservoir
Lafayette, California

NORCAL Job No: NS185079

Attention: Sarah Khosravani, PE, Project Engineer

This report presents the findings of a PS-wave Suspension logging investigation performed by NORCAL Geophysical Consultants, Inc. for A3GEO at the Lafayette Reservoir located in Lafayette, California. The purpose of the investigation is to measure compressional (P-) and shear (S-) wave velocities of the alluvium and underlying bedrock. The logging was performed on September 20, 2018 in one site visit by NORCAL Professional Geophysicist William J. Henrich (PGp No. 893). Logistical support and safety information were provided onsite by Mr. Rob Speidel, a Geologist of A3GEO.

Lafayette Reservoir is impounded by an earth fill dam. The dam is operated by the East Bay Municipal Water District (EBMUD). The PS-wave suspension logging is in support of an on-going geotechnical study to assess foundation response to seismic motion at the dams' intake structure.

1.0 SCOPE OF WORK

The scope of work consisted of conducting a PS-wave logging survey in one geotechnical borehole labeled B-1. The borehole is located over water, as shown on Figure 1, Borehole Location Map. The scope of work included processing and interpreting the geophysical logging data and presenting our results in a written report.

2.0 BOREHOLE CONDITIONS

Borehole B-1 was advanced from a drill deck using a 4 7/8-inch diameter rotary tri-cone bit from the mudline (49-ft below drill deck) through the alluvium to 94-ft below drill deck (bdd). The drill method was switched over to HQ (4-inch diameter) diamond core at the alluvium-rock contact at 94-ft bdd deck and terminated at 172-ft bdd. The drill deck has an elevation of approximately

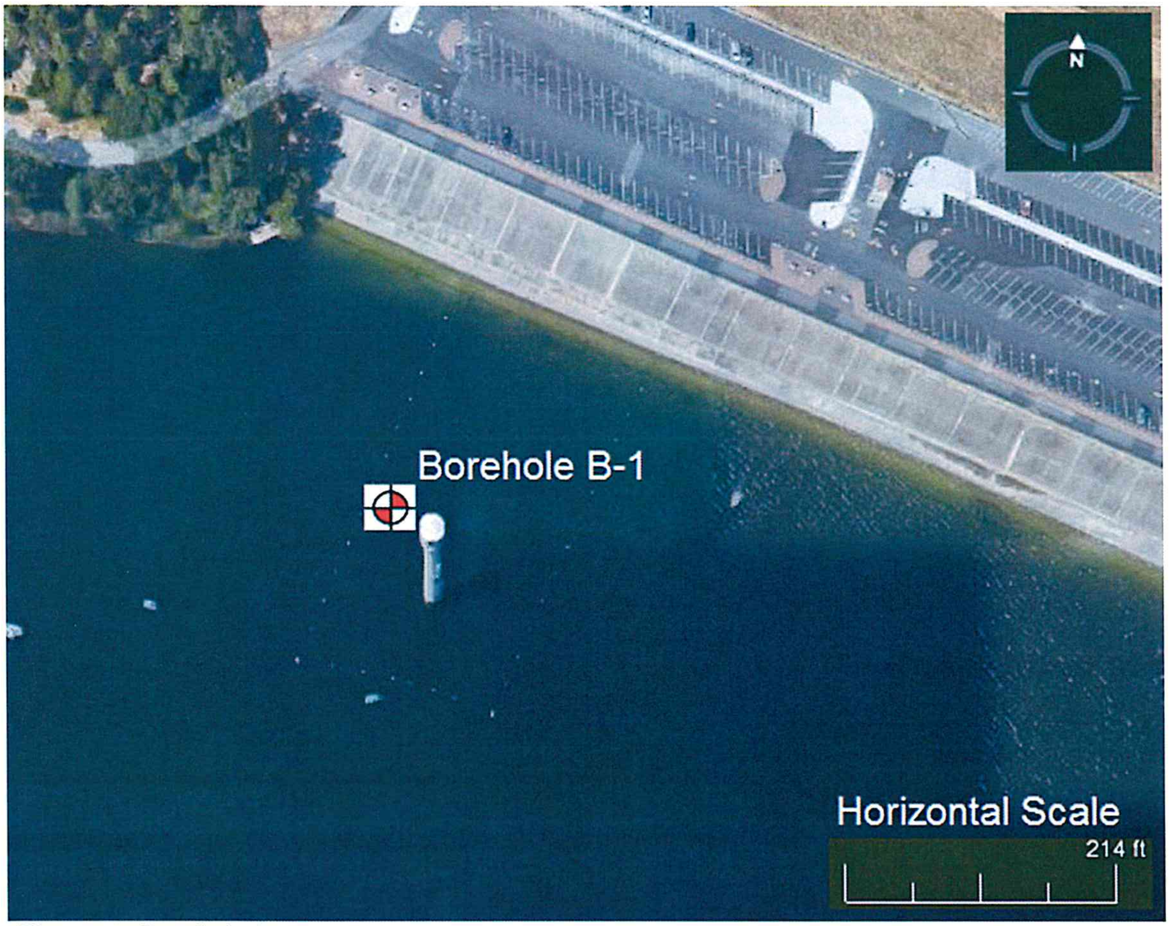


Figure 1. *Borehole Location Map*

438 feet above EBMUD Aqueduct Datum. The bedrock consisted of highly weathered, highly deformable plastic claystone belonging to the Orinda Formation. The bedrock did become more consolidated 50-ft below the top of rock. The stability of the borehole was very good with less than a foot of sediment accumulating at the bottom of the borehole prior to geophysical logging.

3.0 PS-WAVE SUSPENSION LOGGING INVESTIGATION

The PS-wave suspension investigation consisted of two components; 1) a borehole caliper and 2) the PS-wave Suspension Borehole Logging (SBL) survey. The primary objective of the caliper survey was to evaluate the overall condition of the hole prior to committing the much more expensive SBL equipment to the open hole. However, in the process, the caliper logging can provided valuable information on the relative consolidation of the alluvial and bedrock. The primary objective of the SBL survey was to measure the compressional (P-) wave and shear (S-) wave velocities of those materials.

Detailed descriptions of the logging instrumentation, methodology, our data acquisition and data analysis procedures and how the results are presented, are provided in Appendix A. The Appendix also includes a table listing the interval P- and S-wave velocities measured in each borehole.

4.0 RESULTS

4.1 BOREHOLE CALIPER SURVEYS

The results of the caliper logging conducted in Borehole B-1 is illustrated by the Borehole Diameter Graph shown on the right side of the respective velocity log plot on Plate 1. The Borehole Diameter Graph indicates borehole walls ranging in diameter from 4.25 to 5.5-inches. The diameters of approximately 4.25 inches in the lower borehole sections reflect the advance of HQ coring in bedrock. Borehole diameters in excess of 5-inches are the result of tri-cone drilling in the alluvium. Given the geologic materials, slight variations in diameters probably indicate the geologic materials are cohesive with high percentages of clay.

4.2 SUSPENSION BOREHOLE LOGGING SURVEY

The results of the suspension borehole logging (SBL) survey are illustrated on the PS-wave suspension velocity profile shown on Plate 1. The graph depicts the variations in S-wave velocity (Vs) and P-wave velocity (Vp) versus elevation. Vs are indicated by the red triangles and Vp are indicated by the blue squares. These data represent interval Vs and Vp 3-point moving averages (see Appendix A). The averaging was performed to smooth variations in the PS-wave velocities in overlapping interval profiles due to changes in acquisition parameters and borehole conditions (see Appendix A).

The seismic velocities measured in alluvium (~345 to 378-ft above EBMUD Aqueduct Datum) range from 720 to 920 feet per second (fps) for Vs and from 5,300 to 6100 fps for Vp. The seismic velocities measured in bedrock alluvium (~282.5 to 345-ft above EBMUD Aqueduct Datum) range from 850 to 1,970 fps for Vs and from 5,300 to 8,650 fps for Vp. The variations in Vs are a function of the relative rigidity of the geologic formations. In our experience, alluvial soils with Vs less 900 fps are low to medium density, soft to moderately stiff (clay), or loose to moderately consolidated (silt and mixtures). Bedrock with Vs less than 1200 fps generally represents highly weathered, plastic claystone in this case. At about 308-ft above EBMUD

Aqueduct Datum, the bedrock has a Vs of 1900 fps and a Vp of 8500 fps. These higher velocities represent more consolidated, less weathered bedrock, and occur 50-ft below the alluvium/bedrock contact suggesting the depth of substantial weathering is at least 50 feet at this location.

5.0 STANDARD CARE

The scope of NORCAL's services for this project consisted of using geophysical borehole logging methods to characterize the subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the standard of care ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate the opportunity to provide our services to A3GEO for this project. If you have any questions or require additional geophysical services, please do not hesitate to contact us.

Sincerely,

NORCAL Geophysical Consultants, Inc.

REGISTERED GEOPHYSICIST
WILLIAM J. HENRICH
10/08/2018
No. 893
STATE OF CALIFORNIA

William J Henrich
Professional Geophysicist PGp 893

Donald J. Kirker

PROFESSIONAL GEOPHYSICIST
DONALD KIRKER
No. 997
10-8-18
STATE OF CALIFORNIA

Donald J. Kirker
Professional Geophysicist PGp 997

WJH/DJK/tlt

Enclosures: Plate 1. B-1 PS-wave Suspension Velocity Profile with Caliper Log

Appendix A: PS-wave Suspension Borehole Logging (SBL) Survey

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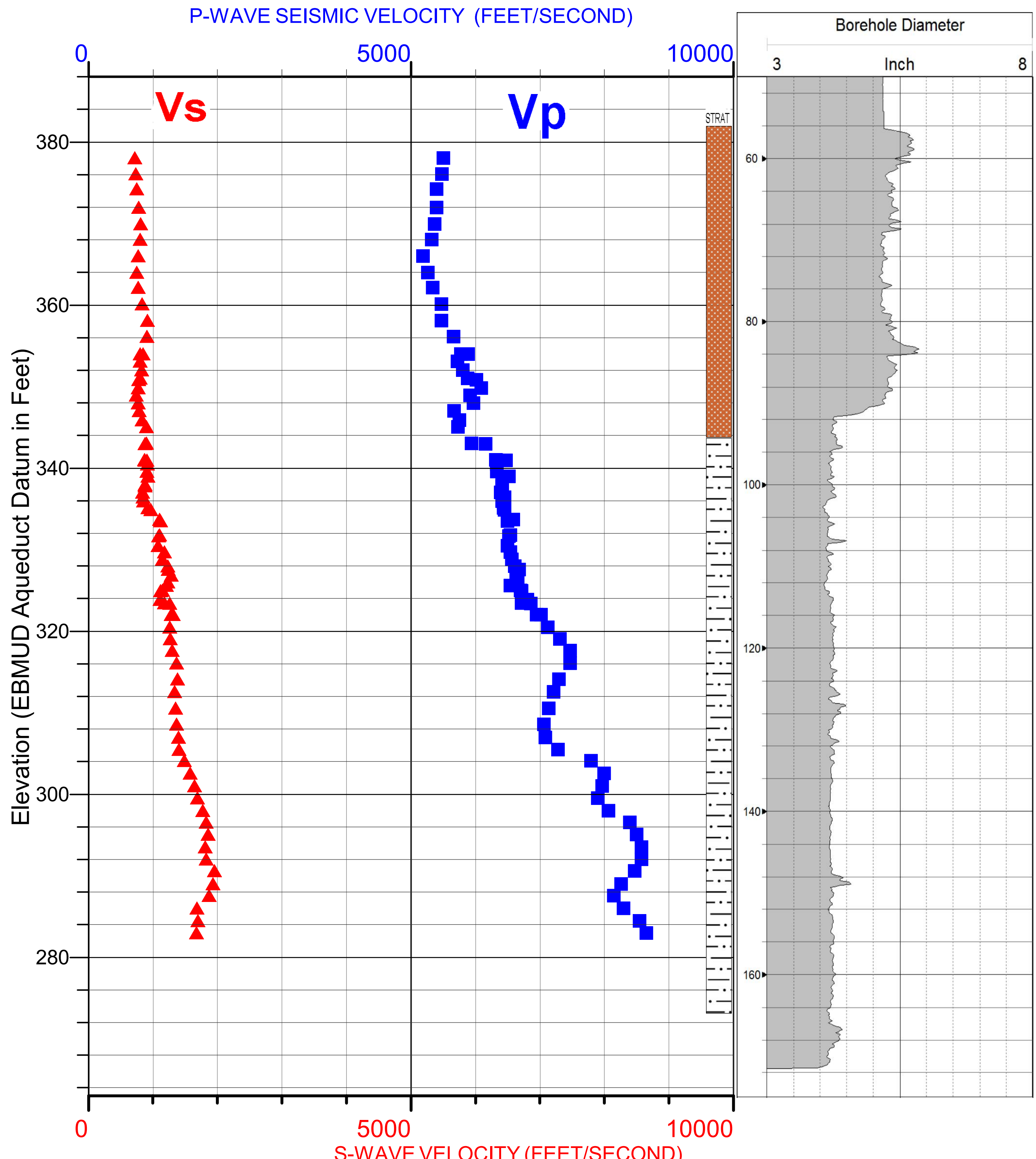
Appendix A:

PS-WAVE SUSPENSION BOREHOLE LOGGING (SBL) SURVEY



P- & S-WAVE VELOCITY LEGEND

▲ ▲ ▲ Vs-3ptAve*

■ ■ ■ Vp-3ptAve*

*Interval velocities should be used to calculate elastic moduli values

Strat Legend

Alluvium

Claystone (Orinda Fm.)

NORCAL

SUSPENSION P- AND S-WAVE VELOCITY PROFILE BOREHOLE B-1

LOCATION: Lafayette Reservoir, Lafayette, CA

CLIENT: A3GEO

JOB #: NS185079

NORCAL GEOPHYSICAL CONSULTANTS INC.

DATE:Sept., 2018

DRAWN BY: W. HENRICH

APPROVED BY: WJH

PLATE

1

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

Appendix A:

PS-WAVE SUSPENSION BOREHOLE LOGGING (SBL) SURVEY

APPENDIX A

BOREHOLE GEOPHYSICAL LOGGING

1.0 INSTRUMENTATION

NORCAL conducted the Borehole Geophysical Logging (BGL) investigation using a digital ***MICROLOGGER2*** System manufactured by **Robertson Geologging, Ltd.** This system consisted of the following components:

- control console
- computer
- motorized cable winch
- P- and S-Wave Suspension logger
- 3-arm caliper tool

2.0 SUSPENSION BOREHOLE LOGGING SURVEY

PS-wave Suspension Borehole Logging (SBL) was conducted in Borehole B-1 as shown on Plate 1. As a preface to the SBL survey, a caliper log was conducted to assess any blockage or excessive washout that might interfere with the passage of the suspension logging tool. The caliper tool consists of three interconnected mechanical arms that are spring loaded against the borehole wall. The horizontal deflections of the arms gauge the borehole diameter in units of inches with depth. Providing that no constrictions were present and that total (as drilled depth) was attained, the SBL survey proceeded with the measurement of compressional (P-) and shear (S-) wave velocities versus depth. Descriptions of the logging methodology, data acquisition and analysis procedures are presented in the following sections. The results of the PS-wave suspension velocities versus depth are listed in Tables 1.

2.1 METHODOLOGY

The PS-wave survey was conducted using a Robertson Geologging, Ltd. digital suspension logging system. A schematic diagram of the suspension logging tool is shown in Figure 1. The tool is equipped with a dipole seismic energy source located near the bottom of the probe and a pair of detectors (receivers) designated as R1 and R2, located within the middle to the upper sections. The distance from the energy source to the closest receiver was 10.3 feet (3.15 meters) when assembled with a detachable 2-meter isolation tube. The in-line distance between the receiver pair was 3.28 feet (1.0 meter). Each receiver contains one horizontal and one vertical oriented element. The horizontal elements preferentially record shear wave motion and the vertical receiver elements preferentially record first arriving P-wave energy.

When assembled with a 2-meter isolation tube, the suspension logging tool is approximately 23-ft long (Figure 1). By definition, the depth reference point of the tool is half-way between the two receivers. Since this point is approximately 15-ft from the probe tip, the maximum depth of a suspension logging survey, given a non-sloughing borehole, will always be reported as 15 feet less than the total depth of the borehole. When in operation, the probe is centralized in the borehole with flexible rubber rings positioned just below the source and just above the receiver section. This is necessary in order to maintain a gap between the probe housing and borehole wall.

Suspension seismic data are collected at discrete depths in the fluid-filled portion of the borehole. At each measurement depth, the energy source is activated via commands from the surface control console. This activation causes a metal solenoid (anvil) to horizontally strike the inner probe housing. This energy propagates through the fluid to the borehole wall, which produces a seismic "flexure" wave in the adjacent formation. As this wave propagates radially into the formation a physical interaction between the seismic wave and the borehole wall creates tube waves together with refracted P-waves that travel up the borehole to the two receivers.

2.2 DATA ACQUISITION

We measured P- and S- wave velocities at stationary depth positions distributed at 1- to 2-ft intervals throughout the accessible depth range of the borehole as indicated by the red triangles and blue squares shown on Plate 2. The maximum measurement depth was 156-feet below the top of the drill deck. The drill deck relates to top of the barge, as this was a marine survey with the depth of the water at 44-feet from the drill deck to the mudline of the reservoir. Our procedure was to lower the tool via a conductor casing that terminated 10-feet past the mud line into the alluvium down to the maximum depth of the open borehole and then take measurements in the up-hole direction until the tip of the steel conductor casing was intersected by the tool receivers. However, in this survey we found that because of the very viscous and heavy weight of the drill mud we had to change acquisition parameters (increase time window and receiver gain) to account for slower Vs (S-wave velocities) in alluvium. When the data quality became so poor, we withdrew the probe, had the driller flush the borehole with clear water and then reintroduce the PS-wave tool shortening its source to receiver configuration to help increase the signal amplitude of the Vs waveforms. By shortening (see Figure 1), we mean that the isolator section was reduced from 2-meter (6.56 ft.) to 1 meter (3.28 ft.). The complete PS-wave survey required three different files with depth overlap.

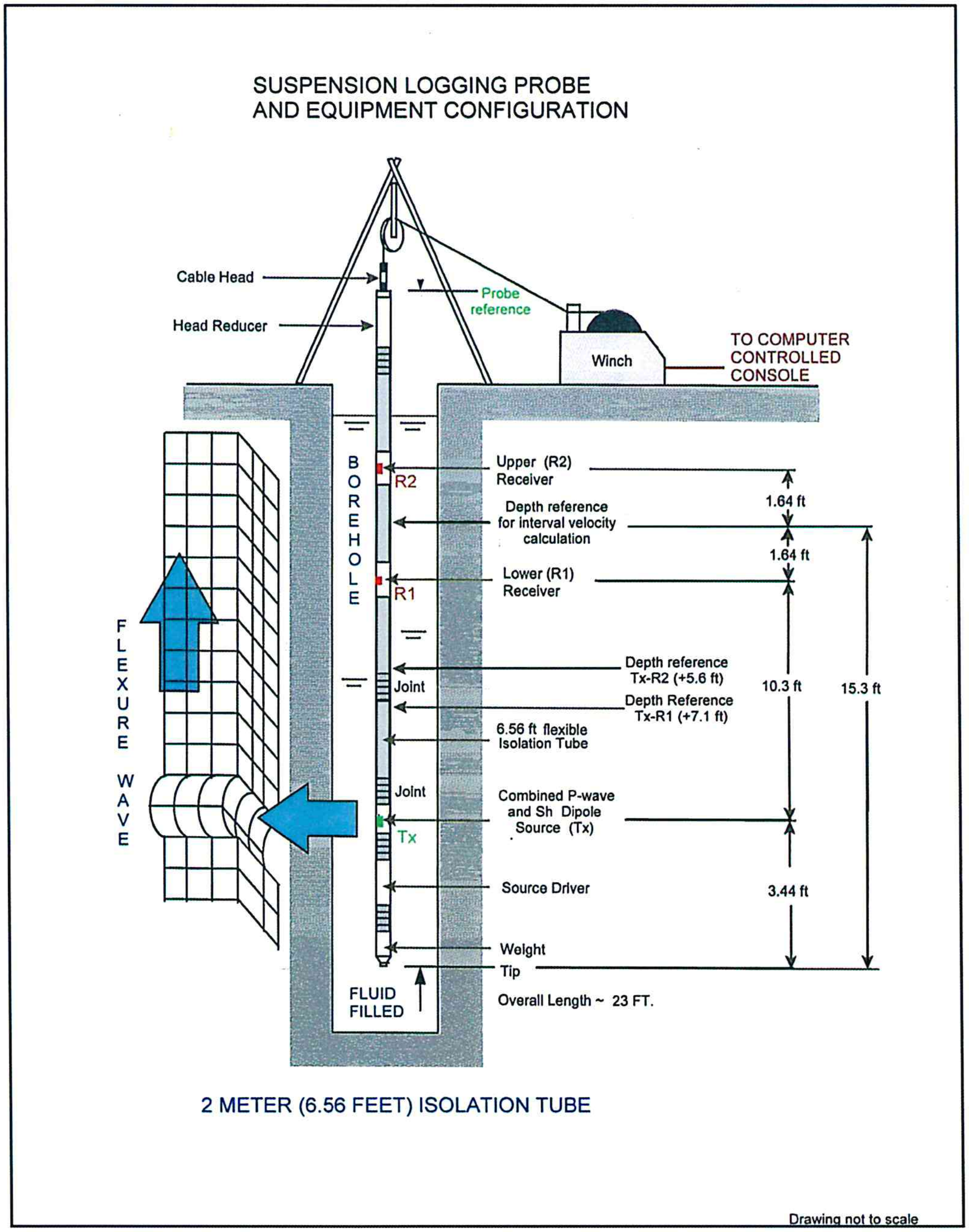


Figure 1: Suspension logger schematic diagram

At each measurement station, we cycled the energy source to fire 1 to 2 times in succession into each of the receiver elements. This cycling algebraically summed (stacked) the seismic energy resulting in an enhanced signal-to-noise ratio. We recorded S-wave velocities by applying a 1.2 KHz low pass filter to the signals detected by the horizontal receiver elements. This filtering reduces high frequency interference from the onset of earlier arriving P-wave energy on the S-wave channels. We recorded P- waveforms using a 20 KHz low pass filter.

2.3 DATA ANALYSIS

2.3.1 Seismic Records

P-wave velocities (Vp) and S-wave velocities (Vs) were calculated with the interpretation computer software programs ***PSLogger Application*** Version 1.121 and ***PSLOG Analysis*** Version 1.0.001. Both programs are published by ***Robertson Geologging, Ltd.*** (2009). Four sample suspension logger seismic records from Borehole B-1 are presented in Figures 2, 3, 4 and 5. The first two figures represent records gathered in alluvium at depths 74- and 88-ft below top of drill deck (btod). Figures 4 and 5 represent records gathered in bedrock (claystone) at depths 137- and 153.5-ft btod. Individual records display six seismic wave-traces. The upper four traces were detected by the horizontal receiver elements and are used to identify S-wave arrivals. These traces are labeled according to the wave type (s=shear), the direction of anvil impact (l=left and r=right) and the relative distance of the receiver from the source (n=near and f=far). For example, the top wave trace is labeled "srf" because it was recorded to identify S-waves using a right strike as detected by the far receiver. The seismic S-wave traces produced by a right hand strike of the source (Cycle 1) are colored red for both the near and far receivers. The seismic S-wave traces produced by a left hand strike of the source (Cycle 2) are colored green for both the near and far receivers.

The lower two traces were detected by the vertical receiver elements and are used to identify P-wave arrivals. These traces are labeled according to the wave type (p=primary) and the relative distance of the receiver from the source (n=near and f=far). For example, the bottom most wave trace is labeled "pn" because it was recorded to identify P-waves by the near receiver. These seismic wave traces are produced during Cycle 3 and are colored blue. Since they were detected by the vertical elements in the receivers, the direction of impact is inconsequential and is not addressed by separate waveforms. Note that the time scale range in Figure 2 for the S-wave recording window (0 to 28,000 microseconds) is eight times larger than the P-wave recording window (0 to 3,500 microseconds). Suspension records that appeared too ambiguous or lacked useable S-wave information were discarded from the final presentation of velocities.

Figure 2. *Sample seismic record Run-3 from 74-ft btod (alluvium)*

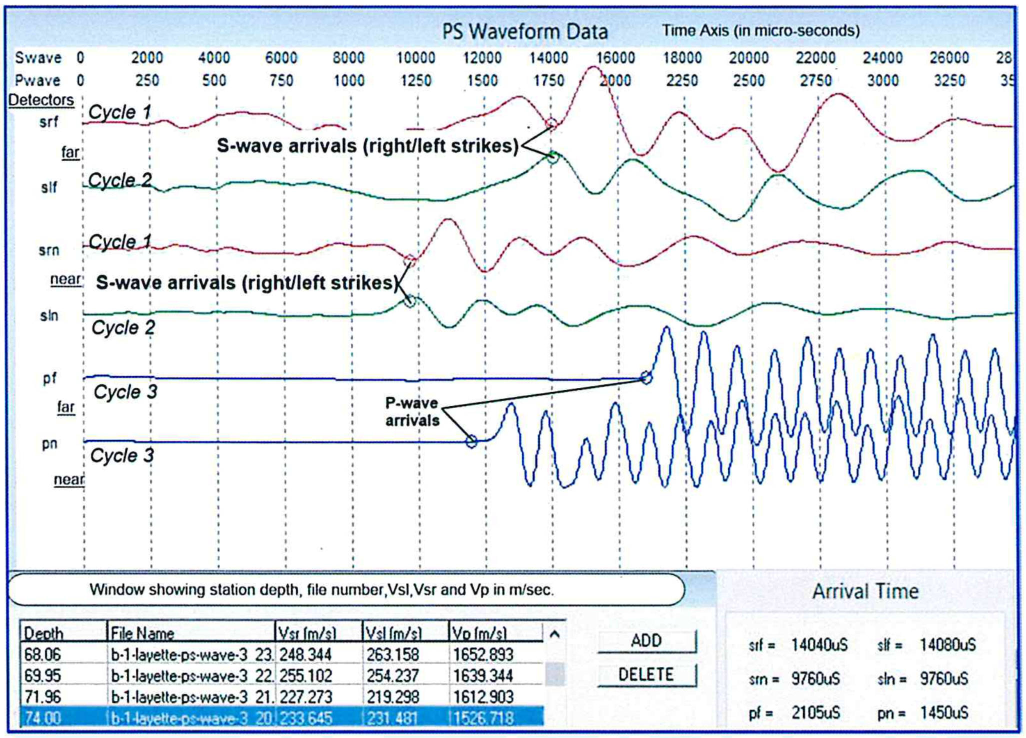


Figure 3. *Sample seismic record Run-3 from 88-ft btod (alluvium)*

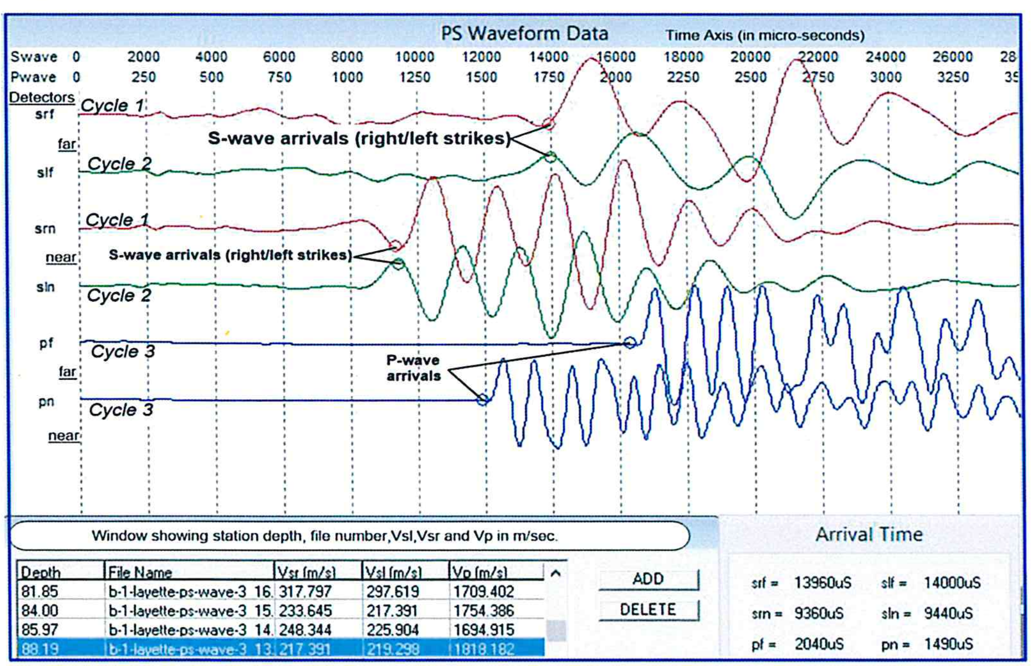


Figure 4. *Sample seismic record Run-3 from 137-ft btod (bedrock)*

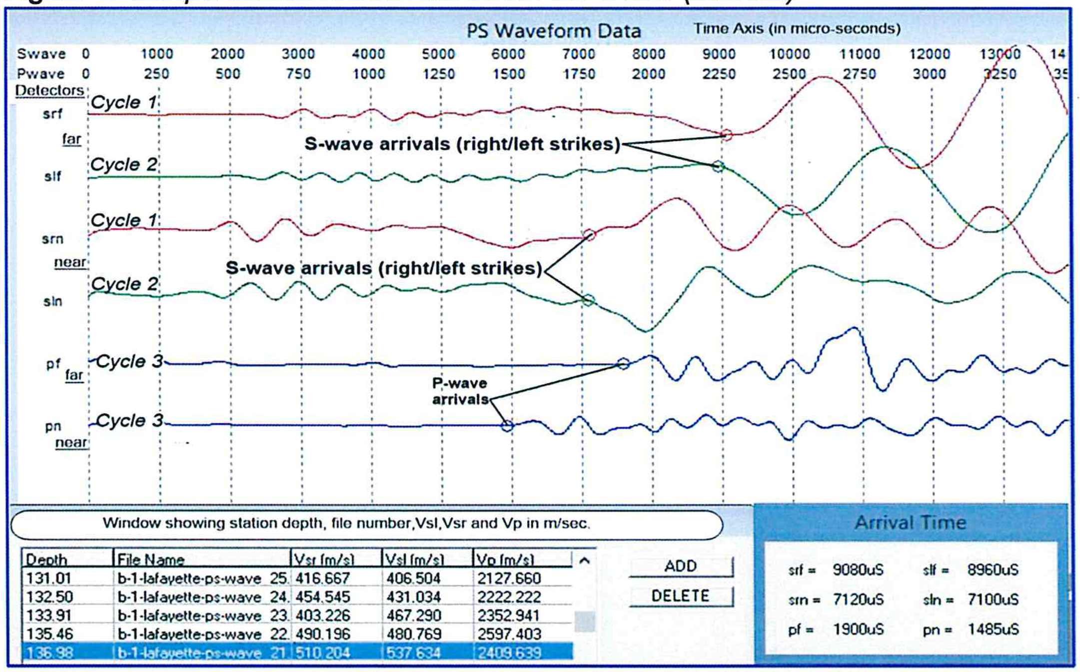
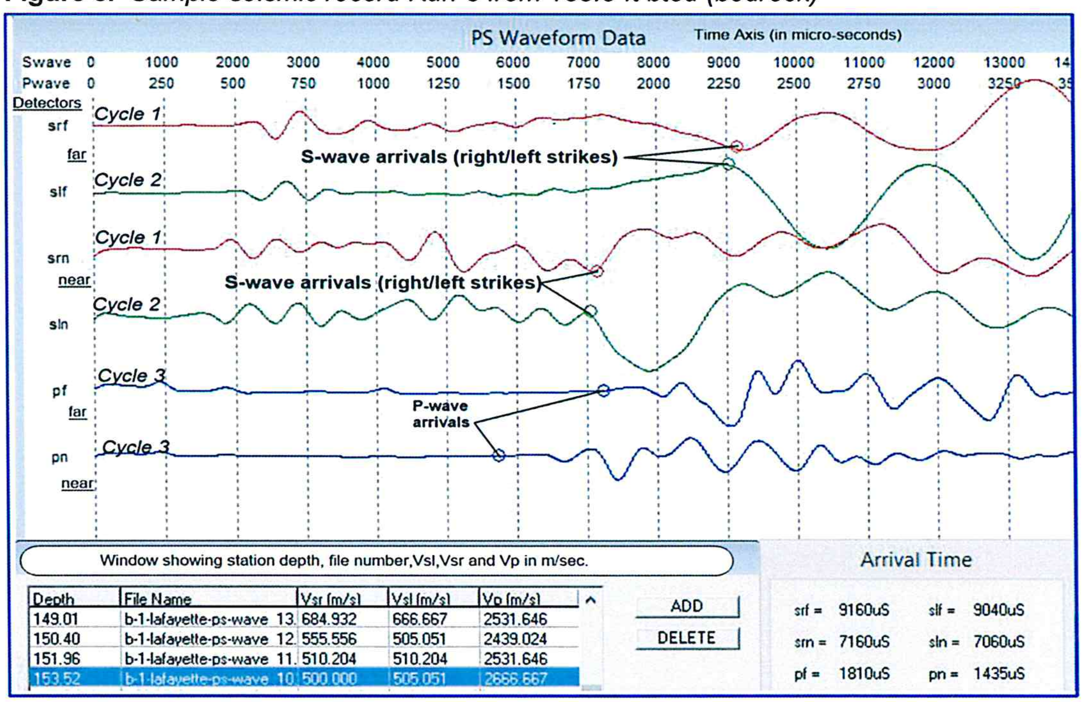


Figure 5. *Sample seismic record Run-3 from 153.5-ft btod (bedrock)*



2.3.2 S-Wave Arrivals

On the seismic records shown in Figures 2, 3, 4 and 5, the red traces (Cycle 1) were created by right anvil impacts and the green traces (Cycle 2) were created by left anvil impacts. Pairing the traces produced by opposite directions of impact reveals a phase reversal that is associated with the onset (arrival) of S-wave energy. However, because there can be slight discrepancies in timing between Cycle 1 and Cycle 2, the reversal point may not occur at the same exact time on both traces. Therefore, the onset of S-wave energy is further defined as the point where there is also a significant increase in amplitude within the phase reversal time window. These arrival times are depicted by open dots on the upper four wave traces.

2.3.3 P-Wave Arrivals

P-wave arrivals are identified as the point where the wave traces produced by Cycle 3 (blue) change from straight lines to sinusoidal wave forms. These points, referred to as “first breaks”, are represented by blue circles on the lower two wave traces.

2.3.4 Seismic Velocity Calculations

Seismic wave velocities (V) are calculated by dividing the distance (X) from the source to a given receiver by the time required (T) for the seismic wave to reach that receiver. Hence, (V) = X/T. This is referred to as a direct path seismic velocity. However, velocities can also be calculated by dividing the distance between the two receivers (R1 and R2) by the difference in travel time to those receivers (T2-T1). This is considered an interval velocity. For example:

$$V_{(R2-R1)} = (X_{R2}-X_{R1}) / (T_{R2}-T_{R1})$$

Where X_{R1} and T_{R1} are the distance and travel time to the first receiver (R1); and X_{R2} and T_{R2} are the distance and time to the second receiver (R2). The separation between receivers is one-meter.

2.3.5 Interval Seismic Velocities

We used the travel times measured in Borehole B-1 to compute four interval velocities; three S-wave (Vs) and one P-wave (Vp) for each borehole. The interval Vs were computed using the Cycle 1 (Vs_{Right}) and Cycle 2 (Vs_{Left}) travel times and their average. The interval Vp were computed using the Cycle 3 travel times. All station depths and velocities are listed in Table 1. The velocities are listed in both metric (meters, meters per second) and Imperial (feet, feet per second) units. As indicated in the Data Acquisition Section (2.2) we collected several PS-velocity profiles (files) depending on boreholes fluid viscosity and whether the data were collected within the alluvium or bedrock sequence. As a result of this data density, the station spacing along the upper portions of the finalized velocity profile are closely spaced and in some cases show station duplication. As a method to integrate the closely spaced data and smooth the velocity distribution, we applied a 3-point running average to both Vs and Vp. The resulting smoothed values are presented in the two far right columns of the table. These values, that are shaded blue in the table and are graphed on Plate 2, represent our interpretation of the Vs and Vp distribution with depth. We have emphasized the interval velocities because they are the least susceptible to variations in triggering and/or coupling errors. The depths of the measurements are based on a probe reference point that is half-way between the near and far receivers.

3.0 RESULTS

The results of the Borehole Geophysical Investigations in Borehole B-1 are illustrated by the Suspension P- and S-wave velocity Profile shown on Plate 2. The results are also tabulated in Tables 1 below.

The Suspension P- and S-wave Profiles contains, from left to right, a seismic velocity vs. depth graph, a stratigraphic (Strat) column and a borehole diameter vs. depth graph. On the seismic velocity vs. depth graph, the horizontal axis represents velocity and ranges from 0-ft/sec on the left to 10,000-ft/sec on the right. The vertical axis represents depth in feet bgs and ranges from 50-ft at the top to 175-ft at the bottom. The stratigraphic column shown on the right hand side of the graph differentiates between embankment on top and bedrock on the bottom according to the “Strat Legend” shown in the lower left hand corner of the plate. Finally, the borehole diameter vs. depth graph is shown on the right hand side of the plate. Here, the horizontal axis represents borehole diameter in inches and ranges from 3-in on the left to 8-in on the right. The vertical axis is the same as on the velocity vs. depth graph.

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Table 1: Borehole 1, P- and S-wave Velocity Table

METRIC UNITS DEPTHS & INTERVAL VELOCITIES					IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES						
Depth	VsLeft	VsRight	VsAvg	Vp	Depth	VsLeft	VsRight	VsAvg	Vp	Vs3pt Ave	Vp3ptAve
Meters	M/sec.	M/sec.	M/sec.	M/sec.	Feet	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.
17.67	212	216	214	1695	57.97	695	707	701	5527		
18.27	229	219	224	1724	59.95	753	719	736	5622	723	5513
18.88	219	227	223	1653	61.94	719	743	731	5390	737	5482
19.44	229	223	226	1667	63.78	753	732	742	5435	749	5405
20.13	234	237	235	1653	66.06	767	779	773	5390	785	5405
20.74	263	248	256	1653	68.06	863	815	839	5390	816	5375
21.32	254	255	255	1639	69.95	834	837	836	5346	802	5332
21.93	219	227	223	1613	71.96	719	746	733	5260	777	5195
22.55	231	234	233	1527	74.00	759	767	763	4978	751	5271
23.12	229	231	230	1709	75.85	753	759	756	5574	778	5344
23.74	237	260	249	1681	77.89	776	854	815	5481	833	5482
24.36	270	296	283	1653	79.93	885	973	929	5390	918	5482
24.95	298	318	308	1709	81.85	976	1043	1010	5574	909	5668
25.60	258	223	240	1852	83.98	846	732	789	6039	846	5778
25.60	217	234	226	1754	84.00	713	767	740	5721	807	5896
25.89	301	243	272	1818	84.93	988	796	892	5929	803	5726
26.20	226	248	237	1695	85.97	741	815	778	5527	824	5813
26.51	240	248	244	1835	86.97	789	812	800	5983	804	5888
26.57	248	260	254	1887	87.16	812	854	833	6153	783	6022
26.88	219	217	218	1818	88.19	719	713	716	5929	773	6098
27.14	238	231	235	1905	89.05	781	759	770	6211	749	5921
27.43	234	229	232	1724	90.00	767	753	760	5622	777	5976
27.72	243	245	244	1869	90.96	796	804	800	6095	793	5673
28.08	250	249	250	1626	92.13	820	817	819	5302	832	5758
28.33	260	275	268	1802	92.96	854	901	878	5875	903	5739
28.94	316	301	309	1852	94.96	1038	988	1013	6039	900	5948
28.96	245	248	246	1818	95.00	804	812	808	5929	882	6163
29.55	245	258	251	2000	96.96	804	846	825	6522	870	6324
29.58	309	287	298	2000	97.05	1013	943	978	6522	881	6479
29.58	256	256	256	1961	97.06	841	841	841	6394	910	6337
29.72	278	278	278	1869	97.51	911	911	911	6095	918	6337

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METRIC UNITS DEPTHS & INTERVAL VELOCITIES					IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES						
Depth	VsLeft	VsRight	VsAvg	Vp	Depth	VsLeft	VsRight	VsAvg	Vp	Vs3pt Ave	Vp3ptAve
Meters	M/sec.	M/sec.	M/sec.	M/sec.	Feet	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.
30.00	313	298	305	2000	98.42	1025	976	1001	6522	913	6337
30.18	251	253	252	1961	99.03	823	831	827	6394	927	6524
30.49	291	291	291	2041	100.02	954	954	954	6655	882	6420
30.54	264	263	264	1905	100.21	866	863	865	6211	879	6420
30.77	245	254	249	1961	100.94	804	833	818	6394	844	6398
30.96	258	260	259	2020	101.57	846	854	850	6588	850	6459
31.10	270	268	269	1961	102.03	885	879	882	6394	856	6418
31.36	254	256	255	1923	102.90	833	841	837	6271	921	6440
31.44	316	321	318	2041	103.14	1038	1052	1045	6655	967	6461
31.78	309	313	311	1980	104.28	1013	1025	1019	6457	1103	6589
31.85	391	368	379	2041	104.49	1282	1206	1244	6655	1119	6502
32.38	342	325	334	1961	106.22	1124	1065	1094	6394	1107	6545
32.41	298	301	299	2020	106.32	976	988	982	6588	1094	6523
32.78	368	368	368	2020	107.54	1206	1206	1206	6588	1087	6502
33.01	333	321	327	1942	108.31	1094	1052	1073	6332	1184	6548
33.27	391	385	388	2062	109.16	1282	1262	1272	6724	1153	6570
33.54	342	338	340	2041	110.03	1124	1108	1116	6655	1231	6612
33.66	417	379	398	1980	110.44	1367	1243	1305	6457	1238	6683
33.87	397	391	394	2128	111.13	1302	1282	1292	6938	1285	6639
34.13	382	385	383	2000	111.98	1252	1262	1257	6522	1234	6661
34.25	361	342	352	2000	112.37	1183	1124	1153	6522	1209	6544
34.45	370	370	370	2020	113.02	1215	1215	1215	6588	1152	6707
34.46	329	333	331	2151	113.05	1079	1094	1086	7013	1121	6722
34.79	321	326	323	2013	114.13	1052	1070	1061	6566	1115	6814
34.91	373	357	365	2105	114.52	1224	1172	1198	6865	1178	6718
34.93	394	385	389	2062	114.59	1292	1262	1277	6724	1266	6867
35.35	407	400	403	2151	115.97	1334	1312	1323	7013	1285	6950
35.35	379	387	383	2182	115.98	1243	1268	1256	7115	1315	7022
35.81	413	420	417	2128	117.50	1356	1379	1367	6938	1263	7127
36.26	355	357	356	2247	118.95	1163	1172	1168	7328	1269	7313
36.69	391	385	388	2353	120.39	1282	1262	1272	7673	1304	7471
37.17	446	450	448	2273	121.95	1465	1478	1471	7411	1366	7471

NORCAL GEOPHYSICAL CONSULTANTS INC.
A Terracon COMPANY

METRIC UNITS DEPTHS & INTERVAL VELOCITIES					IMPERIAL UNITS DEPTHS AND INTERVAL VELOCITIES						
Depth	VsLeft	VsRight	VsAvg	Vp	Depth	VsLeft	VsRight	VsAvg	Vp	Vs3pt Ave	Vp3ptAve
Meters	M/sec.	M/sec.	M/sec.	M/sec.	Feet	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.	Ft./sec.
37.76	407	420	413	2247	123.89	1334	1379	1356	7328	1385	7302
38.24	400	410	405	2198	125.46	1312	1345	1328	7167	1341	7220
38.84	410	407	408	2198	127.44	1345	1334	1339	7167	1359	7141
39.45	431	427	429	2174	129.44	1414	1402	1408	7089	1366	7065
39.93	407	417	412	2128	131.01	1334	1367	1350	6938	1404	7091
40.39	431	455	443	2222	132.50	1414	1491	1453	7246	1410	7286
40.82	467	403	435	2353	133.91	1533	1323	1428	7673	1491	7796
41.29	481	490	485	2597	135.46	1577	1608	1593	8470	1580	8000
41.75	538	510	524	2410	136.98	1764	1674	1719	7858	1649	7970
42.20	515	481	498	2326	138.44	1691	1577	1634	7583	1691	7899
42.67	543	505	524	2532	140.00	1783	1657	1720	8255	1773	8067
43.12	588	610	599	2564	141.46	1930	2001	1965	8361	1830	8399
43.58	575	526	551	2632	142.97	1886	1727	1806	8581	1862	8508
44.03	575	532	553	2632	144.46	1886	1745	1815	8581	1815	8581
44.48	556	556	556	2632	145.95	1823	1823	1823	8581	1825	8581
44.92	588	532	560	2632	147.37	1930	1745	1838	8581	1959	8473
45.42	667	685	676	2532	149.01	2187	2247	2217	8255	1932	8263
45.84	505	556	530	2439	150.40	1657	1823	1740	7953	1877	8155
46.32	510	510	510	2532	151.96	1674	1674	1674	8255	1688	8302
46.79	505	500	503	2667	153.52	1657	1640	1649	8696	1699	8549
47.24	532	549	541	2667	155.00	1745	1803	1774	8696	1680	8658
47.55	505	481	493	2632	156.00	1657	1577	1617	8581		

APPENDIX D

Laboratory Test Data Sheets

A3GEO

MOISTURE & DENSITY TEST

Client : A3GEO

Project : Lafayette Outlet Tower Seismic Retrofit

ISI Lab No.: G-62899

Job no : 1141-4A

Boring #	B-1	B-1	B-1	B-1	B-1	B-1	B-1	B-1
Sample Depth (ft.)	3	5-7	8	14.5	19.5	23.5	25	29.5
Sample Elevation (ft.)	386	384-382	381	374.5	369.5	365.5	364	359.5
Soil type: (visual)	Dark greenish gray clay	Dark greenish gray clay	Dark greenish gray clay	Dark greenish gray clay	Greenish gray sandy clay	Grayish brown clay with sand	Grayish brown clay	Grayish brown clay
Date tested:	10/18/18	10/22/18	10/18/18	10/18/18	10/18/18	10/18/18	10/18/18	10/18/18
Tested by:	JH	JH	JH	JH	JH	JH	JH	JH
Specimen height (in.)		2.73						
Wt. of specimen + tare (gm)		569.66						
Tare wt. (gm)		0.00						
Diameter (in.)		2.84						
Wet wt. of soil + dish wt. (gm)	205.50	493.68	159.30	239.78	198.41	288.13	212.98	243.21
Dry wt. of soil + dish wt. (gm)	164.74	414.26	140.91	201.70	176.80	243.60	181.64	209.25
Wt. of dish (gm)	50.56	85.72	51.26	50.70	85.64	85.29	50.82	50.66
Dish ID								

Wet Density (pcf)		**125.4**						
Dry Density (pcf)		**101.0**						
Moisture Content (%)	**35.7**	**24.2**	**20.5**	**25.2**	**23.7**	**28.1**	**24.0**	**21.4**

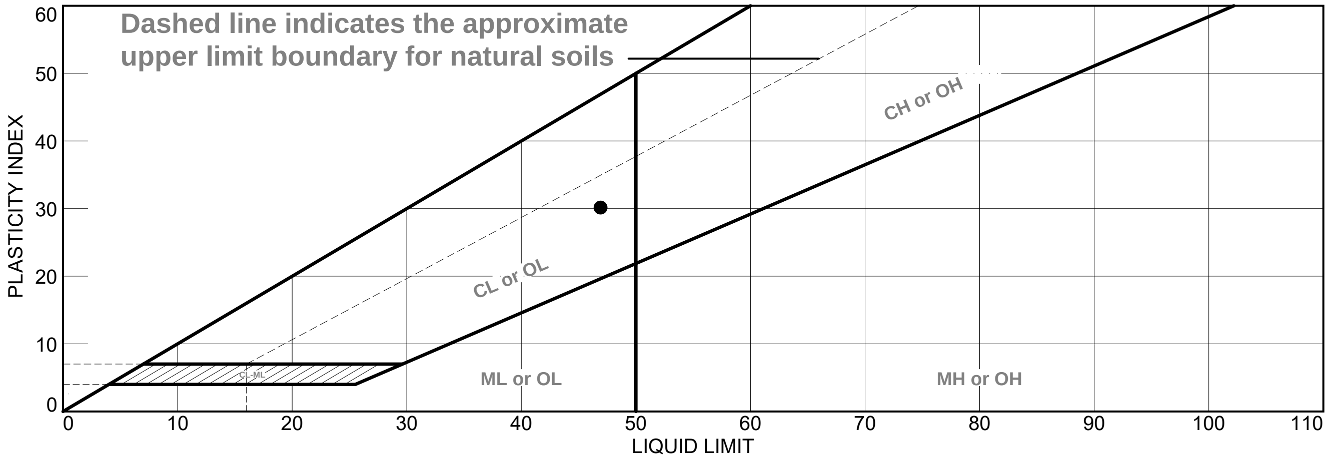
Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio		0.669						
Saturation (%)		97.6						

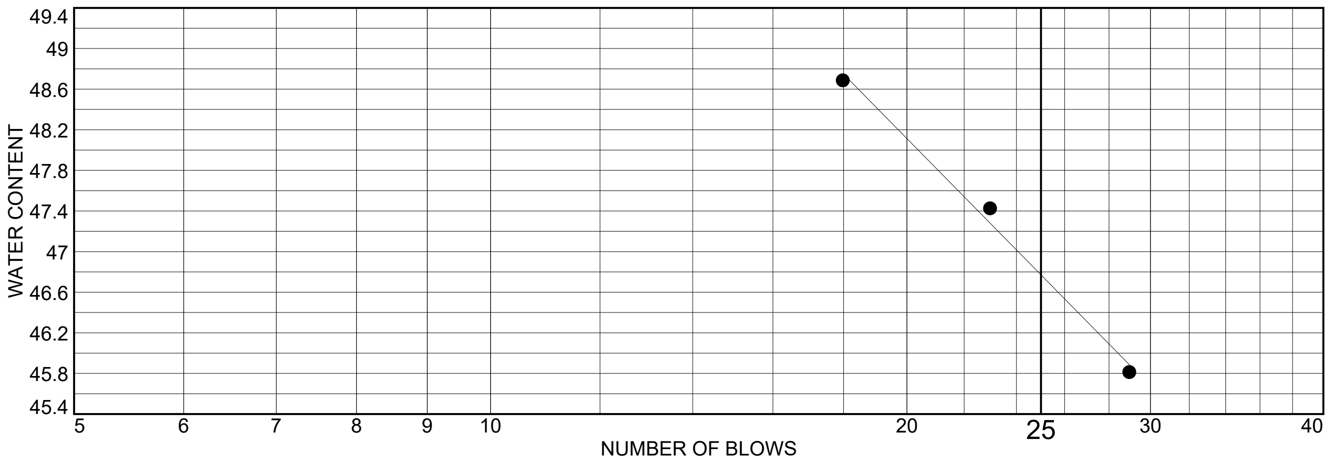
MOISTURE & DENSITY TEST

Client : A3GEO | Project : Lafayette Outlet Tower Seismic Retrofit | ISI Lab No.: G-62899 | Job no : 1141-4A

Boring #	B-1	B-1	B-1					
Sample Depth (ft.)	34	38.5	43.5					
Sample Elevation (ft.)	355	350.5	345.5					
Soil type: (visual)	Olive gray sandy clay	Olive gray sandy clay	Olive gray clay					
Date tested:	10/18/18	10/18/18	10/18/18					
Tested by:	JH	JH	JH					
Specimen height (in.)		2.93						
Wt. of specimen + tare (gm)		423.67						
Tare wt. (gm)		0.00						
Diameter (in.)		2.33						
Wet wt. of soil + dish wt. (gm)	655.15	584.90	478.60					
Dry wt. of soil + dish wt. (gm)	569.83	516.23	410.83					
Wt. of dish (gm)	187.61	187.85	84.59					
Dish ID								
Wet Density (pcf)		**129.1**						
Dry Density (pcf)		**106.8**						
Moisture Content (%)	**22.3**	**20.9**	**20.8**					
Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio		0.578						
Saturation (%)		97.7						

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark greenish gray clay	47	17	30			

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

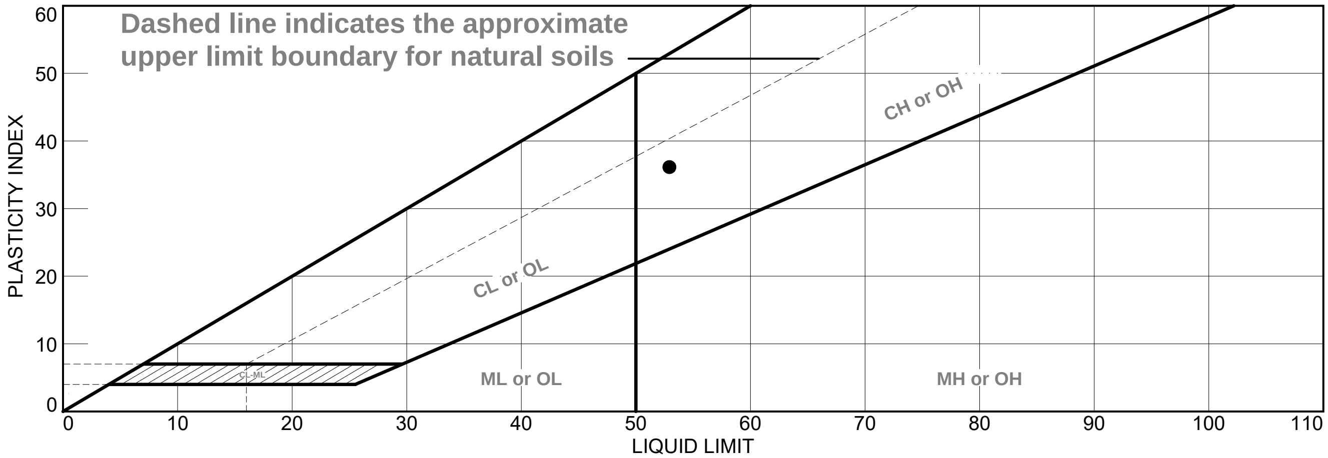
● **Source of Sample:** B-1 **Depth:** 8 ft **Sample Number:** 3

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Greenish gray clay	53	17	36			

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

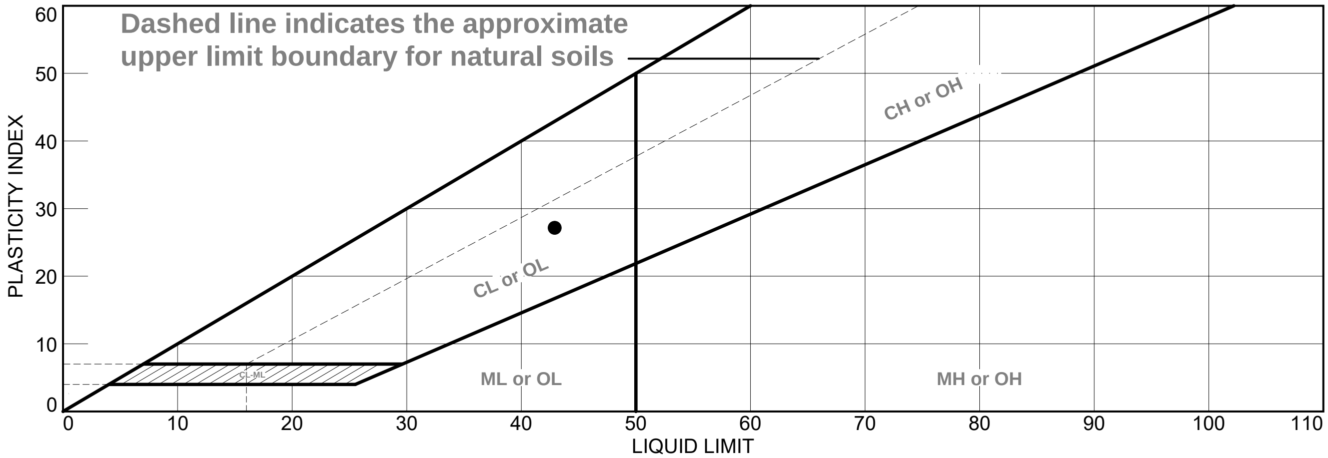
● **Source of Sample:** B-1 **Depth:** 12.5 ft **Sample Number:** 4

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Greenish gray sandy clay	43	16	27		61.5	

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

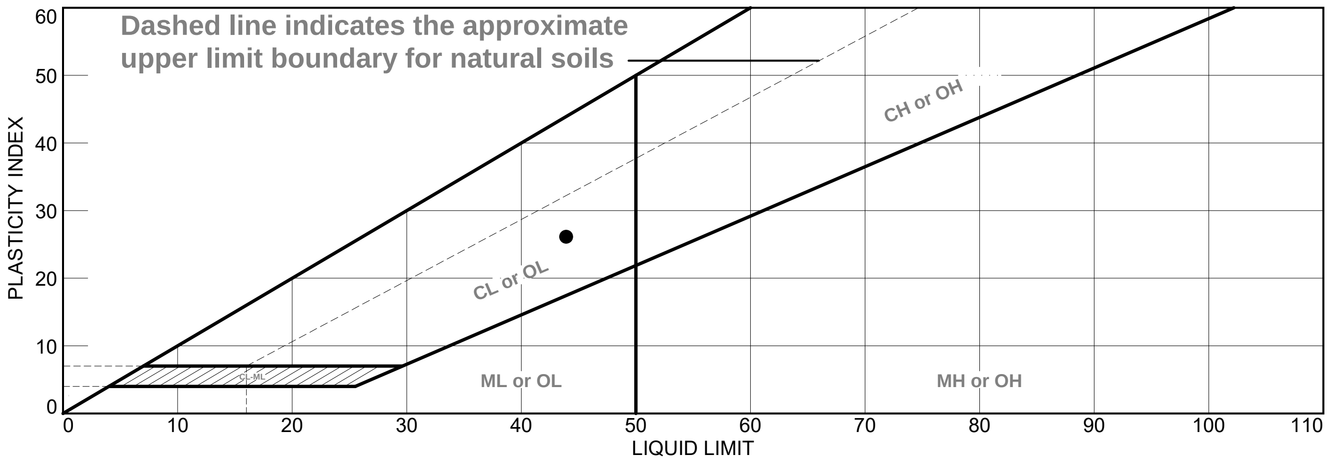
● **Source of Sample:** B-1 **Depth:** 19.5 ft **Sample Number:** 7

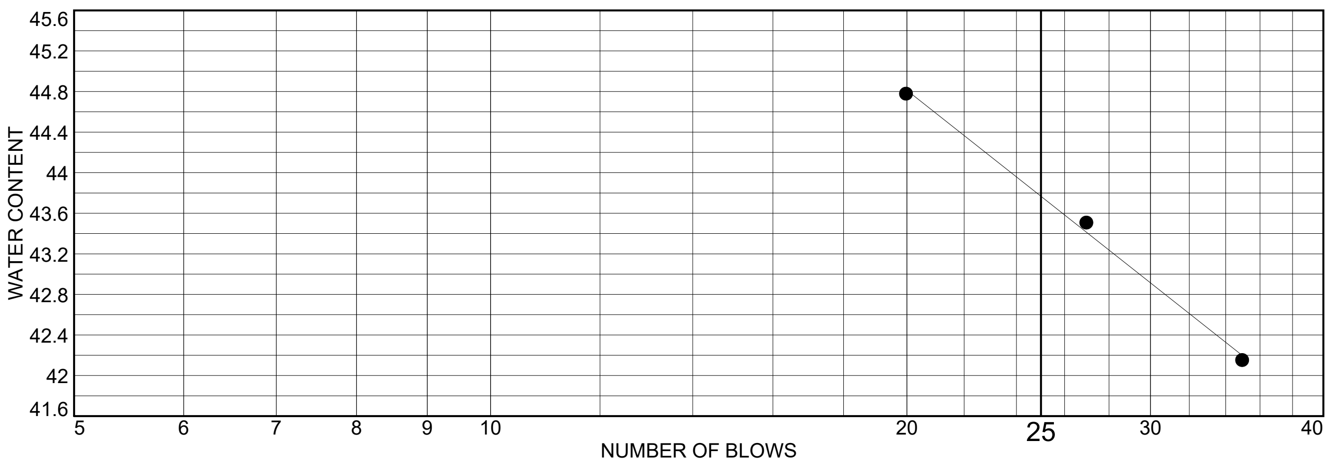
Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Grayish brown clay	44	18	26			

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

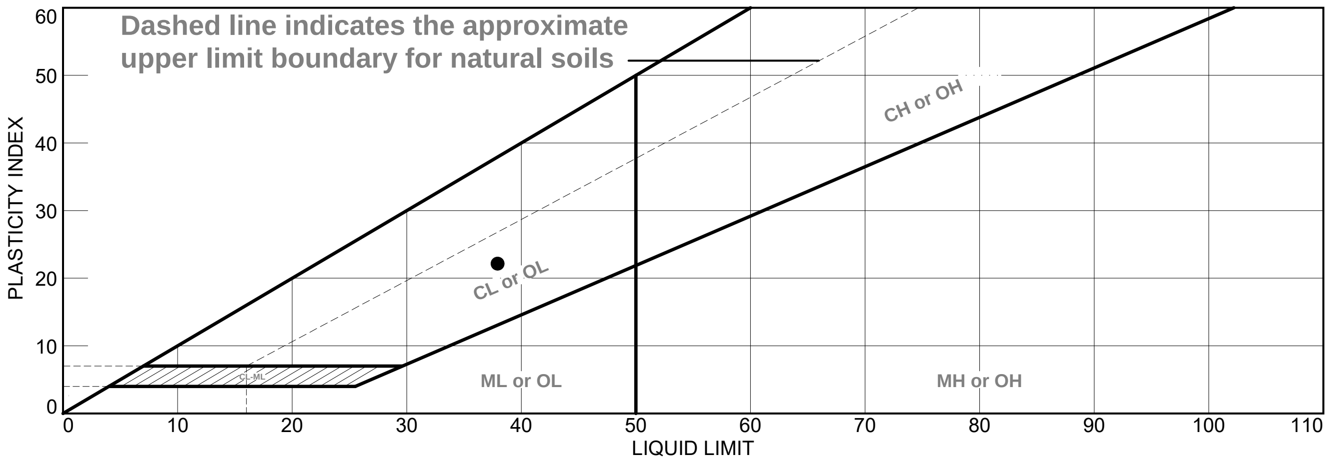
● **Source of Sample:** B-1 **Depth:** 27.5 ft **Sample Number:** 10

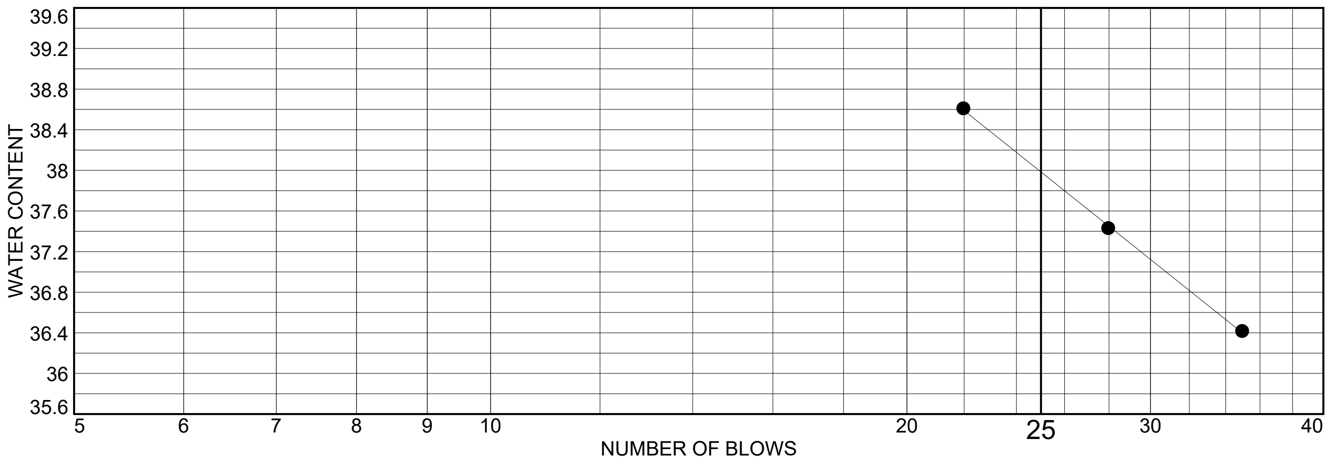
Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive gray clayey sand	38	16	22	70	48	

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

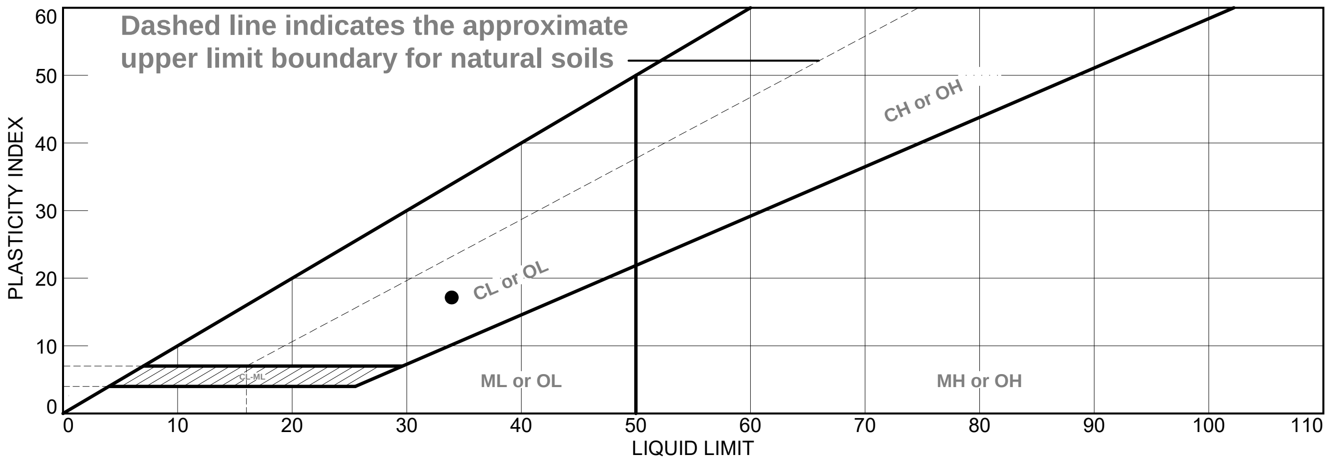
● **Source of Sample:** B-1 **Depth:** 34 ft **Sample Number:** 12

Remarks:

Figure

Tested By: JH **Checked By:** JH

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Olive gray sandy clay	34	17	17	75	50	CL

Project No. 2530-025.0 **Client:** A3GEO

Project: Lafayette Outlet Tower Seismic Retrofit

1141-4A

● **Source of Sample:** B-1 **Depth:** 38.5 ft **Sample Number:** 13

Remarks:

Figure

Tested By: JH **Checked By:** JH

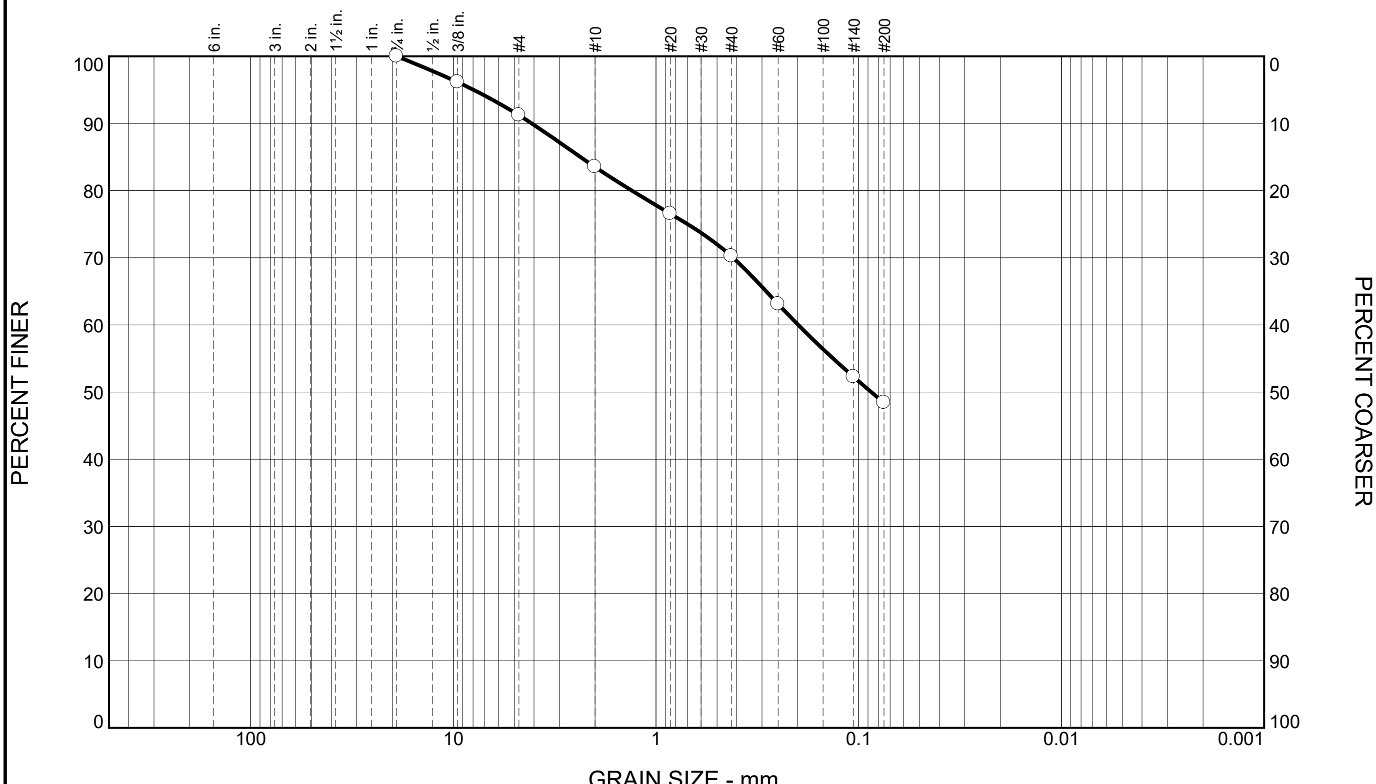
ASTM D-1140
PERCENT PASSING NO. 200 SIEVE REPORT

Method A
Specimens Soaked Overnight without Deflocculating Agent
Dry Mass Determined Directly

Client Name A3GEO
Project Name Lafayette Outlet Tower Seismic Retrofit
Project Number 1141-4A

Boring Number	B-1				
Sample Depth (ft)	19.5				
Sample Elevation (ft)	369.5				
Percent of Soil Finer than No. 200 Sieve	61.5				
Visual Classification	Greenish gray sandy clay				
Date	10/18/18				
Weight of Dry Soil + Pan (before wash)	176.8				
Weight of Dry Soil + Pan (after wash)	120.7				
Weight of Pan	85.6				

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	9	7	14	22	48	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100		
3/8	96		
#4	91		
#10	84		
#20	77		
#40	70		
#60	63		
#140	52		
#200	48		

* (no specification provided)

Material Description

Olive gray clayey sand

Atterberg Limits

PL= 16 LL= 38 PI= 22

Coefficients

D_{90}= 4.1094 D_{85}= 2.3618 D_{60}= 0.1989
D_{50}= 0.0867 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO= A-6(6)

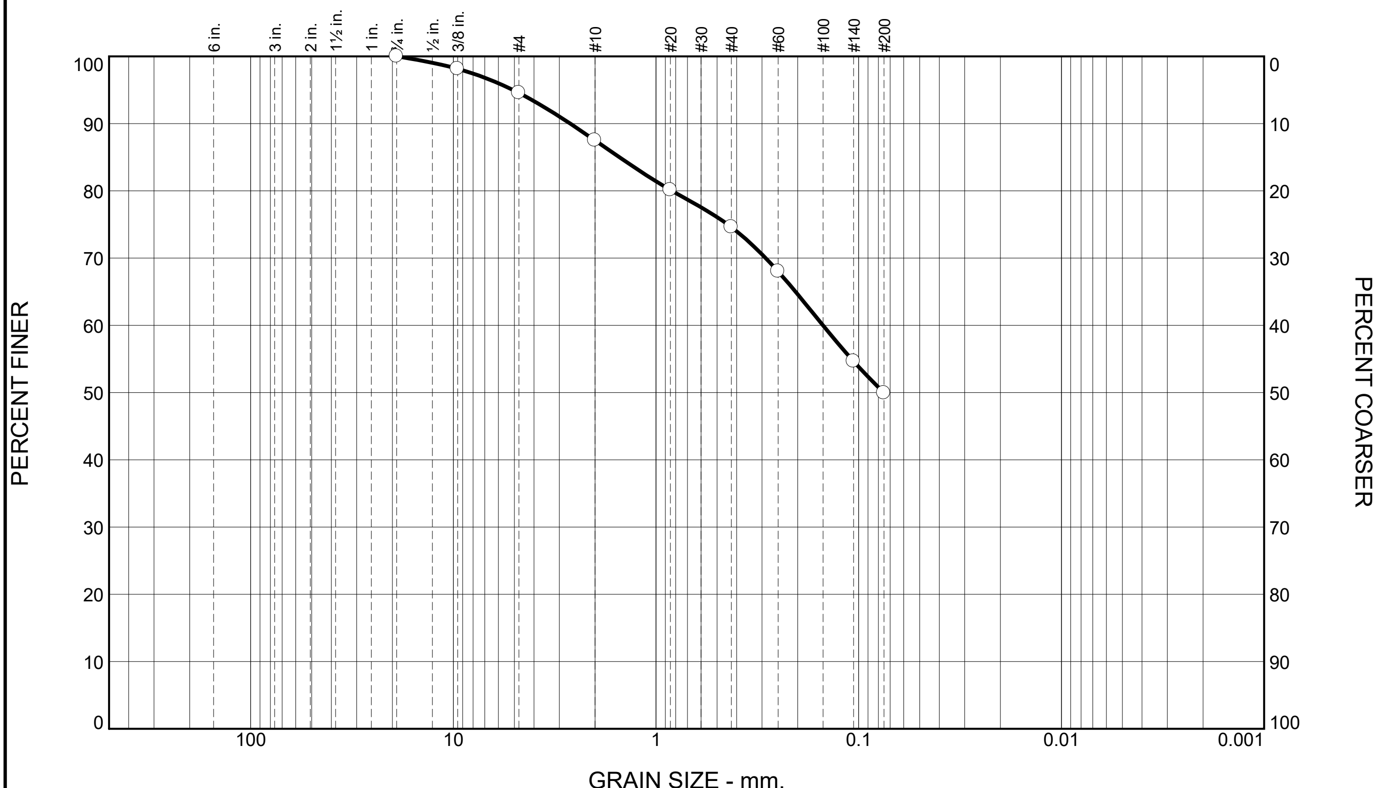
Remarks

Source of Sample: B-1 **Depth:** 34 ft
Sample Number: 12 **Date:** 10-23-18

Client: A3GEO
Project: Lafayette Outlet Tower Seismic Retrofit
1141-4A
Project No: 2530-025.0 **Figure**

Tested By: JH **Checked By:** JH

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	5	8	12	25	50	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100		
3/8	98		
#4	95		
#10	87		
#20	80		
#40	75		
#60	68		
#140	55		
#200	50		

* (no specification provided)

Material Description

Olive gray sandy clay

Atterberg Limits

PL= 17 LL= 34 PI= 17

Coefficients

D_{90}= 2.6544 D_{85}= 1.5161 D_{60}= 0.1501
D_{50}= 0.0754 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-6(5)

Remarks

Source of Sample: B-1 **Depth:** 38.5 ft
Sample Number: 13

Date: 10-23-18

Client: A3GEO
Project: Lafayette Outlet Tower Seismic Retrofit
1141-4A
Project No: 2530-025.0 **Figure**

Tested By: JH **Checked By:** JH

Boring Number	B-1		B-1		B-1	
Sample Number	4		4		4	
Depth (ft)	12.5		12.5		12.5	
Date Tested	10/23/18		10/24/18		10/25/18	
Description	Greenish gray clay		Greenish gray clay		Greenish gray clay	
Sample Condition	Undisturbed		Undisturbed		Undisturbed	
	Initial	**After Consolidation**	**Initial**	**After Consolidation**	**Initial**	**After Consolidation**
Height (in)	5.98	5.98	5.98	5.85	5.98	5.79
Diameter (in)	2.84	2.84	2.87	2.90	2.89	2.92
Height/Diameter Ratio	2.11		2.08		2.07	
Total Weight (g)	1295.37	1303.76	1295.37	1303.76	1295.37	1303.76
Moisture Content (%)	20.37	21.15	20.37	21.15	20.37	21.15
Moisture Content From	entire sample		entire sample		entire sample	
Wet Density (pcf)	130.27	131.41	127.47	128.82	126.06	128.14
Dry Density (pcf)	108.22	108.47	105.90	106.34	104.73	105.77
Area (cm^2)	40.87	40.79	41.77	42.50	42.23	43.16
Total Volume (cc)	620.77	619.37	634.39	631.79	641.48	635.18
Void Ratio	0.5574	0.5539	0.5916	0.5851	0.6094	0.5936
Saturation (%)	98.7	103.1	93.0	97.6	90.2	96.2
Specific Gravity	2.70		2.70		2.70	
Specific Gravity From	Assumption		Assumption		Assumption	
B value Before Consolidation	0.96		0.96		0.96	
Total Back Pressure (psf)	8640		8640		8640	
Rate of Strain (%/min)	0.02		0.02		0.02	
Axial Strain at Failure (%)	2.40		1.10		1.50	
Effective Consolidation Stress (psf)	500		1000		2000	
Major Effective Stress at Failure (psf) σ1	2099		2487		3978	
Minor Effective Stress at Failure (psf) σ3	449		530		1024	
Deviator Stress at Failure (psf)	1650		1957		2954	
Pore Pressure at Failure (psf)	51		470		976	
Failure Sketch	Sketch on Worksheet		Sketch on Worksheet		Sketch on Worksheet	
	ADDITIONAL INFORMATION REQUIRED BY ASTM D 4767					
Classification Based On	Plasticity index, Visual		Plasticity index, Visual		Plasticity index, Visual	
Liquid Limit	53					
Plastic Limit	17					
Remarks	0		0		0	

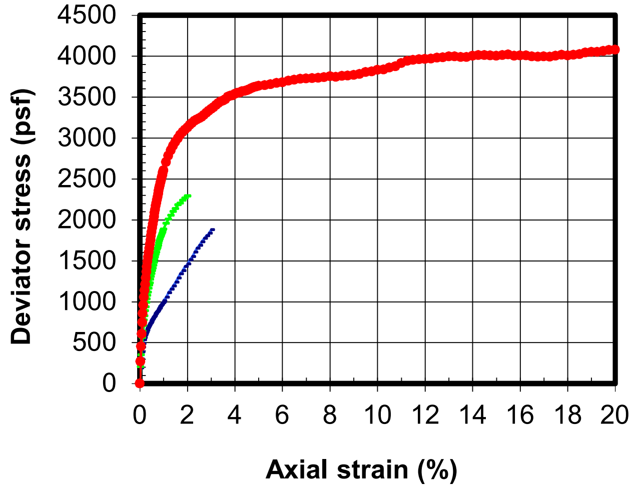
The following information is the same for all samples

Method for Specimen Saturation	Wet
Method used to determine Area after Consolidation	Method A
Failure Criteria	Maximum Effective σ1 / σ3 ratio

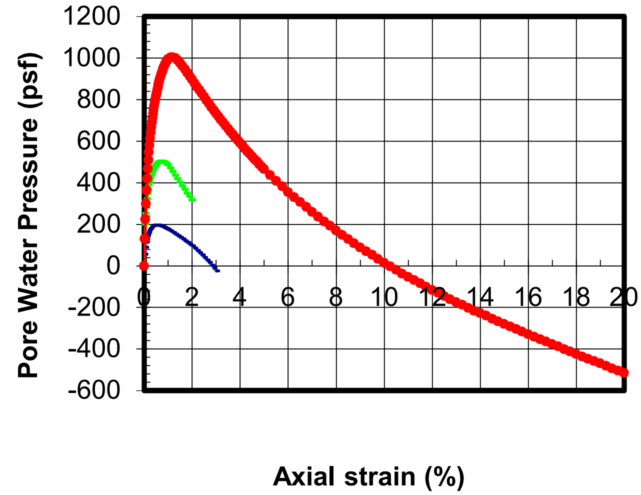
Client: **A3GEO**	Boring #: **B-1**	Sample #: **4**
Project: **Lafayette Outlet Tower Seismic Retrofit**	Depth (ft): **12.5**	
Project #: **1141-4A**	Soil: **Greenish gray clay**	

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

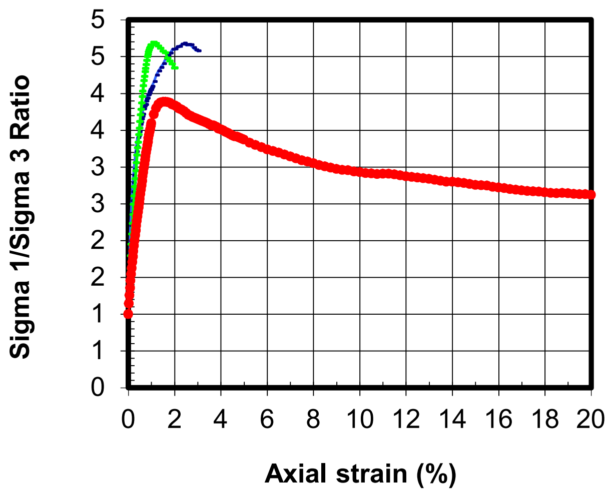
Deviator Stress



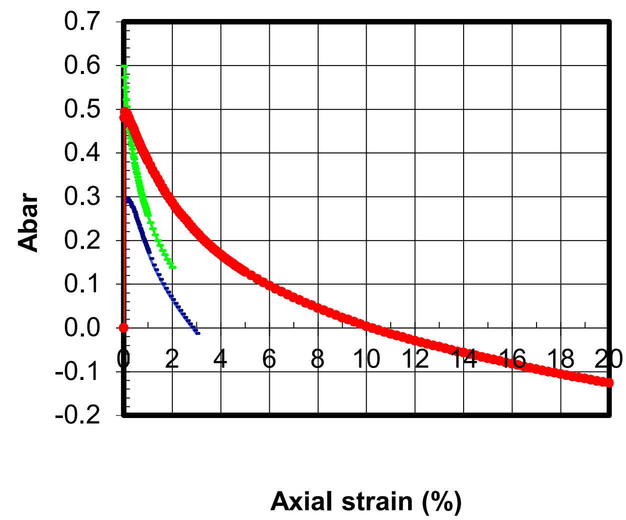
Pore Water Pressure



Sigma 1/Sigma 3 Ratio



Abar



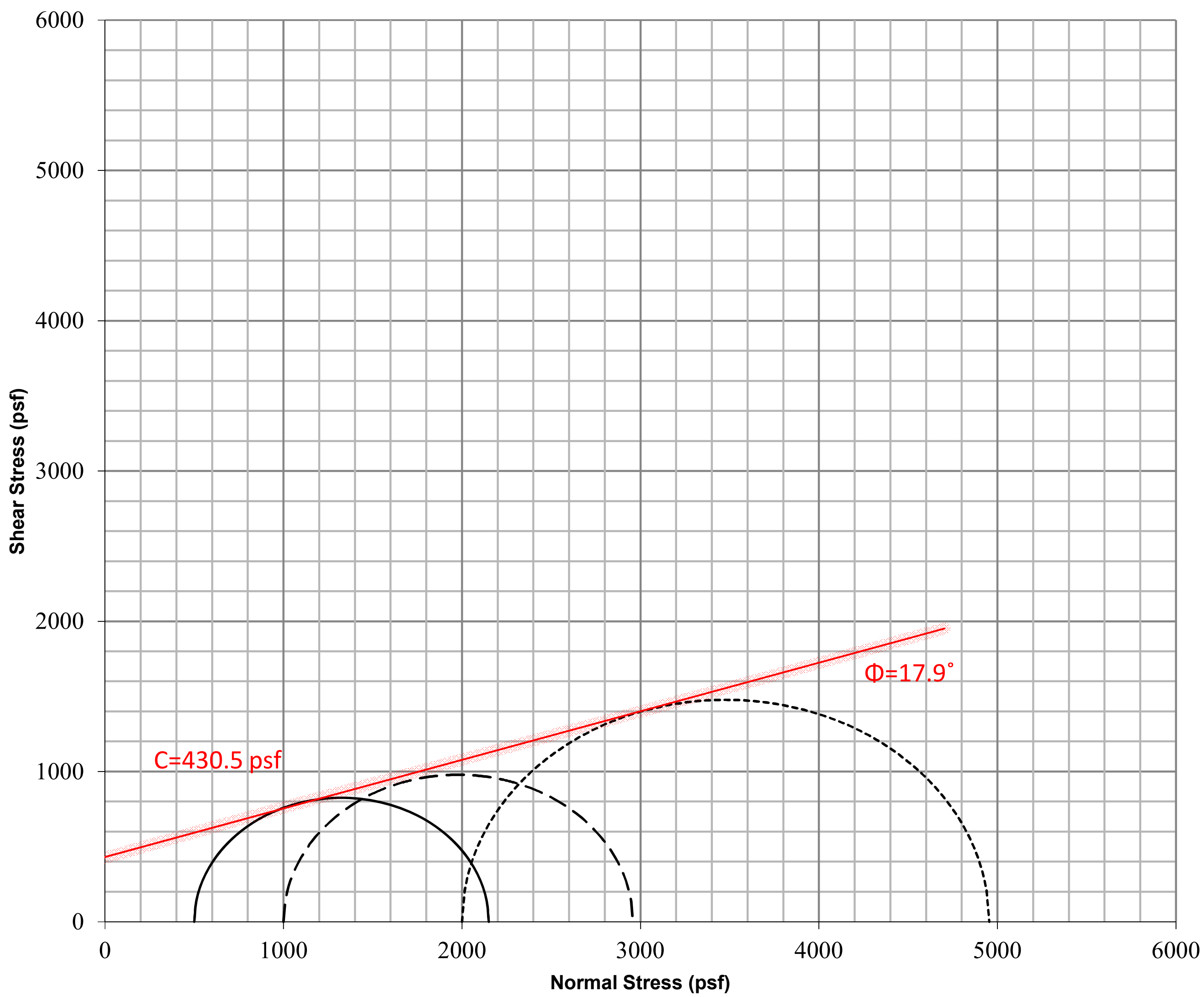
Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Saturation (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1650	2.40	5.98	2.84	20.37	130.3	108.2	0.557	98.7	2.70	0.02	53	17	2.1
dash	1000	1957	1.10	5.98	2.87	20.37	127.5	105.9	0.592	93.0	2.70	0.02			2.1
dot	2000	2954	1.50	5.98	2.89	20.37	126.1	104.7	0.609	90.2	2.70	0.02			2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**4**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**12.5**		
Project #:	**1141-4A**	Soil:	**Greenish gray clay**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

INSPECTION SERVICES INC. **PLATE NUMBER__________**



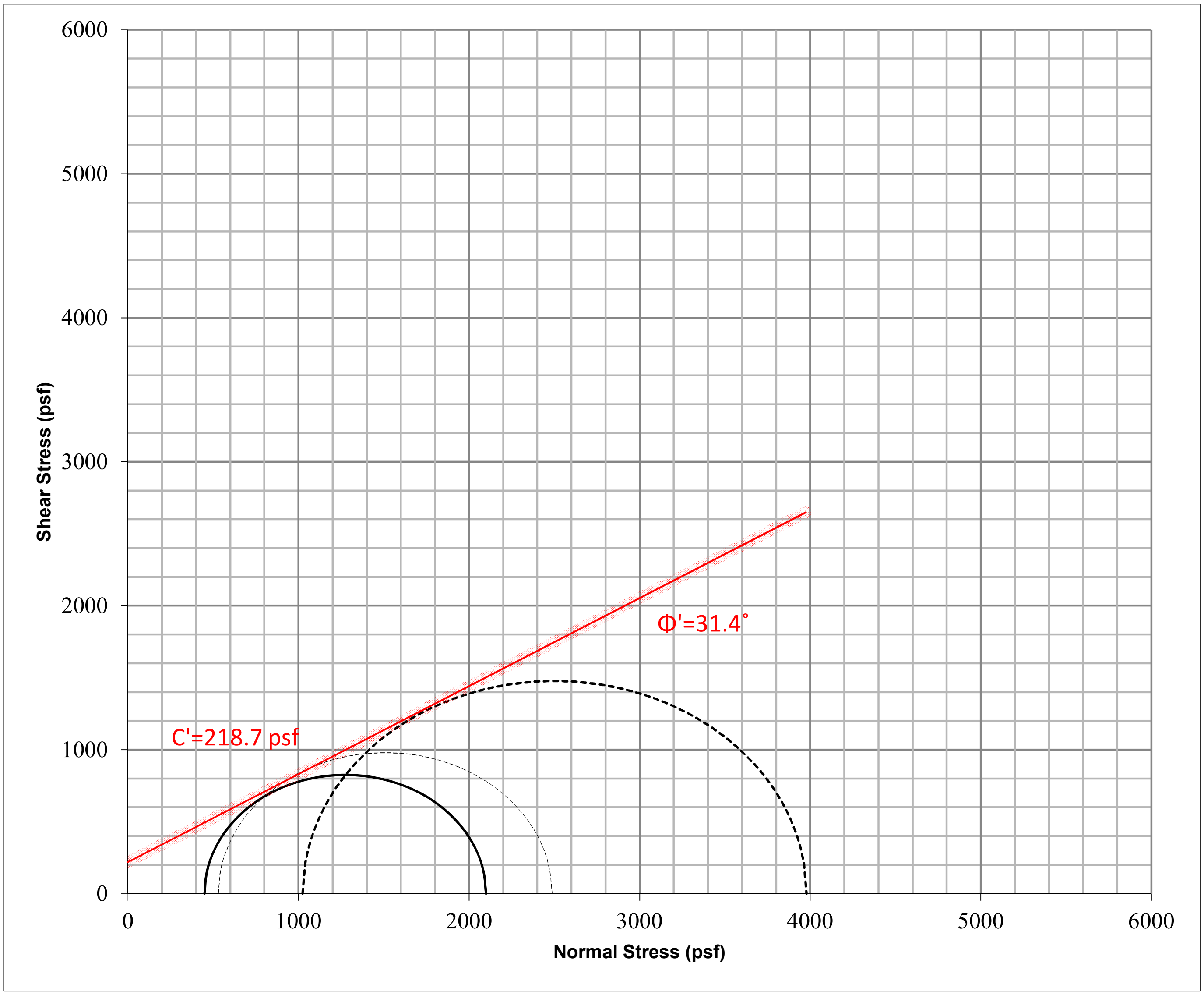
TOTAL MOHR CIRCLES

Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1650	2.40	5.98	2.84	20.37	130.3	108.2	0.557	98.7	2.70	0.02	53	17	2.1
dash	1000	1957	1.10	5.98	2.87	20.37	127.5	105.9	0.592	93.0	2.70	0.02			2.1
dot	2000	2954	1.50	5.98	2.89	20.37	126.1	104.7	0.609	90.2	2.70	0.02			2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**4**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**12.5**		
Project #:	**1141-4A**	Soil:	**Greenish gray clay**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**



EFFECTIVE MOHR CIRCLES

Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1650	2.40	5.98	2.84	20.37	130.3	108.2	0.557	98.7	2.70	0.02	53	17	2.1
dash	1000	1957	1.10	5.98	2.87	20.37	127.5	105.9	0.592	93.0	2.70	0.02			2.1
dot	2000	2954	1.50	5.98	2.89	20.37	126.1	104.7	0.609	90.2	2.70	0.02			2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**4**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**12.5**		
Project #:	**1141-4A**	Soil:	**Greenish gray clay**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

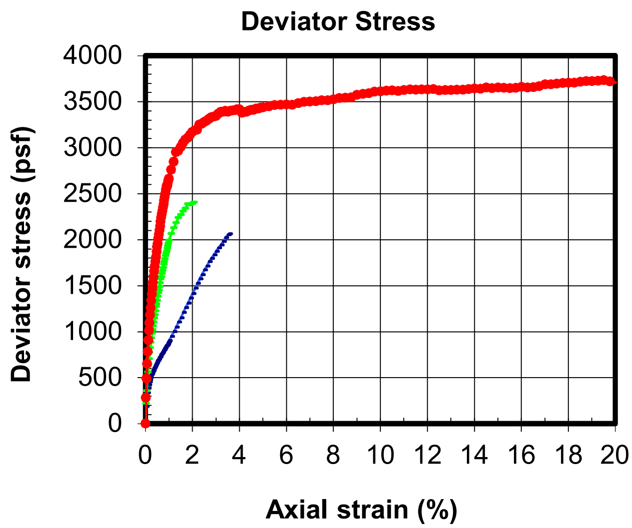
INSPECTION SERVICES INC. **PLATE NUMBER__________**

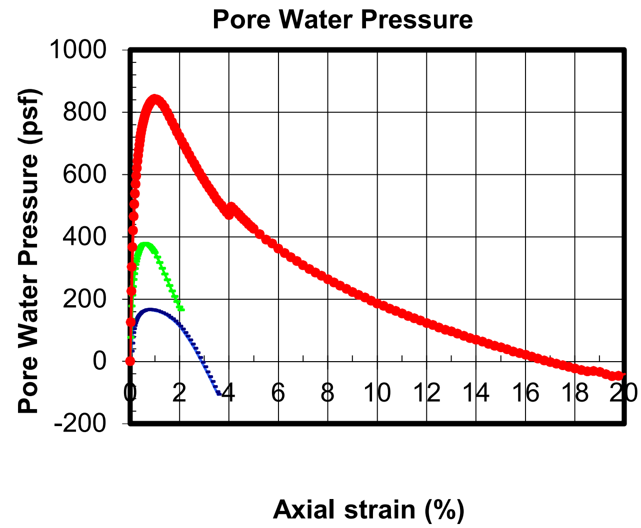
Boring Number	B-1		B-1		B-1	
Sample Number	6		6		6	
Depth (ft)	18		18		18	
Date Tested	10/19/18		10/20/18		10/22/18	
Description	Greenish gray clay with sand		Greenish gray clay with sand		Greenish gray clay with sand	
Sample Condition	Undisturbed		Undisturbed		Undisturbed	
	Initial	**After Consolidation**	**Initial**	**After Consolidation**	**Initial**	**After Consolidation**
Height (in)	5.98	5.98	5.98	5.83	5.98	5.77
Diameter (in)	2.84	2.84	2.88	2.90	2.89	2.92
Height/Diameter Ratio	2.11		2.08		2.07	
Total Weight (g)	1287.01	1284.97	1287.01	1284.97	1287.01	1284.97
Moisture Content (%)	21.96	21.76	21.96	21.76	21.96	21.76
Moisture Content From	entire sample		entire sample		entire sample	
Wet Density (pcf)	129.43	129.62	126.03	126.79	124.64	126.74
Dry Density (pcf)	106.13	106.45	103.34	104.13	102.20	104.09
Area (cm^2)	40.87	40.78	41.97	42.74	42.44	43.21
Total Volume (cc)	620.77	618.87	637.49	632.69	644.60	632.90
Void Ratio	0.5882	0.5834	0.6310	0.6187	0.6492	0.6193
Saturation (%)	100.8	100.7	93.9	95.0	91.3	94.9
Specific Gravity	2.70		2.70		2.70	
Specific Gravity From	Assumption		Assumption		Assumption	
B value Before Consolidation	0.96		0.96		0.96	
Total Back Pressure (psf)	7200		7200		7200	
Rate of Strain (%/min)	0.02		0.02		0.02	
Axial Strain at Failure (%)	2.50		1.20		1.30	
Effective Consolidation Stress (psf)	500		1000		2000	
Major Effective Stress at Failure (psf) σ1	2115		2813		4126	
Minor Effective Stress at Failure (psf) σ3	445		684		1174	
Deviator Stress at Failure (psf)	1670		2129		2952	
Pore Pressure at Failure (psf)	55		316		826	
Failure Sketch	Sketch on Worksheet		Sketch on Worksheet		Sketch on Worksheet	
	ADDITIONAL INFORMATION REQUIRED BY ASTM D 4767					
Classification Based On	Visual		Visual		Visual	
Liquid Limit						
Plastic Limit						
Remarks	0		0		0	

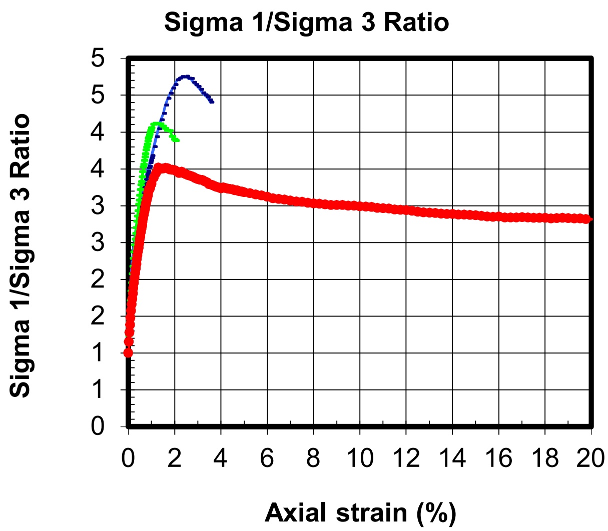
The following information is the same for all samples	
Method for Specimen Saturation	Wet
Method used to determine Area after Consolidation	Method A
Failure Criteria	Maximum Effective σ1 / σ3 ratio

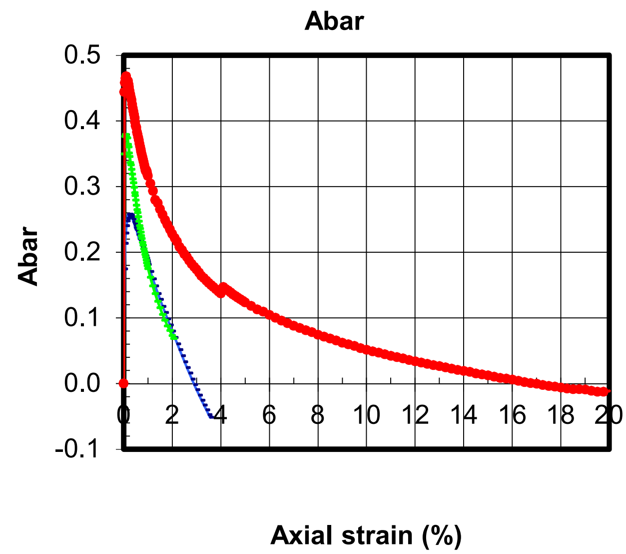
Client: **A3GEO**	Boring #: **B-1**		Sample #: **6**
Project: **Lafayette Outlet Tower Seismic Retrofit**	Depth (ft): **18**		
Project #: **1141-4A**	Soil: **Greenish gray clay with sand**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**









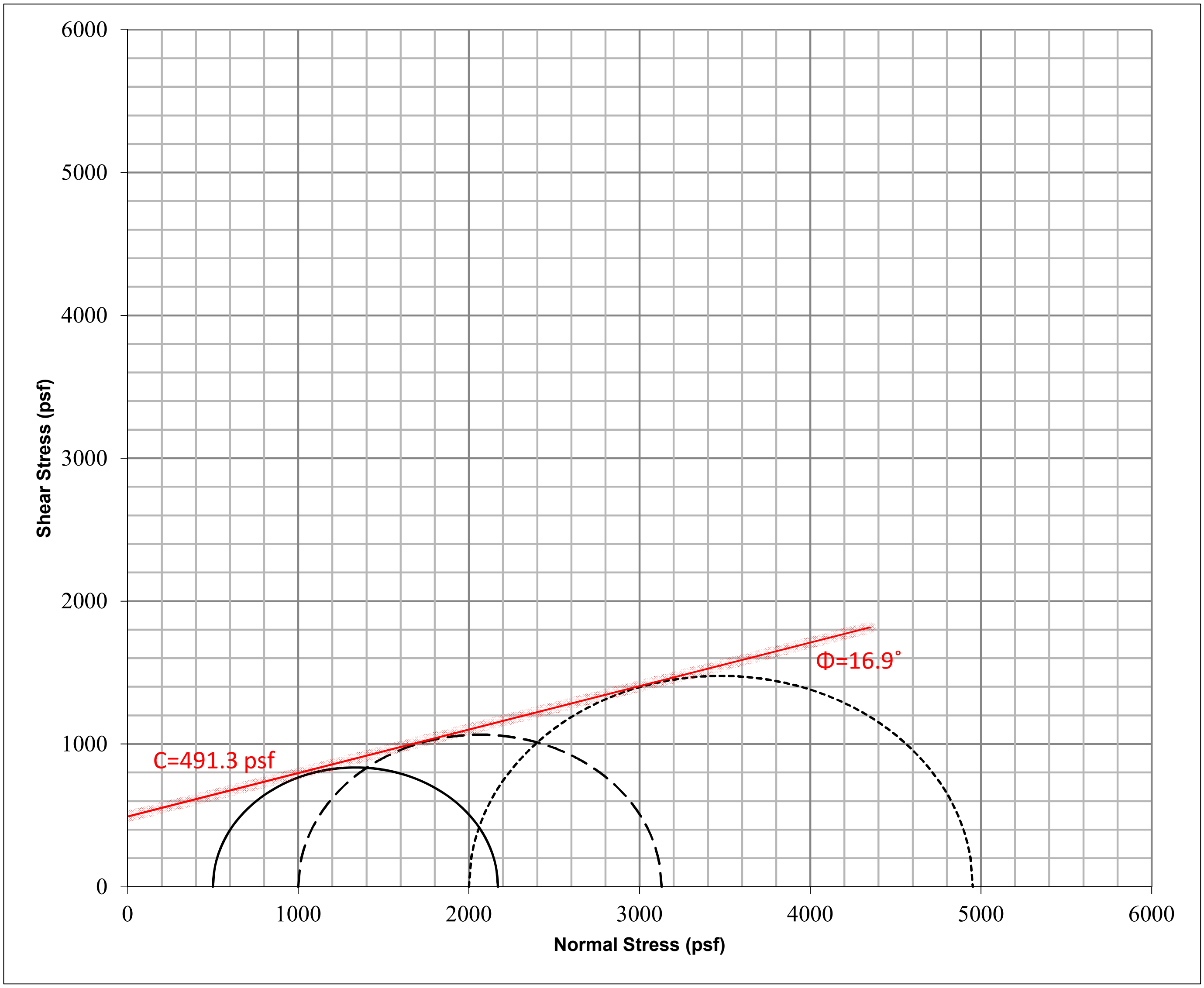
Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Saturation (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1670	2.50	5.98	2.84	21.96	129.4	106.1	0.588	100.8	2.70	0.02			2.1
dash	1000	2129	1.20	5.98	2.88	21.96	126.0	103.3	0.631	93.9	2.70	0.02			2.1
dot	2000	2952	1.30	5.98	2.89	21.96	124.6	102.2	0.649	91.3	2.70	0.02			2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**6**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**18**		
Project #:	**1141-4A**	Soil:	**Greenish gray clay with sand**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

INSPECTION SERVICES INC. **PLATE NUMBER__________**



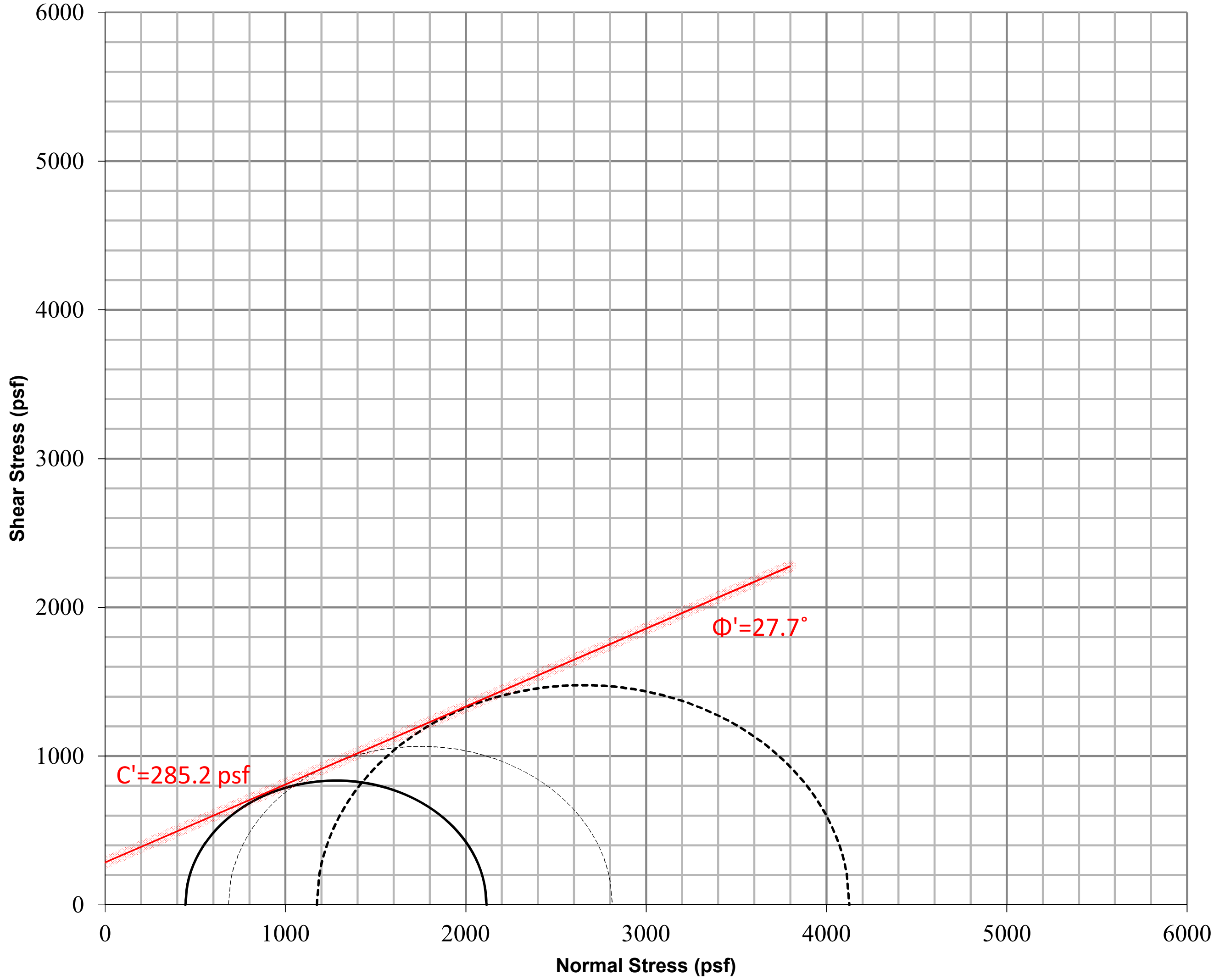
TOTAL MOHR CIRCLES

Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1670	2.50	5.98	2.84	21.96	129.4	106.1	0.588	100.8	2.70	0.02			2.1
dash	1000	2129	1.20	5.98	2.88	21.96	126.0	103.3	0.631	93.9	2.70	0.02			2.1
dot	2000	2952	1.30	5.98	2.89	21.96	124.6	102.2	0.649	91.3	2.70	0.02			2.1

Client: **A3GEO**	Boring #: **B-1**		Sample #: **6**
Project: **Lafayette Outlet Tower Seismic Retrofit**	Depth (ft): **18**		
Project #: **1141-4A**	Soil: **Greenish gray clay with sand**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**



EFFECTIVE MOHR CIRCLES

Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	500	1670	2.50	5.98	2.84	21.96	129.4	106.1	0.588	100.8	2.70	0.02			2.1
dash	1000	2129	1.20	5.98	2.88	21.96	126.0	103.3	0.631	93.9	2.70	0.02			2.1
dot	2000	2952	1.30	5.98	2.89	21.96	124.6	102.2	0.649	91.3	2.70	0.02			2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**6**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**18**		
Project #:	**1141-4A**	Soil:	**Greenish gray clay with sand**		

ASTM D-4767	**STAGED TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

INSPECTION SERVICES INC. **PLATE NUMBER__________**

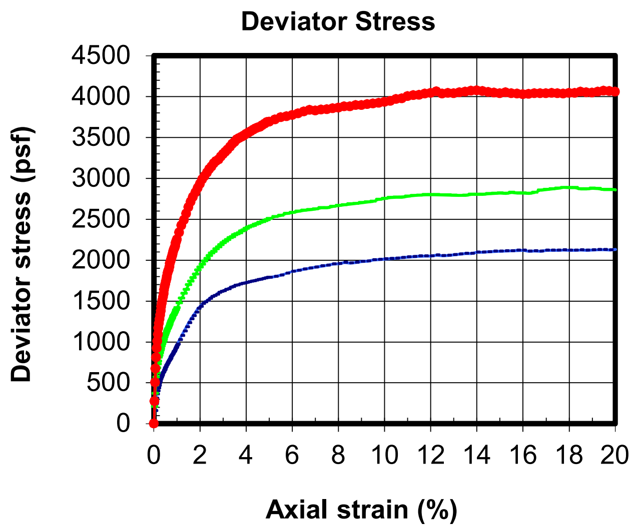
Boring Number	B-1		B-1		B-1	
Sample Number	10		10		10	
Depth (ft)	27.5		27.5		27.5	
Date Tested	10/31/18		10/30/18		10/29/18	
Description	Grayish brown clay		Grayish brown clay		Grayish brown clay	
Sample Condition	Undisturbed		Undisturbed		Undisturbed	
	Initial	**After Consolidation**	**Initial**	**After Consolidation**	**Initial**	**After Consolidation**
Height (in)	6.10	6.08	6.10	6.06	6.10	6.03
Diameter (in)	2.86	2.86	2.86	2.85	2.86	2.83
Height/Diameter Ratio	2.13		2.13		2.13	
Total Weight (g)	1325.12	1332.40	1314.57	1316.93	1321.27	1314.84
Moisture Content (%)	21.15	21.81	21.34	21.55	20.48	19.90
Moisture Content From	entire sample		entire sample		entire sample	
Wet Density (pcf)	128.82	130.30	127.79	130.13	128.44	131.97
Dry Density (pcf)	106.33	106.97	105.32	107.05	106.61	110.07
Area (cm^2)	41.45	41.36	41.45	41.04	41.45	40.60
Total Volume (cc)	642.17	638.37	642.17	631.77	642.17	621.97
Void Ratio	0.5852	0.5758	0.6004	0.5745	0.5811	0.5313
Saturation (%)	97.6	102.3	96.0	101.3	95.2	101.1
Specific Gravity	2.70		2.70		2.70	
Specific Gravity From	Assumption		Assumption		Assumption	
B value Before Consolidation	0.98		0.98		0.98	
Total Back Pressure (psf)	7200		7200		7200	
Rate of Strain (%/min)	0.02		0.02		0.02	
Axial Strain at Failure (%)	1.90		3.00		3.70	
Effective Consolidation Stress (psf)	750		1500		3000	
Major Effective Stress at Failure (psf) σ1	1894		3088		5052	
Minor Effective Stress at Failure (psf) σ3	486		879		1559	
Deviator Stress at Failure (psf)	1407		2210		3493	
Pore Pressure at Failure (psf)	264		621		1441	
Failure Sketch	Sketch on Worksheet		Sketch on Worksheet		Sketch on Worksheet	
	ADDITIONAL INFORMATION REQUIRED BY ASTM D 4767					
Classification Based On	Plasticity index, Visual		Plasticity index, Visual		Plasticity index, Visual	
Liquid Limit	44		44		44	
Plastic Limit	18		18		18	
Remarks	0		0		0	

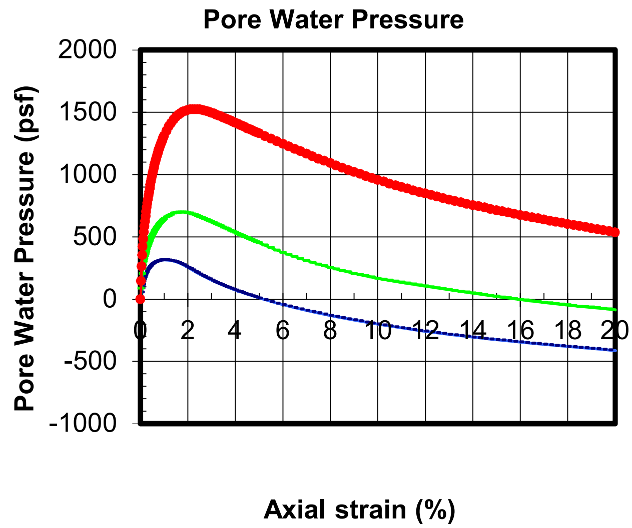
The following information is the same for all samples

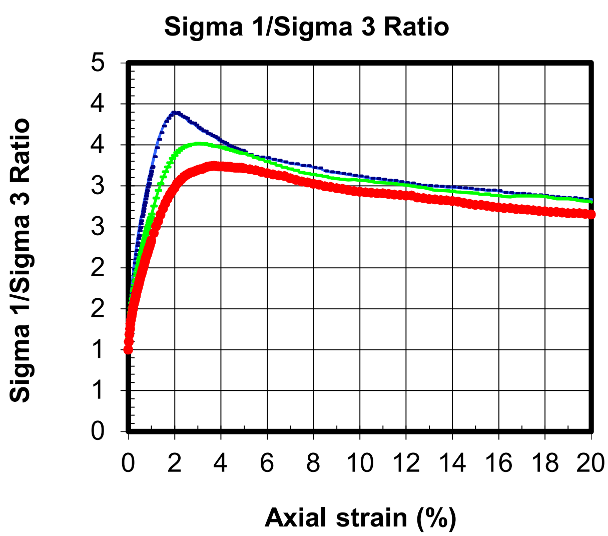
Method for Specimen Saturation	Wet
Method used to determine Area after Consolidation	Method A
Failure Criteria	Maximum Effective σ1 / σ3 ratio

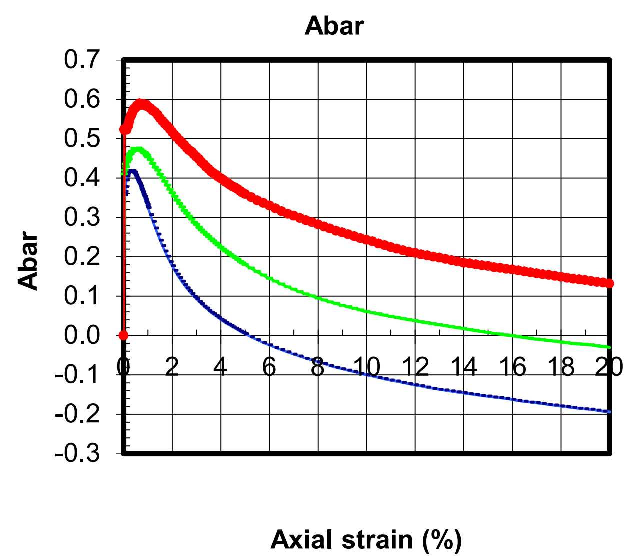
Client: **A3GEO**	Boring #: **B-1**		Sample #: **10**
Project: **Lafayette Outlet Tower Seismic Retrofit**	Depth (ft): **27.5**		
Project #: **1141-4A**	Soil: **Grayish brown clay**		

ASTM D-4767	**TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**







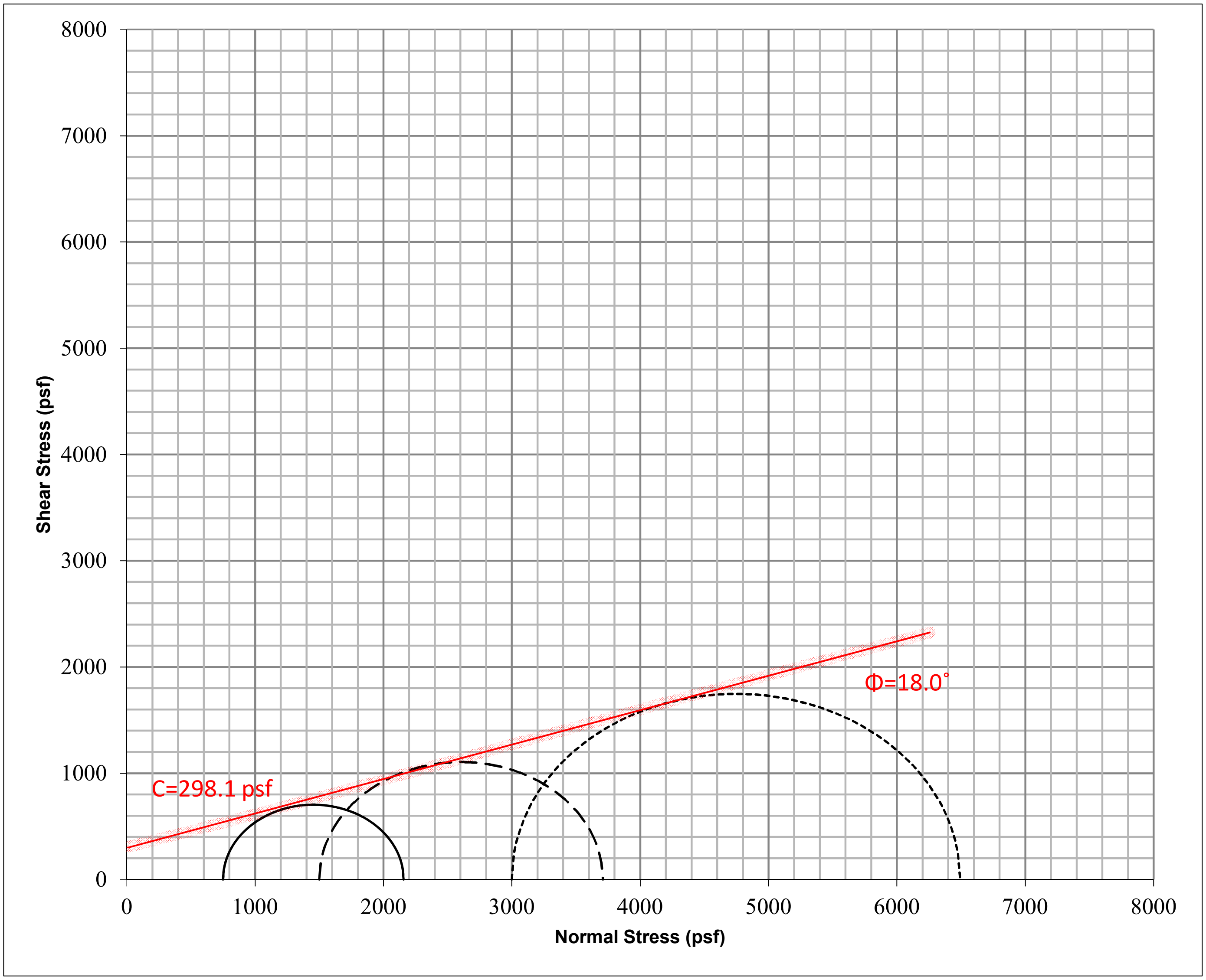


Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Saturation (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	750	1407	1.90	6.10	2.86	21.15	128.8	106.3	0.585	97.6	2.70	0.02	44	18	2.1
dash	1500	2210	3.00	6.10	2.86	21.34	127.8	105.3	0.600	96.0	2.70	0.02	44	18	2.1
dot	3000	3493	3.70	6.10	2.86	20.48	128.4	106.6	0.581	95.2	2.70	0.02	44	18	2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**10**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**27.5**		
Project #:	**1141-4A**	Soil:	**Grayish brown clay**		

ASTM D-4767	**TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**



TOTAL MOHR CIRCLES

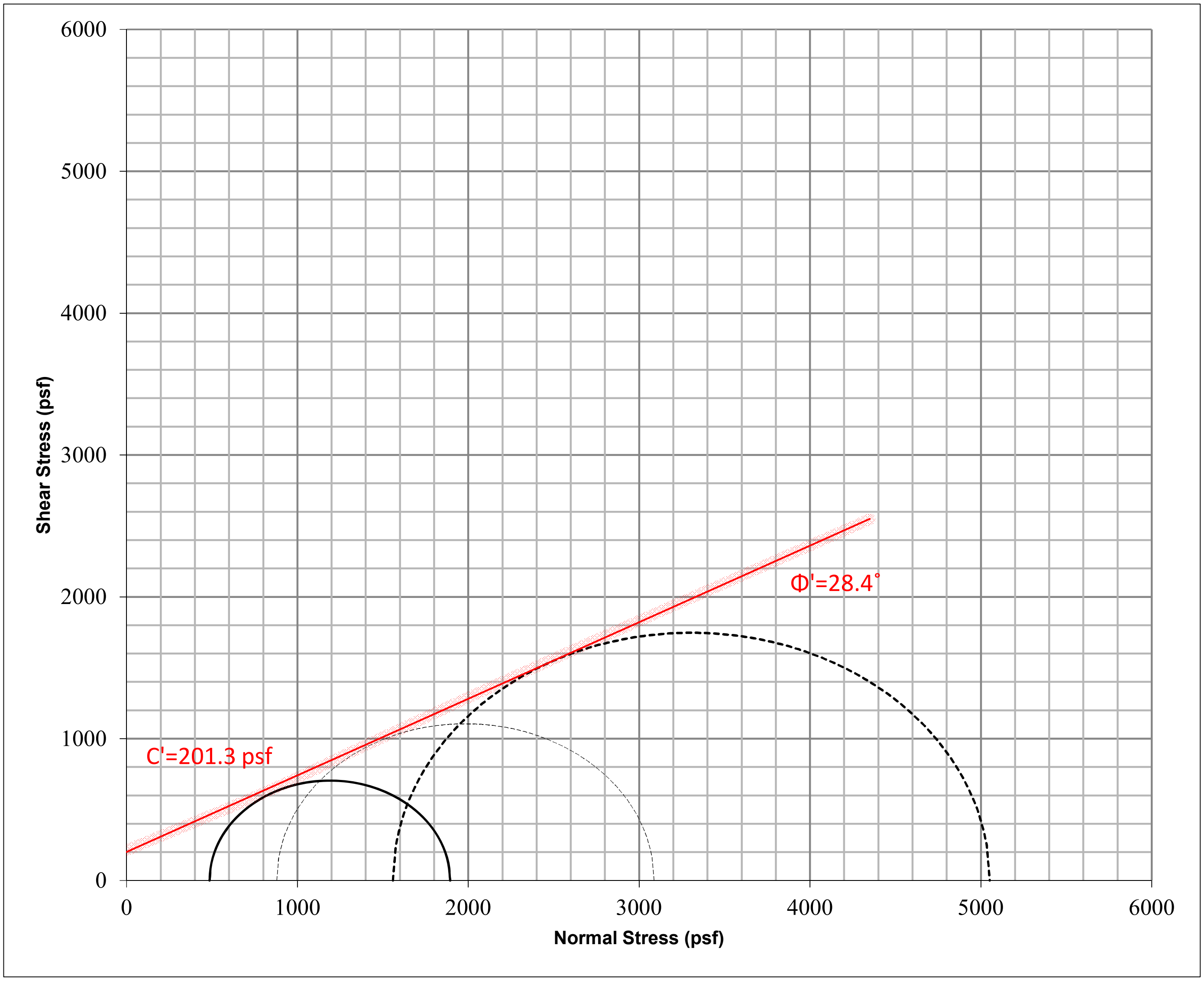
Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	750	1407	1.90	6.10	2.86	21.15	128.8	106.3	0.585	97.6	2.70	0.02	44	18	2.1
dash	1500	2210	3.00	6.10	2.86	21.34	127.8	105.3	0.600	96.0	2.70	0.02	44	18	2.1
dot	3000	3493	3.70	6.10	2.86	20.48	128.4	106.6	0.581	95.2	2.70	0.02	44	18	2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**10**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**27.5**		
Project #:	**1141-4A**	Soil:	**Grayish brown clay**		

ASTM D-4767	**TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

INSPECTION SERVICES INC. **PLATE NUMBER_________**



EFFECTIVE MOHR CIRCLES

Failure Criteria Maximum Effective σ1 / σ3 ratio

Line Type	Minor Principal Stress at failure (psf) σ_3	Maximum Deviator Stress at failure (psf)	Axial Strain at Failure (%)	Initial Height (in.)	Initial Diam. (in.)	Initial Moisture Content (%)	Initial Wet Density (pcf)	Initial Dry Density (pcf)	Initial Void Ratio	Initial Satura-tion (%)	Specific Gravity (assumed)	Rate of Strain (%/min)	Liquid Limit	Plastic Limit	Height to Diameter Ratio
solid	750	1407	1.90	6.10	2.86	21.15	128.8	106.3	0.585	97.6	2.70	0.02	44	18	2.1
dash	1500	2210	3.00	6.10	2.86	21.34	127.8	105.3	0.600	96.0	2.70	0.02	44	18	2.1
dot	3000	3493	3.70	6.10	2.86	20.48	128.4	106.6	0.581	95.2	2.70	0.02	44	18	2.1

Client:	**A3GEO**	Boring #:	**B-1**	Sample #:	**10**
Project:	**Lafayette Outlet Tower Seismic Retrofit**	Depth (ft):	**27.5**		
Project #:	**1141-4A**	Soil:	**Grayish brown clay**		

ASTM D-4767	**TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED**	**TXCU**

UNCONFINED COMPRESSION TEST
ASTM D-2166

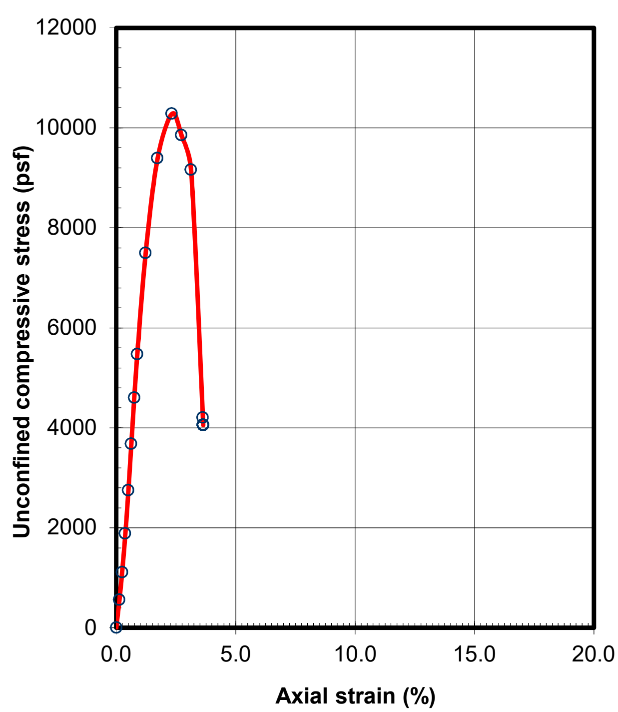
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 15
Depth (ft) : 51.5
Date tested : 10/20/18
Soil : Undisturbed mottled brown rock core

Specimen:

Total wt. =	911.62	gms
Ht. =	5.62	in
Ave dia. =	2.41	in
Area =	4.55	sq.in
Volume =	419.42	c.c.
Shearing rate =	0.22	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.425	
Ht/Dia ratio =	2.34	
Moisture =	14.70	%
Total density =	135.69	pcf
Dry density =	118.30	pcf
Saturation =	93.4	%
Unconfined compress. strength =	10279	psf
Strain @ failure =	2.33	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0042	18.12	0.0	0.00
0.0111	35.73	556.6	0.12
0.0181	53.32	1110.8	0.25
0.0252	77.89	1883.8	0.37
0.0323	105.46	2749.0	0.50
0.0393	135.16	3679.2	0.62
0.0464	164.63	4600.0	0.75
0.0534	192.65	5472.6	0.87
0.0731	257.92	7492.7	1.23
0.1012	320.21	9391.1	1.73
0.1350	350.79	10278.7	2.33
0.1575	338.22	9849.7	2.73
0.1800	316.98	9158.4	3.13
0.2081	155.95	4201.8	3.63
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64
0.2086	151.21	4057.0	3.64

UNCONFINED COMPRESSION TEST
ASTM D-2166

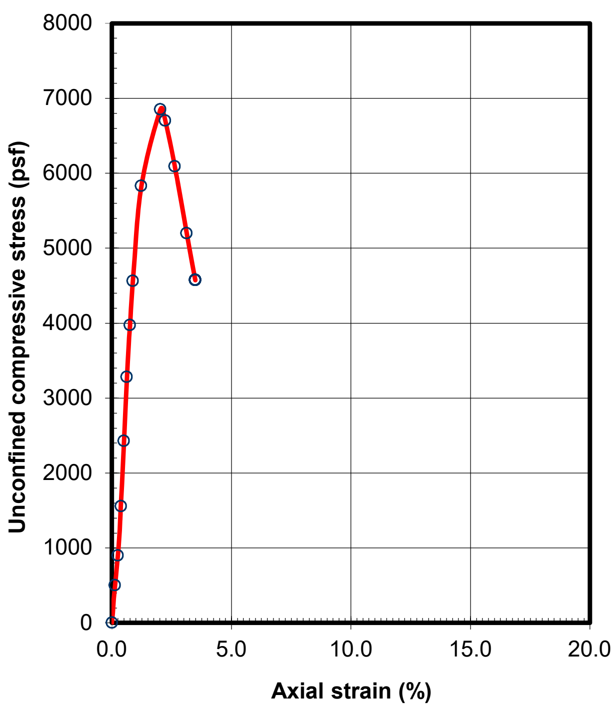
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 17
Depth (ft) : 60.5
Date tested : 10/22/18
Soil : Undisturbed grayish brown rock core

Specimen:

Total wt. =	856.11	gms
Ht. =	5.18	in
Ave dia. =	2.43	in
Area =	4.62	sq.in
Volume =	392.24	c.c.
Shearing rate =	0.21	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.426	
Ht/Dia ratio =	2.14	
Moisture =	15.24	%
Total density =	136.26	pcf
Dry density =	118.24	pcf
Saturation =	96.7	%
Unconfined compress. strength =	6850	psf
Strain @ failure =	2.03	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0039	13.98	0.0	0.00
0.0103	30.04	500.0	0.12
0.0168	42.89	899.1	0.25
0.0233	64.09	1556.4	0.37
0.0299	92.18	2426.0	0.50
0.0364	119.89	3281.2	0.63
0.0428	142.33	3971.5	0.75
0.0493	161.59	4561.8	0.88
0.0675	203.22	5827.5	1.23
0.1090	238.24	6850.2	2.03
0.1194	233.85	6702.1	2.23
0.1401	214.53	6088.3	2.63
0.1660	186.05	5196.8	3.13
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48
0.1844	165.93	4572.4	3.48

UNCONFINED COMPRESSION TEST
ASTM D-2166

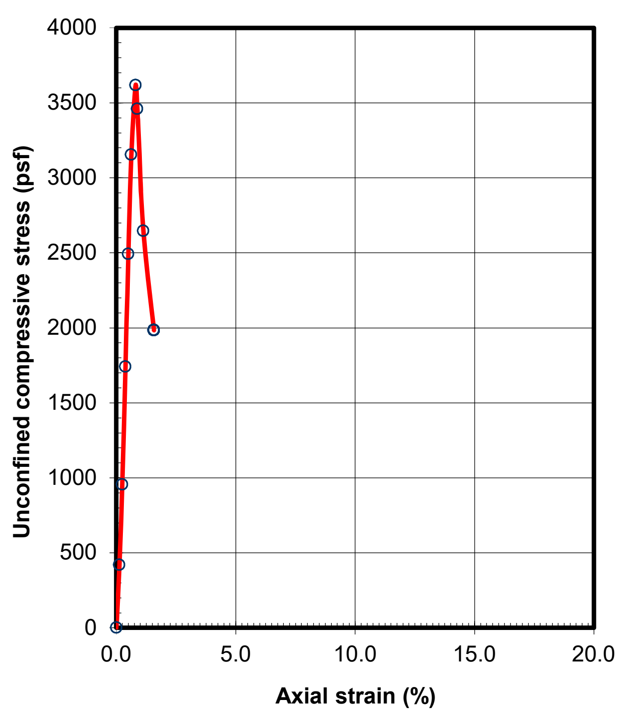
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 19
Depth (ft) : 69.7
Date tested : 10/22/18
Soil : Undisturbed greenish gray rock core

Specimen:

Total wt. =	1093.03	gms
Ht. =	5.97	in
Ave dia. =	2.52	in
Area =	4.98	sq.in
Volume =	487.18	c.c.
Shearing rate =	0.24	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.357	
Ht/Dia ratio =	2.37	
Moisture =	12.79	%
Total density =	140.07	pcf
Dry density =	124.18	pcf
Saturation =	96.6	%
Unconfined compress. strength =	3617	psf
Strain @ failure =	0.80	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0045	16.63	0.0	0.00
0.0119	31.14	419.0	0.13
0.0194	49.73	954.9	0.25
0.0270	77.02	1740.3	0.38
0.0345	103.21	2492.0	0.50
0.0419	126.36	3154.4	0.63
0.0525	142.68	3616.9	0.80
0.0570	137.29	3459.8	0.88
0.0719	109.17	2646.7	1.13
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57
0.0985	86.32	1984.3	1.57

UNCONFINED COMPRESSION TEST
ASTM D-2166

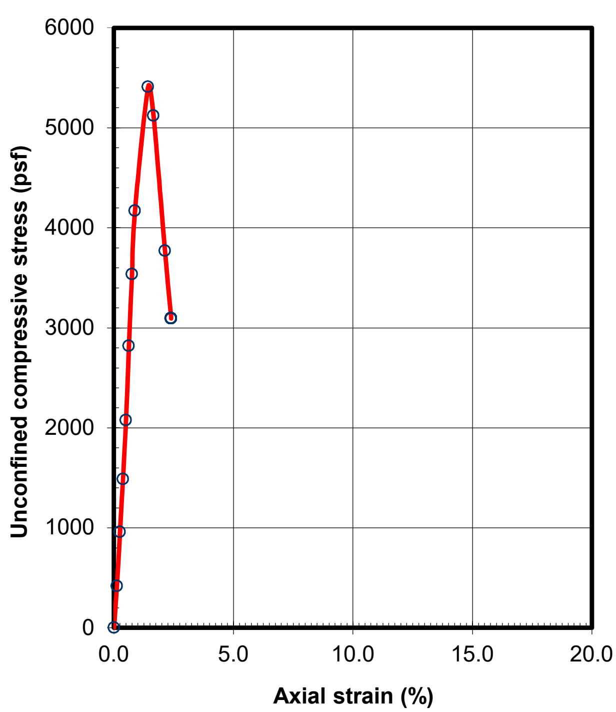
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 21
Depth (ft) : 82
Date tested : 10/20/18
Soil : Undisturbed greenish gray rock core

Specimen:

Total wt. =	810.72	gms
Ht. =	5.03	in
Ave dia. =	2.41	in
Area =	4.57	sq.in
Volume =	376.60	c.c.
Shearing rate =	0.20	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.439	
Ht/Dia ratio =	2.08	
Moisture =	14.73	%
Total density =	134.40	pcf
Dry density =	117.14	pcf
Saturation =	90.6	%
Unconfined compress. strength =	5408	psf
Strain @ failure =	1.43	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0038	9.80	0.0	0.00
0.0101	23.06	417.2	0.13
0.0163	40.35	959.9	0.25
0.0227	57.14	1485.8	0.38
0.0290	76.00	2074.9	0.50
0.0353	99.88	2820.0	0.63
0.0416	122.94	3537.4	0.75
0.0479	143.33	4169.5	0.88
0.0756	183.97	5408.4	1.43
0.0871	175.15	5122.6	1.66
0.1109	132.11	3770.8	2.13
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39
0.1240	110.41	3093.7	2.39

UNCONFINED COMPRESSION TEST
ASTM D-2166

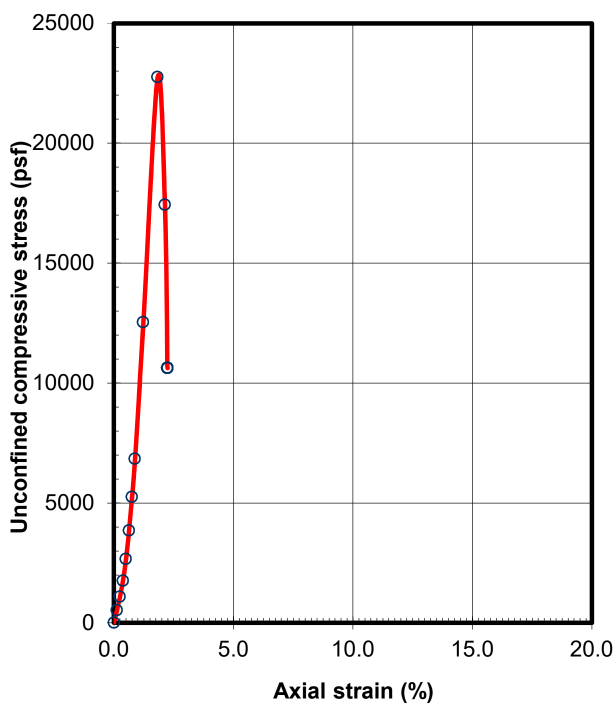
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 22
Depth (ft) : 87.5
Date tested : 10/20/18
Soil : Undisturbed gray rock core

Specimen:

Total wt. =	941.56	gms
Ht. =	5.65	in
Ave dia. =	2.40	in
Area =	4.53	sq.in
Volume =	419.73	c.c.
Shearing rate =	0.23	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.284	
Ht/Dia ratio =	2.35	
Moisture =	6.67	%
Total density =	140.05	pcf
Dry density =	131.29	pcf
Saturation =	63.5	%
Unconfined compress. strength =	22747	psf
Strain @ failure =	1.83	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0041	14.39	0.0	0.00
0.0113	30.99	526.9	0.13
0.0184	48.43	1078.8	0.25
0.0255	69.92	1757.3	0.38
0.0327	98.57	2660.6	0.50
0.0397	136.30	3848.2	0.63
0.0468	180.89	5249.2	0.75
0.0540	231.29	6829.0	0.88
0.0737	413.79	12530.8	1.23
0.1077	743.86	22746.9	1.83
0.1246	575.11	17431.4	2.13
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24
0.1306	356.38	10620.2	2.24

UNCONFINED COMPRESSION TEST
ASTM D-2166

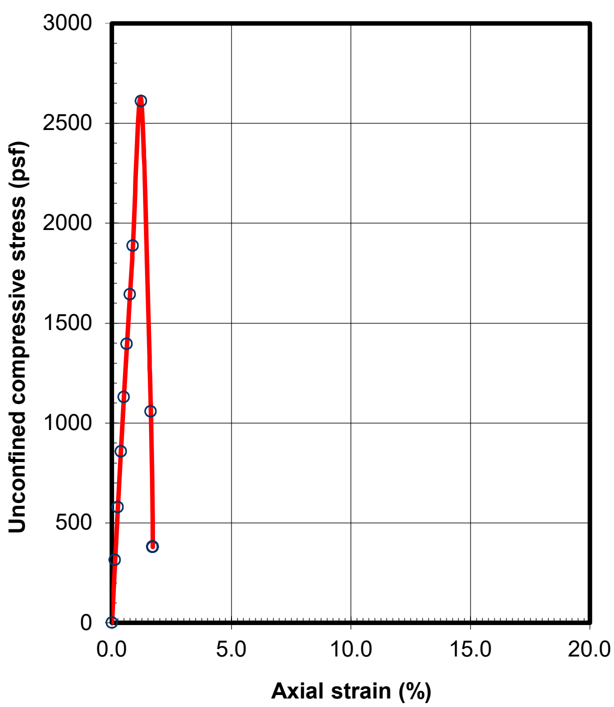
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 24
Depth (ft) : 102
Date tested : 10/20/18
Soil : Undisturbed gray rock core

Specimen:

Total wt. =	986.67	gms
Ht. =	5.63	in
Ave dia. =	2.41	in
Area =	4.54	sq.in
Volume =	419.30	c.c.
Shearing rate =	0.23	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.256	
Ht/Dia ratio =	2.34	
Moisture =	9.45	%
Total density =	146.91	pcf
Dry density =	134.22	pcf
Saturation =	99.7	%
Unconfined compress. strength =	2610	psf
Strain @ failure =	1.23	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0041	13.01	0.0	0.00
0.0112	22.92	313.8	0.13
0.0183	31.29	578.2	0.25
0.0254	40.14	856.9	0.38
0.0324	48.82	1129.5	0.50
0.0395	57.31	1395.6	0.63
0.0465	65.26	1644.0	0.75
0.0535	73.04	1886.2	0.88
0.0733	96.38	2610.3	1.23
0.0958	46.89	1056.5	1.63
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71
0.1005	25.20	380.0	1.71

UNCONFINED COMPRESSION TEST
ASTM D-2166

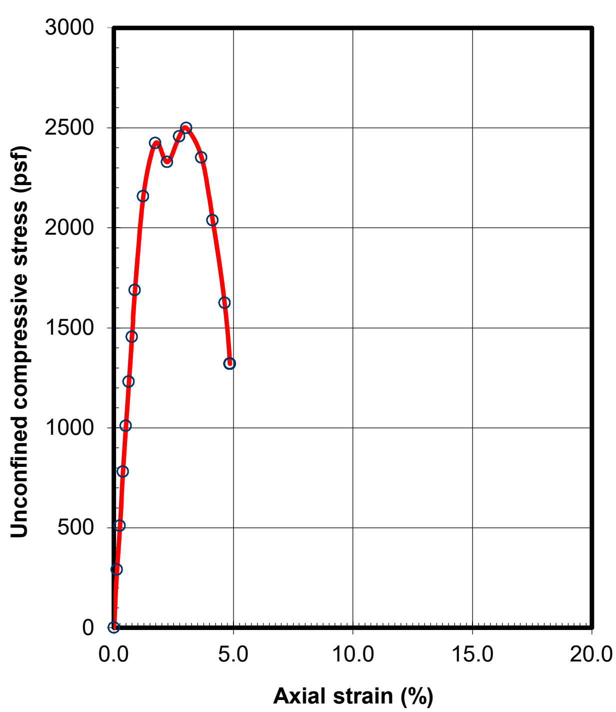
Client : A3GEO
Project Name : Lafayette Outlet Tower Seismic Retrofit
Project Number : 1141-4A
Boring Number B-1
Sample Number : 25
Depth (ft) : 116
Date tested : 10/22/18
Soil : Undisturbed greenish gray rock core

Specimen:

Total wt. =	838.90	gms
Ht. =	5.16	in
Ave dia. =	2.43	in
Area =	4.62	sq.in
Volume =	390.73	c.c.
Shearing rate =	0.21	inch/min
Shearing rate =	0.25	%/min
Gs (assumed) =	2.70	

Test Report:

Void ratio =	0.445	
Ht/Dia ratio =	2.13	
Moisture =	14.87	%
Total density =	134.04	pcf
Dry density =	116.69	pcf
Saturation =	90.3	%
Unconfined compress. strength =	2498	psf
Strain @ failure =	3.03	%



(see worksheet for sketch of failure)

Data Reduction:
(For Graph)

Dial Read.	Load Read.	Deviator Stress (psf)	Axial Strain (%)
0.0038	10.83	0.0	0.00
0.0104	20.17	290.7	0.13
0.0168	27.22	509.5	0.25
0.0233	35.96	780.5	0.38
0.0297	43.37	1009.3	0.50
0.0363	50.53	1229.8	0.63
0.0427	57.84	1454.6	0.75
0.0492	65.46	1688.3	0.88
0.0673	80.90	2157.5	1.23
0.0931	89.90	2422.6	1.73
0.1189	87.24	2329.1	2.23
0.1448	91.83	2456.2	2.73
0.1604	93.47	2498.4	3.03
0.1932	89.10	2350.7	3.67
0.2172	78.98	2036.9	4.13
0.2430	65.43	1623.4	4.63
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85
0.2543	55.32	1319.7	4.85

APPENDIX E

Borings by Others

DRILL RIG BARGE-MOUNTED B-40	HOLE ELEVATION 377'±	LOGGED BY DH-LA
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 4"	DATE DRILLED AUG. 29-SEPT. 7,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
0	CL	0-97.5 ALLUVIUM.		RD	Drilled from barge in reservoir. Water elevation 446'±. Set 81' casing from barge, so that it was 12' below reservoir bottom.
5					
10					
		0-1: CLAY, grayish brown; soft; moist; very plastic. Blocky structure, breaks along old contraction cracks.	0-1 13-15'	P	
15			SPT-1 15-16.5'	DR	3/.5,7/.5,9/.5
				RD	
20		CLAYEY SILT, olive green, sandy.	0-2 19.5-21.5'	P	HOLE ORIGINALLY LOGGED BY EBMUD PERSONNEL IN THE FIELD. ADDITIONAL DESCRIPTIONS OF EXTRUDED TUBE SAMPLES BY WAWA PERSONNEL.
			SPT-2 21.5-23'	DR	4/.5,5/.5 Test incomplete: cat line fouled pulling hammer and rods 3'+ out of hole.
25					

W.A. WAHLER & ASSOCIATES	LAFAYETTE DAM	SOIL EXPLORATION DRILL HOLE LOG	HOLE NO
PALO ALTO • NEWPORT BEACH • CALIF		PROJECT NO. 0722 · DATE OCT. 1973 · SHEET NO. 1 OF 5	SS-25

DRILL RIG BARGE-MOUNTED B-40	HOLE ELEVATION 377'±	LOGGED BY DH-ASB
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 4"	DATE DRILLED AUG. 29-SEPT. 7,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
25		ALLUVIUM--CONTINUED CLAYEY SILT--CONTINUED		RD	
		0-3: Layers of: SILTY CLAY, mottled gray and green; very plastic; damp; stiff; contains some fine sand.	0-3 28-29'	P	
30		SANDY CLAY, mottled greenish gray and yellowish green; damp; firm to stiff; moderately plastic; 0-5% coarse sand.	SPT-3 29-31.5'	DR	4/.5,6/.5,7/.5
				RD	
35					
		SILTY SAND,AND SILTY CLAY mixture, medium gray.	0-4 36.5-38.5'	P	
			SPT-4 38.5-40'	DR	9/.5,12/.5,14/.5
40				RD	
45					
		As above.	0-5 45-47'	P	
			SPT-5 47-48.5'	DR	6/.5,9/.5,13/.5
				RD	
50					

W.A. WAHLER & ASSOCIATES	LAFAYETTE DAM	SOIL EXPLORATION DRILL HOLE LOG	HOLE NO
PALO ALTO • NEWPORT BEACH • CALIF		PROJECT NO. 0722 · DATE OCT. 1973 · SHEET NO. 2 OF 5	SS-25

DRILL RIG	BARGE-MOUNTED B-40	HOLE ELEVATION	377'±	LOGGED BY	DH-LA
GROUNDWATER DEPTH (BELOW GROUND SURFACE)	NOT ENCOUNTERED	HOLE DIAMETER	4"	DATE DRILLED	AUG. 29-SEPT. 7,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
50		ALLUVIUM--CONTINUED		RD	
		0-6: SANDY CLAY, yellow brown to green gray; stiff; moist; moderately plastic. Contains root remnants and carbonaceous matter. Few, very weathered shale granules.	0-6 53.5-55.5'	P	
55			SPT-6 55.5-57'	DR	9/.5,12/.5,12/.5
				RD	
60					
			0-7 63-64.2'	P	
65		SILTY CLAY, gray.	SPT-7 64.2-65.7'	DR	9/.5,9/.5,10/.5
				RD	
70		CLAYEY SILT, gray; slightly sandy.	0-8 70.5-72.5'	P	
			SPT-8 72.5-74'	DR	11/.5,11/.5,14/.5
75				RD	

W A WAHLER & ASSOCIATES — PALO ALTO • NEWPORT BEACH • CALIF.

LAFAYETTE DAM

SOIL EXPLORATION DRILL HOLE LOG

PROJECT NO.	DATE	SHEET NO.	HOLE NO
0722	OCT. 1973	3 of 5	SS-25

DRILL RIG	BARGE-MOUNTED B-40	HOLE ELEVATION	377'±	LOGGED BY	DH
GROUNDWATER DEPTH (BELOW GROUND SURFACE)	NOT ENCOUNTERED	HOLE DIAMETER	4"	DATE DRILLED	AUG. 29-SEPT. 7,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
75		ALLUVIUM--CONTINUED		RD	
		Layers of: SILTY CLAY, gray, some gravel to 3/8" ø and wood fragments.	0-9 79-81'	P	
80		Sandy, clayey.	SPT-9 81-82.5'	DR	8/.5,11/.5,14/.5
				RD	
85					
		SANDY CLAY, silty, gray.	0-10 87.5-89.5'	P	
90			SPT-10 89.5-91'	DR	5/.5,7/.5,11/.5
		Gravel to 3/8" ø at ±91'.			
		92.5-107.0 <u>ORINDA FORMATION</u>. MUDSTONE, 96.0-97.7,, gray, poorly cemented. SILTSTONE, gray, poorly cemented.			
95					
			PB-1 96-97.7'	PB	Refusal of Pitcher Barrel at 97.7'. Rotary drilled to 98'.
			PB-2 98-100.3'	PB	
100					

W A WAHLER & ASSOCIATES — PALO ALTO • NEWPORT BEACH • CALIF.

LAFAYETTE DAM

SOIL EXPLORATION DRILL HOLE LOG

PROJECT NO.	DATE	SHEET NO.	HOLE NO
0722	OCT. 1973	4 of 5	SS-25

DRILL RIG BARGE-MOUNTED B-40	HOLE ELEVATION 377'±	LOGGED BY DH
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 4"	DATE DRILLED AUG. 29-SEPT. 7,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
100		ORINDA FORMATION--CONTINUED			
		SILTSTONE, gray, poorly cemented.	PB-3 100.3-102.9'	PB	
			PB-4 102.9-105.4'	PB	
105			PB-5 105.4-107'	PB	Terminated hole.
		BOTTOM OF HOLE = 107.0 FEET			
110					

W A WAHLER & ASSOCIATES	LAFAYETTE DAM PALO ALTO • NEWPORT BEACH • CALIF	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO SS-25
		PROJECT NO. 0722	DATE OCT. 1973	SHEET NO. 5 of 5	

DRILL RIG	BARGE-MOUNTED B-40	HOLE ELEVATION	369'±	LOGGED BY	DH
GROUNDWATER DEPTH (BELOW GROUND SURFACE)	NOT ENCOUNTERED	HOLE DIAMETER	4"	DATE DRILLED	SEPT. 10-18,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
0	CL	0-58.0 ALLUVIUM.		RD	
		CLAYEY SILT, gray, soft.	0-1 2.5-4.5'	P	
5		CLAYEY SAND to SANDY CLAY.	S-1 4.5-6'	DR	4/.5,6/.5,8/.5
				RD	
10		SILTY, CLAYEY SAND, gray, well graded.	S-1 (Shelby) 11-13'	P	
		SANDY CLAY, brown, stiff.	SPT-2 13-14.5'	DR	8/.5,6/.5,5/.5
15				RD	
20			S-2 19.5-21.5'	P	No recovery. Suspect clean sand.
		SILTY SAND, gray, non-plastic fines.	0-2 21.5-23.5'	P	HOLE ORIGINALLY LOGGED BY EBMUD PERSONNEL IN THE FIELD. ADDITIONAL DESCRIPTIONS OF EXTRUDED TUBE SAMPLES BY WAWA PERSONNEL.
25		SANDY SILT and SILTY CLAY, gray.	SPT-3 23.5-25'	DR	

W A WAHLER & ASSOCIATES	LAFAYETTE DAM	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO
	PALO ALTO • NEWPORT BEACH • CALIF.	PROJECT NO. 0722	DATE OCT. 1973	SHEET NO. 1 of 4	SS-27

DRILL RIG	BARGE-MOUNTED B-40	HOLE ELEVATION	369'±	LOGGED BY	DH-LA
GROUNDWATER DEPTH (BELOW GROUND SURFACE)	NOT ENCOUNTERED	HOLE DIAMETER	4"	DATE DRILLED	SEPT. 10-18,1973

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
25		ALLUVIUM--CONTINUED		RD	
			0-3 28-30'	P	
30		SILTY CLAY, gray, slightly sandy, low to moderate plasticity.	SPT-4 30-31.5'	DR	5/.5,7/.5,10/.5
				RD	
35		Alternating layers: SILTY SAND, medium gray. SANDY SILT, gray, w/ coarse sand and clay.	0-4 36.5-38.5'	P	
		SANDY, SILTY CLAY, gray, low plasticity, soft.	SPT-5 38.5-40'	DR	5/.5,6/.5,7/.5
40				RD	
45		0-5: Layers of: SANDY GRAVEL, clayey, to 45.9' brownish gray, w/pebbles to 1/2" ø, damp.	0-5 45-47'	P	
		SANDY SILT, gray green; damp; low plasticity; very stiff; some sand grains to 1/4" ø.	SPT-6 47-48.5'	DR	7/.5,10/.5,11/.5 HOLE ORIGINALLY LOGGED BY EBMUD PERSONNEL IN THE FIELD. ADDITIONAL DESCRIPTIONS OF EXTRUDED TUBE SAMPLES BY WAWA PERSONNEL.
50		Fossil bone at 46.4'.		RD	

W A WAHLER & ASSOCIATES	LAFAYETTE DAM	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO
	PALO ALTO • NEWPORT BEACH • CALIF.	PROJECT NO. 0722	DATE OCT. 1973	SHEET NO. 2 of 4	SS-27

DRILL RIG BARGE-MOUNTED B-40	HOLE ELEVATION 369'±	LOGGED BY DH	
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 4"	DATE DRILLED SEPT. 10-18, 1973	

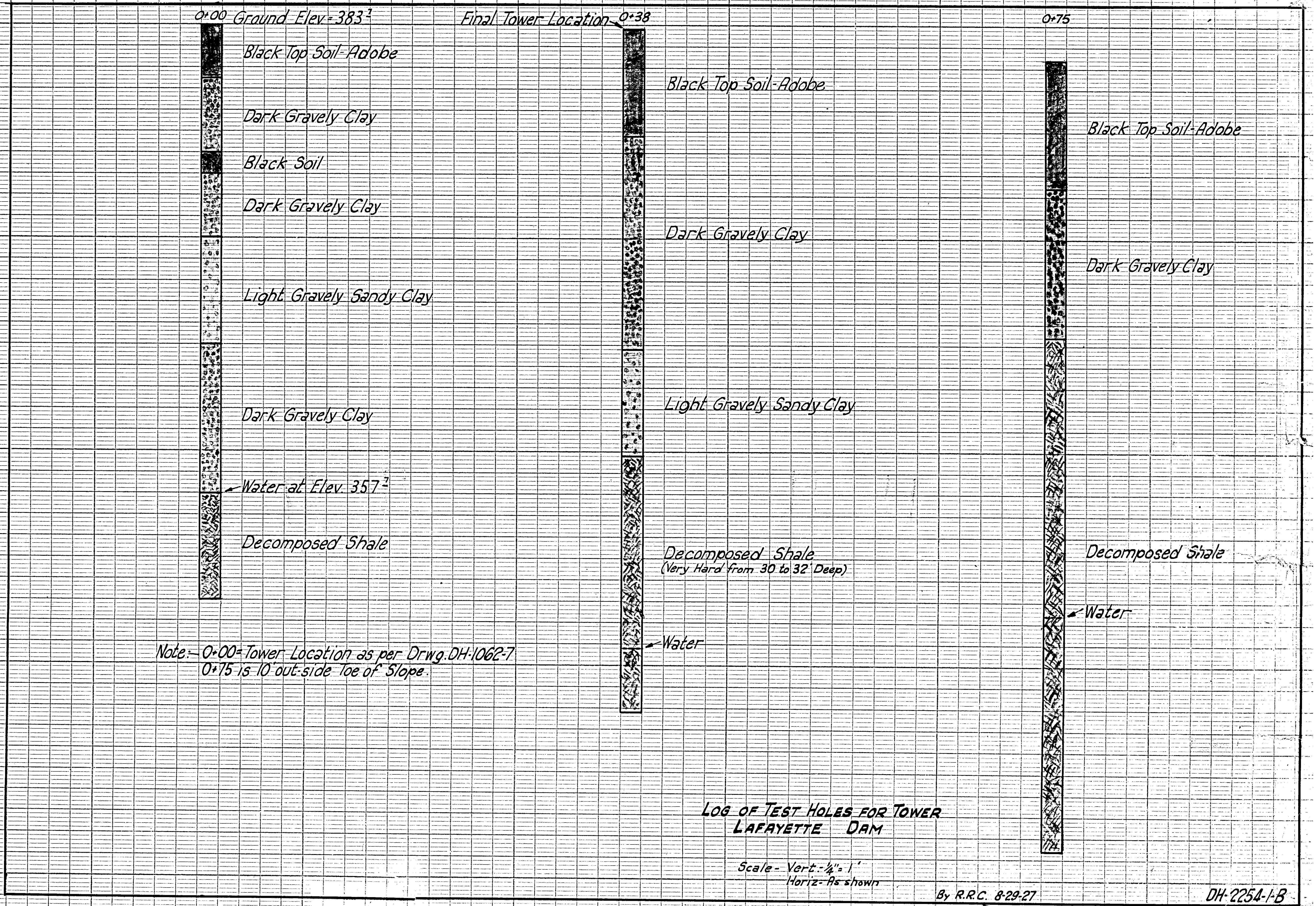
ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
50		ALLUVIUM--CONTINUED		RD	
		SILTY, COARSE SAND, clayey, gray.	O-6 53.5-55.5'	P	
55		SILTY CLAY, gray.	SP-7 55.5-57'	DR	7/.5,8/.5,10/.5
		58.0-77.5 ORINDA FORMATION.		RD	From 58' drilling harder, like through rock or gravel.
60					
		MUDSTONE, gray green, soft, poorly cemented.	O-7 62-63.5'	P	
			SPT-8 63.5-65'	DR	17/.5,17/.5,28/.5
65				RD	
			PB-1 65.5-67.5'	PB	
		SANDSTONE, brown.	C-1	C	
70					
		SILTSTONE, gray, medium hard, weathered fractures.	C-2	C	HOLE ORIGINALLY LOGGED BY EBMUD PERSONNEL IN THE FIELD. ADDITIONAL DESCRIPTIONS OF EXTRUDED TUBE SAMPLES BY WAWA PERSONNEL.
75					

W A WAHLER & ASSOCIATES	LAFAYETTE DAM PALO ALTO • NEWPORT BEACH • CALIF.	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO. SS-27
		PROJECT NO. 0722	DATE OCT. 1973	SHEET NO. 3 of 4	

DRILL RIG BARGE-MOUNTED B-40	HOLE ELEVATION 369'±	LOGGED BY DH	
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 4"	DATE DRILLED SEPT. 10-18, 1973	

ELEVATION (Depth)	CLASS.	DESCRIPTION FIELD IDENTIFICATION	SAMPLE NUMBER	MODE	REMARKS
75		ORINDA FORMATION--CONTINUED SILTSTONE--CONTINUED	C-3	C	Terminated hole.
		BOTTOM OF HOLE = 77.5 FEET			
80					HOLE ORIGINALLY LOGGED BY EBMUD PERSONNEL IN THE FIELD. ADDITIONAL DESCRIPTIONS OF EXTRUDED TUBE SAMPLES BY WAWA PERSONNEL.

W A WAHLER & ASSOCIATES	LAFAYETTE DAM PALO ALTO • NEWPORT BEACH • CALIF.	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO. SS-27
		PROJECT NO. 0722	DATE OCT. 1973	SHEET NO. 4 of 4	



0+00 Ground Elev. = $383^{\underline{7}}$
Black Top Soil-Adobe
Dark Gravely Clay
Black Soil
Dark Gravely Clay
Light Gravely Sandy Clay
Dark Gravely Clay
Water at Elev. $357^{\underline{7}}$
Decomposed Shale
Final Tower Location 0+38
Black Top Soil-Adobe
Dark Gravely Clay
Light Gravely Sandy Clay
Decomposed Shale
(Very Hard from 30 to 32' Deep)
Water
0+75
Black Top Soil-Adobe
Dark Gravely Clay
Decomposed Shale
Water
Note:- 0+00 = Tower Location as per Drwg. DH-1062-7
0+75 is 10' out-side Toe of Slope.
LOG OF TEST HOLES FOR TOWER
LAFAYETTE DAM
Scale - Vert.- ¼" = 1'
Horiz.- As shown
By R.R.C. 8-29-27
DH-2254-1-B

APPENDIX F

Laboratory Tests by Others

W.A. WAHLER & ASSOCIATES

PROJECT NO. 0722

MARCH 1974

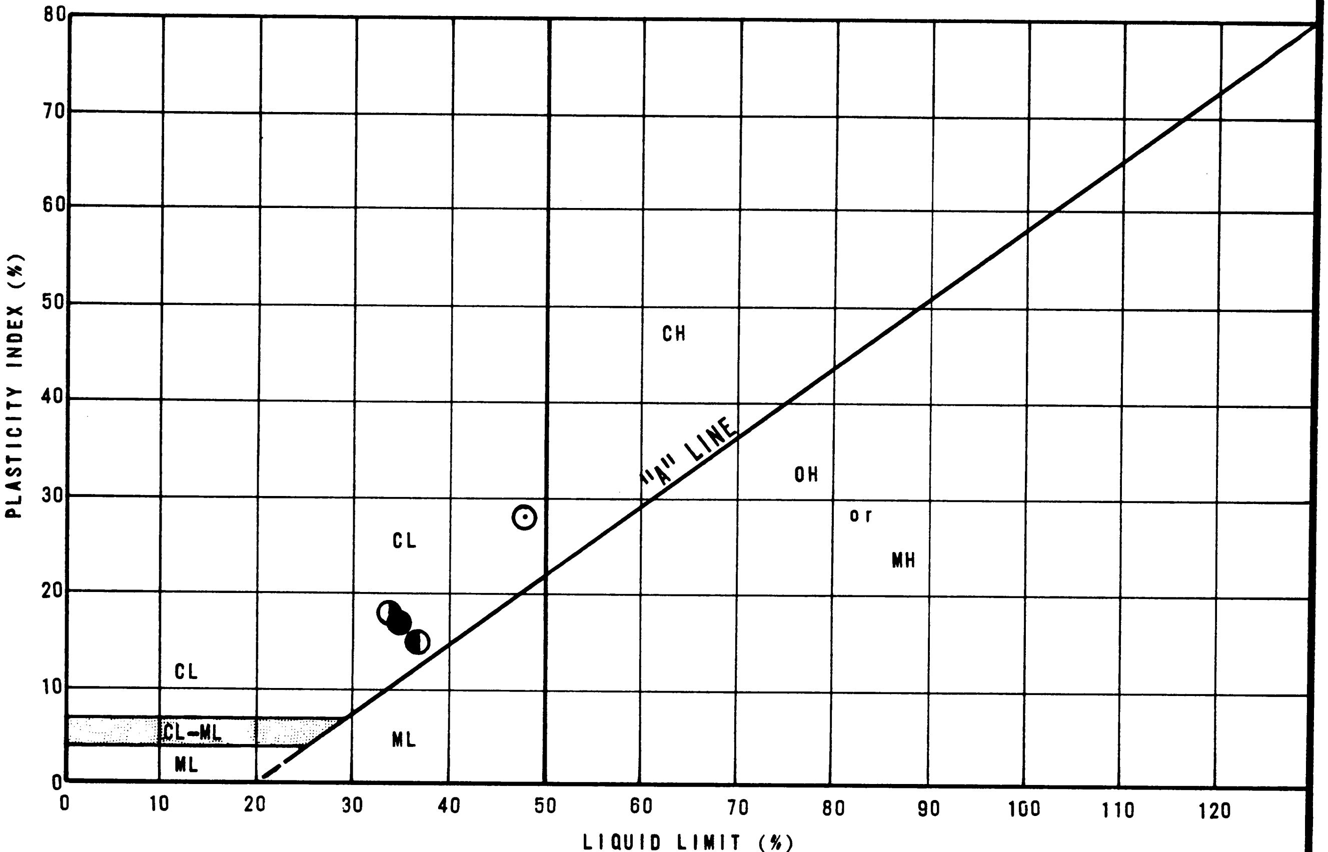
TABLE B-1 — CONTINUED

IN-PLACE WATER CONTENT AND DRY DENSITY DATA

DRILL HOLE NO.	SAMPLE NO.	DEPTH (FT)	WATER CONTENT (%)	DRY DENSITY (pcf)
SS-22	0-1	10-12	18.0	112.1
SS-22A	0-1	23.5-25.5	19.6	110.6
SS-22A	0-2	37-39	16.0	114.9
SS-22A	0-3	50.5-52.5	18.7	109.8
SS-22A	PB-1	86-88.5	18.9	111.9
SS-22A	PB-5	89-91.5	20.8	105.0
SS-22A	PB-5	89-91.5	22.3	102.3
SS-22A	PB-5	89-91.5	17.5	110.9
SS-22A	PB-5	89-91.5	18.0	107.8
SS-22A	PB-8	101.5-104	24.3	99.9
SS-22A	0-5	108-110	26.5	98.6
SS-22A	0-5	108-110	26.6	99.1
SS-22A	0-5	108-110	27.4	96.8
SS-22A	PB-10	113.5-116	20.2	109.8
SS-22A	PB-10	113.5-116	21.5	107.1
SS-22A	PB-12	121.5-124	26.6	98.6
SS-22A	PB-15	132-134.5	21.6	105.3
SS-22A	PB-15	132-134.5	22.8	103.7
SS-22A	PB-18	144-146.5	18.7	112.7
SS-22A	PB-22	160-162.5	17.8	113.6
SS-23	0-2	23.5-25.5	20.1	109.5
SS-23	0-3	37-39	19.5	111.2
SS-23	0-4	50.5-52.5	17.3	114.1
SS-23	0-4	50.5-52.5	20.0	110.2
SS-23	PB-2	78-80.5	18.3	113.2
SS-23	PB-3	87-89.5	22.2	103.6
SS-23	PB-3	87-89.5	23.9	102.8
SS-23	PB-3	87-89.5	22.3	104.5
SS-23	PB-4	96-98.5	21.2	107.4
SS-23	PB-5	105-107.5	23.7	102.7
SS-23	PB-5	105-107.5	23.8	102.8
SS-23	PB-6	114-116.5	21.4	107.3
SS-23	PB-8	125.5-128	25.0	101.7
SS-23	PB-9	134.5-137	23.1	104.7
SS-23	PB-9	134.5-137	22.7	104.9
SS-23	PB-9	134.5-137	24.2	102.8
SS-23	PB-10	143.5-146	19.3	112.0
SS-23	PB-10	143.5-146	20.4	109.5

DRILL HOLE NO.	SAMPLE NO.	DEPTH (FT)	WATER CONTENT (%)	DRY DENSITY (pcf)
SS-24	0-1	19.5-21.5	20.0	110.8
SS-24	0-5	59.5-61.5	22.1	106.0
SS-24	0-6	85-87	25.4	100.6
SS-24	0-6	85-87	26.6	98.6
SS-24	0-6	85-87	26.8	97.2
SS-25	0-1	13-14	26.1	97.2
SS-25	0-3	28-30	23.7	104.2
SS-25	0-6	53.5-55.5	21.2	109.3
SS-25	0-10	87.5-88.8	21.2	107.9
SS-26	0-2	18.5-20.5	21.7	103.9
SS-26	0-2	18.5-20.5	18.9	113.4
SS-26	0-5	44-46	15.9	115.9
SS-26	0-5	44-46	17.6	111.6
SS-26	0-8	69.5-71.5	25.1	---
SS-26	0-8	69.5-71.5	26.3	100.6
SS-27	0-2	21.5-23.5	27.8	97.2
SS-27	0-5	45-47	21.3	108.2
SS-28	PB-2	16.5-19	29.6	93.2
SS-28	PB-2	16.5-19	28.4	95.4
SS-28	PB-3	19-21.5	28.6	94.9
SS-28	PB-3	19-21.5	30.2	92.9
SS-28	PB-3	19-21.5	29.8	94.5
SS-28	PB-4	21.5-24	28.7	94.3
SS-28	PB-4	21.5-24	27.6	96.5
SS-28	PB-5	24-26.5	27.8	96.3
SS-28	PB-5	24-26.5	32.9	90.0
SS-28	PB-5	24-26.5	31.2	92.4
SS-28	PB-6	26.5-29	34.9	87.2
SS-28	PB-6	26.5-29	28.1	96.8
SS-28	PB-7	29-31.5	31.4	91.5
SS-28	PB-7	29-31.5	31.9	90.7
SS-28	PB-7	29-31.5	33.4	89.3
SS-28	PB-7	29-31.5	32.5	90.1

PLASTICITY CHART



PLASTICITY DATA

KEY SYMBOL	HOLE NUMBER	DEPTH (feet)	NATURAL WATER CONTENT W (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX $\left(\frac{W-PL}{LL-PL}\right)$	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	SS-25	28-30	23.7	20	48	28	---	CL
●	SS-25	53.5-55.5	21.2	18	35	17	---	CL
◑	SS-25	87.5-88.8	21.2	16	34	18	---	CL
◐	SS-26	18.5-20.5	20.3(ave.)	22	37	15	---	CL

W. A. WAHLER & ASSOCIATES
PALO ALTO • NEWPORT BEACH • CALIF.

LAFAYETTE DAM

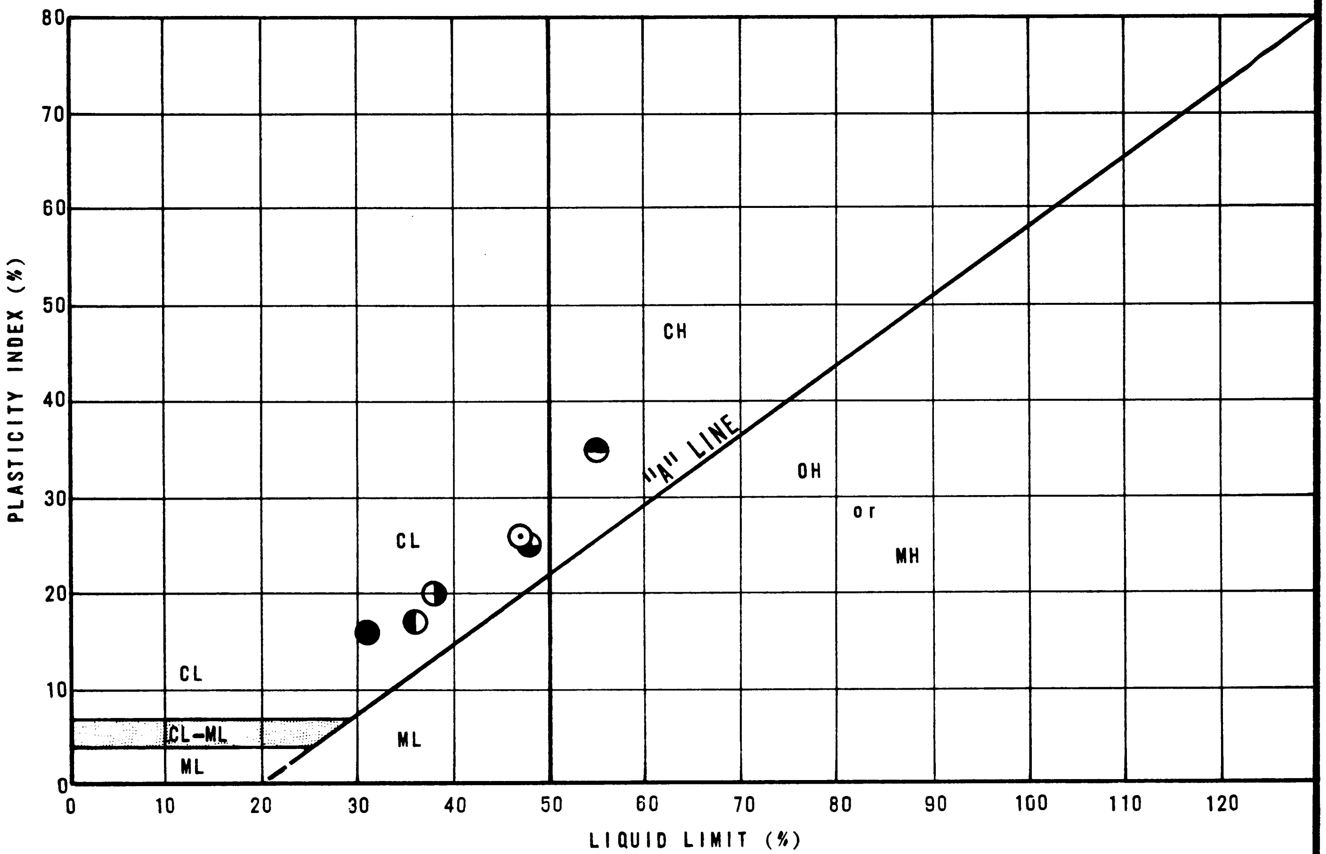
ATTERBERG LIMITS - PLASTICITY DATA

PROJECT NO.	DATE	DRAWING NO.
0722	DECEMBER 1973	B-2

740-3

Sheet 7 of 8

PLASTICITY CHART



PLASTICITY DATA

KEY SYMBOL	HOLE NUMBER	DEPTH (feet)	NATURAL WATER CONTENT W (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX $\left(\frac{W - PL}{LL - PL}\right)$	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	SS-27	21.5-23.5	27.8	21	47	26	---	CL
●	SS-27	45-47	21.3	15	31	16	---	CL
◑	SS-28	16-19.5	29.0(ave.)	18	38	20	---	CL
◐	SS-28	21.5-24	28.2(ave.)	19	36	17	---	CL
◒	SS-28	24-26.5	30.6(ave.)	23	48	25	---	CL
◓	SS-28	29-31.5	32.3(ave.)	20	55	35	---	CH

740-3

W. A. WAHLER & ASSOCIATES

LAFAYETTE DAM

PALO ALTO • NEWPORT BEACH • CALIF.

ATTERBERG LIMITS - PLASTICITY DATA

PROJECT NO.	DATE	DRAWING NO.
0722	DECEMBER 1973	B-2

W.A. WAHLER & ASSOCIATES

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LAFAYETTE DAM

GRADATION TEST RESULTS

HOLE SS-25

PROJECT NO 8722

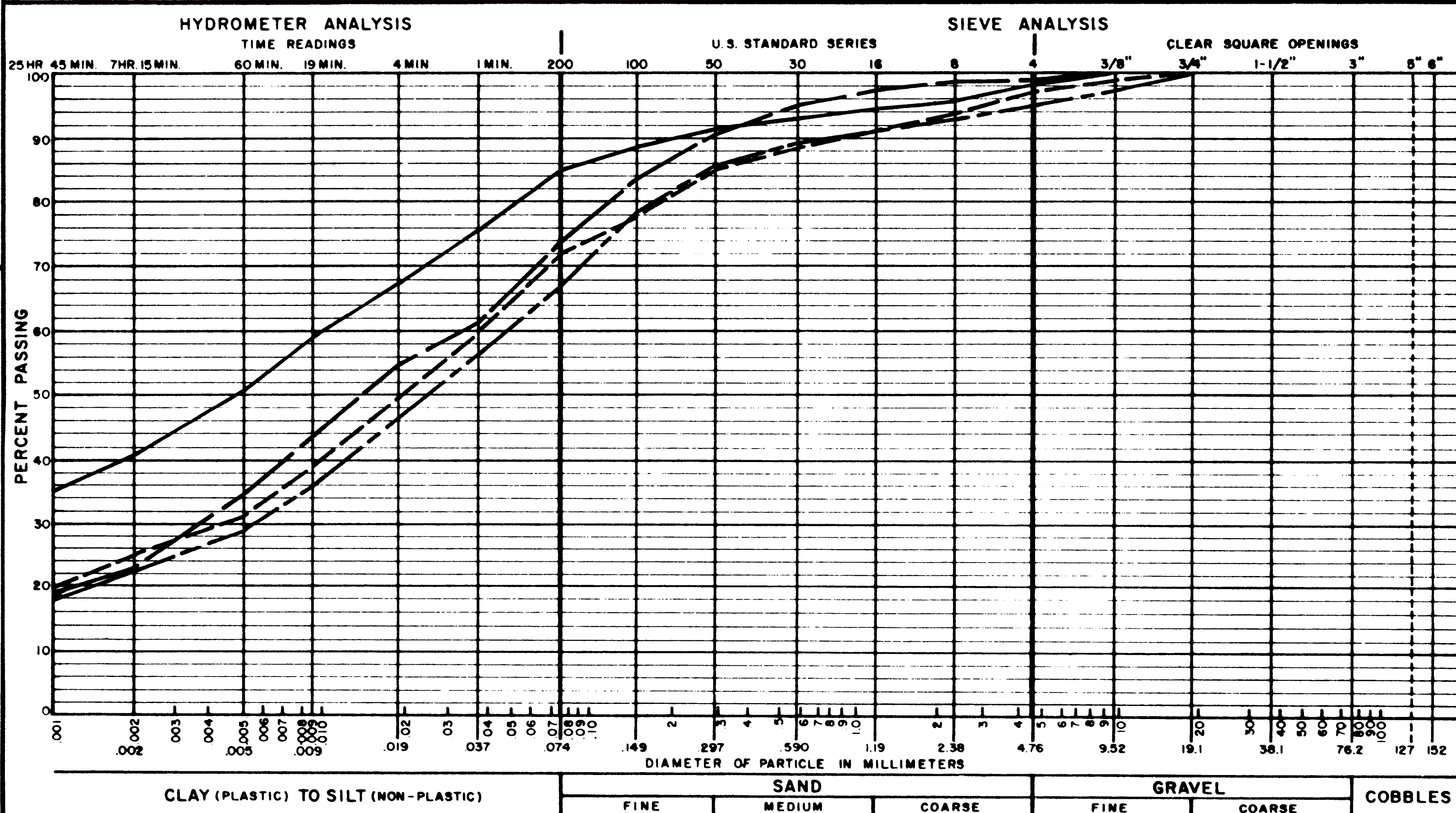
DATE DECEMBER 1973

DRAWING NO I-1

Sheet 28 of 33

KEY: ——— – – – — — — —·—

LL	---	48	35	34
PL	---	20	18	16
PI	---	28	17	18
NAT. W/C	26.1	23.7	21.2	21.2
SPEC. GRAVITY	---	2.74	2.73	---
CLASSIF. SYMB.	CL	CL	CL	CL
SAMPLE NO.	0-1	0-3	0-6	0-10
DEPTH, FT.	13-14	28-30	53.5-55.5	87.5-88.8
HOLE NO.	SS-25	SS-25	SS-25	SS-25



W.A. WAHLER & ASSOCIATES

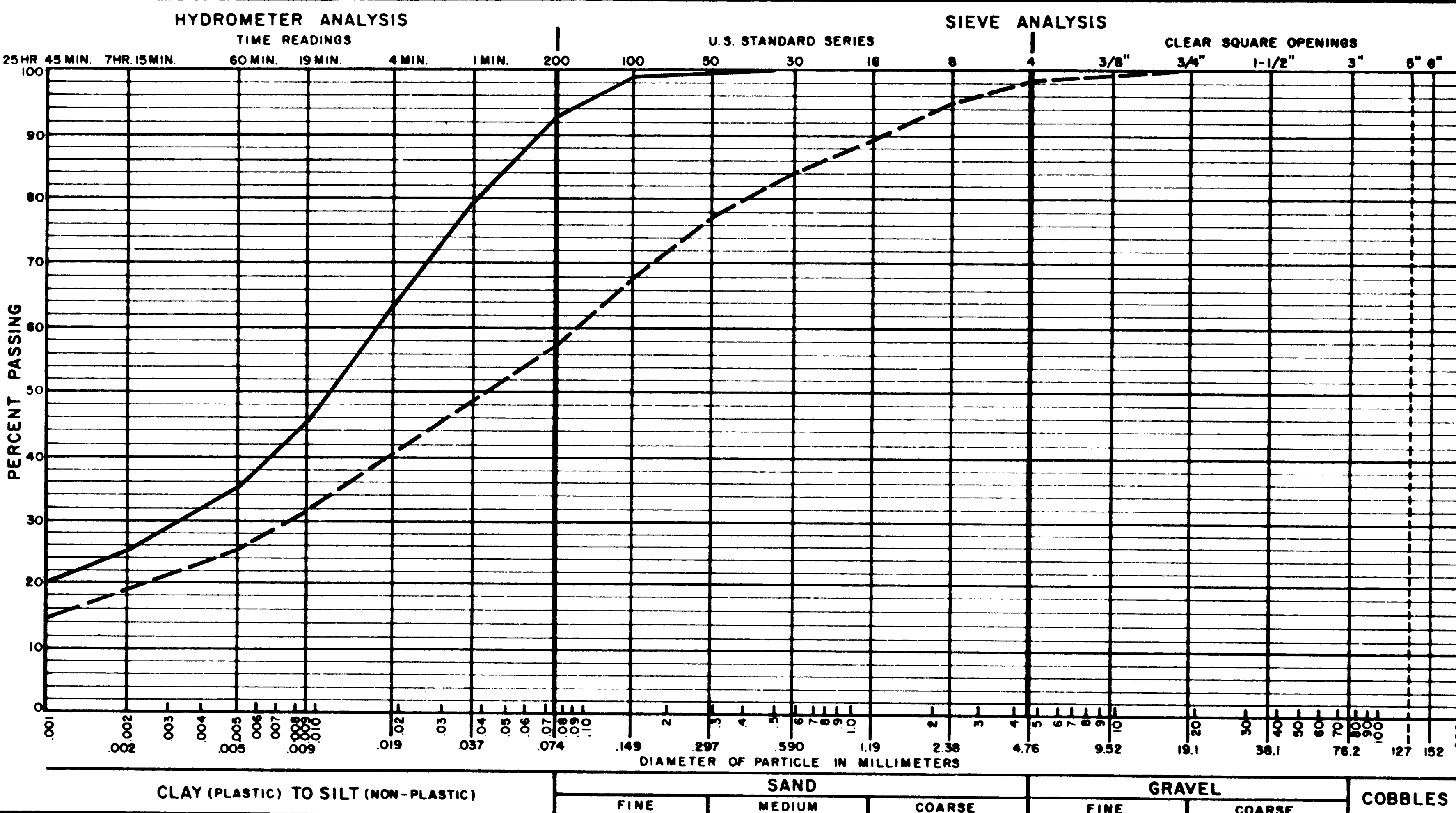
PALO ALTO • NEWPORT BEACH • CALIF.

LAFAYETTE DAM

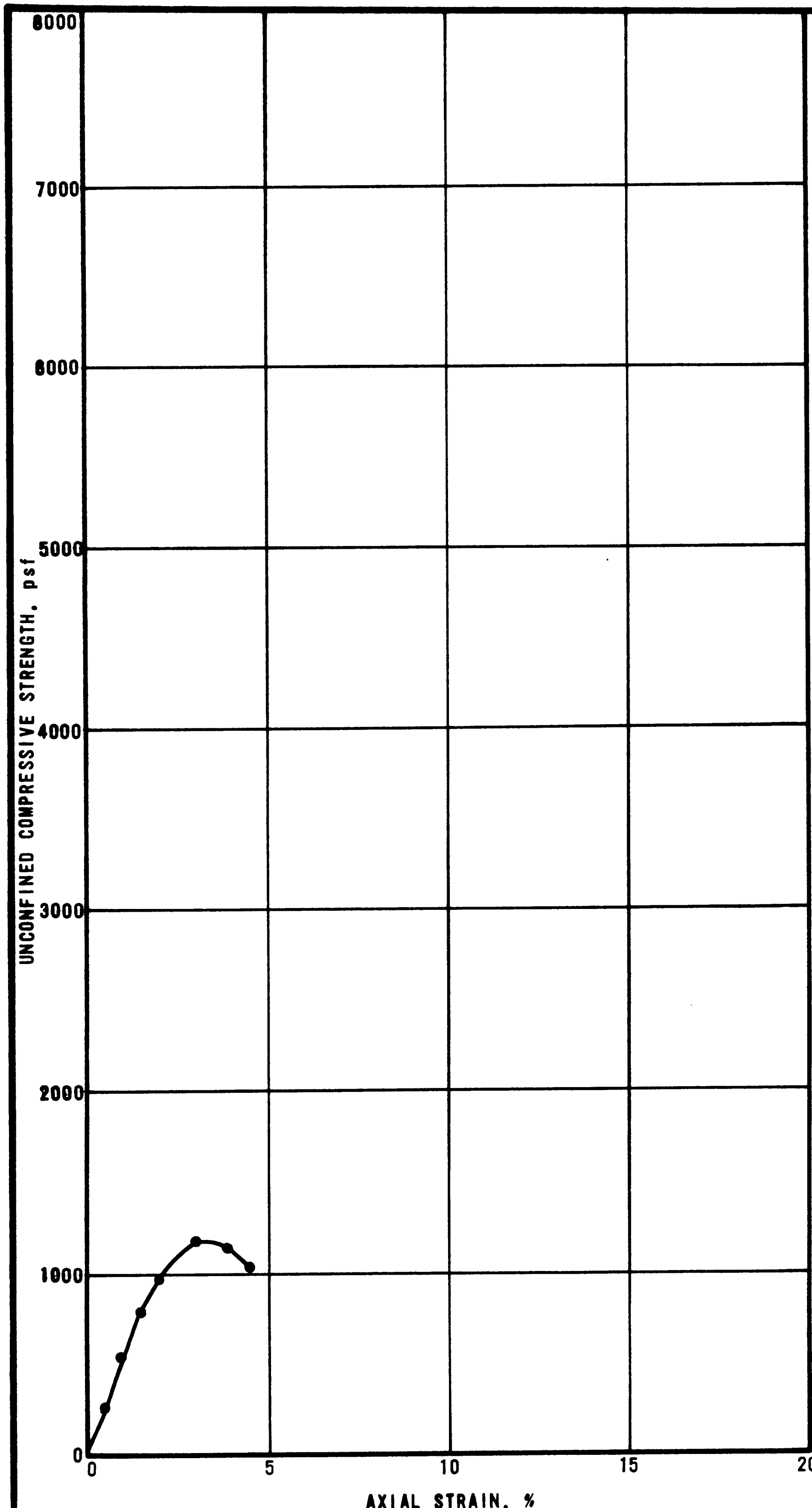
GRADATION TEST RESULTS
HOLE SS-27

KEY: ———— - - - -

LL	47	31							
PL	21	15							
PI	26	16							
NAT. W/C	27.8	21.3							
SPEC. GRAVITY	2.76	---							
CLASSIF. SYMB.	CL	CL							
SAMPLE NO.	0-2	0-5							
DEPTH, FT.	21.5-23.5	45-47							
HOLE NO.	SS-27	SS-27							



PROJECT NO	DATE	DRAWING NO
0722	DECEMBER 1973	B-1



LOCATION:

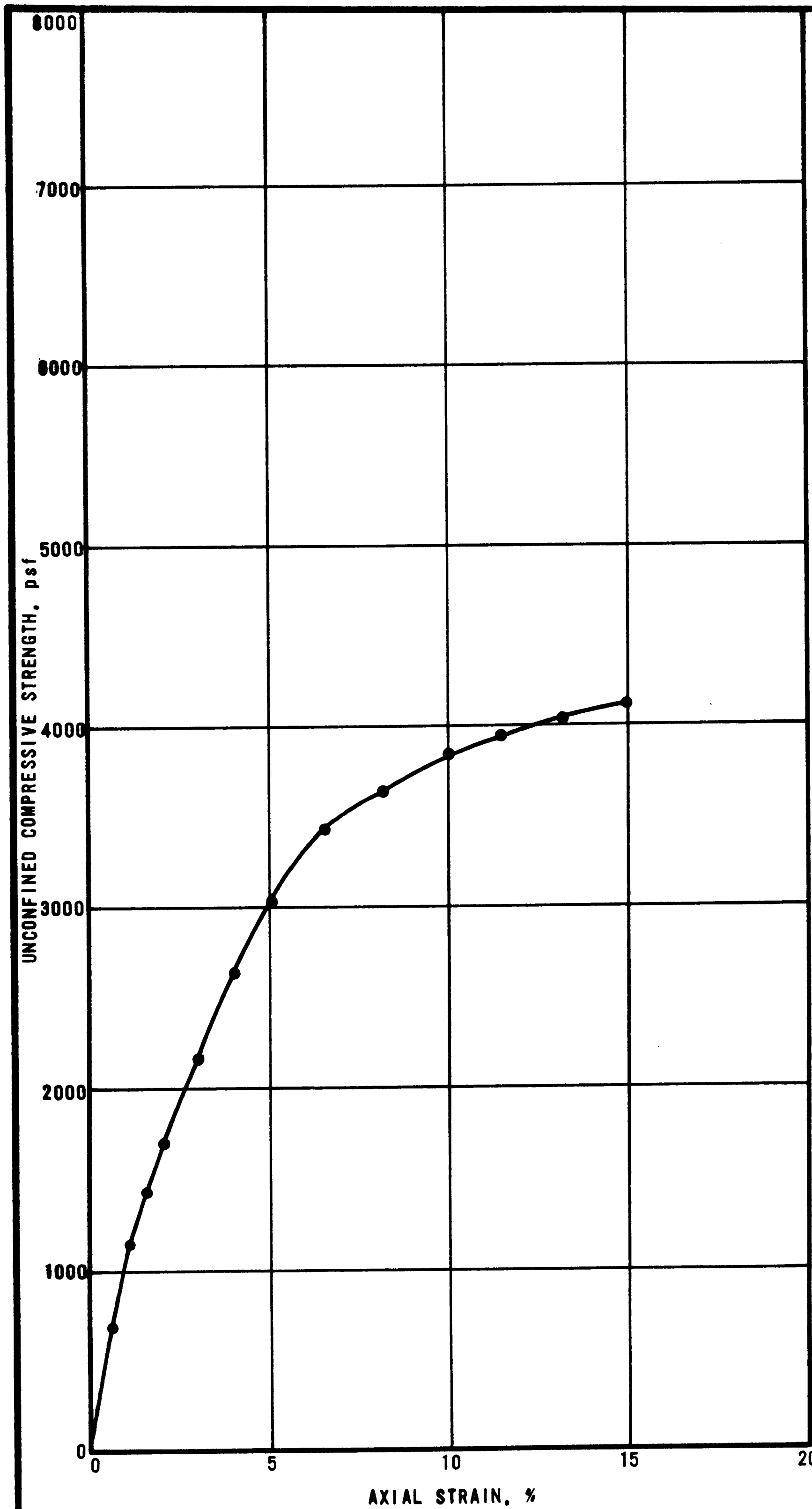
UPSTREAM FOUNDATION

SPECIMEN NO.	0-1		
Drill Hole No.	SS-25		
Depth (ft)	13-14		
Water Cont. %	26.1		
Dry Density, pcf	97.2		
Void Ratio	.759		
Saturation, %	94.2		

NOTE: SATURATION AND VOID RATIO DATA CALCULATED USING AN ESTIMATED SPECIFIC GRAVITY OF 2.74.

$OCR = \frac{600}{13.5 \times 60 \times 2.4} = 1.85$

W.A. WAHLER & ASSOCIATES	LAFAYETTE DAM	UNCONFINED COMPRESSION TEST		
		PROJECT NO.	DATE	DRAWING NO.
	PALO ALTO ● NEWPORT BEACH ● CALIF.	0722	DECEMBER 1973	B-3



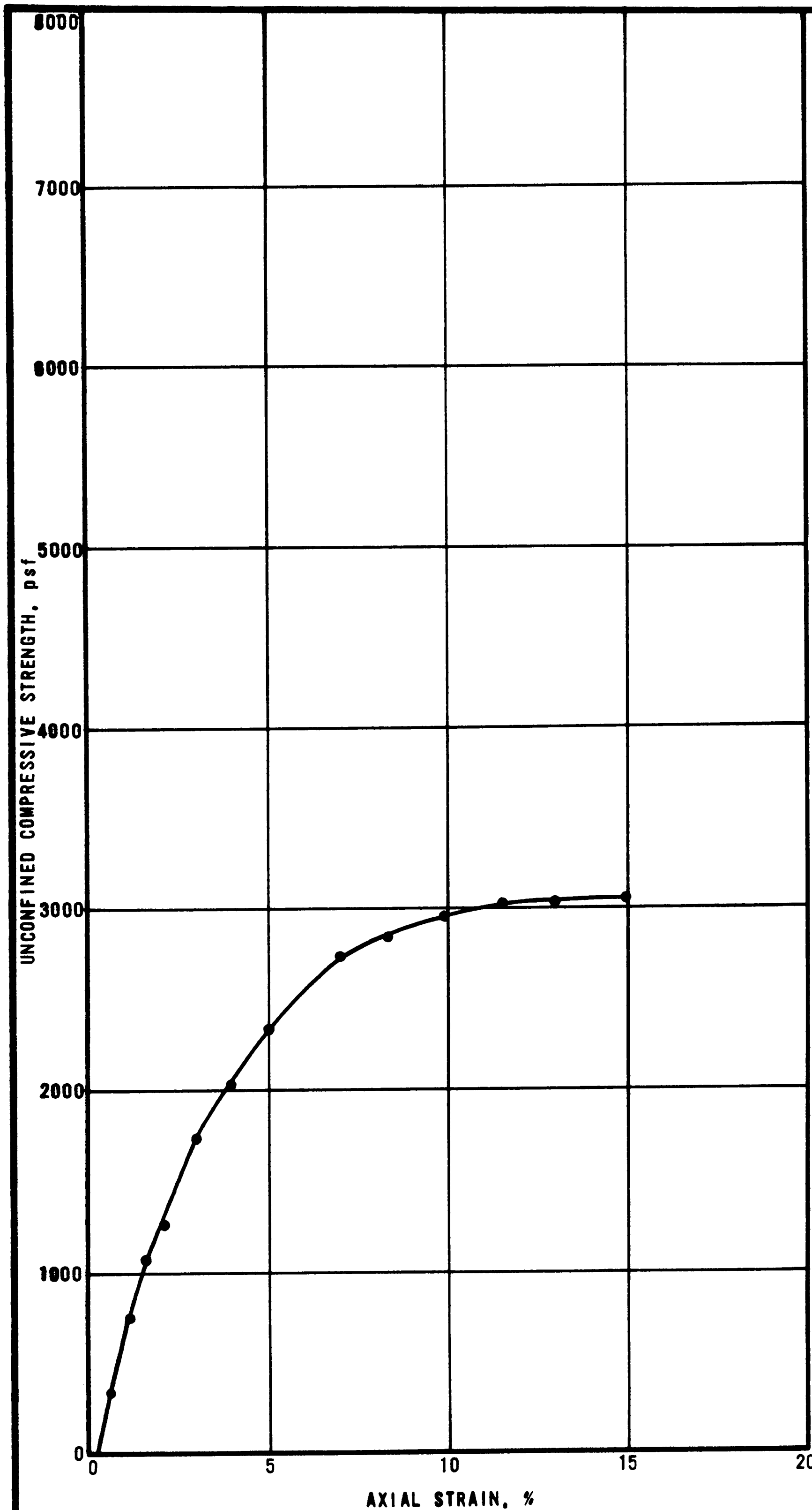
LOCATION:

UPSTREAM FOUNDATION

SPECIMEN NO.	0-6		
Drill Hole No.	SS-25		
Depth (ft)	53.5-55.5		
Water Cont. %	21.2		
Dry Density, pcf	109.3		
Void Ratio	.559		
Saturation, %	100.0		
NOTE: SPECIFIC GRAVITY = 2.73			

$$OCR = \frac{1500}{54 \times 60 \times 0.40} = 1.16$$

W. A. WAHLER & ASSOCIATES	LAFAYETTE DAM	UNCONFINED COMPRESSION TEST		
		PROJECT NO.	DATE	DRAWING NO.
	PALO ALTO ● NEWPORT BEACH ● CALIF.	0722	DECEMBER 1973	D-3



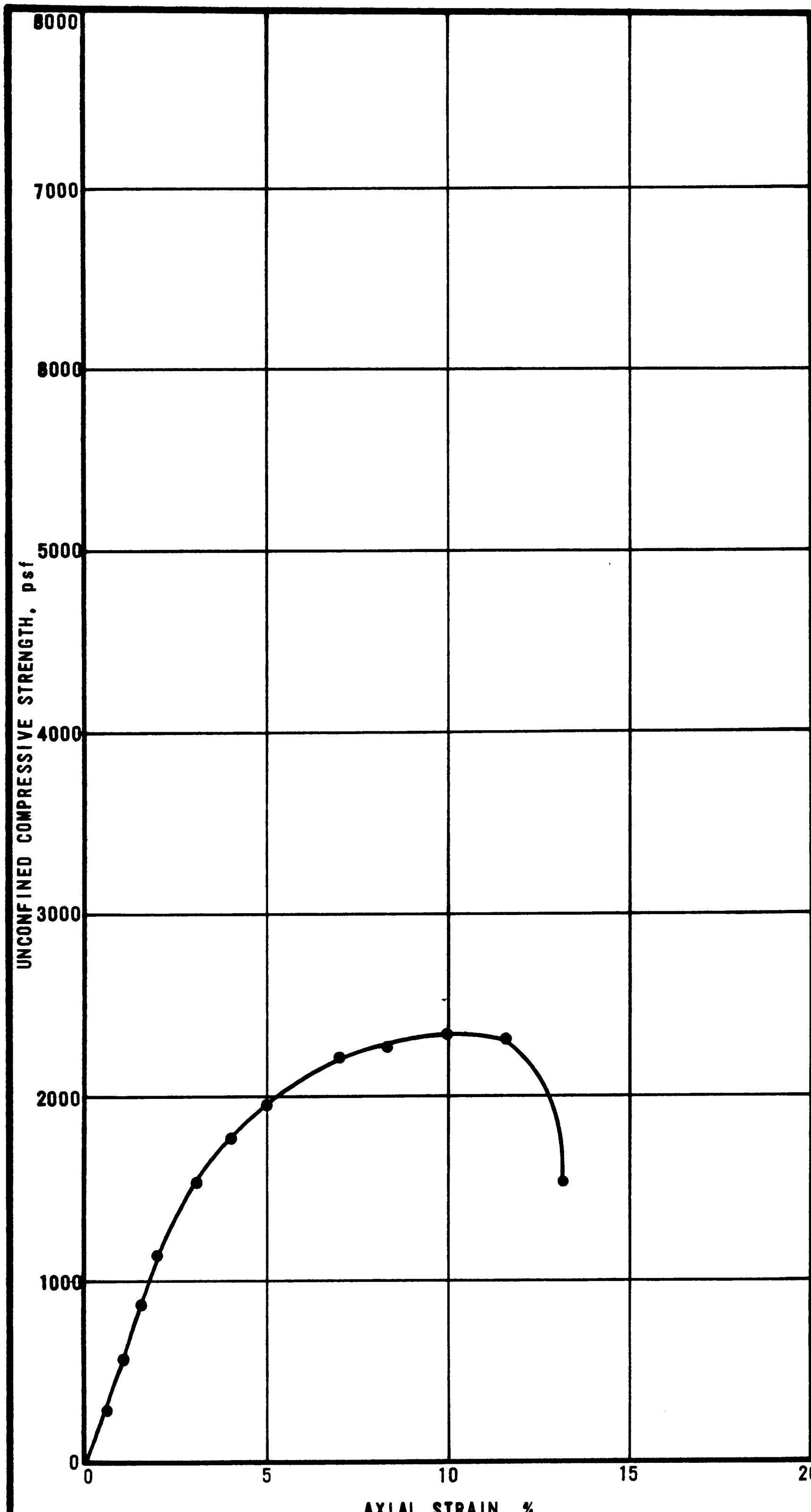
LOCATION:

UPSTREAM FOUNDATION

SPECIMEN NO.	0-10		
Drill Hole No.	SS-25		
Depth (ft)	87.5-88.8		
Water Cont. %	21.2		
Dry Density, pcf	107.9		
Void Ratio	.585		
Saturation, %	99.4		

NOTE: SATURATION AND VOID RATIO DATA CALCULATED USING AN ESTIMATED SPECIFIC GRAVITY OF 2.74.

W. A. WAHLER & ASSOCIATES	LAFAYETTE DAM	UNCONFINED COMPRESSION TEST		
	PALO ALTO ● NEWPORT BEACH ● CALIF.	PROJECT NO. 0722	DATE DECEMBER 1973	DRAWING NO. B-3



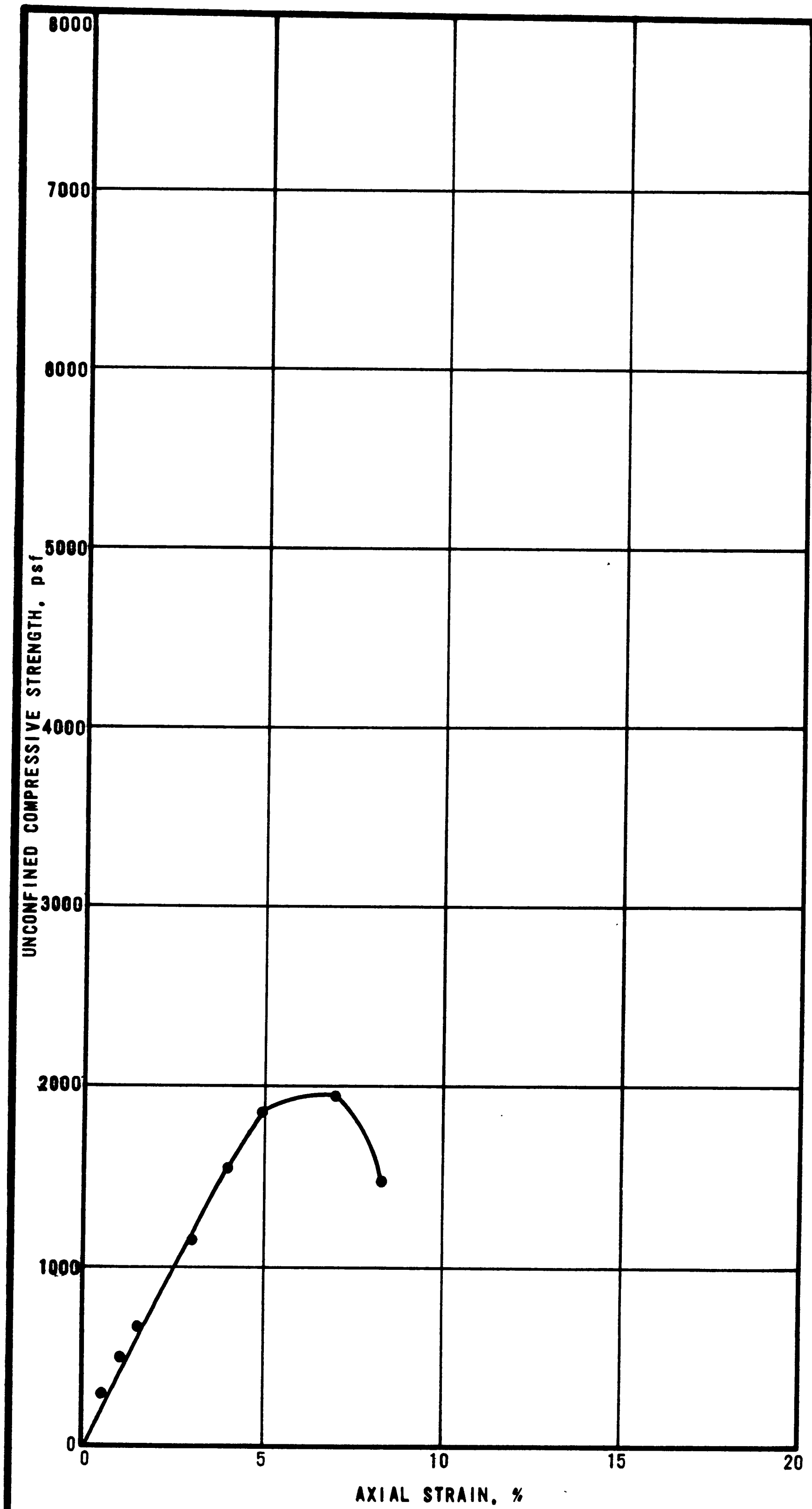
LOCATION:
UPSTREAM FOUNDATION

SPECIMEN NO.	0-2		
Drill Hole No.	SS-27		
Depth (ft)	21.5-23.5		
Water Cont. %	27.8		
Dry Density, pcf	97.2		
Void Ratio	.772		
Saturation, %	99.4		

NOTE: SPECIFIC GRAVITY = 2.76

OCR = $\frac{1000}{22 \times 60 \times 0.4}$ [illegible]

W.A. WAHLER & ASSOCIATES PALO ALTO ● NEWPORT BEACH ● CALIF.	LAFAYETTE DAM	UNCONFINED COMPRESSION TEST
PROJECT NO. 0722	DATE DECEMBER 1973	DRAWING NO. B-3



LOCATION:
UPSTREAM FOUNDATION

SPECIMEN NO.	0-5		
Drill Hole No.	SS-27		
Depth (ft)	45-47		
Water Cont. %	21.3		
Dry Density, pcf	108.2		
Void Ratio	.580		
Saturation, %	100.0		

NOTE: SATURATION AND VOID RATIO DATA CALCULATED USING AN ESTIMATED SPECIFIC GRAVITY OF 2.74.

$OCR = \frac{900}{46 \times 60 \times 0.4} = 0.82$

W.A. WAHLER & ASSOCIATES	LAFAYETTE DAM	UNCONFINED COMPRESSION TEST	
PALO ALTO ● NEWPORT BEACH ● CALIF.	PROJECT NO. 0722	DATE DECEMBER 1973	DRAWING NO. B-3

Appendix F

Deterministic Seismic Hazard Analysis and Spectrally Matched Time Histories Technical Memorandum

AECOM

Technical Memorandum for Deterministic Seismic Hazard Analysis and Spectrally Matched Time Histories

Lafayette Reservoir Outlet Tower Seismic Retrofit Project

East Bay Municipal Utility District

November 26, 2018

Revision History

Revision	Revision date	Details	Name	Position
0	11/7/2018	Initial Draft	Mark Dober	Task Manager
1	11/26/2018	Final	Mark Dober	Task Manager

Prepared for:

East Bay Municipal Utility District
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Oakland, CA 94607

Prepared by:

AECOM
300 Lakeside Drive, Suite 400
Oakland, CA 94612
aecom.com

Table of Contents

Tables

Figures

1. Introduction

As part of the Lafayette Reservoir Outlet Tower Seismic Retrofit Project for the East Bay Municipal Utility District (EBMUD), AECOM has performed a site-specific deterministic seismic hazard analysis (DSHA) for Lafayette Outlet Tower at Lafayette Reservoir. Lafayette Reservoir and its outlet tower are situated in a seismically active portion of central coastal California within the San Andreas fault system (Figure 1). Multiple branches of the San Andreas fault system in the region, such as the Hayward and Calaveras faults, are capable of generating large magnitude earthquakes (moment magnitude (**M**) ≥ 6.5). The tower will also be subjected to strong ground shaking generated by future large events on numerous active faults within a distance of 50 km (Figure 2). This technical memorandum presents the results of a deterministic seismic hazard analysis for ground shaking and the development of spectrally matched time histories as part of the alternative analyses of the Lafayette Outlet Tower.

In this study, the available geologic and seismologic data were used to evaluate and characterize (1) potential seismic sources and (2) maximum ground motions for design. Then time histories were spectrally matched to the resulting response spectra for use in the engineering analysis. The following presents the seismic source characterization, the ground motion prediction models used, the deterministic hazard analysis and the spectrally matched time histories.

1.1 Previous Studies

International Civil Engineering Consultants (ICEC) (1995) performed a seismic evaluation of the Lafayette Reservoir Outlet Tower. ICEC considered three faults, a **M** 7.0 on the Calaveras fault at a distance of 6.5 km, a **M** 7.3 on the Hayward fault at a distance of 9 km, and a **M** 8.0 on the San Andreas fault at a distance of 39 km. The Calaveras fault was the controlling event. The ground motion models were pre-Next Generation of Attenuation (NGA), calculated for rock site conditions. The 84^{th}-percentile peak ground acceleration (PGA) was 0.65 g for the Calaveras fault.

EBMUD (2013) developed site-specific design response spectra for the Lafayette Reservoir Outlet Tower. The controlling maximum event was a **M** 7.25 on the Hayward fault at a distance of 8.8 km. The analysis used the 2008 NGA-West1 models with a V_S30 (time average shear-wave velocity in top 30 m) of 392 m/sec. The 84^{th}-percentile spectrum was modified for near-fault directivity effects, and then adjusted for the maximum rotated component. The resulting PGA was 0.66 g.

2. Deterministic Seismic Hazard Analysis Methodology

The deterministic approach involves the following steps:

- Identification of the potential seismic sources that could produce ground motions of engineering significance at the site and estimation of the maximum earthquake that could reasonably be expected from these sources.
- Characterization of the seismic sources, including fault-to-site distances (rupture distance, Joyner-Boore distance), fault dip, and sense of slip.
- Development of the range of ground motions (median, 84^{th} percentile) that are likely to occur at the site due to the maximum earthquake for each seismic source.
- Enveloping the ground motions from each seismic source to develop the controlling maximum earthquake with the potential for generating the strongest ground motions at the site.

The first step requires a characterization of all significant seismic sources which could produce ground motions of engineering significance at the site (Section 3.1). Required parameters include fault location, geometry, and orientation; sense of slip; and maximum magnitude. In a deterministic analysis, no earthquake recurrence rate information is used. A description of the deterministic analysis is contained in Section 4.

To characterize the ground motions at the project site in the deterministic analysis, we used published empirical ground motion prediction equations for response spectral acceleration. The relationships used in this study were selected on the basis of the appropriateness of the site conditions (Section 3.2) and tectonic environment for which they were developed (Section 3.3).

3. Input to Analysis

The following sections describe the characterization of the seismic sources considered in the seismic hazard analysis, the geologic site conditions at the outlet tower site, and the empirical ground motion prediction models selected and used.

3.1 Seismic Sources

Based on our review of available data, the most significant seismic source to Lafayette Outlet Tower in terms of strong ground shaking is the Hayward fault. The Calaveras fault has a similar maximum magnitude to the Hayward fault, but is at a slightly greater distance. Other nearby faults, such as the Southampton, Franklin, Moraga and Mount Diablo faults, are still being reviewed for inclusion in the DSHA for final design of the tower. These are Latest Pleistocene or Conditionally Active faults with little direct evidence of Holocene activity.

The Hayward fault extends for 106 km from the area of Mount Misery, east of San Jose, to Point Pinole on San Pablo Bay (Figure 2). At Point Pinole, the Hayward fault runs into San Pablo Bay. The northern continuation of this fault system is the Rodgers Creek fault. The two faults are separated by a 5-km-wide right step beneath San Pablo Bay (Figure 2). Systematic right-lateral geomorphic offsets and creep offset of cultural features have been well documented along the entire length of the fault (Lienkaemper, 1992). In addition to undergoing displacement in earthquake ruptures, the Hayward fault also moves by aseismic creep. Measurements along the fault over the last two decades show that the mean creep rate is 4 to 7 mm/yr (Lienkaemper *et al*., 2012).

The last major earthquake on the Hayward fault, in October 1868, occurred along the southern segment of the fault. This **M** 6.8 event caused toppling of buildings in Hayward and other localities within about 5 km of the fault. The surface rupture associated with this earthquake is thought to have extended for approximately 30 km, from Warm Springs to San Leandro, with a maximum reported displacement of 1 m. Recent studies by Lienkaemper and Williams (2007) indicate that there have been 10 earthquakes along the southern Hayward fault since about 170 A.D. resulting in an average recurrence interval of 170 years. The last 5 events have an average recurrence interval of 140 $\pm$ 50 years. Paleoseismic trenching along the northern Hayward fault indicates that the last surface rupturing earthquake along this part of the fault was sometime between 1626 and 1724 (Lienkaemper *et al.*, 1999). This study also indicated at least four surface-rupturing earthquakes in the last 2,250 years. The Third Uniform California Earthquake Rupture Forecast (UCERF3, Field *et al*., 2013) report a best estimate slip rate of 9 mm/yr for the Northern and Southern Hayward fault. Based on consensus fault characterizations, Aagaard *et al.,* (2016) calculated a 33% probability for an earthquake rupture of magnitude **M** $\geq$ 6.7 anywhere on the Hayward fault between 2014 and 2043, with a 72% probability for all faults in the San Francisco Bay Region. The Hayward fault has the highest probability for any fault in the San Francisco Bay Region (Aagaard *et al.,* 2016).

We have adopted a **M** 7.25 for the maximum considered earthquake on the Hayward fault. This magnitude is consistent with the recent magnitude-area relationships utilized in UCERF3 (Field *et al*., 2013) considering a rupture on the combined Northern and Southern Hayward fault.

3.2 Geologic Site Conditions

NORCAL Geophysical Consultants, Inc. performed a borehole geophysical logging investigation for one borehole at Lafayette Reservoir Outlet Tower (NORCAL, 2018) to obtain a site-specific V_S30 (time-averaged shear wave velocity in the upper 30 m). The borehole was advanced from a drill deck from the mudline through the alluvium (94-ft below the drill deck), then continued into rock to a depth of 172-ft below the drill deck. The bedrock consisted of highly weathered, highly deformable plastic claystone belonging to the Orinda Formation.

Shear-wave velocity (V_S) data was acquired using the PS-wave suspension logging method. The V_S profile is summarized on Figure 3. The calculated V_S30 is 320 m/sec (1,050 ft/sec). Also shown on Figure 3 is the V_S profile utilized by ICEC (1995), estimated from a borehole drilled in 1927 at the tower location.

3.3 Ground Motion Prediction Models

To estimate the ground motions for crustal earthquakes at the project site in the DSHA, we have used ground motion prediction models appropriate for tectonically active crustal regions, such as California. The crustal models were developed as part of the NGA-West2 Project sponsored by Pacific Earthquake Engineering Research (PEER) Center Lifelines Program.

The NGA-West1 Project began in 2003 and in 2008, the first set of models became available. The NGA-West1 models had a substantially better scientific basis than past relationships, which generally dated around 1997 (e.g., Abrahamson and Silva, 1997), because they were developed through the efforts of five selected ground motion prediction developer teams working in a highly interactive process with other researchers who: (a) developed an expanded and improved database of strong ground motion recordings and supporting information on the causative earthquakes, the source-to-site travel path characteristics, and the site and structure conditions at ground motion recording stations; (b) conducted research to provide improved understanding of the effects of various parameters and effects on ground motions that are used to constrain models; and (c) developed improved statistical methods to develop ground motion relationships including uncertainty quantification. The NGA-West1 models benefited greatly from extensive new strong motion data from large earthquakes (**M** > 7) at close distances (< 25 km). Data include records from the 1999 **M** 7.6 Chi Chi, Taiwan, 1999 **M** 7.4 Kocaeli, Turkey, and 2002 **M** 7.9 Denali, Alaska earthquakes.

The NGA-West2 models were developed based on an expanded strong motion database compared to the initial NGA database. A number of more recent well recorded earthquakes were added to the NGA-West2 database including the Wenchuan, China, numerous small to moderate magnitude California events down to **M** 3.0, and several Japanese, New Zealand, and Italian earthquakes.

The PEER NGA-West2 models of Abrahamson *et al.* (2014), Boore *et al.* (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014) were used in this DSHA. The Idriss (2014) model is only valid for V_S30 greater than 450 m/sec and was excluded due to the site-specific shear-wave velocity (Section 3.2). The NGA models were weighted equally in this DSHA to estimate the ground motions at the site.

4. Deterministic Seismic Hazard Analysis

The California Division of Safety of Dams (DSOD) has adopted a "consequence-hazard" matrix (Figure 4) that establishes guidelines for selecting the level of ground motions to be used in the seismic design of dams (DSOD, 2018). In this approach, 84^{th} percentile ground motions are required for "extreme consequence" dams such as the Lafayette Dam and very high slip rate faults (9 mm/yr or greater) such as the Hayward fault. The following describes the selection and characterization of the maximum ground motions for Lafayette Outlet Tower.

Based on the ground motion prediction models described in Section 3.3, 5%-damped horizontal acceleration response spectra were calculated for the controlling maximum earthquake on the Hayward fault (Table 1). Figure 5 shows the lognormal average of ground motion prediction models for the median (50^{th} percentile) and 84^{th} percentile acceleration response spectra for the fault parameters listed in Table 1. Other input parameters include $Z_{2.5}$, the depth to a V_S of 2.5 km/sec (a proxy for basin effects), which is only used in one model, Campbell and Bozorgnia (2014). Abrahamson *et al.* (2014) and Chiou and Youngs (2014) use $Z_{1.0}$, the depth of the V_S of 1.0 km/sec. In the absence of site-specific data for $Z_{1.0}$ and $Z_{2.5}$, the authors provide an equation for default values based on the V_S30 at the site. Figure 6 shows the impact of the individual ground motion prediction models for the 84^{th} percentile. Boore et al. (2014) gives the highest ground motions at short periods (< 0.2 sec), Campbell and Bozorgnia (2014) gives the highest ground motions at long periods (> 1.0 sec). The 5%-damped horizontal enveloped median and 84^{th} percentile spectral values are provided in Table 2.

Because the Lafayette Outlet Tower is located at near-field distances of the Hayward fault, the effect of forward rupture directivity needs to be incorporated in the ground motions. We adjusted the 84^{th}-percentile horizontal response spectrum using the model of Bayless and Somerville, developed as part of the NGA-West2 Directivity Working Group (Spudich *et al.*, 2013), which is an update to the widely used model of Somerville *et al.* (1997). The Bayless and Somerville model is a function of magnitude, rupture distance, fraction of the fault rupture that lies between the hypocenter and site, and angle between the direction of fault rupture and the direction of waves travelling from fault to the site. Because it is not known *a priori* where the rupture might be initiated, we have followed the assumption of Fraser and Howard (2002) in which 40 percent of the fault length ruptures toward the site. Figure 7 shows the adjustments to the spectrum for fault normal and fault parallel directivity effects and the values are provided in Table 3. Fraser and Howard (2002) also recommend the standard response spectrum be used for the fault parallel direction when the fault parallel spectrum falls below the standard response spectrum, and this is reflected in Table 3.

Figure 8 compares the 84^{th}-percentile spectra developed in this study, with that developed by EBMUD (2013). As described in Section 1.1, the EBMUD (2013) spectra were developed using the NGA-West1 ground motion models for a V_S30 of 392 m/sec. Forward directivity effects were included to develop the fault normal spectrum using the model of Somerville *et al*. (1997) as corrected by Abrahamson (2000). The spectrum was then scaled to obtain the maximum rotated component (Figure 8). Compared to the EBMUD (2013) maximum rotated spectrum, the 84^{th}-percentile spectrum developed in this DSHA is about 9% lower at PGA, with a shift in the peak from 0.25 sec to 0.3 sec (Figure 8).

Table 1 Fault Parameters for DSHA

	Hayward Fault
M	7.25
Slip rate (mm/yr)	9 ± 2
Rupture Distance (km)	8.8
Joyner-Boore Distance (km)	8.8
R_x (km)	8.8
Sense of Slip	Right Lateral Strike-slip
Dip (deg)	90
Hanging Wall	No
Z_{tor} (km)	0
Width (km)	12
$Z_{1.0}$ (km)	0.44
$Z_{2.5}$ (km)	1.63
Z_{hyp} (km)	default
V_S30 (m/sec)	320

R_x = horizontal distance to top edge of rupture measured perpendicular to fault strike
Z_{tor} = depth to the top of coseismic rupture
$Z_{1.0}$ =Depth to shear-wave velocity of 1.0 km/s at the site
$Z_{2.5}$ =Depth to shear-wave velocity of 2.5 km/s at the site
Z_{hyp} = hypocentral depth
Width = down-dip rupture width

Table 2 5%-Damped Horizontal Acceleration Response Spectra Hayward Fault

	Median	**84th percentile**
Period (sec)	**SA (g)**	**Sa (g)**
0.010	0.36	0.61
0.020	0.36	0.62
0.030	0.37	0.64
0.050	0.41	0.72
0.075	0.50	0.87
0.100	0.58	1.01
0.150	0.72	1.24
0.200	0.80	1.38
0.250	0.85	1.47
0.300	0.86	1.52
0.400	0.82	1.50
0.500	0.76	1.42
0.750	0.58	1.13
1.000	0.47	0.92
1.500	0.31	0.63
2.000	0.23	0.46
3.000	0.15	0.30
4.000	0.10	0.21
5.000	0.075	0.15
7.500	0.037	0.074
10.000	0.021	0.042

See Figure 5

Table 3 84th-Percentile 5%-Damped Horizontal Acceleration Response Spectra Hayward Fault adjusted for Rupture Directivity

	Fault Parallel[1]	Fault Normal
Period (sec)	**SA (g)**	**Sa (g)**
0.010	0.61	0.61
0.020	0.62	0.62
0.030	0.64	0.64
0.050	0.72	0.72
0.075	0.87	0.87
0.100	1.01	1.01
0.150	1.24	1.24
0.200	1.38	1.38
0.250	1.47	1.47
0.300	1.52	1.52
0.400	1.50	1.50
0.500	1.42	1.42
0.750	1.13	1.25
1.000	0.92	1.05
1.500	0.63	0.72
2.000	0.46	0.56
3.000	0.30	0.38
4.000	0.21	0.27
5.000	0.15	0.20
7.500	0.074	0.10
10.000	0.042	0.057

See Figure 7

[1] For design purposes, the standard response spectrum is used for fault parallel direction per Fraser and Howard (2002).

5. Spectral Matching

Three sets of horizontal two-component acceleration time histories were spectrally matched to the 84th percentile fault parallel and fault normal target design spectra. Because the response spectrum of a time history has peaks and valleys that deviate from the design response spectrum (target spectrum), it is necessary to modify the motion to improve its response spectrum compatibility. The procedure proposed by Lilhanand and Tseng (1988), as modified by Al Atik and Abrahamson (2010) and contained in the computer code RSPmatch09, was used to develop the acceleration time histories through spectral matching to the target spectrum. This time-domain procedure has been shown to be superior to previous frequency-domain approaches because the adjustments to the time history are only done at the time at which the spectral response occurs resulting in only localized perturbations on both the time history and the spectra (Lilhanand and Tseng, 1988). This process preserves the non-stationary properties of the original time history, and develops a time history with a realistic displacement waveform.

To match the target design spectrum, seed time histories should be from events of similar magnitude and distance (for duration) and most importantly, spectral shape as the earthquake used to develop the spectrum. Earthquakes of **M** 6.9 to 7.5 at distances of 0 to 25 km were searched for as potential seed time histories. This resulted in 166 stations from 12 events. From these candidates, 5-95% duration and spectral shape were reviewed to verify the time histories have the necessary energy and frequency content, respectively. Table 4 lists the selected seed time histories used in the spectral matching and their properties. Figure 9 compares the response spectra for the three sets of seed time histories scaled to PGA of the target spectrum. The acceleration time histories for these seeds are provided on Figures 10 to 12. The seed time histories are strong motion recordings obtained from the PEER NGA-West2 database that have been rotated into fault normal and fault parallel orientations based on the fault strike provided in the PEER NGA-West2 database. The spectral matches and resulting time histories are shown in Figures 13 to 24.

Time History properties, including 5-95% significant durations and Arias intensities, for the spectrally matched time histories are provided in Table 5. The spectrally matched time histories have durations ranging from 17.4 to 24.5 sec, with an average of 22.0 sec. The spectrally matched time histories have Arias intensities ranging from 6.3 to 8.8 m/sec, with an average of 7.7 m/sec.

Three modern empirical relationships to calculate Arias intensity (AI) are used as a comparison to the spectrally matched time histories: Travasarou *et al*. (2003), Watson-Lamprey and Abrahamson (2006), and Abrahamson *et al.* (2016) for conditional Arias Intensity. The calculated values for a **M** 7.25 and a V_S30 of 320 m/sec at a rupture distance of 8.8 km are listed in Table 6. We recommend using the Abrahamson *et al.* (2016) model for comparison to the spectrally matched time histories. The Abrahamson *et al.* (2016) is based on the more recent NGA-West2 database and includes the more complicated scaling included with the NGA-West2 ground motion models. The Arias Intensity of the spectrally matched time histories slightly exceeds the Abrahamson *et al.* (2016) 84^{th} percentile median of 5.7 m/sec.

Similarly, 5-95% durations were calculated using the model of Kempton and Stewart (2006) with a median and median plus one sigma of 19.9 sec and 30.9 sec, respectively for a **M** 7.25 on the Hayward fault. The spectrally matched durations are less than 30.9 sec, the median plus one sigma duration (Table 5).

Table 4 Seed Time History Properties

PEER RSN	Year	Earthquake Name	Station Name	M	ClstD (km)	V_S30 (m/s)	Comp	Max Acc (g)	Max Vel (cm/sec)	Max Dis (cm)	I_a (m/sec)	Duration 5-95% (sec)
900	1992	Landers	Yermo Fire Station	7.3	23.6	354	FN	0.24	55.94	45.5	1.00	16.8
							FP	0.18	17.06	9.1	0.60	19.6
5991	2010	El Mayor-Cucapah	El Centro Array #10	7.2	20.1	203	FN	0.36	42.20	20.1	3.09	20.5
							FP	0.36	45.34	39.0	3.44	18.4
6930	2010	Darfield, New Zealand	LRSC	7.0	12.5	296	FN	0.10	12.23	7.4	0.21	23.0
							FP	0.08	10.49	6.7	0.17	23.7

Table 5 Spectrally Matched Time History Properties

PEER RSN	Year	Earthquake Name	Station Name	M	ClstD (km)	V_S30 (m/s)	Comp	Max Acc (g)	Max Vel (cm/sec)	Max Dis (cm)	I_a (m/sec)	Duration 5-95% (sec)
900	1992	Landers	Yermo Fire Station	7.3	23.6	354	FN	0.61	111.20	59.2	6.30	17.4
							FP	0.61	76.51	50.5	6.54	19.4
5991	2010	El Mayor-Cucapah	El Centro Array #10	7.2	20.1	203	FN	0.61	73.26	41.5	8.82	24.5
							FP	0.61	67.06	35.0	7.71	23.5
6930	2010	Darfield, New Zealand	LRSC	7.0	12.5	296	FN	0.61	79.05	43.2	8.42	23.2
							FP	0.61	75.91	40.3	8.32	24.2

PEER RSN: Pacific Earthquake Engineering Research Center, NGA-West2 Record Sequence Number
M: Moment Magnitude
ClstD: Closest Distance to fault rupture
Comp: Component - FN=Fault Normal, FP=Fault Parallel
I_a = Arias intensity

Table 6 Arias Intensity

Ground Motion Level	Travasarou *et al*. (2003)	Watson-Lamprey and Abrahamson (2006)	Abrahamson *et al.* (2016)
	AI (m/sec)	AI (m/sec)	AI (m/sec)
Median	2.3	5.6	2.2
84th Percentile	5.6	7.8	5.7

6. Conclusions

As part of the Lafayette Reservoir Outlet Tower Seismic Retrofit Project for EBMUD, AECOM has performed a site-specific DSHA for Lafayette Outlet Tower at Lafayette Reservoir.

A borehole geophysical investigation at the outlet tower obtained a site-specific V_S30 of 320 m/sec.

For the alternative analyses of the outlet tower, an 84^{th}-percentile 5%-damped horizontal acceleration response spectrum for design was developed for a **M** 7.25 event on the Hayward fault at a rupture distance of 8.8 km using the NGA-West2 ground motion models. Additional input parameters are provided in Table 1. Because the outlet tower is located at near-field distances of the Hayward fault, forward directivity effects were incorporated using the model of Bayless and Somerville (Spudich *et al.*, 2013) to develop the fault normal and fault parallel spectra.

Three two-component sets of horizontal time histories were spectrally matched to the fault normal and fault parallel target design spectra. Seeds time histories are selected from the PEER NGA time history database for the appropriate magnitude and distance. The seed time history response spectra were compared to the target design spectra to ensure the seed time histories have the necessary frequency content and spectral shape. Arias Intensity and 5-95% duration was compared to empirical models to confirm the spectrally matched ground motions are appropriate for the site. These time histories will be used in the dynamic analysis of the outlet tower.

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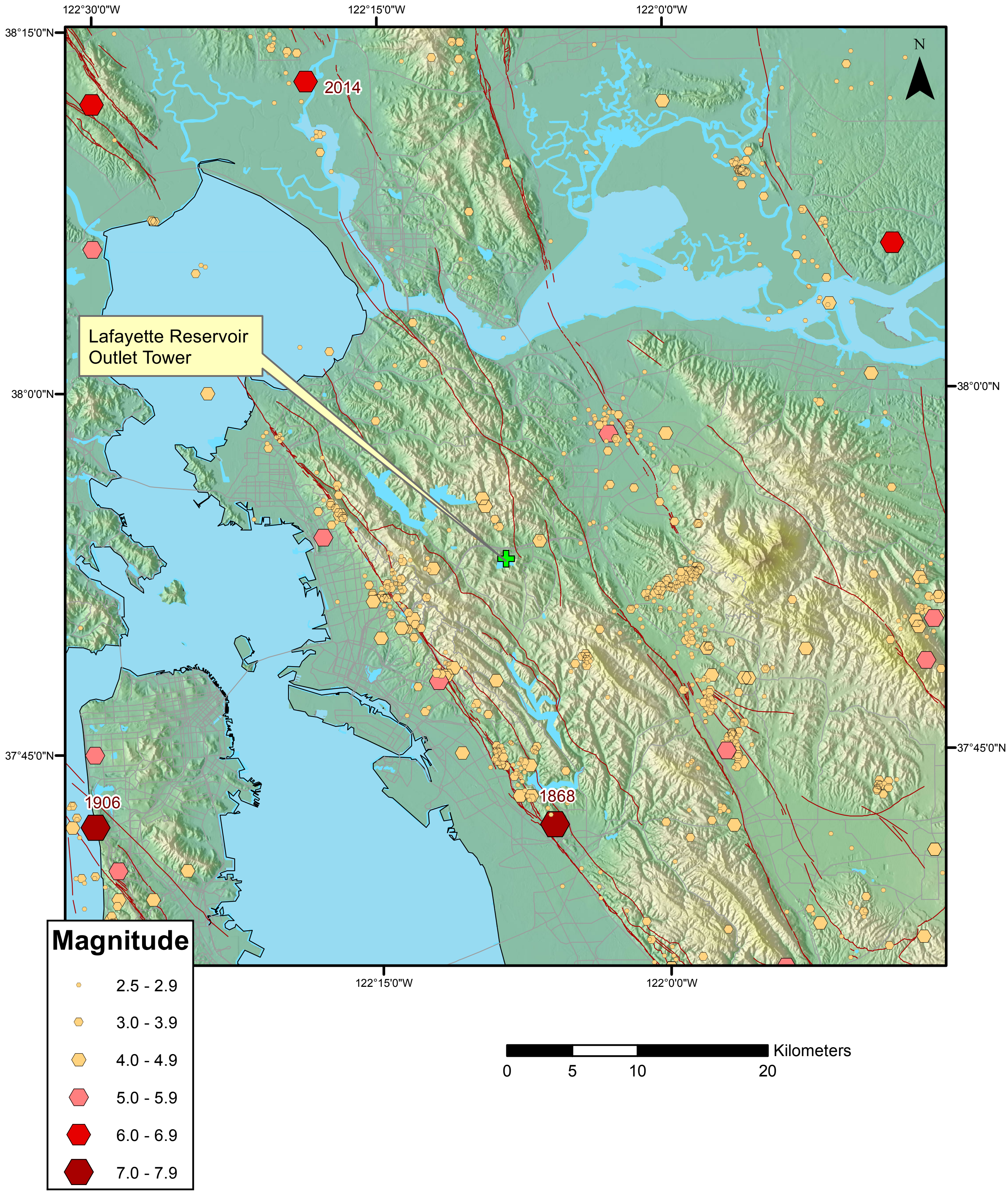
Figures



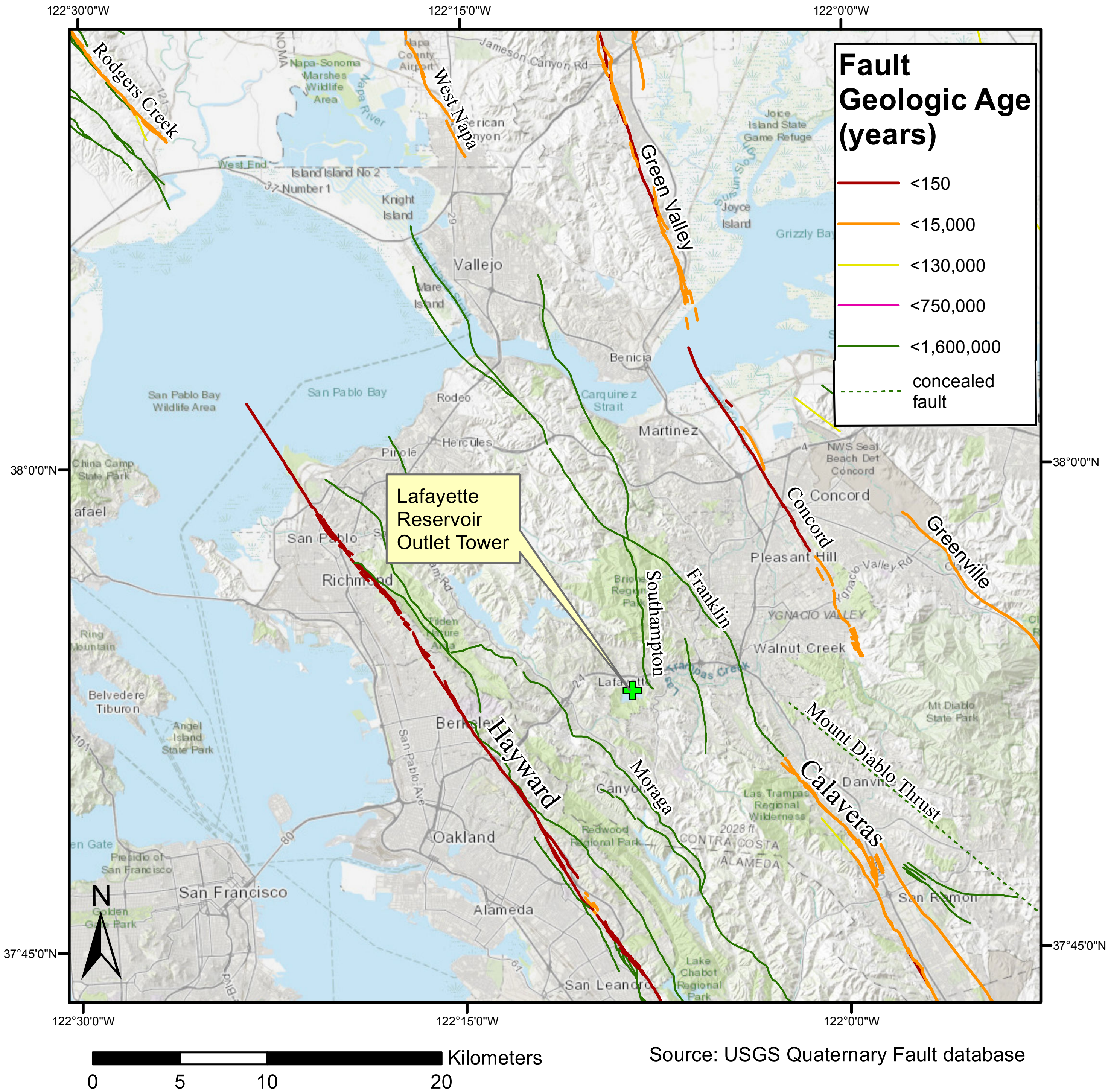
AECOM

Project No. 60572210

Lafayette Reservoir Outlet Tower Seismic Retrofit Project

HISTORICAL SEISMICITY OF THE SAN FRANCISCO BAY AREA (1828 - 2018)

Figure 1



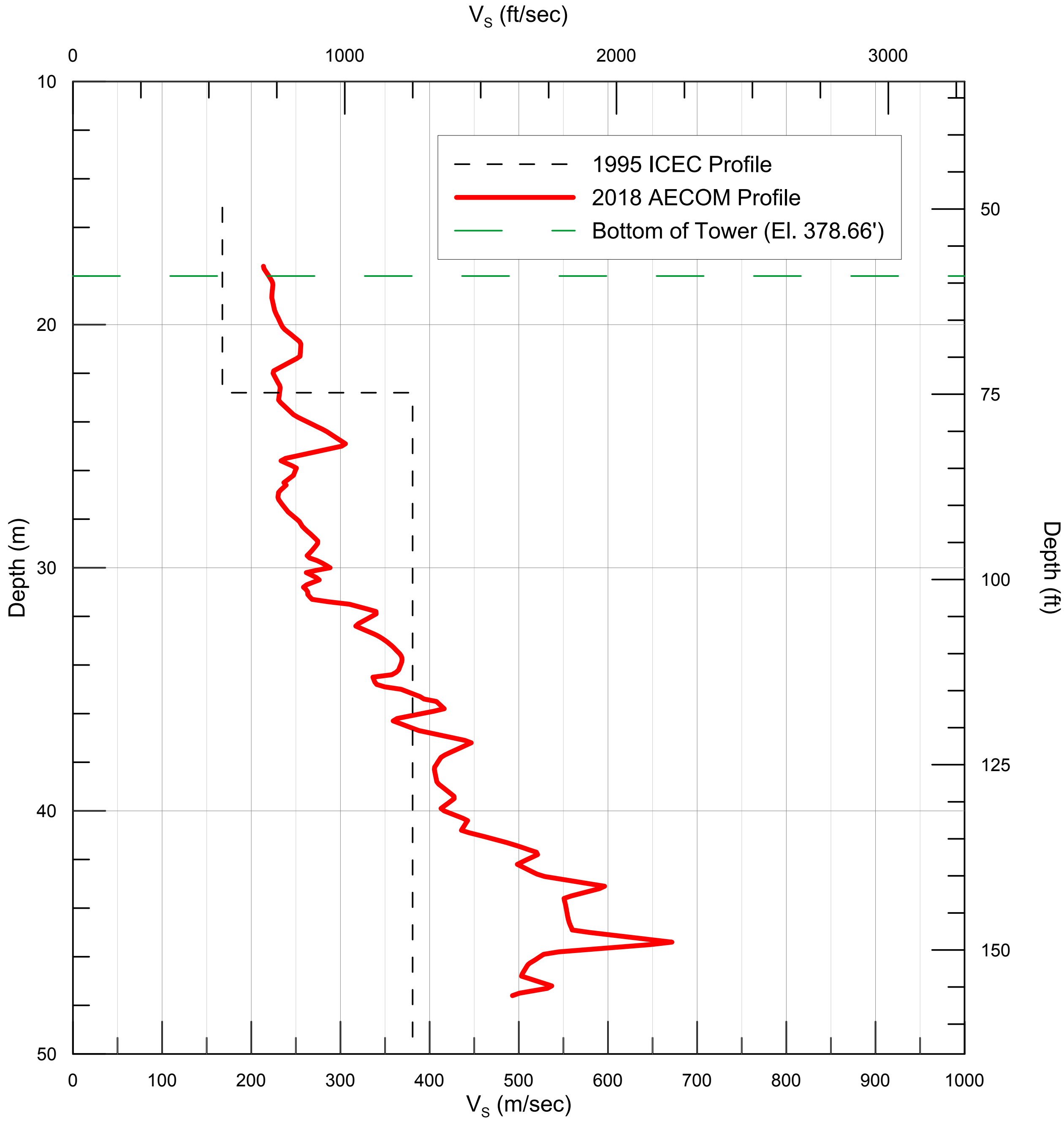
AECOM

Project No. 60572210

Lafayette Reservoir Outlet Tower Seismic Retrofit Project

QUATERNARY FAULTS NEAR LAFAYETTE RESERVOIR

Figure 2



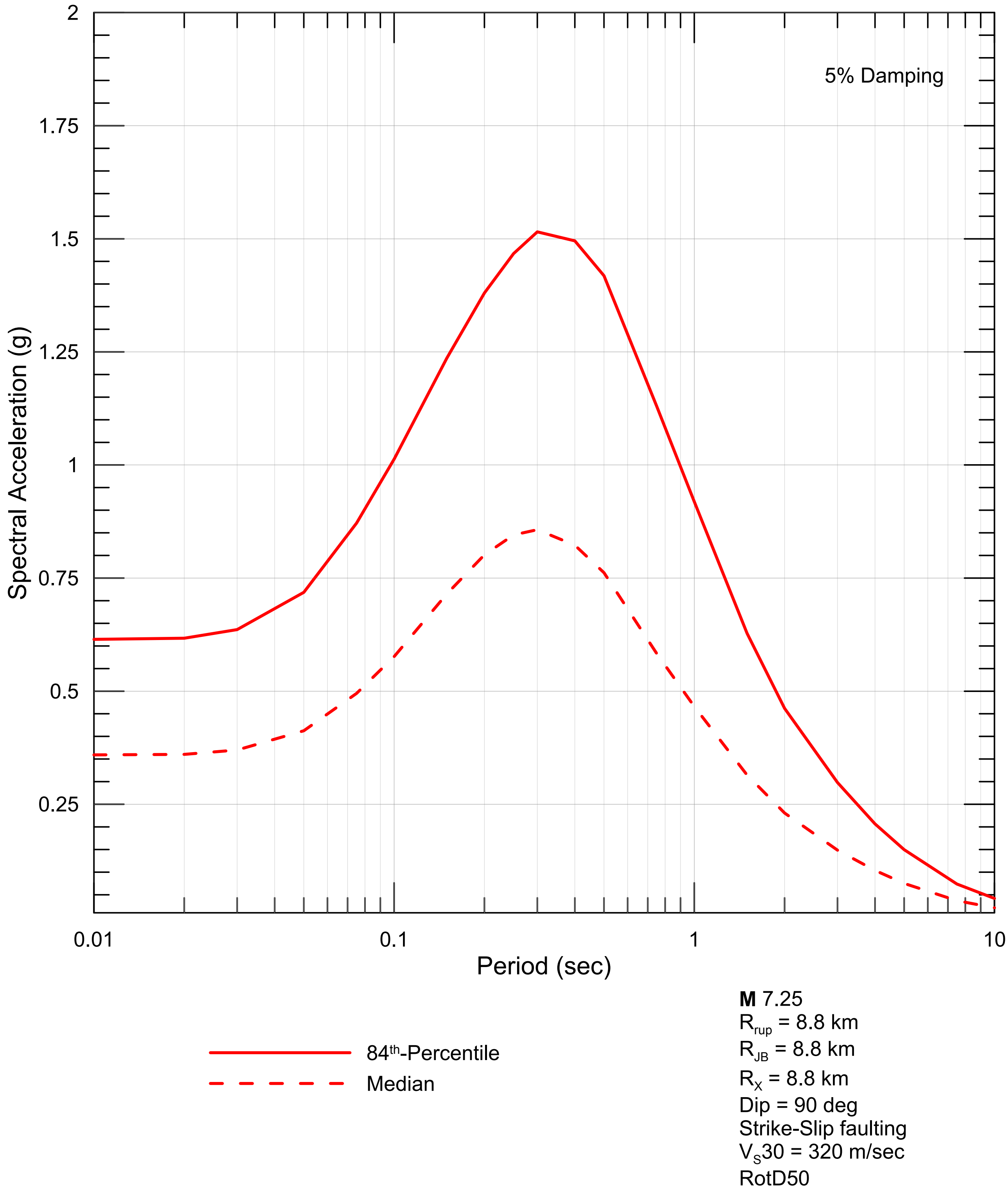
AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	SHEAR-WAVE VELOCITY PROFILE	Figure 3

		SLIP RATE			
		Very High 9 mm/yr or greater	**High** 8.9 to 1.1 mm/yr	**Moderate** 1.0 to 0.1 mm/yr	**Low** Less than 0.1 mm/yr
HAZARD CLASS	**Extremely High**	84th	84th	67th to 84th	50th to 84th
	High	84th	84th	50th to 84th	50th to 84th
	Significant	67th to 84th	50th to 84th	50th to 67th	50th
	Low	50th	50th	50th	50th

DSOD Hazard Matrix

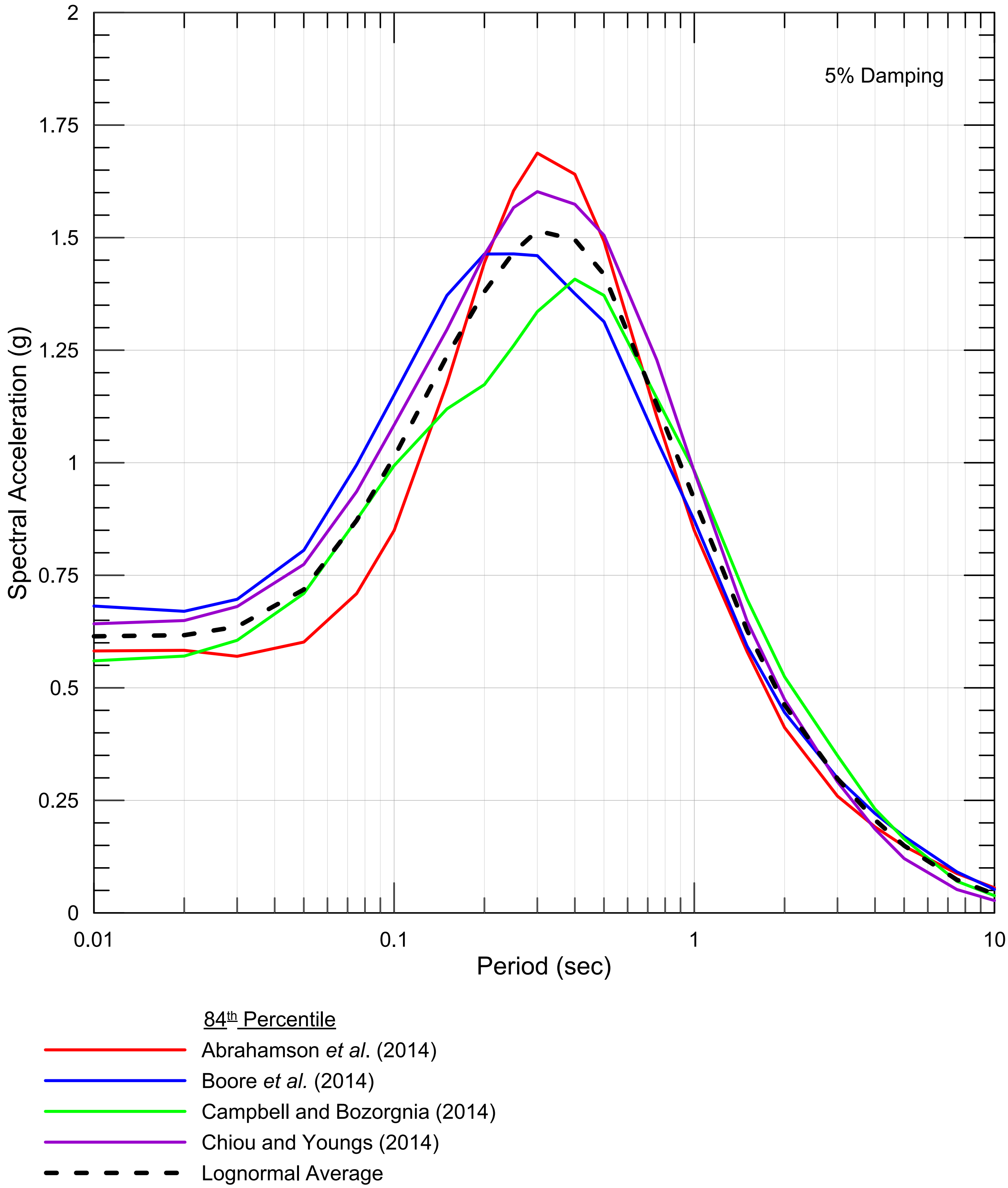
September 2017

Source: DSOD Technical Reference

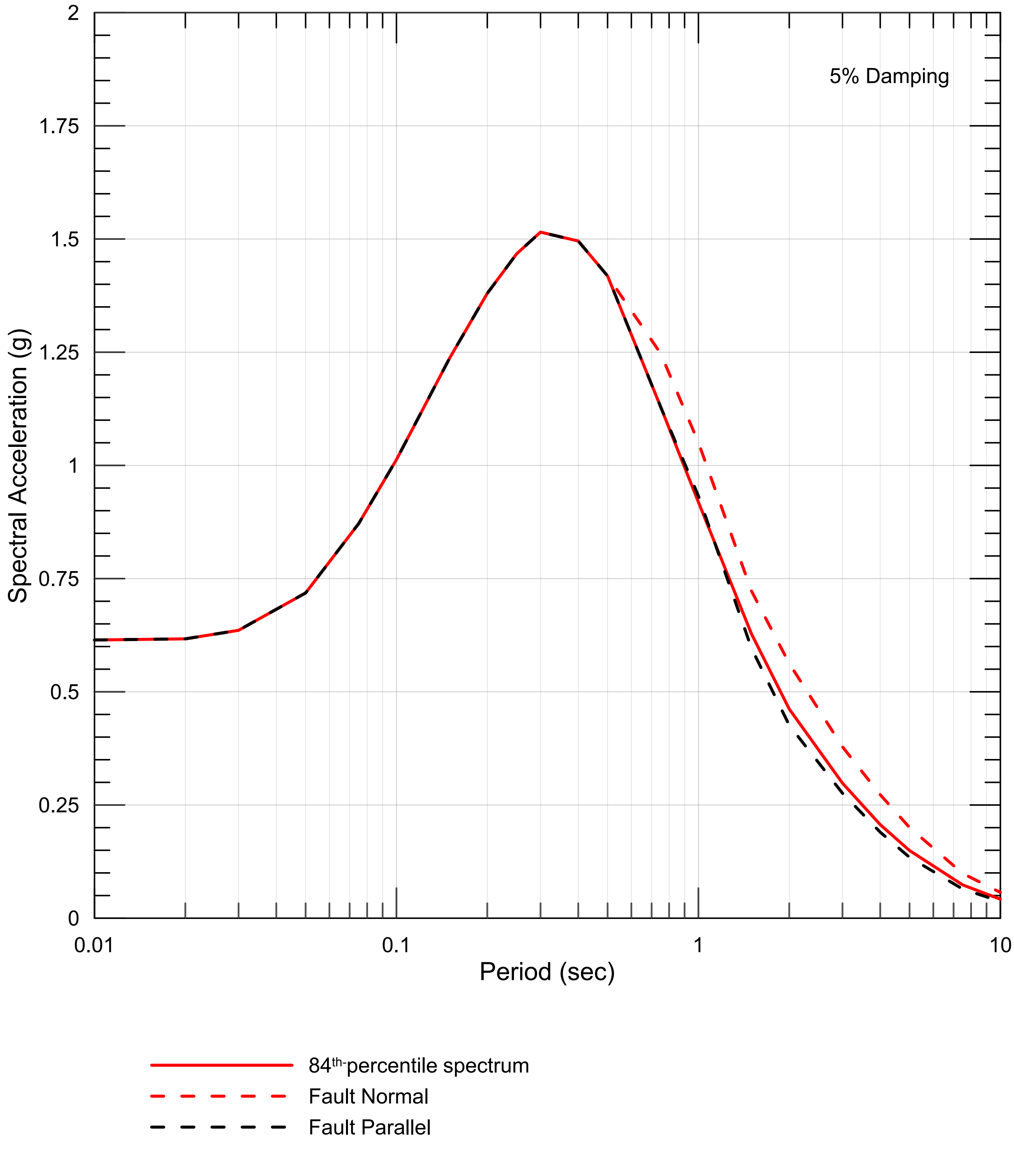


See Table 2 for Tabular Values

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	HAYWARD FAULT MEDIAN AND 84TH PERCENTILE DETERMINISITIC SEISMIC HAZARD SPECTRA	Figure 5

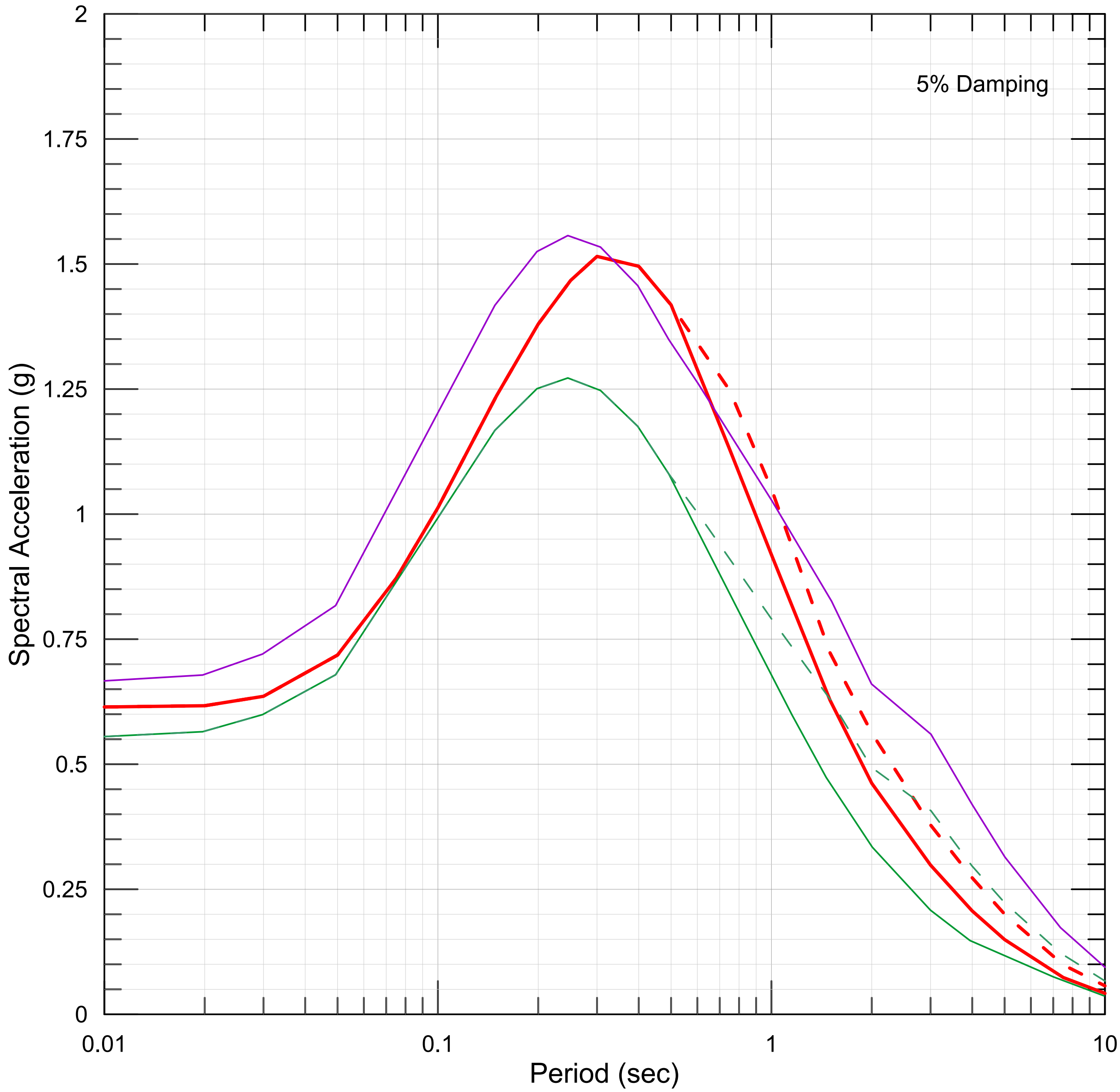


AECOM	Project No. 60572210	SENSITIVITY OF 84TH PERCENTILE DETERMINISTIC SEISMIC HAZARD SPECTRA TO GROUND MOTION MODELS FOR THE HAYWARD FAULT	Figure 6
	Lafayette Reservoir Outlet Tower Seismic Retrofit Project		



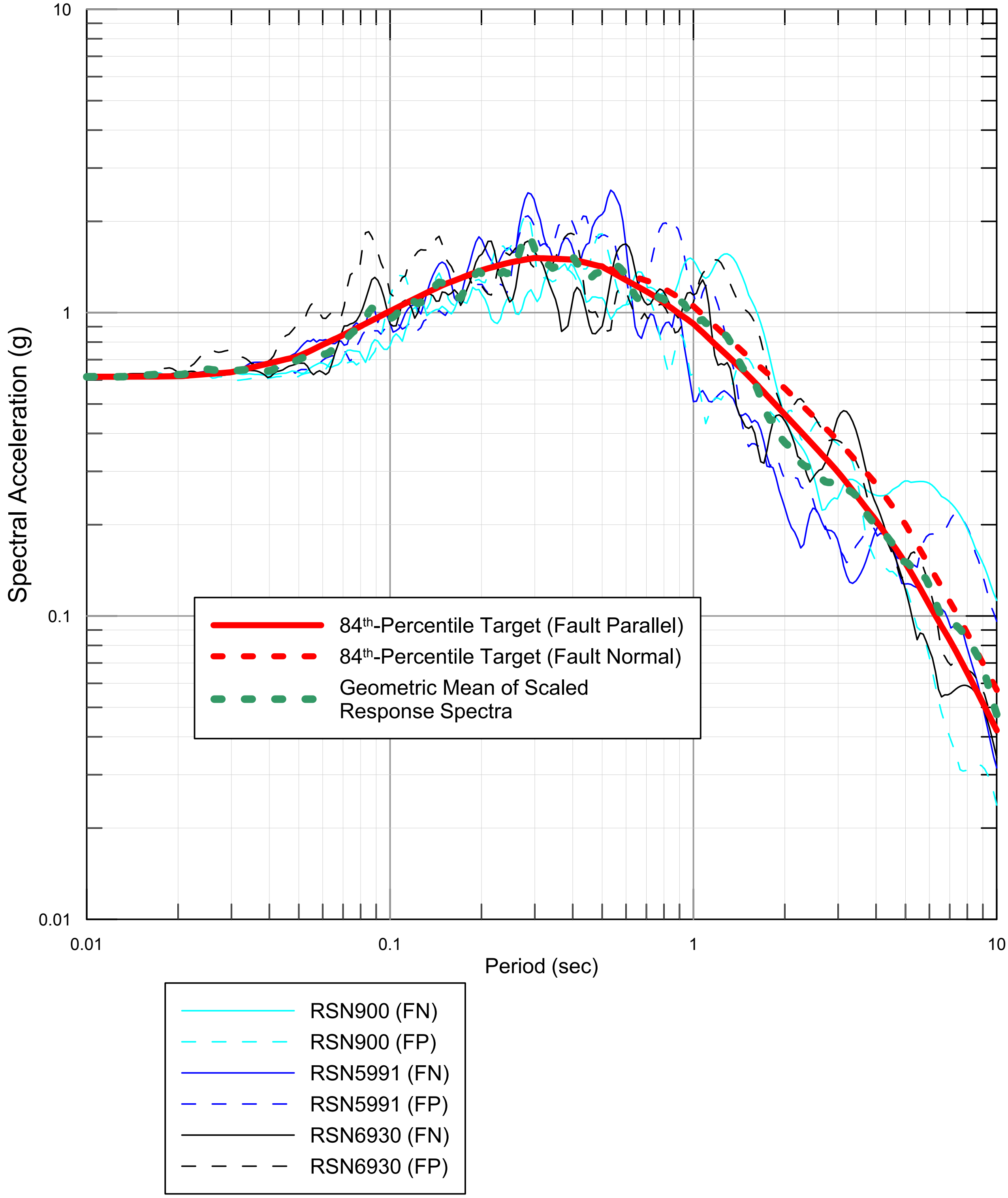
See Table 3 for Tabular Values

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	HAYWARD FAULT 84TH PERCENTILE RESPONSE SPECTRUM ADJUSTED FOR DIRECTIVITY	Figure 7

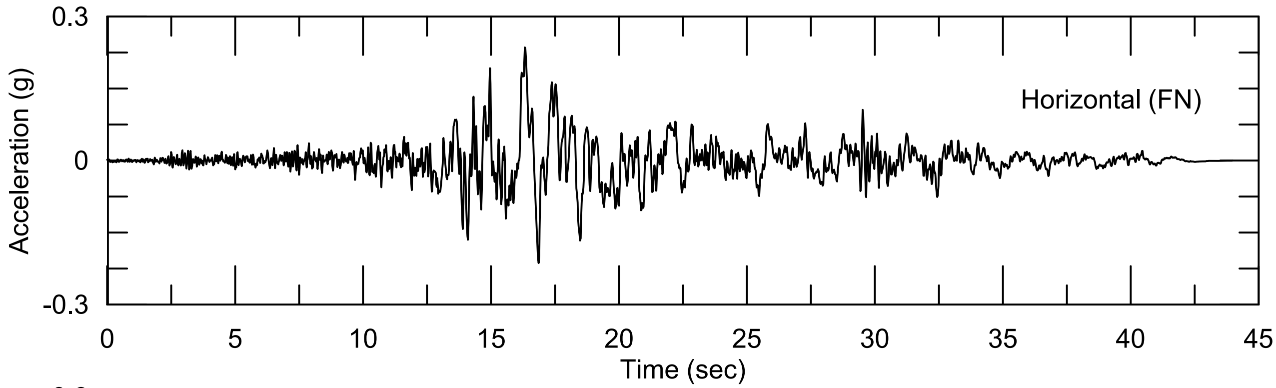


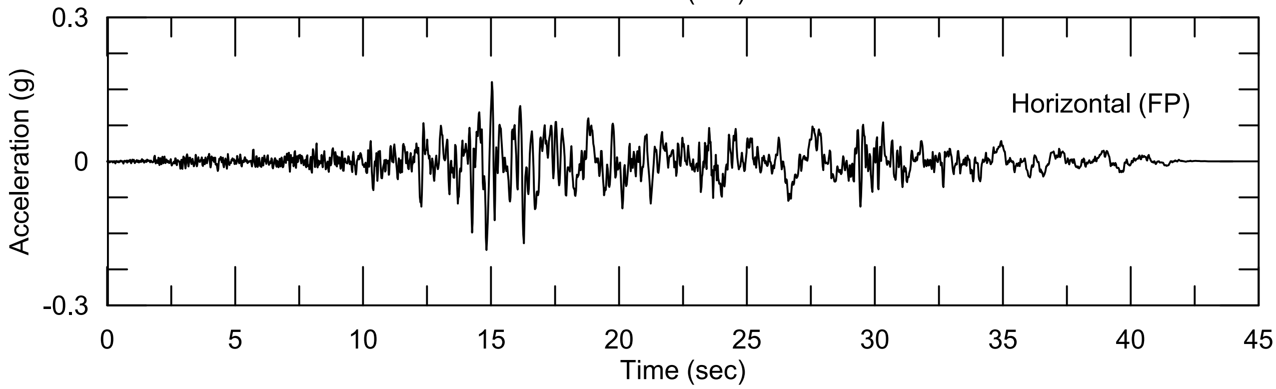
AECOM 84th-Percentile Spectrum (Fault Parallel)
AECOM 84th-Percentile Spectrum (Fault Normal)
EBMUD 84th-Percentile Spectrum (No Forward Directivity)
EBMUD 84th-Percentile Spectrum (Fault Normal)
EBMUD 84th-Percentile Spectrum (Fault Normal + MaxRot)

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	COMPARISON OF HAYWARD FAULT RESPONSE SPECTRA	Figure 8

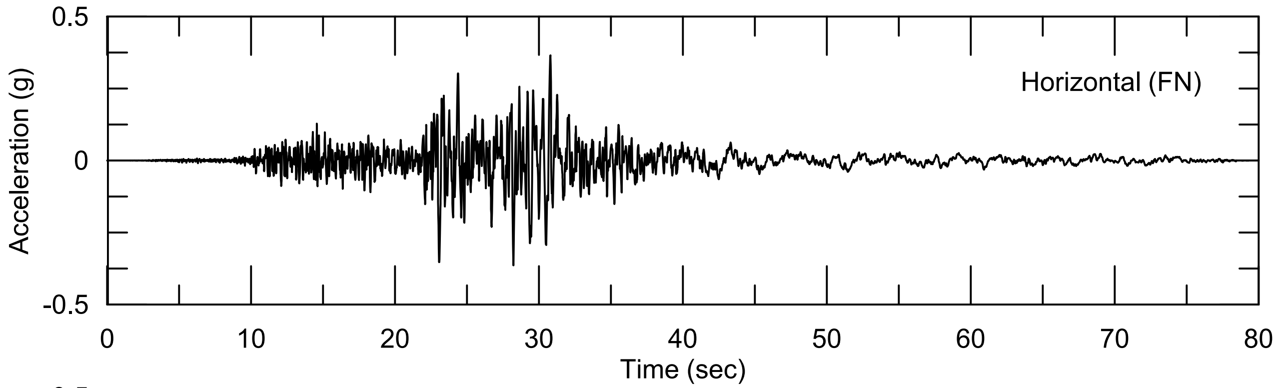


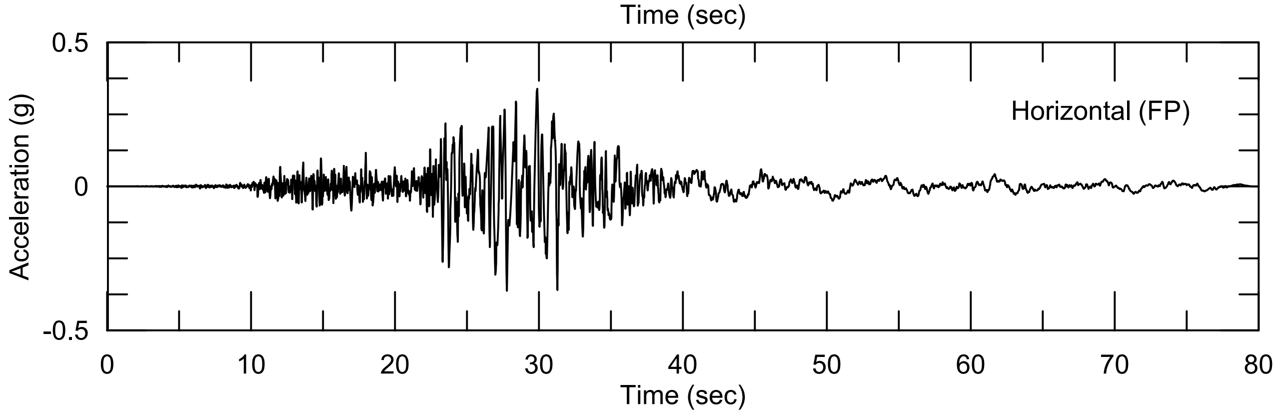
AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	COMPARISON OF HORIZONTAL TARGET RESPONSE SPECTRUM AND SCALED RESPONSE SPECTRA OF SELECTED SEED TIME HISTORIES	Figure 9



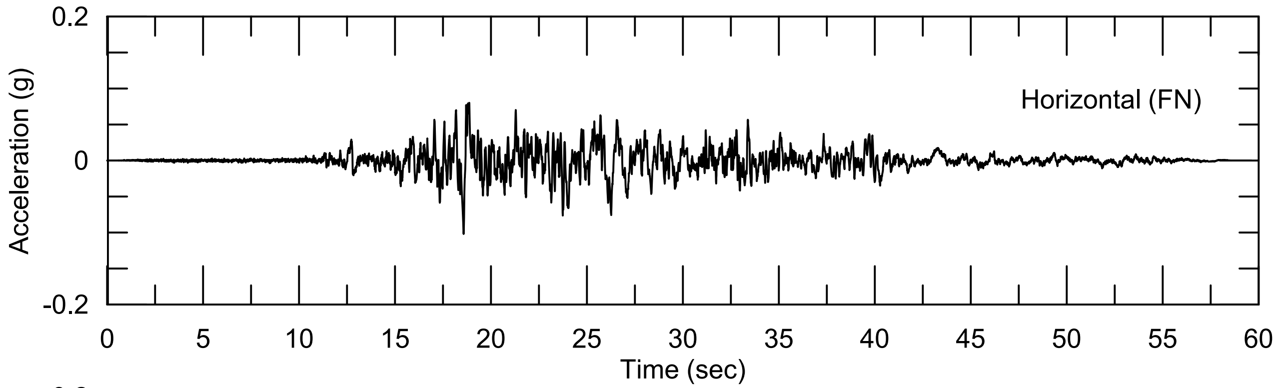


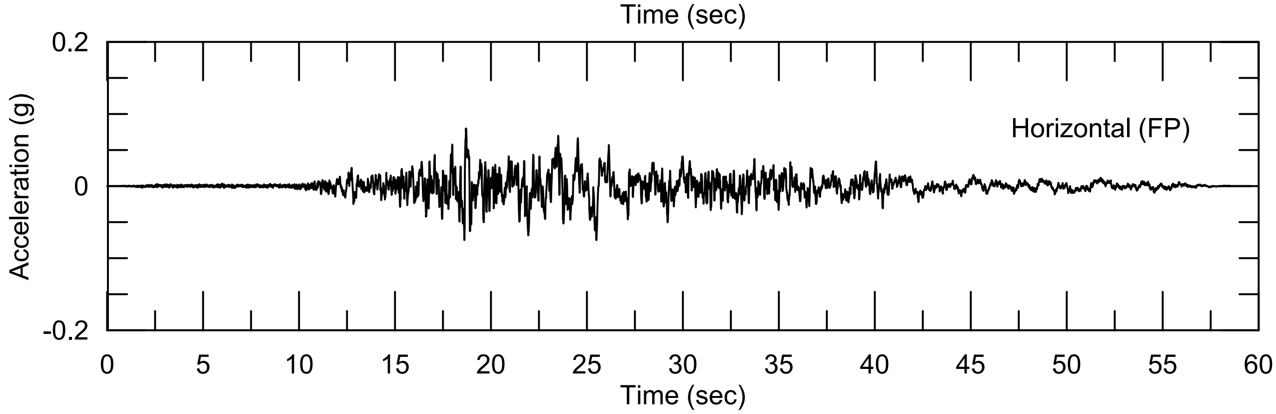
AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	SEED TIME HISTORIES RSN900 - 1992 LANDERS YERMO FIRE STATION	Figure 10



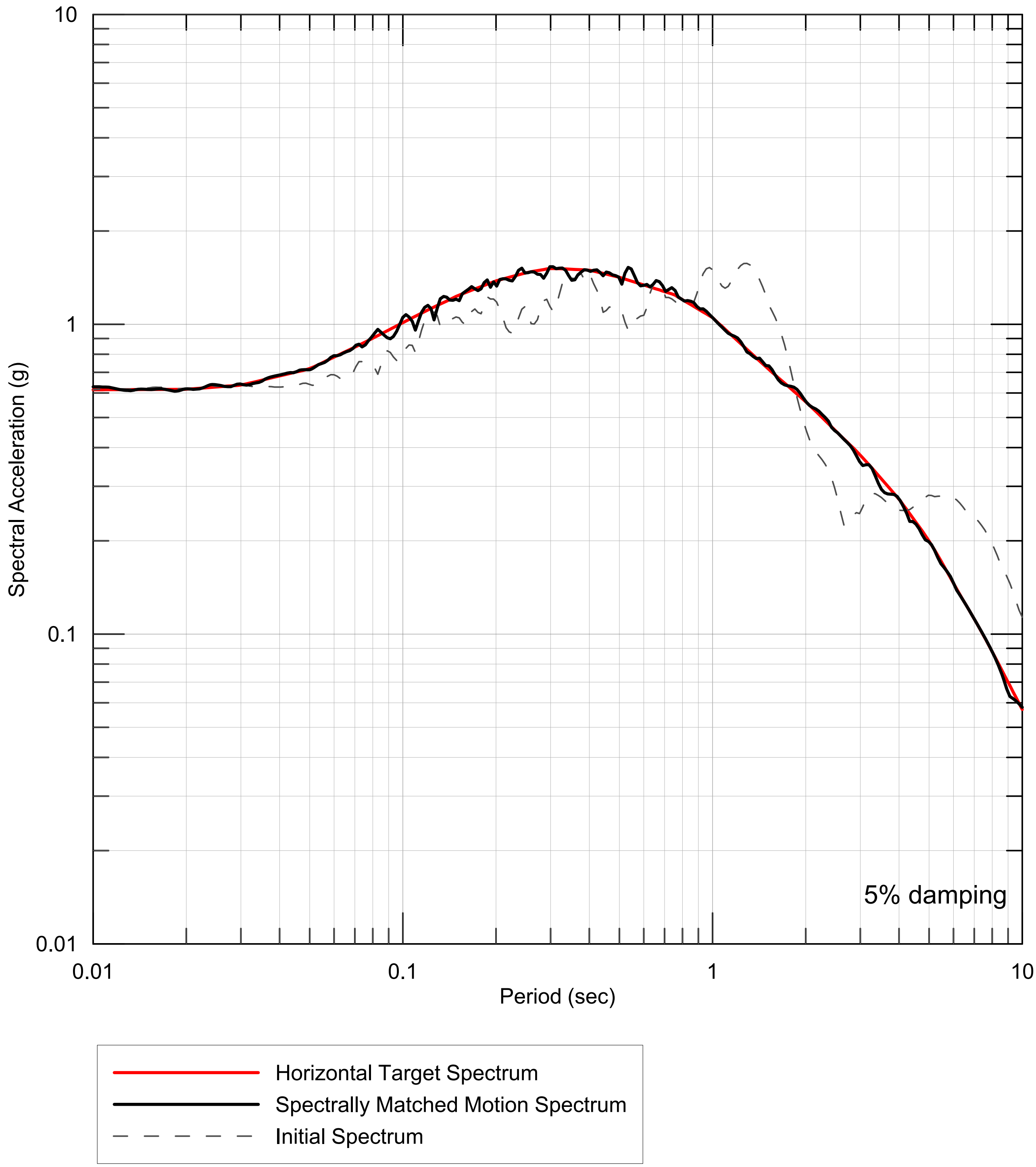


AECOM	Project No. 60572210	SEED TIME HISTORIES RSN5991 - 2010 EL MAYOR-CUCAPAH EL CENTRO ARRAY #10	Figure 11
	Lafayette Reservoir Outlet Tower Seismic Retrofit Project		



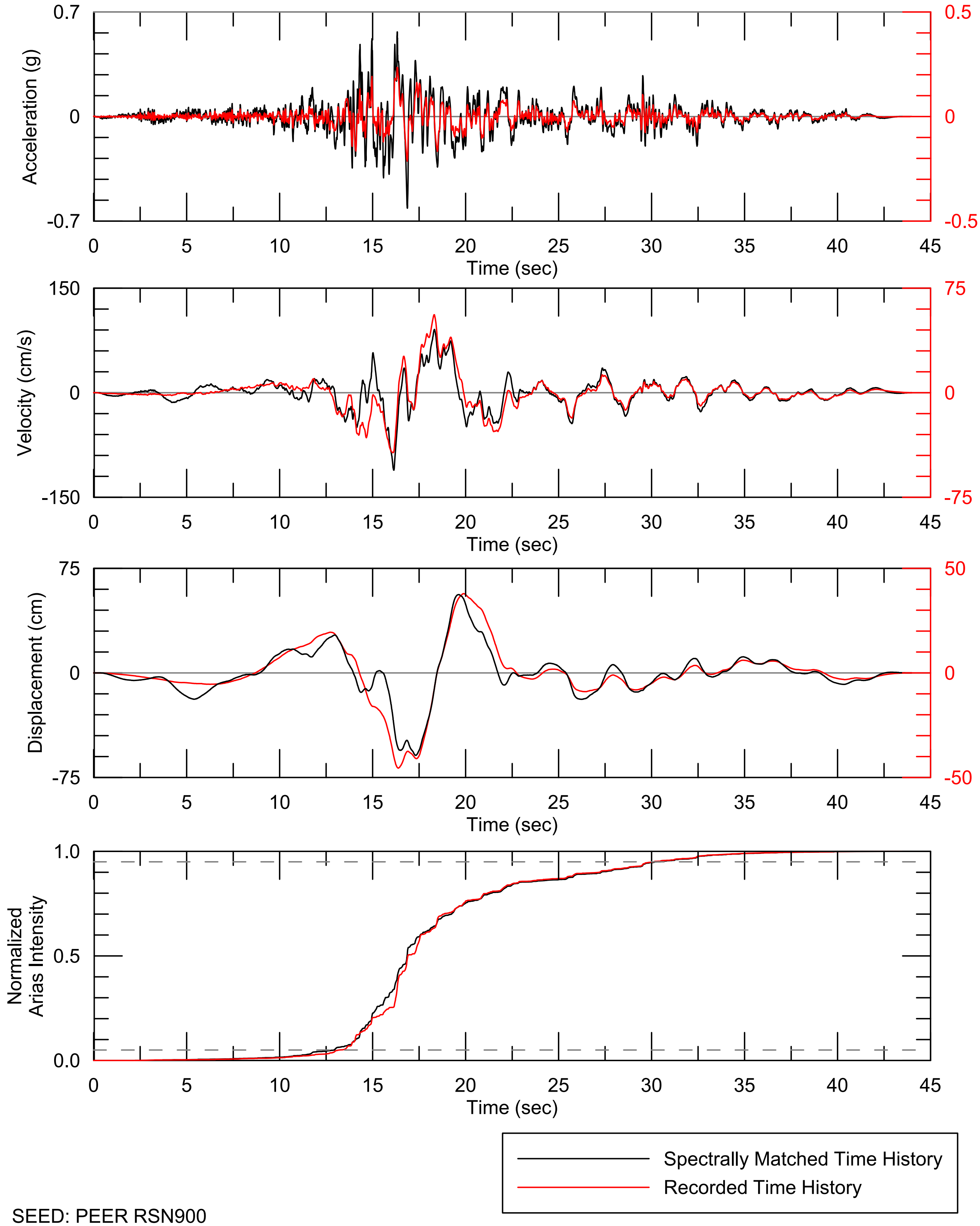


AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	SEED TIME HISTORIES RSN6930 - 2010 DARFIELD LRSC	Figure 12



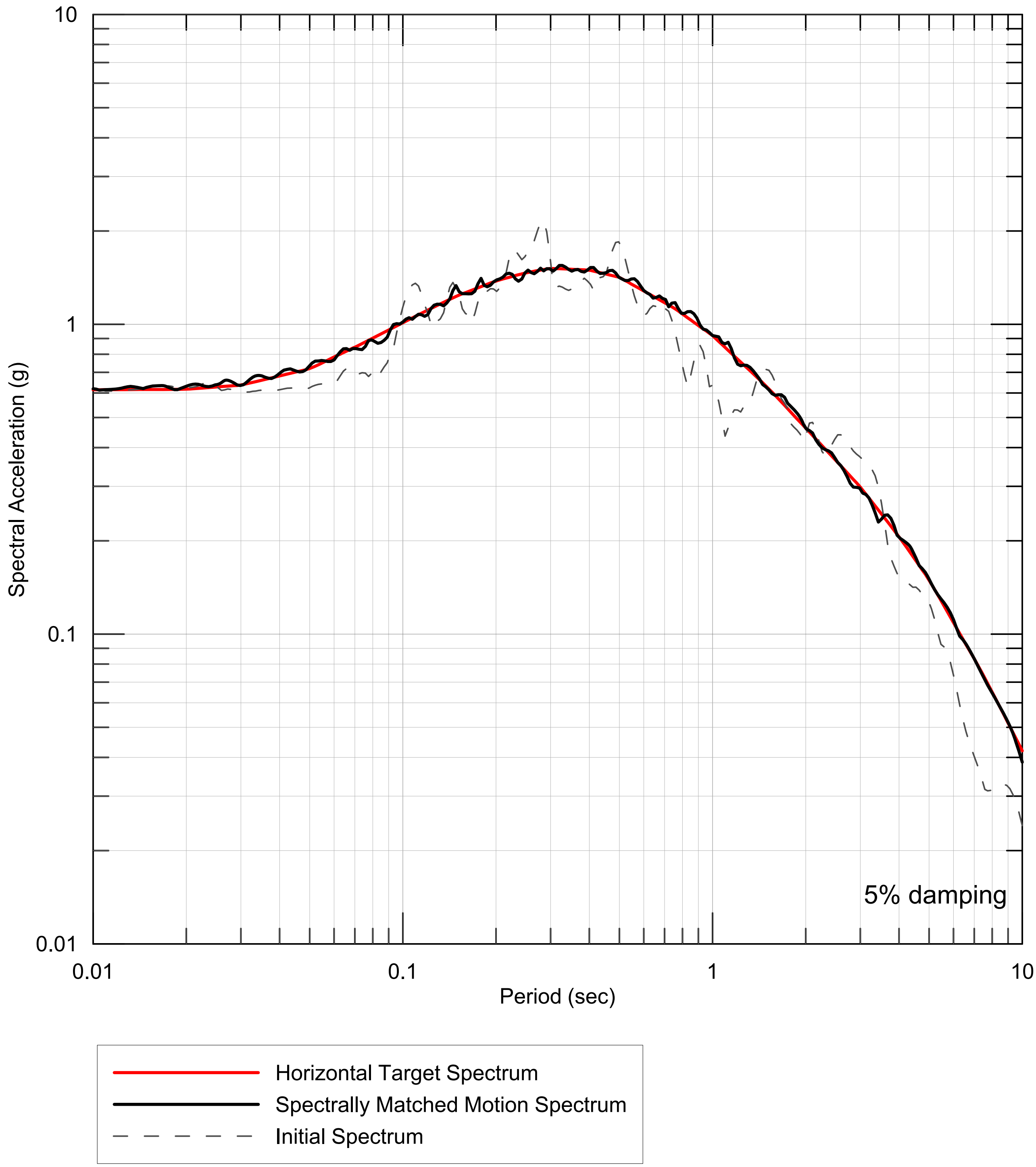
SEED: PEER RSN900

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 1992 LANDERS - YERMO F.S. (FN) SEED	Figure 13



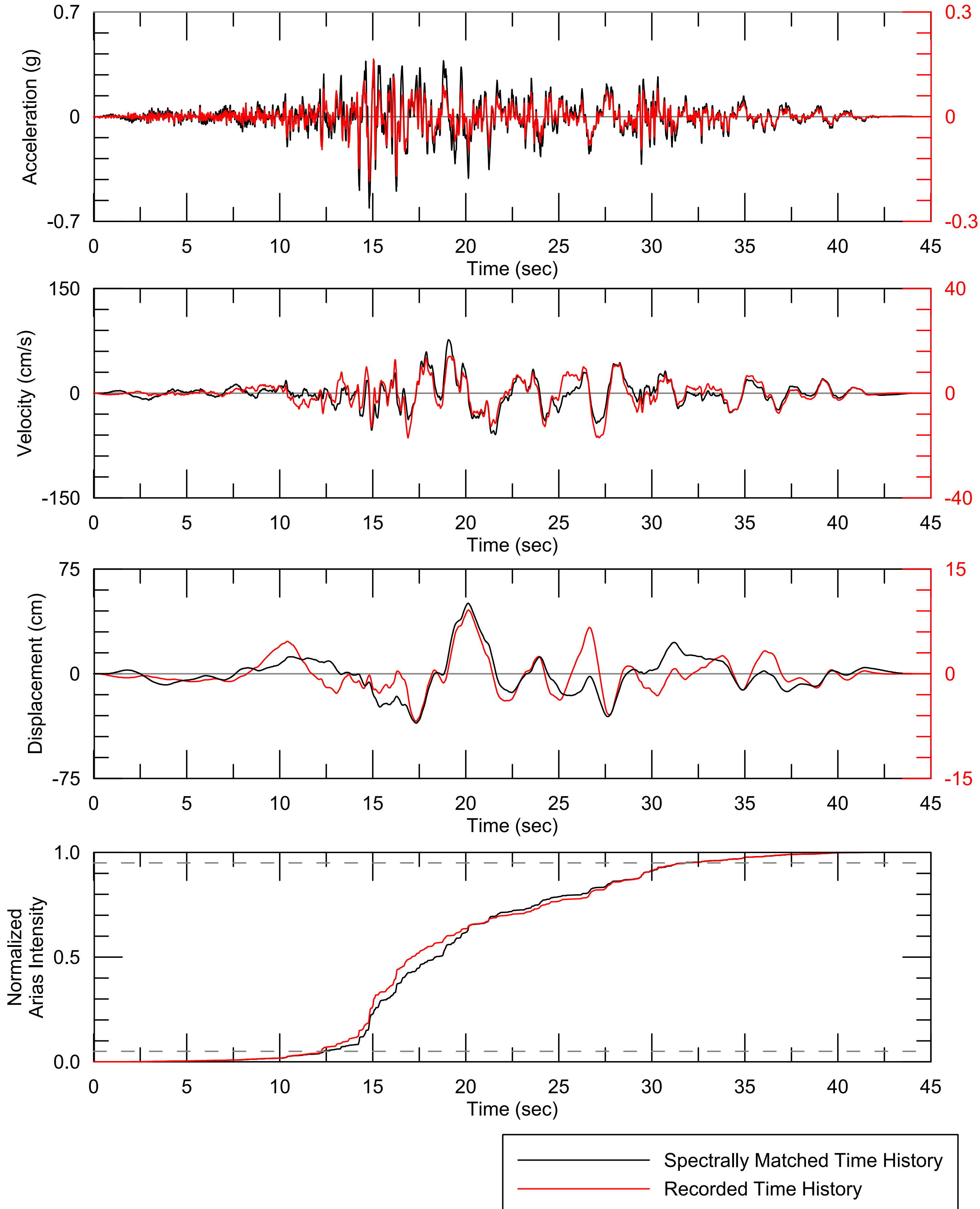
SEED: PEER RSN900

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 1992 LANDERS - YERMO F.S. (FN) SEED	Figure 14



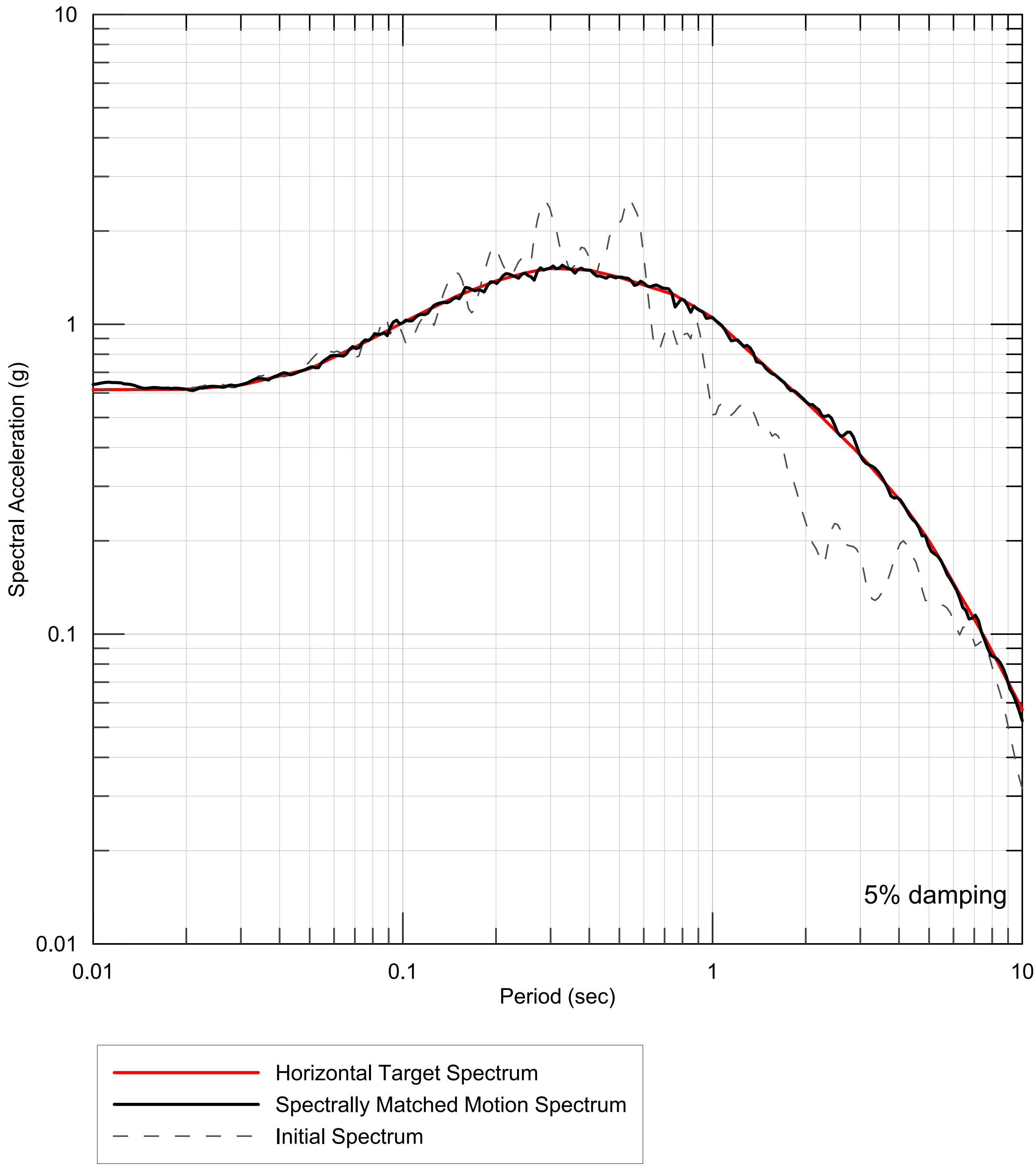
SEED: PEER RSN900

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 1992 LANDERS - YERMO F.S. (FP) SEED	Figure 15



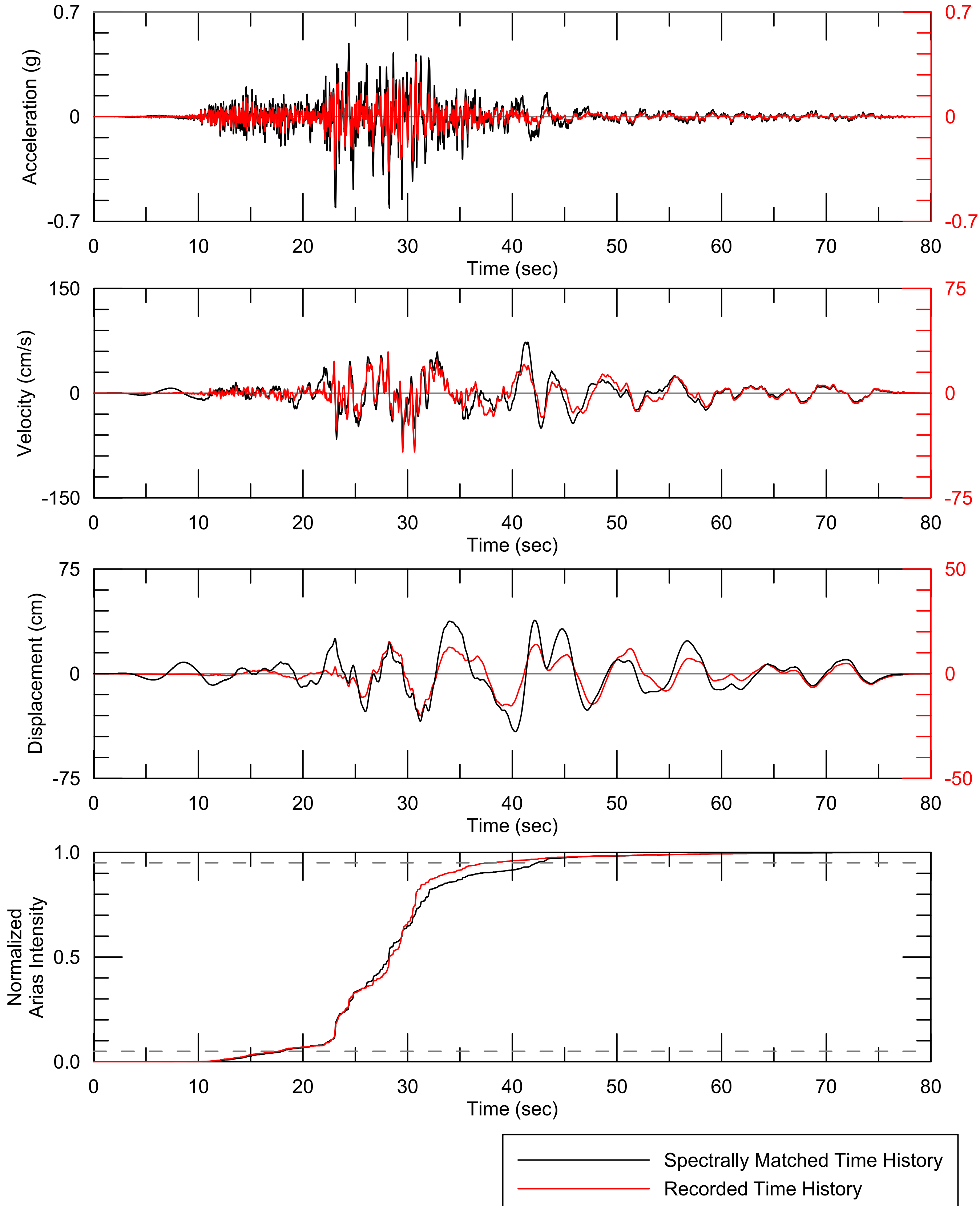
SEED: PEER RSN900

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 1992 LANDERS - YERMO F.S. (FP) SEED	Figure 16



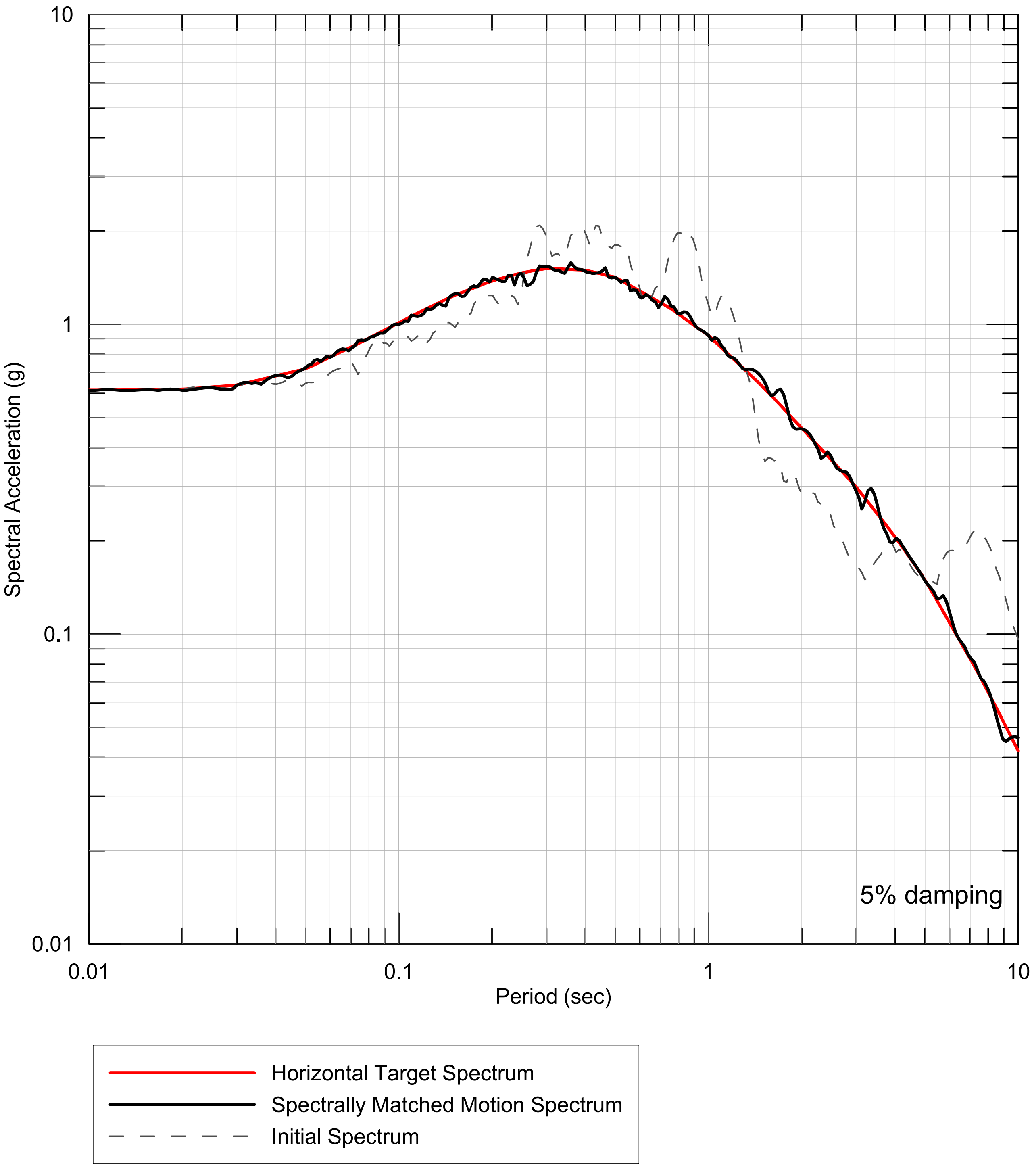
SEED: PEER RSN5991

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 EL MAYOR-CUCAPAH - E.C. ARRAY #10 (FN) SEED	Figure 17



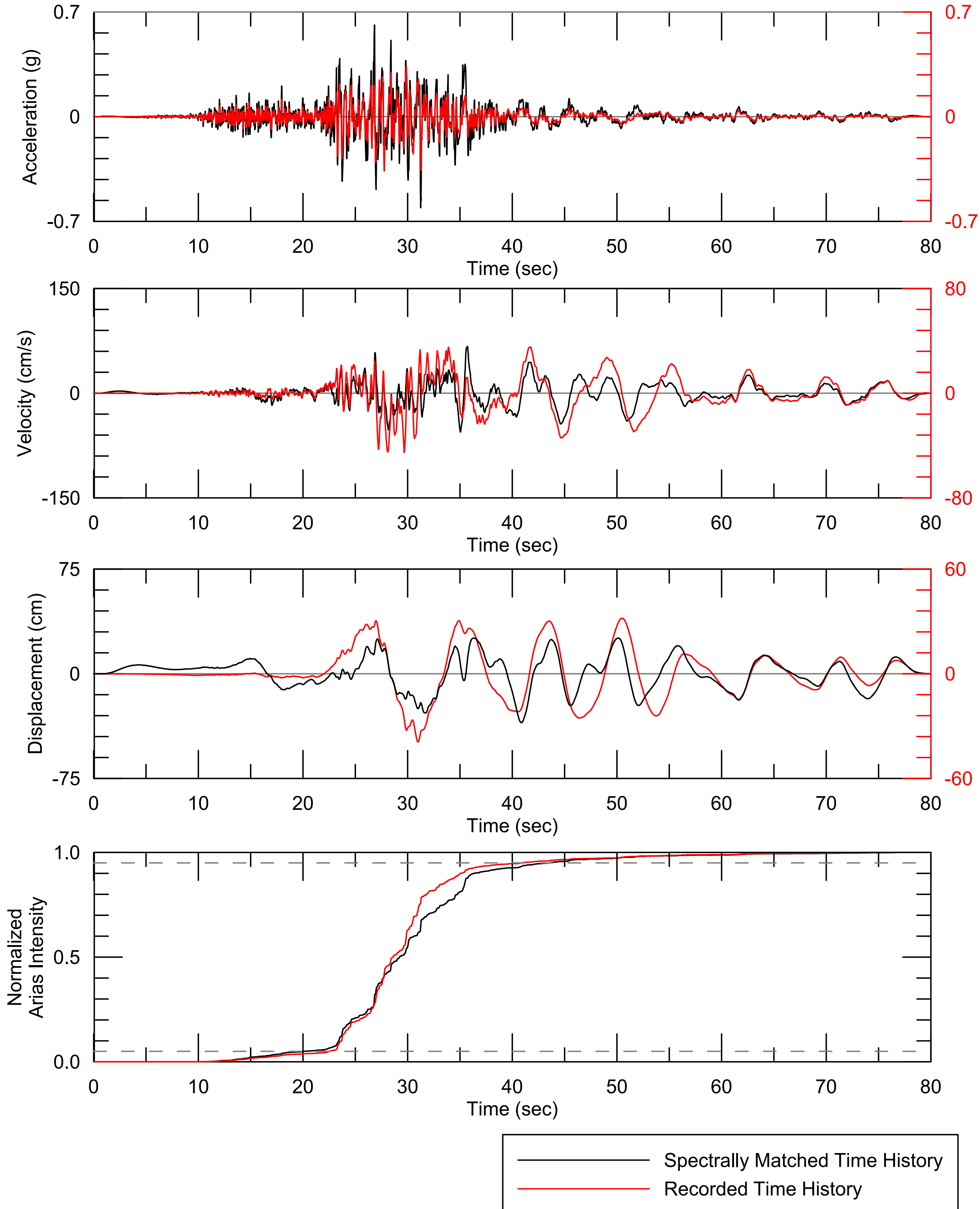
SEED: PEER RSN5991

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 EL MAYOR-CUCAPAH - E.C. ARRAY #10 (FN) SEED	Figure 18



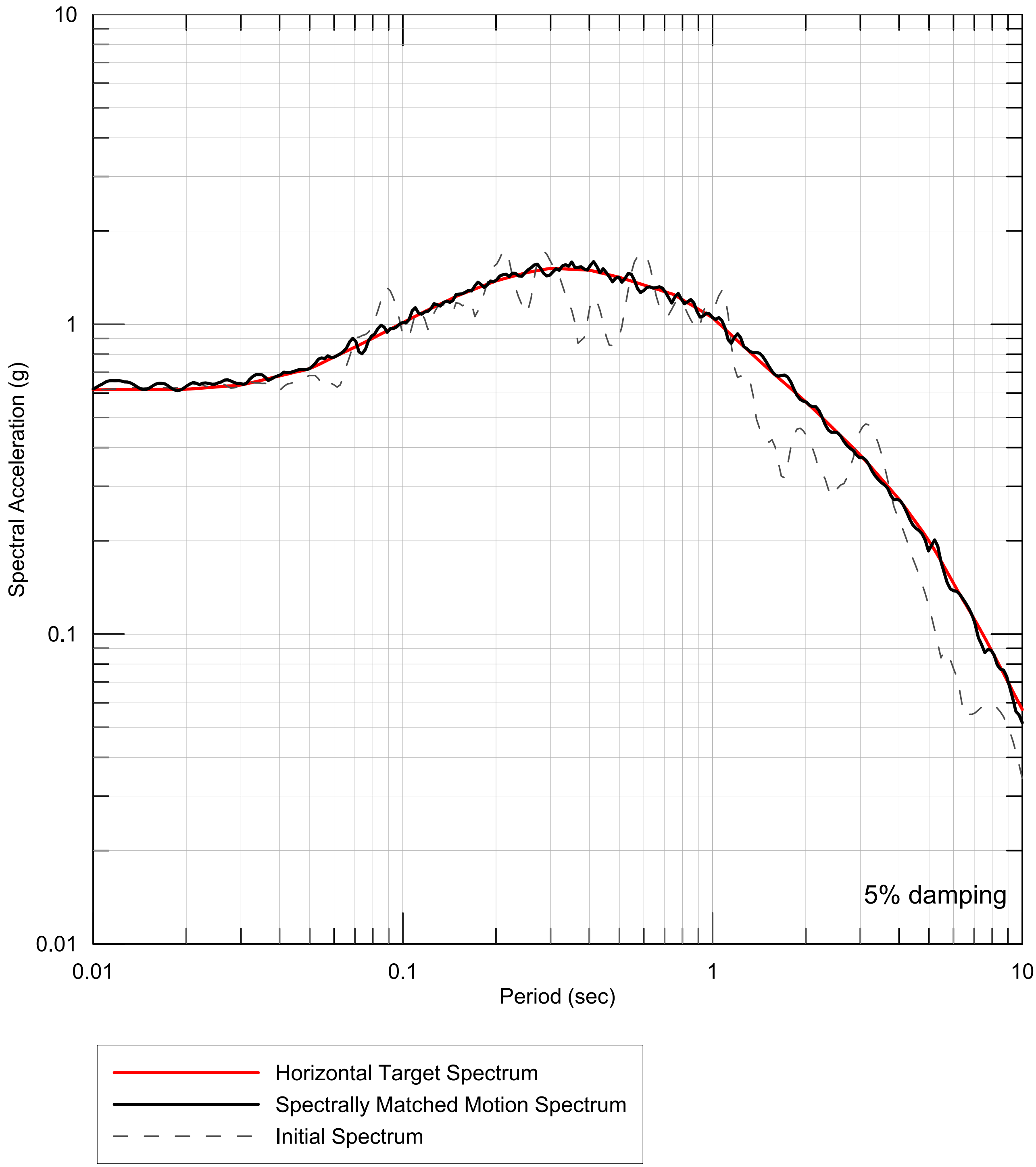
SEED: PEER RSN5991

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 EL MAYOR-CUCAPAH - E.C. ARRAY #10 (FP) SEED	Figure 19



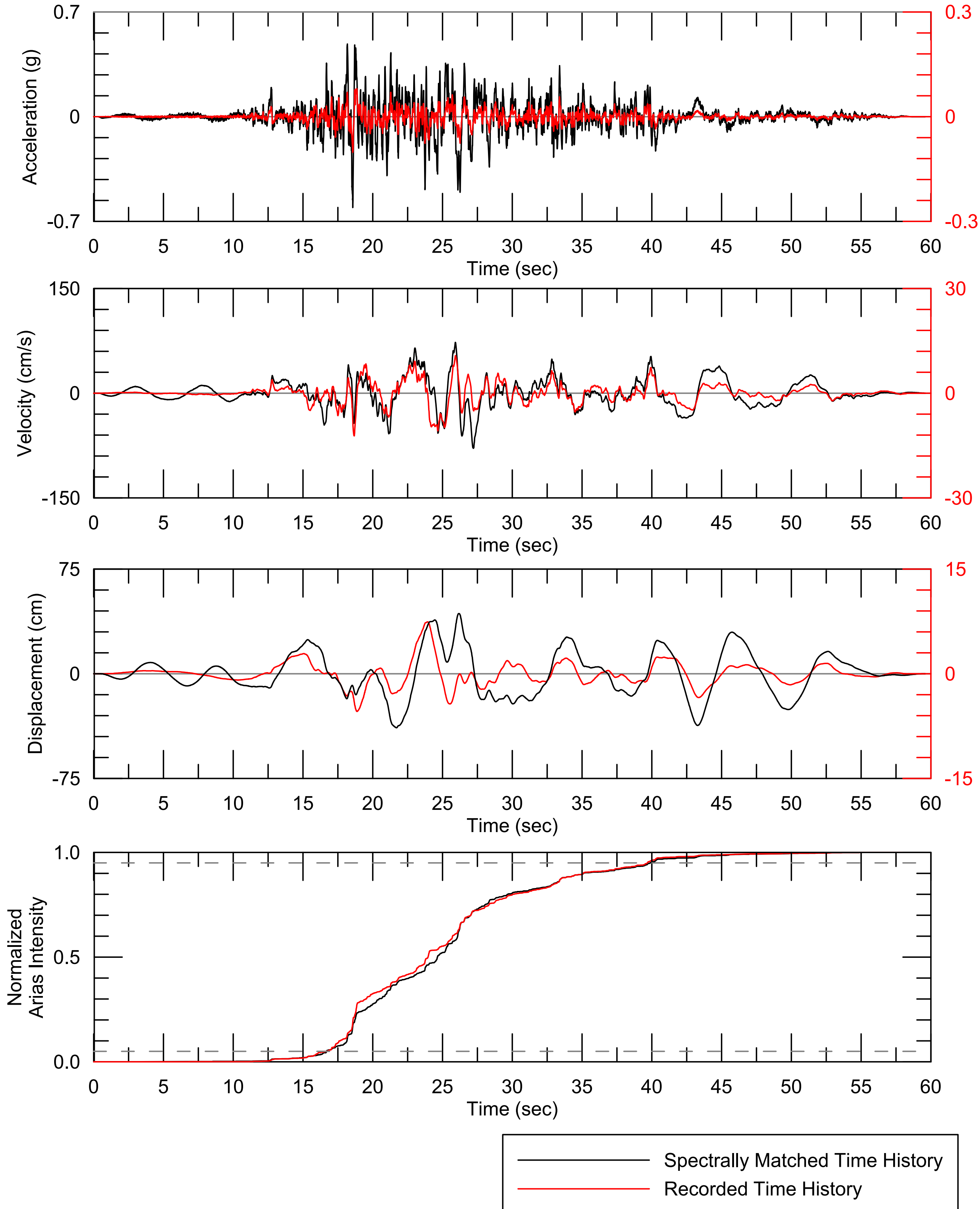
SEED: PEER RSN5991

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 EL MAYOR-CUCAPAH - E.C. ARRAY #10 (FP) SEED	Figure 20



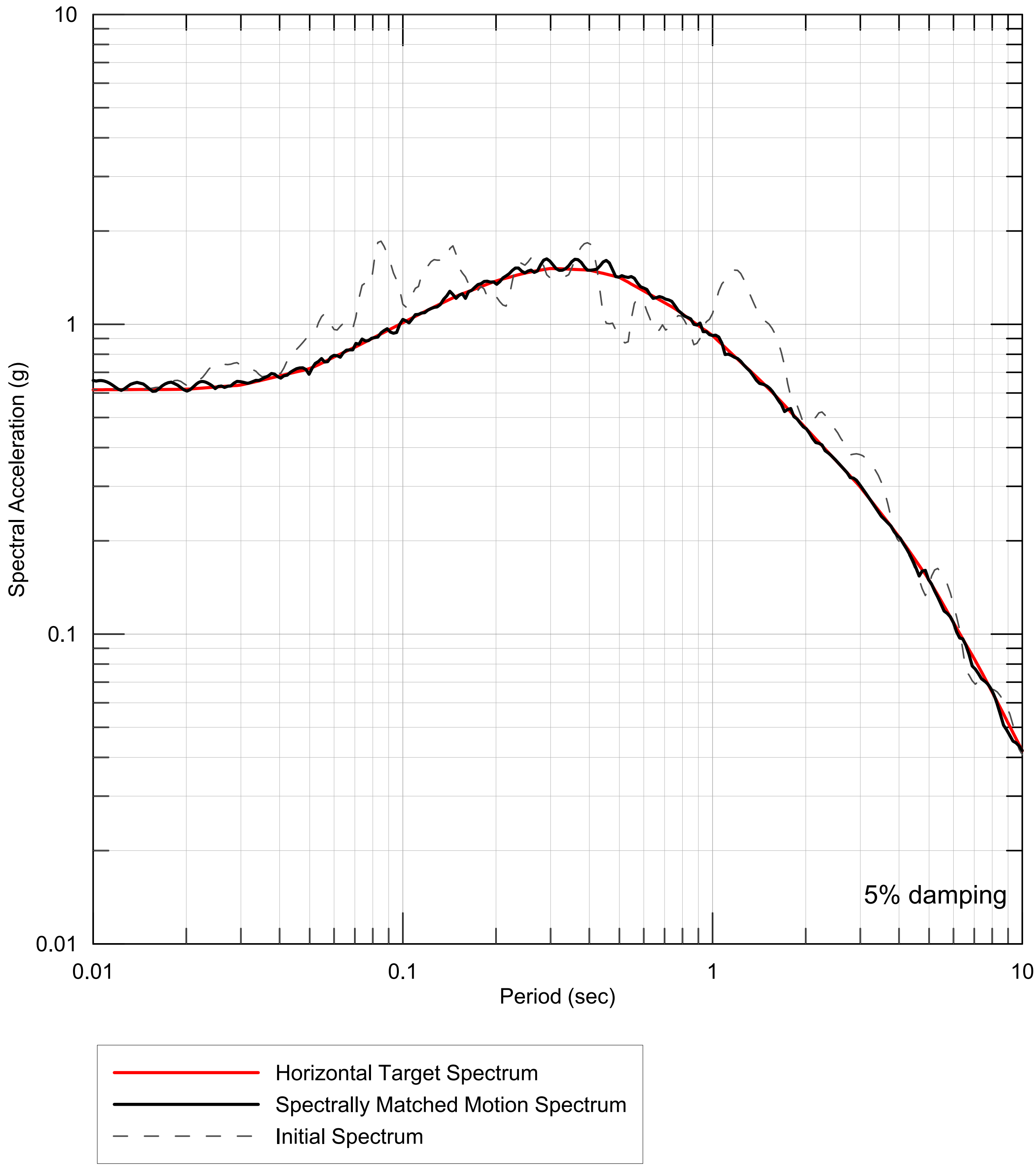
SEED: PEER RSN6930

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 DARFIELD - LRCS (FN) SEED	Figure 21



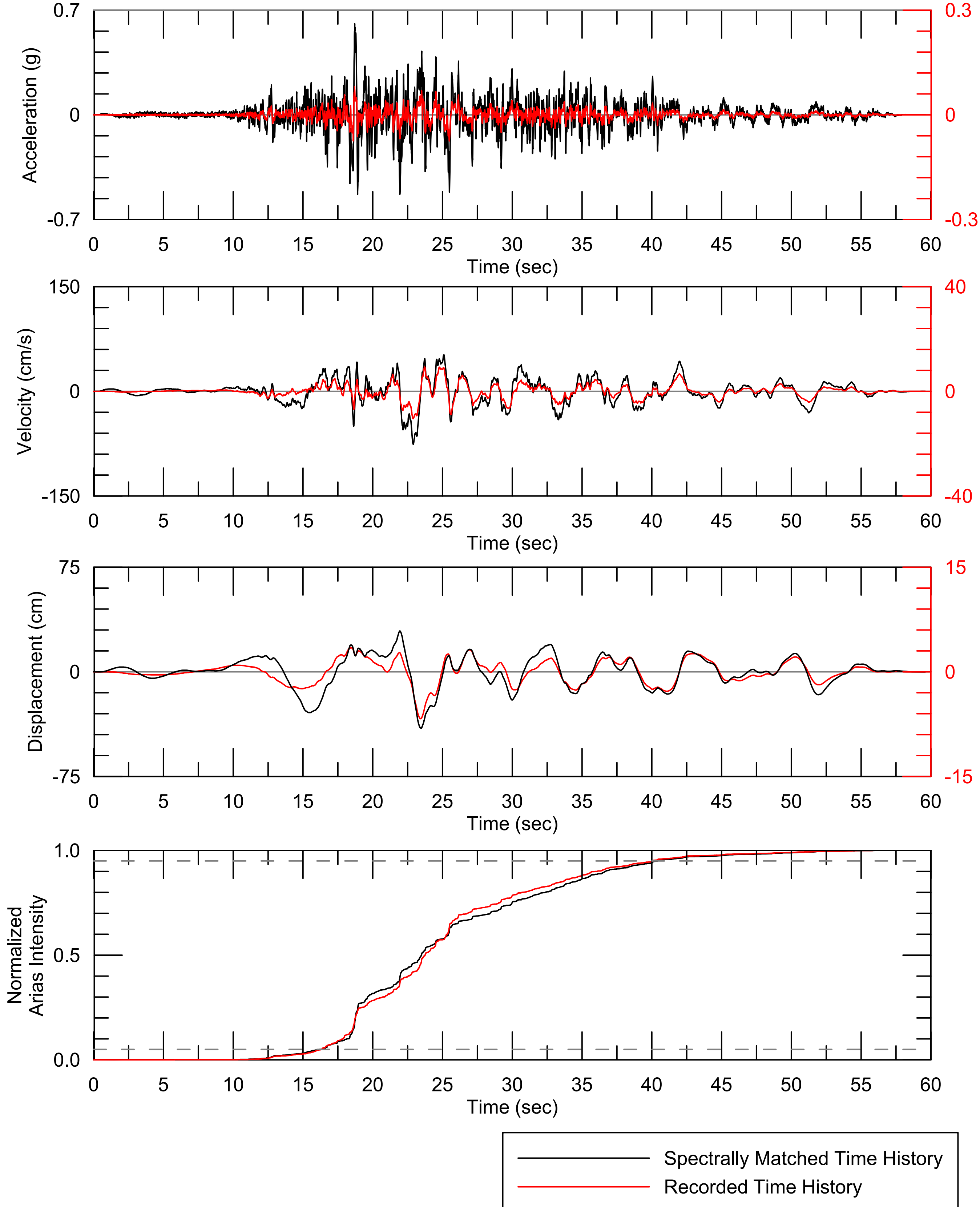
SEED: PEER RSN6930

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 DARFIELD - LRCS (FN) SEED	Figure 22



SEED: PEER RSN6930

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	RESPONSE SPECTRUM FOR TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 DARFIELD - LRCS (FP) SEED	Figure 23



SEED: PEER RSN6930

AECOM	Project No. 60572210 Lafayette Reservoir Outlet Tower Seismic Retrofit Project	TIME HISTORY SPECTRALLY MATCHED TO HORIZONTAL HAYWARD FAULT 84TH-PERCENTILE 2010 DARFIELD - LRCS (FP) SEED	Figure 24

AECOM
300 Lakeside Drive, Suite 400
Oakland, CA 94612
aecom.com

Appendix G

Concrete Testing Results at Lafayette Reservoir Outlet Tower

INSPECTION SERVICES INC.

September 14, 2018
ISI File No.: 2301-065.0
ISI Lab No.: T-62775
MCSS Task / P.O. No.: 103034

Fariborz Vossoughi, Ph.D., P.E.
Senior Engineer/Project Manager
AECOM
300 Lakeside Dr., Suite 400
Oakland, CA 94612

Re: Concrete Testing on Cores from the Lafayette Reservoir Outlet Tower

Dear Mr. Vossoughi:

Per AECOM's request, Inspection Services Inc. (ISI) took nine (9) four-inch nominal diameter concrete cores, roughly 6" to 8" long, from the Lafayette Reservoir Outlet Tower on August 29th and 30th, 2018. We labelled each core and recorded its sampling location, then performed eight (8) compression tests and two (2) E-Modulus tests in our Berkeley laboratory. Ground penetrating radiation (GPR) was used before sampling to determine rebar spacing as well as to avoid cutting into the rebar during the coring process.

1. Reinforcing Steel Mapping

ISI performed GPR scanning on the Lafayette Reservoir Outlet Tower and marked the reinforcement indications on the concrete surface at each location. As requested, there were three (3) sampling locations at approximate Elevations 444, 464, and 484 feet, which correspond to chest-height above each ladder landing platform. We found the reinforcement layout at each sampling location to be consistent: vertical reinforcement was approximately 10" to 12" on center and horizontal reinforcement being approximately 16" on center.

2. Concrete Cores for Compressive Strength and E-Modulus

Avoiding the marked rebar, three (3) cores were taken from each of the sampling elevations for a total of nine (9) samples. As requested, ISI inspectors minimized the runoff from the wet coring process entering the reservoir by using a special wet vacuum attachment. Each core was labelled and its sampling location recorded. The core holes were then patched with high-strength, non-shrink grout as requested.

ISI's lab performed initial compression tests on six (6) cores, two (2) from each sampling location in order to obtain an average compressive strength for each elevation and establish the baseline as is prescribed by the ASTM for E-Modulus testing. The three (3) remaining cores were then tested for modulus of elasticity. Unfortunately, one core (Sample A, from elevation 444') broke prematurely during its initial seating process so that we were unable to get any usable data from this sample. The modulus tests on Samples E and G were subsequently performed and these cores were additionally tested for compressive strength after E-Modulus testing was successfully completed.

All tests were conducted in accordance with the latest editions of ASTM C469 (Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression) and ASTM C42 (Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete).

The following Table 1 presents the results of these tests.

Table 1: Concrete Cores for Compressive Strength and E-Modulus

Core ID	Sampling Elevation [ft]	Diam. [in] x Length [in]	Ult. Load [lbs]	Area [in^2]	Corr. Factor [-]	Comp. Strength [psi]	E-Modulus [psi]	Avg. Comp. Strength per Elevation [psi]
A	444	3.69 x 6.43	-	10.69	0.98	-	*	**6,950**
B	444	3.98 x 7.58	84,500	12.44	1.00	6,790	-	
C	444	3.98 x 7.60	88,500	12.44	1.00	7,110	-	
D	464	3.98 x 8.24	47,900	12.44	1.00	3,850	-	**3,770**
E	464	3.69 x 7.02	41,000	10.69	1.00	3,840	**3,370,000**	
F	464	3.98 x 7.97	44,900	12.44	1.00	3,610	-	
G	484	3.69 x 7.29	57,600	10.69	1.00	5,380	**4,150,000**	**4,630**
H	484	3.99 x 7.19	55,000	12.50	1.00	4,390	-	
I	484	3.96 x 7.34	50,600	12.32	1.00	4,110	-	

*No data available due to shearing of sample during the seating stage

As noted before, Sample A (Elevation 444') prematurely broke during the seating of the E-Modulus test and so that no data could be obtained from this sample. However, the other cores' compressive strengths are fairly consistent within each sampling location and the E-Modulus for each elevation closely matches the theoretical value as shown in Table 2.

Table 2: Concrete Cores E-Modulus % Error

Sample Elevation [ft]	Avg. Comp. Strength [psi]	$E_{Theoretical}$ [psi] = $57{,}000\sqrt{f_c'}$	E_{Actual} [psi]	Abs. Error [%]
444	6,950	4,750,000	-	-
464	3,770	3,500,000	3,370,000	**3.7**
484	4,630	3,880,000	4,150,000	**7.0**

3. Conclusion

The concrete strength among all samples tested per elevation turned out to be very consistent. Furthermore, the tested E-Moduli came very close to the theoretical values as calculated based on compressive strengths.
On the other hand, the concrete strength that we found at the lowest elevation (444') is markedly higher than the concrete strength tested at both higher elevations (464' and 484').

Please contact us should you have any further questions or require additional services.

Respectfully submitted,
Inspection Services, Inc. (ISI)

Prepared by:

Antoine Megevand
Project Manager

Reviewed by:

REGISTERED PROFESSIONAL ENGINEER
CAN S. CELIK
C 60852
GE 2583
Exp. 12/18
CIVIL GEOTECHNICAL
STATE OF CALIFORNIA

Can S. Celik P.E., G.E.
Senior Geotechnical Engineer

Attachments: Individual photos of cores before capping, after capping, and after testing
Excel worksheets of the E-Modulus tests


4


4


A
Inspection Services,
Pier 26, The Embarcadero,


B


B
I


C


NO.C305R

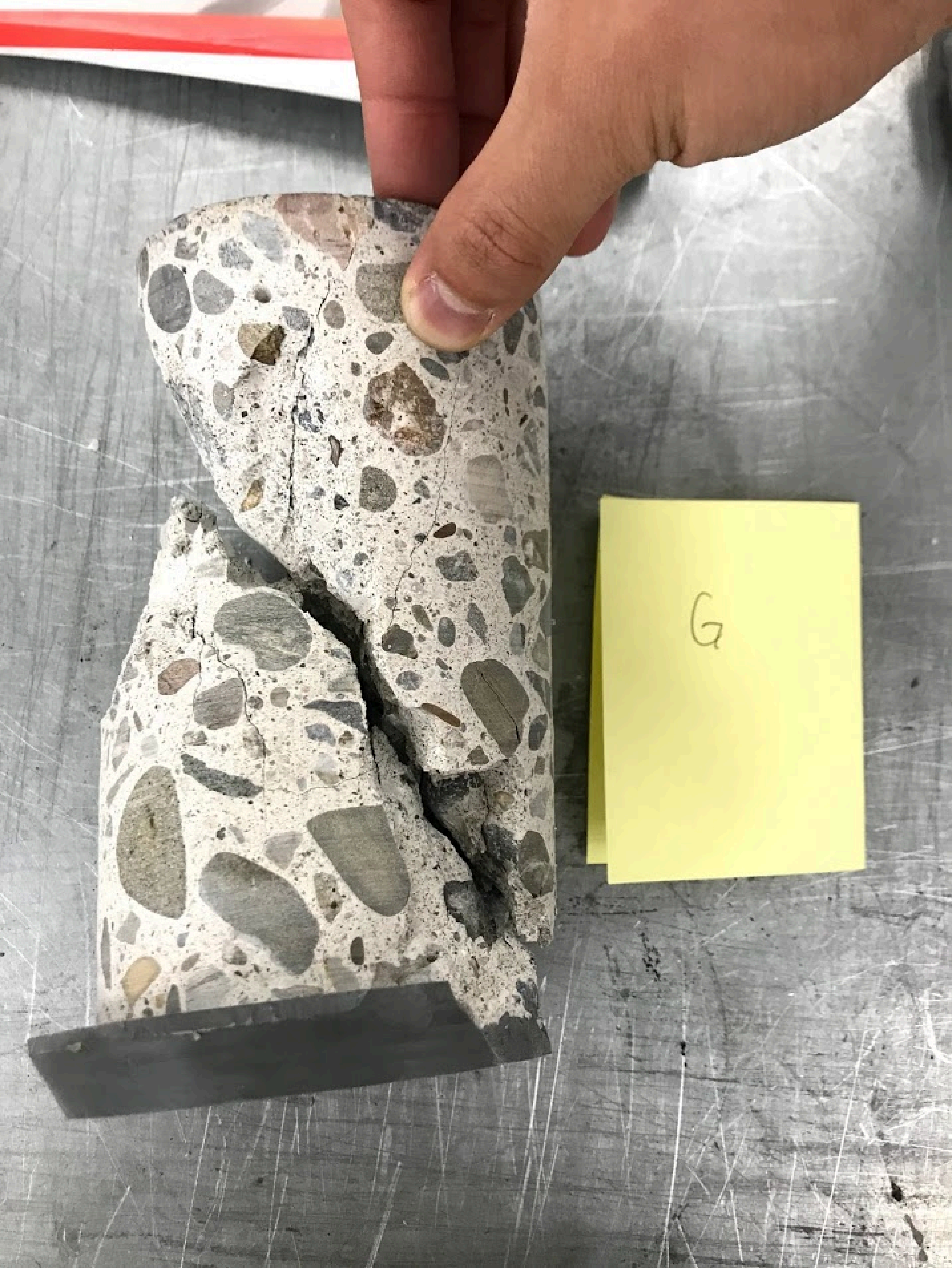

8058


No. C305R


THE L.S.STARRETT
No.C305R


NO. C305R


THE L.S.STARRETT
No.C305R


G


NO. C305R


H


H


H
No. C305R

Sample E

	E Mod (psi)	E Mod avg (psi)
#1	3310344.828	3369458.128
#2	3428571.429	

Trial 1

Stress (psi)	Gauge Reading (in)	Longitudinal Deformation (in)	Longitudinal Strain []
200	0.00015	0.000075	0.0000375
400	0.00025	0.000125	0.0000625
600	0.0005	0.00025	0.000125
800	0.0008	0.0004	0.0002
1000	0.00095	0.000475	0.0002375
1200	0.00115	0.000575	0.0002875
1400	0.0014	0.0007	0.00035
1600	0.0017	0.00085	0.000425

Trial 2

Stress (psi)	Gauge Reading (in)	Longitudinal Deformation (in)	Longitudinal Strain []
200	0.0001	0.00005	0.000025
400	0.00025	0.000125	0.0000625
600	0.0005	0.00025	0.000125
800	0.0008	0.0004	0.0002
1000	0.00095	0.000475	0.0002375
1200	0.0011	0.00055	0.000275
1400	0.00145	0.000725	0.0003625
1600	0.00165	0.000825	0.0004125

Sample G

	E Mod (psi)	E Mod avg (psi)
#1	4148148.148	4148148.148
#2	4148148.148	

Trial 1

Stress (psi)	Gauge Reading (in)	Longitudinal Deformation (in)	Longitudinal Strain []
200	0.0002	0.0001	0.00005
400	0.0005	0.00025	0.000125
600	0.00065	0.000325	0.0001625
800	0.0008	0.0004	0.0002
1000	0.001	0.0005	0.00025
1200	0.0013	0.00065	0.000325
1400	0.0015	0.00075	0.000375
1600	0.0016	0.0008	0.0004
1800	0.00185	0.000925	0.0004625

Trial 2

Stress (psi)	Gauge Reading (in)	Longitudinal Deformation (in)	Longitudinal Strain []
200	0.0002	0.0001	0.00005
400	0.00055	0.000275	0.0001375
600	0.0007	0.00035	0.000175
800	0.00085	0.000425	0.0002125
1000	0.00105	0.000525	0.0002625
1200	0.00135	0.000675	0.0003375
1400	0.0015	0.00075	0.000375
1600	0.00165	0.000825	0.0004125
1800	0.0019	0.00095	0.000475

Analysis By: Joonhee Kim
Date:1/23/2019
Checked By: Chris Abela, PE
Date:1/23/2019

Project: Lafayette Reservoir Outlet Tower Seismic Retrofit

Worksheet: Concrete Compressive Strength Calculation

Codes: ACI 214.4R-10 (ACI, 2010), EM 1110-2-6052 (USACE, 2007)

Description: Calculates equivalent concrete strength from concrete testing data in accordance with codes referenced above.

Notes to QC Reviewer:

1. Concrete strength increased based on relationship between static and dynamic properties in accordance with EM 6053, Section 5-1d. Factor takes into account for effect of increase in material properties
2. Assume standard treatment per ASTM C42
3. Assume all samples L/D are at or appx. 2

References:

1. Concrete Testing Results/Data at Lafayette Reservoir Outlet Tower (2018)

Analysis By: Joonhee Kim
Date:1/23/2019
Checked By: Chris Abela, PE
Date:1/23/2019

Compressive Strength of Concrete Cores

Core	Sample Elevation (ft)	Compressive Strength (psi)	Average Compressive Strength per EL (psi)
A	444	*	
B	444	6,790	6,950
C	444	7,110	
D	464	3,850	
E	464	3,840	3,770
F	464	3,610	
G	484	5,380	
H	484	4,390	4,630
I	484	4,110	

* No data available due to shearing of sample

El 444

$f_1 := 6790\text{psi}$

$f_2 := 7110\text{psi}$

El 464

$f_3 := 3850\text{psi}$

$f_4 := 3840\text{psi}$

$f_5 := 3610\text{psi}$

El 484

$f_6 := 5380\text{psi}$

$f_7 := 4390\text{psi}$

$f_8 := 4110\text{psi}$

	Factor	Mean value	Coefficient of variation V, %
$F_{\ell/d}$: ℓ/d ratio†	Standard treatment‡:	$1-\{0.130-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Soaked 48 hours in water:	$1-\{0.117-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
	Dried§:	$1-\{0.144-\alpha f_{core}\}\left(2-\frac{\ell}{d}\right)^2$	$2.5\left(2-\frac{\ell}{d}\right)^2$
F_{dia}: core diameter	2 in. (50 mm)	1.06	11.8
	4 in. (100 mm)	1.00	0.0
	6 in. (150 mm)	0.98	1.8
F_{mc}: core moisture content	Standard treatment‡:	1.00	2.5
	Soaked 48 hours in water:	1.09	2.5
	Dried§:	0.96	2.5
F_d: damage due to drilling		1.06	2.5

*To obtain equivalent in-place concrete strength, multiply the measured core strength by appropriate factor(s) in accordance with Eq. (9-1).

†Constant α equals $3(10^{-6})$ 1/psi for f_{core} in psi, or $4.3(10^{-4})$ 1/MPa for f_{core} in MPa.

‡Standard treatment specified in ASTM C42/C42M.

§Dried in air at 60 to 70°F (16 to 21°C) and relative humidity less than 60% for 7 days.

Calculate equivalent in place strength, f_{ci}

$F_{ld} := 1.0$ Factor for L/D ratio

$V_{ld} := 0$ Coefficient of variation associated with $F_{l/d}$

$F_{dia} := 1.0$ Factor for diameter of core

$V_{dia} := 0$ Coefficient of variation associated with F_{dia}

$F_{mc} := 1.0$ Factor for moisture content of core

$V_{mc} := 0.025$ Coefficient of variation associated with F_{mc}

$F_d := 1.06$ Factor for damage due to drilling

$V_d := 0.025$ Coefficient of variation associated with F_d

$$f_{ci} := F_{ld} \cdot F_{dia} \cdot F_{mc} \cdot F_d \begin{pmatrix} f_1 \\ f_2 \\ f_3 \\ f_4 \\ f_5 \\ f_6 \\ f_7 \\ f_8 \end{pmatrix} = \begin{pmatrix} 7.197 \times 10^3 \\ 7.537 \times 10^3 \\ 4.081 \times 10^3 \\ 4.07 \times 10^3 \\ 3.827 \times 10^3 \\ 5.703 \times 10^3 \\ 4.653 \times 10^3 \\ 4.357 \times 10^3 \end{pmatrix} \cdot psi$$

Equivalent in place strengths (Eq 9-1)

Calculate mean in-place strength

$n := 8$ Number of samples

$$f_{cbar} := \frac{\sum f_{ci}}{n} = 5.178 \times 10^3 \cdot psi$$

Sample mean in-place strength (Eq 9-2)

$$s_c := \sqrt{\frac{\sum \left(f_{ci} - f_{cbar}\right)^2}{n-1}} = 1.47 \times 10^3 \cdot psi$$

Standard deviation of in-place strength (Eq 9-3)

$$s_a := f_{cbar} \cdot \sqrt{V_{ld}^2 + V_{dia}^2 + V_{mc}^2 + V_d^2} = 183.073 \cdot psi$$

Standard deviation of in-place strength due to empirical nature of strength correction factors (Eq 9-4)

Calculate equivalent design strength of concrete

Assume 75% confidence level. Per EM 1110-2-6053, add 15% increase in concrete strength to account for dynamic factor

Tolerance Factor Method:

$K_{75} := 1.74$

$Z_{75} := 1.28$

$$f_{c.design} := 1.15\left[f_{cbar} - \sqrt{\left(K_{75} \cdot s_c\right)^2 + \left(Z_{75} \cdot s_a\right)^2}\right] = 3.001 \times 10^3 \text{ psi} \quad \text{(Eq 9-7)}$$

This value is too low/conservative because of the high standard devation. The alternate method will be used to determine the equivalent design concrete compressive strength.

Alternate Method:

$T_{90} := 0.71$

$C := 0.85$

$$f_{cCL} := f_{cbar} - \sqrt{\frac{\left(T_{90} \cdot s_c\right)^2}{n} + \left(Z_{75} \cdot s_a\right)^2} = 4.741 \times 10^3 \cdot \text{psi}$$

Lower bound estimate of mean in-place strength (Eq 9-8)

$$f_{c.design.static} := C \cdot f_{cCL} = 4.03 \times 10^3 \cdot \text{psi} \quad \text{(Eq 9-9)}$$

$$f_{c.design.dynamic} := 1.15 f_{c.design.static} = 4.634 \times 10^3 \text{ psi}$$

$$E_{static} := 57000\sqrt{\left(f_{c.design.static} \cdot \text{psi}\right)} = 3.618 \times 10^6 \cdot \text{psi}$$

Static elastic modulus

$$E_{dynamic} := 1.15 E_{static} = 4.161 \times 10^6 \cdot \text{psi}$$

Dynamic elastic modulus

Table 9.2—*K*-factors for one-sided tolerance limits on the 10% fractile (Natrella 1963)

	Confidence level		
n	75%	90%	95%
3	2.50	4.26	6.16
4	2.13	3.19	4.16
5	1.96	2.74	3.41
6	1.86	2.49	3.01
8	1.74	2.22	2.58
10	1.67	2.06	2.36
12	1.62	1.97	2.21
15	1.58	1.87	2.07
18	1.54	1.80	1.97
21	1.52	1.75	1.90
24	1.50	1.71	1.85
27	1.49	1.68	1.81
30	1.48	1.66	1.78
35	1.46	1.62	1.73
40	1.44	1.60	1.70

Note: *n* = number of specimens tested.

Table 9.3—*Z*-factors for use in Eq. (9-7) and (9-8) (Natrella 1963)

Confidence level, %	*Z*
75	0.67
90	1.28
95	1.64

Table 9.4—One-sided *T*-factors for use in Eq. (9-8) (Natrella 1963)

	Confidence level		
n	75%	90%	95%
3	0.82	1.89	2.92
4	0.76	1.64	2.35
5	0.74	1.53	2.13
6	0.73	1.48	2.02
8	0.71	1.41	1.90
10	0.70	1.38	1.83
12	0.70	1.36	1.80
15	0.69	1.34	1.76
18	0.69	1.33	1.74
21	0.69	1.33	1.72
24	0.69	1.32	1.71
30	0.68	1.32	1.70

Note: *n* = number of specimens tested.

Table 9.5—*C*-factors for use in Eq. (9-9)

Structure composed of:		One member	Many members
One batch of concrete		0.91	0.89
Many batches of concrete	Cast-in-place	0.85	0.83
	Precast	0.88	0.87