Subject	**Liquefaction and Ground Improvement Analysis for Bethany Reservoir Alternative (Final Draft)**
Project feature	Geotechnical
Prepared for	California Department of Water Resources (DWR) / Delta Conveyance Office (DCO)
Prepared by	Delta Conveyance Design and Construction Authority (DCA)
Copies to	File
Date/Version	December 23, 2021
Reference no.:	EDM_GE_CE_TMO_Liq-Ground-Imp-Analysis-Bethany_001004_V02_FD_20211223

1. Purpose

This supplementary technical memorandum (TM) was prepared to summarize the liquefaction potential and a possible liquefaction mitigation approach at three sites along the alignment of the Delta Conveyance Project's Bethany Reservoir Alternative. This TM is complementary to the *Liquefaction and Ground Improvement Analysis* for the Central and Eastern (C-E) Corridor Options (DCA, 2021a). The intakes and tunnel shaft sites described for the Eastern Corridor Option from Twin Cities Complex through Lower Roberts Island would be located on the same sites under the Bethany Reservoir Alternative. The Bethany Reservoir Alternative tunnel shaft site on the Upper Jones Tract would be located at a different location than under the Eastern Corridor Option; however, the two locations are relatively close and have similar characteristics related to liquefaction potential. This TM was created in support of developing a conceptual footprint for key elements of the Bethany Reservoir Alternative located south of Upper Jones Tract at Union Island and Bethany Complex. A detailed site-specific liquefaction analysis for these and other key elements, including the tunnels and aqueduct pipelines, would be performed upon acquisition of additional geotechnical information during the future design phase.

1.1 Organization

This TM is organized as follows:

- Overview
- Liquefaction Potential Calculations
- Liquefaction Mitigation Calculations
- Findings
- Conclusions
- References
- Document History and Quality Assurance

2. Overview

The Bethany Reservoir Alternative includes the following main components:

- A tunnel and associated shafts
- The Bethany Reservoir Pumping Plant (BRPP) and Surge Basin located southeast of the Clifton Court Forebay

- The Bethany Reservoir Aqueduct (Aqueduct) four buried pipelines leading from the BRPP to Bethany Reservoir Discharge Structure
- The Bethany Reservoir Discharge Structure
- Buried welded steel discharge pipeline connecting to the Jones Approach Canal Aqueduct for conveying pumped flow to the Jones Pumping Plant approach channel (7,500 cubic feet per second project design capacity only)

This TM presents a conceptual-level, liquefaction potential evaluation and possible foundation soils mitigation at the following locations, as shown in Attachment 1:

- Union Island Tunnel Maintenance Shaft
- BRPP and Surge Basin
- Bethany Reservoir Discharge Structure

For site soils that are susceptible to liquefaction, a deep mechanical mixing (DMM) approach to mitigate liquefaction is considered, as described in DCA (2021a).

2.1 Geotechnical Information

Soil borings and cone penetrometer test (CPT) soundings were completed at locations near the Bethany Reservoir Alternative tunnel alignment and BRPP during the prior environmental assessment phase of the former WaterFix Project, as documented in the *Environmental Impact Report* (DWR 2013) and *Environmental Impact Statement* (DWR, 2018). Additional soil boring and CPT sounding logs were also available from DWR's Atlas database (CCWD, 2008; DWR, 1967) while other data was digitized from a nearby project (SLTP, 2018).

As described in DCA (2021a), Delta geology is characterized by buried river channels, abundant sand lenses, and upper layers of organic-rich soil. The groundwater level is generally about 5 feet below ground surface (bgs) within much of the Delta (DWR 2013; DWR 2018); however, historical boring logs at the Tracy Pumping Plant indicate the groundwater level deepens toward the southern end of the Delta and generally approaches 10 feet bgs near the BRPP (USBR 1947).

South of the BRPP location, the geology changes beyond the margins of the historical Delta and consists of colluvium from the coast range. Farther to the south, the Bethany Reservoir Discharge Structure is underlain by the Panoche Formation, consisting of marine sandstones, clay shales, and minor siltstones. The sandstones are occasionally concretionary, and the clay shales are often thinly bedded, deeply weathered, soft, and friable. These sedimentary formation beds generally dip to the northeast at 20 degrees from horizontal.

2.2 Seismic Ground Motions

The current draft Project seismic design criteria specify a combination of probabilistic and deterministic ground motions for conceptual design, depending on the facility type (DCA, 2021b). Preliminary probabilistic and deterministic seismic hazard analyses were performed using the latest generation of earthquake ground motion attenuation relationships and fault source models, as presented in Attachment 2. Table 1 summarizes the preliminary ground motions by facility that were used in the liquefaction analysis. The PGA values presented in this table are those at the ground surface and were estimated by multiplying the amplification factors obtained from the site response analysis (DCA, 2021c) to the MDE or MCE PGAs at the reference site.

Table 1. Preliminary Ground Motions at Bethany Alternative Facility Sites

Facility	Maximum Design Earthquake[a]	Controlling Magnitude (M_w)[b]	Peak Ground Acceleration (% of g)[c]	Amplification Factor[d]
Union Island Tunnel Maintenance Shaft	Envelope of 2,475-year probabilistic and 84th-percentile deterministic ground motions	6.9	0.55	0.37
Bethany Reservoir Pumping Plant	2019 CBC (MCE)		0.58	0.57
Bethany Reservoir Discharge Structure	975-year probabilistic ground motions		0.59	N/A[e]

[a] Delta Conveyance Draft Seismic Guidelines (DCA, 2021b).
[b] Controlling earthquake magnitude for deterministic ground motions.
[c] Per Attachment 2, for a reference stiff soil site (Site Class D, with reference seismic shear-wave velocity (V_{s30}) = 1,100 feet per second [fps] or 335 meters per second).
[d] Factors calculated using 1-D site response analysis, described in DCA, 2021c.
[e] Ground motions not considered in evaluation, as discussed in Section 5.3.

Notes:
% = percent
g = acceleration due to gravity
M_w = moment magnitude
N/A = not applicable

3. Liquefaction Potential Calculations

3.1 Analytical Standard Penetration Test Procedure

Liquefaction potential was evaluated at the Bethany Reservoir Pumping Plant and Union Island Tunnel Maintenance Shaft sites using the Youd et al. approach (2001), as described in DCA (2021a). As in the Central-Eastern Alignment TM, the standard penetration test (SPT) analysis was used to determine the DMM wall-to-wall spacing.

The cyclic shear resistance at each facility was scaled by an age factor *K*. Preliminary radiocarbon dating tests collected on Bouldin Island and Lower Roberts Island indicate that the average age of the Modesto formation (Qm) is approximately 10,000 years before present, and the age of the Riverbank Formation (Qr) is approximately 40,000 years before present. Following the methodology described in Hayati et al. (2008), the ages of these geologic units result in approximate age factors of *K* = 1.5 and 1.6, respectively. The depth to the Modesto and Riverbank Formation at each site was taken from Maier et al. (2013) and Gatti et al. (2013).

At each site, an unimproved (native) factor of safety (FS) against liquefaction was calculated as the ratio of soil cyclic resistance ratio to cyclic shear stress ratio. An improved FS is also calculated and reflects the improved conditions after implementing a grid of DMM walls.

Attachment 3 summarizes the results of the calculated FS; where the liquefaction potential is indicated as “no” for cases where the unimproved or improved FS is at or above 1.0, and “yes” where the unimproved or improved FS is below 1.0. This approach does not apply for the rock anticipated at the Bethany Reservoir Discharge Structure site, as rock is not susceptible to liquefaction.

3.2 Analytical Cone Penetration Test Procedure

Liquefaction potential was evaluated at the BRPP and Union Island Tunnel Maintenance Shaft sites using CPT data and CLiq 3.0 software (Geologismiki 2020), as described in DCA (2021a). This exercise used data from a representative CPT sounding from the Union Island Tunnel Maintenance Shaft and the BRPP. The resulting FS versus depth plot was analyzed in conjunction with the liquefaction potential index (LPI) plot at each site to produce an anticipated liquefaction depth and compared against the SPT analysis to determine the depth of liquefaction. Attachment 4 provides the calculated FS and LPI plots.

Age factors were also applied to the Qr and Qm soils in the CPT analyses, as described in Section 3.1.

The results of the CPT-based liquefaction analysis were somewhat inconsistent with those of the SPT-based analysis at the BRPP. The CPT-based analysis results suggest significant liquefaction from depths of about 10 to 75 feet, whereas little to no liquefaction is predicted by the SPT data. These are due to soil type determination using the CPT data, which resulted in being classified as sandy/silty areas; and the observed and logged conditions in the soil boring were clayey sand and sandy clay (analyzed at six depths). As sandy/silty soils are susceptible to liquefaction, more widespread liquefaction is predicted by the CPT data. The CPT sounding predicted N_{60} values that were, on average, 25 percent of what was actually measured at the same depth in the adjacent soil boring at the Jones Approach Channel. Attachment 5 provides a comparative N_{60} plot from the boring and paired CPT. As Section 5.3 discusses, an alternative evaluation was completed based on shear wave velocity to confirm the SPT analysis findings; recommendations for future analysis are given in Section 6 of this TM.

4. Liquefaction Mitigation Calculation

The improved FS against liquefaction resulting from an implemented soil-cement grid (DMM panels) for each soil layer within each borehole was estimated using the SPT data and the procedures described in DCA (2021a), with additional cyclic resistance added using the age factor of the geologic unit. For each analysis, DCA estimated the required replacement ratio (A_r) for each sample interval, assuming a DMM panel thickness of 1 meter (36 inches), to achieve an FS against liquefaction of 1.0. Attachment 3 provides the results of these analyses. The recommended depth of liquefiable soils was taken as the more conservative depth between the SPT and CPT analyses. The analyses did not account for the placement and consolidation of fill material during construction of the shaft pads.

5. Findings

This section presents the findings of the analyses.

5.1 Union Island Tunnel Maintenance Shaft

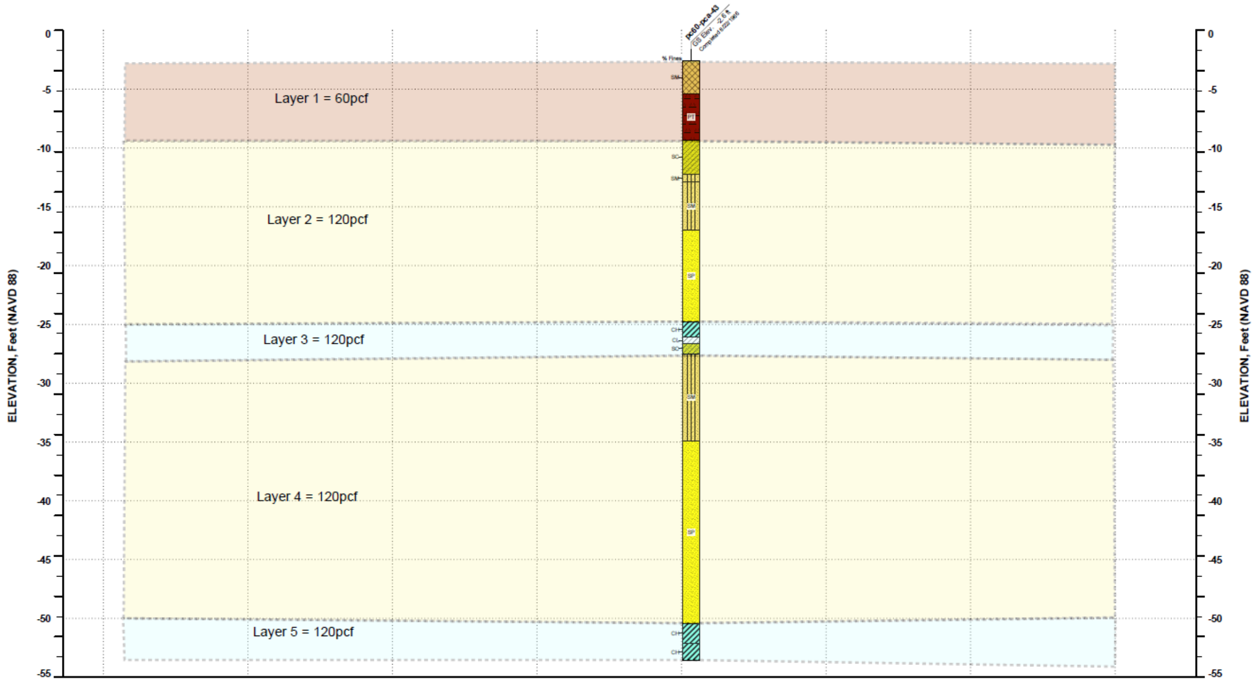

Figure 1. Conceptual Geologic Profile at Union Island Maintenance Shaft

The Union Island Tunnel Maintenance Shaft could be constructed within alternating layers of coarse-grained and fine-grained soils underlaying a surface layer of peat (Figure 1). Figure 2 provides the boring and CPT locations.

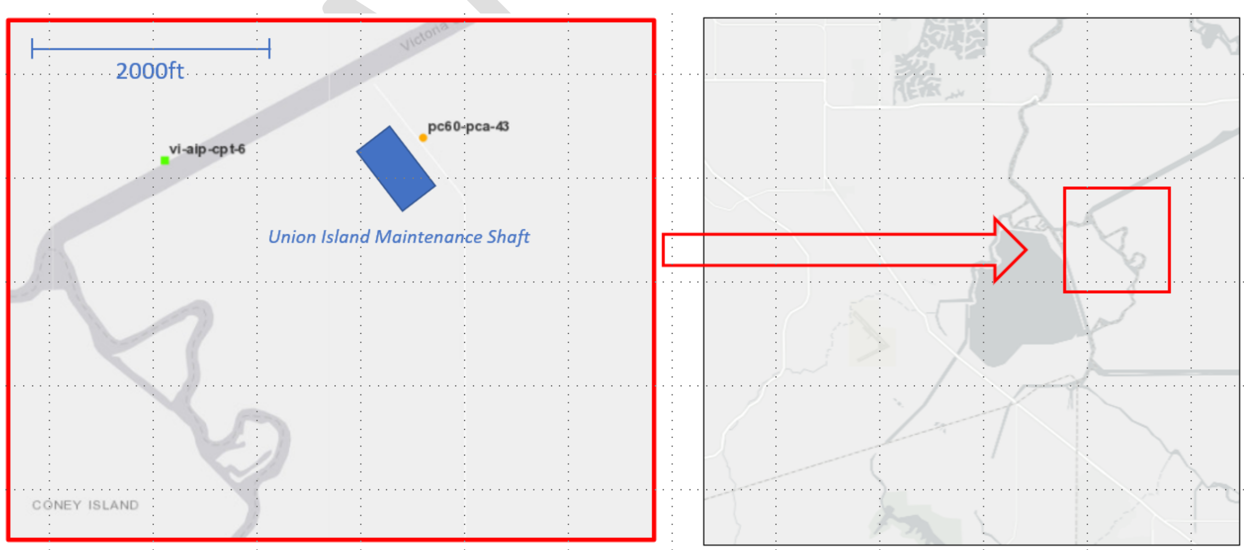

Figure 2. Plan View of Boring and CPT Used at Union Island Maintenance Shaft (Plan view is oriented north-south)

No recent data were available for this site, so data from the Peripheral Canal Project (DWR, 1967) were used. The historic boring was drilled at approximately -3 feet elevation (El.) (all references to elevation are North American Vertical Datum 1988 [NAVD88]) and samples were obtained by driving a 2.5-inch ID sampler. The subsurface soils at this site are characterized by a 7-foot-thick layer of fill and peat, followed by a layer of sand and silty sand terminating around a depth of 22 feet, overlying a thin layer of silty clay that terminates around a depth of 25 feet, followed by an additional layer of sand and silty sand terminating around 44 feet in depth and ending in a high-plasticity clay. For the liquefaction analyses, the earthquake groundwater depth was set to 0 feet bgs to reflect the mean water surface of the nearby slough.

Table 2 summarizes the average soil parameters for each soil layer evaluated in the SPT-based liquefaction analysis at the Union Island Tunnel Maintenance Shaft site. Attachment 3 provides the analysis results, which demonstrate the need to reinforce the soils to prevent liquefaction. Table 3 summarizes the required minimum DMM properties and depth of anticipated liquefaction (greater of SPT and CPT analyses). The minimum calculated spacing is defined as the center-to-center distance between the grid of DMM walls at each site required to maintain an FS greater than or equal to 1.0. The replacement ratio is defined as the percentage of area occupied by the DMM walls, given the corresponding DMM wall spacing and thickness, over a given volume of improved ground.

Table 2. Summary of Soil Parameters at Union Island Tunnel Maintenance Shaft

Explorations Used	Layer (refer to Figure 1)	Top Elev. (feet)	USCS[b]	Total Unit Weight (pcf)[c]	Average SPT N-value	Average Fines Content (%)	Average Shear Wave Velocity (fps)[d]	Shear Modulus G (ksi)
PC60-PCA-43 (BH#2)[a]	1	-3	PT	60	4	50	400	2.0
	2	-10	SM	120	19	15	450	5.5
	3	-25	CH	120	12	100	550	7.1
	4	-28	SM (Qm)	120	41	20	650	11.2
	5	-50	CH (Qm)	120	20	100	600	9.0

[a] DWR (1967)
[b] Unified Soil Classification System
[c] Soil parameters taken from DWR (1967), adjusted to be consistent with complementary explorations used in 1-D site response analysis (DCA, 2021c)
[d] Shear wave velocity taken from nearby seismic CPT measurements
Notes:
pcf = pound(s) per cubic foot
fps = feet per second
ksi = 1,000 pounds per square inch
Qm = Modesto formation

Table 3. Estimated DMM Cell Size, Minimum Replacement Ratio, and Depth of Liquefiable Soils at Union Island Tunnel Maintenance Shaft

Location	Union Island Tunnel Maintenance Shaft
Minimum Calculated Spacing (meters / feet)	14 / 45
Replacement Ratio (%)	14
Predicted Depth of Liquefiable Soils (feet)[a]	30

[a] CPT used from HTE (2008)

5.2 Bethany Reservoir Pumping Plant

The BRPP may be constructed within alternating layers of coarse-grained and fine-grained soils (Figure 3). Figure 4 provides the boring locations.

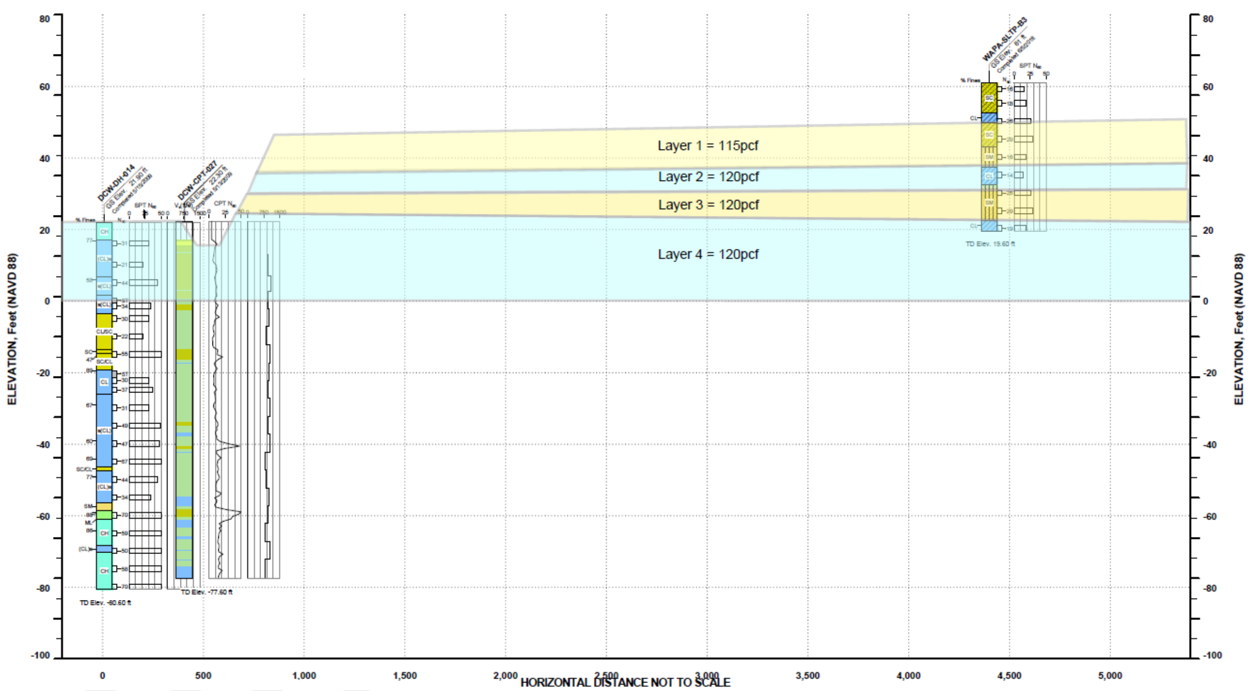

Figure 3. Conceptual Geological Profile at Bethany Reservoir Pumping Plant

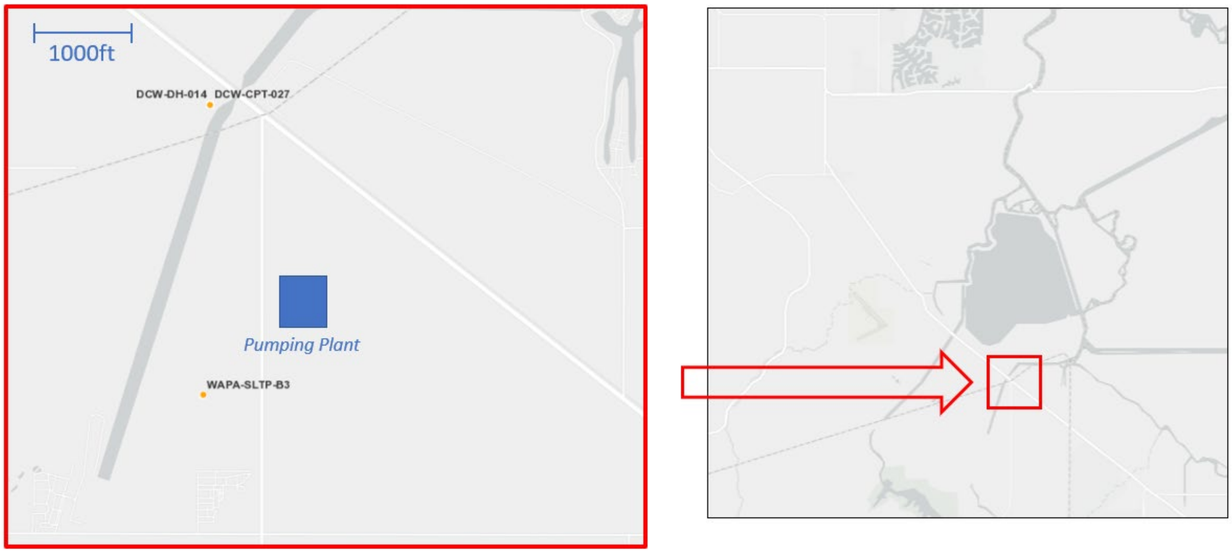

Figure 4. Plan View of Borings and CPTs Used Near Bethany Reservoir Pumping Plant
Plan view is oriented north-south.

The soil borings drilled near this site (DCW-DH-014 and WAPA-SLTP-B3) were drilled at El. 22 and El. 61 feet, respectively (SLTP, 2018). The SLTP exploration was located at an elevation approximately 20 feet higher than the proposed BRPP, so the top 20 feet of soil was disregarded. This SLTP exploration encountered clayey sand to a depth of 7 feet, followed by a 5-foot-thick silty sand layer before reaching an additional 5-foot layer of lean clay and overlying a 10 foot layer of silty sand, ending in a thin layer of sandy lean clay. For the liquefaction analyses, the earthquake groundwater depth was set to 30 feet bgs to reflect the groundwater measurement taken in the field during drilling of a nearby soil boring (DWR, 2013).

Table 4 summarizes the averaged soil parameters for each layer used in the SPT-based liquefaction analysis at the BRPP site. Attachment 3 provides the results of the SPT liquefaction analysis.

Liquefaction was predicted in an isolated zone at a depth of approximately 35 feet in the clayey sand layer using the SPT data. Because of this, the results in Attachment 3 show the need for a minimal soil reinforcement spacing. The presence of clayey fines would mitigate this isolated risk of liquefaction (refer to Table 4). For this reason, no ground improvement is anticipated at the BRPP site to address liquefaction. More refined analyses will be performed to confirm this during future design phases.

Given the significant underprediction of N_{60} in the paired CPT (DCW-CPT-027) (discussed in Section 3.2 and illustrated in Attachment 5), the calculated liquefaction prediction results from the CPT are not considered representative of the conditions at the Bethany Reservoir Pumping Plant. Downhole shear wave velocity normalized for overburden pressure (Vs1) measurements from DCW-CPT-027 reported no values below 700 fps, which indicates the soils are not subject to liquefaction at this site (Andrus & Stokoe, 2000), as Attachment 6 shows.

Table 4. Summary of Soil Parameters at Bethany Reservoir Pumping Plant

Explorations Used	Layer (refer to Figure 3)	Top Elev. (feet)	USCS[c]	Total Unit Weight (pcf)[d]	Average SPT N-value	Average Fines Content (%)	Average Shear Wave Velocity (fps)[e]	Shear Modulus G (ksi)
DCW-DH-014[a], WAPA-SLTP-B3[b]	1	50	SC (Qm)	115	18	39	600	10.2
	2	37	CL (Qm)	120	10	90	550	8.0
	3	33	SM (Qm)	120	18	33	650	10.7
	4	23	CL (Qm)	120	22	75	650	11.0

[a] DWR (2013)
[b] SLTP (2013)
[c] Unified Soil Classification System
[d] Soil parameters taken from DWR (1967), adjusted to be consistent with complementary explorations used in 1-D site response analysis (DCA, 2021c)
[e] Shear wave velocity taken from nearby seismic CPT measurements
Notes:
% = percent
pcf = pound(s) per cubic foot
fps = feet per second
ksi = 1,000 pounds per square inch
Qm = Modesto Formation

5.3 Bethany Reservoir Discharge Structure

The Bethany Reservoir Discharge Structure would be constructed within soft sedimentary rock of the Upper Cretaceous Panoche formation, which consists of interbedded shales, sandstones, and occasional siltstones (DWR, 2016). The groundwater level is anticipated to be near the same elevation as the nearby Bethany Reservoir.

Available shear wave velocities measured from Boring DH-1 (DWR, 2016) were normalized for overburden pressure and reported values no lower than 3,800 fps, which indicate no potential for liquefaction at this site (Andrus & Stokoe, 2000). Given the lack of liquefiable soils at this site, no ground improvement should be required for the purposes of liquefaction mitigation at the Bethany Reservoir Discharge Structure.

6. Conclusions

This TM was created in support of developing a conceptual footprint for key elements of the Bethany Reservoir Alternative located south of Upper Jones Tract, including at Union Island and features within the Bethany Complex. Liquefaction potential was identified at the Union Island Tunnel Maintenance Shaft site for the Bethany Reservoir Alternative, based on preliminary design ground motions that are consistent with the current draft project seismic design criteria (DCA, 2021b).

For this conceptual design TM, DCA recommends incorporating a viable ground improvement scheme at the maintenance shaft location to reduce the risk of liquefaction, strength loss, settlement, and lateral spreading. Installation of a grid of DMM panels is considered a viable option. At Union Island Tunnel Maintenance Shaft, the minimum wall spacing would be set at 45 feet for conceptual engineering and the walls would extend to a depth of 30 feet bgs, or deeper.

Since the soil underlying the BRPP is dominated by clays, DCA concludes no ground improvement to address liquefaction is anticipated at this site. Similarly, given the lack of liquefiable soils near Bethany Reservoir, no ground improvement is anticipated for liquefaction mitigation at the Bethany Reservoir Discharge Structure.

Given the significant differences in N_{60} blow counts predicted from the CPT sounding and those measured in the adjacent soil boring at the Jones Approach Channel, a comparison of all CPT/soil boring pairs in the former WaterFix dataset is encouraged.

7. References

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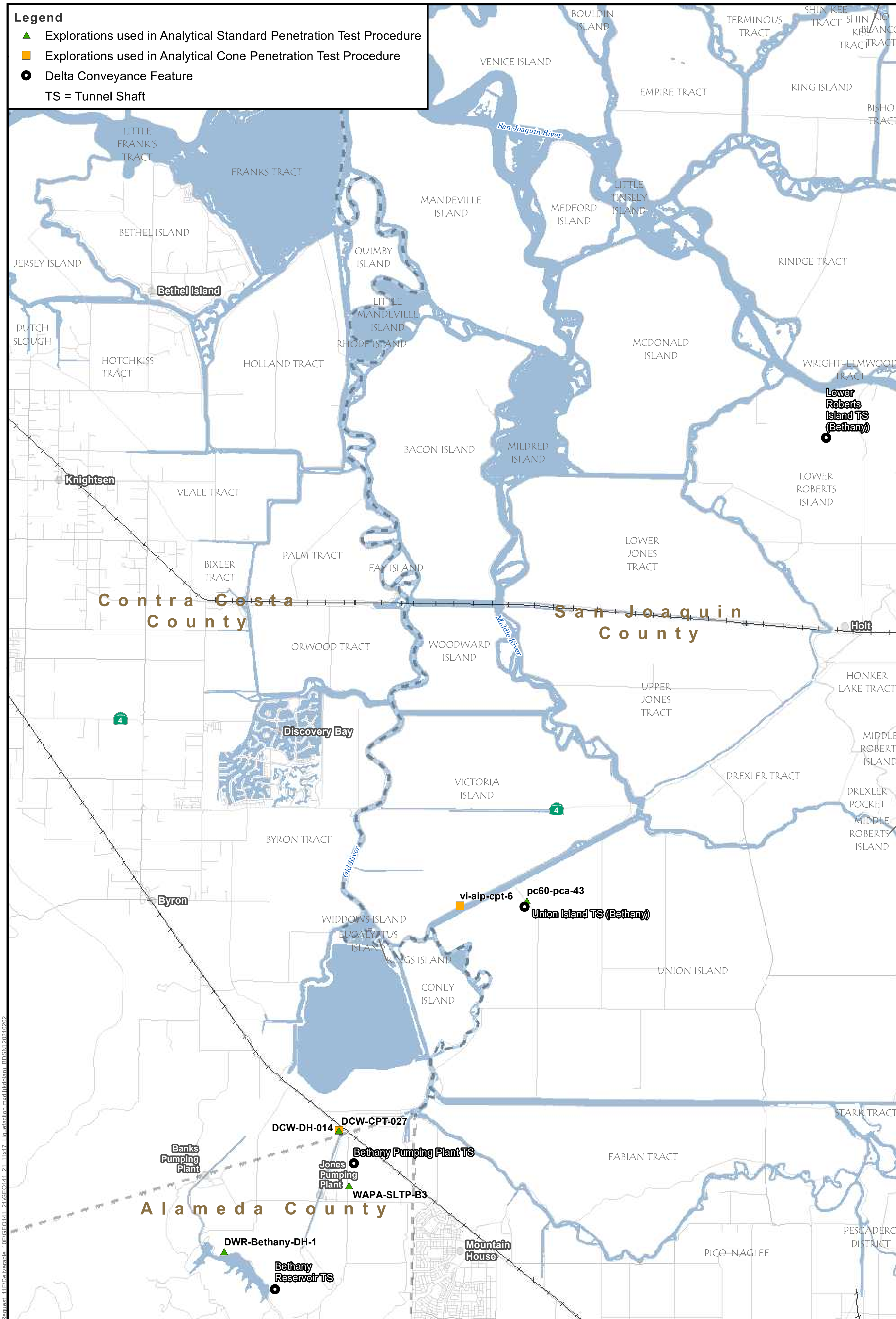
8. Document History and Quality Assurance

Reviewers listed have completed an internal quality review check and approval process for deliverable documents that is consistent with procedures and directives identified by the Engineering Design Manager (EDM) and the DCA.

Approval Names and Roles			
Prepared by	**Internal Quality Control review by**	**Consistency review by**	**Approved for submission by**
John Hinton / EDM Geotechnical Engineer	Dario Rosidi /EDM Seismic Lead Andrew Finney / EDM Geotechnical and Field Work Lead	Gwen Buchholz / DCA Environmental Consultant Phil Ryan / EDM Design Manager	Terry Krause / EDM Project Manager

This interim document is considered preliminary and was prepared under the responsible charge of Andrew Finney, California Geotechnical Engineering License GE2759.

Attachment 1
Map of Analysis Locations



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Attachment 1
Explorations Used in Liquefaction Analysis

Data Source: DCA, DWR

Attachment 2
Probabilistic and Deterministic Ground Motions

LCI
EARTH SCIENCE CONSULTANTS

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(925) 482-1960, fax (925) 482-0961

MEMORANDUM OF TRANSMITTAL

Date: 1 September 2021

To: Andrew Finney and Dario Rosidi

From: Patricia Thomas, Sarah Smith, and Ivan Wong

SUBJECT: Data Transmittal – Delta Conveyance Probabilistic and Deterministic Ground Motions for Bethany Alternative Sites

Lettis Consultants International, Inc. (LCI) is pleased to provide these probabilistic and deterministic peak horizontal ground accelerations (PGAs) for the three sites along the Bethany Alternative of the Delta Conveyance Project (Figure 1). These values supplement the values provided in reports by Wong *et al*. (2021) for 12 sites along the two original alignments. All sites are shown on Figure 1. Consistent with those previous analyses, the ground motions computed herein are for a generic stiff soil site condition with a time-averaged shear-wave velocity in the top 30 m (V_S30) of 1,100 ft/sec (335 m/sec).

The seismic source model used in the May 2019 analyses for WaterFix (LCI, 2019) has since been updated. Specifically the characterizations of the West Tracy, Midland, and Greenville faults were revised based on new information (Figure 1). The updates to the seismic source model are described in Wong *et al*. (2021). Table 1 provides mean and 85th percentile PGAs at 500, 1,000, and 2,475-year return periods.

Table 1. Probabilistic PGAs for California Delta Conveyance[1]

Location[1]	Latitude	Longitude	500-Year PGA		1,000-Year PGA		2,475-Year PGA	
			Mean (g)	85th % (g)	Mean (g)	85th % (g)	Mean (g)	85th % (g)
Bethany Reservoir Shaft	37.779498°	-121.605939°	0.46	0.53	0.59	0.67	0.78	0.89
Pumping Plant	37.801215°	-121.575039°	0.41	0.47	0.53	0.60	0.70	0.79
Union Island Shaft	37.866588°	-121.523912°	0.33	0.37	0.41	0.46	0.54	0.61

[1] Stiff Soil, Site Class D was assumed for each location.

Notes:

% = percentile
PGA = peak horizontal acceleration

The results of the PSHA show that the highest probabilistic hazard is at the Bethany Reservoir shaft followed by the Pumping Plant. The lowest hazard is at the Union Island shaft. The probabilistic PGA hazard at the three sites is controlled by the active faults to the west including the Greenville and Mt. Diablo faults (Figure 1). Unlike the DSHA, the West Tracy fault is not a major contributor to the probabilistic hazard because of its low slip rate.

A DSHA was also performed for the three sites. All three sites are within 6 km of the West Tracy fault, and so, deterministic PGAs are computed for the **M** 6.9 scenario on the West Tracy fault. Deterministic scenarios on other faults in the region result in lower PGAs.

Table 2. Deterministic PGA Values[1]

LOCATION	DETERMINISTIC MEDIAN PGA (g)	DETERMINISTIC 84TH PERCENTILE PGA (g)	DETERMINISTIC 69TH PERCENTILE PGA (g)	DETERMINISTIC 95TH PERCENTILE PGA (g)
Bethany Reservoir Shaft	0.54	0.93	0.71	1.31
Pumping Plant	0.60	1.02	0.78	1.44
Union Island Shaft	0.42	0.72	0.55	1.03

[1] Controlling deterministic scenario for all three sites is **M** 6.9 earthquake on the West Tracy fault.

The Bethany Reservoir Shaft and Pumping Plant sites are located on the hanging wall of the West Tracy fault resulting in larger ground motions than at the Union Island Shaft site. The Pumping Plant hazard is highest because it is closest to the West Tracy fault (Figure 1).

The probabilistic and deterministic ground motions represent free-field motions for a reference site condition of stiff soil (V_S30 = 1,100 ft/sec). These preliminary ground motions should be revised at a later date using site response analysis to model the effects of the softer, near surface materials.

REFERENCES

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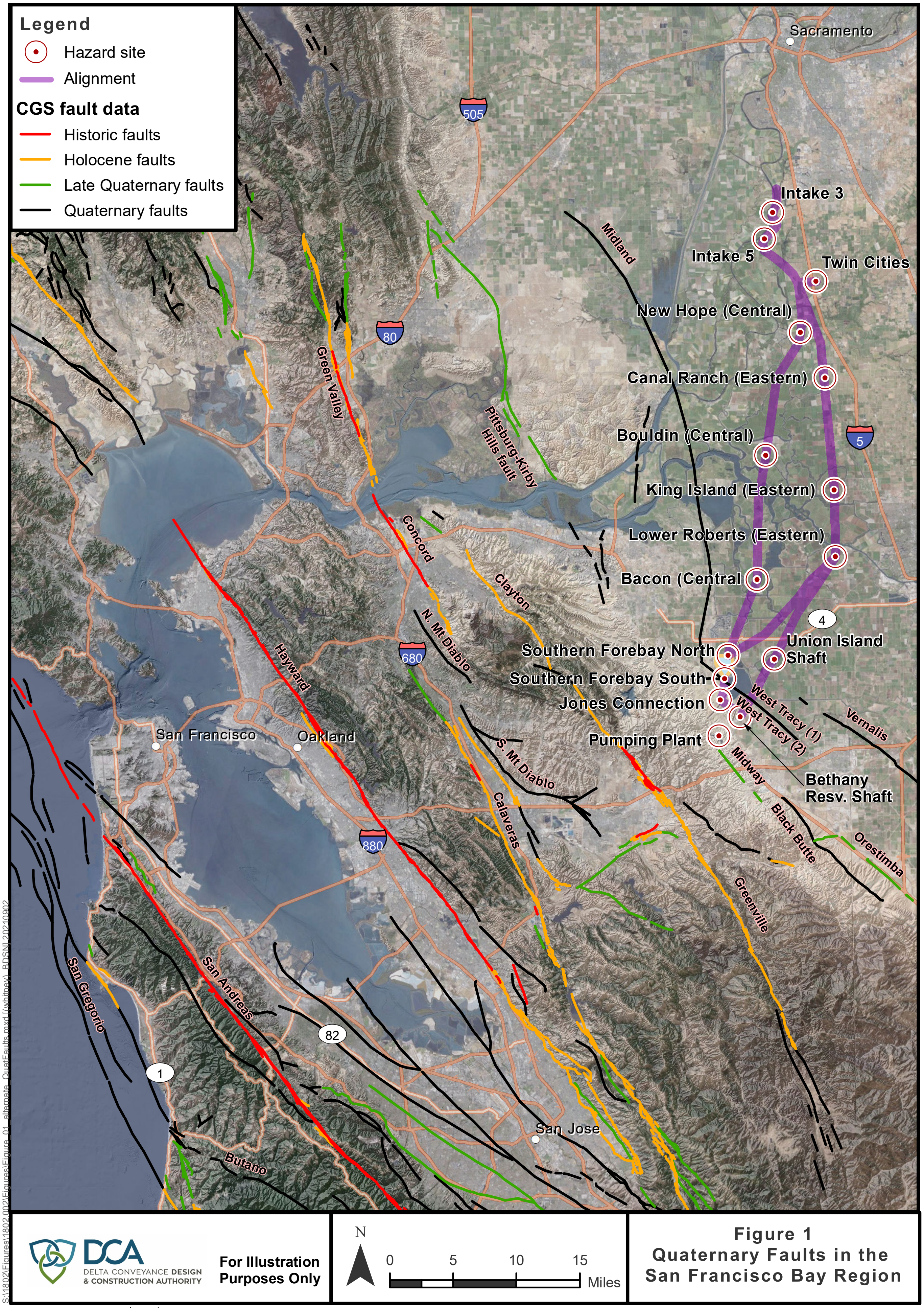
Wong, I., Thomas, P., Zandieh, A., Lewandowski, N., Smith, S., and Unruh, J., 2021, Seismic hazard analyses and development of conceptual seismic design ground motions for the Delta Conveyance, unpublished final report (Rev 2 dated 1 Sep 2021) prepared by Lettis Consultants International for the Delta Conveyance Design and Construction Office.



DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY

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Figure 1
Quaternary Faults in the San Francisco Bay Region

Data Source: DCA, CGS (2005)

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EARTH SCIENCE CONSULTANTS

MEMORANDUM OF TRANSMITTAL

Date: 1 September 2021

To: Andrew Finney and Dario Rosidi

From: Patricia Thomas and Ivan Wong

SUBJECT: **Data Transmittal – Delta Conveyance Probabilistic and Deterministic Ground Motions for Union Island Shaft**

As requested, the following are probabilistic and deterministic peak horizontal ground acceleration (PGA) values and spectral acceleration (SA) values for spectral periods from 0.01 to 10.0 sec, as well as Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) design response spectra for the Union Island Shaft site along the Bethany Alternative alignment of the Delta Conveyance Project (Figure 1). These values supplement the values provided in reports by Wong *et al*. (2021) for 12 sites along the two original alignments and Thomas *et a*l. (2021) for three sites along the Bethany Alternative. All sites are shown on Figure 1. Consistent with those previous analyses, the ground motions computed herein are for a generic stiff soil site condition with a time-averaged shear-wave velocity in the top 30 m (V_S30) of 1,100 ft/sec (335 m/sec).

The seismic source model used in the 2019 analyses for WaterFix (LCI, 2019) was subsequently updated. Specifically the characterizations of the West Tracy, Midland, and Greenville faults were revised based on new information (Figure 1). The updates to the seismic source model are described in the Wong *et a*l. (2021). The results for the three Bethany Alternative presented in Thomas *et al*. (2021) are based on the updated source model, as are the expanded results presented herein for Union Island Shaft site. These results for the Union Island Shaft site supersede those in Thomas *et al*. (2021).

Probabilistic Ground Motion Results

The probabilistic seismic hazard analysis (PSHA) methodology, including documentation of the seismic source model and ground motion models, used to develop the Union Island Shaft ground motions are provided in Wong *et al*. (2021). The results of the PSHA for the Union Island Shaft are presented in terms of ground motion as a function of annual exceedance frequency (AEF). AEF is the reciprocal of the average return period. Figure 2 shows the mean, median (50th percentile), 5th, 15th, 85th, and 95th percentile PGA hazard curves. The range of uncertainty between the the 5th and 95th percentile (fractiles) is a factor of 1.6 at a return period of 2,475 years. These fractiles indicate the range of epistemic uncertainty about the mean hazard. The 1.0 sec horizontal SA hazard curves are shown on Figure 3, which also have a factor of 1.6 at a return period of 2,475 years. Table 1 provides mean and 5th to 95th percentile PGA and 1.0 sec values at return periods of 144, 200, 475, 975, and 2,475 years.

Table 1. Summary of PGA and 1.0 Sec Horizontal Spectral Accelerations[1]

	PGA (g)	1.0 SEC SA (g)
144-Year Return Period		
Mean	0.20	0.25
5th-95th Percentiles	0.16 - 0.25	0.19 - 0.31
200-Year Return Period		
Mean	0.23	0.29
5th-95th Percentiles	0.18 - 0.29	0.22 - 0.36
475-Year Return Period		
Mean	0.32	0.41
5th-95th Percentiles	0.25 - 0.4	0.32 - 0.51
975-Year Return Period		
Mean	0.41	0.54
5th-95th Percentiles	0.32 - 0.5	0.41 - 0.66
2,475-Year Return Period		
Mean	0.54	0.74
5th-95th Percentiles	0.42 - 0.66	0.56 - 0.92

[1] Stiff Soil, Site Class D was assumed for Union Island Shaft site.

Notes: % = percentile
PGA = peak horizontal acceleration

The contributions of the various seismic sources to the mean PGA hazard are shown on Figures 4 and 5 as hazard curves and fractional contribution plots, respectively. Seismic sources that contribute at least 5 percent to the hazard over the period range of 144 to 2,475 years are identified on these figures. Figures 4 and 5 show that the PGA hazard is controlled by the Mt. Diablo fault for return periods between 100 and 10,000 years. Although the site is located 25 km from the Mt. Diablo fault, it has a preferred slip rate of 2.0 mm/year, while the closer faults such as Greenville and West Tracy have significantly lower slip rates. The 1.0 sec SA hazard results are similar with some increased relative contribution from the Greenville and Midway-Black Butte faults (Figures 6 and 7).

The hazard can also be deaggregated in terms of the joint magnitude-distance-epsilon probability conditional on the ground motion parameter (PGA or SA exceeding a specific values). Epsilon is the difference between the logarithm of the ground motion amplitude and the mean logarithm of ground motion (for that M and D) measured in units of standard devition (σ). Thus, positive epsilons indicated larger-than-average ground motions. By deaggregating the PGA and 1.0 sec SA hazard by magnitude, distance, and epsilon bins, we can illustrate the contribution by events at various return periods. Figure 8 shows the deaggregation of the PGA hazard for the return periods of 475 and 2,475 years. The contributions to the PGA hazard are coming from a wide range of M and D reflecting the contribution from several seismic sources (Figures 4 and 5). The majority of the PGA hazard at both the 475 and 2,475 year return periods is coming from events with magnitudes **M** 6.4 to 7.4 at distances less than 60 km. Deaggregation of the 1.0 sec SA hazard shows contribution from events of the same magnitude and distance ranges, but with additional contribution from events of magnitude **M** 7.2 to 8.4 between 80 and 90 km on the San Andreas fault (Figure 9).

Based on the magnitude and distance deaggregated results, the controlling earthquakes as defined by the mean magnitude (M-bar) and modal magnitude (M*), and mean distance (D-bar) and modal distance (D*) can be calculated. Table 2 lists the M-bar, M*, D-bar, and D* for the five return periods (144, 200, 475, 975, and 2,475 years) and for PGA and 1.0 sec horizontal SA.

Table 2. Magnitude and Distance Deaggregation

Period (sec)	PGA	1.0 Sec SA
144-Year Return Period		
Modal M	6.7	6.7
Modal R_{RUP} (km)	25	25
Mean M	6.6	6.8
Mean R_{RUP} (km)	35.6	48.0
200-Year Return Period		
Modal M	6.7	6.7
Modal R_{RUP} (km)	25	25
Mean M	6.6	6.8
Mean R_{RUP} (km)	33.1	45.4
475-Year Return Period		
Modal M	6.7	6.7
Modal R_{RUP} (km)	25	25
Mean M	6.6	6.8
Mean R_{RUP} (km)	27.9	39.2
975-Year Return Period		
Modal M	6.7	6.7
Modal R_{RUP} (km)	25	25
Mean M	6.6	6.8
Mean R_{RUP} (km)	24.6	41.9
2,475-Year Return Period		
Modal M	6.7	6.7
Modal R_{RUP} (km)	25	25
Mean M	6.6	6.8
Mean R_{RUP} (km)	21.4	30.2

Figure 10 shows a suite of mean uniform hazard spectra (UHS) at the return periods of 144, 200, 475, 975, and 2475 years. A UHS depicts the ground motions at all spectral periods with the same annual exceedance frequency or return period. The mean UHS shown on Figure 10 are tabulated in Table 3.

Table 3. Mean Uniform Hazard Spectra

PERIOD (SEC)	144-YEAR RETURN PERIOD, SA (g)	200-YEAR RETURN PERIOD, SA (g)	475-YEAR RETURN PERIOD, SA (g)	975-YEAR RETURN PERIOD, SA (g)	2,475-YEAR RETURN PERIOD, SA (g)
0.01	0.20	0.23	0.32	0.41	0.54
0.03	0.22	0.25	0.34	0.43	0.57
0.05	0.26	0.29	0.41	0.51	0.68
0.075	0.33	0.38	0.52	0.66	0.88
0.10	0.40	0.45	0.63	0.80	1.05
0.15	0.49	0.55	0.76	0.96	1.27
0.20	0.52	0.59	0.82	1.04	1.37
0.25	0.53	0.60	0.83	1.06	1.41
0.30	0.52	0.60	0.83	1.06	1.41
0.40	0.48	0.55	0.76	0.99	1.33
0.50	0.43	0.50	0.70	0.91	1.23
0.60	0.38	0.44	0.62	0.81	1.10
0.75	0.33	0.38	0.53	0.69	0.95
1.0	0.25	0.29	0.41	0.54	0.74
1.5	0.16	0.19	0.27	0.35	0.48
2.0	0.12	0.13	0.19	0.25	0.35
3.0	0.061	0.076	0.12	0.15	0.21
4.0	0.038	0.045	0.073	0.11	0.14
5.0	0.027	0.032	0.049	0.070	0.11
7.5	0.016	0.019	0.027	0.037	0.055
10.0	0.011	0.013	0.018	0.024	0.035

Deterministic Ground Motion Results

A deterministic seismic hazard analysis (DSHA) was also performed for the Union Island Shaft site. The site is on the footwall and within 6 km of the West Tracy fault, and so, deterministic ground motions are computed for the characteristic **M** 6.9 scenario on the West Tracy fault (Figure 11). Deterministic ground motions were also computed for the larger, but more distant, **M** 8.0 scenario for the San Andreas fault (Figure 11). Deterministic scenarios for the San Andreas fault scenario and on other faults in the region result in lower ground motions. Inputs for the DSHA are provided in Table 4 and the resulting deterministic ground motions are provided in Table 5. Median, 69^{th}, 84^{th}, and 95^{th} PGA values were computed to illustrate the range of uncertainty in the computed ground motions due to the aleatory sigma of the ground motion models. Figure 12 compares the enveloped (West Tracy scenario) deterministic ground motions to the suite of UHS for return periods of 144 to 2,475-year return periods. The median deterministic ground motions are similar to the 975-year UHS, while the 84^{th} and 95^{th} percentile deterministic ground motions exceed the 2,475-year UHS (Figure 12).

Table 4. DSHA Inputs

INPUT PARAMETER	INPUT PARAMETER DEFINITION	WEST TRACY	SAN ANDREAS
M	Moment magnitude	6.9	8.0
R_{RUP}	Closest distance to coseismic rupture (km)	5.9	81.9
R_{JB}	Closest distance to surface projection of coseismic rupture (km)	5.9	81.9
R_X	Horizontal distance from top of rupture measured perpendicular to fault strike (km)	-5.9	81.9
R_{y0}	The horizontal distance off the end of the rupture measured parallel to strike (km)	0	0
U	Unspecified-mechanism factor: 1 for unspecified; 0 otherwise	0	0
F_{RV}	Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	1	0
F_N	Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal	0	0
F_{HW}	Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise	0	0
Z_{TOR}	Depth to top of coseismic rupture (km)	0	0
Dip	Average dip of rupture plane (degrees)	70	90
V_{S30}	The average shear-wave velocity (m/s) over a subsurface depth of 30 m	335	335
$F_{Measured}$	0 = inferred, 1 = measured	1	1
Z_{HYP}	Hypocentral depth from the earthquake	Default	Default
$Z_{1.0}$	Depth to Vs=1 km/sec	0.7	0.7
$Z_{2.5}$	Depth to Vs=2.5 km/sec	4.0	4.0
W	Fault rupture width (km)	20.3	13
Region	Specific Regions considered in the models	California	California

Table 5. DSHA Results

Period (sec)	West Tracy Median (g)	West Tracy 69th Perc. (g)	West Tracy 84th Perc.(g)	West Tracy 95th Perc. (g)	San Andreas Median (g)	San Andreas 69th Perc. (g)	San Andreas 84th Perc.(g)	San Andreas 95th Perc. (g)	Envelope Median (g)	Envelope 69th Perc. (g)	Envelope 84th Perc.(g)	Envelope 95th Perc. (g)
0.01	0.42	0.55	0.72	1.03	0.11	0.15	0.19	0.28	0.42	0.55	0.72	1.03
0.02	0.42	0.56	0.73	1.04	0.11	0.15	0.19	0.28	0.42	0.56	0.73	1.04
0.03	0.43	0.57	0.75	1.07	0.11	0.15	0.20	0.29	0.43	0.57	0.75	1.07
0.05	0.48	0.64	0.84	1.21	0.12	0.17	0.22	0.33	0.48	0.64	0.84	1.21
0.075	0.58	0.77	1.02	1.48	0.14	0.20	0.27	0.40	0.58	0.77	1.02	1.48
0.10	0.68	0.91	1.20	1.74	0.17	0.23	0.31	0.47	0.68	0.91	1.20	1.74
0.15	0.85	1.12	1.48	2.11	0.20	0.27	0.37	0.56	0.85	1.12	1.48	2.11
0.20	0.94	1.25	1.65	2.35	0.22	0.31	0.41	0.61	0.94	1.25	1.65	2.35
0.25	1.01	1.33	1.77	2.55	0.24	0.33	0.45	0.67	1.01	1.33	1.77	2.55
0.30	1.03	1.38	1.85	2.70	0.25	0.34	0.47	0.70	1.03	1.38	1.85	2.70
0.40	0.98	1.34	1.82	2.71	0.24	0.33	0.46	0.70	0.98	1.34	1.82	2.71
0.50	0.90	1.24	1.72	2.60	0.23	0.32	0.44	0.68	0.90	1.24	1.72	2.60
0.75	0.69	0.98	1.37	2.13	0.18	0.26	0.36	0.57	0.69	0.98	1.37	2.13
1.0	0.56	0.80	1.13	1.77	0.15	0.21	0.29	0.46	0.56	0.80	1.13	1.77
1.5	0.37	0.53	0.76	1.20	0.11	0.16	0.22	0.35	0.37	0.53	0.76	1.20
2.0	0.27	0.38	0.55	0.86	0.085	0.12	0.17	0.27	0.27	0.38	0.55	0.86
3.0	0.17	0.24	0.34	0.54	0.061	0.087	0.12	0.20	0.17	0.24	0.34	0.54
4.0	0.11	0.16	0.22	0.35	0.047	0.066	0.094	0.15	0.11	0.16	0.22	0.35
5.0	0.076	0.11	0.15	0.24	0.036	0.051	0.073	0.11	0.076	0.11	0.15	0.24
7.5	0.035	0.049	0.070	0.11	0.022	0.032	0.045	0.070	0.035	0.049	0.070	0.11
10.0	0.019	0.027	0.038	0.059	0.014	0.019	0.027	0.043	0.019	0.027	0.038	0.059

Design Ground Motions

MDE and OBE design response spectra were developed for the Union Island Shaft site. In accordance with the Delta Conveyance seismic design criteria (DCA, 2021), MDE for shafts is defined as the envelope of the 2,475-year UHS and 84th percentile deterministic response spectra. Figure 13 compares these spectra and shows that for this site, the MDE is controlled by the 84th percentile deterministic spectra for all spectral periods. The OBE is defined as the 475-year UHS (Figure 10). Table 6 provides the MDE and OBE for the Union Island Shaft site.

Table 6. MDE and OBE Design Ground Motions

PERIOD (SEC)	MDE, SA (g)	OBE, SA (g)
0.01	0.72	0.32
0.02	0.73	0.33
0.03	0.75	0.34
0.05	0.84	0.41
0.075	1.02	0.52
0.10	1.20	0.63
0.15	1.48	0.76
0.20	1.65	0.82
0.25	1.77	0.83
0.30	1.85	0.83
0.40	1.82	0.76
0.50	1.72	0.70
0.60	1.55	0.62
0.75	1.37	0.53
1.0	1.13	0.41
1.5	0.76	0.27
2.0	0.55	0.19
3.0	0.339	0.118
4.0	0.222	0.073
5.0	0.153	0.049
7.5	0.070	0.027
10.0	0.038	0.018

The probabilistic and deterministic ground motions represent free-field motions for a reference site condition of stiff soil (V_S30 = 1,100 ft/sec). These ground motions should be revised at a later date using site response analysis to model the effects of the softer, near-surface materials.

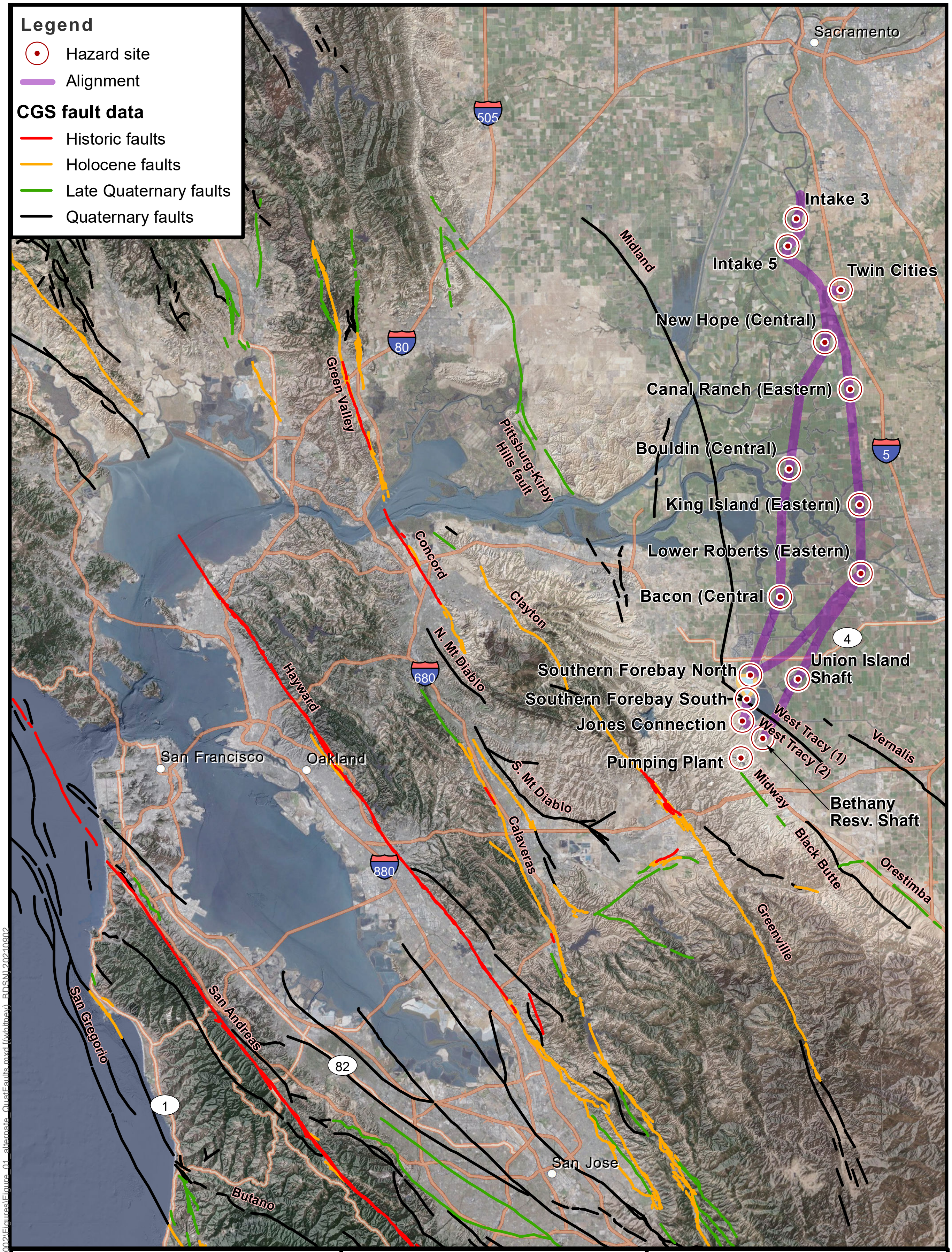
REFERENCES

DCA (Delta Conveyance Design and Construction Authority), 2021, Conceptual-Level Seismic Design Criteria, prepared for DWR/Delta Conveyance Office, 26 p.

Lettis Consultants International, 2019, Data transmittal – WaterFix probabilistic and deterministic ground motions for CER Section 4, letter of transmittal to Andrew Finney dated 1 May 2019.

Thomas, P., Smith, S., and Wong, I., 2021, Data transmittal – Delta Conveyance probabilistic and deterministic ground motions for Bethany Alternative sites, memorandum of transmittal to Andrew Finney and Dario Rosidi prepared by Lettis Consultants International dated 1 September 2021.

Wong, I., Thomas, P., Zandieh, A., Lewandowski, N., Smith, S., and Unruh, J., 2021, Seismic hazard analyses and development of conceptual seismic design ground motions for the Delta Conveyance, unpublished final report (Rev 2 dated 1 Sep 2021) prepared by Lettis Consultants International for the Delta Conveyance Design and Construction Office.



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N 0 5 10 15 Miles

Figure 1
Quaternary Faults in the San Francisco Bay Region

Data Source: DCA, CGS (2005)

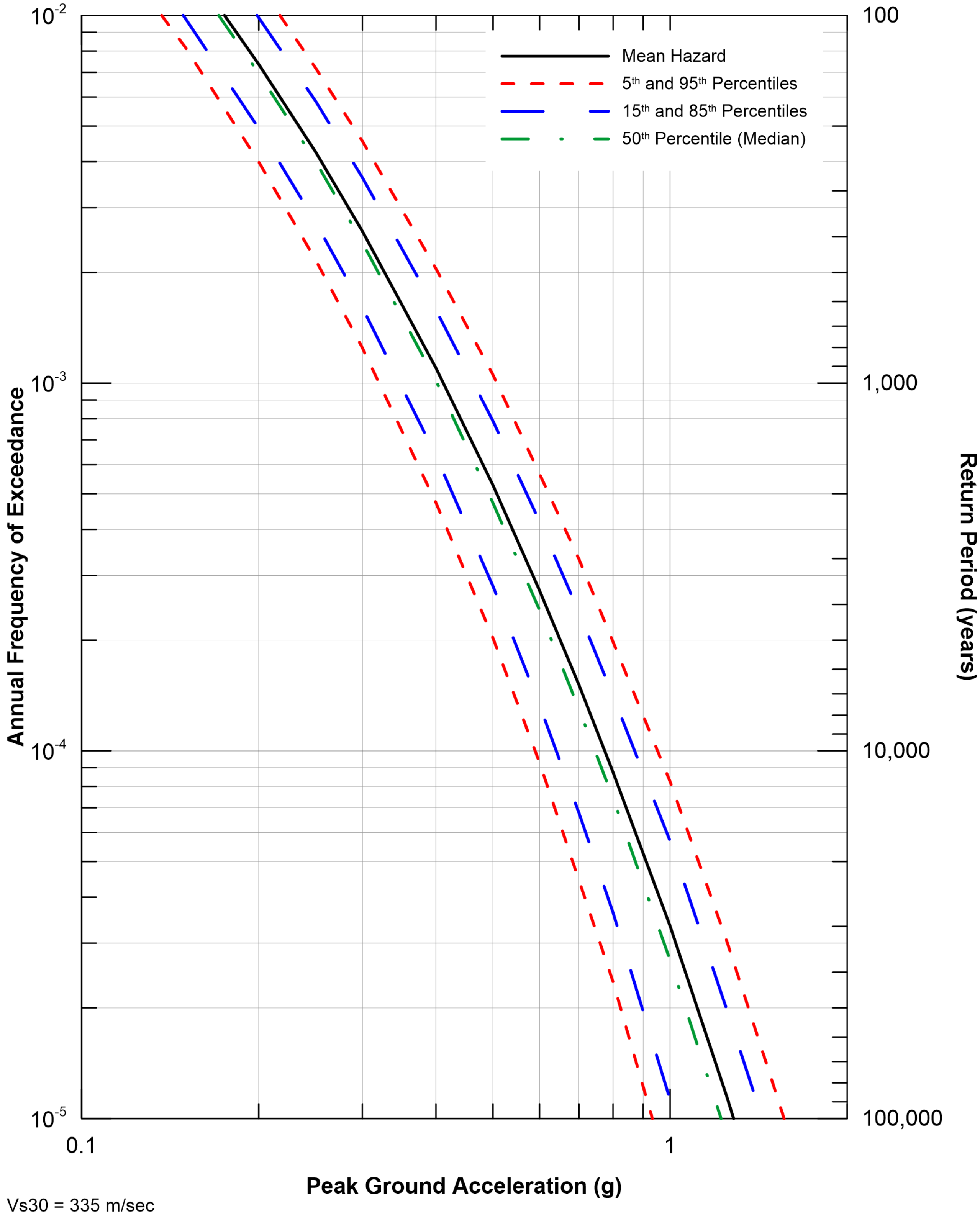

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Figure 2
Seismic Hazard Curves for Peak Horizontal Acceleration for Union Island Shaft

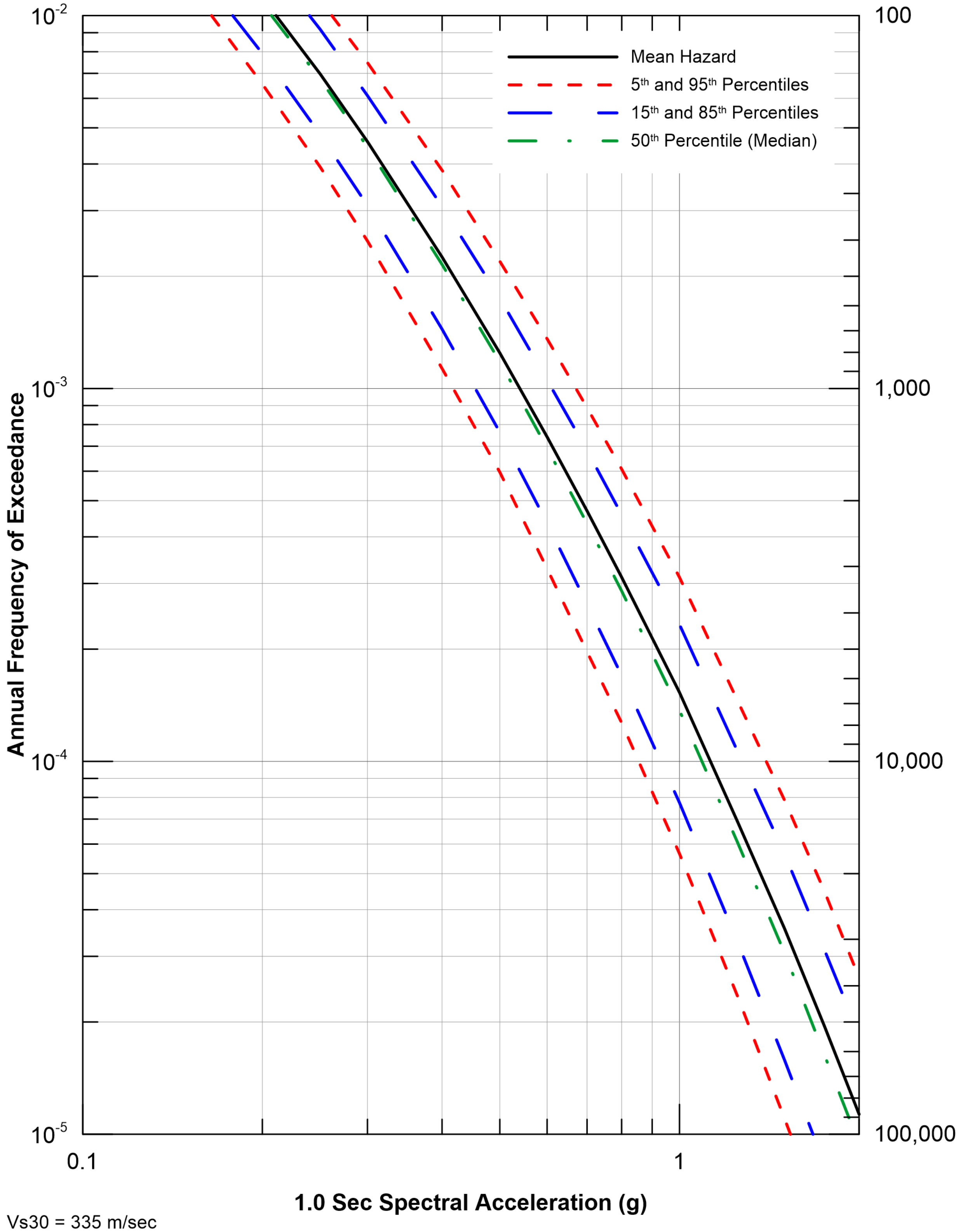

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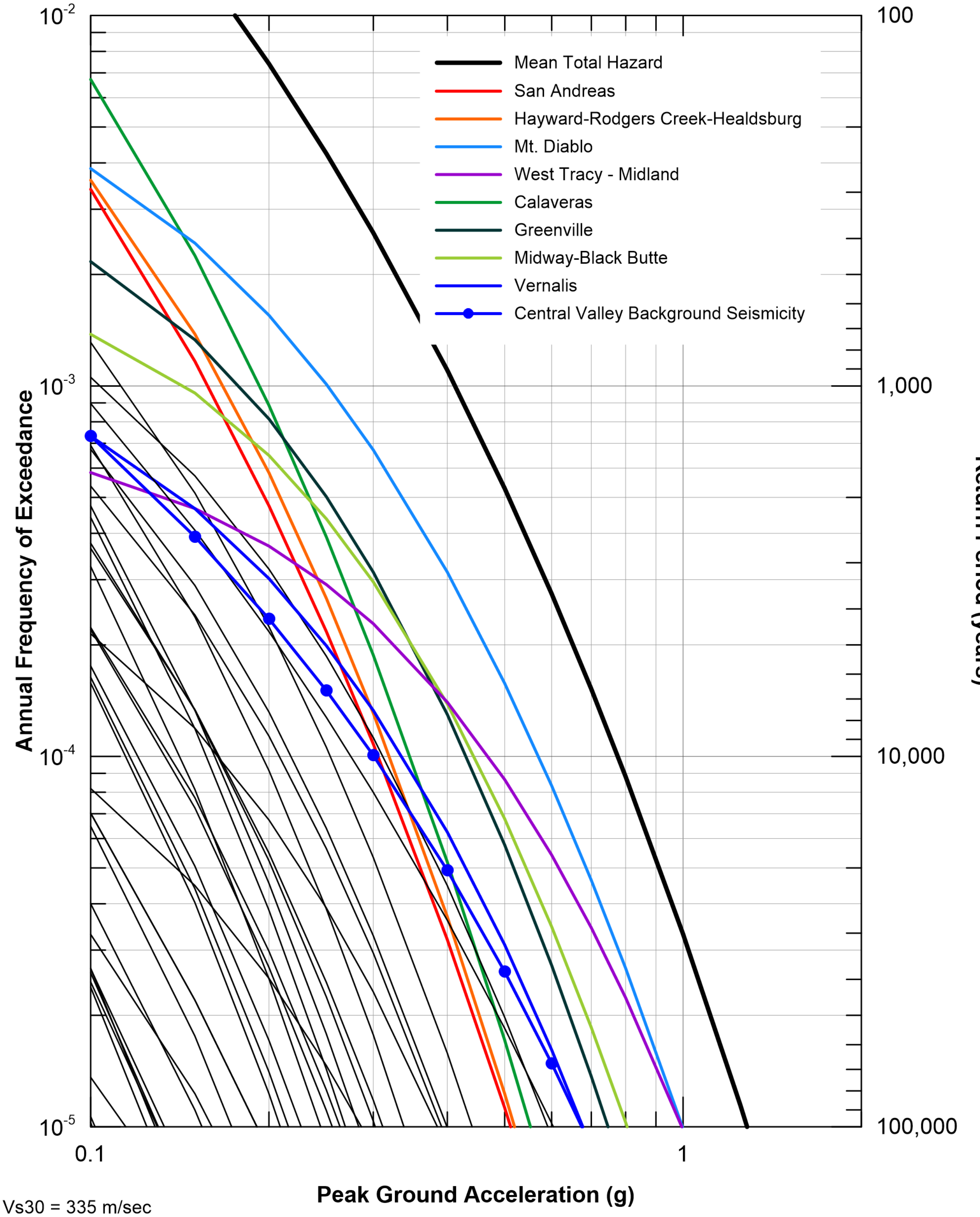
Figure 3
Seismic Hazard Curves for 1.0 Sec Horizontal Spectral Acceleration for Union Island Shaft



Vs30 = 335 m/sec
Sources contributing 5% or more in
144 to 2,475-year return period range listed.
Other less significant sources shown in black
not listed.

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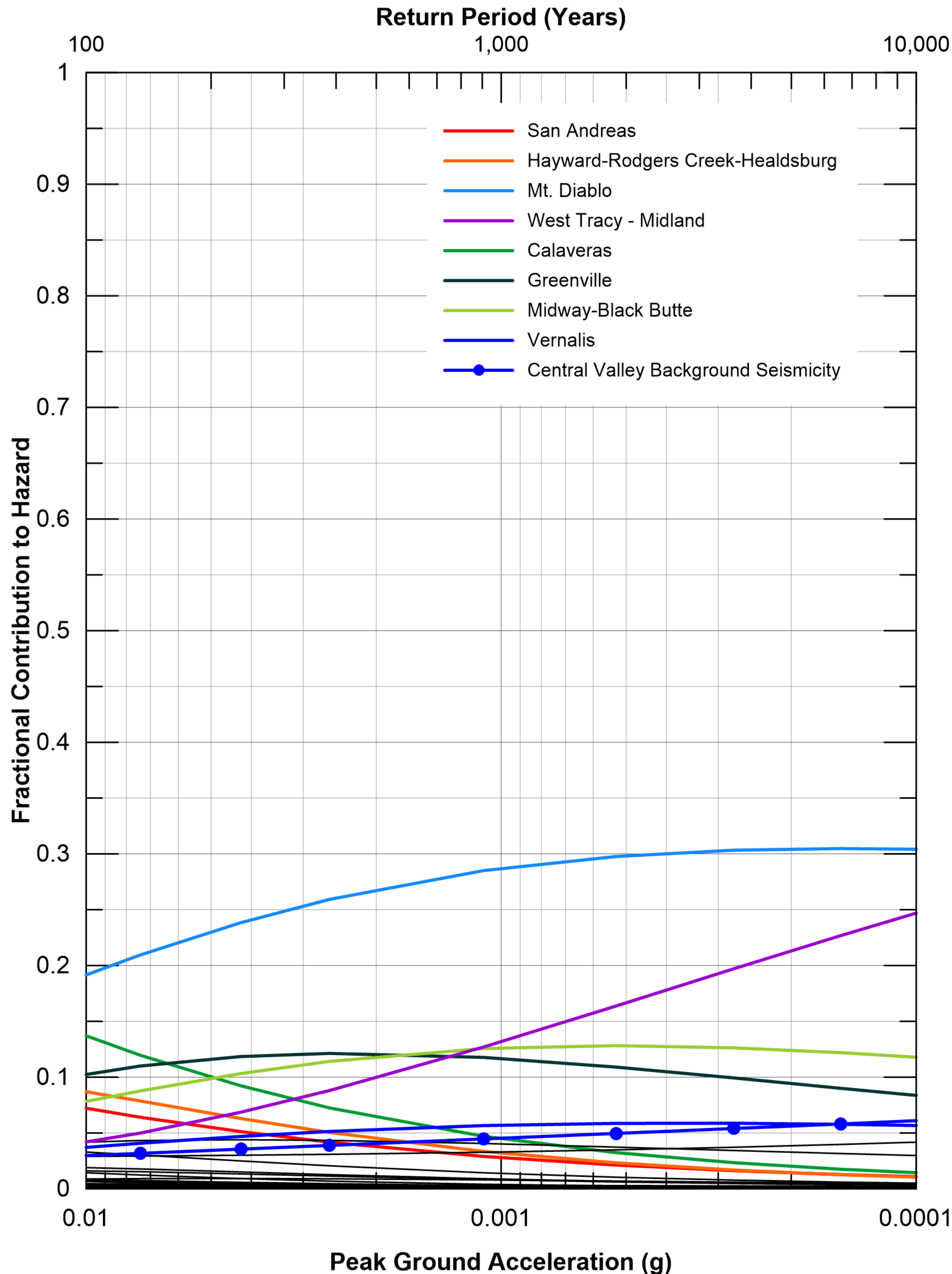
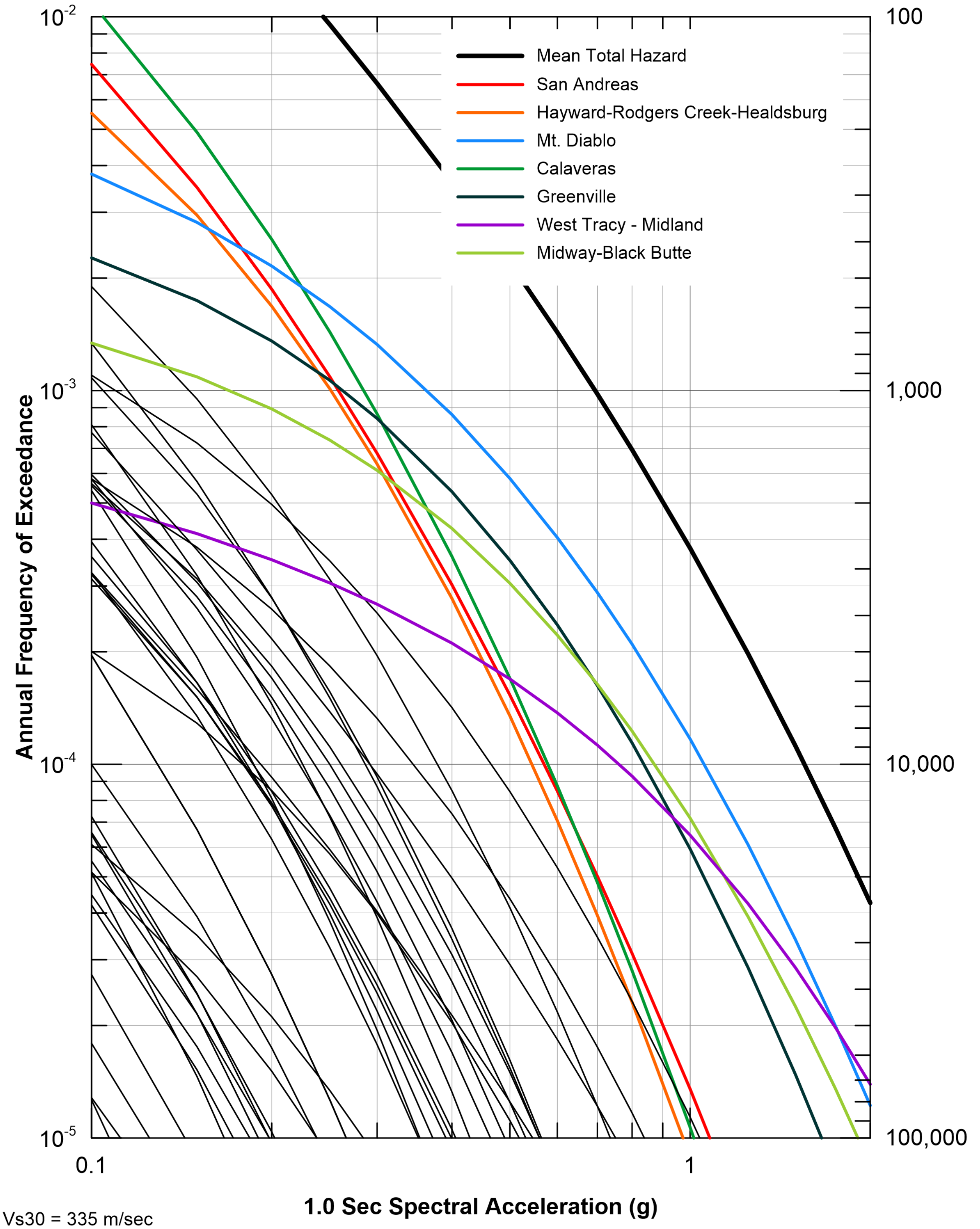
Figure 4
Seismic Source Contributions for Mean Peak Horizontal Acceleration Hazard for Union Island Shaft



Vs30 = 335 m/sec
Sources contributing 5% or more in
144 to 2,475-year return period range listed.
Other less significant sources shown in black
not listed.

DCA DELTA CONVEYANCE DESIGN & CONSTRUCTION AUTHORITY **For Illustration Purposes Only**		**Figure 5** **Seismic Source Fractional Contributions for Mean Peak Horizontal Acceleration Hazard for Union Island Shaft**



Vs30 = 335 m/sec
Sources contributing 5% or more in
144 to 2,475-year return period range listed.
Other less significant sources shown in black
not listed.

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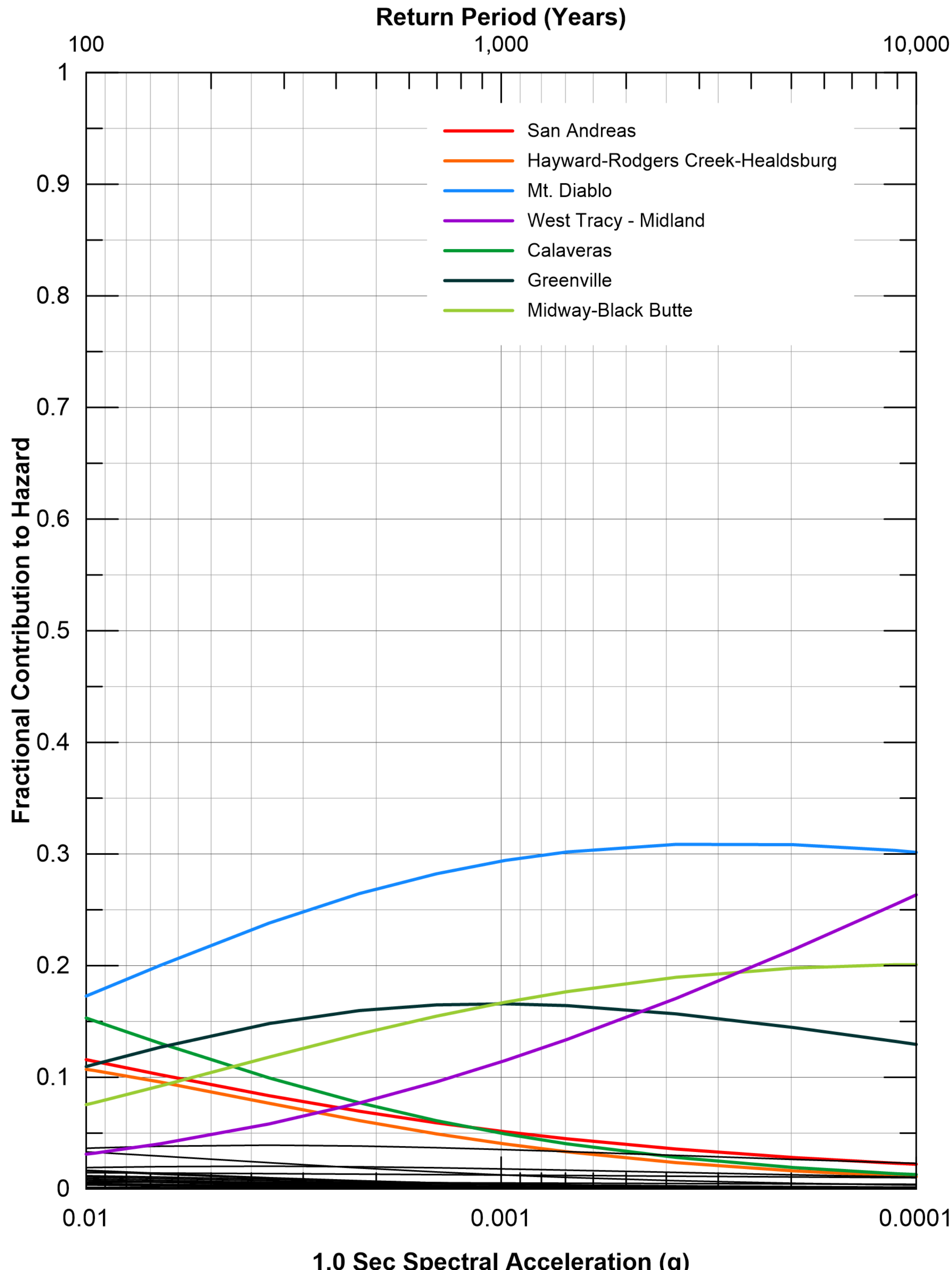
Figure 6
Seismic Source Contributions for Mean 1.0 Sec Horizontal Spectral Acceleration Hazard for Union Island Shaft



Vs30 = 335 m/sec
Sources contributing 5% or more in
144 to 2,475-year return period range listed.
Other less significant sources shown in black
not listed.

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Figure 7
Seismic Source Fractional Contributions for Mean 1.0 Sec Horizontal Spectral Acceleration Hazard for Union Island Shaft

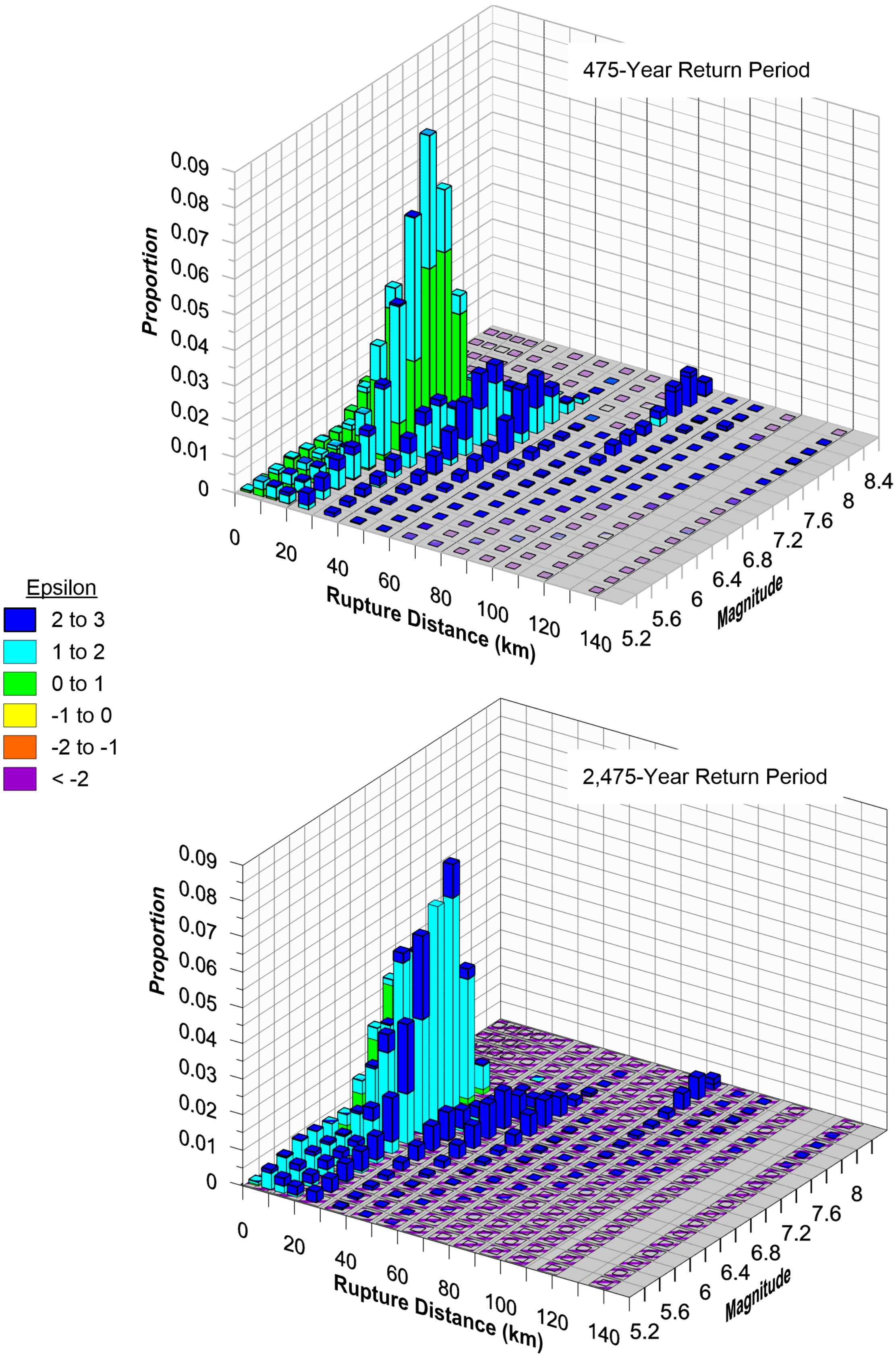

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Figure 8
Magnitude and Distance Contributions to the Mean Peak Horizontal Hazard at 475 and 2,475-Year Return Periods for Union Island Shaft

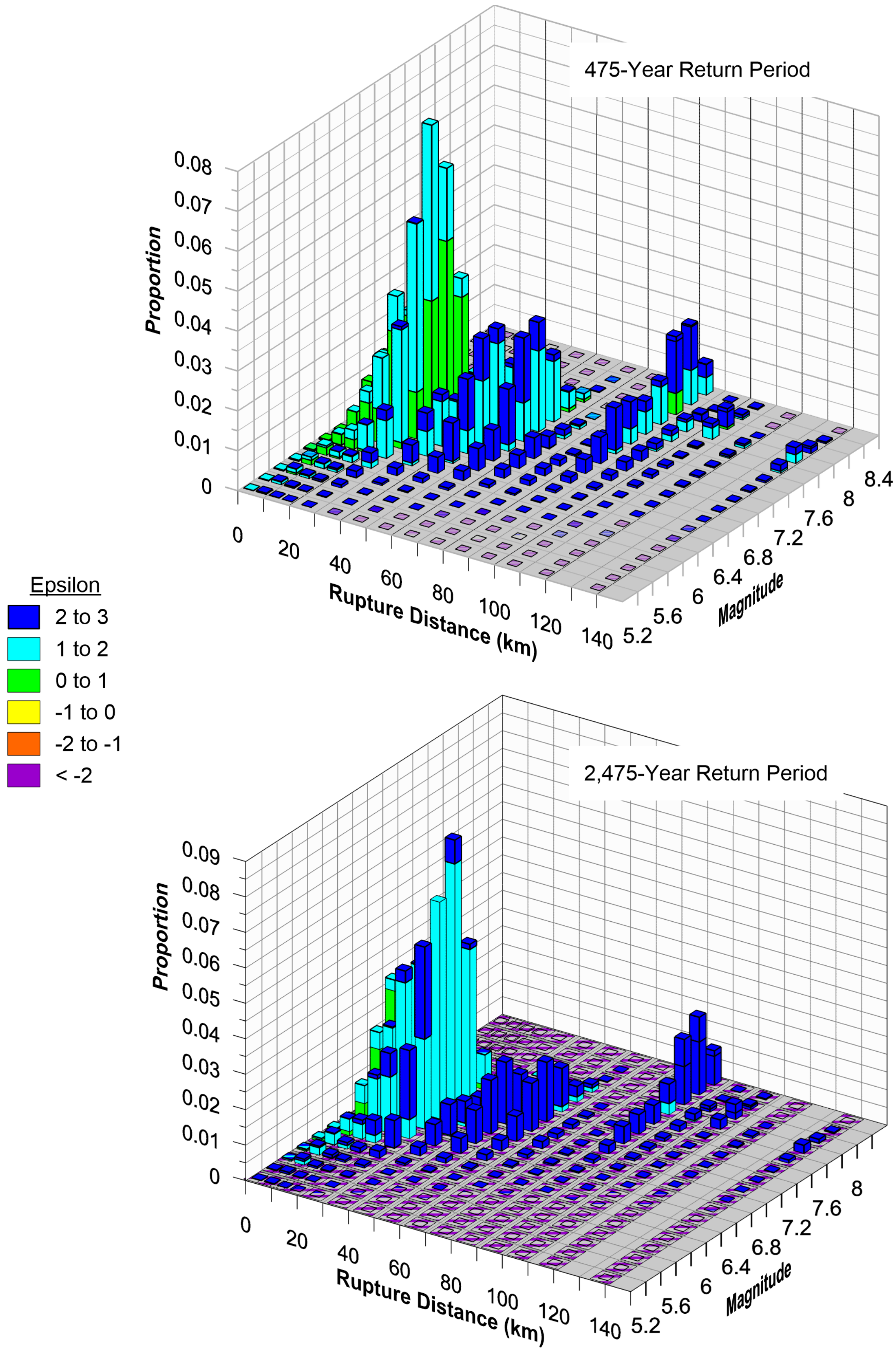

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Figure 9
Magnitude and Distance Contributions to the Mean 1.0 Sec Spectral Hazard at 475 and 2,475-Year Return Periods for Union Island Shaft

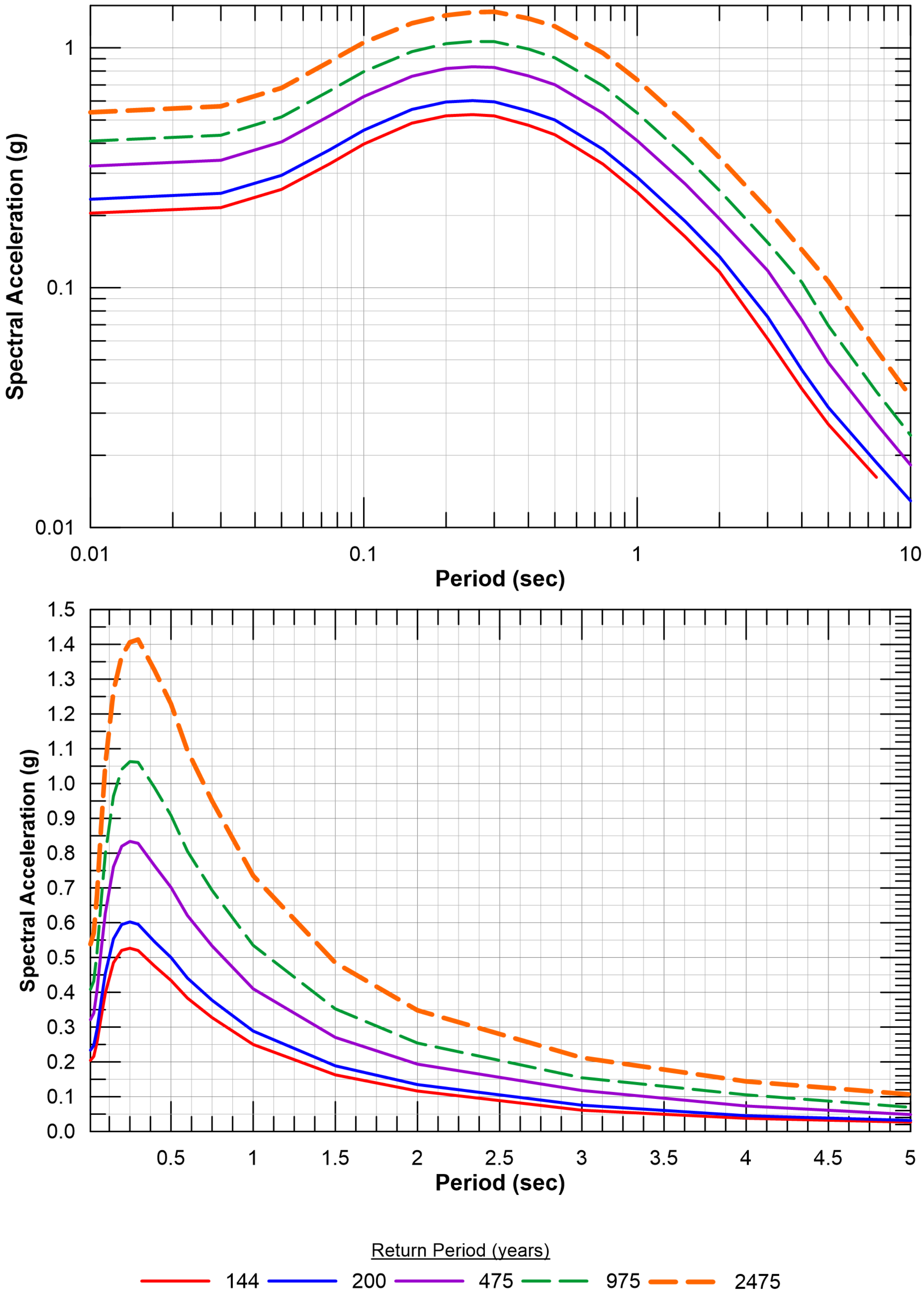

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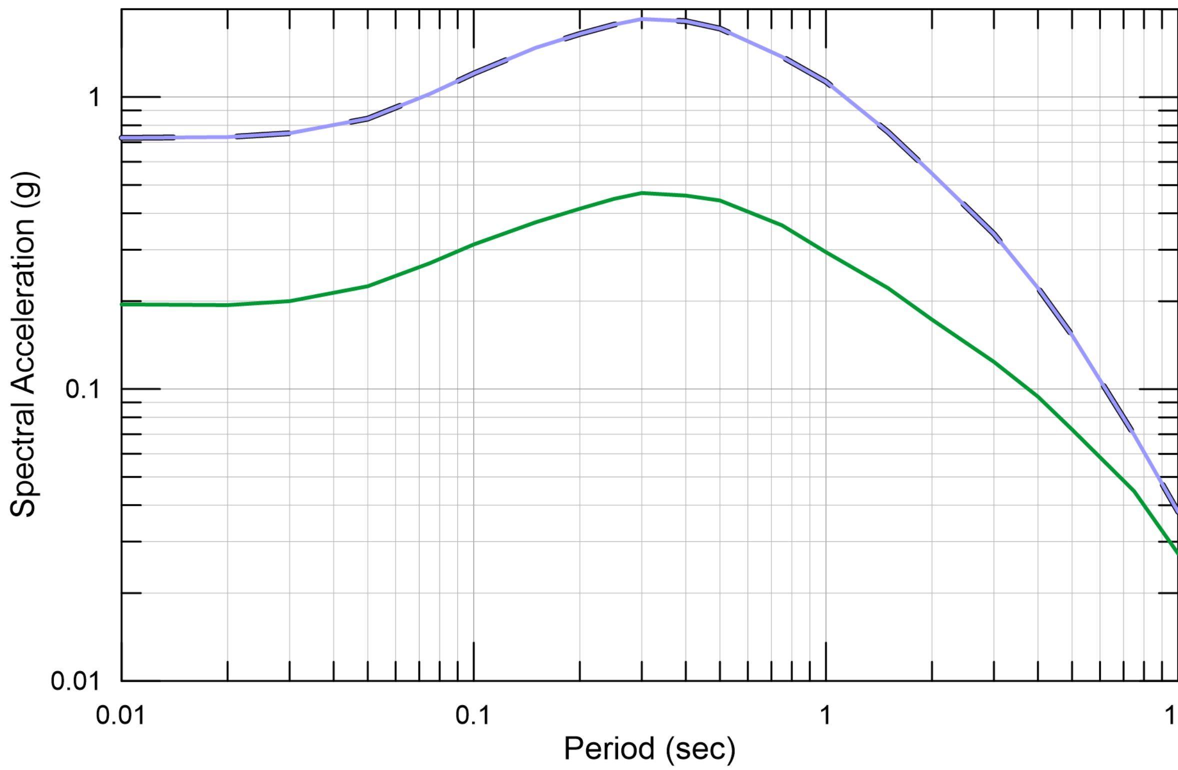
Figure 10
Mean Uniform Hazard Spectra for Union Island Shaft



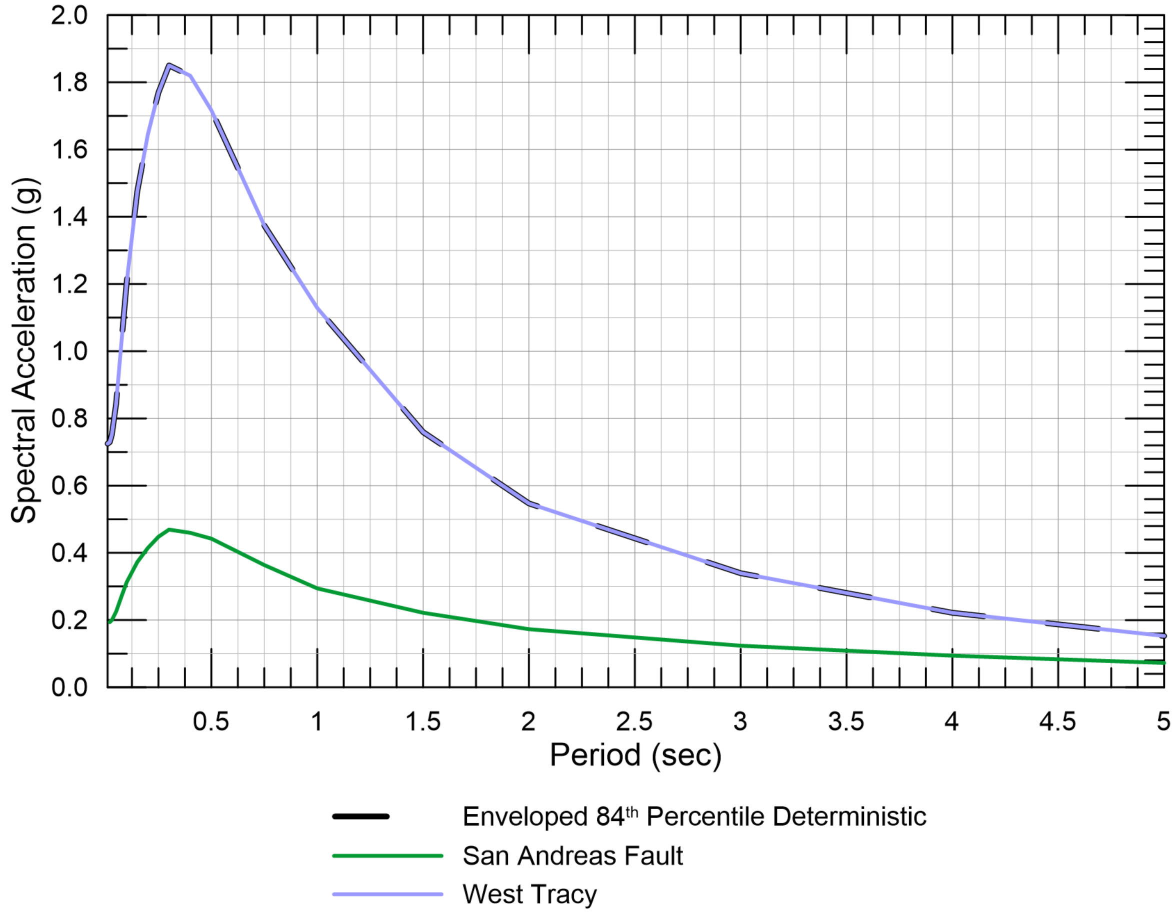


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Figure 11
5%-Damped 84th Percentile Deterministic Spectra for Union Island Shaft

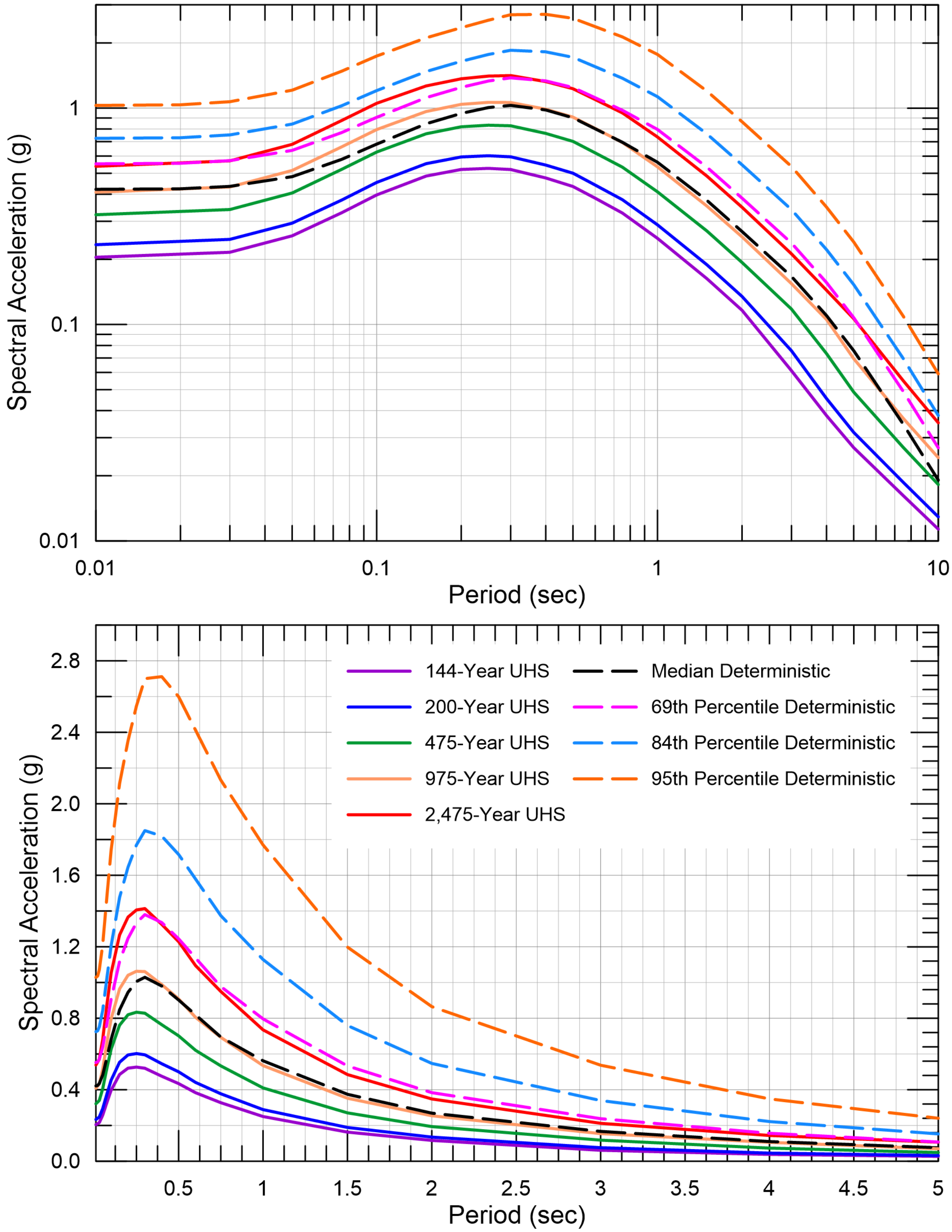

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Figure 12
Comparison of 5%-Damped UHS and Enveloped Deterministic Spectra for Union Island Shaft

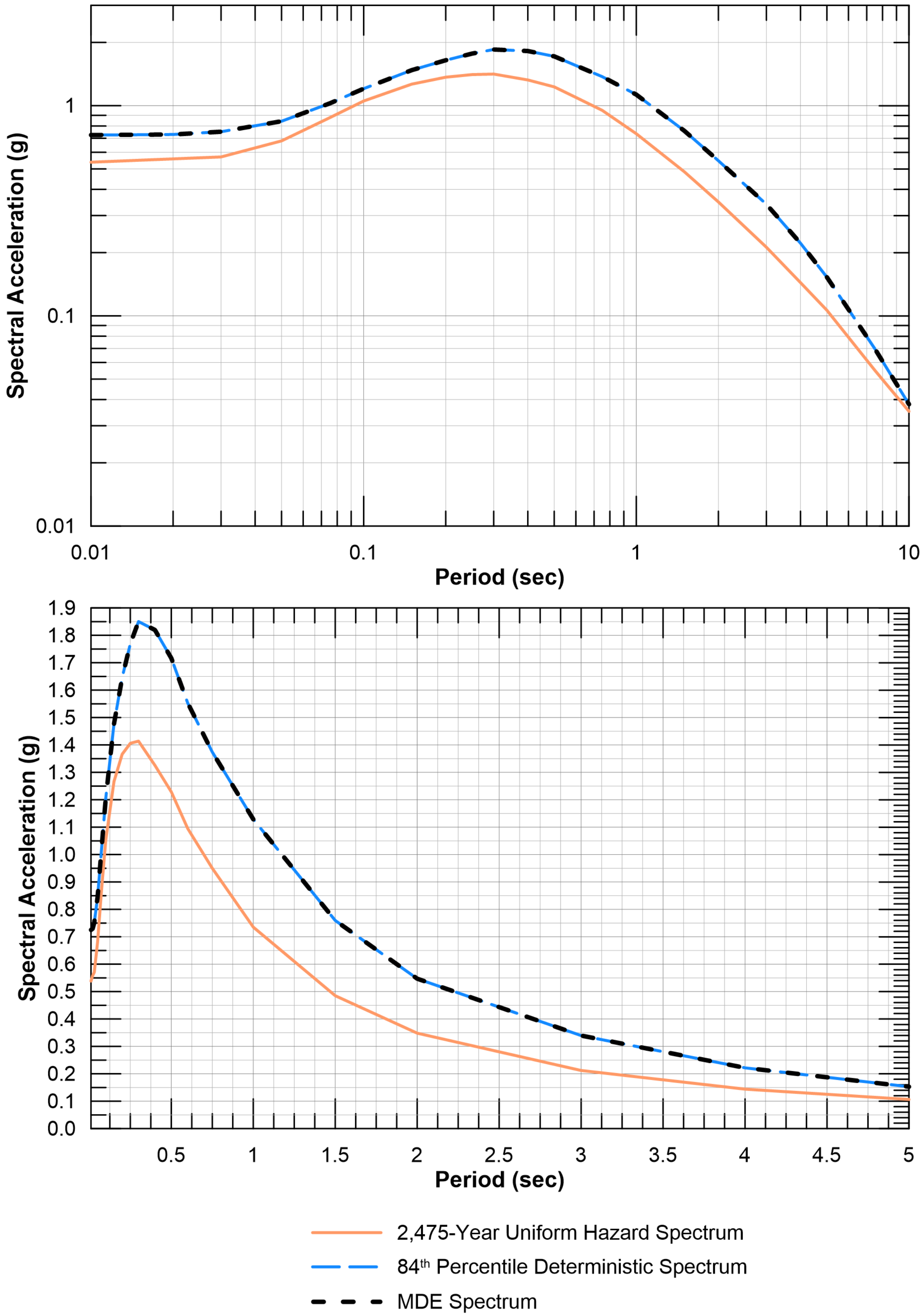

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Figure 13
MDE Response Spectrum for Union Island Shaft

Attachment 3
Documentation of Analytical SPT Procedure

Bethany Alternative Analytical Results

WAPA-SLTP-B3 (Pumping Plant)				DCW-DH-014 (Pumping Plant)				PC60-PCA-43 (Union Island Shaft)			
Min Spacing (ft)			N/A	Min Spacing (ft)			N/A	Min Spacing (ft)			45
Replacement Ratio			0%	Replacement Ratio			0%	Replacement Ratio			14%
Elev (ft)	**Soil Type**	**Unimproved Liquefaction Concern?**	**Improved Liquefaction Concern?**	**Elev (ft)**	**Soil Type**	**Unimproved Liquefaction Concern?**	**Improved Liquefaction Concern?**	**Elev (ft)**	**Soil Type**	**Unimproved Liquefaction Concern?**	**Improved Liquefaction Concern?**
60	SC	**NO-Above GW**	**NO-Above GW**	17	CL	**NO**	**NO**	-10	PT	**YES**	**NO**
56	SC	**NO-Above GW**	**NO-Above GW**	11	CL	**NO**	**NO**	-14	SM	**YES**	**NO**
51	CL	**NO**	**NO**	6	CL	**NO**	**NO**	-17	SP	**YES**	**NO**
46	SC	**NO-Above GW**	**NO-Above GW**	-1	CL	**NO**	**NO**	-26	CH	**NO**	**NO**
41	SM	**NO-Above GW**	**NO-Above GW**	-4	SC	**NO**	**NO**	-28	SP	**YES**	**NO**
36	CL	**NO**	**NO**	-9	SC	**NO**	**NO**	-31	SM	**YES**	**NO**
31	SM	**NO**	**NO**	-14	SC	**NO**	**NO**	-42	SP	**NO**	**NO**
26	CL	**NO**	**NO**	-21	CL	**NO**	**NO**	-51	CH	**NO**	**NO**
21	CL	**NO**	**NO**	-24	CL	**NO**	**NO**				
				-29	CL	**NO**	**NO**				
				-34	CL	**NO**	**NO**				
				-39	CL	**NO**	**NO**				
				-44	CL	**NO**	**NO**				
				-49	CL	**NO**	**NO**				
				-54	CL	**NO**	**NO**				
				-59	ML	**NO**	**NO**				
				-64	CH	**NO**	**NO**				
				-69	CL	**NO**	**NO**				
				-74	CH	**NO**	**NO**				
				-79	CL	**NO**	**NO**				

Using updated PGA and M from LCI(2021) and PGA reduction factor from 1-D response analysis
BPP: Assuming depth to Modesto Fm. (K=1.5) at 10ft, depth to Riverbank Fm. (K=1.6) at 60ft (USGS Profile West)
Union Island: Assuming depth to Modesto Fm. (K=1.5) at 15ft

Note:
NO: FS ≥ 1
YES: FS < 1

Attachment 4
Documentation of Analytical CPT Procedure

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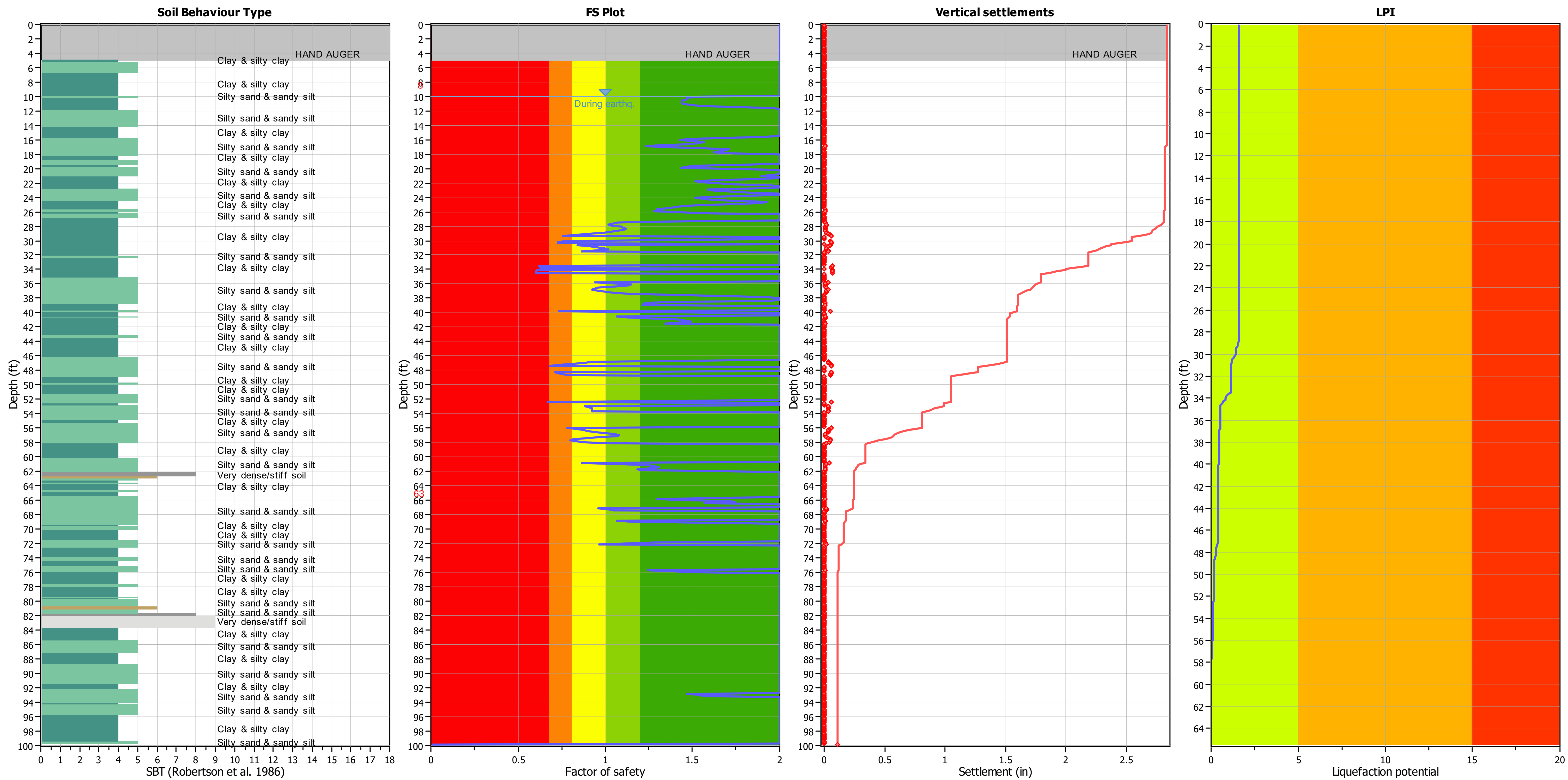
http://www.geologismiki.gr

Project: **DCA Liquefaction Evaluation**

Location:

CPT: Bethany PP_DCW-CPT-027

Total depth: 99.90 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.33	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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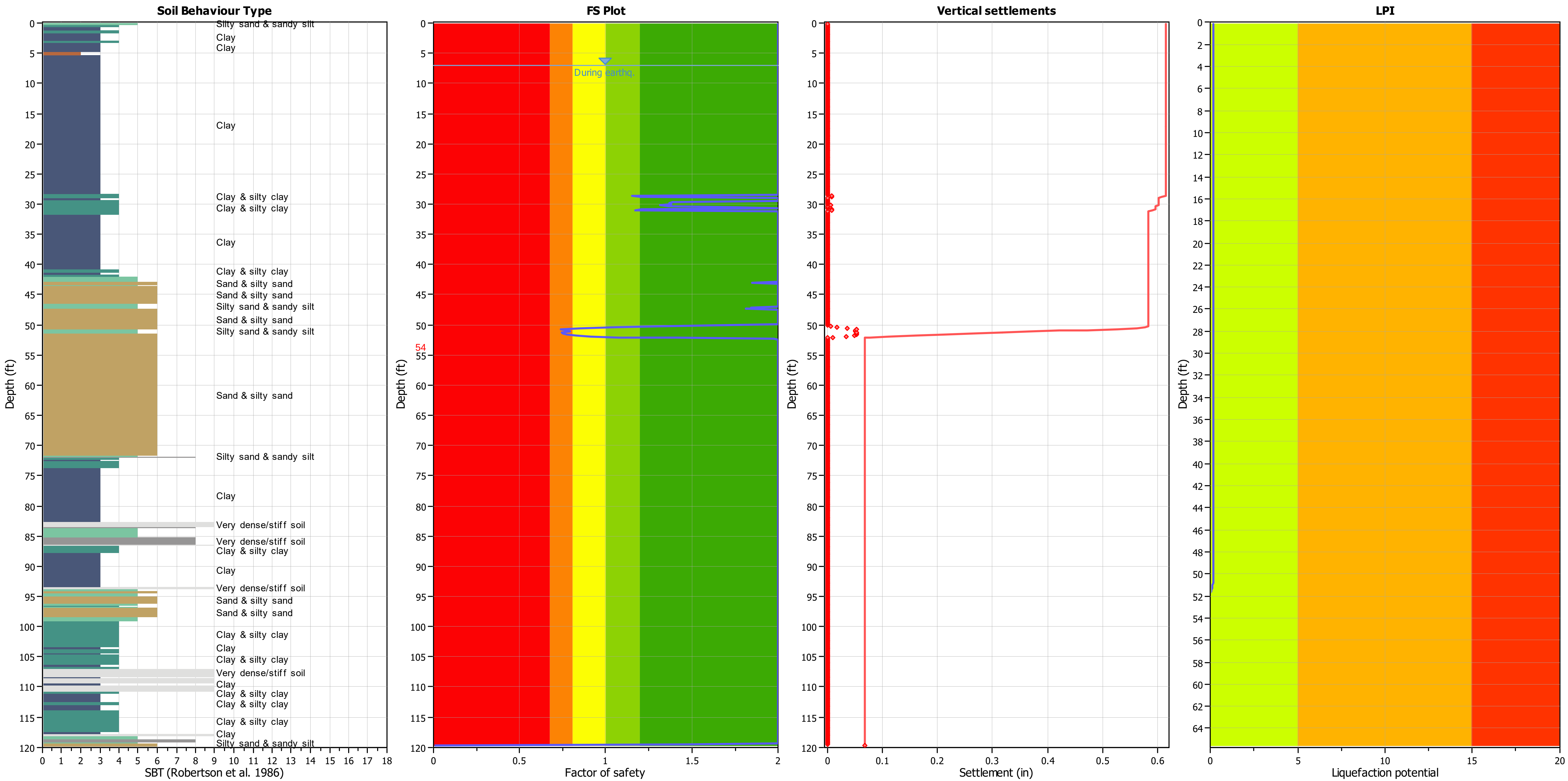
Project: DCA Liquefaction Evaluation

Location:

CPT: Union Is_vi-aip-cpt-6

Total depth: 119.70 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.20	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Attachment 5
Comparative plot of predicted versus measured blow counts (DCW-CPT-027 versus DCW-DH-014)

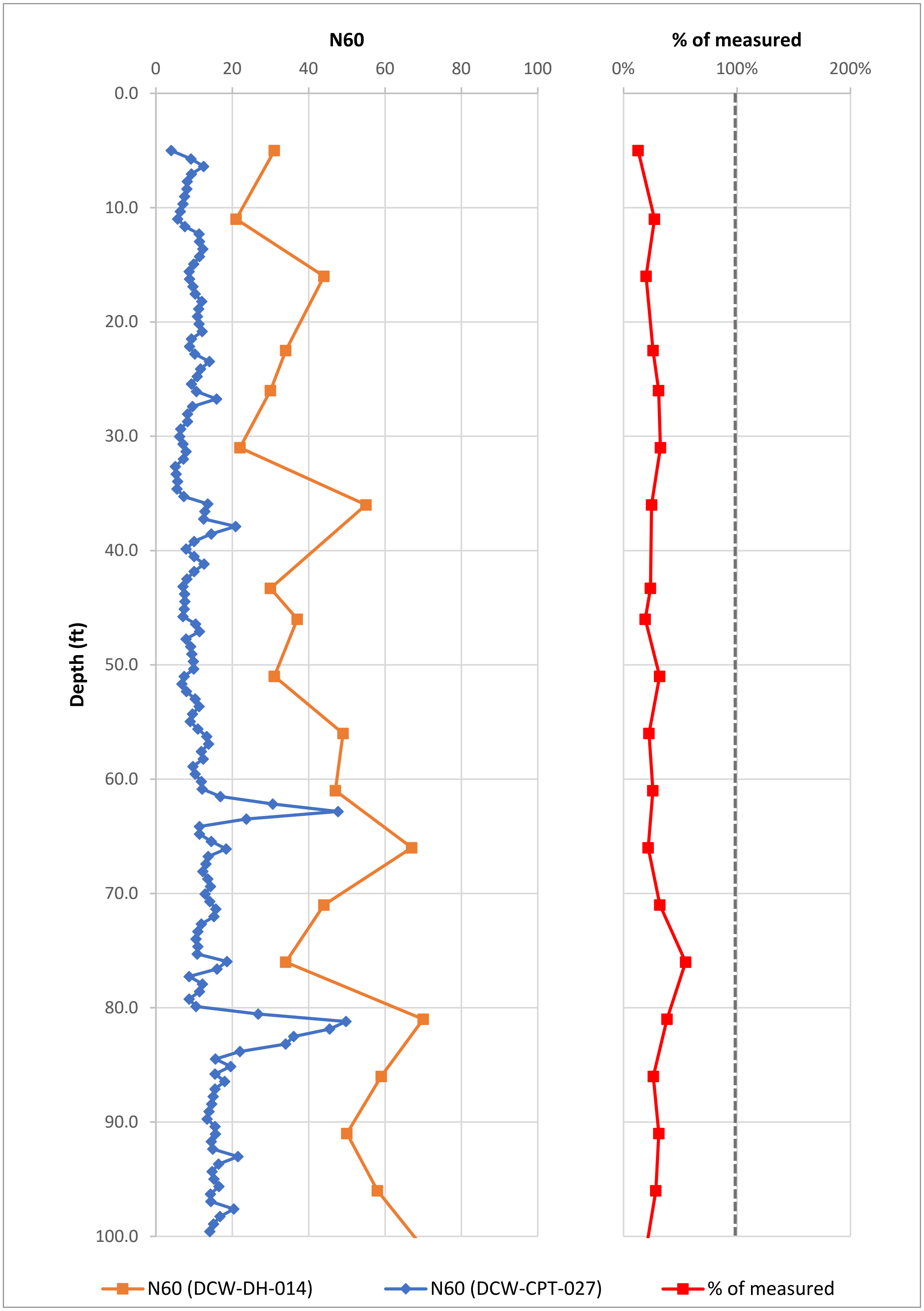


Figure A5: Measured N60 (orange) and calculated N60 (blue) versus depth [left] and percent calculated N60 as compared to measured N60 [right]. In the right plot, a value of 100% indicates that the measured and calculated N60 values are the same at that depth.

Attachment 6
Shear Wave Velocity Measurements from DCW-CPT-027

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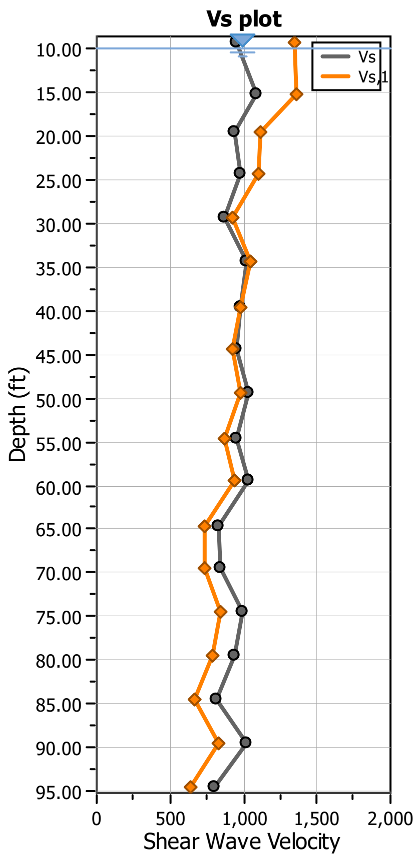
Vs BASED LIQUEFACTION ANALYSIS REPORT (NCEER 1998)

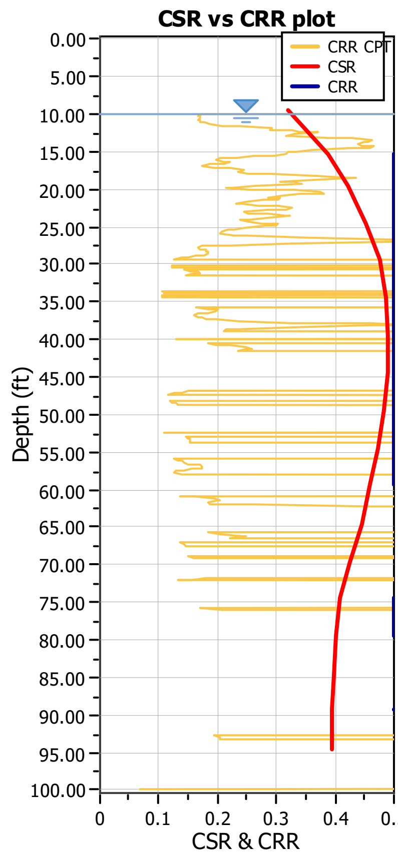
Project title : DCA Liquefaction Evaluation **Location :**

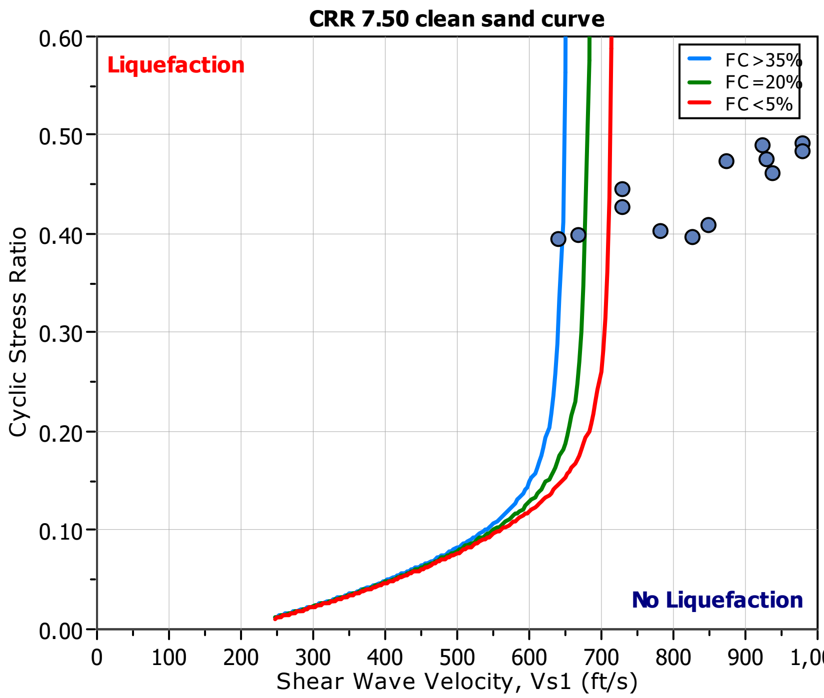
CPT file : DCW-CPT-027

:: Input parameters and analysis properties ::

Calculation m;ethod:	NCEER (1998)
G.W.T. (in-situ):	1.00 ft
G.W.T. (earthq.):	10.00 ft
Earthquake magnitude M_w:	6.90
Peak ground acceleration:	0.62g







:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (ft)	Weight (pcf)	u_o (tsf)	σ_v (tsf)	σ'_v (tsf)	r_d	CSR	K_σ	MSF	CSR*	Can Liquefy
1	9.35	115.00	0.26	0.54	0.28	0.98	0.394	1.00	1.24	2.000	Yes
2	15.25	115.00	0.44	0.88	0.43	0.96	0.478	1.00	1.24	0.386	Yes
3	19.52	115.00	0.58	1.12	0.54	0.95	0.523	1.00	1.24	0.423	Yes
4	24.44	115.00	0.73	1.41	0.67	0.94	0.559	1.00	1.24	0.452	Yes
5	29.36	115.00	0.88	1.69	0.80	0.93	0.585	1.00	1.24	0.475	Yes
6	34.44	115.00	1.04	1.98	0.94	0.89	0.586	0.97	1.24	0.487	Yes
7	39.53	115.00	1.20	2.27	1.07	0.85	0.578	0.95	1.24	0.491	Yes
8	44.45	115.00	1.36	2.56	1.20	0.81	0.566	0.93	1.24	0.489	Yes
9	49.37	115.00	1.51	2.84	1.33	0.77	0.549	0.92	1.24	0.483	Yes
10	54.62	115.00	1.67	3.14	1.47	0.73	0.529	0.90	1.24	0.472	Yes
11	59.38	115.00	1.82	3.41	1.59	0.69	0.508	0.89	1.24	0.460	Yes
12	64.63	115.00	1.99	3.72	1.73	0.65	0.483	0.88	1.24	2.000	Yes
13	69.55	115.00	2.14	4.00	1.86	0.61	0.458	0.87	1.24	2.000	Yes
14	74.47	115.00	2.29	4.28	1.99	0.57	0.432	0.86	1.24	0.407	Yes
15	79.39	115.00	2.45	4.56	2.12	0.55	0.422	0.85	1.24	0.402	Yes
16	84.48	131.00	2.60	4.90	2.29	0.54	0.413	0.84	1.24	2.000	Yes
17	89.40	115.00	2.76	5.18	2.42	0.53	0.406	0.83	1.24	0.396	Yes
18	94.48	115.00	2.92	5.47	2.56	0.51	0.399	0.82	1.24	0.393	Yes

Abbreviations

Depth: Depth from free surface where SPT was performed (ft)
u_0: Water pressure at test point (tsf)
σ_v: Total overburden pressure at test point (tsf)
σ_v': Effective overburden pressure based on GWT during earthquake (tsf)
r_d: Nonlinear shear mass factor
CSR: Cyclic Stress Ratio ()
MSF: Effective overburden stress factor
K_σ: Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (ft)	V_s (ft/s)	Fines %	n	V_{s1} (ft/s)	V_{s1c} (ft/s)	$CRR_{7.5}$	F.S.	Can Liquefy

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (ft)	V_s (ft/s)	Fines %	n	V_{s1} (ft/s)	V_{s1c} (ft/s)	$CRR_{7.5}$	F.S.	Can Liquefy

Abbreviations

Depth: Depth from free surface where Vs was performed (ft)
V_s: Estimated Vs (ft/s)
n: Stress exponent normalization factor
V_{s1}: Normalized Vs (ft/s)
V_{s1c}: Critical value of Vs1, which separates contractive and dilative behavior (tsf)
$CRR_{7.5}$: Cyclic Resistance Ratio for M_w 7.50
F.S.: Factor of safety against liquefaction