COWI

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| **TITLE** | Summary of May 27, 2026, ECRB Meeting of the Berkeley Pier and Ferry Terminal Project; Pre-Application Review Response |
| **TO** | Harriet Lai Ross, Regulatory Director<br>Lesley Ewing, Sr. Coastal Engineer |
| **FROM** | James Connolly, Project Manager, COWI |
| **COPY** | Liza McNulty, Capital Improvement Program Manager, Jessica Rivas Deputy Project Engineer (COWI), Lindsay Martien Principal Shoreline Development Analyst (BCDC) |

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**DATE** July 23, 2026

**REVISION** 1
**OUR REFERECE** A272767

This memorandum and its attachments provide responses to the comments received in a letter summary of the May 27, 2026 Engineering Criteria Review Board (ECRB) meeting transmitted to the City of Berkeley (Liza McNulty) via email on June 23rd, 2026.

**Comment #1: Shoreline Stability** – The applicant determined the critical yield acceleration for different search limits and found no changes in Ky value. The estimated displacement of the rock dike, for an MCE event, is 5”-7”. Geologic conditions for the rock dike were determined from borings through the dike and sonic tests, finding a friction angle of 45 degrees. However, during discussion of the pile sleeve, concerns were raised about the adequacy of the response spectra in capturing local seismic conditions. Since displacement of the rock dike is critical to the pile sleeve design and overall stability of this critical facility, additional information for dike stability has been requested, as discussed in Structural concerns.

Response:

See Attachment 1 – Development of ASCE 7-22 Design Earthquake Response Spectrum, authored by ENGEO. This letter outlines the procedure used to develop the ASCE 7-22 site-specific design response spectrum for the Berkeley Water Transportation Pier Ferry (BWTPF).

**Comment #2: Summary of Coastal Analysis** – ECRB members continued to express uncertainty over the wave analysis and whether a large enough wave was used in the analysis, especially for waves coming from the NW. Waves will affect the project in two significant aspects – ferry operations and structural forces. Regarding operation concerns, WETA noted acceptance of the waves and the anticipated downtimes. It was also noted that if waves were too high at the terminal for ferry service, conditions in the Bay would likely also limit ferry use. With respect to structural stability, the wave forces would be less than other considered forces and these other forces would be used for design conditions. The ECRB recommended that the applicant continue to consider further analysis of wind data but did not recommend that the applicant return again with additional wave analysis.

Response:

We can confirm that northwest wave conditions were included as part of the overall assessment using long-term regional wind and wave datasets, NOAA hindcasts, offshore buoy observations, and spatially varying wind forcing. The wave study was not based on a single design direction but instead evaluated wave conditions approaching the project site from multiple directions representative of the variability observed throughout San Francisco Bay.

The project team acknowledges the ECRB recommendation to continue considering available wind data. While additional local wind measurements may provide further insight into site-specific wind conditions, the wave analysis was intentionally based on longer-term regional datasets because wave generation in San Francisco Bay is controlled not only by local wind speed at the project site, but also by wind conditions over the larger fetch areas that generate and sustain waves before they reach the terminal location. Wind stations located across the Bay and along the primary fetch directions provide a more representative basis for assessing wave generation than short-duration measurements collected directly at the site. The study relied on long-term regional wind and wave datasets, NOAA hindcasts, offshore buoy observations, and spatially varying wind forcing to characterize wave conditions.

The complete coastal assessment provides confidence that the wave analysis appropriately captures the climate relevant to the project and provides a basis for the assessment of ferry operations, wave-induced downtime, structural loading, and coastal design.

**Comment #3: Structural** – The overall pier and breakwater designs raised no concerns; however, the pile sleeve that will be used for the landward most piles were of concern. Since the February 2026 ECRB meeting, the pile sleeve design changed in response to ECRB comments, both with a larger sleeve, no material in the void, and a cover plate to keep debris out of the annulus. While the design changes were clear, the ECRB wanted further information concerning the design and its constructability.

For the design, the ECRB requested drawings of the sleeve details, the proposed sleeve cap, and depth below the rock dike to better understand the proposed sleeve and cap and ensure it would not damage the pile or cause bending.

The sleeve design would use a differential offset from the piles, providing a 10.75” gap at the seaward side and 17.75” on the landward side. These offsets were based upon the response spectra, anticipated rock dike displacement, and explanation of the lateral piles shown in the plans. ECRB requested documentation about whether and how directivity, local conditions and directionality for features such as the Hayward fault were captured in development of the response spectra and considerations of life safety.

In addition to the concerns related to the response spectra, the ECRB requested an examination of constructability and a failure mode analysis for the sleeved piles. This

failure analysis should consider existing or updated response spectra, whichever best captures all appropriate conditions and construction tolerances.

Response:

See Attachment 2 – Landside Pier Interface and Pile Sleeve Details. These drawings provide details for the pile sleeve, the sleeve cover plate, and the sleeve embedment depth. The sleeve is designed to go 2ft past the bottom of the rock dike. The clearance and geometry have been detailed to avoid impact from the design seismic displacements and kinematic movement.

In response to ECRB feedback, the cover plate was revised to an HDPE plate to prevent binding, see attached detail. The cover plate is attached to the pile by bolts. In the event of a demand exceeding design parameters, the cover plate primary failure mode is failure of the connecting bolts, which will not transfer significant structural demands or secondary loads to the structural pile.

A template for pile driving will be required on the landside piles to ensure construction tolerances are limited to within ½" of noted values and impact does not occur during design seismic events. In the event code compliant seismic displacement limits are exceeded, the pile may strike the sleeve. This scenario has been analyzed. In this scenario, the pile striking the sleeve does not result in failure of the structural pile.

The ultimate failure mode of the structural piles is highly ductile, characterized by the formation of two plastic moment hinges—one at the top connection and one below the mudline. This mechanism provides substantial displacement capacity, ensuring collapse prevention.

The installation process utilizes standard, well-established marine construction methodologies, highly comparable to installing a drilled shaft with temporary casing. The sequence is as follows:

1. Drive outer steel sleeve through the rock dike
2. Use an auger to remove soil and rock inside the steel sleeve down to the required clean-out depth, establishing the permanent annulus.
3. Install template to minimize construction tolerances
4. Drive structural steel pile on the inside of the casing
5. Attach the cover plate assembly to seal the annulus.

**Comment #4: Additional Concerns** - During discussion of the proposed project, the ECRB members noted concerns with both the navigation channel and sedimentation shoreward of the breakwater. WETA confirmed that the channel width was adequate for the proposed ferry service and operational needs. ECRB members also noted that the breakwater, while reducing wave energy at the ferry terminals, will also increase sedimentation shoreward of the breakwater. Both the issues with the navigation channel

and sedimentation inland of the breakwater will be brought to the attention of BCDC staff and will be addressed through future permits for maintenance dredging.

Response:

WETA has confirmed that the proposed navigation channel dimensions are adequate to support the planned ferry service and vessel operations. The channel geometry and berth configuration were developed in coordination with WETA and are consistent with the operational requirements of the proposed ferry fleet.

With respect to sedimentation, the reduction in wave energy provided by the breakwater may promote localized sediment deposition within the sheltered area immediately behind the structure. Given the separation between the breakwater and the shoreline, the zone of influence is limited, and any increase in deposition is expected to remain localized rather than extend to the shoreline.

We acknowledge that future maintenance dredging may be required to maintain navigational depths and operational functionality over the life of the project. Any future dredging activities are anticipated to be at least 10-years after the initial dredging episode and would be subject to applicable regulatory approvals and permits.

# Attachment 1
# Development of ASCE 7-22 Design Earthquake Response Spectrum

Project No.
**25022.000.001**

June 4, 2026
Revised July 17, 2026

Mr. James Connolly
COWI
555 12th Street, Suite 1700
Oakland, CA 94601

Subject: Berkeley Water Transportation Pier Ferry
Berkeley, California

## DEVELOPMENT OF ASCE 7-22 DESIGN EARTHQUAKE RESPONSE SPECTRUM

Dear Mr. Connolly:

We prepared this letter to outline our procedure for developing the ASCE 7-22 site-specific design response spectrum for the Berkeley Water Transportation Pier Ferry (BWTPF) project as presented during the San Francisco Bay Conservation and Development (BCDC) Engineering Criteria Review Board (ECRB) meeting on May 27, 2026.

## SITE-SPECIFIC SEISMIC-HAZARD ANALYSIS

We performed our site-specific seismic hazard analysis (SHA) in accordance with Chapter 21 of ASCE 7-22 modified by ASCE 61-26 (draft) commentary. Consistent with the ASCE 61-26 (draft) commentary, "Geometric mean ground motions are considered appropriate for piers and wharves and maximum direction ground motions are not recommended," and "The hazards defined in ASCE 7-10 and more recent editions are now a function of a risk-targeted maximum considered earthquake ($MCE_R$)… the committee felt that this approach is not necessarily applicable to marine structures." We developed an ASCE 7-22-compliant site-specific response spectrum for the median component ($RotD_{50}$) of ground shaking without incorporating fragility factors.

We provide an outline of our general procedure below and subsequently discuss the details of our analysis.

- We performed a probabilistic seismic-hazard analysis (PSHA) to develop a $RotD_{50}$ uniform hazard response spectrum (UHS) corresponding to a 2-percent probability of exceedance in 50 years (2,475-year return period).
- We performed a deterministic seismic-hazard analysis (DSHA) to develop an 84$^{th}$-percentile $ROTD_{50}$ response spectrum.
- We rotated the $RotD_{50}$ deterministic and probabilistic response spectra to a maximum-component ($RotD_{100}$) response spectra by applying orientation-independent rotation factors per Shahi and Baker (2014).

- We applied period-dependent amplification factors to incorporate near-fault directivity effects by calculating an NHR3-recommended weighted mean of the Chiou and Spudich (2013), Bayless and Somerville (2013), and Bayless et al. (2020) models for a 2,475-year return period event.
- We compared the DSHA response spectrum with the deterministic lower limit to develop the deterministic response spectrum in accordance with Section 21.2.2 of ASCE 7-22.
- We compared the probabilistic and deterministic response spectra to obtain the MCE response spectrum.
- We compared the MCE response spectrum with 80 percent of the general response spectrum (i.e., the code minimum) to develop the recommended site-specific MCE response spectrum.
- We multiplied the site-specific MCE response spectrum by two-thirds to obtain the site-specific DE response spectrum.
- We converted the site-specific RotD100 MCE and DE response spectra to a RotD50 response spectra by applying orientation-independent rotation factors per Shahi and Baker (2014).
- We developed an alternative 7 percent and 10 percent damped response spectra by using Rezaeian et al. (2014) damping scale factors.

## Seismic Source Model

At the commencement of the project, the 2018 National Seismic Hazard Model (NSHM) was the most-current available seismic model for analysis. In order to maintain consistency with other response spectrum developed for the project, we continued to utilize the 2018 NSHM, as implemented in the United States Geological Survey *nshmp-haz* software, to perform this analysis. The 2018 NSHM incorporates the Third California Earthquake Rupture Forecast model or UCERF3 (Field et al. 2014 and 2015) for California sources and seismicity rates. Background seismicity, modeled as gridded point sources, is also included in the model. The implementation of the 2018 NSHM in seismic-hazard codes considers many sources of epistemic uncertainty regarding alternate rupture scenarios, maximum magnitudes for individual faults, and alternate magnitude-recurrence relations. This uncertainty affects the mean hazard that is provided by hazard codes implementing the 2018 NSHM and is used in typical applications, including this analysis.

## Ground-Motion Models and Site Parameters

We used four semi-empirical ground-motion models (GMMs) from the Next Generation Attenuation West 2 (NGA West 2) project (Ancheta et al., 2014) to estimate ground motion for shallow crustal sources included within the 2018 NSHM. These models include Abrahamson et al. (2014) [ASK], Boore et al. (2014) [BSSA], Campbell and Bozorgnia (2014) [CB], and Chiou and Youngs (2014) [CY]. We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used a logic-tree approach and assigned equal weight (0.25) to each of the four GMMs in our analysis. Other ground-motion models are considered for different source types within the seismic source model, such as subduction sources; however, the contributions from these sources are negligible at the project site for the return periods considered due their large source-to-site distances.

The ground-motion models incorporate "site parameters" to model how subsurface soil will amplify or attenuate ground motions as they propagate from the bedrock below. For the GMMs we implemented, these site parameters include the following.

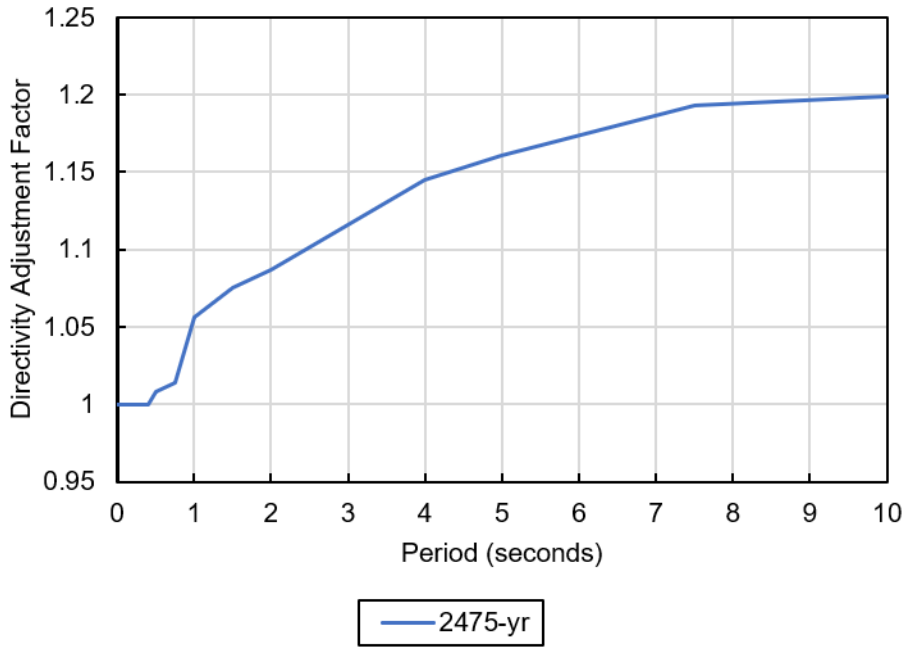
- Time-averaged shear-wave velocity ($V_S$) over the top 100 feet or 30 meters ($V_{S30}$)
- Depth at which $V_S$ reaches 3,280 feet/sec or 1.0 kilometer/sec ($z_{1.0}$)
- Depth at which $V_S$ reaches 8,200 feet/sec or 2.5 kilometers/sec ($z_{2.5}$)

We estimated a $V_{S30}$ value of 690 feet/sec (210 meters/sec) based on the measured and correlated $V_S$ profiles from our exploration program. To estimate $z_{1.0}$ and $z_{2.5}$, we used the USGS Bay Area Velocity Model version 8.3.0 Basin Depth models, as implemented in the USGS Site Data Application available through OpenSHA. We applied $z_{1.0}$ and $z_{2.5}$ values of 311 and 2,789 feet (95 and 860 meters) in our analysis, respectively.

## Near-Fault Source Directivity Effects

Directivity effects can increase long-period ground motions at near-fault sites (Somerville et al., 1997; Abrahamson 2000). We used the period-dependent models by Chiu and Spudich (2013), Bayless and Somerville (2013), and Bayless et al. (2020) to consider directivity effects, as implemented in the Natural Hazards Risk and Resiliency Research Center (NHR3) directivity based PSHA interactive tool (Mazzoni et al., 2023), to estimate the period dependent directivity adjustment factors for a return period of 2,475 years. We calculated a weighted mean of the directivity adjustment factors obtained from the CS13, BS13, and BSS20 models with weights of 0.5, 0.25, and 0.25, respectively, as recommended by NHR3. We present the weighted mean directivity adjustment factors used in Exhibit 1.

**EXHIBIT 1: Weighted Mean Directivity Adjustment Factor used for Probabilistic and Deterministic Response Spectra**



## Probabilistic Seismic-Hazard Analysis

We performed a PSHA for the project site to develop a set of hazard curves and resulting uniform hazard response spectra (UHS) for a return period of 2,475 years. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_{i}^{n} v_i \iint P[A > a|m,r]\, f_{Mi}(\mathrm{m})\, f_{Ri|Mi}(\mathrm{r},\mathrm{m}) dr dm$$

In this equation, the hazard **H*(a)*** is the annual frequency of earthquakes that produce a ground motion amplitude ***A*** higher than ***a***. Amplitude ***A*** may represent peak ground acceleration, velocity, or pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e., over all faults and areas). In the above equation, $v_i$ is the annual rate of earthquakes (with magnitude higher than some threshold $M_i$) in source ***i***, and $f_{Mi}(m)$ and $f_{Ri|Mi}(r,m)$ are the probability density functions for magnitude and distance, respectively. $P[A > a|m, r]$ is the probability that an earthquake of magnitude ***m*** at distance ***r*** produces a ground-motion amplitude ***A*** at the site that is greater than ***a***. Seismic sources may be either faults or background sources; the specification of source geometries and the calculation of $f_{Ri|Mi}$, are performed differently for these two types of sources.

We developed $RotD_{50}$ hazard curves for the peak ground acceleration (PGA) and for oscillator periods (T) ranging from 0.01 to 10.0 seconds. We developed a UHS by calculating the PSa value associated with a 2,475-year return period from the mean hazard curve at each oscillator period. We then converted these risk-targeted $RotD_{50}$ PSa values to maximum-component ($RotD_{100}$) PSa values by applying the maximum-component factors discussed in Shahi and Baker (2014). We additionally applied the directivity adjustment factors from the NHR3 tool. We present the $RotD_{100}$ 2,475-year return period probabilistic response with and without directivity adjustment in Appendix A.

## Disaggregation of the Seismic Hazard

We performed a disaggregation of the seismic hazard associated with the 2,475-year return period at the peak ground acceleration (PGA), and for spectral periods of 0.5, 1.0 and 2.0 seconds. We present the resulting faults and their respective contributions to the seismic hazard at the site for the 2,475-year return period in Table 1.

**TABLE 1: Summary of Disaggregation Results for a 2,475-Year Return Period***

| SOURCE | $R_{RUP}$ | | $M_W$ | CONTRIBUTION (%) | | | |
|---|---|---|---|---|---|---|---|
| | km | miles | | PGA | 0.5 sec | 1.0 sec | 2.0 sec |
| Hayward (North) [2] | 6.3 | 3.9 | 7.3 | 56.7 | 57.7 | 66.2 | 69.4 |
| San Andreas (Peninsula) [13] | 25.0 | 15.5 | 8.0 | 9.3 | 12.1 | 12.2 | 14.2 |
| Hayward (North) [3] | 6.5 | 4.1 | 7.0 | 3.0 | 2.9 | 3.1 | 2.8 |
| Hayward (North) [1] | 7.2 | 4.5 | 6.9 | 2.1 | 2.1 | 2.1 | <1.0 |
| San Gregorio (North) [3] | 28.4 | 17.7 | 7.8 | <1.0 | 2.4 | 2.2 | 1.0 |

*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

These results represent known fault sources contributing at least 1 percent to the seismic hazard at the site for at least one of the spectral periods considered and for the given return period. Background seismicity zones, such as gridded or areal sources, are not presented. The rupture distances ($R_{RUP}$) and mean moment magnitude ($M_W$) values listed are based on values assigned according to the 2018 NSHM and UCERF3, and the bracketed numbers after each fault name correspond to fault subsections assigned by the 2018 NSHM and UCERF3. Magnitudes vary slightly between the two fault models (FM 3.1 and 3.2) utilized by the 2018 NSHM and UCERF3 and from one spectral period to another. Therefore, for each source we present the maximum mean magnitude of the spectral periods considered where that source contributes significantly to the hazard. Note that the above fault tables are not exhaustive lists and other faults in the region may generate seismic shaking at the project site.

## Deterministic Seismic-Hazard Analysis

We performed a DSHA to develop the $RotD_{100}$ 84$^{th}$-percentile (i.e., lognormal mean plus one standard deviation) response spectrum considering scenario earthquakes (excluding background seismicity) with at least 1 percent contribution to the seismic hazard based on the 2,475-year disaggregation results shown in Table 1. Based on these consolidated source contributions, we considered Hayward, San Andreas and San Gregorio fault rupture scenarios in our DSHA.

Similar to the probabilistic response spectrum, we applied the maximum-component factors discussed in Shahi and Baker (2014) and the directivity factors previously discussed. Finally, we compared the $RotD_{100}$ 84$^{th}$-percentile deterministic response spectrum with the deterministic lower-limit response spectrum for Site Class DE specified in Table 21.2-1 of ASCE 7-22 and did not permit it to fall below these values. We present the $RotD_{100}$ deterministic response with and without directivity adjustment in Appendix B.

The deterministic response spectrum represents the weighted mean of the four NGA West 2 GMMs.

## Resulting Ground Surface Response Spectra

According to Section 21.2.3 of ASCE 7-22, the site-specific MCE response spectrum is the lesser of the PSHA and DSHA response spectra. At this site, the final deterministic response spectrum is less than the final probabilistic response spectrum for all oscillator periods considered; the DSHA response spectrum is, therefore, the MCE response spectrum. Additionally, we compared the site-specific MCE response spectrum to 80 percent of the general response spectrum for Site Class DE (i.e., the code minimum response spectrum) and selected the higher value at each oscillator period. We obtained the general response spectrum for Site Class DE from the ASCE Hazard Tool website. Subsequently, we calculated the site-specific DE response spectrum, which is defined as two-thirds of the site-specific MCE.

Lastly, we rotated the final $RotD_{100}$ DE and MCE response spectra to the $RotD_{50}$ spectra by applying orientation-independent rotation factors per Shahi and Baker (2014). We also developed the 7 percent and 10 percent damped response spectra by applying the Rezaeian et al. (2014) damping scale factors to the final 5 percent damped response spectra. In applying this method, we used the deterministically controlling Hayward (North) [2] event for the magnitude and rupture distance input. We present the development of the final $RotD_{100}$ response spectrum in Appendix C and the $RotD_{50}$ response spectrum in Appendix D.

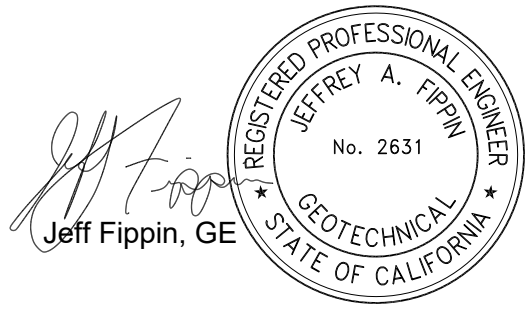
If you have any questions or comments regarding this letter, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

Vlad Zasmolin, PE

Jeff Fippin, GE




vz/jaf/dt

Attachments: Selected References
Appendix A: Probabilistic Seismic Hazard Response Spectra
Appendix B: Deterministic Seismic Hazard Response Spectra
Appendix C: ASCE 7-22 Site Specific $RotD_{100}$ Response Spectra
Appendix D: ASCE 7-22 Site Specific $RotD_{50}$ and $RotD_{100}$ Response Spectra

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# APPENDIX A

## Probabilistic Seismic Hazard Response Spectra

25022.000.001
June 4, 2026
Revised July 17, 2026

## Probabilistic Seismic-Hazard Analysis

We performed a PSHA for the project site to develop a set of hazard curves and resulting uniform hazard response spectra (UHS) for a return period of 2,475 years. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

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## Disaggregation of the Seismic Hazard

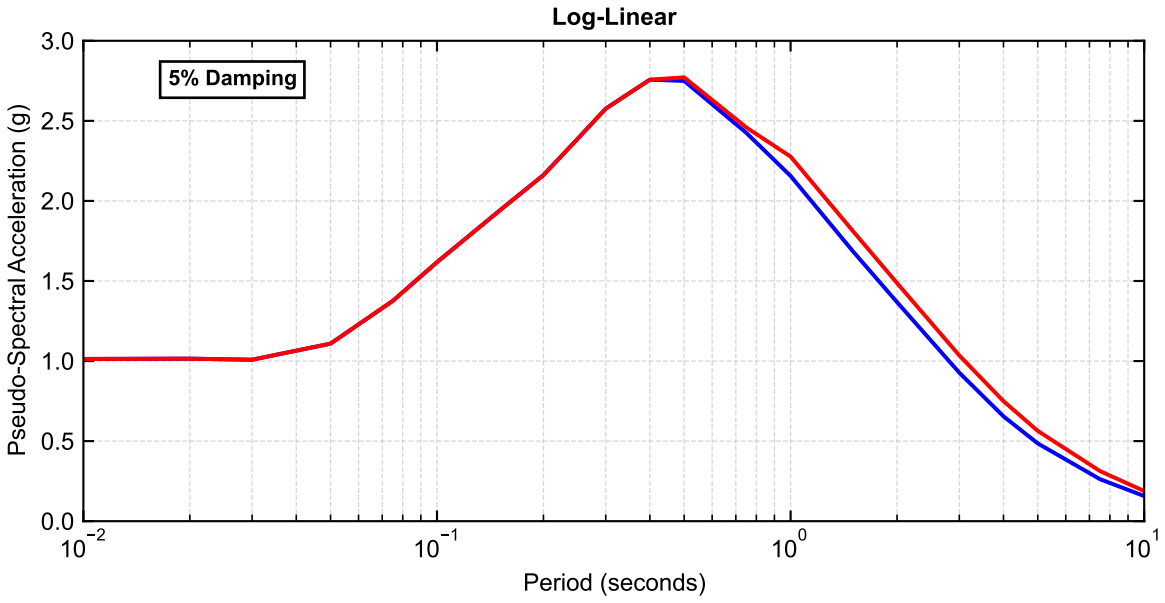
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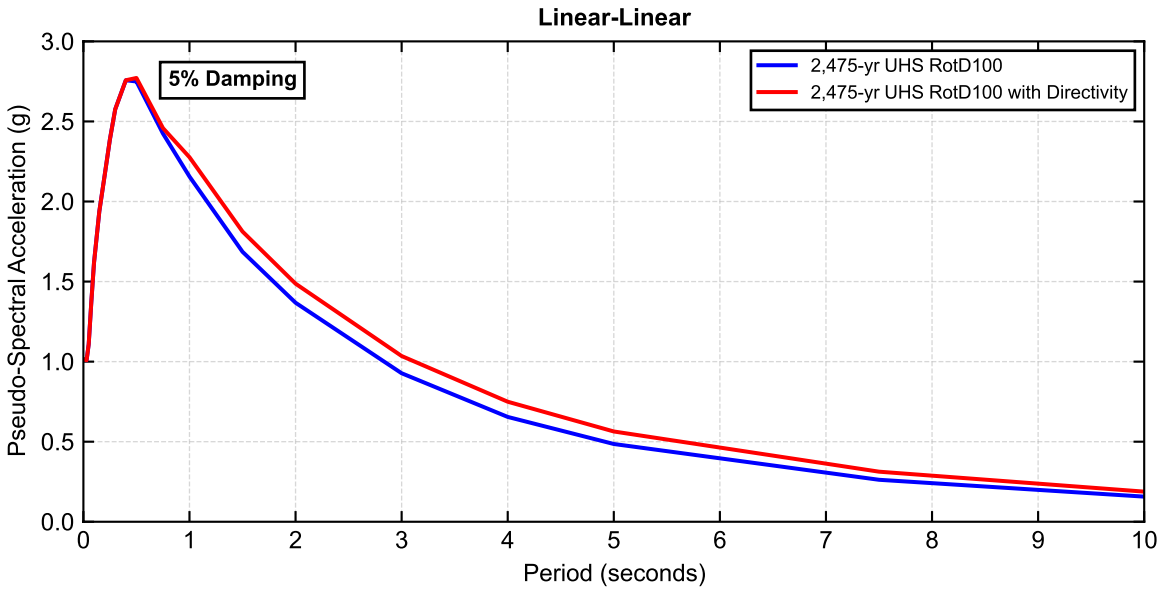
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*Based on the USGS Earthquake Hazard Toolbox: NSHM Conterminous U.S. 2018

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ENGEO Expect Excellence

PROBABLISTIC SEISMIC HAZARD RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

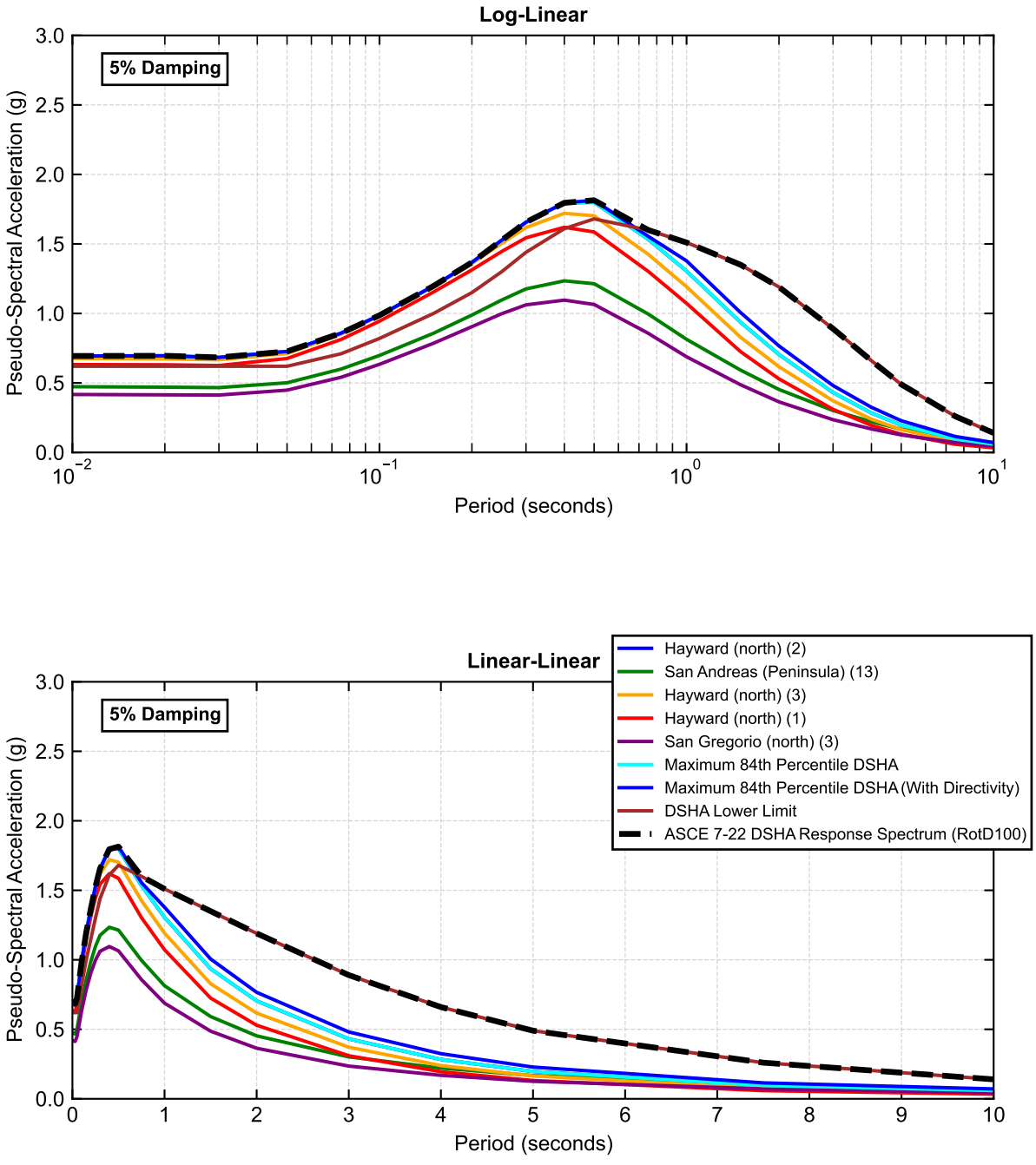
| | | |
|---|---|---|
| PROJECT NO: 25022.000.001 | | FIGURE NO. A |
| SCALE: AS SHOWN | | |
| DRAWN BY: VZ | CHECKED BY: JF | |

# APPENDIX B

## Deterministic Seismic Hazard Response Spectra

25022.000.001
June 4, 2026
Revised July 17, 2026

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DETERMINISTIC SEISMIC HAZARD RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

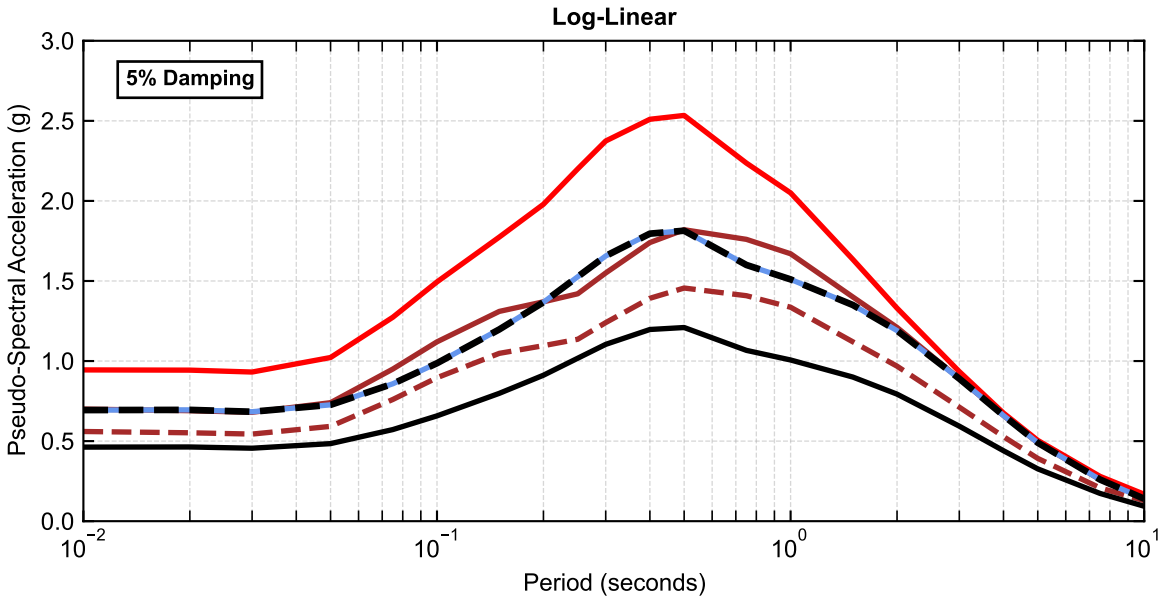
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|---|---|---|
| PROJECT NO: 25022.000.001 | | FIGURE NO. B |
| SCALE: AS SHOWN | | |
| DRAWN BY: VZ | CHECKED BY: JF | |

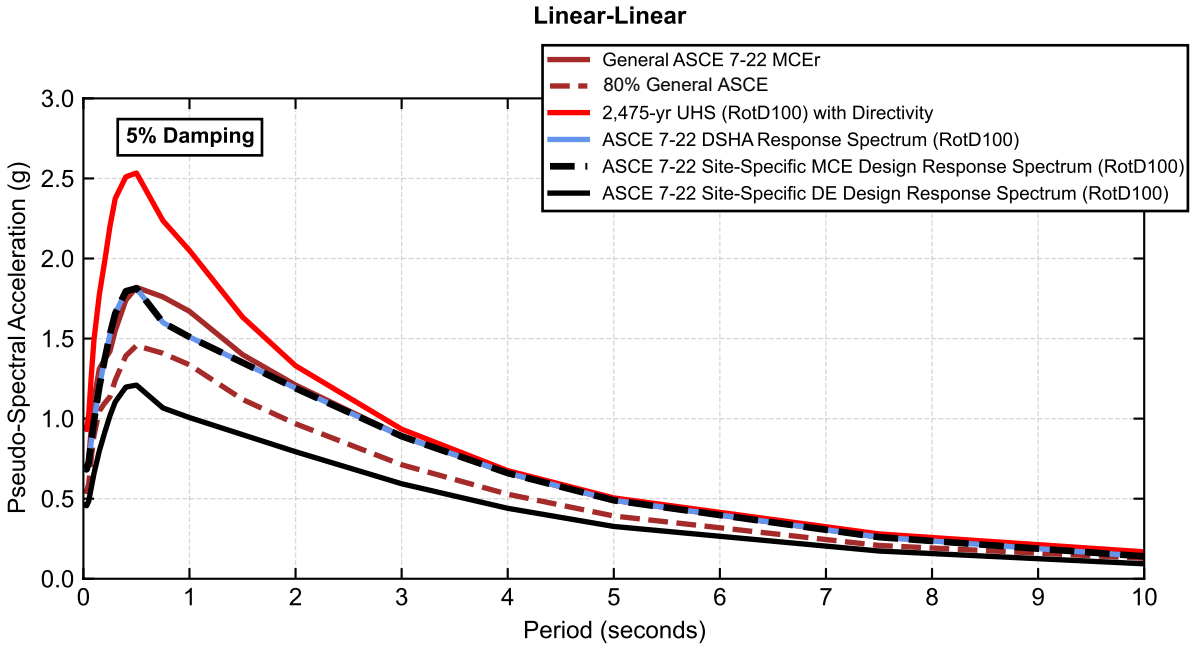
# APPENDIX C

## ASCE 7-22 Site Specific $RotD_{100}$ Response Spectra

25022.000.001
June 4, 2026
Revised July 17, 2026

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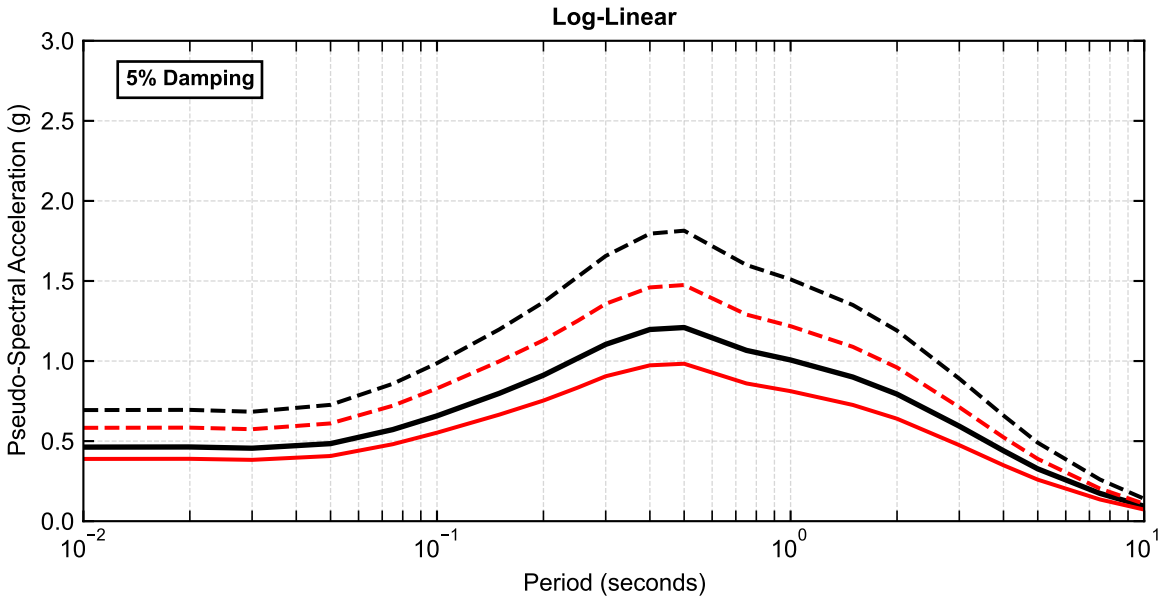

| | | | |
|---|---|---|---|
| ENGEO Expect Excellence | ASCE 7-22 SITE SPECIFIC RotD100 RESPONSE SPECTRA<br>BERKELEY WATER TRANSPORTATION PIER FERRY<br>BERKELEY, CALIFORNIA | PROJECT NO: 25022.000.001<br>SCALE: AS SHOWN<br>DRAWN BY: VZ CHECKED BY: JF | FIGURE NO.<br>C |

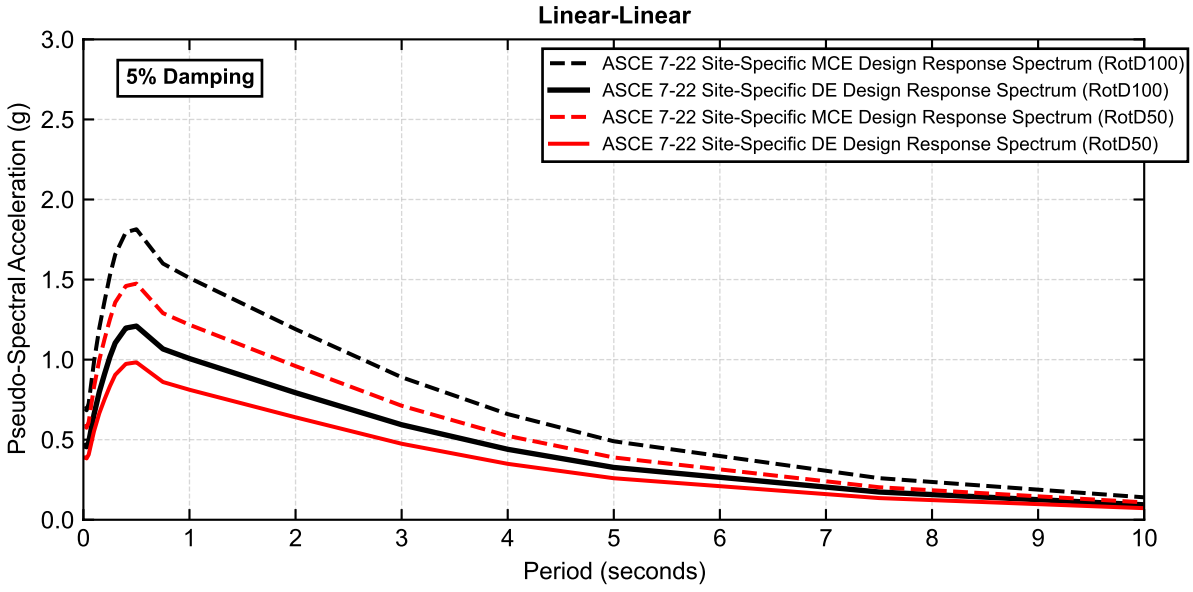
# APPENDIX D

## ASCE 7-22 Site Specific $RotD_{50}$ and $RotD_{100}$ Response Spectra

25022.000.001
June 4, 2026
Revised July 17, 2026

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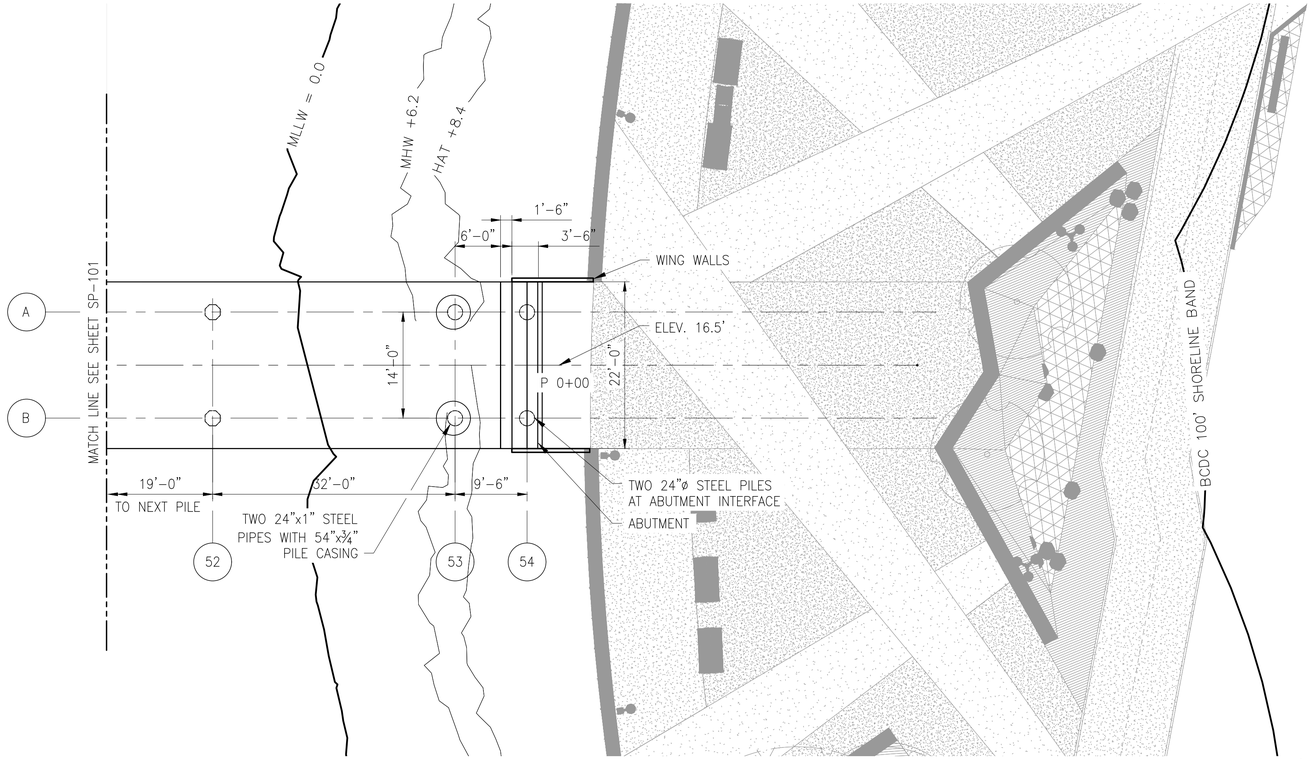

ENGEO Expect Excellence

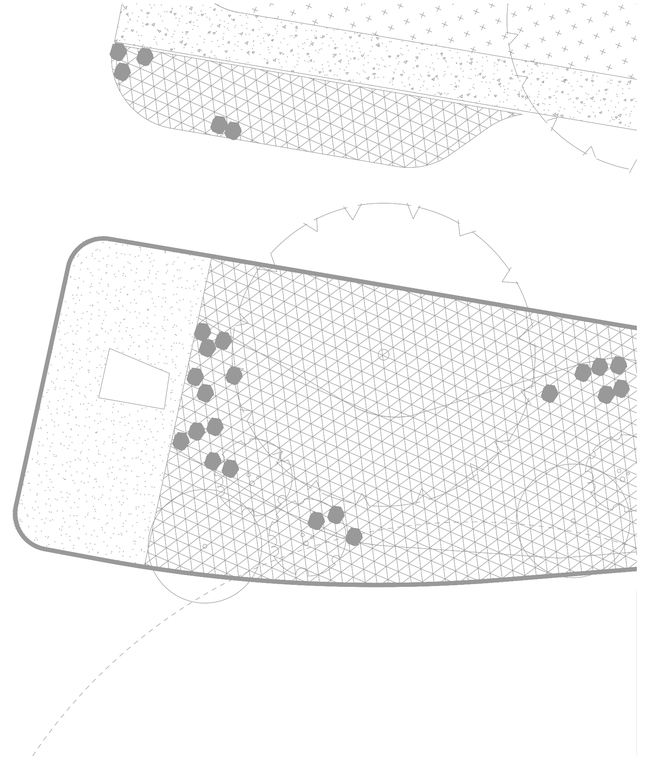
ASCE 7-22 SITE SPECIFIC $RotD_{50}$ AND $RotD_{100}$ RESPONSE SPECTRA
BERKELEY WATER TRANSPORTATION PIER FERRY
BERKELEY, CALIFORNIA

| | | |
|---|---|---|
| PROJECT NO: 25022.000.001 | | FIGURE NO. D |
| SCALE: AS SHOWN | | |
| DRAWN BY: VZ | CHECKED BY: JF | |

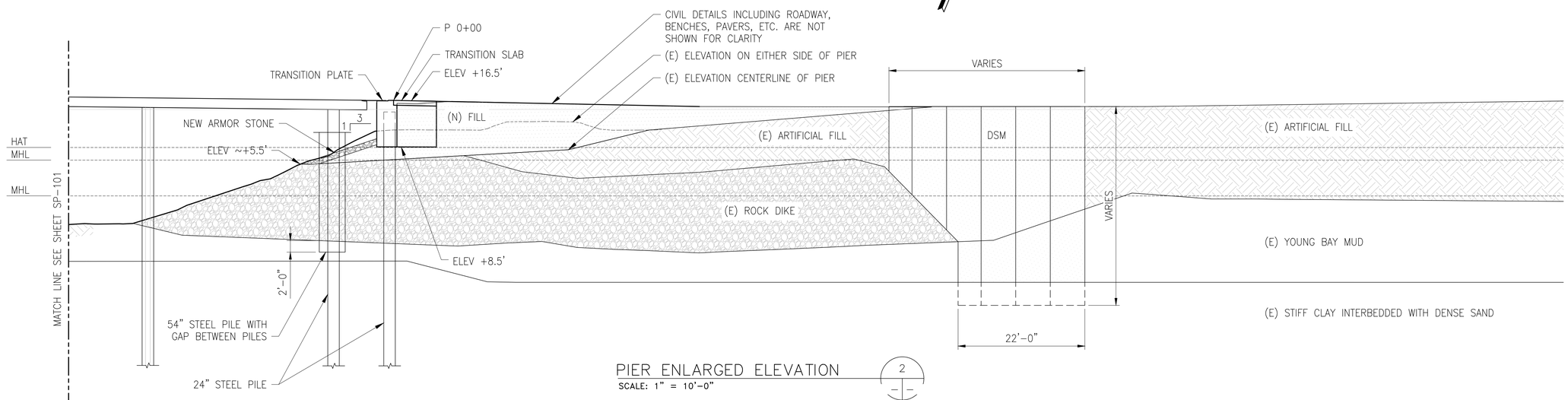
Attachment 2

Landside Pier Interface and Pile Sleeve Details



PIER ENLARGED PLAN
SCALE: 1" = 10'–0"




PIER ENLARGED ELEVATION
SCALE: 1" = 10'–0"

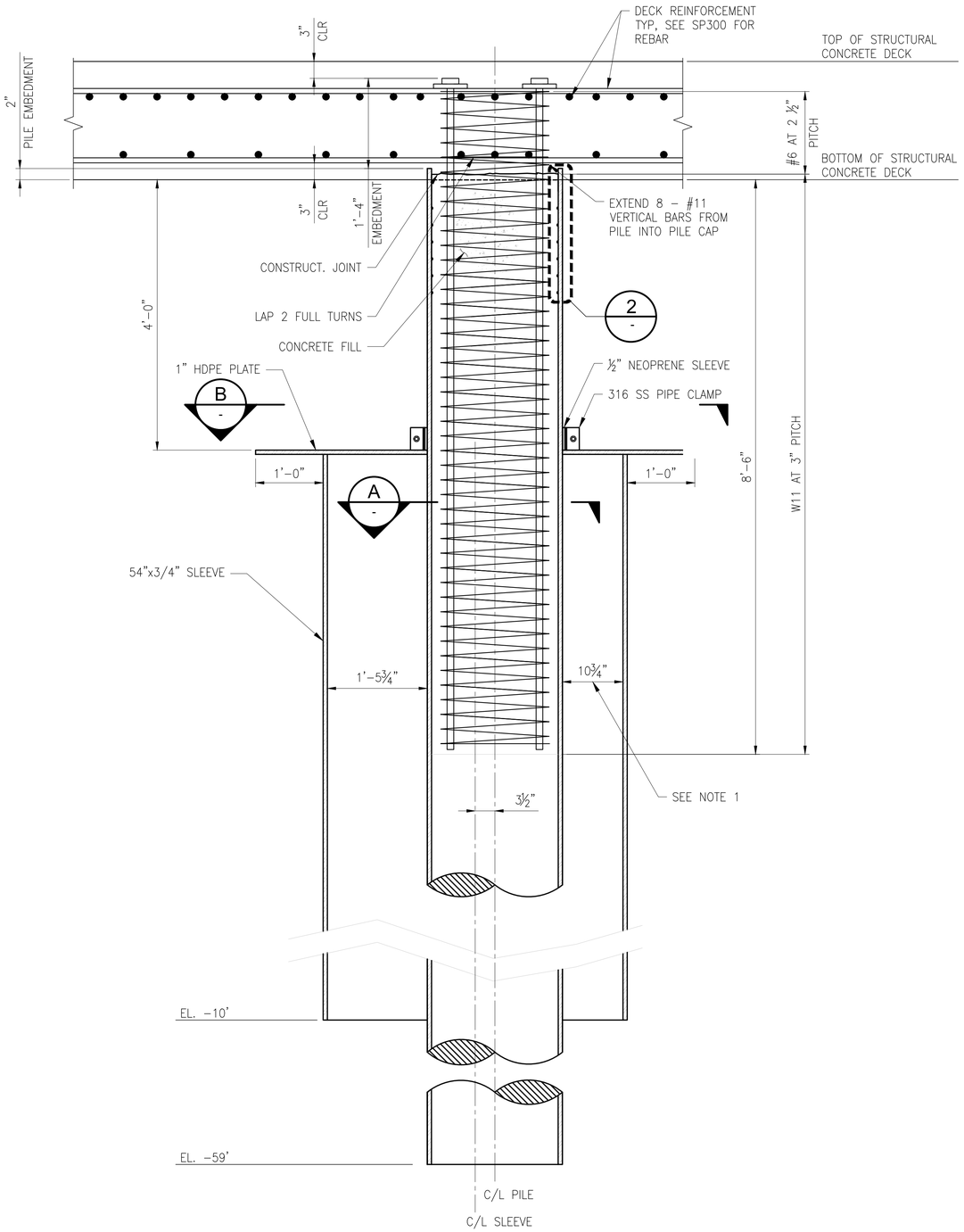
DRAFT

90% SUBMITTAL
AUGUST 21, 2026

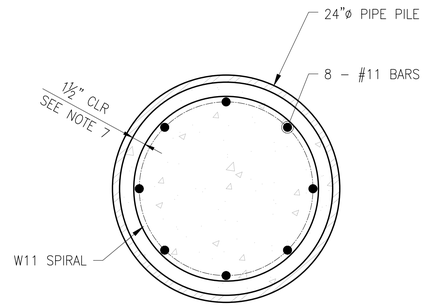
| REVISION | MARK | DATE | DESCRIPTION | APPROVAL |
|---|---|---|---|---|
| B.1 | | 01–11–2026 | 60% SUBMITTAL BCDC ECRB | JMC |
| B | | 12–12–2025 | 60% SUBMITTAL | JMC |
| A | | 06–27–2025 | 30% SUBMITTAL | JMC |

FILE NAME: C:\Users\vmdmo\COWI\A272767 – Berkeley – BWTPF – Documents\07_CAD\A272767–SP–100.dwg LAYOUT NAME: LANDSIDE INTERFACE AND DSM PLAN PLOTTED: Wednesday, July 08, 2026 – 6:59pm USER: MDMO

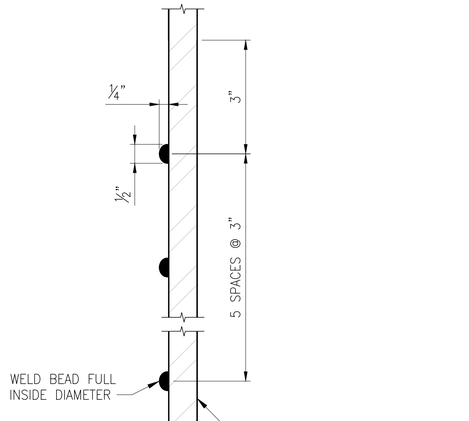
| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| PROJECT MANAGER: DATE | DEPICTION OF MONUMENTS: DATE | SUBMITTED: DATE | DESIGN JRVS | HORIZ. AS SHOWN | CITY OF BERKELEY PARKS RECREATION & WATERFRONT 1947 CENTER ST., 5th FR, BERKELEY CA94704 | COWI 555 12th Street, 17th Flr Oakland, CA 94607 Tel.: 510.839.0572 Fax: 510.839.9155 Website: www.cowi-na.com | BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT<br>CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA<br>LANDSIDE INTERFACE AND DSM PLAN | PLAN |
| | SURVEY PARTY CHIEF | SUPERVISING CIVIL ENGINEER R.C.E. EXP. | DRAWN ENSN | VERT. | | | | FILE |
| 0 1 2 3<br>FOR REDUCED PLANS – ORIGINAL SCALE IS IN INCHES | WATERSHED REVIEW: DATE | APPROVED: DATE | CHECK JBSW | BOOK | | | | SP–100 |
| | | CITY ENGINEER R.C.E. EXP. | AS BUILT | DATE 12/16/24 | | | | SHEET 1 OF xx |




PILE DECK CONNECTION AT STEEL PILES (1)
SCALE: 1" = 1'-0"

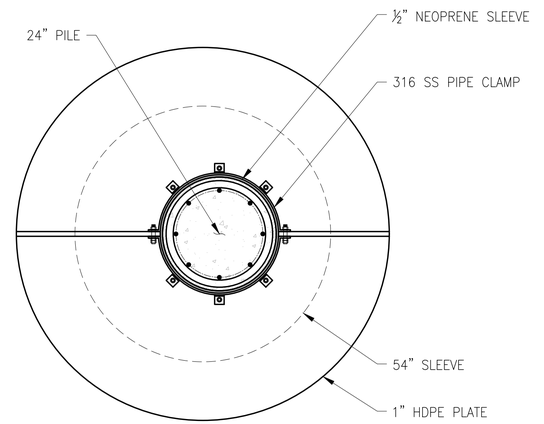



SECTION – PILE (A)
SCALE: 1 1/2"= 1'-0"

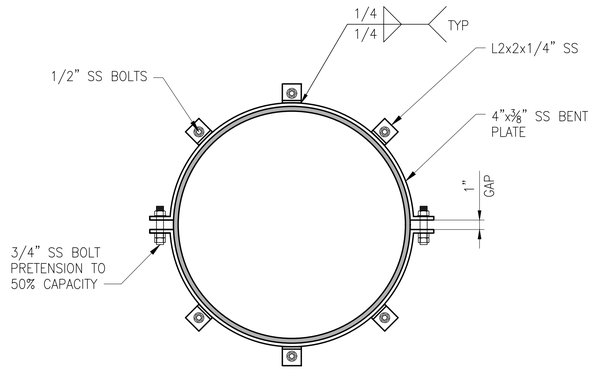



DETAIL – WELD BEAD (2)
SCALE: 6"= 1'-0"

NOTE:
1. MAINTAIN PILE SLEEVE TOP GAPS WITHIN 1/2" OF THE SPECIFIED VALUE. CONTRACTOR SHALL UTILIZE TEMPLATES TO ENSURE TOLERANCES ARE MET.




SECTION – COVER PLATE (B)
SCALE: 3/4"= 1'-0"




DETAIL – PILE CLAMP (3)
SCALE: 1 1/2"= 1'-0"

90% SUBMITTAL
AUGUST 21, 2026
DRAFT

| REVISION | MARK | DATE | DESCRIPTION | APPROVAL |
|---|---|---|---|---|
| B.1 | | 01-11-2026 | 60% SUBMITTAL BCDC EGRB | JMC |

FILE NAME: C:\Users\vmdmo\COWI\A272767 - Berkeley - BWTPF - Documents\07_CAD\A272767-SP-322.dwg LAYOUT NAME: PIER STEEL PIPE DECK CONNECTION PLOTTED: Thursday, July 16, 2026 - 10:43am USER: MDMO

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PROJECT MANAGER: DATE | DEPICTION OF MONUMENTS: DATE | SUBMITTED: DATE | DESIGN JRVS | HORIZ. AS SHOWN | PARKS RECREATION & WATERFRONT 1947 CENTER ST., 5th FR, BERKELEY CA94704 | COWI 555 12th Street, 17th Flr Oakland, CA 94607 Tel.: 510.839.8972 Fax: 510.839.5715 | BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT<br>CITY OF BERKELEY, ALAMEDA COUNTY, CALIFORNIA<br>PIER STEEL PIPE PILE DECK CONNECTION | PLAN |
| | SURVEY PARTY CHIEF | R.C.E. | DRAWN ENSN | VERT. | | | | FILE |
| 0 1 2 3<br>FOR REDUCED PLANS – ORIGINAL SCALE IS IN INCHES | WATERSHED REVIEW: DATE | APPROVED: DATE<br>R.C.E. | CHECK JBSW | BOOK | | | | SP-322 |
| | | CITY ENGINEER EXP. | AS BUILT | DATE 12/16/24 | | | | SHEET 1 OF xx |