COWI

TITLE	BCDC Engineering Criteria Review Board February 25, 2026, Meeting Response	**ADDRESS** COWI 555 12th Street, Suite # 1700, Oakland, CA 94607
TO	Lesley Ewing, Engineer and ECRB Secretary	**PHONE**: 510-839-8972 www.cowi.com
FROM	James Connolly, Project Manager, COWI	**DATE**: April 24, 2026
COPY	Liza McNulty, Capital Improvement Program Manager, Jessica Rivas Deputy Project Engineer (COWI), Harriet Ross, Regulator Director (BCDC), Lindsay Martien Principal Shoreline Development Analyst (BCDC)	**PAGE**: 1/11 **REVISION**: 1 **OUR REFERECE**: A272767

The following are the responses to the questions received via email on March 9th, 2026. We plan to have slides in our presentation for each of these items at the ECRB meeting No. 2 and will be able to answer any follow-up questions.

1. **Datum** - The project datum is Berkeley Mean Lower Low Water (Berkeley MLLW) and people can get easily confused about how to use this correctly. The applicant was advised to carefully check elevations for project heights and dredge depths. Consider preparing a 'cheat table' that shows the difference tidal heights relative to both Berkeley MLLW and also to North American Vertical Datum of 1988 (NAVD88). This will be especially useful in the future when the new tidal epoch is developed or when the North American - Pacific Geopotential Datum of 2022 is introduced, possibly later in 2026.

   No follow-up was requested, but if a 'cheat table' is prepared, please provide a copy to BCDC staff and ECRB members.

   Response: Woolpert (eTrac) performed a topographic and bathymetric survey of the project site. As part of their scope, they prepared a memorandum that provides a description of the vertical datum conversion between Berkeley MLLW and NAVD88. See Attachment No. 1a – Vertical Shift NOAA Berkeley MLLW Datum.

   COWI has carefully checked and confirmed elevations for the project heights and dredge depths. See Attachment No. 1b – email from Woolpert dated 3/19/26.

2. **Codes** - The discuss mentioned several relevant American Society of Civil Engineers (ASCE) codes – ASCE 61-14 (which references ASCE 7-05), ASCE 61-26 (draft), ASCE 7-22, and the California Building Code (CBC) – as well as the 2016 fault risks. As with the Datums, it might be useful to provide a table that compares seismic events for each of these codes and identifies which project components should be designed for which seismic event. As noted during the discussion, codes that are mostly focused on terrestrial development do not always

have obvious marine corollaries. To address this concern, the project design has included elements of conservatism.

Please examine the project components for the CBC design requirements (using ASCE 7-22), note either how the project can meet these requirements, or explain why compliance will not be possible, and what the specific concerns or constraints to compliance are, and what measures will be taken to be as compliant as possible. Likewise, please provide the same examination for those elements that fall under review by ASCE 61-26 (draft), noting either conformance or explaining specific explanations for why compliance is not possible and identifying measures that will be taken to be as compliant as possible.

Response: The project team has incorporated ASCE 7-22 into the design criteria and have added an additional check using the DE response spectrum for ASCE 7-22 checked against life safety in accordance with the CBC. This check applies to all marine structures (inner pier, outer pier, breakwater). The DE has been developed in accordance with ASCE 7-22. Based on our estimated shear wave velocity of 690 feet per second, the site has been classified as Site Class DE, “loose sand or medium stiff clay” in accordance with Chapter 20 of ASCE 7-22. We performed our site-specific seismic hazard analysis (SHA) in accordance with Chapter 21 of ASCE 7-22. The ASCE 7-22 site-specific response spectra at the various damping levels presented in Attachment No. 2 – “ASCE 7-22 Response Spectra Table” A graphical representation of all the response spectra is provided in Attachment No. 3 – “ASCE 7-22 Response Spectra Figure”

The design criteria will continue to use the ASCE 61-14 for the performance based seismic assessment of all marine structures. The ASCE 61-26 (draft) is not released to the public. The document is marked as “*PUBLIC COMMNET DRAFT: DO NOT USE OR CITE*” As such the ASCE 61-26 (draft) cannot be directly referenced in the design criteria. This said, based on preliminary, cursory analysis for the pier, no red flags for conformance with 61-26 were identified, however detailed analysis and modelling of unpublished, draft criteria that is specifically prohibited from use in its current draft form is not within the Project scope.

The following table is the updated seismic event and performance criteria for each project element with the ASCE 7-22 included.

Project Element	ASCE 61-14			ASCE 7-22
	OLE (50% in 50 Yrs)	CLE (10% in 50 Yrs)*	DE (7-05)	DE (7-22)
Inner Pier	Elastic	Minimal	Control and Repairable	Life Safety
Outer Pier	Minimal	Control and Repairable	Life Safety	Life Safety
Breakwater	Minimal	Control and Repairable	Life Safety	Life Safety

* The Breakwater and Outer Pier (CLE) event is classified as a 20% in 50-year event.

3. **<u>Shoreline Stability</u>** - The slope stabilization has focused on the movement of the rock dike since movement of bay mud and alluvium is not expected to have adverse impacts on adjacent piles. The provided seismic slope stability shows a failure plane that terminates at the toe of the rock slope.

   Please provide the other failure planes that were examined and provide evidence that this is the worst-case situation has been identified and included in the project design.

   Response: We extended our slope stability search limits to 60 feet to the waterside of the rock dike toe (approximately three times the slope height) and identified the failure surface with the lowest yield acceleration for failure surfaces in front of the DSM ground improvement. While the analysis identified a failure surface extending beyond the toe of the dike, the critical yield acceleration (ky) remained constant; the failure envelope does not extend to the limits of the search. Consequently, our estimated displacement of 5 to 7 inches during an MCE level event remains unchanged. Our updated slope stability result is presented in Attachment No. 4.

4. **<u>Coastal Analysis</u>** - Wave conditions at the Ferry Terminal area were developed from offshore swell that was propagated into the Bay area, and locally generated waves developed from local wind conditions. The modelling effort was commended, but it was noted that the wave heights used in the coastal analyses seem low.

   1) The FEMA process typically uses a 30-year hindcast record, while the analysis presented used a hindcast record of only 27 years.

   2) The AECOM work for BCDC included 31 years of wave information and had higher waves than those used for Berkeley Pier and Breakwater.

3) Public comments also provided the wave information used in the 1978 U.S. Army Corps of Engineers, Draft Detailed Project Review, Section 107 Small Navigation Project, Berkeley Marina. Alameda County, California.

4) Also, wind data more focused on the Berkeley Pier area is available through a commercial site that the City might investigate.

Please investigate additional available wind and wave data sources and reconsider wave assumptions based on the additional available data. gate additional available wind and wave data sources

Response: 1) We agree that approximately 30 years is generally considered a minimum data period for conducting an extreme wave analysis for the 1% AEP event. However, within San Francisco Bay, wind fields that govern wave generation vary considerably by location. As a result, the representativeness of the wind data is a key factor in the reliability of the analysis. Based on a review of multiple wind stations and sensitivity model analysis, wind data from Alameda, San Francisco, and Richmond were identified as the most representative of wind conditions influencing the Berkeley Ferry Terminal site. Data from other stations are different and would not represent the local wind wave generated waves within the fetch limits for the specific site. The selected stations provided an overlapping 27-year record that coincides with the available offshore wave data, which was also used as input to the numerical model. Between using 31 years of wind data that are not representative of site-specific conditions and 27 years of wind data that reflect the appropriate wind directions and exposure, COWI elected to use the latter. This approach provides more reliable and technically defensible results to use in a detailed design phase. A slightly shorter record with representative wind forcing is preferable to a longer record based on non-representative airport data.

2) COWI reviewed the AECOM San Francisco Bay wave model developed for BCDC. That model is a regional, bay-wide wind-wave model and is not intended for detailed engineering design at the Berkeley site, where wind-generated waves and local wind direction and speed variability are the primary drivers of design conditions. COWI conducted sensitivity analyses and developed a site-specific wave model using wind data from three stations identified as representative of conditions at the Berkeley Ferry Terminal (see details in the Coastal Study Report). Wind stations used in the AECOM analysis are located farther from the site and the data does not correspond to the winds that are potentially generating waves at the site.. As shorelines around San Francisco Bay have different exposures, and wind data is shown to have different directions wind-generated wave analyses for detailed design need to be location-specific, using wind data within the potential fetch limits from the site. For this reason, results from the regional AECOM model are not directly comparable to the site-specific analysis developed for Berkeley Terminal.

3) The referenced 1978 U.S. Army Corps of Engineers Section 107 Draft Detailed Project Review for the Berkeley Marina was prepared to support a small-craft marina project located north of the ferry terminal, within a partially enclosed basin. As such, the wave conditions discussed in that report reflect a setting influenced by marina geometry and local bathymetry, including resonance and wave reflection, and are not representative of conditions at the proposed Berkeley Ferry Terminal and breakwater. The report notes that wave heights of up to approximately 5 ft may have entered the marina from the west-northwest direction. However, the study also highlights limitations related to wind data availability and representativeness. Design wind conditions were primarily derived from the Alameda wind station, with the report acknowledging differences between wind measurements at Alameda and other nearby stations. Those local measurements indicate lower wind speeds for the west-northwest and northwest directions than those adopted for design. The design wave estimated in the 1978 study were based on assumed deep-water wave generation using wind speeds that exceed locally measured values for the dominant west-northwest directions. The report further acknowledges that shallow bathymetry in front of the marina would reduce wave heights as waves propagate shoreward. Even with this adjustment, the methodology relies on simplifying assumptions that were appropriate for the purpose and tools available at the time but differ substantially from the current site-specific numerical modeling approaches used for the Berkeley Ferry Terminal.

The 1978 study applies fetch-limited wave generation methods using simplified representations of wind fields and uniform bathymetry, without resolving spatial variability in wind forcing or wave transformation processes across the Bay. As a result, it does not capture the combined effects of directional sheltering, depth-induced wave transformation, and localized energy dissipation that govern wave conditions at the ferry terminal site. These processes are particularly important in San Francisco Bay, where complex shoreline geometry and variable depths strongly influence wave growth and propagation.

In contrast, the Berkeley Ferry Terminal analysis was developed using a site-specific numerical wave model that accounts for spatially varying, representative wind forcing, water level variability, and detailed local bathymetry. The model explicitly resolves wave generation, propagation, and transformation processes across the domain, including the influence of directional exposure, fetch limitations, and depth-induced breaking. This allows for a more realistic representation of wave conditions at the project site under varying wind and water level scenarios. Water depths at the ferry terminal and breakwater are also generally shallower than those assumed in the marina study and vary with tide, further limiting direct comparison.

For these reasons, the 1978 Berkeley Marina study provides historical context but is not considered an appropriate basis for comparison with the design wave conditions developed for the Berkeley Ferry Terminal. The site-specific modeling approach adopted by COWI allows these factors to be addressed explicitly and provides a more reliable basis for detailed engineering design.

4) Prior to initiating the wave modeling, COWI conducted a regional wind assessment that included comparison of available wind stations and evaluation of their representativeness for the Berkeley Pier area. Following this review, sensitivity analyses were performed for individual wind directions to understand their relative influence on wave generation at the site. From a physical standpoint, wind-generated waves in San Francisco Bay are not necessarily formed directly at the project location. Adequate fetch length is required for waves to develop, and the most influential wind forcing may occur some distance from the site. For this reason, localized point wind measurements at the pier were not used directly as model input. Even if local wind data were considered, the available records do not provide a sufficiently long, continuous time series that overlaps with the offshore wave hindcast data and other regional wind stations to meet minimum requirements for extreme wave analysis. A comparison of available local wind data, along with the associated limitations and implications, is documented in the Coastal Study Report.

5. **Waves** - The waves that were used in the project were noted as being conservative, for example, stacking 1% probability of occurrence swell and 1% probability of exceedance locally generated waves. It was also noted that there was only a 10% friction change applied for the wave transformation across the extensive shallow area.

   Please identify the conservative design assumptions that were used to establish the wave climate for the area and examine them in light of the wave heights identified from the alternative analysis noted above and critical flooding conditions, such as the 500-year event. If other wave heights are higher than what was used, consider a higher wave height in the design criteria, for project height, uplift forces, and runup.

   Response: There are two points in this comment that might reflect a misunderstanding regarding both the application of conservatism in the wave selection and the treatment of bed friction in the model.

   1) Conservatism was not applied by independently stacking sea and swell wave components. Sea and swell were modeled concurrently, and the reported 1% Annual Exceedance Probability (AEP) wave conditions inherently include both components as part of a single combined wave climate. Conservatism instead relates to the method used to establish design conditions. Specifically, the design elevation was determined by combining separately derived 1% AEP estimates for wave conditions and 1% extreme water levels (including storm surge). Because extreme wave conditions and extreme water levels do not generally occur with the same probability, and their statistical dependence was not explicitly quantified, this approach represents a conservative combined-extreme scenario rather than a formal 1% AEP joint-probability event. In effect, this results in a design condition that is more conservative than a typical single-variable 1% AEP estimate. These extremes water levels and wave conditions were used for present and future design criteria.

In standard design practice, present-day conditions are defined by combining the 1% AEP wave condition with the corresponding 1% AEP extreme water level (storm surge), without inclusion of sea level rise. For future scenarios, sea level rise is incorporated alongside the extreme water level, and the corresponding wave condition is reduced relative to the present-day 1% AEP value to maintain a consistent target level of design risk within a probability framework. In practice, maintaining a consistent level of risk under elevated water levels would correspond to a lower associated wave condition when considering same probabilistic risk. However, for this project, the 1% AEP wave condition and the 1% AEP extreme water level (including sea level rise) were retained concurrently, which further increases the level of conservatism in the design conditions.

2) There is a misunderstanding regarding the treatment of bed friction in the wave modeling. A 10% change in friction was not directly applied to the model. Instead, COWI conducted a sensitivity analysis using representative minimum and maximum bed friction coefficients appropriate for a non-calibrated model. The resulting range of wave heights was evaluated, and the analysis showed that wave heights could increase by approximately 10% on average under different friction assumptions. To remain conservative, this increase was subsequently applied to the modeled wave time series. Additional details of this sensitivity analysis are provided in the Coastal Study Report.

Regarding the suggestion to use a 500-year (0.2% AEP) event for design, it is not standard engineering practice to adopt such a high return period for a ferry terminal with a 50-year design life. The selected design criteria were reviewed and approved by the City of Berkeley, with the associated level of risk over the design life considered acceptable and consistent with common practice for similar infrastructure. For completeness, wave-induced uplift forces are not governing for this project, as addressed in a separate response. With respect to wave runup and overtopping, ferry operations would cease under wave conditions well below the 100-year event, and the selected structural materials are resilient to occasional overtopping.

The adopted AEP and sea-level-rise assumptions were applied conservatively across both present-day and future conditions. For future scenarios, sea-level rise was included while retaining a 1% AEP wave condition, which results in a higher level of conservatism than would be required to maintain equivalent risk over the design life. In this context, the use of a 0.2% AEP event would represent a level of conservatism that is not aligned with design practice for this type of facility.

6. **Design Life and Sea Level Rise** - The project established a 50-year life of project and used the intermediate scenario for sea level rise. It was noted in support materials and the presentation that the design is conservative, that the project can be safe for the intermediate high scenario for the 50-year life of project or for a longer life of project with the intermediate scenario. The area that will first experience inundation from sea level rise will be the plaza and landside development.

The applicant was asked to examine using the intermediate high scenario for the project design, including examining the changes to runup, uplift and other flooding aspects that might occur with the higher sea level rise scenario.

Response: The project was established with a 50-year design life and initially referenced the Intermediate sea level rise scenario. As noted in the supporting materials and presentation, the design approach incorporated conservatism, and the structures were evaluated for performance under higher sea level rise scenarios over the project life and beyond (2100). The deck, breakwater, and plaza have already been assessed for the Intermediate-High and High sea level rise scenarios. To provide additional clarity and consistency, COWI is updating the design documentation to remove reliance on the Intermediate scenario and to formally adopt the Intermediate-High scenario as the governing design condition. Revised documents reflecting this update will be provided prior to the next meeting.

7. **Wave Loadings** - The project has not been analyzed for wave loadings since it has been assumed that the seismic loads will dominate the design.

   Using a wave height appropriate for the site (see above discussion), please examine wave loads with both intermediate and intermediate high sea level rise.

   Response: COWI analyzed the project for horizontal wave loadings on both the float and breakwater (refer to "Coastal Study Report" Section 3 - Wave Modeling, page 19 submitted on January 9th, 2026).

   - The horizontal wave loading on the float was 27 kips. Which is less than the controlling 100-kip impact load used to size the piles and other structural elements on the float.
   - The horizontal wave loading was approximately 3 kips per foot of breakwater. Analysis for the breakwater indicates that all structural components remain within allowable limits under the wave load conditions.

   Subsequent to the BCDC ECRB meeting we have performed additional calculations to validate the assumption that the wave loading does not control the design of the pier piles and deck or breakwater pile as summarized below.

   - The horizontal wave force on the pier piles is approximately 5 kips. This is significantly lower than the 120-kip shear force resulting from seismic loads.
   - The calculated maximum wave uplift pressure on the underside of the pier deck and breakwater pile cap does not exceed the self-weight of the slabs, thus does not control the design.

8. **Elevation Constraints** - During the discussion of higher sea level rise scenarios the applicant explained that the elevation of the plaza, slope of the terminal ramps and other project element constraint and limit the terminal ramp length, pier and breakwater elevations. There was also a discussion about what adaptation options might be possible.

   Please provide a write-up of constraints to a higher pier and breakwater and a broad overview of adaptation options that might be possible within the project site, assuming others are addressing offsite constraints

   Response: The elevation of the pier and breakwater are constrained by two elements: 1) Existing elevation of the intersection of University Avenue and Sewall Drive located east of the plaza; 2) Float elevation at low tides and Gangway slope. The following summarizes these constraints.

   1) University Avenue and Seawall Drive intersection are at approximately elevation (EL) +15-feet (City of Berkeley (COB) Datum) east of the Pier / Plaza. The distance between the intersection and the start of the new pier is approximately 80ft. To provide accessible slopes (less than 2%) throughout the plaza the new pier landside elevation is set at EL +16.5-feet (COB Datum). The slope of the plaza with the proposed pier elevation is 1.9%. Raising the landside end of the pier higher will reduce accessibility through the plaza requiring addition of ramps and other feature that reduce the functionality of the plaza.

   2) The 120-ft gangway connects the ferry float to the new pier. Pier and breakwater EL +17.6 feet (COB Datum) is the maximum height the deck can be set and still provides an ADA compliant maximum slope of 1:12 on the gangway. Raising the pier and breakwater higher will result in the gangway not being ADA compliant during low tides. The Pier will transition from EL+17.6 feet over 100-foot length at the shoreside end to match the proposed Plaza elevation of 16.5 feet (COB Datum).

   These constraints, along with a detailed coastal analysis of 100-year storm surge conditions and wind-wave analyses, results in a proposed Pier and breakwater elevation of +17.6 feet (COB Datum), about six feet above the existing pier elevation (11.5 feet). The EL +17.6 feet (COB Datum) elevation is intended to limit overtopping and maintain safe access during all weather conditions in which the ferry is in operation.

   Adaptation options for the new pier and breakwater include changing the handrails on the pier and breakwater to solid elements in the future. The solid barrier along the perimeter of the pier and breakwater would prevent water from reaching the deck. In addition, the pier and breakwater are designed to be resilient. The structures have been purposely designed using concrete which will not be damaged if future wave overtopping occurs.

9. **Structural** - The sleeves that will be used with the most landward piles are important for pier stability and have difficult constraints to provide a protective barrier without creating rigidity. The sleeves and annular material have not yet been designed.

   Please discuss design constraints for the sleeve pile system, provide specs or criteria that will be used for the sleeve material, offset from the pile center, and material characteristics for the sleeve fill.

   Response: Subsequentially from the meeting we have increased the sleeve from 36" to 54". The interstitial space between the pile and 54" diameter steel sleeve shall be left void and free of added materials. A plate will be added to the top to prevent entrance and material from falling in the void. The plate will be welded to the steel pile but not the sleeve to allow for seismic displacement. The gap will be designed to accommodate a calculated seismic displacement of 10" and kinematic movement of 6" see response to question 10. The sleeve will be offset 3.5" from center line of the pile leaving a 10.75" gap on offshore side and a 17.75" gap on the land side.

10. **Cumulative Displacement** - With the inland Deep Soil Mixing (DSM) treatment is being included in the project to reduce the rock dike displacement to about ½ foot for the Maximum Considered Earthquake (MCE) level event and less than about ½ foot for Contingency Level Earthquake (CLE) level event. A CLE level event is likely to reduce the available displacement for any future event.

    Please discuss the concerns related to displacement from multiple events and what responses or modifications might be considered after any major seismic event.

    Response: The displacements presented during our BCDC presentation were characterized as "< 6 inches" to reflect that the analytical and empirical methodologies used provide a relative index of seismic slope performance; predicting exact magnitudes of displacement with these straightforward methods involves inherent uncertainties that preclude higher precision. Our analysis of the displacement of the two earthquake levels is summarized in the following table.

Seismic Event	Kinematic Displacement (inches)	
	NCHRP 611 (2008)	Bray and Macedo (2019)
OLE (72 year)	0	0
CLE (475 year) / DE	0	3

The following summarizes anticipated cumulative movement for two CLE/DE events occurring over the 50-year service life of the pier.

- Cumulative Kinematic Displacement: 0-6 inches
- Seismic Displacement: 10 inches
- Total Displacement: 16 inches < Gap: 17.75 inches: Okay

After a CLE or DE event, the pier will be inspected for damage. During the inspection the sleeve gap can be checked to ensure sufficient clearance remains. If the gap is found to be inadequate, a retrofit may be required. One potential solution involves driving two new piles and sleeves outside the pier that tie into the existing deck. Alternatively, the deck could be partially demolished to remove and replace the sleeve. Note that the probability of having two or more CLE or DE seismic events during the service life of the pier is extremely low.