San Francisco Bay Conservation and Development Commission
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TO: All Engineering Criteria Review Board Members

FROM: Rafael Montes, Senior (Staff) Engineer (415/352-3670; rafael.montes@bcdc.ca.gov)

SUBJECT: Approved Minutes of October 22, 2015 Engineering Criteria Review Board Meeting

1. **Call to Order.** The meeting was called to order by the Chair Dr. Roger Borcherdt at approximately 1:00 p.m., in the Pier 1 Bayside 2 Conference Room, Port of San Francisco, California.

The following Board Members were present: Dr. Roger Borcherdt, Board Chair, Professors Jack Moehle (UC Berkeley) and Mary Comerio (UC Berkeley), Mr. Jim French, G.E., Mr. Frank Rollo, G.E., Mr. William Holmes, S.E., Mr. Bob Battalio, P.E., and Dr. Lou Gilpin, C.E.G., who was present for Item Three. The members of the staff present were Mr. Erik Buehmnan, Permit Analyst, Ms. Jaime Michaels, Principal Analyst, Mr. Bob Batha, Chief of Permits, Mr. John Bowers, Staff Counsel, Mr. Brad McCrea, Regulatory Program Director and Mr. Rafael Montes, Staff Engineer and Board Secretary.

The audience included the following: Mr. James Conolly of COWI, Mr. Sam Yao and Mr. John Sumnicht of SGH, Mr. John Gouchon and Mr. Haze Rogers of Langan Treadwell Rollo, Ms. Ivana Micic of ROMA, Mr. Jim Brady, Mr. Dilip Trivedi, Ms. Azadeh Bozargzadeh and Ms. Ingrid Maloney of Moffatt&Nichol, Mr. Kim von Bluhn and Mr. Steve Reel of the Port of San Francisco and Mr. Mike Gougherty of WETA.

2. **Approval of the Meeting Minutes of August 11, 2015.** Chair Borcherdt solicited comments from the Board members regarding the last meeting minutes of August 11, 2015 with respect to one administrative matters item and the review of two projects: Tesoro Golden Eagle/Avon Refinery MOTEMS-Compliance project and Brooklyn Basin Project (2nd review) in the cities of Martinez and Oakland, California, respectively. The chair had one word correction: replace “not” with “no” on page 13 in the sentence prior to Item 6 (Adjournment). The sentence should have read, “The motion was approved unanimously with no abstentions.” Mr. French made a motion for approval followed by Professor Moehle. The Chair entertained a vote to approve the minutes. They were approved unanimously.

3. **Brooklyn Basin Project.** Mr. Jeff Fippin was present at the meeting to witness any comments from the Board with respect to his material submittal. The submittal was in response to four comments made at the August 11, 2015 review meeting regarding the proposed bulkhead gravity wall at Clinton Basin that included: (1) a check on the kinematic demands over transient dynamic response; (2) a check on the time response period of the site; (3) submittals at a later date, once complete; and (4) of a seismic instrumentation plan and the performance of an analysis check regarding the non-circular slip failure surface and slope stability analysis of the gravity bulkhead with respect to the specific site. The analysis of all the items above should be reported for consensus by the Board at the present meeting.

Prior to any Board deliberations, it was found that information was missing with regard to the slip failure and slope stability analysis, a detailed that went unreported until the time of the meeting. Although a digital version of the analysis had been provided ahead of the meeting to Mr. Montes, its missing was gone undetected when hardcopy submittals were provided for distribution, causing an error of omission.

As a result of this omission error Mr. Fippin did an impromptu presentation of the findings including the analysis checks on the stability of the bulkhead slope at Clinton Basin as requested by the Board in August 2015. During the brief presentation, discussion of the slope stability analysis results ensued between the Board and Mr. Fippin. In summary, he mentioned that the NCHRP methodology remained the same, one that used a displacement-based approach to the analysis of the wall movement. The movement of the gravity wall would be limited to or below six inches -with a factor of safety FS of 1.0. An FS of less than 1 would indicate a greater demand versus the capacity of the system.

Mr. Fippin reiterated that his team performed their slope stability analysis considering a 0.18 pseudo static coefficient, which correlates to approximately 6-inch deformation. According to his explanation, the analysis resulted in a factor of safety greater than 1.0 for both the surface immediately under the bulkhead wall and for the surface that extends into the lower alluvium. He tried to reassure the Board that his team had a very robust solution to ameliorating any soil deficiencies to protect the gravity wall bulkhead in a number of ways including excavating all weaker bay mud in the surroundings and replacing the weak soil with light weight concrete and installing sheet piles, among the many strategies to create the wall.

The Board was hesitant to concur with Mr. Fippin's claims that the upper of the two slip surfaces passed through the controlled low strength material and instead thought that the design of the system had a lower FS than the lower slip surface that passed through the weaker Old Bay Mud. In the end the Board opined that the results of the analysis were inconsistent with the soil strengths as reported and, therefore, it suspected that the FS was less than 1.0. Dr. Borcherdt indicated that although the Board did not reach consensus he suggested the applicant address the Board's concerns about the factor-of-safety issues Dr. Borcherdt moved the motion to the next topic.

4. **Water Emergency Transportation Authority San Francisco Ferry Terminal Expansion**. Mr. Michael Gougherty of WETA introduced the applicant's team that included Ms. Ingrid Maloney, P.E., Mr. Dilip Trivedi, P.E., Mr. Jim Brady, P.E. and Ms. Azadeh Bozorgzadeh, P.E. of Moffatt&Nichol, Mr. James Conolly of COWI, Mr. Sam Yao, S. E. and Mr. John Sumnicht, S.E. of SGH, Mr. Haze Rodgers, G.E. and Mr. John Gouchon, P.E. of Langan, Mr. Kim von Bluhn, P.E. and Mr. Steve Reel, P.E. of the Port of San Francisco, and Ms. Ivana Micic of ROMA.

Mr. Gougherty did an introduction of the project describing a master plan for the San Francisco Ferry Terminal that included two main phases that would add three and two new berths at the South and North basin areas. However, the purpose of this meeting was to review the engineering criteria aspect of the South Basin proposal that would also include a berth promenade and a public access plaza surrounding the water side of the current Agriculture (Ag) Building. The building itself was not part of the project but could be considered for future reconstruction in the future; however, the structure would remain at the site.

Ms. Maloney did a briefing of the existing conditions. She pointed out the site of the historic seawall along the San Francisco promenade. The wall dates back to the early 1900s. Currently, the project was probing the integrity and stability of the structure for any impacts on the proposed facilities. On the other hand, the project would not rely on the seawall for support but rather all the structures would be separated from the new plaza and the berth promenade and connected via sliding joints infill gap. The second constraint is the Ag Building dating back to 1914 that sits on substructure that is quite deteriorated. The project is seeking access for future rehabilitation of that site. She also indicated the creation of seismic joints between the current ferry plaza structure and the new structures. The ferry plaza was seismically retrofitted about 10-15 years ago. A new pedestrian access bridge south of the Ag Building would be built to facilitate access to the new ferry area.

Ms. Maloney described the demolition phase that consisted of removing the current Sinbad's structure including all the buildings, deck and removal of piles. Further, the current South Basin ferry promenade would be partially removed in some areas where piles would be kept while other areas would be completely removed to make way for the new promenade structure. Portions of the old deck and piles between Gate E and the Ferry Plaza would be kept since these were retrofitted in the last 10-15 years and would support the higher elevation of the new promenade deck. The new plaza and promenade would be built to a 14.5 feet NAVD elevation to counter future sea level rise water levels. She indicated that the Ag Building, which is pile-supported has settled over the years to elevations of 8.5-9 feet NAVD. Mr. Battalio commented that since the building is partially supported by the seawall whether it was settling differentially. Ms. Maloney did not have an answer. Mr. Rollo asked whether the grade was being raised via structural or solid fill means and requested to know how the voids left by the pile removal were to be addressed. Ms. Maloney disclosed that the new structures: the plaza and promenade would be on piles. No solid fill would be used for the project. The project would include dredging of the areas of gates F and G to accommodate vessels. The dredged materials would go to Alcatraz or any Dredged Materials Management Office (DMMO) designated areas.

Mr. Trivedi did the presentation on the coastal conditions and explained how the area of the project was protected by a breakwater Pier 14 built in the 1990s, which protected the area from strong fetches from the south; the ferry plaza provides some protection from north fetches and ocean currents. He showed the wind speed conditions as monitored from the island of Alameda station to correlate to wave heights, and it was determined that the west driven winds would be appropriate for design criteria. Further, a spectral wave numerical model for the whole of San Francisco Bay was used to establish conditions, which noted that there would be interference between the floats themselves and deflection off and refraction around the Pier 14 breakwater. With this information the chosen criteria for the project would consist of the 100-year return period design wave equal to 3.4 feet coupled with a 4.6-second period. Finally, in reference to the coastal design criteria he described briefly a motions analysis, waves impacting the project, the wave model depicting the wave reflection off the seawall and the breakwater, the wave attenuation from floats, and the diffraction, refraction, wave transformation, and wave-structure interaction. Mr. Battalio opined that the Board could be interested in knowing the reason and configuration of the Pier 14 breakwater. Mr. Trivedi went on to explain that the south east fetches, which don't happen often, could cause a lot of problems at the shoreline of the site. Therefore, the breakwater was created to protect the basin from the north-east fetches. The structure also accounts for localized sedimentation and reflection off of the structure. As a result, there was a gap left in the design of the breakwater in order to prevent the accumulation of wave energy from impacting the harbor. Mr. Battalio, who'd worked on the design of the Pier 14 breakwater in the 1990s, mentioned that the design took into consideration some of the waves coming from the north. Mr. Trivedi did not have the analysis information regarding the impacts of the north waves but offered it as follow-up. Mr. Battalio indicated he did not have any specific concerns but would be interested in seeing the report nonetheless.

Mr. Trivedi explained now the sea level rise criteria for the project that would raise the structure almost five (5) feet above the current structure and the Embarcadero at 9.5 feet to a target elevation of 14.5 feet NAVD. The revised total water levels (TWL), which combined the effect of tides and waves, as per the Port of San Francisco Building Code, are at 11.4 feet NAVD. Mr. Trivedi and Ms. Maloney indicated the locations of the grade transitions that included two ADA (American with Disabilities Act) ramps and access steps (bleachers) and stairs around the new plaza to get from the Embarcadero to the new structure. Mr. Battalio asked if wave action would be impacting the bottom of the ramps.

Mr. John Gouchon provided the details of the scope of the geotechnical evaluations of the site. These included previous studies by others and by Treadwell & Rollo, results of subsurface explorations, evaluations of potential for lateral spreading and liquefaction, development of vertical capacities (compression and uplift) for new piles, development of lateral soil springs and vertical pile springs for new piles, site-specific response spectra and evaluation of seismic seawall and shoreline stability.

He pointed out and described the location of the soil exploratory sampling. While describing the soil profiles, he noted there were two seawalls: one dating to 1967 used to widen the Embarcadero and an older seawall from 1899 upland of current seawall at the shoreline. The trenches between these two walls were filled with riprap. The current seawall supports part of the Ag Building. The seawall itself is supported by piles. He described the soil strata underneath the mudline and the upland with sand lenses, bay mud, a thin layer of sand and the dense Posey Formation and Old Bay Clay, where the project team proposes to drive the piles to, at an elevation of between -110 feet to below -200 feet and rock estimated at about -245 feet NAVD88. The site was within mapped liquefaction zone; the fill beneath The Embarcadero was comprised of loose to medium dense saturated sand, silty sand, silty clayey sand, sand with gravel and gravel with clay and sand. There was also seawall riprap, which was not liquefiable. He pointed out evidences of liquefaction from the 1906 and the 1989 Loma Prieta earthquakes. So in conclusion, there was potential for liquefaction and liquefaction-induced settlement of several inches behind the seawall. Mr. Rollo inquired about the borings as shown in slide 22 between B-3 and the seawall where there was a gap of information between the two areas; he noted there seemed to be some weak materials as captured by the B-3 and B-4 borings that may reflect into the seawall area and may pose a risk of liquefaction and pile downdrag to the seawall and piles. Since the FLAC analysis had not yet been done, the applicant and the Board concurred that this technical feature of the soil analysis would be important enough to hold a follow-up meeting and discussions.

Mr. Gouchon continued his presentation to describe the proposed pile sizes: designed vertical loads, friction within and below the Bay Mud and end bearing in the Posey Formation. Mr. Rollo asked how the team would rationalize mobilizing all the strength of the friction piles in the Bay Mud while trying to get to the end bearing in the Posey Formation. In other words, the expected friction capacity would never be developed when each pile would go to end bearing in the deeper soils of the Posey Formation. Mr. Gouchon suggested that the piles would instead fail in the Bay Mud before transferring to the end bearing deeper in the mud. Mr. Rollo requested to know whether the piles would be cut or remove. The response—piles would be cut. However, if new and old piles were in conflict of space, attempts would be made to remove them completely and backfill the holes with grout to provide friction on new piles. Mr. Rollo requested clarification on this strategy. His concerns were about the loss of pile strength (lateral, uplift, compression) from the effects of the voids created by the removed piles making the modeling for strength capacity less reliable.

Mr. Gouchon continued his presentation covering pile foundation-vertical capacity, pile foundation p-y curves, lateral pile analyses, vertical springs, site-specific response spectra, probabilistic seismic hazard analysis or PSHA and deterministic analysis, recommended rock spectra, input rock motions, matched time series and shear wave velocity profiles-waterside. At the last slide, discussions ensued regarding the model showing sand layers under the bay mud indicative of a potential liquefaction at the site of the historic rock dike next to the seawall. During the discussions, the team told the Board that the model being used would be able to incorporate liquefiable layers and other soil models and ascertain the conditions for mitigation. As Mr. Gouchon continued to elaborate on the site-specific spectra, Chair Borcherdt requested additional information on liquefaction and site conditions behind the seawall and their potential impacts on the future performance of the access facilities. He noted that it would be beneficial to have similar information of site-specific soil conditions as applied to the areas behind the seawall.

Mr. Rodgers used the rest of the geotechnical presentation to explain the use and preliminary results of the FLAC model. Chair Borcherdt requested an overview of the reasons for doing the analysis. Mr. Rodgers explained that following 1906 Earthquake there was evidence of widespread liquefaction and lateral spreading along the shoreline area. When the observation was made, there was only the inland seawall in place. Prior to the earthquake, the City had embarked on the expansion of The Embarcadero by building the second seawall. Portions of the new seawall had already being constructed when the earthquake struck. Subsequently, the new seawall was backfilled with debris and loose granular material. Now the design team knew the wall is susceptible to liquefaction and ground settlement and because it is relatively tall it had the potential for movement that could pose a real risk.

Further, Mr. Rodgers said that since the new foundation consist of deep piles that the best way to have a high degree of accuracy and confidence in the movement and any impacts of the seawall relative to the new foundations would be to do a soil interaction (FLAC) analysis. The chair asked if the analysis of movement would include the Ag Building as well. Before answering such question directly, Mr. Rodgers added that since the wall was a simple long linear structure, it would be suitable to the two-dimensional soil structure interaction analysis. As a result, a FLAC analysis would be used in the areas of the profile A-A', a cross section drawing that included the Ag Building.

After Mr. Rodger's explanation of the analysis, Chair Borcherdt opined that in light of the "essential facility" classification of the new ferry terminal, it was crucial to keep in mind that the vulnerabilities of the seawall were critical to issues of access to the new structure. Therefore, the stability of the seawall could play a major role in the impacts to the new structure. Compounding the risk, the Ag Building sits partly on the seawall placing additional loads on it. If the seawall were to move to the extent of impacting the stability of the Ag Building, the latter could pose a direct risk to the new structure. Therefore, he asked directly whether the seawall was going to be modeled and analyzed as well. Mr. Rodger explained that the model allows for the analysis of the seawall as an independent structure from the Ag Building and both are allowed to behave differently and interact with each other. The model runs to simulate the displacement at the top and the bottom of the seawall.

Mr. Battalio expressed concerns about the vulnerabilities of the seawall and the impacts of 100-year storms over that seawall. Ms. Maloney replied that the upcoming FLAC analyses would be able to provide the information of the vulnerabilities of the seawall, the Ag Building and the new components. The Board asked whether people would be able to get access to and egress from the new facilities safely during and after an earthquake. Mr. Rollo likened the issue as a conundrum as he referred to similar scenarios involving the life safety earthquake design of fire stations meant to withstand significant larger earthquakes than their particular surrounding neighborhoods susceptible to liquefaction. Therefore, although the stations and equipment could be safe and operational after an earthquake, the fire engines would not be able to go too far over destroyed streets.

Mr. Gougherty explained that WETA had a commitment to make its facilities accessible. The chair turned to Mr. Rodgers to know whether the FLAC analysis would also covered the landside. He responded that the site spectra had been done for the landside, but at this time there was no resolve to do the FLAC analysis on the landside as per the directions of the applicant. Following this response, Mr. Gouchon told the Board that liquefaction was expected behind the seawall on the landside. The chair asks if there were engineering solutions being put forward to deal with the liquefaction potential. Mr. Gougherty repeated the statement that the seawall was not within the scope of the project. Mr. French requested to know the design philosophy for analyzing areas with and without piles such as the access ramps to the plaza and on the south basin and the ferry portals' access to the boat platforms. Mr. Rodgers informed the Board that the limits of the displacement analysis extended to the end of the gangway slope over the ferry loading floats. He further explained that the analysis would cover three rows of the Ag Building piles inland of the seawall and out to the edge of the marginal wharf but reiterated the exclusion of the gangways to the ferry floats. Although the model extended farther than the gangway piles offshore and the MUNI turnaround pile onshore, piles beyond such areas were not covered for analysis. Discussions regarding the scope of the FLAC analysis ensued. Mr. Rodgers summed up the scope of the soil analysis and final recommendations of lateral load for use by the structural engineers.

Next, Mr. Jim Brady made the presentation of the structural design analysis. The structure would consist of a concrete deck and girders, 24-inch, and 36-inch-in-diameter pipe piles with the latter penetrating through the rock dike at seawall. The design approach consisted of weak column/strong beam connectors to protect the integrity and capacity of the deck. Seismic joints allowing +/- 2 feet of movement at interfaces with the seawall, Ag Building and Ferry Plaza would be installed. Mr. Battalio raised again concerns about the Ag Building of impacts on the project and the "L" shape of the main structure that create conflict with the seismic joints. There was discussion about the prospects for the Ag Building and how the project once completed could not only create momentum for the repair of the building but physically provide a platform for moving temporarily the building for the repairs of the substructure and then place back. Similarly, Mr. Gougherty suggested that the WETA project could serve as a potential layout space to facilitate the temporary removal of the Ag Building. He expounded on the idea that some of the features of the project were meant to be of interim nature until the Ag Building provided the final designed access from the ferry portals to the Embarcadero. The South Basin ramp would be one of the temporary structures. This access ramp would be supported by two pier bents running along the south side of the Ag Building. Once the Ag Building was repaired at a later time, the ramp would be removed and a new access along the south side of the Ag Building would be installed.

Mr. Brady continued his presentation of the structural details. The joints separating the new and existing structures would be able to displace up to 2 feet during an earthquake**.** The Ag Building joint would be larger and framed between the building and the new structure. He showed the cross sections of the seawall, the existing ferry plaza and the portion of existing ferry structure (dock and piles) to be retained among the new facilities. He communicated expectations that the earthquake driving movement would be from the west towards the seawall but did not have the final analysis to confirm his comment. If this movement direction was the case, the new structure would move about a foot towards the shore. Mr. Holmes raised questions about whether such displacements were to be permanent in the soil or dynamic in nature. Again, Mr. Brady responded that this view was not yet confirmed. Mr. Battalio asked questions about the deformation of the seawall and how it would react in relation to the new facilities and whether this project was being coordinated with the plans by the Port to secure the seawall from a seismic event. Mr. Steven Reel with the Port replied that FLAC analysis of the seawall was being carried out throughout the San Francisco waterfront and that the Port was eagerly anticipating the analysis as well. A brief mention of concerns regarding the pilings to be driven through the riprap bay ward of the seawall was made, but the inquiry was quickly dismissed by noting that the use of large piles through riprap was standard practice. A case to support this point was the piles driven next to the seawall at the new Exploratorium, not far from this location.

Questions about the gates to be erected around the Ag Building next to the project to limit foot traffic were made. Mr. Gougherty suggested that the area behind the gates in the Ag Building zone would be used by the Port. Discussion regarding the Risk Category IV classification ensued. The Board opined that the area between the Ag Building and the new structure should be noted as something less than Category IV, meaning not designed as an essential facility. Mr. Holmes asked for the reason for not leaving an open hole between the building and the new structure as it would simplify the design. The Board asked to clarify the different areas of risk classification. Ms. Comerio articulated the concerns of the seeming lack of coordination among WETA, the Port, and any other stakeholders to provide safe access especially in light of this being an emergency egress facility. She urged the parties to come to an agreement to devise a plan to get people safely out of the facility during and after an earthquake. Mr. Reel told the Board the Port was cognizant of the issue. Ms. Maloney expressed that the Ag Building was not to be taken as a means of egress during this project. The Board reiterated that this access or egress item as it pertains to the Ag Building and other access means should be stated clearly in the design criteria.

As Mr. Brady continued his presentation, Mr. French suggested that the team reviewed the criteria in relation to the methods of removing the existing piles, whether they would be pulled out of the ground or cut below the mudline. He felt that the removal options could influence the integrity of the new piles in the geotechnical view. The chair had some questions regarding the gangways as they would be critical components to the egress from the facility and urged the team to pay careful attention to the gangway design insofar as taking into account its relative movement and the connecting structures so that it would be undamaged and functional during an emergency. Mr. Brady described the tract system, which would allow for around 10 feet of horizontal movement of the gangways. The chair noted that the design should ensure the gangway end wheels remain within the tracks. Mr. Brady suggested some chains to keep them aligned.

Mr. Brady allowed Mr. John Sumnicht of Gumpertz and Heger to do a briefing on the south basin bridge design. The truss bridge would span about 60 feet and cantilever at each end. The bridge would be supported by two piers consisting of two 36-inch-in-diameter steel piles per pier. Further, the deck would be fixed at the new promenade and would be allowed to slide up to two feet on the seawall end. It would be designed to move independently from the promenade in the event that the lateral displacement of the latter would be out-of-lime with the bridge.

The new plaza and ferry promenade would be supported by 36- and 24-inch-in-diameter steel pipe piles except for the retained section of the ferry promenade that was already supported by concrete piles. The seismic design approach would follow the California Building Code (CBC) guidelines for Essential Facility-Risk Category IV, seismic isolation from existing structure and the seismic codes, including the Port of Long Beach Wharf Criteria. Mr. Brady described the seismic performance objectives that included two events: Life Safety of an approximately 1,000 year return and Immediate Occupancy of 475-year return earthquake. Further, in the same slide, there was a table with the new steel pipe piles component strain for the two events. Professor Moehle asked for the meaning of the strain values for the top of pile hinge concrete strains for the immediate occupancy for the life safety criteria. Mr. Brady explained that the strain values (steel deformation) that were expected by the reinforcement embedded at the top of the steel pile and connected to the concrete pile cap and the deck above. The Board's concern was that the estimated strain values were too high that seem to approach fracture limits. Mr. Brady admitted that the steel strain was high but concrete spalling would be expected where the steel pile connects into the deck and suggested that the two-inch gap between the deck bottom concrete and the top of the piles would ameliorate the condition. Mr. Holmes asked how Mr. Brady rationalize this high strain to be consistent with the project target design of "immediate occupancy." Mr. Brady opined that if this was a concrete pile, this would have extensive damage in cracking; however, he did not expect the same effect on steel piles but rather expected very little damage with the kind of demand displacement and strains. Mr. Holmes responded that there would be damage in the steel at the interface of the top of steel pile. He mentioned that ASCE 41 would not support such expectations.

Supporting the criteria, Ms. Maloney reported that this was in line with the Port of Long Beach Wharf strain criteria for marine oil terminals based on contingency level earthquake (CLE) for immediate occupancy or control damage and operative level earthquakes (OLE) or operations within a short period of time for maintaining operations. She felt that the strain could be higher according to the above criteria, especially when compared to the much higher dead and live loads anticipated by oil cargo facilities that the criteria intended to address. Questions regarding the type of rebar steel reinforcement embedded in the pipe pile and connecting the deck were raised and whether the steel would be brittle, A615, or more elastic such as A706. The latter would be more resilient and last through some fatigue cycles before fracture. Professor Moehle opined that the strain of steel in the concrete were too high for Risk Category IV. He would not even accept those strain values for Risk Category III either as the strains would still be too high. ASCE 41 would not permit more than 0.05 strain ratio for reinforcing steel for ordinary occupancy. The Board would not accept such criteria and would recommend the ratio to be reduced by half. Mr. Brady said they would take it back to the drawing board and thanked the ECRB for its recommendation. The Board opined that at that ratio of 0.06 and 0.08 the structure may have to be rebuilt since it would not be repairable after a strong earthquake and that for the design to fit the immediate occupancy category, the strain ratio would need to be reduced. Finally, the Board opined that such strain criteria were not compatible with the target earthquake events of Immediate Occupancy and Life Safety and whether the CLE and OLE were somewhat related to the Life Safety performance of the project.

Mr. Reel of the Port of San Francisco explained that CLE and OLE referred to the criteria for wharf structures aimed at limited and repairable damage and operational within weeks. The Board suggested instead that the project sought a medium CLE/OLE criterion between the Immediate Occupancy and Life Safety design targets. Settling into the three-prong approach, the criteria should manifest that the Immediate Occupancy target would be too ambitious and that the Life Safety criterion be limited to no-collapse of the deck and safe egress. Further, the Life Safety criterion should state that the deck would be unusable and unrepairable after a strong earthquake. A Board member suggested that such criteria expectations would be more transparent.

Ms. Bozargzadeh covered the last portion of the presentation regarding the seismic analysis methods and response spectrum analysis. The analyses included P-delta effect, soil structure interaction, multi-directional effect on ground motion and torsional plan eccentricity. A non-linear static pushover analysis was done to determine during a seismic event displacement capacity, softening of the structure due to inelastic behavior, overstress (yield) location on the structure, and strain hardening of sections. She showed slides of the maximum displacement (in inches) in two horizontal directions, X and Y for the two earthquake events, design and maximum credible earthquakes ranging from 8.7 to 13.5 inches. Upon explaining the anticipated displacement demands on the structure, she was asked by Prof. Moehle about the estimated plastic (hysteretic) displacement where the steel would be irreversibly deformed. The displacement demand, according to Mr. Bozargzadeh was about 27 inches. Prof. Moehle inquired whether that was reasonable in light of the two-inch gap between the top of the steel pile and deck. He noted that the designed hinge developing at the top of the pile was too big and that with a 27 inches development displacement and a two-inch gap at the top of the pile, the stresses would be too large, resulting in a large impact to the deck.

Ms. Bozargzadeh continued to finish her presentation by illustrating two scenarios of kinematic loadings simulations. The first kinematic loading case consisted of loading occurring on the structure following damage (if any) due to Maximum Credible Earthquakes and Design Earthquakes with no simultaneous inertial response (if hinges due to kinematic load occur close to those from inertial response, then combined response is evaluated in the next case), and loading occurring on the structure following damage due to MCE and DE with simultaneous inertial response equal to 25 percent of the MCE or DE (based on MOTEMS). Prof. Moehle inquired about the timing of lateral spread relative to the maximum ground shaking. Mr. Gouchon opined that lateral spreading would take some time and would be gradual after the earthquake due to the build-up of the soils pore pressures. Mr. Rollo suggested it would be around 15 seconds after the quake before seeing lateral spreading. The Board contended that the inertial damage would occur first at the top; therefore, it would be better to combine kinematic loading with 25 percent of inertial loads at the end of the piles. However, the conundrum was that all loadings would be happening after cracking the top deck-pile connectors when the strain limits (and not-recoverable damages) had already occurred. More discussions regarding the kinematic loading and inertial response ensued making the issue of excessive displacement more relevant by the additional displacement component of lateral spreading. Inquiries about ideas to de-bond the pile rebars into the deck without causing a problem were suggested. However, the Board did not suggest any solution. Instead, it pointed out the issues of the strain values in the criteria.

The Board deliberated on the issues of the project. Mr. Moehle asked Ms. Comerio about her feelings on the raised structure where people standing on the land side of the seawall would not be able to see the Bay. Instead, Mr. McCrea offered a response by saying that the BCDC Design Review Board had looked at the proposal and accepted the proposal. The Board commented on the site response spectra in the geotechnical reports. The chair suggested reports on site response spectra from the SHAKE analysis for evidence of site resonance. Prof. Moehle opined that the seismic joints seemed to have been designed exclusively for horizontal and not vertical movement. He raised the issue of access if the joints were to experience permanent vertical deformations. Mr. Rollo opined that the applicant should come back to the Board with resolutions to the questions raised at the meeting, especially pending analysis that had not been yet reviewed today.

Board's motion: Preamble with the understanding that:

The ECRB review of the engineering criteria for the WETA San Francisco Ferry Terminal Expansion project on October 22, 2015 was conducted when the project was at about the 35 percent design stage. The Applicant's consultants (Langan, Moffat and Nichol and Simpson Gumpertz & Heger) have provided information including:

a. Scope of work detailing the limits of project;

b. Coastal conditions and flood resilience that takes into account sea level rise for the life of the project;

c. Geotechnical evaluation of the project site including:

(1) assessment of liquefaction potential, pile capacity and lateral and vertical pile movement analyses,

(2) site-specific response spectra and recommended spectra,

(3) time-history series to the MCEr rock spectrum,

(4) ground motion analysis results for MCEr and DE, and

(5) dynamic soil structure interaction, modeling and additional evaluations.

d. Structural design approach Including:

(1) seismic joints, seawall/ferry plaza/Ag Building/project layout,

(2) pedestrian bridge at south basin, and

(3) seismic design approach: performance objectives, analysis methods, pile section properties, response spectrum, pushover and kinematic loading analysis.

The following ECRB review comments are based on referenced information.

With the understanding that the project is still at an early design stage, the ECRB suggests that the WETA San Francisco Ferry Terminal Expansion project move forward. Further, the ECRB recommends follow-up discussion of items of several Board members' inquiries and results of analyses not yet discussed that shall be addressed in writing ahead of the next public meeting. Such forthcoming items of discussion are listed in order of specific criteria:

a. Coastal Engineering

(1) Explain whether the design team has compared the coastal design criteria with the original downtown terminal design of the 1990s regarding the similarities in loadings and waves results.

(2) Although the FEMA 100-year return elevation, meant to be provisional, compared well with the URS report as well as the Boston Harbor report, please explain whether there has been a thorough review of the deck elevations in light of potential queries reporting relative small waves.

(3) The project's impacts from wind speeds may have been set based on direction therefore, the Board requests to see a comparison between the 100-year return wind magnitudes applied not solely based on wave height/wave period and segregated by direction but to all directions.

(4) The governing wave/fetch exposure seems to be from the north-east as opposed to from the north where the Pier 14 breakwater would protect the project. Therefore, the Board would like to see the modeling report of the study of wave/fetch exposure that would describe the worst-case wave exposure.

(5) The coastal wind conditions report indicates the design criteria of a 100-year return design wave as being 3.4 feet with a period of 4.6 seconds. However, the table reference, under the slide of the “Coastal Conditions” noting the north wind-direction conditions, seems to indicate that the wave height is higher in correlation with a smaller period; therefore, could it be assumed that the wavelength of the period as opposed to the wave height governs in this region?

(6) Since the criteria involved the use of a dynamic analysis (wave height criteria) when looking at the response of the float, there should be a discussion of the effects and reflection on the guide piles and the interaction between the waves, the float and the guide piles, a naval architecture component, which would inform the structural dynamic analysis.

(7) Explain the effect of the curb at the edge of deck of the ferry promenade and explain the adaptive approach to sea-level rise in the future.

b. Structural Engineering

(1) Evaluate and explain the criteria with regard to damage on the concrete piles on whether there would be moment resistance left after an MCE-type of earthquake. In addition, please explain the results of any kinematic effects on the structure.

(2) Perform a detailed evaluation of the sliding joints with respect to design for horizontal and nominal vertical displacement.

(3) Explain the rationale or purpose of the steel joints between the Ag Building and the new ferry promenade as they are difficult to design. Wouldn’t a pedestrian barrier on the new structure be sufficient?

(4) There were concerns about the occupancy level IV and how people would be able to have access and egress the project site safely. Please explain any contingency emergency plans to safe evacuation?

c. Geotechnical

(1) Provide results of the FLAC analysis.

(2) Provide results of the pore pressure generation in the lower sands.

(3) Did the models used for determining went far enough to the east in ascertaining any potential kinematic loading of the piles at the floats, and whether there is too much boundary effects by cutting the current model up to where it is. Potential movement on top of some of the lower liquefiable and softening sands may be holding it in place at the edge of the model.

(4) Explain how the kinematic loads are applied to the piles. Also, who will calculate the moments at the tops of the piles – the geotech or the structural – and how will this be modeled in FLAC?

(5) With respect to the results of the evaluations of liquefaction and impacts of lateral spreading on the piles and performance of the entire structure, if liquefaction occurs, what effects would it have on pile capacities, i.e. downdrag and what effects would it have on lateral strengths and displacement. In addition, when the assumptions were made for the shear strength for the section of sand in the area, the combinations in the strength parameters (large cohesion plus significant friction angle) seemed pretty high; therefore, please review the parameters again.

(6) Knowing that the structure would be classified an essential facility, the applicant is encouraged to have a strategy for the access to the facility during and immediately following the occurrence strong motion event.

d. Consideration of Seismic Instrumentation. Propose a seismic instrumentation plan appropriate for the project. This shall be coordinated with the California Strong Motion Instrumentation Program run by the California Geological Survey.

5. **Adjournment.** There being no further old or new business, the meeting was adjourned at approximately 5:00 p.m.

Respectfully submitted,

RAFAEL MONTES, P.E.
Board Secretary

Approved, as corrected at the Engineering Criteria
Review Board Meeting of March 30, 2016.