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 February 26, 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 Milton Marks Conference Center—Monterey Room, 455 Golden Gate Avenue, San Francisco, California.

2. **Roll Call.** 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. William Holmes, S.E., Dr. Lou Gilpin, C.E.G. Mr. Frank Rollo, G.E. and Mr. Bob Battalio, P.E., who was only present during the second item of discussion. The members of the staff present were Mr. Erik Buehmann, Permit Analyst, Ms. Rosa Schneider, Permit Analyst, Ms. Heather Perry, Mr. Bob Batha, Chief of Permits, Ms. Jaime Michaels, Permit Analyst, and Mr. Rafael Montes, Staff Engineer and Board Secretary.

Prior to the meeting there were introductions of the audience in the room. In attendance among the several of the applicant representatives were from Moffatt&Nichol, Mr. Christopher Devick, P. E., Mr. Rod Iwashita, P.E., Mr. Carl Schulze, P.E., Mr. Juanito Jamias, P. E., Mr. Jim Brady, P.E. and Mr. Dilip Trivedi, P.E. ; from ENGEO: Mr. Joe Tootle, G.E., Mr. Taylor Hall, Mr. Jeff Fippin, G.E., Mr. Uri Eliahu, G.E.; from the US Army Corps of Engineers: Mr. Michael Onines, Ms. Carolyn Mallory and Mr. Peter Broderick; from Signature Development: Mr. Patrick VanNess and Ms. Deborah Tu; from others: Mr. Malcolm Charles (MOTCO), Mr. Alain Placido (City of Oakland); Dr. Arul Arulmoli, G.E. (Earth Mechanics), Mr. Kevin Treat (KPW).

3. **Approval of the Meeting Minutes of January 22, 2015.** Chair Borcherdt solicited comments from the Board members regarding the last Board's meeting minutes of January 22, 2015 with respect to the review of the Treasure Island Redevelopment project in the City of San Francisco, California. Dr. Lou Gilpin noted some additional wording that were missing from the draft document on page 14, paragraph three in reference to some of his comments about questioning whether there was sufficient undersurface information across the critical access causeway structure and as a result proposed the applicant to do more closely spaced geotechnical

sampling from the rock on Yerba Buena Island to the fill over bay mud along Subphase 1A. Further, Mr. Rollo noted a misquoted name reference where a comment should have been credited to Mr. Bill Rudolph, G.E. with ENGEO, who presented the geotechnical criteria of the Treasure Island project, and not to him. Such misquote was on page 11, fourth paragraph reading, "Mr. Rollo speculated that if the analysis…" The change should read instead, "Mr. Rudolph speculated…" Mr. Rollo commended his Board peers for an excellent job in noting all of the meeting's motion provisions. Mr. Rollo moved the motion to approve the minutes. Mr. Holmes seconded the motion. The Chair entertained the motion and the minutes were approved by a voice vote.

Prior to the beginning of the presentations, Mr. Rollo asked the Chair whether to disregard all the previous unsolicited individual submittals made by the Treasure Island project applicant in an attempt to anticipate the provisions in the motion taken today by the ECRB. Chair Borcherdt agreed with Mr. Rollo's understanding of this action and indicated that the applicant would have now an opportunity to do a formal response based on the approved minutes' provisions. Further, he indicated that there would be no additional discussion of any of those previous responses at this time.

4. **Item Review: Brooklyn Basin project.** Dr. Borcherdt, the Chair of the Board, asked the team leader of the project to introduce himself along with the participants and members of the audience. Mr. Dilip Trivedi introduced Mr. Brady and let Mr. Jeff Fippin (project manager) introduced Mr. Taylor Hall, Mr. Tootle and Mr. Eliahu (president of ENGEO). Further, Mr. Trivedi introduced the developer representative Mr. VanNess and Ms. Tu of Signature Development.

The participants of the second review discussion introduced themselves. Mr. Christopher Devick (project manager) introduced Mr. Iwashita, Mr. Schulze, Mr. Jamias, Mr. Onines, Ms. Mallory and Mr. Broderick, Mr. Charles, Dr. Arulmoli and Mr. Treat.

Prior to the beginning of the review session, Mr. Rollo asked the applicant to state for the record whether it had used Treadwell&Rollo's raw data or rely on calculations and work performed by Treadwell&Rollo for its design parameter recommendations. Mr. Fippin declared that his team had revised the parameters used for the recommendations in the report of the criteria. Further, Moffatt&Nichol stated that it had not relied on the T&R recommendations as well.

Mr. Trivedi described the general area of the project, the development plan, existing conditions and the proposed shoreline treatments. He indicated areas of removal of existing structures along the shoreline such as portions of the 9th Avenue Wharf deck closest to the apron, the existing timber wharf along what would be Shoreline Park. The project would also include installation of riprap and a boardwalk on both sides of Clinton Basin and a new sheet pile bulkhead wall at the land's end of Clinton Basin as a backfill retaining structure to create Gateway Park. He noted that the ECRB had reviewed this project several years ago, and now it had been revised and changed, including new seismic retrofit strategies. Mr. Trivedi introduced Mr. Tootle, who would present the geotechnical side of the project presentation. Mr. Montes noted that each speaker should state his/her name for the public record.

Mr. Tootle described the historical area map of the project dating to 1876, which indicated that the boundaries of the project consisted of areas reclaimed from the historical Bay. No liquefiable soils had been found in the subsurface soils of the waterfront features. His slide of the 9th Avenue wharf land reclamation showed a dike that was placed around the 1920s. The areas behind the dike were filled in with dredged fill from the adjacent channel. He showed slides of the soil explorations, historic and new, soil profiles and design cross sections with respect to the 9th Avenue Wharf. Part of the evaluation was to know the types of soils and of any potential of soft soils underneath the rock dike. The predominant subsurface soils found were fine grains, but the team did not consider the risk of liquefaction significant. It chose to classify the site as Site Class E in reference to the 2013 California Building Code spectra. The risk-based maximum considered earthquake or MCEr estimates were found to be similar to with the geometric mean estimates. The risk-based spectra was used to infer the 2/3 Peak Ground Acceleration design spectra.

Mr. Tootle described the analysis of the slope stability at one cross section of the 9th Avenue Wharf site where the active pressures were on the upslope side of the rock dike and passive pressures were on the downslope. Therefore, the goal was to mitigate by limiting the amount of deformation that could occur on the slope. The next slide would show the proposed dike toe wall that aimed at restraining the movement of the dike during a ground motion event. Mr. Rollo interjected to inquiry about whether the wall would be tied back for lateral support. Mr. Brady replied that the wall would be designed as a cantilever wall, whose daylight would be the midline (completely under the mudline). The depth of the wall would be in the order of 60 feet although the parameters were still being studied. Mr. Fippin elaborated about the acting earth pressures on the wall explaining that the deformations of the wall would be about half a foot or less. Mr. Rollo inquired about the more detailed pressure distribution over the cantilevered wall stating that the Board would need to see that information to provide a thorough assessment of the criteria based on their mandate. However, Mr. Fippin said that this could be provided some time later as the design was still in the process of being optimized. He also tried to explain that the criteria were to limit the deformations on the wall to 2 inches of soil movement so that kinematic loads are nominal. At Clinton Basin where new structures such as a new bulkhead wall and a pile-supported boardwalk on both sides of the channel, the deformation criteria of the soil mass would be 6 inches based on the kinematic loads projected for the types of soils. There was further discussion regarding the design of the wall's movement at six inches and the relation of the piles that would be adjacent to the wall.

Chair Borcherdt asked the applicant representatives to confirm whether the level of design presented today represented 35-percent design criteria. Mr. Trivedi said design has not proceeded to 25-35 percent, but the concept has been selected from all the reviewed alternatives. Based on such response, the Chair ascertained that the project would have to come back to the Board for further review. Mr. Brady noted that although no detail analysis had been done on the selected alternative, the toe wall and bulkhead wall, a fair amount of analysis had already been done for the wharf structures and other portions of the project.

Mr. Tootle continued his presentation of the geotechnical criteria noting that a tie-back type of wall would be recommended for the Bulkhead wall design at Clinton Basin. Finally, he closed his presentation with a description of the proposed seismic motion instrumentation. But first he showed some of the existing ground motion stations in the area as depicted in the Center for Engineering Strong Motion Data (CESMD) website that showed the California Geological Survey (CGS) and the US Geological Survey (USGS) monitoring instruments at the site. It showed quite a bit of free-field instrumentation in the area. His team would propose a service-mounted seismograph on the wharf structure as well as survey monuments both near shore and inland of the project so that information could be gotten from the performance of the wharf structure and be able to monitor any free field that may happen at the site. That was the end of Mr. Tootle's presentation and Mr. Rollo wanted to go back to discussions on the proposed tie-back wall.

Mr. Brady did a presentation on the structural criteria aspect of the project beginning with a description of the existing 9th Avenue Wharf. The slide showed the plan and section views of the historical structure (1930s era) where on the outboard part there were timber pile aprons running along the middle of the structure. In addition, on out outboard side of the structure, there were concrete piles whereas at the very back of the wharf there were green timber piles. Concrete casings of varying depths were used on the concrete and green timber piles. The casings were used as a typical construction method of that era. The wharf measured about 1,200 feet by 225 feet and had a flat slab deck and deepened deck with a railroad girder at the edge of the timber apron. Pictures of an existing timber railroad trestle were shown. This trestle would be retained and repaired as well as the east façade of the shed building and the shed building along one side of the wharf. Only a section of the building shed would be demolished while the rest would be retained. The retrofit concept consisted of two seismic joints across the deck slab on the east and west sides with a 650 feet of separation. Forty-eight-inch steel pile cluster caps would be located at five locations along the alignment of the historic dike. The existing apron deck areas would be removed ranging from 16 to 51-foot sections across the deck. The slides showed the typical sections of the building, middle and west end portions of the wharf. All cross sections drawings showed a proposed sheet toe wall bayward of the historic dike so as to reduce slope movement. Mr. Rollo inquired about the constructability/installation of the sheet pile wall and the effects to the outer piles and how the sheet pile wall to be driven through deck openings. His inquiry was about any measures to be taken to prevent damage and deformation of the outer vertical piles due to the known displacement/deformations. Mr. Tootle expressed that once the toe wall is installed, it would address the outward sliding force of the rock dike and that any failure displacement beyond the toe wall, on the bay side, would be limited to one-half foot of less of lateral movement and only within the accreted soil and the young bay mud. Mr. Rollo asked about the reaction of and stresses on the piles due to the six-inch movement. Mr. Brady said that such pile-versus-stresses analysis would be underway with the recommendations from this meeting. Mr. Rollo pointed out the need to know about this detail due to the team's disclosure that the building over the pier would be retained. Therefore, the displacement of the piles could possibly impact the building over the deck even if the soil movement under it were to displace six inches or less. Mr. Tootle explained that the interpretation of this analysis of displacement was incorrect since the six-inch or less movement would be on the sheet pile, not on the existing piles, which may not move for they were braced at the top by the deck and pinned at the bottom by the soil. Instead a relatively small deflection of the toe sheet pile wall would not translate in to the adjacent piles. However, he said that once the stresses had been calculated and analyzed that it would be presented to the Board for review. Mr. French cautioned about the limitations of the pseudostatic analysis that resulted in the six inches or less of movement, and urged the team to look carefully at most of the forces acting over the toe wall to make the best estimates of potential displacements. Mr. Brady concluded that driving the toe wall in front of the rock dike would reduce significantly its sliding force. Additionally, he mentioned that the deck's loading would not be increased as shown in some of the earlier alternatives presented in the review material, which considered grassy areas that would have required deeper deck sections for soil provisions. The selected alternative did away with the soil in the deck and would include bleachers in most of the deck. Chair Borcherdt inquired whether this extra deck loading was the driving factor for the deletion of the grassy park like alternative. Mr. Brady responded that the loading had its challenges that would have required the installation of additional piles for the design.

Mr. Brady presented now the concept design for the Clinton Basin area, which consisted at the mouth of the basin of a tie-back steel sheet pile bulkhead wall for the retention of fill land mass that as part of the project would become the so-called Gateway Park. The wall would be tie-backed and anchored with a setback "deadman" or weight some distance from the wall. The area between the deadman and the bulkhead wall will be compressed with engineered fill. Further, the deadman itself would be embedded within the engineered fill for anchor support. The wall would be a standard US Army Corps of Engineers sheet pile type wall design. In addition, there would be two boardwalks alongside the Clinton channel consisting of composite deck supported by longitudinal beams and three rows of 24-inch octagonal prestressed concrete piles typically 14 feet apart. Mr. Rollo inquired about the ground profile schematic of the boardwalk section that showed among others an interim final ground, an existing original ground and the ultimate final ground conditions. Mr. Trivedi explained that these ground elevations represented a future marina with some dredging involved; however, the dredging would not be significant if it ever happened, for it was not envisioned and was not part of the project. As for the marina schematics, the applicant was not pursuing its construction at this time. Mr. Rollo understood the situation but cautioned that if modification of the mudline became an eventuality that it should be considered at this stage of the design to preclude changes in the future. Mr. Trivedi agreed that any such potential changes would be included in the analysis.

Mr. Brady explained the seismic demand criteria used throughout the project. Beginning with the 9th Avenue Wharf component, he offered that the design criteria used the 2013 California Building Code for vertical loads and the ASCE 61-14 standard for Seismic Design of Pile-Supported Piers and Wharves for lateral loads. The standard used a displacement-based approach; it would be designed as "Life Safety" and would anticipate a peer review. The ASCE standard was not part of the building code. He felt that the structure would not satisfy a force-based approach. The detailing of the historic structure was not current and as a result does not satisfy the code's force-based approach. There was some discussion regarding what the ASCE 61-14 requirements criteria provided in terms of earthquake levels. Mr. Fippin explained that the peak ground acceleration (PGA) at this site when comparing the results of the 500-year event developed in ASCE 61 versus the building code seismic event the results were about the same. But the CBC's PGA was slightly smaller. The standard used was much closed to the Marine Oil Terminal Engineering Maintenance Standards (MOTEMS), Chapter 31F of the CBC developed for the repair of existing terminal.

He described the shed building structure of the 9th Avenue Wharf as having exterior reinforced concrete walls, steel trusses supporting a wood-framed roof with a lateral force resisting system. The retrofit design standards used would be the 2013 CBC Chapter 34, and the 2010 ASCE 7 "Minimum Design Loads for Buildings and Other Structures" equivalent lateral force procedure.

Regarding the design criteria of the Clinton Basin component of the bulkhead tie-back wall and boardwalks, Mr. Brady said that as the structures are new the criteria would be based on the 2013 California Building Code (IBC) and the ASCE 7-10 standard requirements for Minimum Design Loads for Buildings and Other Structures. Such standard was developed as forced-based approach. Chair Borcherdt asked about the type of structures to be there. Mr. Brady said that in order to satisfy the building code the piles would be 24"-octagonal prestressed concrete, cast-in-place, two-way pile cap system. Professor Moehle inquired about the last bullet of the slide in reference to the performance of a check analysis with displacement-based approach whether it was part of the code and what its applicability was. Mr. Brady said that this displacement-based approach would be done separately to confirm the overall criteria strategy. He opined that this approach was in fact a better analysis than the force-based approach as it predicts more accurately the structure's behavior during a ground motion event.

Lastly, Mr. Trivedi briefed the Board on the Sea Level Rise (SLR) aspect of the project. The constraints of the Clinton Basin side of the project were the limited space for setbacks; therefore, the design would be to build to a 36-inch SLR projection that would take the project beyond the life of the structure. On the other hand, the 9th Ave. terminal was already high enough at four to five feet above estimated SLR projections for its life. Mr. Rollo again inquired about the eventually of future dredging and marina that could affect the SLR estimates at the Clinton Basin. There was some brief discussion as to this inquiry on what it would mean to reshape the shoreline from the original mudline grade to accommodate the anticipated boardwalk.

BOARD DISCUSSION. The Chair began with an inquiry about any known impacts to the proposed areas of the project after the Loma Prieta (LP) earthquake in 1989. Mr. Brady noted that there had been some epoxy repairs on the piles of the wharf although he was not sure whether this was the result of the LP quake. At the same wharf, there may have been some openings created between the piles connections to the deck tied together via reinforcing steel at the top of the piles, which made this area a weak link during a motion event. Further, there were some concrete jackets around the outboard piles that showed deterioration when exposed at MLLW. But Mr. Brady reiterated that there was no conclusion that these damages had been due to Loma Prieta.

He further explained that although many of the historic records of constructions were non-existent, the few available records showed the current wharf structure with a timber apron outside the rock dike and the concrete-jacketed piles, some of them embedded in the rock dike. Chair Borcherdt inquired whether the team felt confident with the level of design criteria addressing all the vulnerabilities of the structure after acknowledging the lack of available information to assess the old structure. Mr. Brady said that the final structure would be significantly relieved from any current loads such as was now with the Port of Oakland's usage of storing shipped goods and the additional load relief of removing other currently permanent features. The Port used the terminal for cotton storage filling up early in and emptying throughout the year; therefore, the live load in the 9th Ave Terminal was likely low at the time of the Loma Prieta EQ in October of 1989. Chair Borcherdt warned that future earthquakes could very likely be more powerful than LP, which could significantly affect this old structure. He further inquired about the potential for liquefaction. Mr. Fippin added that all of the observations from previous ground explorations had not encountered liquefiable- type of soils. Most soils indication alluvium or clay type. Mr. Rollo indicated that previous exploration records indicated the existence of liquefiable soils that could result in as much as 7 inches of liquefaction-induced settlement and several inches of lateral displacement, and that Mr. Fippin likely meant in the area of the subject improvements along the waterfront. Mr. Fippin concurred with Mr. Rollo's clarification and added more soil explorations had been performed specific to the subject improvements and that information indicated soil comprised non-liquefiable soil such as native clay and clay from dredge materials used as fill and accreted materials in front of the rock dike.

Due to the proliferation of soft soils as indicated in the soil reports, Chair Borcherdt inquired whether further soil explorations going much deeper to more competent soils should be done in order to develop a more thorough site specific response analysis in lieu of characterizing the entire project soil site as a Class E site (CBC classification for soft soil sites). Mr. Fippin said that that their explorations had encountered stiff old bay muds and alluvial soil that were primarily clay at significant depths but they had not done a ground motion hazard analysis. The chair reiterated his warning that the site response could be very significant and it could only be to the advantage of the team to do a more thorough estimate of the site response in order to come up with a more precise estimate of the potential loads.

Professor Comerio inquired about the landscaping renderings of the presentation and the lack of definitive information to know what was being proposed in terms of planting and pertinent accommodation within the structures. She also inquired about the height of the buildings as shown in the renderings. Mr. Brady said that there was no more soil or dirt fill for landscaping purposes on the 9th Avenue terminal as seen in some of the previous renderings. Mr. VanNess explained that planting would be done with light-weight fill in the form of planters encased in the deck. The weight would be based on the load limits recommended by the design criteria. He added that all the rows of tree planting shown in the renderings along the shoreline were on firm land and not supported by any overwater structures. He also mentioned that the buildings could be up to 240 feet tall and the overall plane footing could be about 80 feet and pile-supported. Professor Comerio asked her Board colleagues whether there would be any influence issues to the new structures from the piles next to the shoreline. Would the inland building piles have an impact on the shoreline? Mr. Brady interjected to add that the building foundations usually rely of a construction mat with deep foundations with not significant effects to structures beyond the footprints of the buildings.

Mr. Fippin explained that tall building foundations in these situations were more effective with deep pile supports rather than densification of the ground techniques. He said the engineers would design the buildings to bridge any anomalies within the fill. Mr. Rollo wanted to discuss the different loading criteria used for the 9th Ave. Wharf and the Clinton Basin. Why the difference? Mr. Fippin explained the difference in soil profiles between the Wharf and the Clinton Basin based on the recent soils explorations onshore. Therefore, Mr. Rollo asked for the reasons to project those soil findings to the offshore conditions of the proposed structures, these being the focus of this meeting. Mr. Fippin described the process of consolidation of the soils behind the proposed bulkhead wall design and in the Clinton Basin and the process of evaluating soil strength. After further discussion regarding the process of potential means of consolidating the new shoreline, Mr. Rollo requested that the team would have a definitive response on his question for presentation at the next meeting.

Dr. Gilpin requested at this time a summary of the quantities of areas and volumes of Bay fill. Mr. Trivedi suggested no number figures but said that there would be no new marsh habitat development as a result of the US Corps of Engineers case that wrested such project from other agencies. Instead, he said there would be the creation of mud flats at the same elevation of the proposed marsh. He pointed out on the slide regarding Clinton Basin and surroundings indicating the areas of new Bay and those of where Bay was lost with new fill, an approximate total of three acres of Bay would be opened up as a net result. Dr. Gilpin requested final quantities of fill as information for the next meeting. Professor Comerio remarked that the information being presented was not what the Board had been given or at least the information was very difficult to sort out, and that such conveyance of information was unacceptable for a thorough evaluation and assessment of the project. She urged the team to provide updated information going forward as this had been somewhat incoherent for the seeming lack of definition. Mr. Trivedi said that the only information that had changed since the information prepared two weeks ago was minor in relation to the entire project and that the changes were not structural in nature but only pertaining to the BCDC's Design Review Board in relation to public access and changes pertaining to other resource agencies not reviewing the project's engineering aspects.

Mr. Holmes noted that some of the buildings shown in the renderings in the first slide were somewhat out of proportion with the assumption by the team of their small impact on the offshore structures. Mr. Trivedi observed that the building although seeming big next to the shoreline would be set back quite a distance from the street along the shoreline. Mr. Rollo also observed the apparent lack of space at Clinton Basin for the bulkhead wall's installation of a deadman anchor tied to a cable and to such wall to counter the land mass fill loadings, what if the cables snapped or degraded over time making the wall subject to bending or worse. Mr. Trivedi acknowledged the challenge and said they would be prodding for other anchoring system techniques if necessary.

Chair Borcherdt asked that at the next meeting the team present more details of the structure within the BCDC's 100-foot shoreline band jurisdiction to make an assessment of any potential impacts to the overwater structures. Although he deferred to his geotechnical colleagues on the levels of risk soil categories in reference to the building codes, Chair Borcherdt's concerns were about any unforeseen ground surcharges from the building project that may result in a lateral displacement of the shoreline. Mr. Trivedi pointed out that the only sections where building would be closer to the shoreline structures were at the Clinton Basin, and that these structures would be designed as new requiring higher levels of ground motion resistance. As for the wharf terminal, there would be long-distance setbacks between the wharf and the new buildings. Professor Comerio reiterated Chair Borcherdt's point to know more about any anticipated building structures in the vicinity so that the Board were better be able to assess any impacts to the shoreline from a more global aspect.

Mr. Holmes asked questions in reference to the approach of using combined criteria for the of the 9th Ave. building shed of the wharf such as the 2013 CBC Chapter 34 applicable to existing structures and 2010 ASCE 7 on "Minimum Design Loads for Buildings and Other Structures" for equivalent lateral force procedure. Regarding both criteria, his opinion was that ASCE 7 was applicable to new buildings whereas the CBC was not very specific on its use. He thought that the approach would be cumbersome to meet. He suggested instead the use of ASCE 41 as a more appropriate approach to the retrofit of the building, which has a new building's performance level as well as an existing building. It provides for force levels and acceptance criteria. The team noted the suggestion.

Dr. Borcherdt had comments and inquiries regarding the ground motions and asked whether there were efforts to use the Oakland's outer wharf records from LP in getting a feeling for what the site response may look like. Such records had much information about site response. The team replied that it had not although one of its members said that they had available data from USGS of a site in downtown Oakland with PGA information. The Chair suggested a review of the Oakland's Outer wharf record instead. He also had a suggestion regarding instrumentation at the site, which would include sensors in boreholes near the sheet pile walls and instruments located throughout the project site near and on significant structures to measure their response during an earthquake. He recommended this measure as being very valuable to the profession at large, especially since there are no measurements of such structures to date. Mr. Trivedi asked for a better definition of who the end-users of the data would be. Chair Borcherdt replied that the first users would be the scientific and engineering communities as they would learn from the data produced by the instrument to improve engineering design criteria for future structures underlain by bay mud. The chair suggested contacting the California Geological Survey, who may be able to help the team with some ideas as to the best and most efficient seismic instrumentation plan for the project. Mr. Montes offered to provide samples of instrumentation plans and contact information.

Mr. Holmes warned about the global approach to securing the building shed as the controlling factor of shaking would be the wharf, not the walls of the building shed. So he suggested that care should be given to the estimation of the response of the wharf whose displacement would be transmitted to the building. Mr. Treat acknowledged the challenge and said his team was already working in addressing such higher ground accelerations for consideration of the safety criteria of the shed.

The Chair stated that the comments from the Board were intended to be helpful to the applicant. He entertained a motion to move forward with contingencies to guide the applicant in the process of securing a concurrence of the Board concerning the project's engineering criteria. The following are the contingencies brought up at the meeting.

1) The sheet pile walls questions brought up by Mr. Rollo that included a request to know that the team has taken into consideration of the criteria any future dredging that may affect the profile of the bottom of the bay and any repercussion that it may involve with the current design.
2) A request to do a thorough site response analysis.
3) Request for better estimates that would be within the 100-foot shoreline band with respect to the overwater structures reviewed at the meeting.
4) Estimates of the amounts of fill in the Bay.
5) Impacts on the Bay from potential failures of the retaining structures (sheet pile walls).

Mr. French referred to the discussions of the sheet pile wall analysis on Bay mud strength off- and onshore and their transitioning. He asked the team to look carefully at the active and passive strengths (his opinion was that the active looked small) and to look at the dynamic seismic increments. Although the team had looked at Blake's analysis on the pile walls, which is applicable for slopes, he was not sure whether this approach would be directly applicable to walls. He rather suggested Professor Sitar's latest pair reports, one for cohesive and the other for cohesionless cases, as the most suitable approach. He requested in using one or the other to explain the reasons for the selection. Prof. Sitar's may result in bigger pressures than estimated originally. He also reminded the team to look at the time of consolidation brought up during the discussions earlier. If it were 15 feet, it may be drained outwards and figure how long it would be. Mr. Rollo reminded to include the displacement of the existing piles outward of the proposed toe pile walls.

Mr. Rollo asked the Chair that the motion include all the inquiries and issues of concern to be resolved for the next meeting including impact of design at the deadman areas, dynamic site characterization (D, E, F). He added that the Board should not entertain a motion that declared its agreement with the criteria as presented. Mr. Holmes thought that what Mr. Rollo's motion had suggested would not do that. Mr. Rollo reworded his thoughts that the Board did not have enough information to make a motion in favor of the project as it was presented.

The Board spent some time formulating a motion to cover all of its inquiries. After some attempts, the Chair entertained a motion and asked Mr. Rollo to summarize it who added that the Board is generally in agreement with the proposal provided that the issues and inquiries that were put forth and raised were properly addressed before the next presentation. Such issues ranged from selection of parameters, design philosophy and methodology. Chair Borcherdt indicated that it was important that the motion include the various comments and requests brought up by all of the Board members so that these would be documented in the record and so that these suggestions would be useful to the applicant. Mr. Holmes indicated that these points would be in the minutes that would be available. Mr. French seconded the motion. The motion passed unanimously.

5. **Item Review: MOTCO Project**. Mr. Charles, the Director of Public Works at MOTCO began the presentation of the project, which involved the replacement of Pier 2 at the facility. MOTCO was one of two ammunition depots owned by the Department of Defense (DoD), one was in the East Coast in North Carolina and the West Coast was MOTCO. MOTCO, just north of Concord is the only West Coast strategic seaport capable of safely meeting ammunition shipment requirements in support of the Pacific theater. The facility experienced a disaster brought upon by the explosion at Port Chicago, known as the worst military accident in US history. The Port Chicago National Memorial is a monument at the site. As a result of the disaster, safety requirements regarding the handling of explosives were enacted to preclude any such event in the future. Piers 2 – 4 were built after the disaster.

As a result of the disaster history, prior to construction activities on the water side there would be an investigation and clearance of potential munitions and explosives of concern (MEC), which may be remnants of the WWII Port Chicago explosion. The work would consist of demolition of timber portions of Pier 2 that included the old West Trestle and the entire berthing structure of Pier 2, but the project would retain the concrete piles on the East trestle approach along with most of its deck. Only two spans on the East Trestle would be rebuilt to match the elevation of the new Pier 2. The project would construct a new concrete Pier 2 designed for containerized operations with a new West trestle approach and a new Personnel/Forklift Trestle. The East trestle would be repaired to continue serving until replacement was required. When questioned about the reasons for not replacing the East trestle, he replied that the Army does not replace structures that it deemed structurally functional. In 2012, a structural audit/survey revealed that the East trestle was still functional relative to other structures and that would only require minor repairs.

On the land side, the project would demolish building 160 and existing approach ramps; it would repair/raise subsided portions of White Road, which was subject to flooding; it would also construct new pier approach ramps with shoreline protection; and it would construct upgrades to utilities infrastructure, including new 12 kV electrical substation with emergency generators.

There were few questions at this time from the Board about the East trestle conditions, flooding at White Road and location of Building 160. Professor Moehle asked for the type of interpretation of the audit assessment when it declared the East trestle functional and whether it had undergone a seismic evaluation. Mr. Charles said that it was a concrete structure built in 1965 and was relatively not very old, 50 years old. Mr. Onines responded that, as part of this evaluation, their engineers had looked at the structure seismically and found that it did not meet current seismic criteria; therefore, the plan would involve an upgrade of some aspects of the structure but leaving it until replacement was called for when the concrete was closed to no longer being functional. The structure's use would be limited to serve during munition shipments, which did not occur on a daily basis, but only on quarterly year events. As a result, the DoD considered that the chances of a seismic event during active operations would be very rare based on its limited use. Further, Mr. Charles added that the East trestle replacement was not in the 1391 section of federal appropriations from Congress; therefore, it could not be included as another replacement item and as part of this project. Mr. Rollo asked for a description entry/egress operations scenario. Mr. Charles said that access in would be through the east and exit through the west. If the East Trestle were ever disabled during an earthquake, it would have the West trestle structure to serve both accesses.

Chair Borcherdt asked to know more details about the Port Chicago incident. Mr. Charles said that in July 1944 at the height of WWII there were two cargo ships near the shore; one of them had a detonation that took out the adjacent ship. The explosion leveled the town of Port Chicago just south of the shoreline resulting in hundreds of deaths and injuries. Instantly after the accident, the US military had no munition capabilities for the troops in the Pacific scenario. As a result, the Navy almost immediately decided then to rebuild Piers 1 to 4, but this time, it would build them away from the shoreline. Many of the casualties were African Americans. The accident became a historic event part of US military history. The chair asked whether there were some unexploded ordnances in the area. Mr. Charles acknowledged that there could potentially be unexploded munitions in the area that these could be detonated as a result of human intervention. Any detonations would be confined within what he described as an "arc" of influence, which ranged about four miles. So the location of the installations was so far removed from populated sites that it could cause limited damage. This site was very particularly located with no other as suitable sites anywhere else along the West Coast. And this location was so particular due to the relation of the populated areas outside of the arc of influence. Mr. Rollo asked what would happened if this board deemed the site to be inappropriate for such structure whether the DoD would overturn any decisions that may not be in alignment with the project. Mr. Charles said that a simple answer would be yes that the DoD could overturn the Board's decision. Mr. Holmes asked whether the level of design was SUG IV or V, which referred to a level of criticality or risk levels in the military code. Mr. Onines opined that the level design for a seismic event at this site would be a level II. He also said that the criteria were related to how often earthquakes would occur while the US Army was at war of a major Pacific war scenario. Mr. Schulze stated that the level of design was similar to MOTEMS criteria performance-based design with two levels of earthquake (Level 1 and Level 2) parameters.

Dr. Arulmoli presented the geotechnical aspect of the project's criteria. He explained that the applicable code was the federal code UFC 4-152-01, Design for Piers and Wharfs (2012). He described that the MOTEMS (2003) CBC 2013 Chapter 31F and the federal code were by coincidence exactly the same performance criteria. The performance level earthquake would be Level 1 with 50% probability of exceedance in 50 years (72-year return period) and Level 2 with a 10% probability of exceedance in 50 years (475-year return period). The criteria uses site-specific probabilistic seismic hazard analyses (PSHA) and site response analyses (SRA) to develop design acceleration and relative displacement response spectra for Level 1 and Level 2 earthquakes. In addition, the design incorporated new generation attenuation –West 2 models used in the PSHA. He showed a slide with earthquake fault sources relative to the site of the project. He pointed out that Los Medanos/Roe Island fault had been included but that the Concord fault would be the most influential of the faults nearby. He described the historical geotechnical investigation done around the piers dating back to 1966 (Dames & Moore), which included bores in Pier 2 and in 1978 (Woodward Clyde) of further bores at Pier 2 later on. However, these investigations would not be enough to do seismic analyses. EMI did site-specific soil borings in 2014, a total of 13 borings. However, due to the fish windows for work in the water, EMI could not complete all the borings in time and instead rely on the landside borings to generate the soil profiles. The profiles would represent 4 classifications: soil 1A representing soft to medium stiff clayey silt to silty clay, soil 1B representing medium stiff clayey silt to silty clay, soil 2 representing stiff to very stiff elastic silt to fat clay and soil 3 representing dense to very dense silty sand. Further information on the soil boring sampling was presented.

The design findings concluded that liquefaction was not a significant issue and that only few isolated pockets of liquefiable soils on landside could be subject. To determine the pile capacities for axial and lateral movement, the piles would be driven into the dense sand layer and P-y springs models would be developed for soil resistance. Regarding the settlement at the abutment approaches on the landside, he related that there could be up to nine (9) feet of new fill expected and several feet of estimated settlement could occur. Wick drains and surcharge loading would be used to accelerate settlement during construction. Once mitigated, soil settlement would no longer be an issue. Mr. Rollo asked for the estimation of downdrag load. Dr. Arulmoli answered that there would be mitigation for the downdrag before driving piles. He added that the soils surcharge would take about six months. The new fill on the shoreline would be done in coordination with the surcharging of the soils.

Regarding the slope stability, the installation of new piles driven next to existing piles along the slope and outside of the new Pier 2 footprint, would help limit slope deformations to within acceptable limits. Dr. Gilpin inquired how the road and the abutment approach would work for access to the structure in relation to the excessive settlement along the shoreline. Dr. Arulmoli explained that the wick drains would mitigate the settlements and engineered fill would be placed to grade. He reiterated that it would take about six months to reach final settlement of the shoreline and backfill. There was further discussion about this mitigation of the settlement issue at the site. The demolition would cut off the existing piles at the mudline that would serve the purpose of soil support at the slope. The removal of the old structure could be done ahead of the consolidation of the soils in the shoreline. Construction of the new structure would happen only after the soil consolidation process and once final settlement of the ground has been achieved. The Board asked about lateral displacements. Dr. Arulmoli said it would be about 13 to 16 inches without taking into account the leftover piles under the mudline that would help reduce the displacement further, but the analysis of displacement has been done without this additional slope support of the cut-off piles. This was the end of Dr. Arulmoli's presentation.

Mr. Schulze explained the structural aspect of the criteria applied to the main pier. The pier would consist of a cast-in-place concrete deck. There would be a seismic joint separating the trestles to the main pier. The seismic design issues involved were (1) relatively large ground motions, (2) soft soils in the upper layers, (3) relatively heavy deck, (4) long period structure (large displacements), (5) expansion joints and (6) slope stability (kinematic effects). He briefed the Board on the seismic displacement demand model of the piles (depth versus lateral deflection). The demand criteria used Level 1 and Level 2 earthquake responses equivalent to 75- and 475-year return periods, respectively. In addition, the seismic demand model included equivalent depth to fixity, response spectrum analysis, crane tipping loads and displacement demand. The seismic capacity model included pushover analysis, demand displacements of Level 1 and Level 2 earthquake similar to MOTEMS specified criteria. With the pushover model, the team measured displacements of the piles at the two level earthquakes including an analysis with the mounted crane on the deck. Professor Moehle asked whether there had been any efforts to do a more conservative analysis of displacement beyond at 500-year return earthquake response due to the risk significance of the structure. Mr. Charles commented again on the financial constraints (1391) put on the project by the federal mandates. The cost estimates from its inception five years ago were now significantly higher so any other requirements would not be feasible to accommodate due to the limited purse resources. Professor Moehle responded that sometimes the little extra analysis would not add significant resources and could be very beneficial to the project. He opined that the current analysis of the displacement was almost there and that slightly more conservative criteria would not be much more expensive relative to the total cost of the project. There was further discussion as to the merits of analyses of higher level earthquakes relative to the structure.

Mr. Onines opined that designing a more resilient structure (< 500-year return) would not provide additional benefits in the global sense since even though the structure may survive the stronger ground motions, all the surrounding infrastructure, rail lines, utilities, local highway, other feed lines and so forth would be offline rending a more quake-resilient MOTCO facility inoperable in the end. There was some further discussion on this topic. Mr. Schulze ended his presentation with a description of the combined kinematic plus 25% inertial loads (as per MOTEMS) or the dynamic analysis model.

Mr. Jamias did the presentation of the trestles. The East Trestle was a reinforced concrete structure with prestressed piles precast and cast-in-place pile caps and deck. It was designed based on NAVDOCKS DM-25 (US Navy design criteria) for Waterfront Operation Facilities in April 1965. The design was based on seismic lateral load of 6.7%g. it was designed to take the vertical loads of an HS-20 truck, 800 psf and 120-ton locomotive. Its construction dates to 1967-1968. It was inspected in 2012 and rated in good condition. Although the structure was very strong, it needs to be strengthened in order to increase vertical loads for carrying capacity to support bomb carts and reach stacker equipment loads. The strengthening would happen at the deck with fibre-reinforced polymers or FRP to the underside and to the sides of the precast deck beams. Because there would be a grade differential between the new Pier 2 and the existing East Trestle, two spans most adjacent to the pier would be demolished to build a transition ramp to meet the new structure. However, all the existing piles including the ones under the new spans would remain. Only the bent camps and the deck would be new for the two replaced spans. The new transition ramp would go from an existing +11.5 feet to meet the new main platform deck elevation at +13.5 feet MLLW.

He explained further the seismic performance of the East Trestle. The design criteria used in the East Trestle were already out-of-date, and the team found that the structure did not meet performance requirements in current design standards. Therefore, the estimated displacement of the structure as is was unacceptable. The goal would be not to strengthen the structure for seismic loading but just to be able to carry more live load. In addition, the structure would not be SLR compliant even if seismic retrofit were done. The structure would be replaced with a new structure at the end of its useful life.

Professor Comerio wanted to know how the two independent structures of the Pier 2 and the East Trestle would move with respect to one another. Mr. Jamias said that both structures would be joined by a seismic joint that would allow them to move as separate structures. Mr. Holmes noted that the joint's width separating both structures would amount to a couple of feet making the joint design challenging due to the great spacing especially in light of the train loadings. Mr. Charles revealed that although the original plan several years ago would have involved rail transportation over the structure, the revised project includes no such means but truck loading. So the two-foot gap being bridged by the seismic joint plate would not be an issue as far as loadings. Professor Moehle asked whether new pier would not be anticipated or expected to settle as all new structures do. Mr. Jamias said that the settlements would be not significant. This was the end of Mr. Jamias' presentation.

Mr. Devick would do the last presentation of the project regarding the sea level rise analysis. The expected life of the project would be 50 years. The tide station reference used was located at Port Chicago. The projected flood elevations for 1-year to 100-year return for Still Water Levels (SWL) ranged from 6.7 to 8.5 feet MLLW. The wind wave heights for the same year return ranged from 1.8 to 2.3 feet. The SLR projections used for the project were based on the latest US Army Corps SLR projections and the COCAT (Coastal and Ocean Working Group of the California Climate Change Action Team) SLR relative to year 1992. The latter SLR information used indicated a low to high range of 9.7 to 39.7 inches between years 1992 and 2068. His next slide showed the typical cross section of the deck for the new pier indicating a top of deck elevation of +13.50 feet MLLW. Accounting for SLR projections of 36 inches for the life of the project plus 100-year flood events the deck of the main pier would still have 24 inches of freeboard. The utilities under the centerline of the deck at the crane railing location would be just above flood levels by the end of the structure's life, but the team had addressed any potential future issues of inundation of the piping with the use of seawater resilient materials for such components.

The storm drains were features that could be impacted in the future because they would be low in elevation; however, those could easily be modified later on. Overall, the deck was designed not to be inundated over the life of the structure. The 36-inch SLR high estimate, including two feet of freeboard, was chosen for the pier deck since this component could not be easily raised in the future or what the team considered to be a low adaptive capacity of the deck. The moorings and crane lift height were instead designed to be resilient to 22 inches of SLR (medium estimate) since these components could more easily be adapted in the future (medium to high adaptive capacity components). The storm drain and crane operations could be modified to take any impacts of flooding in the future.

Mr. Devick indicated that minor shoreline improvement and repairs would be done on White Road onshore that would raise the road to make it resilient to future flooding. Improvements of the revetments at the abutment areas of the West and Forklift trestles would be included again to protect these areas from erosion and flooding. This was the end of the presentations.

Mr. Bob Battalio inquired about the causes of flooding on the White Road along the pier and whether the road would be raised. Mr. Devick said the flooding was due now to storm and high water levels and that the road would be raised as a result of the problem. Mr. Battalio asked whether the project would be designed to be consistent with the BCDC policies on Climate Change, namely, that it be designed to be flood resilient to mid-century levels, and if expected to be there past year 2050 that the project be designed with an adaptive management plan for end-of-century levels addressing flooding. He also raised issues of life safety in relation to flooding. Mr. Devick said that the structures were designed at height levels that take into account high estimate scenarios of SLR projections. Mr. Battalio asked if waves in relation to deck uplift had been taken into consideration. Mr. Devick said these have not and that such questions of deck uplift could be taken up by the structure engineer.

At this time, Mr. Montes made reference to the policy on the safety of fills section of the BCDC plan that addresses issues of SLR. Specifically, Policy No. 4 relates to the provisions available to the project proponents to address SLR for the time life of the projects. Two of the provisions that could be more related to this project were that the bottom of the structure be designed to be above SLR projections plus 100-year storm events for the life of the project and the second one would be that the project be designed to be resilient to periodic flooding for its expected life. Mr. Devick had already informed the audience that the deck elevation relative to present day water level estimates would be about 60 inches above current water levels. For the 50 year life of the project and a prediction of a high estimate value of almost 40 inches, the pier would be well adapted to be above water. As for the year 2100 projections of 66.5 inches of the COCAT SLR table, the estimates were already on the high side; therefore, the structure could very well be able to be fine even at the end-of-century levels. Mr. Battalio asked about the implications of the uplift on the deck's bottom from waves, and whether there would be any implications to the adjacent marsh from deck overtopping and residue spilling into the Bay, and Mr. Montes added what would happen after the expected life at year 2068, decades before the end of century. Mr. Charles said that the decisions would be made at that time and that the DoD did not see any life safety or environmental implications in the future. Mr. Trivedi explained that the structure would be fine during its design life and that the structure would be there after its design life and operations could just continue indefinitely. Mr. Rollo gave the example of the San Francisco Waterfront, which continues to operate in spite of well exceeding its design life.

Mr. Holmes asked whether the supporting beams would be made of foam cells as these may cause buoyancy of the superstructure during rising waters. Mr. Trivedi suggested that all the design detailing would take into account uplift of the deck and issues of potential superstructure buoyancy by detailing the anchoring of the piles to the deck. There was further discussion over the adaptability of the project after year 2068. Mr. Onines explained that the DoD would continue to monitor the facility after its life. The Department would make a decision to continue operations, replace it or remove it.

Chair Borcherdt acknowledged that this project fell under the purview of the US Army Corps of Engineers. Therefore, he had a few questions regarding the project and the ground response at the site. One of them was regarding the relation between the predominant period of the structure as presented earlier to be near 1.7 seconds and the predominant period of the ground response. He indicated that the ground response analysis based on the soil profile only to a depth of 70 feet might not be adequate to account for all of the resonant amplification at the site. He encouraged the applicant to derive an estimate of the site response based on a complete SWV profile down to rock below the depth of 70 feet to ensure that all predominant periods indicating resonant amplifications had been identified. He was concerned that if resonant amplifications of the soil profile were near those of the loading structure that the engineering criteria for the structure might need to be strengthened..

Dr. Borcherdt also indicated that the report provide a probabilistic seismic hazard analysis (PSHA) for a shear wave velocity of 350 meters per second, which was relatively stiff compared to other SWV in the materials indicated in one of the figures as 160 feet per second. He asked whether there had been an error in the conversion of the data figures.

Another comment by the Chair regarding the EMI calculations in the report involved the resolutions of the periods in Figures 4 and 5 of the ground motion report. He indicated that the sparce resolution of periods between 1 and 2 seconds might be insufficient to identify predominant periods associated with resonant amplifications in this period range. He suggested development of spectral estimates at more closely spaced intervals in period. Dr. Arumoli responded that the 350 meter per second SWV related to the firm ground at 70 feet below ground surface and further that the material was extremely dense. He accepted to take that motion and put it at that level much below 70 feet to come up with an estimate of site response at the site. Further detail discussion of this item ensued with differing opinions on the methodology of performing site response analyses. Dr. Arumoli reiterated his statement to the Board that his team would look into its suggestions and extend the analysis to deeper sections of soil.

Chair Borcherdt brought up the suggestion for the project to take this opportunity to install seismic instrumentation at the site. He indicated that the US Geological Survey already had in place cooperative instrumentation agreements with USACE regarding instrumentation for dams. He thought that the USACE would be amenable to having some instrumentation in other structures to document to ground motions at the site. He also requested to know of any impacts from incidents similar to the Port Chicago Disaster would have on the rest of the San Francisco Bay. Mr. Charles asked the Chair to clarify his request/question. The Chair suggested that an early warning system (EWS) or other similar technologies could be installed to help mitigate a disaster scenario similar to the Port Chicago's and asked the USACE whether this system would be helpful to the agency.

As a way to explain the uses of the EWS, he drew a hypothetical example of a truck loaded with munitions on the way to the loading platform that could be deterred from driving on to the dock area upon being alerted of a P-wave (original ground motion from seismic event) coming in the direction of the site. Dr. Arulmoli exclaimed that the San Andreas Fault was 18 miles away giving 10-20 seconds between seismic waves, P and S, which may not be enough to do anything. The Chair asked to look at it from the perspective of the long-period response of the structures such as the loading platforms and the cranes and the possibility of longer period site response in the range of 1-2 seconds, which could give rise to damaging levels of ground motions at the site. One example was that of Mexico City where the earthquake ground motions that originated 400 Km away from the City gave rise to large amounts of damage at many of the sites which had large resonant soil amplifications with predominant periods near 2-2.5 seconds. In this case an EWS could have provided more than 30 seconds of warning and been helpful in taking some steps to mitigate the pending effects of strong shaking. During this discussion, Mr. Holmes asked how the instrumentation policy applies to this applicant that happens to be the federal government. Mr. Montes opined that the applicant won't be any different from others in complying with State law and BCDC policies. Mr. Malcolm asked whether his agency would be required to pay for the seismic instrumentation arrangements. Chair Borcherdt encouraged the applicant to look into it. He also said the system would not be as expensive as it may be perceived and yet the benefits would outweigh the relatively minor expense. The second comment was regarding the kind of monitoring capability the agency would like to have of the instruments. He suggested as useful to have a few motion recorders at the site when ground motion events happen to record the response of the loading structures. Knowing that the Corps had an interest with regards to dams, it was suggested that this instrumentation could be of benefit to the Corps in a similar fashion. Mr. Onines accepted the suggestion but tried to differentiate between the civil aspects of the agency from a military function of the DoD. He did not remember knowing of any military structures with an array of seismic instruments as described.

Professor Comerio asked Mr. Montes whether there was a policy regarding the installation of instrumentation on new major fills or not. He answered in the affirmative and gave as reference the BCDC Policy No. 3 under the Safety of Fills section of the BCDC Bay Plan. Mr. Malcolm reminded the Board of the financial constraints of Federal Code 1391, the Congress appropriation of funding. He suggested that there could not be a second project to do what was required now.

As to whether this would be a project that the US Army would consider, he was not ready to provide an answer. Mr. Rollo responded that it was part of BCDC policy that could make it contingent of its approval of the overall project. He also indicated that it would be contingent of the Board's approval to at least provide some instrumentation at the new trestle. To assuage the financial fears of this endeavor, Chair Borcherdt suggested very minor costs of purchasing the instruments where maintenance costs could be possibly worked out with the CGS or the USGS. Mr. Malcom said he would look at the suggestion but would do so within the constraints of the available funding. Chair Borcherdt reiterated the policy that requires seismic instrumentation on new fill projects and indicated that this was a fill project because the DoD was expanding the trestles over water.

Mr. Holmes added that there were other issues that needed to be addressed similar to the seismic instrumentation such as the design level assigned to the project. He believed that this project warranted at the minimum a risk level III category for structures that were deemed significant and essential. He quoted Level III's definition such that "buildings and other structures that represent substantial hazards to human life that represent significant economic loss in the amount of failure during an earthquake. " He surmised that even though the DoD or the applicant had a policy for the structures to be deemed Level II, by reading the description of the engineering findings you could make the professional judgment to categorize the project as a Level III if not a Level IV, which would influence the criteria for more stringent requirements of safety. He asked the applicant that in order to satisfy the Board, it would have to make some statement to clarify/justify its chosen risk level or for precluding higher levels of risk. Mr. Malcolm reiterated his position not to interpret his words in the negative, but that the applicant would look again into the suggestions. Mr. Holmes recommended that at the least the team should provide responses to comments made by the Board. However, Mr. Malcolm expressed that he could not commit to anything. Dr. Gilpin noted that the project included removing remnant munitions from the Bay. Mr. Malcolm said that it was a potentiality. Dr. Gilpin thought that such potentiality added a higher level of risk just as discussed. There was some discussion about the potentiality of encountering the remnant munitions during a survey to be carried out prior to the beginning of construction activities.

MOTION. Mr. Holmes formulated the motion by first thanking the applicant for the presentation and follow-up discussions but adding that the Board was not ready to approve the criteria. Mr. Rollo asked Mr. Holmes whether he would consider a motion that spelled out the Board's questions, which the applicant could respond to without necessarily indicating that a second meeting was requested. He suggested this measure to resolve the questions from the Board. Mr. Holmes accepted that initiative. Members of the Board gave their recommendations that were included in Mr. Holmes' motion. The chair included contingency items regarding a recommendation that an early warning system and seismic instrumentation appropriate for the site be considered. He indicated that an instrumentation plan be developed in coordination with CGS to record the response of the new structures and the ground response on land as a reference site. He also indicated items regarding the discussion of the site ground response as stated in the statement below. Mr. French requested the provision of the evaluations of seismic consideration of the performance of the piles and ground motions under the risk-targeted maximum considered earthquake or MCE_R shaking level (collapse-prevention goal) as summarized in the statement below. Mr. Holmes requested a justification for not asserting a target building performance level higher than proposed. Similarly, he requested a consideration of the structural design criteria classification as stated in the statement below. Mr. Battalio made recommendations regarding sea-level rise as summarized below. Mr. Rollo seconded the motion. The chair entertained the following recommendations of the ECRB for the applicant of the MOTCO project as a motion. The motion was approved unanimously with no abstentions.

With the understanding that:

1) MOTCO facility is an important military munitions shipment terminal,
2) The MOTCO project needs to comply with criteria of the US Army Corps of Engineers and the SF BCDC, as well as other relevant MOTEMS and Codes, and
3) The MOTCO project is at the 35% design stage,

Unanimous recommendations of the ECRB as derived from a review of the project in a public meeting of February 26, 2015 are as follows:

Item No. I. The ECRB requests that the applicant provide written responses to the following recommendations and requests for information;

Item No. 2. Approval of the applicant written responses by the ECRB will imply that an additional public review of the project by the ECRB will not be necessary;

Item No. 3. Considering that the natural periods of the shipment platform, underlying soil deposits, and possibly the loading cranes are relatively long between 1.5 and 2 seconds and that large earthquakes at some distance can generate potentially damaging ground motions in this period range, the applicant is encouraged to consider linking its control displays into an Earthquake Early Warning System provided by integrated Seismic Networks in California as operated by the University of California, Cal Tech, and the USGS. This capability could allow advance warnings of pending damaging shaking of a few seconds to more than a 30 seconds, allowing shut down of some potentially hazardous loading operations;

Item No. 4. To be in compliance with the SF Bay Plan policy number 3 on the Safety of Fills, the ECRB encourages the applicant to develop a plan in conjunction with the Strong-Motion Instrumentation Program of the California Geological Survey to record earthquake induced shaking at locations on the new loading platforms and on land as a reference. This recommendation is consistent with efforts of the US ACOE concerning required strong motion instrumentation on dams for purposes of dam safety;

Item No. 5. The applicant is encouraged to thoroughly evaluate and justify criteria chosen for structural design, whether it be from MOTEMS or CBC;

Item No. 6. Considering the considerable thickness of soft and stiff soil deposits beneath the loading platforms and their ability to amplify shaking near the resonant periods of the soil profile and the loading platforms, the applicant is encouraged to evaluate the potential interaction of these resonant periods in more detail. In particular, the applicant is encouraged to develop spectral transfer functions for detailed shear velocity profiles inferred to bedrock for comparison with those inferred for the platforms, and for comparison with the site response spectra inferred with increased resolution in the period band (0.5-2.5 secs) from the 7 sets of selected time histories;

Item No. 7. Provide evaluations of seismic considerations of performance of the piles and ground motions under risk-targeted maximum considered earthquake or MCER shaking level (collapse-prevention goal) and justify reasons for not asserting a target building performance level of at least Life Safety (3-C) of ASCE 41 as prescribed in CBC Section 3401; and

Item No. 8. In regard to sea level rise and relevant BCDC policies, the applicant is encouraged to:

a) Provide information on how the West Trestle's materials are resilient, if inundated, as required by BCDC policies on the Safety of Fills, Policy No. 4 requiring that structures on fill be designed for resilience to flooding (100-year return storms plus sea-level rise projections) for the life of the project and Climate Change policies.

b) Provide information on the life safety implications of potential flooding defense failures and the risk to existing habitat if the new west Trestle and existing East Trestle are regularly inundated, as required BCDC policies on Climate Change Policy No. 2 and 3.

c) Provide information regarding conformation of the project design with BCDC policy No. 3 that indicates if a structure is to remain in place longer than mid-century, an adaptive management plan be developed to address long term impacts based on the projected water levels of the risk assessment?

d) Provide information on any adaptive measures beyond life of the structure, such as proposed alternatives to continue operations, decommissioning, etc. after year 2068.

e) Provide information on whether the deck has capacity to withstand uplift from wave loads once water levels have reached the soffit or beyond? Indicate whether the design has taken into consideration such forces? If so, please explain.

f) Indicate whether the design complies with US Army Corps of Engineers guidance for SLR design?

g) M&N February 09, 2015 Memo, page 6 paragraph 3, reads, *"The deck design includes a one-foot tall curb around the perimeter that has a top elevation of +14.5 feet MLLW for the main platform, west trestle and forklift trestle and 12.5 feet MLLW for the east trestle. For an event such as a 5-year still water level coinciding with a 50-year wind* *wave, assuming 60% of the wave height is above the still water level, a total water level 8.6 feet MLLW would result. This elevation for all SLR scenarios would not result in overtopping of the deck over the service life of the project. Some splash on to the deck would be anticipated for the extreme simultaneous high water level and wave heights."* This information is presumably selected to represent approximately a 50-year event, presumably selected to correspond to the 50-year life of the project. Wouldn't it be more appropriate to model the design for a 100-year event scenario?

h) What would the expected wave and wave crest elevation be during a 100-year water level?

i) Does the structural design accommodate these and other appropriate design wave loads, indicating that the impacts of SLR and storm conditions are limited to operational interruptions and damages below the threshold for life safety structural failure?

6. **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, with no corrections at the
Engineering Criteria Review Board Meeting of May 28, 2015.