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 January 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 1:30 p.m., in the Milton Marks Conference Center—San Diego Room, 455 Golden Gate Avenue, San Francisco, California.

2. **Roll Call.** The following Board Members were present: Dr. Roger Borcherdt, Board Chairman introduced the new active members of the Board, Professors Jack Moehle (UC Berkeley) and Martin Fischer (Stanford University). The other members of the Board included Mr. Bob Battalio, P.E., Mr. Jim French, G.E., Mr. William Holmes, S.E., Dr. Lou Gilpin, C.E.G. Mr. Frank Rollo, G.E. The members of the staff present were Mr. John Bowers, Staff Counsel and Mr. Larry Goldzband, BCDC E.D., both remained temporarily in the meeting, Ms. Ming Yeung, Permit Analyst, Mr. Bob Batha, Chief of Permits, 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 Mr. Brad Porter, P.E., Mr. Jim Brady, P.E. and Mr. Dilip Trivedi, P.E. of Moffatt & Nichol (M&N), Mr. Stefanos Papadopulos, G.E. (ENGEO Project Manager), Mr. Joe Tootle, G.E. (ENGEO Principal-in-charge), Pedro Espinoza (ENGEO Lead Engineer), Mr. Bill Rudolph G.E. (ENGEO Principal-Technical Aspects), and Mr. Uri Eliahu, President of ENGEO, Mr. James Suh (Treasure Island Community Development), Mr. Robert Beck (Treasure Island Development Authority or TIDA).

3. **Approval of the Meeting Summary of June 10, 2014.** Chair Borcherdt solicited comments from the Board members regarding the last Board's meeting minutes of June 10, 2014 with respect to the review of the Tesoro Avon Refinery MOTEMS-compliance project in the unincorporated section of Martinez, California. For the record, Mr. Rollo pointed out his abstention from a vote of the minutes due to his recusal from participation at that meeting. There were no further comments. Mr. Bill Holmes made the motion to approve the minutes; Mr. French seconded the motion. The Chair entertained the motion and the minutes were approved by a voice vote.

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4. **Item Review of the Treasure Island (TI) Redevelopment project:** Mr. Brad Porter outlined the agenda, which included the Project Overview, the Ferry Terminal component, Sea Level Rise (Mr. Trivedi), Soil Conditions (Mr. Rudolph) and Questions. He then introduced Mr. Beck of TIDA, who presented the project.

Overview: the redevelopment project was a former Naval station. In 2011, TIDA completed the environmental work on the project to develop a program of 8,000 new residential home on the island, about half-a-million square feet of commercial and retail space and 300 acres of open space.

Treasure Island was centrally located between the two spans (East and West) of the Bay Bridge in the central Bay. The project would include one major phase on Yerba Buena Island and four major phases beginning the northern half of YBI and southwestern quadrant of TI and progressing eastward and northward with the final phase being the northwestern corner of TI and the open spaces at the northern section of TI. The first major phase would involve the YBI development and the southwestern shore of TI. The timing of the phases would begin as early as 2015 to 2034. The work on TI would require significant amount of geotechnical work.

The Ferry Terminal would consist of the following major components, two breakwaters, a passenger float, fixed structure and a shelter structure. The north breakwater, although not designed at this time, it would be conceptually a vertical panel with panel piles support. The north breakwater would be about 900 feet long whereas the south breakwater would be around 400 feet long. Most likely there would be no public access on top of the breakwaters. The touchdown of the structure would stop shy of the current rock dike along the perimeter of the island.

The terminal itself would consist of a shelter, fixed pier, a gangway and a float. The construction materials have not been selected but gave an indication that the float would be made of concrete or steel, similar to the one at Jack London Square in Oakland. In addition, there would be some dredging done to accommodate larger ships. The design of the shelter would be all wall glass and light roof. The fixed pier would also have glass on the north side to protect users from the north winds. The inhabited portions of the terminal would be designed to comply with the CBC 2013 and ASCE 7-10 standards of force design. The breakwaters would be designed either for displacement based ASCE 61-14 or force base, whichever provides a more practical design. The life span of the structures would be 40 years. The fenders would be about 15 years. He introduced Mr. Trivedi, who would explain the coastal design aspect of the project.

Mr. Trivedi related the fact that there was a dike along the perimeter of the island. In the 1930s, the dike was necessary to confine the sediments being placed to create the island for the World's Fair. The crest is higher on the northwest corner of TI as there was more exposure to wakes and wave swells from the Pacific Ocean. The typical crest elevations along the perimeter of the island ranged from 12 to 14 feet NAVD 88. The typical tide range was six feet, and the existing extreme water level including surge was 9.1 feet. During typical winter storms, the northwest corner experienced some wave run-up and high tides. This level was also known as still water elevation or SWL and does not include wave run-up. There was on-going settlement, especially on the northernmost corner of TI.

Addressing the design life, risk and consequence, Mr. Trivedi explained the coastal flooding zones, as designated by FEMA as Zones V or A, that consisted of areas on shore that are subject of wave attacks and result in 100-year storm inundations. These zones were in reference to inundation areas as a result of overtopping. Slide 10 of the presentation indicated the existing typical section of range of possible sea level rise (SLR) values over 100 years, which showed in the yellow-colored section relative to the crest the risk of overtopping. Although a seismic/tsunami scenario has a low probability of occurrence with significant consequences, SLR has a high probability of occurrence with a low immediate risk. The proposed solution would be to build high and do an adaptive management linked to a monitoring strategy for the future.

In addressing the risk posed by SLR, Mr. Trivedi displayed a graph and pie chart with references to Scripps Institute of Oceanography, October 28, 2011 that showed a predominant factors comparison for years 2010 and 2100 where sea level anomalies, steric sea level and storm surge showed a smaller risk factor (higher range of 1-2 meters) relative to the higher risk of tides and storm waves in the 2 and 4 meter range. The number for steric sea level in the 2-meter range is what they presently use for the tides protection analysis. He showed pie charts with a West versus East Coast of the US comparison whereby for the West Coast tides and waves were the predominant risk factors that need to be addressed. Sea level rise for the year 2100 reflected about half of one third of the pie chart. Instead, for the East Coast, the predominant factors were wave and surge (hurricanes-prone areas) and similarly to the West Coast, SLR is relatively smaller factor where the year 2100 occupies about similarly half of one third of the entire pie.

Mr. Trivedi explained the factors, data iterations and the probability analysis that went into the estimates of water height demand versus an elevation target for the shoreline perimeter in the future. In addition, he explained that the probabilities of occurrence of overtopping varied, such as for instance, the chances of high waves during high tides were low, and the chances were similarly low for high waves at low tides. Therefore, the design criteria to develop the elevations of the perimeter used the procedures known in the industry literature such as the FEMA 100-year storm event: wave run-up and the 2% wave run-up associated with the 100-year storm condition. The design would serve to assess overtopping primarily to protect the public access along the perimeter of TI. With that information, the elevation of the perimeter was established and the next step was to assess the risk factor of SLR.

The methods used to assess SLR were not limited to one study but ranged from a variety of sources, including the original M&N 2009 report that used varied wide estimates from historical measurements to maximum estimate in the Intergovernmental Panel on Climate Change (IPCC), Four Assessment Report (AR4) 2007, and other sources. Semi-empirical studies (Rahmstorf 2007) and scientific literature, a mixed of several methods were used to assess the SLR estimates. Again, no specific study was used for the SLR analysis.

Mr. Trivedi referred to Slide 13 of the presentation, which used as reference the IPCC 5th assessment risk curves showing high curve values for 2050 and 2080 (estimated life of project) of 24 and 36 inches. In turn, the National Research Council (NRC) 2012 had had incorporated a combination of the climate change based models from the IPCC risk curves and further use projected values, which are the most likely scenarios to occur in 2100, which correspond to 36 inches but with the potential of higher elevations of up to 66 inches for the same year scenario although unlikely.

Therefore, with all the available information and the associated numerical figures, the team had to decide the proper height estimate. The owners of the project decided to be conservative in this respect: building high so as to preclude a need to revisit or do significant changes or improvements in the next 60-70 years. Hence, the team used the most conservative numbers. A thirty-six-inch target was chosen as a minimum SLR height for year 2070. This numerical target was higher than present 100-year water level in the Bay and higher than FEMA standards. The perimeter's height for the short term, however, would not be designed at the 3 feet-above-FEMA, as the building development would be since it may obstruct the views. Instead, it would be designed to increase the height over time with plenty of room for adaptation in the future. Therefore, the height to be used for the perimeter would be lower than the buildings for a 35-40 year horizon. The perimeter height target would be 16 inches, in conformance with the NRC 2012 middle projections for SLR. This height strategy, however, would not be pegged to any particular number or study.

Slide 14 of presentation described the current elevation of the perimeter of the TI, which showed that about two-thirds, excluding the south and east sides, of the island were already above the FEMA base flood elevation or BFE levels. So the strategy, according to Mr. Trivedi, would be not to create a levee-protected community, but instead let the perimeter remain as a shoreline. This was the case since no action was needed now to protect the island from flooding. Using the figures shown in the slide, he described the southwest side of the island's existing elevations to be higher than the BFE, but with a 1% wave run-up, with no SLR included, he asserted that this area would be raised by about one foot. He showed another example such as the west side of TI, where the current elevations of the shoreline were also above the BFE. If the 1% wave run-up and no SLR were included, the area would need to be increased roughly by three feet. He stressed the point that the perimeter would require building it higher now, but with no necessary height improvements for at least the next 35-40 years. In addition, the design would increase 16 inches for SLR above the required (FEMA) elevations.

He then explained on the next slide, Slide 15, about the application of the 16-inch increase in the perimeter's height, not as a straight measurement but by flatting its bay front slope to attenuate the water levels by a composite slope of 2:1 and 3:1. Therefore, the initial elevations of the perimeter would be increased to take into account the 1% run-up plus the 16-inch increase. For the future, once it is assessed that water levels would exceed the initial project's height of 16 inches, an adaptation strategy would be put in place to raise it by 20 inches. As a result, the promenade would be raised but only enough as to prevent view obstructions from the public access. In the future, the promenade closer to the city development (west facing San Francisco) would be raised as a component whereas to the north where the shoreline was very far from the development area and ample space, there would be more opportunities for innovations with regards to the public access. Clipper Cove, Mr. Trivedi assessed, would not need major height increase of the perimeter as the area was already developed at an elevation that exceeds the 16-inch scenario. In addition, the area could be adjusted only minorly to accommodate the 36-inch one. Lastly, he mentioned that the funding mechanism for the adaptive measures beyond the 16-inch strategy was in the works. The next part of the presentation would be regarding the geotechnical aspect of the project.

Mr. Bill Rudolph began by drawing the attention to the documents included prior to the meeting, including a 2009 conceptual design report by ENGEO, and more recently his team had submitted a data report that summarizes new and existing data at the site. His team had also prepared an interim site characterization report, which was an interpretative report of subsurface conditions and soil properties. The report was interim in nature in that the data were still being collected and evaluated. As part of the ECRB packet, his team had prepared a shoreline stabilization strategy memorandum and one that indicated the geotechnical design criteria for the ferry plaza. In addition, today his team had brought many more details of the analysis that were run; however, they would not be part of the presentation. He said that in the interest of time and brevity, he wanted to explain and discuss geotechnical aspect of the project, including its original construction, subsurface exploration, soils characterization, seismic design criteria, geotechnical hazards and mitigation strategies and design criteria island-wide.

Treasure Island was developed in the late 1930s in the shallow waters of San Francisco Bay. About two-thirds of the site was underlain by sand shoals and to the east the site was underlain by Bay Mud and no sand (Slide 24). The site was developed by creating a rock dike perimeter, and in many areas the dike was place directly above the sand shoals. At other places with deeper waters, a blanket of sand was placed under the dike but over the sand shoals. The dikes were built incrementally with the initial rock dike at the bottom and build-up layers of rock and fill over or towards the landside and against the dike. Mr. Rudolph showed a map of the sub-phase 1A soil explorations (CPTs, borings, in-situ testing and others) done at the causeway and the west side of the TI where the proposed ferry terminal and breakwaters were projected (Slide 27). He pointed out that based on the explorations, his team had a fair amount of information to assess and characterize the soils at the site. For this area, he described the key strata from mud surface to about 270 below as sand fill, shoal sand (both sands are subject to liquefaction under moderate and strong ground shaking), young bay mud fine-grained, young bay mud sandy deposits, young bay mud offshore deposits, old bay deposits and bedrock.

Mr. Rudolph showed soils cross sections of areas of concern/consideration for the sub-phase 1A areas of development with differing soil conditions such as the City Side Park north of the proposed ferry terminal where there are deep soft bay deposits but with a 300-foot set-back area of open space. Toward the southwest shore at the proposed ferry terminal and plaza, there were sensitive areas of shoreline improvement adjacent to the areas of the shoreline. Such cross section was used to analyze the alternatives to limit the amount of deformation near shoreline. At the Clipper Cove side there exist areas of thinner bay deposits where buildings would be in the proximity of the shoreline representing different conditions to be addressed with different performance criteria considered in the geotechnical design. The other feature was the causeway structure where thicker sand deposits created the ramp connecting YBI to TI. He explained more in detail the laboratory testing done and how the data were optimize to analyze the differing soil conditions of the site.

One aspect of the design criteria at the site (the City Side Park/Ferry Terminal) was the specific selection. There were two separate criteria in the evaluation. The team started out by looking at the CBC 2013 and ASCE 7-10 geotechnical design criteria, the MCE_G (geotechnical map) value. This value was based on the lesser of the 2,500 year return probabilistic motion and capped by mean + 1 standard deviation deterministic event. Using such criteria, he found that the ground motions at the site were almost equally governed by a magnitude 7.3 earthquake at the Hayward and 8.1 earthquake at the San Andreas faults with a peak ground acceleration of about 0.46g, namely, a so-called "life safety non-collapse performance criteria." But the CBC/ASCE 7-10 criteria would be applicable to habitable structures and not to waterfront stability buttresses and deformation analysis of waterfront slopes. The second criteria being looked at to evaluate these types of structures had been the 475-year return period or Level 1 performance. By applying the latter criteria, the results came out to be equal to the PGA of 0.46g. Therefore, by applying the second criteria to the shoreline structure, it would result in uniform criteria for both ferry terminal and waterfront areas. He explained that the team was not setting up separate criteria for the open space, but instead putting the kind of deformation and performance levels into perspective. So the analysis was done to evaluate the expected performance along the shoreline and only with regard to the City Side Park (Subphase I) if there was an event similar to the Loma Prieta earthquake of 1989. The analysis was used after Loma Prieta for communicating risk for a smaller level earthquake but not necessarily as design criteria.

The hazards identify in the analysis included liquefaction (lateral spreading and seismic settlement of sand fills), seismic slope deformation (kinematic loads on offshore structures) and static settlement under the consolidation of the bay muds. Accordingly, the mitigation measures included the use of stone columns and setbacks in feasible areas, deep soil mix buttresses (DSM) where higher level performance was sought along the shoreline, tie-back sheet piles or DSM soldier pile walls, vibro-compaction to mitigate seismic settlement in areas beyond the influence of shoreline deformation, to mitigate kinematic loads on offshore structures, piles would be designed for lateral loads and for areas of static settlement would be surcharged with wick drains to pre consolidate and mitigate for poor-performance soils.

His next slide, Slide 36, showed the remediation strategies along the perimeter of TI. There was a densification zone (stone columns or equivalent) along the City Side Park (half of west side) and portions of the northeast and southeast sides, areas where there would be a 300-foot setback to the shoreline. This strategy was meant to mitigate lateral spreading flow; however, the shoreline would still be exposed to some inertial deformations but that would be limited to a minimum amount. The idea would be to increase the protection at the development but reduce it in the areas of wide setbacks. The next strategy would consist of structural retentions (deep soil mixing or DSM, soldier and sheet pile retention or equivalent) with the intent of limiting deformation of both lateral spreading to less one foot along both sides of the causeway to the south, along Clipper Cove (southeast) and northeast sides of TI, areas critical to access to and fro. Clipper Cove would remain an area with utilities close to the shoreline that needs a higher level of protection. Hence, the recommendation of DSM buttress would be applicable along this portion of the island. Along the rest of the island, similar setback conditions exist to the City Side Park, investigations and remediation strategies continue to be studied where there would be a future edge retention in areas with thick YBM (structural retention to be determined) along the northwest side of TI, and future edge improvement (as needed) to the north and middle east side of the island.

Mr. Rudolph continued to explain the City Side lateral deformations investigations of the soils studies. With regards to the Ferry Terminal, it would have a DSM buttress along the shoreline; however, the outboard of the dike areas, including the seafloor, outside the buttress remediation would still be subject to seismic deformations. As a result, the team evaluated a design for the kinematic loads associated with the yielding of the rock dike beyond the edge of the DSM buttress. With regards to the breakwaters, the strategy would rely on short partially penetrating concrete piles and king and batter piles for vertical and lateral loads. The latter piles would derive their capacity from friction in the older bay deposits in the bay mud.

He touched on the earthquake instrumentation subject that the ECRB would be requesting information on. He said that seismographs would be installed in the DSM buttress zones and that a post construction LiDAR survey would be performed to set a baseline for reference in future potential deformations in strong motion events. This was the end of the applicant's presentation.

Discussion: Chair Borcherdt's initial comment referred to the much added information the applicant had provided from the review packet ahead of the meeting. He indicated that the Board would be interested in discussing the stability of the perimeter with respect to the stone columns, DSM, etc. and their performance during a Loma Prieta-equivalent event. Mr. Rudolph said that during Loma Prieta, there was partial liquefaction of the shoreline resulting in a limited potential for lateral spreading. If the ground motion were somewhat higher than Loma Prieta, there would have probably been more flow-type failures and larger deformations. He then addressed the site of City Side Park with respect to the same question. The stone columns idea would mitigate liquefaction and flow failure in a number of ways such as providing a higher composite strength of the materials around the perimeter, would densify the existing sands, would intercept lower permeability layers that may control lateral spreading, and provide drainage. As a result, there would still exist the potential for inertial ground deformations occurring beneath the stone columns and the potential for flow type failure to occur outboard of the stone columns zone where no improvement would occur.

Mr. Rudolph reminded the Board that the rock dike is sitting on sand fill. So with this remediation flow failure would be limited to the outboard areas of the stone columns. Inland the deformations would be more associated with the inertial deformations within the bay mud. For instance, within the park itself at the shoreline there could be up to 10 feet of horizontal and vertical deformation right at the shoreline. However, immediately inland of the stone column zone the deformation would be in the order of 4 feet. Even further at the 300 feet back where the development zone would be, the deformation would be less than one foot horizontal and less than 3 inches of vertical movement. He indicated that this design was protective of the development areas and that damages within the shoreline perimeter would be repairable following an upper level earthquake event. With regards to a Loma-Prieta-type of event after the installation of the stone columns, the conditions would be so much better than those that exist today or those that existed at the time of that event resulting in higher levels of performance with some settlement and lateral yielding but with no wholesale failures of the shoreline and just minor movements.

The Chair followed up Mr. Rudolph's response in reference to the design criteria based on the MCE_G maps and how the criteria would differ once taking into account a specific site response at the surface or beneath the Ferry Terminal. Mr. Rudolph said that because the design criteria were based on PGA, the impact of modification of ground conditions on site response would not be so relevant in terms of the design of the shoreline buttresses. The Chair inquired about the CBC code occupancy classification for the terminal. Mr. Rudolph said its classification was Site

Class E. The site classification is based on the types of soils present and their engineering properties as defined in Section 1613A.5.2 of the CBC. Site Classifications A through F range from hard rock conditions to soils vulnerable to potential failure or collapse under seismic loading, respectively. Chair Borcherdt strongly disagreed with the designated classification of Class E, and opined that Class F would be more accurate and appropriate, and further that such classification had already been determined in the ENGEO literature that had been provided to the ECRB for review prior to the meeting. The Chair indicated that the site classification of Site Class F as specified in CBC 2013 requires that a site specific analysis procedure be used to derive estimates of design ground motions. He opined that in doing this analysis, Mr. Rudolph's team would find large resonant ground motion amplifications associated with this site. He pointed out that large period-dependent amplifications have been recorded on the Treasure Island borehole seismograph array as described in the reports by de Alba and others (de Alba, P., Benoit, J., Youd, T. L., Shakal, A. F., Pass, D. G., and Carter, J. L. (1994), "Deep instrumentation Array at Treasure Island Naval Station," in *The Loma Prieta, California Earthquake of October 17, 1989 Strong Ground Motion and Ground Failure*, Borcherdt, R. D., ed. U. S. Geological Survey Professional Paper, Washington, D. C., A155-A168). He pointed out that this array, as initially funded by the National Science Foundation and being operated by the California Geological Survey is providing recordings of local earthquakes especially important for understanding and predicting the amplification characteristics of the soil deposits beneath Treasure Island. This particular area, he suggested, needed to be investigated more thoroughly. He opined that the site response throughout the island would vary significantly and could become a major issue with respect to the design criteria for various structures; therefore, this issue needed careful attention, which was absent in the current report.

Mr. Rudolph said there was no disagreement with respect to the structural structures, but in referencing the work of Mr. Kyle Rollins (1989) regarding computed and measured liquefaction-induced settlements after Loma Prieta earthquake, he commented that the issue would be addressed. However, he expressed skepticism that the ultimate results would not change the evaluation and remediation of liquefaction nor would it change the performance and design of gravity shoreline buttress type system. He agreed with the comment and said that it would be part of the outcome of work for the development of the criteria, but that the criteria would not change the strategy for the shoreline structures as they may not be relevant to such structures. Chair Borcherdt noted that in some cases very dense materials over the top of soft materials can improve the response at the surface because the dense material would serve as a reflecting surface back to the source instead of propagating to the surface. But there are many issues that need to be studied and analyzed such as how the soil conditions at this site affect the final ground motions. And because of the resulting ground motions on the perimeter structures, displacement could be an issue; therefore, the team might want to see how the sand soil layer affects the longer period motions.

Mr. Frank Rollo tried to reiterate his perception of intents of the engineering criteria as described. The first intent, as he saw it, involved three option methods for stabilizing the perimeter by DSM, stone columns and walls. The second intent was to eliminate, reduce potential for liquefaction in the development areas and along the causeway. The last intent involved the allowance of the outboard dikes to go adrift because it is believed that the deformations based on the improved soils would be less than one foot. In addressing the causeway component, he believed that such structure should be classified as an essential structure for it serves as primary access to the island. Therefore, he thought it would be designed under essential structure criteria (2,500 year return period). He was then corrected that this structure would be designed as the MCE_G, which is deterministically capped. He asked whether it was designed for up to one foot of movement. Mr. Rudolph asserted that the one foot was the target criteria. Mr. Rollo asked whether the major utilities running along the causeway had been evaluated for such movement. Mr. Rudolph said it would be the intent to do so. Further discussion regarding the details of the one-foot goal ensued, and Mr. Rudolph indicated that such goal was based in professional judgment and that it was rationally protective of the structures for an MCE_G event.

With regard to the intent to reduce potential liquefaction in the improved areas (buildings and habitable spaces), admitting that he believed the areas should be classified as Site Class F, Mr. Rollo inquired whether the intent, once mitigation of liquefaction has been addressed, was to argue for all the areas along the perimeter to be classified as Site Class E. Mr. Rudolph replied that was not the case as not too many structures would be located along the DSM areas; Mr. Rollo asked the same question regarding the habitable areas. Mr. Rudolph responded that the intent of the presentation was not to address the criteria of the development areas (buildings) as they lie outside of the BCDC's 100-foot shoreline band boundary. He further indicated that there would be other level of study to develop criteria considered for the developed areas, but that these would not fall under the purview of BCDC. Mr. Rollo asked whether the intent was to stabilize the entire perimeter in the first phase; the intent, according to Mr. Rudolph, was for Subphase 1A as a stand-alone stabilization project. Other stabilization strategies for the rest of the perimeter areas would follow.

Mr. Rollo inquired whether there would be a Subphase 1A wrap-around protection out 300 feet inland to prevent the liquefiable material from flowing around the stabilized portions. The response was that the direction of flow would be perpendicular to the shoreline and there was no intent at this time to use a wrap-around to protect Subphase 1A components. Mr. Rollo's inquiries were related to any possibilities that any influence of the inland and not-yet mitigated inland portions of the island would have any effects either on the stabilized sections or the dike. Mr. Rudolph responded that the intent would be to stabilize a significant distance beyond the Subphase 1A area from the shoreline to prevent any free flow that could pose a risk. As far as determining the specific distance of mitigation, it was not yet known since the team was at a conceptual level with regards to the remediation scope. Mr. Rudolph indicated that among the mitigation measures to counter settlement, and liquefaction, in general, it would include vibro-compaction (densification) behind the DSM buttress and surcharging with wick drains in areas of new fill (Slide 39). He added that settlement had been originally over estimated as there is indication from the soil exploration that it may range from 8-12 inches rather than 2 feet. With vibro-compaction, there could be up to 8 inches of ground loss, which would then be filled to the new grade. He opined that the installation of wick drains first would facilitate the vibro-compaction and then surcharge. Mr. French indicated some skepticism about the 8-inch settlement statement being at the low end and thought it prudent to check on this as the issue had been discussed by Mr. Kyle Rollins in a previous report. Mr. Rudolph said there would be some ground-truthing worth doing.

Mr. Bob Battalio had questions regarding the sea level rise analysis in relation to the geotechnical analysis. He wanted to know whether the additional 3-foot fill would be done post-settlement (after vibro-compaction). Mr. Rudolph affirmed such concept in that after surcharging, the project would add layers of new fill once the ground had been stabilized. Mr. Rollo asked to check on the stability of the permanent dike when the 10 feet surcharge happen. The surcharges would be far back from the shoreline in any event.

Professor Fischer asked whether more stringent design criteria had been considered for the causeway as it was the only way to get in and out of the island. Mr. Rudolph answered in the negative. Mr. Porter said that in the event of a disability, there would be other ways of accessing the island by boat in the interim.

Mr. Holmes inquired whether the Ferry Terminal would have different coding as the pier and shelter would be complying with CBC 2013 and ASCE 7-10 but with Occupancy Category II and III, respectively, and the reason for not classifying them all as category IV. He also agreed with the other Board members that this component should be considered an essential facility. Mr. Porter said that the causeway was the primary means to the island whereas the ferry terminal would be secondary. In supporting the position, Mr. Rollo asked whether the upslope on the causeway on YBI had been analyzed for failure and speculated as to the consequences of long closure of the structure. The discussion turned to ways of mitigating the risk of deformations offshore.

Mr. Rollo inquired whether in considering the Causeway an essential structure, it would require the same classification for the Ferry Terminal. Mr. Rudolph explained a system used for evaluating risk of offshore movement by analyzing and devising deformation profiles (Figure 4 of the Geotechnical Memo No. 1). These would indicate zones of soil resistance and sand sediments. Mr. Eliahu further explained that they anticipated kinematic loading on the piles within the upper sand layers of the bay mud and that any piles driven for the Ferry Terminal would penetrate through that zone deep into the soils resistance region that would be loaded by the moving soil. The bay mud would flow around the piles. Further analyses would determine whether there existed enough resistance of the soils to support the slope. Explanation of the conceptual analysis regarding this approach continued.

Further information revealed that the DSM buttress would be set about 12 feet inland of the top of the slope of the dike. The distance of the DSM versus the dike was in relation to the sufficient space to avoid hitting the inboard toe of the dike. Further probing of this potential conflict between the DSM and the dike had been suggested to the client to reduce cost risk in the bidding process. Mr. Rollo inquired as to the movement of the dike. Mr. Rudolph said there were two different modes of movement. One would be one of straight settlement, but the one of most concern was regarding the lateral flow and any movement that would be difficult to predict during liquefaction. Such movement could be in the order of 10 feet or more, similar to a block riding on un-compacted sand fill.

Mr. Rudolph speculated that if the analysis showed that movement of the dike would be excessive, large diameter piles offshore may be used to stabilize the rock dike. The other possibility would be to build a very long pier span to bridge over the rock dike width and movement. Both concepts were being looked at but no selection had been made. Mr. Rollo asked him to address that before getting back with the information to the ECRB. Mr. Holmes asked what would happen to areas outside of the ferry facilities, would the instability situation of the dike be address elsewhere as well? Mr. Rudolph said that it would need to be addressed at later stages of design as the presentation was focused now on Sub-phase 1A. Mr. Rollo hypothesized that the Board may need to recommend criteria that would prevent unnecessary fill and protection of the bay from an unstable dike, which is part of the ECRB/BCDC mandate. However, he also said that the Board should be reasonable about it as well. Mr. Rudolph expressed that based on such comment, every shoreline areas along the bay is susceptible to similar conditions, and his team’s challenge was to come up with reasonable criteria at some occurrence interval taking into account feasibility (economically and technically). He agreed that something should be addressed but implored the Board to be reasonable.

Mr. French asked if the analysis was based on NGA with the ground motions and inquired into the Site Class F inquiry raised earlier whether it was applicable to buildings or not, but in short whether the evaluation of the site class based on the ground motion shifted pulses and ground velocities and material softer than bay mud would result in a different classification. Mr. Espinoza was asked to explain the subject. There would be large amplification due to liquefaction at the longer periods and the phenomenon had been reflected in the site response analysis. The rock mass would be considered shallow and relatively small failures compared to high- buildings' whose periods are shorter. The results of the PGA and motions were lower than the Site Class E since the motions were high. If, in the end, liquefaction did not occur, the site would still be protected. Mr. French expanded such comment to confirm that the analysis of the ground motions had followed an enveloped approach. Mr. Rudolph said that there were other analyses not presented today that included further studies of this issue. The presented materials were only a preliminary conclusion of the studies. There were discussions on the issues liquefaction, the MCE_G criteria. Mr. Rudolph made emphasis that the DSM buttress embedment would be deep enough to go through the soft layers fully penetrating into the bay mud with enough capacity to protect the site from lateral spreading and deformation. Mr. Rollo added that the system would be protective to the extent of the limits of the Park. More about this discussion of the criteria ensued. After more probing from Mr. French regarding the specific design criteria, Mr. Rudolph declared that the MCE_G was the design earthquake level.

Because of the uniqueness of the site, Chair Borcherdt reiterated the request for a site specific analysis designed to know the influence of site response not just for the periphery of Subphase 1A as a public record, but also as public knowledge to be used for general planning purposes for development of the entire island in the future. Mr. Espinoza attempted to explain once more the approach to the site classification. Mr. Rudolph related that the memoranda summarized the studies as general seismic design criteria, which he thought it described correctly, but a detailed seismic report was in the works for this part of the project. A full seismic analysis with site specific response analysis would be submitted. He said the team was communicating a basic philosophy approach in the design criteria. Mr. French suggested that a more careful look should be taken at the ground motions that could result in further movement of the shoreline. Mr. Rudolph referred to the aftermath of ground shaking during the Loma Prieta earthquake that did not result in significant movement of the area. Loma Prieta accelerations had been 0.13 and 0.15g. Mr. Rudolph said that the hard structure retention at the shoreline would be designed for the MCEg and there would be no lower level earthquake criteria.

Mr. Rollo pointed to a retention wall in the conceptual models being used in conjunction with the DSM buttress. Mr. Rudolph said that concept would not be used after all. Mr. Rollo asked about the timing for the development of the Subphase 1A and when would there be a completed evaluation prior to the recommended design for slope stability and mitigation of liquefaction, breakwater, pile design, etc.? Mr. Rudolph deferred to the project manager of the project. Mr. Papadopulos and Mr. Eliahu intervened to say that the team had analyzed cross section and come up with design for such; however, the fine details still remained (walkways, utility coordination, shelter design, etc.) but the basic design concepts have been developed and described. He mentioned there were two alternatives for the causeway. His team had settled for one, which was not the steel sheetpile bulkhead solution but something that would result in much higher performance. However, he did not reveal such solution. Mr. Rollo asked when the Board would see that strategy. When would they see the assumptions that went into the design? Mr. Rollo indicated the information regarding downdrag loads, shear strengths used for the old bay clays, and asked the rationale for the details assumptions made regarding the liquefiable sand. Mr. Elahu said that such information was in the materials submitted for review. Mr. Rollo rejected such argument as he said he had done a thorough review of the material. Mr. Eliahu said that there would be a design report to establish such information, but that his team could respond to such questions and provide all the justifications.

Chair Borcherdt asked about the project's percent design stage. This reference would be crucial in order to effectively advise BCDC as to the issues and potential concerns based on its mandate in relation to the policies of bay fill. Mr. Eliahu explained that the objective today was to come to the ECRB with the basis and the premises for the design to reach a consensus on the criteria. The deformation range as described had been conservative. Mr. Rudolph tried to assuage the Board's inquiries and concerns by saying that there would be a much more detail seismic report with regards to the soils and assumptions made regarding the liquefied materials and shear strengths, etc., and he added that a summary could be put together as a response to the inquiries. Mr. Rollo expounded on the many issues resulting from the information that needed to be discussed by the team and know with higher degree of measure of the consequences of any movement of the rock dike bayward and the degree of its potential movement whether this be few inches or 10 feet, etc. of displacement and how this could affect the DSM structure. Would there be erosion protection of the DSM if the rock dike were dislodged? Would there be measures in place to put back the dike to its original location after displacement? Who would pay for the collection of any misplacement of fill in the Bay? He expressed his opinion to have the applicant come back at a later percent design stage (65-70) to know of any resolutions or findings to the issues discussed today. The level of specificity seems lacking at this time. He opined that the applicant needed to come back to the Board with more complete answers. Mr. Rudolph replied that the intent was to produce the later seismic report to provide more specificity.

Mr. French asked about the methodologies for the different deformations of 10 feet in front of the stone columns and four feet on the back of the columns, etc. Mr. Rudolph said they had used yield accelerations for circular and noncircular slip surfaces and did the three-progression models that produced the deformation levels. The shear zones were estimated and based on the deformation, a calculation of shear strain was gotten. Where the shear strains were less than four percent, it was assumed there was no shear softening but the bay strains were appropriate. He mentioned a PLAXIS analysis that showed a similar magnitude of cycle deformation during an earthquake event. Further detail of the analysis on this subject ensued. Mr. Rollo added that the Board wanted to know the input side of the analysis to understand the assumptions made.

Since Mr. Brady, the structural engineer of the project, was about to leave, Mr. Porter prodded the Board for any particular questions regarding the structural criteria. Mr. Holmes returned to the occupancy category question and why the project had not selected a higher level of safety criteria for the ferry terminal components. Why were there all but one structures classified as Two? The shelter was classified as Three. And he asked why not Four to design it as an essential facility?

One of the arguments made for the lower category was that the Bay Bridge was the primary means of egress and entry to the island, not the ferry terminal, which would carry about 20 percent of the islanders at one time. The Class Three of the Shelter alone, according to Mr. Porter, was due to the occupancy level as prescribed by the CBC 2013. The terminal would not be designed to take all of the inhabitants in the island out. More discussion on the vulnerabilities of the entry and exit points followed. Mr. Espinoza added that the viaducts of entry/exit are programmed to be replaced by a different project, including Macalla Road and the Causeway. The causeway would be set deeper into the hill side primarily because of the geotechnical concerns. This roadway project sequencing would be in the years 2016-2017. The Chair inquired about the current percent design stage for the general criteria of the shelter/ferry terminal.

Dr. Gilpin asked whether there was sufficient subsurface information across the critical access causeway structure. The abrupt transition from rock on Yerba Buena Island to fill over Bay Mud along the Phase 1a project shoreline may warrant more closely spaced borings than presently exists (approximately 300 foot spacing). It would not float away. Mr. Rollo asked whether he had enough information to unequivocally support that assertion. Mr. Holmes asked whether the team was relying on the rock dike for geotechnical protection at all, and the answer to Mr. Holmes and Mr. Rollo was in the negative. It would provide wave protection, but after an earthquake, the dike would have to be rebuilt, Mr. Rudolph replied. Mr. Rollo prodded the design with a question about a tsunami scenario where the big wave would hit the island when the dike had already been undermined and separated from the DSM. Mr. Rudolph said the DSM will be about 60 feet in depth where the upper 10 feet would be designed as a retaining wall to protect it from such event. Mr. French asked whether 10 feet was deep enough and how was it estimated. Mr. Rudolph explained the concept level of vibrating full plate I-beams into the DSM columns and would be reinforced on the land side and designed with tie-back system into the DSM. Mr. French reiterated Mr. Rollo's point that these are the criteria the Board liked to see in the next iteration.

Chair Borcherdt wanted to know whether there were plans to monitor settlement. Mr. Rudolph mentioned again the PLAXIS analysis to evaluate the existing confined and unconfined soils conditions. The Chair suggested exploring the possibilities of installing strain gauges along the periphery where soft bay muds are present. Mr. Rudolph said that they would be open to ideas and ways to address this.

Mr. Battalio had some questions regarding the reasons for having four guide piles for the float rather than six, which was the case for a similar float in the San Francisco Ferry Terminal. Mr. Porter did not have an answer at the time and Mr. Brady had already left the room. Therefore, he could respond to this question in writing later on. Mr. Battalio asked further about the coastal engineering criteria of the breakwaters, and more specifically regarding southerly wind waves and whether the team anticipated and considered reflection off the back side of the north breakwater and complex wave patterns at the float that could result in an increase of the peak loads. Mr. Dilip explained there had been a large mole (rock) in the area of public access where the waves were concentrated. The team wants to remove that fill, which would result in further studies of relieving that area from the obstruction while leaving the wave surge component. He explained further the challenges of the design and commented that one of the issues at the site as opposed to the San Francisco Ferry terminal had been sand transport. Further discussion on the coastal engineering studies and issues ensued.

Mr. Battalio requested to raise some SLR questions, but since Mr. Rollo had to leave for an appointment, he was given the chance to provide some last comments. He wanted to know who the client/applicant was and whether a public entity (the City) would ultimately run the development. Although he expressed good progress on the discussion and understanding of the project, he suggested the Board to request a follow-up meeting at 65- or 70-percent design with more specificity. He turned to Mr. Montes for guidance either directly or after consultation with BCDC management about the issue of the rock dike moving 10 feet into the Bay, for if the dike cannot be placed back, it would mean more fill in the Bay to fill the gap between the DSM and the dike. Therefore, he asked how the rock dike sliding into the Bay in the aftermath of an earthquake squares off with BCDC's policies on the safety of fills and other policies regarding the protection of the Bay. Mr. Batha responded to the question to say the staff would consider the extent of the issue from the development along the shoreline affect the Bay. Mr. Rollo left it to BCDC's discretion to weigh the consequences against it's policies, but added his motion that the applicant should come back to the Board for further review at a later design stage. Mr. Holmes questioned whether 65-70 percent design would be too late for a consultation with the Board for advice.

Mr. Battalio came back to the sea level rise inquiries. He asked whether the project as he understood it was being designed for 16 inches rise by year 2050. Mr. Trivedi said that it was 16 inches but not necessarily pegged to a year goal. Mr. Battalio thought it to be consistent with the BCDC's policies and the NRC 2012 middle projection. Ms. Yeung remarked that BCDC had asked the applicant to consider the higher range of the NRC 2012 that would use the 24-inch value instead of the 16 inches as an initial projection. Mr. Trivedi tried to explain and justify the values being proposed versus the BCDC request to use higher values, 16 versus 24 inches. He said that based on available data globally and regionally (West versus East coasts), the higher values may not be substantiated. Mr. Battalio countered that the degree of uncertainty is significant to err on the conservative-(higher)-values side since he quoted studies that referred to the suppression of sea level rise on the America's side of the Pacific partly due to oceanic circulation, heat exchange where sea level rise is higher on the Japan's side. He added that there were some opinions about SLR in the America's side of the Pacific that rising water levels would eventually catch up with global sea level rise levels. Another question regarded whether the studies done

for the project were consistent with the BCDC's policies on Climate Change since the project's studies were done before these policies came into effect in 2012. Mr. Trivedi replied that the design of the water levels was on-going. The adaptive management plan would have triggers to monitor water levels.

The design of the perimeter was such that it would be adaptable to increase elevation even before the 16 inches had already occurred, again a value not pegged to any particular year. However, the chosen elevation was anticipated to hold true for the next 35 years. Mr. Montes quoted part of the Policy No. 3 on Climate Change with regards to any new structures in the Bay. The policy read, in part, that all projects should be designed to be resilient to a mid-century sea level rise projection. So the question to be asked was what was that 2050 number? Mr. Trivedi pointed out that the project used much more conservative values than the most current work from the IPCC. As he read the policy, he interpreted as enabling the professionals to use best available science (Policy No. 2, Bay Plan Climate Change) in the flood risk assessment. Further, he said that the perimeter work would not stop at 16 inches as it would be a completely an adaptable structure. Mr. Battalio asked further what would happen after 2070 when the highest estimated 36-inch value had been used at the project. Would the development be there by 2100? Mr. Trivedi replied that the perimeter was not limited to a particular height or elevation. It would change its function at particular points in time. When SLR reaches 36 or 42 inches in the future, the perimeter becomes a levee. After further questioning from Mr. Battalio regarding the adaptation of the project by the end of century, Mr. Trivedi reported that the year 2070 did not imply a sunset date for the development. Further questioning from Mr. Battalio included a scenario of water running through the street corridors and the contingencies for getting the runoff/overtopping waters out of the island. Mr. Trivedi asserted that the drainage system would be designed to handle any waters inside the island or perimeter. What about the combined conditions in the future of SLR and high tides and rain while the flood gates are closed, where would the flood waters go to? Mr. Trivedi said the San Francisco Public Utilities Commission (SFPUC) has criteria for system designed storm drains. The storm drain system would be separate from the sewage system. He also referred to a library of documents done by the applicant's team to address issues of grading and contingency plans regarding potential water back-up issues. The final grades were to come up three and a half feet above existing grade conditions by placement of fill. Therefore, the crowns of the pipes would not be at the present grade but higher. The PUC criteria would be to have four feet of required freeboard/hydraulic grade line, including an allowance for sea level rise very similar to the perimeter protection. The addition of pumps would be considered later as an additional step to the adaptability process. This additional information had not been made clear in the packet to the ECRB. Mr. Beck pointed out that all existing infrastructure would be demolished to make way to a complete upgrade. There would be a number of "lift" or pump stations to improve the drainage within the development areas. Mr. Battalio would require more information regarding the storm drain system for the next review session. Further discussion of the drainage issues continued.

Chair Borcherdt asked the Board to prepare for a motion. Mr. Battalio requested the Chair for additional time to make further inquiries regarding the sea level rise aspect of the project. Would part of the development fall within the new and to-be published flood hazard zones pointing out that some FEMA flood elevation forecast were higher than today's. How would the new FEMA maps affect the development and implications to the City? Mr. Trivedi said that with the adaptation of SLR in the project, it would be outside of the FEMA zones, and he opined that the project would be covered from the FEMA perspective. Mr. Battalio requested that the applicant provide the provisional FEMA hazard zones for the site, and how these hazard zones relate to the proposed perimeter flood protection. (Clarification: will the proposed perimeter treatments prevent the flooding shown in the provisional FEMA maps, and what freeboard would be available for future sea level rise?) Mr. Battalio confirmed the methodologies used for the reduction of the 16 inches based on the composite slopes as the calculations were provided to the Board, provided that the shore armoring is constructed using a quarry stone armor layer and slopes to provide the runup dissipation indicated in the calculations.

The Chair thanked Mr. Battalio for his comments and complimented the applicant for the information and presentation provided. He acknowledged BCDC's request to the Board to provide recommendations with respect to the safety criteria. Agreeing that the criteria were at about the 35 percent design stage and that additional decisions were needed by the applicant before a significant number of the engineering criteria would be available for review, he expressed that the applicant would need to come back before the board at a later design stage at perhaps 65 percent.) He mentioned that the role of the ECRB was to ensure that the BCDC policies were implemented with the engineering criteria pertaining to seismic safety and minimal impact on San Francisco Bay being of principal concern. Close to the end of the discussions regarding the continuing debate on the lower performance level earthquake (474 year-return), Mr. Rudolph suggested that if an earthquake level would serve as a "benchmark" comparison for sustainability of the project that it should be based on the Loma Prieta event and not on an event not yet experienced and to work with criteria that were measurable and not arbitrary. With regards to the question of the extent of the rock dike movement and any Bay fill resulting from its displacement, he suggested that rather than devising a strategy to limit its movement to come up with conditions to mitigate any end results when the earthquake event occurred. This measure, he opined, would be more manageable from his perspective, for, he thought, there was nothing economically feasible to address/prevent the movement of the dike from an earthquake. He claimed that, in the end, the net benefit of the project safety-wise would be higher than any detriments. Mr. Tootle supported this claim in that if nothing was done to the perimeter from any project, the mass of land that makes up the man-made island would go back into the Bay. In contrast, the amount of fill going into the Bay in the aftermath of a strong earthquake would be miniscule after the project had secured the perimeter as described. The net benefit to the Bay, according to the applicant, would exceed the detriments and any negative effects could be conditioned to be mitigated. Mr. Battalio wondered what would be possible as mitigation from the displacement of the dike. He also inquired about the steps that could be taken to repair the dikes in case of failure without increasing the amount of fill in the Bay.

Mr. Eliahu said that the team would summarize all the assumptions and methodologies as discussed earlier in the meeting.

Chair Borcherdt indicated he would entertain a motion for discussion that would suggest that the project move forward with contingencies and the understanding that the project is at an early design stage near 35 percent with many decisions regarding the final engineering structures to be designed and constructed yet to be made, including decisions pertaining to the DSM barrier, the number of breakwaters, the causeway, potential erosion implied with the installation of new breakwaters, instrumentation to monitor settlement and seismic response to earthquake shaking, and others. In addition, he indicated as pointed out by other Board members that the long-term development plans for the interior of Treasure Island presented some special challenges with potential impacts on the Island perimeter that could result in lateral movements of perimeter material into SF Bay, including such projects near the 100 foot periphery as addition of significant amounts of fill to elevate areas above flooding levels, large densification of soil and structural projects, and earthquake induced liquefaction of materials near the periphery. Contributions of numerous Board Members to the discussion lead to the following motion, which passed with unanimous voice vote as proposed and seconded by Board members Battalio and Gilpin, respectively.

MOTION: With the understanding that:

a. The Treasure Island Development SubPhase 1A Project is at an early design stage near 35 percent;

b. The SubPhase 1A project pertains only to portions of the south and western boundaries of the 100 foot shoreline periphery protection zone, so criteria approval of SubPhase 1A does not necessarily constitute approval of engineering criteria needed for other portions of the shoreline, and

c. The impacts of large development projects to take place in the future may affect these and other portions of the boundaries of the 100 foot shoreline zone for SF Bay, the ECRB suggests that the SubPhase 1A portion of the Treasure Island Redevelopment Project move forward with the following contingencies:

(1) The ECRB requests that the SubPhase 1A project applicants plan to return to the ECRB for review at a more complete design stage, which could be at the 50 percent or 75-80 percent stage depending on responses received concerning the following requests for additional information,

(2) The Board requests the development of site specific response spectra at the locations of each of the major projects in Subphase 1A, including the ferry terminal building, the ferry terminal breakwater structures, the DSM barrier, and the Causeway. Each site specific estimate is requested to be based on detailed shear-wave velocity profiles as inferred from boreholes and geologic cross sections from the surface to bedrock and corresponding soil response models most appropriate for each location, consistent with definitions of Site Class F site-specific procedures provided in ASCE 7-10 and CBC 2013. Additional justification is requested for any sites underlain by water saturated sand layers near the surface, not classified as Site Class F.

(3) The Board recommends that a thorough analysis be conducted regarding the impact of earthquake induced liquefaction and lateral failures on loads and deflections of guide piles, gangway, and potential rock dike movements near the ferry terminal.

(4) The Board requests additional information regarding justifications for occupancy level design categories for the Ferry Terminal of 2 and 3, as opposed to level 4 for essential facilities, that the Terminal could become in some earthquake emergency scenarios.

(5) The Board requests further information concerning the analyses conducted to estimate the lateral movement expected of the DSM barrier and associated retaining wall during design earthquake ground motions from the nearby Hayward and San Andreas faults. Estimates depicting the lateral extent and volume of likely liquefaction induced failure in and along the margin of the Bay are needed, as well as information on scenarios that might be undertaken to repair failure following a damaging earthquake.

(6) The significant wave action anticipated from frequent westerly winds in San Francisco Bay and those generated by large container shipping vessels and ferries implies that a third breakwater protecting the entrance of the terminal harbor will be needed. Design plans for such a break water together with the amount of fill implied by each the breakwaters are requested.

(7) A thorough study of the influence of the ferry terminal breakwaters on shoreline erosion and sand build up in and along the margins of TI over the intended lifetime of the breakwaters is recommended. A summary of the results of this study is requested.

(8) The Causeway is considered a critical lifeline for the island in case of an earthquake emergency. Completion of the causeway's general design and submittal of the plans for ECRB review is requested.

(9) As the design stages for the DSM, Causeway, Dikes, breakwaters, and shoreline retaining structures approach later stages of completion, the applicant is encouraged to provide updated plans for review, including additional geotechnical details.

(10) Additional information is requested on the capacity of plans for Subphase 1A of the project to adapt dikes after 16 inches of SLR for higher levels anticipated in 2100. Information is needed documenting a conceptually feasible adaptation strategy for higher sea levels, consistent with projections for the year 2100, which are 36 inches (mid-range "projection") and 66 inches (high) per NRC 2012, Ocean Protection Council (OPC) 2013. If there was a higher perimeter barrier, would there be enough real estate and adequate slope stability? If overtopping were allowed into the undeveloped shoreline band during extreme events, where would the water go or how would it be contained? Additional information is needed on storm water removal system capacities during overtopping and potential liquefaction induced dike failures

(11) Information is needed on compliance of the various aspects of the Subphase 1A project with special flood hazard zones, the new NFIP/FEMA maps, and requirements of the City of San Francisco. An explanation of the project project's benefits in terms of potentially reducing deformation and lateral movement of existing material into the Bay during earthquake loading could be useful. An explanation of the potential impacts on the Bay with and without the project could also be interest.

(12) For compliance with BCDC SF Bay plan policy # 3, a plan is needed for installation and maintenance of seismic instrumentation to record the response of critical structures associated with project SubPhase1 A. In this regard, the ECRB encourages the applicant to develop a plan in conjunction with the Strong-Motion Instrumentation Program of the California Geological Survey to record the ground response at the Ferry Building, the response of the DSM barrier structure, the ground motion at the base of the causeway, and the strains and the relative motions of the dikes and retaining structures associated with settlement and lateral movement. The applicant is encouraged to consider the installation of two arrays of borehole sensors from the surface to the base of the DSM barrier at the time of installation of the DSM trench to minimize costs of installation.

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 February 26, 2015.