San Francisco Bay Conservation and Development Commission
455 Golden Gate Avenue, Suite 10600, San Francisco, California 94102 tel 415 352 3600 fax 415 352 3606

TO: All Engineering Criteria Review Board Members

FROM: Lawrence J. Goldzband, Executive Director (415/352-3653; larry.goldzband@bcdc.ca.gov)
Rafael Montes, Senior Staff Engineer (415/352-3670; rafael.montes@bcdc.ca.gov)

SUBJECT: Approved Minutes of May 24, 2017, BCDC Engineering Criteria Review Board Meeting

1. **Call to Order.** The meeting was called to order by the Chair, Dr. Roger Borcherdt at 1:00 p.m., in the Benicia Conference Room at 455 Golden Gate Avenue, San Francisco, California.

The following Board Members were present: Dr. Roger Borcherdt, Board Chair, Robert "Bob" Battalio, PE, Professor Mary Catherine Comerio, Richard B. Dornhelm, PE, Lou Gilpin, PhD, CEG, and William Holmes, SE.

The following Board Members were not present: Professor Martin Fischer, James "Jim" French, PE, GE, Professor Jack Moehle, Frank Rollo, PE, GE.

BCDC Staff Members present were: Mr. Brad McCrea, Regulatory Director, Ms. Jaime Michaels, Chief of Permits, Rafael Montes, Senior Staff Engineer and Board Secretary and Elena Perez, Permit Analyst.

The audience included the following: Cleve Livingston, Laconia Development LLC, Sam Yao (Simpson Gumpertz & Heger - SGH), Jeff Fippin (ENGEO), Pedro Espinosa (ENGEO), Todd Bradford (ENGEO), Jason White (BKF Engineers), Justin Aff (CMG Landscape Architecture), Maximo Argo (SGH), Luther Greene (RYC), C. Michael Lederer (Brickyard Cove)

Mr. Montes addressed the meeting's agenda as well as some housekeeping items at the start of the meeting; those included location of restrooms, exits and several other items.

Chair Borcherdt called for introductions.

During introductions Board Member Battalio stated that Environmental Science Associates had done work for the City of Richmond. He asked the applicant if they felt he should recuse himself from the project review and discussion.

Mr. Livingston, the project representative, stated they would have no objection to getting Mr. Battalio's input on this project and would appreciate it.

info@bcdc.ca.gov | www.bcdc.ca.gov
State of California | Edmund G. Brown, Jr. — Governor

2. **Approval of Draft Minutes for March 21, 2017, Engineering Criteria Review Board (ECRB) Meeting.**

MOTION: Board Member Comerio moved approval of the minutes, seconded by Board Member Holmes.

Board Member Dornhelm noted on page 2, the second paragraph from the bottom mentioned the Alameda Naval Weapons Station. He stated it was the Alameda Naval Air Station or Naval Aviation Depot, not a Weapons Station.

Board Chair Borcherdt corrected page 15 as follows:

Strike the last paragraph beginning with "He requested the Board's comments ..." and replace it with two short sentences saying: "The Board unanimously recommended that a letter be drafted to acknowledge the contributions of the CSMIP program. The Chair volunteered to draft a letter for review by the Board that would be sent to the Executive Director of BCDC for consideration."

In the second paragraph change "The Chair acknowledged" to "The Board acknowledged" because there was quite a bit of discussion among the Board Members as to that item. Also move the last sentence in that paragraph beginning with "He mentioned briefly the ongoing work ..." to the preceding paragraph.

In the first paragraph strike out the words "disclosed his intervention" and replace that with "indicated his participation at the invitation of BCDC."

At the end of that sentence also add the phrase "as documented in the technical specification letters provided by CSMIP and the applicant."

VOTE: The motion carried unanimously.

3. **Approval of Letter of Acknowledgement for Outstanding Contributions of CSMIP.**

MOTION: Board Member Dornhelm moved approval of the letter, seconded by Board Member Gilpin.

VOTE: The motion carried unanimously.

4. **Board Discussion: Latitude Project (formerly referred to as the Terminal One Project) (Pre-Application).** Board Chair Borcherdt announced: We had an overview or introduction to this project at a meeting on June 7, 2016, which was a joint meeting between the ECRB and the Design Criteria Review Board. So with that said I will turn it over to the Applicant.

Mr. Livingston addressed the Board: We look forward to getting your input and guidance on the engineering criteria issues that we are dealing with on this project.

This is a project that will involve the repurposing of the Terminal One wharf that is over 100 years old. That repurposed wharf will serve as the centerpiece for a waterfront park that will extend the length of the shoreline of the Terminal One Project.

The structural engineering issues that will be raised by this repurposing of the wharf will be addressed by Simpson Gumpertz & Heger and we have two representatives from SGH here today, Sam Yao and Max Argo. They will be presenting a discussion about the condition assessment that they performed on the wharf and the shoreline protection and in addition they will be sharing their thoughts regarding the results of the structural analysis that they have performed on the wharf.

They have been working very closely with our geotech engineer ENGEO. Here today from ENGEO we have Jeff Fippin, Pedro Espinosa and Todd Bradford. Jeff will be sharing with you some thoughts about the geotechnical analysis that they performed on the site and will also be discussing the deep- soil mixing technology that we will be using to stabilize the shoreline.

In addition, ENGEO and SGH have been working with our civil engineering team that is from BKF to do the analysis that is going to be critical to the improvement of the site. We have here today from BKF Jason White who will be sharing with you some thoughts that we have regarding how we are going to be dealing with the flood risk that is associated with the development of the site as well as with sea level rise.

And then finally we have with us today Justin Aff from CMG. CMG is the landscape architect on the project and I think it is fair to say that the success of this project will largely depend upon the creativity of CMG in terms of coming up with improvements that will significantly and dramatically enhance the public access to the shoreline.

I would like to run through some slides that will give you some historical context for this project and also will set the land use planning stage for the discussion of engineering criteria that will follow.

This first image is a representation and it shows the location of the project site in Richmond. The site is located on a 13-acre portion of land that extends into the Bay. It is a very unusual site, a unique site, because it provides not only panoramic views of the entire the Bay but it also provides direct and intimate opportunity for connecting with the shoreline environment.

This is the Richmond Bridge. Over on the east of the project site is the Rosie the Riveter museum and the reconditioned Ford plant. This is where the new ferry terminal will go. So the property is extremely well-situated in terms of the regional transportation grid.

This is an aerial view with the land plan that we are proposing superimposed upon the adjacent property areas. Again, this project site is highly unusual in the sense that it is surrounded on three sides by natural elements.

The port facility that existed on the site was originally constructed in 1915. There is a 94,000-square-foot warehouse that still exists on the site. This property is all built on fill and there were some additional industrial related uses along the shoreline but Brickyard Cove was undeveloped at the time.

The Yacht Club has been built immediately adjacent to the Terminal One site and then the Cove has been developed into a residential enclave to the east of the Richmond Yacht Club.

This is a picture of the site after the Richmond Redevelopment Agency took steps to prepare the site for redevelopment. In 2002 the Richmond Redevelopment Agency removed the storage tanks you saw in the prior picture and they took steps to remediate the existing contamination on the site. Those activities continued until 2008 and the site has remained relatively unchanged from 2008.

Although the storage tanks have been removed the site is largely still covered by hardscape elements, foundations for the tanks and other materials that will need to be removed as the property is redeveloped.

There are really two major elements of this project. One is a residential development that consists of 316 residential units.

There will be 295 condominium flats and those flats will be located in five four- and five-story buildings that are spread across the site.

The second major component of the project is the waterfront park and that park will extend from the intersection of Dornan Drive and Brickyard Cove Road down to the south and wrap around the residential portion of the site and come back in and reconnect with Brickyard Cove Road.

It includes a couple of very important features:

One, an extension of the Bay Trail, which will pick up Brickyard Cove Road here and run along the outside of the loop roadway that we will be constructing to provide automobile access to the waterfront. The Bay Trail will run along the outside of that loop road repurposing the wharf into a public park feature that will be the centerpiece of this waterfront park.

All of the development within the 100-foot shoreline zone is intended to accommodate the public and public access.

We are currently working on improvements for the wharf. We would like to have a wood deck here and it will provide an elevated platform for viewing and perhaps some picnic activities.

We will have a little, informal amphitheater here. It is just a small, informal area where you could have a wedding or something of that nature.

We will have a turf area which will support some informal activities and a garden section of the deck that will provide some almost-native habitat for viewing and sitting.

So with that what I would like to do is turn the podium over to Sam Yao. Sam will introduce the various elements of the project that we are here today to discuss.

Mr. Yao made the following presentation: Cleve just gave an overview of the project, he is from Laconia.

CMG is going to talk a little about the landscaping, that's a work in progress.

Then the geotech ENGEO is going to present the geotechnical exploration findings and I will talk about the structural condition of the wharf.

Then Jason White is going to discuss the flooding and sea level rise issues.

I will then pick it up with the shoreline protection assessment.

Justin, do you want to say something about it?

Mr. Aff replied: The landscape design for the wharf is a great amenity for providing public access to the waterfront as well as a variety of other amenities that Cleve described.

The main program spaces sit within the historic footprint of the warehouse and it is planned to raise that up between 18 and 24 inches to accept soil, structural elements for the deck and drainage. This design was presented to the BCDC DRB approximately one year ago and we are currently revising the design as we coordinate with the structural and geotechnical engineers and we will be coming back to the BCDC Design Review Board within the next few months to present the revised design.

Mr. Fippin commented: I am Jeff with ENGEO and I will talk about the geotechnical aspects of the project.

ENGEO has completed reports at the site in 2003, 2014 and 2016 and another geotechnical engineer worked on the site in the late '90s.

So most recently we performed three deep borings, one behind the wharf, one in the middle of the wharf through a rock dike and one out near the water's edge as well as some CPTs to better inform the shoreline conditions from a geotech perspective.

So based on all that data we created this cross-section, which is very representative of what we see across the site. The site was built about 1915; so when they first developed the site they reclaimed the land. All of the site at the wharf and the backlands are all reclaimed land; fill over what used to be the Bay.

To first reclaim the site a dredging process was performed; a slot cut was made and then it was backfilled with a rock dike. There is an as-built that shows young Bay mud below the bottom of the rock dike. To confirm that we drilled through the rock dike and confirmed that there was 10 feet of soil below the rock dike, which matches very well with what the as-built documents show. So they placed the rock dike and then put fill behind it, which is a mix really close to the rock dike, it is a mix of Bay mud dredged from here and rock from the hillside cut. As you move further inland it is mostly rock cut.

In this cross section the original shoreline is 1,000 feet beyond the edge to the right. Bedrock at the site is primarily rocks associated with the Franciscan complex.

We did some slope stability analysis. Under a static condition this slope is plenty stable but under seismic loading, under the design earthquake under the current California Building Code we identified both with circular and non-circular failure surfaces a failure surface that wants to travel along the young Bay mud and go well back into the site hundreds of feet and could perhaps displace on the order of feet, which would be unacceptable for both the wharf as well as the structures that we want to build behind; so our mitigation for that is to build a buttress in the ground by doing deep-soil mixing behind the rock dike. The rock dike essentially wraps around the entire shoreline.

It is relatively of similar configuration. So it essentially forms the entire shoreline and then rip rap was placed atop it. So the rock dike itself is a mix. It is a mix of sand and gravel.

We know from the as-built that they cut a slot that looked like this. The bathymetry shows that there is soil there so we have just put in soil that has very low strength and a tiny bit of weight in our modeling because we know that it was in-filled probably through accretion. In our strength modeling that has very low strength. And we drove through it too; we have a boring that was over here and encountered very low strength, very low density soil.

So because of the extremely soft soil at the site and the complexity of having relatively shallow, relatively hard bedrock, we did a site response analysis to determine if the soft soil would affect the ground shaking at the site for structural analysis.

We followed ASCE 7-10 procedures where we came up with a target spectrum that represented bedrock based on the design spectrum from the Building Code, selected five representative ground motions that are commonly used in the area, matched them to the target spectrum and then did one-dimensional linear equivalent site response analysis and came up with different time histories at the ground surface as they traveled through the soil column; and then compared those per the building Code to the 80 percent mapped design response spectrum and came up with the enveloping curve that doesn't go below the 80 percent.

So what we see is that if you were to compare the site class E; at the really low periods the peak ground acceleration is slightly below the target spectrum. In the periods of interest of the structure, somewhere around 0.75 of a second out to about 2 seconds the average spectrum is greater than the Code. Then at really long periods it wants to be lower than the Code and we just match it to the Code.

I think it is soft soil over stiff rock. It comes up with something that in the period of interest of this structure is higher than what the Building Code would tell you. We think we got the spectrum dialed in really well and then SGH used that for their structural analysis.

Mr. Livingston commented: I think what we said at the June 7th presentations were that we had not found the as-builts. BKF went through their files and found as-built plans.

Mr. Yao continued: I will discuss our assessment of the existing structural conditions.

The wharf if about 550 feet by 91 feet and right now there is a warehouse on top of that.

The structure is a very thin concrete deck, about 5 inches, supported on beams and girders and supported by over 500 piles.

This is a general overview cross-section of the wharf. Those piles are essentially on a 10 feet grid. There are some timber piles, battered timber piles. Essentially it's meant to be designed for taking the berthing loads. It used to be berthing a lot of ships and barges.

The back of the structure seismic design is a column supporting a pile cap and supporting a timber pile.

On the land side of the wharf there is a bulkhead wall, a concrete bulkhead wall divided by the 20 inch by 20 inch square columns.

This is the typical bands in the middle. Those beams are running transverse and longitudinal and have a cover connected with the pile underneath and that is the pile cap running.

If you look at the spirals on those piles; those are very much close to our modern designs 100 years later. Back then there was no concept of hinge.

So we have all the details. We know the wharf very well.

We did a core through the deck, through the pile and through the beam. The piles starting from the mean low water up to the underside of the deck, including some of the beams, had shotcrete for protection.

You see some of the cores we have taken are from the concrete piles. Here we found the rebar and we are coring between the rebar. That is the real concrete pile.

This is the core through the back. We have asphalt on the top and the concrete on the deck.

The strength is not real high but it is pretty common by the standard of 1915, 3,000/4,000 psi.

We rated each pile according to the American Society of Civil Engineers Manual of Practice for rating the piles. As you look at this, those are the standards of the industry.

If we rated the pile as moderate deterioration it means that you would see cracks over it.

But if it is major it means that you see spalls, you have very large cracks coming up but it does not show the rebar.

Once you have a major spall exposing the rebar we call it severe. Severe is a wide range. You can show that the concrete spall exposed the old rebar up to the whole section. As we show here the whole section of concrete is gone.

So it is a very wide range but the ratings follow the standard. This is the summary of the pile ratings we found.

This is the shotcrete, here is the shotcrete. But right in the front there is no shotcrete. My guess is by the time they did the shotcrete it was already broken, essentially broken. But this part is very, very near to the wharf, there was not shotcrete. I would say 60 to 70 percent of the piles in the tidal zone were shotcreted to protect the concrete because the tide went up and down. But this is way above the tide.

We did not cut the rebar. But based on the cores we have taken, passing through the shotcrete to get to the reinforced concrete, the strength is good, the concrete was not deteriorated. We did not look at the rebar. But at this point the majority of the piles had 4 to 7 inches of shotcrete protecting them. If the shotcrete stayed around for 10, 15 years there is a significant corrosion of the rebar, that shotcrete is not going to be able to contain expansion of the corrosion, it is going to crack.

We cannot say that partially corroded rebar was protected from further corrosion once they put the shotcrete in. However, the concrete testing shows that the chlorides, the salt intruding into the piles exceeded the threshold for the starting of corrosion within those piles.

The testing report we got back said that there is nothing irregular in the original concrete pile.

So I want to give a summary of the rating: 33 piles are Minor. The majority of the piles are in Moderate condition. Then we have got Major deterioration of about 20 percent of the piles. Then the Severe deterioration is less than 5 percent, about 30 piles are in Severe condition.

It is very hard to tell the conditions of the timber piles. But in general our analysis does not necessarily depend on the timber piles. We understand those timber piles stay around for 100 years. We are not intentionally assigning a significant variance to those timber piles.

The timber piles were put in for the berthing loads.

We looked at the Severe condition of certain piles. Among the Severe rating of piles there is a wide range of conditions. As we move ahead to finalize our design we want to keep this in mind.

We will determine later how many need to be replaced. Right now we are just presenting the conditions. It will very much depend on what we do with the wharf, how much landscaping is on top of this. But in general we would like to repair the Severe damaged piles to meet the safety requirements of the California Building Code.

From a structural perspective, without information from the geotech, from a structural perspective we are working with the landscaping architect to try to come up with a 35 percent design but we are not there yet.

Mr. Montes commented: My assessment is that it will come back to the ECRB for a full assessment of the geotechnical and structural aspects.

Mr. Yao added: I would say that what we present to the Design Criteria Review Board today is what we are going to stick with for a 35 percent design, 75 percent design and a 100 percent design. I will come back and present the seismic criteria and just the design criteria. We are going to stick with that as long as you approve it. But the 35 percent results, typically you are looking for acceptance, approval. We are not quite there yet but we are almost there.

We are looking for your input now at our 35 percent design stage.

Board Member Battalio asked: For those piles that are moderately damaged would you anticipate that they may progress to a more severe level of damage through spalling or other action during the life of the project?

Mr. Yao replied: Laconia hired us to finish the design. I will serve as the structural engineer of record. I would make a formal recommendation to Laconia that you need to have periodical inspections to monitor the deterioration of the piles. Of course all the piles deteriorate, some of them faster, some of them slower. Even the California Building Code, Chapter 31F, does have a recommendation based on the condition of the pile, how often you need to do a pile inspection. So we will as part of our design package give a recommendation to Laconia for inspection and give them all the pile inspection records we made so either we do it or somebody else can do it; that's the base.

Board Member Battalio clarified: So the assumption then is that the moderately deteriorated piles would have a capacity or whatever, exceeding whatever design level is required?

Mr. Yao agreed: That is correct. We think that the shotcrete, the strength is reasonably high. The shotcrete contained a wire mesh. But our structural assessment ignored the shotcrete, just assumed the original concrete piles with 4,000 psi strength, so that is our base. But however you do have shotcrete in a certain portion of the piles; that is a given fact. I think that in general that the piles that do not have section loss, they probably have the capacity to carry the loads. But you have a certain amount of section loss. You are going to reduce the pile capacity. That is the way we would approach it.

All 30 piles over here on this table are rated Severe. But within the Severe rating there is a degree of deterioration so we divided it into three categories within the Severe rating, the 30 piles. Only 3 piles have section loss of over 50 percent, another 3 piles with section loss of 25 to 50 percent, and the rest of them are less than 25 percent.

Board Member Holmes commented: I am just looking at the description of Major. It includes multiple cracks and disintegration due to chemical deterioration. That sounds like something that maybe should not wait for inspection, it sounds like that should be fixed or deterioration stopped immediately.

Mr. Yao agreed: I agree with you, Mr. Holmes. I think that when we approach the pile repair we work with the owner. You study not just the strength repair but start with your preventative measures early. Actually long-term you are saving money. But their approach, people just say, "I only repair what is necessary." So we will work out the options. As long as it is meeting the safety requirement we have multiple options for them to choose. But my personal experience is that if we take the preventative measures early, actually long-term you save money, you save money by maintaining.

Board Member Holmes asked: What does the ASCE guide that you are taking this from suggest about Major?

Mr. Yao explained: The ASCE guide does not recommend a set measure for the Major category. ASCE is just to have apples-to-apples comparison, everybody is going to rate it the same way, have very specific criteria, you rate this Major, you rate this Severe, you rate this Moderate so we have a common language.

In 2005 that inspection was very brief, was not rated according to standard industry practice today so it is very hard to make a comparison. But in general we do see that the structure does understandably deteriorate over the last 11 years.

Board Member Holmes continued: But just looking at these descriptions, I can see Moderate being put off and inspected, that makes a lot of sense, but the description of Major did not appeal to me that way.

Mr. Yao added: Well, the structural assessment, it is debatable if the Major deterioration constitutes a major defect on the structural capacity. But in general we like to address this before it becomes Severe. So that is my experience. The first option is some owners do not want to do that, they want to delay. That is an option, still meeting the Building Code.

We are in the process of doing a capacity analysis with seismic load.

Board Member Battalio stated: I guess the question was whether or not the proposed loading is less than the wharf loading. I guess it might be vertically but I do not know about the lateral.

Mr. Yao replied: Well, this is a very important point I had planned to address later but since you brought it up I will just say this: This wharf was designed for very, very heavy industry loads. For the future public access we will never see such high loads.

But saying that, the public safety should not be put in danger, we still need to meet the Building Code; but the wharf is very sturdy. It was designed for loads several times higher than what we plan for. So the loads we are going to put on the wharf are much lighter.

We take the whole thing into consideration but we are going to prove that we are meeting the Building Code. We will still keep the wharf healthy, at least for several years for the next inspection. That is the goal.

I will present data relating to seismic load a bit later.

Board Member Dornhelm commented: So I infer from this that the development remains responsible for the maintenance which includes the future inspections of the wharf so that we know that what we are counting on being done to protect the public spaces, rather than just the private spaces, will be addressed in the plan that you prepare.

Mr. Livingston stated: That is correct.

Mr. Jason White of BKF Engineering addressed the Board: I will talk a little bit about sea level rise and flooding at the site.

We performed an analysis as part of the EIR for the project for designing the site elevations relative to future sea level rise.

The basis for analysis was an updated FEMA FIRM panel issued in 2015 which set the worst-case base flood elevation for the site at 11 feet for Zone VE, which is a zone that is subject to induced velocity action by a 3-foot or higher wave. So that 11-foot base flood elevation is the starting point for our site elevation.

Then what we did is we took a look at data from BCDC and NOAA for predicted sea level rise. That report indicated that sea level rise was predicted to rise by 16 inches by 2050 and 55 inches by 2100.

So what we have done is we have designed the site to the mid-century and we would intend to employ adaptive measures after mid-century if sea level rise occurred beyond that or when it becomes an issue.

So that 16 inches by 2050 plus the base load elevation of 11 feet gives us a starting point of 12.3 feet. Then we went with the Building Code recommendation for 2 feet above that so our minimum finished floor elevation for the site is 14.3 feet; so 3.3 feet above the base floor elevation of 11 feet. We are actually a couple of tenths higher than that, we are at 14.5 feet.

In addition we have the Bay Trail around the perimeter of the site, which we are also setting at 14.5 feet; the Bay Trail which connects to the wharf park. The Bay Trail will be at 14.5 feet. Currently the wharf sits at an elevation of approximately 13 feet. With the landscaping improvements that Justin mentioned we are going to be plus another 18 inches to 24 inches, which will put us 14.5 feet to 15 feet.

Mr. Aff added: Except for at the waterside part.

Mr. White agreed: Correct. The public improvements, Bay Trail, the majority of the park are also at the 14.5 foot elevation.

Board Member Comerio had a question: Is there a reason why that piece at the Bay side is lower? I'm confused. Why is that?

I don't know how deep it is. Why is that prominent? I am curious, why is it at a lower level.

Mr. Aff explained: The idea is to keep this 15 foot wide promenade at the Bay side at that lower level. I think we are going to have as part of the improvements, in the range of improvements we are going to have an additional 6 inches of concrete topping slab on top of that, so it will get raised up a little bit.

Part of the design idea for the park was to have this raised section sit at the historic footprint of the wharf warehouse. One element of that is it marking the historical use of the site.

Mr. Fippin added: The promenade kind of mimics the existing walkway between the wharf and the building.

Mr. Aff continued: I don't know from the point of view of sea level rise if there is a problem if we get 55 inches of sea level rise in 50 years there will be some overtopping of that slab on that edge.

Mr. Livingston continued: I think the main idea was to create sort of an active promenade along the waterfront where people could walk and stroll and basically take advantage of access directly to the water and behind that create these sort of different elements that would focus on different uses of the land, again within the building footprint of the existing warehouse. So it was to try and create a promenade that would be complemented by what we commonly refer to as rooms that are built within the footprint that would be more passive use.

Board Member Battalio commented: I do have some questions or comments. Let me just give you a heads-up on a couple of my thoughts and we can address them whenever you want.

First of all, the 16 inches and the mid-century time horizon are consistent and maybe a little higher, the 16 inches is a little higher than some of the more recent guidelines and so that is fine but the mid-century time horizon seems a little short for a development that will probably be there much longer than that.

So I guess from that perspective, given that 16 inches is a little high for mid-century with recent guidance, the actual amount is probably low because you really should be looking farther into the future. A lot of what we are seeing now is people using somewhere around a 3 foot value of sea level rise, which is somewhere in the 2070 to 2100 or maybe even a little later than 2100 time frame. Maybe it is high by a little bit but that is the way the sea is going.

So that concerns me a little bit that you are using 16 inches and justifying that as 2050, which I think is both low and soon, in my view.

Mr. White clarified: We are using the 16 inches plus 2 feet.

Board Member Battalio replied: Yes. I do not know where you got the 2 feet but the good thing about adding those 2 feet, which is probably something you did early on which is great, as an engineer I think that was smart, but you are still a couple of tenths of a foot or half a foot shy of adding 3 feet plus some sort of freeboard.

You know, it's funny, when we design beams and the like we usually figure out what we need and over-design a little bit. I think freeboard is not a bad idea, even if we are adding in sea level rise. There is not super clarity in the practice on all that but I would assume from what I have seen that a 3 foot sea level rise as a design criteria would be consistent with what I am seeing elsewhere in the Bay and then some level of freeboard above that, which means that your 14.3 is a little low because 11+3+1 would be 15. Nothing is that prescriptive but that is my first thought.

The second thought is the shore elevation away from the wharf is at 8 feet. I guess that is NAVD or mean over the water, which is kind of close. So that means that part of the shore is over-topped with a 100 year design or a FEMA flood event run-up of 11 feet. So that means you have water rushing landward, so that needs to be addressed.

Mr. White answered: Right. And I think SGH has some improvements that they are proposing. They can speak to that in a moment.

Board Member Battalio continued: On the adaptation we are really looking for a little more detail than what I have seen so far. Maybe we will hear about that in terms of how you adapt. I think there is a reference to a wall on the edge, which is one adaptation strategy. But a problem with a wall, especially the edge, is that it is overtopped and then you have water on the other side of the wall.

What some people are doing is having a place where the water can overtop and run back. The adaptation strategy should actually look like it fits and have some sort of convention to it so that it looks feasible. Not to say it is not but we do not know what it is. Just saying there is a wall, in my mind it is a wall, block, drainage one-way; if it is overtopped it cannot get out. And it is more likely to be overtopped if it is seaward. And it has to be higher, which obstructs the views, which in my mind means that 50 years from now it might not happen.

Mr. Livingston inquired: When you suggest that we use the 3 foot criteria as a design criteria; are you tying that to any particular period of time?

Board Member Battalio explained: Yes, I would say that there is a risk that could be achieved or realized by a time frame somewhere between 2070 or 2130. But do not hold me to that, I would have to look at the curves. We all talk about what is going to happen at certain time horizons but the other way to look at it is to take a certain height, then you have a very wide range of potential time horizons depending on what occurs; we do not really know.

But what I am seeing is that - and this is consistent with what the ECRB has told other folks before you all - is that the 3 feet seems to be reasonable. It is the mid-range projection from the National Research Council 2012 report, which was adopted by the state in 2013, the Ocean Protection Council, is about 3 feet by the year 2100 at the mid-range projected. So that is kind of where the 3 foot comes in.

But if you look at some of the new guidance or other numbers or the new information that is not guidance, that may not happen until later. And there is always a what-if kind of worst case and then it could happen a lot sooner. I cannot be more specific than that in terms of timing.

Mr. Aff asked for clarification: When you said 11+3+1, is the 1 feet freeboard? What is the 1?

Board Member Battalio: Yes, I was thinking freeboard. I think one thing to look at is what FEMA requires of the finished floor elevation or lower structural member, which would be your condition at some future time with certain criteria and whether or not they actually require freeboard.

The other point about a freeboard is on a site like this I am sure there is going to be slopes and maybe even some uncertainty at certain places.

Mr. Aff: Because we would be at half a foot. We are at 14.5 for those elevations.

Mr. White: Is your recommendation for the 3 feet plus 1 foot of freeboard? Is that for the building elevation or the Bay Trail or the wharf or all of the above?

Board Member Battalio: These are my comments, I am not sure the Board has actually provided you a recommendation. The public access area can accommodate some overtopping. One of the adaptation measures could be to close it during an extreme event in the future. A lot of folks are looking at accommodating some of the flooding in the shoreline band that is also used for public access and recreational viewing.

Chief of Permits Jaime Michaels commented: The Commission's current policies on public access do require that it remain, to paraphrase, usable, even in extreme flooding situations. Those policies may change over the next few years but we would want this public access to remain usable.

Board Member Battalio asked: Even during a 100 year event?

Ms. Michaels replied: Ideally. It may not happen but ideally. That is what we are advising. And we know that we have some work to do to make sure it is applicable.

Board Member Battalio continued: In my mind there is some relaxation of design criteria for public access if it is only occasionally impacted and still reusable.

Senior Engineer Montes cited policy: Policy Number 5 of the public access policies of the Bay Plan reads:

"[p]ublic access should be sited, designed, managed and maintained to avoid significant adverse impacts from sea level rise and shoreline flooding."

Ms. Michaels added: It's the goal.

Mr. White commented: The shoreline trail could eventually become a levee. It could be raised to provide protection.

Board Member Comerio spoke: Your plan shows the promenade at a lower elevation and you need be aware of access and design issues as you use it for pedestrians. This will affect all of your other design criteria.

Regulatory Director Brad McCrea commented: It would be helpful to get the Board's advice on how much flexibility we have at the wharf's edge for potential elevations.

Mr. Yao continued: SGH did a site inspection and did some engineering assessments of the shoreline protection. I will address the shoreline protection that exists now.

The existing riprap can be divided into three areas. The first area is directly under the wharf and it is categorized as, "light"; that is, 200 pounds or less. If you go northwest you have pretty big riprap; one-quarter to one ton. The southeast side also has this size of riprap present.

The most exposed area is to the northwest. The slopes in this area vary a bit.

Mr. Montes asked: Are you describing the dike being used as shoreline protection as riprap?

Mr. Yao replied: Yes - correct. We ran a wave analysis and the bathymetry is based upon NOAA's Digital Elevation Model (DEM) for San Francisco Bay.

The Caltrans Shoreline Manual requires protection of a light or small size riprap for the site. We have determined that the riprap at the existing site is adequate for a 100 year storm.

Board Member Battalio observed: You have two different conditions here; you have the shoreline and you have the wharf. At the southeast part of the shore you are looking at it more structurally from an erosion protection perspective. The question then goes back to the flood management for the structures on the shore.

Mr. Yao replied: Our scope of work involves protecting the riprap.

Board Member Battalio added: If the project relies on the shore protection through its project life it also needs to consider sea level rise and the increased overtopping potential. Structural failure potential needs to be addressed if the soil behind it is mobilized.

We would also expect the geotech to take a look at the different slopes to see how stable it might be.

Whatever design criteria is used for the Bay Trail should also extend to the access corridor we see here that goes to the shoreline.

Mr. Yao: We will certainly consider the various comments the Board has made regarding elevations, waves and associated risks when doing our analysis.

Board Member Battalio added: It seems that the water is relatively deep so the added depth of sea level rise should not affect your wind-wave generation or your transformations. But it does affect the elevation of the total water level, the wave run-up, because whatever your run-up value is, it is now on a higher peg. So it does relate to the crest of the structure and the potential for erosion of the soil behind the structure, which is actually part of the structural performance.

Mr. Yao replied: Yes, I understand that.

Board Member Battalio continued: Also the FEMA work is public now, they did publish model wave heights and water levels and stuff so you could always just take a look at that, although I had no comment on your modeling.

Mr. Yao answered: Although the criteria have design for it, the site has historically never recorded a 100-year storm as yet.

Board Member Battalio stated: I think I understand where you are going with that. I do not know to what resolution the FEMA analysis was done and to what extent they considered the wave crest elevation or the wave run-up at the wharf, which is different than the shore. That would be up to you to evaluate if you felt it was pertinent to your criteria.

Mr. Yao continued: The design code we follow is California Building Code 2013, ASCE 61-14. ASCE 61-14 is Seismic Design of Piers and Wharves. We use them as a reference and not a design code. And then ASCE 7-10, everybody knows that is the load design criteria. And then NCHRP 611, that is what ENGEO used for designing the seismic criteria. So we have just four design codes.

Mr. Yao went into varying detail and explanation of different codes and criteria used for the project.

Mr. Yao stated: For seismic load we used 100 percent in one direction plus 30 percent in the other direction. We also looked at kinematic load. The California Building Code allowed us to combine them together.

Board Chair Borcherdt asked: So where does the DSM sit with respect to the failure circle?

Mr. Yao explained: The DSM is sitting over here but there is a much bigger circle behind it if you do not view the DSM. But this circle assumes the DSM has already occurred.

Board Chair Borcherdt inquired further: Is there evidence with respect to how you expect the DSM to respond? A number of different things could transpire. How do you expect it to respond if it experiences a large earthquake?

Mr. Aff replied: So the size that we have given it, our analysis indicates that it is going to displace less than 4 inches, the soil behind it as well as the DSM itself.

Board Chair Borcherdt continued: The DSM is a large mass that is much denser than the surrounding soft soil. From one point of view it could reflect seismic energy in the sense that as seismic waves come in, if it is a large enough structure it is like going from soft soil to hard rock, there will be a tendency for it to reflect high-frequency seismic energy. So the question is; how will that move as the ground around it moves?

Mr. Yao replied: I have worked on sea walls in other projects. That is a very common technology now, designed methodology. It has interconnected buttresses.

Buttresses have been a common way of dealing with it to limit the movement of the ground.

Mr. Espinosa added: DSM, we have used it on Treasure Island and other projects that we have presented to the ECRB. Basically it is a static model where you look at it in two-dimensional as a gravity role. So we believe that it will be a similar case because we have similar soil with the young Bay mud.

Board Chair Borcherdt opined: I would think the DSM would be stronger than the soil.

Mr. Espinosa agreed: Correct. But the incoherence of the waves, they are not strong enough. That incoherence does not create a shear issue on the buttress and the buttress will not break because of that.

Board Member Holmes inquired about the DSM as well: The soil in front of the DSM is what is moving?

Mr. Aff responded: That is what is moving, yes. So the soil behind it is moving maybe some but significantly less than that soil which is moving in front. But the amount of movement is much less than what we predicted without the DSM buttress because we shortened the failure surface. We have a lot less driving force and we are including the pinning forces from the structure.

Mr. Montes inquired: So after a ground motion will the DSM crack and then you have to redo it or what happens?

Mr. Espinosa explained: It is usually designed for the design earthquake so it would not crack under the design earthquake. That is how the majority of the buttresses are designed. In this case we do not foresee any areas where you are going to lose the shoreline and the DSM is going to stay cantilever.

Mr. Fippin stated: So there are four different checks that FHWA requires that end up determining the width of your buttress. That is how we come up to the 1-to-1 depth to width ratio.

Mr. Espinosa added: It is designed for the earthquake and it is designed not to crack.

Mr. Fippin commented: The performance of the soil in front of the wharf is really the critical situation here; which is, how much is the soil going to move below the wharf? We can make that DSM buttress 300 feet wide and it would not make the soil below the wharf move any less.

Board Member Holmes stated: I understand the process. I am not familiar with the design of DSM.

Mr. Aff responded: Right. My response to that would be that we are viewing this from the perspective of protection of the things that are behind the DSM buttress; that is a slope stability issue. And typically slope stability or soils that are not liquefiable soils is performed at the same seismic level that the structures that are affected by the slope is performed.

Board Member Gilpin stated: Pseudo-static analysis of slope stability is usually a factor of safety 1.1 or 1.2.

Board Member Holmes observed: It is not the factor of safety, it is what earthquake are you concerned with.

Mr. Yao continued: So much for the kinematic load evaluation; we just explained the methodology.

I came to the Board and presented the same methodology last time on the Brooklyn Basin; the methodology essentially was accepted on that wharf; and Mr. Holmes, you were there, I remember. So I am going to skip this now.

Inertial loads are essentially a non-linear pushover. We have three points to define the performance of the structure: one is initial yielding of the structure; the second is Level 1 performance, that is required for immediate occupancy; and Level 2 is when the strain limit of the material is reached, the concrete strain limit as defined. At that limit the concrete is going to be crushed so we are going to stay away from that.

Based on this we figure out the demand. The demand is explicit because it is a nonlinear analysis. It is an explicit iteration. Essentially it is that we guess the displacement, we reach the equivalent energy behind this. It is an iteration process until the stiffness is converged and they say, hey, this is the demand, the displacement demand, and compared with the capacity, the Level 2 displacement capacity, what is the demand for capacity ratio? That is essentially the amount.

We are still interacting with the landscape architect. The inertial load analysis heavily depends on how much weight you add on top of the wharf. Of course the landscape architect wants to do good work; they want to make the park beautiful but also add a lot of weight on it.

But what I can present to the Board today is that based on this analysis, the initial analysis with some assumptions of landscape design, what we found out is that this slope coming down has riprap on it and below the riprap is a bunch of gravel fill over here. Based on this analysis that last four rows of the piles attract 80 percent of the lateral loads. The piles in the front, they are just going along for the ride. But the stresses are on those piles; the last three rows are critical. They could potentially shear off. So that is what I am trying to say here is that this column, 20 inch by 20 inch column, plus another two or three rows of piles, are critical to the seismic resistance.

Now, in case the landscape design exceeds the capacity of the wharf; then what are we going to do? I am going to present to you the anticipated recommended option. We haven't reached a conclusion yet but I want to present it.

Board Member Holmes opined: Also you get a big torsion from that. You have a big torsion caused by the fact that the entire load goes to those back piles. The mass is way out here towards the middle and then resistance is all on the back side.

Mr. Yao asked: You are talking about torsion for the earthquake in the other direction?

Board Member Holmes replied: Yes.

Mr. Yao: We counted 100 percent/30 percent, then 100 percent/30 percent; yes, so you are right, there is some torsion. But the wharf is long, 550 feet long, so all those 500-some piles are resisting this torsion.

I agree with you, the center of gravity and center of resistance are not aligned.

Board Member Holmes asked: But stiffness for torsion is the same as stiffness for longitudinal, right?

Mr. Yao: That is correct; I agree with you, thank you, but I had to simplify the presentation. This is covered in our 3D analysis, 100 percent in one direction, 30 percent in the other direction and 100 percent in this direction, 30 percent in the other direction.

Board Member Holmes pressed for clarification: Are those 4 piles essentially modeled the same or do you actually show that that first pile almost has no length and then the next one has a little more, the next one has a little more. You are saying that load is taken by those 4 piles. It seems like the load would all be taken by the first pile.

Mr. Yao explained: The first pile takes a lot of load but keep in mind, those are rock. Those are riprap rocks so they are pretty stiff resistance. Yes, there is a distribution. Obviously this carries the maximum loads and this is slightly less, this is slightly less and this is slightly less. But we did see these three form hinges where there is a Level 1 hinge, level 2 hinges. The jury is still out, are we going to need the Level 2 performance? It very much depends on how much weight we put on it so I am not going to give you the conclusion as of yet, we are interacting with the landscape architect.

Board Member Holmes commented: I am familiar with hillside concrete frames that have the same problem; the upper column wants to take the entire load and it is very hard to make them work.

Mr. Yao agreed: You are right.

Anticipated structural retrofit strategies. This strategy has been used in other terminals right after the Loma Prieta earthquake so it is approved and true and for this project it is probably the easiest to do. As we approach this we need to have a concrete overlay on the existing deck anyway. The concrete overlay is for drainage purposes. The stormwater runoff needs to be drained to someplace else. And also the Bay Trail, you have people going back and forth on this plus the wave slamming forces and other forces coming along. A 5 inch concrete deck with maybe some cracks on it is barely making it.

What we propose is one of two strategies.

One is that you have a warehouse sitting over here, you have a warehouse sitting over here and you have another warehouse sitting over here. Those warehouses are going to move away so we have the existing piles, thousands of existing piles. We could have a concrete overlay to tie this wharf to the back so all of those piles are helping these last few rows of piles to resist; share the loads and reduce the load demand on those piles and wharves.

Or we ignore those piles and just drive a new pile and tie it to the deck and share the load and this new pile we are going to design to take the loads. By analysis we will be able to design it, make sure that none of the wharf piles will fail. So that is the strategy I look for if we are going to move forward.

If the existing wharf does not meet the Building Code requirements I propose one of the two strategies moving forward.

Board Chair Borcherdt asked: The length of the wharf is 550 feet, I think. That's almost the wave length of a one second wave traveling horizontally, like a surface wave. So have you taken into account the differential motions that you would expect over the length of the wharf?

Ground motions could be high on one end and low on the other end or vice versa as the wave propagates.

Mr. Yao answered: I understand. We did those things on the Richmond/San Rafael Bridge, on the Bay Bridge. But the standard of engineering practice associated with wharves and piers, we do not get that sophistication and the Building Code does not require it. There is enough structural conservatism in our approach to it.

But maybe there are 56 bands and the majority of the bands are going to have a pile. We have not chosen the pile, we have not done it, but after the Loma Prieta Earthquake the Port of Oakland, all those wharves, we would go back and drive new piles and tie this back and they are still in use right now.

There are three warehouses shown here. Two warehouses are totally on land. There is another one that doesn't show here that is totally on land. One is sitting on the wharf. So those warehouses are sitting on other piles so there are a lot of piles sitting over here. If I need it I just tie this wharf to those piles because those warehouses can be removed.

Some of the piles are going to be bad and we are going to repair them. We are going to cover those areas. Where there is severe damage we are going to repair them.

Mr. Aff spoke: A DSM buttress would be basically constructed by doing overlapping columns of deep-soil mixing that run perpendicular to the shoreline north/south in general and basically create counter-forts that are in the ground that help to resist soil trying to move towards the water. It would go along the entire shoreline band. It would go in both directions beyond the wharf to protect the land behind and increase the slope stability performance of the entire shoreline.

Board Member Comerio: So are those just in-between those existing landside piles?

Mr. Aff: We would try to center the DSM between existing piles in the ground and we would have a layout that would minimize avoidances; that is 60 feet deep by 60 feet wide.

Board Chair Borcherdt asked: Will the DSM go into BCDC jurisdiction zone. Will it be within the 100 foot shoreline band?

Mr. Aff replied: The best performance happens if it occurs as close to the rock dike as possible to make the failure surfaces as small as possible, which would push you wherever the high water line is on this cross-section. You are probably starting some 30 feet from the beginning of BCDC jurisdiction.

Board Chair Borcherdt added: That is pretty standard technology for installing the DSM; it has been done in quite a few places.

Mr. Aff stated: They did do a lot of it at the Port of Oakland.

Board Chair Borcherdt surmised: But just standing back and looking at the big picture now, if we think about this from a BCDC perspective, it seems to me that basically the principal thing that is transpiring in the BCDC jurisdiction zone will be the construction of the DSM zone as a zone to help resist lateral movement or soil failure and contain any movement that might take place landward of that. The other aspect of the project has to do with the safety of the wharf structure and its public safety issue associated with a high occupancy rate at different times. So from an engineering criteria point of view it is important that the public safety issue with respect to the wharf, the sea level rise issue, and other engineering criteria be addressed.

Construction of the DSM will need to provide as much lateral resistance as possible. I am not sure what other input you would want from the Board with respect to the construction of the DSM. I am curious as to how the DSM will respond with respect to the surrounding soil and if there is any historical evidence or any measurements that have been made in the past with respect to how a DSM responds.

I raise that question thinking in terms of the soft soils in Japan and the artificial fills and the quay walls that were installed to retain the liquefiable soils. It turned out that after the Kobe earthquake the pore pressures did really build up pretty high behind the quay walls resulting in some failures. I am wondering whether that same kind of thing might happen in some instances with respect to the DSM. I don't know, that is just a question.

Mr. Espinosa explained: Most of what we are trying to retain is the young Bay mud failures or deep failures, soft soil failures due to the lateral pressures from the earthquake. We do not expect pore pressure buildup on the young Bay muds, which is most of the DSM. There is going to be some pressure buildup on some of the fill that might be potentially liquefiable. But in those areas we are closing the cells of the DSM, which has shown in the same earthquake, in the Kobe earthquake, where buildings that were founded on connected cells of DSM, liquefaction was prevented underneath those buildings, where right in front of that building everything liquefied and went offshore. So that is our scheme for this shoreline protection.

Board Member Gilpin asked: Could you talk about the liquefaction potential for the rock dike? How did you set up the characteristics for understanding it?

Mr. Bradford answered: The material identified within the rock dike was either the previously mentioned 15 pound max size cobbles intermixed with dredge material and young Bay mud. So there was not anything that we identified as being potentially liquefiable within the rock bank itself.

Board Member Gilpin inquired further: So there is no possibility of sand or concentrations of the granular material?

Mr. Bradford replied: The only thing that we identified in any of the explorations along this cross-section was the sort of teal-colored clayey sand; and it was only identified in the farthest-most boring that was done actually out at the tip of the wharf. So we do not have data past the wharf so we conservatively interpolated that as being plainer.

Board Member Gilpin commented: But if you are sampling a gravelly layer it is really hard to find the sand layer because you are pushing gravel in front of it or CPTs, whatever.

Mr. Bradford replied: We did grab samples within the rock dike, we did not just simply core through it and that's how we were able to identify sort of a matrix within that.

Board Member Gilpin asked: It is a pile of garbage, right, basically?

Mr. Bradford responded: Sure. But it was developed with the dredge material that was pulled out of the incision layer within the young Bay mud.

Board Chair Borcherdt continued: I would like to ask Board Members if they have any additional comments but first, Brad, you raised a point earlier and so I was wondering what you would recommend in terms of what you would like to see commented on further.

Mr. McCrea explained: The questions that are outlined in the staff report you have in front of you, on page 8, are a great start.

Board Member Holmes inquired: What do you know about the piles under the landside warehouse?

Mr. Yao replied: Those are timber piles that were installed later because the structures, those two warehouses on the land side were not constructed in 1919 but in the 1930s and 1940s.

Board Member Holmes asked: One of the retrofit schemes you would have to investigate is those piles, right, because you are going to tie to them?

Mr. Yao replied: That's right, we need to investigate. But those are buried in the ground. This is one of two schemes. If we eventually adopt our scheme we do need to investigate.

Board Member Holmes added: The other comment I have is I noticed in the write-up here you are using a knowledge factor of 1 on the piles.

Mr. Yao: On the concrete piles it is 1. On the timber piles it is 0.7.

Board Member Holmes continued: I think you have done as good an investigation as you can do but I am not sure you have complete knowledge of 1, which implies like a new building where everything is inspected. You have got all this major damage, you do not know what the level of deterioration is, so I am not sure you have perfect knowledge of the piles.

Mr. Yao explained: Well, there are guidelines. We did concrete corings. We did pile ratings. It was actually conservative because we ignored the shotcrete. I think we know enough. We have the original design drawings. We can substantiate why we are saying it.

Board Member Holmes clarified: It is not what the rebar and concrete is, it is how much deterioration is there, is my concern.

Mr. Yao explained: But the truth is we ignored the 6 inch shotcrete so that is significant. In my view it is conservative in our assessment of the piles. That shotcrete, it does contribute to the strength of the pile, especially in those hinge areas. Most of those piles do have 4 to 6 inch shotcrete around it.

Board Member Holmes: I think that advantage may be different than a knowledge factor.

Mr. Yao stated: We could go the other way, counting for the shotcrete and reduce the knowledge factor.

Board Member Comerio commented: This is a very thorough presentation. The one thing that is absolutely missing here is a plan of the area that is within the BCDC jurisdiction, which is why we were asking all those questions at the end about those last few slides; graphic questions came up because you need to see where those DSMs are planned. You need to see where they sit relative to those existing piles or that existing building which is being removed. We are not seeing those layers and it has made it harder to understand some of your presentation.

It is really a minor point but it would really have helped to have had that. It would have helped to understand how the promenade works and connects at the end. Looking at these drawings and I know you are not very far with the design yet, I understand that. But just having a sense of how that all links to the land at the edge of the property would be helpful. So just having that, a plan view with all of those elements, including the ones that are to be removed and the ones that are going in, and seeing those layers would be very helpful.

Mr. Livingston stated: We will bring something like that back to you next time we see you.

Mr. Montes added: Part of it was my mistake because originally they were going to present a lot of the landscaping and then I thought that because the ECRB wanted to zero in on the seismic and flooding that they would concentrate more on that portion.

Board Member Comerio explained further: That plan helps us understand the relationships.

Mr. Aff asked for clarification: Just to clarify, you are not asking for the landscape plan, you are asking for the plan view showing where the DSM buttress is, where are the piers, where the piles are, where are the new proposed piles going to be, where the four critical rows of piles are, all those things.

Board Member Comerio added: Where things are going to be removed. Obviously, as you go forward with the design you would have the landscape design on top of that. In order to understand some of the seismic criteria, understanding the full base layer and plan as well as in-section is very helpful because you cannot always see how all the pieces relate to each other.

Board Member Dornhelm commented: I do think it has been very thorough and it would be wonderful when the old warehouse facility finally gives way to a delightful, urban waterfront as you are proposing. But my only reservation has to do with the early assumption of the Level 2 criteria. That you are comfortable that when you say "no collapse" does that mean in such an event the structure is a loss and then does that mean the development will take on the responsibility of cleaning the site? It is all public uses that are being risked in this case and we want to know if something will come back after the site is cleaned. Or can you repair the wharf if it has gone to a Level 2?

Mr. Yao replied: Every major structure is facing that question right after a major earthquake. Obviously the development is going to hire a qualified engineer to go back and inspect them and see how much it is going to cost to retrofit/restore the structure's integrity. How much to just tear them down and rebuild a new one. I want to say that the majority of the wharves and piers we experience the former, going back and retrofitting them and restoring them if you have an earthquake similar to the Loma Prieta Earthquake.

Board Member Dornhelm stated: That is your design earthquake. It is much higher than Loma Prieta. So you are designing it for no collapse.

Mr. Yao explained: The design of the piles is very close to the modern design. So I think the hinge does perform during a major earthquake and provide a lot of ductility to the structure. Of course, if we adopt one of the retrofit options I think it can sustain a much higher earthquake and still become repairable in the future. But it is really out of our scope to really assess reparability of the wharf. I appreciate the comment; as one of the public I want to know. But at this stage we are looking at just one point. A big earthquake line is one point and we design for it and make sure the public is safe.

Board Member Holmes asked: The elevation here of the beams under the platform. I do not see any positive reinforcing continuous. Are you assuming the top fixity of the piles or is that a pin in your model?"

Mr. Yao explained: The beam is extending typically 18 inches.

Board Member Holmes continued: I just do not see any bottom reinforcing. It is bent up to become negative reinforcing, at least in this. It is a small diagram. But is there bottom steel going right through the connect?

Mr. Yao replied: There are some. There are a band of rebars there but there are some continuous spots also.

Board Member Holmes inquired: And are you considering the diagonal sections?

Mr. Yao replied: No. The cover or the transfer is ignored for now in our analysis.

Board Member Holmes opined: That could be a mistake.

Mr. Yao answered: But it is conservative.

Board Member Holmes disagreed: I do not think that is conservative at all. Caltrans found out they had some decorative chamfers on the columns. They thought they would fall off in an earthquake and it turns out they formed a hinge right at the bottom where they did not expect it. Right below the chamfer. So I would take a look at that.

Mr. Yao replied: I will take a look at that, okay. I will let you know next time.

Board Member Comerio commented: As the design is developed it is going to be important to us to understand. The criterion is life safety but what is the egress? Are there gaps between? Do we have any gaps opening up in the wharf? Do we have any gaps opening up between the wharf and the soil? How do people get off the wharf in the immediate aftermath of an earthquake? What do you anticipate the structural movement so that we can be assured that there is safe egress?

Mr. Yao replied: I think we are prepared for that. Because the last time you made the same comment to our company regarding the Alameda Landing Project.

But this project is different from Alameda Landing because I talk about 4 inch displacement, literally 4 inch displacement. The maximum gap you can ever find is 4 inches so people can just step over this 4 inch gap; egress is not an issue. Alameda Landing is a much larger displacement, 16 inches or 20-some inches. That potentially has a fall hazard. But we are talking about 4 inches here.

Board Chair Borcherdt continued: If there are no further comments I think we would like to thank you for your clear presentation and your effort and desire to come before the Board and receive comments. We hope our comments are helpful and we look forward to seeing a successful project.

Mr. Yao commented: I have a quick question here, Chairman. We think that we gave the Board an introduction from an engineering perspective so possibly the next presentation will be shorter. Do you want us to go through the same presentation or do we just make it more concise in response to your comments?

Board Chair Borcherdt responded: The Board actually provides advice to BCDC. We are asked to focus on the engineering criteria that will be used for the project, with a particular eye toward the BCDC jurisdiction zone, safety of fills, and various BCDC policies.

In that regard, as I mentioned earlier, it seems to me that the critical aspects of this project had to do with the safety of the wharf, especially from the point of view of public safety, the impact of sea level rise with respect to the design criteria being applied, and then also the response or the usefulness of the DSM to the assigned purpose, which is to restrain potential failures along the boundary of the development zone.

So if you were to come back to the Board then it would seem to me that this would depend on BCDC with respect to what you want us to provide additional comments on the criteria. But it does seem to me that there are still a number of issues with respect to the safety of the wharf that are still in the design stage. Usually when we see a project at 35 percent we usually see it later. But this project is a little bit different from that point of view and so it is really, I think, depends on what BCDC wants from the ECRB.

Mr. Montes stated: My view of the assessment that you provided today is you provided some good input regarding Mary's comments on the overlays of different landscape versus seismic layout aspects. Bill raised the questions about the chamfer zone where the hinge on the piles can develop. So those are questions that can be answered at the next meeting.

Also keep in mind that the geotechs were not here today so it will be good to have them give their input on the full geotechnical review.

Regarding the sea level rise, I think that Bob Battalio and Richard Dornhelm raised some very good points about the elevation of the wharf, whether the Bay Trail will become a levee in the future.

Board Member Battalio commented: I felt that a longer time horizon and a greater amount of sea level rise would be more typical. Something on the order of 3 feet would be good to address whether or not there is a freeboard included. I personally like freeboards but I am sure you have other considerations.

The adaptation strategy should have a little more description so that we would understand what the implications are to the public access area as well as the efficacy of protecting the development.

Mr. Montes added: One more note, Bob. The State is about to publish the new guidance on sea level rise and perhaps you are more familiar than anyone else about that.

Board Member Battalio replied: I know something about it. I am not involved in the decision process. I do not believe that is going to come on until January of 2018. I will just say what I have heard is that the projections are similar to or maybe slightly lower than the existing guidance for the medium and high curve, the low curve people are not really paying attention to. But relative to the guidance cited in your application, the new guidance might be a little bit lower but then there is this very high potential that is much higher.

Mr. Montes remarked: After 2050.

Board Member Battalio continued: So it is somewhat confusing, I think, so it is going to be interesting to see what comes out of the actual policy. I would say that the 3 feet is something that I am sticking to what we have been saying. I think the 3 feet is a reasonable criterion for design and then an adaptive plan for something on the order of 5 to 6 feet would be reasonable. I do not know that the 9 to 10 feet that people are talking about is something that we are ready to recommend people having to design for at this point.

Board Chair Borcherdt added: Coming back to your original question which was talking about whether you needed to go through the full presentation again. I would think that from the point of view of efficiency of both your time and our time I think the basics will be sufficient, since you have already presented the overall picture at a preceding meeting I would think your next presentation could be much briefer and addressed to only those specific issues that are of real significance here. I think the issues with respect to this particular project are pretty clear.

Ms. Michaels commented: I want to ask the Board something. I can't remember if we have been bringing risk assessments to the Board for sea level rise for you to see and consider. This will be the subject of a major permit application and for all major permit applications the project proponents have to prepare a formal risk assessment to address sea level rise and flooding for the project site. And I just cannot remember at this point if we have been bringing those formally to the Board.

Mr. Montes stated: We have brought it once or twice; once for Tesoro.

Ms. Michaels continued: We will go back and figure out whether or not we have been bringing them to you; and if we do that would maybe be a part of the next presentation if one is prepared by that time.

Board Chair Borcherdt asked: From a permit point of view do you see any of the engineering criteria that need any additional comment?

Ms. Michaels replied: In terms of what might be in the risk assessment?

Board Chair Borcherdt answered: Yes.

Ms. Michaels explained: Well, because sea level rise is an issue at the site and the project will involve some fill in the Bay they will have to prepare one.

At a minimum we have to receive that and consider it and I just cannot remember now if we have been bringing it to you.

Board Member Battalio stated: I would like to see it; I have not seen it. I think to the extent that we are advising on sea level rise criteria it would be helpful.

Ms. Michaels replied: It has not been prepared yet but we will be working with the project proponents to do that.

Mr. Livingston commented: Would it be possible to get the two Board Members who are not present and have the geotechnical expertise, to provide us with some written comments or inputs on the various reports and other documentation that we have submitted? And the reason I am making that request is it would be very helpful to us if we knew what the concerns of the Board were as we move into this next stage of designing our mitigation solutions to the problems that have been identified. Just to make sure that we are not missing anything from their perspective.

Mr. Montes responded: I understand your concerns, but unfortunately we are bound by the laws that preclude individuals or a few members of the ECRB from giving input unless they happen to be all together in a public forum.

Mr. Espinosa asked: Were they able to review the materials since the material was given to them?

Mr. Montes explained: No, since they were not going to participate in the meeting. Therefore they were not able to view the material unless they committed to be in the meeting.

Board Chair Borcherdt added: We have to meet the Open Meeting Act requirements. This is a public meeting so basically any interaction with the Board Members has to take place in a public forum.

Mr. Montes continued: We have to have another public meeting where there will be the geotechs involvement and then the information will be distributed to all of them for another public meeting. I believe the next public meeting will be in August but I will have to double-check on that.

Board Member Gilpin asked: Could it address just the geotech aspects of the project?

Mr. Montes replied: For the next meeting?

Board Member Gilpin answered: If they wanted to bring just the geotech parts of what we looked at today or does it have to open up the whole project?

Mr. Montes responded: No. We have to gather up the whole team.

Board Member Gilpin explained: I understand that. But it could be an agenda item at the next meeting that is just focused on geotech issues for this one project, in addition to the other project that we are going to look at.

Mr. Montes agreed: Sure, yes, we can have the geotech aspect but you all have to be here.

Mr. Fippin commented: I think that the primary concern is that if there are significant comments that require a third visit. I believe the Applicant would like to shorten the time period of review.

Mr. Livingston stated: Right now BCDC is basically our critical path so we are very focused on doing the analysis that ECRB believes is important to support the proposal that we are going to be making from a planning and safety perspective.

We are also very interested in getting the input from the Board so that we can prepare our formal application and get that submitted and get the process going for the formal consideration of that application.

We were hoping to be able to get in front of the ECRB at this meeting and get your full input so that we can go back and submit our formal application based on that input with the additional work that we are going to be doing as described by Sam and Jeff. But I don't want to wait until after the August meeting to put that formal application together and get it submitted.

Mr. Montes had a suggestion: I have an idea. The Alameda Landing that came to the Board in March, you can look at the minutes. You can look at Jim French's comments regarding the DSM and geotechnical aspect of it, so that could give you an idea.

Mr. Livingston agreed: Rafael, I think that is a great idea. Because at your recommendation I did sit in on that meeting and I know that there were some comments made on the DSM that I think would be relatively applicable here. So that will give us something to go with.

Mr. Montes added: I can also provide you with similar projects that had the two geotechs involved, Treasure Island and some other ones.

Ms. Michaels stated: Also I want to add that you do not need to wait until the Board sees this project further for you to submit your permit application. We will accept it at any time and this can go on parallel along with that process.

Mr. Livingston continued: Hopefully we will be ready to come back to the Board for a second reading with our whole program in place and responses to the considerations and concerns that have been raised today by that August meeting, so perhaps we can get on that agenda to follow up on this meeting.

We very much appreciate the input and the opportunity to present the project. We are pretty excited about moving this project forward. We have gotten most of the approvals that we need and BCDC is the last threshold we need to get across so we are very anxious to work with you on that.

Board Chair Borcherdt added: It looks like a nice project, thank you for coming in.

5. **Adjournment.** There being no further old or new business, the meeting was adjourned at 3:56 p.m.

Respectfully submitted,

RAFAEL MONTES, P.E.
Board Secretary

Approved, with no corrections, at the
Engineering Criteria Review Board Meeting August 8, 2017.