1441 Embarcadero Shoreline Redevelopment – Feasibility Study - DRAFT

November 07, 2017

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Contents

Certifications

This report has been prepared by Mott MacDonald under the supervision of a Professional Engineer, including all findings and recommendations.

Name:	Scott Fenical, PE	John Jacob, PE
Signature:	____________________________	____________________________
Date:	____________________________	____________________________

1 Executive Summary

Mott MacDonald (MM), in coordination with TRC Solutions (TRC), performed a shoreline redevelopment feasibility study (Feasibility Study) and evaluation of shoreline redevelopment alternatives for East Bay Regional Park District (District) at 1441 Embarcadero, Oakland, CA. The site consists of a plot of land approximately 385 feet by 170 feet, with the waterside edge of the area bound by an existing steel sheet pile wall bulkhead to the south. The bulkhead extends from the southwest corner of the site and runs for approximately 325 feet along its southern edge. The remainder of the boundary is made up of rock and rubble, with some timber piles and concrete slabs protruding from the eroding fill. The existing bulkhead is in an extremely deteriorated condition and shows signs of advanced corrosion with voids on the face, displacement along its length, and soil subsidence behind it.

Given that the existing sheet pile bulkhead is highly deteriorated, it would be extremely difficult to repair the bulkhead to its original condition. A comprehensive inspection, including dive inspections, of the existing structure would need to be conducted to determine the extents of corrosion and repairs required. Repair procedures would be complex and would require more time than installing a new structure. Hence, repairing the existing bulkhead was not further considered as a viable alternative for shoreline stabilization.

Shoreline redevelopment alternatives considered included a new rock revetment, a new bulkhead, and a “No Action” alternative which consisted of estimating a safe setback distance (within which public access would be prohibited) to address life safety concerns in the event that the existing damaged bulkhead were to fail. The alternatives were coordinated with TRC and the District in a technical memo dated September 7, 2017.

MM performed preliminary coastal engineering analysis to determine coastal engineering design criteria, such as tides, winds, and wind-waves. Coastal analysis indicates that the site is sheltered and storm wave conditions require relatively small armor stone to resist damage during a 100-year storm event, and that tidal currents are negligible. Armor and bedding stone sizes were assumed to be somewhat larger than required strictly for resisting wave attack during extreme wind events, to ensure slope stability and improve constructability. In addition, a site morphology evaluation was performed which indicated that changes in the bottom elevation adjacent to the existing shoreline have been minimal in the past few years. Calculations also indicate that wave-induced scour expected at the toe of either a revetment or new bulkhead alternative would be minimal.

MM also performed preliminary structural engineering analysis to determine stability and design requirements for a new bulkhead, using geotechnical parameters provided by TRC. Analysis indicates that a cantilevered steel sheet pile wall would be prohibitively expensive to resist the design loads and deformations, and that an anchored steel sheet pile system would be desirable to reduce costs. The new bulkhead would consist of a steel sheet pile wall, anchor rods spaced at regular intervals along the length of the sheet pile wall, and an anchor system (sheet pile wall or concrete deadmen) at some distance behind the bulkhead wall. The anchor rods will extend from the face of the bulkhead sheets to the anchor sheets or deadmen, and tie the entire system in place. Some excavation will be required to install the anchor rods; the excavated area will be filled and compacted with competent structural fill.

Results of the feasibility study indicate that both a rock revetment and a new bulkhead are feasible at the project site, and would provide the site with a long-term stable shoreline. The

revetment alternative is less costly (order of magnitude total cost $1.2M, or roughly $2,200/LF under the assumption that excavated soil would not require costly disposal), and likely to have greater longevity with periodic maintenance. The excavation and disposal, plus generator fees, resulting from the Class I soil would result in an increase in total construction costs by approximately 40-50%. However, the revetment as presently evaluated slightly reduces space in the uplands and results in greater Bay fill.

The new bulkhead alternative is more expensive (order of magnitude total cost $2.8M, or roughly $5,000/LF, also under the assumption that excavated soil would not require costly disposal). The excavation and disposal, plus generator fees, resulting from the Class I soil would result in an increase in total construction costs for the new bulkhead also by approximately 40-50%. However, the new bulkhead provides greater retention of space in the uplands, and results in less Bay fill.

The “No Action” alternative is also feasible, but requires a minimum setback on the order of 30 feet from the waterside edge to ensure safe conditions, as well as restrictive fencing to prevent public access to the area near the shoreline. If any significant additional loading is planned near the setback, the setback distance will likely increase. It is recommended that the setback distance be further evaluated in combination with other site improvements, if the “No Action” alternative is pursued.

Given that the Project’s objectives are to stabilize a decaying/hazardous shoreline, and to provide public access, permitting is not likely a significant impediment to shoreline redevelopment for either the revetment or new bulkhead alternative. The “No Action” alternative may meet resistance given the hazard of existing bulkhead failure unless significant analysis is performed to ensure public safety. During the design process, Bay fill and other properties of any preferred alternative can be modified if they become a concern to agencies.

2 Introduction

Mott MacDonald, in coordination with TRC Solutions, performed a shoreline redevelopment feasibility study and evaluation of shoreline redevelopment alternatives for East Bay Regional Park District at 1441 Embarcadero, Oakland, CA. The site consists of a plot of land approximately 385 feet by 170 feet, with the waterside edge of the area bound by an existing steel sheet pile wall bulkhead to the south.

2.1 Site Description

The bulkhead extends from the southwest corner of the site and runs for approximately 325 feet. along its southern edge. The remainder of the boundary is made up of rock and rubble, with some timber piles and concrete slabs protruding from the eroding fill. Figure 1 shows a photo of the site taken from the southeast.

Figure 1. Site Shoreline Conditions

2.2 Existing Bulkhead Condition

Figure 2 shows a photo of the existing bulkhead, looking southeast. Site observations indicate that the existing bulkhead wall appears to have been partially repaired on multiple occasions during its life, however they were likely modest repairs made without any engineering analysis. Despite these previous repair attempts, the bulkhead is in a state of extreme disrepair, showing signs of extensive corrosion with missing sections along its face, and failure of the tiebacks and waler system. The bulkhead wall is bowing and the ground immediately behind the wall is showing signs of significant subsidence.

Figure 3 shows the existing waler which is broken, and showing extreme signs of corrosion. An extremely corroded anchor rod is also visible in the same figure. Figure 4 shows the corrosion in the sheet pile wall and the ground subsidence behind it.

Figure 2: Existing bulkhead bowing and showing deformation

Figure 3: Damaged waler and anchor rod

Figure 4: Corroded sheet-pile wall and ground subsidence

2.3 Consideration of Existing Bulkhead Repair

The existing bulkhead shows signs of extreme corrosion on the surface, and the extents and magnitude of corrosion below the waterline is unknown. In addition, the waler and tieback system has failed, indicating that the capacity of the system is now below that required for stability (as evidenced by bowing). Repair of the existing bulkhead was considered in this Feasibility Study, but not analyzed in detail based on our understanding of activities that would be required. To repair the existing corroded bulkhead, the following would be needed:

- A comprehensive inspection program would have to be undertaken to determine the full extents of corrosion.
- The corroded sections would need to be repaired to where there is sufficient structural section to resist the imposed loads;
- The sheet-pile wall would need to be re-aligned;
- The corroded walers would need to be replaced with new walers;
- The corroded anchor rods would need to be replaced with new anchor rods; and
- The ground behind the existing sheet pile wall bulkhead would need to be excavated, re-filled, and compacted.

Given the extent of corrosion and deflections in the existing structure, it would be extremely difficult to repair the bulkhead to anything resembling its original condition. Construction procedures for repairing the existing sheet pile bulkhead would be complex and would require more time and effort than installing a new structure. Moreover, there is the added cost of performing a comprehensive inspection of the existing structure to determine the extents of corrosion and repairs required, including dive inspections. Therefore, repairing the existing bulkhead was not further considered as a viable alternative for shoreline stabilization.

3 Proposed Alternatives

On September 9, 2017, MM submitted a memo summarizing the proposed alternatives to be evaluated in the Feasibility Study for review by the District, as well as the Port of Oakland. Results of the review by the District and the Port indicated that a living shoreline type solution was not desired, therefore only the rock revetment, new bulkhead, and the "No Action" alternatives are included in this analysis. The memo is included as Appendix A.

3.1 Rock Revetment Alternative

This alternative consists of installation of an engineered rock slope along the waterside edge of the project site. A portion of the waterside edge of the site would need to be excavated, deleterious materials removed, backfill placed and compacted with competent material (possibly including reuse of excavated fill), and geotextile filter fabric installed to prevent future settlement and enable placement of the rock revetment. It is assumed that the existing bulkhead would be removed. Under this alternative, the Bay Trail could run along the waterside edge of the property, and the upland area of the site could be safely developed. This is likely a cost-effective alternative, consistent with surrounding shorelines, and would require minimal maintenance. However, the rock revetment alternative requires consideration of potential loss of upland space, or an increase in the amount of Bay fill.

3.2 New Bulkhead Alternative

This alternative consists of installing a new sheet pile bulkhead, similar in concept to the existing bulkhead, along the waterside edge of the project site. The new bulkhead would be installed seaward of the existing bulkhead, and would likely not require removal of the existing bulkhead. Similar to the rock revetment alternative, the trail would run along the waterside edge of the property, and the upland area of the site could be developed. This alternative would be most costly, would require periodic maintenance, and would likely need to be replaced in 25-30 years. However, this alternative would provide the most space in the uplands for development, and with only a minor amount of Bay fill.

3.3 No Action (Safe Setback) Alternative

The "No Action" alternative consists of determining a safe offset from the existing bulkhead wall that would result in a reduction in stresses at the wall to safe levels, such that in the event of wall failure (which is assumed to occur), the upland developed area would not incur damage. The safe setback estimation is summarized in Section 5.5 for the "No Action" Alternative.

4 Coastal Engineering Analysis

Coastal engineering analysis was performed to develop design criteria for the rock revetment alternative. The analysis includes tides, winds and wind-waves. Tidal currents, winds, and passing ship effects do not control the design of either the revetment or bulkhead alternatives.

4.1 Tides and Vertical Datum

Tidal datum information near the project site was obtained from Alameda NOAA Station (9414750). Water levels are provided below in Table 1. The project utilizes the NAVD88 vertical datum.

Table 1: Tidal Datums at Alameda (NOAA Station 9414750)

Datum	Value [ft, NAVD88]	Description
MHHW	6.37	Mean Higher-High Water
MHW	5.75	Mean High Water
MTL	3.33	Mean Tide Level
MSL	3.22	Mean Sea Level
DTL	3.07	Mean Diurnal Tide Level
MLW	0.91	Mean Low Water
NAVD88	0	North American Vertical Datum of 1988
MLLW	-0.23	Mean Lower-Low Water

4.2 Winds

A wind analysis was performed to quantify the wind climate in the area and develop extreme winds (and subsequently waves) to be used as coastal and structural engineering design criteria. Wind data for Alameda AAMC1 Station were collected from the Center for Operational Oceanographic Products and Services (CO-OPS) for the period 1996-2015 and evaluated in the coastal engineering analysis. Figure 5 shows the distribution of measured wind speed (2-minute average) and direction at Alameda AAMC1 Station in the form of a wind rose. Winds are most commonly from the west. The wind records were analyzed and the largest measured wind events from each direction were extracted. These extreme events were fit to a Weibull distribution, and sustained wind speeds were predicted for extreme events with recurrence intervals ranging from 1 to 100 years from all directions. Figure 6 shows the predicted extreme wind speeds for all return periods and directions (2-minute average). These extreme winds were used as input to a wind-wave growth and transformation model, described in Section 4.3.

Figure 5: Wind speed and direction distribution (wind rose) at Alameda Station

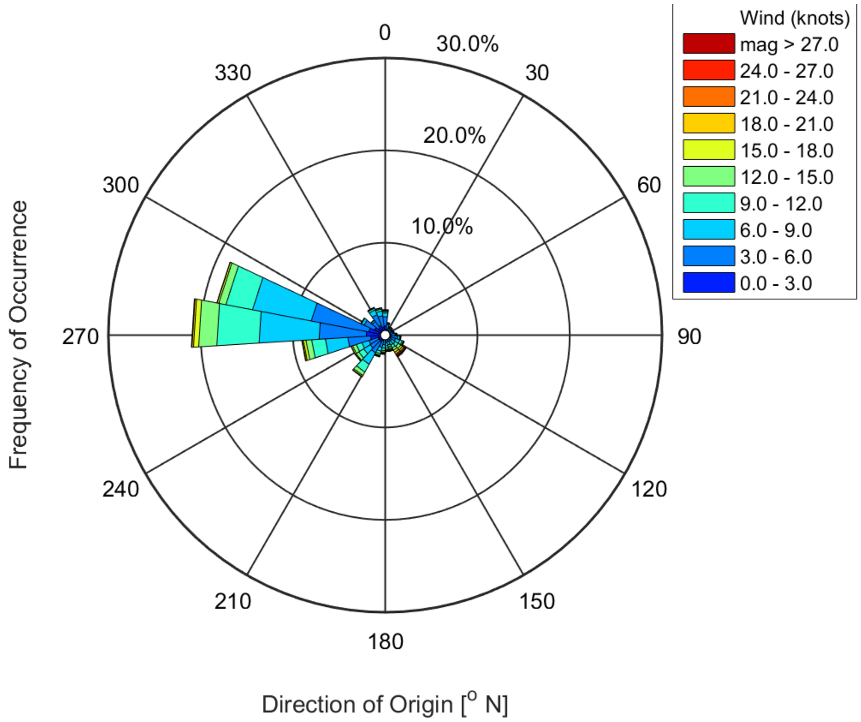
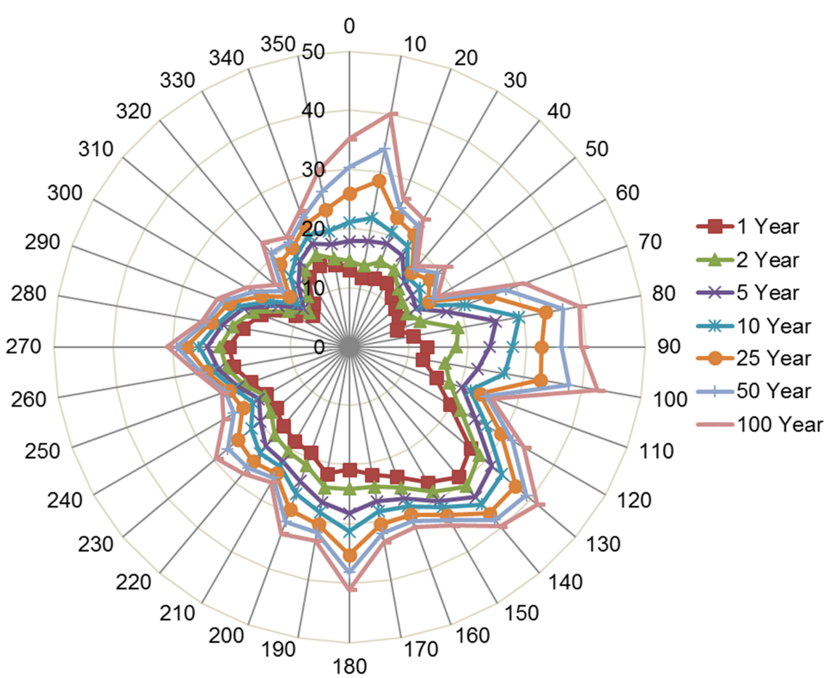


Figure 6: Extreme sustained wind speeds (knots) and directions (degrees) for return periods between 1 and 100 years



4.3 Wind-Waves

The extreme winds developed in Section 4.2 were used develop design waves at the project site. Wind-wave growth and transformation modeling was performed using the two-dimensional spectral wave transformation model SWAN (Holthuijsen *et al.*, 2004). Figure 7 shows the SWAN input bathymetry near the project site, constructed using the NOAA Bathymetric Attributed Grid (BAG) Survey Dataset (NOAA 2013) at 1m resolution.

The results of the SWAN model included spatial distributions of significant wave height and peak wave period, as well as other parameters. 100-year extreme winds were used as input into the SWAN model for all wind directions. All modeling was performed at MHHW (6.6 feet, MLLW) tidal elevation.

Figure 7: Wind-wave modeling domain and color contours of bottom elevation

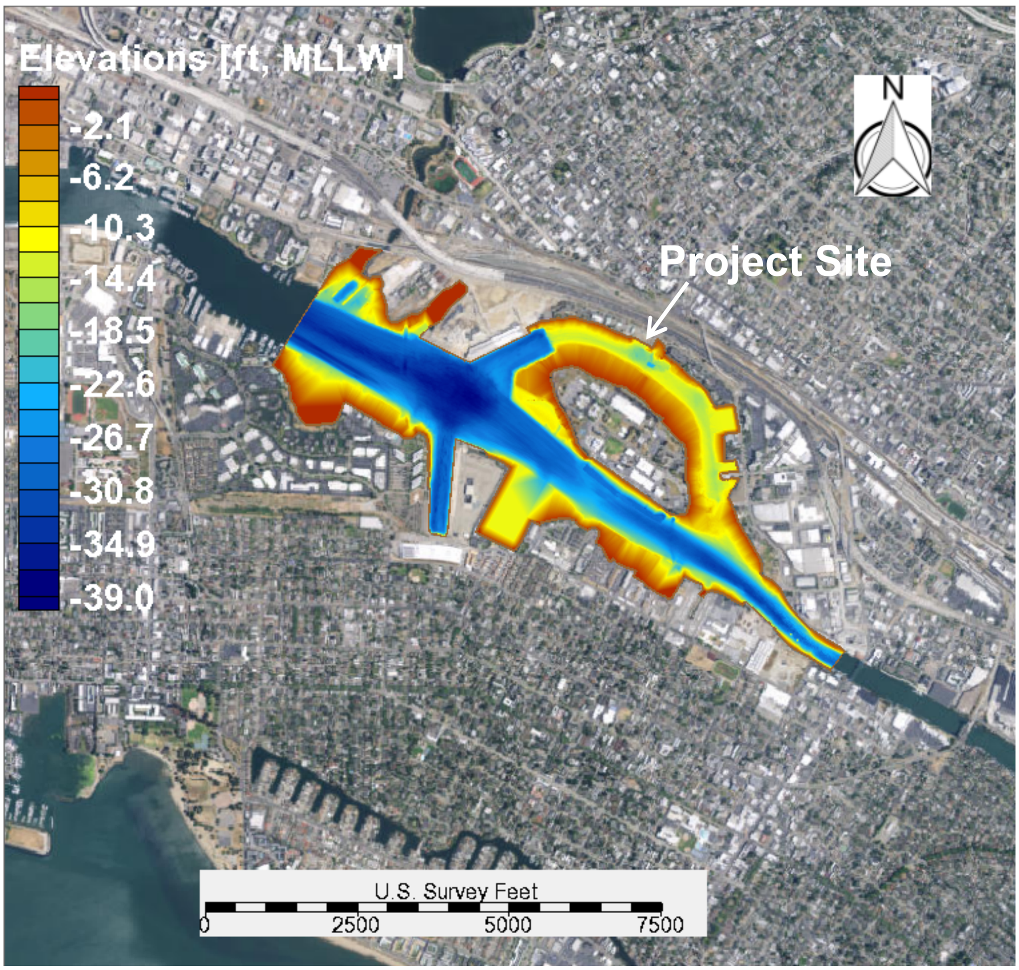
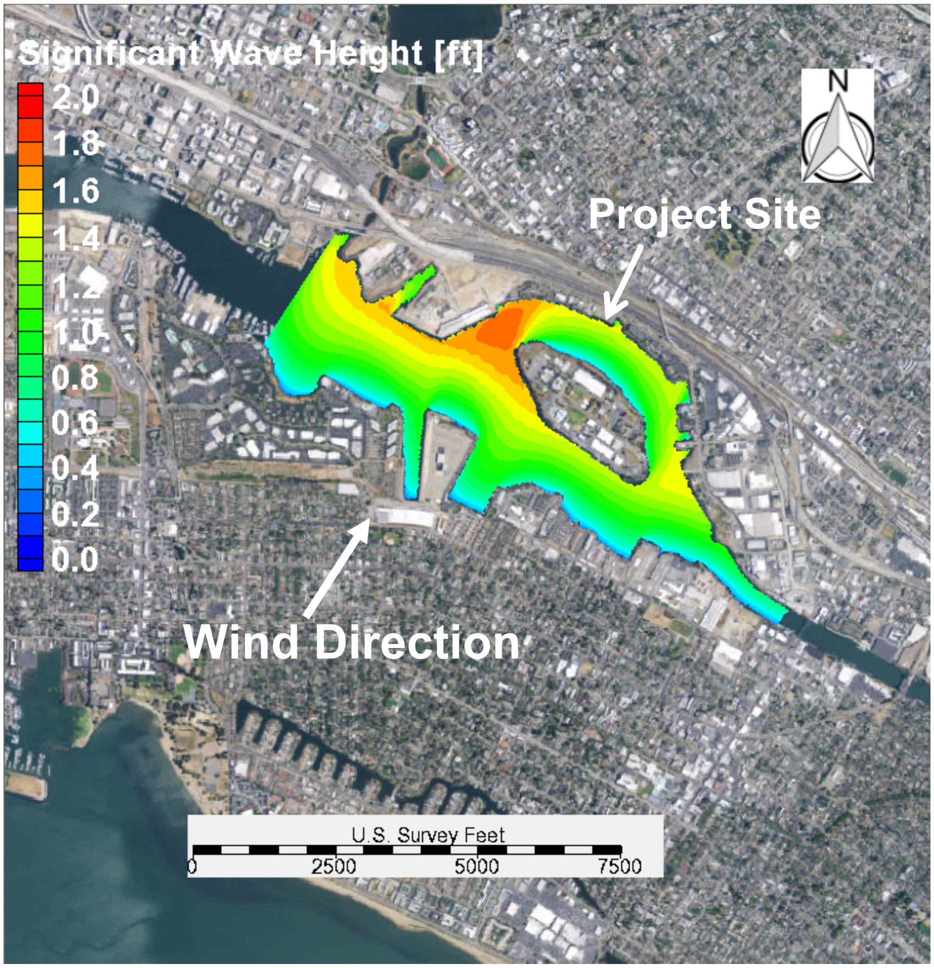


Figure 8 shows example wind-wave growth and transformation modeling results near the project area in the form of significant wave heights for 100-year extreme winds with speed 41 knots (2-minute average) from the southwest. The worst-case direction 100-year significant wave height and peak wave period at the project site were approximately 1.2 feet, 1.9 seconds. Maximum wave heights less than approximately 3 feet should be expected during this event.

Figure 8: 100-year significant wave heights near project site



4.4 Morphology Observations

Site morphology (ongoing seabed elevation changes) were evaluated through comparison of a previous hydrographic survey and the new multibeam hydrographic survey collected by eTrac in September 2017. The survey dataset from NOAA utilized in the wind-wave modeling was used at full 1m resolution for comparison with the eTrac survey originally at 1-ft resolution. Figure 9 shows the elevations surrounding the site collected in September 2017 during the eTrac multibeam hydrographic survey.

Figure 10 shows the computed elevation changes (erosion or deposition) that occurred between October 2012 and September 2017. Changes in elevation near the existing bulkhead are minimal, and likely within survey accuracy. Therefore, it does not appear that any significant morphology changes are ongoing at the project site that would warrant additional consideration in the design. Protection against scour at the toe of each alternative has already been considered.

Figure 9: eTrac 2017 multibeam hydrographic survey elevation contours

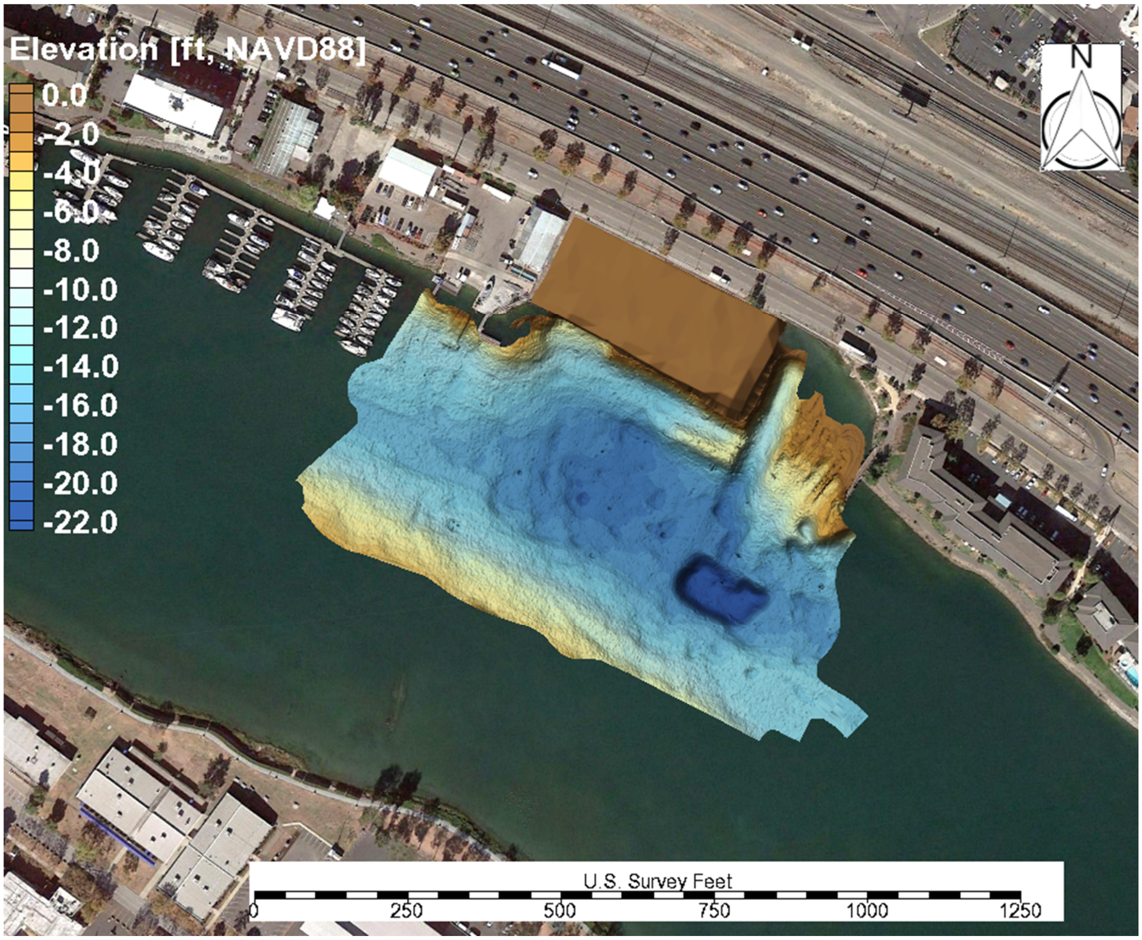
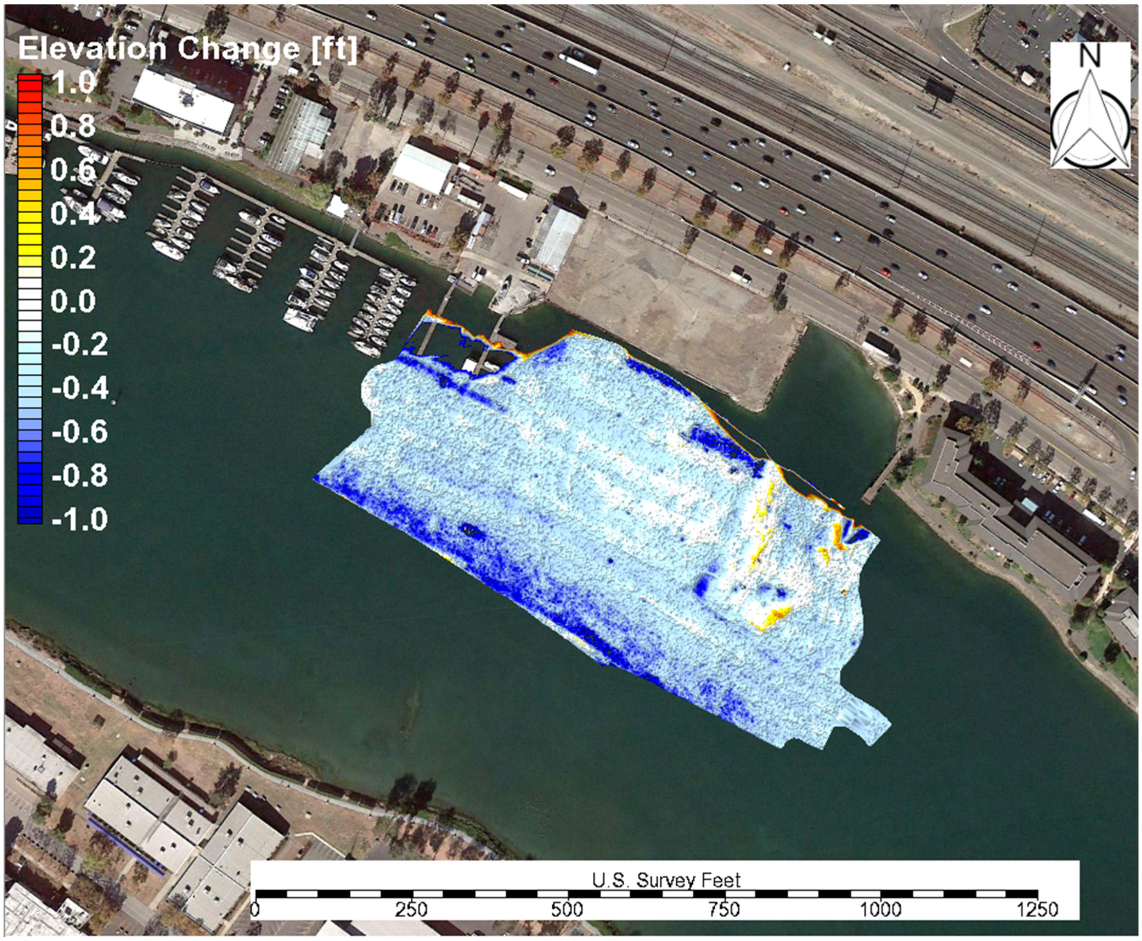


Figure 10: Observed elevation changes (erosion or deposition) from October 2013 to present



Note: Erosion (-) and deposition (+).

4.5 Coastal Design Criteria Summary

Coastal conditions are relatively mild due to the sheltered nature of the site. In general, bulkhead alternatives will not be affected by coastal conditions aside from variations in water levels. Revetment alternative design criteria include only water levels and storm waves, which are minimal and generally in the same size range as the largest vessel wakes to be expected at the site.

4.6 Rock Sizing for Revetment Alternative

Design wave parameters from above analysis were used to design the revetment for scour protection against wave action. The revetment slope was assumed to be 1.5H:1V in order to reduce Bay fill, and to limit encroachment into the uplands. Rock size and weight for the revetment armor layer were computed using the method of Van der Meer (1988b). Bedding layer rock characteristics were determined per recommendations from Coastal Engineering Manual (2006) and CIRIA (2007). Analysis indicates that the minimum armor and bedding stone weights computed to be required to resist storm wave attack with minimal damage are approximately 30 and 5 pounds, respectively. Since the slope is relatively steep, and wave conditions are mild, slope stability considerations are likely to require larger stone than would otherwise be required for armor stone stability under wave attack alone.

5 Structural Engineering Analysis

This section summarizes the analysis performed to determine the feasibility of a sheet pile wall bulkhead, and design criteria used to develop the feasibility-level design.

5.1 Geotechnical Investigation

TRC collected geotechnical information using a series of borings at the site, as shown in Figure 11. TRC analyzed the soil data and provided soil properties to MM for use in bulkhead analysis and design. Soil properties from borehole "B3" were found to be controlling, and were assumed to be representative of the worst-case conditions used for design of the bulkhead. Tables 2-4 show soil properties at Boreholes B1-B3, respectively.

Figure 11: Geotechnical boring locations

Source: TRC Solutions (2017)

Table 2. Soil Properties at Borehole B1

Soil Layer top and bottom elevations below the existing ground (ft)	Soil Type	Internal friction angle (degrees)	Cohesion (psf)
0 - 10	Sand	34	0
10 - 18	Sand	~~15~~ **34** [1]	0
18 – 20	Clay	0	750
20 – 27	Sand	~~15~~ **34** [1]	0
27 – 34	Clay	0	1500
34 – 37	Sand	~~15~~ **34** [1]	0
37 – 45	Clay	0	1000
45 – 50	Clay	0	2000

Table 3. Soil Properties at Borehole B2

Soil Layer top and bottom elevations below the existing ground (ft)	Soil Type	Internal friction angle (degrees)	Cohesion (psf)
0 - 20	Clay	0	500
20 – 27	Clay	0	1000
27 – 30	Clay	0	750
30 – 40	Clay	0	2000
40 – 50	Clay	0	750

Table 4. Soil Properties at Borehole B3 (Controlling)

Soil Layer top and bottom elevations below the existing ground (ft)	Soil Type	Internal friction angle (degrees)	Cohesion (psf)
0 - 10	Sand	33	0
10 – 17	Sand	~~15~~ **33** [1]	0
17 - 30	Clay	0	500
30 – 40	Clay	0	2000
40 – 50	Clay	0	1000

Notes:

1. The lower values of internal friction angles represent a weakened soil condition when the soil has liquefied under seismic conditions. The site does not support any critical structures, nor are there any critical structures in the immediate vicinity. Therefore, seismic forces have not been considered in the design of the bulkhead structure and the higher value of the friction angle values, as would be present during static conditions, have been considered. Moreover, if the sheet-pile bulkhead alternative was to be constructed, the upper few feet of soils, above the water table, on the site would be excavated and filled and compacted with a more competent structural fill. This would result in vastly improved soil conditions at the site.

5.2 Structural Design Approach

The steel sheet pile concept design was based on the loading requirements provided in this section. Loading conditions not specified here relied on accepted industry standards. The requirements provided herein control. The steel sheet pile bulkhead design method used was the Allowable Strength Design (ASD) method. The moment demand and loads in the anchor rods (tie-backs) generated by the design loads were analyzed and compared using three different software platforms: CWALSHT (X0031) 2003, Pilebuck SPW911 v2.40, and Skyline ProSheet. The software platforms were then used to analyze and compare deflections in the design steel sheetpile section imposed by the service loads.

5.3 Structural Design Criteria

5.3.1 Design Elevations

The assumed design elevations used to evaluate the steel sheet pile wall stresses and develop the appropriate material strengths are shown below in Table 5. Tidal elevations were taken from Alameda, CA (NOAA Station 94147509414750).

Table 5: Design Elevations for Bulkhead Analysis

Parameter	Elevation (ft, NAVD88)
NAVD88	0.0
Top of Wall Elev.	11.20
Water Elev. (MHW)	5.75
Water Elev. (MLLW)	-0.23
Mudline Elev.	-15

5.3.2 Design Loads

The dead load consists of the self-weight of the entire structure, including all the permanent attachments such as cap beams, railings, etc. A superimposed dead load of 30psf was considered to account for the unit weight of a future development of a paved surface. The usage of heavy equipment such as mobile cranes will generally produce the controlling loading conditions.

For this feasibility level design, live load criteria included both AASHTO Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic, as well as an alternative surcharge loading of 250 psf, assumed not to act simultaneously with AASHTO loading.

5.3.2.1 Other Loading Considerations

The maximum allowable deflection at the top of the wall considered for this analysis is 2 inches. The maximum allowable deflection along the length of the wall is limited to L/180. To be conservative, no sacrificial thickness or corrosion protection coating, such as galvanization or epoxy coating has been considered in the feasibility level design.

5.3.2.2 Loads Not Considered

The following loads were not considered because they do not significantly affect the design at the feasibility level stage:

- Seismic loads: The site does not support any critical structures, nor are there any critical structures in the immediate vicinity. Therefore, seismic forces have not been considered in the analysis of the bulkhead structure.
- Live load impact and Mooring loads: The site is not intended to provide moorage facilities. As such, no vessels are expected to be in the vicinity of the structure.
- Wind loads and Wave loads: As mentioned in Section 4, the coastal analysis indicates that tidal currents, winds, waves, and passing ship effects are not the controlling loads at the site.

5.4 Toe Scour Analysis

Using the 100-year design wave parameters, the method of Sumer et al. (2001) indicates that the potential scour depth at the toe of the existing bulkhead or a future bulkhead during the design storm would likely be less than 3-4 inches. This is logical given the lack of evidence of toe scour at the existing bulkhead, and indicates that toe scour does not control the embedment depth required for a bulkhead.

5.5 Existing Bulkhead Stability Analysis

An analysis was performed to determine a reasonable setback distance for the “No Action” alternative, within which public access would be prohibited. The analysis assumed a reduced angle of repose of 15 degrees due to liquefaction, a depth of 35 feet to the point of zero moment from the top of the wall, and the assumption that the existing wall fails at the mudline. Assuming that the wall reaches the Rankine Theory state of failure in the Active Case where the wall movement is away from land, the failure zone would extend approximately 30 feet landward from the waterside edge of the site.

In addition to Rankine Theory state of failure approach setback, AASHTO states that “a live load surcharge shall be applied where vehicular load is expected to act on the surface of the backfill within a distance equal to one-half the wall height behind the back face of the wall”. This is interpreted such that no surcharge is applied at an offset distance greater than half the depth of zero moment from the top of the wall, which would be approximately 20 feet.

A load sensitivity analysis was performed using the software platform CWALSHT to confirm the assumption. Using a strip load of 280 psf set back at 30 feet from the wall and comparing it to a “no load” case, it can be observed that no significant stresses are introduced at the existing retaining wall. The intent of the “safe setback” offset is to stay outside of the failure zone as described by Rankine Theory; therefore it is recommended that a minimum safe setback of 30 feet can be used for the “No Action” alternative for the purposes of this Feasibility Study.

6 Feasibility Level Design Summary

6.1 Revetment Alternative

The results of the analysis show that a new rock revetment is a feasible solution. The new revetment would include demolition of the existing bulkhead, backfill where required with coarse granular fill, compaction where feasible, and installation of geotextile fabric, then bedding stone, and finally armor stone. Soils excavated during construction could be re-used on site depending on suitability/competence, but for the purposes of the cost estimate it was assumed they are taken off site with appropriate disposal. The feasibility-level revetment design consists of high-tensile strength geotextile, a roughly 1-ft bedding stone layer, and roughly 2-ft thick armor stone layer. The design includes an approximately 4-ft wide crest, as well as a launched toe for additional slope support, and to guard against any potential scour. Please refer to the drawings in Appendix B showing sketches of the feasibility-level rock revetment alternative. Costs are described in Section 6.4

6.2 New Sheet Pile Bulkhead Alternative

The results of the analysis show that an anchored sheet pile system is a feasible solution. The new sheet pile bulkhead wall could be installed without demolishing the existing bulkhead and leaving it in place. Excavation will be required behind the wall for the installation of the anchor rods and anchor wall. The new bulkhead wall would consist of a steel sheet pile bulkhead, anchor rods spaced at regular intervals along the length of the sheet pile wall, and an anchor sheet pile wall (or concrete deadmen) some distance behind the bulkhead wall. The anchor rods will extend from the face of the bulkhead sheets to the anchor sheets (or concrete deadmen) and tie the entire system in place. Please refer to the drawings in Appendix B showing sketches of the new feasibility-level sheet pile wall bulkhead system.

6.3 No Action (Safe Setback) Alternative

The analysis for the “No Action” or “Safe Setback” option shows that a setback of a minimum of 30 feet from the waterside edge of the existing property would result in safe conditions. Please refer to the drawings in Appendix B showing the extents of the setback area where public access would be prohibited.

6.4 Cost Summary

Feasibility-level cost estimates were prepared using a combination of recent bid prices, discussion with contractors, and pricing from materials suppliers. Conservative assumptions and a 30% design contingency were included, which is typical of estimating at the feasibility level. Considerations made in both cost estimates include mobilization/demobilization, surveying, existing bulkhead demolition, excavation and disposal (assuming Class I disposal costs, and generator fees), structural/granular backfill, armor/bedding stone for the rock revetment, and steel sheet pile and concrete cap for the new bulkhead.

6.4.1 Rock Revetment

The feasibility-level cost estimate for the rock revetment is approximately $1.2M, or $2,200/LF. This cost estimate was prepared under the assumption proposed by TRC that excavated soil would not require costly disposal. It should be noted that disposal of Class I soils would result in significant addition to the costs, if none of the excavated materials can be re-used on-site (backfill for the revetment is all in-water). The excavation and disposal, plus generator fees, resulting from the Class I soil would result in an increase in total construction costs by approximately 40-50%.

6.4.2 New Bulkhead

The feasibility-level cost estimate for the new bulkhead was approximately $2.8M, or $5,000/LF. This cost estimate was also prepared under the assumption proposed by TRC that excavated soil would not require costly disposal. It should be noted that, similar to the rock revetment alternative, the presence of Class I soils would result in a significant addition to the construction costs if none of the excavated materials can be re-used on-site. Given that sheet pile walls are not completely watertight, some dewatering may be required during installation. This water would likely require careful handling and/or disposal due to the presence of Class I soils. The excavation and disposal, generator fees, and costs associated with dewatering, due to the presence of Class I soils would result in an increase of the construction costs by approximately 40-50%.

7 Regulatory Permitting Considerations

7.1 General Permitting Considerations

Regardless of the design, permits from the following agencies will need to be obtained to conduct construction activities:

- USACE – Section 404 and Section 10
- SFRWQCB – Water Quality Certification Section 401
- BCDC – Major Permit
- NMFS and CDFW – Informal or Formal consultation depending on design, net fill volumes, construction methods and schedule. Working within environmental work windows August to November of any given year for this project site would reduce potential impacts.

7.2 General Information Requirements

The rock revetment alternative and new bulkhead alternative would require compilation of similar information to supply in the permits for processing. The following information would need to be included in the application packages:

- Project Description and conceptual design (typically 30% design of selected alternative).
- Net fill volumes (as mentioned previously within the alternatives analysis describe minimization of fill to greatest feasible extent).
- Potential impacts and sensitive species: an informal biological assessment would be sufficient to address evaluation of potential impacts to Endangered Species and Essential Fish Habitat for NMFS and CDFW. Informal consultation could expedite the permitting process.
- Proposed avoidance and minimization measures.
- Proposed mitigation based on any significant impacts identified.
- Other information, to be identified during informal consultations.

7.3 CEQA/NEPA Requirements

CEQA requirements will need to be addressed. A lead agency will need to be identified and typically an IS/Mitigated Negative Declaration is sufficient for this type of project. The Port of Oakland as property owner could take on this role, but the District would need to coordinate and arrange an agreement. NEPA requirements are typically addressed internally by the USACE; similar stabilization projects have not required a comprehensive Environmental Assessment. The USACE will make a preliminary determination that the project neither qualifies for a Categorical Exclusion nor requires the preparation of an Environmental Impact Statement for the purposes of NEPA. At the conclusion of the public comment period, the USACE will assess the environmental impacts of the project.

7.4 Mitigation Requirements

It is not likely that agencies would require compensatory mitigation for impacts even if minor net fill resulted, due to the fact the Project's objectives are to stabilize a decaying/hazardous shoreline, and to provide public access. It should be noted that from a regulatory perspective, an alternative with perceived lower volume and/or area of Bay fill would likely be preferred.

Within an alternatives analysis required under Clean Water Act 401(b) the selected alternative will be based on the least environmentally damaging practicable alternative. Thus, cost would be a significant consideration for practicability.

7.5 Permitting Summary

Given that the Project's objectives are to stabilize a decaying/hazardous shoreline, and to provide public access, permitting is not likely a significant impediment to shoreline redevelopment for either the revetment or new bulkhead alternative. During the design process, Bay fill and other properties of the preferred alternative can be modified if they become a concern to agencies.

8 References

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APPENDIX A

TECHNICAL MEMORANDUM ISSUED ON SEPTEMBER 07, 2017

Liz Musbach
East Bay Regional Park District
2950 Peralta Oaks Court
PO Box 5381
Oakland, CA 94605

Alternatives for 1441 Embarcadero Shoreline Redevelopment Feasibility Study

September 7, 2017

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Dear Ms. Musbach:

Mott MacDonald, in coordination with TRC Solutions, is commencing on a shoreline redevelopment feasibility study and evaluation of shoreline redevelopment alternatives for East Bay Regional Park District at 1441 Embarcadero, Oakland, CA. The site consists of a plot of land approximately 385 ft. by 170 ft, with the waterside edge of the area bound by an existing steel sheet pile wall bulkhead to the south. The bulkhead extends from the southwest corner of the site and runs for approximately 325 ft. along its southern edge. The remainder of the boundary is made up of rock and rubble, with some timber piles and concrete slabs protruding from the eroding fill. Figure 1 shows a photo of the site taken from the southeast.

Figure 1. Site Shoreline Conditions

Based on preliminary observations, the existing bulkhead wall appears to have been partially repaired on multiple occasions during its life. Despite these previous repairs, the bulkhead is in a state of extreme disrepair, showing signs of extensive corrosion and failure of the tiebacks and waler system. The bulkhead wall is bowing and the ground immediately behind the wall is showing signs of significant subsidence. In order to develop the site for public access, the shoreline must be stabilized. Figure 2 shows a photo of the existing bulkhead looking southeast.

Figure 2. Existing Bulkhead

Mott MacDonald, in association with TRC Solutions, proposes to evaluate the following alternatives as part of the feasibility study to stabilize/restore the site's waterfront:

1. <u>Rock Revetment/Shoreline Protection</u>:

   This alternative consists of installation of an engineered rock slope along all or part of the waterside edge of the project site. A portion of the waterside edge of the site would need to be excavated, deleterious materials removed, backfill placed and compacted with competent material (including reuse of onsite fill), and geotextile filter fabric installed to enable placement of the rock revetment and prevent future settlement. The existing bulkhead would either be removed or could remain in place, to be further investigated during the feasibility study.

   Under this alternative, the Bay Trail could run along the waterside edge of the property, and the upland area of the site could be safely developed. This is likely a cost-effective alternative, consistent with surrounding shorelines, and would require minimal maintenance. However, the rock revetment alternative requires consideration of potential loss of upland space, or a moderate amount of Bay fill. Figure 3 shows a concept sketch describing a general rock revetment alternative. This alternative can likely obtain regulatory approval as long as appropriate documentation is provided, Bay fill is minimized in the design, and the project incorporates appropriate Best Management Practices during construction.

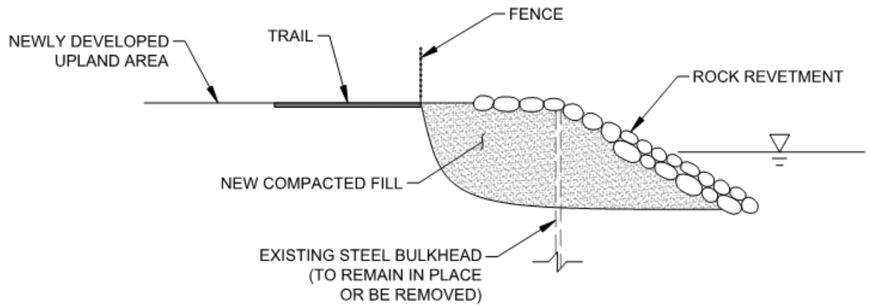

Figure 3. Rock Revetment Alternative

2. New Bulkhead:

   This alternative consists of installing a new sheet pile bulkhead, similar in concept to the existing bulkhead, along all or part of the waterside edge of the project site. The new bulkhead would be installed seaward of the existing bulkhead, and would likely not require removal of the existing bulkhead. Similar to Option 1, the trail would run along the waterside edge of the property, and the upland area of the site could be developed.

   This shoreline stabilization alternative would likely be the costliest, would require some maintenance, and would likely need to be replaced in 25-30 years. However, this alternative would provide the most space in the uplands for development, and with a very minor amount of Bay fill. Figure 4 shows a concept sketch describing a new bulkhead alternative. This alternative can likely obtain regulatory approval as long as appropriate documentation is provided, vibratory hammers are used for the majority of sheet pile installation, and the project incorporates appropriate Best Management Practices during construction.

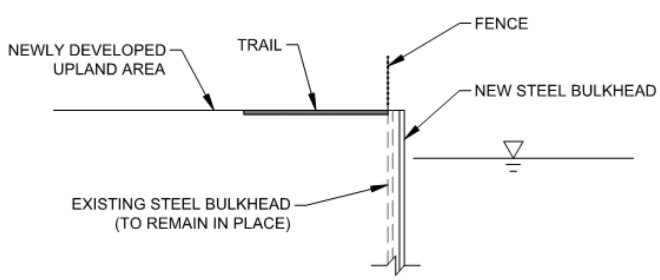

Figure 4. New Bulkhead Alternative

3. Living Shoreline/Habitat Enhancement:

   Redevelopment of the shoreline in this location provides opportunity for not only stabilizing the shoreline, but installation of aquatic enhancement features that can be incorporated into the stabilized shoreline. For instance, as part of an enhanced rock revetment alternative, sheltered areas can be incorporated to provide attractive habitat areas such as small coves, tide pools, pebble beach areas, etc. similar to those constructed by the District at Albany Beach. This would provide habitat value, educational value, and additional attraction for visitors to the site.

This option would be costlier than Option 1 (rock revetment), and would involve either greater Bay fill or further reduced upland area due to flatter slopes required in some sections of the shoreline. However, in general, the Bay fill resulting from habitat elements has not been an impediment to regulatory compliance as long as efforts are undertaken to minimize fill where feasible. Similar to the rock revetment alternative, the living shoreline alternative can also obtain regulatory approval as long as appropriate documentation is provided, Bay fill is minimized in the design, and the project incorporates appropriate Best Management Practices during construction.

No Action:

A "No Action" alternative can be defined as leaving the waterfront in its present condition, which is unstable and represents a safety hazard. The "No Action" alternative would result in the Bay Trail setback from the water a considerable distance. Since the feasibility study focuses on shoreline redevelopment to support the Bay Trail and upland development, the "No Action" alternative will not be considered in the feasibility study.

For the purposes of the shoreline redevelopment feasibility study, we believe that the alternatives listed above are practical solutions which can facilitate proposed future use of the site. Additional alternatives may be feasible and can be evaluated upon further discussion with the East Bay Regional Park District.

Please do not hesitate to contact me with any questions or comments about this memo and/or the Scope of Work for the Feasibility Study submitted by Mott MacDonald on July 18, 2017.

Best Regards,

Scott W. Fenical, PE, D.CE, D.PE
Vice President
Coastal Practice Leader, North/South America
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415.773.2164

APPENDIX B

DRAWINGS

NOTES

1. VERTICAL DATUM: NAVD88
2. HORIZONTAL DATUM: NAD83 CALIFORNIA STATE PLANE, ZONE III, FEET
3. AERIAL SOURCE: WORLD IMAGERY

EXISTING SITE PLAN

0 30 60
SCALE IN FEET

LEGEND

-10 EXISTING CONTOUR

NOT FOR CONSTRUCTION

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Designed	A. Reza	Eng check	S. Fenical
Drawn	C. Taylor	Coordination	
Dwg check	A. Sherma	Approved	
Scale at ANSI D: As Noted	Status	Rev	Security

Project Number	B/O	Total	Drawing Number
388867	1	4	

Title
1441 Embarcadero
Shoreline Redevelopment
Feasibility Study

Existing Site Plan

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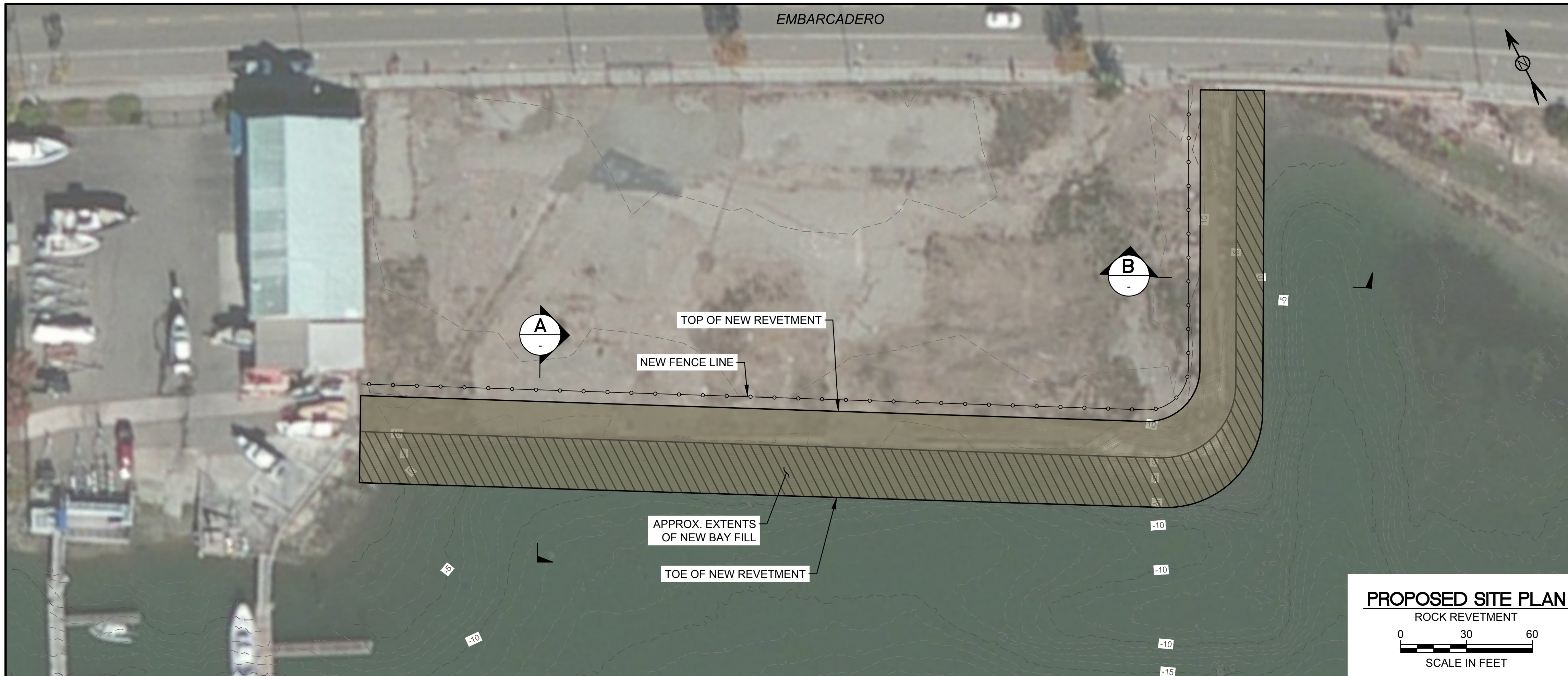


PROPOSED SITE PLAN

ROCK REVETMENT

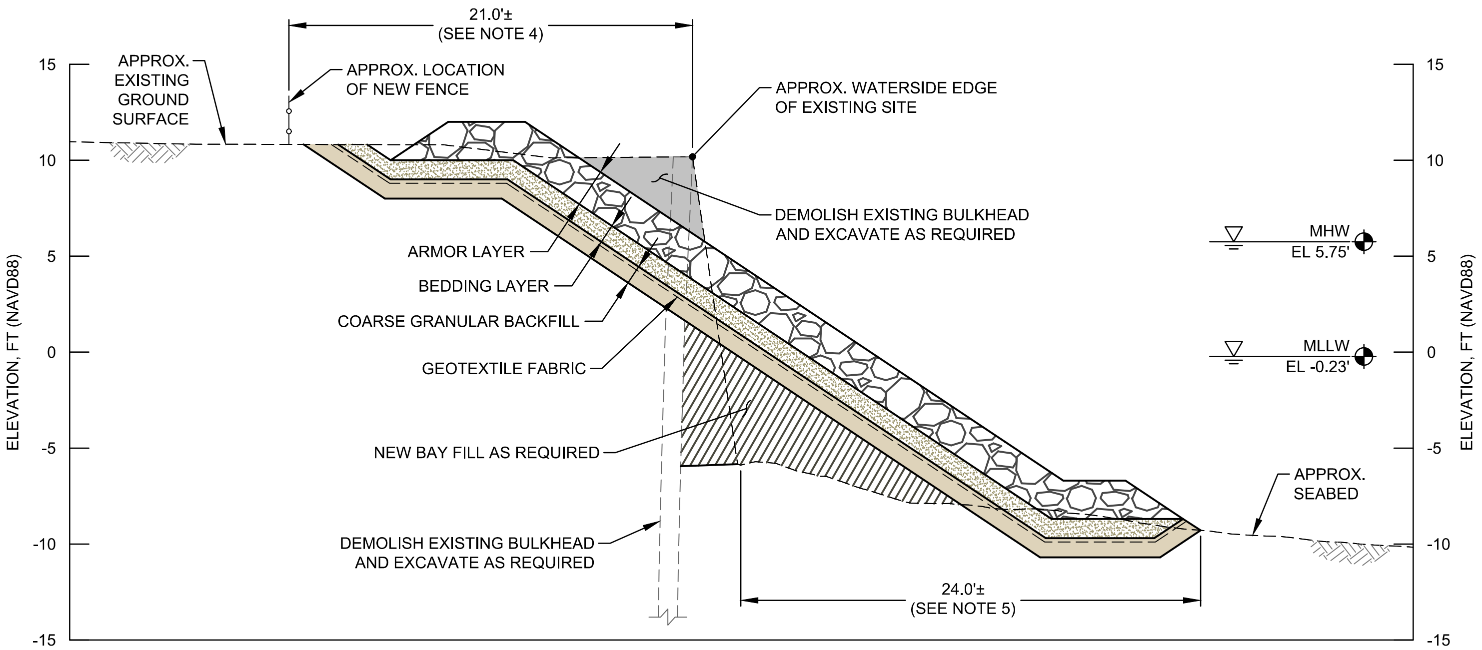
0 30 60
SCALE IN FEET

LEGEND

- EXISTING CONTOUR
- NEW FENCE LINE
- APPROX. EXTENTS OF REVETMENT
- APPROX. EXTENTS OF NEW BAY FILL

NOTES

1. VERTICAL DATUM: NAVD88
2. HORIZONTAL DATUM: NAD83 CALIFORNIA STATE PLANE, ZONE III, FEET
3. AERIAL SOURCE: WORLD IMAGERY
4. LOCATION OF NEW FENCE, AND CONSEQUENTLY ITS DISTANCE FROM THE WATERSIDE EDGE OF THE EXISTING SITE WILL DEPEND UPON THE FINAL LOCATION OF THE NEW ROCK REVETMENT.
5. THE EXTENT OF FILL WILL DEPEND UPON THE FINAL LOCATION OF THE NEW ROCK REVETMENT.



A SECTION

0 5 10
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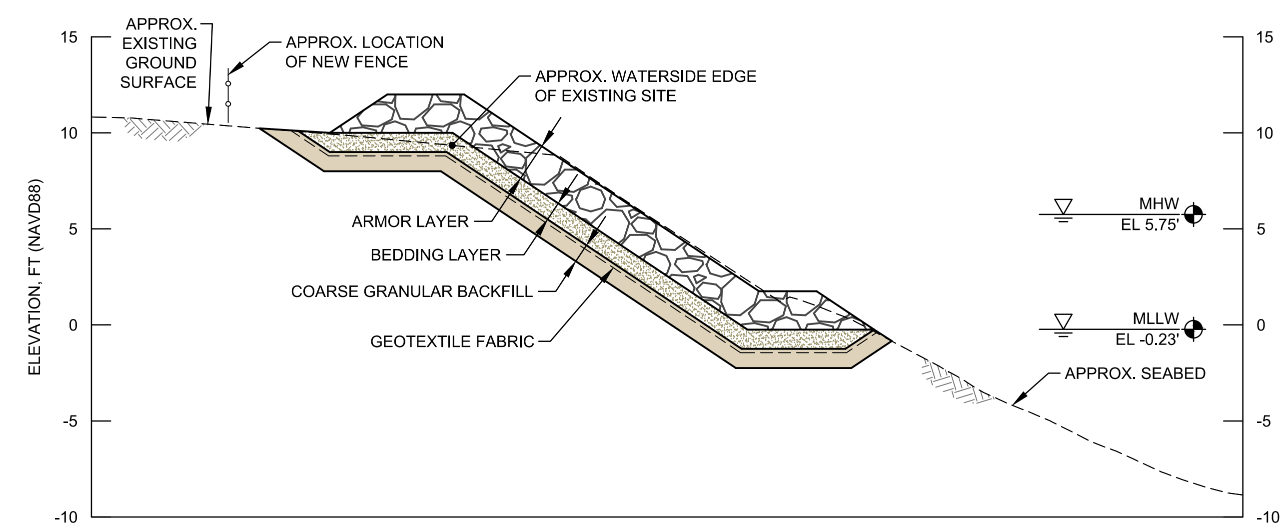

B SECTION

0 5 10
SCALE IN FEET

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Drawn	C. Taylor	Coordination	
Dwg check	A. Sherma	Approved	
Scale at ANSI D: As Noted	Status	Rev	Security

Project Number 388867 | B/O 2 | Total 4 | Drawing Number

Title
1441 Embarcadero
Shoreline Redevelopment
Feasibility Study

Plan and Sections
Proposed Revetment Alternative

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PROPOSED SITE PLAN

SHEETPILE BULKHEAD

0 30 60

SCALE IN FEET

LEGEND

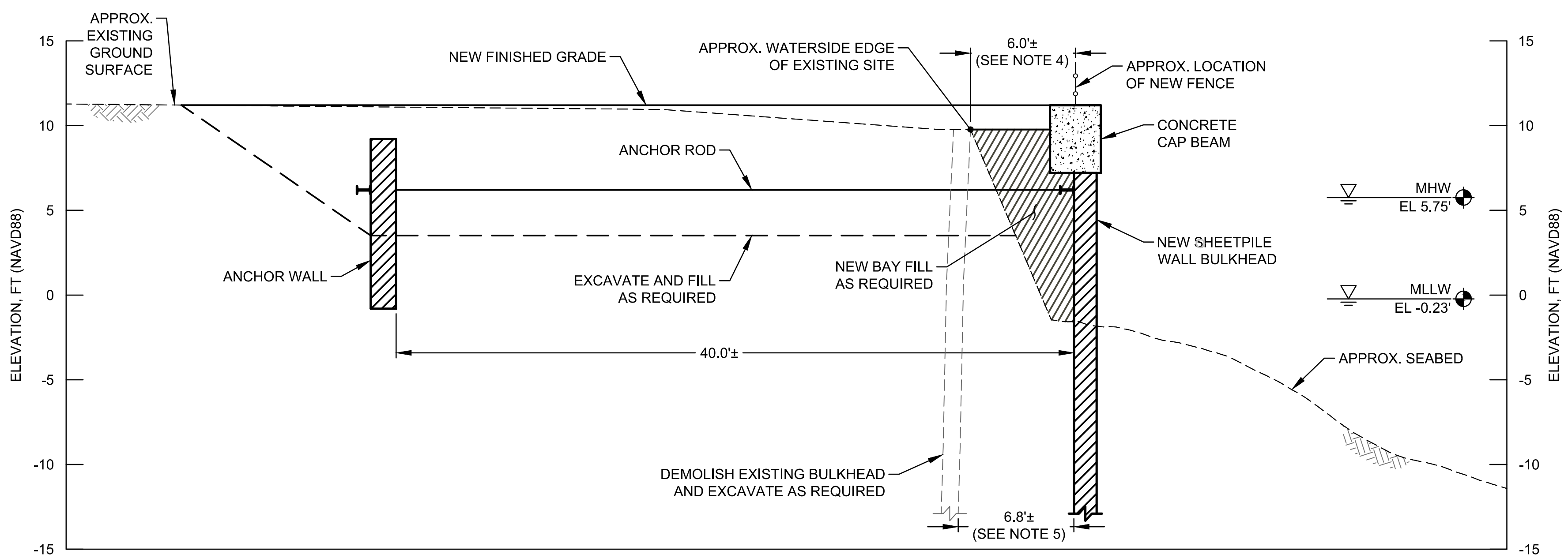
-10 EXISTING CONTOUR

NEW SHEETPILE BULKHEAD

EXTENTS OF FILL

NOTES

1. VERTICAL DATUM: NAVD88
2. HORIZONTAL DATUM: NAD83 CALIFORNIA STATE PLANE, ZONE III, FEET
3. AERIAL SOURCE: WORLD IMAGERY
4. THE LOCATION OF NEW FENCE, AND CONSEQUENTLY ITS DISTANCE FROM THE WATERSIDE EDGE OF THE EXISTING SITE WILL DEPEND UPON THE FINAL LOCATION OF THE NEW SHEETPILE BULKHEAD.
5. THE EXTENT OF FILL WILL DEPEND UPON THE FINAL LOCATION OF THE NEW SHEETPILE BULKHEAD.



C - SECTION

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SCALE IN FEET

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Drawn	C. Taylor		Coordination		
Dwg check	A. Sherma		Approved		
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Project Number	B/O	Total	Drawing Number
388867	3	4	

Title

1441 Embarcadero
Shoreline Redevelopment
Feasibility Study

Plan and Section
Proposed Bulkhead Alternative

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PROPOSED SITE PLAN

NO ACTION

0 30 60
SCALE IN FEET

LEGEND

— — -10- — — EXISTING CONTOUR

NOTES

1. VERTICAL DATUM: NAVD88
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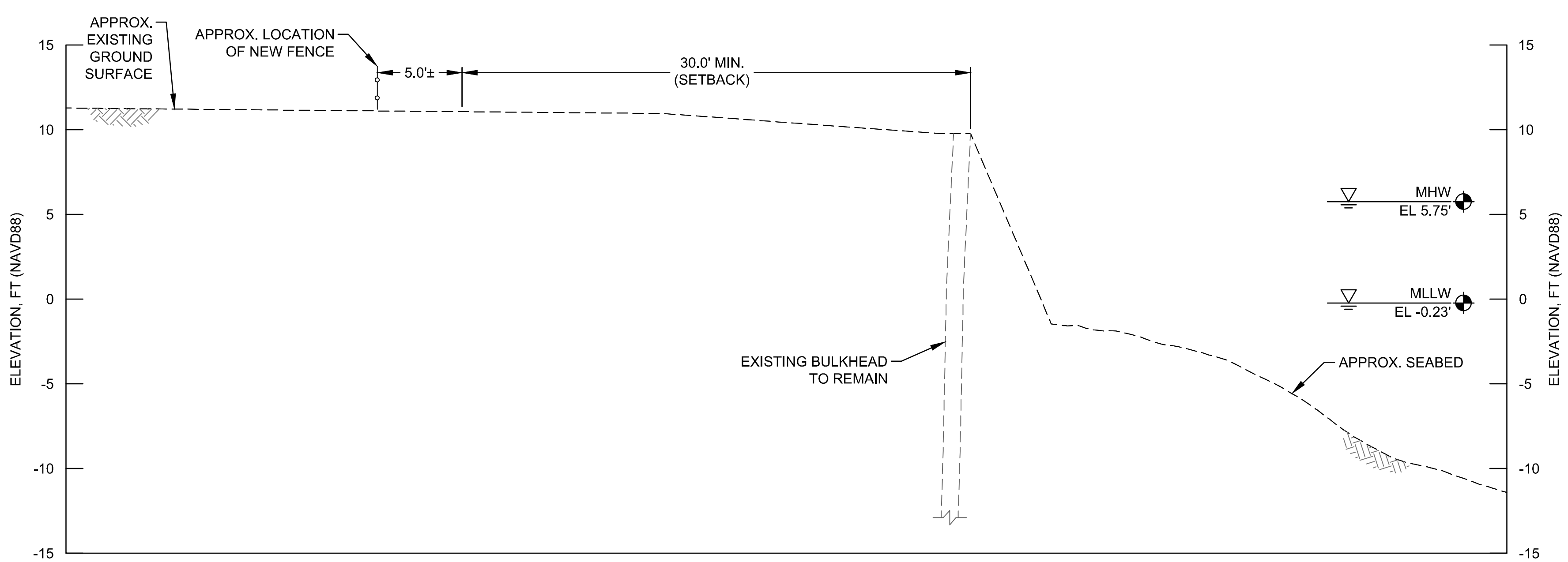

D - SECTION

0 5 10
SCALE IN FEET

NOT FOR CONSTRUCTION

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Drawn	C. Taylor	Coordination	
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Scale at ANSI D: As Noted	Status	Rev	Security

Project Number: 388867 | B/O: 4 | Total: 4 | Drawing Number:

Title
1441 Embarcadero
Shoreline Redevelopment
Feasibility Study

Plan and Section
No Action Alternative

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