Mandalay Bay Seawall Repair Conceptual Study and Feasibility Analysis - FINAL

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PRESENTED TO

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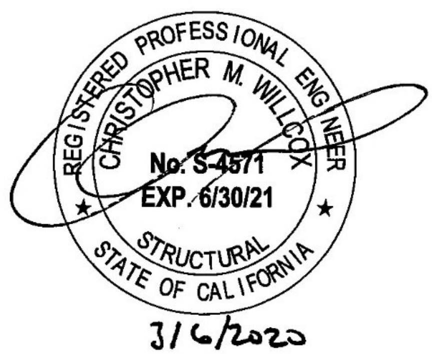

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1.0 EXECUTIVE SUMMARY

This Conceptual Study and Feasibility Report describes the existing condition of the Mandalay Bay Seawalls, develops a number of potential renovations and retrofits, and selects preferred repair options for the two types of walls, Boise and Zurn.

Parts of the existing walls, including a number of Boise wall pilasters and areas of Zurn wall have degraded badly. The most common types of deterioration seen on the walls, are vertical cracking and spalling of the Boise wall pilasters, cracking of the Boise wall panels, and significant surface erosion on the Zurn walls. Areas of rust staining on the concrete surfaces indicate localized corrosion of the steel reinforcing in the walls, previous investigations and testing suggest that the corrosion is not widespread. A petrographic examination of cores taken from the walls in several locations found signs of alkali-silica reaction, an expansive chemical process that causes cracking and spalling in concrete exposed to moisture.

In addition, to the deterioration of the walls, earthquakes present an additional concern. Due to a much better understanding of seismic hazards today, the loads used to design new retaining walls and sea walls are substantially higher now than they were at the time these walls were originally built. Compounding this issue, some of the soils behind and under these retaining walls are expected to liquefy in a severe earthquake. Therefore, these walls are expected to perform poorly in a major earthquake.

Based on the evaluations the same two options were recommended for both the Boise walls and the Zurn walls: Installation of Panels and Tiebacks; and Installation of a New Steel Sheet Pile Wall.

The tieback option, with a 75-year design life, consists of installing new panels in front of the existing wall resting on the existing footing, filling the gap between the new panels and the existing wall with grout and installing tiebacks that extend down to the competent, non-liquefiable sands located approximately 20 to 25 feet below ground level above the seawalls. This is the least disruptive and cheaper option, but further geotechnical investigation is required to be sure that the underlying soils will not flow out under the existing wall and foundation in a major earthquake and that the existing footings and piles are in good enough condition to reuse.

The new cantilever sheet pile option, with a 75-year design life, involves installing new steel sheet pile in front of the existing wall using the press-in method, which does not create noise or vibration. The gap between the new wall and the existing will then be filled. Because the existing walls have battered piles that extend in front of the wall, the new sheet pile will have to be placed about 8 feet in front of the existing walls. This leaves a substantial area to be filled, which adds cost as well as complications in getting regulatory approvals.

The construction costs for the various options are shown below in Table 1. The ranking matrix for the Boise Walls is shown below as Table 2 and the matrix for the Zurn Walls is shown as Table 3.

Table 1 - Construction Costs

BOISE WALLS

Opt	Description	Cost	Unit
A	**Tieback Wall**	**$ 4,277**	**LF**
B	**New cantilever sheet pile wall**	**$ 7,382**	**LF**
C	New cantilever soldier piles and panels	$ 7,037	LF
D	New soldier piles tied to existing pilasters	$ 4,050	LF
E	Concrete pilaster jackets and panel facing	$ 4,000	LF
F	FRP pilaster jackets and panel facing	$ 3,850	LF
G	Riprap stabilization	$ 10,094	LF
H	Epoxy crack injection of panel faces	$ 440	SF
I	Stainless steel reinforcing straps	$ 6,800	EA

ZURN WALLS

Opt	Description	Cost	Unit
J	**Tieback Wall**	**$ 4,155**	**LF**
K	**New cantilever sheet pile wall**	**$ 7,382**	**LF**
L	New cantilever soldier piles and panels	$ 7,037	LF
M	Remove and replace face concrete	$ 2,900	LF
N	New concrete facing	$ 2,600	LF
O	Concrete buttresses	$ 3,150	SF
P	Riprap stabilization	$ 10,094	LF

Table 2 - Evaluation Matrix for Boise Walls

		Construction Cost	Maintenance Cost	Effective Life	Constructability	Disruption	Regulatory Issues	Wall Capacity	Appearance	**Total**	**Ranking**
Weighting Factor		**2**	**1**	**3**	**1**	**1**	**1**	**2**	**1**	**-**	**-**
Options Designed for Seismic Resistance											
Recommended Options											
Scheme A	Tieback Wall	4	3	3	3	4	4	3	4	**41**	**1**
Scheme B	New cantilever sheet pile wall	1	4	5	3	1	2	5	4	**41**	**1**
Other Options Designed for Seismic Resistance											
Scheme C	New cantilever soldier piles and panels	3	3	4	3	2	2	4	3	**39**	**3**
Options with No Increase in Seismic Resistance											
Scheme D	New soldier piles tied to existing pilasters	3	3	3	3	3	3	1	3	**32**	**5**
Scheme E	Concrete pilaster jackets and panel facing	3	4	4	3	3	4	1	5	**39**	**1**
Scheme F	FRP pilaster jackets and panel facing	2	3	4	4	3	4	1	4	**36**	**3**
Scheme G	Riprap stabilization	2	5	5	2	1	1	4	2	**38**	**2**
Scheme H [1]	Epoxy crack injection of panel faces	5	1	2	5	5	5	0	1	**33**	**4**
Scheme I [2]	Stainless steel reinforcing straps	5	1	1	5	5	5	1	1	**32**	**5**

[1] Epoxy injection is only applicable to the panels and needs to be combined with a method of pilaster repair such as Scheme A or B.

[2] Scheme G does not protect areas outside of the strap footprints and should only be applied as a short-term repair or at areas that are suffering from reinforcing corrosion but are otherwise in good condition with minimal surface degradation of the concrete.

Table 3 - Evaluation Matrix for Zurn Walls

		Construction Cost	Maintenance Cost	Effective Life	Constructability	Disruption	Regulatory Issues	Wall Capacity	Appearance	**Total**	**Ranking**
Weighting Factor		**2**	**1**	**3**	**1**	**1**	**1**	**2**	**1**	**-**	**-**
Options Designed for Seismic Resistance											
Recommended Options											
Scheme J	Tieback wall	4	3	3	4	4	4	3	4	**42**	**1**
Scheme K	New cantilever sheet pile wall	1	4	5	3	1	2	5	4	**41**	**2**
Other Options Designed for Seismic Resistance											
Scheme L	New cantilever soldier piles and panels	3	3	4	3	2	2	4	3	**39**	**3**
Options with No Increase in Seismic Resistance											
Scheme M	Remove and replace face concrete	3	4	4	3	3	4	1	5	**39**	**2**
Scheme N	New concrete facing	4	4	4	4	4	4	1	4	**42**	**1**
Scheme O	Concrete buttresses	5	2	3	5	5	4	1	2	**39**	**2**
Scheme P	Riprap stabilization	2	5	5	2	1	1	4	2	**38**	**3**
Scheme Q [1]	Epoxy filling of base joints	N/A									

[1] Epoxy filling of base joints is a maintenance activity that should be performed at areas that are otherwise in good condition but have poor quality construction joints at the base of the wall and are not scheduled for repair in the near future.

2.0 INTRODUCTION

2.1 PURPOSE

The City of Oxnard contracted Tetra Tech, Inc. to conduct a Conceptual Study/Feasibility Analysis (Phase 1) for the Mandalay Bay Seawalls and to prepare Construction Documentation for the wall repairs at 3900 thru 3966 West Hemlock Street (Phase 2). This report documents the work conducted for Phase 1. The purpose of the conceptual study is to analyze the existing seawall construction types, review/outline existing condition, and provide repair options. The feasibility portion of this report focuses on the proposed repair options, how they meet the repair goals, construction phasing and cost.

2.2 BACKGROUND

Mandalay Bay is a waterfront development adjacent to Channel Islands Harbor in Oxnard, California. The area covered by this report is a Waterway Maintenance Assessment District known as Zone 1. Zone 1 contains 743 single family homes and 37 parcels designated as parks. The lots in Zone 1 are protected by reinforced concrete seawalls built in the late 1960's and early 1970's totaling about 7 miles in length.

There are two types of seawall in Zone 1. The walls in the eastern part of the development were built first and are a pilaster-and-panel tie-back wall type known as 'Boise' walls. The Boise walls consist of 8-inch thick precast panels spanning between precast T-shaped pilasters set approximately 11 feet on center on a cast-in-place footing supported by timber piles. The pilasters are supported laterally by tie-rods anchored into a continuous deadman 21 feet behind the wall. The walls stand approximately 10 feet tall above the top of the footing, which is at approximately the mean lower low water (MLLW) elevation.

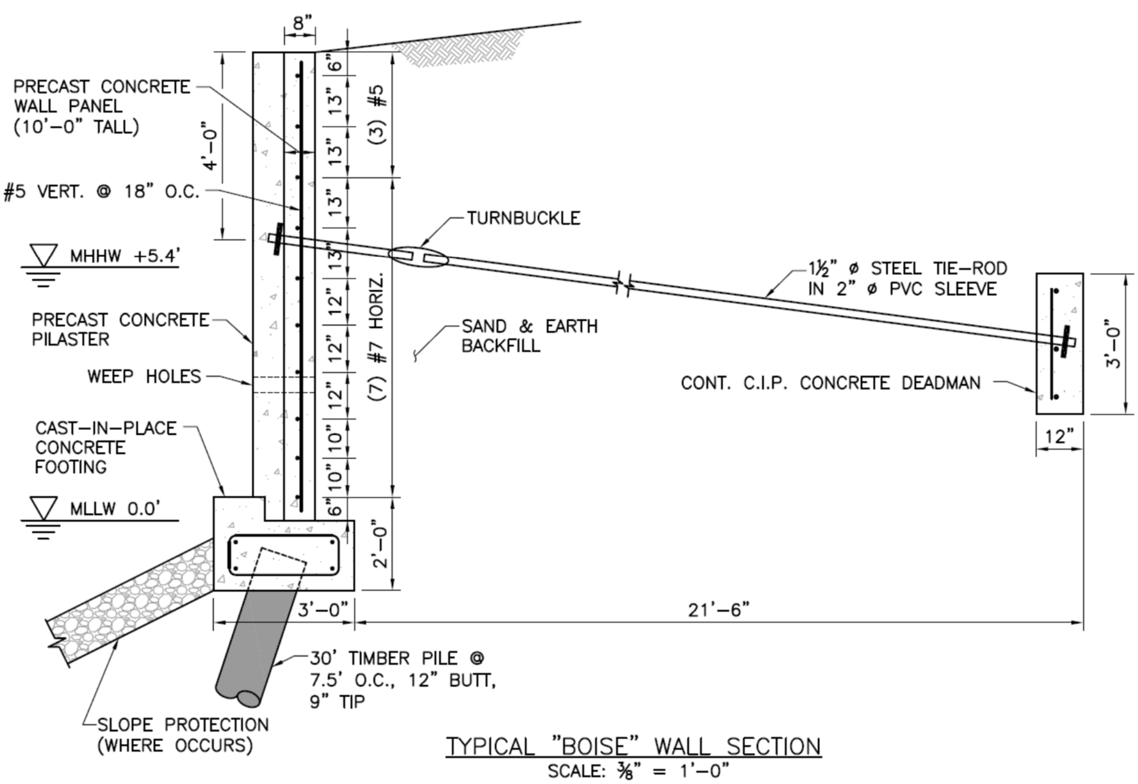

Figure 1 - Typical Boise wall [from TranSystems (2017)]

Built after a change in developers, the seawalls in the western part of the development are a cantilever type referred to as 'Zurn' walls. The Zurn walls are cast-in-place concrete walls supported by two rows of timber piles. The wall

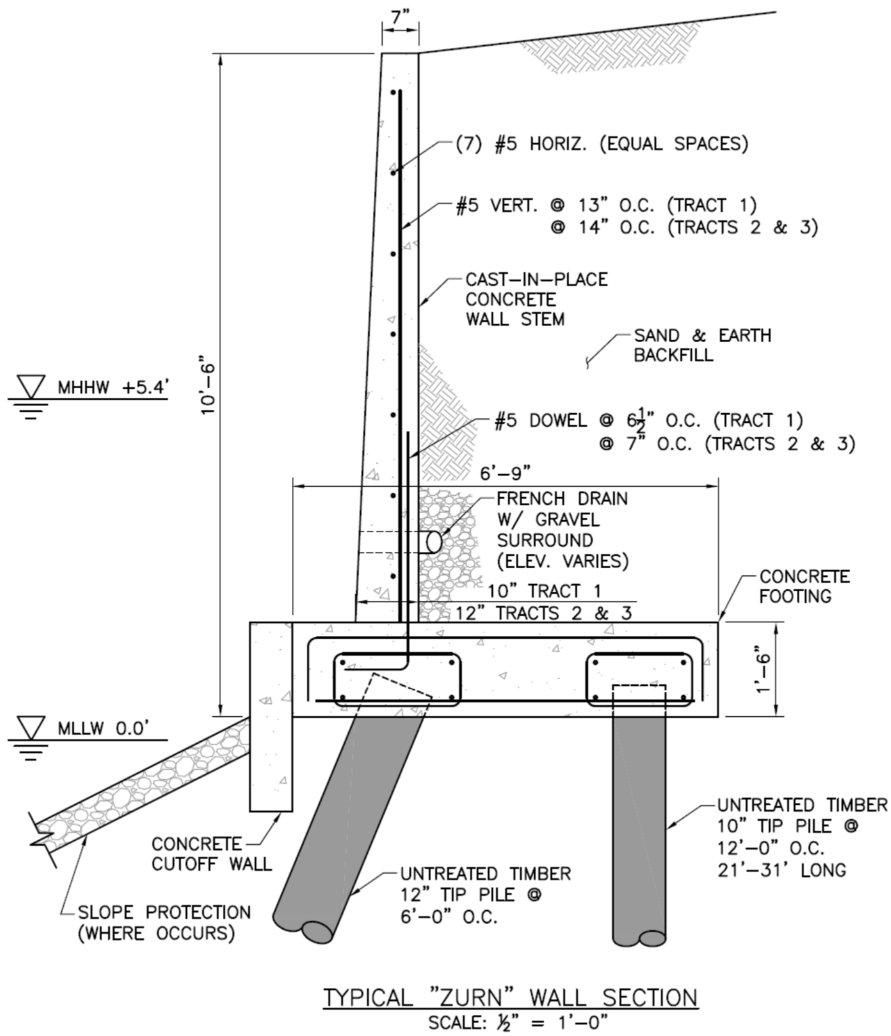

Figure 2 - Typical Zurn wall [from TranSystems (2017)]

stems are 9 feet tall set on a 6'-9" wide footing with the top of the footing, approximately two feet above MLLW. The stem thickness varies from 7 inches at the top to 10 or 12 inches at the bottom depending on the area in the development.

Over the years, repairs have been made in a number of locations. A number of these repairs have been concrete jackets installed on Boise wall pilasters. Other repairs have been to the cutoff walls and channel bottom slopes to prevent undermining of the foundations and to the weep holes to control backfill losses.

From 2010 to 2017, TranSystems Corporation conducted extensive evaluations and testing of the seawalls, followed by development of repair details and a proposed phasing of the repairs. This report relies on the documents prepared by TranSystems for the discussions on the existing condition of the walls and includes the proposed repairs from Phase C of that work in the evaluations of potential repair schemes.

TranSystems also evaluated the walls under seismic loads from the current building code, including liquefaction of the backfill and found that the walls would be overloaded by a code-level seismic event. TranSystems expressed particular concern about the stability of the Boise walls, which are tied back to concrete deadmen that rely on the liquefiable fill to hold them in place. It should be noted that revisions to the code since TranSystems performed their analysis have increased the design seismic ground accelerations in this part of the state that have to be considered.

Figure 3 - Typical pilaster jacket repairs [from TranSystems (2017)]

3.0 CURRENT CONDITIONS

3.1 DESCRIPTION

The walls are approximately 50 years old and are showing their age. The concrete face of the wall exhibits significant erosion in numerous areas, with the erosion being more significant in the cast-in-place Zurn walls. The erosion typically appears to be most significant between 2-3 feet and 5-6 feet above mean lower low water (MLLW). The

Figure 4 - Surface erosion/marine attack on Boise wall panels with cracking and spalling at pilasters

lowest part of the walls and the foundations show some erosion but are in better condition that the areas above. Starting about 7 feet above MLLW, the walls are typically in good condition.

Figure 5 - Spalling and surface erosion on Zurn wall [from TranSystems (2017)]

Horizontal and vertical cracking is common on the Boise wall panels, as is vertical cracking on the pilasters. In some cases, the cracks on the pilasters are several millimeters across. Surface cracking is less prevalent on the Zurn walls and appears to be most common near corners and joints where thermal movement may be contributing.

Figure 6 - Surface erosion and vertical cracking on a Boise wall panel [from TranSystems (2011)]

Figure 7 - Vertical cracking and spalling on Boise wall pilaster [from TranSystems (2017)]

The testing and evaluations performed by TranSystems and the results included the following:

- Petrographic examination of cores taken from various locations showed that the concrete is undergoing alkali-silica reaction (ASR) and this is the likely cause of much of the cracking and erosion in the walls. The severity of the reaction varied between the cores from very severe to minor.
- Impact echo testing found either surface delamination or deeper flaws in a significant proportion of tested locations for both the Zurn walls and the precast panels of the Boise walls, although the rate of surface delamination was lower in the Boise wall panels.
- Electrical resistivity tests generally found higher values indicative of concrete that is largely saturated with a higher level of resistance to corrosion. Combined with the results of half-cell potential tests performed in Phase A, TranSystems concluded that large-scale corrosion was not occurring.
- Excavation behind several wall sections at empty lots found the back faces of the walls to be in generally good condition. Several tieback rods were uncovered and checked for corrosion. Most showed no or negligible corrosion. One tie rod was estimated to have approximately 5 percent section loss at one location.
- The vertical reinforcing at the base of the Zurn wall was exposed at eight locations to check for localized micro-cell corrosion. At one location the dowel bar had fractured, and the adjacent bar had approximately 20% section loss. At a second location, the exposed dowel bar had approximately 50% corrosion loss. At the remaining 6 locations, the exposed bars showed no corrosion. The two locations with bar damage were in walls with significant erosion or degradation at the construction joint.
- Underwater evaluations found several locations where the cutoff wall at the Zurn wall foundation was separating from the footing and areas at the Boise walls where gaps were forming under the footings. Both of these conditions potentially allow seawater to access the piles leaving them susceptible to attack by marine borers.

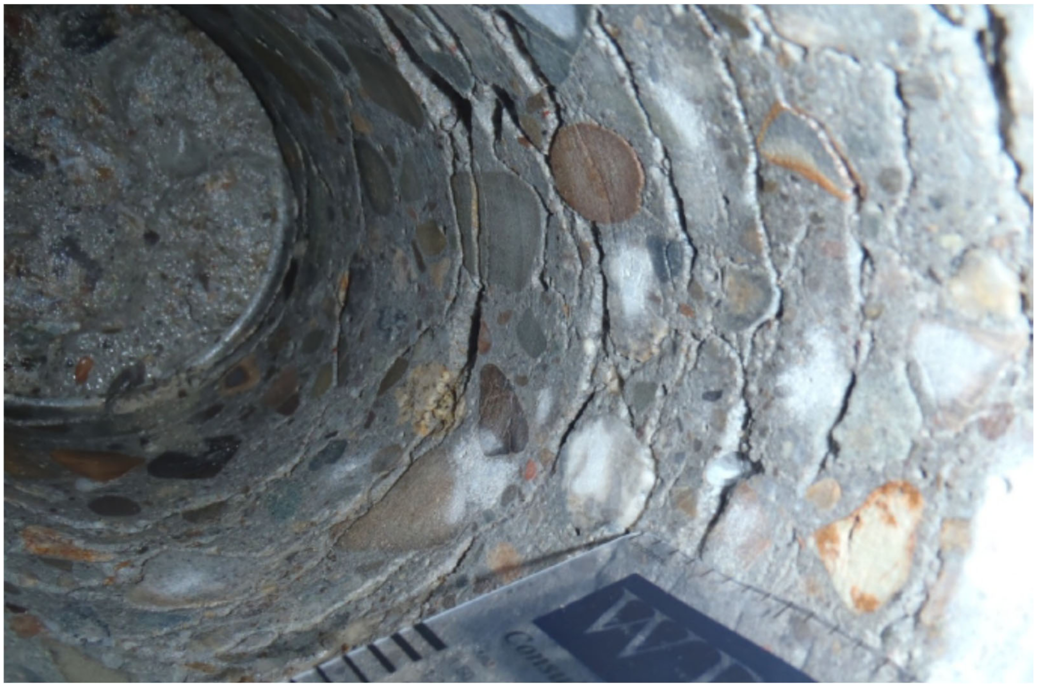

Figure 8 - Inside of core showing expansive cracking [from TranSystems (2011)]

3.2 LIKELY CAUSES OF OBSERVED (AND FUTURE) DEGRADATION

There are several identified and potential causes of the observed and potential future degradation of the wall. One of the major points of evaluation of any potential repairs or retrofits will be the ability of the method to control or limit future progression of the sources of degradation. Therefore, an understanding of these sources and methods of control is an important part of choosing appropriate repair methods.

3.2.1 Alkali-Silica Reaction

The previous evaluation reports identified alkali-silica reaction (ASR) as one of the major sources of degradation in the existing concrete. ASR is a chemical reaction between alkali in the cement paste and silica in the aggregate that creates an expansive gel, which swells in contact water. The swelling of the gel creates an expansive force that causes cracking of the concrete. The concrete cracking then creates a path for further water ingress leading to progression of the reaction and the cracking as well as corrosion of the reinforcing. Petrographic analyses of core samples taken from the sea wall during the evaluation studies confirmed ASR through the presence of silica gel as well as fracturing of aggregates and cement paste. In general, it has been found that in reinforced concrete, the cracking does not extend past the reinforcing layer, but that depth of cracking is sufficient to allow accelerated corrosion of the rebar.

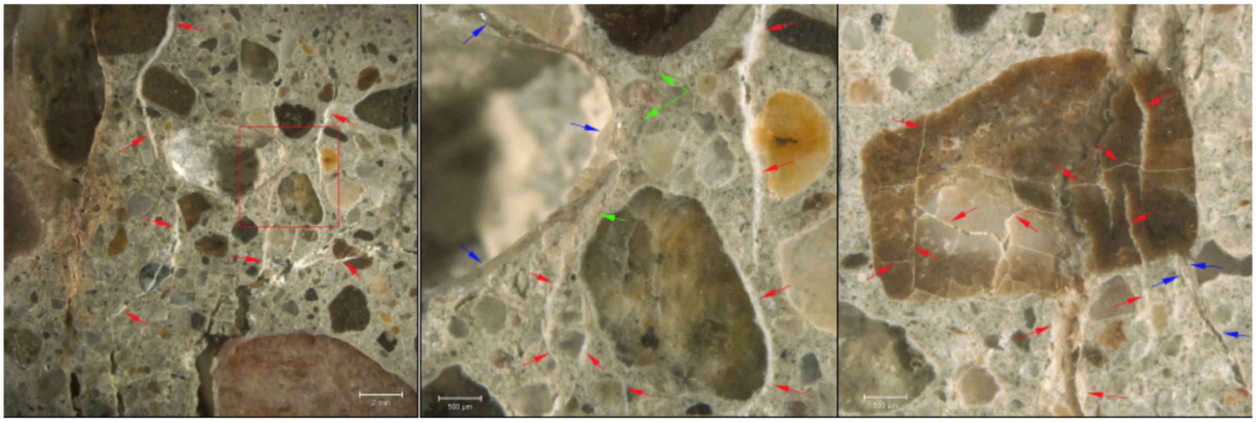

Figure 9 - Photomicrographs of polished core samples showing deposits of ASR gel (indicated by arrows) [from TranSystems (2011)]

ASR requires three elements to occur—alkali, susceptible aggregates and water—and stopping the reaction requires eliminating one of those components. The silica is in the aggregate and so cannot be removed. Likewise, while seawater supplies some alkali ions and probably contributes to the reaction, research indicates that seawater alone is usually not sufficient alone to trigger a severe reaction. Therefore, there is likely sufficient alkali in the concrete to cause a significant portion of the ASR that has been observed. That leaves the water as the only contributing element that can be controlled. While theory indicates that continuous submersion would be the most severe condition, observations have shown that areas subjected to cycles of wetting and drying exhibit more damage than areas that are always wet or always dry. That is consistent with the conditions seen at Mandalay Bay.

3.2.2 Surface Erosion

There are a number of areas on the walls where the surface concrete is deteriorated, but where there is limited cracking and the depth of degradation is limited and relatively uniform (as opposed to isolated areas with deeper losses, which would be more consistent with ASR.) This kind of surface erosion, which appears similar to the surface scaling that occurs on concrete paving that undergoes freeze-thaw cycles in the presence of salts, is not uncommon in seawall structures, particularly those with softer and more permeable concrete. The causes of these losses could include the erosive effect of wave action along with chemical interaction with seawater, including sulfate attack, which is described in more depth in the next section.

The primary mitigation method is to protect the concrete surface from the seawater with a barrier layer of some sort. This can include a layer of concrete that is proportioned to be hard and dense, and relatively immune to chemical attack.

3.2.3 Sulfate Attack

Sulfate attack is a chemical reaction between sulfate ions in water and constituents of the cement paste. The specific chemical reaction depends on the positive ions that are associated with the sulfate ions, but damage manifests in one or both of two ways: expansion and cracking of concrete and/or a decrease in strength and loss of mass. Seawater is classified as a source of moderate sulfate attack and in new construction, it is mitigated through the use of Type II cement. The previous evaluations did not identify sulfate attack as a cause of degradation, and because it manifests in a similar manner to ASR and surface erosion, which are both clearly present, without much in the way of distinguishing characteristics, that is not necessarily surprising. And because the same mitigation and protective measures effective against ASR and surface erosion will be effective against sulfate attack, it is not important to consider it further.

3.2.4 Reinforcing Corrosion

Corrosion, or rust, is simply the oxidation of iron. To progress, it requires iron, oxygen and an electrical connection between the points of reaction. In addition, concrete ordinarily is sufficiently alkaline to prevent the oxidation reaction. However, intrusion of chloride ions reduces the pH of the concrete, and the previous investigations have shown that chloride penetration throughout the walls has progressed far enough to overcome this ‘passivation’ effect. Nonetheless, the previous resistivity testing and destructive evaluations indicated that outside of areas with significant cracking on the front face or construction joint deterioration, there has been limited corrosion of the reinforcing to date. In part this is due to the saturation of the concrete, which limits the oxygen availability. Even in areas with cracking and significant rust staining, there are few spalls exposing rebar, which would be expected if the corrosion losses were severe, due to the expansive force of the rust. (Iron oxide is approximately six times the volume of the iron consumed in the reaction, creating significant internal pressure on the concrete surrounding the rusted reinforcing.)

3.3 POTENTIAL FAILURE MECHANISMS

For recognizing where degradation is becoming critical and in deciding on the most effective repair methods, it is helpful to understand some of the different potential failure mechanisms. Several of the mechanisms are listed below for each of the two wall types. This listing isn’t exhaustive, and many of these mechanisms could act in combination.

Some descriptions include an attempt to predict how ductile the failure mechanism is likely to be. A ductile failure is one where some movement of the wall can occur prior to complete failure. This movement will give some warning that a failure is becoming imminent and usually will reduce the earth pressure on the wall for some amount of time. However, the concrete cracking that will occur with the movement will accelerate corrosion and other deterioration mechanisms, so significant movement of a wall or parts of a wall should be considered a critical situation. In addition, for any mechanism that is expected to be ductile, combinations of effects can occur that lead to a brittle or sudden failure instead.

3.3.1 Boise Walls

3.3.1.1 Flexural Failure of Panel

Failure of the panels in the Boise wall would most likely occur in bending due to corrosion losses in the horizontal reinforcing. Cracking and concrete losses in the panels will increase the likelihood and rate of corrosion. Because the reinforcing is near the front face of the wall, significant corrosion of the reinforcing should be apparent first through rust staining of the concrete at cracks and eventually spalling of the concrete exposing the reinforcing. Since vertical reinforcing can redistribute load in the panel, it is likely that this type of failure will be ductile, possibly with visible bulging of the panel and formation of yield lines occurring before failure.

3.3.1.2 Tieback Failure (at Pilaster)

The pilasters in the Boise wall system rely on the steel tiebacks to hold them in place. There are three locations the tiebacks can fail: at the head, which is embedded in the pilaster; at the deadman end, which is also embedded in concrete; and in the middle section, which is wrapped in a protective sheath. Based on the results of the destructive investigations, the risk of this failure method currently appears low, but the consequences of a tieback failure would be high.

Failure at the head in the pilaster would most likely be a result of corrosion in the nut and bearing plate. As such, this type of failure would likely be sudden and non-ductile. As part of the previous evaluations, at pilasters with rust staining at the anchor head locations, the heads were found to have very light corrosion when they were exposed. Therefore, severe corrosion in the anchor heads is likely to cause significant rust staining on the surface of the pilaster. In addition, corrosion severe enough to fail the anchor may cause spalling that exposes the anchor head. Failure at the deadman would be similar but is less likely because the deadman is in a much less corrosive environment. Corrosion in the length of the tieback will not be readily apparent, but the previous investigations suggest that the existing sheaths have been effective in protecting the tiebacks. It is likely that complete failure in the length of the tieback behind the wall will be preceded by significant stretching of the tieback and leaning of the pilaster, but sudden failures are possible if corrosion creates a notch in the steel.

As noted above, possible signs that this type of failure is occurring or imminent would be severe rust staining and/or concrete bulging on the pilaster surface above the tieback head; a concrete pop out that exposes the anchor head; or a pilaster that is leaning outward, particularly where the adjacent pilasters remain vertical or nearly so.

3.3.1.3 Flexural Failure of Pilaster

Similar to the panels, a bending failure in the pilaster would be the result of reinforcing losses due to corrosion. However, because the panels can redistribute load vertically along the pilaster if part of it weakens, this mechanism is believed to be unlikely on its own to lead to the failure of a wall.

3.3.1.4 Shear Failure at Pilaster Flange

The panels of the Boise walls are held in place by the flanges of the pilasters. Therefore, the flanges of the pilaster were to separate from the body, the panel could fall. This failure of the flange could be in either bending or shear. A flexural failure sufficient to allow the panel to fall would require very significant loss of reinforcing or of the concrete on the pilaster surface, so a shear failure is the more likely mechanism. The most likely form of shear failure would occur when expansive cracks on the front face of a pilaster connect across to flexural cracks forming at the corner on the back face. Simple calculations show that as few as one or two #4 reinforcing bars at the top of the wall provide enough shear friction capacity to hold a wall panel in place. Because there is approximately three feet of concrete in good condition at the top of the pilasters above the tidal zone where the reinforcing is likely to be mostly uncorroded, the likelihood of complete loss of a panel due to this mechanism is currently judged to be low, since the panel will likely fail in bending before the pilaster flange breaks.

Signs of this failure mode may include vertical cracking on the face of the pilaster over all or most of the height along with displacement of the flange outward away from the rest of the pilaster in some combination of rotation and offset across the crack.

3.3.1.5 Shear Failure at Base of Pilaster

The bases of the Boise wall pilasters are held in place by a lip on the footing, which would have to break in order for the pilaster to move. (This lip restrains the bottoms of the panels as well.) It is unclear if this lip is reinforced. Failure by this mechanism is believed to be unlikely unless there are significant concrete losses and/or major cracks in the footing adjacent to a pilaster.

3.3.1.6 Marine Borer Attack on Piles

Timber piles exposed directly to sea water are prone to attack by marine borers that will eat away at the piles reducing their capacity, eventually leading to failure of the wall. While repair of damaged piles is possible, it is an expensive process. Thus, it is best to repair any areas where the ground surface has eroded below the bottom of the footings as quickly as possible.

3.3.2 Zurn Walls

3.3.2.1 Concrete Failure

In this failure mode, degradation of the concrete weakens it to the point where the concrete crushes under the load, or concrete losses are so severe that the reinforcing yields due to the reduced section depth. The ductility of this mode depends on whether the reinforcing yields or the concrete crushes. In the case of reinforcing yielding, the failure should be ductile, but less so if the concrete crushes. Regardless, once failure has begun it will likely progress quickly. Because the concrete degradation is most severe a few feet above the base of the wall, a change in the slope of the wall partway up the face (e.g. the lowest part of the wall remains approximately vertical while the upper portion of the wall is leaning outward) could be an indicator that an area of wall is failing in this way.

3.3.2.2 Reinforcing Corrosion (General)

The reinforcing in the Zurn walls is at the back face, which makes it less prone to accelerated corrosion than the Boise walls, particularly since the investigations of the back faces of the walls found them to be in reasonably good condition. Corrosion will be accelerated if the reinforcing is exposed to seawater. This could occur due to water penetrating from the front surface through ASR-related cracking and general erosion of the concrete, or from the back face through flexural cracks. Water penetrating from the back face will supply less oxygen than water from the front face and has to seep through fairly small cracks, limiting the corrosion rate. Water penetrating through the front face will provide more oxygen due to tidal and wave action but has to travel further to reach the reinforcing.

In general, it is expected that corrosion of the Zurn wall reinforcing should progress slowly until the degradation of the front face progresses significantly into the concrete. This is consistent with the observations from the destructive evaluations. This mechanism is likely to combine with concrete losses on the front face of the wall leading to failure sooner than if there were no reinforcing losses. Typically, this type of corrosion will be distributed along the length of the reinforcing, limiting the amount of losses at any one location and allowing for a more ductile failure when the wall is overloaded.

3.3.2.3 Reinforcing Corrosion (At Base)

In this failure mode, the corrosion in the reinforcing is concentrated at the base as a result of seawater penetrating at the construction joint between the wall and the footing. Typically, this will also involve concrete degradation at the joint to increase the amount of seawater reaching the reinforcing. Because the reinforcing corrosion is likely to be more rapid in this case and to be concentrated at the point of maximum demand, there is a higher risk of a brittle failure of the wall for this failure mode. Therefore, areas where there is an obvious gap or deterioration in the construction joint at the bottom of the wall should be considered a high priority for remediation. The most likely indicator of this failure mode is rotation of the wall at the joint with the footing (i.e. closing of the angle between the wall and the footing so that it is less than 90 degrees.)

3.3.2.4 Marine Borer Attack on Piles

This mechanism is the same as for the Boise walls.

4.0 REPAIR/REHABILITION

4.1 GOALS

4.1.1 Improve Durability/Extend Service Life of Seawalls

The primary goal of the repair and rehabilitation effort is to extend the life of the existing seawalls. The expected effective life of different repair methods will be considered in the assessments, as will the reliability of those estimates. For instance, it is easier to assess service life for repair methods that rely entirely on new construction, as opposed to methods that overlay or strengthen the existing construction.

An additional consideration in extending the life of the seawalls, is addressing their seismic vulnerability. The ability of retrofit schemes to improve the seismic stability of the seawalls will be considered when assessing the pros and cons of different repair methods.

4.1.2 Minimize Impacts to Homeowners

Quality of life for local residents will be impacted by the construction work required for the repairs and their inconvenience and discomfort need to be weighed against the improvements realized by any of the repair schemes being considered.

4.1.2.1 Overhanging Decks

In many locations, homeowners have decks that cantilever over the top of the seawall, complicating the work and limiting the repair options available. However, removal and replacement of the decks, in addition to being a significant expense will be a major inconvenience to the residents. Therefore, options that avoid the need to remove or alter decks are preferred.

Figure 10 - Typical overhanging decks

4.1.2.2 Minimize Noise and Vibration

Noise and vibration, for instance from pile driving or concrete demolition, will be the most noticeable issue with the repair work for most people. Severe vibration could also cause localized liquefaction, which in turn could cause settlement of the surrounding ground, Therefore, schemes that minimize noisy work and vibration are preferred. It should be noted that piles and sheet piles in clay and sand can often be installed by the press-in method, which creates minimal noise and essentially zero vibration in most cases.

4.1.2.3 Work from the Water Side of the Wall

Work from the land side of the wall, by necessity will be done on people's property. Therefore, as much of the work as possible should be done from the water side off of work boats or floats.

4.1.2.4 Limit Amount of Time Docks Are Removed

Loss of access to the water and watercraft will be an annoyance to the residents. Therefore, schemes that minimize the amount of time that docks must be removed and stored are preferred.

4.1.3 Minimize Costs

There are approximately 7 miles of seawall in the area covered by this report. Therefore, even small cost savings will have an impact and potentially will enable acceleration of the repair work.

4.1.4 Project Phasing Plan Based on Severity and Risk

The project should be phased and scheduled both to minimize overall risk to the community and cost of construction. The primary consideration is that work on the areas of wall in the worst condition should be prioritized. This is both because those areas are at the greatest risk of failure and because deterioration will tend to accelerate as the walls degrade, leading to increased costs for repair. A secondary, but important consideration is to phase the work in ways that will reduce the cost of construction. The main example is to repair whole sections of wall at one time instead of skipping around to individual spots, thereby minimizing mobilization costs. Finally, because the repair work will extend over many years, the condition of the walls should be periodically surveyed, and adjustments made to the phasing and scheduling as appropriate to deal with areas that are deteriorating faster than was expected.

For the Phasing Plan, the study area has been broken down into five groups. Four of them are about the same size, with one group—the lowest rated Zurn walls—about half the size of the others. These areas are too large to tackle as stand-alone projects. We anticipate that once we have a better idea of the City's budgetary constraints, we will refine this into more manageable sized projects. The following are general notes to outline the Phasing plan methodology.

- The Boise walls are believed to be more seismically vulnerable and more prone to a sudden failure under static loading than the Zurn walls. Therefore, the most deteriorated Boise walls have been given the highest priority.
- Considering only static loads, the Zurn walls with the highest level of deterioration appear to be the next most critical area. There are a limited number of these, so they have been placed together in a smaller group. (A section of Zurn walls with poor visual ratings on the East side of the fairway was included in the first group due to the proximity to the Boise walls.)
- The remaining Boise walls were placed in the third group. This is primarily because it is felt they are at higher risk of significant displacement in a major earthquake than the Zurn walls.
- The remaining Zurn walls are in the last two groups. Generally speaking, Group 4 is Tracts 2 and 3, and Group 5 is Tract 3 since the walls in Tract 3 are newer and are generally in better condition

OVERALL SITE PLAN
SCALE: 1" = 350'

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CALIFORNIA

CITY OF OXNARD
PUBLIC WORKS DEPARTMENT
305 WEST THIRD STREET
OXNARD, CA 93030

TETRA TECH

MANDALAY BAY SEAWALLS
OXNARD, CA

FIGURE TITLE
PHASING CONCEPT PLAN

PROJECT NO.

SCALE
1" = 350'

DATE
2/3/2020

Figure 11

4.2 REPAIR SCHEMES CONSIDERED

4.2.1 Boise Walls

4.2.1.1 Tieback Wall (Scheme A)

This option would install new precast concrete panels in front of the existing wall on top of the existing footing, with the gap between the new and existing panels filled with grout. The new panels would be held in place with tiebacks. Depending on the panel sizes and the tieback spacing, a waler beam may be required as well. The tiebacks would be angled down at approximately 20 degrees from horizontal and would be long enough to reach the non-liquefiable dense sands starting at approximately El -12 to El -17.

Because this option relies on the existing footing and piles, those elements will need to be maintained and protected. This includes preventing undermining of the footing that would allow marine borers access to the footing.

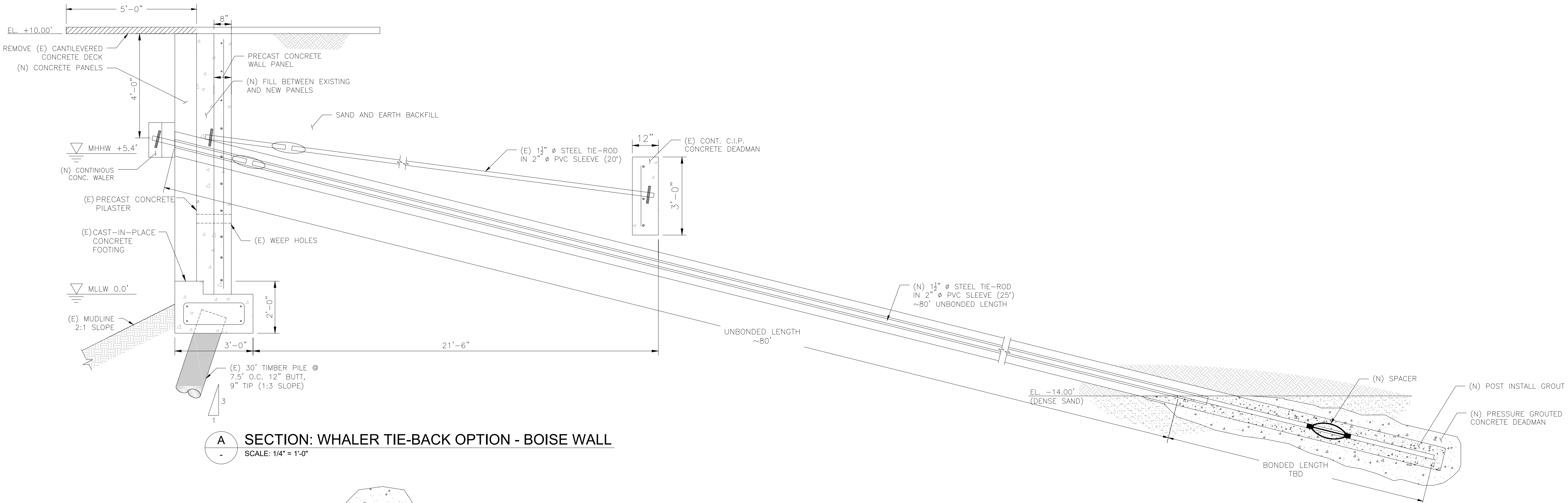
This option has a couple of issues that need to be verified through additional geotechnical analysis to confirm this is a viable option. First, because the footing sits within liquefiable material, it is possible that in a severe earthquake, liquefied sand could flow out underneath the footing and between the piles causing settlement. Second, the tiebacks will create an additional downward load on the existing footing and piles, and the ability of the existing foundation to take that load will need to be confirmed. This is a particular concern for the Boise walls, which have a single line of piles.

Pros:

- Minimal disruption to residents
- Relatively inexpensive
- Can be designed for seismic load

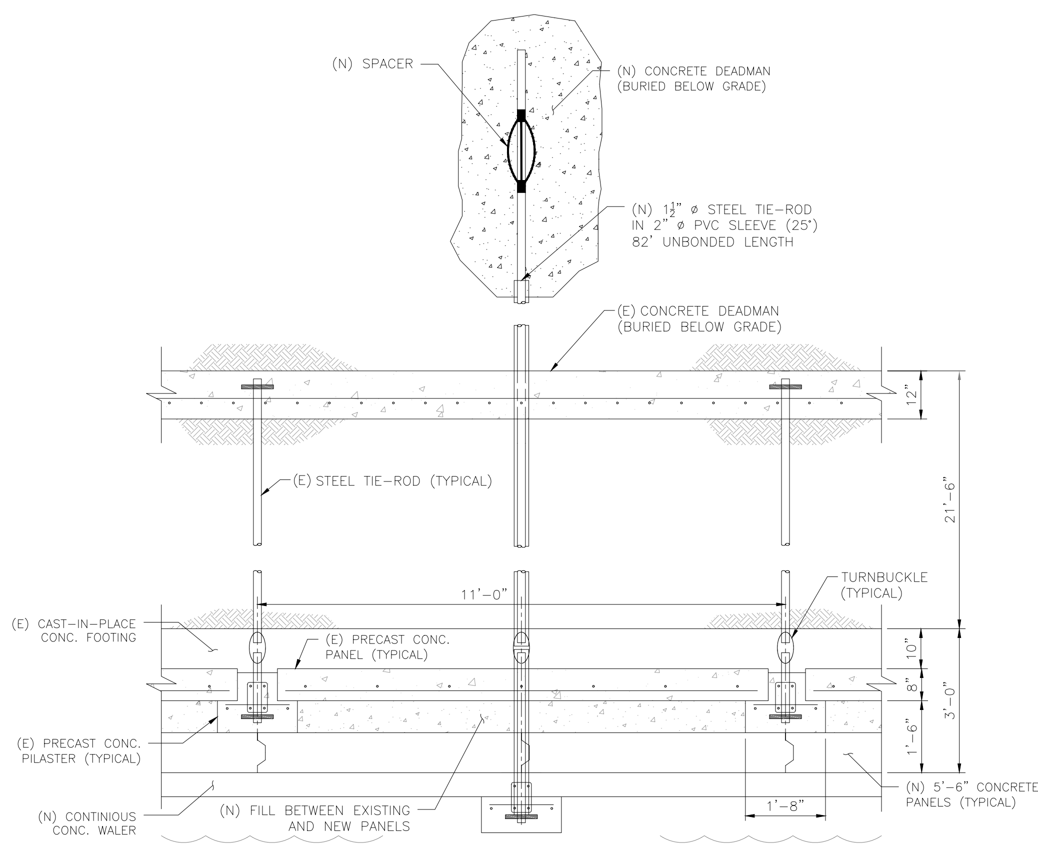
Cons:

- Relies on the existing footing and piles for vertical support.
- Some potential geotechnical issues need to be confirmed to ensure this option is viable



A SECTION: WHALER TIE-BACK OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"



1 PLAN: WHALER TIE-BACK OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"

Figure 12

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4.2.1.2 Steel Sheet Pile Wall Installation (Scheme B)

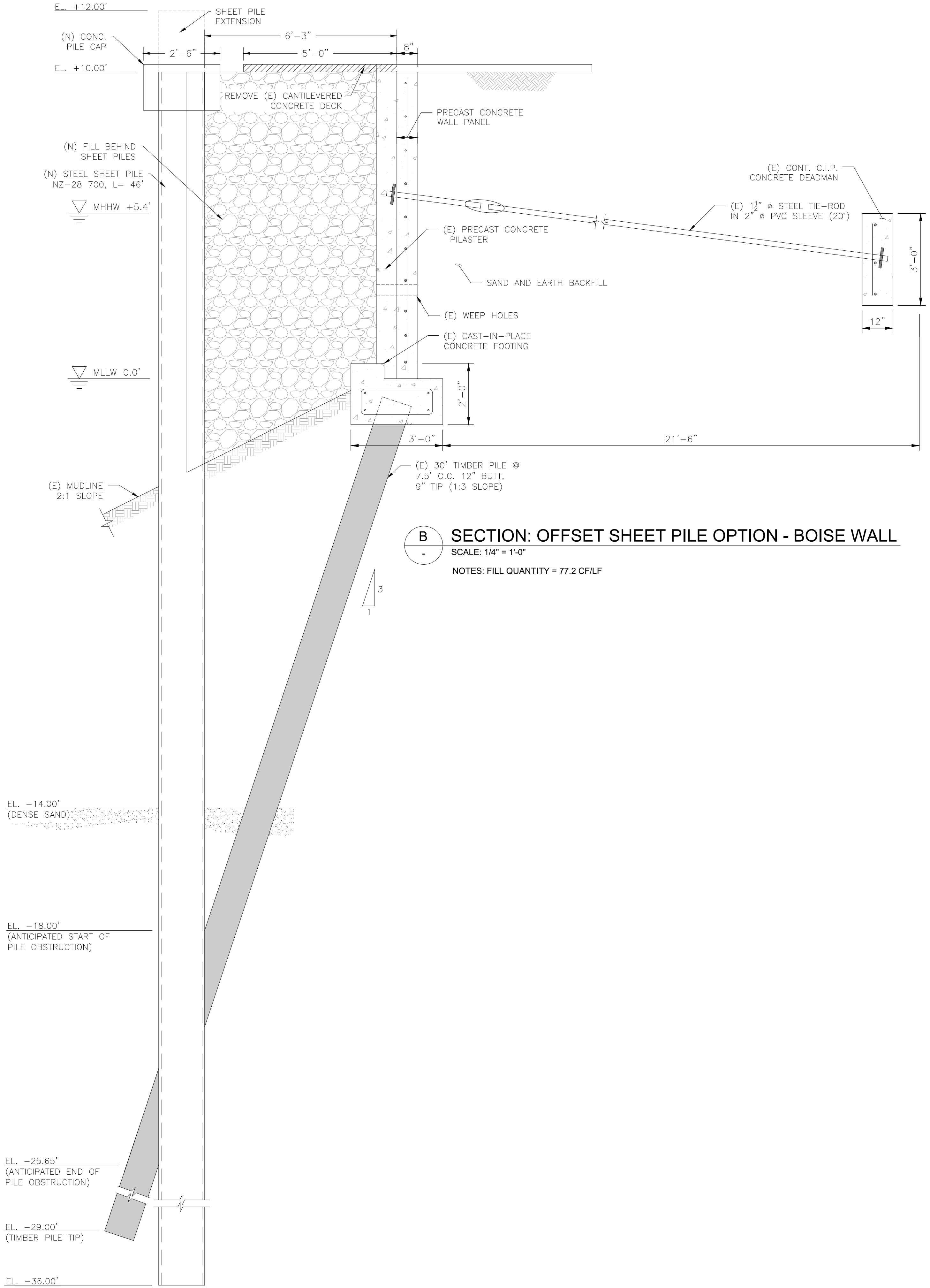
This scheme would install a new steel sheet pile wall in front of the existing seawall with backfill between the new and existing walls. The advantages and disadvantages of this option are similar to Scheme C, but the need to potentially remove overhanging decks makes it somewhat more disruptive. Drawings of the original construction show the piles as 30 feet long at a 3 to 1 batter. Therefore, in order to miss the existing battered piles, the new sheet pile would have to be placed approximately nine feet out from the face of the existing wall if the piles are the full length. It is likely that at the distance, most of the overhanging decks would be far enough from the work that they could be left in place. The primary advantages of this wall type are that it can be designed to resist code-level seismic and liquefaction loading, and it is a very standard type of seawall construction.

Pros:

- Permanent solution that would completely replace the existing seawall with new construction more resistant to deterioration
- Could be designed for liquefaction making the seawall more earthquake resistant than currently
- Very typical seawall construction method

Cons:

- Expensive
- Extends significantly beyond current footprint of wall and footing.
- Could be relatively disruptive to homeowners. Would possibly require removing and replacing some decks that overhang the seawall.
- Large offset will require significant amounts of fill and coverage of existing bottom could make permitting difficult.

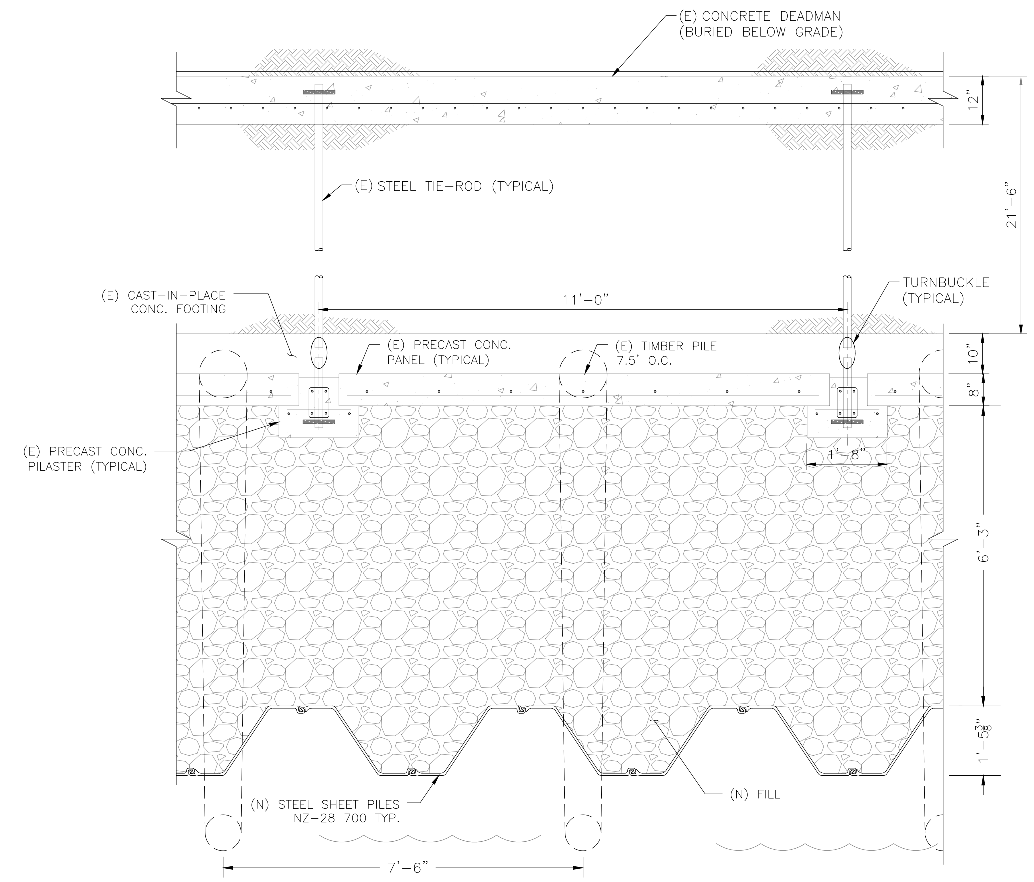


B
-

SECTION: OFFSET SHEET PILE OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"

NOTES: FILL QUANTITY = 77.2 CF/LF



2
-

PLAN: OFFSET SHEET PILE OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"

NOTES: FILL QUANTITY = 77.2 CF/LF

Figure 13

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4.2.1.3 New Cantilever Soldier Piles with New Panels (Scheme C)

In this option, new soldier piles would be driven in front of the footing of the existing wall, panels would be placed between the piles and the gap between the existing wall and the panels would be filled. Because the headroom above the wall is limited where decks overhang it, the panels would be individual pieces of limited height stacked to reach the top height of the new wall. Piles could be either precast concrete tee shapes (most likely prestressed), or stainless steel or carbon steel H-shapes. The panels would most likely be precast concrete, set behind precast concrete piles or between the flanges of steel H-piles. Alternatively, fiberglass may be an option for the panels. The fill would either be sand or gravel, or a very low strength concrete known as CDF, depending on which is most economical. The new piles can be placed to avoid the existing battered piles, so the new wall could be placed close the face of the existing footing at the potential cost of having to adjust panel lengths in a few places if a new pile hits an existing pile and has to be moved.

Because of the limited headroom the piles would have to be driven from above the deck level. This means that holes would have to be cut through any overhanging decks wherever piles are to be driven and then repaired afterward. The tradeoff for this extra level of disruption to the homeowners, is that the new wall would be structurally independent and therefore the life of this construction would not be dependent on the existing wall with its ASR problems. As such, the wall could be designed with an expected life of 75 years or more. In addition, the wall could be designed to resist loads due to seismic events, including liquefaction of the fill, which would be an improvement on the existing situation.

Pros:

- Permanent solution that would completely replace the existing seawall with new construction more resistant to deterioration
- Could be designed for liquefaction making the seawall more earthquake resistant than currently

Cons:

- Expensive
- Extends outside current footprint of wall and footing.
- Disruptive to homeowners. Installing soldier piles will require cutting holes in decks that overhang the seawall.
- Pile driving likely to be objectionable to residents and could lead to liquefaction and settlement. (Alternative pile installation methods to avoid this issue will significantly increase cost for this option.)
- If not carefully constructed, lagging could be uneven and/or allow leaks of backfill, which would be unattractive.

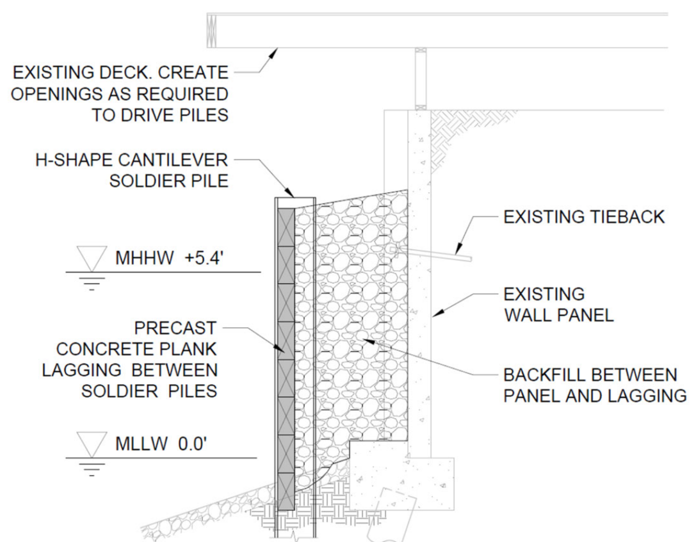


Figure 14 - Cantilever soldier pile and lagging

4.2.1.4 New Soldier Piles Tied to Existing Pilasters with New Panels (Scheme D)

This is similar to the option above, but because the piles would be secured to the existing footings and pilasters, the wall could be installed entirely below the decks. In this case, the piles would be steel H-sections—most likely stainless steel--and would be secured at the base and near the tieback elevation. The options for panels and fill are the same as for the cantilever soldier pile option except that the gap to be filled would be much smaller, increasing the likelihood that CDF would be the more economical option. This option avoids the need to touch the decks, but because it would still rely on the existing footings pilasters and tiebacks it will still be vulnerable to some of the existing degradation modes, although they will be slowed to some extent by the addition of the new layer.

Pros:

- New panels functionally replace the existing panels, which are the hardest and most expensive part of the existing system to protect.
- Relatively quick to install
- Required pile size smaller than for cantilever option
- Low level of disruption

Cons:

- Expensive, but likely less so than the cantilever option
- Does not do anything to protect the pilaster or footings, but still partially relies on them.
- Covers, but does not fully protect existing wall. Hides existing from inspection.
- Does not address seismic vulnerability

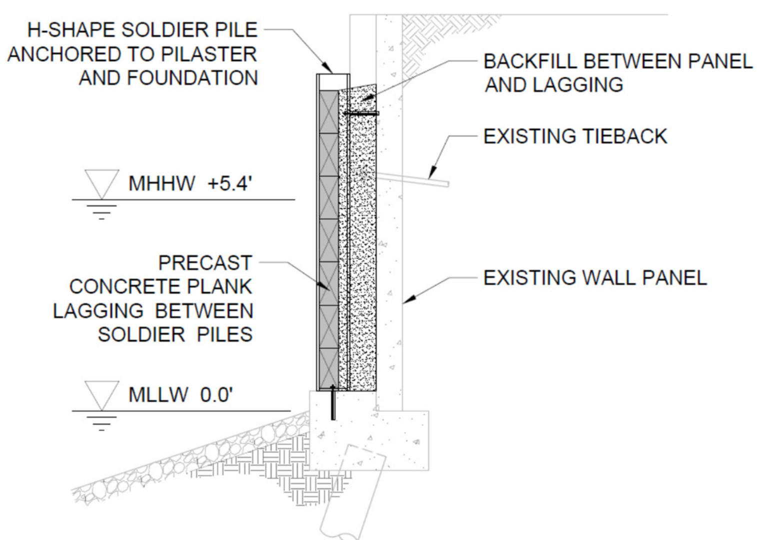


Figure 15 - Soldier piles bolted to existing pilasters

4.2.1.5 New Concrete Pilaster Jackets and Panel Facing (Scheme E)

This is the method proposed in Phase C of the previous work. It consists of a new concrete jacket around the pilaster and a new approximately 4” thick concrete facing on the panels. Surface preparation off the existing concrete would consist of chipping with handheld tools to remove loose concrete. The new concrete should be a low-permeability marine mix proportioned following ACI recommendations for limiting susceptibility to ASR and to marine attack. The pilaster repair is the same as proposed in Phase C of the previous work. The pilaster jackets and panel facings could be done independently depending on the repair priorities at different locations.

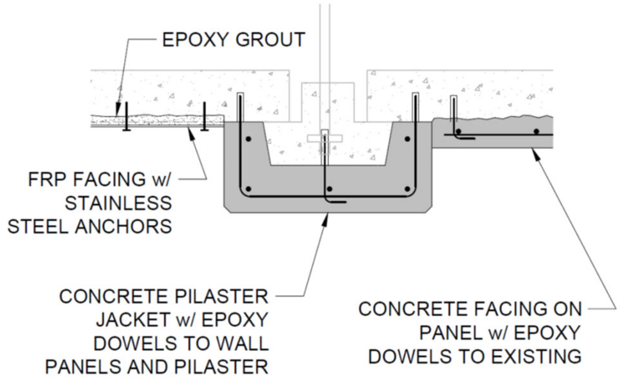


Figure 16 - Concrete pilaster jacket (shown with concrete and FRP panel facings)

Pros:

- Creates a barrier layer limiting seawater penetration to the existing concrete slowing the alkali silica reaction and reinforcing corrosion.
- Reinforcing in pilaster jackets and panel facings will resist expansive forces from continuing ASR.
- Existing jackets done for previous repairs appear to be surviving well.
- Conventional construction allows for a large pool of potential contractors.
- Separates repairs of pilasters and panels, so pilasters (which are generally in worse condition) can be prioritized.

Cons:

- Existing reinforcing may continue to corrode
- Extension of design life unknown
- Does not address seismic vulnerability

4.2.1.6 Fiberglass Pilaster Jackets and Panel Facings with Epoxy Grout (Scheme F)

This option is similar to Scheme A but uses FRP pilaster casings and panel facings with epoxy resin filler instead of concrete. The pilaster casings would be U-shaped and would need to be held in place by bolting to the faces of the panels on either side of the pilaster. The panel facings would be secured to the existing concrete using anchors spaced as required by the manufacturer. The pilaster casings would have a minimal annular space (approx. ½ to 1 inch). Concrete preparation for the pilasters would be the same as Scheme A, but the preparation of the panels would be by hydro blasting to a depth of 1 to 1½ inches (plus removal of any loose concrete. The panel repair is the same as proposed in Phase C of the previous work. As with Scheme A The pilaster jackets and panel facings could be done independently depending on the repair priorities at different locations, and concrete plaster jackets could be matched with FRP panel facings or vice versa.

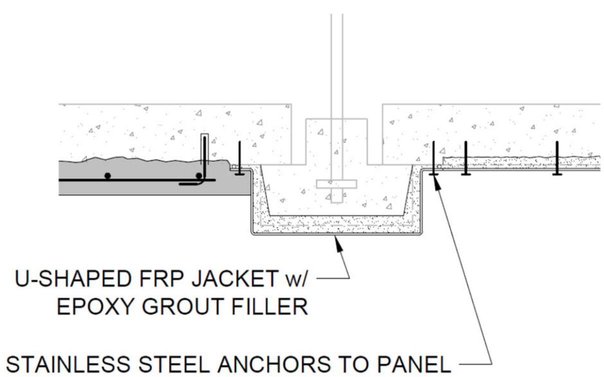


Figure 17 - FRP pilaster jacket (shown with concrete and FRP panel facings)

Pros:

- Creates a barrier layer limiting seawater penetration to the existing concrete slowing the alkali silica reaction and reinforcing corrosion.
- FRP pilaster jackets will resist expansive forces from continuing ASR on front face of pilaster.
- Lighter weight and easier handling may allow use of smaller equipment.
- Separates repairs of pilasters and panels, so pilasters (which are generally in worse condition) can be prioritized.

Cons:

- Limited number of manufacturers for FRP casings
- U-shaped casings not a typical application of FRP
- Likely to be more expensive than concrete repairs
- Existing reinforcing may continue to corrode
- Extension of design life unknown
- Does not address seismic vulnerability

4.2.1.7 Riprap Stabilization (Scheme G)

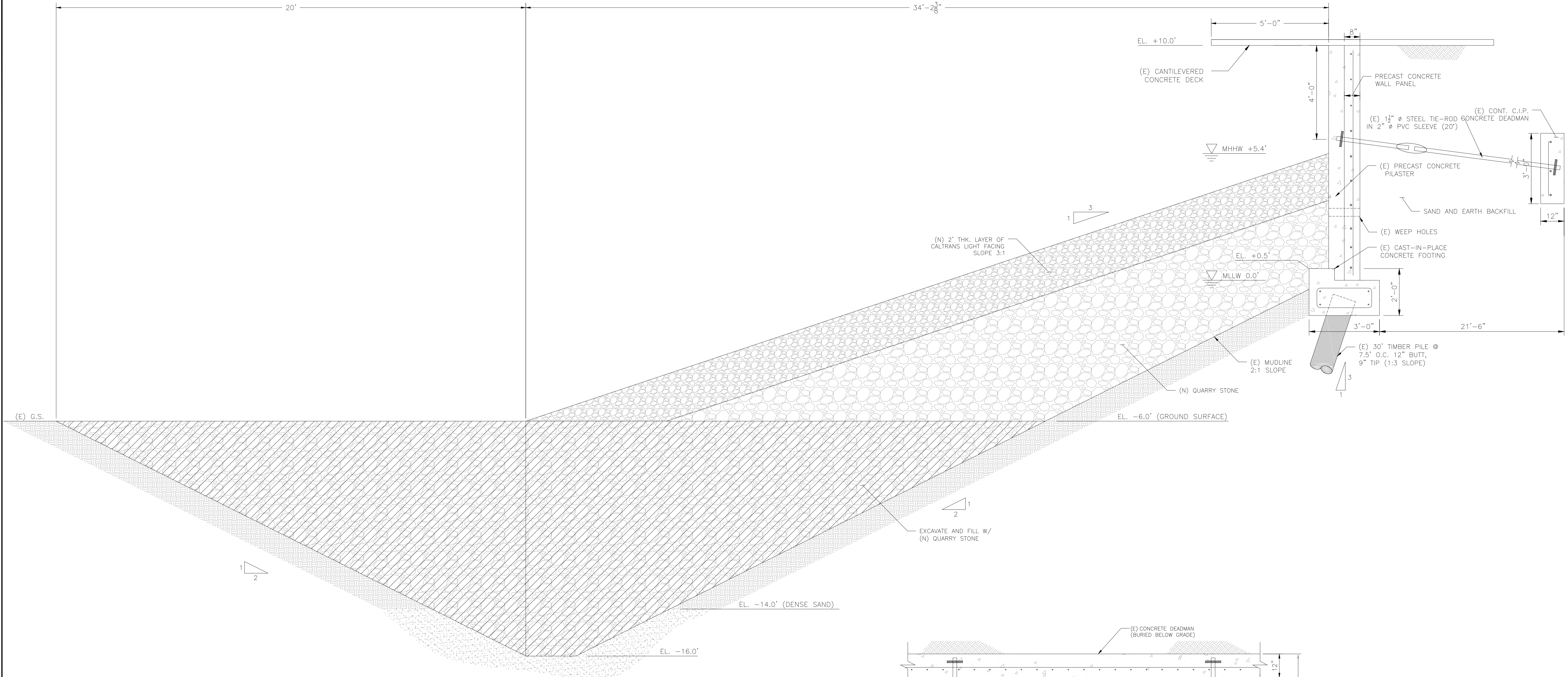
A very simple option for stabilizing the walls would be to install riprap (or other fill protected by rip rap) on the water face of the walls to stabilize them. The height and angle of the fill would be such that the fill would be stable on its own and would not rely on the existing wall. Designing for seismic stability would require a flatter slope (i.e. more material and larger covered area. Seismic stability will also require the new fill to be extended down to the dense soil found around Elevation -15 (approximately 10 to 12 feet below the existing channel bottoms), requiring a large amount of excavation and disposal, and significantly increasing the amount of material required. Further, a conceptual-level geotechnical analysis found that even a slope of 3:1 (horizontal to vertical) would not provide the resistance required to hold the existing ground level once the existing walls have lost capacity. Therefore, it has been judged not feasible to achieve the required seismic resistance using this scheme.

Pros:

- Essentially permanent
- Can be installed quickly with relatively simple mobilization
- Minimal engineering required

Cons:

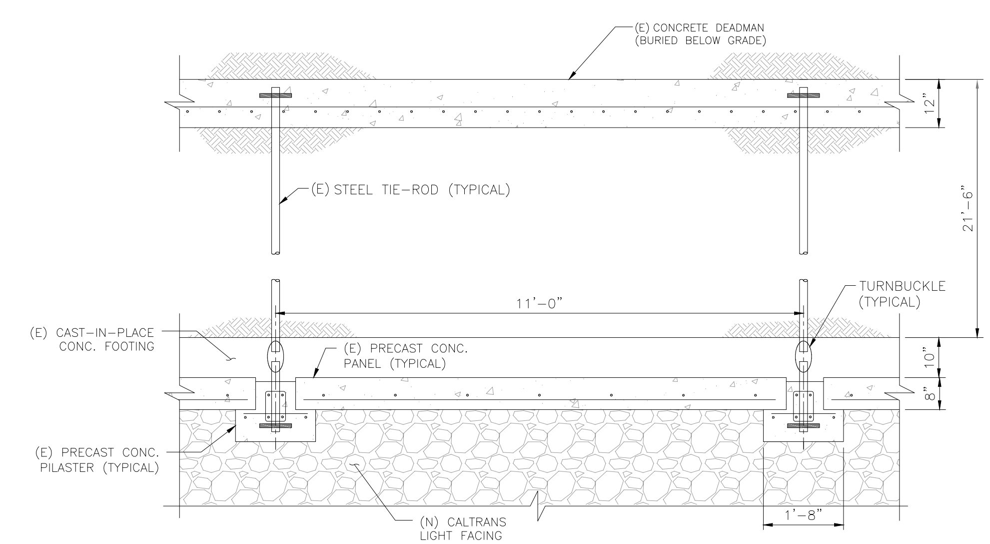
- Would require docks be moved further away from walls, which would require revisions to pier head line. In narrower channels there may not be sufficient room, so might require elimination of docks, which is assumed to be unacceptable to homeowners. Therefore, probably only feasible on wider channels.
- Feasibility depends on slope and depth of channel bottoms at base of walls. Limited information currently available so would require bathymetric survey.
- Large encroachment on existing soft bottom, and significant excavation will cause permitting difficulty.



C
SECTION: ROCK REVETMENT OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"

NOTES: EXCAVATION QUANTITY = 221.3 CF/LF
QUARRY STONE TOTAL = 316.9 CF/LF
CALTRANS LIGHT FACING = 62.4 CF/LF



3
PLAN: ROCK REVETMENT OPTION - BOISE WALL

SCALE: 1/4" = 1'-0"

Figure 18

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4.2.1.8 Epoxy Crack Injection of Panel Faces (Scheme H)

In lieu of refacing the panels, epoxy injection of cracks could be considered. The key caution with this option is that if the conditions that caused the initial cracking are still active, the concrete is likely to crack again adjacent to the repaired cracks. However, given that the wall panels are nearly 50 years old and most are still in serviceable condition, the extension of service life of the panels from this option could be adequate even if they re-crack, as long as the locations of corrosion in the reinforcing are shifted. This option won't do anything to repair or protect the panel surface from further deterioration, so is probably not viable where there is significant degradation of the panel surface.

If the city wishes to investigate this option further, a test program could be instituted where some number of panels with a variety of crack patterns and levels of severity are repaired using this method then evaluated periodically to determine the extent to which the cracking reoccurs.

Note that, if selected, this option should be combined with the pilaster jacketing from Scheme A or Scheme B. Epoxy injection is not considered to be an adequate repair for any but the most lightly deteriorated pilasters.

Pros:

- Relatively low cost
- Quick to accomplish

Cons:

- Does not repair the concrete surface or protect it from further deterioration
- Does not strengthen the wall panels
- Concrete may crack again adjacent to the repaired cracks in a short amount of time.
- Extension of service life is uncertain and will depend on extent, locations and severity of re-cracking
- Repair locations will be obvious and may be considered unattractive
- Does not address seismic vulnerability

4.2.1.9 Stainless Steel Reinforcing Straps (Scheme I)

In this repair, after removing loose concrete in the immediate area, stainless steel straps would be set on a thin bed of epoxy mortar and bolted to the exterior face of a pilaster or panel parallel to the primary reinforcement (vertical on pilasters, horizontal on panels.) These straps would serve as external reinforcing to replace corroded rebar in the concrete. Outside of the concrete directly underneath them, the straps will not repair existing damage or protect the concrete against continuing degradation, so the effectiveness of this repair will decrease as the concrete continues to degrade, around and underneath the straps. Therefore, the effective life in most cases will be limited—potentially in the worst cases to as little as a few years. However, because these straps could be fabricated quickly and installed by a small crew with a minimum mobilization effort, this may be a good option for emergency repairs.

Pros:

- Can be installed quickly with minimal mobilization
- Straps could be prefabricated for use in emergency situations

Cons:

- Does not do anything to slow concrete degradation
- Continuing damage to concrete around straps will eventually reduce effectiveness of repair.
- Extension of design life unknown, but may be short
- Does not address seismic vulnerability

4.2.1.10 Replacement In-Kind (Scheme X)

In-kind replacement of the Boise Walls is not practical. Providing enough length for effective tiebacks in front of the existing wall would place the new walls too far out into the channel. Reuse of the existing tie backs is possible but would require a very complicated sequence of temporary shoring, demolition and new construction; and replacement of damaged tiebacks would be nearly impossible since the anchored ends are now underneath houses in most cases. Even excluding those difficulties, walls supported by tiebacks are susceptible to damage and even collapse due to earthquake-induced liquefaction and the use of tieback walls would usually not be recommended for new permanent construction today in an area like this.

Therefore Zurn-type cantilever walls have been assumed as the 'in-kind' replacement for the Boise Walls. This option is described below in the Zurn Wall section as Scheme X. Use of this scheme for replacement of the Boise Walls would have the same pros and cons as for the replacement of the Zurn walls

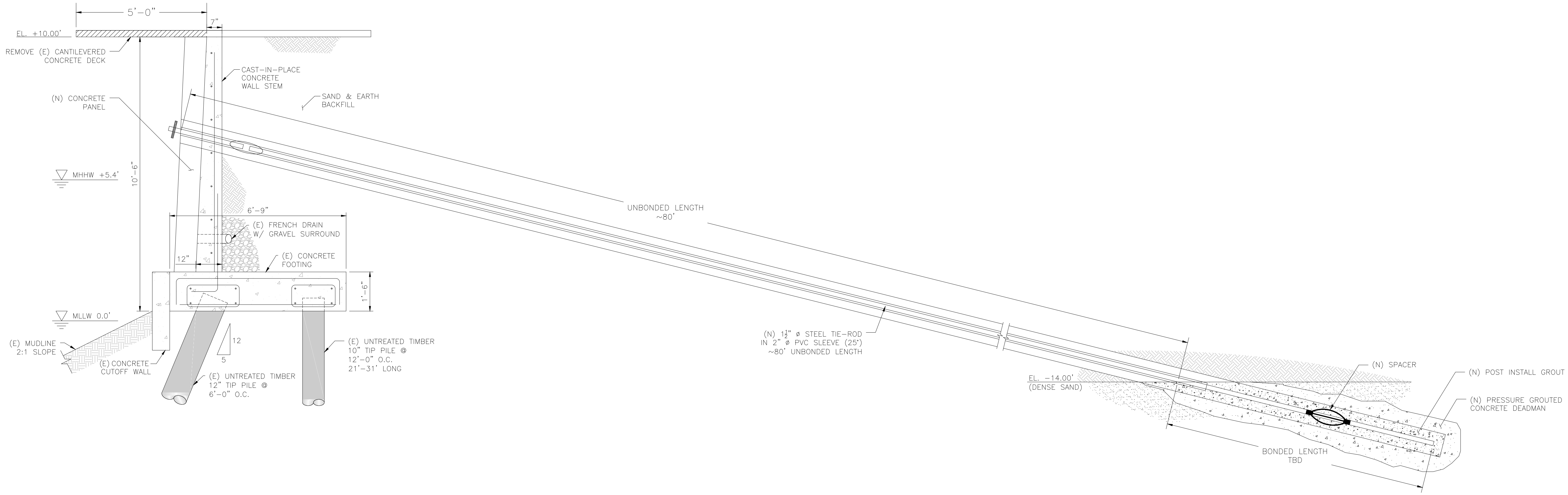
4.2.2 Zurn Walls

4.2.2.1 Tieback Wall (Scheme J)

This scheme is essentially the same as for the Boise walls, but the wider footing with two lines of piles makes the geotechnical issues less likely to preclude this option. (See Figure 19)

4.2.2.2 Steel Sheet Pile Wall Installation (Scheme K)

This scheme is essentially the same as the steel sheet pile wall option discussed for the Boise walls.
(See Figure 20)



D
-
SECTION: WHALER TIE-BACK OPTION - ZURN WALL

SCALE: 1/4" = 1'-0"

Figure 19

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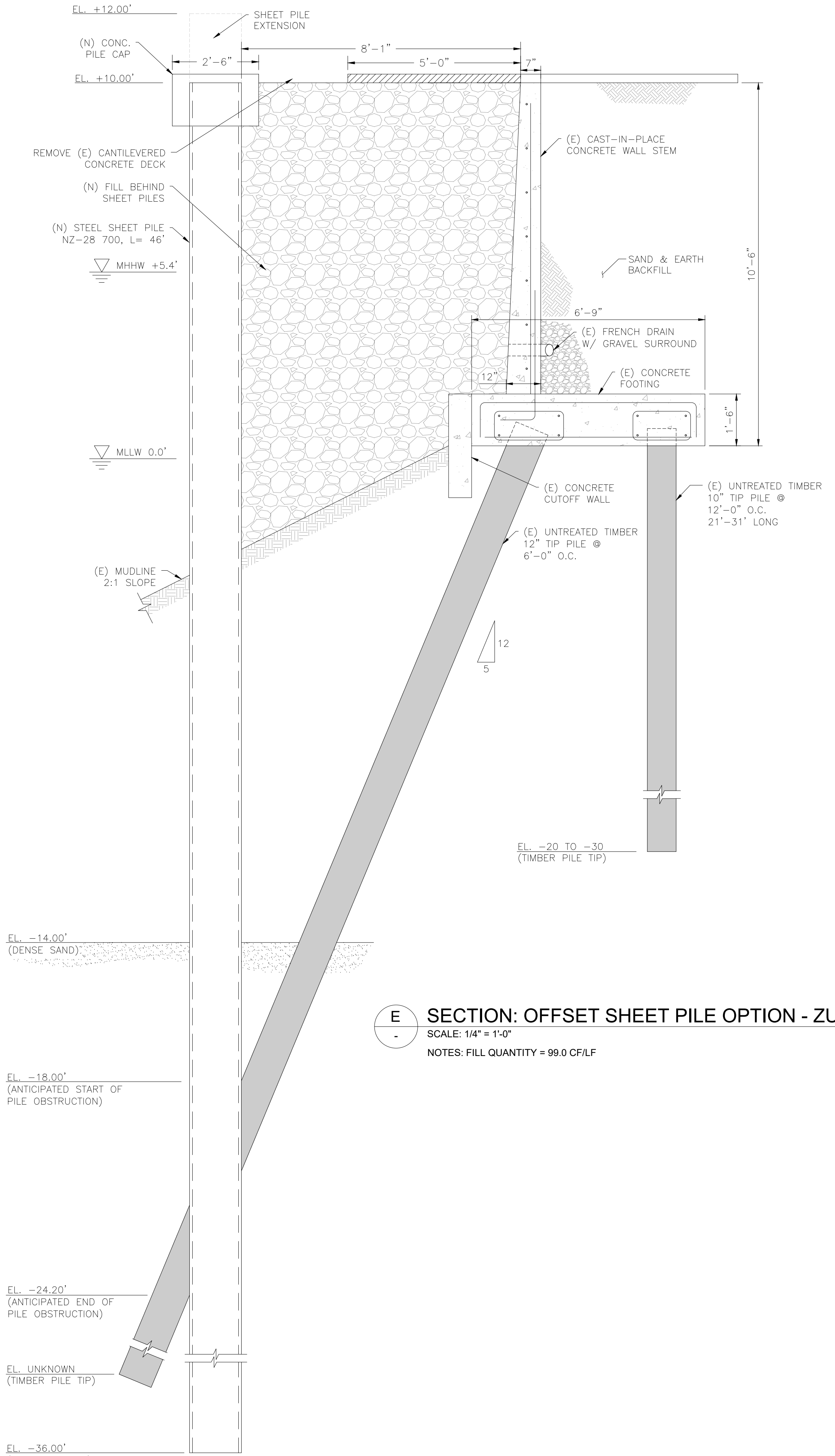
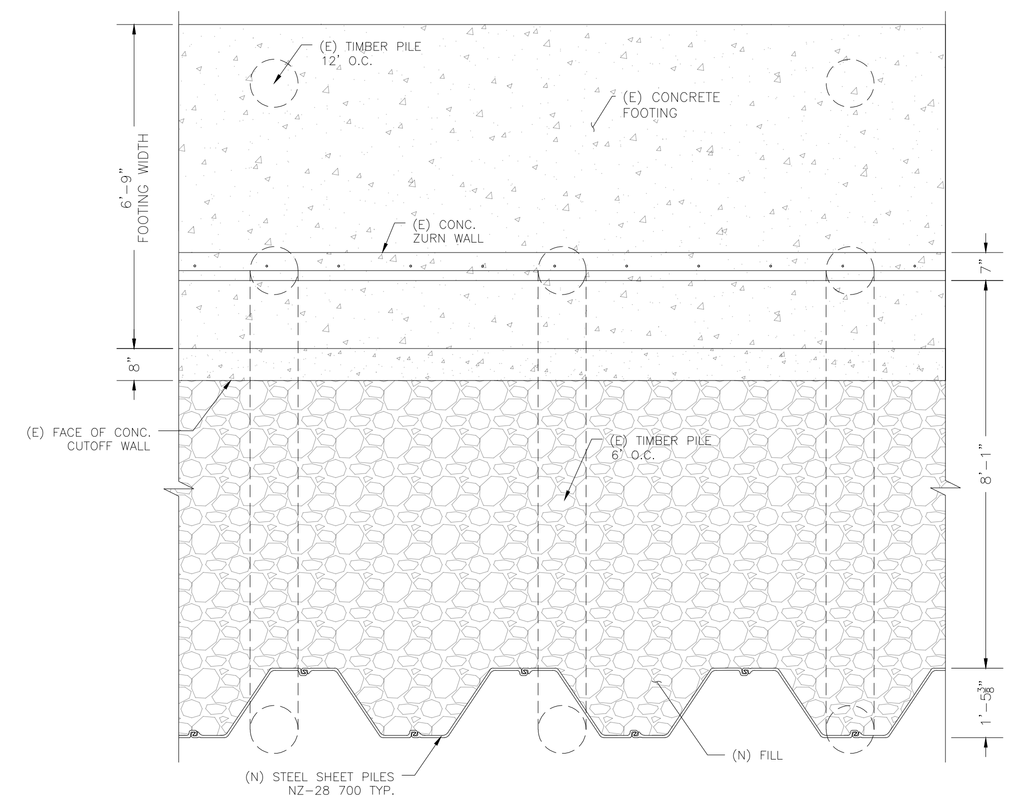


E
-
SECTION: OFFSET SHEET PILE OPTION - ZURN WALL

SCALE: 1/4" = 1'-0"

NOTES: FILL QUANTITY = 99.0 CF/LF



4
-
PLAN: OFFSET SHEET PILE OPTION - ZURN WALL

SCALE: 1/4" = 1'-0"

NOTES: FILL QUANTITY = 99.0 CF/LF

Figure 20

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4.2.2.3 New Cantilever Soldier Piles with New Panels (Scheme L)

This scheme is essentially the same as the cantilever soldier pile option discussed for the Boise walls.

4.2.2.4 Removal and Replacement of Face Concrete (Scheme M)

This is the method proposed in Phase C of the previous work. Approximately four inches of concrete (along with any loose material below that) would be removed and replaced with new pneumatically applied (shotcrete) low-permeability marine concrete proportioned following ACI recommendations for limiting susceptibility to ASR and to marine attack. Dowels into the existing concrete would be provided to protect against delamination of the new concrete from the existing if the expansion due to ASR continued. (This is a modification to the previously proposed detail.)

Pros:

- Creates a barrier layer limiting seawater penetration to the existing concrete slowing the alkali silica reaction and reinforcing corrosion.

Cons:

- May require shoring of the wall until new concrete has cured
- Extension of design life unknown
- Does not address seismic vulnerability

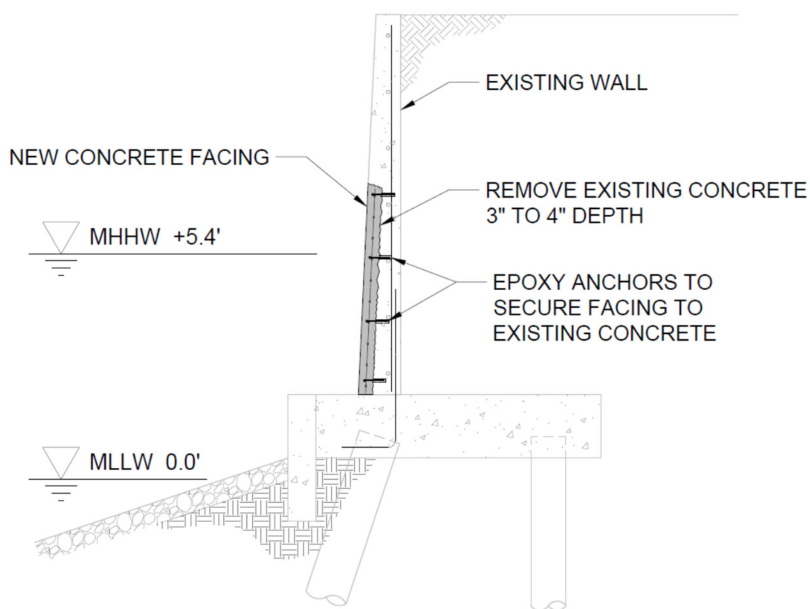

Figure 21 - Remove and replace surface concrete

4.2.2.5 New Concrete Facing (Scheme N)

This is similar to Scheme I, but the concrete removal would be limited to loose material and the new facing would extend further out than the existing concrete. Because it would be a thicker layer, the new concrete could be formed and placed conventionally.

Pros:

- Creates a barrier layer limiting seawater penetration to the existing concrete slowing the alkali silica reaction and reinforcing corrosion.

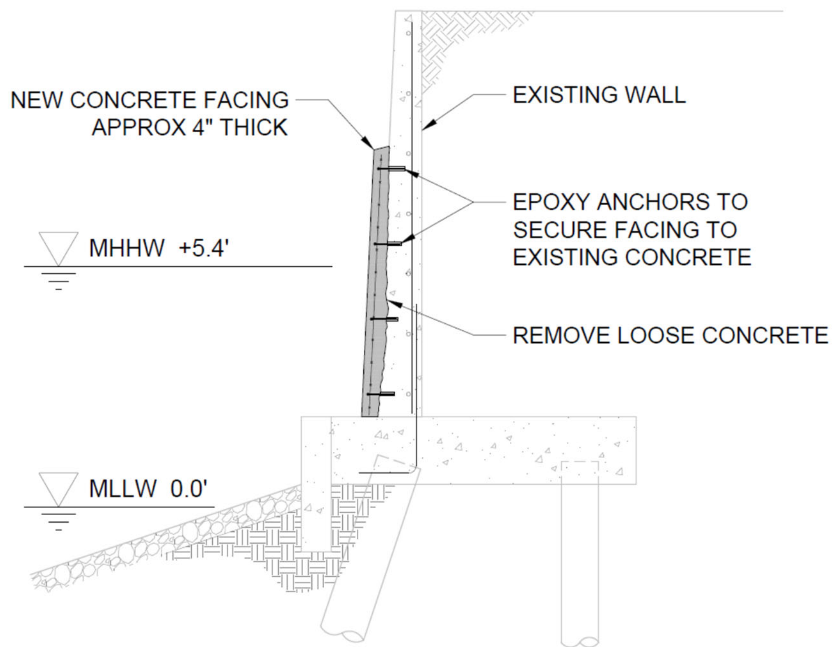

Figure 22 - New concrete facing

Cons:

- May require shoring of the wall until new concrete has cured, but less likely to be necessary than for Scheme (I).
- Extension of design life unknown
- Does not address seismic vulnerability

4.2.2.6 Concrete Buttresses (Scheme O)

In this scheme concrete buttresses would be added in front of the front face of the wall and doweled into it. The buttresses would utilize the existing reinforcing at the back face of the wall, but make the effective section deeper, increasing the strength of the wall. In addition, the buttresses would become the compressive element of the section, reducing the importance of the degrading front face of the existing concrete. The existing wall would span horizontally between buttresses and could tolerate significantly more concrete loss of the front face before requiring repair. Preliminary calculations suggest that the buttresses would need to be spaced about 6 feet on center. The concrete for the buttresses would be a similar mix design as for Scheme J. Alternatively, steel buttresses could be designed that would bolt to the face of the wall and the foundation.

Pros:

- Simple construction
- Unlikely to require shoring
- Repairs to the face concrete between buttresses could be delayed since the demand on the existing concrete would be reduced

Cons:

- Does not protect existing concrete between buttresses, so deterioration of the wall face will continue unabated.
- Deterioration could undermine the interface between the buttress and the existing wall, weakening the repair and shortening its effective life
- Continuing degradation of the wall face will be unattractive and could be concerning to residents
- Does not address seismic vulnerability

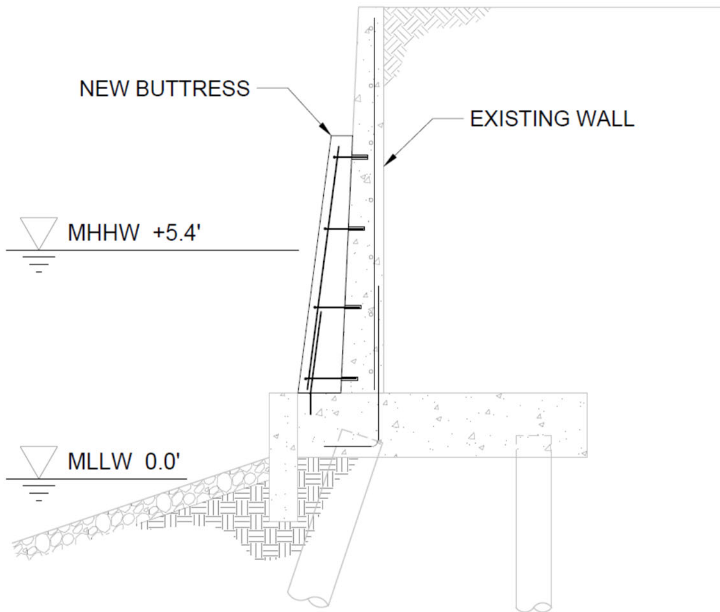

Figure 23 - Buttresses

4.2.2.7 Riprap Stabilization (Scheme P)

This scheme is the same as for the Boise walls.

4.2.2.8 Epoxy Filling of Base Joints (Scheme Q)

Unless a new facing is added to the wall that will protect the base joint (i.e. Scheme I or J), any area where the joint at the base of the Zurn wall is open or significantly deteriorated should have the joint filled by epoxy injection using standard methods. This will slow the corrosion of the reinforcing at the base of the wall.

4.2.2.9 Replacement In-Kind: New Cantilever Concrete Shear Wall (Scheme X)

In this option, the new wall would be similar to the existing Zurn walls—as close as possible to replacement in-kind. In order to avoid the need for shoring, the new walls would be built in front of the existing walls with the old walls left in place. After the walls were completed the gap between the new and old walls would be backfilled. Because of the length of footing required behind the wall the new wall would likely have to be built at least 8 feet in front of the existing wall and part of the sloped channel bottom would have to be excavated to provide a level foundation. In order to protect the existing wall during construction, excavation protection would need to be installed or the new wall would have to be moved even further out into the channel.

Pros:

- Permanent solution that would completely replace the existing seawall with new construction more resistant to deterioration
- Could be designed for liquefaction making the seawall more earthquake resistant than currently

Cons:

- Cast-in-place construction would require cofferdam, significantly increasing cost.
- Extends significantly outside current footprint of wall and footing.
- Required pile driving for foundation likely to be objectionable to residents
- Longer required construction time than other options

4.2.3 Foundation Repairs

Foundation repair work is the same for both types of seawall, so these repairs have been placed in a separate section. Because there are limited options for filling voids under the footings and, if necessary, repairing damaged piles, for this work only the recommended options are presented.

4.2.3.1 Grout Voids under Foundations

This is the same repair presented in previous reports. A piece of sheet pile is used as a stay-in-place form set just in front of the footing and grout is placed behind the form to fill underneath the footing. To limit damage and future undermining from continuing scour, the sheet pile should extend a minimum of six inches and preferably 12 inches or more into the existing bottom. In areas expected to be subject to scour, such as wall corners, the bottom adjacent to the sheet pile should be protected with stone sized to resist the maximum expected current velocities.

4.2.3.2 Jacket Damaged Piles Followed by Grouting Voids

Where the piles have lost material, prior to grouting underneath the footings, the piles should be repaired. The simplest and most cost-effective repair is to install two-piece fiberglass jackets around the pile extending below the damaged area by 12 to 18 inches and up to the underside of the footing and fill the jackets with non-shrink grout. Installing the jackets to the required length below the pile damage will likely require some excavation.

5.0 FEASIBILITY OF REPAIR SCHEMES

The different repair schemes have been compared for the two different wall types using a matrix for each wall type. The matrices compare the different schemes across seven evaluation criteria: construction cost, expected effective life improvement, constructability, disruption to residents, likelihood of regulatory difficulties, potential for capacity improvement, and appearance of the finished work. All of the criteria are rated from a scale of 1 to 5 with 1 being worst and 5 being best. In addition, each criterion has a weighting factor applied to it based on the overall importance of the criterion in assessing the overall ranking of options. The specific criteria are described further below, followed by the evaluation matrices.

5.1 EVALUTION CRITERIA

5.1.1 Construction Cost

This criterion provides a relative ranking of the expected order-of-magnitude construction costs for the different schemes considered. The following are the Rough Order of Magnitude (ROM) construction costs.

Mandalay Bay Repair Scheme Rough Order of Magnitude (ROM) Cost Estimate - Boise Wall (Assume 3.5 Miles)

Scheme	Cost ($USD)	Unit
Scheme A – Tieback Wall	$ 4,277	LF
Scheme B - Steel Sheet Pile Wall Installation	$ 7,382	LF
Scheme C - New Cantilevered Soldier Piles with New Panels	$ 7,037	LF

Scheme D - New Soldier Piles Tied to Existing Pilasters with New Panels	$ 4,050	LF
Scheme E - New Concrete Pilaster Jackets and Panel Facing	$ 4,000	LF
Scheme F - Fiberglass Pilaster Jackets and Panel Facings with Epoxy Grout	$ 3,850	LF
Scheme G - Riprap Stabilization	$ 10,094	LF
Scheme H - Epoxy Crack Injection of Panel Faces	$ 440	SF
Scheme I - Stainless Steel Reinforcing Straps	$ 6,800	EA

* = Designed for Seismic

**=No Seismic Benefit

Mandalay Bay Repair Scheme Rough Order of Magnitude (ROM) Cost Estimate - Zurn Wall (Assume 3.5 Miles)

Scheme	Cost ($USD)	Unit
Scheme J – Tieback Wall	$ 4,155	LF
Scheme K - Steel Sheet Pile Wall Installation	$ 7,382	LF
Scheme L - New Cantilevered Soldier Piles with New Panels	$ 7,037	LF
Scheme M - Removal and Replacement of Face Concrete	$ 2,900	LF
Scheme N - New Concrete Facing	$ 2,600	LF
Scheme O - Concrete Buttress	$ 3,150	SF
Scheme P - Riprap Stabilization	$ 10,094	LF

* = Designed for Seismic

**=No Seismic Benefit

5.1.2 Maintenance Cost

This criterion ranks the estimated maintenance costs over the life of the wall. For all the options the maintenance costs are expected to be low relative to the construction costs.

5.1.3 Design Life

This criterion provides a relative rating of the extent to which each of the schemes is expected to extend the effective life of the wall. Schemes that entirely replace the existing seawall received the highest rating since the design life of those systems can be controlled to the greatest extent and do not rely on any parts of the existing walls. Other schemes have been rated primarily based on the extent to which they protect the wall from the existing degradation mechanisms.

In general, the design life of the replacement schemes can be chosen (Schemes D, F, L and N). For instance, for one project Tetra Tech designed concrete block walls in salt water to have a predicted design life of 100 years using special concrete mixes and high covers. The replacement steel sheet-pile sea wall Tetra Tech designed for the City of Long Beach has a predicted design life of 75 years obtained using corrosion allowances (i.e. extra steel thickness) in the design of the sheet pile. Typically, a predicted design life of 50 to 75 years is easily obtainable for new construction at a reasonable cost. Riprap stabilization (Schemes H and N) is expected to be an essentially permanent solution. The cost estimates for the replacement schemes are based on a design life of 75 years.

Predicting design life for the repair options is more difficult. New elements—for instance the concrete used for pilaster jackets or concrete facing—can be designed much like the new options to reach a desired predicted design life. However, all the repair options rely on parts of the existing walls, as well. These areas are in better shape and less prone to the areas being replaced but will still continue to degrade and could still end up controlling the overall time-to-failure. In addition, the interfaces between new and existing materials can provide a path for ingress of seawater allowing degradation to continue in those areas. This can be seen with some of the existing pilaster jacket repairs where the new concrete is in very good condition, but the existing concrete has spalled leaving gaps. In general, repair options that cover more of the exposed face of the wall, especially in the tidal zone will fare better and last longer. In all cases though, the back faces of the wall will remain exposed to salt water and, although degradation in those areas appears to be progressing slowly based on the excavations described by TranSystems, it is still occurring.

5.1.4 Constructability

This criterion rates the ease-of-construction of the various schemes. Schemes that require lower levels of mobilization/demobilization effort, that require smaller crews and that use common construction techniques received higher ratings.

5.1.5 Disruption

This criterion addresses the extent to which construction of an option is expected to cause disruption to the homeowners. Schemes that can be constructed nearly or entirely from the water side of the wall and that do not affect the existing decks received the highest rantings. Options that required removal or alteration of existing decks received the lowest ratings.

5.1.6 Regulatory

This criterion considers how difficult obtaining regulatory approvals for the work is likely to be. The most likely areas of difficulty are expected to be wetlands permitting and Coastal Commission approval if either of those processes are triggered. Therefore, in general, repair schemes that are entirely within the footprint of the existing wall and footing are assumed to create little or no regulatory difficulty, and that the score for a scheme goes down the more it encroaches onto existing soft bottom.

5.1.7 Wall Capacity

This criterion rates the different schemes on the extent to which they can be designed to increase the capacity of the seawall. The highest ranked schemes could be designed to resist current code levels of seismic loading and liquefaction of the retained fill, which were found to be beyond the capacity of the existing construction. Many of the schemes can be used to strengthen particular portions of the wall, but not increase the overall capacity in significant ways because of the areas (i.e. the foundation) that are not addressed. These options have all been given a rating of 1. Options that provide no strengthening at all, such as epoxy injection, have been given ratings of 0.

As noted above, seismic design of new structural systems (e.g. cantilever sheet pile) would be done to the current Building Code, for which the target performance level is collapse prevention under the maximum credible earthquake (MCE). Starting in 2016, the California Building Code (CBC) has required that liquefaction potential be assessed for the peak ground acceleration from the MCE event. For this site, assuming Site Class D as established by the Geotechnical Report included in the TranSystems Phase B report, the site-modified PGA is approximately 0.84g.

Code-level structural design of the seawalls uses accelerations that are two-thirds of the MCE accelerations based on experience showing that code-level designs can survive ground accelerations approximately 50 percent larger than the design accelerations without collapsing. Therefore, the design PGA for structural design would be approximately 0.56g. Both of these accelerations were determined using the online tool provided by the Applied Technology Council (ATC) at hazards.atcouncil.org.

It should be noted that the seismic accelerations obtained from ASCE 7 are not designated by source. Determining a source fault and magnitude would require a site-specific seismic analysis that is beyond the scope of this report.

5.1.8 Appearance

This criterion judges the appearance of the finished repairs. This is somewhat subjective but attempts to separate out options that residents are more likely to judge as unattractive. Options that leave areas of degrading concrete exposed to view have been given lower ratings.

5.2 EVALUATION MATRICES

For the purpose of evaluating the schemes, options that can be designed to increase the seismic resistance of the walls beyond the original capacity of the existing walls have been split and ranked separately from the options that are not capable of increased resistance (e.g. they rely on the existing foundations or tie backs.)

Separate matrices have been provided for the Boise wall and the Zurn wall options. Scheme P (the new cast-in-place Zurn-type walls) was judged to be non-competitive and was not included in the evaluation matrices.

5.2.1 Boise Walls

Table 4 - Evaluation Matrix for Boise Wall Repair Schemes

		Construction Cost	Maintenance Cost	Effective Life	Constructability	Disruption	Regulatory Issues	Wall Capacity	Appearance	**Total**	**Ranking**
Weighting Factor		**2**	**1**	**3**	**1**	**1**	**1**	**2**	**1**	**-**	**-**
Options Designed for Seismic Resistance											
Recommended Options											
Scheme A	Tieback Wall	4	3	3	3	4	4	3	4	**41**	**1**
Scheme B	New cantilever sheet pile wall	1	4	5	3	1	2	5	4	**41**	**1**
Other Options Designed for Seismic Resistance											
Scheme C	New cantilever soldier piles and panels	3	3	4	3	2	2	4	3	**39**	**3**
Options with No Increase in Seismic Resistance											
Scheme D	New soldier piles tied to existing pilasters	3	3	3	3	3	3	1	3	**32**	**5**
Scheme E	Concrete pilaster jackets and panel facing	3	4	4	3	3	4	1	5	**39**	**1**
Scheme F	FRP pilaster jackets and panel facing	2	3	4	4	3	4	1	4	**36**	**3**
Scheme G	Riprap stabilization	2	5	5	2	1	1	4	2	**38**	**2**
Scheme H [1]	Epoxy crack injection of panel faces	5	1	2	5	5	5	0	1	**33**	**4**
Scheme I [2]	Stainless steel reinforcing straps	5	1	1	5	5	5	1	1	**32**	**5**

[1] Epoxy injection is only applicable to the panels and needs to be combined with a method of pilaster repair such as Scheme A or B.

[2] Scheme G does not protect areas outside of the strap footprints and should only be applied as a short-term repair or at areas that are suffering from reinforcing corrosion but are otherwise in good condition with minimal surface degradation of the concrete.

5.2.2 Zurn Walls

Table 5 - Evaluation Matrix for Zurn Wall Repair Schemes

		Construction Cost	Maintenance Cost	Effective Life	Constructability	Disruption	Regulatory Issues	Wall Capacity	Appearance	**Total**	**Ranking**
Weighting Factor		**2**	**1**	**3**	**1**	**1**	**1**	**2**	**1**	**-**	**-**
Options Designed for Seismic Resistance											
Recommended Options											
Scheme J	Tieback wall	4	3	3	4	4	4	3	4	**42**	**1**
Scheme K	New cantilever sheet pile wall	1	4	5	3	1	2	5	4	**41**	**2**
Other Options Designed for Seismic Resistance											
Scheme L	New cantilever soldier piles and panels	3	3	4	3	2	2	4	3	**39**	**3**
Options with No Increase in Seismic Resistance											
Scheme M	Remove and replace face concrete	3	4	4	3	3	4	1	5	**39**	**2**
Scheme N	New concrete facing	4	4	4	4	4	4	1	4	**42**	**1**
Scheme O	Concrete buttresses	5	2	3	5	5	4	1	2	**39**	**2**
Scheme P	Riprap stabilization	2	5	5	2	1	1	4	2	**38**	**3**
Scheme Q [1]	Epoxy filling of base joints	N/A									

[1] Epoxy filling of base joints is a maintenance activity that should be performed at areas that are otherwise in good condition but have poor quality construction joints at the base of the wall and are not scheduled for repair in the near future.

6.0 CONCLUSION

For the Boise walls, the evaluation matrix suggests that the best non-strengthening repair option is the installation of pilaster jackets and concrete facing (Scheme A), and the best option for enhancing the ability of the seawalls to resist current code-level seismic loading is either a new cantilever sheet pile wall (Scheme F) or a tieback wall (Scheme P). However, there are some geotechnical and capacity issues to be resolved in order to verify that the tieback wall is a feasible option. Overall, the cantilever sheet pile wall receives a higher score than the jacket repair, but the City should consider whether the importance of enhancing the seismic resistance of the seawall is worth the additional cost and disruption entailed by the cantilever wall construction.

For the Zurn walls, the matrix gives the new concrete facing (Scheme J) as the preferred non-strengthening repair option and, again, the cantilever sheet pile wall (Scheme M) and the tieback walls (Scheme Q) as the best potentially earthquake-resistant options. In the case of the Zurn walls, the concrete facing slightly outscores the cantilever sheet pile wall. This is primarily because the new concrete for the Zurn walls is cheaper and easier to install than the corresponding option on the Boise walls (Scheme A) due to the lack of pilasters. As with the Boise walls the City should consider the cost-benefit tradeoff of the earthquake resistance that can be designed into cantilever wall before making a final decision.

Given the closeness of the scores for the different options we would appreciate feedback from the City on the weighting factors used for the different criteria and the assigned ratings.

Finally, it should be noted that, if earthquake shaking results in liquefaction and strength loss in the weak soils prevailing in the upper 22 to 27 feet of the site, structures supported on shallow foundations will experience settlement regardless of the support capacity of the seawalls. Evaluation of such foundation settlement is beyond the scope of any seawall-related evaluations.

APPENDIX A – ENGINEER'S COST ESTIMATE – TIEBACK WALL

CITY OF OXNARD

PUBLIC WORKS - ENGINEERING DIVISION
CONSTRUCTION COST ESTIMATE FOR
MANDALAY BAY SEAWALL REPAIRS

Project Description:
Repair Zurn type seawalls and Boise panel type seawalls
Repair Method:* Tieback Wall *(7.6 Miles)

Date: February 12, 2020

Item No.	Description	Unit of Measure	Estimated QTTY	Unit Cost*	Total Cost
1	High Priority Repairs (Years 1-6)	LF	7,750	$4,277.00	$33,146,750
2	Medium Priority Repairs (Years 7-12)	LF	12,550	$4,277.00	$53,676,350
3	Low Piority Repairs (Years 13-18)	LF	9,400	$4,277.00	$40,203,800
4	Non-Critical Repairs (Years 19-25)	LF	9,300	$4,277.00	$39,776,100
5	Repairs Beyond 25 Years	LF	0	$4,277.00	$0
				Sub Total	**$166,803,000**
CONSTRUCTION COST					

**** Unit Cost Includes:***
Design, Inspection, Construction Management, 15%
Construction Contingency, 25%
Permit Fees & Misc. Soft Costs 5%

Sub Total (Years 1-25)	$166,803,000
Sub Total (Beyond 25 Years)	$0
Project Total	**$166,803,000**

APPENDIX B – ENGINEER'S COST ESTIMATE – CANTILEVERED SHEET PILE

CITY OF OXNARD

PUBLIC WORKS - ENGINEERING DIVISION
CONSTRUCTION COST ESTIMATE FOR
MANDALAY BAY SEAWALL REPAIRS

Project Description:

Repair Zurn type seawalls and Boise panel type seawalls
Repair Method:* Cantilevered Sheet Pile Wall *(7.6 Miles)

Date: February 12, 2020

Item No.	Description	Unit of Measure	Estimated QTTY	Unit Cost*	Total Cost
1	High Priority Repairs (Years 1-6)	LF	7,800	$7,382.00	$57,579,600
2	Medium Priority Repairs (Years 7-12)	LF	12,550	$7,382.00	$92,644,100
3	Low Piority Repairs (Years 13-18)	LF	9,400	$7,382.00	$69,390,800
4	Non-Critical Repairs (Years 19-25)	LF	9,300	$7,382.00	$68,652,600
5	Repairs Beyond 25 Years	LF	0	$7,382.00	$0
				Sub Total	**$288,267,100**
	CONSTRUCTION COST				

**** Unit Cost Includes:***
Design, Inspection, Construction Management, 15%
Construction Contingency, 25%
Permit Fees & Misc. Soft Costs 5%

Sub Total (Years 1-25)	$288,267,100
Sub Total (Beyond 25 Years)	$0
Project Total	**$288,267,100**

APPENDIX C – GEOTECHNICAL EVALUATION OF SEAWALL REPLACEMENT / SUPPORT ALTERNATIVES

1.0 GEOTECHNICAL DATA BASIS

The geotechnical evaluations presented in this report were based primarily on data included in a preliminary geotechnical investigation report prepared for the project by Terra Costa Consulting Group in 2012. The report included data from 9 cone penetration tests (CPTs)to depths ranging from 75 to 100 feet and 4 test pits to depths ranging from 4 to 7 feet. Limited soil sampling was performed in the test pits and at three depths in one of the CPTs. Laboratory testing on the soil samples was limited to grain size analyses, moisture content and plasticity (Atterberg limits).

The results of the preliminary geotechnical investigation indicated a soil profile consisting of three general zones:

a. Shallow man-made fills of variable composition were encountered in the four test pits to the full depths explored. These soils ranged from silty and clayey sands to sandy clays. The sands exhibited highly variable relative density, ranging from very loose to dense, while the clays were typically very stiff to hard.

b. Underlying the fills and extending to approximate depths of 22 to 27 feet a zone of very weak soils was encountered. The soils in this zone included loose silty sands and sandy silts, as well as soft clays. The Terra Costa report concluded that the loose sandy soils in this zone would be susceptible to liquefaction under even moderate earthquake loading conditions, while the weak clayey soils would likely experience further strength loss under cyclic loading from earthquakes.

c. The soils below approximate depths of 22 to 27 feet are much more competent, consisting mostly of dense to very dense sands and stiff to very stiff clays.

The Terra Costa report included seismic design criteria based on the California Building Code in effect at the time of the report, including a design peak ground acceleration of 0.49g to 0.54g. The report included lateral earth pressures for static and seismic conditions. However, the seismic lateral earth pressures were based on the assumption of no liquefaction, which contradicted the conclusion in the same report that liquefaction was likely to occur even under moderate earthquake shaking, well below the design earthquake shaking level. Furthermore, the report stated that the potential for lateral spreading under earthquake loading conditions would likely result in much higher lateral earth pressures on the seawalls than those presented in figures 7 and 9 of the report. We concur with this conclusion.

2.0 LATERAL EARTH PRESSURES

The geotechnical evaluations presented in this report are applicable to any new construction that is to be designed in accordance with the 2019 California Building Code. These evaluations are not applicable to repairs that are considered to be "maintenance" and not subject to the new Code requirements.

Our preliminary evaluations indicate that the design of any new seawall reinforcement or replacement system will be governed by seismic lateral earth pressures. As acknowledged by Terra Costa in their July 11, 2012 report, the forces that would need to be resisted due to liquefaction and lateral spreading can be expected to be much higher than those presented in their report, which would have been appropriate for soils that would not be susceptible to strength loss and liquefaction under seismic ground motion. Furthermore, there have been two code changes since 2012 and seismic design criteria have increased.

In accordance with the 2019 California Building Code liquefaction and ground stability analyses need to be performed for the Maximum Considered Earthquake (MCE) and for sites with soils susceptible to liquefaction (Site Class F) ground motion characteristics, including Peak Ground Acceleration (PGA), typically would need to be determined based on site-specific site response analyses. For certain low-rise structures, the Code allows the use of Site Class D spectra, instead of Class F, because such spectra are conservatively high at low periods. The MCE peak ground acceleration for Class D soil profile at the site is greater than 0.8g versus about 0.5g used for the previous evaluations by Terra Costa. For the soil conditions prevailing at the site, with approximately 20 feet of weak soils the Class D-based peak ground acceleration is unrealistically high. However, a more realistic assessment of ground motion and seismic lateral earth pressures would need to be based on very detailed numerical analyses, which would consider the non-linear behavior of the weak soils under seismic loading. Such analyses are not in the scope of the current study and would require more detailed geotechnical data, partly from more extensive laboratory testing performed on soil samples obtained in borings.

In the absence of comprehensive site-specific numerical analyses, our geotechnical evaluations were based on results of similar studies performed for sites with very similar soil profiles. While these evaluations are adequate for the **comparative** assessment of the various seawall reinforcement or replacement concepts, they are not appropriate for final design. Once a design concept is selected, more detailed geotechnical investigations, with borings, more laboratory testing, and more comprehensive numerical analyses will need to be performed.

The lateral earth pressures to be resisted by the seawalls depend not only on the inertial forces and strength of the soil, but also on how much the wall can yield under pressure. The more rigid and unyielding the wall is, the higher will be the pressures to be resisted. Therefore, evaluation of seismic earth pressures needs to be based on iterative soil-structure interaction analyses. Based on such analyses performed for another site with very similar soil conditions the following seismic lateral loads per lineal foot of wall with corresponding average wall deflections were considered for our current evaluations:

Maximum Considered Earthquake: 18 kips/ft with 8 inches average deflection

Design Earthquake (2/3 MCE): 13 kips/ft with 3 inches average deflection

It should be noted that the total lateral loads for the Design Earthquake case are approximately 2 to 4 times greater than the lateral loads calculated based on the seismic lateral pressures recommended in the 2012 Terra Costa report, which neglected the potential effects of lateral spreading due to liquefaction and strength loss in the weak soils at the site. For comparison purposes, our evaluation of all seawall replacement or reinforcement alternatives were based on the same total lateral earth loads as summarized above. As noted previously, once an alternative is selected, the final design will need to be based on more comprehensive geotechnical investigations and numerical analyses.

3.0 SEAWALL REINFORCEMENT OR REPLACEMENT ALTERNATIVES

The geotechnical evaluations covered four seawall reinforcement or replacement alternatives, which have been used or considered for similar projects. These include additional tie-back anchors, continuous steel sheet pile wall, steel beam and panel wall, and rock buttress. The geotechnical assessment of these alternatives is presented below:

3.1 Tie-back Anchors

Tie-back anchors are commonly used to increase the lateral load capacity of existing seawalls. If the existing seawall panels are structurally sound the anchors can be drilled through and attached to the existing panels. Otherwise, structurally sound new concrete panels can be placed over the existing panels and the anchors attached to the new panels.

Tie-back anchors used for seawall reinforcement typically consist of threaded steel rods, placed in inclined drilled holes and pressure grouted in place. Typically, additional grout tubes are placed in the hole to allow post-grouting, in order to maximize the pull-out capacity. For such anchors to provide adequate capacity under seismic loading conditions, all of their frictional pull-out capacity must be obtained from dense sands and/or stiff clays, which would not be susceptible to liquefaction or strength loss. At the Mandalay Bay site such soils exist at depths greater than

22 to 27 feet below the existing ground surface. Tie-back anchors should be inclined approximately 20 degrees from horizontal with all of their bonded section below depths of 22 to 27 feet (24 feet average).

The actual pull-out capacity of drilled anchors depends to a large extent on the contractor's drilling and installation procedures. Therefore, such anchor installations are typically contracted on a design/build basis. For anchors with nominal diameters of about 4.5 inches, at seawall sites with predominantly sandy soils similar to those found below depths of 22 to 27 feet, we have confirmed allowable sustained pull-out load capacities on the order of 1 kip/foot and ultimate capacities for short term loading of about 1.5 kips/foot of embedment in the dense / stiff soils below depths of 22 to 27 feet. These estimated pull-out capacities can be updated based on data from more comprehensive geotechnical investigations. During construction, all anchors will need to be tested in accordance with Post-Tensioning Institute procedures.

This option has the main advantage that it can be completed without increasing the footprint of the existing wall and possibly be constructed without even removing the overhanging portions of decks present at some locations. The main disadvantage of this option is that the depth interval that would be supported is the depth interval of the existing seawall and not the full depth of the weak soils. If this option is selected, more detailed geotechnical analyses will need to be performed to confirm that the limited support depth interval would be adequate to limit ground deformation under seismic loading conditions. If not, while this is not very likely, additional ground improvement measures such as pressure grouting and deep soil cement mixing may be warranted.

3.2 Continuous Steel Sheet Pile Wall

This option, which has successfully been used for seawall replacement projects at several other sites has the main advantage that it can support the full depth of weak soils. Additionally, sheet piles can be installed by hydraulic press methods which involve minimal noise and vibration as well as minimal impact on adjacent structures. On recent seawall projects sheet piles have been installed by the GIKEN Silent Piler, a hydraulic press method which involves minimal noise and vibration. The main drawback of this method is that at Mandalay Bay the sheet piles will need to be installed beyond the reach of the battered piles supporting the existing walls and the space between the new and old wall will need to be backfilled.

At a recently completed sheet pile wall installation project at a site with very similar soil conditions NZ28 sheet piles were used with tip elevations of -38 feet (MLLW). The calculated average deflections under seismic loads, neglecting any support from the existing walls were on the order of 3 inches for the Design Earthquake (2/3 MCE) and 8 inches for the MCE. For preliminary evaluations of seawall replacement alternatives these sheet pile design parameters may be used. However, final design will need to be based on more detailed geotechnical investigations.

3.3 Steel Beam and Panel

This option is similar to soldier beam and lagging shoring systems used for excavation support. It consists of steel H-piles or wide flange beams to provide the resistance to lateral loads and shorter concrete panels or steel sheet piles to support the soil between the beams. The main advantage of this option is that the beams, which will provide the resistance to lateral loads, can be installed in the spaces between the existing battered piles and the panels can be installed inside the outer (waterside) flange of the beams. The steel beams will need to clear the pile cap (footing) and any cutoff wall of the existing seawalls. Thus, the face of the new wall will be approximately 3 to 4 feet beyond the face of the existing seawalls.

Steel beams are typically driven into the dense soils using vibratory or impact hammers. The vibrations in the adjacent ground induced by such pile driving could cause differential settlement and possibly structural damage at this site, due to the presence of weak soils susceptible to liquefaction. Therefore, this option is not recommended for this project unless measures are taken to minimize vibrations. One possible measure, that has been used on land, is to use in-situ deep soil-cement mixing in 3-foot diameter columns and lower/push the beams into the mix before the soil-cement hardens. The mix can be designed to have strength and stiffness comparable to that of the dense sands once the cement cures and hardens. However, these measures are expected to add approximately $8,000 to $10,000 per beam, probably making this option prohibitively expensive.

We have evaluated the response to lateral loads for several beam options using the computer program LPILE (2028 Version). We considered a range of lateral loads, up to 100 kips per pile. It should be noted that the lateral load per pile is equal to the lateral load per lineal foot times the pile spacing in feet. For example, for a lateral load of 13 kips/ft and spacing of 6 feet the load per pile would be 78 kips. For all cases analyzed adequate fixity was achieved with a tip elevation of -45 feet (MLLW), corresponding to a total beam length of 55 feet. The result of the analyses for 18-inch H-piles and 24- and 30-inch wide-flange beams are graphically presented in the figures that follow. The calculated lateral deflections and bending moments are for free-head conditions. If even partial pile top fixity could be justified, the deflections would be much lower. In the enclosed figures the zero depth corresponds to elevation zero (MLLW), where the resultant of the lateral force was applied.

3.4 Rock Buttress

Rock buttresses have been successfully used to stabilize slopes with marginal stability. At this site a rock buttress would need to resist lateral spreading and extend well below the bottom of the weak soils found at elevations ranging from -12 to -17 feet (average -14 feet MLLW). We performed a preliminary check of the stability of such a buttress with a surface slope of 3:1 (H:V) and a base elevation of -16 feet (2 feet into the dense sands). Despite the massive rock section, our preliminary check indicated that this buttress section would provide less than one half of the resistance needed to prevent lateral spreading under seismic conditions, if any resistance offered by the existing seawalls is neglected. This option was dropped from further consideration.

4.0 SUMMARY

Based on the results of the geotechnical evaluations presented in this report, it is our opinion that the first two seawall reinforcement or replacement options are worth pursuing further. The tie-back anchor alternative offers the two main advantages that it would not extend beyond the footprint of the existing wall and that it can be installed with minimal disruption to the existing improvements. However, the support depth will be limited to the depth range of the existing walls and the adequacy this limited support depth will need to be evaluated by much more detailed soil-structure interaction analyses. The continuous sheet pile option has the main advantage that it would provide continuous support over the full depth interval. However, the trade-off is that it would need to extend well beyond the face of the existing wall, in order to clear the battered piles supporting the existing walls.

Both options will require further geotechnical investigations and numerical analyses before selecting one of the two options and finalizing the design.

Finally, it should be noted that, if earthquake shaking results in liquefaction and strength loss in the weak soils prevailing in the upper 22 to 27 feet of the site, structures supported on shallow foundations will experience settlement regardless of the support capacity of the seawalls. Evaluation of such foundation settlement is beyond the scope of any seawall-related evaluations.