Santa Cruz Wharf

Engineering Report

Prepared for:

Prepared by:

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Engineering Evaluation Summary

The findings of the Santa Cruz Wharf engineering evaluation are presented in this report. The report contains 11 sections that address different aspects of the Wharf structure and function, corresponding to the tasks identified in the Request for Proposals prepared by the City of Santa Cruz for this project. The engineering report was prepared in conjunction with the Master Plan effort by ROMA Design Group who prepared all Master Plan figures presented herein. The engineering study was performed by Moffatt & Nichol with assistance from Mesiti-Miller Engineers (pavement and building engineering) and Axiom Engineers (fire protection engineering).

The engineering evaluation involved a visual and underwater survey of the piles and sub-structure to determine their viability and the need for replacement and repair. What was found is that the Wharf is generally in good and serviceable condition, primarily due to the fact that it has been well-maintained over the years by the Wharf staff. There is a need for some pile replacement, particular under some buildings in locations that have been difficult to access. With the continuation of ongoing maintenance and replacement on an as-needed basis of the structural elements, the life of the Wharf will be extended well into the future.

There is need for general improvement to the pavement and substrate of the Wharf. The asphalt pavement of the driveway and parking areas is severely cracked over the majority of the traffic areas. Vehicular loading, particularly from heavy vehicles and large garbage trucks that service the Wharf, and differential displacement of the decking causes continual cracking of the asphalt paving and damage to the substrate. This is one of the major maintenance costs associated with the operations of the Wharf. Furthermore, no provisions are currently made for the handling and treatment of storm water runoff from vehicular movement and parking areas, which currently flow directly into the bay.

This report documents the piling survey and the evaluation of the general structural condition and identifies the engineering improvements required to further the longevity of the Wharf. In addition, the report documents methods for increasing the resiliency of the structure to reduce potential damage and to enhance public safety in extreme weather conditions related to climate change and rising sea levels as well as seismic events. It further addresses issues related to the weight bearing capacity of the pavement and substrate and describes the improvements that are needed.

Improvement plans, specifications and cost estimates will be produced after review of this report and recommendations by the City of Santa Cruz, as called for in the scope of work. It is intended that this report and the evaluations performed will serve as the basis for the improvement plans with added definition and detail of the improvements. Preliminary budget level numbers for repairs to the existing wharf based upon the recommendations presented in this report are presented below.

Budget Costs:

Piles	$1,000,000 to 1,500,000
Deck structures	$750,000 to $1,100,000
Pavement Replacement (Alt 2-Plywood)	$7,200,000 to $8,700,000
" " (Alt 2.1-Grid Reinf)	$3,600,000 to $4,300,000
Misc. (Sewer, Fire Protection, Landings)	$200,000 to $300,000

The recommendations from each section of the report are summarized as follows:

1. Piles

1.1 Replace all piles with greater than 40% section loss and cap spans larger than 10 ft.

1.2 Replace new piles at all the A-frame locations

1.3 Install T-connections at all dowel-only connections

1.4 Replace T-connection bolts with greater than 30% section loss

2. Structure Evaluation

2.1 Add piles in Master Plan features to increase Wharf lateral stability

2.2 Install side plate connections at unsupported cap splice

2.3 Replace deteriorated stringers, decking, and caps

2.4 Install bolts at stringers laps to provide longitudinal continuity

2.5 Review City operations plan for weight of specific vehicles to access the Wharf in emergencies

2.6 Provide markings to restrict trucks from parking spaces and lower capacity areas

2.7 Test existing wharf timbers to obtain actual allowable stress values if higher load capacity is sought

2.8 Retrofit turnaround areas to increase the load capacity

2.9 Design the new East Promenade for a minimum capacity of 36,000 lbs axle load

2.10 Perform design level seismic analysis for additions to the Wharf

3. Roadway

3.1 Replace asphalt pavement throughout the road and parking areas utilizing plywood under layer (alt 2) or grid reinforcement (alt 2.1)

3.2 Install a test area of improved section to evaluate two alternatives

3.3 Slope new vehicles pavement to drain inlets that provide treatment for runoff water quality

3.4 Limit truck traffic to the greatest extent possible to minimize damage

4. Walkways

4.1 Replace deteriorated timber beneath side walk as part of the sidewalk replacement

4.2 Provide the walkway structure as set out in the companion Master Plan

4.3 East Promenade - Hardwood decking on timber substructure

4.4 Sidewalk in front of buildings and south Commons - Stamped concrete

4.5 West Promenade - Fiberglass Grate on Steel framing

5. Gravity Sewer

5.1 Remove all abandoned piping beneath the Wharf

5.2 Replace all mild steel hangers with hot-dipped galvanized or stainless steel

5.3 Require pressure tests performed on all new laterals installed on the Wharf

5.4 Perform monthly inspections and after large wave events for leaks of the sewage system

6. Fire Suppression

6.1 Consult with SC Fire Dept to identify existing areas that should have extended sprinkler coverage such as the Public Access Dock and Boat Rental

6.2 Provide sprinkler coverage to Wharf substructure expansions per current NFPA 307 in consultation with the SC Fire Dept.

6.3 Provide markers to assist the fire department to locate the access portals at night in the dark

6.4 Limit boat anchorage to outside 200 feet of the west side of the Wharf

7. Wharf Structure Supporting Buildings

7.1 Provide additional stringers beneath bearing wall loads

7.2 Locate column point loads to be directly over pile and limit point load to 40 kips

7.3 Install additional piles to support column point loads that are in excess of 40 kips

8. Existing Landings

8.1 Replace timber decking with fiberglass decking where practical

8.2 Use galvanized or stainless steel bolts for all connections

9. New Landings

9.1 Install a 12x12 timber waler for energy absorption

9.2 Install Fender piles connected back to the Wharf for added stiffness

10. Environmental Loads

10.1 Widen the Wharf with vertical timber piles to increase its resistance to lateral earthquake forces

10.2 Evacuate the Wharf during periods of predicted extreme waves, as occurred in 1985 and 1998

10.3 Utilize open grating for deck surface below the main wharf deck level

11. Permitting

11.1 It is recommended to follow up with agencies based upon items discussed at scoping meeting of August 22, 2014 to further identify agency concerns and confirm level of analysis they may require

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1. PILING SURVEY

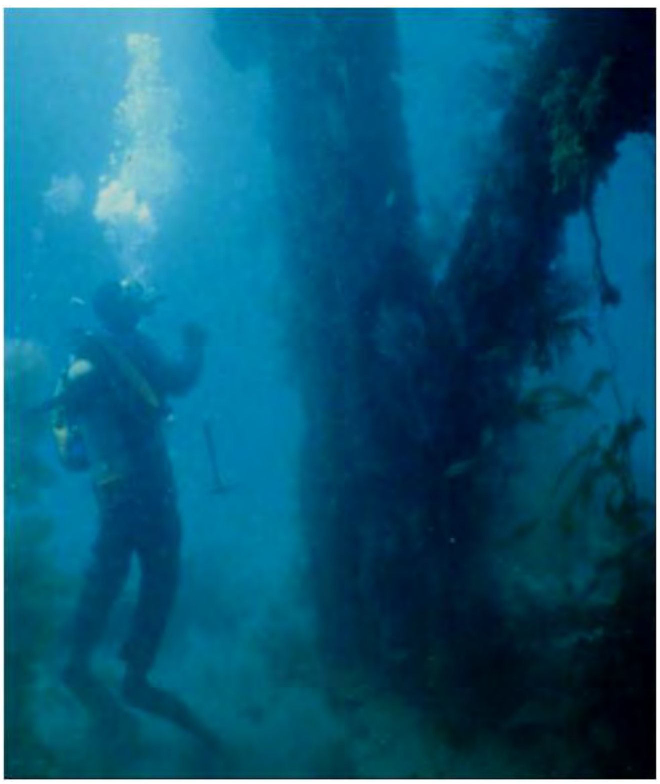
1.1 Summary

An inspection was performed of all 4,450 (approx.) piles of the 100 year old Santa Cruz Wharf. The piles are the most critical element of the structure as they transmit all loads to the supporting seafloor soils. The inspectors were engineer-divers who observed every pile from the ocean floor to the pile top and recorded the results of their observations and testing, which included coring samples of the pile interiors.

The piles are in good condition, overall. Less than 5% of the 4,450 piles need replacement. Notable exceptions are underneath buildings where replacement is difficult with the building structure in place. A major factor contributing to the longevity of the piles has been the practice of using Douglas fir piles treated with preservative (different treatment methods have been used on the existing piles depending on the time period they were installed. Observed damage to the piles is caused by storm waves, floating logs and marine borers. There is some presence of marine borers (Teredo) in the piles as observed in damaged piles during the inspection. The continued replacement of damaged piles by the Wharf staff will allow the continued functioning of the Wharf well into the future.

Recommendations include

- Continuing full time Maintenance crew to replace piles as damaged,
- Use of treated piles for replacement
- Addition of piles for lateral stability and where required for additions to the Wharf



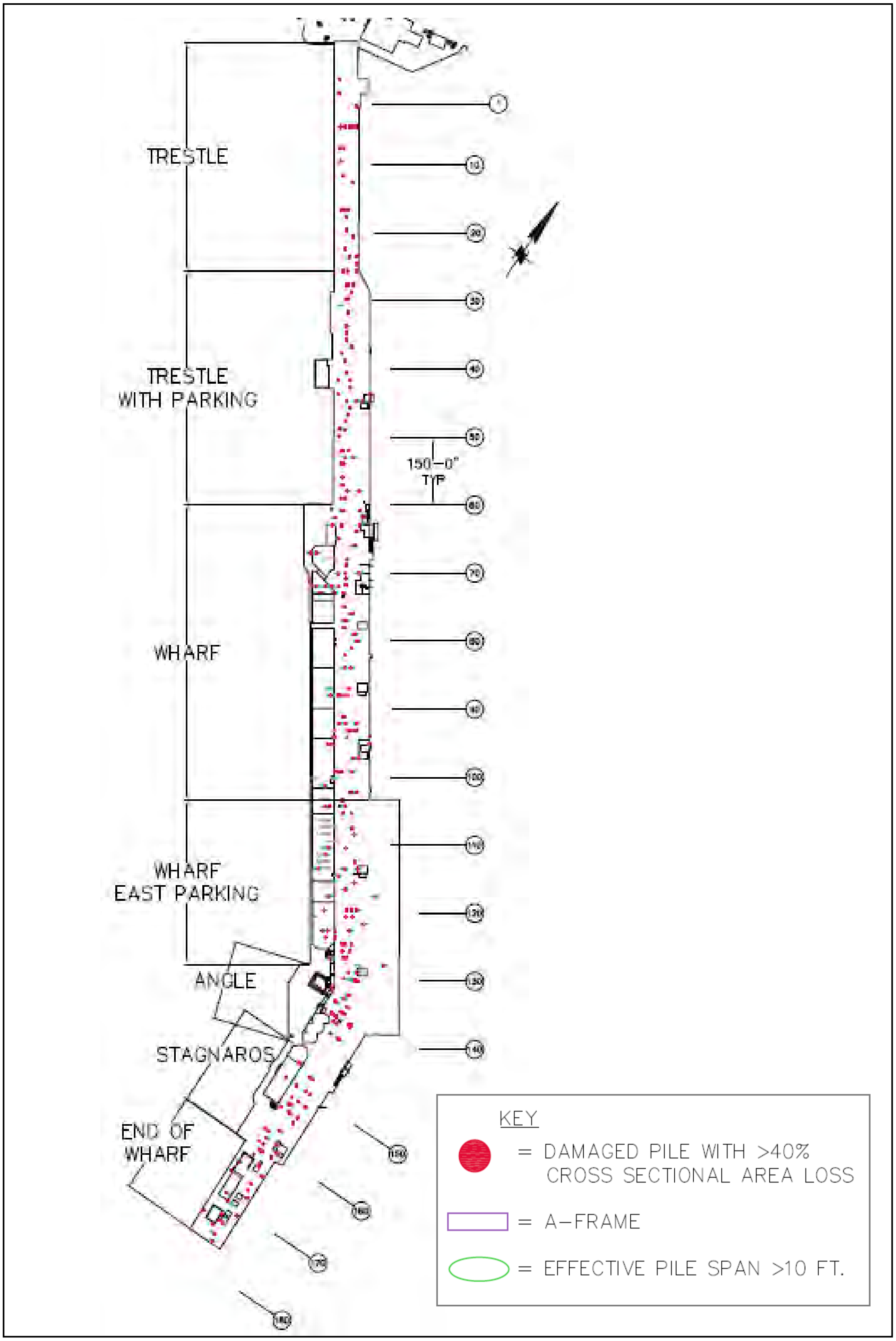


Figure 1-1: Piling Survey Summary Plan

1.2 Introduction

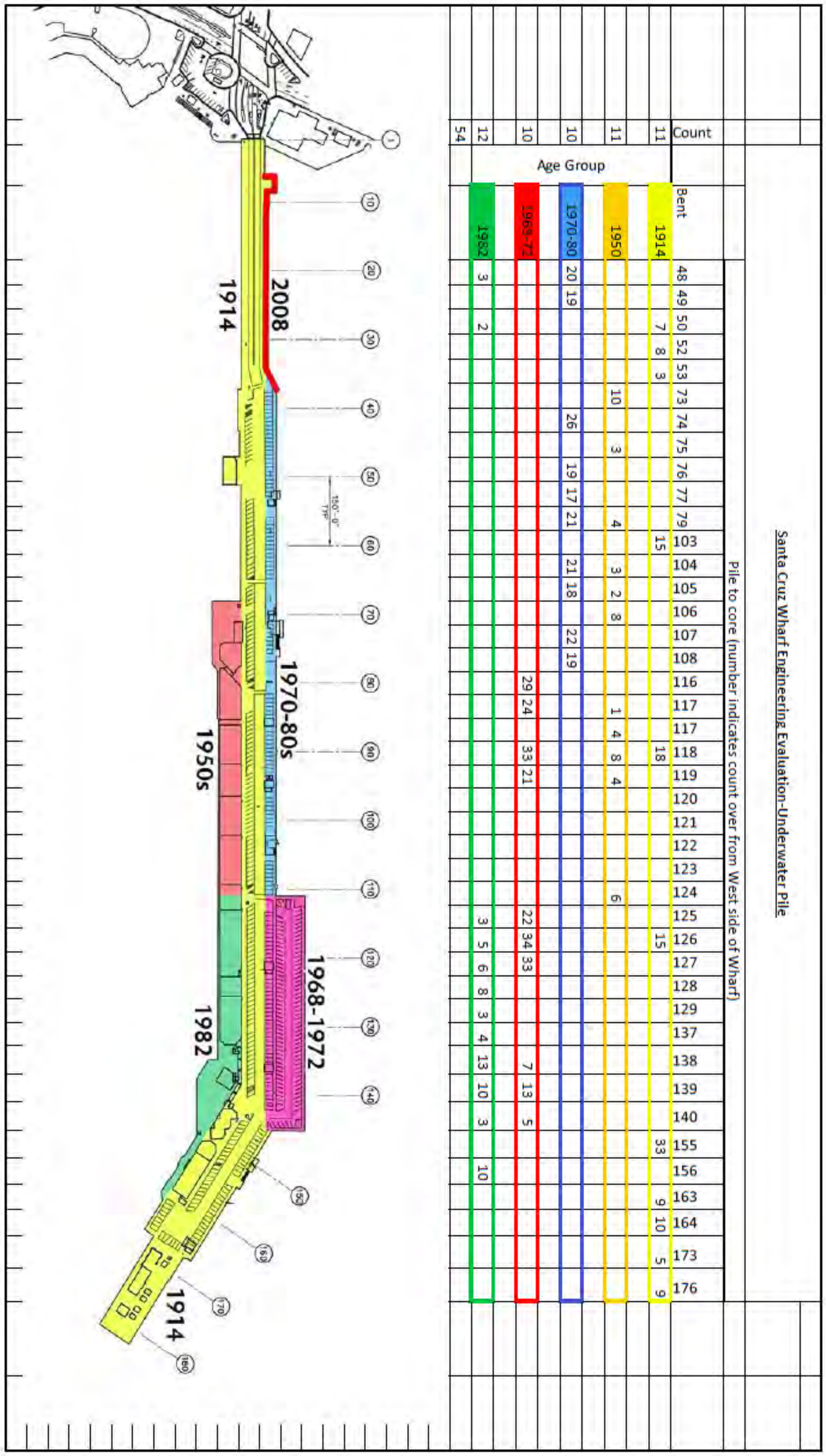
Santa Cruz Wharf was constructed in 1914, and is 2750 ft. long, its original length. It is the longest timber pier on the coast of the United States and one of the 5 longest of such piers in the World. Having been in continuous service for 100 years also gives it the distinction of one of the longest histories for open coast piers in continuous operation in existence.

Additions have been made at various intervals and repairs have been made continuously resulting in pile ages from 2 – 99 years. The Wharf structure is timber construction, with 183 bents (rows) of vertical timber piles. There are a few batter (slanted) piles located throughout the Wharf in an effort to limit lateral deflections (sway) of the Wharf.

1.2.1 Scope

The purpose of this investigation is to evaluate the condition of the timber piles supporting Santa Cruz Wharf. This information will be used for ongoing Wharf maintenance and with the Master Plan study to establish repair priorities for the structure. The following engineering services were performed:

- Perform above and below water inspection to determine the condition of the piling. Develop pile repair or replacement methodology.
- Examine each of the Wharf pilings (approximately 4,450) from top cap to sand line and determine structural viability. Photographs shall be taken to document the underwater inspection. The underwater inspection of the pilings shall be conducted in accordance with the American Society of Civil Engineers (ASCE) “Underwater Investigation Standard Practice Manual”, applying Level I, II and III inspection protocol as follows:
    - Level I – 100% of all pilings, Tactile/visual inspection of all piles
    - Level II – 12% of all pilings
        - Marine growth is removed from three bands and the condition of the underlying pile is inspected
    - Level III – up to 5% of all pilings, need/selection/location is determined based on results of Level I and Level II inspections.
        - Cores are made to investigate for marine borer infestation
        - Extract cores from each age class of piles (see Figure 1-2) on the Wharf (years of construction: 1914, 1950, 1968, 1970s, 1982).



Age Group	Count	48	49	50	52	53	73	74	75	76	77	79	103	104	105	106	107	108	116	117	117	118	119	120	121	122	123	124	125	126	127	128	129	137	138	139	140	155	156	163	164	173	176
1914	11			7	8	3							15									18								15								33		9	10	5	9
1950	11						10		3			4		3	2	8				1	4	8	4					6															
1970-80	10	20	19					26		19	17	21		21	18		22	19																									
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1982	12	3		2																									3	5	6	8	3	4	13	10	3		10				
	54																																										

Figure 1-2: Years of Construction-Pile Core Locations

1.2.2 Description of Structure

The Wharf piles are Douglas Fir timber, 12 inch diameter (nominal), driven tip first, 15 feet into the sand seafloor and beach below. All piles are treated with preservative, the original piles being creosote treatment. The more recently installed, replacement, piles are treated with ACZA and most recently ACZA treated and coated with a polyurea compound. Piles are driven in rows (bents) at 15 ft. centers, and spaced along the row at 6 ft. nominal centers, but varies from 1 foot to 15 feet due to breakage and replacements over the past 100 years. The Wharf maintenance crew maintains a 7 foot maximum spacing on the bent when replacing piles, this spacing was observed during the inspection. Piles are referenced by the location on the bent numbered from the west and the bent number counting from the shore. For example, pile 3 bent 120 is the 3rd pile from the west edge of the Wharf on the 120th row (bent) of piles from the shore, located beneath the buildings in the green area labeled 1982 referring, to Figure 1-2.

The piles support the Wharf structure constructed of timber members described below (see Photo 1-1):

12 x 12 inch pile cap beam (“caps”), spaced at 15 ft. centers of the pile bents

4 x 12 inch beams (“stringers”) at 24 inch maximum centers spanning bents

3 x12 decking laid flat continuously

2 inch asphalt paving on top of decking on road and walkways.

(Note: Photo 1-1 through Photo 1-7 are presented in the report body, additional photographs, Photo 1-8 through Photo 1-76 are presented in Attachment 1A)

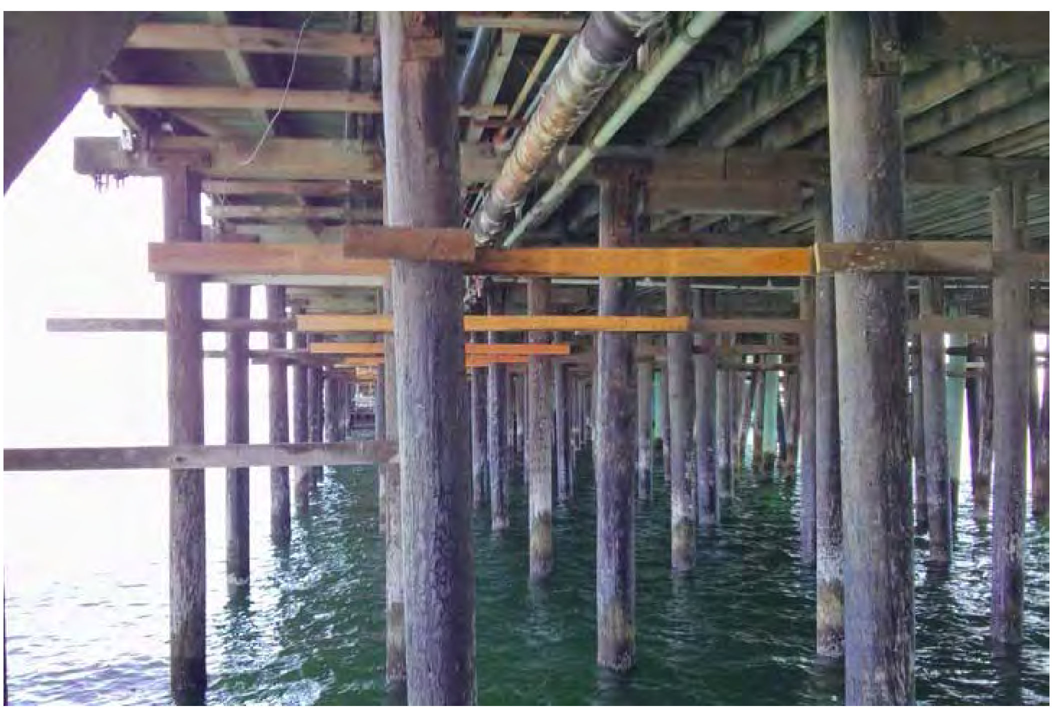


Photo 1-1: Typical Wharf Pile Bent and Structure

The deck elevation of the main Wharf (bent 70-183) is 23 ft. (nominal) above Mean Lower Low Water (MLLW). The northern portion of the wharf slopes up from the shore abutment at elevation 18 ft. (bent 1) to 23 ft. MLLW (bent 70). The water depth ranges from 0 ft. (bent 25) to around 30 ft. at the end (bent 183), resulting in pile lengths extending above the soil (ocean floor) between 20 ft. and 50 ft. From the abutment to bent 25 the pile lengths are 4-18 ft. above the beach sand.

The Wharf supports multiple one- and two-story buildings, the vehicle roadway, pedestrian walkways, and parking. The roadway and parking areas are topped with asphalt concrete (AC). The pedestrian walkways are topped with either AC or concrete. More information on the Wharf substructure, pavement, and wharf buildings and can be found in Sections 2, 3, and 7, respectively.

1.2.3 Methodology

The dive inspection was performed between September 17 and October 4, 2013, and pile core samples were retrieved during March 17 to 21, 2014. The above water inspection was performed between September and November, 2013.

Underwater Dive Inspection:

- Engineer-divers used surface-supplied air to inspect the piles from the waterline down to mud line. Piles were not examined below the mud line.
- The dive crew consisted of either a three or five person team with one or two engineer-divers performing inspections simultaneously.
- Level I, II, and III inspection efforts were performed in accordance with the American Society of Civil Engineers (ASCE) Manuals and Reports on Engineering Practice Number 101, titled "Underwater Investigations Standard Practice Manual", 2001 Edition.
- For the Level II and III inspections, the marine growth was removed by hand with a scraping tool. Typical marine growth consisted of mussels, barnacles, etc.
- Extraction of timber core samples (Level III inspection effort) was performed using an underwater pneumatic-powered rotary coring system. Coring bits were 2 inches in diameter by approximately 9 inches long. Once a core was extracted, a tapered and treated timber cylinder was driven into the hole with a sledge hammer, and any excess plug material was sawn off. Hydraulic-setting grout was used to seal the exterior surface of the plug and the annular space.
- Trestle pile bents 1 through 35 were examined by engineers from the beach during low tide episodes.

All noted elevations are relative to Mean Lower Low Water (MLLW).

Above Water Inspection:

Visual inspection performed on the Wharf underside was performed in a boat and from the horizontal ledgers via hatch access points.

Inspection team members included:

- Brad Porter, P.E. (dive and above water)
- Erica Petersen, P.E. (dive support and above water)
- Heath Pope, P.E. (dive)
- Scott Nordhelm, P.E. (dive)
- James Traber, E.I.T. (dive)
- Nick Ferrante (dive support)
- A.J. Lee (dive support)

1.3 Condition Assessment

The Wharf piles are in satisfactory to good overall condition due to ongoing maintenance. This is noteworthy, considering the age of the wharf. For ease of comparison with the previous pile inspection performed (Hellmers, 1986), the same pile rating system is used. Piles with no damage are identified as "excellent" in this report and the field inspection notes. Piles with deterioration are given a number value between 0 and 100 corresponding to the amount of cross-sectional area that has been lost. A pile is considered "damaged" and in need of replacement if there is more than 40% cross-sectional area loss anywhere along the pile length. Most piles are in excellent condition. A small percentage are in need of replacement. In general, most of the Wharf piles had no visible damage; typical non-damaged piles are shown in Photo 1-2 and Photo 1-3, and Dive inspection data tables are presented in Attachment 1B.

Photo 1-2: Typical non-damaged pile in "excellent" condition

Photo 1-3: Typical non-damaged pile in "excellent" condition

The deterioration of timber structures in a marine environment is most commonly due to physical damage caused by waves or floating logs, fungal attack above water, or marine borers below high tide. Marine borers are organisms that consume wood, thereby destroying the structural integrity of a pile. The two most common types of marine borers in coastal harbors are:

1. Teredo – worm-like, burrow into the interior of the pile, consuming it from the inside out.
2. Limnoria – crustacean, burrow along the surface or exposed part of the pile

Pile damage at Santa Cruz Wharf was observed to follow a pattern (see Photo 1-4):

- A split develops in the grain of pile surface.
- The split deepens and provides access to the untreated interior of the pile for marine borers. The pressure-treated preservative treatment only penetrates the outer circumference (approximately 2 in.) of the pile. The marine borers consume the untreated wood inside the preservative-treated ring.
- It is not uncommon for the exterior preservative-treated pile shell to remain, with the interior hollowed-out by the Teredo.

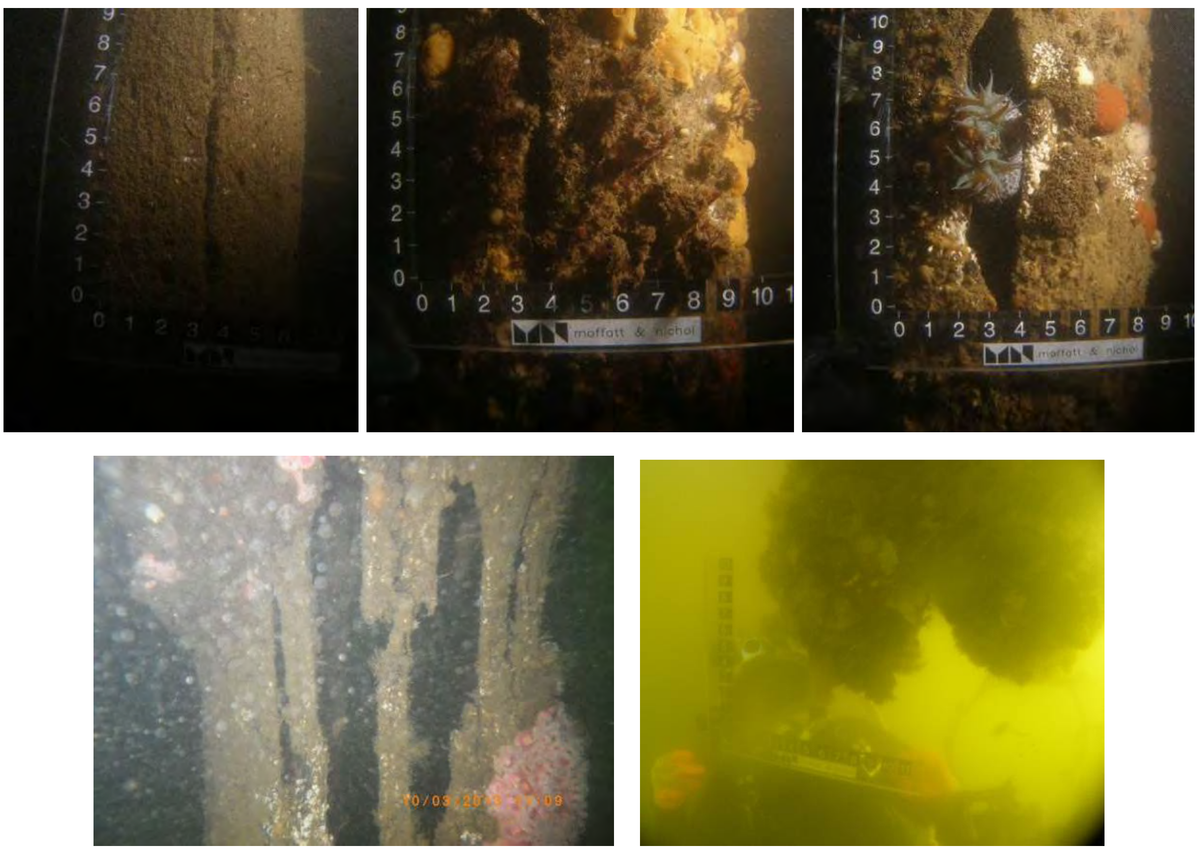


Photo 1-4: Stages of Pile Deterioration

Attachment 1A contains additional underwater photographs (listed on Table 1-1) of Wharf piles with various level of damage.

Table 1-1: Underwater Pile Photo Summary Table

Level of Damage	Photo #
No damage, excellent condition	Photo 1-8 - Photo 1-15
5% section loss	Photo 1-16 - Photo 1-17
Longitudinal splits	Photo 1-18
50% - 80% section loss	Photo 1-19 - Photo 1-23
90% - 100% section loss	Photo 1-24 - Photo 1-28
Pile stubs	Photo 1-29 - Photo 1-30

Pile Connections

The structural members of Wharf support vertical forces ("load" or weight) primarily by simple bearing on the member below. Connections are made between members to transmit lateral forces, generally environmental (waves, tsunami, earthquake, wind). The pile-to-cap connection is critical to the Wharf stability, particularly in resisting earthquake and wave forces. This connection is typically a "T-connection," consisting of a T-shaped steel plate with two bolts on the cap and one bolt on the pile through the stem of the T (Photo 1-34). Many older piles on the Wharf have either "U-strap" or "dowel-only" connections. U-strap connections consist of a steel strap in the shape of an upside-down U; one bolt goes through the pile and strap, and the steel strap loops over top of the cap (Photo 1-35). In dowel-only connections (also called "drift pin"), it appears that the pile has no connection to the pile cap from the outside; there is a dowel in the center of the end grain of the pile connecting through the cap. Dowels can be seen below the cap beams where piles used to be located (Photo 1-36). The dowel-only connections are not as structurally effective, and should be replaced with a T-connection. Many of the T-connections and accompanying bolts are significantly corroded. At the end of the wharf, many have 50% section loss (bents 168 – 183). By Bent 168, the section loss is 30%.

Cores

Pile cores were taken to determine the interior condition of the pile, and detect the presence of marine borers (Teredo) not visible from the exterior. 50 pile cores, total, were taken comprised of 10 cores from each age class (year installed) at representative locations on the Wharf, shown on Figure 1-2 and Attachment 1C. No evidence of marine borers were found in the cores, however the Wharf maintenance crew has occasionally found marine borer damage when removing piles as part of their routine maintenance. A typical example of a core (with no damage) is presented in Photo 1-5, and photographs of all the cores are found in Attachment 1A, Photo 1-37 through Photo 1-76. Photo 1-6 and Photo 1-7 show the cross sections of piles removed in March 2014 that show various levels of damage from Teredo.

Photo 1-5: Typical Core (From Pile 8 on Bent 118)

Photo 1-6: Pile with Teredo damage

Photo 1-7: Pile with Teredo damage

1.4 Analysis

Table 1-2 presents a summary of the pile inspection data. Approximately 6% of the piles are “damaged,” meaning they have lost more than 40% of the cross-sectional area (less than 60% remaining). The damage is primarily caused by marine borers inside the pile.

Many of the damaged piles have an adjacent intact pile, resulting in an effective span within the allowable limits. In these cases, the pile does not need replacement. However, at locations where the effective span is greater than 10 ft., the damaged pile should be replaced. The Wharf maintenance crew maintains a maximum pile spacing of 7 ft. center-to-center. The amount of damaged piles that create an effective of span of between less than 7 ft., between 7 and 10 ft., and greater than 10 ft. are noted in the table. Photo 1-31 shows an example at Bent 137A, where the inspectors observed a large visible span between piles. In addition, the three end piles of the bent at this location are damaged below the water line. (Note: at the time of writing this report, a new pile has been driven in this location).

Table 1-2: Pile Inspection Results (Bents 31-183)

Type	No Noted Damage	Between 5% - 40% Section Loss	≥ 40% Section Loss			Total
			Effective Span < 7 ft.	Effective Span between 7 - 10 ft.	Effective Span > 10 ft.	
# of Piles	4032	148	77	89	93	4439
Percentage of Piles	91%	3%	2%	2%	2%	-

Below some of the buildings, particularly the Miramar Restaurant, there are many missing piles resulting in large spans. This is because driving piles below the buildings is difficult. In many of these locations, the solution has been to install “A-frame” braces to connect adjacent piles and distribute the load. This solution may be acceptable in a few isolated areas, but in the Miramar vicinity, 9 out of 13 bents have A-frames (Photo 1-32). This diminishes structural member redundancy and weakens the vertical load capacity, as each pile is required to support more load. This is especially a problem at Bents 119 and 121, as each of these bent’s A-frames relies upon spreading the load to a damaged pile (pile with more than 40% section loss). Photo 1-33 shows the A-frame at Bent 119.

Drawings showing the locations of damaged piles and A-frames are presented in Attachment 1C.

1.5 Conclusions and Recommendations

1. All the piles with greater than 40% section loss and effective spans larger than 10 ft. should be replaced or repaired.
2. New piles should be installed at all the A-frame locations, especially in the Miramar vicinity. There are methods to install piles through the deck with the building still in place. The building roof and deck are removed, and then the piles are driven.
3. New piles should be installed at locations where the span is greater than 10 ft.
4. T-connections should be added to piles with dowel-only connections.
5. T-connections and bolts with greater than 30% section loss should be replaced.

2. GENERAL STRUCTURAL EVALUATION

2.1 Summary

The structural evaluation of the Santa Cruz Wharf includes assessment of the condition of the existing structural members and analysis of their capacity to safely support the imposed loads (weight, waves, earthquake, etc.). In addition, analyses were performed to inform the Master Plan design. The condition of the structure is good; due to the quality of original construction and continuous maintenance. There are some areas of deterioration, primarily due to water leakage below the deck and vehicle overload in parking areas. Figure 2-1 and Figure 2-2 show overviews of the structural sections and substructure damage on the Wharf, respectively. A summary of recommendations include:

- Replace deteriorated stringers, decking, and caps
- Installation and improvement of connections to stringers and caps
- Reduce large vehicle access onto the Wharf and mark restricted areas
- Retrofit turnaround areas to increase the load capacity and test existing wharf timbers to if higher load capacity is sought
- Design the new East Promenade for a 36,000 lbs. axle load

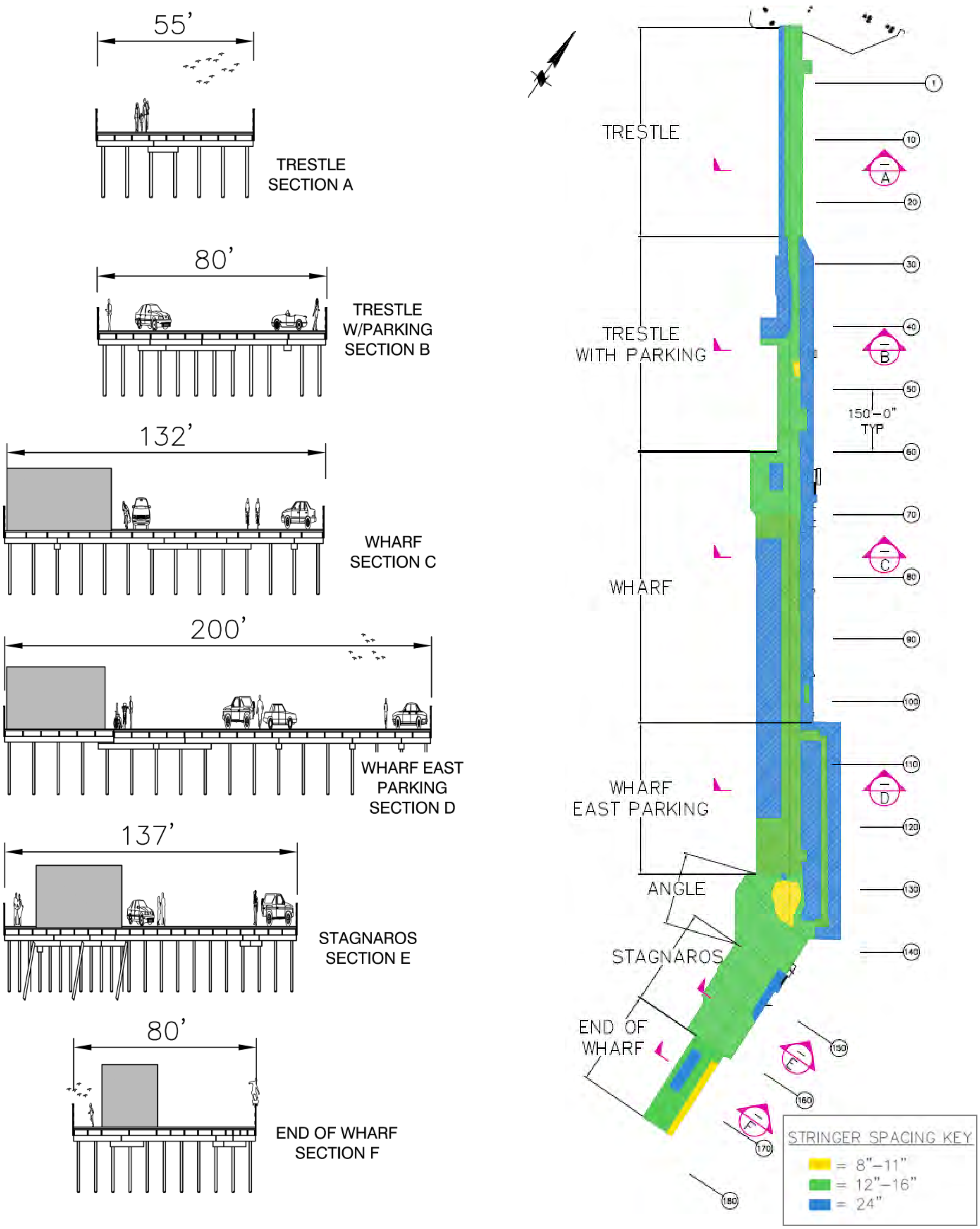


Figure 2-1: Structural Evaluation – Layout Summary Plan

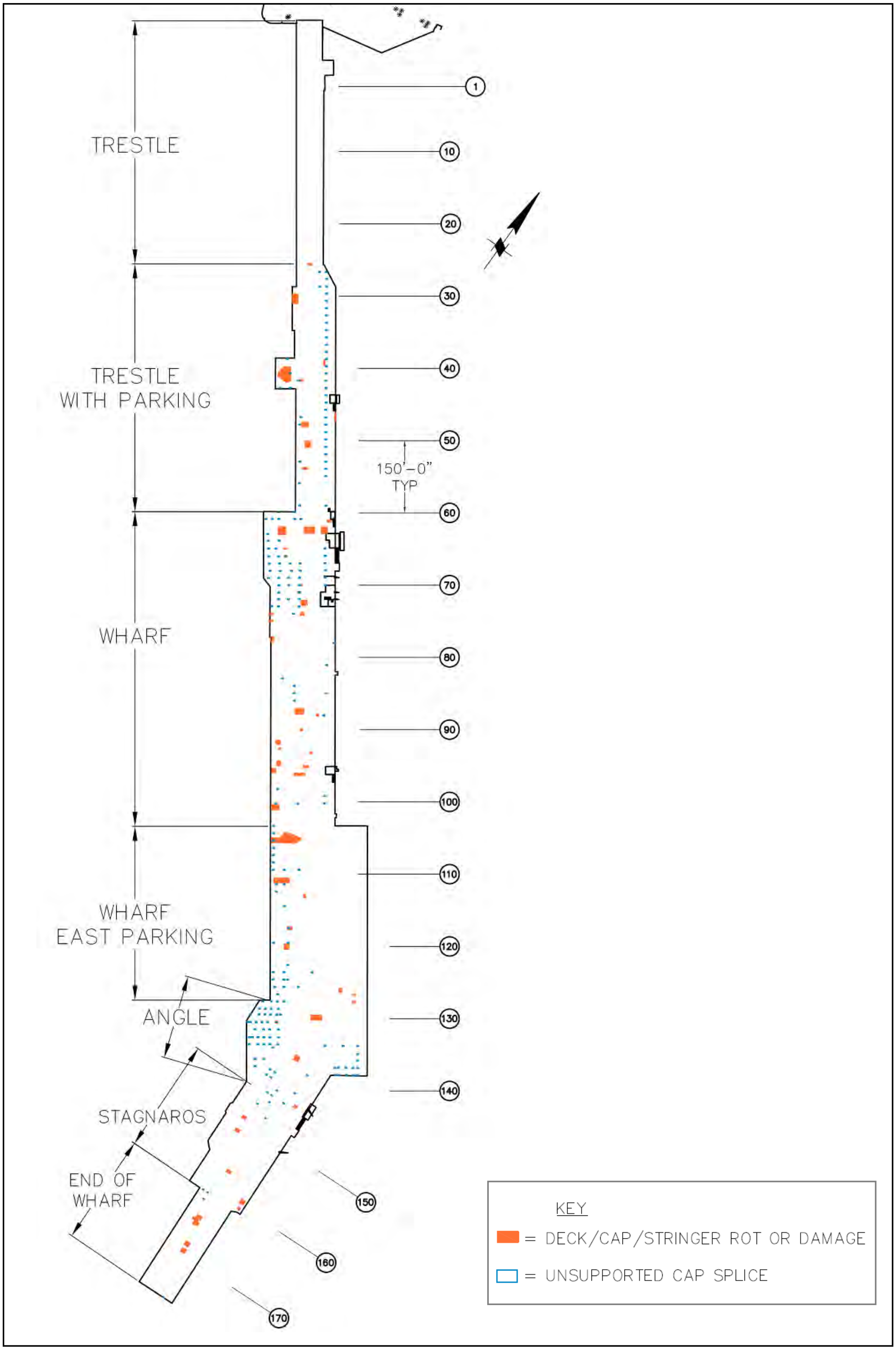


Figure 2-2: Structural Evaluation – Substructure Damage Plan

2.2 Introduction

The Santa Cruz Wharf is a 2,700 ft. long timber structure constructed in 1914. It is the longest timber pier (extends perpendicular to shore) on the coast of the United States, and in continuous use for one of the longest periods of time. Since 1914, it has been widened and modified along its length and has been maintained by a full time wharf crew. The structural members are all timber comprised of 183 rows of piles (bents), supporting beams (caps and stringers) and decking. The decking is then topped with a 2-inch, nominal, layer of asphalt concrete pavement. The arrangement of the structural members is primarily regular, with some variation particularly along joints of different construction periods of the Wharf.

2.2.1 Scope

- Assess the overall structural integrity of the Wharf structure; identify outstanding vulnerability and assess methods to increase resiliency of Wharf structural components relative to existing and changing environmental conditions.

- Generate longevity estimates for components inspected and prioritize necessary repairs and upgrades.

- Provide recommendations and/or strategies for necessary and/or desirable structural improvements, such as widening for stability and/or additional ledgers or bracing.

- Identify structural recommendations and concepts related to (Master Plan).

  - Potential public access and recreational improvements
  - Cantilevered or braced extensions along the Wharf edges
  - Sub-structure improvements for buildings
  - Roadway, parking, sidewalk and public space improvements
  - Light fixtures, guardrails, kiosks, bollards, etc.
  - New refuse collection system

Other sections contain specific evaluations related to the general structural evaluation as follows:

- Roadway--Section 3.
- Wharf Structure Support Buildings-Section 7.
- New Landing (200 ton vessel)-Section 1.

2.2.2 Description of Structure

Structure Type

The Wharf structure is all timber construction. The structure can be separated into 3 functional areas: Foundation (piles and cap), deck (stringers decking and paving) and superstructure (buildings on top). The structural members are connected by "simple" connections—they do not transfer significant bending moments through the connection. An example of a structure that does transfer moments in connections is welded steel structures that act as a frame.

The Wharf foundation (piles) act as cantilever elements. The piles transfer all forces (support) into the seafloor by embedment of approximately 15 ft. The piles support vertical loads (weight) by bearing on the pile tip and friction for the length of embedment. The piles resist lateral loads (wave, earthquake, etc.) by embedment into the soil that produce a bending moment within the embedded portion of the pile. This is similar to a flagpole acting alone.

The Wharf deck (stringers, decking) span as simple beams between pile bents. Decking members span across multiple stringers for vertical loads (weight). The entire timber deck assembly is flexible and acts as a unit to spread load to adjacent members, particular large point loads (such as a truck wheel). For lateral loads, the deck assembly acts as a diaphragm (flexible) to transmit loads across many multiple piles in the foundation.

Structure Members

The details of the structural members are as follows:

The piles support timber cap beams (12x12), stringers (4x12, with some 6x12), and decking (3x12). The arrangement of the caps, stringers, and the connections is varied, particularly along joints of different construction periods of the Wharf. Figure 2-3 and Photo 2-1 present examples of typical wharf framing. (Note: Photo 2-1 through Photo 2-14 are presented in the report, Photo 2-13 through Photo 2-26 are presented in Attachment 2A) The topside of the Wharf has a nominal 2-inch thick layer of asphalt concrete for the road and parking areas. Some of the pedestrian walkways are topped with concrete. The Wharf topside elevation is approximately 23 ft. above Mean Lower Low Water (MLLW). A view of a typical location on the topside of the Wharf is presented in Photo 2-2.

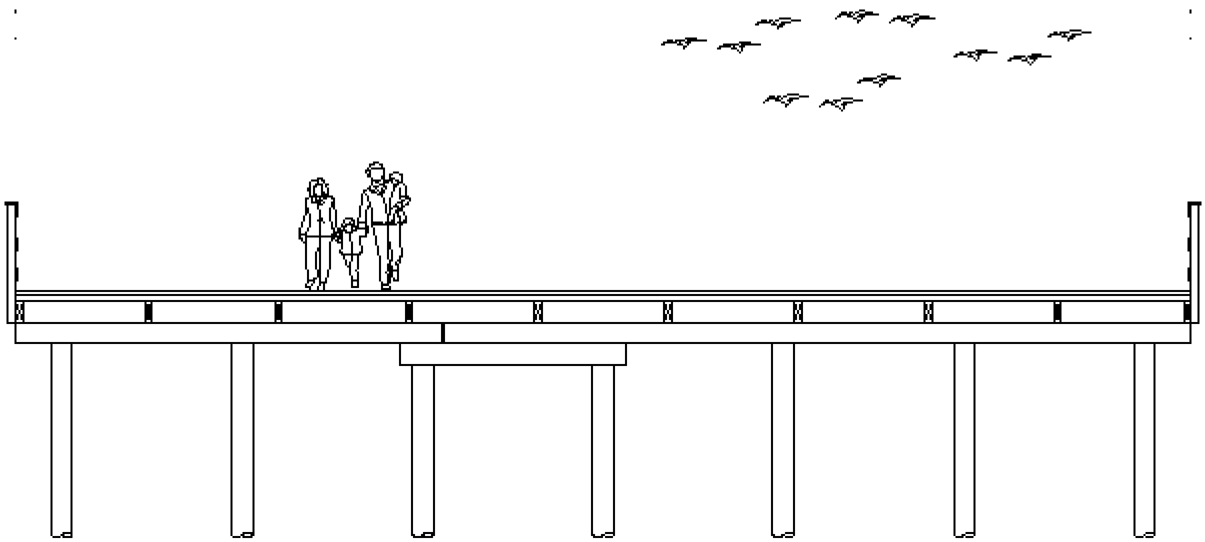


Figure 2-3: Typical Wharf Cross Section

Photo 2-1: Typical Wharf Framing

Photo 2-2: Typical topside view of the Wharf

There are seven distinct areas of the Wharf structure (see layout in Section 2.1):

1. Trestle
2. Trestle with Parking
3. Wharf
4. Wharf East Parking
5. Angle
6. Stagnaro's
7. End of Wharf

A photo of typical bents at each location is presented in Photo 2-3 through Photo 2-9 (west side looking east):

Photo 2-3: Typical Trestle Bent

Photo 2-4: Typical Trestle with Parking Bent

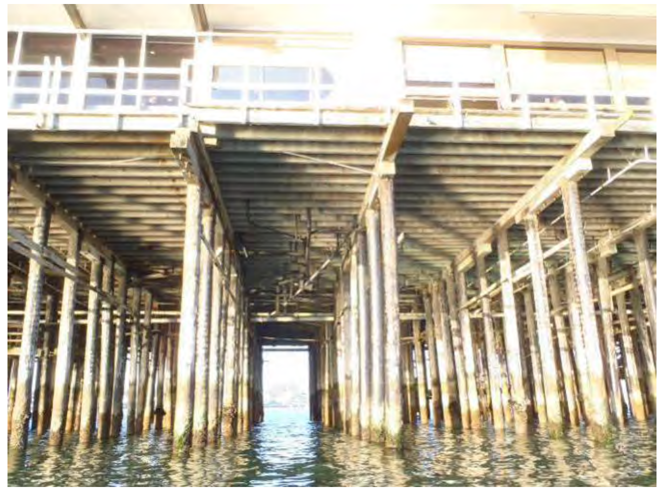


Photo 2-5: Typical Main Wharf Bent

Photo 2-6: Typical East Parking Bent

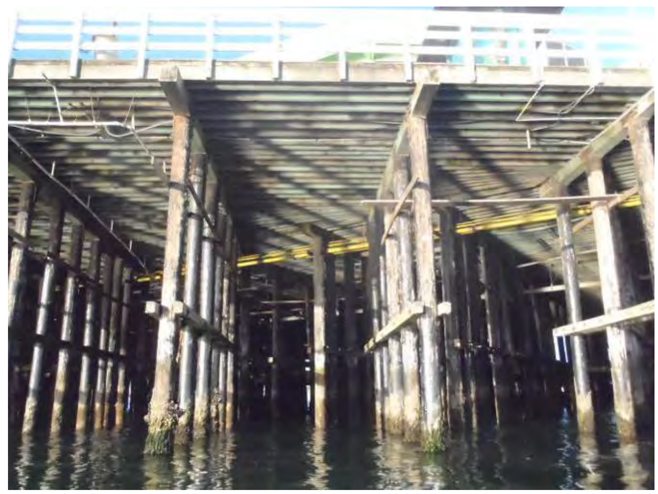


Photo 2-7: Typical Angle Bent

Photo 2-8: Typical Stagnaro's Bent

Photo 2-9: Typical End Bent

Stringers

Stringer size and spacing varies at locations on the Wharf. Beneath buildings and the parking areas stringers are 4x12 at 24 inch centers. In the roadways there are 4x12 with 6x12 added, the spacing is typically 10-12 inches on center, and in some locations 16 in. centers. This provides additional weight capacity in the road. The Wharf staff maintains a maximum spacing of 16 inches on center for replacement, as observed during the inspection. Photo 2-10 shows an example of stringer spacing under the Wharf.

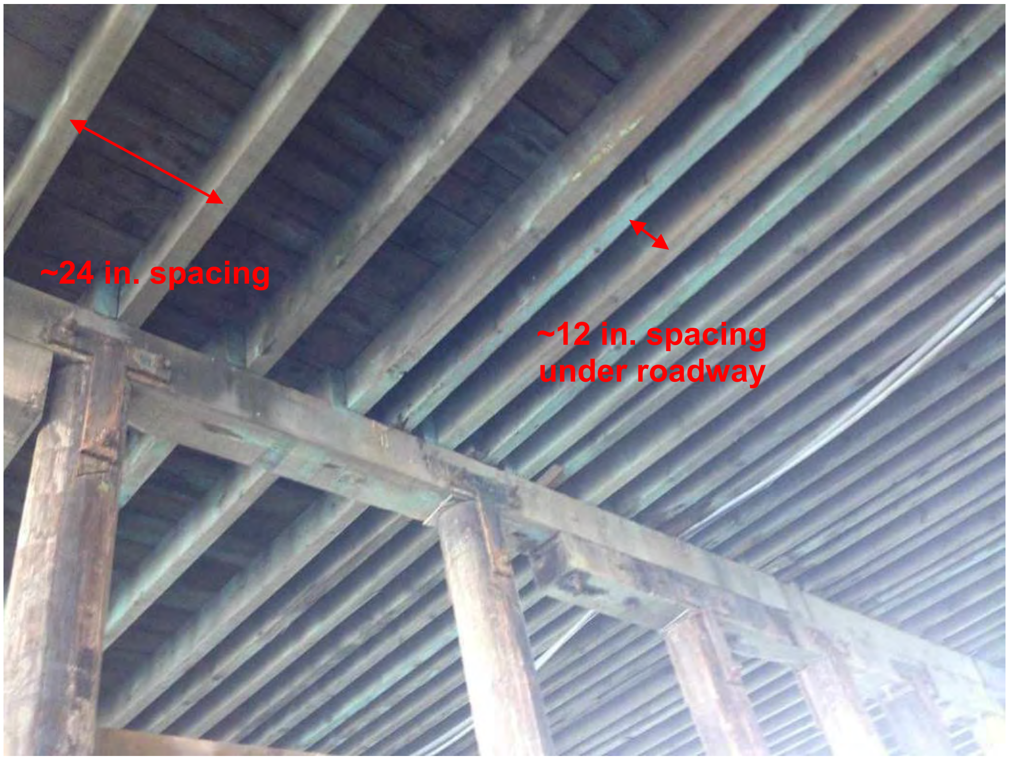


Photo 2-10: Varying Stringer Spacing Under Wharf

Cap Elevation Transition

The west side of the Wharf structure beneath most of the buildings is raised approximately 6 inches above the main deck. This transition is shown in Photo 2-11 and Photo 2-12. The transition occurs between bents 97 and 145A, corresponding with the west expansion of the Wharf in the 1950s and 1980s.

Photo 2-11: Cap/Deck Transition – Bent 99 (Piles 8 and 9)

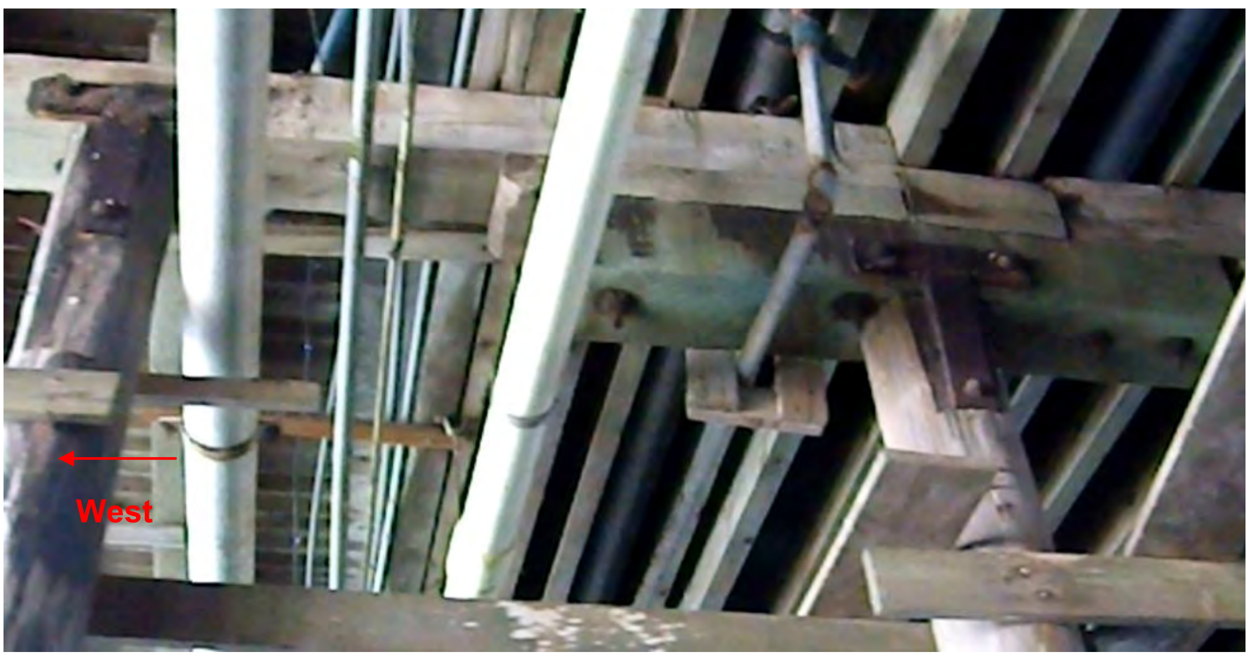


Photo 2-12: Cap/Deck Transition – Bent 125 (Piles 7 and 8)

Cap Splices

In general, cap splices on the Wharf have been created by adding an additional cap below the splice, creating two layers of cap. Ideally, these are located over a pile. A photo showing such a splice is Photo 1-34 (Attachment 1A). The interior portion of most of the Wharf has many segments of double cap, probably due to the Wharf elevation change and/or the addition of new piles on the west side of the Wharf. See Photo 2-13 for a typical example. Near Stagnaro's, there are some locations where there is a triple cap.

Photo 2-13: Long segment of double cap with multiple splices (Bent 165, looking east)

Pile Ledgers

At the south end of the Wharf, horizontal members were installed at elevation 9 ft. MLLW (12 feet below top of pile). These members were installed to provide lateral bracing to the piles, which are longer due to water depth at the end. These horizontal members are 6x6 and 6x8 and run in the longitudinal and transverse direction of the Wharf.

The Master Plan provides additional piles and ledgers at the southern end of the Wharf which will provide additional lateral strength.

2.2.3 Methodology

The above water inspection was performed between August and November, 2013. The inspection was performed on the Wharf underside in a boat and from the horizontal ledgers via hatch access points. A preliminary evaluation was first performed to assess of overall condition of the structure and provide preliminary input to the master plan effort (see memo of Sept 11, 2013 in Attachment 2D).

Inspection team members included:

- Brad Porter, P.E.
- Erica Petersen, P.E.

Structural calculations used the following reference codes:

- American Wood Council National Design Specification for Wood Construction, 2005 (NDS)
- ASCE 7-10
- AASHTO Bridge Design Specifications, 2010
- Previous Santa Cruz Wharf Inspection Report (Hellmer's, 1986)

2.3 Condition Assessment

The caps and stringers that support the decking are in serviceable condition. The structural strength of some members has deteriorated where wetting has occurred due to water leakage, particularly below the buildings and kiosks. Such deterioration was observed on the underside of the Wharf along the east edges of the buildings and beneath the west walkway at the older Wharf joint. Some stringers, decking, and tops of caps appear soft and require replacement. Some caps and cap splices have large splits in them from the edge to the first bolt (Photo 2-14). In general, the wood around the splits is still in adequate condition. These areas are shown on the drawings in Attachment 2B and a summary of the major areas of rot is presented in Table 2-1.

Photo 2-14: Cap Splitting at End – Bent 62, Pile 3

Table 2-1: Deck, Stringer, and Cap Rot Locations

Bent Range	Location of Rot or Damage	Photo #
38-39	West side	
48-50	Under lifeguard building	
54-56	East edge stringers	Photo 2-15
70	Cap between piles 8-10	
99-100	Leaking pipe and drain, localized rot on west side	
103	Cap between piles 13-15	
102-104	Leaking pipe and drain, localized rot on west side	Photo 2-16
104	Fire Damage (not rot) between piles 7-11)	Photo 2-17
108-109	West side	Photo 2-18
112-113	West 60 ft. of wharf	Photo 2-19
118-119	West 40 ft. of wharf	Photo 2-20
120-121	Splitting stringer west part of midbent	Photo 2-21
144A-145A	Deck at midbent	
150-151	Cracks in multiple stringers on east side near platform	Photo 2-22
155-156, 157-158	Leak approx. 40 ft. from west side	
165-166	East side	Photo 2-23
170-172	Under Building, about midbent	
174-176	Between wells and under building	

Stringer Connections

The stringers do not connect to one another at the pile cap where they bear and lap one another from bent to bent in the longitudinal direction. Although for a simple beam, this is not required for the vertical loads, but without a connection, there is reduced longitudinal continuity (diaphragm action) along the Wharf to resist lateral loads. Providing a connection such as a bolt at the end will increase strength during a seismic event.

Unsupported Cap Splices

In some locations, splices between the cap joints are unsupported below. In general, this is not a concern if the bolts connecting the splices are adequate, however in some locations the unsupported splice occurs in a large span between piles or there is an equivalently large span due to damaged piles. There were no locations where the bolts appeared to be failing, however these areas should be monitored. Unsupported cap splices were observed in localized areas and a few repeating areas; the notable areas are described in Table 2-2 and shown in the drawings of Attachment 2B.

Table 2-2: Unsupported Cap Splice Locations

Bent Range	Location of Cap Splices	Photo #
36 to 61	1 unsupported cap splice at 20 ft. inside east edge	-
69	Unsupported cap splice + nearby damaged piles	
69 to 81	Scattered throughout, mostly on west side	
81	2 locations with unsupported splice and large span due to damaged piles	
108	Unsupported cap splice + 13 ft. span	Photo 2-24
111 to 117	1 at 5 ft. inside west edge	Photo 2-25
131 to 141	West side has unsupported splices every 10-20 ft. on west ~60 ft. width of the Wharf	Photo 2-26
144-145	5 unsupported cap splices on east 75 ft. of wharf	-

2.4 Analysis

The following analyses and calculations were performed:

- Determination of structural member capacities
- Vehicle and Live Load Analysis on Roadway
- Truck turning radius analysis
- Preliminary seismic analysis of single pile
- Analysis of pavement sections (see section 3)
- New Building Vertical Load Analysis (see section 7)
- Analyze berthing of vessel at new landing (see section 1)

The full calculations are provided in Attachment 2C, however a summary of the results is provided here.

2.4.1 Capacities of Structural Members

Timber construction is analyzed using “allowable stress design”. Timber members are tested to failure, the load at failure (the “ultimate” load the member can support) is recorded and then this load is reduced by a factor of safety to provide an “allowable” load. Because of the variability in wood members (knots, grain spacing, etc.) a relatively large factor of safety is used to reduce the load allowed referred to as the capacity of the member. The capacities of structural members were calculated using the National Design Specification for Wood Construction (NDS 2005) and are summarized in Table 2-3. During a seismic event, the capacity is allowed to be increased by 1.6 (per NDS 2005 - load duration factor).

Table 2-3: Structural Member Capacities

NON-SEISMIC LOADING

Structural Member	Allowable Bending Moment	Allowable Shear Force	Allowable Axial Force
	lb-ft	lb	lb
Cap, 12x12	38,400	16,300	-
Stringer, 4x12	11,600	5,500	-
Stringer, 6x12	19,200	8,100	29,400 compr/ 68,400 ten
Decking, 3x12	2,600	4,100	-
Pile,2" Diam.	26,600	9,700	41,000

SEISMIC LOADING

Structural Member	Allowable Bending Moment	Allowable Shear Force	Allowable Axial Force
	lb-ft	lb	lb
Cap, 12x12	61,440	26,080	-
Stringer, 4x12	18,560	8,800	-
Stringer, 6x12	30,720	12,960	47,000 compr/ 109,400 ten
Decking, 3x12	4,160	6,560	-
Pile, 12" Diam.	42,560	15,520	65,600

2.4.2 Vehicle Load Analysis

The vertical load analysis results for trucks and vehicles are summarized in this section. A memorandum dated January 23, 2014, provides more details on the calculation methodology and assumptions and is presented in Attachment 2D.

Roadway

The analysis shows the allowable vehicle axle load the Wharf can support (capacity) is 29,000 to 34,400 lbs. The range of axle loads (demand) of Santa Cruz Fire Dept. (FD) trucks is 23,000 to 31,000 lbs. Therefore, some of the FD trucks fall within the allowable range of the existing wharf. With improvements to the existing wharf, the maximum allowable axle would be 35,600 lb., which would include all the FD trucks. These allowable axle loads correspond to the typical 18-21 ton truck (80% total truck weight is on rear axle based on the American Association of State Highway and Transportation Officials (AASHTO)).

Additionally, a FD truck turning radius analysis was performed; some of the FD trucks will be able to turn around within the existing roadway (without having to back up) at the circle at the end of the Wharf and all can turn through the east parking lot.

Parking Area

In the Wharf parking areas, the maximum allowable vehicle is an axle load of 13,500 lbs. corresponding to an 8 ton truck, less than half what the roadway can support. This capacity is sufficient for passenger cars and light trucks but general trucks should not enter the parking areas.

2.4.3 Seismic Analysis

A preliminary analysis of the Wharf performance in a seismic event was performed. The Wharf structure is constructed primarily of vertical timber piles with bolted top connections. In this configuration, it is expected that the piles will behave individually (cantilever structure or “flagpole” described early in this section) during a seismic event. Accordingly, a single pile analysis is considered an appropriate methods.

The pile was modeled in the structural analysis program SAP2000 as a single frame member fixed at the bottom and with a partially fixed condition at the top. The effective length of the pile was estimated as 58 ft., with a fixity in the ground at 5 pile diameters below the mud line (5 ft.). The partial fixity at the top accounts for the restraint that the T-strap connections provide, and was modeled as a rotational restraint. A response spectrum analysis was run using the design response spectrum per USGS and the 2012 International Building Code. The effective wharf mass was calculated and lumped at the top of the pile.

Santa Cruz Wharf is in a seismically active area and has withstood a number of earthquakes during its 100 year in existence. These include the Loma Prieta earthquake whose epicenter was 5 miles away from the Wharf. This earthquake caused significant damage to Santa Cruz including a level of damage to downtown (1/3 mile away) that required a complete reconstruction of downtown. The vertical timber pile construction of the Wharf is inherently flexible and provides consider isolation to the shorter period components of the earthquake motion that are the most intense and cause the greatest damage. There was no damage to the Wharf during the Loma Prieta Earthquake.

The results of the analysis are summarized in Table 2-4.

Table 2-4: Single Pile Seismic Analysis Results

1 Story Building

Demand	unit	Transverse (Y direction)	Longitudinal (X direction)	SRSS (Combine 100% + 30%)	
				30X + 100Y	100X + 30Y
Displacement	in	38	29	39	31
Moment	lb-ft	24,056	24,889	25,188	25,914
Shear	lb	553	760	598	778

2 Story Building

Demand	unit	Transverse (Y direction)	Longitudinal (X direction)	SRSS (Combine 100% + 30%)	
				30X + 100Y	100X + 30Y
Displacement	in	43	32	44	34
Moment	lb-ft	26,667	27,333	27,899	28,480
Shear	lb	607	867	660	886

The estimated displacements of the Wharf during the design seismic event are around 2.5 to 3.5 ft. The moment and shear demands in the piles are less than the capacities during a seismic event.

2.5 Conclusions and Recommendations

The Wharf structure is in good condition overall. There are some areas that have deterioration (rot, corroded steel connections, and other damage). A preliminary structural analysis was performed of the Master Plan elements, along with a general seismic evaluation. The following recommendations are made:

1. Add piles for additions for the Master Plan elements to increase lateral stability to the Wharf.
2. Provide additional bolts or side plates at the unsupported cap splice locations.
3. Replace deteriorated structural members.
4. Review City operations plan for weight of specific vehicles to access the Wharf in emergencies
5. Provide markings to restrict trucks from parking spaces and lower capacity areas
6. Test existing wharf timbers to obtain actual allowable stress values if higher load capacity is sought
7. Retrofit turnaround areas to increase the load capacity
8. Design the new East Promenade for a minimum capacity of 36,000 lbs. axle load
9. Provide connections (bolt) to stringers at lap splice ends to provide longitudinal continuity.
10. Perform design level seismic analysis for additions to the Wharf.

3. ROADWAYS/PARKING /PARKING CONTROL SYSTEM

3.1 Summary

The asphalt pavement of the Wharf roads and parking areas is severely cracked and deteriorated over the majority of the traffic areas. This is due to the flexible substructure (timber framing between piles) that supports the pavement, and the regular travel of large trucks (garbage and delivery) along the road. Resurfacing the entire pavement area using a plywood underlayment to minimize reflective cracking is recommended as well as reducing truck traffic to the extent practical. A method to collect storm water from the vehicle-traveled and parking areas should be incorporated as part of the replacement paving to filter road runoff before discharging into Monterey Bay.

3.2 Introduction

Santa Cruz Wharf has had vehicle access along its length since construction 100 years ago. Providing vehicle access for delivery, public access and emergency vehicles is a continuing requirement of operation. The need for a durable yet flexible road surface is a challenge that requires considerable maintenance. Deck board crack are the source of pavement cracking. Providing a system that bridges across the deck board cracks and reducing large vehicle loads onto the Wharf will help to reduce pavement deterioration and ongoing maintenance.

3.2.1 Scope

- Conduct an assessment of the existing weight-bearing capacities and structural integrity of the pavement and substrate for the Wharf, roadways and parking areas.
- Identify weak and vulnerable areas.
- Provide options for more durable and environmentally friendly paving and surface coating materials.
- Identify methods for meeting Best Management Practices for storm drainage and for the avoidance of ponding and slip and fall accidents as well as for compliance with Title 24 ADA accessibility requirement.
- Conduct a preliminary assessment of construction and maintenance costs relative to the alternative materials and treatments.
- Recommend paving and/or surface coating materials for roadways and parking areas.
- Recommend potential sources for funding for pavement maintenance and develop a maintenance and replacement schedules.

3.2.2 Description of Structure

The Santa Cruz Wharf is a timber structure constructed in 1914. The Wharf deck for road and parking areas is covered with a two inch (nominal) thick asphalt pavement supported by flexible timber decking and framing beneath. The pavement has extensive cracking, reflected up from the decking joints beneath and is essentially porous to rain water. There is no existing storm water collection system—except at localized wash-down and trash enclosures--all storm water runoff flows through the deck into the bay.

3.2.3 Methodology

Pavement Performance

The analysis of pavement performance is largely a function of the support below the pavement section (“subgrade”), which is the Wharf structure. The methodology for the analysis of the Wharf structure is presented in the Vehicle Loading Memorandum (dated Jan 23, 2014- attachment 2D).

Pavement performance analysis is based upon criteria in the following standards:

- Guide for Design of Pavement Structures- American Association of State Highway and Transportation Officials (AASHTO), 1993
- Asphalt Paving of Treated Timber Bridge Decks, US Dept. of Agriculture (USDA), Forest Service, 7100 Engineering 0371-2809P-MTDC

Per the USDA 7100 standard, damage occurs to pavement with decking perpendicular to traffic flow when wheel load deflections to the section (stringers, decking and pavement) are of the following magnitude:

0.00- 0.05 inches	no damage
0.05-0.10 inches	pavement cracks
0.10 and greater	pavement ravels (crumbles)

Storm Water Treatment

The following documents were used as guidance for requirements for handling storm water runoff from Santa Cruz Wharf and apply to the entire wharf including pavement areas and building roofs.

- "Development and Remodeling Projects, Storm Water Best Management Practices (BMP), Chapter 6 of the BMP Manual for the City's Storm Water Management Program" City of Santa Cruz, November 2012
- “Post-Construction Storm water Management Requirements For Development Projects in the Central Coast Region”, California Regional Water Quality Control Board, Central Coast Region, Resolution R3-2013-0032, July 2013

The regulations that apply to Santa Cruz Wharf are twofold; those that apply to the existing conditions and those that would be considered as new construction. Even with the distinction of existing and new, there is considerable area for interpretation, for example: if a second story is added to an existing building and the foot print is not expanded. Meetings were held during September 2013 to obtain input on how these requirements apply (Rodney Cahill, Mesiti-Miller Engineers; Agnes Topp, City of Santa Cruz). Based upon the foregoing, the following approach will be taken for storm water runoff from the Wharf:

1. Peak flow treatment is not considered appropriate at the Wharf, since the Wharf is located in the Monterey Bay, and there are no downstream capacity or erosion concerns.
2. Storm water quality treatment is a consideration for roadways (sediment, oils, grease). Water quality treatment will not be required for paved pedestrian-only areas, nor existing building roof areas.
3. For new buildings, the roof downspouts should direct roof runoff onto vegetated areas or into cisterns/rain barrels for reuse. However, certain building remodels within the existing footprint are exempt: Second-story additions that do not increase the building footprint. Further, building redevelopment is defined, in part, as "creation or addition of impervious surface…the expansion of a building footprint or addition or replacement of a structure".
4. New walkways constructed with decking boards and gaps to allow for drainage would not require treatment since the surface would be pervious.
5. Centralized treatment on land is probably not feasible or desirable. Disadvantages include pumping requirements, space available on land for a treatment system, and any storm drainage outlets discharging onto the beach are highly concerning to the public. The return flow from such a system would essentially return it to the ocean beneath the Wharf, as currently occurs at the existing storm drain discharge onto Cowells Beach. Peak flow treatment (detention/retention) may also be triggered by a land-based system.
6. Decentralized water quality treatment systems with discharges from the Wharf directly into the Monterey Bay are more feasible than a land based centralized system.
7. Water quality treatment would focus on oils and grease and, to a lesser extent, sediment. Trash and litter is also a pollutant of concern. Consider fencing/netting at lower railings to capture windblown litter.
8. If possible, water quality treatment should involve a vegetation based treatment. Given the underlying structural capacity concern and the marine environment, vegetation based treatment may not be feasible.
9. Structural water quality treatment, such as oil and grease chambers, swirl chambers and media filters would likely be the most feasible water quality treatment tool.

Numeric Design:

1. Flow based design is currently 0.2 inches per hour
2. Percentage removal rate will be clarified by the City

3.3 Condition Assessment

The overall condition of the pavement on the Wharf is poor. The asphalt pavement of the Wharf roads and parking areas is severely cracked and deteriorated over the majority of the traffic areas. There is pavement cracking running parallel to the deck boards (reflective cracking) throughout. In many locations the cracks run in the orthogonal direction as well, creating loose pieces of asphalt concrete (AC). The AC was removed in a few locations around the Wharf to examine the condition of the timber decking below. The decking appeared to be in fair condition below the AC, except in some locations where there was splitting and rotting of the timber boards. In these areas, there is a noticeable depression in the pavement in addition to the prevalent cracked condition of the pavement.

The photos in the following table show examples of the pavement condition across the Wharf, going north to south. All photos are found in Attachment 3A.

Location	Description	Photo #
Bents 50 to 100	Pavement general condition	Photo 3-1 through Photo 3-7
Bent 100 vicinity	Timber deck boards with aligned butt joints	Photo 3-8
Bents 100 to 150	Pavement general condition	Photo 3-9 through Photo 3-17Photo 3-16

3.4 Analysis

3.4.1 Truck Loading

(See Section 2.4.2 for the analysis of the structure for truck loading.)

Damage to road pavement is a function of the number and types of wheel/axle loads that the pavement will be subject to over its life. To analyze and allow comparison, the AASHTO Guide uses a method that converts damage from various wheel loads to damage from an equivalent number of “standard” or “equivalent” loads from an 18,000 lb. equivalent single axle load (ESAL). This approach defines how many trips of a different size vehicle would be equivalent to one trip of an 18,000 lb. axle for comparison. Figure 3-2 below lists these comparisons, adjusted to be compared to a 32,000 lb. axle truck instead of an 18,000 lb. axle.

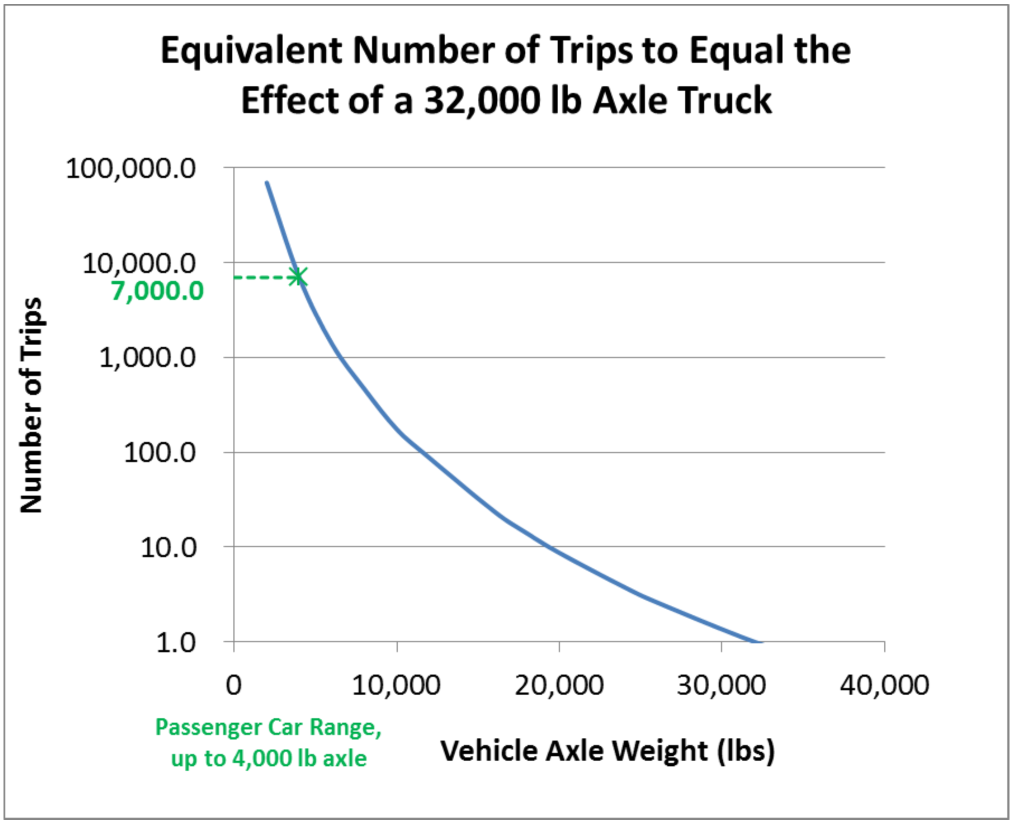


Figure 3-1: Equivalent Number of Trips

It can be seen that the damage caused by a large truck single trip (32,000 lb. axle) is equivalent to thousands or tens of thousands of passenger vehicle trips.

Because timber structures have inherent flexibility, it is challenging to provide a durable surface that will withstand large vehicle loading. . Asphalt is one of the more flexible and cost effective alternatives. For very light loads, such as pedestrian or passenger cars only, cracking can be minimized or avoided entirely for asphalt/timber structures. Avoiding cracking with truck traffic is problematic. Figure 3-2 shows the calculated deflection versus axle weight for various vehicle sizes. It can be seen that axle weights greater than 8,000 lbs. will cause unraveling.

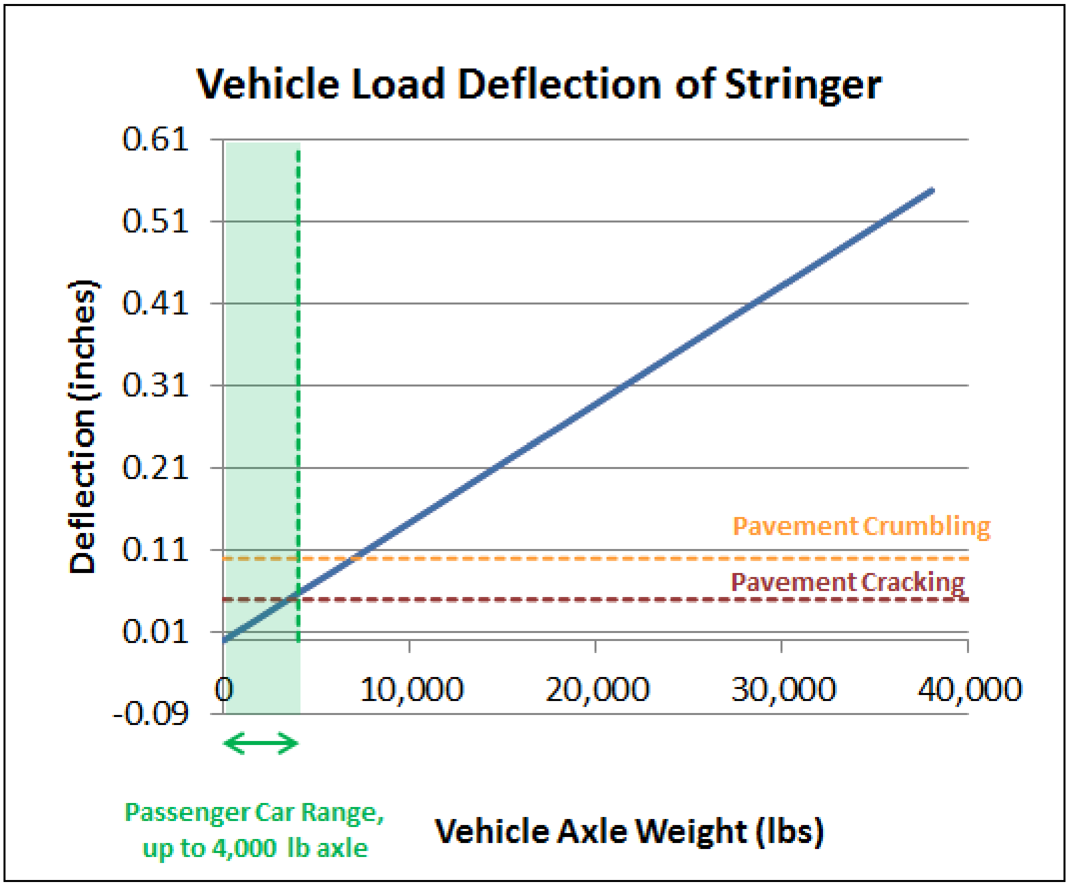


Figure 3-2: Deflection of Stringers for Vehicle Loads

It can be seen that for the range of passenger car axle weights, the deflections are generally less than the cracking deflection. However, the design trucks with axles of around 30,000 lb. cause deflections larger than the cracking deflection and pavement crumbling deflection of 0.1 inch.

3.4.2 Pavement

Based on the observed damage to the pavement, it appears that the following process occurs:

- Initially new pavement is stiff across the deck boards.
- Trucks drive across and over time, the nails loosen and the boards readily bend.
- The timber decking becomes more flexible as the nails loosen and more movement occurs.
- Concurrently, the pavement crack initiates, allowing movement and more cracking. More cracks in the AC provide more flexibility in the section and a degenerative cycle progresses.
- The cracks also allow water seepage onto the timber decking. A small check in the board becomes rotted and adds to degeneration.

The joints at the ends of deck boards are generally staggered, but in some cases they line up where a cut had to be made for repairs. This leads to longitudinal cracks in the road at the butt joint location, as there is the most movement at the end of each deck board.

It is estimated that less than 20% of the existing pavement is in serviceable condition, and it is located in isolated patched areas that would preclude being able to efficiently salvage these areas. Further, any repaving of the existing deck/pavement structure will continue to experience the process of damage described above. Improvement to the structure section throughout the Wharf is required to address the pavement cracking. Considering this, resurfacing of the entire pavement area with a new system is needed.

3.4.3 Pavement Alternatives

A number of alternatives were considered for paving the vehicle areas on Santa Cruz Wharf, which included:

1-Replacement in Kind- Existing 2 inches Asphalt Pavement (AC)

2-Replace Existing AC with added Plywood and Membrane

2.1-Replace Existing AC with added Grid Reinforcement and Membrane

3-Concrete Panels

4-Pervious Pavers on Membrane

4.1- Pervious Pavers on Plywood and Membrane

5-Unit Pavers

6-Unit Pavers to match existing Decking

7-Concrete and Metal Deck Pan

Due to its practicality and cost, the Alternatives 2 and 2.1 that utilize asphalt pavement were selected for further consideration, given the scale of repaving the entire Wharf. Alternative 2 uses plywood to minimize or eliminate the reflective cracking that occurs in the pavement. In addition to bridging across the deck boards, the plywood will add stiffness to the section and further reduce deflection and the cause of cracks. The cost of this system is estimated to be $25-30 per square foot.

Alternative 2.1 uses fiberglass grid embedded in the asphalt (see Figure 3-3) to provide added strength and resist cracking. Although it is not anticipated to provide the same resistance to reflective cracking as the use of plywood, this system has performed well in other applications. It is in use at the Hyde Street Pier in San Francisco, a timber wharf subject to pedestrian and occasional service vehicle loads (ref. memos of Feb 18 and March 5,, 2014, see Attachment 3B). The cost of this system is estimated to be $12-15 per square foot.

Rubberized asphalt should be considered for either of these systems for increased flexibility and crack resistance.

To further assess the performance, constructability and cost effectiveness of these two systems, installation of a test section in the road area for each of these systems should be constructed and evaluated for at least 6 months.

Drainage

The current road surface has no slope and drains through the cracked pavement into the ocean, and allows the constant moisture to accelerate deterioration of the underlying timber structure. When repaving, the pavement can be sloped to collect the water into inlets that can treat the runoff through media (carbon filtration) before discharging it into the bay water. Slopes should be minimized to the extent possible to reduce additional weight. Installation of valley gutter will help do this. The conceptual design of this system is shown in Figure 3-4 and Figure 3-5 . The system will provide a seal over the deck boards to eliminate seepage below and a collection system to allow any trapped water that may collect at the bottom of the asphalt to be drained through a deck “bleeder”. A detail showing the deck bleeder drain detail is provided in Figure 3-6. With the exception of the plywood underlayment, these Figures apply to the grid reinforced pavement system (Alternative 2.1) as well.

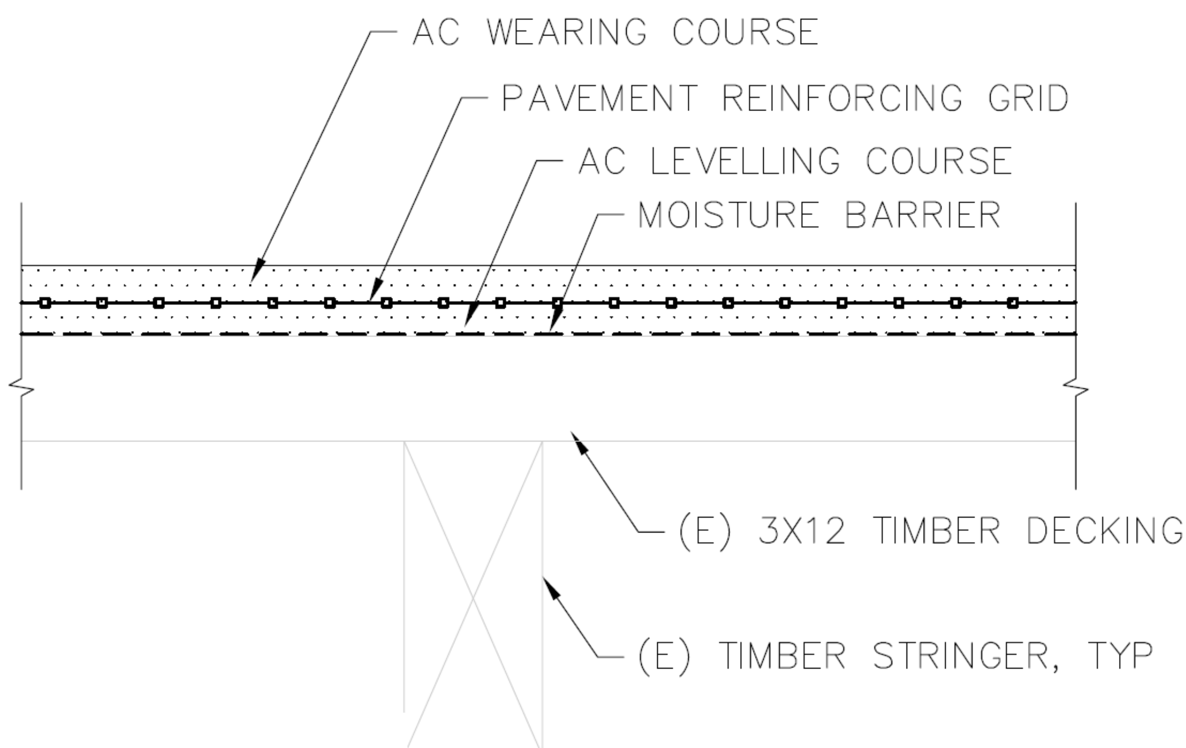
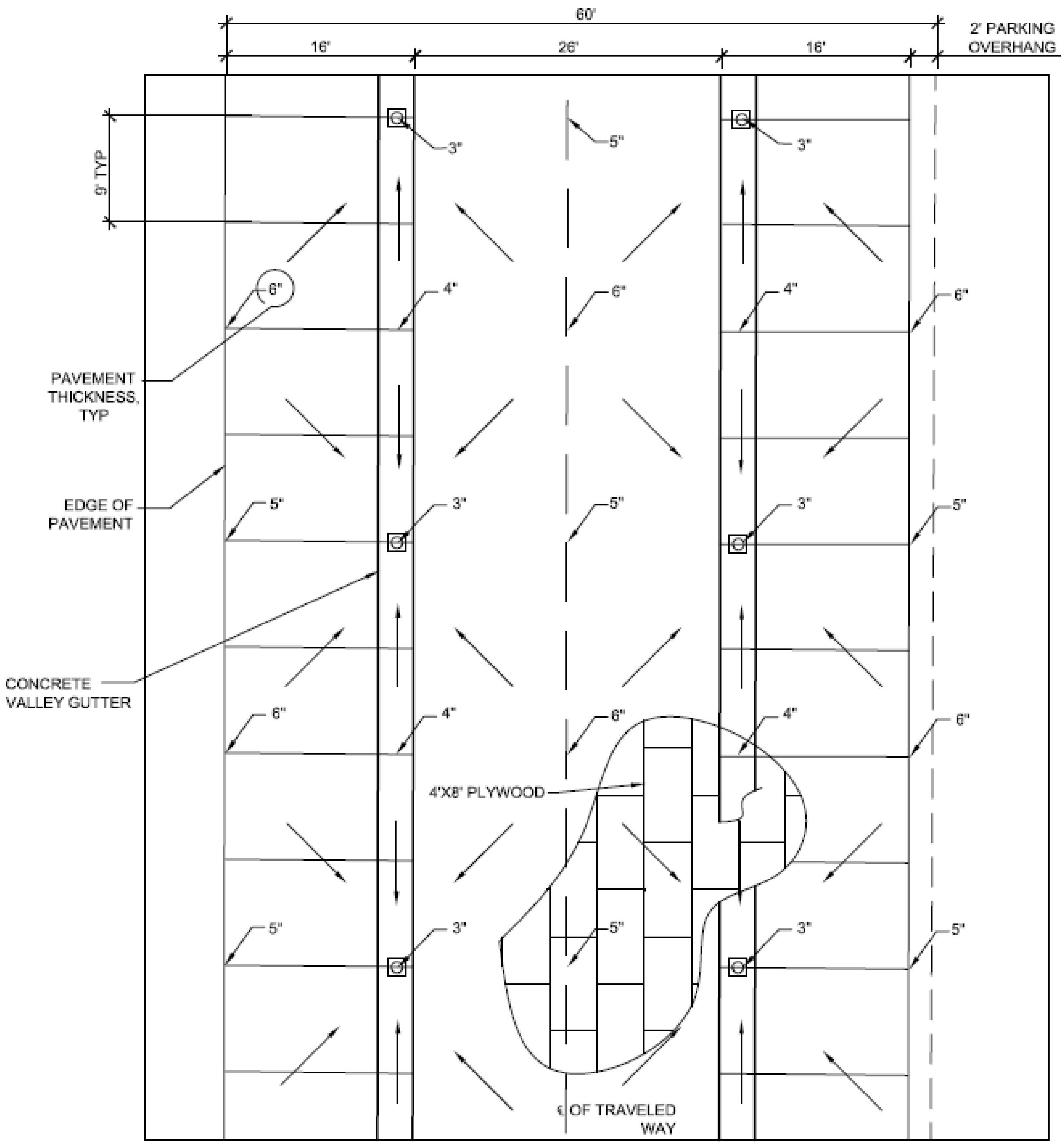


Figure 3-3: Grid Reinforced Pavement Design-Alt 2.1



PLAN

Figure 3-4 : Pavement Conceptual Design (Mesiti-Miller Engineers)-Alt 2

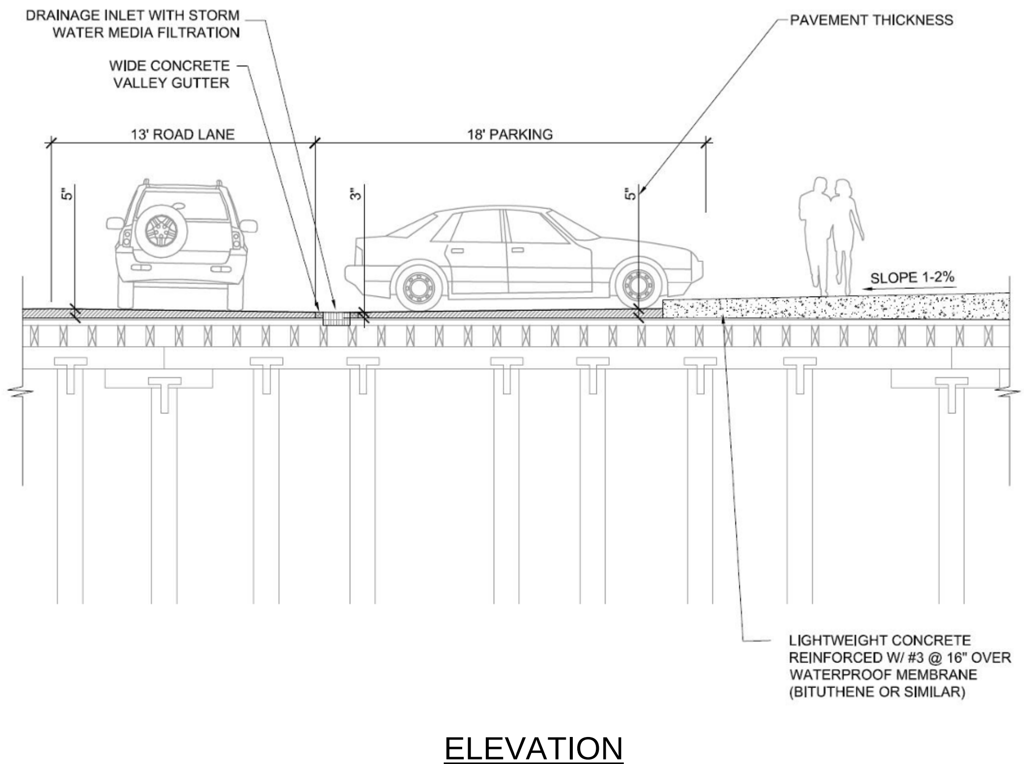


Figure 3-5 : Pavement Conceptual Design (Mesiti-Miller Engineers)

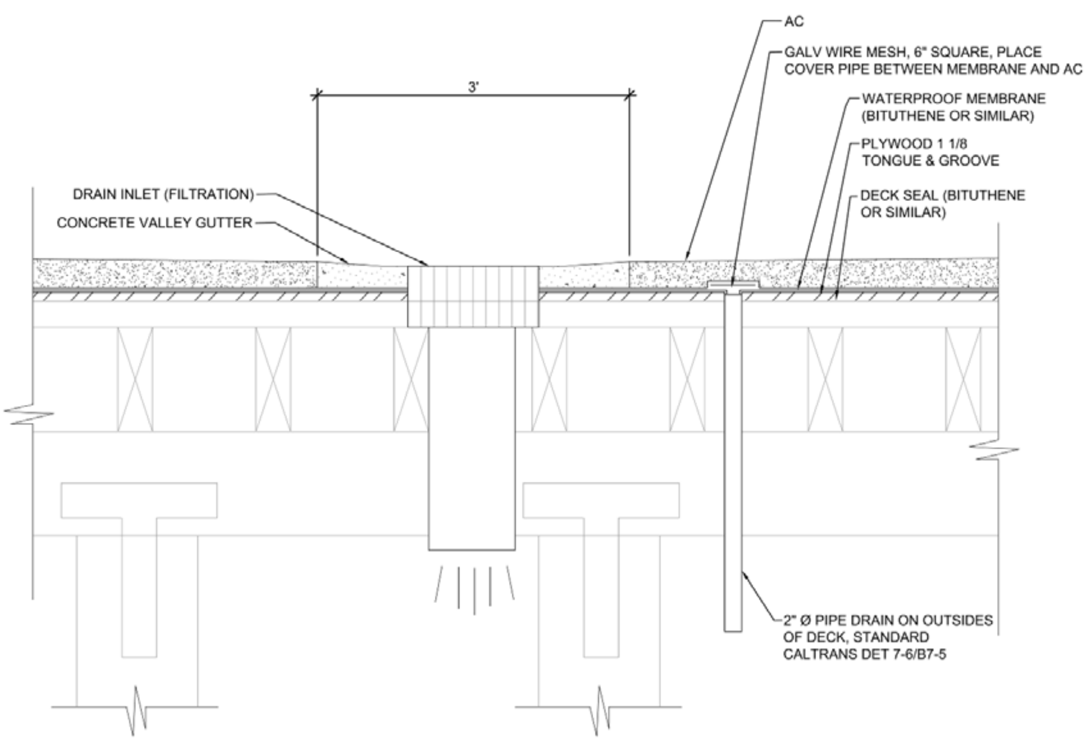


Figure 3-6 Deck Bleeder Drain Detail (Mesiti-Miller Engineers)

Maintenance/Replacement Schedule

As noted above, the existing pavement has deteriorated to a condition that cannot practically be maintained and should be replaced. The replacement cannot likely be performed at a single time and will have to be performed in sections to allow the Wharf to remain in service during the repaving and as funding allows. To utilize rubberized asphalt a minimum batch size of approximately 30,000 sf of asphalt would have to be ordered for the plant to produce the material. Considering these two factors, a plan to minimize disruption and allow efficient re-pavement of the Wharf was developed with the Wharf Staff. Figure 3-7 shows the schedule of priority for replacement of the pavement. It is estimated that each of the 6 areas would require 6-12 weeks to complete the work which would involve the following major tasks:

- Remove existing pavement
- Repair and replace damaged substructure (decking, stringers, caps, piles)
- Place new curb and valley gutter
- Place new asphalt

The new pavement system will have a different thickness than the existing, so a tapered edge will be required at the transition to the existing pavement at each phase, until the final phase is completed.

Funding

The sources of funds for replacement and future maintenance of the Wharf pavement could be a combination of existing City revenues and potential grant funds. Presently fees are collected by the City to drive onto the Wharf; these revenues should be the primary source and proportional to the cost of the pavement and could be supplemented by other City Funds.

The recommended paving system will treat storm water runoff before it is discharged into the Monterey Bay, which does not occur presently. This improvement to water quality may qualify for grant funding that targets such improvements. Specific grants have not been identified, but this approach is the most likely source of grant funding for Wharf pavement replacement.

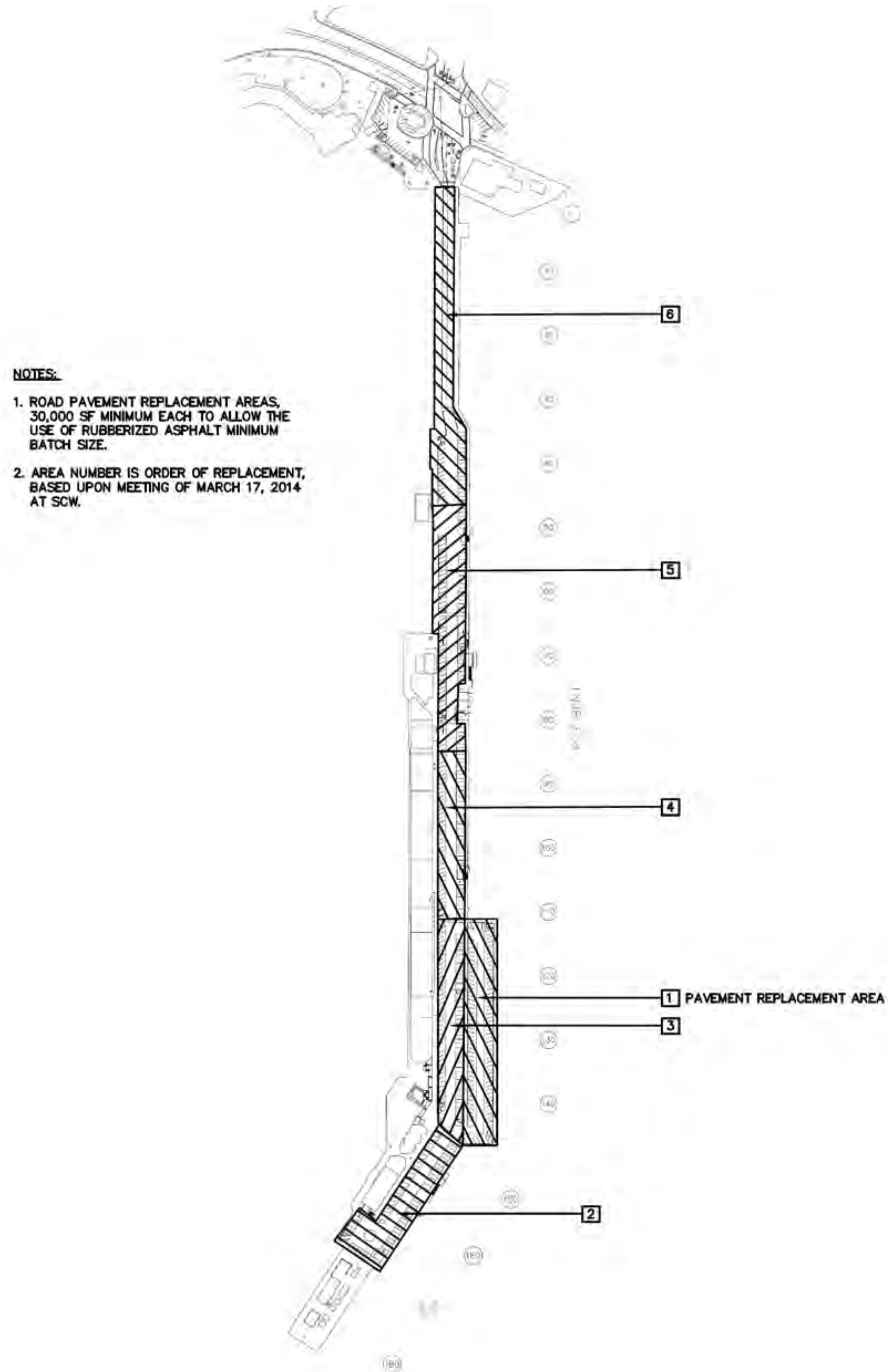


Figure 3-7: Pavement Replacement Schedule of Areas

3.5 Conclusions and Recommendations

1. Pavement should be replaced across the entire wharf.

    - Install a test section to determine most effective asphalt system (plywood under layer Alt 2 or grid reinforcement Alt 2.1)
    - Install selected system (Alt 2 or 2.1) with waterproofing layer between AC and deck boards to minimize cracking.
    - Alternate deck board joint locations
    - Consider rubberized asphalt in place of conventional asphalt.
    - Install by phases in the areas shown on Figure 3-6

2. Install drain inlets in vehicle area to treat runoff with media filtration to address water quality

3. Pursue grant funding for water quality improvement associated with the repavement of the Wharf

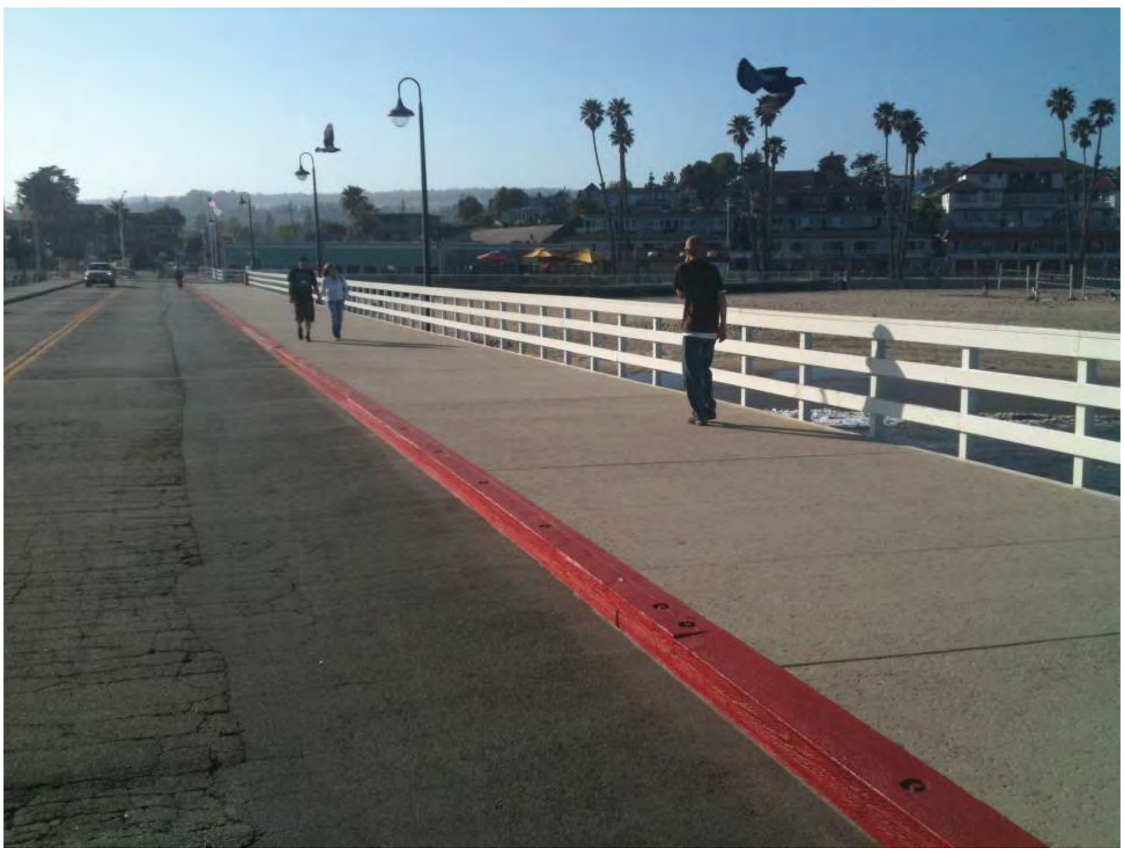
4. Limit truck traffic to the greatest extent possible to minimize damage. This may include the following:

    - Replace garbage collection system with onshore vacuum collection to eliminate garbage truck traffic onto wharf
    - Consider use of smaller, light weight collection vehicles
    - Designate smallest effective Fire Department truck to go onto wharf
    - Require delivery trucks of smallest practical size, or require that deliveries be made at the end of the route when the truck weight is presumably at a minimum.

4. WALKWAYS & PUBLIC COMMON AREAS

4.1 Summary

The walkway areas on Santa Cruz Wharf are in fair to good condition and have sufficient capacity to support the imposed pedestrian load although there are areas of deterioration in the substructure. The walkways are asphalt, concrete or a mixture of the two. The walkways are less able to accommodate pedestrians on the main portion of the Wharf in as they could be. The companion Master Plan recommends to widen the east walkway (East Promenade) and provide a walkway over the water on the west side of the Wharf buildings. These were analyzed as part of the engineering study and are feasible to construct.



4.2 Introduction

4.2.1 Scope

- Conduct an assessment of the existing weight-bearing capacities and structural integrity of the pavement and substrate for the pedestrian walkways and Commons areas.
- Identify weak and vulnerable areas.
- Evaluate widening of the east pedestrian walkway and the south end public area; consider cantilevered walkways along the west side of the Wharf and provide generally applicable construction format and plan drawings.
- Evaluate covering all pedestrian walkways and public use areas with stamped concrete.
- Provide options and recommendations regarding surface coating materials, material applications, and maintenance.

4.2.2 Description of Structure

Walkway widths and surfacing vary along Santa Cruz Wharf. The widest walkways are on the trestle portion at the beginning of the Wharf. The west side is 16 feet wide and has an asphalt surface (see Photo 4-1). The east side is 13 feet wide and has a stamped concrete surface (see Photo 4-2). South of the trestle, the east walkway narrows to 6 ft. wide with an asphalt surface and the west walkway narrows to 13 ft. and continues in front of the buildings (see Photo 4-3). In front of the buildings the walkway surfacing is predominantly asphalt. It is concrete in some locations, which was the original sidewalk constructed in the 1950s. There are also concrete ramps on the sidewalk to accommodate businesses with a floor raised above the sidewalk elevation.

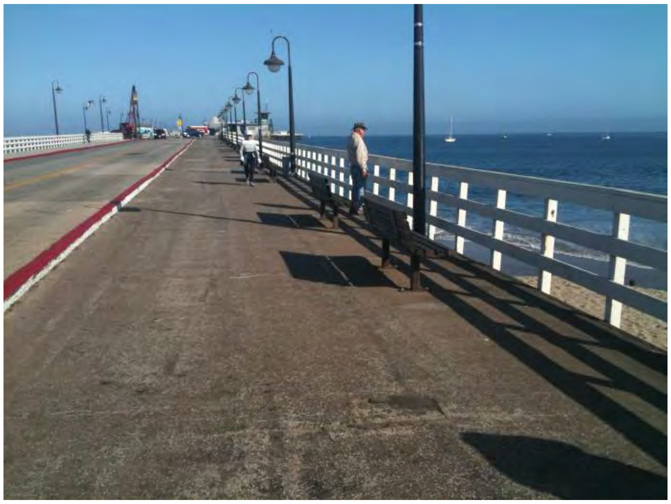


Photo 4-1: Walkway on West Side of Trestle-16 ft width, Asphalt

Photo 4-2: Walkway on East Side of Trestle- 13 ft. Width, Stamped Concrete

Photo 4-3: Walkway at Buildings

All of the walkways are supported by the typical wharf timber structure: asphalt or concrete surface on top of 3x12 timber decking, 4 x 12 timber stringers and 12x12 cap beams. Photo 4-4 shows a section through the existing sidewalk in front of the buildings.

Photo 4-4: Existing Walkway Section at Access Portal

4.3 Condition Assessment

The walkway surface condition was observed from the top of the deck. The supporting substructure was observed from below by boat and the access portals.

Surfacing

The walkway on the west side of the trestle is asphalt and on the east side is recently constructed stamped concrete. The surfacing in front of the buildings is mixed asphalt and concrete. There are a number of ramps and other structures that protrude into this walkway. The walkway surfacing is in good condition along most of the Wharf. Cracks in the surfacing are minor and there are no significant areas of unravelling of the pavement. A notable exception is the south Commons area that has reflective cracks in the asphalt surfacing (see Photo 4-5). These cracks are visually apparent but do not present a significant trip hazard or reduce the function of the walkway.

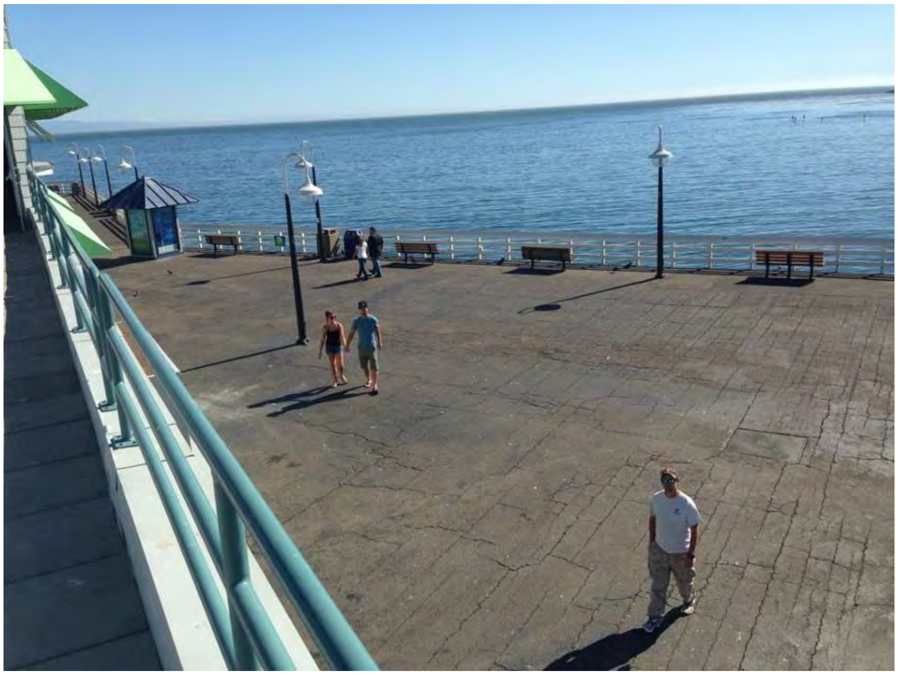


Photo 4-5: Pavement on South Commons

Substructure

The supporting substructure is in fair to good condition and adequate to support the walkway loads. Areas that have deterioration were identified and are presented in Section 2 of the report. Figure 2-2 shows an overview of these areas and Attachment 2B shows detailed locations. There is significant deterioration in some of the stringers and the pile caps beneath the sidewalk, in front of the buildings. This area is the joint between the original 1914 wharf construction and later additions after the 1950s, (see Figure 1-2). The east side of this joint is the edge of the original wharf that was exposed to the weather. It now shows softening and rot in locations. Photo 4-3 shows a patch in this location where the substructure was exposed for repair.

4.4 Analysis

Existing

The walkway areas on Santa Cruz Wharf must support public pedestrian loading (100 psf). The capacity of the Wharf structure to support this load was analyzed in Section 2, and it is found that the Wharf has reserve capacity to support this load (100 psf load Case-1, see calculations in Attachment 2C). The walkways have sufficient capacity to support the imposed load although there are areas of deterioration in the substructure.

The existing pedestrian sidewalk on the west side of the Wharf runs in front of the existing retail/restaurant buildings and on the South Commons Plaza. It is presently a mix of AC and concrete pavement over wood deck. In these areas there are advantages to the use of concrete paving instead of open wood for its ability to be swept and washed in a high traffic area. The walkway can be sloped to drain toward the curb in front of the buildings (see Figure 4-1)

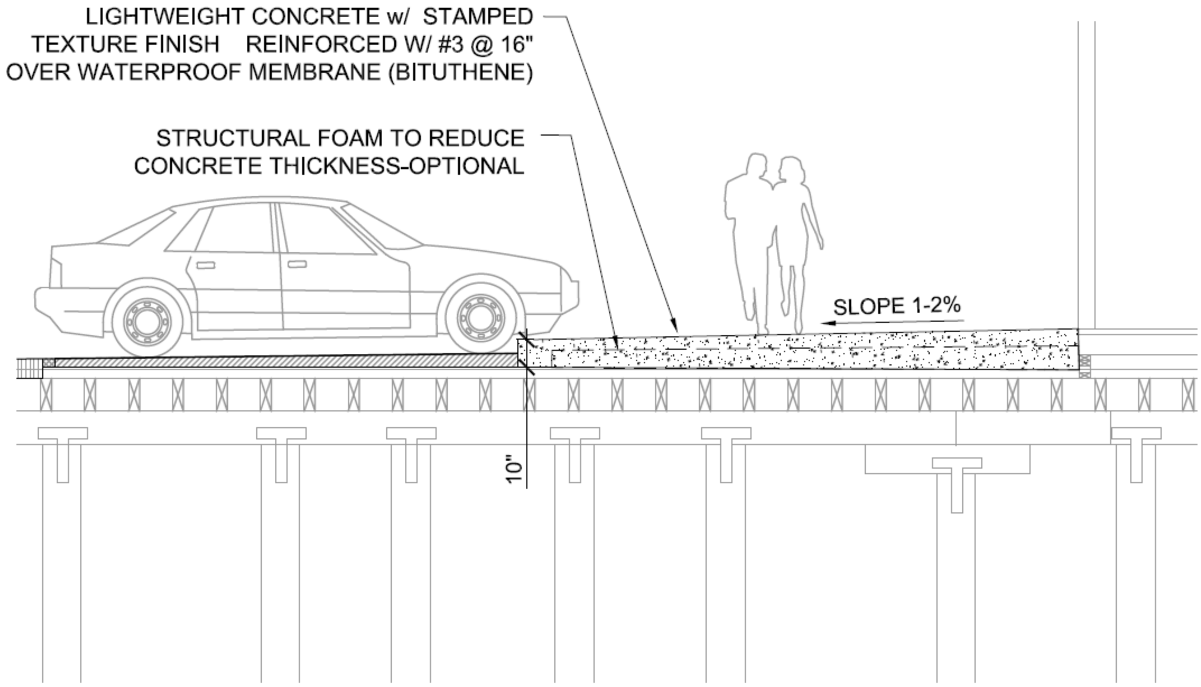


Figure 4-1 Sidewalk Section at Buildings

Master Plan

Because the walkways narrow along the Wharf, they are less able to accommodate pedestrians on the main portion of the Wharf beyond the trestle. The companion Master Plan studied pedestrian circulation on the Wharf and developed methods to improve pedestrian movement and experience along the length of the Wharf. The engineering study is performed in concert with the Master Plan and analyzes the engineering considerations to the Wharf structure for these improvements, presented in the following pages. The resulting recommendations of this study are to widen the east walkway (East Promenade, see Figure 4-2 below and page 11 of the Master Plan) and provide a walkway over the water with the view to Lighthouse Point on the west side of the Wharf buildings (see Figure 4-3).

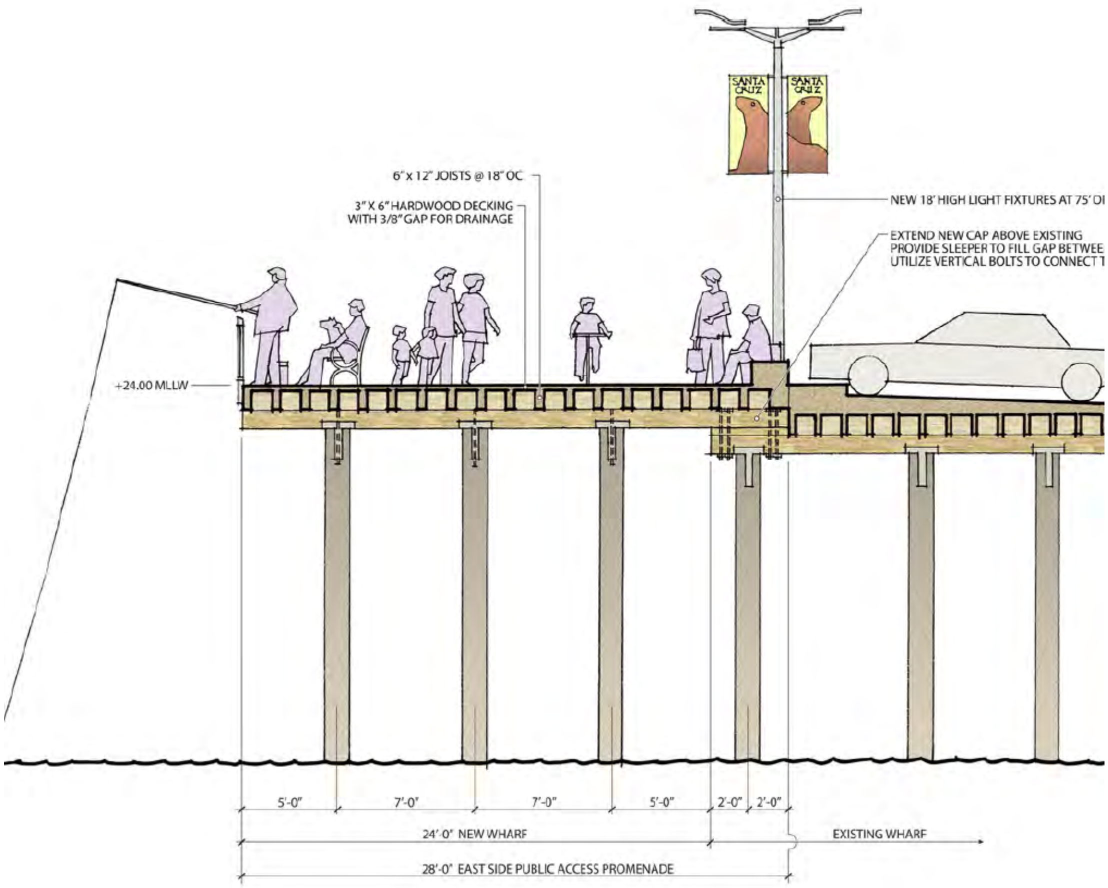


Figure 4-2 East Promenade (from Master Plan)

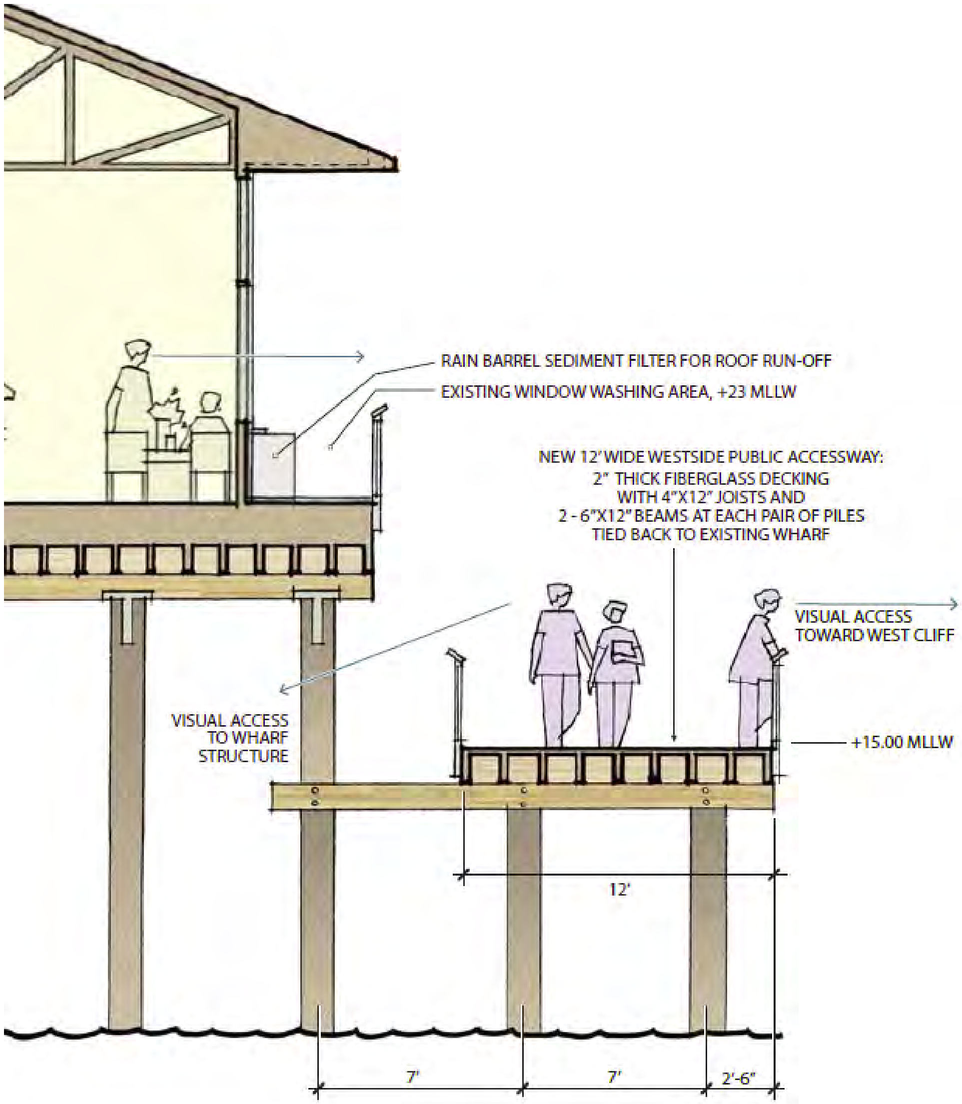


Figure 4-3 West Promenade (from Master Plan)

East Promenade

The East promenade structure is designed of similar material and configuration to the existing timber wharf. This provides elastic compatibility and additional lateral strength to the Wharf to withstand wave and other lateral loads. The Wharf structure acts as a cantilever structure (piles) connected with a diaphragm (deck). By adding piles to the width of the Wharf this increases the stiffness, and reduces deflections and stress in the existing piles. The East Promenade is designed to support pedestrian loading, and in addition, to support emergency vehicles as large as the fire trucks in use by the Santa Cruz Fire Department. This allows it to serve as a secondary means of access should the main roadway become blocked during a fire or other emergency.

West Promenade

The West Promenade (walkway deck) will be lower than the main deck and therefore more exposed to large waves from the prevailing westerly direction. The west side of the Wharf has experienced greater damage to the piles particularly from logs and other large flotsam during large wave storms. Although the west walkway will have greater exposure and resulting damage by wave forces, by absorbing these forces it will also lessen the impact to the existing piles beneath the buildings where it is far more difficult to replace piles. In addition, the walkway will serve to dissipate cresting wave energy that might otherwise strike the existing building walls and picture windows in the restaurants. This will provide increasing benefit as sea level rises. The structure will be steel to provide ductility to wave and object impact, and will tie into the existing wharf to provide added strength and lateral stiffness. The deck material will be fiberglass grating to allow air and water movement through it and reduce wave uplift forces.

Surface Material

Walkway surface materials were evaluated for application on the existing walkways and the new East and West Promenades. The engineering evaluation finds that timber decking (hardwood) and concrete (non-structural) topping are the most suitable materials for the Wharf main deck.

For the East Promenade, the Master Plan favors the use of open wood deck as it offers a rich and distinctive aesthetic that connects the user to the ocean and to the historic fabric of the original Santa Cruz Wharf. At the Santa Monica Pier, a concrete structure, the decision was made to use wood decking over the concrete slab for the marketing and aesthetic appeal that it offered to the visitor. Other advantages are that repair of an open wood deck system will be simpler/cheaper than any similar repair to a concrete topping deck system; stains can be removed from wood by sanding the surface while stains in concrete may be more difficult to remove without the use of chemical cleaners or sandblasting. Timber is considered flammable, but this can be addressed by use of hardwood that will not burn unless a sustained heat source is applied (e.g. a blow torch) and once the heat source is removed the fire will go out.

Alternatively, a concrete walkway surface could be created using a concrete topping system similar to the existing east side of the widened trestle. This will require a stiffer substructure to withstand emergency vehicle loads. The system would be about 27” deep and consist of a 3 in. concrete topping over a 6 in. nail laminated solid wood deck supported by 18 in. deep stringers in turn supported by the pile caps. The concrete topping should be “soft cut” into 2 ft. squares and have contraction joints every 10 ft. While such a concrete surface would have certain advantages in terms of durability, it may be more complicated and expensive to get through the topping and nail-laminated deck to repair the substructure than an open wood deck system. While concrete does offer a canvas for alternative textures, colors and other surface treatments, when repairs are needed, it is difficult to match custom concrete surface treatments and often results in a patchwork appearance. The costs of an open wood deck and a concrete topping are similar.

4.5 Conclusions and Recommendations

- Expose and replace rotted timber below the side walk as part of the sidewalk replacement
- Provide the walkway structure as set out in the companion Master Plan
    - East Promenade - Hardwood decking on timber substructure
    - Sidewalk in front of buildings and south Commons - Stamped concrete
    - West Promenade - Fiberglass Grate on Steel framing

5. GRAVITY SANITARY SEWAGE SYSTEM

5.1 Summary

All sewage on Santa Cruz Wharf is collected by gravity pipes to two large pump stations on the Wharf. These stations pump sewage to the municipal collection system on shore. These pipes are constructed of corrosion resistant PVC and ABS material and are in good condition. Due to the proximity of the Wharf to public beaches it is paramount to ensure that there are no leaks in the sanitary piping. The system should be regularly inspected, as it is currently. Pipe laterals should be pressure tested when installed and some plain steel hanger should be replaced.

5.2 Introduction

The sanitary sewage system on the Wharf conveys the waste to the shore where it connects to the Citywide sewage system. It is unknown when the original piped sewage collection system was installed on the Wharf but it was likely sometime after original construction in 1914. When retail buildings were added, the existing gravity main collectors for the system were installed in 1986 (based upon drawings received from the City of Santa Cruz).

5.2.1 Scope

- Conduct an assessment of the condition of the gravity sanitary sewage system. This would include main and lateral pipelines, cleanouts, hangers and supporting systems for lines and pump station tanks.
- Provide longevity estimates of components inspected.
- Identify deficiencies and recommend repair and upgrade requirements to support both existing and potential new uses.

5.2.2 Description of System

The sewage collection system on the Santa Cruz Wharf has four major components:

- Lateral pipelines
- Main pipelines
- Pump station
- Force main

The lateral pipelines (laterals) collect and convey waste from the plumbing fixtures (toilets, sinks, etc.) used on the Wharf to the main pipelines (mains) which run north and south beneath the sidewalk on the Wharf. The laterals and mains are sloped to convey liquid waste to flow by force of gravity (“gravity lines”) to the collection point in the holding tank beneath the pump station. There are two main pump stations on the Wharf. These are duplex pump stations (one main pump and a standby pump for large flow and redundancy) that pump under pressure to the shore via a pipeline (force main) that discharges into the City sewer system for conveyance to the treatment plan. A plan of the system is presented in Figure 5-1.

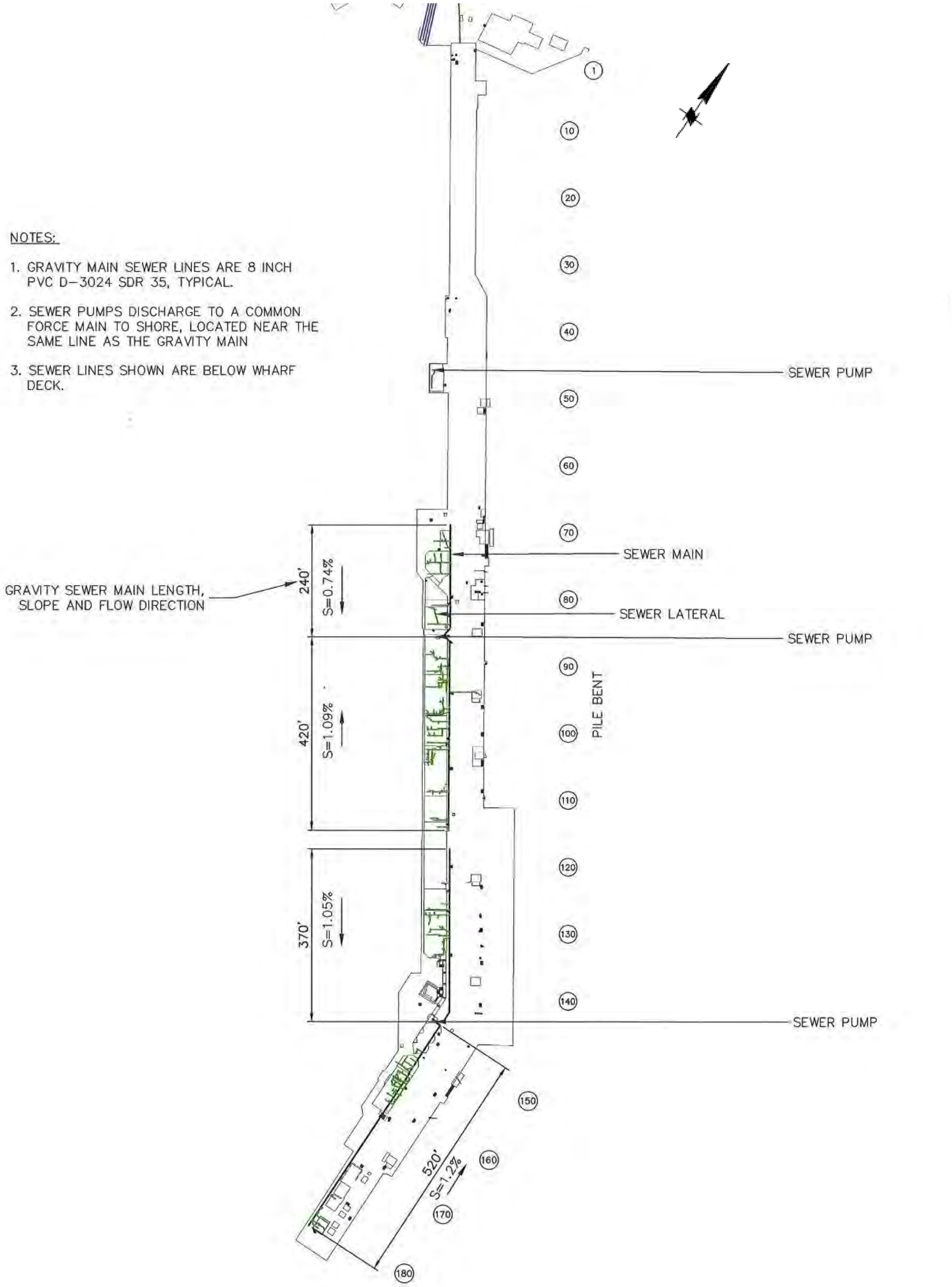


Figure 5-1: Gravity Sanitary Sewer System

With few exceptions, the laterals beneath the businesses are made of ABS pipe using solvent weld joints. The sizes of the lateral pipes vary from 2 – 6 inches depending upon the fixture and the number of fixtures that connect. Fixtures include sinks, toilets, floor drains that connect to the main gravity lines. The gravity mains are PVC with rubber gaskets (ASTM 3034) and joints. The sewer laterals are generally installed by the Wharf tenants. All the gravity mains and beyond (pump stations and force main) are maintained by the City.

The sewer collection system beneath the Wharf is regularly inspected by Wharf Staff for leaks and damage.

5.3 Condition Assessment

On-site observations were made of the gravity sewer system from a boat on March 18-21, 2014. In addition, general observations were made of the system during the Dive inspection of the piles from September 17, 2013 through October 3, 2013. Observations were made for the condition of the pipe, the pipe supports (“hangers”) and to detect any leaks that may be occurring from the pipes. Since all of the existing laterals are solvent welded pipe, and if the joints are made properly, a solid corrosion resistant seal is made with the ABS pipe. No leaks were observed from the pipe joints during these observations. At one location leaks were observed coming from the floor at one of the restaurants. This is most likely from where a floor drain in the kitchen had been damaged from above. This problem has been since remedied.

Leakage from the sewer pipes have been a high priority for the Wharf staff to correct especially due to the location of Santa Cruz Wharf next to popular beach locations and the nearby Monterey Bay Sanctuary.

The hangers that support the pipes are varying types of steel. The predominant type is hot-dipped galvanized. Other types are stainless steel, which is very corrosion resistant, and mild steel that corrodes readily. The hangers that are on the gravity main are in good condition and are all hot-dip galvanized. These were installed to resist seismic forces as prescribed in the building code at the time of construction. The piping supports for the laterals have varied materials and spacings.

Cleanouts

There are numerous cleanouts on the gravity mains to allow clearing of blockages along the entire length. There are not as many cleanouts on the lateral lines, as most blockages would be cleared by snaking the lateral line from the fixture above. There are some cleanouts at the upper end of the laterals (terminal clean out).

Photo 5-1: Sewer Lateral at bent 92 and Abandoned Cast Iron Line

Photo 5-2: Lateral line at Bent 128, New Construction

Photo 5-3: Cleanout at High End of Gravity Line

Photo 5-4: Pump at Lifeguard

Photo 5-5:Sewer Pump Station at Bent 86

Abandoned lines

In many locations beneath the Wharf there are remaining pipelines that were abandoned when new pipes were installed. In a few locations there are older cast-iron pipes and utility lines (PVC and steel water lines, PVC electric conduit, etc.) that have been left in place and abandoned. This makes it confusing to identify which lines are active and which lines or not.

5.4 Analysis

An analysis is performed to determine the flow capacity of the existing sewage system. An estimate of the total demand on the system is made by surveying the fixtures (sinks toilets, drains) in building spaces and applying a weighted formula that assigns the flow contribution from each type of fixture. This method is prescribed in the plumbing code that prescribes a set fixture unit number for each fixture type according to the relative flow it discharges to the sewer drain. For example, a drinking fountain has a relatively low fixture unit of 0.5 and a valve-operated (no water tank) toilet has a value of 8. Using this method, the analysis shows that the existing gravity mains are well within their capacity carrying about 50% of the total possible flow rate that they could carry.

In discussions with wharf staff, there have been no problems or indications of overload in the sewage system. The main problems come from kitchen grease deposits on the pipe wall due to improper disposal in the sewer system.

Given the present age and condition of the existing gravity mains, it is estimated they have at least 25 years of serviceable life remaining. The age and condition of the laterals is more varied and a single value cannot be assigned. The ABS laterals, installed within the past 10 years are estimated to have a 20 year minimum remaining service life. The gravity sewer lines are subject to wave impact during large storms because they are designed to slope and have a lower elevation under the wharf deck. All of these lines may experience damage within the remaining service life that would require repair.

5.5 Conclusions and Recommendations

1. Remove all abandoned piping beneath the Wharf.
2. Replace all mild steel hangers with hot-dipped galvanized or stainless steel.
3. Require pressure tests performed on all new laterals installed on the Wharf.
4. Perform monthly inspections for leaks of the sewage system for leaks along the entire Wharf during times of highest demand – typically 11 AM – 2 PM.
5. Perform inspections for leaks of the sewage system within 3 days (wave conditions permitting) after major wave storm events where wave crests have been observed within 5 feet of the Wharf deck.

6. FIRE WARNING & SUPPRESSION SYSTEMS

6.1 Summary

Santa Cruz Wharf was originally constructed in 1914, and has been widened in various locations from the 1950s to the 1980s. During this time, there were no codes or standards directly applicable to fire suppression systems on piers and wharfs. Over the past 35 years, fire warning and suppression systems have been added to the Wharf as use increased and applicable codes and standards were established. Currently the Wharf is protected by a fire suppression system (see Figure 6-1) along the full length that includes hydrants, sprinklers (full coverage in buildings and partial coverage on the substructure), access hatches to the substructure, fire truck access on the Wharf and a zoned fire alarm system. The Wharf is classified as combustible construction under the fire code because it is timber construction. The existing suppression and alarm system are in overall good condition and is tested regularly.

Recommendations include increasing the coverage of the substructure sprinkler system, providing complete sprinkler coverage in future additions to the Wharf and limiting boat mooring near the Wharf to reduce risk of ignition.

6.2 Introduction

The Santa Cruz Wharf has gone through several expansions, improvements and upgrades during the 1950s to 1980s, since opening in 1914. Additions to the fire suppression (sprinklers, hydrants), warning (alarm) system, and the approximate dates of construction are summarized as follows:

- 1979 Hydrants and 10 inch water main from Bent 1-132
- 1984 “ “ “ “ Bent 132 -183 (end)
- 1986 Fire Access Portals to Substructure
- 1990 Underdeck Fire Suppression System in 1990
- 2001 Fire Alarm System.

(Based upon drawings received from the City of Santa Cruz)

6.1.1 Scope

Conduct an assessment of the existing fire warning and suppression systems and make recommendations for improvements and upgrades.

6.1.2 Description of Fire Suppression System

The following description is based upon the following drawings and site observations:

- 1990 Wharf Underdeck Fire Suppression Drawings
- 2001 First Alarm Fire System
- 2011 Wharf Utility Survey

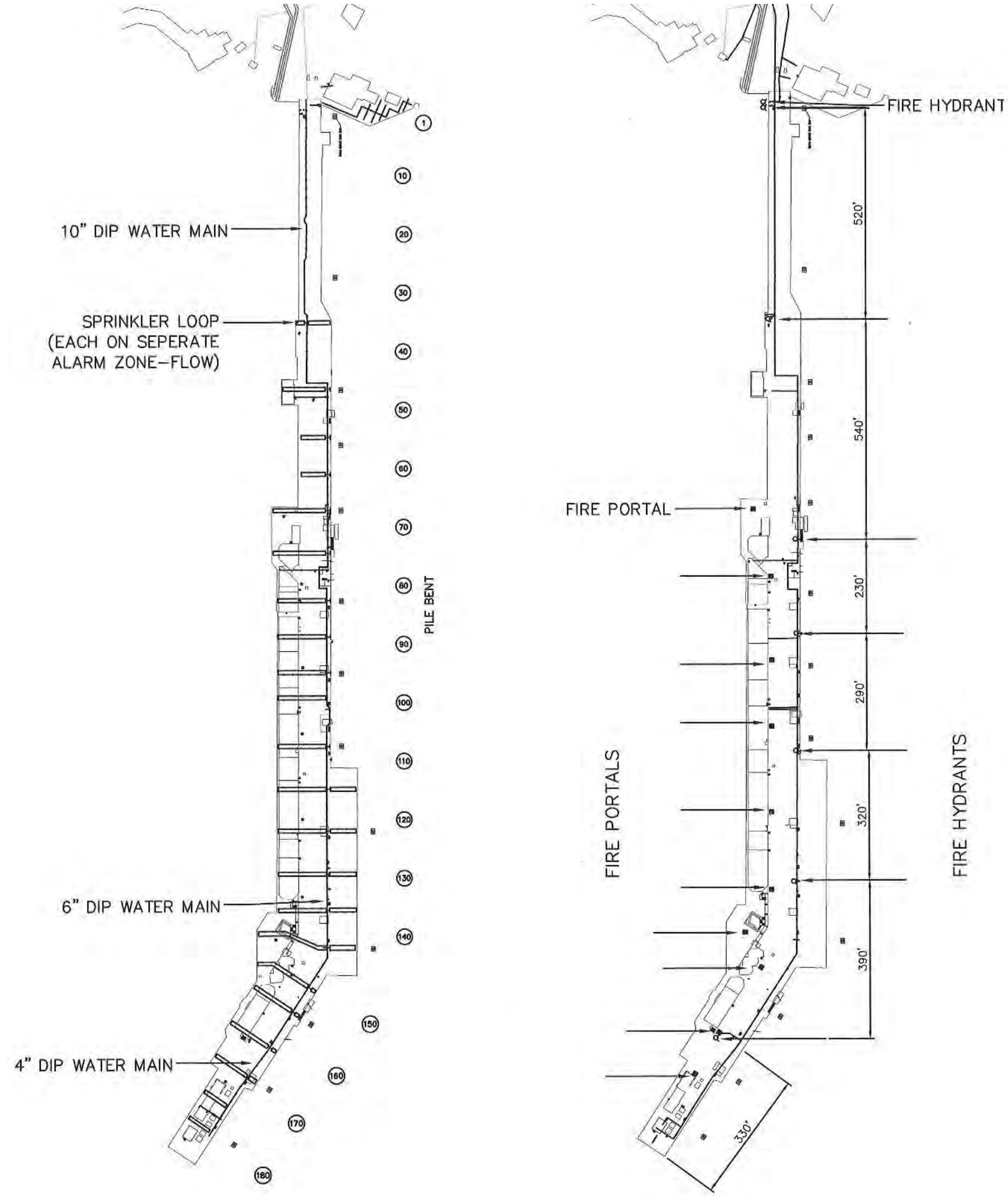


Figure 6-1: Wharf Fire Protection System

The Wharf fire protection (warning and suppression) system is designed to protect the Wharf structure, which is approximately 328,000 square feet. It is 2,700 feet long and the width varies from 55 feet to 250 feet. The supporting structure of the Wharf (substructure) is constructed of timber decking over stringers, pile caps, and 183 rows (bents) of piles 15 feet apart. The construction is classified as combustible in the applicable fire code.

Approximately 257,000 square feet of the Wharf deck is open (road and walkways) and is used for parking, emergency vehicle access, and pedestrian thoroughfare. This portion is constructed of approximately 2 inches of asphalt on timber decking.

The superstructure (buildings) consists of approximately 71,000 square feet (along a length of 1,300 feet) of Type V-B non-rated wood frame buildings. The buildings uses are a mixture of restaurant, retail, public use, and support services.

The underdeck fire suppression system with fire alarm system provides protection for the municipal water main under the Wharf that supplies fire hydrants along the Wharf and sprinkler systems under the Wharf and to buildings on the Wharf. The superstructure overhead fire suppression systems have alarm detection through sprinkler activation and structure protection throughout the buildings.

<u>Water Supply and Fire Hydrant Distribution</u>

The ductile iron municipal water main below the Wharf is 10” (see Figure 6-1 and Photo 6-1) from the point of connection in Beach Street to bent 132 where it reduces to 6” and again reduces to 4” at bent 160. The municipal water main supplies seven hydrants along the length of the Wharf near bents 2, 37, 74, 90, 109, 131, and 160. The average hydrant spacing is 380 feet with the largest spacing between bents 37 and 74 at about 640 feet. The water main supplies the wet fire sprinkler systems that protect the substructure and buildings on the Wharf. Table B105.1 of the California Fire Code requires 7,250 gpm of fire flow for the 71,000 square feet of Type V-B buildings and allows up to a 75% reduction for buildings with sprinklers (1800 gpm minimum fire flow). The Wharf has a robust water supply available. Hydrant flow test show static pressures above 110 psi with 5,800 gpm available at Beach Street and over 3,000 gpm (41% reduction) available to hydrant distribution along the Wharf.

Photo 6-1: 10 Inch Ductile Iron Water Main Near Bent 70

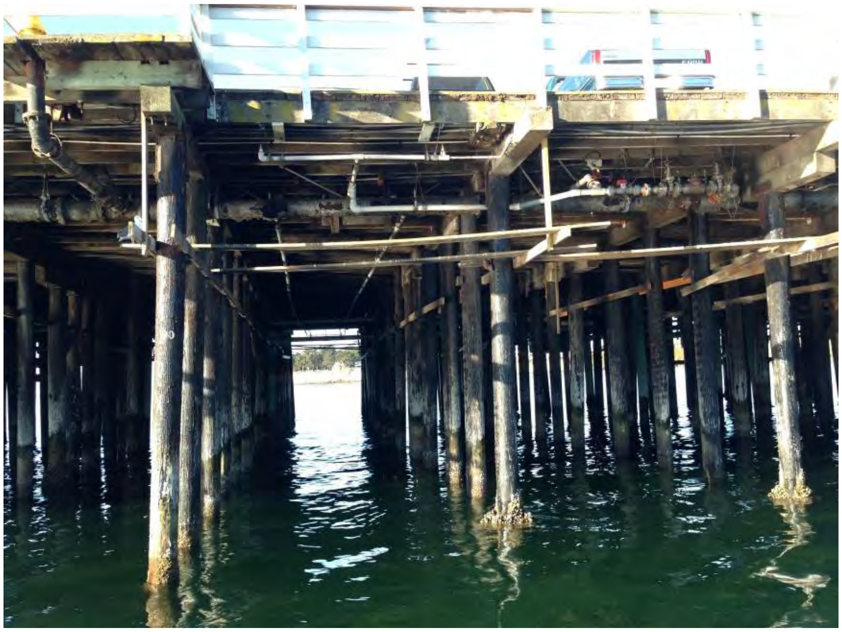


Photo 6-2: Fire Hydrant Bent 90

Blue dot hydrant markers are provided along the vehicle access route to help the fire department locate fire hydrants at night.

The underdeck (substructure) fire suppression system is isolated from the municipal water supply with double check backflow preventers. The underdeck (substructure) fire suppression system cross main parallels the municipal main and is isolated by individual double check back flow preventers along the length of the main as shown on the 1990 Wharf Underdeck Fire Suppression drawings. Fire department connections are provided downstream of the double check assemblies in accordance with the California Department of Health Standards (Title 17).

Substructure Fire Protection

There are 10 access portals (hatches) on the Wharf deck located in the sidewalk, near the center of the Wharf, that provide access to a landing directly below (see Photo 6-3 and Photo 6-4). This allows firefighters access to fight fire in the substructure from the platform below or to lower a fire nozzle below to fight fire. Access portals are numbered, but fire department markers were not evident.

Photo 6-3: Fire Access Portal Platform at Bent 80

Photo 6-4: Fire Access Portal Bent 120

In addition to the access portals sprinklers, the substructure is protected by a wet sprinkler system below the Wharf. The sprinkler coverage is provided beneath 22 bents: 38, 48, 56, 62, 68, 75, 83, 89, 98, 101, 107,114, 121, 127, 134, 140, 146, 152, 159, 165, 171 and 176. Sprinkler coverage is partial and is intended to form a break or fire-stop at these bents. The sprinkler system at these bents consists of two CPVC branch lines which extend across the width of the Wharf (see Photo 6-5 through Photo 6-7). The zone branch lines are supplied by a 3 inch CPVC cross main that runs the length of the Wharf parallel to the ductile iron municipal main. The sprinklers are spaced at 8 feet along the branch lines with branch line spacing at 8 feet resulting in a head coverage area of 64 square feet. The largest zones occur near bents 121 and 134, each have 50 heads covering 3,200 square feet.

Sprinklers are also along the length of the cross main with 8 foot spacing in an effort to protect the CVPC cross main and the ductile iron municipal main alongside it. The sprinkler heads on the 1990 record drawings are listed as Viking Microfast Model M quick response, standard spray, ordinary temperature, Teflon coated, ½" orifice heads with pendent deflector. The pendent heads are shown installed in the upright position with the top of deflector located a maximum of 6 inches below the bottom of the 4x12 stringers and 18" below 3x12 deck planking.

Photo 6-5: Bent 62 Sprinklers

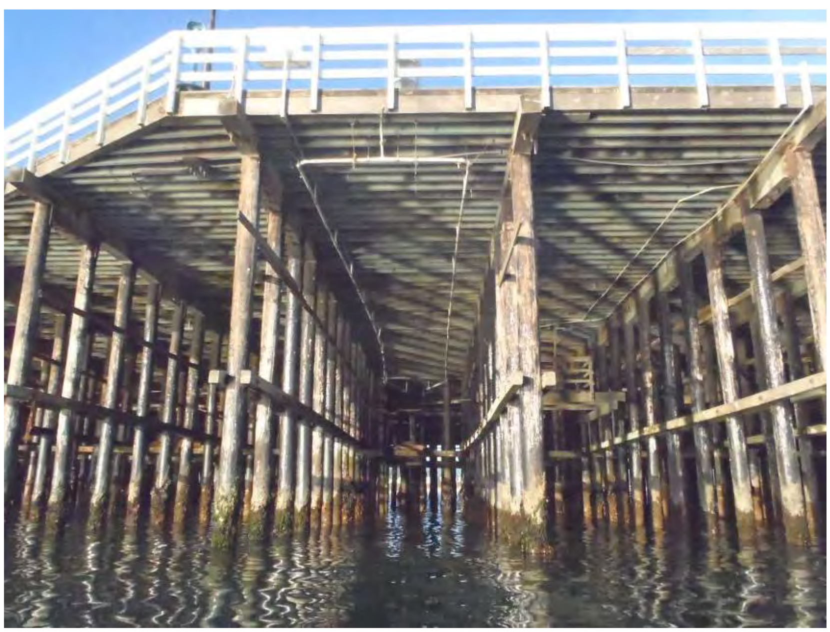


Photo 6-6: Bent 138 Sprinklers

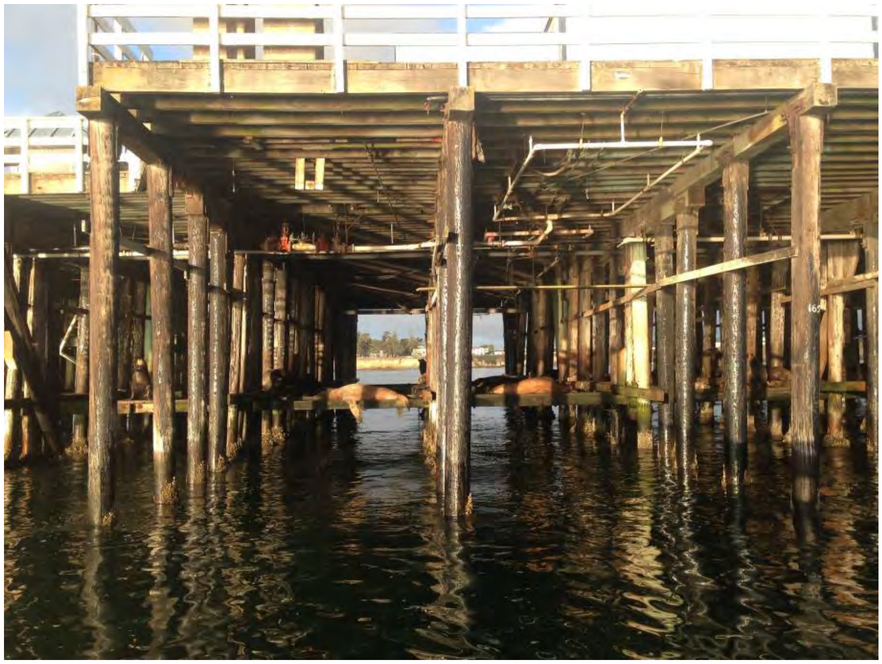


Photo 6-7: Sprinkler and BFP Bent 165

Superstructure Fire Protection

The superstructure or buildings above the Wharf are protected by overhead sprinklers systems. The separate tenant spaces have dedicated wet sprinkler risers complete with double check backflow preventer, flow switch, alarm bell, and fire department connection. The wet sprinkler risers and fire department connections are conspicuously located with alarm bell and signs (see Photo 6-8). Building fire sprinkler shop drawings were not reviewed.

Fire Alarm System

The building and under deck fire suppression systems are monitored by a central station fire alarm system. The FACP (fire alarm control panel) is located in the Agora building utility room, near bent 70. A smoke detector and manual pull switch are provided at the FACP. The FACP monitors 24 water flow switches serving the underdeck fire suppressions system, 11 water flow switches serving the superstructure wet sprinkler risers, and associated tamper switches.

The under deck fire suppression system provides alarm zone detection through sprinkler activation at fire suppression zones located at an average zone spacing of 225 feet along the length of the Wharf. The 22 sprinkler fire breaks, located at an average spacing of 90 feet along the Wharf, effectively protect 47,340 square feet or 14% of the total 328,000 square foot under deck area.

Photo 6-8: Typical Riser, Fire Department Connection, Bell, and Signage.

<u>Fire Boat and Emergency Response Plans</u>

In addition to the fire suppression system, the City of Santa Cruz operates a fireboat to respond to Wharf fires and has a number of established response plans in place to respond to fire and other public safety emergencies on the Wharf.

The City of Santa Cruz Fire Department has recently placed a 34 foot Willard fireboat into service. The boat has a roof-mounted firefighting turret nozzle supplied by a 500 gallon per minute capacity pump. The boat is moored in the Santa Cruz Yacht Harbor with the primary focus to protect the Wharf and vessels in the harbor area.

The City has the following Response Plans (Standard Operating Procedures-SOP) in place for Wharf emergencies, which are summarized (full text contained in attachment 5-A):

SOP 1-3.5 Unit 771/778 Utilization—Establishes unit (fire truck) 771 to respond to Wharf fires.

SOP 2-7.0 Establishes firefighting response plan on the Wharf and deployment of the fire boat (per Standard Operating Guideline #5)

SOP 2-7.1 Establishes guidelines for use of truck 3170 for medical response to the Wharf

SOP 2-7.2 Establishes procedures for evacuation of the Wharf in emergencies of fire, tsunami, high sea, damage, bomb threat, protests, and crime.

SOP 3-1.0 Establishes guidelines for medical response to the Wharf

6.1.3 Methodology

As noted previously, the Wharf was constructed when the contemporary building codes did not have provisions for piers and wharves. The applicable code that developed and is in use is the National Fire Protection Code (NFPA), which contains provisions and exceptions for existing wharves at the time of adoption. Further, the NFPA code provides for the determination on many provisions by the Authority Having Jurisdiction (AHJ), in this case the City of Santa Cruz Fire Marshall. In analyzing the existing fire protection system, the 1990 NFPA code is used, as it is closest to the time that the majority of systems had been installed. For future additions to the Wharf the current code is used.

Current Code

The current adopted code for the City of Santa Cruz is the 2013 California Fire Code as amended by Title 19 of the City of Santa Cruz Municipal Code.

The 2013 California Fire Code references the following applicable standards:

2013 NFPA13 Installation of Sprinkler Systems
2013 NFPA 24 Installation of Private Fire Service Mains and Their Appurtenances

2013 NFPA72 National Fire Alarm and Signaling Code

The most current NFPA code available that addresses wharf construction is:

2011 NFPA307 Standard for the Construction and Fire Protection of Marine Terminals, Piers, and Wharves

1990 Code

Applicable code at or near time of wharf fire suppression improvements:

1988 Uniform Fire Code
1989 NFPA13 Installation of Sprinkler Systems
1990 NFPA307 Construction and Fire Protection of Marine Terminals, Piers, and Wharves
1999 NFPA72 National Fire Alarm and Signaling Code

Any changes to the Wharf fire suppression systems need to be submitted the local fire marshal or AHJ is Eric Aasen at the City of Santa Cruz fire prevention office located at 230 Walnut Avenue.

6.3 Condition Assessment

Site visits were performed by Axiom Engineers (Steve Rawson, PE) and Moffatt & Nichol (Brad Porter, PE) on April 28 and May 14, 2014, accompanied by Wharf staff, to observe the condition of the suppression system. The underdeck portion was observed from a boat and access portals and the above deck system from the top of the Wharf. In addition, the testing records of the system were reviewed. Testing is performed regularly on the system by a testing firm. Discussions were conducted with Wharf and City Fire Department staff to get further information on the background and operation of the fire suppression system.

Based upon these observations, the fire suppression system is in good condition and fully functional.

6.4 Analysis

Existing System

The 1990 NFPA 307 (section references shown in parenthesis) code provides that the code is not intended to be applied to existing structures prior to code adoption unless the AHJ determines there is a distinct hazard to life or property or unless the code specifically refers to existing structures (chap 1-2). This is applicable to Santa Cruz Wharf and the basis of analysis.

A complete automatic sprinkler system is required to protect all combustible substructures (3-3.3.1), but this requirement may be waived for existing substructures that have “Other Extinguishing Facilities” (3-3.3.5), where the “AHJ deems installation or maintenance of an automatic sprinkler system clearly impractical” and provides the following:

- Deck Openings that permit the use of rotating nozzles and other firefighting devices (3-3.3.4) in conjunction with :
    - Structural Barriers (3-3.3.6 through 3-3.3.9)
        - Fire walls (3-3.3.6, .7)
        - Fire stops (3-3.3.8)

Further, the code (3-3.3.5) provides that “Consideration shall be given to any built-in extinguishing equipment that is practical to install and maintain, such as partial automatic sprinkler equipment or manual sprinkler equipment with particular emphasis on preserving the integrity of required structural barriers under fire conditions.”

The existing wharf fire protection system utilizes deck openings (3-3.3.4) with below deck platforms that allow the fire department to protect the substructure as an alternate means for a complete automatic sprinkler system in accordance with(3-3.3.5). A partial automatic sprinkler system has been provided and is intended to act as an alternate means of a fire stop as they extend the width of the Wharf deck at the 22 bent locations.

Notwithstanding the provisions and exceptions that apply to existing wharves, the following are requirements that would apply to complete sprinkler coverage:

3-3.3 Sprinklers:

- Complete coverage
- hydraulically designed in accordance with NFPA13
- ½” orifice sprinklers
- Pendent heads in upright position not more than 18” below deck
- 80 square feet coverage area per sprinkler,
- minimum design area of 5,000 square feet.

Hydraulic design calculations, flow data, and hydraulic node data have not been made available for the 1990 Under Deck Fire Suppression System.

All buildings are to be protected by a NFPA13 automatic sprinkler system, except those that do not exceed 500 square feet with minimum separation from other structures of 30 feet. All buildings currently have full sprinkler systems in place.

3-3.3.6-Fire Stops and walls:

Requires combustible substructures be subdivided by transverse fire walls with a maximum spacing of 450 feet and transverse fire stops between fire walls with a maximum spacing of 150 feet. Fire walls or fire stops were not found during review of the record drawings.

3-3.3.3(d) Bracing:

All sprinkler piping 3 inch and larger within 50 feet parallel and within 50 of the face of wharf subject heavy fireboat nozzle streams require horizontal and vertical bracing at intervals less than 20 feet. The record drawings show existing underdeck sprinkler cross main with lateral bracing every 15 feet and at back flow preventer locations. Longitudinal bracing is not shown. Hangers on the 3” CPVC support main are provided at a 10 foot maximum spacing. There is no reference for bracing of the municipal water main. The 15 foot lateral bracing and 10 foot hanger spacing meet the intent of NFPA307.

3-3.3.3(e) Corrosion Protection:

Requires sprinkler systems to be protected throughout against corrosion. The Teflon coated heads are typically used for corrosive environments. CPVC piping is currently used under the deck and has good corrosion resistance in a marine environment. NFPA13 allows the use on CPVC piping in accordance with its listing. CPVC is listed for use exposed with sprinkler deflectors located within 8” of the ceiling, for light hazard applications, and ordinary hazard applications limited to rooms that do not exceed 400 square feet. The underdeck installation does not meet the NFPA13 requirements for use of CPVC piping. A mock fire test was performed by the Wharf staff and city fire department for this application prior to installation as approved by the AHJ.

6-1 Hydrants:

Hydrants are to be spaced no further apart than 300 feet and no more than 150 feet from a dead end. The average hydrant spacing is 380 feet with the largest spacing between bents 37 and 74 at about 540 feet. The last hydrant at bent 160 is roughly 330 feet from the dead end. Final location of hydrants and their distance from fire department connection are subject to approval of the AHJ.

New Construction

The latest edition of NFPA307 is essentially the same as the 1990 version regarding to wharf fire suppression and fire walls. Full sprinkler protection of both the new superstructure and new substructure are required be developed with the AHJ in accordance with NFPA307.

Means to provide protection by fire wall and fire stops are required to be developed with the AHJ in accordance with NFPA307 for any new substructure.

Access portals to the underside of new substructure need be provided and be compatible with the existing system.

2013 CFC 907.2.2 requires Group B (Business) occupancies to have occupant notification with sprinkler flow and manual pull stations are not required. In CFC 907.2.7 requires Group M (Mercantile) occupancies with sprinklers throughout to occupancies to have occupant notification with sprinkler flow and manual pull stations are not required. The fire suppression and alarm system would fully meet the intent of CFC 907 if the underdeck fire suppression coverage was throughout the entire deck area.

The City of Santa Cruz Title 19 section 903.2.2 requires an automatic sprinkler system for modifications to existing structures over 6000 square feet that involve a 10% increase in area, or change of use, or augmentation of an existing sprinkler system.

6.5 Conclusions and Recommendations

The existing overhead fire suppression systems and alarm systems meet the coverage and notification intent of the current California Fire Code, NFPA 13 Installation of Sprinkler Systems, NFPA 307 2011 NFPA307 Standard for the Construction and Fire Protection of Marine Terminals, Piers, and Wharves.

The following are recommended:

1. Consideration should be given to extend the coverage of under deck fire suppression system in the hazard areas of the Public Access Dock and Boat Rental. In areas where extension of coverage cannot be achieved alternative protection in accordance with NFPA307 should be developed with the authority having jurisdiction to develop a strategy to establish goals and timetables to accomplish this.

2. All expansions to the Wharf be fully sprinklered as provided in the current NFPA code. A strategy should be developed to provide sprinkler coverage in

conjunction with firewalls/ firestops throughout the expansion of the substructure with the AHJ in accordance with NFPA307.

3. Provide markers to assist the fire department to locate the access portals at night in the dark.

4. Limit anchorage to outside 200 feet of the west side of the Wharf to minimize the risk of boat collision and one of the largest potential ignition sources to the substructure.

7. WHARF STRUCTURE SUPPORTING BUILDINGS

7.1 Summary

The Wharf structure has adequate structural capacity to support one- and two-story buildings, including those identified in companion Master Plan. Future buildings on the Wharf, should follow the recommendations in this section to avoid loads from being improperly located on the Wharf structure. When buildings are replaced, the supporting wharf structure should be thoroughly inspected and all deteriorated members (decking, stringers, caps and piles) replaced. The Wharf structure beneath the building occupied by Miramar Restaurant was examined for consideration of replacement during this report.

7.2 Introduction

The Santa Cruz Wharf currently supports multiple buildings, including a life guard building and many commercial spaces. Part of the Master Plan is to evaluate the construction of new buildings or alterations to the existing ones. Preliminary calculations were performed (Mesiti-Miller Engineers) for new one- and two-story vertical building loads. These loads were then analyzed for the capacity of the Wharf substructure to support these loads.

7.2.1 Scope

- Evaluate structural integrity of substrate and identify any weak or vulnerable areas.
- Provide longevity estimates for existing substrate.
- Provide general construction recommendations for structural support of new single and two-story buildings at various points along the Wharf where the water depth varies.

7.2.2 Description of Structure

The Santa Cruz Wharf currently supports numerous buildings; most of them are along the west edge of the Wharf, starting at bent 70 to end of the Wharf. All of the buildings are either one or two stories.

7.2.3 Methodology

The condition assessment of the Wharf substructure is described in Sections 1 and 2. Vertical load calculations were performed (Mesiti-Miller Engineers) for new one- and two-story buildings, these loads are analyzed for the capacity of the Wharf substructure to support these loads.

7.3 Condition Assessment

See Sections 1 and 2 for discussion of the Wharf structural condition. In general, no deficiencies were observed that would prevent new buildings being constructed. The most deterioration of members and missing piles (with replacement A-frames) were found under the existing buildings, in particular the Miramar Restaurant.

7.4 Analysis

Loads for one- and two-story buildings, were analyzed that include floor live load, bearing wall line loads and interior column point loads and are provided in Attachment 7A. Using these loads the substructure is analyzed to determine whether the existing structure can support new building loads. These calculations can be found in Attachment 2C.

- Building columns are assumed to be on a 20 ft. by 20 ft. grid.

- A one story building is assumed to be 14 ft. in height
- A two-story building is assumed to be 24 ft. in height.

Table 7-1 presents the results regarding these locations on the existing structure. It is important to note that the pile soil axial capacities and pile buckling capacities are estimated to be approximately 20 tons. The two-story building interior column load is estimated to be 25 tons, based on 20 ft. spacing of interior columns. Considering this, future two-story buildings may require additional piles be driven (as was done at Stagnaro's) or reduced column spacing to decrease the interior column load going into the existing piles.

Table 7-1: Building Load Analysis Conclusions

Building Type and Load Type	Adequate Support of Wharf Structure?		
	4x12 Stringer	6x12 Stringer	Pile Cap
1 - Story Building			
Type of Load			
Bearing Wall	Yes, if 3+ are bundled together	Yes, if 2+ are bundled together	Yes, up to 10 ft. span
Interior Column	No	No	Yes: For 9 ft. span, it must be within 3.5 ft. of the pile centerline; for an 8 ft. span or less it can land anywhere on the cap
2 - Story Building			
Type of Load			
Bearing Wall	Yes, if 8+ are bundled together	Yes, if 4+ are bundled together	Yes, up to 10 ft. span
Interior Column	No	No	Yes: For 9 ft. span, it must be within 6 in. of the pile centerline

7.5 Conclusions and Recommendations

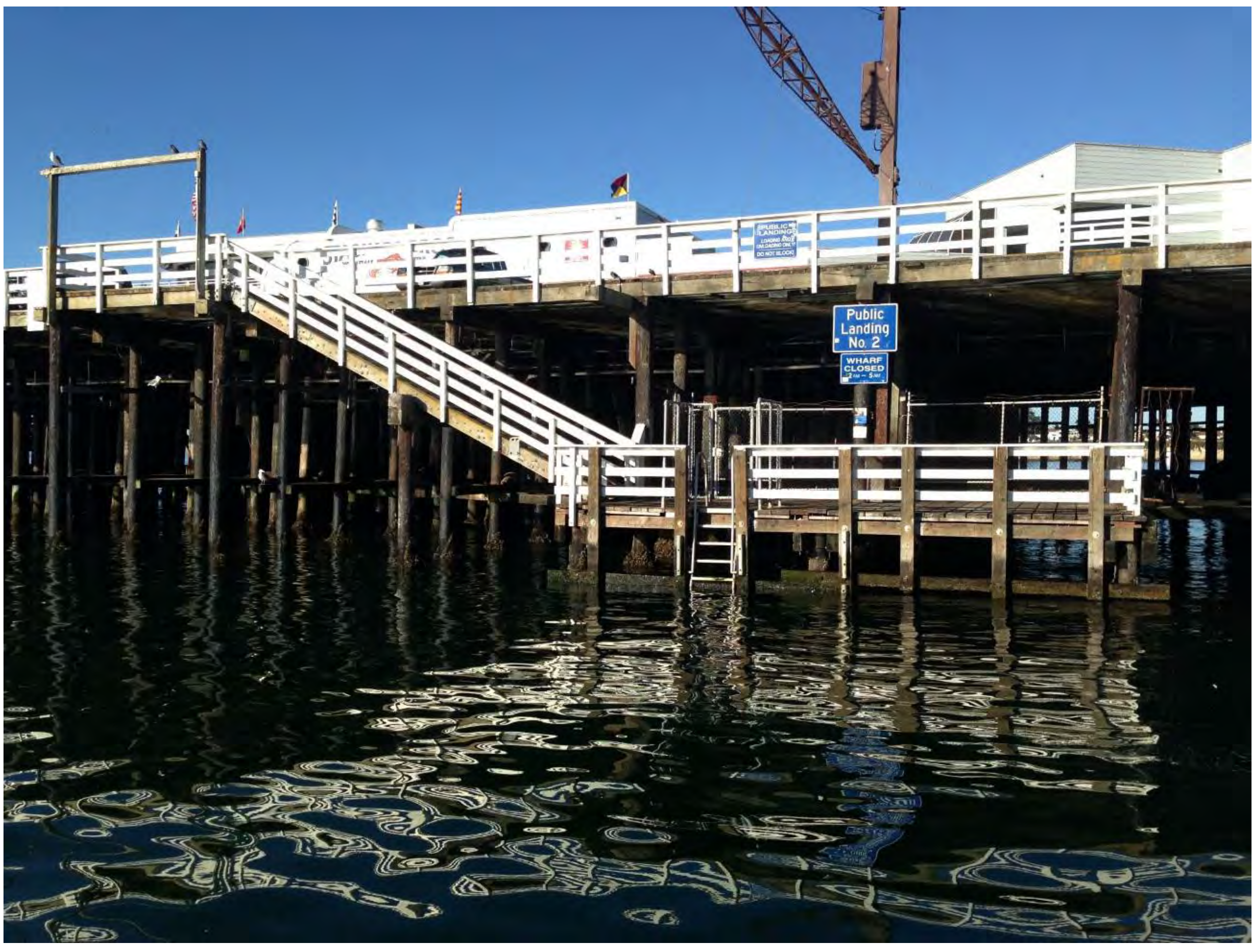
Based on the analysis, the locations of bearing wall and columns on the Wharf substructure effect whether certain structural members would be overstressed. The following recommendations are made:

1. Provide additional stringers beneath bearing wall loads
2. Locate column point loads to be directly over pile and limit point load to 40 kips
3. Install additional piles to support column point loads that are in excess of 40 kips

8. EXISTING LANDINGS & DOCK SERVICEABILITY

8.1 Summary

There are 5 active landings presently on the Wharf for boat access. Two are available to the public, two for boat and kayak rental, and the landing used by Wharf Staff. These landings are all functional but they are subject to seasonal wave damage to the inherent location. Floating docks are in use and convenient for small boats, but must be removed in winter. Fiberglass decking is recommended as an improvement to the decking material on the landings.



8.2 Introduction

8.2.1 Scope

- Conduct an assessment of the existing small craft landings and floating docks.
- Determine structural integrity, hardware conditions and service life.
- Identify recommendations for accessibility and functional ergonomics.
- Make recommendations for improving the general utility and usability of the landings and docks.

8.2.2 Description of Structure

There are five locations on the Wharf with water access to small craft (boats), provided by landings and docks. The Wharf structure at each of these locations is similar: stairway access from the Wharf deck down to a fixed platform (landing) at elevation 8 ft., MLLW. The structural members are timber with bolted connections to the piles and 3x12 timber decking nailed to the stringers. In addition to the five operating access points there is an abandoned landing near bent 103 where the Stagnaro party fishing boats have operated in the past. The existing landings are described below (going north to south).

Kayak access (Bent 52)

This landing is used by the kayak rental concession on the Wharf (Photo 8-1). There is a small floating dock accessed by a ladder from the fixed landing. There are storage shelves beneath the Wharf for kayaks and equipment. The dock is used only by patrons of the kayak rental and has a locked gate at the top of the stair when the business is closed.

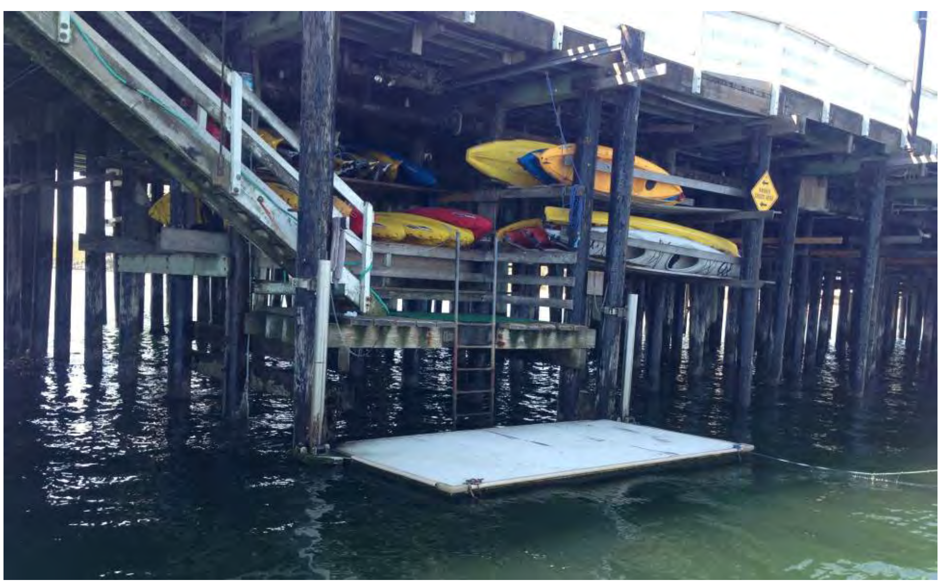


Photo 8-1: Kayak Dock

Boat Rental landing (Bent 68)

There is a small landing used by the boat rental concessionaire (Photo 8-2). Directly above the landing is a crane used to lower the wooden rental boats into the water. This facility is used solely by patrons of the boat rental concessionaire.

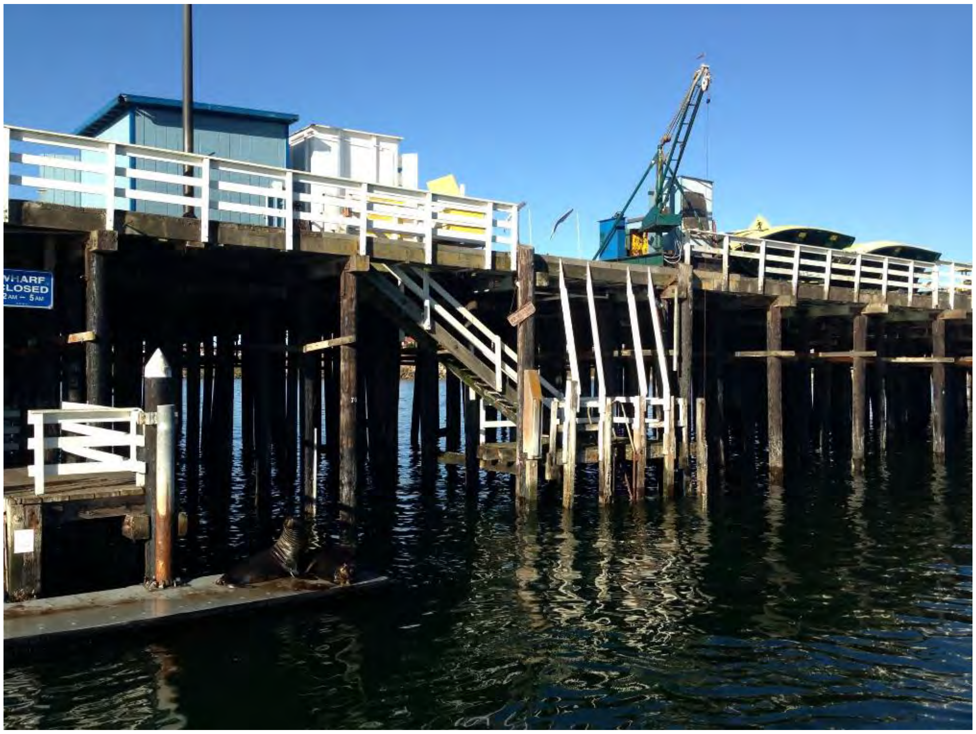


Photo 8-2: Boat Rental Hoist

Public Landing 1 (Bent 72)

At Bent 72 is a larger landing with a floating dock the length of the landing (Photo 8-3). This landing is available for public use for short-term loading and offloading to Santa Cruz Wharf.

Photo 8-3: Public Landing No. 1

Wharf Staff Landing (Bent 80)

This landing is used by the Wharf staff to launch their boats for access to the Wharf underside for maintenance and repair work (Photo 8-4). There is a 3-ton jib crane that was recently installed above to launch their boat and other municipal boats for emergencies.

Photo 8-4: Wharf Staff Landing

Stagnaro Landing (Bent 103)

There are the remnants of the landing that was used by the party fishing boat operated by Stagnaros that accessed Santa Cruz Wharf (Photo 8-5). The fixed landing is still in place but the dock was removed and is not accessible.

Photo 8-5: Stagnaros Landing (abandoned)

Public Landing 2 (Bent 150)

This is the second public and most southerly landing available to the public for short-term loading and offloading to Santa Cruz Wharf (Photo 8-6). There is no floating dock and access is by a ladder that is used at all water levels.

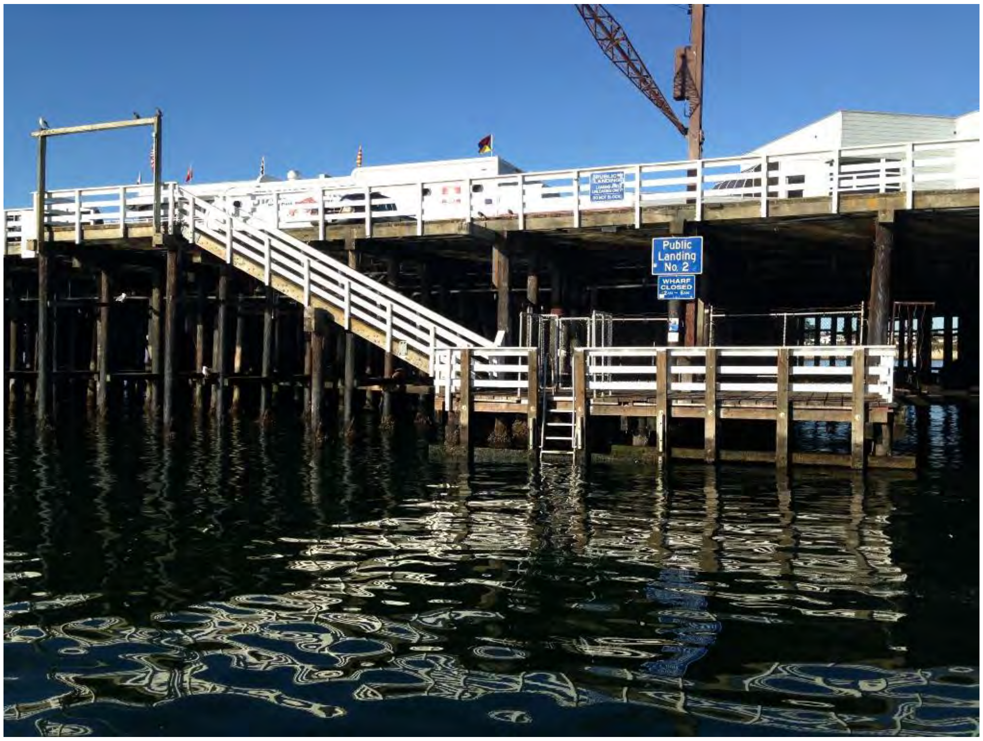


Photo 8-6: Public Landing No. 2

8.3 Condition Assessment

The floating docks at the Kayak dock and Public Landing 1 are frequently occupied by sea lions to haul out. This is both a source of enjoyment to visitors to observe and an impediment to boaters wishing to utilize the docks. Fencing or some physical barrier is required to block sea lion access onto the docks and even the landings (fixed platform). Moss Landing Harbor, nearby, has a similar if not larger sea lion population. The North Harbor guest dock that was installed within the past 10 years has become completely overtaken by sea lions. A study was performed at that facility that utilized low current to cause the sea lions to get off the dock without injury to them.

All of the fixed landings are at elevation 8 ft., MLLW, above high tide. Waves will overtop or more commonly strike the bottom of the deck boards of the landing and lift them out of place (see Photo 8-7). The floating docks are all removed in the winter to avoid damage in the large wave storms.

Photo 8-7: Wave Uplift at Existing Wharf Staff Landing

At each of these landings (Kayak, Boat Rental, Public No. 1 and No.2) the hardware used to connect structural framing are steel bolts with hot dip galvanized (HDG) coating. There is some uncoated steel, or older HDG that has lost much of the protective coating. Replacement bolts and construction that occurred within the past 10 years all have HDG bolts. Although some connection bolts that connect the stringers on the lower landings to the piles show heavy rust corrosion, none had failed or had lost more than 30% of their material.

Each of the landings, except the Wharf Staff Landing, have guard rails on the perimeter of the landing, with an opening for ladder access to boats or a float (Landing No.1). Although it was not observed during the investigation, they are often damaged during winter waves as these railings are not removable.

The landings have been configured to provide access for their present functions: kayak and boat launch, temporary boat mooring and visitor access close the water. However, all of the landings are accessible only via a stairway. This is the largest limitation for accessibility to the landings. To install an accessible ramp to one of the landings on the side of the wharf would require a length of over 200 feet run on the ramp. Providing accessible landings would require significant reconfiguration of the existing wharf to the existing landings.

Improvement to the accessibility and ergonomic function of the landings is addressed in the companion Master Plan. The reconfiguration would provide universal accessibility and a single access point for the boat and kayak rentals and a larger landing below deck. An excerpt of this plan is shown on Figure 8-8.

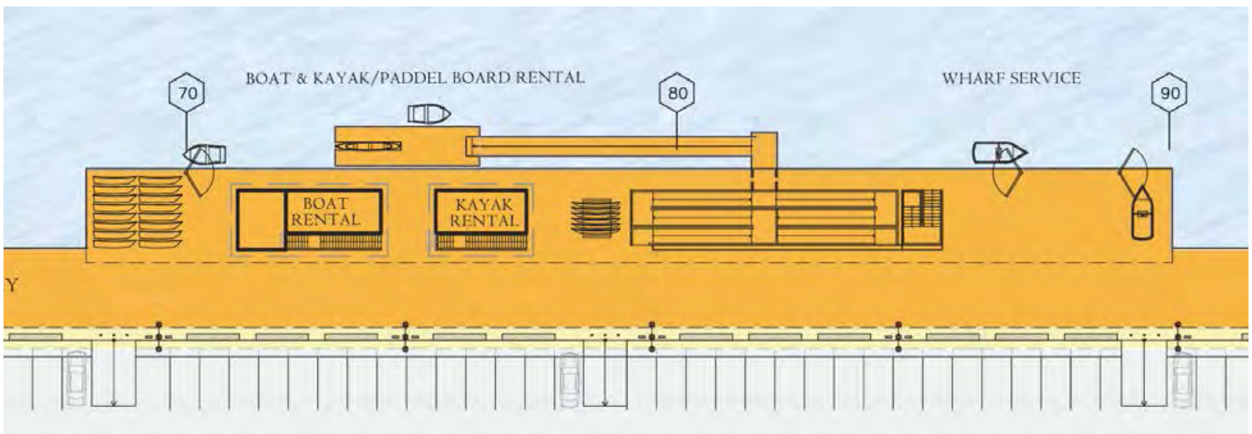


Figure 8-8 Landing Reconfiguration (from Master Plan, ROMA Design Group)

In the interim period of implementation of this plan, the existing landings should provide service for at least 10 years, provided continued maintenance is performed as in the past. The following recommendations apply to the existing landings during this interim period to maintain their serviceability.

8.4 Conclusions and Recommendations

1. Replace timber decking on the existing landings with fiberglass where practical.
2. Utilize HDG connection bolts when performing replacements to the connections, and consider 316 grade stainless steel in lower ledgers subject to wave exposure.

9. POTENTIAL FOR CONSTRUCTION OF NEW LANDING

9.1 Summary

It is very desirable to provide a landing for research and visitor-serving vessels up to approximately 120 ft. in length at Santa Cruz Wharf. To accomplish this, the Master Plan incorporates such a landing at the south east end of the Wharf. Public landing No. 2 is at this approximate location and has served smaller vessels visiting the Wharf for loading and off-loading in the past. In addition to these vessels an improved landing will add to the ability to evacuate the end of the Wharf of people in the event of an emergency.

9.2 Introduction

It is very desirable to provide a landing for research and visitor-serving vessels up to approximately 120 ft. in length at Santa Cruz Wharf. To accomplish this, the Master Plan incorporates such a landing at the south east end of the Wharf. Public landing No. 2 is at this approximate location and has served smaller vessels visiting the Wharf for loading and off-loading in the past.

9.2.1 Scope

- Conduct an assessment and make a recommendation of the optimal location for the construction of a new landing capable of handling research and visitor-serving vessels up to 100 tons.
- Describe the size, shape and type of landing to be utilized.
- Evaluate the existing Wharf structure at the proposed location for enhancements/reinforcements/revisions necessary to accompany a new landing.

9.2.2 Description of Structure

The potential location for the new landing would be on the south east end of the Wharf where the current Landing No. 2 is located. There is currently a landing that is approximately 30 ft. by 15 ft. The water depth at this location is approximately 25 ft. below MLLW.

9.2.3 Methodology

For the berthing analysis, a sample design vessel was chosen that is approximately 200 LT (long tons) in size. The berthing energy and capacity of the existing piles at the landing were calculated.

- A structural analysis model was created using the program SAP2000 to determine the berthing energy absorption of the structural system.
- The timber structural code NDS 2005 was used to determine the member capacities.

9.3 Condition Assessment

The condition of the existing landing is discussed in section 8.

9.4 Analysis

A preliminary berthing analysis was performed in order to determine the berthing energy demand for the design of the landing structure. The design vessel was a 200 LT Coast Guard Marine Protector Class vessel, with a 110 ft. length overall. The berthing energy was calculated using equations from United Facilities Criteria 4-152-01: Piers and Wharves (UFC), and is found to be 12 ft.-kips. The structural response was modeled in

SAP2000. The capacities of the structural members were calculated and compared to the demands. The calculations are provided in Attachment 2C.

There are three methods that might be utilized for boats to access Santa Cruz Wharf:

- Floating dock
- Crane (davits)
- Fixed landing

Floating Dock

Due to its location on the open coast, Santa Cruz Wharf can be exposed to winter waves in excess of 20 feet in height (see Section 10.4.7). To provide a floating dock for all year use (remains in winter) would require a very large system. Such a floating dock would be massive in size and would require large steel piles to anchor it and withstand the forces of winter waves. The scale of such a structure would be on the order of hundreds of feet in length for the dock, and constructed of steel with restraining piles 3 to 5 feet in diameter. This design concept is determined to be impractical at this location.

Crane

Boats can be accessed from the water by use of a crane that lifts them from the water back onto the Wharf deck. This system was used when fishing boats were prevalent on the Wharf and is the method used presently to launch the rental boats and the Wharf boat. This design concept is feasible for smaller boats, but not for the 200 LT design vessel.

Fixed Landing

The fixed landing system is the one that has been used and is the most feasible for all year access at the Wharf.

Elevation of the fixed landing is critical, as it will be exposed to waves all year round. The critical component of the landing system is decking that must withstand wave forces due to its inherent location. As mentioned in the preceding section, the existing timber boards will be lifted during winter storm waves (see Photo 8-7). The use of fiberglass grating will withstand the saltwater environment and will allow much of the wave pressure to be relieved in the wave up-rush.

The new landing to accommodate visiting vessels will be a fixed platform that is accessed from the main wharf by a series of gangways (walkways) that form an accessible route (1:12 slope with landings). The landing will be situated near the same elevation as the existing landing in order to be above the high tide yet be low enough to minimize the difference in heights between the landing and boat deck. Access onto the vessel will be accomplished by means of a 30 ft. long gangway (see Figure 9-1) that is hinged on the landing and will be lowered and raised to match the elevation of the boat deck access over the range of tides the boat can access the Wharf. The size vessels that can be accommodated will be up to 200 tons, which corresponds to a vessel of approximately 120 ft. in length, maximum. Due to the location of the Wharf, it is anticipated that most vessels will approach from the south such that the port (left) side of the vessel contacts the Wharf.

The gangway down to the landing will be supported by framing on new timber pile(s) that align with the existing rows (bents) of piles. The landing will be constructed of fiberglass (Fiber Reinforced Plastic-FRP) decking material, to allow wave energy to pass through the deck, which reduces uplift pressure during winter storm waves (Photo 8-7), that have lifted and dislodged the landing deck boards in the past. It also has excellent resistance to decay. There are 2 types of FRP decking: 1. pultruded which is made of individual bars such that a continuous gap is formed (Photo 9-1) and 2. molded that is made in a grid configuration (Photo 9-2). The support framing directly below the grating would be either timber, similar to the existing landing framing, or fiberglass channels. The framing would be supported on timber piles—either the piles for the existing landing or new piles. The piles along the waterside face of the landing will be extended above the Wharf deck to act as fender piles (cushioning) for the boat while it is approaching (berthing) the landing to tie lines to the landing (moor). Additional piles will be added for the purpose of guiding the boat in while berthing outside of the landing, but in line with the face of the landing.

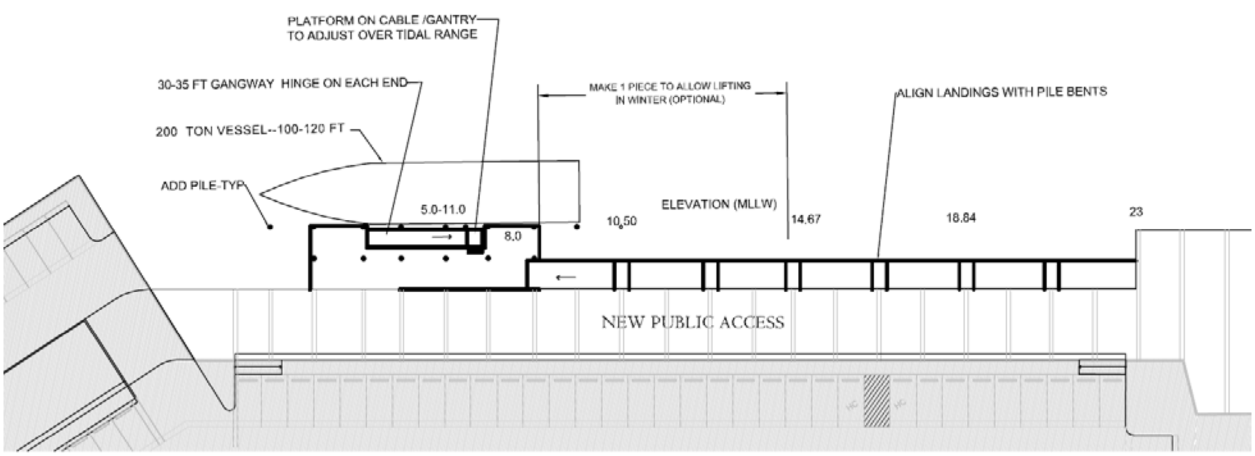


Figure 9-1: New Vessel Landing

Photo 9-1: Pultruded FRP Decking

Photo 9-2: Molded FRP Decking

9.5 Conclusions and Recommendations

The existing timber piles and framing system at Landing No. 2 are adequate capacity to absorb the berthing energy from a 200 ton vessel. If the existing landing structure is to be used to berth the larger vessel, the following are recommended.:

1. Install a 12x12 timber waler for energy absorption.
2. Install Fender piles connected back to the Wharf for added stiffness.

10. ENVIRONMENTAL IMPACTS

10.1 Summary

Santa Cruz Wharf was located behind Lighthouse Point for the natural shelter provided at this location. However, this location is on the open coast and subject to inherent forces of the sea. The elevation of Wharf deck (23 ft. MLLW) is sufficient to keep the Wharf deck above all but the infrequent, highest waves which can be up to 20 ft. in height. As sea level rises waves will be closer to the Wharf deck more frequently. Additional piles to widen the Wharf will increase the Wharf's ability to withstand these waves and other lateral forces. These elements have been incorporated into the Master Plan for the Wharf. This combined with the continued maintenance performed by the Wharf staff will allow the Wharf to continue to resist the forces of the sea..

10.2 Introduction

10.2.1 Scope

The scope of this report section is to provide a summary of the following existing and changing environmental conditions and their potential impacts to the Wharf:

- Bathymetry
- Tide levels (astronomical, surge, extreme)
- Future climate change
- Increased Storminess
- Coastal erosion
- Sea Level Rise
- Tsunamis and Earthquakes
- Coastal Flooding
- Tidal Currents (winds, wave environment, currents)

Identify mitigation measures to minimize potential damage due to collisions with recreational and commercial vessel traffic. Consider placement of navigational aids and warning devices.

10.2.2 Description of Structure

Santa Cruz Wharf is located in Santa Cruz Harbor protected from the prevailing waves by Lighthouse Point to the west. The Wharf is approximately 2300 ft. long and the end is angled to orient into the predominant wave direction, therefore minimizing the impact on the structure. This location and the deck elevation of 23 feet (nominal) above mean lower low water (MLLW) are the two most important elements protecting Santa Cruz Wharf from environmental forces or impacts. Santa Cruz municipal wharf was constructed in 1914, and is the last surviving, of four other adjacent wharves, located in Santa Cruz Harbor.

10.2.3 Methodology

The method of analysis in determining environmental forces was to use existing data from reputable sources (see references at end of section) and using established methods of coastal engineering for forecasting future events.

10.3 Condition Assessment

The Wharf is in good structural condition, due to the location, material of construction and continuous maintenance performed on the structure. The deck elevation of 23 feet (MLLW), puts it above all but the most extreme waves. In addition, the vertical timber piles and their inherent flexibility have withstood many earthquakes over the past 100 years, including the 1989 Loma Prieta earthquake whose epicenter was 5 miles away.

10.3.1 Bathymetry

Figure 10-1 shows the multi-beam survey conducted between October 17 and October 20, 2009 for the scwd[2] Seawater Desalination Project (EcoSystems Management Associates, Inc., 2010). (Note: Figure 10-1 through Figure 10-10 are presented in the report, Figure 10-11 through Figure 10-33 are presented in Attachment 10A) Generally, the water depth at the pier end is approximately 26- to 28-ft below MLLW.

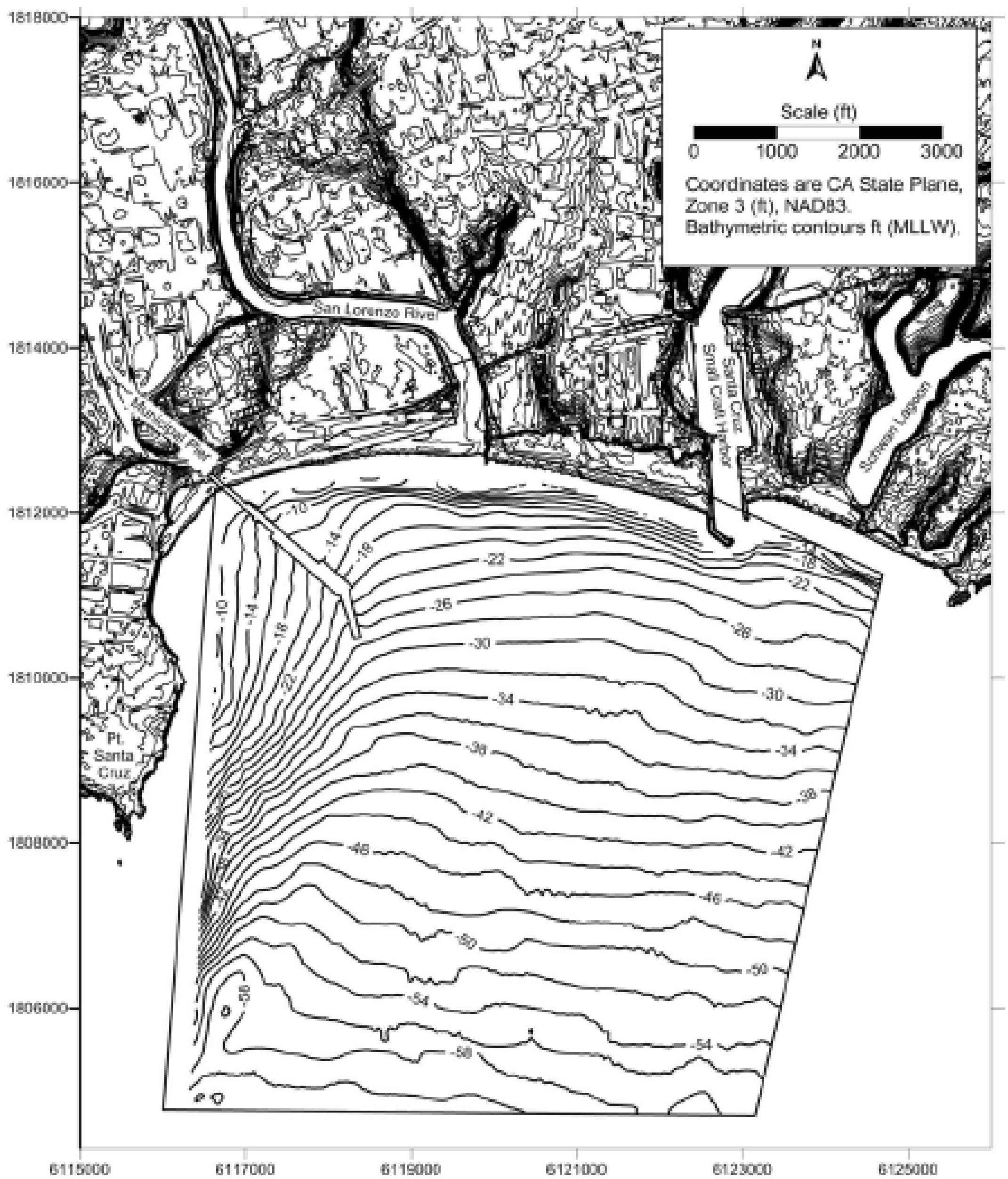


Figure 10-1: Multi-beam Survey Conducted in October 2009 for the scwd[2] Seawater Desalination Project (EcoSystems Management Associates, Inc., 2010)

10.3.2 Tide Levels

There is no permanent tide gauge in the Santa Cruz area. The closest is the National Oceanic and Atmospheric Administration (NOAA) tide gauge at Monterey (NOAA CO-OPS 9413450), approximately 25 miles to the south. A spot check for predicted tides at Santa Cruz and Monterey gives similar results. Therefore, this study assumes that the Santa Cruz area and Monterey tide gauge have the same water levels and under the same meteorological effects (same residual tides). Table 10-1 lists the tidal datum at Monterey tide gauge. Figure 10-11 plots the 40-year time series of astronomical tides and residual tides (the measured values less the predicted values). All figures not in the main body of text are found in Attachment 10A. It is noted the 2011 Japan Tsunami shows a clear drop-down of sea surface in the residual plot.

Table 10-1: Tidal Datum at Monterey, CA (NOAA CO-OPS 9413450)

Tidal Plane	Elevation in feet	
Mean Higher High Water (MHHW)	5.34	5.48
Mean High Water (MHW)	4.64	4.78
Mean Tide Level (MTL)	2.87	3.01
National Geodetic Vertical Datum 1929 (NGVD29)	2.60	2.74
Mean Low Water (MLW)	1.10	1.24
Mean Lower Low Water (MLLW)	0	0.14
North American Vertical Datum 1988 (NAVD88)	-0.14	0

The annual maximum measured tides (including both astronomical tide and local storm effects) were used to conduct the extreme tides analysis. Figure 10-12 shows the estimated results with a Gumbel best fit and Table 10-2 lists the values. For a 50- and 100-year return period, the estimated tides are 7.8 and 8.0 ft. above MLLW, respectively.

Table 10-2. Extreme Tides Analysis for the Monterey Tide Gauge

Return Period (years)	Extreme Tides (ft., MLLW)
2	7.0
5	7.3
10	7.4
25	7.7
50	7.8
100	8.0

10.4 Analysis

10.4.1 Climate Change

Although human civilizations has been adjusting to the changing climate for centuries, it is not until recently that climate adaptation planning becomes a standard procedure for a community or society to plan for future climate changes. The majority of recent scientific research indicates that climate change outside the range of past human experiences is occurring. This further increases the complexity of planning to an even uncertain level (NAS-NRC, 2010). A detailed assessment of impacts and regulations on

Greenhouse gases emissions and climate change are summarized in the City of Santa Cruz General Plan 2030 (City, 2012). Future predictions of climate scenarios or trajectories provide insight about the range of future possibilities, rather than a certain single value.

Potential climate change processes that lead to impacts summarized in the following sections includes:

- Global warming
- Increased frequency and intensity of coastal storms
- Continuing and accelerated sea level rise
- Changing patterns of precipitation, fog, and winds

10.4.2 Increased Storm Intensity

Studies have found a progressive increase in wave-energy levels in the North Atlantic and North Sea since the 1950s and in the North Pacific since the late 1970s, possibly due to global climate change. Over the last 15 years, the U.S. West Coast has experienced unusually intense wave conditions and the storm frequency and magnitude seem increasing. Allan and Komar analyzed measured wave statistics at six wave buoys from the NOAA NDBC along the U.S. West Coast, including one buoy at Half Moon Bay, approximately 30 miles to the north. Although some variations exist, the general trend indicates an increase of average significant wave height and average peak wave period from the analysis. The same study also suggests strong evidence that major El Niño corresponds to years of higher wave conditions off the coasts of California and Oregon (Allan and Komar, 2000).

A similar effort by the U.S. Geological Survey (USGS) investigates specifically off the central coast, where the Project is located (Storlazzi and Wingfield, 2005). By analyzing 22-year buoy data off the central coast, they observed a trend of significant wave height increased by 2 cm/yr. on average, which is equivalent to an increase of 1.4-ft over the past 22 years. The long-term trend also suggests greater storminess and storm intensity over the study period. During El Niño months, the mean significant wave height is higher and larger waves are more frequent (30% more frequent than average for waves exceeding 4 meters). In contrast, during La Niña months, their increases are less profound.

10.4.3 Coastal Erosion

Coastal erosion includes both cliff erosion and beach erosion and may result from the rising sea level and severe storm waves, exacerbated by episodic El Niño and even infrequent tsunami occurrences. Because at the back of the Wharf lies one flat, wide, essentially continuous beach from Cowell's Beach, Main Beach to the mouth of San Lorenzo River, it is highly vulnerable to coastal erosion. Historical photos show beach erosion due to severe El Niño winters in 1926, 1983, and 1997-1998 (Griggs and Haddad, 2011).

Figure 10-13 through Figure 10-16 illustrate a sequence from the base case in 2008 to a 3-ft rise of sea level scenario for the Main Beach (Griggs and Haddad, 2011). Figure 10-2 below shows an example for the 3-ft. sea level rise case. In these figures, blue indicates areas that are inundated under each scenario. They noted that because the beach is relatively flat, even a foot of sea level rise would cover much of the Main Beach and consequently lead to beach erosion. However, regular overtopping of existing concrete retaining wall/seawall along the back of the Main Beach will probably not occur until sea level rises at least an additional two feet.

Near the Wharf, just south of the Cowell's Beach, the West Cliff Drive consists primarily of sea cliffs that front a flat marine terrace. Because most of this portion has been armored with riprap (visible from the Google Earth©), it is not considered to be threatened immediately. However, a continuous rising sea level with severe storms might potentially cause cliff erosion and consequently retreat, especially where the riprap is not functionally intact during the events.

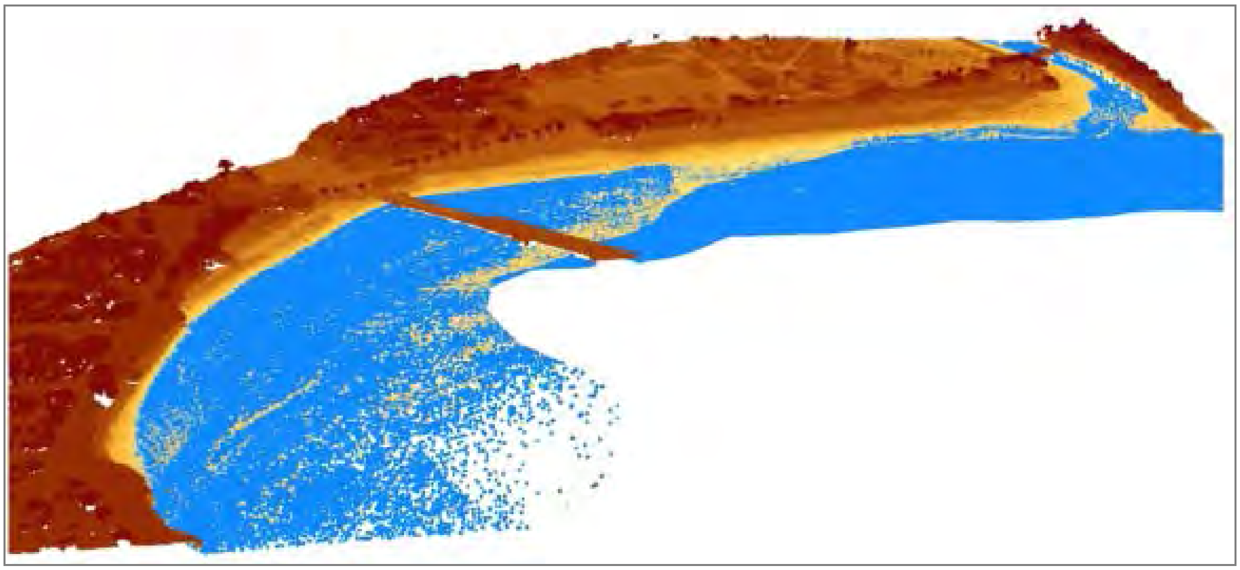


Figure 10-2: Main Beach and the Wharf at Mean High Tide with a 3-ft SLR Scenario (Extracted from Griggs and Haddad, 2011)

10.4.4 Sea Level Rise

A vulnerability assessment concludes that historically sea level rise (SLR) probably generated the most obvious and visible effects in the City of Santa Cruz and will continue to produce some significant impacts on the City's coastline (Griggs and Haddad, 2011). A range of hazards due to SLR include landward inundation of low-lying areas, erosion of coastal cliffs and beaches, and intrusion of sea water. This memo focuses on relative or local sea level change (could be rising or dropping, dependent on the local tectonic movements), rather than absolute or global sea level change.

The two closest tidal measurements are from the NOAA tide gauges at Monterey (NOAA CO-OPS 9413450), approximately 25 miles to the south, and at San Francisco (NOAA CO-OPS 9414290), approximately 65 miles to the north. NOAA Center of Operational Oceanographic Products and Services has tracked mean sea level trend at some relatively long-term tide gauges on all U.S. coasts (NOAA, 2012). Figure 10-17 indicates a rise of 0.93 mm/yr. ± 0.99 mm/yr. with a 95% confidence interval at Monterey gauge between 1973 and 2012 (40-year data), which is equivalent to an increase of 0.31-ft in 100 years. Figure 10-18 indicates a rise of 1.92 mm/yr. ± 0.20 mm/yr. with a 95% confidence interval at San Francisco gauge between 1897 and 2012 (116-year data), which is equivalent to an increase of 0.63-ft in 100 years. Although the sea level trends are not that much different between these two gauges, it is noted that the San Francisco gauge has longer and less variant records.

However, most of the scientists and researchers agree that future sea level change estimate has a highly uncertain characteristic and relying only on the past tide records is not sufficient. Therefore, based on the most recent update, the Coastal and Ocean Working Group of the California Climate Action Team (CO-CAT) 2013 SLR guidance recommends a sea-level rise range from 5 to 24 inches by year 2050 (16 inches suggested in CO-CAT, 2010) and a range from 17 to 66 inches by year 2100 (55 inches suggested in CO-CAT, 2010), at the region south of Cape Mendocino, California. This recommendation should be taken into account for project planning and decision-making in California.

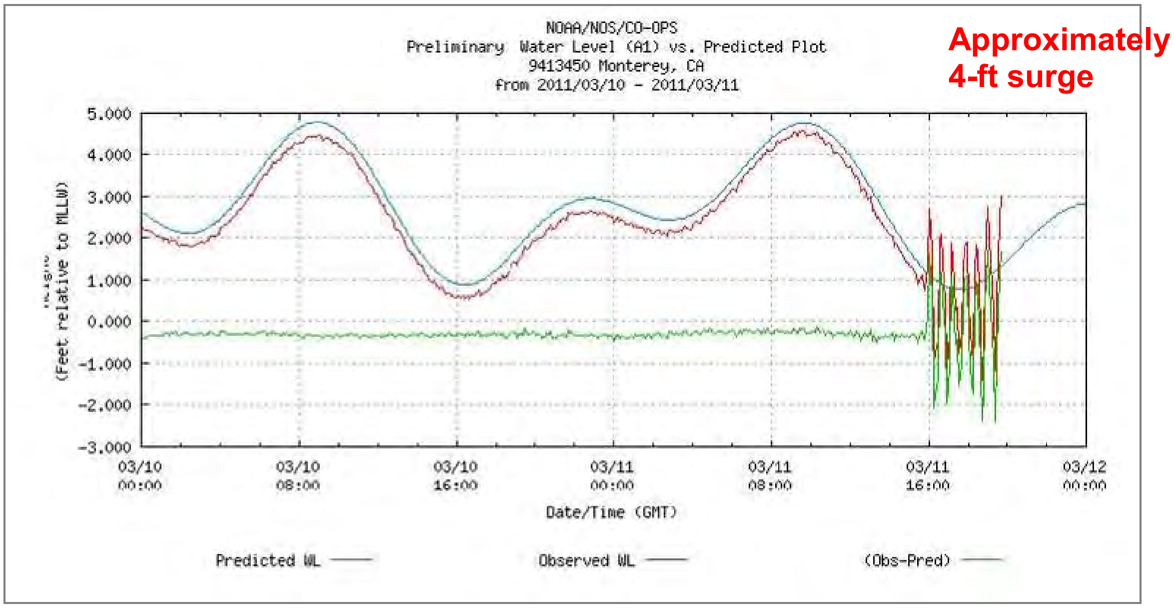
Table 10-3: Sea Level Rise Projections using Year 2000 as the Baseline

Time Period	North of Cape Mendocino	South of Cape Mendocino
2000 – 2030	-4 to 23 cm (-0.13 to 0.75 ft.)	4 to 30 cm (0.13 to 0.98 ft.)
2000 – 2050	-3 to 48 cm (-0.1 to 1.57 ft.)	12 to 61 cm (0.39 to 2.0 ft.)
2000 - 2100	10 to 143 cm (0.3 to 4.69 ft.)	42 to 167 cm (1.38 to 5.48 ft.)

10.4.5 Tsunamis and Earthquakes

The tsunami hazard is not new to the California Coast because the entire Pacific Rim is highly seismically active. During the most recent 9.0 magnitude earthquake in Japan in March 2011, a tsunami reached Santa Cruz and caused substantial damage to the Santa Cruz Small Craft Harbor. Figure 10-3 shows the observed tsunami-induced surge at NOAA's Monterey and San Francisco tide gauges, just hours after the earthquake hit Japan and sent the tsunami waves to the U.S. West Coast. Although the Santa Cruz Yacht Harbor sustained considerable damage, the Wharf was undamaged. This is largely due to the location on the open coast without a narrowing of the waterway, as was the case at the Yacht Harbor and also the damaged harbor in

Crescent city. Approximately 4- and 3.5-ft surge were recorded at Monterey and San Francisco, respectively.



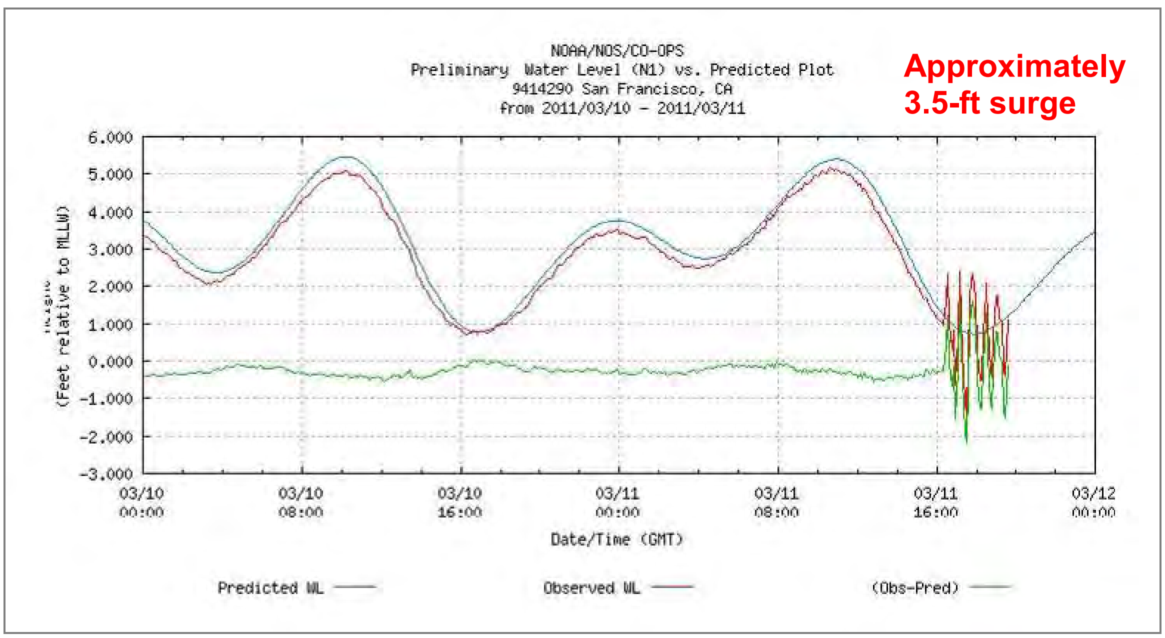


Figure 10-3: Tsunami-Induced Surge Observed at NOAA CO-OPS Gauges Hours after Japan 9.0 Magnitude Earthquake

Below is excerpted from the City's General Plan 2030, which provides a description of the potential tsunami hazard within the City. "Several active and potentially active earthquake faults are located within or near Santa Cruz. Even a moderate earthquake occurring in or near any of the nearby faults could result in local source tsunamis from submarine landsliding in Monterey Bay, such as the one after 1989 Loma Prieta

Earthquake. Additionally, distant source tsunamis from the Cascadia Subduction Zone to the north, or Teletsunamis from elsewhere in the Pacific Ocean are also capable of causing significant destruction in the City. An U.S. Army Corps of Engineers 1975 study, cited by the City's General Plan 2030, estimated a tsunami wave with a probability of occurrence of one in every 100 years would be about 5.9 feet high and one in every 500 years would be about 11.5 feet high. In addition, a model of a locally-generated landslide in the Monterey submarine canyon predicts about 23 feet of runup and strikes the coastline in as little as 10 minutes."

A joint effort by the California Emergency Management Agency, the California Geological Survey, and the University of Southern California modeled a suite of tsunami source events, representing realistic local and distant earthquakes and hypothetical extreme undersea, near-shore landslides (CGS, 2009). The produced maps are designed for emergency planning along the California coast. However, each map does not represent inundation from a single event, but an ensemble of potential source events affecting each specific region. Figure 10-4 illustrates the tsunami inundation zone within the City of Santa Cruz. The Wharf, the entire beach, and the majority of the Downtown area are within the potential tsunami inundation zone.

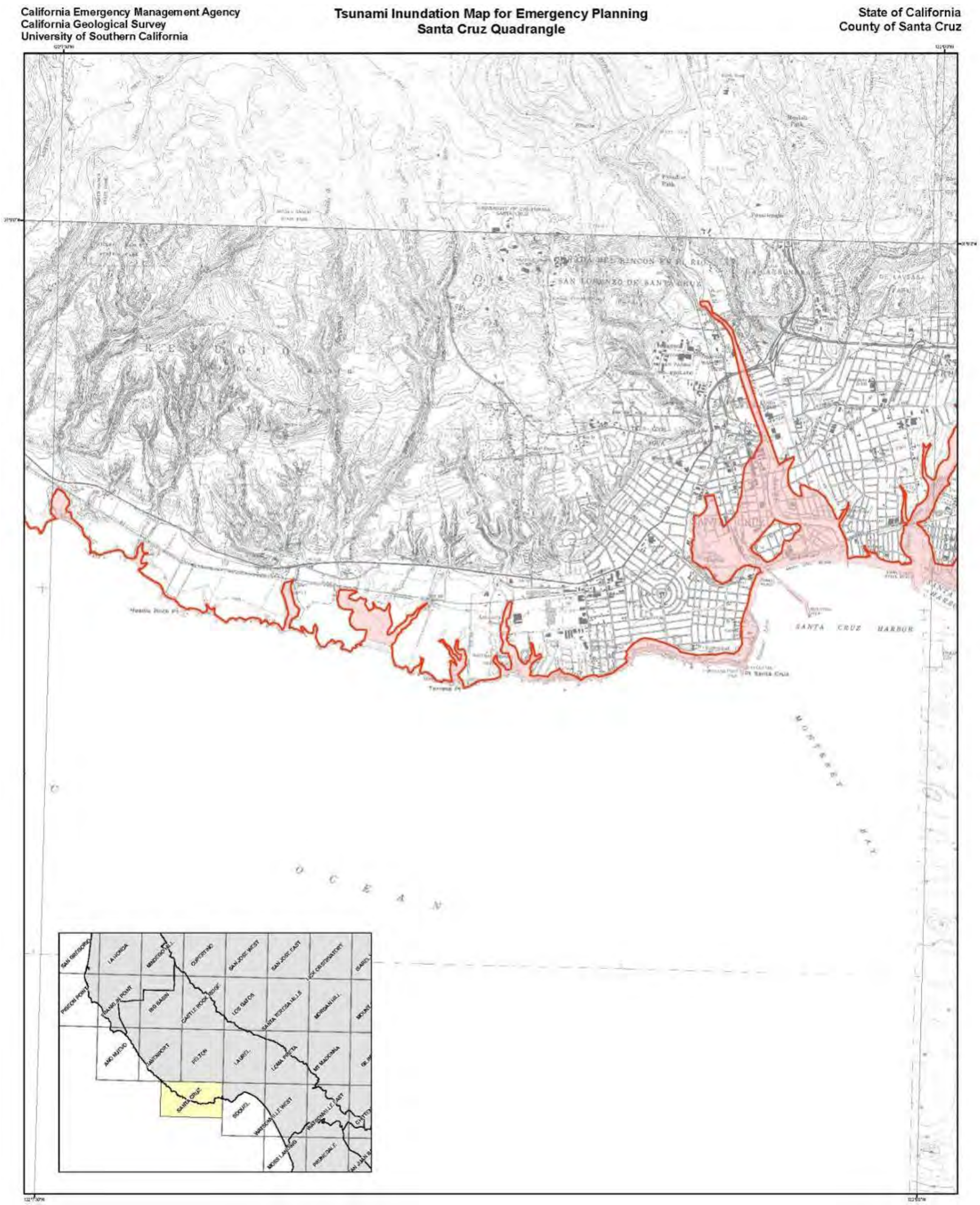


Figure 10-4: Tsunami Inundation Zones within the City of Santa Cruz (Extracted from the California Geological Survey Website)

10.4.6 Coastal Flooding

With the current trend of climate change, an increase of rainfall intensity and changing precipitation pattern is expected. Much concentrated runoff due to changing precipitation pattern as well as the SLR, larger coastal storms, and tsunami may cause flooding at the low-lying areas.

The City of Santa Cruz leads an effort to update the City's General Plan extending to year 2030 (City of Santa Cruz, 2012). The General Plan notes that although the City has been working to improve the flood capacity of the San Lorenzo River levees over the past two decades and reaches significant improvements, the risk of flooding may still occur. However, the flood improvements were recognized by the Federal Emergency Management Agency (FEMA) and they re-issued the Digital Flood Insurance Rate Map (DFIRM) to re-designate much of the Downtown area to the A-99 Flood Zone, an area protected from a 1% chance of flooding by a Federal flood control system under construction. Figure 10-5 illustrates the re-issued FEMA DFIRM, as extracted from the City's General Plan 2030.

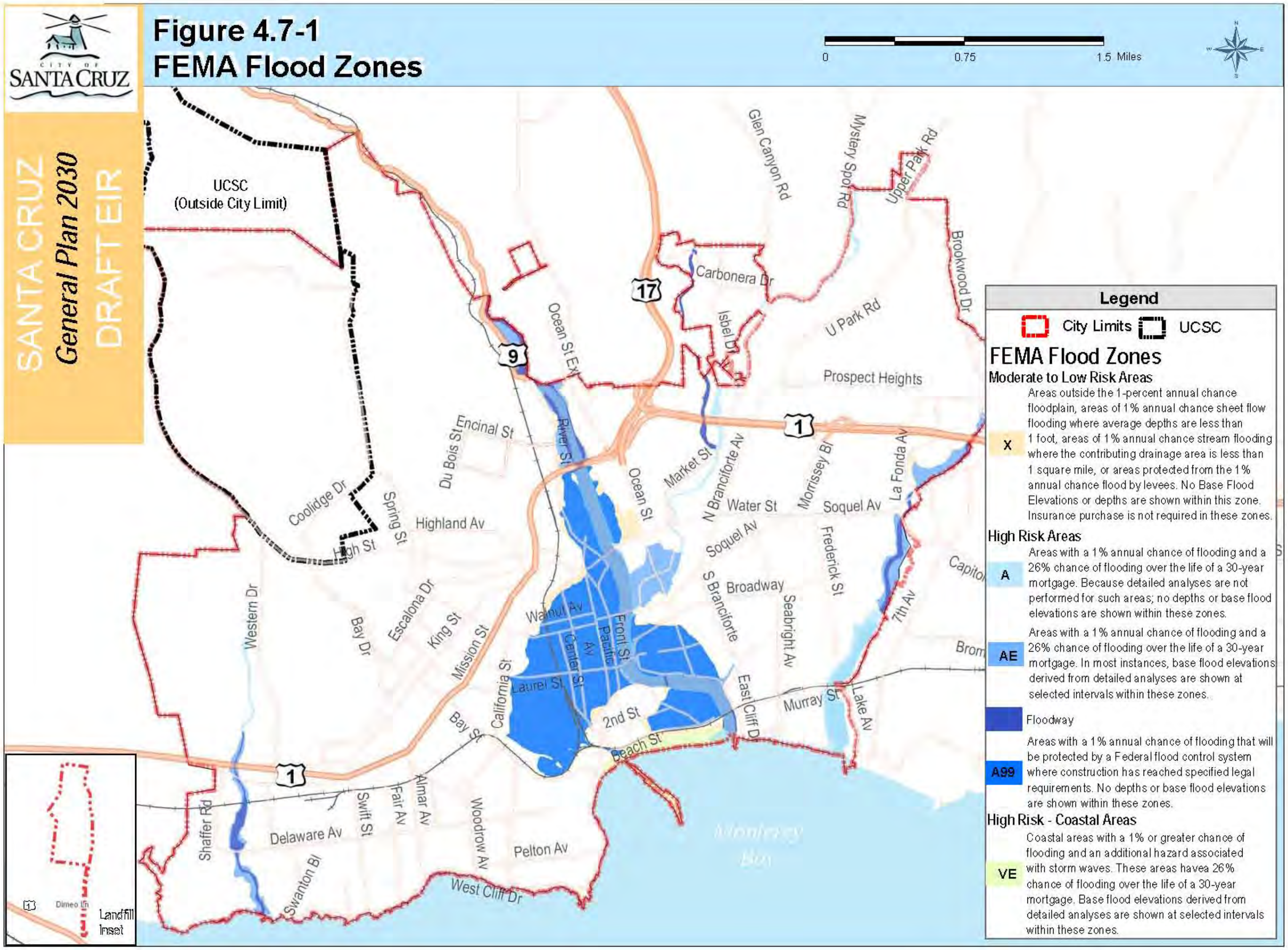


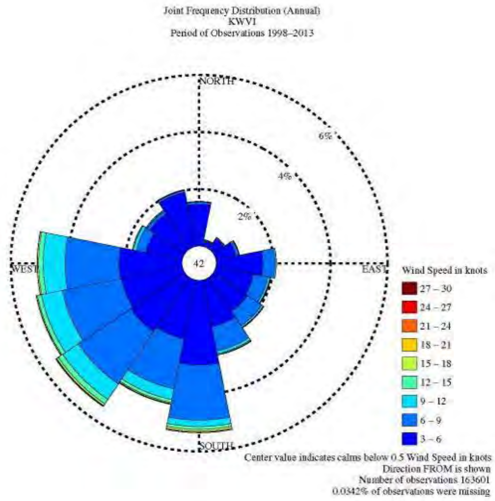
Figure 10-5: FEMA Flood Zones within the City of Santa Cruz (Extracted from the City of Santa Cruz General Plan 2030)

10.4.7 Tidal Currents

Wave

The offshore wave climate along central California can be characterized by three dominant modes: the Northern Hemisphere swell, the Southern Hemisphere swell, and local wind-driven seas. The Northern Hemisphere swell typically is generated by cyclones in the North Pacific Ocean off the Aleutian Islands during the winter months (November-March) and can attain deep-water wave heights exceeding 8 m. The Southern Hemisphere swell is generated by storms off New Zealand, Indonesia, or Central and South America during summer months and, although generally it produces smaller waves than the Northern Hemisphere swell, this swell often has very long period (15+ seconds). The local seas typically develop rapidly when low-pressure systems track near central California in the winter months or when strong sea breezes are generated during the spring and summer. Storms with deep-water wave heights in excess of 5 m occur five times a year on average (Storlazzi and Wingfield, 2005; Storlazzi et. al., 2011).

The NOAA National Data Buoy Center (NDBC) Buoy 46042 is located approximately 30 miles southwest to the Wharf. It is a deep water buoy with water depth over 6500 feet. The offshore wave climate was analyzed from 1991 to 2012. Figure 10-19 through Figure 10-28 show the annual and seasonal significant wave height rose and peak wave period rose. Figure 10-6 below shows an example rose for the annual wave height. At the buoy, long-period swells are clearly dominant rather than the short-period seas.



Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	45	0.74	1.09	1.39	3.13	2.61	2.77	3.38	6.49	5.23	6.29	6.02	6.15	2.82	3.47	3.47	100
27–30																	
24–27																	0.01
21–24																	0.03
18–21									0.03	0.01	0.02						0.1
15–18								0.02	0.07	0.04	0.07	0.03	0.04				0.29
12–15						0.02	0.02	0.03	0.1	0.11	0.19	0.14	0.16	0.03			0.84
9–12	0.02	0.01		0.02	0.05	0.07	0.06	0.1	0.22	0.39	0.66	0.75	0.76	0.12	0.04	0.02	3.3
6–9	0.07	0.03	0.05	0.11	0.42	0.52	0.61	0.87	1.94	1.79	2.22	2.1	1.9	0.34	0.12	0.07	13.2
3–6	1.47	0.2	0.44	0.7	1.63	1.34	1.43	1.67	2.97	2.1	2.27	2.18	2.2	1.3	1.47	1.93	25.3
0–3	43.4	0.5	0.6	0.56	1.02	0.65	0.63	0.7	1.14	0.78	0.84	0.79	1.06	1.04	1.83	1.44	57

Figure 10-6: Annual Wave Height Rose at the Buoy 46042

In terms of seasonality, three groups could be distinguished: spring/fall, summer, and winter. During spring and fall, the predominant swell directions are from northwest and west-northwest, with slightly more frequent from the northwest. During summer, the predominant swell direction is mainly from the northwest. During winter, a broader band of incoming swells occur between west and northwest, with the most frequent from the west-northwest. Therefore, the offshore swells are originated from the west in winter. Additionally, the winter experiences more frequent storms with swell height exceeding 15 ft. 5.5% of the time, followed by the spring and fall with 2.3% and 1.7%, respectively. In contrast, the summer experiences these swell heights less than 0.1% of the time. Similarly, swell period exceeding 15 seconds occurs 19.3%, 12.1%/11.5%, and 8.8% of the time for winter, spring/fall, and summer, respectively. Table 10-4 summarizes the distinct wave climate seasonality at the buoy.

Table 10-4: Summary of Offshore Wave Climate Seasonality at Buoy 46042

Season	Predominant Swell Directions	Percent of time swells exceeding 15 ft.	Percent of time swell period exceeding 15 seconds
Spring	WNW – NW	2.3%	12.1%
Fall	WNW – NW	1.7%	11.5%
Summer	NW	0.1%	8.8%
Winter	W – WNW - NW	5.5%	19.3%

The Coastal Data Information Program (CDIP) of the Scripps Institution of Oceanography developed a regional swell model off the coast of the Monterey Bay. Two snapshots were downloaded from the CDIP website: one is during west-northwest swells and the other is during south swells (Figure 10-7 and Figure 10-8). Historically, during predominant west through northwest swells, a significant wave reduction was observed at the Wharf due to refraction around Point Santa Cruz. The USGS model shows an approximately 35% of wave energy remaining near the Wharf during west-northwest swells (Storlazzi et. al., 2011). The CDIP snapshot also shows a significant amount of wave dissipation near the Wharf. However, during south swells, the CDIP snapshot shows the wave energy can propagate into the area with minor or no reduction, due to its direct exposed to the south. The annual wave rose shows potential waves up to 30 ft. from the west – northwest quadrant and 20 ft. from the south. With the assumption of 35%-40% of energy remaining, the west - northwest swells yield an approximate maximum wave of 12 ft. near the Wharf. With the assumption of no wave reduction for south swells, the potential south swells result in an approximate maximum wave of 20 ft. near the Wharf. Given the seabed is at 26-ft below MLLW, the waves are possibly not breaking near the end of the Wharf.

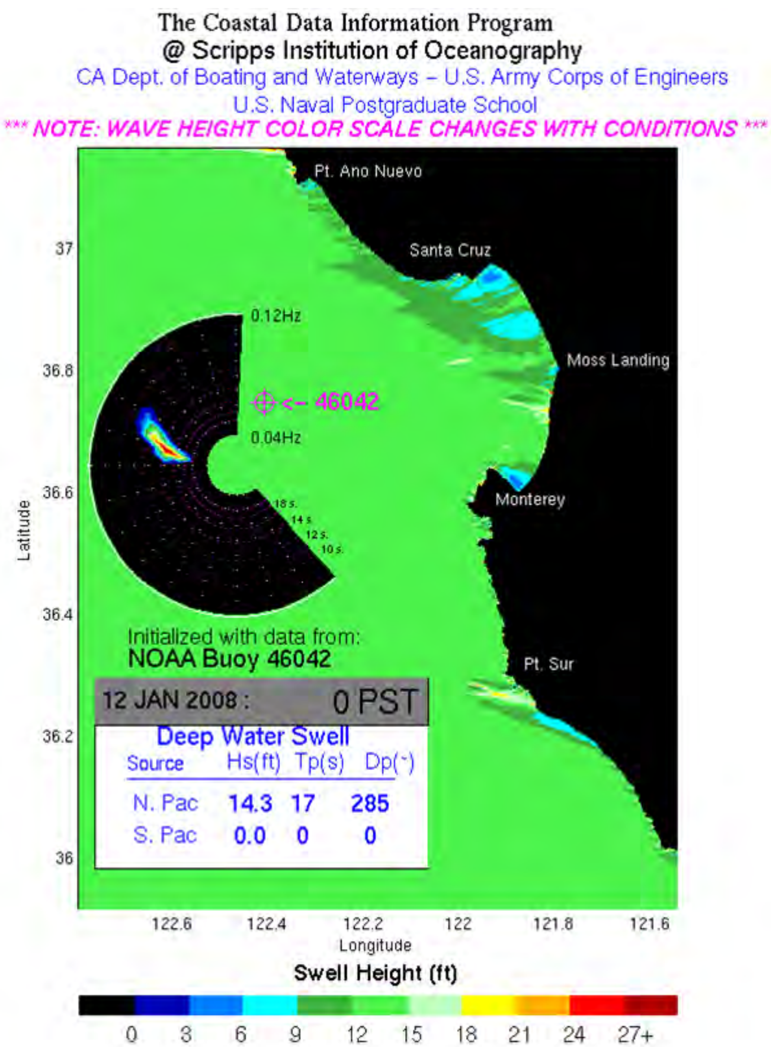


Figure 10-7: CDIP Monterey Bay Swell Model – a Snapshot during WNW Swells

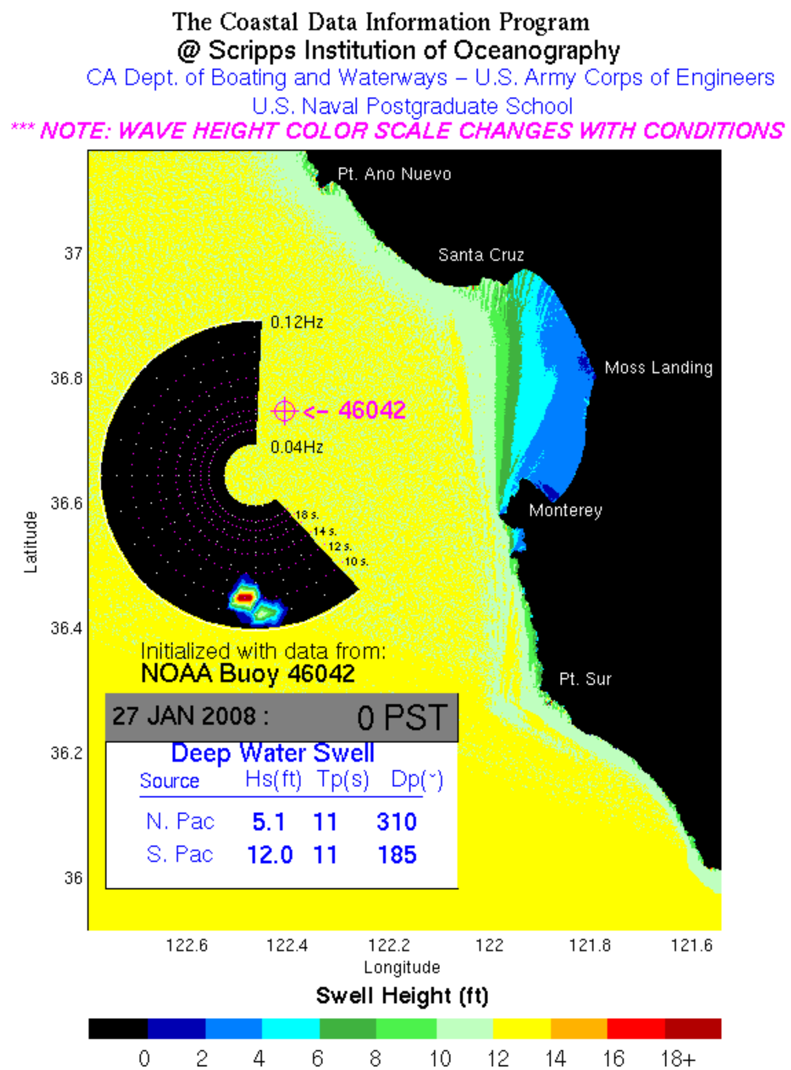
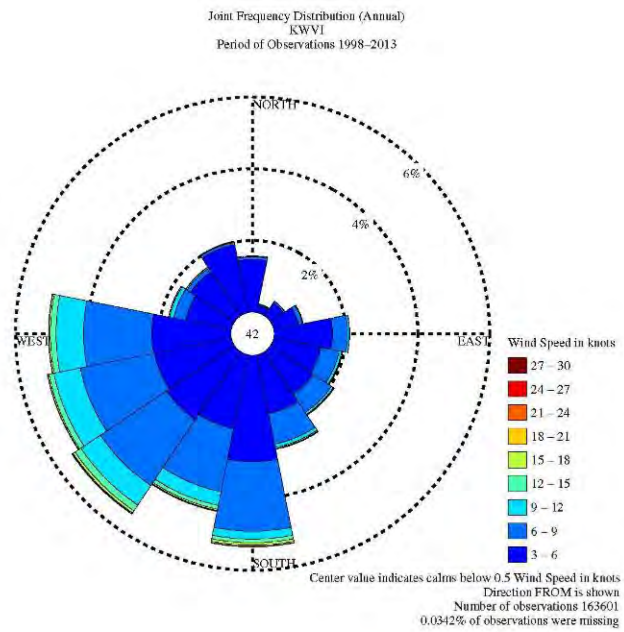


Figure 10-8: CDIP Monterey Bay Swell Model – a Snapshot during South Swells

Wind

The Watsonville Airport, approximately 13 miles to the east, is one of the Meteorological Terminal Aviation Routine weather report (METAR) network stations (Station ID: KWVI). The 2-minute duration winds were analyzed from 1998 to the present. Figure 10-9 (below) and Figure 10-29 through Figure 10-32 (Attachment) show the annual and seasonal wind roses for the Airport. Wind speeds less than 0.5 knots are considered calm conditions.



Percentage of Occurrence

Wind Speed in knots	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	Total
Total	45	0.74	1.09	1.39	3.13	2.61	2.77	3.38	6.49	5.23	6.29	6.02	6.15	2.82	3.47	3.47	100
27–30																	
24–27																	0.01
21–24																	0.03
18–21									0.03	0.01	0.02						0.1
15–18								0.02	0.07	0.04	0.07	0.03	0.04				0.29
12–15						0.02	0.02	0.03	0.1	0.11	0.19	0.14	0.16	0.03			0.84
9–12	0.02	0.01		0.02	0.05	0.07	0.06	0.1	0.22	0.39	0.66	0.75	0.76	0.12	0.04	0.02	3.3
6–9	0.07	0.03	0.05	0.11	0.42	0.52	0.61	0.87	1.94	1.79	2.22	2.1	1.9	0.34	0.12	0.07	13.2
3–6	1.47	0.2	0.44	0.7	1.63	1.34	1.43	1.67	2.97	2.1	2.27	2.18	2.2	1.3	1.47	1.93	25.3
0–3	43.4	0.5	0.6	0.56	1.02	0.65	0.63	0.7	1.14	0.78	0.84	0.79	1.06	1.04	1.83	1.44	57

Figure 10-9: Annual Wind Rose at the Watsonville Airport

Results show the predominant winds are from the south through west quadrant, especially during the spring, summer, and fall seasons. During late fall to early spring, the north and northwest winds start increasing their occurrences, with the greatest during the winter season. Additionally, the winter has the most frequent storms, with 0.4% of the time winds exceeding 18 knots, followed by the spring and fall season with 0.08% and 0.11%, respectively. None of the winds exceeding 18 knots in summer during 16-year data analyzed for the Watsonville Airport. Given the Wharf is directly exposed to the south, the potentially largest winds from the south through southwest might need to be taken into account.

The annual maximum winds, irrespective of wind directions, were used to conduct the extreme winds analysis. Figure 10-33 shows the estimated results with a Weibull best fit and Table 10-5 lists the values. For a 50- and 100-year return period, the estimated wind speeds are 34.3 and 36.5 knots, respectively.

Table 10-5: Extreme Winds Analysis for the Watsonville Airport

Return Period (years)	Extreme Winds (knots)
2	24.4
5	27.2
10	29.4
25	32.2
50	34.3
100	36.5

Currents

Tidal circulation in the Monterey Bay National Marine Sanctuary (MBNMS) is driven by the California Current System, which mixes cool, lower-salinity subarctic water with warm, saltier subtropical water. The ocean is often stratified with density/temperature varying with depth. Three distinct oceanographic seasons were recognized:

Upwelling period from early spring to late summer, when surface waters are cool;

Oceanic or California Current period from late summer to early fall, which allows previously upwelled water to sink and be replaced by warm oceanic waters from offshore; and

Davidson Current period from late fall to late winter, which is characterized by winter storm conditions.

Tidal circulation near the Wharf is affected by a combination of open coastal circulation, wave-induced turbulence, and the Monterey Bay Gyre. The prevailing current direction in the shallow, nearshore areas of Santa Cruz is dependent on the circulation pattern within the Monterey Bay and is predominant to the west. Below are some findings summarized in the Desalination Project (Tenera Environmental, 2010; URS, 2013):

Subsurface currents were parallel to shore and out of the Bay, roughly opposite of the wind driven surface flow;

Currents are dominated by semi-diurnal and diurnal tidal signals that lag the surface tides by roughly three hours on average. These flows over the course of a tidal cycle are very asymmetric, with the surface flow to the southeast during flooding tide lasting only one-third as long as the flow to the northwest during ebbing tide;

The transitions of the tide from ebbing to flooding cycle are very rapid and bore-like in nature.

Additionally, the USGS Sea Floor Observatory (Tripod A) and three additional seabed tripods, mounted with four upward-looking acoustic Doppler current meters (ADCM) and three downward-looking ADCMs, were deployed between 9 m and 30 m isobaths near the Wharf (Storlazzi et. al., 2011). Figure 10-10 presents the measured current results at various depths during the study period between October 2009 and December 2009. The mean currents offshore were much stronger and more uniform (Tripods C & D) than those closer to shore (Tripods A & B), where the mean currents were weaker and more variable. Closer to the seabed, the currents primarily were oriented cross-shore (in north-south direction) closer to shore, but alongshore (in east-west direction) at Tripod D. Currents near the surface primarily were oriented alongshore (Storlazzi et. al., 2011). Except the mean currents near seabed at Tripod C, the mean currents are primarily oriented out of the Bay, which is consistent with the observation in the Desalination Project. The largest measured current speed during this period is approximately 0.5 knots (0.28 m/s) at the seabed near the Wharf.

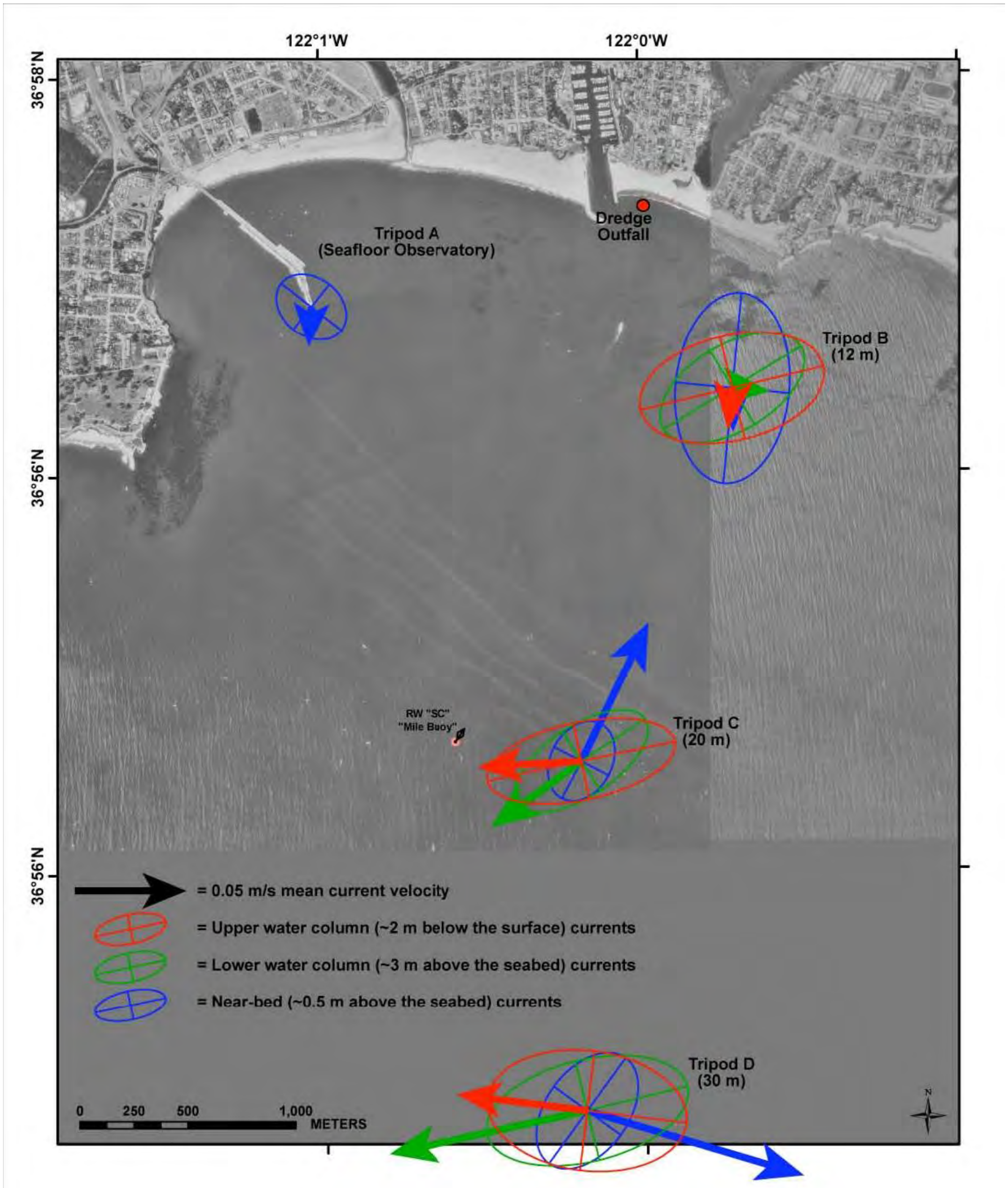


Figure 10-10: Illustration of Principal Axis Ellipses and Mean Current Speeds and Directions by Depths (Extracted from Storlazzi et. al., 2011)

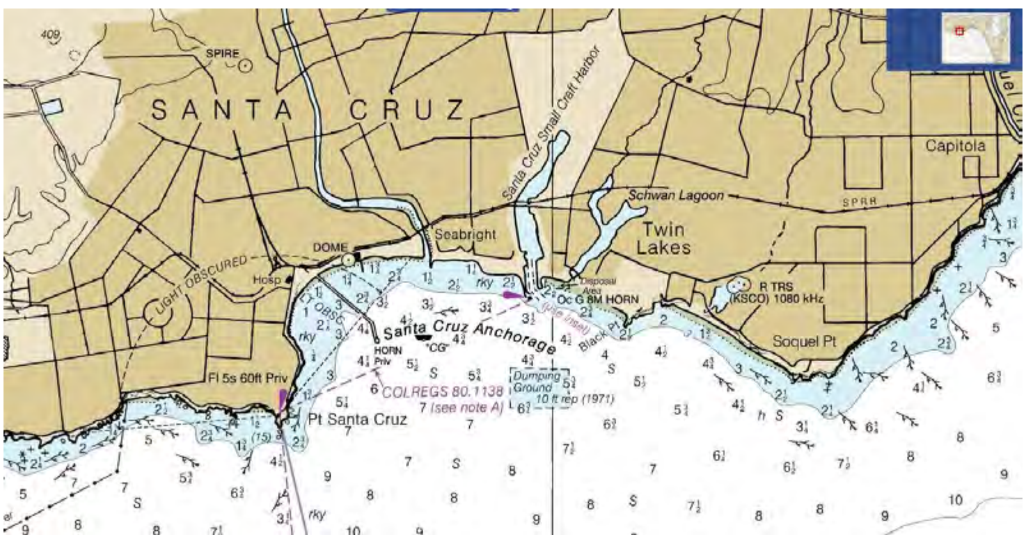
10.4.8 Collision Risk, Navigation Aids

Santa Cruz Wharf is the longest timber pier on the coast of the United States. While it is away from major shipping lanes or designated channels, it is near the entrance to Santa

Cruz Yacht Harbor which has a large volume of boat traffic, being the first harbor on the coast south of Pillar Point, 40 miles to the North. Considering this, the risk of collision (precise term is "allision"- vessel strikes a fixed object) is significant.

Santa Cruz Wharf is clearly identified on the nautical chart, as shown on Figure 10-11. The Wharf is well lit, and has 2 "obstruction" lights on each corner of the end (however these 2 lights are not indicated on the nautical chart-see recommendations). With these fixed aids to navigation (on chart, light and horn) the risk from collision due navigation error is low and adding additional fixed aids would do little to reduce this risk.

The greater risk, and has occurred in the past, is from vessels that are moored at anchor in the vicinity and the mooring breaks leaving the vessel adrift that then strikes the wharf, particularly if the vessel is moored on windward side (generally the west side). Further, most of the west side has buildings up against the edge of the Wharf adding greater risk of collapse and threat to life in the event of a large vessel impact. There is essentially no fendering (piles designed to take vessel impact) on the Wharf edges. The companion Master Plan provides a new walkway on the West side of Wharf that would act to provide fendering in the event of such a vessel collision. This feature would provide the greatest mitigation to the most likely risk of vessel collision at the Santa Cruz Wharf.



COLREGS Demarcation Lines

(253) The lines established for Santa Cruz Anchorage (Santa Cruz Harbor) are described in **80.1138**, chapter 2.

(257)

Anchorage

(258) Good anchorage can be had anywhere off the pier in 5 fathoms, sand bottom. Santa Cruz Anchorage provides good shelter in N weather, but in NW weather a heavy swell is likely to sweep into the anchorage. In S weather there is no protection in the harbor; vessels must run for Monterey or Moss Landing Harbor or take refuge in Santa Cruz Municipal small-craft harbor.

Wharves

(263) The municipal pier, 0.8 mile W of the entrance to the small-craft harbor, is over 0.4 mile long with 26 feet alongside at its outer end; a private seasonal sound signal in on the outer end of the pier. Landings can be made in all

but heavy S weather, but few vessels land except fishing boats. Due to the ocean swell sweeping around the point, there is usually considerable surge. The pier is lined with restaurants and stores. A small-boat hoist is on the pier.

Figure 10-11 Nautical Chart of Santa Cruz Harbor and COLREGS Excerpt

10.5 Conclusions and Recommendations

Because of its location and deck elevation (+23 ft., MLLW) Santa Cruz Wharf should continue to function well into the future, as it has for the past 100 years with continued maintenance and strengthening. With a sea level rise of 3.5 ft., the deck of Santa Cruz Wharf would be approximately at the same present elevation of Capitola Wharf.

The following are recommended to add to the resilience of the Wharf to withstand environmental forces and improve safety:

1. Widen the Wharf with vertical timber piles to increase its resistance to lateral wave and tsunami forces.
2. Evacuate the Wharf during periods of predicted extreme waves, as occurred in 1985 and 1998
3. Apply for a correction to the US Coast Guard to correct Chart 18685 (Monterey Bay) to indicate obstruction lights at the end of the Wharf
4. Limit anchorage on the west side of the Wharf to outside 200 feet (see recommendation 4, Section 6). Notify the US Coast Guard of such to update the Nautical Chart and US Coast Pilot (companion text book to the Chart)
5. Implement the West Walkway as called for in the Master Plan to protect the west side of the Wharf and buildings

11. PERMITS, CONSULTATIONS, STUDIES, & SUPPORT MATERIALS

11.1 Summary

The purpose of this section is to describe the anticipated environmental review and permitting efforts required to support federal grant funding for the Santa Cruz Wharf Master Plan (April 2014) as discussed at the meeting on March 17, 2014 (at City of Santa Cruz, Economic Development Office).

11.2 Introduction

This section describes the anticipated environmental review and permitting efforts required to support Federal grant funding for the Santa Cruz Wharf Master Plan (April 2014). This is based upon the Administrative Draft Master Plan and discussions with the City (meeting of March 17, 2014).

11.2.1 Scope

- Identify permitting requirements for recommendations, initiatives, design/development standards and best practices as well as correction of deficiencies, repairs and for improvements and upgrades to the Wharf identified in the Engineering Report and the Master Plan.
- Identify all local, state (with emphasis on the Coastal Commission) and federal agencies with regulatory or permitting authority and those agencies requiring stand-alone or interagency consultations.
- Identify all required studies for submittals to permitting agencies for the proposed engineering recommendations and Master Plan improvements.
- Prepare studies in support of required permitting for selected early action components to be implemented within a two-year time frame and determined in consultation with the City and mutual agreement of the Consultant to be within budget parameters for this task. Examples of these kind of studies include noise and water quality turbidity impacts related to pile driving.

11.3 Permitting

The following permitting efforts would be anticipated for the project.

NEPA

The project would be required to comply with the National Environmental Policy Act (NEPA) if any of the following are involved:

- Federal funding
- Federal action (i.e. if a Corps permit for significant fill of waters of the US is required)

- The project has significant impacts on federally listed/threatened/endangered species.

It is likely that one or more of these criteria would apply to the project and would therefore require analysis under NEPA, which could be any one of the following (in increasing order of complexity):

- Categorical Exclusion (CE)
- Environmental Assessment (EA) and Finding of No Significant Impact (FONSI)
- Environmental Impact Statement (EIS)

The NEPA analysis will require the following elements:

1. Identification of a Federal lead agency who will prepare the document
2. Purpose and Need Statement for the Project
3. Development and detailed analysis of alternatives (e.g.: no project, other location, various widths, etc.) showing avoidance, minimization and/or mitigations measures have been considered and that the preferred alternative is the least environmentally damaging practicable alternative (LEDPA).
4. Supporting technical information and/or studies to provide information on the existing setting (cultural/historical/archaeological, biological such as macroalgae and habitat surveys, hydrological, geotechnical, coastal and sea-level rise, noise, traffic, etc.) Much of this information may be available from existing studies (Wharf Master Plan/Engineering report; Desalination DEIR cited below), anticipated additional supporting studies are listed below.
5. A Biological Assessment could be required for species listed under the Endangered Species Act if present within the project area (to be determined at pre-application meeting).
6. Identification of potential impacts of the project (both beneficial and adverse) and any proposed minimization or mitigation measures (including Best Management Practices, monitoring, etc.).

Supporting Technical Studies:

Biological Survey's

a. Macro Algae (Caulerpa/Eel grass)
b. Noise study (possibly, pile driving)
c. Essential Fish Habitat Assessment (possibly)

Sea Level Rise

The Project will most likely require a US Army Corps of Engineers (USACE) Individual Permit and NEPA compliance should be coordinated early to clearly outline the lead and requirements for compliance.

It is desirable and planned to have a pre-application meeting with the likely regulatory agencies that would be involved. Many of the specifics of the process would likely be identified or strongly suggested during that meeting:

1. Level of NEPA analysis (anticipate an EA/FONSI)
2. Federal Lead agency for NEPA review (USACE or other)
3. Identification of species of concern (benthic, fisheries, marine mammals, etc.)
4. Potential impacts to sensitive species and habitat, historical/archaeological resources, public uses, the human environment, etc.
5. Other specific concerns

The following agencies, their jurisdictional authority and permit that may be required for the project are shown in the following table:

Agency	Authority	Permit Required
U.S. Army Corps of Engineers Consultation with:	Section 10 Rivers and Harbors Section 404 of the Clean Water Act	Individual Permit
NOAA/National Marine Fisheries Services	Marine Mammal Protection Act, Magnuson-Stevens Fisheries Conservation and Management Act	
NOAA/NMFS/Monterey Bay Marine Sanctuary		
US Fish and Wildlife Service	Endangered Species Act	
California Department of Fish And Wildlife	California Endangered Species Act	
California Coastal Commission	California Coastal Act 1976	Coastal Development Permit/Inclusion to Local Coastal Plan
Regional Water Quality Control Board	Section 401 of the Clean Water Act, Porter-Cologne Water Quality Control Act	Water Quality Certification
Lease of State Lands (was Santa Cruz Harbor deeded to City by State Lands?)	California State Lands Commission	
City And County of Santa Cruz	Municipal Code	Building Permit Planning Approval

The project will also need to comply with the state California Environmental Quality Act (CEQA). Both NEPA and CEQA review and documentation can often be combined even though some discrepancies exist between the regulations. Consideration of joint documentation needs should be addressed early within the planning phase for all supporting studies.

The pending desalination project for the Santa Cruz-Soquel Creek Water District (scwd2) has recently completed studies under the CEQA process. A significant element to that proposed project is the consideration of an open water intake pipeline under the Santa Cruz Wharf. It is anticipated that the information gathered for the scwd2 project would also provide recent baseline data for the Wharf Master Plan NEPA/CEQA efforts.

11.4 Conclusions and Recommendations

It is recommended that a preliminary scoping meeting be conducted with the agencies described in this section to identify agency concerns and confirm level of analysis they may require.

though some discrepancies exist between the regulations. Consideration of joint documentation needs should be addressed early within the planning phase for all supporting studies.

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11.4 Conclusions and Recommendations

It is recommended to follow up with agencies based upon items discussed at preliminary scoping meeting held August 22, 2014 to further identify agency concerns and confirm level of analysis they may require.