
The existing Anthony's Fish Grotto restaurant, looking northeast.

WATERFRONT FACILITY INSPECTION AND ASSESSMENT
FUTURE PORTSIDE PIER RESTAURANT

TUCKER SADLER ARCHITECTS
ARCHITECTURE > PLANNING > INTERIORS > DEVELOPMENT SERVICES

Prepared By:

moffatt & nichol

1660 Hotel Circle North, Suite 500
San Diego, CA 92108

March 18, 2016

moffatt & nichol

1660 Hotel Circle North, Suite 500
San Diego, CA 92108
(619) 220-6050 Fax (619) 220-6055

www.moffattnichol.com

March 18, 2016

Tucker Sadler Architects
34675 Golden Lantern St
Dana Point, CA 92629

Attention: Mr. Greg Mueller, Principal

Subject: Waterfront Facility Inspection and Assessment (WFI)
Future Portside Pier Restaurant San Diego, CA

Dear Mr. Mueller,

In accordance with contract requirements, a field investigation and assessment of the existing Anthony's Restaurant pile-supported platform (platform) has been completed. The inspection scope of work was taken from Moffatt & Nichol's (Matthew Martinez) email fee proposal dated January 12, 2016 – see the "Inspection Scope of Work" paragraphs below.

BACKGROUND AND DESCRIPTION OF FACILITIES

The current Anthony's Fish Grotto lease with the Unified Port of San Diego (Port) expires in January, 2017. As part of an ongoing effort to promote quality development of the Embarcadero at San Diego Bay, the Port is awarding redevelopment rights for the leasehold to The Brigantine Inc. Brigantine has retained Tucker Sadler Architects (TSA) to develop a design that has met with the Port's preliminary approval. The purpose of this waterfront facility inspection (WFI) is to establish the condition of the existing Anthony's Restaurant pile-supported platform and its suitability (or lack thereof) to support the proposed Brigantine design – referred-to as "Portside Pier".

Anthony's Fish Grotto is constructed on a pile-supported platform located in San Diego Bay adjacent to the Embarcadero Wharf near the intersection of W. Ash St. and Harbor Drive. The platform structure is approximately 216-ft long by 104-ft wide, see Photo 1. The structure is supported by prestressed concrete piles with a timber superstructure, see typical below deck framing arrangement as shown in Photo 2.

A cantilevered deck addition to the original structure (Photo 3) is located on the north side that was part of the "Star of the Sea" room. Another deck extension is located on the south side for Anthony's Fishette (patio-style dining). Three prestressed concrete guide piles are located adjacent to the Fishette patio. These piles once supported a floating dock and gangway used for recreational boat access to the restaurant. The floating dock was destroyed during a storm in early 2016. A current view of the south side can be seen in Photo 4.

Vertical loads from the building and platform structure are carried by 16-in. octagonal prestressed concrete piles. Lateral loads were designed to be resisted by and four concrete pile caps each supported by four 24-in. batter piles. The timber deck is composed of glue-laminated ("glulam") pile caps spanning along Gridlines A, C, E, G, and J and around the perimeter, see Figure No. 1. Glulam and timber girders are anchored by steel angle brackets to the pile caps and span intermittently along Gridlines 1 through 17. The pile caps and girders support 3x16 stringers at 16 in. on-center spacing with plywood decking on top as an interior floor underlayment. 2x timber (or plastic) decking is used at exterior locations.

Photo 1. Anthony's Fish Grotto in San Diego Bay along the Embarcadero.

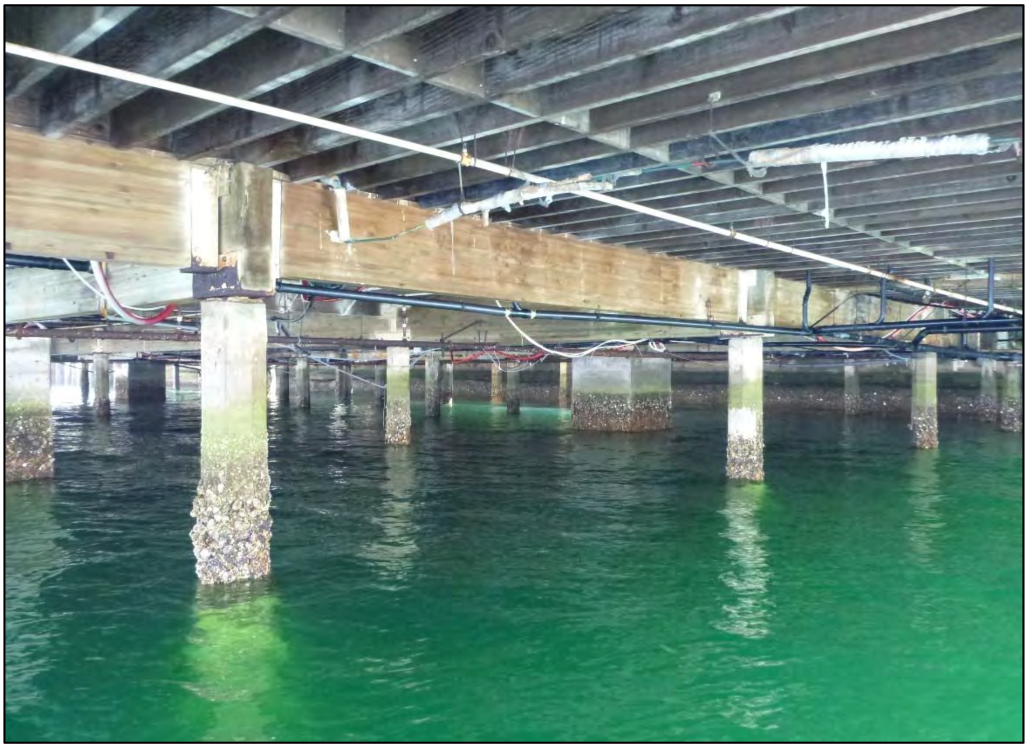


Photo 2. Typical below deck framing along girder at Gridline 3.

Photo 3. Cantilevered deck on north side. An inverted steel truss is added to the glue laminated beam supporting the deck framing.

Photo 4. South side of the structure. The arrows point to three guide piles where floating docks were previously attached.

PERTINENT DOCUMENTS

The following documents were considered as a part of this investigation:

1. Letter Report titled, "North Embarcadero Visionary Plan – Phase 1 Inspection of Embarcadero Facilities," dated December 2008, prepared by Moffatt & Nichol.
2. Plans titled "Anthony's Fish Grotto Embarcadero Harbor Drive", San Diego, CA, dated 6 30, 1965, prepared by Frederick Liebhardt, Eugene Weston III AIA Architects and John Kariotis and Associates (sheets S-1 through S-11).
3. Plans titled "Reconstruction of Apron Wharf", dated July 31, 1974, prepared by Blaylock - Willis and Associates (District Drawing No. 1081). These are the structural drawings for the mid-1970s reconstruction of the Embarcadero Wharf structure.

INSPECTION SCOPE OF WORK

The Scope for this effort included above and below water inspection of the pile supported platform. Specific requirements for the effort are as follows:

- ***Background.*** *This work is to be performed in support of the aforementioned Portside Pier project under development by Tucker Sadler Architects. The existing substructure (prestressed concrete piling), deck platform and building superstructure were built from plans titled*

"Anthony's Fish Grotto – Embarcadero Wharf" in 1966 - making the existing structure 50-years old. The field investigation is to be conducted in order to establish the condition of the existing piling, deck structure and certain utilities, and to determine the suitability of these elements for incorporation into the proposed design.

- ***Field investigation**. Perform Inspection in general conformance to the requirements of a "Routine Inspection" as defined by the American Society of Civil Engineers (ASCE) Manual of Practice 130 "Waterfront Facilities Investigations and Assessments". Inspect the Anthony's Leasehold structure and the Embarcadero Bulkhead structure directly adjacent to the building. Exception – the marina floats adjacent to the restaurant are scheduled for replacement and will not be inspected (floats were lost in a storm prior to the inspection).*
    - ***Underwater inspection**. Perform Level I inspection (general swim-by observation) of 100% of the piling. Perform Level II (limited removal of marine growth and concrete substrate examination of 10% of the piling. Take representative photos.*
    - ***Above water** – Below deck. Examine all visible portions of the underside of the restaurant and adjacent Embarcadero Wharf structures. Also examine and locate visible portions of utilities (gas, potable water, waste water, and electrical conduits) extending out of the Embarcadero bulkhead for general condition and to determine suitability for re-use with the proposed improvements.*
- ***Above Deck**. Inspect exposed portions of the Embarcadero Wharf Deck and exterior decks of the restaurant. Restaurant interiors are not to be inspected.*
- ***Letter Report.** Provide a letter report containing narrative description of findings with a particular emphasis of the suitability of re-use of the piling and pile cap elements for support of the new structure (to be designed for a fifty-year service life). Provide recommendations for repair of the piling and pile caps (if necessary). Provide discussion regarding condition of the deck structure as relates to potential re-use with the proposed improvements, provide photographs and figures. Use the existing structural record drawings to record location of photos and damage.*

CONDITION ASSESSMENT RATINGS

The Naval Facilities Engineering Command (NAVFAC) has developed an overall condition rating system that provides standard condition rating classifications for all waterfront facilities. In the use of this system, each facility is given an overall rating based on the observed conditions. The six terms used to describe the conditions of a structure are described below and will be used in describing structural elements in this report.

- **"Good"** – No problems or only minor problems noted. Structural elements may show some very minor deterioration, but no overstressing observed.
- **"Satisfactory"** – Minor to moderate defects and deterioration observed, but no overstressing observed.

- **"Fair"** – All primary structural elements are sound, but minor to moderate defects and deterioration observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the load-bearing capacity of the structure.
- **"Poor"** – Advanced deterioration or overstressing observed on widespread portions of the structure.
- **"Serious"** – Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible.
- **"Critical"** – Very advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur.

FINDINGS

The field investigation was conducted between February 25th and March 1st, 2016 by M&N engineer-divers. In general, the structure is in fair condition. Reference is made to the photos which provide general descriptions for findings of this investigation. The photos will be used to guide the basic narrative for this report with additional supplemental discussion as necessary.

An inspection of the utility services extending out from the Embarcadero Wharf bulkhead and providing service connections to the restaurant was also performed. The utilities are as follows: fire water, potable water, electrical, telecom, natural gas, and sewer. The utilities were inspected to verify their current condition and capacity to handle future improvements. It can be noted that the utilities investigation unveiled some areas of concern regarding the condition of the existing pipe, pipe hangers, and pipe fittings.

Item 1 – Concrete Piles

Piles. The structure is supported by 16 in. prestressed concrete octagonal piles. The piles were inspected above and below water. A typical Level II inspection location (removal of marine growth) is seen in Photo 5. Other than the two piles described in "Pile Encasements" below, the piles are in good condition with no defects noted. See Figure 1 in Appendix A: Figures for pile plan layout.

Grout Caps. The piles have a built-up (approximately 2.5-in. high) grout cap which provides consistent elevation for the pile cap connections, see Photo 6. The built-up grout caps are in poor condition with corrosion spalls or cracks noted on approximately 40% of the total. The spalls are a result of steel expanding as a result of corrosion, which tends to delaminate and "pops-off" the concrete. The corroding steel is the result of a combination of chlorides, moisture, and oxygen which serves to create a galvanic cell. This type of degradation is common in a marine environment and is often exacerbated by cracking that originates from impact or pile driving. See Appendix B: Background Information for further discussion of concrete deterioration.

The grout caps provide bearing for a partially embedded steel bearing plate and encase a 1-in. steel pin which connects the pile to the superstructure, see Item 7 – Connections, for in-depth discussion.

Corrosion of the bearing plate and pin connector is contributing to the spalling and cracking of the grout caps, see Photo 7.

Pile Encasements. There are two concrete pile encasements at Gridlines 14:A and 15:E that extend from the top of the pile down approximately 5 ft. The pile encasements exhibit significant cracking at Pile 15:E (Photo 8) and one has failed (Pile 14:A, Photo 9).

Both piles with encasements have steel bracing that extends from below the encasements up to the pile caps, (seen in Photo 8 and Photo 9). The braces are severely corroded and are in serious condition. The purpose of these braces was to provide additional stability for the pile, while the concrete encasement repair was being performed.

Guide Piles. Three guide piles located on the south side of the restaurant once supported a floating dock to be used by restaurant patrons. The floating dock has since been destroyed earlier this year and remnants of the dock/pile connection were found below water at each guide pile. The guide pile near Grid 0:C had an 8-ft portion of timber stringer attached to the guide pile bracket (Photo 10) as well as additional framing and debris at the bottom.

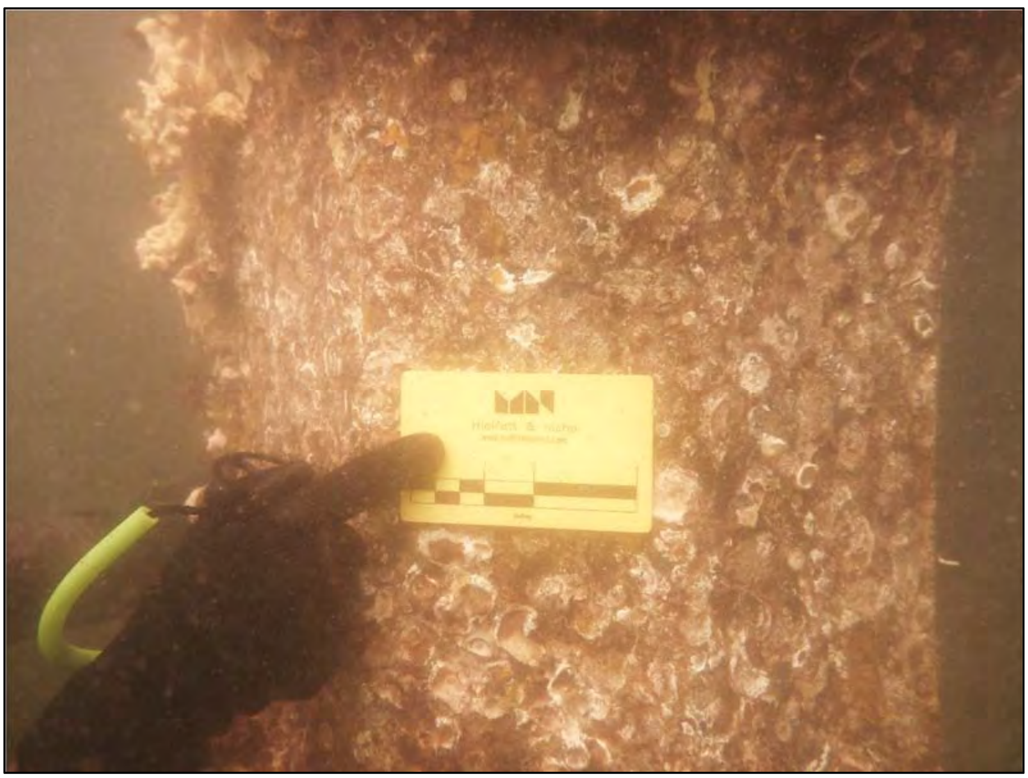


Photo 5. 16-in. octagonal Pile 0:F at a typical Level II cleaning location below water.

Photo 6. The Engineer points to a diagonal crack in the 2.5-in. grout cap at Pile 3:E.

Photo 7. Both the pile cap bracket and the bearing plate are heavily corroded causing spalling of the grout cap at Pile 3:C.

Photo 8. The red arrows point to cracks found in the pile encasement at Pile 15:E.

Photo 9. Failed pile encasement at Pile 14:A.

Photo 10. Guide pile in the vicinity of Grid 0:C. The engineer is holding the guide pile bracket and attached timber stringer.

Item 2 – Batter Piles

Lateral stability for the platform is provided by four strategically located concrete pile caps (5:C, 5:G, 13:C, and 13:G). Each pile cap is supported by four battered piles driven at an angle approximately 15 degrees from vertical see Photo 11. These batter piles brace the pile cap and platform/building above against wind and earthquake loads. See Figure 1 in Appendix A: Figures for pile plan layout.

Batter Piles. The Record Drawings (Pertinent Document 2) indicate that the batter piles are 16-in. octagonal prestressed concrete piles, e.g. similar to the vertical piles. Level I and II inspection determined that the piles are actually 24-in. diameter concrete-filled steel pipe. This is a deviation from the Record Drawings. At the mid-height Level II elevation, the engineer-diver hammered through the partially corroded steel pipe to expose the competent concrete fill, see Photo 12. The cleaning band at the top revealed steel pipe as well, see Photo 13. The remaining batter piles were then checked to determine that this pile was not an anomaly. All batter piles were found to be 24-in. diameter steel pipe, see Photo 14 and Photo 15. At the majority of the piles, the engineer was able to expose competent concrete beneath the steel pipe, see Photo 16.

Concrete Pile Cap. The concrete pile caps are 6-ft wide by 7-ft and 3.75-in. tall and are in poor condition. Both outboard (Gridline C) pile caps exhibit significant corrosion spalling on the top corners. The pile cap at Grid 5:C exhibits an open corrosion spall 2-ft high and 10-in. deep along the entire west face, see Photo 17. At Grid 13:C, the pile cap has a similar defect and has been previously repaired. The repair is also failing, see Photo 18.

Photo 11. Typical batter pile group at Grid 13:C.

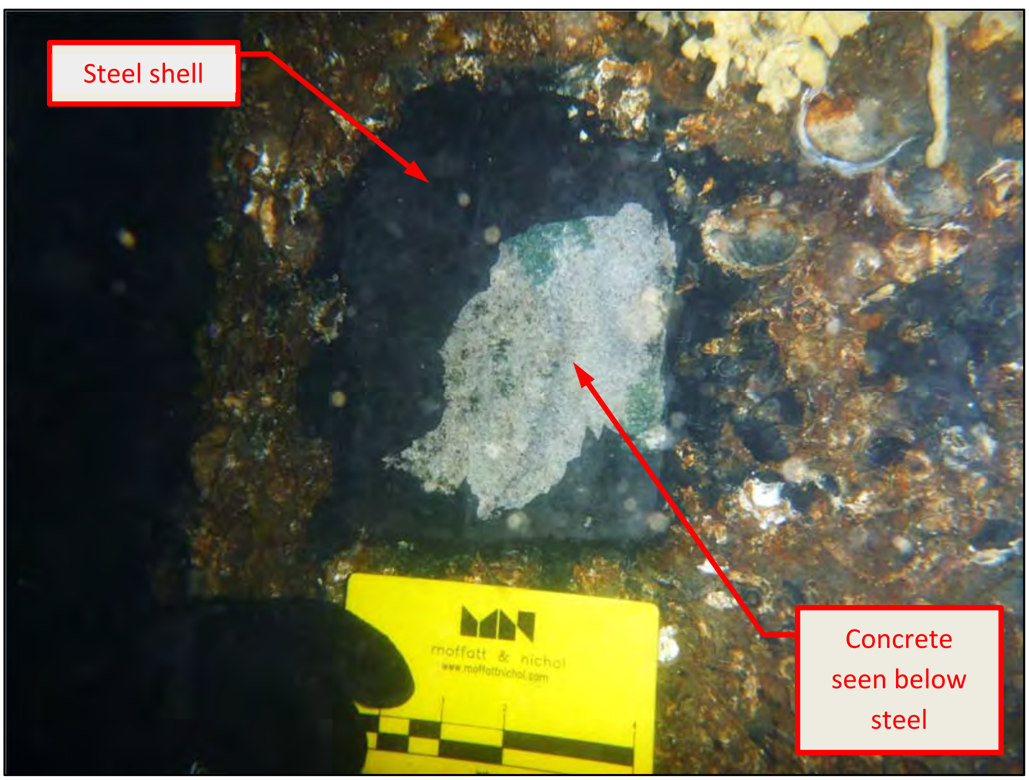


Photo 12. Mid-height Level II at batter Pile 13:G. The partially corroded steel pipe was removed to expose the interior concrete fill.

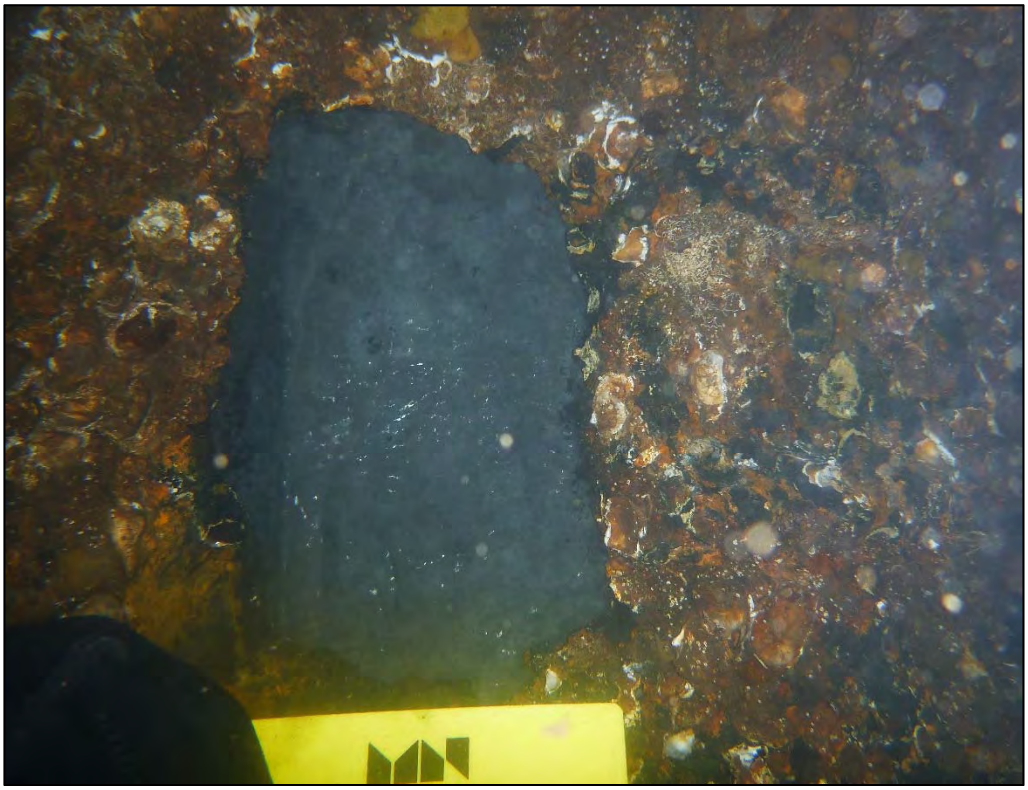


Photo 13. Cleaning band at the top of Pile 13:G where a steel shell was exposed.

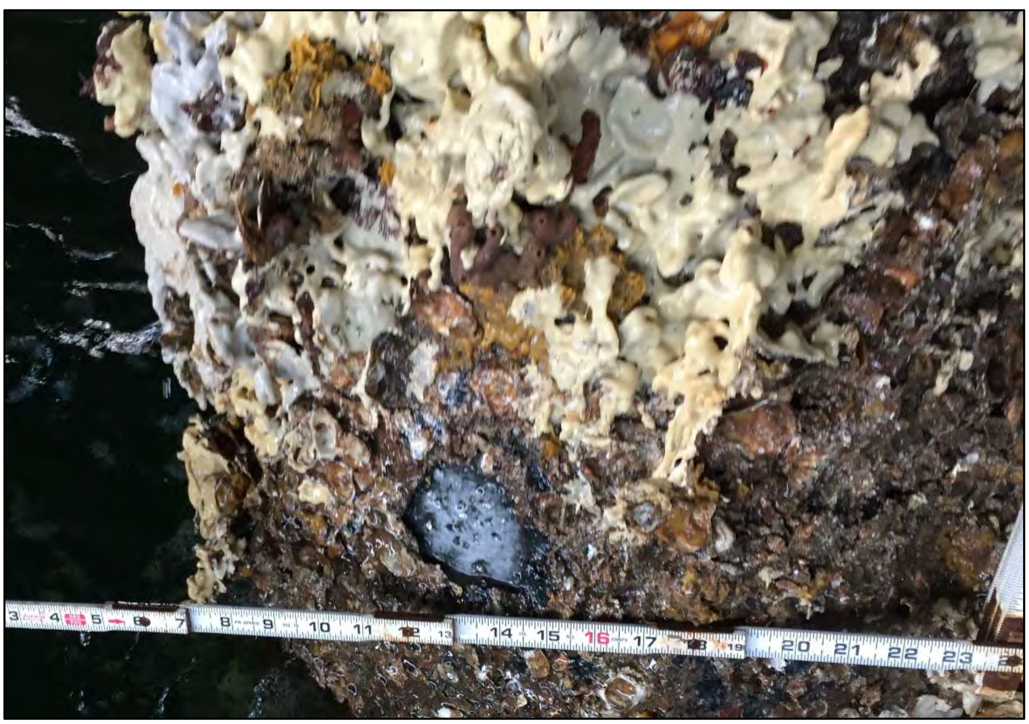


Photo 14. The photo shows the top of the southeast pile of the batter pile group at Grid 5:G exposed at extreme low tide. The steel pipe pile is visible after removal of the marine growth.

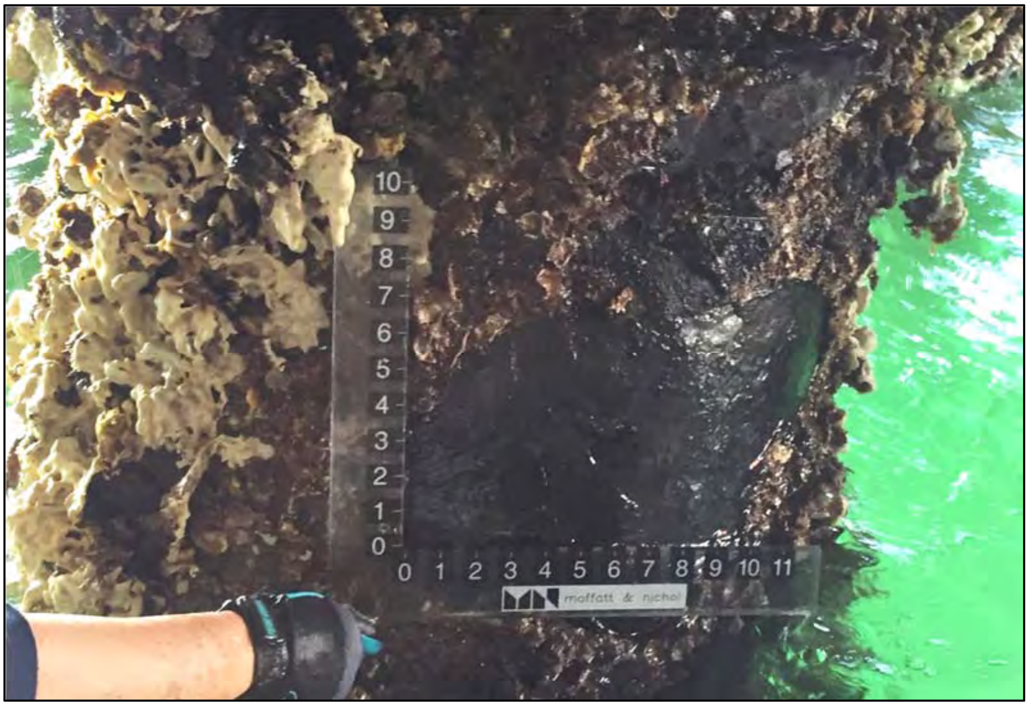


Photo 15. The steel pipe is exposed at the southeast pile of the group at Grid 13:C after removal of marine growth at low tide.

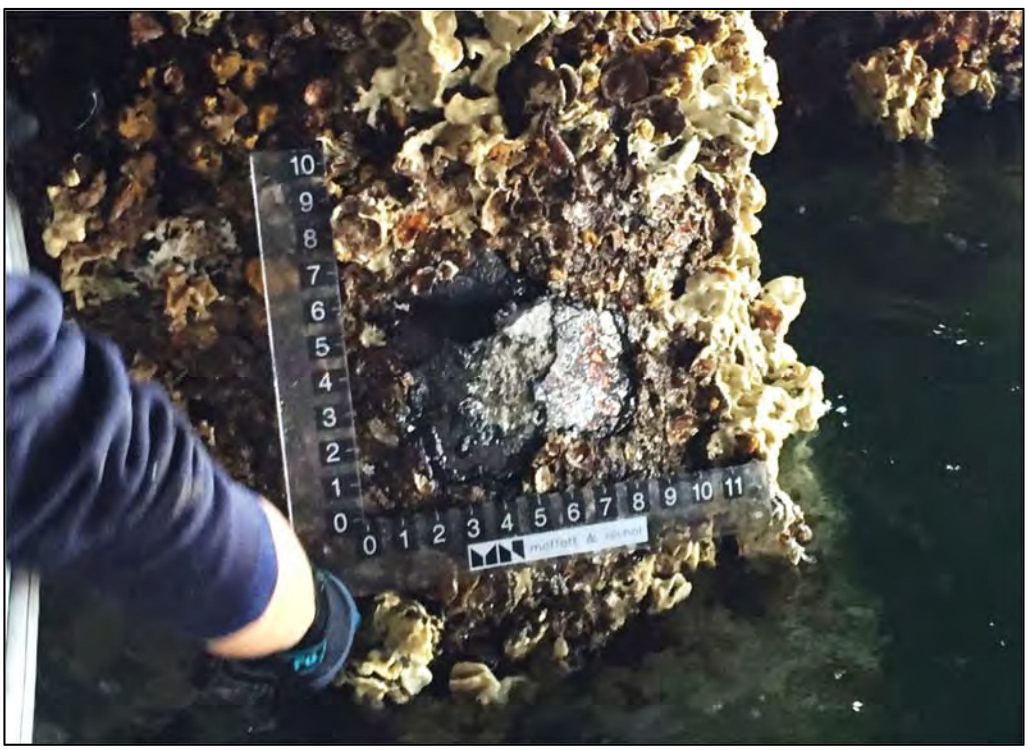


Photo 16. The corroding and delaminating steel pipe is peeled back revealing a concrete substrate (Gridline 13:G, NE Pile).

Photo 17. Open corrosion spall at Pile Cap at 5:C.

Photo 18. Failed repair and open corrosion spalling at Pile Cap at 13:C.

Item 3 – Glulam Pile Caps

The pile caps are in satisfactory condition with minor defects found, primarily due to soft rot. All of the pile caps are pressure treated. The pile caps located on the perimeter of the building are painted brown to improve appearance. See Figure No. 2 in Appendix A: Figures for the framing plan layout.

Soft Rot. The paint on the perimeter pile caps is failing in several areas with soft rot noted beneath (Photo 19). Soft Rot is caused by fungi related to molds. Soft rot typically is relatively shallow; the affected wood is greatly degraded and often soft when wet, but immediately beneath the zone of rot, the wood is typically firm. Because soft rot usually is rather shallow, it is most likely to damage relatively thin pieces of wood such as stringers. It is favored by wet situations but is also prevalent on surfaces that have been alternately wet and dry over a substantial period.

The pile caps that are affected by soft rot typically exhibit less than 10% of section loss on the bottom face. The rot is seen on the perimeter glulam pile caps or underneath areas that have been exposed to prolonged moisture (leaks or drains).

Rotation. The pile cap at Bent A spanning from Gridline 1 to 5 is rotated inward, see Photo 20.

Splits. In several places, splits were noted in the pile cap at the top bolts of the pile cap brackets, see Photo 21.

Photo 19. Pile Cap at Bent A spanning from Gridline 1 to 3 with failing paint and soft rot noted at the areas of exposed wood.

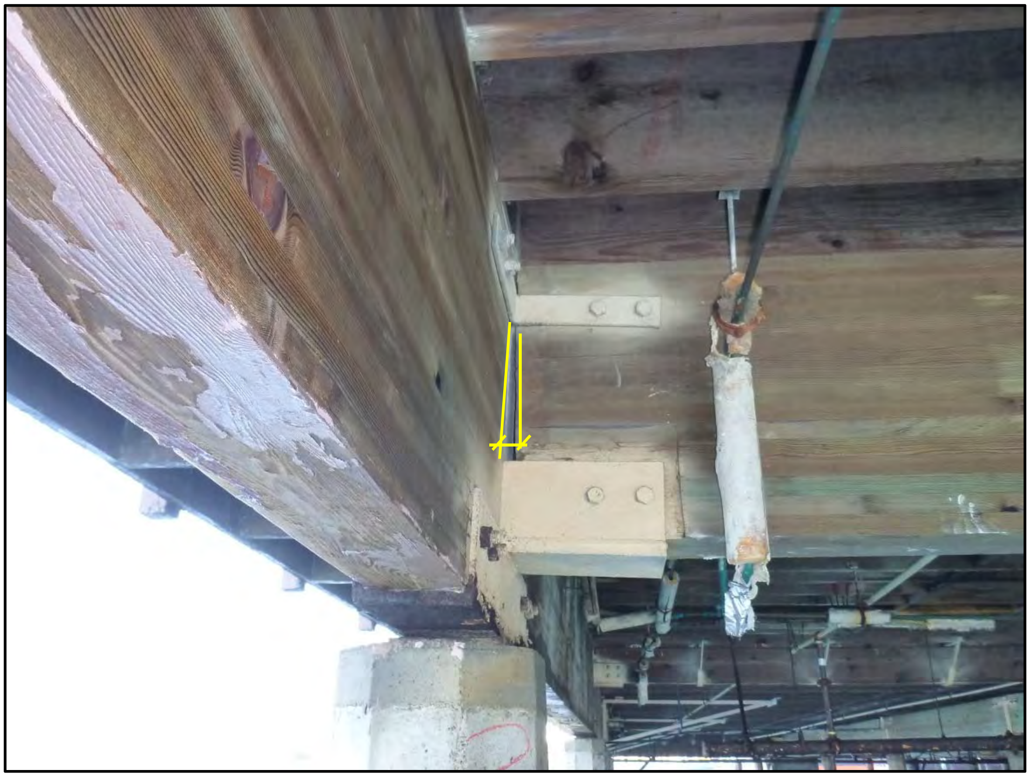


Photo 20.The lines highlight rotation of the pile cap (left-hand side) at the connection above Pile 3:A.

Photo 21. Split noted in the pile cap just to the right of Pile 17:E.

Item 4 – Girders

For the purpose of this discussion, "girders" are defined as smaller glue laminated members or 6x timbers intersecting with the glulam pile caps, but not bearing directly on top of the pile. Girders typically support smaller framing such as beams or stringers. The girders are in satisfactory condition with dry rot affecting less than 10% of the members. In the few areas where dry rot was noted, the damage is primarily on the bottom and affects less than 10% of the cross-section. Some splits were noted in the girders, the worst being located at Gridline 13 between Bents G and H. The 6x14 member spans 12 feet and has a split originating at the girder-to-pile cap connection and extends 5 feet, see Photo 22. See Figure No. 2 in Appendix A: Figures for the framing plan layout.

Photo 22. Split in the 6x14 girder above the batter pile group at Grid 13:G.

Item 5 – Timber Stringers

The stringers are in satisfactory condition (Photo 23) with the exception of some members affected by dry rot. The affected stringers are in fair condition. Areas of soft rot were primarily limited to the center portion of the structure and overhangs (Photo 24). An example of stringers affected by soft rot is seen in the vicinity of Grid 3:J in Photo 25. In several cases where soft rot is prevalent, stringers have been "sistered" (spliced with an additional member) or replaced entirely, see Photo 26. See Figure No. 2 in Appendix A: Figures for the framing plan layout.

Photo 23. Example of stringers in satisfactory condition (in the vicinity of Grid 15:E).

Photo 24. Soft rot noted on stringers supporting the exterior back in the vicinity of Grid 1:J.

Photo 25. Soft rot on stringers noted in the vicinity of Grid 3:J.

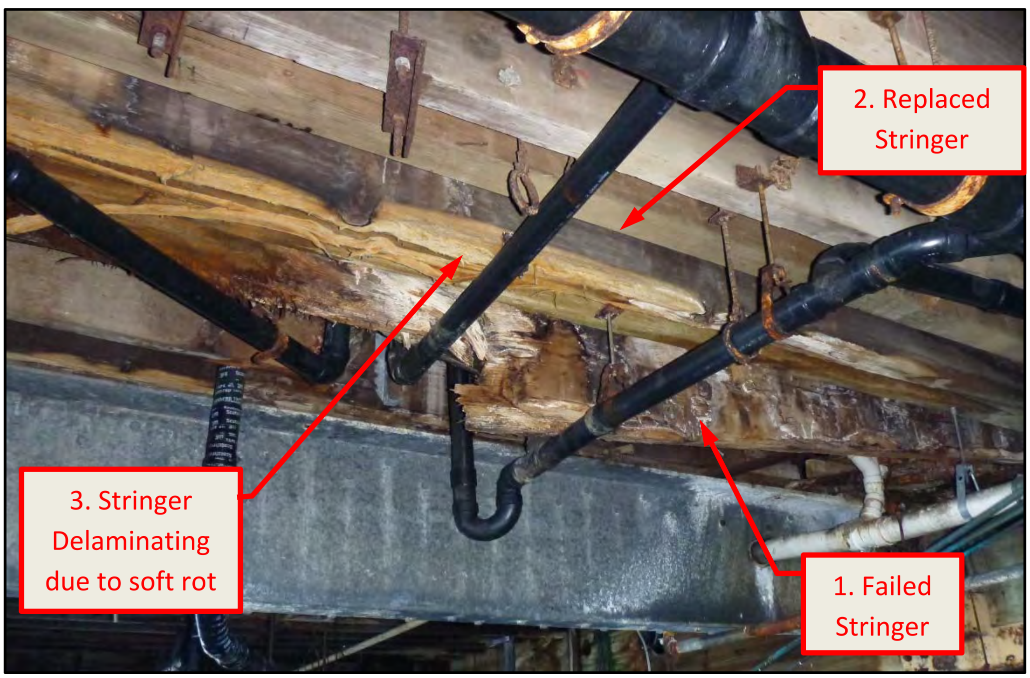


Photo 26. A failed stringer (1) has been replaced with a newer stringer (2). Damage due to soft rot is seen on an adjacent stringer (3).

Item 6 – Plywood

The plywood is in fair condition with the exception of some areas affected by dry rot that are in poor condition. Areas of soft rot was noted and primarily limited to the center portion of the structure where utility penetrations occur. An example of plywood in poor condition is seen in the vicinity of Grid 10:F in Photo 27.

Photo 27. The plywood surrounding the utility is severely affected by soft rot.

Item 7 – Connections

There are several types of steel connecting hardware under the structure. In the marine environment these elements typically fail as a result of steel corrosion. Galvanizing typically delays the onset of corrosion, and application of coatings can be somewhat effective in mitigating rust. Most of the connectors are galvanized, with many of the larger pieces having been coated as a maintenance measure. Appendix B defines corrosion loss by four levels of degradation, i.e. “minor”, “moderate”, “major”, and “severe”. The corrosion rating for most of the connectors falls within the “minor” or “moderate” categories with up to 100% coating loss and moderate corrosion on the bottom of connections, see Photo 28. A lesser percentage of the connectors are rated as “major” or “severe”. Some of the specific components are listed and rated below.

Pile Cap Connection & Brackets. Reference is made to detail 1/S11 taken from the Record Drawings and presented as Photo 29. The glulam pile caps are anchored to the piles with a 1-in. drift pin embedded in both the concrete pile and the glulam cap. A 9-in. square by 1 1/8-in. thick bearing plate sits on the grout cap. A 3 1/2-in. diameter by 3/8-in. plate washer is placed between the bearing plate and the glulam connector. See Photo 30 for labeled components. While many of the connector surfaces are in "fair" condition as defined in Appendix B, the connectors are assigned an overall corrosion loss rating of "moderate" because of the advanced corrosion noted on the pile cap connection hardware and the bottom plate of the connection bracket. An example of moderate corrosion is seen in Photo 31.

Batter Pile Cap Connection. The batter pile cap connection utilizes a 10-in. square tube which is embedded in the concrete pile cap and bolted to the glulam pile cap with steel brackets, see Detail 1-S9 from Pertinent Document 2 for more detail. The connection is in fair condition with minor to moderate corrosion noted on the bottom portions of the lower bracket, see Photo 32. Several of the bolts at the bracket to concrete cap connection are heavily corroded.

Cantilever Girder Hinge Connectors. The hinge connectors are in fair condition with moderate corrosion limited to the bottom of the hardware, see Photo 33.

Girder Knee Braces. The girders have knee braces to stiffen the deep cross-section (five in each bay) on Gridlines 5 and 13. The braces are in satisfactory condition with the exception of the easternmost bracket in each bay which exhibits moderate corrosion, see Photo 34.

Joist Hangers. The joist hangers are in poor condition – primarily because the hangers were fabricated from thin sheet metal. Severe corrosion was noted on the bottom sections of joist hangers and in some cases the bottom section has completely corroded away, see Photo 35.

Built-up Pile Cap. An inverted steel truss comprised of Hollow Structural Sections (HSS) has been added underneath the pile cap spanning from G to J on Gridline 17. This was added to provide additional support for the cantilevered deck addition associated with the Star of the Sea room. The truss is in fair condition with moderate corrosion, see Photo 36.

Steel Retrofit Section. Galvanized steel cold-formed Z-sections have been added in the bay bounded by Grids E, G, 7, and 9. The timber framing in this area shows signs of more advanced soft rot, and it is speculated that the cold-formed framing was added to provide supplemental load capacity. The retrofit, seen in Photo 37, is in fair condition with localized areas of minor corrosion.

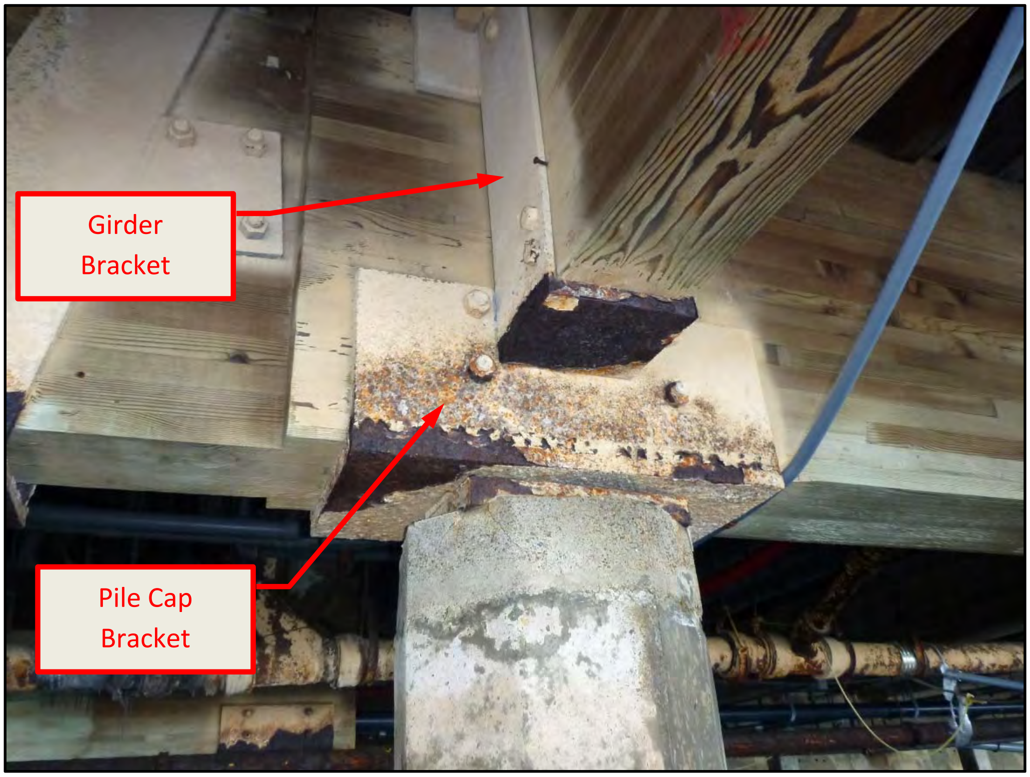


Photo 28. Typical condition where minor corrosion is noted on most surface areas of the steel connectors and moderate corrosion on the bottom plates (Pile 7:G).

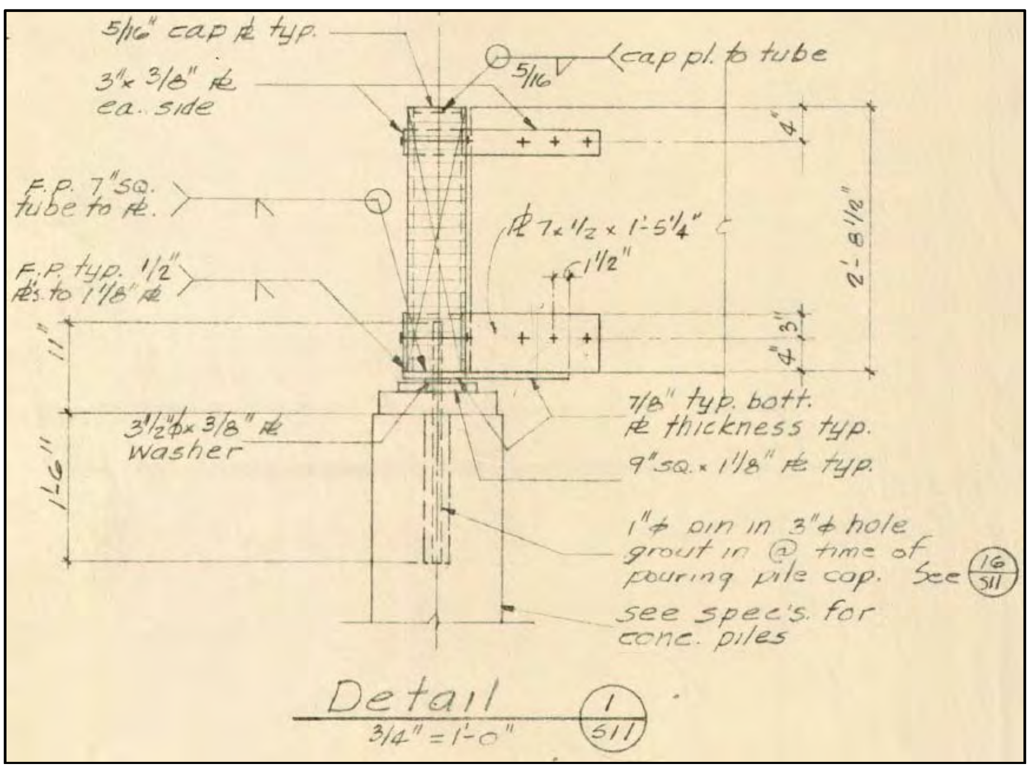


Photo 29. Pile to glulam connection detail from Pertinent Document 2.

Letter to Greg Mueller, Principal
Waterfront Facility Inspection and Assessment – Future Portside Pier Restaurant
March 18, 2016

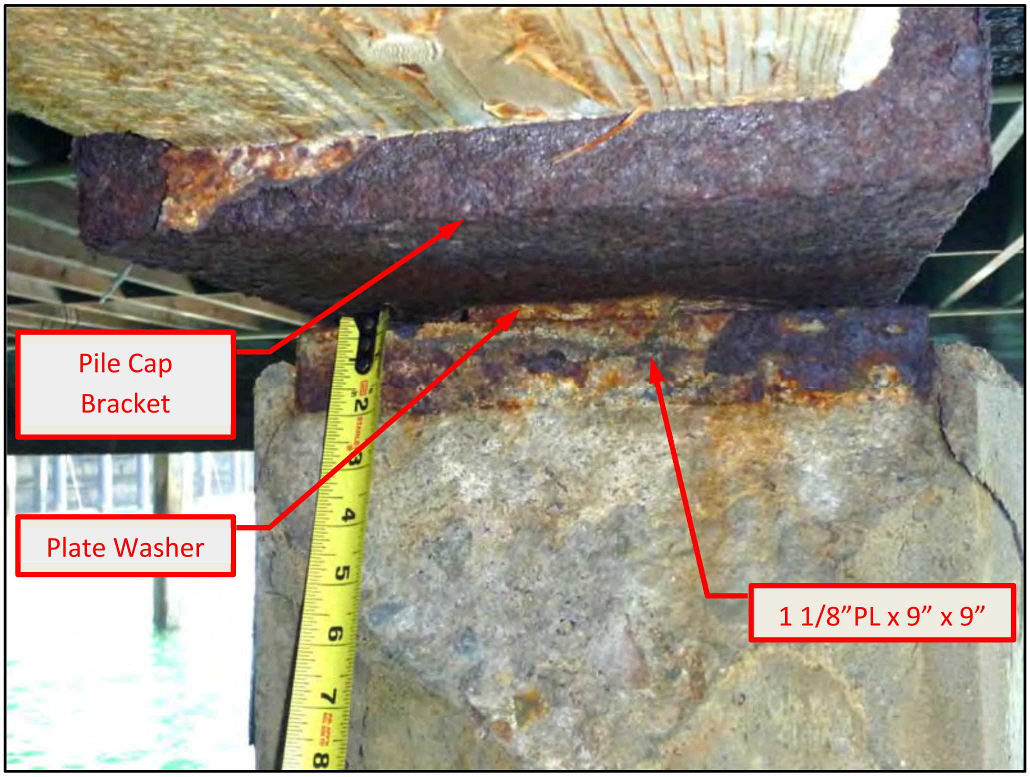


Photo 30. Pile Connection and bracket at Pile 3:C.

Photo 31. Pile 3:J. Moderate corrosion noted on the pile cap connection hardware and the bottom plates of the pile cap bracket and girder bracket.

Photo 32. Pile cap connection above batter pile group at Grid 5:C.

Photo 33. The arrow points to a typical cantilever hinge connector located to the north of Grid 7:G.

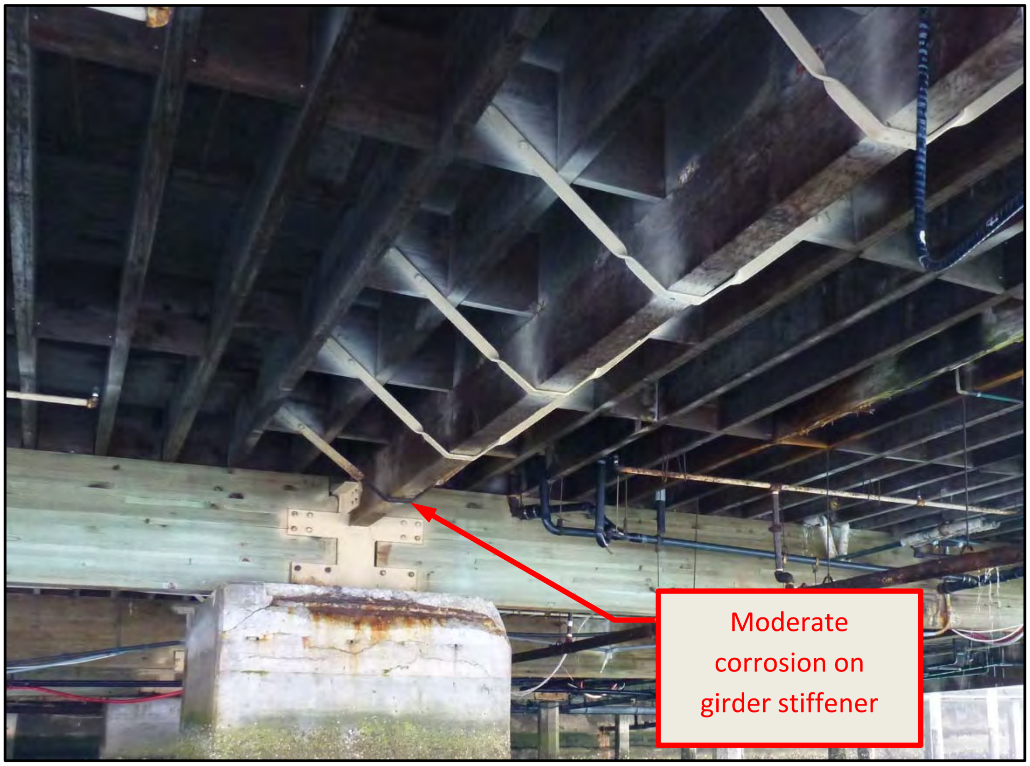


Photo 34. Girder stiffeners along the girder spanning from A to C on Gridline 5.

Photo 35. Corroded Joist hangers in the vicinity of Grid 2:B. the red arrow points to a hanger which has lost 100% of the bottom section.

Photo 36. Inverted truss pile cap stiffener at Grid H:17.

Photo 37. The photo shows the retrofit with cold-formed steel Z-sections in the area bounded by Grids E, G, 7, and 9.

Item 8 – Fire Water

An existing fire water system was found approximately 3 ft below the bottom of the wharf deck, approximately 1-ft north from Embarcadero Wharf Pile No. 69 (reference Pile ID in Pertinent Document No. 3). The system is composed of a 6-in. Cement Mortar Pipe (CMP), galvanized hangers and epoxy coated ductile iron fittings. The pipe is generally in fair condition. There are areas of deterioration that can be seen on the pipe – particularly the segment extending through the Embarcadero bulkhead that shows evidence of rust. The rust is indicative of corrosion beneath the mortar lining. This segment of the pipe is in poor condition.

The pipe hangers under the Embarcadero Wharf are in fair condition with some corrosion evident in some of the hardware. The pipe fittings show evidence of heavy corrosion and are in poor condition – see Photo 38 through Photo 40.

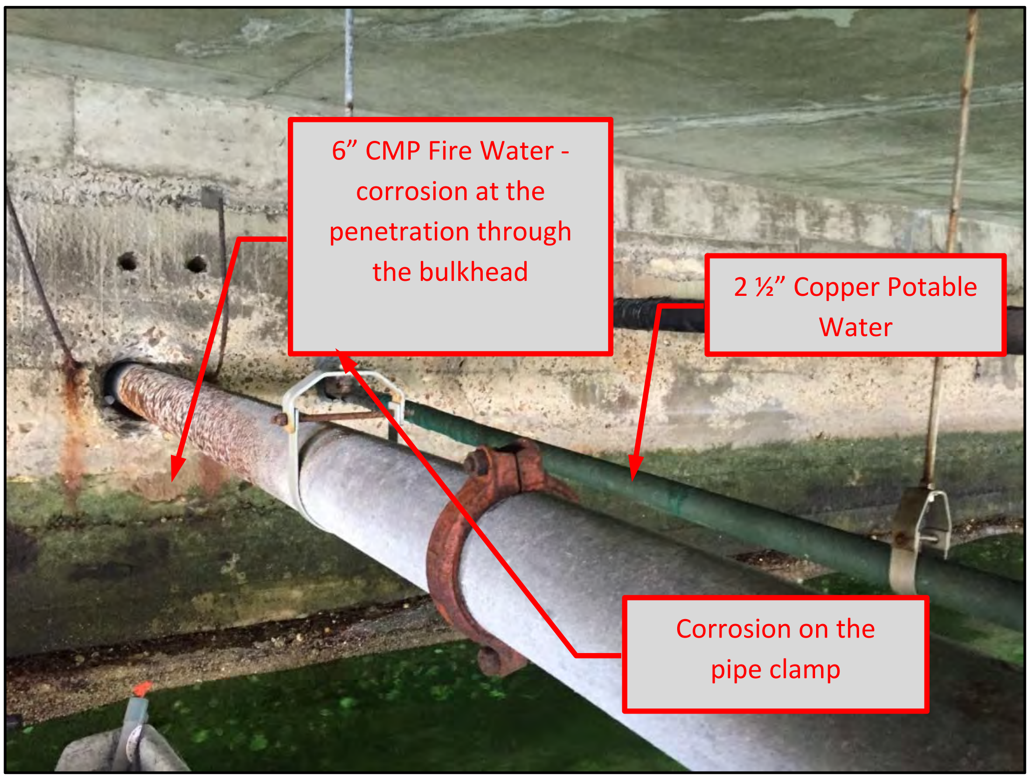


Photo 38. Existing 6-in. CMP Fire Water line showing corrosion on the landside connection of the pipe. The 2 ½-in. potable water line is seen behind the 6-in. pipe.

Photo 39. The 6 in fire water line and 2 ½-in. potable water line is seen extending beyond the Embarcadero Wharf towards the underside of the platform.

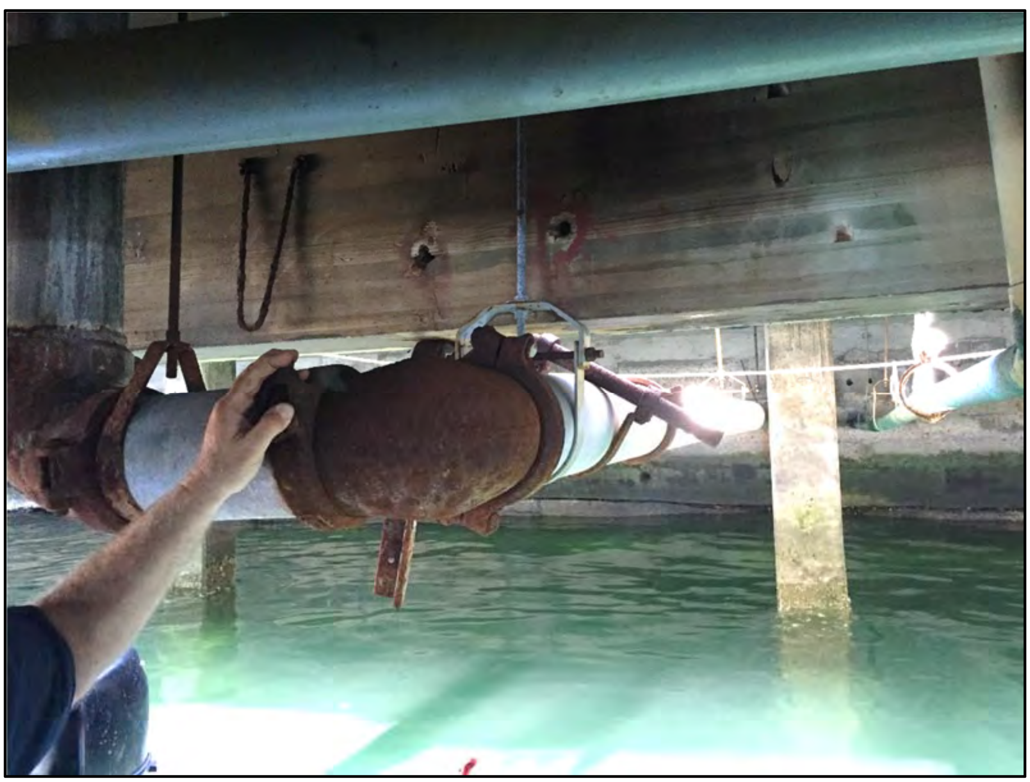


Photo 40. The photo shows corrosion of the elbow fitting on the 6-in. fire water line. The photo was taken looking from beneath the platform back at the Embarcadero bulkhead.

Item 9 – Potable Water

An existing Potable Water system was found approximately 3'-5" below the bottom of the structure and 1-ft south of Embarcadero Wharf Pile No. 69. The system is composed of a 2 ½-in. copper pipe, galvanized hangers and copper fittings. Some areas of deterioration were seen on the pipe. Due to only minor corrosion being identified on the potable water line, the line is in fair condition. The segment of pipe extending through the bulkhead also shows signs of corrosion and can be seen in Photo 38.

Corrosion on the pipe hangers can be seen in Photo 40 and Photo 41. In addition, Photo 41 shows evidence of bending at the lateral fitting. These signs of both bending and corrosion warrants a condition rating of poor for the hangers and fittings.

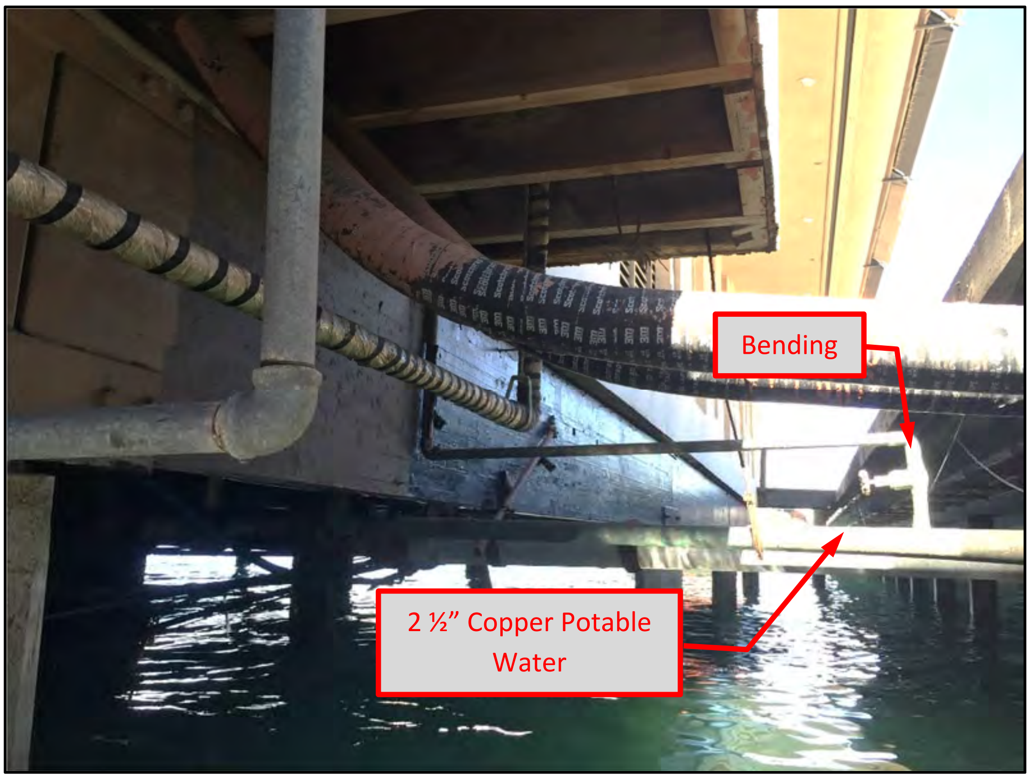


Photo 41. Bending shown in the lateral coming off the 2 ½-in. Potable Water line.

Item 10 – Electrical

An existing electrical system was found approximately 2'-2" below the bottom of the structure and 7-ft south of Embarcadero Wharf Pile No. 69. The system is composed of two 4-in. wrapped conduits, galvanized hangers and epoxy coated ductile iron fittings. Both electrical conduits showed areas of extensive corrosion (see Photo 43) and deterioration of the conduit wrap was also identified. Thus these conduits are classified in poor condition.

Photo 42 and Photo 43 show that the pipe hangers and fittings are in fair condition with corrosion evident in only some of the hardware elements. It is important to note that the spacing and number of the pipe hangers need to be validated.

Letter to Greg Mueller, Principal
Waterfront Facility Inspection and Assessment – Future Portside Pier Restaurant
March 18, 2016

Photo 42. View of the electrical conduits.

Photo 43. Underside view of the electrical conduits showing corrosion damage.

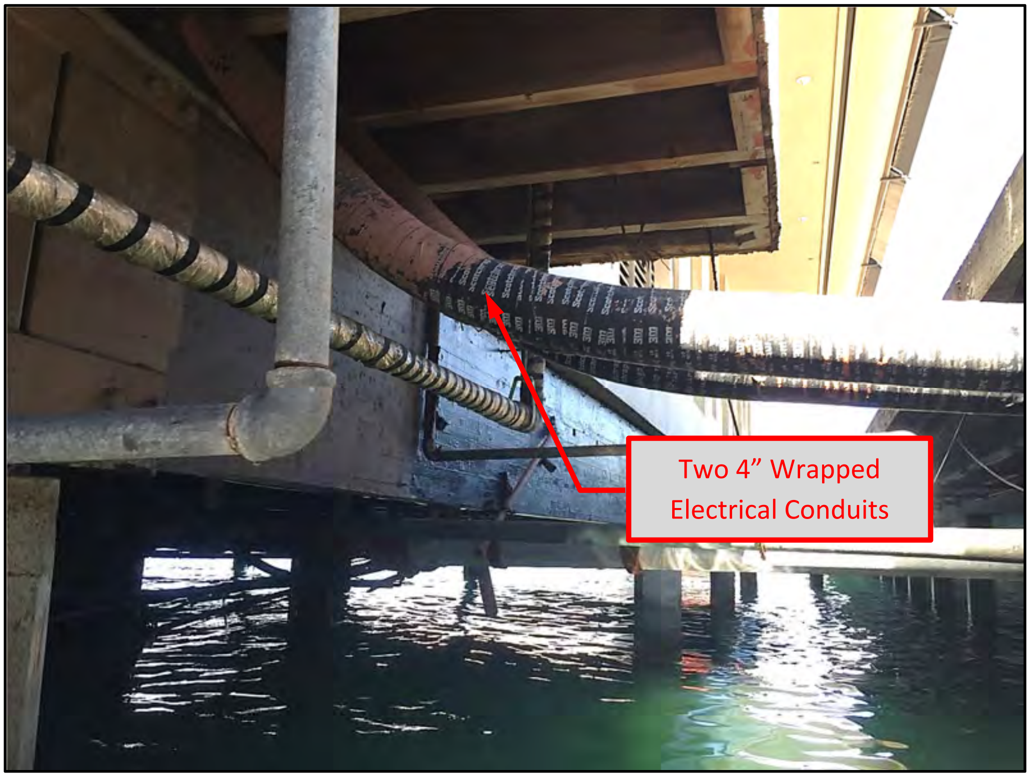


Photo 44. Restaurant connection of the electrical conduits.

Item 11 – Telecom

An existing Telecom conduit was found approximately 1'-8" below the bottom of the existing structure and 9-ft south of Embarcadero Wharf Pile No. 68. The system is composed of a 2 ½-in. PVC pipe, galvanized hangers and PVC fittings. Only minor deterioration was noticed on the PVC pipe which can be seen in Photo 45 through Photo 47. Based on our investigation, the pipe is in satisfactory condition.

Inspection of the pipe hangers show evidence of both bending and corrosion on the hangers which can be seen in Photo 45. Evidence of the corrosion and bending of the hangers justify a condition rating of poor.

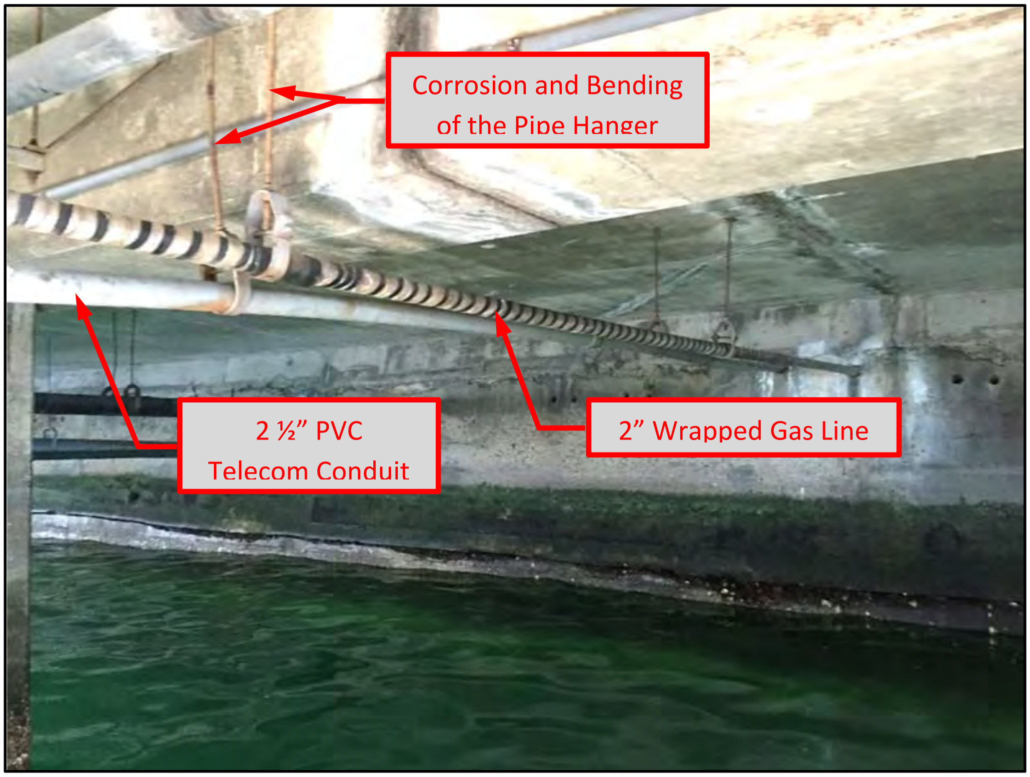


Photo 45. View of the Telecom and Gas line showing bending and corrosion on the hanger.

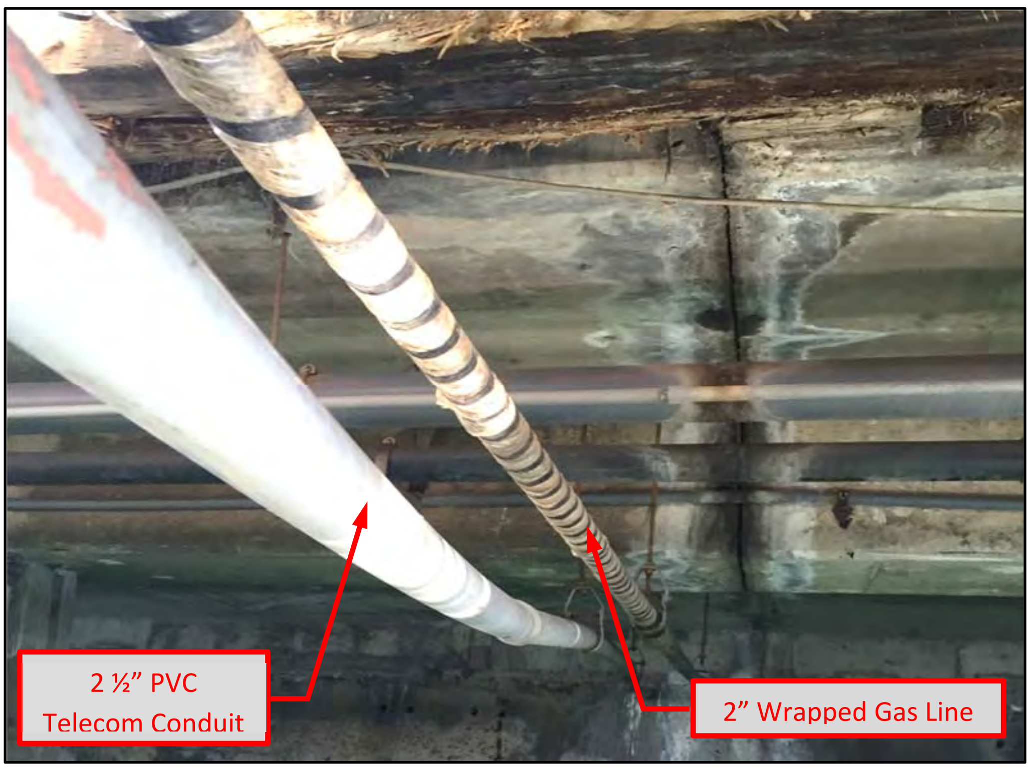


Photo 46. Underside view of the telecom and gas lines.

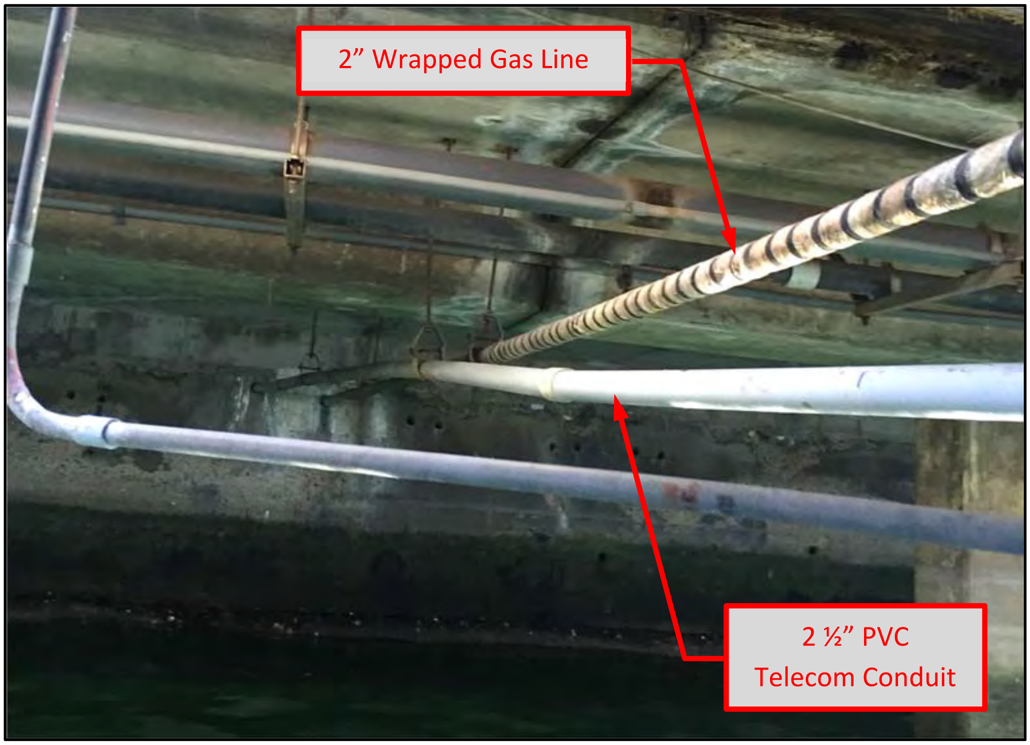


Photo 47. South facing view of the telecom and gas line

Item 12 – Gas

An existing gas system was found approximately 1'-9" below the bottom of the structure and 8-ft south of Embarcadero Wharf Pile No. 68. The system is composed of a wrapped 2-in. pipe and galvanized hangers. Upon inspection, the pipe appeared to be in satisfactory condition. Only a small amount of deterioration appeared to be present on the gas line (see Photo 45 through Photo 48).

Inspection of the pipe hangers show evidence of both bending and corrosion on the hangers which can be seen in Photo 45. Evidence of the corrosion and bending of the hangers justify a poor condition rating. However, the saddles had little deterioration and are in fair condition.

Photo 48. View of the wrapping on the gas line

Item 13 – Sewer

An existing Sewer System was found approximately 3'-10" below the bottom of the structure and 6-ft south of Embarcadero Wharf Pile No. 60. The system is composed of a 6-in. High Density Polyethylene Pipe (HDPE), a 4-in. HDPE lateral, 4-in. steel pipe (landside), galvanized hangers and HDPE fittings. Upon inspection, both the 4-in. and the 6-in. HDPE sewer line generally appeared to be in satisfactory condition and had little to no deterioration (see Photo 49 - Photo 51). However, corrosion was noted at the landside connection of the sewer system.

Photo 49 – Photo 51 also show that the pipe hangers and fittings are in satisfactory condition with little corrosion evident on the sewer system components.

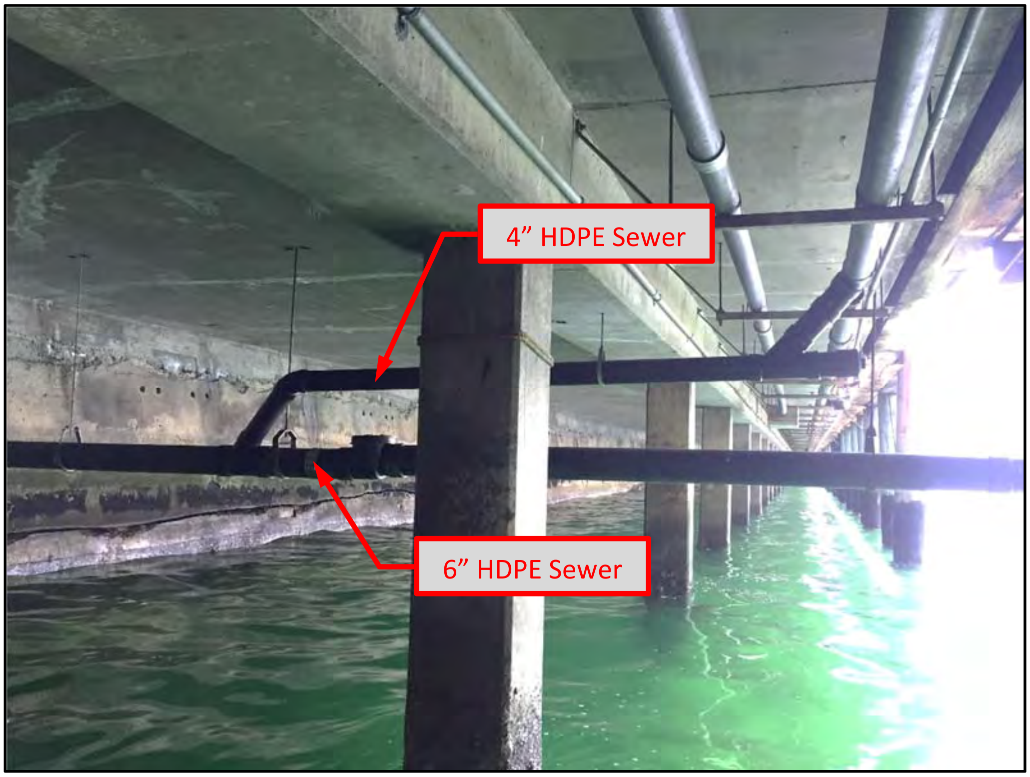


Photo 49. South View of the Sewer System.

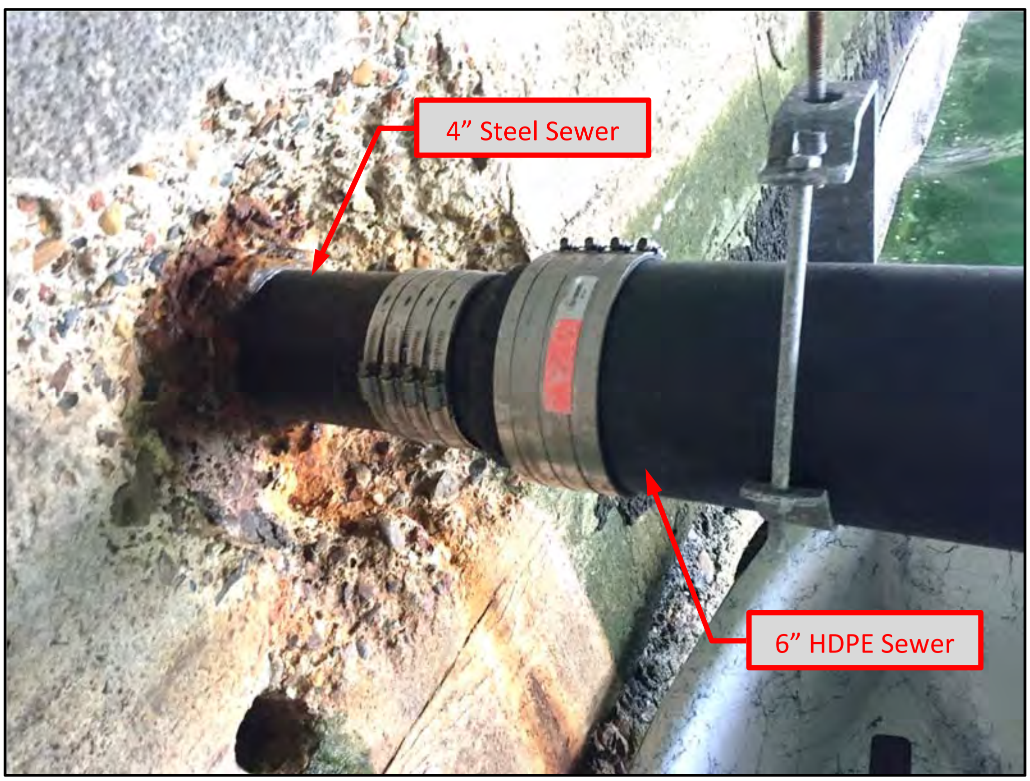


Photo 50. View of the Reducer at the Landside Connection.

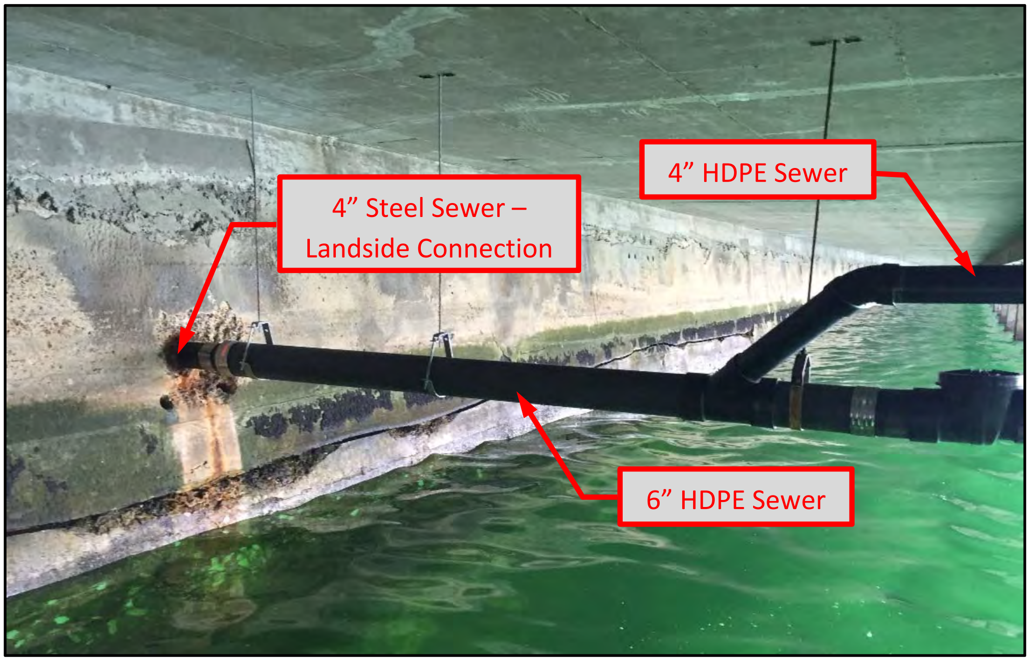


Photo 51. View of the Pipe Hangers and Sewer System.

GENERAL REPAIR AND SERVICE LIFE DISCUSSION

It is appropriate to consider the following definitions developed by the US Navy and currently being used in regards to waterfront facilities repair:

> ***Sustainment*** *- Maintenance and repair activities necessary to keep a typical inventory of facilities in good working order. Sustainment includes regularly scheduled maintenance as well as cyclical major repairs or replacement of components that occur periodically over the expected service life of the facility. Due to obsolescence, sustainment alone does not keep facilities "like new" indefinitely, nor does it extend their service lives. A lack of full sustainment results in a reduction in service life that is not recoverable in the absence of recapitalization funding.*
>
> ***Restoration*** *- Restoration of real property to such a condition that it can be used for its intended purpose. Includes repair or replacement work to restore facilities damaged by inadequate sustainment, excessive age, natural disaster, fire, accident, or other causes.*
>
> ***The key difference between sustainment and restoration is "service life." If the facility <u>has not exceeded</u> its service life and is being repaired; it is "sustainment." If the facility <u>has exceeded</u> its service life and is being repaired; it is "restoration."***
>
> ***Modernization*** *- Alteration or replacement of facilities solely to implement new or higher standards (typically regulatory changes), to accommodate new functions, or to replace structure components that typically last more than 50 years.*

The service life of a well-constructed timber pier or wharf structure in the Southern California marine environment is considered to be on the order of 50 years. The common definition of service life used in reference to an engineering structure like this is - "Service life – the length of time during which a structure, or facility, can be used economically before emergent damage causes increasing interruptions in facility operations or becomes a threat to public health and safety." In this context, the structure is approaching its useful service life.

Restoration and sustainment of existing structure. In order to attain an additional 50 years of service life for the existing platform, significant "restoration" i.e., "…repair or replacement work to restore facilities damaged by inadequate sustainment, excessive age…" would be required with a program of ongoing "sustainment" over the duration of the Brigantine Lease. Presuming the existing piles and platform framing are adequate to resist current code prescribed vertical and lateral load requirements (which is doubtful) the expected restoration and sustainment activities required to meet a 50 year life span for the existing structure are as follows:.

- Provide for complete repair of deficiencies found in the field investigation.
- Conduct above and below water engineering inspections of the platform at six year intervals.
- Make additional repairs to the grout cap and prestressed concrete piles at 15 year intervals.
- Perform repair or replacement of timber framing affected by soft rot at 15 year intervals
- Refurbish steel connectors at 12 year intervals.

Capacity of Existing Structure. It is understood that TerraCosta Consulting Group is under contract to prepare a desktop geotechnical study that will provide estimates of the soil-derived load capacity for the existing piles. Based on this information, and given the existing condition of the prestressed concrete plumb piles, the load capacity of the piles can be predicted with a fair degree of accuracy. Because the deck framing closely matches the record drawings, it is also possible to compute the load capacity of the existing platform structure, presuming a full rehabilitation of the damaged elements noted in this report.

Unfortunately, the lateral load capacity of the concrete-filled steel battered pipe piles is impossible to accurately predict because the as-built condition deviates from the record drawings. There are methodologies (sometimes unreliable) that could be used to estimate pile tip elevation, but the method of connecting the pipe piles to the pile cap is unknown. In modern waterfront structural engineering, pile-to-cap connection is a key element in estimating lateral load capacity. As noted earlier in this report, the steel pipe shell is heavily corroded in the tidal zone, and to a lesser extent there is also a loss of steel thickness below water. The inability to characterize the lateral load capacity of the existing system makes it impossible to incorporate this system into a design to support the proposed Brigantine improvements.

Existing Utilities. The size and general arrangement of utilities that extend out from the bulkhead and are located underneath the Embarcadero Wharf are shown on Figure 3 in Appendix A: Figures. The condition of the utilities underneath the wharf range from satisfactory to poor. Preliminary demand capacities for the various utilities to be used with the proposed Brigantine design should be developed and compared to the existing capacity to determine if there are discrepancies. Research should be performed to see if any of the utilities also provide service to the nearby Maritime Museum. If this is the case, potential alteration of the utilities for Brigantine should consider the effects on the Museum.

RECOMMENDATIONS

Despite the overall condition rating of "fair" assigned to the existing structure, it is recommended that the existing platform be demolished and a replacement structure and associated utilities be designed to support the proposed Brigantine improvements. This recommendation is based on the following considerations:

A. Service life. The Brigantine lease is on the order of 40 years duration. The Tucker Sadler designed building will commence with a service life of a new facility located in a harsh marine environment. To construct a new building on top of an existing platform already beyond its service life, will require an inordinate amount of restoration and ongoing sustainment in order to last another 40+ years. Likewise, the utility services should be re-worked (and upgraded in capacity if necessary) to provide a service life consistent with the new construction.

B. Functionally obsolete. Reusing the existing structure will require the architectural program and superimposed weight of the new design to be in conformance with the load capacity and structural load path of the existing structure. This would likely result in considerable restriction of the proposed architectural solution. If the utility requirements for the Brigantine program require upgrades in capacity to the existing lines, this would serve to reinforce the "functionally obsolete" discussion.

C. Lateral load capacity. The inability to ascertain the load capacity is described in detail above. This fact, in itself, negates the option for reuse of the existing facility.

The opportunity to be of assistance in this matter is appreciated. If there are any questions in regards to this information, please do not hesitate to call.

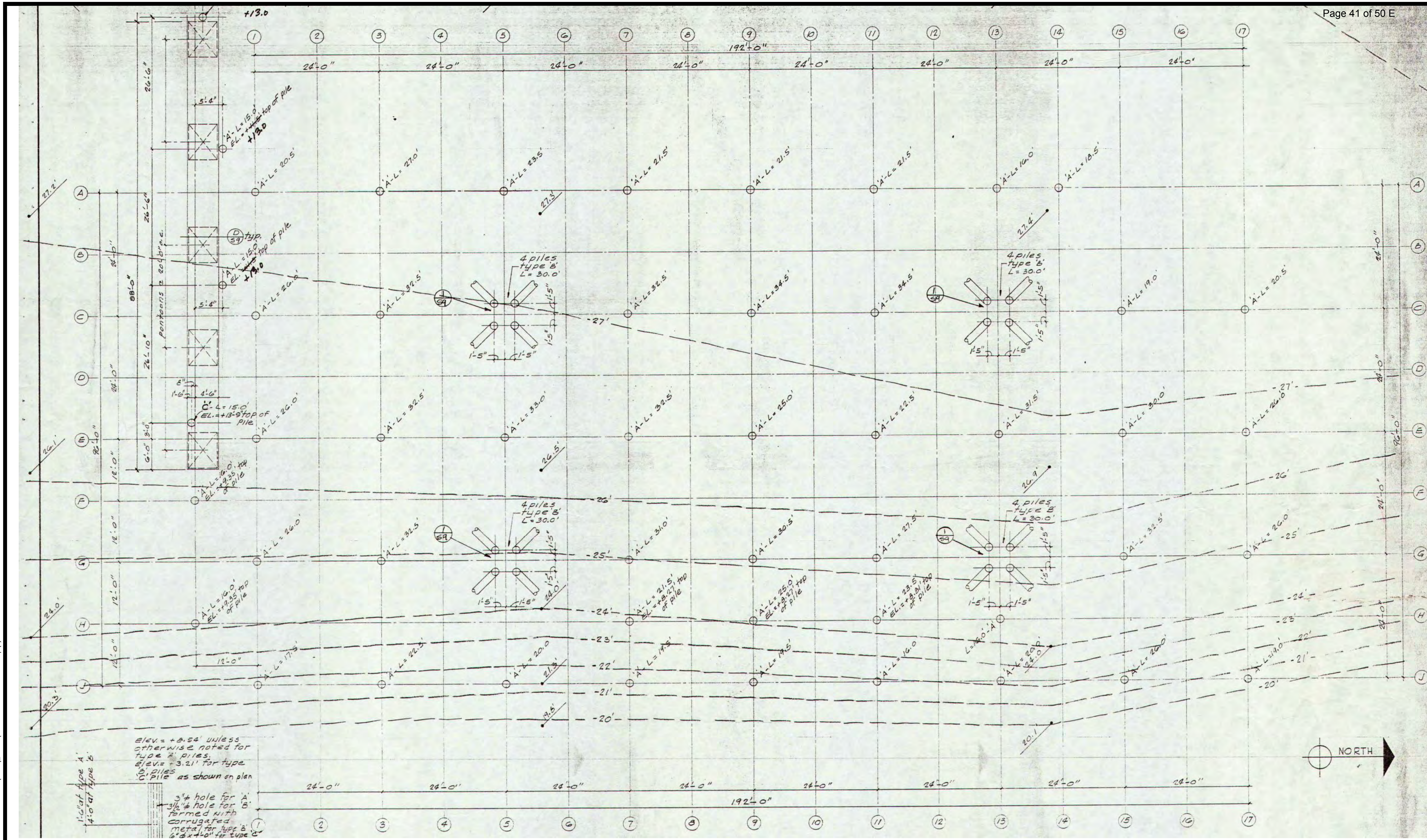
Very truly yours,

Moffatt & Nichol

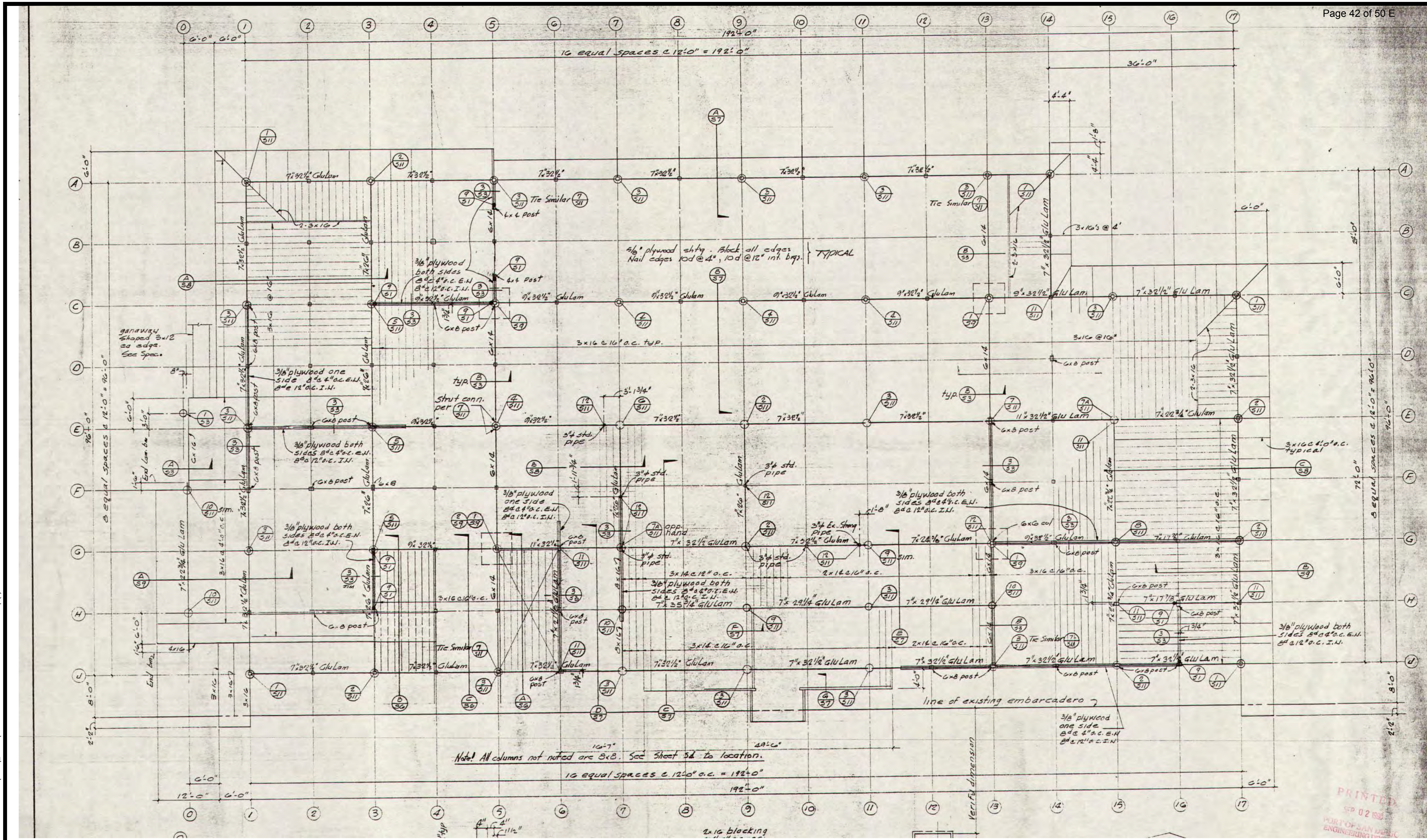
Matthew N. Martinez, S.E.

ADB:mnm

APPENDIX A: FIGURES



DRAWING BY: FREDERICK LIEBHARDT AND EUGENE WESTON III AIA ARCHITECTS TITLE: ANTHONY'S FISH GROTTO DATE: JUNE 30, 1965	DATE MAR 2016	moffatt & nichol 1660 HOTEL CIRCLE NORTH, STE 500 SAN DIEGO, CA 92108 (619) 220-6050	WFI AND ASSESSMENT - FUTURE PORTSIDE PIER RESTAURANT PILE PLAN FIG. No. 1



DRAWING BY: FREDERICK LIEBHARDT AND EUGENE WESTON III AIA ARCHITECTS

TITLE: ANTHONY'S FISH GROTTO

DATE: JUNE 30, 1965

DATE
MAR 2016

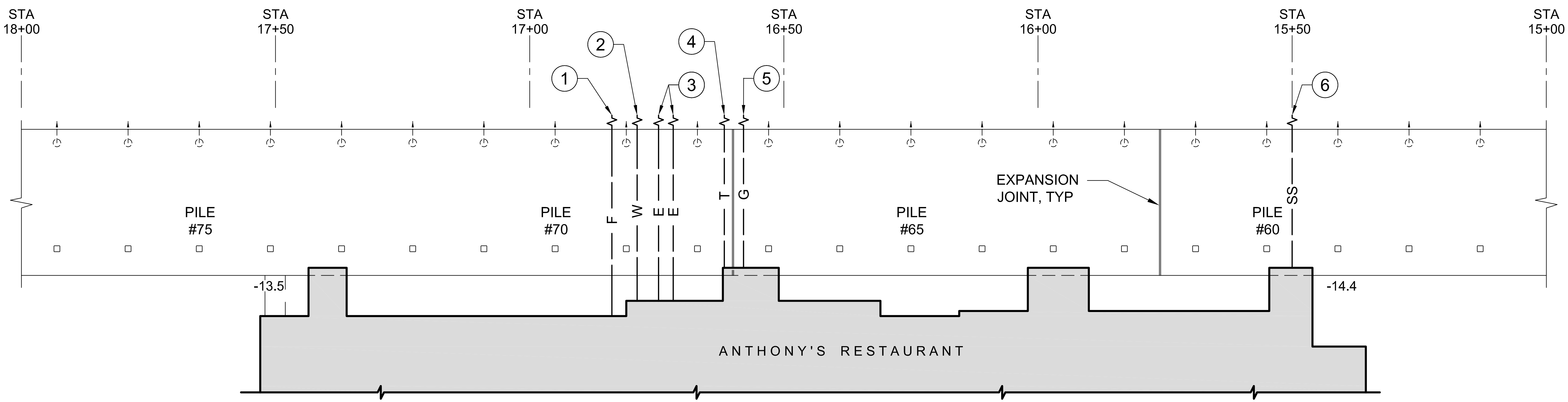
moffatt & nichol
1660 HOTEL CIRCLE NORTH, STE 500
SAN DIEGO, CA 92108
(619) 220-6050

WFI AND ASSESSMENT - FUTURE PORTSIDE PIER RESTAURANT

MAIN FLOOR FRAMING PLAN

FIG. No.
2

DWG INFO: P:\9220 - BRIGANTINE INVESTIGATION REPORT\CADD_ACTIVE\FIGURE 2.DWG - MAIN FLOOR FRAMING PLAN | (c) MOFFATT AND NICHOL ENGINEERS



NOTES:

1. 6" CMP FIRE WATER
2. 2½" COPPER POTABLE WATER
3. 2x4" WRAPPED ELECTRICAL CONDUITS
4. 2½" PVC TELECOM CONDUIT
5. 2" WRAPPED GAS CONDUIT
6. SEWER SYSTEM - 6" HDPE SEWER, 4" HDPE LATERAL, 4" STEEL PIPE

LEGEND:

—— —— F —— ——	EXISTING FIRE WATER
—— —— W —— ——	EXISTING POTABLE WATER
—— —— SS —— ——	EXISTING SEWER
—— —— E —— ——	EXISTING ELECTRICAL
—— —— G —— ——	EXISTING GAS
—— —— T —— ——	EXISTING TELECOM
□	EXISTING 14" SQUARE PILE
⊖	EXISTING 16" DIA CONC PILE

N

20' 0' 20' 40'

SCALE: 1"=20'

DATE MAR 2016	moffatt & nichol 1660 HOTEL CIRCLE NORTH, STE 500 SAN DIEGO, CA 92108 (619) 220-6050	WFI AND ASSESSMENT - FUTURE PORTSIDE PIER RESTAURANT	
		UTILITY LOCATION PLAN	FIG. No. 3

APPENDIX B: BACKGROUND INFORMATION

BACKGROUND INFORMATION

TIMBER DETERIORATION

The deterioration of timber structures in a marine environment is primarily due to biological factors in addition to the physical damage caused by impact loadings. There are two general types of organisms responsible for the biological deterioration of timber members. Typically underwater damage consists of deterioration due to marine borer attack, while above water damage is generally due to fungal attack.

Fungal Decay

Under favorable fungal support conditions, decay can rapidly destroy wood substance and seriously reduce the strength of wood. This may happen before there is any pronounced change in external appearance of the wood; thus even early decay can dangerously weaken a wood structural component. Advanced decay, of course, can render it entirely useless.

There are three types of decay typically encountered in marine timber structures, each of which is caused by different fungi. The most common type of fungal decay typically observed is brown rot, which is commonly referred to as dry rot. With dry rot, only the cellulose is extensively removed; the wood takes on a browner color and tends to crack across the grain, shrink and collapse. As with marine borer damage, the fungus prefers the untreated wood in the center portion of the timber pile. In many instances it can be visually observed as a large deteriorated section of pile, though in some instances, particularly at the top of the pile, it can only be detected by probing the pile. Splits, bolt holes, staple holes and abrasion areas allow easy access through the treated exterior portion of the timber member, to the untreated wood of the pile.

Less commonly encountered are soft rot and white rot. With soft rot, the fungus attacks the wood surface and there is a gradual softening of the wood from the surface inward. Usually they are found in wood exposed to wet conditions and can develop in wood too wet to be attacked by other fungi. White rot destroys the cell structure and gives the wood a white bleached appearance. The wood is spongy to the touch and stringy when broken. Both of these types of decay develop slowly and cause a slow loss in the section properties of the timber elements.

CONCRETE DETERIORATION

Corrosion of Reinforcing Steel

Concrete deterioration in the marine environment takes on many forms. The most prevalent of these is corrosion of the steel reinforcing within the concrete structure. As steel corrodes, it undergoes a volumetric expansion, swelling to more than nine times the original volume. Since the steel is restrained by the surrounding concrete, an outward pressure is exerted on the concrete. This outward pressure is inherently a tensile force, and as concrete is relatively weak in this mode of loading; cracks and “spalling” of the concrete eventually occurs. Spalling leads to exposure of the reinforcing steel to the marine environment, which exacerbates the problem.

Mechanism of Steel Corrosion. Corrosion of steel reinforcing is governed by two processes - the first of these being the pacification of the highly alkaline concrete composition. The second process is the actual corrosion of the reinforcing bar by oxidation.

When first placed, concrete has a high pH value usually ranging from 12.5 to 13.2. This highly alkaline environment allows an oxidized film (Fe_2O_3) to form on the reinforcing steel. This film provides a protective layer around the steel, minimizing the potential for reactions with chloride ions from sea water. Above a pH of 13, the protective film is retained. However, the alkalinity is pacified over time by two processes - the ingression of sea salts and/or by carbonation of the concrete. Sea salts penetrate the concrete through capillary action, and therefore the time to pacification is dependent on the porosity of the concrete. Carbonation is a chemical reaction by which carbon dioxide reacts with calcium hydroxide, the alkaline compound found in fresh concrete, to form calcium carbonate. Calcium carbonate is a neutralized (pH=7) compound, and therefore reduces the high pH concrete environment needed to maintain the beneficial oxidized iron film.

Once the concrete structure has been pacified to the depth of the reinforcing steel, and the oxidized iron film is destabilized, the reinforcement is allowed to corrode. This corrosion is a continual oxidation of the steel bars and is dependent on the availability of oxygen. Since corrosion requires pacification as well as oxidation, the corrosion critical areas of any structural concrete in the marine environment will be those elements in the tidal or splash zones. These areas provide a constant supply of both aggressive salts and oxygen needed for a sustained corrosive attack. All concrete elements located in the marine environment however are susceptible, with varying rates of corrosion based on the level of exposure to corrosive elements.

As stated in the introduction, steel reinforcement expands as it corrodes. The volume of the oxidized iron product can be more than nine times that of the parent material. The pressure induced by the expansion of corroded steel eventually leads to cracking of the concrete. A condition known as “staining” or “bleeding” is usually apparent when deterioration of this sort is encountered, and consists of red rust leaching out of the concrete cracks. As the corrosion of the reinforcing continues, and outward pressure increases, the concrete covering the reinforcing bar eventually spalls out (See Figure I-2). The loss of cover over the bar leads to increased rate of corrosion, and loss of cross-sectional area of the bar.

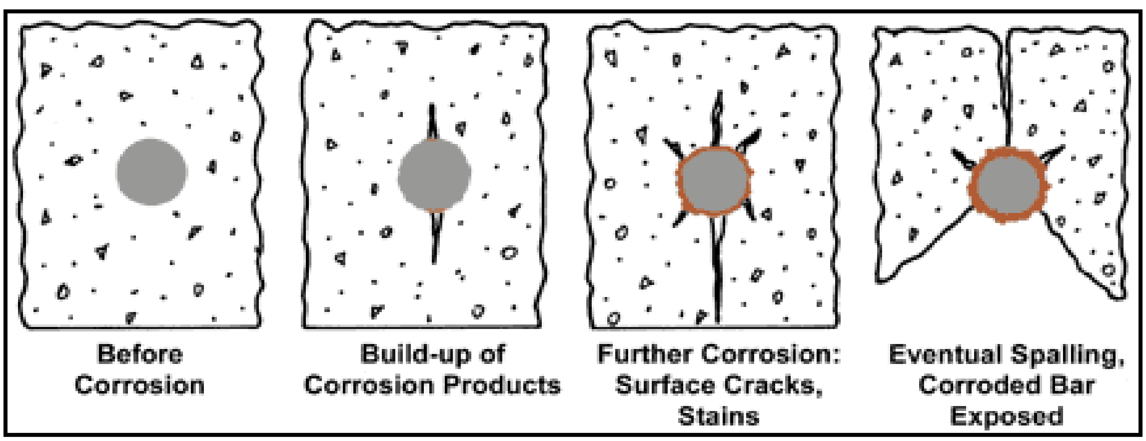


Figure J – Process of Steel Corrosion-Related Concrete Damage

Deterioration of concrete marine structures may be caused by physical and/or chemical interaction with seawater. "If the structure is fully immersed, the attack on the material by seawater is essentially chemical. In alternating immersion and exposure conditions, the attack is of chemical and physical nature. The mechanical action of the waves, the swelling and shrinkage caused by the alternate saturation and drying, atmospheric conditions (wind, exposure to the sun, freezing) and the electrochemical corrosion of steel reinforcement are physical processes which add to the chemical destruction processes."

Submerged deterioration of the concrete as observed by this firm has been limited to what has been identified as secondary ettringite formation, sulfate attack, alkali-silica reaction, and corrosion. The electrochemical corrosion of the reinforcing steel is most active in the tidal range and splash zone where both oxygen and the chloride ion are readily available. Below water, the concentration of chlorides and oxygen are less than in the splash zone. However, in time it will reach the reinforcing steel and initiate corrosion.

"The mechanism of concrete corrosion (deterioration) is extremely complex for it depends on a certain number of parameters which are not always easy to isolate and which react in varying degrees according to the composition and the exposure of the material."

Secondary Ettringite Formation

Secondary ettringite formation is defined as ettringite formed by reaction of sulfate ion and aluminate in concrete that has hardened and developed its intended strength. The sulfate which fuels the reaction is supplied from within the concrete. The reaction has also been referred to as “delayed ettringite formation” in the literature.

Ettringite is formed when sulfates (SO3) react with the free lime (calcium hydroxide (CaOH2) to form gypsum (CaSO4). The gypsum then reacts with tricalcium aluminate (CaAl2) and water to form ettringite (Ca6Al2(SO4)3(OH12)). Many of these reactants are in the cement and/or seawater.

There are two theories as to the mechanism of expansion caused by this phenomenon. In the swelling theory ettringite forms by a through-solution mechanism. In a saturated CH environment ettringite crystals are gel-like and colloidal in size. The high surface area results in adsorption of significant quantities of water and strong swelling pressures develop. It has been observed that a higher proportion of ettringite is found at the transition zone between the aggregate and steel than in the bulk matrix. This finding supports the through-solution mechanism of expansion, since constituents must dissolve and diffuse towards the steel/aggregate surface where the ettringite is precipitated. In the crystal growth theory, expansion is caused by the formation of ettringite at the surface of the reactant grains. The growth of this inner layer pushes other particles out and thus causes expansion. Estimates of crystal growth pressures have been as high as 35,000 psi.

There is some experimental evidence into the various causes and rate of ettringite formation. Some of the components which may affect ettringite formation are elevated temperatures during curing, (SO3)/(Al2O3) ratios, geometry and humidity.

It appears that sufficiently high heat treatment, temperatures above 60-70o C, contributes to the secondary ettringite formation. When concrete is cured at elevated temperatures, ettringite disappears into a calcium-sulfate-hydrate gel and/or monosulfate, this results in the sulfate being unusually bound. The bond is such that it allows a later slow release of the sulfate ion into the pore solution which then combines with tricalcium aluminates to produce ettringite.

The ratios of the aluminum oxide (Al2O3) and sulfur trioxide (SO3) in the cement have shown potentials for expansion when the (SO3)/(Al2O3) is greater than 0.67. Later experiments indicate that the sulfur trioxide may have a greater contribution to the expansion. Therefore, the ratio indicating the potential for expansion has been adjusted to (SO3)2/(Al2O3) greater than 2.

Other items which could contribute to expansion are geometry and humidity. 10x40x160 mm cubes produced much earlier expansions than 40x40x160 mm cubes and specimens in a water soak had earlier expansions than specimens in 60% humidity.

Air-entrainment of the concrete has been shown to reduce the observed expansions due to secondary ettringite formation when comparison is made to non-entrained concrete. The air voids allow the formation of ettringite within the void and prevents the associated microcracking caused by expansion in the paste. In a similar fashion, the addition of silica fume has found to be beneficial by increasing the density of the paste in the transition zone at the aggregate/matrix interface.

It should be mentioned that ettringite formation is part of the hydration process used to make concrete. This formation of ettringite is while the concrete is in a plastic state and helps the concrete develop strength - therefore, this formation is beneficial. This reaction is often referred to as “primary ettringite formation”.

Sulfate Attack

Sulfate attack is a type of secondary ettringite formation. It results from the reaction of sulfate ions and aluminates in hardened concrete. The sulfate is typically from an external source - in the case of marine structures the sulfate is in the seawater It is generally accepted that the primary aggressive constituents of seawater, relative to attack upon the cementitious matrix of portland cement concrete, are magnesium and sulfate ions.

"Magnesium sulfate also reacts with aluminates that are a constituent of the portland-cement, primarily tricalcium aluminate, with consequent production of ettringite (high sulfate calcium sulfoaluminate, 3Ca0.Al203.3CaS04.31H20). Formation of ettringite as a solid-state reaction within the cement-paste matrix can be highly destructive to portland cement concrete because of the increase of solid volume that accompanies the process. Contrariwise, formation of ettringite by a through-solution process whereby the crystals are precipitated within pre-existing openings, such as air voids and cracks, is not harmful."

This reaction can be accompanied by considerable expansion, which causes cracking and spalling of the concrete.

Alkali-Silica Reaction

In the alkali-silica reaction, the alkalis are the metal alkalis sodium and potassium, both of which are present in seawater. For the reaction to occur, reactive silica, sodium and potassium alkalis and water must all be present. It is primarily a reaction between the hydroxyl ions in the pore water of a concrete and certain forms of silica which occasionally occur in significant quantities in aggregate.

"In the alkaline environment within a concrete, an acid/alkali reaction occurs at the accessible surfaces of the silica forming a hydrous silicate. Hydroxyl ions are imbibed into the silica particle and some of the silica oxygen linkages are attacked, weakening the bonding locally. Sodium and potassium cations then diffuse to maintain an electrical neutrality and attract water to form gelatinous alkali-metal-ion hydrous silicate."

The gelatinous silicate increases the solid volume of the concrete. This can cause microcracking and macrocracking, which is destructive to the concrete. If the gel forms in pre-existing air voids, water voids, or when the concrete is in the fresh state, the reaction is not harmful. If the gel forms in the hardened solid concrete, the reaction is harmful.

Sodium and potassium ions and water, two of the constituents of this reaction, are present in seawater. If reactive silicas are present in the concrete, the alkali-silica reaction can occur. However, if the reactive silica content is low and gel growth after the concrete has hardened is of insufficient intensity to induce cracking, the "gel growth occurs without any adverse effect on the concrete. When the reactive silica content is above this level, cracking induced by the gel occurs.

The width of the macrocracks induced by alkali-silica reaction at the exposed surface of a concrete member can range from less than 0.004 in. to 0.40 in. in extreme cases. The macrocracks are generally located within 1-2 in. of the exposed surface of a concrete member and are aligned perpendicular to the exposed surface. However, there are exceptions, in the case of a prestressed column a crack depth of approximately 4 ¾" has been recorded.

One example of severe alkali-silica deterioration has occurred at the Friant Dam, constructed during the period 1939 to 1942. In 1980, Boggs noted that alkali-aggregate reaction had occurred to some extent since construction but that the reaction progress appeared to have accelerated from excellent-looking concrete in the late 1960's to wide cracks on the crest and the appurtenant structures in 1980. Deterioration has not yet reached the point of jeopardizing the safe operation of the dam but eventually will.

"Cracking due to ASR (alkali-silica reaction) has been observed within 3 months in one batch of concrete specimens containing a UK (United Kingdom) aggregate stored under water at 20o C, whereas a similar concrete stored in the open took approximately 3.6 years to crack."

This is only one observation; however, it affirms the observed underwater crack predominance. If it is presumed that the observed rate of dry cracking to underwater cracking (14:1) is correct, than the underwater cracks caused by the alkali-silica reaction should occur in a shorter period of time compared to cracks forming above water – given the same concrete material.

During a previous underwater investigation in San Diego, cracks were observed during the initial inspection of the piles. The inspected piles were approximately 12 years of age. Using the above-mentioned 14:1 rate, this would correlate to above water cracks becoming visible at 168 years of age. This would indicate that it is possible for an aggregate to have a good above water history and not be acceptable for underwater use.

This reaction can be accompanied by considerable expansion, which causes cracking of the concrete, a reduction in the concrete compressive strength and a reduction in the modulus of elasticity.

"Alkali-silica reactivity by itself seldom results in the need to rebuild the structure but, rather, it may weaken or degrade the condition of the structure to the extent that other factors, such as traffic loading, cause premature failure.

Damage Ratings for Steel Elements.

Table 2-5. Damage Ratings for Steel Elements

Damage Rating		Existing Damage[a]	Exclusions [Defects Requiring Elevation to the Next Higher Damage Rating(s)]
NI	Not Inspected	• Not inspected, inaccessible, or passed by[b]	
ND	No Defects	• Protective coating or wrap intact • Light surface rust • No apparent loss of material	
MN	Minor	• Protective coating or wrap damaged and loss of thickness up to 15% of nominal at any location • Less than 50% of perimeter or circumference affected by corrosion at any elevation or cross section • Loss of thickness up to 15% of nominal at any location	Minor damage not appropriate if • Changes in straight line configuration or local buckling • Corrosion loss exceeding fabrication tolerances (at any location)
MD	Moderate	• Protective coating or wrap damaged and loss of thickness 15 to 30% of nominal at any location • More than 50% of perimeter or circumference affected by corrosion at any elevation or cross section • Loss of thickness 15 to 30% of nominal at any location	Moderate damage not appropriate if • Changes in straight line configuration or local buckling • Loss of thickness exceeding 30% of nominal at any location
MJ	Major	• Protective coating or wrap damaged and loss of nominal thickness 30 to 50% at any location • Partial loss of flange edges or visible reduction of wall thickness on pipe piles • Loss of nominal thickness 30 to 50% at any location	Major damage not appropriate if • Changes in straight line configuration or local buckling • Perforations or loss of wall thickness exceeding 50% of nominal
SV	Severe	• Protective coating or wrap damaged and loss of wall thickness exceeding 50% of nominal at any location • Structural bends or buckling, breakage and displacement at supports, loose or lost connections • Loss of wall thickness exceeding 50% of nominal at any location	

[a] Any defect listed is sufficient to identify relevant damage grade.
[b] If not inspected due to inaccessibility or passed by, note as such.

Source: ASCE Manuals and Reports on Engineering Practice No. 130 “Waterfront Facilities Inspection and Assessment”.