TETRA TECH

April 21, 2017

Submitted via email:
kolson@cityofkeywest-FL.gov

Karen Olson
Deputy Director
City of Key West
Port & Marine Services
201 William Street
Key West, FL 33040

Subject: Seawall Condition Assessment
From Turtle Kraals to William Street
Resolution No. 17-017

Dear Ms. Olson:

Tetra Tech is pleased to submit this seawall condition assessment report for your review. The report discusses the condition of the seawall from in front of the Turtle Kraals restaurant (Margaret Street) west to William Street and provides recommendations for its continued monitoring or repair. This report includes a copy of the project topographic survey incorporated into the deficiency plans sheets, and a copy of the benthic resource survey that will be used for permitting in the future.

Please review this report and its attachments at your convenience and let's schedule a meeting to discuss. If you have any questions or need any additional information, please contact me.

Sincerely,

STUART E. McGAHEE LICENSE No 57536 STATE OF FLORIDA PROFESSIONAL ENGINEER

Stuart E. McGahee PE
Project Engineer
FL PE No. 57536

cc: Doug Bradshaw, Director, Ports & Marine Services
Shauna Stotler-Hardy, Tetra Tech

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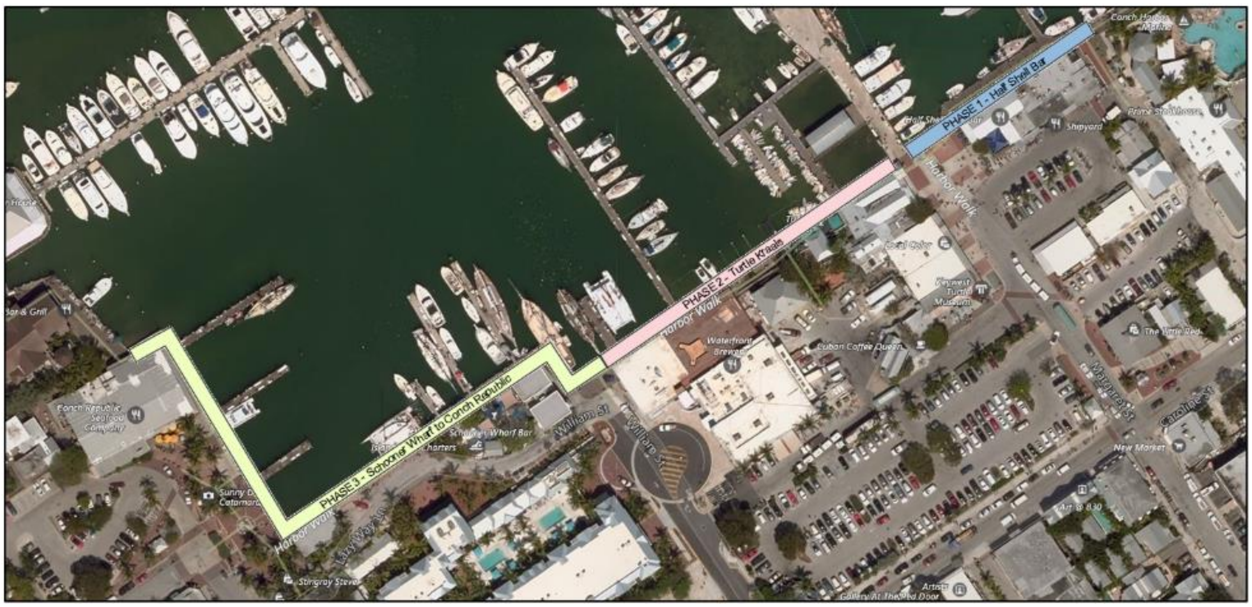
CA #: 2429

2017

CITY OF KEY WEST
PORT & MARING SERVICES

PHASE 2

SEAWALL CONDITION ASSESSMENT
WATERFRONT MARKET TO TURTLE KRAALS RESTAURANT



RESOLUTION NO. 17-017

SUBMITTED
APRIL 21, 2017

STUART E. McGAHEE
LICENSE
No 57536
STATE OF FLORIDA
PROFESSIONAL ENGINEER

04/21/17

Stuart E. McGahee PE
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1.0 REPORT/INSPECTION TERMINOLOGY

Abrasion: The process of eroding, rubbing or wearing away a surface by friction.
Active cracks: Those cracks which the mechanism causing the cracking is still at work.
Aggregate: Granular material such as crushed stone in the concrete mix.
Bugholes: (slang) Industry term used to describe small cavities resulting from entrapment of air bubbles in the surface of concrete.
Concrete cover: The distance between the surface of embedded reinforcement and the surface of the concrete.
Corrosion: Destruction of metal by chemical, electrochemical or electrolytic reaction within its environment.
Crack: A complete or incomplete separation of concrete into two or more parts produced by breaking or fracturing.
Damage: Impairment to the value or usefulness of an element or component.
Deformation: A change in dimension or shape, see distortion.
Deflection: A movement of a structural element measured as linear displacement.
Delamination: A horizontal or planar separation of the surfaces of concrete.
Depression: A lowering of the surrounding surfaces
Deterioration: The decomposition of material during exposure to service.
Diagonal crack: A crack forming an angle other than 90 degrees with the centerline of the concrete member.
Discoloration: A departure of color from what is normal.
Disintegration: The deterioration into small fragments or particles due to any cause.
Dislodged: The movement of an object due to impact or force.
Distortion: A change in alignment of the components of a structure, see deformation.
Distress: The cracking or distortion in a concrete structure as the result of stress.
Dormant cracks: Cracks which are not currently moving.
Efflorescence: A white deposit on concrete caused by crystallization of soluble salts brought to the surface by moisture in the concrete due to capillary action.
FRP: Fiber reinforced plastic composites rebar
Galvanic corrosion: An electrochemical process in which one metal corrodes preferentially to another when both metals are in electrical contact and immersed in an electrolyte (seawater).
Gouges: A groove or hole caused by the impact or action of a hard object.
Hairline crack: A crack not greater than 0.003 inch in width or barely perceptible.
High tide: The highest level of the tide or the time at which the tide is highest.
Hollow area: An area of concrete which when struck with a hammer gives off a hollow sound indicating the existence of a horizontal fracture below the surface.
Honeycomb: Voids in concrete due to failure of the mortar to effectively fill the spaces between coarse aggregate.
Incrustation: A crust of coating, generally hard, formed on the surface of concrete.
Life safety: An act to protect people based on occupancy features and conditions.
Low tide: The lowest level of the tide or the time at which the tide is lowest.
Map crack: An interconnected crack forming networks of any size and similar to those see in dried mud flats.
Pile: A slender structural element that is embedded on end in the ground to support a load.

Pile batter: A pile installed at an angle to the vertical.
Pile bent: A row of bearing piles with a continuous concrete cap.
Pile cap: A structural element that transfers load to the top of one or more supporting piles.
Pile jacket: A prefabricated protective covering placed around the circumference of a pile for the purpose of preservation.
Pitting: Relatively small cavities in concrete or localized corrosion evident as minute cavities in steel.
Popouts: Shallow typical conical depressions in a concrete surface.
Preservation: The process of maintaining a structure in its present condition of arresting further deterioration.
PVC: Polyvinyl chloride used in the manufacture of conduit.
Random crack: A crack that meanders irregularly on the surface of concrete having no particular form.
Raveling: The wearing away of the concrete surface caused by the dislodging of aggregate particles.
Reflective cracks: A propagation of stresses in a concrete topping slab or asphalt layer due to traffic loads
Rehabilitation: The process of modifying a structure to a desired useful condition.
Repair: To replace or correct deteriorated or damaged components or elements of a structure.
Scaling: The local flaking or peeling away of the near-surface of hardened concrete.
Settlement: The lowering in elevation of pavement or structures.
Shrinkage crack: Cracking of a structure due to failure in tension caused by reduction on moisture content.
Sound: The absence of deficiencies or defects which would lessen the structural integrity or performance of the structural element.
Spall: A chip of concrete broken from the surface of a concrete member.
Small spall: A spall not larger than 0.8 depth or than 6 inches in any dimension
Large spall: A spall deeper that 0.8 and/or 6 inches in any dimension.
Splash zone: The area on an offshore structure that is regularly wetted by seawater but is not continuously submerged.
Substrate: Any material on the surface of which another material is placed.
Substructure: All of that part of a marine structure below the deck elevation.
Tidal range: The difference between high and low tide elevations.
Urgency: Priority or a pressing necessity of importance
Void: Volume of concrete that is missing. Term is used to describe an area near the toe of the wall where a considerable amount of concrete is missing.

2.0 EXECUTIVE SUMMARY

From February 20-24, 2017, Tetra Tech, Inc. (Tt) performed routine structural assessment and benthic resources survey of the Turtle Kraals bulkhead on behalf of the City of Key West Port & Marine Services. The survey was performed along the 440 linear foot segment of the Key West Bight perimeter seawall that extends from the north end of Williams Street east in front of the Waterfront Market & Brewery, the public restroom facilities, and the Turtle Kraals restaurant. Data collected during the inspection included:

- Location of all buildings, boardwalks and docks along/above the seawall
- Surface elevations along the top and toe of the seawall and water depths along the seawall
- Elevations of the basin bottom on the water ward side of the dock structures,
- Locations of utilities hanging off of the seawall of docks associated with the seawall,
- Location, quantity, and class of stony corals on or near the seawall
- Seawall condition and deficiencies

The following exhibit is intended to show how the inspection transect was configured and referenced to provide a synopsis of the wall conditions. Figure 1 shows how the divers marked the basin bottom along the toe of the wall.

Figure 1: Survey and Transect locations

Tape measurement wheels where used to mark the site from 0+00 feet (at William Street) to 4+40 feet (at Margaret Street) by placing measurement tape along the mudline of the wall. Measurements were then taken as needed perpendicular to the tape-marker at the mudline up the wall to the points of interest so that each deficiency could be cataloged. The inspection included both a structural and a benthic resource component sufficient to determine existing conditions, develop preliminary remedial measures and to satisfy permitting requirements associated with any additional monitoring, proposed repair or fortification of the bulkhead. Qualified staff were onsite to coordinate survey logistics and catalogue structural deficiencies and benthic resources within the designated survey area.

The purpose of the **seawall assessment** was to perform a visual inspection of the seawall conditions and develop an existing conditions report that could be coupled with the recently completed shoreline topographic survey. This report catalogs the deficiencies of the seawall and discusses possible monitoring or remedial measures that can then be used to develop engineering plans for the repair of the wall. The purpose of the **benthic resource survey** was to determine the quantity of stony corals, spatial extent, size-class, relocation candidates, and total coral tissue area within the project area. Additionally, the survey identified any other biological resources within the project area. This survey will supply the regulatory agencies with the most up-to-date resource data for the project site. Further, data collected from this survey may be used during the planning phase of the project to minimize and avoid impacts to stony corals and/or other resources within the project area.

It is estimated that some portions of the Key West Bight perimeter seawall are close to one hundred years old. Other areas along the perimeter have been repaired or possibly even reconstructed over the years and the exact configuration is uncertain and/or unknown. Portions of the bulkhead adjacent to the waterfront market near Williams Street have been modified as recently as a couple of years ago adding a new outfall pipe. In 1995, the entire upland in this area was redeveloped and engineering plans prepared for that project at the time, and show the wall in its current configuration was already in place. The plans from the 1995 redevelopment were reviewed with the Port and Marine Services staff and show the existing bulkhead consisting of a concrete gravity wall with a key wall under the toe.

However, none of the dimensions of the existing wall or its exact configuration or embedment depth were specified and during the inspection some of the observations did not match exactly what was shown on the 1995 plans. In some places there was clearly a steel key wall located under the toe. In other places no key wall was apparent or the sediment was too deep to tell for certain. Because of this uncertainty, attempts were made to visually check for issues with stability and assumptions were made to perform a basic stability check and make repair recommendations. Without exception, the wall appeared to be stable with no indication of sliding or rotation.

TYPICAL SEAWALL ASSESSMENT SUMMARIES

The following summary assessment sheets describe the general conditions for wall segments and proposed remedial or monitoring measures recommended. The wall segments were grouped together where conditions were found to be similar. For example, the first condition assessment was for the first 40-feet of wall (from 0+00 to 0+40). This segment of wall was grouped together because the steel key wall located under the toe of the wall is visually exposed and a similar remedial procedure is being recommended for this portion of the wall. This grouping methodology was repeated for the entire length.

Assessment Report – Station 0+00 to 0+40

from William Street toward the Waterfront Market

Date of Inspection: February 20-24, 2016

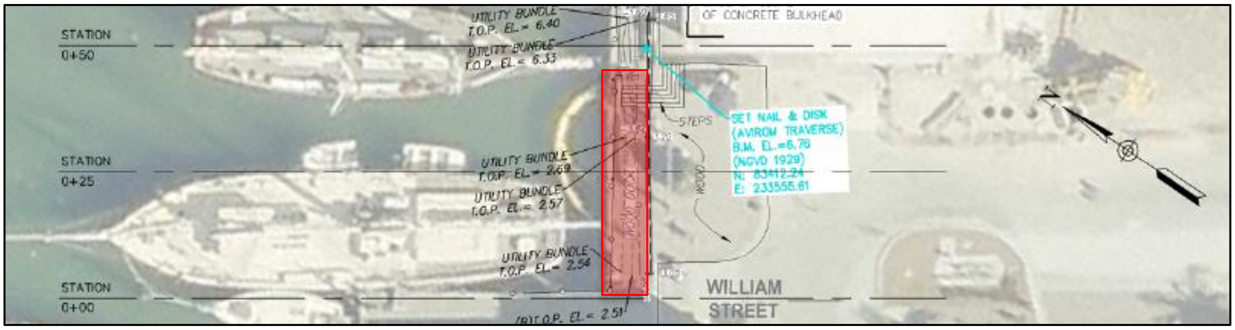


Figure 2: Station Exhibit, 0+00 to 0+50


Figure 3: Station 0+40, looking west

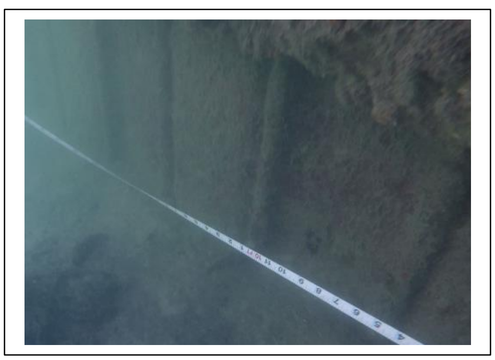


Figure 4:   Station 0+10, steel key wall at toe

Note: Full size copies of these images are included in Appendix 1

Description	Turtle Kraals (0-40)
Condition Index	Good: Station 0+00 to 0+40
Service Life	25 years
Age of Structure	Unknown
Next Inspection	2020
3-Year inspection Cost	$12,500
Near Term Repair Cost	$84,000
Immediate Repair Cost	$ 0

Condition: The condition of the bulkhead located in front of the William Street/Key West Bight terminus (see Figure 2above) appears to be in stable condition. However, because the design parameters of this portion of the wall are not known, it is difficult to perform an accurate stability check. The 1995 upland redevelopment plans show a gravity wall with a key wall located below the toe, but no dimensions or specifications were shown. Figure 3 shows the wall above the waterline and Figure 4 shows what appears to be a steel key wall located just below the toe of the wall.

In front of the Waterfront Market

Date of Inspection: February 20-24, 2016

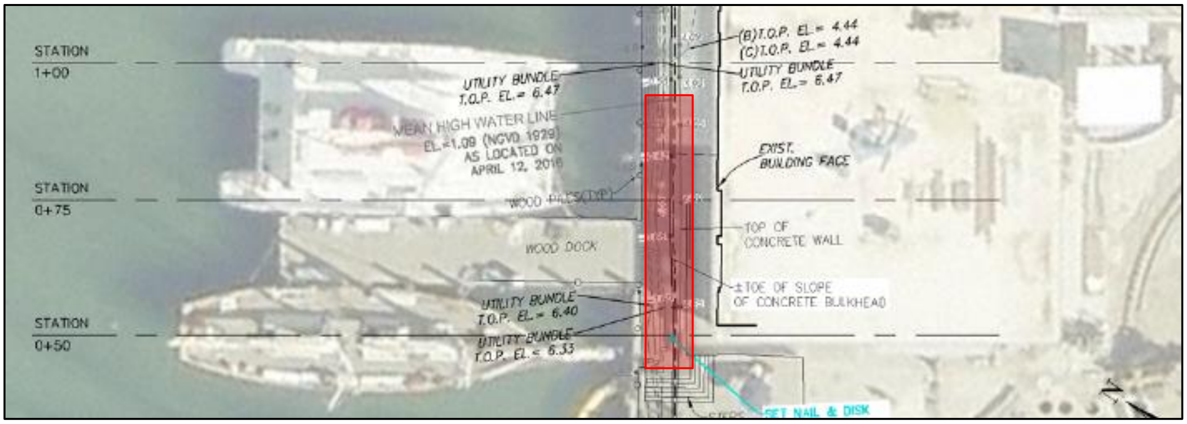


Figure 5: Station Exhibit, 0+40 to 0+89

Figure 6: Station 0+55, looking east at existing toe

Figure 7: Station 0+89, looking east

Note: Full size copies of these images are included in Appendix 1

Description	Turtle Kraals (40-89)
Condition Index	Moderate: Station 0+40 to 0+89
Service Life	25 years
Age of Structure	Unknown
Next Inspection	2020
3-Year Inspection	$ 12,500
Near Term Repair Cost	$ 0
Immediate Repair Cost	$ 0

<u>Condition</u>: The bulkhead located in front of the Waterfront Market & Brewery appears to be in stable condition. The 1995 upland redevelopment plans show a gravity wall with a key wall located below the toe, but no dimensions or specifications were shown. Figure 6 shows the wall above the waterline. Figure 7 shows a previously installed toe wall at the mudline.

Assessment Report – Station 0+82 to 1+31

In front of the Waterfront Market & Brewery

Date of Inspection: February 20-24, 2016

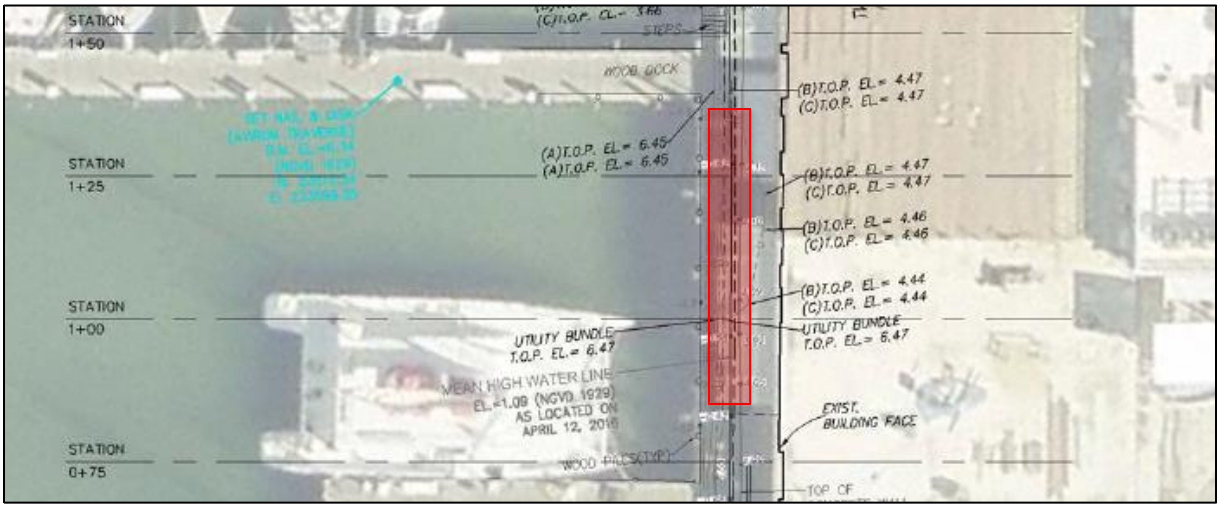


Figure 8: Station Exhibit, 0+82 to 1+32

Figure 9: Sediment depths vary, here shown 12in

Figure 10: Station 1+10, Post construction toe wall

Note: Full size copies of these images are included in Appendix 1

Description	Turtle Kraals (82-131)
Condition Index	Moderate: Station 0+82 to 1+31
Service Life	25 years
Age of Structure	Unknown
Next Inspection	2020
3-Year Inspection Cost	$ 12,500
Near Term Repair Cost	$ 0
Immediate Repair Cost	$ 0

<u>Condition</u>: The condition of the bulkhead located in front of the Waterfront Brewery appears to be in stable condition. For this approximately 50 foot segment a concrete/steel sheet pile combination wall was installed along the toe of the existing wall, presumably to fill any voids that had developed.

From Waterfront Brewery past the Public Restrooms

Date of Inspection: February 20-24, 2016

Figure 11: Station Exhibit, 1+31 to 3+00

Figure 12: Station 1+40, looking east

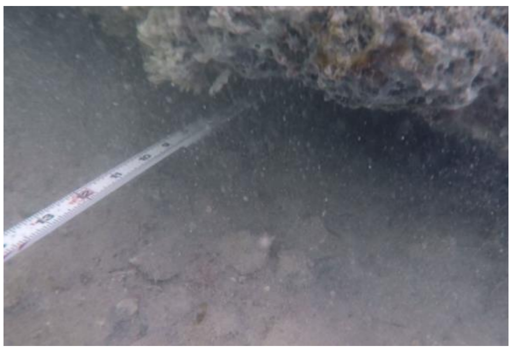


Figure 13: Station 2+33, looking east at existing toe

Note: Full size copies of these images are included in Appendix 1

Description	Turtle Kraals (131-300)
Condition Index	Moderate: Station 1+31 to 3+00
Service Life	25 years
Age of Structure	Unknown
Next Inspection	2020
3-Year Inspection Cost	$ 12,500
Near Term Repair Cost	$ 0
Immediate Repair Cost	$ 0

<u>Condition</u>: The bulkhead located in front of the Waterfront Market & Brewery appears to be in stable condition. There are discontinuous voids in this area but the penetration was nominal. Divers were unable to confirm the presence of a key wall but the depth of the void match those seen in front of Williams Street. It is recommended that this segment of the wall be inspected again in three years to determine if this condition has worsened.

From Public Restrooms to Turtle Kraals

Date of Inspection: February 20-24, 2016

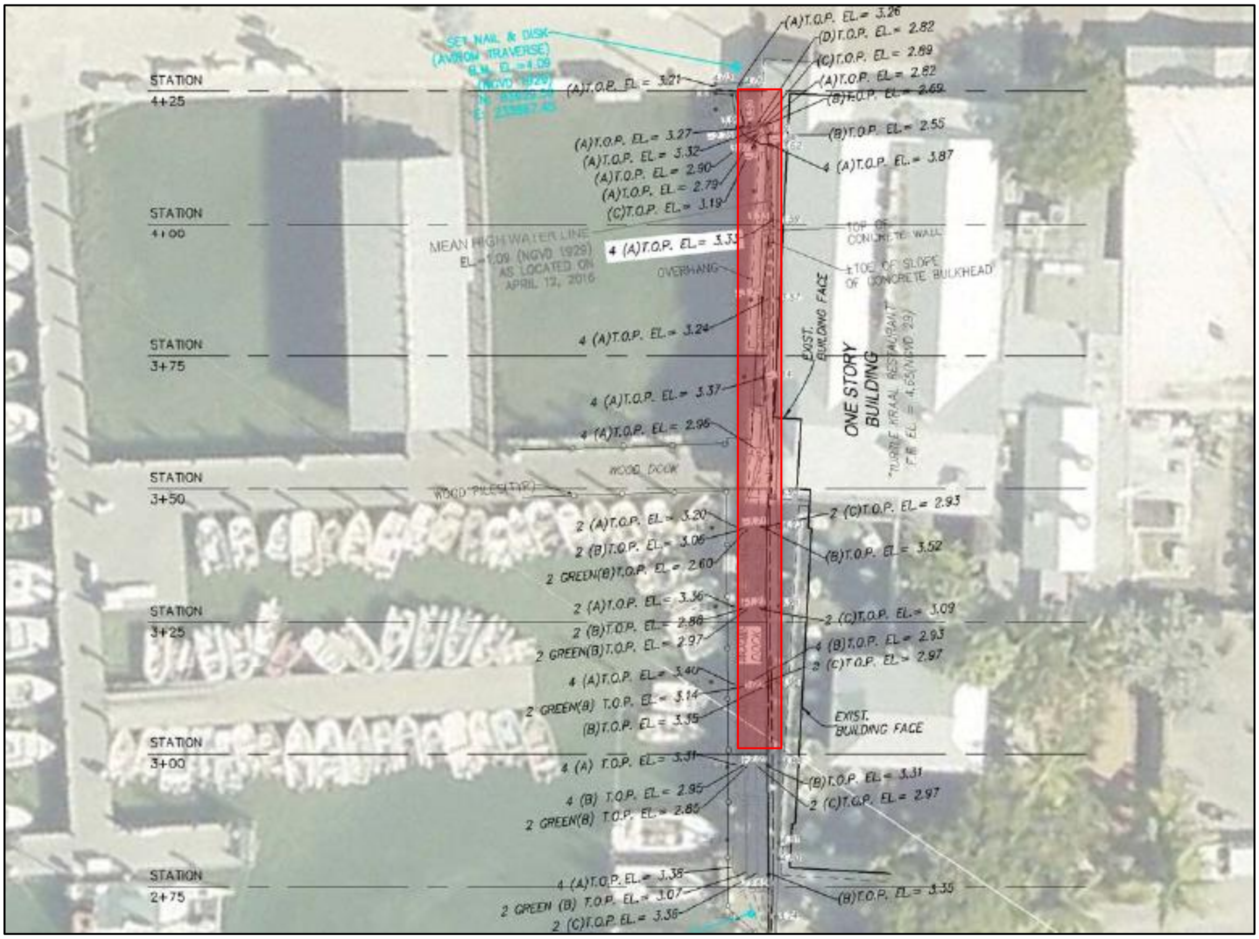


Figure 14: Station Exhibit, 3+00 to 4+40

Figure 15: Station 3+18, looking east

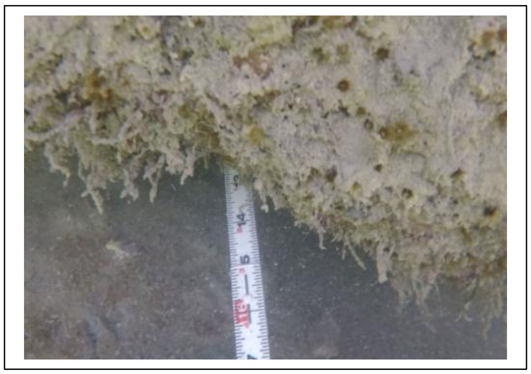


Figure 16: Station 4+05, looking east at existing toe

Note: Full size copies of these images are included in Appendix 1

Description	Turtle Kraals (300-440)
Condition Index	Moderate: Station 3+00 to 4+40
Service Life	25 years
Age of Structure	Unknown
Next Inspection	2020
3-Year Inspection	$12,500
Near Term Repair Cost	$ 5,000
Immediate Repair Cost	$ 0

Condition: The bulkhead located in front of the Waterfront Market & Brewery appears to be in stable condition. The above water line photo (Figure 15) shows a large crack that extends from the mudline to the top of the wall. This crack does not appear to be a construction joint and judging from the condition of concrete in the spalled areas, appears to have been there for a long time. We recommend that this portion of the wall continue to be monitored and re-inspected in three years.   We also recommend that this large crack and several others along the face of the wall be sealed during the next phase of construction.

End of Executive Summary

3.0 INTRODUCTION

The project area is located along the northwestern shore of Key West and lies within Key West Bight adjacent to the Key West Marina and the Key West Ferry terminal. Global Positioning System (GPS) coordinates for the site are as follows: Latitude 24° 33.691' North, Longitude 81°48.055' West. This is considered Phase 2 of the seawall assessment project EXTENDING from William Street in front of the Waterfront Market and Brewery, past the public restrooms up to the intersection of Turtle Kraals Restaurant at Margaret Street.

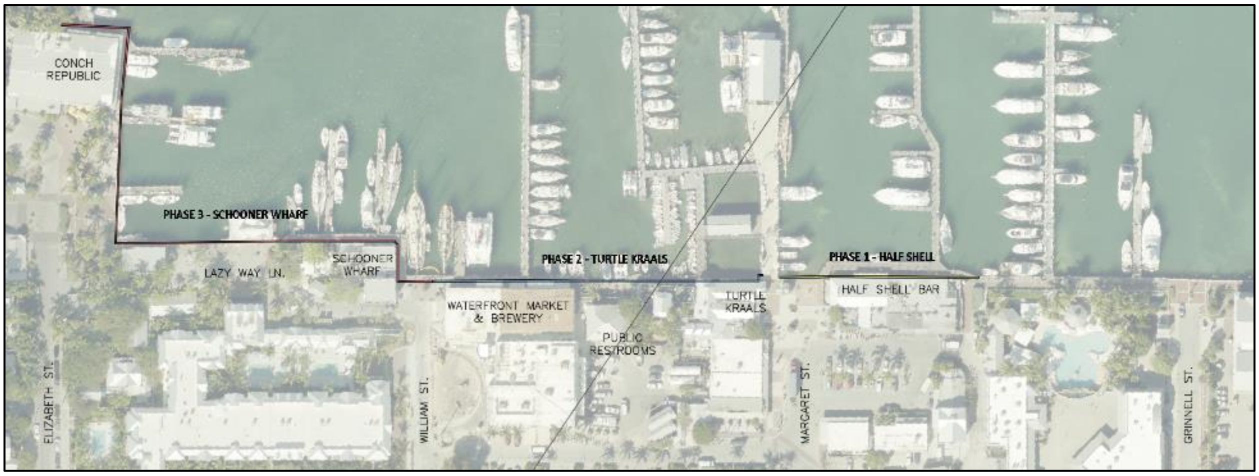


Figure 17: Project limits and surrounding area

The waters adjacent of the Key West Bight and project area are classified by the Florida Department of Environmental Protection (FDEP) as a Class III water body (Recreation, Propagation and Maintenance of a Healthy, Well-Balanced Population of Fish and Wildlife), and they are located inside the limits of the National Oceanic and Atmospheric Administration (NOAA) Florida Keys National Marine Sanctuary (FKNMS).

Permitting along the shoreline, inside the FKNMS, involves submitting an environmental resource permit application to the Florida Department of Environmental Protection (FDEP) and the US Army Corp of Engineers (ACOE). This application along with the project benthic resource survey and coral inventory is submitted by the ACOE directly to the FKNMS and then forwarded to the National Marine Fisheries Service (NMFS), pursuant to Section 7 of the Endangered Species Act (ESA). During the permitting process, coral colonies are identified and mitigated for; in some cases the corals may be relocated and in others a fee mitigation is required.

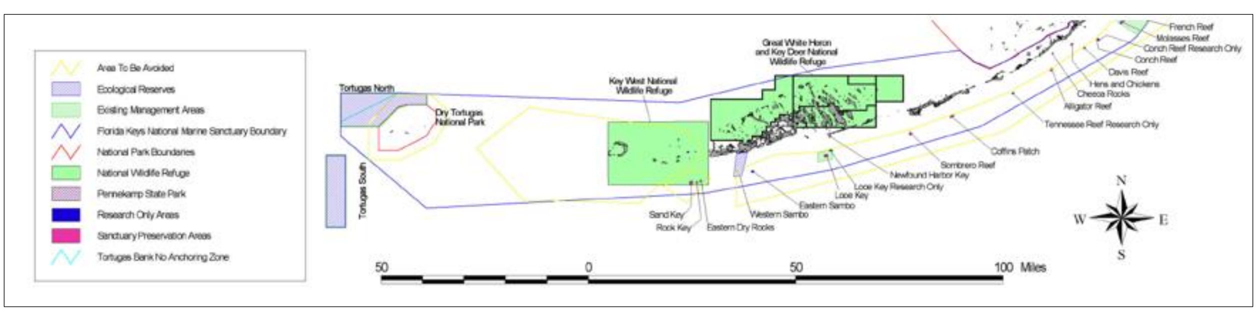


Figure 18: Limits of the FKNMS (http://floridakeys.noaa.gov/fknms_map/sanctuaryzoneboundaries.pdf)

4.0 SITE DESCRIPTION

The survey area is an approximately 440 linear feet of bulkhead with an existing discontinuous dock and boardwalk structures along its length (see Figure 1). The bulkhead is a part of hardened shoreline surrounding the marina basin that supports a variety of amenities. Water depths ranged from 3 to 10 feet of seawater (fsw). The deepest water in Phase 2 is located near Williams Street and the shallowest water is located in front of Turtle Kraals. Sediment composition along the base of the bulkhead is predominantly biogenic mud/fine sand mixed with debris (concrete, rubble, pipes, etc.). Underwater visibility was approximately 5 feet; water temperature was 76° Fahrenheit, with no noticeable current detected during the survey.

The shoreline of the Key West Bight in the project area is known locally as the "Harbor Walk" and is frequented by tourist and patrons visiting the many shops, sightseeing tours, and restaurants located along the seawall overlooking the marina. The Harbor Walk itself is composed of a winding walkway located on the seawall and boardwalks over the water running parallel to the bulkhead.

5.0 INSPECTION METHODS

Field activities were performed using a 19-ft. catamaran in order to safely access the survey area. A three man dive team (including two divers and a tender) and an engineer were on site during the inspection. The certified scientific divers trained in structure inspection and marine biological resource identification used scuba to visually inspect the wall and cataloged deficiencies. They cataloged all stony corals and noted other marine benthic flora and fauna along the face of the approximately 3,500 square-foot bulkhead face and a 10-ft buffer on each end of the bulkhead and along the seafloor apron. Data were collected along a single transect positioned from southwest (0 ft.) to northeast (440 ft.) along the base of the bulkhead as shown on Figure 1.

The transect tape was used by divers to mark and ultimately record the location of each seawall deficiency and coral colony along the bulkhead and buffer area by first marking the location of the observation linearly and then its location vertically from the mudline. Videos of the bulkhead and upland facilities were taken above and below the waterline and include the interspatial area between the waterline and the underside of the docks and boardwalks. Still images were collected from these video feeds and combined with underwater photography which were used together to record specific and representative images of wall condition and benthic resources present within the survey area. Divers followed a systematic approach measuring all deficiencies along the wall using the tape measure and distances from the bottom. This information was then combined with the previously collected georeferenced surveyed data provided by Avirom & Associates, Inc.

During the scientific portion of the inspection, the divers also followed the National Oceanic and Atmospheric Administration (NOAA) Protocol for Benthic Surveys of Coral Resources in Florida Keys National Marine Sanctuary (FKNMS) for seawalls and shoreline structures (rip rap, bulkheads, boat ramps, bridges) dated April 29, 2011. The results of the Benthic and Coral Survey is attached to this report as Appendix 4 (Coral Inventory and Benthic Resource Survey for Turtle Kraals, Key West, FL).

6.0 OBSERVATIONS AND RECOMMENDATIONS

The following table catalogs the observed deficiencies along the face of the bulkhead and provides the estimated size and volume of each. These defects are identified from A-AC and are located on Appendix 3 (Seawall Plan and Cross Section Views, w/ Seawall Elevation (Front) Views with Survey).

An example of how to read this table: Defect A is located at station 0+02, defect B is located from 0+03 to 0+36 and so on.

DEFECT	DEFICIENCY TYPE	DESCRIPTION	APPROXIMATE LOCATION (STATIONS)	HEIGHT (IN)	WIDTH (IN)	DEPTH/ PENETRATION (IN)	APPROX. VOLUME (CF)
A	VOID/SPALL	VOID	0+02 TO 0+03	8	12	12	0.7
B	SCOUR	EXPOSED SHEETPILE	0+03 TO 0+36	24	396	12	66.0
C	OVERPOUR	EXCESS CONCRETE	0+36	N/A	N/A	N/A	
D	OPEN CULVERT	OPEN CULVERT WITH NO MANATEE GRATE	0+41 TO 0+44	36	36	N/A	
E	VOID/SPALL	VOID	0+53 TO 0+55	0.5	24	17	0.1
F	CRACK	CRACK/SPALL AT CAP WITH SOME EXPOSED REBAR	0+65 TO 0+75	29	120	N/A	
G	VOID/SPALL	SPALL WITH SOME EXPOSED REBAR	0+80 TO 0+82	24	24	N/A	
H	VOID/SPALL	VOID IN TOE/BENCH	0+83	3	10	8	0.1
I	EXPOSED SHEETPILE	POSSIBLE SHEETPILE FOR REPAIR WITH CONCRETE CAP	1+08 TO 1+31	76	276	N/A	
J	VOID/SPALL	AREA OF VOIDS	1+33 TO 1+50	16	204	34	64.2
K	CRACK	2-3" TOP-TO-BOTTOM CRACK	1+51	82	3	N/A	
L	VOID/SPALL	VOID	1+51 TO 1+54	12	36	28	7.0
M	CRACK	6" TOP-TO-BOTTOM CRACK	2+04	72	6	N/A	
N	VOID/SPALL	AREA OF VOIDS	2+09 TO 2+29	8	240	6	6.7
O	VOID/SPALL	VOID	2+32 TO 2+35	8	36	33	5.5
P	VOID/SPALL	SPALL	2+58 TO 2+62	32	48	5	4.4
Q	VOID/SPALL	VOID	2+67 TO 2+70	9	36	17	3.2
R	VOID/SPALL	SPALL	2+70 TO 2+73	32	36	5	3.3
S	VOID/SPALL	PIPE WITH SPALLING	2+90	8	8	2	0.1
T	CRACK	2.5" TOP-TO-BOTTOM CRACK	3+19	82	2.5	N/A	
U	LEAKING CULVERT	MOSTLY-SEALED 36" CULVERT	3+38 TO 3+41	36	36	N/A	
V	VOID/SPALL	SPALL	3+71 TO 3+73	24	17	3	0.7
W	VOID/SPALL	SPALL	3+74 TO 3+76	6	24	3	0.3
X	CRACK	1-2" DIAGONAL CRACK	3+73 TO 3+78	60	2	N/A	
Y	VOID/SPALL	VOID	3+86	8	12	UNKNOWN	
Z	VOID/SPALL	SPALL	3+93 TO 3+99	36	72	3	4.5
AA	VOID/SPALL	SPALL	4+00 TO 4+02	24	24	12	4.0
AB	VOID/SPALL	AREA OF VOIDS	4+02 TO 4+17	12	180	8	10.0
AC	VOID/SPALL	SPALL	4+20 TO 4+24	24	48	8	5.3
						TOTAL AREA OF VOID/SPALL	120.1
						Total Volume (CY)	4.4

TYPE OF DEFICIENCY	
(yellow)	EXPOSED SHEETPILE WALL BOTTOM
(orange)	VOID/SPALL
(green)	CRACK
(grey)	OTHER/POSSIBLE DEFICIENCY

Table 1 – Deficiencies Table

The most concerning observation was the approximately 40-feet long segment of wall identified as Defect B where the steel key wall was completely exposed. At this location we used the "QuickRWall Version 5.0" design software to perform a basic stability check on the wall. For the computer analysis, we assumed the wall was a monolithic, unreinforced concrete wall set on top of a steel key set just below the toe (as shown in Figure 19). Because of the fractured limestone behind the wall and the various voids and cracks along its face we assumed the passive and active water pressures cancel each other out and no water pressures were applied. This portion of the wall is only loaded by light pedestrian traffic an active surcharge pressure was applied. In other segments of the wall a stem axial load was added to estimate the load from the columns added to the top of the wall that now support the Harbor Walk (Boardwalk).

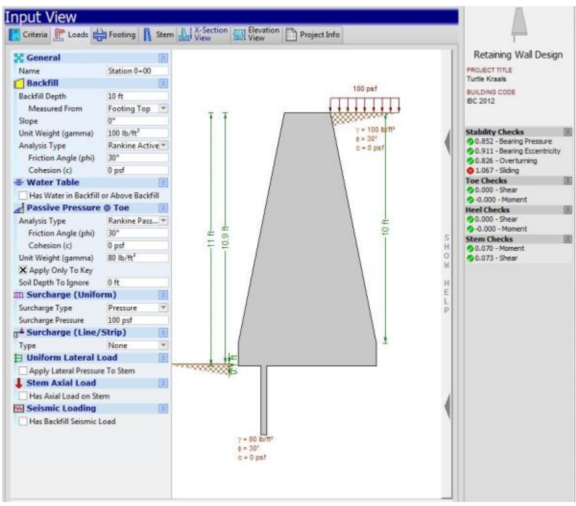


Figure 19 – TK Wall before repair

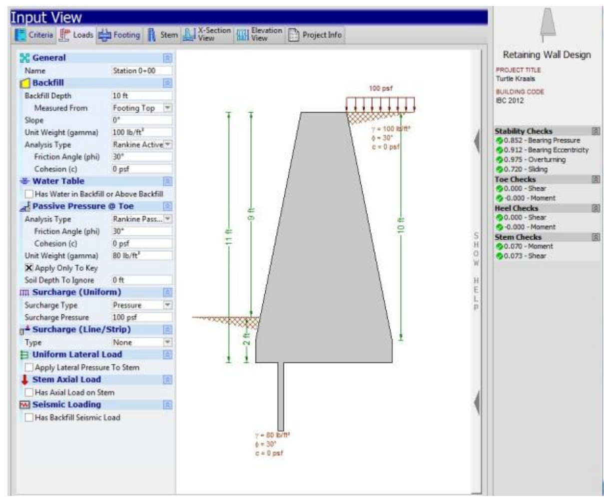


Figure 20 – TK Wall after repair

In its existing condition the analysis shows the wall does not pass the stability check for sliding and remedial measures are required to improve its long term stability. The key wall is actually more exposed than shown below and in some cases the mudline is 12-inches or more below the base of the concrete portion of the wall. The analysis shows that by adding mass back in front of the wall at the toe (Figure 20) the stability can be improved. It is believed that this portion of the basin was excavated deeper along the wall to accommodate the large keel sailboats presently moored there. The basin was most likely excavated at a descending angle from the base of the base of the wall to the current bottom and the sloped portion at the toe has since been eroded by prop wash or other local conditions.

This portion of the wall is the most exposed and therefore we are proposing a concrete toe wall be installed from station 0+00 to 0+40 feet at the toe of the wall extending from a point approximately two feet above the wall toe (base of the concrete portion of the wall near the top of key) down to the hard bottom of the basin. The basin bottom is now relatively flat at this point and minimal additional scour or limestone degradation is expected in the foreseeable future.

This portion of the wall (from 0+00 to 0+40) is the deepest part of the basin inspected and as the inspection moved to the east, the water depths decreased and the toe of the wall disappeared below the mudline for the most part. Stability checks were performed again for representative segments of the wall and no additional concerns were seen.

There were still a few patches of the wall with voids at the mudline but these voids were relatively small and not continuous. In each location where a void was discovered the divers attempted to determine if a key wall was located under the toe by probing into the void. In every case it was impossible to visually determine the presence of a steel key. In many cases the depth of the void matched the depth from the face of the wall at the toe to the face of the steel key seen at 0+00 to 0+40 but observations were inconclusive. In a few cases however, the voids identified extended a little further under the wall indicating that the key was not present. Where the toe of the wall extended below the mudline it would be impossible to determine for sure without limited excavation. No excavation (even limited to minor inspection) can be conducted without a permit from the FKNMS. It is difficult to say if the key wall extends for the entire length of the Turtle Kraals project area. It is important to note here that during the engineering inspection performed (by others) for the Half Shell Bar wall segment, no key wall was observed. And in that case the voids identified at the toe were much more considerable than those seen here.

The wall segment extending from 0+53 to 0+65 included a monolithic concrete toe wall approximately 36-inches x 36-inches that disappeared into the mudline. This indicates that this portion of the wall had a considerable void that needed to be rehabilitated at some point in the past. The visible portion of the wall in this area appeared to be in good and stable condition. There are several spots along the top of the wall (near 0+69 and 0+79) where the cap of the wall is cracked or a large spall has occurred. These portions of the wall are located under the boardwalk and do not pose a structural risk. It is recommended that they be monitored again during the next inspection and scheduled for patchwork in future budgets. No immediate repairs are recommended.

The wall segment extending from 0+82 to 1+32 includes a continuous toe wall composed of both concrete and steel sheets with a concrete cap. Again, it is assumed that this portion of the wall must have had a large void at the toe that needed to be repaired. The age of the repair is unknown.

The water depths along the face of the wall continued to shallow for the remaining portions of the wall from 1+32 to 4+40 with several cracks and voids along the mudline. The voids were discontinuous and relatively small and should be scheduled for re-inspection in 3-5 years.

7.0 CONCLUSION

Based on our visual inspection, a review of the 1995 shoreline reconstruction plans and the conceptual stability analysis we recommend that the toe wall be installed in the area shown below. This work should be scheduled as soon as practical but may be deferred until the remainder of the wall is inspected and then coupled with that repair. The following exhibit (Figure 21) shows the plan view of the area with the bulkhead and existing dock identified in the plan view and an isometric view of the proposed repair in the face view.

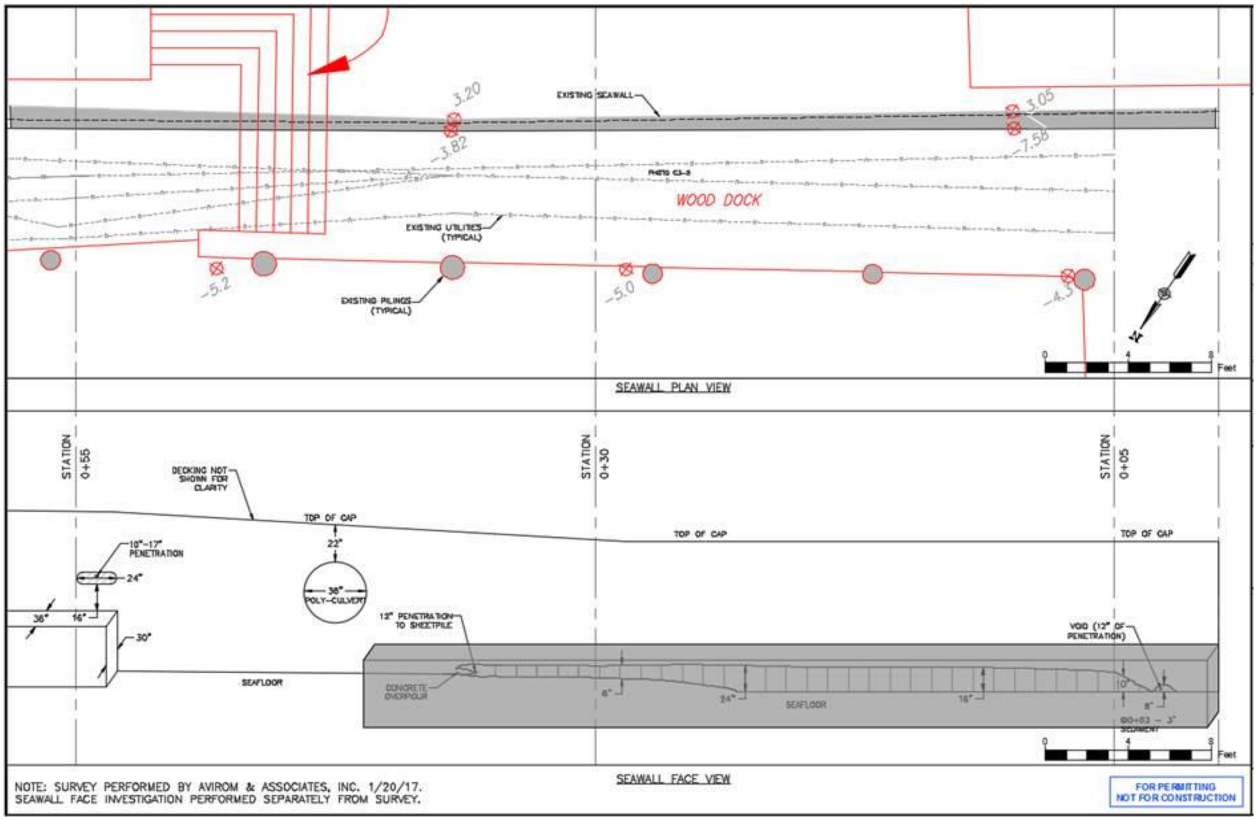


Figure 21 – Plan view with wall repair isometric


Photo 1: At 5' from start of Phase 2 wall segment beginning at corner of William St.

As you can see in Figure 22 below, at (0+05), the elevations captured by the survey show the seafloor sloping down from the basin toward wall face. This is an indication that at some time in the past a large vessel was moored relatively close to wall and it is possible that turbulent prop wash helped to create the scour that is now observed. During the inspection process the sailboat now moored at this location ties up bow first and its prop is over fifty feet from the wall face. At 0+30 in the same Figure 22, the seafloor is shown to slope away from the wall and the key is far less exposed.

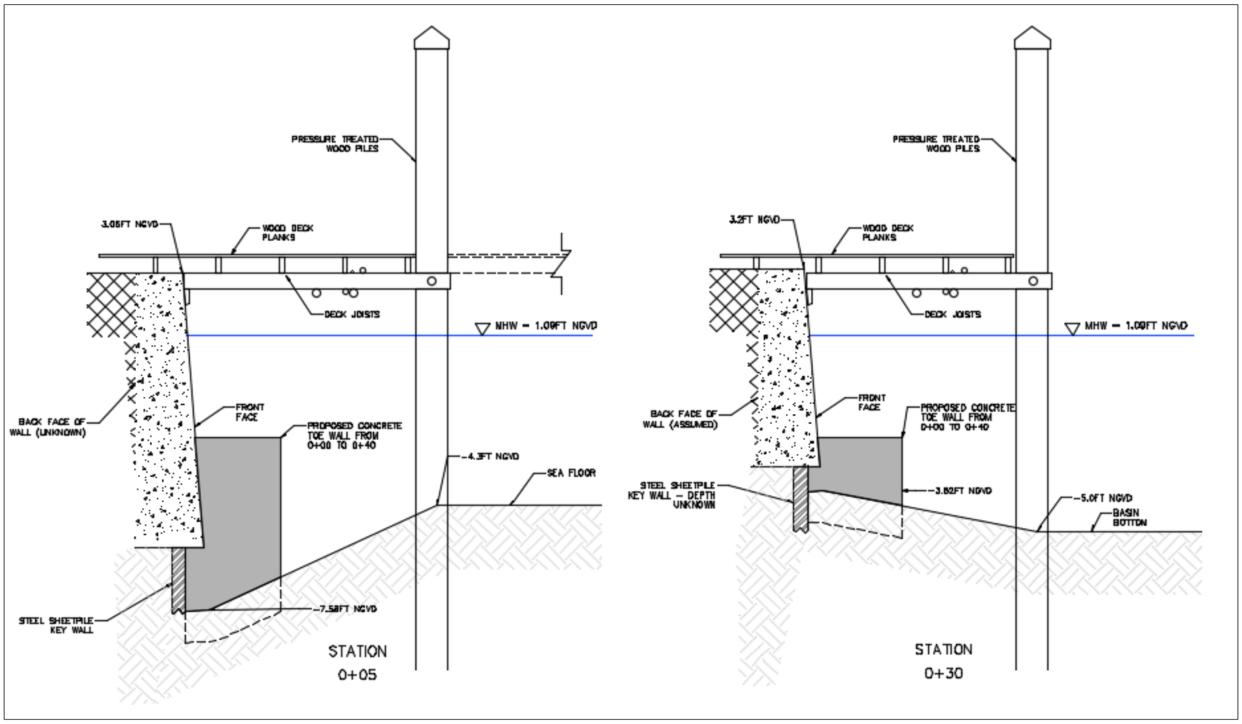


Figure 22 – Cross sections of proposed wall repair

Photo 2: At 30' from the start of Phase 2 wall segment beginning at the corner of Williams St.

The remaining portions of the wall, although not in perfect condition remain stable with no observed adverse conditions. It is recommended that the wall be re-inspected in 3-5 years to check for further deterioration at the mudline. Ultimately the same repair will be required for this entire segment but at this time it can be deferred.

ENVIRONMENTAL & PERMITTING DISCUSSION

A full riparian survey with limited bathymetry has been collected with the preparation of this report. This will need to be provided with the Environmental Resource application to the FDEP and the ACOE along with an engineered construction design so they can evaluate any risks or impacts to the environment. In general the nationwide permitting process allows for improvements that extend up to 18-inches in front of the existing wall. There is some flexibility in this guideline depending on engineering constraints and/or local site conditions.

The National Oceanic and Atmospheric Administration (NOAA) will need to be consulted because of the corals known to exist on the wall and in the area. During the bulkhead inspection benthic resource data was collected and a full resource survey report has been completed. This report will need to be submitted to NOAA FKNMS and the NMFS for consultation and mitigation measures will need to be negotiated.

LIFE CYCLE EXPECTATIONS

The existing wall in its current condition is stable. It has remained in place for decades and with periodic maintenance in the form of the repairs described above should remain stable for decades to come. However, based on the constant use of this facility and possible changes to mooring configurations it is recommended that a periodic inspection program be implemented that will capture and catalog wall conditions. This data can be cross referenced every 3-5 years and areas of concern can be scheduled and budgeted for maintenance.

8.0 COST ESTIMATE

The estimated cost to construct the toe wall from Station 0+00 to 0+40 is $84,000. A cost breakdown is provided in the table below. This cost estimate assumes the contractor will have access to the site for approximately 30-45 days from the William Street location. The contractor will be required to use divers to clean the face of the wall and prepare the seafloor for the installation of formwork. Concrete will then need to be placed and the formwork removed. It is recommended that this work be performed over the summer to minimize any impacts to the businesses in the area.

Turtle Kraals - Cost Estimate Tetra Tech Project #: 194-5365.10.1				
Pay Item	**QTY**	**UNIT**	**Cost/Unit**	**Cost - Total**
FDOT Class IV Concrete (including formwork)	27	CY	1,500	40,500
Reinforcement - GFRP	600	LF	6.50	3,900
			DIRECT COST	**44,400**
FOOH & HOOH (Overhead) Combined (6% Typical)	20%	LS	8,880	53,280
Mobilization / Demobilization (10% Typical)	10%	LS	5,328	58,608
Contractor Profit (17% Typical)	17%	LS	9,963	68,571
Bonds, Permits, and Insurance (2% Typical)	2%	LS	1,371	69,943
Contigency	20%	LS	13,989	83,931
			PROJECT COST	**83,931**
				Say 84,000
Commercial Dive Inspection w/ Report (per 200 LF)	1	LS	12,500	12,500

Table 2 – Cost estimate for wall repair

Appendix 1

Site Photographs

TURTLE KRAALS
Photo Log

Photo 1 – Turtle Kraals

Photo 2 – Turtle Kraals

Photo 3 – Turtle Kraals

Photo 4 – Turtle Kraals

Photo 5 – Turtle Kraals

Photo 6 – Turtle Kraals

Photo 7- Turtle Kraals

TURTLE KRAALS
Photo Log

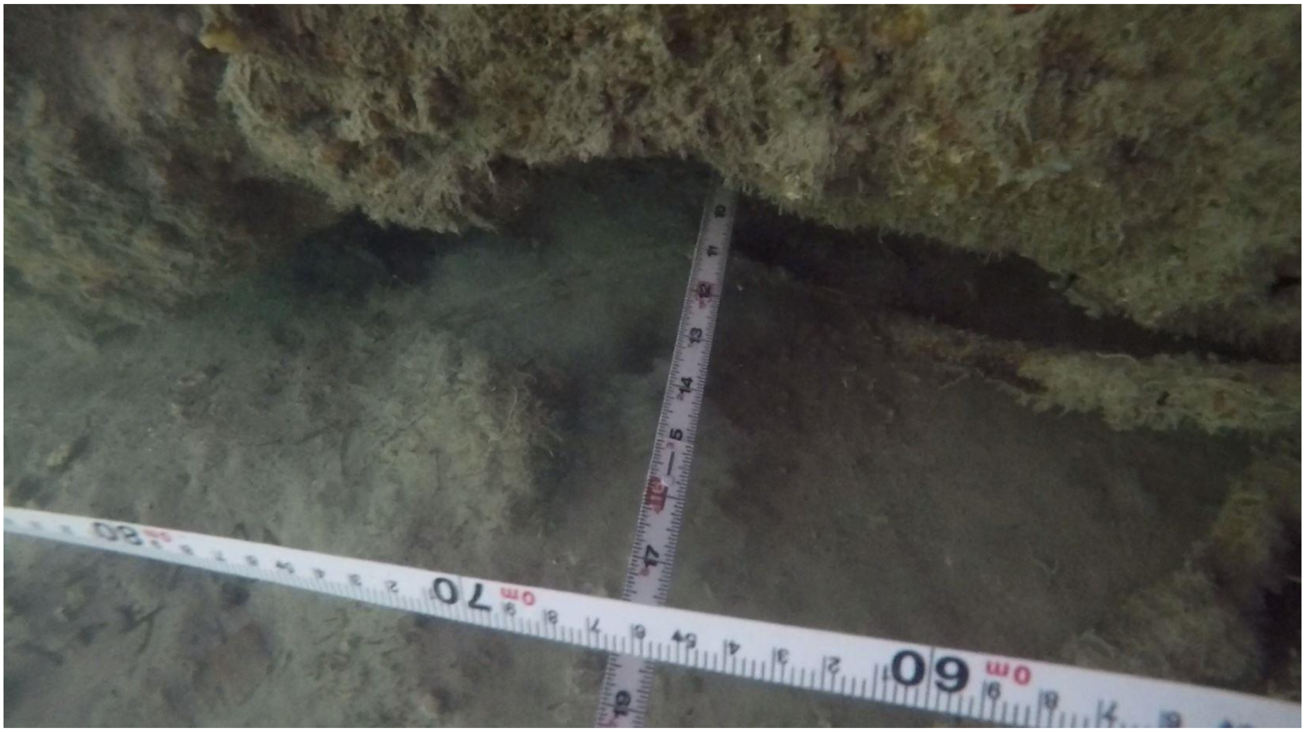
Photo 8 – Turtle Kraals

Photo 9 – Turtle Kraals

TURTLE KRAALS
Photo Log

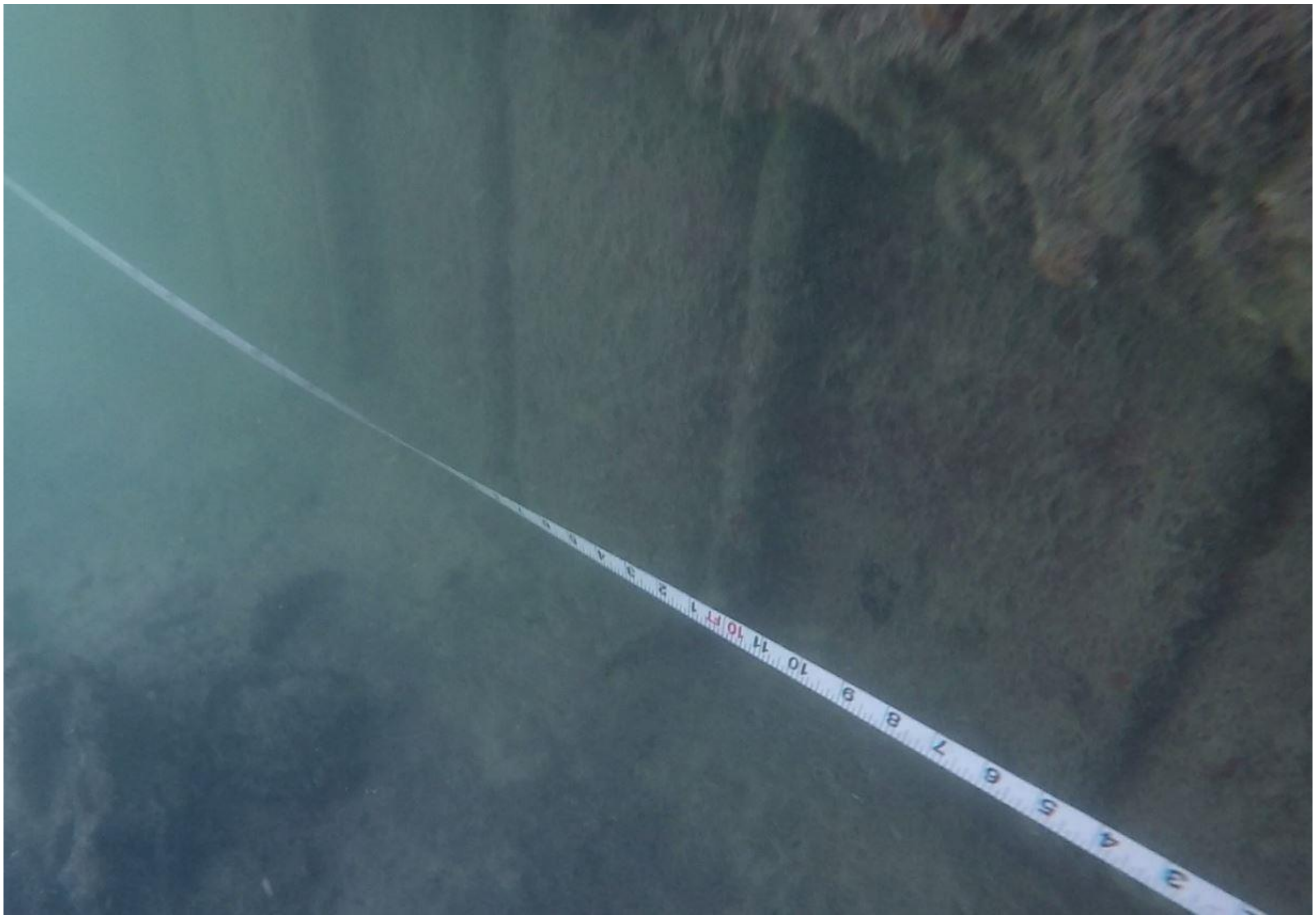
Photo 10 – Turtle Kraals

Photo 11 – Turtle Kraals

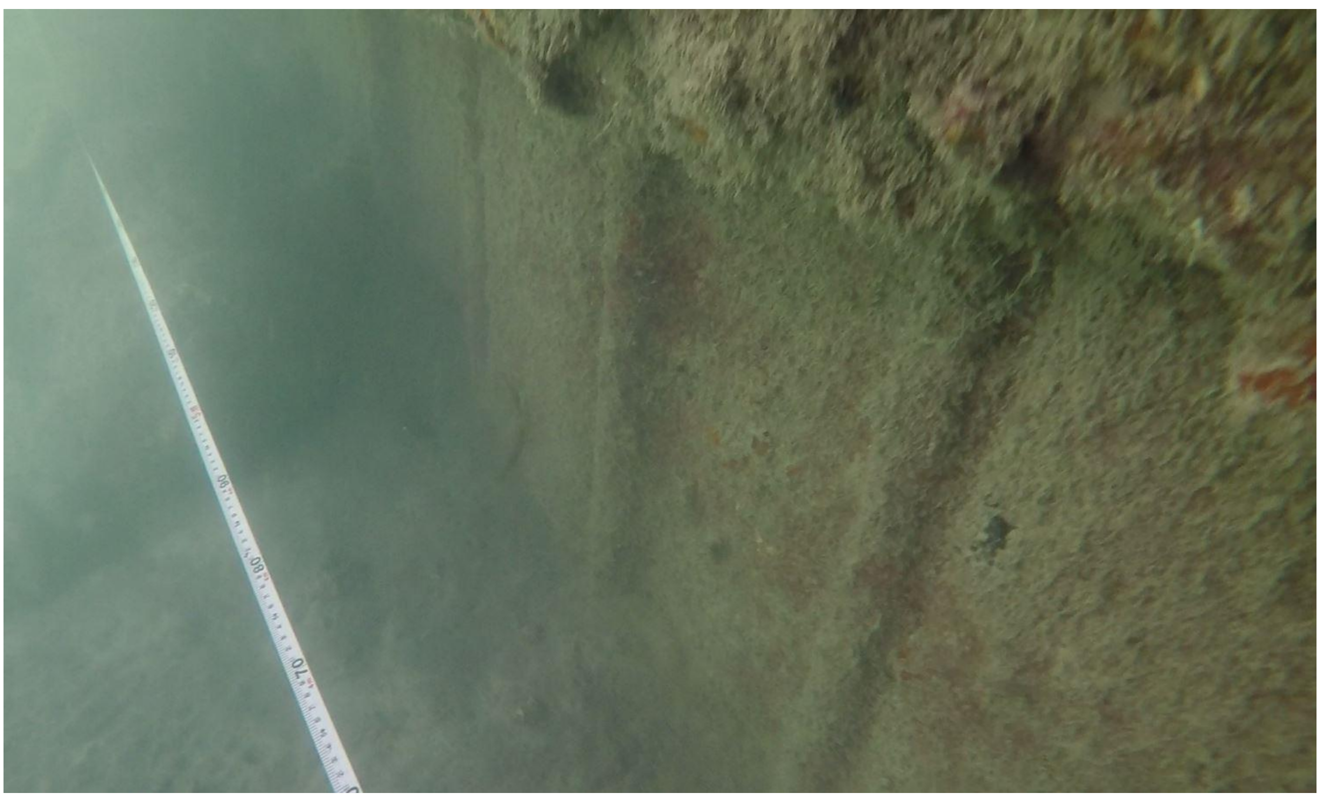


Underwater 1 - at 2ft 6in w 2ft wide x 12in deep void to sheetpile behind

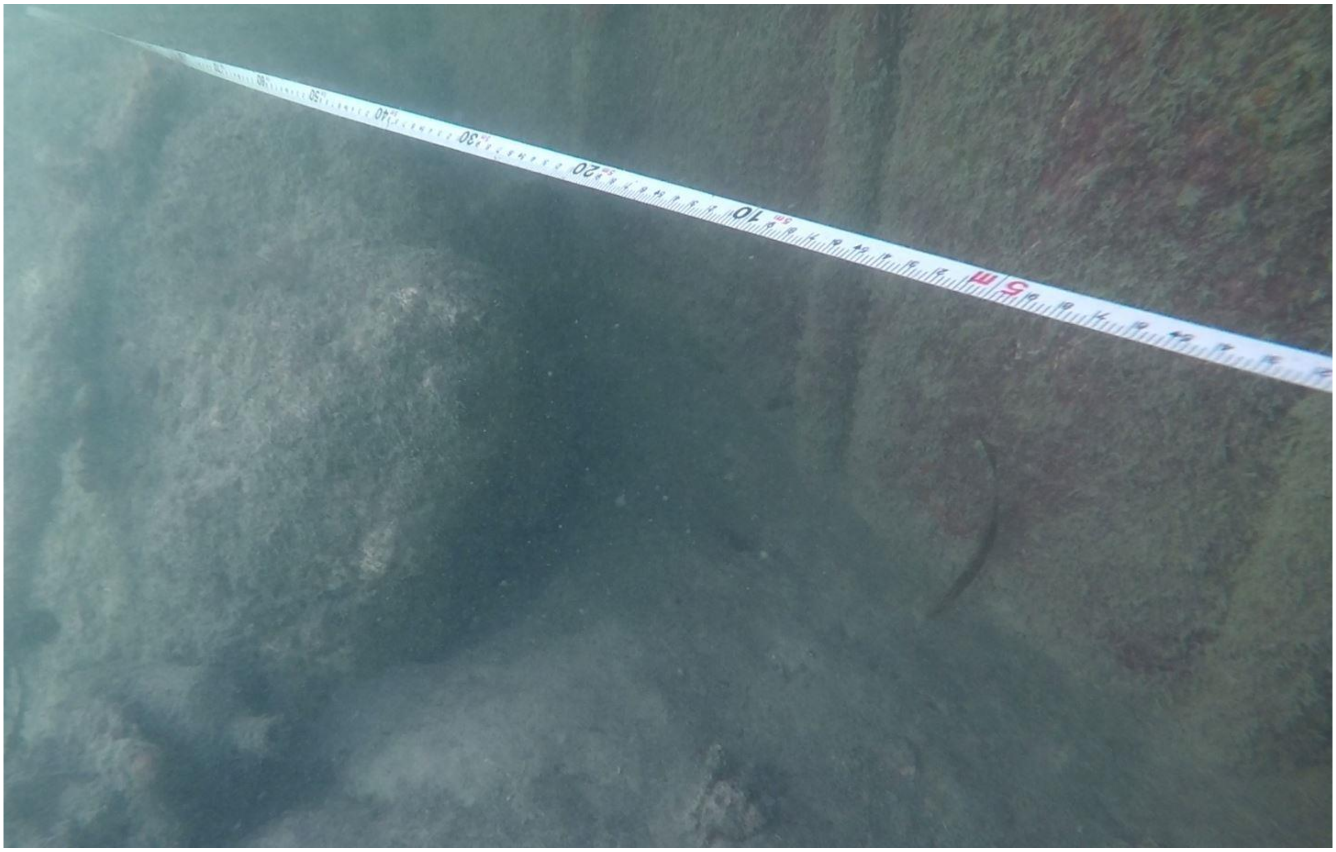
Underwater 2 - at 4ft 0in w continuous undermining at the toe and 4in sediment on top of rock



Underwater 3 - at 10ft shows steel sheetpile with concrete cap in good condition

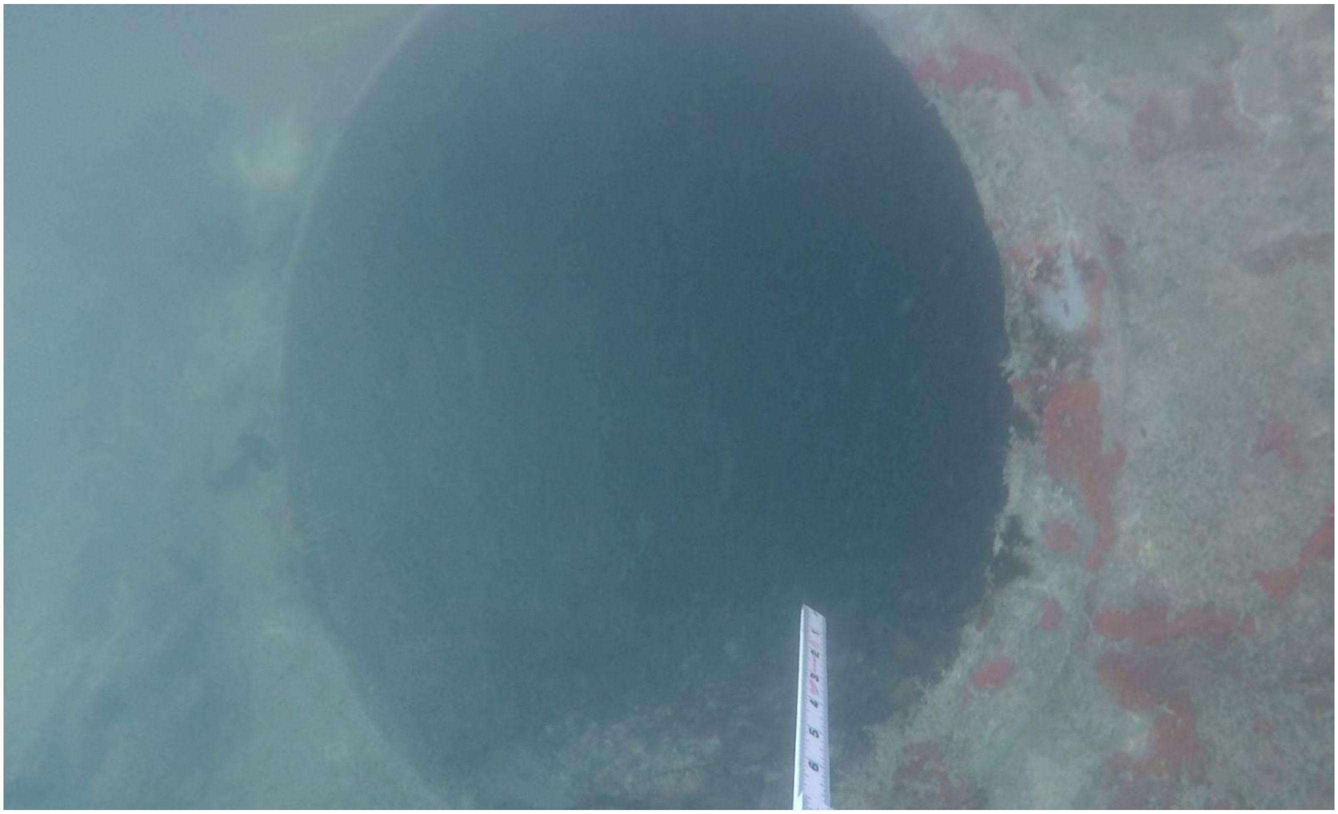


Underwater 4 - at 15ft 8in shows steel sheetpile and concrete cap in good condition



Underwater 5 - at 18ft 0in shows steel sheets driven through rock, rock sheered off left in front of wall

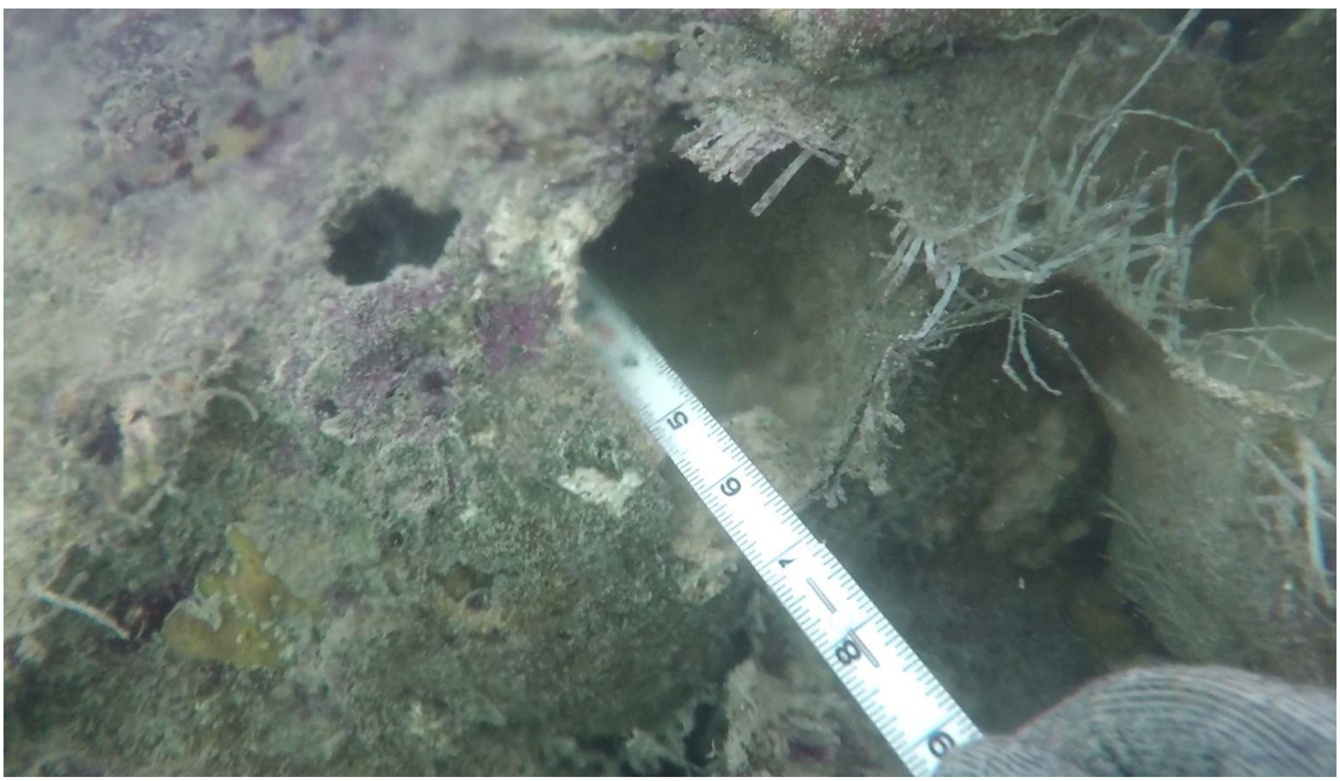
Underwater 6 - at 36ft steel sheet pile seems to have a void but it is only 6in deep at 36ft 8in



Underwater 7 - at 44ft shows corrugated HDPE storm drain

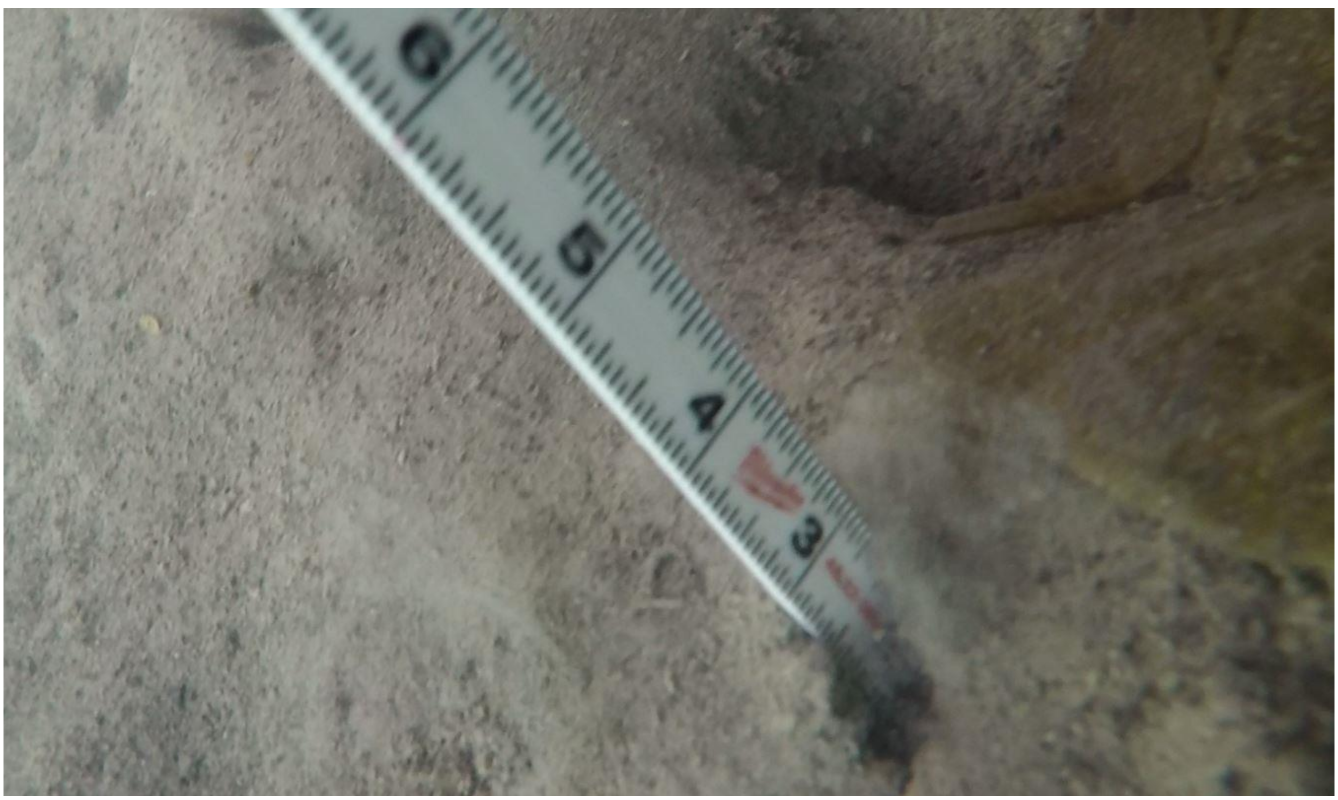
Underwater 8 - at 45ft above waterline looking east shows condition of concrete in good shape

Underwater 9 - at 45ft above waterline looking west shows condition of concrete in good shape



Underwater 10 - at 50ft shows fiberglass sheet could not see wall

Underwater 11 - at 55ft shows concrete toe wall

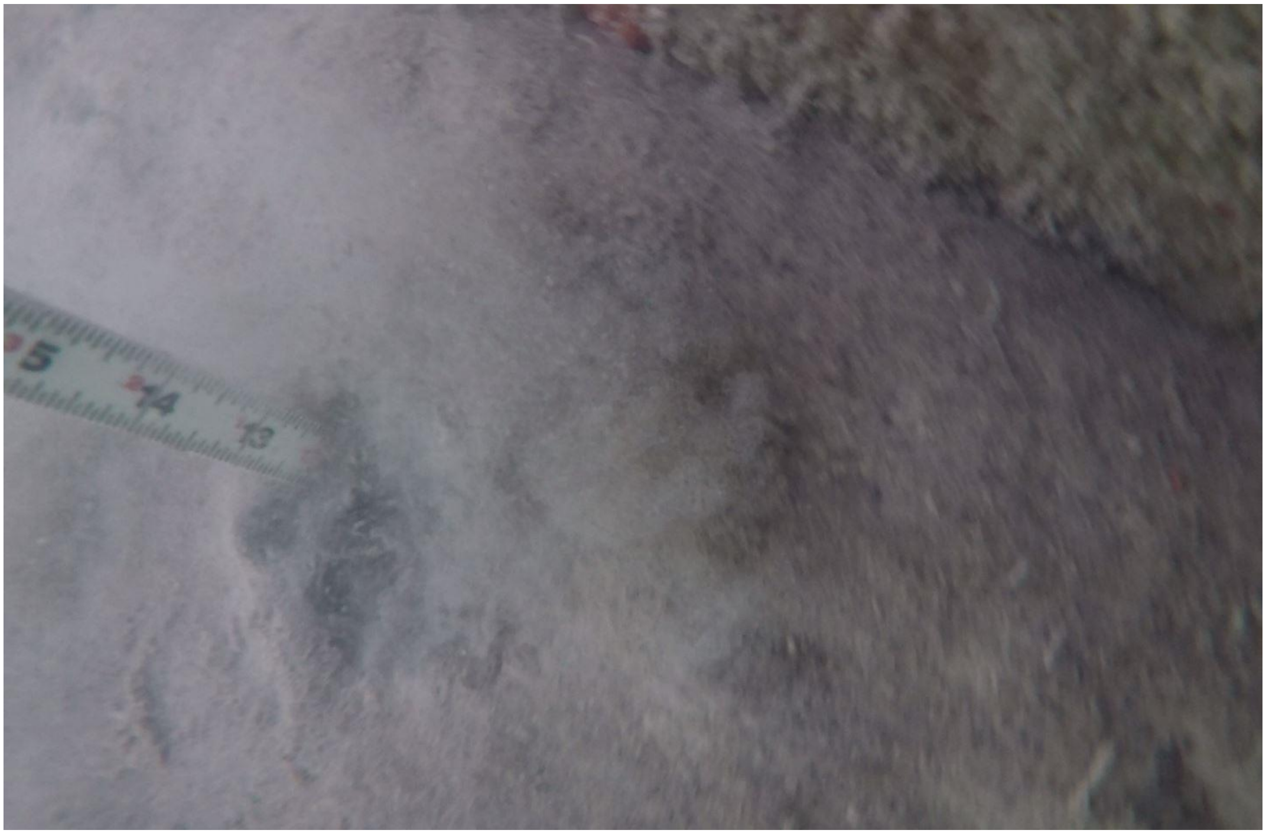


Underwater 12 - at 69ft shows sediment thickness at approximately 3in

TURTLE KRAALS
Photo Log

Underwater 13 - at 89ft above waterline looking east shows wall condition and utilities

Underwater 14 - at 89ft above waterline looking west shows wall and utilities

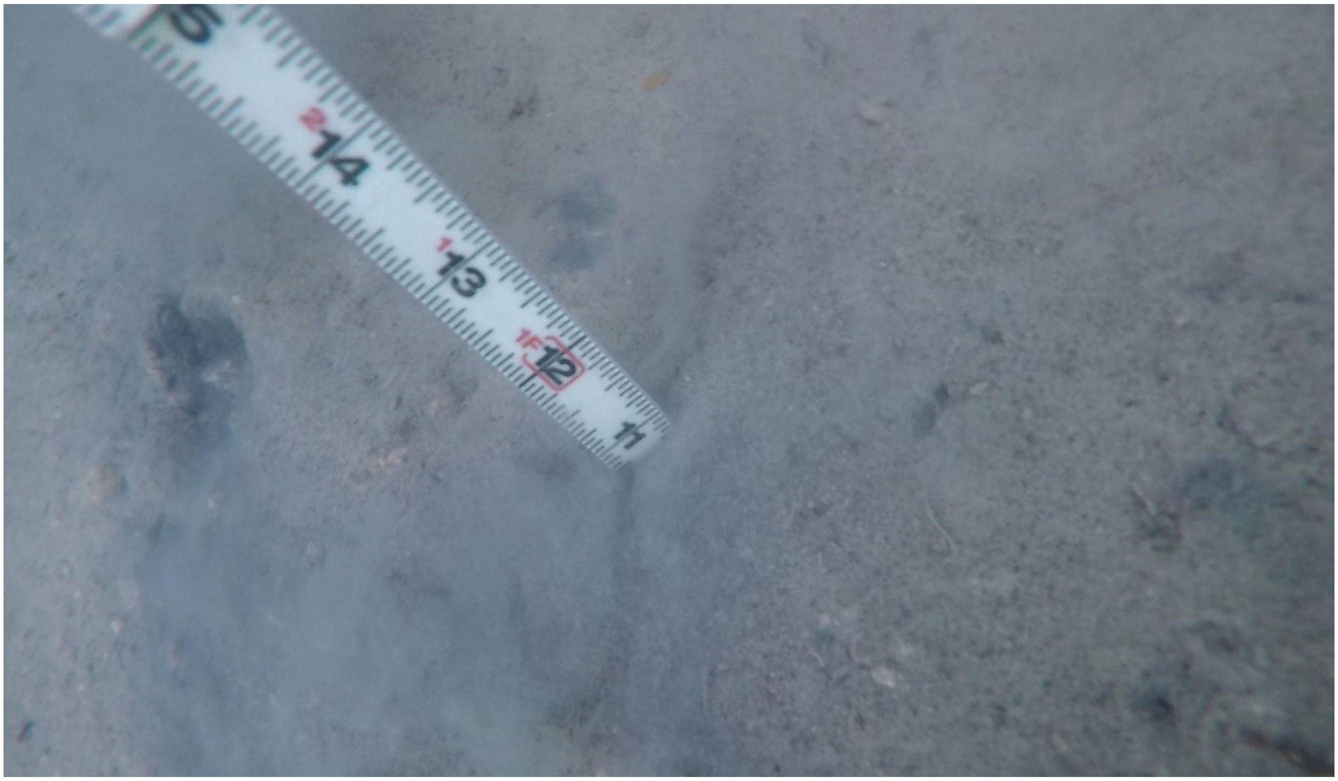


Underwater 15 - at 89ft shows sediment in front of toe wall 12in deep


Underwater 16 - at 110ft shows steel sheet toe wall with concrete cap

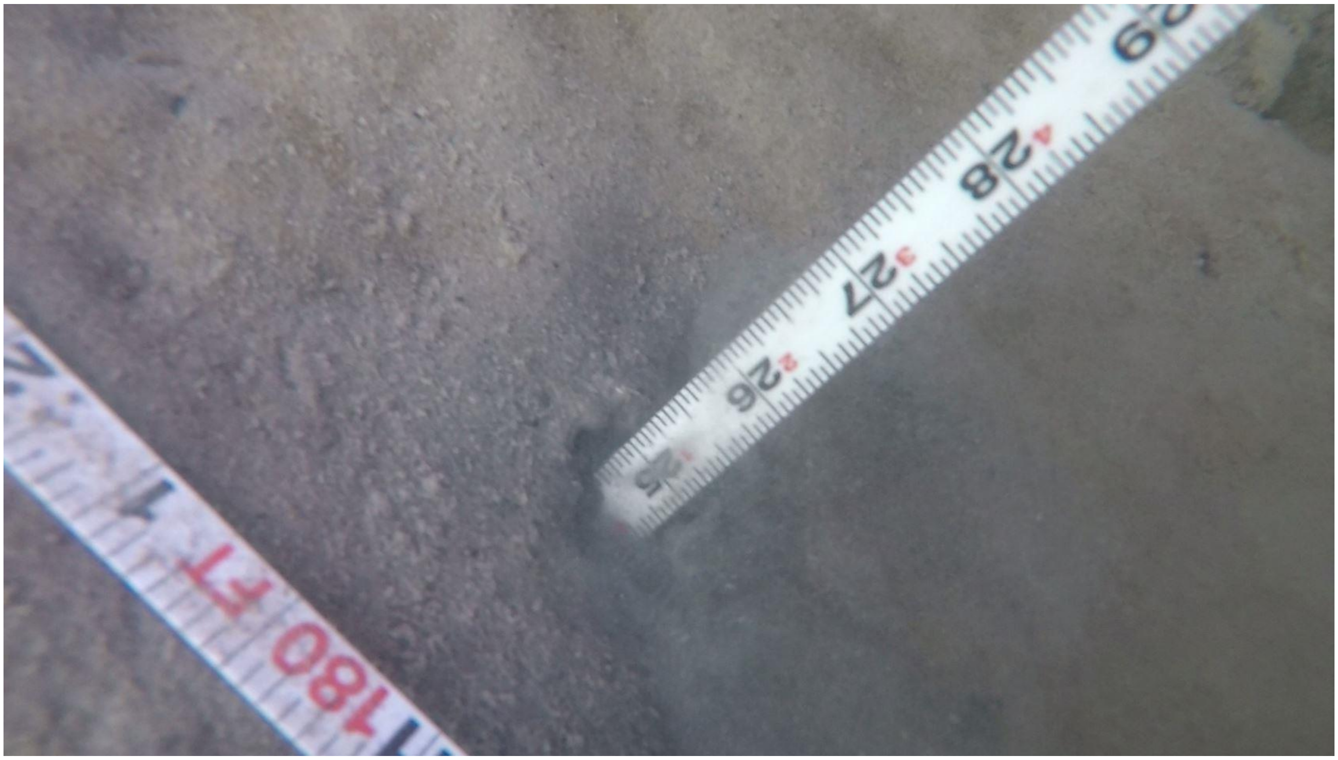
Underwater 17 - at 140ft above waterline looking east shows previous repairs and concrete piles supporting structures above wall

Underwater 18 - at 140ft above waterline looking west shows wall condition and utilities



Underwater 19 – at 146ft shows 11in of sediment


Underwater 20 - at 151ft above waterline shows large vertical crack

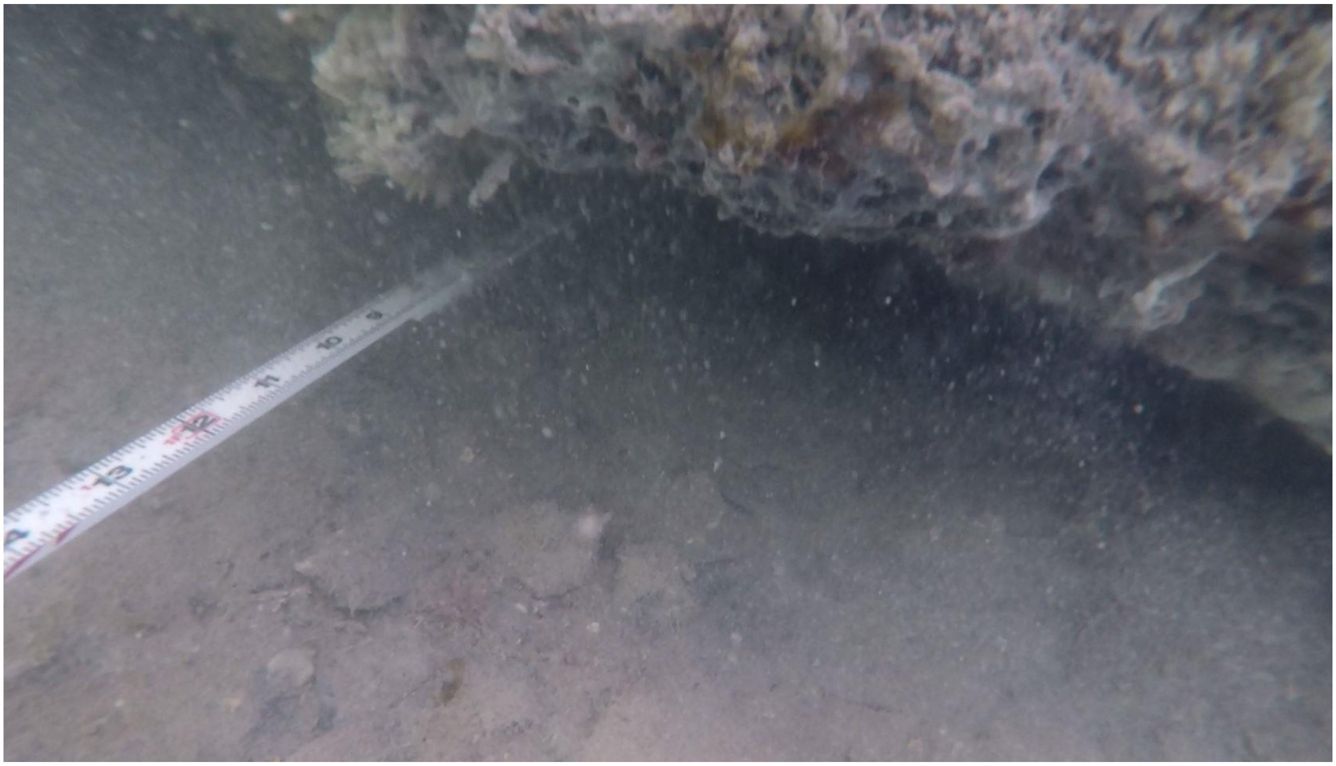


Underwater 21 - at 180ft the soft sediment is 25in deep

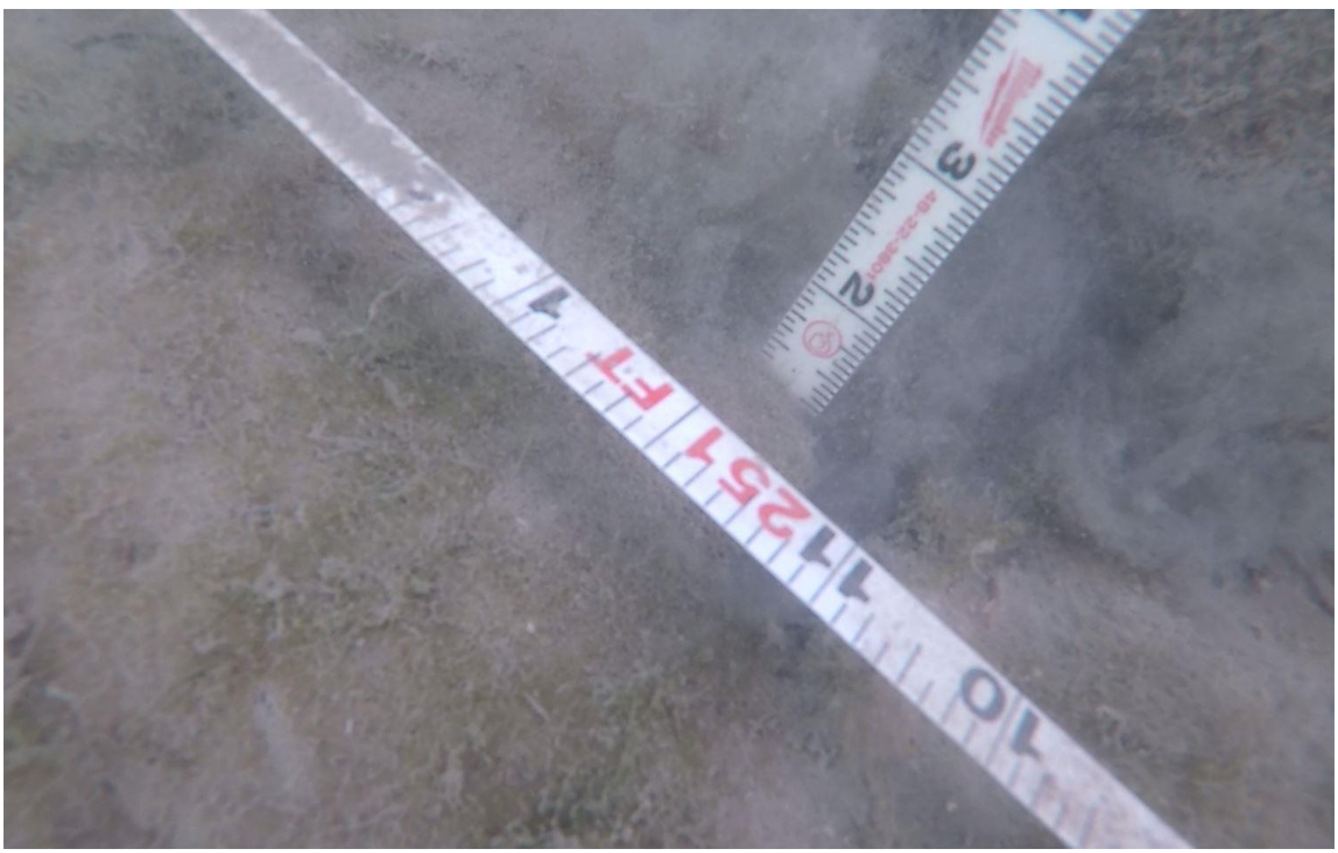
Underwater 22 - at 204ft above waterline looking east

Underwater 23 - at 204ft above waterline looking west

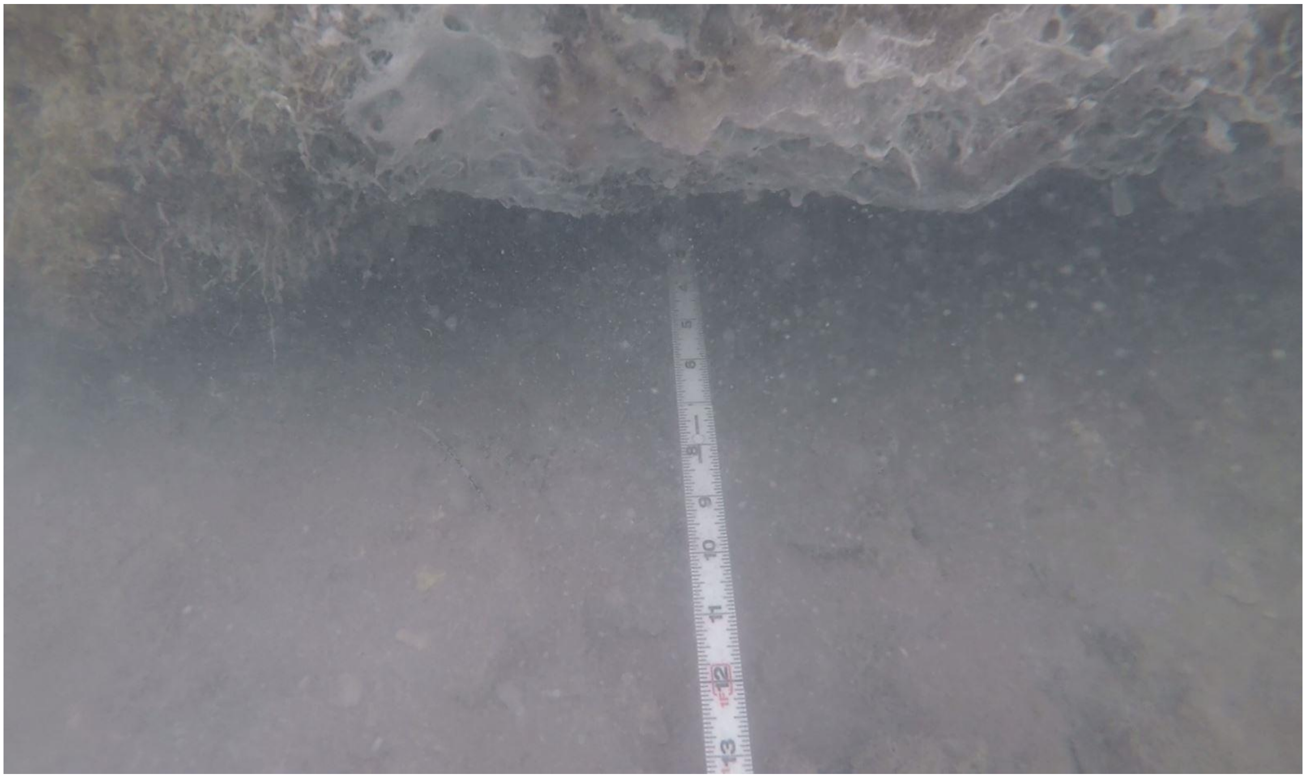
Underwater 24 - at 204ft shows old construction joint and spalling above the waterline



Underwater 25 - at 233ft shows 10ft long 8in deep void at toe of wall

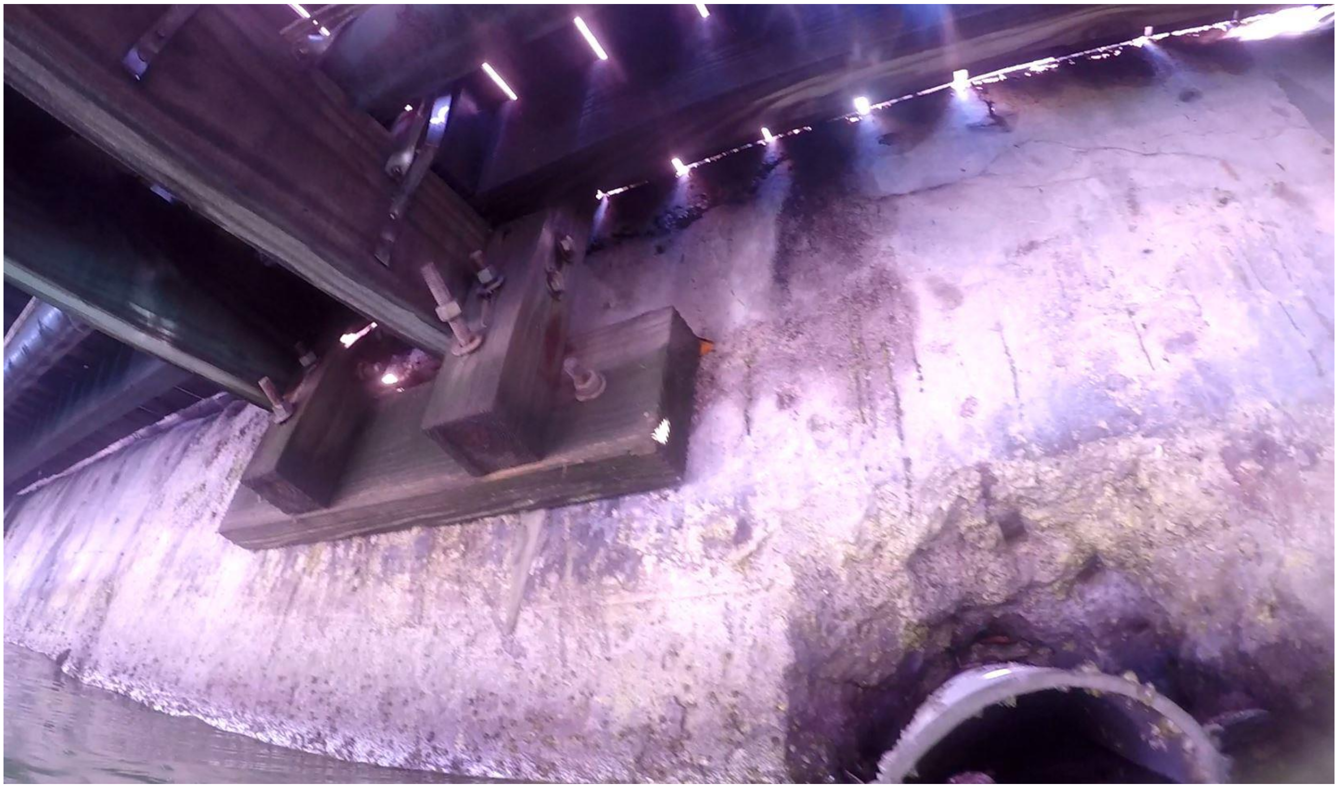


Underwater 26 - at 251ft shows 1in of sediment at toe



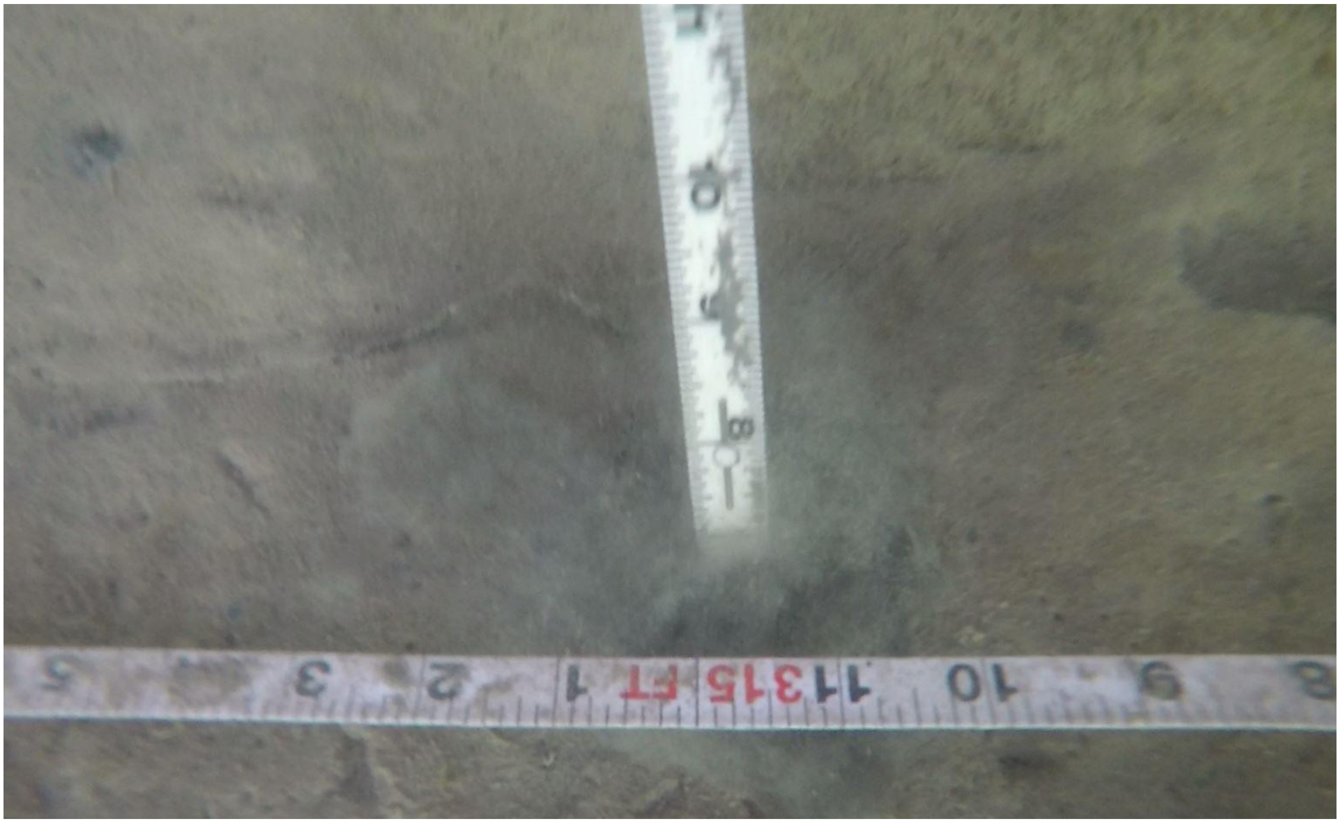
Underwater 27 - at 269ft shows 8in void at toe of wall

Underwater 28 - at 290ft above water line shows 12in diameter HDPE pile full of barnacles



Underwater 29 - at 290ft above water looking east

Underwater 30 - at 295ft shows sediment 19in deep



Underwater 31 - at 315ft shows 7in sediment at toe

Underwater 32 - at 318ft above water shows large crack from top off wall all the way to the bottom

Underwater 33 - at 339ft above water looking at plug

Underwater 34 - at 339ft above water looking east above plug

Underwater 35 - at 339ft shows a plug of some kind, with water leaking on both sides at top

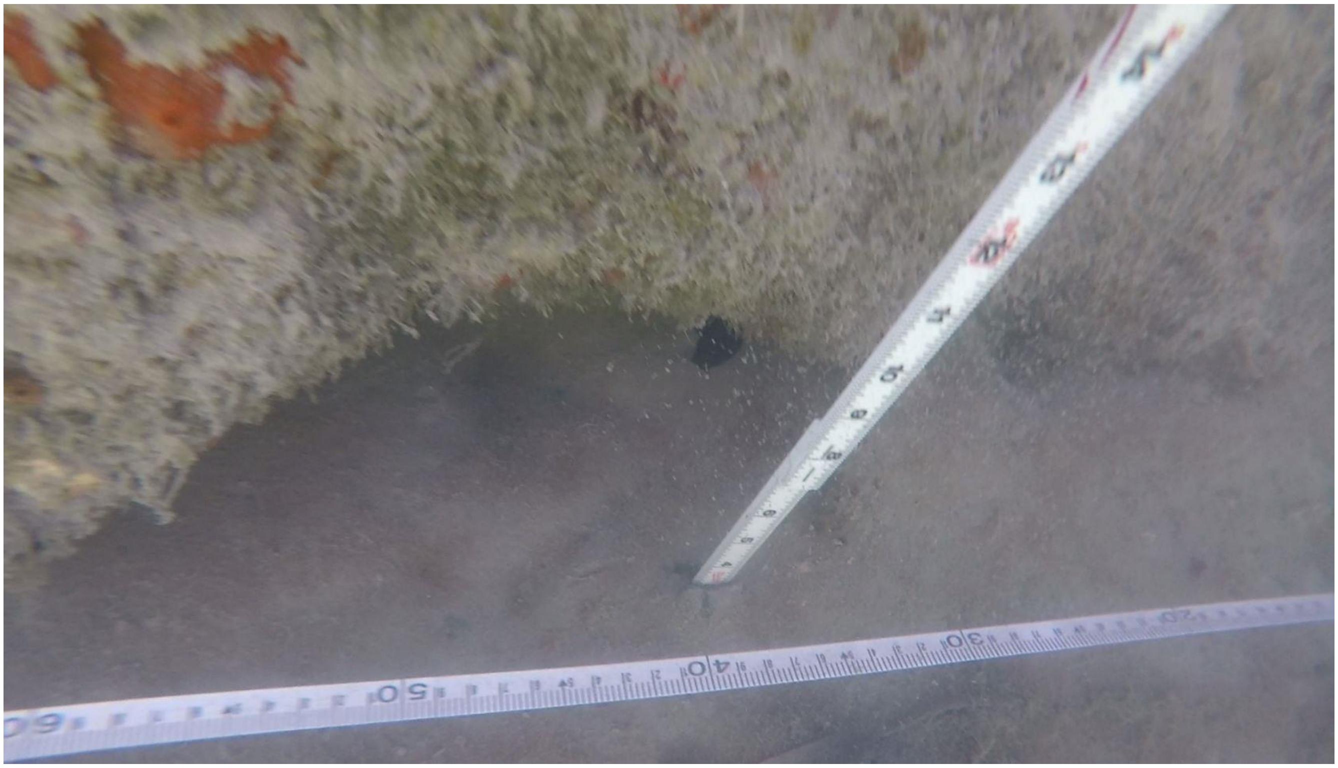

Underwater 36 - at 350ft above waterline looking east at old turtle pen fence

Underwater 37 - at 350ft showing 4in sediment

Underwater 38 - at 375ft inside turtle pen above waterline looking at wall showing utilities

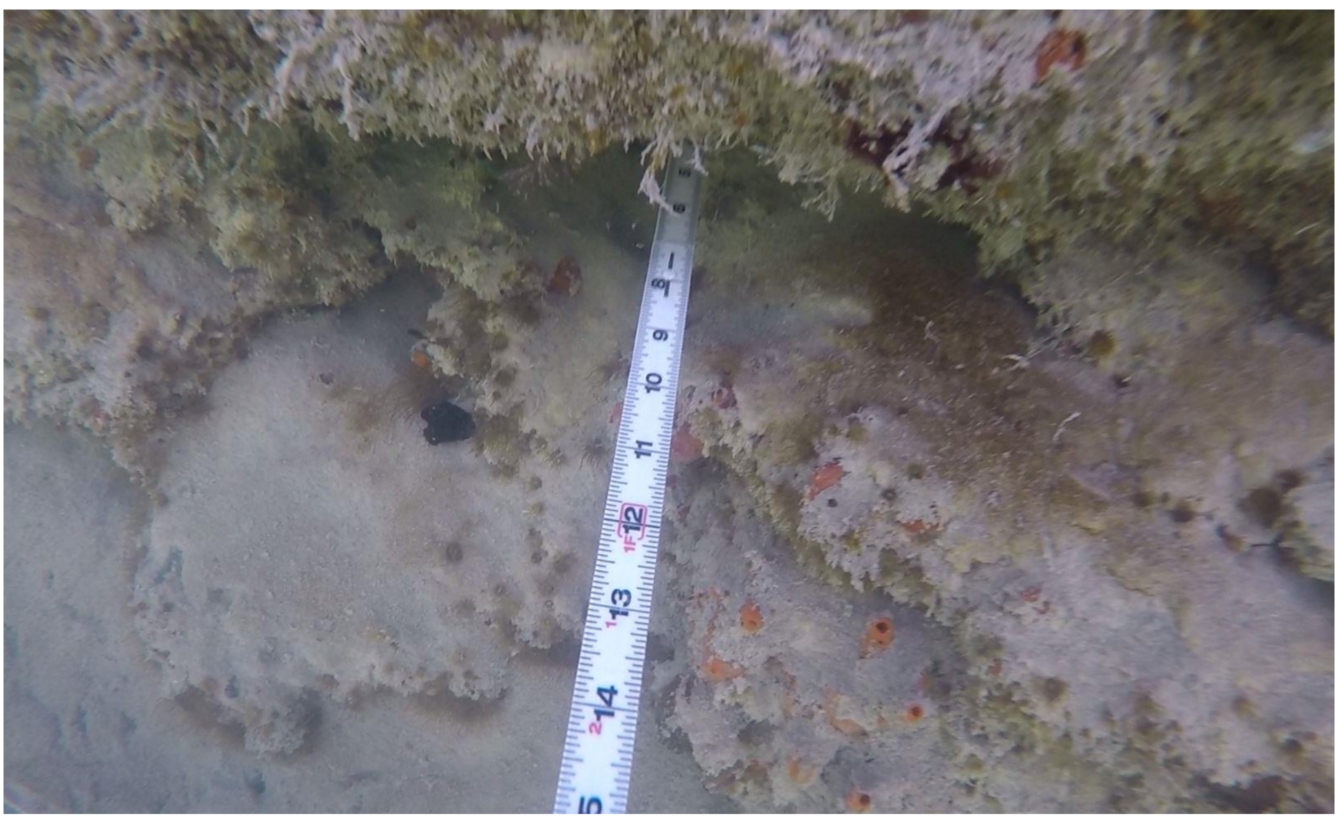
TURTLE KRAALS
Photo Log

Underwater 39 - at 380ft above waterline looking east

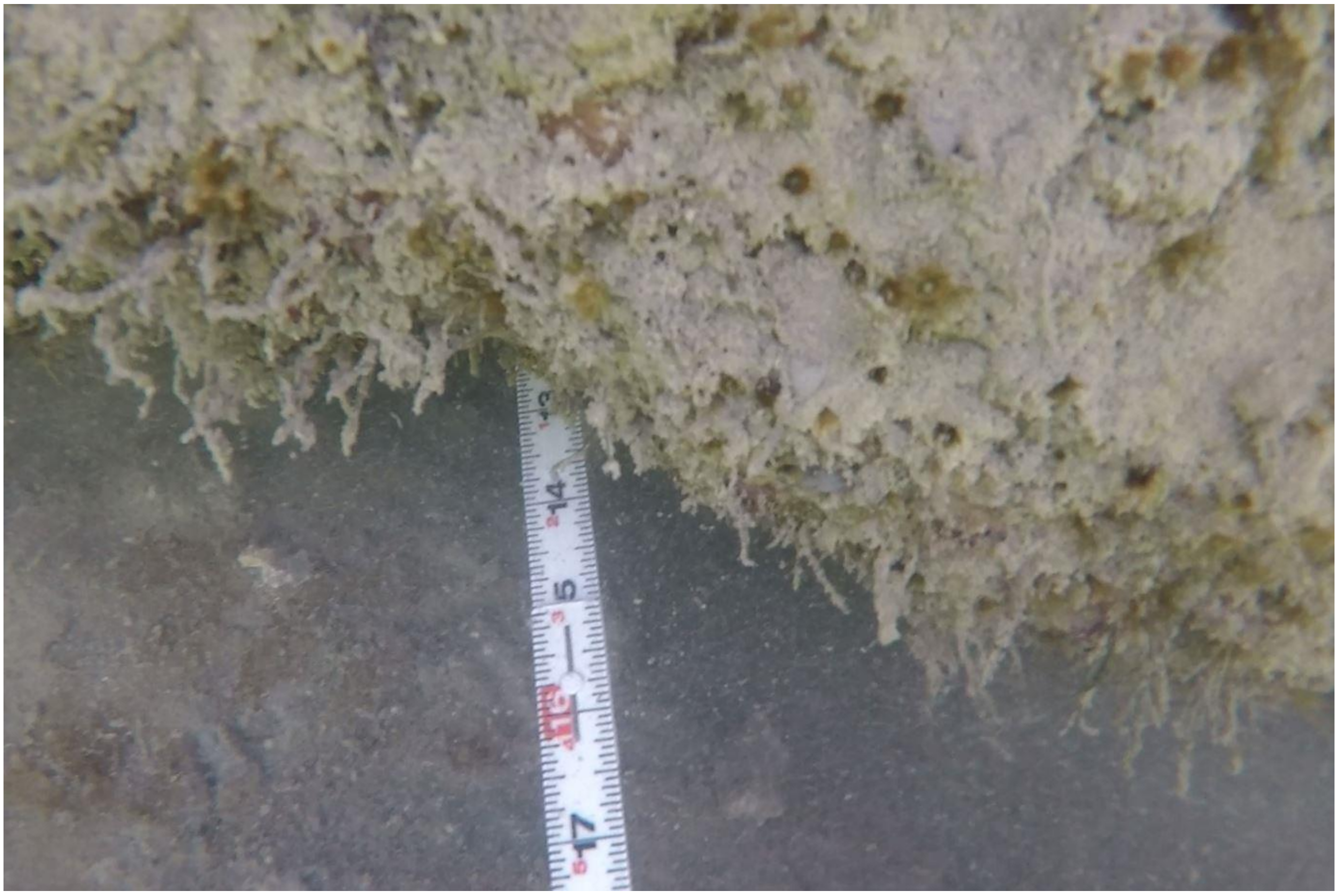


Underwater 40 - at 387ft showing 3in sediment in front of an 8in void

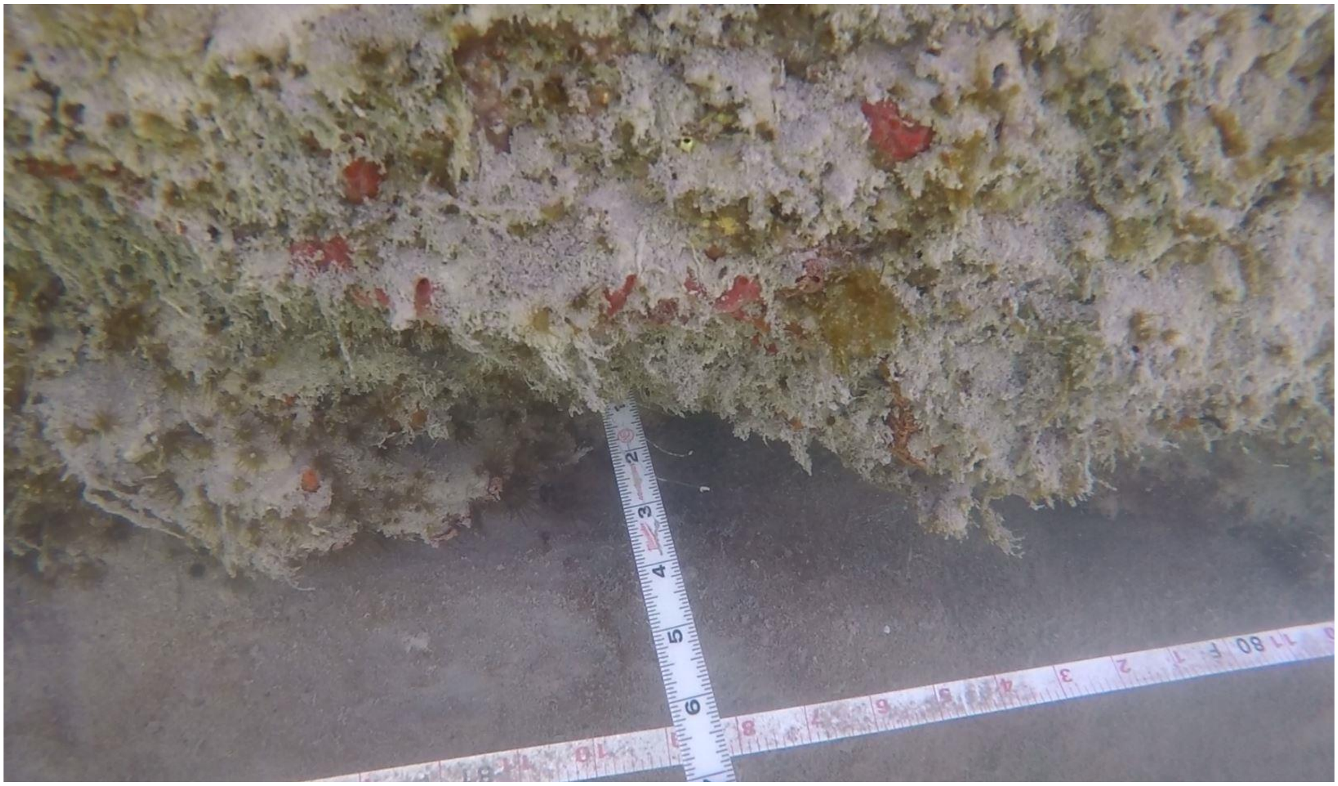
Underwater 41 - at 390ft above waterline showing 8in diameter drain and utilities



Underwater 42 - at 400ft shows 8in void with 4in sediment



Underwater 43 - at 405ft shows 16in void



Underwater 44 - at 411ft shows 3in void

Appendix 2

Project Site Survey (Prepared by Avirom & Associates)

LOCATION SKETCH
(NOT TO SCALE)

SURVEYOR'S REPORT:

1. Reproductions of this Sketch are not valid without the signature and the original raised seal of a Florida licensed surveyor and mapper.
2. No Title Opinion or Abstract to the subject property has been provided. It is possible that there are Deeds, Easements, or other instruments (recorded or unrecorded) which may affect the subject property. No search of the Public Records has been made by the Surveyor.
3. No underground improvements were located.
4. The electrical conduit banks and other utilities shown have been located from field survey information. The surveyor makes no guaranties that the electrical and other utilities shown comprise all such utilities in the area, either in service or abandoned. The surveyor further does not warrant that the electrical and other utilities shown are in the exact location indicated although he does certify that they are located accurately as possible from information available. Every effort has been made to obtain utilities pipe type and size, as shown. It is the responsibility of the end user to verify the invert and pipe size dimensions prior to design of new facilities.
5. The Grid coordinates shown hereon are based on the North American Datum of 1927, of the Florida State Plane Coordinate System (Transverse Mercator Projection), East Zone, established by a Real–Time Kinematic Survey (RTK) GPS Control Survey which is certified to a 2 centimeter local accuracy, relative to the nearest control point within the National Geodetic Survey (NGS) Geodetic Control Network.
   a. Method: Wide Area Continuously Operation GPS Reference Station Network.
   b. Equipment Used: Trimble R10 GNSS, Serial Number 5413460373 (Dual Frequency Receiver)
   c. Processing Software: Trimble Business Center, Version 3.51
6. Elevations shown hereon are in feet and based on the National Geodetic Vertical Datum of 1929 (NGVD 1929). To convert NGVD 1929 elevations to North American Vertical Datum of 1988 (NAVD 1988) for this site, the model value of (–)1.34 must be added algebraically to the NGVD 1929 height.
7. Benchmark Description: National Geodetic Survey Station Designation "872 4580 TIDAL 25" (PID AA0004). Elevation=5.11 (NGVD 1929). Elevation=3.77 (NAVD 1988).
8. Data shown hereon was compiled from instrument(s) of record and does not constitute a Boundary Survey.
9. a. This is NOT a Mean High Water Survey in accord with Chapter 177, Part II Florida Statutes.
   b. Mean High Water Elevation is 1.09 feet, NGVD 1929 (–0.25 feet, NAVD 1988), as located on April 12, 2016, and is shown for informational purposes only.
   c. The Mean High Water Elevation as shown hereon was established by extending the tidal datum shown at Mean High Water Interpolation Point No. 3260, which was obtained from the Florida Department of Environmental Protection Web Site, http://www.labins.org.
10. This map is intended to be displayed at a scale of 1:240 (1"=20').
11. Units of measurement are in U.S. Survey Feet and decimal parts thereof. Well identified features in this survey were field measured to a horizontal positional accuracy of 0.10'. The elevations on impervious surfaces were field measured to 0.05' and on ground surfaces to 0.1'.
12. Symbols shown hereon and in the legend may have been enlarged for clarity. These symbols have been plotted at the center of the field location and may not represent the actual shape or size of the feature.
13. Site Address: 231 Margaret Street, Key West, FL 33040
14. Abbreviation Legend: B.M. = Benchmark; E = Easting; EL. = Elevation; F.B. = Field Book; F.F. = Finished Floor; GPS = Global Positioning System; ID. = Identification; L.B. = Licensed Business; N = Northing; NAD = North American Datum; NAVD = North American Vertical Datum; NGVD = National Geodetic Vertical Datum; PID = Point Identifier; P.L.S. = Professional Land Surveyor; PVC = Polyvinyl–Chloride Pipe; RTK = Real–Time Kinematic; T.O.P. = Top of Pipe.

CERTIFICATION:

I HEREBY CERTIFY that the attached Topographic Survey of the hereon described property is true and correct to the best of my knowledge and belief as surveyed in the field under my direction. I FURTHER CERTIFY that this Topographic Survey meets the Standards of Practice set forth in Chapter 5J–17, Florida Administrative Code, pursuant to Section 472.027, Florida Statutes.

Date: ____________________

____
KEITH M. CHEE–A–TOW, P.L.S.
Florida Registration No. 5328
AVIROM & ASSOCIATES, INC.
L.B. No. 3300

AVIROM & ASSOCIATES, INC. ESTABLISHED 1981

AVIROM & ASSOCIATES, INC.
SURVEYING & MAPPING
50 S.W. 2nd AVENUE, SUITE 102, BOCA RATON, FLORIDA 33432
TEL. (561) 392-2594, FAX (561) 394-7125
www.AVIROMSURVEY.com

REVISIONS	DATE	F.B. / PG.	BY	CK'D	REVISIONS	DATE	F.B. / PG.	BY	CK'D

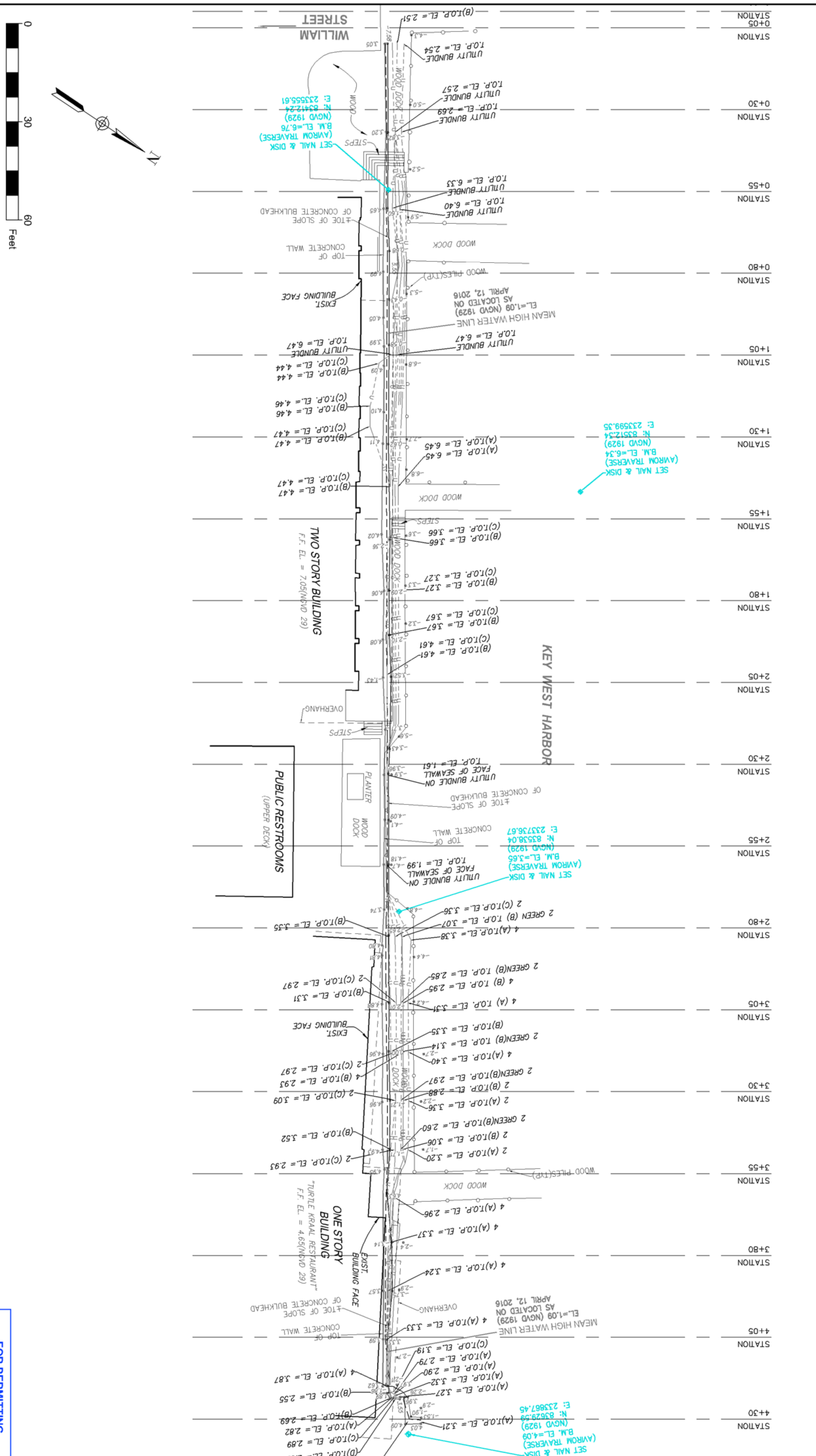
TOPOGRAPHIC SURVEY
TURTLE KRAALS
PORTION OF
WILLIAM A. WHITEHEAD'S MAP OF KEY WEST (UNRECORDED)
SECTION 31, TOWNSHIP 68 SOUTH, RANGE 25 EAST
CITY OF KEY WEST
MONROE COUNTY, FLORIDA

JOB #:	10157-1
SCALE:	1" = 20'
DATE:	01/20/2017
BY:	KSB
CHECKED:	K.M.C.
F.B. 1797	PG. 7
SHEET:	1 OF 1

Appendix 3

Seawall Plan and Cross Section Views
With Seawall Elevation (Front) Views with Survey

NOTE: SURVEY PERFORMED BY AVIROM & ASSOCIATES, INC. 1/20/17.



FOR PERMITTING
NOT FOR CONSTRUCTION

CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL PLAN VIEW

KEY WEST, MONROE COUNTY, FLORIDA

TETRA TECH INC.
759 SOUTH FEDERAL HWY
SUITE 314
STUART, FL 34994-2936
TEL: (772) 781-3400
FAX: (772) 781-3411
CERTIFICATE OF AUTHORIZATION
NO. 2429

Designed by: S. MCGAHEE
Drawn By: F. MARTINEZ
Checked By: --
Reviewed By: --
Design file no: TT_TURTLEKRAALS_V4.DWG
Scale: AS SHOWN

Mark	Description	Date	Appr.
–	–	–	–

Stuart E McGahee
Florida PE No. 57536

Sheet Reference:
C-001
Sheet 1 of –

NOTE: SURVEY PERFORMED BY AVIROM & ASSOCIATES, INC. 1/20/17.

FOR PERMITTING
NOT FOR CONSTRUCTION

Mark	Description	Date	Appr.
–	–	–	–

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CERTIFICATE OF AUTHORIZATION NO. 2429

Stuart E McGahee
Florida PE No. 57536

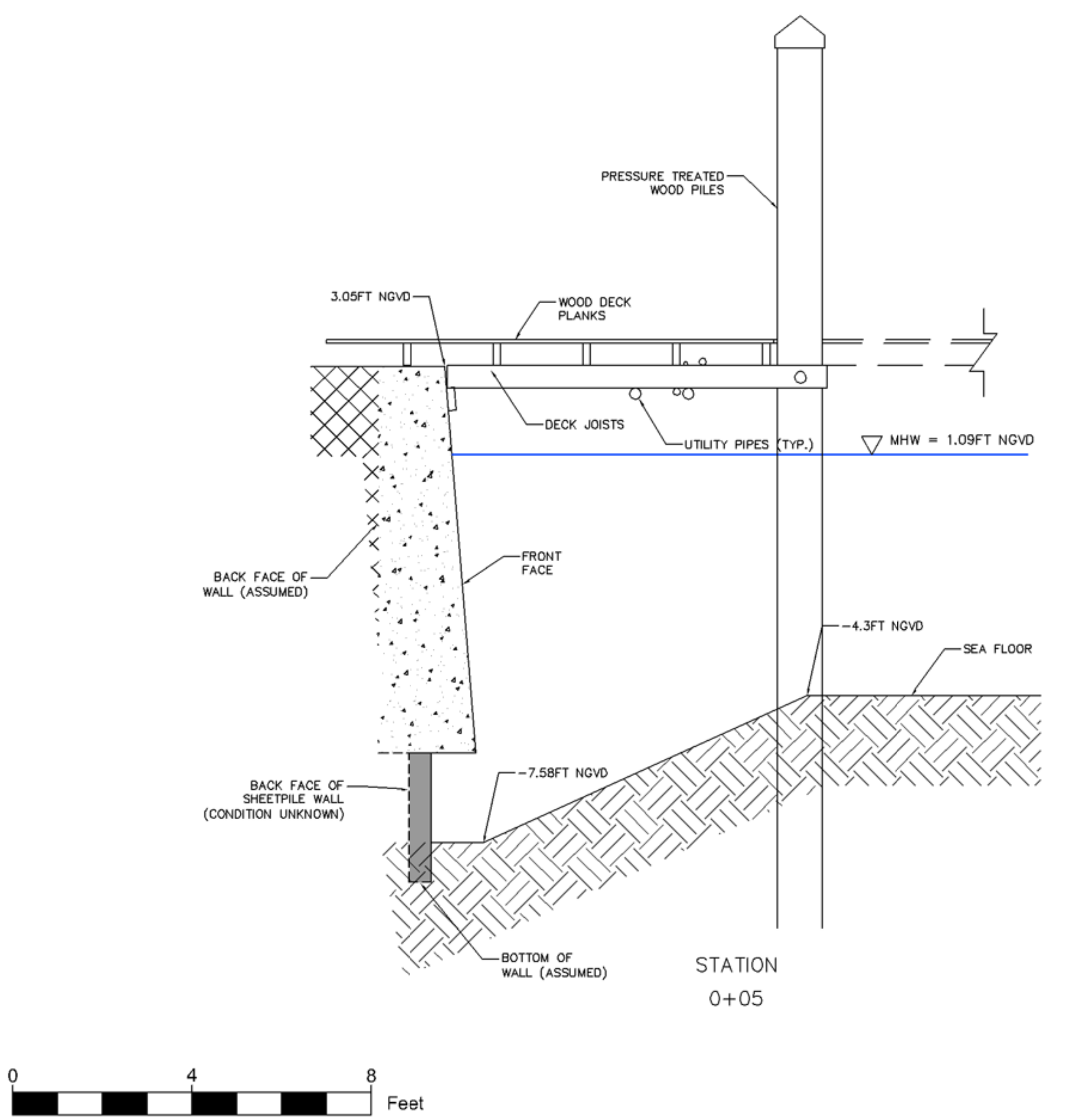
SEAL OF THE CITY OF KEY WEST, FLORIDA

CITY OF KEY WEST
TURTLE KRAAL SEAWALL

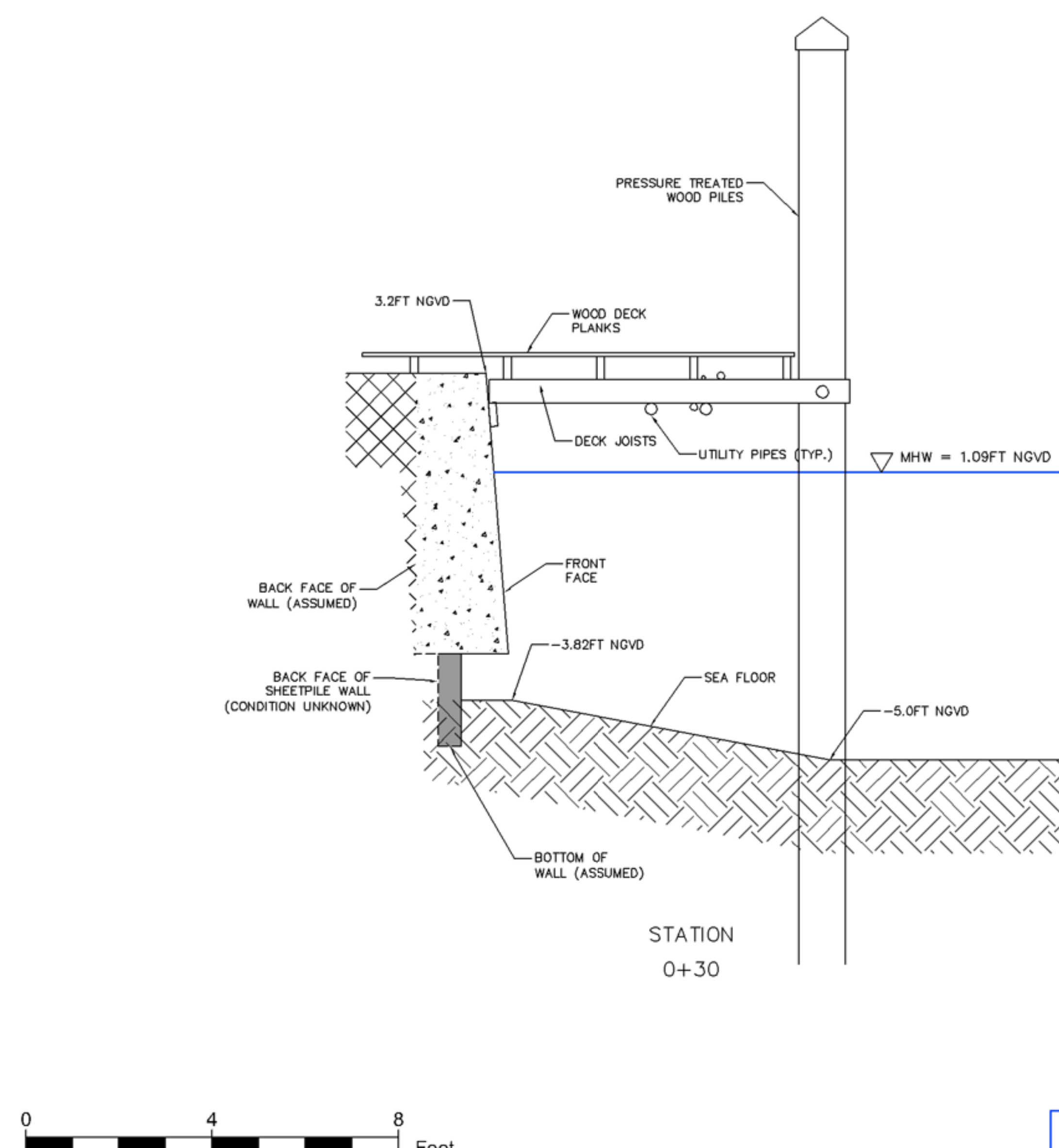
SEAWALL PLAN VIEW WITH AERIAL

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-002
Sheet 2 of –



STATION
0+05



STATION
0+30

FOR PERMITTING
NOT FOR CONSTRUCTION

CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

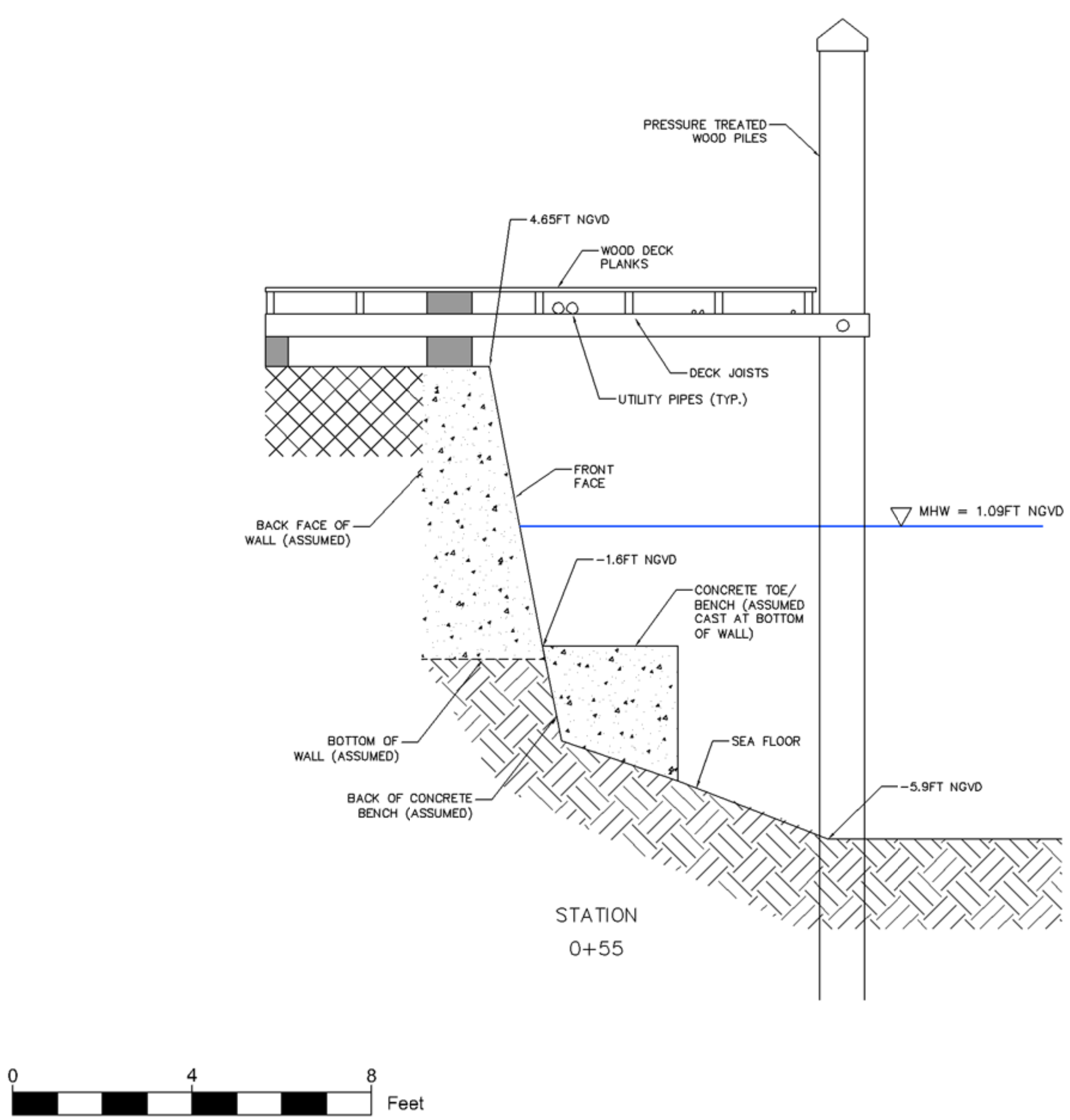
TETRA TECH INC.
759 SOUTH FEDERAL HWY
SUITE 314
STUART, FL 34994-2936
TEL: (772) 781-3400
FAX: (772) 781-3411
CERTIFICATE OF AUTHORIZATION NO. 2429

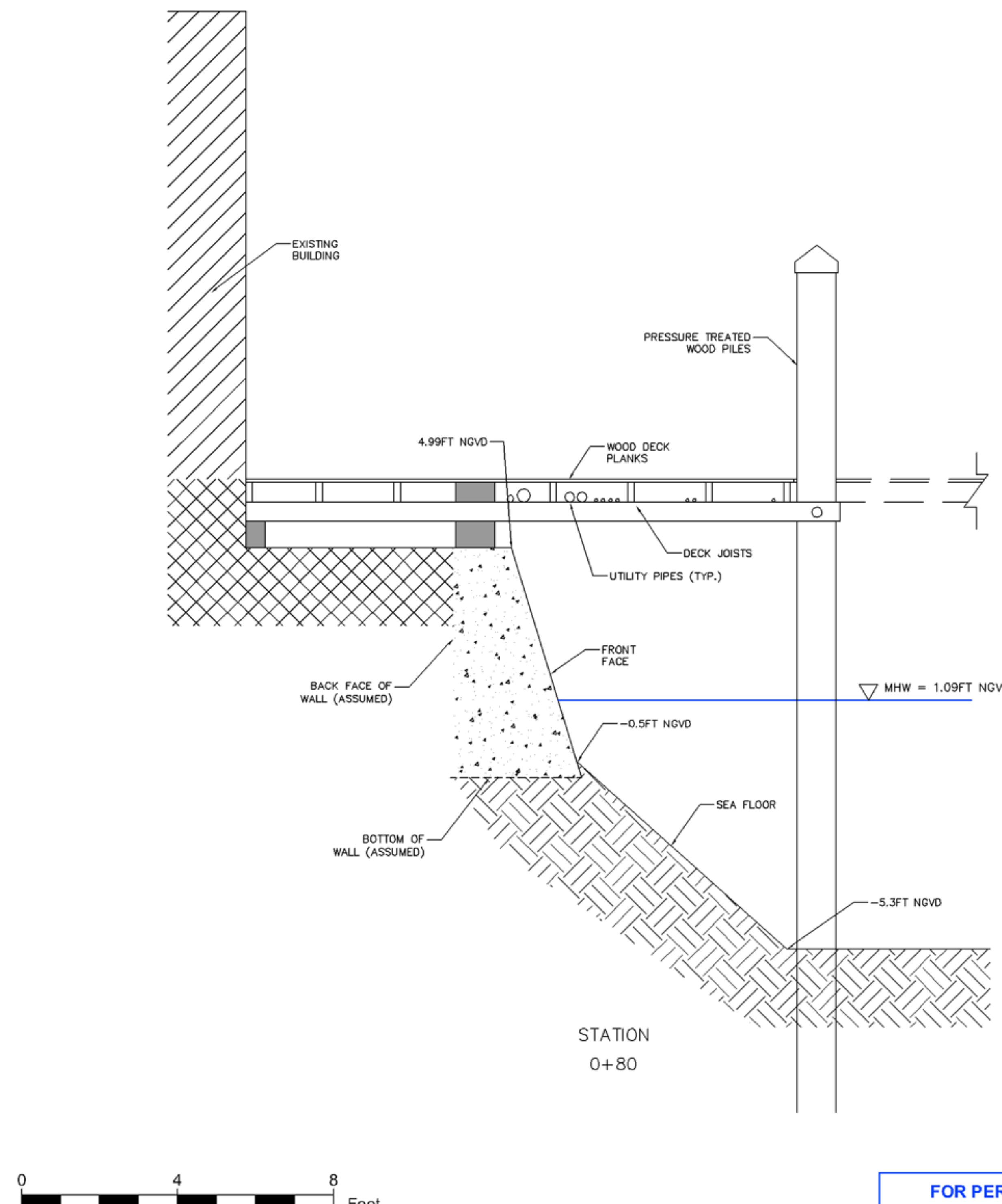
Designed by: S. MCGAHEE
Drawn By: F. MARTINEZ
Checked By: --
Reviewed By: --
Design file no: TT_TURTLEKRAALS_VS_FACE.DWG
Scale: AS SHOWN

Mark	Description	Date	Appr.
-	-	-	-

Stuart E McGahee
Florida PE No. 57536

Sheet Reference:
C-003
Sheet 3 of –





FOR PERMITTING
NOT FOR CONSTRUCTION

Stuart E McGahee
Florida PE No. 57536

Mark	Description	Date	Appr.
–	–	–	–

Designed by: S. MCGAHEE
Drawn By: F. MARTINEZ
Checked By: --
Reviewed By: --
Design file no: TT_TURTLEKRAALS_V5_FACE.DWG
Scale: AS SHOWN

TETRA TECH INC.
759 SOUTH FEDERAL HWY
SUITE 314
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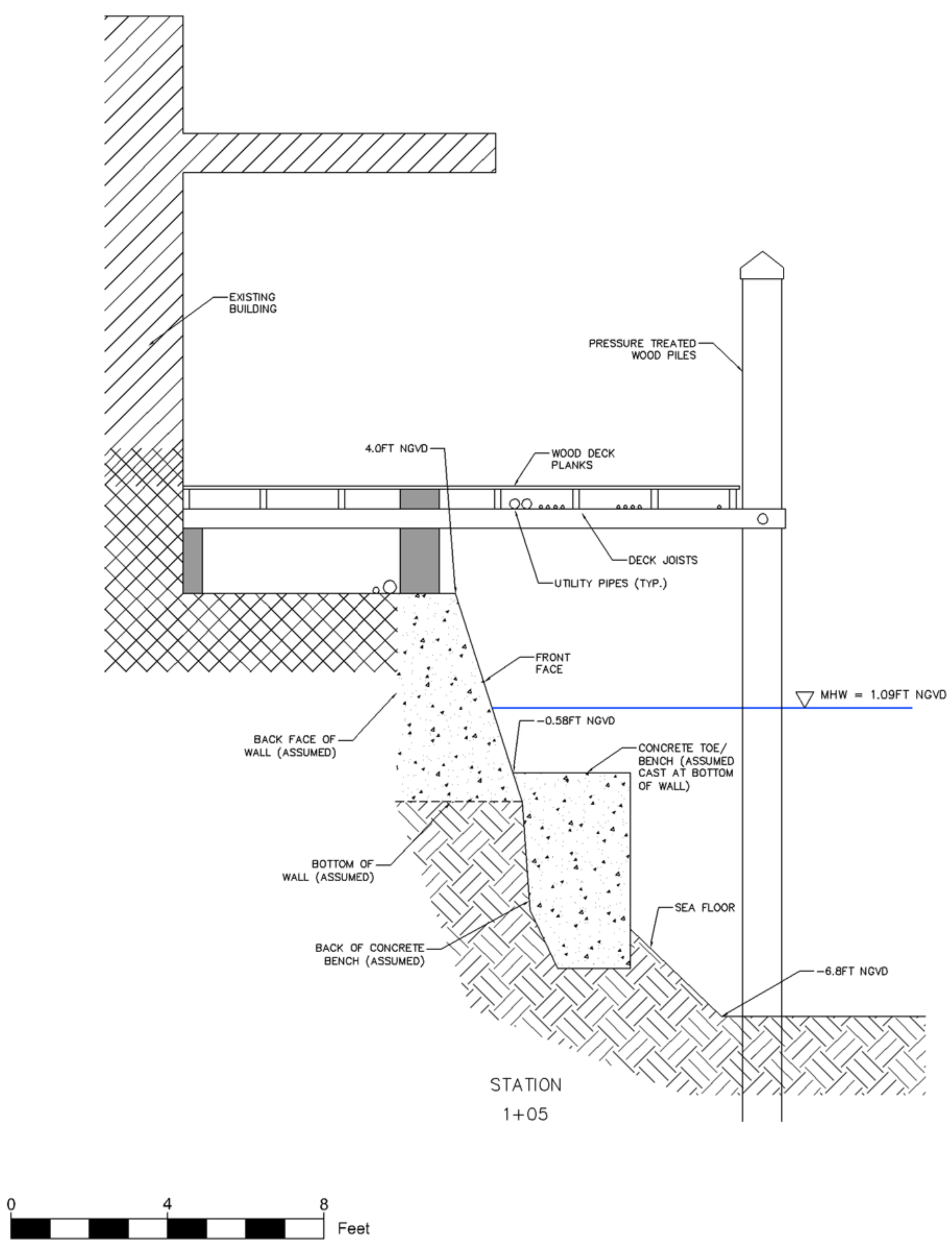
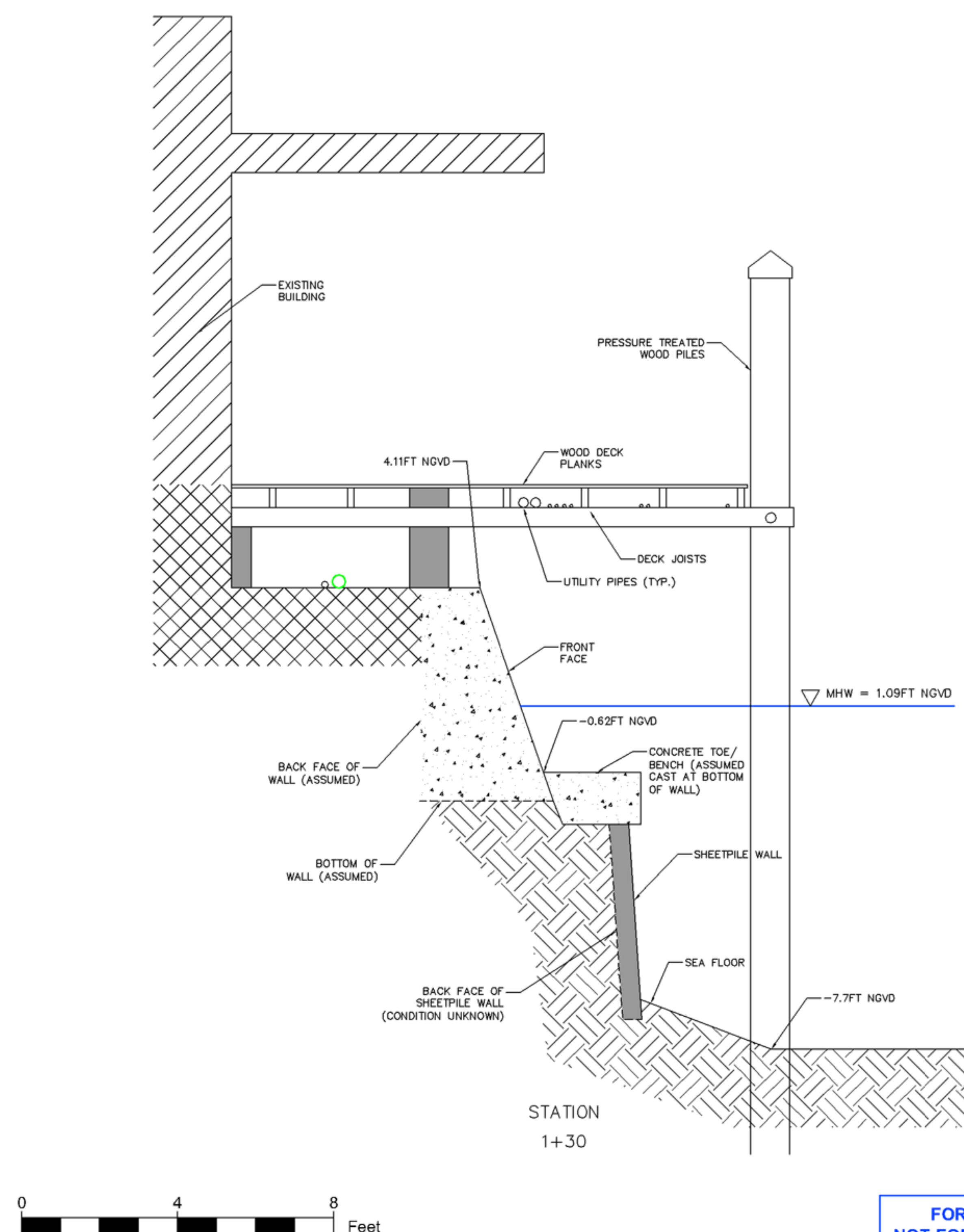
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-004
Sheet 4 of –





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CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

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Drawn By: F. MARTINEZ
Checked By: --
Reviewed By: --
Design file no: TT_TURTLEKRAALS_V5_FACE.DWG
Scale: AS SHOWN

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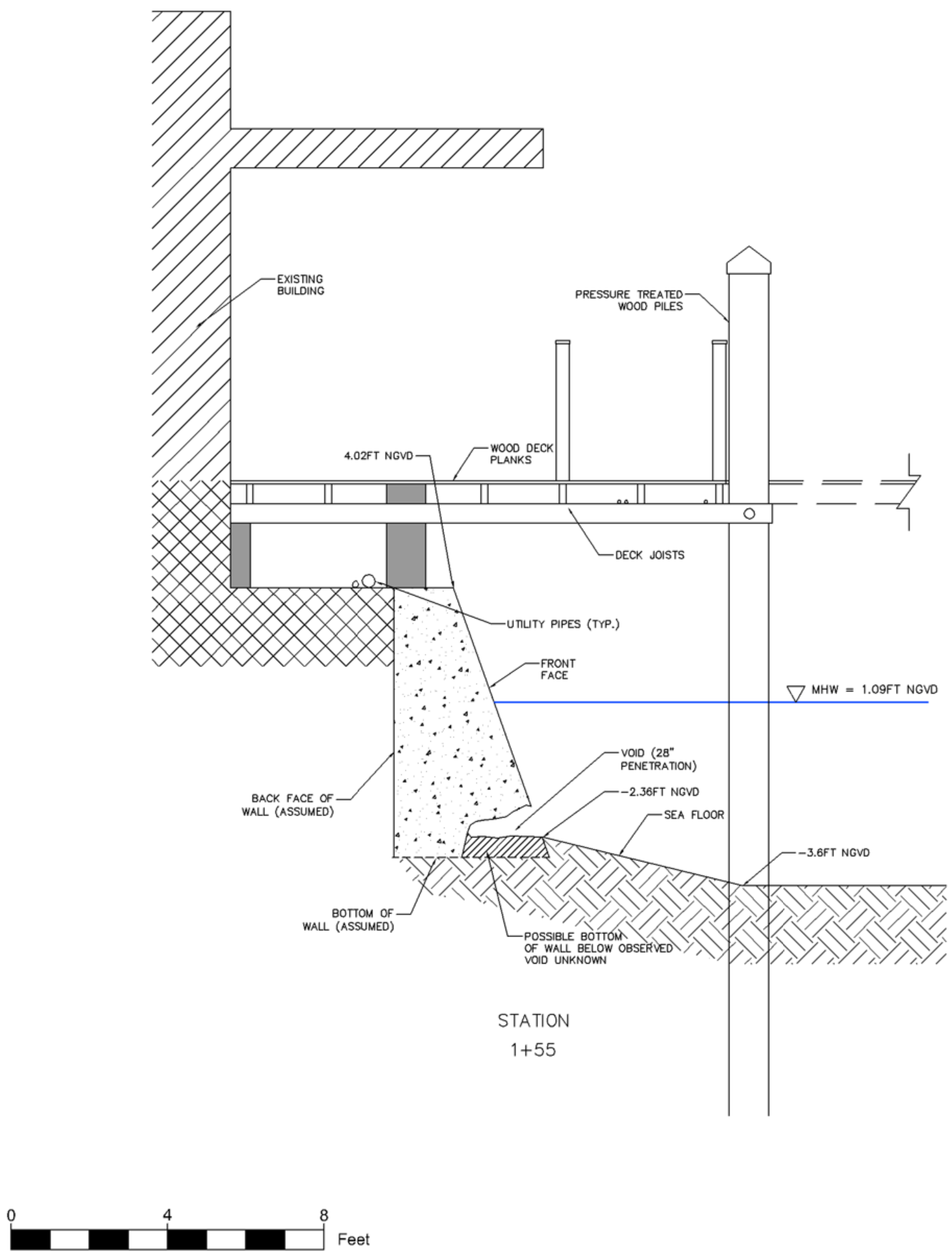
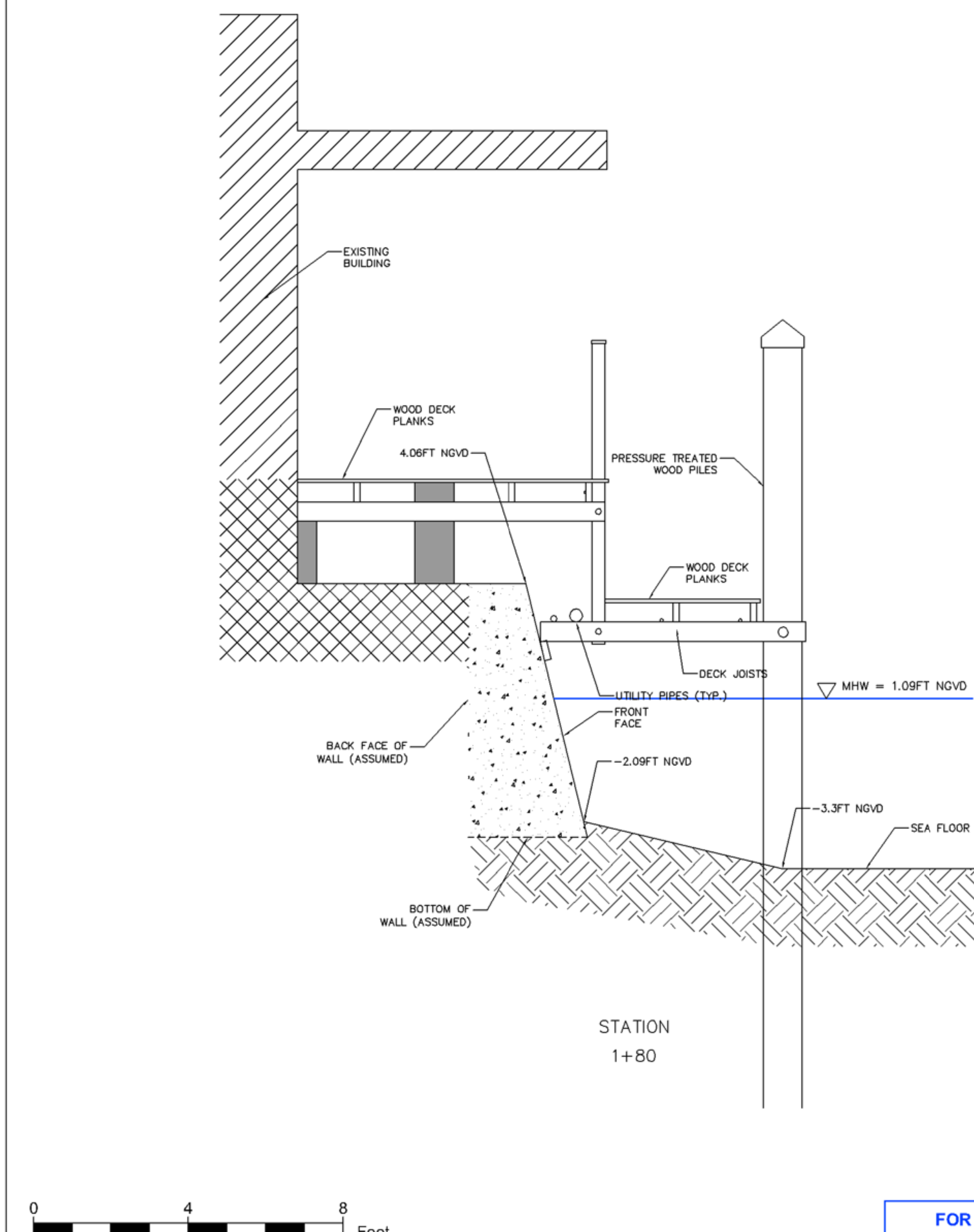
Sheet Reference:
C-005
Sheet 5 of –



STATION
1+55



STATION
1+80

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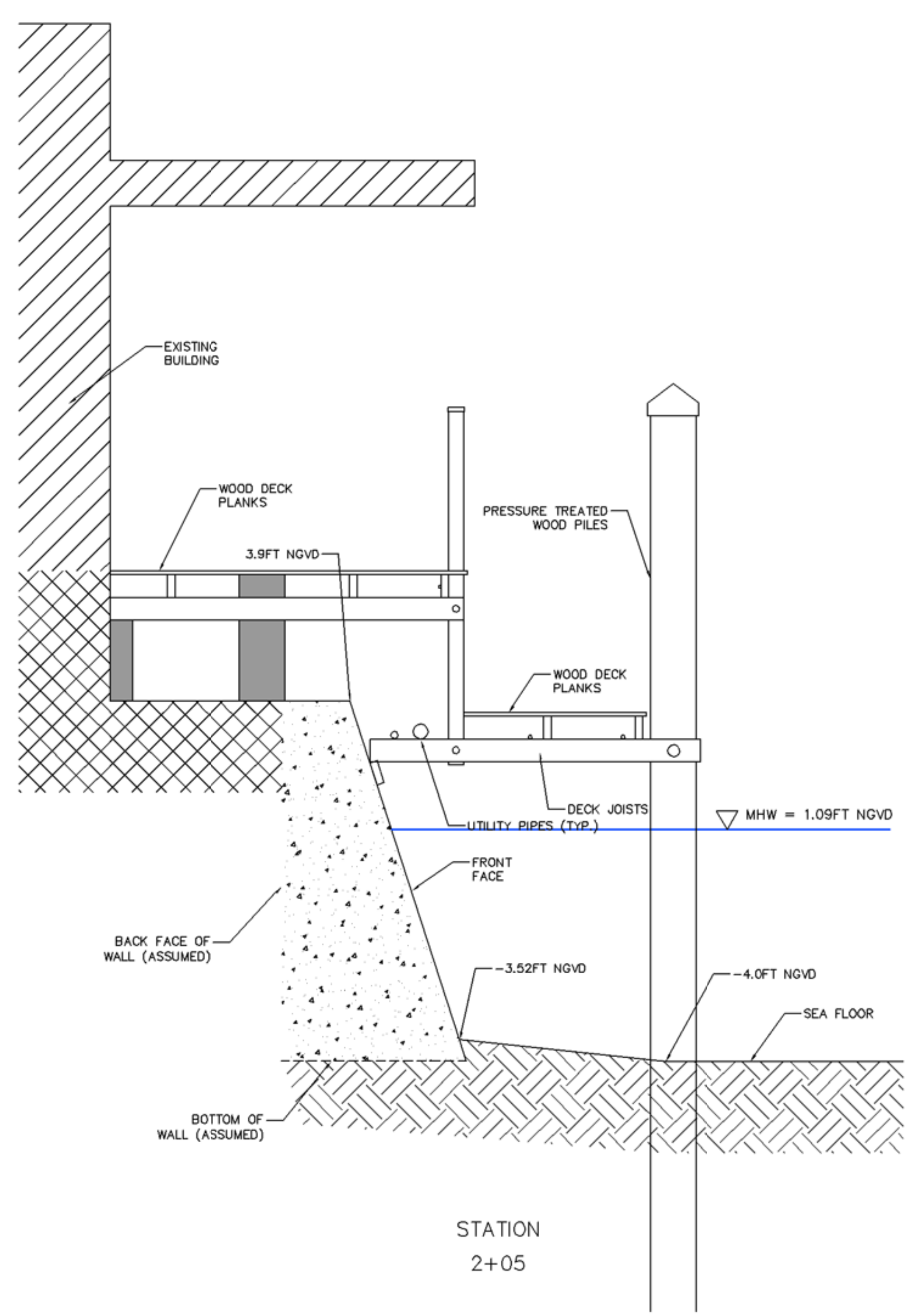
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

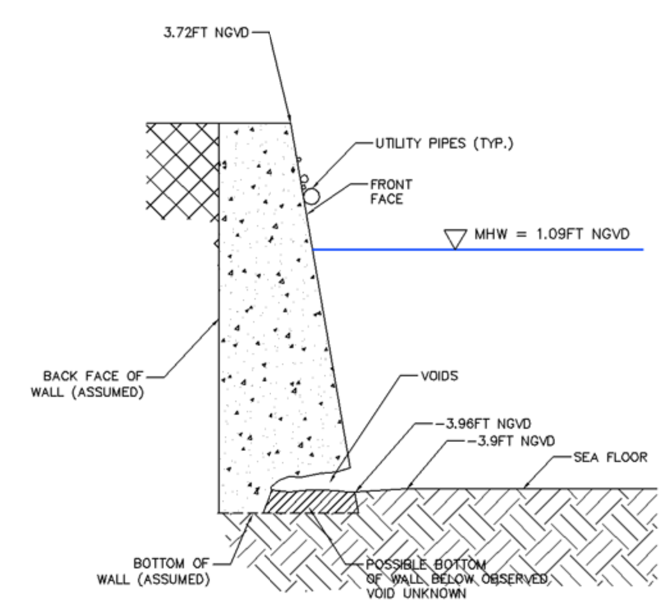
SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-006
Sheet 6 of –



STATION
2+05



STATION
2+30

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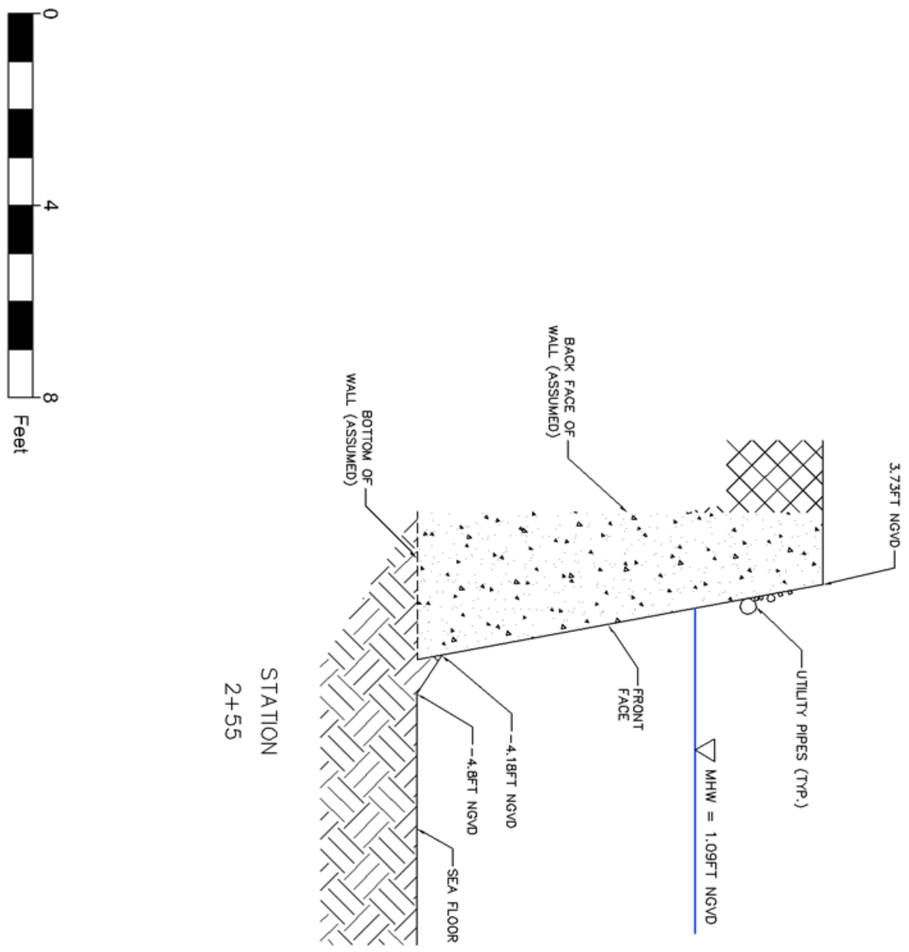
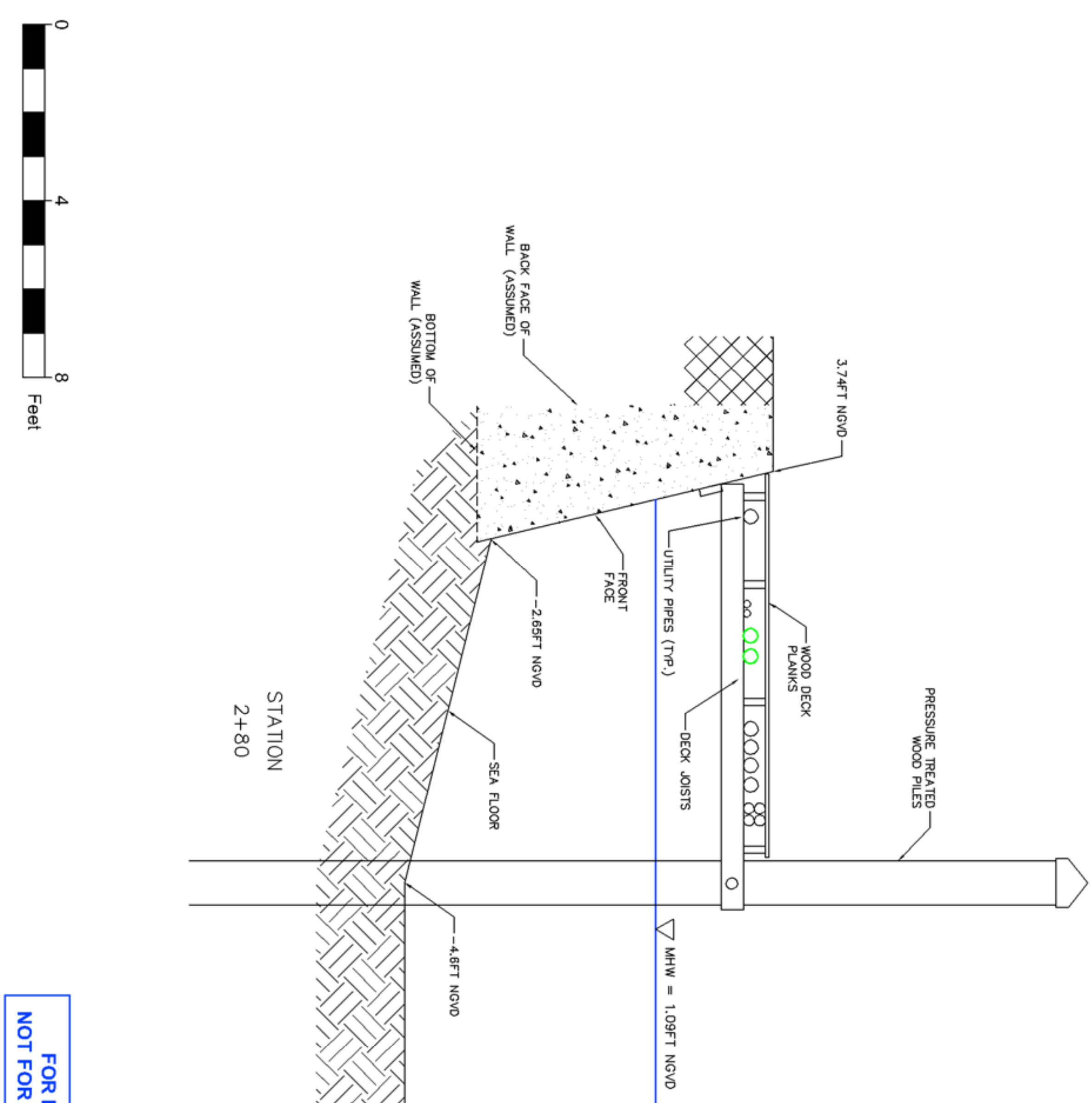
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-007
Sheet 7 of –





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Sheet 8 of –

CITY OF KEY WEST
TURTLE KRAAL SEAWALL

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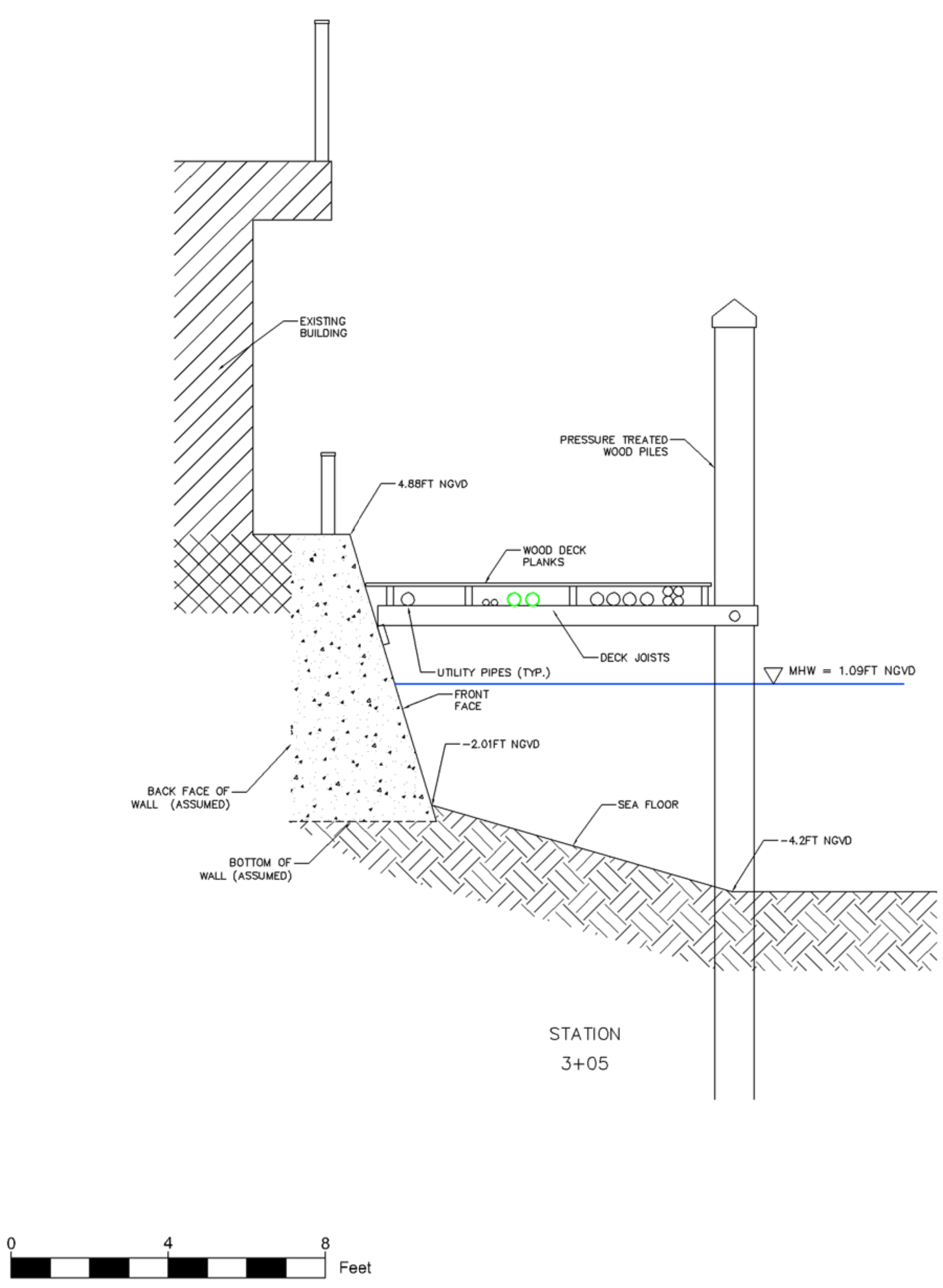

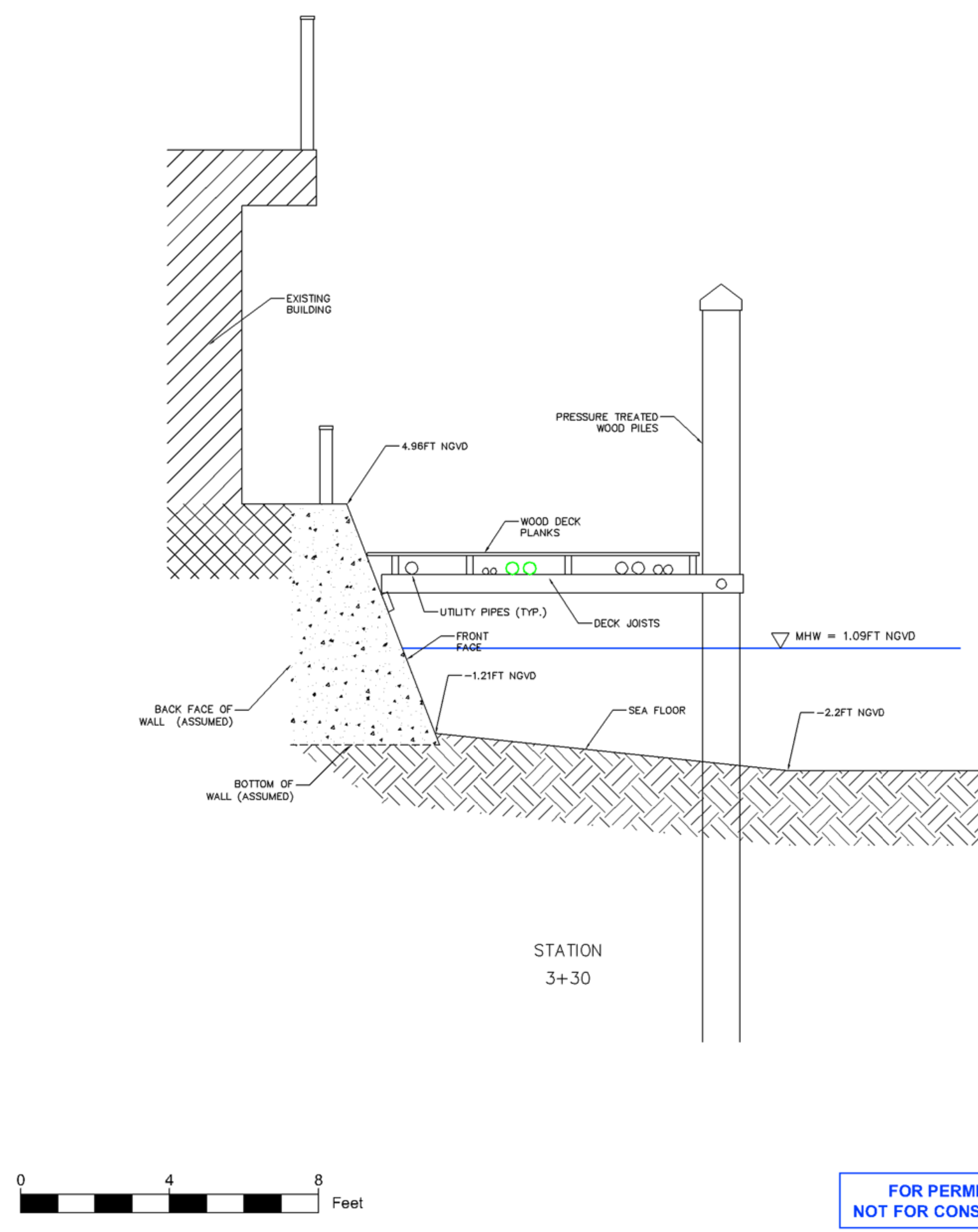

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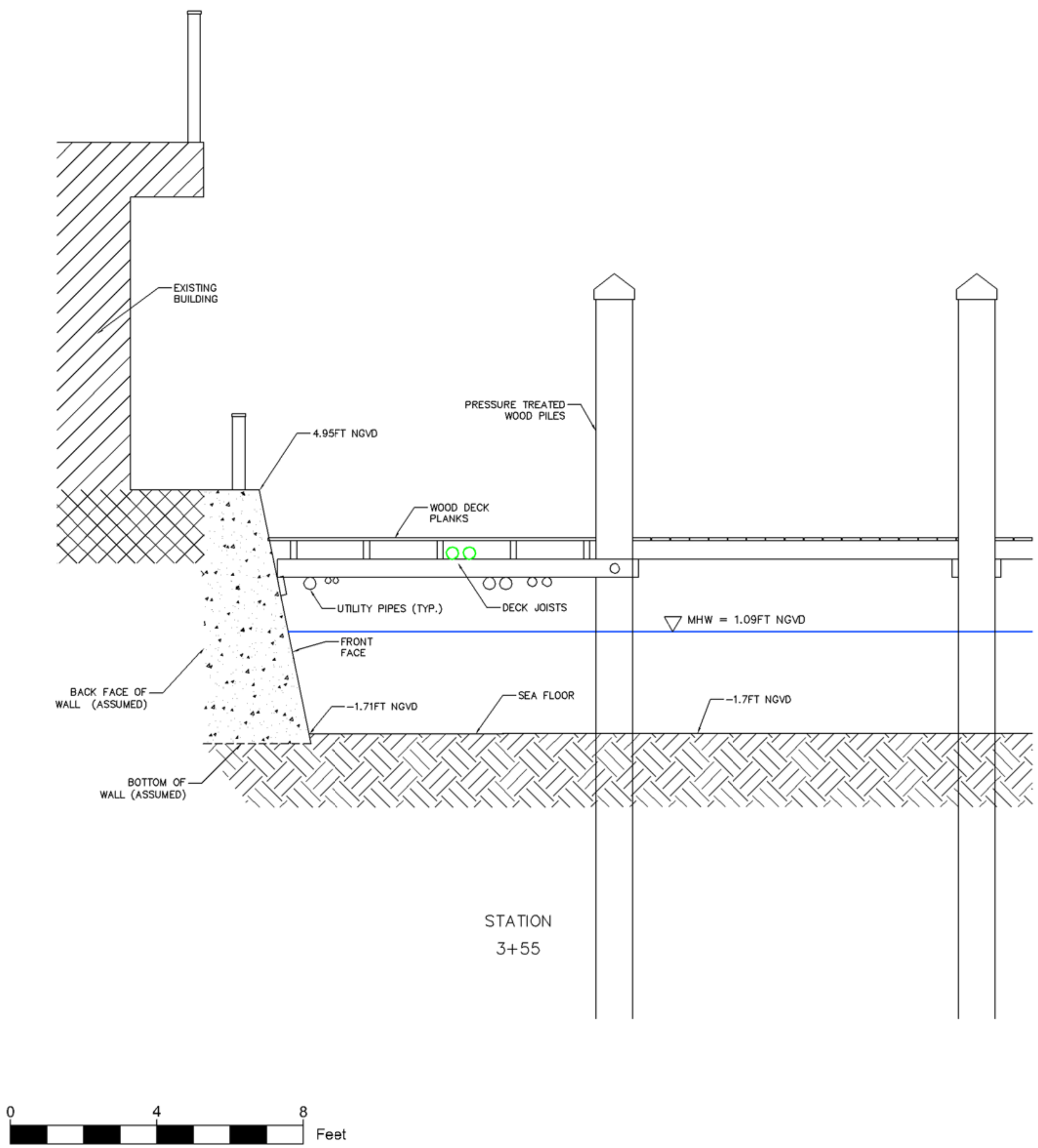
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

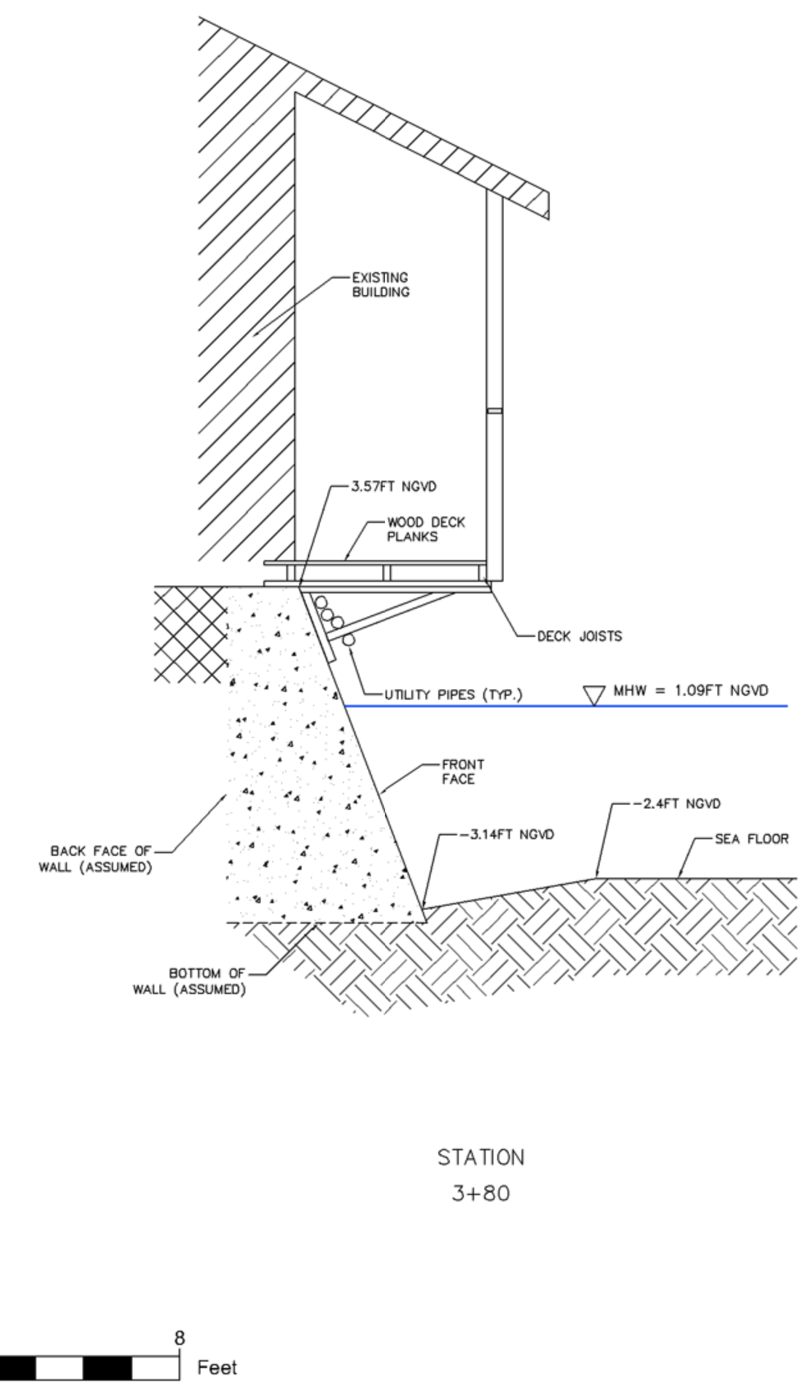
SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-009
Sheet 9 of –



STATION
3+55



STATION
3+80

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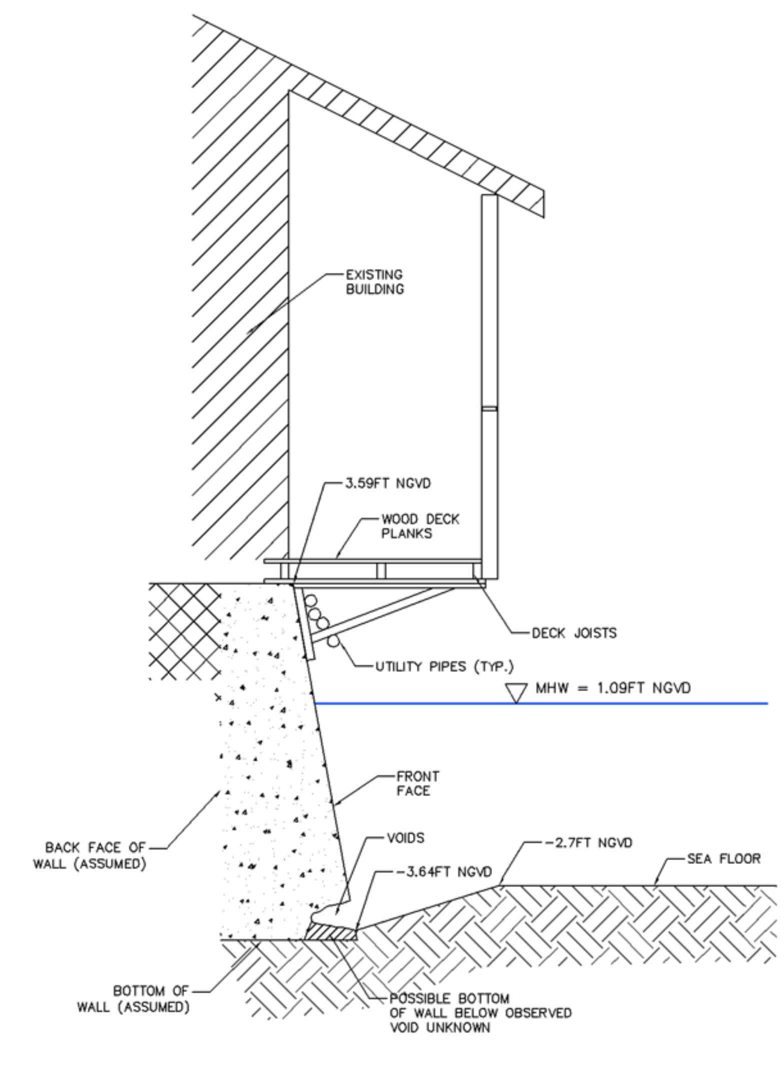
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

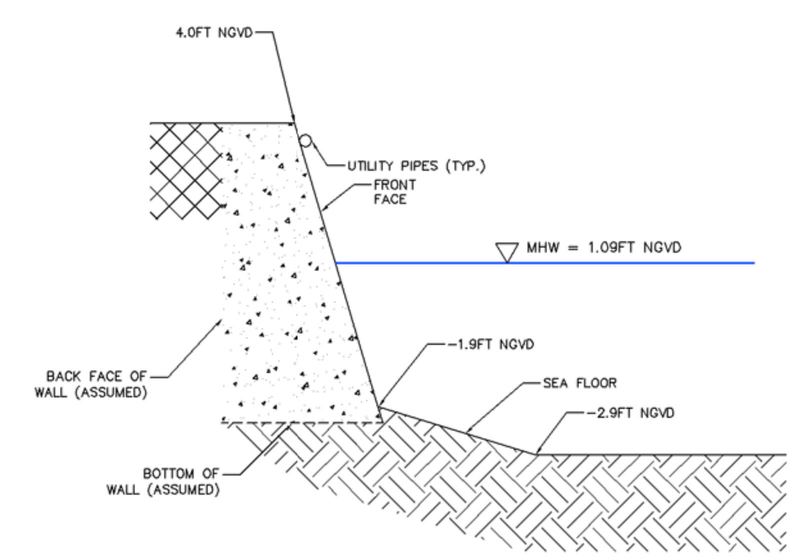
SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-010
Sheet 10 of –



STATION
4+05



STATION
4+30

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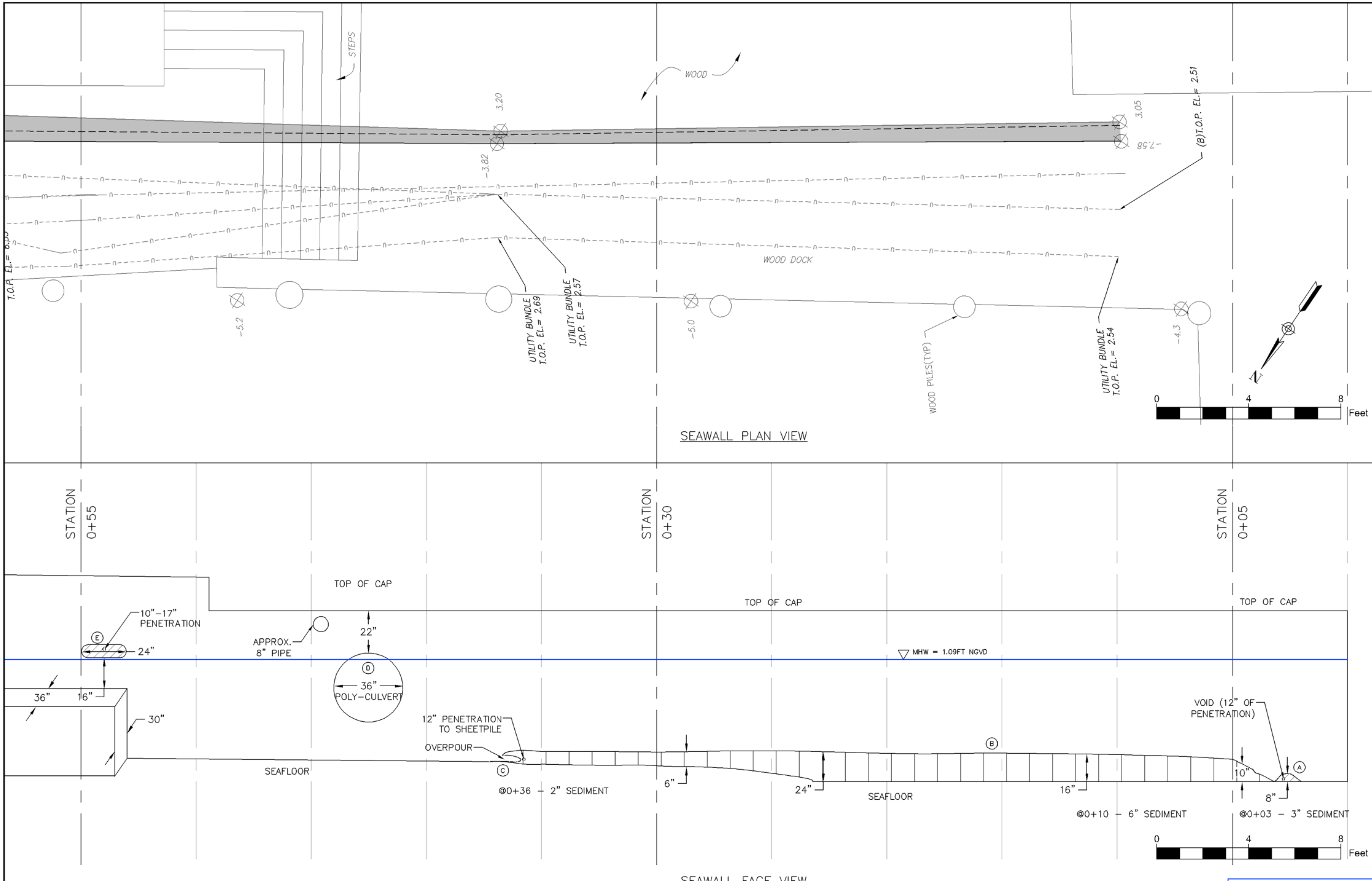
CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL CROSS-SECTIONS

KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-011
Sheet 11 of –



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CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL FACE VIEW

KEY WEST, MONROE COUNTY, FLORIDA

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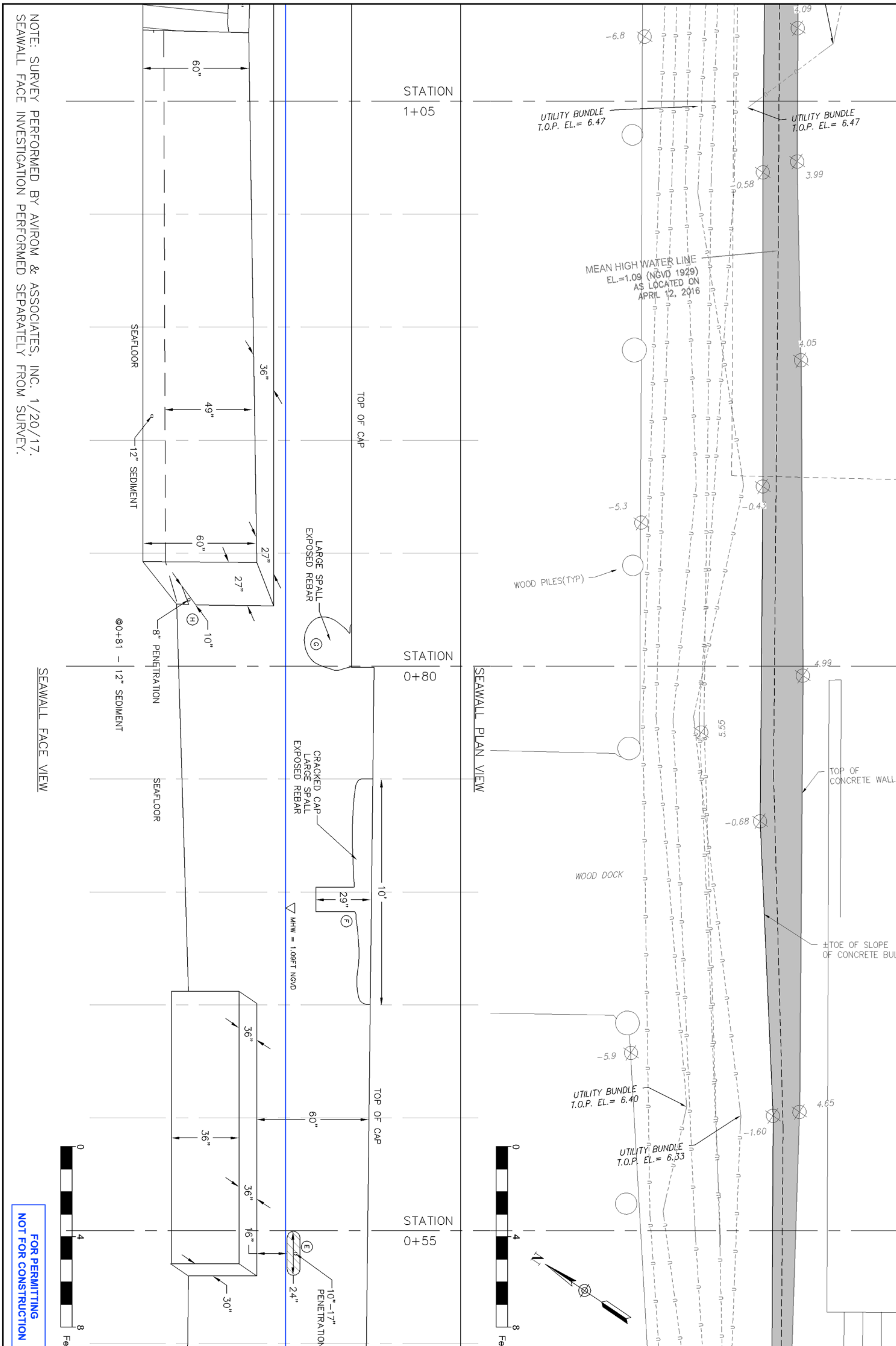
SEAL OF THE CITY OF KEY WEST, FLORIDA

Sheet Reference:
C-012
Sheet 12 of -



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Sheet Reference: C-013 Sheet 13 of –	**CITY OF KEY WEST** **TURTLE KRAAL SEAWALL** SEAWALL FACE VIEW **KEY WEST, MONROE COUNTY, FLORIDA**	**TETRA TECH INC.** 759 SOUTH FEDERAL HWY SUITE 314 STUART, FL 34994–2936 TEL: (772) 781–3400 FAX: (772) 781–3411 CERTIFICATE OF AUTHORIZATION NO. 2429	Designed by: S. MCGAHEE; Drawn By: F. MARTINEZ; Checked By: ––; Reviewed By: ––; Design file no: TT_TURTLEKRAALS_VS_FACE.DWG; Scale: AS SHOWN

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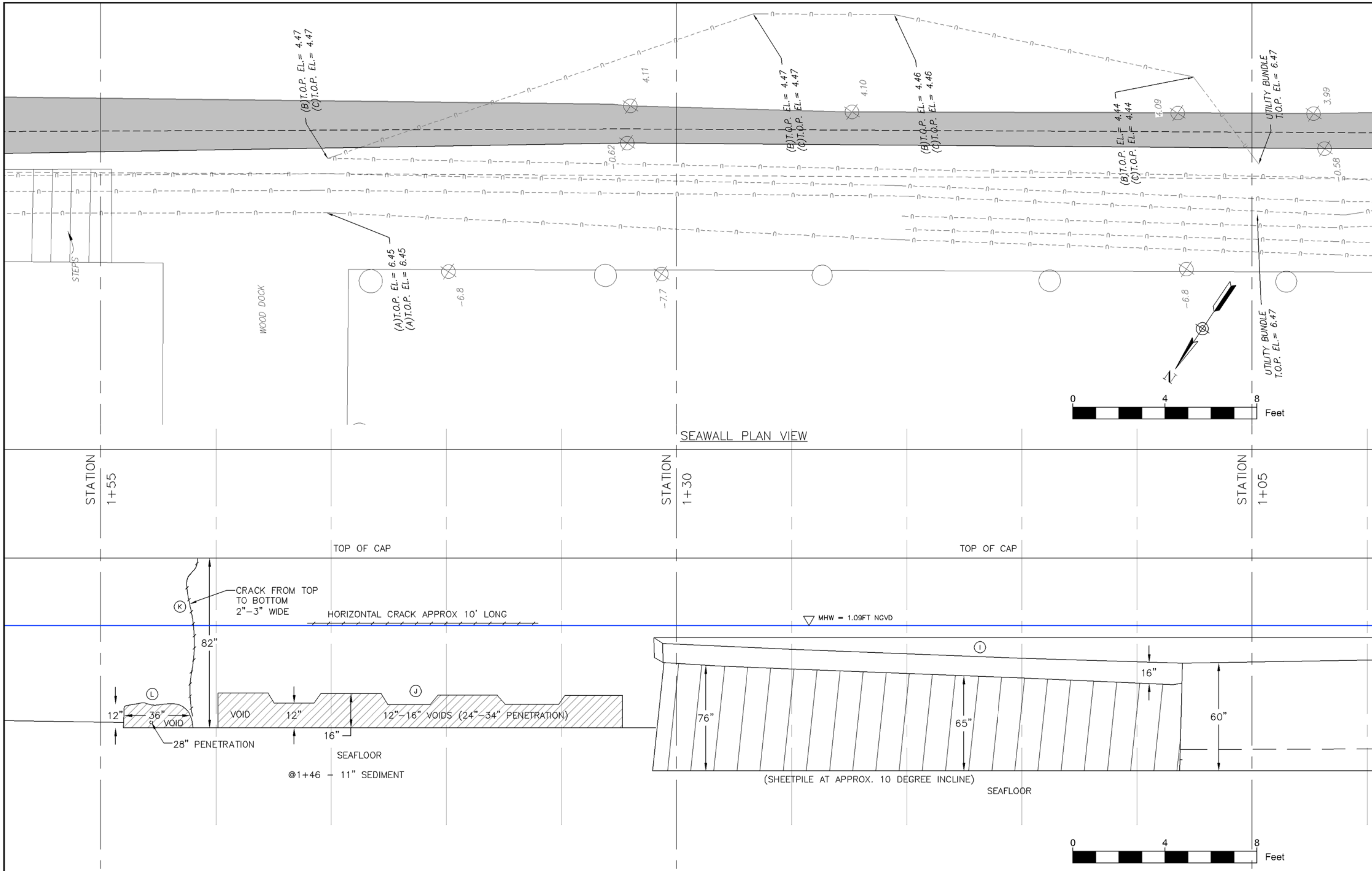
Florida PE No. 57536

SEAL OF THE CITY OF KEY WEST, FLORIDA



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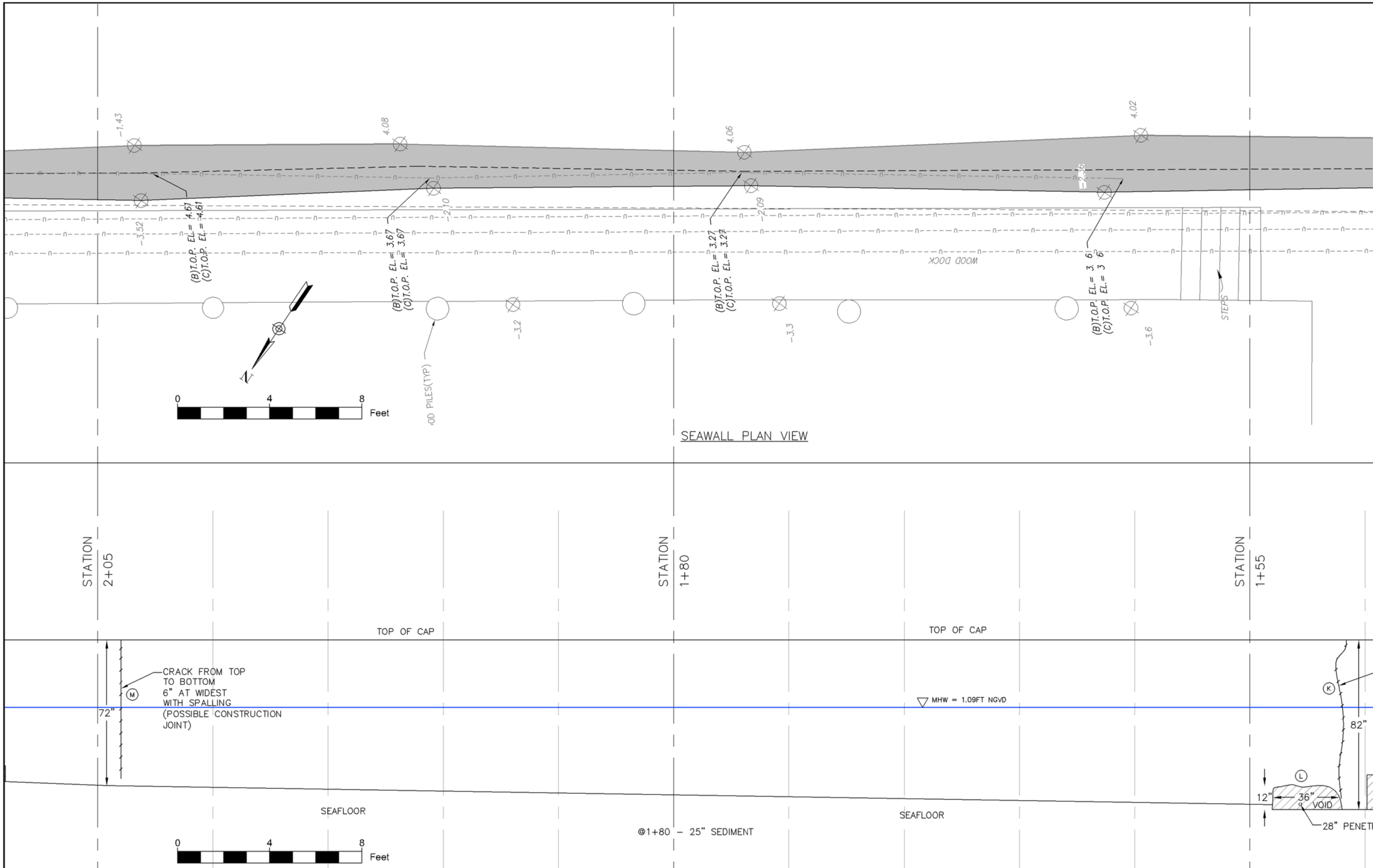
SEAL OF THE CITY OF KEY WEST, FLORIDA

Sheet Reference:
C-014
Sheet 14 of –



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SEAWALL FACE VIEW

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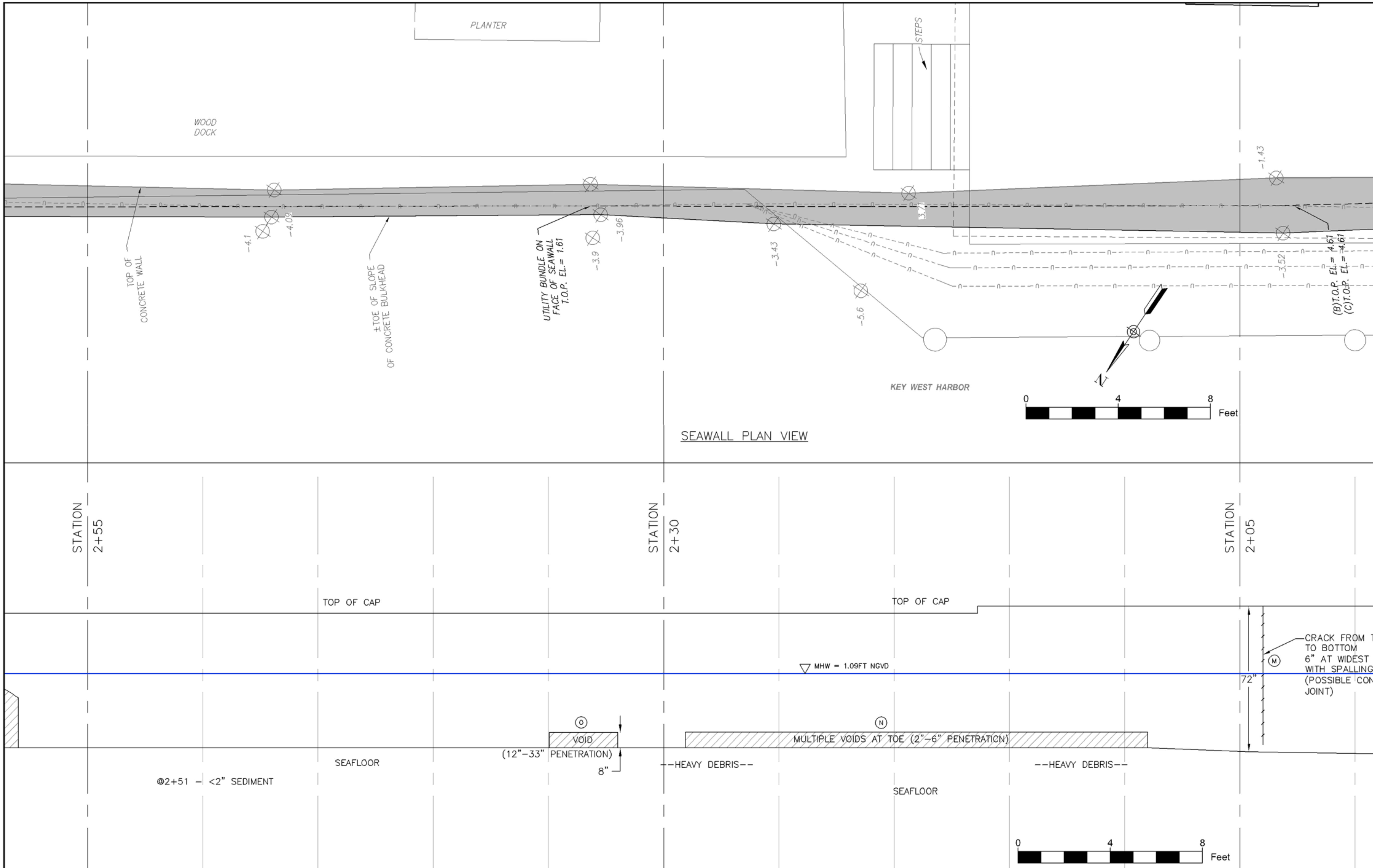
Sheet Reference:
C-015
Sheet 15 of –



SEAWALL FACE VIEW

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SEAWALL FACE VIEW

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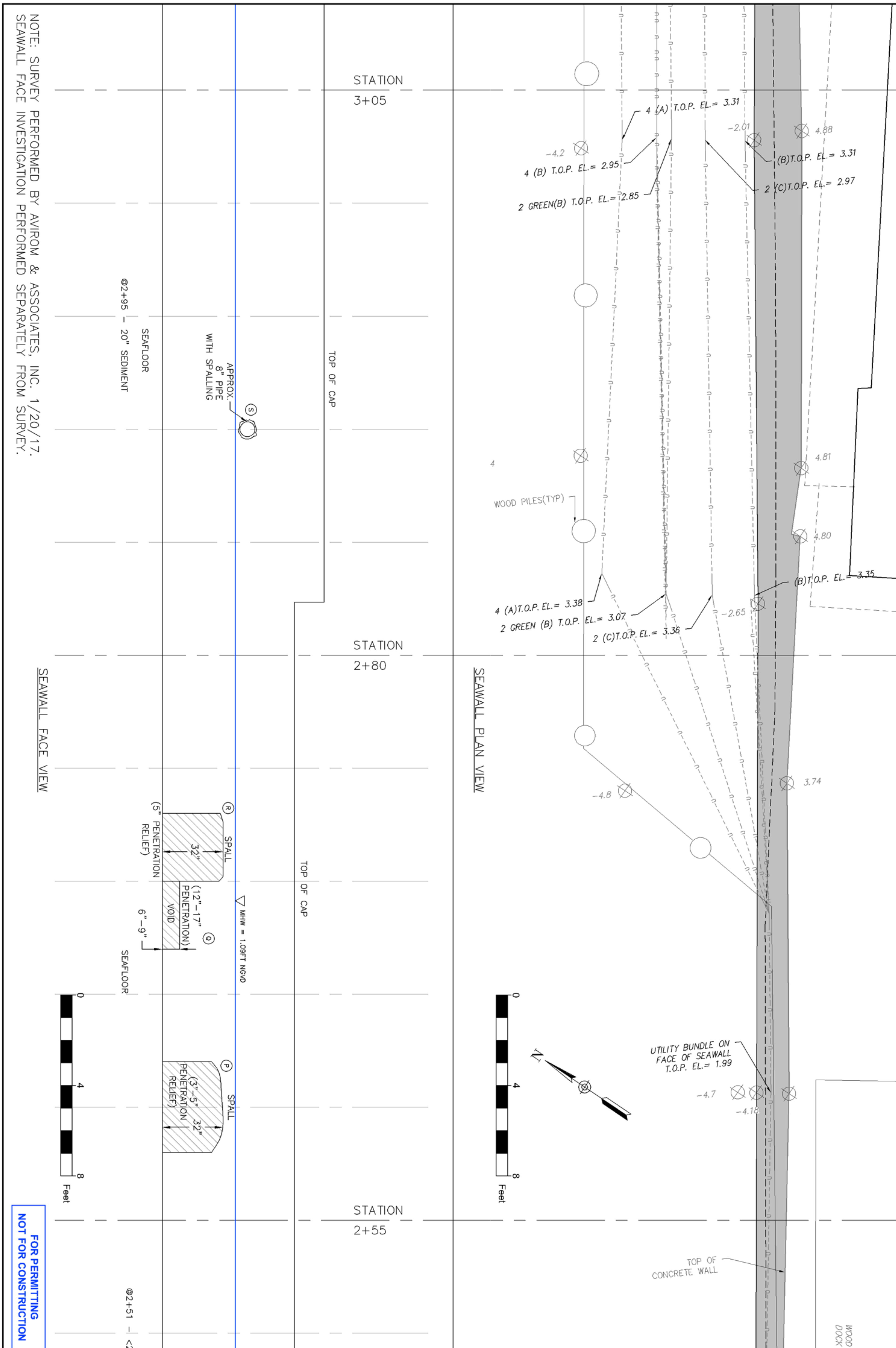
SEAL OF THE CITY OF KEY WEST, FLORIDA

Sheet Reference:
C-016



Sheet Reference: C-017

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CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL FACE VIEW

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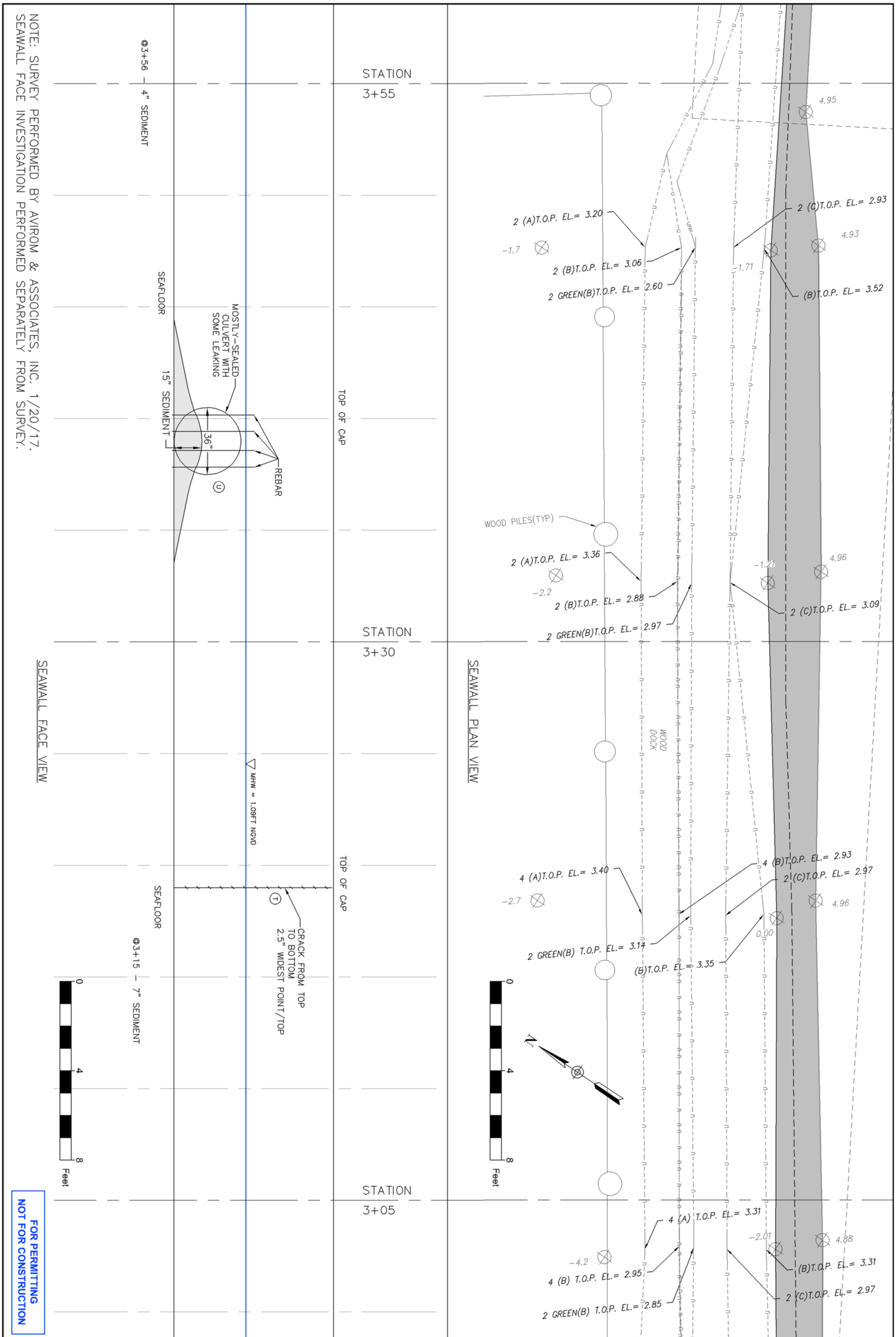
Florida PE No. 57536

SEAL OF THE CITY OF KEY WEST, FLORIDA



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CITY OF KEY WEST
TURTLE KRAAL SEAWALL

SEAWALL FACE VIEW

KEY WEST, MONROE COUNTY, FLORIDA

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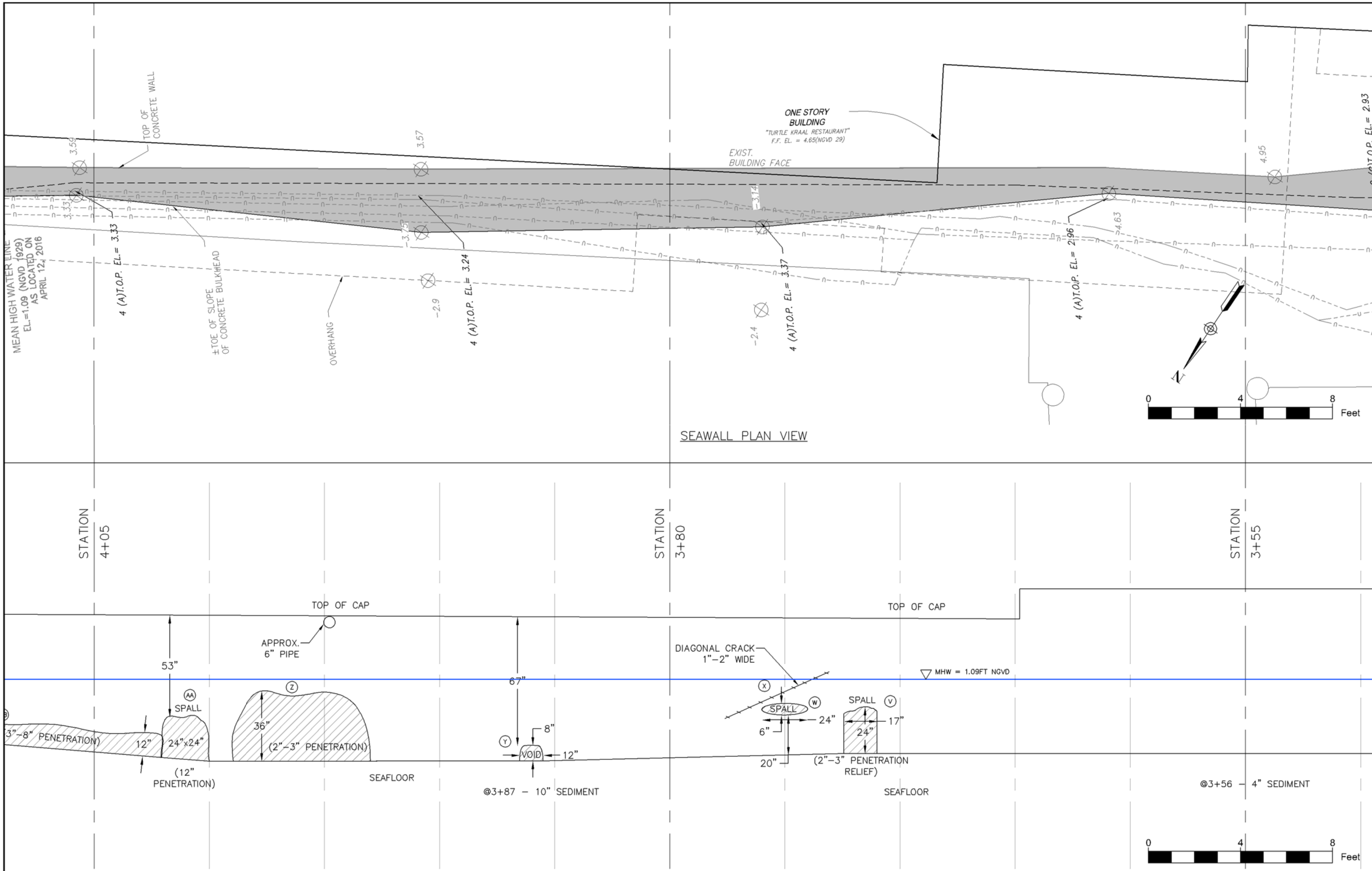
SEAL OF THE CITY OF KEY WEST, FLORIDA



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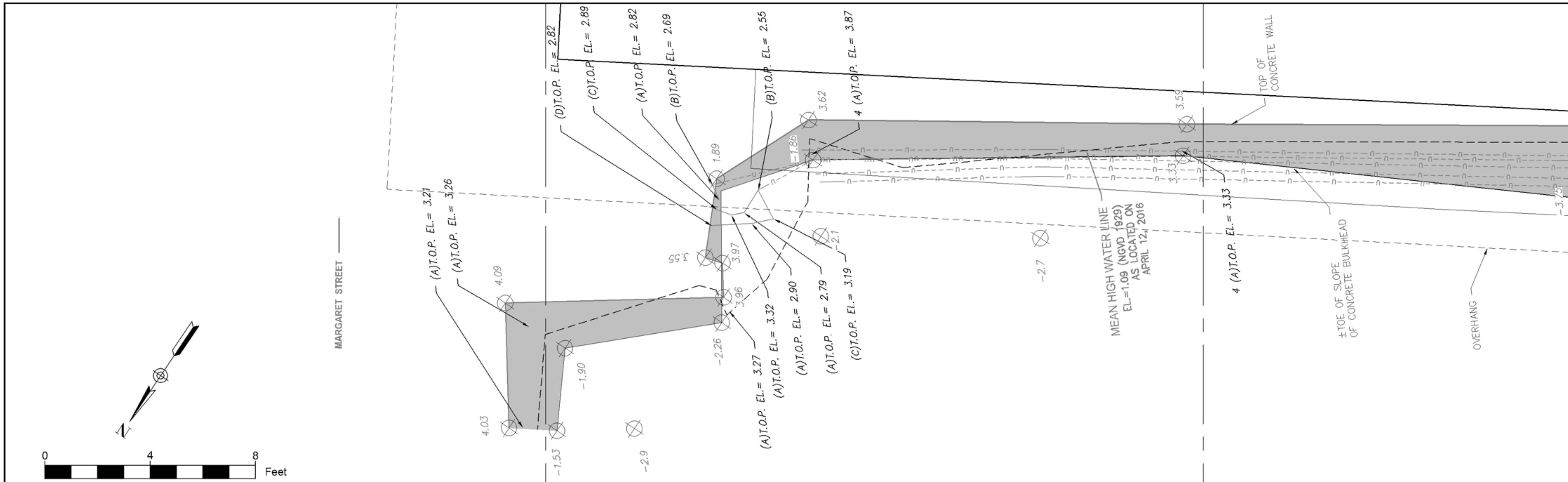
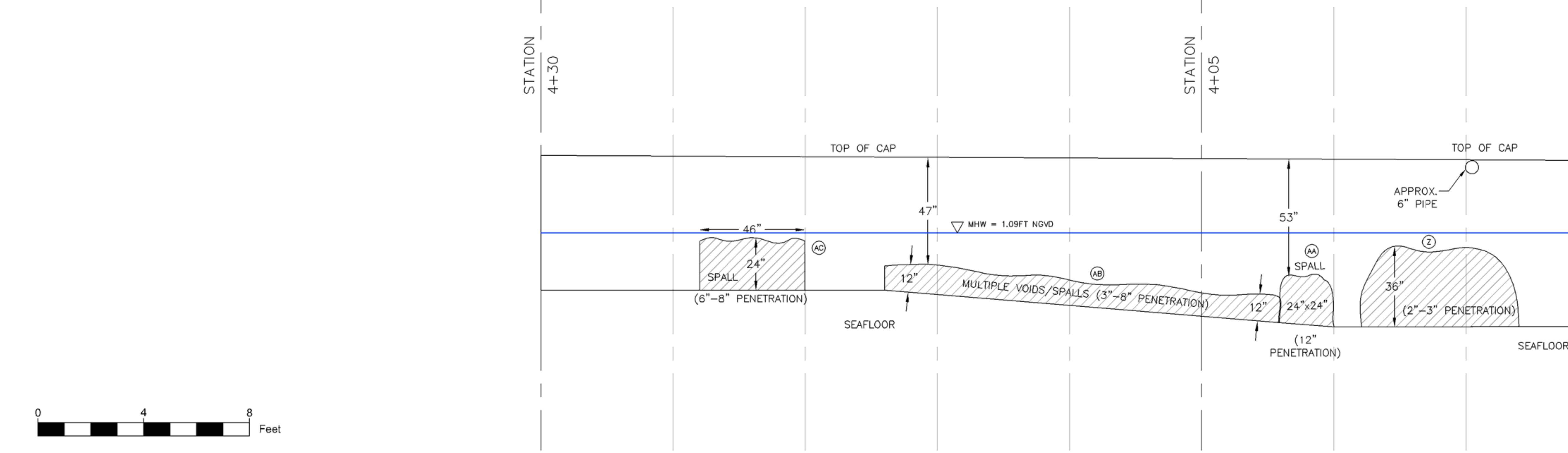
Sheet Reference:
C-019
Sheet 19 of –



SEAWALL PLAN VIEW



SEAWALL FACE VIEW

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Stuart E McGahee
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SEAL OF THE CITY OF KEY WEST, FLORIDA

Sheet Reference:
C-020
Sheet 20 of -

DEFECT	DEFICIENCY TYPE	DESCRIPTION	APPROXIMATE LOCATION (STATIONS)	HEIGHT (IN)	WIDTH (IN)	DEPTH/ PENETRATION (IN)	APPROX. VOLUME (CF)
A	VOID/SPALL	VOID	0+02 TO 0+03	8	12	12	0.666667
B	SCOUR	EXPOSED SHEETPILE	0+03 TO 0+36	24	396	12	66
C	OVERPOUR	EXCESS CONCRETE	0+36	N/A	N/A	N/A	
D	OPEN CULVERT	OPEN CULVERT WITH NO MANATEE GRATE	0+41 TO 0+44	36	36	N/A	
E	VOID/SPALL	VOID	0+53 TO 0+55	0.5	24	17	0.118056
F	CRACK	CRACK/SPALL AT CAP WITH SOME EXPOSED REBAR	0+65 TO 0+75	29	120	N/A	
G	VOID/SPALL	SPALL WITH SOME EXPOSED REBAR	0+80 TO 0+82	24	24	N/A	
H	VOID/SPALL	VOID IN TOE/BENCH	0+83	3	10	8	0.138889
I	EXPOSED SHEETPILE	POSSIBLE SHEETPILE FOR REPAIR WITH CONCRETE CAP	1+08 TO 1+31	76	276	N/A	
J	VOID/SPALL	AREA OF VOIDS	1+33 TO 1+50	16	204	34	64.22222
K	CRACK	2-3" TOP-TO-BOTTOM CRACK	1+51	82	3	N/A	
L	VOID/SPALL	VOID	1+51 TO 1+54	12	36	28	7
M	CRACK	6" TOP-TO-BOTTOM CRACK	2+04	72	6	N/A	
N	VOID/SPALL	AREA OF VOIDS	2+09 TO 2+29	8	240	6	6.666667
O	VOID/SPALL	VOID	2+32 TO 2+35	8	36	33	5.5
P	VOID/SPALL	SPALL	2+58 TO 2+62	32	48	5	4.444444
Q	VOID/SPALL	VOID	2+67 TO 2+70	9	36	17	3.1875
R	VOID/SPALL	SPALL	2+70 TO 2+73	32	36	5	3.333333
S	VOID/SPALL	PIPE WITH SPALLING	2+90	8	8	2	0.074074
T	CRACK	2.5" TOP-TO-BOTTOM CRACK	3+19	82	2.5	N/A	
U	LEAKING CULVERT	MOSTLY-SEALED 36" CULVERT	3+38 TO 3+41	36	36	N/A	
V	VOID/SPALL	SPALL	3+71 TO 3+73	24	17	3	0.708333
W	VOID/SPALL	SPALL	3+74 TO 3+76	6	24	3	0.25
X	CRACK	1-2" DIAGONAL CRACK	3+73 TO 3+78	60	2	N/A	
Y	VOID/SPALL	VOID	3+86	8	12	UNKNOWN	
Z	VOID/SPALL	SPALL	3+93 TO 3+99	36	72	3	4.5
AA	VOID/SPALL	SPALL	4+00 TO 4+02	24	24	12	4
AB	VOID/SPALL	AREA OF VOIDS	4+02 TO 4+17	12	180	8	10
AC	VOID/SPALL	SPALL	4+20 TO 4+24	24	48	8	5.333333
						TOTAL AREA OF VOID/SPALL	120.1435

TYPE OF DEFICIENCY	
(yellow)	EXPOSED SHEETPILE WALL BOTTOM
(orange)	VOID/SPALL
(green)	CRACK
(gray)	OTHER/POSSIBLE DEFICIENCY

FOR PERMITTING
NOT FOR CONSTRUCTION

Stuart E McGahee
Florida PE No. 57536

Designed by: S. MCGAHEE
Drawn By: F. MARTINEZ
Checked By: --
Reviewed By: --
Design file no: TT_TURTLEKRAALS_VS_FACE.DWG
Scale: AS SHOWN

Mark	Description	Date	Appr.
-	-	-	-

TETRA TECH INC.
759 SOUTH FEDERAL HWY
SUITE 314
STUART, FL 34994-2936
TEL: (772) 781-3400
FAX: (772) 781-3411
CERTIFICATE OF AUTHORIZATION NO. 2429

CITY OF KEY WEST
TURTLE KRAAL SEAWALL
SEAWALL FACE DEFICIENCIES LIST
KEY WEST, MONROE COUNTY, FLORIDA

Sheet Reference:
C-021
Sheet 21 of –

Appendix 4

Coral Inventory and Benthic Resource Survey

CORAL INVENTORY AND BENTHIC RESOURCE SURVEY
TURTLE KRAALS
KEY WEST, FLORIDA

Prepared By:

Tetra Tech, Inc.
759 South Federal Highway
Stuart, FL 34994

Prepared For:

City of Key West.
3140 Flagler Avenue
Key West, FL 33040

TETRA TECH

March 2017

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1.0 Introduction

From February 21-23, 2017, Tetra Tech, Inc. (Tt) performed a coral inventory and benthic resource survey of the Turtle Kraals bulkhead on behalf of the city of Key West to satisfy permitting requirements associated with the repair and fortification of the bulkhead. Qualified staff were onsite to coordinate survey logistics and catalogue benthic resources within the designated survey area. Presented in this report are the findings of the survey.

The purpose of the survey is to determine the quantity of stony corals, spatial extent, size class, relocation candidates, and total coral tissue area within the project area. Additionally, the survey will identify any other biological resources within the project area. This survey will supply the regulatory agencies with the most up-to-date resource data for the project site. Further, data collected from this survey may be used during the planning phase of the project to minimize and avoid impacts to stony corals and/or other resources within the project area.

2.0 Site Description

The project area is located along the northwestern shore of Key West and lies within Key West Bight, adjacent to the Key West Marina and the Key West Ferry terminal (see Figure 1). Global Positioning System (GPS) coordinates for the site are as follows: Latitude 24º 33.691' North, Longitude 81º 48.055' West. The waters adjacent to the project area are classified by the Florida Department of Environmental Protection (FDEP) as a Class III (Recreation, Propagation and Maintenance of a Healthy, Well-Balanced Population of Fish and Wildlife) water body.

The survey area is an approximately 430-foot (ft.) bulkhead with an existing discontinuous dock and boardwalk structures along its length (see Figure 2). The bulkhead is a part of hardened shoreline with the marina basin that supports a variety of amenities. Water depths ranged from 3 to 10 feet of seawater (fsw). Sediment composition along the base of the bulkhead is predominantly biogenic mud/fine sand mixed with debris (concrete, rubble, pipes, etc.). Underwater visibility was approximately 2-5 feet; water temperature was 76° Fahrenheit, with no noticeable current detected during the survey.

3.0 Methods

Field activities were performed using a 19-ft. catamaran in order to safely access the survey area. Two (2) Tt certified scientific divers trained in marine biological resource identification cataloged stony corals and noted other marine benthic flora and fauna along the face of the approximately 430-ft. bulkhead and a 5 to 10-ft. buffer on each end of the bulkhead and along the seafloor apron. Data were collected along a single transect positioned from southwest (0 ft.) to northeast (430 ft.) along the base of the bulkhead. Figure 2 presents the survey area and transect location graphically.

The transect tape was used by divers to record the location of each colony along the bulkhead and buffer area. Underwater photography was used to record representative images of resources and conditions present within the survey area. Divers followed the National Oceanic and Atmospheric Administration (NOAA) Protocol for Benthic Surveys of Coral Resources in Florida Keys National Marine Sanctuary (FKNMS) for seawalls and shoreline structures (rip rap, bulkheads, boat ramps, bridges) dated April 29, 2011.

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Figure 1. Project vicinity map

Back of figure

Figure 2. Survey area and transect location

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4.0 Results

No species listed as threatened or endangered, soft corals (gorgonians), or species of concern were documented during the survey. Silt-covered filamentous diatoms makes up the dominant vegetative cover followed by discontinuous seagrass (*Halophila decipiens*) along the silt-covered hard substrate (concrete). Other functional groups documented during the survey include sponges, bryozoans, tunicates, tube worms, fin fish and motile invertebrates. The bottom of the marina basin is comprised of barren mud and detritus with no vegetative cover. Flora and fauna observed during the survey are presented in Table 1.

Table 1. List of marine species observed at the project site

Functional Group	Common Name	Scientific Name
Scleractinia	Lesser Starlet Coral	*Siderastrea radians*
Sponges	Yellow calcareous sponge	*Clathrina canariensis*
		Haliclona sp.
	Orange lumpy encrusting sponge	*Scopalina rutzleri*
Bryozoans	Fan bryozoan	*Reteporellina evelinae*
Tunicates	Black solitary tunicate	*Phallusia nigra*
	Colonial tunicate	*Botryllus* sp.
	Mangrove tunicate	*Ecteinascidia turbinata*
Seagrass	Paddle grass	*Halophila decipiens*
Algae	White scroll algae	*Padina jamaicensis*
Fin Fish	Tarpon	*Megalops atlanticus*
	Mangrove snapper	*Lutjanus griseus*
Other invertebrates	Spiny Lobster	*Panulirus argus*
	Eastern oyster	*Crassostrea virginica*

One species of stony coral, *Siderastrea radians*, was documented during the survey. A total of 119 individual coral colonies accounting for 2,700 square centimeters (cm^2) of live tissue were catalogued: 94 attached to a bulkhead shelf, 10 on the vertical face of the bulkhead, and 15 on debris at the base of the bulkhead. Table 2 summarizes the coral inventory by abundance and size class for the aforementioned corals. A comprehensive list of all stony corals catalogued during the survey is provided in Appendix A; representative photographs are illustrated in Appendix B.

Table 2. Coral Inventory by abundance & size class: Corals encrusting bulkhead and adjacent concrete/debris

Size Class (cm)	0 to <5	5 to <10	10 to <15	TOTAL
SPECIES NAME				
Siderastrea radians	77	31	11	119
TOTAL by size class	77	31	11	119

The highest abundance of stony corals (53 colonies) are along the east end of the bulkhead (229-267 ft. segment) encrusted to the bulkhead shelf (33 colonies), bulkhead (8 colonies), and debris (12 colonies). The second highest abundance of coral colonies are along the middle of the bulkhead (the

90-112 ft. segment) encrusted to the bulkhead shelf (38 colonies). Corals between the 213-215 ft. and 257-263 ft. segments are attached to movable debris adjacent to the bulkhead and/or flat concrete (15 colonies total). Figure 3 presents the coral inventory by transect segment in 10-ft. increments.

The majority (65 percent) of the stony corals documented were in the less than 5 cm size class, 26 percent in the 5 to 10 cm size class, and 9 percent in the 11 to 15 cm size class. Table 3 summarizes the coral inventory by size class and location along each 10-ft. section of the bulkhead. Figures 4 through 6 present coral distribution by size class along the bulkhead.

Table 3. Coral Inventory by size class and location along length of bulkhead (10-ft. increments): Corals encrusting bulkhead and adjacent concrete/debris

Location (ft.)	0 to <5cm	5 to 10cm	11 to 15cm	Total
0-10	–	–	–	0
11-20	–	–	–	0
21-30	–	–	–	0
31-40	–	–	–	0
41-50	–	–	–	0
51-60	–	–	–	0
61-70	–	–	–	0
71-80	–	–	–	0
81-90	4	4	1	9
91-100	8	2	1	11
101-110	15	4	1	20
111-120	1	2	–	3
121-130	2	1	–	3
131-140	5	7	1	13
141-150	2	–	–	2
151-160	–	–	–	0
161-170	–	–	–	0
171-180	–	–	–	0
181-190	–	–	–	0
191-200	–	–	–	0
201-210	1	–	–	1
211-220	3	–	–	3
221-230	3	–	–	3
231-240	2	1	1	4
241-250	17	3	3	23
251-260	8	5	2	15
261-270	6	2	–	8
271-280	–	–	–	0
283	–	–	1	1
TOTAL	77	31	11	119

Discontinuous seagrass (*Halophila decipiens*) cover was documented along the length of the bulkhead. Seagrasses were encountered growing on the bulkhead shelf on accumulated sediments as well as within the buffer zone adjacent to the base of the bulkhead. Seagrass cover ranges from less than 5 to 75%. Table 4 summarizes the abundance and location of seagrass along the bulkhead/concrete and 5-10 ft. buffer area. Figure 7 presents the locations of seagrass cover graphically.

Table 4. Seagrass Inventory by location & abundance along length of bulkhead (10-ft. increments)

Location (ft.)	Species	Location	B-B Score	Percent Conversion
59-60	*H. decipiens*	Bulkhead shelf	1	2.5
107-118	*H. decipiens*	Bulkhead shelf	4	62.5
120-128	*H. decipiens*	Bulkhead shelf	3	37.5
130	*H. decipiens*	Bulkhead shelf	1	2.5
311-367	*H. decipiens*	buffer	1-4	
372-391	*H. decipiens*	buffer	1-3	

5.0 Conclusions and Recommendations

A total of 15 corals (all *S. radians*) that are <15 centimeters (cm) in maximum dimension were identified as potential candidates for relocation. Relocation status is based on the apparent unattached nature of the substrate type. Relocation candidates are located on various types of debris (concrete rubble, rope, etc.); therefore require minimal effort to move. The total area of coral tissue suitable for relocation is 336 cm^2.

There are 104 <15cm corals attached to the flat concrete and bulkhead that are not recommended for relocation due to their size and morphology; these stony corals are encrusted on the concrete/bulkhead and cannot be relocated without damage. No corals were documented with a maximum dimension ≥15cm. The total area of coral tissue not suitable for relocation is 2,364 cm^2. Table 5 summarizes the coral inventory by relocation potential.

Seagrass (*Halophila decipiens*) is present within the 5-10 ft. buffer zone and along an elevated bulkhead shelf. The shelf is approximately 4-5 ft. above the seafloor and approximately 1.5 ft. wide. Over time, suspended sediments have accumulated on the shelf providing suitable substrate for the recruitment of seagrass. Seagrasses located along the bulkhead shelf may be relocated to a nearby suitable substrate. Impacts to mangroves will not occur as a result of the project as they are not within the proposed project area or adjacent buffer zone.

Table 5. Coral Inventory by relocation potential: Corals encrusting bulkhead and adjacent concrete/debris

Size Class (cm) and Surface Area (cm^2)	# <15 cm Coral candidates for relocation	Total Surface Area <15 cm corals to relocate (cm^2)	# <15 cm Corals not able to be relocated	Total Surface Area <15 cm corals not relocated (cm^2)	# >15 cm Corals to be relocated	Total Surface Area >15 cm corals to relocate (cm^2)	TOTAL # relocatable corals	TOTAL # unrelocatable corals	TOTAL # corals
SPECIES NAME									
Siderastrea radians	15	190	104	2542	n/a	n/a	15	104	119
TOTAL by size/area	15	190	104	2542	n/a	n/a	15	104	119

[1] Relocation potential determined by substrate type for corals (i.e., unattached versus attached)

Figure 3. Coral inventory by transect segment (10-ft.)

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Figure 4. Coral inventory of stony corals in <5 cm size class by transect segment (10-ft)

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Figure 5. Coral inventory of stony corals in 5–10 cm size class by transect segment (10-ft)

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Figure 6. Coral inventory of stony corals in 11–15 cm size class by transect segment (10-ft)

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Figure 7. Seagrass cover by transect segment (10-ft)

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APPENDIX A
Stony Coral Inventory

ID No.	Species	Max dimension (cm)	Min dimension (cm)	Area (cm^2)	Location (ft)	Comment
1	Srad	3	2	6	84	on flat concrete
2	Srad	3	2	6	84	on flat concrete
3	Srad	13	9	117	84	on flat concrete
4	Srad	7	5	35	84	on flat concrete
5	Srad	4	3	12	84	on flat concrete
6	Srad	8	6	48	90	on flat concrete
7	Srad	3	3	9	90	on flat concrete
8	Srad	8	8	64	90	on flat concrete
9	Srad	6	6	36	90	on flat concrete
10	Srad	2	2	4	91	on flat concrete
11	Srad	5	5	25	92	on flat concrete
12	Srad	2	2	4	93	on flat concrete
13	Srad	5	3	15	96	on flat concrete
14	Srad	3	2	6	98	on flat concrete
15	Srad	1	1	1	98	on flat concrete
16	Srad	2	2	4	98	on flat concrete
17	Srad	1	1	1	98	on flat concrete
18	Srad	13	8	104	98	on flat concrete
19	Srad	4	4	16	100	on flat concrete
20	Srad	3	3	9	100	on flat concrete
21	Srad	5	2	10	102	on flat concrete
22	Srad	2	2	4	102	on flat concrete
23	Srad	3	3	9	103	on flat concrete
24	Srad	3	3	9	105	on flat concrete
25	Srad	7	4	28	108	on flat concrete
26	Srad	6	4	24	108	on flat concrete
27	Srad	2	2	4	108	on flat concrete
28	Srad	3	3	9	108	on flat concrete
29	Srad	3	3	9	108	on flat concrete
30	Srad	9	7	63	109	on flat concrete
31	Srad	3	3	9	109	on flat concrete
32	Srad	1	1	1	109	on flat concrete
33	Srad	1	1	1	109	on flat concrete
34	Srad	4	2	8	109	on flat concrete
35	Srad	12	8	96	109	on flat concrete
36	Srad	3	3	9	110	on flat concrete
37	Srad	3	3	9	110	on flat concrete
38	Srad	2	2	4	110	on flat concrete
39	Srad	1	1	1	110	on flat concrete
40	Srad	1	1	1	110	on flat concrete
41	Srad	7	7	49	112	on flat concrete
42	Srad	7	7	49	112	on flat concrete
43	Srad	2	2	4	112	on flat concrete
44	Srad	7	7	49	126	on flat concrete
45	Srad	3	3	9	130	on face of wall

ID No.	Species	Max dimension (cm)	Min dimension (cm)	Area (cm^2)	Location (ft)	Comment
46	Srad	3	3	9	130	on face of wall
47	Srad	13	10	130	136	on flat concrete
48	Srad	7	3	21	136	on flat concrete
49	Srad	7	5	35	137	on flat concrete
50	Srad	4	4	16	137	on flat concrete
51	Srad	8	8	64	138	on flat concrete
52	Srad	3	3	9	139	on flat concrete
53	Srad	5	5	25	139	on flat concrete
54	Srad	1	1	1	139	on flat concrete
55	Srad	1	1	1	139	on flat concrete
56	Srad	2	2	4	139	on flat concrete
57	Srad	5	5	25	140	on flat concrete
58	Srad	5	5	25	140	on flat concrete
59	Srad	6	6	36	140	on flat concrete
60	Srad	3	2	6	142	on flat concrete
61	Srad	2	2	4	142	on flat concrete
62	Srad	2	2	4	206	on flat concrete
63	Srad	2	2	4	213	on debris at base of wall
64	Srad	4	4	16	215	on debris at base of wall
65	Srad	2	2	4	215	on debris at base of wall
66	Srad	3	3	9	229-230	on base of wall
67	Srad	2	2	4	229-230	on base of wall
68	Srad	1	1	1	229-230	on base of wall
69	Srad	9	5	45	236	on base of wall
70	Srad	1	1	1	236	on base of wall
71	Srad	10	10	100	239	on flat concrete
72	Srad	2	2	4	239	on flat concrete
73	Srad	13	3	39	241	on flat concrete
74	Srad	1	1	1	245	on flat concrete
75	Srad	6	6	36	245	on flat concrete
76	Srad	2	2	4	247	on flat concrete
77	Srad	3	3	9	247	on flat concrete
78	Srad	2	2	4	247	on flat concrete
79	Srad	2	2	4	247	on flat concrete
80	Srad	11	11	121	248	on flat concrete
81	Srad	2	2	4	248	on flat concrete
82	Srad	2	2	4	248	on flat concrete
83	Srad	2	2	4	248	on flat concrete
84	Srad	3	3	9	248	on flat concrete
85	Srad	2	2	4	248	on flat concrete
86	Srad	2	2	4	248	on flat concrete
87	Srad	1	1	1	249	on flat concrete
88	Srad	1	1	1	249	on flat concrete
89	Srad	1	1	1	249	on flat concrete
90	Srad	1	1	1	249	on flat concrete

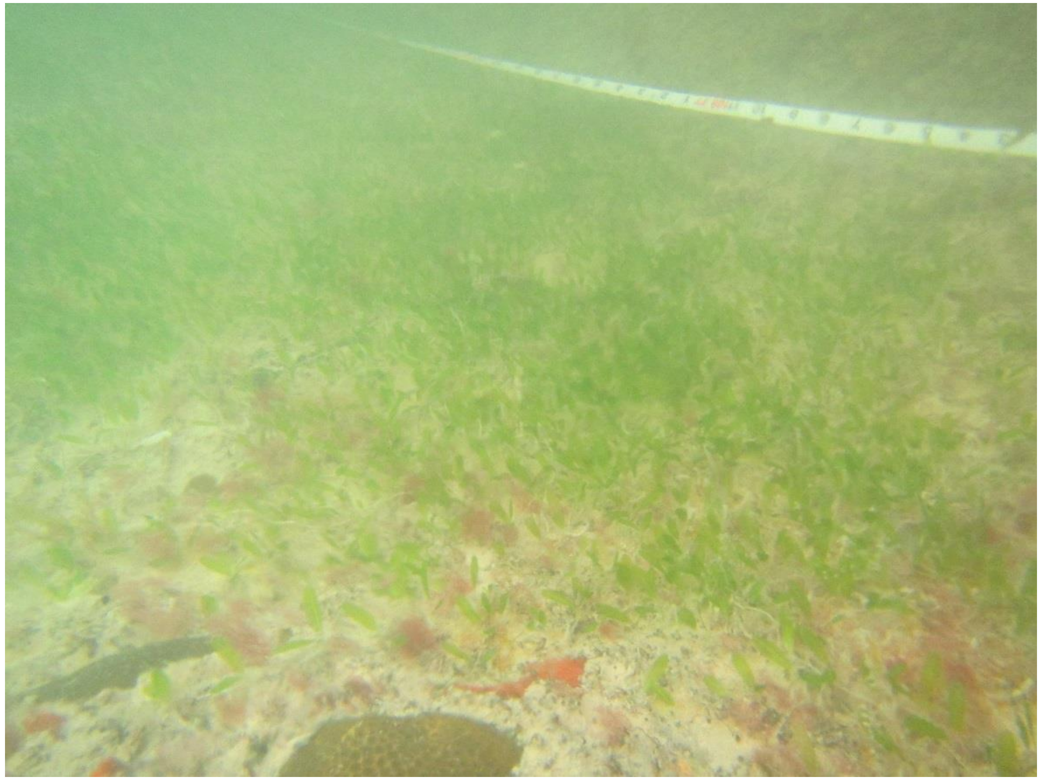
ID No.	Species	Max dimension (cm)	Min dimension (cm)	Area (cm^2)	Location (ft)	Comment
91	Srad	1	1	1	249	on flat concrete
92	Srad	3	3	9	249	on flat concrete
93	Srad	5	3	15	249	on flat concrete
94	Srad	5	5	25	249	on flat concrete
95	Srad	13	3	39	249	on flat concrete
96	Srad	4	4	16	251	on flat concrete
97	Srad	11	11	121	251	on flat concrete
98	Srad	6	6	36	252	on flat concrete
99	Srad	2	2	4	254	on flat concrete
100	Srad	4	4	16	258	on wall
101	Srad	3	3	9	257	on debris
102	Srad	3	3	9	258	on debris
103	Srad	10	9	90	258	on debris
104	Srad	3	3	9	258	on debris
105	Srad	6	6	36	258	on debris
106	Srad	2	2	4	258	on debris
107	Srad	8	4	32	258	on debris
108	Srad	6	6	36	259	on debris
109	Srad	2	2	4	259	on debris
110	Srad	5	5	25	259	on debris
111	Srad	7	7	49	263	on debris
112	Srad	3	3	9	263	on debris
113	Srad	4	4	16	263	on wall
114	Srad	4	4	16	264	on wall
115	Srad	4	4	16	265	on flat concrete
116	Srad	4	4	16	265	on flat concrete
117	Srad	5	5	25	267	on flat concrete
118	Srad	2	2	4	267	on flat concrete
119	Srad	13	13	169	283	on flat concrete

APPENDIX B
Photographic Documentation: Typical Photos of Representative Resources

Resources on Flat Concrete

Siderastrea radians

Halophila decipens



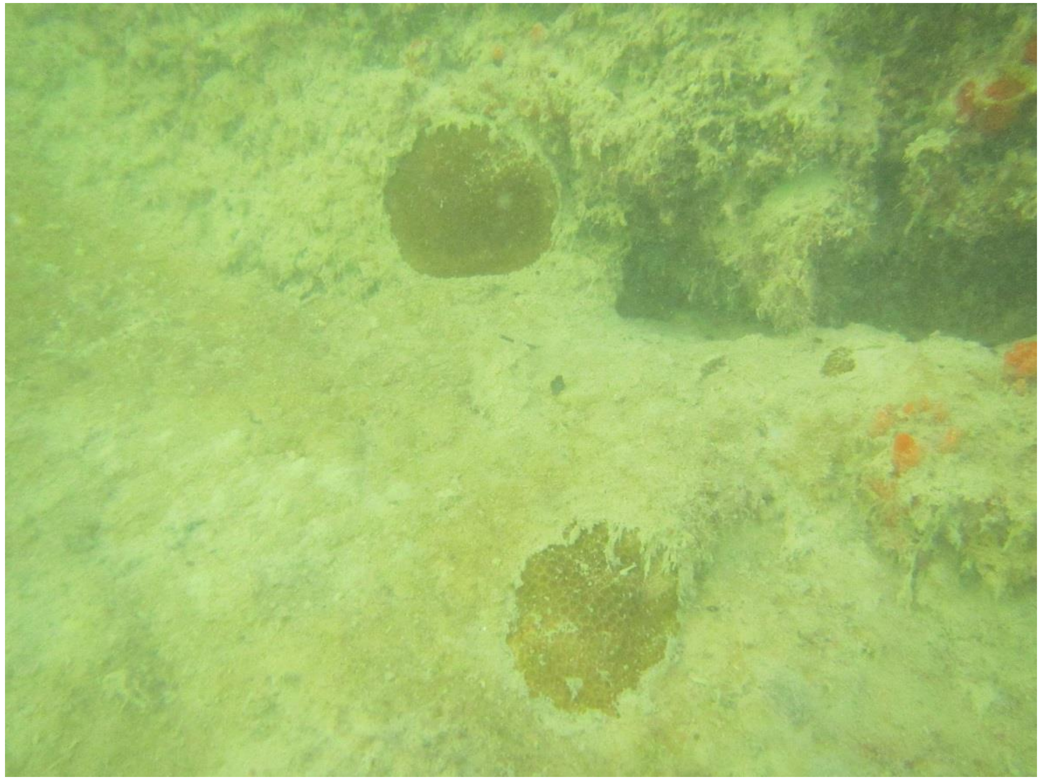
Resources on Flat Concrete

Halophila decipiens

Crassostrea virginica

Resources on Bulkhead

Siderastrea radians

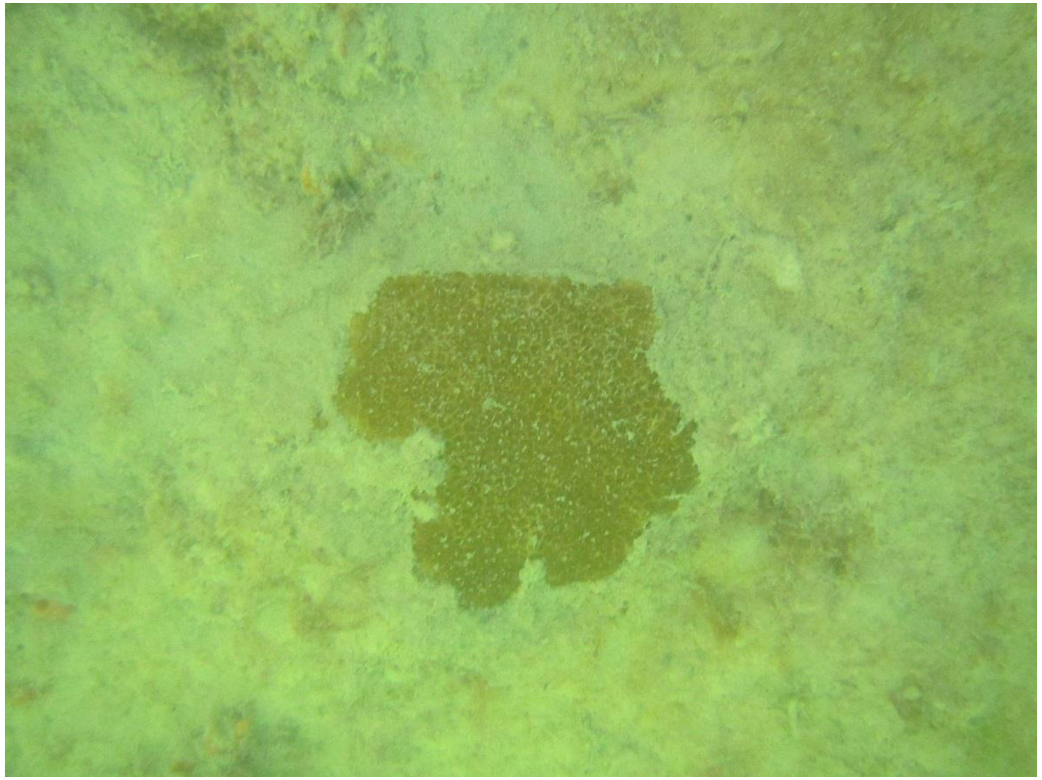


Padina jamaicensis

Resources on Bulkhead

***Botryllus* sp.**

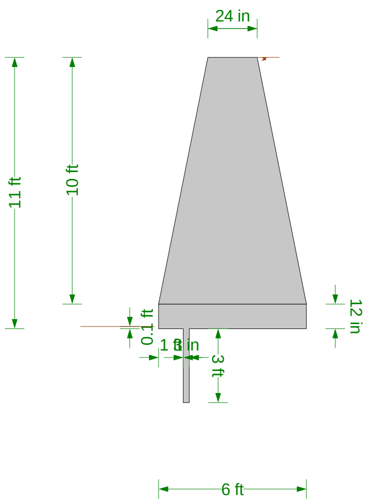
Siderastrea radians



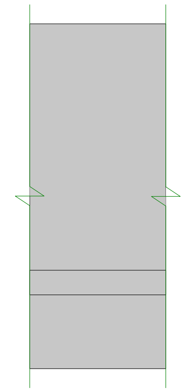
Appendix 5

Seawall Stability Check
(QUICKRWALL)

Design Detail



Concrete f'c = 4000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



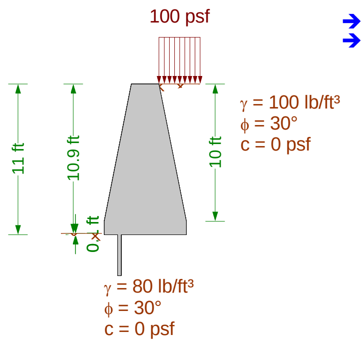
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.826	Overturning	1.82	1.50	0.6D + 1.0H
✗ 1.067	Sliding	1.41	1.50	0.6D + 1.0H
✓ 0.852	Bearing Pressure	3000 psf	2555 psf	1.0D + 1.0F + 1.0L + 1.0H
✓ 0.911	Bearing Eccentricity	10.94 in	12 in	1.0D + 1.0F + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.000	Shear	6072 lb/ft	0 lb/ft	1.4D + 1.4F
✓ -0.000	Moment	3.16 ft·k/ft	-0 ft·k/ft	1.4D + 1.4F
----- Heel Checks -----				
✓ 0.000	Shear	6072 lb/ft	0 lb/ft	1.4D + 1.4F
✓ -0.000	Moment	3.16 ft·k/ft	-0 ft·k/ft	1.4D + 1.4F
----- Stem Checks -----				
✓ 0.070	Moment	163.9 ft·k/ft	11.56 ft·k/ft	1.2D + 1.2F + 1.6L + 1.6H
✓ 0.073	Shear	43715 lb/ft	3200 lb/ft	1.2D + 1.2F + 1.6L + 1.6H

Criteria

Use basic criteria from common project settings	Yes
Building Code	IBC 2012
Concrete Load Combs	IBC 2012 (Strength)
Masonry Load Combs	ASCE 7-10 (ASD)
Stability Load Combs	ASCE 7-10 (ASD)
Apply Sds Factor to Seismic Combinations for Ev	No
Restrained Against Sliding	No
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bottom	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seismic	No
Allowable Bearing Pressure	3000 psf
Req'd Bearing Location	Middle third
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

- Passive pressure neglects top 0 ft of soil.
- Passive pressure is applied only to key.

Load Combinations

IBC 2012 (Strength)

1.4D + 1.4F
1.2D + 1.2F + 1.6L + 1.6H
1.2D + 1.2F + 1.6L + 0.9H
1.2D + 1.2F + 0.5L + 1.6H
1.2D + 1.2F + 0.5L + 0.9H
1.2D + 1.2F + 1.6H
1.2D + 1.2F + 0.9H
0.9D + 1.6H
0.9D + 0.9H
0.9D + 0.9F + 1.6H
0.9D + 0.9F + 0.9H

Notes

The retaining wall stability checks assume:
- No reinforcing
- Steel sheetpile Key of unknown embedment. It is unknown if this key continues for the entire length of the wall?
- 3 FT embedment (estimated)

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (lb/ft)	Stem V-allow (lb/ft)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)
1.4D + 1.4F	0	0	0	0	-0	3.16
1.2D + 1.2F + 1.6L + 1.6H	11.56	163.9	3200	43715	-0	3.16
1.2D + 1.2F + 1.6L + 0.9H	7.67	163.9	2033	43715	-0	3.16
1.2D + 1.2F + 0.5L + 1.6H	9.72	163.9	2833	43715	-0	3.16
1.2D + 1.2F + 0.5L + 0.9H	5.83	163.9	1667	43715	-0	3.16
1.2D + 1.2F + 1.6H	8.89	163.9	2667	43715	-0	3.16
1.2D + 1.2F + 0.9H	5	163.9	1500	43715	-0	3.16
0.9D + 1.6H	8.89	163.9	2667	43715	-0	3.16
0.9D + 0.9H	5	163.9	1500	43715	-0	3.16
0.9D + 0.9F + 1.6H	8.89	163.9	2667	43715	-0	3.16
0.9D + 0.9F + 0.9H	5	163.9	1500	43715	-0	3.16

Load Combination	Heel V-applied (lb/ft)	Heel V-allow (lb/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (lb/ft)	Toe V-allow (lb/ft)
1.4D + 1.4F	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6L + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6L + 0.9H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.5L + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.5L + 0.9H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.9H	0	6072	0	3.16	0	6072
0.9D + 1.6H	0	6072	0	3.16	0	6072
0.9D + 0.9H	0	6072	0	3.16	0	6072
0.9D + 0.9F + 1.6H	0	6072	0	3.16	0	6072
0.9D + 0.9F + 0.9H	0	6072	0	3.16	0	6072

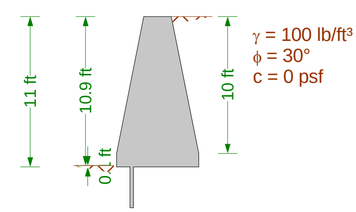
Stability Check Results Summary

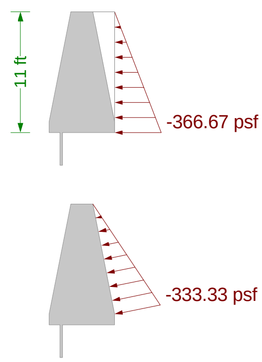
Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0F + 1.0L + 1.0H	9.41	23.89	2.539	1.500	1.500	198.6	329.9	1.661
1.0D + 1.0F + 1.0L + 0.6H	6.45	24.8	3.843	1.500	1.500	131.4	291.5	2.219
1.0D + 1.0F + 1.0H	7.39	23.89	3.231	1.500	1.500	168.1	329.9	1.963
1.0D + 1.0F + 0.6H	4.44	24.8	5.590	1.500	1.500	100.8	291.5	2.891
1.0D + 1.0F + 0.75L + 1.0H	8.91	23.89	2.682	1.500	1.500	191	329.9	1.727
1.0D + 1.0F + 0.75L + 0.6H	5.95	24.8	4.168	1.500	1.500	123.8	291.5	2.355
0.6D + 1.0H	7.39	13.43	1.816	1.500	1.500	168.1	236.3	1.406
0.6D + 0.6H	4.44	14.33	3.231	1.500	1.500	100.8	197.9	1.963
0.6D + 0.6F + 1.0H	7.39	13.43	1.816	1.500	1.500	168.1	236.3	1.406
0.6D + 0.6F + 0.6H	4.44	14.33	3.231	1.500	1.500	100.8	197.9	1.963

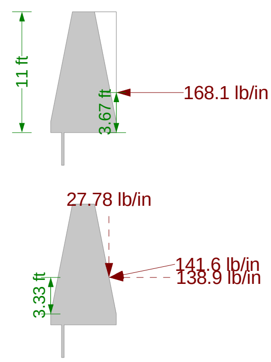
Stability Check Results Summary (continued)

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0F + 1.0L + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 1.0L + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.75L + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.75L + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
0.6D + 1.0H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6F + 1.0H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6F + 0.6H	1.500	1.500	1533	3000	10.94	12	0.28

Backfill Pressure







Lateral Earth Pressure

Rankine Active Earth Pressure Theory

$$K_a = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left[45° - \frac{(30°)}{2}\right] = 0.3333$$

$$\sigma_a = \gamma H K_a - 2c\sqrt{K_a} = (100\ lb/ft^3)(11\ ft)(0.3333) - 2(0\ psf)\sqrt{0.3333} = 366.7\ psf$$

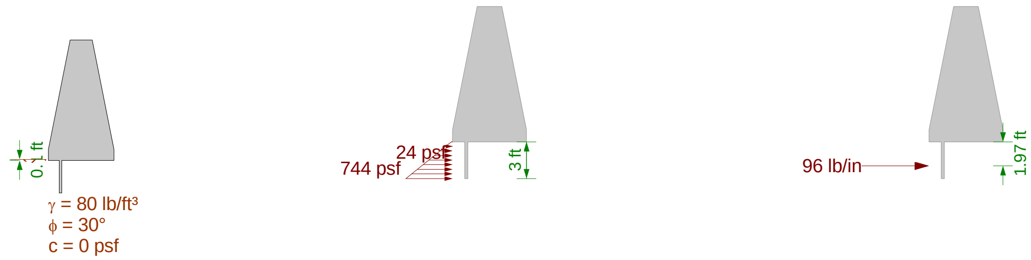
$\alpha_P = \alpha = (0°) = 0°$ (resultant force angle with horizontal)

Lateral Earth Pressure (stem only)

$$\sigma_a = \gamma H K_a - 2c\sqrt{K_a} = (100\ lb/ft^3)(10\ ft)(0.3333) - 2(0\ psf)\sqrt{0.3333} = 333.3\ psf$$

$\alpha_P = \alpha = (0°) = 0°$ (resultant force angle with horizontal)

Passive Pressure



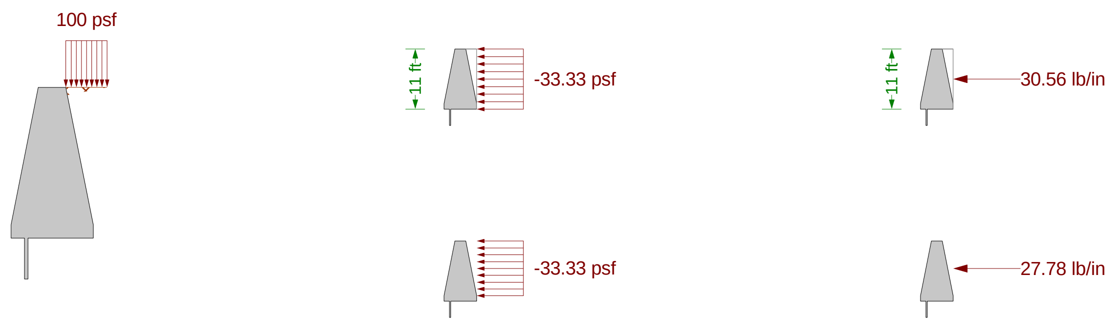
Lateral Earth Pressure

Rankine Passive Earth Pressure Theory

$K_p = \tan^2\left(45° + \frac{\phi}{2}\right) = \tan^2\left[45° + \frac{(30°)}{2}\right] = 3.0$

$\sigma_p = \gamma\, H\, K_p + 2\, c\, \sqrt{K_p} = (80\ \text{lb / ft}^3)(3.1\ \text{ft})(3.0) + 2\,(0\ \text{psf})\,\sqrt{3.0} = 744\ \text{psf}$

Uniform Surcharge Pressure



Lateral Surcharge Pressure

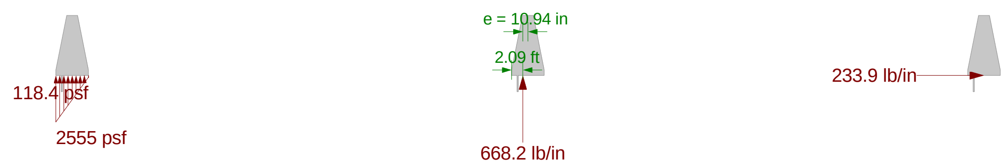
Rankine Active Earth Pressure Theory

$K_a = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left[45° - \frac{(30°)}{2}\right] = 0.3333$

$\sigma_{sur} = K_a\, q = (0.3333)(100\ \text{psf}) = 33.33\ \text{psf}$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ R = (0.350)(668.2 lb / in) = 233.9 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-168.06 lb/in	3.67 ft	88733 in·lb/ft
Uniform Surcharge Pressure	-0 lb/in	-	-30.56 lb/in	5.5 ft	24200 in·lb/ft
Footing Weight	-75 lb/in	3 ft	0 lb/in	-	-32400 in·lb/ft
Stem Weight	-250 lb/in	3 ft	0 lb/in	-	-108000 in·lb/ft
Stem Weight	-125 lb/in	1.33 ft	0 lb/in	-	-24000 in·lb/ft
Stem Weight	-125 lb/in	4.67 ft	0 lb/in	-	-84000 in·lb/ft
Key Weight	-9.38 lb/in	1.13 ft	0 lb/in	-	-1518.75 in·lb/ft
Backfill Weight	-0 lb/in	-	0 lb/in	-	-0 in·lb/ft
Backfill Weight	-83.33 lb/in	5.33 ft	0 lb/in	-	-64000 in·lb/ft
Soil over toe Weight	-0.54 lb/in	-0.05 ft	0 lb/in	-	3.5 in·lb/ft
	-668.25 lb/in				-200981.92 in·lb/ft

$$\frac{-\ 200981.92\ \text{in·lb / ft}}{-\ 668.25\ \text{lb / in}} = 2.09\ \text{ft}$$

Stability Checks [1.0D + 1.0F + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	168.1 lb/in	3.67 ft	88733 in·lb/ft
Surcharge (uniform) lateral pressure	30.56 lb/in	5.5 ft	24200 in·lb/ft
		Total:	112933 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Surcharge (uniform) vertical pressure	0 lb/in	6 ft	0 in·lb/ft
Passive pressure @ toe	96 lb/in	-1.97 ft	-27216 in·lb/ft
Footing Weight	-75 lb/in	3 ft	32400 in·lb/ft
Stem Weight	-250 lb/in	3 ft	108000 in·lb/ft
Stem Weight	-125 lb/in	1.33 ft	24000 in·lb/ft
Stem Weight	-125 lb/in	4.67 ft	84000 in·lb/ft
Key Weight	-9.38 lb/in	1.13 ft	1519 in·lb/ft
Backfill Weight	-0 lb/in	6 ft	0 in·lb/ft
Backfill Weight	-83.33 lb/in	5.33 ft	64000 in·lb/ft
Soil over toe Weight	-0.54 lb/in	-0.05 ft	-3.5 in·lb/ft
		Total:	286699 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{286699 \text{ in·lb / ft}}{112933 \text{ in·lb / ft}} = 2.539 > 1.50 \text{ (OK)}$$

Sliding Check

Sliding Force(s)

Backfill pressure	168.1 lb/in
Surcharge (uniform) lateral pressure	30.56 lb/in
Total:	198.6 lb/in

Resisting Force(s)

Passive pressure @ toe	96 lb/in
Friction	233.9 lb/in
Total:	329.9 lb/in

$$F.S. = \frac{RF}{SF} = \frac{329.9 \text{ lb / in}}{198.6 \text{ lb / in}} = 1.661 > 1.50 \text{ (OK)}$$

Bearing Capacity Check

Bearing pressure < allowable (2555 psf < 3000 psf) - OK
Bearing resultant eccentricity < allowable (10.94 in < 12 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.282 in
Total deflection at top of wall (positive towards toe)	0.282 in

Stability Checks [0.6D + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	168.1 lb/in	3.67 ft	88733 in·lb/ft
Surcharge (uniform) lateral pressure	0 lb/in	5.5 ft	0 in·lb/ft
		Total:	88733 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Surcharge (uniform) vertical pressure	0 lb/in	6 ft	0 in·lb/ft
Passive pressure @ toe	96 lb/in	-1.97 ft	-27216 in·lb/ft
Footing Weight	-45 lb/in	3 ft	19440 in·lb/ft
Stem Weight	-150 lb/in	3 ft	64800 in·lb/ft
Stem Weight	-75 lb/in	1.33 ft	14400 in·lb/ft
Stem Weight	-75 lb/in	4.67 ft	50400 in·lb/ft
Key Weight	-5.63 lb/in	1.13 ft	911.3 in·lb/ft
Backfill Weight	-0 lb/in	6 ft	0 in·lb/ft
Backfill Weight	-50 lb/in	5.33 ft	38400 in·lb/ft
Soil over toe Weight	-0.32 lb/in	-0.05 ft	-2.1 in·lb/ft
		Total:	161133 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{161133 \text{ in·lb / ft}}{88733 \text{ in·lb / ft}} = 1.816 > 1.50 \text{ (OK)}$$

Sliding Check

Sliding Force(s)

Backfill pressure	168.1 lb/in
Surcharge (uniform) lateral pressure	0 lb/in
Total:	168.1 lb/in

Resisting Force(s)

Passive pressure @ toe	96 lb/in
Friction	140.3 lb/in
Total:	236.3 lb/in

$$F.S. = \frac{RF}{SF} = \frac{236.3 \text{ lb / in}}{168.1 \text{ lb / in}} = 1.406 < 1.50 \text{ (FAILS)}$$

Bearing Capacity Check

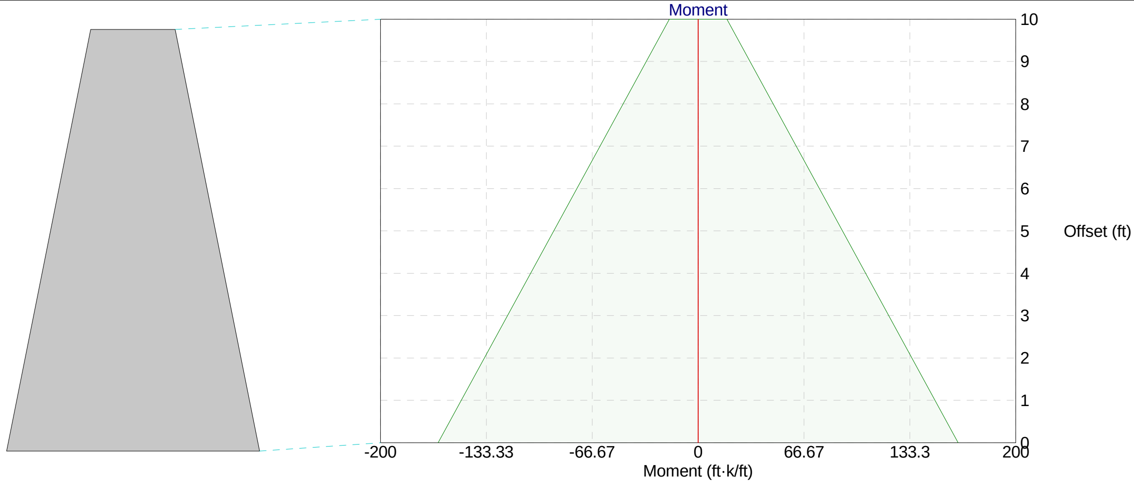
Bearing pressure < allowable (1533 psf < 3000 psf) - OK
Bearing resultant eccentricity < allowable (10.94 in < 12 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.282 in
Total deflection at top of wall (positive towards toe)	0.282 in

Stem Flexural Capacity



Capacity (ACI 318-11 10.2) @ 0 ft from base [Negative bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5\sqrt{F'_c}\,S = 5\sqrt{4000\text{ psi}}\,(10368\text{ in}^3/\text{ft}) = 273.2$ ft·k / ft (as limited by tension)

$M_n = 0.85\,F'_c\,S = 0.85\,(4000\text{ psi})(10368\text{ in}^3/\text{ft}) = 2938$ ft·k / ft (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\,M_n = (0.60)(273.2\text{ ft·k/ft}) = 163.9$ ft·k / ft

Capacity (ACI 318-11 10.2) @ 0 ft from base [Positive bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5\sqrt{F'_c}\,S = 5\sqrt{4000\text{ psi}}\,(10368\text{ in}^3/\text{ft}) = 273.2$ ft·k / ft (as limited by tension)

$M_n = 0.85\,F'_c\,S = 0.85\,(4000\text{ psi})(10368\text{ in}^3/\text{ft}) = 2938$ ft·k / ft (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\,M_n = (0.60)(273.2\text{ ft·k/ft}) = 163.9$ ft·k / ft

Capacity (ACI 318-11 10.2) @ 10 ft from base [Negative bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5\sqrt{F'_c}\,S = 5\sqrt{4000\text{ psi}}\,(1152\text{ in}^3/\text{ft}) = 30.36$ ft·k / ft (as limited by tension)

$M_n = 0.85\,F'_c\,S = 0.85\,(4000\text{ psi})(1152\text{ in}^3/\text{ft}) = 326.4$ ft·k / ft (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\,M_n = (0.60)(30.36\text{ ft·k/ft}) = 18.21$ ft·k / ft

Capacity (ACI 318-11 10.2) @ 10 ft from base [Positive bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5\sqrt{F'_c}\,S = 5\sqrt{4000\text{ psi}}\,(1152\text{ in}^3/\text{ft}) = 30.36$ ft·k / ft (as limited by tension)

$M_n = 0.85\,F'_c\,S = 0.85\,(4000\text{ psi})(1152\text{ in}^3/\text{ft}) = 326.4$ ft·k / ft (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\,M_n = (0.60)(30.36\text{ ft·k/ft}) = 18.21$ ft·k / ft

Stem Shear Capacity

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 0 ft from base [Positive shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(72\text{ in}) = 72859\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi\, V_n = (0.60)(72859\text{ lb / ft}) = 43715\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 0 ft from base [Negative shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(72\text{ in}) = 72859\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi\, V_n = (0.60)(72859\text{ lb / ft}) = 43715\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 10 ft from base [Positive shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(24\text{ in}) = 24286\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi\, V_n = (0.60)(24286\text{ lb / ft}) = 14572\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 10 ft from base [Negative shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(24\text{ in}) = 24286\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi\, V_n = (0.60)(24286\text{ lb / ft}) = 14572\text{ lb / ft}$

Toe Checks [1.4D + 1.4F]

Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3} (1.0) \sqrt{4000 \text{ psi}}\, (10 \text{ in}) = 10119 \text{ lb / ft}$

ϕ = 0.60

$\phi V_n = \phi V_n$ = (0.60) (10119 lb / ft) = 6072 lb / ft

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (200 \text{ in}^3 / \text{ft}) = 5.27 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85 F'_c S = 0.85 (4000 \text{ psi}) (200 \text{ in}^3 / \text{ft}) = 56.67 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi M_n$ = (0.60) (5.27 ft·k / ft) = 3.16 ft·k / ft

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Toe Unfactored Loads

Unfactored Loads

Toe Factored Loads

1.4D + 1.4F

Heel Checks [1.4D + 1.4F]

Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = - 0 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3}(1.0)\sqrt{4000 \text{ psi}}\,(10 \text{ in}) = 10119 \text{ lb / ft}$

ϕ = 0.60

$\phi V_n = \phi V_n$ = (0.60) (10119 lb / ft) = 6072 lb / ft

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\,(200 \text{ in}^3 / \text{ft}) = 5.27 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\,(4000 \text{ psi})\,(200 \text{ in}^3 / \text{ft}) = 56.67 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi M_n$ = (0.60) (5.27 ft·k / ft) = 3.16 ft·k / ft

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Heel Unfactored Loads

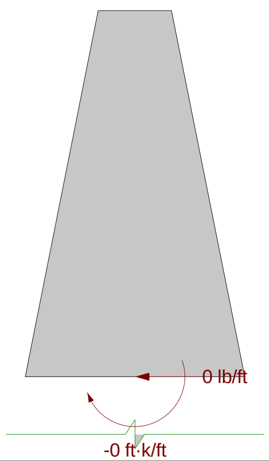
Unfactored Loads

Heel Factored Loads

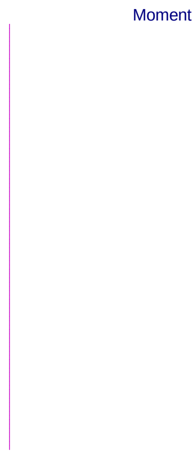
1.4D + 1.4F

Stem Forces [1.4D + 1.4F]

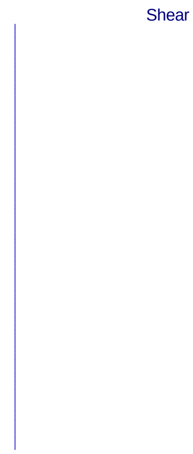
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

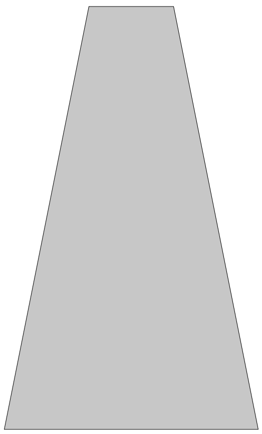


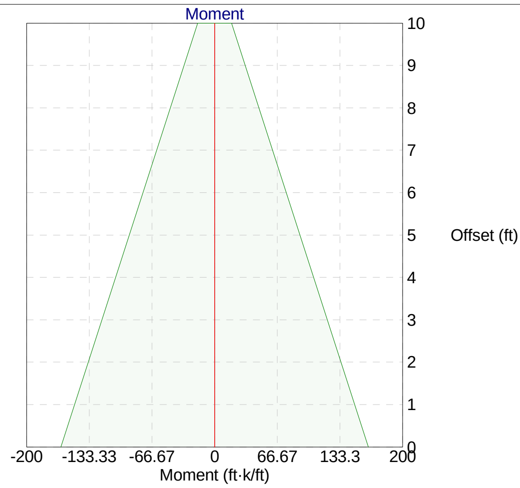
Stem Joint Force Transfer

Location	Force
@ stem base	0 lb/ft

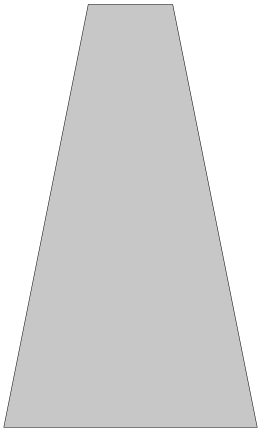
Stem Internal Forces

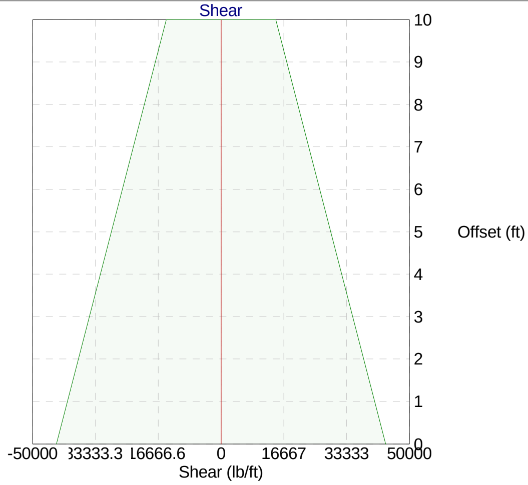
Stem Moment Checks [1.4D + 1.4F]





Stem Shear Checks [1.4D + 1.4F]





Stem Miscellaneous Checks [1.4D + 1.4F]

Minimum Steel Check (ACI 318-11 10.5.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 163.9 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (0 ft·k / ft) = 0 ft·k / ft

Check is waived per ACI 10.5.3 ✓

Maximum Steel Check (ACI 318-11 10.3.5) @ 0 ft from base [Stem in negative flexure]

β_1 = 0.850 (F'$_c$ ≤ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(72 \text{ in})}{(0 \text{ in}) / (0.850)} - 1\right] = \text{INF}$$

ε_t = INF ≥ 0.004 ✓

Toe Checks [1.2D + 1.2F + 1.6L + 1.6H]

Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0 ft·k / ft < M_{stem} = 11.56 ft·k / ft

M_u = 0 ft·k / ft (stem moment does not control)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3}(1.0)\sqrt{4000 \text{ psi}}\,(10 \text{ in}) = 10119 \text{ lb / ft}$

ϕ = 0.60

$\phi V_n = \phi\, V_n = (0.60)(10119 \text{ lb / ft}) = 6072 \text{ lb / ft}$

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5\sqrt{F'_c}\, S = 5\sqrt{4000 \text{ psi}}\,(200 \text{ in}^3 / \text{ft}) = 5.27 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\,(4000 \text{ psi})(200 \text{ in}^3 / \text{ft}) = 56.67 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi\, M_n = (0.60)(5.27 \text{ ft·k / ft}) = 3.16 \text{ ft·k / ft}$

ϕM_n = 3.16 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Toe Unfactored Loads

Unfactored Loads

Toe Factored Loads

1.2D + 1.2F + 1.6L + 1.6H

Heel Checks [1.2D + 1.2F + 1.6L + 1.6H]

Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = - 0 ft·k / ft < M_{stem} = 11.56 ft·k / ft

M_u = - 0 ft·k / ft (stem moment does not control)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3} (1.0) \sqrt{4000 \text{ psi}}\, (10 \text{ in}) = 10119 \text{ lb / ft}$

ϕ = 0.60

$\phi V_n = \phi\, V_n = (0.60)(10119 \text{ lb / ft}) = 6072 \text{ lb / ft}$

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (200 \text{ in}^3 / \text{ft}) = 5.27 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85 (4000 \text{ psi}) (200 \text{ in}^3 / \text{ft}) = 56.67 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi\, M_n = (0.60)(5.27 \text{ ft·k / ft}) = 3.16 \text{ ft·k / ft}$

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Heel Unfactored Loads

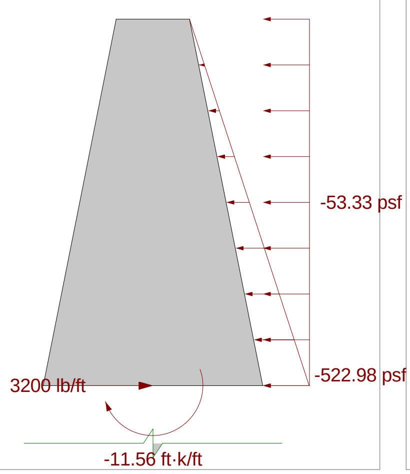
Unfactored Loads

Heel Factored Loads

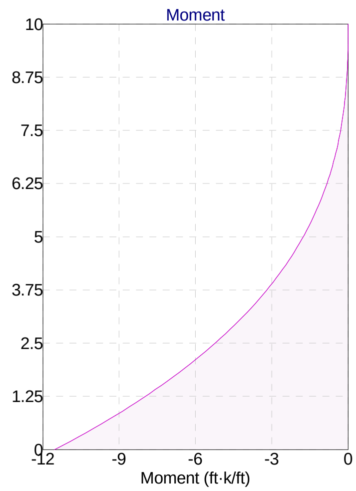
1.2D + 1.2F + 1.6L + 1.6H

Stem Forces [1.2D + 1.2F + 1.6L + 1.6H]

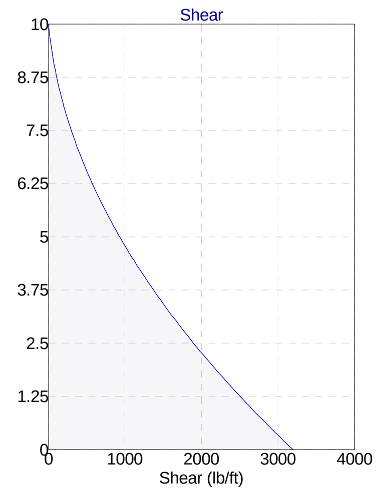
Stem Internal Forces



Stem Internal Forces



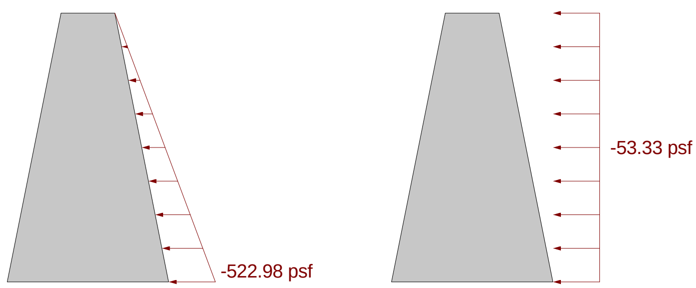
Stem Internal Forces



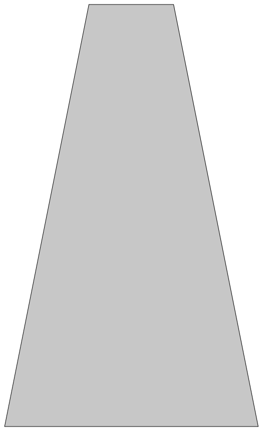
Stem Joint Force Transfer

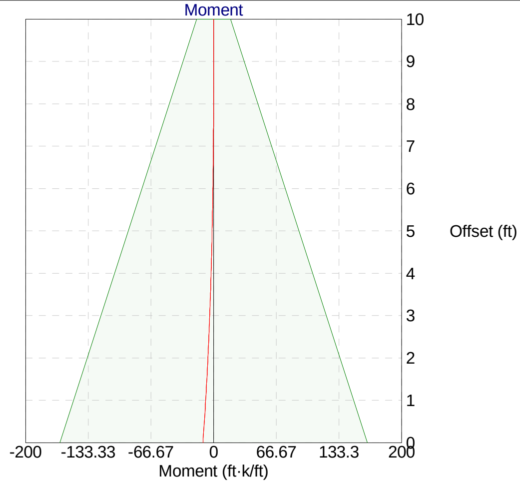
Location	Force
@ stem base	3200 lb/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.2F + 1.6L + 1.6H]

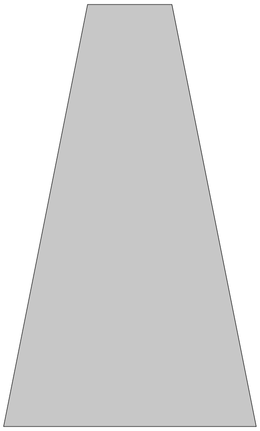


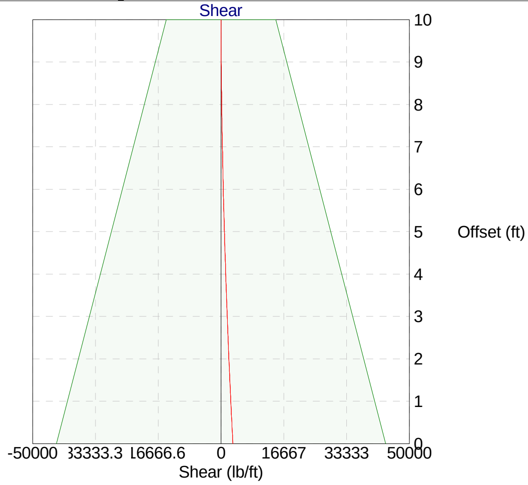


Check (ACI 318-11 Ch 10) @ 0 ft from base

ϕM_n = 163.9 ft·k / ft ≥ M_u = 11.56 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.2F + 1.6L + 1.6H]





Shear Check (ACI 318-11 Ch 11.1.1) @ 0 ft from base

ϕV_n = 43715 lb / ft ≥ V_u = 3200 lb / ft ✓

Stem Miscellaneous Checks [1.2D + 1.2F + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-11 10.5.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 163.9 ft·k / ft $\geq$ (4 / 3) M_u = [4 / 3] (11.56 ft·k / ft) = 15.41 ft·k / ft

Check is waived per ACI 10.5.3 ✓

Maximum Steel Check (ACI 318-11 10.3.5) @ 0 ft from base [Stem in negative flexure]

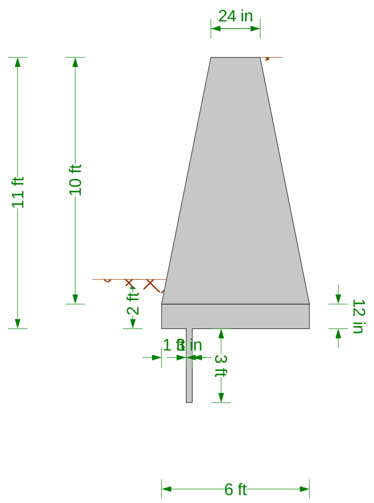
β_1 = 0.850 (F'$_c$ ≤ 4000 psi)

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0 \text{ in}$$

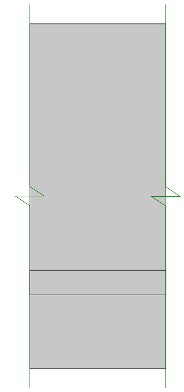
$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1 \right) = 0.003 \left[\frac{(72 \text{ in})}{(0 \text{ in}) / (0.850)} - 1 \right] = \text{INF}$$

ε_t = INF $\geq$ 0.004 ✓

Design Detail



Concrete f'c = 4000 psi
Rebar Fy = 60000 psi
Unit Weight = 150 lb/ft³



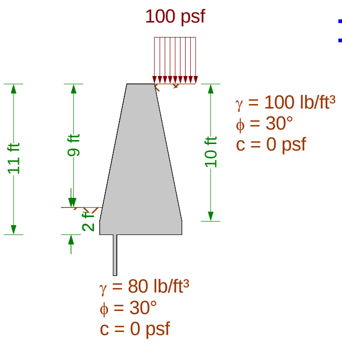
Check Summary

Ratio	Check	Provided	Required	Combination
----- Stability Checks -----				
✓ 0.975	Overturning	1.54	1.50	0.6D + 1.0H
✓ 0.720	Sliding	2.08	1.50	0.6D + 1.0H
✓ 0.852	Bearing Pressure	3000 psf	2555 psf	1.0D + 1.0F + 1.0L + 1.0H
✓ 0.912	Bearing Eccentricity	10.94 in	12 in	1.0D + 1.0F + 1.0L + 1.0H
----- Toe Checks -----				
✓ 0.000	Shear	6072 lb/ft	0 lb/ft	1.4D + 1.4F
✓ -0.000	Moment	3.16 ft·k/ft	-0 ft·k/ft	1.4D + 1.4F
----- Heel Checks -----				
✓ 0.000	Shear	6072 lb/ft	0 lb/ft	1.4D + 1.4F
✓ -0.000	Moment	3.16 ft·k/ft	-0 ft·k/ft	1.4D + 1.4F
----- Stem Checks -----				
✓ 0.070	Moment	163.9 ft·k/ft	11.56 ft·k/ft	1.2D + 1.2F + 1.6L + 1.6H
✓ 0.073	Shear	43715 lb/ft	3200 lb/ft	1.2D + 1.2F + 1.6L + 1.6H

Criteria

Use basic criteria from common project settings	Yes
Building Code	IBC 2012
Concrete Load Combs	IBC 2012 (Strength)
Masonry Load Combs	ASCE 7-10 (ASD)
Stability Load Combs	ASCE 7-10 (ASD)
Apply Sds Factor to Seismic Combinations for Ev	No
Restrained Against Sliding	No
Neglect Bearing At Heel	Yes
Use Vert. Comp. for OT	No
Use Vert. Comp. for Sliding	No
Use Vert. Comp. for Bearing	Yes
Use Surcharge for Sliding & OT	Yes
Use Surcharge for Bearing	Yes
Neglect Soil Over Toe	No
Neglect Backfill Wt. for Coulomb	No
Factor Soil Weight As Dead	Yes
Use Passive Force for OT	Yes
Assume Pressure To Top	Yes
Extend Backfill Pressure To Key Bottom	No
Use Toe Passive Pressure for Bearing	No
Required F.S. for OT	1.50
Required F.S. for Sliding	1.50
Has Different Safety Factors for Seismic	No
Allowable Bearing Pressure	3000 psf
Req'd Bearing Location	Middle third
Wall Friction Angle	25°
Friction Coefficent	0.35
Soil Reaction Modulus	172800 lb/ft³

Loads



Loading Options/Assumptions

- Passive pressure neglects top 0 ft of soil.
- Passive pressure is applied only to key.

Load Combinations

IBC 2012 (Strength)

1.4D + 1.4F
1.2D + 1.2F + 1.6L + 1.6H
1.2D + 1.2F + 1.6L + 0.9H
1.2D + 1.2F + 0.5L + 1.6H
1.2D + 1.2F + 0.5L + 0.9H
1.2D + 1.2F + 1.6H
1.2D + 1.2F + 0.9H
0.9D + 1.6H
0.9D + 0.9H
0.9D + 0.9F + 1.6H
0.9D + 0.9F + 0.9H

Notes

The retaining wall stability checks assume:
- No reinforcing
- Steel sheetpile Key of unknown embedment. It is unknown if this key continues for the entire length of the wall?
- 3 FT embedment (estimated)

Strength Check Results Summary

Load Combination	Stem M-applied (ft·k/ft)	Stem M-allow (ft·k/ft)	Stem V-applied (lb/ft)	Stem V-allow (lb/ft)	Heel M-applied (ft·k/ft)	Heel M-allow (ft·k/ft)
1.4D + 1.4F	0	0	0	0	-0	3.16
1.2D + 1.2F + 1.6L + 1.6H	11.56	163.9	3200	43715	-0	3.16
1.2D + 1.2F + 1.6L + 0.9H	7.67	163.9	2033	43715	-0	3.16
1.2D + 1.2F + 0.5L + 1.6H	9.72	163.9	2833	43715	-0	3.16
1.2D + 1.2F + 0.5L + 0.9H	5.83	163.9	1667	43715	-0	3.16
1.2D + 1.2F + 1.6H	8.89	163.9	2667	43715	-0	3.16
1.2D + 1.2F + 0.9H	5	163.9	1500	43715	-0	3.16
0.9D + 1.6H	8.89	163.9	2667	43715	-0	3.16
0.9D + 0.9H	5	163.9	1500	43715	-0	3.16
0.9D + 0.9F + 1.6H	8.89	163.9	2667	43715	-0	3.16
0.9D + 0.9F + 0.9H	5	163.9	1500	43715	-0	3.16

Load Combination	Heel V-applied (lb/ft)	Heel V-allow (lb/ft)	Toe M-applied (ft·k/ft)	Toe M-allow (ft·k/ft)	Toe V-applied (lb/ft)	Toe V-allow (lb/ft)
1.4D + 1.4F	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6L + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6L + 0.9H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.5L + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.5L + 0.9H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 1.6H	0	6072	0	3.16	0	6072
1.2D + 1.2F + 0.9H	0	6072	0	3.16	0	6072
0.9D + 1.6H	0	6072	0	3.16	0	6072
0.9D + 0.9H	0	6072	0	3.16	0	6072
0.9D + 0.9F + 1.6H	0	6072	0	3.16	0	6072
0.9D + 0.9F + 0.9H	0	6072	0	3.16	0	6072

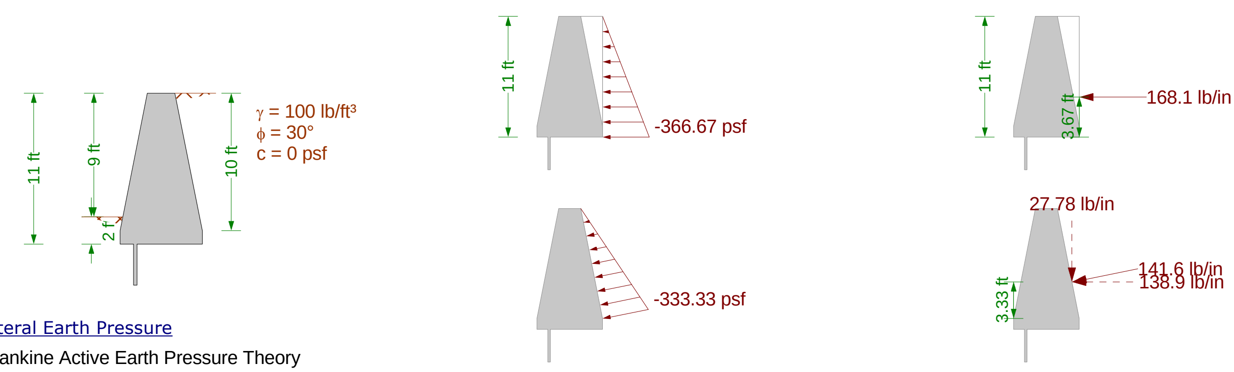
Stability Check Results Summary

Load Combination	Overturning Moment (ft·k/ft)	Resisting Moment (ft·k/ft)	Overturning F.S.	Overturning F.S. Req'd	Overturning F.S. Req'd Seismic	Sliding Force (lb/in)	Resisting Force (lb/in)	Sliding F.S.
1.0D + 1.0F + 1.0L + 1.0H	9.41	21.84	2.321	1.500	1.500	198.6	443.9	2.235
1.0D + 1.0F + 1.0L + 0.6H	6.45	23.57	3.652	1.500	1.500	131.4	359.9	2.739
1.0D + 1.0F + 1.0H	7.39	21.84	2.954	1.500	1.500	168.1	443.9	2.642
1.0D + 1.0F + 0.6H	4.44	23.57	5.312	1.500	1.500	100.8	359.9	3.570
1.0D + 1.0F + 0.75L + 1.0H	8.91	21.84	2.452	1.500	1.500	191	443.9	2.325
1.0D + 1.0F + 0.75L + 0.6H	5.95	23.57	3.962	1.500	1.500	123.8	359.9	2.909
0.6D + 1.0H	7.39	11.38	1.538	1.500	1.500	168.1	350.4	2.085
0.6D + 0.6H	4.44	13.1	2.954	1.500	1.500	100.8	266.4	2.642
0.6D + 0.6F + 1.0H	7.39	11.38	1.538	1.500	1.500	168.1	350.4	2.085
0.6D + 0.6F + 0.6H	4.44	13.1	2.954	1.500	1.500	100.8	266.4	2.642

Stability Check Results Summary (continued)

Load Combination	Sliding F.S. Req'd	Sliding F.S. Req'd Seismic	Bearing Pressure Actual (psf)	Bearing Pressure Allowable (psf)	Bearing Eccentricity Actual (in)	Bearing Eccentricity Allowable (in)	Wall Top Actual Deflection (in)
1.0D + 1.0F + 1.0L + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 1.0L + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.75L + 1.0H	1.500	1.500	2555	3000	10.94	12	0.28
1.0D + 1.0F + 0.75L + 0.6H	1.500	1.500	2555	3000	10.94	12	0.28
0.6D + 1.0H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6F + 1.0H	1.500	1.500	1533	3000	10.94	12	0.28
0.6D + 0.6F + 0.6H	1.500	1.500	1533	3000	10.94	12	0.28

Backfill Pressure



Lateral Earth Pressure

Rankine Active Earth Pressure Theory

$K_a = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left[45° - \frac{(30°)}{2}\right] = 0.3333$

$\sigma_a = \gamma H K_a - 2c\sqrt{K_a} = (100\ lb / ft^3)(11\ ft)(0.3333) - 2(0\ psf)\sqrt{0.3333} = 366.7\ psf$

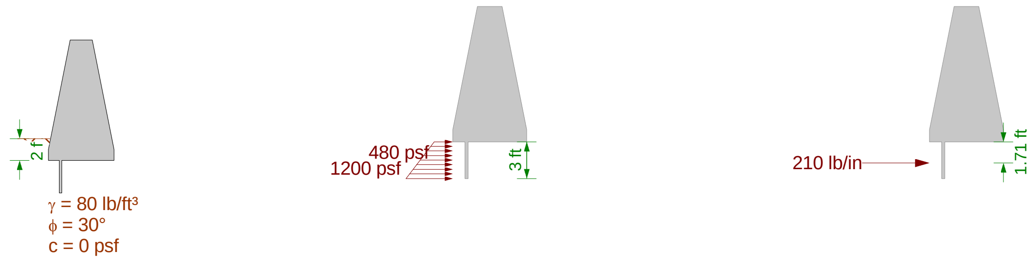
$\alpha_P = \alpha = (0°) = 0°$ (resultant force angle with horizontal)

Lateral Earth Pressure (stem only)

$\sigma_a = \gamma H K_a - 2c\sqrt{K_a} = (100\ lb / ft^3)(10\ ft)(0.3333) - 2(0\ psf)\sqrt{0.3333} = 333.3\ psf$

$\alpha_P = \alpha = (0°) = 0°$ (resultant force angle with horizontal)

Passive Pressure



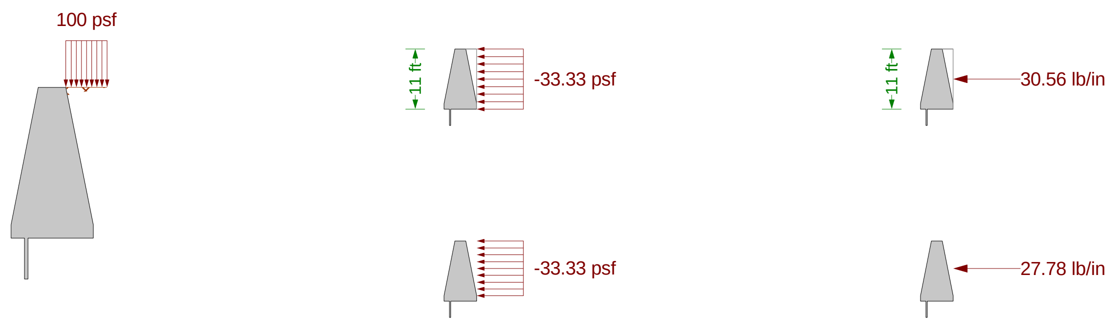
Lateral Earth Pressure

Rankine Passive Earth Pressure Theory

$K_p = \tan^2\left(45° + \frac{\phi}{2}\right) = \tan^2\left[45° + \frac{(30°)}{2}\right] = 3.0$

$\sigma_p = \gamma\, H\, K_p + 2\, c\, \sqrt{K_p} = (80\ \text{lb}/\text{ft}^3)(5\ \text{ft})(3.0) + 2\,(0\ \text{psf})\sqrt{3.0} = 1200\ \text{psf}$

Uniform Surcharge Pressure



Lateral Surcharge Pressure

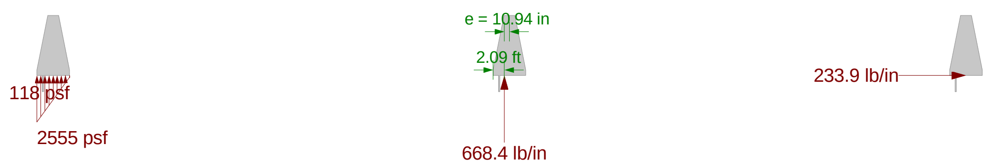
Rankine Active Earth Pressure Theory

$K_a = \tan^2\left(45° - \frac{\phi}{2}\right) = \tan^2\left[45° - \frac{(30°)}{2}\right] = 0.3333$

$\sigma_{sur} = K_a\, q = (0.3333)(100\ \text{psf}) = 33.33\ \text{psf}$

Wall/Soil Weights

Bearing Pressure



Friction

F = μ R = (0.350) (668.4 lb / in) = 233.9 lb / in

Bearing Pressure Calculation

Contributing Forces

	Vert Force	...offset	Horz Force	...offset	OT Moment
Backfill Pressure	-0 lb/in	-	-168.06 lb/in	3.67 ft	88733 in·lb/ft
Uniform Surcharge Pressure	-0 lb/in	-	-30.56 lb/in	5.5 ft	24200 in·lb/ft
Footing Weight	-75 lb/in	3 ft	0 lb/in	-	-32400 in·lb/ft
Stem Weight	-250 lb/in	3 ft	0 lb/in	-	-108000 in·lb/ft
Stem Weight	-125 lb/in	1.33 ft	0 lb/in	-	-24000 in·lb/ft
Stem Weight	-125 lb/in	4.67 ft	0 lb/in	-	-84000 in·lb/ft
Key Weight	-9.38 lb/in	1.13 ft	0 lb/in	-	-1518.75 in·lb/ft
Backfill Weight	-0 lb/in	-	0 lb/in	-	-0 in·lb/ft
Backfill Weight	-83.33 lb/in	5.33 ft	0 lb/in	-	-64000 in·lb/ft
Soil over toe Weight	-0.67 lb/in	0.05 ft	0 lb/in	-	-4.8 in·lb/ft
	-668.38 lb/in				-200990.22 in·lb/ft

$$\frac{-\ 200990.22\ \text{in·lb / ft}}{-\ 668.38\ \text{lb / in}} = 2.09\ \text{ft}$$

Stability Checks [1.0D + 1.0F + 1.0L + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	168.1 lb/in	3.67 ft	88733 in·lb/ft
Surcharge (uniform) lateral pressure	30.56 lb/in	5.5 ft	24200 in·lb/ft
		Total:	112933 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Surcharge (uniform) vertical pressure	0 lb/in	6 ft	0 in·lb/ft
Passive pressure @ toe	210 lb/in	-1.71 ft	-51840 in·lb/ft
Footing Weight	-75 lb/in	3 ft	32400 in·lb/ft
Stem Weight	-250 lb/in	3 ft	108000 in·lb/ft
Stem Weight	-125 lb/in	1.33 ft	24000 in·lb/ft
Stem Weight	-125 lb/in	4.67 ft	84000 in·lb/ft
Key Weight	-9.38 lb/in	1.13 ft	1519 in·lb/ft
Backfill Weight	-0 lb/in	6 ft	0 in·lb/ft
Backfill Weight	-83.33 lb/in	5.33 ft	64000 in·lb/ft
Soil over toe Weight	-0.67 lb/in	0.05 ft	4.8 in·lb/ft
		Total:	262084 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{262084 \text{ in·lb / ft}}{112933 \text{ in·lb / ft}} = 2.321 > 1.50 \text{ (OK)}$$

Sliding Check

Sliding Force(s)

Backfill pressure	168.1 lb/in
Surcharge (uniform) lateral pressure	30.56 lb/in
Total:	198.6 lb/in

Resisting Force(s)

Passive pressure @ toe	210 lb/in
Friction	233.9 lb/in
Total:	443.9 lb/in

$$F.S. = \frac{RF}{SF} = \frac{443.9 \text{ lb / in}}{198.6 \text{ lb / in}} = 2.235 > 1.50 \text{ (OK)}$$

Bearing Capacity Check

Bearing pressure < allowable (2555 psf < 3000 psf) - OK
Bearing resultant eccentricity < allowable (10.94 in < 12 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.282 in
Total deflection at top of wall (positive towards toe)	0.282 in

Stability Checks [0.6D + 1.0H]

Overturning Check

Overturning Moments

	Force	Distance	Moment
Backfill pressure (horz)	168.1 lb/in	3.67 ft	88733 in·lb/ft
Surcharge (uniform) lateral pressure	0 lb/in	5.5 ft	0 in·lb/ft
		Total:	88733 in·lb/ft

Resisting Moments

	Force	Distance	Moment
Surcharge (uniform) vertical pressure	0 lb/in	6 ft	0 in·lb/ft
Passive pressure @ toe	210 lb/in	-1.71 ft	-51840 in·lb/ft
Footing Weight	-45 lb/in	3 ft	19440 in·lb/ft
Stem Weight	-150 lb/in	3 ft	64800 in·lb/ft
Stem Weight	-75 lb/in	1.33 ft	14400 in·lb/ft
Stem Weight	-75 lb/in	4.67 ft	50400 in·lb/ft
Key Weight	-5.63 lb/in	1.13 ft	911.3 in·lb/ft
Backfill Weight	-0 lb/in	6 ft	0 in·lb/ft
Backfill Weight	-50 lb/in	5.33 ft	38400 in·lb/ft
Soil over toe Weight	-0.4 lb/in	0.05 ft	2.88 in·lb/ft
		Total:	136514 in·lb/ft

$$F.S. = \frac{RM}{OTM} = \frac{136514 \text{ in·lb / ft}}{88733 \text{ in·lb / ft}} = 1.538 > 1.50 \text{ (OK)}$$

Sliding Check

Sliding Force(s)

Backfill pressure	168.1 lb/in
Surcharge (uniform) lateral pressure	0 lb/in
Total:	168.1 lb/in

Resisting Force(s)

Passive pressure @ toe	210 lb/in
Friction	140.4 lb/in
Total:	350.4 lb/in

$$F.S. = \frac{RF}{SF} = \frac{350.4 \text{ lb / in}}{168.1 \text{ lb / in}} = 2.085 > 1.50 \text{ (OK)}$$

Bearing Capacity Check

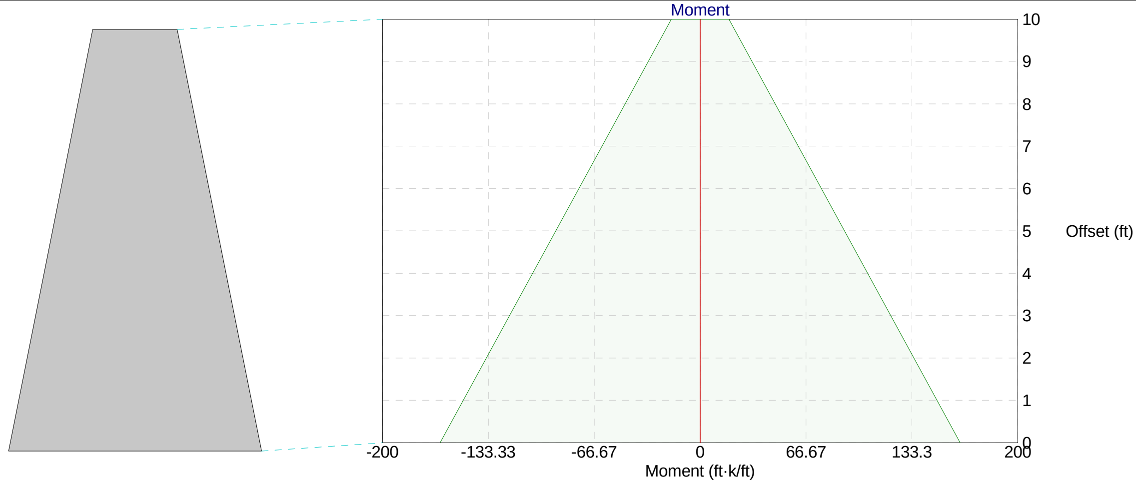
Bearing pressure < allowable (1533 psf < 3000 psf) - OK
Bearing resultant eccentricity < allowable (10.94 in < 12 in) - OK

Wall Top Displacement

(based on unfactored service loads)

Deflection due to stem flexural displacement	0 in
Deflection due to rotation from settlement	0.282 in
Total deflection at top of wall (positive towards toe)	0.282 in

Stem Flexural Capacity



Capacity (ACI 318-11 10.2) @ 0 ft from base [Negative bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (10368 \text{ in}^3 / \text{ft}) = 273.2 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85 F'_c\, S = 0.85\, (4000 \text{ psi})\, (10368 \text{ in}^3 / \text{ft}) = 2938 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\, M_n = (0.60)\, (273.2 \text{ ft·k / ft}) = 163.9 \text{ ft·k / ft}$

Capacity (ACI 318-11 10.2) @ 0 ft from base [Positive bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (10368 \text{ in}^3 / \text{ft}) = 273.2 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85 F'_c\, S = 0.85\, (4000 \text{ psi})\, (10368 \text{ in}^3 / \text{ft}) = 2938 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\, M_n = (0.60)\, (273.2 \text{ ft·k / ft}) = 163.9 \text{ ft·k / ft}$

Capacity (ACI 318-11 10.2) @ 10 ft from base [Negative bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (1152 \text{ in}^3 / \text{ft}) = 30.36 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85 F'_c\, S = 0.85\, (4000 \text{ psi})\, (1152 \text{ in}^3 / \text{ft}) = 326.4 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\, M_n = (0.60)\, (30.36 \text{ ft·k / ft}) = 18.21 \text{ ft·k / ft}$

Capacity (ACI 318-11 10.2) @ 10 ft from base [Positive bending]

Unreinforced, use plain concrete provisions: ACI 22.5.1

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (1152 \text{ in}^3 / \text{ft}) = 30.36 \text{ ft·k / ft}$ (as limited by tension)

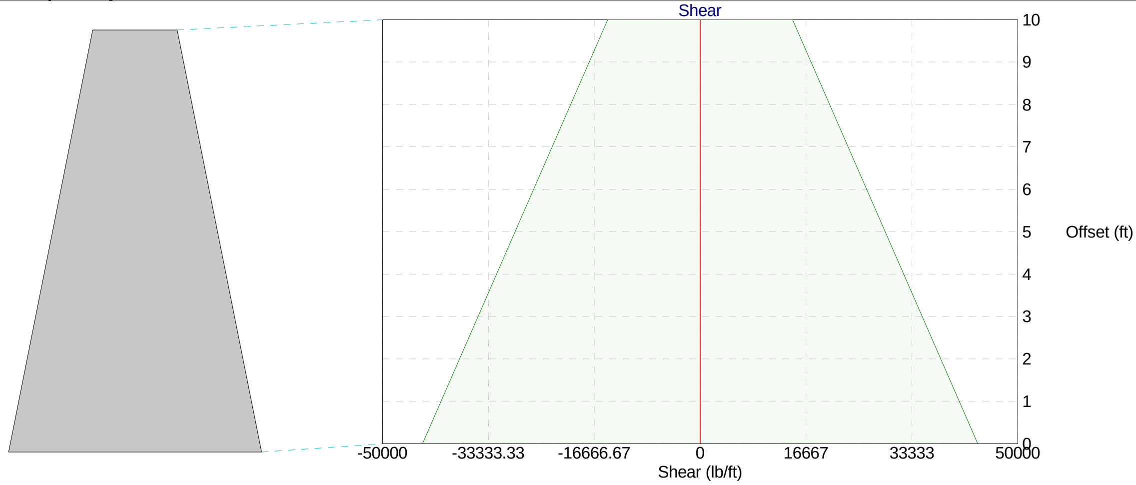
$M_n = 0.85 F'_c\, S = 0.85\, (4000 \text{ psi})\, (1152 \text{ in}^3 / \text{ft}) = 326.4 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

$\phi = 0.60$

$\phi M_n = \phi\, M_n = (0.60)\, (30.36 \text{ ft·k / ft}) = 18.21 \text{ ft·k / ft}$

Stem Shear Capacity



Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 0 ft from base [Positive shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(72\text{ in}) = 72859\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi V_n = (0.60)(72859\text{ lb / ft}) = 43715\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 0 ft from base [Negative shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(72\text{ in}) = 72859\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi V_n = (0.60)(72859\text{ lb / ft}) = 43715\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 10 ft from base [Positive shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(24\text{ in}) = 24286\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi V_n = (0.60)(24286\text{ lb / ft}) = 14572\text{ lb / ft}$

Shear Capacity (ACI 318-11 11.1.1, 11.2.1) @ 10 ft from base [Negative shear]

Unreinforced, use plain concrete provisions: ACI 22.5.4

$V_n = \frac{4}{3}\lambda\sqrt{F'_c}\,h = \frac{4}{3}(1.0)\sqrt{4000\text{ psi}}\,(24\text{ in}) = 24286\text{ lb / ft}$

$\phi = 0.60$

$\phi V_n = \phi V_n = (0.60)(24286\text{ lb / ft}) = 14572\text{ lb / ft}$

Toe Checks [1.4D + 1.4F]

Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3} (1.0) \sqrt{4000 \text{ psi}}\, (10 \text{ in}) = 10119$ lb / ft

ϕ = 0.60

$\phi V_n = \phi V_n$ = (0.60) (10119 lb / ft) = 6072 lb / ft

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\, (200 \text{ in}^3 / \text{ft}) = 5.27$ ft·k / ft (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\, (4000 \text{ psi})\, (200 \text{ in}^3 / \text{ft}) = 56.67$ ft·k / ft (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi M_n$ = (0.60) (5.27 ft·k / ft) = 3.16 ft·k / ft

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Toe Unfactored Loads

Unfactored Loads

Toe Factored Loads

1.4D + 1.4F

Heel Checks [1.4D + 1.4F]

Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = - 0 ft·k / ft ≥ M_{stem} = - 0 ft·k / ft

M_u = - 0 ft·k / ft (stem base moment controls)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3}(1.0)\sqrt{4000 \text{ psi}}\,(10 \text{ in}) = 10119$ lb / ft

ϕ = 0.60

$\phi V_n = \phi\, V_n = (0.60)(10119 \text{ lb / ft}) = 6072$ lb / ft

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5 \sqrt{F'_c}\, S = 5 \sqrt{4000 \text{ psi}}\,(200 \text{ in}^3 / \text{ft}) = 5.27$ ft·k / ft (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\,(4000 \text{ psi})(200 \text{ in}^3 / \text{ft}) = 56.67$ ft·k / ft (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi\, M_n = (0.60)(5.27 \text{ ft·k / ft}) = 3.16$ ft·k / ft

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Heel Unfactored Loads

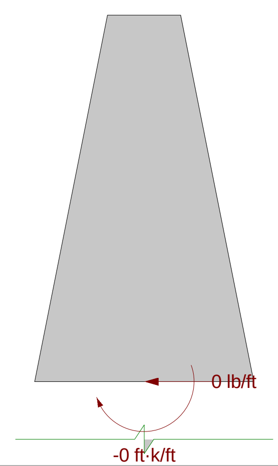
Unfactored Loads

Heel Factored Loads

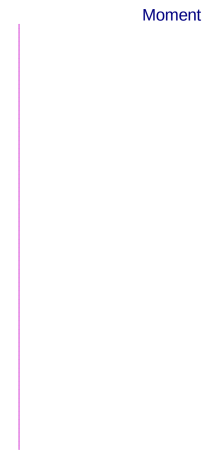
1.4D + 1.4F

Stem Forces [1.4D + 1.4F]

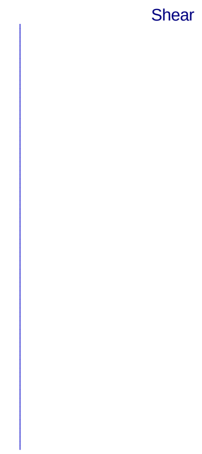
Stem Internal Forces



Stem Internal Forces



Stem Internal Forces

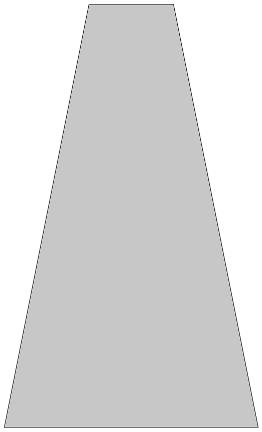


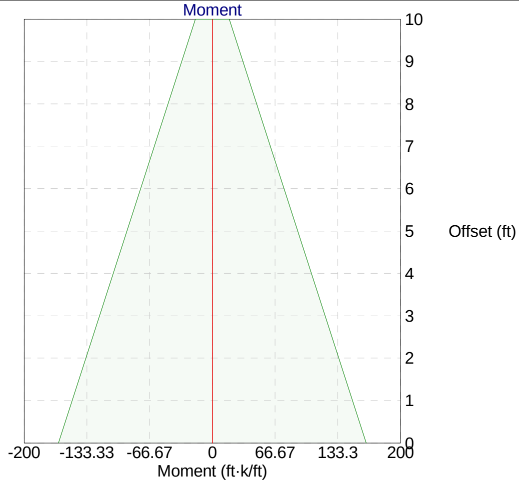
Stem Joint Force Transfer

Location	**Force**
@ stem base	0 lb/ft

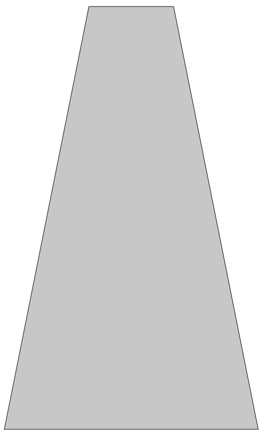
Stem Internal Forces

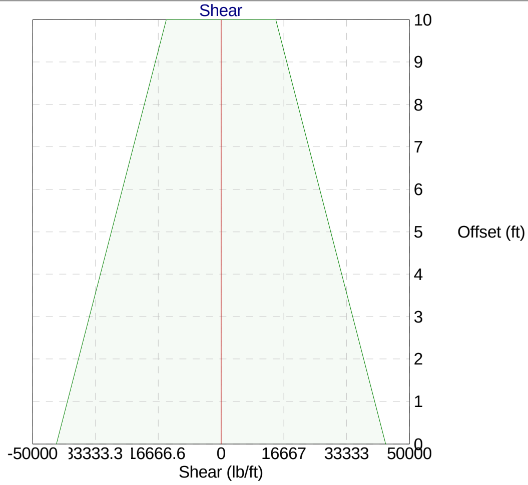
Stem Moment Checks [1.4D + 1.4F]





Stem Shear Checks [1.4D + 1.4F]





Stem Miscellaneous Checks [1.4D + 1.4F]

Minimum Steel Check (ACI 318-11 10.5.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 163.9 ft·k / ft ≥ $(4/3)\,M_u$ = $[4/3](0$ ft·k / ft$)$ = 0 ft·k / ft

Check is waived per ACI 10.5.3 ✓

Maximum Steel Check (ACI 318-11 10.3.5) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850 \qquad (F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s\, f_y}{0.85\, F'_c} = \frac{(0 \text{ in}^2/\text{in})(60000 \text{ psi})}{0.85\,(4000 \text{ psi})} = 0 \text{ in}$$

$$\varepsilon_t = 0.003\left(\frac{d}{a/\beta_1} - 1\right) = 0.003\left[\frac{(72 \text{ in})}{(0 \text{ in})/(0.850)} - 1\right] = \text{INF}$$

ε_t = INF ≥ 0.004 ✓

Toe Checks [1.2D + 1.2F + 1.6L + 1.6H]

Controlling Moment

Design moment M_u for toe need not exceed moment at stem base:

M_{toe} = 0 ft·k / ft < M_{stem} = 11.56 ft·k / ft

M_u = 0 ft·k / ft (stem moment does not control)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3}(1.0)\sqrt{4000 \text{ psi}}\,(10 \text{ in}) = 10119 \text{ lb / ft}$

ϕ = 0.60

$\phi V_n = \phi\, V_n = (0.60)(10119 \text{ lb / ft}) = 6072 \text{ lb / ft}$

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5\sqrt{F'_c}\, S = 5\sqrt{4000 \text{ psi}}\,(200 \text{ in}^3 / \text{ft}) = 5.27 \text{ ft·k / ft}$ (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\,(4000 \text{ psi})(200 \text{ in}^3 / \text{ft}) = 56.67 \text{ ft·k / ft}$ (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi\, M_n = (0.60)(5.27 \text{ ft·k / ft}) = 3.16 \text{ ft·k / ft}$

ϕM_n = 3.16 ft·k / ft ≥ M_u = 0 ft·k / ft ✓

Toe Unfactored Loads

Unfactored Loads

Toe Factored Loads

1.2D + 1.2F + 1.6L + 1.6H

Heel Checks [1.2D + 1.2F + 1.6L + 1.6H]

Controlling Moment

Design moment M_u for heel need not exceed moment at stem base:

M_{heel} = - 0 ft·k / ft < M_{stem} = 11.56 ft·k / ft

M_u = - 0 ft·k / ft (stem moment does not control)

Shear Check (ACI 318-11 11.1.1, 11.11.3.1)

λ = 1.0 (normal weight concrete)

Unreinforced, use plain concrete provisions: ACI 22.5.4

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$V_n = \frac{4}{3} \lambda \sqrt{F'_c}\, h = \frac{4}{3}(1.0)\sqrt{4000 \text{ psi}}\,(10 \text{ in}) = 10119$ lb / ft

ϕ = 0.60

$\phi V_n = \phi\, V_n$ = (0.60)(10119 lb / ft) = 6072 lb / ft

ϕV_n = 6072 lb / ft ≥ V_u = 0 lb / ft ✓

Flexure Check (ACI 318-11 10.2)

Unreinforced, use plain concrete provisions: ACI 22.5.1

Note: Effective thickness reduced by 2 inches for concrete cast on soil (ACI 22.4.8)

$M_n = 5\sqrt{F'_c}\, S = 5\sqrt{4000 \text{ psi}}\,(200 \text{ in}^3 / \text{ft}) = 5.27$ ft·k / ft (as limited by tension)

$M_n = 0.85\, F'_c\, S = 0.85\,(4000 \text{ psi})(200 \text{ in}^3 / \text{ft}) = 56.67$ ft·k / ft (as limited by compression)

Tension controls

ϕ = 0.60

$\phi M_n = \phi\, M_n$ = (0.60)(5.27 ft·k / ft) = 3.16 ft·k / ft

ϕM_n = 3.16 ft·k / ft ≥ M_u = - 0 ft·k / ft ✓

Heel Unfactored Loads

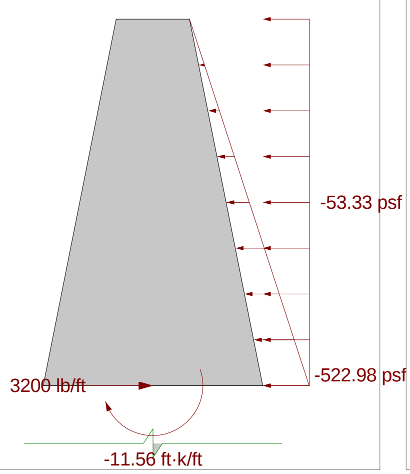
Unfactored Loads

Heel Factored Loads

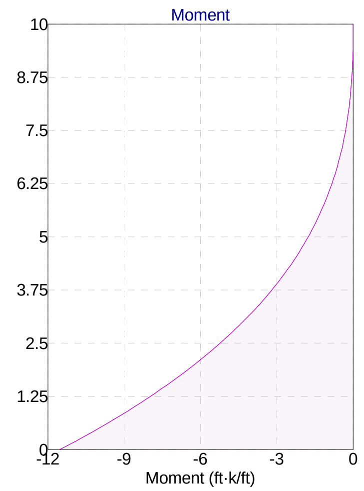
1.2D + 1.2F + 1.6L + 1.6H

Stem Forces [1.2D + 1.2F + 1.6L + 1.6H]

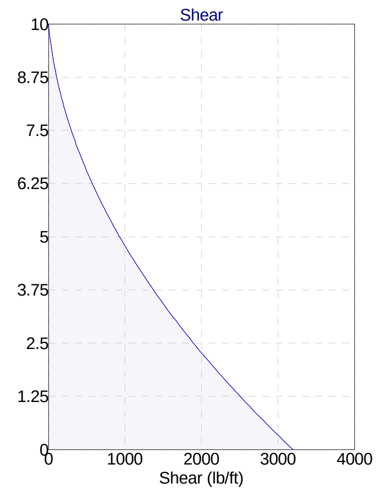
Stem Internal Forces



Stem Internal Forces



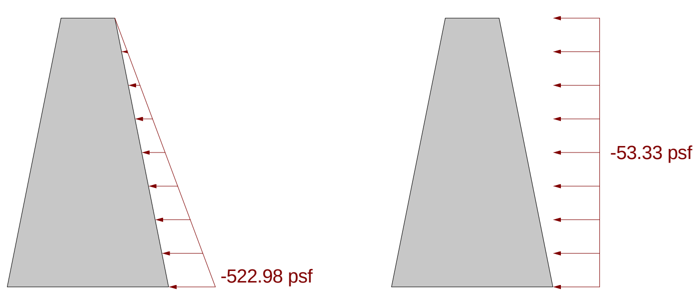
Stem Internal Forces



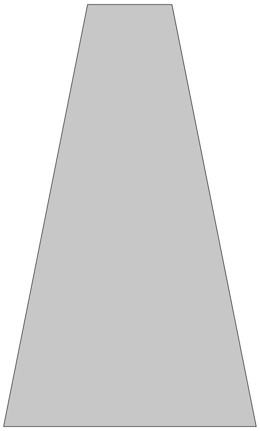
Stem Joint Force Transfer

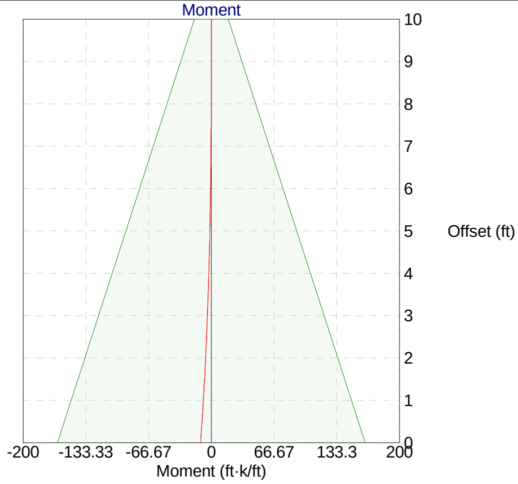
Location	**Force**
@ stem base	3200 lb/ft

Stem Internal Forces



Stem Moment Checks [1.2D + 1.2F + 1.6L + 1.6H]

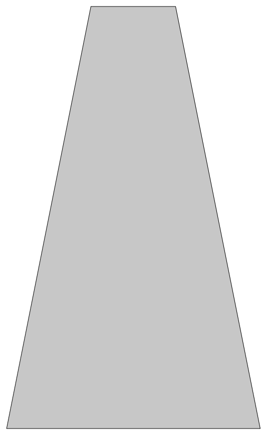


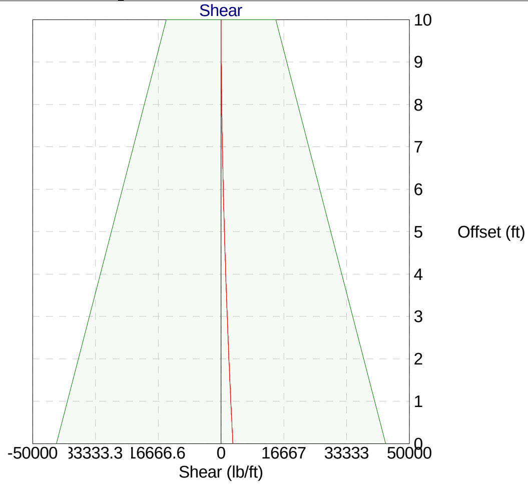


Check (ACI 318-11 Ch 10) @ 0 ft from base

ϕM_n = 163.9 ft·k / ft ≥ M_u = 11.56 ft·k / ft ✓

Stem Shear Checks [1.2D + 1.2F + 1.6L + 1.6H]





Shear Check (ACI 318-11 Ch 11.1.1) @ 0 ft from base

ϕV_n = 43715 lb / ft ≥ V_u = 3200 lb / ft ✓

Stem Miscellaneous Checks [1.2D + 1.2F + 1.6L + 1.6H]

Minimum Steel Check (ACI 318-11 10.5.1) @ 0 ft from base [Stem in negative flexure]

ϕM_n = 163.9 ft·k / ft ≥ (4 / 3) M_u = [4 / 3] (11.56 ft·k / ft) = 15.41 ft·k / ft

Check is waived per ACI 10.5.3 ✓

Maximum Steel Check (ACI 318-11 10.3.5) @ 0 ft from base [Stem in negative flexure]

$\beta_1 = 0.850$ $(F'_c \leq 4000 \text{ psi})$

$$a = \frac{A_s f_y}{0.85 F'_c} = \frac{(0 \text{ in}^2 / \text{in})(60000 \text{ psi})}{0.85 (4000 \text{ psi})} = 0 \text{ in}$$

$$\varepsilon_t = 0.003 \left(\frac{d}{a / \beta_1} - 1\right) = 0.003 \left[\frac{(72 \text{ in})}{(0 \text{ in}) / (0.850)} - 1\right] = \text{INF}$$

ε_t = INF ≥ 0.004 ✓

Appendix 6

Inspection Videos
(Attached via Thumb Drive)