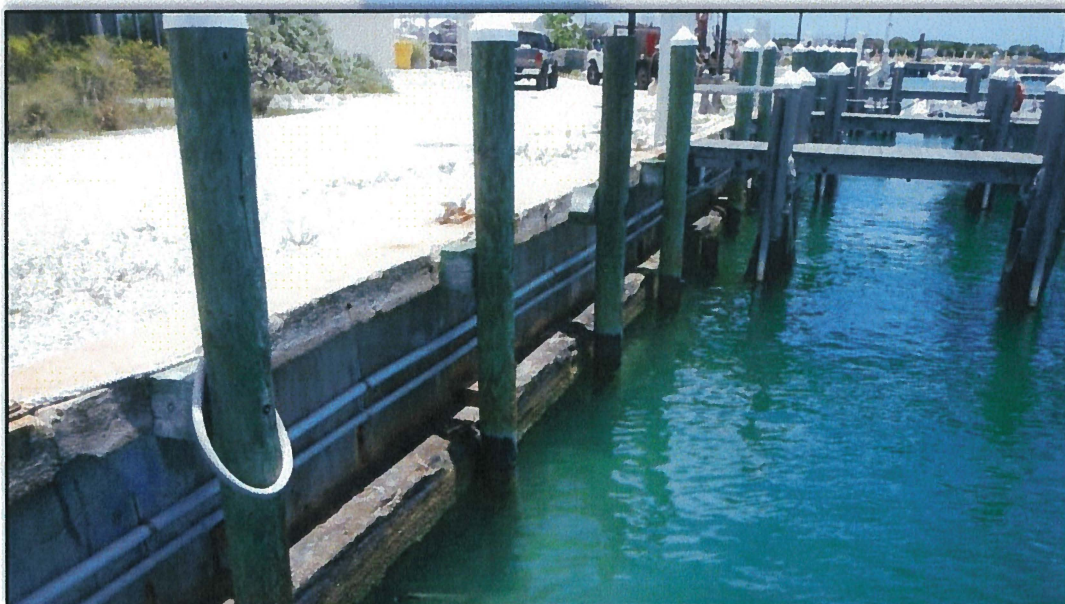
2015

CITY OF KEY WEST

PROJECT: GR1503 / SEAWALL REPAIR & REPLACEMENT OPTIONS

SEPTEMBER 2015



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

9.3.15

Stuart E. McGahee, PE
Florida PE # 57536

TETRA TECH

Tetra Tech, Inc.
759 South Federal Highway, #314
Stuart, FL 34994
Florida COA # 2429

759 South Federal Highway, Suite 314 Stuart, FL 34994
Tel 772.781.3400 Fax 772.781.3411 www.tetratech.com

BACKGROUND

The Truman Annex basin is located on the old U.S. Naval Submarine Base on the southwestern most point of Key West. The basin is approximately 35-40 feet deep and was used as a diesel-powered submarine base known as Marker 57 until it was closed in 1974. Tetra Tech was hired to conduct an assessment of a 350 +/- feet long segment of the existing seawall located on the west side of the existing boat ramp.

The existing seawall appears to be constructed out of concrete sheet pile. It is not clear from the available information, but the thickness and length of the piles seems to vary depending on the location and depth of the water at the toe. The embedment depth along the wall is also unknown, but it appears the wall panels were jetted or driven into the medium density limestone rock (with a minimum amount of embedment) and now the panels have been undermined in several places. The basin slopes steeply away from the toe of the existing wall. It is possible that the wall was originally installed in a shallower basin and the basin was deepened after the wall was already in place. The video provided to Tetra Tech clearly shows the pointed ends of the sheet pile completely exposed. This indicates the wall was initially installed with several feet of embedment.

There is an existing wale beam located just above the waterline (during the inspection) that has been encapsulated in concrete. The concrete covering the wale beam shows some deterioration, i.e. spalling and cracks, etc. The condition of the tie-back system is unknown and was not investigated as a part of this assessment. The erosion of the seawall toe has been monitored for several years and there are voids and undermining that have been documented and continue to worsen. The erosion at the point nearest the boat ramp on the east end of the seawall is shown in the following photo.

Insert 1: Video (0.22 Minutes) East End of Seawall near Boat Ramp

Based on a review of the video provided to Tetra Tech, the panels of the seawall appear to be in relatively good condition for their age. The issue is that at several points along the seawall, those panels are completely unsupported at the toe. That means the only thing keeping them in place is the wale beam and whatever connections are made in the concrete cap. If the support being provided in these locations along the wall fails those panels in jeopardy could drop several feet and roll into the basin.

At the point nearest the boat ramp, the video shows newer steel sheets (on the left hand side of Insert 1 above). This freeze frame shows the voids behind and under the wall. Because the slope is so steep in front of this wall, it will be very difficult to stabilize the erosion in this area without using a driven sheet in front of the old wall.

An April 24, 2012 report by TranSystems (see Exhibit 17) was reviewed by Tetra Tech that documents that several attempts have been made over the years to stabilize the toe but it continues to worsen. The City of Key West has asked Tetra Tech to provide an alternative repair to the one described in the TransSystems report, and to also prepare an assessment for full a wall replacement if deemed necessary.

Insert 2: Video (7.27 Minutes), shows the point where the wall transitions from sack-crete toe protection to the cast in place toe protection. Notice in the photo above where the bottom of the panels are becoming visible. The pole looks like it is sticking in between the toes of two panels.

SUMMARY & OVERVIEW OF EXHIBITS

Tetra Tech performed a site inspection on Tuesday June 23, 2015 beginning at 11:30 am and walked the entire length of the 340 foot long segment of the seawall to observe the proposed boring locations and the buried utility markers. The water level in the basin was measured between 11:30 am and 12:30 pm on a flooding tide and was found to be approximately 7-feet below the top of the seawall cap at that time (low tide occurred at 9:23 am on June 23, 2015 at an approximate elevation of 0.5 feet).

Measurements of the water depths along the face of the seawall were collected using the top of the seawall as a relative elevation. Additional measurements were taken using a weighted tape waterward of the seawall at each of the existing docks. The results of our measurements are shown in **Exhibit 1** - Depths.

Exhibit 2 shows a plan view of a new sheet pile wall installed waterward of the existing seawall. The wall assumes a linear installation until it approaches the existing boat ramp. At that point the wall jogs slightly to tie in at a more perpendicular angle. The plan shows 10-foot returns on either end of the wall and cast in place concrete corners extending down to the toe of the wall.

Measurements collected during the inspection were used to create five wall-basin cross sections that can be used to establish the slope and develop the wall load conditions during final design. For this report they were used to develop conceptual wall repair sketches. The repair sketches were then shared with a marine contractor who performed a constructability review and provided a means and methods assessment (Exhibit 14).

Exhibit 3 shows one stabilization repair method that includes the application of expanding spray foam injected from the land surface behind the wall. The foam is installed under pressure and expands once it comes into contact with water or air. Injection rods are typically installed from the ground surface down to the bottom of the wall at 3-5 feet intervals along the entire wall. The foam is injected until it is observed out in front of the wall. The rods are then slowly extracted while the foam continues to be injected on the way up effectively filling all of the voids behind the wall.

Exhibit 4 shows the same stabilization repair method used in Exhibit 3 with the addition of the installation of an 8-foot Fabriform Unimat along the toe of the wall. This installation will require a specialty contractor with a commercial dive team to install the mat and anchors. There are at least two possible ways to anchor the mat: attach the mat to the existing wall panels using long expansion anchors, or pin the mat to the existing slope.

Exhibit 5 shows a typical cross section with a steel sheet pile replacement wall installed landward of the existing wall. This method would not damage the existing docks, but it would be necessary to cut through the existing deadmen to remove the old wall. This option was eliminated from consideration due to excessive cost and the environmental risk that part of the old wall could slip into the basin and would be difficult to recover.

Exhibit 6 shows a typical cross section with a steel sheet pile wall installed waterward of the existing wall. The new panels are typically permitted to be installed no more that 12-inches in front of the old wall. The permitting agencies will usually allow the panels up to 12-inches in front of the portion of the wall that is sticking out the furthest which in this case would be the wale beam and/or the concrete toe repair. This method will require that the existing concrete cap be removed and possibly part of the existing panels so a new cap and tieback system can be installed.

Pile Buck Sheet Pile Design Software SPW911 was used to conceptually design the steel sheet pile that would be required. **Exhibit 7** is the print out from the Pile Buck Sheet Pile Design Software SPW911. This exhibit was based on the soil borings collected by the geotechnical engineer using the worse-case condition located at the point immediately adjacent to the boat ramp.

The basin cross sections and wall reconstruction schematics are included in this report as **Exhibits 8 – 12**. The geotechnical engineering report (Exhibit 15) has indicated that the contractor should be able to vibrate steel sheets without punching or boring into the limestone. Using a steel sheet pile section, **Exhibit 8** shows the cross section near Dock 1, Exhibit **9** shows the cross section near Dock 2, and **Exhibit 10** shows the cross section near Dock 3. The water depths along the face of the wall increase in this area and the slope away from the toe is steeper. The previous repairs to the existing wall will require that the new steel sheet pile wall be placed a little farther waterward and/or require the previous repairs to be removed. **Exhibit 11** shows the cross section near Dock 4. Water depths at the face continue to increase and the slope away from the face of the wall continue to increase. **Exhibit 12** shows the cross section near Dock 5. This particular cross section shows the steepest slope away from the face of the wall which may require longer sheets.

Exhibit 13 includes product specification data for the DYWIDAG system and sheet pile specifications for both the hot-rolled PZ-27 and the cold rolled XZ-95. **Exhibit 14** includes project specific quotes requested from vendors and contractors.

The boring logs collected during the geotechnical investigation and shared with a marine contractor are included as **Exhibit 15**. Site Photos taken during the inspection are included as **Exhibit 16**. The underwater photos are found in **Exhibit 17** (TransSystems report).

GEOTECHNICAL REVIEW & WALL REPAIR RECOMMENDATIONS

Standard Penetration Tests (SPT) were performed in two locations behind the existing seawall to a depth of 30 feet below land surface (BLS). Groundwater was found to correspond with the observed sea level and was located at 7 feet BLS. The cemented limestone layer is shown in the geotechnical boring logs to be relatively soft for the first 12 feet BLS, with SPT resistance (measured in blows per foot, N), in the 7-13 range. This corresponds to the location (depth) of the scour observed along the toe of the existing seawall.

Once the SPT reached 14 feet BLS the N values increase to 33 and 42 blows per foot. This appears to be the point near where the existing concrete sheet pile stops. An N value of 33-42 is pretty stiff but still "drivable" with steel sheet pile and this may be the hard-rock layer that was used as a base for the original seawall. At 19 feet BLS the limestone becomes cemented and N values increase to 83 and 62 blows per foot which could represent a point close to refusal. Any recommendation for seawall replacement should prescribe the toe of the wall to be embedded substantially into this layer.

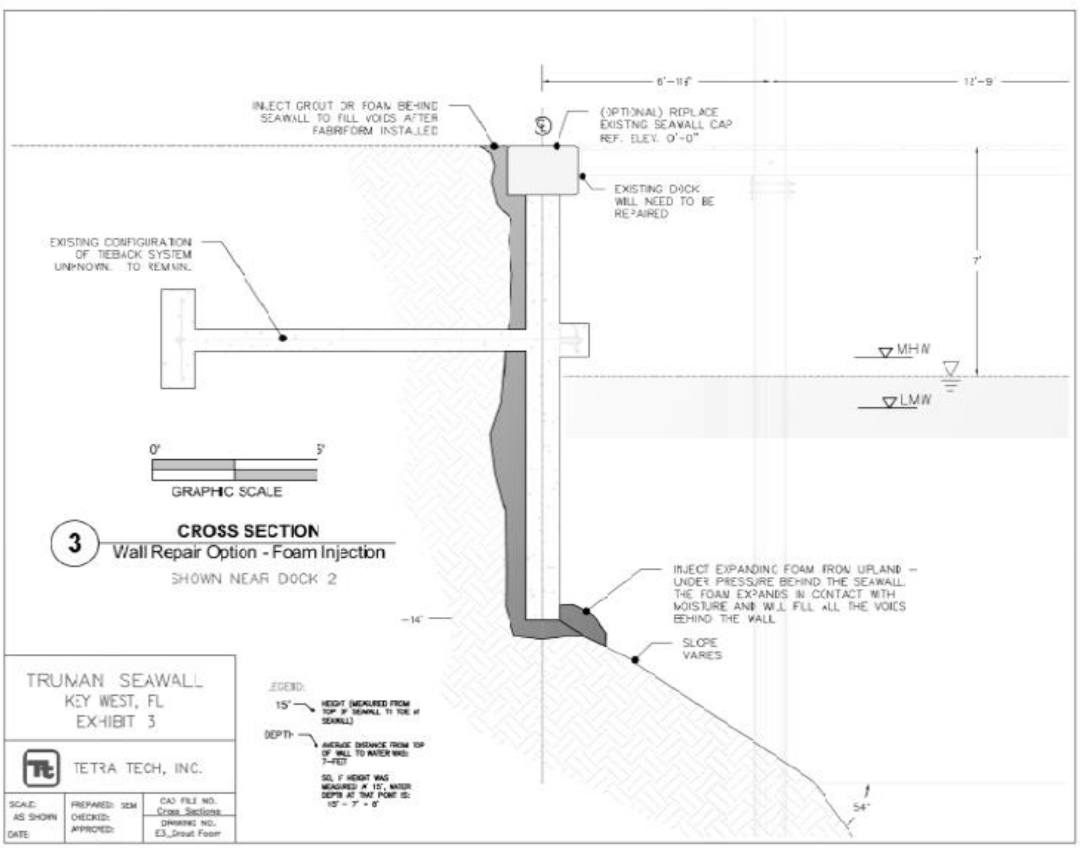
The scouring along the toe of the existing wall gets worse and the water depths increase as you move from east to west. At some point in the past, wall repairs were made that included the placement of formwork and the pouring of a concrete toe reinforcement. These concrete repairs appear to have been effective for many years but now create a constructability issue as shown schematically in **Exhibits 8-12**. The location of this concrete mass and the increasing water depths will require the sheet pile to extend out further and deeper into the basin. This will possibly require additional sheet pile length to ensure the piles are sufficiently toed-into the rock. The constructability review and discussions with the geotechnical consultant indicate that the sheets can be driven through the rock for the entire length of the wall.

WALL REPAIR OPTIONS

Before the wall replacement options are discussed, two in situ wall repair options were considered. The following two seawall repair approaches were selected because they should not add additional loads to the wall. For both cases, a new seawall cap is recommended. In several locations along the wall, in particular the east end near the boat ramp it would be necessary for the contractor to install a temporary screen (like plywood) onto the face of the wall because the voids are so large.

Expanding Polyurethane Foam

Exhibit 3 shows a typical wall condition with an exaggerated void behind the wall. The repair method proposed in this exhibit is called Expanding Polyurethane Injection and can be performed by at least one relatively local (Florida) contractor. This contractor was contacted and provided with the characteristics of the existing wall and asked to submit a quote for this application. The quote from Stable Soils for $150,720 and has been included with other estimates in **Exhibit 14**. The foam is injected in a tremie-type application by placing rods behind the wall at 3-5 feet on center, from the ground surface down to near the toe of the existing wall. As the foam is injected, the operator monitors the front of the wall looking for foam to float up indicating the foam has made it all the way through the opening. A floating turbidity curtain is used out in front of the wall to prevent foam from escaping into the basin. Total repair method cost is estimated to be **$150,720**, with only a minimal amount of permitting or engineering cost which can be completed by the contractor.



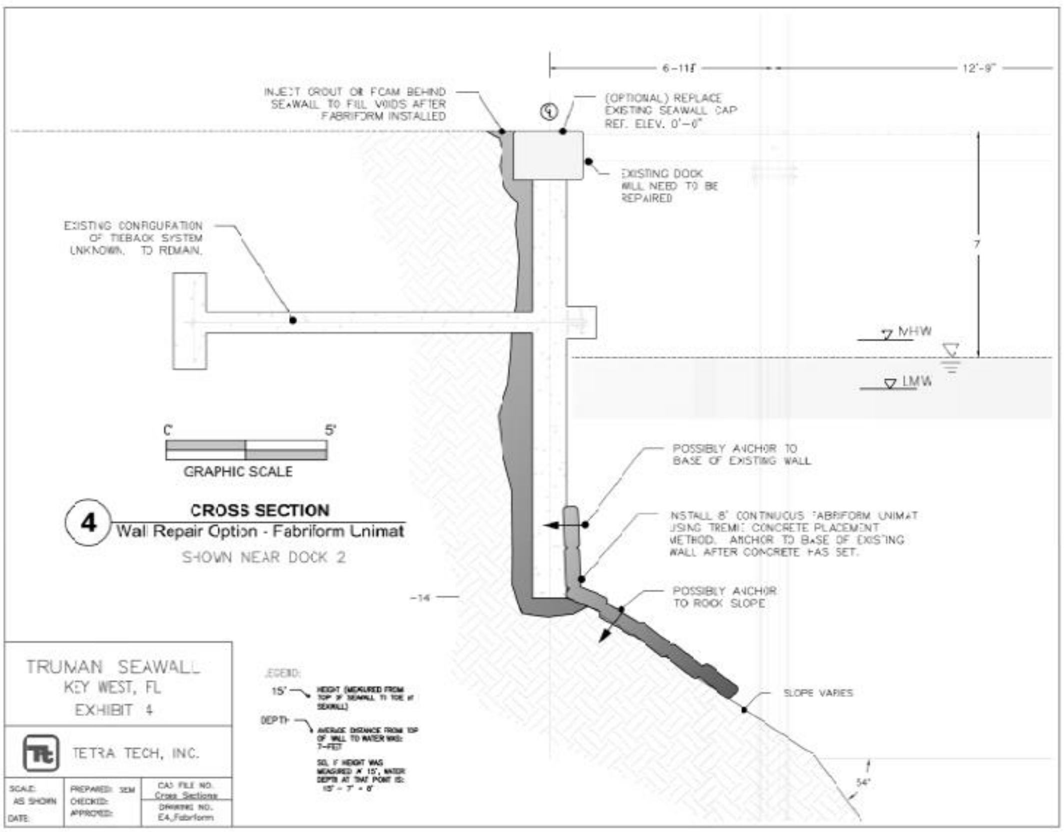
Insert 3: [Exhibit 3] “Stable Soils” Seawall Repair

Expanding Polyurethane with Fabriform Toe Protection

Exhibit 4 has the same general cross section but adds the installation of toe protection to the wall in the form of a Fabriform Unimat. For this preliminary repair study Tetra Tech asked for a quote from Underwater Engineering Services, Inc. (UESI) for the installation of the Fabriform mat (using their commercial dive team) and was given a "verbal" estimate for approximately $50,000 for the mat only. Any repair option should also include some method of pinning the mat in place. The estimate for the anchoring would add at least another $25-50,000 and require the use of helical piles installed hydraulically along the toe of the wall or some other soil pinning method. The video shows the existing concrete bags pinned to the ground by what appears to be rebar hammered into the slope.

We do not recommend this option without attempting to fill the voids behind the wall as well. So, adding the previous repair option (expanding polyurethane injection, $150,720K) to the Fabriform installation the estimated cost is: $150,720 + $50,000 + $50,000 = **<u>$250,720</u>**.

A repair near the east end (boat ramp) of this wall would be difficult using this technique because the fabriform would simply be too vertical. Although this would be suitable for repair along a majority of the wall, it does not appear to be suitable for the entire length.



Insert 4: [Exhibit 4] "Stable Soils" Seawall Repair with Fabriform Toe Protection

WALL REPLACEMENT OPTIONS

Two wall replacement options were considered, concrete panel and steel sheet pile. Based on the following order of magnitude estimates, the concrete panel option was eliminated from consideration due to budgeting restrictions.

Concrete Panel Wall

The budget number for an augured concrete panel wall using 12" x 3' x 25' panels, 2' x 2' reinforced concrete cap, 1.25" x 20' hot dipped galvanized DYWIDAG Threadbar anchor rods, and precast deadman anchors is: ***$ 3,000 - $ 4,000*** per linear foot; for a 325 LF wall that comes to ***$ 975,000 - $ 1,300,000***.

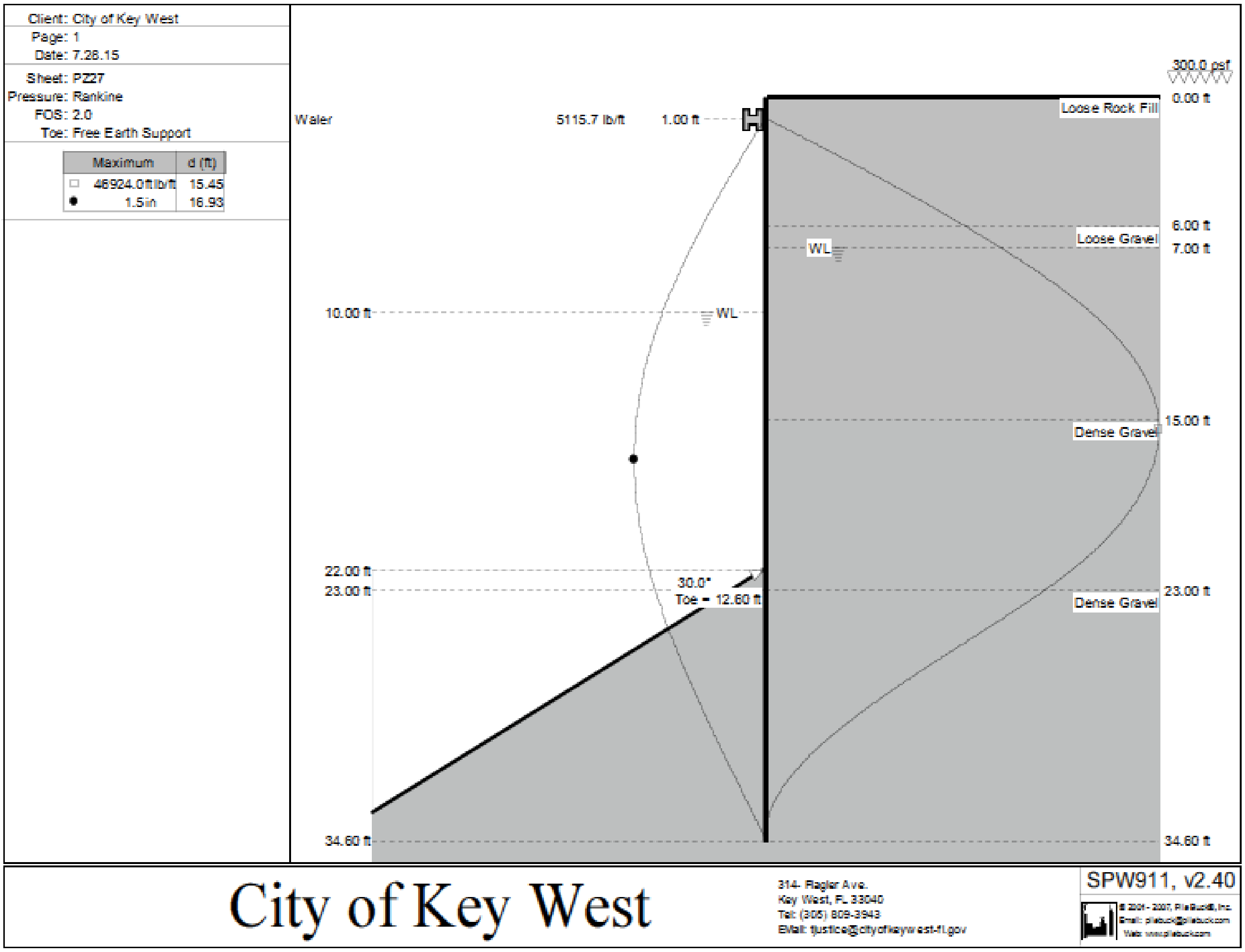
Steel Sheetpile Wall

The budget number for using 25 ' epoxy-coated hot-rolled steel sheetpile with a 2.5' x 2, reinforced concrete cap, 1.25" x 20' hot dipped galvanized DYWIDAG Threadbar anchor rods, and precast deadman anchors is: ***$ 1,700 - $ 2,000*** per linear foot; for a 325 LF wall that comes to ***$ 525,500 - $ 650,000***.

Going forward with the steel sheet pile option and using sheet pile design software (SPW911), a steel sheet was selected that provided both drivability and the required section modulus for the worst case scenario. The steel sheet selected was the hot-rolled, PZ-27 (XZ-95 is the cold-rolled structural equivalent). **Exhibit 5** shows a typical landward installation and **Exhibit 6** shows a typical water ward installation.

As shown in **Exhibit 5**, the installation of a new wall behind an old concrete wall that is not continuously toed into the rock could be problematic. As the new wall goes in and the old wall's tie back system is removed the concrete panels that are not secured in front of the new wall could become dislodged and find their way to the bottom of the basin creating unnecessary environmental considerations. In addition, even though a heavy turbidity curtain would be proposed for this application, any sediments in front of the new wall could become liberated and end up in the basin as well. In order to stabilize the old wall and/or remove it at the same time as the installation of the new wall would unnecessarily complicate the work and has therefore been eliminated from any further consideration. **Exhibit 6** then, showing a typical wall section in front of the old wall, was used as an ideal wall replacement option.

Exhibit 7 shows one design calculation using the PZ-27 sheet pile installed in general soil conditions that are similar to those shown in the geotechnical engineer's report. Using this sheet (PZ-27) and the prescribed soil characteristics, a 35 foot long sheet pile was calculated by the software.



Insert 5: Crop view of **Exhibit 7** (SPW911 Design output)

You will note on this exhibit that the worst case scenario at the expected lowest tide condition was used to produce the highest loads possible in this application. In addition a surcharge load of 300 PSF was applied to the soil profile immediately behind the wall. This surcharge load can be increased and the wall section recalculated if the City plans to use the area immediately behind the wall for heavy vehicular traffic or material storage.

Using these inputs, the PZ-27 is expected to support the loads applied with a maximum deflection of 1.5 inches at a distance of 15-feet below land surface. The linear load on a wale or brace located 1-foot below land surface will generate a maximum load for 5,115 pounds per foot. If the tiebacks, for example, are spaced at 10-feet on center, they will need to support this load (51,150 lbs/deadman).

GENERAL WALL CHARACTERISTICS

For estimating purposes (below), the new steel sheet pile wall was estimated to be constructed with the following general characteristics:

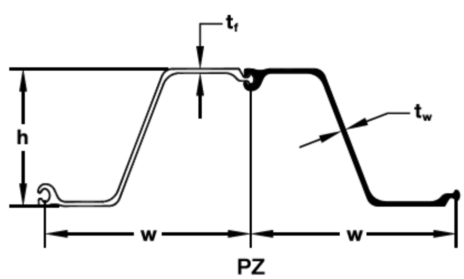
A. <u>Steel Sheet Pile</u>

- PZ-27 (hot rolled)
- XZ-95 (cold rolled equivalent)

The structural equivalent to the hot-rolled PZ-27 sheet pile was selected because it is approximately 30% cheaper per ton. The material selected was **<u>XZ-95</u>** and includes a 16-mil epoxy coating. The biggest difference between hot-rolled and cold-rolled steel sheet pile is the way the knuckles come together. The hot-rolled steel has a tighter fit and will not allow as much water or sediment to pass. Because the Truman seawall is being placed into a dense soil environment and gravel backfill is expected, the cold-rolled opening will work as required.

PZ/PS

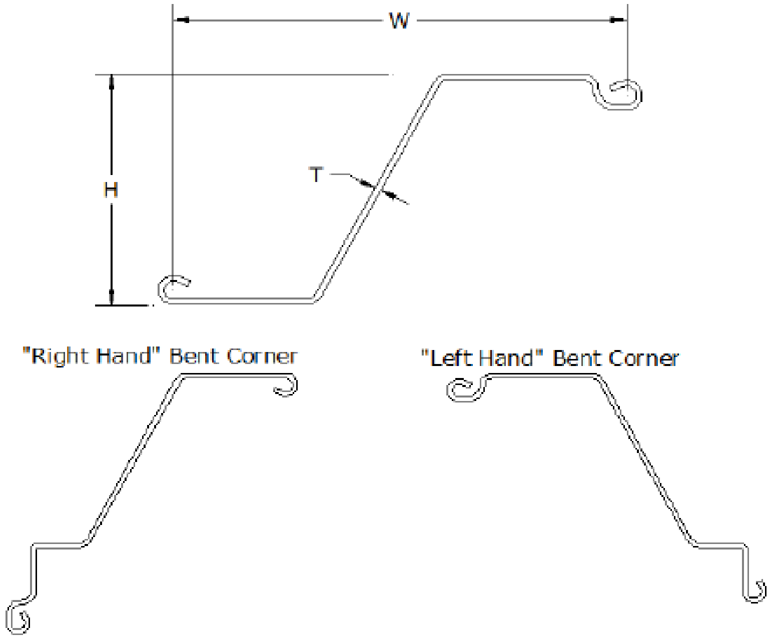
PZ/PS Hot Rolled Steel Sheet Pile



SECTION	Width (w) in (mm)	Height (h) in (mm)	THICKNESS Flange (t_f) in (mm)	Wall (t_w) in (mm)	Cross Sectional Area in²/ft (cm²/m)	WEIGHT Pile lb/ft (kg/m)	Wall lb/ft² (kg/m²)	SECTION MODULUS Elastic in³/ft (cm³/m)	Plastic in³/ft (cm³/m)	Moment of Inertia in⁴/ft (cm⁴/m)	COATING AREA Both Sides ft²/ft of single (m²/m)	Wall Surface ft²/ft² of wall (m²/m²)
PZ 22	22.0 559	9.0 229	0.375 9.50	0.375 9.50	6.47 136.9	40.3 60.0	22.0 107.4	18.1 973	21.79 1171.4	84.38 11500	4.48 1.37	1.22 1.22
PZ 27	18.0 457	12.0 305	0.375 9.50	0.375 9.50	7.94 168.1	40.5 60.3	27.0 131.8	30.2 1620	36.49 1961.9	184.20 25200	4.48 1.37	1.49 1.49
PZ 35	22.6 575	14.9 378	0.600 15.21	0.500 12.67	10.29 217.8	66.0 98.2	35.0 170.9	48.5 2608	57.17 3073.5	361.22 49300	5.37 1.64	1.42 1.42
PZ 40	19.7 500	16.1 409	0.600 15.21	0.500 12.67	11.77 249.1	65.6 97.6	40.0 195.3	60.7 3263	71.92 3866.7	490.85 67000	5.37 1.64	1.64 1.64

Insert 6: PZ-27 Sheetpile Section Properties

XZ-95 Steel Sheet Piling



ASTM A572 Grade 50 Steel   Melted and Manufactured in the US.

Section	Width	Height	Thickness	Cross Sectional Area	Weight		Section Modulus	Moment of Inertia	Coating Area Both Sides
					Pile	Wall			
	W	H	T						
	in	in	in	in^2/ft	lb/ft	lb/ft^2	in^3/ft	in^4/ft	ft^2/ft
	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^3/m	cm^4/m	m^2/m
XZ-95	25.00	14.12	0.375	15.20	51.70	24.80	33.50	237.00	6.03
	635	359	9.50	98.2	76.9	121	1800	32400	1.84

Insert 7: XZ-95 Sheetpile Section Properties

B. <u>Concrete Cap</u>

- Concrete cap: 2.5 feet wide and 2 feet tall,
- 65 CY Concrete for a 350 LF cap
- 6,000 psi concrete,
- Corrosion inhibitors,
- (7) #5 longitudinal bars
- use #4 stirrups at 12-inches on center

C. <u>Backfill (Between) the walls</u>

- Assumed 1.5 feet wide x 10 ft deep, shallow
- Assumed 1.5 feet wide x 20 ft deep, deeper
- Total loose fill: 291 CY (364 Ton)

COST BREAKDOWN

ENGINEER'S WALL REPLACEMENT COST ESTIMATE					
Description	**Unit**	**No. of Units**	**Unit Price**	**Material**	**Labor / Installation**
Mobilization	LS	1	15,000.00	15,000.00	15,000.00
Demolition	LS	1	15,000.00	15,000.00	30,000.00
XZ-95 Sheetpile (FOB Key West)	LF	350	536.40	187,740.00	375,480.00
Concrete Cap (6,000 PSI)	CY	65	265.00	17,225.00	34,450.00
Rebar #5 & #4	LF	350	9.51	3,330.00	6,660.00
Dock & Mooring Pile Repairs	LS	1	20,000.00	20,000.00	40,000.00
Backfill (Local Pea Gravel)	TON	364	30.00	10,920.00	21,840.00
			Subtotal	269,215.00	523,430.00
XZ-95 Total	LF	350	2,264.70		792,645.00

Insert 8: Engineers Cost Estimate

A copy of the specifications for the PZ-27 and XZ-95 sheet pile, and the DWYDAG specification are attached to **Exhibit 7**. A copy of the material quotes received for the steel sheet pile, concrete, backfill, #57 stone, sand, pea gravel, #5 straight rebar and #4 stirrups/hooks are attached as Exhibit 14. The quotes are all FOB Key West from Piling Products (Jacksonville), DECO Truss Company (Miami) or Charlie Topino & Sons (Coppitt Key). A summary list of materials available from these suppliers is provided below.

UNIT PRICES				
Description	**Unit**	**Unit Price**	**No. of Units**	**Subtotal**
Sand (Local Screening)	Ton	30.00	364	10,920.00
Beach Sand (Immokolee)	Ton	45.00	364	16,380.00
Clean Sand (Miami)	Ton	38.00	364	13,832.00
#57 (Miami)	Ton	40.00	364	14,560.00
#57 (Local Screeening)	Ton	31.00	364	11,284.00
Pea Gravel (Local Screening)	Ton	30.00	364	10,920.00
Backfill Dirt (Local)	Ton	28.00	364	10,192.00
Concrete 6,000 PSI (Local)	CY	265.00	65	17,225.00
#5 Rebar	LB	0.42	3338	1,401.96
#4 Rebar (Saddle)	EA	4.26	394	1,678.44
#3 Rebar (Saddle) Alt.	EA	2.05	394	807.70
Fuel Charge (Miami)	EA	250.00	1	250.00
XZ-95 Sheetpile	LF	536.40	350	187,740.00
PZ-27 Sheetpile	LF	697.32	350	244,062.00

Insert 9: Unit Prices from Material Suppliers

ENVIRONMENTAL & PERMITTING

A full riparian survey with bathymetry will need to be provided to the Army Corp of Engineers and the Florida Department of Environmental Protection with a complete construction design so they can evaluate any risks or impacts to the environment. Because of the long history associated with this basin and the fact that we will ultimately be proposing to install a new wall in front of the old wall, limiting transport of sediment the reviewers should have few comments.

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. NOAA is a partner in the seawall remediation and will be coordinating any coral rescue operations that are needed. It is unknown at this time how the relocations will be managed and procured. All of the agencies will require confirmation that no additional resources will be impacted by the placement of the new wall. It would be advisable to submit the existing pre-construction underwater inspection video to the contractors and the reviewing agencies, and a post-construction video to the agencies after it has been completed.

Tetra Tech has had preliminary discussions with NOAA regarding the small coral on the exiting concrete panels and the corals that had been placed by NOAA near the toe of the wall. It is our understanding that NOAA will be performing a site survey (dive) in a few weeks to determine what type of rescue will be required prior to construction. The following picture shows some of the corals that have been placed on the toe of the wall by NOAA.

Insert 10: Toe of Existing Seawall – NOAA placed Corals

If possible it would be beneficial during the survey to posthole a few places behind the seawall to confirm the tieback spacing (approximately 10 feet on center) so that it can be shown on the plan that goes out for bid. It is not necessary to find every tieback, just a few to determine the spacing. The bidders will be instructed to keep the old tiebacks if possible in order to limit the amount of soil that will be displaced. For budgeting, the following are the additional services that will be required prior to construction:

1. A riparian and bathymetric survey
2. A benthic habitat survey identifying corals
3. Preparation of plans and specifications
4. Regulatory pre-application field visits
5. Permitting application
6. Responding to regulatory request for information
7. Procurement assistance and, Construction services.

LIFE CYCLE EXPECTATIONS

Steel sheet pile walls are expected to have a life expectance of over 25-years. The sheet pile walls (even though they will be coated) will eventually show corrosion; especially around the knuckles and joints since the installation of the piles will almost certainly damage part of the protective coatings. For steel in particular the region in and above the splash zone will show the most corrosion.

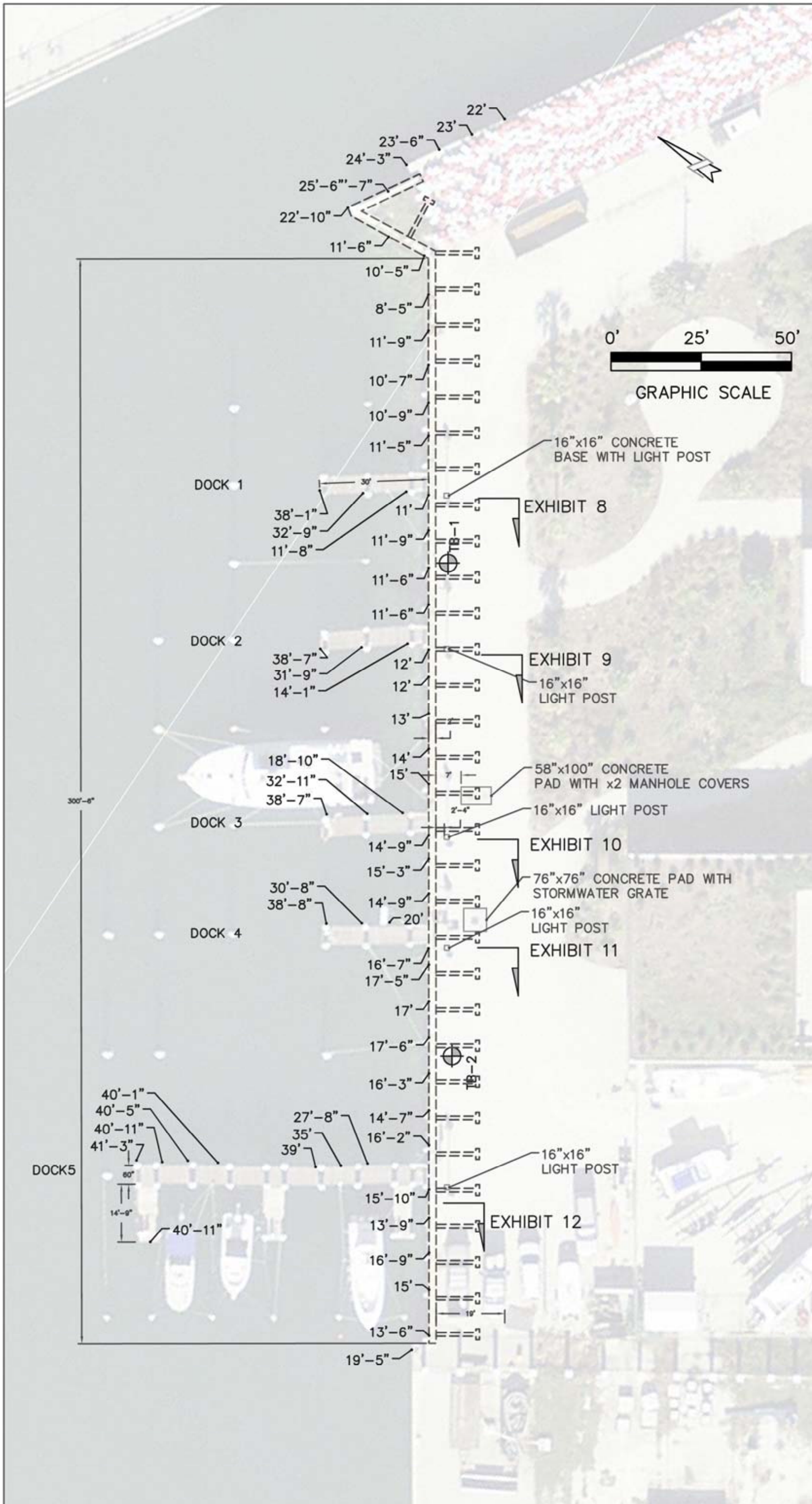
Life expectancy beyond 25-years is uncertain but if care is taken during installation and the face of the walls are coated with epoxy after the installation, few issues should develop. Any stray current (powering the boat slips for example) should be tested for and eliminated as the current will cause a significant increase in corrosion.

RECOMMENDATION

Based on our review of the video and the existing wall conditions at the site, it is expected that any additional repairs made to the toe of the wall would be temporary and therefore we recommended that a new steel sheet pile wall be installed in front of the existing concrete wall.

Exhibits 1-6

Plan View and Design Alternatives



LEGEND:

60"	DIMENSIONS (FIELD–MEASURED)
15'	HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH	AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7–FEET SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' – 7' = 8'
	BORING LOCATION
	EXISTING WALL

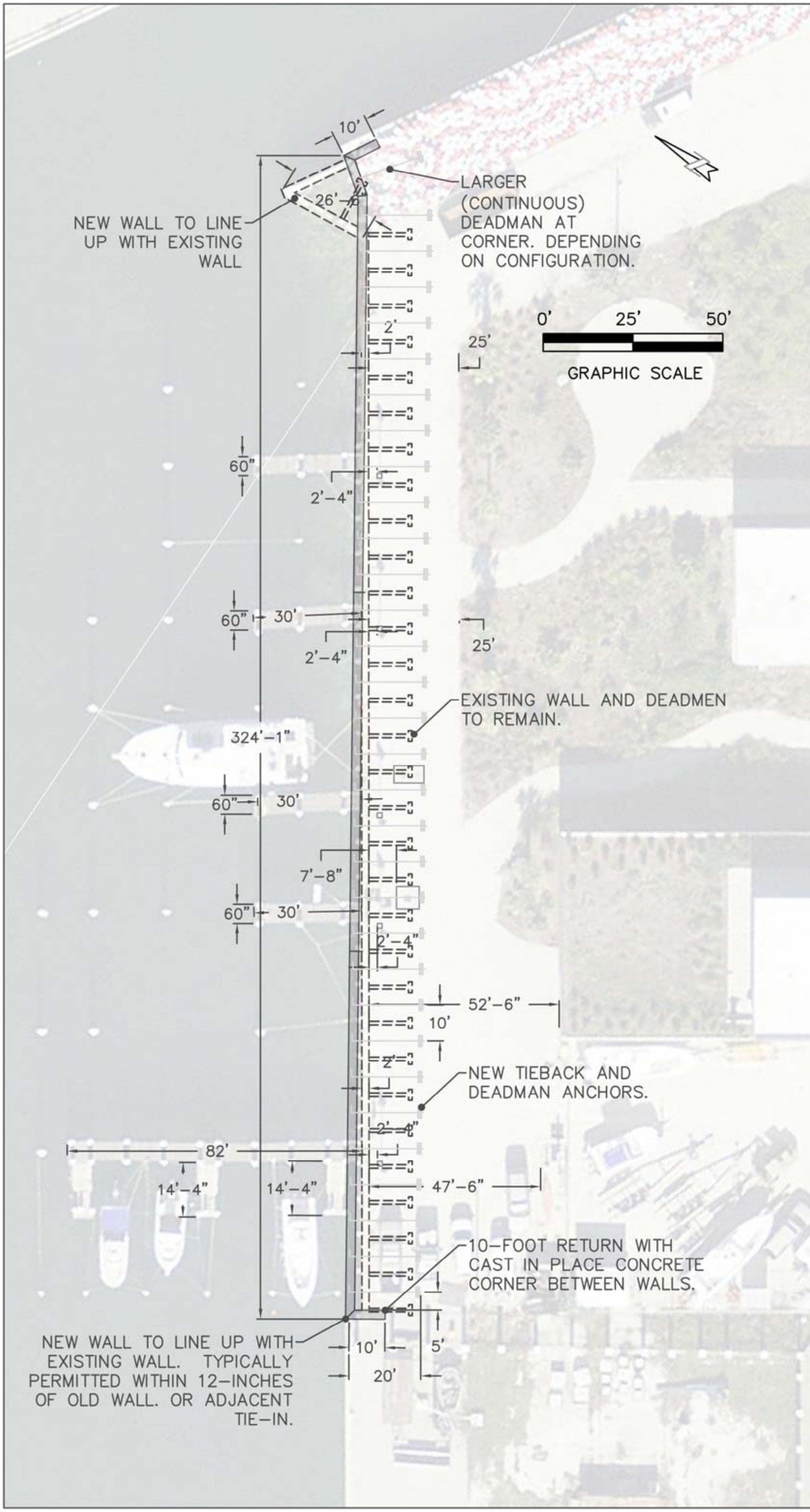
NOTES:

1. ALL DIMENSIONS AND ELEVATIONS SHOWN WERE FIELD–MEASURED USING 25' AND 100' TAPE MEASURES. THE SEAWALL LENGTH WAS MEASURED USING A WHEEL MEASURE.
2. DATA COLLECTED ON 6/23/15. ELEVATION DATA AND WATER DATA COLLECTED BETWEEN 11:30AM TO 12:30PM.
3. ALL ELEVATIONS SHOWN ARE MEASURED COMPARED TO THE ELEVATION OF THE SEAWALL AT EACH LOCATION. ELEVATIONS AT DOCKS WERE COMPARED TO THE RELATIVE ELEVATION AT THE SEAWALL IN FRONT OF THE DOCK.
4. THE SEAWALL CAP IS 2' WIDE BY 1' TALL. THE SEAWALL APPEARS TO BE MADE OF REINFORCED CONCRETE.
5. MEASUREMENTS ARE ROUNDED TO THE NEAREST INCH.
6. DISTANCE TO WATER IS MEASURED FROM THE TOP OF THE EXISTING WALL TO THE TOP OF WATER AT THE TIME TAKEN.

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 1 – DEPTHS

TETRA TECH, INC.

SCALE: AS SHOWN	PREPARED: FM	CAD FILE NO. Site Visit 062315
DATE:	CHECKED: APPROVED:	DRAWING NO. E1_DEPTH



LEGEND:

60" DIMENSIONS (APPROXIMATED)

EXISTING WALL

PROPOSED WALL

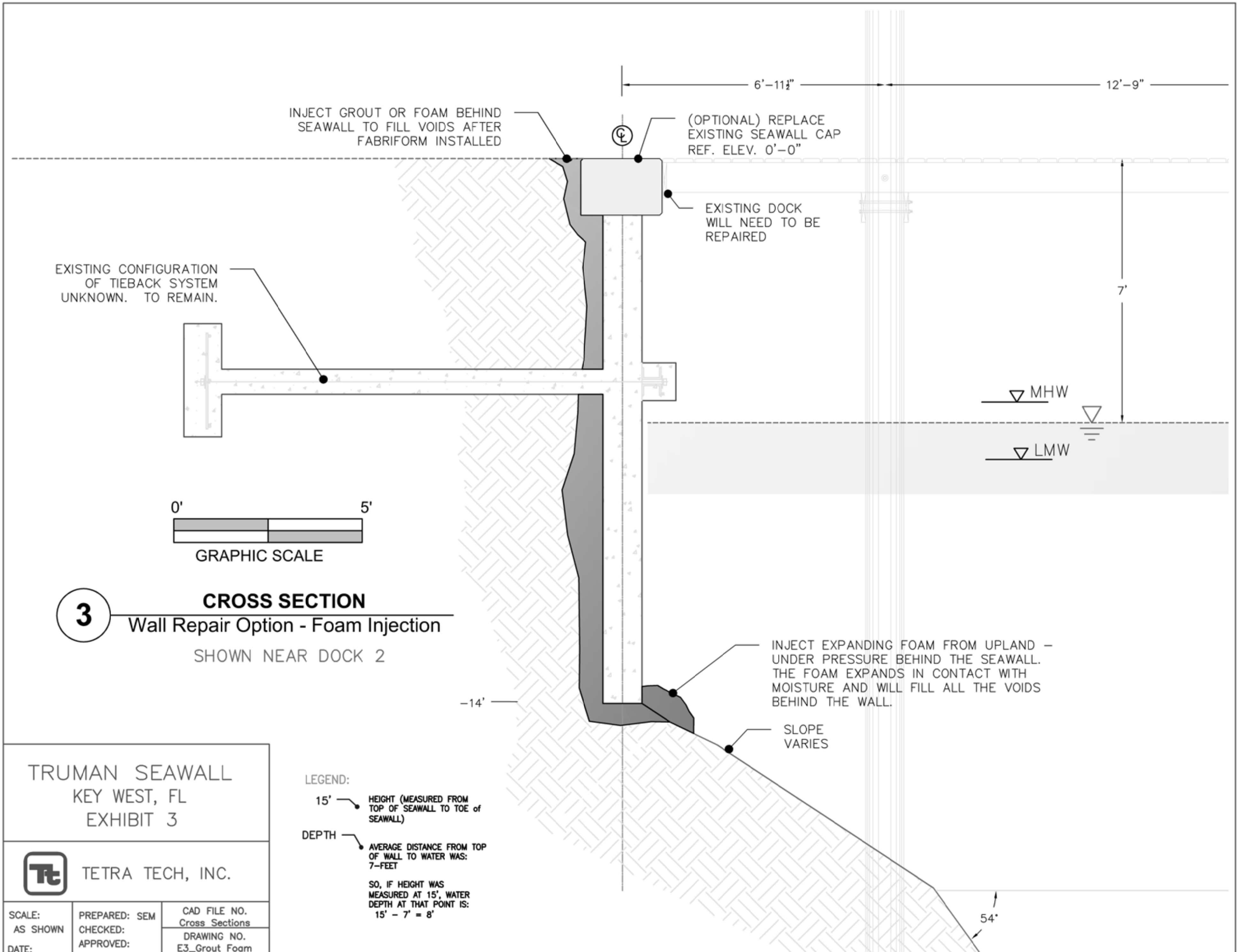
NOTES:

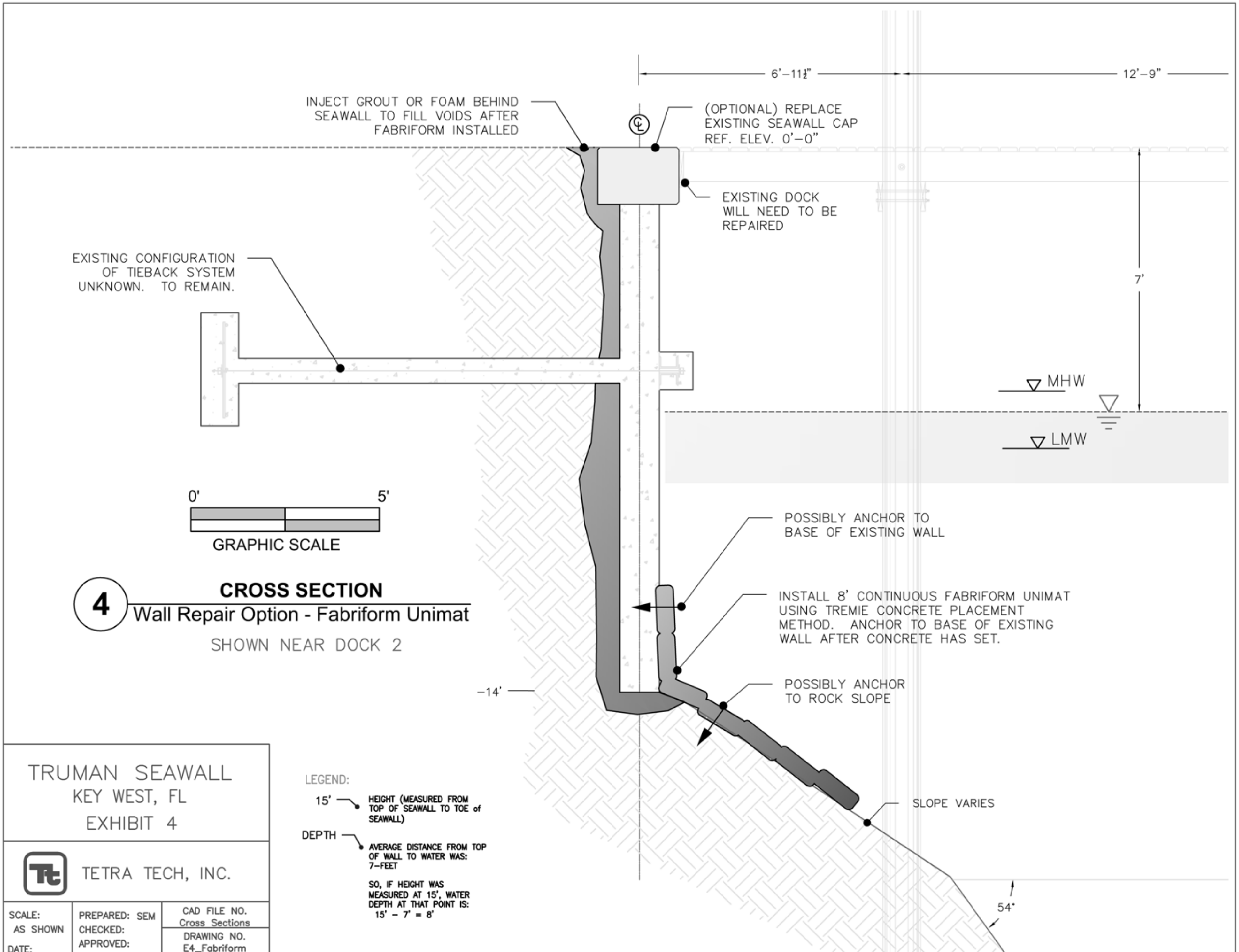
1. ALL DIMENSIONS AND ELEVATIONS SHOWN WERE FIELD-MEASURED USING 25' AND 100' TAPE MEASURES. THE SEAWALL LENGTH WAS MEASURED USING A WHEEL MEASURE.
2. DATA COLLECTED ON 6/23/15. ELEVATION DATA AND WATER DATA COLLECTED BETWEEN 11:30AM TO 12:30PM.
3. MEASUREMENTS ARE ROUNDED TO THE NEAREST INCH.

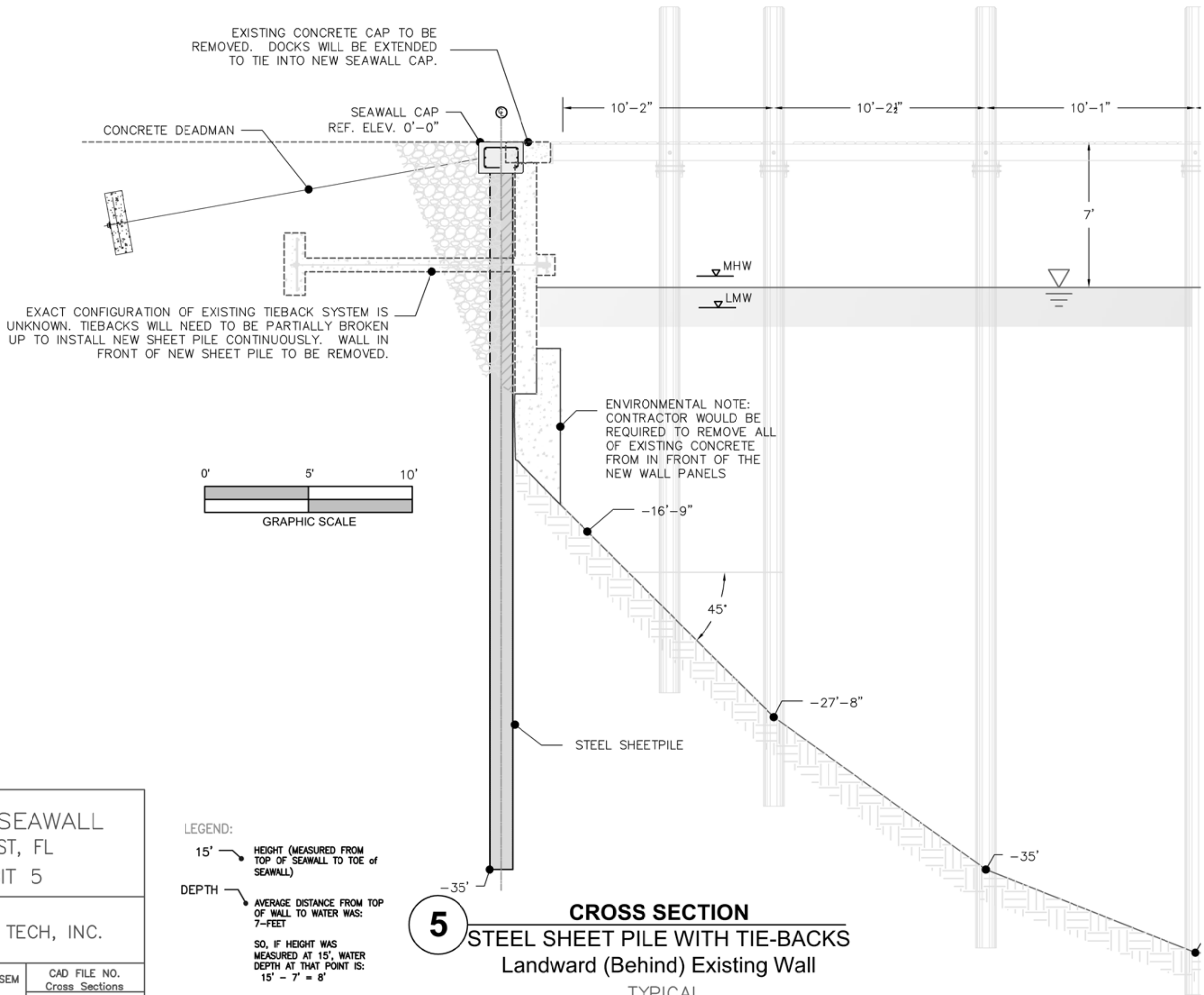
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 2 – PROPOSED WALL PLAN VIEW

TETRA TECH, INC.

SCALE: AS SHOWN	PREPARED: FM	CAD FILE NO. Site Visit 062315
DATE:	CHECKED: APPROVED:	DRAWING NO. E2_PLAN


6'-11½"
12'-9"
INJECT GROUT OR FOAM BEHIND SEAWALL TO FILL VOIDS AFTER FABRIFORM INSTALLED
(OPTIONAL) REPLACE EXISTING SEAWALL CAP REF. ELEV. 0'-0"
EXISTING DOCK WILL NEED TO BE REPAIRED
EXISTING CONFIGURATION OF TIEBACK SYSTEM UNKNOWN. TO REMAIN.
7'
MHW
LMW
0'
5'
GRAPHIC SCALE
3
CROSS SECTION
Wall Repair Option - Foam Injection
SHOWN NEAR DOCK 2
INJECT EXPANDING FOAM FROM UPLAND – UNDER PRESSURE BEHIND THE SEAWALL. THE FOAM EXPANDS IN CONTACT WITH MOISTURE AND WILL FILL ALL THE VOIDS BEHIND THE WALL.
-14'
SLOPE VARIES
54°
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 3
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E3_Grout Foam
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' - 7' = 8'


6'–11½"
12'–9"
INJECT GROUT OR FOAM BEHIND SEAWALL TO FILL VOIDS AFTER FABRIFORM INSTALLED
(OPTIONAL) REPLACE EXISTING SEAWALL CAP REF. ELEV. 0'–0"
EXISTING DOCK WILL NEED TO BE REPAIRED
EXISTING CONFIGURATION OF TIEBACK SYSTEM UNKNOWN. TO REMAIN.
7'
MHW
LMW
0'
5'
GRAPHIC SCALE
4
CROSS SECTION
Wall Repair Option - Fabriform Unimat
SHOWN NEAR DOCK 2
POSSIBLY ANCHOR TO BASE OF EXISTING WALL
INSTALL 8' CONTINUOUS FABRIFORM UNIMAT USING TREMIE CONCRETE PLACEMENT METHOD. ANCHOR TO BASE OF EXISTING WALL AFTER CONCRETE HAS SET.
–14'
POSSIBLY ANCHOR TO ROCK SLOPE
SLOPE VARIES
54°
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7–FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' – 7' = 8'
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 4
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E4_Fabriform

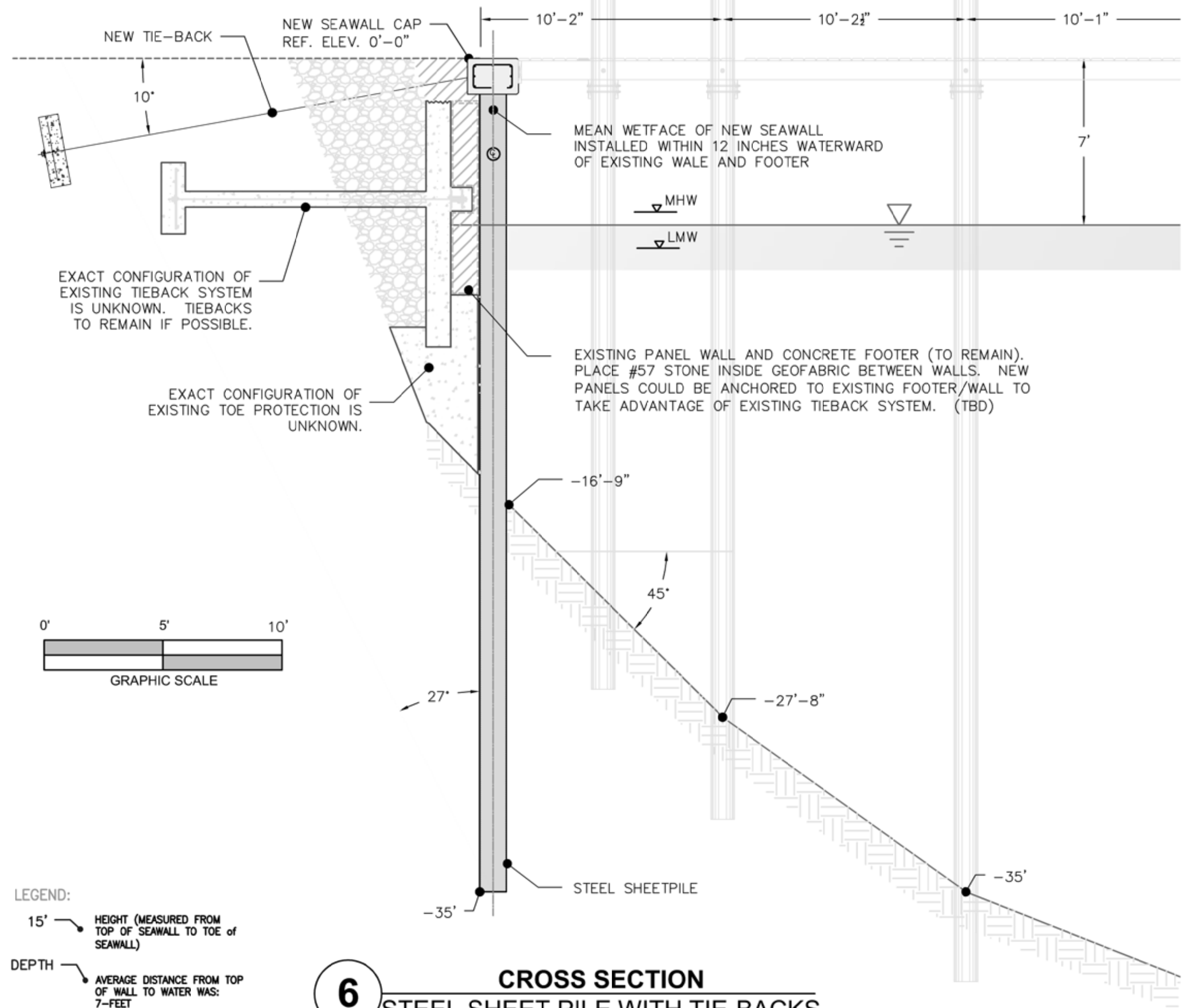


5 **CROSS SECTION**
STEEL SHEET PILE WITH TIE-BACKS
Landward (Behind) Existing Wall
TYPICAL

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 5

TETRA TECH, INC.

SCALE: AS SHOWN	PREPARED: SEM	CAD FILE NO. Cross Sections
DATE:	CHECKED: APPROVED:	DRAWING NO. E5_Landward



LEGEND:

15' — HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)

DEPTH — AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET

SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' - 7' = 8'

6

CROSS SECTION
STEEL SHEET PILE WITH TIE-BACKS
Waterward of Existing Wall
TYPICAL

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 6

TETRA TECH, INC.

SCALE: AS SHOWN	PREPARED: SEM	CAD FILE NO. Cross Sections
DATE:	CHECKED: APPROVED:	DRAWING NO. E6_Waterward

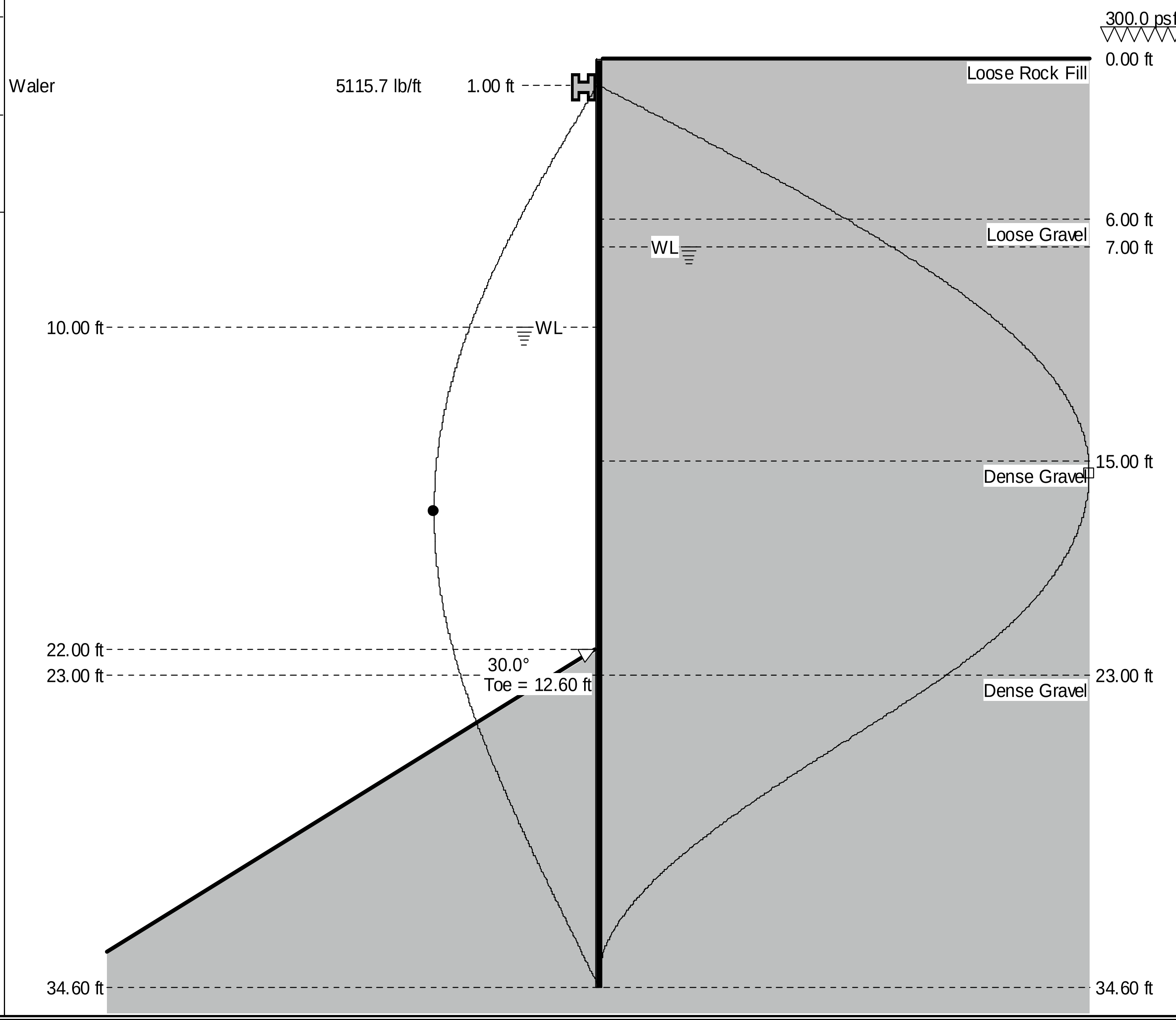
Exhibit 7

Sheet Pile Design, SPW911

Client: City of Key West
Page: 1
Date: 7.28.15

Sheet: PZ27
Pressure: Rankine
FOS: 2.0
Toe: Free Earth Support

	Maximum	d (ft)
□	46924.0 ftlb/ft	15.45
●	1.5 in	16.93



City of Key West

314- Flagler Ave.
Key West, FL 33040
Tel: (305) 809-3943
EMail: tjustice@cityofkeywest-fl.gov

SPW911, v2.40

Client: City of Key West
Page: 2
Date: 7.28.15

Sheet: PZ27
Pressure: Rankine
FOS: 2.0
Toe: Free Earth Support

Input Data

Depth Of Excavation = 22.00 ft | Depth Of Active Water = 7.00 ft | Water Density = 62.43 pcf
Surcharge = 300.0 psf | Depth Of Passive Water = 10.00 ft | Minimum Fluid Density = 31.82 pcf
Slope (passive) = 30.0 degrees

Soil Profile

Depth (ft)	Soil Name	γ (pcf)	γ' (pcf)	C (psf)	C_a (psf)	ϕ(°)	δ(°)	K_a	K_{ac}	K_p	K_{pc}
0.00	Loose Rock Fill	93.55	62.37	0.0	0.0	40.0	0.0	0.22	0.00	4.60	0.00
6.00	Loose Gravel	99.91	65.55	0.0	0.0	35.0	0.0	0.27	0.00	3.69	0.00
15.00	Dense Gravel	109.46	65.55	0.0	0.0	40.0	0.0	0.22	0.00	4.60	0.00
23.00	Dense Gravel	109.46	65.55	0.0	0.0	40.0	0.0	0.22	0.00	4.60	0.00

Solution

Sheet

Sheet Name	I (in^4/ft)	E (psi)	Z (in^3/ft)	f (psi)	Maximum Bending Moment (ftlb/ft)	Upstand (ft)	Toe (ft)	Pile Length (ft)
PZ27	187.50	3.04E+07	31.00	24970.3	64506.5	0.00	12.60	34.60

Load Model: Area Distribution

Supports

Depth (ft)	Type	Linear Load (lb/ft)
1.00	Waler	5115.7

Maxima

	Maximum	Depth
Bending Moment	46924.0 ftlb/ft	15.45 ft
Deflection	1.5 in	16.93 ft
Pressure	615.1 psf	22.00 ft
Shear Force	5036.4 lb/ft	1.00 ft

City of Key West

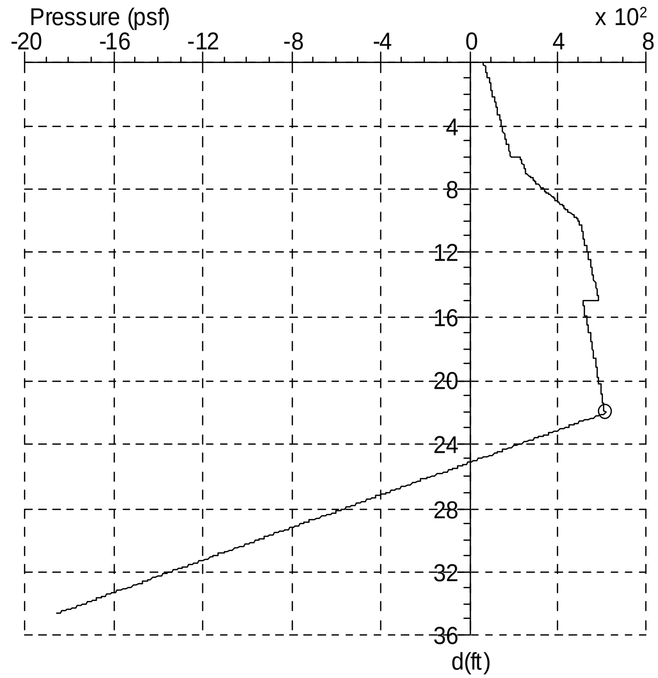
314- Flagler Ave.
Key West, FL 33040
Tel: (305) 809-3943
EMail: tjustice@cityofkeywest-fl.gov

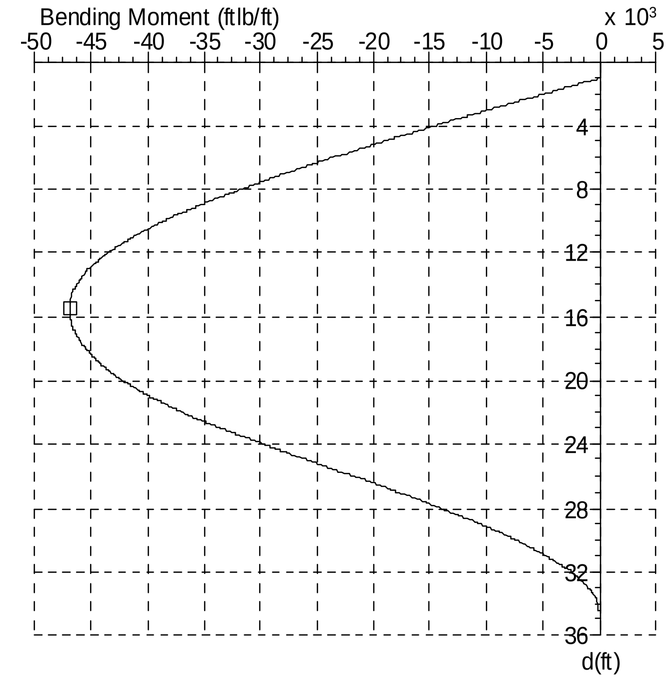
SPW911, v2.40

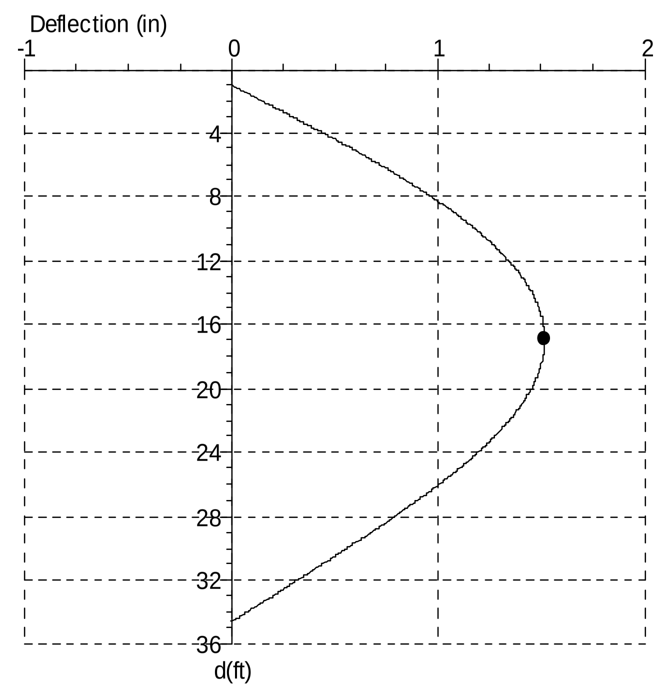
Client: City of Key West
Page: 3
Date: 7.28.15

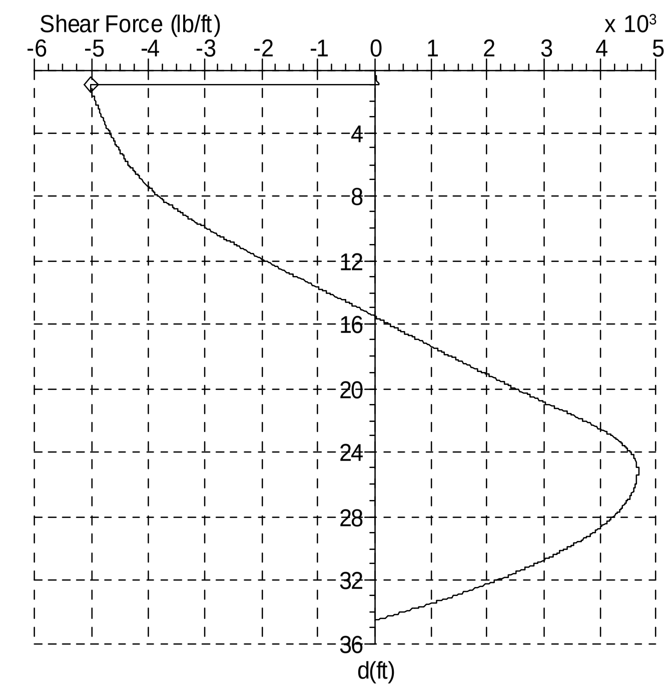
Sheet: PZ27
Pressure: Rankine
FOS: 2.0
Toe: Free Earth Support

	Maximum	d (ft)
○	615.1 psf	22.00
□	46924.0 ftlb/ft	15.45
◇	5036.4 lb/ft	1.00
●	1.5 in	16.93









City of Key West

314- Flagler Ave.
Key West, FL 33040
Tel: (305) 809-3943
EMail: tjustice@cityofkeywest-fl.gov

SPW911, v2.40

Client: City of Key West
Page: 4
Date: 7.28.15

Sheet: PZ27
Pressure: Rankine
FOS: 2.0
Toe: Free Earth Support

depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)
0.00	66.0	0.0	0.0	0.0
0.31	72.5	3.2	0.0	22.6
0.61	78.5	12.6	0.0	45.0
0.92	85.0	30.7	0.0	71.7
1.22	90.9	-979.2	0.0	-5018.4
1.53	97.5	-2568.7	0.1	-4988.4
1.84	104.0	-4148.4	0.1	-4956.4
2.14	109.9	-5575.4	0.2	-4925.5
2.45	116.4	-7134.5	0.2	-4889.5
2.76	123.0	-8681.9	0.3	-4851.4
3.06	128.9	-10077.8	0.3	-4815.0
3.37	135.4	-11601.0	0.4	-4773.0
3.67	141.4	-12973.9	0.4	-4733.0
3.98	147.9	-14470.4	0.4	-4687.1
4.29	154.4	-15952.1	0.5	-4639.0
4.59	160.4	-17285.6	0.5	-4593.6
4.90	166.9	-18737.0	0.6	-4541.6
5.21	173.4	-20171.6	0.6	-4487.5
5.51	179.3	-21460.6	0.7	-4436.6
5.82	185.9	-22861.3	0.7	-4378.5
6.12	235.6	-24118.1	0.7	-4319.0
6.43	244.2	-25478.9	0.8	-4242.8
6.74	252.7	-26815.0	0.8	-4163.9
7.04	262.4	-28007.6	0.9	-4089.7
7.35	287.8	-29294.0	0.9	-4002.1
7.66	310.9	-30438.4	0.9	-3915.5
7.96	336.3	-31667.2	1.0	-3812.5
8.27	361.7	-32862.2	1.0	-3701.4
8.57	384.8	-33917.0	1.0	-3593.5
8.88	410.2	-35040.2	1.1	-3467.0
9.19	435.7	-36122.0	1.1	-3332.5
9.49	458.8	-37067.5	1.1	-3203.2
9.80	484.2	-38063.4	1.2	-3053.3
10.10	501.5	-38926.5	1.2	-2910.4
10.41	507.2	-39827.7	1.2	-2750.4
10.72	512.8	-40677.9	1.2	-2588.5
11.02	517.9	-41405.9	1.3	-2439.9
11.33	523.5	-42157.0	1.3	-2274.6

depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)
11.64	529.1	-42855.3	1.3	-2107.6
11.94	534.2	-43443.9	1.3	-1954.3
12.25	539.8	-44040.0	1.4	-1783.9
12.55	544.9	-44534.8	1.4	-1627.4
12.86	550.5	-45026.6	1.4	-1453.6
13.17	556.2	-45462.9	1.4	-1278.0
13.47	561.3	-45811.0	1.4	-1116.9
13.78	566.9	-46139.9	1.4	-937.9
14.09	572.0	-46389.4	1.5	-773.6
14.39	577.6	-46608.8	1.5	-591.2
14.70	583.2	-46770.0	1.5	-407.1
15.00	588.4	-46865.6	1.5	-238.1
15.31	518.6	-46917.6	1.5	-74.3
15.62	523.2	-46918.3	1.5	91.0
15.92	527.3	-46878.9	1.5	242.5
16.23	531.9	-46792.0	1.5	410.5
16.53	536.1	-46673.0	1.5	564.6
16.84	540.7	-46498.0	1.5	735.4
17.15	545.2	-46276.1	1.5	907.7
17.45	549.4	-46033.6	1.5	1065.5
17.76	554.0	-45721.4	1.5	1240.6
18.07	558.5	-45361.3	1.5	1417.1
18.37	562.7	-44992.0	1.5	1578.8
18.68	567.3	-44539.3	1.5	1758.0
18.98	571.4	-44085.3	1.5	1922.2
19.29	576.0	-43538.6	1.5	2104.3
19.60	580.6	-42942.1	1.5	2287.7
19.90	584.7	-42356.3	1.5	2455.8
20.21	589.3	-41663.7	1.4	2642.1
20.52	593.9	-40920.0	1.4	2829.8
20.82	598.0	-40199.5	1.4	3001.6
21.13	602.6	-39357.4	1.4	3192.1
21.43	606.8	-38546.7	1.4	3366.5
21.74	611.3	-37604.9	1.4	3559.8
22.05	606.7	-36610.2	1.3	3753.9
22.35	545.2	-35661.0	1.3	3918.4
22.66	483.7	-34572.0	1.3	4080.7
22.97	427.8	-33545.6	1.3	4211.3

depth (ft)	P (psf)	M (ftlb/ft)	D (in)	F (lb/ft)
23.27	365.5	-32381.4	1.2	4336.4
23.58	303.2	-31185.5	1.2	4441.6
23.88	246.5	-30075.5	1.2	4520.0
24.19	184.1	-28834.2	1.2	4587.3
24.50	121.7	-27577.1	1.1	4634.9
24.80	65.0	-26424.9	1.1	4661.0
25.11	2.6	-25152.4	1.1	4670.9
25.41	-54.1	-23994.9	1.1	4666.4
25.72	-116.4	-22724.2	1.0	4651.3
26.03	-178.8	-21458.9	1.0	4625.5
26.33	-235.5	-20315.8	1.0	4592.8
26.64	-297.9	-19069.0	0.9	4546.6
26.95	-360.3	-17836.1	0.9	4489.7
27.25	-417.0	-16729.8	0.9	4428.7
27.56	-479.3	-15531.6	0.8	4351.4
27.86	-536.0	-14461.7	0.8	4271.9
28.17	-598.4	-13308.8	0.8	4174.1
28.48	-660.8	-12183.8	0.7	4065.8
28.78	-717.5	-11187.9	0.7	3957.9
29.09	-779.9	-10124.4	0.7	3829.1
29.40	-836.6	-9189.2	0.6	3702.8
29.70	-898.9	-8197.8	0.6	3553.6
30.01	-961.3	-7248.4	0.6	3393.7
30.31	-1018.0	-6424.3	0.5	3239.0
30.62	-1080.4	-5563.2	0.5	3058.7
30.93	-1142.8	-4752.7	0.4	2867.7
31.23	-1199.5	-4062.1	0.4	2684.8
31.54	-1261.8	-3356.0	0.4	2473.4
31.84	-1318.5	-2765.2	0.3	2271.9
32.15	-1380.9	-2174.2	0.3	2040.1
32.46	-1443.3	-1647.7	0.3	1797.6
32.76	-1500.0	-1227.6	0.2	1567.9
33.07	-1562.4	-832.4	0.2	1304.9
33.38	-1624.7	-510.3	0.1	1031.3
33.68	-1681.4	-283.3	0.1	773.3
33.99	-1743.8	-108.6	0.1	479.3
34.29	-1800.5	-20.5	0.0	202.7
34.60	-1862.9	0.0	0.0	0.0

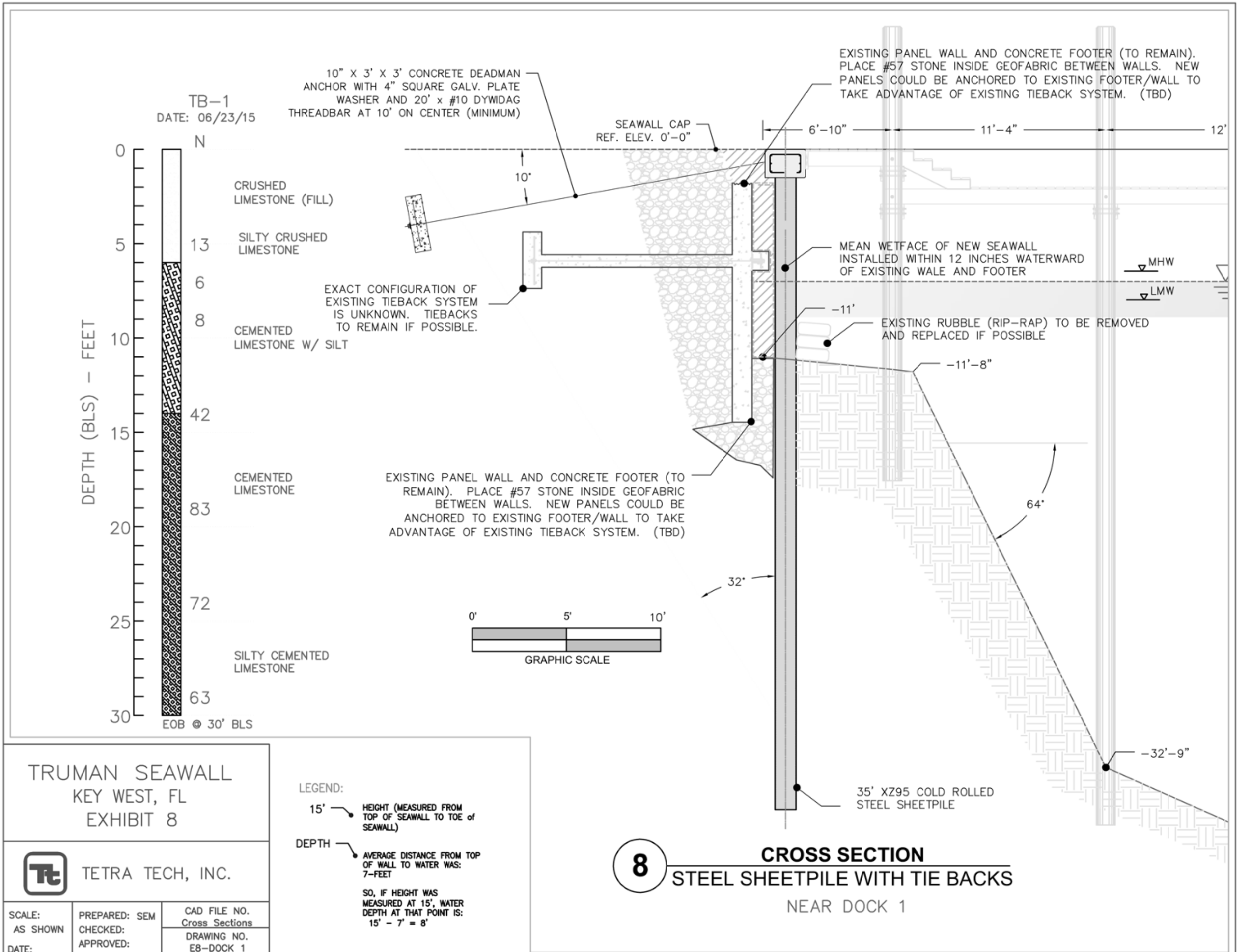
City of Key West

314- Flagler Ave.
Key West, FL 33040
Tel: (305) 809-3943
EMail: tjustice@cityofkeywest-fl.gov

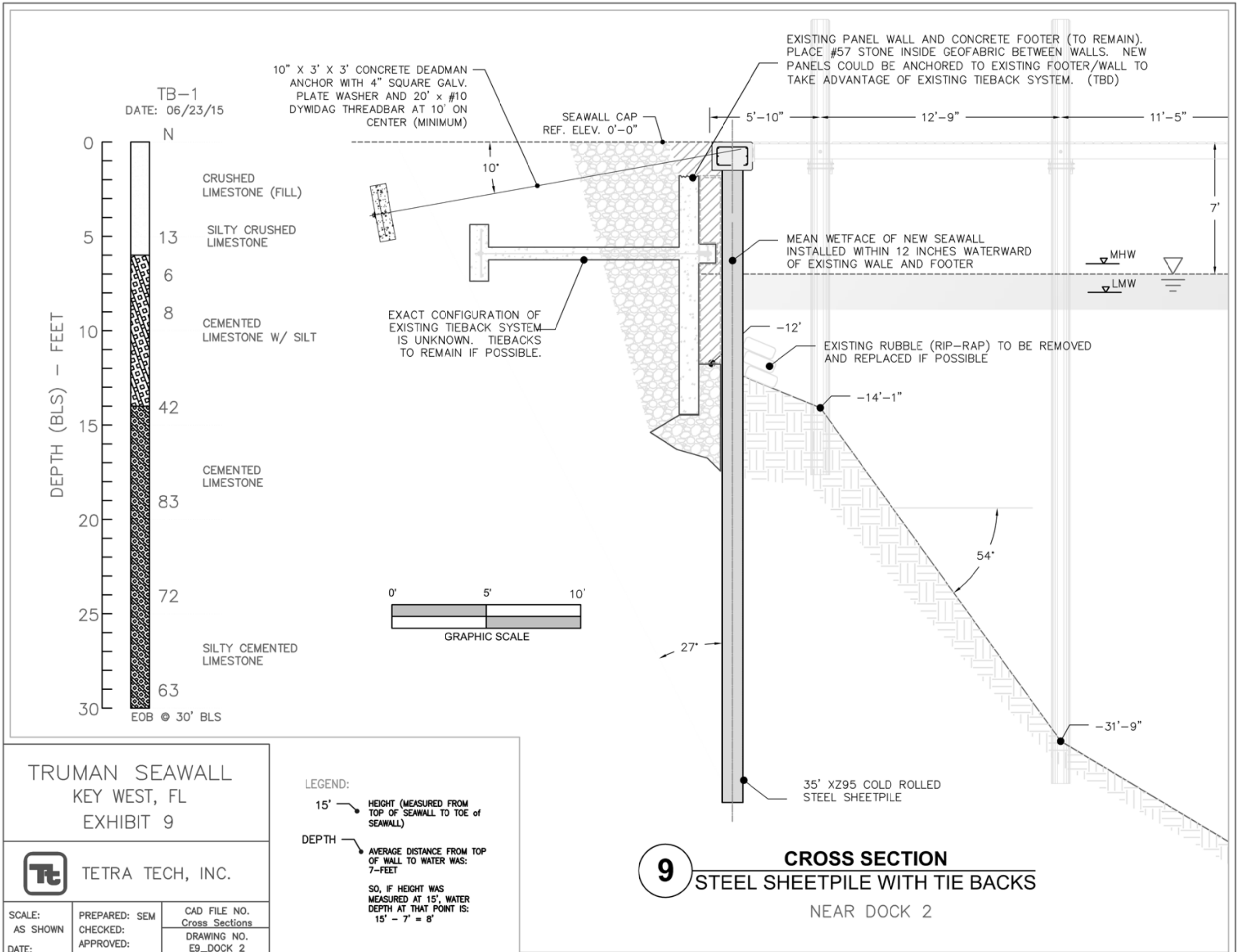
SPW911, v2.40

Exhibit 8-12

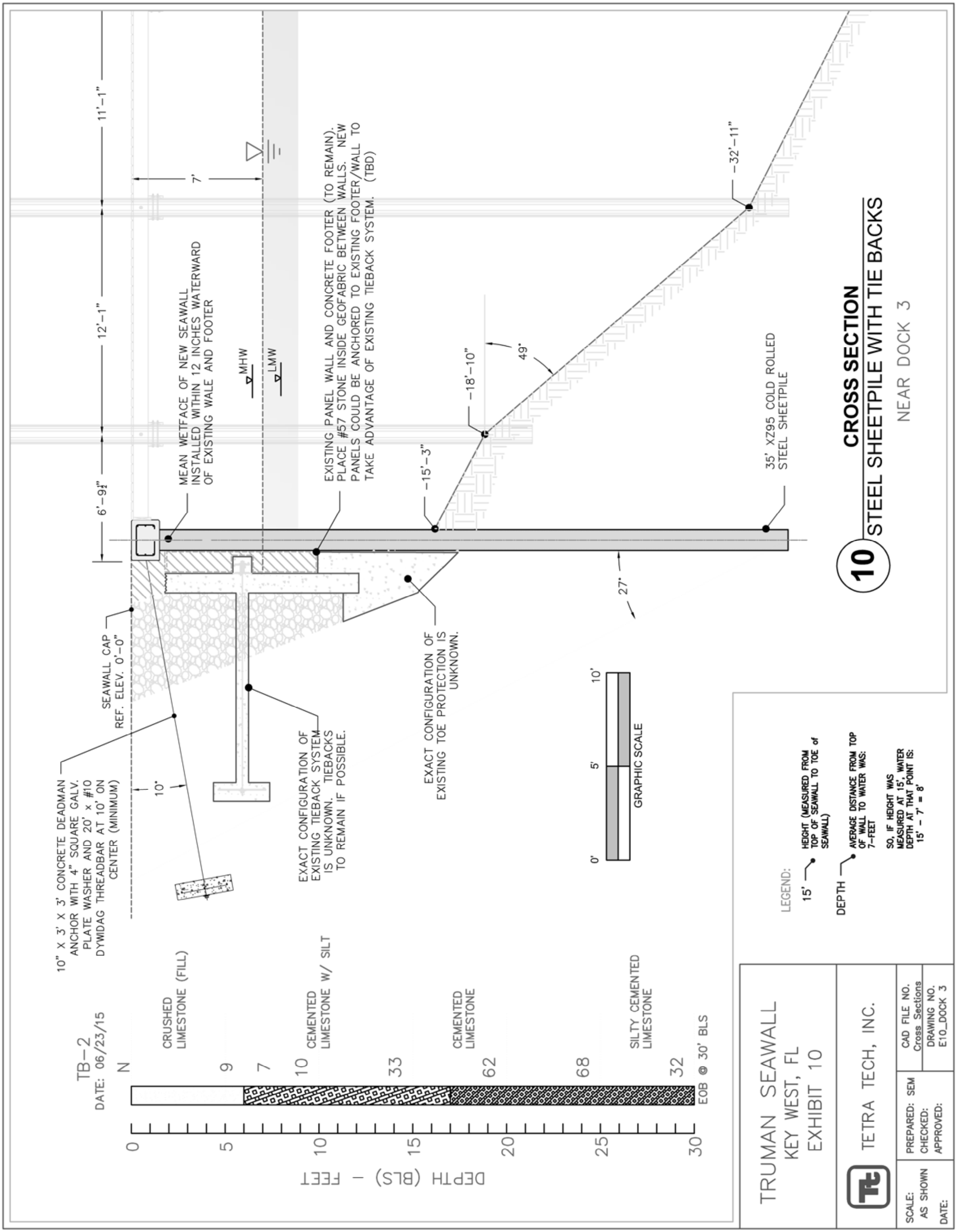
Wall Cross Sections



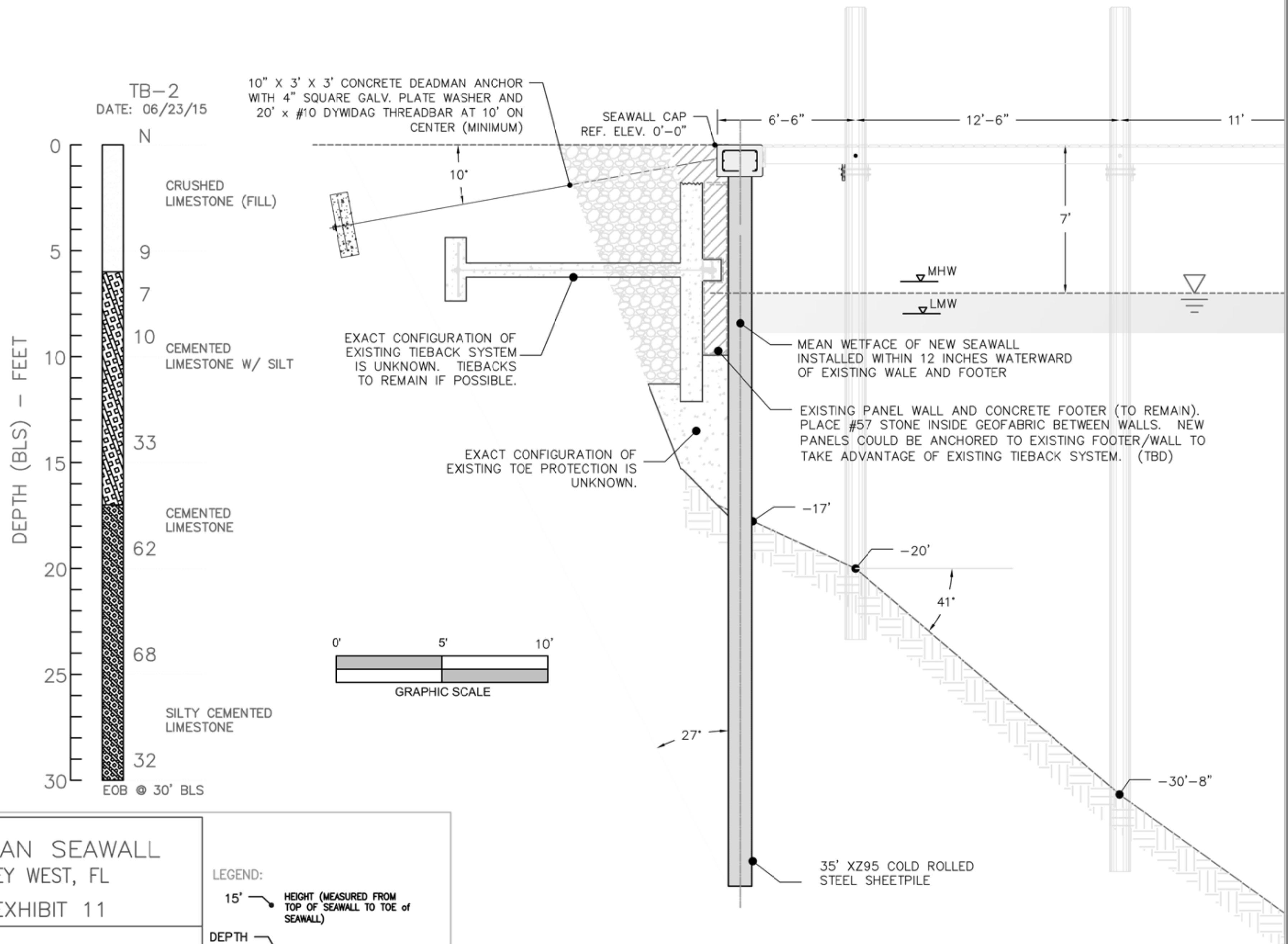
TB-1
DATE: 06/23/15
DEPTH (BLS) - FEET
0
5
10
15
20
25
30
N
13
6
8
42
83
72
63
EOB @ 30' BLS
CRUSHED LIMESTONE (FILL)
SILTY CRUSHED LIMESTONE
CEMENTED LIMESTONE W/ SILT
CEMENTED LIMESTONE
SILTY CEMENTED LIMESTONE
10" X 3' X 3' CONCRETE DEADMAN ANCHOR WITH 4" SQUARE GALV. PLATE WASHER AND 20' x #10 DYWIDAG THREADBAR AT 10' ON CENTER (MINIMUM)
10'
SEAWALL CAP REF. ELEV. 0'-0"
EXISTING PANEL WALL AND CONCRETE FOOTER (TO REMAIN). PLACE #57 STONE INSIDE GEOFABRIC BETWEEN WALLS. NEW PANELS COULD BE ANCHORED TO EXISTING FOOTER/WALL TO TAKE ADVANTAGE OF EXISTING TIEBACK SYSTEM. (TBD)
6'-10"
11'-4"
12'
EXACT CONFIGURATION OF EXISTING TIEBACK SYSTEM IS UNKNOWN. TIEBACKS TO REMAIN IF POSSIBLE.
MEAN WETFACE OF NEW SEAWALL INSTALLED WITHIN 12 INCHES WATERWARD OF EXISTING WALE AND FOOTER
MHW
LMW
-11'
EXISTING RUBBLE (RIP-RAP) TO BE REMOVED AND REPLACED IF POSSIBLE
-11'-8"
EXISTING PANEL WALL AND CONCRETE FOOTER (TO REMAIN). PLACE #57 STONE INSIDE GEOFABRIC BETWEEN WALLS. NEW PANELS COULD BE ANCHORED TO EXISTING FOOTER/WALL TO TAKE ADVANTAGE OF EXISTING TIEBACK SYSTEM. (TBD)
64°
32°
0'
5'
10'
GRAPHIC SCALE
-32'-9"
35' XZ95 COLD ROLLED STEEL SHEETPILE
8
CROSS SECTION
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 1
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' - 7' = 8'
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 8
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E8-DOCK 1



TB-1
DATE: 06/23/15
N
DEPTH (BLS) - FEET
0
5
10
15
20
25
30
13
6
8
42
83
72
63
CRUSHED LIMESTONE (FILL)
SILTY CRUSHED LIMESTONE
CEMENTED LIMESTONE W/ SILT
CEMENTED LIMESTONE
SILTY CEMENTED LIMESTONE
EOB @ 30' BLS
10" X 3' X 3' CONCRETE DEADMAN ANCHOR WITH 4" SQUARE GALV. PLATE WASHER AND 20' x #10 DYWIDAG THREADBAR AT 10' ON CENTER (MINIMUM)
10°
SEAWALL CAP REF. ELEV. 0'-0"
EXISTING PANEL WALL AND CONCRETE FOOTER (TO REMAIN). PLACE #57 STONE INSIDE GEOFABRIC BETWEEN WALLS. NEW PANELS COULD BE ANCHORED TO EXISTING FOOTER/WALL TO TAKE ADVANTAGE OF EXISTING TIEBACK SYSTEM. (TBD)
5'-10"
12'-9"
11'-5"
7'
MHW
LMW
MEAN WETFACE OF NEW SEAWALL INSTALLED WITHIN 12 INCHES WATERWARD OF EXISTING WALE AND FOOTER
EXACT CONFIGURATION OF EXISTING TIEBACK SYSTEM IS UNKNOWN. TIEBACKS TO REMAIN IF POSSIBLE.
-12'
EXISTING RUBBLE (RIP-RAP) TO BE REMOVED AND REPLACED IF POSSIBLE
-14'-1"
54°
27°
-31'-9"
35' XZ95 COLD ROLLED STEEL SHEETPILE
0'
5'
10'
GRAPHIC SCALE
9
CROSS SECTION
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 2
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' - 7' = 8'
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 9
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E9_DOCK 2



CROSS SECTION
10 STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 3
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 10
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E10_DOCK 3
TB-2
DATE: 06/23/15
N
CRUSHED LIMESTONE (FILL)
CEMENTED LIMESTONE W/ SILT
CEMENTED LIMESTONE
SILTY CEMENTED LIMESTONE
9
7
10
33
62
68
32
EOB @ 30' BLS
DEPTH (BLS) – FEET
0
5
10
15
20
25
30
10" X 3' X 3' CONCRETE DEADMAN ANCHOR WITH 4" SQUARE GALV. PLATE WASHER AND 20' x #10 DYWIDAG THREADBAR AT 10' ON CENTER (MINIMUM)
10'
SEAWALL CAP REF. ELEV. 0'-0"
EXACT CONFIGURATION OF EXISTING TIEBACK SYSTEM IS UNKNOWN. TIEBACKS TO REMAIN IF POSSIBLE.
EXACT CONFIGURATION OF EXISTING TOE PROTECTION IS UNKNOWN.
0'
5'
10'
GRAPHIC SCALE
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' – 7' = 8'
6'-9½"
12'-1"
11'-1"
7'
MEAN WETFACE OF NEW SEAWALL INSTALLED WITHIN 12 INCHES WATERWARD OF EXISTING WALE AND FOOTER
MHW
LMW
EXISTING PANEL WALL AND CONCRETE FOOTER (TO REMAIN). PLACE #57 STONE INSIDE GEOFABRIC BETWEEN WALLS. NEW PANELS COULD BE ANCHORED TO EXISTING FOOTER/WALL TO TAKE ADVANTAGE OF EXISTING TIEBACK SYSTEM. (TBD)
-15'-3"
-18'-10"
49°
-32'-11"
27°
35' XZ95 COLD ROLLED STEEL SHEETPILE



11 **CROSS SECTION**
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 4

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 11

TETRA TECH, INC.

SCALE: AS SHOWN	PREPARED: SEM	CAD FILE NO. Cross Sections
DATE:	CHECKED: APPROVED:	DRAWING NO. E11_DOCK 4

LEGEND:

15' — HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)

DEPTH — AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7−FEET

SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' − 7' = 8'

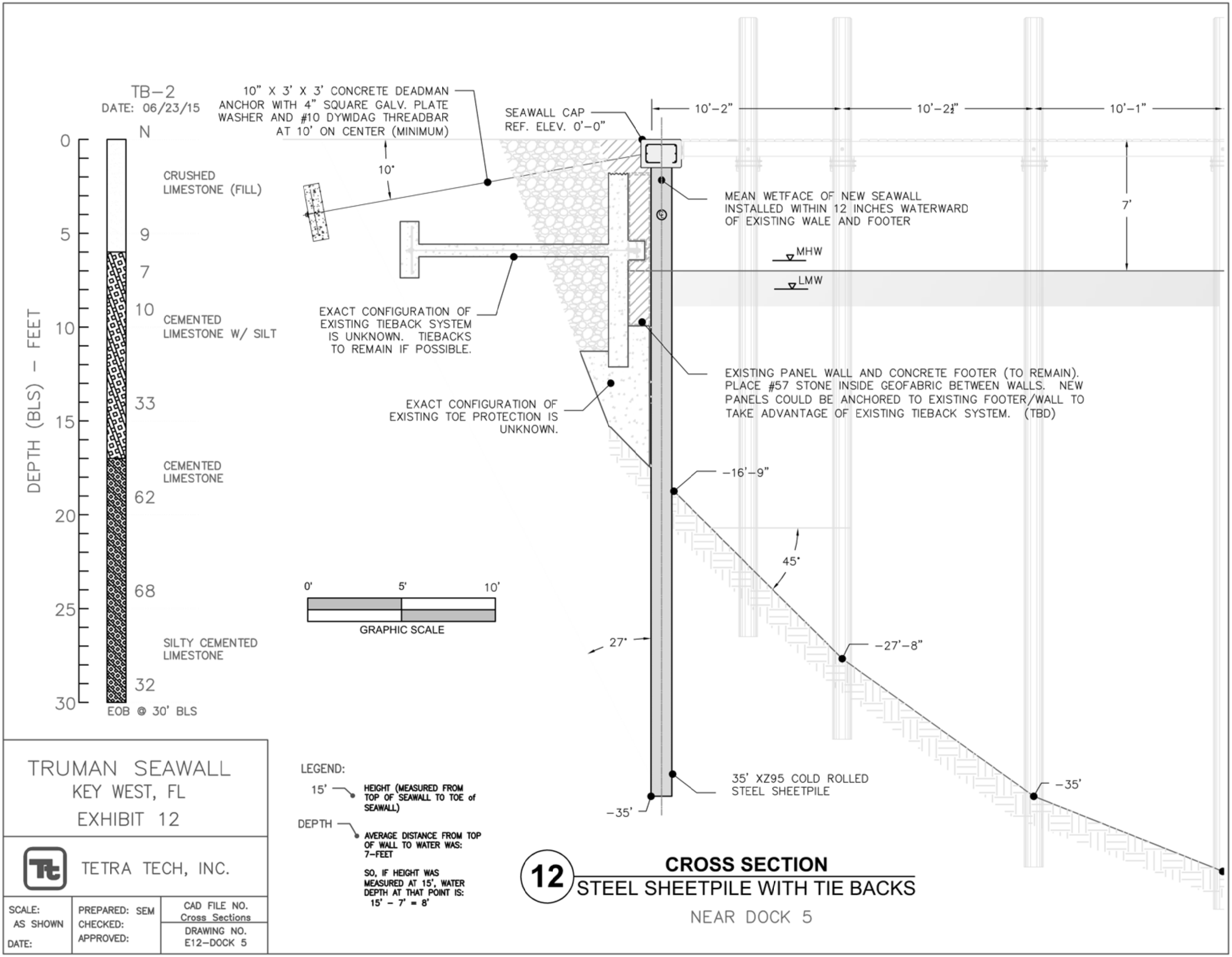
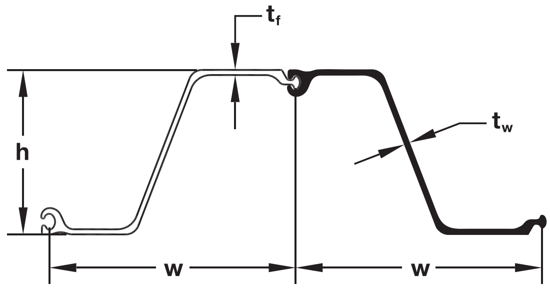

TB-2
DATE: 06/23/15
N
CRUSHED LIMESTONE (FILL)
9
7
10
CEMENTED LIMESTONE W/ SILT
33
CEMENTED LIMESTONE
62
68
SILTY CEMENTED LIMESTONE
32
EOB @ 30' BLS
DEPTH (BLS) – FEET
0
5
10
15
20
25
30
10" X 3' X 3' CONCRETE DEADMAN ANCHOR WITH 4" SQUARE GALV. PLATE WASHER AND #10 DYWIDAG THREADBAR AT 10' ON CENTER (MINIMUM)
10'
SEAWALL CAP REF. ELEV. 0'-0"
10'-2"
10'-2½"
10'-1"
7'
MEAN WETFACE OF NEW SEAWALL INSTALLED WITHIN 12 INCHES WATERWARD OF EXISTING WALE AND FOOTER
MHW
LMW
EXACT CONFIGURATION OF EXISTING TIEBACK SYSTEM IS UNKNOWN. TIEBACKS TO REMAIN IF POSSIBLE.
EXISTING PANEL WALL AND CONCRETE FOOTER (TO REMAIN). PLACE #57 STONE INSIDE GEOFABRIC BETWEEN WALLS. NEW PANELS COULD BE ANCHORED TO EXISTING FOOTER/WALL TO TAKE ADVANTAGE OF EXISTING TIEBACK SYSTEM. (TBD)
EXACT CONFIGURATION OF EXISTING TOE PROTECTION IS UNKNOWN.
-16'-9"
45°
0'
5'
10'
GRAPHIC SCALE
27'
-27'-8"
35' XZ95 COLD ROLLED STEEL SHEETPILE
-35'
-35'
LEGEND:
15' HEIGHT (MEASURED FROM TOP OF SEAWALL TO TOE of SEAWALL)
DEPTH AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET
SO, IF HEIGHT WAS MEASURED AT 15', WATER DEPTH AT THAT POINT IS: 15' - 7' = 8'
12
CROSS SECTION
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 5
TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 12
TETRA TECH, INC.
SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. E12-DOCK 5

Exhibit 13

Product Specifications

PZ/PS

PZ/PS Hot Rolled Steel Sheet Pile



PZ

SECTION	Width (w) in (mm)	Height (h) in (mm)	THICKNESS: Flange (t_f) in (mm)	THICKNESS: Wall (t_w) in (mm)	Cross Sectional Area in^2/ft (cm^2/m)	WEIGHT: Pile lb/ft (kg/m)	WEIGHT: Wall lb/ft^2 (kg/m^2)	SECTION MODULUS: Elastic in^3/ft (cm^3/m)	SECTION MODULUS: Plastic in^3/ft (cm^3/m)	Moment of Inertia in^4/ft (cm^4/m)	COATING AREA: Both Sides ft^2/ft of single (m^2/m)	COATING AREA: Wall Surface ft^2/ft^2 of wall (m^2/m^2)
PZ 22	22.0 559	9.0 229	0.375 9.50	0.375 9.50	6.47 136.9	40.3 60.0	22.0 107.4	18.1 973	21.79 1171.4	84.38 11500	4.48 1.37	1.22 1.22
PZ 27	18.0 457	12.0 305	0.375 9.50	0.375 9.50	7.94 168.1	40.5 60.3	27.0 131.8	30.2 1620	36.49 1961.9	184.20 25200	4.48 1.37	1.49 1.49
PZ 35	22.6 575	14.9 378	0.600 15.21	0.500 12.67	10.29 217.8	66.0 98.2	35.0 170.9	48.5 2608	57.17 3073.5	361.22 49300	5.37 1.64	1.42 1.42
PZ 40	19.7 500	16.1 409	0.600 15.21	0.500 12.67	11.77 249.1	65.6 97.6	40.0 195.3	60.7 3263	71.92 3866.7	490.85 67000	5.37 1.64	1.64 1.64

PS

SECTION	Width (w) in (mm)	Web (t_w) in (mm)	Maximum Interlock Strength k/in (kN/m)	Minimum Cell Diameter* ft (m)	Cross Sectional Area in^2/ft (cm^2/m)	WEIGHT: Pile lb/ft (kg/m)	WEIGHT: Wall lb/ft^2 (kg/m^2)	Elastic Section Modulus in^3/sheet (cm^3/sheet)	Moment of Inertia in^4/sheet (cm^4/sheet)	COATING AREA: Both Sides ft^2/ft of single (m^2/m)	COATING AREA: Wall Surface ft^2/ft^2 of wall (m^2/m^2)
PS 27.5	19.69 500	0.4 10.2	20 3500	30 9.14	8.09 171.2	45.1 67.1	27.5 134.3	3.3 54	5.3 221	3.65 1.11	1.11 1.11
PS 31	19.69 500	0.5 12.7	20 3500	30 9.14	9.12 193.0	50.9 75.7	31.0 151.4	3.3 54	5.3 221	3.65 1.11	1.11 1.11

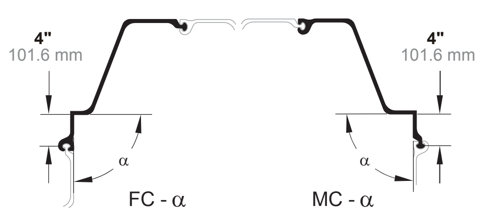
* Minimum cell diameter cannot be guaranteed for piles over 65 feet (19.81 m) in length, or if piles are spliced. 58 Piles are needed to make a 30 foot diameter cell.

PZ/PS

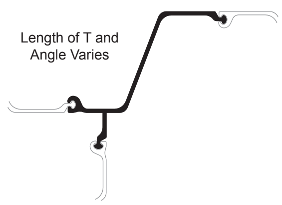
PZ/PS Hot Rolled Steel Sheet Pile

Available Steel Grades						
ASTM	PZ		PS			
	YIELD STRENGTH		YIELD STRENGTH		INTERLOCK STRENGTH	
	(ksi)	(MPa)	(ksi)	(MPa)	(k/in)	(kN/m)
A 328	39	270	39	270	16	2800
A 572 Grade 50	50	345	50	345	20	3500
A 572 Grade 60	60	415	-	-	-	-
A 588	50	345	50	345	20	3500
A 690	50	345	50	345	20	3500

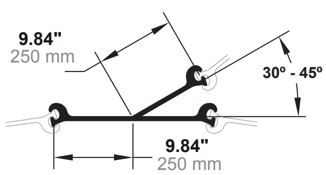
Corner and Junction Piles



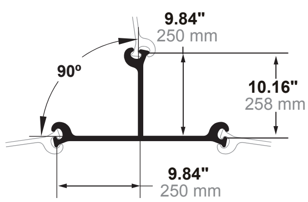
Female or Male Corner



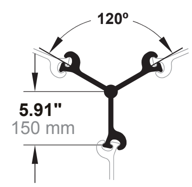
T Pile



30° Y Pile



90° T Pile



120° Y Pile

Delivery Conditions & Tolerances

	ASTM A 6	
Mass	± 2.5%	
Length	+ 5 inches	– 0 inches

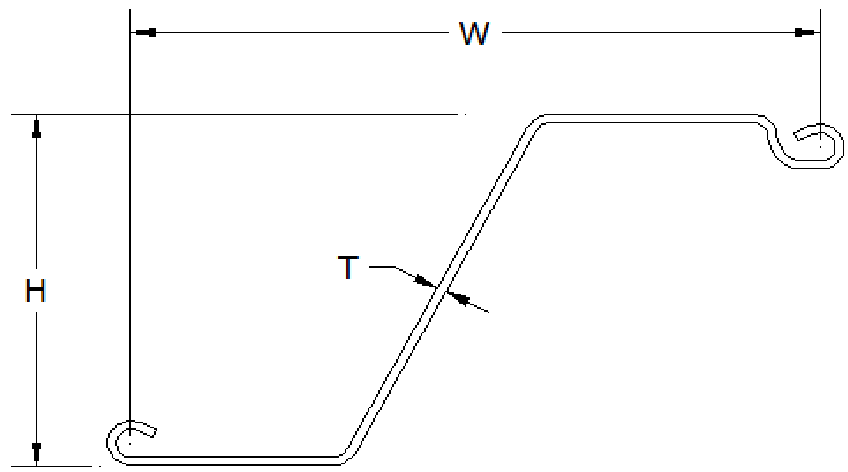
Maximum Rolled Lengths*

PZ	85 feet for singles, 70 feet for pairs	(25.9 m, 21.3 m)
PS	65 feet	(19.8 m)

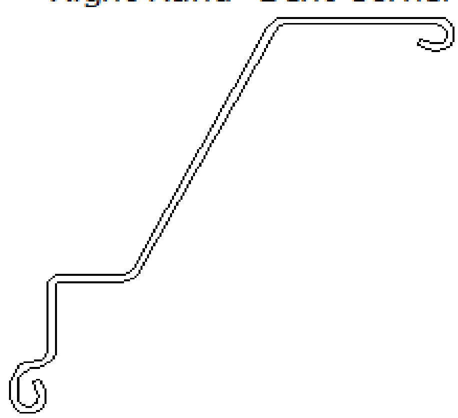
* Longer lengths may be possible upon request.

Piling Products, a division of
Roll Form Group (U.S.), Inc.

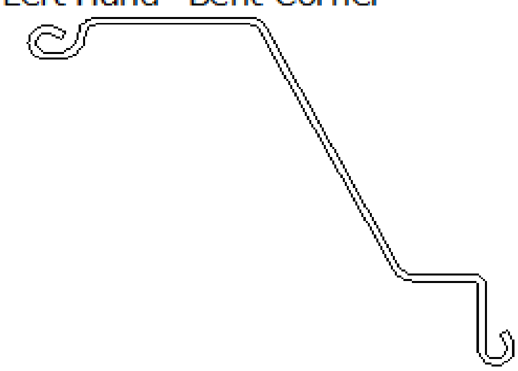
XZ-95 Steel Sheet Piling



"Right Hand" Bent Corner



"Left Hand" Bent Corner



ASTM A572 Grade 50 Steel **Melted and Manufactured in the US.**

Section	Width W in mm	Height H in mm	Thickness T in mm	Cross Sectional Area in^2/ft cm^2/m	Weight Pile lb/ft kg/m	Weight Wall lb/ft^2 kg/m^2	Section Modulus in^3/ft cm^3/m	Moment of Inertia in^4/ft cm^4/m	Coating Area Both Sides ft^2/ft m^2/m
XZ-95	25.00 635	14.12 359	0.375 9.50	15.20 98.2	51.70 76.9	24.80 121	33.50 1800	237.00 32400	6.03 1.84

Piling Products, a division of Roll Form Group (U.S.), Inc.
945 Center Street * Green Cove Springs * Florida * 32043
(904) 287-8000 Fax (904) 529-7757
sales@pilingproducts.com www.pilingproducts.com

DYWIDAG Tie Rods

DYWIDAG Tie Rods for Military Wharf in Guam

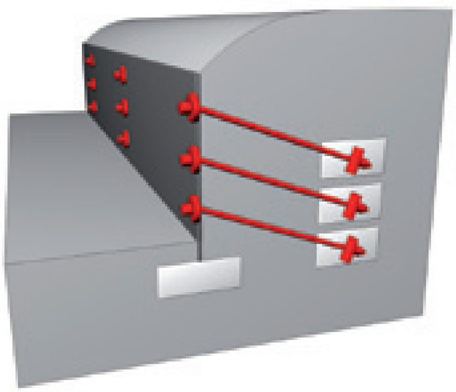
Owner US Navy, Naval Base Guam, USA +++ **General Contractor** Black Construction Corporation, Guam, USA +++ **Architect-Engineer** Wilson Okamoto Corporation, Honolulu, Hawaii, USA

DSI Unit DSI USA, BU Geotechnics, Long Beach, CA, USA

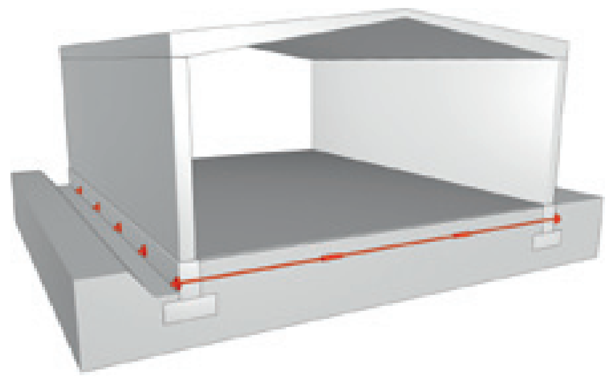
DSI Scope Supply of approx. 8,000 m (26,000 ft.) of Tie Rods with Polyken tape wrap

Contents

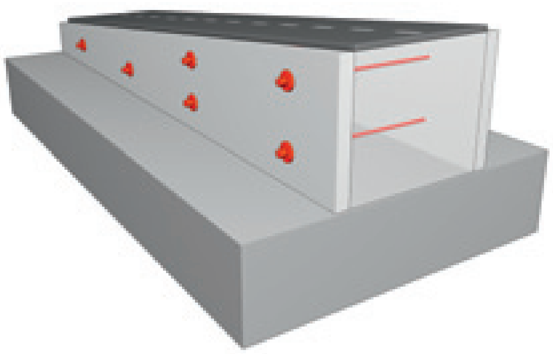
TYPICAL APPLICATIONS



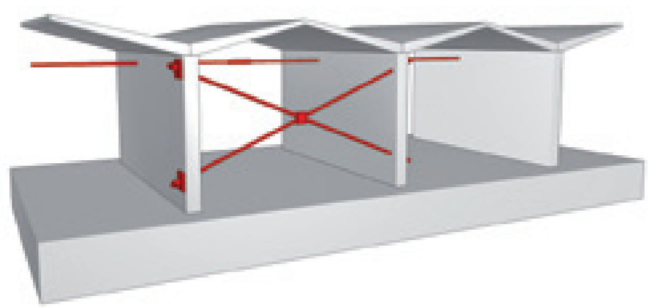
Tie Back Retaining Wall
with Deadman



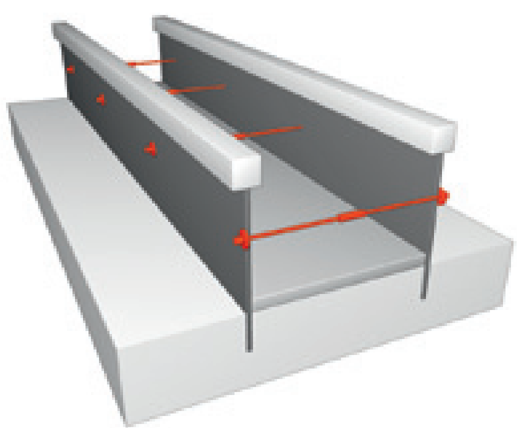
Foundation Tie Rods



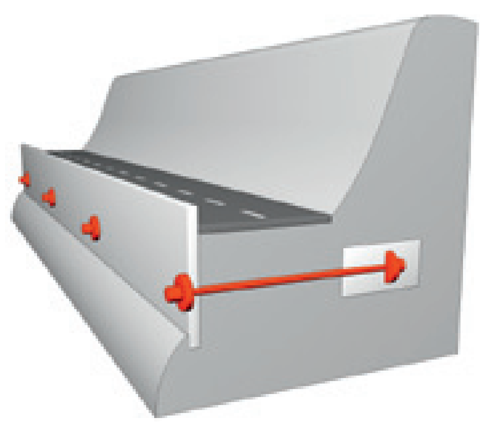
Bracing for Grade Separation



Large Horizontal Load Resistance
Roof Structures



Tie Rods for Sheet Pile
Constructed Wharf

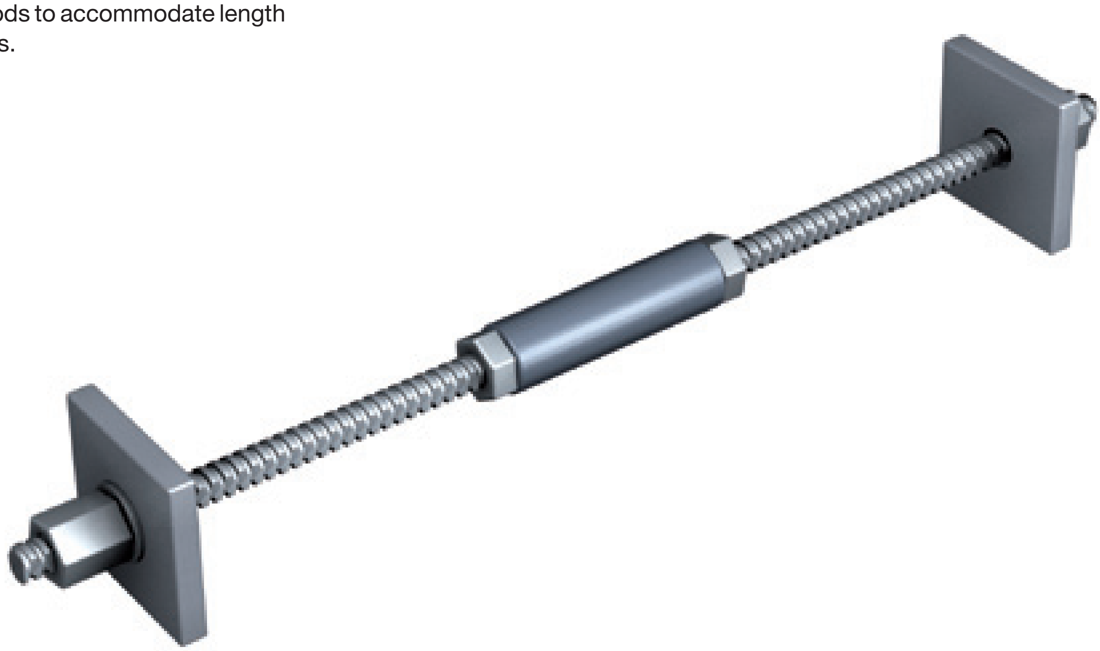


Roadway Embankment
Stabilization

Tie Rods produced from DYWIDAG THREADBAR® may be used for a variety of marine applications. Construction of marine bulkheads for various docking facilities have, for many years, benefited from the use of DYWIDAG Tie Rods. Facilities such as barge and ship docks as well as offshore platforms have found the system to be a cost effective alternative to large diameter A36 Tie Rods with upset threads.

Tie Rods produced from DYWIDAG THREADBAR® offer various economical and technical advantages:

- Easy handling due to the coarse thread over the entire length
- Up to almost 50% less weight than A36 bars.
- No threadability or assembly problems resulting from damaged or dirty threads, because of the rugged hot rolled coarse DYWIDAG threadform.
- DYWIDAG Tie Rods can be ordered slightly longer than necessary to accommodate misalignments on sheet piling. They can easily be cut to the desired length eliminating need to cut and re-weld upset rods to accommodate length changes.



DYWIDAG THREADBAR® Tie Rod Properties

DYWIDAG Grade 75 (ASTM A615) THREADBAR®

THREADBAR® Designation		Maximum THREADBAR® Diameter		Yield Stress (fy)		Cross Section Area (As)		Yield Load (fy x As)		Nominal Weight	
[in]	[mm]	[in]	[mm]	[ksi]	[MPa]	[in²]	[mm²]	[kips]	[kN]	[lbs/ft]	[kg/m]
#6	19	0.86	22	75	517	0.44	284	33.0	147	1.50	2.23
#7	22	0.99	25	75	517	0.60	387	45.0	200	2.04	3.04
#8	25	1.12	28	75	517	0.79	510	59.3	264	2.67	3.97
#9	29	1.26	32	75	517	1.00	645	75.0	334	3.40	5.06
#10	32	1.43	36	75	517	1.27	819	95.3	424	4.30	6.40
#11	36	1.61	41	75	517	1.56	1,006	117.0	520	5.31	7.90
#14	43	1.86	47	75	517	2.25	1,452	168.8	751	7.65	11.38
#18	57	2.50	64	75	517	4.00	2,581	300.0	1,335	13.60	20.24
#20	63	2.72	69	80	552	4.91	3,168	393.0	1,748	16.91	25.16
#24	75	3.18	81	75	517	7.06	4,555	529.5	2,355	24.09	35.85
#28	90	3.68	94	75	517	9.62	6,207	721.5	3,209	32.79	48.79

Warning: Avoid Welding near A722 steel.
Note: Mill length = 60'-0" for #6 through #24 bars and 48'-0" for #28 bars

DYWIDAG Grade 150 (ASTM A722) THREADBAR®

THREADBAR® Designation		Maximum THREADBAR® Diameter		Ultimate Stress (fu)		Cross Section Area (As)		Ultimate Load (fu x As)		Nominal Weight	
	[mm]	[in]	[mm]	[ksi]	[MPa]	[in²]	[mm²]	[kips]	[kN]	[lbs/ft]	[kg/m]
1"	26	1.20	31	150	1,034	0.85	548	127.5	567	3.01	4.48
1-1/4"	32	1.44	36	150	1,034	1.25	806	187.5	834	4.39	6.53
1-3/8"	36	1.63	41	150	1,034	1.58	1,019	237.0	1,054	5.56	8.27
* 1-3/4"	46	2.01	51	155	1,069	2.58	1,664	400.0	1,779	9.22	13.72
* 2-1/2"	66	2.79	71	150	1,034	5.16	3,355	774.0	3,443	18.20	26.36
* 3"	75	3.15	80	150	1,034	6.85	4,419	1,027.0	4,568	24.09	35.85

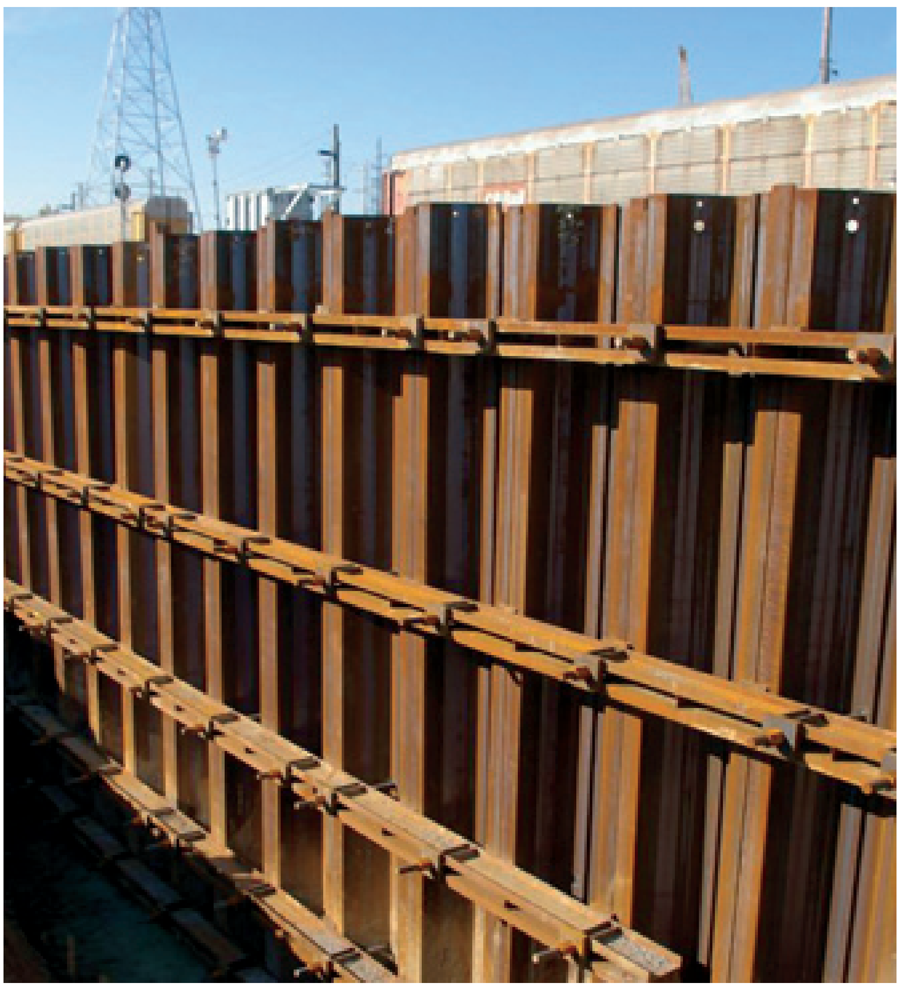
* Meets the strength requirements of the A 722.
Warning: Avoid Welding near A722 steel.
Note: Mill length = 60'-0" for 1", 1¼" and 1 ⅜ " Threadbars and 45'-0" for 1¾", 2½" and 3" bars

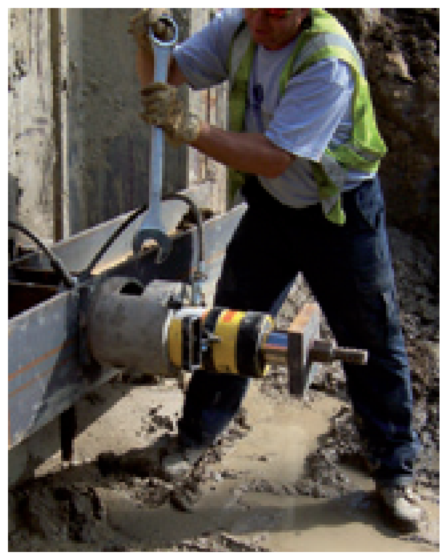
Coupler and hexnut develop the full load of the bar ultimate load. Standard or custom made wedge washers are available for all sizes. Bearing plates can be custom made in any size and from steel material conforming to ASTM A36 or ASTM A572 grade 50. All bars and accessories can be produced with double corrosion protection, hot-dip galvanized, coal-tar or fusion bonded epoxy coated.

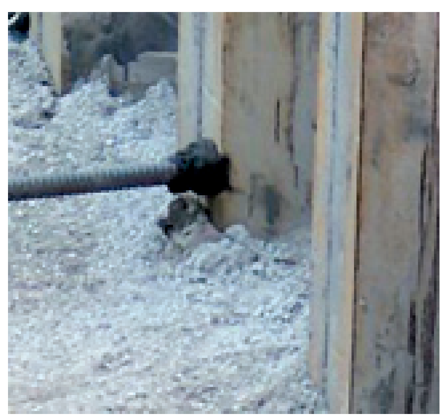
Replacing A36 Round Bars with Threaded Ends to Equivalent DYWIDAG THREADBAR®

A36								DYWIDAG THREADBAR® ASTM A615				
A36 BAR		Tensile Stress Area*		Yield* Strength		Nominal Weight		Equivalent THREADBAR® Size	Yield Load (fy x As)		Nominal Weight	
[inches]	[mm]	[in²]	[mm²]	[kips]	[kN]	[lbs/ft]	[kg/m]	[mm/designation]	[kips]	[kN]	[lbs/ft]	[kg/m]
3/4	19	0.33	215	12.0	53	1.50	2.2	19 (#6) Grade 75	33.0	147	1.50	2.23
1	25	0.61	391	21.8	97	2.67	4.0	19 (#6) Grade 75	33.0	147	1.50	2.23
1-1/4	32	0.97	625	34.8	155	4.17	6.2	22 (#7) Grade 75	45.0	200	2.04	3.04
1-1/2	38	1.41	910	50.7	226	6.01	8.9	25 (#8) Grade 75	59.3	264	2.67	3.97
1-3/4	44	1.90	1,226	68.4	304	8.18	12.2	29 (#9) Grade 75	75.0	334	3.40	5.06
2	51	2.50	1,613	90.0	400	10.68	15.9	32 (#10) Grade 75	95.3	424	4.30	6.40
2-1/4	57	3.25	2,097	117.0	520	13.52	20.1	36 (#11) Grade 75	117.0	520	5.31	7.90
2-1/2	64	4.00	2,581	144.0	641	16.69	24.8	43 (#14) Grade 75	168.8	751	7.65	11.38
2-3/4	70	4.93	3,181	177.0	787	20.19	30.1	57 (#18) Grade 75	300.0	1,335	13.60	20.24
3	76	5.97	3,852	214.9	956	24.03	35.8	57 (#18) Grade 75	300.0	1,335	13.60	20.24
3-1/4	83	7.10	4,581	255.6	1,137	28.21	42.0	57 (#18) Grade 75	300.0	1,335	13.60	20.24
3-1/2	89	8.33	5,374	299.9	1,334	32.71	48.7	57 (#18) Grade 75	300.0	1,335	13.60	20.24
3-3/4	95	10.11	6,523	364.0	1,620	37.55	55.9	63 (#20) Grade 80	393.0	1,748	16.70	24.85

* Round bar with thread cut into bar. For comparison with upset ended tie rods use area of rod x 36 ksi to get yield strength







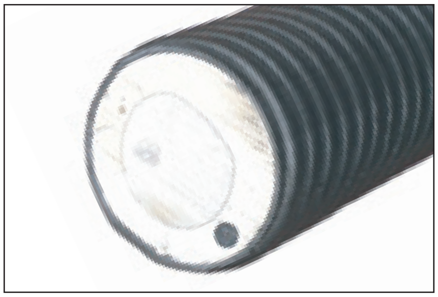
DYWIDAG Tie Rod Corrosion Protection

An important element in the long term durability of a Tie Rod system is corrosion protection. DSI offers various corrosion protection systems to meet on site conditions and degree of exposure.

Possible solutions are:

- The well known DYWIDAG Double Corrosion Protection (DCP) System. Ideal for the most aggressive environments
- Hot dip galvanizing in accordance with ASTM A153
- Fusion bonded epoxy coating per ASTM A775 or ATSM A934
- Coal tar epoxy
- Petroleum wax tape wrapping
- Polyken 980/955 tape coating system

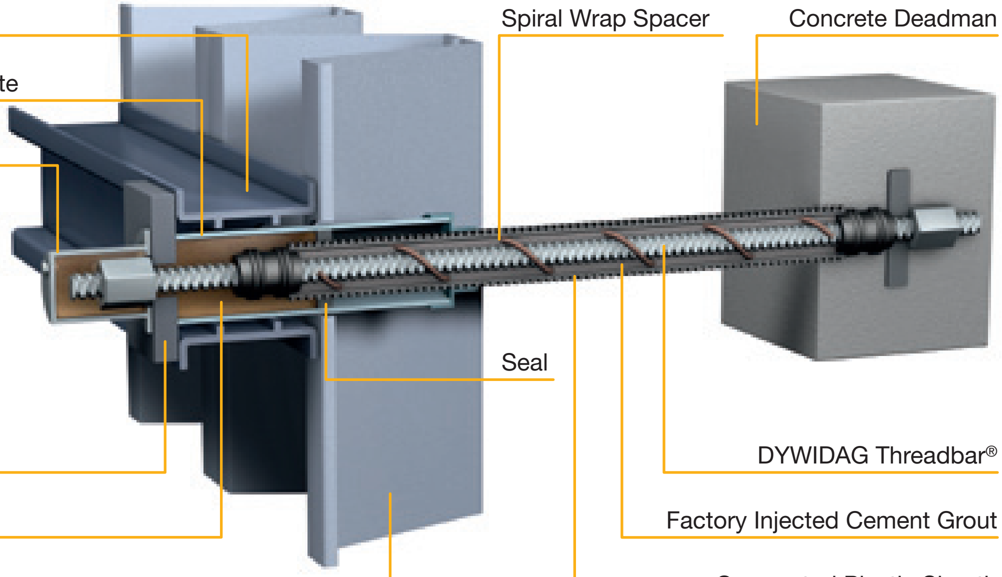
DCP THREADBAR® - exposed section



DCP THREADBAR® - cross section area

Typical Permanent Tie Rod with Double Corrosion Protection (DCP)

(Highly Recommended for Permanent Applications in Agressive Enviroments)



Short Tie Rod Connection

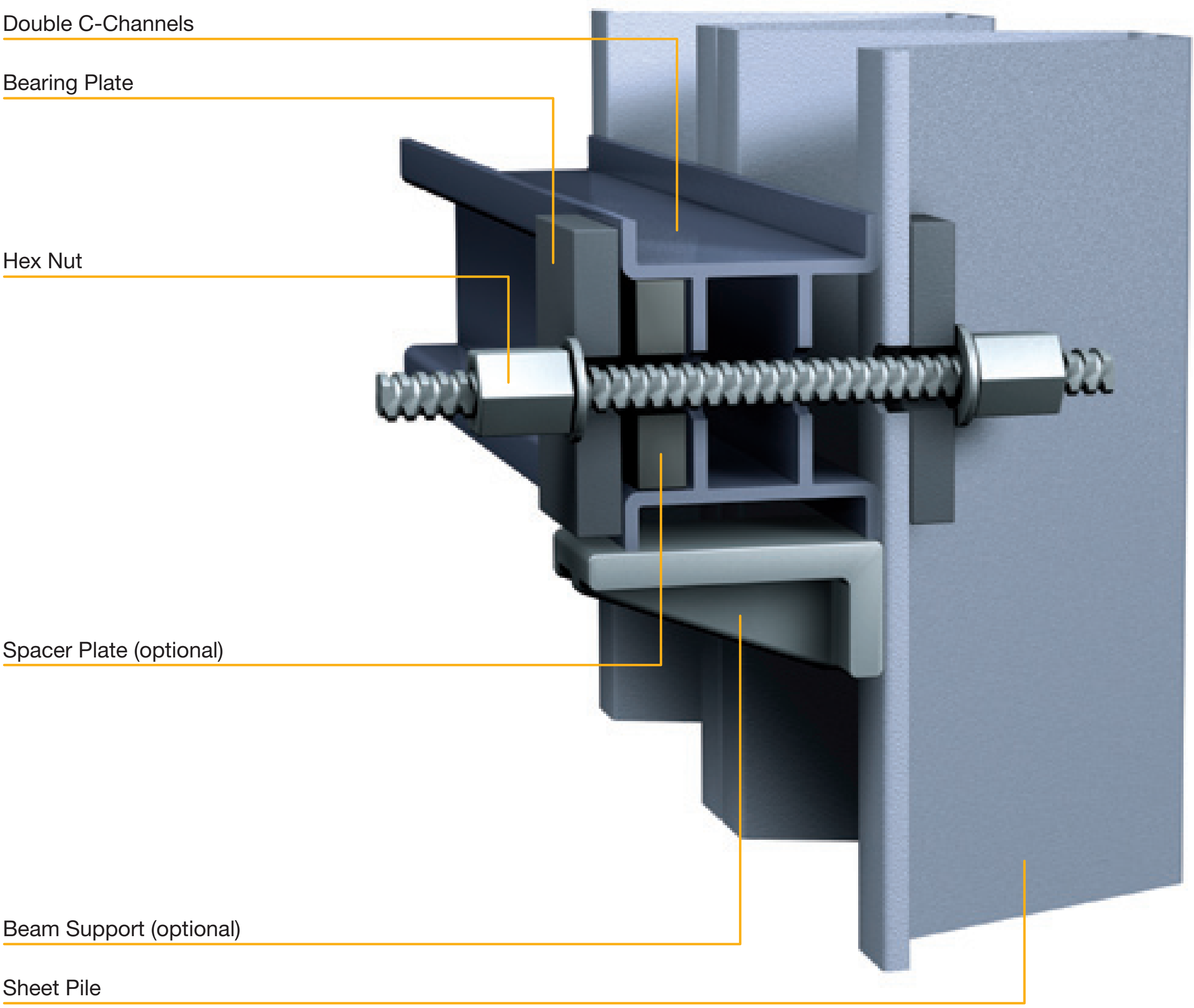
Short bolts are needed to connect a standard sheet pile wall or a modular sheet pile wall to a load distributing double channel beam. This can be done with DYWIDAG THREADBAR®.

The required length of the bar depends on the height of the sheet pile profile, the width of the beam, the plate thickness and the nut length.

DYWIDAG Tie Rods can also be prestressed, to align sheet pile walls.

Features and advantages:

- Continuous coarse DYWIDAG thread
- Can be cut to any length
- Weldable accessories
- Wide range of accessories available
- Available with double corrosion protection system
- Approved by many construction authorities worldwide
- Conformance to ASTM A615 and ASTM A722
- Stock lengths up to 60 ft, but can be cut to any lengths.



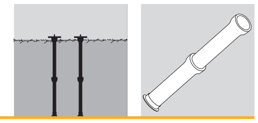
References

Walker Road Canadian Pacific Railway
Temporary Railroad Diversion Structure in Windsor, Ontario

Owner Canadian Pacific Railway +++ **General Contractor** FACCA Inc. 747 County Road, Ruscom, Ontario N0R 1R0 +++
Engineer MTO Contract 2007-3042 Walker Road CPR Grade Separation, Dillon Consulting / Facca Incorporated
DSI Unit DSI Canada Ltd., Eastern Division, Gormley, Canada
SUSPA-DSI Scope Supply of 1000 m of 63 mm diameter *GEWI*® Threadbar

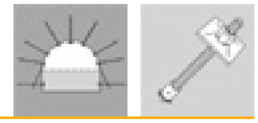
Additional Geotechnical Products from DSI

(Downloadable versions available at www.dsiamerica.com)

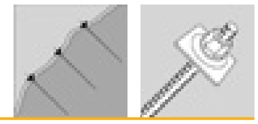


DYWIDAG Driven Ductile Iron Pile

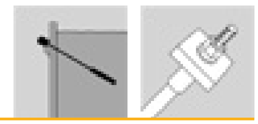
DYNA Force™ Elasto Magnetic Sensor



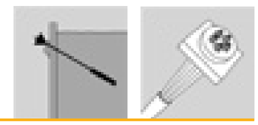
DYWI® Drill Hollow Bar Systems



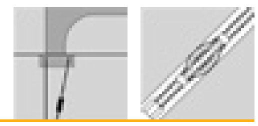
DYWIDAG Soil Nails



DYWIDAG Bar Anchor Systems



DYWIDAG Strand Anchor Systems



DYWIDAG Micropiles

DYWIDAG-Systems International USA Inc.

320 Marmon Drive
Bolingbrook, IL 60440
Phone (630) 739-1100
Fax (630) 739-5517
E-mail dsiamerica@dsiamerica.com

1591 E. Atlantic Blvd #200
Pompano Beach, FL 33060
Phone (954) 532-1326
Fax (954) 532-1330
E-mail dsiamerica@dsiamerica.com

5139 South Royal Atlanta Drive
Tucker, GA 30084
Phone (770) 491-3790
Fax (770) 938-1219
E-mail dsiamerica@dsiamerica.com

2400 Hwy 287 N.
Suite 106
Mansfield, TX 76063
Phone (817) 473-6161
Fax (817) 473-1453
E-mail dsiamerica@dsiamerica.com

2154 South Street
Long Beach, CA 90805
Phone (562) 531-6161
Fax (562) 531-3266
E-mail dsiamerica@dsiamerica.com

1314 Central Ave South
Suite 100
Kent, WA 98032
Phone (253) 859-9995
Fax (253) 859-9119
E-mail dsiamerica@dsiamerica.com

1263 Newark Road
Toughkenamon, PA 19374
Phone (610) 268-2221
Fax (610) 268-3053
E-mail dsiamerica@dsiamerica.com

DYWIDAG-Systems International Canada Ltd.

Eastern Division
37 Cardico Drive
Gormley, ON L0H 1G0
Phone (905) 888-8988
Fax (905) 888-8987
E-mail ecd@dsiamerica.com

Quebec Office
C.P. 412
St. Bruno,
Quebec, QC, J3V 5G8
Phone (450) 653-0935
Fax (450) 653-0977
E-mail ecd@dsiamerica.com

Western Division
19433 96th Avenue
Suite 103
Surrey, BC V4N 4C4
Phone (604) 888-8818
Fax (604) 888-5008
E-mail wcd@dsiamerica.com

Calgary Office
2816 - 21st Street NE., #204
Calgary, Alberta T2E 6Z2
Phone (403) 291-4414
Fax (403) 250-5221
E-mail wcd@dsiamerica.com

ARGENTINA
AUSTRALIA
AUSTRIA
BELGIUM
BOSNIA AND HERZEGOVINA
BRAZIL
CANADA
CHILE
CHINA
COLOMBIA
COSTA RICA
CROATIA
CZECH REPUBLIC
DENMARK
EGYPT
ESTONIA
FINLAND
FRANCE
GERMANY
GREECE
GUATEMALA
HONDURAS
HONG KONG
INDONESIA
ITALY
JAPAN
KOREA
LEBANON
LUXEMBOURG
MALAYSIA
MEXICO
NETHERLANDS
NORWAY
OMAN
PANAMA
PARAGUAY
PERU
POLAND
PORTUGAL
QATAR
RUSSIA
SAUDI ARABIA
SINGAPORE
SOUTH AFRICA
SPAIN
SWEDEN
SWITZERLAND
TAIWAN
THAILAND
TURKEY
UNITED ARAB EMIRATES
UNITED KINGDOM
URUGUAY
USA
VENEZUELA

Please note:
This brochure serves basic information purposes only. Technical data and information provided herein shall be considered non-binding and may be subject to change without notice. We do not assume any liability for losses or damages attributed to the use of this technical data and any improper use of our products. Should you require further information on particular products, please do not hesitate to contact us.

www.dsiamerica.com
www.dsicanada.ca

04309-1US/10.12-web ud

Exhibit 14

Project Quotes

Stable Soils
4265 Northwest 44th Ave., Suite 7, Ocala, Florida 34482
PHONE: 352.433.2323 • TOLL FREE: 866.622.3723 • FAX: 352.732.5459
www.StableSoils.com
The Poly Grout Professionals

HERCU-LIFT
EARTH LIFTING TECHNOLOGY

August 3, 2015

Customer: Stuart McGahee
Truman Seawall, Key West, FL
759 S. Federal Highway, Suite 314
Stuart, FL 34994

Site / Project: Florida Keys National Marine Sanctuary
Seawall Repair - Chemical Grout – Void Fill / Soil Stabilization

PROPOSAL/PROJECT INFORMATION

Stable Soils of Florida, Inc. is pleased to present this proposal for void fill and soil stabilization behind your seawall by chemical grouting the settled areas at the above referenced location. On July 31, 2015 a representative from our company performed a visual site inspection of your property to observe the issues and concerns with your seawall.

Following you will find only a few of the benefits we have to offer:

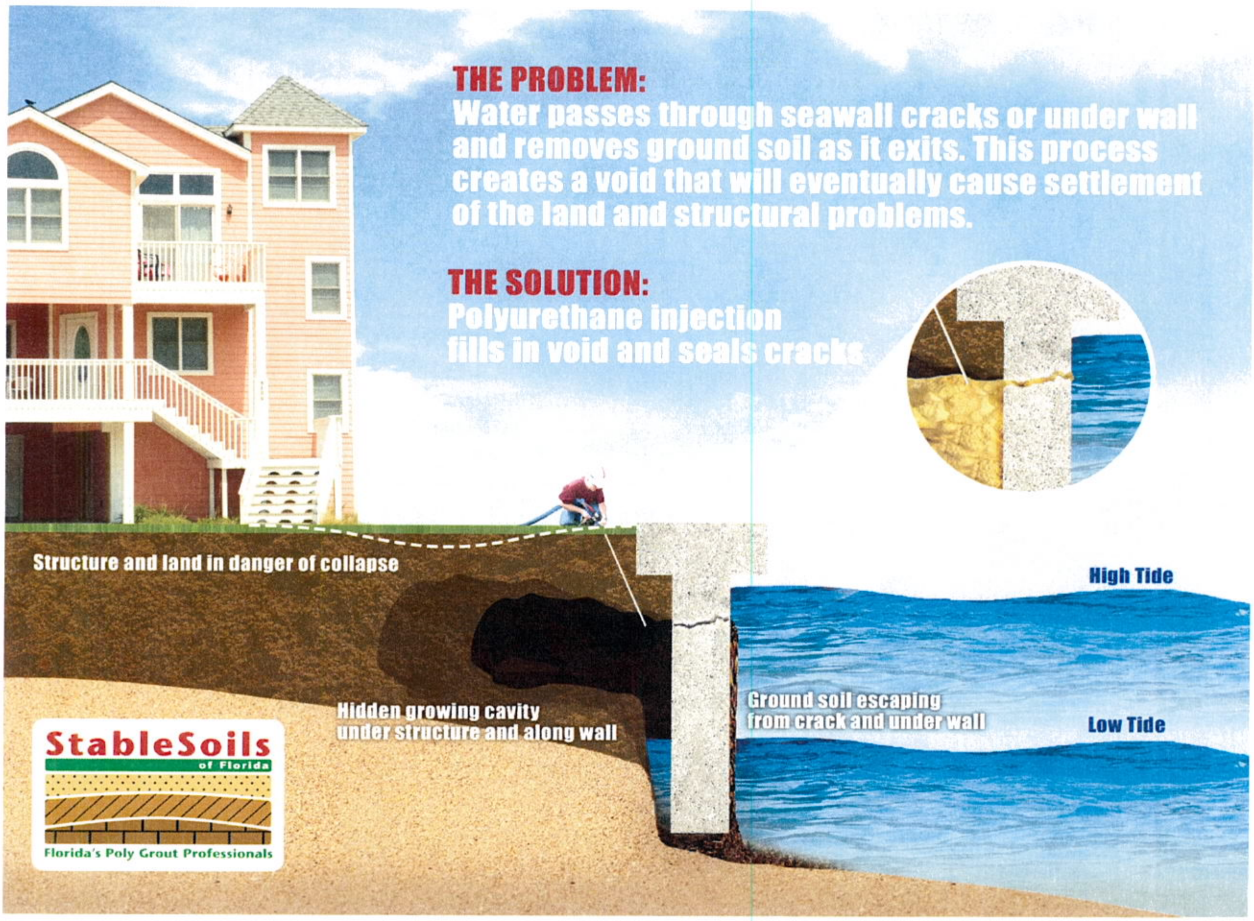
Features and Benefits:

- ✓ All employees are Professionally Trained
- ✓ All employees are Background Checked
- ✓ All employees are Drug Tested
- ✓ Great relationship with all local/most state Engineers
- ✓ We are Fully Insured - General Liability, Workers Comp and Automobile
- ✓ We are Innovators-Always looking for Breakthrough Technology
- ✓ **ZERO** excavation required
- ✓ Experience in all chemical grouting jobs.

Thank you again for your time and consideration,

Stable Soils of Florida, Inc.

CHEMICAL GROUTING



GROUTING:

Stable Soils of Florida carefully installs injection rods and pumps chemical grout behind your seawall and to fill any voids and seal cracks. Upon completion, all equipment will be removed and site area cleaned.

SCHEDULE OF WORK/STIPULATIONS

SCHEDULE OF WORK:

1. Stable Soils of Florida will contact Sunshine State ONE CALL to have all underground utilities located as required by law.
2. Stable Soils of Florida will make every effort to preserve all landscaping, grass and work area.

SCHEDULE:

We can typically mobilize **1 - 2 weeks** from the written notice to proceed.

ITEMS TO BE FURNISHED BY OTHERS:

- Continuous access to & from work area
- Source of electricity (110 volts) & water

STIPULATIONS:

Stable Soils of Florida, Inc., will not be responsible or liable for any damage whatsoever that may occur as a result of our work. Separation of pipes or conduits may occur or may be exaggerated during or after the work is performed. Although unlikely, our product may intrude into plumbing which may require replacement by a Licensed Plumber. The owner agrees to fully release Stable Soils from any and all responsibility stemming from our work or consequential damages to the structures caused by plumbing damages. Stable Soils recommends a thorough test of all plumbing lines prior to and after the job is complete.

Owner understands that any rigid materials (flooring, brick, stucco or exterior coatings, sheetrock, tile, countertops, window and door framings, cabinets, appliances, wall coverings, etc.) may be damaged due to our work. Stable Soils will not be held liable for any damages or cost of repair to any such damage. Owner understands that Stable Soils carries Workmen's Compensation and General Liability Insurance as required by Law.

Owner understands that there may be known and/or unknown structural defects that exist prior to the start of our work. All care and consideration will be given to known defects but Stable Soils will not be responsible for the damages or any consequential damages as a result of these defects. These defects include (but are not limited to) lack of steel reinforcement within foundations, slabs or walls, slabs that are poured to a thickness of less than 4" or inconsistent/varying thickness, decayed wood support structures, etc.

Owner understands there always is the possibility of future movements or stresses that may occur after Stable Soils completes the work. Plumbing leaks (especially in clayey soils), large trees/shrubbery, continually wet soils, foundations constructed on extremely dry soils, improperly compacted soils and sandy/silty soils will continually provide future stresses on structures. Stable Soils will not be responsible for any future damages. Additional work in the future may be required and is NOT COVERED BY ANY FURTHER WARRANTY, EXPRESSED OR IMPLIED, UNDER THE TERMS OF THIS CONTRACT.

SCOPE OF WORK

Inject 1 Part AP 700 on 3' centers with Catalyst to approximately 17' depth average. (1-2' below seawall panel depth) This process involves installing 1/2 inch diameter rods to below seawall toe. Hammer drills are used from a man lift to insert rods as needed. A sacrificial tip is used to keep rod from being plugged with debris. Once rods are in place along a section of wall the catalyst will be added to the chemical and the pumping will begin. We place 1 gal of material on the bottom, let sit for a moment and then place 3/4 of a gallon per vertical foot rise until 1 foot from surface or the chemical rises to the surface.

For this project there will be approximately 116 injection points and approximately 13 gals. Per injection for a total of 1508 gals. Any material used + or – this amount will be an add or deduct item at $90.00 per Gal. Depth will be the determining factor.

This process will take approximately 2 – 4 weeks.

SCHEDULE OF PRICES

Stable Soils of Florida, Inc. proposes to perform the work described above for the unit prices listed below. Due to the nature of this type of work and the unknown quantities of materials & depths actually required to complete the project, the unit price quotations are estimates only. Actual quantities may alter these estimates and final payment is determined upon actual quantities supplied.

DESCRIPTION	EST. QUANITY	UNIT COST	EXTENDED COST
Mobilization/Site Restoration (Includes gals.)	1	Lump Sum	$15,000.00
Chemical Grout Material	1508	$90.00 per gals.	$135,720.00
4 man crew, equipment approximately 3 weeks			
Total Estimated Cost Range:			$150,720.00

NOTE:
Any additional product over 1508 gallons will be charged at a rate of $90.00 per gallon.

TERMS OF PAYMENT:
TBD % deposit due upon acceptance of proposal. Balance of contract due upon completion. Owner agrees to pay cost of collection should any action be taken to collect any/all amounts due and unpaid, to include court costs and attorney's fees.

Thank you very much for considering us for this project. If we can be of further service please do not hesitate to contact us. This proposal is good for 30 days.

Respectfully submitted:

Robert Stephenson, Managing Partner

Date 8-3-15

ACCEPTED BY:

Date

THIS PROPOSAL IS TO BECOME AN INTEGRAL PART OF ANY SUBCONTRACT OR PURCHASE ORDER YOU MAY ISSUE.

Piling Products, a division of
Roll Form Group (U.S) Inc.
945 Center Street
Green Cove Springs, Florida 32043
(904) 287-8000 Fax (904) 529-7757
www.pilingproducts.com

QUOTATION

PAGE 1 *of* 2

Project: Truman Basin Seawall

Location: Key West, FL

Customer:	Tetra Tech
Attn:	Mr. Stuart McGahee
Phone:	772-781-3404
Fax:	
Cell:	
Email:	stuart.mcgahee@tetratech.com

Date:	7/28/2015		
Inquiry Date:	7/28/2015		
Proposed Ship Date:	4-6 Weeks after order		
Terms:	Net 30 days	**Shipped Via:**	Truck
F.O.B.:	Ship Point		
PPD:		**Collect:**	XX
Bid Date:			
Sales Person:	Koslow		

Here is our quotation on the products named, subject to the conditions noted: Prices are based on costs and conditions existing on date of quotation and are subject to changes by the Seller before final acceptance. Typographical errors are subject to correction. Unless otherwise specified, all sales/rentals are subject to all applicable sales and use taxes.

Quantity	Description	Price	Per	Amount
Sale of:	**Sheet Piling per Contractor's Requirements: PZ-27 Equal 255 Wall Feet X 35' long w/ Corners & Coating:**			
124 pieces	XZ-95 Sheet Piling ASTM A-572 Gr.50 X 35' long = 112.2 tons	$965.00	Ton	$108,273.00
70 LF	Corner Bending of Above Wall Pieces (2 each X 35')	$36.00	LF	$2,520.00
26,170 SF	Coal Tar Epoxy Coating; 16 mils after SP-10 Blast; Full Length/Both Sides	$1.82	SF	$47,629.40
26,170 SF	Inorganic Zinc Primer (if required); Full Length/Both Sides	$0.55	SF	$14,393.50
	Freight Charges to Jobsite: 5 loads	$2,985.00	Load	$14,925.00
				$187,740.90

Freight Charge: Please See Above **Note: Sales Taxes are not Charged on Freight Costs.**

Steel Prices are subject to mill and freight charge increases.

Vendor makes no warranty of any kind, expressed or implied, concerning the properties, merchantability or fitness for a particular use of the products herein. Vendee acknowledges that it relied on its own judgement and expertise in choosing the equipment or material.

Quote Valid for 30 Days

Presented By:

Lane Koslow; General Manager

Above prices do not include applicable sales and use taxes. Please sign in the space provided below to indicate you're acceptance and approval of the above, returning a copy by fax and original by mail.

Accepted By: ____________________

Date: ____________________

From: O'Connor, Jeff <joconnor@uesi.com>
Sent: Friday, July 31, 2015 4:51 PM
To: McGahee, Stuart
Subject: RE: Underwater Installation Estimate

Stuart:
I am busy Monday, so we got it done now for you. Best estimate is around $50K. We assumed a 6" thick mat with reinforcing that allows it to articulate. Could be done in 3 to 5 days.

Just saw the anchors – that price does not include anchors.

Hope this helps.

Jeffrey O'Connor, PE
Underwater Engineering Services, Inc.
d 772.429.9332 c 407.709.0004

From: McGahee, Stuart [mailto:Stuart.McGahee@tetratech.com]
Sent: Friday, July 31, 2015 3:45 PM
To: O'Connor, Jeff <joconnor@uesi.com>
Subject: Underwater Installation Estimate

Jeff,

I have a client (TBD) that is asking for rough cost for repairs to his seawall. The project is down in South Florida and I was going to try and give him at least one repair option using fabriform. I propose the installation of a Fabriform Unimat at the toe of a seawall to prevent scour and undermining. It is in a deep basin that slopes up to the existing wall. The wall was originally installed in a rather shallow basin and the basin was later deepened. It is stiff rock all the way down. Can you try and get me a BALLPARK estimate to install this repair using the following.

Located in the Keys.

- 350 LF
- Fabriform Unimat
- 8' Wide x 350 LF long = 2,800 SF = 311 SY
- Water depth = 10 ' (MAX)

Here is a draft of what I had in mind. **Can you give me a BALLPARK Price**?

Stuart E. McGahee PE | Tetra Tech
Direct: 772.781.3404 | Cell: 772.200.5113
759 S. Federal Highway, Suite 314 | Stuart, FL 34994

Page 1 Estimate# 40079 07/30/15

DECO TRUSS COMPANY, INC.

13980 SW 252 STREET MIAMI,FL 33032 (305)257-1910 * FAX 305-257-1911

TETRA TECH KEYWEST

E S T I M A T E

9999 OUR TRUCK 07/30/15

112 - VICENTE R. GARZA 998 - HOUSE ACCOUNT

Ln#	Quantity	Um	Sku#	Description	SF/BF	Price	Extended
001	3338	LB	58STF	#5 REBAR 5/8"		0.42	1401.96
002				80 PCS 40' LONG			
003	394	EA	07	6X18X18X18 W/4" HOOK #4		4.26	1678.44
004	394	EA	07	6X18X18X18 W/4" HOOK #3		2.05	807.70
005	1	EA	FC	FUEL DELIVERY CHARGE		250.00	250.00

THIS PRODUCT HAS BEEN SPECIALLY ORDERED PER YOUR SPECIFICATIONS AND IS THEREFORE NON-CANCELABLE AND NON-RETURNABLE.
PLEASE SIGN, PRINT AND FAX BACK. ***NOTE*** FULL PAYMENT REQUIRED ON ALL SPECIAL ORDERS, PRIOR TO ORDER BEING PLACED

Taxable	Nontaxable	Area	Taxamt	Total
3888.10	250.00	002	291.61	4429.71

ENGINEER'S WALL REPLACEMENT COST ESTIMATE

Description	Unit	No. of Units	Unit Price	Material	Labor / Installation
Mobilization	LS	1	15,000.00	15,000.00	15,000.00
Demolition	LS	1	15,000.00	15,000.00	30,000.00
XZ-95 Sheetpile (FOB Key West)	LF	350	536.40	187,740.00	375,480.00
Concrete Cap (6,000 PSI)	CY	65	265.00	17,225.00	34,450.00
Rebar #5 & #4	LF	350	9.51	3,330.00	6,660.00
Dock & Mooring Pile Repairs	LS	1	20,000.00	20,000.00	40,000.00
Backfill (Local Pea Gravel)	TON	364	30.00	10,920.00	21,840.00
			Subtotal	269,215.00	523,430.00
XZ-95 Total	LF	350	2,264.70		792,645.00

UNIT PRICES

Description	Unit	Unit Price	No. of Units	Subtotal
Sand (Local Screening)	Ton	30.00	364	10,920.00
Beach Sand (Immokolee)	Ton	45.00	364	16,380.00
Clean Sand (Miami)	Ton	38.00	364	13,832.00
#57 (Miami)	Ton	40.00	364	14,560.00
#57 (Local Screeening)	Ton	31.00	364	11,284.00
Pea Gravel (Local Screening)	Ton	30.00	364	10,920.00
Backfill Dirt (Local)	Ton	28.00	364	10,192.00
Concrete 6,000 PSI (Local)	CY	265.00	65	17,225.00
#5 Rebar	LB	0.42	3338	1,401.96
#4 Rebar (Saddle)	EA	4.26	394	1,678.44
#3 Rebar (Saddle) Alt.	EA	2.05	394	807.70
Fuel Charge (Miami)	EA	250.00	1	250.00
XZ-95 Sheetpile	LF	536.40	350	187,740.00
PZ-27 Sheetpile	LF	697.32	350	244,062.00

Exhibit 15

Geotechnical Report

SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA

AACE FILE NO. 15-148

AACE

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

573 SW Biltmore Street
Port St. Lucie, Florida 34983
Ph: 772-807-9191   Fx: 772-807-9192
www.aaceinc.com

TABLE OF CONTENTS

SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT - TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA

AACE FILE NO. 15-148

PAGE #

AACE File No. 15-148
July 15, 2015

Tetra Tech, Inc.
759 South Federal Highway, Suite 314
Stuart, FL 34994

Attn: Mr. Stuart McGahee, P.E.

SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT - TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA

1.0 INTRODUCTION

In accordance with the request and authorization of Tetra Tech, Inc. (TT), Andersen Andre Consulting Engineers, Inc. (AACE) has completed a subsurface exploration and preliminary geotechnical engineering evaluation for the above referenced project. The purpose of performing this exploration was to explore soil types and groundwater levels, and restrictions which these may place on the proposed seawall replacement project. Our work included Standard Penetration Test (SPT) borings, limited laboratory testing, and engineering analysis. This report documents our explorations and presents our findings, and summarizes our preliminary conclusions and recommendations.

2.0 SITE INFORMATION AND PROJECT UNDERSTANDING

2.1 Site Location and Project Description

The subject seawall segment which is proposed to be replaced (i.e. the site) is fronting the NOAA National Marine Sanctuary regional office, located at 33 East Quay Road in Key West, Monroe County, Florida. A Site Vicinity Map (2014 aerial photograph) which depicts the location of the site is included on the attached Sheet No. 1. The site location is further shown superimposed on the "Key West, FL" USGS topographic quadrangle map (1971), also included on Sheet No. 1.

The existing, approximately 325-ft long seawall segment appears to consists of reinforced concrete panels with an approximately 2-ft wide by 1-ft thick concrete cap (top), and with an approximately 2-ft deep by 1-ft wide horizontal concrete beam located along the seawall near the water level (or, near mid-height) in the adjacent basin. The embedment depths of the concrete panels are unknown, and we expect the fronting horizontal beam is acting as a waler as part of a tieback system. The upland side of the seawall is an approximately 25-30 foot wide unpaved pier, which consists of limerock fill with storm drains and inlets, various utilities, lighting, etc.

Based on measurements collectively made by representatives of TT and AACE, the water depth in front of the seawall ranges from about 2 feet to about 13 feet, and sand-cement bags appear to have been placed along the toe of the wall, at least on the eastern approximate one-half of the seawall segment and possibly more. Further, an apparent concrete toe wall is visible on the western approximately one-half of the seawall. Measurements taken along the extent of the five existing wooden docks fronting the seawall indicate that the bottom of the basin slopes away from the seawall at a slope of 1H:1V or steeper.

Representative photographs of the site are presented in Appendix I.

At this point in time, the design of the replacement seawall is in progress and, as such, no specific details are available for a geotechnical engineering evaluation. This report includes general comments and pertinent soil properties to assist in the design, and it is anticipated that a final geotechnical engineering review of the ultimate design will be required.

2.2 Review of USDA Soil Survey

According to the USDA NRCS Web Soil Survey, the soils within the Truman Waterfront area (as well as the majority of Key West) are identified as *Urban land (Map Unit ID 11)*, which is a term used to describe areas which have been altered (by grading, shaping, covering, etc.) to an extent where the original soils cannot easily be identified. In general, the lower keys (including Key West) are underlain by an oolitic limestone formation that varies in density and composition, and which in some areas contains coral and invertebrate fossils.

The approximate location of the site is shown superimposed on a copy of the USDA Web Soil Survey aerial photograph, presented on Sheet No. 1, and the summary report obtained from the USDA Web Soil Survey is included in Appendix II.

3.0 FIELD EXPLORATION PROGRAM

To explore subsurface conditions at the site relative to the proposed seawall replacement/construction, two (2) Standard Penetration Test (SPT) borings were completed to depths of 30 feet below the existing ground surface. This work was performed on June 23, 2015 at the approximate locations shown on the Boring Location Plan on Sheet No. 1.

The soil boring locations shown on Sheet No. 1 were determined in the field by our field crew using a combination of hand-held GPS and tape and wheel measurements, obtained aerial photographs, and existing site features as references. The locations should be considered accurate only to the degree implied by the method of measurement used. We preliminarily anticipate that the actual locations are within 15 feet of those shown on Sheet No. 1.

Summaries of AACE's field procedures are included in Appendix III, and the individual boring profiles are presented on the attached Sheet No. 2. Samples obtained during performance of the borings were visually classified in the field, and representative portions of the samples were transported to our laboratory in sealed sample jars for further classification. The soil samples recovered from our explorations will be kept in our laboratory for 60 days, then discarded unless you specifically request otherwise.

4.0 OBSERVED SUBSURFACE CONDITIONS

4.1 General Soil Conditions

Detailed subsurface conditions are illustrated on the soil boring profiles presented on the attached Sheet No. 2. The stratification of the boring profiles represents our interpretation of the field boring logs and the results of laboratory examinations of the recovered samples. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

In brief, at the locations and depths explored, our borings encountered loose to moderately dense crushed limerock fill to depths of about 6 feet, followed by loose to very dense cemented oolitic limestone with varying degree of silt and reaching the termination depths of our borings.

Despite the relatively high SPT 'N' values, refusal to the SPT sampler was not encountered at any depth (with refusal defined as needing more than 50 blows of a 140-pound hammer dropped from a height of 30 inches to penetrate 6 inches). Hence, the encountered oolitic limestone is generally considered to be a relatively "soft" and friable rock formation, and the recovered SPT split-spoon samples were observed to be in a very friable condition. Following completion of the two SPT borings, it was the opinion of the Drill Crew Chief that the encountered oolitic limerock formation was not suitable (i.e. strong enough) to allow coring to be completed in accordance with ASTM D2113.

4.2 Measured Groundwater Level

The groundwater table was encountered at depths of 7 feet below the existing grades. In general, fluctuations in groundwater levels should be anticipated throughout the year primarily due to tidal fluctuations and possibly other factors that may vary from the time the borings were conducted.

5.0 LABORATORY TESTING PROGRAM

Our drillers observed the soil recovered from the borings, placed the recovered soil samples in moisture proof containers, and maintained a log for each boring. The recovered soil samples, along with the field boring logs, were transported to our Port St. Lucie soils laboratory where they were visually examined by AACE's project engineer to determine their engineering classification.

6.0 PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION

Based on the findings of our subsurface soil exploration, our evaluation of the encountered soil conditions, and judgment based on our experience with similar seawall design projects, it is our opinion that the encountered oolitic limerock formation is suitable for facilitating the seawall design.

We understand that the new seawall is proposed to be installed in front of the existing seawall which is to remain in place, however, modified or partially demolished to allow for a potential tieback system. Various design options are currently being discussed, including utilizing concrete panels embedded into an augered or excavated toe trench, and possibly equipped with a tie-back system. Also, a heavy gauge steel sheet pile wall (either cantilevered or with a tie-back system) could possibly be utilized. Should the existing, partial toe wall and the existing seawall conditions adversely affect the installation of the new seawall in front of the old seawall, consideration is also being given to installing the new seawall upland of the existing bulkhead. In that case, it will be necessary to work around the existing tieback system as it should not be removed entirely before the new sewall has been constructed.

As mentioned in the previous, the encountered oolitic limestone formation is not considered a "strong" limerock formation and it is likely that steel sheet piles could be vibrated in place. We do recommend that any bidding Contractor review this report as well as physically inspecting the recovered soil samples.

The soil parameters summarized below are provided for others to use in the seawall design, both with regards to active and passive earth pressures acting on the wall, the toe embedment and any potential deadman anchors associated with a tie back system. We remain available to provide additional engineering consulting with respect to the design of the seawall components. Further, additional estimates of rock properties can be provided, is needed.

Soil Parameters for Seawall Design

Depth below existing grade (feet)	Average SPT 'N' Value	Unit Weight, γ (pcf)	Angle of Internal Friction, ϕ	Cohesion (psf)	Wall Friction Angle, δ [(B)]
0-6 (limerock fill)	11	113	32	NA	18
6-10 (upper limestone)	8	118	35	1000	23
10-30 (lower limestone)	55	135	38	5000	25

Notes: (A) Assumes vertical backface of wall, and wall directly against granular backfill.

The Rankine coefficients of lateral pressures can be obtained from the following equations:

Active pressure: $K_a = \tan^2 (45 - \phi/2)$
Passive pressure: $K_p = \tan^2 (45 + \phi/2)$
where ϕ is the friction angle of the soil.

We recommend that appropriate safety factors be used in the sheet pile design. The safety factors selected should be based on design and construction considerations which are beyond the scope of this report.

- -Balance of page left blank intentionally - -

7.0 CLOSURE

The preliminary geotechnical evaluation submitted herein is based on the data obtained from the soil borings presented on Sheet No. 2, and our understanding of the proposed construction as previously described. We remain available to complete additional geotechnical engineering analysis for the desired seawall design. Limitations and conditions to this report are presented in Appendix IV.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices for the exclusive use of Tetra Tech, Inc. for the subject project. No other warranty, expressed or implied, is made.

We are pleased to be of assistance to you on this phase of your project. When we may be of further service to you or should you have any questions, please contact us.

Sincerely,

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
Certificate of Authorization No. 26794

Peter G. Andersen, P.E.
Principal Engineer 7-15-2015
Fla. Reg. No. 57956

PETER GREGERS ANDERSEN LICENSE No. 57956 STATE OF FLORIDA PROFESSIONAL ENGINEER

David P. Andre, P.E.
Principal Engineer
Fla. Reg. No. 53969
7/15/15

PGA/DPA/ma

SITE VICINITY MAP

USGS MAP - "KEY WEST, FL" (1971)

USDA SOIL SURVEY MAP

NOT TO SCALE
Latitude: 24.55115
Longitude: -81.80786

USDA SOIL TYPES ON SITE:

No. 11: URBAN LAND

BORING LOCATION PLAN

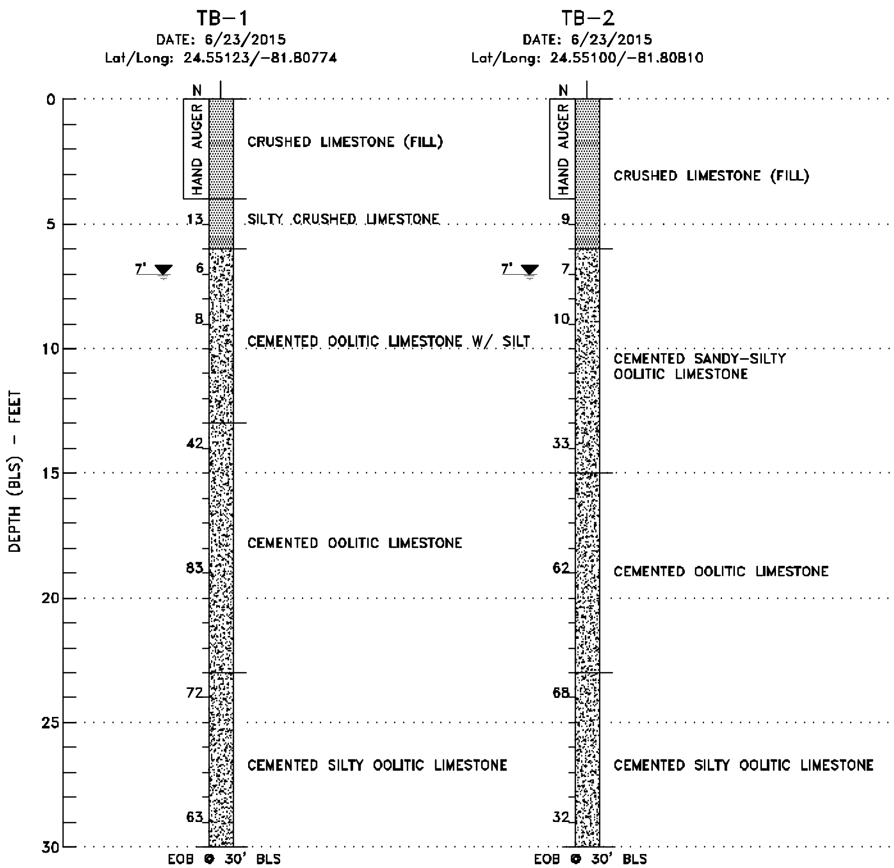
APPROXIMATE SCALE [11x17]: 1"=50'
0 25 50 100
FEET

NOTES AND LEGEND

TB-# Approximate Standard Penetration Test (SPT) Boring Location

Shown boring locations are approximate, and were determined using a combination of hand-held GPS, tape and wheel measurements, obtained aerial photographs, and existing site features as references. The shown boring locations should be considered accurate only to the degree implied by the method of measurement used.

ANDERSEN ANDRE CONSULTING ENGINEERS, INC. 573 SW Biltmore Street, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com Certificate of Authorization No. 26794	VICINITY MAPS AND BORING LOCATION PLAN	SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA	Drawn by: PGA — Date: June/July 2015 Checked by: DPA — Date: June/July 2015 AACE File No: 15-148 — **Sheet No. 1**



CRUSHED LIMESTONE (FILL) [TB-1 @ 3 FT]

CEMENTED OOLITIC LIMESTONE W/ SILT [TB-1 @ 8 FT]

CEMENTED OOLITIC LIMESTONE [TB-1 @ 18 FT]

CEMENTED SILTY OOLITIC LIMESTONE [TB-1 @ 25 FT]

NOTES:

TB-#	STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
N	SPT RESISTANCE IN BLOWS PER FOOT
X' ▼	GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB	END OF BORING
BLS	BELOW LAND SURFACE

DRILL CREW FIRM: CL
DRILL CREW CHIEF: BS
DRILL RIG: CME-45
DRILL METHOD: ROTARY-WASH/BENTONITE SLURRY
CASING: NOT NEEDED
HAMMER TYPE: MANUAL

SOIL LEGEND:

CRUSHED LIMESTONE FILL (GP)
CEMENTED LIMESTONE

ANDERSEN ANDRE CONSULTING ENGINEERS, INC. 573 SW Biltmore Street, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com Certificate of Authorization No. 26794	SPT BORING PROFILES	SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA	Drawn by: PGA	Date: June/July 2015
			Checked by: DPA	Date: June/July 2015
			AACE File No: 15-148	Sheet No. 2

APPENDIX I

Representative Site Photographs

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
573 SW Biltmore Street
Port St. Lucie, Florida 34983
Phone: 772.807.9191 Fax: 772.807.9192
www.aaceinc.com

NOAA Seawall - Truman Waterfront - Key West, FL

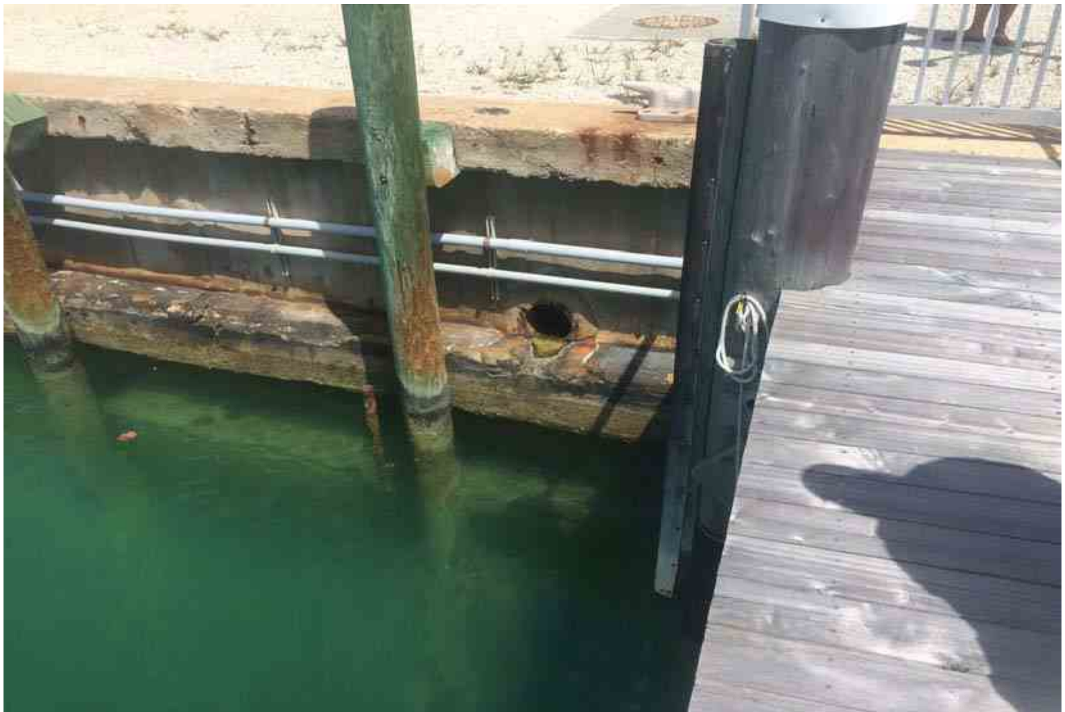
Representative Site Photographs (06/23/2015)

1) Typical View of Existing Seawall

2) Typical View of Existing Seawall

NOAA Seawall - Truman Waterfront - Key West, FL

Representative Site Photographs (06/23/2015)



3) Typical View of Existing Seawall with Drain Crossing

4) Sand-Cement Bags by Toe of Seawall

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
573 SW Biltmore Street
Port St. Lucie, Florida 34983
Phone: 772.807.9191 Fax: 772.807.9192
www.aaceinc.com

NOAA Seawall - Truman Waterfront - Key West, FL

Representative Site Photographs (06/23/2015)

5) Close-Up View of Seawall Front

6) East End of Seawall

Appendix II

USDA Web Soil Survey Summary Report

Soil Map—Monroe County, Keys Area, Florida
(NOAA - KW)

Map Scale: 1:2,340 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0 30 60 120 180

Feet: 0 100 200 400 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 17N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Monroe County, Keys Area, Florida
Survey Area Data: Version 5, Sep 9, 2014

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Data not available.

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Monroe County, Keys Area, Florida (FL687)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
11	Urban land	16.4	58.7%
100	Waters of the Atlantic Ocean	11.5	41.3%
Totals for Area of Interest		**27.9**	**100.0%**

Monroe County, Keys Area, Florida

11—Urban land

Map Unit Setting

National map unit symbol: vryh
Elevation: 0 to 10 feet
Mean annual precipitation: 30 to 51 inches
Mean annual air temperature: 72 to 82 degrees F
Frost-free period: 358 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Urban land: 95 percent
Minor components: 5 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Setting

Landform: Islands
Landform position (three-dimensional): Interfluve, talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: No parent material

Properties and qualities

Slope: 0 to 1 percent
Frequency of flooding: Rare

Interpretive groups

Land capability classification (irrigated): None specified
Other vegetative classification: Forage suitability group not assigned (G156AC999FL)

Minor Components

Udorthents

Percent of map unit: 3 percent
Landform: Islands
Landform position (three-dimensional): Interfluve
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Forage suitability group not assigned (G156AC999FL)

Beaches, tidal

Percent of map unit: 2 percent
Landform: Beaches on islands
Landform position (three-dimensional): Rise
Down-slope shape: Convex
Across-slope shape: Linear

Other vegetative classification: Forage suitability group not assigned (G156AC999FL)

Data Source Information

Soil Survey Area: Monroe County, Keys Area, Florida
Survey Area Data: Version 5, Sep 9, 2014

APPENDIX III

General Notes

SOIL BORING, SAMPLING AND TESTING METHODS

GENERAL

Andersen Andre Consulting Engineers, Inc. (AACE) borings describe subsurface conditions only at the locations drilled and at the time drilled. They provide no information about subsurface conditions below the bottom of the boreholes. At locations not explored, surface conditions that differ from those observed in the borings may exist and should be anticipated.

The information reported on our boring logs is based on our drillers' logs and on visual examination in our laboratory of disturbed soil samples recovered from the borings. The distinction shown on the logs between soil types is approximate only. The actual transition from one soil to another may be gradual and indistinct.

The groundwater depth shown on our boring logs is the water level the driller observed in the borehole when it was drilled. These water levels may have been influenced by the drilling procedures, especially in borings made by rotary drilling with bentonitic drilling mud. An accurate determination of groundwater level requires long-term observation of suitable monitoring wells. Fluctuations in groundwater levels throughout the year should be anticipated.

The absence of a groundwater level on certain logs indicates that no groundwater data is available. It does not mean that groundwater will not be encountered at that boring location at some other point in time.

STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is a widely accepted method of in situ testing of foundation soils (ASTM D-1586). A 2-foot (0.6m) long, 2-inch (50mm) O.D. split-barrell sampler attached to the end of a string of drilling rods is driven 24 inches (0.60m) into the ground by successive blows of a 140-pound (63.5 Kg) hammer freely dropping 30 inches (0.76m). The number of blows needed for each 6 inches (0.15m) increments penetration is recorded. The sum of the blows required for penetration of the middle two 6-inch (0.15m) increments of penetration constitutes the test result of N-value. After the test, the sampler is extracted from the ground and opened to allow visual description of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density and, for cohesive soils, an approximate unconfined compressive strength (Qu):

Cohesionless Soils:	N-Value	Description
	0 to 4	Very loose
	4 to 10	Loose
	10 to 30	Medium dense
	30 to 50	Dense
	Above 50	Very dense

Cohesive Soils:	N-Value	Description	Qu
	0 to 2	Very soft	Below 0.25 tsf (25 kPa)
	2 to 4	Soft	0.25 to 0.50 tsf (25 to 50 kPa)
	4 to 8	Medium stiff	0.50 to 1.0 tsf (50 to 100 kPa)
	8 to 15	Stiff	1.0 to 2.0 tsf (100 to 200 kPa)
	15 to 30	Very stiff	2.0 to 4.0 tsf (200 to 400 kPa)
	Above 30	Hard	Above 4.0 tsf (400 kPa)

The tests are usually performed at 5 foot (1.5m) intervals. However, more frequent or continuous testing is done by AACE through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid. After completion of a test borings, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed by backfilling, either with accumulated cuttings or lean cement.

Representative split-spoon samples from each sampling interval and from different strata are brought to our laboratory in air-tight jars for classification and testing, if necessary. Afterwards, the samples are discarded unless prior arrangement have been made.

POWER AUGER BORINGS

Auger borings (ASTM D-1452) are used when a relatively large, continuous sampling of soil strata close to the ground surface is desired. A 4-inch (100 mm) diameter, continuous flight, helical auger with a cutting head at its end is screwed into the ground in 5-foot (1.5m) sections. It is powered by the rotary drill rig. The sample is recovered by withdrawing the auger our of the ground without rotating it. The soil sample so obtained, is classified in the field and representative samples placed in bags or jars and returned to the AACE soils laboratory for classification and testing, if necessary.

HAND AUGER BORINGS

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5-foot [1.5m]) depth or when access is not available to power drilling equipment. A 3-inch (75mm) diameter hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved at approximately 6-inch (0.15m) interval and its contents emptied for inspection. On occasion post-hole diggers are used, especially in the upper 3 feet (1m) or so. Penetrometer probings can be used in the upper 5 feet (1.5m) to determine the relative density of the soils. The soil sample obtained is described and representative samples put in bags or jars and transported to the AACE soils laboratory for classification and testing, if necessary.

UNDISTURBED SAMPLING

Undisturbed sampling (ASTM D-1587) implies the recovery of soil samples in a state as close to their natural condition as possible. Complete preservation of in situ conditions cannot be realized; however, with careful handling and proper sampling techniques, disturbance during sampling can be minimized for most geotechnical engineering purposes. Testing of undisturbed samples gives a more accurate estimate of in situ behavior than is possible with disturbed samples.

Normally, we obtain undisturbed samples by pushing a 2.875-inch (73 mm) I.D., thin wall seamless steel tube 24 inches (0.6 m) into the soil with a single stoke of a hydraulic ram. The sampler, which is a Shelby tube, is 30 (0.8 m) inches long. After the sampler is retrieved, the ends are sealed in the field and it is transported to our laboratory for visual description and testing, as needed.

ROCK CORING

In case rock strata is encountered and rock strength/continuity/composition information is needed for foundation or mining purposes, the rock can be cored (ASTM D-2113) and 2-inch to 4-inch diameter rock core samples be obtained for further laboratory analyses. The rock coring is performed through flush-joint steel casing temporarily installed through the overburden soils above the rock formation and also installed into the rock. The double- or triple-tube core barrels are advanced into the rock typically in 5-foot intervals and then retrieved to the surface. The barrel is then opened so that the core sample can be extruded. Preliminary field measurements of the recovered rock cores include percent recovery and Rock Quality Designation (RQD) values. The rock cores are placed in secure core boxes and then transported to our laboratory for further inspection and testing, as needed.

SFWMD EXFILTRATION TESTS

In order to estimate the hydraulic conductivity of the upper soils, constant head or falling head exfiltration tests can be performed. These tests are performed in accordance with methods described in the South Florida Water Management District (SFWMD) Permit Information Manual, Volume IV. In brief, a 6 to 9 inch diameter hole is augered to depths of about 5 to 7 feet; the bottom one foot is filled with 57-stone; and a 6-foot long slotted PVC pipe is lowered into the hole. The distance from the groundwater table and to the ground surface is recordedand the hole is then saturated for 10 minutes with the water level maintained at the ground surface.

If a constant head test is performed, the rate of pumping will be recorded at fixed intervals of 1 minute for a total of 10 minutes, following the saturation period.

LABORATORY TEST METHODS

Soil samples returned to the AACE soils laboratory are visually observed by a geotechnical engineer or a trained technician to obtain more accurate description of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to help define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain size distributions or selected other test results may be presented on separate tables, figures or plates as discussed in this report.

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

The soil descriptions shown on the logs are based upon visual-manual procedures in accordance with local practice. Soil classification is performed in general accordance with the United Soil Classification System and is also based on visual-manual procedures.

BOULDERS (>12" [300 MM]) and COBBLES (3" [75 MM] TO 12" [300 MM]):

GRAVEL:	Coarse Gravel:	3/4" (19 mm) to 3" (75 mm)
	Fine Gravel:	No. 4 (4.75 mm) Sieve to 3/4" (19 mm)

Descriptive adjectives:

0 - 5%	– no mention of gravel in description
5 - 15%	– trace
15 - 29%	– some
30 - 49%	– gravelly (shell, limerock, cemented sands)

SANDS:

COARSE SAND: No. 10 (2 mm) Sieve to No. 4 (4.75 mm) Sieve
MEDIUM SAND: No. 40 (425 µm) Sieve to No. 10 (2 mm) Sieve
FINE SAND: No. 200 (75 µm) Sieve to No. 40 (425 µm) Sieve

Descriptive adjectives:

0 - 5%	– no mention of sand in description
5 - 15%	– trace
15 - 29%	– some
30 - 49%	– sandy

SILT/CLAY: < #200 (75µM) Sieve

SILTY OR SILT: PI < 4
SILTY CLAYEY OR SILTY CLAY: 4 ≤ PI ≤ 7
CLAYEY OR CLAY: PI > 7

Descriptive adjectives:

< - 5%	– clean (no mention of silt or clay in description)
5 - 15%	– slightly
16 - 35%	– clayey, silty, or silty clayey
36 - 49%	– very

ORGANIC SOILS:

Organic Content	Descriptive Adjectives	Classification
0 - 2.5%	Usually no mention of organics in description	See Above
2.6 - 5%	slightly organic	add “with organic fines” to group name
5 - 30%	organic	SM with organic fines
		Organic Silt (OL)
		Organic Clay (OL)
		Organic Silt (OH)

Organic Clay (OH)

HIGHLY ORGANIC SOILS AND MATTER:

Organic Content	**Descriptive Adjectives**	**Classification**
30 - 75%	sandy peat	Peat (PT)
	silty peat	Peat (PT)
> 75%	amorphous peat	Peat (PT)
	fibrous peat	Peat (PT)

STRATIFICATION AND STRUCTURE:

Descriptive Term		**Thickness**
with interbedded		
seam	--	less than ½ inch (13 mm) thick
layer	--	½ to 12-inches (300 mm) thick
stratum	--	more than 12-inches (300 mm) thick
pocket	--	small, erratic deposit, usually less than 1-foot
lens	--	lenticular deposits
occasional	--	one or less per foot of thickness
frequent	--	more than one per foot of thickness
calcareous	--	containing calcium carbonate (reaction to diluted HCL)
hardpan	--	spodic horizon usually medium dense
marl	--	mixture of carbonate clays, silts, shells and sands

ROCK CLASSIFICATION (FLORIDA) CHART:

Symbol	Typical Description
LS	Hard Bedded Limestone or Caprock
WLS	Fractured or Weathered Limestone
LR	Limerock (gravel, sand, silt and clay mixture)
SLS	Stratified Limestone and Soils

LEGEND FOR BORING LOGS

N:	Number of blows to drive a 2-inch OD split spoon sampler 12 inches using a 140-pound hammer dropped 30 inches
R:	Refusal (less than six inches advance of the split spoon after 50 hammer blows)
MC:	Moisture content (percent of dry weight)
OC:	Organic content (percent of dry weight)
PL:	Moisture content at the plastic limit
LL:	Moisture content at the liquid limit
PI:	Plasticity index (LL-PL)
qu:	Unconfined compressive strength (tons per square foot, unless otherwise noted)
-200:	Percent passing a No. 200 sieve (200 wash)
+40:	Percent retained above a No. 40 sieve
US:	Undisturbed sample obtained with a thin-wall Shelby tube
k:	Permeability (feet per minute, unless otherwise noted)
DD:	Dry density (pounds per cubic foot)
TW:	Total unit weight (pounds per cubic foot)

APPENDIX IV

Project Limitations and Conditions

Project Limitations and Conditions

Andersen Andre Consulting Engineers, Inc. has prepared this report for our client for his exclusive use, in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made herein. Further, the report, in all cases, is subject to the following limitations and conditions:

VARIABLE/UNANTICIPATED SUBSURFACE CONDITIONS

The engineering analysis, evaluation and subsequent recommendations presented herein are based on the data obtained from our field explorations, at the specific locations explored on the dates indicated in the report. This report does not reflect any subsurface variations (e.g. soil types, groundwater levels, etc.) which may occur adjacent or between borings.

The nature and extent of any such variations may not become evident until construction/excavation commences. In the event such variations are encountered, Andersen Andre Consulting Engineers, Inc. may find it necessary to (1) perform additional subsurface explorations, (2) conduct in-the-field observations of encountered variations, and/or re-evaluate the conclusions and recommendations presented herein.

We at Andersen Andre Consulting Engineers, Inc. recommend that the project specifications necessitate the contractor immediately notifying Andersen Andre Consulting Engineers, Inc., the owner and the design engineer (if applicable) if subsurface conditions are encountered that are different from those presented in this report.

No claim by the contractor for any conditions differing from those expected in the plans and specifications, or presented in this report, should be allowed unless the contractor notifies the owner and Andersen Andre Consulting Engineers, Inc. of such differing site conditions. Additionally, we recommend that all foundation work and site improvements be observed by an Andersen Andre Consulting Engineers, Inc. representative.

SOIL STRATA CHANGES

Soil strata changes are indicated by a horizontal line on the soil boring profiles (boring logs) presented within this report. However, the actual strata's changes may be more gradual and indistinct. Where changes occur between soil samples, the locations of the changes must be estimated using the available information and may not be at the exact depth indicated.

SINKHOLE POTENTIAL

Unless specifically requested in writing, a subsurface exploration performed by Andersen Andre Consulting Engineers, Inc. is not intended to be an evaluation for sinkhole potential.

MISINTERPRETATION OF SUBSURFACE SOIL EXPLORATION REPORT

Andersen Andre Consulting Engineers, Inc. is responsible for the conclusions and recommendations presented herein, based upon the subsurface data obtained during this project. If others render conclusions or opinions, or make recommendations based upon the data presented in this report, those conclusions, opinions and/or recommendations are not the responsibility of Andersen Andre Consulting Engineers, Inc.

CHANGED STRUCTURE OR LOCATION

This report was prepared to assist the owner, architect and/or civil engineer in the design of the subject project. If any changes in the construction, design and/or location of the structures as discussed in this report are planned, or if any structures are included or added that are not discussed in this report, the conclusions and recommendations contained in this report may not be valid. All such changes in the project plans should be made known to Andersen Andre Consulting Engineers, Inc. for our subsequent re-evaluation.

USE OF REPORT BY BIDDERS

Bidders who are reviewing this report prior to submission of a bid are cautioned that this report was prepared to assist the owners and project designers. Bidders should coordinate their own subsurface explorations (e.g.; soil borings, test pits, etc.) for the purpose of determining any conditions that may affect construction operations. Andersen Andre Consulting Engineers, Inc. cannot be held responsible for any interpretations made using this report or the attached boring logs with regard to their adequacy in reflecting subsurface conditions which may affect construction operations.

IN-THE-FIELD OBSERVATIONS

Andersen Andre Consulting Engineers, Inc. attempts to identify subsurface conditions, including soil stratigraphy, water levels, zones of lost circulation, “hard” or “soft” drilling, subsurface obstructions, etc. However, lack of mention in the report does not preclude the presence of such conditions.

LOCATION OF BURIED OBJECTS

Users of this report are cautioned that there was no requirement for Andersen Andre Consulting Engineers, Inc. to attempt to locate any man-made, underground objects during the course of this exploration, and that no attempts to locate any such objects were performed. Andersen Andre Consulting Engineers, Inc. cannot be responsible for any buried man-made objects which are subsequently encountered during construction.

PASSAGE OF TIME

This report reflects subsurface conditions that were encountered at the time/date indicated in the report. Significant changes can occur at the site during the passage of time. The user of the report recognizes the inherent risk in using the information presented herein after a reasonable amount of time has passed. We recommend the user of the report contact Andersen Andre Consulting Engineers, Inc. with any questions or concerns regarding this issue.

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

IIGER03135.0MRP

Exhibit 16

Photo Logs

Site Photos
Key West NOAA Seawall

Key West, Monroe County, Florida
Tetra Tech, Inc.
Project: 194-5363

Photo: Key Sheet

Description:

This is the plan view of the site with direction arrow to be used for reference.

Photo: 1

Description:

Next to the Northeast end of the seawall, this is the adjacent seawall, leading toward the boat ramp. Looking Southeast.

Photo: 2 **Description:** Next to the Northeast end of the wall, this is the adjacent seawall which forms the triangle toward the end of the NOAA seawall. This side of the seawall is made with concrete capped steel sheet piles. Looking Northwest. 	

Photo: 3 **Description:** The seawall cap at the tip of the triangle on the Northeast end of the wall is broken and damaged. A buoy is covering a hole. Looking Northwest. 	

Site Photos
Key West NOAA Seawall

Key West, Monroe County, Florida
Tetra Tech, Inc.
Project: 194-5363

Photo: 4

Description:
The Northeast end of the NOAA seawall. The concrete seawall cap does not extend all the way to the edge (damage is apparent). Quikrete bags appear to have been placed along the line where the cap should run. Looking Northeast.

Photo: 5

Description:
The first dock (Dock 1) along the NOAA seawall, going from Northeast end to Southwest end. The dock steps down a few feet after passing the gate.

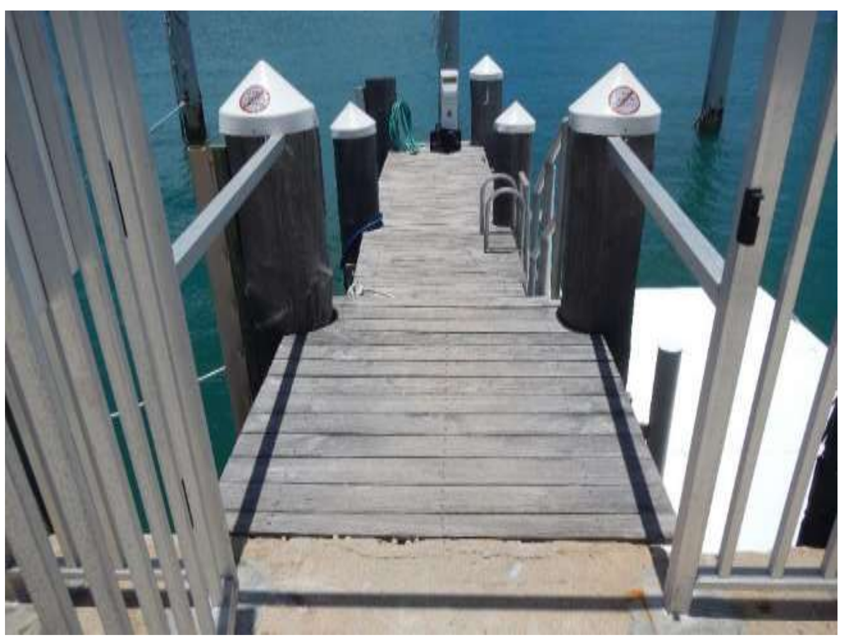


Photo: 6

Description:

View of the seawall from Dock 1 – left.

Photo: 7

Description:
View of the seawall from Dock 1 – right.

Photo: 8

Description:
The second dock along the NOAA seawall, Dock 2.

Photo: 9

Description:
View of the seawall from Dock 2 – right. Quikrete bags are visible along the bottom edge of the seawall.

Site Photos
Key West NOAA Seawall

Key West, Monroe County, Florida
Tetra Tech, Inc.
Project: 194-5363

Photo: 10

Description:
The third dock along the NOAA seawall, Dock 3.

Photo: 11

Description:
View of the seawall from Dock 3 – left. The quikrete bags appear to stop between Docks 2 and 3, and a concrete wall takes its place, running from the edge of the seawall to the top of grade.

Site Photos
Key West NOAA Seawall

Key West, Monroe County, Florida
Tetra Tech, Inc.
Project: 194-5363

Photo: 12

Description:
The fourth dock along the NOAA seawall, Dock 4.

Photo: 13

Description:
View of the seawall from Dock 4 – left. The concrete wall along the bottom appear to continue through the end of the seawall.

Photo: 14

Description:
View of the seawall from Dock 4 – right.

Photo: 15

Description:
The fifth and last dock along the NOAA seawall, Dock 5. This dock goes out about 82 feet, and has 3 sets of steps leading to slips along the left of the dock.

Photo: 16

Description:
View of the seawall from Dock 5 – left. The location of the second soil boring can also be seen here.

Photo: 17

Description:
View of the seawall from Dock 5 – right. The NOAA seawall ends, leading to the Navy seawall next door, on the Southwest end of the seawall.

Photo: 18

Description:
General view of the gravel path behind the seawall. The path is about 25 feet wide, with light posts by the entrance gate of each dock. Looking West.

Photo: 19

Description:
General view of the gravel path leading toward the docks. Looking South.

Photo: 20

Description:
View of the neighboring Navy property on the Southwest end of the NOAA seawall.

Photo: 21
Description:

At least two concrete pads were spotted on site, outside of the gates for Docks 3 and 4, with a storm water drain and utility manholes.

Exhibit 17

TranSystems Report

TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

April 24, 2012

Rob L. McWilliams, R.A.
U.S. Dept of Commerce/NOAA
Project Planning Management Div/Eastern Region
601 E. 12th St., Room 1749
Kansas City, MO 64106
Tel. No. (816) 426-7812
E-mail: rob.l.mcwilliams@noaa.gov

RE: Sea Wall Restoration at Florida Keys National Marine Sanctuary, Key West Florida
Contract Number: WC1330-07-CQ-0057
Reference to Previous Task Order Number: T0010
TranSystems Project Number P101120119

Mr. McWilliams:

In reference to the above project, this report documents the discussions and recommendations made at the site visit on March 27, 2012 in regard to the Sea Wall/Bulkhead at the Florida Keys National Marine Sanctuary's Nancy Foster Center as well as stating analysis of design approach and parametric cost estimate for going forward. The following individuals with NOAA were in attendance: Rob McWilliams (U.S. Dept of Commerce/NOAA), Chris Ostrom (National Ocean Service), Craig Hollingsworth (NOS Florida Keys National Marine Sanctuary), Mary Tagliareni (NOS Florida Keys National Marine Sanctuary), and Sean Morton (NOS Florida Keys National Marine Sanctuary). Also present, in addition to me, was Mr. Jeff Konczak with SuperGrout.

The purpose of the meeting was to familiarize all parties with the current state of the bulkhead and discuss the method of repairing the seawall put forth previously by Mr. Konczak. The past and present condition of the bulkhead is extensively documented in previous reports and will not be restated here. I refer the reader to the *Bulkhead Condition Reinvestigation* report submitted by TranSystems July 10, 2007.

On February 16, 2012 additional underwater video of bulkhead was taken to provide an up to date visual inspection of current conditions. This video, viewed during the site visit, does show the continued degradation of limestone material at the toe/dredge line of the bulkhead and some undermining of the tremie concrete repairs which were made in the Spring of 2006. Measurements indicate undermining in some areas of around 6". Presumably the undermining continues due to previously documented conditions of wave reflection resulting in erosion of the soft limestone bottom.

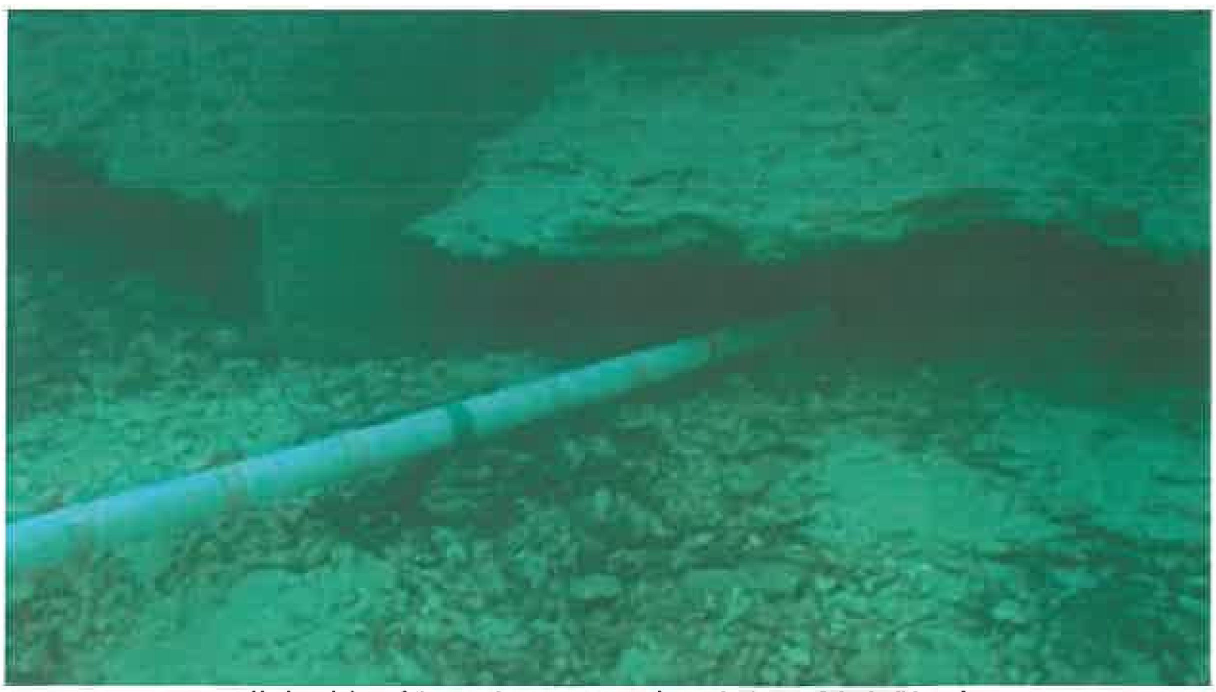
It should be noted that although this tremie concrete repair is performing as intended by preventing fill material from migrating through the sheets, there have been no hurricanes in the 6 years since its installation. Therefore its survivability and ability to perform in the wake of a hurricane event is unknown.

TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

Tremie concrete repair viewed from the top.



Undermining of the tremie concrete repairs at the base of the bulkhead.

TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

Undermining of the tremie concrete repairs at the base of the bulkhead. Note the steep slope of the dredge line.

Mr. Konczak presented his idea to the group for trenching and excavating behind the existing concrete bulkhead to an elevation at or below the bottom of the sheet piling. This excavation would then be filled with concrete. The newly poured concrete would bond to the rear of the existing sheets and fill in any cavities at the bottom. Formwork placed at the front of the wall would prevent the concrete from running out in to the dredged basin. In addition, concrete tieback anchors would be excavated and poured. See attached sketch of the Konczak repair. It was proposed that the repairs be done in segments in order to reduce the unsupported length of bulkhead and potential collapse. These exposed shorter segments would span laterally until reinforced with the repair.

At the time these sketches were developed by Mr. Konczak, he was unaware of the existing repairs made in the Spring 2006 therefore, these sketches do not show the tremie pour or Quikrete sacks placed at the outboard toe of the bulkhead. Though excavation and pouring concrete behind the bulkhead will plug some of the holes near the Quikrete sacks, the tremie pour repairs are still working to plug voids in the area of their repair. The erosion beneath the existing tremie repair has not yet migrated behind the wall.

In addition to the excavating and backfilling with concrete repair option, Jet Grouting was briefly discussed as a potential alternative to stabilizing the waterfront. Jet Grouting involves pressure injecting a cement rich grout in to the soils behind the bulkhead in order to strengthen the structural properties. The end result is the ability to analyze the grout stabilized section of soil as a gravity structure with enough mass and base adhesion to resist load transfer to the existing bulkhead. Both Mr. Konczak and I dismissed this idea due to applicability issues with the types of soil/rock/limestone material expected to be encountered, indeterminate quantity of grout material, and potential for high cost.

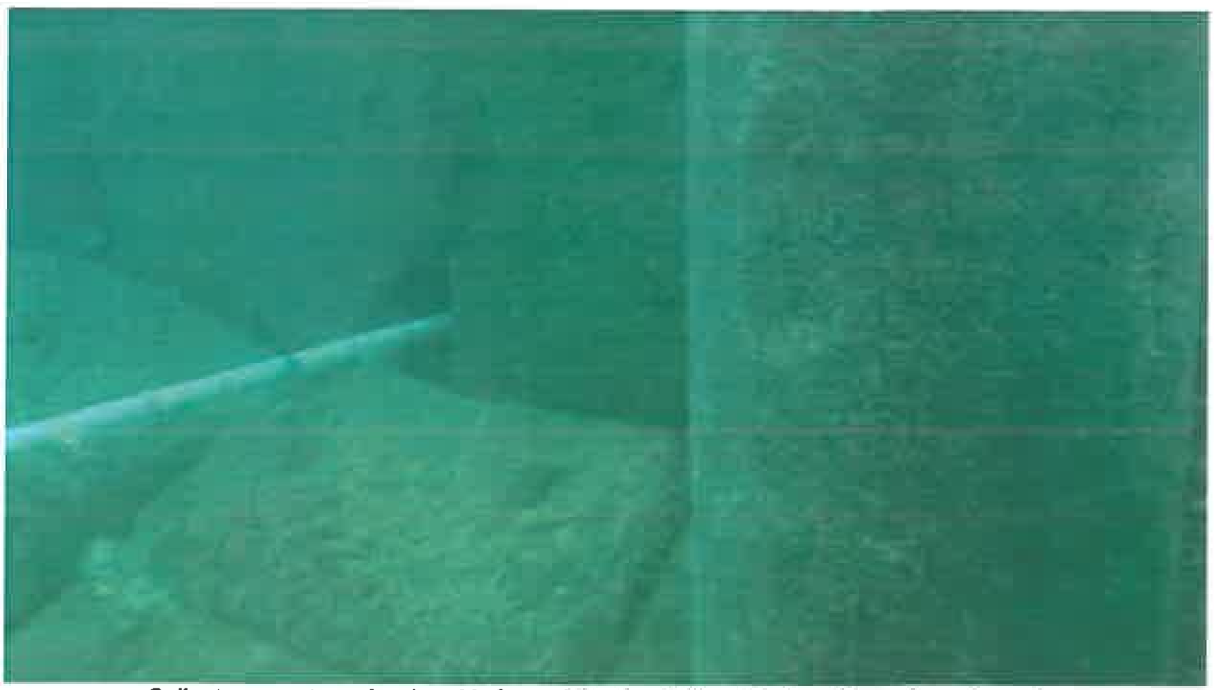
TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

From an engineering perspective, the main issue with the bulkhead at present is stability. The concrete sheet piles do not have adequate toe embedment to provide the fixity of the bulkhead at the base necessary for an earth retaining structure. Compounding this issue is the continued erosion of the limestone material at the toe. Therefore, my advice is not to proceed with trenching behind the bulkhead and filling with concrete as an alternative engineered solution.

If NOAA desires to pursue a near term solution that will plug the holes at the base of the bulkhead and reinforce to some degree the soft limestone bottom, I would recommend a tremie pour similar to what was done before. It is understood that this is not an engineered solution but rather a preventative maintenance measure against sinkholes. This approach was discussed openly amongst the group and the final concept is detailed in sketches SK-1, SK-2, and SK-3 attached. The tremie pour will be wide enough to encompass the Quikrete sacks on the eastern half of the site and high enough to cap the existing tremie pour on the west half.



Quikrete concrete sacks placed in front of the sheet piling. Cavity evident above the sacks.

Sean Morton and Mary Tagliareni indicated that currently there are no corals in the area of the suggested repair therefore there should be no impact in this regard. The inference here is that permitting should not be an issue.

The tremie concrete mix must be ideally suited for placing and performing in an underwater salt water environment. Mr. Konczak has made some very good suggestions in regards to the admixtures that should be specified in order to get a high performance product. These include the addition of micro-silica, anti-washout, and high range water reducer (super plasticizer) to a low slump Type 2 cement concrete mix. At the time of this report Mr. Konczak is researching the availability of this type of mix from the local batch plant near Key West. Should the local plant be unable to produce the mix required an option would be to have the dry mix material delivered and mixed with potable water on-site just prior to placement.

TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

For cost estimating purposes I have assumed the tremie pour will be on average 5' high by 3' wide and run the entire 300' of bulkhead. A general breakdown of estimated construction cost is as follows:

General Conditions (Mob, De-mob, etc.):	$50,000
Formwork (Dive team, placement underwater):	$57,500
Tremie (Material, placement underwater):	$34,000
Subtotal:	$141,500
Contingency (15%)	$21,225
Total:	**$162,725**

It should be noted that the estimated construction contract budget shown above only includes construction cost (labor, equipment, taxes, overhead, profit) and does not include escalation over time, contingency, A/E design fees, or government management fees. These additional costs should all be considered when determining overall funding requirements.

It has been my pleasure to work with NOAA and the Dr. Nancy Foster Florida Keys Environmental Complex personnel on this project as well as on our previous endeavors. I look forward to our continuing relationship. Please feel free to contact me at any time. Thank you.

Sincerely,

Robert Duke Snyder, P.E.
Structural Engineer
rdsnyder@transystems.com
Direct: (757) 963-8955
Cell: (757) 675-8907

TranSystems

101 West Main St., Suite 900
Norfolk, VA 23510
Tel 757 627 1112
Fax 757 627 1113

www.transystems.com

Distribution:

William Becker (William.J.Becker@NOAA.gov)
Contracting Officer/Supervisor, Eastern Acquisitions Division – Kansas City
601 East 12th Street, Room 1756
Kansas City, MO 64106-2808
(816) 426-7453

Carey Marlow (Carey.M.Marlow@NOAA.gov)
Contract Specialist, Eastern Acquisitions Division – Kansas City
601 East 12th Street, Room 1756
Kansas City, MO 64106-2808
(816) 426-7460

Jack Klaus (jack.c.klaus@noaa.gov)
Project Planning & Management Division – Eastern Region
601 East 12th Street, Room 1756
Kansas City, MO 64106-2808
(816) 426-7813

Sean A. Morton (sean.morton@noaa.gov)
Sanctuary Superintendent, NOS Florida Keys National Marine Sanctuary
33 East Quay Road
Key West, FL 33040
(305) 809-4700 x233

Chris Ostrom (chris.ostrom@noaa.gov)
National Ocean Service
1305 East West Highway; SSMC4 Room 11443
Silver Spring, MD 20910-3278
(301) 713-3125 x260

Jeff L. Reeder, PE (jlreeder@transystems.com)
Principal, Senior Vice President
TranSystems
2400 Pershing Road, Suite 400
Kansas City, MO 64108
(816) 329-8690

Tim L. Scott, PE (tscott@gdsengr.com)
Gibbons Drake Scott, Inc.
9201 E. 63rd Street, Suite 100
Raytown, MO 64133
(816) 358-1790

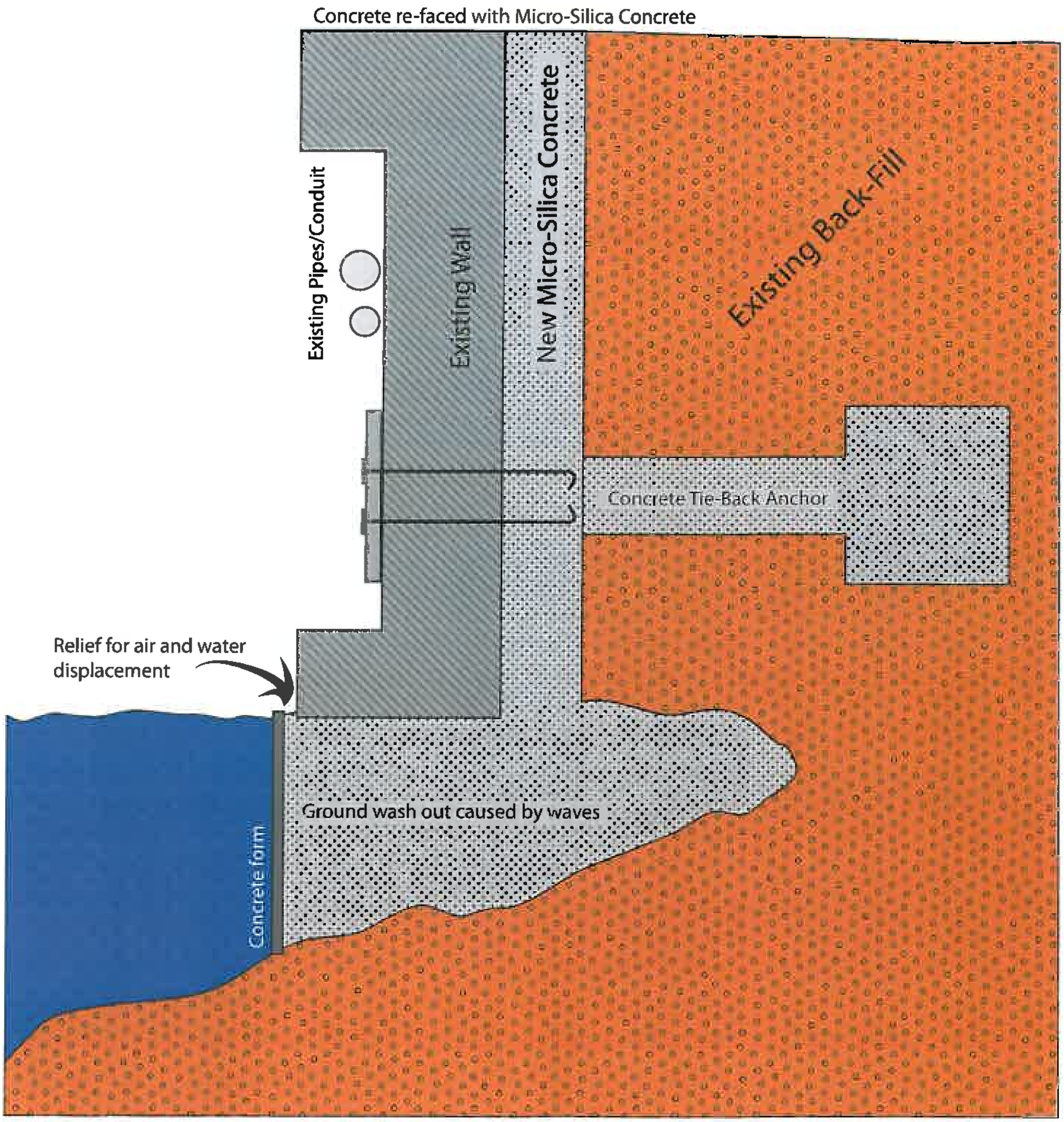
Nancy Foster Center

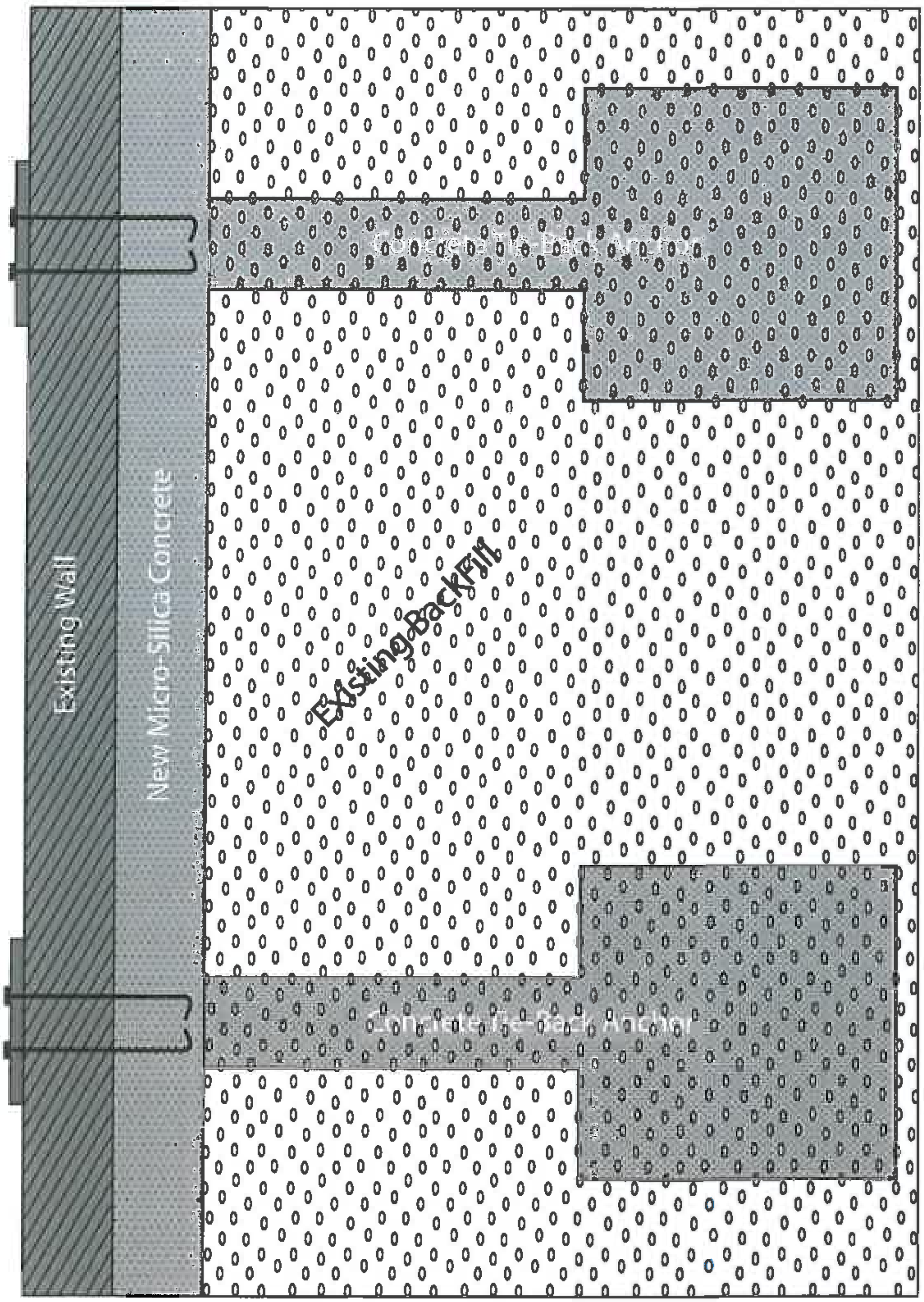
Florida Keys National Marine Sanctuary

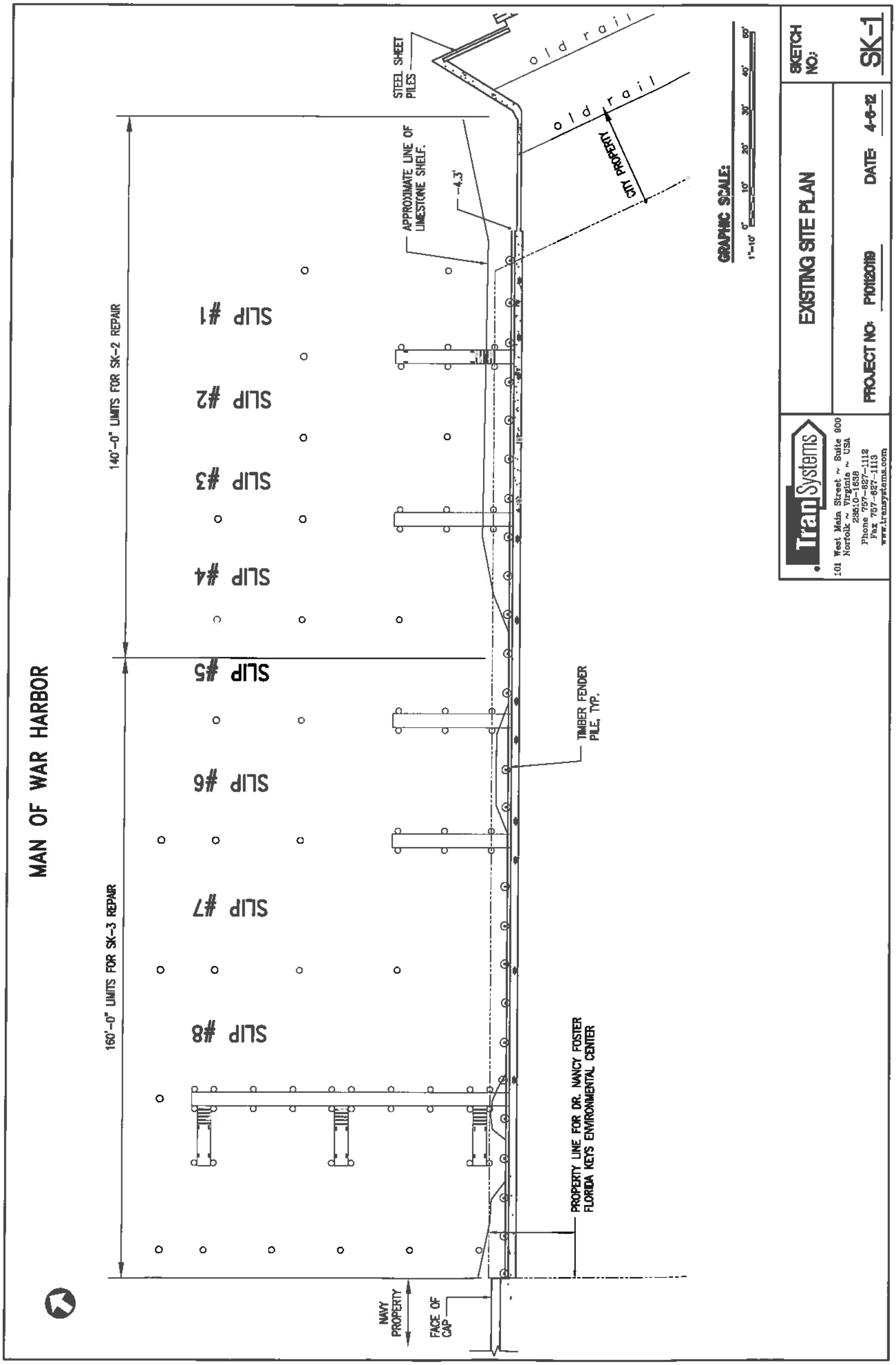
Conceptual Break Wall Repair

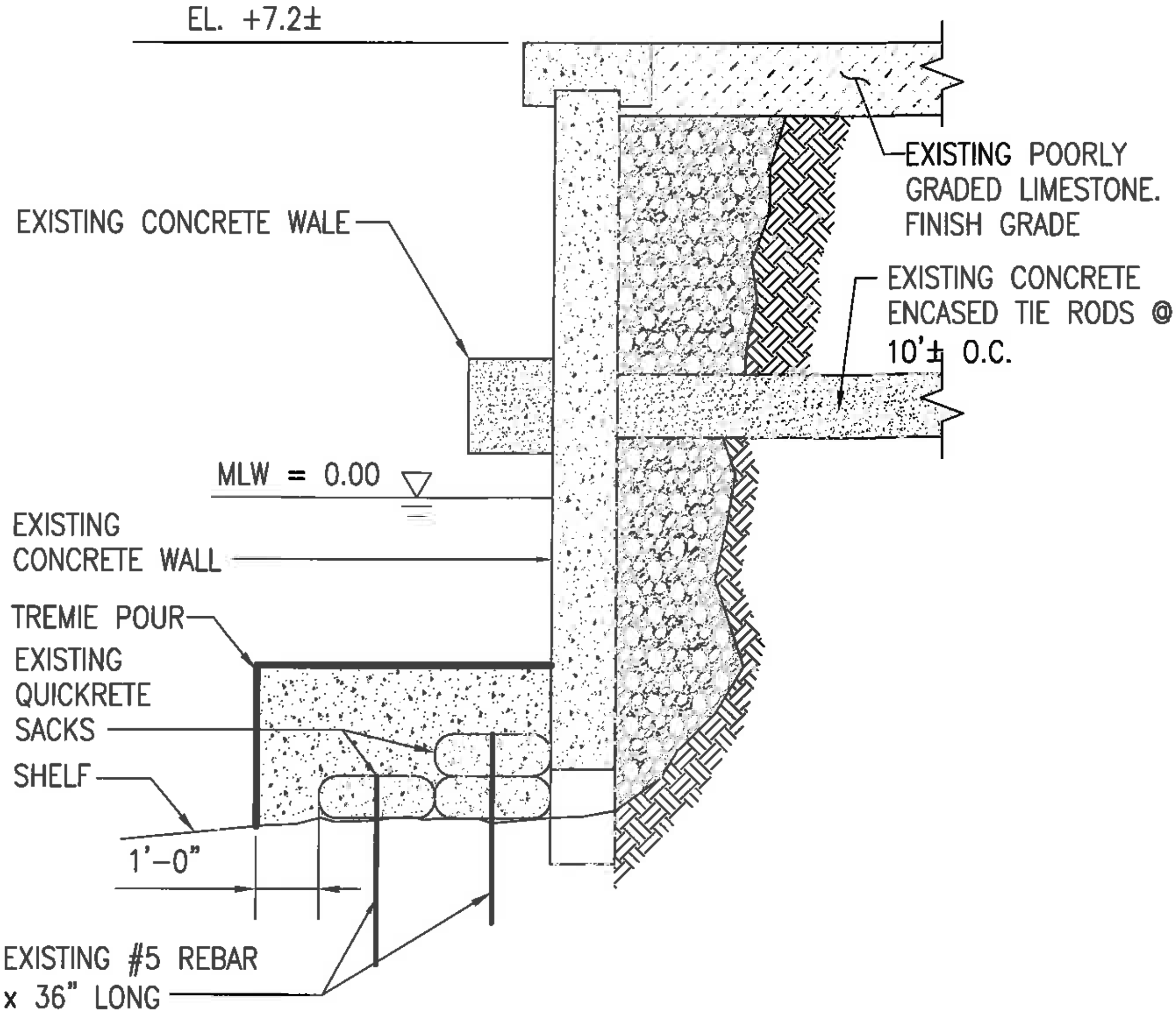
Jeff Konczak
Alpena Marc LLC

Micro-Silica Concrete
with BASF Anti-Washout Admixture
will fill already washed out areas even while water is still present




Existing Wall
New Micro-Silica Concrete
Existing BackFill
Concrete Tie-Back Anchor
Concrete Tie-Back Anchor


MAN OF WAR HARBOR
140'-0" LIMITS FOR SK-2 REPAIR
160'-0" LIMITS FOR SK-3 REPAIR
SLIP #1
SLIP #2
SLIP #3
SLIP #4
SLIP #5
SLIP #6
SLIP #7
SLIP #8
STEEL SHEET PILES
old rail
old rail
APPROXIMATE LINE OF LIMESTONE SHELF.
-4.3'
CITY PROPERTY
TIMBER FENDER PILE, TYP.
PROPERTY LINE FOR DR. NANCY FOSTER FLORIDA KEYS ENVIRONMENTAL CENTER
NAVY PROPERTY
FACE OF CAP
GRAPHIC SCALE:
1"=10'
0'
10'
20'
30'
40'
50'
TranSystems
101 West Main Street ~ Suite 900
Norfolk ~ Virginia ~ USA
23510-1638
Phone 757-627-1112
Fax 757-627-1113
www.transystems.com
EXISTING SITE PLAN
PROJECT NO: P101120119
DATE: 4-6-12
SKETCH NO:
SK-1



TYPICAL SECTION THRU BULKHEAD

SCALE: 3/8" = 1'-0"

TranSystems
101 West Main Street ~ Suite 900
Norfolk ~ Virginia ~ USA
23510-1638
Phone 757-627-1112
Fax 757-627-1113
www.transystems.com

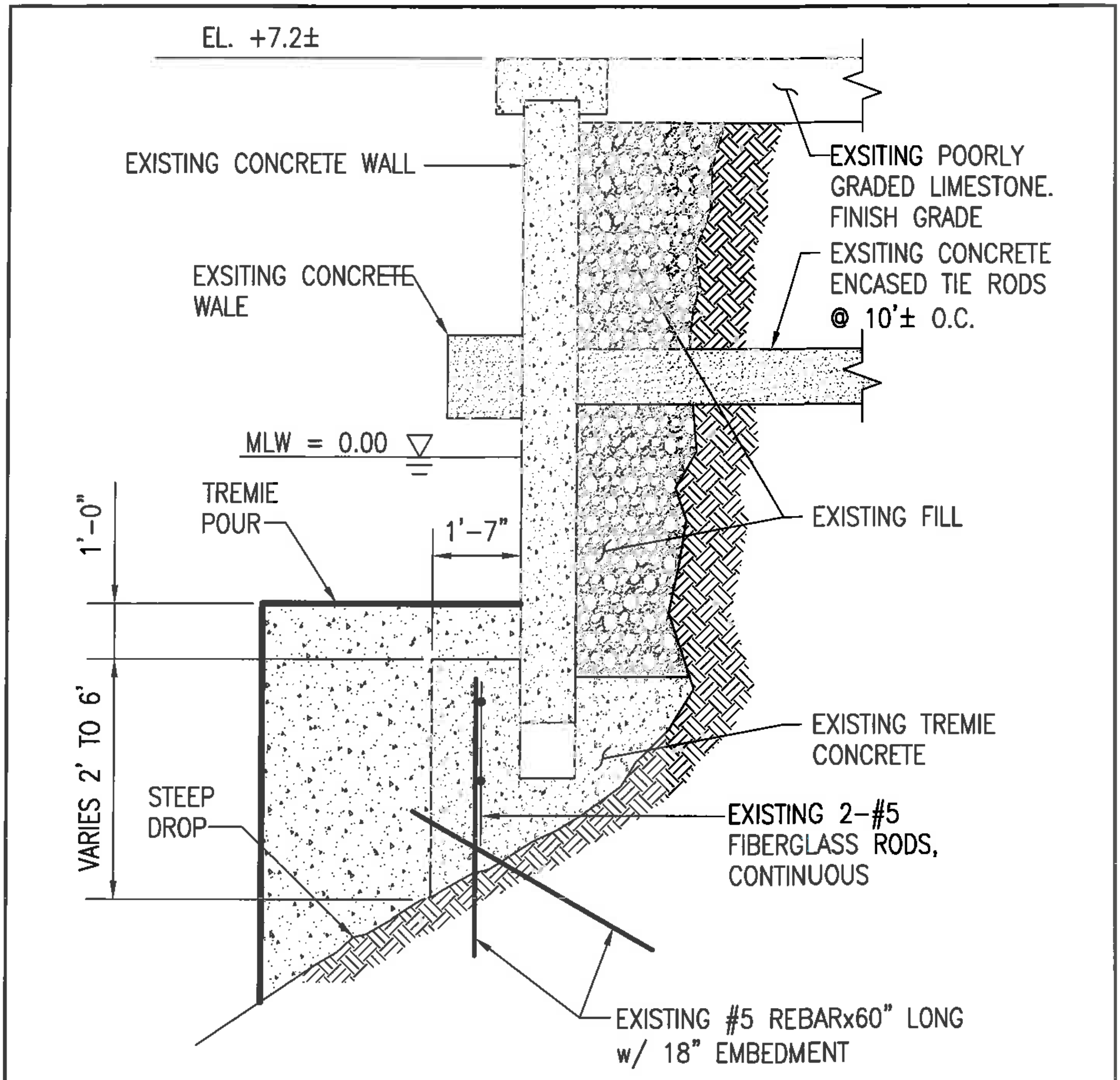
TYPICAL SECTION THRU BULKHEAD

1' 0 1' 5'

PROJECT NO: P101120119

DATE: 4-6-12

SKETCH NO.: SK-2



TYPICAL SECTION THRU BULKHEAD

SCALE: 3/8" = 1'-0"

TranSystems

101 West Main Street ~ Suite 900
Norfolk ~ Virginia ~ USA
23510-1638
Phone 757-627-1112
Fax 757-627-1113
www.transystems.com

TYPICAL SECTION THRU BULKHEAD

1' 0 1' 5'

PROJECT NO: P101120119

DATE: 4-6-12

SKETCH NO.: SK-3