TETRA TECH

July 22, 2015

James W. Bouquet P.E.
Director of Engineering
City of Key West
3140 Flagler Ave
Key West, FL 33040

Subject: Truman Seawall Repair Recommendations (PO No. 079755)

Dear Mr. Bouquet,

Please find attached our updated engineering assessment for the repair or replacement of the Truman Seawall in the City of Key West (City). Tetra Tech and our geotechnical engineering sub-consultant (Anderson Andre) conducted a field visit on June 23, 2015 to collect measurements and soil borings and the results of these field visits are included as attachments to this letter. In addition we meet with a qualified marine contractor, Custom Built Marine Construction (CBMC), who reviewed the boring logs, and provided a constructability analysis and budget level cost estimate for two wall repair options.

It is our intention that this report and its attachments be used by the City to determine the type and configuration of the seawall upgrade that best suits the site conditions and funding available. After a wall repair method has been selected, a site specific riparian survey will need to be ordered and final engineering analysis performed to select the appropriate material characteristics. The survey and the construction plans produced at that time can then be submitted for permitting the repair. Please feel free to call me to discuss this assessment or if you have any questions or need any additional information.

Sincerely,

7.22.15

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

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

cc: Terrence Justice, City of Key West
Shauna Stotler-Hardy, Tetra Tech

759 South Federal Highway, Suite 314 Stuart, FL 34994
Tel 772.781.3400 Fax 772.781.3411 www.tetratech.com
CA #: 9645

2015

TRUMAN SEAWALL

CITY OF KEY WEST

SEAWALL REPAIR & REPLACEMENT OPTIONS

TETRA TECH

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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 and is approximately 40 feet deep. It is my understanding that it was used as a diesel-powered submarine base known as Marker 57 until it was closed in 1974. Tetra Tech was hired to conduct a remediation assessment of a 340 feet segment of the 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 information we have reviewed but the thickness and length of the piles seems to vary depending on the location and depth of water at the toe. The embedment depth along the wall is also unknown but it appears the wall panels were placed on the top of the limestone rock (with little or no embedment) and now the panels have been undermined in several places. There is an existing wale beam located just above the waterline that has been encapsulated in concrete. The condition and configuration of the wale beam and the tie-back system is unknown and was not investigated as a part of this assessment.

The panels may have been “toed-in” to the limestone when they were originally installed but the steep slopes in the front of the seawall, possible wave refraction, and idle prop-wash seems to have caused certain areas to degrade. The erosion of the toe has been monitored for several years and there are voids that have been documented and continue to worsen. An April 24, 2012 report by TranSystems was reviewed buy Tetra Tech that shows degradation of limestone at the toe and undermining of the seawall. 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.

Summary

Tetra Tech performed a site visit on Tuesday June 23, 2015 beginning at 10:30 am and walked the length of the seawall to observe the proposed boring locations and the buried utility markers. The water level in the basin was measured between 10:30 am and 11:30 am and found to be approximately 7-feet below the top of the seawall cap at that time.

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 water ward of the seawall at each of the existing docks. A copy of the results of our measurements are attached as **Exhibit 1** - Depths. Site Photos taken during the inspection are included as **Exhibit 2**.

We used the measurements collected during the inspection 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 that were then shared with a marine contractor who performed a constructability review and provided a means and methods assessment.

The basin cross sections and wall repair / reconstruction schematics are included in this report as **Exhibits 3 – 7**. **Exhibit 3** shows the cross section near Dock 1 and used a concrete panel wall in the cross section. This was done to show that the contractor will need to bore through the limestone to set the panels. **Exhibit 4** shows the cross section near Dock 2 and shows a steel sheet pile section. The geotechnical engineer has indicated the contractor should be able to vibrate steel sheets without punching or boring into the limestone. **Exhibit 5** shows the cross section near Dock 3 and uses a steel cross section. The water depths along the face of the wall increase in this area and the slop away from the toe is steeper. Previously made repairs will require the wall to be placed a little farther water ward and/or require the previous repairs to be removed. **Exhibit 6** shows the cross section near Dock 4 and uses a steel sheet pile. Water depths at the face continue to increase. Slope away from the face of the wall continue to increase. **Exhibit 7** shows the cross section near Dock 5 and uses a steel section. This particular cross section shows the steepest slope away from the face of the wall which may require longer sheets.

The boring logs collected during the geotechnical investigation and shared with a marine contractor are included as **Exhibit 8**. The TranSystems report is included as **Exhibit 9** because of the underwater photos that are included.

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 these 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 is indicative of the scour conditions 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 where the existing concrete sheet pile typically stops. An N value of 33-42 is pretty stiff but still "drivable" with steel sheet pile. I suspect that this was 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 of refusal for standard pile driving methods. Any recommendation for seawall replacement should prescribe the toe of the wall to be embedded into this layer.

Exhibit 3 shows how a concrete panel wall could be used easily in front of the exiting wall if placed immediately water ward of the protruding wale beam cap. Standard FDOT concrete panels would be specified at 10-12" wide and 20-22' long for this segment. A large excavator will be needed with and auger attachment to core into the rock prior to placement of the concrete panels. The voids left between the installed panel and the core hole will need to be filled with stone, pea gravel, or grout depending on the budget.

Exhibit 4 has the same general cross section but uses a steel sheet pile in the cross section to distinguish between the types of installation impacts that may be encountered. Specifically it eliminates the need for the wet-coring that would be required to place the concrete panels water ward of the existing wall.

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 pouring a concrete toe reinforcement. This concrete repair creates a constructability issue and is shown schematically in Exhibits 5-7. 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 insure 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. We will recommend that this be field verified with a series of test piles.

Another alternative for installation of either the concrete panel wall or a steel sheet pile wall would be the installation of the replacement wall landward of the existing wall. The contractor we contacted (CBMC) has indicated that this would allow them to bench down behind the wall during construction and would allow more embedment into the limestone. The old wall could be removed after installation was completed and rip-rap could be added to toe to mitigate scour risks in the future.

Budgeting

Stuart McGahee from Tetra Tech met with the lead estimator, Lee Corrigan from Custom Built Marine Construction, CBMC (772-333-2385) to discuss constructability and develop a cost estimate range for the wall construction methods discussed above. We discussed the geotechnical engineering results and both the concrete panel wall and steel sheet pile wall construction options.

Concrete Panel Wall
The budget number for a 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 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***.

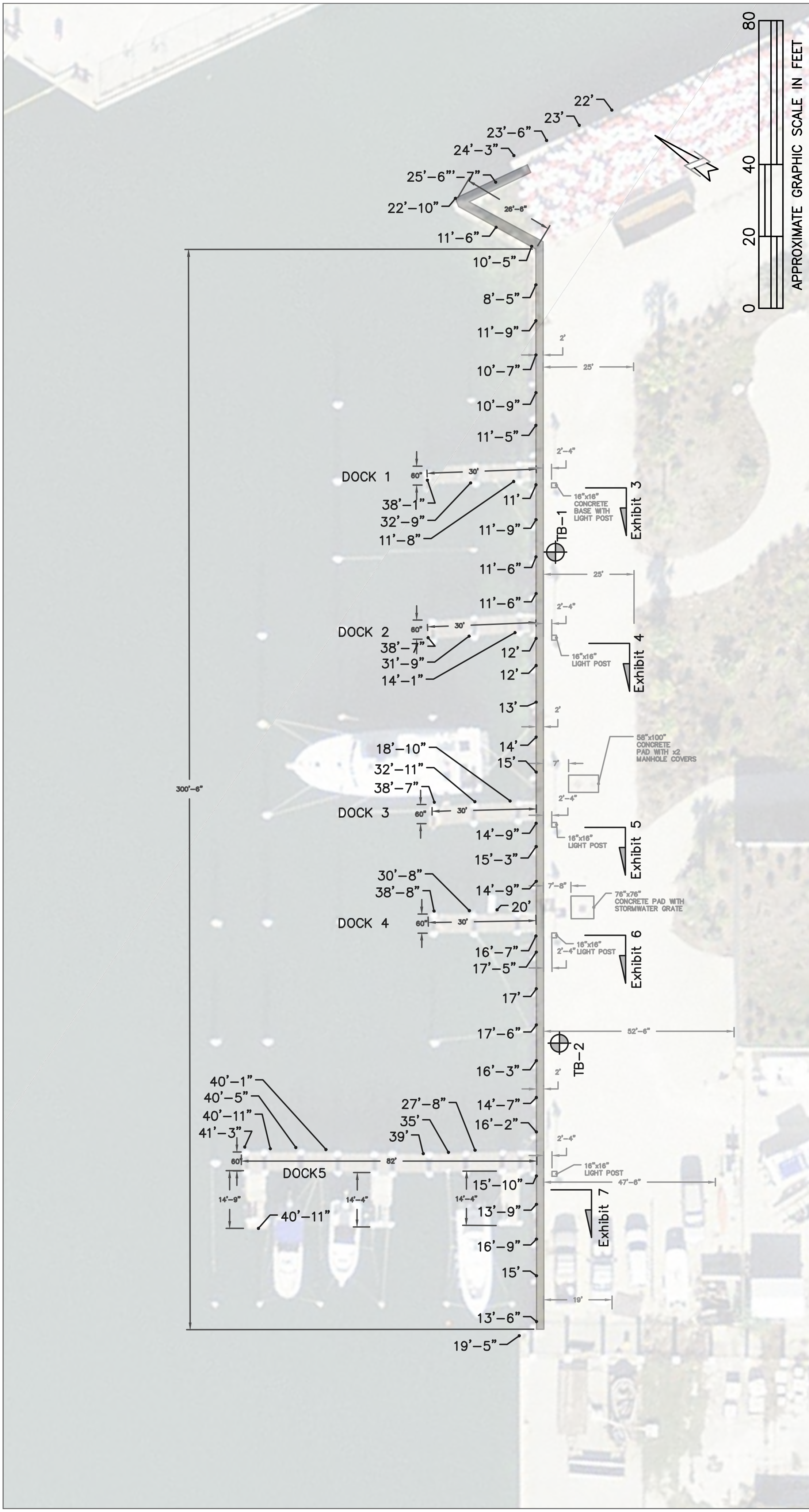
These estimates were provided by CBMC on a preliminary basis and represent only one contractor's expectations. Any contractor should be allowed to perform a field visit prior to submittal of a formal bid for the work.

For budgeting the following additional services that will be required prior to construction include:

1. A formal riparian and bathymetric survey,
2. Preparation of Plans and Specifications
3. Regulatory pre-application field visits
4. Permitting application
5. Responding to regulatory request for information
6. Preparation of Final Plans and Specifications
7. Procurement assistance and,
8. Construction services.

Life Cycle Expectations

Both the Concrete and Steel sheetpile walls are expected to have a life expectance of over 25-years. Life expectancy beyond that is uncertain. The concrete will not corrode as visibly but the chloride interaction with the concrete and the tie-back system will eventually cause spalling and other degradation. The sheetpile walls (even though they will be coated) will quickly begin to show corrosion. Especially around the knuckles 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.



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

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
	CHECKED:	DRAWING NO. G-01
DATE:	APPROVED:	

EXHIBIT 2 - 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.

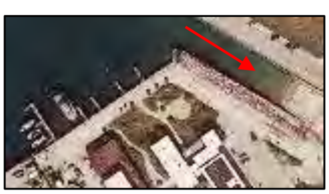


Photo: 2

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.

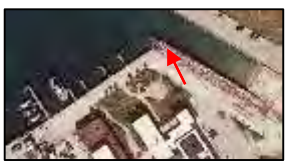


Photo: 3

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.

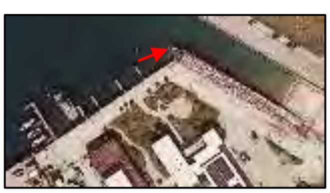


EXHIBIT 2 - Site Photos
Key West NOAA Seawall

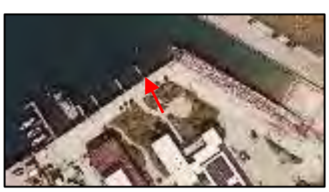
Key West, Monroe County, Florida
Tetra Tech, Inc.
Project: 194-5363

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.



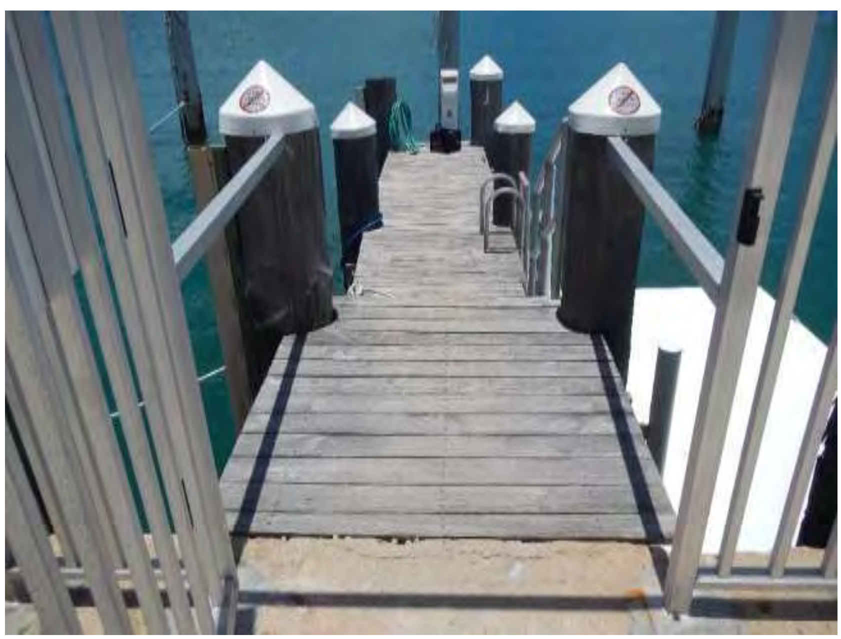


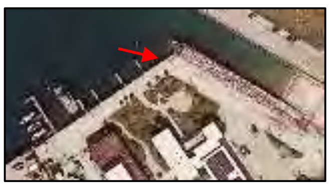
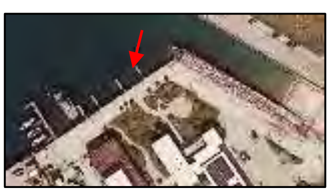
Photo: 6 **Description:** View of the seawall from Dock 1 – left. 	
Photo: 7 **Description:** View of the seawall from Dock 1 – right. 	

Photo: 8

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

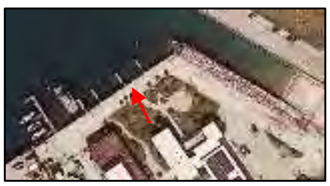


Photo: 9

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

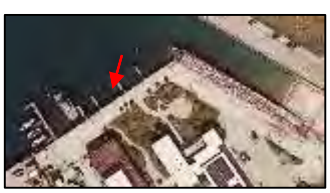


Photo: 10

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

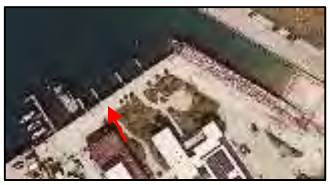


Photo: 11

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.

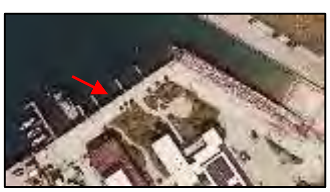


Photo: 12

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

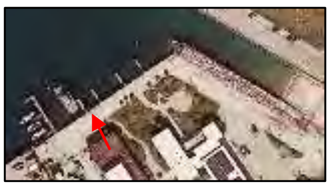


Photo: 13

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

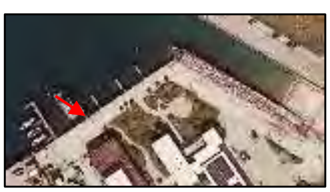


EXHIBIT 2 - Site Photos
Key West NOAA Seawall

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

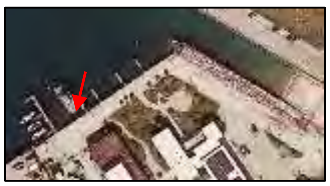
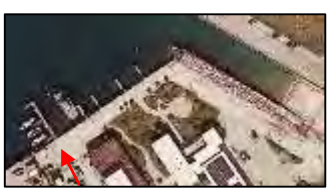
Photo: 14 **Description:** View of the seawall from Dock 4 – right. 	
Photo: 15 **Description:** The fifth and last dock along the NOAA seawall, Dock 5. This dock goes out about 82 feet, and has 3 sets of steps leading to slips along the left of the dock. 	

Photo: 16

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

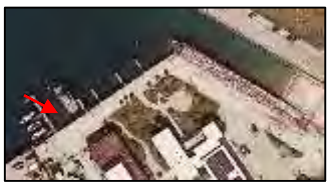


Photo: 17

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.

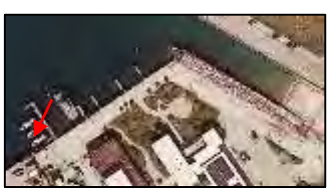


Photo: 18

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.

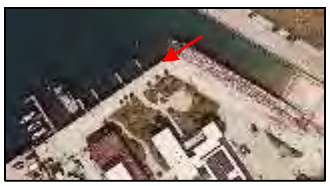


Photo: 19

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

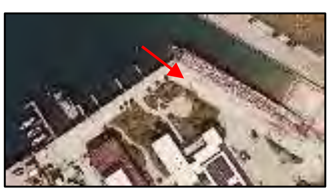


Photo: 20

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

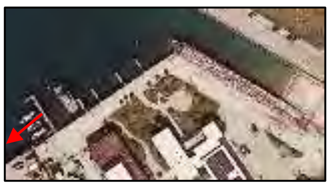
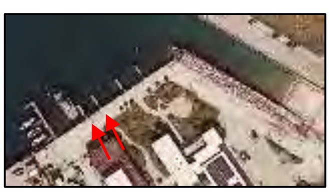
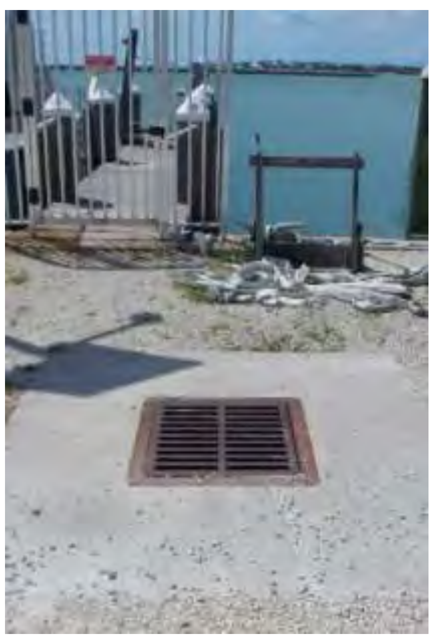
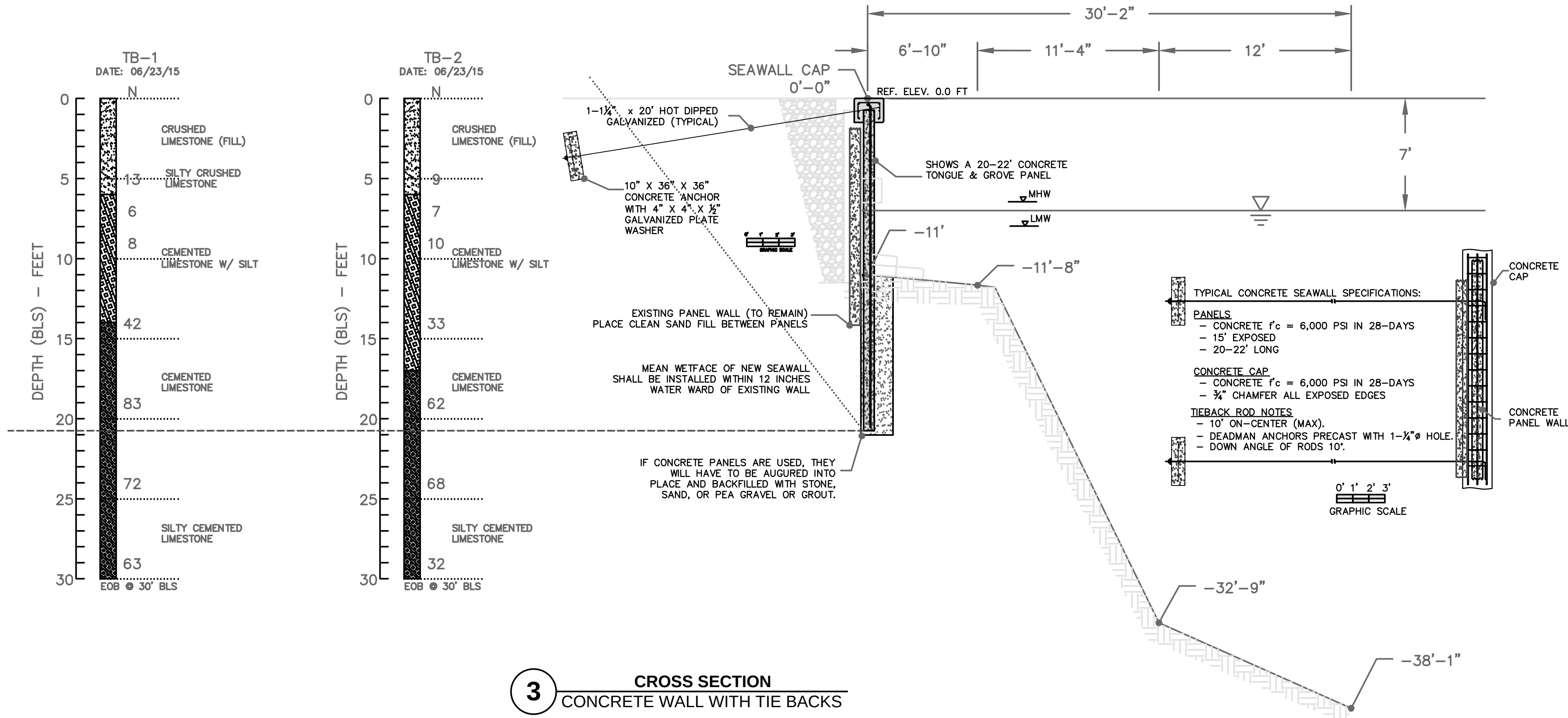


Photo: 21
Description:

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.







3 **CROSS SECTION**
CONCRETE WALL WITH TIE BACKS
NEAR DOCK 1

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.

LEGEND:

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

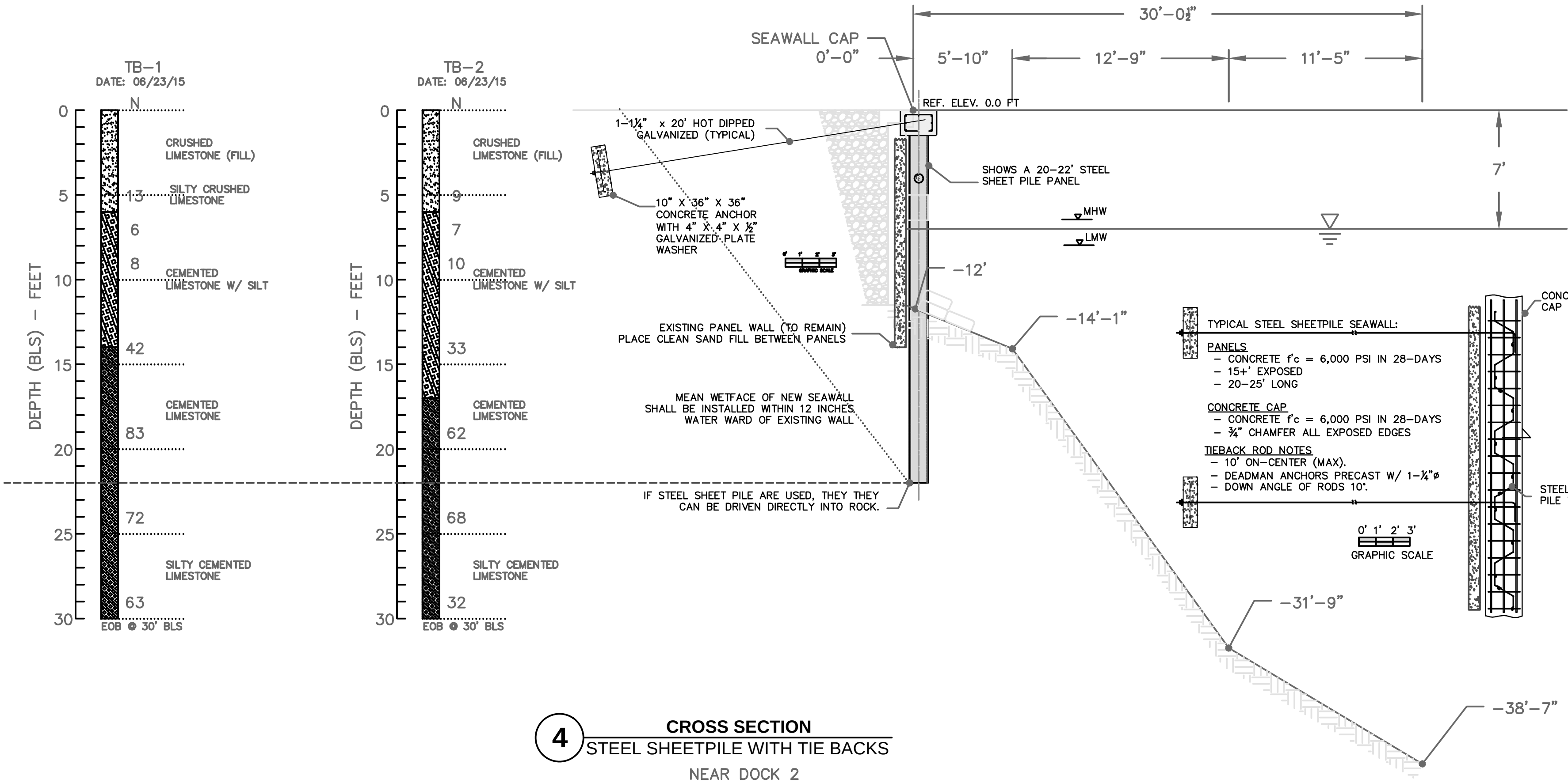
DEPTH — AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET

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

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 3

TETRA TECH, INC.

SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. CS-DOCK 1



4 CROSS SECTION

STEEL SHEETPILE WITH TIE BACKS

NEAR DOCK 2

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.
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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'

SCALE: AS SHOWN
DATE:

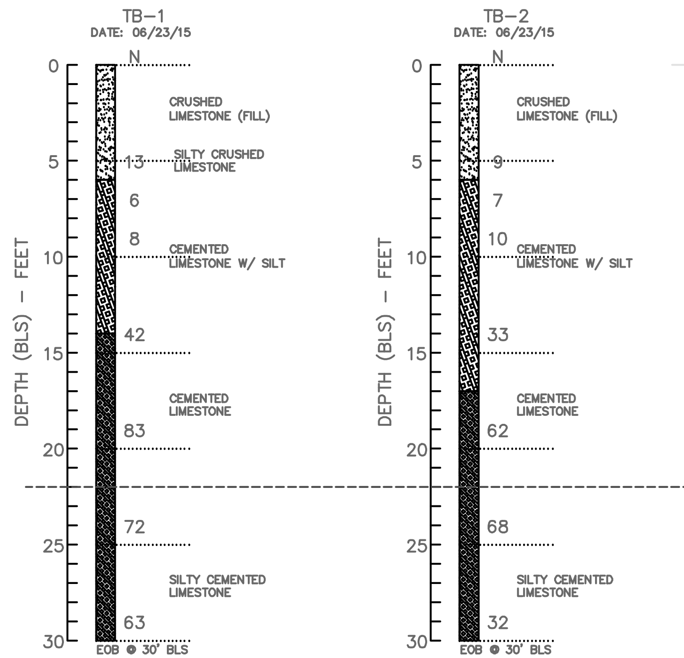
PREPARED: SEM
CHECKED:
APPROVED:

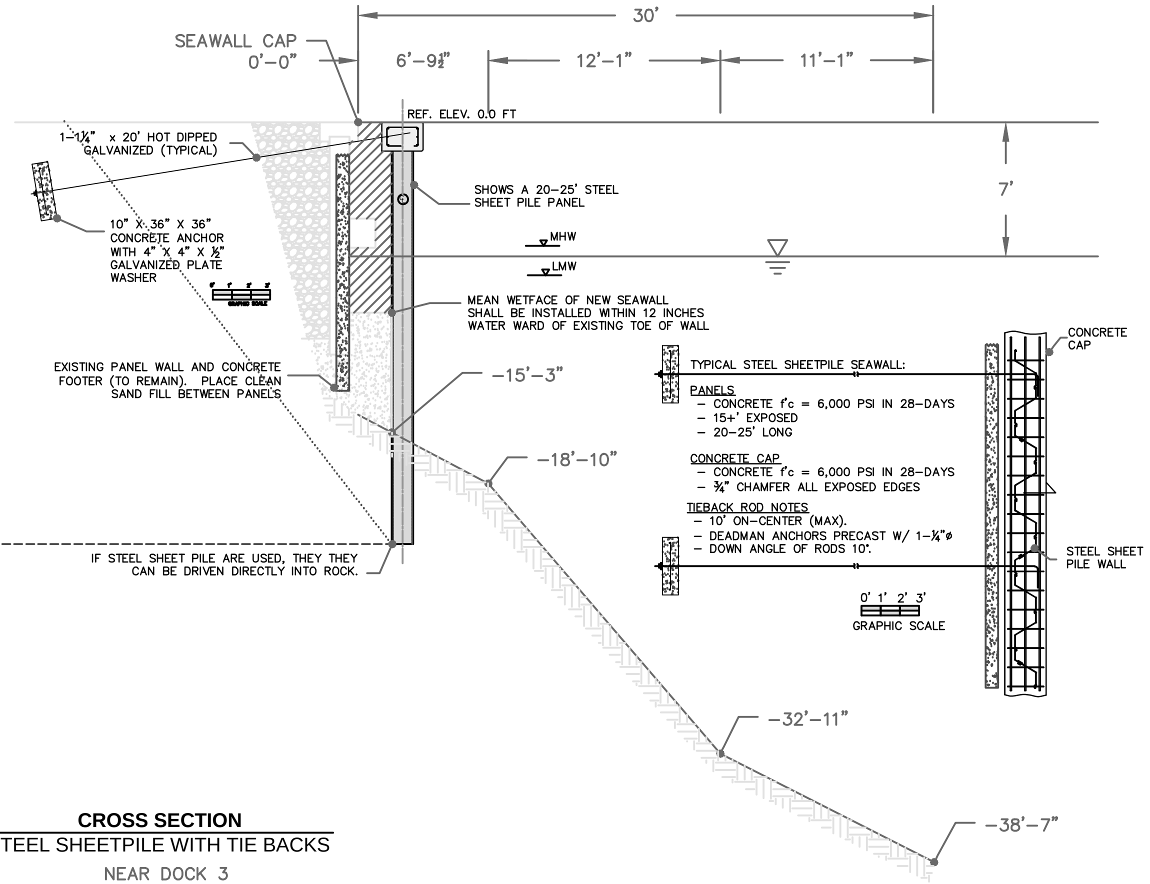
CAD FILE NO. Cross Sections
DRAWING NO. CS-DOCK 2

TETRA TECH, INC.

TRUMAN SEAWALL
KEY WEST, FL

EXHIBIT 4





5 **CROSS SECTION**
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 3

NOTES:

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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.

LEGEND:

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

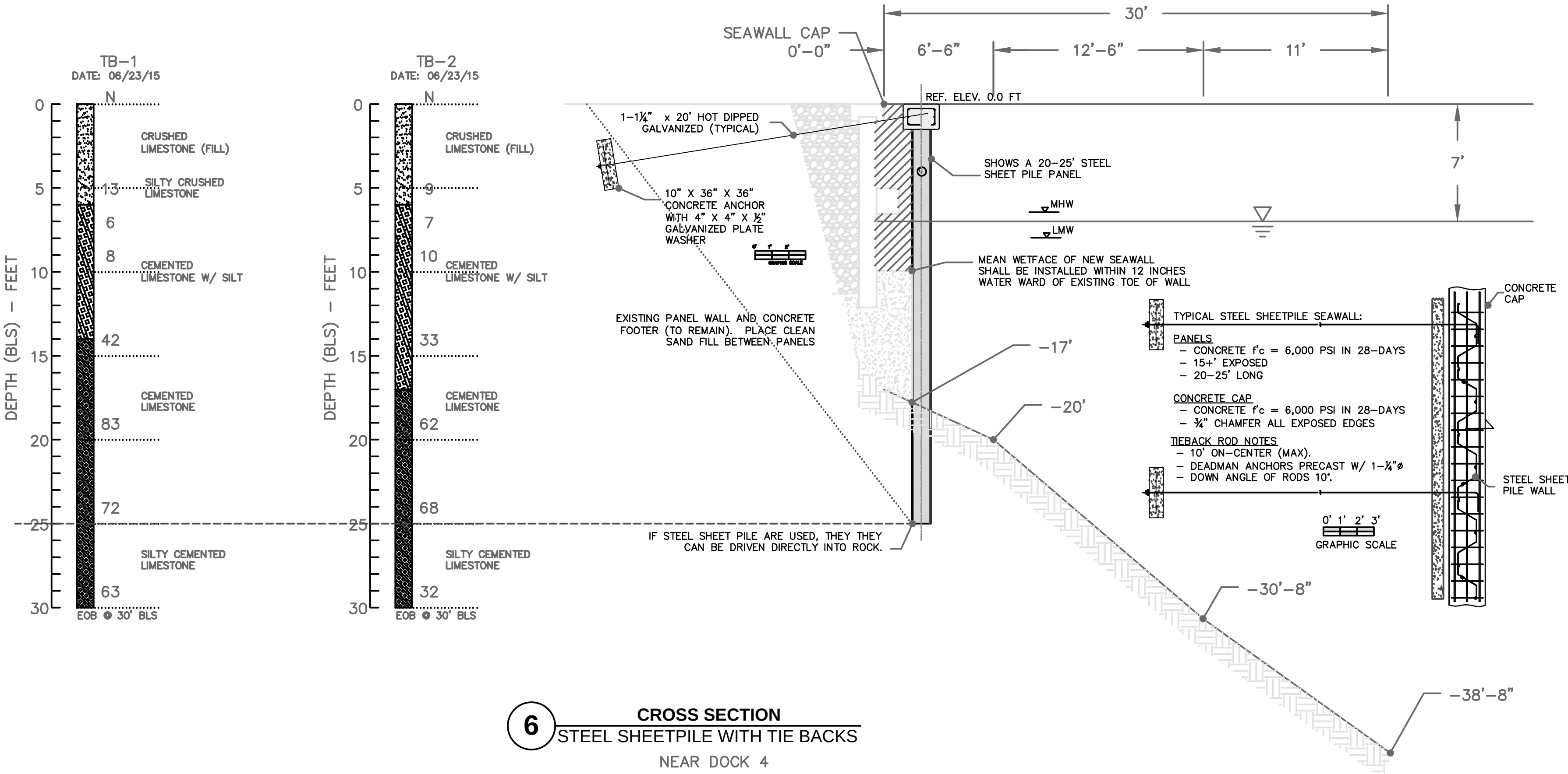
DEPTH — AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET

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

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 5

TETRA TECH, INC.

SCALE: AS SHOWN
DATE:
PREPARED: SEM
CHECKED:
APPROVED:
CAD FILE NO. Cross Sections
DRAWING NO. CS-DOCK 3



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

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.

LEGEND:

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

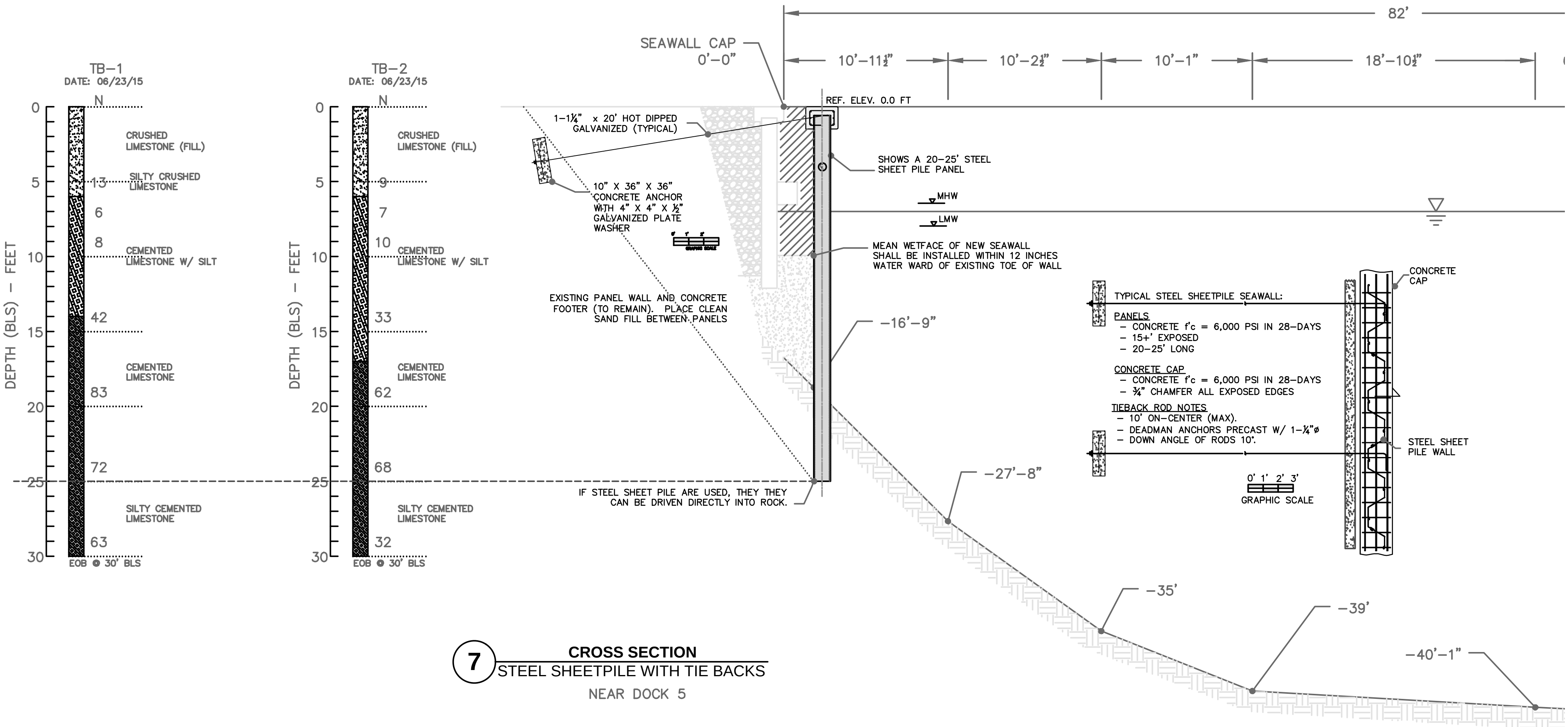
DEPTH — AVERAGE DISTANCE FROM TOP OF WALL TO WATER WAS: 7-FEET

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

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

TETRA TECH, INC.

TRUMAN SEAWALL
KEY WEST, FL
EXHIBIT 6



7 **CROSS SECTION**
STEEL SHEETPILE WITH TIE BACKS
NEAR DOCK 5

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.

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 7

TETRA TECH, INC.

SCALE: AS SHOWN
DATE:

PREPARED: SEM
CHECKED:
APPROVED:

CAD FILE NO. Cross Sections
DRAWING NO. CS-DOCK 5

EXHIBIT 8

Geotechnical Engineering 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

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
Geotechnical Engineering
Construction Materials Testing
Environmental Consulting

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.

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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 GREGERS ANDERSEN LICENSE No. 57956 STATE OF FLORIDA PROFESSIONAL ENGINEER

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

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

N
W E
S

NOT TO SCALE

Latitude: 24.55115
Longitude: -81.80786

USDA SOIL TYPES ON SITE:

No. 11: URBAN LAND

BORING LOCATION PLAN

Source: Google Earth Pro

N
W E
S

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

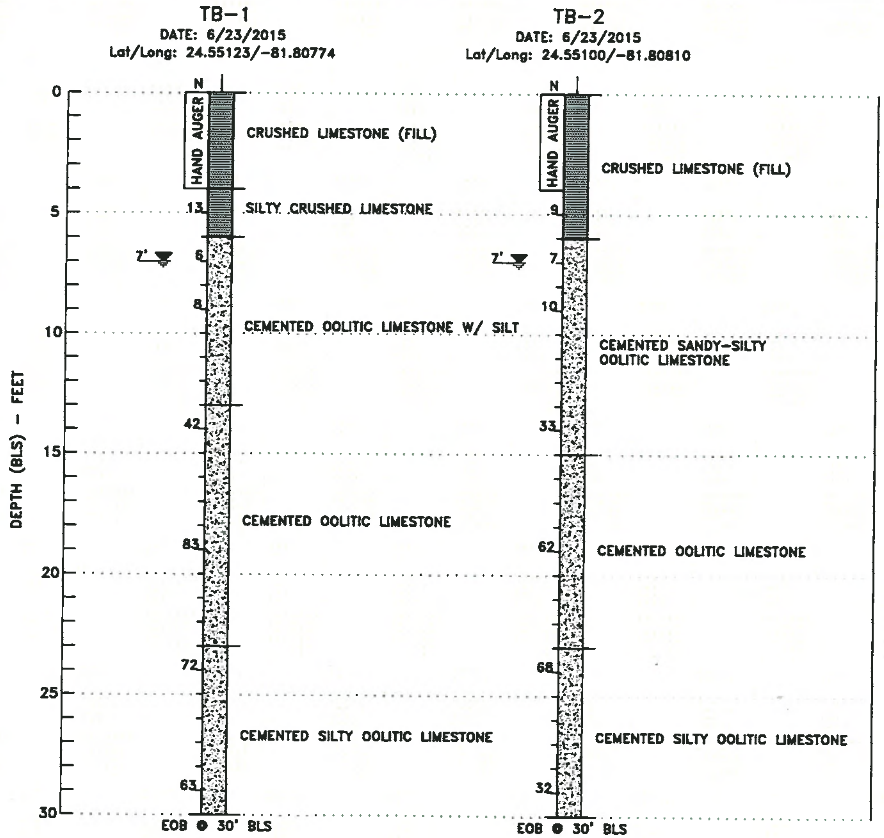
NOTES AND LEGEND

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

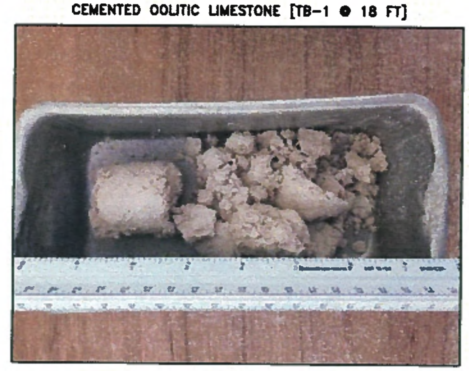
VICINITY MAPS AND BORING LOCATION PLAN	SUBSURFACE SOIL EXPLORATION AND PRELIMINARY GEOTECHNICAL ENGINEERING EVALUATION PROPOSED SEAWALL REPLACEMENT TRUMAN WATERFRONT KEY WEST, MONROE COUNTY, FLORIDA	Drawn by: PGA	Date: June/July 2015
		Checked by: DPA	Date: June/July 2015
		AACE File No: 15-148	Sheet No. 1



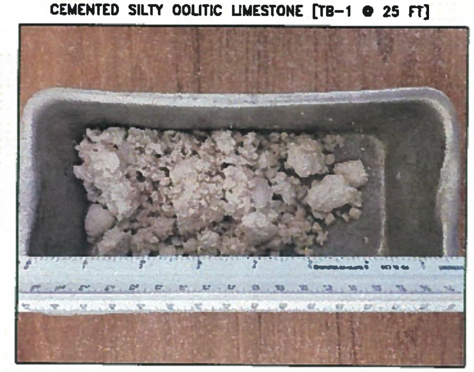
CRUSHED LIMESTONE (FILL) [TB-1 @ 3 FT]

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 Checked by: DPA AACE File No: 15-148	Date: June/July 2015 Date: June/July 2015 **Sheet No. 2**

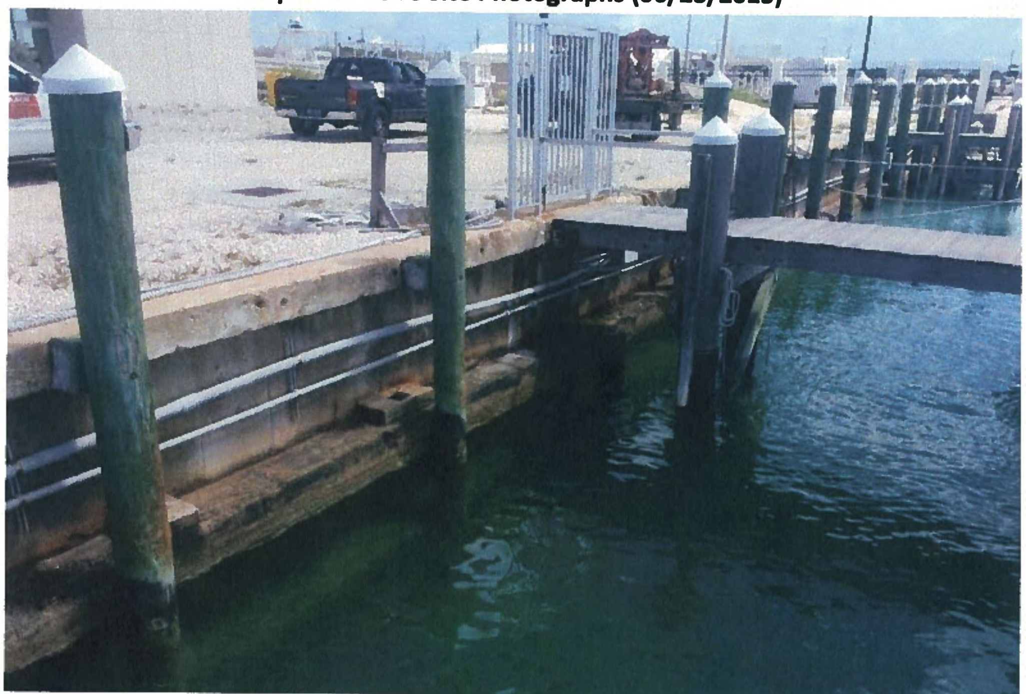
<u>APPENDIX I</u>

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

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Phone: 772.807.9191 Fax: 772.807.9192
www.aaceinc.com

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

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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

<u>Appendix II</u>

USDA Web Soil Survey Summary Report

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

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

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
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)

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

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)

<u>APPENDIX IV</u>

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 9

TranSystems Report (for Reference)

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

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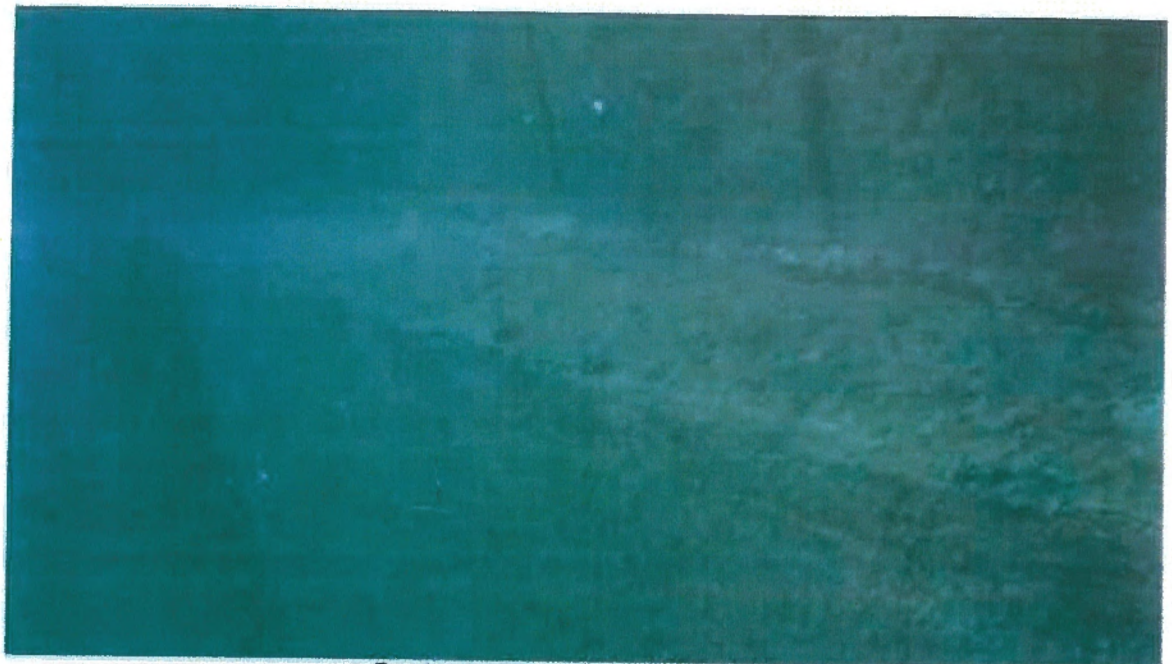
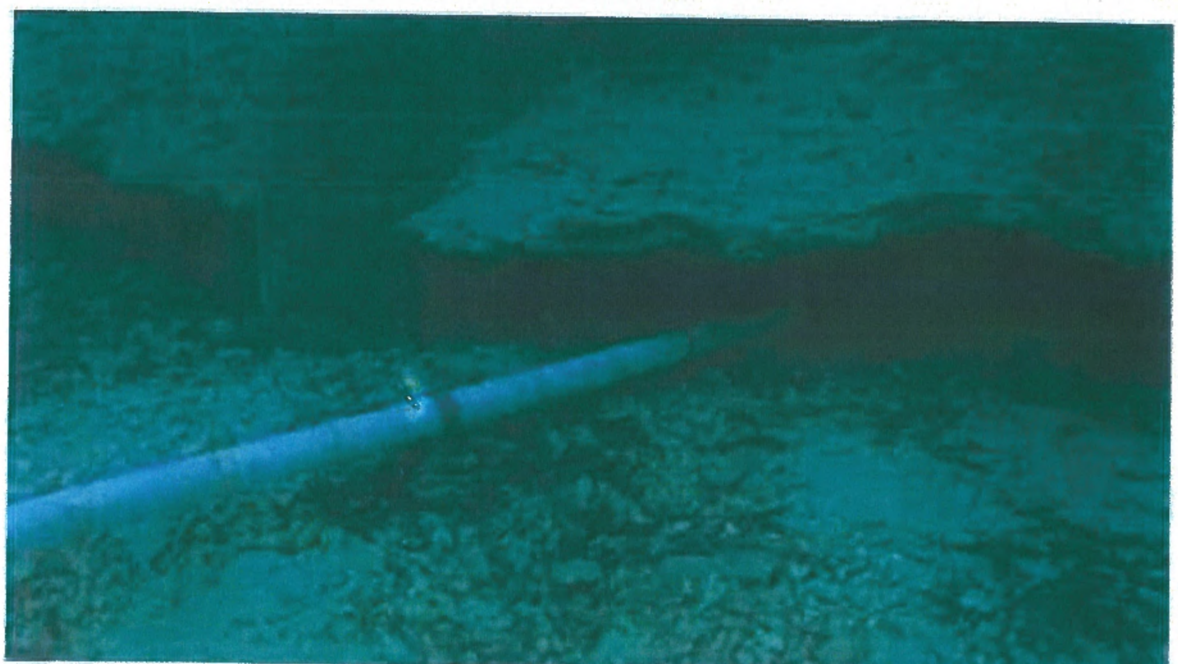


Tremie concrete repair viewed from the top.



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

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

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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.

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

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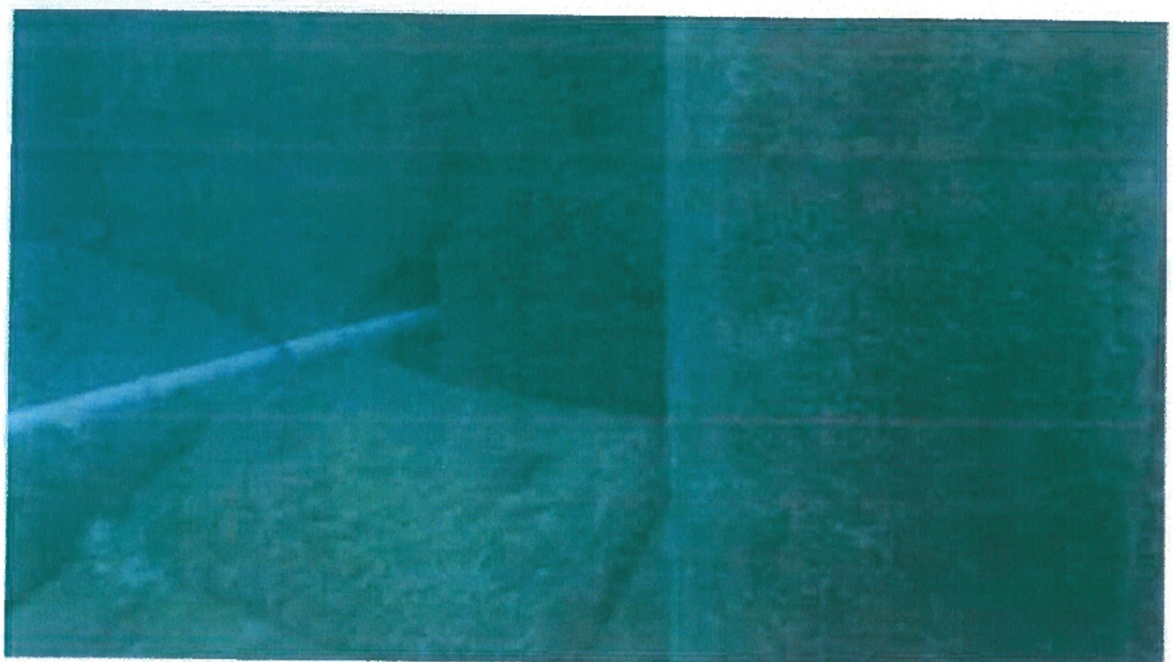
From an engineering perspective, the main issue with the bulkhead at present is stability. The concrete sheet piles do not have adequate toe embedment to provide the fixity of the bulkhead at the base necessary for an earth retaining structure. Compounding this issue is the continued erosion of the limestone 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.

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

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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

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

TranSystems

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

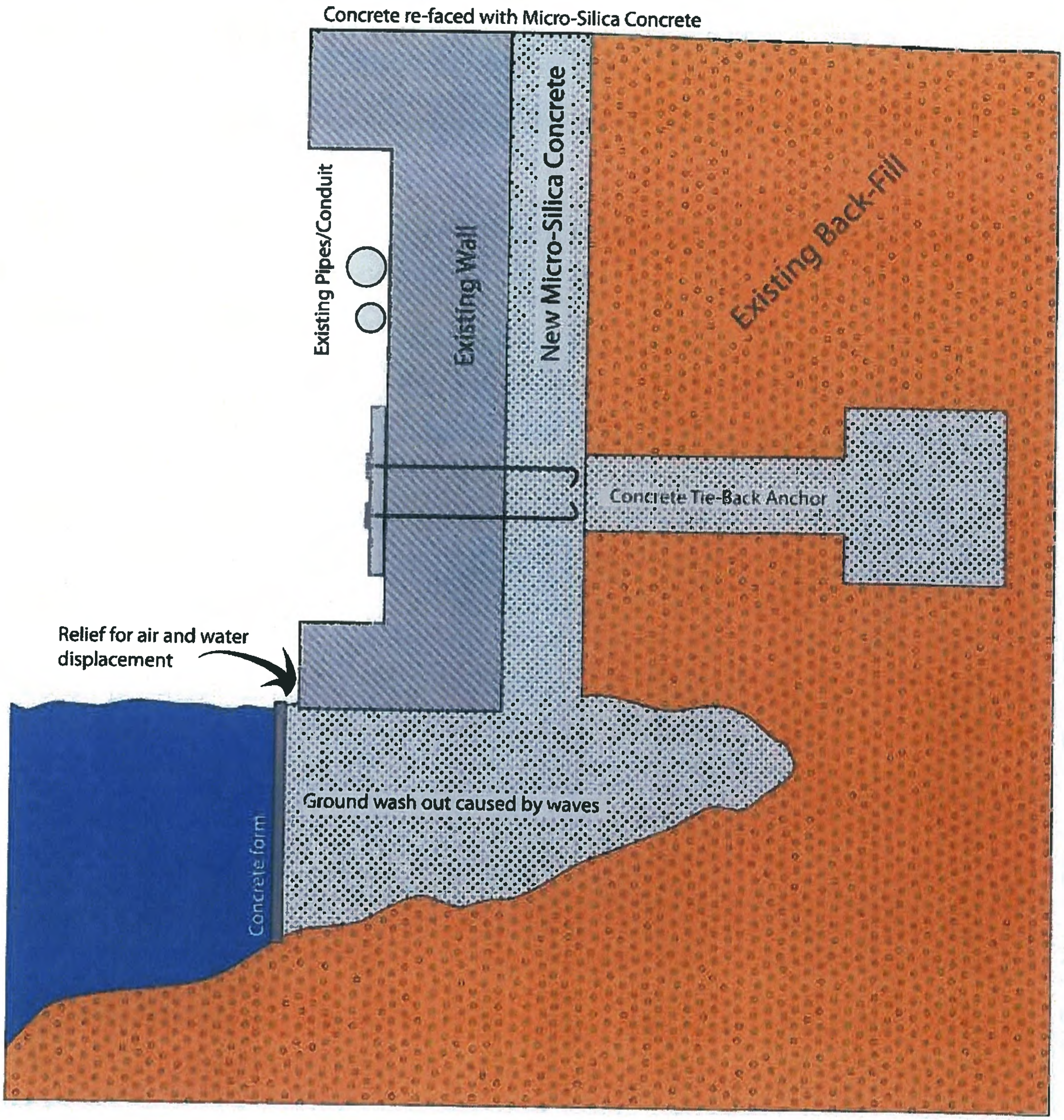
Nancy Foster Center

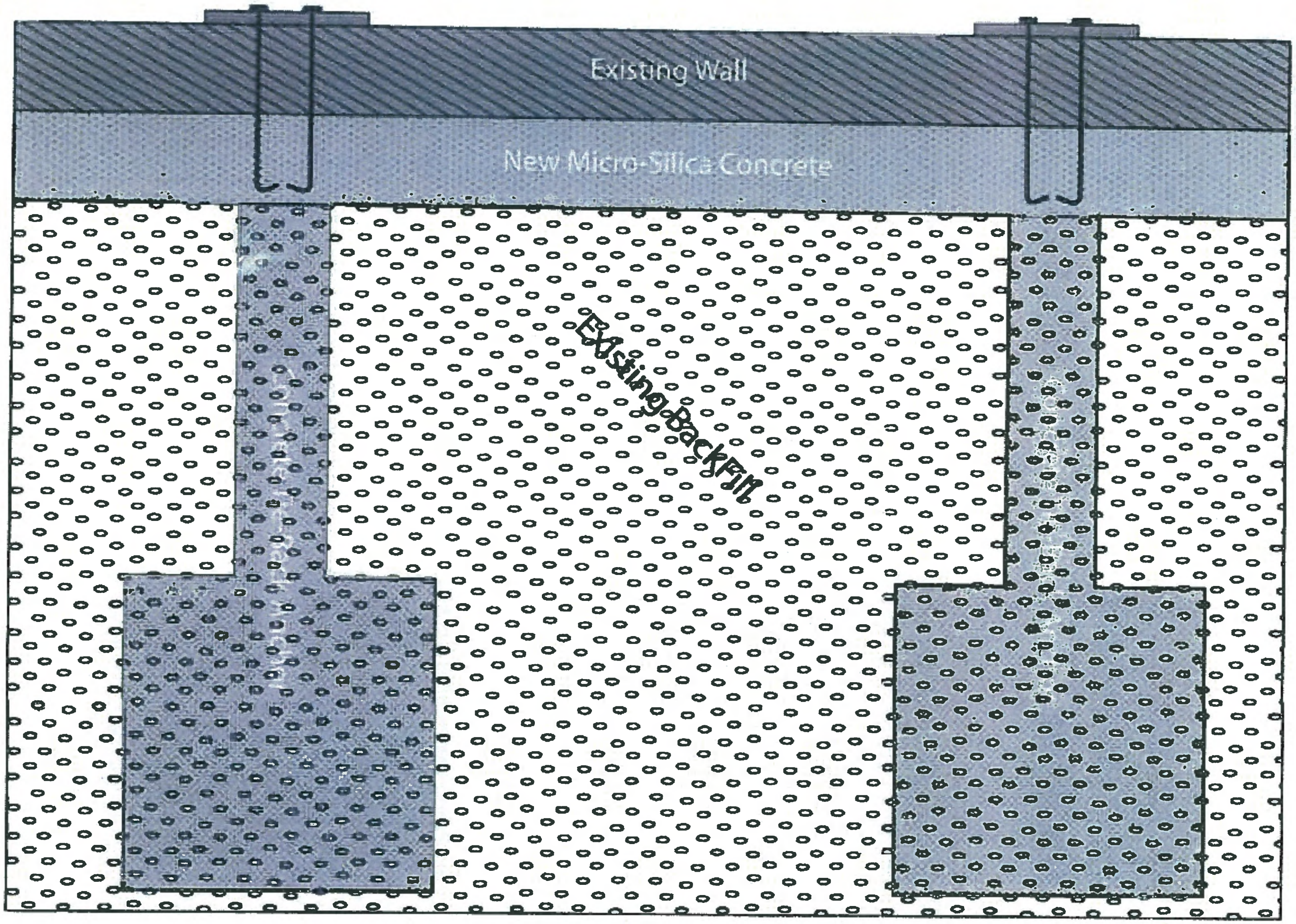
Florida Keys National Marine Sanctuary

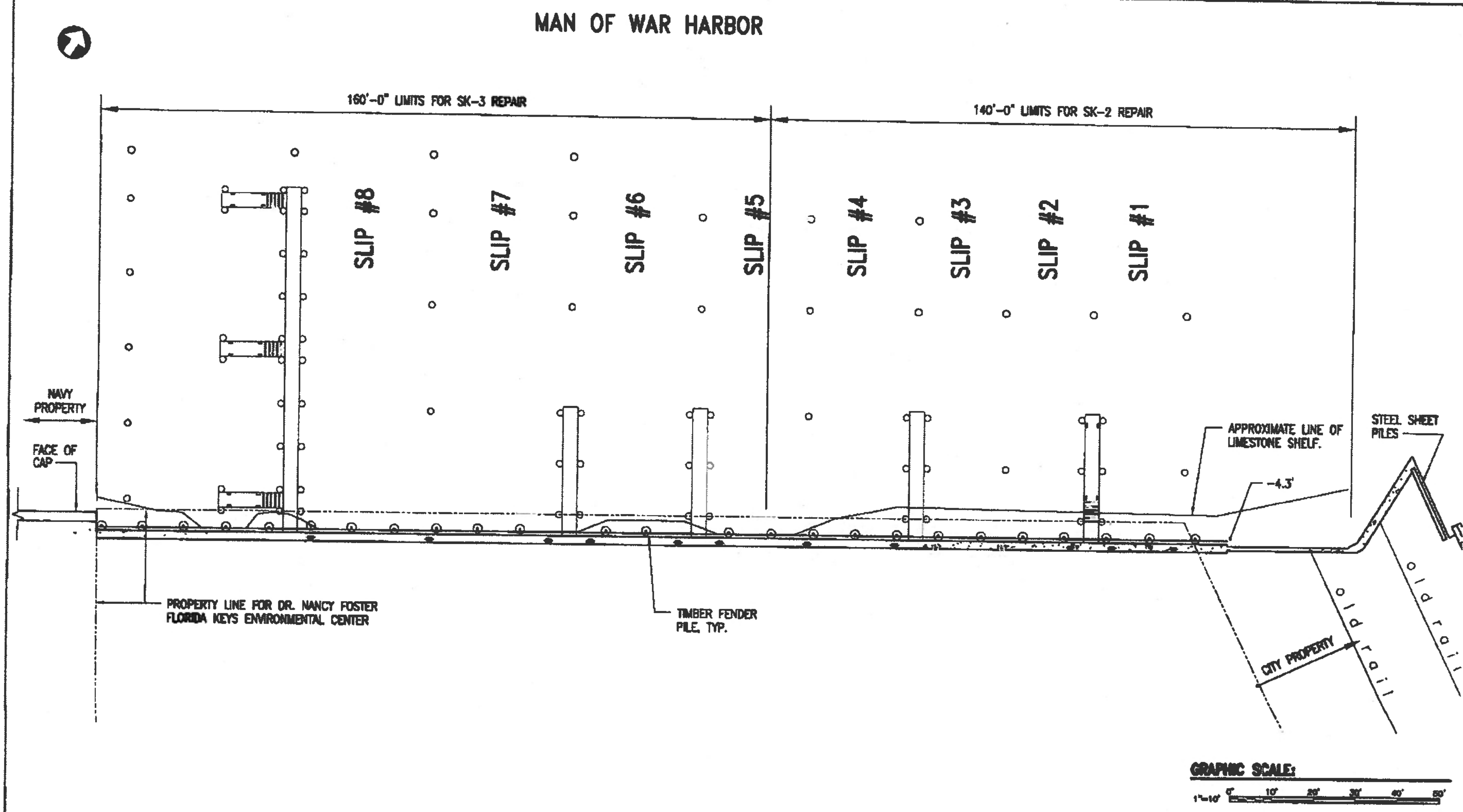
Conceptual Break Wall Repair

Jeff Konczak
Alpena Marc LLC

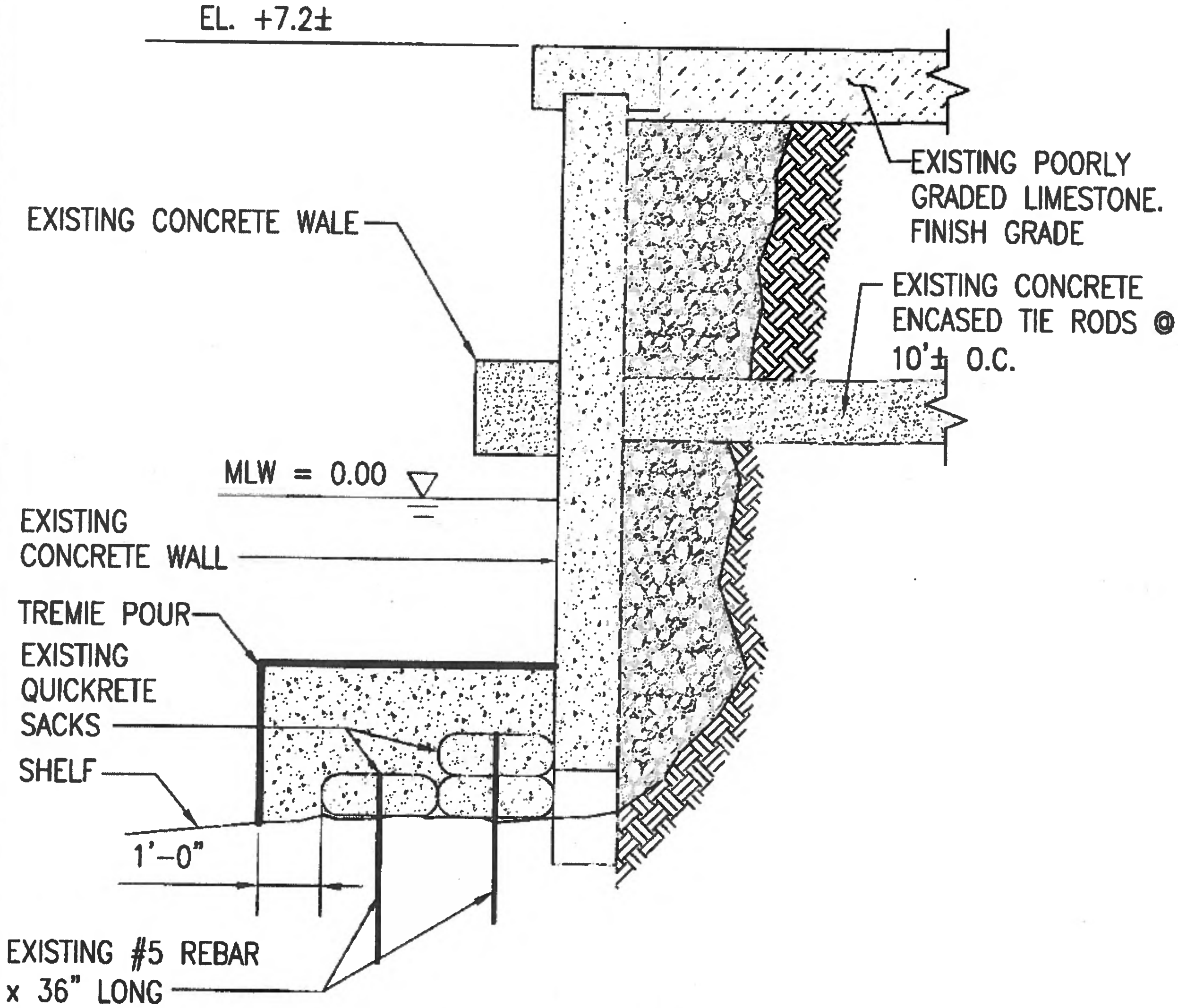
Micro-Silica Concrete
with BASF Anti-Washout Admixture
will fill already washed out areas even while water is still present




Existing Wall
New Micro-Silica Concrete
Existing BackFill



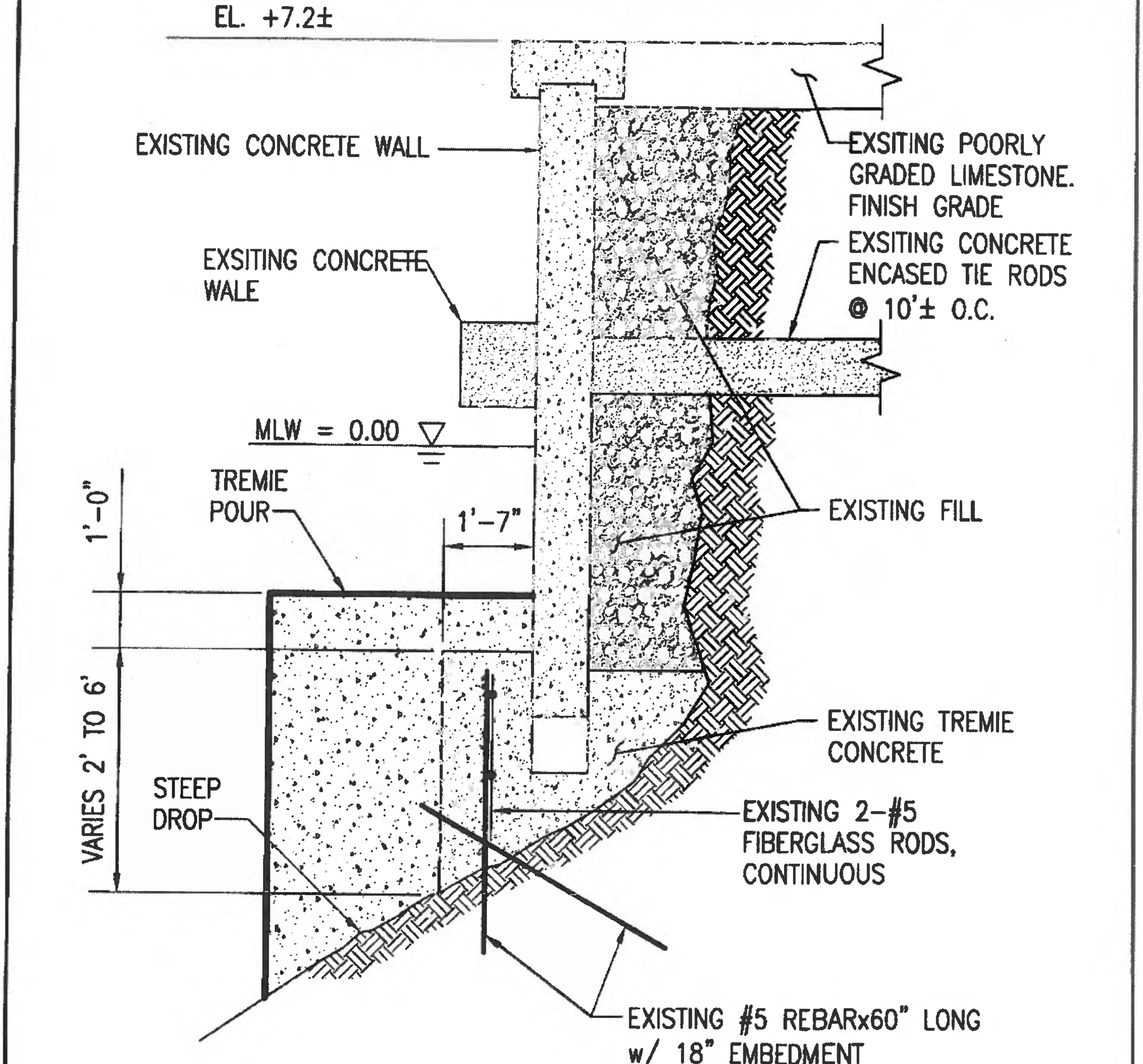
TranSystems 101 West Main Street ~ Suite 900 Norfolk ~ Virginia ~ USA 23510-1638 Phone 757-627-1112 Fax 757-627-1113 www.transystems.com	EXISTING SITE PLAN PROJECT NO: P101120119 DATE: 4-6-12	SKETCH NO: SK-1



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: P10112011 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