city of longbeach CA

TranSystems

NAPLES SEAWALL STABILITY INVESTIGATION AND REPAIR RECOMMENDATIONS

LONG BEACH, CALIFORNIA

(VOLUME 1 of 2)

Prepared for:
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TABLE OF CONTENTS

VOLUME 1 OF 2 – INVESTIGATION & REPAIR RECOMMENDATIONS

LIST OF FIGURES

LIST OF FIGURES

LIST OF FIGURES

LIST OF FIGURES

LIST OF PHOTOGRAPHS

LIST OF PHOTOGRAPHS

LIST OF PHOTOGRAPHS

LIST OF PHOTOGRAPHS

LIST OF PHOTOGRAPHS

1

2

3

4

5

6

7

8

9

10

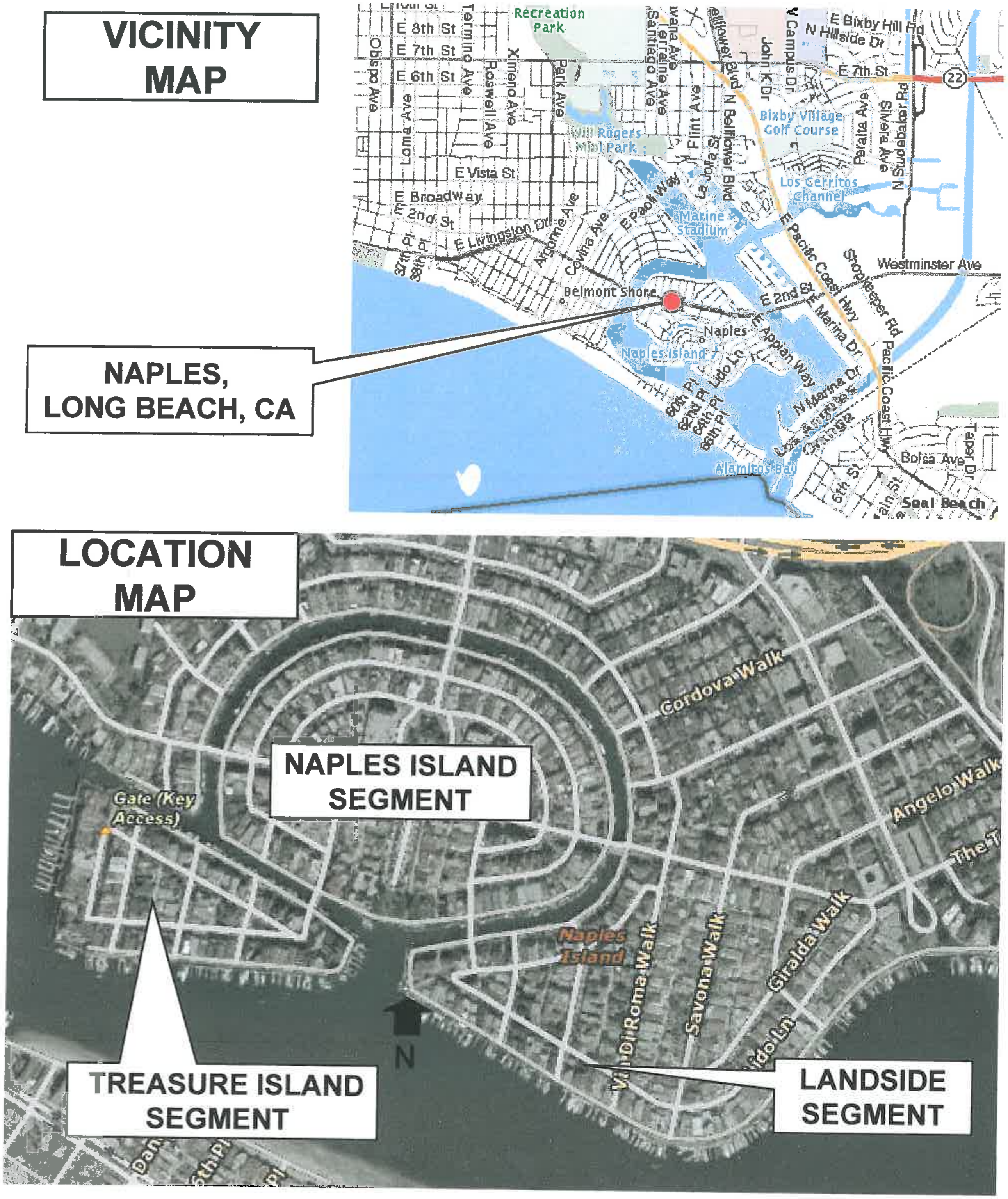

FIGURE 1 – 1 LOCATION AND VICINITY MAPS

1.0 Introduction

1.1 Objective

The primary intent of this investigation is to identify viable options to stabilize the seawall in its present position and extend the life of the seawall which have not been considered in previous engineering investigations. Interim repairs are also made to close the gap between current conditions and the time when funding is available for Long Range repairs of the seawall. In late August through early November, 2008, TranSystems' staff investigated the distressed concrete sheet pile seawall, analyzed its' stability, developed repair concepts and made repair priorities for the Naples seawalls located within the community of Naples, Long Beach, California. Through a combination of background research, field investigation, structural and geotechnical analysis, Transystems engineers have assessed the various factors contributing to the seawall's horizontal, vertical and rotational displacements from its original constructed positions. Areas of subsidence behind the seawalls, due to the loss of backfill soil and/or sheet displacement, are affecting sidewalks behind the seawall. The City is also considering continuing a program to repair/replace deteriorated tieback rods. These issues are also addressed as part of this investigation.

Background research included review of available as-built construction documents, past condition reports, and historical field data. Field investigation was done by rapid visual and representative measurements. It was not the intent or within the scope of services to document all seawall deterioration such that contract documents can be created directly from the data. The collection of field data is limited to that necessary to evaluate the existing conditions and failure mechanisms with a reasonable level of certainty. The project goals were to identify the most viable means of repair suitable for the existing site conditions and allow repair volumes to be quantified, to produce a budgetary level cost estimate.

1.2 Scope of Services

A) Project Start-Up: Included a Kickoff Meeting, Preliminary Site Walk & Project Development Plan.

B) Review Existing Documentation and Reports: Review all available construction documents, past condition reports and historical field data.

B.1 Document Review

Engineering Review/Investigations to Identify and Quantify the Cause(s) of Seawall Deterioration on Naples Island of Long Beach, California, by Concept Marine Associates, August 2004

Basis of Design, City of Long Beach Bulkhead Survey, by Cash & Associates, January 18, 1993

Condition Evaluation and Repair and Maintenance Plan for Seawalls and Bulkheads, by Moffatt & Nichol, July 1990

Guard Rail Survey Records, by Naples HOA, for 2000 to 2007

Engineers' Notebook Article "Pre-Cast Concrete Piles of Canal Walls at Long Beach, CA, by John Blackman, November 1940

History of Naples; Internet Website, http:\www.beachcalifornia.com

Long Beach Record Drawings:

B-725 Wall Replacement (Not Constructed), B-761 Wall Replacement (Current Wall), B-2675 Repair & Replacement of Tie Rods, B-2544 Tie Rod Repair & Replace, B-3375 Waterproofing of Bulkheads, B-4331 Naples Island Soil Anchors, B4371 Soil Anchors Phase V, B-4414 Naples Island Soil Anchors Phase VI

City of Long Beach, Refurbish Alamitos Bay Marina, Marina Rehabilitation Project, TranSystems Corporation, 2008 Design Build

Request for Proposal Drawings – Hydrographic Survey Data & Dredging Information.

Personal Interviews

Naples Home Owners, On-site an estimated 20 long time residents provided construction and wall performance recounts dating back to the late 1940's

Reyes Construction, Clint Larison – descriptions of past efforts installing prestress tiebacks and helical anchors

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C) Underwater Condition Survey: TranSystems' crew of licensed Engineer/Divers performed a "Level I Swim-By" inspection of the entire seawall below the mean tide line. The sheet pile joint gaps were evaluated for soil leaking; and active leakage was noted by measurement of the soil pile heights at the base of the sheets. Major structural damage such as broken, missing or large concrete spalls of the concrete sheets were also noted during this survey. Areas of undermined sheets or loss of rock protection along the main channels were noted when present, (Refer to Figure 6-2, Seawall Mudline Profiles). Underwater photographs were taken of damage and typical conditions when water clarity permitted, (Refer to Section 4.0, Waterside Survey).

D) Above Water Condition Survey: TranSystems' engineers performed a visual survey of the face of the concrete sheet piling, pile cap and tieback connections, where exposed. This inspection was done on the water side of the wall from the mean tide elevation to the top of the cap. Major structural damage such as open joints, broken tongue and grove sheet joints, broken sheets, wide cracks and large concrete spalls were recorded. Sheet rotation was measured with a "Smart Level" rail survey and compared to previous recordings provided by the Naples Home Owners Association. Due to the constricted work space between the dock systems, boats and the seawall, this work was performed from the deck of the floating dock systems and by swimming between the dock systems, (Refer to Section 2.0, Top Deck Survey).

E) Top Deck & Void Survey: TranSystems' staff performed a non-destructive visual survey of the entire seawall cap, sidewalks and existing grade adjacent to the seawall cap, (Refer to Section 2.1, Flatwork & Subsidence). Major structural damage such as open joints, breaks in the concrete pile cap, wide corrosion cracks and large concrete spalls, (Refer to Section 2.2, Seawall Cap) were recorded. Areas of suspect or obvious soil loss and sink holes behind the seawall were probed with a 4' compaction testing hand probe to determine the nature and extent of voids and soft soil conditions, (Refer to Suspect Sinkhole List, page 2-20).

F) Topographic and Hydrographic Survey

F.1 Topographic Survey – TranSystems' staff recorded evidence of subsidence, uplift and horizontal displacements using a hand tape, mason's string, plumb bob and "Smart Level" on the various improvements and grade behind the seawall. This data was used to evaluate the magnitude of subsidence, uplift, and/or horizontal displacement of the cap, sidewalk or grade, (Refer to Section 3.0, Rail Survey & Section 3.4, Seawall Cap Construction Joints).

F.2 Hydrographic Survey – TranSystems' Engineer Divers used a surveyor's rod to measured water depths at standard intervals of distance off the seawall face. This information was used to produce hydrographic mudline profiles of the seafloor at 16 locations representing both typical and common problem areas, such as near channel entrances, (Refer to Section 6.0, Hydrographic Survey Profiles). Recordings were taken at uniform distances from the wall face such that Profiles could be created of the mudline perpendicular to the wall. At each station, the water depth was measured at the wall face, at 5' off the wall, 10' off the wall, 15' off the wall and 20' off the wall. These Profiles were used to model typical conditions, (Refer to Figure 6-1 on page 6-1), estimate the original dredge depths and evaluate areas of erosion concern in the wall stability calculations, (Refer to Section 7.0. pages 7-7 & 7-7).

G) Meetings and Presentations: TranSystems' Project Manager and Principal–in-Charge were available to attend meetings with the City of Long Beach Staff to discuss our recommendations and meetings with Community Representatives, when requested.

H) Geotechnical Investigation

H.1 Subsurface Evaluations – Considerable geotechnical information exists in the form of a Geotechnical Investigation performed by Kleinfelder (Project No. 166-02), titled "Naples Island Seawall Deterioration", in June of 2002 in which (9) Boring Logs, Groundwater Monitoring, Corrosion Potential, Particle Size, Plasticity & Direct Shear Testing were performed. These (9) existing borings by Kleinfelder were supplemented with (6) new borings by AESCO Technologies, a sub consultant to Transystems. The (6) new Landside Borings were taken at representative locations distributed throughout the site to confirm the quality of the Kleinfelder work and to gain the necessary confidence to utilize the total accumulated geotechnical information as the basis for the current design concept effort. The dominant issue requiring quantification on the waterside is to determine the amount of material which has been removed by tidal, wind and wave action eroding the sheet pile embedment depth, (Refer to Figures 6-1 & 6-2). We have made the assumption that soil conditions below the original dredge depth will be generally consistent on both the land and water side of the seawall for this coastal bay sedimentary soil site.

H.2 Evaluate Wall Stability – Transystems and AESCO have worked together to evaluate the static and seismic stability of the existing seawalls for future expected movements, (Refer to Section 7.0 Seawall Stability Analysis).

H.3 Wall Movement – This report include comments by AESCO regarding whether adverse subsurface conditions have contributed to wall movements, (Refer to Geotech Report, Appendix D, Tab 4, bound separately)

H.4 Review Concept Repairs – AESCO reviewed the Concept Repairs provided by TranSystems and provided design passive and active soil pressures anticipated during both static and seismic conditions for use by Transystems to perform wall stability calculations, (Refer to Section 8.0, Repair Recommendations)

H.5 Liquefaction Potential – It is known that the site experienced damage from the 1933 Long Beach earthquake. This report includes comments by AESCO on the sites' liquefaction potential and potential for damage to the current seawall construction, (Refer to Appendix D, Tab 4, page 4-1)

I) Written Report: includes the following information:

I.1 Condition Descriptions – with photographs of typical conditions. (Refer to Section 2.0 Top Deck Survey & Section 4.0 Waterside Survey)

I.2 Structural Stability Calculations – of seawall conditions, with and without surcharge loads of adjacent homeowner improvements such as concrete decks and raised planters, (Refer to Section 7.0 Seawall Stability Analysis).

I.3 Discussion – of previous repair programs and recommendations, (See Section 4.1, page 1-8).

I.4 Recommendations for Repair – with Concept Level Details, (See Section 8.0, Figures 8-2 thru 8-10).

I.5 Budgetary Cost Estimate – for each Concept Repair, (Refer to Section 9.0)

I.6 Pro's and Con's – of each Concept, including, schedule, environment impact, permitting regulatory agencies requirements, aesthetics, impact on public and private property, impact on marina and sidewalk use during construction, (Refer to Section 8.0, pages 8-10 thru 8-16).

I.7 Field Data – Refer to Appendix Sections 1, 2 & 3 of Field Notes and Geotech Report, bound separately.

I.8 Geotechnical Report – Refer to Appendix D, Tab 4 of Field Notes & Geotech Report, bound separately.

1.3 Description of Site

The approximate 11,000 linear feet of seawall line the manmade canals and Alamitos Bay in the Naples residential development. The seawalls create a means for adjacent residential property owners to have direct access to their own private boat dock, which the great majority of Owners have constructed.

The system of seawalls is divided into three segments for the purposes of this report. See Figure 1-1 for a Location Map of the three segments described below:

NAPLES ISLAND – is an oval shaped island located in the center of the Naples community. It is accessed by 5 concrete bridges. Approximately 82 properties and Colonnade Park front the water on this island.

LANDSIDE – is the mainland waterfront. In general terms this segment runs from the west channel of Alamitos Bay to the East main Channel of Alamitos Bay interrupted by the large crescent shape of Naples Island. Approximately 133 properties front the water on the Landside Segment

TREASURE ISLAND – is a narrow triangular shaped island located at the west end of the Naples development. The north and east faces form part of the Naples Canal system and the southwest face fronts the west channel of Alamitos Bay. A single bridge provides access to the island from Naples Island. Approximately 39 properties front the water on Treasure Island.

Photo 1 – Naples. Canal between Naples Island and Landside Segments. Taken from Ravenna Bridge looking southeast

1.4 Description of Seawall Construction

The seawalls are constructed of precast concrete sheet piles 2'-4" wide x 10" thick with lengths of 22', 24' and 26', (see Figures 1-12, 1-13 & 7-1). The wall is topped with a 1'-8" tall x 1'-8" wide cast-in-place reinforced concrete cap. An 18" extension was added to the cap in the late 60's and an anodized aluminum guard rail provides pedestrian safety atop the cap. Steel tie-back rods 1-3/8" diameter at 10' on-center brace the top of the wall from outward rotation due to soil pressures at the 22' & 24' sheets and 1 ½" diameter rods at the 26' sheets. The rods are buried in the soil behind the wall and extend 18 feet back to a 15" square precast concrete soldier pile, which were also buried beneath the earth. Seawalls of this nature are typically constructed by first driving the sheet piles, installing the soldier piles and tieback rods then dredging the waterside and using the dredge spoils as fill soil behind the wall to create a level elevated grade. The top of the current wall is approximately +9.0' MLLW. Mudline depths vary but original dredge depths are estimated as -5.0' MLLW inside the canals, -6.5' at the canal entrance channel and along the Main Channel of Alamitos Bay, and -8.0' along the West Channel of Alamitos Bay.

1.5 Construction History

Transystems reviewed previous reports by Moffat Nichol in 1990, Cash & Associates in 1993 and Concept Marine in 2004. Each of these reports had a collection of historical details of the wall construction and some of the work which has been done over the life of the structure. No "as-built" drawings were made available for this investigation but the information in the previous reports provided a substantial amount of background information. Transystems also researched historical web based data, interviewed long time resident's and also meet with Contractor's who have performed work on the facility.

The Naples Canals were first developed in 1905-1906. Originally constructed of timber lagging with concrete facing these walls were prone to marine borer and dryrot deterioration. The Long Beach Earthquake in 1933 caused widespread and severe failures of the timber wall systems and slope failures with the sand fill collapsing back into the canals. This devastating earthquake left the shoreline unsafe and inaccessible for boaters and residents. The City of Long Beach, Naples Community and Work Projects Administration (WPA) collaborated to raise the funding needed to rebuild the canals in 1938-1939 using the precast concrete sheet pile system which exists to this day.

The developing oil industry pumped so much oil out of the Seal Beach and Willmington Oil fields, in and near Long Beach, that the Naples Community subsided and estimated 17" between 1928 and 1970. Seawater would overtop the seawall cap and flood the adjacent streets and walks at extreme high tides.

In response to this subsidence an 18" extension was added to the top of the existing cap in 1967-68. During the mid 1990's to early 2000's, new tie rods were installed below the existing cap. We understand that both helical anchors and grouted anchors have been utilized in conjunction with small steel wales bolted to the waterside face of the existing sheet piling. Starting in the mid 1950's to present, various soil grouting and sheet pile joint grouting efforts have taken place to respond to localized soil loss through the sheet pile joints and various gaps in the seawall construction.

The City is attempting to control the loading on the seawall by publishing Seawall Guidelines on subjects which may have an adverse effect on the structure. Examples include heavy raised planters and hardscape decks adjacent to the seawall cap. The planting of large trees near the wall or mounting guide piles and gangways on the cap are not permitted. The Seawall Guidelines follow and should be a considered as part of the basis for loading assumptions made for the design of future repairs and mitigation or analysis of current conditions.

Seawall Guidelines

The Naples Seawall Committee has been working with the City of Long Beach for almost eight years to see that the seawall is maintained and repaired. The following list of guidelines will help to preserve our walls for future generations.

1. The sidewalk, parkway, and seawalls are city property and access needs to be maintained. Any improvements in the parkway may at some time in the future be damaged or require removal in the course of making seawall repairs. If the city determines that some structure in front of the walls is causing damage, they will require the homeowners to modify and remove it.

2. The seawalls are supported by steel tiebacks at approximately 10' centers. To avoid any damage to these tiebacks, any excavation in the parkway area should be done with great care. At the back face of the seawall the tiebacks are very close to grade. Care should be taken to not disturb the tiebacks or their coatings.

3. Any structure built above the current grade adds load onto the tiebacks. Since the condition of the tiebacks is unknown, it is desirable to keep structures, whether a deck, flower bed, or other improvement, low and of lightweight materials.

4. The City of Long Beach seals leaks in the seawalls by pumping grout behind the walls. To enable them to easily perform this task, an unpaved area of approximately one foot wide should be left immediately behind the wall.

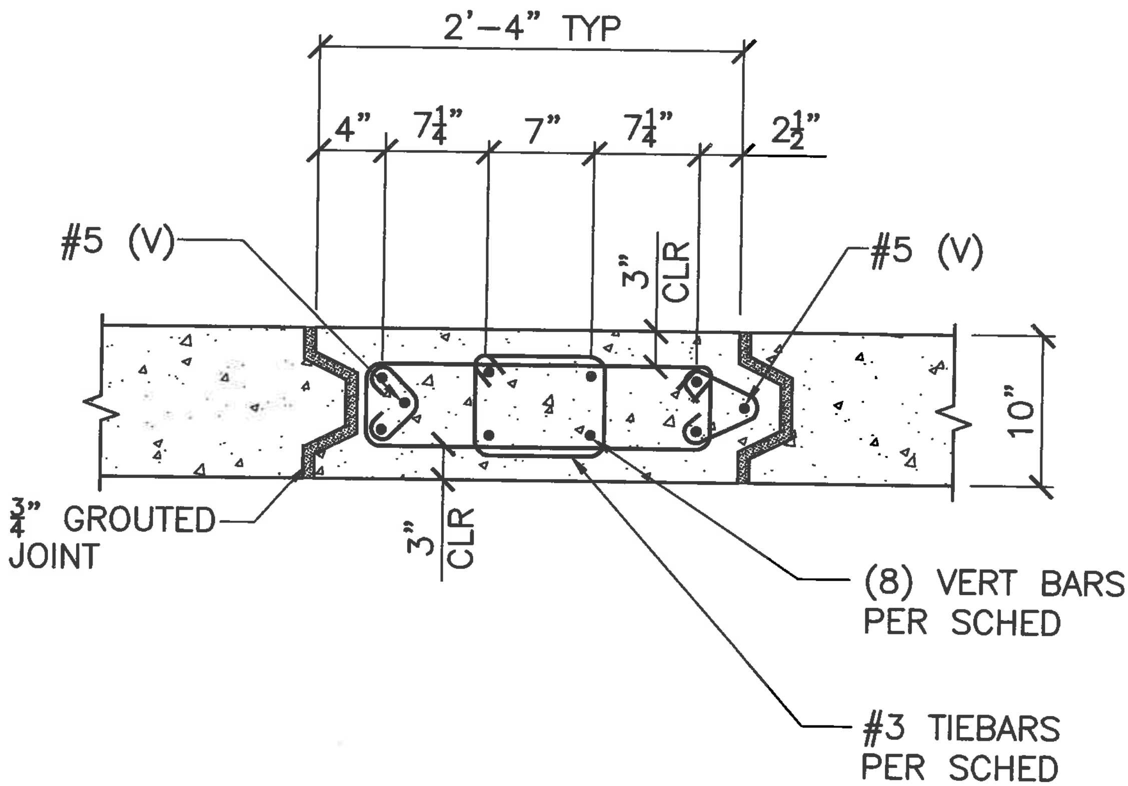
5. Adequate drainage is important to avoid flooding. This is best accomplished by leaving an adequate amount of unpaved area at sidewalk grade.

6. Drilling into the wall or fastening things to the wall will damage it and is prohibited by the City of Long Beach.

7. Property owners constructing docks must adhere to the Marine Department specifications for the placement of electrical and water lines, platforms, piles, davits, and ladders. In addition, owners of such structures should be observant and repair any structure that is in any way damaging or endangering the seawall.

8. If you aren't sure if something will jeopardize the wall or if you see some construction that appears in that area, please contact Robert Maldonato, Public Works Department, Bureau of Engineering at 570-6256.

REINFORCING SCHEDULE		
22' SHEETS	8–#6 VERT.	#3 TIES @ 15"
24' SHEETS	8–#7 VERT.	#3 TIES @ 15"
26' SHEETS	8–#8 VERT.	#3 TIES @ 15"



TranSystems

PROJ NO:
SCALE: 1" = 1'-0"
DATE: 2-5-09
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE

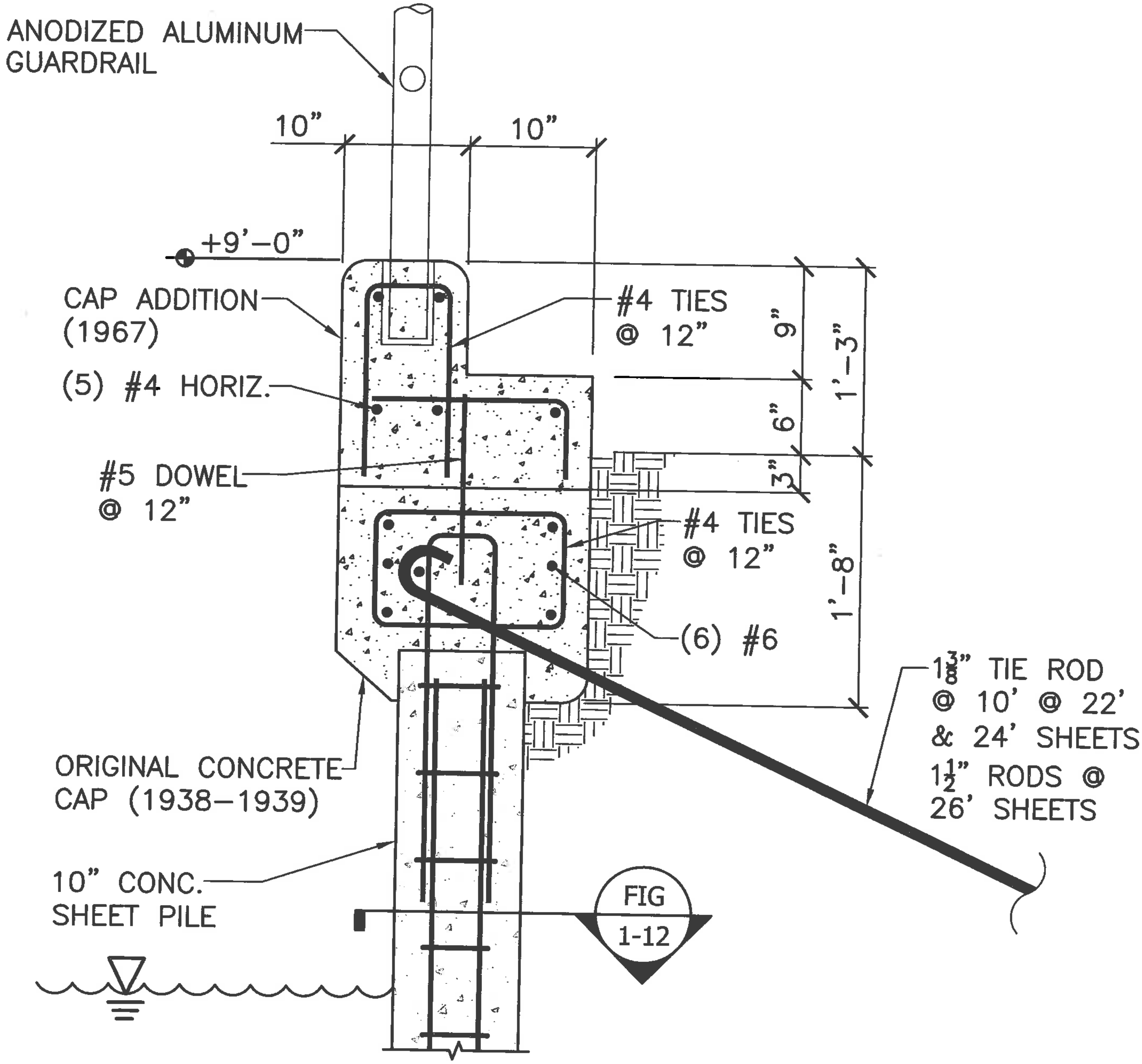
TYPICAL SHEET PILE SECTION

PROJECT

NAPLES SEAWALL INVESTIGATION

FEBRUARY 2009

FIG. 1–12



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SHEET TITLE
TYPICAL SHEET PILE SECTION

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NAPLES SEAWALL INVESTIGATION

FEBRUARY 2009

FIG. 1-13

2.0 Top Deck Survey

2.1 Flatwork & Subsidence

The entire waterfront of the Naples community is accessible by public walkways approximately 5' inboard of the seawall. The condition of these sidewalks was visually surveyed for horizontal and vertical movement which can be an indicator of soil loss or seawall movement. The results are generalized for each property and walks with vertical displacements in excess of ½" are displayed in Red on Figure 2-1 (Naples Island), Figures 2-4a, b, c, d, e & f for (Landside) and Figure 2-7 for (Treasure Island). ½" is a common threshold of acceptance by professionals checking for ADA compliance and trip hazards. Due to the advanced age of the community we found no consistency in the condition of the walkways. Many walks are very new and as-such yield little information on ground movement. Other walks appear to be 40 or more years old and are riddled with cracks, (see Photo's 2-23 thru 2-26), offsets and separations in response to settlement from soil loss or heave in areas where mud jacking or tieback grouting has taken place. The sidewalk survey information provides the ability to estimate the cost to repair walkways, if it is a desire to restore all walks after global or localized seawall repairs mitigate the soil loss.

A 4' soil compaction probe was used to test for sinkholes and soft soil conditions, both are strong indicators of soil loss and tidal saturation/drawdown "pumping action" which can remove a small amount of soil fines with each tidal drawdown. All suspect openings in paving and planting areas were probed, (see Photo's 2-1 thru 2-4) for sinkholes, loose soil and suspect voids. The findings of this survey are displayed in Purple on Figure 2-1 (Naples Island), Figures 2-4a, b, c, d, e & f for (Landside) and Figure 2-7 for (Treasure Island). These findings are also tabled in a list on Page 2-20 to assist in prioritizing repairs and cross checking with evidence collected from the Waterside Survey, Section 4.0.

2.2 Seawall Cap

The concrete sheet piles are capped with an approximate 1'-8" wide x 3' tall concrete cap constructed in two increments, (see Figure 1-13 on page 1-13). The seawall cap holds the top of the concrete sheet piles in alignment and transfers the top reaction of the soil pressure to steel tie-back rods occurring at 10' on center in the original wall construction, and approximately half that spacing in areas which have been retrofitted, (see Photo 4-8 on page 4-6). The original lower cap is in fair condition on the land side of the wall and in very poor condition on the waterside. The newer, upper increment of the cap, is in a severely deteriorated condition with very few areas not exhibiting signs of advanced corrosion deterioration such as cracks, corner spalls, face spalls, rail spalls, insufficient localized concrete cover "pop spalls" and corrosion bleeding, (see Photo's 2-6 thru 2-18). These findings are displayed graphically for the entire community, (see Figure 2-10 on page 2-24). This figure shows the over 95% of the seawall cap is in advanced deterioration. Statistically, an estimated 10,730 linear feet of cracks and spalls occur on the 3 visible surfaces of the 11,000 foot seawall cap, (see Figure's 2-3, 2-6, 2-9, 4-2, 4-4, 4-6 Estimates of Spalling Quantities). It is not material cost effective or performance wise to consider piece-meal repairs for the caps. Partial repairs will be challenging for contractors to find sound concrete to stop demolition work and adjacent corroding reinforcement will form an electrical bond with the new reinforcing creating a circuit which accelerates the corrosion, typically at the interface between the new and original construction. We recommend that the cap be replaced at the same time the seawall is replaced or that interim tieback repairs be considered, when practical, to transfer the load directly from the sheet piles to new tie back rods, (see Figure 8-2 & 8-3) . The cap deterioration is not currently considered the "weak link" in the existing seawall system and will not be the driving force instigating repairs. We estimate it will be more common for the tie-rods to fail first from corrosion deterioration, (see Photo 2-5 on page 2-5) or areas of concrete sheet pile deterioration will yield in shear or bending and those conditions will drive repair funding.

2.3 Raised Decks and Planters

Figure 2-1 (Naples Island), Figures 2-4a, b, c, d, e & f for (Landside) and Figure 2-7 for (Treasure Island) indicate the approximate extent which Raised Planters and solid Raised Decks occur throughout the Naples community. Raised decks & planters which match or exceed the height of the seawall cap create a "surcharge load" on the seawall which is estimated to increase the forces on the seawall system 11% to 14%, (see Section 7.3 of the Seawall Stability Analysis). Large trees can further increase this "surcharge load" in direct proportion to their weight. For this reason the City has created a set of Seawall Guidelines for waterside residents, (see page 1-11).

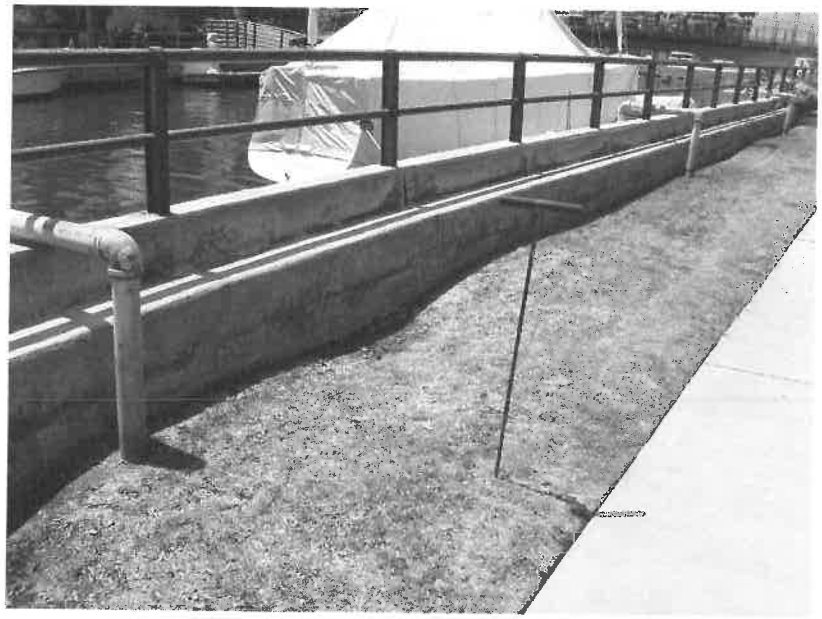


Photo 2 – 1, Naples, Top Deck Survey: **Naples Island, #15 Colonnade Canal.** Typical grass sink hole. Probe used to test for soft soil.

Photo 2 – 2, Naples, Top Deck Survey: **Rivo Alto Canal, #82** Loose soil occurring in rock bed. Probe shows extent of soft soil.

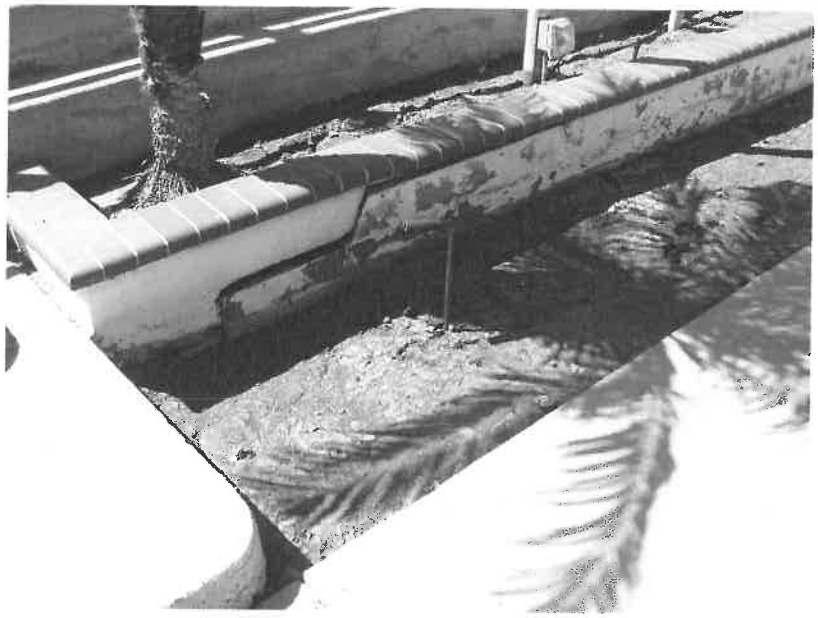


Photo 2 – 3, Naples, Top Deck Survey: Rivo Alto Canal, #24
Typical dirt planter with loose soil. Probe shows extent of soft soil.

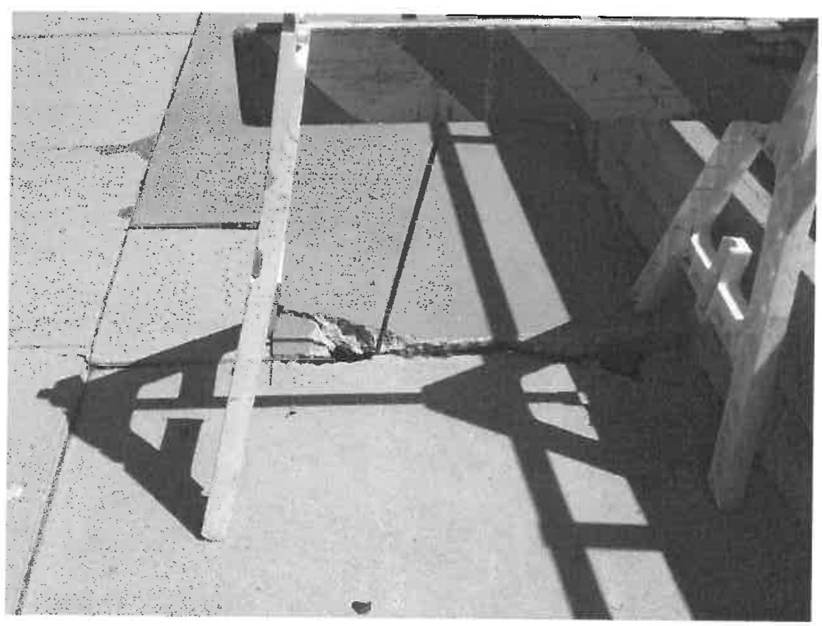


Photo 2 – 4, Naples, Top Deck Survey: Rivo Alto Canal, #133
Sinkhole under concrete sidewalk. Probe is in sinkhole.

Photo 2 – 5, Naples, Top Deck Survey: Vista Del Golfo, #15
Tie rod protruding above ground.

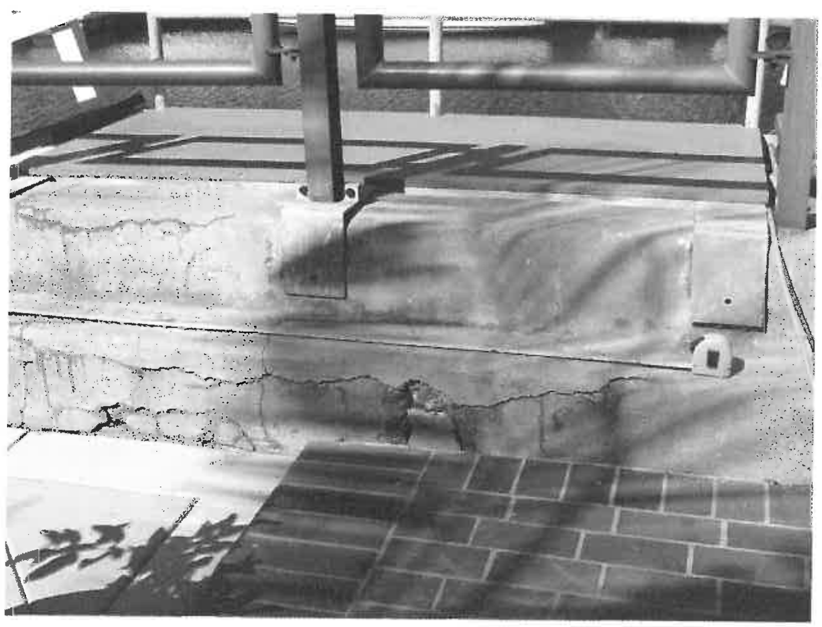


Photo 2 – 6, Naples, Top Deck Survey: Rivo Alto Canal, #118
Cap cracking on the inboard side of the lower cap.

Photo 2 – 7, Naples, Top Deck Survey: Rivo Alto Canal, #47
Cap cracking on the inboard side of the lower cap.

Photo 2 – 8, Naples, Top Deck Survey: Rivo Alto Canal, #185
Cap cracking on the inboard side of the lower cap. Rebar is exposed.

Photo 2 – 9, Naples, Top Deck Survey: Rivo Alto Canal, #20
Spalling on the inboard side of the new top cap. Rebar is rusted and exposed.

Photo 2 – 10, Naples, Top Deck Survey: Rivo Alto Canal, #179
Spalling on the inboard side of the new top cap & bottom cap. Rebar is rusted and exposed.

Photo 2 – 11, Naples, Top Deck Survey: <u>Rivo Alto Canal, #183</u>
Spalling on the inboard side of the new top cap & bottom cap at seawall joint. Rebar is rusted and exposed.

Photo 2 – 12, Naples, Top Deck Survey: <u>Corso Di Napoli, #5733</u>
Patch of a spall occurring in top cap.

Photo 2 – 13, Naples, Top Deck Survey: **Rivo Alto Canal, #221**
Rail spalling at outboard side of seawall. Spalling occurs on top cap.

Photo 2 – 14, Naples, Top Deck Survey: **Rivo Alto Canal, #144**
Rail spalling at inboard side of seawall. Spalling occurs on top cap.

Photo 2 – 15, Naples, Top Deck Survey: Corso Di Napoli, #5639
Rail cracking on top side of seawall. Cracking occurs on top cap.

Photo 2 – 16, Naples, Top Deck Survey: Rivo Alto Canal, #213
Grout reaction – exposed concrete and rebar.

Photo 2 – 17, Naples, Top Deck Survey: **Rivo Alto Canal, #213**
Typical cracking of seawall cap at gangway gate.

Photo 2 – 18, Naples, Top Deck Survey: **Rivo Alto Canal, #185**
Typical spalling of seawall cap at gangway gate. Rebar exposed.

Photo 2 – 19, Naples, Top Deck Survey: **Rivo Alto Canal, #95**
Typical seawall joint in good condition.

Photo 2 – 20, Naples, Top Deck Survey: **Naples Canal, #5598**
Typical seawall joint in fair condition.

Photo 2 – 21, Naples, Top Deck Survey: Rivo Alto Canal, #172
Horizontal joint offset at failed seawall section.

Photo 2 – 22, Naples, Top Deck Survey: Rivo Alto Canal, #131
Joint compression.

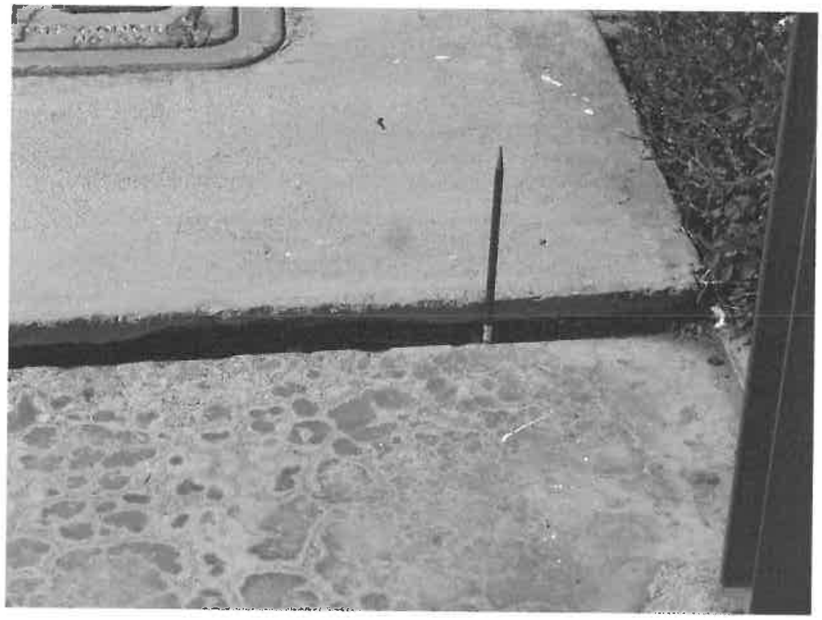


Photo 2 – 23, Naples, Top Deck Survey: Plaza Park
Vertical offset in sidewalk.

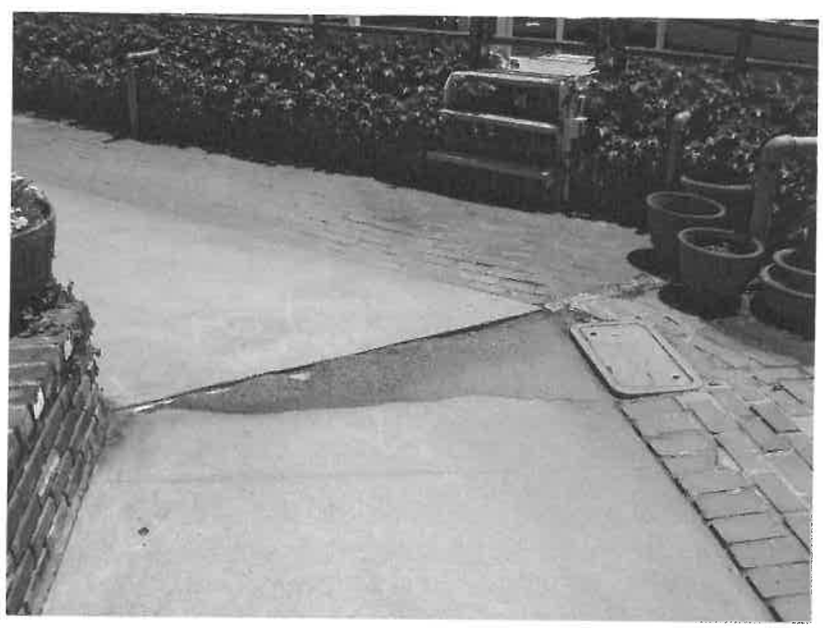


Photo 2 – 24, Naples, Top Deck Survey: Naples Canal, #5607
Sidewalk patching.

Photo 2 – 25, Naples, Top Deck Survey: **Corso Di Napoli, #5679**

Horizontal gaps in sidewalk. Note previous patching efforts.

Photo 21 – 26, Naples, Top Deck Survey: **Rivo Alto Canal, #81**

Sidewalk cracking.

Photo 2 – 27, Naples, Top Deck Survey: Rivo Alto Canal, #86
Example of raised wood deck & planter.

Photo 2 – 28, Naples, Top Deck Survey: Rivo Alto Canal, #115
Example of typical raised planter and decks with hard finishes.

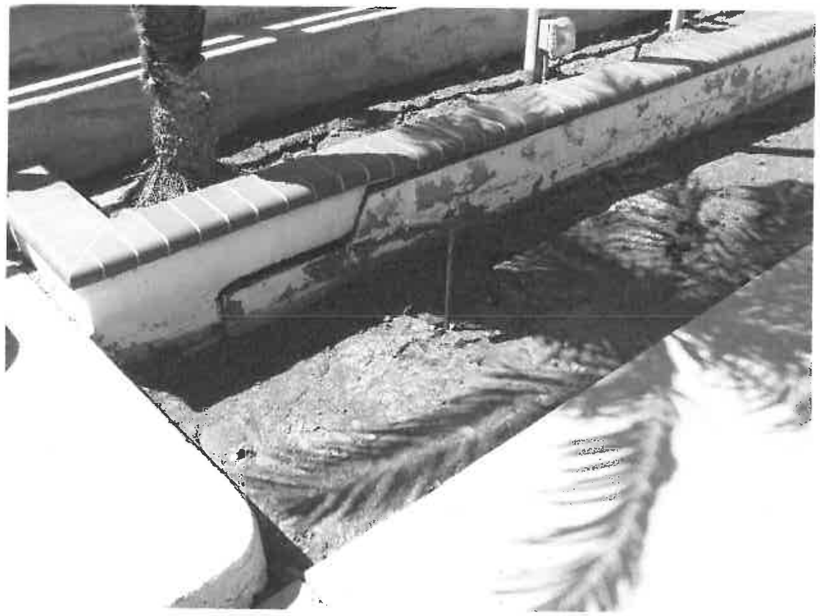


Photo 2 – 29, Naples, Top Deck Survey: Rivo Alto Canal, #24
Example of raised planter that has cracked due to subsidence. Note how deep the probe is sunk into the loose soil.

Photo 2 – 30, Naples, Top Deck Survey: Rivo Alto Canal, #171
Example of raised deck with stone finish.

Photo 2 – 31, Naples, Top Deck Survey: **Rivo Alto Canal, #183**
Example of typical top deck planting area & sidewalk.

Photo 2 – 32, Naples, Top Deck Survey: **Corso Di Napoli, #5655**
Example of concrete wall in planter, adjacent to seawall.

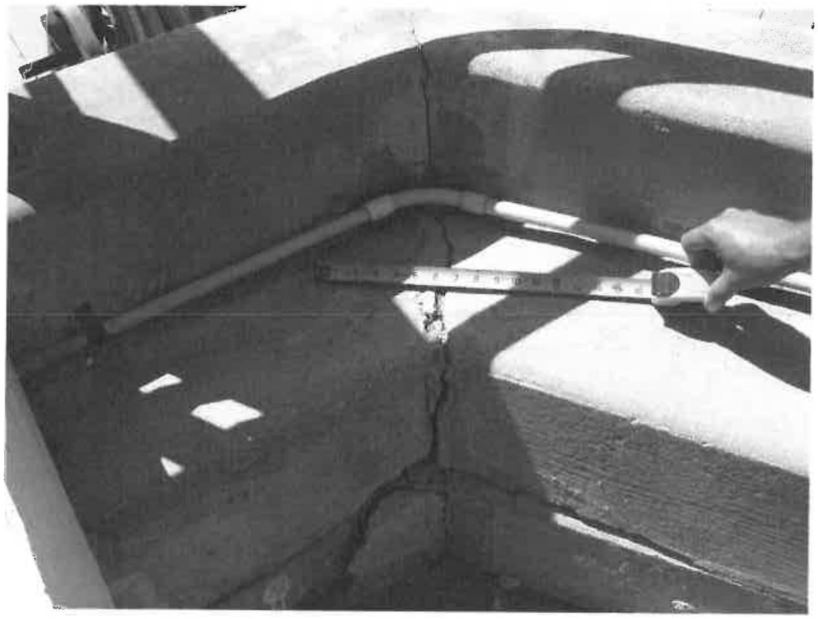


Photo 2 – 33, Naples, Top Deck Survey: **Corso Di Napoli, #5705**
Example of corner crack in the seawall cap.

Naples Seawall Investigation – Suspect Sinkhole locations

Note the word "sinkhole is only used where is there is a significant probability of existence or it is confirmed to exist. The word "sinkhole", "void" and "depression" are synonymous. All other notes, such as "soft soil", "loose soil", "very soft soil" or "extremely soft soil", are indications of conditions which have varying degrees of potential for voids and sinkholes to occur at a future date. Sinkhole and Subsidence Repair methods are discussed in Section 8.5.

Treasure Island

5561 E. Corso Di Napoli
Sinkhole under brick paving and depression in brickwork

5571 E. Corso Di Napoli
Sinkhole at end of Geneva Walk adjacent to property

1 Palmero Walk
3 depressions in grass planter

2 Palmero Walk
1 depression in grass planter near property line with 5607 E. Corso Di Napoli

5655 E. Corso Di Napoli
Possible void, "extremely soft soil adjacent to raised planter

5680 E. Corso Di Napoli
1 small sinkhole and "extremely soft soil" in grass planter adjacent to canal entrance.
"Very soft" soil at end of Alley on Canal side of property

Naples Island

15 Colonnade
3 sinkholes, 2" to 5" deep in grass planter

20 & 24 Rivo Alto Canal
2" deep sinkhole in earth adjacent to planter of #24 Rivo Alto Canal which is also adjacent to the raised concrete deck of #20 Rivo Alto Canal. Sidewalk settlement and site wall masonry damage are evidence of suspect sinkhole extending under concrete deck of #20

26 Rivo Alto Canal
1.5" deep Sinkhole adjacent to brick paving near property line with #24

50 Rivo Alto Canal
Est. 6" deep Sinkhole under wood deck adjacent to planter

Naples Island – continued

70 Rivo Alto Canal
Very soft soil beneath raised brick deck adjacent to seawall

82 Rivo Alto Canal
Possible void, "Very loose soil" under stones & Paving near seawall joint

84 Rivo Alto Canal
Suspect void under planter adjacent to brick paving. "Extremely loose soil"

98 Rivo Alto Canal
"Loose soil" below planter near seawall joint

144 Rivo Alto Canal
"Loose soil" in front of planter near property line with #140

156 Rivo Alto Canal
Very soft soil in grass planter adjacent to seawall

182 Rivo Alto Canal
Very soft soil in planter adjacent to seawall

194 Rivo Alto Canal
Suspect sinkhole under concrete paving near property line with #192

208 Rivo Alto Canal
Possible void, "very loose" soil in planter near property line with #206

210 Rivo Alto Canal
Possible void, "very loose" soil in planter between steps and seawall joint

Landside

55 Rivo Alto Canal
"Extremely soft" soil near property line with # 53, "very soft" soil in planter

63 Rivo Alto Canal
"Extremely soft" soil in planter along seawall

75 Rivo Alto Canal
Depression and "very soft" soil in grass planter

<u>95 Rivo Alto Canal</u>
Depressed paving and "very soft" soil in rock landscape, major drainage area

<u>97 Rivo Alto Canal</u>
Est. 6" deep sinkhole under brick paving. Bricks low and uneven, major drainage area

<u>107 Rivo Alto Canal</u>
Suspect sinkhole under wood deck. "Extremely soft" soil

<u>109 Rivo Alto Canal</u>
"Very soft" soil near property line with # 111, between brick and tile decks

<u>133 Rivo Alto Canal</u>
8" deep sinkhole x 12' x 5' beneath failed paving

<u>143 Rivo Alto Canal</u>
"Very soft" soil under brick paving near seawall joint

<u>149 Rivo Alto Canal</u>
Possible void, "extremely soft" soil under concrete paving near property line with #145

<u>157 Rivo Alto Canal</u>
Depression in brick paving and "very soft" soil near property line with #159

<u>183 Rivo Alto Canal</u>
"Very soft" soil at end of Sicilian Walk under brick paving

<u>185 Rivo Alto Canal</u>
Sink hole beneath brick paving, "very soft" soil at end of Sicilian Walk under brick paving

<u>189 Rivo Alto Canal</u>
"Very soft" soil under wood deck

<u>191 Rivo Alto Canal</u>
Depression in brick paving

<u>219 Rivo Alto Canal</u>
Suspect sinkhole, "extremely soft" soil under brick near alley

<u>2 Colonnade</u>
Depression and soft soil beneath paving near property line with 5705 CDN

Landside – continued

1 Via Di Roma
Depression in grass planter and "very soft" soil at end of street surrounding storm drain outfall

1 Savona Walk
"Loose soil" near property line with Savona Walk

15 Vista Del Golfo
8" deep sinkhole near seawall joint in rock planter

41 Vista Del Golfo
Depressed paving between seawall joint and property line with # 35

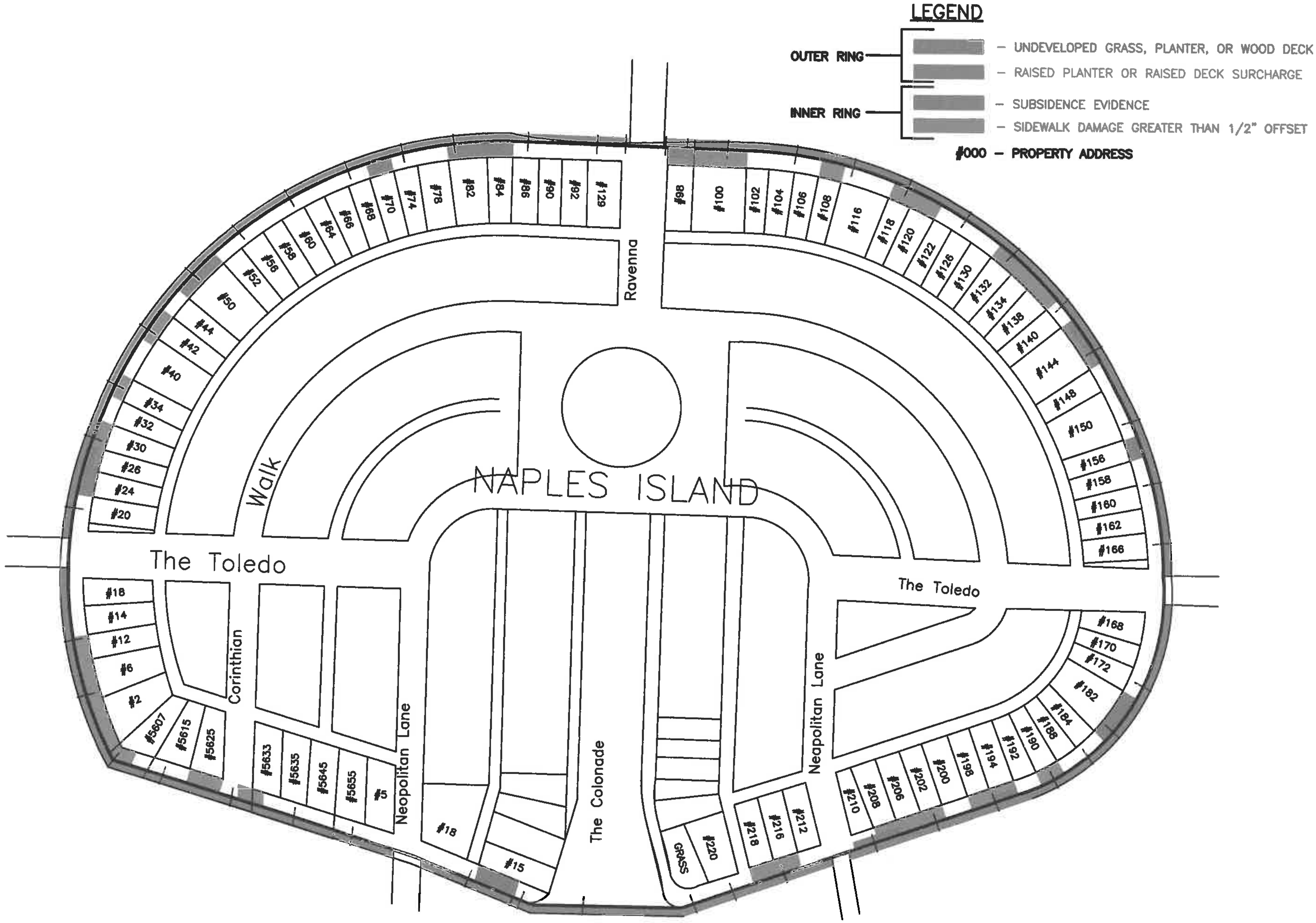


FIGURE 2–1 NAPLES ISLAND
SIDEWALK & PLANTER
KEY PLAN

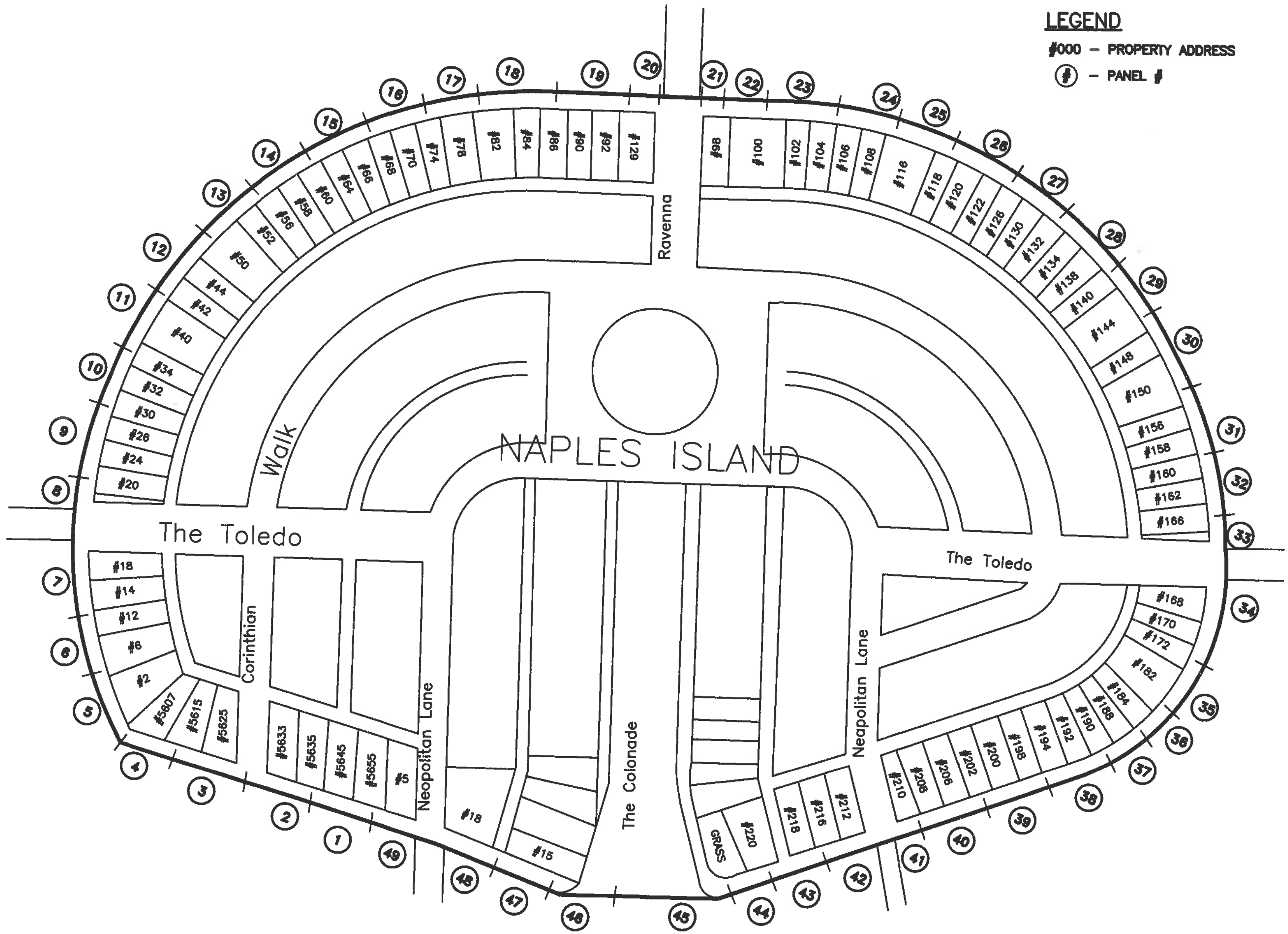


FIGURE 2–2 NAPLES ISLAND SEAWALL CAP & RAIL CONDITION KEY PLAN

Figure 2-3 Naples Island Seawall Cap and Rail Condition Totals		
Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 1	18'	17'
Panel 2	28'	6'
Panel 3	50'	20'
Panel 4	10'	6'
Panel 5	8'	6'
Panel 6	5'	2'
Panel 7	28'	19'
Panel 8	0	0
Panel 9	43'	19'
Panel 10	43'	6'
Panel 11	30'	0
Panel 12	38'	14'
Panel 13	35'	6'
Panel 14	25'	10'
Panel 15	23'	28'
Panel 16	20'	6'
Panel 17	35'	2'
Panel 18	23'	0'
Panel 19	40'	24'
Panel 20	8'	20'
Panel 21	3'	4'
Panel 22	5'	8'
Panel 23	13'	10'
Panel 24	(7) - BOLTED = 0'	5'
Panel 25	8'	5'

Figure 2-3 Naples Island
Seawall Cap and Rail Condition Totals

Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 26	28'	19'
Panel 27	28'	40'
Panel 28	8'	17'
Panel 29	(5) - BOLTED = 0'	9'
Panel 30	15'	6'
Panel 31	30'	23'
Panel 32	20'	27'
Panel 33	3'	2'
Panel 34	8'	8'
Panel 35	35'	11'
Panel 36	30'	12'
Panel 37	30'	25'
Panel 38	30'	14'
Panel 39	5'	8'
Panel 40	10'	2'
Panel 41	30'	2'
Panel 42	25'	4'
Panel 43	38'	7'
Panel 44	35'	7'
Panel 45	15'	3'
Panel 46	35'	3'
Panel 47	20'	4'
Panel 48	10'	4'
Panel 49	30'	6'
TOTAL	**1057'**	**506'**

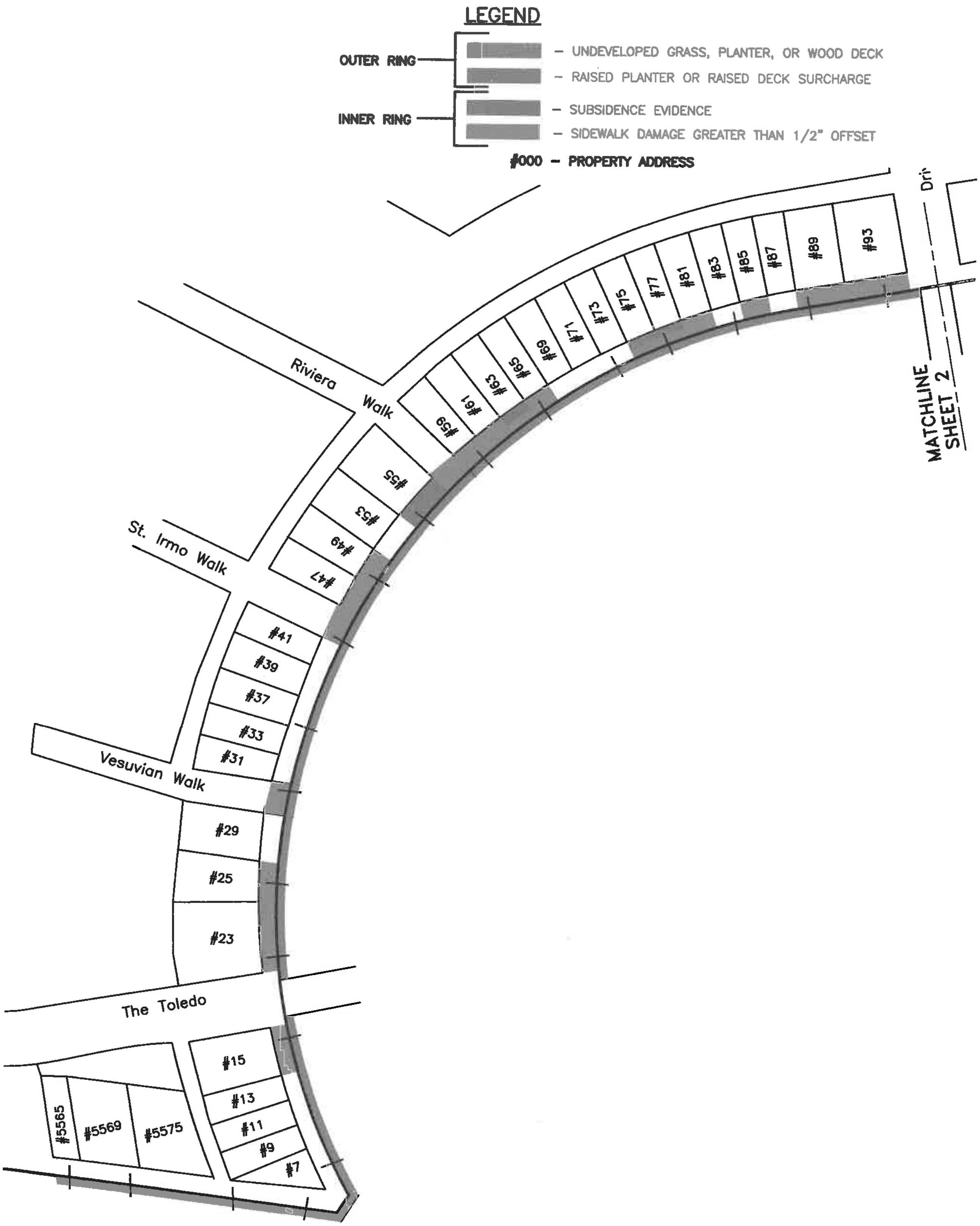


FIGURE 2-4a NAPLES LANDSIDE
SIDEWALK & PLANTER
KEY PLAN
SURVEY DATE: 8/2008

SHEET 1 OF 6

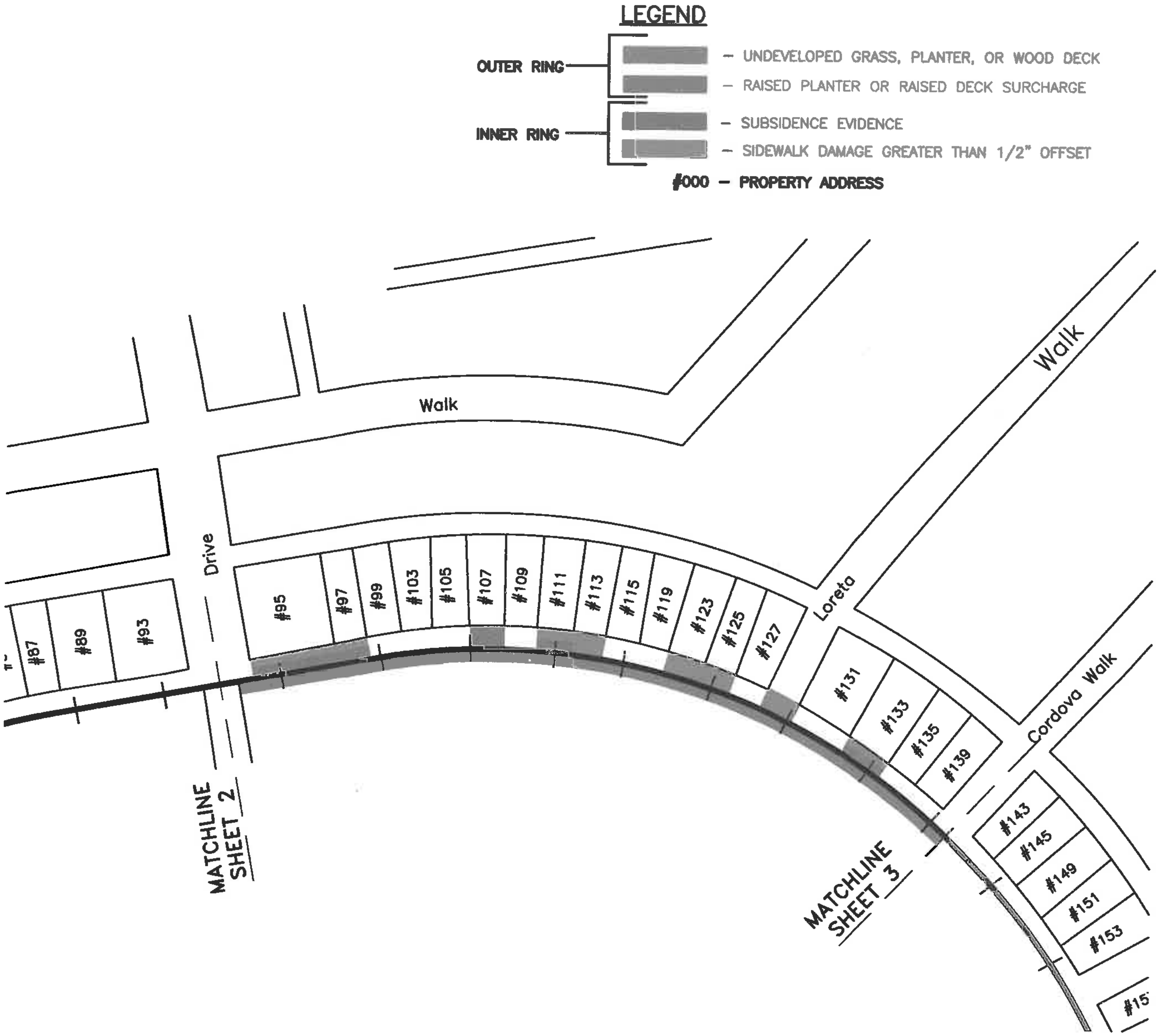


FIGURE 2–4b NAPLES LANDSIDE
SIDEWALK & PLANTER
KEY PLAN
SURVEY DATE: 8/2008

SHEET 2 OF 6

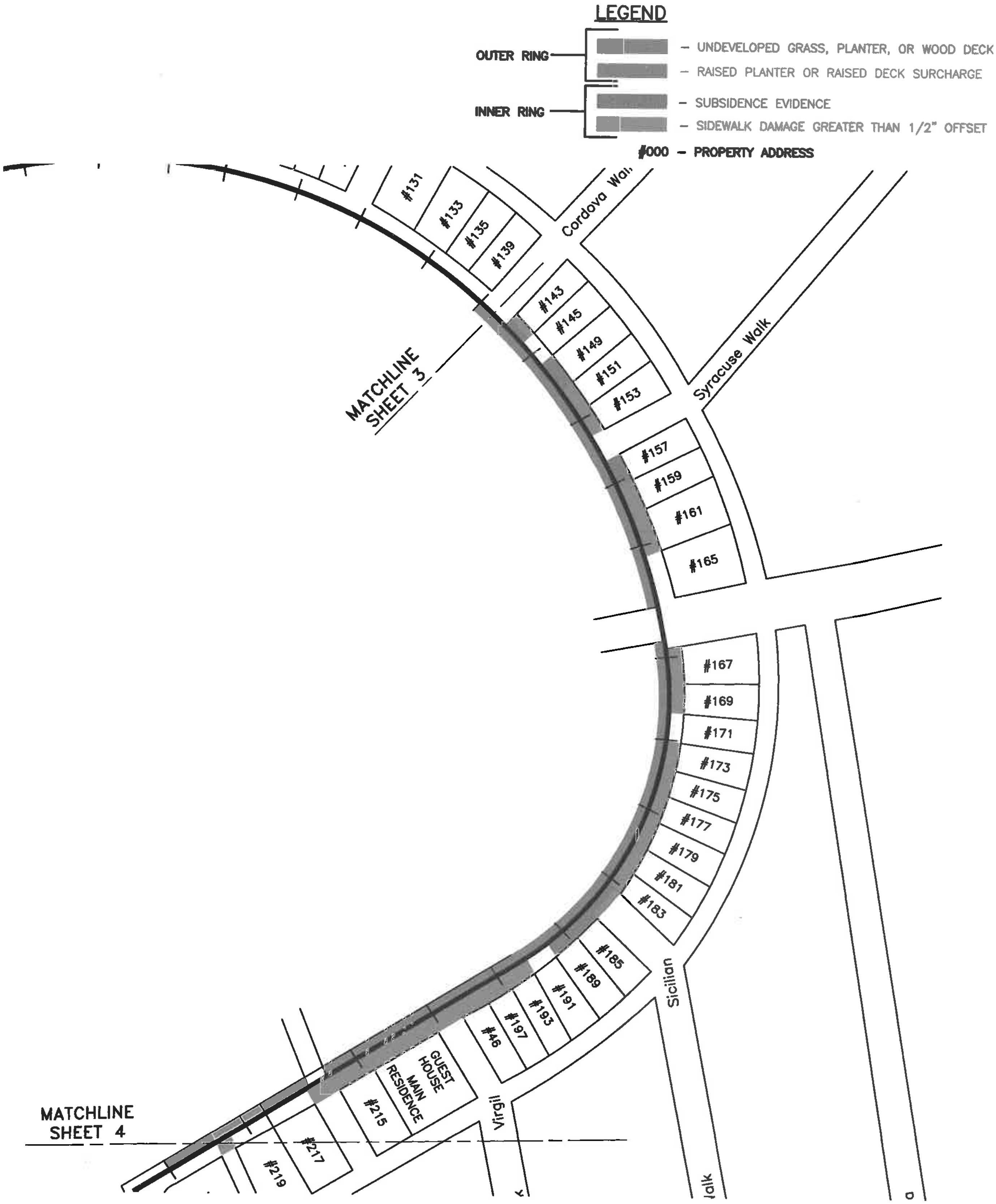


FIGURE 2–4c NAPLES LANDSIDE SIDEWALK & PLANTER KEY PLAN

SURVEY DATE: 8/2008

SHEET 3 OF 6

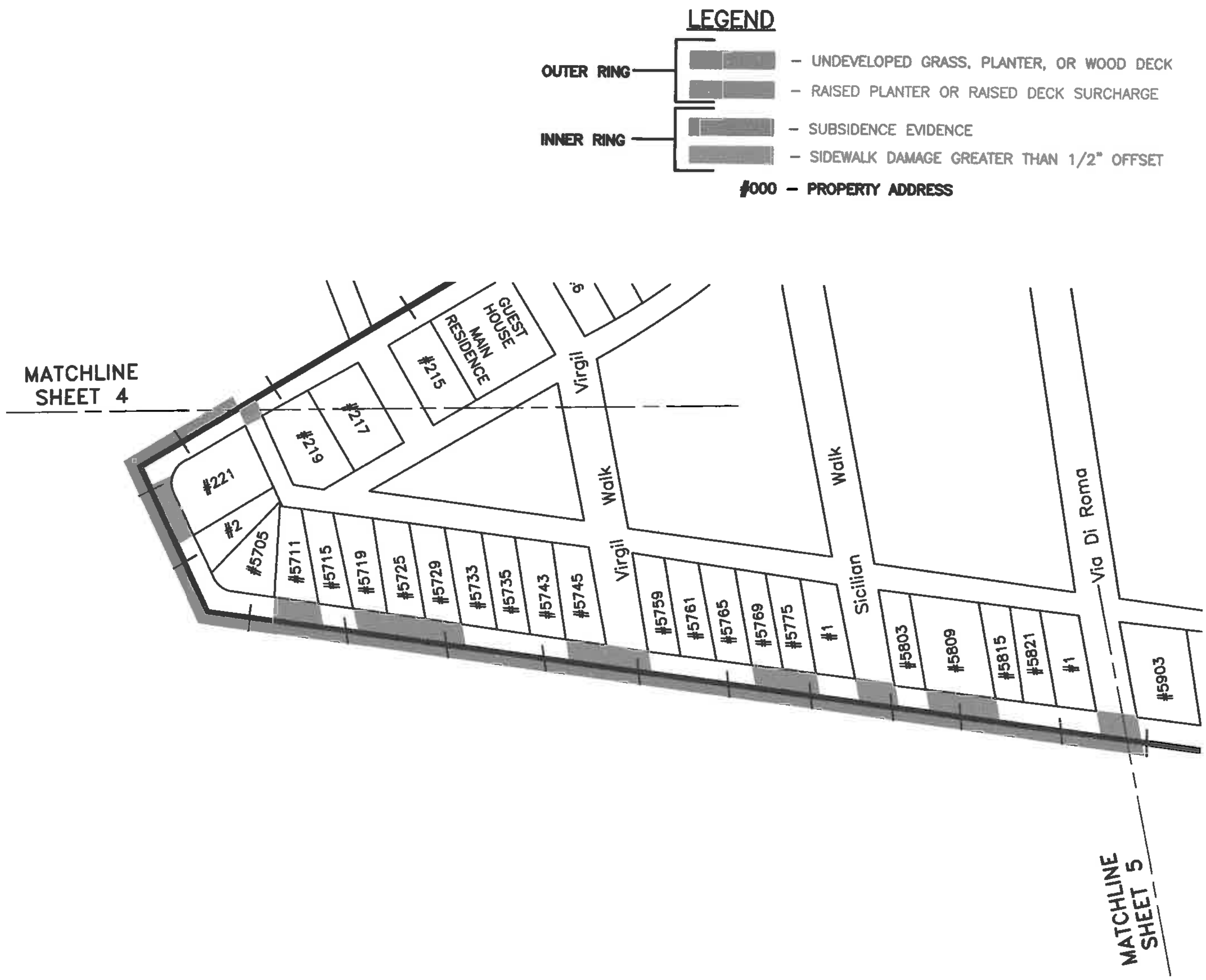


FIGURE 2-4d NAPLES LANDSIDE SIDEWALK & PLANTER KEY PLAN

SURVEY DATE: 8/2008

SHEET 4 OF 6

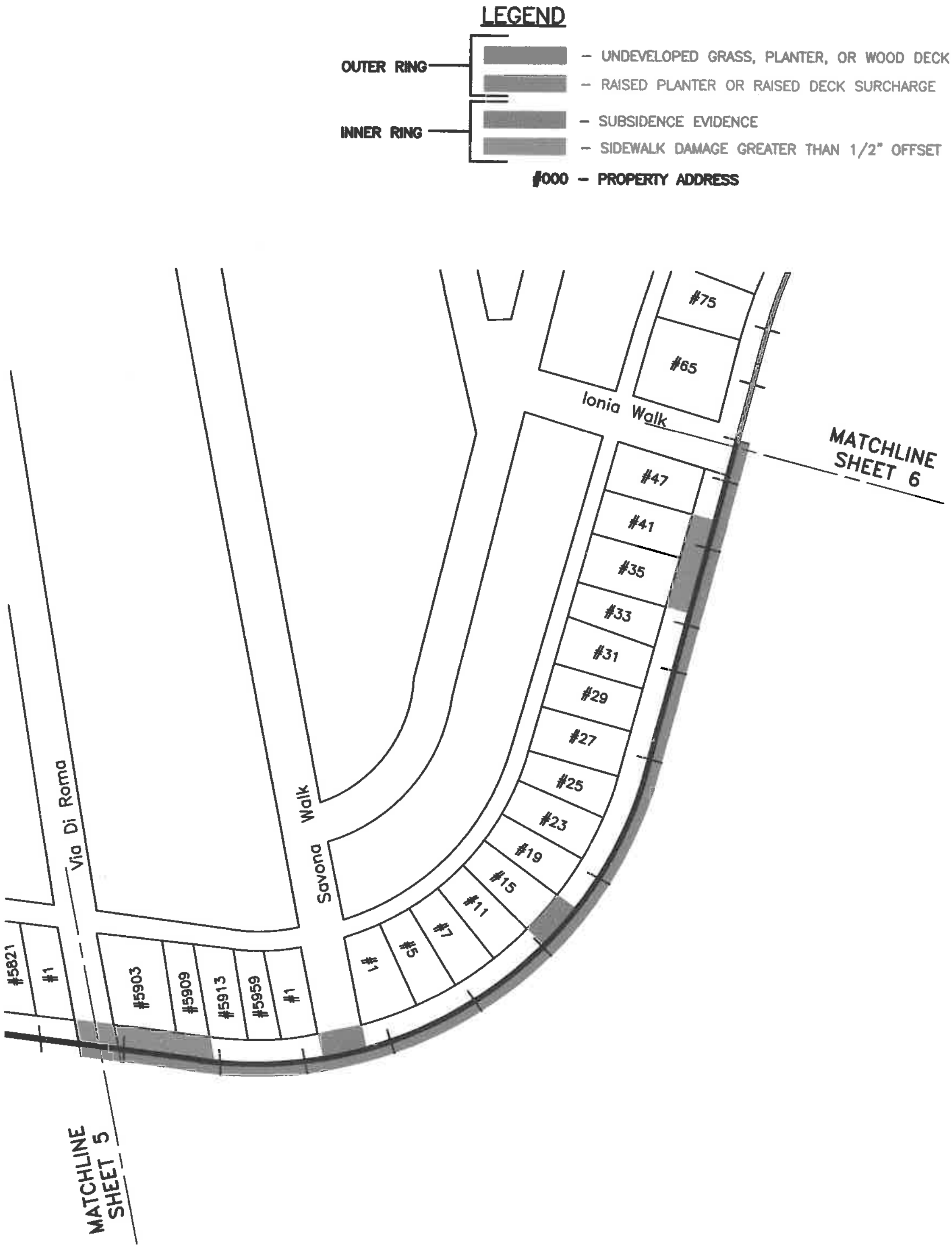


FIGURE 2–4e NAPLES LANDSIDE
SIDEWALK & PLANTER
KEY PLAN
SURVEY DATE: 8/2008

SHEET 5 OF 6

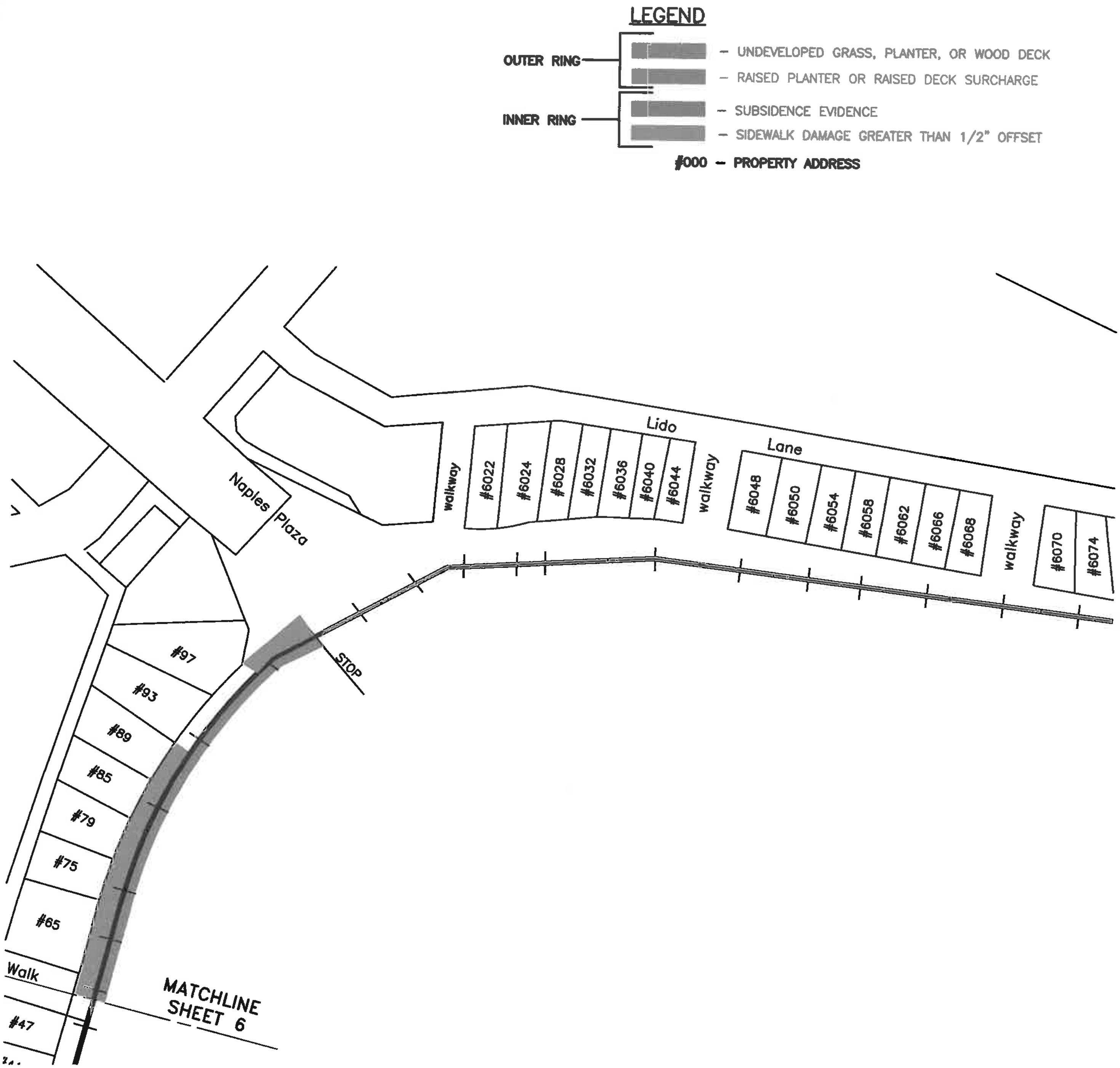


FIGURE 2–4f NAPLES LANDSIDE
SIDEWALK & PLANTER
KEY PLAN

SURVEY DATE: 6/2008

SHEET 6 OF 6

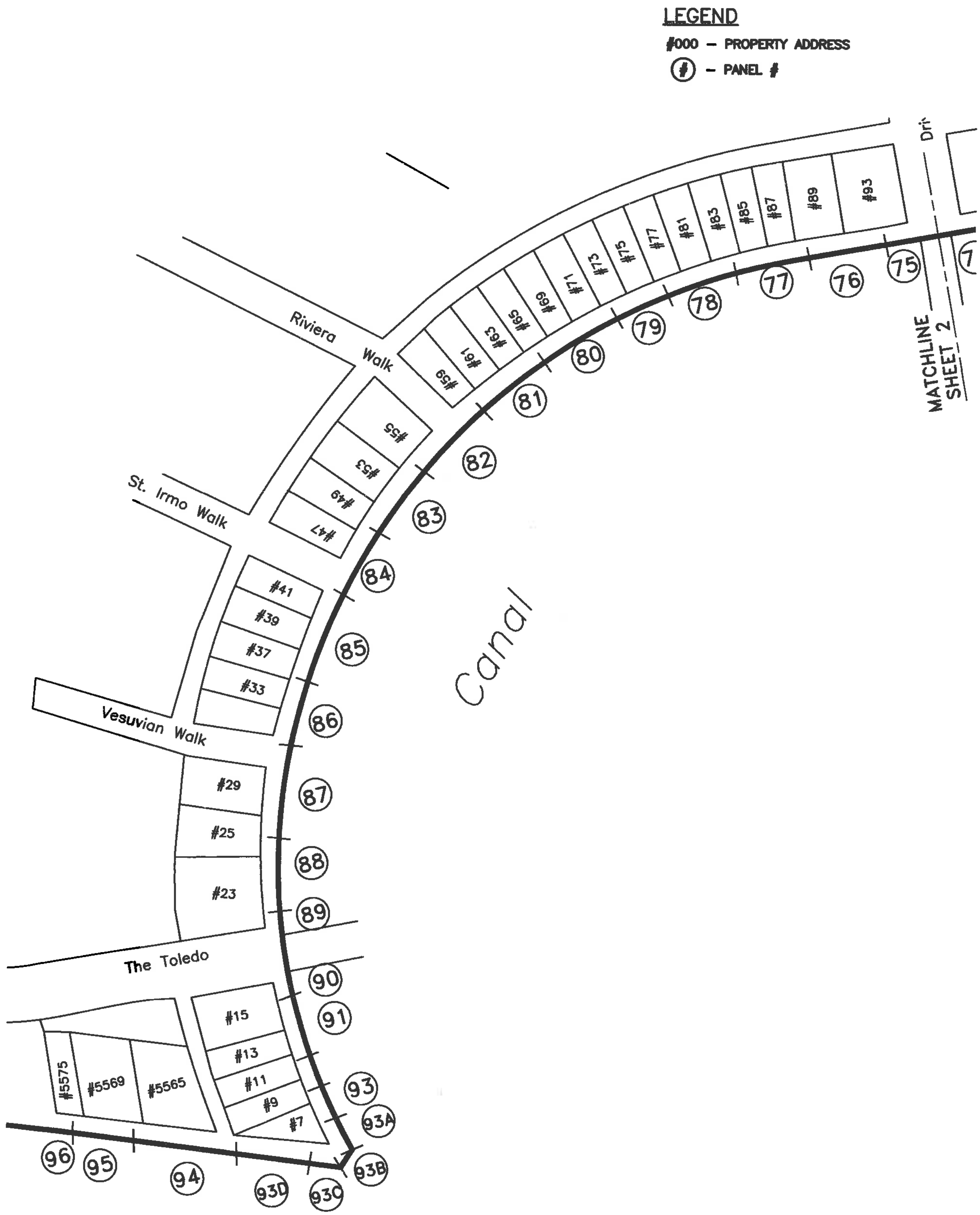


FIGURE 2-5a NAPLES – LANDSIDE SEAWALL CAP & RAIL CONDITION KEY PLAN

SURVEY DATE: 8/2006

SHEET 1 OF 6

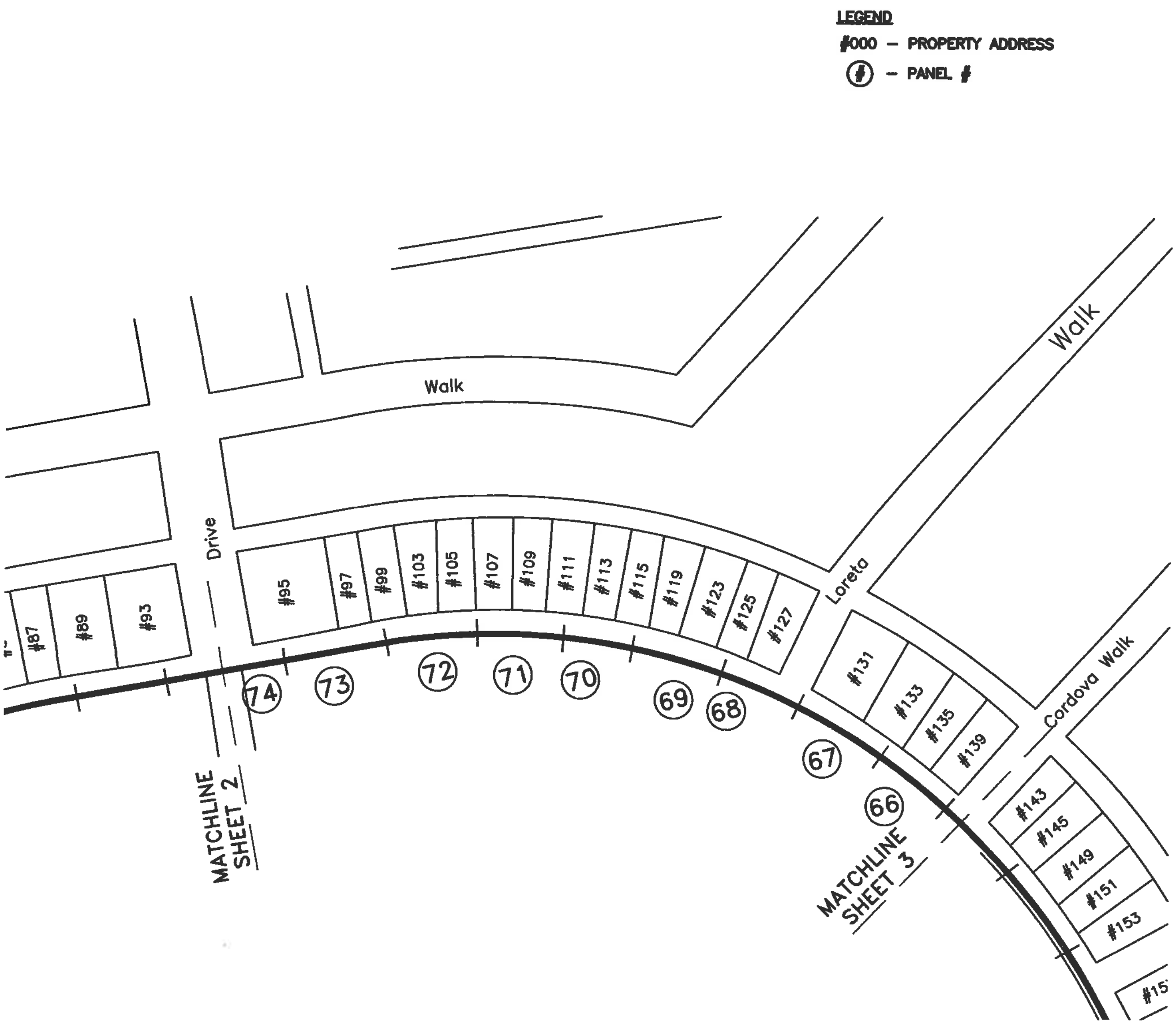


FIGURE 2–5b NAPLES – LANDSIDE SEAWALL CAP & RAIL CONDITION KEY PLAN

SURVEY DATE: 6/2008

SHEET 2 OF 6

LEGEND
#000 – PROPERTY ADDRESS
(#) – PANEL #

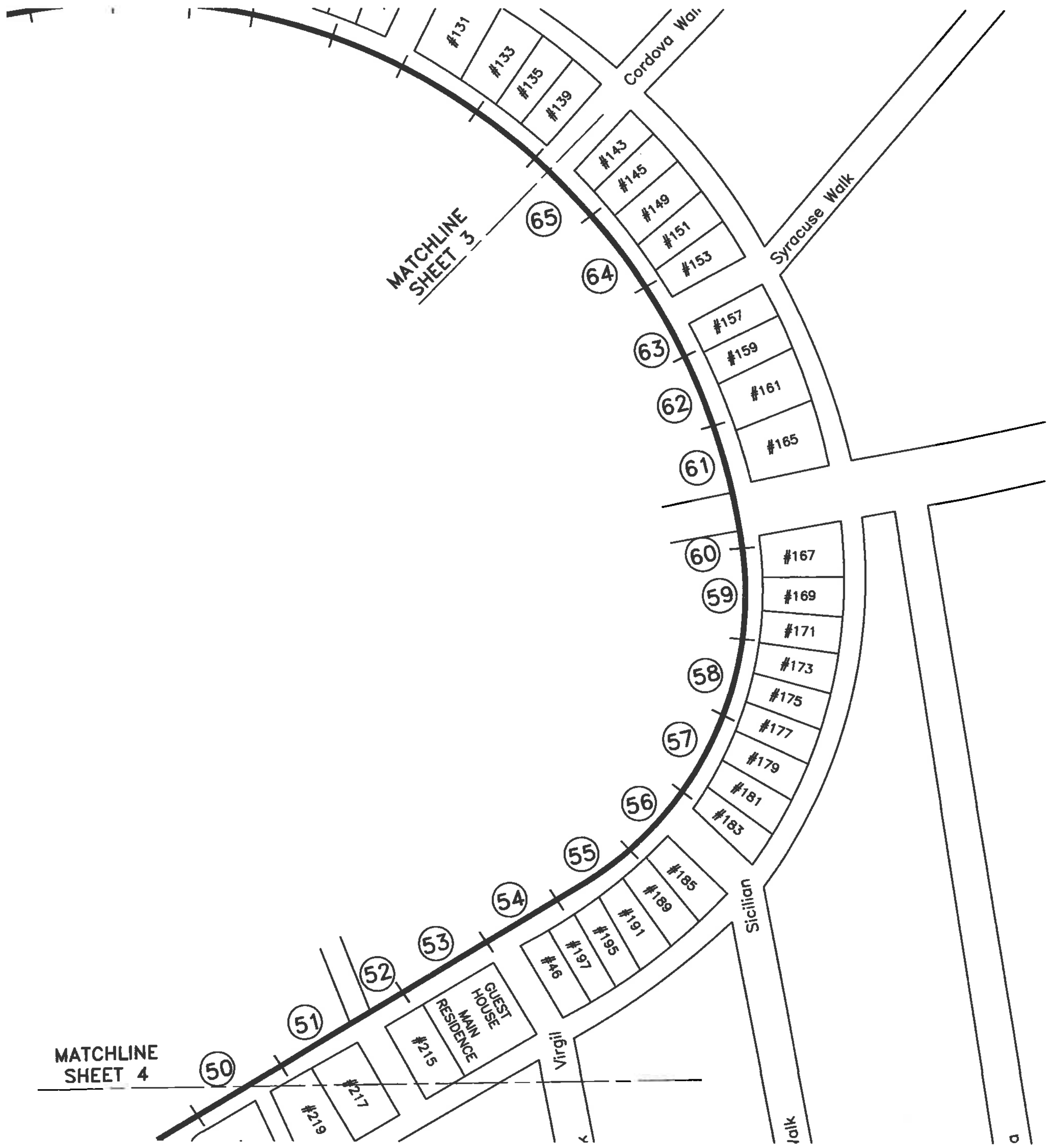


FIGURE 2–5c NAPLES – LANDSIDE SEAWALL CAP & RAIL CONDITION KEY PLAN

SURVEY DATE: 8/2008

SHEET 3 OF 6

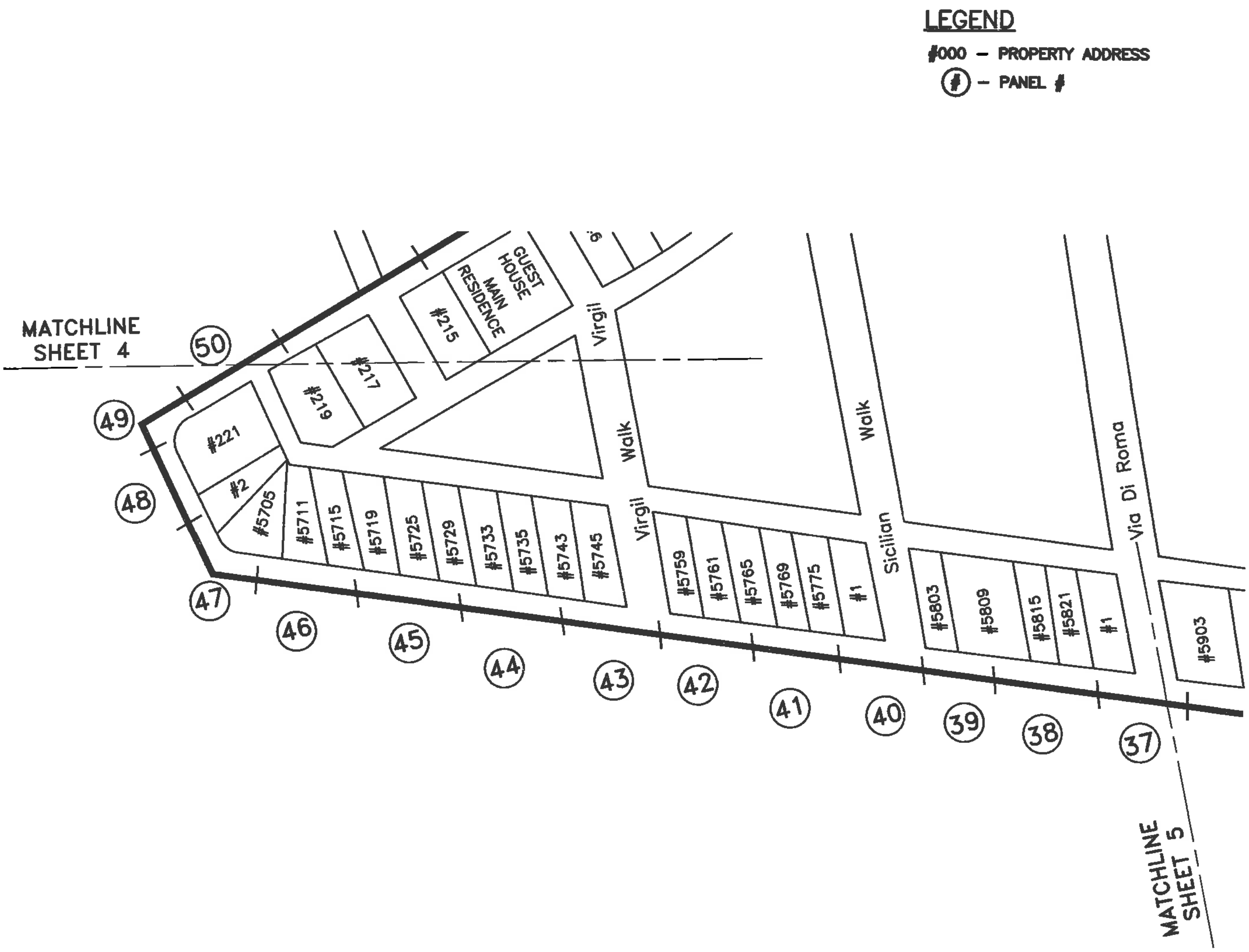


FIGURE 2–5d NAPLES – LANDSIDE SEAWALL CAP & RAIL CONDITION KEY PLAN

SURVEY DATE: 8/2008

SHEET 4 OF 6

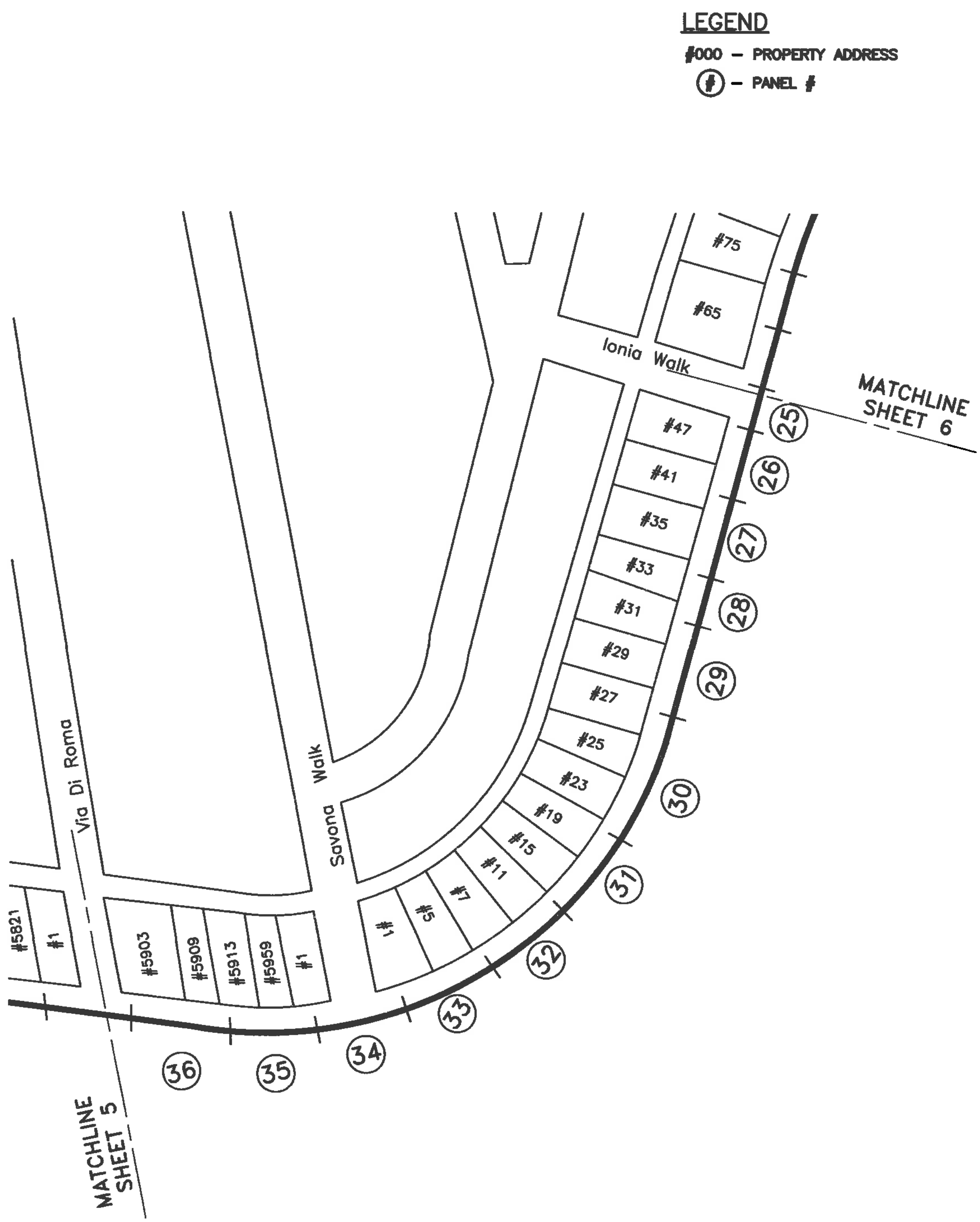


FIGURE 2–5e NAPLES – LANDSIDE SEAWALL CAP & RAIL CONDITION KEY PLAN

SURVEY DATE: 8/2008

SHEET 5 OF 6

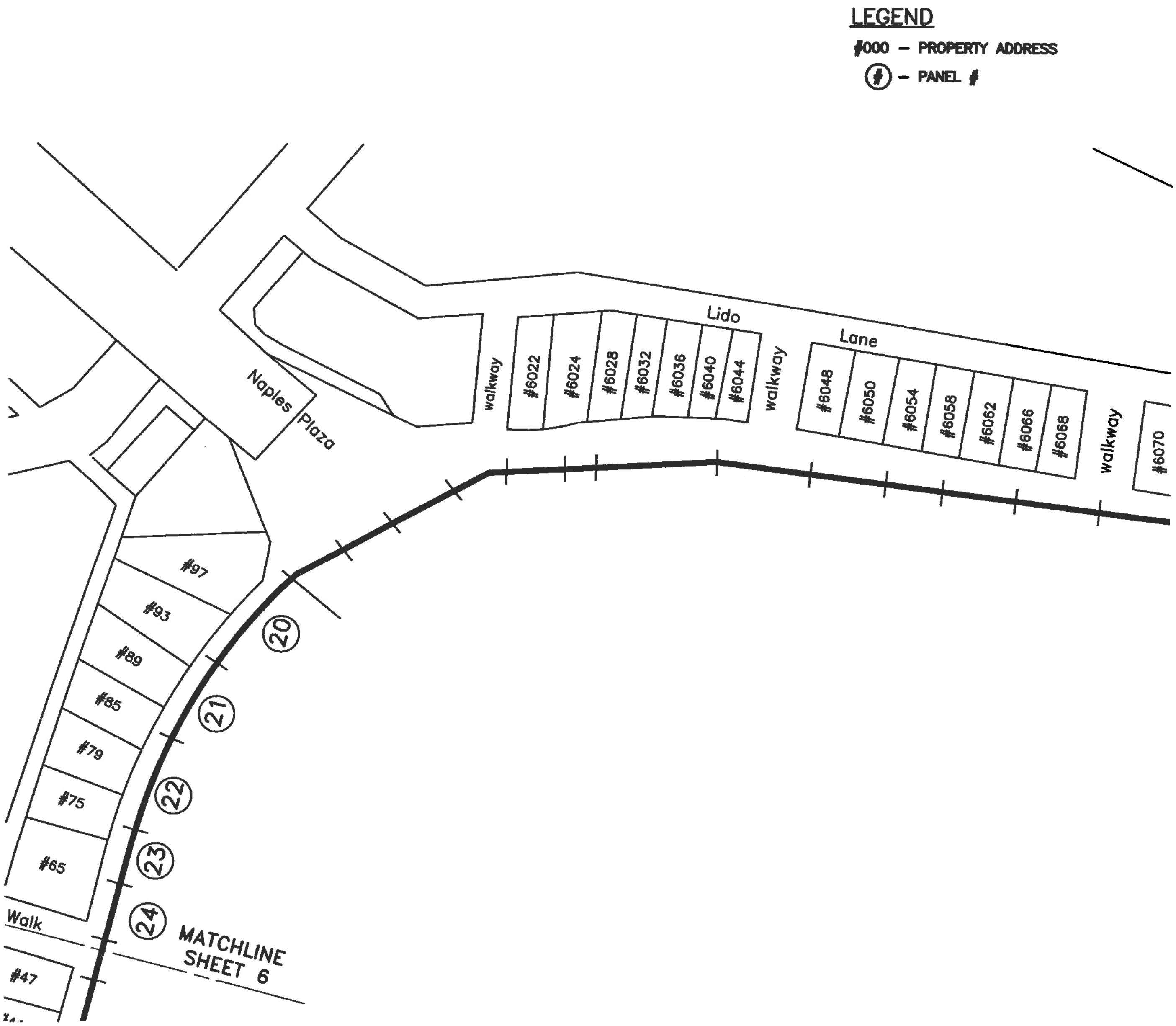


FIGURE 2–5f NAPLES – LANDSIDE
SEAWALL CAP & RAIL CONDITION
KEY PLAN

SURVEY DATE: 8/2008

SHEET 6 OF 6

Figure 2-6 Naples - Landside Top Cap - Rail & Spalling Condition Notes		
Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 20	30'	20'
Panel 21	38'	4'
Panel 22	28'	0
Panel 23	30'	0
Panel 24	33'	6'
Panel 25	15'	8'
Panel 26	15'	3'
Panel 27	35'	5'
Panel 28	15'	4'
Panel 29	28'	3'
Panel 30	45'	5'
Panel 31	8'	N/A
Panel 32	23'	0
Panel 33	30'	0
Panel 34	(9) -BOLTED- = 0'	0
Panel 35	15'	0
Panel 36	0	0
Panel 37	0	0
Panel 38	20'	2'
Panel 39	20'	2'
Panel 40	3'	0
Panel 41	20'	4'
Panel 42	20'	4'
Panel 43	23'	7'
Panel 44	25'	5'

Figure 2-6 Naples - Landside Top Cap - Rail & Spalling Condition Notes		
Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 45	25'	0
Panel 46	28'	1'
Panel 47	18'	2'
Panel 48	13'	5'
Panel 49	13'	5'
Panel 50	33'	11'
Panel 51	23'	7'
Panel 52	5'	2'
Panel 53	40'	6'
Panel 54	10'	8'
Panel 55	28'	18'
Panel 56	40'	20'
Panel 57	33'	58'
Panel 58	25'	8'
Panel 59	23'	26'
Panel 60	3'	11'
Panel 61	33'	63'
Panel 62	30'	50'
Panel 63	15'	57'
Panel 64	33'	46'
Panel 65	13'	0
Panel 66	38'	22'
Panel 67	38'	19'
Panel 68	20'	23'
Panel 69	28'	2'
Panel 70	23'	6'

Figure 2-6 Naples - Landside Top Cap - Rail & Spalling Condition Notes		
Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 71	30'	17'
Panel 72	45'	6'
Panel 73	28'	6'
Panel 74	3'	6'
Panel 75	13'	12'
Panel 76	18'	19'
Panel 77	43'	16'
Panel 78	33'	0
Panel 79	35'	0
Panel 80	(12) -BOLTED- = 0	0
Panel 81	(12) -BOLTED- = 0	6'
Panel 82	25'	20'
Panel 83	38'	28'
Panel 84	28'	28'
Panel 85	38'	35'
Panel 86	40'	13'
Panel 87	33'	8'
Panel 88	23'	10'
Panel 89	15'	6'
Panel 90	13'	8'
Panel 91	58'	21'
Panel 92	0	0
Panel 93	8'	0
Panel 94	33'	5'
Panel 95	25'	12'
Panel 96	13'	7'
TOTALS:	**1788'**	**847'**

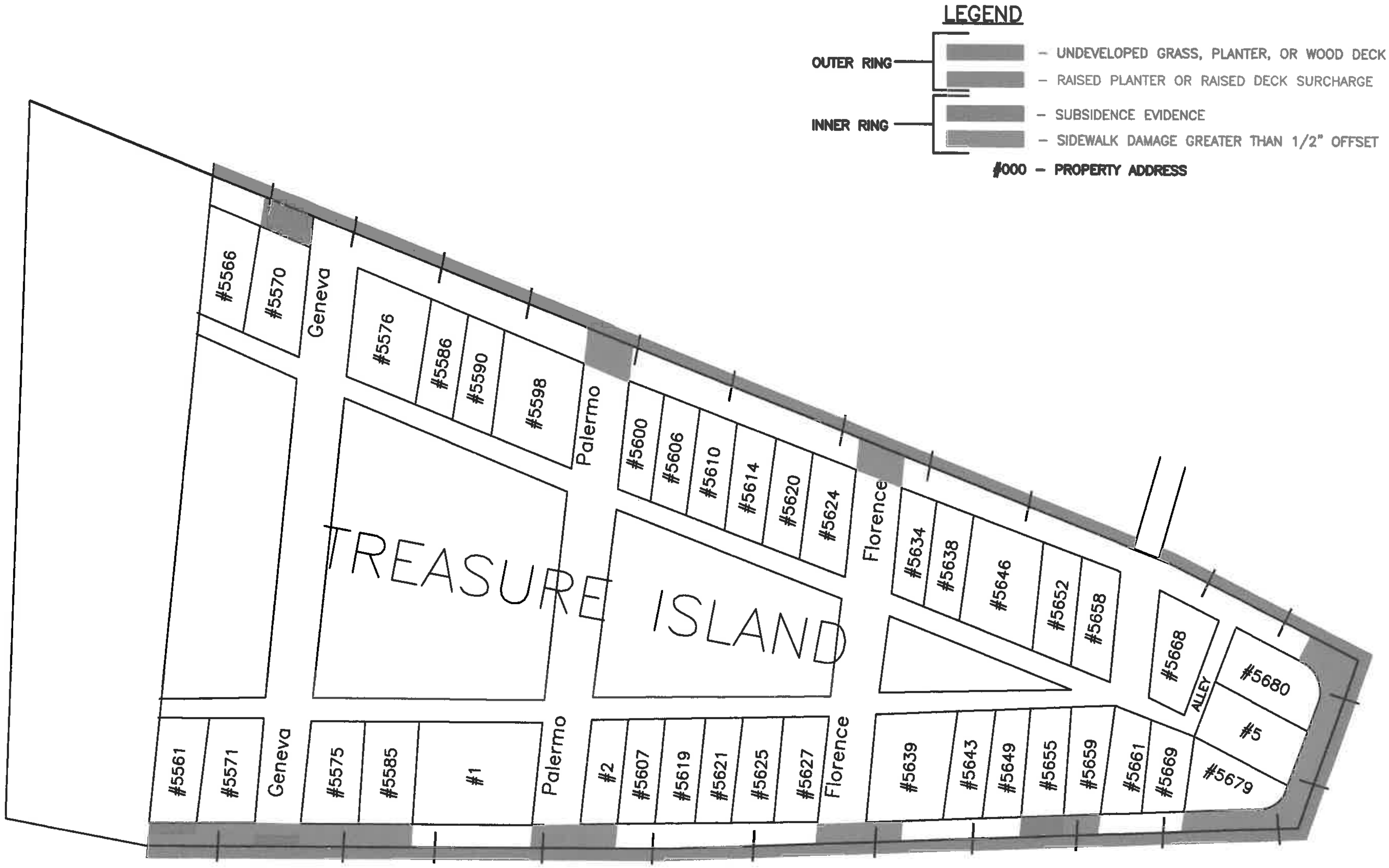


FIGURE 2–7 TREASURE ISLAND
SIDEWALK & PLANTER
KEY PLAN
SURVEY DATE: 8/2008

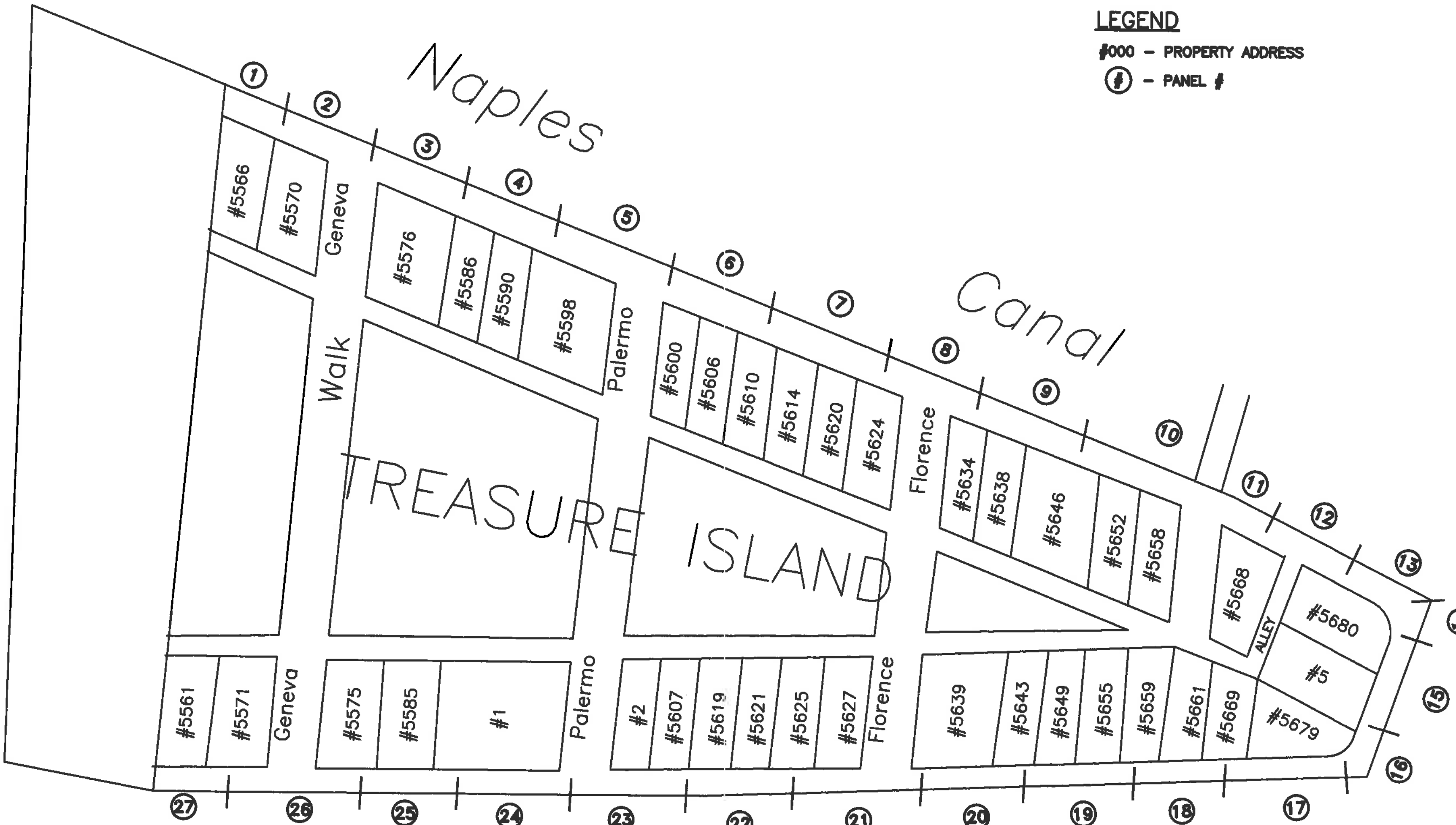
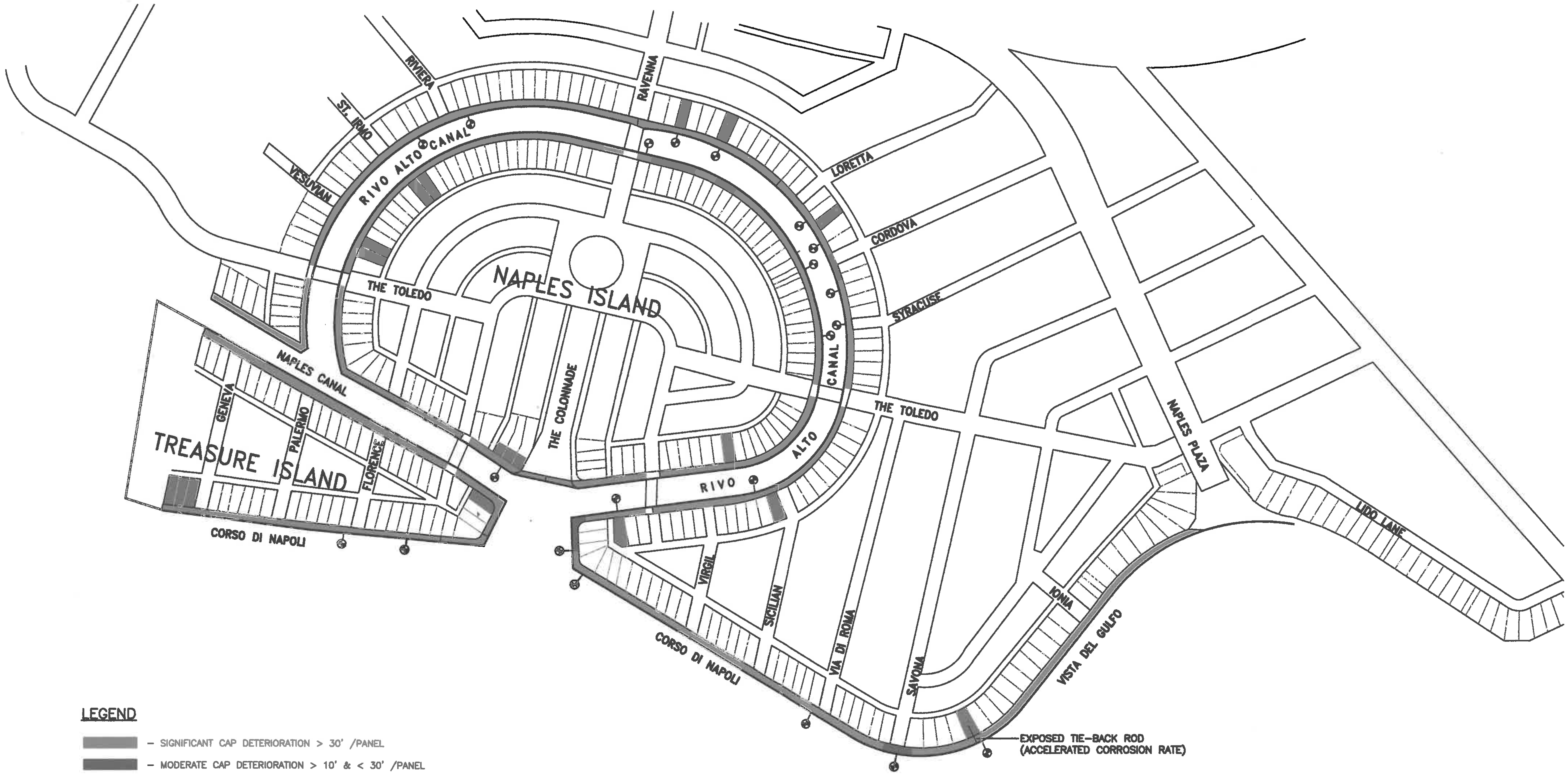


FIGURE 2-8 TREASURE ISLAND
SEAWALL CAP & RAIL CONDITION
KEY PLAN
SURVEY DATE: 6/2008

Figure 2-9 Treasure Island Seawall Cap and Rail Condition Totals		
Panel #	Top Cap - Rail Spalling	Top Cap - Spalling
Panel 1	20'	3'
Panel 2	8'	2'
Panel 3	23'	10'
Panel 4	28'	5'
Panel 5	13'	2'
Panel 6	23'	1'
Panel 7	30'	5'
Panel 8	8'	4'
Panel 9	25'	2'
Panel 10	20'	7'
Panel 11	18'	2'
Panel 12	28'	0
Panel 13	13'	0
Panel 14	13'	0
Panel 15	25'	0
Panel 16	25'	0
Panel 17	35'	5'
Panel 18	33'	9'
Panel 19	35'	0
Panel 20	15'	0
Panel 21	35'	0
Panel 22	18'	0
Panel 23	35'	0
Panel 24	20'	2'
Panel 25	35'	2'
Panel 26	30'	1'
Panel 27	18'	7'
Total Linear Ft.	**629'**	**69'**



LEGEND

- SIGNIFICANT CAP DETERIORATION > 30' /PANEL
- MODERATE CAP DETERIORATION > 10' & < 30' /PANEL
- CAP IN FAIR CONDITION < 10' /PANEL
- INDICATES SINKHOLE OR SUSPECT SINKHOLE IN PLANTING & OR SIDEWALK AREA
- EVIDENCE OF ACTIVE SOIL LOSS (ABOVE WATER SOFT SOILS OR VOIDS & UNDER WATER SAND PILES @ BASE OF WALL)

N

NTS

FIGURE 2-10 NAPLES SEAWALL SUMMARY OF ABOVE WATER CONDITIONS
FEBRUARY 25, 2009

FIGURE 2-10

3, 0 Rail Survey

3.1 History

The Naples Home Owners Association (HOA) has a very capable group of retired technical minded members who have developed a program of annually monitoring the angle of inclination of the guard rail posts which are mounted on the seawall cap. Measurements are taken using a "Smart Level" which is a digital hand level with a reading to the 1/10th degree, (see Photo's 3-1 & 3-2). The ability to track which rails are being revisited for measurement is very challenging. The group has placed a small file mark below the top rail to assist them but often plant growth, dock gate relocations, utilities and decay make finding the starting points a challenge on many properties and the work is best repeated by the same team member using the same protocol.

3.2 Survey Goal

The value in this survey is, once a baseline has been recorded, future readings can be compared to it and it may be possible to identify seawall rotation resulting from tie-rod yielding. If Transystems or the HOA team can identify a failing seawall panel before multiple tie rods break, they have the opportunity to facilitate repairs and protect their property before wall collapse. The HOA team has been surveying the rails since 2000.

3.3 Interpretation of Results

Transystems was able to confirm the accuracy of the data supplied by the HOA Survey Team. The only recommendation we have for future data collection and interpretation are with the distribution of graphics to the public indicating minor movement in or out. While it is easy to say a concrete wall is moving a few tenths of a degree it can mislead outsiders to believe all walls are in motion. It is fairer to indicate that the great majority of walls are stationary and most small changes are due to the accuracy of recordings. We have found readings change for tidal influence, temperature, season water table, impact to the rails, corrosion at the base connection, recalibration of the level, installation of new dock gates and the ability of a technician to place the level on the same clean surface as years past. We would recommend that an approximate ½ degree tolerance be justifiable to account for the above variations before a conclusion is drawn that a wall panel has moved in or out. Should multiple adjacent measurements support a conclusion than it may be appropriate to consider a smaller number Transystems utilized an "In" and "Out" designation for a lean "outboard" towards the water or "inboard" towards shore, (see Figure 3-0 on page 3-3). Conclusions were not drawn for isolated measurements contradicted by measurements on adjacent rails.

3.4 Survey Results

Survey results are shown graphically in Figures 3-1, 3-3 and 3-5 with the raw field data provided in Figures 3-2, 3-4 & 3-6. No significant data supporting a conclusion of "out" rotation was found. The following "In" findings require discussion before being dismissed.

Treasure Island: 1.0 degree or less "In" minor rotation at panel 10, (see Figure 3-5 on page 3-25). "In" movement does not concern tie-rods. Our dive team saw no evidence that any sheets have significant soil loss to allowing movement. It is considered safe to dismiss this finding since contradicted by other data.

Naples Island: No conclusion of change occurring was found on this island.

Landside: 1.8 degrees or less "In" minor rotation at panel 88, (see Figure 3-3a on page 3-9). "In" movement does not concern tie-rods. Our dive team saw no evidence that any sheets have significant soil loss to allowing movement. It is considered safe to dismiss this finding since contradicted by other data.

Landside: 4.4 degrees or less "In" significant rotation at panel 82, (see Figure 3-3a on page 3-9). "In" movement does not concern tie-rods. Our dive team saw no evidence that any sheets have significant soil loss to allow movement. The next adjacent rail had only moved 0.7 degrees. It is considered safe to dismiss this finding since contradicted by other data. The rail may have been struck or we measured a different rail than in past.

3.5 Seawall Cap Construction Joints

Measurements were recorded for horizontal and vertical movement of the seawall cap construction joints for evaluation of seawall movement, (see Photo's 2-19 thru 2-22). The findings appear typical for sheet pile bulkhead construction. This data will gain value if the City or HOA use it for comparison with future surveys. It can also confirm or be the primary indicator of tie rod yielding. We expect fewer measurement variables affecting the concrete cap than the aluminum guard rail system.

LEGEND

- SIGNIFICANT ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- MINOR ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- APPROXIMATE PLUMB CONDITION
- MINOR ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007
- SIGNIFICANT ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007

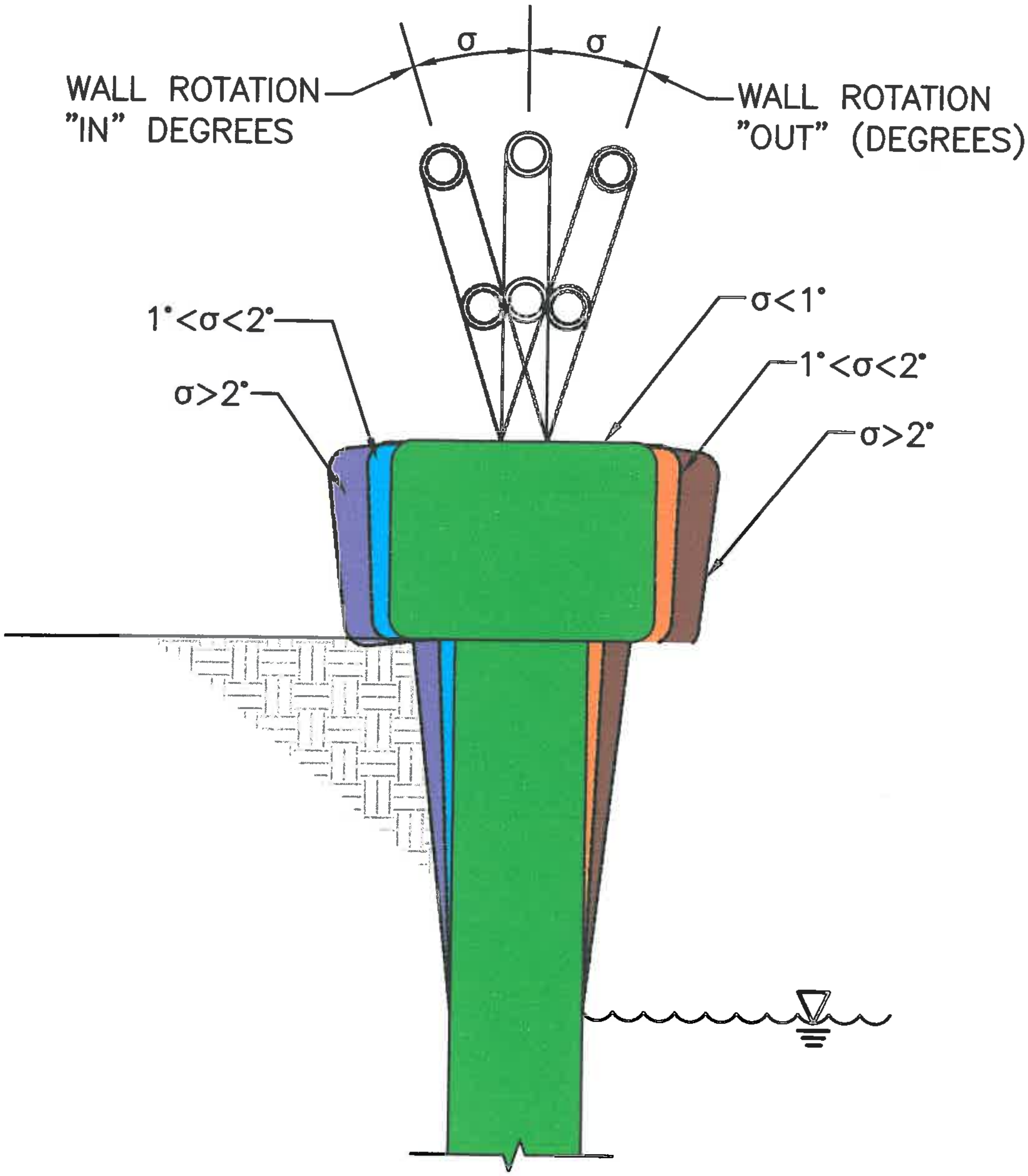


FIGURE 3-0 ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 8/2008

LEGEND

NO SIGNIFICANT MOVEMENT OBSERVED SINCE 11/2007

(#) - PANEL #

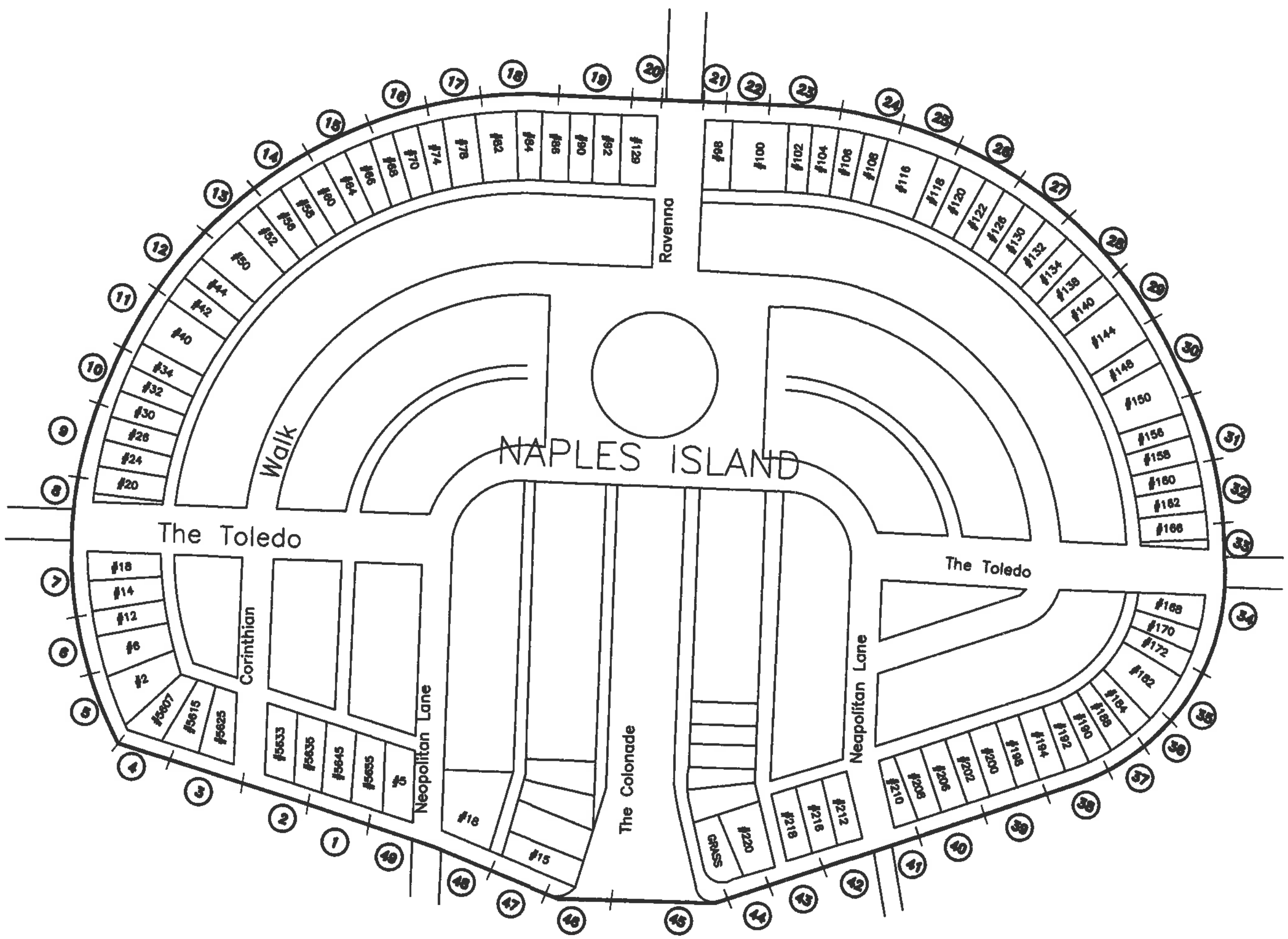


FIGURE 3-1 NAPLES ISLAND
RESULTS OF WALL ROTATION ANALYSIS
FROM SURVEY OF GUARDRAIL POSTS
SURVEY DATE: 8/2008

Naples Island - Naples Canal

11/2007 SEAWALL SURVEY

Address			S1	S2	S3	S4	S5	S6
Napels Canal - The Colonade								
Colonnade Park			87.2	87.9	87.2	89.4	89.7	89.4
			S7 89.9	S8 88.6				
Napels Canal - #15			89.8	88.2	89.9	89.6	87.8	89.7
			S7 88.4	S8 89.3	S9 90.2	S10 89.1	S11 92	
Napels Canal - #18			90.7	90.6	90.5	90.1	89	89.9
			S7 90.7	S8 90.3				
Napels Canal - #5			90.6	90.4	90.1	89.9	89.6	89.5
Napels Canal - #5655			90	90	90.8			
Napels Canal - #5645			91	90.3	90.3			
Napels Canal - #5635			90	91.2	90.1	90.7		
Napels Canal - #5633			90.5	91.1	89.7			
Napels Canal - Corinthian Walk			90.1	90.3	89.1			
Napels Canal - #5625			88.6	89.7	89	90.9	91.9	
Napels Canal - #5615			90.9	91.4	89.9	89.2		
Napels Canal - #5607			89	88.1	88.7	88.4	90.6	

Naples Island - Rivo Alto Canal

Address			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #2			92.5	89.6	90.7	89.9	89.8	88.7
			S7 89.3	S8 88.9				
Rivo Alto Canal - #6			89.9	90.4	89.2	89.6	92.2	
Rivo Alto Canal - #12			91.3	89.6	88			
Rivo Alto Canal - #14			88.9	91.6	90.2			
Rivo Alto Canal - #18			89.9	89.7	90.6	90.8		

Naples Island - Naples Canal

11/2008 SEAWALL SURVEY

Address			S1	S2	S3	S4	S5	S6
Napels Canal - The Colonade								
Colonnade Park		IN .6 DEGREES	87.6 OUT	88.4 OUT	87.7 OUT	89.9 OUT	90	90
			S7 89.4 IN	S8 89.2 OUT				
Napels Canal - #15			89.5 IN	88.8 OUT	89.5 IN	90	88.3 OUT	89.7 IN
		IN .7 DEGREES	S7 89.0 OUT	S8 90	S9 90.0 IN	S10 89.8 OUT	S11 87.3 IN	
Napels Canal - #18			88.8 IN	88.9 IN	89.0 IN	89.5 IN	89.3 OUT	89.6 IN
			S7 89.2 IN					
Napels Canal - #5			88.8 IN		89.2 IN		90	
Napels Canal - #5655			89.4 IN	89.4 IN	88.4 IN			
Napels Canal - #5645			88.3 IN	89.1 IN	88.1 IN			
Napels Canal - #5635			89.4 IN	88.2 IN	89.4 IN	88.7 IN		
Napels Canal - #5633			88.9 IN	88.2 IN	89.7 IN			
Napels Canal - Corinthian Walk			89.4 IN	89.0 IN	89.7 OUT	89.1 OUT		
Napels Canal - #5625		IN .8 DEGREES	89.2 OUT	89.7 IN	89.6 OUT	88.5 IN	87.3 IN	
Napels Canal - #5615			88.5 IN	87.8 IN	89.4 IN	90		
Napels Canal - #5607			89.6 OUT	88.7 OUT	89.3 OUT	89.1 OUT	88.7 IN	

Naples Island - Rivo Alto Canal

Address			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #2		IN .7 DEGREES	86.8 IN	89.8 IN	88.8 IN	89.5 IN	89.5 IN	89.4 OUT
			S7 89.5 OUT	S8 89.5 OUT				
Rivo Alto Canal - #6		IN .8 DEGREES	89.0 IN	88.8 IN	89.6 OUT	89.0 OUT	87.0 IN	
Rivo Alto Canal - #12			88.0 IN	89.7 IN	88.6 OUT			
Rivo Alto Canal - #14		IN .8 DEGREES	89.6 OUT	87.6 IN	89.0 IN			
Rivo Alto Canal - #18			89.3 IN	89.6 IN	88.8 IN	88.5 IN		

FIGURE 3-2a

Address			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - WEST								
Rivo Alto Canal - #20			89.7	91.5	91.6	91.2	90.2	90.9
Rivo Alto Canal - #24			89.6	90.7	89.9			
Rivo Alto Canal - #26			89.7	90.4	90.6			
Rivo Alto Canal - #30			89.9	90.4	90.4			
Rivo Alto Canal - #32			90.2	90.3	89.2			
Rivo Alto Canal - #34			90	90	90.4			
Rivo Alto Canal - #40			90.1	90.9	88.5	90.7	89.3	
Rivo Alto Canal - #42			90.3	89.5	89.5	89.9		
Rivo Alto Canal - #44			89.6	89.8	89.5			
Rivo Alto Canal - #50			89.9	90.9	90.5	89.1	89.7	88.8
Rivo Alto Canal - #52			89.9	90.3	89.6			
Rivo Alto Canal - #56			89.3	90.1	88.4			
Rivo Alto Canal - #58			87.5	87.8	88.3			
Rivo Alto Canal - #60			88.4	88.2	88.7			
Rivo Alto Canal - #64			88.8	87.7	88.5			
Rivo Alto Canal - #66			88.6	87.4	88.3			
Rivo Alto Canal - #68			88.7	88.4	88			
Rivo Alto Canal - #70			87.7	88.1	86	86.9		
Rivo Alto Canal - #74			88	87.1	87.1			
Rivo Alto Canal - #78			88.7	89.1	89	88.2	89.4	
Rivo Alto Canal - #82			89.3	87.8	89.7	88.1		
Rivo Alto Canal - #84			88.7	89.3	89.1			
Rivo Alto Canal - #86			89.1	88.1	88.3			
Rivo Alto Canal - #90			87.8	88.7	89.1	88.5	88.1	87.3
Rivo Alto Canal - #92			87.7	88.3	89	89.7	89.3	88.6
Rivo Alto Canal - #129			89.7	91.5	91	89.9	89.8	90.9
			S7 91	S8 90.5	S9 91.2	S10 91.2	S11 90	

.	s		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - WEST								
Rivo Alto Canal - #20		IN .8 DEGREES	89.6 IN	87.8 IN	87.6 IN	88.0 IN	89.1 IN	88.3 IN
Rivo Alto Canal - #24			90	88.6 IN	89.2 IN			
Rivo Alto Canal - #26			89.7 IN	89.9 IN	88.7 IN			
Rivo Alto Canal - #30			89.5 IN	89.0 IN	89.0 IN			
Rivo Alto Canal - #32			89.1 IN	89.1 IN	89.8 OUT			
Rivo Alto Canal - #34			89.4 IN	89.4 IN	88.8 IN			
Rivo Alto Canal - #40			89.3 IN	88.4 IN	88.1 OUT	88.7 IN	89.9 OUT	
Rivo Alto Canal - #42			89.2 IN	90	90	89.4 IN		
Rivo Alto Canal - #44			89.5 IN	89.4 IN	89.7 IN	89.8 IN		
Rivo Alto Canal - #50			90	88.8 IN	89.5 OUT	90	89.4 IN	
Rivo Alto Canal - #52			89.8 IN	89.1 IN	89.7 IN			
Rivo Alto Canal - #56			89.8 OUT	90.1 IN	89.2 OUT			
Rivo Alto Canal - #58		IN .3 DEGREES	87.8 OUT	88.1 OUT	88.7 OUT			
Rivo Alto Canal - #60			88.7 OUT	88.4 OUT	89.0 OUT			
Rivo Alto Canal - #64			89.1 OUT	88.0 OUT	88.9 OUT			
Rivo Alto Canal - #66		IN .5 DEGREES	89.0 OUT	87.9 OUT	88.7 OUT			
Rivo Alto Canal - #68			89.3 OUT	88.9 OUT				
Rivo Alto Canal - #70		IN .5 DEGREES	88.5 OUT	88.3 OUT	86.5 OUT			
Rivo Alto Canal - #74		IN .6 DEGREES	88.5 OUT	87.7 OUT	87.5 OUT			
Rivo Alto Canal - #78			89.3 OUT	89.7 OUT	89.5 OUT	88.6 OUT	89.6 OUT	
Rivo Alto Canal - #82			88.9 OUT	88.6 OUT	89.8 IN	88.7 OUT		
Rivo Alto Canal - #84			89.4 OUT	89.9 OUT	89.7 OUT			
Rivo Alto Canal - #86			89.7 OUT	88.7 OUT	89.0 OUT			
Rivo Alto Canal - #90		IN .5 DEGREES		89.2 OUT		89.2 OUT		87.8 OUT
Rivo Alto Canal - #92				88.9 OUT		89.5 IN		89.3 OUT
Rivo Alto Canal - #129		OUT 1.2 DEGREES		87.3 OUT		89.1 IN		89.1 IN
				S8 88.6 IN		S10 89.2 IN		

FIGURE 3-2b

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Ravena						
Rivo Alto Canal - #98	89.2	90.4	89.6	88.9		
Rivo Alto Canal - #100	89.4	89.7	90.5	89.2	87.2	
Rivo Alto Canal - #102	88.9	88.5	89.2			
Rivo Alto Canal - #104	89	89.5	89.9			
Rivo Alto Canal - #106	90.2	89.1	89.9			
Rivo Alto Canal - #108	90.9	89.2	90.2	90.6		
Rivo Alto Canal - #116	89.3	88	89.1	88.9	89.8	90.7
Rivo Alto Canal - #118	90.2	90.6	89.2			
Rivo Alto Canal - #120	89.6	90.4	89.8			
Rivo Alto Canal - #122	90.3	89	89.3			
Rivo Alto Canal - #126	89.6	89.2	88.3	88.6	89.9	90.1
Rivo Alto Canal - #130	90.4	89.4	89.3	91	90.8	89.5
Rivo Alto Canal - #132	89.9	89.1	90.2	90.2	89.3	
Rivo Alto Canal - #134	88.3	88.8	87.8	88.8	89.9	89.8
	S7 89.9	S8 89.7				
Rivo Alto Canal - #138	89.8	89.4	88.5	87.8	88.1	89.2
Rivo Alto Canal - #140	90.4	90.4	89.8	90.2	89.2	89.5
	S7 90.4	S8 89.6	S9 90.1			
Rivo Alto Canal - #144	89.5	89	89.5	90.3	88.8	89.1
	S7 88.3	S8 90				
Rivo Alto Canal - #148	88.3					
Rivo Alto Canal - #150	88.3	89.1	87.5	91.1	87	89.3
Rivo Alto Canal - #156	89.7	88.2	88.8			
Rivo Alto Canal - #158	90	89.4	89	88.5	88.5	
Rivo Alto Canal - #160	88.1	88.5	90.5			
Rivo Alto Canal - #162	88	88.1	89.4			
Rivo Alto Canal - #166	88.2	87.6	89.8	90.3	89.9	

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Ravena							
Rivo Alto Canal - #98		90	89.7 OUT	90	89.7 OUT		
Rivo Alto Canal - #100		90	89.5 IN	88.5 IN	89.9 OUT	88.0 OUT	
Rivo Alto Canal - #102		89.0 OUT	89.2 OUT	89.8 OUT			
Rivo Alto Canal - #104		89.9 OUT	89.5 IN	88.1 IN			
Rivo Alto Canal - #106		89.4 IN	89.8 OUT	89.0 IN			
Rivo Alto Canal - #108		88.8 IN	90	89.4 IN	88.7 IN		
Rivo Alto Canal - #116		89.7 OUT	88.8 OUT	89.3 OUT	89.6 OUT	89.5 IN	88.8 IN
Rivo Alto Canal - #118		89.0 IN	88.6 IN	90			
Rivo Alto Canal - #120		88.8 IN	88.8 IN	89.5 IN			
Rivo Alto Canal - #122		89.1 IN	88.5 OUT	90			
Rivo Alto Canal - #126		89.8 OUT		88.6 OUT		89.4 IN	
Rivo Alto Canal - #130		88.9 IN		90		88.4 IN	
Rivo Alto Canal - #132			89.7 OUT		89.1 IN		86.7 OUT
Rivo Alto Canal - #134			88.1 OUT		89.6 IN		89.0 IN
Rivo Alto Canal - #138		89.3 IN		89.2 OUT		88.4 OUT	
Rivo Alto Canal - #140			89.9 IN		89.0 IN		89.9 OUT
			S8 90				
Rivo Alto Canal - #144		90		88.0 OUT		89.1 OUT	
		S7 89.0 OUT					
Rivo Alto Canal - #148		HEDGE	HEDGE	HEDGE	HEDGE	HEDGE	HEDGE
Rivo Alto Canal - #150	IN .7 DEGREES	88.7 OUT	89.0 OUT	88.0 OUT	88.1 IN	87.7 OUT	90
Rivo Alto Canal - #156		89.8 IN	89.9 OUT	89.5 OUT			
Rivo Alto Canal - #158		89.3 IN			89.7 OUT		89.2 OUT
Rivo Alto Canal - #160		88.8 OUT	89.2 OUT	89.6 IN			
Rivo Alto Canal - #162		88.9 OUT	88.8 OUT	90			
Rivo Alto Canal - #166	IN .3 DEGREES	88.5 OUT	87.9 OUT	89.4 IN	89.2 IN	89.6 IN	

FIGURE 3-2c

Rivo Alto Canal - The Toledo Bridge - EAST								
Rivo Alto Canal - #168			90.6	90.3	89.5	89.2	89.8	89.8
Rivo Alto Canal - #170			90.5	89.2	90.7			
Rivo Alto Canal - #172			91.3	90.9	88.2	90.4		
Rivo Alto Canal - #182			88.4	88.5	89.4	88.9	90	90.2
Rivo Alto Canal - #184			90.7	90.1	90			
Rivo Alto Canal - #188			90.2	90.6	89.7			
Rivo Alto Canal - #190			90.3	89.8	88.9			
Rivo Alto Canal - #192			88.1	90.1	91.8	90.9	90.5	90.1
Rivo Alto Canal - #194			89.3	89.9	90	90.7		
Rivo Alto Canal - #198			90.7	90.6	90.6			
Rivo Alto Canal - #200			91	89.7	90.9			
Rivo Alto Canal - #202			91.1	90.6	91.8			
Rivo Alto Canal - #206			90.2	89.1	90			
Rivo Alto Canal - #208			91.3	89.7	89.4			
Rivo Alto Canal - #210			91	90.4	89.8	90.4		
Napels Canal - Neopolitan Lane - EAST								
Rivo Alto Canal - #212			90	90.2	90.1	90		
Rivo Alto Canal - #216			89.3	90.8	90.7			
Rivo Alto Canal - #218			89.4	89.3	89.6			
Napels Canal - Alley			88.5					
Rivo Alto Canal - #220			89.5	89.2	90.6	89.8		
Rivo Alto Canal - GRASS			91.3	88.8	90.2	89.4	88.4	87.5
			S7 87.5					
Napels Canal - The Colonade								

Rivo Alto Canal - The Toledo Bridge - EAST								
Rivo Alto Canal - #168			88.7 IN	89.0 IN	89.8 IN	89.5 OUT	89.9 IN	89.5 IN
Rivo Alto Canal - #170			88.7 IN	88.4 IN	88.5 IN			
Rivo Alto Canal - #172		IN .8 DEGREES	87.9 IN	88.6 IN	88.8 OUT	89.0 IN		
Rivo Alto Canal - #182			89.1 OUT	89.3 IN	89.5 IN	89.7 OUT	89.1 IN	89.0 IN
Rivo Alto Canal - #184				89.6 IN		89.2 IN		89.3 IN
Rivo Alto Canal - #188			89.1 IN	88.7 IN	89.6 IN			
Rivo Alto Canal - #190			88.9 IN	89.5 IN	89.4 OUT			
Rivo Alto Canal - #192		IN .9 DEGREES	89.0 OUT		87.3 IN		88.7 IN	
Rivo Alto Canal - #194			90	89.4 IN	89.3 IN	88.6 IN		
Rivo Alto Canal - #198			88.6 IN	88.7 IN	88.6 IN			
Rivo Alto Canal - #200			88.3 IN	88.8 IN	88.4 IN			
Rivo Alto Canal - #202		IN .7 DEGREES	88.3 IN	88.7 IN	87.5 IN			
Rivo Alto Canal - #206			89.1 IN	89.4 IN	89.2 IN			
Rivo Alto Canal - #208		IN .8 DEGREES	87.8 IN	89.2 IN	89.2 IN			
Rivo Alto Canal - #210			88.2 IN	89.1 IN	89.6 IN	89.1 IN		
Napels Canal - Neopolitan Lane - EAST								
Rivo Alto Canal - #212			89.1 IN	89.0 IN	89.2 IN	89.3 IN		
Rivo Alto Canal - #216			90	88.8 IN				
Rivo Alto Canal - #218			88.6 IN	90	89.5 OUT	89.5 OUT		
Napels Canal - ALLEY			88.7 IN					
Rivo Alto Canal - #220			89.8 OUT	89.6 OUT	88.7 IN	89.4 IN		
Rivo Alto Canal - GRASS			88.9 IN	89.4 OUT	89.1 IN	90	89.1 OUT	88.2 IN
			S7 88.2 OUT					
Napels Canal - The Colonade								

FIGURE 3-2d

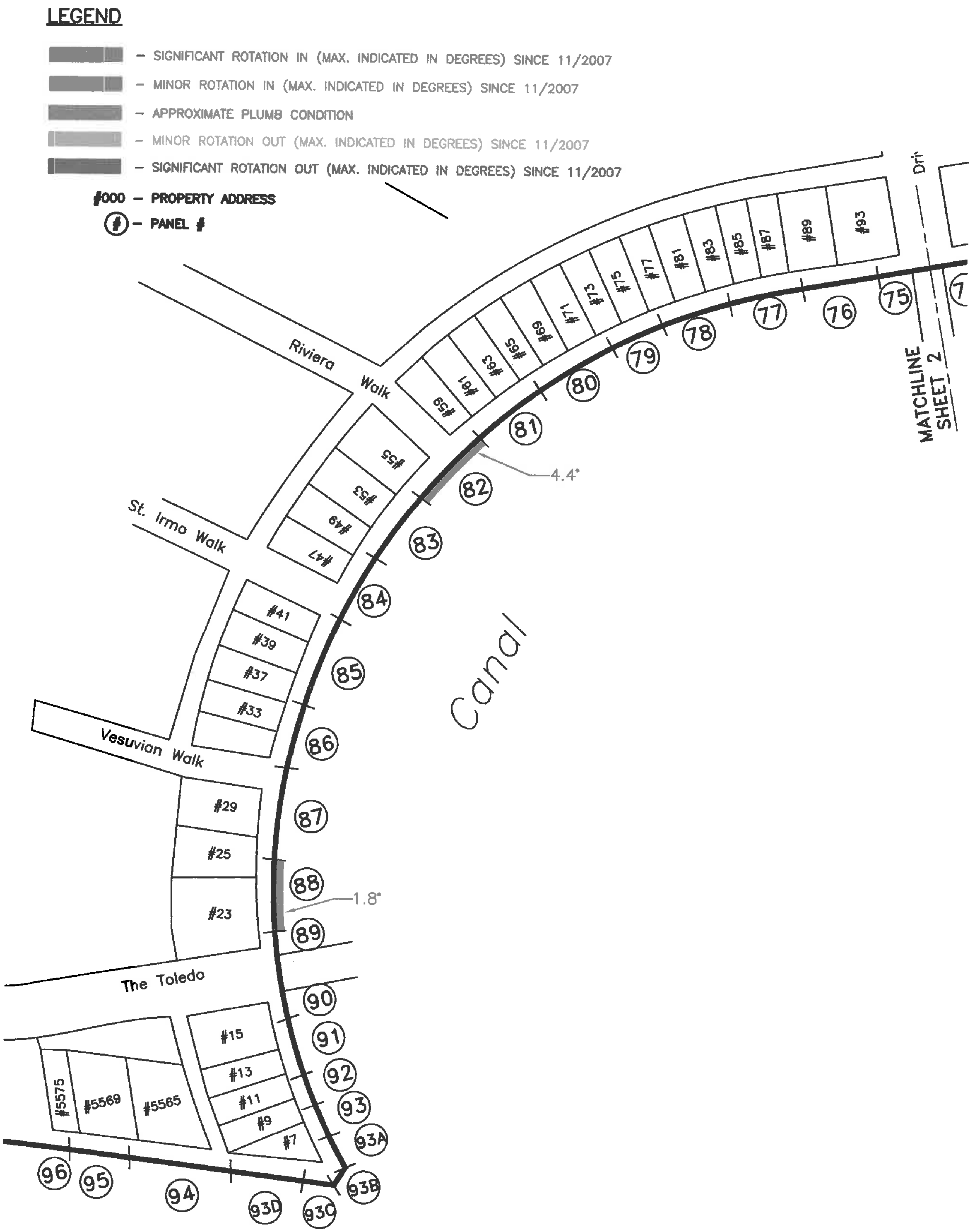


FIGURE 3–3a RESULTS OF WALL ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS NAPLES LANDSIDE

SURVEY DATE: 8/2008

SHEET 1 OF 6

LEGEND

- SIGNIFICANT ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- MINOR ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- APPROXIMATE PLUMB CONDITION
- MINOR ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007
- SIGNIFICANT ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007

#000 – PROPERTY ADDRESS

(#) – PANEL #

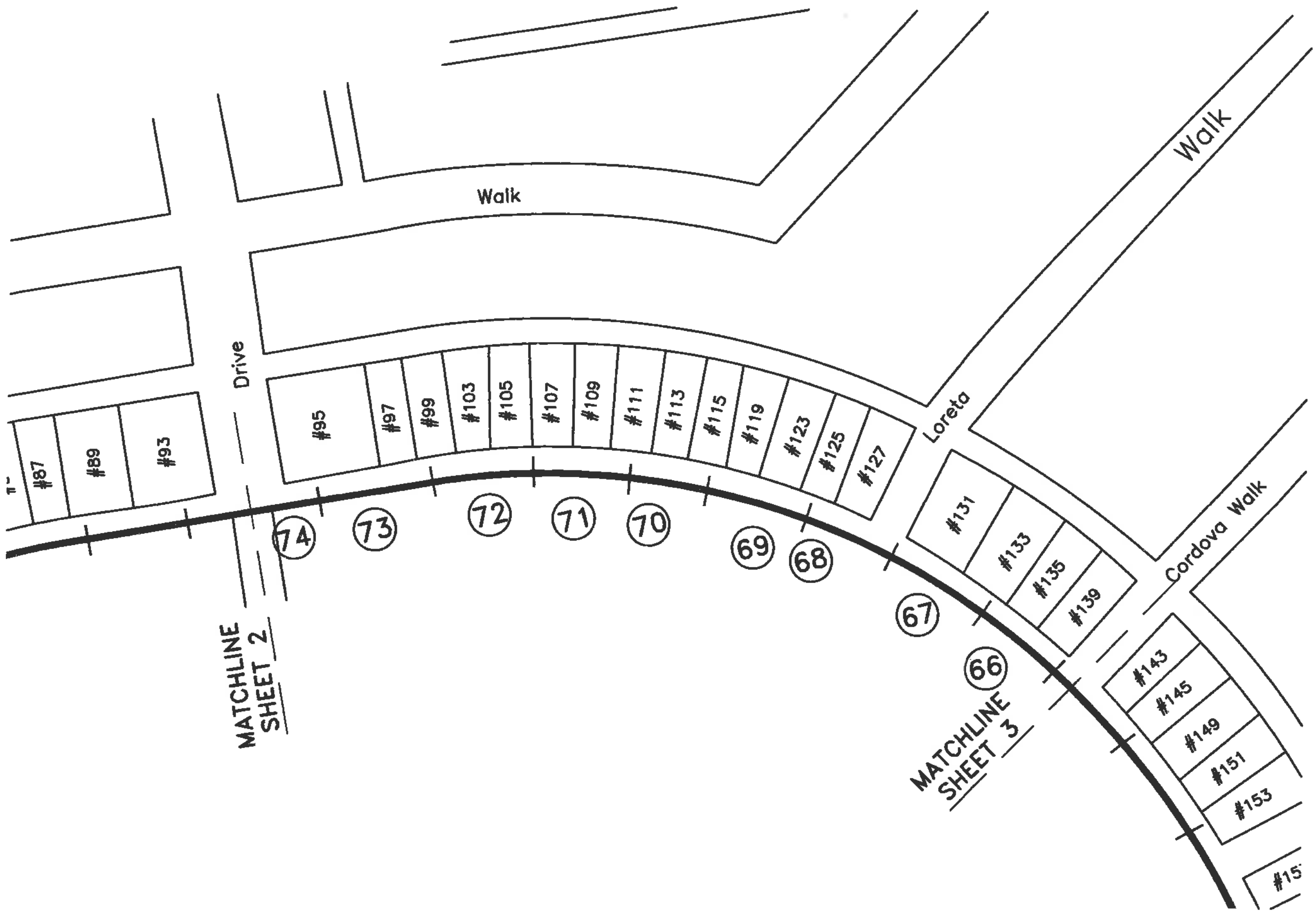


FIGURE 3–3b NAPLES LANDSIDE RESULTS OF WALL ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 8/2008

SHEET 2 OF 6

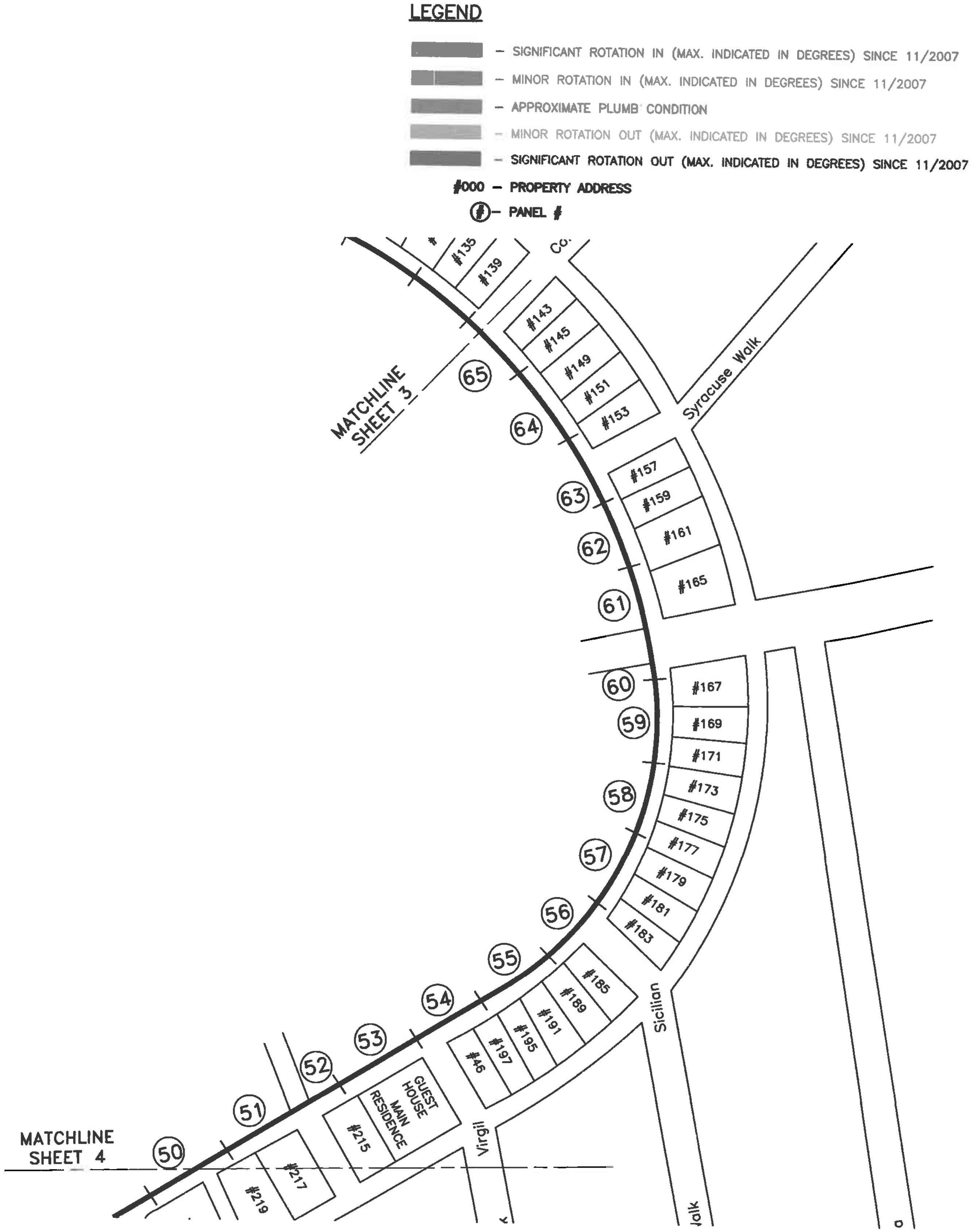


FIGURE 3–3c NAPLES LANDSIDE RESULTS OF WALL ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 8/2008

SHEET 3 OF 6

LEGEND

- SIGNIFICANT ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- MINOR ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- APPROXIMATE PLUMB CONDITION
- MINOR ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007
- SIGNIFICANT ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007

#000 – PROPERTY ADDRESS

(#) – PANEL #

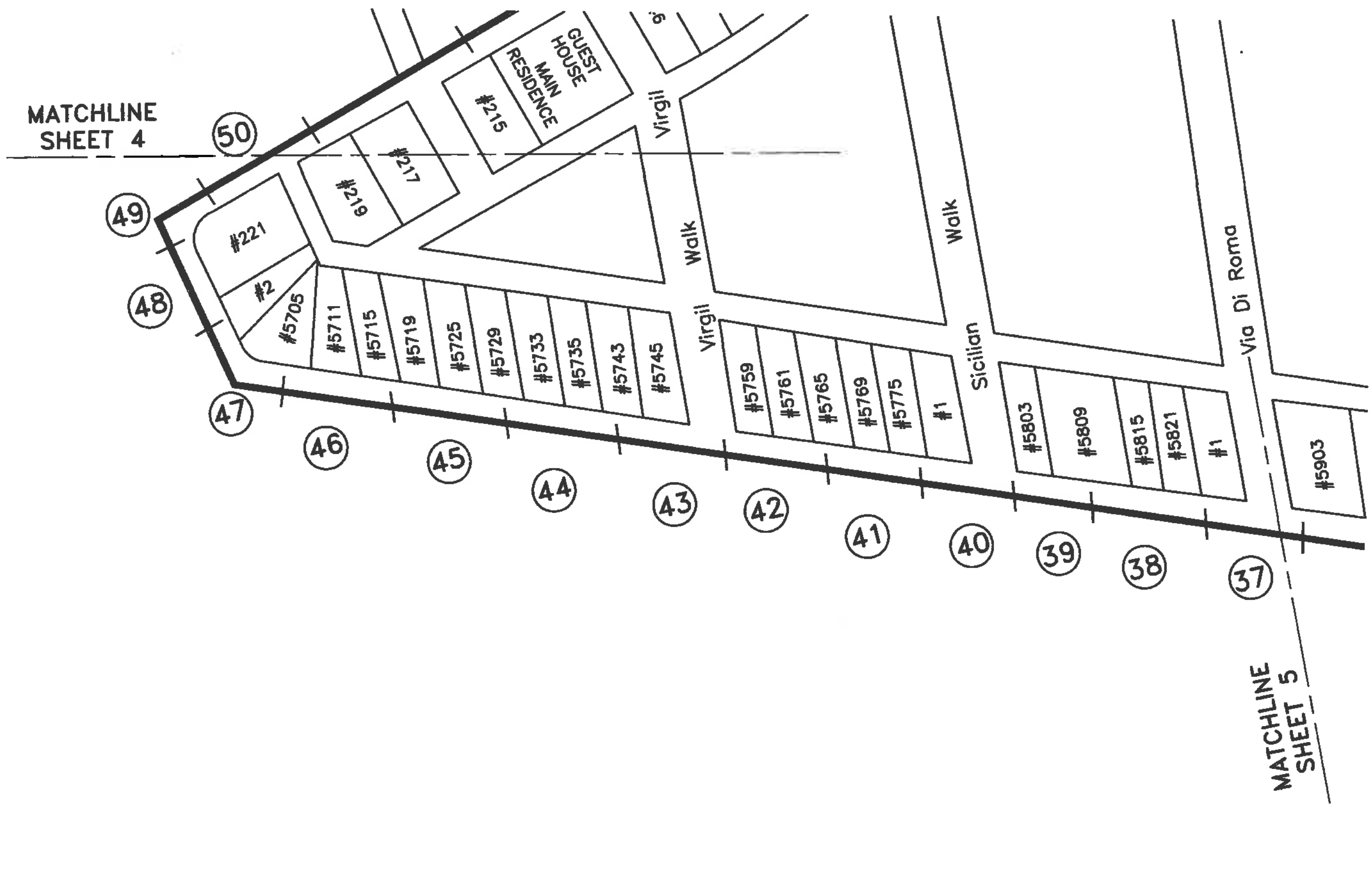


FIGURE 3-3d NAPLES LANDSIDE
RESULTS OF WALL ROTATION ANALYSIS
FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 8/2008

SHEET 4 OF 6

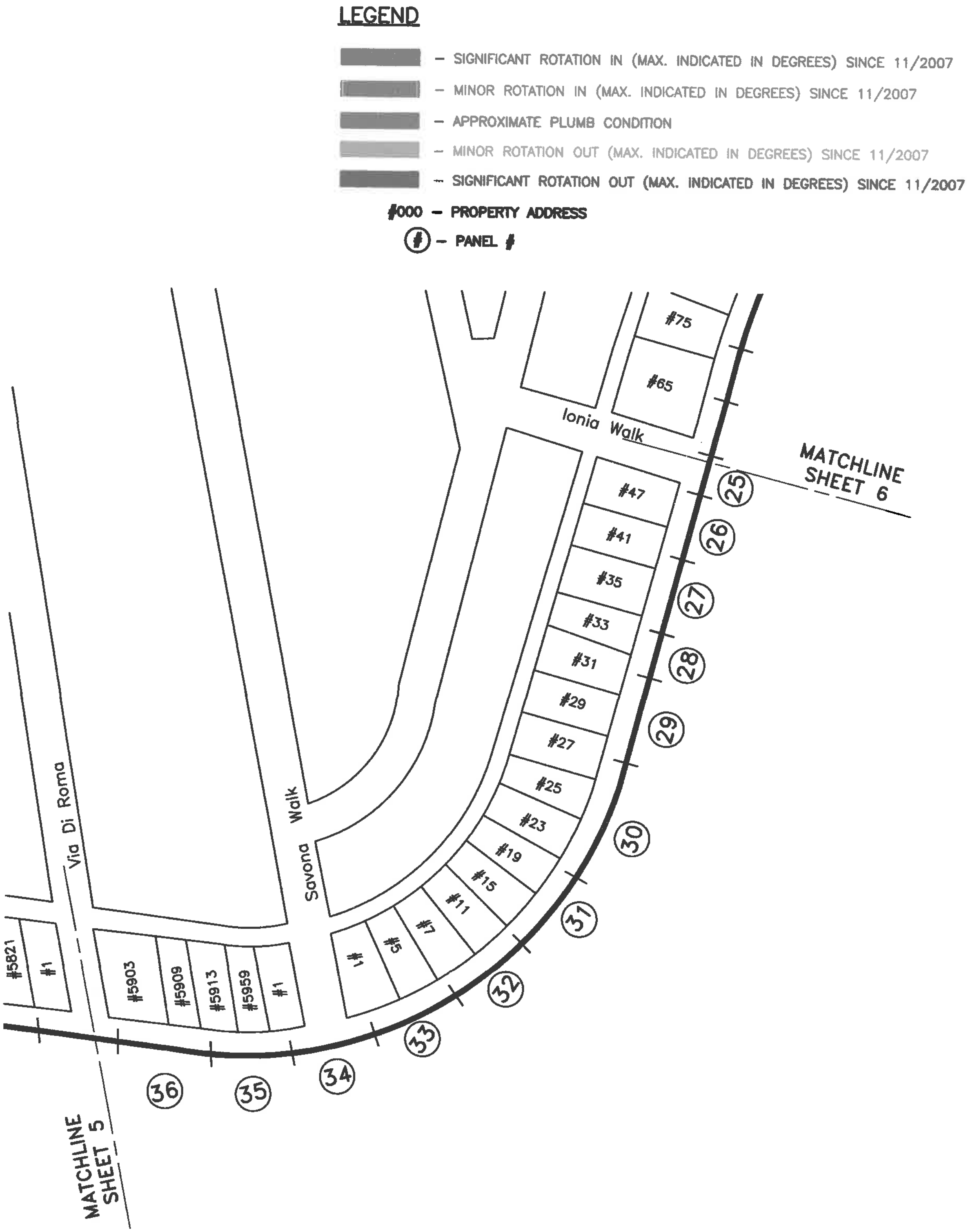


FIGURE 3–3e NAPLES LANDSIDE
RESULTS OF WALL ROTATION ANALYSIS
FROM SURVEY OF GUARDRAIL POSTS
SURVEY DATE: 8/2008

SHEET 5 OF 6

LEGEND

- SIGNIFICANT ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- MINOR ROTATION IN (MAX. INDICATED IN DEGREES) SINCE 11/2007
- APPROXIMATE PLUMB CONDITION
- MINOR ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007
- SIGNIFICANT ROTATION OUT (MAX. INDICATED IN DEGREES) SINCE 11/2007

#000 – PROPERTY ADDRESS

(#) – PANEL #

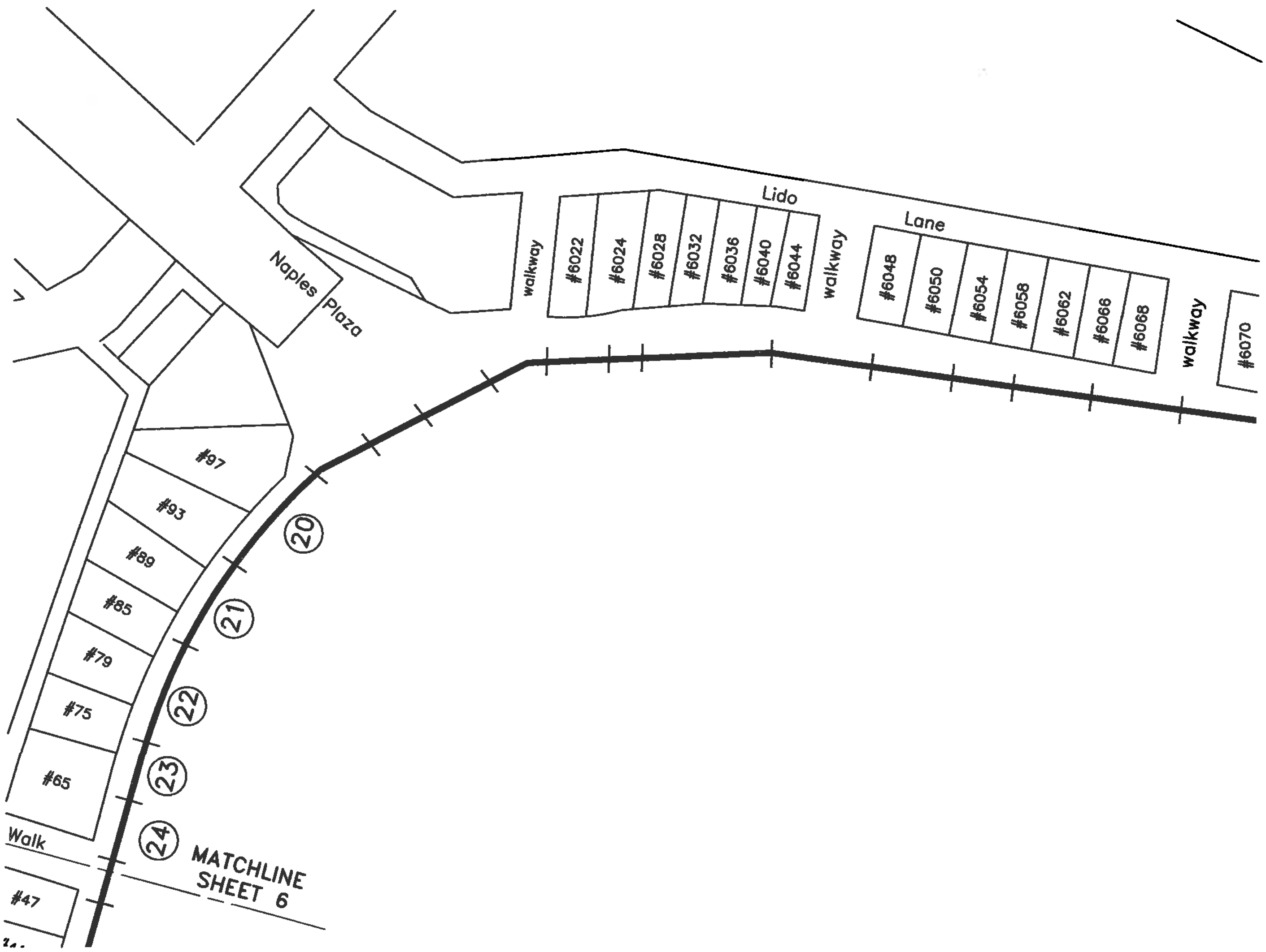


FIGURE 3–3f NAPLES LANDSIDE RESULTS OF WALL ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 8/2008

SHEET 6 OF 6

Naples Landside - Naples Canal

11/2007 SEAWALL SURVEY

Address		S1	S2	S3	S4	S5	S6
Napels Canal - End of Public Access							
Napels Canal - #5565		89.1	89.9	90.6	90.1		
Napels Canal - #5569		89.3	90.8	90.1	90.9	89.5	88.8
Napels Canal - #5575		90.6	90.4	90.2	91.1	91.6	

Address		S1	S2	S3	S4	S5	S6
Napels Canal - Girabaldi Lane		90.3					
Napels Canal - #7		90.1	~~89.3~~	~~89.8~~	90.5	90.1	90.1
	* SUSPECT	S7 90.3	S8 92.4	S9 88.8 *	S10 90	S11 89.8	S12 89.5

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal							
Rivo Alto Canal - #7		90.6	91.2	90.1	89.8	91	
Rivo Alto Canal - #9		~~89.8~~	90.3	90.6			
Rivo Alto Canal - #11		88.2	89.5				
Rivo Alto Canal - #13		91.2	89.3				
Rivo Alto Canal - #15		89.5	90.3	90.6	89.8	91.3	89.4
		S7 90.8	SPALL				

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - WEST							
Rivo Alto Canal - #21		89.7	90	91	90.5	91	
Rivo Alto Canal - #23		90.8	90.3	89.6			
Rivo Alto Canal - #25		88.6	90.7	89.9	90.4		
Rivo Alto Canal - #29		89.4	90.9	89.1	90.9		

Naples Landside - Naples Canal

11/2008 SEAWALL SURVEY

Address		S1	S2	S3	S4	S5	S6
Napels Canal - End of Public Access							
Napels Canal - #5565		89.7 OUT	89.4 IN		89.1 IN		
Napels Canal - #5569		89.9 OUT	88.4 IN	89.2 IN	88.3 IN	89.8 IN	89.4 OUT
Napels Canal - #5575	IN 0.8 DEGREES	88.7 IN	89.0 OUT	89.1 IN	88.1 IN	87.6 IN	

Address		S1	S2	S3	S4	S5	S6
Napels Canal - Girabaldi Lane		88.9 IN					
Napels Canal - #7		89.1 IN	88.9 IN	88.6 IN	88.9 IN	89.2 IN	89.2 IN
	IN 0.8 DEGREES C = CORNER	S7 89.0 IN	S8 86.8 IN	S9 88.1 IN	S10 89.3 IN	S11 89.8 IN ←C	S12 90 C→

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal							
Rivo Alto Canal - #7		88.6 IN	88.2 IN	89.2 IN	89.5 IN	88.3 IN	
Rivo Alto Canal - #9		HEDGE	HEDGE	HEDGE			
Rivo Alto Canal - #11		89.0 OUT	89.8 IN				
Rivo Alto Canal - #13		88.0 IN	89.7 OUT				
Rivo Alto Canal - #15		89.8 IN	89.0 IN	88.6 IN	89.5 IN	88.1 IN	90
		S7 88.4 IN					

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - WEST							
Rivo Alto Canal - #23	IN 1.8 DEGREES	89.7 IN	89.4 IN	89.0 IN	87.7 IN	89.6 IN	
	IN 1.3 DEGREES	87.9 IN	88.7 IN	89.5 OUT			
Rivo Alto Canal - #25		88.4 IN	89.3 IN	88.8 IN			
Rivo Alto Canal - #29		90	88.3 IN	89.9 IN	88.4 IN		

FIGURE 3-4a

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Vesuvian Walk		90.3	91.6	89			
Rivo Alto Canal - #31		90.8	90.2				
Rivo Alto Canal - #33		90.5	91.1	89.4			
Rivo Alto Canal - #37		90.8	90.7	90.9			
Rivo Alto Canal - #39		90.9	90.5	91.2			
Rivo Alto Canal - #41		90.5	90.2	89			

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - St. Irmo Walk		90.7	91.1	91.2			
Rivo Alto Canal - #47		90.4	89.4	89.1			
Rivo Alto Canal - #49		89.7	90.7	90.5			
Rivo Alto Canal - #53		89	90.5	90.3	90.1		
Rivo Alto Canal - #55	* SUSPECT	91.7	88.9 *	91.2	90.8		

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Riviera Walk		92.3	90.4				
Rivo Alto Canal - #59		89.6	90.6	90.8			
Rivo Alto Canal - #61		90.8	91.5	88.9			
Rivo Alto Canal - #63		90.3	90.6	90			
Rivo Alto Canal - #65		91.3	89	91.2			
Rivo Alto Canal - #69		88	90.2				
Rivo Alto Canal - #71		91.7	89.4	90.3			
Rivo Alto Canal - #73		90.2	90.5	89.3			

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Vesuvian Walk		89.1 IN	88.1 IN	89.3 IN			
Rivo Alto Canal - #31		88.5 IN	89.1 IN				
Rivo Alto Canal - #33		89.1 IN	88.1 IN	90			
Rivo Alto Canal - #37		88.5 IN	88.4 IN	88.3 IN			
Rivo Alto Canal - #39		88.6 IN	88.7 IN	88.1 IN			
Rivo Alto Canal - #41		89.2 IN	HEDGE	89.3 OUT			

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - St. Irmo Walk		88.9 IN	88.1 IN	88.1 IN			
Rivo Alto Canal - #47		88.8 IN	90	89.2 OUT			
Rivo Alto Canal - #49		89.2 IN	88.1 IN	88.8 IN			
Rivo Alto Canal - #53		89.7 OUT	89.2 IN	89 IN	88.5 IN		
Rivo Alto Canal - #55	IN 0.7 DEGREES	87.6 IN	89.8 IN	88 IN	88.5 IN		

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Riviera Walk	IN 4.4 DEGREES	87.0 IN	85.2 IN				
Rivo Alto Canal - #59	NEW LOCATIONS	89.9 IN	89.2 IN	88.9 IN			
Rivo Alto Canal - #61	NEW LOCATIONS IN 0.7 DEGREES	88.8 IN	87.8 IN	89.5 OUT			
Rivo Alto Canal - #63		89.0 IN	89.5 -BOLT- IN	89.2 -BOLT- IN			
Rivo Alto Canal - #65		88.0 IN	89.0 OUT	88.0 IN			
Rivo Alto Canal - #69	NEW RAILS	88.5 IN	88.0 -BOLT- IN	88.6 -BOLT- IN			
Rivo Alto Canal - #71	IN 0.6 DEGREES	87.7 IN	90	89.0 IN			
Rivo Alto Canal - #73		89.3 IN	89.0 IN	89.8 OUT			

FIGURE 3-4b

			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #75			91.3	91.5	90.9	89.2		
Rivo Alto Canal - #77			90.6	90.4	90.8	90.9		
Rivo Alto Canal - #81			88.5	91.1	89.8			
Rivo Alto Canal - #83			90.5	90	89.5			
Rivo Alto Canal - #85			90.1	90.1	90.6			
Rivo Alto Canal - #87			91	90.6	90.2			
Rivo Alto Canal - #89			90.5	90.1	91.7			
Rivo Alto Canal - #93			90.9	87.2	89.8	90.7	90.4	
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Ravenna Bridge								
Rivo Alto Canal - #95			89.8	89.8	91.3	89.7	89.9	
Rivo Alto Canal - #97			90.1	88.2	88.7			
Rivo Alto Canal - #99			91	89.5	90.2			
Rivo Alto Canal - #103			89.5	89.4	90.9			
Rivo Alto Canal - #105			90.7	90.4	90.3			
Rivo Alto Canal - #107			89.2	90.4	91.4			
Rivo Alto Canal - #109			89.9	90.4	90.1			
Rivo Alto Canal - #111			90.5	90.8	90.7			
Rivo Alto Canal - #113			90.9	90.8	89.6			

			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #75			88.1 IN	88.0 IN	88.5 IN	89.7 OUT		
Rivo Alto Canal - #77			88.8 IN	89.0 IN	88.7 IN	88.5 IN		
Rivo Alto Canal - #81			89.1 IN	88.3 IN	89.6 OUT			
Rivo Alto Canal - #83			88.9 IN	89.7 IN				
Rivo Alto Canal - #85			89.8 OUT	89.3 IN	89.4 IN	88.8 IN		
Rivo Alto Canal - #87			88.3 IN	88.8 IN	89.2 IN			
Rivo Alto Canal - #89		IN 0.8 DEGREES	88.9 IN	89.4 IN	87.5 IN			
Rivo Alto Canal - #93		IN 0.6 DEGREES	88.5 IN	87.7 OUT	89.6 IN	88.6 IN	89.0 IN	
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Ravenna Bridge								
Rivo Alto Canal - #95			89.8 IN	89.8 IN	88.3 IN	89.7 IN	89.6 IN	
Rivo Alto Canal - #97		IN 0.5 DEGREES	89.4 IN	88.7 OUT	89.2 OUT			
Rivo Alto Canal - #99			88.5 IN	90	89.2 IN			
Rivo Alto Canal - #103			90	90	88.5 IN			
Rivo Alto Canal - #105			88.7 IN	89.0 IN	89.4 IN			
Rivo Alto Canal - #107			89.8 OUT	89.0 IN	88.1 IN			
Rivo Alto Canal - #109			89.6 IN	89.1 IN	89.5 IN			
Rivo Alto Canal - #111			88.9 IN	89.7 IN				
Rivo Alto Canal - #113			88.7 IN	88.5 IN	88.6 IN	89.8 IN		

FIGURE 3-4c

Address			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #115			89.5	90.5	88	90.9		
Rivo Alto Canal - #119			89.6	89.4	90.5	89.6		
Rivo Alto Canal - #123			90	90.6				
Rivo Alto Canal - #125			90.6	89.8	91.1			
Rivo Alto Canal - #127			89.6	89.4	89.7			
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Loreta Walk			89.8	91.8				
Rivo Alto Canal - #131			90.3	88.6	90.1	89		
Rivo Alto Canal - #133			91.3	90.8	91.7	90.7		
Rivo Alto Canal - #135			91.6	90	90.2	90.2		
Rivo Alto Canal - #139			91.9	91.6	91.2			
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Cordova Walk			89.5	90.3				
Rivo Alto Canal - #143			90.7	90.6	91			
Rivo Alto Canal - #145			90.6	91.6	90.6			
Rivo Alto Canal - #149			91.2	88.3	90			
Rivo Alto Canal - #151			90.6	91.4	90			
Rivo Alto Canal - #153			89.8	89.9				

Address			S1	S2	S3	S4	S5	S6
Rivo Alto Canal - #115			90	88.9 IN	88.5 IN	88.5 IN		
Rivo Alto Canal - #119			90	89.7 OUT	88.9 IN	89.7 IN		
Rivo Alto Canal - #123			89.4 IN	88.6 IN				
Rivo Alto Canal - #125			88.8 IN	89.5 IN	88.2 IN			
Rivo Alto Canal - #127			90	90	89.2 OUT			
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Loreta Walk IN 0.7 DEGREES			89.5 IN	87.5 IN				
Rivo Alto Canal - #131			89.2 IN	89.1 OUT	89.3 IN	89.8 OUT		
Rivo Alto Canal - #133		IN 0.7 DEGREES	88.1 IN	88.5 IN	87.6 IN	88.6 IN		
Rivo Alto Canal - #135			89.3 -BOLT- IN	89.3 IN	89.2 IN	89.4 -BOLT- IN		
Rivo Alto Canal - #139		IN 0.7 DEGREES	87.4 IN	88.0 IN	88.2 IN			
Address			**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Rivo Alto Canal - Cordova Walk			90	89.3 IN				
Rivo Alto Canal - #143			88.6 IN	88.8 IN	89.3 IN			
Rivo Alto Canal - #145		IN 0.7 DEGREES	88.8 IN	87.7 IN	89.0 IN			
Rivo Alto Canal - #149			88.1 IN	89.0 OUT	89.4 IN			
Rivo Alto Canal - #151		IN 0.7 DEGREES	88.8 IN	87.9 IN	89.4 IN			
Rivo Alto Canal - #153			89.6 IN	89.5 IN				

FIGURE 3-4d

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Syracuse Walk		89.6	88.9				
Rivo Alto Canal - #157		88.8	87.4	91.8			
Rivo Alto Canal - #159		90.3	89.2	90.9	90		
Rivo Alto Canal - #161		87.7	89.5	89.3	90		
Rivo Alto Canal - #165		90.5	89.3	89.4	90.6	89.9	

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - EAST							
Rivo Alto Canal - #167		88.9	91.9	90.5	92.2		
Rivo Alto Canal - #169		92	90.5	91.2			
Rivo Alto Canal - #171		91.1	91.4	90.4			
Rivo Alto Canal - #173		89.9	90.7	89.9			
Rivo Alto Canal - #175		90.8	89.6	90.8			
Rivo Alto Canal - #177		90	90.2				
Rivo Alto Canal - #179		91.5	90.8	90.4			
Rivo Alto Canal - #181		89.9	90.3	89.5			
Rivo Alto Canal - #183		90	91				

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Syracuse Walk		89.7 IN	89.5 IN				
Rivo Alto Canal - #157	IN 0.8 DEGREES	89.4 OUT	88.1 OUT	87.4 IN			
Rivo Alto Canal - #159		89.1 IN	89.8 OUT	88.7 IN	89.3 IN		
Rivo Alto Canal - #161	IN 0.7 DEGREES	88.3 OUT	89.6 IN	90	89.3 IN		
Rivo Alto Canal - #165		89.0 IN	89.9 OUT	90	88.7 IN	89.4 IN	

Address		S1	S2	S3	S4	S5	S6
Rivo Alto Canal - The Toledo Bridge - EAST							
Rivo Alto Canal - #167	IN 0.7 DEGREES	89.1 OUT	87.5 IN	88.9 IN	87.1 IN		
Rivo Alto Canal - #169	IN 0.7 DEGREES	87.4 IN	88.8 IN	88.2 IN			
Rivo Alto Canal - #171		88.3 IN	88.0 IN	89.0 IN			
Rivo Alto Canal - #173		89.5 IN	88.5 IN	89.5 IN			
Rivo Alto Canal - #175		88.6 IN	89.7 IN	88.5 IN			
Rivo Alto Canal - #177		89.4 IN	89.2 IN				
Rivo Alto Canal - #179	IN 0.7 DEGREES	87.9 IN	88.5 IN	89.0 IN			
Rivo Alto Canal - #181		89.5 IN	89.1 IN	90			
Rivo Alto Canal - #183		89.4 IN	88.4 IN				

FIGURE 3-4e

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Sicilian Walk	91.4	90.7				
Rivo Alto Canal - #185	89.6	90.4	89.2	90.8		
Rivo Alto Canal - #189	90.4	90.4				
Rivo Alto Canal - #191	90.7	90.2	90.8			
Rivo Alto Canal - #195	89.7	90.1	89.9			
Rivo Alto Canal - #197	88.7	90.1				
Rivo Alto Canal - #46	89.9	89.4	89.5			

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Virgil Walk	88	89.8	90.9			
Rivo Alto Canal - GUEST HOUSE						
Rivo Alto Canal - MAIN RESIDENCE	89.5	90.9	91	91.3	90.8	90.3
Rivo Alto Canal - #215	89.5	89.8	91.3	89.7		

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Neopolitan Lane - EAST						
Rivo Alto Canal - #217	89.2	89.7	89.7	90.3	90.6	
Rivo Alto Canal - #219	90.5	90.2	90.1	89.6		

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Alley	88.3					
Rivo Alto Canal - #221	88.6	90.3	90.4	89.7	90.2	90.1
	S7 90.1	S8 89.3	S9 90.9			

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Sicilian Walk	88.0 IN	88.7 IN				
Rivo Alto Canal - #185	90	89.0 IN	89.8 OUT	88.6 IN		
Rivo Alto Canal - #189	89.0 IN	88.9 IN				
Rivo Alto Canal - #191	88.7 IN	89.2 IN	88.6 IN			
Rivo Alto Canal - #195	89.7 IN	89.3 IN	89.5 IN			
Rivo Alto Canal - #197	89.3 OUT	89.3 -BOLT- IN				
Rivo Alto Canal - #46	89.5 IN	89.9 IN	90			

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Virgil Walk	88.6 IN	89.5 IN	88.5 IN			
Rivo Alto Canal - GUEST HOUSE	90	88.5 IN	88.4 IN			
Rivo Alto Canal - MAIN RESIDENCE	88.1 IN	88.6 IN	89.0 IN			
Rivo Alto Canal - #215	89.9 IN	89.5 IN	88.1 IN	89.6 IN		

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Neopolitan Lane - EAST						
Rivo Alto Canal - #217	89.6 OUT	89.7 IN	89.7 IN	89.1 IN	88.8 IN	
Rivo Alto Canal - #219	88.8 IN	89.2 IN	89.2 IN	90		

Address	S1	S2	S3	S4	S5	S6
Rivo Alto Canal - Alley	88.0 OUT					
Rivo Alto Canal - #221	89.3 OUT	89.0 IN	89.0 IN	89.6 IN	89.1 IN	89.3 IN
	89.4 IN	90	88.5 IN	CORNER		

FIGURE 3-4f

Address			S1	S2	S3	S4	S5	S6
Colonnade Canal								
Colonnade Canal #221			89.9	89.3	90.4	89.4	90.3	90.5
			S7 88.3					
Colonnade Canal #2			89.3	89.7	90	90.6		
Colonnade Canal #5705			88.8	88.9				

Address			S1	S2	S3	S4	S5	S6
Alamitos Bay								
Alamitos Bay - #5705			91.1	89.2	89.6	90.5	90.3	91.5
Alamitos Bay - #5711			89.7	90.2	89.2	89.7		
Alamitos Bay - #5715			90.3	90.3	90.8			
Alamitos Bay - #5719			90.2	90.1				
Alamitos Bay - #5725			89.1	89.6	89.3			
Alamitos Bay - #5729			89.6	89.9	89.5			
Alamitos Bay - #5733			88.8	89.8	90.5			
Alamitos Bay - #5735			90.5	89.8	90.1			
Alamitos Bay - #5743			90.9	90.2	90.4			
Alamitos Bay - #5745			90.9	91.1	92.1			

Address			S1	S2	S3	S4	S5	S6
Colonnade Canal								
Colonnade Canal - #221			89.6 IN	89.8 OUT	89.1 IN	89.9 OUT	89.1 IN	88.9 IN
		IN 0.6 DEGREES	88.8 OUT					
Colonnade Canal - #2			89.5 OUT	89.7 IN	89.8 IN	89.0 IN		
Colonnade Canal - #5705			89.2 OUT	89.4 OUT	CORNER			

Address			S1	S2	S3	S4	S5	S6
Alamitos Bay								
Alamitos Bay - #5705		IN 0.6 DEGREES	88.3 IN	89.4 OUT	89.9 IN	88.9 IN	89.1 IN	87.9 IN
Alamitos Bay - #5711		MARKS DO NOT MATCH UP	89.6 IN	88.7 IN	89.9 IN	89.7 IN		
Alamitos Bay - #5715			89.8 IN	89.1 IN	89.0 IN			
Alamitos Bay - #5719			88.5 IN	89.1 IN	89.2 IN			
Alamitos Bay - #5725			89.7 OUT	89.7 IN	90			
Alamitos Bay - #5729			89.7 IN	89.6 IN	89.8 OUT			
Alamitos Bay - #5733			89.4 OUT	89.5 IN	88.7 IN			
Alamitos Bay - #5735			88.8 IN	89.5 IN	89.2 IN			
Alamitos Bay - #5743			88.3 IN	89.2 IN	89.0 IN			
Alamitos Bay - #5745		IN 0.7 DEGREES	88.4 IN	88.2 IN	87.2 IN			

FIGURE 3-4g

Address	S1	S2	S3	S4	S5	S6
Alamitos Bay - Virgil Walk	91.8	90.1	89.9			
Alamitos Bay - #5759	90	91.2	90.9			
Alamitos Bay - #5761	91.2	89.8	90.4			
Alamitos Bay - #5765	90.6	88.7	90.3			
Alamitos Bay - #5769	90.4	91.7				
Alamitos Bay - #5775	91.4	91.2	90.3			
Alamitos Bay - #1	91.4	89.8	89.7			

Address		S1	S2	S3	S4	S5	S6
Alamitos Bay - Virgil Walk	IN 0.8 DEGREES	87.4 IN	89.2 IN	89.4 IN			
Alamitos Bay - #5759		89.3 IN	SHRUBS	SHRUBS			
Alamitos Bay - #5761		88.5 IN	88.2 IN	89.6 IN	89.3 IN		
Alamitos Bay - #5765		88.8 IN	89.4 OUT	89.0 IN			
Alamitos Bay - #5769	IN 0.7 DEGREES	88.9 IN	87.6 IN				
Alamitos Bay - #5775		88.0 IN	88.1 IN	89.1 IN			
Alamitos Bay - #1		88.1 IN	89.5 IN	SHRUBS			

Address	S1	S2	S3	S4	S5	S6
Alamitos Bay - Sicilian Walk	90.5	89.9	90.4			
Alamitos Bay - #5803	90.5	90.3	89.7			
Alamitos Bay - #5809	90.5	91.3	89.9	91.4	90.1	
Alamitos Bay - #5815	91.2	90.2	90.5			
Alamitos Bay - #5821	90.8	90.3	92			
Alamitos Bay - #1	90.8	91.5	90.5			

Address		S1	S2	S3	S4	S5	S6
Alamitos Bay - Sicilian Walk		89.1 IN	89.5 IN	88.9 IN			
Alamitos Bay - #5803		88.8 IN	89.1 IN	89.6 IN			
Alamitos Bay - #5809		88.9 IN	88.1 IN	89.5 IN	88.0 IN	89.2 IN	
Alamitos Bay - #5815		88.2 IN	89.2 IN	88.8 IN			
Alamitos Bay - #5821		SHRUBS	SHRUBS	SHRUBS			
Alamitos Bay - #1	IN 0.7 DEGREES	88.6 IN	87.8 IN	89.0 IN			

FIGURE 3-4h

Address	S1	S2	S3	S4	S5	S6
Alamitos Bay - Via Di Roma Walk	90.5	89.9	90.3			
Alamitos Bay - #5903	89.8	90.6	90.7	91.3	91.1	90.6
	S7 90.2					
Alamitos Bay - #5909	90.2	90.2	90			
Alamitos Bay - #5913	89.4	90.1	90	88.3		
Alamitos Bay - #5959	89.1	92	89.1			
Alamitos Bay - #1	88.6	90.3	88.2			

Address		S1	S2	S3	S4	S5	S6
Alamitos Bay - Via Di Roma Walk		88.9 IN	89.4 IN	89.1 IN			
Alamitos Bay - #5903		88.6 IN	88.7 IN	88.6 IN	88.0 IN	89.2 IN	88.9 IN
		89.4 IN					
Alamitos Bay - #5909		89.1 IN	89.2 IN	89.4 IN			
Alamitos Bay - #5913		90	88.0 IN	89.4 IN	88.8 IN		
Alamitos Bay - #5959	IN 0.6 DEGREES	89.6 OUT	87.4 IN	89.7 OUT			
Alamitos Bay - #1		89.6 OUT	89.0 IN	90 -BOLT-			

Address	S1	S2	S3	S4	S5	S6
Alamitos Bay - Savona Walk	88.9	90.2	91			
Alamitos Bay - #1	89.7	90.6	89.9	90.9	89.5	89.4
Alamitos Bay - #5	88.4	89.3	88.3	90.2		
Alamitos Bay - #7	89.5	88	90			
Alamitos Bay - #11	91.2	90.4	89.9	89.7		
Alamitos Bay - #15	93.3	~~89.1~~	89.4			
Alamitos Bay - #19	88.9	89.4	90.2	91.1		
Alamitos Bay - #23	89.3	90.3	90.7	89.3		
Alamitos Bay - #25	90.8	~~88.8~~	91.7	91		
Alamitos Bay - #27	89.8	89.8	92.1			
Alamitos Bay - #29	92.8	88.6	89.9	90.5		
Alamitos Bay - #31	90.3	90.5	90.1			

Address		S1	S2	S3	S4	S5	S6
Alamitos Bay - Savona Walk		89.4 IN	89.2 IN	88.4 IN			
Alamitos Bay - #1		90	88.9 IN	89.5 IN	88.5 IN	90	90
Alamitos Bay - #5		89.0 OUT	89.9 OUT	89.9 OUT	89.3 IN		
Alamitos Bay - #7	IN 0.6 DEGREES	89.9 IN	88.5 OUT	89.4 IN			
Alamitos Bay - #11		88.3 IN	89.0 IN	89.6 IN	89.9 IN		
Alamitos Bay - #15		SHRUBS	SHRUBS	89.4 OUT			
Alamitos Bay - #19		89.5 OUT	90	89.2 IN	88.8 IN		
Alamitos Bay - #23		89.9 OUT	89.1 IN	88.6 IN	89.9 OUT		
Alamitos Bay - #25	IN 0.6 DEGREES	88.4 IN	89.5 (N) POST OUT	87.7 IN	88.4 IN		
Alamitos Bay - #27		89.6 IN	89.5 IN	87.3 IN			
Alamitos Bay - #29	IN 0.8 DEGREES	86.4 IN	89.1 OUT	89.2 IN	88.9 IN		
Alamitos Bay - #31		89.0 IN	88.8 IN	89.3 IN			

FIGURE 3-4i

Address			S1	S2	S3	S4	S5	S6
Alamitos Bay - #33			90.6	89.1	88.2	90		
Alamitos Bay - #35			89.1	88.8	89.5	89.3		
Alamitos Bay - #41			91.5	91.5	89.9	88.6		
Alamitos Bay - #47			91.3	88.9	88.3	88.2		
Alamitos Bay - Ionia Walk			89.6	89.7	89.7			
Alamitos Bay - #65			90.7	88.5	88.9	89.9	91.4	91.2
			S7 91.2	S8 89	S9 90.2	S10 90.6	S10 91.8	
Alamitos Bay - #75			90.2	89	91.7	89.3		
Alamitos Bay - #79			89.9	91.4	90.3	90.7		
Alamitos Bay - #85			91.3	90.4	90.1			
Alamitos Bay - #89			91.4	89.9	91.1	91.6		
Alamitos Bay - #93			91.4	91.2	89.6	91.1	91.2	
Alamitos Bay - #97			90.3	88.6	90.1			
Alamitos Bay - #99			89.6	90.2	88.9	90.4		

Address			S1	S2	S3	S4	S5	S6
Alamitos Bay - #33		IN 0.6 DEGREES	88.7 IN	89.4 OUT	88.5 OUT	89.4 IN		
Alamitos Bay - #35			89.7 OUT	89.4 OUT	90	90		
Alamitos Bay - #41		IN 0.7 DEGREES	87.8 IN	87.8 IN	89.4 IN	89.3 OUT		
Alamitos Bay - #47		IN 0.7 DEGREES	88.1 IN	89.5 OUT	88.8 OUT	88.7 OUT		
Alamitos Bay - Ionia Walk			89.8 IN	89.6 IN	89.6 IN			
Alamitos Bay - #65			88.6 IN	89.1 OUT	89.4 IN	89.9 IN	88.4 IN	
		IN 0.6 DEGREES	S7 89.3 IN		S9 88.8 IN		S11 87.6 IN	
Alamitos Bay - #75		IN 0.7 DEGREES	89.2 IN	89.5 OUT	87.6 IN	90		
Alamitos Bay - #79		IN 0.7 DEGREES	89.5 IN	87.9 IN	89.0 IN	88.6 IN		
Alamitos Bay - #85			88.1 IN	89.1 IN	89.2 IN			
Alamitos Bay - #89		IN 0.7 DEGREES	88.0 IN	89.5 IN	88.2 IN	87.7 IN	88.0 IN	
Alamitos Bay - #93			88.0 IN	89.5 IN	88.6 IN	88.2 IN		
Alamitos Bay - #97			89.0 IN	89.2 OUT	89.2 IN			
Alamitos Bay - #99			89.8 IN	89.2 IN	89.5 IN	89.0 IN		

FIGURE 3-4j

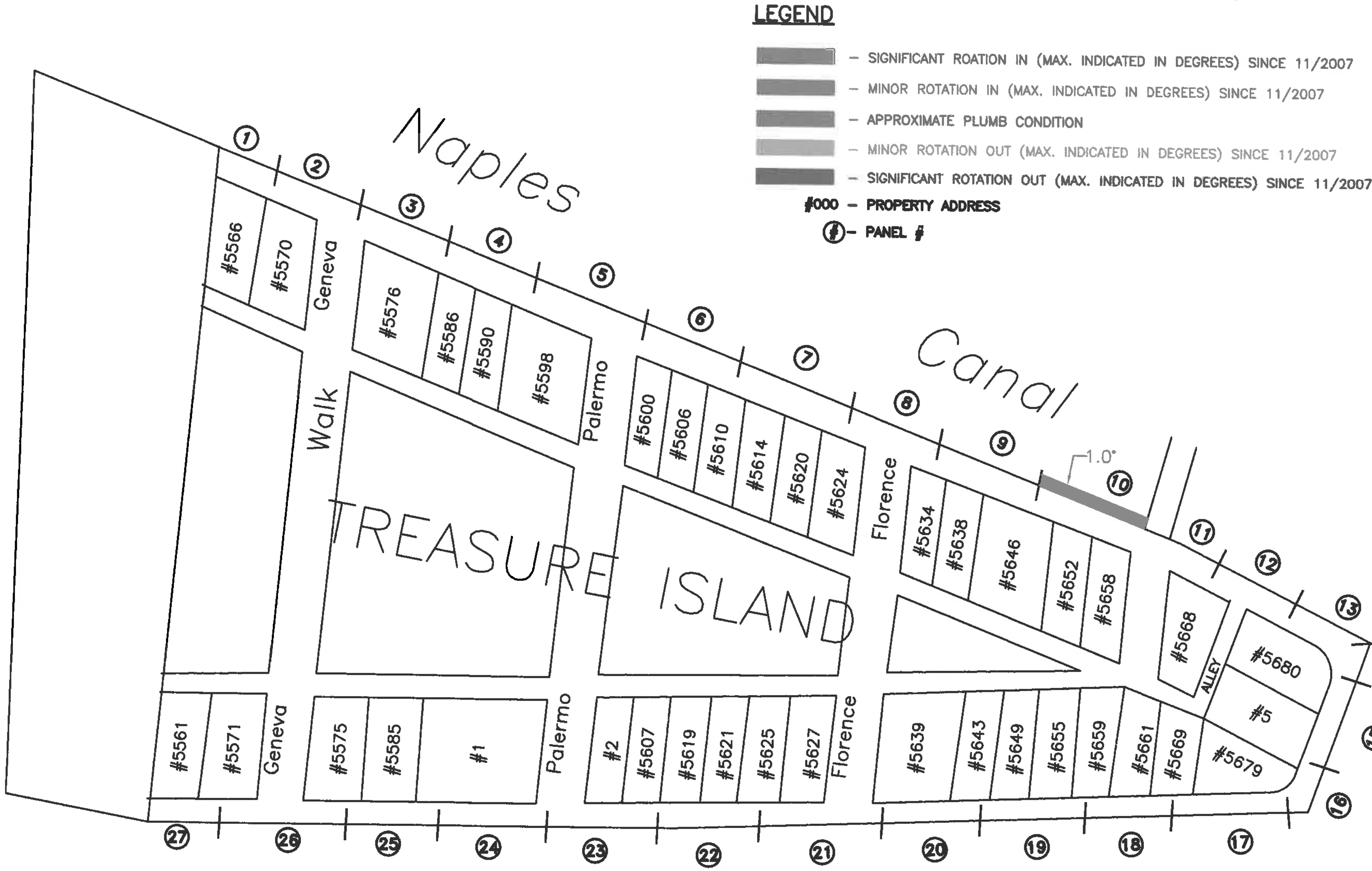


FIGURE 3–5 NAPLES – TREASURE ISLAND RESULTS OF WALL ROTATION ANALYSIS FROM SURVEY OF GUARDRAIL POSTS

SURVEY DATE: 6/2008

Treasure Island - Naples Canal

TREASURE ISLAND 11/2007 SEAWALL SURVEY

Address			S1	S2	S3	S4	S5	S6
Napels Canal - End of Public Access								
Napels Canal - #5566			90.4	90.8	91.8	90.5		
Napels Canal - #5570			90.3	90.7	90.5	89.5		
Napels Canal - Geneva Walk			91.1	89.8	90.2			
Napels Canal - #5576			90.3	90.2	89.8	89.8	90.1	89.3
Napels Canal - #5586			90.6	89.7	89.9			
Napels Canal - #5590			90.8	90.1	89.6	91.4	SPALL	
Napels Canal - #5598			89.1	91.1	90	90.5	SPALL	
Napels Canal - Palermo Walk			88.5	89.5	91.1		SPALL	
Napels Canal - #5600			90.9	90.8	89.2			
Napels Canal - #5606			89.2	89.7	89.7			
Napels Canal - #5610			90.8	89.6	89.9			
Napels Canal - #5614			90.2	89.6	90.3			
Napels Canal - #5620			89.2	89.9	89.2			
Napels Canal - #5624			90.3	89.7	90.1			
Napels Canal - Florrence Walk			89.5	89	88.5			
Napels Canal - #5634			89.7	89.4	90.5			
Napels Canal - #5638			89.2	89.2	89.9			
Napels Canal - #5646			88.4	89.2	89	90.6	91.4	
Napels Canal - #5652			90.5	87.9	89.8	88.8		
Napels Canal - #5658			89.5	89.8	89.9			

Treasure Island - Naples Canal

TREASURE ISLAND 08/2008 SEAWALL SURVEY

Address			S1	S2	S3	S4	S5	S6
Napels Canal - End of Public Access								
Napels Canal - #5566	IN 0.8 DEGREES		89.1 IN	88.4 IN	87.4 IN	88.8 IN		
Napels Canal - #5570			89.1 IN	88.7 IN	88.7 IN	89.5 IN		
Napels Canal - Geneva Walk			88.4 IN	89.6 IN	89.1 IN			
Napels Canal - #5576			89.1 IN	89.2 IN	89.5 IN	89.5 IN	88.2 IN	90
Napels Canal - #5586			88.8 IN	89.6 IN	89.5 IN			
Napels Canal - #5590			88.5 IN	89.2 IN	89.7 IN	88 IN		
Napels Canal - #5598			89.8 OUT	88.2 IN	89.4 IN	89.7 OUT		
Napels Canal - Palermo Walk			89.7 IN	89.8 IN	88.2 IN			
Napels Canal - #5600			88.4 IN	88.5 IN	89.8 IN			
Napels Canal - #5606				89.7 IN	89.5 IN			
Napels Canal - #5610			89.4 IN	89.8 IN	89.4 IN			
Napels Canal - #5614			89 IN	89.7 IN	89.1 IN			
Napels Canal - #5620			90	89.3 IN	90			
Napels Canal - #5624			89 IN	89.8 IN	89.2 IN			
Napels Canal - Florrence Walk			89.9 OUT	89.6 OUT	89.2 OUT			
Napels Canal - #5634			89.5 IN	89.8 IN	88.6 IN			
Napels Canal - #5638			90	89.3 IN	90			
Napels Canal - #5646	IN 1.0 DEGREES		89.2 OUT	88.9 OUT	89.7 OUT	88.5 IN	87.6 IN	
Napels Canal - #5652			88.8 IN	88.7 OUT	89.5 IN			
Napels Canal - #5658			89.5 OUT	89.7 IN	89.5 IN			

TREASURE ISLAND 11/2007 SEAWALL SURVEY

Napels Canal - W Neapolitan Ln Bridge

Napels Canal - #5668			89.2	90.4	90.4	89.7	90.2	90.4
Napels Canal - #5680			90.4	90.4	89.2	90.7	88.7	90.5
			S7 90.2	S8 89.1	S9 90	S10 89.4	S11 90.2	SPALL

Treasure Island - Colonade Canal (NOTE: CHANGE IN INSPECTION DIRECTION)

Address			S1	S2	S3	S4	S5	S6
Colonade Canal - #5680			88.9	90.8	90.4	89.9	90.5	
Colonade Canal - #5			91.2	89.7	88.2	90.2	90.5	
Colonade Canal - #5679			90.4	89.1	90.3			

Treasure Island - Alamitos Bay

Address			S1	S2	S3	S4	S5	S6
Alamitos Bay - #5561			90.7	90	91	90.1		
Alamitos Bay - #5571			89.9	91	90.4	90.2		
Alamitos Bay - Geneva Walk			90.7	90.8	90.1			
Alamitos Bay - #5575			90.1	91.8	90.6	92		
Alamitos Bay - #5585			91.7	90.3	89.2	90		
Palermo Walk #1			90.2	91.1	90.2	89.2	90.7	89.6
			S7 90.7	S8 90.3				
Alamitos Bay - Palermo Walk			88.5	89.2	88.9			
Palermo Walk - #2			89.4	88.4	89.3			
Alamitos Bay - #5607			89.4	88.9	88.9	89.7		

TREASURE ISLAND 08/2008 SEAWALL SURVEY

Napels Canal - W Neapolitan Ln Bridge

Napels Canal - #5668	IN 0.8 DEGREES	89.7 OUT	88.9 IN	88.9 IN	89.5 IN	89.1 IN	88.8 IN
Napels Canal - #5680		88.9 IN	89.4 IN	89.6 IN	89.6 IN	88.9 IN	89.2 IN
		S7 89.6 OUT	S8 89.3 IN	S9 89.9 IN	S10 89.1 IN	S11 88.4 IN	S12 89 IN

Treasure Island - Colonade Canal (NOTE: CHANGE IN INSPECTION DIRECTION)

Address		S1	S2	S3	S4	S5	S6
Colonade Canal - #5680		89.5 IN	88.6 IN				
Colonade Canal - #5		88.1 IN	89.6 IN	88.9 OUT	89.1 IN	88.8 IN	
Colonade Canal - #5679		88.8 IN	89.7 IN	89 IN			

Treasure Island - Alamitos Bay

Address		S1	S2	S3	S4	S5	S6
Alamitos Bay - #5561		88.6 IN	89.5 IN	88.6 IN	89.2 IN		
Alamitos Bay - #5571		89.3 IN	88.1 IN	88.9 IN	89.3 IN		
Alamitos Bay - Geneva Walk		88.7 IN	88.5 IN	89.2 IN			
Alamitos Bay - #5575	IN 0.7 DEGREES	89.3 IN	87.2 IN BOLTED	88.8 IN	87.4 IN		
Alamitos Bay - #5585	IN 0.7 DEGREES	87.6 IN	89.1 IN	88.6 OUT	89.2 IN		
Palermo Walk - #1		89.3 IN	88.3 IN	89.2 IN	89.8 OUT	88.7 IN	89.8 IN
		S7 88.6 IN	S8 89.1 IN				
Alamitos Bay - Palermo Walk		89.1 OUT	90	89.5 IN			
Palermo Walk - #2		89.6 IN	89.0 OUT	90			
Alamitos Bay - #5607		89.5 OUT	89.5 OUT				

Alamitos Bay - #5619			90	90.1					
Alamitos Bay - #5621			91	90.4	90.6				
Alamitos Bay - #5625			89.9	90.2	91				
Alamitos Bay - #5627			92.2	91.6	90.8				
Alamitos Bay - Florence Walk			91	90.1	89.4	90	91	91.6	
Alamitos Bay - #5639			91.3	90.7	90.3	90.6	89.3	91.2	
			S7 90.3	S8 91.3	S9 92.3	S10 91.8			
Alamitos Bay - #5643			91.6	90.4	91.7	91.7	90.7	91.7	
Alamitos Bay - #5649			90.8	90.6	90.6	90.2	90	90.7	
			S7 90.2	S8 90.2					
Alamitos Bay - #5655			90	90.2	91.1	90.5	91.2	91.9	
			S7 90.5						
Alamitos Bay - #5659			90.5	91.1	91.5	90.5	90.2	90.2	
Alamitos Bay - #5661			91	91	91	90.9	90.5	90.1	
Alamitos Bay - #5669			91.1	90.9	89.7	89.8	89.5	89.3	
			S7 90.4	S8 90.4	S9 91.2				
Alamitos Bay - #5679			90.9	89.85	90.1	89.6	90.3	90.6	
			S7 90.10	S8 90.8					
Alamitos Bay - End of Public Access									

Alamitos Bay - #5619		89.7 IN	88.3 IN	89.2 IN			
Alamitos Bay - #5621	IN 0.7 DEGREES	88.3 IN	88.9 IN	87.9 IN			
Alamitos Bay - #5625		89.5 IN	89.1 IN	88.3 IN			
Alamitos Bay - #5627	IN 0.8 DEGREES	87.0 IN	87.8 IN	88.7 IN			
Alamitos Bay - Florence Walk			88.3 IN		89.8 IN		88.4 IN
Alamitos Bay - #5639			88.6 IN		88.7 IN		88.0 IN
	IN 0.8 DEGREES		S8 88.0 IN		S10 87.4 IN		
Alamitos Bay - #5643	IN 0.8 DEGREES		88.9 IN		87.5 IN		87.6 IN
Alamitos Bay - #5649			88.6 IN		89.1 IN		88.6 IN
			S8 88.1 IN				
Alamitos Bay - #5655		89.3 IN		88.1 IN		88.0 IN	
	IN 0.8 DEGREES	S7 85.6 IN					
Alamitos Bay - #5659	IN 0.8 DEGREES	88.7 IN	88.2 IN	87.7 IN	88.8 IN	89.3 IN	89.1 IN
Alamitos Bay - #5661		88.4 IN	88.4 IN	88.2 IN	88.5 IN	88.8 IN	89.2 IN
Alamitos Bay - #5669		88.2 IN	88.4 IN	89.7 IN	89.6 IN	89.9 IN	90
		S7 88.9 IN	S8 88.9 IN	S9 88.2 IN			
Alamitos Bay - #5679		88.4 IN	88.6 IN	88.9 IN	89.1 IN	89.6 IN	85.8 IN
		S7 89.0 IN	S8 89.4 IN				
Alamitos Bay - End of Public Access							

4.0 Waterside Survey

4.1 Storm Drains

Storm Drains penetrate the seawall at every street and alley in Naples. A visual inventory of these drains reveals that less than half have operable dampers to prevent the backflow of water during extreme tides, (see Photo's 4-1 thru 4-6). For additional information see Section 5.0.

4.2 Seawall Cap

The concrete sheet piles are capped with an approximate 1'-8" wide x 3' tall concrete cap constructed in two increments, (see Figure 1-13 on page 1-13). The cap was examined from both the top deck and the water side. Photo's 4-11 thru 4-20 provide typical examples of corrosion deterioration occurring on the outboard face of the cap. In general terms the upper cap is experiencing corrosion spalls due to insufficient cover. The bottom cap typically has long corrosion spalls where boat wake and wind chop provide the chlorides and wet and dry sequences ripe for corrosion. For more detailed information on the Seawall Cap, refer to Section 2.2 on page 2-1 and Figure 2-10.

4.3 Dock Guide Piles

Photo 4-21 depicts the most common dock guide pile system in use at Naples. Notice how it is not contacting or loading the seawall. Photo 4-22 depicts a guide pile method the City attempts to avoid by educating, and enforcing when necessary, the Seawall Guidelines on page 1-11. It is the intent of the City to not introduce any new loads to the seawall structure.

4.4 Seawall Corners

The seawall turns have suffered more abuse than typical panels. Photo's 4-24 and 4-25 depict a combination of corrosion and impact damage. A small impact will accelerate corrosion by opening small cracks which allow chlorides in seawater to migrate further into the concrete member, reaching the steel sooner. Corners are inherently strong locations in a structure and we do not recommend repairs for this damage beyond what is planned, long range, for the entire community

4.5 Sheet Pile Gaps & Voids

Concrete sheet piles utilize tongue and grove joints (see Figure 1-12, page 1-12) to provide an earth seal. In many sheet pile designs the joint is intentionally left open ¾" so that grout may be placed in the joint after installation. The Naples Seawalls typically have a ¾" gap between sheets. During our waterside survey we find that in most cases the grout is still in place above water and missing under water, (see Picture 4-29 & 4-30). Open joints provide an ideal path for the transport of fine soils by tidal "pumping" action. Figure 2-10 on page 2-24 indicates areas where our divers have matched up sand piles at the bottom of the seawall joints with indication of subsidence and/or sinkhole potential on the top deck. These locations would gain the most benefit from a chemical grouting interim solution, see Figure 8-4.

4.6 Sheet Pile Sulfate Deterioration Our Structural Engineer Divers have identified what they are reasonably confident is sulfate deterioration of the below water portion of the concrete sheet piles in Rivo Alto Canal, see Figure 4-7 on page 4-40 for a plan of the limits of this deterioration. Sulfate deterioration typically first effects column or sheet pile corners by softening the concrete which allows more seawater to penetrate deeper into the member and once the reinforcing is aggressively corroding that becomes the mode of failure. In some circumstances where the corners get some protection, such as the Naples sheet piles, when the grout is till present between sheets, the sulfates continue to weaken the entire concrete surface until is sheds off in large spongy chunks. Long before material begins to fall from the member, chlorides have penetrated and are corroding the reinforcing. This appears to be the deterioration process at Naples. The corners, when exposed, are the first to weaken, (see Photo 4-46), then round, then corrode, (see Photo 4-20) then small voids along the tongue and grove joints form, (see Photo's 4-34 & 4-35) which may allow soil loss, then the entire sheet face begins to shed, (see Photo's 4-37 thru 4-42). In the pile faces where we removed some loose concrete we were unable to find any remaining horizontal ties in the concrete section. Photo 4-44 shows sulfate deterioration slowly eating at the northwest quarter of Rivo Alto Canal. The early stages look like a pebbled surface which is the concrete aggregate after the loss of ½" of the sand and cement binder. We recommend replacement of the severely damaged sheet pile bulkhead as soon as practical with either Figure 8-6 or 8-8. If funding will be delayed, Interim repair 8-5 has the potential for reducing the sheet pile stresses by 50% which will buy a few years.

Photo 4 – 25, Naples, Waterside Survey: **Rivo Alto Canal #5607**
Broken/raveled sheet pile joint at corner near canal entrance.

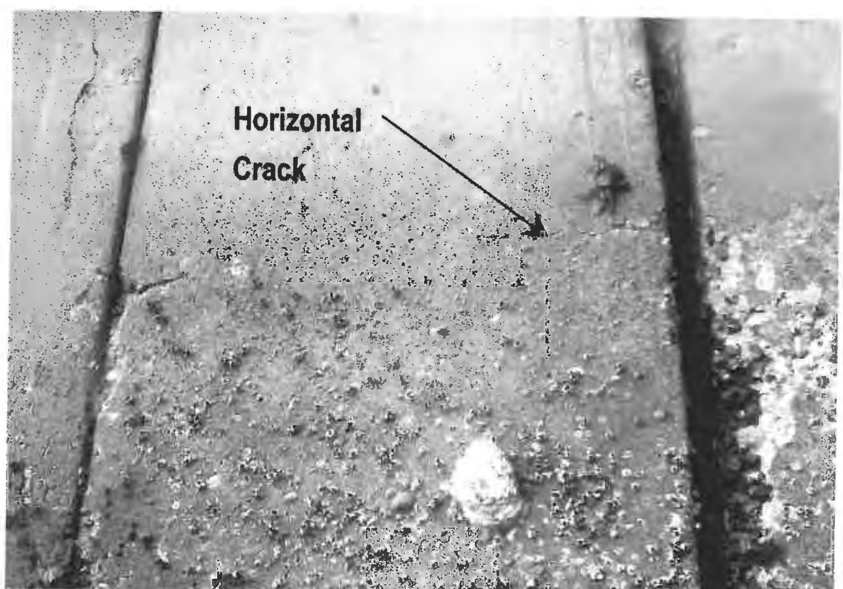


Photo 4 – 26, Naples, Waterside Survey: **Rivo Alto Canal #173**
Typical sheet pile horizontal crack near mean high tide elevation.

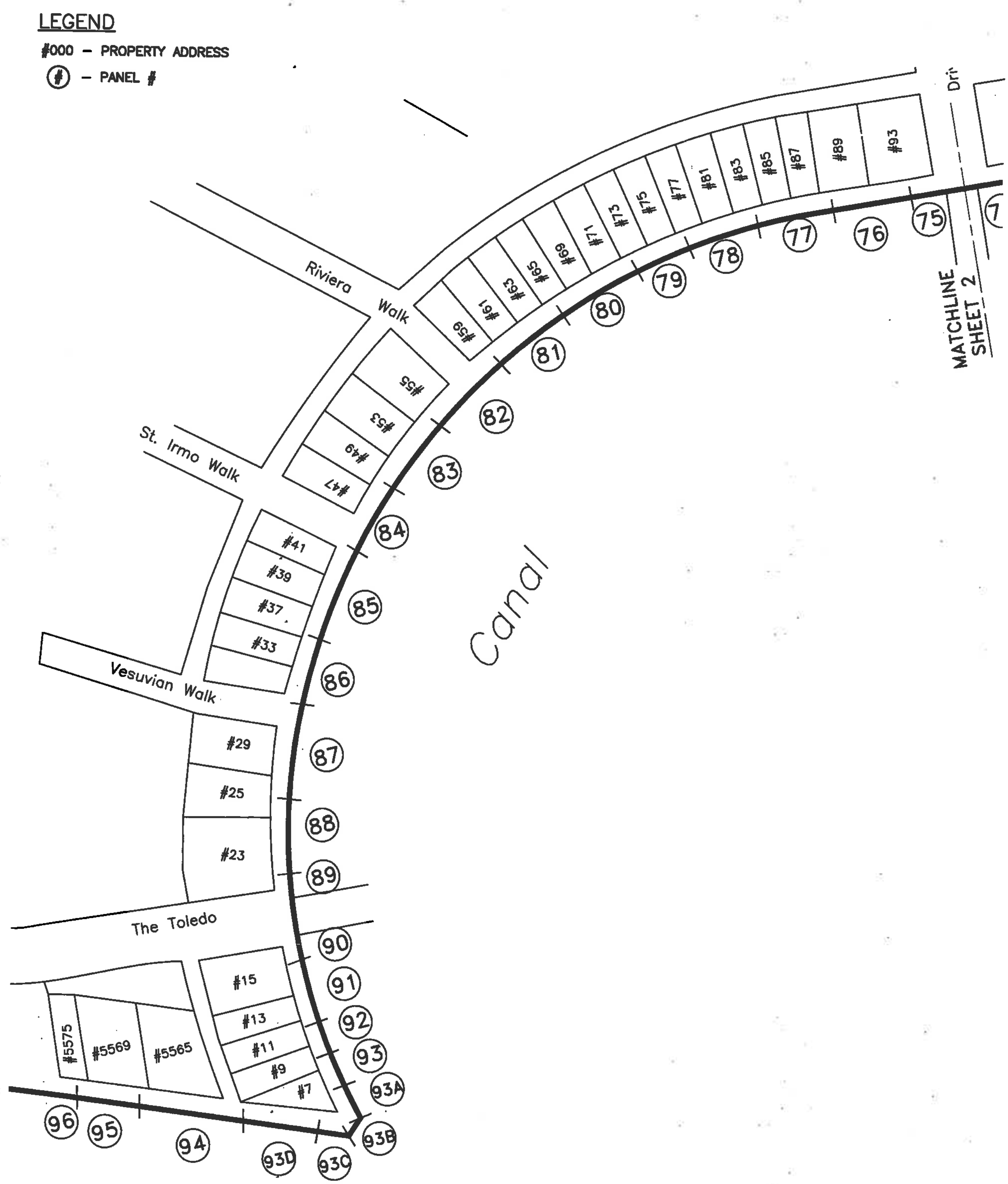


FIGURE 4-3a NAPLES LANDSIDE JOINT PANEL KEY PLAN

SURVEY DATE: 8/2006

SHEET 1 OF 6

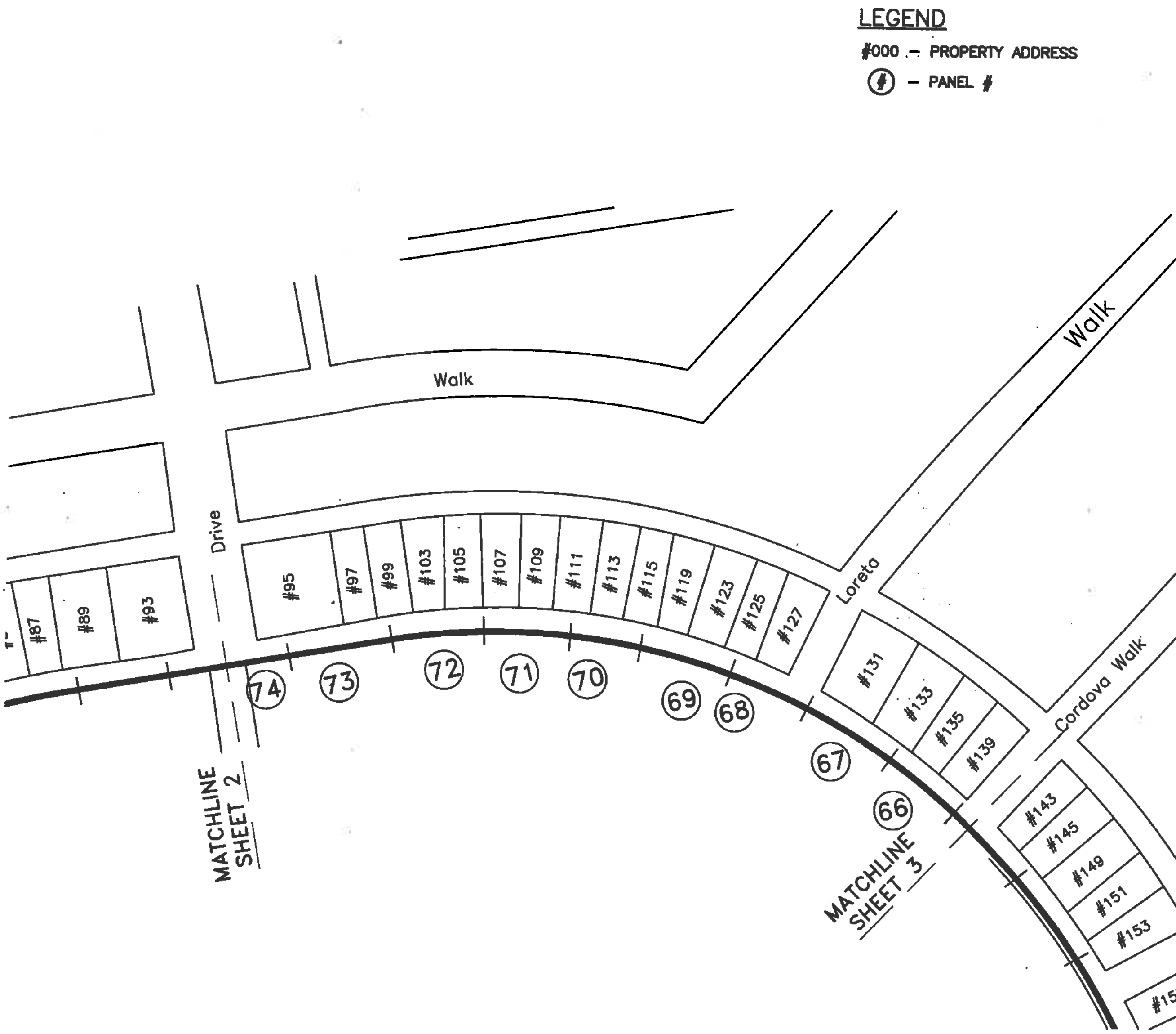


FIGURE 4–3b NAPLES LANDSIDE JOINT PANEL KEY PLAN

SURVEY DATE: 8/2006

SHEET 2 OF 6

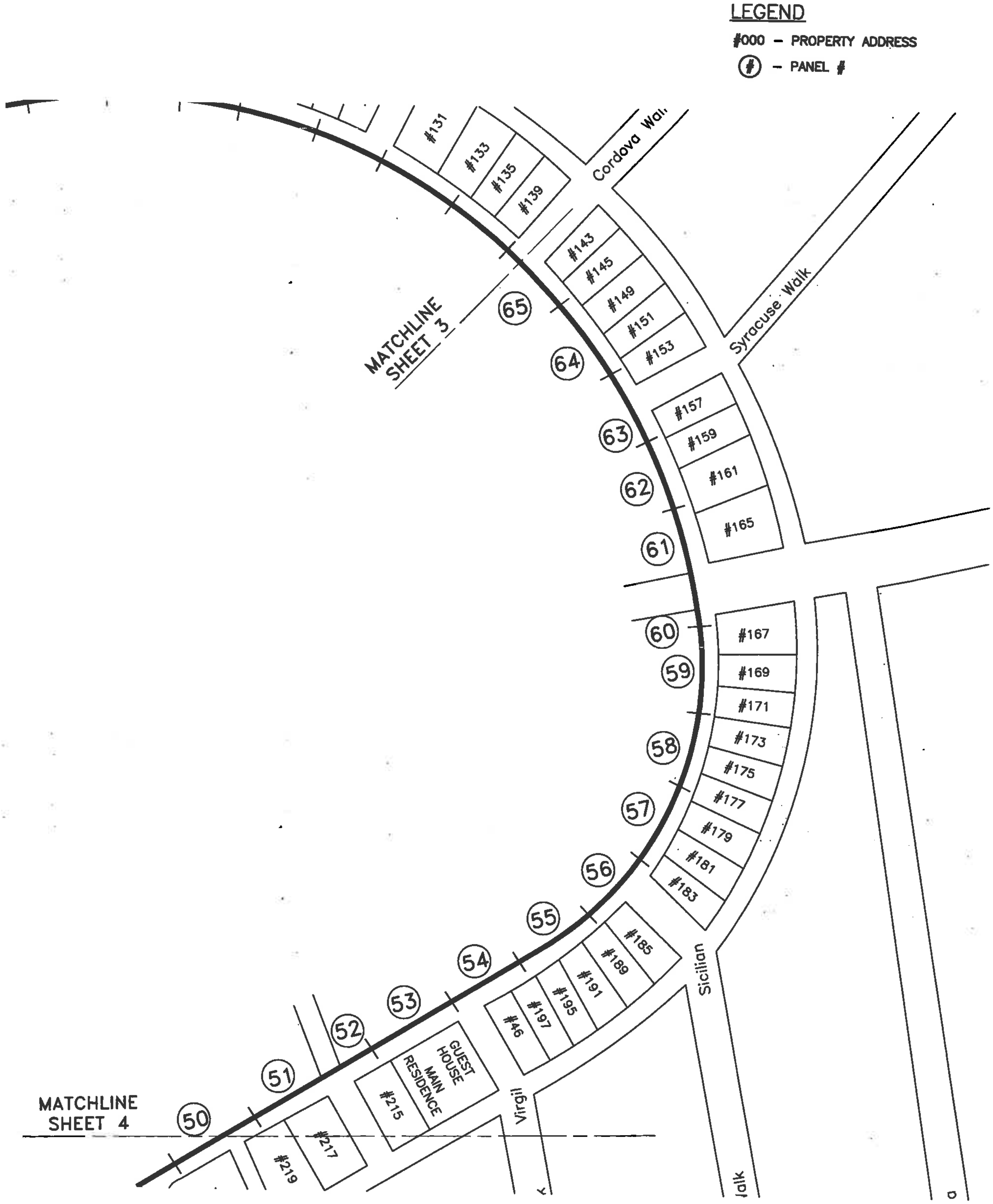


FIGURE 4–3c NAPLES LANDSIDE JOINT PANEL KEY PLAN

SURVEY DATE: 8/2008

SHEET 3 OF 6

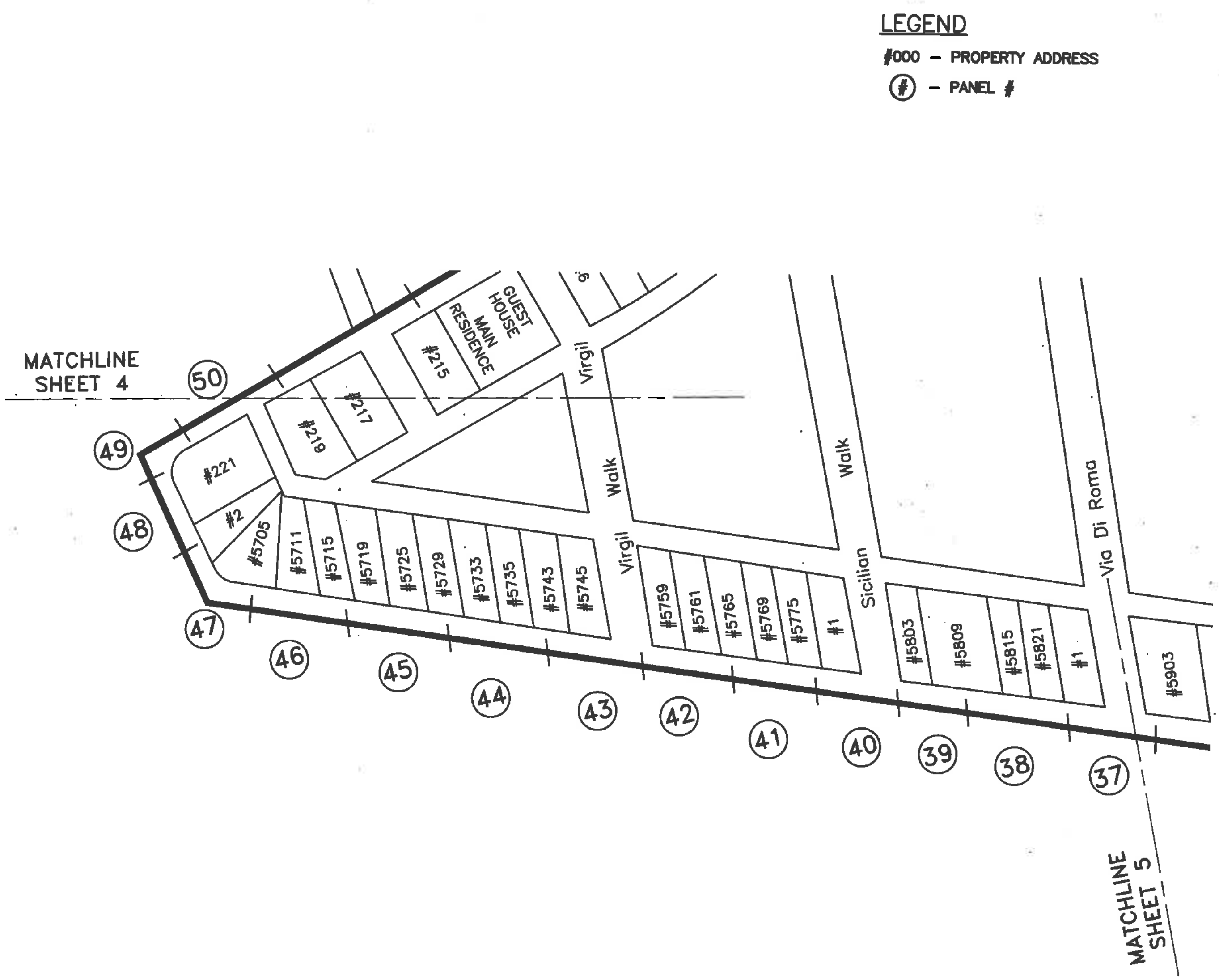


FIGURE 4–3d NAPLES LANDSIDE JOINT PANEL KEY PLAN

SURVEY DATE: 8/2008

SHEET 4 OF 6

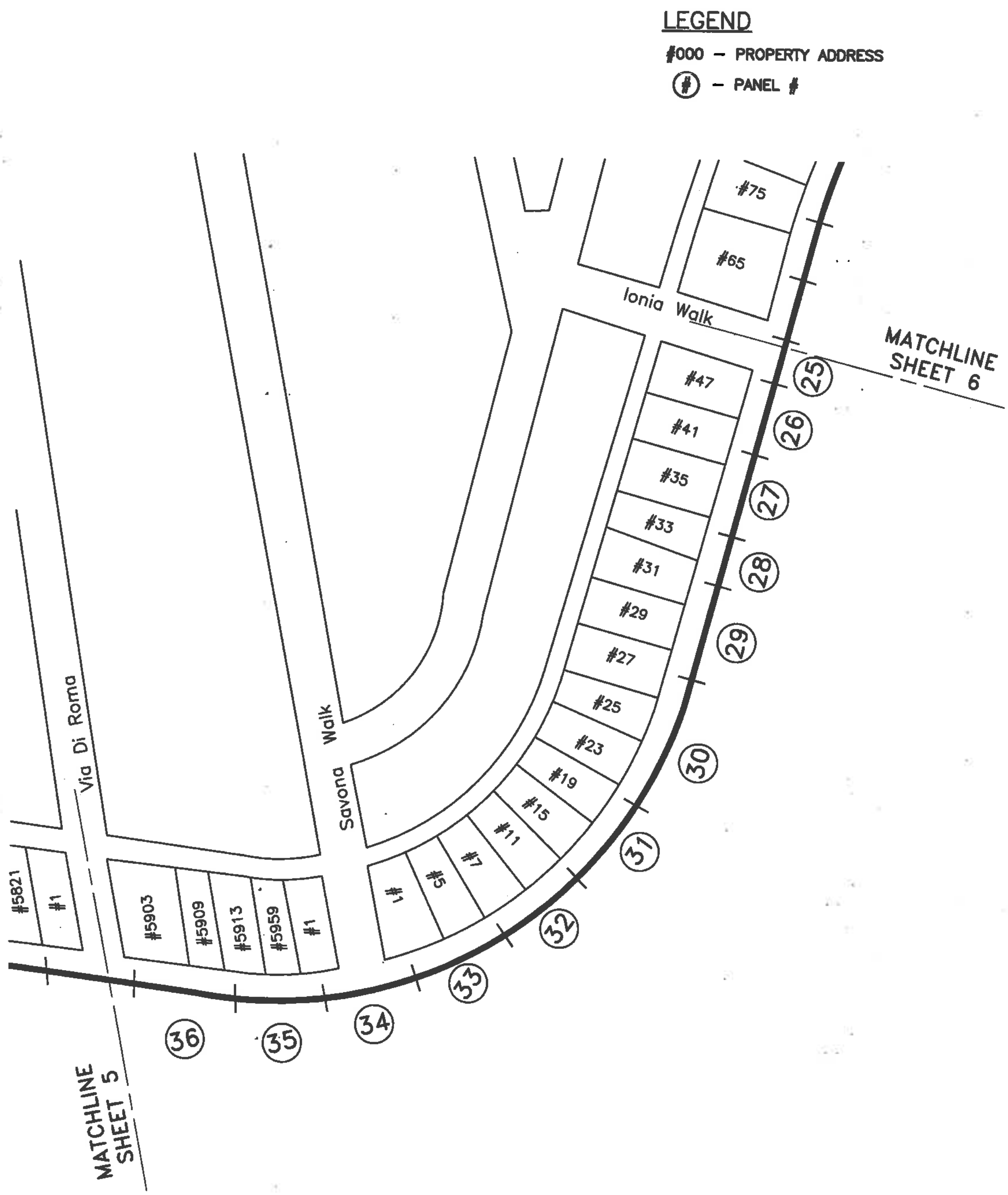


FIGURE 4–3e NAPLES LANDSIDE
JOINT PANEL KEY PLAN

SURVEY DATE: 8/2008

SHEET 5 OF 6

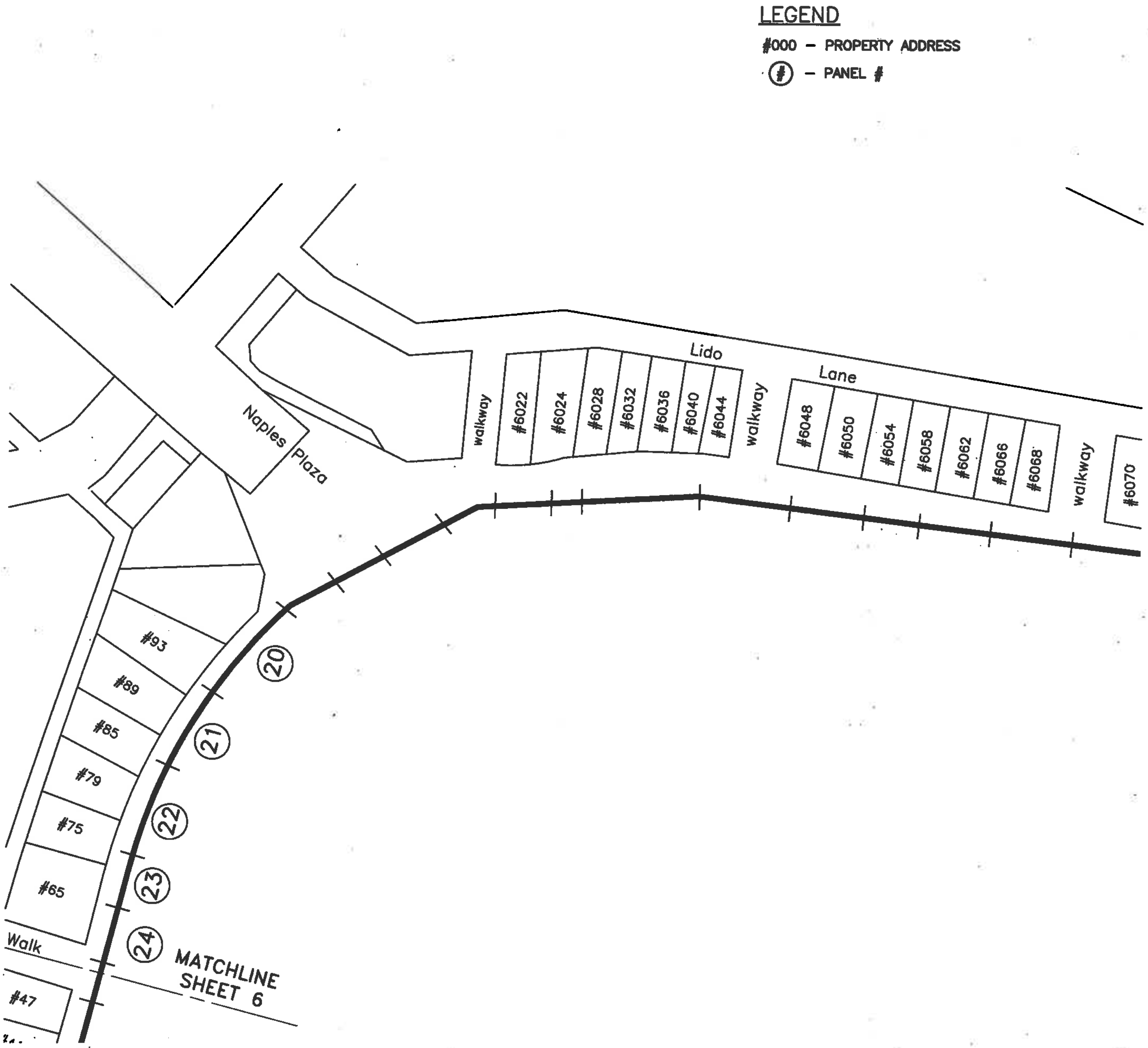


FIGURE 4–3f NAPLES LANDSIDE JOINT PANEL KEY PLAN

SURVEY DATE: 8/2008

SHEET 6 OF 6

Figure 4-4 Naples - Landside Waterside Seawall Cap Cracking Quantities		
Panel #	Top Cap - Cracking	Bottom Cap - Cracking
Panel 20	0	43.5
Panel 21	24	26
Panel 22	6	20
Panel 23	18	47
Panel 23A	16	66
Panel 24	6	31
Panel 25	3	68
Panel 26	18	40
Panel 27	12	53.5
Panel 28	8	25
Panel 29	10	34
Panel 30	10	8
Panel 31	4	5
Panel 32	6	37.5
Panel 33	20	29.5
Panel 34	2	0
Panel 35	26	20
Panel 36	10	30.5
Panel 37	6	42.5
Panel 38	9	50.5
Panel 39	8	102.5
Panel 40	2	42.5
Panel 41	5	47.5
Panel 42	8	48.5
Panel 43	14	22
Panel 44	4	55.5
Panel 45	7	39
Panel 46	10	36
Panel 47	4	27
Panel 48	4	4
Panel 49	2	4
Panel 50	6	34
Panel 50A	12	23
Panel 51	15	18
Panel 52	8	0
Panel 53	8	31.5
Panel 54	11	23.5

Figure 4-4 Naples - Landside Waterside Seawall Cap Cracking Quantities

Panel #	Top Cap - Cracking	Bottom Cap - Cracking
Panel 55	7	32
Panel 56	16	37.5
Panel 57	12	54
Panel 58	12	35
Panel 59	6	62.5
Panel 60	0	15
Panel 61	14	50
Panel 62	15	25
Panel 63	12	10
Panel 64	4	65
Panel 65	3	0
Panel 66	9	66
Panel 67	7	49
Panel 68	2	21
Panel 69	10	70
Panel 70	6	48
Panel 71	10	23
Panel 72	7	41
Panel 73	19	13
Panel 74	3	32
Panel 75	11	23
Panel 76	28	22
Panel 77	14	36
Panel 78	25	19
Panel 79	17	52
Panel 80	9	29
Panel 81	12	51
Panel 82	2	48
Panel 83	15	60
Panel 84	14	36
Panel 85	19	44
Panel 86	14	35
Panel 87	16	8
Panel 88	8	34
Panel 89	0	16

Figure 4-4 Naples - Landside Waterside Seawall Cap Cracking Quantities		
Panel #	**Top Cap - Cracking**	**Bottom Cap - Cracking**
Panel 90	11	18
Panel 91	10	48
Panel 92	4	6
Panel 93	7	23
Panel 93A	4	24
Panel 93B	2	13
Panel 93C	6	15
Panel 93D	12	39
Panel 94	2	24
Panel 95	3	16
Panel 96	10	32
Total Linear Ft.	**791'**	**2756'**

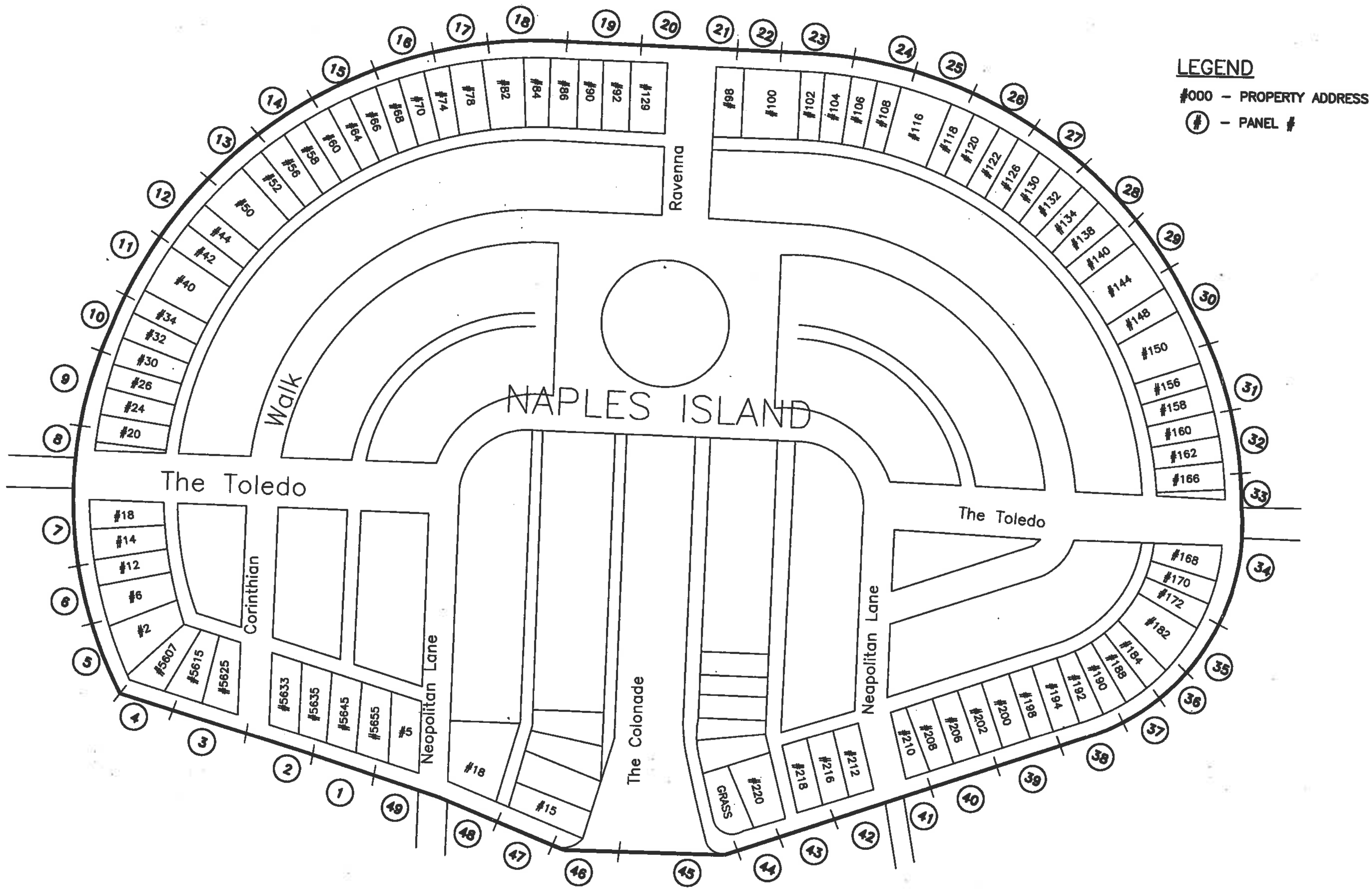


FIGURE 4-5 NAPLES ISLAND JOINT PANEL KEY PLAN

Figure 4-6 Naples Island Waterside Seawall Cap Cracking Quantities

Panel #	Top Cap - Cracking	Bottom Cap - Cracking
Panel 1	0	44'
Panel 2	0	30'
Panel 3	6'	31.5'
Panel 4	0	48'
Panel 5	0	38'
Panel 6	0	30'
Panel 7	0	38'
Panel 8	13'	1.5'
Panel 9	2'	3.5'
Panel 10	0	42'
Panel 11	1	6'
Panel 12	0	28'
Panel 13	0	32'
Panel 14	0	25'
Panel 15	0	18'
Panel 16	0	5'
Panel 17	0	6'
Panel 18	4.5'	19'
Panel 19	0	12'
Panel 20	1'	12'
Panel 21	10	24'
Panel 22	4'	1'
Panel 23	2'	4'
Panel 24	2'	43'
Panel 25	0	5'

Figure 4-6 Naples Island Waterside Seawall Cap Cracking Quantities		
Panel #	Top Cap - Cracking	Bottom Cap - Cracking
Panel 26	6'	36'
Panel 27	0	3'
Panel 28	6'	45'
Panel 29	8'	34.5'
Panel 30	0	18'
Panel 31	4'	37'
Panel 32	17'	23'
Panel 33	4'	37'
Panel 34	5'	47'
Panel 35	0	5'
Panel 36	0	17'
Panel 37	0	14'
Panel 38	1'	43'
Panel 39	7'	22.5'
Panel 40	4'	16'
Panel 41	0	56'
Panel 42	0	62.5'
Panel 43	0	37'
Panel 44	4'	2'
Panel 45	13'	21'
Panel 46	4'	62'
Panel 47	8'	31'
Panel 48	2'	38'
Panel 49	1'	44'
Total Linear Ft.	**133'**	**1297.5'**

Figure 5-1 Naples Island Storm Drain Survey	
LOCATION	TYPE / CONDITION
CORINTHIAN WALK	**12" DIA, ELBOW TYPE, INFLATABLE SEAL INSIDE**
#56 RIVO ALTO CANAL	**12" DIA, BAFFLE TYPE, FAIR CONDITION**
#84	**12" DIA, BAFFLE TYPE, SEAL MISSING**
#129	**12" DIA, BAFFLE TYPE, GOOD CONDITION**
#118	**12" DIA, BAFFLE TYPE, INFLATABLE SEAL INSIDE**
#182	**12"DIA, BAFFLE TYPE, GOOD CONDITION**
#212	**12" DIA, BAFFLE TYPE, FROZEN OPEN**
#220	**12" DIA, BAFFLE TYPE, FAIR CONDITION**
COLONNADE PARK #1	**12" DIA, BAFFLE TYPE, INFLATABLE SEAL INSIDE**
WALKWAY / #15	**12" DIA, BAFFLE TYPE, OLD BUT WORKS**
#5	**10" DIA, OPEN HOLE**

Figure 5-2 Naples Landside Storm Drain Survey	
LOCATION	Description
#93	12" DIA BAFFLE TYPE, FOULED WITH BARNACLES, TORN SEAL
IONIA WALK BAY SIDE)	12" DIA, BAFFLE TYPE, INFLATABLE PLUG INSIDE
SAVONA WALK (BAY SIDE)	12' DIA, ELBOW TYPE, FOULED WITH MUSSELS
VIA DI ROMA WALK (BAY SIDE)	12' DIA, BAFFLE TYPE, FAIR CONDITION
SICILIAN WALK (BAY SIDE)	12' DIA, BAFFLE TYPE, FAIR CONDITION
VIRGIL WALK (BAY SIDE)	12' DIA, ELBOW TYPE, FOULED WITH MUSSELS
ALLEY (BAY SIDE)	12' DIA, ELBOW TYPE, FOULED WITH MUSSELS
VIRGIL WALK (CANAL SIDE)	12' DIA, ELBOW TYPE, FOULED WITH MUSSELS
SICILIAN WALK (CANAL SIDE)	12" DIA, BAFFLE TYPE, INFLATABLE PLUG INSIDE
#169	12' DIA, OPEN HOLE
#167	6" DIA, OPEN HOLE
#165	12' DIA, BAFFLE TYPE, GOOD CONDITION
SYRACUSE WALK	12' DIA, OPEN HOLE
CORDOVA WALK #1	30" DIA, BAFFLE TYPE, FROZEN OPEN
LORETTA WALK	10" DIA, OPEN HOLE
TOLEDO BRIDGE	12" DIA, BAFFLE TYPE, GOOD CONDITION
#93	12" DIA, BAFFLE TYPE, GOOD CONDITION
#65	12" DIA, BAFFLE TYPE, NEW CONDITION
RIVIERA WALK	12" DIA, ELBOW TYPE, FAIR CONDITION
ST IRMO WALK	12" DIA, ELBOW TYPE, FAIR CONDITION
VESUVIAN WALK	12" DIA, ELBOW TYPE, FAIR CONDITION
TOLEDO	12" DIA, ELBOW TYPE, FAIR CONDITION
#5575	12" DIA, HORIZONTAL COMPRESSION GASKET, FAIR CONDITION
#119	12" DIA, BAFFLE TYPE, FROZEN OPEN

Figure 5-3 Treasure Island Storm Drain Survey	
LOCATION	TYPE / CONDITION
#5570	12" DIA, BAFFLE TYPE, GOOD CONDITION
GENEVA WALK (BAY SIDE)	12" DIA, BAFFLE TYPE, GOOD CONDITION
#5576	12" DIA, ELBOW TYPE, FAIR CONDITION
PALERMO WALK	12" DIA, BAFFLE TYPE, GOOD CONDITION
FLORENCE WALK (BAY SIDE)	12" DIA, BAFFLE TYPE, CONC OBSTRUCTS LID FROM FULL OPNG
#5680 & ALLEY	12" DIA, BAFFLE TYPE, FOULED WITH BARNACLES
FLORENCE WALK (CANAL SIDE)	12" DIA, BAFFLE TYPE, FAIR CONDITION
PALERMO WALK (CANAL SIDE)	12" DIA, BAFFLE TYPE, GOOD CONDITION
GENEVA WALK (CANAL SIDE)	12" DIA, BAFFLE TYPE, FAIR CONDITION
#5561	6" DIA, OPEN HOLE

Figure 5-4 Naples Island
Light Pole Location & Condition Report

LOCATION	# OF POLES	CONDITION / ANGLE
CORINTHIAN WALK	1	**SLIGHT /** TILTED 5 DEG. INBOARD
#2	1	**GOOD** / STRAIGHT
#18	1	**GOOD** / STRAIGHT
#20	1	**GOOD** / STRAIGHT / CORROSION AT BASE
BRIDGE	1	**GOOD** / STRAIGHT
#30	1	**GOOD** / STRAIGHT
#42	1	**GOOD** / STRAIGHT
#56	1	**GOOD** / STRAIGHT
#74	1	**GOOD** / STRAIGHT
#90	1	**SLIGHT** / TILTED 10 DEG. OUTBOARD
#98	1	**SLIGHT** / TILTED 5 DEG. INBOARD
#102	1	**MAJOR-MODERATE** / TILTED 10-15 DEG. OUTBOARD
#116	1	**SLIGHT** / TILTED 5 DEG. INBOARD
#126	1	**GOOD** / STRAIGHT
#129	1	**GOOD** / STRAIGHT
#144	1	**GOOD** / STRAIGHT
#158	1	**GOOD** / STRAIGHT
#166	1	**GOOD** / STRAIGHT
#168 & TOLEDO ST.	1	**GOOD** / STRAIGHT
#170	1	**SLIGHT** / TILTED 3 DEG. INBOARD
#184	1	**GOOD** / STRAIGHT
#202	1	**SLIGHT** / TILTED 5 DEG. INBOARD
#210	1	**GOOD** / STRAIGHT
#220	1	**GOOD** / STRAIGHT
NEAPOLITIAN BRIDGE	2	**GOOD** / STRAIGHT
#15	1	**GOOD** / STRAIGHT
#18	1	**GOOD** / STRAIGHT

Figure 5-5 Naples Landside Light Pole Location & Condition Notes		
LOCATION	# OF POLES	CONDITION / ANGLE
APPIAN & LIDO	1	**GOOD** / STRAIGHT
#6076 LIDO LANE	1	**GOOD** / STRAIGHT
#6062 LIDO LANE	1	**GOOD** / STRAIGHT
#6036 LIDO LANE	1	**GOOD** / STRAIGHT
#6022 LIDO LANE	1	**GOOD** / STRAIGHT
#55	1	**SLIGHT** / TILTED
IONIA WALK	1	**GOOD** / STRAIGHT
#31	1	**SLIGHT** / TILTED INBOARD
#19	1	**GOOD** / STRAIGHT
SAVONA WALK	1	**GOOD** / STRAIGHT
VIA DI ROMA	1	**GOOD** / STRAIGHT
SICILIAN WALK	1	**SLIGHT** / TILTED INBOARD
VIGIL WALK	1	**MODERATE** / TILTED INBOARD
#5725 CORSO DE NAPLES	1	**GOOD** / STRAIGHT
#5705 CORSO DE NAPLES	1	**SLIGHT** / TILTED
#221 RIVO ALTO CANAL	1	**GOOD** / STRAIGHT
#219 RIVO ALTO CANAL	1	**GOOD** / STRAIGHT
#217 RIVO ALTO CANAL	1	**GOOD** / STRAIGHT
VIRGIL WALK @ RIO ALTO CANAL	1	**SIGNIFICANT** / TILT (LEANING LEFT WHEN FACING WATER)
SICILIAN WALK	1	**GOOD** / STRAIGHT
#173 RIVO ALTO CANAL	1	**GOOD** / STRAIGHT
#167 RIVO ALTO CANAL	1	**GOOD** / STRAIGHT (BASE CRUMBLING, PIC. #573)
#165	2	**GOOD** / STRAIGHT
SYRACUSE WALK	1	**GOOD** / STRAIGHT
#139	1	**SLIGHT** / TILTED 10 DEG.
LORETTA WALK	1	**SLIGHT** / TILTED 10-15 DEG. INBOARD
#103	1	**MODERATE** / TILTED15 DEG. INBOARD
#93	1	**SLIGHT** / TILTED
RIVIERA WALK	1	**MODERATE** / TILTED 15-20 DEG. INBOARD
ST. IRMO WALK	1	**SLIGHT** / TILTED 10 DEG. INBOARD
#75	1	**SLIGHT** / TILTED 5 DEG. OUTBOARD
TOLEDO RD. BRIDGE	4	**GOOD** / STRAIGHT
CORNER OF #7 & #5565 (ALLEY)	1	**SLIGHT** / TILTED 5 DEG. INBOARD

Figure 5-6 Treasure Island Light Pole Location & Condition Report		
LOCATION	# OF POLES	CONDITION / ANGLE
#5570 & GENEVA WALK	1	**SLIGHT** / TILTED 5 DEG. INBOARD
#5598 & PALERMO WALK	1	**GOOD** / STRAIGHT
#5624 & FLORENCE WALK	1	**GOOD** / STRAIGHT
#5658	1	**GOOD** / STRAIGHT
#5668 & ALLEY	1	**GOOD** / STRAIGHT
#5680	1	**SLIGHT** / TILTED 10 DEG. INBOARD
#5679	1	**SLIGHT** / TILTED 5 DEG. INBOARD
#5659 & #5655	1	**SLIGHT** / TILTED 5-10 DEG. INBOARD & LEFT
#5627 & FLORENCE WALK	1	**SLIGHT** / TILTED 0-5 DEG. OUTBOARD
#1 & PALMERO WALK	1	**MODERATE** / TILTED 10-15 DEG. INBOARD
#5571 & GENEVA WALK	1	**SLIGHT** / TILTED 10 DEG. INBOARD

6.0 HYDROGRAPHIC SURVEY PROFILES

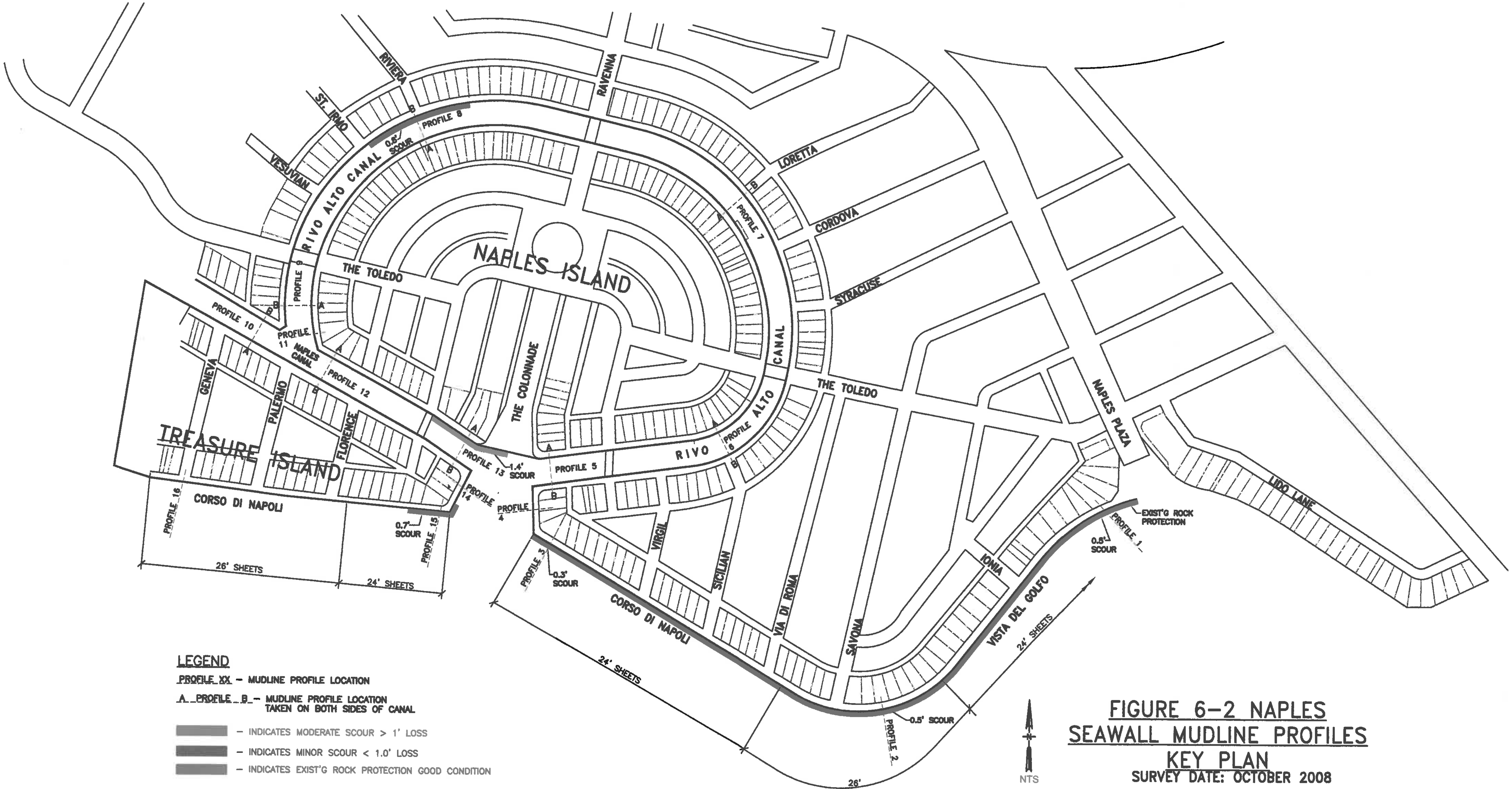
PROFILES TAKEN OCTOBER 2008

PROFILE #	ELEV @ WALL		ELEV @ 20' AWAY	ESTIMATED SILTATION	SITE LOCATION
LANDSIDE					
1	+1.5	ROCK	-6.2	0.5' SCOUR	EAST BAYSIDE
2	-5.5		-6.2	0.5' SCOUR	EAST BAYSIDE
3	-5.3		-7.1	0.3' SCOUR	EAST BAYSIDE
4	-4.6		-6.3	1.8'	CHANNEL ENTRANCE
5B	-5.2		-7.6	0.8'	CHANNEL ENTRANCE
6B	-4.8		-6.9	0.4	CANAL
7B	-4.1		-6.2	1.1'	CANAL
8B	-5.8		-7.4	0.8' SCOUR	CANAL
9B	-4.1		-6.3	1.0'	CANAL
10B	-4.4		-7.6	0.2'	CANAL
NAPLES ISLAND					
5A	-5.3		-7.7	0.7'	CHANNEL ENTRANCE
6A	-4.3		-6.6	0.8'	CANAL
7A	-3.4		-5.8	1.6'	CANAL
8A	-4.2		-6.2	1.0'	CANAL
9A	-3.2		-6.8	1.2'	CANAL
11	-4.6		-7.4	0.2'	CANAL
12A	-3.6		-6.8	1.0'	CANAL
13A	-6.5		-8.9	1.4' SCOUR	CHANNEL ENTRANCE
TREASURE ISLAND					
10A	-3.8		-7.2	0.7'	CANAL
12B	-3.7		-7.1	0.8'	CANAL
13B	-6.4		-7.9	0.1'	CHANNEL ENTRANCE
14	-5.0		-7.6	0.9'	CHANNEL ENTRANCE
15	-8.7		-9.7	0.7' SCOUR	WEST BAYSIDE
16	-4.6		-8.0	2.7'	WEST BAYSIDE

FIGURE 6-1

* DATUM MEAN LOWER LOW WATER (MLLW)

ESTIMATED ORIGINAL DREDGE DEPTH BEFORE SILTATION		**CONC. SHEET PILE LENGTH**
EAST BAYSIDE	-5.0 @ WALL TO - 7.5 @ 20' OFF WALL	24 & 26 FEET
CHANNEL ENTRANCE	-6.5 @ WALL TO - 8.0 @ 20' OFF WALL	22 FEET
CANAL	-5.0 @ WALL TO - 7.5 @ 20' OFF WALL	22 FEET
WEST BAYSIDE	-8.0 @ WALL TO -10.0 @ 20' OFF WALL	24 & 26 FEET



LEGEND

PROFILE XX – MUDLINE PROFILE LOCATION

A PROFILE B – MUDLINE PROFILE LOCATION TAKEN ON BOTH SIDES OF CANAL

– INDICATES MODERATE SCOUR > 1' LOSS

– INDICATES MINOR SCOUR < 1.0' LOSS

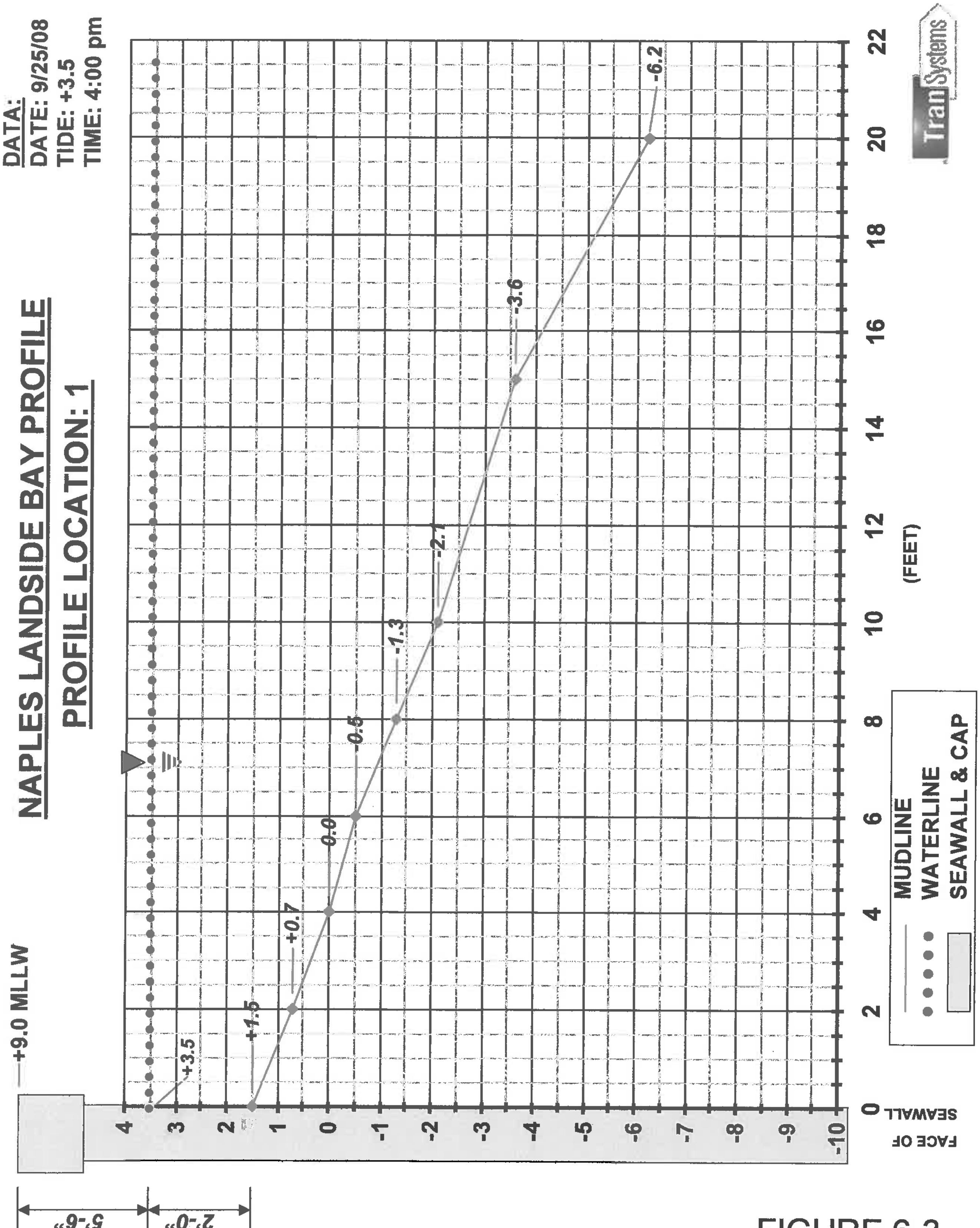
– INDICATES EXIST'G ROCK PROTECTION GOOD CONDITION

NOTE: ALL SHEET PILES 22' IN LENGTH UNLESS NOTED OTHERWISE ON PLAN.

FIGURE 6–2 NAPLES SEAWALL MUDLINE PROFILES KEY PLAN

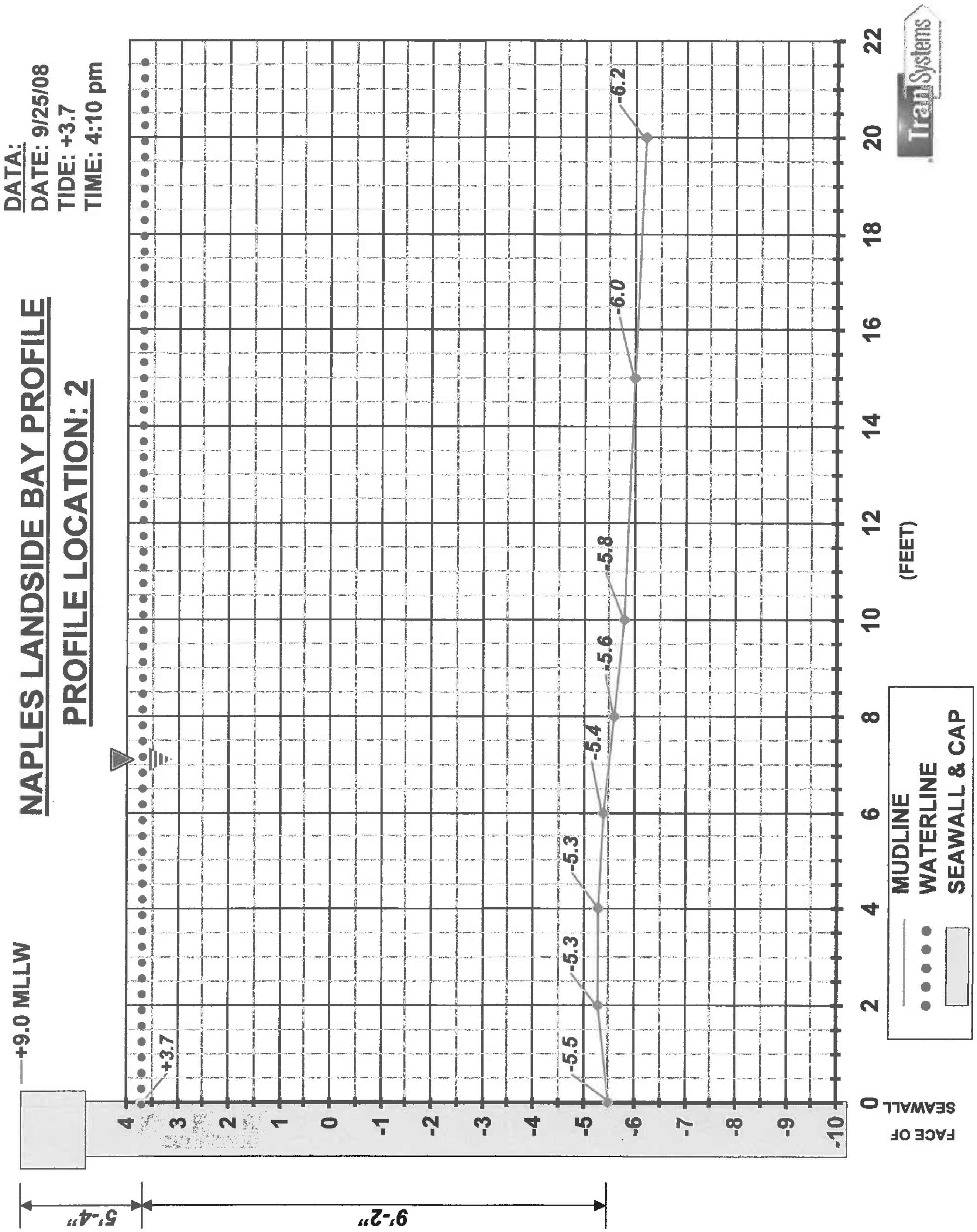
SURVEY DATE: OCTOBER 2008

FIGURE 6–2



NAPLES LANDSIDE BAY PROFILE
PROFILE LOCATION: 1
DATA:
DATE: 9/25/08
TIDE: +3.5
TIME: 4:00 pm
+9.0 MLLW
5'-6"
2'-0"
+3.5
+1.5
+0.7
0.0
-0.5
-1.3
-2.1
-3.6
-6.2
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
FACE OF SEAWALL 0
2
4
6
8
10
12
14
16
18
20
22
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-3



NAPLES LANDSIDE BAY PROFILE
PROFILE LOCATION: 2
DATA:
DATE: 9/25/08
TIDE: +3.7
TIME: 4:10 pm
+9.0 MLLW
5'-4"
9'-2"
FACE OF SEAWALL
(FEET)
+3.7
-5.5
-5.3
-5.3
-5.4
-5.6
-5.8
-6.0
-6.2
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-4

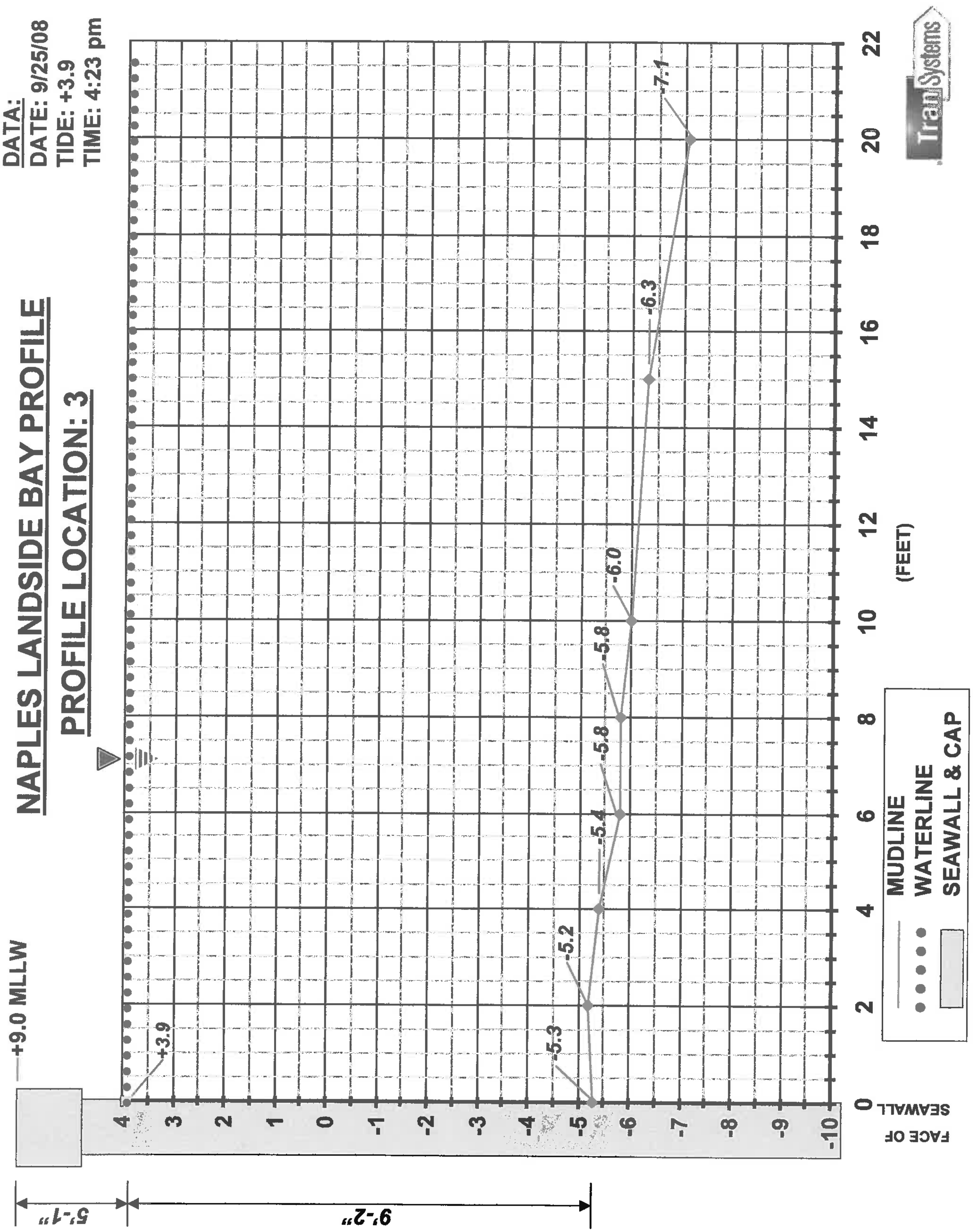

NAPLES LANDSIDE BAY PROFILE
PROFILE LOCATION: 3
DATA:
DATE: 9/25/08
TIDE: +3.9
TIME: 4:23 pm
+9.0 MLLW
+3.9
-5.3
-5.2
-5.4
-5.8
-5.8
-6.0
-6.3
-7.1
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
5'-1"
9'-2"
TranSystems

FIGURE 6-5

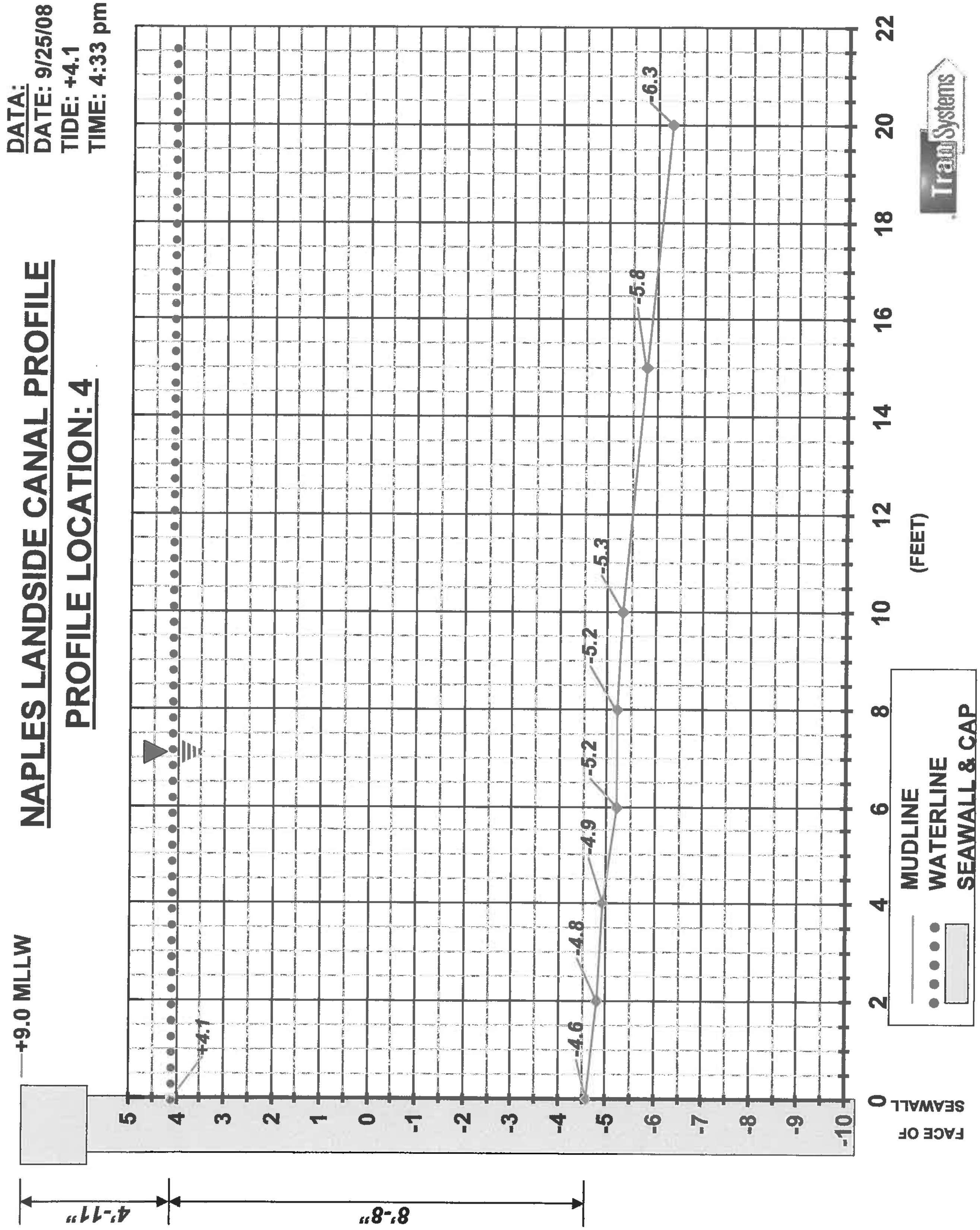

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 4
DATA:
DATE: 9/25/08
TIDE: +4.1
TIME: 4:33 pm
+9.0 MLLW
4'-11"
8'-8"
+4.1
-4.6
-4.8
-4.9
-5.2
-5.2
-5.3
-5.8
-6.3
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-6

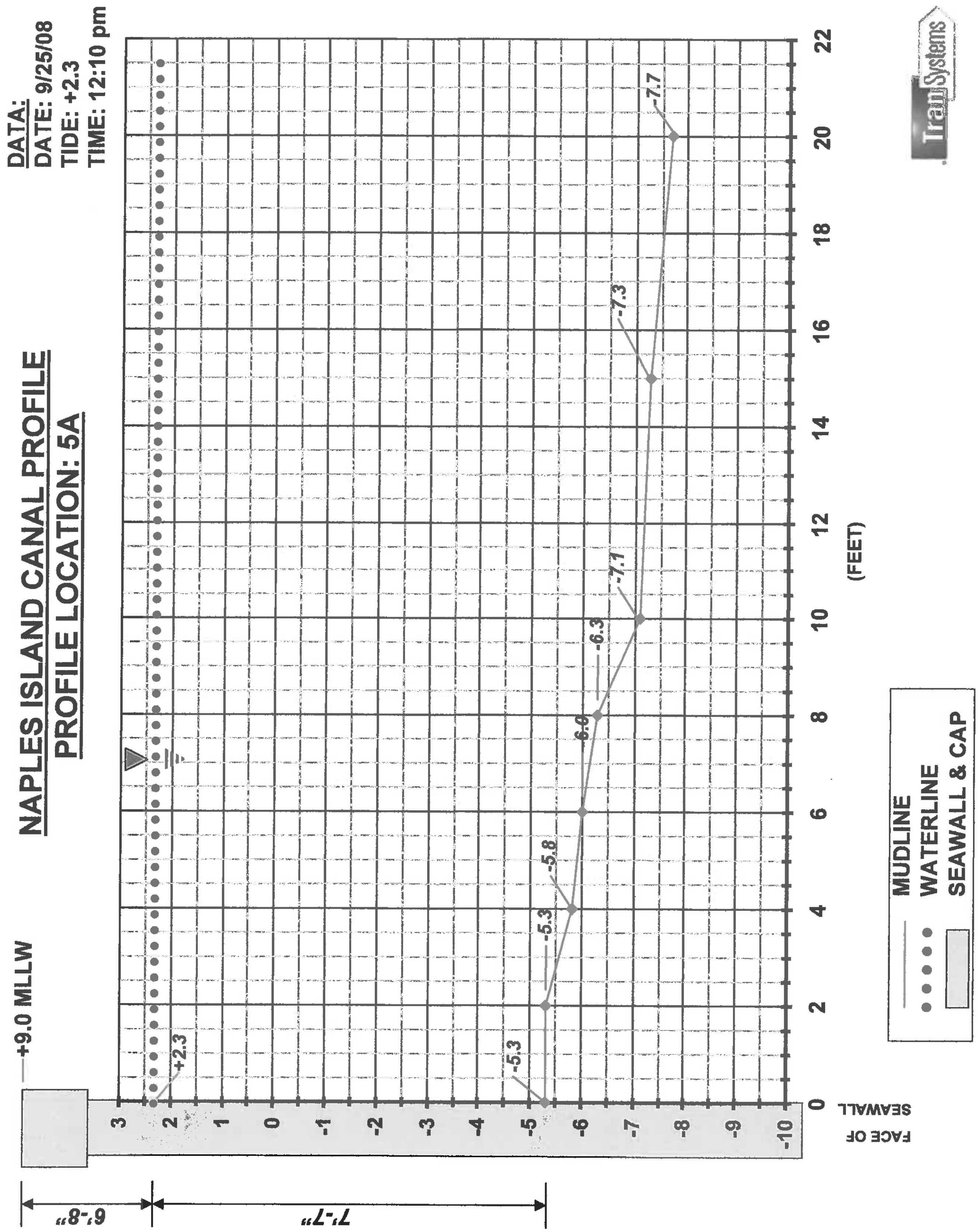
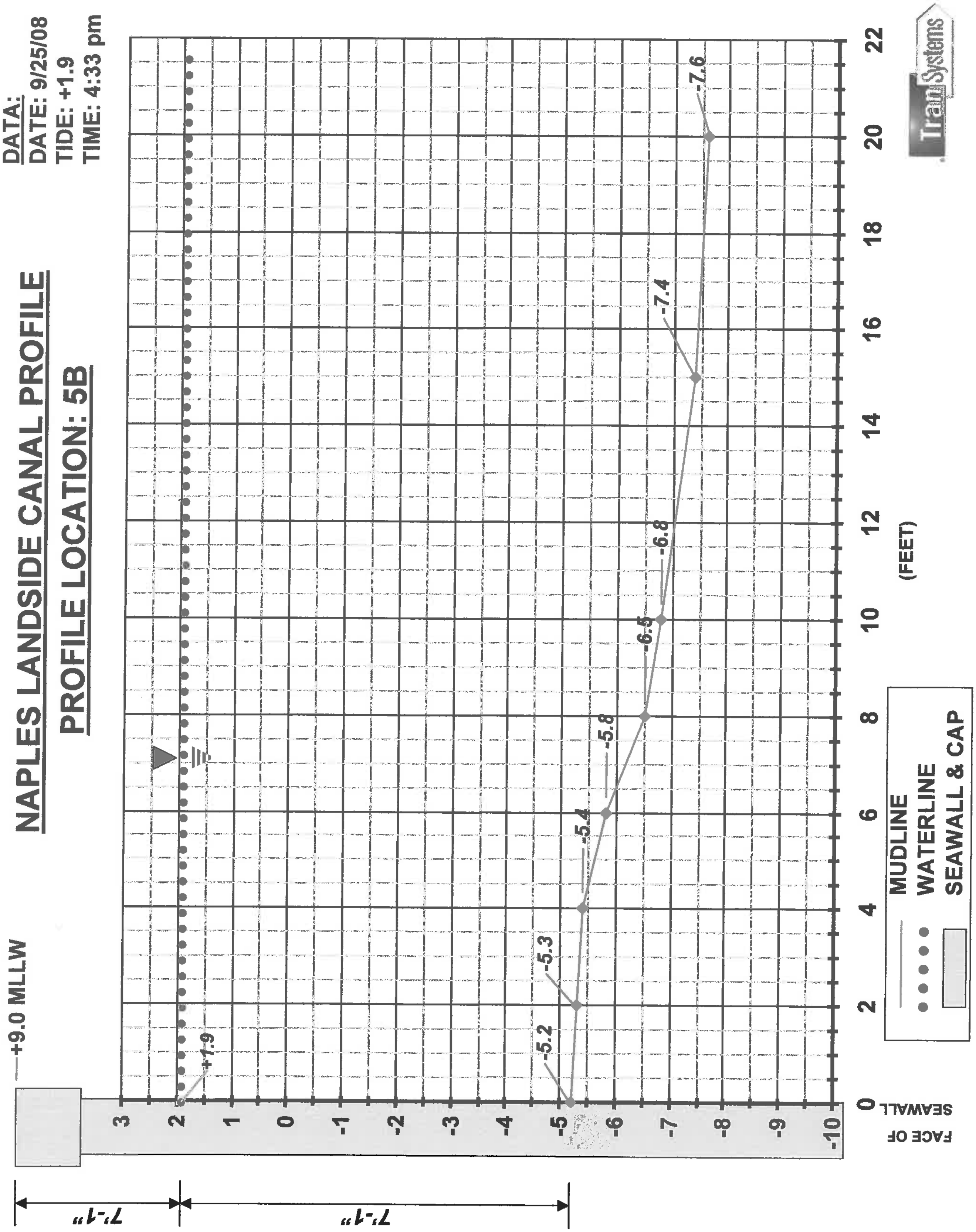

NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 5A
DATA:
DATE: 9/25/08
TIDE: +2.3
TIME: 12:10 pm
+9.0 MLLW
6'-8"
7'-7"
+2.3
-5.3
-5.3
-5.8
-6.0
-6.3
-7.1
-7.3
-7.7
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP

FIGURE 6-7



NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 5B
DATA:
DATE: 9/25/08
TIDE: +1.9
TIME: 4:33 pm
+9.0 MLLW
+1.9
-5.2
-5.3
-5.4
-5.8
-6.5
-6.8
-7.4
-7.6
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
7'-1"
7'-1"

FIGURE 6-8

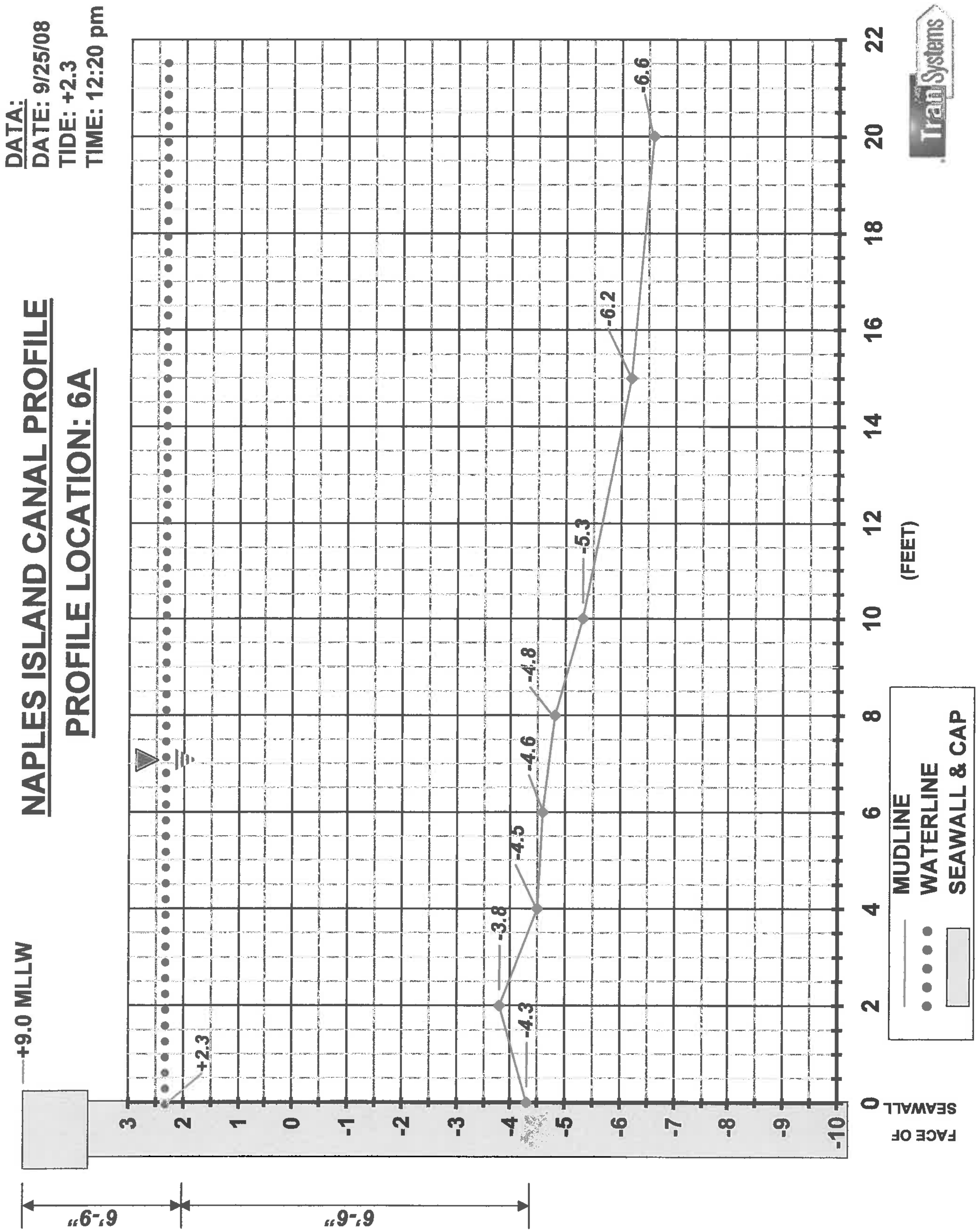

NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 6A
DATA:
DATE: 9/25/08
TIDE: +2.3
TIME: 12:20 pm
+9.0 MLLW
+2.3
-4.3
-3.8
-4.5
-4.6
-4.8
-5.3
-6.2
-6.6
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
6'-9"
6'-6"

FIGURE 6-9

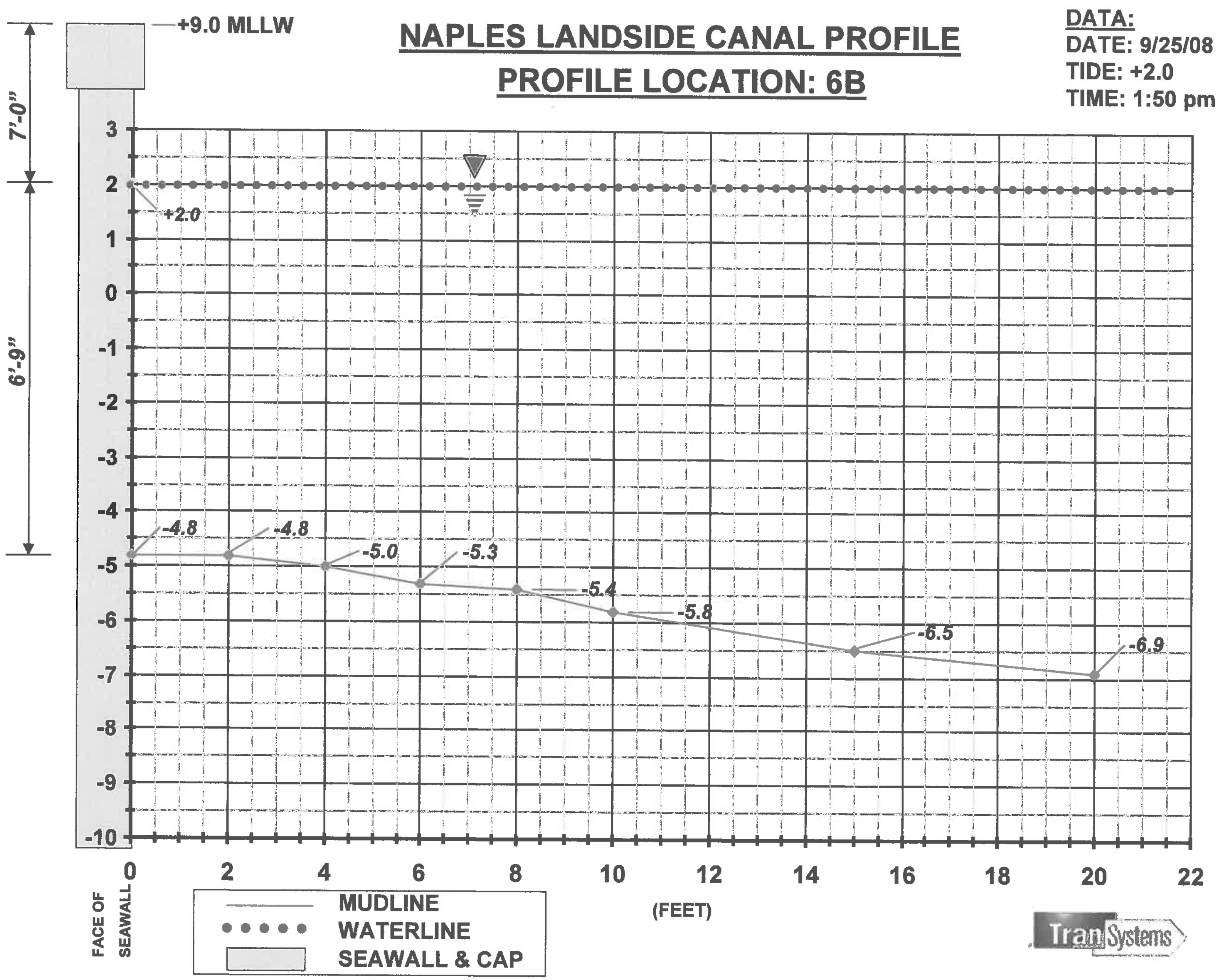

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 6B
DATA:
DATE: 9/25/08
TIDE: +2.0
TIME: 1:50 pm
+9.0 MLLW
7'-0"
6'-9"
+2.0
-4.8
-4.8
-5.0
-5.3
-5.4
-5.8
-6.5
-6.9
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP

FIGURE 6-10

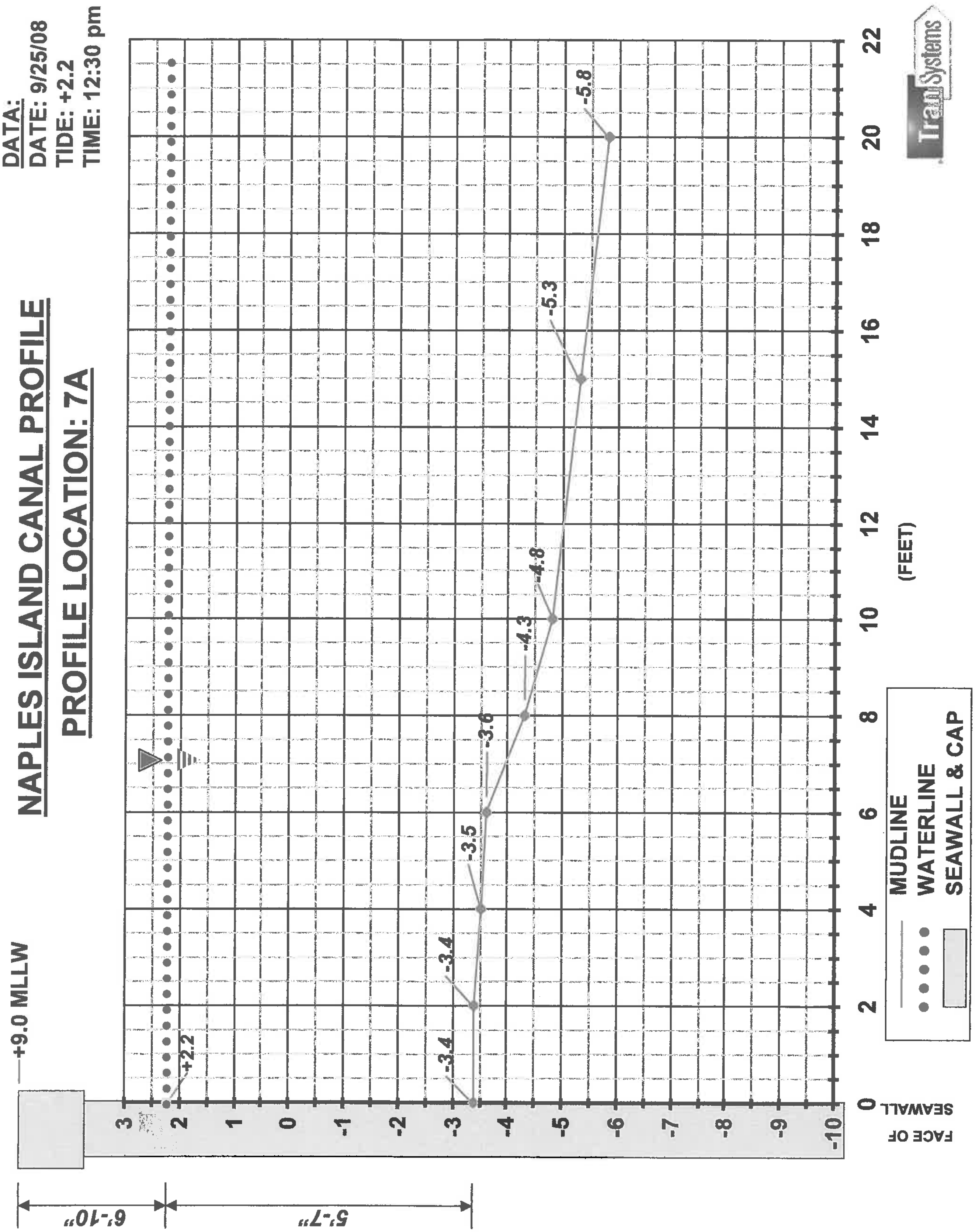

NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 7A
DATA:
DATE: 9/25/08
TIDE: +2.2
TIME: 12:30 pm
+9.0 MLLW
+2.2
-3.4
-3.4
-3.5
-3.6
-4.3
-4.8
-5.3
-5.8
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
6'-10"
5'-7"
TranSystems

FIGURE 6-11

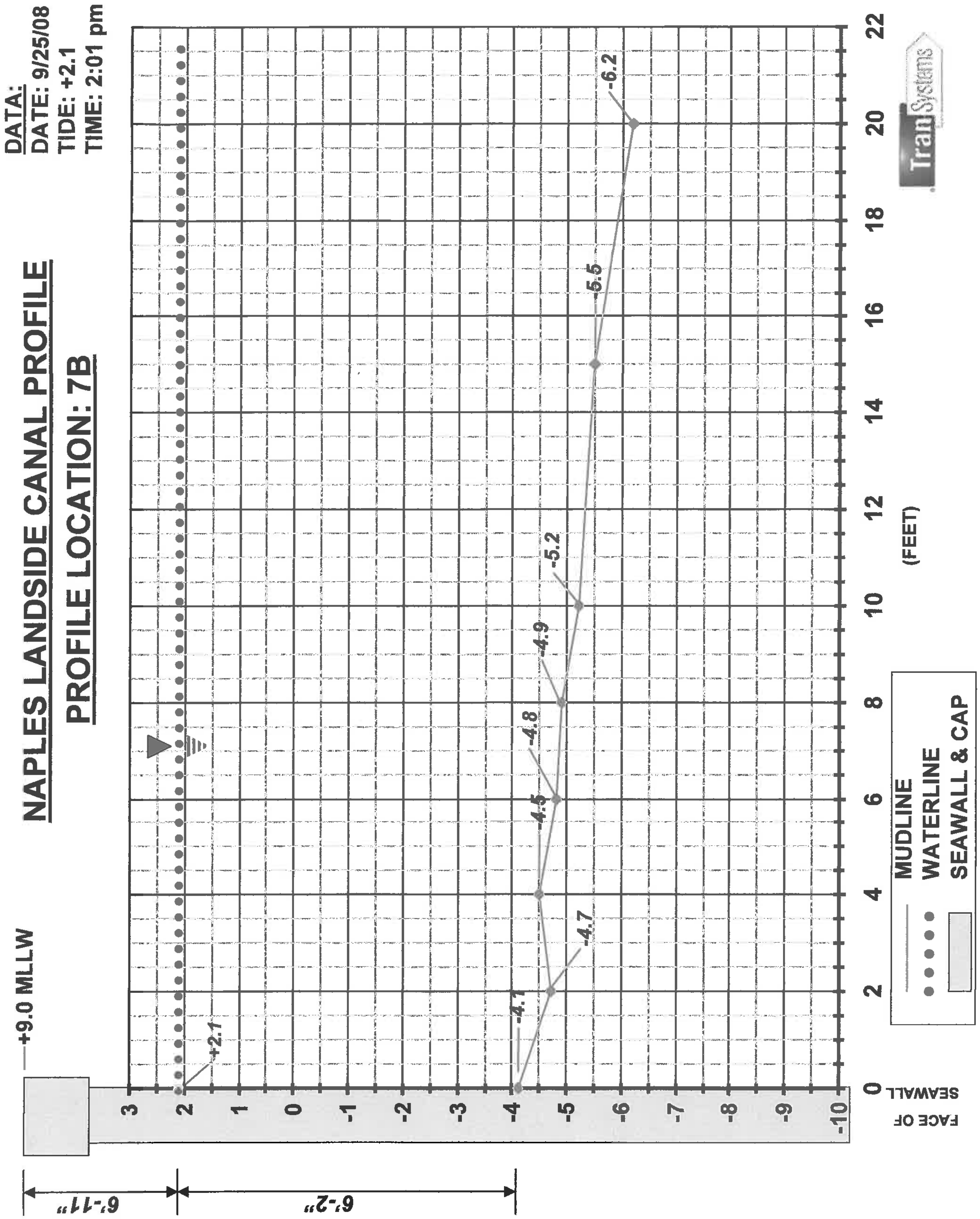
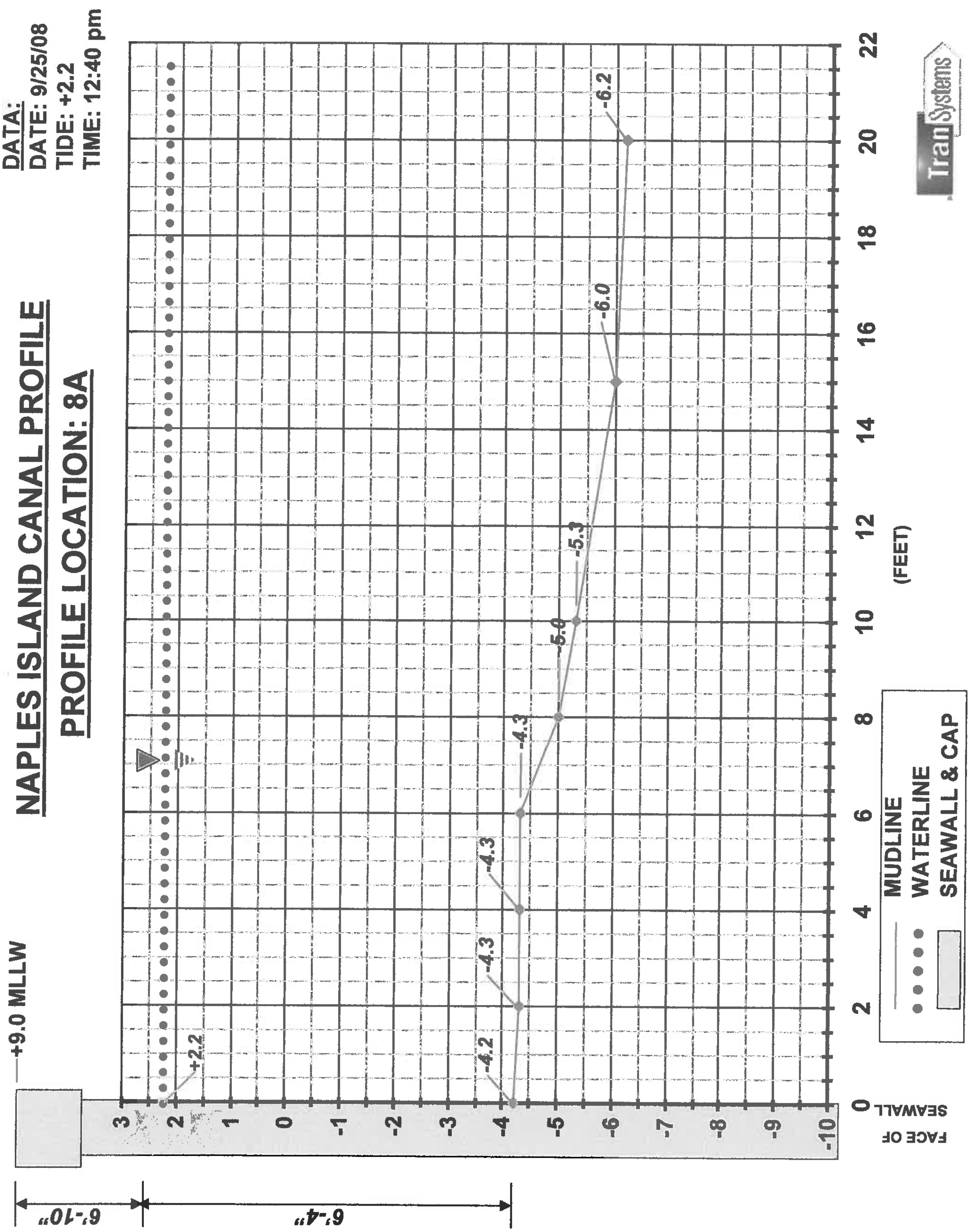

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 7B
DATA:
DATE: 9/25/08
TIDE: +2.1
TIME: 2:01 pm
+9.0 MLLW
6'-11"
6'-2"
+2.1
-4.1
-4.7
-4.5
-4.8
-4.9
-5.2
-5.5
-6.2
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP

FIGURE 6-12



NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 8A
DATA:
DATE: 9/25/08
TIDE: +2.2
TIME: 12:40 pm
+9.0 MLLW
6'-10"
6'-4"
+2.2
-4.2
-4.3
-4.3
-4.3
-5.0
-5.3
-6.0
-6.2
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-13

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 8B

DATA:
DATE: 9/25/08
TIDE: +2.3
TIME: 2:40 pm

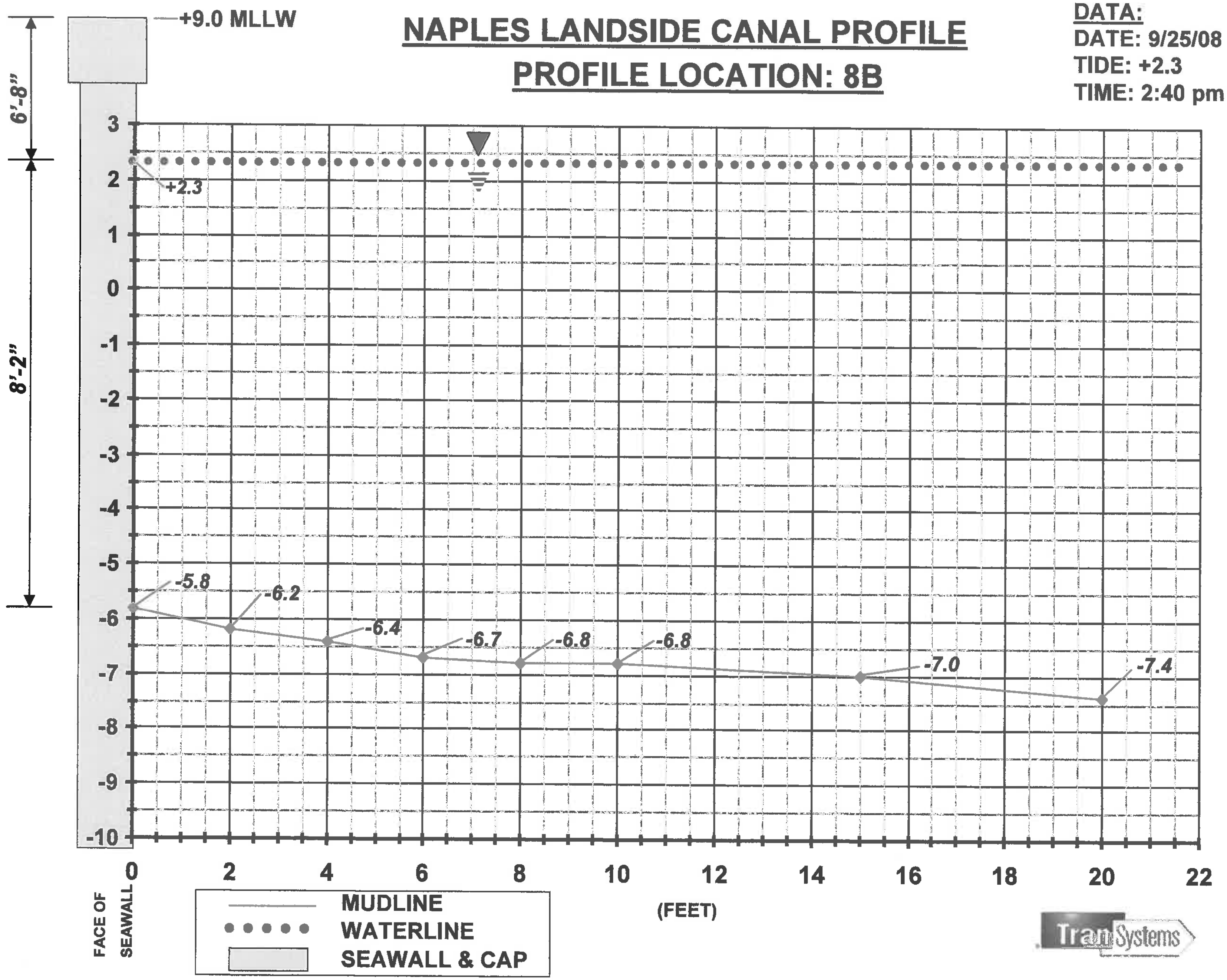


FIGURE 6-14

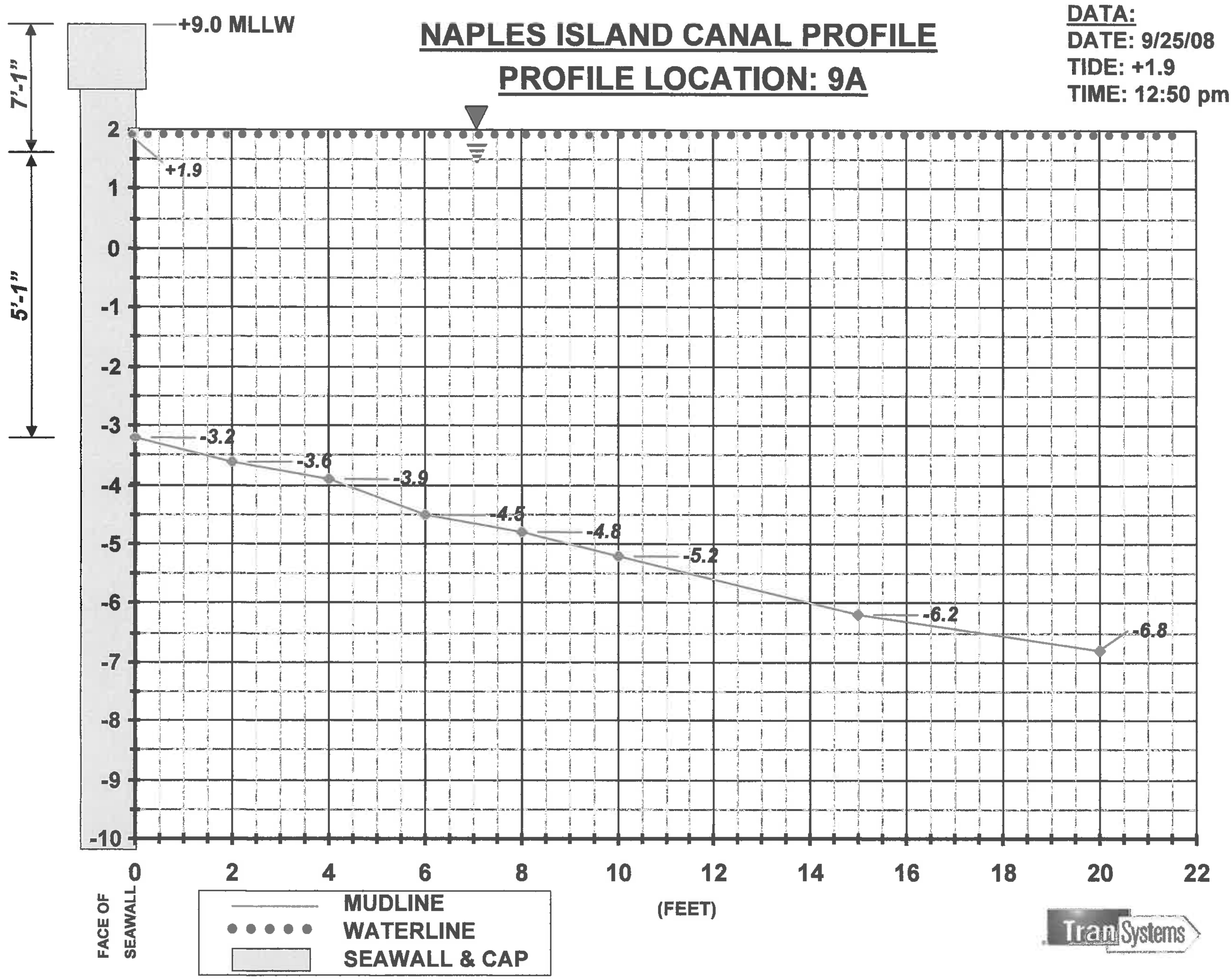

NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 9A
DATA:
DATE: 9/25/08
TIDE: +1.9
TIME: 12:50 pm
+9.0 MLLW
7'-1"
5'-1"
+1.9
-3.2
-3.6
-3.9
-4.5
-4.8
-5.2
-6.2
-6.8
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-15

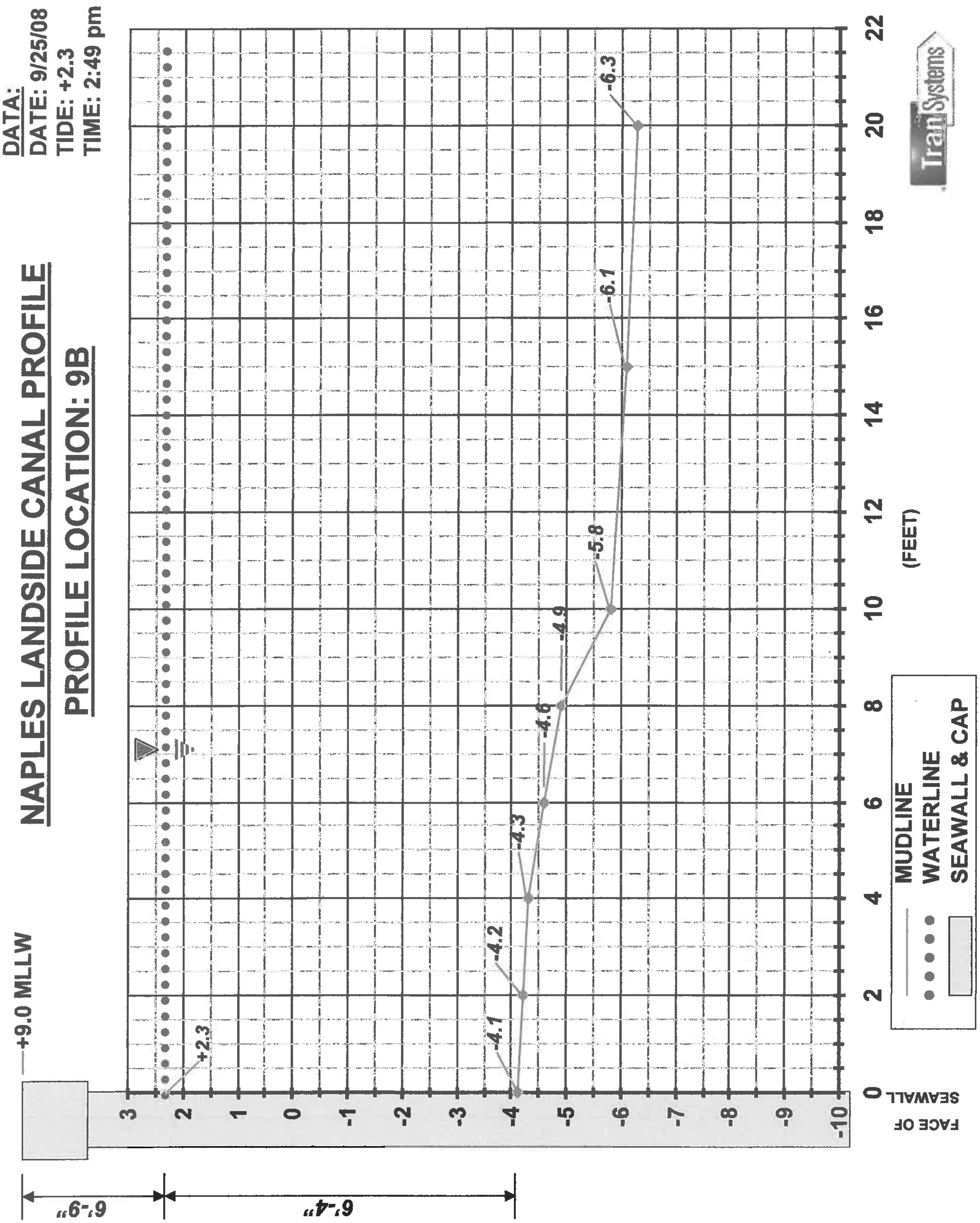

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 9B
DATA:
DATE: 9/25/08
TIDE: +2.3
TIME: 2:49 pm
+9.0 MLLW
+2.3
-4.1
-4.2
-4.3
-4.6
-4.9
-5.8
-6.1
-6.3
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
6'-9"
6'-4"
TranSystems

FIGURE 6-16

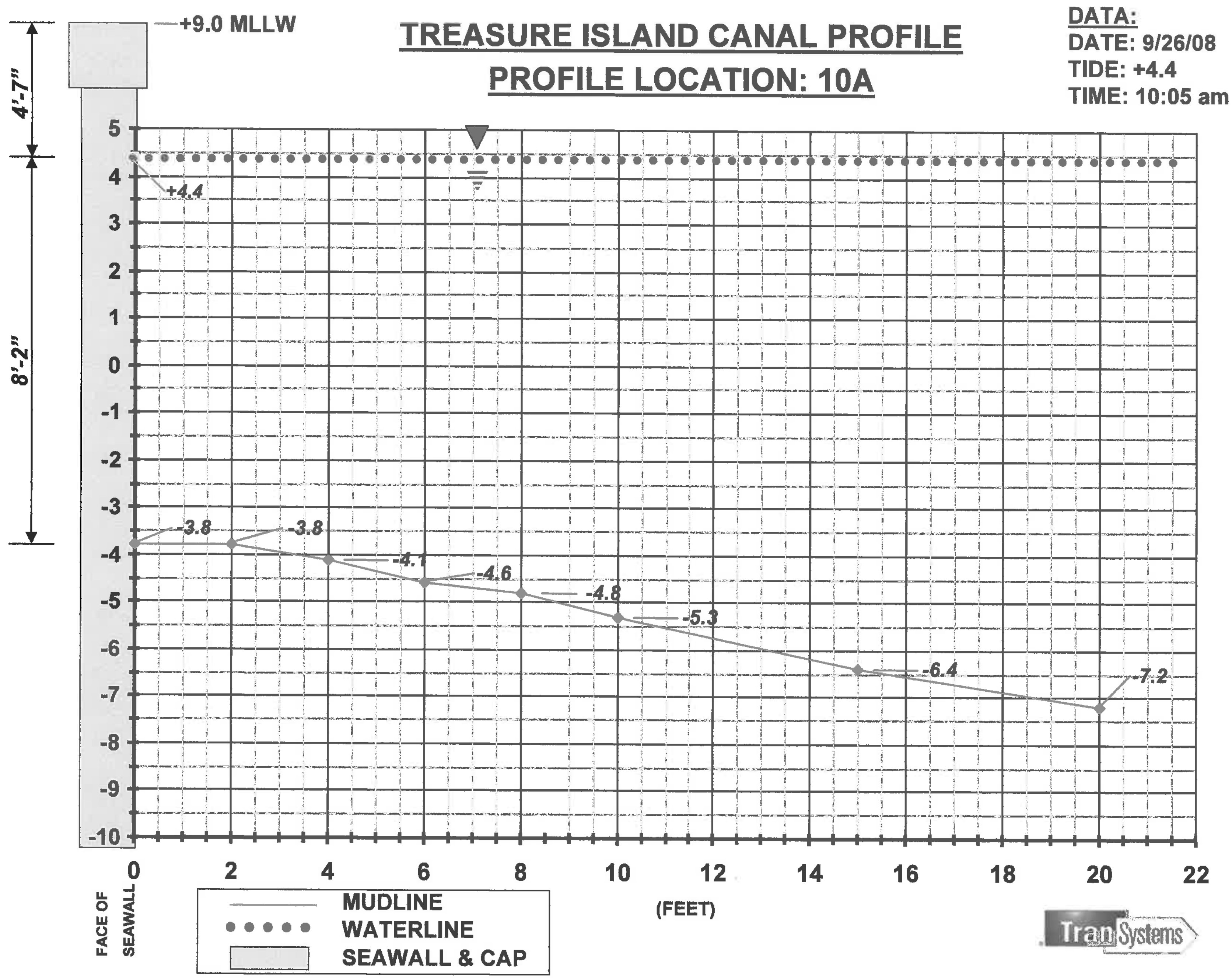

TREASURE ISLAND CANAL PROFILE
PROFILE LOCATION: 10A
DATA:
DATE: 9/26/08
TIDE: +4.4
TIME: 10:05 am
+9.0 MLLW
4'-7"
8'-2"
+4.4
-3.8
-3.8
-4.1
-4.6
-4.8
-5.3
-6.4
-7.2
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP

FIGURE 6-17

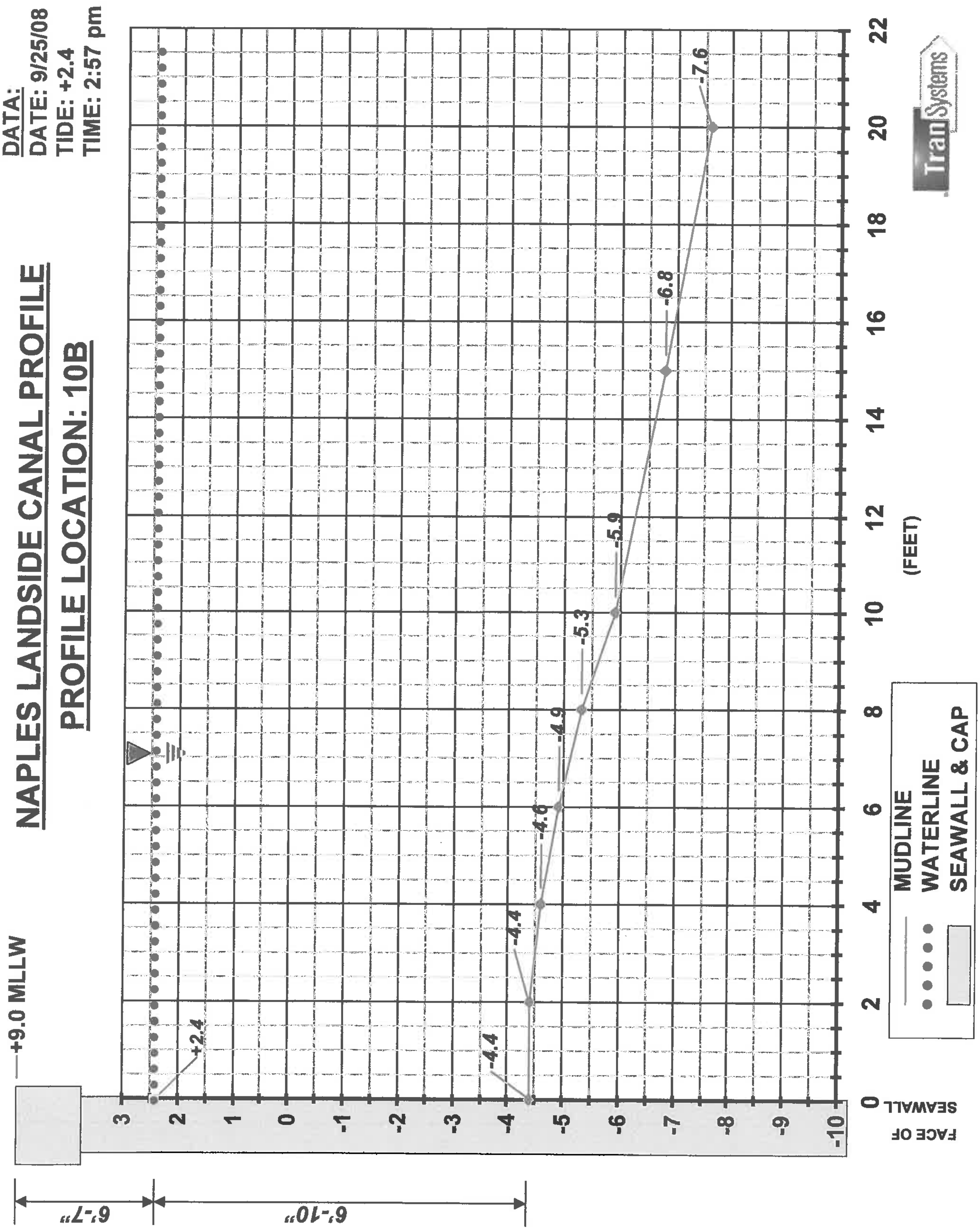
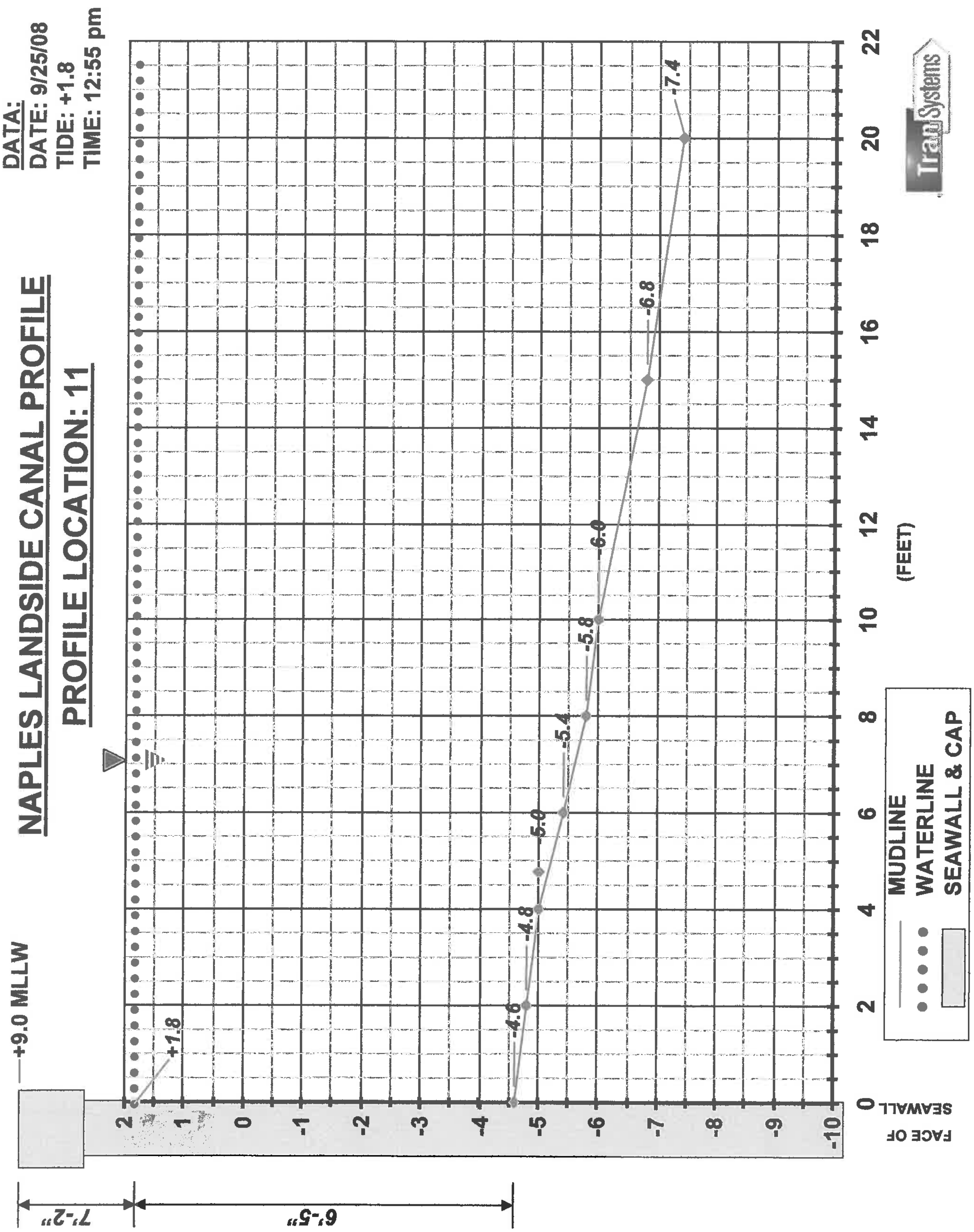

NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 10B
DATA:
DATE: 9/25/08
TIDE: +2.4
TIME: 2:57 pm
+9.0 MLLW
6'-7"
6'-10"
+2.4
-4.4
-4.4
-4.6
-4.9
-5.3
-5.9
-6.8
-7.6
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-18



NAPLES LANDSIDE CANAL PROFILE
PROFILE LOCATION: 11
DATA:
DATE: 9/25/08
TIDE: +1.8
TIME: 12:55 pm
+9.0 MLLW
7'-2"
6'-5"
+1.8
-4.6
-4.8
-5.0
-5.4
-5.8
-6.0
-6.8
-7.4
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP

FIGURE 6-19

NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 12A

DATA:
DATE: 9/25/08
TIDE: +1.8
TIME: 1:05 pm

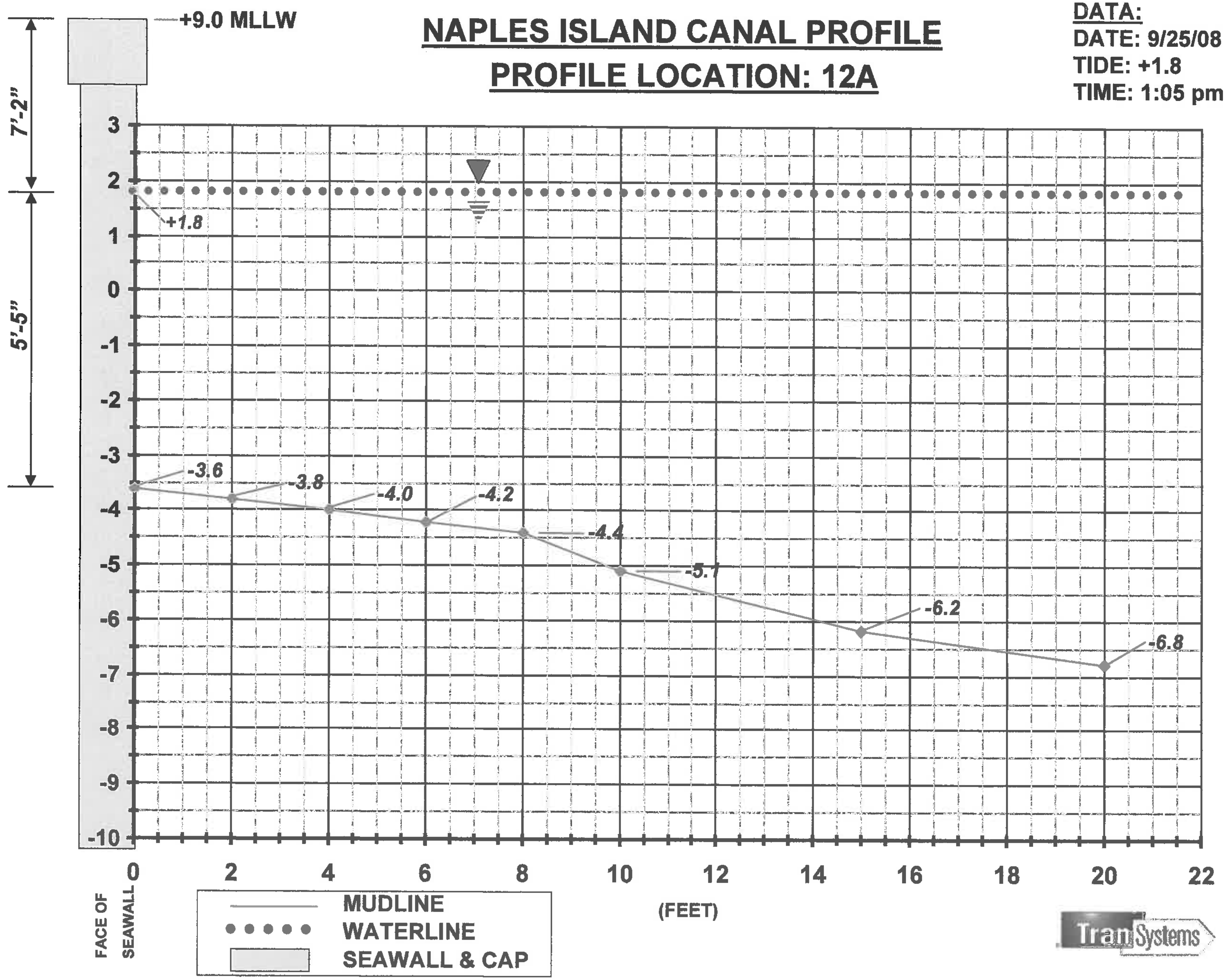
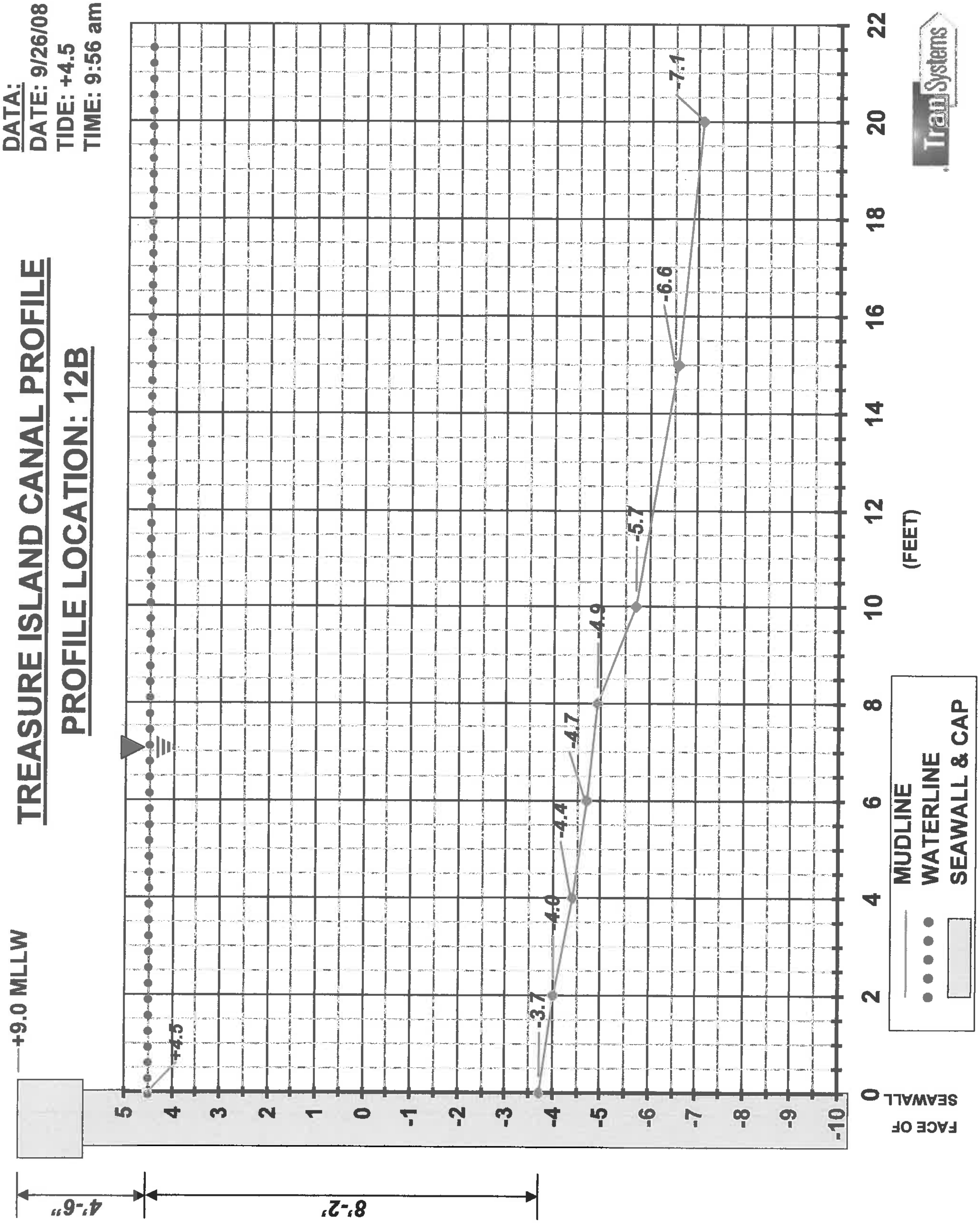
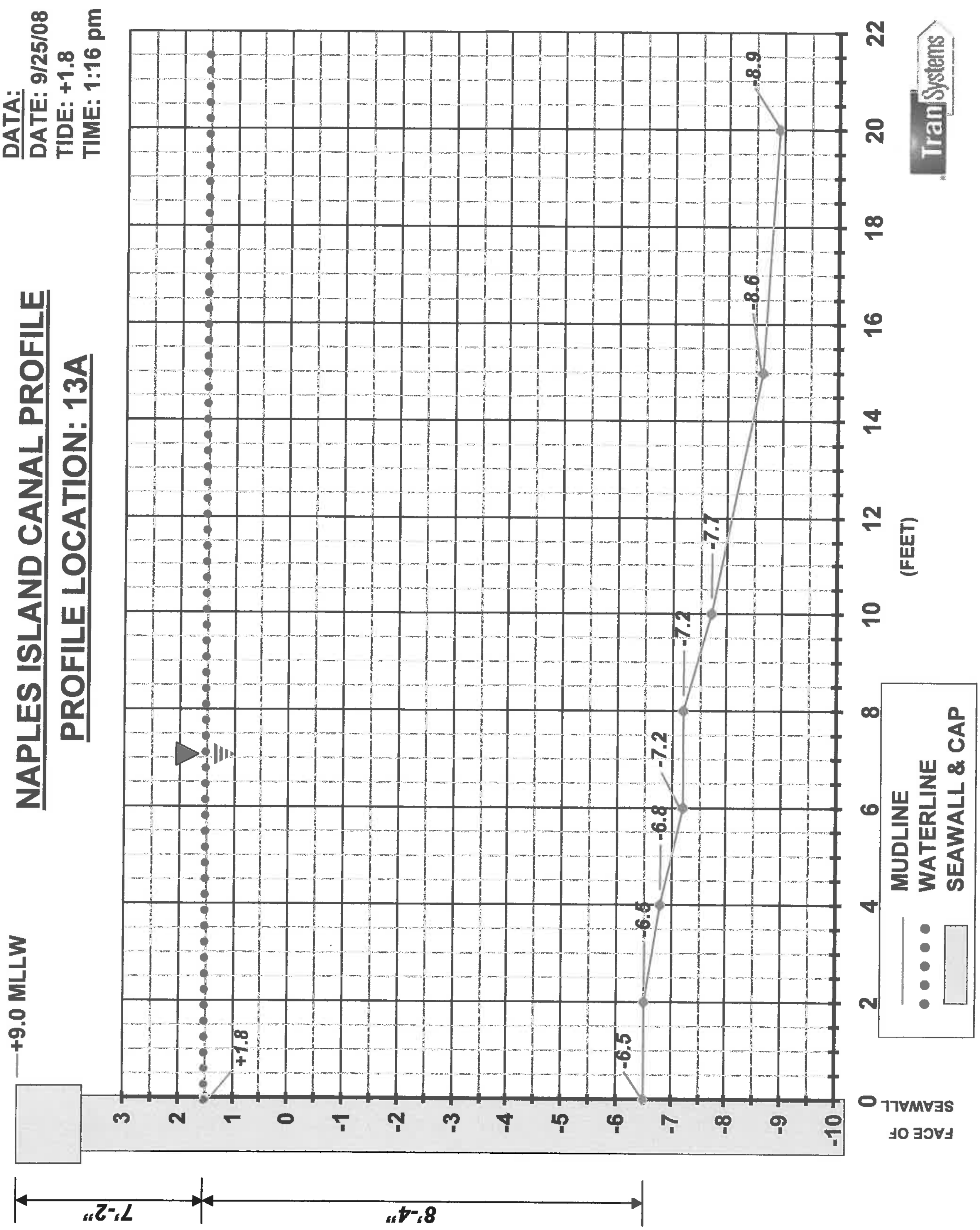


FIGURE 6-20



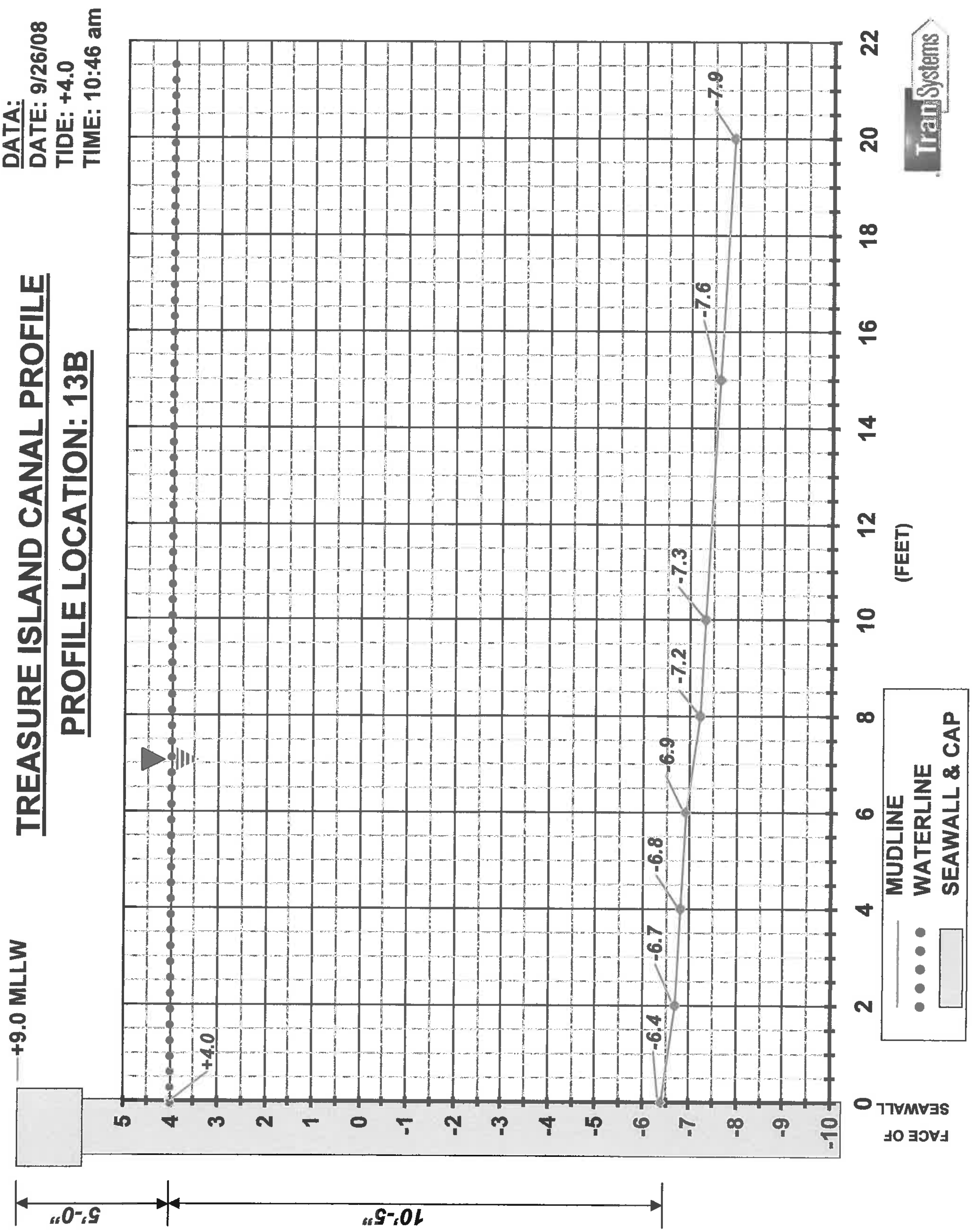
TREASURE ISLAND CANAL PROFILE
PROFILE LOCATION: 12B
DATA:
DATE: 9/26/08
TIDE: +4.5
TIME: 9:56 am
+9.0 MLLW
4'-6"
8'-2'
+4.5
-3.7
-4.0
-4.4
-4.7
-4.9
-5.7
-6.6
-7.1
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-21



NAPLES ISLAND CANAL PROFILE
PROFILE LOCATION: 13A
DATA:
DATE: 9/25/08
TIDE: +1.8
TIME: 1:16 pm
+9.0 MLLW
+1.8
-6.5
-6.5
-6.8
-7.2
-7.2
-7.7
-8.6
-8.9
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
7'-2"
8'-4"

FIGURE 6-22



TREASURE ISLAND CANAL PROFILE
PROFILE LOCATION: 13B
DATA:
DATE: 9/26/08
TIDE: +4.0
TIME: 10:46 am
+9.0 MLLW
+4.0
-6.4
-6.7
-6.8
-6.9
-7.2
-7.3
-7.6
-7.9
5'-0"
10'-5"
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-23

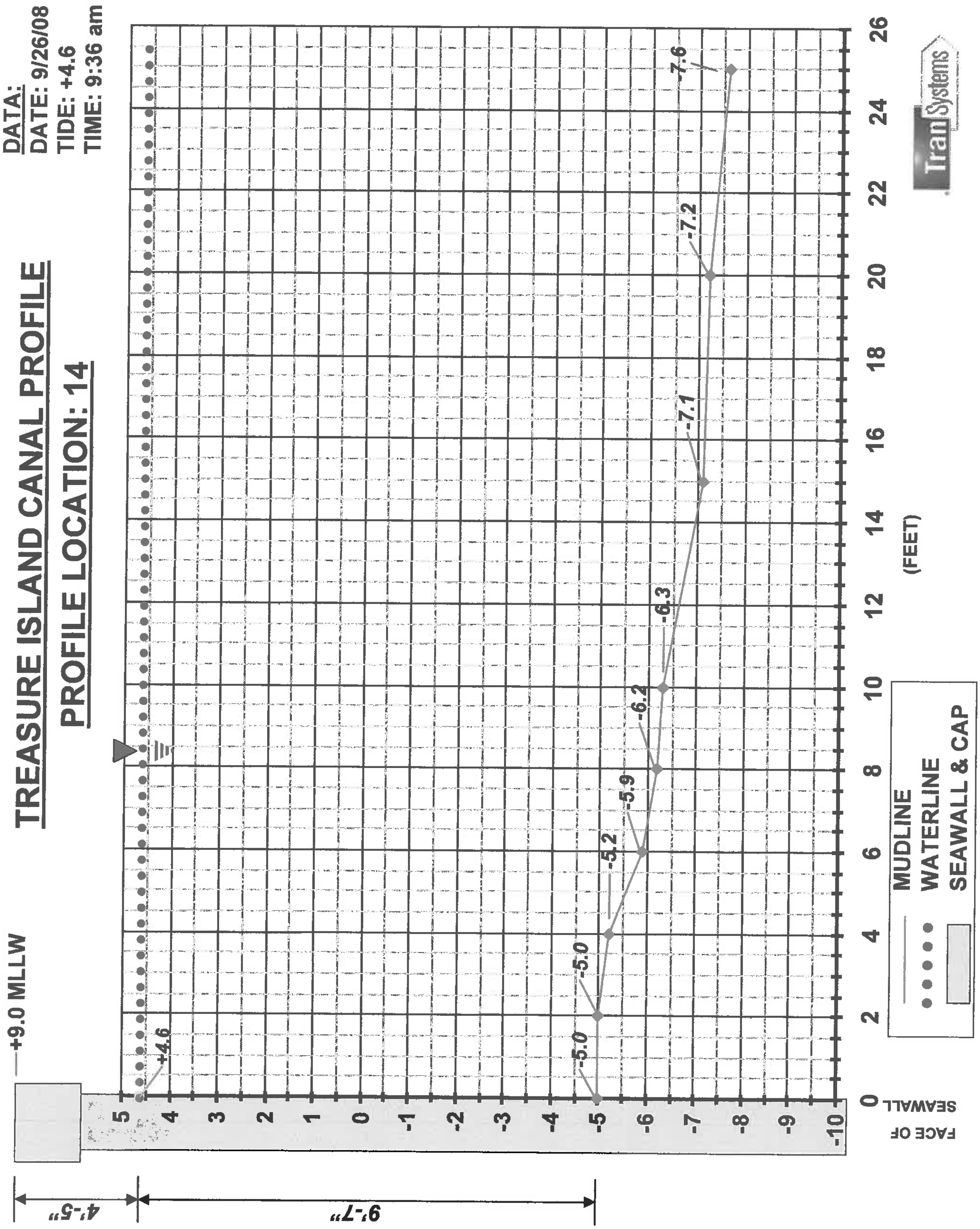

TREASURE ISLAND CANAL PROFILE
PROFILE LOCATION: 14
DATA:
DATE: 9/26/08
TIDE: +4.6
TIME: 9:36 am
+9.0 MLLW
4'-5"
9'-7"
+4.6
-5.0
-5.0
-5.2
-5.9
-6.2
-6.3
-7.1
-7.2
-7.6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
0
2
4
6
8
10
12
14
16
18
20
22
24
26
(FEET)
FACE OF SEAWALL
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-24

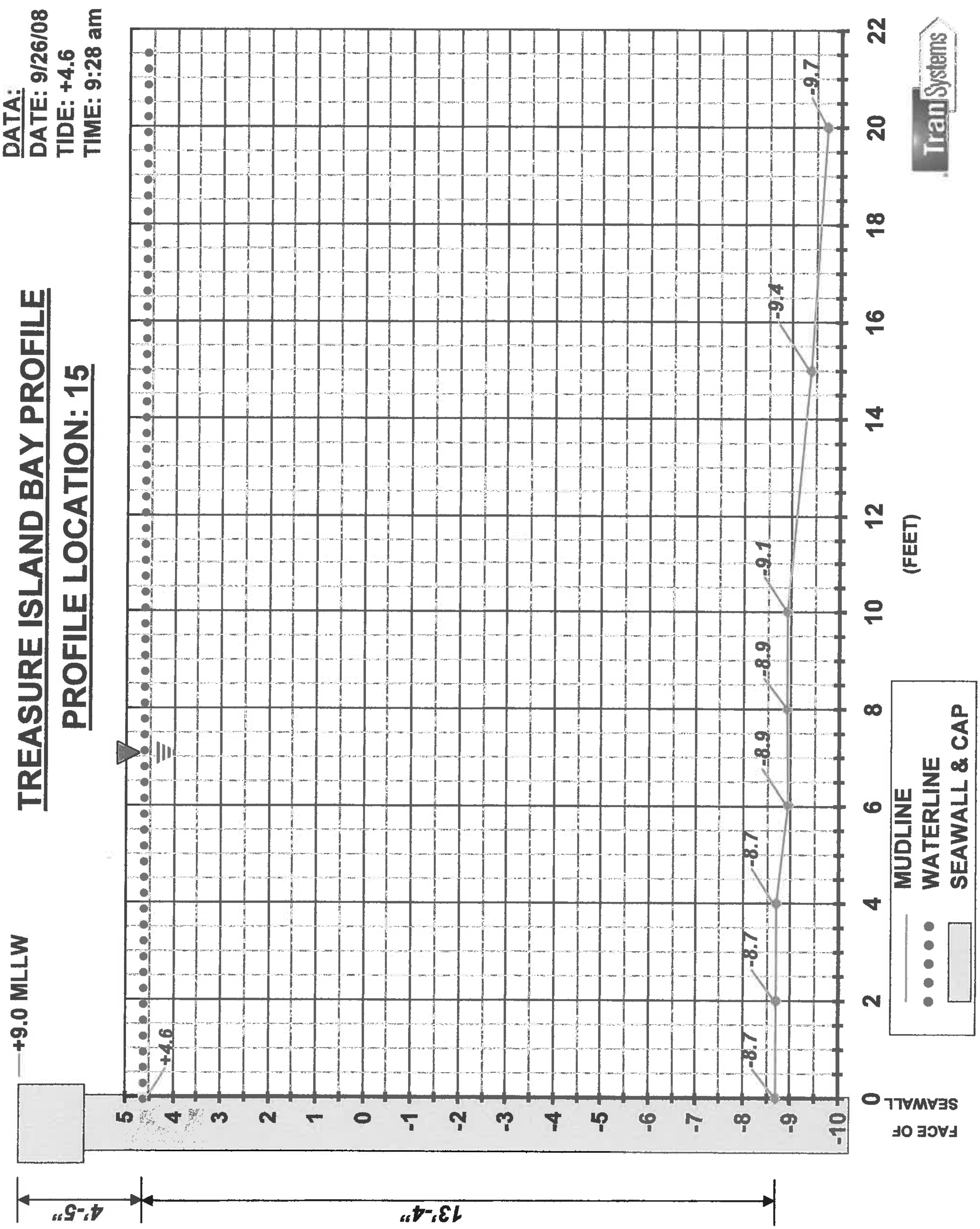

TREASURE ISLAND BAY PROFILE
PROFILE LOCATION: 15
DATA:
DATE: 9/26/08
TIDE: +4.6
TIME: 9:28 am
+9.0 MLLW
+4.6
-8.7
-8.7
-8.7
-8.9
-8.9
-9.1
-9.4
-9.7
MUDLINE
WATERLINE
SEAWALL & CAP
(FEET)
FACE OF SEAWALL
4'-5"
13'-4"

FIGURE 6-25

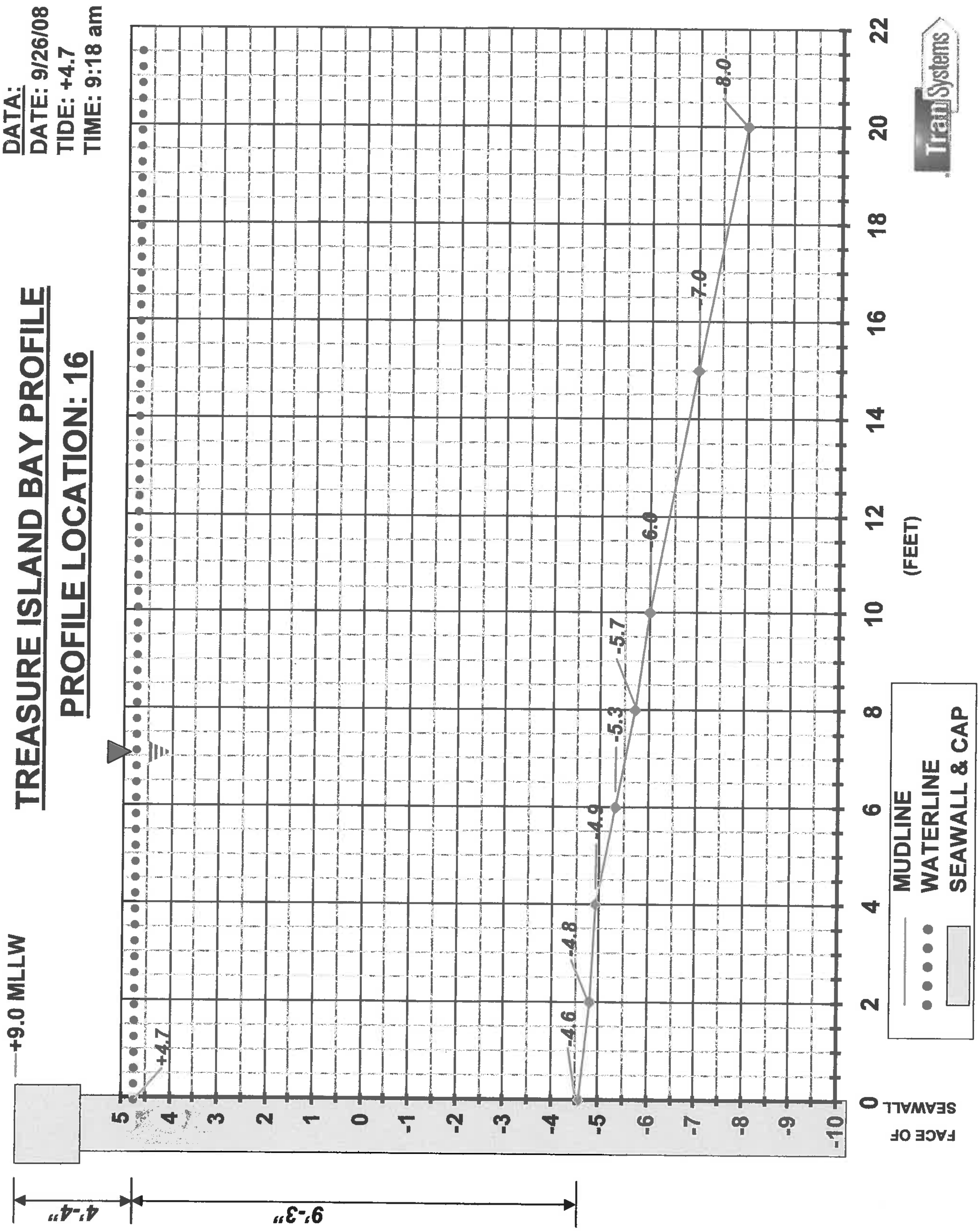

TREASURE ISLAND BAY PROFILE
PROFILE LOCATION: 16
DATA:
DATE: 9/26/08
TIDE: +4.7
TIME: 9:18 am
+9.0 MLLW
4'-4"
9'-3"
+4.7
-4.6
-4.8
-4.9
-5.3
-5.7
-6.0
-7.0
-8.0
FACE OF SEAWALL
(FEET)
MUDLINE
WATERLINE
SEAWALL & CAP
TranSystems

FIGURE 6-26

7.0 Seawall Stability Analysis

The Naples Seawalls are constructed with 3 different length concrete sheet piles. A review of original drawing details, City Drawing number B-761 and an excellent description of the wall construction in an engineering article titled "Precast Concrete Pile for Canal Walls at Long Beach" by John Blackman, November 19, 1940 provide sufficient information of the sheet pile construction that the sheet pile strength and stiffness can be modeled with reasonable accuracy. Efficient engineering design would have necessitated the use of various length sheet piles to address the various dredge depths or anticipated future dredge depths occurring in the project. For this analysis the sheet pile lengths are shown on Figure 7-1, pg 7-7.

This analysis includes 20 Load Cases of typical seawall conditions covering the range of dredge depths and loading configurations occurring in the project and one load case of the specific conditions at Profile 15 on Naples Island where scour is a concern. Scour is also a concern in the canal entrance where all load cases for this small wall area assume a depth 1.5' deeper than the typical canal walls. Siltation or scour are considered localized anomalies for purposes of the Long Range Repair recommendations in Section 8.0 but the two areas of scour mentioned above are addressed by Interim Repairs in Section 8.0. In the case of siltation, the benefit of the additional soil is ignored, since in most cases this is very soft mud of little benefit to brace the wall. In the case of scour, the analysis provides sufficient results for areas with greater dredge depth that the significance of the soil loss can be judged and repairs recommended, if appropriate.

7.1 Load Cases

Each of the four seawall regions (East Bayside; Channel Entrance, Canal & West Bayside) were analyzed for the following 4 loading conditions.

Soil Load Only (Case 1, 5, 9 &13). Soil load is the active soil pressure due to the height of retained earth behind the seawall.

Soil Load + Surcharge (Case 2, 6, 10 & 14). Surcharge is the effect of a 5' wide x 167 pounds per square foot earth filled planter or concrete deck constructed adjacent to the seawall. 167 psf = 1.25' tall planter or deck x 134 pounds per cubic foot (average weight of soil/plants and/or a 4" concrete deck over earth fill). This case is run for a comparison to Cases 1, 5, 9 & 13 to evaluate the relative effect of the raised planters and deck surcharge on wall stability during static conditions. The City may utilize these finding to assist future revisions and enforcement of the Seawall Guidelines, see Page 1-11.

Soil Load + Seismic (Case 3A, 7A, 11A & 15A). Seismic is the additional horizontal load of the retained earth behind the seawall accelerating horizontally due to seismic ground motions. Seismic Load is a very short term increase in the Soil Load and is represented by a load equal to 20X the height of retained earth (see Volume 2, Appendix D, Tab 4, page 4-3, paragraph 3). This case is run for a comparison to Cases 3, 7, 11 & 15 to evaluate the relative effect of the raised planters and deck surcharge on wall stability during seismic conditions. The City may utilize these finding to assist future revisions and enforcement of the Seawall Guidelines, see Page 1-11.

Soil Load + Surcharge + Seismic (Case 3, 7, 11 & 15). Seismic is the additional horizontal load of the retained earth behind the seawall accelerating horizontally due to seismic ground motions. Seismic Load is a very short term increase in the Soil Load and is represented by a load equal to 20X the height of retained earth (see Volume 2, Appendix D, Tab 4, page 4-3, paragraph 3).

New Tie Rods (Soil Load + Surcharge + Seismic) (Case 4, 8, 12 & 16). Same as above load case except this analysis considers the effect of new tie rods installed at an elevation of +5', which is 2 feet below the +7' elevation of the existing tie rods. New tie rods are spaced at 4'-8" on center compared to the existing tie rods spacing of 10'-0".

Scour Condition on West Bayside near Channel Entrance (Case 17). This case evaluates a specific localized condition which differs significantly from the remainder of the load cases occurring in the project.

7.2 Failure Modes:

The analysis results were evaluated for 4 possible failure modes.

Soil Sliding – The standard Factor of Safety for passive soil pressure to resist sliding is 1.50 for static loads. When short term seismic loads are considered, a Factor of Safety of 1.20 is acceptable. A result between 1.20 and 0.95 represents a potential for failure during a seismic event and a result less than 0.95 represents an expected failure condition. Figure 7-2 tables the analysis results for each load case.

Tie Rod Tension – Steel elements are most commonly designed using "Allowable Stress" capacities which include Factors of Safety which are approved and accepted by the design industry. Building Codes permit a 1/3 increase in "allowable stress" capacities for short term loading conditions such as Seismic. The existing tie rods are 1 3/8" diameter and estimated to be Grade A6, Fy=35ksi steel, in use during that time period, which has an allowable load of 31,080# for static loads and 41,340# for load cases which include seismic. Tension values which exceed these limits for static or seismic conditions have a potential for failure. Existing Tie-rod conditions are believed to range from Fair to Very Poor, as evidenced by the tie-rod survey performed by Concept Marine Associates in 2004. For this reason it is not possible to quantify with accuracy the rod failure load. 1 3/8" diameter tie-rods in Good condition are expected to fail at 46,600# for all load cases. 1 ½" diameter tie-rods, used at the 26' long sheet piles, are expected to fail at 55,500# for all load cases if in Good condition. Figure 7-2 tables the tension load on the tie rods for each load case.

Concrete Sheet Pile Bending – Concrete elements are most commonly designed using "Ultimate Strength" capacities which include Factors of Safety built into the coefficients used in the calculation of both the demand load and the allowable load. Figure 7-2 tables the ratio of the bending Moment Capacity vs. the maximum bending Moment Demand in the sheet pile. A ratio below 1.00 represents a Factor of Safety less than prescribed by the Code. A ratio between 0.95 and 0.75 are considered as have a significant potential to fail and reinforced concrete in new condition will fail at a ratio near 0.75 for all load cases. Note: The existing concrete sheet piling has areas of significant concrete sulfate deterioration and reinforcing corrosion deterioration which should be considered in any interpretation of the analysis results.

Concrete Sheet Pile Shear - Concrete elements are most commonly designed using "Ultimate Strength" capacities which include Factors of Safety built into the coefficients used in the calculation of both the demand load and the allowable load. Figure 7-2 tables the ratio of concrete Shear Capacity vs. the maximum Shear Demand. A ratio below 1.00 represents a Factor of Safety less than prescribed by the Code. A ration between 0.95 and 0.75 are considered as having a significant potential for failure and Reinforced concrete in new condition will fail at a ratio near 0.75 for all load cases. Note: The existing concrete sheet piling has localized areas of significant concrete sulfate deterioration which should be considered in any interpretation of the analysis results.

7.3 Seawall Stability Analysis Conclusions

7.3.1 East Bayside Segment - Results of Load Cases 1, 2, 3A, 3 & 4

Sliding - This segment has an acceptable Factor of Safety for sliding for all load cases. The Factors of Safety for sliding for Load Case 2 is 1.46 which is just below the standard for new design and represents a very low risk of failure under static conditions.

Surcharge Effect - The additional surcharge weight has the net effect of increasing all stresses by an average of 12% and the Tension Rod load by 5,400#.

Tie Rod Tension – Demand exceeds the failure point of a new rod by 18% for Load Case 3 and 6% for Load Case 3A. Tie Rod failure is expected in a major earthquake due to the condition of the rods. The Interim Repair of new tie rods, as shown in Figures 8-2 and 8-3 and Load Case 4, would satisfactorily mitigate this risk.

Sheet Pile Bending has an acceptable Factor of Safety for all load cases.

Sheet Pile Shear has an acceptable Factor of Safety for static load cases 1 & 2 but show a potential for failure during seismic load cases 3. The addition of new tie rods, as described above, would reduce this shear by 7%. No other work is recommended.

The East Bayside was analyzed for 24' sheet piles which cover the majority of the region. The only significant shortcoming of this region is the tiebacks. The recommendation for new tie-backs is appropriate for both the 24' and 26' sheets which comprise the East Bayside seawall when consideration is given that numerous tiebacks are expected to be in poor condition.

7.3.2 Channel Entrance Segment – Results of Load Cases 5, 6, 7A, 7 & 8

Sliding - This segment has a Factor of Safety for sliding of 1.10 and 0.97 for static load cases 5 & 6 respectfully. These are below the standard for design and represent a minor risk of failure if additional scour occurs unabated. The Factor of Safety for Load case 7 is 0.76 which represents significant potential for failure during a major earthquake. The addition of new tie rods lessens this risk by only a few percentage points, which is still unacceptable by most geotechnical standards. We recommend that Interim Repair rock protection be installed in the areas where scour is occurring, see Figure 8-10.

Surcharge Effect - The additional surcharge weight has the net effect of increasing all stresses by an average of 13% and the Tension Rod load by 5,400#.

Tie Rod Tension – Demand exceed Failure point of a new rod by 33% & 45% for Load Case 7A & 7, respectfully. Tie Rod failure is expected. The Interim Repair of new tie rods, as shown in Figures 8-2 and 8-3 and as shown in Load Case 8, would satisfactorily mitigate this risk.

Sheet Pile Bending has a ratio of Capacity/Demand of 0.66 and .63 for Load Case 7A & 7, respectfully, which represents a failure condition during a major earthquake. Failure is likely considering the effects of minor reinforcing corrosion, typical of most sheet piles. The combination of rock protection, new tiebacks and removal of surcharge loads can significantly reduce the risk of failure.

Sheet Pile Shear has a ratio of Capacity/Demand of 0.82 & 0.72 for Load Case 7A & 7, respectfully, which indicates that a failure is expected to occur during a major earthquake. The addition of new tiebacks, as shown by Load Case 8, only mitigates this ratio to a 0.79, with is still unacceptable. We recommend strengthening by a combination of rock protection, new tiebacks and removal of surcharge loads, if wall replacement is delayed.

We recommend that this segment be replaced on a higher priority basis due to these findings and the importance of uninterrupted boater access at the channel entrance.

7.3.3 Canal Segment – Results of Load Cases 9, 10, 11A, 11 & 12

Sliding - This segment has a Factor of Safety for sliding of 1.41 and 1.24 for static load cases 9 & 10 respectfully. These are below the standard for new design but represent a low risk of failure, in consideration that none of the approximate 2 miles of sheet piles have failed in sliding since construction 75 years ago. The Factor of Safety for Load case 11A and 11 are 1.13 & 1.02, respectfully, which represents minor potential of failure during a major earthquake. The addition of newer tie rods on the Naples Island portion and some areas of the Landside portion results in a small reduction of risk by improving the Factor of Safety to 1.05. This is still considered marginally acceptable by most geotechnical standards. This condition is a low priority when compared to the numerous other shortcomings requiring funding. No repair recommendations are made to address this sole issue.

Surcharge Effect - The additional surcharge weight has the net effect of increasing all stresses by an average of 14% and the Tension Rod load by 5,400#.

Tie Rod Tension – Demand exceed Failure point of a new rod by 6% & 18% for Load Case 11A & 11, respectfully. Tie Rod failure is expected considering the condition of the rods. The addition of newer tie rods on Naples Island has mitigated this risk to an acceptable level. The Interim Repair of new tiebacks as shown on Figures 8-2 & 8-3 installed on the remainder of the canal walls would satisfactorily mitigate this risk, as shown in Load Case 12.

Sheet Pile Bending has a ratio of Capacity/Demand of 0.93 & 0.87 for Load Case 11A & 11 respectfully, which is below the standard Factor of Safety and nearing the expected failure point at 0.75. Failures are expected to occur at the areas weakened by sulfate deterioration and reinforcing corrosion. This weakness can be mitigated to a satisfactory level by the addition of new tiebacks at areas of concrete in fair condition but new tiebacks will not provide a satisfactory solution for the regions with significant sulfate deterioration. If wall replacement is delayed, we recommend an Interim repair of the installation of an underwater bulkhead to relieve stress from the seawall, see Figure 8-5.

Sheet Pile Shear has a ratio of Capacity/Demand of 0.88 for Load Case11 for the canal walls without newer tiebacks, which is below the standard Factor of Safety. The addition of new tiebacks to the remaining sheet piles on the Landside of the canals, as shown by Load Case 12 mitigates this condition to an acceptable level. New tiebacks will not provide a satisfactory solution for the regions with significant sulfate deterioration. . If wall replacement is delayed, we recommend an Interim repair of the installation of an underwater bulkhead to relieve stress from the seawall, see Figure 8-5.

7.3.4 West Bayside Segment – Results of Load Cases 13, 14, 15A, 15, 16 & 17

Sliding - This segment has a Factor of Safety for sliding of 1.23 and 1.13 for static load cases 13 & 14 respectfully. These are below the standard for new design but represent a low risk of failure, in consideration that none of the approximate 2 miles of sheet piles have failed in sliding since construction 75 years ago. The Factor of Safety for Load case 15 is 0.90 which represents an expected failure during a major earthquake. The addition of new tie rods, as shown in Load Case 16, improves the Factor of Safety on 3%. Consideration should be given by the City to request removal of surcharge loads from this segment of the seawall to gain a 9% improvement. An acceptable option would be the placement of a few feet of rock protection to reduce the unbraced wall height similar to Figure 8-10.

Surcharge Effect - The additional surcharge weight has the net effect of increasing all stresses by an average of 9% and the Tension rod force by 5,400#.

Tie Rod Tension – Demand exceed Failure point of a new rod by 36% & 46% for Load Case 15A & 15, respectfully. This assumes 1 ½" diameter tie-rods. Considering that half of this wall has 1 3/8" diameter rods and that many rods are deteriorated, tie-rod failure is expected during a major earthquake and is also possible during a moderate earthquake. The addition of new tie rods, as shown in Figures 8-2 & 8-3 and supported by the calculations for Load Case 16, satisfactorily mitigate this risk

Sheet Pile Bending has a ratio of Capacity/Demand of 0.77 for Load Case 15 which is very close to the expected failure point of 0.75. Failures are expected, when consideration is given to account for the typical minor reinforcing corrosion conditions. This weakness can be mitigated to a satisfactory level, 0.95, by the addition of new tiebacks or rock protection as shown by Load Case 16.

Sheet Pile Shear has a ratio of Capacity/Demand of 0.60 for Load Case15. Failure is expected during a major earthquake and also possible during a moderate earthquake. The addition of new tiebacks improves this condition to 0.66 which is still unsatisfactory. We recommend that this segment be replaced on a higher priority basis and consideration be given to strengthening by a combination of rock protection/new tiebacks and/or removal of surcharge loads, if wall replacement is delayed.

Scour Condition near channel entrance. The findings of Load Case 17 indicate that this localized area, see Figure 6-2, may become unstable during static conditions if scour continues unabated. We recommend the placement of rock protection per Figure 8-10 on a higher priority basis.

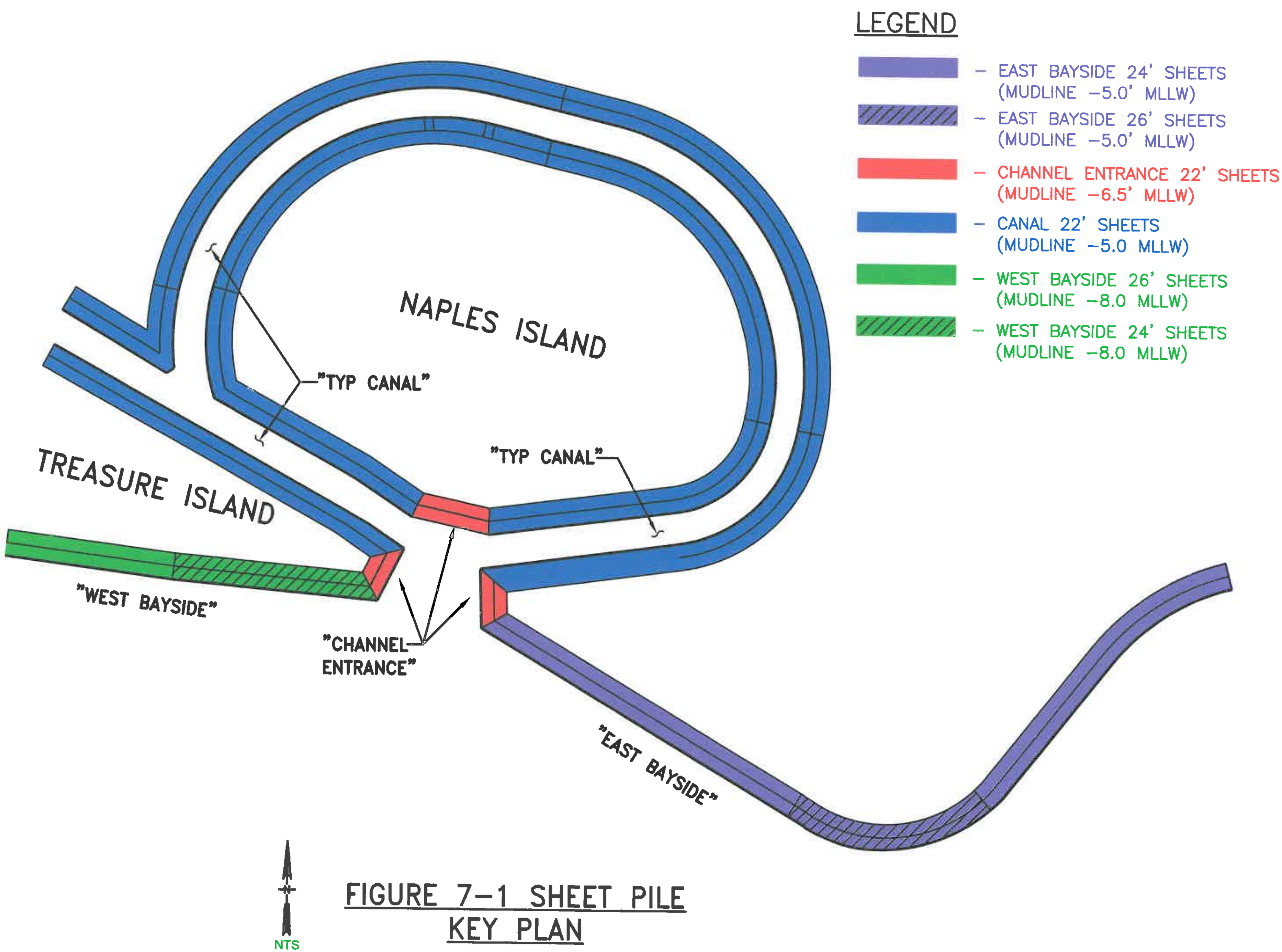
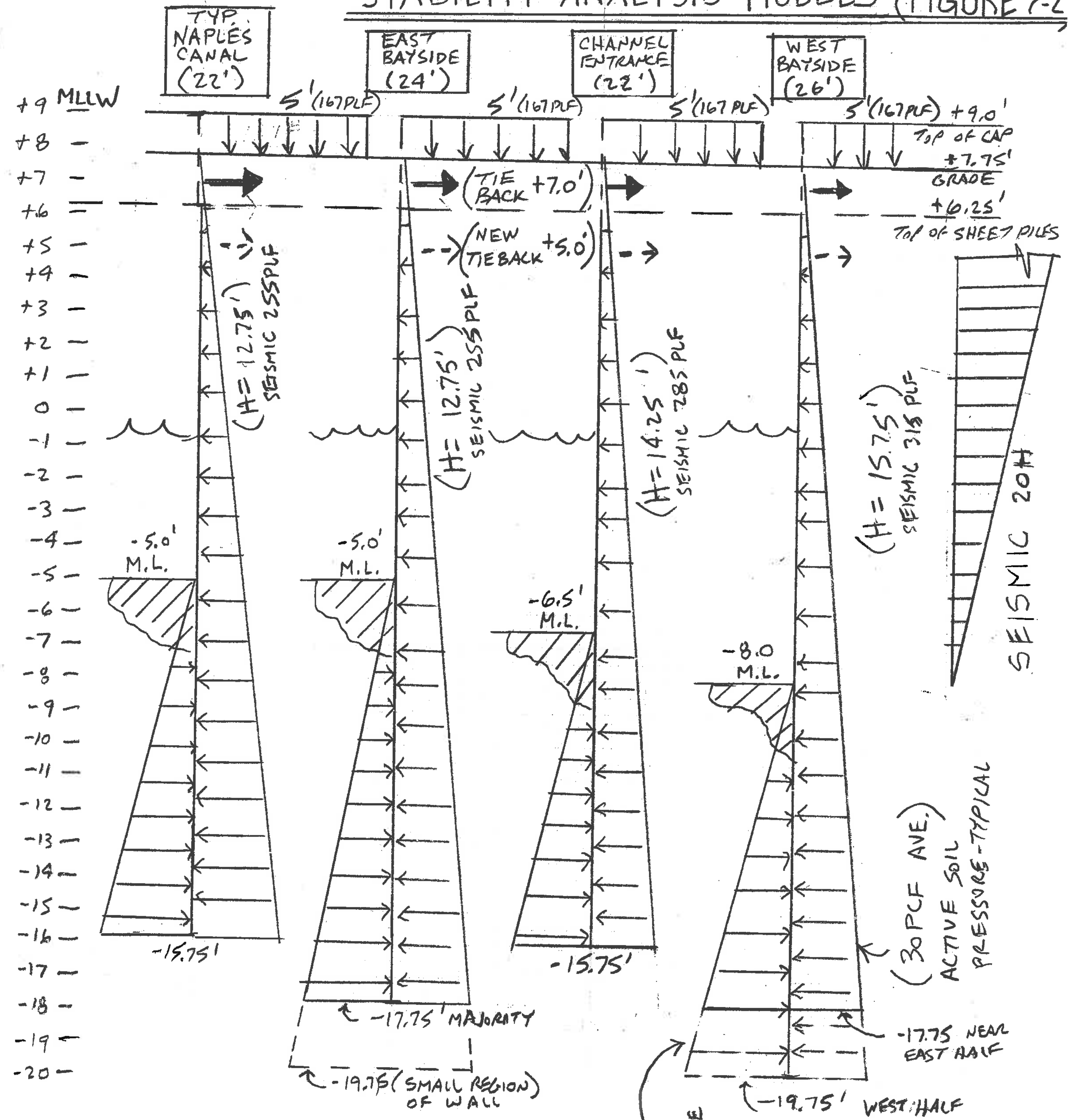


FIGURE 7–1 SHEET PILE KEY PLAN

STABILITY ANALYSIS MODELS (FIGURE 7-2)



LOAD CASES

1) D+L
2) D+L + SURCHARGE
3) D+L + SURCHARGE + SEISMIC
3A) DL + SEISMIC
4) NEW TIEBACKS

FIG 7-2

(FIGURE 7-2)

Calculations for 1/8 US Grid

SUMMARY OF SEAWALL STABILITY ANALYSIS (FIGURE 7-3)

(COMPUTER ANALYSIS BY "RETAIN WALL PRO 6.0")

LOAD CASE	MODEL DESCRIPTION	SLIDING F. O. S.	TIE ROD TENSION	SHEET PILE BENDING	SHEET PILE SHEAR
1	EAST BAYSIDE SEAWALL (24' SHEETS), MUDLINE -5.0', NO DECK SURCHARGE				
		1.63	5,420#	OK	OK
2	EAST BAYSIDE SEAWALL (24' SHEETS), MUDLINE -5.0', WITH DECK SURCHARGE				
		1.46	10,870#	OK	OK
3A	EAST BAYSIDE SEAWALL (24' SHEETS), MUDLINE -5.0' SOIL + SEISMIC				
		1.35	49,500#	OK	OK
3	EAST BAYSIDE SEAWALL (24' SHEETS), MUDLINE -5.0', SOIL+ SURCHARGE + SEISMIC				
		1.23	54,950#	OK	0.88
4	EAST BAYSIDE NEW TIE-BACKS (24' SHEETS), MUDLINE -5.0', SOIL + SURCHARGE + SEISMIC				
		1.27	26,820#	OK	0.95
5	CHANNEL ENTRANCE SEAWALL (22' SHEETS), MUDLINE -6.5', NO DECK SURCHARGE				
		1.10	6,690#	OK	OK
6	CHANNEL ENTRANCE SEAWALL (22' SHEETS), MUDLINE -6.5', WITH DECK SURCHARGE				
		0.97	12,140#	OK	OK
7A	CHANNEL ENTRANCE SEAWALL (22' SHEETS), MUDLINE -6.5', SOIL + SEISMIC				
		0.83	61,980#	0.66	0.82
7	CHANNEL ENTRANCE SEAWALL (22' SHEETS), MUDLINE -6.5', SOIL+ SURCHARGE + SEISMIC				
		0.76	67,430#	0.63	0.72
8	CHANNEL ENTRANCE NEW TIE-BACKS (22' SHEETS), MUDLINE -6.5', SOIL + SURCHARGE + SEISMIC				
		0.79	32,780#	0.78	0.79
9	TYP CANAL SEAWALL (22' SHEETS), MUDLINE -5.0', NO DECK SURCHARGE				
		1.41	5,420#	OK	OK
10	TYP CANAL SEAWALL (22' SHEETS), MUDLINE -5.0', WITH DECK SURCHARGE				
		1.24	10,870#	OK	OK
11A	TYP CANAL SEAWALL (22' SHEETS), MUDLINE -5.0', SOIL + SEISMIC				
		1.13	49,500#	0.93	OK
11	TYP CANAL SEAWALL (22' SHEETS), MUDLINE -5.0', SOIL + SURCHARGE + SEISMIC				
		1.02	54,950#	0.87	0.88
12	TYP CANAL NEW TIE-BACKS (22' SHEETS), MUDLINE -5.0', WITH DECK SURCHARGE + SEISMIC				
		1.05	26,820#	OK	0.95
13	WEST BAYSIDE SEAWALL (26' SHEETS), MUDLINE -8.0', NO DECK SURCHARGE				
		1.23	8,100#	OK	OK
14	WEST BAYSIDE SEAWALL (26' SHEETS), MUDLINE -8.0', WITH DECK SURCHARGE				
		1.13	13,550#	OK	OK
15A	WEST BAYSIDE SEAWALL (26' SHEETS), MUDLINE -8.0', SOIL + SEISMIC				
		0.96	75,520#	0.80	0.67
15	WEST BAYSIDE SEAWALL (26' SHEETS), MUDLINE -8.0', SOIL + SURCHARGE + SEISMIC				
		0.90	80,970#	0.77	0.60
16	WEST BAYSIDE NEW TIE-BACKS (26' SHEETS), MUDLINE -8.0', SOIL + SURCHARGE + SEISMIC				
		0.93	39,160#	0.95	0.66
17	WEST BAYSIDE - SCOUR CONDITION (24' SHEETS), MUDLINE -8.7', SOIL ONLY				
		0.93	9,530	OK	OK

LEGEND

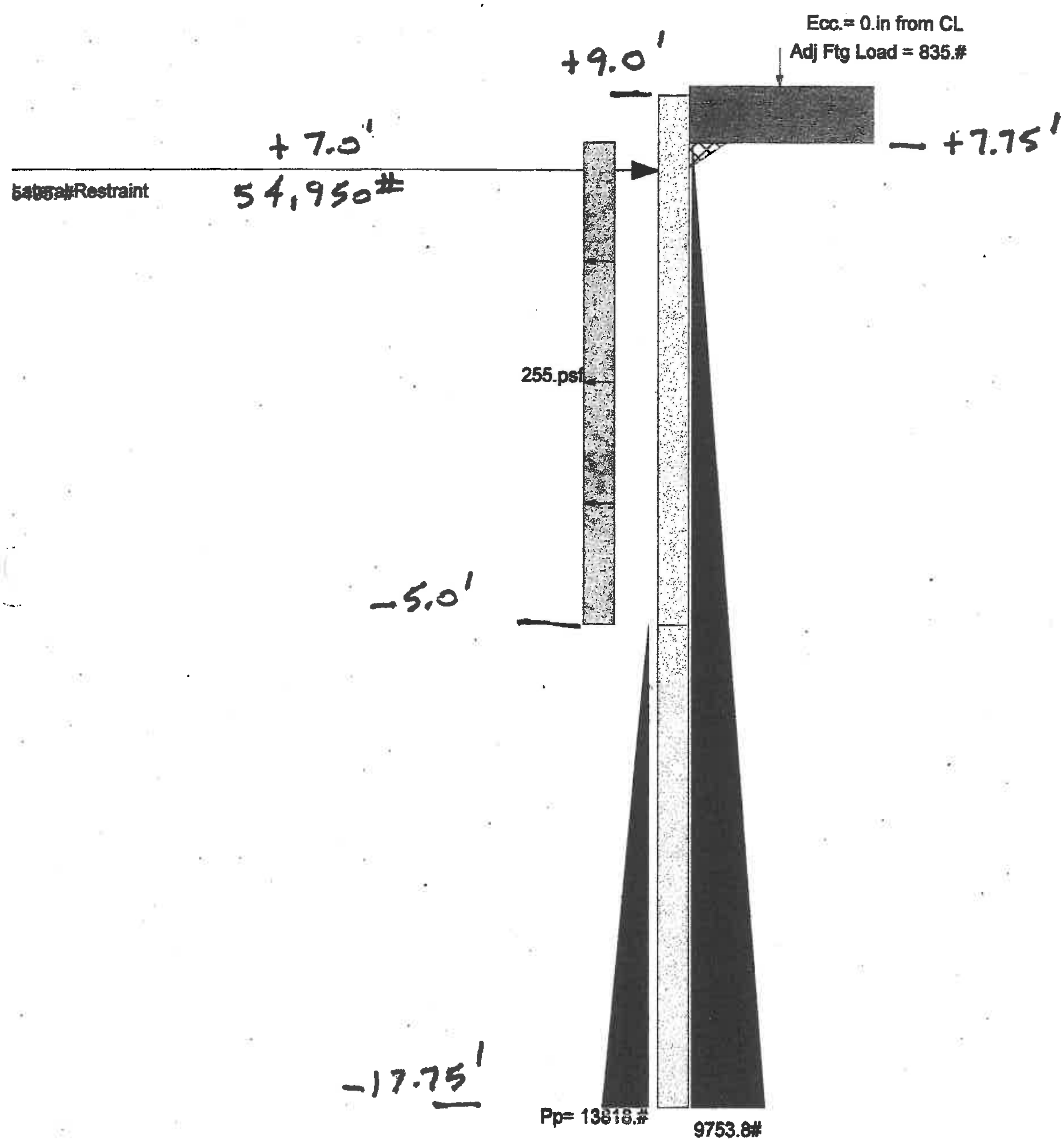
(yellow) INDICATES F. O. S. LESS THAN REQUIRED BY CURRENT CODE BUT FAILURES ARE NOT EXPECTED
CRITERIA: SLIDING FOR STATIC LOAD CASES< 1.5, BENDING <1.0, SHEAR <1.0

(red) INDICATES POTENTIAL FOR FAILURE DURING SEISMIC LOADING
CRITERIA: SLIDING FOR SEISMIC LOAD CASES < 1.2; BENDING<0.95; SHEAR < 0.95

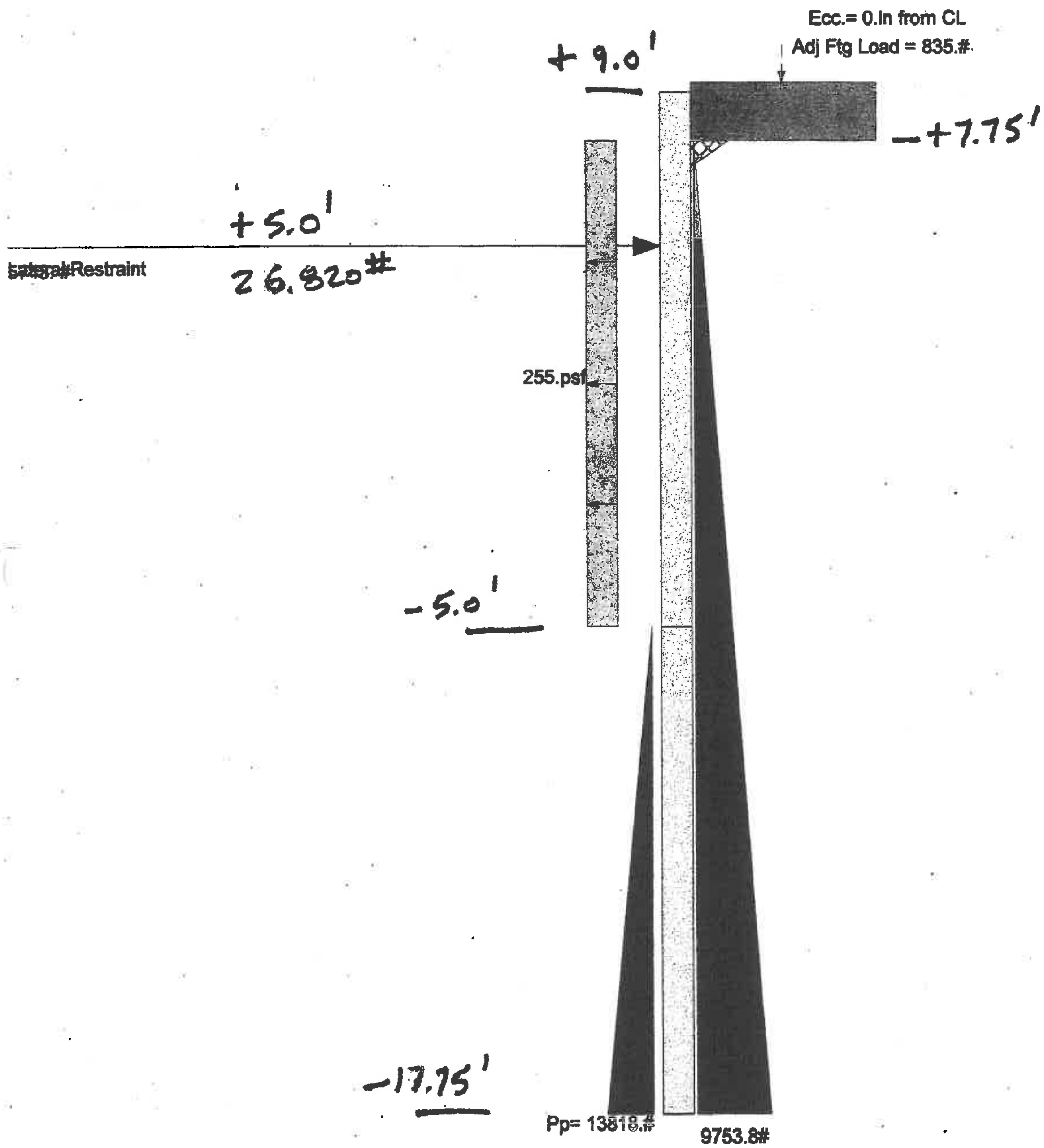
(brown) INDICATES EXPECTED FAILURE DURING SEISMIC LOADING
CRITERIA: SLIDING FOR SEISMIC LOAD CASES < 0.95, BENDING < 0.75, SHEAR < 0.75
TIE ROD LOAD EXCEEDS FAILURE LOAD OF 46,600# @ 22' & 24' , 55,500# @ 26' SHEETS

(blue) INDICATES POTENTIAL SOIL FAILURE UNDER STATIC LOAD CONDTIONS.

EAST BAYSIDE SEAWALL F.B.D.



EAST BAYSIDE W/ NEW TIEBACKS F.B.D.



TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION Page: ______
Job # : CASE 1 Dsgnr: C.DUNCA Date: FEB 22,2009
Description....
EAST BAYSIDE SEAWALL - GRAVITY LOADS, -5' ML, 24' SHEETS
This Wall in File: c:\rp6\naples seawall case 1.rp5

.etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	153.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Design Summary

Total Bearing Load	=	3,344 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	541.9 lbs
Reaction at Bottom	=	9,211.8 lbs
Sliding Stability Ratio	=	1.63 OK
Sliding Calcs		
Lateral Sliding Force	=	9,211.8 lbs
less 100% Passive Force	= -	13,817.8 lbs
less 100% Friction Force	= -	1,170.3 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	0.0 lbs OK

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	12.00 ft	6.75 ft	0.00 ft
Rebar Size	=	# 7	# 7	# 7
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.000	0.171	0.374
Mu....Actual	=	3.6 ft-#	3,000.9 ft-#	6,562.0 ft-#
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#
Shear Force @ this height	=	0.0 lbs		3,224.0 lbs
Shear.....Actual	=	0.00 psi		41.33 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	68.25 in	68.25 in	
Rebar embedment into footing	=			7.75 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	-1,896.5 lbs
Heel Active Pressure	=	-7,315.3
Sliding Force	=	9,211.8 lbs

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem	=			-3,860.0 ft-#
Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	1,750.0 lbs	0.42 ft	729.2 ft-#
Soil Over Heel	=	lbs	0.83 ft	ft-#
Footing Weight	=	1,593.8 lbs	0.42 ft	664.1 ft-#
Total Vertical Force	=	3,343.8 lbs	Base Moment =	-2,466.8 ft-#

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION Page: ______
Job # : CASE 2 Dsgnr: C.DUNCA Date: FEB 22,2009
Description....
EAST BAYSIDE SEAWALL - GRAVITY LOADS W/ SURCHARGE, -5' ML, 24' SHEETS
This Wall in File: C:\RP6\NAPLES SEAWALL CASE 2.RP5

etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	153.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,344 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	1,087.2 lbs
Reaction at Bottom	=	10,252.3 lbs
Sliding Stability Ratio	=	1.46 Ratio < 1.5
Sliding Calcs		
Lateral Sliding Force	=	10,252.3 lbs
less 100% Passive Force	= -	13,817.8 lbs
less 100% Friction Force	= -	1,170.3 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	390.4 lbs NG

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	12.00 ft	6.89 ft	0.00 ft
Rebar Size	=	# 7	# 7	# 7
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.030	0.188	0.432
Mu....Actual	=	519.7 ft-#	3,305.8 ft-#	7,578.2 ft-#
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#
Shear Force @ this height	=	0.0 lbs		4,291.5 lbs
Shear.....Actual	=	0.00 psi		55.02 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	68.25 in	68.25 in	
Rebar embedment into footing	=			7.75 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : Naples Seawall Investigation
Page: ______
Job # : CASE 3A Dsgnr: C. Dunca Date: MAR 15,2009
Description....
WEST BAYSIDE SEAWALL - GRAVITY + SEISMIC. -5 ML, 24' SHEETS
This Wall in File: c:\rp6\naples seawall case 3.rp5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtł	=	0.83
Footing Thickness	=	153.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,344 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	4,949.7 lbs	
Reaction at Bottom	=	11,115.3 lbs	
Sliding Stability Ratio	=	1.35 Ratio	< 1.50
Sliding Calcs			
Lateral Sliding Force	=	11,115.3 lbs	
less 100% Passive Force	= -	13,817.8 lbs	
less 100% Friction Force	= -	1,170.3 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	1,684.9 lbs	NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall	
		Stem OK	Stem OK	Stem OK	
Design height	=	12.00 ft	7.18 ft	0.00 ft	
Rebar Size	=	# 7	# 7	# 7	
Rebar Spacing	=	7.00 in	7.00 in	7.00 in	
Rebar Placed at	=	Edge	Edge	Edge	
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in	
Design Data					
fb/FB + fa/Fa	=	0.007	0.415	0.815	
Mu....Actual	=	125.5 ft-#	7,290.4 ft-#	14,304.1 ft-#	ok!
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#	
Shear Force @ this height	=	0.0 lbs		6,460.0 lbs	
Shear.....Actual	=	0.00 psi		82.82 psi	ok!
Shear.....Allowable	=	85.00 psi		85.00 psi	
Rebar Lap Required	=	68.25 in	68.25 in		
Rebar embedment into footing	=			7.75 in	

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	153.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 40,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 3.00 in @ Btm.= 3.00 in

Surcharge Loads

Surcharge Over Heel = 0.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
Used for Sliding & Overturning

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottorr	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,344 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	5,494.9 lbs	
Reaction at Bottom	=	12,155.9 lbs	
Sliding Stability Ratio	=	1.23	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	12,155.9 lbs	
less 100% Passive Force	= -	13,817.8 lbs	
less 100% Friction Force	= -	1,170.3 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	3,245.7 lbs	NG

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	12.00 ft	7.23 ft	0.00 ft
Rebar Size	=	# 7	# 7	# 7
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.023	0.434	0.872
Mu....Actual	=	397.7 ft-#	7,613.1 ft-#	15,320.2 ft-#
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#
Shear Force @ this height	=	0.0 lbs		7,527.5 lbs
Shear.....Actual	=	0.00 psi		96.51 psi >0.88
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	68.25 in	68.25 in	
Rebar embedment into footing	=			7.75 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	10.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	153.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottorr	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,344 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	5,743.3 lbs	
Reaction at Bottom	=	11,834.8 lbs	
Sliding Stability Ratio	=	1.27	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	11,834.8 lbs	
less 100% Passive Force	= -	13,817.8 lbs	
less 100% Friction Force	= -	1,170.3 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	2,764.1 lbs	NG

Concrete Stem Construction

Thickness = 10.00 in   Fy = 40,000 psi
Wall Weight = 125.0 pcf   f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	10.00 ft	6.39 ft	0.00 ft
Rebar Size	=	# 7	# 7	# 7
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.110	0.369	0.697
Mu....Actual	=	1,936.5 ft-#	6,471.8 ft-#	12,245.0 ft-#
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#
Shear Force @ this height	=	0.0 lbs		6,955.7 lbs
Shear.....Actual	=	0.00 psi		89.18 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	68.25 in	68.25 in	
Rebar embedment into footing	=			7.75 in

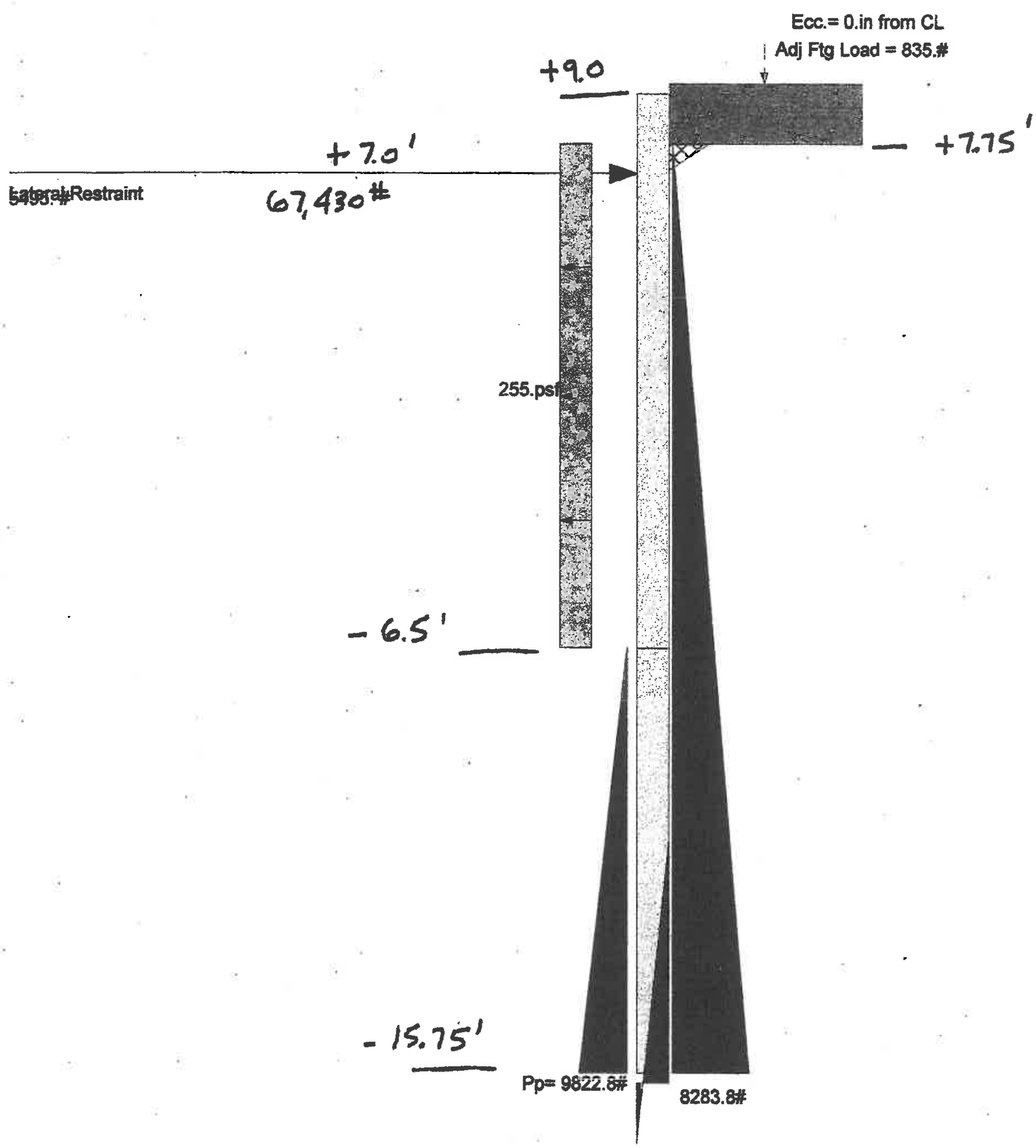
Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

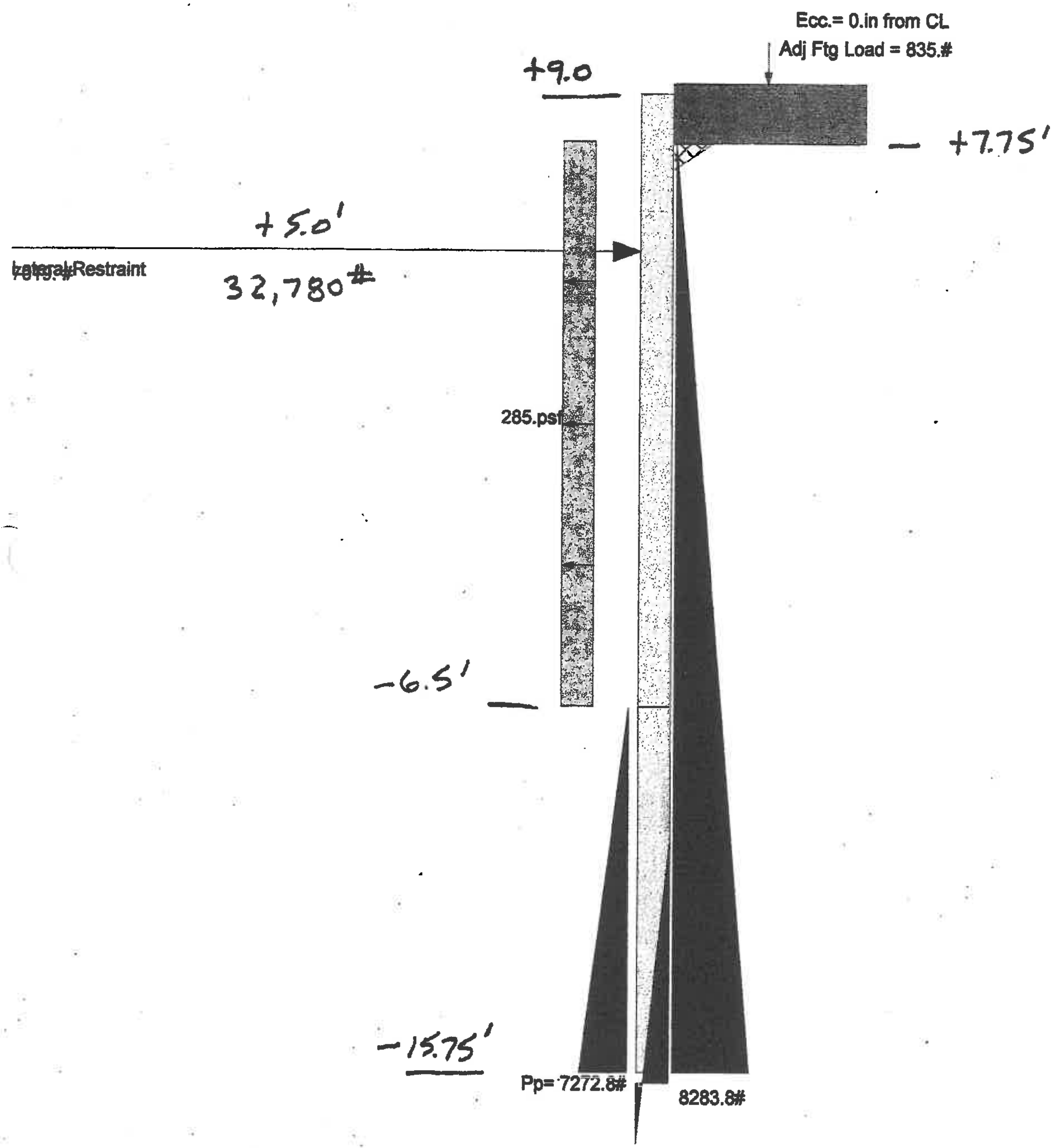
Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

CHANNEL ENTRANCE SEAWALL F.B.D



CHANNEL ENTERANCE WITH
NEW TIEBACKS F.B.D.



TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION   Page: ______
Job # : CASE 5   Dsgnr: C. DUNC.   Date: FEB 22,2009
Description....
CHANNEL ENTRANCE SEAWALL - GRAVITY LOADS -6.5 ML, 22' SHEETS
This Wall in File: c:\rp6\naples seawall case 1.rp5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	14.25 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	15.50 ft
Top Support Height	=	13.50 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtt	=	0.83
Footing Thickness	=	111.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Design Summary

Total Bearing Load	=	3,094 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	669.3 lbs	
Reaction at Bottom	=	7,614.4 lbs	
Sliding Stability Ratio	=	1.10	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	7,614.4 lbs	
less 100% Passive Force	= -	7,272.8 lbs	
less 100% Friction Force	= -	1,082.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	3,066.0 lbs	NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	13.50 ft	7.59 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.000	0.315	0.691
Mu....Actual	=	3.6 ft-#	4,213.8 ft-#	9,235.0 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		4,040.2 lbs
Shear.....Actual	=	0.00 psi		51.80 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	-2,376.6 lbs
Heel Active Pressure	=	-5,237.8
Sliding Force	=	7,614.4 lbs

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem	=			-5,432.4 ft-#
Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	1,937.5 lbs	0.42 ft	807.3 ft-#
Soil Over Heel	=	lbs	0.83 ft	ft-#
Footing Weight	=	1,156.3 lbs	0.42 ft	481.8 ft-#
Total Vertical Force	=	3,093.8 lbs	Base Moment =	-4,143.3 ft-#

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	14.25 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	15.50 ft
Top Support Height	=	13.50 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	111.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,094 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	1,214.2 lbs
Reaction at Bottom	=	8,629.7 lbs
Sliding Stability Ratio	=	0.97 UNSTABL
Sliding Calcs		
Lateral Sliding Force	=	8,629.7 lbs
less 100% Passive Force	= -	7,272.8 lbs
less 100% Friction Force	= -	1,082.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	4,588.9 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	13.50 ft	7.70 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.039	0.337	0.769
Mu....Actual	=	519.7 ft-#	4,509.3 ft-#	10,281.9 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		5,079.4 lbs
Shear.....Actual	=	0.00 psi		65.12 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

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Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	14.25 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	15.50 ft
Top Support Height	=	13.50 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	111.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	285.0 #/ft
...Height to Top	=	14.30 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,094 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	6,197.8 lbs	
Reaction at Bottom	=	10,009.0 lbs	
Sliding Stability Ratio	=	0.83	UNSTABL
Sliding Calcs			
Lateral Sliding Force	=	10,009.0 lbs	
less 100% Passive Force	= -	7,272.8 lbs	
less 100% Friction Force	= -	1,082.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	6,657.9 lbs	NG

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	13.50 ft	8.08 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.012	0.770	1.511
Mu....Actual	=	158.6 ft-#	10,286.7 ft-#	20,195.0 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		8,111.0 lbs
Shear.....Actual	=	0.00 psi		103.99 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

'etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	14.25 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	15.50 ft
Top Support Height	=	13.50 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	111.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	285.0 #/ft
...Height to Top	=	14.30 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,094 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	6,742.7 lbs
Reaction at Bottom	=	11,024.2 lbs
Sliding Stability Ratio	=	0.76 UNSTABL
Sliding Calcs		
Lateral Sliding Force	=	11,024.2 lbs
less 100% Passive Force	= -	7,272.8 lbs
less 100% Friction Force	= -	1,082.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	8,180.7 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	13.50 ft	8.13 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.027	0.793	1.589
Mu....Actual	=	364.6 ft-#	10,601.3 ft-#	21,241.9 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		9,150.2 lbs
Shear.....Actual	=	0.00 psi		117.31 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION Page: ______
Job # : CASE 8 Dsgnr: C. DUNC. Date: MAR 16,2009
Description....
CHANNEL ENTRANCE - NEW TIEBACKS GRAVITY LOADS
W/ SURCHARGE & SEISMIC
This Wall in File: c:\rp6\naples seawall case 4.rp5

.etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	14.25 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	15.50 ft
Top Support Height	=	11.50 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtl	=	0.83
Footing Thickness	=	111.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel = 0.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	285.0 #/ft
...Height to Top	=	14.30 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,094 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	7,018.6 lbs
Reaction at Bottom	=	10,607.6 lbs
Sliding Stability Ratio	=	0.79 UNSTABL
Sliding Calcs		
Lateral Sliding Force	=	10,607.6 lbs
less 100% Passive Force	= -	7,272.8 lbs
less 100% Friction Force	= -	1,082.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	7,555.7 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

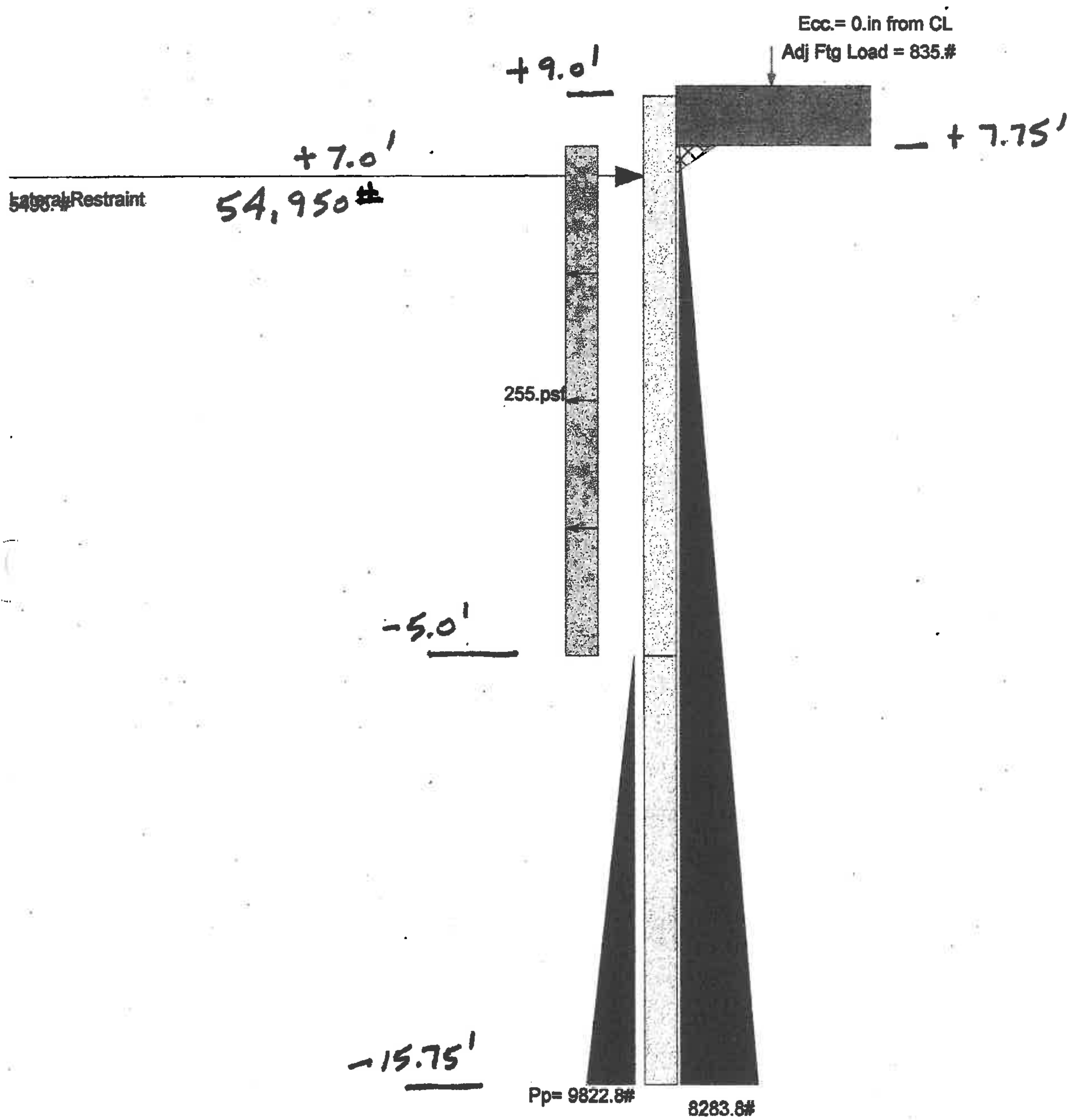
Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	11.50 ft	7.20 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.125	0.664	1.278
Mu....Actual	=	1,676.5 ft-#	8,879.6 ft-#	17,080.9 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		8,419.7 lbs
Shear.....Actual	=	0.00 psi		107.94 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

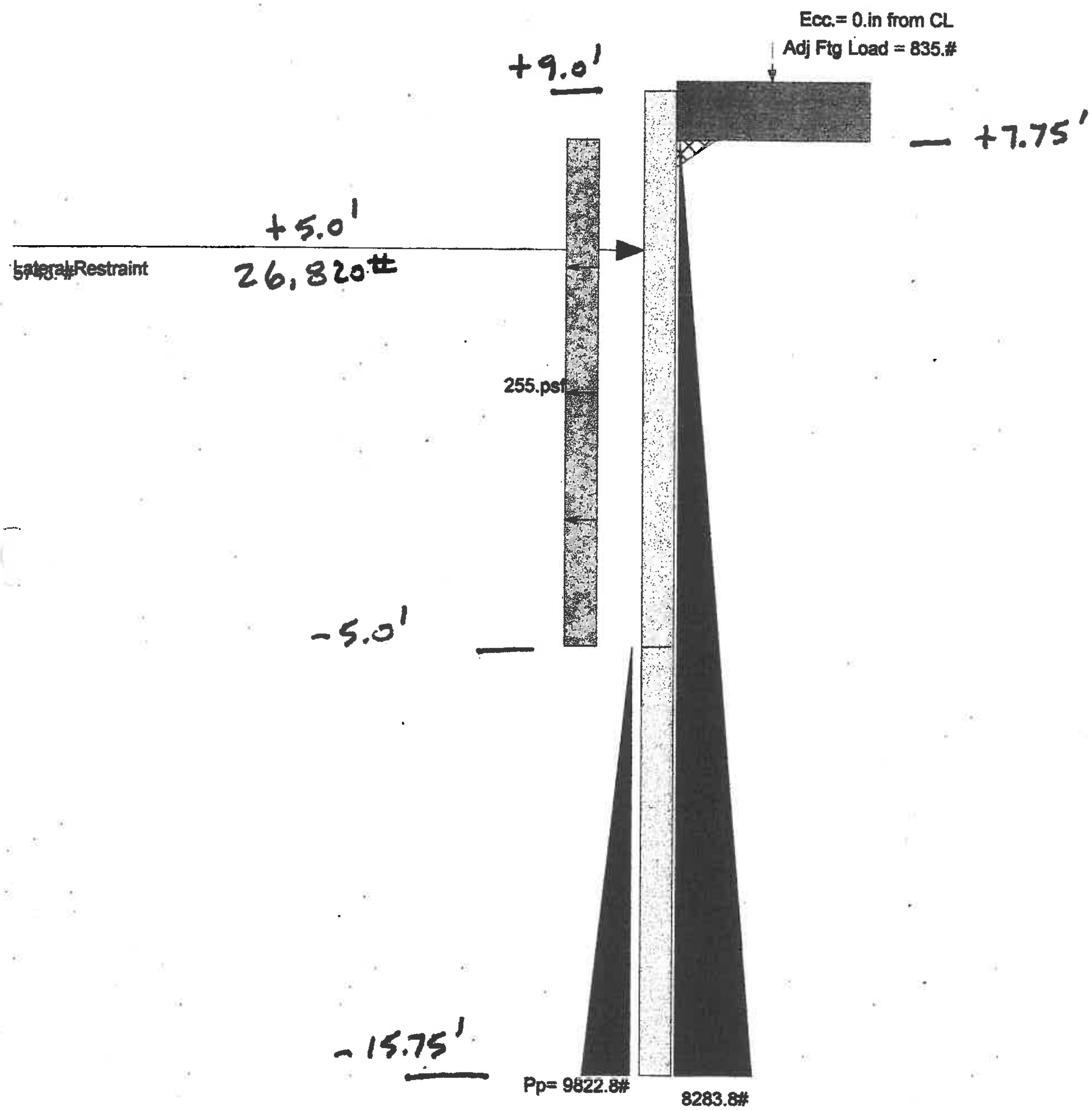
Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

CANAL SEAWALL F.B.D.



CANAL SEAWALL w/ NEW TIEBACK F.B.D.



TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION Page: ______
Job # : CASE 9 Dsgnr: C. DUNC. Date: FEB 22,2009
Description....
NAPELS CANAL SEAWALL - GRAVITY LOADS, -5' ML, 22' SHEETS
This Wall in File: c:\rp6\naples seawall case 1.rp5

etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1.
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	129.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Design Summary

Total Bearing Load	=	3,094 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	541.9 lbs	
Reaction at Bottom	=	7,741.8 lbs	
Sliding Stability Ratio	=	1.41	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	7,741.8 lbs	
less 100% Passive Force	= -	9,822.8 lbs	
less 100% Friction Force	= -	1,082.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	707.1 lbs	NG

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	12.00 ft	6.75 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.000	0.225	0.491
Mu....Actual	=	3.6 ft-#	3,000.9 ft-#	6,562.0 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		3,224.0 lbs
Shear.....Actual	=	0.00 psi		41.33 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	-1,896.5 lbs
Heel Active Pressure	=	-5,845.3
Sliding Force	=	7,741.8 lbs

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem = -3,860.0ft-#

Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	1,750.0 lbs	0.42 ft	729.2 ft-#
Soil Over Heel	=	lbs	0.83 ft	ft-#
Footing Weight	=	1,343.8 lbs	0.42 ft	559.9 ft-#
Total Vertical Force	=	3,093.8 lbs	Base Moment =	-2,571.0 ft-#

TRANSYSTEMS
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Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION   Page: ____
Job # : CASE 10   Dsgnr: C. DUNC,   Date: FEB 22,2009
Description....
NAPELS CANAL SEAWALL - GRAVITY LOADS, W/ SURCHARGE, -5' ML, 22' SHEETS
This Wall in File: C:\RP6\NAPLES SEAWALL CASE 2.RP5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	129.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,094 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	1,087.2 lbs	
Reaction at Bottom	=	8,782.3 lbs	
Sliding Stability Ratio	=	1.24	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	8,782.3 lbs	
less 100% Passive Force	= -	9,822.8 lbs	
less 100% Friction Force	= -	1,082.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	2,267.9 lbs	NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	12.00 ft	6.89 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.039	0.247	0.567
Mu....Actual	=	519.7 ft-#	3,305.8 ft-#	7,578.2 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		4,291.5 lbs
Shear.....Actual	=	0.00 psi		55.02 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : Naples Canal
Job # : CASE 11A Dsgnr: C. Dunca
Page: ______
Date: FEB 22,2009
Description....
Gravity, No Surcharge + Seismic for comparrison to Case 11 - to quantify the effect of surcharge
This Wall in File: c:\rp6\naples seawall case 3.rp5

etainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	129.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottorr	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,094 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	4,949.7 lbs	
Reaction at Bottom	=	9,645.3 lbs	
Sliding Stability Ratio	=	1.13	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	9,645.3 lbs	
less 100% Passive Force	= -	9,822.8 lbs	
less 100% Friction Force	= -	1,082.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	3,562.4 lbs	NG

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	12.00 ft	7.18 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.009	0.545	1.070
Mu....Actual	=	125.5 ft-#	7,290.4 ft-#	14,304.1 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		6,460.0 lbs
Shear.....Actual	=	0.00 psi		82.82 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION
Job # : CASE 11 Dsgnr: C. DUNC, Date: FEB 22,2009 Page: ______
Description....
NAPELS CANAL SEAWALL - GRAVITY LOADS, W/ SURCHARGE & SEISMIC ,-5' ML,22' SHEETS
This Wall in File: C:\RP6\NAPLES SEAWALL CASE 3.RP5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	12.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Surcharge Loads

Surcharge Over Heel = 0.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing‖Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottom	=	0.00 ft

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	129.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 40,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 3.00 in @ Btm.= 3.00 in

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,094 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	5,494.9 lbs
Reaction at Bottom	=	10,685.9 lbs
Sliding Stability Ratio	=	1.02 Ratio < 1.5
Sliding Calcs		
Lateral Sliding Force	=	10,685.9 lbs
less 100% Passive Force	= -	9,822.8 lbs
less 100% Friction Force	= -	1,082.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	5,123.2 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	12.00 ft	7.23 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.030	0.570	1.146
Mu....Actual	=	397.7 ft-#	7,613.1 ft-#	15,320.2 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-# 0.87
Shear Force @ this height	=	0.0 lbs		7,527.5 lbs
Shear.....Actual	=	0.00 psi		96.51 psi
Shear.....Allowable	=	85.00 psi		85.00 psi 0.88
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	12.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	14.00 ft
Top Support Height	=	10.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	129.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	255.0 #/ft
...Height to Top	=	12.75 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,094 lbs
...resultant ecc.	=	0.00 in.
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	5,743.3 lbs
Reaction at Bottom	=	10,364.8 lbs
Sliding Stability Ratio	=	1.05 Ratio < 1.5
Sliding Calcs		
Lateral Sliding Force	=	10,364.8 lbs
less 100% Passive Force	= -	9,822.8 lbs
less 100% Friction Force	= -	1,082.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	4,641.6 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

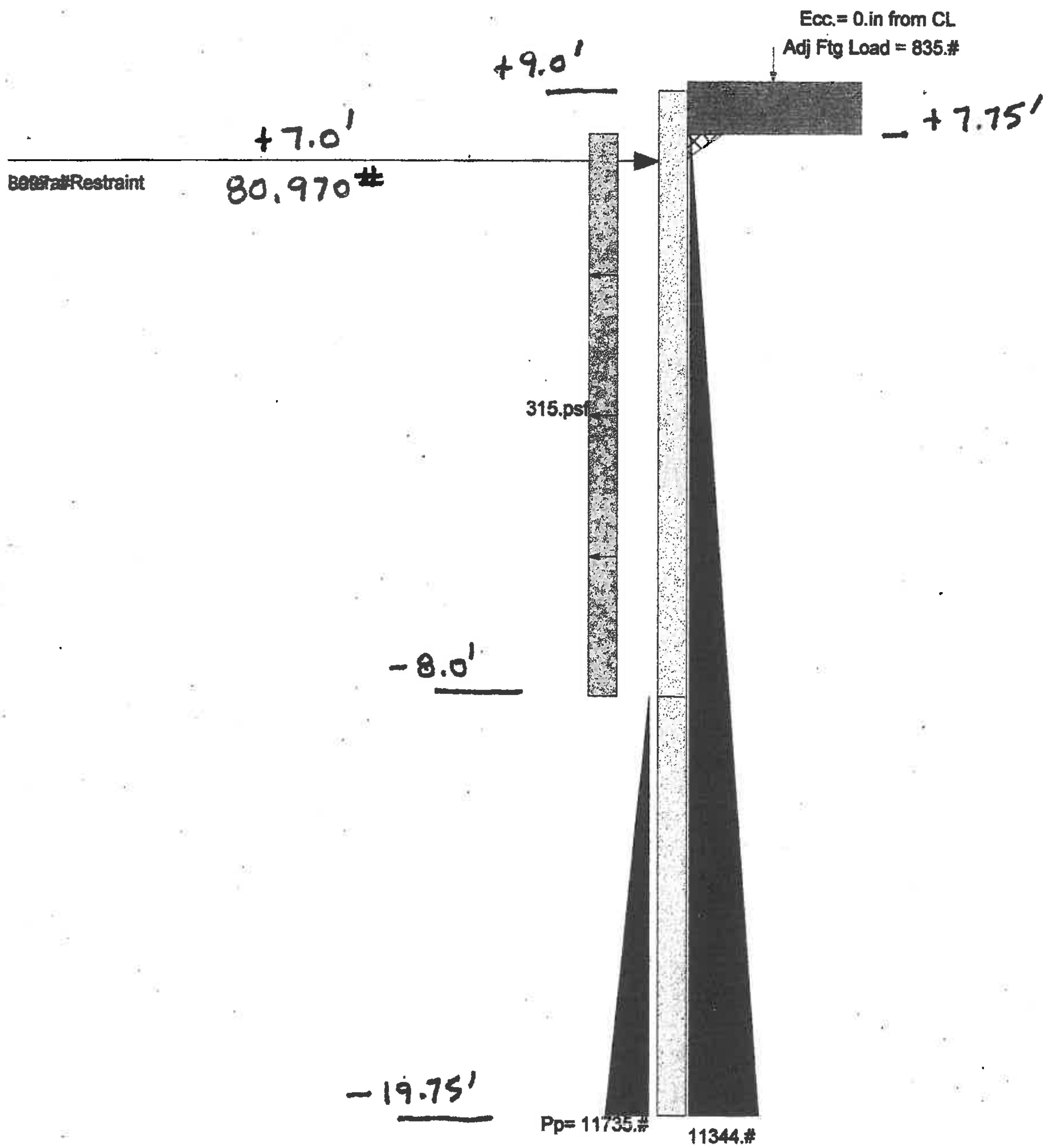
Thickness = 10.00 in   Fy = 40,000 psi
Wall Weight = 125.0 pcf   f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	10.00 ft	6.39 ft	0.00 ft
Rebar Size	=	# 6	# 6	# 6
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.145	0.484	0.916
Mu....Actual	=	1,936.5 ft-#	6,471.8 ft-#	12,245.0 ft-#
Mn * Phi.....Allowable	=	13,365.9 ft-#	13,365.9 ft-#	13,365.9 ft-#
Shear Force @ this height	=	0.0 lbs		6,955.7 lbs
Shear.....Actual	=	0.00 psi		89.18 psi
Shear.....Allowable	=	85.00 psi		85.00 psi 7.95
Rebar Lap Required	=	46.80 in	46.80 in	
Rebar embedment into footing	=			6.64 in

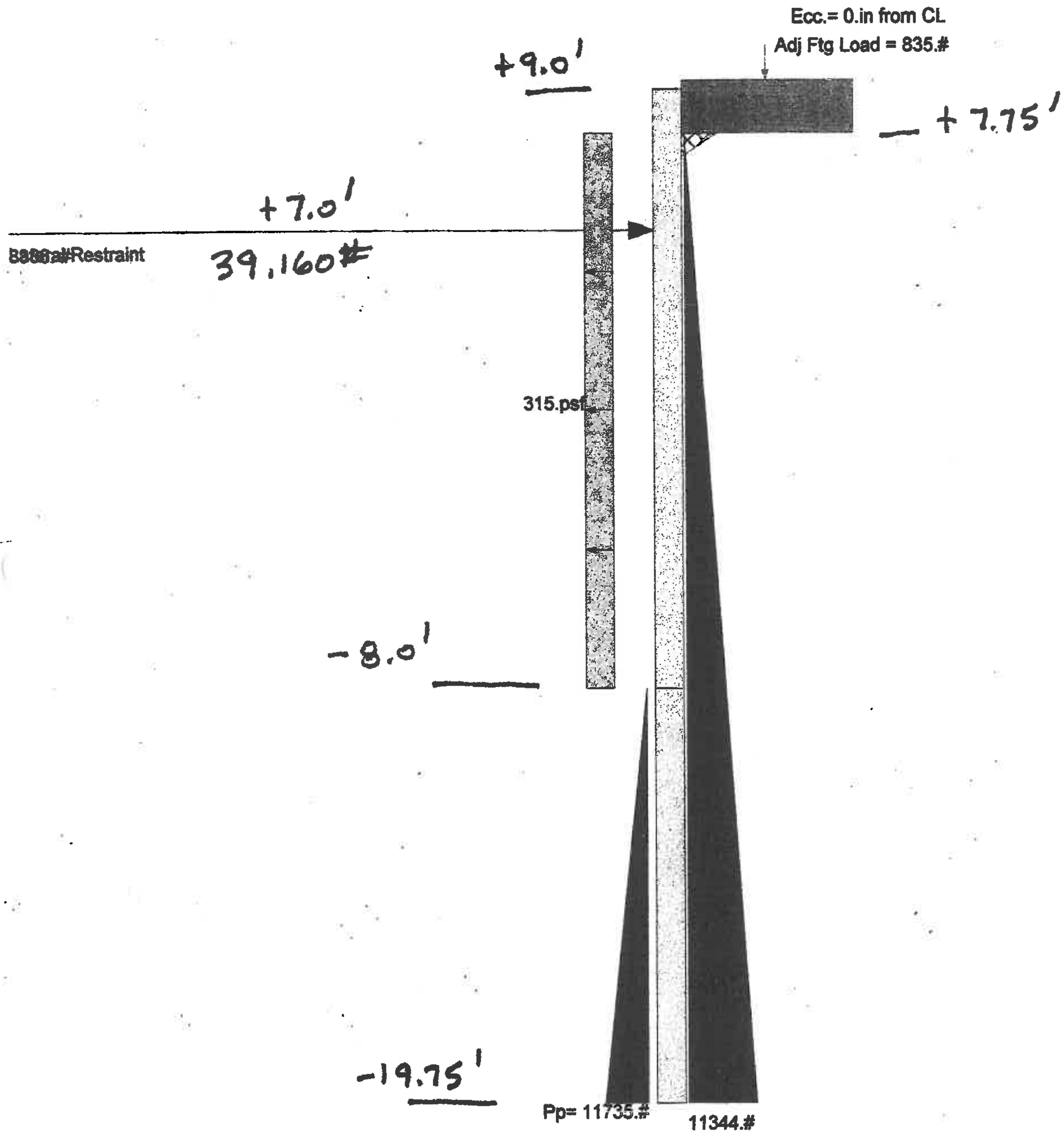
Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

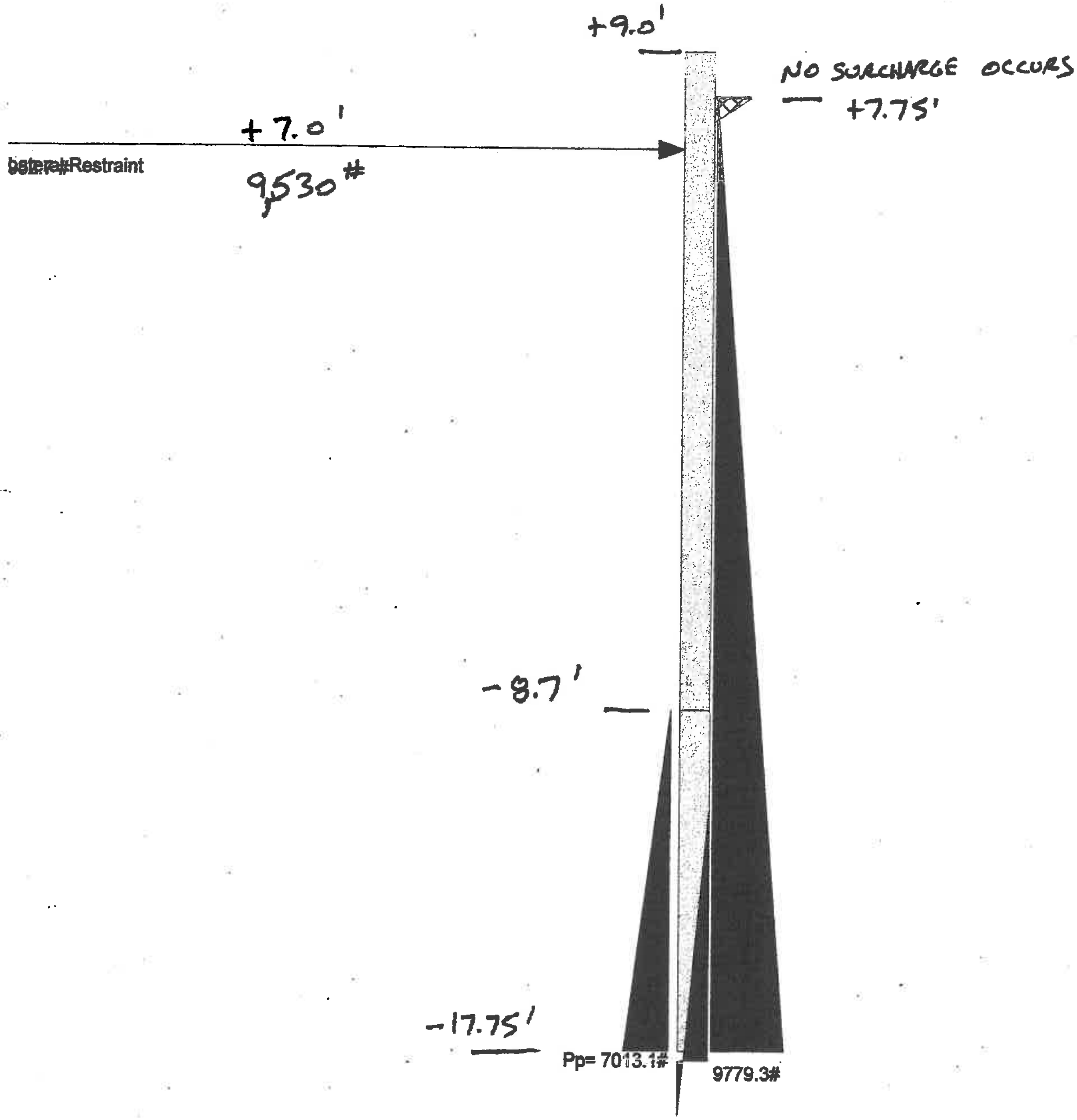
WEST BAYSIDE SEAWALL F.B.D.



WEST BAYSIDE W/ NEW TIEBACKS F.B.D.



WEST BAYSIDE AT SCOUR CONDITION
NEAR CHANNEL ENTRANCE
(24' SHEETS, - 8.7 MUDLINE)



TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION
Job # : CASE 13 Dsgnr: C. DUNC. Date: FEB 22,2009
Page: ______
Description....
WEST BAYSIDE SEAWALL - GRAVITY LOADS, -8' ML, 26' SHEETS
This Wall in File: c:\rp6\naples seawall case 1.rp5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	15.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.00 ft
Top Support Height	=	15.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtt	=	0.83
Footing Thickness	=	141.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Design Summary

Total Bearing Load	=	3,594 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	810.2 lbs	
Reaction at Bottom	=	10,533.5 lbs	
Sliding Stability Ratio	=	1.23	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	10,533.5 lbs	
less 100% Passive Force	= -	11,735.3 lbs	
less 100% Friction Force	= -	1,257.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	2,807.2 lbs	NG

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	15.00 ft	8.37 ft	0.00 ft
Rebar Size	=	# 8	# 8	# 8
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.000	0.259	0.568
Mu....Actual	=	3.6 ft-#	5,715.4 ft-#	12,549.2 ft-#
Mn * Phi.....Allowable	=	22,080.1 ft-#	22,080.1 ft-#	22,080.1 ft-#
Shear Force @ this height	=	0.0 lbs		4,948.3 lbs
Shear.....Actual	=	0.00 psi		63.44 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	78.00 in	78.00 in	
Rebar embedment into footing	=			8.85 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	-2,910.7 lbs
Heel Active Pressure	=	-7,622.8
Sliding Force	=	10,533.5 lbs

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem = -7,381.9ft-#

Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	2,125.0 lbs	0.42 ft	885.4 ft-#
Soil Over Heel	=	lbs	0.83 ft	ft-#
Footing Weight	=	1,468.8 lbs	0.42 ft	612.0 ft-#
Total Vertical Force	=	3,593.8 lbs	Base Moment =	-5,884.5 ft-#

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	15.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.00 ft
Top Support Height	=	15.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottorr	=	0.00 ft

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	141.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,594 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	1,355.1 lbs	
Reaction at Bottom	=	11,521.4 lbs	
Sliding Stability Ratio	=	1.13	Ratio < 1.5
Sliding Calcs			
Lateral Sliding Force	=	11,521.4 lbs	
less 100% Passive Force	= -	11,735.3 lbs	
less 100% Friction Force	= -	1,257.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	4,289.0 lbs	NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	15.00 ft	8.49 ft	0.00 ft
Rebar Size	=	# 8	# 8	# 8
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.024	0.272	0.617
Mu....Actual	=	519.7 ft-#	6,002.6 ft-#	13,620.9 ft-#
Mn * Phi.....Allowable	=	22,080.1 ft-#	22,080.1 ft-#	22,080.1 ft-#
Shear Force @ this height	=	0.0 lbs		5,957.6 lbs
Shear.....Actual	=	0.00 psi		76.38 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	78.00 in	78.00 in	
Rebar embedment into footing	=			8.85 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION
Job # : CASE 15A Dsgnr: C. DUNC. Date: MAR 16,2009
Page: ______
Description....
WEST BAYSIDE SEAWALL - GRAVITY + SEISIMIC, 26' SHEETS, -8 ML
This Wall in File: c:\rp6\naples seawall case 3.rp5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	15.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.00 ft
Top Support Height	=	15.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Surcharge Loads

Surcharge Over Heel = 0.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	315.0 #/ft
...Height to Top	=	15.75 ft
...Height to Bottom	=	0.00 ft

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Width	=	0.83
Footing Thickness	=	141.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 40,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 3.00 in @ Btm.= 3.00 in

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs
Footing Width	=	0.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	0.00 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,594 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	7,552.2 lbs	
Reaction at Bottom	=	13,477.8 lbs	
Sliding Stability Ratio	=	0.96	UNSTABL
Sliding Calcs			
Lateral Sliding Force	=	13,477.8 lbs	
less 100% Passive Force	=	- 11,735.3 lbs	
less 100% Friction Force	=	- 1,257.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	7,223.6 lbs	NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall	
		Stem OK	Stem OK	Shear NG!	
Design height	=	15.00 ft	8.98 ft	0.00 ft	
Rebar Size	=	# 8	# 8	# 8	
Rebar Spacing	=	7.00 in	7.00 in	7.00 in	
Rebar Placed at	=	Edge	Edge	Edge	
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in	
Design Data					
fb/FB + fa/Fa	=	0.007	0.635	1.247	
Mu....Actual	=	154.2 ft-#	14,024.4 ft-#	27,534.9 ft-#	
Mn * Phi.....Allowable	=	22,080.1 ft-#	22,080.1 ft-#	22,080.1 ft-#	0.80
Shear Force @ this height	=	0.0 lbs		9,953.5 lbs	
Shear.....Actual	=	0.00 psi		127.61 psi	
Shear.....Allowable	=	85.00 psi		85.00 psi	0.67
Rebar Lap Required	=	78.00 in	78.00 in		
Rebar embedment into footing	=			8.85 in	

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION   Page: ____
Job # : CASE 15   Dsgnr: C. DUNC   Date: FEB 22,2009
Description....
WEST BAYSIDE SEAWALL - GRAVITY LOADS, W/ SURCHARGE & SEISMIC, -8' ML,26' SHEETS
This Wall in File: C:\RP6\NAPLES SEAWALL CASE 3.RP5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	15.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.00 ft
Top Support Height	=	15.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	141.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf
>>>Used To Resist Sliding & Overturning		
Surcharge Over Toe	=	0.0 psf
Used for Sliding & Overturning		

Uniform Lateral Load Applied to Stem

Lateral Load	=	315.0 #/ft
...Height to Top	=	15.75 ft
...Height to Bottom	=	0.00 ft

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Design Summary

Total Bearing Load	=	3,594 lbs	
...resultant ecc.	=	0.00 in	
Soil Pressure @ Toe	=	0 psf	OK
Soil Pressure @ Heel	=	0 psf	OK
Allowable	=	3,000 psf	
Soil Pressure Less Than Allowable			
ACI Factored @ Toe	=	0 psf	
ACI Factored @ Heel	=	0 psf	
Footing Shear @ Toe	=	0.0 psi	OK
Footing Shear @ Heel	=	0.0 psi	OK
Allowable	=	107.5 psi	
Reaction at Top	=	8,097.1 lbs	
Reaction at Bottom	=	14,465.7 lbs	
Sliding Stability Ratio	=	0.90	UNSTABL
Sliding Calcs			
Lateral Sliding Force	=	14,465.7 lbs	
less 100% Passive Force	= -	11,735.3 lbs	
less 100% Friction Force	= -	1,257.8 lbs	
Added Force Req'd	=	0.0 lbs	OK
....for 1.5 : 1 Stability	=	8,705.4 lbs	NG

Concrete Stem Construction

Thickness	=	10.00 in	Fy	=	40,000 psi
Wall Weight	=	125.0 pcf	f'c	=	2,500 psi

Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	15.00 ft	9.04 ft	0.00 ft
Rebar Size	=	# 8	# 8	# 8
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.017	0.649	1.296
Mu....Actual	=	369.1 ft-#	14,331.5 ft-#	28,606.6 ft-#
Mn * Phi.....Allowable	=	22,080.1 ft-#	22,080.1 ft-#	22,080.1 ft-#
Shear Force @ this height	=	0.0 lbs		10,962.9 lbs
Shear.....Actual	=	0.00 psi		140.55 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	78.00 in	78.00 in	
Rebar embedment into footing	=			8.85 in

0.77

70.60

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in	-or-	Not req'd, Mu < S * Fr
Heel: None Spec'd	-or-	Not req'd, Mu < S * Fr
Key: No key defined	-or-	No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION
Job # : CASE 16 , Dsgnr: C. DUNC. Page: ______ Date: FEB 22,2009
Description....
WEST BAYSIDE SEAWALL - NEW TIEBACKS GRAVITY LOADS, W/ SURCHARGE & SEISMIC
This Wall in File: C:\RP6\NAPLES SEAWALL CASE 4.RP5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	15.75 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.00 ft
Top Support Height	=	13.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Surcharge Loads

Surcharge Over Heel = 0.0 psf
>>>Used To Resist Sliding & Overturning
Surcharge Over Toe = 0.0 psf
Used for Sliding & Overturning

Axial Load Applied to Stem

Axial Dead Load	=	0.0 lbs
Axial Live Load	=	0.0 lbs
Axial Load Eccentricity	=	0.0 in

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Uniform Lateral Load Applied to Stem

Lateral Load	=	315.0 #/ft
...Height to Top	=	15.75 ft
...Height to Bottom	=	0.00 ft

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Width	=	0.83
Footing Thickness	=	141.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft

f'c = 4,000 psi Fy = 40,000 psi
Footing Concrete Density = 150.00 pcf
Min. As % = 0.0018
Cover @ Top = 3.00 in @ Btm.= 3.00 in

Adjacent Footing Load

Adjacent Footing Load	=	835.0 lbs
Footing Width	=	5.00 ft
Eccentricity	=	0.00 in
Wall to Ftg CL Dist	=	2.50 ft
Footing Type		Line Load
Base Above/Below Soil at Back of Wall	=	0.0 ft

Design Summary

Total Bearing Load	=	3,594 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	8,385.8 lbs
Reaction at Bottom	=	13,972.9 lbs
Sliding Stability Ratio	=	0.93 UNSTABL
Sliding Calcs		
Lateral Sliding Force	=	13,972.9 lbs
less 100% Passive Force	= -	11,735.3 lbs
less 100% Friction Force	= -	1,257.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	7,966.3 lbs NG

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Shear NG!
Design height	=	13.00 ft	8.04 ft	0.00 ft
Rebar Size	=	# 8	# 8	# 8
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.070	0.545	1.055
Mu....Actual	=	1,550.8 ft-#	12,032.4 ft-#	23,296.0 ft-#
Mn * Phi.....Allowable	=	22,080.1 ft-#	22,080.1 ft-#	22,080.1 ft-# >0.95
Shear Force @ this height	=	0.0 lbs		10,104.2 lbs
Shear.....Actual	=	0.00 psi		129.54 psi
Shear.....Allowable	=	85.00 psi		85.00 psi >0.66
Rebar Lap Required	=	78.00 in	78.00 in	
Rebar embedment into footing	=			8.85 in

Other Acceptable Sizes & Spacings:
Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

TRANSYSTEMS
1650 Hotel Circle North
Suite 201
San Diego, CA 92108

Title : NAPLES SEAWALL INVESTIGATION Page: ______
Job # : CASE 17 Dsgnr: C. DUNC. Date: MAR 16,2009
Description....
WEST BAYSIDE SEAWALL SCOUR CONDITION GRAVITY ONLY 24' SHEETS, -8.7' MUDLINE
This Wall in File: c:\rp6\naples seawall case 1.rp5

RetainPro Version 6.0
Build Date : 10-SEP-2001, (c) 1989-2001

Restrained Retaining Wall Design

Criteria

Retained Height	=	16.45 ft
Wall height above soil	=	1.25 ft
Total Wall Height	=	17.70 ft
Top Support Height	=	15.00 ft
Slope Behind Wal	=	0.00 : 1
Height of Soil over Toe	=	0.00 in
Soil Density	=	105.00 pcf
Wind on Stem	=	0.0 psf

Soil Data

Allow Soil Bearing	=	3,000.0 psf
Equivalent Fluid Pressure Method		
Heel Active Pressure	=	30.0 psf/ft
Toe Active Pressure	=	30.0 psf/ft
Passive Pressure	=	170.0 psf/ft
Water height over heel	=	0.0 ft
Footing||Soil Frictior	=	0.350
Soil height to ignore for passive pressure	=	0.00 in

Footing Strengths & Dimensions

Toe Width	=	0.00 ft
Heel Width	=	0.83
Total Footing Widtr	=	0.83
Footing Thickness	=	109.00 in
Key Width	=	0.00 in
Key Depth	=	0.00 in
Key Distance from Toe	=	0.00 ft
f'c = 4,000 psi	Fy =	40,000 psi
Footing Concrete Density	=	150.00 pcf
Min. As %	=	0.0018
Cover @ Top = 3.00 in	@ Btm.=	3.00 in

Design Summary

Total Bearing Load	=	3,348 lbs
...resultant ecc.	=	0.00 in
Soil Pressure @ Toe	=	0 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	3,000 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	0 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	0.0 psi OK
Footing Shear @ Heel	=	0.0 psi OK
Allowable	=	107.5 psi
Reaction at Top	=	952.7 lbs
Reaction at Bottom	=	8,826.5 lbs
Sliding Stability Ratio	=	0.93 UNSTABL
Sliding Calcs		
Lateral Sliding Force	=	8,826.5 lbs
less 100% Passive Force	= -	7,013.1 lbs
less 100% Friction Force	= -	1,171.8 lbs
Added Force Req'd	=	0.0 lbs OK
....for 1.5 : 1 Stability	=	5,054.9 lbs NG

Concrete Stem Construction

Thickness = 10.00 in Fy = 40,000 psi
Wall Weight = 125.0 pcf f'c = 2,500 psi
Stem is FIXED to top of footing

		@ Top Support	Mmax Between Top & Base	@ Base of Wall
		Stem OK	Stem OK	Stem OK
Design height	=	15.00 ft	8.49 ft	0.00 ft
Rebar Size	=	# 7	# 7	# 7
Rebar Spacing	=	7.00 in	7.00 in	7.00 in
Rebar Placed at	=	Edge	Edge	Edge
Rebar Depth 'd'	=	6.50 in	6.50 in	6.50 in
Design Data				
fb/FB + fa/Fa	=	0.001	0.356	0.771
Mu....Actual	=	25.9 ft-#	6,257.0 ft-#	13,542.1 ft-#
Mn * Phi.....Allowable	=	17,560.4 ft-#	17,560.4 ft-#	17,560.4 ft-#
Shear Force @ this height	=	0.0 lbs		5,280.7 lbs
Shear.....Actual	=	0.00 psi		67.70 psi
Shear.....Allowable	=	85.00 psi		85.00 psi
Rebar Lap Required	=	68.25 in	68.25 in	
Rebar embedment into footing	=			7.75 in

Other Acceptable Sizes & Spacings:

Toe: # 6 @ 18.00 in -or- Not req'd, Mu < S * Fr
Heel: None Spec'd -or- Not req'd, Mu < S * Fr
Key: No key defined -or- No key defined

Footing Design Results

		Toe	Heel
Factored Pressure	=	0	0 psf
Mu' : Upward	=	0	0 ft-#
Mu' : Downward	=	0	0 ft-#
Mu: Design	=	0	0 ft-#
Actual 1-Way Shear	=	0.00	0.00 psi
Allow 1-Way Shear	=	0.00	0.00 psi

Summary of Forces on Footing : Slab is NOT providing sliding, stem is FIXED at footing

Forces acting on footing for sliding & soil pressure....

Sliding Forces

Stem Shear @ Top of Footing	=	-3,106.3 lbs
Heel Active Pressure	=	-5,720.2
Sliding Force	=	8,826.5 lbs

Load & Moment Summary For Footing : For Soil Pressure Calcs

Moment @ Top of Footing Applied from Stem = -7,966.0ft-#

Surcharge Over Heel	=	lbs	ft	ft-#
Adjacent Footing Load	=	lbs	ft	ft-#
Axial Dead Load on Stem	=	lbs	ft	ft-#
Soil Over Toe	=	lbs	ft	ft-#
Surcharge Over Toe	=	lbs	ft	ft-#
Stem Weight	=	2,212.5 lbs	0.42 ft	921.9ft-#
Soil Over Heel	=	lbs	0.83 ft	ft-#
Footing Weight	=	1,135.4 lbs	0.42 ft	473.1 ft-#
Total Vertical Force	=	3,347.9 lbs	Base Moment =	-6,571.0ft-#

8.0 Repair Recommendations

8.1 Existing Seawall Repair or Replace?

New concrete waterfront construction for the US Navy and Port Districts in Southern California has a typical expectation of a 75 year service life. Much has been learned about concrete durability in the past 25 years and with the advent of a variety of concrete admixtures, rebar coatings and researched concrete mix designs, a 75 to 100 year service live may be projected for new marine concrete structures. For structures built before 1950, it is very typical for repairs to proceed as follows: In the first 25 years very limited repairs are expected, in the middle 25 years repairs are typically required on a 10 year cycle, in the last 25 years the cycle drops to 7 and then 5 year cycles. After 75 years the cost of piecemeal repairs, interruption to operations, and uncertain performance of the patched structure create a period where consideration is be given to when and how the structure will be replaced. It is critical to develop a plan for replacement of major structures so the construction can occur when it is favorable to the Client as opposed to responding to localized failures in an emergency fashion.

Emergency repairs typically have a very high unit cost and have no master planning for continuity for future repair interfacing. Since this structure is currently 70 year old, it is recommended that the City and Homeowner's consider the timing of a replacement program and wisely appropriate future funds in consideration of the limited remaining service life. The magnitude of a complete wall replacement project is on the order of $56 Million dollars and expected to require phasing construction and raising bonds to cover a construction period of 6 years, (see Cost Estimates in Section 9.0 and Figure 8-1, Phasing Plan on page 8-19).

To best direct the limited annual funding for Seawall Maintenance and Repair, the following list of problems is compiled and ranked purely on structural need and public safety.

8.2 Phasing

The Phases have been divided into roughly 6 equal work areas, separated by the bridges. It is anticipated that one Phase would be constructed each year and require approx 6 months of active work. The cost of mobilizing and demobilizing equipment around a bridge is significant since few items will be able to clear below the bridges.

All seawall caps are past the point of repair and any program for cap repair will have a short life since it will be demolished when the wall is replaced in the near future. Transystems welcomes the opportunity to discuss phasing and timing of repair recommendations with City Staff and Homeowner's. A preliminary Repair Priority and Phasing Plan is provided in Figure 8-1 for review and to serve as a starting point for development of a cohesive phasing plan which includes all stakeholders input.

8.3 Priorities

1. Sulfate Deterioration of the concrete sheet piling below the waterline, (see Figure 4-7, page 4-40). The northeast quarter of Rivo Alto Canal, colored in Figure 4-7 relates to the severity of the concrete deterioration observed by the inspection team. A few hundred feet of these walls are considered to be in "serious", marked in red and bending failures are "probable" anytime within the next 5 years and "expected" should the site experience a moderate earthquake before the sheet piling can be replaced. For this reason this area is ranked as the number 1 priority shown in red on Figure 8-1, Phasing Plan. The Naples Island walls in this quadrant are at slightly less risk than the landside walls due to the installation of the newer tie-back rods which shorten the bending span of the sheet pile approx 13%. If sub prioritization is needed to match funding, the Landside regions marked in red should precede the Naples Island red regions.

2. There is one area of moderate scour near the main channel entrance, (see Figure 6-2 on page 6-2). It is marked in red on Figure 6-2. The lower mudline has the effect of increasing the bending and tieback loads on the sheet pile system and reducing the embedment into the soil. Based upon the results of the Seawall Stability Analysis, page 7-8, the channel entrance is also an area at greater risk of failure during an earthquake. If a major earthquake were to occur near or in Long Beach, these sheets may slide out rapidly into the canal, as happened during the 1933 earthquake. The West Bayside Segment, if added to the Channel Entrance works into an efficient size project for a contractor. The West Bayside seawalls have a risk of failure in an earthquake, second only to the Channel Entrance seawalls. Since they are both in the same work space, the combined area shown in brown on Figure 8-1, Phasing Plan, is ranked as the number 2 priority.

3. Minor sulfate deterioration was also observed in the northwest quarter of Rivo Alto Canal, colored green in Figure 4-7, page 4-40, but this deterioration is currently limited to the outer ½" of the sheet pile surface and no areas of major erosion or rebar loss is evidenced yet so it is anticipated that this is okay for static loads but that it presents a weak section which will fail under a lesser earthquake the balance of the canal walls. For this reason the west quarter of Rivo Alto Canal is rated as priority number 3, colored purple on Figure 8-1, Phasing Plan.

4. The Naples Island segment is almost completely encompassed by a system of recently installed tiebacks and wales. Since these tiebacks are designed to carry 100% of the lateral load requirement, the Island is considered less susceptible to failure than areas without this work. For this reason Naples Island was generally ranked as a 4th or 5th priority.

   (Phase 4 and Phase 5 are relatively equal in ranking and either one could go first. Refer to Figure 8-1, light blue and dark blue colors.)

5. Phase 6 is overall in the best condition and the mudline is consistent with no areas of any significant scour and only a very uniform loss of 0.3 to 0.5 feet of material. This segment has an estimated remaining service life of 20 years; while by comparison, priority 1 has an estimated 0 to 5 year remaining service life. It is believed that the sheets are a couple feet longer, along the bayfront properties to account for the possibility of deeper water conditions and this extra pile length increases the Factor of Safety for this Phase a few percentage points above the others, See Figure 7-2, page 7-8. For these reasons, the East Bayside is ranked 6th priority, colored in green on Figure 8-1. If this Phase needs to wait the longest for replacement, it may be appropriate to consider chemical grouting the couple sink holes identified on Figure 2-10, page 2-46.

8.4 Existing Tieback Repair Program

The existing tieback systems should be considered as an interim repair for areas where the concrete is in fair condition and may last another 20 years.

A review of the existing Tie-back Condition Survey and current program to replace the tiebacks draws the following conclusions. No reliable predictions can be made in regards to which original tie-backs have been damaged by construction activities or have weaknesses in the protective coating allowing corrosion to occur. If the reader "steps back" and considers the shallow depth of the original tie-backs, 1-3 feet beneath the surface, and that 80% of the properties along the canals have had major remodels, with the majority adding 1 or 2 additional stories to the original structures, it can reasonably be concluded that the majority of original tie-backs have been exposed to construction activities with the potential to damage the bars or protective coating. The rail monitoring program along with recommended monitoring of the seawall joints will be partially effective at identifying when a single tie-back has failed. The value of this monitoring is only realized if it can affect the timely repair of a single failed tieback prior to failure of a 2nd adjacent tie-back. If a second consecutive tie-back fails, the probable result is a local failure of a 50' to 100' wall segment. The replacement of all tiebacks is a viable intermediate solution to address the high probability of existing tieback failures. This repair method has a close spacing which braces every sheet pile and is expected to perform fair during moderate earthquakes. This repair method does not address the following equally important issues effecting the longevity and performance of the concrete in the sheet piling.

1) The advanced age of the concrete sheet piles will soon experience bending failures at the areas of severe deterioration.

2) The steel wale system for the new tie-backs is at a very corrosive tidal elevation and the first series of wale's will soon require maintenance repairs to the coating and anchor bolts, see Photo 8-1.

3) Advanced Concrete Cap deterioration is not addressed.

4) Loss of soil fines through the sheet pile joints and cracks due to tidal pumping which cause sinkholes and subsidence are not addressed.

Photo 8.1 – Naples. Rivo Alto Canal #58, Replacement tie-backs have rapid corrosion due to their elevation and will require maintenance in the next five years.

8.5 Interim Repairs (see Figure 8-11)

Transystems has considered the following menu of Interim Repairs for application prior to funding of a program to replace the seawalls:

Tieback System: See Figures 8-2 & 8-3. This concept continues the tieback concept in-place on Naples Island. We recommend using grouted rods. We have considered Helical tiebacks as an option but they require destructive excavations to install them on the tieback shaft after it passes thru a small hole in the sheet piling. For this reason we prefer the grouted tie-back rod solution. Tieback solutions may be considered for the Channel Entrance and West Bayside segments. See Section 9 for cost estimate.

Sinkhole Repairs: A list of locations with conditions favorable to the formation of sinkholes and confirmed subsidence/sinkholes is compiled on Page 2-20. Figure 2-10 displays the locations of the confirmed subsidence/sinkholes and locations of active soil migration evidenced by sand mounds at the sheet pile joints. The deepest sinkhole is 8" deep occurring in front of #133 Rivo Alto Canal, see Photo 2-4. Recommended subsidence repair areas are shown on Figure 8-11. This work may be done as funding permits but sidewalk sinkholes represent a safety hazard and should be barricaded when they reach a condition which poses a trip hazard and then fixed on a priority basis. We recommend a gravel filled bag system as a cost effective means of restoring the sidewalks with sinkholes to a safe condition for the public access. The gravel will have some ability to accommodate future movement but also strength to bridge smaller voids and protect the sidewalk for more years than earth fill. The gravel can be easily excavated for future utility work or seawall replacement efforts (see Figure 8-4.)

The planting area adjacent to the sidewalk, (see Figure 8-4) can most efficiently be maintained by adding topsoil at landscaped areas or adding decorative rock/gravel at bare earth locations. It is anticipated that a few sacks of new fill material would be required once every 3-5 years to maintain an approximate level condition, (see Photo 2-3 for an example of a repair area suitable for decorative rock fill). Upon completion of work in the planting strip it would be acceptable to install a wood or plastic lumber deck system as per the City's Seawall Guidelines on page 1-11. See Section 9 for sinkhole repair cost estimate.

Mudjacking or chemical soil mixing has been utilized in prior repair projects to mitigate the loss of backfill material and resultant subsidence. When mudjacking is performed in a uniform continuous method it can be successful. A shortcoming of mudjacking is the ground water trapped behind the wall seeks a new route after the repair, typically at the end of the repair or an inconsistency in the repair, creating new subsidence locations. To mitigate this problem, mudjacking should be performed in as long and continuous repair as possible which carries a very high cost and does not improve the stability of the seawall which is the primary objective of this investigation. The mudjacking process exerts high pressure on the seawall and should only be considered for application on wall sections with new tie rods and with concrete sheets in good condition (see Geotech Report Vol 2, Appendix D, Section 4.4.3). At this time we do not recommend mudjacking as a typical sinkhole repair method. The formation of sinkholes is currently occurring over decades. Should the future loss of soil be observed as occurring rapidly then a mudjacking or soil mixing repair may be considered on a case-by case basis by the designer.

Light Pole Foundation Repairs: See Manufactures information sheets at the end of Section 9.0 from A.B. Chance, a construction supply company which specializes in helical anchor foundations. We propose either a single long helical pipe or a trapeze foundation of three smaller helical anchors. Both systems are very fast and will have minimal impact on the site. Both are capable of being installed in poor soils. The deepest sink hole is 8" so a 10' screw footing will work fine. See Section 9 for cost estimate and examples of product and applications and details.

Scour Repair at base of Seawall: Grout filled bags or rock could be placed in the scour areas as a temporary mitigation but this would pose a navigation hazard unless clearly marked with a warning pile at the limits of the work area. This is an appropriate repair to address the scour in the Channel Entrance. See Figure 8-10 and 8-11 and Section 9 for cost estimate. This repair can be built up to an elevation of -5.0 MLLW if it stays under the dock width and will not effect navigation or the performance of the docks. See Section 9 for cost estimate. The cost of this work varies wildly dependent upon the volume of the repair. A small repair of 50 feet may cost $1,500/ft where as a large repair of 2,000 feet may only cost $800/ft.

Bulkhead Repair: An underwater bulkhead constructed with PZ27 steel sheet piling is proposed as an interim solution to reduce bending stress in areas with significant sulfate damage. This new bulkhead will have the effect of raising the elevation of the bottom support for the existing sheet piling. A new bulkhead with a top elevation of -5 MLLW will stay beneath the typical floating dock system at extreme low tide, see Figure 8-5. The distance of the bulkhead from the existing wall will be limited to 5' or less, as required to avoid interference with boat hull contact. Backfill with rock is proposed since it can easily be dug out for future wall replacement. The steel sheet piling typically has a salvage value, if removed for a permanent wall replacement in the future. The interim bulkhead is expected to mitigate the risk of concrete sheet pile failures under (non-seismic conditions) for a minimum of 5 years. See Section 9.0 for a cost estimate and Manufacture's data on PZ-27 sheet piling at the end of section 9.

Note: The original tie rods, throughout the Naples community, are known to be in varying states of decay. It is not possible to assess these rods without extensive destructive investigation of every property. Tie rod failures are still possible anywhere in the community where newer rods have not yet been installed.

8.6 Seismic Performance of Existing Seawalls

Based upon our waterfront experience and knowledge gained from this investigation, it is our opinion that some sections of the canal seawalls have a significant risk of "global" failure due to their present deteriorated condition, if the site experiences a "moderate" near-source earthquake.

Upon review of our seawall stability analysis results, all of the seawalls are at risk of failure in a "major" near-source earthquake. A "global" failure of a seawall at Naples is described as a failure of the tieback rod system with a rapid unzippering of multiple tieback rods and pullout or breakage of the soldier piles buried 18' behind the seawall. The tie rods which are corroded or connected to a deteriorated section of the seawall cap are expected to fail at those points. After the first couple tiebacks snap, the seawall cap will break apart and a series of sheet piles in 50' to 200' sections, dependent upon the seawall construction joint spacing, will rotate towards the bay. The grade behind the wall will cave down to an approximate 1:1 slope for an estimated 5 to 10 of feet of depth. Slope failures behind the seawall are expected to be limited to the first 13 feet but if liquefaction occurs, all typical residential spread footing foundations in the bay and beach areas are prone to distortion and settlement. The Geotechnical Engineer estimates the liquefaction potential for settlement at Naples to be 7" due to the depth and density of the fill material.

Mitigation Measures – "Surcharge" Removal & "Rock-Seismic"

In the interim period, before the seawall is replaced, the seawall segments at greatest risk can be improved to a level of risk equal to that of the balance of the community by the addition of rock at the base of the wall and by removing surcharge loads, where they occur, see "Rock Seismic" and "Surcharge" descriptions on Figure 8-11. Refer to Section 2.3 for description of "Surcharge".

8.7 Rising Sea Elevation

The design of a replacement seawall system should consider the reality that the sea level has been rising steadily for the past recorded 100 years. There is much discussion and political perspective on whether human activities and green-house gasses are to blame but for purposes of designing a seawall it does not matter why, it only matters that the best estimate of future sea levels be evaluated in regards to the proper cap elevation for the anticipated service life of a new or replacement structure. The following graphics have been compiled to provide some historical data and prediction envelopes for future sea levels in the next 75 – 100 years.

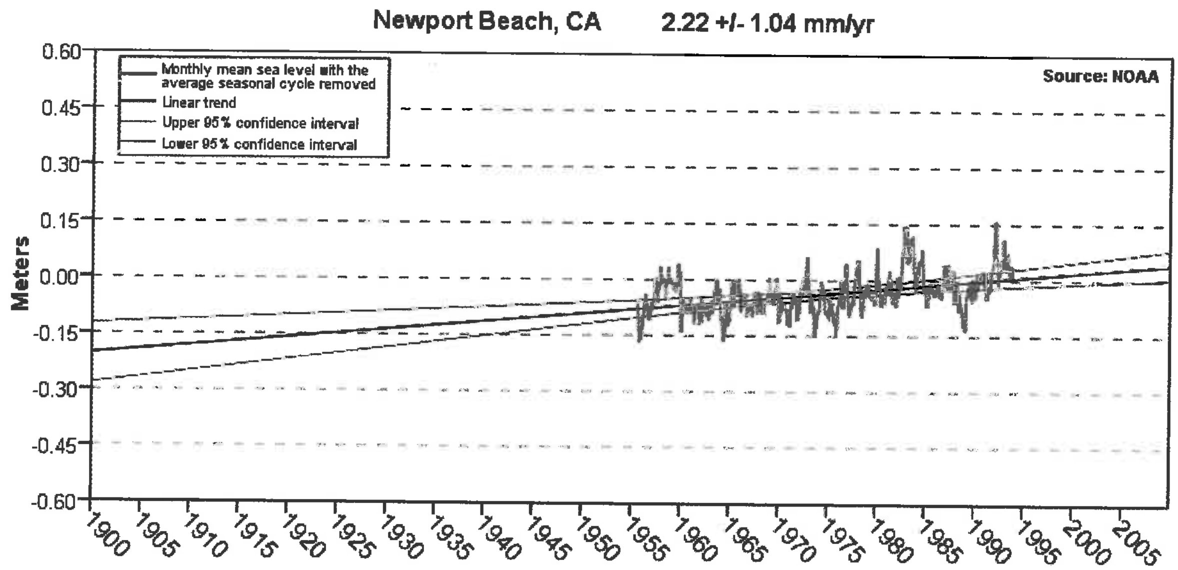


Figure 8-1a. Local Newport Beach Data from NOAA shown above and indicates 2.2 mm average increase in sea level which extrapolates to a 6.5" increase in the next 75 years, if the past 100 year trend continues.

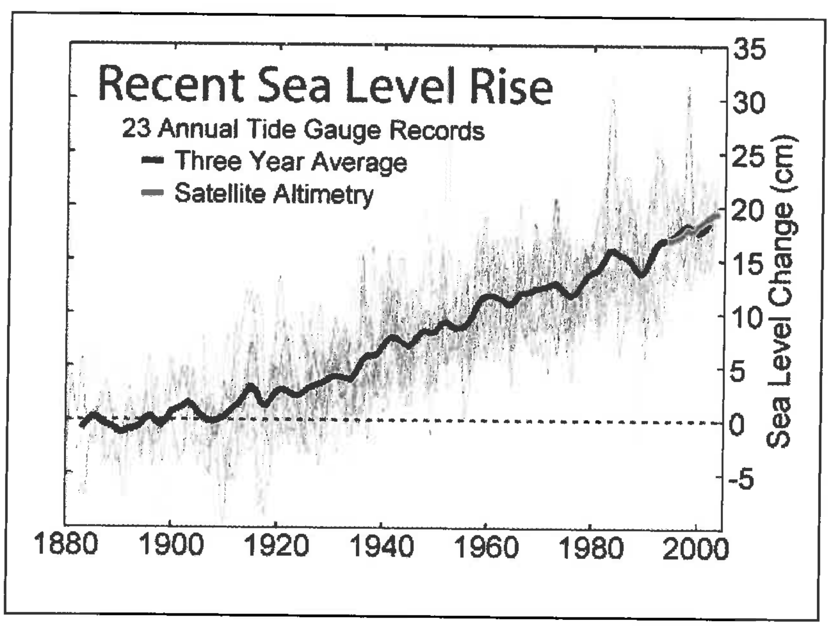


Figure 8-2b. One hundred plus years of tidal gauge records are now being supplemented with satellite recordings to provide another resource of historical sea level trends. These records indicate a 20 cm rise in 100 years which equates to 5.9" sea level rise in a 75 year period.

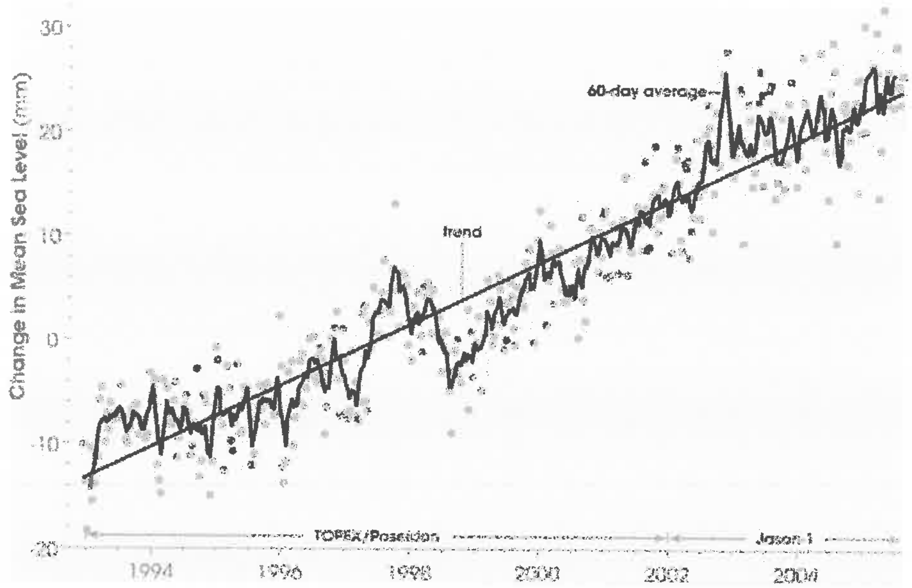


Figure 8-2c. Using only satellite data, the most recent sea level trend is 37 mm rise in 12.5 years which equates to 3 mm/year = 8.7" sea level rise in 75 years.

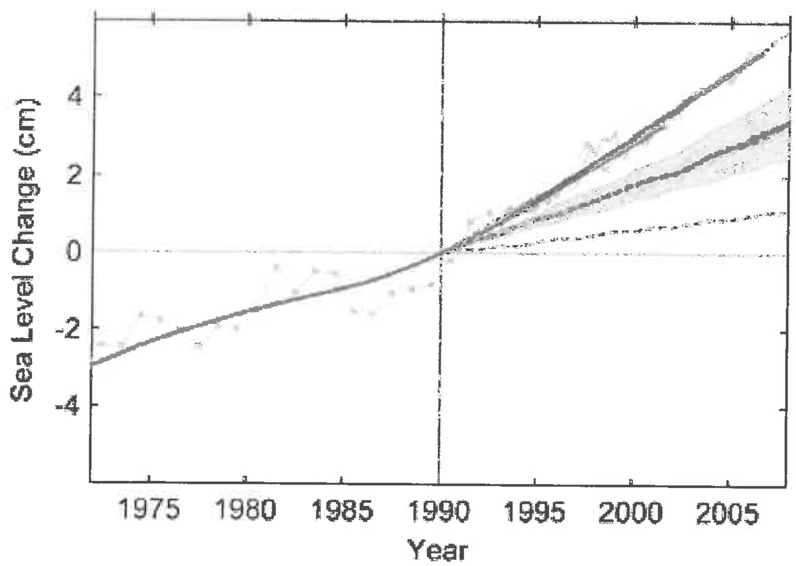


Figure 8-2d. This graphic bounds the cumulative predictions by research groups. This graph indicates that all researchers believe the sea level is rising and that the average of the current modeling (the blue dashed line) indicates 3 cm of rise in 15 years which equates to 5.9" rise in 75 years.

The current extreme high tide for Alamitos Bay is approximately +8.0' MLLW. This is predicted to increase to +8.5' in the next 75 years. If the City and Homeowners' elect to pursue a long term repair involving the seawall cap, it is recommended that the cap height be increased 6" from the current cap elevation of +9.0 MLLW to a new elevation of +9.5' MLLW. This is intended to mitigate flooding of the roads and walks during future extreme tides in combination with wind waves. This is a concern development wide since a 6" wave could crest the current wall at an extreme future tide but the Bay front lots have a greater exposure to wave action and this recommendation is very important to those properties.

Since the entire seawall cap must be demolished for any repair, there is no premium for raising the cap elevation 6". Most steps re concrete, brick or tile and will be rebuilt by each homeowner. The gangways and any utilities on the cap get removed for any new wall solution.

8.8 Long Range Repair Options (see Figure 8-1)

8.8.1 Waterside Repair Option

The Waterside Repair method addresses replacement of the aging seawall, cap and tie-rods with the least intrusive method to the residential properties and dock systems. Almost all activities associated with this repair can be accomplished from the waterside, thus its title. This Option consists of:

Pressure washing the existing wall face to remove potential voids of marine growth if trapped in the repair

Excavating a shallow trench adjacent to the seawall to remove the sediments down to the level of harder original dredge depth plus 1 foot min for bearing

Driving new steel "H" piles at a spacing to be optimized by the designer between 5' and 10' on center

Installing recycled plastic spacers to the inboard flange of the "H" pile

Setting the precast concrete planks into the space between the "H" piles and securing them plumb with solid plastic blocking on the outboard leg of the "H" pile flange

Grouting the gap between the new concrete planks and the existing seawall with a pea gravel concrete mix up to the base of the existing cap. (Where grout is placed below water it shall use a tremme method and the concrete shall have an anti-washout admixture)

Demolish the existing cap and construct a new cap to encapsulate both the new "H" pile top and the existing sheet piles

Install a new guard rail, sidewalks and repair subsidence areas behind the wall

Refer to Figures 8-6 & 8-7 on the following pages for graphics of the Waterside Repair Option & Section 9 for cost estimate.

8.8.1.1 Waterside Repair Positive Factors

A) Least intrusive on existing homeowner improvements and public walkway during construction.

B) Steel is the major cost of this construction and the price is dropping rapidly.

C) Precast concrete panels allow construction to progress rapidly

D) Method can be started and stopped at any location for phasing or work interruptions

E) Repair will seal all voids, gaps and cracks in the existing seawall to alleviate the ongoing loss of soil and settlement due to tidal pumping.

F) Repair will replace the deteriorated seawall cap and be an aesthetic improvement.

G) Repair will improve the seismic capacity of the seawall and protect the value of adjacent residential properties.

H) Repair will significantly reduce future maintenance costs for many years.

I) The sidewalk system will have few, if any closures for wall or tie-back repairs for the next 25-50 years.

J) Repair can address sea level rise, if desired, at minimal cost.

K) No effect on floating dock vertical clearances.

L) There is an opportunity to reduce the steel costs inside the Naples Island circular canal if 16" square precast concrete pile struts are installed just under the mudline to brace the steel piles on each side of the canal. This will reduce the steel "H" pile lengths and estimated 25% in that region minus the cost of the struts has the potential to save $1 million in construction cost but will necessitate coordination of the pile layout on each side of the canal and frequently interfere with boat access during construction.

M) Storm drains can reasonably be extended thru precast concrete.

N) Steel H pile drive is expected to go very quickly, estimate 10 minutes of hammer time per pile. The hammer will be 60 to 80 decibels for approx 10 minutes 45 minutes. Residents within 2 blocks of any pile driving may feel a shock wave from the hammer blows as the pile reaches competent bearing strata. A vibratory hammer may also be used but those produce a different type of discomfort and residents within 1 block may need to take the crystal off the shelf. Personal Items tend to walk around on shelves and the vibration can also be physically uncomfortable for those with 50' of the work. Vibratory hammers typically require more driving time per pile but it is not expected to exceed 15 minutes per pile and they will have a similar production rate of 1 pile every 45 minutes.

O) All equipment is expected to work from "flexi float" platforms built for this project and resold for what they can get when the job is done.

P) The City can put restrictions on the type of equipment the contractor can use. Diesel equipment is the most common and will result in the lowest fee if all other issues wore equal. If the City or HOA do not want diesel, then an electric hammer is feasible.

8.8.1.2 Waterside Repair Negative Factors

A) Cost - Current steel cost makes this an expensive option due to the significant length of the "H" piles cantilevering out of the soft seafloor to brace the wall

B) Steel corrodes and the "H" piles require epoxy coating and sacrificial cathodic protection anodes to fight this. The first 15 years will be maintenance free but after that a program of recoating on 12-15 year intervals is anticipated. Active impressed current is a reasonable cost option to be considered by the design team but requires maintenance which has historically been a weakness of maritime facilities.

C) "H" piles will reduce the channel width an estimated 16" on each side.

D) Requires temporary relocation of the floating dock systems and a mid-sized floating crane to drive the piles.

E) Storm drains must miss "H" pile locations.

F) Gangway platforms and float guide piles require replacement at cost to the City.

8.8.2 Landside Repair Option

The Landside Repair method attempts to address replacement of the aging seawall, cap and tie-rods with the least impact on the adjacent properties, channel depth, channel width and eliminate corrosive materials from the construction, as practical. This method utilizes a combination of waterside fiberglass sheet piling to seal the wall and composite caisson on the landside to brace the tieback rods required for the fiberglass sheet pile wall. This Option consists of:

Pressure washing the existing wall face to remove potential voids of marine growth if trapped in the repair

Removing the sidewalks adjacent to the seawall to excavate the 24" diameter caissons at 5' to 10' on-center, to be optimized by the designer

Lining the caisson excavation with a hollow fiberglass pipe pile

Driving new fiberglass sheet piling a few inches clear of the existing wall

Installing the rebar cage inside the fiberglass lined caisson

Marking and drilling holes thru the existing wall and fiberglass pipe to install (2) new tie rods into each caisson

Dewater and fill the caisson with concrete

Attach a new high and low dense plastic composite wale to the face of the new sheet piling and secure with the new tie rods

Fill the void between the existing and new seawall with concrete grout up to the base of the existing cap

Demolish the existing seawall cap and construct a new cap to encapsulate both the top of the new fiberglass sheets and the existing sheet piles

Install a new guard rail on the cap

Bury the caissons tops, re-grade and construct new sidewalks

Refer to Figures 8-8 & 8-9 on the following pages for graphics of the Landside Repair Option & Section 9 for cost estimate.

8.8.2.1 Landside Repair Positive Factors

A) Does not require major landside excavation and only requires minimal removal of existing homeowner improvements for construction.

B) Highly resistant to corrosion results in minimal maintenance for the service life of the structure.

C) Method can be started and stopped easily for phasing or work interruptions

D) Repair will seal all voids, gaps and cracks in the existing seawall to alleviate the ongoing loss of soil and settlement due to tidal pumping.

E) Repair will replace the deteriorated seawall cap and be an aesthetic improvement.

F) Repair will improve the seismic capacity of the seawall and protect the value of adjacent residential properties.

G) The sidewalk system will have few, if any closures for wall or tie-back repairs for the next 25-50 years.

H) Repair can address sea level rise, if desired, at minimal cost.

I) No effect on floating dock vertical clearances.

J) Can use a smaller crane than the Waterside Repair Option to perform work.

K) Lesser construction cost than Waterside Repair.

8.8.2.2 Landside Repair Negative Factors

A) Requires longer disruption of public walkway.

B) The fiberglass sheet pile work requires a slightly longer construction schedule than Waterside Repair Option.

C) Reduces the channel width and estimated 12" on each side + 8.5" more at each whaler.

D) Requires temporary relocation of the floating dock systems.

E) Ultraviolet light on the waterfront will fade the color pigment in the fiberglass sheet piling and wales but this does not weaken the members.

F) Gangway platforms and float guide piles require replacement at cost to the City

G) Continuous sheet pile systems may hit more obstruction buried below the mud and have to work around them rather than beat thru them, which an H pile can often do.

H) Storm drains require careful coordination of special integrated drain extension sheet pile sections.

I) Sidewalk work will be dirty work with removal of wet soil during caisson construction

Preferred Option: The Consultant prefers the Waterside Option, if the steel can be coated in plastic or high performance epoxy. This option's simplicity will result in fewer construction conflicts and change orders for unknown conditions.

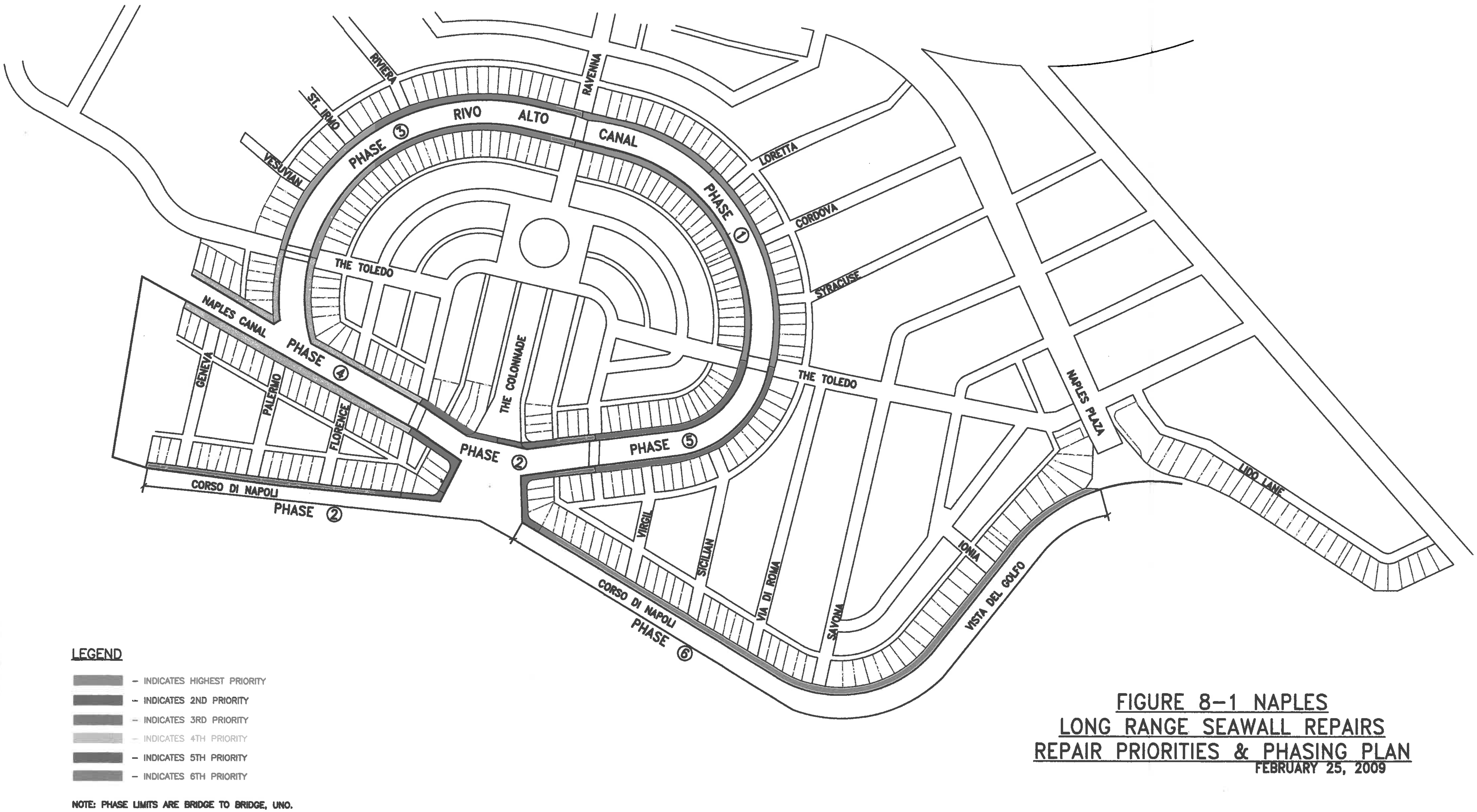
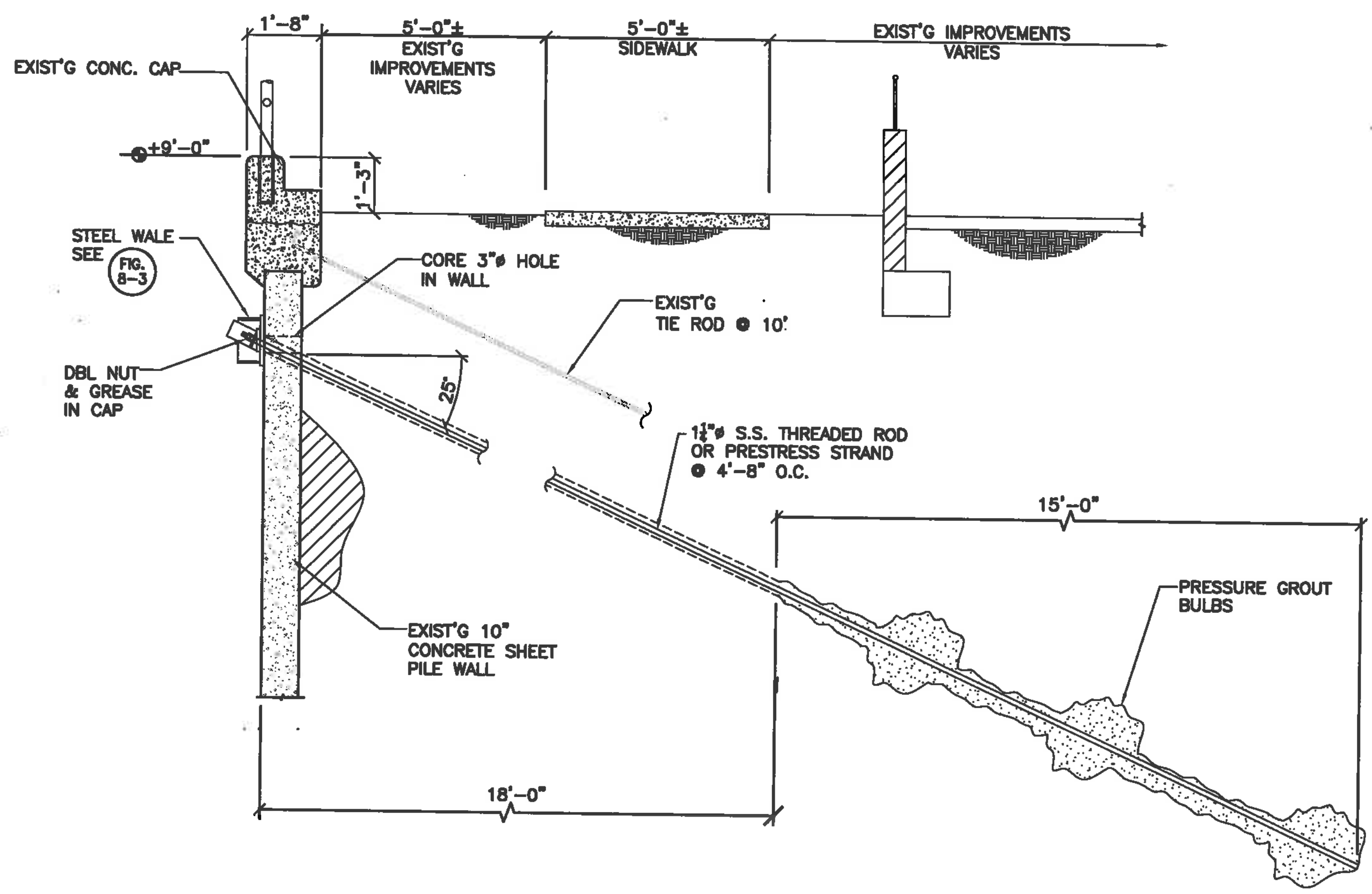
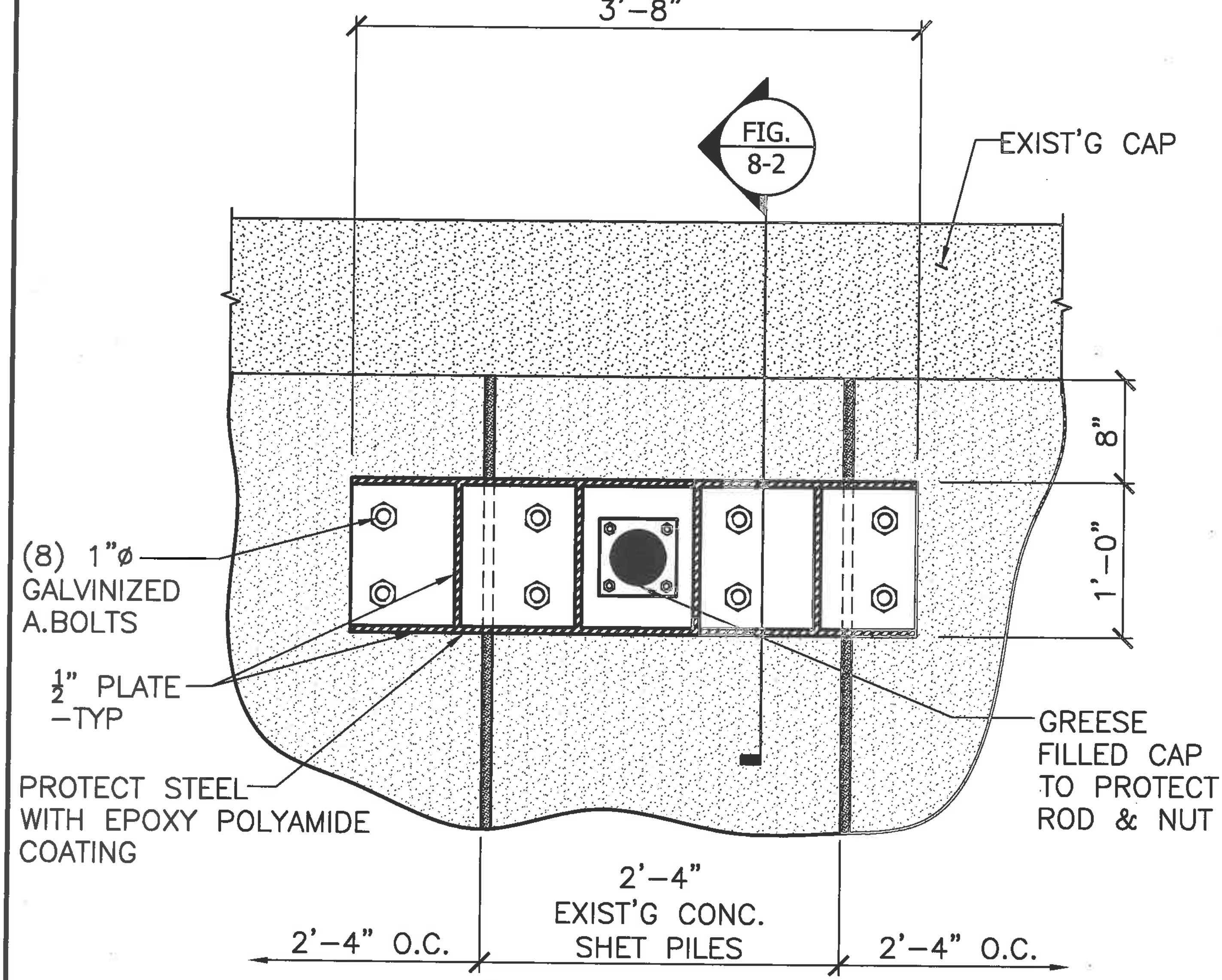


FIGURE 8-1 NAPLES
LONG RANGE SEAWALL REPAIRS
REPAIR PRIORITIES & PHASING PLAN
FEBRUARY 25, 2009

FIGURE 8-1



TranSystems PROJ NO: SCALE: 3/16" = 1'-0" DATE: 11/20/08 DESIGNED BY: C.D.D. DRAWN BY: D.V.G. CHECKED BY:		SHEET TITLE INTERIM REPAIRS TIEBACK REPAIR - SECTION PROJECT NAPLES SEAWALL INVESTIGATION FEBRUARY 2009 FIG. 8-2



TranSystems

PROJ NO:	
SCALE:	1" = 1'-0"
DATE:	2-5-09
DESIGNED BY:	C.D.D.
DRAWN BY:	D.V.G.
CHECKED BY:	

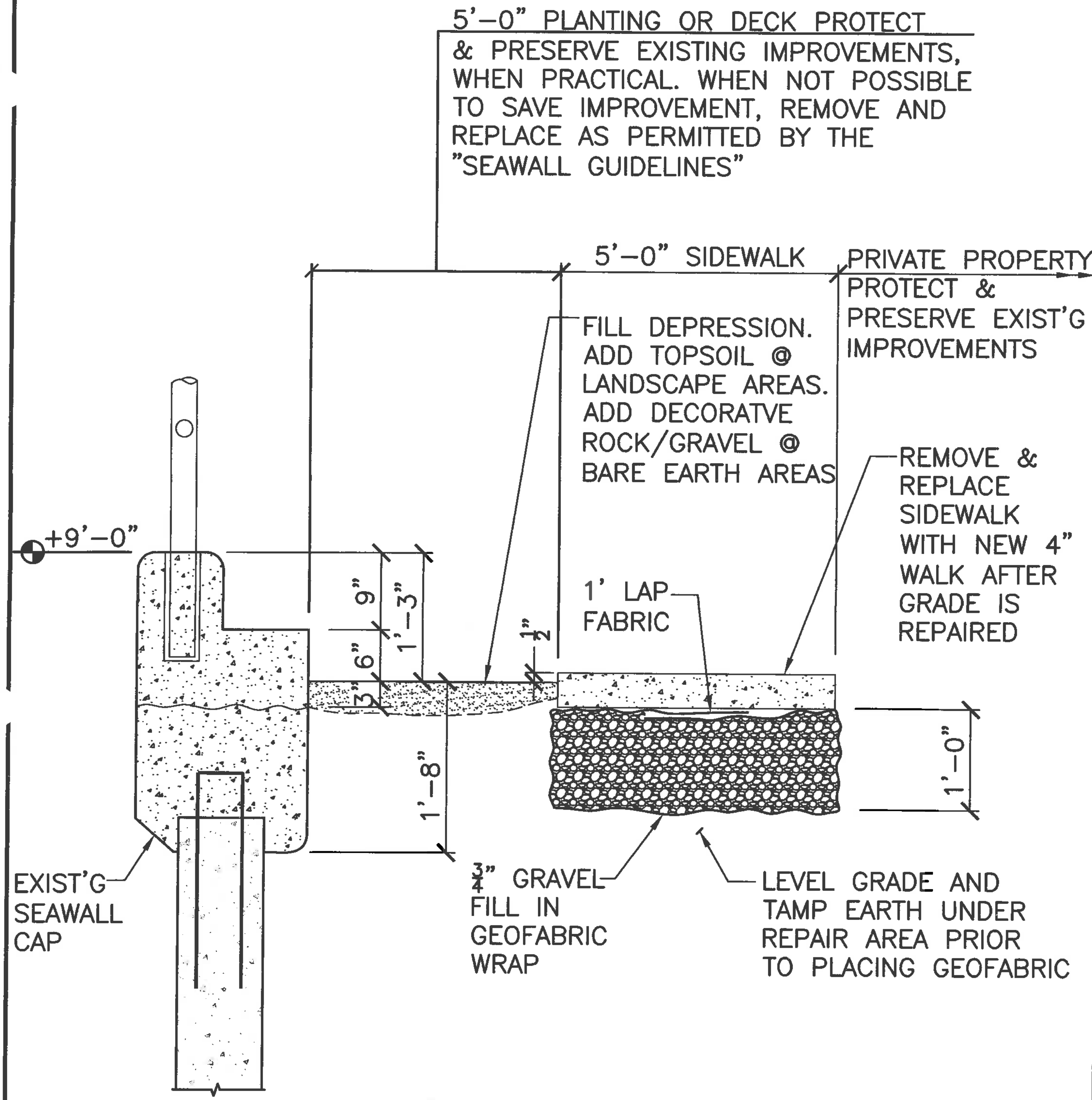
SHEET TITLE

INTERIM REPAIRS
TIEBACK WALE - ELEVATION

PROJECT

NAPLES SEAWALL
INVESTIGATION

FEBRUARY 2009 | **FIG. 8-3**



TranSystems

PROJ NO:
SCALE: 3/4" = 1'-0"
DATE: 2-5-09
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE

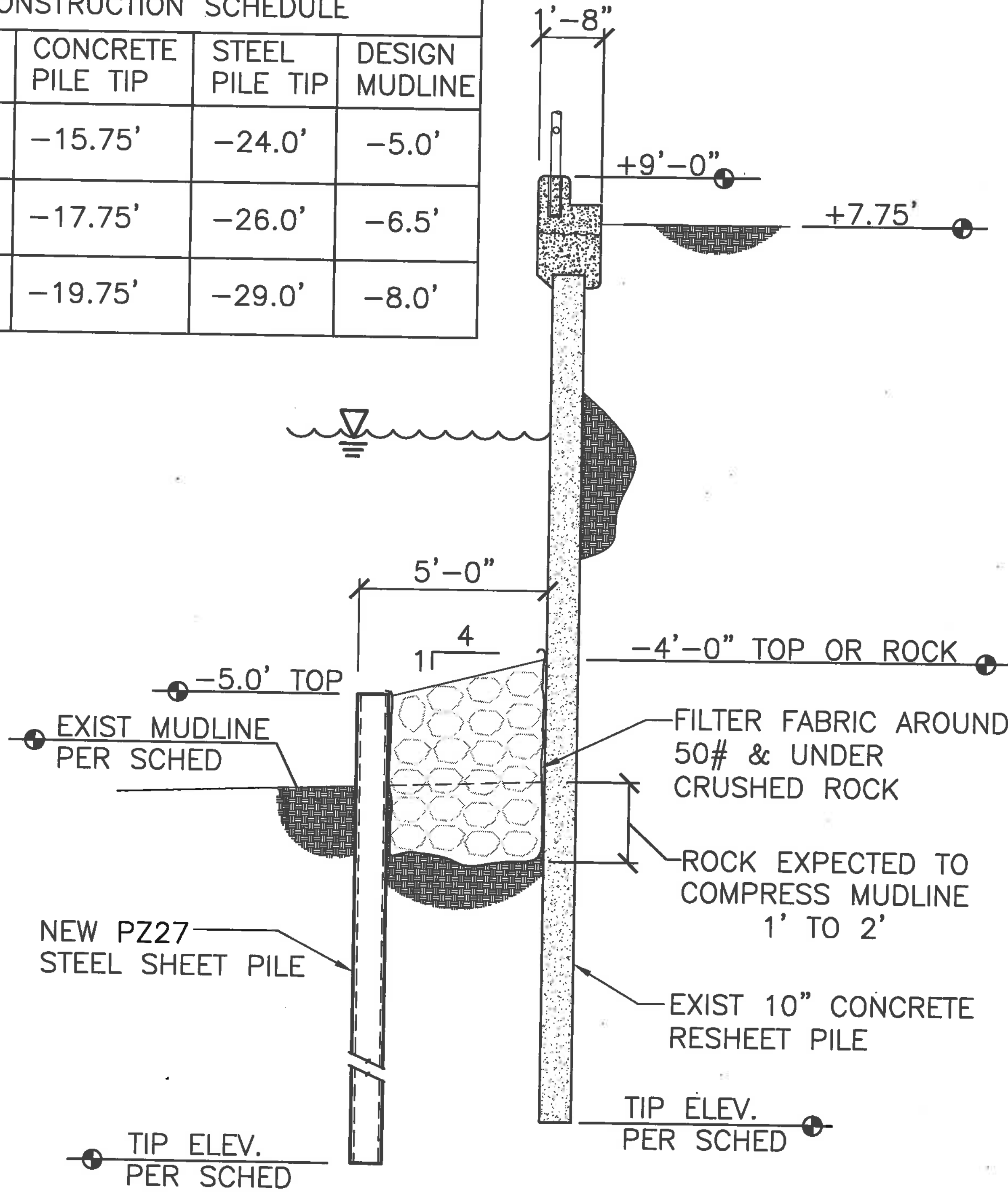
INTERIM REPAIRS
SINKHOLE REPAIR – SECTION

PROJECT

NAPLES SEAWALL INVESTIGATION

FEBRUARY 2009 **FIG. 8–4**

CONSTRUCTION SCHEDULE			
LOC.	CONCRETE PILE TIP	STEEL PILE TIP	DESIGN MUDLINE
22' SHEET	−15.75'	−24.0'	−5.0'
24' SHEET	−17.75'	−26.0'	−6.5'
26' SHEET	−19.75'	−29.0'	−8.0'



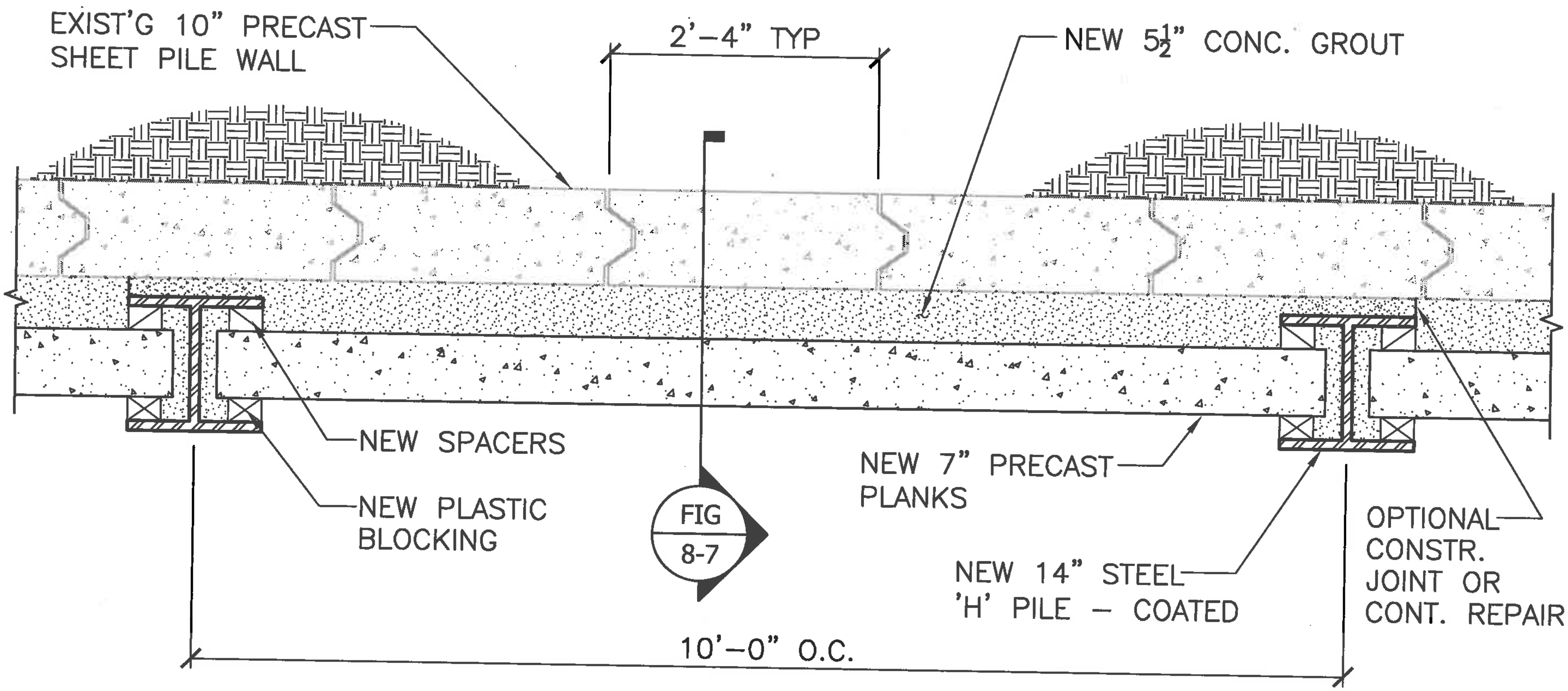
TranSystems

PROJ NO:
SCALE: 1/4" = 1'-0"
DATE: 2-5-09
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE
INTERIM REPAIRS BULKHEAD REPAIR

PROJECT
NAPLES SEAWALL INVESTIGATION

FEBRUARY 2009 | **FIG. 8–5**



TranSystems

PROJ NO:
SCALE: 3/4" = 1'-0"
DATE: 11/20/08
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

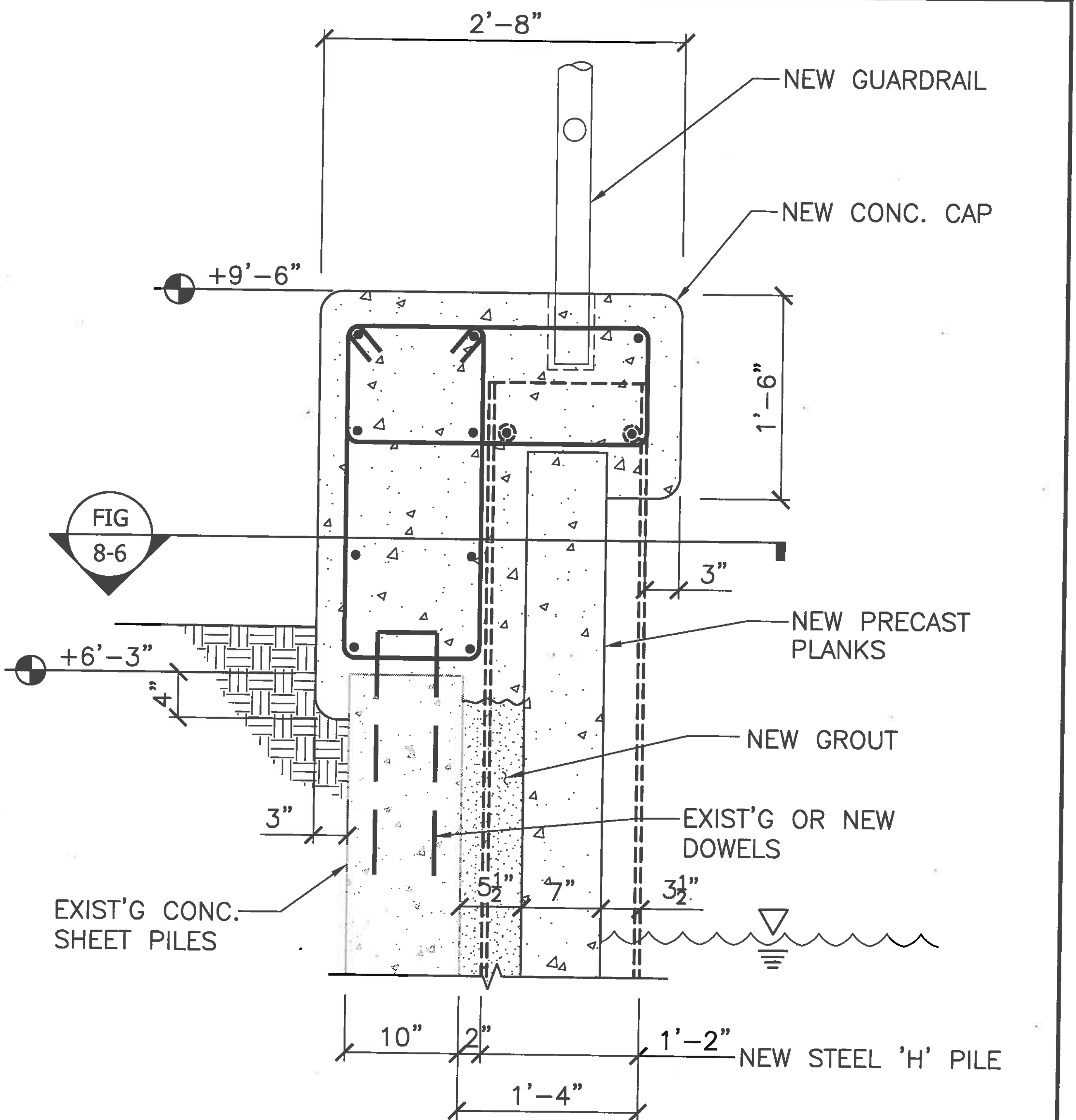
SHEET TITLE

LONG RANGE WATERSIDE REPAIR – PLAN

PROJECT

NAPLES SEAWALL INVESTIGATION

FIG. 8–6



TranSystems

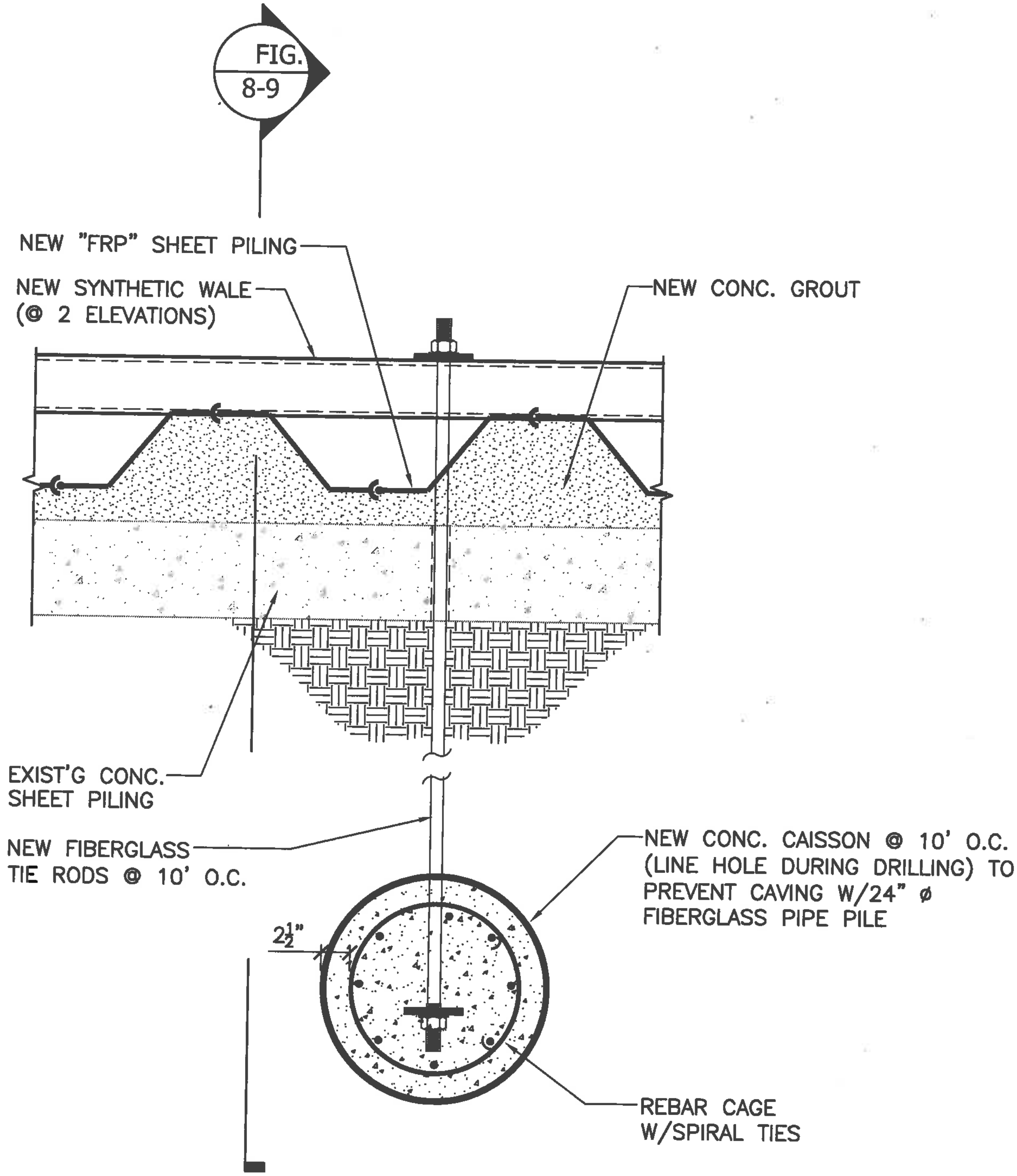
PROJ NO:
SCALE: 3/4" = 1'-0"
DATE: 11/20/08
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE
LONG RANGE
WATERSIDE REPAIR – SECTION

PROJECT
NAPLES SEAWALL
INVESTIGATION

FIGURE 8–7

FIG. 8–7



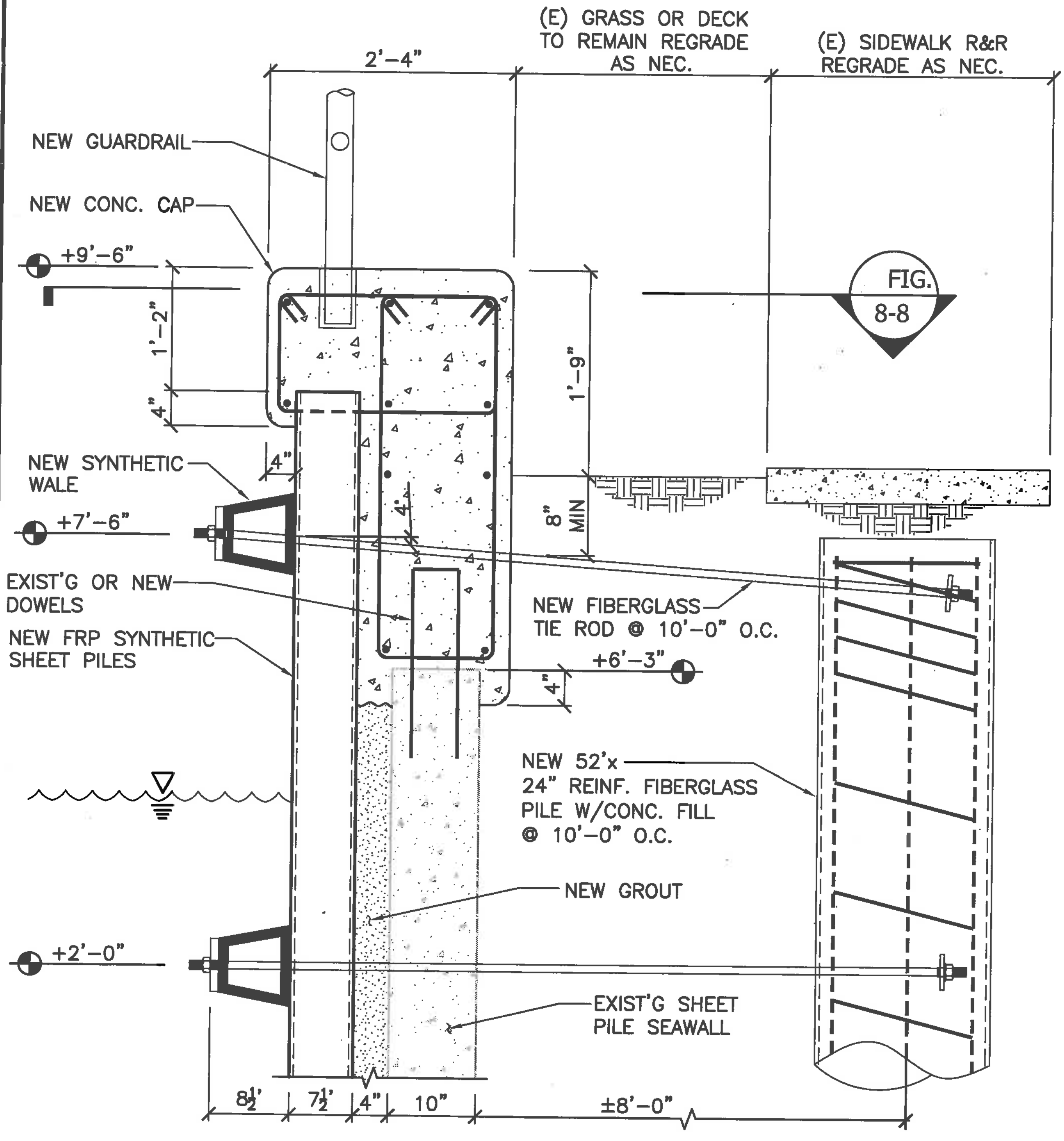
TranSystems

PROJ NO:
SCALE: 3/4" = 1'-0"
DATE: 11/20/08
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE
LONG RANGE
LANDSIDE REPAIR – PLAN

PROJECT
NAPLES SEAWALL
INVESTIGATION

FIG. 8–8



TranSystems

PROJ NO:	
SCALE:	3/4" = 1'-0"
DATE:	11/20/08
DESIGNED BY:	C.D.D.
DRAWN BY:	D.V.G.
CHECKED BY:	

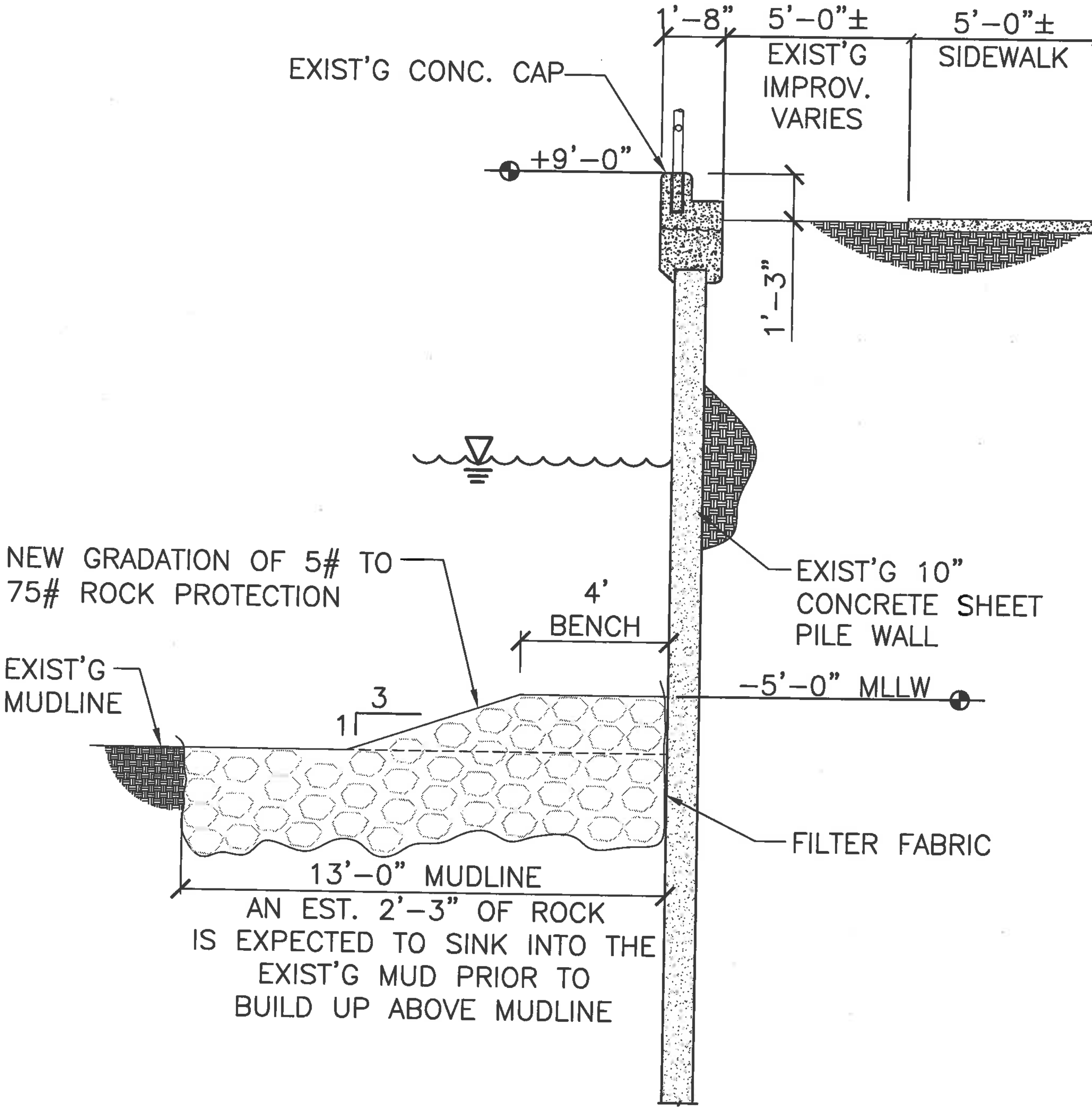
SHEET TITLE

LONG RANGE LANDSIDE REPAIR – SECTION

PROJECT

NAPLES SEAWALL INVESTIGATION

FIG. 8–9



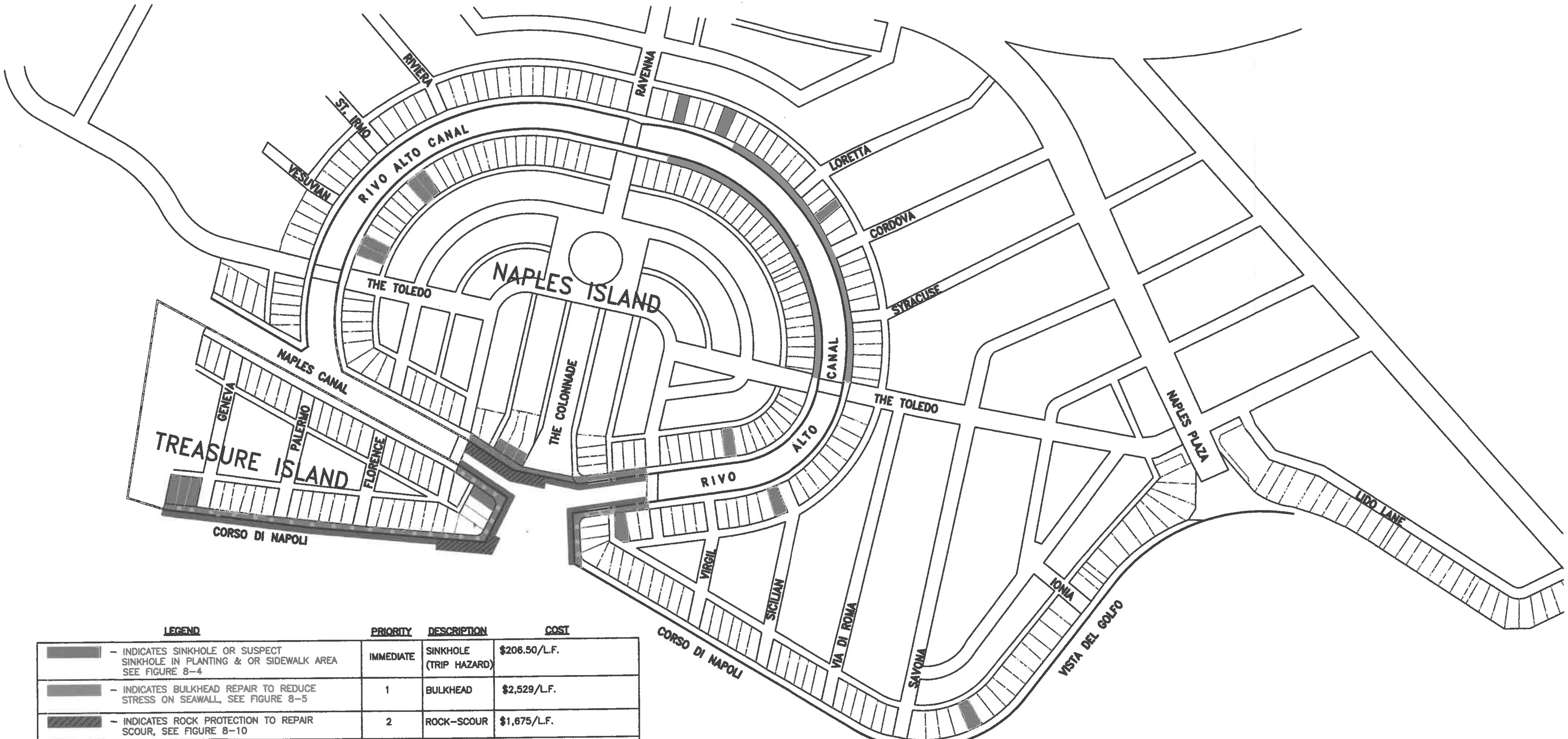
TranSystems

PROJ NO:
SCALE: 1/4" = 1'-0"
DATE: 2-5-09
DESIGNED BY: C.D.D.
DRAWN BY: D.V.G.
CHECKED BY:

SHEET TITLE
INTERIM REPAIRS
SCOUR REPAIR – SECTION

PROJECT
NAPLES SEAWALL INVESTIGATION

FEBRUARY 2009
FIG. 8–10



LEGEND	PRIORITY	DESCRIPTION	COST
– INDICATES SINKHOLE OR SUSPECT SINKHOLE IN PLANTING & OR SIDEWALK AREA SEE FIGURE 8-4	IMMEDIATE	SINKHOLE (TRIP HAZARD)	$206.50/L.F.
– INDICATES BULKHEAD REPAIR TO REDUCE STRESS ON SEAWALL, SEE FIGURE 8-5	1	BULKHEAD	$2,529/L.F.
– INDICATES ROCK PROTECTION TO REPAIR SCOUR, SEE FIGURE 8-10	2	ROCK-SCOUR	$1,675/L.F.
– INDICATES REMOVE SURCHARGE (WHERE OCCURS). SEE PG. 8-9, PARA. 3, OF REPORT	3	SURCHARGE	N/A
– INDICATES PROPOSED TIEBACK LOCATIONS TO STRENGTHEN WALL. (WHERE CURRENTLY DO NOT OCCUR), SEE FIGURES 8-2 & 8-3	4	TIEBACKS	$2,039/L.F.
– INDICATES ROCK PROTECTION TO REDUCE SEISMIC RISK, SEE FIGURE 8-10	5	ROCK-SEISMIC	$1,340/L.F. (80% OF SCOUR REPAIR)
– INDICATES SINKHOLE OR SUSPECT SINKHOLE IN PLANTING & OR SIDEWALK AREA SEE FIGURE 8-4	6	SINKHOLE (NO TRIP HAZARD)	$206.50/L.F.

NTS

FIGURE 8-11 NAPLES INTERIM REPAIRS – SITE KEY PLAN

FEBRUARY 25, 2009

FIGURE 8-11

Long Range Landside Repair - Option	COST ESTIMATE	DATE PREPARED: February 25, 2009	SHEET: 1 OF 2
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-8 & 8-9)	A/E CONTRACT NUMBER Project No. PW 8110-14		IDENTIFICATION NUMBER
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations	ESTIMATED BY: TRANSYSTEMS CORPORATION		CATEGORY CODE NUMBER
	STATUS OF DESIGN: PRELIMINARY DESIGN		DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Landside Repair - Option								
Mobilization - Required for each Phas	6	LS			75000.00	450,000	75,000.00	450,000
Barge Crane - Rental	42	MO	40000.00	1,680,000			40,000.00	1,680,000
Materials Barge - Rental	42	MO	10000.00	420,000			10,000.00	420,000
A) Docks								
Remove existing docks & guide pipes	225	EA			600.00	135,000	600.00	135,000
Reinstall docks w/ new guide piles	225	EA	1000.00	225,000	1800.00	405,000	2,800.00	630,000
(after wall work is complete)								
R & R Gangways & Platforms	225	EA	200.00	45,000	1000.00	225,000	1,200.00	270,000
B) FRP Sheet Piling								
Install new FRP sheet piles	319200	SF	8.00	2,553,600	8.00	2,553,600	16.00	5,107,200
Install new Synthetic Wales	22800	LF	30.00	684,000	30.00	684,000	60.00	1,368,000
Install new FRP rods	3800	EA	60.00	228,000	60.00	228,000	120.00	456,000
Install new hardware	15200	EA	6.00	91,200	6.00	91,200	12.00	182,400
C) Composite Soldier Piles								
Auger holes	98800	LF			40.00	3,952,000	40.00	3,952,000
Install 24" dia fiberglass shell	98800	LF	75.00	7,410,000	25.00	2,470,000	100.00	9,880,000
Install rebar cage	1900	EA	700.00	1,330,000	700.00	1,330,000	1,400.00	2,660,000
Place concrete fill	11500	CY	125.00	1,437,500	125.00	1,437,500	250.00	2,875,000
Extend existing storm drains	45	EA	2500.00	112,500	2500.00	112,500	5,000.00	225,000
D) Grout Fill								
Temp Silt Curtain	250	MOV	150.00	37,500	200.00	50,000	350.00	87,500
Power wash existing seawall	13000	SY			20.00	260,000	20.00	260,000
(prior to placing wall panels)								
Place Concrete Grout	3488	CY	120.00	418,560	120.00	418,560	240.00	837,120

Long Range Landside Repair - Option	COST ESTIMATE	DATE PREPARED: February 25, 2009	SHEET: 2 OF 2
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-8 & 8-9)	A/E CONTRACT NUMBER Project No. PW 8110-14		IDENTIFICATION NUMBER
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations	ESTIMATED BY: TRANSYSTEMS CORPORATION		CATEGORY CODE NUMBER
	STATUS OF DESIGN: PRELIMINARY DESIGN		DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
E) Guard Railing								
Remove existing Railing	11400	LF			2.50	28,500	2.50	28,500
Install new Railing	11400	LF	35.00	399,000	35.00	399,000	70.00	798,000
(after new cap is complete)								
F) Concrete Cap								
Remove existing Cap	53067	CF			15.00	796,005	15.00	796,005
Construct new Cap (after grout fill)	68172	CF	15.00	1,022,580	30.00	2,045,160	45.00	3,067,740
G) Subsidence Repairs								
Remove existing damaged sidewalks	68400	SF			2.25	153,900	2.25	153,900
Regrade, Add soil as needed	68400	SF	1.00	68,400	2.00	136,800	3.00	205,200
Construct new sidewalks	68400	SF	2.00	68,400	4.00	273,600	6.00	410,400
Construct new light pole foundations	6	EA	300.00	1,800	800.00	4,800	1,100.00	6,600
Relocate lights to new foundations	6	EA	6200.00	37,200	800.00	4,800	7,000.00	42,000
Raw Cost SUBTOTAL								**36,983,565**
General Requirements	10%							3,698,400
Contractor Overhead and Profit	15%							5,547,500
Burdened Cost SUBTOTAL								**46,229,465**
Design/Construction Contingency	15%							6,934,400
Design & Permitting	6%							2,773,800
Construction Administration (6%)	6%							2,773,800
TOTAL PROGRAM COST IN 2009 DOLLARS								**$58,711,465**
	divide by 11,400 feet =				**UNIT PRICE $5,150 / LF**			

Long Range
Waterside Repair - Option

COST ESTIMATE

DATE PREPARED: February 25, 2009

SHEET: 1 OF 2

ACTIVITY AND LOCATION:
Naples Seawall Repairs
Long Beach, CA (FIGURE 8-6 & 8-7)

A/E CONTRACT NUMBER
Project No. PW 8110-14

IDENTIFICATION NUMBER

PROJECT TITLE:
Naples Seawall Stability Investigation and Repair Recommendations

ESTIMATED BY:
TRANSYSTEMS CORPORATION

CATEGORY CODE NUMBER

STATUS OF DESIGN:
PRELIMINARY DESIGN

DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Waterside Repair - Option								
Mobilizaton - Required for each Phase	6	LS			75000.00	450,000	75,000.00	450,000
Barge Crane - Rental	36	MO	55000.00	1,980,000			55,000.00	1,980,000
Materials Barge - Rental	36	MO	10000.00	360,000			10,000.00	360,000
A) Docks								
Remove existing docks & guide pipes	225	EA			600.00	135,000	600.00	135,000
Reinstall docks w/ new guide piles	225	EA	1000.00	225,000	1800.00	405,000	2,800.00	630,000
(after wall work is complete)								
R & R Gangways & Platforms	225	EA	1000.00	225,000	600.00	135,000	1,600.00	360,000
B) Soldier Piles								
Install new steel H piles @ 6'	1900	EA	7175.00	13,632,500	900.00	1,710,000	8,075.00	15,342,500
(W14x193 x 52' epoxy coated top half)								
Install cathodic protection anodes	1900	EA	750.00	1,425,000	750.00	1,425,000	1,500.00	2,850,000
C) Precast Wall Panels								
Trench for precast conc panels	11400	LF			20.00	228,000	20.00	228,000
(prior to installing H piles)								
Install precast concrete panels	2280	EA	1475.00	3,363,000	400.00	912,000	1,875.00	4,275,000
Install plastic lumber blocking	2280	EA	750.00	1,710,000	400.00	912,000	1,150.00	2,622,000
Extend existing stormdrains	45	EA	500.00	22,500	2500.00	112,500	3,000.00	135,000
D) Grout Fill								
Temp Silt Curtain	250	MOV	150.00	37,500	200.00	50,000	350.00	87,500
Powerwash existing seawall	13000	SY			20.00	260,000	20.00	260,000
(prior to placing wall panels)								
Place Concrete Grout	2111	CY	120.00	253,320	120.00	253,320	240.00	506,640

Long Range Waterside Repair - Option	COST ESTIMATE	DATE PREPARED: February 25, 2009	SHEET: 2 OF 2
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-6 & FIGURE 8-7)	A/E CONTRACT NUMBER Project No. PW 8110-14		IDENTIFICATION NUMBER
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations	ESTIMATED BY: TRANSYSTEMS CORPORATION		CATEGORY CODE NUMBER
	STATUS OF DESIGN: PRELIMINARY DESIGN		DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
E) Guard Railing								
Remove existing Railing	11400	LF			2.50	28,500	2.50	28,500
Install new Railing	11400	LF	35.00	399,000	35.00	399,000	70.00	798,000
(after new cap is complete)								
F) Concrete Cap								
Remove existing Cap	53067	CF			15.00	796,005	15.00	796,005
Construct new Cap (after grout fill)	76351	CF	15.00	1,145,265	30.00	2,290,530	45.00	3,435,795
G) Subsidence Repairs								
Remove existing damaged sidewalks	68400	SF			2.25	153,900	2.25	153,900
Regrade, Add soil as needed	68400	SF	1.00	68,400	2.00	136,800	3.00	205,200
Construct new sidewalks	68400	SF	2.00	68,400	4.00	273,600	6.00	410,400
Construct new lightpole foundations	6	EA	300.00	1,800	800.00	4,800	1,100.00	6,600
Relocate lights to new foundations	6	EA	6200.00	37,200	800.00	4,800	7,000.00	42,000
Raw Cost SUBTOTAL								**36,098,040**
General Requirements	10%							3,609,800
Contractor Overhead and Profit	15%							5,414,700
Burdened Cost SUBTOTAL								**45,122,540**
Design/Construction Contingency	15%							6,768,400
Design & Permitting	6%							2,707,400
Construction Administration (6%)	6%							2,707,400
TOTAL PROGRAM COST IN 2009 DOLLARS								**$57,305,740**
				UNIT PRICE $5,027 / LF				

Interim Repair - Sink Holes			COST ESTIMATE		DATE PREPARED: February 6, 2009		SHEET: 1	
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-4)			A/E CONTRACT NUMBER Project No. PW 8110-14				IDENTIFICATION NUMBER	
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations			ESTIMATED BY: TRANSYSTEMS CORPORATION				CATEGORY CODE NUMBER	
			STATUS OF DESIGN: PRELIMINARY DESIGN				DELIVERY ORDER NUMBER	
	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
ITEM DESCRIPTION	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Intermediate Repair - Sink Holes								
Mobilization - Required for each Phas	1	LS			10000.00	10,000	10,000.00	10,000
Concrete demolition & disposal	5000	SF	1.00	5,000	5.00	25,000	6.00	30,000
(assume arbitrary 1000 feet of walk)								
Level fill & Tamp	5000	SF			4.00	20,000	4.00	20,000
Gravel & Filter Fabric Wrap	5000	SF	1.00	5,000	3.50	17,500	4.50	22,500
New 4" Concrete Walk	5000	SF	2.00	10,000	7.50	37,500	9.50	47,500
Raw Cost SUBTOTAL								**130,000**
General Requirements	10%							13,000
Contractor Overhead and Profit	15%							19,500
Burdened Cost SUBTOTAL								**162,500**
Design/Construction Contingency	15%							24,400
Design & Permitting	6%							9,800
Construction Administration (6%)	6%							9,800
TOTAL PROGRAM COST IN 2009 DOLLARS								**$206,500**
	divide by arbitrary 1000 feet of walk =				**UNIT PRICE $206.50 / L. F.**			
					Assumes 5' wide sidewalk			

Interim Repair - Bulkhead Repair	COST ESTIMATE	DATE PREPARED: February 25, 2009	SHEET: 1 OF 1
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-5)	A/E CONTRACT NUMBER Project No. PW 8110-14		IDENTIFICATION NUMBER
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations	ESTIMATED BY: TRANSYSTEMS CORPORATION		CATEGORY CODE NUMBER
	STATUS OF DESIGN: PRELIMINARY DESIGN		DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Intermediate Repair - Bulkhead								
assume arbitrary 400' repair								
Mobilization - Required for each Phase	1	LS			45000.00	45,000	45,000.00	45,000
Move Boat Docs twice	8	LS	4000.00	32,000			4,000.00	32,000
Install PZ 27 x 21'	226800	SF	1.05	238,140	0.45	102,060	1.50	340,200
place filter fabric	8000	SF	1.50	12,000	4.50	36,000	6.00	48,000
Rock Rip Rap	700	ton	80.00	56,000	80.00	56,000	160.00	112,000
Barge & Crane Rental	1.5	m0	40000.00	60,000	80.00	120	40,080.00	60,120
Raw Cost SUBTOTAL								**637,320**
General Requirements	10%							63,700
Contractor Overhead and Profit	15%							95,600
Burdened Cost SUBTOTAL								**796,620**
Design/Construction Contingency	15%							119,500
Design & Permitting	6%							47,800
Construction Administration (6%)	6%							47,800
TOTAL PROGRAM COST IN 2009 DOLLARS								**$1,011,720**
	divide by 400 foot assumed repair =				**UNIT PRICE $2,529 / L. F.**			

Interim Repair - Tie Backs	COST ESTIMATE	DATE PREPARED: February 25, 2009	SHEET: 1 OF 1
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-2 & 8-3)	A/E CONTRACT NUMBER Project No. PW 8110-14		IDENTIFICATION NUMBER
	ESTIMATED BY: TRANSYSTEMS CORPORATION		CATEGORY CODE NUMBER
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations	STATUS OF DESIGN: PRELIMINARY DESIGN		DELIVERY ORDER NUMBER

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Intermediate Repair - Tie Backs								
Mobilization - Required for each Phas	1	LS			50000.00	50,000	50,000.00	50,000
Barge Crane - Rental	1.5	MO	13000.00	19,500			13,000.00	19,500
Drill Rental	1.5	MO	7000.00	10,500			7,000.00	10,500
Tieback Installation								
Drill & Grout tieback rods **(arbitrary #)**	40	EA	1250.00	50,000	2750.00	110,000	4,000.00	160,000
includes core drill, rod, wale & grout.								
Raw Cost SUBTOTAL								**240,000**
General Requirements	10%							24,000
Contractor Overhead and Profit	15%							36,000
Burdened Cost SUBTOTAL								**300,000**
Design/Construction Contingency	15%							45,000
Design & Permitting	6%							18,000
Construction Administration (6%)	6%							18,000
TOTAL PROGRAM COST IN 2009 DOLLARS								**$381,000**
	divide by **(arbitray #)** of tie-backs =				**UNIT PRICE $9,525 / EACH**			

Interim Repair - Light Pole		COST ESTIMATE		DATE PREPARED: February25, 2009		SHEET: 1 OF 1		
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (SEE MANUF. DATA AT END OF SECTION 9)		A/E CONTRACT NUMBER Project No. PW 8110-14				IDENTIFICATION NUMBER		
		ESTIMATED BY: TRANSYSTEMS CORPORATION				CATEGORY CODE NUMBER		
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations		STATUS OF DESIGN: PRELIMINARY DESIGN				DELIVERY ORDER NUMBER		

ITEM DESCRIPTION	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Intermediate Repair - Light Pole								
Mobilization - Required for each Phas	1	LS			10000.00	10,000	10,000.00	10,000
Pedestal Footing demo & disposal	15	EA	75.00	1,125	800.00	12,000	875.00	13,125
assume arbitrary 15 poles replaced)								
Level fill & Tamp	15	EA	25.00	375	400.00	6,000	425.00	6,375
Install new Chance Anchor Base	15	EA	500.00	7,500	500.00	7,500	1,000.00	15,000
Build new conc Pedestal & Walkway	15	EA	250.00	3,750	1200.00	18,000	1,450.00	21,750
Set light and electrical	15	EA	200.00	3,000	850.00	12,750	1,050.00	15,750
Raw Cost SUBTOTAL								**82,000**
General Requirements	10%							8,200
Contractor Overhead and Profit	15%							12,300
Burdened Cost SUBTOTAL								**102,500**
Design/Construction Contingency	15%							15,400
Design & Permitting	6%							6,200
Construction Administration (6%)	6%							6,200
TOTAL PROGRAM COST IN 2009 DOLLARS								**$130,300**
	divide by 15 pole repair assumption =				**UNIT PRICE $8,687 / EACH.**			

Interim Repair - Scour Repair		COST ESTIMATE			DATE PREPARED: February 25, 2009		SHEET: 1 OF 1	
ACTIVITY AND LOCATION: Naples Seawall Repairs Long Beach, CA (FIGURE 8-10)			A/E CONTRACT NUMBER Project No. PW 8110-14				IDENTIFICATION NUMBER	
PROJECT TITLE: Naples Seawall Stability Investigation and Repair Recommendations			ESTIMATED BY: TRANSYSTEMS CORPORATION				CATEGORY CODE NUMBER	
			STATUS OF DESIGN: PRELIMINARY DESIGN				DELIVERY ORDER NUMBER	
	QUANTITY		MATERIAL COST		LABOR COST		ENGINEERING ESTIMATE	
ITEM DESCRIPTION	NUMBER	UNIT	UNIT COST	TOTAL	UNIT COST	TOTAL	UNIT COST	TOTAL
Intermediate Repair - Scour								
assume arbitrary 200' repair								
Mobilization - Required for each Phas	1	LS			20000.00	20,000	20,000.00	20,000
Move Boat Docs twice	2	LS	4000.00	8,000			4,000.00	8,000
place filter fabric	4000	SF	1.50	6,000	4.50	18,000	6.00	24,000
Rock Rip Rap	850	ton	80.00	68,000	80.00	68,000	160.00	136,000
Barge Rental	0.77	m0	30000.00	23,100			30,000.00	23,100
Raw Cost SUBTOTAL								**211,100**
General Requirements	10%							21,100
Contractor Overhead and Profit	15%							31,700
Burdened Cost SUBTOTAL								**263,900**
Design/Construction Contingency	15%							39,600
Design & Permitting	6%							15,800
Construction Administration (6%)	6%							15,800
TOTAL PROGRAM COST IN 2009 DOLLARS								**$335,100**
	divide by assumed 200' repair =				**UNIT PRICE $1675 / L. F.**			

SUPPLEMENTAL INFORMATION

MANUFACTURES CATALOG DATA
CONSTRUCTION COMPONENTS

DATA SHEETS

"NEXGEN" FRP SHEET PILING

"SUPER WALE" FOR FRP SHEET PILING

"SUPER LOC" ACCESSORIES FOR WALE SPLICING

"CHANCE" LIGHT POLE FOUNDATIONS

"CHANCE" TRIPOD FOUNDATION SHOP DRAWING

"BETHLEHEM STEEL" PZ 27 STEEL SHEET PILE DATA

Instant Foundations for Decorative and Architectural Lighting

Hassle-free
- No concrete
- No hole digging
- No weather delays

Preferred efficiency
- Immediate pole mounting
- Direct bolt-up and wiring

Simple installation
- By backhoe or skid steer
- Before landscaping
- Through asphalt

CHANCE®

CHANCE® Instant Foundations for De

Advantages versus Poured-in-Place Foundations

Install during all types of weather . . . foundations can be screwed down when temperatures are below freezing.	Installation must be made during relatively dry periods and above-freezing temperatures.
Foundation can be salvaged for reuse when lighting standard is relocated merely by reversing foundation out of the ground.	Virtually no reuse possibility . . . often times the concrete foundation must be removed when lighting standard is relocated. (Also true of pre-cast units.)
Lower installed costs. In most locations, it requires 10 minutes with a two-man crew.	Extensive excavations required – more than one craft required for forming, rebars, finishing, etc.
Easy to transport – weigh between 130 to 215 pounds.	Poured-in-place foundations require several pieces of equipment.
Eliminates need for expensive anchor bolts – transformer base or lighting standard can be bolted direct to foundation base . . . no problem of misaligning anchor bolts.	Long anchor bolts could add $20 or more to each pole installation cost . . . anchor bolts can be set "wrong" requiring expensive repairs.
Clean, no-excavation installation in residential and commercial areas.	Excess dirt (spoils) must be hauled away . . . heavy concrete trucks can mar lawns and parkways.
One-trip convenience . . . in less than 10 minutes, the crew is ready to set the standard.	Several trips to the site are required to excavate, form, pour and then, after curing, set the standard.
No requirement for leveling side slope of right-of-way.	Top of concrete foundation must be leveled on sloping right-of-way.

Nostalgic ligh
foundations
with renovati
and gutters.

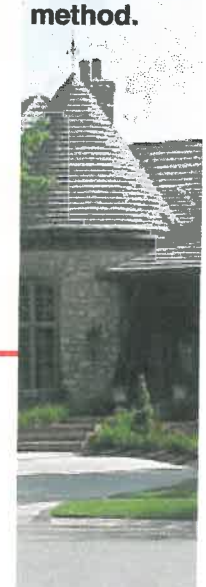
Foundations for signage supports also install immediately!

Easy to transport and store, for staging right on site!

Advantages versus Pre-Cast Foundations

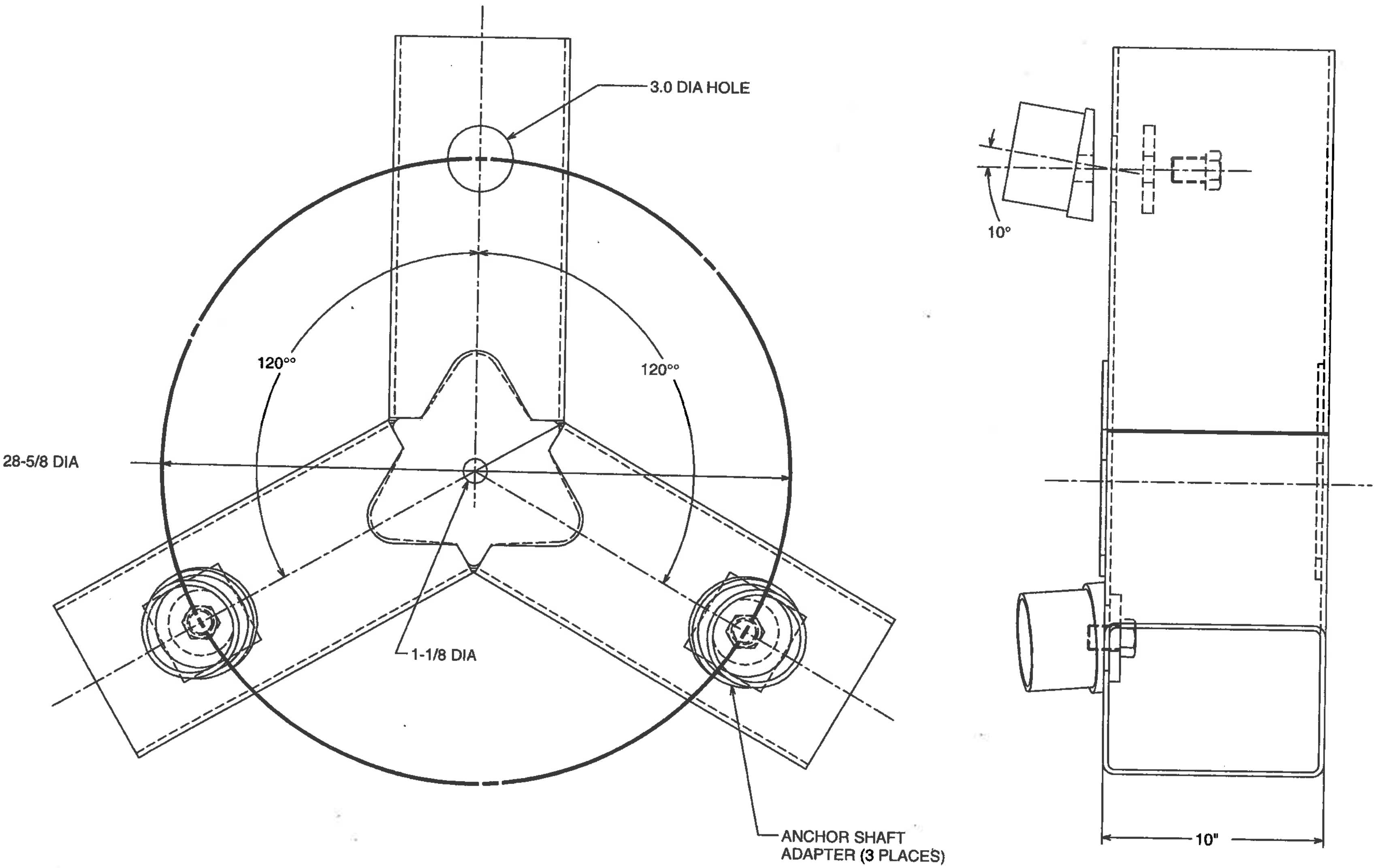
Easy to transport . . . several foundations can be delivered to the job site.	Extremely heavy and bulky units. Must be carefully handled when loading and unloading.
Simplified storage – can be stored in same yard adjacent to light standards and bases, stacked in relatively compact area.	Considerable storage space required for each individual pre-cast base.
All-weather installation using typical construction equipment.	In wet seasons, excavation can collapse before foundation is set in place.

Leaning light
direct-burial
can result fro
method.



Advantages versus Direct-Buried Pole Bases

Greater overturning moment resistance.	Low overturning moment resistance, prone to leaning.
Use same installing equipment that augers holes.	Results vary widely with crew competence.
Higher production rate, pole attaches immediately.	Require field inspection for adequate capacity.
Smaller crews.	Labor-intensive, time-consuming backfill compaction.



-NOTES-

1. HOT DIP GALV.
2. SHIPPED UNASSEMBLED.

PERMISSABLE ECCENTRICITY AND/OR ANGULAR DEVIATION IS RELATED TO LOAD, SOIL CONDITIONS AND ELEVATION ABOVE GRADE, AND SHOULD BE DISCUSSED WITH CHANCE APPLICATION ENGINEERS ON A PROJECT SPECIFIC BASIS.

MAX. COMPRESSION 125 KIPS.
GROSS WEIGHT 130 LBS.

LATEST EC NO:	REV.	DATE	CHANGED BY	RESP ENGINEER
EC02819	B	07-23-01	MARTIN	WEST

MARTIN 07-23-01

TRIPOD SUBASSEMBLY

SA107-000203

SuperWale™ Data Sheet

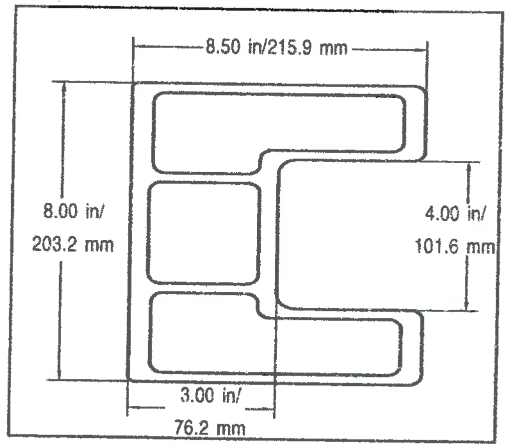
(Part Number SS810)

Refer to the SuperLoc™ Design/Installation Manual for Comprehensive Information

**Wale & Retaining Wall System (US Patent #6,893,191 B2/May 17, 2005)*

Physical Properties

Depth of Section	6.00 in.
	152.4 mm
Width of Section	8.00 in.
	203.2 mm
Minor Moment of Inertia	49.22 in^4
	2.05E+07 mm^4
Minor Section Modulus	14.40 in^3
	1.97E+07 mm^3
Weight	9.70 lbs./ft.
	14.44 kg/m
Area of Web	6.00 in^2
	3,870.96 mm^2



Note: Values are not factored, an appropriate safety factor must be applied

Mechanical Properties		Test Method	Average Values Imperial	Average Values Metric
Full Section Modulus of Elasticity	***	Lab	3.80E+06 psi	26,200 MPa
Full Section Modulus of Rigidity	***	Lab	450,000 psi	3,103 MPa
Ultimate Shear Capacity	***	Lab/Calculated	42,000 lbs.	19,051 kg.
Ultimate Moment Capacity	***	Lab/Calculated	48,000 ft.-lbs.	65,079 N-m
Tensile Strength LW	***	ASTM D638	43,000 psi	296 MPa
Compression Strength LW	***	ASTM D695	40,000 psi	276 MPa
Compression Strength CW	***	ASTM D695	13,000 psi	90 MPa
In-Plane Shear	***	Mod. D23441	7,000 psi[1]	48 MPa

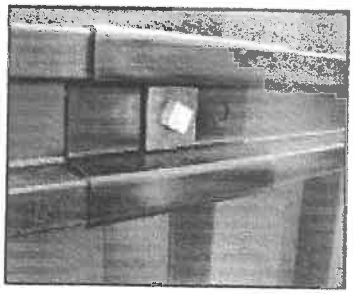
CW = Crosswise **LW = Lengthwise**

1. Follow ASTM D2344, but rotate the coupon 90 degrees (cut section of coupon length faces up)
2. Values are published as ultimate. Appropriate Safety Factors must be applied.

Mitered corner connectors and wale splices are available in ASTM 240 Grade Black Stainless Steel.

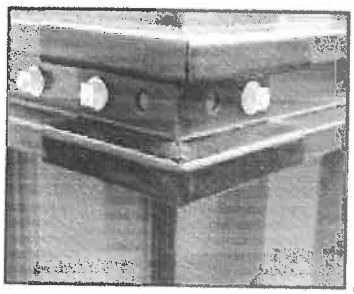
See Back for Additional Information on SuperLoc™ Accessories.

SuperLoc™ Accessories



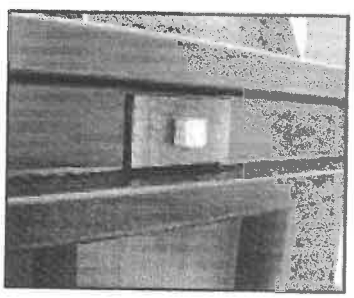
SuperWale™ W-Splice

Part No.: Fab.095
Material: ASTM 240 Black Stainless Steel
Dimension: 12" L x 8.4" W (305mm x 213mm), Hole Diameter 1.125" (29 mm)
Working Load Capacity: 12,000 lbs. (5,443 kg.)
Note: Tie-rod must be backed with a 3-1/4" x 3-1/4" x 1/4" (83 mm x 83 mm x 6 mm) steel washer.
Weight: 11 lbs. (5 kg)



SuperWale™ W-Corner Connector

Part No.: 90° Connector Fab.093
45° Connector Fab.094
Note: Special angles are available upon request; however, a 2-week lead time applies.
Material: ASTM 240 Black Stainless Steel
Dimension: Both sides 12" L x 8.4" W (304.8 mm x 213.36 mm), Hole Diameter drilled at 13/16" (20.64 mm) for a 3/4" x 4.5" (19.05 mm x 114.3 mm) bolt.
Weight: 23 lbs. (10 kg)



SuperWale™ ASTM 240 Black Stainless Steel Washers

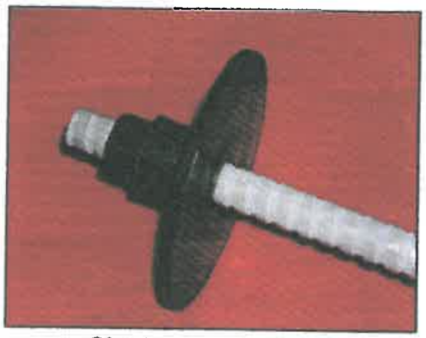
Part No.: 3-1/4" x 3-1/4" x 1/4" (82.55 mm x 82.55 mm x 6.35 mm) Washer Fab.096
3-1/4" x 3-1/4" x 3/8" (82.55 mm x 82.55 mm x 9.53 mm) Washer Fab.097
3-1/4" x 6" x 1/2" (82.55 mm x 152.4 mm x 12.7 mm) Washer Fab.098
Material: ASTM 240 Black Stainless Steel
Dimension: Hole diameter 1.125" (28.58 mm)

Working Load Capacity When Used With SuperWale™

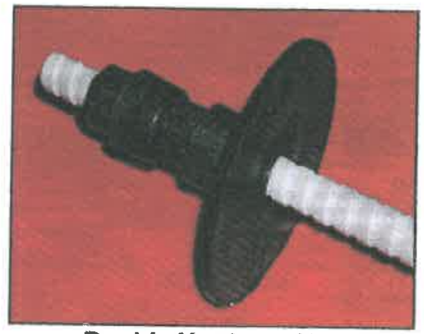
Allowable Tie-Rod Force with Specified Washers	ASTM 240 Steel	Weight
Allowable Tie-Rod Force	Steel Washer Dimensions	
4.250 lbs. (1,928 kg)	3-1/4" x 3-1/4" x 1/4" (82.55 mm x 82.55 mm x 6.35 mm)	0.75 lbs. (.34 kg)
6,700 lbs. (3,309 kg)	3-1/4" x 3-1/4 "x 3/8" (82.55 mm x 82.55 mm x 9.53 mm)	1.13 lbs. (.51 kg)
11,000 lbs. (4,990 kg)	3-1/4" x 6 "x 1/2" (82.55 mm x 152.4 mm x 12.7 mm)	2.75 lbs. (1.25kg)

Working Loads

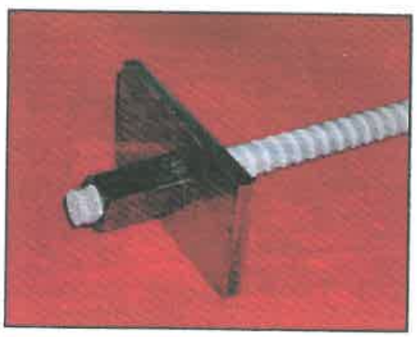
SuperRod™ has been subject to extensive mechanical testing. A Safety Factor of 1.60 has been applied to the Failure Load. Maximum recommended torque not to exceed 175 lbs-ft (260.43 kg/m).



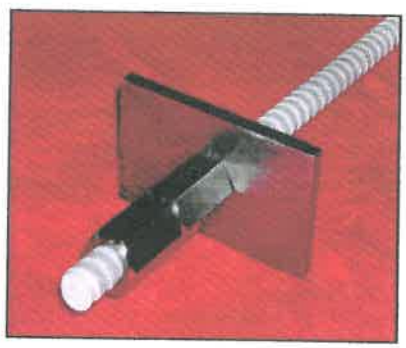
Single Urethane Nut
Working Load: 8,000 lbs. (3,629 kg.)



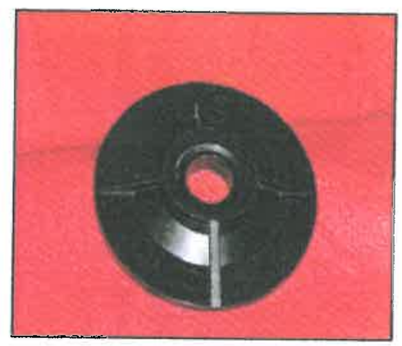
Double Urethane Nuts
Working Load: 13,750 lbs. (6,237 kg.)



SuperRod™
with One Stainless Steel Nut
Working Load: 15,450 lbs. (7,008 kg.)



SuperRod™
with Two Stainless Steel Nuts
Working Load: 26,250 lbs. (11,907 kg.)



SuperRod™
Washers: 6" (152 mm)
Glass Filled Polymer Washer.

Official Manufacturer's Representative

Lee COMPOSITES INC

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Spring, TX 77391-1286
Phone 281.376.6146
Fax 281.251.8461
E-mail: boblee@leecomposites.com
Web site: www.leecomposites.com

NAPLES SEAWALL INVESTIGATION
KEY PERSONNEL ROSTER

CITY OF LONG BEACH- CLIENT
DIVISION OF PUBLIC WORKS

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