Naval Facilities Engineering Command

ENGINEERING SERVICE CENTER
Port Hueneme, California 93043-4370

CONTRACTOR REPORT

DESKTOP ENGINEERING ANALYSIS AND LOAD ANALYSIS AT AUTEC – MAIN PIER FACILITY ANDROS ISLAND, BAHAMAS

Submission Date: Feb-17-2012 (100%)

Prepared By:

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DESKTOP ENGINEERING ANALYSIS AND LOAD ANALYSIS AT

AUTEC – MAIN PIER FACILITY ANDROS ISLAND, BAHAMAS

Submission Date: Feb-17-2012

Prepared for:

NAVFAC ESC
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Funding Provided By: CNIC
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AUTEC – Main Pier Facility
Andros Island, Bahamas

Section 1 - Executive Summary

Contract: N62473-06-D-3029 **Submission Date**: 02/17/2012 [100%]

Contractor: Halcrow, Inc. **Funding Provided By:** CNIC

22 Cortlandt Street, 31st Floor
New York, New York 10007-3107

Executive Summary:

The purpose of this desktop engineering analysis is to determine the operational load rating capacity of the pier based on the existing level of deterioration. The analysis is based on the original construction (1965) and pier modification (1983) design drawings and the inspection findings in the January 2010 Waterfront Facilities Inspection (WFI) report performed by Halcrow, Inc.

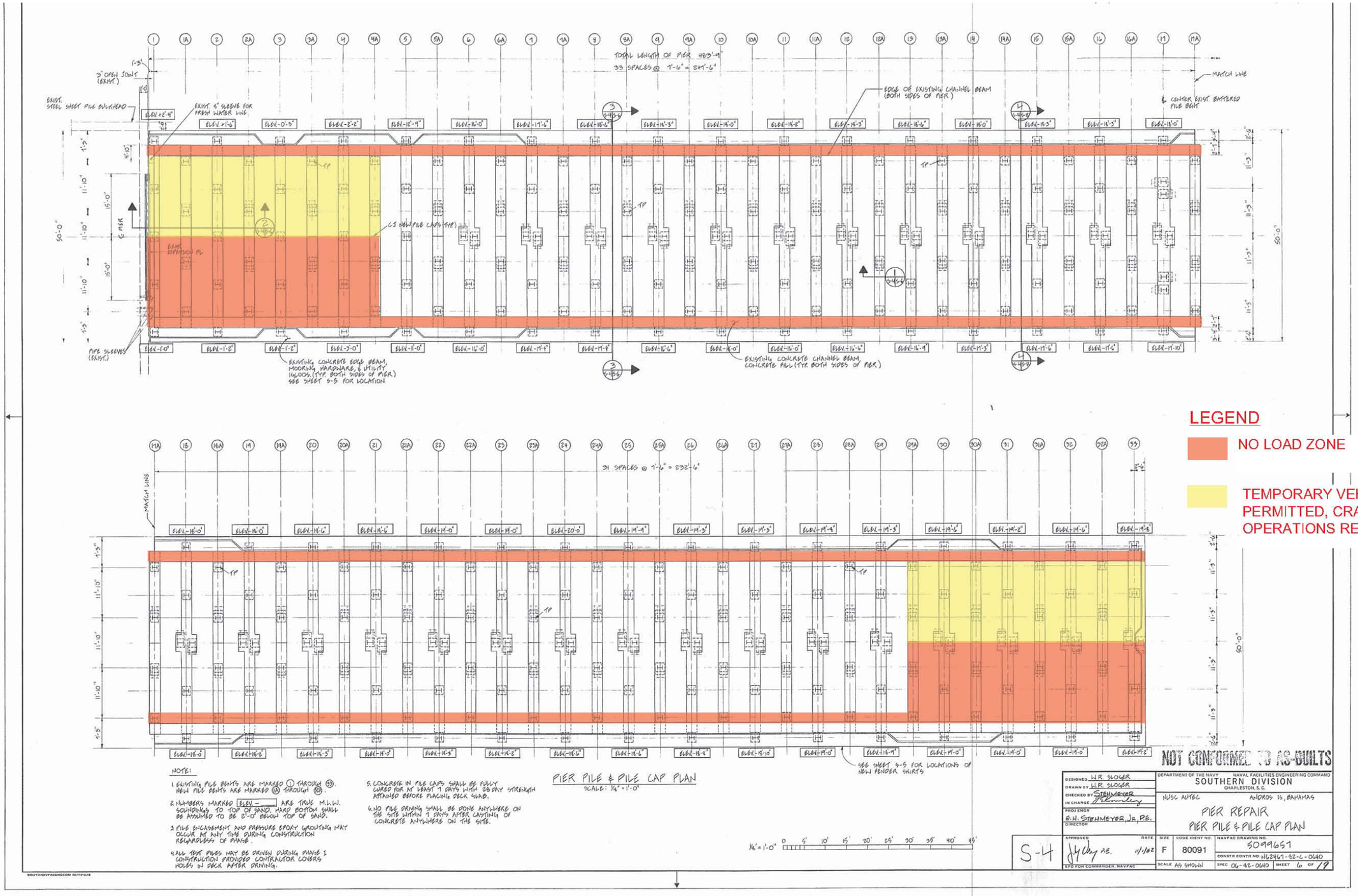
The Main Pier at AUTEC in Andros Island, Bahamas, is a steel H-pile supported reinforced concrete structure that is 484 ft long by 50 ft wide. There is a total of 327 concrete encased steel H-piles arranged in 66 bents. The piles support cast-in-place pile caps and precast reinforced concrete deck planks with a cast-in-place deck overlay.

In all load analysis cases, the outer 6 ft of the northern and southern longitudinal edges of the pier, and the southern half (25 ft) of the inshore 50 ft (from Bents 1 to 4A) and the offshore 50 ft (from Bents 29A to 33) should not be loaded due to severe deterioration noted during the most recent Halcrow inspection. In addition to the severe deterioration, there are two broken deck planks located at the southern half of the inshore 50 ft and one broken deck plank at the offshore 50 ft. These areas will be referred to throughout as “NO LOAD ZONES” and are depicted in Figure 1.

Outside of the NO LOAD ZONES, 8 kip lifts by the 28 ton National 11105 crane are permitted without restrictions.

For 30 kip lifts by the 80 ton Grove RT880E, the crane outriggers should be located 12 ft away from 1983 pile caps (i.e. Bents with “A” designations) in the longitudinal direction and along the centerline of the pier in the transverse direction. Prior to the analysis, the facility stated that their preferred locations for the 80 ton crane are distances of 185 ft east of the bulkhead line and 145 ft west of the pier’s eastern edge. Based on the analysis, the centerline of the 80 ton crane should be located at Bents 13A or 23A. Bent 13A corresponds to 188.75 ft east of the bulkhead line and Bent 23A corresponds to 145 ft west of the pier’s eastern edge. Figure 2 indicates the crane’s locations at Bents 13A and 23A and Figure 3 specifies the outrigger locations relative to the crane locations.

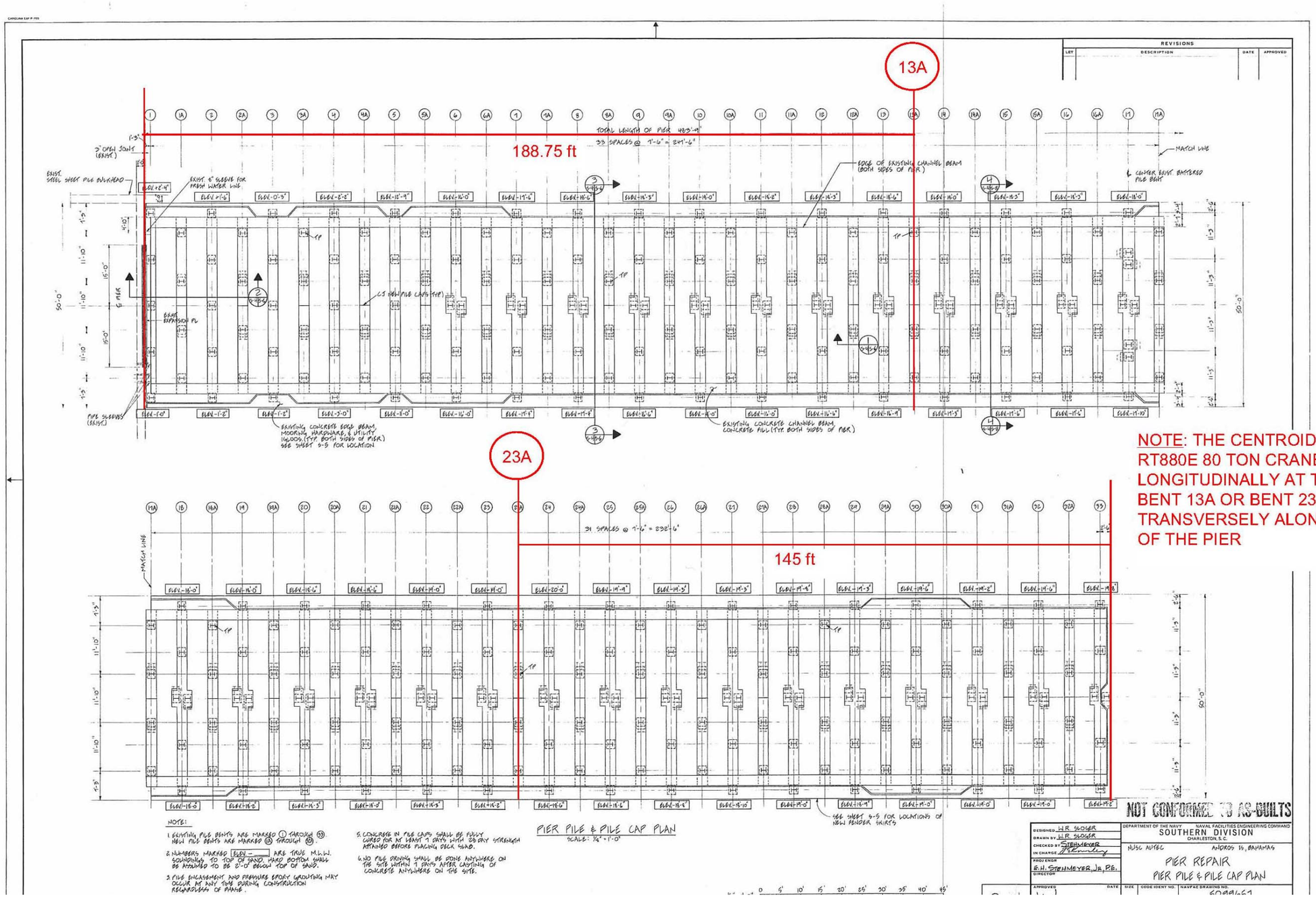
The 130 ton Grove RT9130E is restricted from use on the pier. This restriction is a result of calculated shear forces that exceed the shear capacity of the pile caps. These large calculated shear forces occur even when this crane is driven onto the pier with no lifting operations occurring.



LEGEND
NO LOAD ZONE
TEMPORARY VEHICLE LOADS PERMITTED, CRANE LIFTING OPERATIONS RESTRICTED
PIER PILE & PILE CAP PLAN
SCALE: 1/8" = 1'-0"
NOT CONFORMED TO AS-BUILTS
SOUTHERN DIVISION
PIER REPAIR
PIER PILE & PILE CAP PLAN
S-4

FIGURE 1

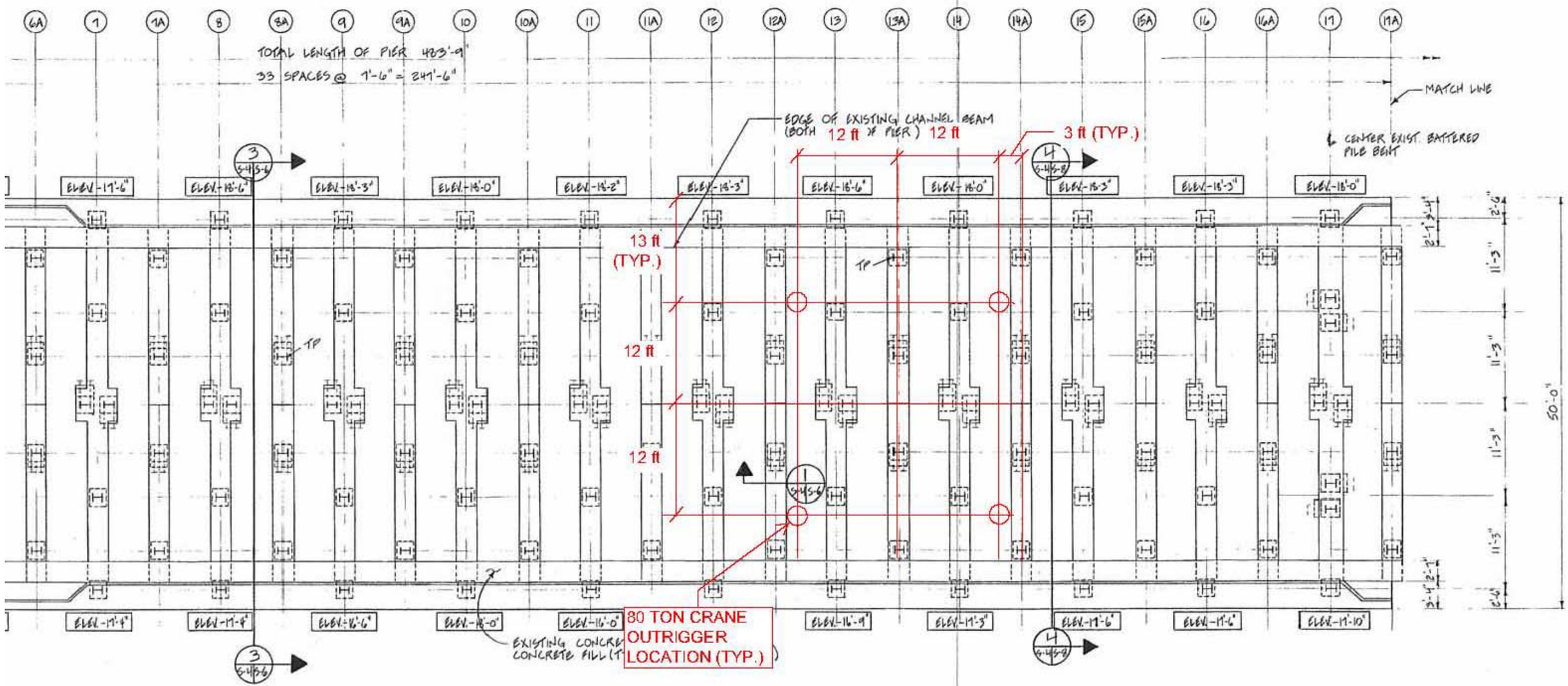
80 TON CRANE LOCATION PLAN



NOTE: THE CENTROID OF THE GROVE RT880E 80 TON CRANE TO BE LOCATED LONGITUDINALLY AT THE CENTERLINE OF BENT 13A OR BENT 23A AND TRANSVERSELY ALONG THE CENTERLINE OF THE PIER

FIGURE 2

80 TON CRANE OUTRIGGER LOCATIONS



NOTE: THE OUTRIGGER POSITIONS SHOWN ARE APPLICABLE FOR BOTH 80 TON CRANE LOCATIONS (I.E. BENTS 13A AND 23A)

FIGURE 3

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SECTION 2 - GENERAL

2.1 Purpose

The purpose of this desktop engineering analysis is to determine the operational load rating capacity of the Main Pier at AUTEC based on the existing level of deterioration. The analysis is based on the original construction (1965) and pier modification (1983) design drawings and the inspection findings in the January 2010 Waterfront Facilities Inspection (WFI) report performed by Halcrow, Inc. The lateral load capacity due to vessel mooring and berthing loads and environmental loads are not included in this analysis.

2.2 Facility Description

The Main Pier at AUTEC in Andros Island, Bahamas, is a steel H-pile supported reinforced concrete structure that is 484 ft long by 50 ft wide. There is a total of 327 concrete encased steel H-piles arranged in 66 bents. At original construction, circa 1965, the pier was constructed with 33 piles bents spaced 15 ft on center with five piles typically spaced 11 ft 3 in. on center. In 1983, the pier was modified by adding intermediate bents between the original bents resulting in 66 pile bents spaced 7.5 ft on center. The additional bents consist of four piles typically spaced 11 ft 10 in. on center. Figure 2-1 depicts a plan and section views of the 1965 original configuration and Figure 2-2 depicts a plan view of the pier with the 1983 modification. The 1983 intermediate bents are designated with an “A” after the bent number.

2.3 Facility Operation

The Main Pier supports mobile crane operations for the transfer of ammunition and other military equipment from small craft vessels berthed along the pier to flat-bed trucks. Facility operations require the use of a National 28-ton truck crane and a flat-bed truck for daily use and larger cranes (potentially a Grove 80-ton or 130-ton mobile crane) for intermittent usage. Fire truck access is also required in case of emergencies.

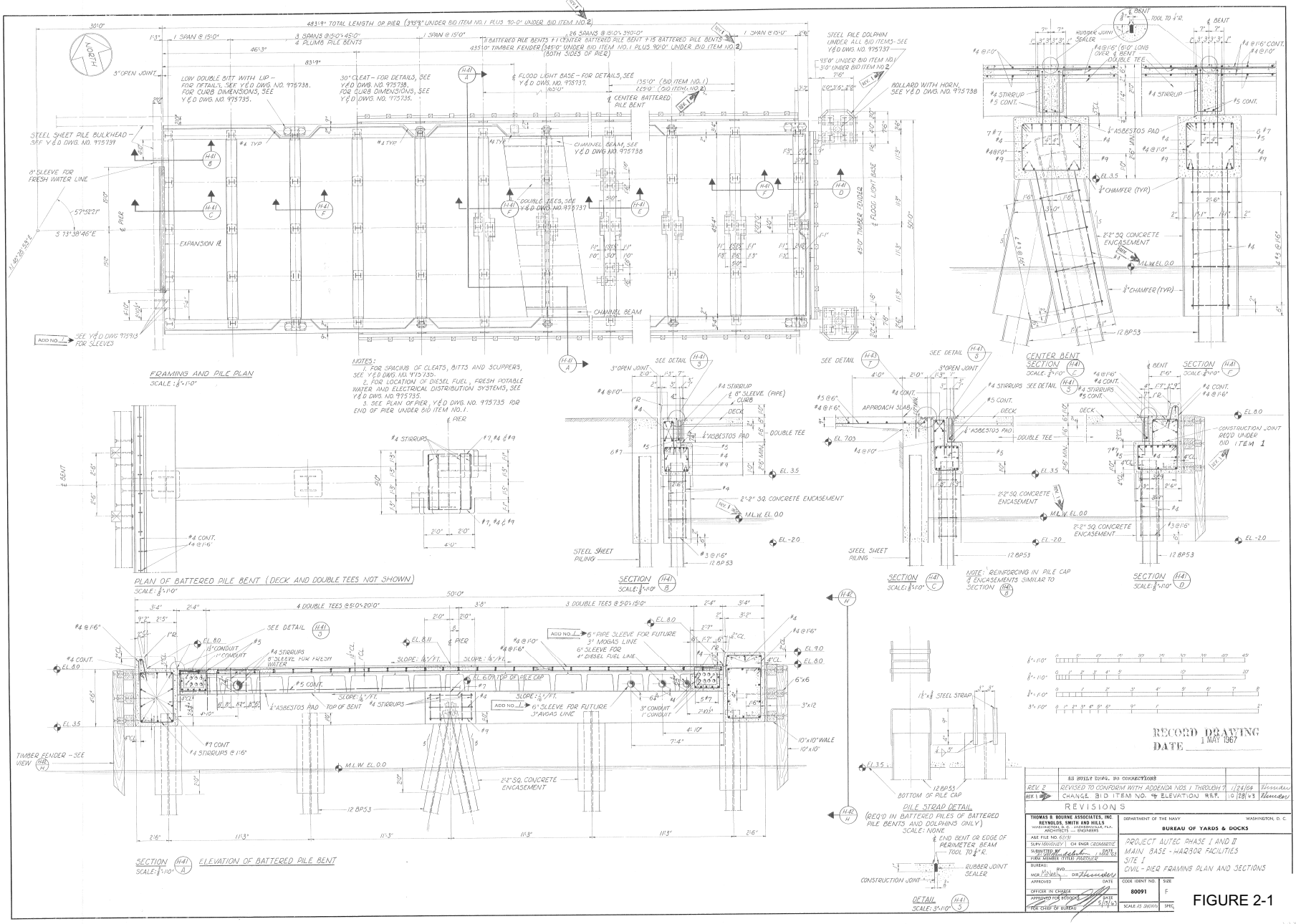
FRAMING AND PILE PLAN
SCALE: 1/8"=1'-0"
PLAN OF BATTERED PILE BENT (DECK AND DOUBLE TEES NOT SHOWN)
SCALE: 1/4"=1'-0"
SECTION (H-41 A) ELEVATION OF BATTERED PILE BENT
SCALE: 1/8"=1'-0"
SECTION (H-41 B)
SCALE: 1/8"=1'-0"
SECTION (H-41 C)
SCALE: 1/8"=1'-0"
CENTER BENT SECTION (H-41 E)
SCALE: 1/8"=1'-0"
SECTION (H-41 F)
SCALE: 1/8"=1'-0"
SECTION (H-41 D)
SCALE: 1/8"=1'-0"
PILE STRAP DETAIL
(REQ'D IN BATTERED PILES OF BATTERED PILE BENTS AND DOLPHINS ONLY)
SCALE: NONE
DETAIL (H-41 S)
SCALE: 3"=1'-0"
NOTES:
1. FOR SPACING OF CLEATS, BITTS AND SCUPPERS, SEE Y&D DWG. NO. 975735.
2. FOR LOCATION OF DIESEL FUEL, FRESH POTABLE WATER AND ELECTRICAL DISTRIBUTION SYSTEMS, SEE Y&D DWG. NO. 975735.
3. SEE PLAN OF PIER, Y&D DWG. NO. 975735 FOR END OF PIER UNDER BID ITEM NO. 1.
RECORD DRAWING
DATE 1 MAY 1967
REVISIONS
REV 2 REVISED TO CONFORM WITH ADDENDA NOS. 1 THROUGH 7 10/24/64
REV. 1 CHANGE BID ITEM NO. & ELEVATION REF. 10/28/63
THOMAS R. BOURNE ASSOCIATES, INC.
REYNOLDS, SMITH AND HILLS
DEPARTMENT OF THE NAVY
WASHINGTON, D. C.
BUREAU OF YARDS & DOCKS
PROJECT AUTEC PHASE I AND II
MAIN BASE - HAZSOR FACILITIES
SITE I
CIVIL - PIER FRAMING PLAN AND SECTIONS
CODE IDENT NO. 80091
SIZE F
SCALE AS SHOWN
SPEC.

FIGURE 2-1

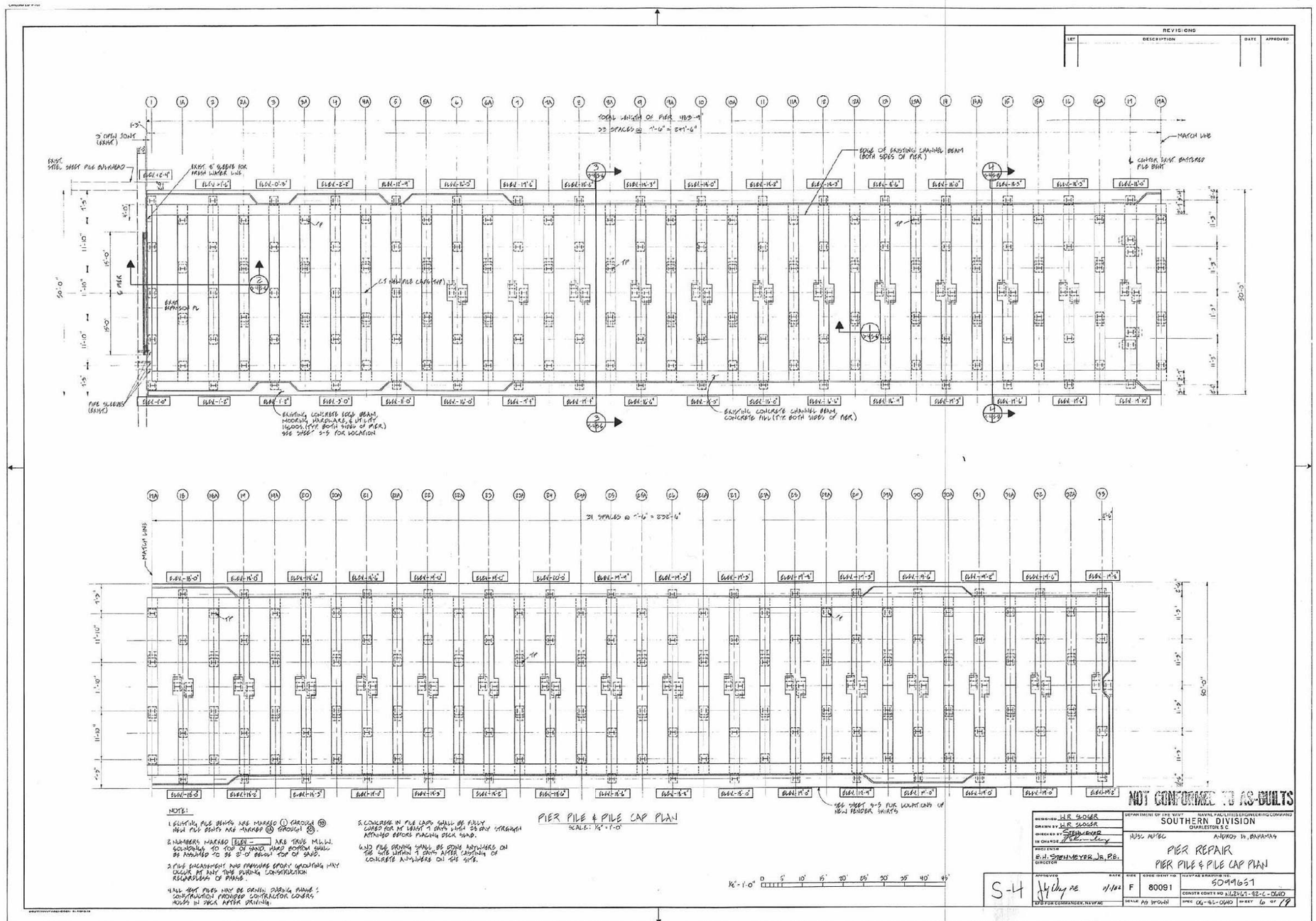
REVISIONS
PIER PILE & PILE CAP PLAN
NOT CONFORMED TO AS-BUILTS
SOUTHERN DIVISION
PIER REPAIR
PIER PILE & PILE CAP PLAN
50491651

FIGURE 2-2

SECTION 3 – DESIGN CRITERIA

3.1 Structure Geometry

The pier is 484 ft long by 50 ft wide with 66 bents at 7.5 ft on center. The original 1965 construction was designed with 33 bents at 15 ft on center. The modification in 1983 added 33 additional bents placed between the original bents. The plan view depicting the overall configuration of the existing pier is provided in Figure 2-2 of the previous section. The overall geometry of the pier is summarized in Table 3-1

Table 3-1: Pier Geometry

Item	Dimensions
Pier Footprint	484 ft long by 50 ft wide
No. of Bents	66
Bent Spacing	7.5 ft
1965 Bent Pile Spacing	11 ft 3 in. (typical)
1983 Bent Pile Spacing	11 ft 10 in. (typical)

The scope of the 1983 pier modification included removing the original prestressed double-tee deck planks, installing new intermediate pile caps supported on four piles and installing a new deck constructed of reinforced concrete deck planks and deck overlay. The original 1965 piles and the newer 1983 piles are essentially the same section (i.e. 12BP53 versus 12HP53, respectively) and have the same allowable capacity of 60 tons each. The 1965 and 1983 pile caps have the same concrete cross section and top reinforcing bar arrangement but the bottom reinforcing bar arrangement varies slightly. The bottom reinforcing in the 1965 and 1983 pile caps consists of four (4) #9 bars and (4) #10 bars, respectively. Based on the time period of construction (1965) and the facility being located in the Bahamas, a concrete strength of 3,000 psi and steel grade of 40 ksi has been assume for the pile caps constructed in 1965. The concrete elements constructed in 1983 are 4,000 psi and 6,000 psi for the cast-in-place and precast concrete, respectively, with a steel grade of 60 ksi. Figure 2-1 includes sections for the 1965 piles and pile caps and Figures 3-1 and 3-3 provide a plan view of the 1983 deck planks, and sections for the 1983 pile caps and deck planks. The element geometry is summarized in Table 3-2.

Table 3-2: Element Geometry

Item	Piles (1965)	Piles (1983)	Pile Caps (1965)	Pile Caps (1983)	Deck Planks	Deck Overlay
Material	Steel - 12BP53	Steel 12HP53	CIP Concrete	CIP Concrete	Precast Concrete	CIP Concrete
Strength	60 tons (Allowable)	60 tons (Allowable)	3,000 psi	4,000 psi	6,000 psi	4,000 psi
Concrete Section	26 in. wide by 26 in. deep	26 in. wide by 26 in. deep	30 in. high by 30 in. wide	30 in. high by 30 in. wide	18 in. high by 48 in. wide	6 in. high
Reinforcing Steel Grade	n/a	n/a	Grade 40	Grade 60	Grade 60	Grade 60
Concrete Cover	4 in.	4 in.	4 in. (bot.) 3 in. (top)	4 in. (bot.) 3 in. (top)	3 in.	n/a
Top Reinforcing	n/a	n/a	(6) #7	(6) #7	n/a	2 layers - #4 @ 1'-6" O.C.
Bottom Reinforcing	n/a	n/a	(4) #9	(4) #10	(10) #7	n/a
Shear Reinforcing	n/a	n/a	#4 @ 1'-0" o.c.	#4 @ 1'-2" o.c.	#3 @ 10" o.c.	n/a

3.2 Design Load Rating

The original 1965 pier construction was design to withstand a vehicle live load of AASHO H-15 (plus 15% impact) and a uniform load of 400 psf. The 1983 pier modification increased the uniform live load capacity to 600 psf but no increase in vehicle live load is specified on the design drawings. Table 3-3 summarizes the design live loads. The lateral load design criteria are provided for information only.

Table 3-3: Design Live Loads

Item	1965 Original Construction	1983 Modification
Uniform Load	400 psf	600 psf
Vehicular Load	H-15 (plus 15% impact)	(not documented)
Lateral	1,200 lbs per lf above EL. 0.0	Wind Loads: Normal Mooring 70 knots Hurricane Mooring 120 knots

3.3 Vehicular Loads

Day-to-day operations at AUTEC require the use of a National 11105 28-ton truck crane, typically used to lift ammunition weighing 8 kips. The facility would also like the flexibility to lift winches and other equipment up to 30 kips with a Grove RT880E mobile crane, and small vessels up to 44 kips with a Grove RT9130E 130-ton mobile crane. In case of emergencies, the pier must also be accessible by AUTEC's two fire trucks, Fire Rescue E-1 and Fire Rescue E-2. Table 3-4 provides the specifications for the vehicle loads.

Table 3-4: Vehicular Loads

Vehicle	Rated Capacity	Vehicle Weight	Maximum Lift	Typical Lift	Axles and Wheel Spacing		Outrigger Spacing	
					Length	Width	Length	Width
National 11105	28.4 ton	58 kips	56 kips	8 kips	22.25 ft	7 ft (front) 6 ft (rear)*	20 ft**	23 ft / 13 ft***
Grove RT880E	80 ton	108.2 kips	160 kips	30 kips	13.83 ft	8.5 ft	24 ft	24 ft
Grove RT9130E	130 ton	174 kips	260 kips	44 kips	16.21 ft	9.5 ft	27.83 ft	27.83 ft
Flat-bed Tractor Trailer	n/a	13.3 kips (tractor) 13.2 kips (trailer)	44 kips	(4) 4.4 kips (or 17.6 kips)	16 ft / 48 ft****	7 ft (front) 6 ft (middle & rear)*	n/a	n/a
Fire Rescue E-1	n/a	48.45 kips	n/a	n/a	20.25 ft	7 ft (front) 6 ft (rear)*	n/a	n/a
Fire Rescue E-2	n/a	65 kips	n/a	n/a	19 ft*****	7 ft (front) 6 ft (rear)*	n/a	n/a

* Rear axle consists of two wheels on either side (four total)
** Distance between main A-frame outriggers and RSOD (Rear Stabilizer)
*** 23 ft between A-frame footings, 13 ft between RSOD footings
**** 16 ft between front tractor axle and tractor rear double axles (four tires per axle), 48 ft between front tractor axle and trailer rear double axles (four tires per axle)
***** 19 ft between front axle and two rear axles (rear axles are 4.5 ft o.c.)

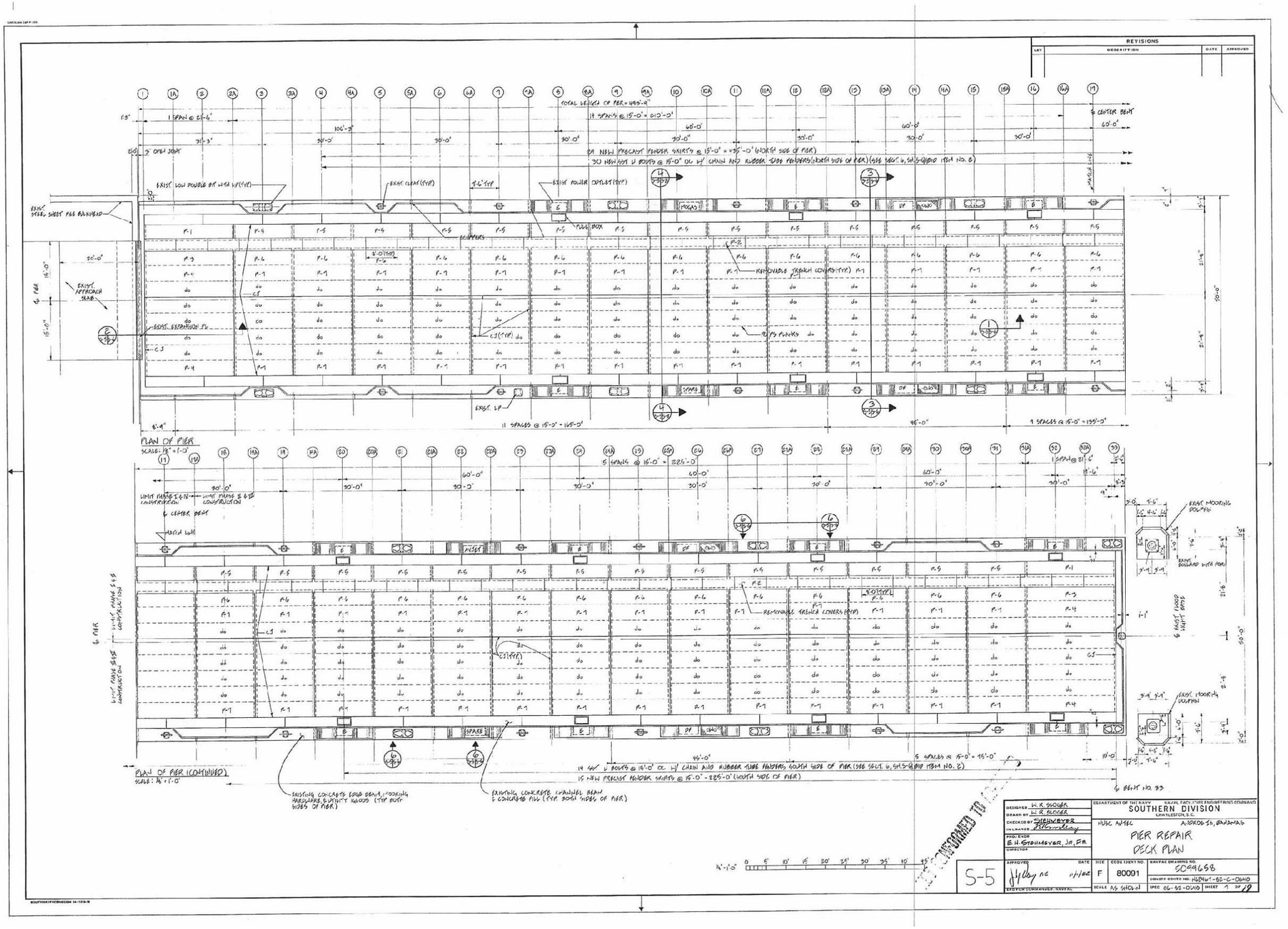
REVISIONS
PLAN OF PIER
PLAN OF PIER (CONTINUED)
SOUTHERN DIVISION
PIER REPAIR
DECK PLAN
S-5

FIGURE 3-1

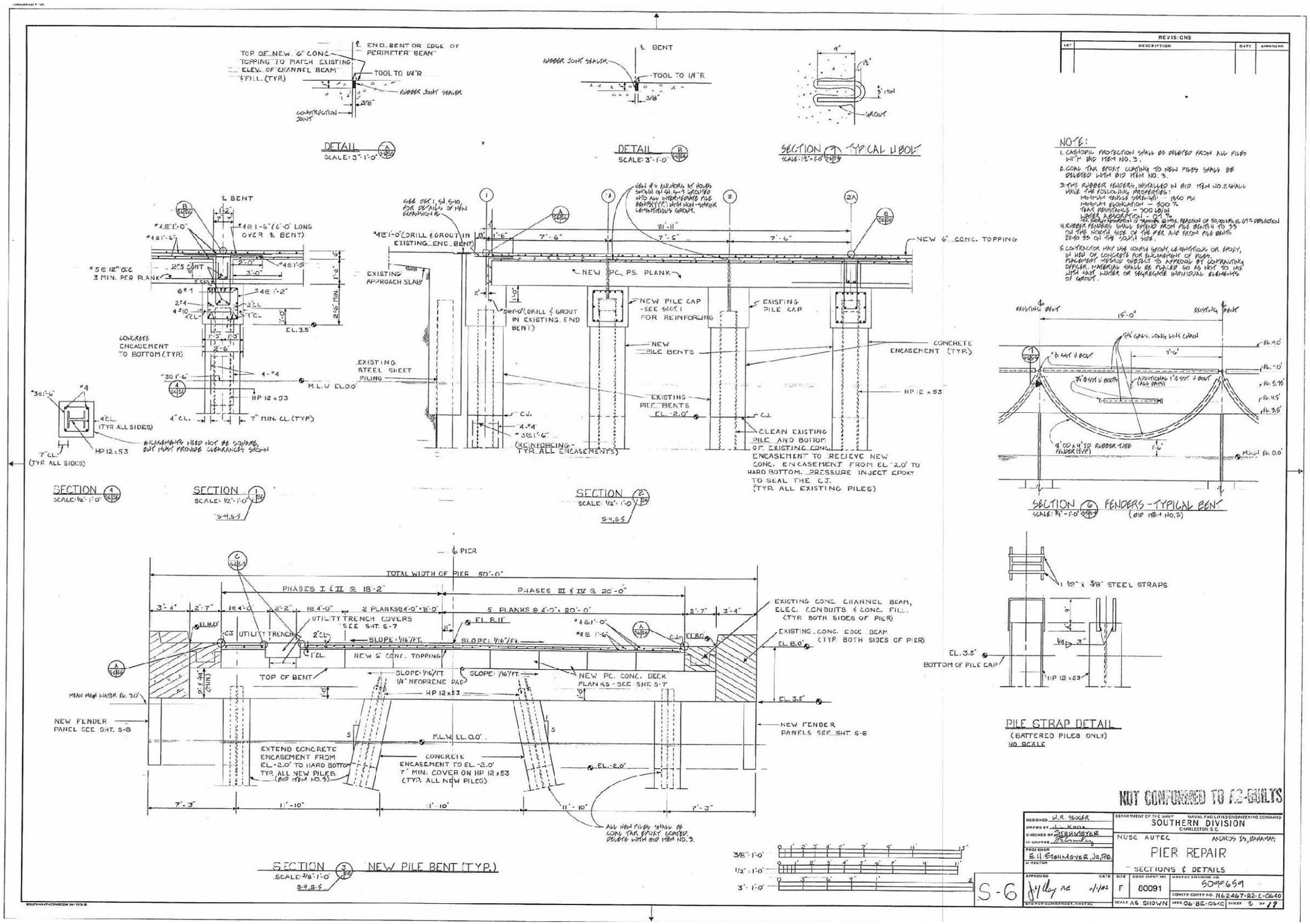
DETAIL
SCALE: 3"-1'-0"
DETAIL
SCALE: 3"-1'-0"
SECTION 7 TYPICAL U BOLT
NOTE:
SECTION 4
SECTION 1
SECTION 2
SECTION 6 FENDERS - TYPICAL BENT
SECTION 3 NEW PILE BENT (TYP.)
PILE STRAP DETAIL
(BATTERED PILES ONLY)
NO SCALE
NOT CONFORMED TO AS-BUILTS
SOUTHERN DIVISION
NUSC AUTEC
PIER REPAIR
SECTIONS & DETAILS
S-6

FIGURE 3-2

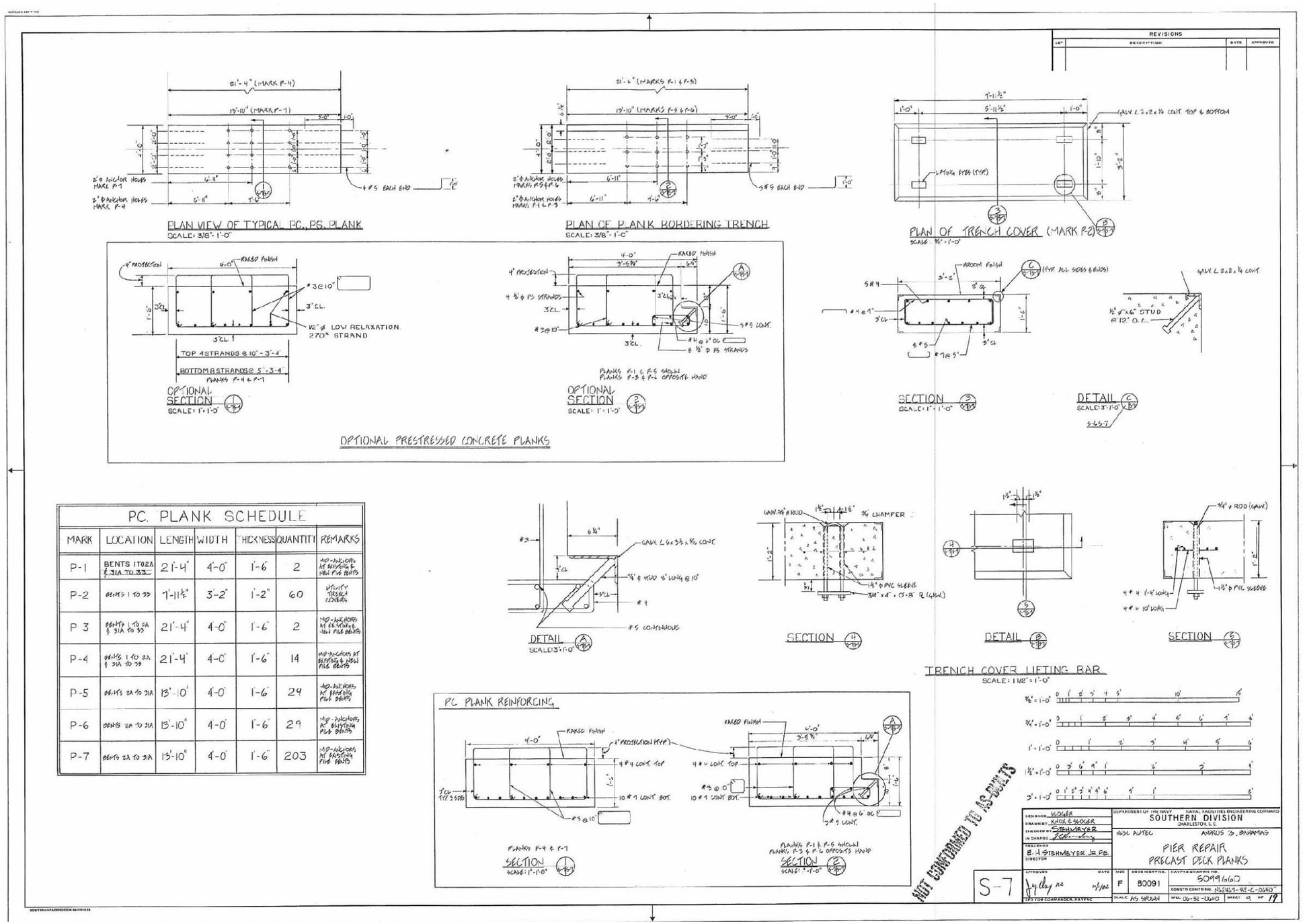
REVISIONS
PLAN VIEW OF TYPICAL P.C. PS. PLANK
SCALE: 3/8" = 1'-0"
PLAN OF PLANK BORDERING TRENCH
SCALE: 3/8" = 1'-0"
PLAN OF TRENCH COVER (MARK P-2)
SCALE: 3/4" = 1'-0"
OPTIONAL SECTION
SCALE: 1" = 1'-0"
OPTIONAL PRESTRESSED CONCRETE PLANKS
SECTION
SCALE: 1" = 1'-0"
DETAIL
SCALE: 3" = 1'-0"
PC. PLANK SCHEDULE
MARK | LOCATION | LENGTH | WIDTH | THICKNESS | QUANTITY | REMARKS
P-1 | BENTS 1T02A & 31A TO 33 | 21'-4" | 4'-0" | 1'-6" | 2 | MID-ANCHORS AT BENTS 1 & 33 & HAV PILE BENTS
P-2 | BENTS 1 TO 33 | 7'-11½" | 3'-2" | 1'-2" | 60 | VICINITY TRENCH COVERS
P-3 | BENTS 1 TO 2A & 31A TO 33 | 21'-4" | 4'-0" | 1'-6" | 2 | MID-ANCHORS AT EX. BULKHEAD & HAV PILE BENTS
P-4 | BENTS 1 TO 2A & 31A TO 33 | 21'-4" | 4'-0" | 1'-6" | 14 | MID-ANCHORS AT BENTS 1 & HAV PILE BENTS
P-5 | BENTS 2A TO 31A | 13'-10" | 4'-0" | 1'-6" | 29 | MID-ANCHORS AT EXISTING PILE BENTS
P-6 | BENTS 2A TO 31A | 13'-10" | 4'-0" | 1'-6" | 29 | MID-ANCHORS AT EXISTING PILE BENTS
P-7 | BENTS 2A TO 31A | 13'-10" | 4'-0" | 1'-6" | 203 | MID-ANCHORS AT EXISTING PILE BENTS
PC PLANK REINFORCING
SECTION
SCALE: 1" = 1'-0"
TRENCH COVER LIFTING BAR
SCALE: 1 1/2" = 1'-0"
NOT CONFORMED TO AS-BUILTS
DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
SOUTHERN DIVISION
CHARLESTON, S.C.
NDSC AUTEC
ANDROS IS., BAHAMAS
PIER REPAIR
PRECAST DECK PLANKS
S-7

FIGURE 3-3

SECTION 4 – ANALYSIS

4.2 Structural Capacities

The analysis of the structural capacities includes the steel H-piles, concrete pile caps and the composite deck consisting of precast reinforced concrete deck planks and a CIP overlay. The deteriorated capacities included for the concrete pile caps and deck analyses are based on observations from the 2010 Waterfront Facilities Inspection by Halcrow Inc.

4.2.1 Piles

The allowable capacity of the steel H-piles is 60 tons. This value is provided on both the original (constructed 1965) and modification (constructed 1983) design drawings. At original construction, the tops of the steel H-piles were encased in concrete to the -2.0 ft elevation. In 1983, the newly installed piles and existing piles were fully encased to the mudline except the first four original bents, Bents 1, 2, 3 and 4. During an inspection performed in 1998, it was noted that Bent 1A Pile B1, Bent 2A Pile A1, Bent 4 Piles A, and D had moderate to severe corrosion on the short sections of pile that were not encased. Since then, all piles except Pile E in Bent 4 have been encased. Pile E in Bent has 6 ft to 8 ft of exposed steel H-pile and also has severe corrosion include through holes.

The 60 ton (120 kips) allowable capacity provided on the drawings is a geotechnical capacity. The geotechnical pile capacities typically are calculated with a minimum safety factor 2, which would correspond to an ultimate capacity of 120 tons. Although, there are severely corroded sections of the steel H-piles within Bents 1 through 4, no structural overloading was observed (i.e. buckling). Based on the short exposed height of the piles in this area of the pier the 60 ton geotechnical capacity is not affected. For example an HP12x53 pile with a 20 ft effective length has working capacity of 276 kips, or 2.3 times greater than the 120 kips geotechnical capacity. Nonetheless, to prevent potential local buckling of the piles, it is advised that crane operation (i.e. lifting) should be restricted between Bents 1 and 4 (i.e. the inshore 100 ft of the pier).

Table 4-1: Pile Capacity

Item	Location	Deterioration	Allowable Capacity	Operational Restrictions
P1	Bents 1 through 4	Isolated severe corrosion	120 kips	No crane operations
P2	Bents 4A to 33	None	120 kips	None

4.2.2 Pile Caps

The full and deteriorated capacities of the pile caps are provided in Table 4-2. The two cases that conservatively estimate the reduced structural capacities of the pile cap based on the observed deterioration are sections with 25 percent section loss of the bottom steel reinforcing and the loss of the 4 in. wide concrete cover on one vertical face of the cap cross section. Additional cases of reduced capacity including sections with 50 percent section loss of the bottom steel reinforcing and the loss of the 4 in. wide concrete cover on both vertical faces are also provided for information.

Table 4-2: Pile Cap Capacities

Capacity / Applied Load		Positive Moment (k-ft)		Negative Moment (k-ft)		Shear (kips)	
		1965	1983	1965	1983	1965	1983
Capacity	**Full Capacity**	**286.68**	**534.26**	**271.31**	**405.06**	**86.4**	**102.74**
	No Cover on One Vertical Face	**284.75**	**529.00**	**269.75**	**402.42**	**78.2**	**93.3**
	No Cover on Both Vertical Faces	282.12	521.84	267.61	398.82	70.0	83.87
	25% Section Loss	**217.37**	**407.1**	**205.39**	**307.01**	**80.16**	**94.74**
	50% Section Loss	145.48	275.67	138.2	206.82	73.93	86.75

*Values in bold are representative of the level of deterioration reported in the January 2010 WFI report.

4.2.3 Deck

Based on the inspection findings in the 2010 WFI report, the deck plank soffits have cracks and delaminations with rust staining, isolated areas of spalling with exposed steel reinforcing. In addition there are two broken deck planks located between Bents1 and 1A and Bents 31A and 32. All the deck spalls with severe corrosion are located along the northern and southern edges of the pier on the utility trenches that were not rehabilitated in 1983. These severely deteriorated utility trenches and the areas with broken deck planks are to be designated as NO LOAD ZONES (see Figure 1 of the Executive Summary).

Outside the NO LOAD ZONE areas, the remaining deficiencies include cracks and delaminations with rust staining. Based on these deficiencies a reduced structural capacity correlating to 25 percent section loss of the tension steel reinforcing has been conservatively assumed. The full and deteriorated capacities of the deck are provided in Table 4-3. For information, the reduced deck capacity with 50 and 75 percent section loss of the tension steel reinforcing is also provided.

Table 4-3: Deck Capacities

Capacity / Applied Load		Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)
Capacity	**Full Capacity**	**522.51**	**95.78**	**132.57**
	25% Section Loss	**393.61**	**83.16**	**127.58**
	50% Section Loss	266.91	70.51	122.58
	75% Section Loss	134.86	57.83	117.58
*Values in bold are representative of the level of deterioration reported in the January 2010 WFI report at locations outside the areas of the NO LOAD ZONES.				

4.3 Loads

Since the early 1960s, concrete structures have been designed based on strength or variations of the load and resistance factor design (LRFD) method. The load factors applied to the dead and live loads are considered are factors of 1.2 and 1.6, respectively, as referenced Equation 9-2 of ACI 318-05. No other load types are applicable to the analysis. No lateral loadings were taken into account during this analysis.

4.3.1 Dead Loads

The dead loads based on the design properties are provided in Table 4-4. The tributary area for the 1965 and 1983 bents are 84.4 sf (7.5 ft by 11.25 ft) and 88.8 sf (7.5 ft by 11.83 ft). The concrete is assumed to weigh 150 pcf and piles are assumed to be 35 ft long HP 12x53 with a 25 ft long 26 in. by 26 in. concrete encasement. For the pile capacity check, the larger tributary area value of 88.8 sf was used for conservatism. All dead loads are factored by 1.2.

Table 4-4: Dead Loads

Analyzed Element	Element	Dimensions	Material Density	Unfactored Dead Load	Factored* Dead Load
Deck	Deck	4 ft wide by 2 ft high	150 pcf	**1.2 klf**	**1.44 klf**
Pile Cap	Deck	7.5 ft wide by 2 ft high	150 pcf	2.25 klf	2.7 klf
	Pile Cap	2.5 ft by 2.5 ft	150 pcf	0.94 klf	1.13 klf
	Total			**3.19 klf**	**3.83 klf**
Pile (35 ft long)	Deck	7.5 ft by 11.83 ft by 2 ft	150 pcf	26.7 kips	32.04 kips
	Pile Cap	2.5 ft by 2.5 ft by 11.83 ft	150 pcf	11.1 kips	13.32 kips
	Pile – Concrete Encasement	25 ft x 4.59 sf	150 pcf	12.1 kips**	14.48 kips
	Pile – Steel	35 ft	53 plf	1.9 kips	2.28 kips
	Total			**56.9 kips**	**68.3 kips**

* Load factor of 1.2 is applied to dead loads

** Reduced value accounting for buoyancy force (21.5 ft of concrete encasement below MLW)

4.3.2 Live Loads

The live loads based on tributary areas discussed in Section 4.2.1 are provided in Table 4-5. All live loads are factored by 1.6. Table 4-5 provides the loads used in the 600 psf uniform load analysis.

Table 4-5: Uniform Live Loads

Load	Analyzed Element	Dimensions	Unfactored Live Load	Factored Live Load
600 psf	Deck	4 ft wide	2.4 klf	3.84 klf
	Pile Cap	7.5 ft wide	4.5 klf	7.2 klf
	Pile	11.83 ft long by 7.5 ft wide	53.25 kips	85.2 kips

* Load factor of 1.6 is applied to dead loads

Table 4-6 provides the loads during lifting operations for the 28 ton, 80 ton and 130 ton crane load analyses, respectively. Loads during crane lift operations are factored by the 1.6 live load factor. Table 4-7 provides the vehicular and crane loads when they are moving. The moving loads are factored by the 1.6 live load factor and a 15% impact factor.

Table 4-6: Crane Lifting Operation Live Loads

Crane	Lift	Factored Outrigger Loads (kips)			
		Fwd Left	Fwd Right	Aft Left	Aft Right
28 ton	8 kips	52.8	31.7	21.2	0
80 ton	30 kips	0	47	48.5	118.9
130 ton	44 kips	13.2	80.5	83	163.8

Table 4-7: Moving Loads

Crane	Front Axle (kips)	Intermediate Axle (kips)	Rear Axle (kips)
28 ton	36.8	n/a	70
80 ton	99.3	n/a	101.9
130 ton	146.3	n/a	174.1
Fire Rescue No. 1	35.7	n/a	53.5
Fire Rescue No. 2	35.9	n/a	83.4
Tractor Trailer with 17.6 k load	12.3	40.6	28.4
Tractor Trailer with 44 kip load	12.3	64.9	52.7

* Axle loads were split to tire loads in the analyses.

4.4 600 PSF Analysis Results

Outside the NO LOAD ZONES, there is no reduction in the pier's design uniform load rating of 600 psf. Under this applied load, the deck reaches only 8 percent of its reduced structural capacity (in shear); the pile caps reach 72 percent of their reduced capacity (in negative bending moment capacity and the piles reach 63 percent of their allowable capacity. A summary of the 600 psf uniform live load analysis results for the deck planks, pile caps and piles are provided in Tables 4-8 to 4-10, respectively.

It should be noted that the maximum calculated negative moment on the 1983 pile cap is a result of the large dead load at the ends of the pile caps. The pile caps cantilever 7.25 ft beyond the northernmost and southernmost piles and support a 4.5 ft deep by 3.33 ft wide edge beam.

The 600 psf analysis for the deck and pile caps was performed using a 2-D STAAD model. The pile forces were taken as the support reactions of the pile cap analysis.

Table 4-8: 600 PSF – Deck Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)
Capacity	Full Capacity	522.51	95.78	132.57
	25% Section Loss	393.61	83.16	127.58
Applied	**Dead Load**	**10**	**2**	**3***
	600 psf	**35**	**5**	**10***

1. The 600 psf load case includes dead load
2. (*) Shear value taken "d" (2.5 ft) away from the support

Table 4-9: 600 PSF - Pile Cap Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)		Negative Moment (k-ft)		Shear (kips)	
		1965	**1983**	**1965**	**1983**	**1965**	**1983**
Capacity	Full Capacity	286.68	534.26	271.31	405.06	86.4	102.74
	No Cover on One Vertical Face	284.75	529.00	269.75	402.42	78.2	93.3
	25% Section Loss	217.37	407.1	205.39	307.01	80.16	94.74
Applied	**Dead Load**	**25**	**55**	**44**	**220**	**14***	**31***
	600 psf	**95**	**81**	**139**	**220**	**45***	**47***

1. The 600 psf load case includes dead load
2. (*) Shear value taken "d" (2.5 ft) away from the support

Table 4-10: 600 PSF – Pile Analysis Results

Capacity / Applied Load		Force (kips)				
		Pile 1	Pile 2	Pile 3	Pile 4	Pile 5
Capacity		199.4*				
Applied	1965 Pile Cap Dead Load	51	45	42	45	51
	600 psf 1965 Pile Cap	83	137	119	137	83
	Dead Load 1983 Pile Cap	89	29	28	90	n/a
	600 psf on 1983 Pile Cap	124	122	121	124	n/a
	1. All applied load cases include dead load 2. (*) The corrected value for pile capacity based on an allowable capacity of 120 kips (i.e. 120 kips x 2 x 0.9) and factored dead load of pile subtracted (16.4 kips)					

4.5 Moving Load Analysis Results (vehicles driving on the deck)

Outside the NO LOAD ZONES, the pier in its current condition can support the moving loads of the 28 ton crane, flat-bed tractor trailer (for loads up to 44 kips), and Fire Rescue No.1 without restrictions. The moving loads from the 80 ton crane and Fire Rescue No. 2 result in shear forces that exceed the reduced shear capacities of the pile caps by less than 10 percent. Halcrow recommends restricting the speed of the 80 ton crane to 5 mph in order to minimize the likelihood of impact loads on the pier. As for Fire Rescue No. 2, because the need for a fire truck on the pier would typically be in response to an urgent matter, it is recommended that Fire Rescue No. 2 should generally not be permitted unless Fire Rescue No. 1 is not operational. The moving loads from the 130 ton crane results in shear forces that exceed the reduced shear capacity of the pile caps by approximately 50 percent, therefore, the 130 crane should be restricted from access to the pier.

4.5.1 Moving Load Analysis - Deck

The reduced capacity of the concrete deck can adequately support all moving loads including the Fire Rescue No. 2 and the 130 ton crane. A summary of the moving load analysis results for the deck are provided in Table 4-11.

The deck was modeled as a 2-D member in STAAD with moving load cases for one and two wheels of the heaviest axle.

Table 4-11: Moving Loads - Deck Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)
Capacity	Full Capacity	522.51	95.78	132.57
	25% Section Loss	393.61	83.16	127.58
Applied	**Dead Load**	**10**	**2**	**6**
	Fire Rescue No.1 & Tractor Trailer with 17.6 kip load	**59**	**15**	**36**
	Tractor Trailer with 44 kip load	**69**	**18**	**43**
	28 ton crane	**74**	**19**	**46**
	Fire Rescue No.2	**86**	**18**	**54**
	80 ton crane	**103**	**22**	**57**
	130 ton crane	**181**	**35**	**93**

1. All applied load cases include dead load.

4.5.2 Moving Load Analysis - Pile Caps

The reduced capacity of the concrete pile caps can adequately support the moving loads of the 28 ton crane, flat-bed tractor trailer (for loads up to 44 kips), and Fire Rescue No.1 without restrictions. The moving loads from the 80 ton crane and Fire Rescue No. 2 result in shear forces that exceed the reduced shear capacities of the pile caps by less than 10 percent. The moving loads from the 130 ton crane results in shear forces that exceed the reduced shear capacity of the pile caps by approximately 50 percent and the positive and negative moment capacities of the 1965 pile caps by less than 10 percent. The 130 crane should be restricted from accessing pier based on these conditions. A summary of the moving load analysis results for the pile caps are provided in Table 4-12.

The pile caps were modeled as 2-D members in STAAD with moving load cases for one and two wheels of the heaviest axle.

Table 4-12: Moving Loads – Pile Cap Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)		Negative Moment (k-ft)		Shear (kips)	
		1965	1983	1965	1983	1965	1983
Capacity	Full Capacity	286.68	534.26	271.31	405.06	86.4	102.74
	No Cover on One Vertical Face	284.75	529.00	269.75	402.42	78.2	93.3
	25% Section Loss	217.37	407.1	205.39	307.01	80.16	94.74
Applied	**Dead Load**	**25**	**55**	**44**	**220**	**26**	**49**
	Fire Rescue No. 1 & Tractor Trailer with 17.6 kip load	**92**	**110**	**88**	**234**	**65**	**76**
	Tractor Trailer with 44 kip load	**102**	**123**	**103**	**238**	**74**	**85**
	28 ton crane	**108**	**128**	**108**	**255**	**78**	**88**
	Fire Rescue No. 2	**124**	**142**	**120**	**243**	**86**	**98**
	80 ton crane	**141**	**157**	**145**	**285**	**81**	**100**
	130 ton crane	**223**	**230**	**217**	**270**	**119**	**136**

1. All applied load cases include dead load.
2. Numbers in blue indicate results that are greater than the reduced capacity of the element by less than 10%. Numbers in red exceed the reduced capacity of the member by more than 10%.

4.5.3 Moving Load Analysis - Piles

The allowable capacity of the concrete encased steel H-piles can adequately support all moving loads including the Fire Rescue No. 2 and the 130 ton crane. A summary of the moving load analysis results for the piles are provided in Tables 4-13 and 4-14.

The allowable pile capacity of 120 kips correlates to 216 kips when converted to load factor design. After subtracting away the dead load of the pile, 199.6 kips is the remaining capacity available for supporting the dead load of the pile caps and deck, and all live loads. This value was determined so that the STAAD analysis results could be easily compared.

The pile forces were taken as the support reactions for the 1965 and 1983 pile cap analyses.

Table 4-13: Moving Loads - Pile Analysis Results for 1965 Pile Cap Bents

Capacity / Applied Load	Force (kips)				
	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5
Capacity	199.6*				
Dead Load 1965 Pile Cap	51	45	42	45	51
Fire Rescue No. 1 & Tractor Trailer with 17.6 kip load	**69**	**93**	**89**	**93**	**70**
Tractor Trailer with 44 kip load	**73**	**103**	**99**	**103**	**74**
28 ton crane	**74**	**107**	**104**	**107**	**76**
Fire Rescue No. 2	**79**	**119**	**115**	**119**	**81**
80 ton crane	**81**	**125**	**119**	**125**	**81**
130 ton crane	**102**	**173**	**165**	**173**	**102**

1. All applied load cases include dead load.
2. (*) The corrected value for pile capacity based on an allowable capacity of 120 kips (i.e. 120 kips x 2 x 0.9) and factored dead load of pile subtracted (16.4 kips)

Table 4-14: Moving Loads - Pile Analysis Results for 1983 Pile Cap Bents

Capacity / Applied Load	Force (kips)				
	Pile 1	Pile 2	Pile 3	Pile 4	
Capacity	199.6*				
Dead Load 1983 Pile Cap	89	29	28	90	
Fire Rescue No. 1 & Tractor Trailer with 17.6 kip load	**130**	**77**	**76**	**128**	
Tractor Trailer with 44 kip load	**138**	**87**	**86**	**137**	
28 ton crane	**142**	**91**	**91**	**145**	
Fire Rescue No. 2	**152**	**104**	**103**	**150**	
80 ton crane	**154**	**110**	**110**	**162**	
130 ton crane	**196**	**161**	**160**	**198**	

1. All applied load cases include dead load.
2. (*) The corrected value for pile capacity based on an allowable capacity of 120 kips (i.e. 120 kips x 2 x 0.9) and factored dead load of pile subtracted (16.4 kips)

4.6 Lifting Load Analysis Results

Outside the NO LOAD ZONES, the pier in its current condition can support 8 kip lifts by the 28 ton crane without restrictions. Thirty (30) kip lifts by the 80 ton crane result in significant positive moment overloads on the 1965 pile caps and significant shear overloads on all pile caps when the crane is in certain positions on the pier. Based on this initial analysis and feedback from the site regarding preferred lifting locations, 30 kip lifts by the 80 ton crane are permissible only at Bents 13A and 23A as indicated on Figures 2 through 3 of the executive summary. Although based on the moving load analysis, the 130 ton crane is not permitted on the pier, 44 kip lifts by the 130 ton crane was also analyzed. These lifts also results in significant moment and shear overloads on the pile caps. But similar to the 30 kip lift 80 ton crane load case, 44 kip lifts with the 130 ton are feasible at Bents 13A and 23A.

4.6.1 Lifting Load Analysis - Deck Planks

The reduced capacity of the concrete deck can adequately support 8 kip lifts by the 28 ton crane without restrictions and 30 kip lift by the 80 ton crane with the outriggers fully extended to 24 ft and located at mid-pier and at Bents 13A or 23A. A summary of the lifting load analysis results for the deck are provided in Table 4-15.

Firstly, the deck was modeled as a 2-D member in STAAD with load cases for one outrigger and two outriggers applied as moving loads. The one outrigger load cases used the heaviest outrigger load and the two outrigger load cases used the two heaviest outrigger loads.

The deck analysis for the 80 ton crane at Bents 13A and Bents 23A was modeled as a 3-D structure in STAAD with all outrigger loads applied.

Table 4-15: Lifting Loads - Deck Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)	Negative Moment (k-ft)	Shear (kips)
Capacity	Full Capacity	522.51	95.78	132.57
	25% Section Loss	393.61	83.16	127.58
Applied	**Dead Load**	**10**	**2**	**6**
	28 ton, 8 kip Lift	**107**	**22**	**59**
	80 ton, 30 kip Lift at mid-pier and at Bents 13A and 23A	**185**	**80**	**67**
	80 ton, 30 kip Lift	**189**	**90**	**77**
	130 ton, 44 kip Lift at mid-pier and at Bents 13A and 23A	**213**	**80**	**115**
	130 ton, 44 kip Lift	**375**	**120**	**170**

1. All applied load cases include dead load.
2. Numbers in blue indicate results that are greater than the reduced capacity of the element by less than 10%. Numbers in red exceed the reduced capacity of the member by more than 10%.

4.6.2 Lifting Load Analysis - Pile Caps

The reduced capacity of the concrete pile caps can adequately support 8 kip lifts by the 28 ton crane without restrictions. Thirty (30) kip lifts by the 80 ton crane result in shear forces that exceed the reduced shear capacity of the 1965 pile caps by less than 10 percent. Due to the overload, Halcrow further refined the analysis to model the load distribution more accurately. Based on the additional justification below, the shear forces reduces from 87 kips to 32 kips which falls well below the reduced shear capacity 78.2 kips.

1. The load from the deck plank onto the pile cap in the 3-D STAAD model is applied as a point load. In reality the load will be applied to the pile cap will be a uniform load. A revised model was created to distribute the point load over 4 ft which is the width of one deck plank. This assumption is also conservative as this load may distribute over additional deck planks.

2. Furthermore, the ACI concrete code includes a provision which allows the shear load to be taken a distance equaling the depth of the member away from the face of the support as long as a concentrated load is not located within that same distance from the face of the support. The pile cap is 2.5 ft deep and as shown in Figure 3 of the Executive Summary the outrigger loads are spaced 4.5 ft away from the 1965 pile caps and distribute the load as uniform load as discussed above.

A summary of the lifting load analysis results for the pile caps are provided in Table 4-16. The 130 ton lifting analysis is provided for information only.

Firstly, the pile cap was modeled as a 2-D member in STAAD with load cases for one outrigger and two outriggers applied as moving loads. The one outrigger load cases used the heaviest outrigger load and the two outrigger load cases used the two heaviest outrigger loads.

The pile cap analysis for the 80 ton crane at Bents 13A and Bents 23A was modeled as a 3-D structure in STAAD with all outrigger loads applied then further refined to determine the more accurate shear force.

Table 4-16: Lifting Loads – Pile Cap Analysis Results

Capacity / Applied Load		Positive Moment (k-ft)		Negative Moment (k-ft)		Shear (kips)	
		1965	1983	1965	1983	1965	1983
Capacity	Full Capacity	286.68	534.26	271.31	405.06	86.4	102.74
	No Cover on One Vertical Face	284.75	529.00	269.75	402.42	78.2	93.3
	25% Section Loss	217.37	407.1	205.39	307.01	80.16	94.74
Applied	**Dead Load**	**25**	**55**	**44**	**220**	**26**	**49**
	28 ton, 8 kip Lift	**145**	**161**	**102**	**268**	**75**	**73**
	80 ton, 30 kip Lift at mid-pier and at Bents 13A and 23A	**130**	**144**	**37**	**104**	**87 (32*)**	**60**
	130 ton, 44 kip Lift at mid-pier and at Bents 13A and 23A	**125**	**280**	**27**	**104**	**83**	**89**
	80 ton, 30 kip Lift over pile caps	**296**	**294**	**176**	**329**	**141**	**168**
	130 ton, 44 kip Lift over pile caps	**398**	**384**	**226**	**371**	**185**	**213**

1. All analysis results include dead loads
2. Numbers in blue indicate results that are greater than the reduced capacity of the element by less than 10%. Numbers in red exceed the reduced capacity of the member by more than 10%.
3. (*) 87 kips is the value determined by using the 3-D STAAD model. 32 kips is the value determined by refining the analysis and is more representative of the actual shear value on the 1965 pile cap.

4.6.3 Lifting Load Analysis - Piles

The allowable capacity of the concrete encased steel H-piles can adequately support 8 kip lifts by the 28 ton crane without restrictions and 30 kip lift by the 80 ton crane with the outriggers fully extended to 24 ft and located at mid-pier and at Bents 13A or 23A. A summary of the lifting load analysis results for the piles are provided in Tables 4-17 and 4-18. The 130 ton lifting analysis is provided for information only.

The allowable pile capacity of 120 kips correlates to 216 kips when converted to load factor design. After subtracting away the dead load of the pile, 199.6 kips is the remaining capacity available for supporting the dead load of the pile caps and deck, and all live loads. This value was determined so that the STAAD analysis results could be easily compared.

The pile forces were taken as the support reactions for the pile cap analyses.

Table 4-17: Lifting Loads - Pile Analysis Results for 1965 Pile Cap Bents

Capacity / Applied Load	Force (kips)				
	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5
Capacity	199.6*				
Dead Load 1965 Pile Cap	51	45	42	45	51
80 ton, 30 kip Lift at mid-pier and at Bents 13A and 23A	**51**	**111**	**52**	**80**	**45**
130 ton, 44 kip Lift at mid-pier and at Bents 13A and 23A	**51**	**108**	**51**	**77**	**54**
28 ton, 8 kip Lift on 1965 Pile Cap	**87**	**98**	**96**	**98**	**86**
80 ton, 30 kip Lift	**121**	**165**	**162**	**164**	**120**
130 ton, 44 kip Lift	**147**	**210**	**206**	**210**	**146**
	1. All analysis results include dead loads 2. Numbers in blue indicate results that are greater than the reduced capacity of the element by less than 10%. Numbers in red exceed the reduced capacity of the member by more than 10%. 3. (*) The corrected value for pile capacity based on an allowable capacity of 120 kips (i.e. 120 kips x 2 x 0.9) and factored dead load of pile subtracted (16.4 kips)				

Table 4-18: Lifting Loads - Pile Analysis Results for 1983 Pile Cap Bents

Capacity / Applied Load	Force (kips)				
	Pile 1	Pile 2	Pile 3	Pile 4	
Capacity	199.6*				
Dead Load 1983 Pile Cap	89	29	28	90	
80 ton, 30 kip Lift at mid-pier and at Bents 13A and 23A	**86**	**62**	**51**	**63**	
130 ton, 44 kip Lift at mid-pier and at Bents 13A and 23A	**115**	**83**	**52**	**92**	
28 ton, 8 kip Lift on 1983 Pile Cap	**145**	**82**	**81**	**148**	
80 ton, 30 kip Lift	**216**	**148**	**147**	**220**	
130 ton, 44 kip Lift	**263**	**193**	**192**	**269**	
	1. All analysis results include dead loads 2. Numbers in blue indicate results that are greater than the reduced capacity of the element by less than 10%. Numbers in red exceed the reduced capacity of the member by more than 10%. 3. (*) The corrected value for pile capacity based on an allowable capacity of 120 kips (i.e. 120 kips x 2 x 0.9) and factored dead load of pile subtracted (16.4 kips)				

SECTION 5 – CONCLUSIONS

5.1 Overall Condition

As evaluated in the January 2010 WFI report, the overall condition of the AUTEC Main Pier is Poor but mainly due to the severe spalling along the utility trenches that have remained since the original construction in 1965 and the isolated areas with broken deck planks. These areas have been designated as NO LOAD ZONES. The other elements held over from the original 1965 construction include the pile caps and steel H-piles at the bent locations without the "A" designation. The H-piles have been encased for protection against further corrosion but the pile caps exhibit moderate levels of deterioration; primarily delaminations and cracks with rust staining. These elements are considered to be in Fair condition. Considering the age of these elements and the fact that warmer temperature waters accelerate concrete deterioration, the condition of these elements is relatively good. This is most likely due to the fact that the concrete cover was specified in the drawings to be 4 in. thick.

The elements installed in 1983 including the piles and pile caps with the "A" designation and the deck planks are generally in Satisfactory condition with almost no defects on the encased steel H-piles and concrete pile caps, and isolated areas of delamination and cracking on the deck planks. Similar to the 1965 elements, the 1983 pile encasements and pile caps were designed with 4 in. of concrete cover. The slightly higher frequency of deterioration discovered on the deck planks maybe because these elements were designed with 3 in. of concrete cover versus 4 in. for the pile encasements and pile caps.

5.2 Limiting Structural Element

The limiting structural element at the main pier is the 1965 pile caps for a number of reasons. Based on their age and type of construction, these elements exhibit the greatest level of deterioration compared to either the 1983 concrete elements or the original steel H-piles that have been protected by being encased in concrete. Furthermore, because the design drawings did not specify the concrete strength or grade of steel reinforcing, Halcrow conservatively assumed 3,000 psi for the concrete strength and 40 ksi for the steel reinforcing yield strength.

Based on the facility's need to operate heavy crane lifts, Halcrow recommends further investigation so that the concrete strength and steel grade of these older pile caps can be firmly established. If the concrete and steel strengths do in fact reflect similar values to the ones assumed in this analysis, Halcrow recommends strengthening the 1965 pile caps so that they are comparable in strength to the 1983 pile caps.

Due to the high construction costs associated with doing work in the Bahamas and the aggressive environment, all repair or strengthen efforts should also incorporate the most durable and cost effective repair. Halcrow recommends installing LifeJackets on the concrete pile caps which are fiberglass forms with zinc mesh attached to the inner face. This product not only cathodically protects the steel reinforcing from further corrosion but also provides a physical barrier preventing additional chlorides from penetrating the concrete as well as also serving as formwork when pumping the concrete repair material.

5.2.1 Uniform Live Load and Moving Loads

Outside the NO LOAD ZONES, there is no reduction in the pier's design uniform load rating of 600 psf and the pier in its current condition can support the moving loads of the 28 ton crane, flat-bed tractor trailer (for loads up to 44 kips), and Fire Rescue No.1 without restrictions. The moving loads from the 80 ton crane and Fire Rescue No. 2 result in shear forces that exceed the reduced shear capacities of the pile caps by less than 10 percent. Halcrow recommends restricting the speed of the 80 ton crane to 5 mph in order to minimize the likelihood of impact loads on the pier. As for Fire Rescue No. 2, because the need for a fire truck on the pier would typically be in response to an urgent matter, it is recommended that Fire Rescue No. 2 restricted from use unless Fire Rescue No. 1 is not operational. The moving loads from the 130 ton crane results in shear forces that exceed the reduced shear capacity of the pile caps by approximately 50 percent, therefore, the 130 crane should be permanently restricted from the pier.

5.2.2 Lifting Loads

Outside of the NO LOAD ZONES, 8 kip lifts by the 28 ton National 11105 crane are permitted without restrictions.

For 30 kip lifts by the 80 ton Grove RT880E, the crane outriggers should be located 12 ft away from 1983 pile caps (i.e. Bents with "A" designations) in the longitudinal direction and along the centerline of the pier in the transverse direction. Prior to the analysis, the facility stated that their preferred locations for the 80 ton crane are distances of 185 ft east of the bulkhead line and 145 ft west of the pier's eastern edge. Based on the analysis, the centerline of the 80 ton crane shall be located at Bents 13A or 23A. Bent 13A corresponds to 188.75 ft east of the bulkhead line and Bent 23A corresponds to 145 ft west of the pier's eastern edge.