PBS&J

Final Design Report

Navarre Pier

Santa Rosa County, Florida

Prepared:
October 25, 2007

FINAL DESIGN REPORT
NAVARRE PIER
SANTA ROSA COUNTY, FLORIDA

October 25, 2007

TABLE OF CONTENT

b. Beams
c. Connections
d. Handrails

2. Substructure
a. Pile Caps
b. Connections
c. Bearing Pads
d. Piles

3. Pile Reactions & Embedment

4. Miscellaneous Wood Structures

IV. Appendices

A. Geotechnical Report

B. Design Forces & Loads
B.1 Wave Forces
B.2 Wind Forces
B.3 Live Load
B.4 Dead Load
B.5 Construction Loads
B.6 Load Combinations & Factors
B.7 Crane Configuration
B.8 Auto turn – Run-On and Run-Off approach angle analysis of vehicles

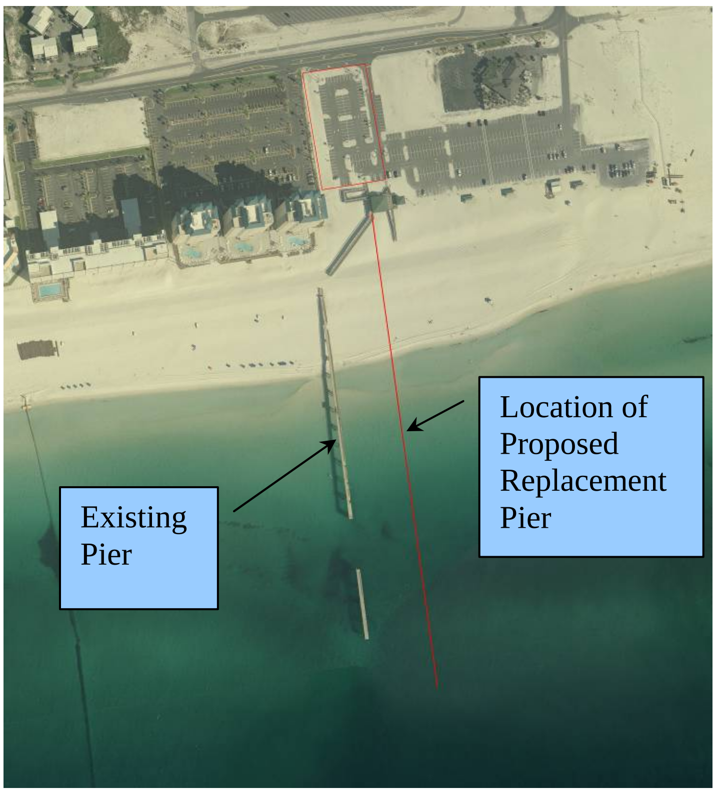
C. STAAD Analysis
C.1 Summary of loads from STAAD analysis
C.1.1 STA 15+ 1^{st} Order Design file
C.1.2 STA 15+ 1^{st} Order Results
C.1.3 STA 15+ 2^{nd} Order Design File
C.1.4 STA 15+ 2^{nd} Order Results
C.1.5 STA 15+ 2^{nd} Order Reactions – reduced envelope
C.2.1 STA 12+ 1^{st} Order Design File
C.2.2 STA 12+ 1^{st} Order Results
C.2.3 STA 12+ 2^{nd} Order Design File
C.2.4 STA 12+ 2^{nd} Order Results
C.2.5 STA 12+ 2^{nd} Order Reactions – reduced envelope
C.2.6 STA 12+ 2^{nd} Order Results with reduced envelope and specific load cases

D. Pile Design
D.1 Pile design with FDOT MATHCAD pile interactions
D.1.1 STA 15+ Pile design for envelope

I. Background Information

The Navarre Pier Design Report summarizes the design criteria and design development process that has generated the new pier design. The report is mainly a summary of the pier main structure design, including constructability, geotechnical and wave load criteria that affected the design and the recommendation for the final structure.

The current Navarre Pier was damaged by several recent hurricanes in the Gulf of Mexico. The pier is located in Navarre Beach, Florida in Santa Rosa County in the western portion of the Florida panhandle. This is midway between Pensacola and Destin, Florida on the longest reach of unbroken coast in Florida. The photo below depicts the existing pier and the proposed pier location.



The new replacement pier incorporates methods to reduce damage from storms and increase the reliability and performance of the structure. These methods include increasing the height of the structure to be primarily above the design breaking wave, use of high performance concrete, use of high performance rebar, use of “simple made continuous” design, use of performance based design concepts to un-load the system, use of full factored loads for design basis loads, use of both pre-stress and non-pre-stressed steel in hi-moment piles.

The project development team consisted of PBS&J for wave loads as well as all structural engineering analysis and design, and Larry M. Jacobs & Associates for the geotechnical investigation and recommendations.

II. Geotechnical Design Criteria

A. General Description and Subsurface Conditions

The subsurface exploration was performed by Larry M. Jacobs (LMJ) and Associates. The complete report of their findings, dated August 29, 2007 is located in Appendix A. The soil exploration for this site revealed that the soils are primarily sands of various compaction levels with some anomalies. The most significant anomaly is a peat layer that will have some affect on the piles performance since it occurs at the mid-height of the piles embedment. The peat layer occurs generally between -23 to -35 ft. The peat layer was taken into account in the design recommendations.

Layers of medium dense to very dense sand were found beneath this layer of peat; these are the soil layers that will provide the most downward resistance and are the soil layers that we will terminate the compression piles in

The pile factor of safety is 1.9 to 2.0.

The recommendations for the area from shore out to STA9+00 are as follows:

Estimated Allowable Pile Capacities - Beach to Halfway Between B-5A and B-6			
Pile Size/Description	**Estimated Pile Tip Elevation (ft)**	**Estimated Allowable Pile Capacity (Kips)**	
		Compression	**Tension**
24"x24" Prestressed Concrete	-42	370	39
24"x24" Prestressed Concrete - Tension Pile	-59	224	104
30"x30" Prestressed Concrete	-42	566	48
30"x30" Prestressed Concrete - Tension Pile	-59	318	130

The recommendations for the area from STA9+00 to the terminal end of the pier are as follows:

Estimated Allowable Pile Capacities - Halfway between B-5A and B-6 to Southern End			
Pile Size/Description	**Estimated Pile Tip Elevation (ft)**	**Estimated Allowable Pile Capacity (Kips)**	
		Compression	**Tension**
24"x24" Prestressed Concrete	-50	480	49
24"x24" Prestressed Concrete - Tension Pile	-57	233	84
30"x30" Prestressed Concrete	-50	737	61
30"x30" Prestressed Concrete - Tension Pile	-57	338	108

All recommendations in the August 29, 2007 LMJ geotechnical report will be followed.

B. Pile/Soil Spring

Linear pile-soil springs were modeled to better simulate the soil-structure interaction. The elastic soil structure interaction allows for a better representation of the redistribution of loads within the structure. A small amount of movement can allow for a large amount of force redistribution. This is particularly important when the structure is relatively rigid with respect to the soils.

Different spring rates were determined for vertical downward forces and vertical upward forces. The spring rate was established in conjunction with LMJ to achieve agreement with historical and calculated allowable movement parameters. The piles were allowed to move approximately ½" laterally and ½" vertically during extreme factored loading. The service level deformations are lower.

III. Structural Design

A. Design Criteria

The codes governing the design for this structure are the Florida Building Code 2004 Chapters 16 and 31, ASCE 7-02, ASCE 24-98, ACI 318-05, and NDS 2005. Many of the design aspects of structures such as the proposed Navarre Pier area not covered in state and local building codes and other design standards. Where appropriate, we have used our judgment and experience with similar structures to define the load parameters that should reasonably apply to this structure.

The required design event is a 20 year Mean Recurrence Interval (MRI) storm design event for components of the structure. The pier design uses traditional design concepts of strength design as well as using some techniques taken from performance based design, such as load relief (lift-off) panels and handrails. The pier is designed with appropriate Load and Resistance factors and should incur little damage to its main structure during the 20 year event. Where practical, measures have also been taken to improve the pier's performance in events exceeding the 20 year event.

The design load combinations cover the load combinations and load factors so that the pier can be evaluated in terms of service and in terms of ultimate extreme loads. These combinations are ultimate factored loads, and serviceability based unity loads. The deck panels are designed to lift off when subject to sufficient upward force. This allows the wave loads to pass around/through the structure with much less affect to the main structure.

1. Environmental Considerations

In an effort to build a replacement pier in the most environmentally responsible manner, the construction process will use methods that least affect the beach system. This technique is called "top-down" construction. This method minimizes the traffic and the material on the beach.

Using this approach, installation of piles, pile caps, deck beams, and decking work begins landward of the dune system. As work proceeds, sections of the pier structure are completed behind the dune. Construction equipment then works from on top of the completed sections of the structure to complete the next seaward section. Using this approach, all work seaward of the dune line can be completed from above the beach and water surface.

2. Construction Issues

The pier is designed to be built "Top Down". The members are primarily pre-cast components that are set in place with heavy equipment, and then joined together. This process helps to reduce construction times and increase the quality of the constructed product. There are some limitations with respect to connections but these are a necessary trade off for the simplicity and speed of pre-cast construction.

The size and weight of the members are one of the major factors governing the design. As the pieces get larger and heavier, the crane needed to move these pieces also increases. This has ramifications on the width of the pier and the size and weight of the pieces. There is an iterative process to find the most efficient size and weight pieces that a commonly available crane can handle. The design crane for this pier is a Linkbelt 238H5 (or similar) crawler crane. The width of the pier is driven by this machine. The beams, caps, piles, and soil interactions have all been designed using this machine size as a basis.

The bay spacing was chosen after discussions with marine contractors familiar with this type of construction. They strongly advised on limiting the bay spacing to 35 feet for constructability issues

3. Operation, Maintenance and Emergency Considerations

The pier has been designed to allow maintenance access for equipment with tire loads of up to 6,000# per tire. The entire pier and maintenance access have been designed for a live load of 125 PSF as well as 6,000# point loads. This will allow for the operation of light equipment, pick-up trucks or, if needed, an ambulance on the pier.

It is anticipated that all people entering or leaving the pier will go through the existing office building. ADA requirements for pier access have been addressed by relocating the current ADA ramp to the front of the existing buildings. The ADA ramps are per the Florida Building Code and have the required slope and landing intervals. The guardrails for the entire pier are at 42" which is consistent with the Florida building code. Provisions for ADA compliant guardrails at specified fishing areas are incorporated via a chain system. This allows for a lower hand rail height at the ADA fishing areas when needed and allows the guard rail to be maintained at 42" the balance of the time.

Access by maintenance vehicles will require opening a gate at the top of the maintenance ramp. The maintenance ramp has a slope of approximately 20%. This is acceptable since it is not for regular use.

4. Mean Recurrence Interval (MRI) in Design Consideration

The design storm event is a 20 year storm and the main pier structure is designed to withstand these forces. Individual components such as guardrails, handrails, handrail posts and deck panels pay yield and unload stress during these events. The purpose of this behavior is to relieve as much excess load as possible while preserving the integrity of the base structure. The height was chosen to allow most of the waves to past beneath the structure.

The last portion of the structure, the octagon-shaped terminus, is designed such that the top 1 ½ ft. of a breaking wave just reaches the bottom of the pile cap. The beams, deck, and guard rails are placed above this expected wave height. While this is a reasonable design approach, it should be stressed that this section of the pier can be expected to experience the most extreme wave and wind forces during storm events.

5. Modeling Limitations

As a result of the high level of variance associated with the modeled wave loads, high load factors (2.0) are used with the data in the analysis.

6. Reinforcing Steel Strength

The steel proposed for use in this project is a new material that is highly corrosion resistant. It has the added benefit of very high strength. We were able to take advantage of both of these issues. Based on current design practice, we are only allowed to design concrete structures with a maximum reinforcing steel yield stress of 80KSI. The reinforcing steel used has a yield stress of 120KSI. This provides a reserve capacity of almost 50% in all the members designed with this material.

7. Detailing

Some concepts from the ACI manual that are used in design of structures in seismic zones have been incorporated in the design of the pier. These concepts relate to detailing of the reinforcing. These are concepts for top bar continuity and for mechanical anchorages. This will allow for tension forces to be transmitted through the diaphragm and not have the beams pull apart. The mechanical anchorages are helpful in not relying on concrete to transfer or develop the force transfer between bars. This is helpful in transferring axial tension loads in load reversal regions or regions of negative moment. These two ideas allow for large deformation without structural failure.

8. Simple Made Continuous

This idea allows the structure to be pre-cast and loaded in "simple" spans, but then transformed into the equivalent of a continuous beam. The design allows for significant moment redistribution due to elastic and plastic deformations. Each section has been designed for construction loads as well as environmental loads. The most important loads during construction are the crane loads, while the most important loads during the remainder of the structures life are the environmental loads – specifically the wave impact loads. All the loads were considered as portions of the design envelopes that were generated. There are a total of 62 load combinations.

9. P-Delta effects

When designing structures that have slender components, it is important to consider second order effects. The ACI code encourages the use of a refined second-order analysis. The structure was analyzed using the second-order method and was compared to results from the first order method. The model included consideration of variability in member properties per the code provisions as well as soil structure interaction effects. This structure was analyzed by a first order elastic analysis; a second order analysis was then conducted. The first order results were compared to the second order results to become aware of the magnification of forces and displacements as a result of second order affects. The second order force increases were typically in the 5-10% range. The second order displacements were lower. The stability index Q is 0.0375 which is less than 0.05, thus the frame is considered non-sway (ACI 10.11.4.2). Since second order moments were about 5%, deflection amplification between first and second order effects were about 5%, and the stability index was below 5%, these effects are low enough that they are easily accommodated. The second order loads and forces were used in the design of the components.

10. Frame action

It is interesting to note that being “structurally efficient” is not helpful in this case. Structurally efficient structures have high load capacity to self weight ratios. In our case however, the lighter our structure, the worse it performed. The decision to go to with conventional reinforcing was partially a result of this effect since conventionally reinforced sections are heavier than their pre-stressed equivalent. Post tensioned structures allow highly efficient structures, but in our case, structural efficiency must be balanced with performance and serviceability. Due to the high lateral loads and the general configuration of the structure, greater weight and mass are helpful to provide lateral resistance and impact resistance.

The pier bents [the structural name for the pile and pile cap combination] cannot resist the lateral loads developed by breaking waves as stand-alone structures. The wave loads generated are in the same order of magnitude as the self-weight of a single bay of the structure. Due to the narrow foot print of the bent, this leads to force couples that far exceed both the downward and withdrawal capacities of the piles. In order to resist the large breaking wave forces, the structure must use the frame action of multiple bays of the pier. The pier therefore is designed with 10 bays grouped together to allow for the self-weight of 10 bays to resist the wave impact loads. The grouping of 10 bays is also less than the design wave length so that no more than 2/3 to 3/4 of a wave length affects the structure at maximum conditions. The design wave length is 420 feet, while 10 bents or 9 bays creates a sub-structure that is 315 ft long.

11. Design Load Combinations

The applicable design codes requires that the design load conditions be evaluated using certain load factors and combinations. These factors and combinations help to create a series of solutions that can have a better structural performance due to better treatment of loads with differing levels of uncertainty regarding their intensity and recurrence intervals. Higher load factors are used with loads with higher uncertainly. Lower load factors are used with loads with well defined parameters.

The ASCE 7 load combinations are as follows:

1. 1.4 (D + F)
2. 1.2 (D + F + T) + 1.6 (L + H) + 0.5 (Lr or S or R)
3. 1.2 D + 1.6 (Lr or S or R) + (L or 0.8W)
4. 1.2 D + 1.6W + 2.0 Fa + L + 0.5 (Lr or S or R)
5. 1.2 D + 1.0 E + L + 0.2S
6. 0.9 D + 1.6 W + 2.0 Fa + 1.6 H

7. 0.9 D + 1.0 E + 1.6 H

Where:

D = Dead Load
E = Earthquake Load
F = Loads due to fluids with well defined pressures and maximum heights
Fa = Flood Load – ***This is our biggest lateral load to consider.*** This is to be considered with the maximum wind load. This will not be considered with the maximum construction load since we will be able to remove the crane from the pier prior to landfall of a hurricane.
H = Load due to lateral earth pressure, ground water pressure, or pressure of bulk materials
L = Live Load – This is Live or Construction Load. ***The Crane loading is our biggest vertical load to consider***.
Lr = Roof Live Load
R = Rain Load
S = Snow Load
T = Self-straining Force
W = Wind Load.
Wi = Wind-on-ice determined in accordance with ASCE 7 section 10

Values for above variables:

L = Live Load = 125 PSF
D = Dead Load = concrete at 150 PCF, wood is considered at 50 PCF
L = Live or Construction Load = 125 PSF or 6,000 LBS point load from ambulance tire. This also includes crane plus load on hook at least favorable orientation on outrigger; crane load on running gear (this will be generated by data obtained from discussions with contractors and in consideration of the required equipment and process) See Appendix B.
S = Snow load = 0.0 PSF
H = 0.0 PSF
Rain = trivial
Fa = See Appendix B
T = See Appendix J on Thermal Expansion
W = 110 PSF See Appendix B Wind – Technically the design wind speed is 0.83*the basic wind speed of 150 MPH. In the interest of conservative and reasonable design the 50 year MRI wind will be used for our analysis.
Wi = 0.0 PSF

Pier Deck Design height = 30.0 feet NAVD

12. Material Properties

The strengths of the materials that will be used for the Navarre Pier replacement are summarized below:

<u>Concrete (28-day compressive strength)</u>	
Reinforced Concrete Superstructure	6,000 psi
Prestressed Concrete Piles	6,500 psi
<u>Reinforcement</u>	
Beams, ASTM 1074, Grade 120	used at 80 ksi
Caps, ASTM 1074, Grade 120	used at 80 ksi
Piles, ASTM 1074, Grade 120	used at 80 ksi
Pile Dowels, ASTM A706	60 ksi
Spiral reinforcement, ASTM A 82	70 ksi
<u>Prestressing steel</u>	
ASTM A416 Seven Wire Uncoated, Stress relieved	270 ksi

<u>Wood components</u>
Pressure Treated #2 or better SYP

<u>Wood Piles</u>
10” diameter or larger ASTM D25

B. Analysis Approach

The structure was analyzed using both global structural analysis and component analysis. The global analysis utilized a STAAD 2006 PRO structural analysis software package. The component analysis utilized several methods such as MATHCAD, EXCEL and hand solutions. The design was produced in MATHCAD and EXCEL spreadsheets. The results for all these are compiled in the attached appendices.

<u>1. Global Structural Analysis</u>
The pier design utilizes a structural model to help deal with the complexity associated with multiple load cases (62 in our case), soil/structure interactions and structural behaviors. The generation of numerical models allows for cycling through the design process in a more efficient manner. One aspect of the model results is vast amounts of data.

One method for managing large amounts of model results data is to make use of bounding envelopes and graphical information representations. Graphs that depict bounding envelopes can clearly display hundreds of pieces of information on one page. It should be noted that a envelope is not an actual load case but a summary of worst cases. Design to the bounding envelope necessitates that the design covers all possible solutions contained in the envelope. If a solution can be found for the envelope, then the design is complete. If the full envelop cannot be solved, one method is to reduce the envelope to certain load cases such as construction phase loads only and long term loads only. In the event that the reduced envelope cannot be solved, the next step is to identify the actual loads cases that form the envelope and design for the individual cases. These issues occurred during the design of the pier. The pile design for the STA12+ piles required that the load set be reduced to long term loads and to construction loads. The long term loads then had to be further broken into actual design load cases. Once the actual load cases were identified, the solutions were acceptable.

The elastic analysis makes use of a two phase approach to highlight non-linear P-Delta effects. The first design uses a first order elastic analysis where all the base loads are impressed on the structure. The results are then tabulated and combined algebraically since the assumption is that the structure is remaining linear and elastic. The next step is to conduct a second order analysis where the loads are combined during the analysis. The process iterates the lateral deformation and adds addition loads to the load set. The structure is deemed to be stable when the successive forces and deformations converge from one cycle to the next. It is termed non-linear since the solution can not be solved by a linear set of equations.

The next step is to compare first order to second order results for forces and for deformations. This allows for the second order/non-linear effects to be identified. If the second order affects are in the 5% to 10% range they are very small and are easily managed.

2. Load Representation

Graphical representations of the loads as seen by the analysis program are attached in Appendix C.

Wave Loads

The wave loads are represented by multiple trapezoidal loads on each member. The loads change at each bay and are equivalent to the magnitude and direction depicted in the wave load report. The model uses 9 bays or 10 bents and is 315 ft long. The design wave length is 420 feet.

The wave loads are taken in two orthogonal directions, South and East. The proper combinations of the two loads vectors generate resultants that are equivalent to loads at various angles for design. This allows for two primary wave loads cases for each depth. The waves at our location tend to become depth limited and under-go a behavior known as refraction. The effect of refraction is to turn the wave face toward the shore or to become more shore parallel. The wave propagation becomes more shore normal as it approaches the shore. This behavior is more pronounced when the depth is less than the wave length. In our case, the depth (25ft) is much less than the wavelength (420ft). A graph showing the wave attack angle for two locations on the coast shows that the waves tend to approach shore within (+/-) 30 degrees of shore normal. This is the basis for modeling the load effects as straight on the end of the pier (shore normal or project South or 180d00m00s). For the purposes of our design, the pier extends in the direction of “Project South”. Project South is actually about 172d00m00s from true North. This is approximately 8 degrees from true south and is correctly aligned to project perpendicular to the shore in this location.

Dead Load

The dead load consists of self weight and the load of decking and crane mats. The dead load is a blanketed uniform load in the analysis.

Live Load

The live load was taken as 125 PSF. This is used as a blanket uniform live load. This load is

The live load of a 12,000 # ambulance is actually a lower overall load than a uniform 125 PSF over the same projected area of a truck. Therefore the ambulance load does not control global design. The ambulance load does control the deck panel design.

Construction Loads

The crane load is represented by eight primary loads. The crane load occurs primarily on the outer beams. This places the crane in its worst case orientations. This is useful when combining multiple load patterns to identify worst case effects.

Truck wheel loads are taken in several locations along the interior beams to create a bounded worse case loading set for the interior beams.

Wind Load

The wind loads are taken similar to the wave loads. The primary load cases are generated in two orthogonal directions and the full compliment of load vectors can be generated by proper use of their component geometric summation.

C. Analysis Results

The results from analysis are:

1. Station 15+ with terminus

Exterior/Crane Beam Maximum Positive Moment	26,023	K-in
Exterior/Crane Beam Maximum Negative Moment	-5,372	K-in
Exterior/Crane Beam Maximum Shear	352	Kip
Interior/Truck Beam Maximum Moment	8,428	K-in
Interior/Truck Beam Maximum Shear	104	K
Terminus Beam Maximum Moment	5,517	K-in
Terminus Beam Maximum Shear	58.1	Kip
Pile Cap Maximum Positive Moment	28,493	K-in
Pile Cap Maximum Negative Moment	-23,110	K-in
Pile Cap Maximum Torsion	10,251	K-in
Pile Cap Maximum Shear	352	Kip

Full Envelope

Factored Ultimate level Reactions	571, -57	Kip
Factored Ultimate level Long term Reactions	415, -57	Kip

2. STA 12+, STA 9+

Exterior/Crane Beam Maximum Positive Moment	27,017	K-in
Exterior/Crane Beam Maximum Negative Moment	-6,134	K-in
Exterior/Crane Beam Maximum Shear	360	Kip
Interior/Truck Beam Maximum Moment	8,404	K-in
Interior/Truck Beam Maximum Shear	105	Kip
Pile Cap Maximum Positive Moment	15,960	K-in
Pile Cap Maximum Negative Moment	-11,551	K-in

Pile Cap Maximum Torsion	10,341	K-in
Pile Cap Maximum Shear	285	Kips

Full Envelope

Factored Ultimate level Reactions	530, -45.9	Kip
Factored Ultimate level Long term Reactions	481, -46	Kip

3. Component Analysis

The secondary components such as handrails, handrail posts, deck panels, wood joists, and wood beams are analyzed and designed using Mathcad worksheets. The analysis and design of these components is contained in Appendix H.

D. Design Conclusions & Recommendations

The design recommendations are incorporated into the design drawings and details. They are as follows:

1. Superstructure

A. Deck – The deck will be pressure treated 3x6 planks fastened together with 3 runners underneath them. These units will span from beam to beam and the unit will not be less than 60" long. Fasteners will be ½" carriage bolts, with counter sunk heads.

B. Beams – The exterior beams are to be conventionally reinforced concrete beams. The exterior dimensions are to be 24" wide x 45" deep at mid span. The ends will step down to 36" deep to allow for similar height construction with the interior beams.

The interior beams are to be conventionally reinforced concrete beams. The exterior dimensions are to be 24" wide x 36" deep at all sections.

C. Connections – The reinforcing steel will use mechanical splices at the joint over the pile cap. These are to be Lenton Lock mechanical splices. Additional steel will be placed between the beams and the diaphragm formed and poured to create a continuous beam effect.

D. Handrails – The handrails are to be pressure treated wood.

2. Substructure

A. Bent cap – The pile caps are to be pre-cast members with sockets for alignment on the piles. The pile caps will precast with 6,000 PSI concrete. The dimensions will be 24'-2" long x 4'-6" wide x 3'-0" high.

B. Connections – The cap to pile connections shall be cast with high strength shrinkage compensating grout.

C. Bearing pads – The pre-cast beam and cap elements shall be separated by an elastomeric bearing pad.

D. Piles – The piles are to be pre-cast pre-stressed 24” x 24” square piles. The piles are to be reinforced with both conventional and pre-stressed reinforcement. The piles are to have 2” diameter jetting tubes cast into them. Piles are to be jetted to within 5 feet of elevation then driven to final elevation. The pile are to be reinforced with prestressed and conventional reinforcement.

3. Pile Reactions & Embedment

The pile loads in both compression and tension are derived by the structural design program, taking into account all load combinations. In accordance with standard structural design practice, all full load combinations were examined. This includes various combinations of construction loads (e.g., the weight of the crane during construction) and the effects of waves and wind from the 20 year storm event.

Appendix C includes the results of all load analyses. The combinations analyzed ranged from “full envelope” loads which represent the combination of all construction loads and those from the 20 year storm event to “reduced envelope” loads which represent worst case loads expected from either construction loads or the 20 year storm event.

Since removal of the crane from the pier will be required before any large storm event, the pier will not experience the combination of peak construction loads and peak storm loads at the same time. The “reduced envelope” loads were therefore used to determine the required capacities of each pile. These “reduced envelope” loads are shown in the following table.

It should be noted that the calculated loads shown below are all factored loads; these loads are believed to be conservatively high. Available capacities are taken from the LMJ report and subsequent detailed discussions and are believed to be conservatively low.

From Station 15 up to Station 12 on the pier,, the seaward pile in the pile group generates the most upward force or tension. The piles caps in this section of the pier have four piles per bent and the lead pile is the primary tension pile and is set to a deeper elevation to generate the additional tensions forces needed.

From Station 12 to the landward end of the pier, the wave heights get lower and the forces are reduced. In this section of the pier, the pile caps shift to three pile bents. .

Location	Ultimate Calculated Compression (Kips)	Ultimate Calculated Tension (Kips)	Nominal Available Compression (Kips)	Nominal Available Tension (Kips)	Tip Elevation (feet)	Comments
Station 15 up to Station 12						
Pile Reaction - Full envelope	571	-57	480	-49/-84	-50/-57	
Pile Reaction	415	-57	480	-49/-84	-57	
Station 12 to landward end						
Pile Reaction - Full envelope	530	-46	480	-49/-84	-50/-57	
Pile Reaction	481	-46	480	-49/-84	-57	

4. Miscellaneous Wood Structures

The deck boards for all the decking will be 3”x 6” planking. The guardrail posts will all be 4”x6” posts. The top rail will be a 2”x6” board with a 2”x4” face board. The kick rail will be a 2”x 4” board. All joists are to be 2”x12”, all girders are to be doubled 2”x12” members. All board to pile connections are to use waffle grid spike connectors to increase shear transfer. All wood piles are to be 10” diameter minimum. The full wood design is contained in Appendix H.