A SunCam online continuing education course

Recreational and Commercial Boat Docking Facilities

Continuing Education Course

Part 2: Timber Pier Design

Course Summary:

This course assumes that the continuing education engineer has completed Part 1 of this course titled "Recreational & Commercial Boating Facilities – Part 1, Site Analysis". As part of that course the engineer learned essential site analysis procedures that will be used as a basis for this course, which is Part 2. The course will now continue by taking the engineer thorough the process of designing the main components of a basic light commercial boat docking facility and "wave break". In Part 1, a sample site was analyzed for Wind & Wave Exposure, Water Tidal (or Stage) levels, Possible Major Storm Conditions, and Site Soil Conditions. This continuing education program is intended to provide the design engineer with the essentials for the next logical steps of that process, which are:

1. Overview & Basic Layout of a Facility
2. Basic Assumptions, Design Loads & Formulae
3. Design of the Basic Pier Cross Section
4. Soil Conditions for Pile Supports
5. Lateral Load Considerations & Design
6. Design of a "Wave Break" Wall

These principals may be applied to a range of structures from simple recreational piers to light commercial facilities, with or without wave attenuation features. Each of the listed subjects will take the reader through the step by step process of performing that phase of the design and analysis and will discuss the respective level of service of the docking facility. The procedures laid out herein are suitable for very simple recreational docks to more sophisticated procedures required for light commercial docking facilities. Use of this course material for design purposes is strictly subject to the limitations and disclaimers set forth which are as follows:

This course is intended only as a study guide of design considerations and is limited to maritime facilities of the size and exposure discussed within this specific course. It is not intended nor is it possible within the confines of such a course to cover all aspects of maritime design. It is not intended that the materials included herein be used for design of facilities that exceed the size or exposure limitations as demonstrated by the examples. Nor is it intended that an engineer that is inexperienced in maritime design should study this course and immediately undertake design of marine structures without some oversight or guidance from someone more experienced in this field. This is especially important for design of facilities that are exposed to hurricane, high river stages, storm surge or tornado level storms. Rather it is intended to build the engineer's understanding of maritime design so that he or she can work with other engineers who are more experienced in this area and to allow the student contribute meaningfully to a project. The author has no control or review authority over the subsequent use of this course material, and thus the author accepts no liability for secondary damages that may result from its inappropriate use. In addition this document does not discuss environmental or regulatory permitting, which is a key component of maritime design projects – these matters are best taken up with professionals who routinely perform these functions as regulatory issues can dramatically affect design.

Portions of this document refer to the US Army Corps of Engineers Shore Protection Manual and Coastal Engineering Manual; we wish to formally thank the COE and acknowledge the contributions and research done by the US Army Waterways Experimental Station, & Coastal Engineering Research Center, Vicksburg, Mississippi for there work in producing these manuals.

1. Overview & Basic Layout of a Facility:

The first part of this course will take a design of a simple light commercial pier that would be used to moor two boats for year round use. As a point of reference, from Part 1 of this course the pier will be situated in a small harbor with somewhat limited exposure to the west (Figure 1).

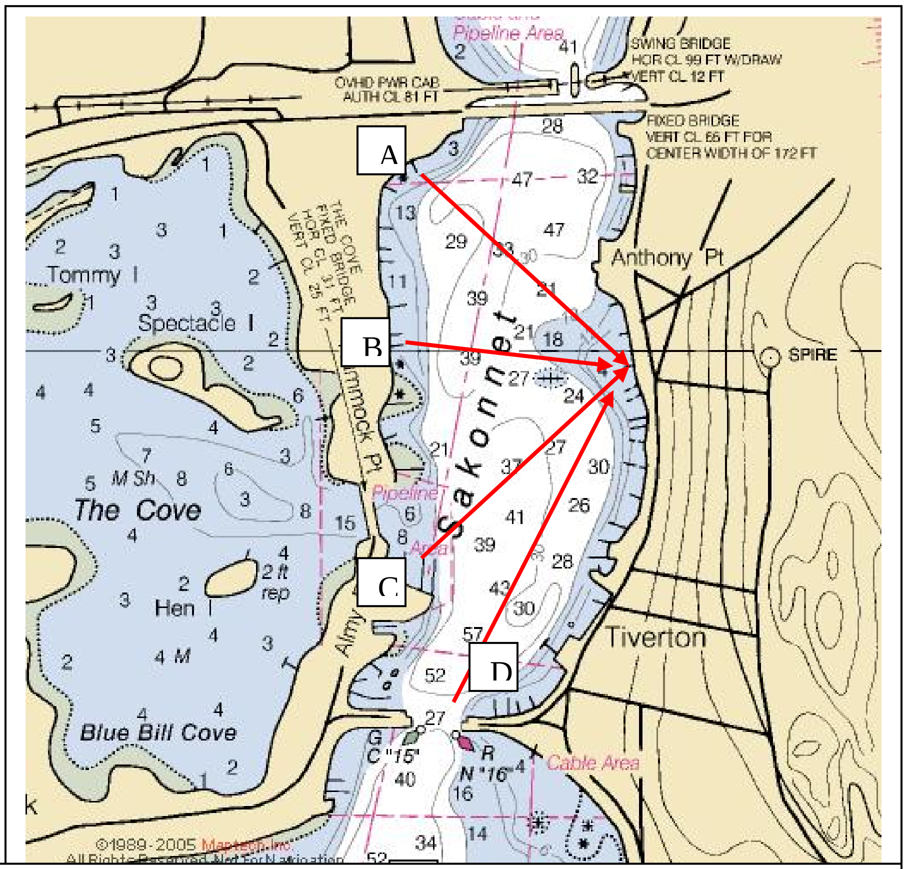

Figure 1: Sample Study Area – with wind/wave exposure shown

Because of the commercial use of the pier, and the wave exposure from the west, a “wave wall” will be utilized to protect the moored boats.

For purposes of this study we have assumed that the client has a commercial use for the boats, and that they will be moored at the pier on a year-round basis. From Part 1, we know that there are two sources of waves that are of concern, the Northwest/ due West exposure (Fetches A and B) and the steepness of the chop that would come from the Northwest is the primary concern, secondarily the Fetch “D” which is known to be a common spring and fall wave direction. For this reason, the client has decided to build a wave break that will be in the form of a timber “wave wall” consisting of horizontal timbers called upper & lower “wales” with tight vertical boards attached to the wales, thus forming the wall. He will dock two each 35 foot research boats at the facility – and will use a floating dock and gangway for access to the boats. The figure below is the preliminary layout developed for the design to proceed from. There will be a main walkway, 5 feet wide and 140 feet long that will end about the -10 foot (MLW) contour, there will also be a 55 foot “L” walkway at the end of the pier that will also be 5 feet wide. About the center of this walkway there will be a 32 foot gangway going down to an 8 foot by 32 foot floating dock that will serve as the primary mooring point for the boats. There will be a 60 foot wave break on the southerly side of the pier that will be 60 feet long, also a wave break on the westerly face of the pier that will be 55 feet long, and a 36 foot long wave break return leg that will not incorporate a walkway. The process of developing the design of the structure will take the individual components of the proposed pier and assess them for the various load conditions that will occur during normal seasonal conditions. There will be commentary throughout the text on design consideration for survival during hurricanes or other severe conditions.

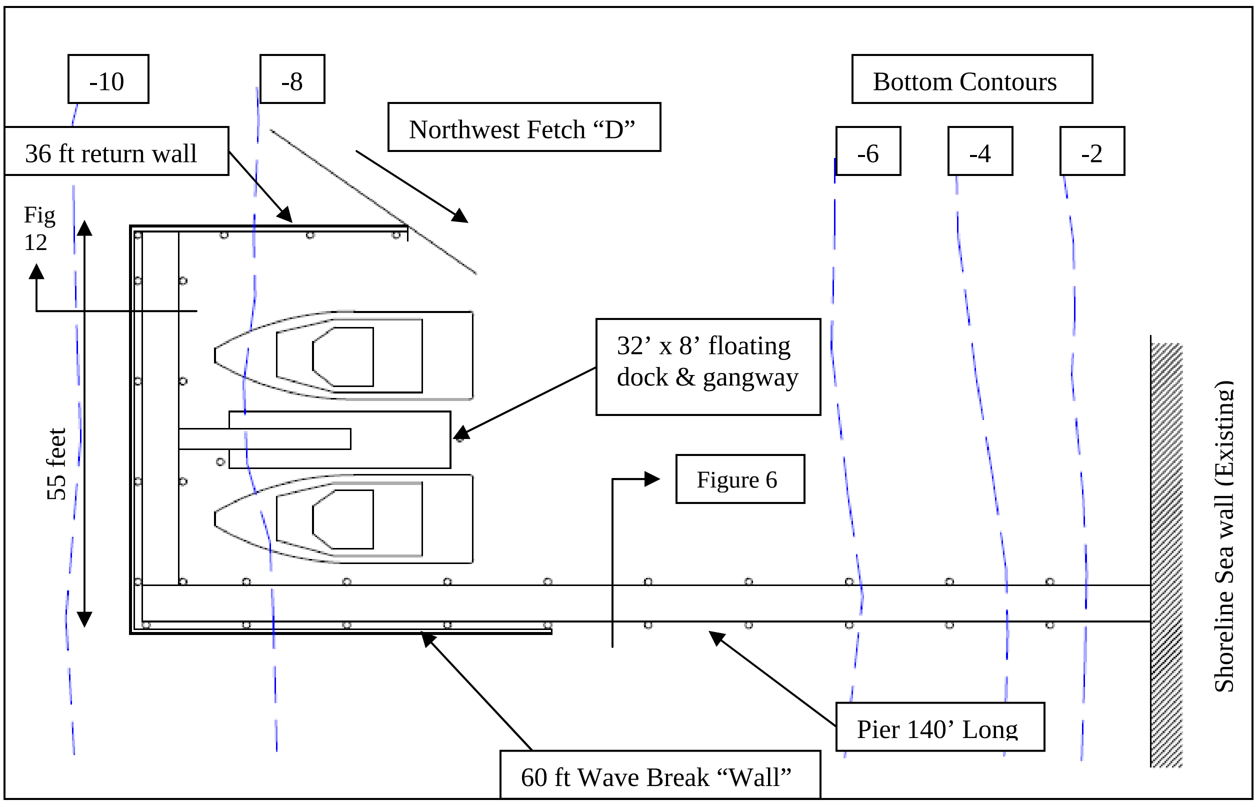

Figure 2: Preliminary Pier Layout of proposed Pier

Based on the survey, water depths are determined to be 10.0' (MLW) at the outer end of the pier, -5.5' MLW at the 2nd bent, -3.0' MLW at the 1st bent, and 0.0' MLW at the seawall (shoreline). Based on this data the water depths at the various water stage conditions would be as follows:

	Water Depth			
Location	MLW	Mean HW	Max HW	Hurricane
Outer Bent	10.0’	13.8’	15.6’	25.5’
2nd Bent	5.5’	9.3’	11.1’	21.0’
1st Bent	3.0’	6.8’	8.6’	18.5’
Seawall	0.0’	3.8’	5.6’	15.5’

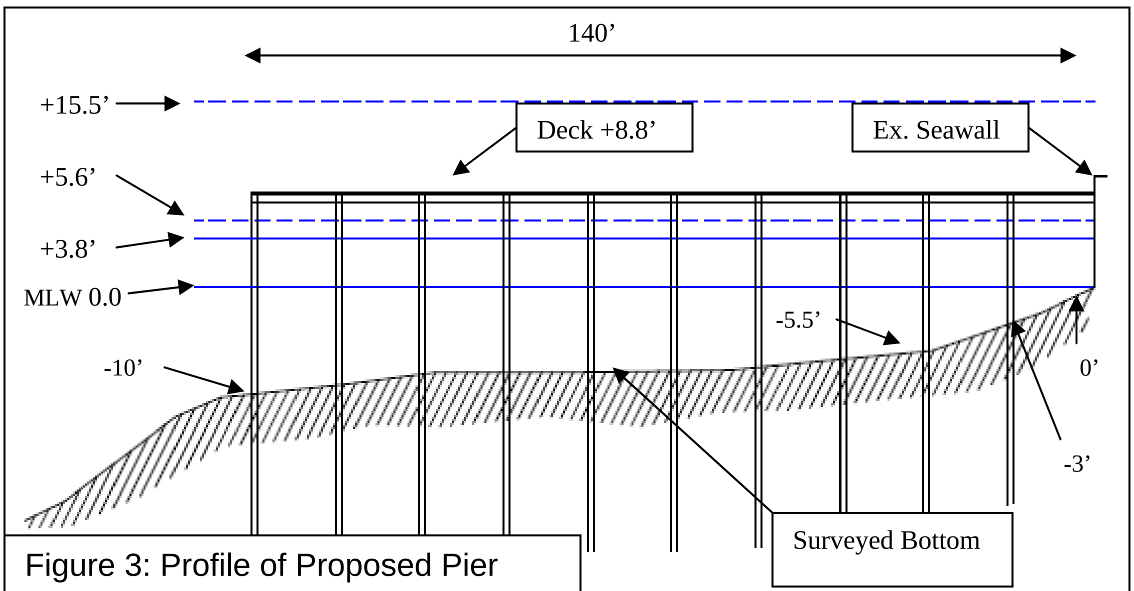

Figure 3: Profile of Proposed Pier

These figures along with the deepwater depths will be used to evaluate the supported and unsupported lengths of the piles to determine pile materials as well as X-Bracing requirements.

2. Basic Assumptions, Design Loads & Formulae

The following will be the basic design parameters to be followed in designing the pier: (Note: Because the wood in most marine structures is under constant weathering attack, the rate of which is very unpredictable and may be subjected to occasional loads in excess of the design parameters, it has been our policy on simple pier structures subject only to relatively light loads to avoid the more laborious detailed methodologies. Rather focusing on simple design procedures, but also allowing for higher factors of safety and system redundancy in the process. The reasoning being that unlike upland structures – marine piers face degradation in many forms – thus a higher level of caution is exercised)

Design Loading Assumptions:

1. Live Load on Deck = 60 pounds/ square foot – uniform load, or a 200 pound concentrated load on one plank.
2. Dead load of wet treated wood = 55 pounds/ cubic foot.
3. Allowable bending stress in treated southern pine wood = 1200 psi, Shear = 135 psi, Modulus of Elasticity = 1.2×10^6. The allowable bending stress in Greenhart Piles (South American Hardwood) = 2200 psf; Modulus of Elasticity = $1.7 \times 10^{6.}$
4. Allowable increases for short term loadings such as Waves 33%; for Deck Loads such as personnel, carts etc 25%.
5. Reductions in allowable unit stresses due to treatment & weathering of wood – Treatment 10%, weathering case by case.
6. Specific Gravity of wood 50.

7. Design Wave H = 2 feet, T = 2.0 seconds. Survival Condition 4.3 foot wave, T = 2.6 seconds

Design Basic Formulae:

Formula for Maximum Bending Moment

Uniform Load: $M_{max} = WL/8$ [Formula 5]

Concentrated Load: $M_{max} = PL/4$ [Formula 6]

Formula for Maximum Deflection

Uniform Load: = $D_{max} = 5\ WL^3/384\ EI$ [Formula 7]

Concentrated Load: $D_{max} = PL^3/\ 48\ EI$ [Formula 8]

Section Modulus of rectangular wood

$S = bh^2/6$ [Formula 9]

Moment of Inertia of rectangular wood

$I = bh^3/12$ [Formula 10]

Piles in bending (unrestrained at top)

$M_{max} = PL$ [Formula 11]

Moment restrained at top

$M_{max} = PL/2$ [Formula 12]

Section Modulus of pile (at bottom grade)

$S = 0.785R^3 = 170$ for 12" pile [Formula 13]

Moment of Inertia of pile

$I = 0.785 R^4 = 1017$ for 12" pile. [Formula 14]

Terms used in Formulae:

W = Uniformly distributed Load of Given Length (weight per foot x length in feet)

P = Concentrated Load (Pounds or Kips as desired)

L = Length of Span (in the cases posed in this text "L" is in inches

D_{max} = Maximum Deflection (in the cases posed in this text "D" is in inches)

M_{max} = Maximum Bending Moment (In the cases in this text Inch-Pounds (or Inch-Kips)

S = Section Modulus (inches^3)

I = Moment of Inertia (inches^4)

b = Width of rectangular timber in Inches (Perpendicular to direction of bending force)

h = Height of rectangular timber in Inches (Parallel to direction of bending force)

R = Radius of round Timber (inches) Note that piles are tapered, so one has to specify or at least take into account what the diameter will be at the point of maximum bending moment – which is normally a few feet below the bottom elevation – also referred to as the “mud line”.

H = Significant wave Height (also called H_s) in feet

T = Period of Significant Wave in Seconds

2×6^N = Denotes Nominal Timber size, generally deduct ½” to obtain actual timber size (i.e. 2 x 6 is actually 1.5” x 5.5” – which are the dimensions that need to be used as “b” and “h” or “R” in calculation of S, I and D)

3. Design of the Basic Pier Cross Section

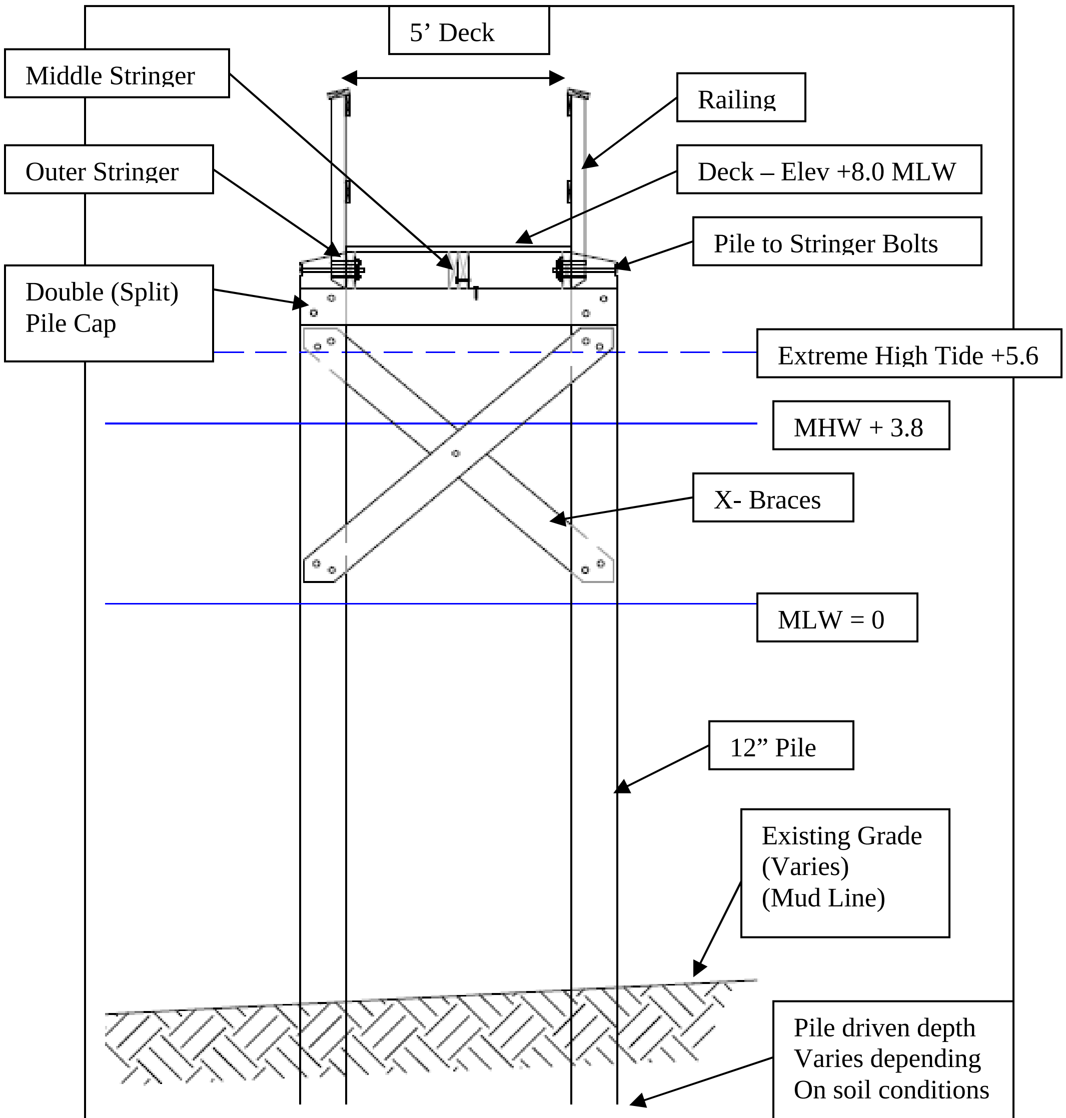

Figure 4: Typical Pier Section – without wave break. Span between bents = 14.0'

Figure 4 above used in conjunction with Figures 2 and 3 is the usual starting point for the design process, and is a generic pier design. Also shown are the major components, exclusive of the Wave Break.

Details will be discussed at a later point in the text; in this segment we will size the major components.

Major Component Design Process:

a. Determine deck plank size. From Figure 3, the overall deck width from outside to outside of stringers is 5'-0". Based on the design configuration, the maximum span condition for an individual plank is the dimension from the centerline of the pier to the outside deck face of the assumed outer 3 x 10^N stringer. Thus, the plank span will be (5.0' x 0.5) = 2.5 feet span on a single 2 x 8^N plank. The design loads are 60 psf or 200 pound concentrated load in center (Design Assumption 1). [Note: It is generally good practice to round spans and conditions upwardly to account for nominal inaccuracies that can in marine construction. In addition surface loads are usually applied to the nominal timber surface versus the actual timber surface; i.e. the load would be applied to the 8" width as opposed to the 7.5" width – as deck planks generally have at least a ¼" gap between them.]

b. Uniform Load would be the span x the width of the plank in feet (8" or 0.667'); W = 60 psf x 0.667 x 2.5 = 100.0#, this is less than the Bending Moment that would be developed by a single 200 pound concentrated load - therefore single concentrated load controls.

(1) Maximum Bending Moment = PL/4 = (200 x 2.5 x 12)/4 = 1500 inch pounds (Formula 6).

(2) Section Modulus of a 2 x 8N deck plank where the actual "b" is 7.5" and the actual "h" is 1.5"; S = (1.5^2 x 7.5)/6 = 2.81"^{3} (Formula 9)

(3) Unit stress on plank from concentrated load = 1500/ 2.81 = 534 psi < 1500 psi (Assumption 3 = 1200 x 1.25 = 1500 psi) the plank is okay

c. Next take a typical section of the Pier's walking surface from between the pile "bents" (A "bent" is the term for the two- pile and cap configuration shown in Figure 4). For this design we have selected a span between bents of 14.0'. So the load on the "stringers" between the pile bents will be calculated using the combined live and dead loads. First Calculate the Dead Loads, then the Live Loads:

(1) Dead Load of Deck (2 x 8N planks) = thickness of 1.5" or 0.125' x 5.0'(width) x 14.0' (length) x 55 pcf (weight of wet wood) = 481.25#

(2) Dead Load of Stringers (assume 3 each 3 x 10N) = [(2.5 x 9.5)/144] (Area of 3 x 10 in inches) x 14.0' x 3 (each) x 55 pcf = 381 pounds

(3) Live Load is 60 psf (Assumption 1) x 5.0' x 14.0' = 4200 pounds

(4) Total Load on 14' span = 481# + 381# + 4200# = 5062# (W)

d. Determine suitability of 4 x 10 center stringer and 3 x 10 outboard stringers (2 each per bent) at 14 foot span.

(Note: For most of these calculations we tend not to segregate live & dead loads to retain simplicity of analysis, as an above normal factor of safety needs to be considered for the weathering of wood and other environmental factors. However if the reader wishes to make the factored analysis it is also completely appropriate – however additional thickness should be considered. Answers to test questions will take this into account – or be very close to the factored result).

The center stringer will take half the load, and the two outer stringers will take ¼ of the load. Therefore using Formula 5, the maximum bending stress on the center stringer will be:

(1) Calculate Maximum Bending for Center & Outboard Stringers

$M_{max(ctr)}$ = (5062# x .5 x 14 x 12)/8 = 53,151 inch pounds.

$M_{max(outer)}$ = (5062# x .25 x 14 x 12)/8 = 26,576 inch pounds.

(2) Section Modulus of 4 x 10 & 3 x 10 stringer:

$S_{4x10} = (9.5^2 \times 3.5)/6 = 52.6''^3$ (Formula 9)

$S_{3x10} = (9.5^2 \times 2.5)/6 = 37.6''^3$ (Formula 9)

(3) Moment of Inertia of 4 x 10 & 3 x 10 stringer:

$I_{4x10} = (9.5^3 \times 3.5)/12 = 250.0''^4$ (Formula 10)

$I_{3x10} = (9.5^3 \times 2.5)/12 = 178.6''^4$ (Formula 10)

(4) Unit stress on Center Stringer:

53,151/ 52.6 = 1010.5 psi < 1500 (1200 x 1.25) okay.

(5) Check Deflection Center Stringer using Formula 7:

D_{max} = 5 x 5062# x 0.5 x (14.0' x 12")^3 / 384 x E x 250.0 = 0.52" (About 1:320 okay)

(Cross-Check Deflection of 3 x 10)

D_{max} = 5 x 5062# x 0.5 x (14.0' x 12")^3 / 384 x E x 178.6 = 0.73"

(About 1:230 – This would be a little bouncy considering commercial use – so we will stick with the use of a 4 x 10)

(6) Unit stress on Outboard Stringer:

26,576/ 37.6 = 706.8 psi < 1500 (1200 x 1.25) okay.

(7) Check Deflection Outboard Stringer using Formula 7:

D_{max} = 5 x 5062# x 0.25 x (14.0' x 12")^3 / 384 x E x 178.6 = 0.36"

(Less than 1:400 okay)

Note: In most cases on longer span wood piers deflection tends to govern over unit stress – we tend to prefer a stiffer design for commercial structures because they tend to get overloaded with equipment from time to time. Also, since outer stringers could also take lateral impact loads from drifting boats, debris, etc. we prefer to use a 3 x section as a minimum; this also provides a little more weathering protection. Thus the outer stringers will be 3 x 10 sections.

(8) Shear load of the stringers at pile cap tends to be quite low. i.e. Shear = (5061 x 0.5)/ 2 = 1265#. Shear area of stringer = 3.5 x 9.5 = 33.25 square inches, therefore shearing stress is 38 psi < 135 psi therefore okay.

(9) Lateral Bolted connection to pile. Vertical Load on outer one outer stringer at one end is 5062#/4 = 1265#. From Appendix 2 for bolted connections we can see that a 1" bolt can take a load of 1715# which is well above the requirement. Besides reducing the load on the split cap, this connection would also take a considerable amount of uplift from storm driven waves combined with flooding, thus they serve the dual purpose of stabilizing the deck structure as well as providing uplift protection. Also see the additional discussion under "Pile Cap" for more information.

e. Check Split Pile Caps: A Pile cap is the cross member that the stringers sit on, a split cap consists of two timbers – in this case 3 x 10s which are bolted through the piles (see Figures 4

& 5), with one 3 x 10 on either side of the pile therefore acting like a clamp. Good practice is to flatten the areas where the caps bolt against the piles using a chain saw; this is called "dapping". The cut is usually about 1 to 1 ½" deep and allows for a better "flat fit area" for the bolted connection to the pile, and also provides a seat for the cap timbers. Simplified details of this connection are shown in Figure 5, and since bolt calculations can be somewhat lengthy – calculations for the bolting is attached in Appendix 3. Note also that the outside stringers bolted laterally to piles, which serves the dual purpose of protecting against uplift during floods, and also taking some of the vertical load off of the cap. Properly constructed the Cap only takes concentrated load from the center stringer.

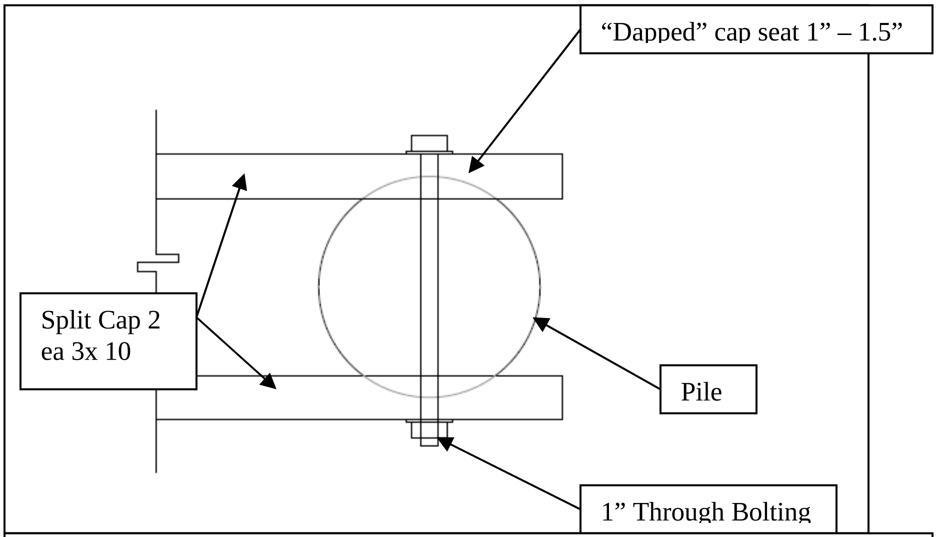

Figure 5: Typical Split Pile Cap detail – looking down on Pile

(1) From Step c. 1 to 4, the load from the deck and stringers is 5062#, the load on the outboard stringers is 0.25 x 5062# = 1265.5#, and the load on the center of one half of split cap = 2531#. Span from center to center of pile bolt connections is 6.0 feet. Therefore the maximum bending moment on one half of the split cap is:

M_{max} = (2531 x 6 x 12)/ 4 = 45,558 inch pounds, (Formula 6)

Since the load is divided evenly between the two caps, or 22,779 inch pounds per 3 x 10.

(2) Check one side 3 x 10 pile cap S = 37.6, therefore:

22,779/ 37.6 = 605.8 psi < 1500 psi (1200 x 1.25) okay.

(3) Check Bolt loading (assuming no Dap): Vertical load on one pile = 5062# / 2 = 2531#. For simplicity we prefer to use a 1" diameter bolt; from Appendix 3 bolt connection to Pile Cap in double shear, we can see that the allowable load on a single bolt is 2895#, which is sufficient. However, if there is room we will usually specify two bolts, even though one bolt will do. There are a few reasons for this, one being that even hot dipped galvanized bolts begin to corrode in less than ten years, in twenty years

they can show considerable metal loss from corrosion. Another reason is that over time the bolt holes tend to open up and make the bolts loose, and thus allow the nuts to work loose. Thus the second bolt adds a degree of security and longevity to the connection. Note that no downward load reduction consideration has been given to the dapped pile connection. For instance, a 1” dap on a 12” pile is about 4.24 square inches, a 1 ½” dap is about 8.16 square inches, unless under inspection contractors sometimes tend to be a little careless making these fit ups, so tend to ignore the benefit, thus the dap usually ends up being an additional factor of safety in case the pier becomes neglected and the connections are allowed to deteriorate. A 1” dap at 4.24 square inches, using an allowable bearing load of 350 psi would produce about 1484# of bearing for each side of the split cap or 2968# total which would sustain the entire design load by itself, this is considered cheap insurance. It should be obvious that the dap alone provides no uplift protection. It is very important to note that uplift protection is a critical and often neglected factor in pier design. Storm driven flood waves can produce considerable uplift forces – which could lift the deck off of the piles and turn it into a floating raft and create a considerable hazard for moored boats, emergency service crews and other structures; taking these factors into consideration it becomes easy to understand the importance that this connection be secure.

The 3 x 10 cap is more than ample, so it is not necessary to do a separate evaluation that includes the outer stringers as the short offset only increases the maximum bending moment by about 20%. We tend to use this larger section for more practical reasons mainly because the 9.5 inch face leaves more room for the bolts that must go through the piles.

4: Soil Conditions for Pile Supports

The work thus far in the structure design determines the basic components, except for the Piles and X-Bracing. In Part 1 of this course the various types of sub-surface investigations were discussed in detail and thus will not be repeated here.

Once soil information is obtained, it is important to have it reviewed by an individual experienced in marine geotechnical analysis to determine the minimum pile embedment length as well as the lateral support characteristics. This information must them be weighed against the service level of the anticipated structure, obviously the more severe the service and exposure the more important this information becomes in the design process. We will now apply all of the above logic this into our test case as follows: Because of the commercial use of the pier, and the loads that the wave wall will encounter, more sophisticated soil evaluation is considered important, thus split spoon samples were obtained. We will assume that the tests were reasonably consistent and that the top 6 inches of soil was loose marine sediment, and that under that was medium dense sand with some gravel with blow counts in the 20 to 30 blow per foot range down to 20 feet, and that very dense sand and gravel was encountered at that depth – this layer would be where the piles would most likely achieve bearing capacity. It should be noted that we recommend a minimum of 15 feet embedment into firm soils in northern climates to protect from ice pull-out, and 10 feet for lightly loaded piers in southern ice-free climates. On more complex projects with long piles and extreme lateral loads there are calculation procedures that should be followed to determine the depth of pile fixity – the cases discussed herein do not require that level of analysis. In general we recommend ignoring any soft sediment layers with respect to lateral support, and also ignoring at least the first eighteen inches of firm surface soil.

5. Lateral Load Considerations & Design

Once the above information is obtained - the pile design can proceed, we will evaluate pile lengths taken from the profile shown in Figure 3, which range in exposed length from sea bed to the center of bolted connection on the cap from10.0 feet to 14.5 feet. Because of the simplicity of these calculations we will perform them in a spreadsheet program and determine the unit stress produced at one foot increments. This will require Formula 9 to determine the pile Section Modulus and Formula 10 to determine the bending moment assuming that the pile is pin connected at the top.

This section will only deal with the pier itself, and will neglect the wave-break for the time being – since that is a separate study of its own. We sited earlier that we would design for the normal weather conditions which are extreme conditions that occur several times a year, after which we would look at some of the considerations given to the survival condition of an extreme storm event. First some thought must be given as to what might happen in the life of the pier, which one would expect to last at least 40 to 50 years. The most common lateral load occurrence is wave impact from the side at an extreme high tide however wave impact is a function of the exposed vertical face, which in this case is only the side face of the stringers and deck. This totals only about 12 inches at the deck line, and about a 12 inch pile profile every 14 feet (Figure 6). While this is a condition worth considering under the worst of circumstances it would only generate a lateral load of about 300# per lineal foot (the basic methods and formulas for obtaining this figure will be discussed in the section on the wave break). A less common condition, but one worth considering would be if a boat were to be docked in this section of the pier during a storm. This is an exercise worth exploring, even though the planned design does not intend for this area to be used for docking – one can never tell what logical condition might take place during the life of the pier - and as such conditions should be allowed for in the design loadings. Such a condition would not only transmit wave energy against the pier, but it would also bring a wind component with it. For this design we will consider the loads that a 35 foot power boat would bring to bear if it were to be moored to the pier during a wind storm that was producing 40 knot winds and two foot waves. Windage areas vary considerably from boat to boat, but the average 35 foot fishing boat would have about 340 square feet of windage profile area. The formula for wind pressure is the dynamic drag equation, which for this application can be reduced to the following for purposes of expedient design:

$$P = .0034\ V^2\ Co \qquad \text{[Formula 15]}$$

Where P is the wind pressure against the boat hull in pounds per square foot, V wind speed in Knots, and Co is a factor based on the boats shape (in this case we will use 1.0).

Entering the 40 knot wind into the formula we get about 5.44 psf of pressure applied to the boat profile of 340 square feet, or about 1850 pounds; which would reduce to about 53 pounds per lineal foot of boat. In addition to the wind force a two foot wave on a boat drafting about 2.5 feet of water would produce about 490 pounds of lateral force per lineal foot of boat (This is a very conservative assumption, however calculation of wave loads will be discussed later in this text). Since the boat is longer than the pile spacing we would apply the total load per lineal foot or about 550 pounds per lineal foot (rounded), times the distance between piles; or 14 x 550 = 7700 pounds (again a very conservative assumption). We will now apply this load to the top of the pile at the deck elevation and first check without the X-Bracing to see if the piles can withstand the loads as a pure cantilever.

The following table was developed using the formulas 11 & 12

Bending Moment Calculations - Cantilever Piles

Pile Length (Feet)	"Soft Mat'l" Allowance (Feet)	Total "L" (Inch)	Total "P" (#)	Total Moment (In #)
10.5	2.0	150	7700	1,155,000
11.5	2.0	162	7700	1,247,400
12.5	2.0	174	7700	1,339,800
13.5	2.0	186	7700	1,432,200
14.5	2.0	198	7700	1,524,600

Bending Stress - Cantilever Piles

Total Moment (In #)	Pile Sect Mod	Unit Stress (PSI)
1,155,000	170.0	6794
1,247,400	170.0	7338
1,339,800	170.0	7881
1,432,200	170.0	8425
1,524,600	170.0	8968

Table 1 (a & b): Calculation of pile stress at one foot increments – un-braced condition, assuming boat moored along side. Using Formulas 11 & 12

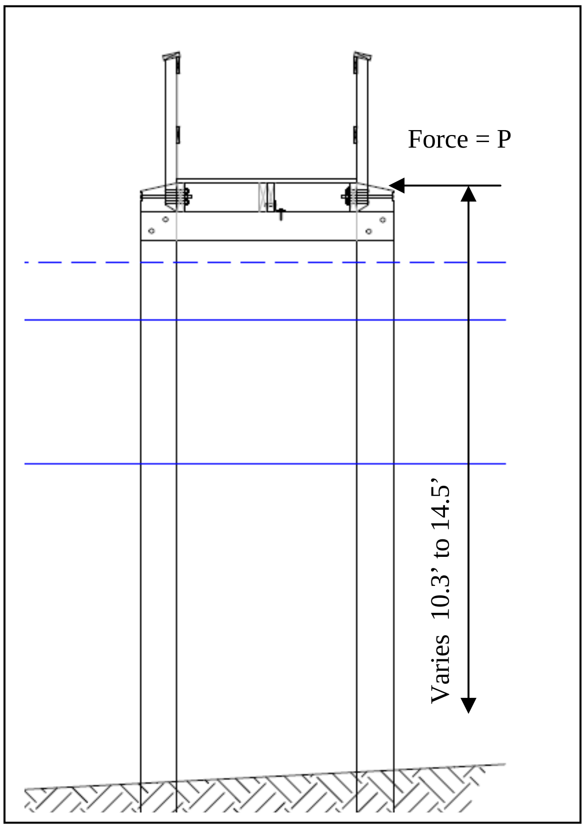

Figure 6: Typical Pile Bent, without X-Bracing

In all cases it can be seen that the bending stress exceeds the allowable unit stress of either greenheart piles (2200 psi x 2 = 4400 psi x 1.25[2] = 5500 psi allowable), or pressure treated southern pine piles (1200 psi x 2 = 2400 psi x 1.25 = 3000 psi allowable).

[Note (2): normally we would use 1.33% for wind or wave loads, however since a moored or drifted boat constitutes a more consistent load state, the factor was reduced to 1.25%].

Based on the above, it becomes evident that if the case were assumed for the drifted or moored boat, one would have to assume that the non-wave break pile bents would require X- bracing. For purposes of this study however, let us also explore the possibility of a lesser condition, that being the wave load only – which was about 300# per lineal foot, or 4200 pounds.

Bending Moment Calculations - Cantilever Piles

Pile Depth (Feet)	"Soft Mat'l" Allowance (Feet)	Total "L" (Inch)	Total "P" (#)	Total Moment (In #)
10.5	2.0	150	4200	630,000
11.5	2.0	162	4200	680,400
12.5	2.0	174	4200	730,800
13.5	2.0	186	4200	781,200
14.5	2.0	198	4200	831,600

Table 2a: Calculation of maximum bending on piles, without boat moored along side (Formula 11 & 12)

Bending Stress - Cantilever Piles

Total Moment (In #)	Pile Sect Mod	Unit Stress (PSI)
630,000	170.0	3706
680,400	170.0	4002
730,800	170.0	4299
781,200	170.0	4595
831,600	170.0	4892

Table 2b: Calculation of resulting bending stress distributed between 2 piles based on Table 2a results

In this case, if Greenhart piles were used the X-Bracing would not be necessary, however if treated southern pine were be used, X-Bracing would still be required in all cases. Since Treated Southern Pine is considerably cheaper than Greenhart, and the X-Bracing would make for a much more durable pier, the choice of treated pine and X-bracing makes the most financial sense. However it should be noted, with the advent of hardwood piles that are being sustainably farmed, and regulators in many areas of the country are mandating the use hardwoods over the use of the more toxic treated wood to limit the leaching of treatment chemicals into the waterbody. Thus it is very important to check the environmental regulations on this issue before proceeding with the design.

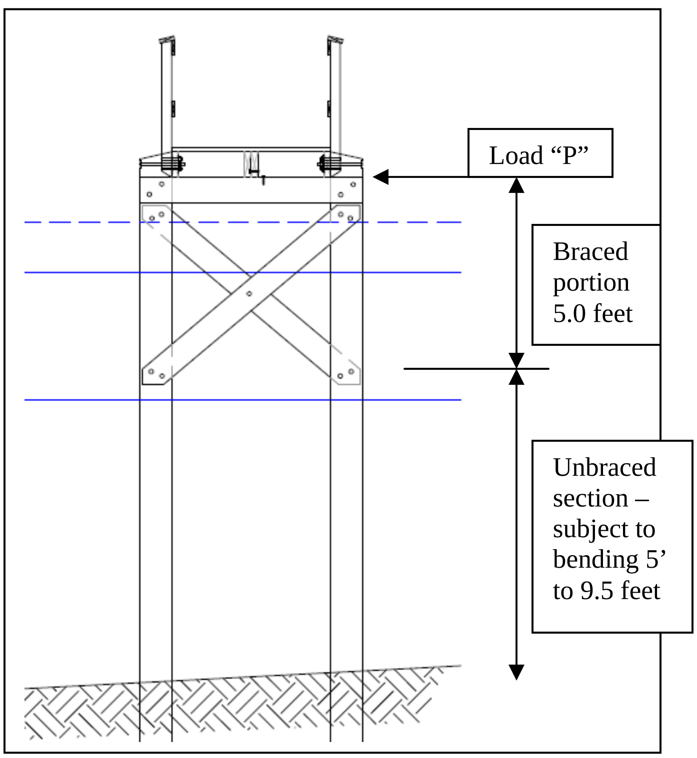

Figure 7: X-Braced Pier Pile Bent

Thus we need to consider the bending moments of the piles with X-Bracing and the loads that will be generated within the braces (Figure 7). In the case of X-Bracing we treat the upper portion of the pile as a moment resisting connection, which it functionally is. In addition we will use vector analysis to calculate the compression and tension in the X-Braces as well as the connecting bolts. Tables 3a & 3b below show the results of the revised calculations:

Bending Moment Calculations - X-Braced Cantilever Piles

Pile Depth (Feet)	"Soft Mat'l" Allowance (Feet)	Total "L" (Inch)	Total "P" (#)	Total Moment (In #)
5.5	2.0	90.0	7700	346,500
6.5	2.0	102.0	7700	392,700
7.7	2.0	116.4	7700	448,140
8.5	2.0	126.0	7700	485,100
9.5	2.0	138.0	7700	531,300

Table 3a: Bending moments recalculated using Formulas 12 & 13, and shorter unbraced length

Bending Stress - Cantilever Piles

Total Moment (In #)	Pile Sect Mod	Unit Stress (PSI)
346,500	170.0	2038
392,700	170.0	2310
448,140	170.0	2636
485,100	170.0	2854
531,300	170.0	3125

Table 3b: Revised calculation of bending stress distributed over two piles based on Table 3a results. (divide the unit stress in the table be 2 for the stress in the individual pile)

Based on the tabular data presented above all of the piles except the longest (9.5 feet unbraced length) fall within the allowable unit stress for the two Southern Pine piles of 1500 psi per pile (1200 psi x 1.25) [i.e. 3125 psi/ 2 piles = 1562.5 psi]. In the case of the pile that is somewhat overstressed we will extend the X-Brace down about six more inches to bring it within allowances. Note that the use of X-bracing, which was determined to be advantageous in any event, also allows for the application of the higher lateral forces (7700#) as discussed earlier. Thus additional durability has been built in at very low additional cost.

The next step is to calculate the timber sizes for the X-Brace and determine the number of bolts required per connection. This will be done using simple graphic vector analysis.

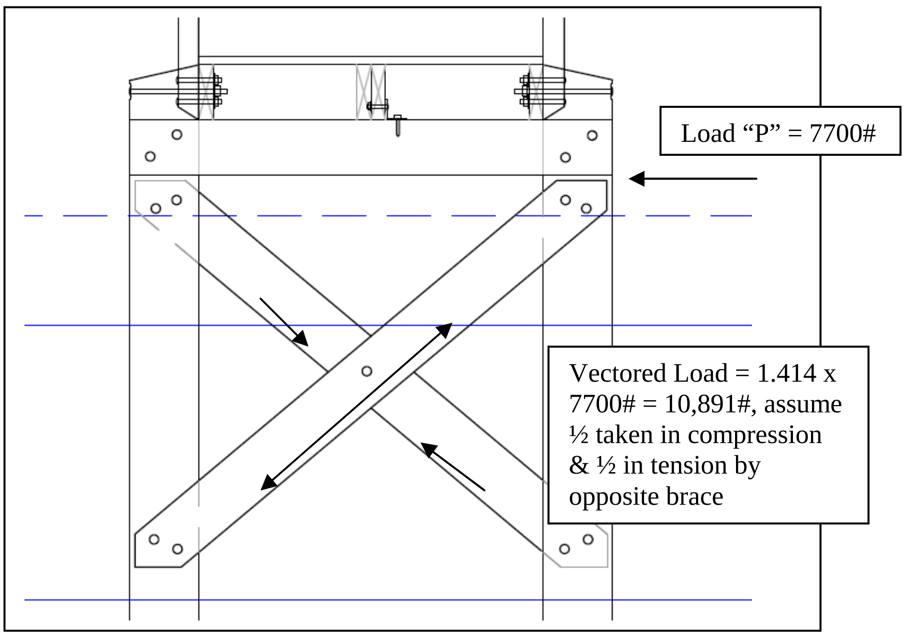

Figure 8: Detail of X-Braced Bent, showing simple load distribution

Based on Figure 8, each of the X-Braces shares a portion of the 10,891# load, or 5446# in either tension or compression. For the tension load this would be distributed over the cross section of 2.5" x 9.5" or 23.75 square inches, or about 230 psi., which is well below both allowable tension and compression loads for this short distance (refer to standard timber design manuals for longer length braces in compression).

The X-Brace is probably the most often under-designed component of a pier structure, which is probably the reason that most inspections of older pier reveal broken X-Brace connections. Referring to Appendix 1 we will find that a 1-3/8" bolts in the configuration shown can carry a load of 2717#. However, since the loads on X-Braces are of very short duration most of the time, we should be able to increase their capacity by 25%; which would be the same as reducing the load to 2178#. Inserting these figures into the formulae in this same appendix we see that 1 ¼" bolts would have a capacity of 2314#, and 1 1/8" bolts would have a capacity of 1947#. Since the maximum loads on these bolts would only be approached during an extreme condition, and factoring in the limited exposure of this site, we would opt for two 1 ¼"bolts per connection. It is advisable to also detail this connection carefully so that the contractor knows how to place the bolts with proper spacing, and edge distance within the allotted space.

6. Design of a "Wave Break" Wall

The first thing that one must consider is the lateral loads that will be generated by the design wave on the vertical face of the wave break. These loads can be significant and they must be carefully considered in design. From Section 1 of this study we have determined that a 2 foot wave with a 2 second period will be the design wave, and the survival condition will be the 4.3 foot wave with a 2.6 second period.

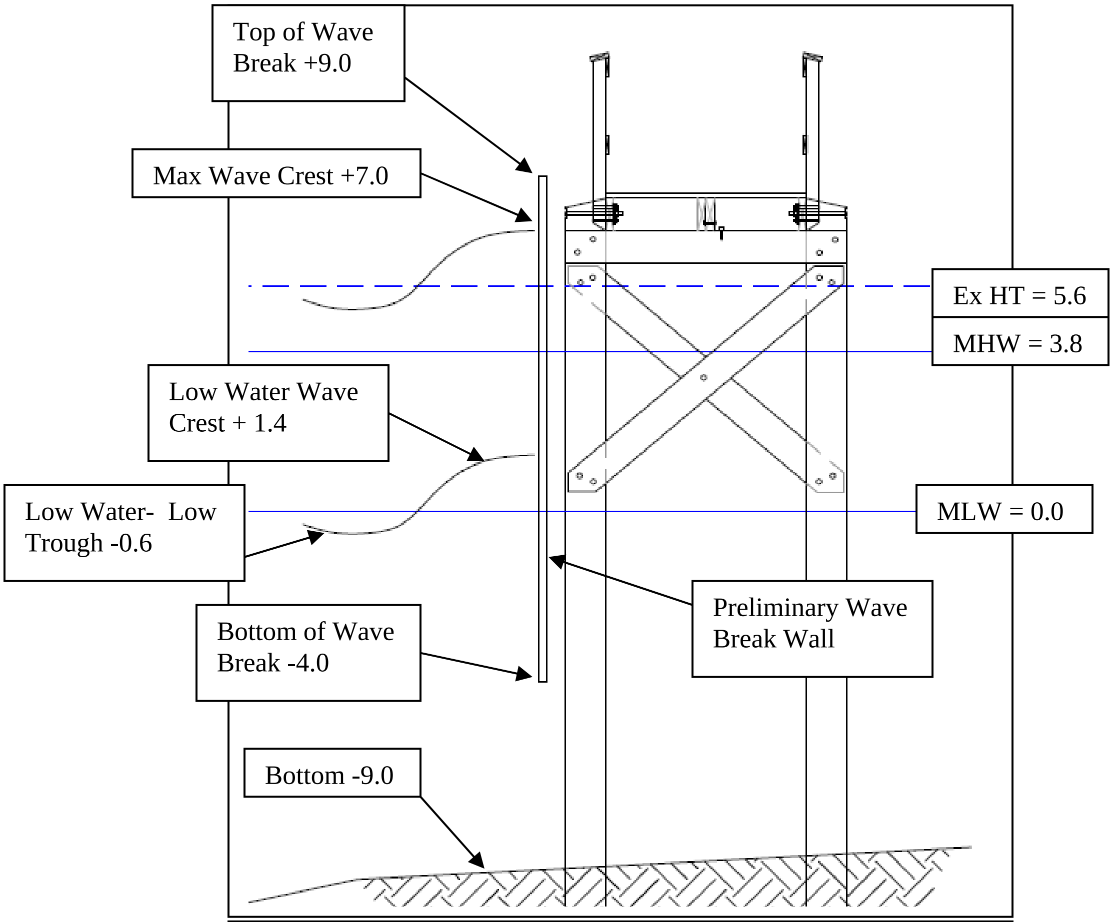

Figure 9: Preliminary Section of Wave-Break

To determine the lateral loads generated by waves on vertical fixed wave walls, which do not extend all the way to the bottom, we recommend consulting the COE Coastal Engineering Manual (EM 1110-2-1100, Part VI), Figure 13 is the basic description of the condition that we will be investigating for the wave break wall above.

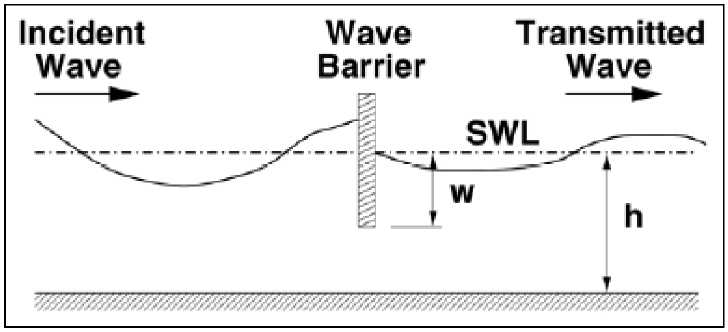

Figure VI-5-61. Wave barrier definition sketch

Figure 10: Taken from COE Coastal Engineering Manual Part VI, Figure VI-5-61; Diagram of fixed, vertical wave wall, open at bottom

The following procedures were developed and tank tested for conditions where w/h = 0.4, 0.5, 0.6 and 0.7, in 3 meters (9.84 ft) of water depth. The following Formulae were taken from the Coastal Engineering Manual Formulas VI-5-163, 164 & 165:

$$F_o = \rho g \, H_{mo} \, \frac{\sinh k_p h}{k_p \cosh k_p h}$$ (Formula 16)

where

ρ = water density

g = gravity

H_{mo} = incident significant wave height

k_p = wave number associated with the spectral peak period, T_p

h = water depth at the barrier

Where "Fo" is the Significant force per unit width of the Vertical Wall

In Formula 16 the "wave number" k_p is found by the following formula:

$k_p = 2\pi / L$ (Formula 17)

$$F_{mo} = F_o \left(w/h \right)^{0.386 \left(h/L_p \right)^{-0.7}}$$

where

F_{mo} = significant force per unit width of barrier

F_o = significant force per unit width of vertical wall (Equation VI-5-163)

w = barrier penetration depth

h = water depth

L_p = local wavelength associated with the peak spectral period, T_p

(Formula 18)

Note: The term "h" in this set of formulas is the same as the term "d" in Part 1, and should not be confused

Once obtaining F_{mo} from figure 18, F_{design} is calculated using formula 19. It is worth noting that these formulas were tank tested, and thus should serve as a good basis for designing fixed wave break walls, however tank testing does not always emulate actual storm conditions. In the writer's experience the horizontal load forces produced by this method would be suitable for some floating breakwaters, which tend to ride over portions of the wave, especially when longer periods occur. However, in more exposed wave climates – and fixed breakwaters the designer may be well advised to factor in an abundance of caution and add an additional factor of 2.0 to the F_{design} loads from Formula 19 for the later cases structures.

$$F_{design} = 1.8 F_{mo}$$

(Formula 19)

To produce the above calculation series one will also need the formula for wave length which is given here as Formula 19.

$$Ls := \frac{g \cdot Ts^2}{2 \cdot \pi} \cdot \sqrt{\tanh\left(\frac{4 \cdot \pi^2 \cdot d}{Ts^2 \cdot g}\right)}$$

(Formula 20)

Variable List:

Ls = Significant wave Length (Distance between Wave Crests – (feet) (also referred to as Ld in this section of the CEM)

g = Gravity = 32.2 feet per second squared

Ts = Significant Wave Period (Seconds)

d = Water Depth (Equivalent Still-Water in feet)

Note that Significant Wave Height does not enter into the formula; the wave length is determined only by the wave Period (Ts) and water depth (d or h). Our first step in determining the wave load on the wave wall is to find wave length. The easiest way to do this is to enter the formula into a spreadsheet, and use the mathematical formulas built into those programs, or use a packaged programs such as MathCad™. In this case since MathCad™ shows the progress of calculations Figure 11 shows an example of how to set up the formula. This will be done for the normal design condition, then for the hurricane condition. The design wave (Hs) was calculated earlier to be 2.0 feet, with a period (Ts) of 2.0 seconds.

(Note that Ls from the SPM is the same as Lp in the CPM)

Wave Length - Normal Tidal Range - non hurricane

$$d1 := 30 \qquad Hs1 := 2.0 \qquad Ts1 := 2.0 \qquad g1 := 32.2$$

$$Ls1 := \left(\frac{g1 \cdot Ts1^2}{2 \cdot \pi}\right) \cdot \sqrt{\tanh\left(\frac{4 \cdot \pi^2 \cdot d1}{Ts1^2 \cdot g1}\right)} \qquad Ls1 = 20.499$$

Figure 11: Application of Formula 20 using MathCad™, for design wave of 2.0 ft & 2.0 seconds, and using variables listed above

Obviously, these results can also be obtained using hand calculations. As a matter of interest the hurricane wave with a period (Ts) of 2.6 seconds, and "d" of 40 feet to allow for the additional water depth from the storm surge would have a length (Ls) of 34.3 feet.

The next step in the process is to calculate the wave number "k_p" using Ls of 20.5 (rounded), and using Formula 17, which would compute as follows:

Kp = 2 x 3.1416/ 20.5 = 0.306

The next step is then to compute the horizontal force on the wave barrier as if it were a solid wall; this is accomplished by using Formula 16. As in the steps above, because of the complexity of these calculations it is a lot easier to use either an electronic spreadsheet or a packaged software program. Here again we will demonstrate using MathCad™, inserting the variables as follows in Figure 12:

Fo - Significant Wave Force per Unit Width of a full depth Vertical Wall

$$\rho 1 := \frac{64}{62.4} \quad \rho 1 = 1.026 \quad Hmo1 := 2.0 \quad kp = 0.307 \quad h1 := 5.6 + 9.0$$

$$g1 = 32.2 \qquad h1 = 14.6 \quad \text{feet}$$

$$Fo2 := \rho 1 \cdot g1 \cdot Hmo1 \cdot \frac{\sinh(kp) \cdot h1}{kp \cdot \cosh(kp) \cdot h1} \qquad Fo2 = 64.058 \quad \text{pounds per lin foot}$$

Figure 12: Application of Formula 16 to calculate horizontal wave force

Note that the density of salt water is taken as 64 (salt water)/62.4 (fresh water) pounds per cubic foot, H_{mo} and k_p have been taken from prior calculations, and "h" is the sum of the Extreme Tide (without storm surge) of 5.6 feet and the water depth measured from the Still Water Elevation (SWE) to the bottom under the breakwater 9.0 feet for a total "h1" of 14.6 feet. The computed depth is then 64.058 pounds per lineal foot of wall.

Note: If this were a fresh water project the weight of water would be taken as 1.0 (62.4 pounds per cubic foot).

The next step in this series is to apply the Fo1 horizontal load to Formula 18, to adjust the horizontal load to fit the actual wave wall that we have designed. The wall in the figure has a top elevation of +9.0 and a bottom elevation of -4.0, for a total height of 13 feet. The +9.0 elevation was derived from the Extreme High predicted tide with a SWE of +5.6 feet and factoring in the 2.0 foot design wave. For simplicity of calculation it should be noted that the crest of a typical non-breaking wave rises about 0.7 times the wave height Hs, thus the crest of a 2.0 foot wave would rise about 1.4 feet above the SWE, or about elevation +7.0 as shown in Figure 12. When waves hit a vertical wall they tend to "run up" the surface, and some more significant waves might actually be higher. The design shown was based on experience, i.e. raising the top of the wave wall to an elevation somewhat higher than the deck elevation, thus protecting the deck and stringers from the full horizontal force of a possible overtopping wave.

To continue with the horizontal force calculations and using the peak tidal condition SWE, we are ready to compute "w" in Formula 18. The variable "w" in the example (or "w1" in Figure 16 below), is the sum of the Extreme High Tide from Figure 12, which is +5.6 feet added to the depth below MLW to the bottom of the proposed wall, which is -4.0 feet. The "w" then equals 9.6 feet. Thus using the other variable from the preceding the MathCad™ version of the Formula 18 would appear as follows:

Fmo = Significant Force per unit width of Breakwater

$$w1 := 5.6 + 4.0 \quad w1 = 9.6 \quad \text{feet}$$

$$Fmo2 := Fo2 \cdot \left(\frac{w1}{h1}\right)^{0.386 \cdot \left(\frac{h1}{Ls1}\right)^{-0.7}} \qquad Fmo2 = 52.173 \quad \text{pounds per lin foot}$$

Figure 13: Application of Formula 18 to calculate the net horizontal wave force on the actual wave wall

The horizontal force "F_{mo}" calculates to about 52 pounds per lineal foot of wall for a two foot wave, from this Goya (1985) suggested adding a factor of safety of 1.8 to allow for larger waves within the spectrum. In addition to Goya's adjustment we also recommend an additional factor of safety of 2.0 to account for the any of several "unknowns" that can occur during a storm. Thus the final step is to use Formula 19 to determine the F_{design} actual design load:

> $F_{design} = 1.8 \times 2.0 \times F_{mo} = 1.8 \times 2.0 \times 52.173 = 191.42$ or 192 pounds per lineal foot of wall (this should be rounded off to <u>200 pounds per lineal foot</u>).

Experience dictates that the design wave would focus most of its energy of the very small percentage of the vertical surface. Typically 80% of the wave's energy is focused in the top 20% of the wave crest, which when factoring in the highly variable tidal elevation could occur at virtually any location on the wall from the low water line to the extreme high water line. Thus when doing the bending analysis on the vertical wave wall members it is good practice to assume several horizontal load locations in the vertical plane and to consider them as a point load condition. Certainly these should be the locations that by observation would cause the highest bending stresses, such as mid span or any cantilevered extensions.

The next step in the wave wall design is to position the horizontal wale members that the vertical wave wall planks will be attached to, then to size these members, as well as the vertical planks and determine the bolting requirements. Positioning the wales for practical installation purposes would be as follows: The top wale is shown about even with the pile cap, which is a flexible location; that is to say, we can shift the location of this wale up or down to accommodate the loading conditions to gain the most efficient vertical plank size. The lower wale is not as easy, and its placement is the most critical; typically it requires a workman in the water (dock builder in a wet or dry suit or diver) to install. This is because the wale should be a little above the MLW elevation and positioned so that an in-water diver can install the bolts, preferably working without an air hat" and thus be able to move much more freely and still see what he is doing. That is to say, working at water level is much easier from

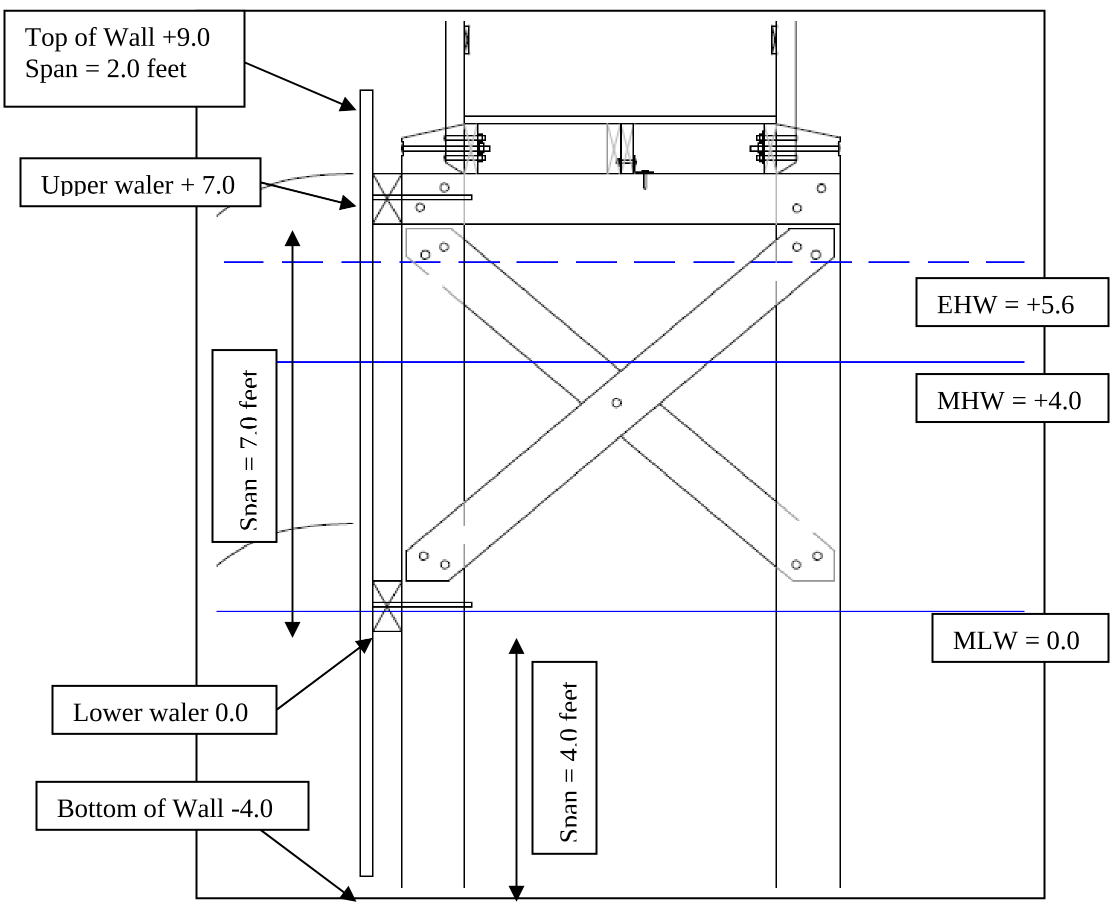

Figure 14: Placement of Wales on Wave Wall for load calculations

an installation standpoint than putting the wale so deep that the diver must work underwater. Components that are totally underwater also make subsequent inspection by the owner much more difficult too.

In some areas where tidal change is very small – such as from Florida to the Caribbean, where the total range can be in the one to two foot range – placement of the lower wale below MLW may be unavoidable in order to get enough separation.

In the case of our study design (Figure 14), the range of possible loading locations is shown - by observation we can see that one obvious location of maximum bending moment would be the mid-span location between the wales; the second location would be the two foot cantilevered section above the upper wale. There is also some concern for the rather long four foot cantilevered span below the lower wale – but since this cantilever is below the MLW line, and 80% of a wave's energy is focused in the top 20% of the wave – the residual energy in the lowest portion of the wave would be minimal. Further, even if significant waves should occur at an abnormally low time (below MLW), the crest of the wave would still not focus its energy on the lower portions of the cantilever. Thus it is typically assumed that this area is not subjected to heavy loadings (See Figure 17).

If we assume that the maximum wave force that is 80% of the wave energy is focused in an area that is the upper 20% of the wave height then the wave force would be 80% of 200 plf or 160 plf. This force would be focused on an area that is 20% of 2.0 feet of the wave height, or 0.4 feet centered at the wave crest location – which we will place at the mid span of the two wales. For simplicity we will consider this as a 160 plf point load on the wall planks, and spread the rest of the load uniformly over the wave height. This is not technically correct in the finite sense, but it makes the calculations easier, and considering that we rounded the wave load up almost 20 plf, it would allow a little leeway to simplify the application. The rest of this exercise is simply plugging the forces into a bending moment formula which can be obtained from almost any engineering manual. For purposes of this study we plugged the loads into an inexpensive beam calculation software program and obtained the following results.

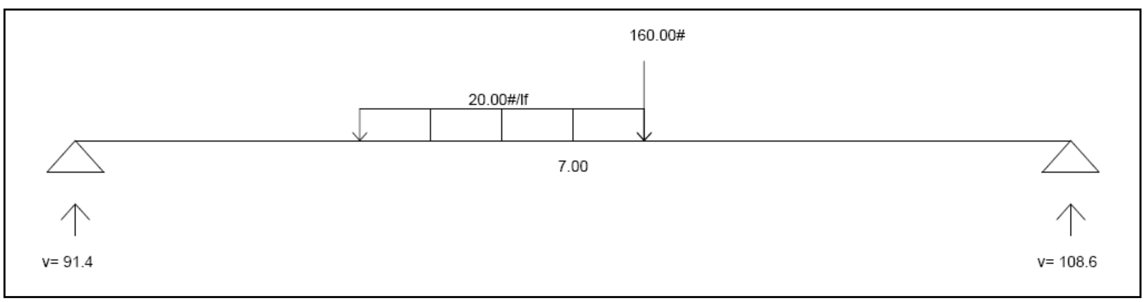

Figure 15: Load diagram for one horizontal foot of wave break, with wave energy focused about mid-span – using beam calculation software.

Using this software the maximum bending moment produced was about 3900 inch pounds, with a maximum deflection of 0.55 inches for a 2x plank; or 0.12 inches if a 3x plank were used. Likewise applying the same wave to the two foot cantilever the following load diagram was generated:

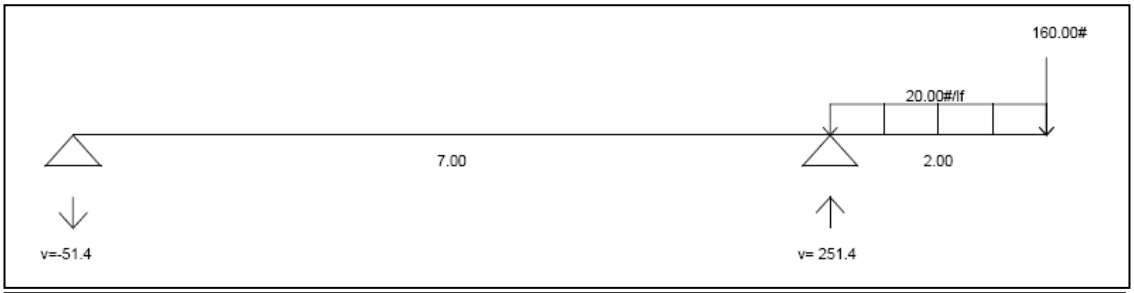

Figure 16: Load diagram for one foot of wave break, with wave energy focused at the top overhang – using beam calculation software.

In this case the maximum moment generated was 4320 inch pounds (slightly higher than the mid span load), with a maximum deflection of 0.82” if a 2x member were used; and 0.34” if a 3x vertical member were used. In this case it can easily be observed that the cantilevered portion of the plank should be the controlling factor, and although the available Section Modulus of 12.5 for a 12” wide section of 3x plank would only reach 345 psi of bending stress, the predicted deflection of 0.34” in 24” (1:70) is pushing the limit of a treated wood plank’s elasticity – thus we will opt for the nominal 3x wood plank.

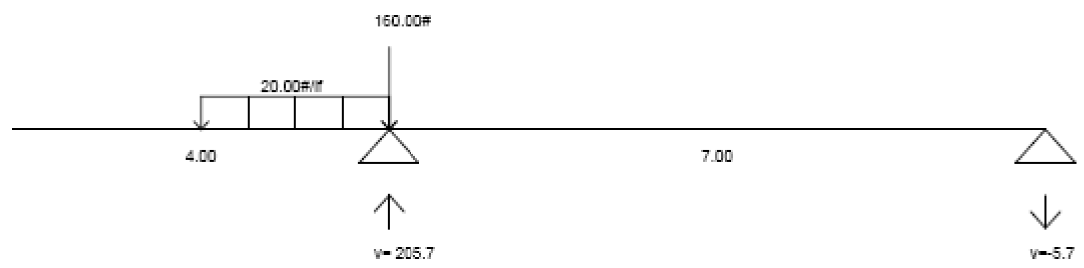

Figure 17: Load diagram for one foot of wave break, with wave energy focused at the top of overhang – using beam calculation software. This represents an extreme low water (ELW) case where the astronomical low tide might be a foot or more below MLW, and the crest of the wave would focus its energy on the lowest beam.

The next phase of the design is to size the horizontal beams (or wales) that the vertical wave wall planks will be attached to. From the above examples we can see that the maximum load on the upper beam would be 251.4 pounds per lineal foot of beam (Figure 16). The design load for the lower beam would be the case shown in Figure 17 of 205.7 pounds per lineal foot of beam. Referring back to Figure 2, the pier layout plan, from observation we can see that the maximum span of the breakwater between pile bents is 14 feet, thus we will use this span in our beam calculations for the wave wall. We will use the worst case of a uniform wave striking the wave wall perpendicular to its alignment, which would produce the maximum impact. However practical experience in these cases is that such events are extremely rare. Normally the wave will strike the wall at some angle other than 90^o, albeit the differential angle might be very small. It is beyond the scope of this document to go into the details of the reductions in wave forces that occur as the angle from the perpendicular increases, but let it suffice to say, that in some cases the reductions become significant and may be worth investigating. Another factor to consider, especially in areas of limit fetch, is that non-breaking waves are rarely uniform in shape, thus the full force of the wave does not strike the face of the wave all at once, but rather is spread over a period of one or more seconds. Taking these two factors into consideration, the actual forces at any given moment during a storm will typically be lower than those calculated here. Be that as it may, because the marine environment is usually less than predictable, these force reductions are usually "banked" as uncalculated factors of safety. Thus considering the foregoing - using the maximum loads of 251.4 and 205.7 (rounded to 255 plf and 210 plf) the maximum bending moments would appear thus in Figures 18 and 19.

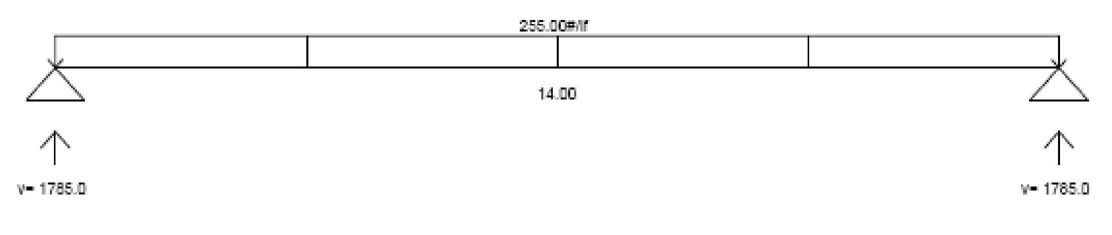

Figure 18: Uniform wave load on upper wave wall beam (wale) of 255#/ lineal foot, producing maximum bending moment of 74,970"#, and 0.52" of deflection bending assuming the use of a single 8 x 8 horizontal beam.

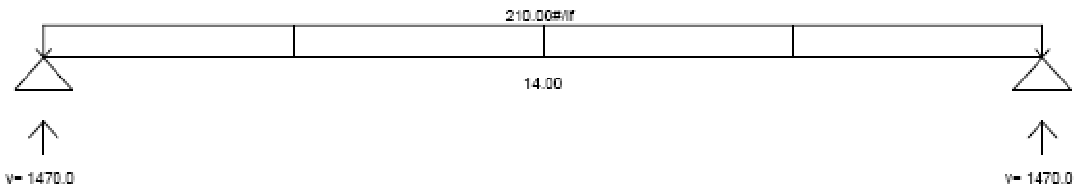

Figure 19: Uniform wave load on Lower wave wall beam (wale) of 210#/ lineal foot, producing maximum bending moment of 61,740"#, and 0.4297" of deflection bending assuming the use of a single 8 x 8 horizontal beam.

Taking the upper beam and dividing the allowable unit stress of 1200 psi into the maximum bending moment of 74,970"#, we obtain a required minimum section modulus of 62.47 for a timber section. Inserting the dimensions of a rough treated 8x8 (7.5" x 7.5"), into the Section Modulus (S) Formulas 9 and 10 we obtain an S for the 8x8 of 70.31, which is suitable for the requirement. There are two other factors that must be considered, there is an allowable reduction of 33% to 25% allowed for short term wave loads, offset to some degree by a loss in section modulus that will occur over time due to weathering of the wood which is higher in the tidal zone because of continual wetting and drying. It should also be noted that the maximum load condition would only occur at the extreme peak of a rare astronomical tide, and then typically only lasts short time. Taking all of the above factors into consideration, we will consider the available S of 70.31 to be a reasonably good choice to meet the long term design requirement. For purposes of simplicity of construction we will make both the upper and lower beams 8x8s.

Checking the maximum lateral loads generated on the wall, from Figure 21 we find that the maximum load for one half of the pile connection is 1912#, thus by multiplying this figure by 2 the maximum lateral load on the pile would be 3824# focused at the top beam. From reviewing Tables 3a and 3b, we find that the X-Braced design is capable of withstanding up to 7700# of lateral load, thus we consider the X-Braced section as detailed adequate for the wave wall.

Course Recap:

In Part I of this course we have learned the basic steps for designing and sizing the basic components of a pier and wave break structure as they would be used in Recreational and Light Commercial Piers and other equivalent structures of Maritime usage. Upon completing this course the Engineer should have a basic understanding of the six most important components of design for these basic level marine structures, these are:

1. Overview & Basic Layout of a Facility
2. Basic Assumptions, Design Loads & Formulae
3. Design of the Basic Pier Cross Section
    a. Deck
    b. Stringers
    c. Pile Caps
4. Soil Conditions for Pile Supports
    a. Free standing pile support
5. Lateral Load Considerations & Design

a. X-Braced Pile Supports

b. Lateral Loads on Stringers

6. Design of a "Wave Break" Wall

a. Design of Wale Section

b. Design of Plank Wall Section

Once the Engineer has developed an understanding of these components, he or she should be in a position to go on to study other levels of maritime design. Future Continuing Education Courses will go on to undertake other areas of design, such as layout of maritime facilities, design of floating docks, wave attenuation, coastal revetments, and bulkheads, as well as more advanced subjects such as design for storm survivability in more exposed waters.

APPENDICIES

Appendix 1

Wood Connection Design

Baseline Data X Brace - Single Shear Load acting Parallel to Thinner Piece & at 45 deg to Thicker Pile

Bolt Dia $D1 := 1.375$ inch dia

SG for wood $G1 := .50$

Fy for Bolt $Fyb := 45000$ psi

Thickness Main Member $tm := 11.5$ inches

Angle of Main conn from parallel $\theta m := 90$ degrees

Fes for Wood(Fe Parl for bin DF-L) $Fe1 := 11200 \cdot G1$ $Fe1 = 5600$

Fem for Wood (Fe perp for bolt in DF-L) $Fe2 := 6100\left(G1^{1.45}\right) \cdot D1^{-0.5}$ $Fe2 = 1904.079$

Assume Load of thinner member acting at 45 degrees to pile, therefore thinner member is parallel load and thicker member (pile) is thicker member at 45 degrees

Feθ gives Fe for wood at angle of load to grain θ $\theta 3 := 45$

$$Fe\theta := \frac{Fe1 \cdot Fe2}{\left[Fe1 \cdot (\sin(\theta 3))^2\right] + \left[Fe2 \cdot (\cos(\theta 3))^2\right]} \qquad Fe\theta = 2328.1$$

Thickness of Secondary Member $ts := 2.5$ inches

Angle of Sec conn from parallel $\theta s := 0$ degrees

$$Re1 := \frac{Fe\theta}{Fe1} \qquad Re1 = 0.416$$

$Re2 := 1 + Re1$ $Re2 = 1.416$

$Re3 := 2 + Re1$ $Re3 = 2.416$

$$Rt1 := \frac{tm}{ts} \qquad Rt1 = 4.6$$

$Rt2 := 1 + Rt1$ $Rt2 = 5.6$

$Rt3 := 1 + Rt1 + Rt1^2$ $Rt3 = 26.76$

$\theta 1 := \theta m$ $\theta 1 = 90$ Note: $\theta 1$ is the larger of θm and θs

$$K\theta 1 := 1 + \frac{\theta 1}{360} \qquad K\theta 1 = 1.25$$

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$$k1 := \frac{\sqrt{Rel + 2 \cdot Rel^{2} \cdot \left(1 + Rtl + Rtl^{2}\right) + \left(Rtl^{2} \cdot Rel^{3}\right)} - Rel \cdot (1 + Rtl)}{(1 + Rel)} \qquad k1 = 0.718$$

$$k2 := -1 + \sqrt{2 \cdot (1 + Rel) + \frac{2Fyb \cdot (1 + 2Rel) \cdot D1^{2}}{3 \cdot Fe2 \cdot tm^{2}}} \qquad k2 = 0.801$$

$$k3 := -1 + \sqrt{\frac{2 \cdot (1 + Rel)}{Rel} + \frac{2 \cdot Fyb \cdot (2 + Rel) \cdot D1^{2}}{3 \cdot Fe2 \cdot ts^{2}}} \qquad k3 = 3.281$$

Yield Limit Equations

Mode Im (NDS eq 11.3-1) $Z1 := \frac{D1 \cdot tm \cdot Fe\theta}{4 \cdot K\theta 1}$ $Z1 = 7362.617$ lb

Mode Is (NDS eq 11.3-2 $Z2 := \frac{D1 \cdot ts \cdot Fe1}{4K\theta 1}$ $Z2 = 3850$ lb

Mode II (NDS eq 11.3-3 $Z3 := \frac{k1 \cdot D1 \cdot ts \cdot Fe1}{3.6 \cdot K\theta 1}$ $Z3 = 3071.373$ lb

Mode IIIm (NDS eq 11.3-4) $Z4 := \frac{k2 \cdot D1 \cdot tm \cdot Fe\theta}{3.2(1 + 2 \cdot Rel) \cdot K\theta 1}$ $Z4 = 4025.63$ lb

Mode IIIs (NDS eq 11.3-5) $Z5 := \frac{k3 \cdot D1 \cdot ts \cdot Fe\theta}{3.2 \cdot (2 + Rel) \cdot K\theta 1}$ $Z5 = 2717.076$ lb

Mode IV (NDS eq 11.3-6) $Z6 := \frac{D1^{2}}{3.2 \cdot K\theta 1} \cdot \sqrt{\frac{2 \cdot Fe\theta \cdot Fyb}{3 \cdot (1 + Rel)}}$ $Z6 = 3319.834$ lb

Z = lowest of the above values

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Wood Connection Design

Baseline Data Pile to Stringer Single Shear Connection - Load Parallel to Pile & perp to thinner Stringer

Bolt Dia $D1 := 1.0$ inch dia

SG for wood $G1 := .50$

Fy for Bolt $Fyb := 45000$ psi

Thickness Main Member $tm := 11.5$ inches

Angle of Main conn from parallel $\theta m := 0$ degrees

Fes for Wood(Fe Parl for bin DF-L) $Fe1 := 11200 \cdot G1$ $Fe1 = 5600$

Fem for Wood (Fe perp for bolt in DF-L) $Fe2 := 6100\left(G1^{1.45}\right) \cdot D1^{-0.5}$ $Fe2 = 2232.731$

Feθ gives Fe for wood at angle of load to grain θ $\theta 3 := 45$

$$Fe\theta := \frac{Fe1 \cdot Fe2}{\left[Fe1 \cdot (\sin(\theta 3))^2\right] + \left[Fe2 \cdot (\cos(\theta 3))^2\right]} \qquad Fe\theta = 2676.93$$

Thickness of Secondary Member $ts := 3.5$ inches

Angle of Sec conn from parallel $\theta s := 90$ degrees

$Re1 := \frac{Fe2}{Fe1}$ $Re1 = 0.399$

$Re2 := 1 + Re1$ $Re2 = 1.399$

$Re3 := 2 + Re1$ $Re3 = 2.399$

$Rt1 := \frac{tm}{ts}$ $Rt1 = 3.286$

$Rt2 := 1 + Rt1$ $Rt2 = 4.286$

$Rt3 := 1 + Rt1 + Rt1^2$ $Rt3 = 15.082$

$\theta 1 := \theta s$ $\theta 1 = 90$ Note: $\theta 1$ is the larger of θm and θs

$K\theta 1 := 1 + \frac{\theta 1}{360}$ $K\theta 1 = 1.25$

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$$k1 := \frac{\sqrt{Re1 + 2\cdot Re1^2\cdot\left(1 + Rt1 + Rt1^2\right) + \left(Rt1^2\cdot Re1^3\right)} - Re1\cdot(1 + Rt1)}{(1 + Re1)} \qquad k1 = 0.512$$

$$k2 := -1 + \sqrt{2\cdot(1 + Re1) + \frac{2Fyb\cdot(1 + 2Re1)\cdot D1^2}{3\cdot Fe2\cdot tm^2}} \qquad k2 = 0.726$$

$$k3 := -1 + \sqrt{\frac{2\cdot(1 + Re1)}{Re1} + \frac{2\cdot Fyb\cdot(2 + Re1)\cdot D1^2}{3\cdot Fe2\cdot ts^2}} \qquad k3 = 2.106$$

Yield Limit Equations

Mode Im (NDS eq 11.3-1) $$Z1 := \frac{D1\cdot tm\cdot Fe2}{4\cdot K\theta 1} \qquad Z1 = 5135.281\ \text{lb}$$

Mode Is (NDS eq 11.3-2 $$Z2 := \frac{D1\cdot ts\cdot Fe1}{4K\theta 1} \qquad Z2 = 3920\ \text{lb}$$

Mode II (NDS eq 11.3-3 $$Z3 := \frac{k1\cdot D1\cdot ts\cdot Fe1}{3.6\cdot K\theta 1} \qquad Z3 = 2228.664\ \text{lb}$$

Mode IIIm (NDS eq 11.3-4) $$Z4 := \frac{k2\cdot D1\cdot tm\cdot Fe2}{3.2(1 + 2\cdot Re1)\cdot K\theta 1} \qquad Z4 = 2593.752\ \text{lb}$$

Mode IIIs (NDS eq 11.3-5) $$Z5 := \frac{k3\cdot D1\cdot ts\cdot Fe2}{3.2\cdot(2 + Re1)\cdot K\theta 1} \qquad Z5 = 1715.255\ \text{lb}$$

Mode IV (NDS eq 11.3-6) $$Z6 := \frac{D1^2}{3.2\cdot K\theta 1}\cdot\sqrt{\frac{2\cdot Fe2\cdot Fyb}{3\cdot(1 + Re1)}} \qquad Z6 = 1730.04\ \text{lb}$$

Z = lowest of the above values

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Appendix 3

Wood Connection Design

__Baseline Data__ Pile Cap - Double Shear Load acting Parallel to Pile & Perp to Split Cap (Thinner)

Bolt Dia $D1 := 1.0$ inch dia

SG for wood $G1 := .50$

Fy for Bolt $Fyb := 45000$ psi

Thickness Main Member $tm := 11.5$ inches

Angle of Main conn from parallel $\theta m := 0$ degrees

Fes for Wood(Fe Parl for bin DF-L) $Fe1 := 11200 \cdot G1$ $Fe1 = 5600$

Fem for Wood (Fe perp for bolt in DF-L) $Fe2 := 6100\left(G1^{1.45}\right) \cdot D1^{-0.5}$ $Fe2 = 2232.731$

Feθ gives Fe for wood at angle of load to grain θ $\theta 3 := 45$

$$Fe\theta := \frac{Fe1 \cdot Fe2}{\left[Fe1 \cdot (\sin(\theta 3))^2\right] + \left[Fe2 \cdot (\cos(\theta 3))^2\right]} \qquad Fe\theta = 2676.93$$

Thickness of Secondary Member $ts := 2.5$ inches

Angle of Sec conn from parallel $\theta s := 90$ degrees

$Re1 := \frac{Fe2}{Fe1}$ $Re1 = 0.399$

$Re2 := 1 + Re1$ $Re2 = 1.399$

$Re3 := 2 + Re1$ $Re3 = 2.399$

$Rt1 := \frac{tm}{ts}$ $Rt1 = 4.6$

$Rt2 := 1 + Rt1$ $Rt2 = 5.6$

$Rt3 := 1 + Rt1 + Rt1^2$ $Rt3 = 26.76$

$\theta 1 := \theta s$ $\theta 1 = 90$ Note: θ1 is the larger of θm and θs

$K\theta 1 := 1 + \frac{\theta 1}{360}$ $K\theta 1 = 1.25$

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$$k1 := \frac{\sqrt{Re1 + 2\cdot Re1^2\cdot\left(1 + Rt1 + Rt1^2\right) + \left(Rt1^2\cdot Re1^3\right)} - Re1\cdot(1 + Rt1)}{(1 + Re1)} \qquad k1 = 0.692$$

$$k2 := -1 + \sqrt{2\cdot(1 + Re1) + \frac{2Fyb\cdot(1 + 2Re1)\cdot D1^2}{3\cdot Fe2\cdot tm^2}} \qquad k2 = 0.726$$

$$k3 := -1 + \sqrt{\frac{2\cdot(1 + Re1)}{Re1} + \frac{2\cdot Fyb\cdot(2 + Re1)\cdot D1^2}{3\cdot Fe2\cdot ts^2}} \qquad k3 = 2.489$$

Yield Limit Equations Double Shear Connections

Mode Im (NDS eq 11.3-1) $Z1 := \frac{D1\cdot tm\cdot Fe2}{4\cdot K\theta 1}$ $Z1 = 5135.281$ lb

Mode Is (NDS eq 11.3-2 $Z2 := \frac{2D1\cdot ts\cdot Fe1}{4K\theta 1}$ $Z2 = 5600$ lb

Mode IIIs (NDS eq 11.3-5) $Z5 := \frac{2k3\cdot D1\cdot ts\cdot Fe2}{3.2\cdot(2 + Re1)\cdot K\theta 1}$ $Z5 = 2895.971$ lb

Mode IV (NDS eq 11.3-6) $Z6 := \frac{2\,(D1)^2}{3.2\cdot K\theta 1}\cdot\sqrt{\frac{2\cdot Fe2\cdot Fyb}{3\cdot(1 + Re1)}}$ $Z6 = 3460.081$ lb

Z = lowest of the above values

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