Non-Linear Dynamics of Caisson Well-Protectors During Hurricane Andrew

Report to U.S. Minerals Management Service

Herndon, Virginia

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August 1997

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1.0 INTRODUCTION

The Gulf of Mexico is home to thousands of offshore structures. In the early stages of offshore development, most installations were large drilling and production platforms. Later, as fields matured, and energy companies decided to produce from small fields, smaller structures were designed to support only a few wells, without any drilling or production equipment. Hydrocarbons are piped from these "minimal structures" to larger platforms for processing. "Minimal structures" don't have a strict definition; however, three types of structures usually fall in this category. They are: 1) caissons, 2) braced caissons, and 3) tripods (listed in order of size from smallest to largest).
This study focuses only on caissons, which consist of a driven pile, from 3 to 8 feet in diameter, that support a maximum of four wells (most caissons support only one well).

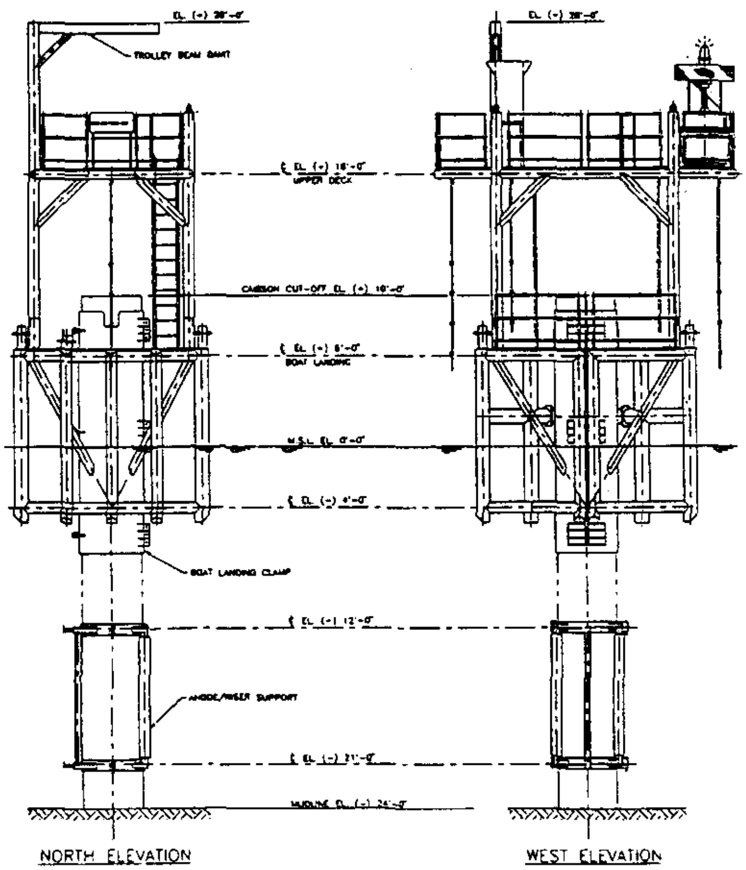


Figure 1
Typical Caisson Elevation

They usually have only one deck, used for maintenance, and a boat landing. Some of the larger caissons can have a helideck and some test equipment. Figure 1 is an elevation of a typical 48 inch diameter caisson. Note that the only equipment it supports is a small davit, and a navigation light/horn.

2.0 BACKGROUND

In the aftermath of Hurricane Andrew, there was an urgent need to re-assess the capacity of minimal structures in the Gulf of Mexico. Many of the structures damaged were near-new, and were designed for a storm the size of Andrew. Many structural analysts believed that design criteria needed to be updated, a lengthy and expensive process.

This study focuses on caissons that were in close proximity to the path of Hurricane Andrew (within 50 miles of the storm track). Hundreds of caissons were rendered inoperable in this area. Damages range from a few degrees of lean for some structures, to complete toppling for others.

3.0 PROBLEM STATEMENT

The goal of this study is to determine, if possible, the role that dynamics played in the failure of many of the caissons subjected to Hurricane Andrew, and specifically, if failure of individual caissons was due to dynamic effects alone, and not just simple overloading. Previous studies [3, 9] did not explicitly consider dynamics in evaluations of caisson performance during hurricane Andrew. A secondary goal of this project is to develop a simple tool to analyze caissons for dynamic effects using Newmark's method. [14] This is intended to be a quick check for structures that may be overloaded due to dynamic effects.

The first structure studied was located in Block 10-South Pelto. This 36 inch diameter caisson suffered an 11 degree lean after the storm passed. Specific attention will be paid to this caisson because it failed. If this caisson was able to withstand the maximum static wave forces generated at its location, it must have failed due to dynamic effects.

The second structure studied was located in Block 52 - South Timbalier. It was a 96 inch diameter "coke bottle" caisson standing in sixty feet of water. It was severed five feet above the mudline.

The third structure studied was located on Block 120-Ship Shoal. It stands in 40 feet of water and has a diameter of 4 feet. This structure was toppled by the storm, but the data does not give its failure mode.

4.0 LITERATURE REVIEW.

The structural data used in this project was taken by Barnett and Casbarian, following Hurricane Andrew, in 1994. Under contract to the Minerals Management Service (MMS), Barnett and Casbarian collected data for thousands of caissons in the Gulf of Mexico, including the condition of the caissons after the storm event.

Starting in the 1970's designers began to adopt a methodology in which a caisson was designed for an extreme static wave load, and then resized for effects of dynamics, using a Dynamic Amplification Factor (DAF). Hong suggested a DAF of 1.4 be used. [8] The API adopted this procedure for its guidelines. [4]

After model testing, in 1996. Kriebel et al. determined that the API guidelines overpredicted the water particle velocities and forces by 10% to 15% in most cases. However, random breaking waves sometimes generated forces that were 1.5 to 2.2 times as large as were analytically predicted. [9] According to Kriebel, "For these and other breaking waves, measured wave loads were strongly effected by dynamic amplification effects due to ringing of the structure following wave impact." In this case, Dynamic Amplification Factors (DAF's) ranged from 1.15, for five times the caisson's natural period, to very large values, at resonance (no values larger than 1.15 were measured, only predicted). [9]

To capture second order effects, and resonance, some type of time dependent approach needs to be used. Recently, time-history analysis using the finite-difference method has been accepted as the preferred method of dynamic analysis. The data from the time-history analysis will be compared with the actual condition of the three caissons, as recorded by Barnett and Casbarian, to evaluate the analytical model.

4.1 Governing Equations Of Dynamics

Single degree of freedom systems are often represented as a spring, dashpot, and mass (Figure 2).

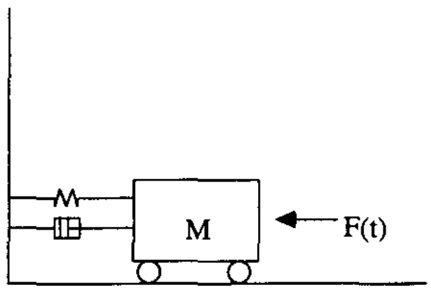


Figure 2
SDOF Oscillator

When the mass is perturbed, it oscillates back and forth at it's natural period, and if a damper (dashpot) is present, the oscillations die off until the mass returns to its equilibrium position. The equation describing this motion is Newton's second law:

$$\sum f = ma \qquad [10]$$

Summing forces and differentiating twice with respect to time gives,

$$mx = f(t) - cx - kx \qquad [10]$$

Rearranging terms yields a familiar second order differential equation,

$$mx + cx + kx = f(t) \qquad [10]$$

Where:
m = Mass of oscillator
c = Damping coefficient
k = Spring stiffness constant
f(t) = Arbitrary force

This equation has a solution of the form $x = A\sin(\omega t) + B\cos(\omega t)$

The natural period of this system depends only on k and m. It can be shown that for a SDOF system, the natural period, denoted T_n, is:

$$T_n = 2\cdot\pi\cdot\sqrt{\frac{M}{K}} \qquad [10]$$

When the force perturbing the system is periodic, its response can take many different forms. When the period of the forcing function is close to the natural period of the structure, its motion starts to grow exaggerated. This phenomenon, called dynamic amplification (DA), can lead to structural overload. In the case of offshore oil platforms, periodic wave forces can cause overloading, leading to brace buckling and yielding of structural members. At this point, the stiffness of the system can change drastically. Non-linear analysis dictates that at each time step in the analysis, the stiffness must be reevaluated, and the equation must be solved again. This is necessary for complex, multi-degree of freedom systems, but is not necessary for caissons, as detailed in section 4.2.

4.2 Choice Of Single Degree Of Freedom System

A caisson well protector very closely resembles the classic "mass on a flexible rod" system analyzed by students worldwide. The bending of this rod is the system's only degree of freedom; it can be analyzed in a very simple manner using software that is readily available. The properties of the system--such as mass, stiffness, and strength--can be varied easily, and the effects of these changes are readily interpreted. The dynamic behavior of a single degree of freedom (SDOF) system can be well-captured using Newmark's constant acceleration method. [14] For these reasons, a SDOF system was chosen for this analysis.

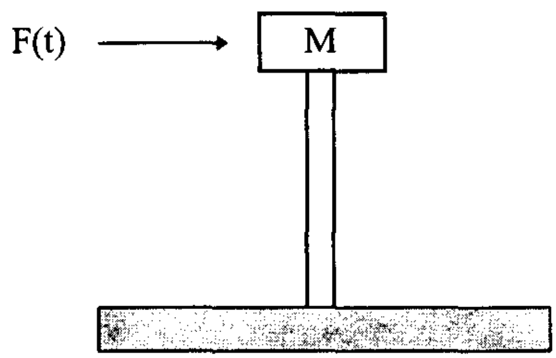


Figure 3
SDOF Structural Model

4.3 Choice of EPP System

For the non-linear portion of the analysis, it was decided to use an elasto-perfectly plastic system. That is to say, once the system reaches a certain load, it loses its stiffness. This is a good model for this structure, because once the ultimate plastic moment is reached, the structure forms a plastic hinge.

4.4 Ultimate Limit State Condition

Many oil platforms have failed in hurricanes, at least 200 major failed due to Hurricane Andrew – even more minimal structures failed. Most importantly, the failure -- or ultimate limit state condition -- for each type of offshore structure needs to be defined. In this study, failure is defined in two ways. Total failure is defined as the point at which the structure has been deflected so much that it can no longer support its vertical gravity loads, and collapses. The second type of failure is a loss of serviceability. Caissons are designed to produce oil or gas; if they are not producing, then they have failed. Caissons that are deformed plastically, so that they are left with a permanent set, may not be able to produce hydrocarbons because the well may not be able to be controlled or worked-over. This is termed serviceability failure. Total failure occurs well after the structure has ceased fulfilling its service requirements.

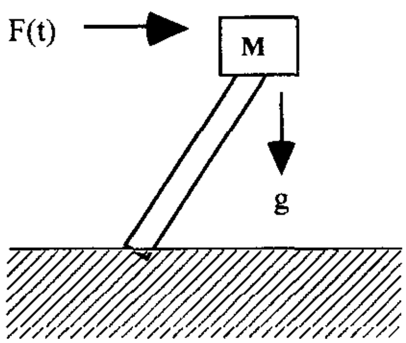


Figure 4
SDOF System at Failure

In order to understand the processes leading up to failure, a few terms need to be defined. When loads on a structure are small, it behaves linear-elastically. However, the structure has a defined yield point; when internal stresses reach the yield stress, the structure will start to deform plastically. A measure of the magnitude of these stresses is called the overload ratio:

$$\eta = \frac{f_{max}}{f_{yield}} \qquad [10]$$

Where F_y is the minimum force that causes yielding and F_{max} is the force applied to the structure.

In this study, the structures are assumed to behave elasto-perfectly plastically. That is to say, once the structures' yield point has been reached, it loses all its stiffness. This is a valid assumption for this simple study, when one considers that when overloaded dramatically, a caisson may buckle locally, or fail the soil due to cyclic degradation. Because of this zero post-yielding stiffness, the structure forms a failure mechanism, and starts to accelerate when it reaches an overload ration of one or greater. This phenomenon is displayed in Figure 5.

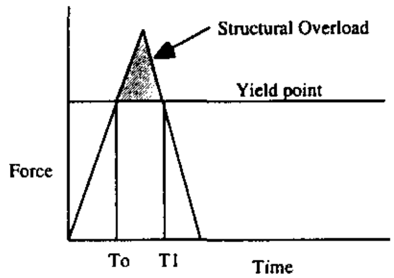


Figure 5
Wave Force Causing Overload

During the time $t_1 - t_0 = \Delta t$ the structure is deforming plastically. The length of this duration (Dt) determines the amount of plastic deformation.

The extent of a structure's plastic deformation can be described as an amount of displacement. However, usually yielding is defined as a ratio of the structures total deformation compared to its max. elastic deformation. This ratio is the structures ductility demand, and is represented below as μ.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \qquad [10]$$

Most structures are designed to fail in a ductile manner, and caissons nearly always fail in this way, because they do not have any joints to fracture, or braces to buckle.

4.5 Wave Loading – Method of Determining Wave Heights for the Analysis.

A sea-state spectrum for Hurricane Andrew, considered by many to be a 200 year storm, was used as a basis for determining the wave heights at the sites studied. Combined with hindcast significant waveheights generated by the Minerals Management Service, a water surface profile was developed that reflected the confused nature of the sea generated by the storm. Wave energy was concentrated in three periods: T= 14s, 12s, and 10s, with heights of 10, 15, and 10 ft. respectively. By superimposing these three waves it was possible to generate a representative water surface profile. The profile indicated that waves traveling out of phase with eachother would super-impose to form "packets" of three large waves (called freak waves by sailors). These large wave-heights were used in MATLAB to do the dynamic analysis.

Using the maximum wave heights for each site -- in all cases this was determined by the breaking criteria -- horizontal forces on the structure were determined, using depth stretched linear wave theory. This force was used to perform a static pushover analysis to determine the structure's ultimate moment capacity, based on the assumption that the maximum moment in a pile occurs 3-5 diameters below the mudline [4]. Applying the dynamic structural analysis will show if these wave forces are able to fail the structure.

4.6 Review of Linear Wave Theory

While it is thought that linear wave theory is not a very good predictor of the water surface condition generated by a hurricane, it is well known and documented that it is excellent for modeling the wave induced motions over submerged cylindrical members. The velocity and acceleration of particles in the water column can be expressed as:

$$u_x = \frac{pi \cdot H}{T} \cdot \frac{\cosh(k \cdot s)}{\sinh(k \cdot d)} \qquad a_x = \frac{2 \cdot pi^2 \cdot H}{T^2} \cdot \frac{\cosh(k \cdot s)}{\sinh(k \cdot d)}$$

This equation yields the kinematics near the still water level only. In order to extrapolate these values above or below the still water level, some form of stretching needs to be used. The most accurate way to accomplish this is the use of depth stretching, [1] where the SWL kinematics are stretched up to the instantaneous water surface, then brought down to the desired level. Analytically, this is accomplished by substituting $s = z + d$ into the above equation.

Once the kinematics have been determined, the forces on individual members can be calculated using the Morisson, O'Brien, Johnson and Schaaf equation:

$$F_{tot} = C_d \cdot \frac{\rho}{2} \cdot D \cdot L \cdot u_x \cdot \left(\left| U_x \right| \right) + C_m \cdot \rho \cdot V \cdot a_x \qquad [10]$$

For slender tubular members, such as the ones shown in figure 1, the wave forces are dominated by drag (the structure does not alter the characteristics of the wave). In fact, over 90% of the total force is due to drag. [1] The selection of a proper Cd becomes critical. Five steps need to be taken to determine the proper coefficient for each member. These steps are taken in the analysis to account for: Reynolds and KC number variations, member orientation, member roughness, and proximity to the free surface and/or mudline.

5.0 ANALYSIS

The analysis of the subject structures is a three-stage process, using three different software programs: Excel, Mathcad, and Matlab. Excel is used to superimpose the three waves derived from the sea-state spectrum, and to plot the water surface profiles. Mathcad is used for derivation of the caissons' structural characteristics, such as bending capacity, mass, stiffness, and resistance to local buckling. It is also used to determine the two structures natural periods. Finally, Matlab is used to perform a time-history analysis of the structures' behavior under periodic loading.

5.1 Environmental Conditions

The first step in the analysis is to determine the maximum wave-height at each of the three sites. Data from the Minerals Management Service shows the track that Andrew took through the Gulf of Mexico, and then over the Mississippi Delta. It also shows the significant wave-heights as contours (See figure 6).

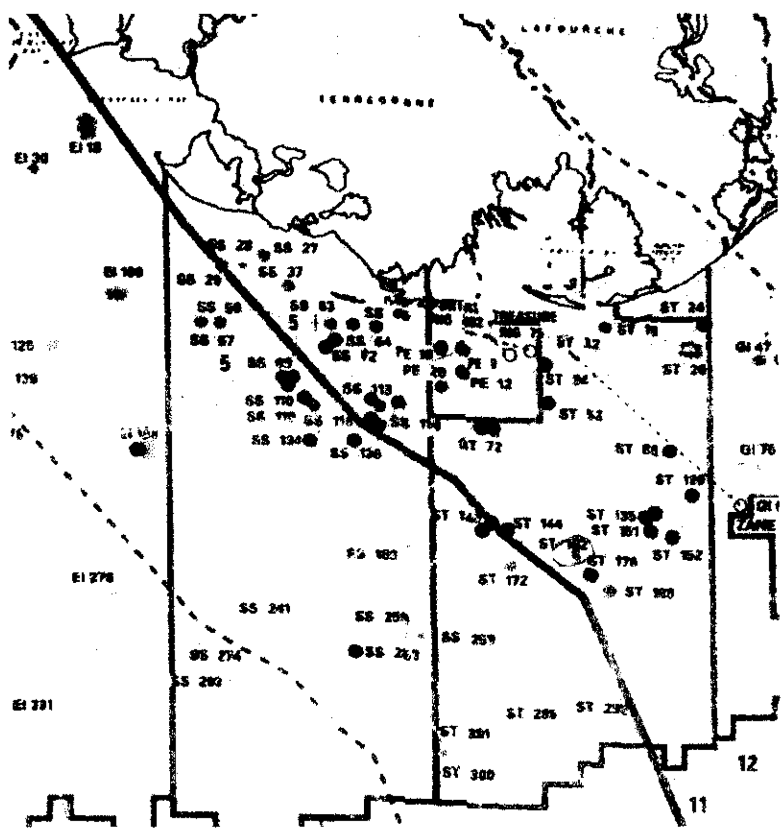


Figure 6

Waveheight Contours at SS and ST Area

Using a lognormal distribution of wave-heights [1] and based on a sample of 200 waves (an average number for a big storm), one can determine the maximum wave-height.

$$H_{max} = H_s \sqrt{\frac{\ln(N)}{2}}$$

$N = 200$

In some cases, this maximum height exceeded the breaking criteria, and had to be lowered in order to give accurate results.

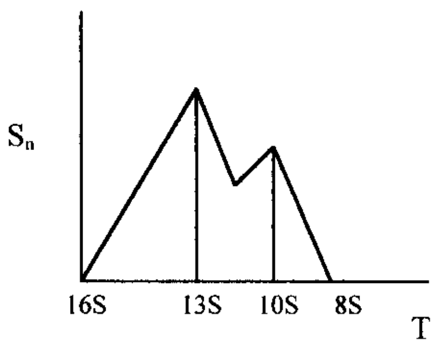


Figure 7
Simplified Spectrum

A Hurricane Andrew wave spectrum was also used to determine the environmental conditions at the sites. As explained in section 4.5, it was not used so much to give the wave-heights, as to express the characteristics of the water-surface fluctuations.

A simplified spectrum is shown at left as figure 7. Figure 8 shows the sea surface eight hours after the center of the storm passed the location of the photographer. Note that there are distinct large swells. These swells are represented by the large peak in the spectrum. The other wave periods passed this location before the photo was taken, leaving only the large regular waves. Regardless, Figure 8 still shows the enormity of the long period waves.

The simplified spectrum was used to generate representative water surface profiles, given

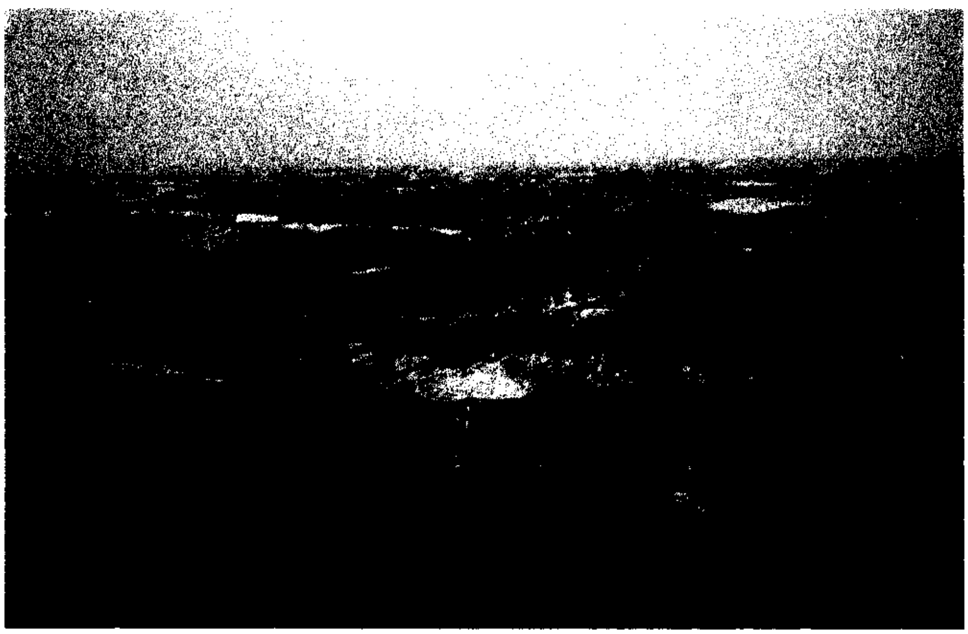


Figure 8
Surface Conditions 10 Hours After the Storm Center Passed

in the appendix. The profiles show that when the three different waves, of random phases, are superimposed, they form distinct large wave "packets." These packets usually consist of two or three very large waves, which rapidly die off, to be followed by another packet. This is significant for the dynamic analysis, in that only three cycles of a large periodic force should be applied to the structures.

5.2 Structural Analysis

The three caisson structures were analyzed using Mathcad, because the program is visual and also very good at carrying units. The following characteristics were determined for each caisson: 1) point of apparent fixity, 2) stiffness, 3) weight, including inner casings and added mass, 4) natural period, 5) elastic and ultimate capacity, 6) possibility of local buckling, and 7) overload ratio. A summary table of the structural characteristics for the three caissons follows.

	Water Depth (ft)	Diameter (in)	Length to Fixity (ft)	Stiffness (K/ft)	Natural Period (Sec)	Ultimate Capacities (kips)	Overload Ratio
Caisson 1	36	36	48.5	45	1.5	F_e=35.0 F_p=44.5	.98
Caisson 2	60	96	100	128.6	2.4	F_e=221 F_p=280	.588
Caisson 3	46	48	66	65.3	1.7	F_e=68.1 F_p=86.5	.787

The analyses for the three caissons is given in the appendix.

Because Caisson #2 is so much stiffer and stronger, it was analyzed using Lpile+, to ensure that the structure behaves in a ductile manner and does not simply rotate or "kick" due to soil failure. The results of this analysis are given in the appendix; they show that the caisson bends and deflects in a ductile manner.

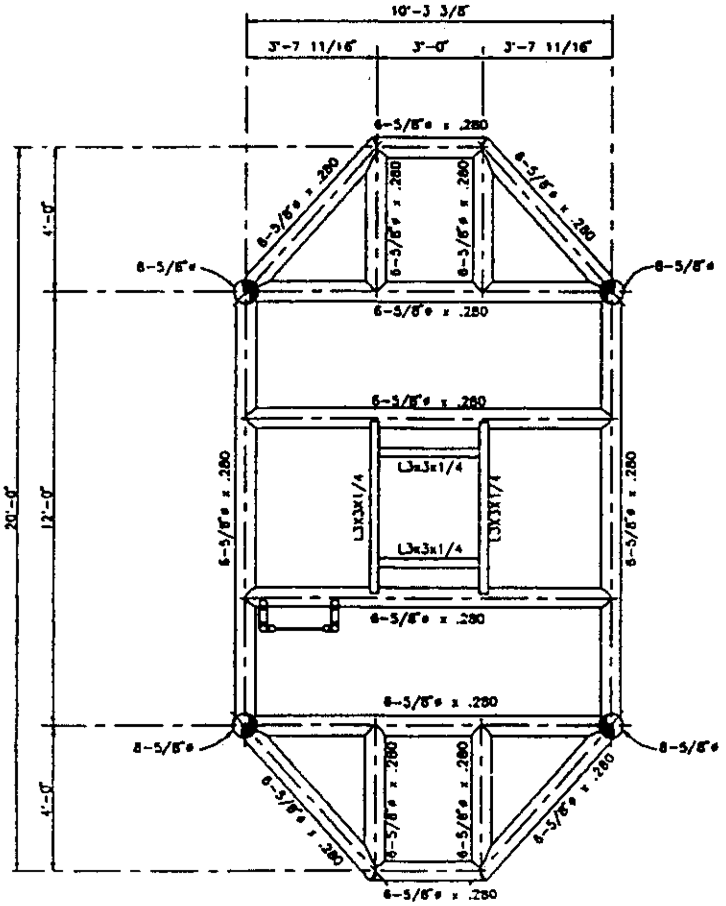
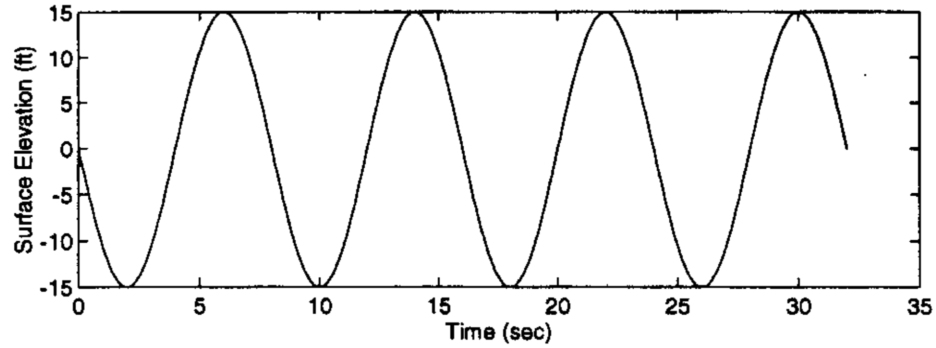


Figure 9
Caisson Deck Plan

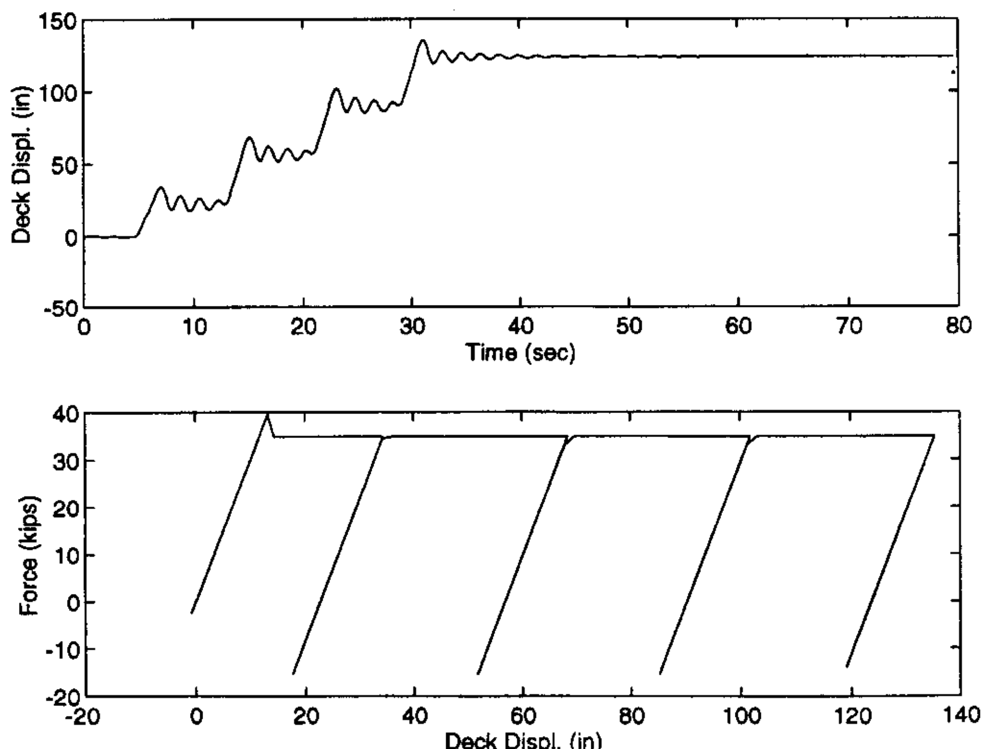
5.3 Time History Analysis

Following are the results of the time-history analysis. The Matlab code, first coded by J. Stear and G. Fenves, first calculates a sinusoidal water surface profile, with the number of waves defined by the user (in these cases, three or four). The program then uses the structures' geometry to determine wave forces. Since the caissons all have boat landings, the program adds on a deck/boatlanding force, when the wave is in contact with the landing. This landing was assumed to be 10 feet tall, and 12 feet wide. Since the structure is heavily framed, it is modelled as a block, with a C_d of 2.5 (See Figure 9).

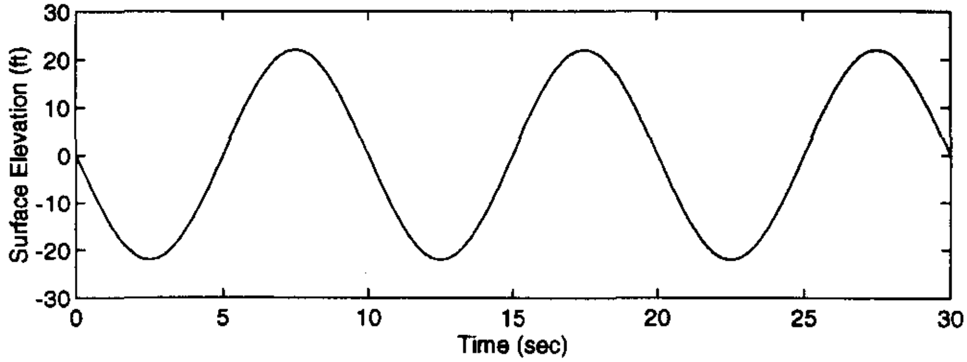
Caisson #1 – Water Surface Profile and Force-Time Relationship

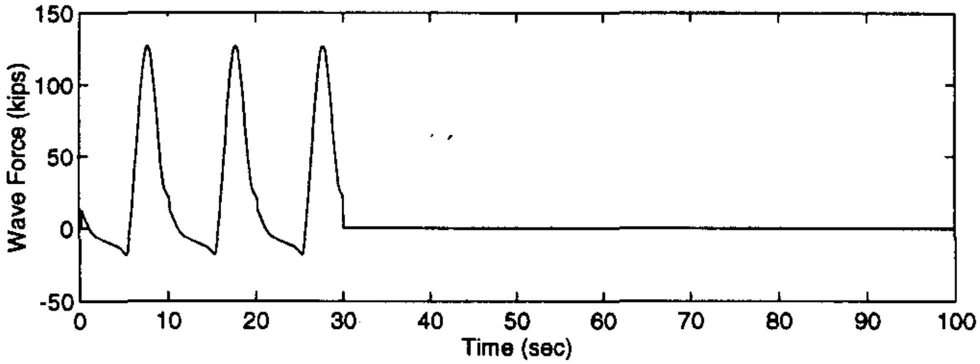


Caisson #1 – Displacement History and Force Displacement Relationship

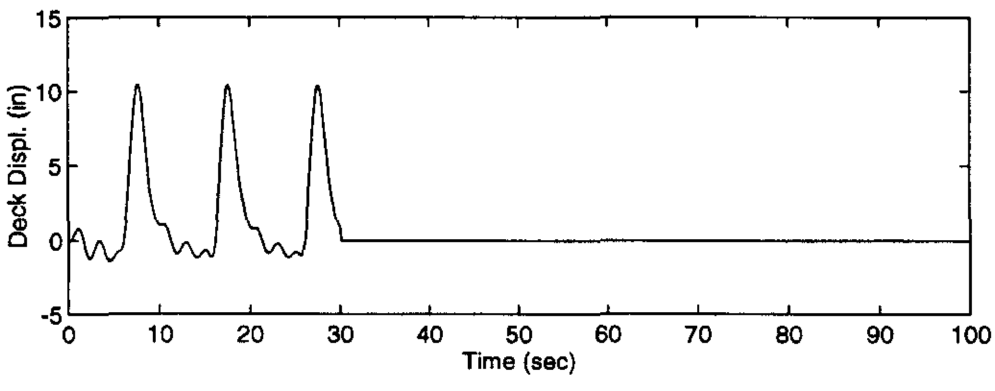


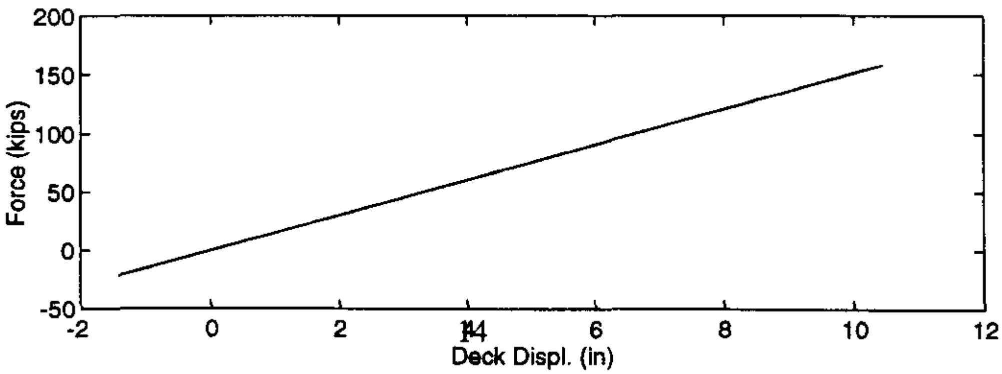
Caisson #2 – Water Surface Profile and Force-Time Relationship



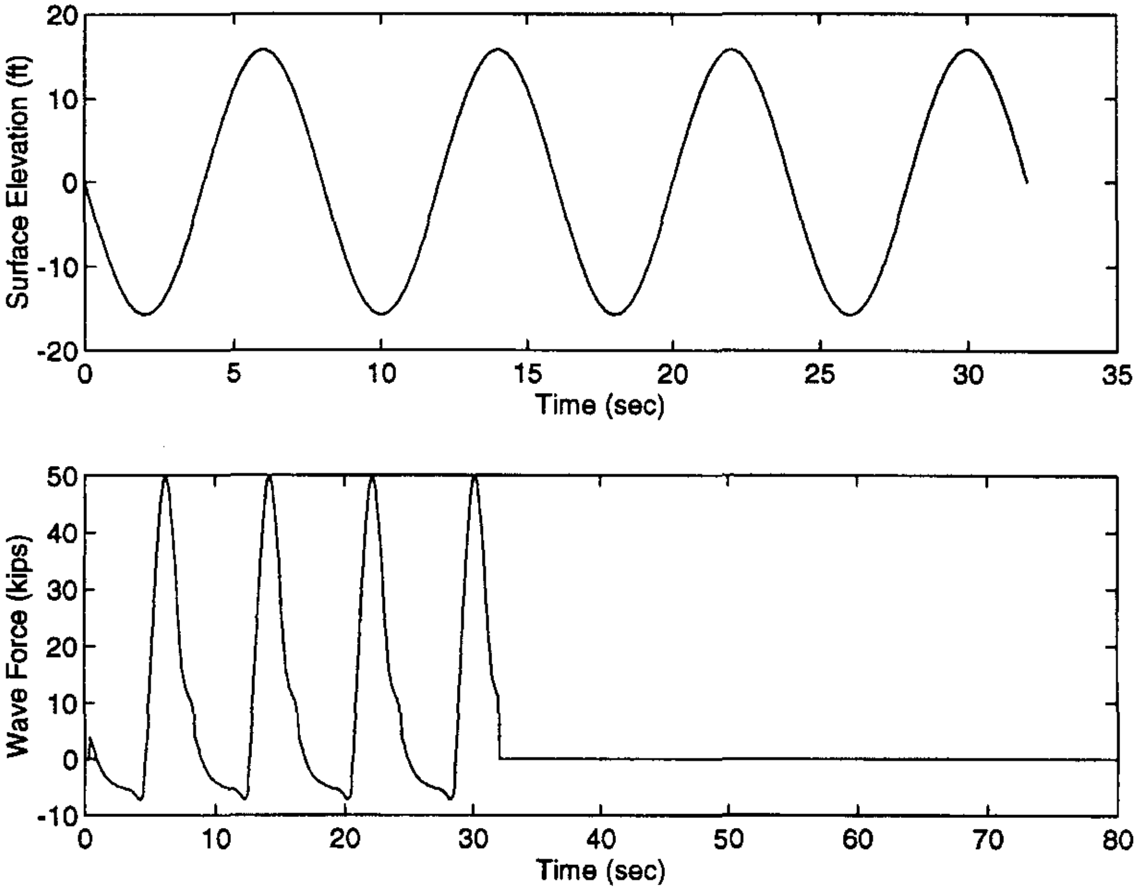


Caisson #2 – Displacement History and Force Displacement Relationship

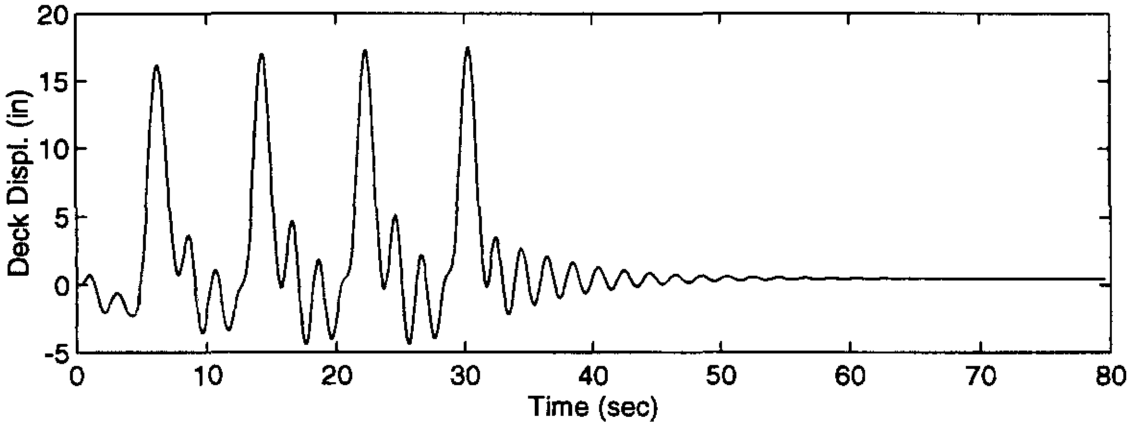




Caisson #3 – Water Surface Profile and Force-Time Relationship



Caisson #3 – Displacement History



References

[1] Bea, R. G., "Wind and Wave Forces on Marine Structures" Class notes for CE205B, UC Berkeley, Fall 1996.

[2] Smith, C.E., "Offshore Platform Damage Assessment in the Aftermath of Hurricane Andrew" Proceedings, 25th Meeting USNR Panel on Wind and Seismic Effects. Tsukuba, Japan May 17-20, 1993

[3] Barnett & Casbarian, Inc. "Development of an Acceptance Criteria for Caisson Structures After Extreme Environmental Loading - Draft Report" August 1994, Houston, Texas

[4] American Petroleum Institute, "API RP 2A-LRFD Section D 2.2.4a Local Buckling" APE, April 1, 1994

[5] Personal Communication. Robert G. Bea, Professor and Vice Chair of Civil Engineering. University of California, Berkeley. December 1996.

[6] Petrauskas C., Botelho D.L., Krieger W.F., and Griffin J.J, "A reliability Model for Offshore Platforms and its Application to ST151 "H" and "K" Platforms During Hurricane Andrew (1992)" Chevron Petroleum Technology Company La Habra, CA

[7] Stear, James. Response of an Offshore Structure to Single and Series Waves. CE 205b, University of California, Berkeley. December 14, 1995

[8] Hong, S. T. and Brooks, J. C., "Dynamic Behavior and Design of Offshore Caissons," Offshore Technology Conference, OTC 2555, 1976

[9] Kriebel, D.L., Berek, E.P., Chakrabarti, S.K. and Waters, J.K, "Wave-Current Loading on a Shallow Water Caisson" Offshore Technology Conference, OTC 8067, 1996

[10] D. T. McDonald, K. Bando, R. G. Bea, and R. J. Sobey. Near Surface Wave Forces on Horizontal Members and Decks of Offshore Platforms, Final Report, Coastal and Hydraulic Engineering, Department of Civil Engineering, University of California, Berkeley, December 18, 1990

[11] MATLAB Version 4.2c, The Mathworks

[12] MATHCAD Version 6.0+, Mathsoft Apps

[13] Stahl, B. and Baur, M.P. "Design Methodology for Offshore Platform

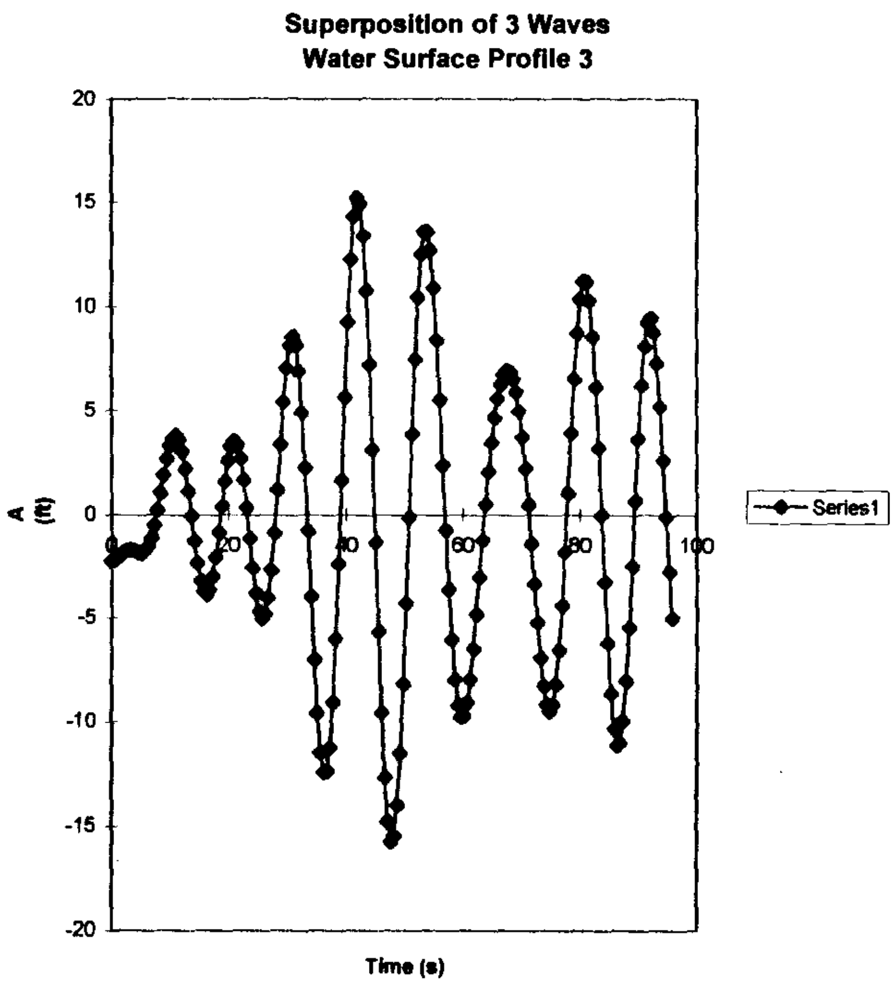
Conductors" Offshore Technology Conference, OTC 3902, 1980

[14] Newmark, N.M. "A Method of Computation for Structural Dynamics," Transactions ASCE, 1962

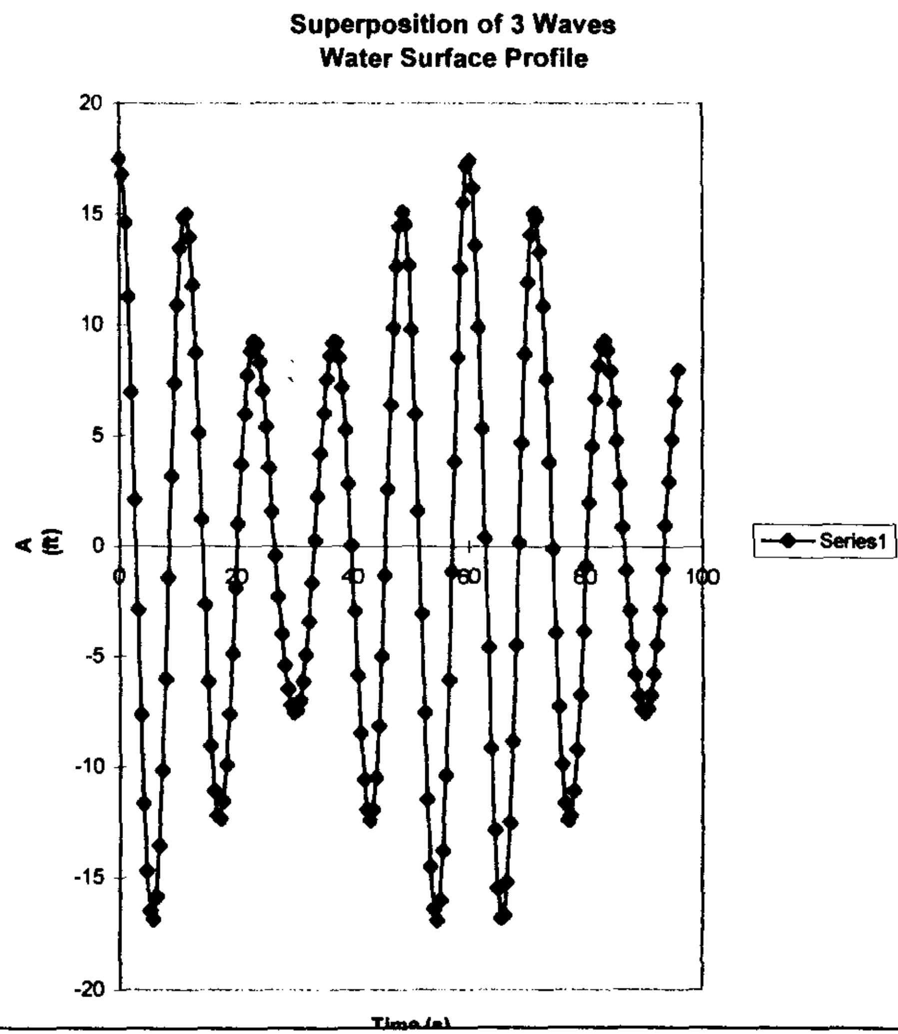
Appendix

Water Surface Profiles
Structural Analysis Using Mathcad [12]
Lpile+ Plots
Matlab Scripts

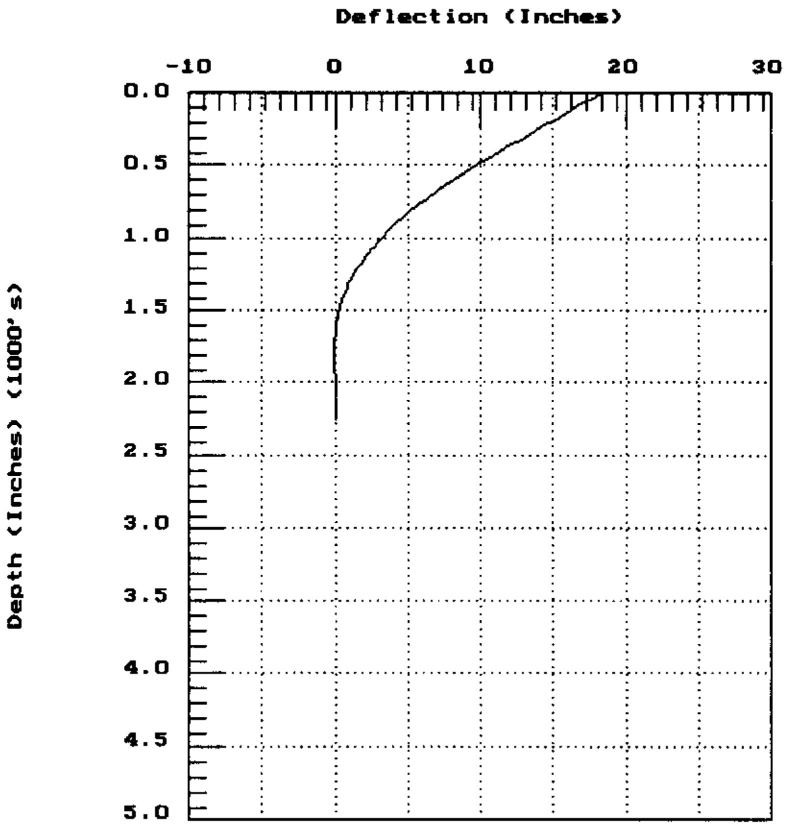
Degree	t (rad)	Wave 1	Wave 2	Wave 3	Sigma(A)	HMax=2A
		T=14s	T=12s	T=10s		34.26349
		H=10Ft	H=15Ft	H=10Ft		
		A1	A2	A3	Total	
0	0	3.299916	-6.93227	1.418311	-2.21404	
30	0.523583	2.336485	-7.44806	2.890594	-2.22098	
60	1.047167	1.245087	-7.40884	4.053163	-2.11059	
90	1.57075	0.085497	-6.81753	4.781453	-1.95058	
120	2.094333	-1.07878	-5.71819	4.997431	-1.79953	
150	2.617917	-2.18396	-4.19274	4.677956	-1.69875	
180	3.1415	-3.16954	-2.35487	3.857259	-1.66715	
210	3.665083	-3.98152	-0.34151	2.623273	-1.69976	
240	4.188667	-4.57544	1.697296	1.108215	-1.76993	
270	4.71225	-4.91877	3.609622	-0.52558	-1.83473	
300	5.235833	-4.9927	5.252968	-2.10307	-1.8428	
330	5.759417	-4.79318	6.504875	-3.45522	-1.74352	
360	6.283	-4.33115	7.272054	-4.43716	-1.49625	
390	6.806583	-3.6319	7.497335	-4.94367	-1.07824	
420	7.330167	-2.73374	7.163933	-4.9205	-0.4903	
450	7.85375	-1.68585	6.29669	-4.37011	0.240732	
480	8.377333	-0.54563	4.960233	-3.35149	1.063118	
510	8.900917	0.624473	3.25415	-1.97376	1.90486	
540	9.4245	1.760374	1.305576	-0.38456	2.681389	
570	9.948083	2.799861	-0.74029	1.245845	3.305419	
600	10.47167	3.686003	-2.73099	2.742764	3.697781	
630	10.99525	4.370265	-4.51818	3.945808	3.797895	
660	11.51883	4.815171	-5.96868	4.726076	3.572562	
690	12.04242	4.996356	-6.97442	4.999965	3.021901	
720	12.566	4.903894	-7.46044	4.73813	2.181588	
750	13.08958	4.542851	-7.39052	3.968625	1.120957	
780	13.61317	3.933	-6.76988	2.773899	-0.06298	
810	14.13675	3.107742	-5.64476	1.281962	-1.25505	
840	14.66033	2.112276	-4.09901	-0.34733	-2.33406	
870	15.18392	1.001122	-2.24781	-1.93941	-3.18609	
900	15.7075	-0.16486	-0.2291	-3.32369	-3.71765	
930	16.23108	-1.32182	1.806672	-4.35185	-3.867	
960	16.75467	-2.40638	3.707817	-4.91373	-3.61229	



Degree	t (rad)	Wave 1	Wave 2	Wave 3	Sigma(A)	HMax=2A
		T=14s	T=12s	T=10s		34.99754
		H=10Ft	H=15Ft	H=10Ft		
		A1	A2	A3	Total	
0	0	5	7.5	5	17.5	
30	0.523583	4.81347	7.220204	4.732136	16.76581	
60	1.047167	4.267796	6.401694	3.957244	14.62673	
90	1.57075	3.403693	5.10554	2.758351	11.26758	
120	2.094333	2.285633	3.42845	1.263912	6.977996	
150	2.617917	0.997038	1.495556	-0.36595	2.126645	
180	3.1415	-0.36595	-0.54892	-1.9566	-2.87147	
210	3.665083	-1.70163	-2.55245	-3.33761	-7.59169	
240	4.188667	-2.91035	-4.36553	-4.36101	-11.6369	
270	4.71225	-3.90192	-5.85289	-4.91715	-14.672	
300	5.235833	-4.60237	-6.90355	-4.94643	-16.4523	
330	5.759417	-4.95942	-7.43912	-4.44573	-16.8443	
360	6.283	-4.94643	-7.41965	-3.46869	-15.8348	
390	6.806583	-4.56439	-6.84658	-2.11999	-13.531	
420	7.330167	-3.84178	-5.76267	-0.54414	-10.1486	
450	7.85375	-2.83253	-4.2488	1.090006	-5.99132	
480	8.377333	-1.61194	-2.41791	2.607365	-1.42249	
510	8.900917	-0.27108	-0.40662	3.845356	3.167656	
540	9.4245	1.090006	1.63501	4.671334	7.39635	
570	9.948083	2.369765	3.554648	4.996798	10.92121	
600	10.47167	3.47271	5.209066	4.786878	13.46865	
630	10.99525	4.31655	6.474824	4.064064	14.85544	
660	11.51883	4.838322	7.257483	2.905804	15.00161	
690	12.04242	4.999096	7.498644	1.436199	13.93394	
720	12.566	4.786878	7.180317	-0.18729	11.77991	
750	13.08958	4.2175	6.32625	-1.79071	8.753042	
780	13.61317	3.333445	5.000168	-3.20226	5.131352	
810	14.13675	2.200676	3.301013	-4.27071	1.230982	
840	14.66033	0.903709	1.355563	-4.88156	-2.62229	
870	15.18392	-0.46069	-0.69103	-4.96938	-6.1211	
900	15.7075	-1.79071	-2.68606	-4.52475	-9.00152	
930	16.23108	-2.98712	-4.48068	-3.59532	-11.0631	
960	16.75467	-3.96066	-5.94099	-2.28066	-12.1823	



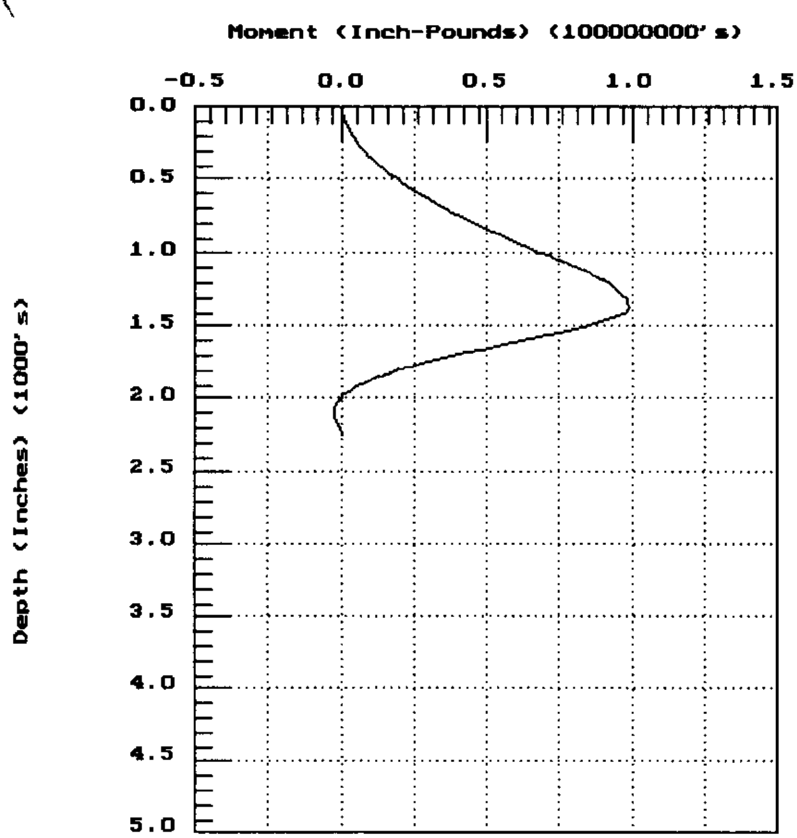
1A – 20 cycles
w/sand



CAIS2.GRF

Cntl-P to Print Screen

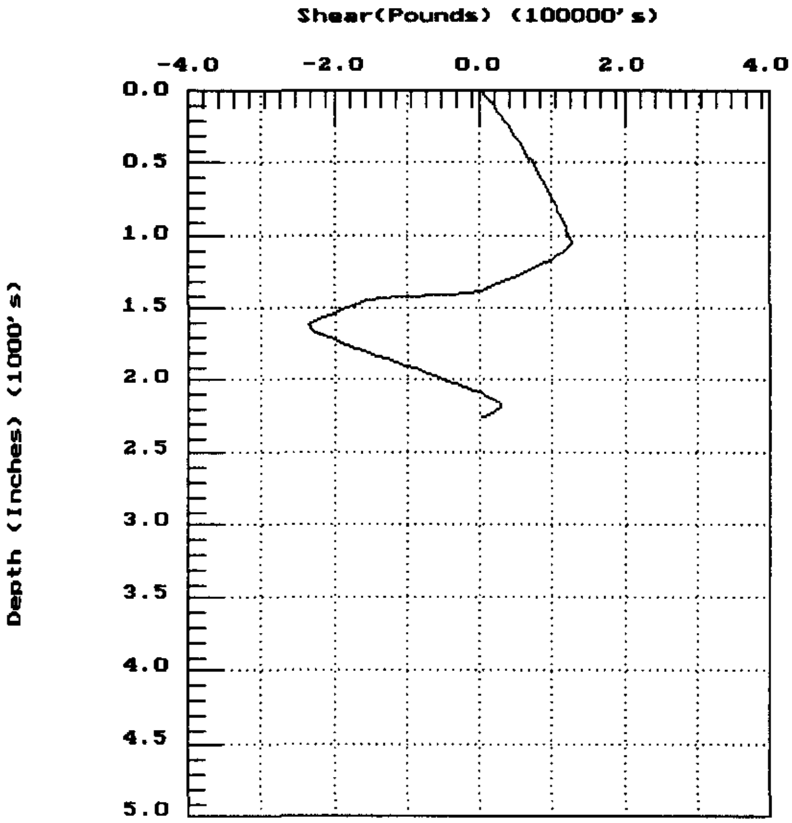
1A - 20 cycles w/sand



CAIS2.GRF

Cntl-P to Print Screen

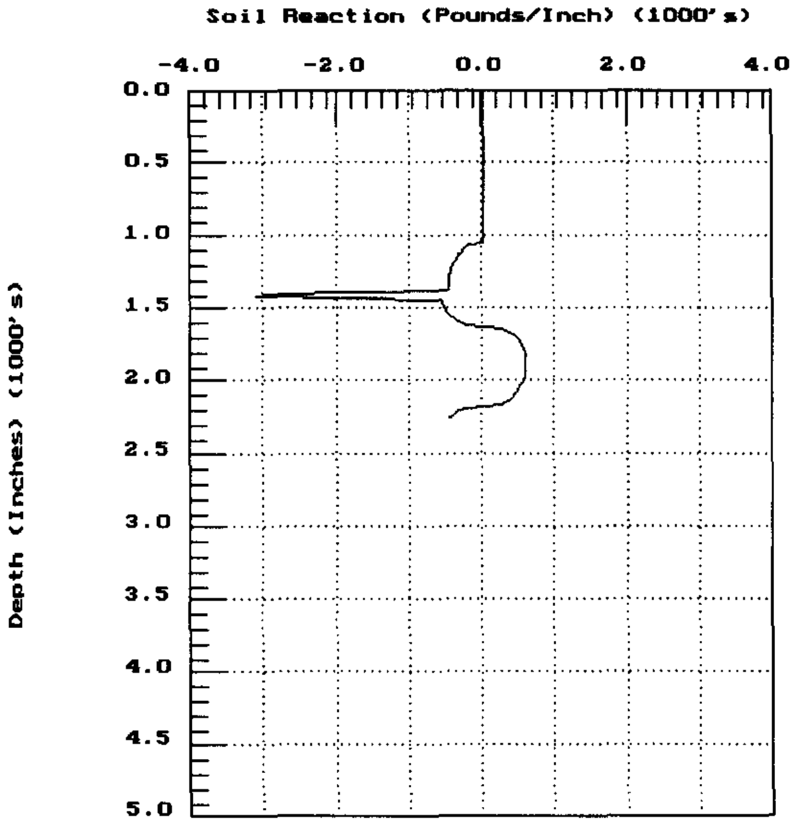
1A - 20 - cycles
w/ sand



CAIS2.GRF

Cntl-P to Print Screen

1A - 20 cycles w/sand



CAIS2.GRF

Cntl-P to Print Screen

Caisson #1 South Pelto 10 — 30" diameter caisson in 36 feet of water.

1) Define geometry and constants for Mathcad

$kips := 1000 \cdot lbf$ $\quad E := 29000 \cdot \frac{kips}{in^2}$ $\quad I := .049087 \cdot (D_o^4 - D_i^4)$

$g := 32.2 \cdot \frac{ft}{sec^2}$ $\quad A := .785398 \cdot (D_o^2 - D_i^2)$

$S := .098175 \cdot \left(\frac{D_o^4 - D_i^4}{D_o} \right)$

2) Define caisson structural characteristics

$L_1 := 6 \cdot ft$	$WT_1 := 1.375 \cdot in$	$D_{i1} := 30 \cdot in - WT_1$	$I_1 := .049087 \cdot (D_o^4 - D_{i1}^4)$	$I_1 = 0.328 \cdot ft^4$
$L_2 := 10 \cdot ft$	$WT_2 := 1.375 \cdot in$	$D_{i2} := 30 \cdot in - WT_2$	$I_2 := .049087 \cdot (D_o^4 - D_{i2}^4)$	$I_2 = 0.328 \cdot ft^4$
$L_3 := 10 \cdot ft$	$WT_3 := 1.625 \cdot in$	$D_{i3} := 30 \cdot in - WT_3$	$I_3 := .049087 \cdot (D_o^4 - D_{i3}^4)$	$I_3 = 0.383 \cdot ft^4$
$L_4 := 30 \cdot ft$	$WT_4 := 1.75 \cdot in$	$D_{i4} := 30 \cdot in - WT_4$	$I_4 := .049087 \cdot (D_o^4 - D_{i4}^4)$	$I_4 = 0.41 \cdot ft^4$
$L_5 := 10 \cdot ft$	$WT_5 := 1.675 \cdot in$	$D_{i5} := 30 \cdot in - WT_5$	$I_5 := .049087 \cdot (D_o^4 - D_{i5}^4)$	$I_5 = 0.394 \cdot ft^4$
$L_6 := 10 \cdot ft$	$WT_6 := 1.375 \cdot in$	$D_{i6} := 30 \cdot in - WT_6$	$I_6 := .049087 \cdot (D_o^4 - D_{i6}^4)$	$I_6 = 0.328 \cdot ft^4$
$L_7 := 10 \cdot ft$	$WT_7 := .875 \cdot in$	$D_{i7} := 30 \cdot in - WT_7$	$I_7 := .049087 \cdot (D_o^4 - D_{i7}^4)$	$I_7 = 0.214 \cdot ft^4$
$L_8 := 145 \cdot ft$	$WT_8 := .5 \cdot in$	$D_{i8} := 30 \cdot in - WT_8$	$I_8 := .049087 \cdot (D_o^4 - D_{i8}^4)$	$I_8 = 0.125 \cdot ft^4$

Determine point of apparent fixity:

$d := 5 \cdot D_o$ $\quad d = 12.5 \cdot ft$

This point lies in depth 4

3) Determine caisson stiffness

$L_{eff} := 36 \cdot ft + 12.5 \cdot ft$ $\quad L_{eff} = 48.5 \cdot ft$ $\quad I_{av} := .049087 \cdot (D_o^4 - D_{i4}^4)$ $\quad I_{av} = 0.41 \cdot ft^4$

$K := \frac{3 \cdot E \cdot I_{av}}{L_{eff}^3}$ $\quad K = 44.997 \cdot \frac{kips}{ft}$

4) Calculate the cantilever's weight for dynamics calculations

$L_1 := 75 \cdot ft$	$WT_1 := 1.375 \cdot in$	$D_{i1} := 30 \cdot in - WT_1$	$A_1 := \pi \cdot (D_o - WT_1) \cdot WT_1$	$A_1 = 0.859 \cdot ft^2$
$L_2 := 10 \cdot ft$	$WT_2 := 1.375 \cdot in$	$D_{i2} := 30 \cdot in - WT_2$	$A_2 := \pi \cdot (D_o - WT_2) \cdot WT_2$	$A_2 = 0.859 \cdot ft^2$
$L_3 := 10 \cdot ft$	$WT_3 := 1.625 \cdot in$	$D_{i3} := 30 \cdot in - WT_3$	$A_3 := \pi \cdot (D_o - WT_3) \cdot WT_3$	$A_3 = 1.006 \cdot ft^2$
$L_4 := 22.5 \cdot ft$	$WT_4 := 1.75 \cdot in$	$D_{i4} := 30 \cdot in - WT_4$	$A_4 := \pi \cdot (D_o - WT_4) \cdot WT_4$	$A_4 = 1.079 \cdot ft^2$

$W_1 := 490 \cdot \frac{lbf}{ft^3} \cdot A_1 \cdot L_1$ $\quad W_1 = 31.557 \cdot kips$

$W_2 := 490 \cdot \frac{lbf}{ft^3} \cdot A_2 \cdot L_2$ $\quad W_2 = 4.208 \cdot kips$

$W_3 := 490 \cdot \frac{lbf}{ft^3} \cdot A_3 \cdot L_3$ $\quad W_3 = 4.929 \cdot kips$

$W_4 := 490 \cdot \frac{lbf}{ft^3} \cdot A_4 \cdot L_4$ $\quad W_4 = 11.891 \cdot kips$

Inner casings:

$W_{c1} := \pi \cdot (20 \cdot in - .44 \cdot in) \cdot .44 \cdot in \cdot (69 \cdot ft + 12.5 \cdot ft) \cdot 490 \cdot \frac{lbf}{ft^3}$

$W_{c2} := \left(68 \cdot \frac{lbf}{ft}\right) \cdot 81.5 \cdot ft$

$W_{c3} := \left(47 \cdot \frac{lbf}{ft}\right) \cdot 81.5 \cdot ft$ $\quad W_{st} := W_1 + W_2 + W_3 + W_4 + W_{c1} + W_{c2} + W_{c3}$

$W_{st} = 69.455 \cdot kips$

$W_{deck} := 15 \cdot kips$

$W_{bl} := 5 \cdot kips$

Calculate added hydrodynamic mass:

$V := \pi \cdot \frac{(30 \cdot in)^2}{4} \cdot 36 \cdot ft$ $\quad W_H := 2 \cdot 64 \cdot \frac{lbf}{ft^3} \cdot V$ $\quad W_H = 22.619 \cdot kips$ $\quad$ [1]

$W_{tot} := W_{st} + W_{deck} + W_{bl} + W_H$ $\quad W_{tot} = 112.075 \cdot kips$

$M := \frac{W_{tot}}{32.2 \cdot \frac{ft}{sec^2}}$ $\quad M = 3.481 \cdot ft^{-1} \cdot sec^2 \cdot kips$ $\quad T_n := 2 \cdot \pi \cdot \sqrt{\frac{M}{K}}$ $\quad T_n = 1.747 \cdot sec$

These natural periods fall within the acceptable range of one to five seconds.

5) Determine caisson's ultimate elastic and plastic capacity
Note: Steel is A36

$S_4 := .098175 \cdot \left(\frac{D_o^4 - D_{i4}^4}{D_o}\right)$ $\quad M_{el} := 36 \cdot \frac{kips}{in^2} \cdot S_4$ $\quad M_{el} = 2.039 \cdot 10^4 \cdot kips \cdot in$

$F_{el} := \frac{M_{el}}{48.5 \cdot ft}$ $\quad F_{el} = 35.039 \cdot kips$

$Z := 1.27 \cdot S_4$ $\quad M_{plas} := 36 \cdot \frac{kips}{in^2} \cdot Z$ $\quad M_{plas} = 2.59 \cdot 10^4 \cdot kips \cdot in$

$F_{plas} := \frac{M_{plas}}{48.5 \cdot ft}$ $\quad F_{plas} = 44.499 \cdot kips$

6) Check for local buckling of the caisson.

$\frac{D_o}{WT_4} = 17.143$ $\quad F_y \quad 36 \quad \frac{2070}{F_y} = 57.5$

17.14 < 57.5
This section is not likely to buckle locally
AISCC-LRFD

6) Calculate caisson's overload ratio (η)

$F_{wave} \cdot 40 \cdot kips$ $\quad F_{plas} \cdot 40.8 \cdot kips$

(This value was used in the analysis, instead of 44 kips.)

$\eta \cdot \frac{F_{wave}}{F_{plas}}$ $\quad \eta = 0.98$

This caisson is not overloaded by the maximum static wave force. This will prove significant in the dynamic analysis.

Caisson #2 South Timbalier 52

96" diameter caisson in 60 feet of water. Casings are not grouted.

1) Define geometry and constants for Mathcad

$kips := 1000 \cdot lbf$ $\qquad E := 29000 \cdot \frac{kips}{in^2}$ $\qquad I := .049087 \cdot \left(D_o^4 - D_i^4\right)$

$A := .785398 \cdot \left(D_o^2 - D_i^2\right)$

$S := .098175 \cdot \left(\frac{D_o^4 - D_i^4}{D_o}\right)$

2) Define caisson structural characteristics

$L_1 := 15.5 \cdot ft$ $\quad WT_1 := 1.25 \cdot in$ $\quad D_{i1} := 72 \cdot in - WT_1$ $\quad I_1 := .049087 \cdot \left(D_o^4 - D_{i1}^4\right)$ $\quad I_1 = 4.304 \cdot ft^4$

$L_2 := 9.5 \cdot ft$ $\quad WT_2 := 1.25 \cdot in$ $\quad D_{i2} := 72 \cdot in - WT_2$ $\quad I_2 := .049087 \cdot \left(D_o^4 - D_{i2}^4\right)$ $\quad I_2 = 4.304 \cdot ft^4$

$L_3 := 25 \cdot ft$ $\quad WT_3 := 1.25 \cdot in$ $\quad D_{i3} := 84 \cdot in - WT_3$ $\quad I_3 := .049087 \cdot \left(D_o^4 - D_{i3}^4\right)$ $\quad I_3 = 6.86 \cdot ft^4$

$L_4 := 20 \cdot ft$ $\quad WT_4 := 1.25 \cdot in$ $\quad D_{i4} := 96 \cdot in - WT_4$ $\quad I_4 := .049087 \cdot \left(D_o^4 - D_{i4}^4\right)$ $\quad I_4 = 10.269 \cdot ft^4$

$L_5 := 30 \cdot ft$ $\quad WT_5 := 1.5 \cdot in$ $\quad D_{i5} := 96 \cdot in - WT_5$ $\quad I_5 := .049087 \cdot \left(D_o^4 - D_{i5}^4\right)$ $\quad I_5 = 12.275 \cdot ft^4$

$L_6 := 25 \cdot ft$ $\quad WT_6 := 1.25 \cdot in$ $\quad D_{i6} := 96 \cdot in - WT_6$ $\quad I_6 := .049087 \cdot \left(D_o^4 - D_{i6}^4\right)$ $\quad I_6 = 10.269 \cdot ft^4$

$L_7 := 5 \cdot ft$ $\quad WT_7 := 1.0 \cdot in$ $\quad D_{i7} := 96 \cdot in - WT_7$ $\quad I_7 := .049087 \cdot \left(D_o^4 - D_{i7}^4\right)$ $\quad I_7 = 8.248 \cdot ft^4$

$L_8 := 10 \cdot ft$ $\quad WT_8 := .75 \cdot in$ $\quad D_{i8} := 96 \cdot in - WT_8$ $\quad I_8 := .049087 \cdot \left(D_o^4 - D_{i8}^4\right)$ $\quad I_8 = 6.21 \cdot ft^4$

$L_9 := 20 \cdot ft$ $\quad WT_9 := .5 \cdot in$ $\quad D_{i9} := 96 \cdot in - WT_9$ $\quad I_9 := .049087 \cdot \left(D_o^4 - D_{i9}^4\right)$ $\quad I_9 = 4.156 \cdot ft^4$

Determine point of apparent fixity:

$d := 5 \cdot D_o$ $\qquad d = 40 \cdot ft$

This point lies in depth 5

3) Determine caisson stiffness

$L_{eff} := 60 \cdot ft + 40 \cdot ft$ $\qquad L_{eff} = 100 \cdot ft$ $\qquad I_{av} := .049087 \cdot \left(D_o^4 - D_{i4}^4\right)$ $\qquad I_{av} = 10.269 \cdot ft^4$

$K := \frac{3 \cdot E \cdot I_{av}}{L_{eff}^3}$ $\qquad K = 128.652 \cdot \frac{kips}{ft}$

4) Calculate the Cantilever's weight

$L_1 := 105 \cdot ft$ $\quad WT_1 := 1.25 \cdot in$ $\quad D_{i1} := 72 \cdot in - WT_1$ $\quad A_1 := \pi \cdot \left(D_o - WT_1\right) \cdot WT_1$ $\quad A_1 = 1.929 \cdot ft^2$

$L_2 := 9.5 \cdot ft$ $\quad WT_2 := 1.25 \cdot in$ $\quad D_{i2} := 72 \cdot in - WT_2$ $\quad A_2 := \pi \cdot \left(D_o - WT_2\right) \cdot WT_2$ $\quad A_2 = 1.929 \cdot ft^2$

$L_3 := 25 \cdot ft$ $\quad WT_3 := 1.25 \cdot in$ $\quad D_{i3} := 84 \cdot in - WT_3$ $\quad A_3 := \pi \cdot \left(D_o - WT_3\right) \cdot WT_3$ $\quad A_3 = 2.257 \cdot ft^2$

$L_4 := 20 \cdot ft$ $\quad WT_4 := 1.25 \cdot in$ $\quad D_{i4} := 96 \cdot in - WT_4$ $\quad A_4 := \pi \cdot \left(D_o - WT_4\right) \cdot WT_4$ $\quad A_4 = 2.584 \cdot ft^2$

$L_5 := 30 \cdot ft$ $\quad WT_5 := 1.5 \cdot in$ $\quad D_{i5} := 96 \cdot in - WT_5$ $\quad A_5 := p \cdot \left(D_o - WT_5\right) \cdot WT_5$ $\quad A_5 = 3.093 \cdot ft^2$

$W_1 := 490 \cdot \frac{lbf}{ft^3} \cdot A_1 \cdot L_1$ $\qquad W_1 = 99.268 \cdot kips$

$W_2 := 490 \cdot \frac{lbf}{ft^3} \cdot A_2 \cdot L_2$ $\qquad W_2 = 8.981 \cdot kips$

$W_3 := 490 \cdot \frac{lbf}{ft^3} \cdot A_3 \cdot L_3$ $\qquad W_3 = 27.644 \cdot kips$

$W_4 := 490 \cdot \frac{lbf}{ft^3} \cdot A_4 \cdot L_4$ $\qquad W_4 = 25.322 \cdot kips$

$W_5 := 490 \cdot \frac{lbf}{ft^3} \cdot A_5 \cdot L_5$ $\qquad W_5 = 45.46 \cdot kips$

Inner casings:

$W_{c1} := \pi \cdot (30 \cdot in - .4 \cdot in) \cdot .4 \cdot in \cdot (129.5 \cdot ft) \cdot 490 \cdot \frac{lbf}{ft^3}$

$W_{c2} := \pi \cdot (22 \cdot in - .25 \cdot in) \cdot .25 \cdot in \cdot (129.5 \cdot ft) \cdot 490 \cdot \frac{lbf}{ft^3}$

$W_{c3} := \pi \cdot (20 \cdot in - 1 \cdot in) \cdot 1 \cdot in \cdot (129.5 \cdot ft) \cdot 490 \cdot \frac{lbf}{ft^3}$

$W_{deck} := 50 \cdot kips$

$W_{bl} := 7 \cdot kips$

$W_{st} := W_1 + W_2 + W_3 + W_4 + 3 \cdot (W_{c1} + W_{c2} + W_{c3})$

$W_{st} = 311.881 \cdot kips$

Calculate added hydrodynamic mass:

$V := \pi \cdot \frac{(96 \cdot in)^2}{4} \cdot 60 \cdot ft$ $\qquad W_H := 2 \cdot 64 \cdot \frac{lbf}{ft^3} \cdot V$ $\qquad W_H = 386.039 \cdot kips$ $\qquad$ [1]

$W_{tot} := W_{st} + W_{deck} + W_{bl} + W_H$ $\qquad W_{tot} = 754.919 \cdot kips$

$M := \frac{W_{tot}}{32.2 \cdot \frac{ft}{sec^2}}$ $\qquad M = 23.445 \cdot ft^{-1} \cdot sec^2 \cdot kips$ $\qquad T_n := 2 \cdot \pi \cdot \sqrt{\frac{M}{K}}$ $\qquad T_n = 2.682 \cdot sec$

These natural periods fall within the acceptable range of one to five seconds.

5) Determine caisson's ultimate elastic and plastic capacity
Note: Steel is Grade 50

$S_5 := .098175 \cdot \left(\frac{D_o^4 - D_{i5}^4}{D_o} \right)$ $M_{el} := 50 \cdot \frac{kips}{in^2} \cdot S_5$ $M_{el} = 2.651 \cdot 10^5 \cdot kips \cdot in$

$F_{el} := \frac{M_{el}}{100 \cdot ft}$ $F_{el} = 220.949 \cdot kips$

$Z := 1.27 \cdot S_5$ $M_{plas} := 50 \cdot \frac{kips}{in^2} \cdot Z$ $M_{plas} = 3.367 \cdot 10^5 \cdot kips \cdot in$

$F_{plas} := \frac{M_{plas}}{100 \cdot ft}$ $F_{plas} = 280.605 \cdot kips$

6) Check for local buckling of the caisson.

$\frac{D_o}{WT_5} = 64$ $F_y := 50$ $\frac{2070}{F_y} = 41.4$

64 > 41.4
This section has the possibility to buckle locally
[AISCC-LRFD]

6) Calculate caisson's overload ratio (η)

$F_{wave} := 130 \cdot kips$ $F_{plas} := 221 \cdot kips$

$\eta := \frac{F_{wave}}{F_{plas}}$ $\eta = 0.588$

This caisson is not overloaded by the maximum static wave force.

Caisson #3 Ship Shoal 113

48" diameter caisson in 46 feet of water. Casings are not grouted.

1) Define geometry and constants for Mathcad

$kips := 1000 \cdot lbf$ $\quad E := 29000 \cdot \frac{kips}{in^2}$ $\quad I := .049087 \cdot (D_o^4 - D_i^4)$

$A := .785398 \cdot (D_o^2 - D_i^2)$

$S := .098175 \cdot \left(\frac{D_o^4 - D_i^4}{D_o}\right)$

2) Define caisson structural characteristics

$L_1 := 21 \cdot ft$	$WT_1 := .75 \cdot in$	$D_{i1} := 48 \cdot in - WT_1$	$I_1 := .049087 \cdot (D_o^4 - D_{i1}^4)$	$I_1 = 0.767 \cdot ft^4$
$L_2 := 10 \cdot ft$	$WT_2 := 1.25 \cdot in$	$D_{i2} := 48 \cdot in - WT_2$	$I_2 := .049087 \cdot (D_o^4 - D_{i2}^4)$	$I_2 = 1.259 \cdot ft^4$
$L_3 := 10 \cdot ft$	$WT_3 := 1.5 \cdot in$	$D_{i3} := 48 \cdot in - WT_3$	$I_3 := .049087 \cdot (D_o^4 - D_{i3}^4)$	$I_3 = 1.499 \cdot ft^4$
$L_4 := 45 \cdot ft$	$WT_4 := 1.75 \cdot in$	$D_{i4} := 48 \cdot in - WT_4$	$I_4 := .049087 \cdot (D_o^4 - D_{i4}^4)$	$I_4 = 1.735 \cdot ft^4$
$L_5 := 10 \cdot ft$	$WT_5 := 1.5 \cdot in$	$D_{i5} := 48 \cdot in - WT_5$	$I_5 := .049087 \cdot (D_o^4 - D_{i5}^4)$	$I_5 = 1.499 \cdot ft^4$
$L_6 := 10 \cdot ft$	$WT_6 := 1.25 \cdot in$	$D_{i6} := 48 \cdot in - WT_6$	$I_6 := .049087 \cdot (D_o^4 - D_{i6}^4)$	$I_6 = 1.259 \cdot ft^4$
$L_7 := 5 \cdot ft$	$WT_7 := 1.0 \cdot in$	$D_{i7} := 48 \cdot in - WT_7$	$I_7 := .049087 \cdot (D_o^4 - D_{i7}^4)$	$I_7 = 1.015 \cdot ft^4$
$L_8 := 40 \cdot ft$	$WT_8 := .75 \cdot in$	$D_{i8} := 48 \cdot in - WT_8$	$I_8 := .049087 \cdot (D_o^4 - D_{i8}^4)$	$I_8 = 0.767 \cdot ft^4$

Determine point of apparent fixity:

$d := 5 \cdot D_o$ $\quad d = 20 \cdot ft$

This point lies in depth 4

3) Determine caisson stiffness

$L_{eff} := 46 \cdot ft + 20 \cdot ft$ $\quad L_{eff} = 66 \cdot ft$ $\quad I_{av} := .049087 \cdot (D_o^4 - D_{i5}^4)$ $\quad I_{av} = 1.499 \cdot ft^4$

$K := \frac{3 \cdot E \cdot I_{av}}{L_{eff}^3}$ $\quad K = 65.307 \cdot \frac{kips}{ft}$

4) Calculate the Cantilever's weight

$L_1 := 108 \cdot ft$	$WT_1 := .75 \cdot in$	$D_{i1} := 48 \cdot in - WT_1$	$A_1 := \pi \cdot (D_o - WT_1) \cdot WT_1$	$A_1 = 0.773 \cdot ft^2$
$L_2 := 10 \cdot ft$	$WT_2 := 1.25 \cdot in$	$D_{i2} := 48 \cdot in - WT_2$	$A_2 := \pi \cdot (D_o - WT_2) \cdot WT_2$	$A_2 = 1.275 \cdot ft^2$
$L_3 := 10 \cdot ft$	$WT_3 := 1.5 \cdot in$	$D_{i3} := 48 \cdot in - WT_3$	$A_3 := p \cdot (D_o - WT_3) \cdot WT_3$	$A_3 = 1.522 \cdot ft^2$
$L_4 := 25 \cdot ft$	$WT_4 := 1.75 \cdot in$	$D_{i4} := 48 \cdot in - WT_4$	$A_4 := \pi \cdot (D_o - WT_4) \cdot WT_4$	$A_4 = 1.766 \cdot ft^2$

$W_1 := 490 \cdot \frac{lbf}{ft^3} \cdot A_1 \cdot L_1 \qquad W_1 = 40.914 \cdot kips$

$W_2 := 490 \cdot \frac{lbf}{ft^3} \cdot A_2 \cdot L_2 \qquad W_2 = 6.247 \cdot kips$

$W_3 := 490 \cdot \frac{lbf}{ft^3} \cdot A_3 \cdot L_3 \qquad W_3 = 7.456 \cdot kips$

$W_4 := 490 \cdot \frac{lbf}{ft^3} \cdot A_4 \cdot L_4 \qquad W_4 = 21.631 \cdot kips$

Inner casings:

$W_{c1} := \pi \cdot (30 \cdot in - .44 \cdot in) \cdot .44 \cdot in \cdot (107 \cdot ft) \cdot 490 \cdot \frac{lbf}{ft^3}$

$W_{c2} := \left(65 \cdot \frac{lbf}{ft}\right) \cdot 107 \cdot ft$

$W_{c3} := \left(40 \cdot \frac{lbf}{ft}\right) \cdot 107 \cdot ft$

$W_{c4} := \left(26 \cdot \frac{lbf}{ft}\right) \cdot 107 \cdot ft$

$W_{deck} := 20 \cdot kips$

$W_{bl} := 5 \cdot kips$

$W_{st} := W_1 + W_2 + W_3 + W_4 + W_{c1} + W_{c2} + W_{c3} + W_{c4}$

$W_{st} = 105.142 \cdot kips$

Calculate added hydrodynamic mass:

$V := \pi \cdot \frac{(48 \cdot in)^2}{4} \cdot 46 \cdot ft \qquad W_H := 2 \cdot 64 \cdot \frac{lbf}{ft^3} \cdot V \qquad W_H = 73.991 \cdot kips$ [1]

$W_{tot} := W_{st} + W_{deck} + W_{bl} + W_H \qquad W_{tot} = 204.133 \cdot kips$

$M := \frac{W_{tot}}{32.2 \cdot \frac{ft}{sec^2}} \qquad M = 6.34 \cdot ft^{-1} \cdot sec^2 \cdot kips$

$T_n := 2 \cdot \pi \cdot \sqrt{\frac{M}{K}} \qquad T_n = 1.958 \cdot sec$

These natural periods fall within the acceptable range of one to five seconds.

4) Determine caisson's ultimate elastic and plastic capacity
Note: Steel is Grade 36

$S_5 := .098175 \cdot \left(\frac{D_o^4 - D_{i4}^4}{D_o}\right) \qquad M_{el} := 36 \cdot \frac{kips}{in^2} \cdot S_5 \qquad M_{el} = 5.396 \cdot 10^4 \cdot kips \cdot in$

$F_{el} := \frac{M_{el}}{66 \cdot ft} \qquad F_{el} = 68.13 \cdot kips$

$Z := 1.27 \cdot S_5 \qquad M_{plas} := 36 \cdot \frac{kips}{in^2} \cdot Z \qquad M_{plas} = 6.853 \cdot 10^4 \cdot kips \cdot in$

$F_{plas} := \frac{M_{plas}}{66 \cdot ft} \qquad F_{plas} = 86.525 \cdot kips$

6) Check for local buckling of the caisson.

$\frac{D_o}{WT_4} = 27.429$ $\quad F_y := 36 \quad \frac{2070}{F_y} = 57.5$

<u>27.43 < 57.5</u>
This section is not likely to buckle locally [AISCC-LRFD]

6) Calculate caisson's overload ratio (η)

$F_{wave} := 50 \cdot kips \qquad F_{plas} := 68.1 \cdot kips$

68.1 kips was used in the analysis.

$\eta := \frac{F_{wave}}{F_{plas}} \qquad \eta = 0.734$

This caisson is not overloaded by the maximum static wave force. This will prove significant in the dynamic analysis.

```
function   Cais1

global Tw kw H Cd Cm theta Cddeck wide wkf rho D l ld hs fy fr;

psi=0.05;
w=87.5;
k=3;
fr=40.8;
fy=40.8;
g=32.2*12;
mu=1;

hwave=[30];
cyc=[4];
Tw=8;
D=30;
rho=(64/32.2)*(1/(1000*144*144));
wavenum=length(hwave);
cycnum=length(cyc);
Cd=1.2;
Cm=1.5;
Cddeck=2.5;
d=36*12;
hs=32*12;
wkf=0.88;
wide=12*12;

dt=Tw/100;
L=g*Tw*Tw/(2*pi);
m=w/g;
wn=sqrt(k/m);
c=2*m*wn*psi;
kw=2*pi/L;

for i=1:wavenum

  H=hwave(i)*12;

  for j=1:cycnum

    p=[zeros(1000,1)];

    for ii=1:cyc(j)*100+1

      theta=(2*pi/Tw)*((ii-1)*dt)+(pi/2);

      eta(ii)=(H/2)*cos(theta);

      if ((eta(ii)+d)<=hs)
        l=round(eta(ii)+d);
        p1=sum(PFD(0:1:l));
        p2=sum(PFI(0:1:l));
        p3=0;
      else
```

```
        l=round(eta(ii)+d);
        p1=sum(PFD(0:1:l));
        p2=sum(PFI(0:1:l));
        ld=round((eta(ii)+d)-hs);
    if (ld>10*12) ld=10*12;
      disp('Deck Inundation')
            p3=sum(PFDdeck(0:1:ld));
    else p3=0;
          end
      end

      p(ii)=p1+p2+p3;

    end
time=[0:dt:(length(p)-1)*dt];
time1=[0:dt:(length(eta)-1)*dt];

figure(1)

clf

subplot(2,1,1)
plot(time1,eta./12)
xlabel('Time (sec)')
ylabel('Surface Elevation (ft)')

subplot(2,1,2)
plot(time,p)
ylabel('Wave Force (kips)')
xlabel('Time (sec)')


Deckmax=max(abs(p3))
pause

%    figure(1)

%    clf

%    plot(eta)

%    figure(2)

%    clf

%    plot(p)

    disp('p assembled')

    fy=40.8;

    fr=40.8;

    [u,f]=NNL1(m,c,k,fy,fr,p,dt);
```

```
    epp(j,i)=max(abs(u));

%    figure(3)

%    clf

%    plot(u)

%    figure(4)

%    clf

%    plot(u,f)

    disp('epp done')

%    [u,f]=NNL1(m,c,k,sigmay,sigmar2,p,dt);

%    deg(i,j)=max(abs(u));

%    figure(5)

%    clf

%    plot(u)

%    figure(6)

%    clf

%    plot(u,f)

%    disp('deg done')

  end

end

time=[0:dt:(length(u)-1)*dt];

figure(2)

subplot(2,1,1)
plot(time,u)
xlabel('Time (sec)')
ylabel('Deck Displ. (in)')

 subplot(2,1,2)
 plot(u,f)
 ylabel('Force (kips)')
 xlabel('Deck Displ. (in)')

% mesh(hwave,cyc',epp./16);
```

```
% title('Ductility Demand on B/EPP Structure')
% xlabel('Wave Height (ft)')
% ylabel('Number of Waves')
```

```
function Cais2

global Tw kw H Cd Cm theta Cddeck wide wkf rho D l ld hs fy fr;

psi=0.05;
w=727;
k=15.16;
fr=221;
fy=221;
g=32.2*12;
mu=1;

hwave=[44];
cyc=[3];
Tw=10;
D=72;
rho=(64/32.2)*(1/(1000*144*144));
wavenum=length(hwave);
cycnum=length(cyc);
Cd=1.2;
Cm=1.5;
Cddeck=2.5;
d=60*12;
hs=56*12;
wkf=0.88;
wide=20*12;

dt=Tw/100;
L=g*Tw*Tw/(2*pi);
m=w/g;
wn=sqrt(k/m);
c=2*m*wn*psi;
kw=2*pi/L;

for i=1:wavenum

  H=hwave(i)*12;

  for j=1:cycnum

    p=[zeros(1000,1)];

    for ii=1:cyc(j)*100+1

      theta=(2*pi/Tw)*((ii-1)*dt)+(pi/2);

      eta(ii)=(H/2)*cos(theta);

      if ((eta(ii)+d)<=hs)
        l=round(eta(ii)+d);
        p1=sum(PFD(0:1:l));
        p2=sum(PFI(0:1:l));
        p3=0;
      else
```

```
        l=round(eta(ii)+d);
        p1=sum(PFD(0:1:l));
        p2=sum(PFI(0:1:l));
        ld=round((eta(ii)+d)-hs);
    if (ld>10*12) ld=10*12;
      p3=sum(PFDdeck(0:1:ld));
      disp('Deck Inundation')
     else p3=0;
          end
          end

     p(ii)=p1+p2+p3;

   end
time=[0:dt:(length(p)-1)*dt];
time1=[0:dt:(length(eta)-1)*dt];

figure(1)

clf

subplot(2,1,1)
plot(time1,eta./12)
xlabel('Time (sec)')
ylabel('Surface Elevation (ft)')

subplot(2,1,2)
plot(time,p)
ylabel('Wave Force (kips)')
xlabel('Time (sec)')

Deckmax=max(p3)
pause

%    figure(1)

%    clf

%    plot(eta)

%    figure(2)

%    clf

%    plot(p)

    disp('p assembled')

    fy=221;

    fr=221;

    [u,f]=NNL2(m,c,k,fy,fr,p,dt);
```

```
% title('Ductility Demand on B/EPP Structure')
% xlabel('Wave Height (ft)')
% ylabel('Number of Waves')
```

```
    epp(j,i)=max(abs(u));
%    figure(3)
%    clf
%    plot(u)
%    figure(4)
%    clf
%    plot(u,f)
    disp('epp done')
%    [u,f]=NNL2(m,c,k,sigmay,sigmar2,p,dt);
%    deg(i,j)=max(abs(u));
%    figure(5)
%    clf
%    plot(u)
%    figure(6)
%    clf
%    plot(u,f)
%    disp('deg done')
  end
end
time=[0:dt:(length(u)-1)*dt];
figure(2)

subplot(2,1,1)
plot(time,u)
xlabel('Time (sec)')
ylabel('Deck Displ. (in)')

 subplot(2,1,2)
 plot(u,f)
 ylabel('Force (kips)')
 xlabel('Deck Displ. (in)')

% mesh(hwave,cyc',epp./16);
```

```
function Cais3

global Tw kw H Cd Cm theta Cddeck wide wkf rho D l ld hs fy fr;

psi=0.05;
w=156.1;
k=4;
fr=68.1;
fy=68.1;
g=32.2*12;
mu=1;

hwave=[31.5];
cyc=[4];
Tw=8;
D=48;
rho=(64/32.2)*(1/(1000*144*144));
wavenum=length(hwave);
cycnum=length(cyc);
Cd=1.2;
Cm=1.5;
Cddeck=2.5;
d=48*12;
hs=44*12;
wkf=0.88;
wide=10*12;

dt=Tw/100;
L=g*Tw*Tw/(2*pi);
m=w/g;
wn=sqrt(k/m);
c=2*m*wn*psi;
kw=2*pi/L;

for i=1:wavenum

  H=hwave(i)*12;

  for j=1:cycnum

    p=[zeros(1000,1)];

    for ii=1:cyc(j)*100+1

      theta=(2*pi/Tw)*((ii-1)*dt)+(pi/2);

      eta(ii)=(H/2)*cos(theta);

      if ((eta(ii)+d)<=hs)
        l=round(eta(ii)+d);
        p1=sum(PFD(0:1:l));
        p2=sum(PFI(0:1:l));
        p3=0;
      else
```

```
            l=round(eta(ii)+d);
            p1=sum(PFD(0:1:l));
            p2=sum(PFI(0:1:l));
            ld=round((eta(ii)+d)-hs);
                if (ld>10*12) ld=10*12;
                p3=sum(PFDdeck(0:1:ld));
                end
                disp('Deck Inundation')
        end

        p(ii)=p1+p2+p3;

    end
time=[0:dt:(length(p)-1)*dt];
time1=[0:dt:(length(eta)-1)*dt];

figure(1)

clf

subplot(2,1,1)
plot(time1,eta./12)
xlabel('Time (sec)')
ylabel('Surface Elevation (ft)')

subplot(2,1,2)
plot(time,p)
ylabel('Wave Force (kips)')
xlabel('Time (sec)')

Deckmax=max(p3)
pause

%    figure(1)

%    clf

%    plot(eta)

%    figure(2)

%    clf

%    plot(p)

    disp('p assembled')

    fy=68.1;

    fr=68.1;

    [u,f]=NNL3(m,c,k,fy,fr,p,dt);

    epp(j,i)=max(abs(u));
```

```
%     figure(3)

%     clf

%     plot(u)

%     figure(4)

%     clf

%     plot(u,f)

    disp('epp done')

%     [u,f]=NNL3(m,c,k,sigmay,sigmar2,p,dt);

%     deg(i,j)=max(abs(u));

%     figure(5)

%     clf

%     plot(u)

%     figure(6)

%     clf

%     plot(u,f)

%     disp('deg done')

  end

end

time=[0:dt:(length(u)-1)*dt];

figure(2)


subplot(2,1,1)
plot(time,u)
xlabel('Time (sec)')
ylabel('Deck Displ. (in)')

% subplot(2,1,2)
% plot(u,f)
% ylabel('Force (kips)')
% xlabel('Deck Displ. (in)')

% mesh(hwave,cyc',epp./16);
% title('Ductility Demand on B/EPP Structure')
```

```
% xlabel('Wave Height (ft)')
% ylabel('Number of Waves')
```